GEOTECHNICAL INVESTIGATION REPORT

1.5 MG Irwin Road Reservoir

Irwin Road (34.936016N, 117027812W)
City of Barstow, San Bernardino County, California

Converse Project No. 20 81 128 01

Prepared For:

Golden State Water Company
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May 12, 2020

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

May 12, 2020

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Subject: **GEOTECHNICAL INVESTIGATION REPORT**
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City of Barstow, San Bernardino County, California
Converse Project No. 20-81-128-01

Dear Mr. Mim Mack:

Converse Consultants (Converse) is pleased to submit this Geotechnical Investigation Report for the 1.5 MG Irwin Road Reservoir Project, located in the City of Barstow, San Bernardino County, California. This report was prepared in accordance with our proposal dated March 2, 2020 and your Purchase Order number 7013396 - SP from Golden State Water Company dated March 10, 2020.

We appreciate the opportunity to be of continued service to Golden State Water Company (GSWC). 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/Regional Manager

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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 reservoir site and the subsequent design recommendations for a 1.5 MG water reservoir located on Irwin Road in the City of Barstow, San Bernardino County, California. The approximate location of the proposed project site is shown in Figure No. 1, *Approximate Project Location Map.*

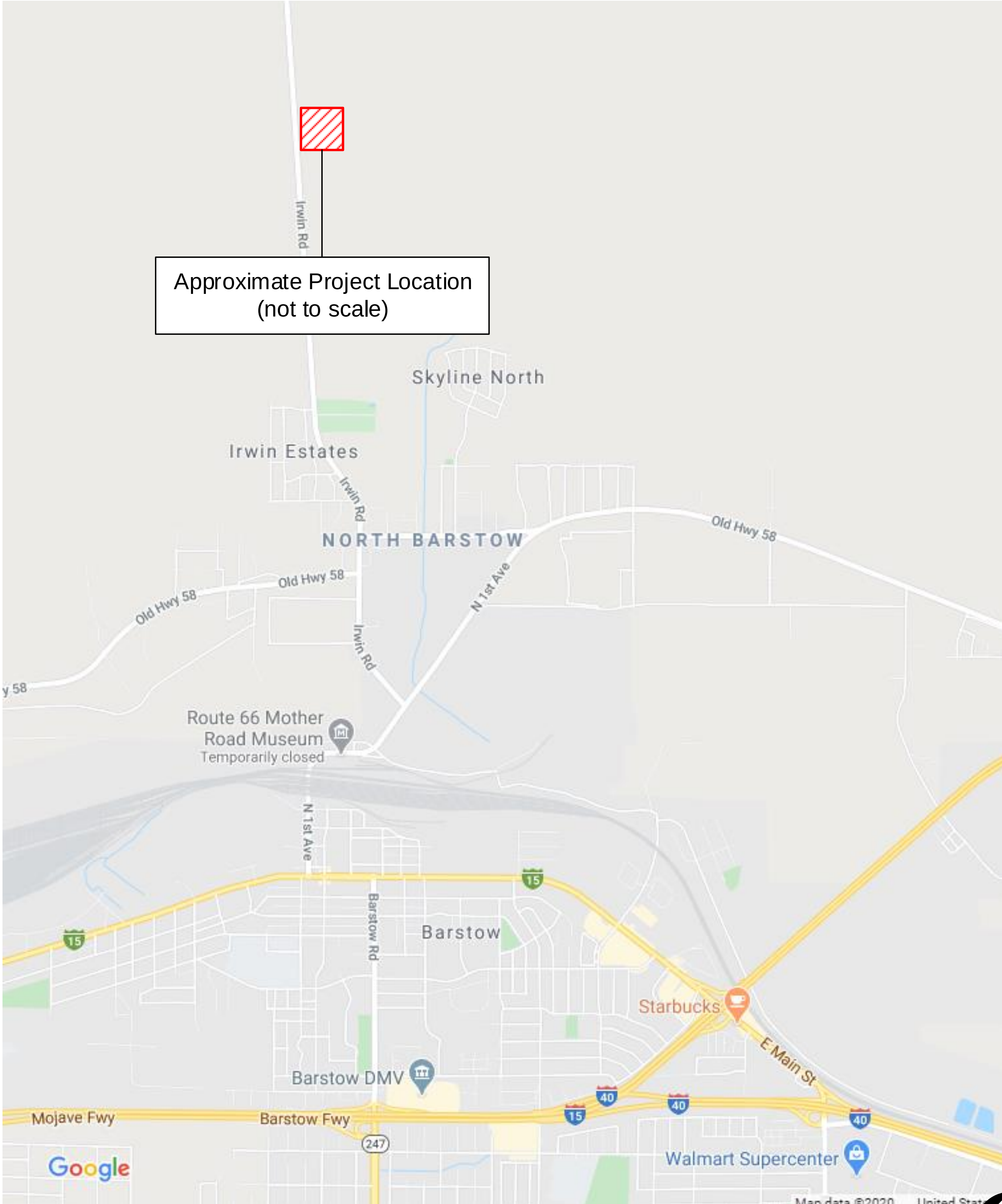
Currently, the site of the proposed reservoir is vacant. Golden State Water Company intends to construct a new reservoir at this location. The purposes of this investigation were to determine the nature and engineering properties of the subsurface soils, and to provide earthwork, design and construction recommendations.

This report is prepared for the project described herein and is intended for use solely by Golden State Water Company 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 proposed 1.5 MG water reservoir will be constructed adjacent to the east side of Irwin Road, at the northeast corner of section 19 T10N R1W, approximately 0.9 miles north of Spadra Street in the City of Barstow, San Bernardino County, California. The new reservoir will be constructed on a vacant land. Based on information provided through email by Mr. Tim Mim Mack with Golden State Water Company, and the Reservoir Grading and Paving Plans (GSWC, 2020) prepared by Golden State Water Company, the project will include the following.

- An above ground steel potable water reservoir 102.0 feet in diameter by 35.5 feet in height. We understand that the water reservoir will be founded on a continuous footing (ring foundation) and, if required, the roof supported on columns resting on isolated spread footings. The reservoir bottom will consist layers of compacted oil sand with 6-8 lbs of SC-800 liquid asphalt, 3/8” drain rock, impermeable membrane and sand (SE 30).
- Grading up to approximately 6 feet of cut into existing soil and up to approximately 5 feet of fill. In addition, approximately maximum 7-foot-high 2H:1V (horizontal:vertical) cut slopes and approximately maximum 5-foot-high 2H:1V (horizontal:vertical) fill slopes are proposed.
- A 15 feet wide asphalt paved access road without shoulders will be constructed from Irwin road through the western edge of the project site. A 20 feet wide asphalt paved ring road will be constructed surrounding the reservoir.
- On-site pipelines, appurtenances, drainage system components, entrance gate and perimeter fence. A total of 5 catch basins are proposed.



Approximate Project Location Map

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Project: 1.5 MG Irwin Road Reservoir
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FIGURE NO.
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- A 175,500-gallon retention basin is proposed along the southern project site boundary.

3.0 SITE DESCRIPTION

The site is currently vacant and rugged terrain with sparse to moderate vegetation consisting of grasses and bushes. Gravel, cobbles and free scattered boulders cover the remaining surface of the site. It is bounded to the west by Irwin Road and to the north, east, and south by vacant open terrain with small shrubs. Overhead electricity lines traverse in a north/south 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 south from between approximately 2,344 feet above mean sea level (amsl) to approximately 2,335 feet amsl. Present site conditions are shown in the photographs no. 1 through 3.

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

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

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

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 Golden State Water Company, Google Earth aerial photographs, groundwater data, 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.

- Conducted a field reconnaissance and marked the boring locations such that drill rig access to all locations was available.
- Notified Underground Service Alert (USA) at least 48 hours prior to drilling to clear the boring locations of any conflict with existing underground utilities.
- Engaged a California-licensed drill rig to conduct drilling of borings.

