Converse Consultants
Geotechnical Engineering
Environmental & Groundwater Science
Inspection & Testing Services

PRELIMINARY GEOTECHNICAL INVESTIGATION REPORT

UPPER RESERVOIR REPLACEMENT PROJECT
13655 FOOTHILL BOULEVARD
SYLMAR, LOS ANGELES COUNTY, CALIFORNIA

CONVERSE PROJECT NO. 19-31-168-01

Prepared For:

JOHN ROBINSON CONSULTING, INC.
Mr. John Robinson
1055 East Colorado Boulevard
Suite 500
Pasadena, California 91106

Presented By:

CONVERSE CONSULTANTS
717 South Myrtle Avenue
Monrovia, California 91016
626-930-1200

September 29, 2020

September 29, 2020

Mr. John Robinson
John Robinson Consulting, Inc.
1055 East Colorado Boulevard, Suite 500
Pasadena, California 91106

Subject: **PRELIMINARY GEOTECHNICAL INVESTIGATION REPORT**
Upper Reservoir Replacement Project
13655 Foothill Boulevard
Sylmar, Los Angeles County, California
Converse Project No. 19-31-168-01

Dear Mr. Robinson:

Converse Consultants (Converse) is pleased to present this Preliminary Geotechnical Investigation Report for the Upper Reservoir Replacement Project located at 13655 Foothill Boulevard in Sylmar, Los Angeles County, California.

The purpose of the study was to investigate the geotechnical site conditions and provide recommendations for the Upper Reservoir Replacement Project at the above-referenced project site. Our services were performed in accordance with our proposal dated April 3, 2019.

Based on our field exploration, laboratory testing, geologic evaluation and geotechnical analysis, the site is suitable from a geotechnical standpoint for the proposed project, provided our conclusions and recommendations are implemented during design and construction.

We appreciate this opportunity to be of service to John Robinson Consulting, Inc. If you should have any questions regarding this report, please contact us at (626) 930-1200.

Sincerely,

CONVERSE CONSULTANTS

Sivathasan

Siva K. Sivathasan, PhD, PE, GE, DGE, QSD, F. ASCE
Senior Vice President / Principal Engineer

REGISTERED PROFESSIONAL ENGINEER
KATHIRAVETPILLAI SIVATHASAN
No. 2708
Exp. 6-30-22
GEOTECHNICAL
STATE OF CALIFORNIA

PROFESSIONAL CERTIFICATION

This report for the Upper Reservoir Replacement Project located at 13655 Foothill Boulevard, in the Sylmar neighborhood of the Los Angeles County, California, has been prepared by the staff of Converse under the professional supervision of the individuals whose seals and signatures appear hereon.

The findings, recommendations, specifications, or professional opinions contained in this report were prepared in accordance with generally accepted professional engineering and engineering geologic principles and practice in this area of Southern California. There is no warranty, either expressed or implied.

In the event that changes to the property occur, or additional, relevant information about the property is brought to our attention, the conclusions contained in this report may not be valid unless these changes and additional relevant information are reviewed, and the recommendations of this report are modified or verified in writing.

Babak Abbasi, PhD, EIT
Senior Staff Engineer

PROFESSIONAL GEOLOGIST
MARK B. SCHLUTER
No. 1415
CERTIFIED ENGINEERING GEOLOGIST
STATE OF CALIFORNIA

Mark B. Schluter, PG, CEG, CHG
Senior Engineering Geologist

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Siva K. Sivathasan, PhD, PE, GE, DGE, QSD, F.ASCE
Senior Vice President/Principal Engineer

EXECUTIVE SUMMARY

The following is a summary of our geotechnical investigation, conclusions and recommendations as presented in the body of this report. Please refer to the appropriate sections of the report for complete conclusions and recommendations. In the event of a conflict between this summary and the report, or an omission in the summary, the report shall prevail.

- The proposed project site is located at 13655 Foothill Boulevard, in the Sylmar neighborhood of the Los Angeles County, California. The subject site's surface elevations range from approximately 1300 feet to 1315 feet relative to mean-sea-level (MSL), with a surface gradient sloping towards the southwest.

- The proposed project consists of replacing the existing 1 MG reservoir and related site improvements with a new reservoir. The existing Upper Reservoir is a partially buried circular reinforced concrete reservoir. The proposed reservoir will be replacing the existing reservoir in place at the Upper Reservoir site. The new 1 MG reservoir is partially buried square reinforced concrete reservoir, 78 feet long, and approximately 40-feet high. The continuous square foundation is going to support the reservoir at depth of approximately 20 feet below finished grade along north east side and at depth of approximately 10 feet below finished grade along south east side of new reservoir. The partially buried L-shaped 2.5 MG reservoir is located at distance of approximately 15 feet from the new reservoir on both north west and north east sides. The depth of foundations for 2.5 MG reservoir is approximately 20 feet below existing ground level.

- Two (2) exploratory borings (BH-1 and BH-2) were drilled within the project site to evaluate the subsurface earth materials for the proposed project on April 3, 2020. The borings were drilled using truck mounted drill rig with an 8-inch diameter hollow stem auger to a maximum depth of 47 feet below the existing ground surface (bgs). Each boring was visually logged by a Converse engineer and sampled at regular intervals and at changes in subsurface soils, in accordance with the Unified Soil Classification System.

- Groundwater was not encountered during drilling of our subsurface exploration borings to maximum depth of 47 feet bgs. Groundwater is not anticipated during construction.

- The project site is not located within a currently designated State of California Earthquake Fault Zone (formerly Alquist-Priolo Special Studies Zones) for surface fault rupture. The Alquist-Priolo Earthquake Fault Zoning Act requires the California Geological Survey to zone "active faults" within the State of California.

- Seismic design parameters based on the CBC 2019 code can be found in Section 6.1 Seismic Design Parameters.

- Site soils consisted primarily of silty sand and sand with layers and lenses of gravels and cobbles. These material types should be excavatable with heavy-duty earth moving, drilling, and trenching equipment.

- The sulfate content of the sampled soil corresponds to American Concrete Institute (ACI) exposure category S0 (soluble sulfate in soil is less than 0.1, percent by weight). Based on the site location and the results of chloride testing, we do not anticipate concrete structures will be exposed to external sources of chlorides, such as deicing chemicals, salt, brackish water, or seawater. ACI specifies exposure category C1 where concrete is exposed to moisture, but not to external sources of chlorides. The minimum electrical resistivity when saturated was 5,800 Ohm-cm. The value indicates that the tested soil is slightly corrosive to ferrous metals (if any) in contact with the soil.

- Continuous square footings are considered suitable for structure support provided the recommendations in this report are incorporated into the project plans and specifications are followed during site construction

- For non-building structures (e.g. signs, fence walls, short retaining walls, etc.), conventional footings can be used. Cast in drilled Hole (CIDH) deep foundation can be used for fence or retaining walls on the eastern boundary to avoid overburden pressure on the over-steepened slope descending to neighboring property.

- Footings may be designed based on an allowable net bearing capacity of 3,500 pounds per square foot (psf).

- A coefficient of friction of 0.35 may be used with the dead load forces. Passive earth pressure of 250 psf per foot of depth may be used for the sides of footings poured against compacted native soils. The maximum value of the passive earth pressure should be limited to 2,500 psf.

- The static settlement of reservoir supported on continuous and/or spread footings founded on compacted fill is anticipated to be less than 1.0 inch. Differential settlement is expected to be up to one-half (1/2) of the total settlement over a 30-foot span. Most of the footing settlement at the project site is expected to occur immediately after the application of the load.

Results of our investigation indicate that the site is suitable from a geotechnical standpoint for the proposed development, provided that the recommendations contained in this report are incorporated into the design and construction of the project.

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1.0 INTRODUCTION

This report contains the findings and recommendations of our geotechnical study performed at the site of the Upper Reservoir Replacement Project located at 13655 Foothill Boulevard, in the Sylmar neighborhood of the City of Los Angeles, Los Angeles County, California, as shown on Drawing No. 1, *Site Location Map*.

The purpose of the study was to evaluate the subsurface soil conditions and provide geotechnical recommendations and design recommendations for the design and construction of the proposed project, consistent with the current edition of California Building Code and other local jurisdiction requirements.

This report is written for the project described herein and is intended for use solely by John Robinson Consulting, Inc., City of San Fernando, and their design team. It should not be used as a bidding document but may be made available to the potential contractors for information on factual data only. For bidding purposes, the contractors should be responsible for making their own interpretation of the data contained in this report.

2.0 SITE AND PROJECT DESCRIPTION

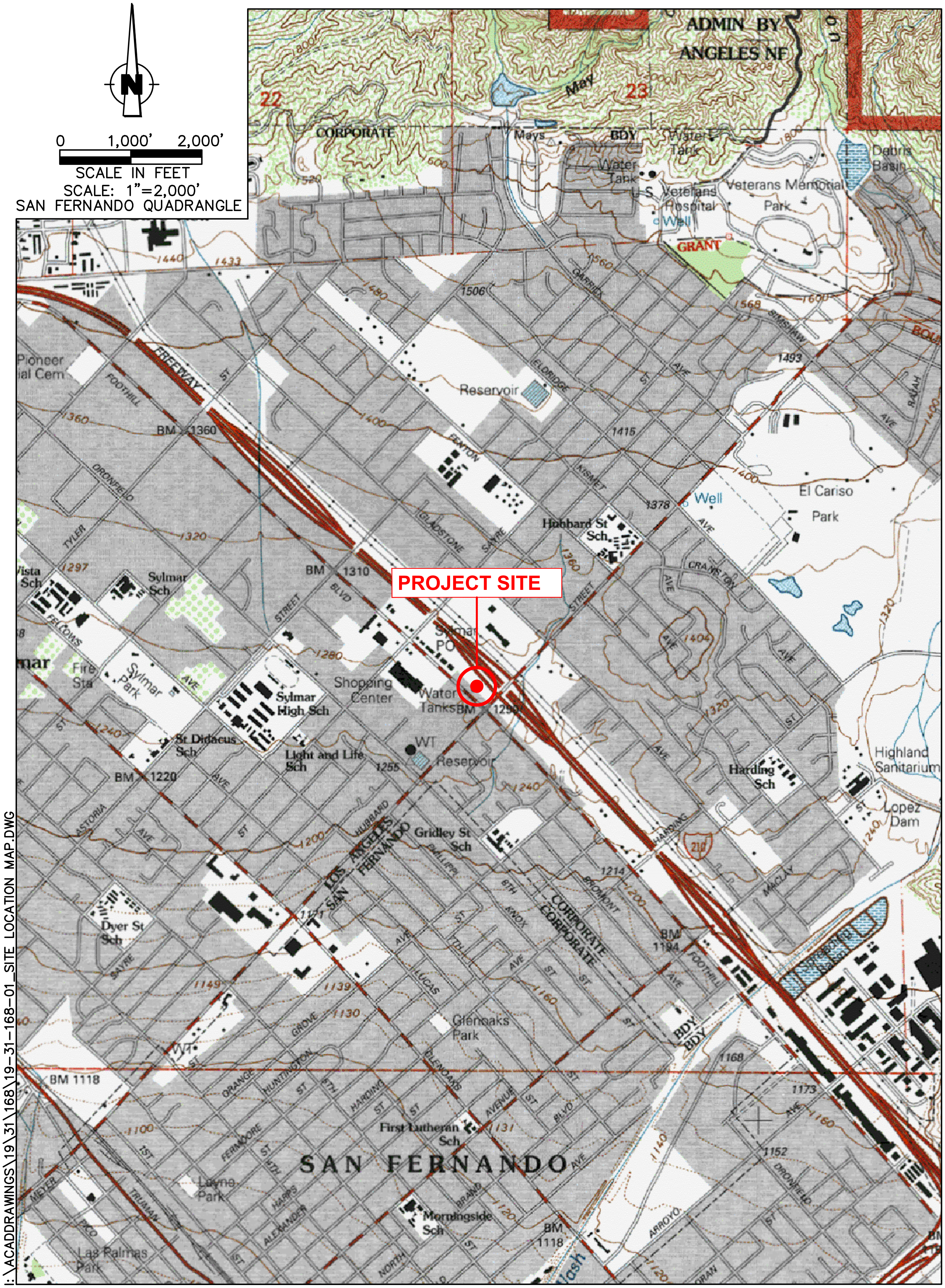
2.1 Site Description

The project site is located at 13655 Foothill Boulevard, in the Sylmar neighborhood of the Los Angeles County, California. The site is located west of Hubbard Street and north of Foothill Boulevard and south of the Interstate 210 Foothill Freeway. The Upper Reservoir Replacement Project is planned to be within the existing water reservoir facility site in Sylmar, California. The proposed new reservoir is planned to replace the current 1MG circular reservoir along the south side of the facility, as shown on Drawing No. 2, *Boring Location Map.* The subject site has surface elevations ranging from approximately 1300 to 1315 feet relative to mean-sea-level (MSL), respectively, with general surface gradients down toward the southwestward direction.

The site coordinates for the Upper Reservoir Replacement Project are: 34.30821 degrees North Latitude, 118.43078 degrees West Longitude. The site coordinates were centered on the subject site and used to calculate earthquake ground motions.

2.2 Project Description

The proposed project consists of replacing the existing 1 MG reservoir and related site improvements with a new reservoir. The existing Upper Reservoir is a partially buried circular reinforced concrete reservoir. The proposed reservoir will be replacing the existing reservoir in place at the Upper Reservoir site. The project site is shown on Drawing No. 2, *Boring Location Map*. The new 1 MG reservoir is partially buried square reinforced concrete reservoir, approximately 74 by 88 feet, and approximately 40-feet high. The continuous square foundation is going to support the reservoir at depth of



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SITE LOCATION MAP

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Project No.	Drawing No.
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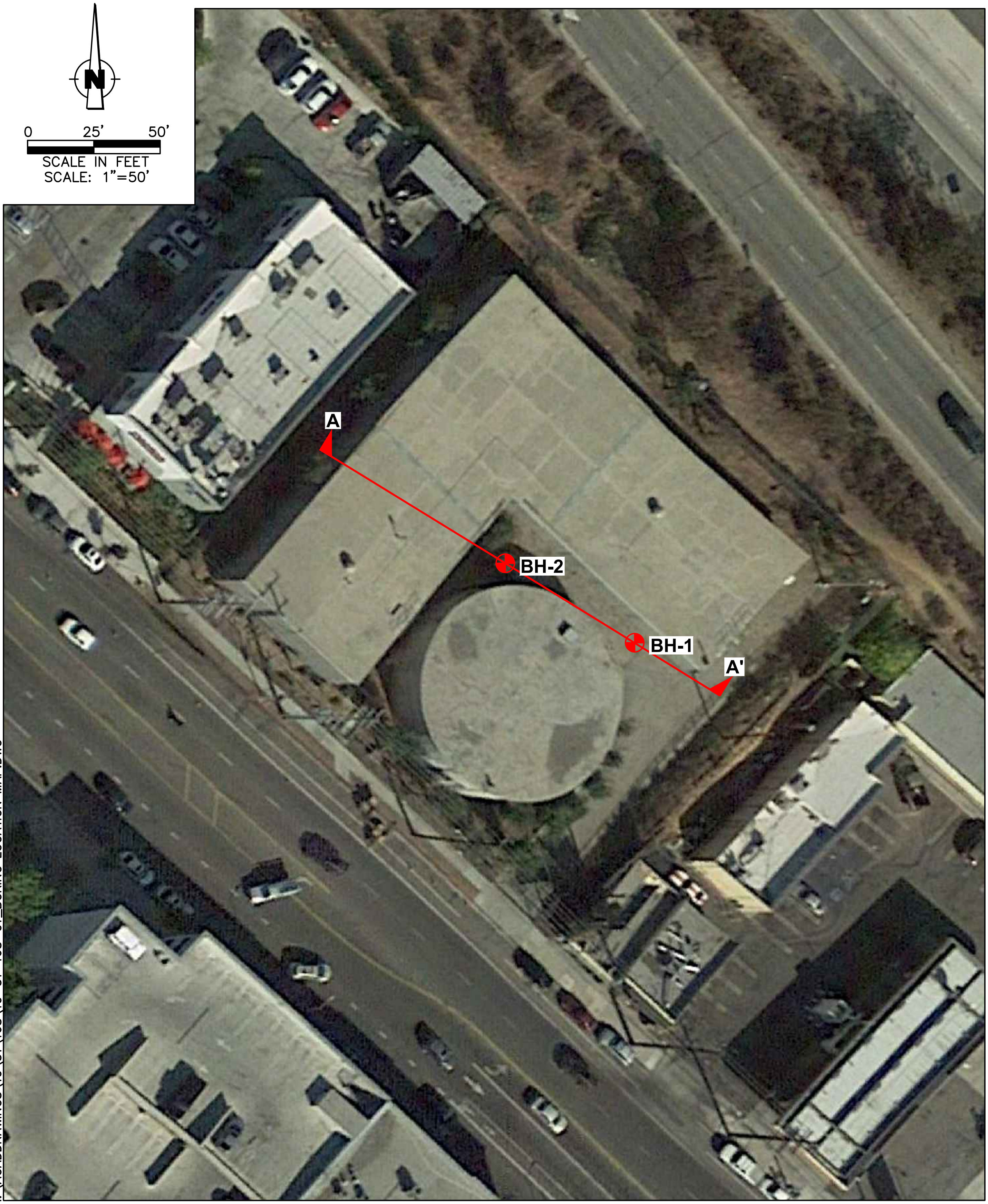

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BORING LOCATION MAP

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Project No.	Drawing No.
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approximately 20 feet below finished grade along north east side and at depth of approximately 10 feet below finished grade along south east side of new reservoir. The partially buried L-shaped 2.5 MG reservoir is located at distance of approximately 15 feet from the new reservoir on both north west and north east sides. The depth of foundations for 2.5 MG reservoir is approximately 20 feet below existing ground level.

3.0 SCOPE OF WORK

Our scope of work consists of the tasks described in the following subsections.

