GEOTECHNICAL EXPLORATION
PROPOSED WATER TANK, GOLDEN MEADOWS – TRACT 31194
MENIFEE AREA, RIVERSIDE COUNTY, CALIFORNIA

Prepared for

RICHLAND COMMUNITIES, INC.
4100 Newport Place Drive
Newport Beach, California 92672

Project No. 11813.002

April 7, 2021

Leighton and Associates, Inc.
A LEIGHTON GROUP COMPANY

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Richland Communities, Inc.
4100 Newport Place Drive
Newport Beach, California 92672

Attention: Mr. Brian Hardy, Vice President – Land Entitlement

Subject: Geotechnical Exploration
Proposed Water Tank, Golden Meadows – Tract 31194
Menifee Area, Riverside County, California

In accordance with our February 11, 2021, Leighton and Associates, Inc. (Leighton) is pleased to present this geotechnical exploration report for the proposed welded steel water tank to be constructed on the north side of the proposed Golden Meadows residential development, Tract 31194, located in the Menifee Area of Riverside County, California. This report presents our findings and conclusions, and geotechnical recommendations for design and construction of this proposed water tank.

Based on the results of this geotechnical exploration, the site of the proposed tank is generally a ridge top with moderate to steep surrounding slopes underlain by Cretaceous-Aged granitic rock (Gabbro). The site is not located within a currently designated Alquist-Priolo Special Studies Zone or a Riverside County fault zone.

We appreciate the opportunity to be of additional service. If you have any questions or if we can be of further assistance, please contact us at your convenience.

Respectfully Submitted,

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TABLE OF CONTENTS

List of Accompanying Tables, Figures and Appendices

Tables

Figures

Appendices

1.0 INTRODUCTION

1.1 Site Location and Description

The proposed water tank is located on a ridge top along the northern portion of Tract 31194, east of the intersection of Garbani Road and Evans Road in the Menifee Area of Riverside County, California *(see Site Location Map, Figure 1)*. This site is currently a vacant/undeveloped hilltop/ridgeline that slopes moderately to steeply in southerly and westerly directions. Site access is by a steep dirt road located to the south side of the site.

1.2 Proposed Water Tank

We understand that an approximately 103-foot diameter by 40-foot high welded steel water tank is proposed to be constructed at the Site as depicted on the provided conceptual grading plan (see Figure 3) with expansion pad for a potential future tank. Based on this plan, a desired pad elevation of 1,660 feet and cut slopes of up to approximately 60 feet in height may be constructed to create the required pad (future tank). Site access is expected to extend from an extension of Grabani Road and will also require cut slopes up to approximately 20 feet in height and fill slopes of up to approximately 40 feet in height. For the purpose of bearing capacity evaluation and slope stability analyses, a static pressure of 2,500 pound-per-square-foot (PSF) is assumed to be exerted by the proposed tank.

1.3 Purpose and Scope of Work

The purpose of our geotechnical study is to explore subsurface conditions at this proposed tank site and provide geotechnical recommendations for design and construction. In accordance with our proposal, the scope of this exploration has included the following tasks:

- **Desktop Review**: We reviewed relevant geotechnical literature, reports and aerial photographs for this tank site. These documents are referenced at the end of this report.
- **Geologic Mapping**: On February 17, 2021, we performed a site reconnaissance to observe site conditions and map any pertinent geologic features (i.e. bedding, joints, foliation, etc.) in existing/exposed cut slopes or natural bedrock exposures. We also collected surface samples for the purpose of laboratory testing and evaluation.

- **Geophysical Survey**: Four (4) seismic refraction lines were performed by our sub-consultant Atlas Technical Consultants, LLC. The purpose of this survey is to obtain readings/points for both vertical and lateral velocities so "tomography models" can be provided. Tomography is an enhanced seismic refraction method that allows changes in layer velocity to be revealed as gradients rather than discrete contacts. The seismic refraction survey report is presented in whole in Appendix B. The approximate locations of the survey transects are shown on Figure 3, Geotechnical Map. Three survey lines were conducted at the tank site and one survey line was conducted along the cut area for the planned access road.
- **Geotechnical Laboratory Testing**: Geotechnical laboratory tests were performed on surficial earth material collected during our site reconnaissance. Tests performed are included in Appendix A.
- **Geotechnical Analyses**: Data obtained from our background review, field exploration and geotechnical laboratory testing was evaluated to develop geotechnical conclusions and recommendations presented in this report.
- **Report Preparation**: Results of our geotechnical exploration have been summarized in this report to address geotechnical conditions encountered at the site, including our geotechnical findings, conclusions and recommendations for tank design and construction.

Important information about limitations of geotechnical reports is presented in Appendix E, *GBA Important Information About This Geotechnical Engineering Report*.

2.0 FINDINGS

2.1 Regional Geology/Settings

The subject property is located within a prominent natural geomorphic province in southwestern California known as the Peninsular Ranges. This province is characterized by steep, elongated ranges and valleys that generally trend northwestward. Tectonic activity along the numerous faults in the region has created the geomorphology present today. Specifically, the property is situated in the southern portion of the Perris Block, a stable, eroded mass of Cretaceous and older crystalline and metamorphic rock. Thin sedimentary, metamorphic and volcanic units locally mantle the bedrock with alluvial deposits filling in the lower valley and drainage areas. The Perris Block is bounded by the San Jacinto Fault Zone to the northeast, the Elsinore Fault Zone to the southwest, the Cucamonga Fault Zone to the northwest and the poorly-defined northern boundary of the Temecula Basin to the southeast. The Temecula segment of the active Lake Elsinore Fault Zone is located approximately 6.4 miles to the southwest.

2.2 Site Geology

As regionally mapped on Figure 2, the site is underlain by gabbroic bedrock. Our field exploration indicates that this bedrock is generally covered with a relatively thin layer of surficial soils as further described below.

2.2.1 Surficial Soils

Surficial soils including topsoil and localized artificial fill should be expected within the site. These soils are expected to be relatively shallow (<3 feet), but they may be deeper in localized areas such as current access road. Expansion Index (EI) testing was performed on a representative soil sample indicate that these materials (silty sand) possess a very low expansion potential (EI=0). Test results are included in Appendix A.

2.2.2 Cretaceous-Aged Gabbroic Bedrock

Bedrock is exposed on outcrops and on steeper hillsides throughout the site. This granular granitic-type bedrock is generally dark gray in color, massive and jointed. Where exposed on the surface, the bedrock is moderately weathered and creates boulders along the hillsides up to 10 feet in diameter. The elevated portion of the site (north of future tank) has been cut into a relatively flat pad with bedrock exposed at the surface. The bedrock breaks down into granular silty to well-graded sand with gravel.

Jointing is common within this bedrock unit, limited exposures of undisturbed in-place bedrock were encountered throughout the site. Jointing typically dips to the south or southwest. Bedrock cut slopes will need to be geologically mapped as they are excavated to confirm the anticipated structural pattern and long-term stability.

2.3 Rippability and Excavation Characteristics

Review of provided conceptual site design indicates cuts (excavation below existing ground surface) up to 60 feet may be required to create the proposed pad for future tank and up to 20 feet for the proposed access road.

Based on our seismic refraction survey data performed by Atlas Technical Consultants, LLC. (Appendix B), rippable bedrock using Caterpillar D-9 dozer with a single shank should be anticipated to a depth of 10 to 20 feet BGS or may vary depending on location. However, very difficult ripping or blasting (or other rock reducing techniques) should be anticipated for deeper excavations or where measured shear wave velocities exceed 4,000 foot-per-second (fps) as shown on the “Tomograpgy Models” included in Appendix B. The relatively shallow and hard rock zones (green to red color) are likely due to buried corestones/remnant boulders, dikes, and/or less weathered rock.

A summary of the seismic refraction survey including rippability criteria based a Caterpillar D-9 dozer with a single shank is further provided in Appendix B. Trench excavation characteristics using conventional excavators may vary based on the specific equipment used. It is important that a contractor with excavation experience in similar conditions be consulted for the proper excavation methodology, equipment, and production rate based on the findings of this report.

2.4 Surface and Groundwater

Surface water was not encountered on this site during our field exploration. Groundwater is not expected to be encountered within the depth of excavation. However, localized seeps may occur at Formation contacts or in fractured zones immediately after rain events.

2.5 Faulting and Seismicity

Seismic hazards in Southern California could include strong ground shaking and fault rupture. No currently-known active surface faults cross or trend towards this project site. The subject site is not included within an Earthquake Fault Zone as created by the Alquist-

Priolo Earthquake Fault Zoning Act (Bryant and Hart, 2007). The nearest zoned active faults are the Temecula Segment of the Elsinore Fault Zone, approximately 6.4 miles southwest of the site and the Glen Ivy Segment of the Elsinore Fault Zone is located approximately 15.6 miles northwest of the site

A detailed review of vertical, sequential, stereo aerial photograph pairs was conducted to identify possible geomorphic evidence of faulting and landsliding. Various photos taken between 1949 and 1999 were reviewed (see references). Our review of aerial photographs and subsequent field observations do not provide geomorphic evidence supporting the existence of faulting or reveal any photo-lineaments that are typically associated with faulting in this region. The recent (<11,000 years) geologic history of this area reflects that this site is undergoing a regressive, erosional sequence. As observed in the aerial photographs, there are several deeply cut active, drainage channels that do not show any horizontal displacement that may be associated with active faulting.

2.6 Secondary Seismic Hazards

In general, secondary seismic hazards for sites in the region could include soil liquefaction, earthquake-induced settlement, lateral displacement and landsliding. The potential for secondary seismic hazards at the site is discussed below.

2.6.1 Seismically Induced Settlement

Seismically induced settlement consists of dry dynamic settlement (above groundwater) and liquefaction-induced settlement (below groundwater). During a strong seismic event, seismically induced settlement can occur within loose to moderately dense sandy soil due to reduction in volume during and shortly after a large, long-duration local earthquake. Settlement caused by ground shaking is often non-uniformly distributed, which can result in differential settlement. Based on the results of our exploration, seismic settlement is not considered a geotechnical constraint for this tank.

2.6.2 Seismically Induced Landslides

Based on the underlying bedrock formation and our review of aerial photographs and field observations, the site is not susceptible to seismically induced landslides.

2.7 Slope Stability

Proposed cut slopes up to 60 feet in height at 2:1 (horizontal:vertical) gradients are considered grossly stable for static and pseudo-static conditions. Compacted fill slopes up to 20 feet in height at 2:1 gradients are also considered grossly stable for static and

pseudo-static conditions. In addition, 1.5:1 cut slope in the granitic rock are also considered acceptable from a gross slope stability perspective provided no adverse geologic conditions observed during grading. As such, these cut slopes should be observed by an engineering geologist during grading to verify jointing or fracture patterns and recommend remedial measures, if needed.

3.0 CONCLUSIONS AND RECOMMENDATIONS

Based on results of this geotechnical exploration, the proposed tank site pad is underlain by granitic rock/Gabbro. The site is not located within a currently designated Alquist-Priolo Special Studies Zone or a Riverside County fault zone. However, as is the case for most of Southern California, strong ground shaking has and will occur at this site. Our geotechnical recommendations for design and construction of this proposed water tank are presented in the following sections.

3.1 Tank Foundation Location

Based on proposed pad elevations, the tank pad will be entirely cut into the granitic rock. Due to potentially weathered bedrock material along the shallow cut (daylight) areas we recommend a setback of 15 feet horizontally from daylight to ring foundation.

3.2 Earthwork

Earthwork is expected to generally consist of cut pad and access road excavation, pad surface preparation, and footing and pipeline construction. In addition, minor filling (~15 feet) may be required on the downhill side for access road. Specific earthwork recommendations are provided in the following subsections:

3.2.1 Site Preparation

Based on proposed grading concept, the tank pad is expected to expose dense gabbroic rock. If highly weathered bedrock/loose rocks or any undesirable geologic features are exposed within portions of the tank pad and/or subgrade for the access road, then such conditions should be addressed by the project geotechnical engineer or geologist prior to foundation construction.

