# GEOTECHNICAL EXPLORATION
# PROPOSED WATER TANK, BELLE TERRE - FORMER TTM 39883, FRENCH VALLEY AREA UNINCORPORATED RIVERSIDE COUNTY, CALIFORNIA

Prepared for

REGENT FRENCH VALLEY, LLC
11990 San Vicente Boulevard, Suite 200
Los Angeles, California 90049

Project No. 10034.003

February 23, 2015
Revised July 11, 2018

Leighton and Associates, Inc.
A LEIGHTON GROUP COMPANY

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Regent French Valley, LLC
11990 San Vicente Boulevard, Suite 200
Los Angeles, California 90049

Attention: Marinel Robinson

**Subject: Geotechnical Exploration**
**Proposed Water Tank, Belle Terre - Former TTM 39883**
**Unincorporated Riverside County, California**

In accordance with our December 19, 2014 proposal, authorized on January 7, 2015, 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 east side of former TTM 39883, located in the French Valley area of unincorporated 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 consists of moderately to steeply sloping terrain underlain by Jurassic-Aged metasedimentary rock. The site is not located within a currently designated Alquist-Priolo Special Studies Zone or a Riverside County fault zone. This proposed water tank may be founded on conventional ring-wall footings, bearing directly on undisturbed bedrock. In addition, our slope stability analyses indicate that proposed cut and natural slopes should be stable under short- and long-term conditions.

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,
LEIGHTON AND ASSOCIATES, INC.

PROFESSIONAL ENGINEER
SIMON I SAIID
2641
GEOTECHNICAL
STATE OF CALIFORNIA

Simon I. Saiid, GE 2641
Principal Engineer

REGISTERED GEOLOGIST
ROBERT F. RIHA
No. 1921
CERTIFIED ENGINEERING GEOLOGIST
STATE OF CALIFORNIA

Robert F. Riha, CEG 1921
Senior Principal Geologist

Distribution: (1) addressee (PDF via email)

41715 Enterprise Circle N., Suite 103 ▪ Temecula, CA 92590-5661
951.296.0530 ▪ Fax 951.296.0534

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# 1.0 INTRODUCTION

## 1.1 Site Location and Description

The proposed water tank is located on a ridge top along the eastern portion of former tentative tract map 29883, northwest of the intersection of the existing Fields Drive and Rebecca Street, unincorporated Riverside County, California *(see Site Location Map, Figure 1)*. This site is currently a vacant/undeveloped hilltop/ridgeline that slopes moderately to steeply in easterly and westerly directions. Site access is by a steep dirt road located to the east of the hillside. As shown on Figure 1, the site is located along the east side of the San Diego Aqueduct.

## 1.2 Proposed Water Tank

We understand that an approximately 200-foot diameter by 40-foot high welded steel water tank is proposed to be constructed at Site 1 as depicted on the provided conceptual grading plan (see Figure 3). Based on this plan, a desired pad elevation of 1,585 feet and cut slopes of up to 45 feet in height may be constructed to create the required pad. Site access is expected to extend from Fields Drive and will also require cut slopes up to 45 feet in height and fill slopes of up to 10 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 January 19, 2015, 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. Logs of two test pits from a previous exploration on this site are also included in Appendix A.

- **Geophysical Survey**: Three (3) seismic refraction lines were performed by our sub-consultant Southwest Geophysics, Inc. (SGI). 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 (such as previous survey). 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. Two 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. We performed a site-specific probabilistic seismic hazard analyses (PSHA) and developed site-specific response spectra to be used by the tank designer (see Appendix C). In addition, slope-stability analyses were performed for most critical slopes and results are presented in Appendix D.

- **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 10 miles to the southwest.

## 2.2 Site Geology

As regionally mapped on Figure 2, the site is underlain by metasedimentary rock formation, locally known as Bedford Canyon Formation. Our field exploration indicates that this Formation 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 (clayey/silty sand) possess a low expansion potential (EI=29). Test results are included in Appendix A.

2.2.2 Jurassic Metasedimentary Bedrock: Metamorphic Bedrock locally known as Bedford Canyon Formation is exposed on existing cuts and on steeper hillsides throughout the site. This slate-type metasedimentary bedrock is generally dark gray in color and well-foliated, or structured. Where seen in our test holes, the “near-surface” bedrock is moderately weathered and generally broke into small fragments (<12 inches) upon excavation. However, a very resistant steeply dipping quartzite bed is observed to cross the site and may produce some oversize fragments (> 12 inches) upon excavation.