3.1 Site Reconnaissance and Data Review

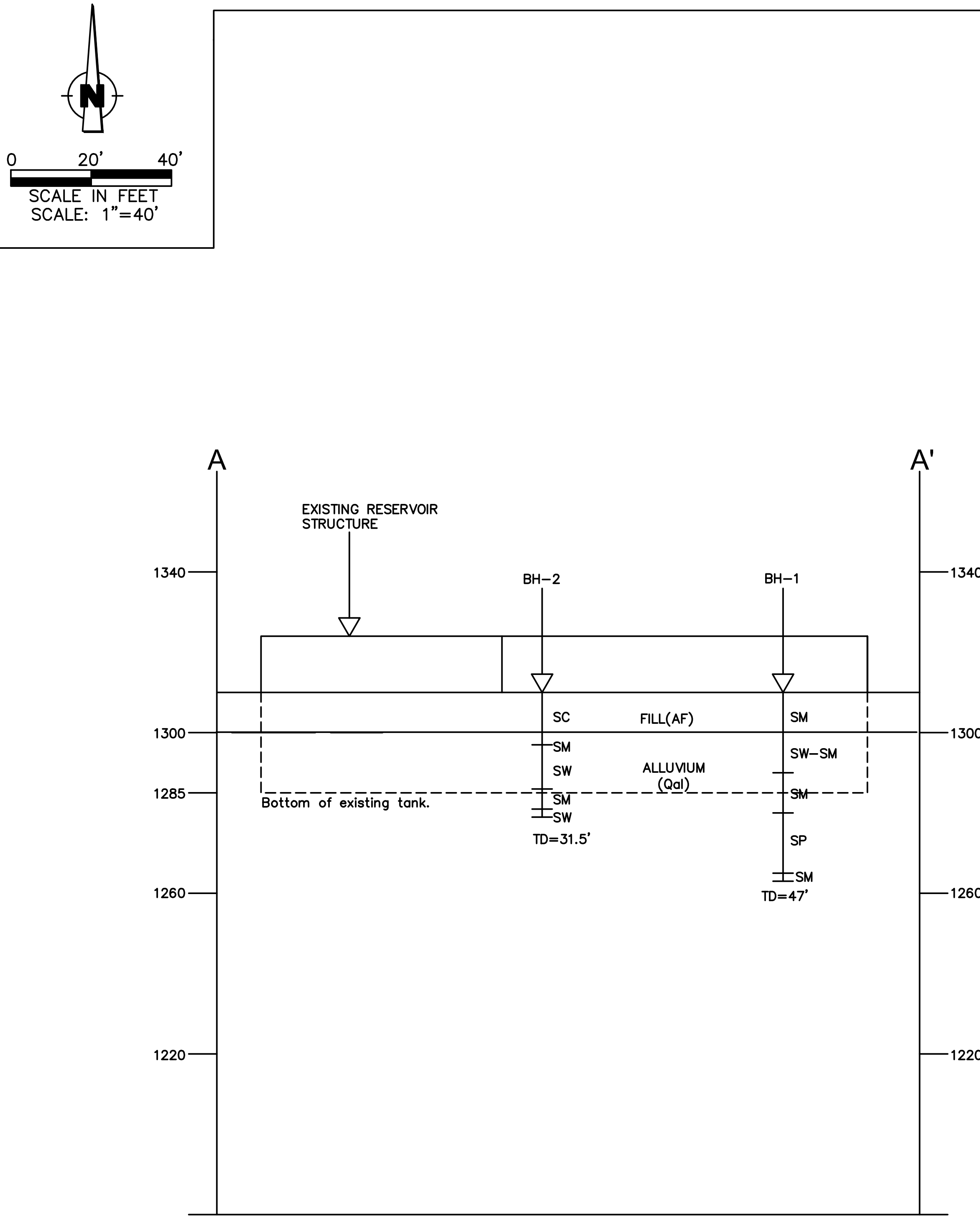
During the site reconnaissance on March 12, 2020, the surface conditions were noted, and the locations of the borings were determined. The borings were located using existing boundary features as a guide and should be considered accurate only to the degree implied by the method used. Underground Service Alert (USA) of Southern California was notified of our proposed drilling locations at least 48 hours prior to initiation of the subsurface field work.

3.2 Subsurface Exploration

Two (2) exploratory borings (BH-1 and BH-2) were drilled within the project site to evaluate the subsurface earth materials for the proposed project on April 3, 2020. The borings were drilled using truck mounted drill rig with an 8-inch diameter hollow stem auger to a maximum depth of 47 feet below the existing ground surface (bgs). Difficult drilling conditions were encountered in the gravels and cobbles at a depth of 47 feet. Each boring was visually logged by a Converse engineer and sampled at regular intervals and at changes in subsurface soils, in accordance with the Unified Soil Classification System. Detailed descriptions of the field exploration and sampling program are presented in Appendix A, *Field Exploration*.

California Modified Sampler (ring samples), Standard Penetration Test samples, and bulk soil samples were obtained for laboratory testing. Standard Penetration Tests (SPTs) were performed in selected borings at selected intervals using a standard (1.4 inches inside diameter and 2.0 inches outside diameter) split-barrel sampler. Borings extending into groundwater or deeper than 10 feet were backfilled with cement grout and capped to match surface conditions.

The approximate locations of the exploratory borings are shown in Drawing No. 2, *Boring Location Map.* Drawing No. 3, *Geologic Cross Section A-A'* shows geologic cross sections of the project site. For a description of the field exploration and sampling program, see Appendix A, *Field Exploration*.



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GEOLOGIC CROSS SECTION A-A'

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3.3 Laboratory Testing

Representative samples of the site soils were tested in the laboratory to aid in the classification and to evaluate relevant engineering properties. The tests performed included:

- In situ moisture contents and dry densities (ASTM Standard D2216)
- Grain size distribution (ASTM Standard C136)
- Maximum dry density and optimum-moisture content relationship (ASTM Standard D1557)
- Direct shear (ASTM Standard D3080)
- Soil corrosivity tests (Caltrans 643, 422, 417, and 532)

All the laboratory test results are presented in Appendix B, *Laboratory Testing Program* and Appendix A, *Field Exploration*. The log of the exploratory boring is presented in Drawing Nos. A-2 and A-3, *Log of Borings*. The test results are also summarized below.

Stratum A (Fill)- Silty Sand (SM) to Clayey Sand (SC)
As determined by laboratory testing, the United Soil Classification System (USCS) Group Symbol for this material was SM at BH-1, and SC at BH-2. The associated ASTM Group name was “Silty Sand with Gravel” at BH-1, and “Clayey Sand” at BH-2. The natural moisture content of Stratum A was measured at 8% to 9%. The sample tested from this stratum contained 23.9 percent material by weight finer than the No. 200 sieve.

Stratum B (Alluvial)- Sand (SP) to Silty Sand (SM)
As determined by laboratory testing, the United Soil Classification System (USCS) Group Symbol for this material varied at depths between 10 and 30 feet bgs from Sand (SP) to Silty Sand (SM). The associated ASTM Group name was “Poorly graded Sand” to “Silty Sand”. The natural moisture content of this material was measured in range of 5% to 8%.

Stratum C (Alluvial)- Sand (SP) to Silty Sand (SM)
As determined by laboratory testing, the United Soil Classification System (USCS) Group Symbol for this material varied at depths between 30 and 47 feet bgs from Sand (SP) to Silty Sand (SM). The associated ASTM Group name was “Poorly graded Sand” to “Silty Sand”. The natural moisture content of this material was measured in range of 5% to 6%. Stratum C has lighter brown color, more gravel content, and higher SPT blow counts compared to Stratum B.

3.4 Analyses and Report

Data obtained from the exploratory fieldwork and laboratory-testing program were analyzed and evaluated with respect to the planned construction. This report was prepared to provide the findings, conclusions and recommendations developed during our study and evaluation.

4.0 GEOLOGIC CONDITIONS

4.1 Regional Geologic Setting

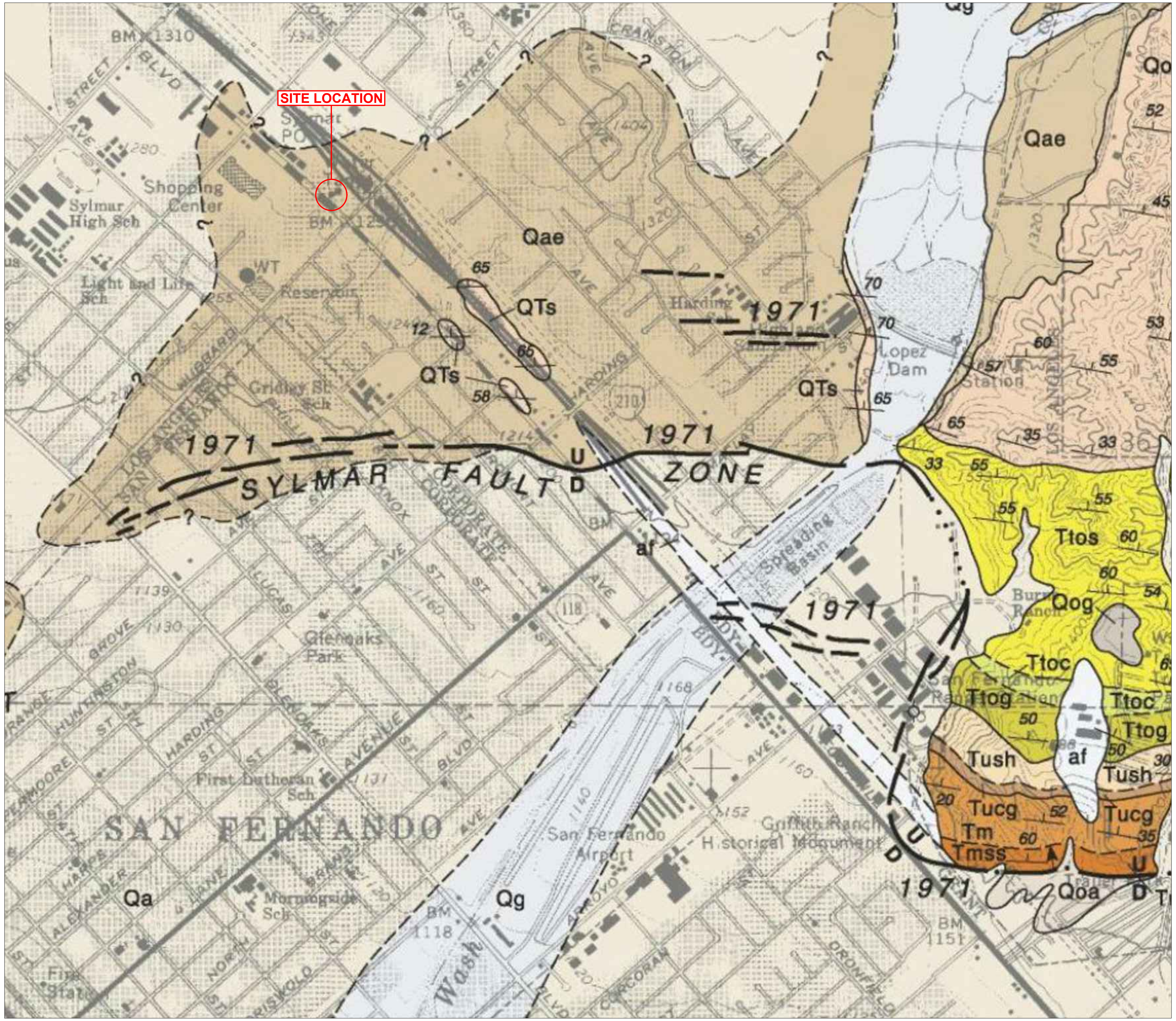
The project site is located within the northern portion of the San Fernando Valley basin, a broad sediment-filled basin located in the Transverse Ranges geomorphic province along the northern convergence of the Peninsular Ranges geomorphic province of California. Local stream channels and drainages have deposited stream and flood sediments across the valley basin along southern flank of the San Gabriel Mountains during Holocene time (0 – 11,000 years) to form a gently sloping alluvial fan that descends into the lower valley basin. Soils underlying the project site include deep alluvial deposits consisting of silty sands, sands, clayey sands with lenses and layers of gravels and cobbles. These alluvial sediments have been deposited over time by rivers and local stream tributaries which once drained across the valley basin to the Los Angeles River. Most of these natural river and stream channels are now controlled by dams, debris basins and flood control channels that collect surface runoff and convey storm water to the Los Angeles River channel and ocean. Drawing No. 4, *Regional Geologic Map*, has been prepared to show the project site with respect to regional geology of Sylmar and the northern San Fernando Valley.

4.2 Subsurface Profile of Project Site

Based on our soil borings drilled at the site on April 3, 2020, the subsurface conditions generally consist of existing fill soils placed during previous site grading operations over natural alluvial sediments. The exploratory borings were drilled to the maximum depth of 47 feet below the ground surface (bgs). Difficult drilling conditions were encountered in the gravel, cobble, and possible boulder size rock materials at various depths. Cobbles were also observed in drill cuttings. The observed fill soils consist primarily of silty sand and clayey sand. The depth of the fill observed was up to depths of approximately ten (10) feet below existing ground surfaces. The alluvial sediments consist predominately of silty sands and sands with layers and lenses of gravels and cobbles. Following table summarizes different soil layers at site.

Table No. 1, Summary of Strata at Site

Stratum A: (FILL)	From beneath the topsoil to depths of 10.0 feet.	dark brown, SILTY SAND (SM) to CLAYEY SAND (SC), fine to coarse-grained; loose to dense compact density, (N = 11 to 27)
Stratum B: (Alluvial)	Below Stratum A to a depth of 30 feet.	Brown, Sand (SP) to Silty Sand (SM), fine to coarse-grained, some gravel and trace clay, (N is in range of 55 to 80)
Stratum C: (Alluvial)	Below Stratum B to the maximum depth of penetration, 47 feet.	Yellow Brown, Sand (SP) to Silty Sand (SM), Some cobbles, increased drilling resistance and difficulty, (N = 100+)



N

NOT TO SCALE

SAN FERNANDO AND NORTH 1/2 VAN NUYS MAP (DF-33)

LEGEND

af / Qg / Qa

SURFICIAL SEDIMENTS

af *Artificial cut and fill*
Qg *Gravel and sand of major stream channels*
Qa *Alluvial gravel, sand and clay of valley areas*

Qls

LANDSLIDE DEBRIS

Qls *Landslide debris*

Qae / Qoa / Qog

OLDER DISSECTED SURFICIAL SEDIMENTS

Qae *Similar to* **Qa**, *but slightly elevated and dissected*
Qoa *Remnants of older, slightly consolidated, dissected alluvial gravel and sand*
Qog *High remnants of slightly consolidated, brown coarse alluvial gravel*

— *UNCONFORMITY* —

Qp

PACOIMA FORMATION
(Of Oakeshott 1958)
Terrestrial, weakly consolidated; Pleistocene age

Qp *Weakly consolidated brown alluvial gravel and sand derived from adjacent mountains*

— *UNCONFORMITY* —

QTs / Ts / Tsr

SAUGUS FORMATION
Terrestrial, weakly consolidated; Pliocene and Pleistocene age

QTs *Light gray pebble cobble conglomerate, sandstone and greenish to reddish claystone, interbedded; composed of locally derived granitic and gneissic detritus including some of anorthosite and gabbro-norite; deposited by streams; Pliocene and Pleistocene age*
Ts *(In Whitney Canyon area) same as* **QTs**, *but correlative in age (Pliocene) with units* **Tsr** *and* **Tps** *(in part lower member of Pico Formation of Oakeshott 1958)*
Tsr *Sunshine Ranch Member (of Oakeshott 1958; Winterer and Durham 1962, in part) in Van Norman Lake area; light reddish to greenish claystone and sandstone and interbedded pebble-cobble conglomerate similar to that of* **QTs**

Tps

PICO FORMATION
(Of Oakeshott 1958; Winterer and Durham 1962)
Shallow marine regressive; Pliocene (?) age

Tps *(At Los Angeles aqueduct) light gray, semi-friable, cross-bedded medium to coarse grained sandstone and some pebble-cobble conglomerate of granitic detritus*

Ttoc / Ttos / Ttog

TOWSLEY FORMATION
(Of Winterer and Durham 1962; Pico Formation of Hill 1930; "Repetto" Formation including Elsmere Member, of Oakeshott 1958)
Terrestrial and shallow marine regressive; latest Miocene age and/or early Pliocene age

Ttos *Mostly light gray to tan sandstone; minor gray silty claystone and conglomerate*
Ttoc *Gray silty claystone and siltstone, minor sandstone*
Ttog *Gray to brown crudely bedded cobble conglomerate of mostly granitic and gneissic detritus and coarse grained sandstone; probably terrestrial, deposited by streams as a fan delta*

Holocene | Pleistocene | Pliocene | QUATERNARY | ?ZOIC | ? | ? | ? | ?

REFERENCE: THOMAS W. DIBBLE, Jr. (DF-33)
GEOLOGIC MAP OF THE SAN FERNANDO AND VAN NUYS (NORTH 1/2) QUADRANGLES 1991

REGIONAL GEOLOGIC MAP

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13655 Foothill Boulevard
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Drawing No. 3, *Geologic Cross Section A-A'*, has been drawn across the project site to illustrate the subsurface conditions. For additional information on the subsurface conditions, see the Logs of Boring Data in Appendix A, Field Exploration.

4.3 Groundwater

Groundwater was not encountered during our subsurface exploration. Based on review of Historically Highest Groundwater Map, Plate No. 1.2 in the Seismic Hazard Zone report for the San Fernando 7.5-Minute Quadrangle (CDMG, 1998), the historically highest groundwater contour level is approximately 150 feet below existing ground surface.

In general, groundwater levels fluctuate with the seasons and local zones of perched groundwater may be present within the near-surface deposits due to local conditions or during rainy seasons. Groundwater conditions below any given site vary depending on numerous factors including seasonal rainfall, local irrigation, storm water recharge, groundwater basin recharge and pumping, among other factors. The regional groundwater table is not expected to be encountered during the planned construction.

4.4 Subsurface Variations

Based on results of the subsurface exploration and our experience, some variations in the continuity and nature of subsurface conditions within the project site should be anticipated. Because of the uncertainties involved in the nature and depositional characteristics of the earth material at the site, care should be exercised in interpolating or extrapolating subsurface conditions between or beyond the boring locations. If, during construction, subsurface conditions differ significantly from those presented in this report, this office should be notified immediately so that recommendations can be modified, if necessary.

5.0 FAULTING AND GEOLOGIC HAZARDS

Geologic hazards are defined as geologically related conditions that may present a potential danger to life and property. Typical geologic hazards in Southern California include earthquake ground shaking, fault surface rupture, liquefaction and seismically induced settlement, lateral spreading, landslides, earthquake induced flooding, tsunamis and seiches, and volcanic eruption hazard.

Results of a site-specific evaluation for each type of possible seismic hazards are discussed in the following sections.