3.2.2 Fill Placement and Compaction

Onsite very low expansive (EI<21) soils free of organics, debris, and oversized material less than (≤) 3-inches in largest dimension are suitable for use as structural fill on this site. Soils to be placed as fill, whether onsite or import material, should be reviewed by Leighton and tested if and as necessary.

To provide uniform subgrade and fill any potential voids created from removal of loose rock/materials, we recommend that a minimum of 6-inch layer consisting of granular base (Caltrans Class 2 or equivalent) be placed prior to construction of concrete floor slab. However, if removal of rock or loose material creates voids larger than 2 feet in depth, such areas should be subject to further evaluation as potentially needing additional filling procedures.

Where fill is being placed at slopes steeper-than (>) 5:1 (horizontal:vertical), proper surface preparations and benching should be implemented in accordance with latest edition of the "Greenbook", and approved by Leighton during construction. A 15-foot wide minimum fill slope keyway should be prepared to support the tank pad slope and access road fill slope (see Figure 5). As such, all areas to receive fill, including processed areas, fill slope, and benches, should be observed, mapped, and approved or tested by Leighton prior to proceeding with placement of fill.

3.2.3 Utility Trench Backfill

Utility trenches should be backfilled with compacted fill in accordance with Sections 306-1.2 and 306-1.3 of the "Greenbook". 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:

- **Sand**: A uniform, granular material that has a Sand Equivalent (SE) of (≥) 30 or greater and a maximum particle size of ¾-inches (or as specified by the pipe manufacturer), water densified in place, or
- **CLSM**: One sack cement slurry/Controlled Low Strength Material (CLSM) conforming to Section 201-6 of the "Greenbook".

Pipe bedding should extend at least 4-inches below any pipeline invert and at least 12 inches over the top of the pipeline. Native soils 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 D 1557 laboratory maximum density.

3.3 Seismic Design Parameters

It is our understanding that the proposed water tank will be constructed of steel and the structural design of the tank will follow ANSI/AWWA D100-11. Therefore, the purpose of the seismic hazard evaluation is to identify and assess potential seismic hazards at the site in general accordance with the requirements of ANSI/AWWA D100, which generally follows the requirements of ASCE 7-16. Our seismic hazard evaluation includes development of peak ground acceleration (PGA) and design response spectra by using a seismic source model based on proximity of the site to active faults, major historical earthquakes, regional seismicity, and subsurface soil conditions at the site. Specifically, our scope includes estimation of peak horizontal ground acceleration and the response spectra at the site for the Maximum Considered Earthquake (MCE) and the Design Earthquake (DE) Site-specific ground motion parameters derived based on the

requirements of ASCE 7-16, Chapters 11 and 21. These seismic coefficients were calculated utilizing an interactive program on current United States Geological Survey (USGS) website (referred to as USGS General Procedure). Based on our site-specific seismic refraction survey, this site is classified as a Class **B** site:

Table 1. 2019 CBC Site-Specific Seismic Parameters

2013 CBC Site-Specific Seismic Design Parameters	Value
Site Longitude (decimal degrees)	-117.18905
Site Latitude (decimal degrees)	33.6555
Site Class Definition	**B**
Mapped Spectral Response Acceleration at 0.2s Period, S_s	1.41
Mapped Spectral Response Acceleration at 1s Period, S_1	0.52
Short Period Site Coefficient at 0.2s Period, F_a	*0.9*
Long Period Site Coefficient at 1s Period, F_v	*0.8*
Adjusted Spectral Response Acceleration at 0.2s Period, S_{MS}	1.26
Adjusted Spectral Response Acceleration at 1s Period, S_{M1}	0.42
Design Spectral Response Acceleration at 0.2s Period, S_{DS}	**0.84**
Design Spectral Response Acceleration at 1s Period, S_{D1}	**0.28**
Long-Period Transitions, T_L	**8 sec**

The results of this analysis also indicate that the adjusted Peak Ground Acceleration (PGA_m) for the MCE_G is 0.54g.

3.4 Tank Spread/Ring Footing Foundations

The proposed foundations and slabs should be designed in accordance with the structural consultants' design, the minimum geotechnical recommendations presented herein, and applicable ANSI/AWWA D100-11 requirements.

3.4.1 Minimum Embedment and Width

Conventional shallow spread/ring footings may be used to support the proposed tank, bearing solely on an undisturbed granitic rock approved by the geotechnical consultant. Tank footings should be embedded at least 12-inch below lowest adjacent grade, with a minimum width of 12-inch. These footings should have a minimum of 15 feet setback from adjacent descending slope/daylight.

3.4.2 Allowable Bearing Pressure

An allowable bearing pressure of 5,000 pounds-per-square-foot (psf) may be used for static and sustained live loads, based on minimum embedment depth and widths recommended above. The bearing pressure value may be increased by

250 psf for each additional foot of embedment or each additional foot of width to a maximum vertical bearing value of 6,000 psf. These allowable bearing pressures are for total dead loads and frequently applied live loads, and can be increased by one-third for short duration wind and seismic loads. Where applicable, a modulus of subgrade reaction of 450 pci may be used for design of footings/pads or any structures founded on this Formation.

All continuous footings should be reinforced with top and bottom reinforcing steel to provide structural continuity and to permit spanning of local irregularities. It is essential that we observe tank pad and footing excavations before reinforcing steel is placed.

3.4.3 Lateral Load Resistance

Lateral (horizontal) loads on foundations may be resisted by both frictional resistance along the base of the footing and passive resistance in properly compacted fill adjacent to the sides of footings. Frictional resistance between the base of footings poured (cast) directly on native rock or aggregate base may be computed using a coefficient of friction of 0.35, or 35-percent of sustained dead loads. Passive resistance may be computed using an equivalent fluid pressure of 300 pounds-per-cubic-foot (pcf) for undisturbed Pauba and/or new properly compacted fill. Passive pressure should not exceed 3,000 psf. These values may be increased by one-third when considering wind and seismic forces. Both friction and passive values have already been reduced by a factor-of-safety of 1.5, and can be used in combination.

3.4.4 Settlement Estimates

Based on the tank hydrostatic pressures presented in Section 1.2 (< 2,500 psf) and bearing on native rock, the settlement is expected to be less-than (<) ½-inch at the center of the tank and on the order of ¼-inch to negligible at the edge/perimeter.

3.5 Lateral Pressures for Retaining Structures

The lateral earth pressures below are provided for the design permanent retaining structures/walls. Earth pressures provided are ultimate values and a safety factor should be applied as appropriate.

Table 2. Static Lateral Earth Pressures

Conditions	Equivalent Fluid Weight (pcf)		
	Level Backfill[2]	2:1 Slope Backfill	1.5:1 Slope Backfill
Active (cantilever)	35	55	70
At-Rest (braced)	50	75	95
Passive[3]	250	-	-

Notes:
(1) Assumes drained condition
(2) Assumes a level condition behind and in front of wall foundation of project.
(3) Maximum passive pressure = 3,500 psf, level conditions.

Determination of appropriate design conditions (active or at-rest) depends on wall flexibility. If a rotation of more than 0.001 radian (0.06 degrees) is allowed, active pressure conditions apply; otherwise, at-rest condition governs.

Surcharge due to above grade loads on the wall backfill, such as traffic, should be considered in design of retaining walls. Vertical surcharge loads behind the retaining wall on or in the backfill within a 1:1 (horizontal:vertical) 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.

Additional lateral earth pressures due to seismic shaking should also be considered in the design. In accordance with current engineering practices and research, an increment of lateral earth pressure equal to $14H^2$ where H is the height of the wall, may be applied at a distance of 0.5H above the toe of the wall. If the wall is restrained, the above increment of lateral earth pressure should be doubled. Under the combined effects of static and earthquake loads on the wall, a factor of safety between 1.1 and 1.2 is acceptable when evaluating the stability (sliding, overturning) of the wall (NAVFAC DM 7.2).

Where applicable, a coefficient of friction of 0.35 may be considered between the concrete/shotcrete walls and the backfill surrounding the tank to estimate downward drag forces.

3.6 Asphalt Paving for Driveway / Access Road

Pavement construction associated with the proposed access road should conform to latest version of *Caltrans Standard Specifications* or the *Standard Specifications for Public Works Construction* (Greenbook), and applicable County Standards. Based on design procedures outlined in the current Caltrans *Highway Design Manual*, recommended flexible (asphalt) pavement section is tabulated below for an assumed Traffic Index (TI) of 4.0 and R-value of 40.

Table 3. Preliminary Asphalt Pavement Section

Traffic Index	Thickness (inches)	
	Asphalt Concrete	Class 2 Aggregate Base
4.0	3.0	4.0

Representative samples of the actual subgrade materials for R-value testing, during subgrade preparation or prior to pavement construction, can be performed to refine this pavement design. An appropriate Traffic Index (TI) should be selected or verified by the project Civil Engineer prior to finalizing this pavement section design, based on anticipated truck traffic. This TI is based on only light auto and pickup-truck traffic.

Pavement subgrade soils should be prepared in accordance with Section 3.1 above. The Aggregate Base (AB) should be compacted to a minimum of 95 percent relative compaction (modified Proctor, ASTM D 1557). Aggregate base may be waived is asphalt concrete is placed directly on granitic rock.

3.7 Soil Corrosivity

3.7.1 Sulfate Attack

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

Table 4. Sulfate Concentration and Sulfate Exposure

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

3.7.2 Ferrous Corrosivity

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

Table 5. Relationship between Soil Resistivity and Soil Corrosivity

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

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

3.7.3 Soil Corrosivity Test Results Summary

As a preliminary screening process for sulfates in soils, we have performed laboratory tests on a representative surface soil-sample. As summarized in Table 6 (below), our laboratory test results indicated negligible concentration of soluble sulfates. No special measures to mitigate sulfate exposure are recommended

based on the test results. Import soils (if any) should also be tested for sulfate content.

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

Table 6. Soil Corrosivity Test Results Summary

Boring Number	Sample Depth (feet)	Sulfate Content (ppm)	Chloride Content (ppm)	pH	Resistivity (ohm-cm)
S-1	0 to 1	**111**	**20**	6.7	13,000

4.0 CONSTRUCTION CONSIDERATIONS

4.1 Trench Excavations

Based on our field observations, caving of cohesionless and sandy soils will likely be encountered in unshored trench excavations. To protect workers entering excavations, excavations should be performed in accordance with OSHA and Cal-OSHA requirements, and the current edition of the California Construction Safety Orders, see: http://www.dir.ca.gov/title8/sb4a6.html

Contractors should be advised that fill and cohesionless alluvial/colluvial soils should be considered Type C soils as defined in the California Construction Safety Orders. As such, excavations less-than (<) 20 feet deep within Type C soils should be sloped back no steeper than 1½:1 (horizontal:vertical), where workers are to enter the excavation. Weathered granitic rock may be classified as OSHA soil Type A. Therefore, unshored temporary cut slopes should be no steeper than ¾:1 (horizontal:vertical), for a height no-greater-than (≤) 10 feet. These recommended temporary cut slopes assume a level ground surface for a distance equal to one-and-a-half (x1.5) the depth of excavation. However, unshored excavations may be impractical near adjacent existing utilities and structures; so shoring may still be required depending on trench locations.

During construction, soil 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. should be maintained to facilitate construction while providing safe excavations.

4.2 Temporary Trench Shoring

Typical cantilever shoring can be designed based on the active equivalent fluid pressure of 33 pounds-per-cubic-foot (pcf) where there is no adverse bedding. If excavations are braced at the top and at specific depth intervals, then braced earth pressure may be approximated by a uniform rectangular soil pressure distribution. This uniform pressure expressed in pounds-per-square-foot (psf), may be assumed to be 20 multiplied by H for design, where H is equal to the depth of the excavation being shored, in feet. These recommendations are valid only for trenches not exceeding 10-feet in depth at this site.

