ADDENDUM NO. 3
TO THE REQUEST FOR BIDS
CO# 1602-U
HOLLY DRIVE RESERVOIR – PHASE II

September 2, 2020

TO: ALL PROSPECTIVE BIDDERS

This Addendum forms a part of the Request for Bid (RFB) for the above-identified project and modifies the original RFB as noted below. Portions of the RFB, not specifically mentioned by the Addendum, remain in force.

Where text within the RFB is to be revised, within this addenda, for clarity, new text to be inserted is shown in underlined red text and old text to be deleted is shown in ~~strikeout~~.

ITEM 1: ADD

Appendix 'A' containing the design related Geotechnical Report dated December 5, 2016 and addendum to the Geotechnical Report dated September 11, 2018.

Attachments: 1. Appendix A

c: Project File

APPENDIX ‘A’

DESCRIPTIONS:

1) **Addendum dated September 11, 2018** for Preliminary Geotechnical Investigation, Proposed SAWCO New 100,000 Gallon Water Tank, Upland, San Bernardino County, California, Project No. 63314.12, dated December 5, 2016, prepared by LOR Geotechnical Group, Inc.

2) LOR Geotechnical Group, Inc., Preliminary Geotechnical Investigation, Proposed SAWCO New 100,000 Gallon Water Tank, Upland, San Bernardino County, California, Project No. 63314.12, dated December 5, 2016

September 11, 2018

TKE Engineering, Inc.
2305 Colton Avenue
Riverside, California 92507

Project No. 63314.11

Attention: Mr. Christopher Deiter, P.E.

Subject: Proposed MSE Walls, Proposed SAWCO New 100,000 Gallon Water Tank, Upland, San Bernardino County, California.

Reference: LOR Geotechnical Group, Inc., 2016, Preliminary Geotechnical Investigation, Proposed SAWCO New 100,000 Gallon Water Tank, Upland, San Bernardino County, California, Project No. 63314.12, dated December 5, 2016.

As requested by you, we have prepared this letter to address geotechnical considerations pertaining to the proposed use of MSE walls at the subject site during construction of the subject new 100,000 gallon water tank pad. A question had been raised by project engineer Hai Liao, of Knapp Associates, Inc., regarding the required horizontal setback for proposed footings.

Our above referenced report does recommend a horizontal setback from slope faces of 5 feet, as is typical for standard foundations. However, MSE walls do not include footings and the walls themselves does not impart a significant load upon the materials outside of or away from the base of the wall. Therefore, the 5 horizontal feet to daylight requirement does not apply to the proposed construction.

Drawings (untitled) provided to us this date by Mr Liao indicate that the MSE walls will be constructed such that the lower approximately two feet of the walls will be embedded relative to the lowest adjacent natural grade. Provided that the excavation for the wall exposes competent bedrock material, this depth should be adequate. However, there are some undocumented fill materials known to be present within the site and deeper excavation may be required locally in order to ensure that the wall is founded entirely upon competent bedrock.

Should you have any questions regarding this report, please do not hesitate to contact this firm at your convenience.

Respectfully submitted,
LOR Geotechnical Group, Inc.

REGISTERED PROFESSIONAL ENGINEER
JOHN P. LEUER
NO. 2030
EXPIRATION DATE
09/30/19
GEOTECHNICAL
STATE OF CALIFORNIA

John P. Leuer, GE 2030
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PRELIMINARY GEOTECHNICAL INVESTIGATION
PROPOSED SAWCO NEW 100,000
GALLON WATER TANK
CITY OF UPLAND
SAN BERNARDINO COUNTY, CALIFORNIA

PROJECT NO. 63314.1
DECEMBER 5, 2016

Prepared For:

TKE Engineering, Inc.
2305 Chicago Avenue
Riverside, California 92507

Attention: Mr. Christopher Deiter, P.E.

LOR GEOTECHNICAL GROUP, INC.
Soil Engineering ▲ Geology ▲ Environmental

December 5, 2016

TKE Engineering, Inc.
2305 Colton Avenue
Riverside, California 92507

Project No. 63314.1

Attention: Mr. Christopher Deiter, P.E.

Subject: Preliminary Geotechnical Investigation, Proposed SAWCO New 100,000 Gallon Water Tank, Upland, San Bernardino County, California

LOR Geotechnical Group, Inc. is pleased to present this report summarizing our geotechnical investigation for the subject project. This report was based upon a scope of services generally outlined in our Proposal dated November 3, 2016, and other written and verbal communications with you.

In summary, it is our opinion that the site can be developed from a geotechnical perspective, provided the recommendations presented in the attached report are incorporated into design and construction. The following executive summary reviews some of the important elements of the project. However, this summary should not be solely relied upon.

The data developed during this investigation indicates that removals on the order of 1 to 3 feet, and possibly more locally, are anticipated across the area of improvements, including the water tank and retaining wall. The actual depths of removals should be determined during the grading operation by observation and in place density testing. Very low expansive soils and soils with negligible sulfate content were encountered on the site.

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INTRODUCTION

During November and December of 2016, a Preliminary Geotechnical Investigation was performed by LOR Geotechnical Group, Inc. for a proposed 100,000 gallon water storage tank to be constructed within the central portion of APN: 1003-561-06-0000 southeast of an existing water tank and north of Holly Drive in the Upland area of San Bernardino County, California. The purpose of this investigation was to provide a technical evaluation of the geologic setting of the site and to provide geotechnical design recommendations for the proposed construction. The scope of our services included:

- Review of available pertinent geotechnical literature, reports, maps, and agency information pertinent to the study area;
- Geologic field reconnaissance mapping to verify the areal distribution of earth units and significance of surficial features as compiled from documents, literature, and reports reviewed;
- A subsurface field investigation to determine the physical soil/bedrock conditions pertinent to the proposed improvements;
- Laboratory testing of selected soil/bedrock samples obtained during the field investigation;
- Development of geotechnical recommendations for site grading and foundation design; and,
- Preparation of this report summarizing our findings, and providing conclusions and recommendations for site development.

PROJECT CONSIDERATIONS

To orient our investigation at the site, a grading plan and cross-sections for Alternative 3, were prepared and provided for our use. A plan view drawing was utilized as a base for our Enclosure A-2 within Appendix A. We understand that the 100,000 gallon tank will be 10 feet in height, 42 feet in diameter, and at the approximate same elevation, 2,388 feet, as the adjacent existing tank. The steel tank will be constructed upon fill with retaining walls up to approximately 20 feet high proposed for construction to the northeast, southeast and southwest of the tank. These walls will continue to the northwest around the existing 32 foot diameter water tank which, at a later date, will be demolished and replaced by a 42 foot diameter tank. That phase of work will require the construction of a retaining wall to support the up-gradient slope. Excluding any over-excavations and/or removals, fills of up to approximately 19 feet are anticipated to reach the proposed design finish grade for the new tank. Minimal cut, only in the area near the existing tank and for the proposed retaining to the northwest, will be required.

It is our understanding that the foundation for the proposed tank may consist of a concrete ringwall footing approximately 2 to 4 feet in both width and thickness which may extend 1 to 2 inches above the ground surface. Also, preliminary plans for the proposed retaining walls include reinforced concrete cantilever walls and Mechanically Stabilized Earth (MSE) walls. Moderate foundation loads are expected with such structures.

EXISTING SITE CONDITIONS

The site of the new proposed water tank is located southeast of the currently existing water tank located in the central portion of APN: 1003-561-06-0000, on the north side of Holly Drive, in the northern portion of the City of Upland, California. The property is located upon an elevated ridgeline that descends from northwest to southeast. The site of the proposed new tank consists of largely natural, undisturbed ground that slopes at a gradient of approximately 2.5:1 (horizontal to vertical) and is covered by a light to moderate growth of brush. A paved road that extends to the northwest from Holly Drive provides access to the existing, fence enclosed, water tank on the northwest side of the site. The approximate location of the site is shown on the attached Index Map, Enclosure A-1, within Appendix A.

SUBSURFACE FIELD INVESTIGATION

Our subsurface field exploration program was conducted on November 18, 2016. The work consisted of excavating a total of three exploratory trenches using a tractor mounted backhoe equipped with a 24-inch bucket. The approximate locations of our exploratory trenches are presented on the enclosed Plot Plan, Enclosure A-2, in Appendix A. The subsurface conditions encountered in the exploratory trenches were logged by a geologist from this firm. The trenches were excavated to depths of approximately 3.5 to 5 feet below the existing ground surface. Bulk samples were obtained at selected intervals and returned to our geotechnical laboratory in sealed containers for further testing and evaluation.

A detailed description of the subsurface field exploration program and the trench logs are presented in Appendix B.

LABORATORY TESTING PROGRAM

Selected soil and bedrock samples obtained during the field investigation were subjected to laboratory testing to evaluate their physical and engineering properties. Laboratory testing included laboratory compaction, direct shear, and soluble sulfate. A detailed description of the laboratory testing program and the test results are presented in Appendix C.

GEOLOGIC CONDITIONS

Regional Geologic Conditions

The subject site lies along the base of the San Gabriel Mountains which, in turn, lie within a series of modest-to-high-elevation ranges in southern California that exhibit east-westerly structural trends at odds with the general northwesterly structural grain of the remainder of the state. Therefore these mountain ranges are collectively referred to as the Transverse Ranges geomorphic province.

The Transverse Ranges are generally grouped into three divisions, the western, central, and eastern. The western included the Santa Monica Mountains, and others north of the Los Angeles basin and stretch offshore into the channel islands, while the eastern portions include the San Bernardino Mountains and the Little San Bernardino Mountains north of Palm Springs. The central portion generally includes the San Gabriel Mountain ranges that lie between the San Andreas fault on the east and the San Gabriel fault on the west. This area generally extends from Devore west to the Santa Clarita-Newhall pass region.

The central Transverse Ranges are underlain by metamorphic and granitic basement rock referred to as the San Gabriel Mountain type, typified by green and gray schists (Pelona Schist) of Late Mesozoic age, overlain tectonically by a 1,700-million- year-old Precambrian gneissic and granitic basement that had been intruded by Mesozoic (Triassic and Cretaceous) granitic plutons prior to the thrusting that juxtaposed this unit above the schist.