5.1 Seismic Characteristics of Nearby Faults

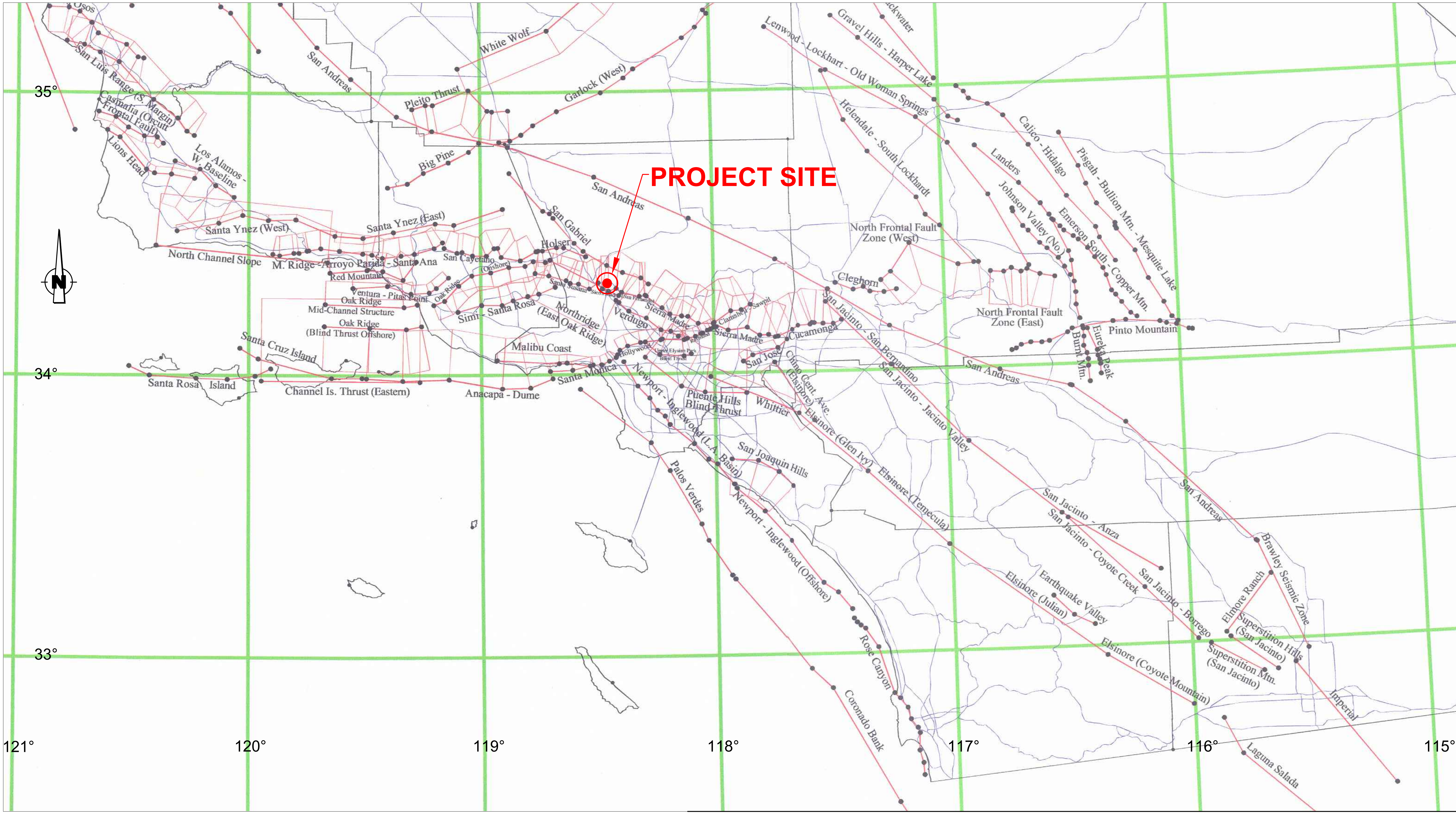
The subject site is situated within a seismically active region. As is the case for most areas of Southern California, strong ground-shaking resulting from earthquakes associated with nearby and more distant faults may occur at the project site. During the life of the project, seismic activity associated with active faults can be expected to generate moderate to strong ground shaking at the project site.

The project site is not located within a currently designated State of California Earthquake Fault Zone (formerly Alquist-Priolo Special Studies Zones) for surface fault rupture. No surface faults are known to project through or towards the site. The closest known fault with the potential for surface rupture is the Sylmar Fault that ruptured in 1971 during the M6.6 San Fernando Earthquake. The Sylmar Fault trace is located approximately 0.47 mile to the south of the project site. As a result, the potential for surface rupture resulting from the movement of this fault or other nearby faults is considered to be low. The approximate locations of local and regional active faults with respect to the project site are shown on Drawing No. 5, *Southern California Regional Fault Map*. The mapped epicenters of earthquakes with magnitude 5.0 or greater in Southern California during the past 200 years are shown on Drawing No. 6, *Epicenter Map of Southern California Earthquakes (1800-1999)*.

There are a number of regional fault systems, which could produce ground shaking at the site during a major earthquake. Table No. 1, *Summary of Regional Faults,* shows the location of the known most capable faults with respect to the site within 50 kilometers. The data presented below are based on updated fault data from “2008 National Seismic Hazard Maps” from U.S. Geological Survey (USGS) website.

Table No. 2, Summary of Regional Faults

Fault Name and Section	Approximate Distance to Site (kilometers)	Max. Moment Magnitude (Mmax)	Slip Rate (mm/yr)
Elsinore;W	3.76	7.03	2.5
Puente Hills (Santa Fe Springs)	8.85	6.7	0.7
San Jose	9.72	6.7	0.5
Puente Hills (Coyote Hills)	13.42	6.9	0.7
Elysian Park (Upper)	13.75	6.7	1.3
Puente Hills (LA)	14.5	7	0.7
Sierra Madre	16.72	7.2	2
Sierra Madre Connected	16.72	7.3	2
Raymond	16.9	6.8	1.5
Clamshell-Sawpit	19.62	6.7	0.5
Chino, alt 2	21.4	6.8	1
Chino, alt 1	21.42	6.7	1
Verdugo	21.9	6.9	0.5
Cucamonga	26.46	6.7	5
Hollywood	26.97	6.7	1
Newport Inglewood Connected alt 2	27.97	7.5	1.3
Newport-Inglewood, alt 1	28.63	7.2	1



REFERENCE: PORTION OF CGS 2002 CALIFORNIA FAULT MODEL MODIFIED FOR USE WITH FRISKSP AND EQFAULT BY THOMAS F. BLAKE, AUGUST 2004

FAULT SOURCES

BLIND THRUST FAULT, POLYGONS INDICATE RUPTURE PLANES AND DIP DIRECTION

SOUTHERN CALIFORNIA REGIONAL FAULT MAP

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Upper Reservoir Replacement Project
Geotechnical Investigation
13655 Foothill Boulevard
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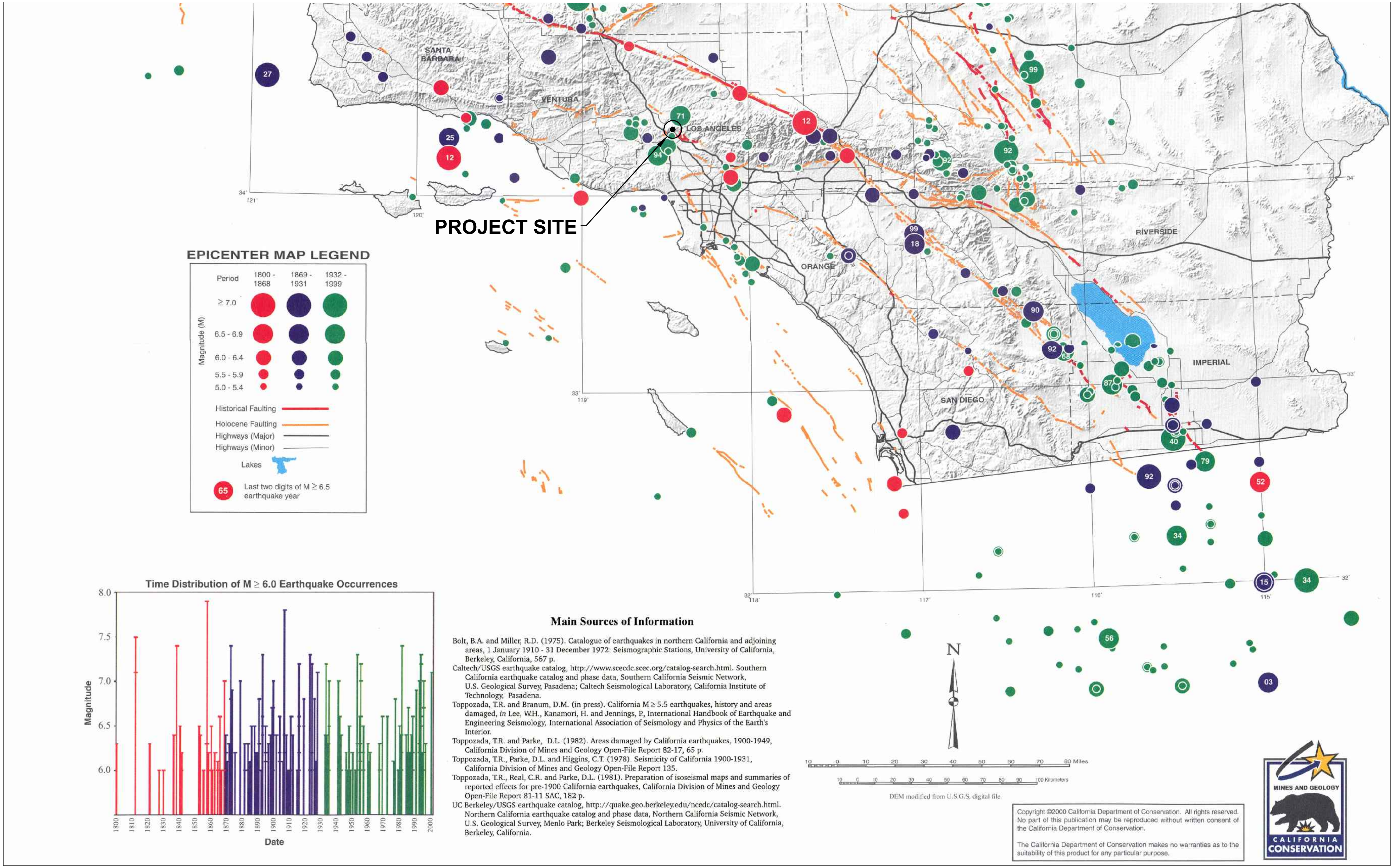
Drawing NO. 5

Date APRIL 2020



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Toppozada, T.R. and Branum, D.M. (in press). California M ≥ 5.5 earthquakes, history and areas damaged, *in* Lee, W.H., Kanamori, H. and Jennings, P., International Handbook of Earthquake and Engineering Seismology, International Association of Seismology and Physics of the Earth's Interior.

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Toppozada, T.R., Real, C.R. and Parke, D.L. (1981). Preparation of isoseismal maps and summaries of reported effects for pre-1900 California earthquakes, California Division of Mines and Geology Open-File Report 81-11 SAC, 182 p.

UC Berkeley/USGS earthquake catalog, http://quake.geo.berkeley.edu/ncedc/catalog-search.html. Northern California earthquake catalog and phase data, Northern California Seismic Network, U.S. Geological Survey, Menlo Park; Berkeley Seismological Laboratory, University of California, Berkeley, California.

REFERENCE: PORTION OF EPICENTERS AND AREAS DAMAGED BY M≥5 CALIFORNIA EARTHQUAKES, 1800-1999 CALIFORNIA DEPARTMENT OF CONSERVATION, MAP SHEET 49 DATED 2000.

EPICENTER MAP OF SOUTHERN CALIFORNIA EARTHQUAKES (1800-1999)

Converse Consultants

Upper Reservoir Replacement Project
Geotechnical Investigation
13655 Foothill Boulevard
Sylmar, CA 91342

Project No.	Drawing No.
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Fault Name and Section	Approximate Distance to Site (kilometers)	Max. Moment Magnitude (Mmax)	Slip Rate (mm/yr)
Newport Inglewood Connected alt 1	28.63	7.5	1.3
Santa Monica Connected alt 2	30.98	7.4	2.4
San Joaquin Hills	34.26	7.1	0.5
Palos Verdes Connected	38.33	7.7	3
Palos Verdes	38.33	7.3	3
Elsinore;GI	40.38	6.89	5
Elsinore;GI+T	40.38	7.29	5
Santa Monica Connected alt 1	41.08	7.3	2.6
Santa Monica, alt 1	41.08	6.6	1
Sierra Madre (San Fernando)	42.61	6.7	2
San Gabriel	44.98	7.3	1
Newport-Inglewood (Offshore)	45.91	7	1.5
S. San Andreas;SM	48.86	7.31	29
San Jacinto;SBV	49.7	7.06	6

5.2 Seismic History

We have reviewed California Geologic Survey Map Sheet 49; *Epicenters and Areas Damaged by M ≥ 5 California Earthquakes,* 1800-1999, (CGS, Toppozada et al., 2000). The mapped epicenters of earthquake with magnitude 5.0 or greater in Southern California during the past 200 years are shown on Drawing No. 6, *Epicenter Map of Southern California Earthquakes (1800-1999).*

5.3 Surface Fault Rupture

The project site is not located within a currently designated State of California Earthquake Fault Zone (formerly Alquist-Priolo Special Studies Zones) for surface fault rupture. The Alquist-Priolo Earthquake Fault Zoning Act requires the California Geological Survey to zone "active faults" within the State of California. An "active fault" has exhibited surface displacement with Holocene time (within the last 11,000 years) hence constituting a potential hazard to structures that may be located across it. The active fault set-back distance is measured perpendicular from the dip of the fault plane. The project site is located approximately 0.47 miles north of the mapped active fault trace of the Sylmar Fault that ruptured in 1971. Based on a review of existing geologic information, no known active faults project through or toward the site. The potential for surface rupture resulting from the movement of the nearby faults is considered low.

5.4 Liquefaction and Seismically-Induced Settlement

Liquefaction is the sudden decrease in the strength of cohesionless soils due to dynamic or cyclic shaking. Saturated soils behave temporarily as a viscous fluid (liquefaction) and, consequently, lose their capacity to support the structures founded on them. The potential for liquefaction decreases with increasing clay and gravel content but increases as the ground acceleration and duration of shaking increase. Liquefaction potential has been found to be the greatest where the groundwater level and loose sands occur within 50 feet of the ground surface.

The site is not located within potential liquefaction zones per the State of California Seismic Hazard Zones Map for the San Fernando Quadrangle as shown in Drawing No. 7, *Seismic Hazard Zones Map*. Based on our field data at BH-1, there is no liquefiable layer within 47 feet below ground level, and since highest historical ground water contour level is approximately 150 feet below ground surface, the project site is not susceptible to liquefaction.

5.5 Lateral Spreading

Seismically induced lateral spreading involves primarily lateral movement of earth materials due to ground shaking. It differs from the slope failure in that complete ground failure involving large movement does not occur due to the relatively smaller gradient of the initial ground surface. Lateral spreading is demonstrated by near-vertical cracks with predominantly horizontal movement of the soil mass involved. The project site is located within a gently sloping alluvial valley. The Interstate 210 Foothill Freeway and Hubbard Street off ramp have been graded with slopes along the north side of the project site. The potential for seismically induced lateral spreading to affect the proposed site is considered to be low.

5.6 Seismically-Induced Slope Instability

Seismically induced landslides and other slope failures are common occurrences during or soon after earthquakes. The project sites are also not shown with any earthquake induced landslide areas due to the relatively flat condition of the valley basin and site topography. In the absence of significant ground slopes, the potential for seismically induced landslides to affect the proposed site is considered to be low.

5.7 Earthquake-Induced Flooding

Review of the Flood Insurance Rate Map (FIRM), Map Number 06037C1075F, effective date September 26, 2008, from the Map Service Center (MSC) viewer, indicates that the site is designated as Zone “X”, “Areas of minimal flood hazard”.

The potential of earthquake induced flooding of the subject site is considered to very low.

EARTHQUAKE FAULT ZONES

Earthquake Fault Zones
Zone boundaries are delineated by straight-line segments; the boundaries define the zone encompassing active faults that constitute a potential hazard to structures from surface faulting or fault creep such that avoidance as described in Public Resources Code Section 2621.5(a) would be required.

Active Fault Traces
Faults considered to have been active during Holocene time and to have potential for surface rupture: Solid Line in Black or Red where Accurately Located; Long Dash in Black or Solid Line in Purple where Approximately Located; Short Dash in Black or Solid Line in Orange where Inferred; Dotted Line in Black or Solid Line in Rose where Concealed; Query (?) indicates additional uncertainty. Evidence of historic offset indicated by year of earthquake-associated event or C for displacement caused by fault creep.

1906 C ?

SEISMIC HAZARD ZONES

Liquefaction Zones
Areas where historical occurrence of liquefaction, or local geological, geotechnical and ground water conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693(c) would be required.

Earthquake-Induced Landslide Zones
Areas where previous occurrence of landslide movement, or local topographic, geological, geotechnical and subsurface water conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693(c) would be required.

Overlapping Liquefaction and Earthquake-Induced Landslide Zones
Areas that lie within zones of required investigation for both liquefaction and earthquake-induced landslides.