4.3 Geotechnical Services during Construction

Our geotechnical recommendations presented in this report are based on subsurface conditions as interpreted from limited subsurface explorations and limited geotechnical laboratory testing. Our geotechnical recommendations provided in this report are based on information available at the time the report was prepared and may change as plans are developed. Additional geotechnical exploration, testing and/or analysis may be required based on final plans. Leighton and Associates, Inc. should review site grading, foundation and shoring (if any) plans when available, to comment further on geotechnical aspects of this project and check to see general conformance of final project plans to recommendations presented in this report.

Leighton and Associates, Inc. should be retained to provide geotechnical observation and testing during excavation and all phases of earthwork. Our conclusions and recommendations should be reviewed and verified by us during construction and revised accordingly if geotechnical conditions encountered vary from our findings and interpretations. Geotechnical observation and testing should be provided:

- During all cut excavation,
- During compaction of all fill materials,
- After excavation of all footings and prior to placement of concrete,
- During utility trench backfilling and compaction,
- During pavement subgrade and base preparation (if any), and/or
- If and when any unusual geotechnical/geologic conditions are encountered.

5.0 LIMITATIONS

This report was necessarily based in part upon data obtained from a limited number of observances, site visits, soil samples, tests, analyses, histories of occurrences, spaced subsurface explorations and limited information on historical events and observations. Such information is necessarily incomplete. The nature of many sites is such that differing characteristics can be experienced within small distances and under various climatic conditions. Changes in subsurface conditions can and do occur over time. This exploration was performed with the understanding that this subject site is proposed for development as described in Section 1.2 of this report. Please refer to Appendix E, ASFE's *Important Information About Your Geotechnical Report*, prepared by the Associated Soil and Foundation Engineers (ASFE) presenting additional information and limitations regarding geotechnical engineering studies and reports.

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

R E F E R E N C E S

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Aerial Photos Reviewed:

3-12-99 C135 – 39 – 135-136

9-11-97 C116 – 39 – 46-47

10-4-95 CAP – 125-127

1-30-95 C104 – 39 – 10

2-2-93 C86 – 39 – 13

10-12-90 90205 – 125-126

7-30-86 86084

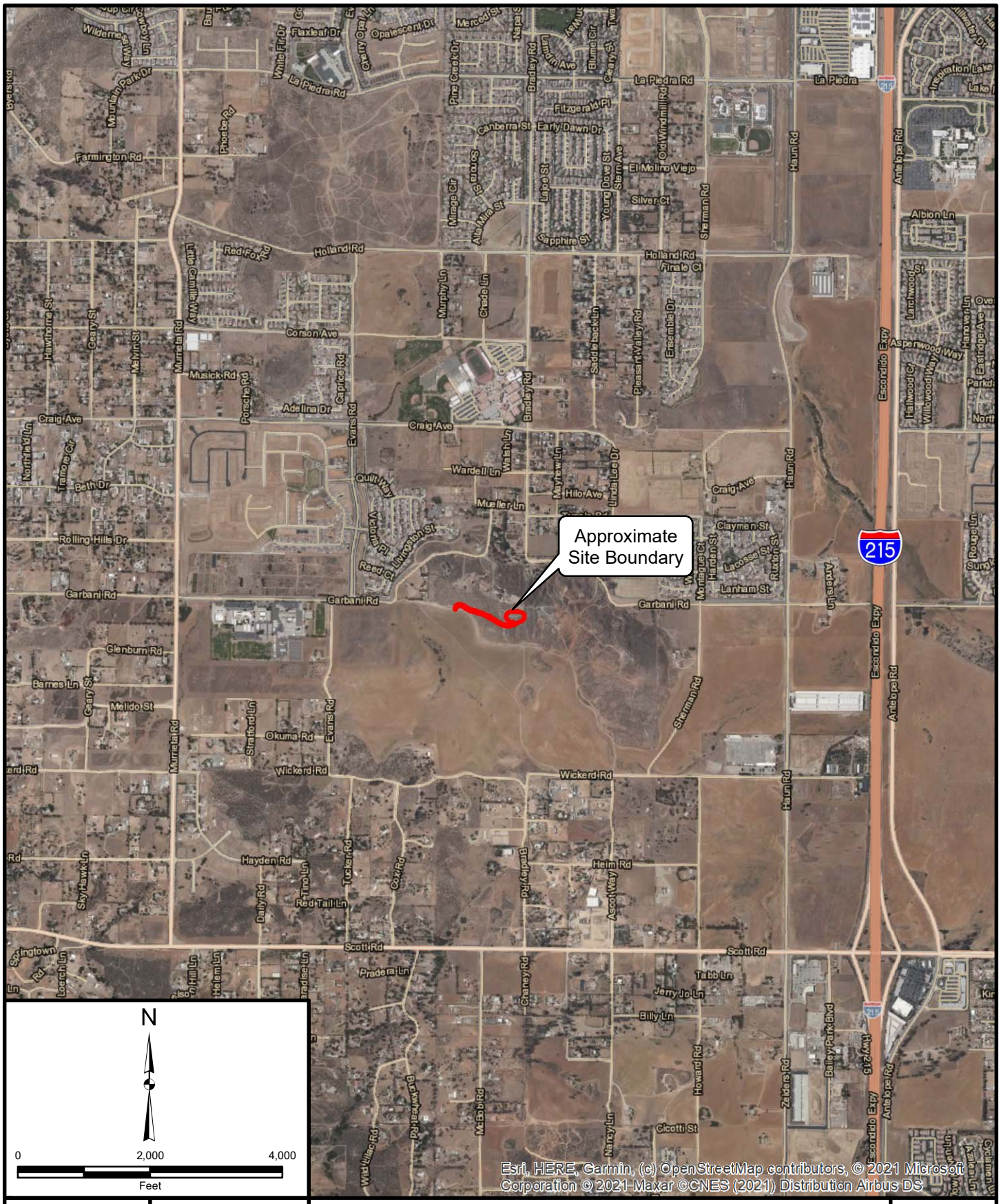
6-14-83 83045 – 101

12-14-79 79274 – 161A

5-15-67 2HH – 191-192

5-15-67 3HH – 20

5-23-49 9F – 124-125



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Scale: 1 " = 2,000 '	Date: March 2021

Base Map: ESRI ArcGIS Online 2021
Thematic Information: Leighton
Author: Leighton Geomatics (btran)

SITE LOCATION MAP

Golden Meadows Tank Site
Tract 31194
Menifee Area, Riverside County, California

Figure 1

Leighton

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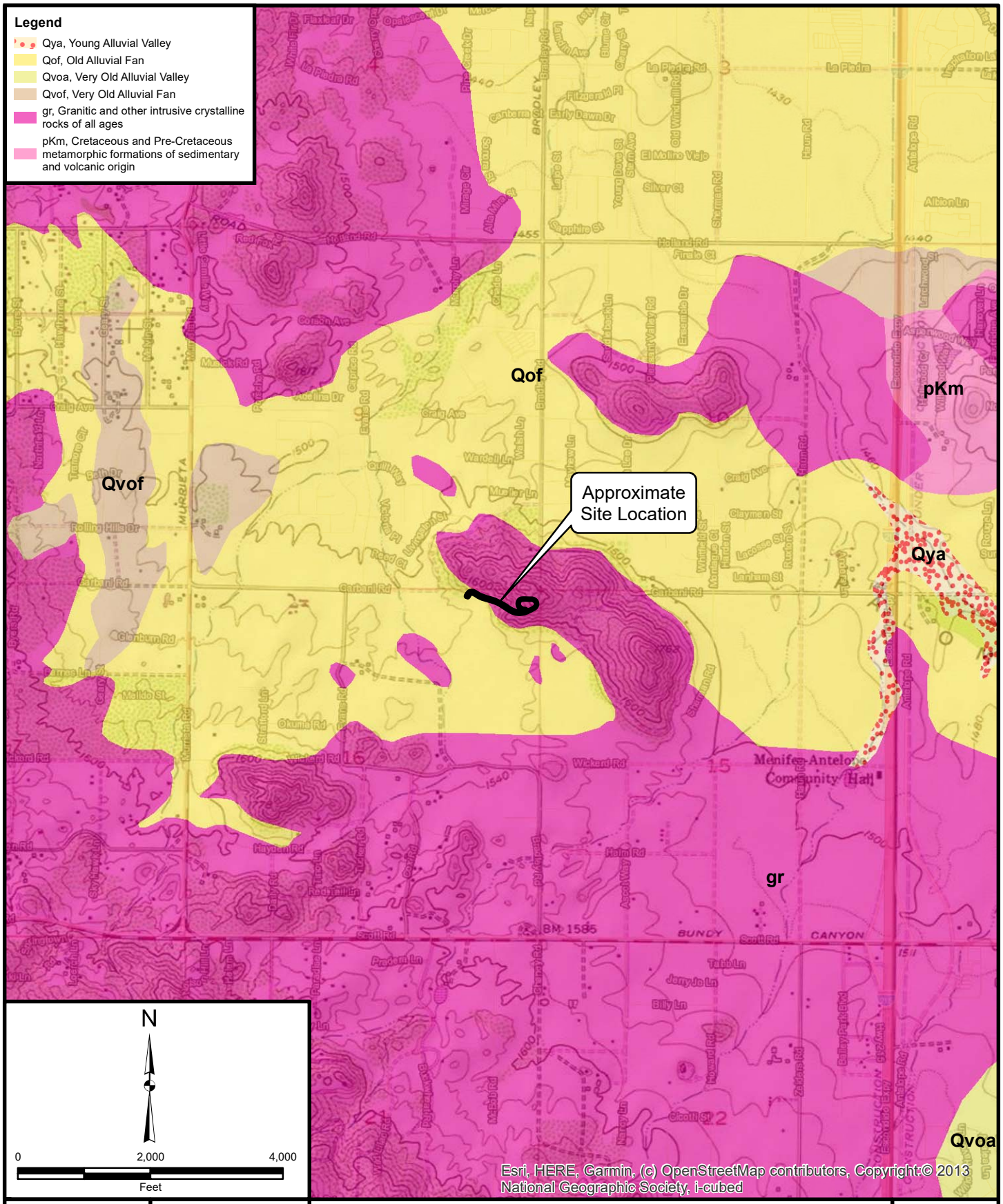

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Scale:1 " = 2,000 '	Date: March 2021

Base Map: ESRI ArcGIS Online 2021
Thematic Information: Leighton, USGS
Author: Leighton Geomatics (btran)

REGIONAL GEOLOGY MAP

Golden Meadows Tank Site
Tract 31194
Menifee Area, Riverside County, California

Figure 2

Leighton

Map Saved as V:\Drafting\11813\002\Maps\11813-002_F02_RGM_2021-03-10.mxd on 2/23/2021 1:16:13 PM