4.3 Subsurface Exploration

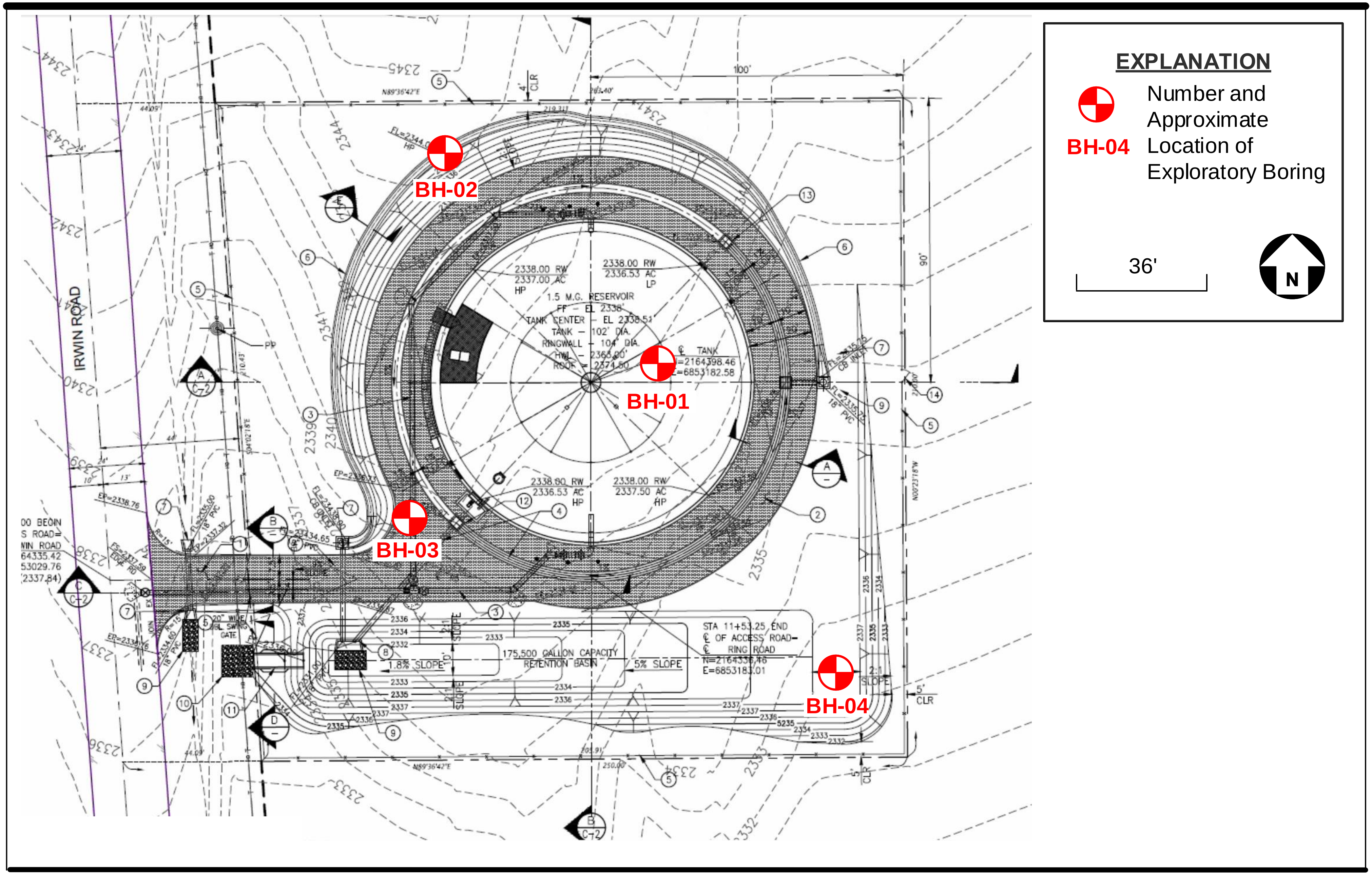
Four exploratory borings (BH-01 through BH-04) were drilled on March 26, 2020 to investigate subsurface conditions at the proposed reservoir site. The borings were drilled to the depths ranging from 17.3 feet to 50.8 feet below existing ground surface (bgs).

The boring locations are presented in Figure No. 2, *Approximate Boring 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)
- Collapse potential (ASTM D4546)
- Grain size analysis (ASTM D6913)



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Approximate Boring Locations Map

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Figure No.
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- Maximum dry density and optimum-moisture content (ASTM D1557)
- Direct shear (ASTM D3080)

For *in situ* moisture data, see the Log of Borings 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.

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 borings and laboratory test results, the subsurface soil at the site consists primarily of a mixture of sand, silt and gravel. Some gravel up to 3 inches in largest dimension was encountered in all the borings. Due to the friable and dry nature of the soil as well as the high gravel content some of the in-place samples were disturbed or difficult to collect. Though not encountered, larger materials (>3 inches) may be present within the project site.

For a detailed description of the subsurface materials encountered in the exploratory borings, see Drawings No. A-2 through A-5, *Logs of Borings,* in Appendix A, *Field Exploration.*

5.2 Groundwater

Groundwater was not encountered during the investigation to the maximum explored depth of 50.8 feet bgs. The GeoTracker and National Water Information System databases (SWRCB, 2020; USGS, 2020) were reviewed for groundwater data from sites within a one-mile radius of the project site and no data was available.

Numerous sites with groundwater data were identified to the south of the project site, however they are located below the hills in and around the alluvial valleys of the Mojave River and are not representative of the subsurface conditions at the site. For comparison, data from two of the closest sites was recorded and is presented in the table below.

Table No. 1, Summary of USGS Groundwater Depth Data

Alignment No.	Location	Groundwater Depth Range (ft. bgs)	Date Range
345521117012501	5500 ft. South of Proposed Reservoir Site	109.97	1959
3545520117012501	5600 ft. South of Proposed Reservoir Site	101.34	1959

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 50.8 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. Difficult excavation will occur where high concentration of gravel and larger materials (> 3 inches) are encountered.

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 boring locations.

5.5 Flooding

Review of National Flood Insurance Rate Maps (FEMA, 2020) 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, 2020a) 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 1.5 MG Irwin Rd Reservoir Site 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 Site Geology

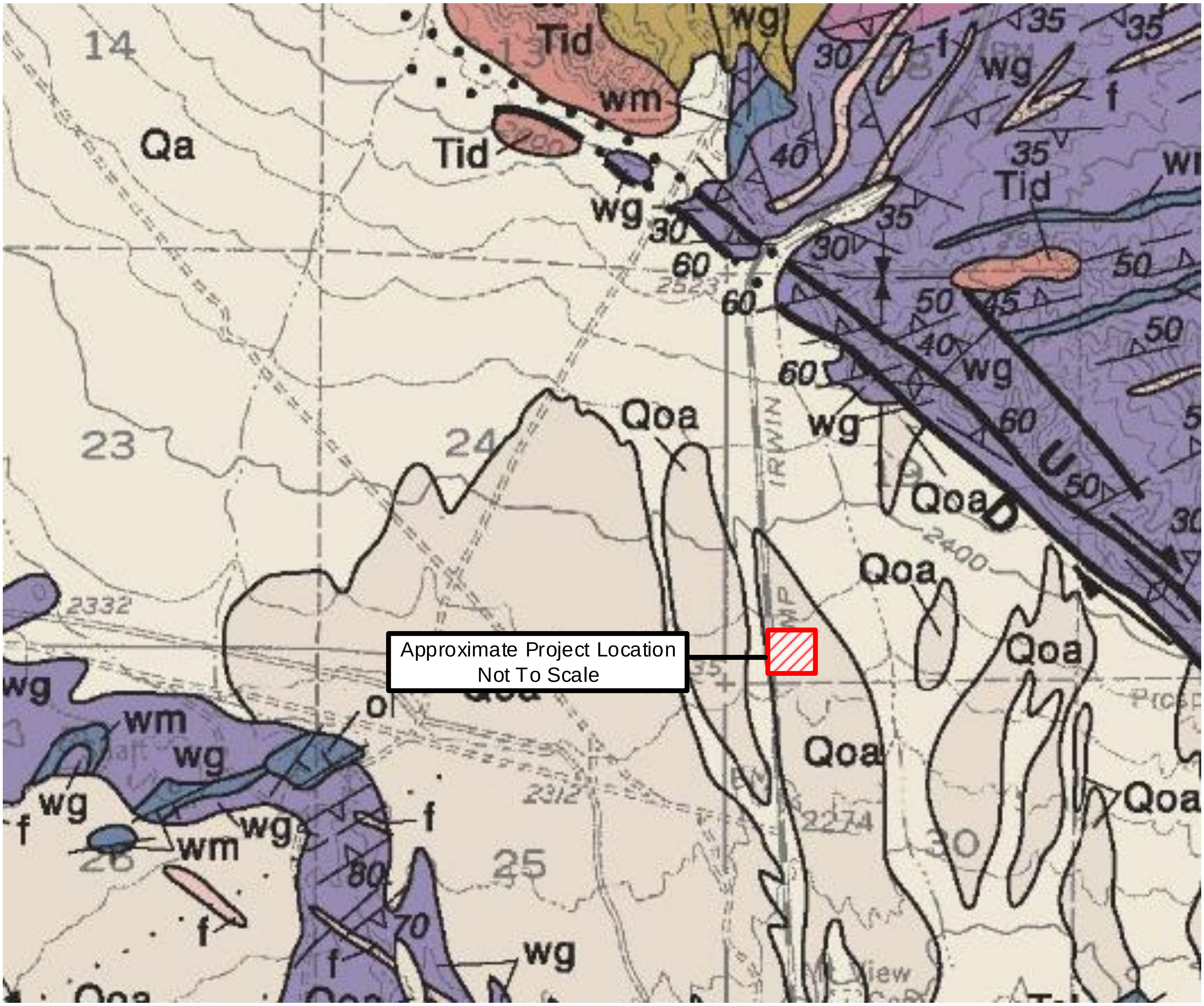
Review of the latest regional mapping (Dibblee, 2008) indicates that the subsurface material below the proposed reservoir is mostly Pleistocene-aged older alluvial deposit (Qoa) defined by loose to moderately consolidated, gray, silt, sand and gravel that is poorly bedded and undeformed. Due to the nature of the soils and topography in this area, it is possible that some of the near surface soils may consist of a significant amount of alluvial sand. A detailed view of the above referenced geologic map of the proposed project site is shown in Figure No. 3, *Geologic Reference Map.*

7.0 FAULTING AND SEISMICITY

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

7.1 Faulting

The proposed reservoir 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



LEGEND

Qoa	**Qoa** *Older alluvial gravel, sand, and silt, light gray, poorly bedded, undeformed*

N

Geologic Reference Map

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Figure No.
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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 34.9360158°N latitude and -117.0278122°W longitude, the National Seismic Hazard Maps Database (USGS, 2008) and other published geologic data.