OVERLAPPING EARTHQUAKE FAULT AND SEISMIC HAZARD ZONES

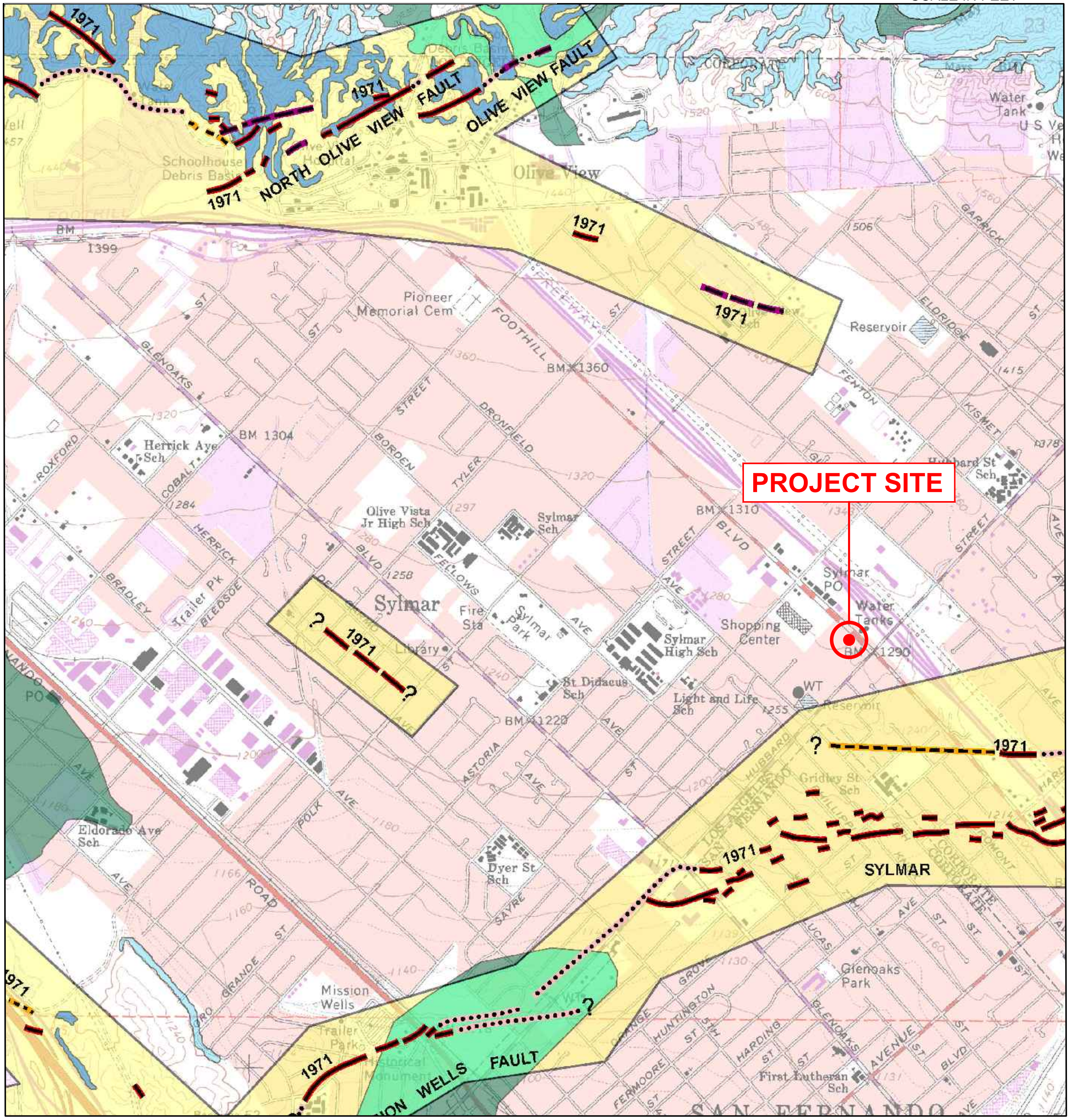
Overlap of Earthquake Fault Zone and Liquefaction Zone
Areas that are covered by both Earthquake Fault Zone and Liquefaction Zone.

Overlap of Earthquake Fault Zone and Earthquake-Induced Landslide Zone
Areas that are covered by both Earthquake Fault Zone and Earthquake-Induced Landslide Zone.

Note: Mitigation methods differ for each zone –
AP Act only allows avoidance; Seismic Hazard Mapping Act allows mitigation by engineering/geotechnical design as well as avoidance.

N

0 1,000' 2,000'
SCALE IN FEET



REFERENCE:SAN FERNANDO QUADRANGLE 1999
SEISMIC HAZARD ZONES STATE OF CALIFORNIA

SEISMIC HAZARD ZONES MAP

Converse Consultants

Upper Reservoir Replacement Project
Geotechnical Investigation
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5.8 Tsunami and Seiches

Tsunamis are tidal waves generated by fault displacement or major ground movement. Based on the location of the site from the ocean and the project site elevation, tsunamis do not pose a hazard. Seiches are large waves generated in enclosed bodies of water in response to ground shaking. The project site contains two water storage reservoirs. Based on site elevations and distances from offsite reservoirs, offsite seiches pose a low hazard to the project site. A potential seiche could occur within the onsite reservoirs from strong ground shaking and should be mitigated with appropriate design measures.

5.9 Volcanic Eruption Hazard

There are no known volcanoes near the site. According to Jennings (1994), the nearest potential hazards from future volcanic eruptions is the Amboy Crater-Lavic Lake area located in the Mojave Desert more than 112 miles northeast of the site. Volcanic eruption hazards are not present.

6.0 SEISMIC ANALYSIS

6.1 CBC Seismic Design Parameters

General seismic parameters based on the 2019 California Building Code and ASCE 7-16 with Supplement 1 are calculated using the ATC hazard, *Seismic Design by location* website application and the site coordinates for (34.30821 degrees North Latitude, 118.43078 degrees West Longitude). The seismic parameters are presented below.

Table No. 3, CBC Seismic Design Parameters

Seismic Parameter	Value
Site Class	C
Mapped Short period (0.2-sec) Spectral Response Acceleration, S_S	2.649 g
Mapped 1-second Spectral Response Acceleration, S_1	0.870 g
Site Coefficient (from Table 1613.5.3(1)), F_a	1.2
Site Coefficient (from Table 1613.5.3(2)), F_v	1.4
MCE 0.2-sec period Spectral Response Acceleration, S_{MS}	3.179 g
MCE 1-second period Spectral Response Acceleration, S_{M1}	1.218 g
Design Spectral Response Acceleration for short period, S_{DS}	2.119 g
Design Spectral Response Acceleration for 1-second period, S_{D1}	0.812 g

6.2 Site-Specific Response Spectra

A site-specific response spectrum was developed for the project for a Maximum Considered Earthquake (MCE), defined as a horizontal peak ground acceleration that has a 2 percent probability of being exceeded in 50 years (return period of approximately 2,475 years).

In accordance with ASCE 7-16, Section 21.2 the site-specific response spectra can be taken as the lesser of the probabilistic maximum rotated component of MCE ground motion and the 84th percentile of deterministic maximum rotated component of MCE ground motion response spectra. The design response spectra can be taken as 2/3 of site-specific MCE response spectra but should not be lower than 80 percent of CBC general response spectra. The risk coefficient C_R has been incorporated at each spectral response period for which the acceleration was computed in accordance with ASCE 7-16, Section 21.2.1.1.

The 2019 CBC mapped acceleration parameters are provided in the following table. These parameters were determined using the *ATC hazard by location Seismic Design Maps* website application, and in accordance with ASCE 7-16 Sections 11.4, 11.6, 11.8 and 21.2.

Table No. 4, 2019 CBC Mapped Acceleration Parameters

Site Class	C	Seismic Design Category	E
S_s	2.649	**C_{RS}**	0.905
S_1	0.870	**C_{R1}**	0.892
F_a	1.2	**0.08 F_v/F_a**	0.093
F_v	1.4	**0.4 F_v/F_a**	0.467
S_{MS}	3.179	**T_0**	0.077
S_{M1}	1.218	**T_S**	0.383
S_{DS}	2.119	**T_L**	8
S_{D1}	0.812		

A site-specific response analysis, using faults within 200 kilometers of the sites, was developed using the computer program EZ-FRISK Version 8.06 (Fugro, 2019).

The weighted mean maximum-rotated horizontal spectral acceleration values were computed by multiplying the weighted mean geometric spectral values derived from four next-generation attenuation (NGA) West 2 ground motion attenuation models by Abrahamson et al. (2014), Boore et al. (2014), Campbell and Bozorgnia (2014), and Chiou and Youngs (2014) with the scale factors provided in ASCE 7-16 Section 21.2. An average shear wave velocity at upper 30 meters of soil profile (V_{s30}) of 550 meters per second, depth to bedrock of with a shear wave velocity 1,000 meters per second at 150 meters below grade, and depth of bedrock where the shear wave velocity is 2,500 meters per second at 3,000 meters below grade were selected for EZ-Frisk Analysis.

The probabilistic response spectrum results and peak ground acceleration for each attenuation relationship are presented in the following table.

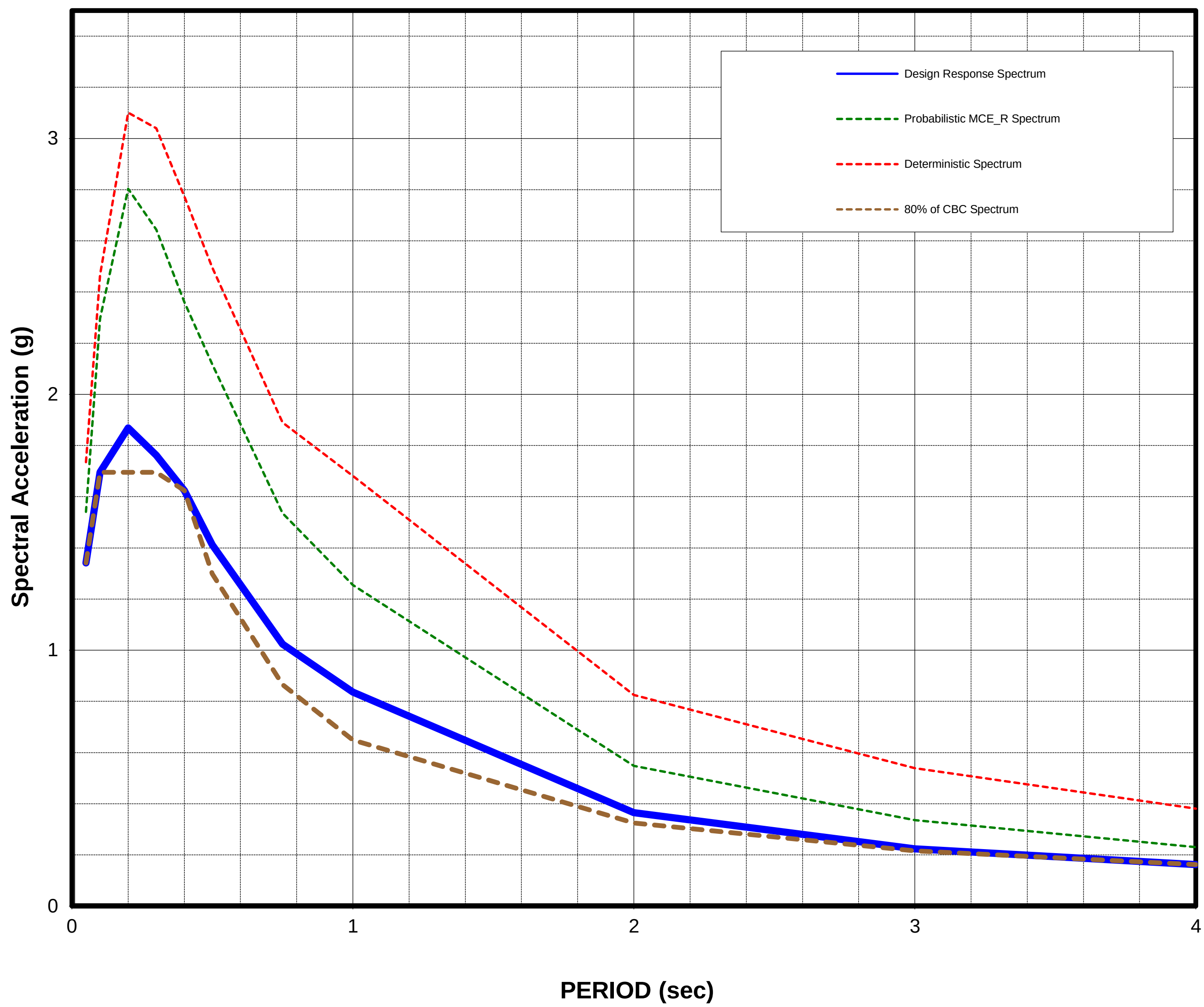
Table No. 5, Probabilistic Response Spectrum Data

Attenuation Relationship	Probabilistic Mean	Abrahamson et al. (2014)	Boore et al. (2014)	Campbell-Bozorgnia (2014)	Chiou-Youngs (2014)
Peak Ground Acceleration (g)	1.116	1.168	1.076	1.069	1.150
Spectral Period (sec)	**2% in 50yr Probabilistic Spectral Acceleration (g)**				
0.050	1.548	1.276	1.511	1.695	1.677
0.100	2.306	2.001	2.418	2.294	2.499
0.200	2.816	3.215	2.631	2.359	3.025
0.300	2.601	2.666	2.384	2.486	2.868
0.400	2.276	2.084	2.116	2.330	2.566
0.500	2.002	1.700	1.849	2.031	2.300
0.750	1.385	1.125	1.286	1.488	1.622
1.000	1.082	0.869	1.027	1.166	1.224
2.000	0.456	0.397	0.389	0.509	0.519
3.000	0.269	0.235	0.233	0.317	0.283
4.000	0.178	0.164	0.169	0.205	0.170

Applicable response spectra data are presented in the table below and on Drawing No. 8, *Site-Specific Design Response Spectrum.* These curves correspond to response values obtained from above attenuation relations for horizontal elastic single-degree-of-freedom systems with equivalent viscous damping of 5 percent of critical damping.

Table No. 6, Probabilistic MCE_R Spectral Acceleration (g)

Period (sec)	2% in 50yr Probabilistic Spectral Acceleration (g) Geometric Mean	Risk Coefficient C_R	Scale Factors for MCE_R	Probabilistic MCE_R Spectral Acceleration (g)
0.05	1.55	0.905	1.100	1.541
0.10	2.31	0.905	1.100	2.296
0.20	2.82	0.905	1.100	2.803
0.30	2.60	0.903	1.125	2.643
0.40	2.28	0.902	1.150	2.360
0.50	2.00	0.900	1.175	2.117
0.75	1.39	0.896	1.238	1.536
1.00	1.08	0.892	1.300	1.255
2.00	0.46	0.892	1.350	0.549
3.00	0.27	0.892	1.400	0.336
4.00	0.18	0.892	1.450	0.230



Note: Calculated using EZFRISK program Risk Engineering, version 8.06

SITE SPECIFIC DESIGN RESPONSE SPECTRUM

Upper Reservoir Replacement Project Sylmar For : City of Sylmar/ / John Robinson Consulting, Inc.	Project Number: 19-31-168-01
Converse Consultants	Drawing No. 8

Table No. 7, Site-Specific Response Spectrum Data

Period (sec)	84th Percentile Deterministic Response Spectrum, (g) Geometric Mean	Scale Factors for MCE_R	84th Percentile Deterministic MCE Response Spectrum (g)	Site Specific MCE_R Spectral Acceleration (g)	80% CBC Design Response Spectrum	Site Specific Design Spectral Acceleration (g)
0.05	1.577	1.100	1.735	1.541	1.342	1.34
0.10	2.242	1.100	2.466	2.296	1.695	1.70
0.20	2.819	1.100	3.101	2.803	1.695	1.87
0.30	2.702	1.125	3.040	2.643	1.695	1.76
0.40	2.411	1.150	2.773	2.360	1.624	1.62
0.50	2.124	1.175	2.496	2.117	1.299	1.41
0.75	1.527	1.238	1.890	1.536	0.866	1.02
1.00	1.293	1.300	1.681	1.255	0.650	0.84
2.00	0.611	1.350	0.825	0.549	0.325	0.37
3.00	0.385	1.400	0.539	0.336	0.217	0.22
4.00	0.263	1.450	0.381	0.230	0.162	0.16

The site-specific design response parameters are provided in the following table. These parameters were determined from Design Response Spectra presented in table above and following guidelines of ASCE Section 21.4.

The site-specific parameters (Table No. 7) can be used for design instead of mapped parameters (Table No. 2 and No. 3).

Table No. 8, Site-Specific Seismic Design Parameters

Parameter	Value (5% Damping)	Lower Limit, 80% of CBC Design Spectra
Site-Specific 0.2-second period Spectral Response Acceleration, S_{MS}	2.543	2.543
Site-Specific1-second period Spectral Response Acceleration, S_{M1}	1.255	0.974
Site-Specific Design Spectral Response Acceleration for short period S_{DS}	1.682	1.695
Site-Specific Design Spectral Response Acceleration for 1-second period, S_{D1}	0.836	0.650

7.0 EARTHWORK AND SITE GRADING RECOMMENDATIONS

7.1 General Evaluation

Based on our field exploration, laboratory testing, and analyses of subsurface conditions at the site, remedial grading will be required to prepare the sites for support of the proposed reservoir that is planned to be constructed with conventional shallow footings. To reduce differential settlement, variations in the soil type, degree of compaction, and thickness of the compacted fill, the thickness of compacted fill placed underneath the shallow footings should be kept uniform.

Site grading recommendations provided below are based on our experience with similar projects in the area and our evaluation of this investigation. Site preparation for the proposed development will require removal of existing structures, improvements, and other existing underground manmade structures and utilities.

The site soils can be excavated utilizing conventional heavy-duty earth-moving equipment. The excavated site soils, free of vegetation, organics, debris, and oversize rock materials may be placed as compacted fill in structural areas after proper processing. Rocks larger than three (3.0) inches in the largest dimension should not be placed as fill.

On-site fine-grained soils (clays and silts) and with an expansion index exceeding 20 should not be re-used for compaction within 2 feet below the proposed shallow foundations and slabs on grade. Soils containing organic materials (roots, grass, plants, etc.) should not be used as structural fill. The extent of removal should be determined by the geotechnical representative based on soil observation during grading.

7.2 Over-Excavation and Re-compaction

Prior to the start of construction, all loose soils, fill and soils disturbed during demolition should be removed to firm and unyielding compacted fill soils or to acceptable native materials. Uncertified fill at the bottom of existing reservoir is not anticipated, however, in case of encountering such fills they should be removed and treated as mentioned above. In order to provide uniform support for the structures on shallow foundations, the minimum depth of over-excavation should be at least 5 feet below the ground surface, or 2 feet below bottom of proposed shallow foundations, whichever is deeper. Deeper over-excavation will be needed if soft, yielding soils or earth materials are exposed on the excavation bottoms. Over-excavation should extend at least 5 feet laterally beyond the limits of footings or as limited by the existing structures and improvements to remain in place. These recommendations also hold for the over excavation beneath the bottom of footings and membrane floor slab of the new tank.

Over-excavation and re-compaction for retaining walls, if any, should be two (2) feet below bottom of footings and should extend two (2) feet laterally beyond the retaining wall area. The upper 24-inches of site soils should be removed and re-compacted in areas of sidewalks and surface parking. The over-excavation should extend two (2) feet laterally beyond the sidewalk and surface parking areas. If loose, disturbed, or otherwise unsuitable materials are encountered at the bottom of excavation, deeper removal will be required until firm native soils are encountered.

The exposed bottom of the over-excavation area should be scarified at least 6.0 inches, moisture conditioned as needed to near-optimum moisture content and compacted to ninety percent (90%) relative compaction. The upper 12- inches of subgrade below new pavement should be compacted to 95 percent relative compaction. Over-excavation should not undermine adjacent improvements. Remedial grading should not extend within a projected 1:1 (horizontal to vertical) plane projected down from the outer edge of adjacent improvements. If loose, yielding soil conditions are encountered at the excavation bottom, the following options can be considered:

a. Over-excavate until a firm bottom is reached.
b. Scarify or over-excavate an additional 18 inches deep, and then place at least 18-inch-thick layer of compacted base material (CAB or equivalent) to bridge the soft bottom. Base materials should be compacted to 95% relative compaction.
c. Over-excavate an additional 18 inches deep, and then place a layer of geotextile reinforcement (i.e. Mirafi HP570, or equivalent), then place an 18-inch-thick layer of compacted base material (CAB or equivalent) to bridge the soft bottom. Base material should be compacted to 95% relative compaction. An additional layer of geotextile reinforcement may be needed on top of base material depending on the actual site conditions.