TRACT 31194 – TANK SITE GRADING EXHIBIT

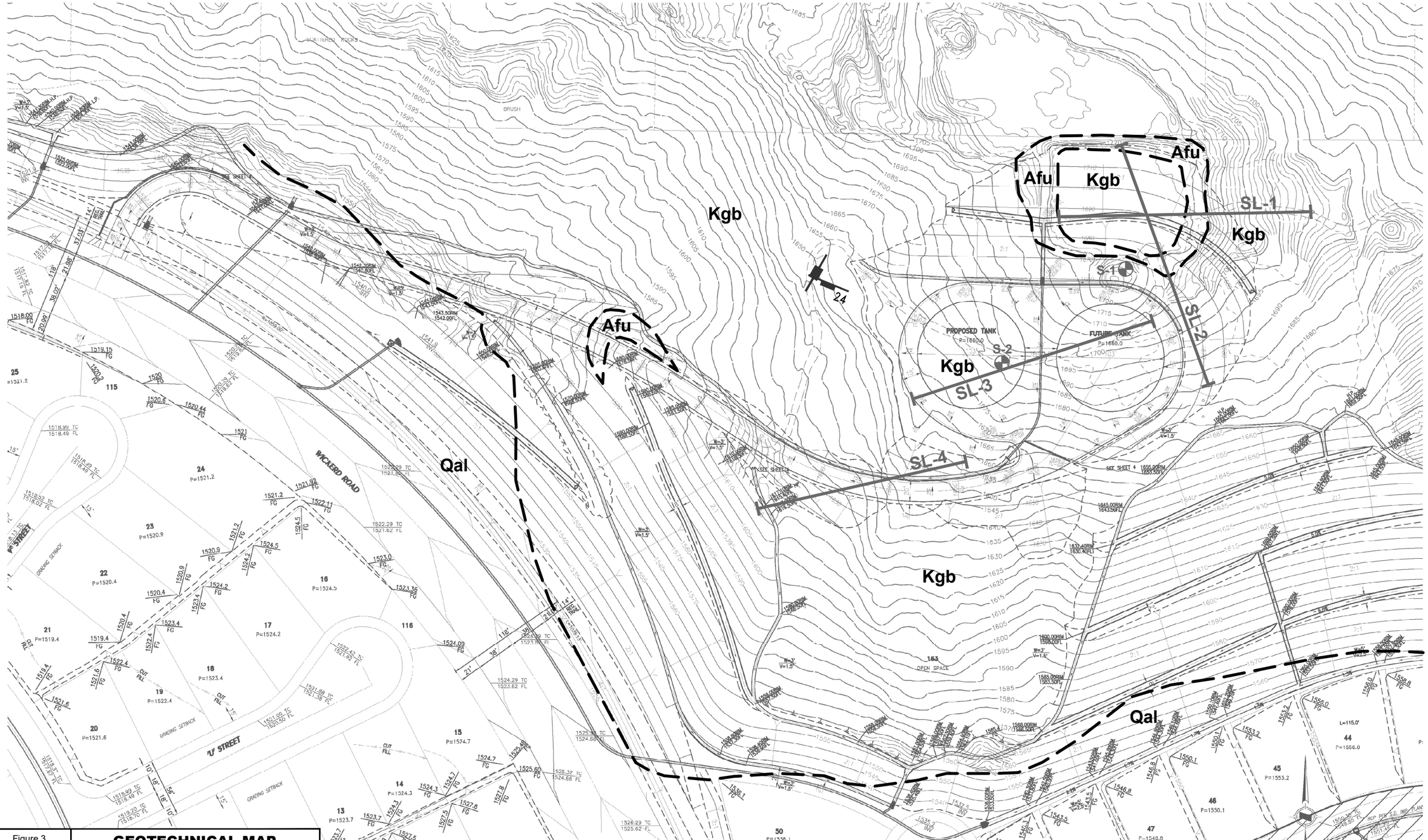

Figure 3

Leighton

GEOTECHNICAL MAP

Golden Meadows Tank Site
Tract 31194
Menifee Area, Riverside County, California

Proj: 11813.002	Eng/Geol: SIS/RFR
Scale: As Shown	Date: March 2021

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LEGEND

Afu	ARTIFICIAL FILL UNDOCUMENTED	24	STRIKE AND DIP OF BEDROCK JOINT
Qal	QUATERNARY ALLUVIUM	SL-4	APPROXIMATE LOCATION OF SEISMIC REFRACTION LINE
Kgb	CRETACEOUS AGED GABBRO	S-2	APPROXIMATE LOCATION OF SURFACE SAMPLE

0 20 40 80
SCALE: 1" = 40'

JANUARY 2020

APPENDIX A

Laboratory Test Results

Leighton

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TESTS for SULFATE CONTENT
CHLORIDE CONTENT and pH of SOILS

Project Name: Golden Meadows TR31194 Tank Geo Tested By : M. Vinet Date: 02/22/21

Project No. : 11813.002 Data Input By: M. Vinet Date: 02/23/21

Boring No.	S-1			
Sample No.	S-1			
Sample Depth (ft)	0 - 1.0			
Soil Identification:	Poorly Graded Sand with Silt (SP-SM)			
Wet Weight of Soil + Container (g)	100.00			
Dry Weight of Soil + Container (g)	100.00			
Weight of Container (g)	0.00			
Moisture Content (%)	0.00			
Weight of Soaked Soil (g)	100.00			

SULFATE CONTENT, DOT California Test 417, Part II

Beaker No.	1			
Crucible No.	1			
Furnace Temperature (°C)	850			
Time In / Time Out	Timer			
Duration of Combustion (min)	45			
Wt. of Crucible + Residue (g)	25.1122			
Wt. of Crucible (g)	25.1095			
Wt. of Residue (g) (A)	0.0027			
PPM of Sulfate (A) x 41150	111.11			
PPM of Sulfate, Dry Weight Basis	**111**			

CHLORIDE CONTENT, DOT California Test 422

ml of Extract For Titration (B)	30			
ml of AgNO3 Soln. Used in Titration (C)	0.4			
PPM of Chloride (C -0.2) * 100 * 30 / B	20			
PPM of Chloride, Dry Wt. Basis	**20**			

pH TEST, DOT California Test 643

pH Value	**6.70**			
Temperature °C	21.0			

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SOIL RESISTIVITY TEST

DOT CA TEST 643

Project Name:	Golden Meadows TR31194 Tank Geo	Tested By :	M. Vinet	Date:	02/22/21
Project No. :	11813.002	Data Input By:	M. Vinet	Date:	02/23/21
Boring No.:	S-1	Depth (ft.) :	0 - 1.0		
Sample No. :	S-1				

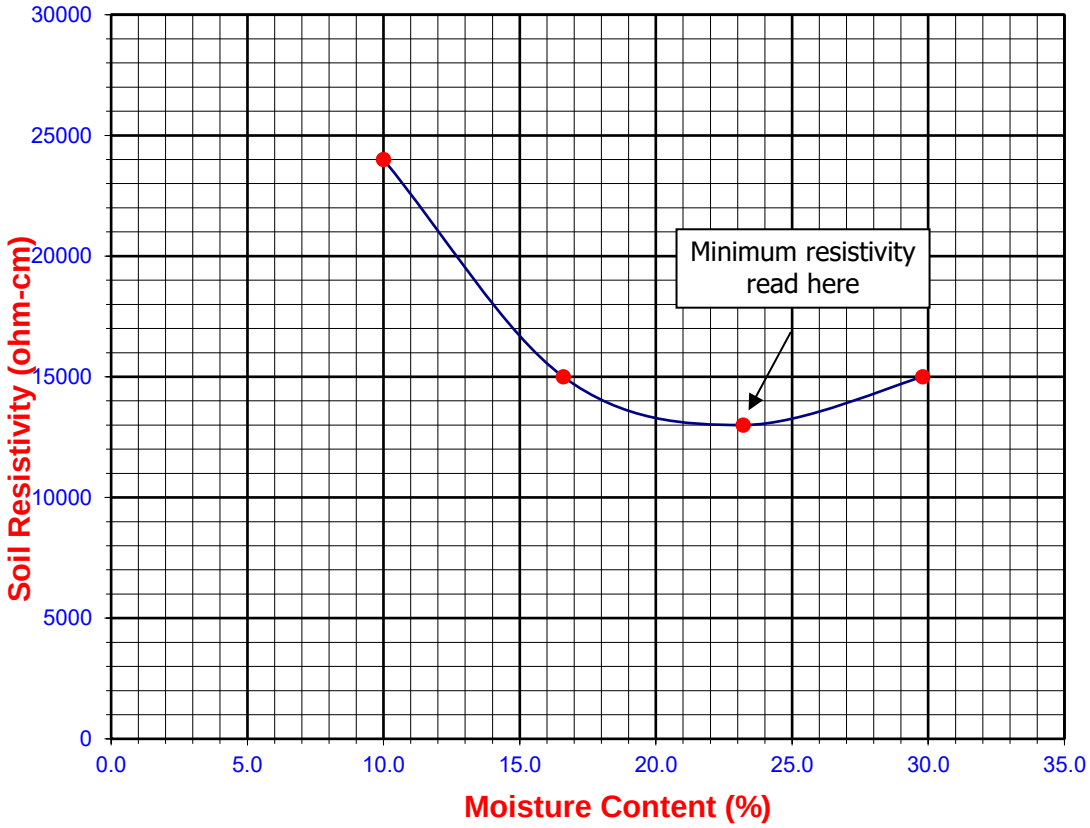
Soil Identification:* Poorly Graded Sand with Silt (SP-SM)

*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	24000	24000
2	83	16.60	15000	15000
3	116	23.20	13000	13000
4	149	29.80	15000	15000
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	Temp. (°C)
DOT CA Test 643		DOT CA Test 417 Part II	DOT CA Test 422	DOT CA Test 643	
13000	**23.2**	**111**	**20**	**6.70**	**21.0**



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EXPANSION INDEX of SOILS

ASTM D 4829

Project Name:	Golden Meadows TR31194 Tank Geo	Tested By:	M. Vinet	Date:	2/18/21
Project No. :	11813.002	Checked By:	M. Vinet	Date:	2/19/21
Boring No.:	S-1	Depth:	0 - 1.0		
Sample No. :	S-1	Location:	N/A		
Sample Description:	Poorly Graded Sand with Silt (SP-SM), Olive Brown.				

Dry Wt. of Soil + Cont. (gm.)	3350.7
Wt. of Container No. (gm.)	0.0
Dry Wt. of Soil (gm.)	3350.7
Weight Soil Retained on #4 Sieve	124.3
Percent Passing # 4	96.3

MOLDED SPECIMEN	Before Test	After Test
Specimen Diameter (in.)	4.01	4.01
Specimen Height (in.)	1.0000	0.9935
Wt. Comp. Soil + Mold (gm.)	590.0	619.8
Wt. of Mold (gm.)	200.0	200.0
Specific Gravity (Assumed)	2.70	2.70
Container No.	11	11
Wet Wt. of Soil + Cont. (gm.)	627.8	619.8
Dry Wt. of Soil + Cont. (gm.)	599.3	352.9
Wt. of Container (gm.)	327.8	200.0
Moisture Content (%)	10.5	18.9
Wet Density (pcf)	117.6	127.5
Dry Density (pcf)	106.5	107.2
Void Ratio	0.584	0.573
Total Porosity	0.368	0.364
Pore Volume (cc)	76.3	74.9
Degree of Saturation (%) [S meas]	**48.6**	**89.2**

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.)
2/18/21	11:30	1.0	0	0.5000
2/18/21	11:40	1.0	10	0.5000
	Add Distilled Water to the Specimen			
2/19/21	7:00	1.0	1160	0.4935
2/19/21	8:00	1.0	1220	0.4935

Expansion Index (EI meas) = ((Final Rdg - Initial Rdg) / Initial Thick.) x 1000	-6.5
Expansion Index (Report) = Nearest Whole Number or Zero (0) if Initial Height is > than Final Height	**0**

Rev. 03-08

APPENDIX B

Results of Seismic Refraction Survey

Leighton

SEISMIC REFRACTION STUDY

GOLDEN MEADOWS TANK EXPLORATION

Menifee, California

PREPARED FOR:

Jeffery Deland
Leighton & Associates, Inc.
41715 Enterprise Circle North, Suite 103
Temecula, CA 92590

PREPARED BY:

Atlas Technical Consultants, LLC
6280 Riverdale Street
San Diego, CA 92120

March 9, 2021

6280 Riverdale Street
San Diego, CA 92120
(877) 215-4321 | oneatlas.com

March 9, 2021

Atlas No. 121058SWG
Report No. 1

MR. JEFFERY DELAND
LEIGHTON & ASSOCIATES, INC.
17781 COWAN
IRVINE, CA 92614

Subject: Seismic Refraction Study
Golden Meadows Tank Exploration
Menifee, California

Dear Mr. Deland:

In accordance with your authorization, Atlas Technical Consultants has performed a seismic refraction study pertaining to the Golden Meadows Tank Exploration project located in Menifee, California. Specifically, our evaluation consisted of performing four seismic P-wave refraction traverses at the project site. The purpose of our evaluation was to develop subsurface velocity profiles of the study areas in order to assess the depth to bedrock and apparent rippability of the subsurface materials. Our field services were conducted on February 17th, 2021. This data report presents our methodology, equipment used, analysis, and results.