Foliation within the bedrock is generally consistent across the site. Foliation, or relict bedding planes, follow a consistent northwest trend across the site and dip very steeply to both the north and south. Such an orientation produces “bedding” planes that are expected to dip more steeply than the proposed cut slope surfaces and should therefore be supported in the proposed west and south-facing cut slopes. However, 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 45 feet may be required to create the tank pad and up to 20 feet for the proposed access road.

Based on our seismic refraction survey data performed by Southwest Geophysics, Inc. (Appendix B), rippable bedrock using Caterpillar D-9 dozer with a single shank should be anticipated to a depth of 10 to 25 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 Figure 4. The relatively shallow and hard rock zones (Green Color – Figure 4) are likely due to resistant quartzite bed, buried corestones/ remnant boulders, dikes, and/or less weathering.

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 9.8 miles southwest of the site and the Anza Segment of the San Jacinto Fault Zone is located approximately 12.5 miles northeast of the site (see Appendix C – EQFAULT program output). Historical records of seismic activity in the region indicate that a peak, horizontal ground acceleration (PHGA) of 0.21g has not been exceeded at this site in recent history and closest fault is approximately 5.7 miles away from the site (see Appendix C – EQSEARCH program output).

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 1997 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. Results of our site-specific ground motion analyses are presented in Section 3.2 of this report.

## 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

Our slope stability analyses were performed using *SLIDE* 6.0 software licensed to Leighton. Both static and pseudo-static analyses were performed. Our cross-sectional model was selected to represent worst case scenario or steepest/highest cut slope for circular type failures to simulate potential failure through surficial weathered rock. Analyses output and sections are included in Appendix D. Soil parameters used in our analysis are generally based on results of our laboratory direct shear testing and published data for similar soil types. A summary of soil parameters used in our analyses is tabulated below:

**Table 1. Slope Stability Analyses Soil Parameters**

| Soil Description | Shear Strength | | Moist Unit Weight (pcf) | Source/Reference |
|---|---|---|---|---|
| | Friction (Degrees) | Cohesion (psf) | | |
| Surficial Soils (SM/ML) | 30 | 100 | 120 | Laboratory direct-shear testing (remolded samples) and published data |
| metasedimentary bedrock - weathered | 37 | 200 | 130 | |

Stability analyses results are summarized in the following subsections:

2.7.1 Cut Slopes Stability: As presented in Appendix D, proposed cut slopes up to 45 feet in height at 2:1 and 1.5:1 (horizontal:vertical) gradients are considered grossly stable for static and pseudo-static conditions. Compacted fill slopes up to 15 feet in height at 2:1 (horizontal:vertical) gradients are also considered grossly stable for static and pseudo-static conditions. Cut slopes, especially the steeper 1.5:1 slopes, should be observed by an engineering geologist during grading to verify jointing or fracture patterns and recommend remedial measures, if needed.

2.7.2 Natural Slopes Stability: Natural slopes were also evaluated for short- and long-term stability incorporating the surcharge load exerted by the proposed tank. The results of our evaluation yielded adequate factor of safety for both static and pseudo-static conditions. Results of our analyses are presented in Appendix D.

# 3.0 CONCLUSIONS AND RECOMMENDATIONS

Based on results of this geotechnical exploration, the proposed tank site pad is underlain by dense metasedimentary rock formation, locally known as Bedford Canyon Formation. 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.

This proposed above-grade potable water tank may be founded on conventional ring-wall footings, bearing directly on undisturbed dense rock. In addition, our slope stability analyses indicate that proposed cut slopes should be grossly stable at 2:1 and 1.5:1 (horizontal:vertical) gradients, and fill slopes should be constructed no-steeper-than 2:1 (horizontal:vertical).

Our geotechnical recommendations for design and construction of this proposed water tank are presented in the following sections.