Table No. 2, Summary of Regional Faults

Fault Name and Section	Closest Distance (km)	Slip Sense	Length (km)	Slip Rate (mm/year)	Maximum Magnitude
Gravel Hills-Harper Lk	1.59	strike slip	65	0.7	6.5
Blackwater	8.73	strike slip	60	0.5	6.5
Lenwood-Lockhart-Old Woman Springs	8.91	strike slip	145	0.9	6.5
Calico-Hidalgo	11.24	strike slip	117	1.8	6.5
Landers	12.34	strike slip	95	0.6	6.5
Helendale-So Lockhart	31.16	strike slip	114	0.6	6.5

(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 34.936016N latitude and -117.027812W 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. 3, CBC Mapped Seismic Design Parameters

Seismic Parameters	Values
Site Coordinates	34.936016N 117.027812W
Risk Category	III
Site Class	D
Mapped Short period (0.2-sec) Spectral Response Acceleration, S_s	1.491g
Mapped 1-second Spectral Response Acceleration, S_1	0.508g
Site Coefficient (Table 11.4-1), F_a	1.0

Table No. 3, CBC Mapped Seismic Design Parameters (Continued)

Seismic Parameters	Values
Site Coefficient (Table 11.4-2), F_v	1.792
MCE 0.2-sec period Spectral Response Acceleration, S_{Ms}	1.491g
MCE 1-second period Spectral Response Acceleration, S_{M1}	0.910g
Design Spectral Response Acceleration for short period, S_{Ds}	0.994g
Design Spectral Response Acceleration for 1-second period, S_{D1}	0.607g
Site-Modified Maximum Peak Ground Acceleration, PGA_M	0.723g

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.

Table No. 4, Recommended Site-Specific Seismic Acceleration Parameters

Seismic Parameters	Values
MCE 0.2-sec period Spectral Response Acceleration, S_{Ms}	1.377g
MCE 1-second period Spectral Response Acceleration, S_{M1}	1.054g
Design Spectral Response Acceleration for short period, S_{Ds}	0.918g
Design Spectral Response Acceleration for 1-second period, S_{D1}	0.703g
Site-Specific Maximum Peak Ground Acceleration, MCE_G PGA_M	0.850g

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, 2020b). 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 located within an area not designated as a liquefaction risk by the State of California and San Bernardino County (CGS, 2007; SBC, 2020b). Based on the absence of shallow groundwater and site-specific analysis presented in Appendix D, *Liquefaction and Settlement Analyses*, the potential of liquefaction induced settlement is considered negligible.

Seismic Settlement: Seismically induced settlement occurs in loose, granular sediments during ground shaking associated with earthquakes. Based on the high blow counts and site-specific analysis presented in Appendix D, *Liquefaction and Settlement Analyses*, the potential of dry seismic settlement is up to 0.11 inches.

Landslides: Seismically induced landslides and other slope failures are common occurrences during or soon after earthquakes. The reservoir site is located within a designated State of California or San Bernardino County landslide hazard zone (CGS, 2007; SBC, 2020b). 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, 2019 and SBC, 2020a). 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.

Seismic Slope Stability: The proposed maximum 7-foot high cut slope ascending away from the proposed reservoir pad to the north and the 5-foot high fill slope southeast of the proposed reservoir pad are expected to be constructed with a ratio of approximately 2H:1V (horizontal:vertical) or flatter. The project site and surrounding proposed cut and fill slopes are expected to be stable during seismic events.

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 on the Logs of Borings 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 10 feet soils ranged from 113 to 118 pounds per cubic foot (pcf) with moisture contents ranging from 1 to 3 percent. Results are presented in the log of borings in Appendix A, *Field Exploration.*
- Expansion Index – One representative sample from the upper 4 feet of soils in boring BH-01 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.
- Collapse Potential – The collapse potential of three relatively undisturbed samples were tested under a vertical stress of up to 2.0 kips per square foot (ksf) in accordance with the ASTM Standard D4546 test method. The test results showed collapse potential of between 1.5 and 6.3 percent, indicating slight to moderate collapse potential.
- 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 test was performed on two 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 131.0 and 132.0 (with rock correction 135.4 and 136.3) pcf and moisture content of 7.0 (with rock correction of 6.0 percent) percent.
- Direct Shear – Two direct shear tests were performed on samples remolded to 90 percent of the maximum dry 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*.

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 8.4 and 8.6.
- The sulfate contents of the samples tested were 0.0035 and 0.0121 percent by weight (35 and 121 ppm).
- The chloride concentrations of the samples tested were 57 and 99 ppm.
- The minimum electrical resistivities of the samples when saturated were 1,575 and 2,706 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

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, concrete, 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/Removal

Approximately 2 feet to 5 feet of the upper older alluvium may be prone to future settlement under the surcharge of reservoir foundations, improvements, and/or fill loads. These materials should be overexcavated to competent older alluvium and replaced with compacted fill soils.

Reservoir Foundation Areas
In order to provide uniform support and mitigate potential adverse differential settlement, overexcavations within the entire reservoir areas should extend at least 5 feet below existing grade or at least 2 feet below the lowest depth of proposed footings, whichever is deeper. The overexcavation should be uniform and should also extend to at least 5 feet beyond the footprint of the ring footings.

Pavement and other Improvements Areas
Overexcavations within proposed pavement areas or other improvements outside of the proposed reservoir foundation can be limited to 2 feet below the existing ground surface.
Localized, deeper overexcavation could be required where recommended by the geotechnical consultant based on observations during grading.

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 utilities.

9.3 Backfill Materials

No fill or aggregate base should be placed until excavations and/or natural ground preparation have been observed by the geotechnical consultant. Backfill materials should be based on the latest edition of GSWC Standard Drawings and Specifications. 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.

9.4 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.

9.5 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 GSWC Standards or County of San Bernardino requirements whichever is more stringent.

9.6 Foundation Type and Bearing Pressure

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 3,000 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 4,000 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. 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.

9.7 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.

9.7.1 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. 5, 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)	36
At-rest (wall is restrained)	56

These pressures assume a level ground surface around the reservoir structure for a distance greater than the reservoir height with 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.

9.7.2 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 3,000 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.

9.8 Permanent Cut Slopes

Cut slopes approximately 7 feet high are proposed on the north and northwest and northeast of 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 older alluvium, 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. This could be achieved with a 10 foot wide and 2 foot deep keyway at the toe of the proposed slope, with a backcut no steeper than a 1.5:1 (H:V) slope ratio. We anticipate properly constructed cut slopes will be stable.

9.9 Permanent Fill Slopes

Fill slopes approximately height of 5 feet are proposed on the south and southwest 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 9.4, *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. We anticipate properly constructed fill slopes will be stable.

9.10 Catch Basin

Catch basin walls and slab should be design based on net allowable bearing capacity presented in Section 9.6, *Foundation Type and Bearing Pressure* and lateral earth pressures presented in Section 9.7, *Lateral Earth Pressures and Resistance to Lateral Loads*. Modulus of subgrade can be taken as 200 kcf for slab design.

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 feet of soils is estimated. An average value of 8 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

The total settlement of shallow footings from static structural loads and short-term settlement of properly compacted fill is anticipated to be 1 inch or less. The differential settlement resulting from static loads is anticipated to be 0.5 inches or less over a horizontal distance of 40 feet.

Based on the absence of shallow groundwater and high blow counts, and site-specific analysis presented in Appendix D, *Liquefaction and Settlement Analyses*, the site has up to 0.11 inches of dry seismic settlement with negligible amount of liquefaction induced settlement. The dynamic differential settlement can be taken as half of the total seismic settlement.

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 was 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 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.

The measured value of the minimum electrical resistivity of the samples when saturated were 1,575 and 2,706 ohm-cm. This indicates that the samples tested were corrosive to severely 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.

9.14 Asphalt Concrete Pavement

Paving should be conducted as per GSWC Standards or the County of San Bernardino requirements, whichever is more stringent. Based on the grading and paving plans, the structural section of pavement at the site is 3 inches of asphalt concrete over 6 inches of aggregate base.

Additionally, several other pavement sections are presented, if needed. Based on soil type and experience on similar type of projects, pavement design was based on an assumed R-value of 40 and design Traffic Indices (TIs) ranging from 5 to 8.