If you have any questions, please call us at 619.280.4321.

Respectfully submitted,
Atlas Technical Consultants LLC

REGISTERED PROFESSIONAL GEOPHYSICIST
PATRICK F. LEHRMANN
No. 1043
Exp. 1/31/2022
STATE OF CALIFORNIA

Evan C. Anderson
Senior Staff Geophysicist

Patrick F. Lehrmann, P.G., P.Gp.
Principal Geologist/Geophysicist

ECA:PFL:ds

Distribution: Mr. Jeffery DeLand at @leightongroup.com

CONTENTS

TABLES

FIGURES

1. INTRODUCTION

In accordance with your authorization, Atlas Technical Consultants has performed an additional seismic refraction study pertaining to the Golden Meadows Tank Exploration project located in Menifee, California (Figure 1). Specifically, our evaluation consisted of performing four seismic P-wave refraction traverses at the project site. The purpose of our evaluation was to develop subsurface velocity profiles of the study areas in order to assess the depth to bedrock and apparent rippability of the subsurface materials. Our field services were conducted on February 17th, 2021. This data report presents our methodology, equipment used, analysis, and results.

2. SCOPE OF SERVICES

Our scope of services included:

- Performance of four seismic P-wave refraction traverses at the project site.
- Compilation and analysis of the data collected.
- Preparation of this data report presenting our results and conclusions.

3. SITE AND PROJECT DESCRIPTION

The project site is generally located west of State Route 215 and east of State Route 15 in Menifee, California (Figure 1). Specifically, the seismic traverses are located at the termination of Garbani Road along the southern slope of the adjacent hill. The seismic traverses were performed over slight to moderately steep terrain. Granite outcrops were present throughout the study areas. The study site was marked by dry, knee-high brush throughout. Figures 2 and 3 depict the general site conditions in the areas of the seismic traverses.

Based on our discussions with you, it is our understanding that new construction activities are proposed at the site and the results of our study may be used for the design and construction parameters pertaining to the project.

4. STUDY METHODOLOGY

A seismic P-wave (compression wave) refraction study was conducted at the project site to evaluate the rippability characteristics of the subsurface materials and to develop subsurface velocity profiles of the area studied. The seismic refraction method uses first-arrival times of refracted seismic waves to estimate the thicknesses and seismic velocities of subsurface layers. Seismic P-waves generated at the surface, using a hammer and plate, are refracted at boundaries separating materials of contrasting velocities. These refracted seismic waves are then detected by a series of surface vertical component 14.5-Hz geophones and recorded with a 24-channel Geometrics Geode seismograph. The travel times of the seismic P-waves are used in conjunction with the shot-to-geophone distances to obtain thickness and velocity information on the subsurface materials.

Four seismic traverses (SL-1 and SL-4) were conducted in the study area. The general locations and lengths of the lines were determined by surface conditions, site access, and depth of investigation. Shot points (signal generation locations) were conducted along the lines at the ends, midpoint, and intermediate points between the ends and the midpoint.

The seismic refraction theory requires that subsurface velocities increase with depth. A layer having a velocity lower than that of the layer above will not generally be detectable by the seismic refraction method and, therefore, could lead to errors in the depth calculations of subsequent layers. In addition, lateral variations in velocity, such as those caused by core stones, intrusions, or boulders can also result in the misinterpretation of the subsurface conditions. In general, the effective depth of evaluation for a seismic refraction traverse is approximately one-third to one-fifth of the length of the spread.

In general, the seismic P-wave velocity of a material can be correlated to rippability (see Table 1 below), or to some degree "hardness." Table 1 is based on published information from the Caterpillar Performance Handbook (Caterpillar, 2018), as well as our experience with similar materials, and assumes that a Caterpillar D-9 dozer ripping with a single shank is used. We emphasize that the cutoffs in this classification scheme are approximate and that rock characteristic, such as fracture spacing and orientation, play a significant role in determining rock quality or rippability. The rippability of a mass is also dependent on the excavation equipment used and the skill and experience of the equipment operator.

For trenching operations**,** the rippability values should be scaled downward. For example, velocities as low as 3,500 feet/second may indicate difficult ripping during trenching operations. In addition, the presence of boulders, which can be troublesome in narrow trenching operations, should be anticipated.

Table 1 – Rippability Classification

Seismic P-wave Velocity	Rippability
0 to 2,000 feet/second	Easy
2,000 to 4,000 feet/second	Moderate
4,000 to 5,500 feet/second	Difficult, Possible Blasting
5,500 to 7,000 feet/second	Very Difficult, Probable Blasting
Greater than 7,000 feet/second	Blasting Generally Required

It should be noted that the rippability cutoffs presented in Table 1 are slightly more conservative than those published in the Caterpillar Performance Handbook. Accordingly, the above classification scheme should be used with discretion, and contractors should not be relieved of making their own independent evaluation of the rippability of the on-site materials prior to submitting their bids.

5. DATA ANALYSIS

The collected data was processed using SIPwin (Rimrock Geophysics, 2003), a seismic interpretation program, and analyzed using SeisOpt Pro (Optim, 2008). SeisOpt Pro uses first arrival picks and elevation data to produce subsurface velocity models through a nonlinear optimization technique called adaptive simulated annealing. The resulting velocity model provides a tomography image of the estimated geologic conditions. Both vertical and lateral velocity information is contained in the tomography model. Changes in layer velocity are revealed as gradients rather than discrete contacts, which typically are more representative of actual conditions.

6. RESULTS AND CONCLUSIONS

As previously indicated, four seismic traverses were conducted as part of our study. Figures 4a, 4b, 4c, and 4d present the velocity models generated from our analysis. Based on the results, it appears the project site is underlain by moderately low-velocity materials in the very near-surface and higher velocity materials in the shallow surface. Distinct vertical and lateral velocity variations are evident in the models. Moreover, the degree of weathering and the depth to possible bedrock varies across the study area.

Based on the refraction results, variability in the excavatability (including depth of rippability) of the subsurface materials should be expected across the project areas. In addition, oversized materials should be expected. A contractor with excavation experience in similarly difficult conditions should be consulted for expert advice on excavation methodology, equipment, and production rate.

7. LIMITATIONS

The field evaluation and geophysical analyses presented in this report have been conducted in general accordance with current practice and the standard of care exercised by consultants performing similar tasks in the project area. No warranty, express or implied, is made regarding the conclusions, recommendations, and opinions presented in this report. There is no evaluation detailed enough to reveal every subsurface condition. Variations may exist and conditions not observed or described in this report may be present. Uncertainties relative to subsurface conditions can be reduced through additional subsurface exploration. Additional subsurface surveying will be performed upon request.

This document is intended to be used only in its entirety. No portion of the document, by itself, is designed to completely represent any aspect of the project described herein. Atlas should be contacted if the reader requires additional information or has questions regarding the content, interpretations presented, or completeness of this document. This report is intended exclusively for use by the client. Any use or reuse of the findings, conclusions, and/or recommendations of this report by parties other than the client is undertaken at said parties' sole risk.

8. SELECTED REFERENCES

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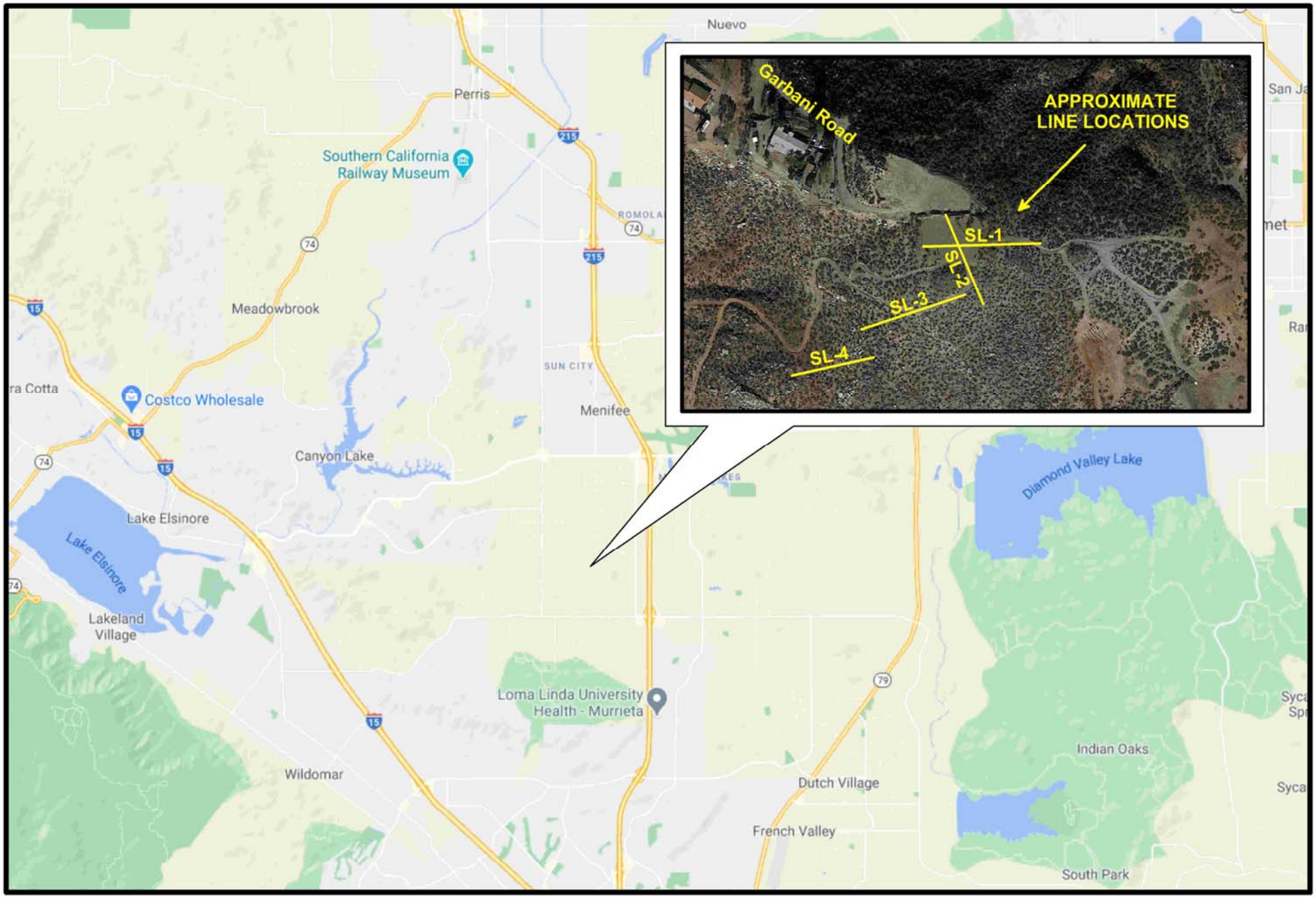


SITE LOCATION MAP	N	Golden Meadows Tank Exploration Menifee, California	ATLAS Figure 1
		Project No.: 121058SWG / Date: 03/21	