Excavation activities should not disturb adjacent utilities or undermine any adjacent structures to remain. Existing utilities should be removed and adequately capped at the project boundary line or salvaged/rerouted as designed.

The actual depth of removal should be based on recommendations and observation made during grading. Therefore, some variations in the depth and lateral extent of over-excavation recommended in this report should be anticipated.

7.3 Engineered Fill

Following observation of the excavation bottom, subgrade soil surfaces should be scarified to a depth of at least 6 inches. The scarified soil should be moisture-conditioned to within three percent (3%) of optimum moisture for granular soils and to approximate three percent (3%) above the optimum moisture for fine-grained soils. Scarified soils shall be compacted to a minimum ninety percent (90%) of the laboratory maximum dry density as determined by the ASTM Standard D1557 test method.

Any import fill should be tested and approved by Project Geotechnical Consultant. The import fill should have an expansion potential less than 20. The imported materials should be thoroughly mixed and moisture conditioned within three percent (3%) above the optimum moisture. All fill, if not specified otherwise elsewhere in this report, should be compacted to at least 90 percent of the laboratory dry density in accordance with the ASTM Standard D2922 test method.

Where the fill is not within the areas specified above or is not to support any structures, excavated site soils, free of deleterious materials and rock particles larger than 3.0 inches in the largest dimension, should be suitable for placement as compacted fill. The site materials should be thoroughly mixed, and moisture conditioned to approximately three percent (3%) above the optimum moisture, and then compacted to at least ninety percent (90%) of relative compaction.

7.4 Foundation Bottom Subdrains

In the event of leaks in the storage tank and water accumulation near the foundation bottoms, installation of subdrains is recommended to prevent potential hydrostatic uplift. Subdrain systems should be constructed of a minimum 4-inch diameter, rigid ABS (SDR-35) or Schedule 40 PVC pipe with glued manufactured pipe fitting and caps. The drainpipe should be sloped at a minimum 2% gradient to provide gravity flow to the approved outlet location with sump pump lift station or gravity outflow. Perforated pipes shall be laid with perforations down. Schedule 40 PVC perforated pipe may have to be custom fabricated.

Surface drain systems should not be connected to the subdrain system. Introduction of surface water in the subdrain system could be recharge water into the compacted fill soils. Surface and subsurface drainage systems should be kept separate. Appropriate backflow preventers should be installed to prevent accidental discharge.

A State of California Department of Transportation (Caltrans) Class 2 Permeable Material is recommended for the permeable drain material. The percentage composition by weight of permeable material in place shall conform to the following gradations:

Table No. 9, Caltrans Class 2 Permeable Material

Sieve Size	Percentage Passing
1”	100
3/4”	90 - 100
3/8”	40 - 100
No. 4	25 – 40
No.8	18 – 33
No. 30	5 – 15
No. 50	0 – 7
No. 200	0 – 3

Note: Class 2 permeable material shall have a Sand Equivalent value of not less than 75.

7.5 Excavatability and Rippability

Based on our field exploration, most of the earth materials at the site should be excavatable and rippable with conventional heavy-duty earth moving equipment in good working condition. However, areas of harder, cemented layers are anticipated to be encountered during excavation and grading. These areas may require the use of larger heavy-duty dozers, excavators, track-mounted hydraulic breakers and/or single shank rippers to loosen, rip, cross-rip, downsize, crush, breakdown, mix and process the excavated materials into soil size materials suitable for use as structural fill. Every effort shall be made during excavation, transport and grading to reduce the size of the materials to particle sizes less than three (3) inches in size to be adequately placed as structural fill.

7.6 Pipeline Backfill Recommendations

Any soft and/or unsuitable material encountered at the pipe invert should be removed and replaced with an adequate bedding material. The pipe subgrade should be level, firm, uniform and free of loose materials and properly graded to provide uniform bearing and support to the entire section of the pipe placed on bedding material. Oversize particles larger than 2.0 inches the largest dimension, if any, should be removed from the trench bottom and replaced with compacted materials. During the digging of depressions for proper sealing of the pipe joints, the pipe should rest on a prepared bottom for as near its full length as is practicable. The bedding zone is defined as that portion of the pipe trench from 4.0 inches below the pipe invert to one foot above the top of the pipe, in accordance with section 306-1.2.1 of the latest edition of the Standard Specifications for public Works Construction (SSPWC)

7.7 Trench Zone Backfill

The trench zone is defined as the portion of the trench above the pipe bedding extending up to the final grade level of the trench surface.

The following specifications are recommended to provide a basis for quality control during the placement of trench backfill.

Trench excavations to receive backfill shall be free of trash, debris or other unsatisfactory materials at the time of backfill placement. Excavated on-site soils free of oversize particles, defined as larger than 1.0 inch in maximum dimension in the upper 12.0 inches of subgrade soils and larger than 3.0 inches in the largest dimension in the trench backfill below, and deleterious matter after proper processing may be used to backfill the trench zone. Imported trench backfill, if used, should be approved by the project soils consultant prior to delivery at the site. No more than thirty percent (30%) of the backfill volume should be larger than 3/4 inches in the largest dimension.

Trench backfill shall be compacted to ninety percent (90%) of the laboratory maximum dry density as per ASTM Standard D1557 test method. At least the upper 12.0 inches of trench underlying pavements should be compacted to at least ninety-five percent (95%) of the laboratory maximum dry density.

Trench backfill shall be compacted by mechanical methods, such as sheepsfoot, vibrating or pneumatic rollers, or mechanical tampers, to achieve the density specified herein. The backfill materials shall be brought to within three percent (3%) of optimum moisture content and then placed in horizontal layers if the expansion index is less than or equal to 30. Should the expansion index be greater than 30, backfill materials shall be brought to approximately three percent (3%) above optimum moisture content. The thickness of uncompacted layers should not exceed 8.0 inches. Each layer shall be evenly spread, moistened or dried as necessary, and then tamped or rolled until the specified density has been achieved.

The contractor shall select the equipment and processes to be used to achieve the specified density without damage to adjacent ground and completed work. The field density of the compacted soil shall be measured by the ASTM Standard D1556 or ASTM Standard D6938 test methods or equivalent. Observation and field tests should be performed by Converse during construction to confirm that the required degree of compaction has been obtained. Where compaction is less than that specified, additional compactive effort shall be made with adjustment of the moisture content as necessary, until the specified compaction is obtained. It should be the responsibility of the contractor to maintain safe conditions during cut and/or fill operations. Trench backfill shall not be placed, spread or rolled during unfavorable weather conditions. When the work is interrupted by heavy rain, fill operations shall not be resumed until field tests by the project's geotechnical consultant indicate that the moisture content and density of the fill are as previously specified.

7.7.1 Selected Fill Materials for Trench Zone Backfill

Imported soils, if any, used as compacted trench backfill should be predominantly granular and meet the following criteria:

- Expansion Index less than 20
- Free of all deleterious materials
- Contain no particles larger than 3.0 inches in the largest dimension
- Contain less than thirty percent (30%) by weight retained on 3/4-inch sieve
- Contain at least fifteen percent (15%) fines (passing #200 sieve)
- Have a Plasticity Index of 10 or less

Any import fill should be tested and approved by the geotechnical representative prior to delivery to the site.

7.8 Expansive Soil Mitigation

The on-site soil materials will be mixed during the grading and the expansion potential might change. Therefore, the expansion potential of site soils should be verified after the grading for slabs, foundations and pavements. If the expansion potential of mixed soil is found to have an Expansion Index (EI) above 20, Converse recommends mixing on-site soil used for support of foundations, walkways, and pavements with 4 percent Lime to reduce expansion potential.

Any proposed import fill should have an Expansion Index (EI) less than 20 and should be evaluated and approved by Converse prior to import to the site.

7.9 Shrinkage and Subsidence

Soil shrinkage and/or bulking as a result of remedial grading depends on several factors including the depth of over-excavation, and the grading method and equipment utilized, and average relative compaction. For preliminary estimation, bulking and shrinkage factors for various units of earth material at the site may be taken as presented below:

- The approximate shrinkage factor for the native alluvial soils is estimated to range from five to fifteen percent (5-15%).
- For estimation purposes, ground subsidence may be taken as 0.15 feet as a result of remedial grading.

Although these values are only approximate, they represent our best estimates of the factors to be used to calculate lost volume that may occur during grading. If more accurate shrinkage and subsidence factors are needed, it is recommended that field-testing using the actual equipment and grading techniques be conducted.

7.10 Subgrade Preparation

Final subgrade soils for structures should be uniform and non-yielding. To obtain a uniform subgrade, soils should be well mixed and uniformly compacted. The subgrade soils should be non-expansive and well-drained. The near-surface site soils should be free draining. We recommend that at least the upper 4.0 inches of subgrade soils underneath the tank should be comprised of well-drained granular soils such as sands, gravel or crushed aggregate satisfying the following criteria:

- Maximum size ≤ 1.5 inches
- Percent passing U.S. #200 sieve ≤ twelve percent (12%)
- Sand equivalent ≥ 30
- The subgrade soils should be moisture conditioned before placing concrete.

8.0 DESIGN RECOMMENDATIONS

8.1 General Evaluation

Based on the results of our background review, subsurface exploration, laboratory testing, geotechnical analyses, and understanding of the planned site development, it is our opinion that the proposed project is feasible from a geotechnical standpoint, provided the following conclusions and recommendations are incorporated into the project plans, specifications, and are followed during site construction. The proposed water tank and site improvements may be supported by shallow continuous ring and isolated square footings.

8.2 Shallow Foundations

8.2.1 Vertical Capacity

Continuous ring and square footings should be founded at least 24 inches below lowest adjacent final grade on the recommended earth materials. A minimum footing width of 24 inches is recommended for continuous and square footings. The net allowable dead plus live load bearing value for isolated square and continuous footings is 3,500 psf. The net allowable bearing pressure can be increased by 400 psf for each additional foot of excavation depth and width up to a maximum value of 5,000 psf.

The net allowable bearing values indicated above are for the dead loads and frequently applied live loads and are obtained by applying a factor of safety of 3.0 to the net ultimate bearing capacity.

8.2.2 Lateral Capacity

Resistance to lateral loads can be provided by friction acting at the base of the foundation and by passive earth pressure. A coefficient of friction of 0.35 may be assumed with normal dead load forces. An allowable passive earth pressure of 250 psf per foot of depth up to a maximum of 2,500 psf may be used for footings poured against properly compacted fill. The values of coefficient of friction and allowable passive earth pressure include a factor of safety of 1.5.

8.2.3 Settlement

The static settlement of reservoir supported on continuous and/or spread footings founded on compacted fill and native soil will depend on the actual footing dimensions and the imposed vertical loads. Most of the footing settlement at the project site is expected to occur immediately after the application of the load. Based on the maximum allowable net bearing pressures presented above, static settlement is anticipated to be less than 1.0 inch. Differential settlement is expected to be up to one-half (1/2) of the total settlement over a 30-foot span.

8.2.4 Dynamic Increases

Bearing values indicated above are for total dead load and frequently applied live loads. The above vertical bearing may be increased by thirty-three percent (33%) for short durations of loading which will include the effect of wind or seismic forces. The allowable passive pressure may be increased by thirty-three percent (33%) for lateral loading due to wind or seismic forces.

8.3 Modulus of Subgrade Reaction

For the subject project, design of the reservoir supported on compacted fill subgrade prepared in accordance with the recommendations provided in this report may be based on a soil modulus of subgrade reaction of (k_s) of 125 pounds per square inch per inch.

8.4 Lateral Earth Pressure

The following provisional design values may be used for any utility vaults and/or walls below grade that are less than 6 feet high.

The earth pressure behind any buried wall depends primarily on the allowable wall movement, type of soil behind the wall, backfill slopes, wall inclination, surcharges, and any hydrostatic pressure. The following earth pressures are recommended for vertical walls with no hydrostatic pressure.

Table No. 10, Lateral Earth Pressures for Retaining Wall Design

Backfill Slope (H:V)	Cantilever Wall Equivalent Fluid Pressure (psf)	Restrained Wall Equivalent Fluid Pressure (psf)
Level	40 (triangular pressure distribution)	60 (triangular pressure distribution)

The recommended lateral pressures assume that the walls are fully back-drained to prevent build-up of hydrostatic pressure. Adequate drainage could be provided by means of permeable drainage materials wrapped in filter fabric installed behind the walls. The drainage system should consist of perforated pipe surrounded by a minimum one square foot per lineal feet of free draining, uniformly graded, permeable material aggregate, and wrapped in filter fabric such as Mirafi 140N or equivalent. The filter fabric should overlap approximately 12.0 inches or more at the joints. The subdrain pipe should consist of perforated, 4-inch diameter, rigid ABS (SDR-35) or Schedule 40 PVC , or equivalent, with perforations placed down. Alternatively, a prefabricated drainage composite system such as the Miradrain G100N or equivalent can be used. The subdrain should be connected to solid pipe outlets, with a maximum outlet spacing of 100 feet. Waterproofing membranes should be added to the subterranean wall levels for moisture sensitive areas to mitigate moisture migration through the walls.

In addition, walls with inclined backfill should be designed for an additional equivalent fluid pressure of one pound per cubic foot for every 2 degrees of slope inclination. Walls subjected to surcharge loads located within a distance equal to the height of the wall should be designed for an additional uniform lateral pressure equal to 1/3 or 1/2 the anticipated surcharge load for unrestrained or restrained walls, respectively. These values are applicable for backfill placed between the wall stem and an imaginary plane rising 45 degrees from below the edge (heel) of the wall footings.

Retaining walls taller than 6 feet should be designed to resist additional earth pressure caused by seismic ground shaking based on Section 1615A.1.6 of CBC 2019. A seismic earth pressure of 20H (psf), based on an inverted triangular distribution, can be used for design of wall. See Drawing No. 9, *Lateral Earth Pressure for Retaining Walls*.

8.5 Slabs-on-Grade

Slabs-on-grade should have a minimum thickness of five (5) inches nominal for support of normal ground-floor live loads. Minimum reinforcement for slabs-on-grade should be No. 4 reinforcing bars, spaced at 18 inches on-center each way. The thickness and reinforcement of more heavily loaded slabs will be dependent upon the anticipated loads and should be designed by a structural engineer. A static modulus of subgrade reaction equal to 125 pounds per square inch per inch may be used in structural design of concrete slabs-on-grade.

It is critical that the exposed subgrade soils should not be allowed to desiccate prior to the slab pour. Care should be taken during concrete placement to avoid slab curling. Slabs should be designed and constructed as promulgated by the ACI and Portland Cement Association (PCA). Prior to the slab pour, all utility trenches should be properly backfilled and compacted.

In areas where a moisture-sensitive floor covering (such as vinyl tile or carpet) is used, a 15-mil-thick moisture retarder/barrier can be used between the bottom of slab and subgrade that meets the performance criteria of ASTM E1745 Class A material. Retarder/barrier sheets should be overlapped a minimum of six inches and should be taped or otherwise sealed per the product specifications.

8.6 Cast-In-Drilled-Hole Pile Foundations for Non-building Structures

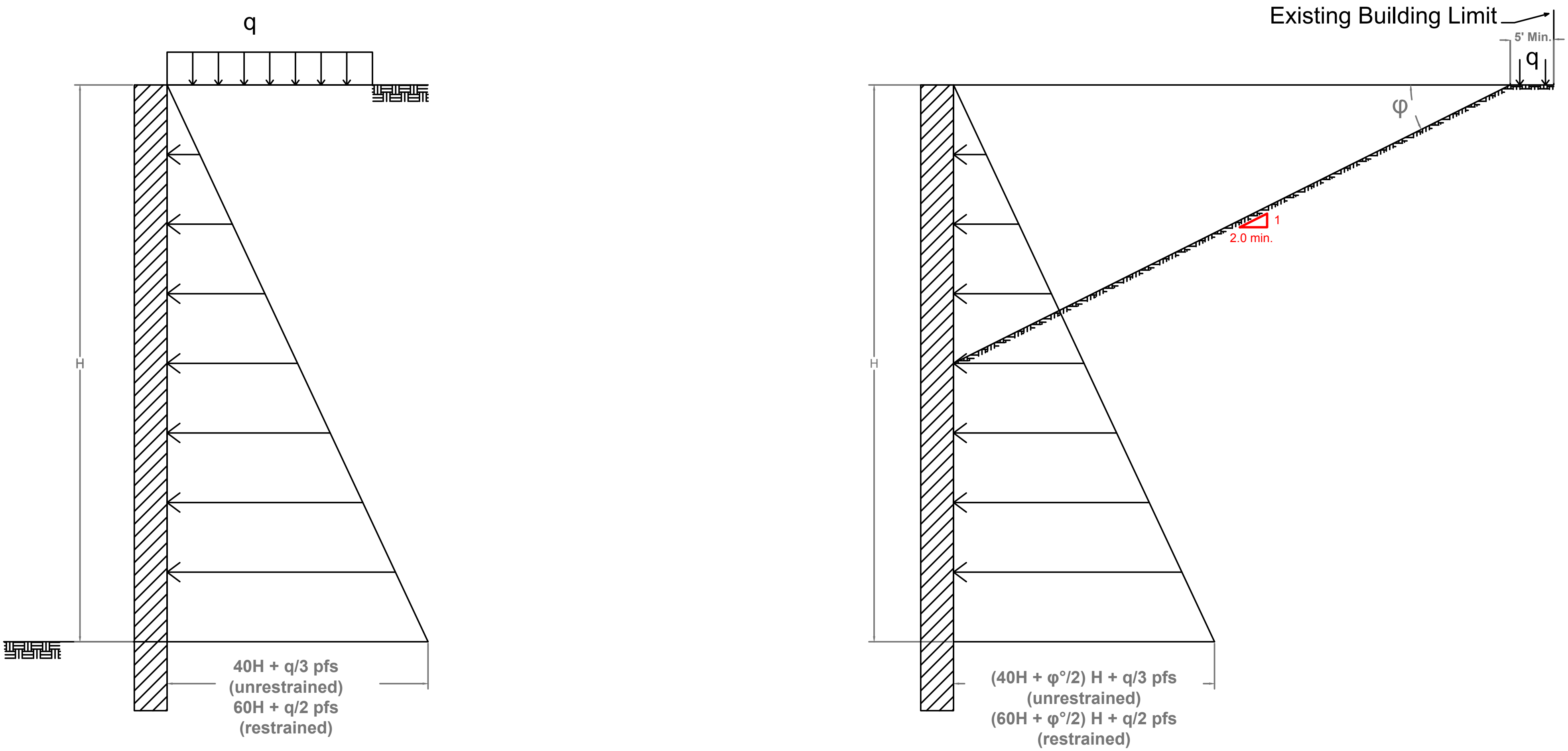
The planned non-building structures (e.g. lighting for parking lot, walkway, and court, fence walls, signs, etc.) may be supported on a Cast-In-Drilled-Hole (CIDH) pile foundation provided the following recommendations are incorporated into design and construction.