Faulting, both bounding and interior, is common in the Transverse Ranges. The San

Andreas fault marks the eastern boundary and northern boundary, while the Cucamonga fault marks the southern boundary along the southeastern portion from Lytle Creek to San Antonio canyon. West of San Antonio Canyon, the southern boundary is marked by the Sierra Madre fault. The interior of the central Transverse Ranges is cut by many interior faults as well. The San Antonio Canyon fault is one example, coexisting with the general location of the canyon.

The regional geology of the site and immediate surrounding region, as mapped by the U.S.G.S. (Morton and Miller, 2003), is shown on Enclosure A-3, within Appendix A.

Site Geologic Conditions

A thin layer of topsoil, averaging approximately one foot thick, covers the majority of the site. This material consists of silty sand with minor gravel and organic matter. In many areas it rests directly upon bedrock while in some areas it is built upon older colluvial soils. The colluvium was found to be present in the area of our exploratory trenches, T-2 and T-3 and consists of silty sand with a fair percentage of angular, gravel sized rocks. Locally, minor amounts of surficial fill soils are present and these are related to earlier water tank and access road construction activities. The fill soils consist of locally derived materials and appear to be up to about 2 feet thick but generally less than one foot thick in the area of the proposed new tank.

As noted above, the subject site is underlain by crystalline metamorphic and igneous bedrock materials. The underlying rocks along the base of the San Gabriel Mountains, east of San Antonio Canyon, have been mapped by the United States Geological Survey (Morton and Miller, 2003) as composed of various rock types consisting of mylonite, mylonitized tonalite, granulitic gneiss, cataclasite. These are all various types of crystalline rocks that have been dramatically altered by intense pressure, typically along a fault zone.

Our observations and trenching indicated various units of mylonitized tonalite, and granulitic gneiss in the site area. A tonalite is a type of igneous cyrstalline rock, similar in nature to granite but lacking in quartz, containing less than 10 percent quartz. Mylonitization is a process of severe deformation of the original rock texture from mechanical forces applied in a primary direction resulting is a flinty, banded, or streaked appearance. Granulitic gneiss is the result of similar pressure on metamorphic rocks that produce even-sized interlocking mineral crystals which have a preferred orientation, or foliation. As typical of these materials, the bedrock at the subject site was noted to be foliated, highly fractured, weathered near the surface into light reddish hues, becoming less weathered and lighter in color at depth. It was noted during excavation within these materials, the upper, approximately one foot, was typically weathered and moderately soft but the bedrock became harder quickly below this depth. Most joint surfaces within the bedrock have steep dip components to the north and south while the foliation generally dips moderately to the north and east. These orientations are considered favorable as related to the proposed retaining wall configuration.

Groundwater Hydrology

Groundwater was not encountered within any of our exploratory trenches at the site as advanced to a maximum depth of 5 feet below the existing ground surface.

Information pertaining to groundwater depths in the local area is lacking as the site area is underlain by bedrock at depth and no wells for which data is available was found in our search.

An unnamed tributary to Frankish Canyon is present just west of the site and it, most likely, is the main influence on groundwater conditions in the immediate site area. Considering that the flowline for this stream is approximately 70 feet below the site elevation at its nearest point, the depth to groundwater could be at or near this depth below the site.

Mass Movement

The site lies on relatively steep mountainous terrain and these areas tend to be prone to factors of mass movement including creep, rock falls, debris flows, avalanches, and landslides. However the crystalline rocks underlying the site tend to have high strength characteristics which lowers the potential for mass movement, except where fractures occur. While the rocks at the site were noted to be highly fractured, the fractures noted tended to be in steeply dipping sub-parallel sets. No areas of deep seated landslides, slumps, or surficial failures were noted on or adjacent to the site, or on the aerial photographs reviewed.

Faulting

As previously noted, the San Gabriel Mountains are boarded by and cut internally by numerous faults, many of which are considered to be active. However there are no known active faults on the subject site. In addition, according to the Official Maps of Alquist-Priolo Earthquake Fault Zones of California (Hart and Bryant, 2010) the subject site does not lie within a current State of California Earthquake Fault Zone (see Enclosure A-4 within Appendix A).

The closest known active fault is the Cucamonga fault located southeast of the site at a distance of approximately 0.8 miles (1.3 km). The Cucamonga fault is part of the Sierra Madre fault system which marks the southern boundary of the San Gabriel Mountains. This system is comprised of steeply north dipping, range-front thrust faults along which most of the uplift of the San Gabriel Mountains has occurred. The Cucamonga fault marks the eastern portion of the Sierra Madre fault system while the San Fernando fault marks the western portion. It is believed that the Cucamonga fault is capable of producing an earthquake on the order of 7.0 or greater.

Other faults in the region include the San Antonio fault located just under a mile (1.5 km) to the west, the Chino fault located approximately 9.5 miles (15km) to the southwest, the San Jacinto fault located approximately 11 miles (18 km) to the east, the San Andreas fault located approximately 13 miles (21 km) to the northeast, and the Whittier/Elsinore fault located approximately 19.5 miles (31km) to the southwest.

The San Antonio fault is generally known by offset materials located along either side of San Antonio canyon, and may join the San Jose fault to the southwest. The motion on this fault is left lateral strike slip.

The Chino fault is generally considered to be a western branch off the Whittier/Elsinore fault zone. The Whittier/Elsinore fault zone is one of the larger faults in southern California. The primary sense of slip along the Elsinore fault is right lateral strike-slip. It is believed that the Elsinore fault zone is capable of producing an earthquake magnitude on the order of 6.5 to 7.5.

The San Jacinto fault zone is a sub-parallel branch of the San Andreas fault zone, extending from the northwestern San Bernardino area, southward into the El Centro region. This fault has been active in recent times with several large magnitude events. It is believed that the San Jacinto fault is capable of producing an earthquake magnitude on the order of 6.5 or greater.

The San Andreas fault is considered to be the major tectonic feature of California, separating the Pacific Plate and the North American Plate. While estimates vary, the San Andreas fault is generally thought to have an average slip rate on the order of 24mm/yr and capable of generating large magnitude events on the order of 7.5 or greater.

Recent standards of practice have included a discussion of all potential earthquake sources within a 100 kilometer (62 mile) radius. However, while there are other large earthquake faults within a 100 kilometer (62 mile) radius of the site, none of these are considered as relevant to the site as the faults described above, due to their greater distance and/or smaller anticipated magnitudes.

Historical Seismicity

In order to obtain a general perspective of the historical seismicity of the site and surrounding region a search was conducted for seismic events at and around the area within various radii. This search was conducted utilizing the historical seismic search program by EPI Software, Inc (Reeder, 2000). This program conducts a search of a user selected cataloged seismic events database, within a specified radius and selected magnitudes, and then plots the events onto an overlay map of known faults. For this investigation the database of seismic events utilized by the EPI program was obtained from the Southern California Seismic Network (SCSN) available from the Southern California Earthquake Center. At the time of our search the data base contained data from January 1, 1932 through December 31, 2010.

In our first search, the general seismicity of the region was analyzed by selecting an epicenter map listing all events of magnitude 4.0 and greater, recorded since 1932, within a 100 kilometer (62 mile) radius of the site, in accordance with guidelines of the California Division of Mines and Geology. This map illustrates the regional seismic history of moderate to large events. As depicted on Enclosure A-5, within Appendix A, the site lies within a relatively active region associated with the San Jacinto and San Andreas faults trending southeast to northwest. Of all the events plotted, the closest was a magnitude 4.5 located approximately 4 kilometers (2.4 miles) to the southwest of the site.

In the second search, the micro seismicity of the area lying within a 15 kilometer (9.3 mile) radius of the site was examined by selecting an epicenter map listing events on the order of 1.0 and greater since 1978. In addition, only the "A" events, or most accurate events were selected. Caltech indicates the accuracy of the "A" events to be approximately 1 km. The results of this search is a map that presents the seismic history around the area of the site with much greater detail, not permitted on the larger map. The reason for limiting the events to the last 35 ± years on the detail map is to enhance the accuracy of the map. Events recorded prior the mid 1970's are generally considered to be less accurate due to advancements in technology. As depicted on this map, Enclosure A-6, a great number of small earthquakes has occurred in areas to the southwest of the site and throughout the local area.

In summary, the historical seismicity of the site entails numerous small to medium magnitude earthquake events occurring around the subject site, predominately associated with the presence of the San Andreas fault and related faults. Any future planned improvements within the property should anticipate that moderate to large seismic events could occur very near the site.

Secondary Seismic Hazards

Other secondary seismic hazards generally associated with severe ground shaking during an earthquake include liquefaction, seiches and tsunamis, earthquake induced flooding, landsliding and rockfalls, and seismically-induced settlement.

Liquefaction: The potential for liquefaction generally occurs during strong ground shaking within granular loose sediments where the groundwater is usually less than 50-feet. Due to the lack of such loose sediments within the site, the potential for liquefaction is considered nil.

Seiches/Tsunamis: Since the site is not located near any large bodies of water, the potential for a seiche or tsunami is considered nil.

Flooding (Water Storage Facility Failure): Other than the existing, onsite water tank, there are no large water storage facilities located near the site which could possibly rupture during in earthquake and affect the site by flooding.

Seismically-Induced Landsliding and Rockfalls: The relatively steep relief across the site increases the potential for landslides. However the crystalline bedrock materials have relatively high strength characteristics, thus reducing the potential for deep seated slides. In addition, no evidence for landslides or rockfalls was noted during our site reconnaissance or during our review of aerial photographs of the site and vicinity.

Seismically-Induced Settlement: Settlement generally occurs within areas of loose, granular soils with relatively low density. Since the site is underlain by relatively hard igneous bedrock, the potential for settlement is considered nil.

SOILS AND SEISMIC DESIGN CRITERIA (California Building Code)

Section 1613 of Chapter 16 of the 2016 California Building Code (CBC) contains the procedures and definitions for the calculations of the earthquake loads on structures and non structural components that are permanently attached to structures and their supports and attachments.