SEISMIC LINE LOCATION MAP

N

Golden Meadows Tank Exploration
Menifee, California

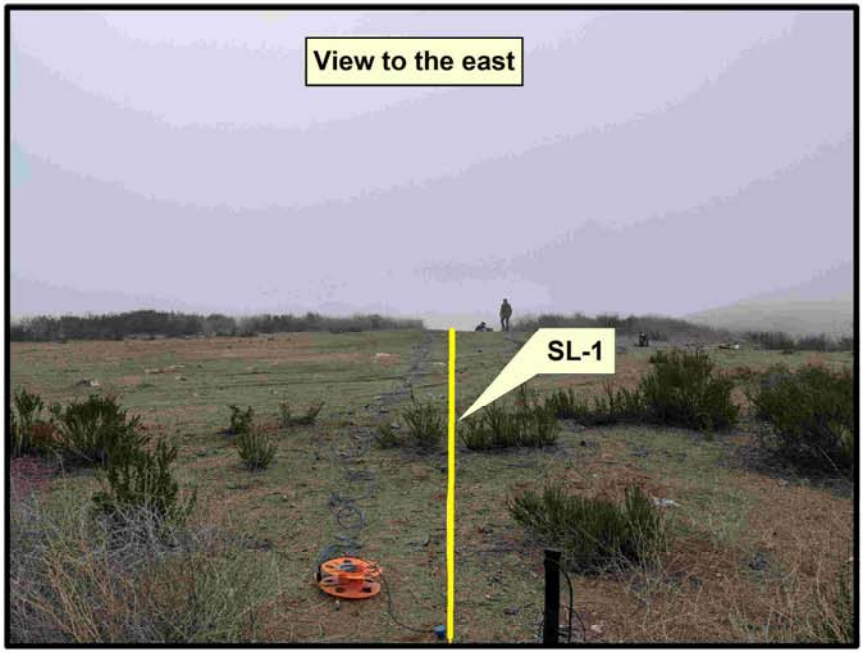
Project No.: 121058SWG | Date: 03/21

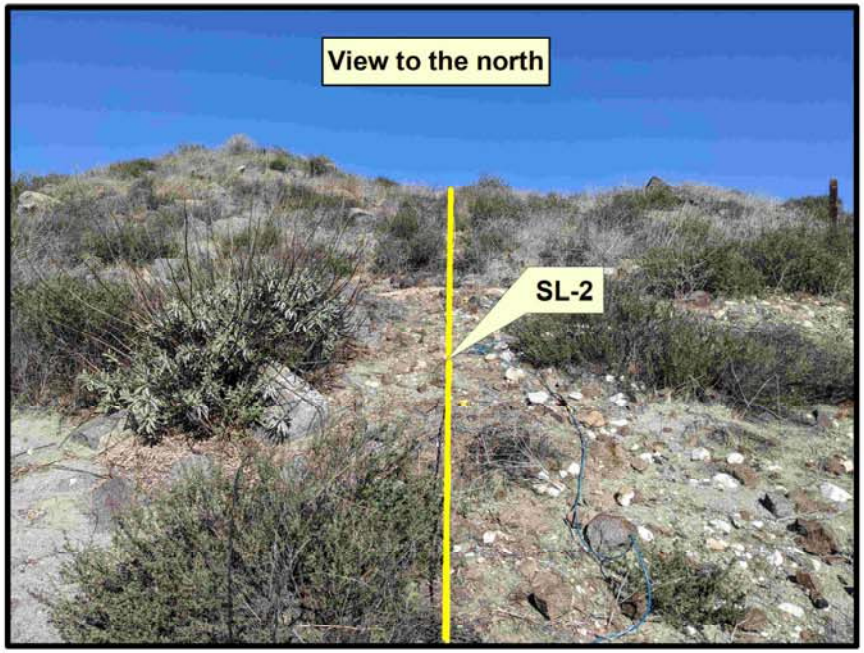
ATLAS

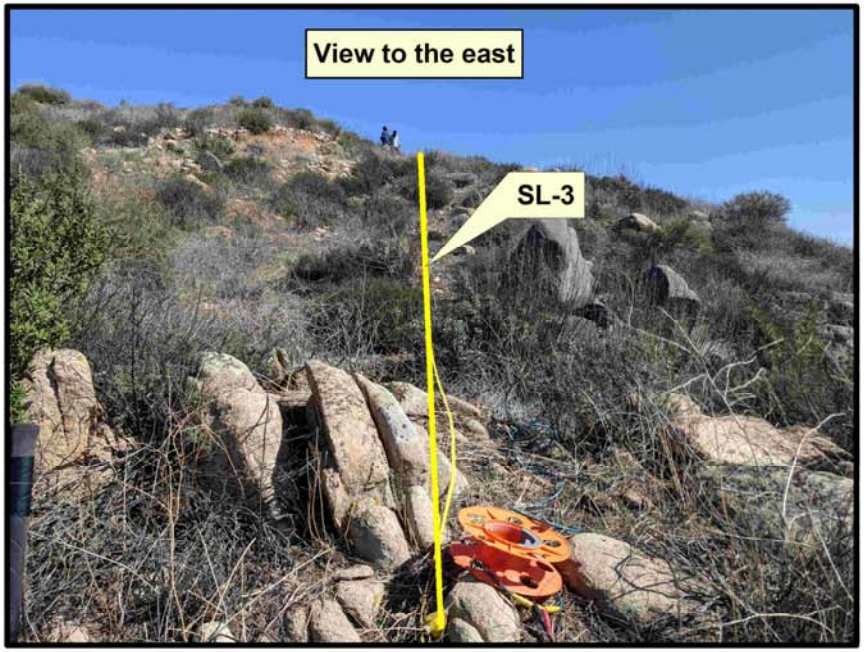
Figure 2

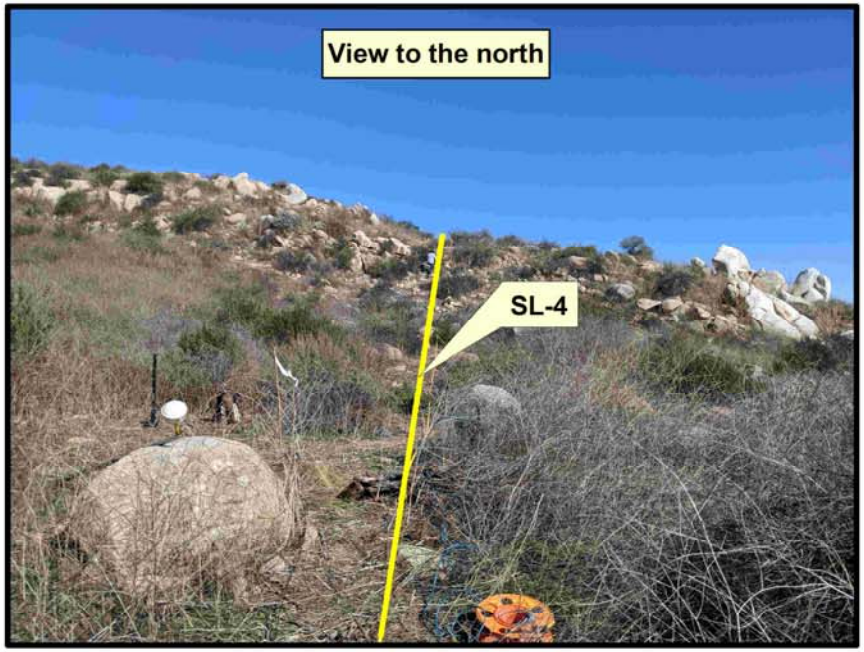
0 100 200

approximate scale in feet





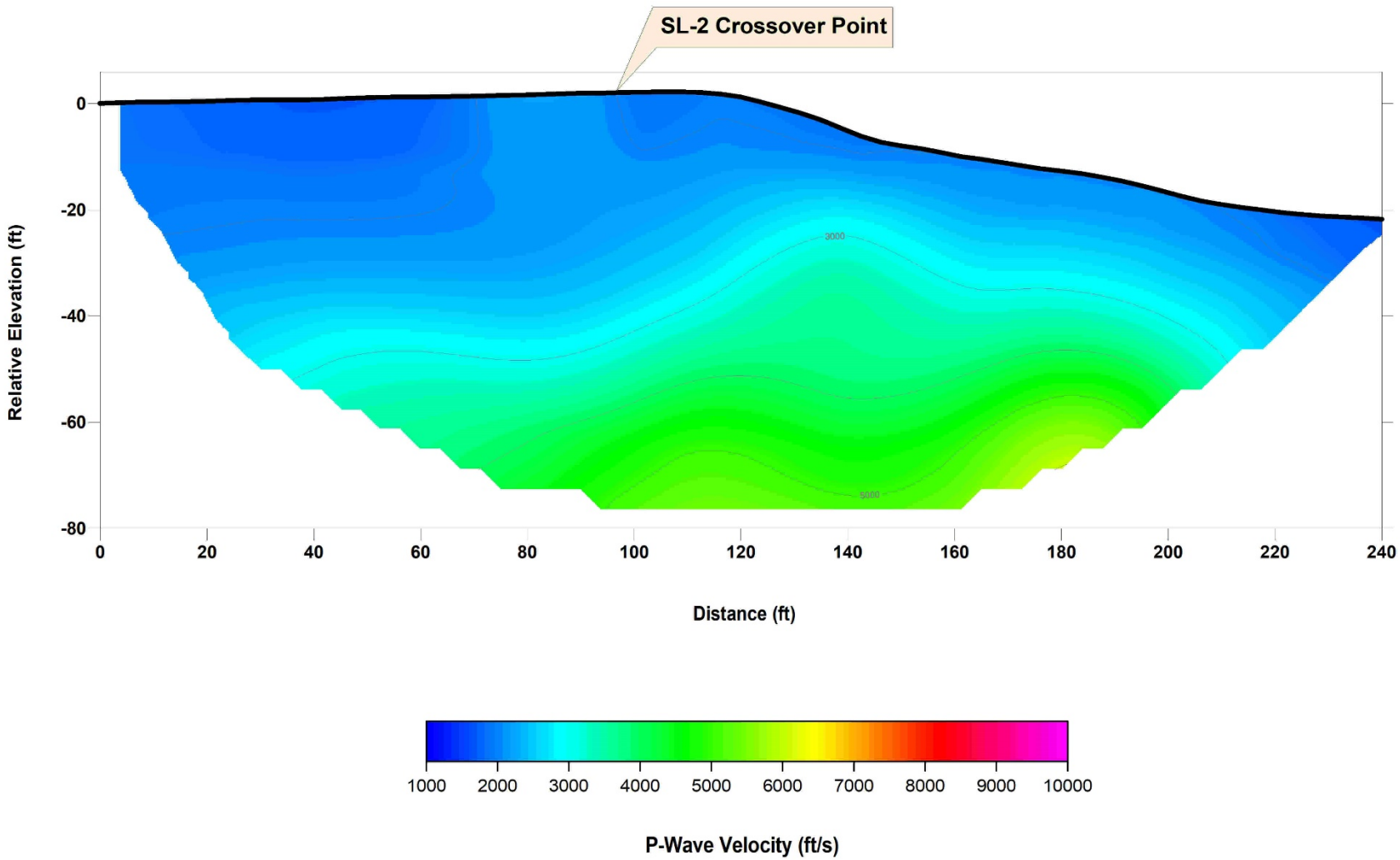




SITE PHOTOGRAPHS	Golden Meadows Tank Exploration Menifee, California	ATLAS Figure 3
	Project No.: 121058SWG / Date: 03/21	