8.6.1 Vertical Capacity

CIDH piles should be at least 18-inches in diameter and can be designed for an allowable skin friction of 200 psf against the perimeter of pile. The diameter and length of CIDH pile shall be determined by the structural engineer based on design loads. The uplift capacities can be taken as one-half of compressive capacities for pile design.

8.6.2 Lateral Capacity

Resistance to lateral loads can be provided by friction acting at the base of the foundation and by passive earth pressure. A coefficient of friction of 0.30 may be assumed with normal dead load forces. An allowable passive earth pressure of 200 psf per foot of depth up to a maximum of 2,000 psf may be used for foundations poured against compacted



Notes:
1. Assume No Hydrostatic Pressure
2. H- Piles should be spaced a maximum of eight feet on center.
3. Excavation sheeting may be retained by soil anchor.
4. Care should be exercised in placement of logging to provide contact between the logging and the retained soil.
5. For H>6 feet, additional earth pressure caused by seismic ground shaking equal to 20H(psf) should be used.

LATERAL EARTH PRESSURE FOR RETAINING WALLS

Converse Consultants

Upper Reservoir Replacement Project
Geotechnical Investigation
13655 Foothill Boulevard
Sylmar, CA 91342

Project No. 19-31-168-01

Drawing No. 9

fill. The values of coefficient of friction and allowable passive earth pressure include a factor of safety of 1.5.

For ground surface restrained by concrete slab, the passive resistance may be calculated from the ground surface. For unrestrained ground condition, the passive resistance of the upper one (1) foot of earth material should be neglected in design.

8.6.3 Settlement

Based on the maximum allowable net vertical capacity presented above, static settlement is anticipated to be less than 0.5 inch.

8.7 Soil Corrosivity Evaluation

Converse retained Environmental Geotechnology Laboratory, Inc., located in Arcadia, California, to test one sample taken in the general area of the proposed reservoir. The tests included minimum resistivity, pH, soluble sulfates, and chloride content, with the results summarized on the following table:

Table No. 11, Soil Corrosivity Test Results

Boring No.	Sample Depth (feet)	pH (Caltrans 643)	Soluble Chlorides (Caltrans 422) ppm	Soluble Sulfate (Caltrans 417) % by Weight	Saturated Resistivity (Caltrans 532) Ohm-cm
BH-1	0-5	7.86	105	0.005	5,800

In accordance with the Caltrans Corrosive Guidelines (2012), the pH, soluble sulfate, and chloride content values of the sample tested are in the “non-corrosive” range. The minimum saturated resistivity is not in the corrosive range to ferrous metal. Mitigation measures to protect ferrous metal pipes in contact with the soils may be anticipated.

The test results presented herein are considered preliminary. If advanced corrosivity study is desired by the design team, a corrosion engineer can be consulted for appropriate mitigation procedures and construction design. In general, conventional corrosion mitigation measures may include the following:

- Steel and wire concrete reinforcement should have at least three inches of concrete cover where cast against soil, unformed.
- Below-grade ferrous metals should be given a high-quality protective coating, such as 18-mil plastic tape, extruded polyethylene, coal-tar enamel, or Portland cement mortar.
- Below-grade metals should be electrically insulated (isolated) from above-grade metals by means of dielectric fittings in ferrous utilities and/or exposed metal structures breaking grade.

8.8 Site Drainage

Adequate positive drainage should be provided away from the reservoir to prevent ponding and to reduce percolation of water into structural backfill. We recommend that the landscape area immediately adjacent to the foundation shall be designed sloped away from the building with a minimum five percent (5%) slope gradient for at least 10 feet measured perpendicular to the face of the wall. Impervious surfaces within 10 feet of the foundation shall have a minimum two percent (2%) slope away from the building per 2016 CBC.

Planters and landscaped areas adjacent to the building perimeter should be designed to minimize water infiltration into the subgrade soils. Gutters and downspouts should be installed on the roof, and runoff should be directed to the storm drain through non-erosive devices. Lower level areas may require special drainage provisions and sump pumps to provide suitable drainage.

9.0 CONSTRUCTION CONSIDERATIONS

9.1 General

Site soils should be excavatable using conventional heavy-duty excavating equipment. Temporary sloped excavation is feasible if performed in accordance with the slope ratios provided in Section 9.2, *Temporary Excavations*. Existing utilities should be accurately located and either protected or removed as required. For steeper temporary construction slopes or deeper excavations, shoring should be provided by the contractor as necessary, to protect the workers in the excavation.

9.2 Temporary Excavations

Based on the sandy materials encountered in the exploratory borings, sloped temporary excavations (if necessary) may be constructed according to the slope ratios presented in Table No. 12, *Slope Ratios for Temporary Excavations*. Any loose utility trench backfill or other fill encountered in excavations will be less stable than the native soils. Temporary cuts encountering loose fill or loose dry sand may have to be constructed at a flatter gradient than presented in the following table:

Table No. 12, Slope Ratios for Temporary Excavations

Maximum Depth of Cut (feet)	Maximum Slope Ratio* (horizontal: vertical)
0 – 4	vertical
4 – 8	1:1
8+	1.5:1

*Slope ratio assumed to be uniform from top to toe of slope.

Surfaces exposed in slope excavations should be kept moist but not saturated to minimize raveling and sloughing during construction. Adequate provisions should be made to protect the slopes from erosion during periods of rainfall. Surcharge loads, including construction, should not be placed within 5 feet of the unsupported trench edge. The above maximum slopes are based on a maximum height of 6 feet of stockpiled soils placed at least 5 feet from the trench edge.

All applicable requirements of the California Construction and General Industry Safety Orders, the Occupational Safety and Health Act of 1987 and current amendments, and the Construction Safety Act should be met. The soils exposed in cuts should be observed during excavation by the project's geotechnical consultant. If potentially unstable soil conditions are encountered, modifications of slope ratios for temporary cuts may be required.

If the excavation occurs near existing structures, special construction considerations would be required during excavation to protect these existing structures during construction. The proposed excavation should not cause loss of bearing and/or lateral supports of the existing structures.

9.3 Shoring Design

Temporary shoring will be required for the recommended excavation due to space limitations and property line boundaries and because of nearby existing structures or facilities and traffic loading. Temporary shoring may consist of the use of conventional soldier piles and lagging. Shoring should ultimately be designed by a qualified structural engineer considering the recommendations below in their final design and others which are applicable. Existing structures adjacent to excavation should be monitored for distress or excessive vibration during excavation.

Drilled excavations for soldier piles, which are recommended to create the proposed 25-foot-high excavation, may require the use of drilling fluids or temporary casing to prevent caving and to maintain an opened hole for pile installation.

9.3.1 Cantilevered Shoring

Cantilevered shoring systems may include soldier piles with lagging to maintain temporary support of vertical wall excavations. Shoring design must consider the support of adjacent underground utilities and/or structures and should consider the effects of shoring deflection on supported improvements. Due to sandy nature of on-site soils, some caving during the drilling of soldier-pile borings should be anticipated. A soldier pile system will require continuous lagging to control caving and sloughing in the excavation between soldier piles.

Temporary cantilevered shoring should be designed to resist a lateral earth pressure equivalent to a fluid density of 40 pounds per cubic foot (pcf) for non-surcharged condition. This pressure is valid only for shoring retaining level ground.

In addition to the lateral earth pressure, surcharge pressures due to miscellaneous loads, such as soil stockpiles, existing structures, vehicular traffic or construction equipment located adjacent to the shoring, should be included in the design of the shoring. A uniform lateral pressure of 100 psf should be included in the upper 10 feet of the shoring to account for normal vehicular and construction traffic within 10 feet of the excavation. Surcharge pressures from the existing structures should be added to the above earth pressures for surcharges within a horizontal distance less than or equal to the wall height. Surcharge coefficients of 50% of any uniform vertical surcharge should be added as a horizontal earth pressure for shoring design. All shoring should be designed and installed in accordance with state and federal safety regulations.

The minimum embedment depth for piles is ten (10) feet from the lowest adjacent grade into firm alluvium, below the bottom of the excavation. Vertical skin friction against soldier piles may be taken as 250 psf. Fixity may be assumed at two (2) feet below the excavation into firm native alluvium or bedrock. For the design of soldier piles spaced at least 3.0 diameters on-center, the passive resistance of the soils adjacent to the piles may be assumed to be 250 psf per foot of embedment depth. Soldier pile members placed in drilled holes should be properly backfilled with a sand/cement slurry or lean concrete in order to develop the required passive resistance.

Caving soils should be anticipated between the piles. To limit local sloughing, caving soils can be supported by continuous lagging or guniting. The lagging between the soldier piles may consist of pressure-treated wood members or solid steel sheets. In our opinion, steel sheeting is expected to be more expedient than wood lagging to install. Although soldier piles and any bracing used should be designed for the full-anticipated earth pressures and surcharge pressures, the pressures on the lagging are less because of the effect of arching between the soldier piles. Accordingly, the lagging between the piles may be designed for a nominal pressure of up to a maximum of 400 psf. All lumber to be left in the ground should be treated in accordance with Section 204-2 of the "Standard Specifications for Public Works Construction" (Latest Edition).

9.3.2 Tie-Back Shoring

A tie-back soldier-pile shoring system may be used to maintain temporary support of deep vertical walled excavations. Braced or tied-back shoring, retaining a level ground surface, should be designed for a uniform pressure of 20H psf, where H is the height of the retained cut in feet.

Surcharge pressures should be added to this earth pressure for surcharges within a distance from the top of the shoring less than or equal to the shoring height. A surcharge coefficient of 50 percent of any uniform vertical surcharge should be added as a horizontal shoring pressure for braced shoring. A uniform lateral pressure of 100 psf should be included in the upper 10 feet of the shoring to account for normal vehicular and construction traffic within 10 feet of the trench excavation.

9.3.3 Tie-Backs

For design of tie-back shoring, it should be assumed that the potential wedge of failure is determined by a plane at 30 degrees from the vertical, through the bottom of the excavation. Tie-back anchors may be installed at angles of 15 to 40 degrees below a horizontal plane. Soil friction values, for estimating the allowable capacity of drilled friction anchors, may be computed using the following equation:

$q = 40H$; $q \leq$ *500 pounds-per-square-foot (psf)*

where:

H = average depth of anchor below ground surface, shown on Drawing No. 10, *Schematic Typical Tie-Back Design*
q = anchor surface area resistance, in psf (excluding tip),

Only the frictional resistance developed beyond the assumed failure plane should be included in the tie-back design for resisting lateral loads. After shoring/tie-back is no longer needed to support the excavation, stress should be carefully released and shoring system including tieback may be able to be left in place.

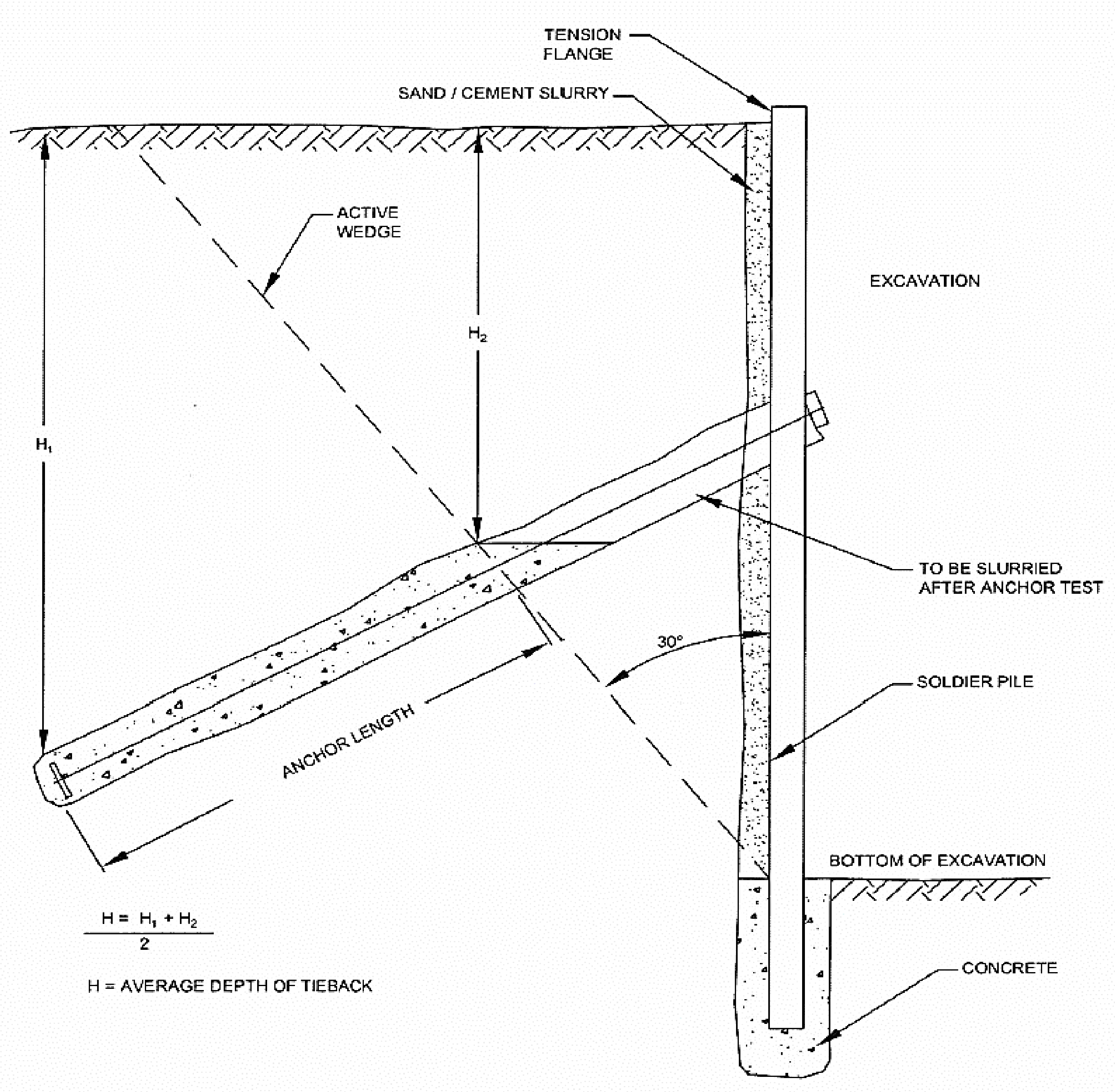
All shoring and tie-back should be designed by experienced California licensed Civil Engineer and installed by experienced contractors. Shoring/tie-back design should also be reviewed by a geotechnical consultant to verify the soil parameters used in the design are in conformance with geotechnical report.

All applicable requirements of the California Construction and General Industry Safety Orders, the Occupational Safety and Health Act of 1987 and current amendments, and the Construction Safety Act should be met. The soils exposed in cuts should be observed during excavation by a competent person employed by the contractor. If potentially unstable soil conditions are encountered, modifications of slope ratios for temporary cuts may be required.

It is recommended that Converse review plans and specifications for proposed shoring and that a Converse representative observes the installation of shoring. A licensed surveyor should be retained to establish monuments on shoring and the surrounding ground prior to excavation. Such monuments should be monitored for horizontal and vertical movement during construction. Results of the monitoring program should be provided immediately to the project Structural (shoring) Engineer and Converse for review and evaluation. Adjacent building elements should be photo-documented prior to construction.

9.4 Geotechnical Services During Construction

This report has been prepared to aid in the site preparation and site grading plans and specifications, and to assist the architect, civil and structural engineers in the design of



SCHEMATIC TYPICAL TIE-BACK DESIGN

Upper Reservoir Replacement Project

Sylmar

For : City of Sylmar/ / John Robinson Consulting, Inc.

Project Number:

19-31-168-01

Converse Consultants

Drawing No.

10

the proposed reservoir. It is recommended that this office be provided an opportunity to review final design drawings and specifications to verify that the recommendations of this report have been properly implemented.

Recommendations presented herein are based upon the assumption that adequate earthwork monitoring will be provided by the geotechnical engineer of record. Excavation bottoms should be observed by a geotechnical engineer or his/her representative prior to the placement of compacted fill. Structural fill and backfill should be placed and compacted during continuous observation and testing. Footing excavations should be observed prior to placement of steel and concrete so that footings are founded on satisfactory materials and excavations are free of loose and disturbed materials.

During construction, the geotechnical engineer and/or their authorized representatives should be present at the site to provide a source of advice to the client regarding the geotechnical aspects of the project and to observe and test the earthwork performed. Their presence should not be construed as an acceptance of responsibility for the performance of the completed work, since it is the sole responsibility of the contractor performing the work to ensure that it complies with all applicable plans, specifications, ordinances, etc.

This firm does not practice or consult in the field of safety engineering. We do not direct the contractor's operations and cannot be responsible for other than our own personnel on the site; therefore, the safety of others is the responsibility of the contractor. The contractor should notify the owner if he considers any recommended actions presented herein to be unsafe.

10.0 CLOSURE

The findings and recommendations of this report were prepared in accordance with generally accepted professional engineering and engineering geologic principles and practice. We make no other warranty, either expressed or implied. Our conclusions and recommendations are based on the results of the field and laboratory investigations, combined with an interpolation and extrapolation of soil conditions between and beyond boring locations. If conditions encountered during construction appear to be different from those shown by the borings, this office should be notified.

Design recommendations given in this report are based on the assumption that the earthwork and site grading recommendations contained in this report are implemented. Additional consultation may be prudent to interpret Converse's findings for contractors, or to possibly refine these recommendations based upon the review of the final site grading and actual site conditions encountered during construction. If the scope of the project changes, if project completion is to be delayed, or if the report is to be used for another purpose, this office should be consulted.

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Appendix A

Field Exploration

APPENDIX A: FIELD EXPLORATION

Field exploration included a site reconnaissance and subsurface exploration program. During the site reconnaissance, the surface conditions were noted, and the approximate locations of the borings were determined and shared with design team. The exploratory borings were approximately located using existing boundary and other features as a guide and should be considered accurate only to the degree implied by the method used. The various field study methods performed are discussed below.

Exploratory Borings

Two (2) exploratory borings (BH-1 and BH-2) were drilled within the project site to evaluate the subsurface earth materials for the proposed project on April 3, 2020. The borings were drilled using truck mounted drill rig with an 8-inch diameter hollow stem auger to a maximum depth of 47 feet below the existing ground surface (bgs). Each boring was visually logged by a Converse engineer and sampled at regular intervals and at changes in subsurface soils, in accordance with the Unified Soil Classification System. Where appropriate, field descriptions and classifications have been modified to reflect laboratory test results.

Ring samples of the subsurface materials were obtained at frequent intervals in the exploratory borings using a drive sampler (2.4-inches inside diameter and 3.0-inches outside diameter) lined with sample rings. The steel ring sampler was driven into the bottom of the borehole with successive drops of a 140-pound driving weight falling 30 inches, using an automatic hammer. Samples are retained in brass rings (2.4-inches inside diameter and 1.0-inch in height). The central portion of the samples were retained and carefully sealed in waterproof plastic containers for shipment to the Converse laboratory. Blow counts for each sample interval are presented on the logs of borings. Bulk samples of typical soil types were also obtained.

Standard Penetration Test (SPT) was also performed using a standard split-barrel sampler (1.4-inches inside diameter and 2.0-inches outside diameter). The mechanically driven hammer for the SPT sampler was 140 pounds, falling 30 inches for each blow. The recorded blow counts for every six inches for a total of 1.5 feet of sampler penetration are shown on the Logs of Borings in the “BLOWS" column. The standard penetration test was performed in accordance with the ASTM Standard D1586 test method.

It should be noted that the exact depths at which material changes occur cannot always be established accurately. Changes in material conditions that occur between driven samples are indicated in the logs at the top of the next drive sample. A key to soil symbols and terms is presented as Drawing No. A-1, *Soil Classification Chart*. The log of the exploratory boring is presented in Drawing Nos. A-2 through A-3, *Log of Borings*.

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

BORING LOG SYMBOLS

SAMPLE TYPE

STANDARD PENETRATION TEST
Split barrel sampler in accordance with ASTM D-1586-84 Standard Test Method

DRIVE SAMPLE 2.42" I.D. sampler.

DRIVE SAMPLE No recovery

BULK SAMPLE

GROUNDWATER WHILE DRILLING

GROUNDWATER AFTER DRILLING

LABORATORY TESTING ABBREVIATIONS

TEST TYPE
(Results shown in Appendix B)

CLASSIFICATION

Plasticity	pi
Grain Size Analysis	ma
Passing No. 200 Sieve	wa
Sand Equivalent	se
Expansion Index	ei
Compaction Curve	max
Hydrometer	h

STRENGTH

Pocket Penetrometer	p
Direct Shear	ds
Direct Shear (single point)	ds*
Unconfined Compression	uc
Triaxial Compression	tx
Vane Shear	vs
Consolidation	c
Collapse Test	col
Resistance (R) Value	r
Chemical Analysis	ca
Electrical Resistivity	er

UNIFIED SOIL CLASSIFICATION AND KEY TO BORING LOG SYMBOLS

Converse Consultants

Project Name	Project No.	Figure No.
Upper Reservoir Replacement Project **Geotechnical Investigation** **13655 Foothill Boulevard** **Sylmar, CA 91342**	**19-31-168-01**	**A-1**

Log of Boring No. BH-1

Dates Drilled: 4/3/2020 Logged by: Babak Checked By: MBS

Equipment: 8" HOLLOW STEM AUGER Driving Weight and Drop: 140 lbs / 30 in

Ground Surface Elevation (ft): 1310 Depth to Water (ft): NOT ENCOUNTERED

Depth (ft)	Graphic Log	SUMMARY OF SUBSURFACE CONDITIONS This log is part of the report prepared by Converse for this project and should be read together with the report. This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	SAMPLES DRIVE	SAMPLES BULK	BLOWS	MOISTURE (%)	DRY UNIT WT. (pcf)	OTHER
0		**3-inch GRASS, LANDSCAPED AREA** **FILL (Af)** **SILTY SAND (SM):** fine to medium-grained, some gravel, dark brown.						ma,max, ca (fc=23.9%)
5					12/21/21	9	127	
10		**ALLUVIUM (Qal):** **POORLY-GRADED SAND WITH SILT (SP-SM):** fine to coarse-grained, some gravel, brown.			20/27/38	8	135	
15		some clay,			20/37/49	7	111	
20		**SILTY SAND (SM):** fine to medium-grained, some gravel, brown.			10/12/43			
25					50@6"	8	113	
30		**POORLY GRADED SAND (SP):** fine to medium-grained, some silt, yellow brown.			50@5"			

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Project Name
Upper Reservoir Replacement Project
Geotechnical Investigation
13655 Foothill Boulevard
Sylmar, CA 91342

Project No. **19-31-168-01**

Figure No. **A-2a**

Log of Boring No. BH-1

Dates Drilled: 4/3/2020 Logged by: Babak Checked By: MBS

Equipment: 8" HOLLOW STEM AUGER Driving Weight and Drop: 140 lbs / 30 in

Ground Surface Elevation (ft): 1310 Depth to Water (ft): NOT ENCOUNTERED

Depth (ft)	Graphic Log	SUMMARY OF SUBSURFACE CONDITIONS This log is part of the report prepared by Converse for this project and should be read together with the report. This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	SAMPLES DRIVE	SAMPLES BULK	BLOWS	MOISTURE (%)	DRY UNIT WT. (pcf)	OTHER
		ALLUVIUM (Qal): **POORLY GRADED SAND (SP):** fine to medium-grained, some gravel , brown			50@6"	6	118	
40		increased drilling resistance and difficulty			33/50@4"			
45		**SILTY SAND (SM):** fine to medium-grained, some gravel, brown.			15/50@5"	5	106	

Boring terminated at 47 feet below ground surface due to refusal on Gravels and Cobbles.
No groundwater was encountered.
Borehole was backfilled with cement grout on 4/3/2020.

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Project Name
Upper Reservoir Replacement Project
Geotechnical Investigation
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Sylmar, CA 91342

Project No. **19-31-168-01**

Figure No. **A-2b**

Log of Boring No. BH-2

Dates Drilled: 4/3/2020 | Logged by: Babak | Checked By: MBS

Equipment: 8" HOLLOW STEM AUGER | Driving Weight and Drop: 140 lbs / 30 in

Ground Surface Elevation (ft): 1310 | Depth to Water (ft): NOT ENCOUNTERED

SUMMARY OF SUBSURFACE CONDITIONS

This log is part of the report prepared by Converse for this project and should be read together with the report. This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.

Depth (ft)	Summary of Subsurface Conditions	Samples: Drive	Samples: Bulk	Blows	Moisture (%)	Dry Unit Wt. (pcf)	Other
0	**3' GRASS, LANDSCAPED AREA**		X (0–5)				
	FILL (Af): **CLAYEY SAND (SC):** fine to coarse-grained, some gravel and cobbles, dark brown.						
5		X		6/10/6	8	107	
8	**ALLUVIUM (Qal):** **SILTY SAND (SM):** fine to coarse-grained, brown						
10		X		15/20/40	8	123	
15	**POORLY-GRADED SAND (SP):** fine to coarse-grained, some gravel, brown.	X		27/32/37	7	124	
20		X		17/33/50@5"	5	108	ds
25		X		29/43/35			
26	**SILTY SAND (SM):** fine-grained, brown.						
30	**POORLY-GRADED SAND (SP):** fine to coarse-grained, some gravel.	X		32/50@5"	6	116	ds

End of boring at 31.5 feet below ground surface.
No groundwater was encountered.
Borehole backfilled with cement grout on 4/3/2020.

Converse Consultants

Project Name
Upper Reservoir Replacement Project
Geotechnical Investigation
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Sylmar, CA 91342

Project No. **19-31-168-01**

Figure No. **A-3**

Appendix B

Laboratory Testing Program

APPENDIX B: LABORATORY TESTING PROGRAM

Tests were conducted in our laboratory on representative soil samples for the purpose of classification and evaluation of their relevant physical characteristics and engineering properties. The amount and selection of tests were based on the geotechnical requirements of the project. Test results are presented herein and on the Logs of Borings in Appendix A, *Field Exploration*. The following is a summary of the laboratory tests conducted for this project.

Moisture Content and Dry Density

Results of moisture content and dry density tests performed on relatively undisturbed ring samples were used to aid in the classification of the soils and to provide quantitative measure of the *in-situ* dry density. Data obtained from this test provides qualitative information on strength and compressibility characteristics of site soils. For test results, see the Logs of Borings in Appendix A, *Field Exploration*.

Grain-Size Analysis

To assist in classification of soils, mechanical grain-size analysis was performed on one (1) selected sample. Testing was performed in general accordance with the ASTM Standard C136 test method. Grain-size curve is shown in Drawing No. B-1, *Grain Size Distribution Results*.

Maximum Dry Density Test

One (1) laboratory maximum dry density-moisture content relationship test was performed on a representative bulk sample of the upper 5 feet of soil material. The testing was conducted in accordance with ASTM Standard D1557 laboratory procedure. The test result is presented on Drawing No. B-2, *Moisture-Density Relationship Results*.

Direct Shear

Direct shear tests were performed on two (2) undisturbed soil samples. For the test, three samples contained in brass sampler rings were placed, one at a time, directly into the test apparatus and subjected to a range of normal loads appropriate for the anticipated conditions. The samples were then sheared at a constant strain rate of 0.025 inch/minute. Shear deformation was recorded until a maximum of about 0.50-inch shear displacement was achieved. Ultimate strength was selected from the shear-stress deformation data and plotted to determine the shear strength parameters. For test data, including sample density, see Drawing No. B-3a and B3b, *Direct Shear Test Results,* and the following table:

Table No. B-1, Direct Shear Test Results

Boring No.	Depth (feet)	Soil Classification	Peak Strength Parameters	
			Friction Angle (degrees)	Cohesion (psf)
BH-2	20	Poorly-Graded Sand (SP)	31	50
BH-2	30	Poorly-Graded Sand (SP)	36	50

Soil Corrosivity

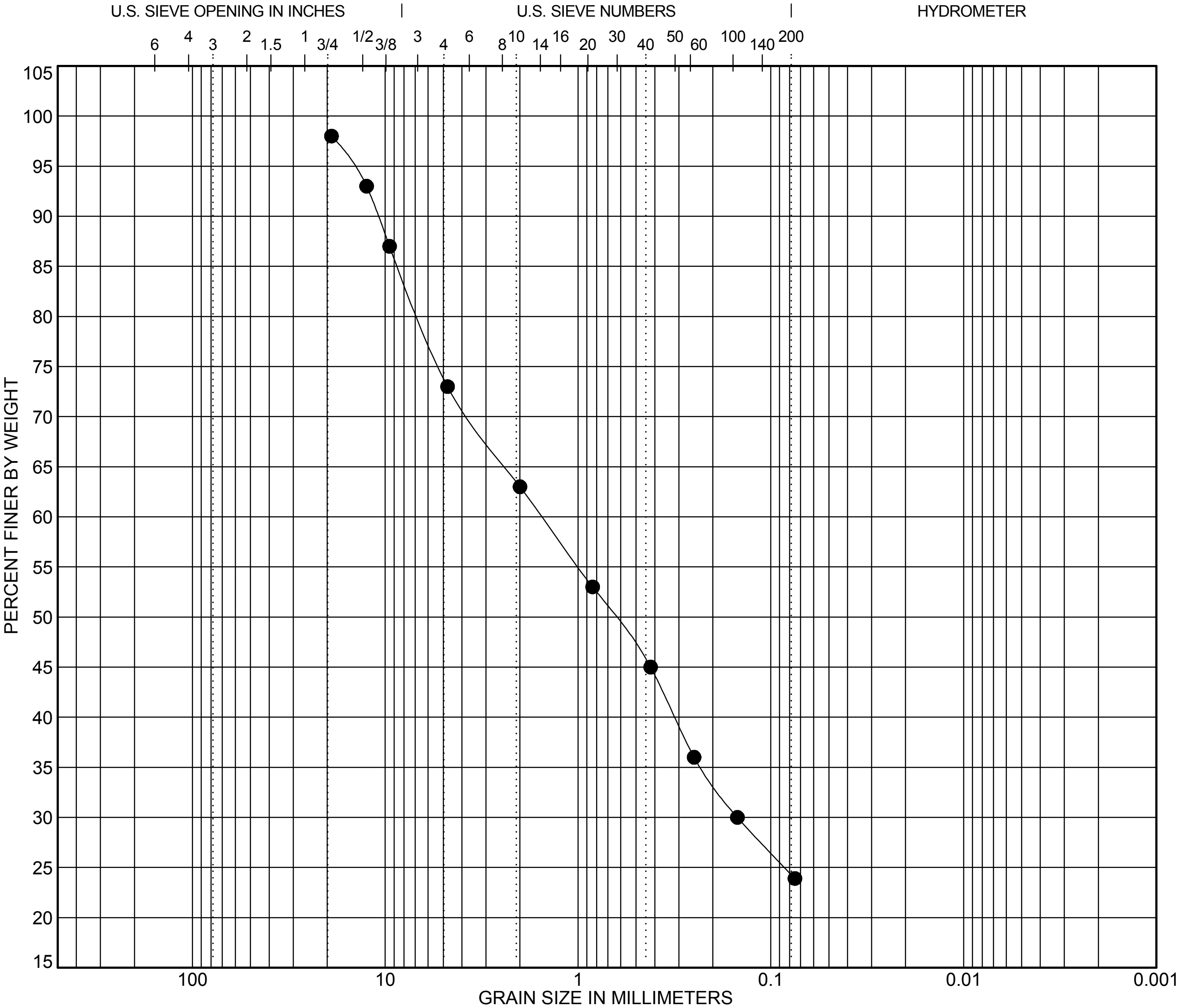
One (1) representative soil sample was tested to determine minimum electrical resistivity, pH, and chemical content, including chloride concentrations, and soluble sulfate. The purpose of these tests is to determine the corrosion potential of site soils when placed in contact with common construction materials. These tests were performed by EGL in Arcadia, California. The test results received from EGL are included in the following table:

Table No. B-2, Corrosivity Test Results

Boring No.	Sample Depth (feet)	pH (Caltrans 643)	Soluble Chlorides (Caltrans 422) ppm	Soluble Sulfate (Caltrans 417) (%)	Saturated Resistivity (Caltrans 643) Ohm-cm
BH-1	0-5	7.86	105	0.005	5,800

Sample Storage

Soil samples presently stored in our laboratory will be discarded 30 days after the date of this report, unless this office receives a specific request to retain the samples for a longer period of time.



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

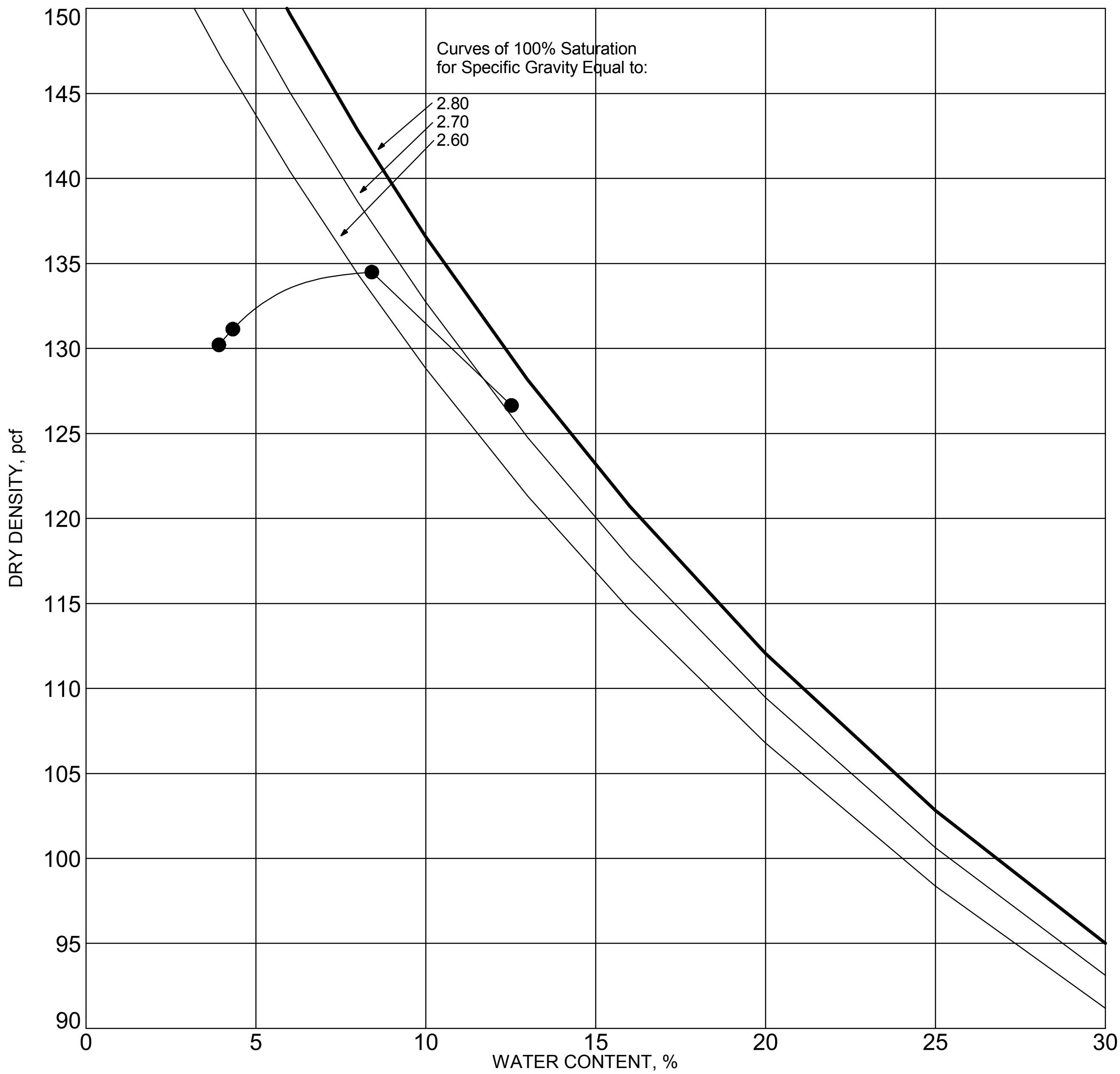
	Boring No.	Depth (ft)	Description	LL	PL	PI	Cc	Cu
●	**BH-1**	**5**	**Silty Sand (SM)**					

	Boring No.	Depth (ft)	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**BH-1**	**5**	**19**	**1.542**	**0.149**		**25.0**	**49.1**	**23.9**	

GRAIN SIZE DISTRIBUTION RESULTS

Converse Consultants

Project Name	Project No.	Drawing No.
Upper Reservoir Replacement Project **Geotechnical Investigation** **13655 Foothill Boulevard** **Sylmar, CA 91342**	**19-31-168-01**	**B-1**



SYMBOL	BORING NO.	DEPTH (ft)	DESCRIPTION	ASTM TEST METHOD	OPTIMUM WATER, %	MAXIMUM DRY DENSITY, pcf
●	BH-1	5	Silty Sand (SM)	D1557 Method B	7.2	135.8

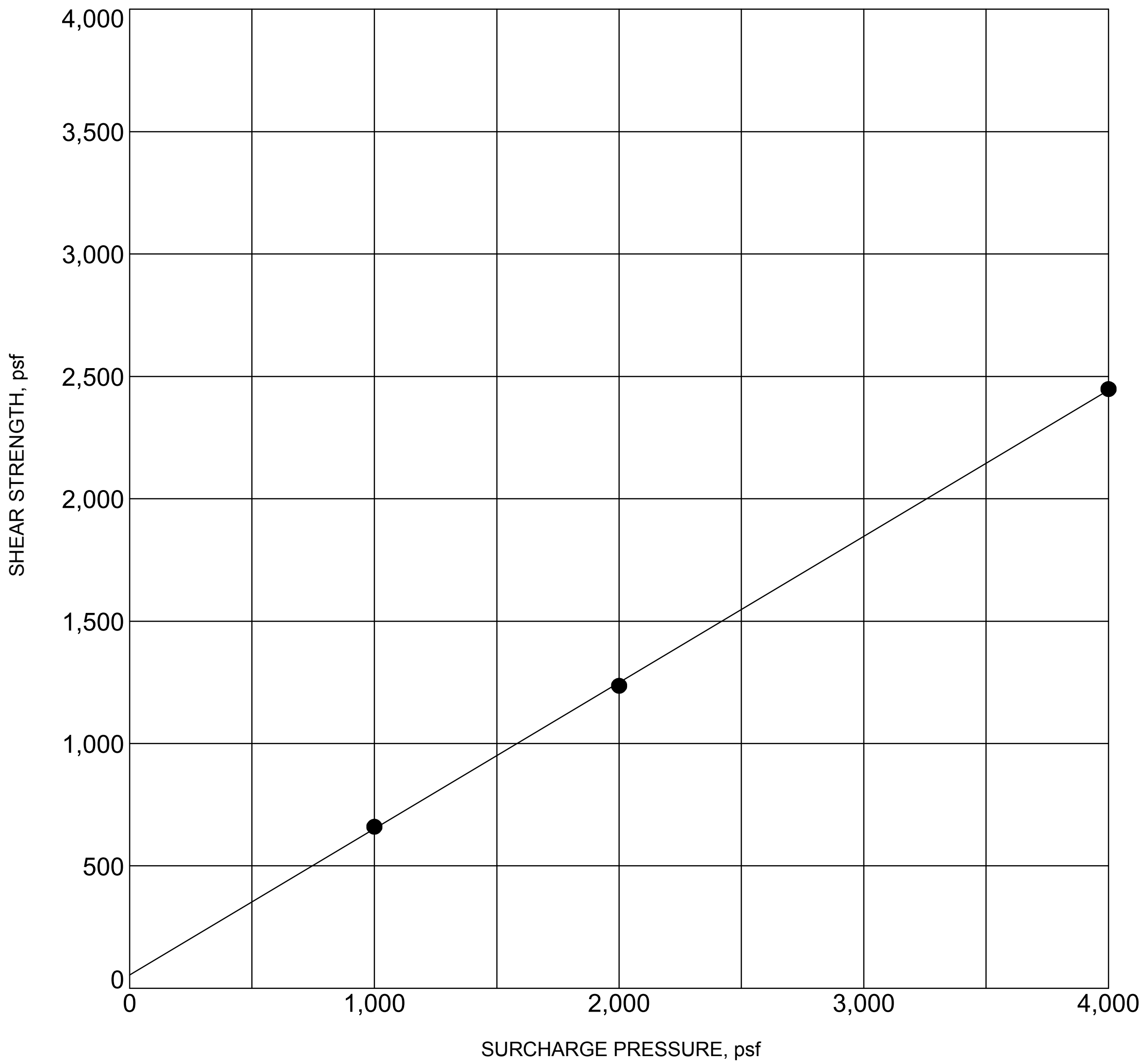
MOISTURE-DENSITY RELATIONSHIP RESULTS

Converse Consultants

Project Name
Upper Reservoir Replacement Project
Geotechnical Investigation
13655 Foothill Boulevard
Sylmar, CA 91342

Project No.
19-31-168-01

Drawing No.
B-2

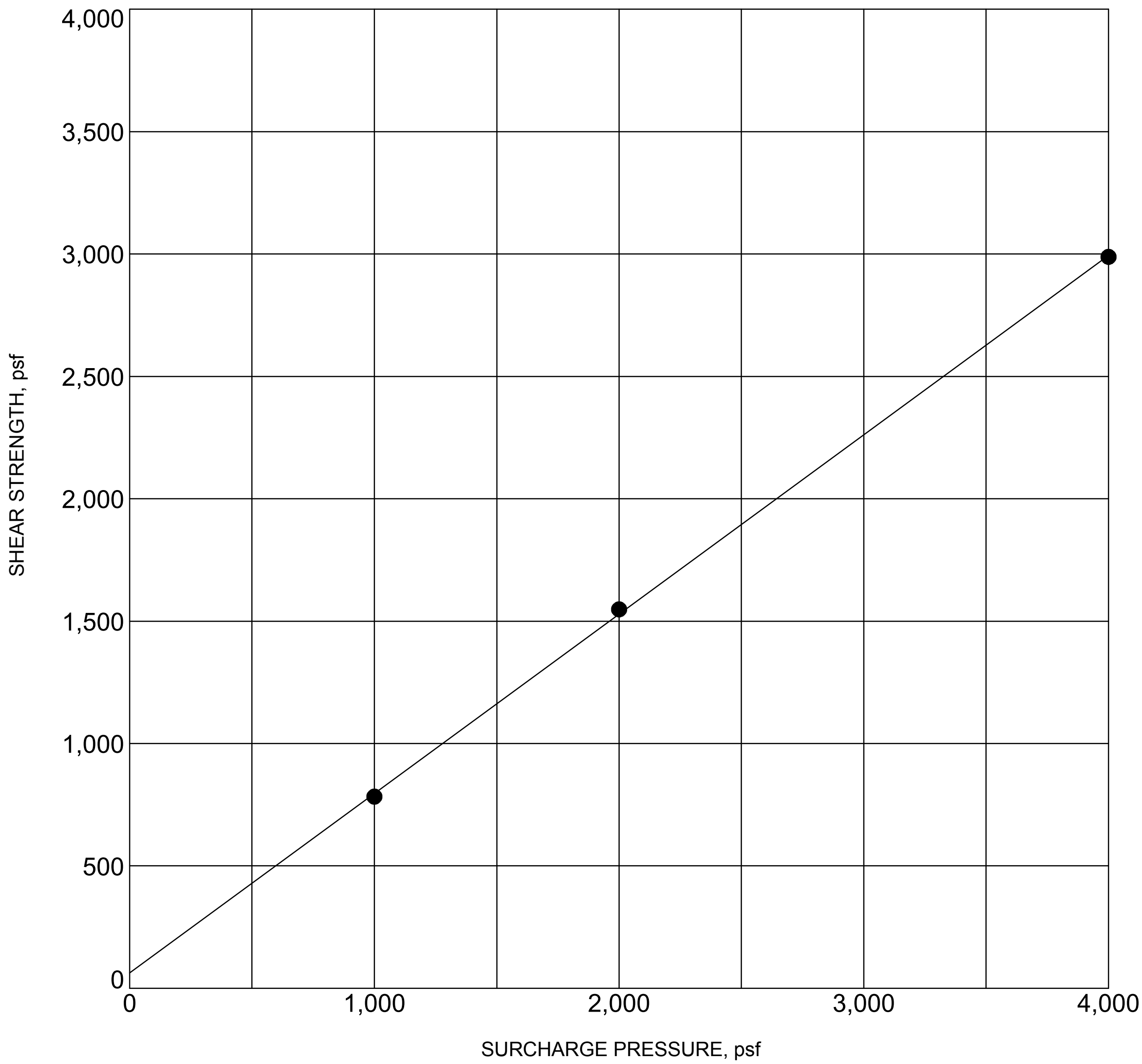


BORING NO. :	**BH-2**	DEPTH (ft) :	**20**
DESCRIPTION :	**Well-Graded Sand (SW)**		
COHESION (psf) :	**50**	FRICTION ANGLE (degrees):	**31**
MOISTURE CONTENT (%) :	**5**	DRY DENSITY (pcf) :	**108**

DIRECT SHEAR TEST RESULTS

Converse Consultants

Project Name	Project No.	Drawing No.
Upper Reservoir Replacement Project **Geotechnical Investigation** **13655 Foothill Boulevard** **Sylmar, CA 91342**	**19-31-168-01**	**B-3a**



BORING NO. :	**BH-2**	DEPTH (ft) :	**30**
DESCRIPTION :	**WELL-GRADED SAND (SW)**		
COHESION (psf) :	**50**	FRICTION ANGLE (degrees):	**36**
MOISTURE CONTENT (%) :	**6**	DRY DENSITY (pcf) :	**116**

DIRECT SHEAR TEST RESULTS

Converse Consultants

Project Name

Upper Reservoir Replacement Project
Geotechnical Investigation
13655 Foothill Boulevard
Sylmar, CA 91342

Project No. **19-31-168-01**

Drawing No. **B-3b**

Appendix C

Earthwork Specifications

APPENDIX C: EARTHWORK SPECIFICATIONS

Scope of Work

The work includes all labor, supplies and construction equipment required to construct the building pads in a good, workman-like manner, as shown on the drawings and herein specified. The major items of work covered in this section include the following:

- Site Inspection
- Authority of Geotechnical Engineer
- Site Clearing
- Excavations
- Preparation of Fill Areas
- Placement and Compaction of Fill
- Observation and Testing

Site Inspection

- The Contractor shall carefully examine the site and make all inspections necessary, in order to determine the full extent of the work required to make the completed work conform to the drawings and specifications. The Contractor shall satisfy himself as to the nature and location of the work, ground surface and the characteristics of equipment and facilities needed prior to and during prosecution of the work. The Contractor shall satisfy himself as to the character, quality, and quantity of surface and subsurface materials or obstacles to be encountered. Any inaccuracies or discrepancies between the actual field conditions and the drawings, or between the drawings and specifications must be brought to the Owner's attention in order to clarify the exact nature of the work to be performed.

- This G*eotechnical Study Report* by Converse Consultants may be used as a reference to the surface and subsurface conditions on this project. The information presented in this report is intended for use in design and is subject to confirmation of the conditions encountered during construction. The exploration logs and related information depict subsurface conditions only at the particular time and location designated on the boring logs. Subsurface conditions at other locations may differ from conditions encountered at the exploration locations. In addition, the passage of time may result in a change in subsurface conditions at the exploration locations. Any review of this information shall not relieve the Contractor from performing such independent investigation and evaluation to satisfy himself as to the nature of the surface and subsurface conditions to be encountered and the procedures to be used in performing his work.

Authority of the Geotechnical Engineer

- The Geotechnical Engineer will observe the placement of compacted fill and will take sufficient tests to evaluate the uniformity and degree of compaction of filled ground.

- As the Owner's representative, the Geotechnical Engineer will (a) have the authority to cause the removal and replacement of loose, soft, disturbed and other unsatisfactory soils and uncontrolled fill; (b) have the authority to approve the preparation of native ground to receive fill material; and (c) have the authority to approve or reject soils proposed for use in building areas.

- The Civil Engineer and/or Owner will decide all questions regarding (a) the interpretation of the drawings and specifications, (b) the acceptable fulfillment of the contract on the part of the Contractor and (c) the matters of compensation.

Site Clearing

- Clearing and grubbing shall consist of the removal from building areas to be graded of all existing grass, trees, structures, pavement, utilities, and other vegetation.

- Organic and inorganic materials resulting from the clearing and grubbing operations shall be hauled away from the areas to be graded.

Excavations

- Based on observations made during our field explorations, the surficial soils can be excavated with conventional earthwork equipment.

Preparation of Fill Areas

- All organic material, grass, organic soils, incompetent fill soils and debris should be removed from the proposed building areas.

- In order to provide uniform support for the new reservoir, the minimum depth of over-excavation should be 5 feet below the existing grade, or 2 feet below proposed shallow foundations whichever is deeper. Deeper over-excavation will be needed if soft, yielding soils are exposed on the excavation bottom. The actual depth of removal should be determined based on observations made during grading. Over-excavation should extend a least 5 feet beyond the limits of footings, or equal distance of over-excavation depth, whichever is greater, or as limited by the existing structures. Excavation activities should not disturb existing utilities, buildings, foundations and remaining structures to be protected in place. Existing utilities should be removed and adequately capped at the project boundary line or salvaged/rerouted as designed for sidewalks and flatwork area, at least the upper 24.0 inches of existing soils should be scarified and recompacted to at least ninety percent (90%) of compaction. Deeper over-excavation will be needed if soft, yielding soils are exposed on the excavation bottom. The excavation should be extended to at least 12.0 inches beyond the driveway and flatwork limit where space is permitted.

- The subgrade in all areas to receive fill shall be scarified to a minimum depth of 6.0 inches, the soil moisture adjusted within three percent (3%) above optimum, and then compacted to at least ninety percent (90%) of the laboratory maximum dry density as determined by ASTM Standard D1557 test method.

- Compacted fill may be placed on native soils that have been properly scarified and re-compacted as discussed above.

- All areas to receive compacted fill will be observed and approved by the Geotechnical Engineer before the placement of fill.

Placement and Compaction of Fill

- Compacted fill placed for the support of footings, slabs-on-grade, exterior concrete flatwork, and driveways will be considered structural fill. Structural fill may consist of approved on-site soils or imported fill that meets the criteria indicated below.

- Fill consisting of selected on-site earth materials or imported soils approved by the Geotechnical Engineer shall be placed in layers on approved earth materials. Soils used as compacted structural fill shall have the following characteristics:

    - All fill soil particles shall not exceed 3.0 inches in nominal size and shall be free of organic matter and miscellaneous inorganic debris and inert rubble.

    - Imported fill materials shall have an Expansion Index (EI) less than 20. All imported fill should be compacted to at least ninety percent (90%) of the laboratory maximum dry density (ASTM Standard D1557) at about to three percent (3%) above optimum moisture.

    - Fill soils shall be evenly spread in maximum 8-inch lifts, watered or dried as necessary, mixed and compacted to at least the density specified below. The fill shall be placed and compacted on a horizontal plane, unless otherwise approved by the Geotechnical Engineer.

    - All fill placed at the site shall be compacted to at least ninety percent (90%) of the laboratory maximum dry density as determined by ASTM Standard D1557 test method. The on-site soils shall be moisture conditioned at approximate three percent (3%) above the optimum moisture content.

- Representative samples of materials being used, as compacted fill will be analyzed in the laboratory by the Geotechnical Engineer to obtain information on their physical properties. Maximum laboratory density of each soil type used in the compacted fill will be determined by the ASTM Standard D1557 compaction method.

- Fill materials shall not be placed, spread or compacted during unfavorable weather conditions. When site grading is interrupted by heavy rain, filling operations shall not resume until the Geotechnical Engineer approves the moisture and density conditions of the previously placed fill.

- It shall be the Grading Contractor's obligation to take all measures deemed necessary during grading to provide erosion control devices in order to protect slope areas and adjacent properties from storm damage and flood hazard originating on this project. It shall be the contractor's responsibility to maintain slopes in their as-graded form until all slopes are in satisfactory compliance with job specifications, all berms have been properly constructed, and all associated drainage devices meet the requirements of the Civil Engineer.

Trench Backfill

The following specifications are recommended to provide a basis for quality control during the placement of trench backfill.

- Trench excavations to receive backfill shall be free of trash, debris or other unsatisfactory materials at the time of backfill placement.

- Trench backfill shall be compacted to a minimum relative compaction of ninety percent (90%) as per ASTM Standard D1557 test method.

- Rocks larger than 1.0 inch should not be placed within 12.0 inches of the top of the pipeline or within the upper 12.0 inches of pavement or structure subgrade. No more than thirty percent (30%) of the backfill volume shall be larger than 3/4-inches in largest dimension. Rocks shall be well mixed with finer soil.

- The pipe design engineer should select bedding material for the pipe. Bedding materials generally should have a Sand Equivalent (SE) greater than or equal to 30, as determined by the ASTM Standard D2419 test method.

- Trench backfill shall be compacted by mechanical methods, such as sheepsfoot, vibrating or pneumatic rollers, or mechanical tampers, to achieve the density specified herein. The backfill materials shall be brought to between optimum and three percent (3%) above optimum, then placed in horizontal layers. The thickness of uncompacted layers should not exceed 8.0 inches. Each layer shall be evenly spread, moistened or dried as necessary, and then tamped or rolled until the specified density has been achieved.

- The contractor shall select the equipment and processes to be used to achieve the specified density without damage to adjacent ground and completed work.

- The field density of the compacted soil shall be measured by the ASTM Standard D1556 or ASTM Standard D2922 test methods or equivalent.

- Observation and field tests should be performed by Converse during construction to confirm that the required degree of compaction has been obtained. Where compaction is less than that specified, additional compactive effort shall be made with adjustment of the moisture content as necessary, until the specified compaction is obtained.

- It should be the responsibility of the Contractor to maintain safe conditions during cut and/or fill operations.

- Trench backfill shall not be placed, spread or rolled during unfavorable weather conditions. When the work is interrupted by heavy rain, fill operations shall not be resumed until field tests by the project's geotechnical consultant indicate that the moisture content and density of the fill are as previously specified.

Observation and Testing

- During the progress of grading, the Geotechnical Engineer will provide observation of the fill placement operations.

- Field density tests will be made during grading to provide an opinion on the degree of compaction being obtained by the contractor. Where compaction of less than specified herein is indicated, additional compactive effort with adjustment of the moisture content shall be made as necessary, until the required degree of compaction is obtained

- A sufficient number of field density tests will be performed to provide an opinion to the degree of compaction achieved. In general, density tests will be performed on each one-foot lift of fill, but not less than one for each 500 cubic yards of fill placed.