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

GEOTECHNICAL INVESTIGATION REPORT

RICE CANYON RESERVOIR PERMANENT ACCESS ROAD AND NEW CONDUIT

WEST OF DALE COURT AND LINCOLN STREET
CITY OF LAKE ELSINORE, RIVERSIDE COUNTY, CALIFORNIA

CONVERSE PROJECT NO. 21-81-268-01

Prepared For:
DUDEK
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July 12, 2022

July 12, 2022

Charles Greely, PE, LEED AP, QSD
Principal
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Subject: **GEOTECHNICAL INVESTIGATION REPORT**
Rice Canyon Reservoir Permanent Access Road and New Conduit
West of Dale Court and Lincoln Street
City of Lake Elsinore, Riverside County, California
Converse Project No. 21-81-268-01

Dear Mr. Charles:

Converse Consultants (Converse) is pleased to submit this geotechnical investigation report to assist with the design and construction of the Rice Canyon Reservoir Permanent Access Road and New Conduit project, located at West of Dale Ct and Lincoln Street, City of Lake Elsinore, Riverside County, California. The report was prepared in accordance with our proposal dated November 2, 2021, and your Subconsultant Agreement for Professional Services dated February 15, 2022.

Based upon our field investigation, laboratory data, and analyses, the project is considered feasible from a geotechnical standpoint, provided the recommendations presented in this report are incorporated into the design and development of the project.

We appreciate the opportunity to be of service to Dudek. 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

Dist.: 1-Electronic Pdf /Addressee
SR/KN/HSQ/kvg

PROFESSIONAL CERTIFICATION

This report has been prepared by the following professionals whose seals and signatures appear hereon.

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

SK Syfur Rahman, PhD, EIT
Senior Staff Engineer

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Senior Staff Geologist

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

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1.0 INTRODUCTION

This report presents the results of our geotechnical investigation performed for Rice Canyon Reservoir Permanent Access Road and New Conduit project, located at the West of Dale Ct and Lincoln Street, City of Lake Elsinore, Riverside County, California. The approximate location of the site is shown on Figure No. 1, *Approximate Site Location Map*.

The purposes of this investigation were to determine the nature and engineering properties of the subsurface soils and to provide recommendations for site earthwork, and design and construction of the proposed improvements.

This report is prepared for the project described herein and is intended for use solely by Dudek, Elsinore Valley Municipal Water District (EVMWD) 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.

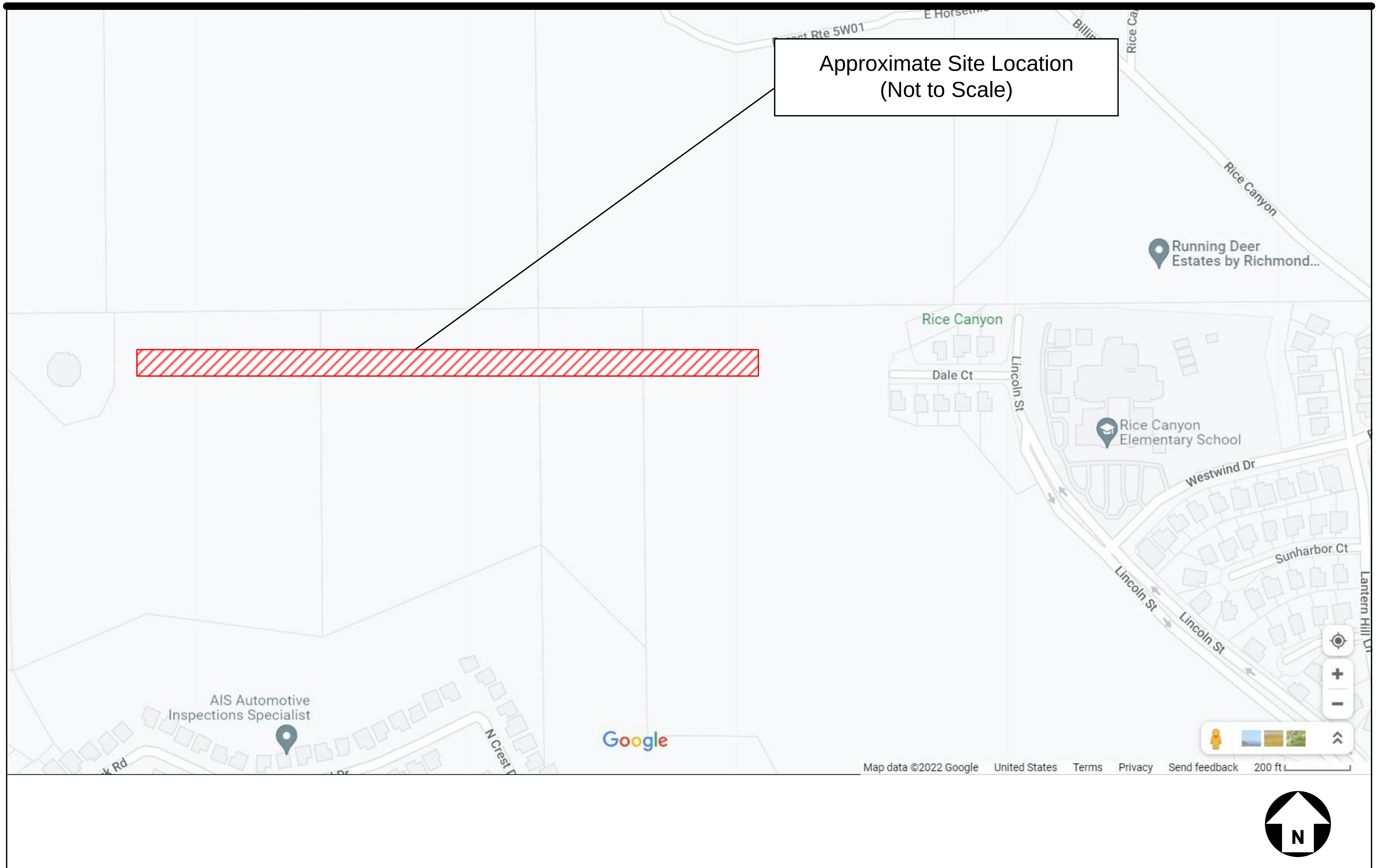
2.0 PROJECT BACKGROUND AND DESCRIPTION

In 2018, the City of Lake Elsinore and the area surrounding the Rice Canyon Reservoir endured damage from the 2018 Holy Fire. Subsequent to the Holy Fire, the following winter season flooding hit the region causing mud slides through Rice Canyon. This resulted in the reservoir losing its power source and remote connection to the district monitoring system. The post-Holy Fire flooding also undercut and damaged 3 concrete Arizona crossings located within the canyon which made the access road inaccessible by vehicles. Interim repairs to the access road were done in 2019, including an assessment of the conduit and tank.

To make permanent repairs for the access to the reservoir and enable communication and power to the reservoir, the improvements that are required are as follows.

- Replace the existing electrical conduit that is within the existing access road from Dale Court to Rice Canyon Reservoir potable water tank using open trench method.
- Expand 3 existing concrete Arizona crossings, allowing permanent access to the Rice Canyon Reservoir and protection of the existing 16-inch water main and the new electrical conduct for vehicle accessing the reservoir.

The focus of this investigation is to provide recommendations for the expansion of the 3 Arizona Crossings (culvert crossings). According to the information provided by Mr. Charles Greely with Dudek, the cutoff depths for the Arizona crossings will be in the range of 5 to 9 feet below ground surface.



Project: Rice Canyon Reservoir Permanent Access Road and New Conduit

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Approximate Site Location Map

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Figure No. 1

3.0 SITE DESCRIPTION

Rice Canyon Reservoir Permanent Access Road and New Conduit project is located at West of Dale Court and Lincoln Street, City of Lake Elsinore, Riverside County, California. The site is bounded by Lincoln Street in the east and hilly vacant land in the north, south and west. The site presently consists of existing reservoirs and associated structures, 3 damaged Arizona crossing and access road. Tress, large boulders, gravels were visible in both side of the existing access road. Present site conditions are depicted in Photograph Nos. 1 and 2.

Photograph No. 1: Present site conditions facing west.