TOMOGRAPHY MODEL

SL-1

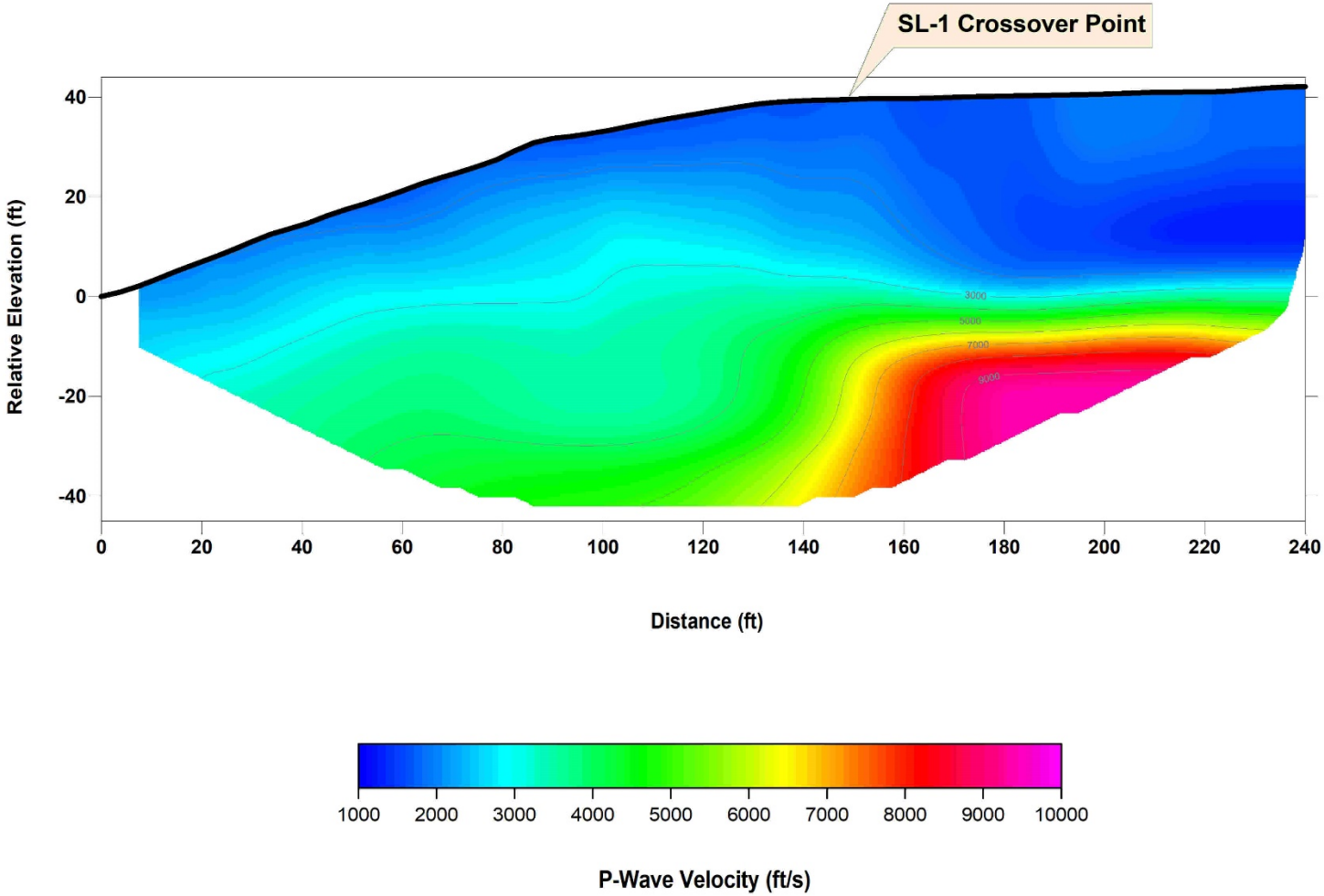


SEISMIC PROFILE SL-1	Golden Meadows Tank Exploration Menifee, California	ATLAS Figure 4a
	Project No.: 121058SWG / Date: 03/21	

Note: Contour Interval = 1,000 feet per second

TOMOGRAPHY MODEL

SL-2

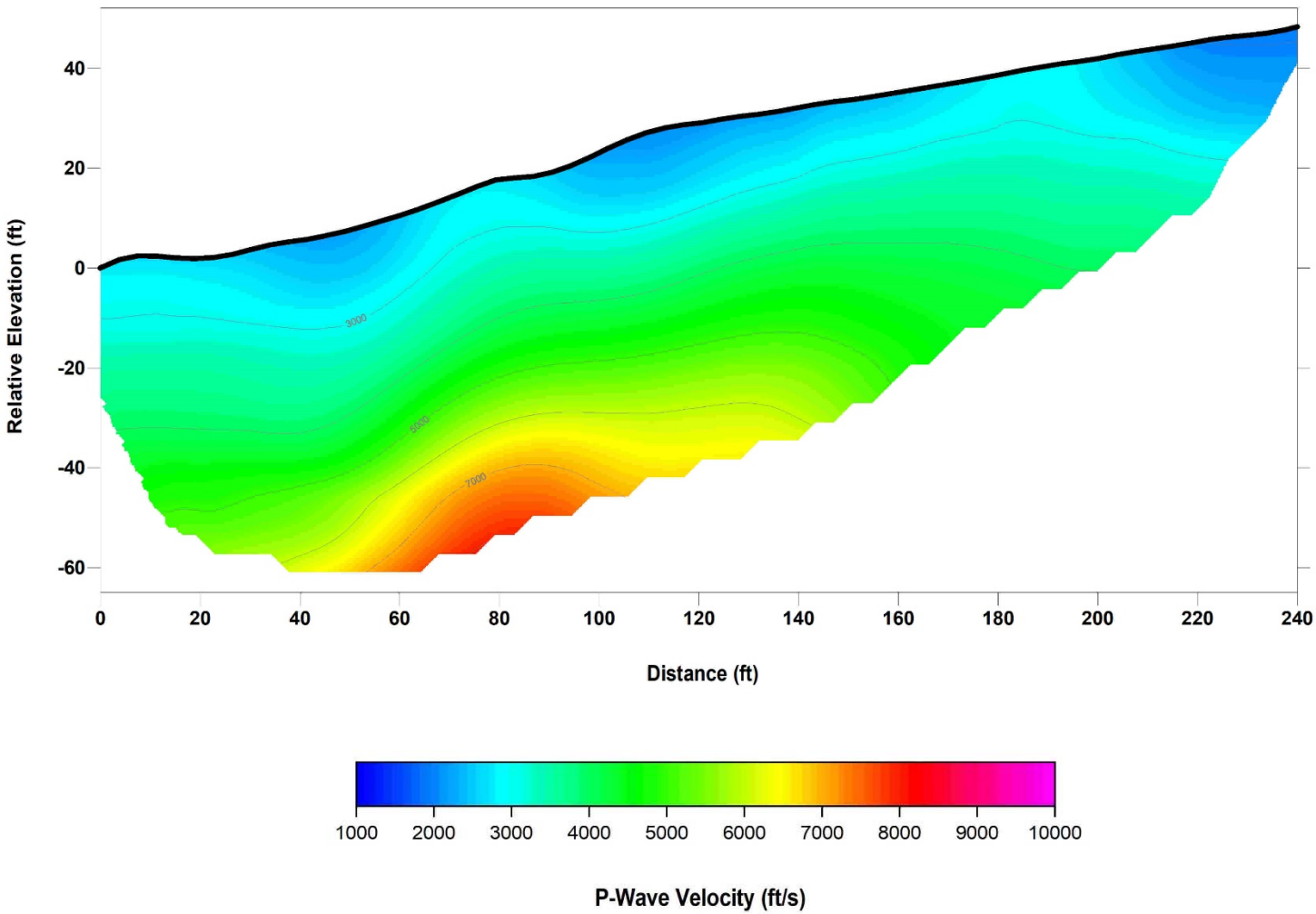


SEISMIC PROFILE SL-2	Golden Meadows Tank Exploration Menifee, California	ATLAS Figure 4b
	Project No.: 121058SWG / Date: 03/21	

Note: Contour Interval = 1,000 feet per second

TOMOGRAPHY MODEL

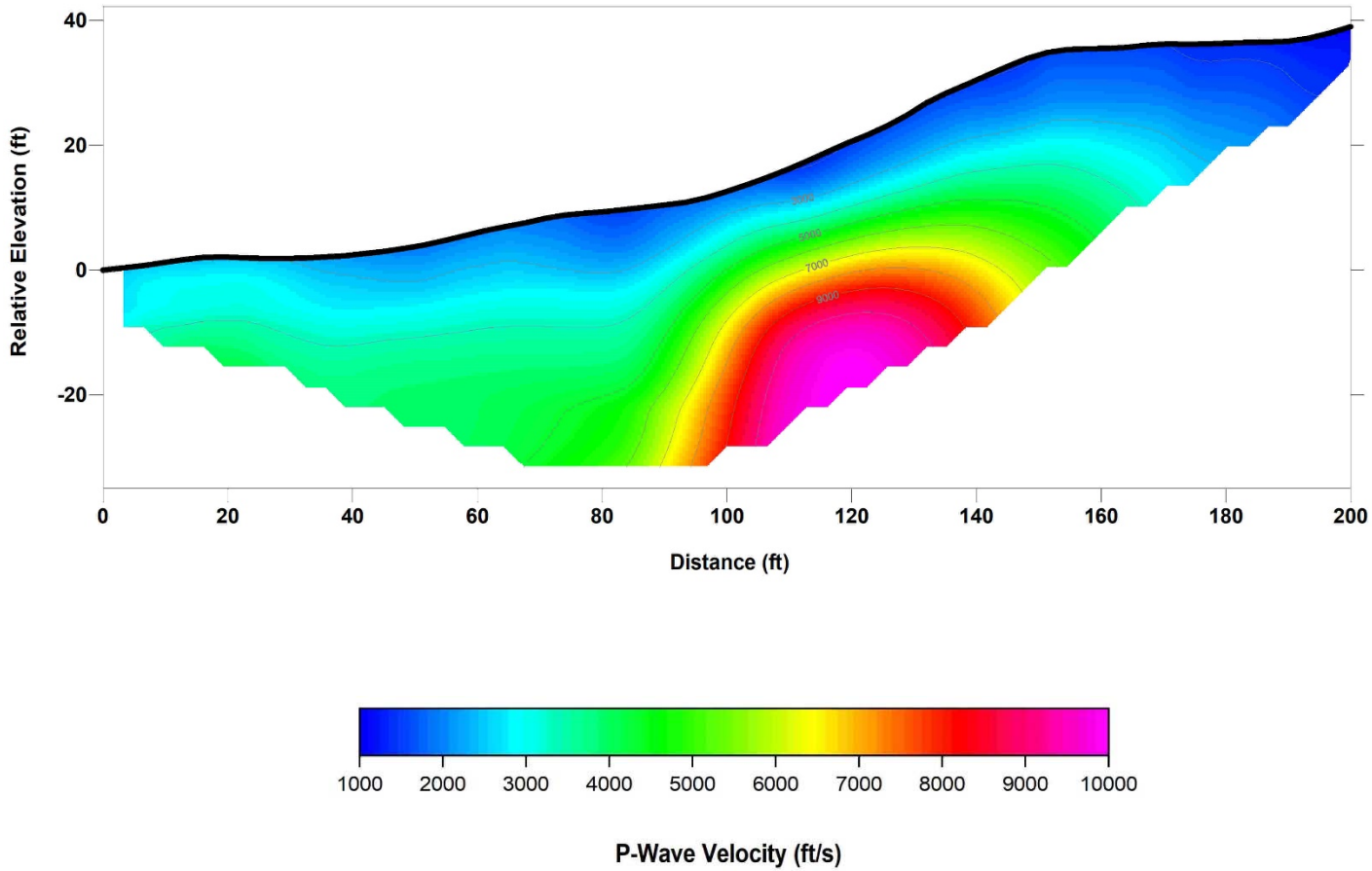
SL-3



SEISMIC PROFILE SL-3	Golden Meadows Tank Exploration Menifee, California	ATLAS Figure 4c	Note: Contour Interval = 1,000 feet per second
	Project No.: 121058SWG / Date: 03/21		

TOMOGRAPHY MODEL

SL-4



SEISMIC PROFILE SL-4	Golden Meadows Tank Exploration Menifee, California	ATLAS Figure 4d	Note: Contour Interval = 1,000 feet per second
	Project No.: 121058SWG / Date: 03/21		

APPENDIX C

Site-Specific Seismic Coefficients

Leighton

Golden Meadows Tank Site

Latitude, Longitude: 33.6555, -117.18905

Date	4/1/2021, 5:36:15 PM
Design Code Reference Document	ASCE7-16
Risk Category	IV
Site Class	B - Rock

Type	Value	Description
S_S	1.405	MCE_R ground motion. (for 0.2 second period)
S_1	0.52	MCE_R ground motion. (for 1.0s period)
S_{MS}	1.264	Site-modified spectral acceleration value
S_{M1}	0.416	Site-modified spectral acceleration value
S_{DS}	0.843	Numeric seismic design value at 0.2 second SA
S_{D1}	0.277	Numeric seismic design value at 1.0 second SA

Type	Value	Description
SDC	D	Seismic design category
F_a	0.9	Site amplification factor at 0.2 second
F_v	0.8	Site amplification factor at 1.0 second
PGA	0.597	MCE_G peak ground acceleration
F_{PGA}	0.9	Site amplification factor at PGA
PGA_M	0.537	Site modified peak ground acceleration
T_L	8	Long-period transition period in seconds
SsRT	1.405	Probabilistic risk-targeted ground motion. (0.2 second)
SsUH	1.516	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration
SsD	1.501	Factored deterministic acceleration value. (0.2 second)
S1RT	0.52	Probabilistic risk-targeted ground motion. (1.0 second)
S1UH	0.568	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration.
S1D	0.6	Factored deterministic acceleration value. (1.0 second)
PGAd	0.638	Factored deterministic acceleration value. (Peak Ground Acceleration)
C_{RS}	0.926	Mapped value of the risk coefficient at short periods
C_{R1}	0.916	Mapped value of the risk coefficient at a period of 1 s

DISCLAIMER

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APPENDIX D

General Earthwork Grading Guidelines

Leighton

LEIGHTON AND ASSOCIATES, INC.