Based on the above information, Converse has performed an analysis to evaluate the required pavement section thickness. Asphalt concrete and aggregate base thickness results are presented using the *Caltrans Highway Design Manual (Caltrans, 2020)*, Chapter 630 with a safety factor of 0.2 for Asphalt concrete/aggregate base section and 0.1 for full depth asphalt concrete section. Preliminary asphalt concrete pavement sections are presented in the following table.

Table No. 6, Recommended Preliminary Pavement Sections

R-value	Traffic Index (TI)	Pavement Section		
		Option 1		Option 2
		Asphalt Concrete	Aggregate Base	Full AC Section
	5	3.0	4.0	5.0
	6	4.0	4.5	6.5
	7	5.0	5.5	8.0
	8	5.0	8.0	10.0

At or near the completion of grading, subsurface samples should be tested to evaluate the actual subgrade R-value for final pavement design.

Prior to placement of aggregate base, at least the upper 2 feet of subgrade soils should be scarified, moisture-conditioned if necessary, and recompacted to at least 95 percent of the laboratory maximum dry density as defined by ASTM Standard D1557 test method.

Base and asphaltic concrete materials should conform with the Standards of GSWC or the County of san Bernardino.

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 the Golden State Water Company 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

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

Four exploratory borings (BH-01 through BH-04) were drilled on March 26, 2020 to investigate subsurface conditions at the proposed reservoir site. The borings were drilled to the depths ranging from 17.3 feet to 50.8 feet below existing ground surface (bgs).

The borings were advanced using a truck-mounted drill rig equipped with 8-inch diameter hollow-stem augers for soils sampling. 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 into the bottom of the borehole with successive drops of a 140-pound driving weight falling 30 inches. Blow counts at each sample interval are presented on the boring logs. 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. Bulk samples of typical soil types were also obtained. Numerous ring samples collected from each borehole were disturbed or contained no soil recovery as a result of the poor consolidation and large grain sizes

Standard Penetration Testing (SPT) was also performed in the 50 feet and 30 feet deep borings in accordance with the ASTM Standard D1586 test method at 10-foot intervals beginning at 20 feet bgs using a standard (1.4 inches inside diameter and 2.0 inches outside diameter) split-barrel sampler. The mechanically driven hammer for the SPT sampler was 140 pounds, falling 30 inches for each blow. The recorded blow counts for every 6 inches for a total of 1.5 feet of sampler penetration are shown on the Logs of Borings.

Following the completion of logging and sampling, all borings were backfilled with soil cuttings and compacted by pushing down with augers using the drill rig weight. If construction is delayed, the surface may settle over time. So, we recommend the owner monitor the boring locations and backfill any depressions that might occur or provide protection around the boring locations to prevent trip and fall injuries from occurring near the area of any potential settlement.

For a key to soil symbols and terminology used in the boing logs, refer to Drawing No. A-1, *Unified Soil Classification and Key to Boring Symbols*. Borings are presented in Drawings No. A-2 through A-5.

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 BORING LOG SYMBOLS

Converse Consultants

1.5 MG Irwin Road Reservoir
Irwin Road (34.936016N, 117.027812W)
City of Barstow, San Bernardino County, California
For: Golden State Water Company

Project No. 20-81-128-01

Drawing No. A-1

Log of Boring No. BH-01

Dates Drilled: 3/26/2020 Logged by: Catherine Nelson Checked By: Robert Gregorek

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

Ground Surface Elevation (ft): 2340 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 AND GRAVEL (SP-SM):** fine to coarse-grained, some gravel up to 3" in largest dimension, reddish-brown.			12/19/28	1	117	ei, ca, er, ma, max
								ds
5					37/50-6"			No Recovery
					33/50-4"			No Recovery
10		- gray			12/50-3"	2	118	
15		- reddish brown			50-4"	4	104	dist.
20					13/27/27	2		
25					50-4"			No Recovery
30					50-6"	3		

Log of Boring No. BH-01

Dates Drilled: 3/26/2020 | Logged by: Catherine Nelson | Checked By: Robert Gregorek

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

Ground Surface Elevation (ft): 2340 | 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 AND GRAVEL (SP-SM):** fine to coarse-grained, some gravel up to 3" in largest dimension, gray.			50-3"	4	107	dist.
40					50-6"	3		
45					50.3"	3	107	dist.
50					41/50-3"	2		
		End of boring at 50.8 feet bgs. No groundwater encountered. Borehole backfilled with soil cuttings and compacted by pushing down with augers using drill rig weight on 3/26/2020.						

Converse Consultants

1.5 MG Irwin Road Reservoir
Irwin Road (34.936016N, 117.027812W)
City of Barstow, San Bernardino County, California
For: Golden State Water Company

Project No. **20-81-128-01**

Drawing No. **A-2b**

Log of Boring No. BH-02

Dates Drilled: 3/26/2020 Logged by: Catherine Nelson Checked By: Robert Gregorek

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

Ground Surface Elevation (ft): 2344 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		**ALLUVIUM** **SAND WITH SILT AND GRAVEL (SP-SM):** fine to coarse-grained, some gravel up to 3" in largest dimension, reddish-brown to gray						
					15/22/30	2	114	
5					27/35/36	1	117	col
					23/50-4"	3	104	dist.
10		- gray			50-3"	3	106	dist.
15					50-6"	2		No Recovery
20					21/40/37			
25					50-5"	2	113	
30					12/20/50-4"			No Recovery

1.5 MG Irwin Road Reservoir
Irwin Road (34.936016N, 117.027812W)
City of Barstow, San Bernardino County, California
For: Golden State Water Company

Converse Consultants

Project No. 20-81-128-01

Drawing No. A-3a

Project ID: 20-81-128-01.GPJ; Template: LOG

Log of Boring No. BH-02

Dates Drilled: 3/26/2020 Logged by: Catherine Nelson Checked By: Robert Gregorek

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

Ground Surface Elevation (ft): 2344 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
		End of boring at 32.3 feet bgs. No groundwater encountered. Borehole backfilled with soil cuttings and compacted by pushing down with augers using drill rig weight on 3/26/2020.						

Converse Consultants

1.5 MG Irwin Road Reservoir
Irwin Road (34.936016N, 117.027812W)
City of Barstow, San Bernardino County, California
For: Golden State Water Company

Project No. 20-81-128-01

Drawing No. A-3b

Log of Boring No. BH-03

Dates Drilled: 3/26/2020 Logged by: Catherine Nelson Checked By: Robert Gregorek

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

Ground Surface Elevation (ft): 2338 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 AND GRAVEL (SP-SM):** fine to coarse-grained, some gravel up to 3" in largest dimension, reddish-brown.						
					15/16/18	1	113	
		- light gray						
5					10/9/26	2	115	col, ds ca,er,ma, max
					50-5"			No Recovery
10					50-5"			No Recovery
					22/50-6"	2	111	
15					50-5"			No Recovery
20					22/38/50-3"	2		
25					50-6"			No Recovery
30					21/50-5"	2		
		End of boring at 30.9 feet bgs. No groundwater encountered. Borehole backfilled with soil cuttings and compacted by pushing down with augers using drill rig weight on 3/26/20						

Converse Consultants

1.5 MG Irwin Road Reservoir
Irwin Road (34.936016N, 117.027812W)
City of Barstow, San Bernardino County, California
For: Golden State Water Company

Project No. 20-81-128-01

Drawing No. A-4

Log of Boring No. BH-04

Dates Drilled: 3/26/2020 Logged by: Catherine Nelson Checked By: Robert Gregorek

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

Ground Surface Elevation (ft): 2332 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 AND GRAVEL (SP-SM):** fine to coarse-grained, some gravel up to 3" in largest dimension, reddish-brown.			24/27/50-6"	2	114	
5		- light gray			38/50-3"	3	116	
					50-6"	3	109	dist.
10					50-6"			No Recovery
		- reddish brown			50-6"	3	112	col
15					18/40/50-4"	3	107	dist.
		End of boring at 17.3 feet bgs. No groundwater encountered. Borehole backfilled with soil cuttings and compacted by pushing down with augers using drill rig weight on 3/26/2020.						

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 borings, in Appendix A, *Field Exploration*. The following is a summary of the various laboratory tests conducted for this project.

Moisture Content

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 Borings 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

Boring No.	Depth (feet)	Soil Description	Expansion Index	Expansion Potential
BH-01	0-4	Sand with Silt and Gravel (SP-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

Boring No.	Depth (feet)	pH	Soluble Sulfates (CA 417) (% by weight)	Soluble Chlorides (CA 422) (ppm)	Min. Resistivity (CA 643) (Ohm-cm)
BH-01	0-4	8.6	0.0035	57	2,706
BH-03	4-10	8.4	0.0121	99	1,575

Collapse

To evaluate the moisture sensitivity (collapse/swell potential) of the encountered soils, three collapse tests were performed in accordance with the ASTM Standard D4546 laboratory procedure. The samples were loaded to approximately 2 kips per square foot (ksf), allowed to stabilize under load, and then submerged. The test results are presented in the following table.