## 3.1 Tank Foundation Location

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 <u>Site Preparation:</u> Based on proposed grading concept, the tank pad is expected to expose dense metasedimentary 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 <u>Fill Placement and Compaction:</u> Onsite low expansive (EI<51) 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 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 7-10. Our seismic hazard evaluation also includes development of site specific ground motions in terms of peak ground accelerations (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 7-10, Chapters 11 and 21. At the discretion of the designing Structural Engineer, either of the following seismic design methodologies/ parameters can be used.

3.3.1 2016 CBC Seismic Parameters: Seismic design parameters per the 2016 Edition of the California Building Code (CBC) are provided in Table 2 below. These seismic coefficients were calculated utilizing an interactive program on current United States Geological Survey (USGS) website using ASCE 7-10 procedures (referred to as USGS General Procedure). Based on our site specific seismic refraction survey, this site is classified as a Class **B** site:

**Table 2. 2013 CBC Site-Specific Seismic Parameters**

| 2013 CBC Site-Specific Seismic Design Parameters | Value |
|---|---|
| Site Longitude (decimal degrees) | -117.0760 |
| Site Latitude (decimal degrees) | 33.6198 |
| **Site Class Definition** | **B** |
| Mapped Spectral Response Acceleration at 0.2s Period, $S_s$ | 1.50 |
| Mapped Spectral Response Acceleration at 1s Period, $S_1$ | 0.60 |
| *Short Period Site Coefficient at 0.2s Period, $F_a$* | *1.0* |
| *Long Period Site Coefficient at 1s Period, $F_v$* | *1.0* |
| Adjusted Spectral Response Acceleration at 0.2s Period, $S_{MS}$ | 1.50 |
| Adjusted Spectral Response Acceleration at 1s Period, $S_{M1}$ | 0.60 |
| **Design Spectral Response Acceleration at 0.2s Period, $S_{DS}$** | **1.00** |
| **Design Spectral Response Acceleration at 1s Period, $S_{D1}$** | **0.40** |
| **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 0.5g.

3.3.2 Site-Specific Probabilistic Seismic Hazard Analysis: A site-specific probabilistic seismic hazard analysis was also performed using the computer

program EZ-FRISK (Risk Engineering, 2003) to estimate peak horizontal ground acceleration (PHGA) that could occur at the site, and to develop design response spectra. Various probabilistic density functions were used in this analysis to assess uncertainty inherent in these calculations with respect to magnitude, distance and ground motion. An averaging of the following four next-generation attenuation relationships (NGAs) was used with equal weights to calculate site-specific PHGA and spectra:

- Abrahamson-Silva (2008)
- Boore-Atkinson (2008),
- Campbell-Bozorgnia (2008), and
- Chiou-Youngs (2007)

The MCE Peak Horizontal Ground Acceleration (PHGA) for various probability of exceedance is presented in Table 3 below:

**Table 3. Probabilistic PHGA Vs. Probability of Exceedance**

| Return Period (years) | Definition | Peak Horizontal Ground Acceleration (g) | Reference |
|---|---|---|---|
| 1237 | 2% probability of exceedance in 25 years | 0.50 | Appendix C |
| 2475 | 2% probability of exceedance in 50 years | 0.61 | Appendix C |
| 3712 | 2% probability of exceedance in 75 years | 0.69 | Appendix C |
| 4950 | 2% probability of exceedance in 100 years | 0.74 | Appendix C |
| 475 | 10% probability of exceedance in 50 years | 0.40 | App. C, C-1 |
| 975 | 10% probability of exceedance in 100 years | 0.46 | Appendix C |
| 2475 | 2% probability of exceedance in 50 years | 0.50 | $PGA_m$ –USGS General Procedure, |
| 2475 | 2% probability of exceedance in 50 years | 0.54 | PSH Deaggreg. |

Probabilistic seismic hazard-analysis acceleration values and probabilities should only be considered reasonable best estimates. All of the influences affecting attenuation and occurrence rates are not yet known. Furthermore, there are uncertainties in every parameter used to obtain such results. At the present time, there is no test available to verify validity of these ground motions and probability data. Therefore, significant deviations from indicated values are possible due to geotechnical and geological uncertainties and other site-specific conditions.