It should be noted that the classification of use and occupancy of all proposed structures at the site, and thus design requirements, shall be the responsibility of the structural engineer and the building official.

CBC Earthquake Design Summary

The following earthquake design criteria have been formulated for the site utilizing the source referenced above.

However, these values should be reviewed by the building official (Risk Category) and structural engineer and the final design should be performed by a qualified structural engineer familiar with the region.

CBC 2016 SEISMIC DESIGN SUMMARY (ASCE 7-10)* Site Location (USGS WGS84) 34.1645, -117.6619, Risk Category II	
Site Class Definition Chapter 20 ASCE 7	D
S_s Mapped Spectral Response Acceleration at 0.2s Period, (Figure 1613.3.1(1))	3.228
S_1 Mapped Spectral Response Acceleration at 1s Period, (Figure 1613.3.3(2))	1.125
F_a Short Period Site Coefficient at 0.2s Period, (Table 1613.3.3(1))	1.0
F_v Long Period Site Coefficient at 1s Period,(Table 1613.3.3(2))	1.5
S_{MS} Adjusted Spectral Response Acceleration at 0.2s Period, (eq .16-37)	3.228
S_{M1} Adjusted Spectral Response Acceleration at 1s Period, (eq .16-38)	1.688
S_{DS} Design Spectral Response Acceleration at 0.2s Period,(eq .16-39)	2.152

CBC 2016 SEISMIC DESIGN SUMMARY (ASCE 7-10)* Site Location (USGS WGS84) 34.1645, -117.6619, Risk Category II	
S_{D1} Design Spectral Response Acceleration at 1s Period, (eq .16-40)	1.125
Seismic Design Category - Short Period (Table 1613.3.5(1))	E
Seismic Design Category - Long Period (Table 1613.3.5(2))	E
*Values obtained from U.S.G.S. online U.S. Seismic Design Maps tool	

CONCLUSIONS

General

This investigation provides a broad overview of the geotechnical and geologic factors which are expected to influence future site planning and development. On the basis of our field investigation and testing program, it is the opinion of LOR Geotechnical Group, Inc. that construction of the proposed water tank as proposed is feasible from a geotechnical standpoint, provided the recommendations presented in this report are incorporated into design and implemented during grading and construction.

The subsurface conditions encountered in our exploratory trenches are indicative of the locations explored. The subsurface conditions presented here are not to be construed as being present the same everywhere on the site. If conditions are encountered during the construction of the project which differ significantly from those presented in this report, this firm should be notified immediately so we may assess the impact to the recommendations provided.

Foundation Support

Based upon the field investigation and test data, it is our opinion that the existing surficial soils and the upper portions of the bedrock will not, in their present condition, provide uniform and/or adequate support for the proposed water tank, retaining walls, or for the proposed fill soils. Our observations indicate generally loose in-situ conditions for the native soils. Left as is, these conditions could cause unacceptable differential and/or overall settlements upon application of the anticipated foundation loads.

To provide adequate support for the proposed tank and retaining walls, removal of the surficial soils and the upper approximately one foot of the bedrock materials is recommended.

The proposed water tank ringwall foundation system will provide adequate support for the anticipated downward and lateral loads when utilized in conjunction with the recommended removals and fill mat.

Retaining walls supporting fill around the proposed water tank are anticipated to be MSE walls or reinforced concrete cantilever walls. Either type should be founded on competent bedrock materials following the wall component manufacturers guidelines and our recommendations as presented herein. The wall to be constructed at a later date to the northwest of the existing water tank will support materials exposed by cutting into the bedrock. Provided that competent bedrock materials are exposed at proposed footing grades, these foundations may also be founded upon the bedrock materials. Deepening of the footings in local areas may required to expose competent bedrock.

Geologic Mitigations

No special mitigation methods are deemed necessary at this time, other than the geotechnical recommendations provided in the following sections.

Seismicity

Seismic ground rupture is generally considered most likely to occur along pre-existing active faults. Since no active faults are known to exist at, or project into the site, the probability of ground surface rupture occurring at the site is considered nil.

It is reasonable to expect a moderately strong ground motion seismic event to occur during the lifetime of the proposed structure on the site. The effects of ground shaking anticipated at the subject site should be mitigated by the seismic design requirements and procedures outlined in Chapter 16 of the California Building Code. However, it should be noted that the current building code requires the minimum design to allow a structure to remain standing after a seismic event, in order to allow for safe evacuation. A structure built to code may still sustain damage which might ultimately result in the demolishing of the structure (Larson and Slosson, 1992).

RECOMMENDATIONS

General Site Grading

It is imperative that no clearing and/or grading operations be performed without the presence of a qualified geotechnical engineer. An on-site, pre-job meeting with the developer, the contractor, the jurisdictional agency, and the geotechnical engineer should occur prior to all grading related operations. Operations undertaken at the site without the geotechnical engineer present may result in exclusions of affected areas from the final compaction report for the project.

Grading of the subject site should be performed in accordance with the following recommendations as well as applicable portions of the California Building Code, and/or applicable local ordinances.

All areas to be graded should be stripped of significant vegetation and other deleterious materials.

All uncontrolled fills encountered during site preparation should be completely removed, cleaned of significant deleterious materials, and are anticipated to be adequate for reuse as compacted fill.

Cavities created by removal of subsurface obstructions should be thoroughly cleaned of loose soil, organic matter and other deleterious materials, shaped to provide access for construction equipment, and backfilled as recommended in the following Engineered Compacted Fill section of this report.

Initial Site Preparation

Any undocumented fill and all loose surficial soil materials should be removed from structural areas and areas to receive engineered compacted fill. The data developed during this investigation indicates that removals on the order of 1 to 3 feet, and perhaps more locally, will be required in most areas. The actual depths of removal should be determined during the grading operation by observation and in-place density testing, as necessary.

Preparation of Fill Areas

After conducting the removals discussed above and prior to placing fill, the surfaces of all areas to receive fill should be scarified to a depth of at least 6 inches, where practical. The scarified ground should be brought to near optimum moisture content and recompacted to a relative compaction of at least 90 percent (ASTM D 1557).

Preparation of Foundation Areas

The proposed ringwall footing for the water tank should rest upon at least 24 inches of properly compacted fill consisting of granular, non-expansive material. In areas where the required fill thickness is not accomplished by removals and site rough grading, the footing areas should be further subexcavated to a depth of at least 24 inches below the proposed footing base grade, with the subexcavation extending at least 5 feet beyond the footing lines. This firm should review the final foundation plans for the tank, when available, and specific recommendations regarding over-excavations and temporary stability of the excavation for the footing segment in the vicinity of the existing tank should be addressed at that time, as necessary.

All wall footings should rest upon competent bedrock materials which are anticipated to be found at approximately 1 to 3 feet below the existing ground surface. Removals for retaining wall foundations should extend at a minimum of 1 to 1 (horizontal to vertical) from the bottom of the retaining wall footings an amount equal to the depth below, or 2 feet beyond the outside of the foundation. In addition, if MSE walls are used near the proposed water tank, the top layer of geosynthetic soil reinforcement shall be deeper than the tank footing depth.

Engineered Compacted Fill

Provided they are free from significant amounts of organic matter or other deleterious materials, the on-site soils should provide adequate quality fill material. Unless approved by the soils engineer, rock or similar irreducible material with a maximum dimension greater than 6 inches should not be buried or placed in fills.

Import fill, which will be required, should be inorganic, non-expansive, granular soils free from rocks or lumps greater than 6 inches in maximum dimension. Sources for import fill should be approved by the geotechnical engineer prior to their use.

Fill should be spread in maximum 8-inch thick, uniform, loose lifts, with each lift brought to near optimum moisture content and compacted to a relative compaction of at least 90 percent in accordance with ASTM D 1557.

Based upon the relatively loose condition of the near surface soils and the relative compaction anticipated for compacted fill soil, we estimate a compaction shrinkage factor of approximately 5 to 10 percent. Therefore, 1.05 cubic yards to 1.10 cubic yards of in-place materials would be necessary to yield one cubic yard of properly compacted fill material. In addition, we would anticipate subsidence to be nil. These values are for estimating purposes only, and are exclusive of losses due to stripping or the removal of subsurface obstructions and/or oversize materials. These values may vary due to differing conditions within the project boundaries and the limitations of this investigation.

Short-Term Excavations

Following the California Occupational and Safety Health Act (CAL-OSHA) requirements, excavations 5 feet deep and greater should be sloped or shored. All excavations and shoring should conform to CAL-OSHA requirements.

Short-term excavations 5 feet deep and greater shall conform to Title 8 of the California Code of Regulations, Construction Safety Orders, Section 1504 and 1539 through 1547. Based on our exploratory trenches and field observations, it appears that much of the site exposes shallow bedrock which can be classified as Stable Rock with lesser site areas containing granular soils which can be classified as Type C materials. These are the predominant types of soils on the project and all short-term excavations should be based on these types of soils. Deviation from the standard short-term slopes are permitted using option 4, Design by a Registered Professional Engineer (Section 1541.1).

Short-term slope construction and maintenance are the responsibility of the contractor, and should be a consideration of his methods of operation and the actual soil conditions encountered.

Slope Construction

All cut and fill slopes should be constructed no steeper than two horizontal to one vertical. Fill slopes should be overfilled during construction and then cut back to expose fully compacted soil. A suitable alternative would be to compact the slopes during construction, then roll the final slopes to provide dense, erosion-resistant surfaces.

Where fills are to be placed against existing slopes steeper than five horizontal to one vertical, the fill should be properly benched into competent bedrock materials. Benches should be constructed at approximately 2- to 4-feet vertical intervals.

Slope Protection

Since the native materials are susceptible to erosion by running water, measures should be provided to prevent surface water from flowing over slope faces. Slopes at the project should be planted with deep rooted ground cover as soon as possible after completion. The use of succulent ground covers, such as iceplant or sedum, is not recommended. If watering is necessary to sustain plant growth on slopes, then the watering operation should be monitored to ensure proper operation of the irrigation system and to prevent over watering.