Table No. B-3, Collapse Test Results

Boring No.	Depth (feet)	Soil Classification	Percent Swell (+) Percent Collapse (-)	Collapse Potential
BH-02	5.0-6.5	Sand with Silt and Gravel (SP-SM)	-1.7	Slight
BH-03	4.0-5.5	Sand with Silt and Gravel (SP-SM)	-6.3	Moderate
BH-04	13.0-13.5	Sand with Silt and Gravel (SP-SM)	-1.5	Slight

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-4, Grain Size Distribution Test Results

Boring No./ Report	Depth (ft)	Soil Classification	% Gravel	% Sand	%Silt	%Clay
BH-01	0-4	Sand with Silt and Gravel (SP-SM)	30.0	61.2	8.8	
BH-03	4-10	Sand with Silt and Gravel (SP-SM)	32.0	61.0	7.0	

Laboratory Maximum Dry Density and Optimum Moisture Content

Two 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-5, Summary of Moisture-Density Relationship Results

Boring No.	Depth (feet)	Soil Description	Maximum Dry Density (pcf)	Optimum Moisture (%)
BH-01	0-4	Sand with Silt and Gravel (SP-SM)	131.0 (135.4*)	7.0 (6.0*)
BH-03	4-10	Sand with Silt and Gravel (SP-SM)	132.0 (136.3*)	7.0 (6.0*)

(* Rock correction: BH-01 = 14.44% and BH-03 = 14.70%)

Direct Shear

Two direct shear tests were performed on samples remolded to 90 percent of the laboratory maximum dry density under soaked moisture condition in accordance with ASTM D3080 method. 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-6, Direct Shear Test Results

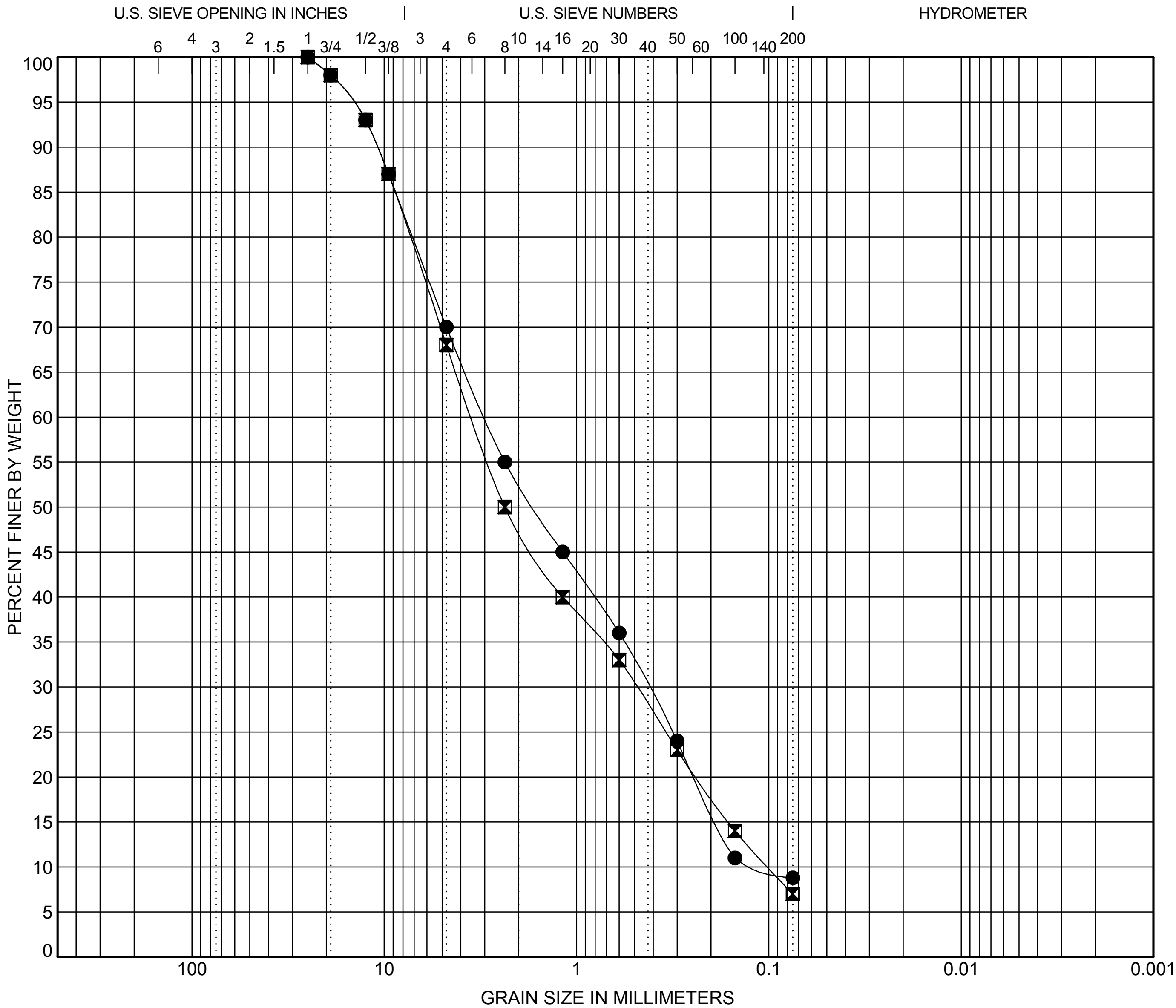
Boring No.	Depth (feet)	Soil Description	Ultimate Strength Parameters	
			Friction Angle (degrees)	Cohesion (psf)
*BH-01	0-4	Sand with Silt and Gravel (SP-SM)	38	40
+BH-03	4.0-5.5	Sand with Silt and Gravel (SP-SM)	35	20

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

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

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
●	**BH-01**	**0-4**	**SAND WITH SILT AND GRAVEL (SP-SM)**				**0.55**	**27.22**
⊠	**BH-03**	**4-10**	**SAND WITH SILT AND GRAVEL (SP-SM)**				**0.68**	**34.48**

	Boring No.	Depth (ft)	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**BH-01**	**0-4**	**25**	**2.98**	**0.424**	**0.109**	**30.0**	**61.2**	**8.8**	
⊠	**BH-03**	**4-10**	**25**	**3.481**	**0.487**	**0.101**	**32.0**	**61.0**	**7.0**	