3.3.3 <u>Site-Specific Response Spectra:</u> Site-specific response spectra for this proposed site was developed based on a uniform-hazard approach. The uniform-hazard approach assumes that the same level of hazard is uniformly applied to the entire response spectra. Spectral values for the DE and MCE

events were computed using the same probabilistic analysis approach described in previous section. Near-source and directivity effects were included using techniques proposed by Sommerville et al. (1997) and Abrahamson (2000). Response spectra values were calculated for 5% damping using the EZ-FRISK program.

The results of this analysis are presented in Appendix C. In accordance with ASCE 7-10, the site-specific Maximum Considered Earthquake ($MCE_R$) was derived as the lesser of the probabilistic and deterministic $MCE_R$ (see Figures C-2 and C-3, Appendix C) and the site-specific design response spectrum curve is shown on Figure C-1, Appendix C. The MCE and DE seismic coefficients listed in Table 4 below are slightly lower than those derived from the USGS general procedure (Table 2). We recommend that the values presented below be used in structural design of the tank. However, the structural engineer may consider the values included in Table 2 for a more conservative approach.

**Table 4. Seismic Coefficients per ASCE Chapter 21**

| Seismic Coefficient | Design Value (g) |
|---|---|
| Spectral Response Acceleration at 0.2s Period, $S_s$ | **1.33** |
| Spectral Response Acceleration at 1s Period, $S_1$ | **0.53** |
| Design Spectral Response Acceleration at 0.2s Period, $S_{DS}$ | **0.89** |
| Design Spectral Response Acceleration at 1s Period, $S_{D1}$ | **0.35** |

* g- Gravity acceleration

Since the probabilistic spectrum is less than the deterministic spectrum, the site is governed by the probabilistic analysis and a moment magnitude of 6.85Mw is recommended for this site.

## 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 metasedimentary 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 4,000 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 500 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 appurtenant 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 5. Static Lateral Earth Pressures**

| Conditions | Equivalent Fluid Weight (pcf) | | |
|---|---|---|---|
| | Level Backfill[2] | 2:1 Slope Backfill | 1.5:1 Slope Backfill |
| Active (cantilever) | 36 | 53 | 70 |
| At-Rest (braced) | 55 | 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, assumed due to the presence of expansive clays.

**Table 6. 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).

## 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 7. 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 8. 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 9 (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 severely-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 9. Soil Corrosivity Test Results Summary**

| Boring Number | Sample Depth (feet) | Sulfate Content (ppm) | Chloride Content (ppm) | pH | Resistivity (ohm-cm) |
|---|---|---|---|---|---|
| S-1 | 0 to 2 | **80** | **122** | 6.6 | 2,080 |



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# 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 rock within upper 10 feet BGS 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 30 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 Regent French Valley, LLC 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 Regent French Valley, LLC, 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.

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**Aerial Photos Reviewed:**

9-11-97 C116-40 204-205

10-12-90 90 205 – 140

2-8-88 88045 – 38-39

5-9-67 1HH – 98-99

5-23-49 10F – 86-88


| | |
|---|---|
| Project: 10034.003 | Eng/Geol: SIS/RFR |
| Scale: 1 " = 2,000 ' | Date: January 2015 |

Base Map: ESRI ArcGIS Online 2015
Thematic Information: Leighton
Author: Leighton Geomatics (mmurphy)