Photograph No. 2: Present site conditions facing northeast.

4.0 SCOPE OF WORK

The scope of this investigation includes the following tasks presented below.

4.1 Project Set-up

As part of the project set-up, our staff performed the following tasks.

- Prepared the test pit locations map and submitted to Mr. Charles Greely with Dudek for review and approval.
- Conducted a site reconnaissance and staked/marked the test pit locations such that backhoe access to all the locations was available.
- Notified Underground Service Alert (USA) at least 48 hours prior to excavating to clear the test pit locations of any conflict with existing underground utilities.
- Engaged a California-licensed backhoe to excavate exploratory test pits.

4.2 Subsurface Exploration

Three exploratory test pits (TP-01 through TP-03) were excavated using a backhoe equipped with 24-inch-wide bucket to investigate the subsurface conditions on April 21, 2022. The test pits were excavated between 5.0 feet and 10.0 feet below the existing ground surface (bgs).

No drive sample could be collected due to the presence of gravelly soil. Representative bulk samples were collected from selected depths and placed in large plastic bags for delivery to our laboratory.

The approximate locations of the test pits are shown on Figure No. 2, *Approximate Test Pit Locations Map.* A detailed discussion of the subsurface exploration is presented in Appendix A, Field Exploration.

4.3 Laboratory Testing

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

- Soil corrosivity (California Tests 422, 417, and 643)
- Grain size distribution (ASTM D6913)
- Maximum dry density and optimum moisture content (ASTM D1557)
- Direct shear (ASTM D3080)

For a description of the laboratory test methods and test results, see Appendix B, *Laboratory Testing Program*.

Project: Rice Canyon Reservoir Permanent Access Road and New Conduit

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Approximate Test Pit Locations Map

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Figure No.
2

4.4 Report Preparation

Data and information obtained from the document review, 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 improvements.

5.0 SITE CONDITIONS

The subsurface conditions encountered at the site during our investigation are described in the following sections.

5.1 Subsurface Profile

Based on the exploratory test pits and laboratory test results, the subsurface soils at the site consisted primarily of a mixture of sand, gravel, cobble, and boulders and occasional silty/clay. Gravel, cobble, and boulders up to a 3-foot maximum dimension were observed in all the test pits. Boulders up to a 4-foot maximum dimension were observed in TP-02 and up to 8 feet maximum dimension were observed TP-01. Larger boulders could be present within the project site limits.

Discernible fill soils were identified in TP-01 and TP-03 as a result of previous grading for the existing structures and access road. Where present, the fill soils were likely derived from on-site sources and are similar to the native alluvial soils in composition and density.

For a detailed description of the subsurface materials encountered in the exploratory test pits, see Drawings No. A-2 through A-4 *Logs of Test Pits,* in Appendix A, *Field Exploration.*

5.2 Groundwater

Groundwater was not encountered in any test pits during the field investigation. No standing water was observed. For comparison, the following databases were reviewed for relevant groundwater information.

The GeoTracker database (SWRCB, 2022) was reviewed to evaluate the current and historical groundwater levels from sites within approximately a 1.0-mile radius of the project site. No site with groundwater data was found in this review.

The National Water Information System (USGS, 2022) was reviewed to evaluate current and historical groundwater levels from sites within approximately a 1.0-mile radius of the project site. No site with groundwater data was found in this review.

The California Department of Water Resources database (DWR, 2022) was reviewed to evaluate current and historical groundwater data from sites within a 1.0-mile radius of the project site. No site with groundwater data was found in this review.

Based on available data, the historical high groundwater level at the project site is unknown. Based on our field investigation, current groundwater is expected to be deeper than 10 feet bgs. However, this project site traverses an active blue line stream channel which indicates that groundwater could be present at shallow depths or above the ground surface at any time. Therefore, it should be noted that the groundwater level could vary depending upon the seasonal precipitation, recent precipitative events, and possible groundwater pumping activity in the site vicinity.

5.3 Excavatability

The subsurface materials at the site are expected to be marginally excavatable by conventional heavy-duty earth moving and trenching equipment. However, excavation will be difficult where any concentration of gravel, cobble, or boulders 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 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.

6.0 ENGINEERING GEOLOGY

The regional and local geology within the proposed project area is discussed below.

6.1 Regional Geology

The project site is located on the northwestern margin of the Elsinore Trough in the northern Peninsular Ranges Geomorphic Province of Southern California.

The Peninsular Ranges Geomorphic Province consists of a series of southeast-trending mountain ranges and valleys bounded on the north by the San Bernardino and San Gabriel Mountains, on the west by the Los Angeles Basin, and on the south by the Pacific Ocean.

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

Topography within the province is generally characterized by broad alluvial valleys separated by linear mountain ranges. This southeast-trending linear fabric is created by the regional faulting within the granitic basement rock of the Southern California Batholith. Broad, linear, alluvial valleys have been formed by erosion of these principally granitic mountain ranges.

The Elsinore Trough is a graben valley between the Santa Ana Mountains to the west and the Perris Block to the east. The valley was formed by movement within the northwest-trending Elsinore Fault Zone. In the vicinity of Lake Elsinore, the valley is bounded by the Glen Ivy North fault to the northeast and the Willard and Wildomar faults to the southwest.

6.2 Local Geology

The project site is located in Rice Canyon approximately 3 miles northeast of the modern shore of Lake Elsinore. Regional geologic mapping (Morton and Miller, 2006) indicates that the project site is underlain by young (Holocene and late Pleistocene) alluvial channel deposits. This alluvium generally consists of slightly to moderately consolidated sand, silt, clay, and gravel. Since the project site is in an active drainage channel, cobbles and boulders of surrounding geologic units can be expected to be present at and below the surface.

7.0 CBC 2019 SEISMIC DESIGN PARAMETERS

Seismic parameters based on the 2019 California Building Code (CBSC, 2019) and ASCE 7-16 are provided in the following table. These parameters were determined using the generalized coordinates for the location and the Seismic Design Maps ATC online tool.

Table No. 1, CBC Seismic Design Parameters

Parameter	Value
Site Coordinates	33.697631 N, 117.409734 W
Risk Category	II
Site Class	D
Mapped Short period (0.2-sec) Spectral Response Acceleration, S_S	2.471g
Mapped 1-second Spectral Response Acceleration, S_1	0.932g
Site Coefficient (from Table 1613.5.3(1)), F_a	1.0
Site Coefficient (from Table 1613.5.3(2)), F_v	1.7
MCE 0.2-sec period Spectral Response Acceleration, S_{MS}	2.471g
MCE 1-second period Spectral Response Acceleration, S_{M1}	1.584g
Design Spectral Response Acceleration for short period S_{DS}	1.647g
Design Spectral Response Acceleration for 1-second period, S_{D1}	1.056g
Peak Ground Acceleration, PGA_M	1.144g

8.0 LABORATORY TEST RESULTS

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 Test Pits in Appendix A, *Field Exploration.* The results are also discussed below.

8.1 Physical Testing

The results of laboratory tests on samples obtained from the site is presented below.

- Grain Size Analysis – One representative soil sample was tested to determine the relative grain size distribution in accordance with the ASTM Standard D6913. The test result is graphically presented in Drawing No. B-1, *Grain Size Distribution Results.*
- Maximum Dry Density and Optimum Moisture Content – The moisture-density relationship of a representative soil sample was tested in according to ASTM Standard D1557 and the results are presented in Drawing No. B-2, *Moisture-Density Relationship Results*, in Appendix B, *Laboratory Testing Program.* The laboratory maximum dry density was 129.5 pounds per cubic feet (pcf) with optimum moisture content of 7.8 percent.
- Direct Shear – One remolded direct shear test was performed in accordance with ASTM Standard D3080. The results of the direct shear test are presented in Drawing No. B-3, *Direct Shear Test Results* in Appendix B, *Laboratory Testing Program*.

8.2 Chemical Testing - Corrosivity Evaluation

One representative soil sample was 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 construction materials. These tests were performed by AP Engineering and Testing, Inc. (Pomona, CA) in accordance with California Tests 643, 422, and 417. The test results presented in Appendix B, *Laboratory Testing Program* and summarized below.

- The pH measurement of the sample was 7.8.
- The soluble sulfate content of the sample was 24 ppm (0.0024 percent by weight).
- The chloride concentration of the sample was 19 ppm.
- The minimum electrical resistivity (wet condition) of the sample when saturated was 19,079 ohm-cm.

9.0 EARTHWORK RECOMMENDATIONS

Earthwork recommendations for the project are presented below.

9.1 General

This section contains our general recommendations regarding earthwork and grading for the proposed improvements. These recommendations are based on the results of our field exploration, laboratory tests, our experience with similar projects, and data evaluation as presented in the preceding sections. These recommendations may require modification by the geotechnical consultant based on findings during the final investigation or observation of the actual field conditions during grading.

All existing underground utilities and appurtenances should be located at the site. 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 structures or utilities.

All debris, deleterious material, demolished material, and artificial fill (if any) and surficial soils containing roots and perishable materials should be stripped and removed from the site. 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 Overexcavation/Removal

Structural footings and slabs-on-grade should be uniformly supported by compacted fill. In order to provide uniform support, structural areas should be overexcavated, scarified, and recompacted as follows.

Table No. 2, Overexcavation Depths

Structure/Pavement	Minimum Excavation Depth
Mat Foundation/ Wall Footings	15 inches below foundation/footing bottoms, or 5 feet bgs, whichever is deeper
Pavement	12 inches below pavement or 3 feet bgs, whichever is deeper

The depth of over excavation below the footings (retaining wall)/mat foundation, and pavement should be uniform. The over excavation should extend to at least 2 feet beyond the footprint of the footings (retaining wall) and mat foundation. The over excavation bottom should be scarified and compacted as described in Section 9.4, *Compacted Fill Placement*.

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. Consideration should be given to using slot cuts or other excavation methods which preserve lateral support during excavation operations near the existing structures.

9.3 Engineered/Structural Fill

No fill or aggregate base should be placed until excavations and/or natural ground preparation have been observed by the geotechnical consultant. The native soils encountered within the site are generally considered suitable for re-use as compacted fill. Excavated soils should be processed, including cleaning roots and debris, removal of oversized particles, mixing, and moisture conditioning, before placing as compacted fill. On-site soils used as fill should meet the following criteria.

- No particles larger than 3 inches in largest dimension.
- Rocks larger than 1 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 in 3/4-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 structural/engineered fill materials if they are process according to this section.

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

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 1 feet of subgrade soils underneath pavements intended to support vehicle loads should be scarified, moisture conditioned, and 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 Compaction Behind Wall

The retaining wall (associated with the Arizona crossing) backfill should be compacted to 90 percent of laboratory the maximum dry density. Compaction of backfill adjacent to wall can produce excessive lateral pressures. Improper types and locations of compaction equipment and/or compaction techniques may damage the wall. The use of heavy compaction equipment should not be permitted within a horizontal distance of 5 feet from the wall. Backfill behind any wall within the recommended 5-foot zone should be compacted using lightweight construction equipment such as handheld compactors to avoid overstressing the wall.

9.6 Trench Zone Backfill

The trench zone is defined as the portion of the trench above the pipe (electrical conduit) bedding extending up to the final grade level of the trench surface. Excavated on-site soils free of oversize particles and deleterious matter may be used to backfill the trench zone. Trench zone backfill should follow EVMWD or City of Lake Elsinore Standards, whichever is applicable. If additional recommendations beyond EVMWD or City of Lake Elsinore Standards are needed, the following specifications can be used for trench backfills.

- Trench excavations to receive backfill should be free of trash, debris or other unsatisfactory materials at the time of backfill placement.
- Trench zone backfill should be compacted to at least 90 percent of the laboratory maximum dry density as per ASTM D1557 test method. At least the upper 1 foot of trench backfill underlying pavement should be compacted to at least 95 percent of the laboratory maximum dry density as per ASTM D1557 test method.
- Particles larger than 1 inch should not be placed within 12 inches of the pavement subgrade. No more than 30 percent of the backfill volume should be larger than ¾-inch in the largest dimension. Gravel should be well mixed with finer soil. Rocks larger than 3 inches in the largest dimension should not be placed as trench backfill.
- Trench backfill should be compacted by mechanical methods, such as sheepsfoot, vibrating or pneumatic rollers or mechanical tampers to achieve the density specified herein. The backfill materials should be brought to within $\pm$ 3 percent of optimum moisture content for coarse-grained soil, and between optimum and 2 percent above optimum for fine-grained soil, then placed in horizontal layers. The thickness of uncompacted layers should not exceed 8 inches. Each layer should be evenly spread, moistened or dried as necessary, and then tamped or rolled until the specified density has been achieved.
- The contractor should select the equipment and processes to be used to achieve the specified density without damage to adjacent ground, structures, utilities and completed work.
- The field density of the compacted soil should be measured by the ASTM D1556 (Sand Cone) or ASTM D6938 (Nuclear Gauge) or equivalent.
- Trench backfill should not be placed, spread or rolled during unfavorable weather conditions. When the work is interrupted by heavy rain, fill operations should not resume until field tests by the project's geotechnical consultant indicate that the moisture content and density of the fill are in compliance with project specifications.

9.7 Shrinkage and Subsidence

The in-situ moisture and dry density of the site soils were not determined due to the lack of drive samples. Therefore, it was not possible to estimate the shrinkage and subsidence of the site soils after compaction. However, since the site soil contains a large number of particles greater than 3 inches, there will be a reduction in the volume of compactable soil after removing large particles. To determine actual shrinkage and subsidence factors, it is recommended that field-testing using the actual equipment and grading techniques be conducted.

9.8 Site Drainage

Adequate positive drainage should be provided away from the structure and excavation areas to prevent ponding and to reduce percolation of water into the foundation soils. The building pad should have a gradient of at least 2 percent towards drainage facilities. The drainage gradient should be 1 percent for paved areas and 2 percent in landscaped areas. Surface drainage should be directed to suitable non-erosive devices.

10.0 DESIGN RECOMMENDATIONS

Design recommendations are presented in the following sections.

10.1 General Evaluation

The various design recommendations provided in this section are based on the assumptions that in preparing the site, the above earthwork recommendations will be implemented.

10.2 Mat Foundation Design Parameters

The Arizona crossing may be supported by mat foundation. The modulus of subgrade reaction (k) for design of flexible mat foundation can be estimated from the available soil compressibility data and published charts. For design of flexible mat foundation, the following equation may be used.

$$k = k_1[(B+1)/2B]^2$$

Where:
k= vertical modulus of subgrade reaction for mat foundation, kips per cubic feet
k_1= 200 kcf, normalized modulus of subgrade reaction for 1-square-foot footing
B= foundation width, feet

Other necessary parameters (modulus of elasticity and Poisson's ratio) for mat foundation design are as follows.

$E = 33\ W_c^{1.5}\ f_c^{\ 0.5}$ psi
Where, E = Modulus of Elasticity of Concrete (psi)
W_c = weight of concrete (pcf)
f_c = compressive strength of concrete at 28 days (psi)
ν = 0.35, Poisson's Ratio

An allowable net bearing capacity of 2,500 psf may be used for mat foundations founded on compacted native soil. The mat should be reinforced with top and bottom steel, as appropriate, to provide structural continuity and to permit spanning of local irregularities. The mat foundation dimensions, and reinforcement should be based on structural design. For design purposes, the self-weight of the mat foundation can be negligible.

10.3 Lateral Earth Pressures and Resistance to Lateral Loads

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

10.3.1 Active Earth Pressures

The active earth pressure behind any buried wall or foundation depends primarily on the allowable wall movement, type of backfill materials, backfill slopes, wall or foundation inclination, surcharges, and any hydrostatic pressures. The recommended lateral earth pressures without surcharge for the site are presented in the following table.

Table No. 3, Active and At-Rest Earth Pressures

Loading Conditions	Lateral Earth Pressure (psf/ft 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 ground surface around the structure for a distance greater than the structure height, 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.

10.3.2 Passive Earth Pressure

Resistance to lateral loads can be assumed to be provided by a combination of friction acting at the base of foundations and by passive earth pressure. A coefficient of friction of 0.35 between concrete and soil may be used with the dead load forces. An allowable passive earth pressure of 275 psf per foot of depth may be used for the sides of footings poured 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.

10.4 Seismic Earth Pressure

Parameters for seismic earth pressure behind the buried wall or foundation are listed in the following table.

Table No. 4, Seismic Earth Pressure

Parameters	Value
Horizontal Seismic Coefficient	0.22H*
Existing Wall Footing Pressure (psf)	2000
Soil Unit Weight (pcf)	120.0
Soil Internal Friction Angle (degree)	32.0
Active Earth Pressure (psf)	40

*H = height of buried wall

10.5 Settlement

The total settlement of mat foundation 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. Dynamic settlement analysis was not part of the scope of the project.

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.

10.6 Soil Corrosivity

The results of chemical testing of one representative soil sample 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 general discussion pertaining to soil corrosivity are presented below.

The sulfate contents of the sampled soil correspond 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 site locations and the results 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 sample when saturated was 19,079 Ohm-cm. This indicates that the soil tested of the site is mildly corrosive to ferrous metals in contact with the soils (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 and site soils.

10.7 Flexible Pavement Recommendations

R-values of the site soil was not tested. However, for pavement design, we have utilized a design subgrade R-value of 40 and design Traffic Indices (TIs) ranging from 5 to 8.

Based on the above information, 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 below.

Table No. 6, Recommended Preliminary Pavement Sections

R-value	Traffic Index (TI)	Pavement Section		
		Option 1		Option 2
		Asphalt Concrete (inches)	Aggregate Base (inches)	Full AC Section (inches)
40	5	4.0	3.5	4.0
	6	4.0	4.5	5.0
	7	4.0	4.5	6.5
	8	4.5	6.0	7.5

At or near the completion of grading, subsurface samples should be tested to evaluate the actual subgrade R-value for final pavement design. *Please note that the minimum standard section required by the City of Lake Elsinore for local roads is 3.5-inch asphalt over 4-inch aggregate base.*

Prior to placement of aggregate base, appropriate earthworks should be performed according to specifications provided in section 9.0 earthwork recommendations.

Base materials should conform with Section 200-2.2,"*Crushed Aggregate Base*," of the current Standard Specifications for Public Works Construction (SSPWC; Public Works Standards, 2018) and should be placed in accordance with Section 301.2 of the SSPWC.

Asphaltic concrete materials should conform to Section 203 of the SSPWC and should be placed in accordance with Section 302.5 of the SSPWC.

10.8 Rigid Pavement Recommendations

Rigid pavement design recommendations were provided in accordance with the Portland Cement Association's (PCA) Southwest Region Publication P-14, Portland Cement Concrete Pavement (PCCP) for Light, Medium and Heavy Traffic Rigid Pavement. For pavement design, we have utilized a design subgrade R-value of 40 and design Traffic Indices (TIs) ranging from 5 to 9. We recommend that the project structural engineer consider the loading conditions at various locations and select the appropriate pavement sections from the following table.

Table No. 7, Rigid Pavement Structural Sections

Design R-Value	Design Traffic Index (TI)	PCCP Pavement Section (inches)
40	5.0	6.5
	6.0	6.5
	7.0	7.0
	8.0	7.5
	9.0	7.5

The above pavement section is based on a minimum 28-day Modulus of Rupture (M-R) of 550 psi and a compressive strength of 3,750 psi. The third point method of testing beams should be used to evaluate modulus of rupture. The concrete mix design should contain a minimum cement content of 5.5 sacks per cubic yard. Recommended maximum and minimum values of slump for pavement concrete are 3.0 inches to 1.0 inch, respectively.

Transverse contraction joints should not be spaced more than 10 feet and should be cut to a depth of 1/4 the thickness of the slab. Longitudinal joints should not be spaced more than 12 feet apart. A longitudinal joint is not necessary in the pavement adjacent to the curb and gutter section.

Prior to placement of concrete, at least the upper 12.0 inches of subgrade soils below rigid pavement sections should be compacted to at least ninety-five percent (95%) relative compaction as defined by the ASTM D 1557 standard test method.

Positive drainage should be provided away from all pavement areas to prevent seepage of surface and/or subsurface water into pavement base and/or subgrade.

10.9 Concrete Flatwork

Except as modified herein, concrete walks, driveways, access ramps, curb and gutters should be constructed in accordance with Section 303-5, *Concrete Curbs, Walks, Gutters, Cross-Gutters, Alley Intersections, Access Ramps, and Driveways*, of the Standard Specifications for Public Works Construction (Public Works Standards, 2018).

The subgrade soils under the above structures should consist of compacted fill placed as described in this report. Prior to placement of concrete, the upper 2 feet of subgrade soils should be moisture conditioned to between within 3 percent of optimum moisture content for coarse-grained soils and 0 to 2 percent above optimum for fine-grained soils.

The cement concrete thickness of driveways for passenger vehicles should be at least 4 inches, or as required by the civil or structural engineer. Transverse control joints for driveways should be spaced not more than 10 feet apart. Driveways wider than 12 feet should be provided with a longitudinal control joint.

11.0 CONSTRUCTION RECOMMENDATIONS

Temporary sloped excavation recommendations are presented in the following sections.

11.1 General

Prior to the start of construction, all existing underground utilities should be located at the project site. Such utilities should either be protected in-place or removed and replaced during construction as required by the project specifications.

Sloped excavations may not be feasible in locations adjacent to existing utilities or pavement. Recommendations pertaining to temporary excavations are presented in this section.

Excavations near existing utilities or pavement may require vertical side wall excavation. Where the side of the excavation is a vertical cut, it should be adequately supported by temporary shoring to protect workers and any adjacent structures.

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

11.2 Temporary Sloped Excavations

Temporary open-cut trenches may be constructed with side slopes as recommended in the following table. Temporary cuts encountering soft and wet fine-grained soils; dry loose, cohesionless soils or loose fill from trench backfill may have to be constructed at a flatter gradient than presented below.

Table No. 8, Slope Ratios for Temporary Excavations

Soil Type	OSHA Soil Type	Depth of Cut (feet)	Recommended Maximum Slope (Horizontal: Vertical) [1]
Silty Sand (SM), Sand with Gravel (SP), Gravelly Sand (GP)	C	0-10	1.5:1

[1] Slope ratio is assumed to be constant from top to toe of slope, with level adjacent ground.

Shallow excavations up to 4 feet bgs can be vertical. For steeper temporary construction slopes or deeper excavations, or unstable soil encountered during the excavation, shoring or trenches should be provided by the contractor to protect the workers in the excavation. Design recommendations for temporary shoring can be provided if necessary.

Surfaces exposed in slope excavations should be kept moist but not saturated to retard raveling and sloughing during construction. Adequate provisions should be made to protect the slopes from erosion during periods of rainfall. Surcharge loads, including construction materials, should not be placed within 5 feet of the unsupported slope edge. Stockpiled soils with a height higher than 6 feet will require greater distance from trench edges.

12.0 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. Testing should be performed to determine density and moisture of the compacted soils. 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.

13.0 CLOSURE

This report is prepared for the project described herein and is intended for use solely by Dudek, EVMWD 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 is 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.

14.0 REFERENCES

AMERICAN CONCRETE INSTITUTE (ACI), 2014, Building Code Requirements for Structural Concrete (ACI 318-14) and Commentary, October 2014.

CALIFORNIA BUILDING STANDARDS COMMISSION (CBSC), 2019, California Building Code (CBC).

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DAS, B.M., 2011, Principles of Foundation Engineering, Seventh Edition, published by Global Engineering, 2011.

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ROMANOFF, MELVIN, 1957, Underground Corrosion, National Bureau of Standards Circular 579, dated April 1957.

U.S. GEOLOGICAL SURVEY (USGS), 2022, National Water Information System: Web Interface (https://maps.waterdata.usgs.gov/mapper/index.html), accessed June 2022.

Appendix A

Field Exploration

APPENDIX A

FIELD EXPLORATION

Our field investigation included site reconnaissance and a subsurface exploration program consisting of drilling of test pits. During the site reconnaissance, the surface conditions were noted, and the test pits were marked at the locations approved by Mr. Charles (Chuck) Greely with Dudek. The approximate test pit locations were established in the field with reference to existing sites, street centerlines and other visible features. The locations should be considered accurate only to the degree implied by the method used.