Soil Expansiveness

The upper materials encountered during this investigation were observed to be granular and are considered to have a very low expansion potential. Although specialized construction procedures to specifically resist expansive soil activity are not anticipated at this time, additional evaluation of on-site and any imported soils for their expansion potential should be conducted during the rough grading operations.

Foundation Design

If the site is prepared as recommended, the proposed concrete ring wall foundation for the tank should be founded so as to lie entirely on a compacted fill mat of at least 24 inches in thickness. For the minimum 24-inch width and depth, footings may be designed using an allowable soil bearing pressure of 3,000 pounds per square foot (psf) for dead plus live loads.

The planned retaining walls at the site may consist of reinforced concrete retaining walls or MSE walls. The wall footing/base shall rest on competent, approved bedrock materials. Vertically, the wall footing/base should be placed at least 24 inches below the lowest adjacent grade and 1 foot into firm bedrock. Horizontally, the wall footing/base should be setback from the slope face a minimum of 5 feet. For these conditions and accounting a minimum wall footing/base width of 7 feet, an allowable bearing pressure of 6,000 psf is recommended for foundation design.

The above values are net pressures; therefore, the weight of the foundations and the backfill over the foundations may be neglected when computing dead loads. The values apply to the maximum edge pressure for foundations subjected to eccentric loads or overturning. The recommended pressures apply for the total of dead plus frequently applied live loads, and incorporate a factor of safety of at least 3.0. The allowable bearing pressures may be increased by one-third for temporary wind or seismic loading. The resultant of the combined vertical and lateral seismic loads should act within the middle one-third of the footing width. The maximum calculated edge pressure under the toe of foundations subjected to eccentric loads or overturning should not exceed the increased allowable pressure.

Resistance to lateral loads will be provided by passive earth pressure and base friction. For footings bearing against compacted fill or firm bedrock materials, passive earth pressure may be considered to be developed at a rate of 350 pounds per square foot per foot of depth. For footings against 2.5:1 sloping compacted fill or bedrock, passive earth pressure may be considered to be developed at a rate of 260 pounds per square foot per foot of depth. Base friction may be computed at 0.35 times the normal load. Base friction and passive earth pressure may be combined without reduction. These values are for dead load plus live load and may be increased by one-third for wind or seismic.

Settlement

Maximum settlement of the tank foundation designed and constructed in accordance with the preceding recommendations are estimated to be on the order of 0.5 inch. Differential settlement across the tank pad should be about one-half of the total settlement. Settlement of all foundations is expected to occur rapidly, primarily as a result of elastic compression of supporting soils as the loads are applied, and should be essentially completed shortly after initial application of the loads.

Settlement of the proposed retaining walls resting on firm bedrock materials should be negligible.

Concrete Cantilever Wall Design

The design of footings for walls below grade (basement or retaining structures) should be performed in accordance with the recommendations described earlier under Preparation of Foundation Areas and Foundation Design. For design of retaining wall footings, the resultant of the applied loads should act in the middle one-third of the footing, and the maximum edge pressure should not exceed the basic allowable value without increase.

For design of retaining walls unrestrained against movement at the top, we recommend an equivalent fluid density of 35 pounds per cubic foot (pcf) be used. This assumes level backfill consisting of compacted, non-expansive, granular imported soils placed against the structures and within the back cut slope extending upward from the base of the stem at 35 degrees from the vertical or flatter.

Walls below grade that are restrained against free movement at the top of the walls and that have level backfill should be designed to resist a lateral earth pressure between active and at-rest conditions. For this condition, we recommend an equivalent fluid density of 45 pcf be used if compacted, granular imported soil is used for backfill.

Retaining structures subject to uniform surcharge loads within a horizontal distance behind the structures equal to the structural height should be designed to resist additional lateral loads equal to 0.28 times the surcharge load for unrestrained retaining walls and equal to 0.38 times the surcharge load for restrained retaining walls. Any isolated or line loads from adjacent foundations or vehicular loading will impose additional wall loads and should be considered individually.

The above parameters were estimated assuming that granular, non-expansive, imported materials with a compacted unit weight of 120 pcf and internal friction angle of 34 degrees will be used as wall backfill.

To avoid over stressing or excessive tilting during placement of backfill behind walls, heavy compaction equipment should not be allowed within the zone delineated by a 45 degree line extending from the base of the wall to the fill surface. The backfill directly behind the walls should be compacted using light equipment such as hand operated vibrating plates and rollers. No material larger than three inches in diameter should be placed in direct contact with the wall.

Wall pressures should be verified prior to construction, when the actual backfill materials and conditions have been determined. Recommended pressures are applicable only to level, non-expansive, properly drained backfill with no additional surcharge loadings. If inclined backfills are proposed, this firm should be contacted to develop appropriate active earth pressure parameters.

MSE Wall Design

The parameters of bearing capacity and lateral earth pressure given in the Foundation Design section should also be applicable to verify the external stability (i.e. stability against a bearing capacity, overturning, sliding failures) of a tentative reinforced soil wall. Additional parameters to confirm the internal stability of the reinforced soil mass will depend on the properties of the geogrid reinforcement and actual backfill used and should be supplied by the wall manufacturer.

Wall Drainage

Because water increases earth pressures drastically, good drainage measures need to be maintained at the project to prevent the build up of water behind the walls. These could include installation of weep holes in the face of the wall or perforated pipe drains installed behind the walls, which would be allowed to drain to a non-invasive area.

Sulfate Protection

The results of the soluble sulfate tests conducted on selected subgrade soils expected to be encountered at foundation levels are presented in Appendix C.

Based on the test results the sulfate exposures of on site soils is considered negligible by the CBC. Therefore, no specific recommendations are given for concrete elements to be in contact with on site soils.

Construction Monitoring

Post investigative services are an important and necessary continuation of this investigation. Project plans and specifications should be reviewed by this firm prior to construction to confirm that the intent of the recommendations presented herein have been incorporated into the design. This is particularly important for this project due to the close proximity of the existing tank to the northwest and this firm should review the final foundation plans for the tank, when available. Specific recommendations regarding over-excavations in the vicinity of the existing tank should be addressed at that time, as necessary.

During construction, sufficient and timely geotechnical observation and testing should be provided to correlate the findings of this investigation with the actual subsurface conditions exposed during construction. Items requiring observation and testing include, but are not necessarily limited to, the following:

- Site preparation-stripping and removals.
- Excavations, including approval of the bottom of excavation prior to backfilling.
- Scarifying and recompacting prior to fill placement.
- Subgrade preparation for pavements and slabs-on-grade.
- Placement of engineered compacted fill and backfill, including approval of fill materials and the performance of sufficient density tests to evaluate the degree of compaction being achieved.
- Foundation excavations, including footings.

LIMITATIONS

This report contains geotechnical conclusions and recommendations developed solely for use by TKE Engineering, Inc., and their design consultants, for the purposes described earlier. It may not contain sufficient information for other uses or the purposes of other parties. The contents should not be extrapolated to other areas or used for other facilities without consulting LOR Geotechnical Group, Inc.

The recommendations are based on interpretations of the subsurface conditions concluded from information gained from subsurface explorations, and a surficial site reconnaissance. The interpretations may differ from actual subsurface conditions, which can vary horizontally and vertically across the site. Due to possible subsurface variations, all aspects of field construction addressed in this report should be observed and tested by the project geotechnical consultant.

If parties other than LOR Geotechnical Group, Inc. provide construction monitoring services, they must be notified that they will be required to assume responsibility for the geotechnical phase of the project being completed by concurring with the recommendations provided in this report or by providing alternative recommendations.

The report was prepared using generally accepted geotechnical engineering practices under the direction of a state licensed geotechnical engineer. No warranty, express or implied, is made as to conclusions and professional advice included in this report. Any persons using this report for bidding or construction purposes should perform such independent investigations as deemed necessary to satisfy themselves as to the surface and subsurface conditions to be encountered and the procedures to be used in the performance of work on this project.

The intent of this geotechnical investigation was to provide the geotechnical foundations data for design and grading procedures and mitigation criteria for the client only. Our studies did not include an evaluation of the potential for hazardous materials to exist at the site or in the region around the site. This report may not contain sufficient information for uses or the purposes of other parties and the contents therefore should not be utilized, or extrapolated to other uses or areas or used for other purposes than those specifically stated above.

TIME LIMITATIONS

The findings of this report are valid as of this date. Changes in the condition of a property can, however, occur with the passage of time, whether they be due to natural processes or the work of man on this or adjacent properties. In addition, changes in the Standards of Practice and/or Governmental Codes may occur. Due to such changes, the findings of this report may be invalidated wholly or in party by changes beyond our control. Therefore, this report should not be relied upon after a significant amount of time without a review by LOR Geotechnical Group, Inc. verifying the suitability of the conclusions and recommendations.

CLOSURE

It has been a pleasure to assist you with this project. We look forward to being of further assistance to you as construction begins. Should conditions be encountered during construction that appear to be different than indicated by this report, please contact this office immediately in order that we might evaluate their effect.

Should you have any questions regarding this report, please do not hesitate to contact this firm at your convenience.

Respectfully submitted,
LOR Geotechnical Group, Inc.