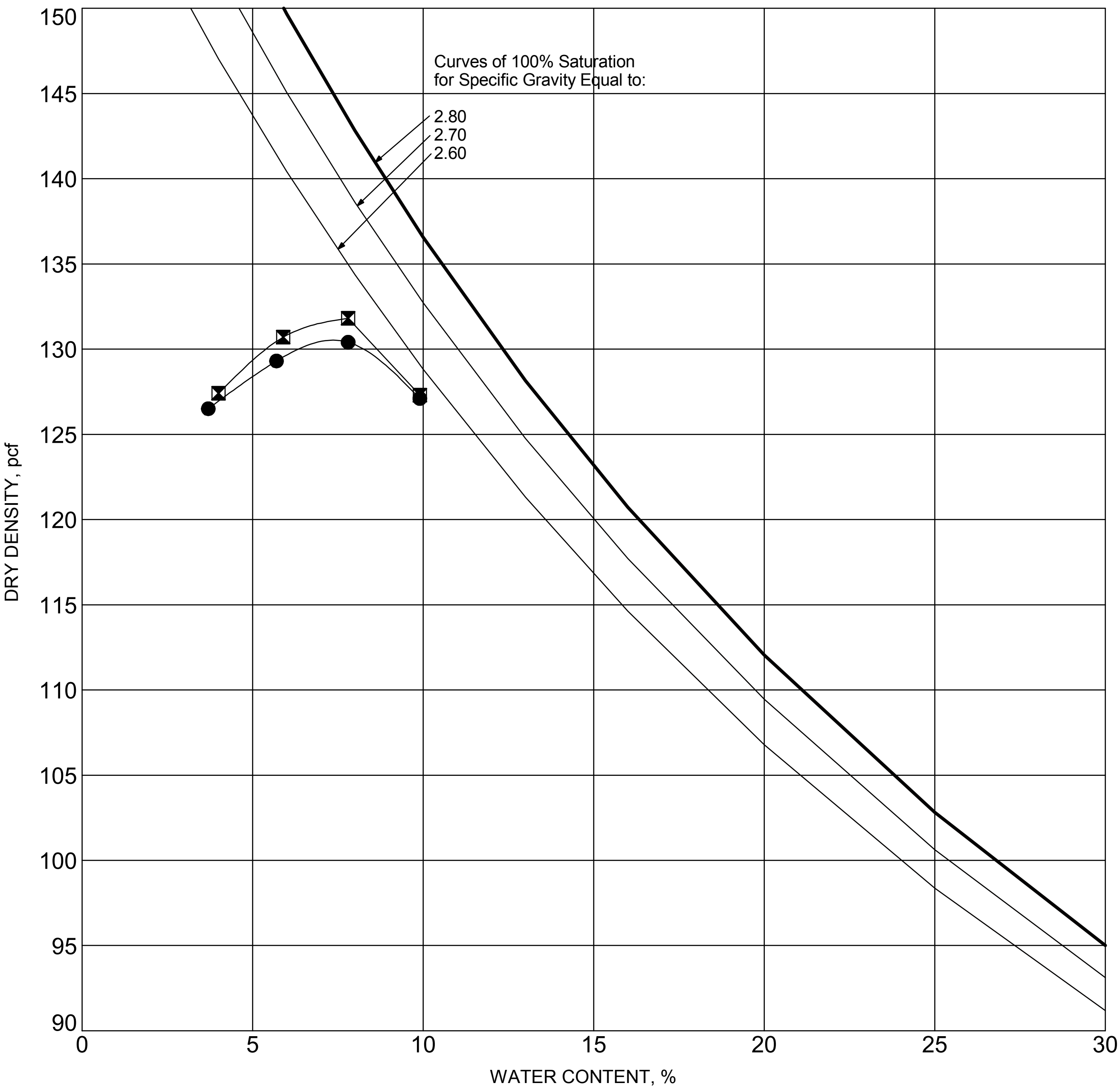
GRAIN SIZE DISTRIBUTION RESULTS

Converse Consultants

1.5 MG Irwin Road Reservoir
Irwin Road (34.936016N, 117.027812W)
City of Barstow, San Bernardino County, California
For: Golden State Water Company

Project No. **20-81-128-01**

Drawing No. **B-1**



SYMBOL	BORING NO.	DEPTH (ft)	DESCRIPTION	ASTM TEST METHOD	OPTIMUM WATER, %	MAXIMUM DRY DENSITY, pcf
●	**BH-01**	**0-4**	**SAND WITH SILT AND GRAVEL (SP-SM), REDDISH BROWN**	**D1557 - A**	**7.0 (6.0*)**	**131.0 (135.4*)**
⧗	**BH-03**	**4-10**	**SAND WITH SILT AND GRAVEL (SP-SM), REDDISH BROWN**	**D1557 - A**	**7.0 (6.0*)**	**132.0 (136.3*)**

(*Rock correction: BH-01 = 14.44% and BH-03 = 14.7%)

MOISTURE-DENSITY RELATIONSHIP RESULTS

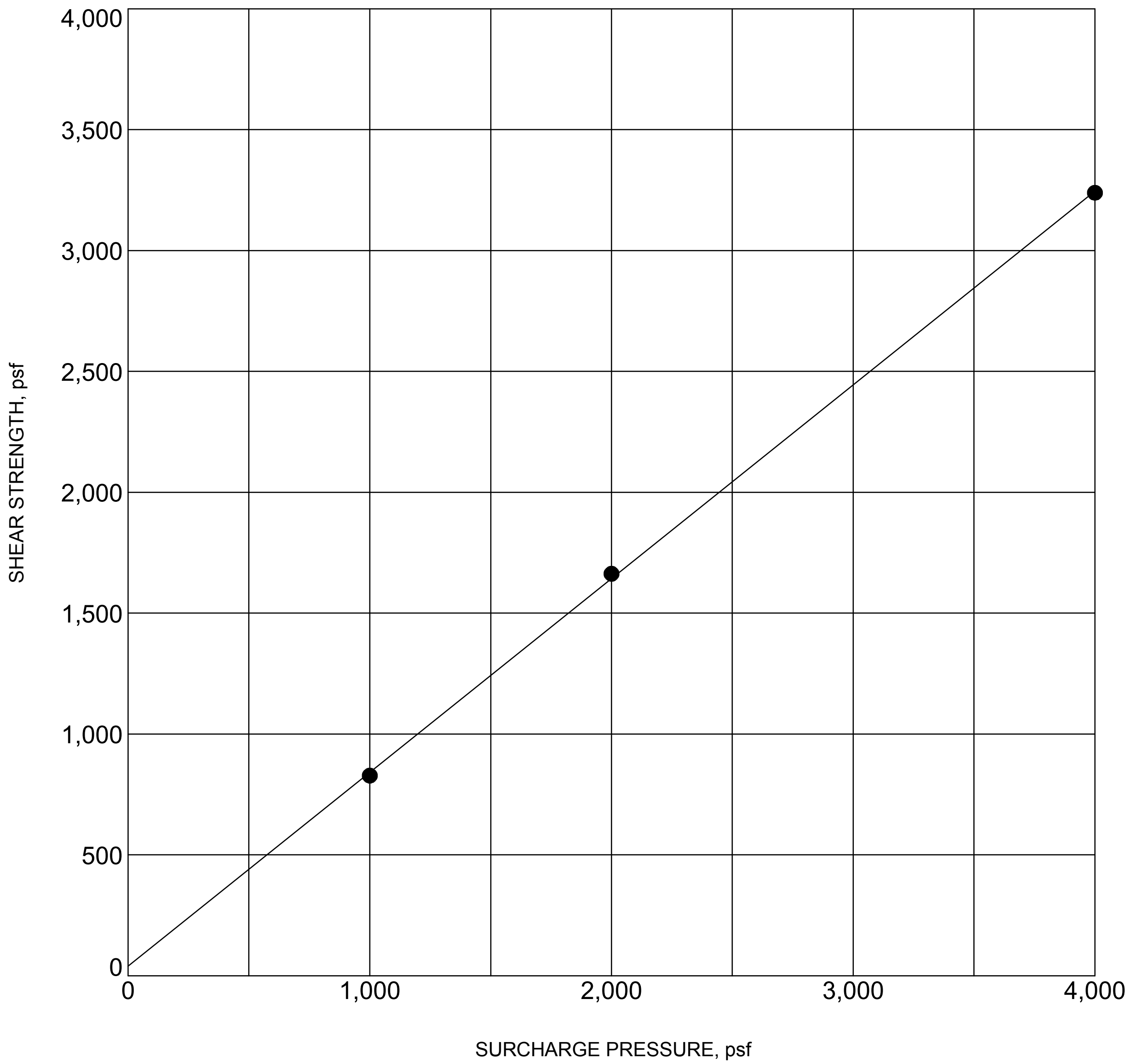
Converse Consultants

1.5 MG Irwin Road Reservoir
Irwin Road (34.936016N, 117.027812W)
City of Barstow, San Bernardino County, California
For: Golden State Water Company

Project No. **20-81-128-01**

Drawing No. **B-2**

Project ID: 20-81-128-01.GPJ; Template: COMPACTION



BORING NO.	:	**BH-01***	DEPTH (ft)	:	**0-4**
DESCRIPTION	:	**SAND WITH SILT AND GRAVEL (SP-SM)**			
COHESION (psf)	:	**40**	FRICTION ANGLE (degrees):		**38**

(*Samples remoldedto 90% of the maximum dry density)

NOTE: Ultimate Strength.

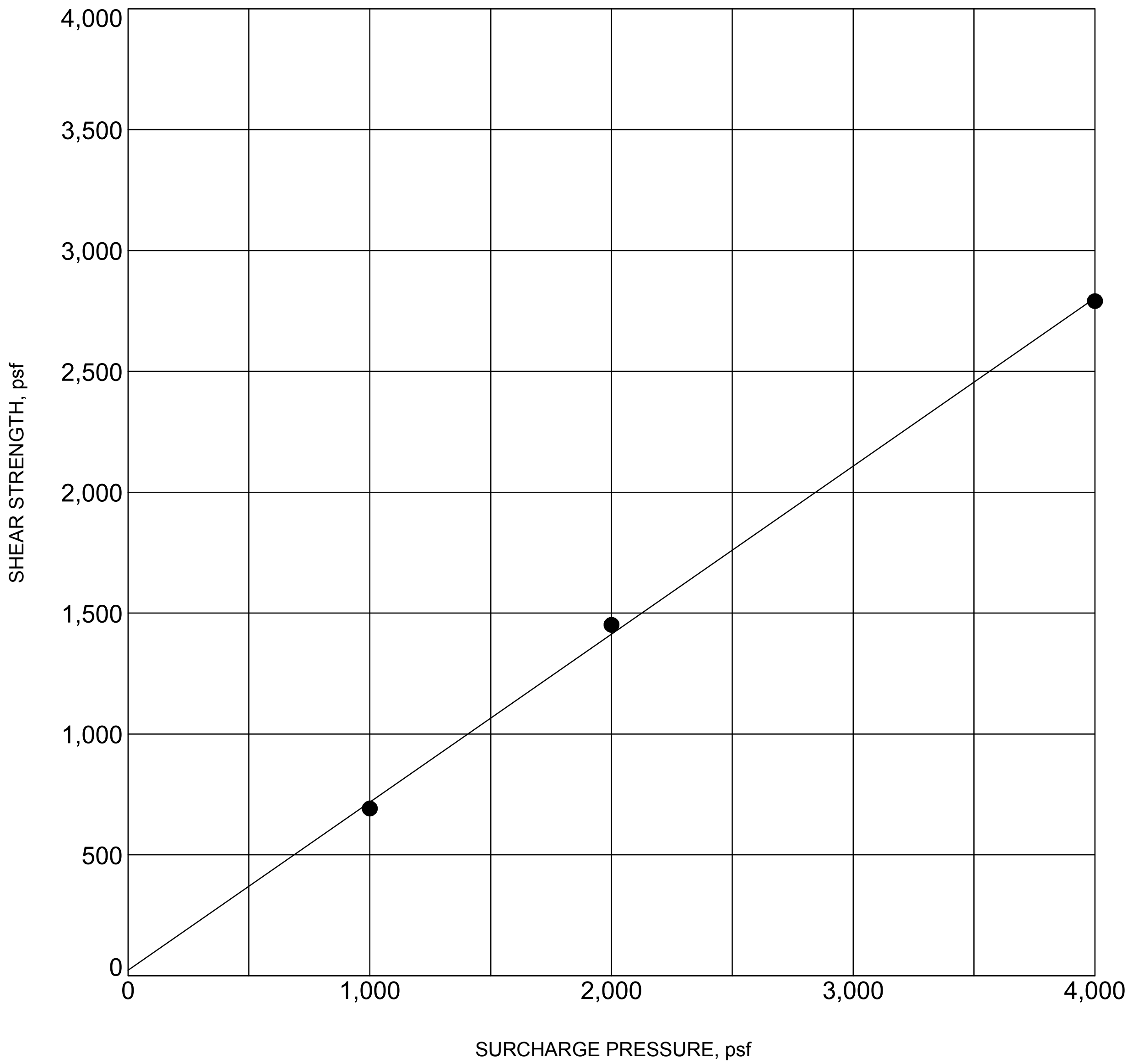
DIRECT SHEAR TEST RESULTS

Converse Consultants

1.5 MG Irwin Road Reservoir
Irwin Road (34.936016N, 117.027812W)
City of Barstow, San Bernardino County, California
For: Golden State Water Company

Project No. **20-81-128-01**

Drawing No. **B-3**



BORING NO.	:	**BH-03***	DEPTH (ft)	:	**4.0-5.5**
DESCRIPTION	:	**SAND WITH SILT AND GRAVEL (SP-SM)**			
COHESION (psf)	:	**20**	FRICTION ANGLE (degrees):		**35**
MOISTURE CONTENT (%)	:	**5.0**	DRY DENSITY (pcf)	:	**103.0**

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

NOTE: Ultimate Strength.

DIRECT SHEAR TEST RESULTS

Converse Consultants

1.5 MG Irwin Road Reservoir
Irwin Road (34.936016N, 117.027812W)
City of Barstow, San Bernardino County, California
For: Golden State Water Company

Project No. **20-81-128-01**

Drawing No. **B-4**

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	33.936016N, 117.027812 W
Site Class	D
Risk Category	III
Mapped Short period (0.2-sec) Spectral Response Acceleration, S_s	1.491g
Mapped 1-second Spectral Response Acceleration, S_1	0.508g
Site Coefficient (from Table 11.4-1), F_a	1.0
Site Coefficient (from Table 11.4-2), F_v	1.792
MCE 0.2-sec period Spectral Response Acceleration, S_{MS}	1.491g
MCE 1-second period Spectral Response Acceleration, SM_1	0.910g
Design Spectral Response Acceleration for short period S_{DS}	0.994g
Design Spectral Response Acceleration for 1-second period, S_{D1}	0.607g
Mapped Risk coefficient at Short Period (0.2 Sec), C_{RS}	0.896
Mapped Risk coefficient at Period 1-second, C_{R1}	0.902

Seismic Parameters	
Long-period transition period in seconds, T_L	8
Site Modified Peak Ground Acceleration, PGA_M	0.723g

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 at 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)	0.65	0.65	0.74	0.52	0.68
Spectral Period (sec)	**2% in 50yr Probabilistic Spectral Acceleration (g)**				
0.05	0.75	0.66	0.89	0.66	0.79
0.10	1.07	0.93	1.31	0.94	1.08
0.20	1.39	1.46	1.55	1.08	1.45
0.30	1.51	1.64	1.51	1.20	1.65
0.40	1.48	1.59	1.40	1.27	1.64
0.50	1.40	1.40	1.35	1.25	1.57
0.75	1.12	1.03	1.07	1.11	1.26
1.00	0.90	0.79	0.83	0.96	1.00
2.00	0.43	0.39	0.38	0.55	0.40
3.00	0.27	0.24	0.24	0.37	0.22
4.00	0.19	0.17	0.17	0.26	0.13
5.00	0.14	0.14	0.13	0.19	0.08
6.00	0.10	0.11	0.10	0.13	0.06
7.00	0.08	0.10	0.08	0.10	0.04
8.00	0.07	0.09	0.07	0.07	0.03
9.00	0.06	0.08	0.06	0.06	0.02
10.00	0.05	0.07	0.05	0.05	0.02

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	0.75	0.896	1.1	0.741
0.10	1.07	0.896	1.1	1.057
0.20	1.39	0.896	1.1	1.374
0.30	1.51	0.897	1.125	1.524
0.40	1.48	0.898	1.15	1.530
0.50	1.40	0.898	1.175	1.472
0.75	1.12	0.900	1.238	1.243
1.00	0.90	0.902	1.3	1.050
2.00	0.43	0.902	1.35	0.527
3.00	0.27	0.902	1.4	0.341
4.00	0.19	0.902	1.45	0.244
5.00	0.14	0.902	1.50	0.184
6.00	0.10	0.902	1.50	0.141
7.00	0.08	0.902	1.50	0.113
8.00	0.07	0.902	1.50	0.094
9.00	0.06	0.902	1.50	0.079
10.00	0.05	0.902	1.50	0.069

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	0.94	1.1	1.039	0.741	0.513	0.51
0.10	1.27	1.1	1.397	1.057	0.709	0.71
0.20	1.72	1.1	1.889	1.374	0.795	0.92
0.30	1.98	1.125	2.230	1.524	0.795	1.02
0.40	2.07	1.15	2.381	1.530	0.795	1.02
0.50	2.03	1.175	2.379	1.472	0.795	0.98
0.75	1.70	1.238	2.105	1.243	0.647	0.83
1.00	1.47	1.3	1.912	1.050	0.486	0.70
2.00	0.81	1.35	1.089	0.527	0.243	0.35
3.00	0.53	1.4	0.746	0.341	0.162	0.23
4.00	0.37	1.45	0.532	0.244	0.121	0.16

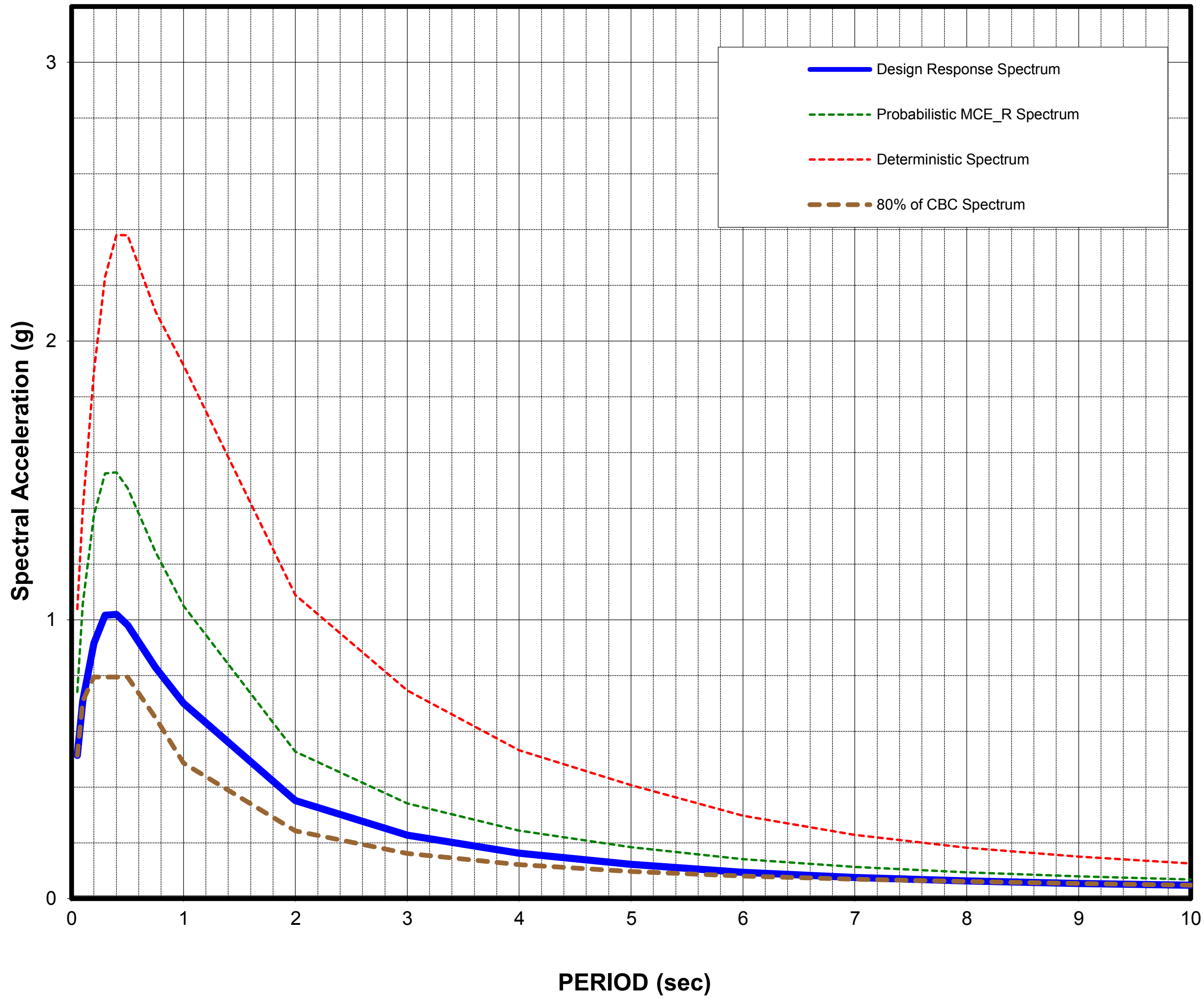
Table No. C-4, Site-Specific Response Spectrum Data (Continued)