Three exploratory test pits (TP-01 through TP-3) were excavated using a backhoe equipped with 24-inch-wide bucket to investigate the subsurface conditions on April 21, 2022. The test pits were excavated between 5.0 feet and 10.0 feet below the existing ground surface (bgs).

During exploration, 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.

No drive sample could be collected due to the presence of gravelly soil. Representative bulk samples were collected from selected depths and placed in large plastic bags for delivery to our laboratory.

Test pits were backfilled in lifts with excavated soil, tamped, and then wheel rolled at the surface using the bucket under the weight of the backhoe. If construction is delayed the ground surface at the test pit locations may settle over time. 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 pits, refer to Drawing Nos. A-1a and A-1b, *Unified Soil Classification and Key to Test Pit Log Symbols*. For Logs of Test Pits, see Drawings No. A-2 through A-4, *Logs of Test Pits*.

SOIL CLASSIFICATION CHART

MAJOR DIVISIONS			SYMBOLS GRAPH	SYMBOLS LETTER	TYPICAL DESCRIPTIONS
COARSE GRAINED SOILS	GRAVEL AND GRAVELLY SOILS	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
	MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	GRAVELS WITH FINES (APPRECIABLE AMOUNT OF FINES)		GM	SILTY GRAVELS, GRAVEL - SAND - SILT MIXTURES
				GC	CLAYEY GRAVELS, GRAVEL - SAND - CLAY MIXTURES
MORE THAN 50% OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	SAND AND SANDY SOILS	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
	MORE THAN 50% OF COARSE FRACTION PASSING ON NO. 4 SIEVE	SANDS WITH FINES (APPRECIABLE AMOUNT OF FINES)		SM	SILTY SANDS, SAND - SILT MIXTURES
				SC	CLAYEY SANDS, SAND - CLAY MIXTURES
FINE GRAINED SOILS	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
MORE THAN 50% OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE	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

FIELD AND LABORATORY TESTS

C	Consolidation (ASTM D 2435)
CL	Collapse Potential (ASTM D 4546)
CP	Compaction Curve (ASTM D 1557)
CR	Corrosion, Sulfates, Chlorides (CTM 643-99; 417; 422)
CU	Consolidated Undrained Triaxial (ASTM D 4767)
DS	Direct Shear (ASTM D 3080)
EI	Expansion Index (ASTM D 4829)
M	Moisture Content (ASTM D 2216)
OC	Organic Content (ASTM D 2974)
P	Permeablility (ASTM D 2434)
PA	Particle Size Analysis (ASTM D 6913 [2002])
PI	Liquid Limit, Plastic Limit, Plasticity Index (ASTM D 4318)
PL	Point Load Index (ASTM D 5731)
PM	Pressure Meter
PP	Pocket Penetrometer
R	R-Value (CTM 301)
SE	Sand Equivalent (ASTM D 2419)
SG	Specific Gravity (ASTM D 854)
SW	Swell Potential (ASTM D 4546)
TV	Pocket Torvane
UC	Unconfined Compression - Soil (ASTM D 2166)
	Unconfined Compression - Rock (ASTM D 7012)
UU	Unconsolidated Undrained Triaxial (ASTM D 2850)
UW	Unit Weight (ASTM D 2937)
WA	Passing No. 200 Sieve

BORING LOG SYMBOLS

DRILLING METHOD SYMBOLS

Auger Drilling

Mud Rotary Drilling

Dynamic Cone or Hand Driven

Diamond Core

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

UNIFIED SOIL CLASSIFICATION AND KEY TO TEST PIT LOG SYMBOLS

CONSISTENCY OF COHESIVE SOILS						
Descriptor	Unconfined Compressive Strength (tsf)	SPT Blow Counts	Pocket Penetrometer (tsf)	CA Sampler	Torvane (tsf)	Field Approximation
Very Soft	<0.25	< 2	<0.25	<3	<0.12	Easily penetrated several inches by fist
Soft	0.25 - 0.50	2 - 4	0.25 - 0.50	3 - 6	0.12 - 0.25	Easily penetrated several inches by thumb
Medium Stiff	0.50 - 1.0	5 - 8	0.50 - 1.0	7 - 12	0.25 - 0.50	Can be penetrated several inches by thumb with moderate effort
Stiff	1.0 - 2.0	9 - 15	1.0 - 2.0	13 - 25	0.50 - 1.0	Readily indented by thumb but penetrated only with great effort
Very Stiff	2.0 - 4.0	16 - 30	2.0 - 4.0	26 - 50	1.0 - 2.0	Readily indented by thumbnail
Hard	>4.0	>30	>4.0	>50	>2.0	Indented by thumbnail with difficulty

APPARENT DENSITY OF COHESIONLESS SOILS		
Descriptor	SPT N_{60}- Value (blows / foot)	CA Sampler
Very Loose	<4	<5
Loose	4- 10	5 - 12
Medium Dense	11 - 30	13 - 35
Dense	31 - 50	36 - 60
Very Dense	>50	>60

MOISTURE	
Descriptor	Criteria
Dry	Absence of moisture, dusty, dry to the touch
Moist	Damp but no visible water
Wet	Visible free water, usually soil is below water table

PERCENT OF PROPORTION OF SOILS	
Descriptor	Criteria
Trace (fine)/ Scattered (coarse)	Particles are present but estimated to be less than 5%
Few	5 to 10%
Little	15 to 25%
Some	30 to 45%
Mostly	50 to 100%

SOIL PARTICLE SIZE		
Descriptor		Size
Boulder		> 12 inches
Cobble		3 to 12 inches
Gravel	Coarse	3/4 inch to 3 inches
	Fine	No. 4 Sieve to 3/4 inch
Sand	Coarse	No. 10 Sieve to No. 4 Sieve
	Medium	No. 40 Sieve to No. 10 Sieve
	Fine	No. 200 Sieve to No. No. 40 Sieve
Silt and Clay		Passing No. 200 Sieve

PLASTICITY OF FINE-GRAINED SOILS	
Descriptor	Criteria
Nonplastic	A 1/8-inch thread cannot be rolled at any water content.
Low	The thread can barely be rolled, and the lump cannot be formed when drier than the plastic limit.
Medium	The thread is easy to roll, and not much time is required to reach the plastic limit; it cannot be rerolled after reaching the plastic limit. The lump crumbles when drier than the plastic limit.
High	It takes considerable time rolling and kneading to reach the plastic limit. The thread can be rerolled several times after reaching the plastic limit. The lump can be formed without crumbling when drier than the plastic limit.

CEMENTATION/ Induration	
Descriptor	Criteria
Weak	Crumbles or breaks with handling or little finger pressure.
Moderate	Crumbles or breaks with considerable finger pressure.
Strong	Will not crumble or break with finger pressure.

NOTE: This legend sheet provides descriptions and associated criteria for required soil description components only. Refer to Caltrans Soil and Rock Logging, Classification, and Presentation Manual (2010), Section 2, for tables of additional soil description components and discussion of soil description and identification.

UNIFIED SOIL CLASSIFICATION AND KEY TO TEST PIT LOG SYMBOLS

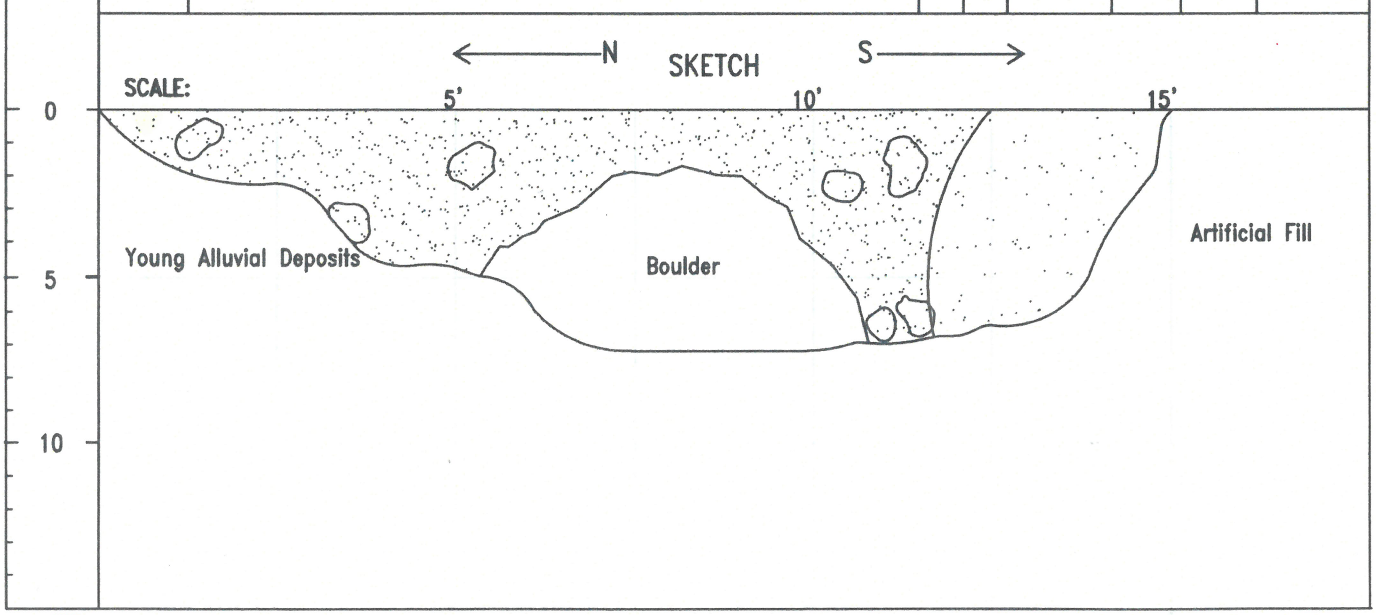
Log of Test Pit No. TP-01

Dates Drilled: 4/21/2022 Logged by: Catherine Nelson Checked By: Hashmi S. Quazi

Equipment: BACKHOE WITH 24" WIDE BUCKET Driving Weight and Drop: N/A

Ground Surface Elevation (ft): 1669 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 test pit and at the time of excavation. 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/FOOT	MOISTURE (%)	DRY UNIT WT. (pcf)	OTHER
0–8		YOUNG ALLUVIAL DEPOSITS/ARTIFICIAL FILL **Sand with Gravel (SP): medium to coarse grained, mostly gravel, cobbles, and boulders up to 8' maximum dimension, loose, moist, light grayish brown.** Interbedded very thin (1"-2") layers of silty/clayey sand,			- Native (0-5) - Fill (5-8)			CP, DS
5								
		End of test pit at 8 feet bgs due to refusal on boulder. No groundwater encountered Test Pit filled with soil and tamped with backhoe bucket on 04/21/22. * No ring samples due to refusal on gravel and cobbles.						



Log of Test Pit No. TP-02

Dates Drilled: 4/21/2022 | Logged by: Catherine Nelson | Checked By: Hashmi S. Quazi

Equipment: BACKHOE WITH 24" WIDE BUCKET | Driving Weight and Drop: N/A

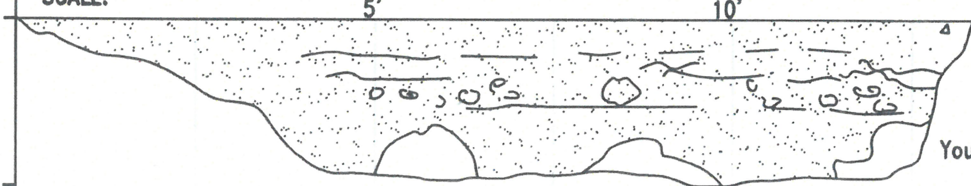
Ground Surface Elevation (ft): 1746 | Depth to Water (ft): NOT ENCOUNTERED

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 test pit and at the time of excavation. 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/FOOT	MOISTURE (%)	DRY UNIT WT. (pcf)	OTHER
		YOUNG ALLUVIAL DEPOSITS **Sand with Gravel (SP): fine to coarse grained, mostly gravel, cobbles, and boulders up to 4' maximum dimension, loose to medium dense, moist, light grayish brown to brown.** Interbedded with very thin (1" -2") layers of silty/clayey sand.		▓				PA
5		End of test pit at 5 feet bgs due to refusal on boulder. No groundwater encountered. Test Pit filled with soil and tamped with backhoe bucket on 04/21/22. * No ring samples due to refusal on gravel and cobbles.						

←N SKETCH S→

SCALE: 5' 10' 15'

0



Young Alluvial Deposits

5

10

Converse Consultants

Rice Canyon Reservoir Permanent Access Road and New Conduit West of Dale Ct and Lincoln Street
City of Lake Elsinore, Riverside County, California
For: Dudek

Project No. 21-81-268-01

Drawing No. A-3

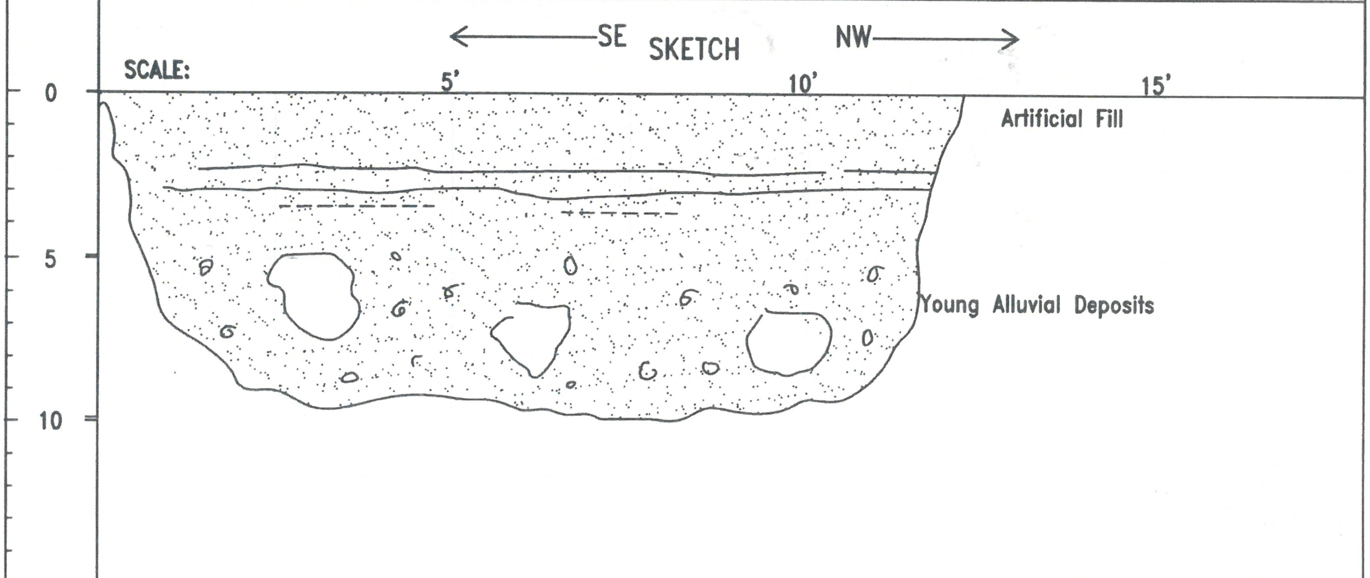
Log of Test Pit No. TP-03

Dates Drilled: 4/21/2022 | Logged by: Catherine Nelson | Checked By: Hashmi S. Quazi

Equipment: BACKHOE WITH 24" WIDE BUCKET | Driving Weight and Drop: N/A

Ground Surface Elevation (ft): 1763 | 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 test pit and at the time of excavation. 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/FOOT	MOISTURE (%)	DRY UNIT WT. (pcf)	OTHER
		ARTIFICIAL FILL: **Silty Sand (SM): medium to coarse grained, some gravel, cobbles up to 8" maximum dimension, medium dense, moist, brown to yellowish brown.**						
5		YOUNG ALLUVIAL DEPOSITS **Silty Sand /Gravelly Sand (SM/GP): medium to coarse grained, some gravel, cobbles and boulders up to 3' maximum dimension, loose to medium dense, moist, brown to light grayish brown.**						**CR**
10		End of test pit 10 feet bgs below ground level. No groundwater encountered. Test Pit filled with soil and tamped with backhoe bucket on 4/21/22. * No ring samples due to refusal on gravel and cobbles.						



Converse Consultants

Rice Canyon Reservoir Permanent Access Road and New Conduit West of Dale Ct and Lincoln Street
City of Lake Elsinore, Riverside County, California
For: Dudek

Project No. 21-81-268-01

Drawing No. A-4

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

Soil Corrosivity Test

One representative soil sample was tested to determine minimum electrical resistivity (wet condition), 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. The tests were performed by AP Engineering and Testing, Inc. (Pomona, CA) in accordance with Caltrans Tests 643, 422 and 417. Test results are presented in the following table.

Table No. B-1, Summary of Soil Corrosivity Test Results

Boring No.	Depth (feet)	pH	Soluble Sulfates (CA 417) (ppm)	Soluble Chlorides (CA 422) (ppm)	Min. Resistivity (CA 643) (Ohm-cm)
TP-03	2-10	7.8	24	19	19,079

Grain-Size Analyses

To assist in classification of soils, mechanical grain-size analysis was performed on one select sample in accordance with the ASTM Standard ASTM D6913 test method. Grain-size curve is shown in Drawing No. B-1, *Grain Size Distribution Result*.

Table No. B-2, Grain Size Distribution Test Results

Boring No.	Depth (ft)	Soil Classification	% Gravel	% Sand	%Silt	%Clay
TP-02	0-5	Sand with Gravel (SP)	21.0	74.8	4.2	

Maximum Density and Optimum Moisture Content

Laboratory maximum dry density-optimum moisture content relationship test was performed on one representative bulk soil sample. The test was conducted in accordance with the ASTM Standard D1557 test method. The test results are presented in Drawing No. B-2, *Moisture-Density Relationship Results,* and are summarized in the following table.

Table No B-3, Summary of Moisture-Density Relationship Results

Boring No.	Depth (feet)	Soil Description	Optimum Moisture (%)	Maximum Density (lb/cft)
TP-01	0-5	Sand with Gravel (SP), Brown	7.8	129.5

Direct Shear

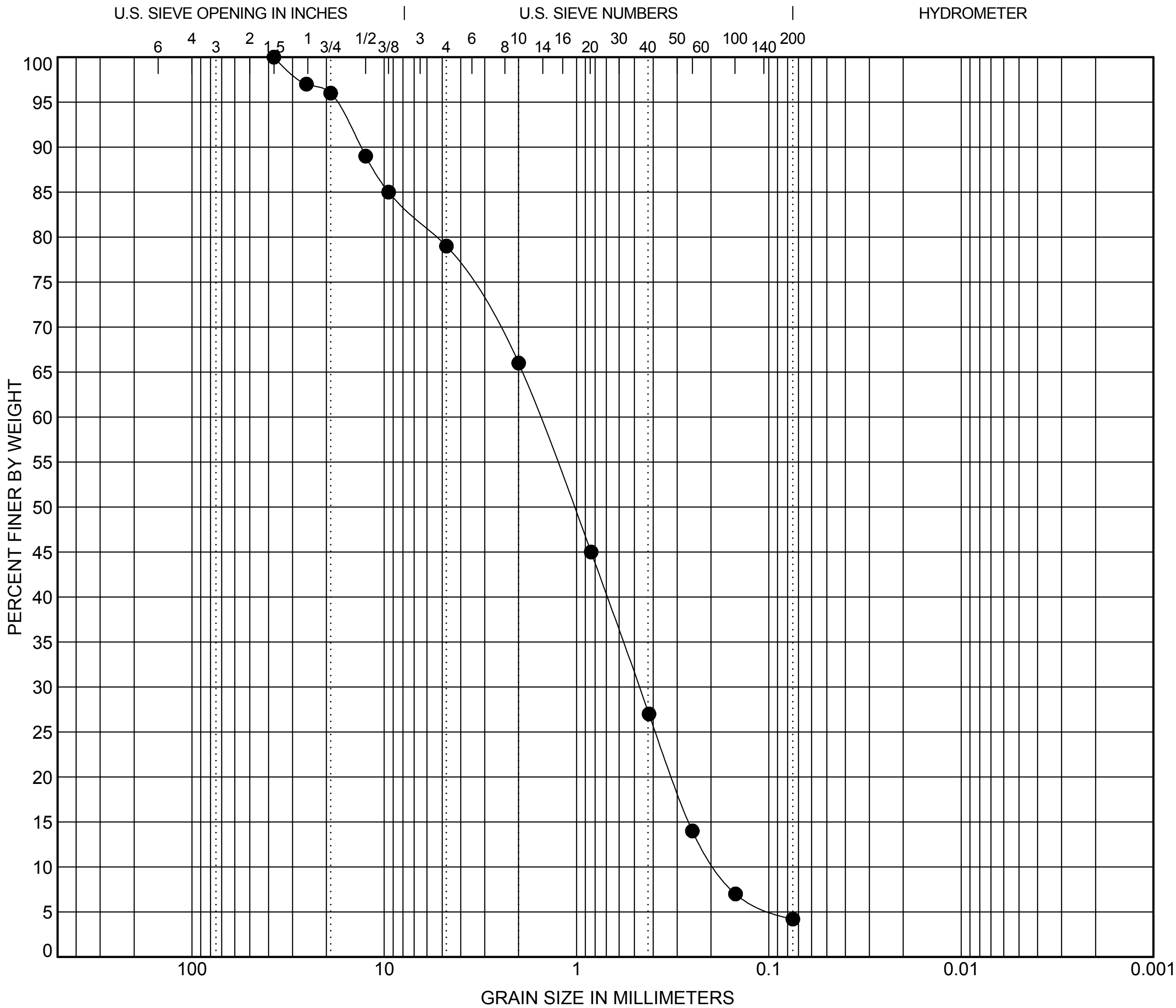
One remolded direct shear test was performed under soaked moisture condition in accordance with ASTM D3080. The samples were remolded to 90% of the laboratory maximum dry density. For the test, three samples contained in brass sampler rings were placed, one at a time, directly into the test apparatus and subjected to a range of normal loads appropriate for the anticipated conditions. The samples were then sheared at a constant strain rate of 0.02 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 data, including sample density and moisture content, see Drawing No. B-3*, Direct Shear Test Result,* and the following table.

Table No. B-4, Summary of Remolded Direct Shear Test Results

Boring No.	Depth (feet)	Soil Description	Peak Strength Parameters	
			Friction Angle (degrees)	Cohesion (psf)
TP-01	0-5	Sand with Gravel (SP)	32	120

Sample Storage

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



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	Test Pit No.	Depth (ft)	Description	LL	PL	PI	Cc	Cu
●	**TP-02**	**0-5**	**Sand with Gravel (SP)**				**0.77**	**8.39**

	Test Pit No.	Depth (ft)	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**TP-02**	**0-5**	**37.5**	**1.561**	**0.471**	**0.186**	**21.0**	**74.8**	**4.2**	

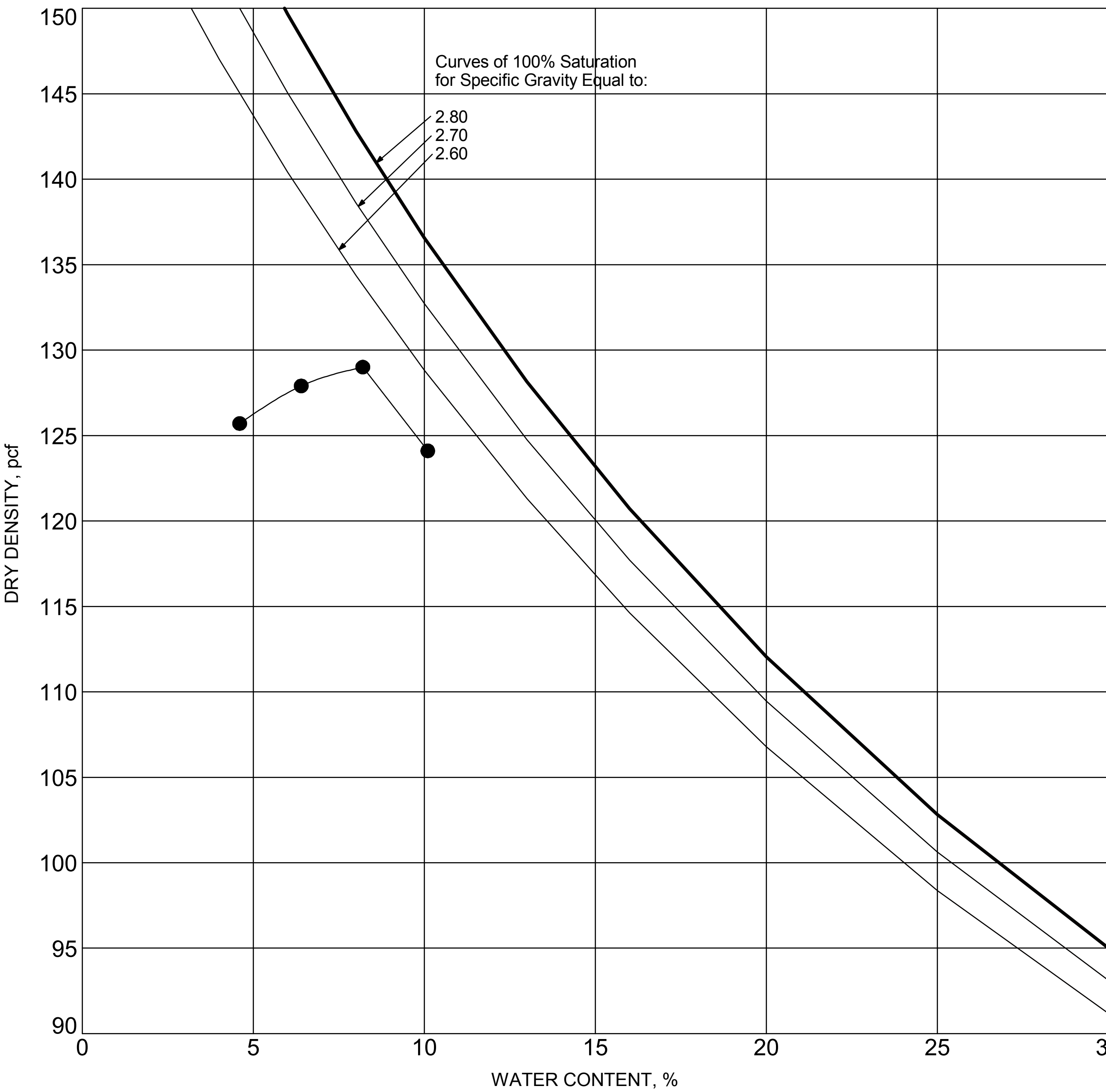
GRAIN SIZE DISTRIBUTION RESULTS

Converse Consultants

Rice Canyon Reservoir Permanent Access Road and New Conduit
West of Dale Ct and Lincoln Street
City of Lake Elsinore, Riverside County, California
For: Dudek

Project No. **21-81-268-01**

Drawing No. **B-1**



SYMBOL	TEST PIT NO.	DEPTH (ft)	DESCRIPTION	ASTM TEST METHOD	OPTIMUM WATER, %	MAXIMUM DRY DENSITY, pcf
●	TP-01	0-5	Sand with Gravel (SP), Brown	D1557 Method B	7.8	129.5

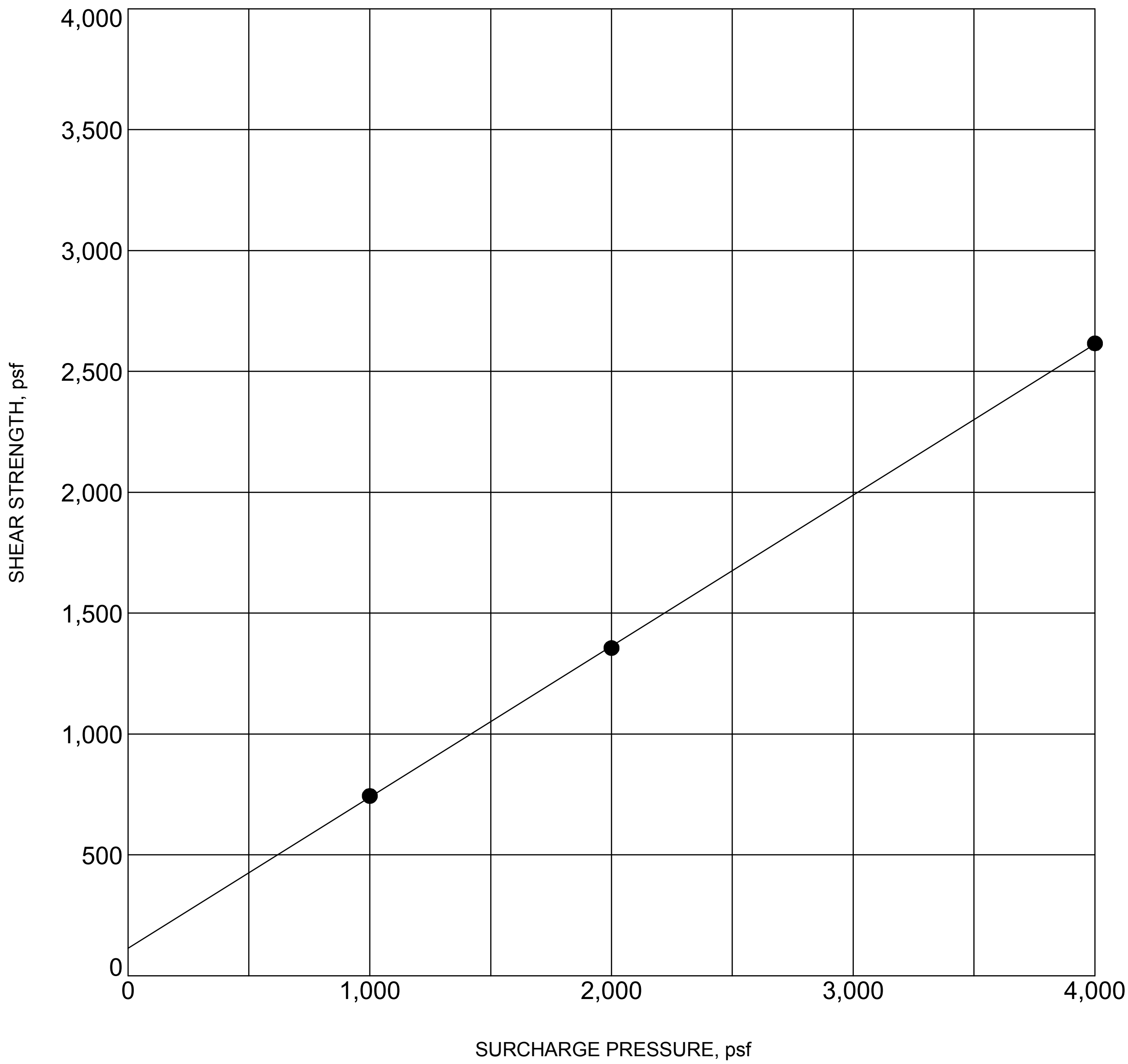
MOISTURE-DENSITY RELATIONSHIP RESULTS

Converse Consultants

Rice Canyon Reservoir Permanent Access Road and New Conduit
West of Dale Ct and Lincoln Street
City of Lake Elsinore, Riverside County, California
For: Dudek

Project No.
21-81-268-01

Drawing No.
B-2



TEST PIT NO. :	**TP-01**	DEPTH (ft) :	**0-5**
DESCRIPTION :	**Sand with Gravel (SP): some silt /clayey sand**		
COHESION (psf) :	**120**	FRICTION ANGLE (degrees):	**32**
MOISTURE CONTENT (%) :	**7.8**	DRY DENSITY (pcf) :	**129.5**

NOTE: Ultimate Strength.

DIRECT SHEAR TEST RESULTS

Converse Consultants

Rice Canyon Reservoir Permanent Access Road and New Conduit
West of Dale Ct and Lincoln Street
City of Lake Elsinore, Riverside County, California
For: Dudek

Project No.
21-81-268-01

Drawing No.
B-3