Robert M. Markoff, CEG
Engineering Geologist

John P. Leuer, GE 2030
President

RMM:GB:JPL:ss

Distribution: Addressee (2) and via email: cdeiter@tkeengineering.com

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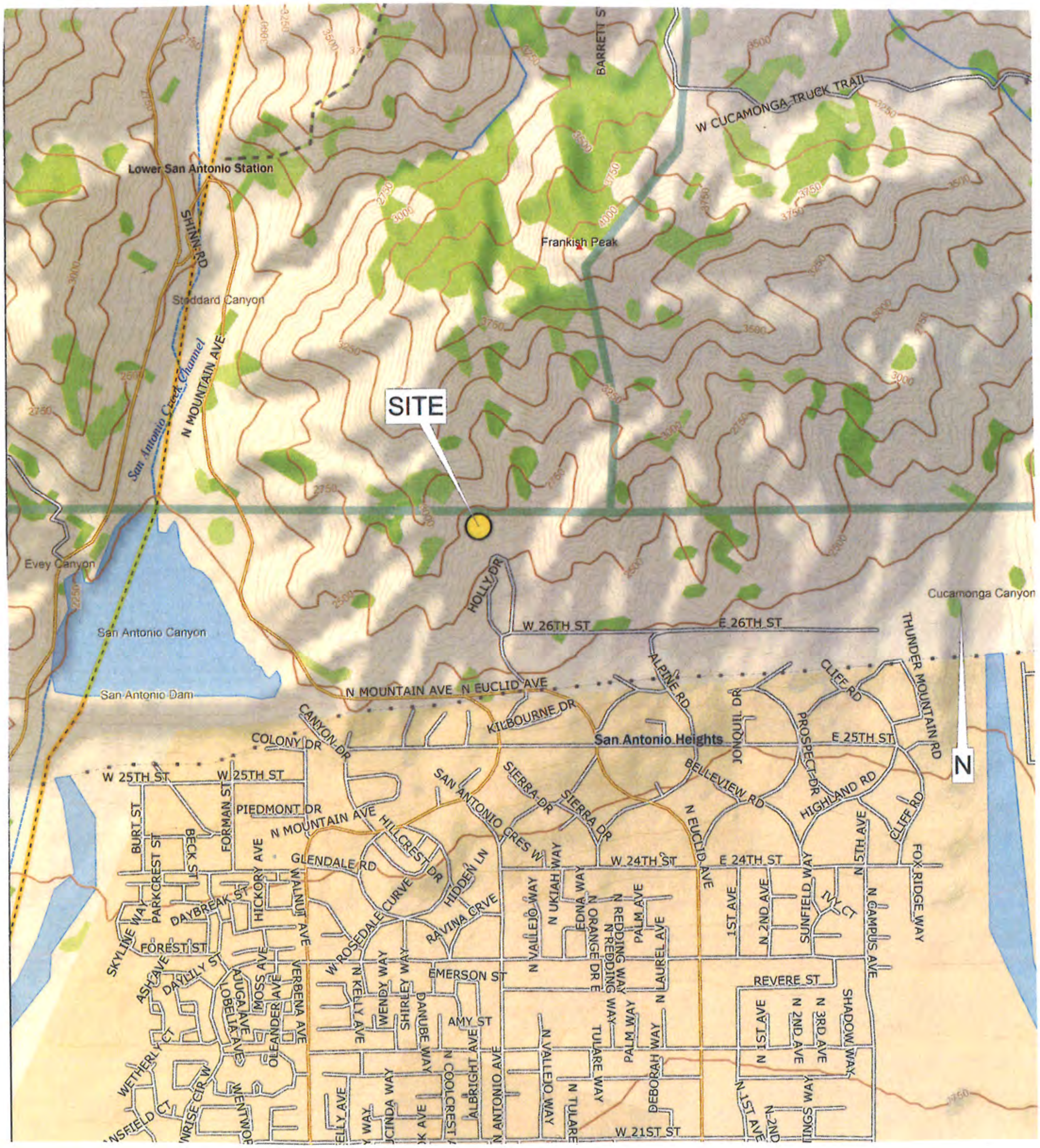
AERIAL PHOTOGRAPHS

(San Bernardino County Flood Control and Water Conservation District)

DATE	FLIGHT NO.	PHOTO NO(S).	SCALE
1938	W-65	D-2-8:9	1" = 1,000'
November 10, 1955	F-34	6-39;41	1" = 1,200'
October 15, 1959	AXL-17W	52-53	1" = 1,667'
February 7, 1970	C-297	72-73	1" = 2,000'
October 8, 1971	C-186	1-3	1" = 2,000'
January 21, 1978	C-279	161-162	1" = 2,000'
February 25, 1985	C-450	161-163	1" = 2,000'
April 20, 1996	C-528	178-179	1" = 2,000'
January 19, 2005	C-553	16-3;4	1" = 1,200'

APPENDIX A

Index Map, Trench Location Map, Regional Geologic Map, Earthquake Fault Zone Map, and Historical Seismicity Maps



INDEX MAP

PROJECT:	**PROPOSED SAWCO 100,000 GALLON WATER TANK**	**PROJECT NO.:**	**63314.1**
CLIENT:	**TKE ENGINEERING, INC.**	**ENCLOSURE:**	**A-1**
LOR Geotechnical Group, Inc.		**DATE:**	**NOVEMBER 2016**
		SCALE:	**1" = 2,000'**

Legend

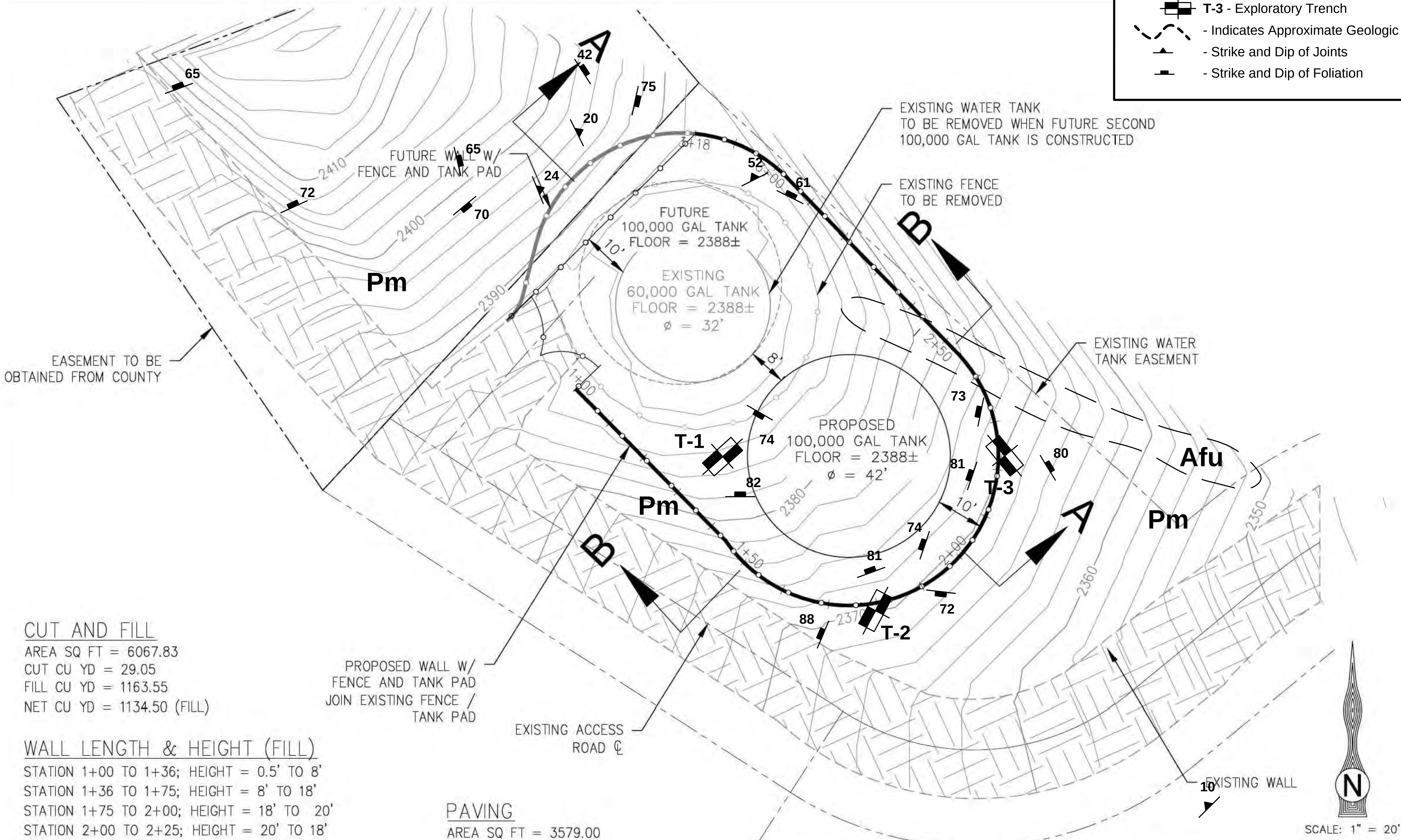
(Locations Approximate)

Map Symbols

- **Afu** - Undocumented Fill
- **Pm** - Bedrock

Map Symbols

- **T-3** - Exploratory Trench
- Indicates Approximate Geologic Contact
- Strike and Dip of Joints
- Strike and Dip of Foliation



CUT AND FILL

AREA SQ FT = 6067.83
CUT CU YD = 29.05
FILL CU YD = 1163.55
NET CU YD = 1134.50 (FILL)

WALL LENGTH & HEIGHT (FILL)

STATION 1+00 TO 1+36; HEIGHT = 0.5' TO 8'
STATION 1+36 TO 1+75; HEIGHT = 8' TO 18'
STATION 1+75 TO 2+00; HEIGHT = 18' TO 20'
STATION 2+00 TO 2+25; HEIGHT = 20' TO 18'
STATION 2+25 TO 2+70; HEIGHT = 18' TO 12'
STATION 2+70 TO 3+18; HEIGHT = 8' TO 0.5'

PAVING

AREA SQ FT = 3579.00

FENCING

LF = 293

TKE ENGINEERING, INC.
2305 CHICAGO AVENUE
RIVERSIDE, CA 92507
(951) 680-0440
FAX: (951) 680-0490

San Antonio Water Company Since 1882

SAN ANTONIO WATER COMPANY

ALTERNATIVE 3 - PLAN

NEW 100,000 GALLON TANK AND FUTURE 2ND 100,000 GAL TANK

PLOT PLAN

PROJECT: SAWCO 100,000 GALLON WATER TANK, UPLAND, CALIFORNIA

CLIENT: TKE ENGINEERING, INC.