# SITE LOCATION MAP

French Valley North - Belle Terre Tanksite
French Valley Area, California

Figure 1

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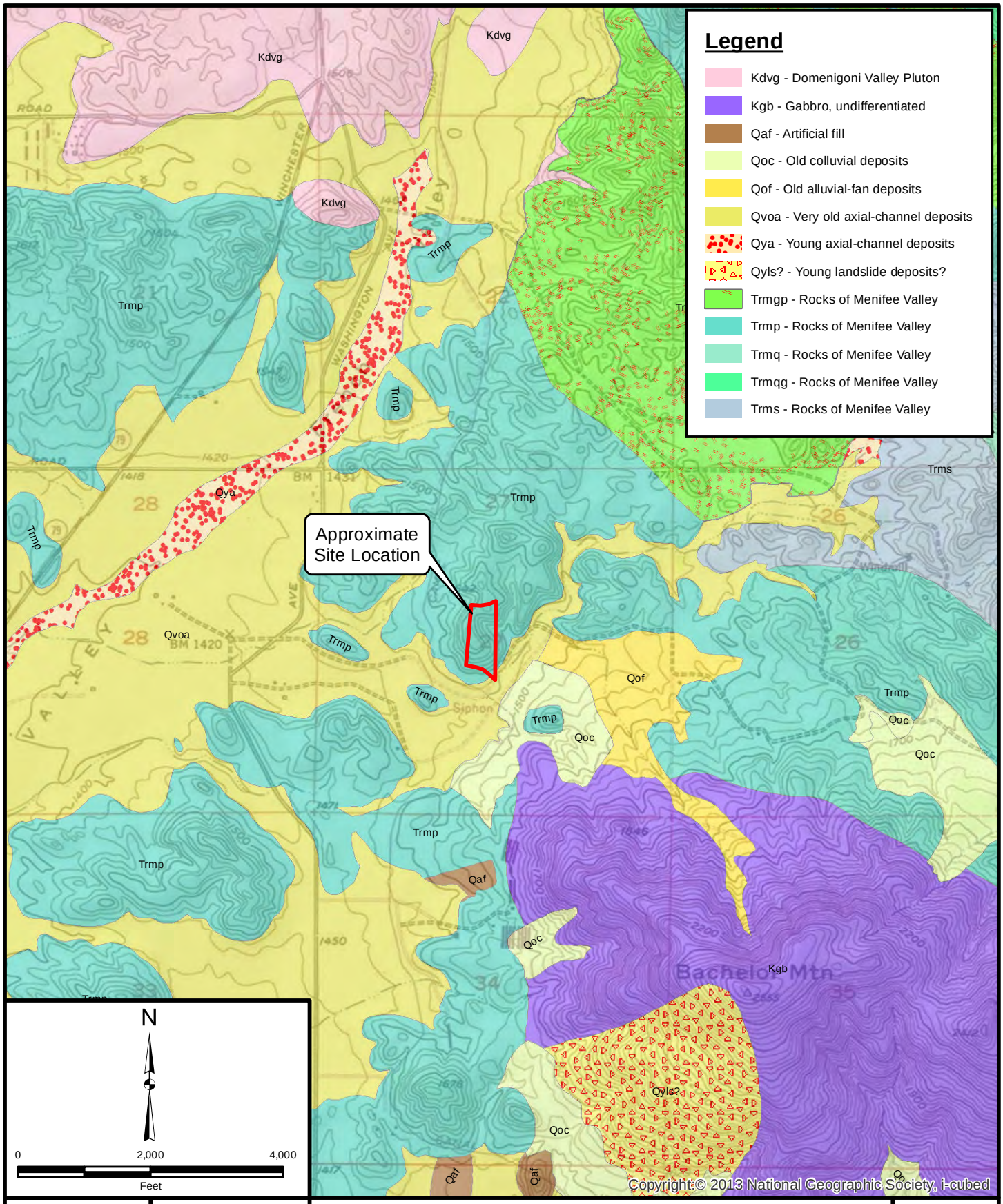


| | |
|---|---|
| Project: 10034.003 | Eng/Geol: SIS/RFR |
| Scale: 1 " = 2,000 ' | Date: January 2015 |

Base Map: ESRI ArcGIS Online 2015
Geology: USGS, 2006, Geologic map of the San Bernardino Sand anta Ana 30'x60' quadrangles, California, Version 1.0, Open File Report 2006-1217
Author: Leighton Geomatics (mmurphy)