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)
5.00	0.27	1.50	0.406	0.184	0.097	0.12
6.00	0.20	1.50	0.297	0.141	0.081	0.09
7.00	0.15	1.50	0.229	0.113	0.069	0.08
8.00	0.12	1.50	0.183	0.094	0.061	0.06
9.00	0.10	1.50	0.150	0.079	0.054	0.05
10.00	0.08	1.50	0.126	0.069	0.049	0.05

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}	1.377	1.193
Site-Specific1-second period Spectral Response Acceleration, S_{M1}	1.054	0.728
Site-Specific Design Spectral Response Acceleration for short period S_{DS}	0.918	0.795
Site-Specific Design Spectral Response Acceleration for 1-second period, S_{D1}	0.703	0.486



Note: Calculated using EZFRISK program Risk Engineering, version 8.06

SITE SPECIFIC DESIGN RESPONSE SPECTRUM

Project: 1.5 MG Irwin Road Reservoir
Location: Irwin Road (34.936016N, 117.027812W)
City of Barstow, San Bernardino County, California
For: Golden State Water Company

Project Number: 20-81-128-01

Converse Consultants

Drawing No. C-1

APPENDIX D

LIQUEFACTION AND SETTLEMENT ANALYSES

The subsurface data obtained from the boring BH-01 was used to evaluate the liquefaction potential and associated dry seismic settlement when subjected to ground shaking during earthquakes.

A simplified liquefaction hazard analysis was performed using the program SPTLIQ and CLiq (InfraGEO Software, 2019) using the liquefaction triggering analysis method by Boulanger and Idriss (2014). A modal earthquake magnitude of M 7.48 was selected based on the results of the seismic deaggregation analyses using the USGS interactive online tool (https://earthquake.usgs.gov/hazards/interactive/).

A site-specific peak ground acceleration (PGA_M) of 0.85g for the MCE design event, where g is the acceleration due to gravity, was selected for this analysis. The PGA was based on site-specific seismic design parameters presented in Section 7.3, *Site-Specific Seismic Analysis.*

The result of our analysis is presented on Plate D-1 and summarized in the following table.

Table D-1, Estimated Dynamic Settlement

Location		Groundwater Depth (feet bgs)	Dry Seismic Settlement (inches)	Liquefaction Induced Settlement (inches)
BH-01	Current	> 50.0	0.11	Negligible
	Historical	> 50.0		

Based on our analysis, the site has up to 0.11 inches of dry seismic settlement with negligible amount of liquefaction induced settlement. The dynamic differential settlement can be taken as half of the total seismic settlement.

SIMPLIFIED LIQUEFACTION HAZARDS ASSESSMENT USING STANDARD PENETRATION TEST (SPT) DATA

(Copyright © 2015, 2018, SPTLIQ, All Rights Reserved; By: InfraGEO Software)

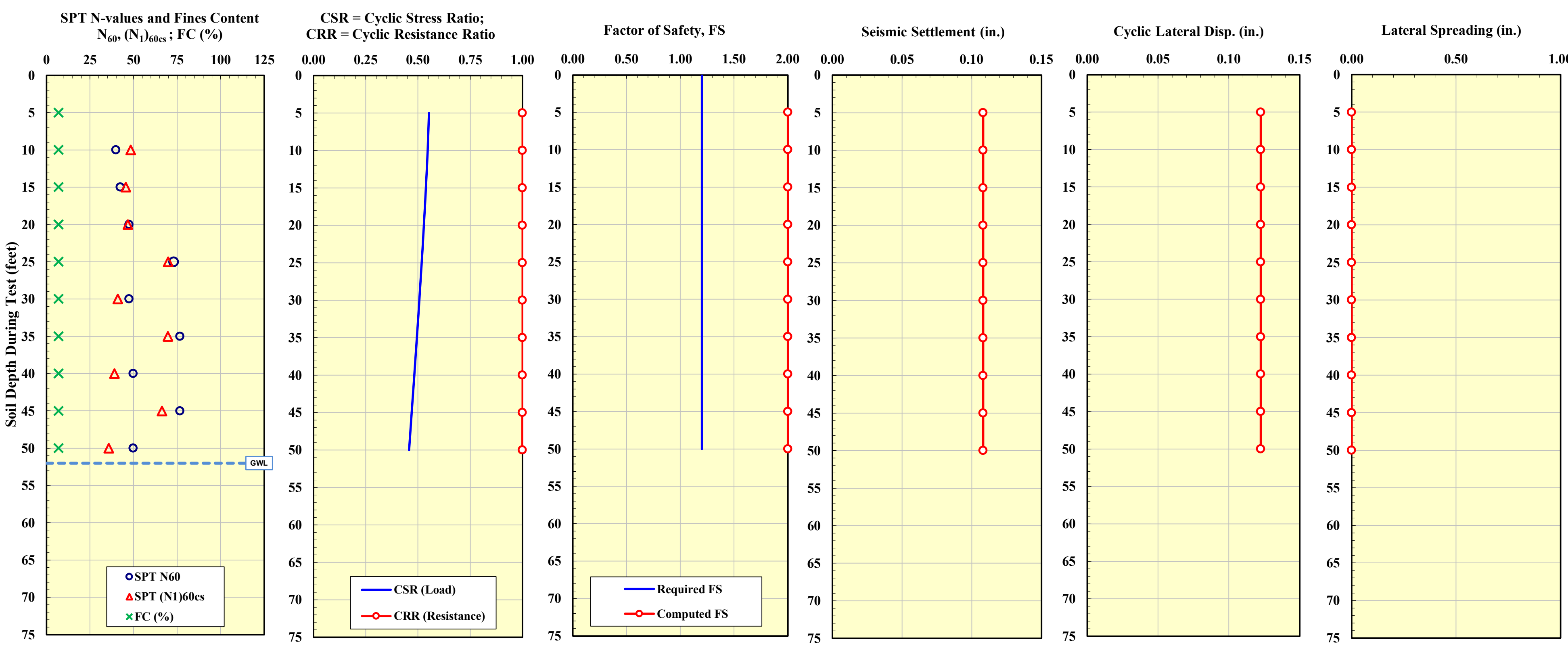
PROJECT INFORMATION	
Project Name	1.5 MG Irwin Road Reservoir
Project No.	20-81-128-01
Project Location	Irwin Road, Barstow, CA
Analyzed By	Z. Alam
Reviewed By	H. Quazi

TOPOGRAPHIC CONDITIONS		
Ground Slope, S	0.00 %	
Free Face (L/H) Ratio	N/A	H = 0.00 feet

GROUNDWATER LEVEL DATA	
GWL Depth Measured During Test	52.00 feet
GWL Depth Used in Design	52.00 feet

BORING DATA	
Boring No.	BH-01
Ground Surface Elevation	2,340.00 feet
Proposed Grade Elevation	2,340.00 feet
Borehole Diameter	8.00 inches
Hammer Weight	140.00 pounds
Hammer Drop	30.00 inches
Hammer Energy Efficiency Ratio, ER	80.00 %
Hammer Distance to Ground Surface	5.00 feet

SEISMIC DESIGN PARAMETERS	
Earthquake Moment Magnitude, M_w	7.48
Peak Ground Acceleration, A_{max}	0.85 g
Required Factor of Safety, FS	1.20



Analysis Methods Used ==>>

Liquefaction Triggering:	Seismic Settlements:	Cyclic Lateral Displacements:	Lateral Spreading:
Boulanger-Idriss (2014)	Above GWL: Pradel (1998)	Above GWL: Pradel (1998)	Zhang et al. (2004)
	Below GWL: Ishihara and Yoshimine (1992)	Below GWL: Tokimatsu and Asaka (1998)	