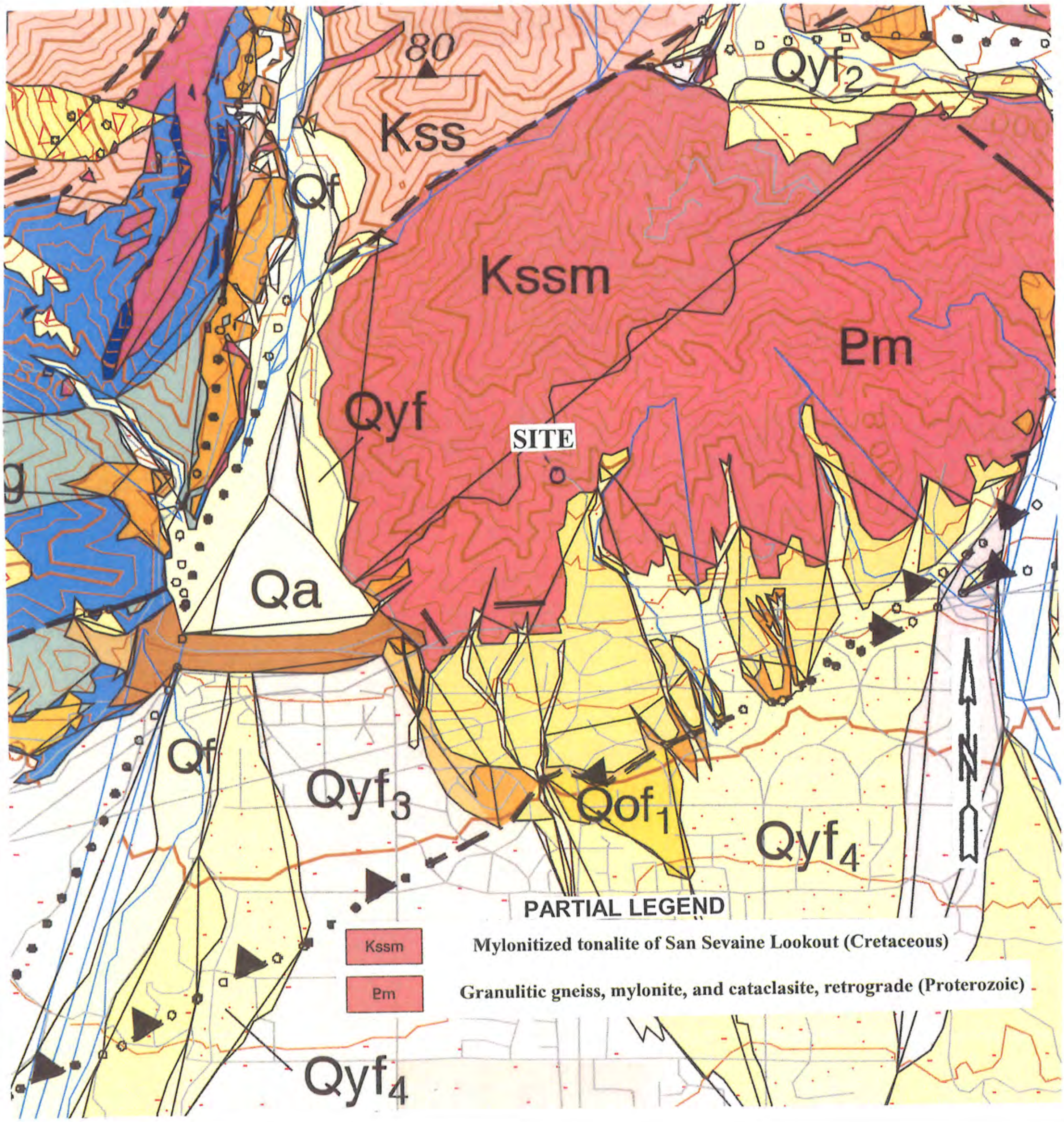
PROJECT NO: 63314.1

ENCLOSURE: A-2

DATE: DECEMBER 2016

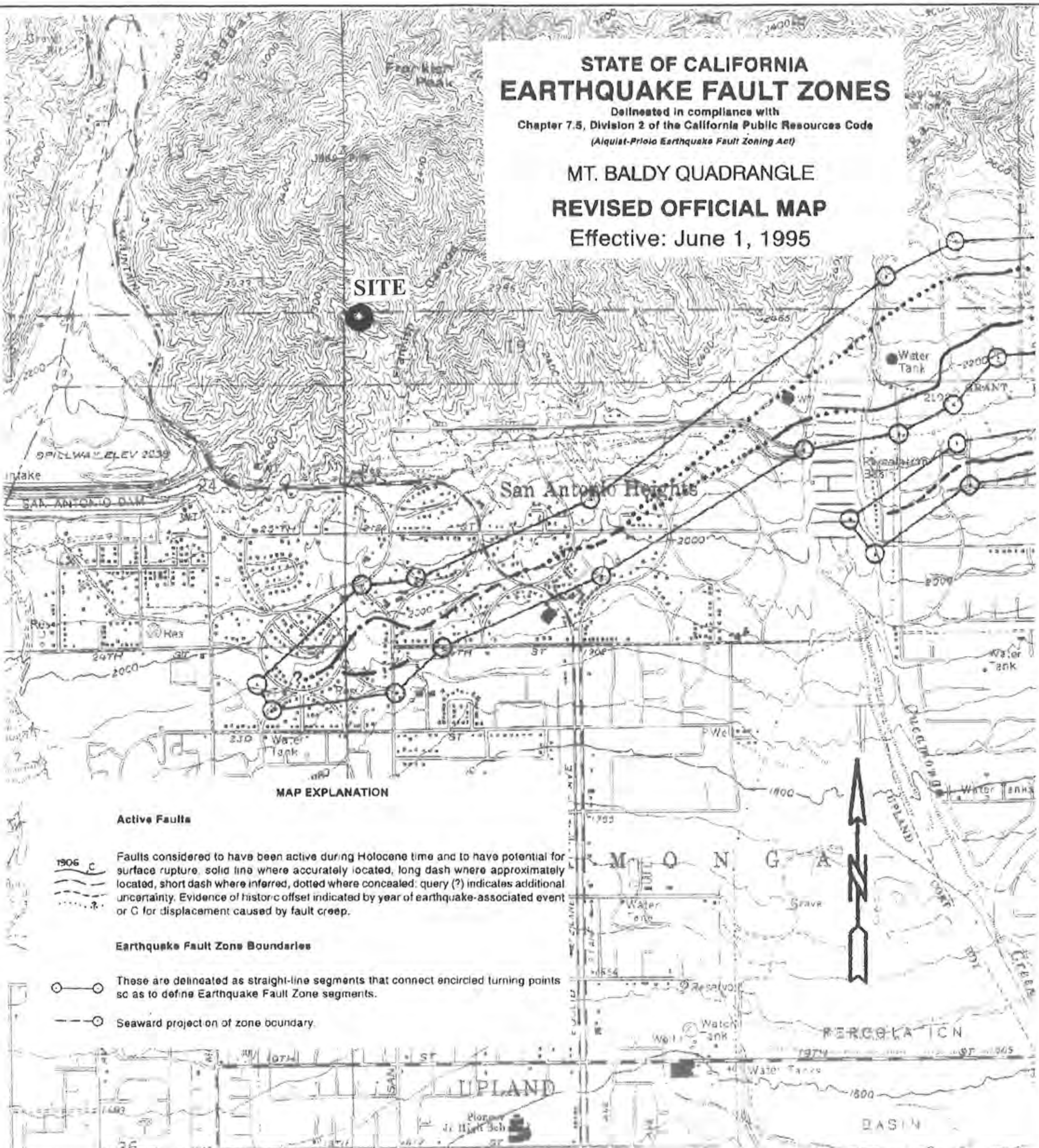
SCALE: 1" = 20'

LOR Geotechnical Group, Inc.