# REGIONAL GEOLOGY MAP

French Valley North - Belle Terre Tanksite
French Valley Area, California

Figure 2

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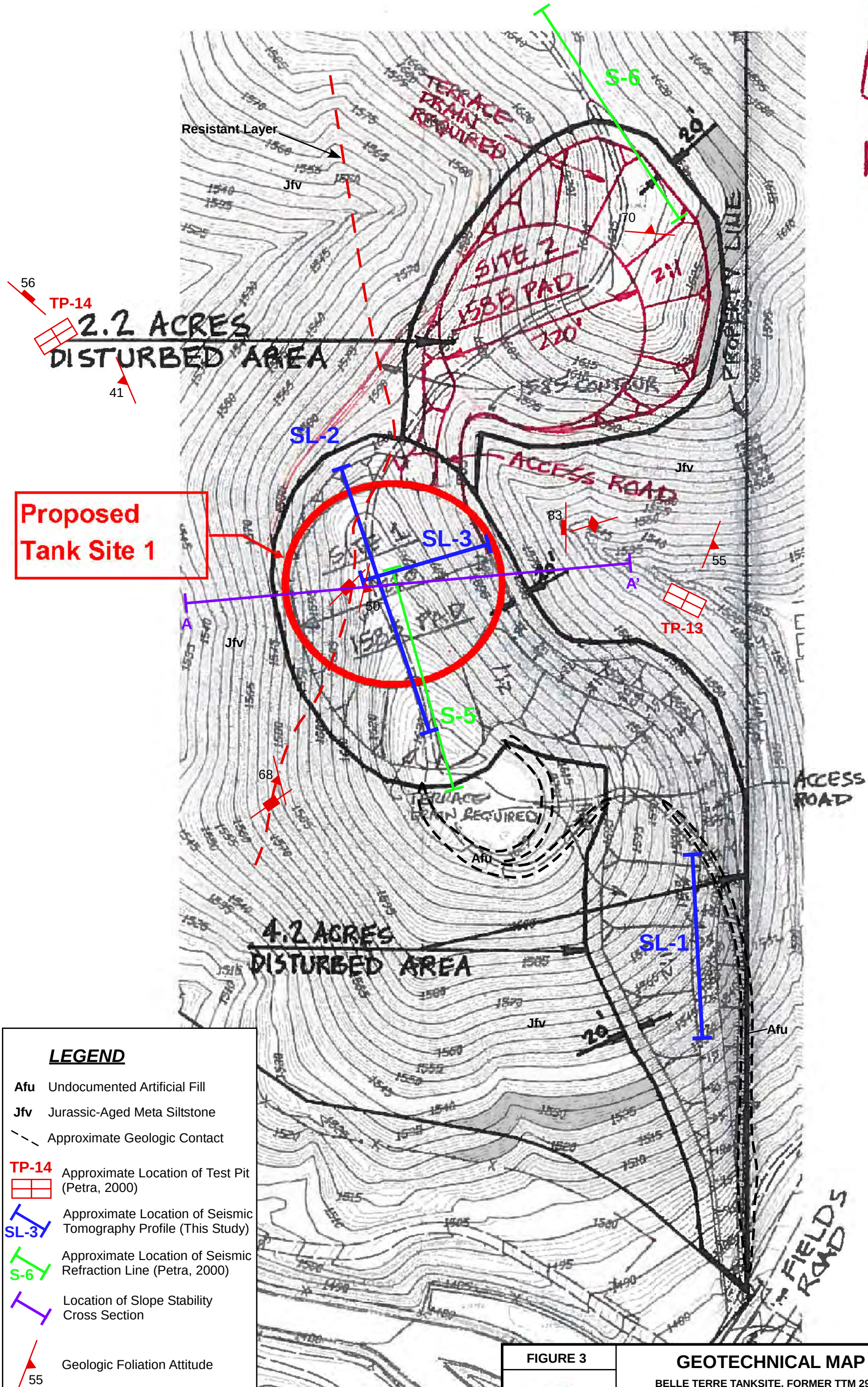
Map Saved as V:\drafting\10034\003\Maps\10034.003_Fig2_RGM_2015-01-26.mxd on 1/26/2015 11:21:21 AM




## LEGEND

| | |
|---|---|
| Afu | Undocumented Artificial Fill |
| Jfv | Jurassic-Aged Meta Siltstone |
| | Approximate Geologic Contact |
| TP-14 | Approximate Location of Test Pit (Petra, 2000) |
| SL-3 | Approximate Location of Seismic Tomography Profile (This Study) |
| S-6 | Approximate Location of Seismic Refraction Line (Petra, 2000) |
| | Location of Slope Stability Cross Section |
| 55 | Geologic Foliation Attitude |
| 83 | Geologic Joint Attitude |

FIGURE 3

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# GEOTECHNICAL MAP

BELLE TERRE TANKSITE, FORMER TTM 29883
RIVERSIDE COUNTY, CALIFORNIA

| | |
|---|---|
| Proj: 10034.003 | Eng/Geol: SIS/RFR |
| Scale: 1" = 100' | Date: 01/2015 |

Drafted By: JTD | Checked By: