FINAL GEOTECHNICAL INVESTIGATION REPORT

Reservoir No. 3A, County Service Area 70, Zone1
East of Columbine Road
Oak Hills Area, San Bernardino County, California

Converse Project No. 19-81-275-01

Prepared For:

San Bernardino County Special Districts Department
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May 10, 2021

May 10, 2021

Mr. Philip Krause
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Subject: **FINAL GEOTECHNICAL INVESTIGATION REPORT**
Reservoir No. 3A, County Service Area 70, Zone 1
East of Columbine Road
Oak Hills Area, San Bernardino County, California
Converse Project No. 19-81-275-01

Dear Mr. Krause:

Converse Consultants (Converse) is pleased to submit this Final Geotechnical Investigation Report for the Oak Hills Reservoir No. 3A project, located in the Oak Hills Area, San Bernardino County, California. This report was prepared in accordance with our proposal dated October 18, 2019 and your Work Order number 18407-107 dated November 25, 2019. A draft copy of the report, dated March 13, 2021, was issued for review and comment(s). no comments were received.

We appreciate the opportunity to be of continued service to San Bernardino County Special Districts Department (SBCSDD). Should you have any questions, please do not hesitate to contact us at 909-796-0544.

CONVERSE CONSULTANTS

Hashmi S. E. Quazi, PhD, PE, GE
Principal Engineer

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PROFESSIONAL CERTIFICATION

This report 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.

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1.0 INTRODUCTION

This report contains the findings of the geotechnical investigation performed to assess the suitability of the proposed site and the subsequent design recommendations for Reservoir No. 3A located east of Columbine Road in the Oak Hills Area, San Bernardino County, California. The approximate location of the proposed project site is shown in Figure No. 1, *Approximate Project Location Map.*

Currently, southwest of the proposed reservoir site is occupied by two existing water reservoirs, that will remain. San Bernardino County Special Districts Division (SBCSDD) intends to construct a new reservoir at this location. The purposes of this investigation were to determine the nature and engineering properties of the subsurface soils, and to provide earthwork, design and construction recommendations.

This report is prepared for the project described herein and is intended for use solely by SBCSDD and their authorized agents for design purposes. 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 PROJECT DESCRIPTION

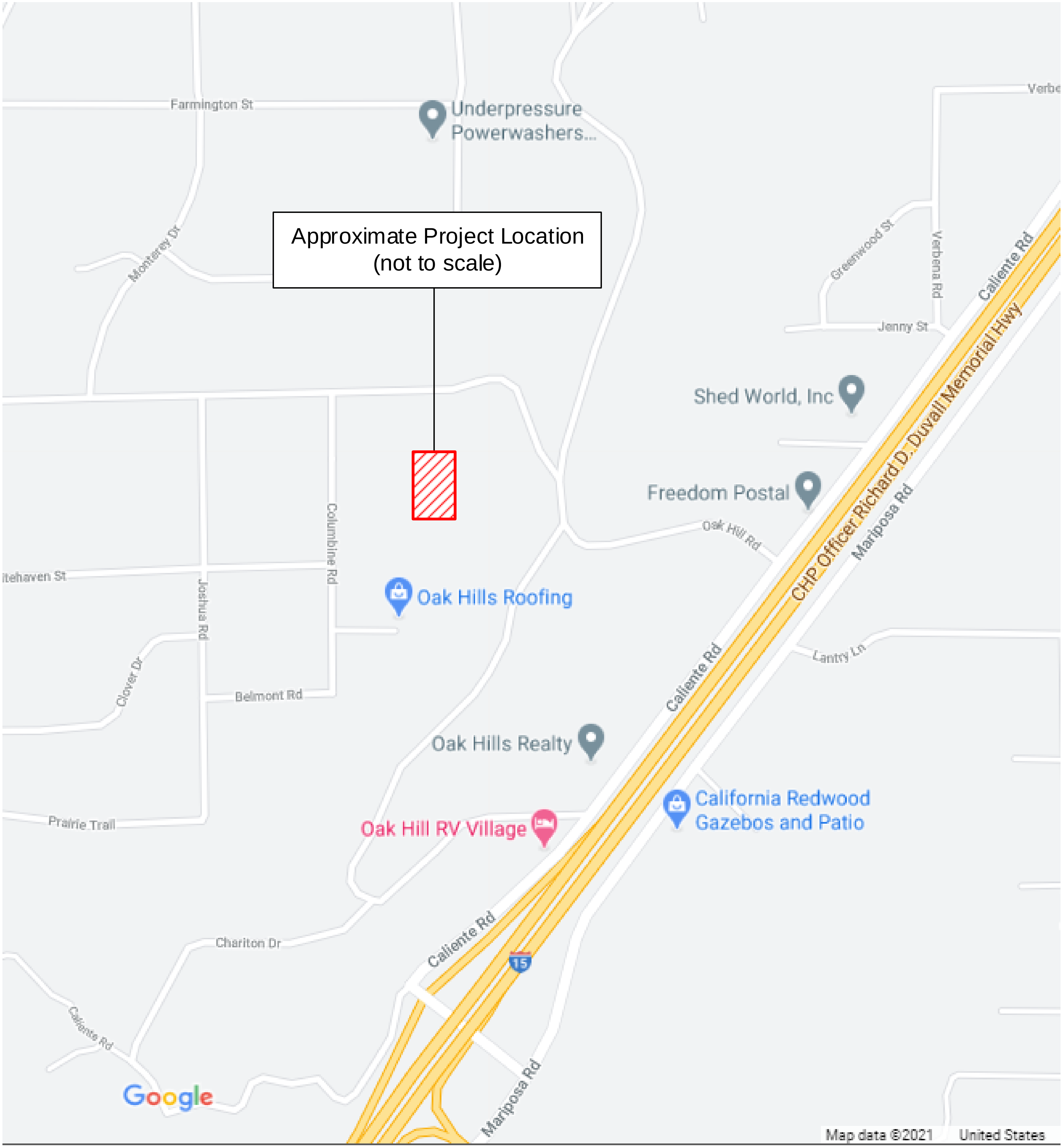
The District intends to construct a new 2 million gallon, 120-foot-diameter, 28-foot-tall, steel reservoir. We understand that the reservoir will be founded on a continuous footing (ring foundation) and the roof supported on columns resting on isolated spread footing.

As indicated on the referenced site layout plan and topographic map prepared by ERSC Inc., the reservoir proposed depths of cuts and fills will be up to approximately 2 feet and 22 feet, respectively. The proposed cut and fill slope will be approximately 2-feet-high and 29-feet-high, respectively, at slope ratios of 2H:1V (horizontal:vertical).

3.0 SITE DESCRIPTION

The site is located approximately 0.5 miles northwest of Interstate Freeway 15, and accessed via Columbine Road, in the Oak Hills area of San Bernardino County, California. The approximate coordinates of the new reservoir are 34.3702 N, 117.4340 W. The site is currently occupied with two existing reservoirs surrounded by rugged terrain with moderate to dense vegetation consisting of grasses, bushes, and trees. Gravel, cobbles, and free scattered boulders cover the remaining surface of the site.

The proposed reservoir will be located on the side of a hill and two steep drainage swales southeast of the existing reservoirs. It is bounded to the north by a private residence, to the east by vacant terrain with small bushes, to the south by the existing reservoirs and a private residence, and to the west by an unpaved access road.



Approximate Project Location Map

Project: Reservoir 3A, Service Area 70, Zone 1
Location: Oak Hills Area, San Bernardino County, California

For: San Bernardino Special Districts Department

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FIGURE NO.
1

Overhead electricity lines traverse in a northeast/southwest direction just beyond the western edge of the project site property boundary.

Elevation of the ground surface varies within the site and generally slopes to the north from between approximately 4,060 feet above mean sea level (amsl) to approximately 4,020 feet amsl. Present site conditions are shown in the photographs no. 1 through 5.

Photograph No. 1: Project site showing proposed reservoir location, facing north.

Photograph No. 2: Project site location showing proposed reservoir location, facing west.

Photograph No. 3: Project site showing existing access road and overhead utilities facing south.

Photograph No. 4: Project site showing dense vegetation facing east.

Photograph No. 5: Project site showing dense vegetation in ravine below existing tanks facing south.

4.0 SCOPE OF WORK

The scope of this investigation included project set-up, subsurface exploration, laboratory testing, engineering analysis, and preparation of this report, as described in the following sections.

4.1 Document Review

We reviewed geologic maps, proposed project plans submitted to us by ERSC, Google Earth aerial photographs, groundwater data, online jurisdictional records, and other information pertaining to the project site to assist in the evaluation of geologic hazards that may be present. We used pertinent information (the documents cited in Section 12, *Reference*s) to understand the subsurface conditions and plan the investigation for this project.

4.2 Project Set-up

The project set-up consisted of the following tasks.

- A field reconnaissance and to mark the test pit locations such that backhoe access to all locations was available.
- Obtained a right-of-entry permit to access site.
- Coordinated with county-appointed ecologist on nesting bird survey for approved test locations.
- Notified Underground Service Alert (USA) at least 48 hours prior to excavation to clear the test pit locations of any conflict with existing underground utilities.
- Engaged a California-licensed backhoe company to excavate test pits.

4.3 Subsurface Exploration

Seven exploratory test pits (TP-01 through TP-07) were excavated on December 4 and 21, 2020 to investigate the subsurface conditions at the proposed reservoir site. The test pits were excavated to the depths ranging from 4 feet to 8 feet below existing ground surface (bgs).

The test pit locations are presented in Figure No. 2, *Approximate Test Pit and Overexcavation Locations Map*. For a description of the field exploration and sampling see Appendix A, *Field Exploration*.

4.4 Laboratory Testing

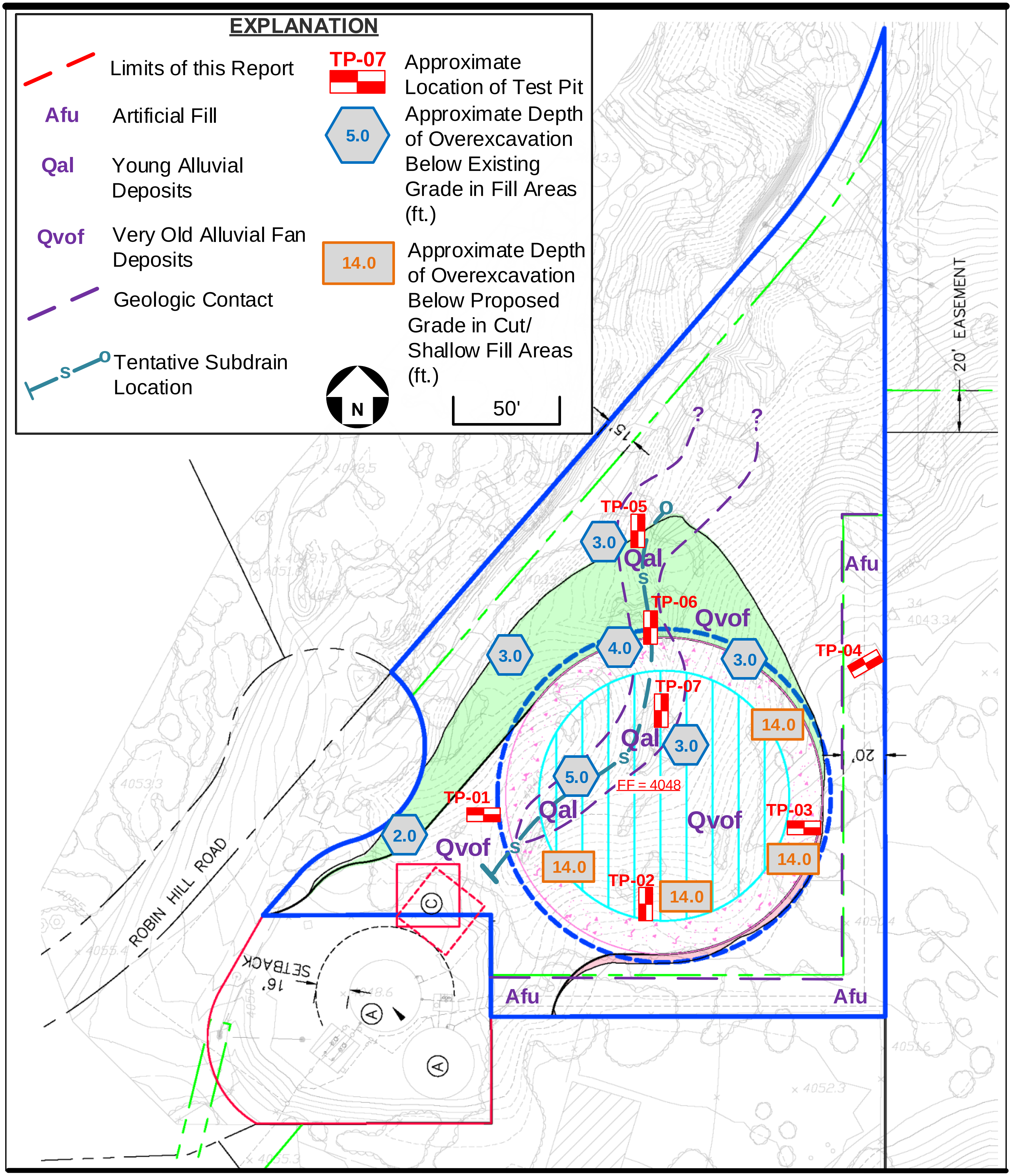
Representative samples were tested in the laboratory to aid in the soils classification and to evaluate the relevant engineering properties of the site soils. These tests included the following.

- *In*-situ moisture contents and dry densities (ASTM D2216 and ASTM D2937)
- Expansion index (ASTM D4829)
- Soil corrosivity (California Test Methods 643, 422, and 417)
- Grain size analysis (ASTM D6913)
- Maximum dry density and optimum-moisture content (ASTM D1557)
- Direct shear (ASTM D3080)
- Consolidation (ASTM D2435)

For *in situ* moisture data, see the Log of Test Pits in Appendix A, *Field Exploration*. For a description of the laboratory test methods and test results, see Appendix B, *Laboratory Testing Program.*

4.5 Analysis and Report Preparation

Data obtained from the field exploration and laboratory testing program were compiled and evaluated. Geotechnical analyses of the compiled data were performed, and this report was prepared to present our findings, conclusions and recommendations for the proposed water reservoir.



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Approximate Test Pit and Overexcavation Locations Map

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FIGURE NO.
2

5.0 SITE CONDITIONS

A general description of the subsurface conditions and various materials encountered during our field exploration are presented in this section.

5.1 Subsurface Profile

Based on the exploratory test pits and laboratory test results, the subsurface soil at the site consists primarily of a mixture of sand, silt and gravel. Scattered to few gravel up to 3 inches was encountered in all the test pits. Few cobble between 4 and 6 inches was encountered in TP-05, TP-06, and TP-07. Due to the dense and dry nature of the soil as well as the gravel content, some of the in-place samples were disturbed or difficult to collect.

For a detailed description of the subsurface materials encountered in the exploratory test pits, see Drawings No. A-2 through A-8, *Logs of Test Pits,* in Appendix A, *Field Exploration.*

5.2 Groundwater

Groundwater was not encountered during the investigation to the maximum explored depth of 8 feet bgs. The GeoTracker and National Water Information System databases (SWRCB, 2021) were reviewed for groundwater data from sites within a one-mile radius of the project site and no data was available.

The historical depth to groundwater, if present at the proposed reservoir site, is not known with certainty. The current groundwater level at the project site is expected to be deeper than 8 feet bgs. Based on the absence of groundwater during our investigation, groundwater is not expected to be encountered during excavation. It should be noted that the groundwater level could vary depending upon the seasonal precipitation and possible groundwater pumping activity in the site vicinity. Shallow perched groundwater may be present locally, particularly following precipitation.

5.3 Excavatability

The subsurface materials at the site are expected to be excavatable by conventional heavy-duty earth moving equipment. <u>Difficult excavation will occur in excavations greater than about 5 feet where high concentrations of gravel and larger materials or cobbles (> 3 inches) as well as dense soils are encountered.</u>

The phrase “conventional heavy-duty excavation equipment” is intended to include commonly used equipment such as excavators, scrapers, and trenching machines. It does not include hydraulic hammers (“breakers”), jackhammers, blasting, or other specialized equipment and techniques used to excavate hard earth materials. Selection of an appropriate excavation equipment models should be done by an experienced earthwork contractor.

5.4 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, care should be exercised in interpolating or extrapolating subsurface conditions between or beyond the test pit locations.

5.5 Flooding

Review of National Flood Insurance Rate Maps (FEMA, 2021) indicates that the project site is within an area defined as a Flood Hazard Zone “D” with the criteria of “Area of Undetermined Flood Hazard”. The entire project site is located outside any San Bernardino County (SBC, 2021a) designated flood or dam inundation zones. Based on the project elevation, flooding is not considered a risk.

6.0 GEOLOGIC SETTING

The regional and local geology are discussed in the following subsections.

6.1 Regional Geology

The Reservoir No. 3A project site is located in the Mojave Desert Geomorphic Province of Southern California. The Mojave Desert is a broad interior region of isolated mountain ranges separated by wide desert plains. The area is roughly triangular shaped and bounded by the Garlock Fault on the north, the San Andreas Fault on the southwest, and the Colorado River on the east. The drainages are primarily closed and terminate in playas within the valley floors.

The province is a seismically active region primarily characterized by a series of northwest-southeast-trending strike-slip faults and east-west trending secondary faults. The most prominent of the nearby fault zones include the Helendale, Lenwood, Landers, and San Andreas Fault Zones, all of which have been known to be active during Quaternary time.

Extension of the region has resulted in exposure of basement rocks dating to the Precambrian age, deposition of young Holocene-aged sedimentary basins, and eruptions of volcanic units.

6.2 Local Geology

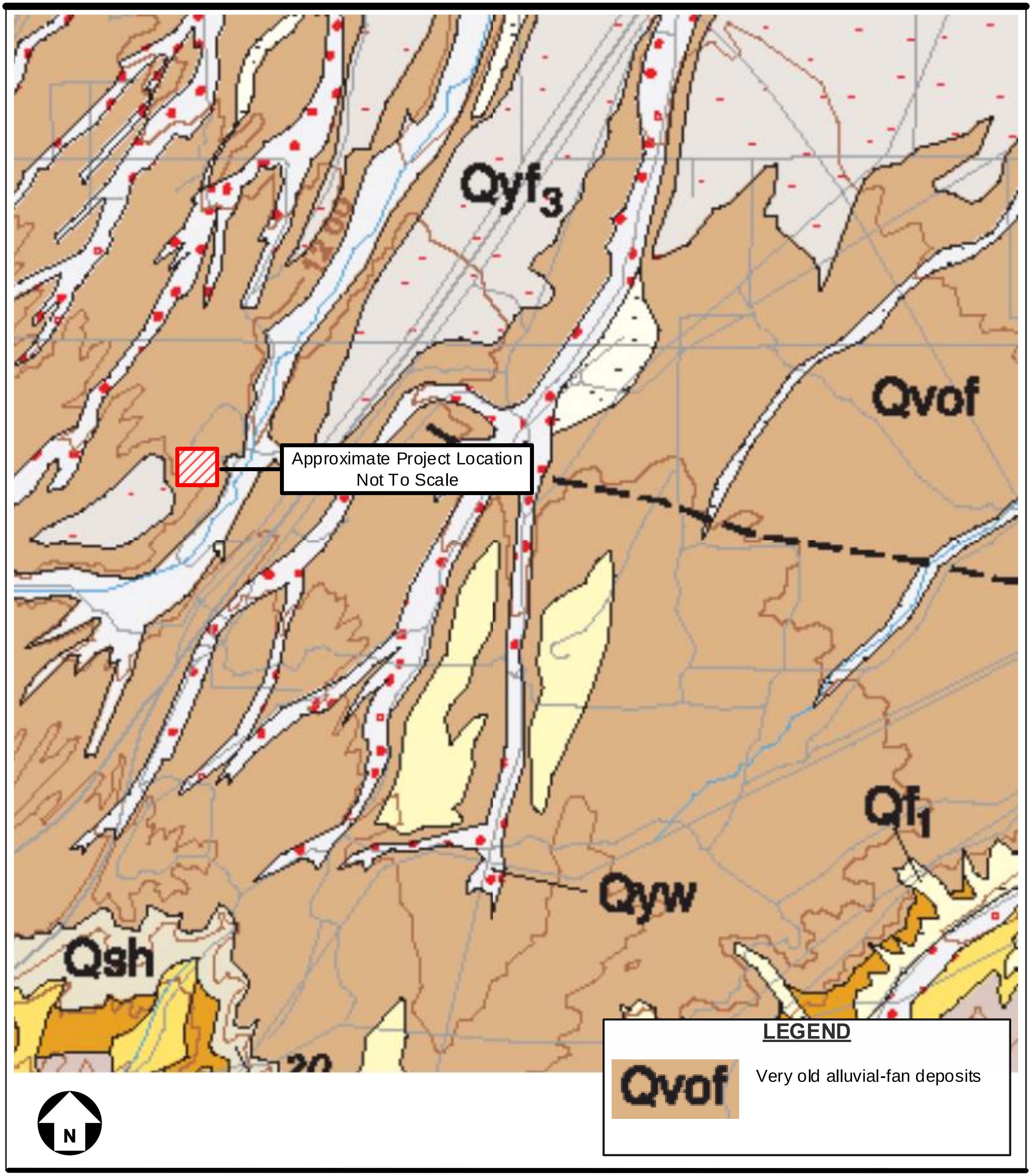
Based on our review of the regional mapping (Morton and Miller, 2006), available geotechnical literature, and our current investigation, it is our understanding that the proposed reservoir site is primarily underlain by Pleistocene-aged very old alluvial fan deposits (Qvof) consisting mainly of massive debris flow deposits of unsorted, unbedded, angular and subrounded gravel and cobbles derived from the San Gabriel mountain terrane to the south. A detailed view of the above referenced geologic map of the proposed project site is shown in Figure No. 3, *Geologic Reference Map.* The location and distribution of the different geologic units is indicated on Figure No. 2, *Approximate Test Pit and Overexcavation Locations Map*. A description of the earth material soils encountered are described below:

Artificial Fill, Undocumented (Afu): Undocumented non-engineered artificial fills were encountered along the eastern edge of the proposed site in TP-04 and is associated with backfill material from an existing water distribution line and access road that originates from the on-site existing reservoirs. Based on our exploration and geologic mapping the approximate depth of the fill soils in this area is expected to be present to approximately 5.0 feet bgs. Where observed these non-engineered fill soils are generally comprised of silty sand, which are very fine to coarse-grained, few to scattered gravel and cobbles, various shades of orangish to yellowish brown and some root and rootlets.

Topsoil (no map symbol): Topsoil was encountered in test pits TP-01, TP-02, and TP-03 to a depth of approximately 0.2 inches bgs. This material is comprised of silty sand which was fine to coarse-grained with few to scattered gravel up to 1.5 inches, roots and rootlets, various shades of brown and reddish brown.

Alluvium (Qal): Alluvium was encountered in test pits TP-05, TP-06, and TP-07 from the surface to depths ranging from 2.0 to 4.0 feet bgs. This material was comprised of silty sand and sand with silt which was fine to coarse-grained with few to scattered gravel up to 3 inches and few cobbles up to 6 inches, roots and rootlets, various shades of brown.

Very Old Alluvial Fan Deposits (Qvof): Very old alluvial fan deposits were encountered in all test pits from below the surface layer to the bottom of each test pit. Where observed, this material was generally comprised of silty sand which was fine to coarse-grained, with few to scattered gravel and scattered cobbles, and were various shades of orangish to reddish browns. Cobbles up to 6 inches were present in this unit. The dense and dry properties of the soil created difficult excavation conditions in areas greater than about 5 feet bgs. The upper 1.0 to 1.5 of this layer is lower in density due to the weathering of this unit.



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Geologic Reference Map

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Figure No.
3

MORTON, D.M. and MILLER, F.K., 2006, Geologic Map of the San Bernardino and Santa Ana 30' x 60' Quadrangles, California, U.S. Geological Survey Open-File Report 2006-1217, scale 1:100,000.

7.0 FAULTING AND SEISMICITY

The location of the proposed Reservoir 3A site in relation to active faults and their seismic activity is discussed below.

7.1 Faulting

The proposed Reservoir 3A site is situated in a seismically active region. As is the case for most areas of Southern California, 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 site. Review of recent seismological and geophysical publications indicates that the seismic hazard for the project is high.

The proposed reservoir site is not located within a currently mapped State of California Earthquake Fault Zone for surface fault rupture. Table No. 2, *Summary of Regional Faults,* summarizes selected data of known faults capable of seismic activity within 100 kilometers of the site. The data presented below was calculated using site coordinates 34.3702°N latitude and 117.4340°W longitude, the National Seismic Hazard Maps Database (USGS, 2008) and other published geologic data.

Table No. 1, Summary of Regional Faults

Fault Name and Section	Closest Distance (km)	Slip Sense	Length (km)	Slip Rate (mm/year)	Maximum Magnitude
Cleghorn	7.14	strike slip	25	3	6.80
S. San Andreas	10.42	strike slip	548	n/a	8.18
San Jacinto	14.28	strike slip	241	n/a	7.88
North Frontal (West)	16.21	reverse	50	1.0	7.20
Cucamonga	21.06	thrust	28	5.0	6.70
San Jose	36.98	strike slip	20	0.5	6.70
Helendale-So Lockhart	38.77	strike slip	114	0.6	7.40
Sierra Madre Connected	39.37	reverse	76	2.0	7.30
Sierra Madre	39.37	reverse	57	2.0	7.20
Clamshell-Sawpit	40.62	reverse	16	0.5	6.70
Chino, alt 2	47.17	strike slip	29	1.0	6.80
Chino, alt 1	47.31	strike slip	24	1.0	6.70
Raymond	56.20	strike slip	22	1.5	6.80
North Frontal (East)	58.38	thrust	27	0.5	7.00
Elsinore	60.55	strike slip	241	n/a	7.85
Lenwood-Lockhart-Old Woman Springs	62.10	strike slip	145	0.9	7.50
Puente Hills (Coyote Hills)	65.96	thrust	17	0.7	6.90

Fault Name and Section	Closest Distance (km)	Slip Sense	Length (km)	Slip Rate (mm/year)	Maximum Magnitude
Landers	69.94	strike slip	95	0.6	7.40
Elysian Park (Upper)	69.96	reverse	20	1.3	6.70
Johnson Valley (No)	70.55	strike slip	35	0.6	6.90
Verdugo	71.45	reverse	29	0.5	6.90
Gravel Hills-Harper Lk	71.99	strike slip	65	0.7	7.10
Puente Hills (Santa Fe Springs)	72.66	thrust	11	0.7	6.70
Pinto Mtn	74.42	strike slip	74	2.5	7.30
Puente Hills (LA)	77.86	thrust	22	0.7	7.00
San Gabriel	78.14	strike slip	71	1.0	7.30
Hollywood	78.52	strike slip	17	1.0	6.70
Sierra Madre (San Fernando)	79.99	thrust	18	2.0	6.70
Blackwater	80.14	strike slip	60	0.5	7.10
Calico-Hidalgo	83.08	strike slip	117	1.8	7.40
Santa Monica Connected alt 2	83.56	strike slip	93	2.4	7.40
San Joaquin Hills	83.79	thrust	27	0.5	7.10
So Emerson-Copper Mtn	83.97	strike slip	54	0.6	7.10
Northridge	87.20	thrust	33	1.5	6.90
Newport Inglewood Connected alt 2	91.77	strike slip	208	1.3	7.50
Newport Inglewood Connected alt 1	91.95	strike slip	208	1.3	7.50
Newport-Inglewood, alt 1	91.95	strike slip	65	1.0	7.20
Santa Monica, alt 1	96.16	strike slip	14	1.0	6.60
Santa Monica Connected alt 1	96.16	strike slip	79	2.6	7.30
Newport-Inglewood (Offshore)	97.19	strike slip	66	1.5	7.00
Santa Susana, alt 1	97.88	reverse	27	5.0	6.90

(Source: https://earthquake.usgs.gov/cfusion/hazfaults_2008_search/)

7.2 CBC Seismic Design Parameters

CBC seismic design parameters based on the 2019 California Building Code (CBC, 2019), ASCE 7-16 and site coordinates 34.3702N latitude and 117.4340W longitude are provided in the following table. These parameters were determined using the ATC Hazards online calculator. The coordinates are in reference to approximately central portion of the referenced project area.

Table No. 2, CBC Mapped Seismic Design Parameters

Seismic Parameters	
Site Coordinates	34.3702N, 117.4340 W
Site Class	D
Risk Category	III
Mapped Short period (0.2-sec) Spectral Response Acceleration, S_s	1.500g
Mapped 1-second Spectral Response Acceleration, S_1	0.606g
Site Coefficient (from Table 11.4-1), F_a	1.0
Site Coefficient (from Table 11.4-2), F_v	1.7
MCE 0.2-sec period Spectral Response Acceleration, S_{MS}	1.500g
MCE 1-second period Spectral Response Acceleration, SM_1	1.030g
Design Spectral Response Acceleration for short period S_{DS}	1.000g
Design Spectral Response Acceleration for 1-second period, S_{D1}	0.687g
Site Modified Peak Ground Acceleration, PGA_M	0.715g

7.3 Site-Specific Seismic Analysis

To develop site-specific seismic design parameters, a site-specific ground motion study was performed in accordance with the 2019 CBC and ASCE 7-16 design guidelines. The methodology and results of this study are presented in Appendix C. Based on the results of this study, site-specific seismic acceleration parameters were developed and summarized in the table below, on the next page.

Table No. 3, Recommended Site-Specific Seismic Acceleration Parameters

Seismic Parameters	Values
MCE 0.2-sec period Spectral Response Acceleration, S_{Ms}	2.137g
MCE 1-second period Spectral Response Acceleration, S_{M1}	2.177g
Design Spectral Response Acceleration for short period, S_{Ds}	1.425g
Design Spectral Response Acceleration for 1-second period, S_{D1}	1.451g
Site-Specific Maximum Peak Ground Acceleration, MCE_G PGA_M	0.719g

7.4 Secondary Effects of Seismic Activity

Generally, in addition to ground shaking, effects of seismic activity on a project site may include surface fault rupture, soil liquefaction, and settlement due to earthquake shaking, landslides, lateral spreading, tsunamis, seiches, and flooding due to earthquake-induced dam failure. The site-specific potential for each of these seismic hazards is discussed in the following sections.

Surface Fault Rupture*:* The proposed reservoir site is not located within a State of California or San Bernardino County designated earthquake fault zone (CGS, 2007; SBC, 2021b). The risk of surface fault rupture is considered low.

Liquefaction: Liquefaction is defined as the phenomenon in which a cohesion-less soil mass suffers a substantial reduction in its shear strength due to the development of excess pore pressures. During earthquakes, excess pore pressures in saturated soil deposits may develop as a result of induced cyclic shear stresses, resulting in liquefaction.
Soil liquefaction generally occurs in submerged granular soils and non-plastic silts located within 50 feet of the ground surface during or after strong ground shaking. There are several general requirements for liquefaction to occur. They are as follows.

- Soils must be submerged.
- Soils must be loose to medium-dense.
- Soils must be relatively near the ground surface.
- Ground motion must be intense.
- Duration of shaking must be sufficient for the soils to lose shear resistance.

The proposed reservoir site is not located within an area not designated as a liquefaction risk by the State of California and San Bernardino County (CGS, 2007; SBC, 2021b). A site-specific liquefaction analysis was beyond of our scope of work. Based on the absence of shallow groundwater and dense subsurface conditions, the potential of liquefaction induced settlement is anticipated negligible.

Seismic Settlement: Seismically induced settlement occurs in loose, granular sediments during ground shaking associated with earthquakes. A site-specific seismic settlement analysis was beyond of our scope of work. Based on the dense subsurface conditions, the potential of seismic settlement is anticipated negligible.

Landslides: Seismically induced landslides and other slope failures are common occurrences during or soon after earthquakes. The Reservoir 3A site is not located within a designated State of California or San Bernardino County landslide hazard zone (CGS, 2007; SBC, 2021b). The slopes within and surrounding the site were observed for slumps, scarps, fissures, deformation or seepage. No visible indications of potential slope movement or instability were observed during our site reconnaissance.

Lateral Spreading: Seismically induced lateral spreading involves primarily lateral movement of earth materials over deeper layers which have liquefied 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. Due to the low risk of liquefaction and dense nature of the soil materials, the risk of lateral spreading is considered low.

Tsunamis: Tsunamis are large waves generated in large bodies of water by fault displacement or major ground movement. Based on the inland location of the proposed reservoir site, tsunamis do not pose a hazard.

Seiches: Seiches are large waves generated in enclosed bodies of water in response to ground shaking. The proposed reservoir site is not located near any large, enclosed bodies of water and is not at risk for flooding due to off-site seiches. Seiching within the new reservoir may result in flooding within the site after construction and filling of the reservoir is complete.

Earthquake-Induced Flooding: Dams or other water-retaining structures may fail as a result of large earthquakes, resulting in flooding. The reservoir site is not located within a State of California or San Bernardino County designated dam inundation area (DWR, 2021 and SBC, 2021a). The risk of earthquake-induced flooding at the reservoir site due to failure of offsite dams is considered low. Failure of the reservoir during an earthquake would result in flooding of the site and surrounding areas.

8.0 LABORATORY TEST RESULTS

Results of physical and chemical tests performed for this project are presented below.

8.1 Physical Testing

Results of the various laboratory tests are presented in Appendix B, *Laboratory Testing Program*, except for the results of in-situ moisture and dry density tests which are presented in the Logs of Test Pits in Appendix A, *Field Exploration*. The results are also discussed below.

- In-situ Moisture and Dry Density – *In-situ* dry density and moisture content of the soils were determined in accordance with ASTM Standard D2216 and D2937. Dry densities (ignoring disturbed samples) of the upper 8 feet soils ranged from 93 to 121 pounds per cubic foot (pcf) with moisture contents ranging from 1 to 8 percent. Results are presented in the Log of Test Pits in Appendix A, *Field Exploration.*
- Expansion Index – One representative sample from the upper 5 feet of soils in test pit TP-03 was tested to evaluate the expansion potential in accordance with ASTM Standard D4829. The test result indicated an EI of 0, corresponding to very low.
- Grain Size Analysis – Two representative samples were tested to determine the relative grain size distribution in accordance with the ASTM Standard D6913. The test results are graphically presented in Drawing No. B-1, *Grain Size Distribution Results.*
- Maximum Dry Density and Optimum Moisture Content – Typical moisture-density relationship tests were performed on three representative soil samples in accordance with ASTM Standard D1557. The result is presented in Drawing No.

B-2, *Moisture-Density Relationship Results*, in Appendix B, *Laboratory Testing Program*. The laboratory maximum dry density the samples tested were 126.4, 131.5 and 126.1 (with rock correction 127.7) pcf and moisture contents of 12.0, 7.5, and 8.5 (with rock correction 8.1 percent) percent, respectively.

- Direct Shear – Two direct shear tests were performed on one sample remolded to 90 percent of the maximum dry density and one to in-situ density under soaked moisture condition in accordance with ASTM Standard D3080. The test results are presented in Drawing No. B-3 and B-4, *Direct Shear Test Results* in Appendix B, *Laboratory Testing Program*.
- Consolidation Test – One consolidation test was performed on a relatively undisturbed sample of the site soil, in accordance with ASTM Standard D2435. The test result is shown on Drawing No. B-5, *Consolidation Test Results,* in Appendix B, *Laboratory Testing Program.*

8.2 Chemical Testing - Corrosivity Evaluation

Two representative soil samples were tested to determine minimum electrical resistivity, pH, and chemical content, including soluble sulfate and chloride concentrations. The purpose of these tests was to determine the corrosion potential of site soils when placed in contact with common pipe materials. The test was performed by AP Engineering and Testing, Inc. (Pomona, CA) in accordance with California Test Methods 643, 422, and 417. The test results are presented in Appendix B, *Laboratory Testing Program* and summarized below.

- The pH measurements of the samples tested were 7.4 and 7.9.
- The sulfate contents of the samples tested were 0.0036 and 0.0037 percent by weight (36 and 37 ppm).
- The chloride concentrations of the samples tested were 84 and 96 ppm.
- The minimum electrical resistivities of the samples when saturated were 12,665 and 7,318 ohm-cm.

9.0 CONCLUSION AND RECOMMENDATIONS

Recommendations for the design and construction of the proposed water reservoir are presented in the following subsections. Our recommendations are based on the subsurface investigation and the results of laboratory testing.

9.1 General Earthwork

Prior to the start of construction, all existing underground utilities and appurtenances should be located within the project area. Such utilities should either be protected in-place or removed and replaced during construction as required by the project specifications. All excavations should be conducted in such a manner as not to cause loss of bearing and/or lateral support of existing utilities.

All debris, surface vegetation, deleterious material, surficial soils containing roots and perishable materials should be stripped and removed from the site. The site should be stripped to the bottom of the roots of vegetation. The actual stripping depth required depends on site usage prior to construction and should be established in the field at the time of construction. Deleterious material, including organics, and debris generated during excavation, should not be placed as fill.

The final bottom surfaces of all excavations should be observed and approved by the project geotechnical consultant prior to placing any fill. Based on these observations, localized areas may require remedial grading deeper than indicated herein. Therefore, some variations in the depth and lateral extent of excavation recommended in this report should be anticipated.

9.2 Over-excavation

Overexcavation recommendations within the different areas of the site is presented below.

General (Fill Areas)

The undocumented artificial fill, topsoil, alluvium and approximately 1 foot to 1.5 feet of the upper weathered very old alluvial fan deposits may be prone to future adverse settlement under the surcharge of reservoir foundations, all improvements, and/or fill loads. These materials should be overexcavated to competent very old alluvial fan deposits and replaced with compacted fill soils.

Reservoir Foundation Areas (Cut or Shallow Fill)

In order to provide uniform support and mitigate potential adverse differential settlement, overexcavations within shallow fill and cut areas of the entire reservoir foundation area should also extend at least 14 feet below existing grade or at least 12 feet below the lowest depth of proposed footings, whichever is deeper. The overexcavation should be uniform and should also extend to at least 10 feet beyond the footprint of the ring footings.

Pavement Areas (Cut)

Overexcavations within proposed cut areas for pavement or other improvements outside of the proposed reservoir foundation can be limited to approximately 2 feet below the existing ground surface, provided natural ground is exposed.

The final bottom surfaces of all excavations should be observed and approved by the project geotechnical consultant prior to placing any fill or structures. However, localized deeper over-excavation could be encountered, based on observations and density testing by the geotechnical consultant during grading of the final bottom surfaces of all excavations.

The estimated locations and approximate depths of over-excavation of unsuitable, compressible soil materials are indicated on Figure No. 2, *Approximate Test Pit and Overexcavation Locations Map*.

If isolated pockets of very soft, loose, eroded, or pumping soil are encountered, the unstable soil should be excavated as needed to expose undisturbed, firm, and unyielding soils.

The contractor should determine the best manner to conduct the excavations, such that there are no losses of bearing and/or lateral support to the existing structures or utilities (if any).

Areas to receive fill and/or other surface improvements should be scarified to a minimum depth of 6 inches, brought to a near-optimum moisture condition, and recompacted to at least 90 percent relative compaction (based on ASTM Test Method D1557).

9.3 Subdrains

A canyon subdrain should be placed in the drainage course located in the northwestern portion of the site, following overexcavation of unsuitable soil materials to competent very old alluvial fan deposits. The subdrain should consist of a 6-inch perforated schedule 40 PVC pipe, or equivalent, encased in at least 9 cubic feet/linear foot of 1/2-inch to 3/4-inch crushed gravel, wrapped in filter fabric (Mirafi 140 or equivalent). The last 20 feet of the lower end of the subdrain should be solid pipe, encased in compacted fill soils. The estimated location of the subdrain is indicated on Figure No. 2, *Approximate Test Pit and Overexcavation Locations Map*.

9.4 Backfill Materials

No fill or aggregate base should be placed until excavations and/or natural ground preparation have been observed by the geotechnical consultant. <u>Backfill materials should be based on SBCSDD Specifications</u>. Additional information is presented below.

Excavated soils should be processed, including cleaning roots and debris, removal of oversized particles, mixing, and moisture conditioning, before placing as compacted fill. Screening may be required to remove oversized particles from some on-site soils. On-site soils used as fill should meet the following criteria.

- No particles larger than 3 inches in largest dimension.
- Rocks larger than one inch should not be placed within the upper 12 inches of subgrade soils.

- Free of all organic matter, debris, or other deleterious material.
- Expansion index of 20 or less.
- Sand Equivalent greater than 15 (greater than 30 for pipe bedding).
- Contain less than 30 percent by weight retained on ¾-inch sieve.
- Contain less than 40 percent fines (passing #200 sieve).

Based on field investigation and laboratory testing results, on-site soils may be suitable as fill materials.

Imported materials, if required, should meet the above criteria prior to being used as compacted fill. Any imported fills should be tested and approved by geotechnical representative prior to delivery to the site.

9.5 Compacted Fill Placement

All surfaces to receive structural fills should be scarified to a depth of 6 inches. The soil should be moisture conditioned to within ±3 percent of optimum moisture content for coarse soils and 0 to 2 percent above optimum moisture content for fine soils. The scarified soils should be recompacted to at least 90 percent of the laboratory maximum dry density.

Fill soils should be thoroughly mixed, and moisture conditioned to within ±3 percent of optimum moisture content for coarse soils and 0 to 2 percent above optimum moisture content for fine soils. Fill soils should be evenly spread in horizontal lifts not exceeding 8 inches in uncompacted thickness.

All fill placed at the site should be compacted to at least 90 percent of the laboratory maximum dry densities as determined by ASTM Standard D1557 test method, unless a higher compaction is specified herein. At least the upper 12 inches of structural bedding materials below the reservoir areas and subgrade soils below finish grade underneath pavement should be compacted to at least 95 percent of the laboratory maximum dry density.

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

9.6 Pipeline Recommendations

On-site pipe trench subgrade preparation and backfill recommendations are presented below.

Pipeline Subgrade Preparation

The final subgrade surface 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. Protruding oversize particles larger than 2 inches in dimension, if any, should be removed from the trench bottom and replaced with compacted on-site materials.

Any loose, soft and/or unsuitable materials encountered at the pipe subgrade should be removed and replaced with an adequate bedding material. 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.

Pipe Bedding and Trench Backfill

Pipe bedding and trench backfill should be as per SBCSDD requirements.

9.7 Foundation Type and Bearing Pressure

The width of ring wall footing should be at least 18 inches. The embedment of ring wall foundation should be at least 18 inches below the adjacent grade to the top of footing. The actual ring wall foundation dimensions and reinforcement should be based on structural design. A ring footing may be designed based on an allowable net bearing capacity of 2,500 psf. The allowable bearing capacity can be increased by 500 pounds per square foot (psf) with each foot of additional embedment and 150 psf with each foot of additional width up to a maximum of 3,500 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. If normal code requirements are applied for design, the above vertical bearing value may be increased by 33 percent for short duration loadings, which will include loadings induced by wind or seismic forces.

9.8 Lateral Earth Pressures and Resistance to Lateral Loads

The following subsections outline lateral earth pressures and resistance to lateral loads. Lateral earth pressures and resistance to lateral loads are estimated by using on-site native soils strength parameters obtained from laboratory testing.

9.8.1 Active Earth Pressures

The active earth pressure behind any buried structure depends primarily on the allowable structure movement, type of backfill materials, backfill slopes, surcharges, and any hydrostatic pressure. We recommend that the reservoir foundation be designed based on the following lateral earth pressure.

Table No. 4, Lateral Earth Pressures

Loading Conditions	Lateral Earth Pressure (psf/ft. of depth)
Active earth conditions (wall is free to deflect at least 0.001 radian)	40
At-rest (wall is restrained)	60

These pressures assume a level backfill, no surcharge and no hydrostatic pressure. If water pressure is allowed to build up behind the structure, the active pressures should be reduced by 50 percent and added to a full hydrostatic pressure to compute the design pressures against the structure.

9.8.2 Passive Earth Pressure

Resistance to lateral loads can be assumed to be provided by a combination of friction acting at the base of reservoir and by passive earth pressure. A coefficient of friction of 0.35 between mass concrete and soil may be used with the dead load forces. An allowable passive earth pressure of 280 psf per foot of depth may be used for the sides of the reservoir against recompacted soils. A factor of safety of 1.5 was applied in calculating passive earth pressure. The maximum value of the passive earth pressure should be limited to 2,500 psf for compacted fill.

Vertical and lateral bearing values indicated above are for the total dead loads and frequently applied live loads. If normal code requirements are applied for design, the above vertical bearing and lateral resistance values may be increased by 33 percent for short duration loading, which will include the effect of wind or seismic forces.

Due to the low overburden stress of the soil at shallow depth, the upper 1-foot of passive resistance should be neglected unless the soil is confined by pavement or slab.

9.9 Permanent Cut Slopes

Cut slopes approximately 2 feet high are proposed on the south and southeast portions the site. Cut slopes should be constructed with slope ratios no steeper than 2:1 (H:V).

Geologic observation of all cut slopes should be conducted during grading to observe whether low-density very old alluvial fan deposits, significant layers of relatively non-cohesive alluvium, adversely oriented planes of weakness or other unfavorable conditions may be exposed. If these conditions are exposed in proposed cut slopes during grading, stabilization fills may be required, which will likely require overexcavation and replacement with compacted fill. Based on the cut slope height and ratio, we anticipate properly constructed cut slopes will be stable.

9.10 Permanent Fill Slopes

Fill slopes approximately 29 feet high are proposed on the north and northwest portions of the site. Fill slopes should be constructed with slope ratios no steeper than 2:1 (H:V).

Fill slopes should be constructed on compacted fill prepared in accordance with Section 9.5, *Compacted Fill Placement*. Fill slopes should be properly compacted out to the slope face. This may be achieved by either overbuilding then cutting back to the compacted core, frequent backrolling, or by utilizing other methods that meet the intent of the project specifications. The fill slope face should be track rolled to achieve compaction. Based on our slope stability analysis presented in Appendix D, *Slope Stability Analysis*, we understand properly constructed fill slopes will be stable.

9.11 Shrinkage and Subsidence

The volume of excavated and recompacted soils will decrease as a result of grading. The shrinkage would depend on, among other factors, the depth of cut and/or fill, and the grading method and equipment utilized. Based on our previous experience in the other projects in close vicinity of this site, for the preliminary estimation, shrinkage factors for various units of earth material at the site may be taken as presented below.

- The shrinkage factor (defined as a percentage of soil volume reduction when moisture conditioned and compacted to the average of 92 percent relative compaction) for the upper 5 to 8 feet of soils is estimated. An average value of 7 percent may be used for preliminary earthwork planning.
- Subsidence (defined as the settlement of native materials from the equipment load applied during grading) would depend on the construction methods including type of equipment utilized. Ground subsidence is estimated to be approximately 0.1 foot to 0.15 foot.

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.

9.12 Settlement

Provided the previous earthwork recommendations in *Section 9.1 through 9.3* are implemented during construction, the total settlement of shallow footings from static structural loads and short-term settlement of properly compacted fill is anticipated to be 0.5 inch or less. The differential settlement resulting from static loads is anticipated to be 0.25 inches or less over a horizontal distance of 40 feet.

A site-specific liquefaction and seismic settlement analysis were beyond of our scope of work. Based on the absence of shallow groundwater and dense subsurface conditions, the potential of liquefaction and seismic induced settlement is anticipated negligible.

Generally, the static and dynamic settlement does not occur at the same time. For design purposes, the structural engineer should decide whether static and dynamic settlement will be combined or not.

9.13 Soil Corrosivity

Two representative soil samples from the site were evaluated for corrosivity with respect to common construction materials such as concrete and steel. The test results are presented in Appendix B, *Laboratory Testing Program* and design recommendations pertaining to soil corrosivity are presented below.

The sulfate content of the sampled soils corresponds to American Concrete Institute (ACI) exposure category S0 for these sulfate concentrations (ACI 318-14, Table 19.3.1.1). No concrete type restrictions are specified for exposure category S0 (ACI 318-14, Table 19.3.2.1). A minimum compressive strength of 2,500 psi is recommended.

We anticipate that concrete structures such as footings, slabs, and flatwork (if any) will be exposed to moisture from precipitation and irrigation. Based on the location and the result of chloride testing of the site soils, we do not anticipate that 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 (ACI 318-14, Table 19.3.1.1). ACI provides concrete design recommendations in ACI 318-14, Table 19.3.2.1, including a compressive strength of at least 2,500 psi and a maximum chloride content of 0.3 percent.

According to Romanoff, 1957, the following table provides general guideline of soil corrosion based on electrical resistivity.

Table No. 5, Correlation Between Resistivity and Corrosion

Soil Resistivity (ohm-cm) per Caltrans CT 643	Corrosivity Category
Over 10,000	Mildly corrosive
2,000 – 10,000	Moderately corrosive
1,000 – 2,000	corrosive
Less than 1,000	Severe corrosive

The measured value of the minimum electrical resistivity of the samples when saturated were 7,318 and 12,665 ohm-cm. This indicates that the samples tested were moderately corrosive to ferrous metals in contact with the soil (Romanoff, 1957). Converse does not practice in the area of corrosion consulting. If needed, a qualified corrosion consultant should provide appropriate corrosion mitigation measures for any ferrous metals in contact with the site soils.

10.0 GEOTECHNICAL SERVICES DURING CONSTRUCTION

The project geotechnical consultant should review plans and specifications as the project design progresses. Such review is necessary to identify design elements, assumptions, or new conditions which require revisions or additions to our geotechnical recommendations.

The project geotechnical consultant should be present to observe conditions during construction. Geotechnical observation and testing should be performed as needed to verify compliance with project specifications. Additional geotechnical recommendations may be required based on subsurface conditions encountered during construction.

11.0 CLOSURE

This report is prepared for the project described herein and is intended for the sole use of SBCSDD and their authorized agents, to assist in the design and construction of the proposed project. Our findings and recommendations were obtained in accordance with generally accepted professional principles practiced in geotechnical engineering. We make no other warranty, either expressed or implied.

Converse Consultants is not responsible or liable for any claims or damages associated with interpretation of available information provided to others. Site exploration identifies actual soil conditions only at those points where samples are taken, when they are taken. Data derived through sampling and laboratory testing is extrapolated by Converse employees who render an opinion about the overall soil conditions. Actual conditions in areas not sampled may differ. In the event that changes to the project occur, or additional, relevant information about the project is brought to our attention, the recommendations 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. In addition, the recommendations can only be finalized by observing actual subsurface conditions revealed during construction. Converse cannot be held responsible for misinterpretation or changes to our recommendations made by others during construction.

As the project evolves, continued consultation and construction monitoring by a qualified geotechnical consultant should be considered an extension of geotechnical investigation services performed to date. The geotechnical consultant should review plans and specifications to verify that the recommendations presented herein have been appropriately interpreted, and that the design assumptions used in this report are valid.

Where significant design changes occur, Converse may be required to augment or modify the recommendations presented herein. Subsurface conditions may differ in some locations from those encountered in the explorations, and may require additional analyses and, possibly, modified recommendations.

Design recommendations given in this report are based on the assumption that the 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 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.

12.0 REFERENCES

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Appendix A

Field Exploration

APPENDIX A

FIELD EXPLORATION

Our field investigation included a site reconnaissance and a subsurface exploration program consisting of excavating test pits. During the site reconnaissance, the surface conditions were noted, and the test pit locations were marked in the field with reference to property boundaries, and other visible features. The test pit locations should be considered accurate only to the degree implied by the method used to mark them in the field.

Seven exploratory test pits (TP-01 through TP-07) were excavated on December 4 and 21, 2020 with a backhoe equipped with a 24-inch-wide bucket to investigate subsurface conditions at the proposed reservoir site. The test pits were excavated to the depths ranging from 4 feet to 8 feet below existing ground surface (bgs).

Encountered materials were continuously logged by a Converse geologist and classified in the field by visual classification in accordance with the Unified Soil Classification System. Where appropriate, the field descriptions and classifications have been modified to reflect laboratory test results.

Relatively undisturbed samples were obtained using California Modified Samplers (2.4 inches inside diameter and 3.0 inches outside diameter) lined with thin sample rings. The steel ring sampler was driven at selected level benches in the test pit using manual hand-operated equipment with successive drops of a 35-pound driving weight falling 30 inches. Samples were retained in brass rings (2.4 inches inside diameter and 1.0 inch in height) and carefully sealed in waterproof plastic containers for shipment to the Converse laboratory. In-situ dry densities and moisture contents of the site soils were also determined in the test pits with a nuclear testing gauge in accordance with ASTM Standard D6938. Representative bulk samples were collected from selected depths within the test pits and placed in large plastic bags for delivery to our laboratory.

Following the completion of logging and sampling, test pits were backfilled with excavated soil and compacted by tamping with the bucket of the backhoe and rolling the weight of the equipment over the surface. The ground surface at the test pit locations may settle over time. If construction is delayed, we recommend the owner monitor the test pit locations and backfill any depressions that occur or provide protection around the test pit locations to prevent trip and fall injuries from occurring.

For a key to soil symbols and terminology used in the test pit logs, refer to Drawing No. A-1, *Unified Soil Classification and Key to Test Pit Symbols*. Test pit logs are presented in Drawings No. A-2 through A-8.

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 (CMS).

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
Disturb	Dist.

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
Permeability	perm
Soil Cement	sc

Apparant Density	Very Loose	Loose	Medium	Dense	Very Dense
SPT (N)	< 4	4 - 11	11 - 30	31 - 50	> 50
CA Sampler	< 5	5 - 12	13 - 35	36 - 60	> 60
Relative Density (%)	< 20	20 - 40	40 - 60	60 - 80	> 80

Consistency	Very Soft	Soft	Medium	Stiff	Very Stiff	Hard
SPT (N)	< 2	2-4	5-8	9-15	16-30	> 30
CA Sampler	< 3	3-6	7-12	13-25	26-50	> 50

UNIFIED SOIL CLASSIFICATION AND KEY TO TEST PIT SYMBOLS

Converse Consultants

Reservoir 3A, County Service Area 70, Zone 1
East of Columbine Road
Oak Hills Area, San Bernardino County, California
For: San Bernardino County Special Districts Division

Project No.
19-81-275-01

Drawing No.
A-1

Project ID: 19-81-275-01.GPJ; Template: KEY

Log of Test Pit No. TP-01

Dates Drilled: 12/4/2020 Logged by: Catherine Nelson Checked By: Robert Gregorek

Equipment: Backhoe with 24" wide Bucket Driving Weight and Drop: N/A

Ground Surface Elevation (ft): 4046 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
		TOPSOIL **SILTY SAND (SM):** fine to coarse-grained, few gravel up to 1.5" in largest dimension, roots and rootlets, light reddish brown.		X		6	111	NUC @ 1'
		VERY OLD ALLUVIAL FAN DEPOSITS **SILTY SAND (SM):** fine to coarse-grained, few gravel up to 3" in largest dimension, moderately to very desiccated, reddish brown.	X		disturbed	3		
5		@3': decrease in gravel size.		X		8	108	NUC @ 5' ca, er
		@5': very few gravel up to 1" in largest dimension, slightly desiccated, increase in moisture, larger sand grains, dark reddish brown.		X				
		End of Test Pit at 8 feet bgs. No groundwater encountered. Backfilled with soil cuttings and compacted by pushing down the bucket using the weight of the backhoe on 12/04/2020.						

Log of Test Pit No. TP-02

Dates Drilled: 12/4/2020 Logged by: Catherine Nelson Checked By: Robert Gregorek

Equipment: Backhoe with 24" wide Bucket Driving Weight and Drop: N/A

Ground Surface Elevation (ft): 4046 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
		TOPSOIL **SILTY SAND (SM):** fine to coarse-grained, few gravel up to 1.5" in largest dimension, roots and rootlets, brown.						
5		**VERY OLD ALLUVIAL FAN DEPOSITS** **SILTY SAND (SM):** fine to coarse-grained, few gravel up to 2" in largest dimension, very desiccated, dark reddish brown.		X		5	106	NUC @ 4'
		End of Test Pit at 5 feet bgs. No groundwater encountered. Backfilled with soil cuttings and compacted by pushing down the bucket using the weight of the backhoe on 12/04/2020.						

Log of Test Pit No. TP-03

Dates Drilled: 12/4/2020 Logged by: Catherine Nelson Checked By: Robert Gregorek

Equipment: Backhoe with 24" wide Bucket Driving Weight and Drop: N/A

Ground Surface Elevation (ft): 4045 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
		TOPSOIL **SILTY SAND (SM):** fine to coarse-grained, scattered gravel up to 1.5" in largest dimension, roots and rootlets, brown.				5	115	ei, ma, max, ds NUC @ 2'
5		**VERY OLD ALLUVIAL FAN DEPOSITS** **SILTY SAND (SM):** fine to coarse-grained, few gravel up to 2" in largest dimension, very desiccated, dark reddish brown.						
		End of Test Pit at 5 feet bgs. No groundwater encountered. Backfilled with soil cuttings and compacted by pushing down the bucket using the weight of the backhoe on 12/04/2020.						

Log of Test Pit No. TP-04

Dates Drilled: 12/21/2020 Logged by: Catherine Nelson Checked By: Robert Gregorek

Equipment: Backhoe with 24" wide Bucket Driving Weight and Drop: N/A

Ground Surface Elevation (ft): 4047 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
		ARTIFICIAL FILL **SILTY SAND (SM):** fine to coarse-grained, few gravel up to 2" in largest dimension, dry, slightly desiccated, some pinholes, roots and rootlets, light orangish brown to yellowish gray.		X				max, ca, er
				X		4	117	NUC @ 2'
				X		4	117	NUC @ 4'
5		**VERY OLD ALLUVIAL FAN DEPOSITS** **SILTY SAND (SM):** fine to coarse-grained, few gravel up to 2" in largest dimension, more fines, very desiccated, reddish brown.	X		disturbed	3		
		End of Test Pit at 5.5 feet bgs. No groundwater encountered. Backfilled with soil cuttings and compacted by pushing down the bucket using the weight of the backhoe on 12/21/2020.						

Log of Test Pit No. TP-05

Dates Drilled: 12/21/2020 Logged by: Catherine Nelson Checked By: Robert Gregorek

Equipment: Backhoe with 24" wide Bucket Driving Weight and Drop: N/A

Ground Surface Elevation (ft): 4026 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 **SILTY SAND (SM):** fine to coarse-grained, few gravel up to 3" and cobble up to 4" in largest dimension, slight consolidation, roots and rootlets, brown.						
						2	98	
5		**VERY OLD ALLUVIAL FAN DEPOSITS** **SILTY SAND (SM):** fine to coarse-grained, scattered gravel up to 2" in largest dimension, larger sand grains, moderately desiccated, roots, reddish brown.				5	109	NUC @ 4'
						8	118	ds
		End of Test Pit at 7 feet bgs. No groundwater encountered. Backfilled with soil cuttings and compacted by pushing down the bucket using the weight of the backhoe on 12/21/2020.						

Log of Test Pit No. TP-06

Dates Drilled: 12/21/2020 Logged by: Catherine Nelson Checked By: Robert Gregorek

Equipment: Backhoe with 24" wide Bucket Driving Weight and Drop: N/A

Ground Surface Elevation (ft): 4029 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 **SAND WITH SILT (SP-SM):** fine to coarse-grained, scattered gravel up to 3" and cobbles up to 6" in largest dimension, roots and rootlets, brown.	■	☒		1	96	ma
5		**VERY OLD ALLUVIAL FAN DEPOSITS** **SILTY SAND (SM):** fine to coarse-grained, scattered gravel up to 2.5" in largest dimension, slightly desiccated, roots, reddish brown.						
		End of Test Pit at 4 feet bgs. No groundwater encountered. Backfilled with soil cuttings and compacted by pushing down the bucket using the weight of the backhoe on 12/21/2020.						

Log of Test Pit No. TP-07

Dates Drilled: 12/21/2020 Logged by: Catherine Nelson Checked By: Robert Gregorek

Equipment: Backhoe with 24" wide Bucket Driving Weight and Drop: N/A

Ground Surface Elevation (ft): 4032 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 **SILTY SAND (SM):** fine to coarse-grained, few gravel up to 3" and cobbles up to 6" in largest dimension, roots and rootlets, brown.				2	104	NUC @ 1'
		VERY OLD ALLUVIAL FAN DEPOSITS **SILTY SAND (SM):** fine to coarse-grained, few gravel up to 3" in largest dimension, moderately desiccated, larger sand grains, roots, reddish brown.				3 7	121 118	NUC @ 3' c
5		End of Test Pit at 4 feet bgs. No groundwater encountered. Backfilled with soil cuttings and compacted by pushing down the bucket using the weight of the backhoe on 12/21/2020.						

Converse Consultants

Reservoir 3A, County Service Area 70, Zone 1
East of Columbine Road
Oak Hills Area, San Bernardino County, California
For: San Bernardino County Special Districts Division

Project No. **19-81-275-01**

Drawing No. **A-8**

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 physical properties and engineering characteristics. The amount and selection of tests were based on the geotechnical parameters required for this project. Test results are presented herein and on the Log of the test pits, in Appendix A, *Field Exploration*. The following is a summary of the various laboratory tests conducted for this project.

In-Situ **Moisture Content and Dry Density**

In-situ dry density and moisture content tests were performed on relatively undisturbed ring samples, in accordance with ASTM Standard D2216 and D2937 to aid soils classification and to provide qualitative information on strength and compressibility characteristics of the site soils. For test results, see the Logs of Test pits in Appendix A, Field Exploration.

Expansion Index

One sample was tested to evaluate the expansion potential of material encountered at the site. The test was conducted in accordance with ASTM Standard D4829. The test result is presented in the following table.

Table No. B-1, Summary of Expansion Index Test Result

Test pit No.	Depth (feet)	Soil Description	Expansion Index	Expansion Potential
TP-03	0-5	Silty Sand (SM)	0	Very Low

Soil Corrosivity

Two soil samples were tested to determine minimum electrical resistivities, pH, and chemical content, including soluble sulfate and chloride concentrations. The purpose of these tests was to determine the corrosion potential of site soils when placed in contact with common construction materials. This test was performed by AP Engineering and Testing, Inc. (Pomona, CA) in accordance with California Test Methods 643, 422 and 417. The test result is presented on the following table.

Table No. B-2, Summary of Corrosivity Test Results

Test pit No.	Depth (feet)	pH	Soluble Sulfates (CA 417) (ppm)	Soluble Chlorides (CA 422) (ppm)	Min. Resistivity (CA 643) (Ohm-cm)
TP-01	5-6	7.4	36	84	12,665
TP-04	0-3	7.9	37	96	7.318

Grain-Size Analyses

To assist in classification of soils, mechanical grain-size analyses were performed on two select samples in accordance with the ASTM Standard D6913 test method. Grain-size curves are shown in Drawing No. B-1, *Grain Size Distribution Results*.

Table No. B-3, Grain Size Distribution Test Results

Test pit No./ Report	Depth (ft)	Soil Classification	% Gravel	% Sand	%Silt	%Clay
TP-03	0-5	Silty Sand (SM)	0	63	37	
TP-06	0-3	Sand with Silt (SP-SM)	2	86	12	

Laboratory Maximum Dry Density and Optimum Moisture Content

Three laboratory maximum dry density and optimum moisture content relationship tests were performed on bulk samples in accordance with ASTM Standard D1557 method. The test results are presented on Drawing No. B-2, Moisture-Density Relationship Results, and are summarized in the following table.

Table No. B-4, Summary of Moisture-Density Relationship Results

Test pit No.	Depth (feet)	Soil Description	Maximum Dry Density (pcf)	Optimum Moisture (%)
TP-03	0-5	Silty Sand (SM), reddish brown	126.4	12.0
TP-04	0-3	Silty Sand (SM), orangish brown	131.5	7.5
TP-06	0-3	Silty Sand (SM), brown	126.1 (127.7)*	8.5 (8.1)*

(* Rock correction: TP-06 = 5.4%)

Direct Shear

Two direct shear tests were performed on one sample remolded to 90 percent of the maximum dry density and one to in-situ density under soaked moisture condition in accordance with ASTM Standard D3080. For each 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 sample was then sheared at a constant strain rate of 0.02 inch/minute. Shear deformation was recorded until a maximum of about 0.25-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 results, including sample density and moisture content, see Drawings No. B-3 and B-4, *Direct Shear Test Results,* and the following table.

Table No. B-5, Direct Shear Test Results

Test pit No.	Depth (feet)	Soil Description	Ultimate Strength Parameters	
			Friction Angle (degrees)	Cohesion (psf)
*TP-03	0-5	Silty Sand (SM)	33	60
+TP-05	6-7	Silty Sand (SM)	34	20

(*Samples remolded to 90% of the maximum dry density)

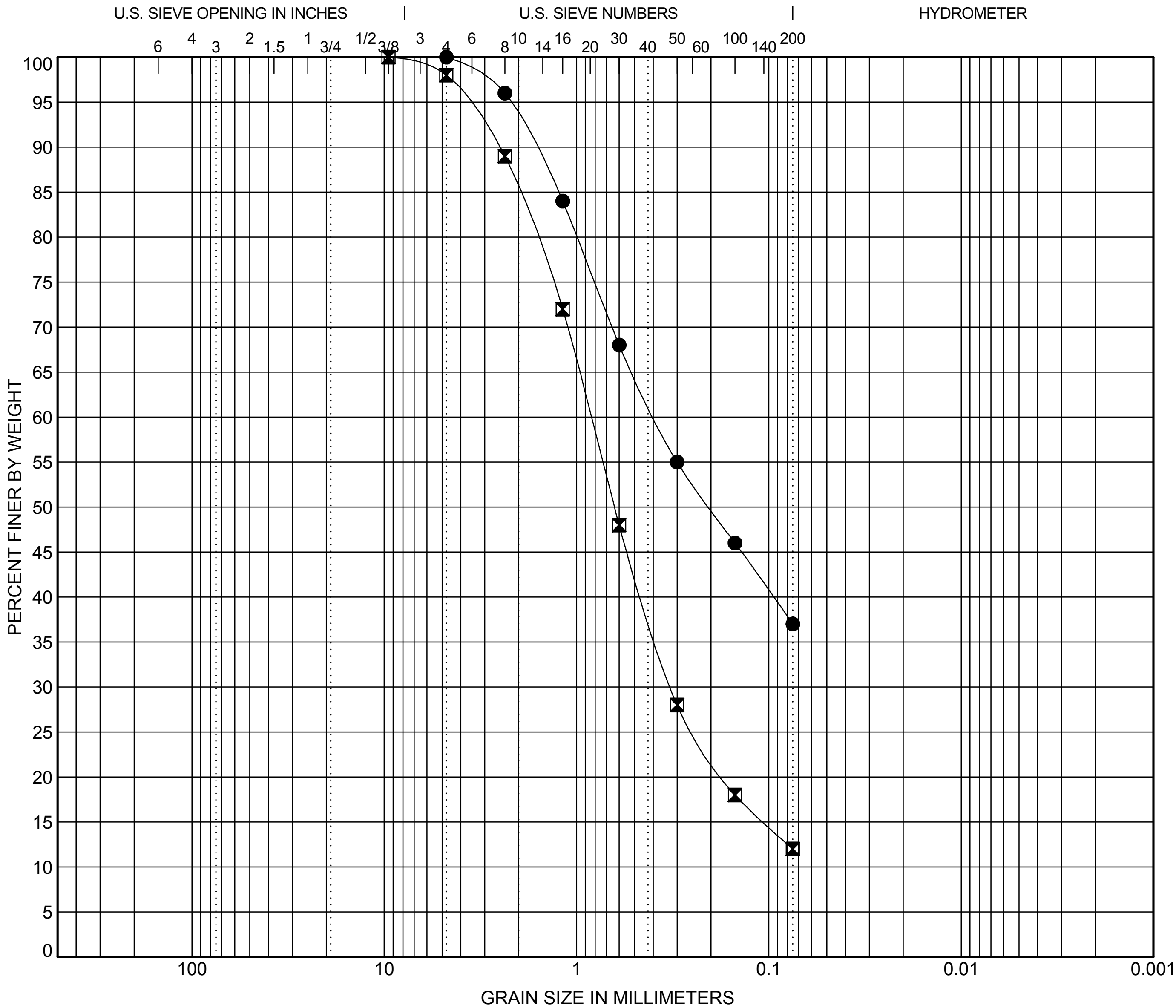
(+Samples remolded to the in-place dry density)

Consolidation

One test was conducted in accordance with ASTM Standard D2435 method. Data obtained from this test on a relatively undisturbed ring sample was used to evaluate the settlement characteristics of the on-site soils under load. Preparation for this test involved trimming the sample, placing it in a 1-inch-high brass ring, and loading it into the test apparatus, which contained porous stones to accommodate drainage during testing. Normal axial loads were applied to one end of the sample through the porous stones, and the resulting deflections were recorded at various time periods. The load was increased after the sample reached a reasonable state of equilibrium. Normal loads were applied at a constant load-increment ratio, successive loads being generally twice the preceding load. For test result, including sample density and moisture content, see Drawing No. B-5, *Consolidation Test Result*.

Sample Storage

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



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	Boring No.	Depth (ft)	Description	LL	PL	PI	Cc	Cu
●	**TP-03**	**0.0 to 5.0**	**Silty Sand (SM)**					
⊠	**TP-06**	**0.0 to 3.0**	**Sand with Silt (SP-SM)**				**2.06**	**14.14**

	Boring No.	Depth (ft)	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**TP-03**	**0.0 to 5.0**	**4.75**	**0.392**			**0.0**	**63.0**	**37.0**	
⊠	**TP-06**	**1.0 to 2.0**	**9.5**	**0.841**	**0.322**		**2.0**	**86.0**	**12.0**	

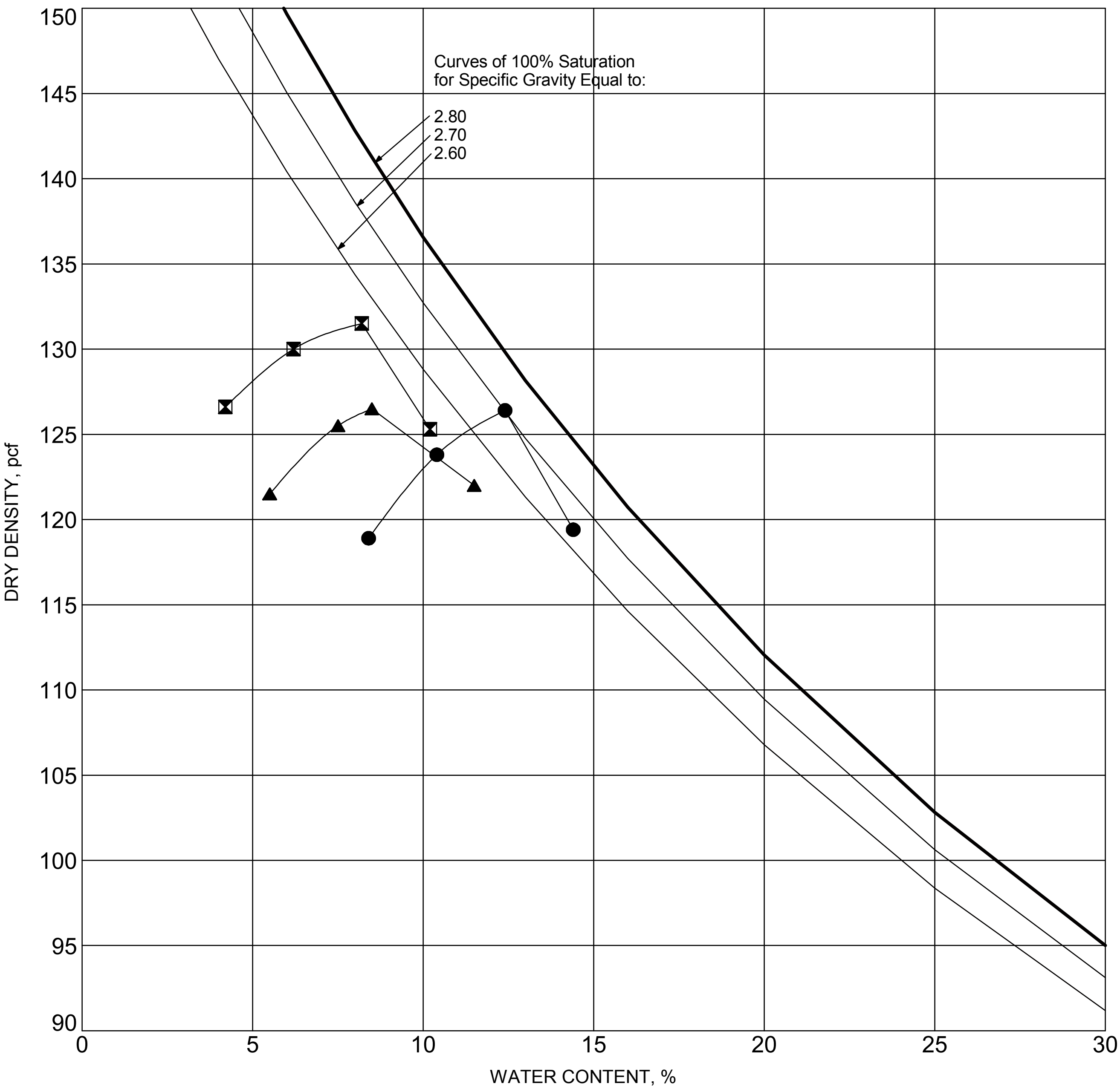
GRAIN SIZE DISTRIBUTION RESULTS

Converse Consultants

Reservoir 3A, County Service Area 70, Zone 1
East of Columbine Road
Oak Hills Area, San Bernardino County, California
For: San Bernardino County Special Districts Division

Project No.
19-81-275-01

Drawing No.
B-1



SYMBOL	BORING NO.	DEPTH (ft)	DESCRIPTION	ASTM TEST METHOD	OPTIMUM WATER, %	MAXIMUM DRY DENSITY, pcf
●	**TP-03**	**0 to 5**	**Silty Sand (SM), reddish brown**	**D1557 - A**	**12**	**126.4**
⊠	**TP-04**	**0 to 3**	**Silty Sand (SM), orangish brown**	**D1557 - A**	**7.5**	**131.5**
▲	**TP-06**	**1 to 2**	**Silty Sand (SM), brown**	**D1557 - A**	**8.5 (8.1*)**	**126.1 (127.7*)**

(*with rock correction 5.4%)

MOISTURE-DENSITY RELATIONSHIP RESULTS

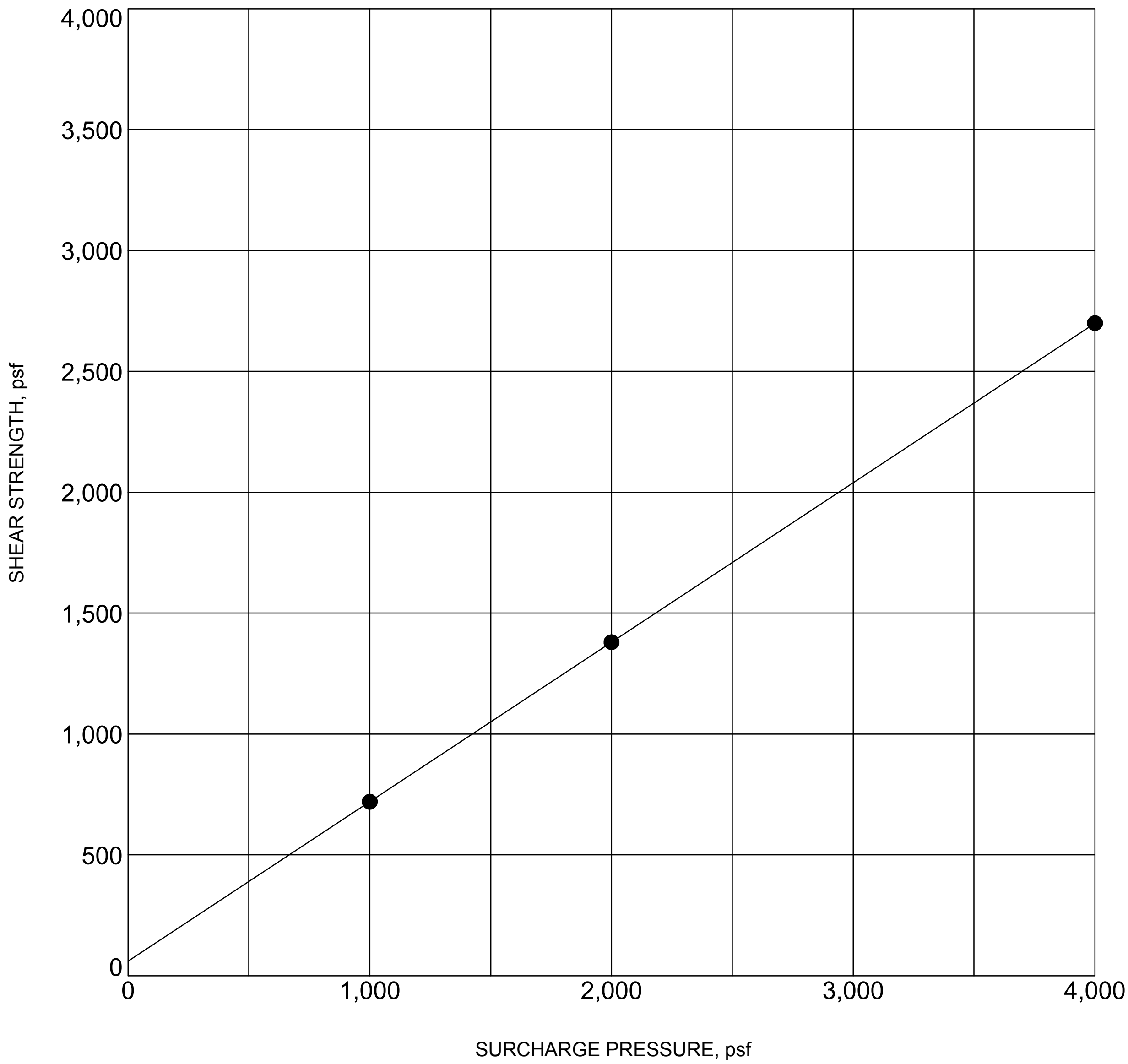
Converse Consultants

Reservoir 3A, County Service Area 70, Zone 1
East of Columbine Road
Oak Hills Area, San Bernardino County, California
For: San Bernardino County Special Districts Division

Project No. **19-81-275-01**

Drawing No. **B-2**

Project ID: 19-81-275-01.GPJ; Template: COMPACTION



BORING NO. :	**TP-03***	DEPTH (ft) :	**0.0 to 5.0**
DESCRIPTION :	**Silty Sand (SM)**		
COHESION (psf) :	**60**	FRICTION ANGLE (degrees):	**33**
MOISTURE CONTENT (%) :	**5.0**	DRY DENSITY (pcf) :	**115.0**

(*Remolded to 90% of the maximum dry density)

NOTE: Ultimate Strength.

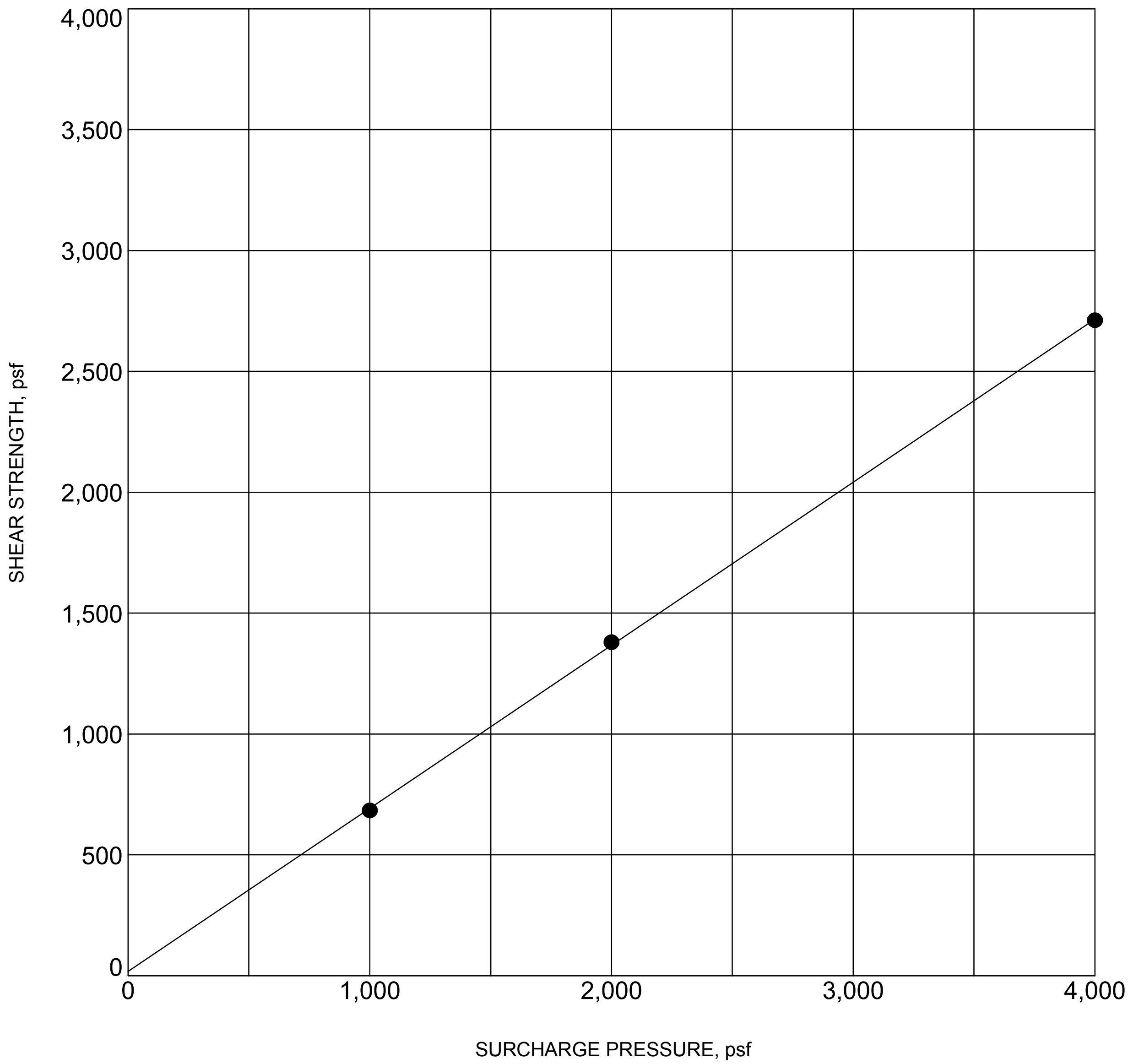
DIRECT SHEAR TEST RESULTS

Converse Consultants

Reservoir 3A, County Service Area 70, Zone 1
East of Columbine Road
Oak Hills Area, San Bernardino County, California
For: San Bernardino County Special Districts Division

Project No. **19-81-275-01**

Drawing No. **B-3**



BORING NO. :	**TP-05***	DEPTH (ft) :	**6.0 to 7.0**
DESCRIPTION :	**Silty Sand (SM)**		
COHESION (psf) :	**20**	FRICTION ANGLE (degrees):	**34**
MOISTURE CONTENT (%) :	**8.0**	DRY DENSITY (pcf) :	**118.0**

(*Remolded to in-situ density)

NOTE: Ultimate Strength.

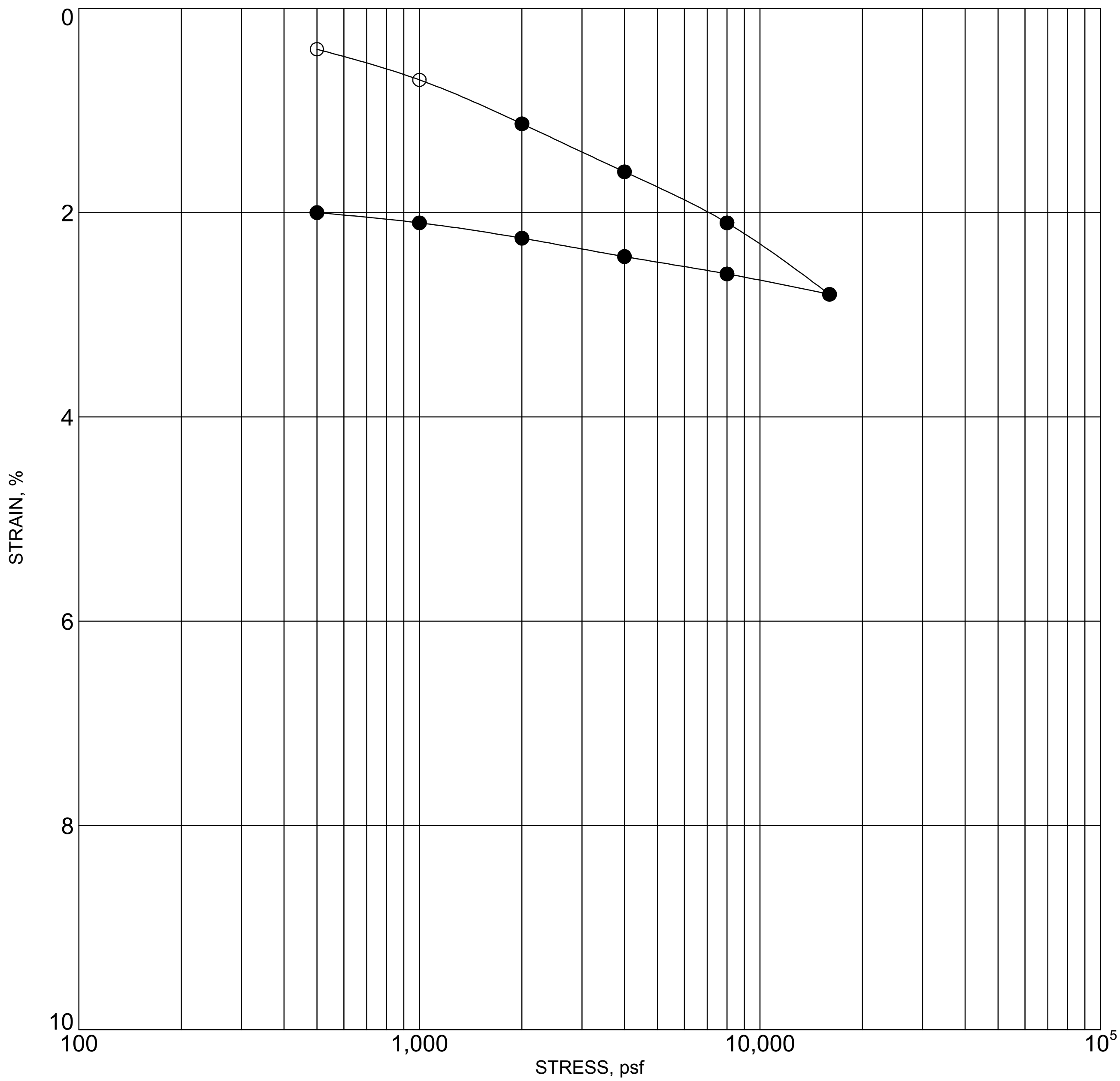
DIRECT SHEAR TEST RESULTS

Converse Consultants

Reservoir 3A, County Service Area 70, Zone 1
East of Columbine Road
Oak Hills Area, San Bernardino County, California
For: San Bernardino County Special Districts Division

Project No. **19-81-275-01**

Drawing No. **B-4**



BORING NO. :	TP-07	DEPTH (ft) :	3.0 to 4.0
DESCRIPTION :	Silty Sand (SM)		

	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	PERCENT SATURATION	VOID RATIO
INITIAL	6.6	118	44	0.396

NOTE: SOLID CIRCLES INDICATE READINGS AFTER ADDITION OF WATER

CONSOLIDATION TEST RESULTS

Converse Consultants

Reservoir 3A, County Service Area 70, Zone 1
East of Columbine Road
Oak Hills Area, San Bernardino County, California
For: San Bernardino County Special Districts Division

Project No. **19-81-275-01**

Drawing No. **B-5**

Appendix C

Site-Specific Ground Motion Study

APPENDIX C

SITE-SPECIFIC GROUND MOTION STUDY

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. C-1, CBC Mapped Seismic Design Parameters

Seismic Parameters	
Site Coordinates	34.3702N, 117.4340 W
Site Class	D
Risk Category	III
Mapped Short period (0.2-sec) Spectral Response Acceleration, S_s	1.500g
Mapped 1-second Spectral Response Acceleration, S_1	0.606g
Site Coefficient (from Table 11.4-1), F_a	1.0
Site Coefficient (from Table 11.4-2), F_v	1.7
MCE 0.2-sec period Spectral Response Acceleration, S_{MS}	1.500g
MCE 1-second period Spectral Response Acceleration, SM_1	1.030g
Design Spectral Response Acceleration for short period S_{DS}	1.000g
Design Spectral Response Acceleration for 1-second period, S_{D1}	0.687g
Mapped Risk coefficient at Short Period (0.2 Sec), C_{RS}	0.919
Mapped Risk coefficient at Period 1-second, C_{R1}	0.897
Long-period transition period in seconds, T_L	12
Site Modified Peak Ground Acceleration, PGA_M	0.715g

A site-specific response analysis, using faults within 200 kilometers of the site, 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). The scale factors provided in ASCE 7-16 Section 21.2 were utilized. An average shear wave velocity within the upper 30 meters of soil profile (Vs30) of 280 meters per second was utilized. Based on the site-specific analysis, seismic acceleration parameters are summarized in the table below.

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

Table No. C-2, 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)	0.93	0.88	1.07	0.72	0.93
Spectral Period (sec)	2% in 50yr Probabilistic Spectral Acceleration (g)				
0.05	1.06	0.88	1.29	0.90	1.04
0.10	1.50	1.13	2.03	1.25	1.38
0.20	2.03	2.03	2.28	1.45	2.06
0.30	2.22	2.34	2.26	1.75	2.37
0.40	2.26	2.38	2.18	2.01	2.43
0.50	2.21	2.17	2.18	2.05	2.40
0.75	1.83	1.59	1.78	1.83	2.05
1.00	1.48	1.27	1.41	1.62	1.60
2.00	0.87	0.72	0.81	1.10	0.72
3.00	0.61	0.48	0.59	0.84	0.42
4.00	0.46	0.36	0.47	0.63	0.26
5.00	0.37	0.30	0.40	0.49	0.17
7.50	0.21	0.21	0.25	0.23	0.08
10.0	0.13	0.14	0.16	0.12	0.05

Applicable response spectra data are presented in the table below and on Figure No. C-1, *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. C-3, Probabilistic MCE_R Spectral Acceleration (g)

Period (sec)	2% in 50 yr. Probabilistic Spectral Acceleration (g) Geometric Mean	Risk Coefficient C_R	Scale Factors for MCE_R	Probabilistic MCE_R Spectral Acceleration (g)
0.05	1.06	0.919	1.100	1.075
0.10	1.50	0.919	1.100	1.520
0.20	2.03	0.919	1.100	2.049
0.30	2.22	0.916	1.125	2.291
0.40	2.26	0.914	1.150	2.374
0.50	2.21	0.911	1.175	2.361
0.75	1.83	0.904	1.238	2.047
1.00	1.48	0.897	1.300	1.728
2.00	0.87	0.897	1.350	1.050
3.00	0.61	0.897	1.400	0.768
4.00	0.46	0.897	1.450	0.603
5.00	0.37	0.897	1.500	0.495
7.50	0.21	0.897	1.500	0.281
10.0	0.13	0.897	1.500	0.172

Deterministic response spectra parameters were determined using PEER spread sheet and presented in Table No. C-4. The following fault parameters were used to calculate the spectrum.

South San Andreas Fault, Mw–8.04, RRUP–10.4 km, RJB–10.4 km, Rx–10.4 km and dip angle are 86 degree.

Applicable response spectra data are presented in the table below and on Drawing No. C-1, 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.

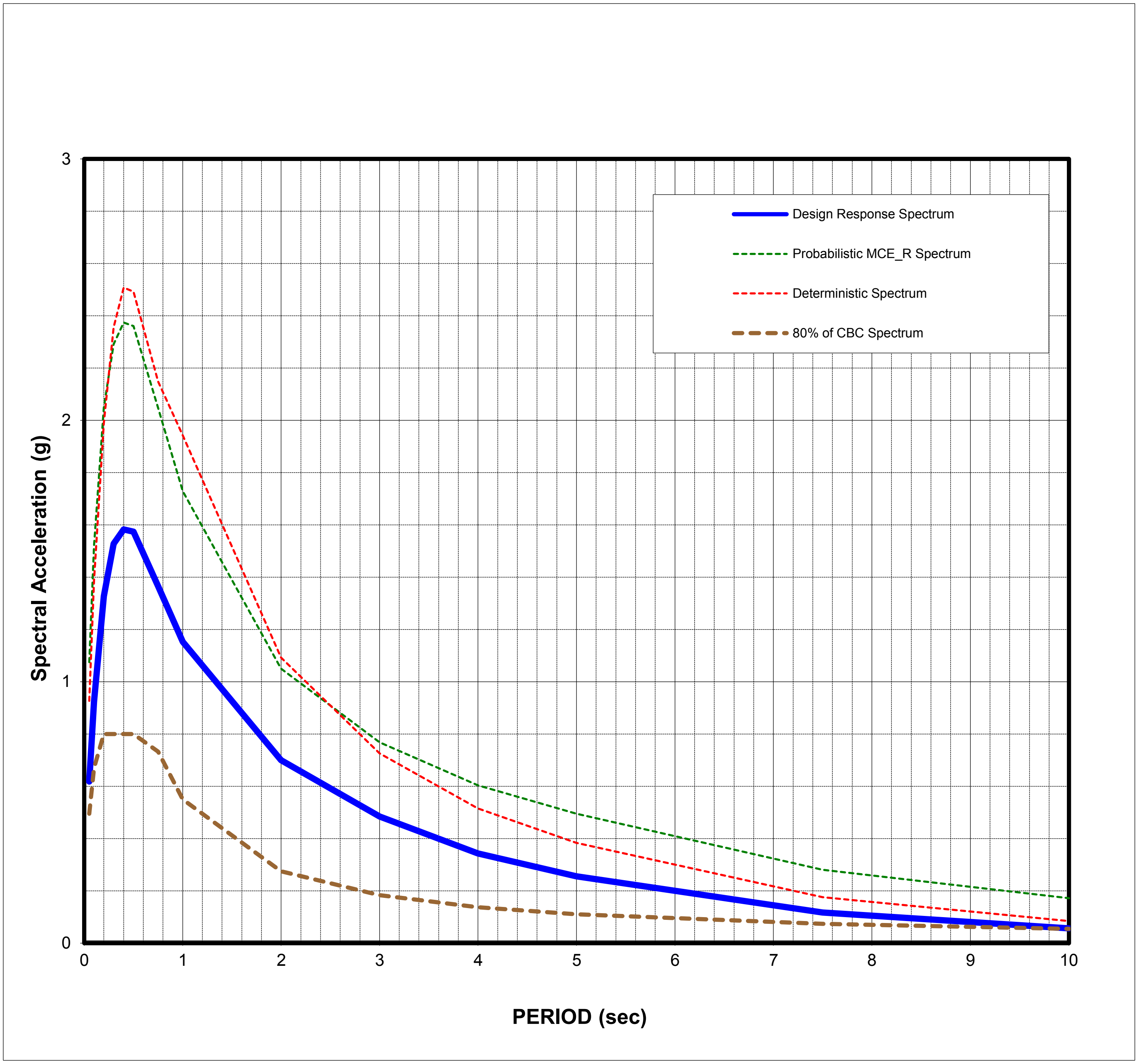
Table No. C-4, Site-Specific Response Spectrum Data

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	0.84	1.100	0.927	0.927	0.495	0.62
0.10	1.26	1.100	1.386	1.386	0.669	0.92
0.20	1.81	1.100	1.989	1.989	0.800	1.33
0.30	2.09	1.125	2.356	2.291	0.800	1.53
0.40	2.18	1.150	2.508	2.374	0.800	1.58
0.50	2.12	1.175	2.492	2.361	0.800	1.57
0.75	1.74	1.238	2.148	2.047	0.733	1.36
1.00	1.49	1.300	1.943	1.728	0.549	1.15
2.00	0.81	1.350	1.092	1.050	0.275	0.70
3.00	0.52	1.400	0.726	0.726	0.183	0.48
4.00	0.36	1.450	0.515	0.515	0.137	0.34
5.00	0.26	1.500	0.383	0.383	0.110	0.26
7.50	0.12	1.500	0.175	0.175	0.073	0.12
10.0	0.06	1.500	0.085	0.085	0.055	0.06

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 ASCE 7-16 Section 21.4 guidelines.

Table No. C-5, 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}	2.137	1.200
Site-Specific1-second period Spectral Response Acceleration, S_{M1}	2.177	0.824
Site-Specific Design Spectral Response Acceleration for short period S_{DS}	1.425	0.800
Site-Specific Design Spectral Response Acceleration for 1-second period, S_{D1}	1.451	0.549



Note: Calculated using EZFRISK program Risk Engineering, version 8.06 and NGAW2_GMPE Spreadsheets V5.7

SITE SPECIFIC DESIGN RESPONSE SPECTRUM

Project: Reservoir No. 3A, County Service Area 70, Zone 1
Location: East of Columbine Road
Oak Hills Area, San Bernardino County, California
For: San Bernardino County Special Districts Department

Project Number: 19-81-275-01

Converse Consultants

Drawing No. C-1

Appendix D

Slope Stability Analysis

APPENDIX D

SLOPE STABILITY ANALYSIS

The anticipated stability of the proposed slopes (embankment and cut slopes) under static conditions (dry, steady seepage, and rapid drawdown) were evaluated using the Slide 8.0 software (RocScience, 2020). Pseudostatic analyses using a seismic co-efficient of 0.2g were performed in order to evaluate the stability of the slopes during a large earthquake. Slope above access road was evaluated only for dry and pseudostatic conditions. These slopes were selected as a worst-case condition due to their heights, slope ratio, materials encountered, and possible impoundment of water. The purpose of the analyses was to evaluate the anticipated factors of safety against failure of the proposed slopes under a variety of conditions.

For all slope conditions, a Mohr-Coulomb soil strength model was assumed, and Factors of Safety (FOS) for slope stability were evaluated using different methods solution procedures: Bishop Simplified, Janbu Simplified, and Spencer.

The relevant soil parameters for the proposed slope including unit weight, friction angle, cohesion was derived from field and laboratory test data.

Grid searches within predefined areas were utilized to determine the critical slip surface in each case. Slip surface limits (entrance and exit zones) were implemented to avoid modeling surficial slope failures which have a marginally lower overall factor of safety compared to deeper seated slip surfaces, but which are less relevant to the slope design.

Limit equilibrium methods for evaluating slope stability consider the static equilibrium of a soil mass above a potential failure surface. For conventional, two-dimensional methods of analysis; the slide mass above an assumed failure surface is first divided into vertical slices, then stresses are evaluated along the sides and base of each slice. The factor of safety against a slope failure (FS_{slope}) is defined as:

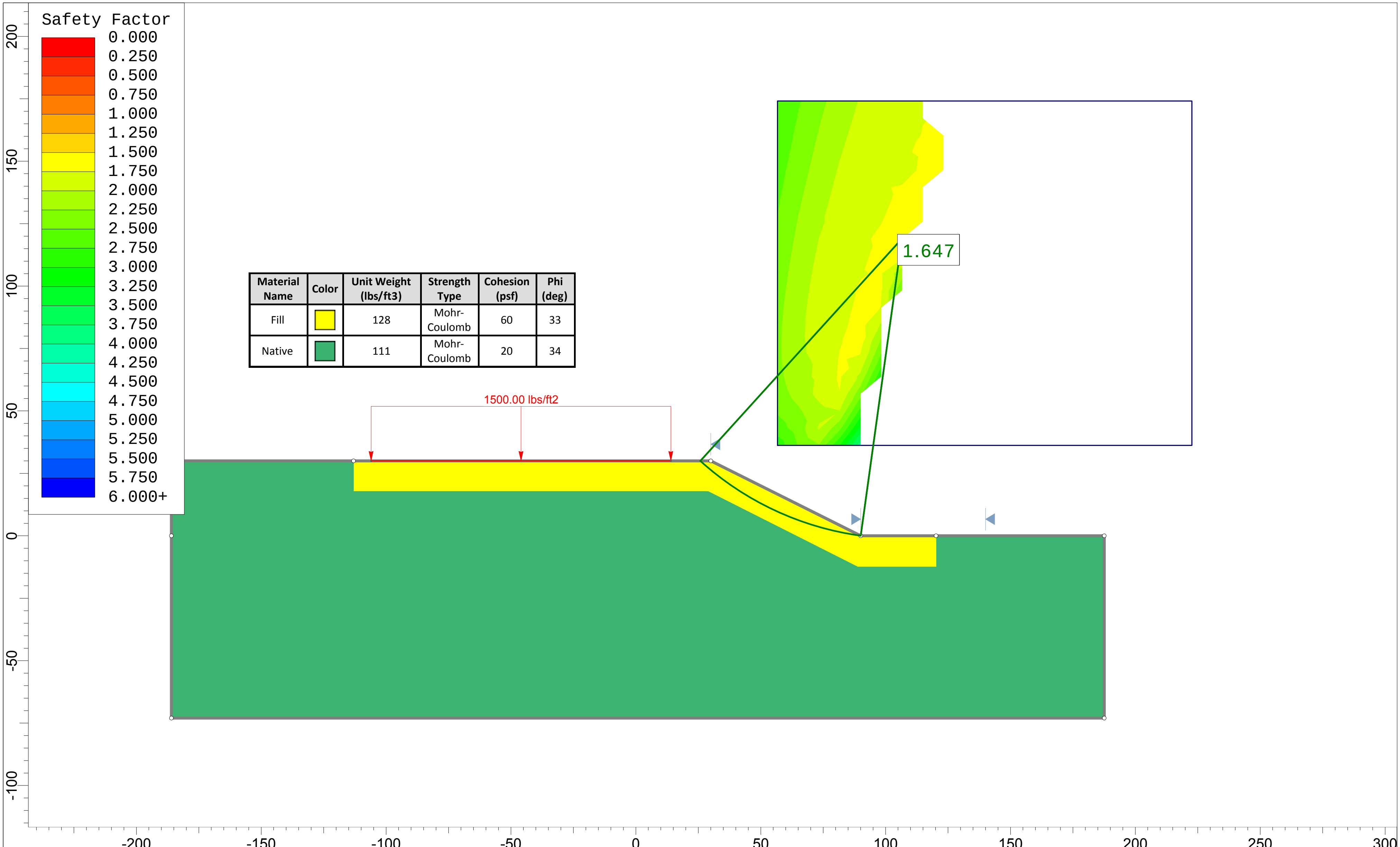
$$FS_{slope} = \frac{\text{shear strength of soil}}{\text{shear stress required for equilibrium}}$$

The strengths and stresses are computed along a defined failure surface located at the base of the vertical slices. The shearing resistance along the potential slip surface is computed, with appropriate Mohr-Coulomb strength parameters, as a function of the effective normal stress.

The following table and pages include figures presenting the results of the analyses.

Table No. D-1, Factors of Safety Against Slope Failure

Slope	Condition		Static FOS	Remarks
Fill Slope	Static	2H:1V	1.65>1.5	Stable
	Pseudo-Static		1.08≈1.1	Stable
Note: H = Horizontal, V = Vertical				



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Fill		128	Mohr-Coulomb	60	33
Native		111	Mohr-Coulomb	20	34

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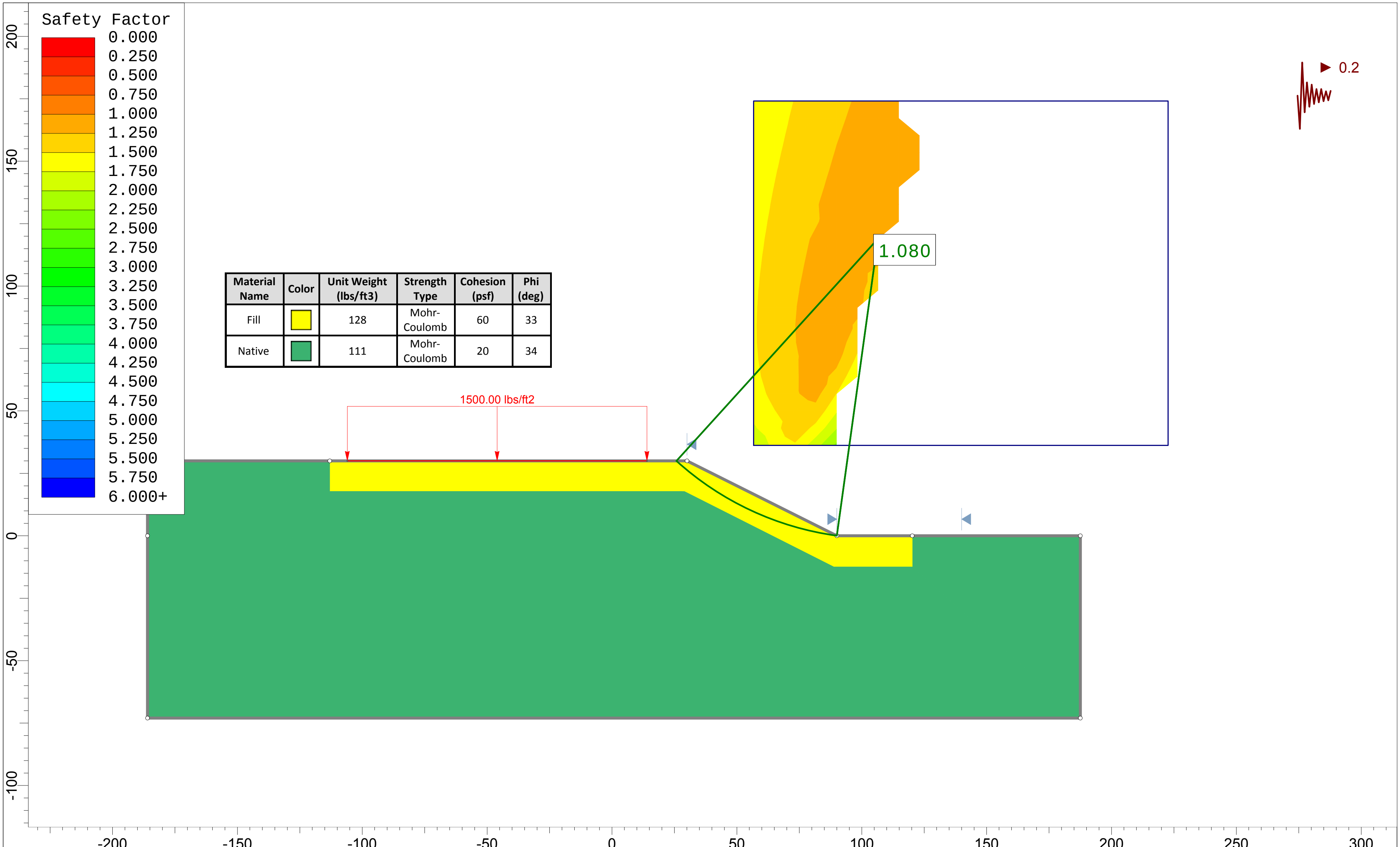
Project	Reservoir No. 3A, County Service Area 70, Zone 1		
Group	Fill Slope	*Scenario*	Static
Drawn By	Mahmoud Suliman	*Company*	Converse Consultants
Date	3/9/2021	*File Name*	Slide1.slmd



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Fill		128	Mohr-Coulomb	60	33
Native		111	Mohr-Coulomb	20	34

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SLIDEINTERPRET 9.008

Project	Reservoir No. 3A, County Service Area 70, Zone 1		
Group	Fill Slope	*Scenario*	Pseudo-Static
Drawn By	Mahmoud Suliman	*Company*	Converse Consultants
Date	3/9/2021	*File Name*	Slide1.slmd