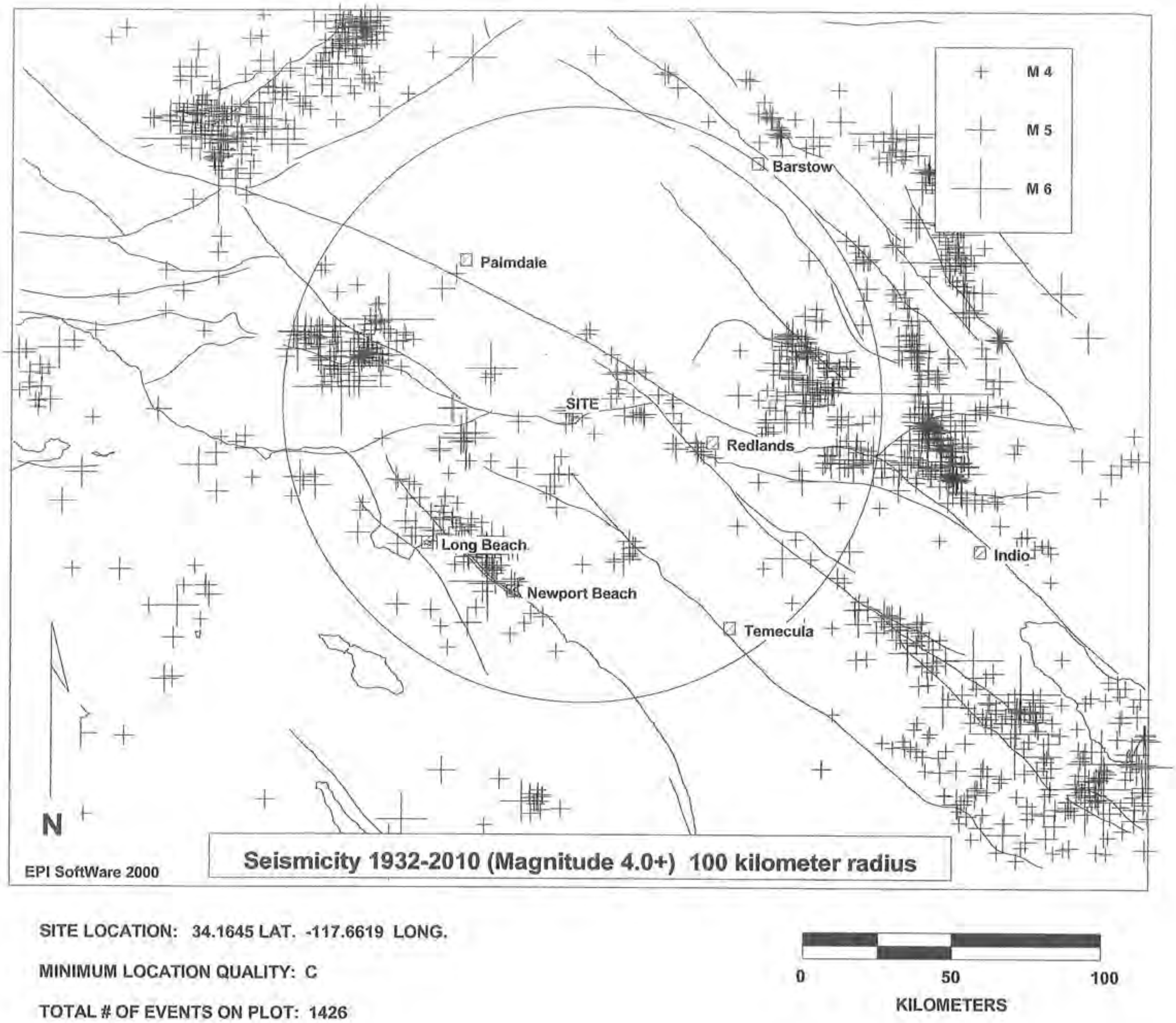
REGIONAL GEOLOGIC MAP

PROJECT:	PROPOSED SAWCO 100,000 GALLON WATER TANK	PROJECT NO.:	63314.1
CLIENT:	TKE ENGINEERING, INC.	ENCLOSURE:	A-3
LOR Geotechnical Group, Inc.		DATE:	NOVEMBER 2016
		SCALE:	1" = 2,000'



EARTHQUAKE FAULT ZONE MAP

PROJECT:	**PROPOSED SAWCO 100,000 GALLON WATER TANK**	**PROJECT NO.:**	**63314.1**
CLIENT:	**TKE ENGINEERING, INC.**	**ENCLOSURE:**	**A-4**
LOR Geotechnical Group, Inc.		**DATE:**	**NOVEMBER 2016**
		SCALE:	**1" = 2,000'**



SITE LOCATION: 34.1645 LAT. -117.6619 LONG.

MINIMUM LOCATION QUALITY: C

TOTAL # OF EVENTS ON PLOT: 1426

TOTAL # OF EVENTS WITHIN SEARCH RADIUS: 499

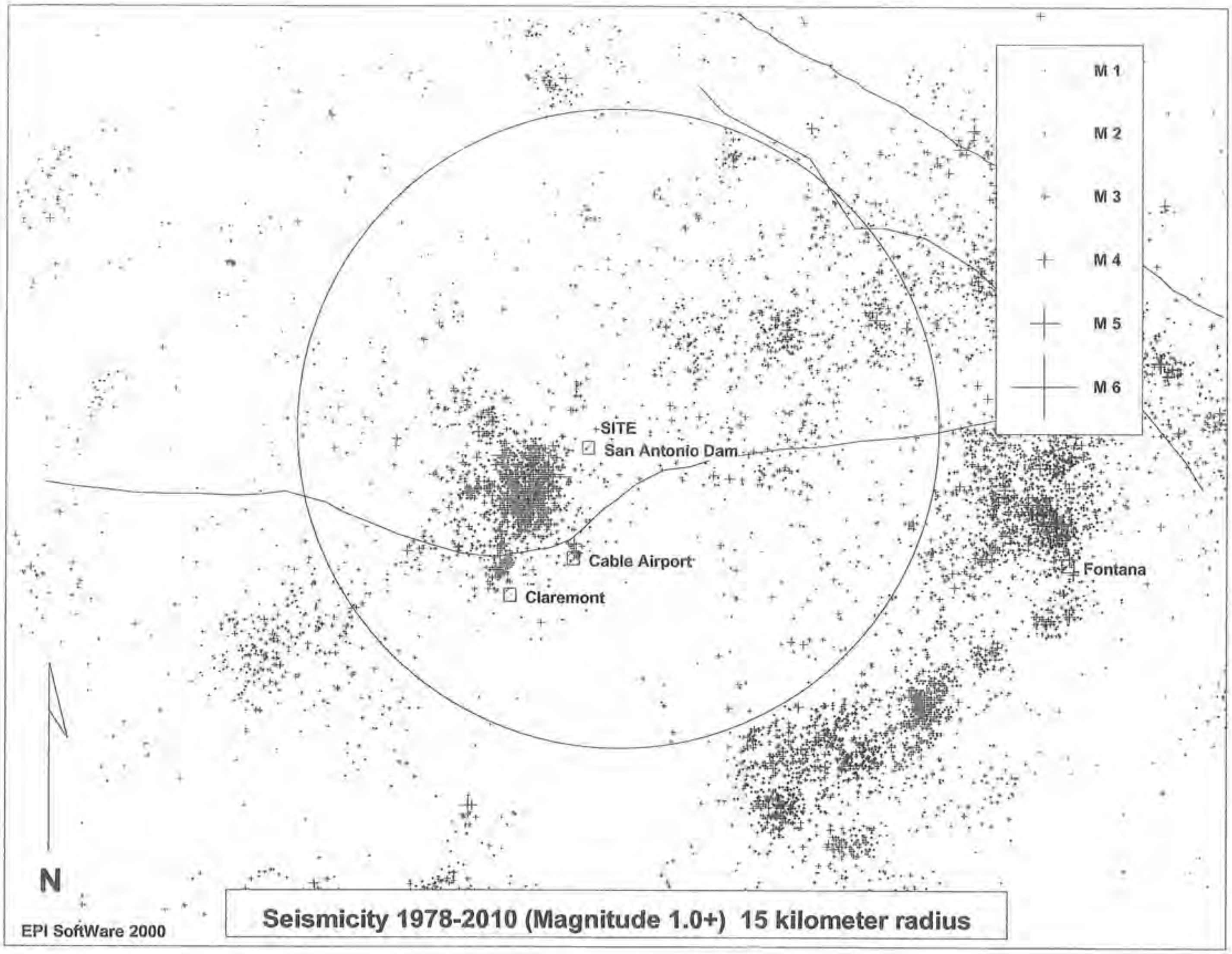
MAGNITUDE DISTRIBUTION OF SEARCH RADIUS EVENTS:

4.0- 4.9 : 453
5.0- 5.9 : 42
6.0- 6.9 : 4
7.0- 7.9 : 0
8.0- 8.9 : 0

CLOSEST EVENT: 4.5 ON FRIDAY, MARCH 02, 1990 LOCATED APPROX. 4 KILOMETERS SOUTHWEST OF THE SITE

LARGEST 5 EVENTS:

6.7 ON MONDAY, JANUARY 17, 1994 LOCATED APPROX. 80 KILOMETERS WEST OF THE SITE
6.6 ON TUESDAY, FEBRUARY 09, 1971 LOCATED APPROX. 70 KILOMETERS NORTHWEST OF THE SITE
6.4 ON SATURDAY, MARCH 11, 1933 LOCATED APPROX. 67 KILOMETERS SOUTHWEST OF THE SITE
6.3 ON SUNDAY, JUNE 28, 1992 LOCATED APPROX. 76 KILOMETERS EAST OF THE SITE
5.9 ON MONDAY, JANUARY 17, 1994 LOCATED APPROX. 77 KILOMETERS WEST OF THE SITE



SITE LOCATION: 34.1645 LAT. -117.6619 LONG.

MINIMUM LOCATION QUALITY: A

TOTAL # OF EVENTS ON PLOT: 8676

TOTAL # OF EVENTS WITHIN SEARCH RADIUS: 2675

MAGNITUDE DISTRIBUTION OF SEARCH RADIUS EVENTS:

1.0- 1.9 : 1800
2.0- 2.9 : 809
3.0- 3.9 : 59
4.0- 4.9 : 6
5.0- 5.9 : 1
6.0- 6.9 : 0
7.0- 7.9 : 0
8.0- 8.9 : 0

CLOSEST EVENT: 1.6 ON THURSDAY, MARCH 01, 1990 LOCATED APPROX. .1 KILOMETER OF THE SITE

LARGEST 5 EVENTS:

5.5 ON WEDNESDAY, FEBRUARY 28, 1990 LOCATED APPROX. 3 KILOMETERS SOUTHWEST OF THE SITE
4.7 ON THURSDAY, MARCH 01, 1990 LOCATED APPROX. 5 KILOMETERS WEST OF THE SITE
4.7 ON SUNDAY, JUNE 26, 1988 LOCATED APPROX. 5 KILOMETERS SOUTHWEST OF THE SITE
4.5 ON TUESDAY, APRIL 17, 1990 LOCATED APPROX. 8 KILOMETERS SOUTHWEST OF THE SITE
4.5 ON FRIDAY, MARCH 02, 1990 LOCATED APPROX. 3 KILOMETERS SOUTHWEST OF THE SITE

APPENDIX B

Field Investigation Program and Boring Logs

APPENDIX B
FIELD INVESTIGATION

Subsurface Exploration

The site was investigated on November 18, 2016 and consisted of excavating three exploratory trenches to depths of between 3 and 5 feet below the existing ground surface. The approximate locations of the trenches are shown on Enclosure A-2 in Appendix A.

The trench exploration was conducted using a New Holland backhoe equipped with a 24-inch bucket. The soils were continuously logged by an engineering geologist from this firm who inspected the site, maintained detailed logs of the trenches, obtained undisturbed, as well as disturbed, soil samples for evaluation and testing, and classified the soils by visual examination in accordance with the Unified Soil Classification System.

Disturbed soil samples were obtained at soil changes and other selected levels within the trenches. The samples were placed in sealed containers for transport to the laboratory.

All samples obtained were taken to our laboratory for storage and testing. Detailed logs of the trenches are presented on the enclosed Trench Logs, Enclosures B-1 through B-3. A Trench Log Key and Soil Classification Chart are presented on Enclosures B-i and B-ii, respectively.