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. 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).

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). 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.

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. 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 OSHA and Cal/OSHA requirements for safety of trench excavations.

7.2 Bedding and Backfill

All bedding and backfill of utility trenches shall be performed 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 relative compaction 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.

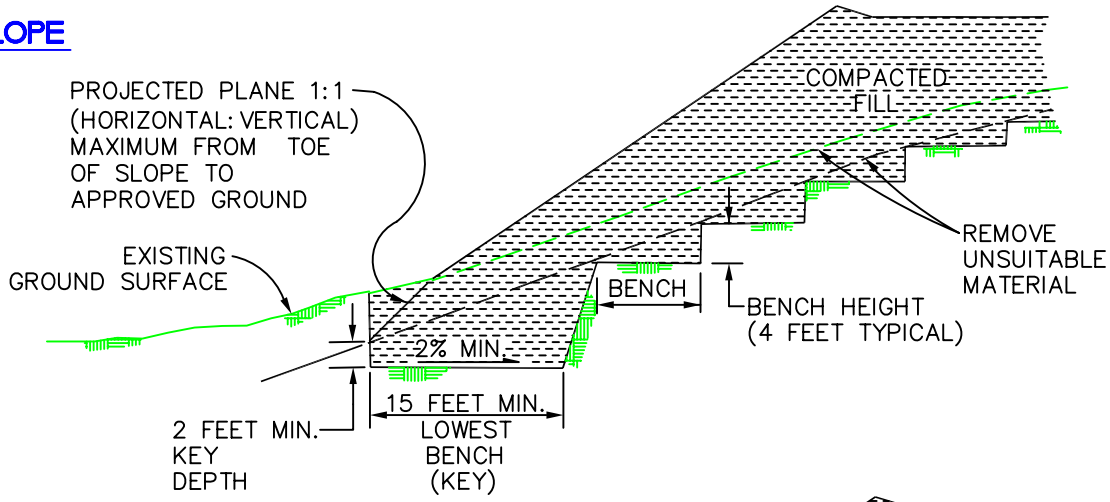
7.3 Lift Thickness

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.

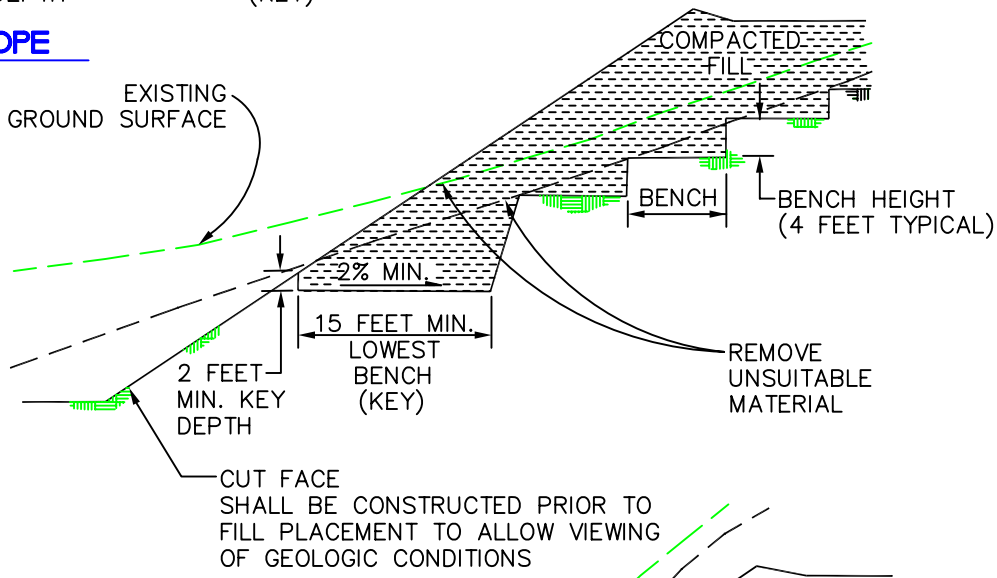
7.4 Observation and Testing

The jetting of the bedding around the conduits shall be observed by the Geotechnical Consultant.

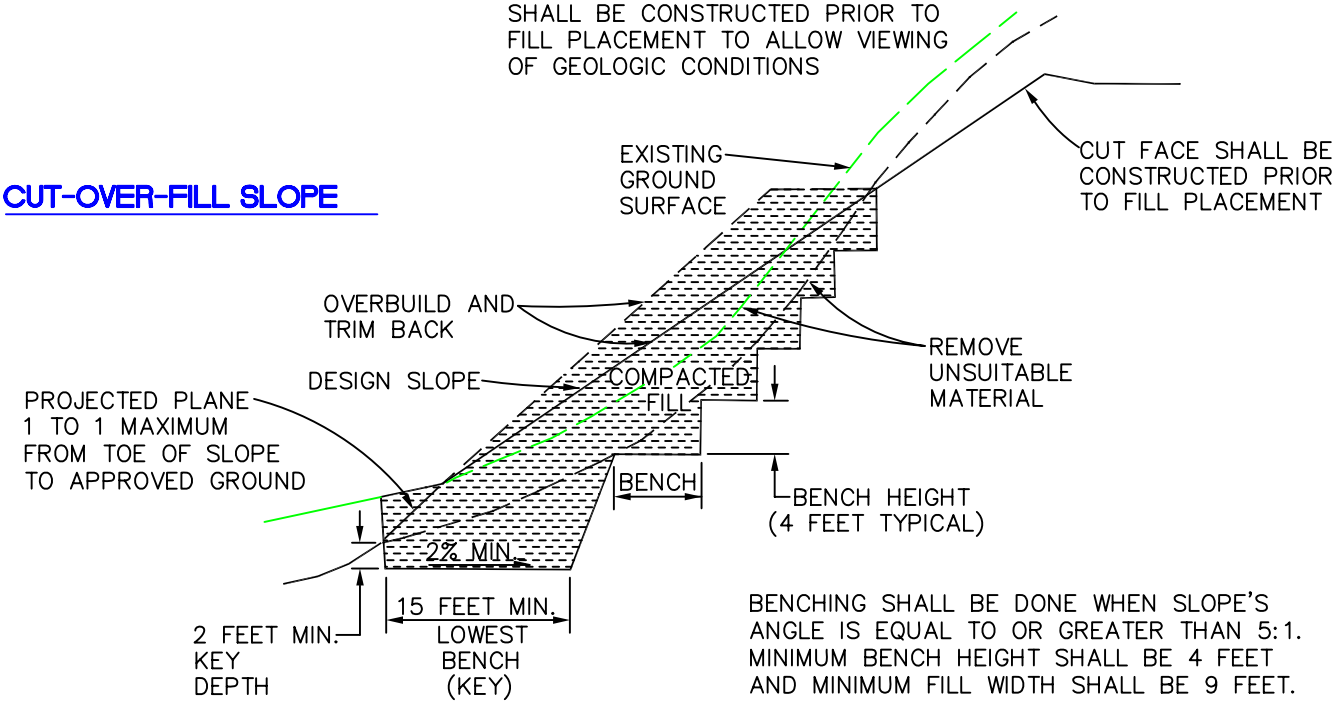
FILL SLOPE



FILL-OVER-CUT SLOPE



CUT-OVER-FILL SLOPE



BENCHING SHALL BE DONE WHEN SLOPE'S ANGLE IS EQUAL TO OR GREATER THAN 5:1. MINIMUM BENCH HEIGHT SHALL BE 4 FEET AND MINIMUM FILL WIDTH SHALL BE 9 FEET.

KEYING AND BENCHING	GENERAL EARTHWORK AND GRADING SPECIFICATIONS STANDARD DETAILS A	Leighton

APPENDIX E

ASFE - Important Information About Your Geotechnical Report

Leighton

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, you can benefit from a lowered exposure to problems associated with subsurface conditions at project sites and development of them that, for decades, have been a principal cause of construction delays, cost overruns, claims, and disputes. If you have questions or want more information about any of the issues discussed herein, contact your GBA-member geotechnical engineer. Active engagement in GBA exposes geotechnical engineers to a wide array of risk-confrontation techniques that can be of genuine benefit for everyone involved with a construction project.

Understand the Geotechnical-Engineering Services Provided for this Report

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

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

Geotechnical-Engineering Services are Performed for Specific Purposes, Persons, and Projects, and At Specific Times

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

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

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

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

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

Read this Report in Full

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

You Need to Inform Your Geotechnical Engineer About Change

Your geotechnical engineer considered unique, project-specific factors when developing the scope of study behind this report and developing the confirmation-dependent recommendations the report conveys. Typical changes that could erode the reliability of this report include those that affect:

- the site's size or shape;
- the elevation, configuration, location, orientation, function or weight of the proposed structure and the desired performance criteria;
- the composition of the design team; or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project or site changes – even minor ones – and request an assessment of their impact. *The geotechnical engineer who prepared this report cannot accept*

responsibility or liability for problems that arise because the geotechnical engineer was not informed about developments the engineer otherwise would have considered.

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

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

This Report's Recommendations Are 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 judgement and opinion to do so. Your geotechnical engineer can finalize the recommendations *only after observing actual subsurface conditions* exposed during construction. If through observation your geotechnical engineer confirms that the conditions assumed to exist actually do exist, the recommendations can be relied upon, assuming no other changes have occurred. *The geotechnical engineer who prepared this report cannot assume responsibility or liability for confirmation-dependent recommendations if you fail to retain that engineer to perform construction observation.*

This Report Could Be Misinterpreted

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

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

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

Give Constructors a Complete Report and Guidance

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

Read Responsibility Provisions Closely

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

Geoenvironmental Concerns Are 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 provide environmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated subsurface environmental problems have led to project failures.* If you have not obtained your own environmental information about the project site, ask your geotechnical consultant for a recommendation on how to find environmental risk-management guidance.

Obtain Professional Assistance to Deal with Moisture Infiltration and Mold

While your geotechnical engineer may have addressed groundwater, water infiltration, or similar issues in this report, the engineer's services were not designed, conducted, or intended to prevent migration of moisture – including water vapor – from the soil through building slabs and walls and into the building interior, where it can cause mold growth and material-performance deficiencies. Accordingly, ***proper implementation of the geotechnical engineer's recommendations will 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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