CONSISTENCY OF SOIL

SANDS

SPT BLOWS	CONSISTENCY
0-4	Very Loose
4-10	Loose
10-30	Medium Dense
30-50	Dense
Over 50	Very Dense

COHESIVE SOILS

SPT BLOWS	CONSISTENCY
0-2	Very Soft
2-4	Soft
4-8	Medium
8-15	Stiff
15-30	Very Stiff
30-60	Hard
Over 60	Very Hard

SAMPLE KEY

Symbol — **Description**

INDICATES CALIFORNIA SPLIT SPOON SOIL SAMPLE

INDICATES BULK SAMPLE

INDICATES SAND CONE OR NUCLEAR DENSITY TEST

INDICATES STANDARD PENETRATION TEST (SPT) SOIL SAMPLE

TYPES OF LABORATORY TESTS

1 Atterberg Limits

2 Consolidation

3 Direct Shear (undisturbed or remolded)

4 Expansion Index

5 Hydrometer

6 Organic Content

7 Proctor (4", 6", or Cal216)

8 R-value

9 Sand Equivalent

10 Sieve Analysis

11 Soluble Sulfate Content

12 Swell

13 Wash 200 Sieve

TRENCH LOG LEGEND

PROJECT: SAWCO 100,000 GALLON WATER TANK	PROJECT NO.: 63314.1
CLIENT: TKE ENGINEERING, INC.	ENCLOSURE: B-i
LOR Geotechnical Group, Inc.	DATE: NOVEMBER 2016

SOIL CLASSIFICATION CHART

MAJOR DIVISIONS			SYMBOLS GRAPH	SYMBOLS LETTER	TYPICAL DESCRIPTIONS
COARSE GRAINED SOILS MORE THAN 50% OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	GRAVEL AND GRAVELLY SOILS MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	CLEAN GRAVELS (LITTLE OR NO FINES)		GW	WELL-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
				GP	POORLY-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
		GRAVELS WITH FINES (APPRECIABLE AMOUNT OF FINES)		GM	SILTY GRAVELS, GRAVEL - SAND - SILT MIXTURES
				GC	CLAYEY GRAVELS, GRAVEL - SAND - CLAY MIXTURES
	SAND AND SANDY SOILS MORE THAN 50% OF COARSE FRACTION PASSING ON NO. 4 SIEVE	CLEAN SANDS (LITTLE OR NO FINES)		SW	WELL-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
				SP	POORLY-GRADED SANDS, GRAVELLY SAND, LITTLE OR NO FINES
		SANDS WITH FINES (APPRECIABLE AMOUNT OF FINES)		SM	SILTY SANDS, SAND - SILT MIXTURES
				SC	CLAYEY SANDS, SAND - CLAY MIXTURES
FINE GRAINED SOILS MORE THAN 50% OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE	SILTS AND CLAYS	LIQUID LIMIT LESS THAN 50		ML	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS OR CLAYEY SILTS WITH SLIGHT PLASTICITY
				CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
				OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS	LIQUID LIMIT GREATER THAN 50		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SAND OR SILTY SOILS
				CH	INORGANIC CLAYS OF HIGH PLASTICITY
				OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
HIGHLY ORGANIC SOILS				PT	PEAT, HUMUS, SWAMP SOILS WITH HIGH ORGANIC CONTENTS

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS

PARTICLE SIZE LIMITS

BOULDERS	COBBLES	GRAVEL		SAND			SILT OR CLAY
		COARSE	FINE	COARSE	MEDIUM	FINE	

12" 3" 3/4" No. 4 No. 10 No. 40 200
(U.S. STANDARD SIEVE SIZE)

SOIL CLASSIFICATION CHART

PROJECT:	SAWCO 100,000 GALLON WATER TANK	PROJECT NO.:	63314.1
CLIENT:	TKE ENGINEERING, INC.	ENCLOSURE:	B-ii
LOR Geotechnical Group, Inc.		DATE:	NOVEMBER 2016

LOG OF TRENCH T-1

DEPTH IN FEET	LABORATORY TESTS		ESTIMATED COMPACTION (%)	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	SAMPLE TYPE	LITHOLOGY	U.S.C.S.	DESCRIPTION
0								SM	@ 0 feet, TOPSOIL: **SILTY SAND with GRAVEL, approximately 20% angular gravel, 10% coarse grained sand, 20% medium grained sand, 30% fine grained sand, 20% silty to clayey fines brown, dry, loose.**
									@ 1 foot, GRANITIC BEDROCK:**fine to coarse grained, highly fractured and moderately to highly weathered, easy digging in upper 1' with moderate to hard digging below that depth.**
									@ 2 feet, joints: N87W, 82E and N30E, 74S
									@ 3.5 feet, pratical refusal.
									END OF TRENCH
									No fill **Slight caving** **No groundwater** **Bedrock @1'**
5									
10									

PROJECT:	**Sawco 100,000 Gallon Water Tank**	PROJECT NUMBER:	**63314.1**
CLIENT:	**TKE Engineering, Inc.**	ELEVATION:	
LOR GEOTECHNICAL GROUP INC.		DATE EXCAVATED:	**November 18, 2016**
		EQUIPMENT:	**New Holland LB75B**
		BUCKET W.: **24"**	ENCLOSURE: **B-1**

LOG OF TRENCH T-2

DEPTH IN FEET	LABORATORY TESTS		ESTIMATED COMPACTION (%)	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	SAMPLE TYPE	LITHOLOGY	U.S.C.S.	DESCRIPTION
0								SM	@ 0 feet, TOPSOIL: **SILTY SAND, fine to coarse grained, dark brown, dry, loose.**
									@ 1 foot, COLLUVIUM: **SILTY SAND with GRAVEL, approximately 25% fine to coarse angular gravel, 15% coarse grained sand, 20% medium grained sand, 25% fine grained sand, 15% silty fines, light brown, dry, loose to medium dense.**
									@ 3 feet GRANITIC BEDROCK: **fine to coarse grained, moderately fractured, light brown, dry, hard, joint: N20E, 81N**
									@ 4 feet, joints: N84W, 72S and N20E, 23N.
5									**@ 5 feet, refusal**
									END OF TRENCH
									No fill **Slight caving** **No groundwater** **Bedrock @ 3'**
10									

PROJECT:	**Sawco 100,000 Gallon Water Tank**	PROJECT NUMBER:	**63314.1**
CLIENT:	**TKE Engineering, Inc.**	ELEVATION:	
LOR GEOTECHNICAL GROUP INC.		DATE EXCAVATED:	**November 18, 2016**
		EQUIPMENT:	**New Holland LB75B**
		BUCKET W.: **24"**	ENCLOSURE: **B-2**

LOG OF TRENCH T-3

DEPTH IN FEET	LABORATORY TESTS		ESTIMATED COMPACTION (%)	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	SAMPLE TYPE	LITHOLOGY	U.S.C.S.	DESCRIPTION
0								SM	@ 0 feet, TOPSOIL: SILTY SAND, fine to coarse grained, dark brown, dry, loose.
									@ 1 foot, COLLUVIUM: SILTY SAND with GRAVEL, approximately 25% fine to coarse gravel, 15% coarse grained sand, 20% medium grained sand, 25% fine grained sand, 15% silty fines, light brown, dry, loose to medium dense.
									@ 2.5 feet, GRANITIC BEDROCK:fine to coarse grained, moderately fractured, light brown, dry, hard. @ 3 feet, joint: N10E, 73N. @ 3.5 feet, joints: N34W, 80N and N15E, 81N @ 4.5 feet, refusal.
5									**END OF TRENCH** No fill Slight caving No groundwater Bedrock @ 2.5'
10									

PROJECT: **Sawco 100,000 Gallon Water Tank**	PROJECT NUMBER: **63314.1**
CLIENT: **TKE Engineering, Inc.**	ELEVATION:
LOR GEOTECHNICAL GROUP INC.	DATE EXCAVATED: **November 18, 2016**
	EQUIPMENT: **New Holland LB75B**
	BUCKET W.: **24"** ENCLOSURE: **B-3**

APPENDIX C

Laboratory Testing Program and Test Results

APPENDIX C
LABORATORY TESTING

General

Selected soil and bedrock samples obtained from the exploratory trenches were tested in our geotechnical laboratory to evaluate the physical properties of the soils/bedrock affecting foundation design and construction procedures. The laboratory testing program performed in conjunction with our investigation included moisture content, dry density, laboratory compaction, direct shear, and soluble sulfate content. Descriptions of the laboratory tests are presented in the following paragraphs:

Laboratory Compaction

Selected soil samples were tested in the laboratory to determine compaction characteristics using the ASTM D 1557 compaction test method. The results are presented in the following table:

LABORATORY COMPACTION				
Trench Number	**Sample Depth (feet)**	**Soil Description (U.S.G.S.)**	**Maximum Dry Density (pcf)**	**Optimum Moisture Content (percent)**
T-3	3-4	Remolded Bedrock	146.0	3.5

Direct Shear Tests

Shear tests are performed, in accordance with AMT. D 3080, with a direct shear machine at a constant rate-of-strain (usually 0.04 inches/minute). The machine is designed to test a sample partially extruded from a sample ring in single shear. Samples are tested at varying normal loads in order to evaluate the shear strength parameters, angle of internal friction and cohesion. Samples are tested in a remolded state and soaked, to represent the most unfavorable conditions expected in the field.

DIRECT SHEAR TESTS				
Trench Number	Sample Depth (feet)	Soil Description (U.S.G.S.)	Cohesion (psf)	Angle of Internal Friction (degrees)
T-3	3-4	Remolded Bedrock	550	30

Soluble Sulfate Content Tests

The soluble sulfate content of selected subgrade soils was evaluated. The concentration of soluble sulfates in the soils was determined by measuring the optical density of a barium sulfate precipitate. The precipitate results from a reaction of barium chloride with water extractions from the soil samples. The measured optical density is correlated with readings on precipitates of known sulfate concentrations. The test results are presented on the following table:

SOLUBLE SULFATE CONTENT TESTS			
Trench Number	Sample Depth (feet)	Soil Description	Sulfate Content (percent)
T-2	1-3	(SM) Silty Sand	<0.005
T-3	3-4	Remolded Bedrock	<0.005