SECTION H

GEOTECHNICAL REPORT

LAKE GREGORY REGIONAL PARK SITEWIDE SEDIMENT MANAGEMENT PROJECT

FOR

LAKE GREGORY REGIONAL PARK
24171 LAKE DRIVE, CRESTLINE, CA 92325

PROJECT NO.: 30.30.0169

GEOTECHNICAL INVESTIGATION REPORT

Lake Gregory Regional Park Sitewide Sediment Management
City of Crestline, San Bernardino County, California

Converse Project No. 23-81-115-01

Prepared For:

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August 11, 2023

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Subject: **GEOTECHNICAL INVESTIGATION REPORT**
Lake Gregory Regional Park Sitewide Sediment Management
City of Crestline, County of San Bernardino, California
Converse Project No. 23-81-115-01

Dear Mr. Muli:

Converse Consultants (Converse) is pleased to submit our Geotechnical Investigation Report for the evaluation of the stockpile soils at the Lake Gregory Regional Park Sitewide Sediment Management Project, located in the City of Crestline, San Bernardino County, California. This report was prepared in accordance with our proposal dated January 23, 2023, and your Subconsultant Agreement for Professional Services dated June 9, 2023.

We appreciate the opportunity to be of service to J. Smith/T. Muli and the County of San Bernardino. Should you have any questions, please do not hesitate to contact us at 909-474-2847.

CONVERSE CONSULTANTS

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

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PROFESSIONAL CERTIFICATION

This report has been prepared by the individuals whose seals and signatures appear herein.

The findings, recommendations, specifications, or professional opinions contained in this report were prepared in accordance with generally accepted professional engineering, 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 geotechnical investigation report was prepared by Converse for the Lake Gregory Regional Park Sitewide Sediment management project, located in the City of Crestline, San Bernardino County, California. The approximate location of the proposed project is shown in Figure No. 1, *Approximate Project Location Map.*

The purpose of this investigation was to evaluate dirt stockpiles and to provide geotechnical recommendations for design and construction of the project.

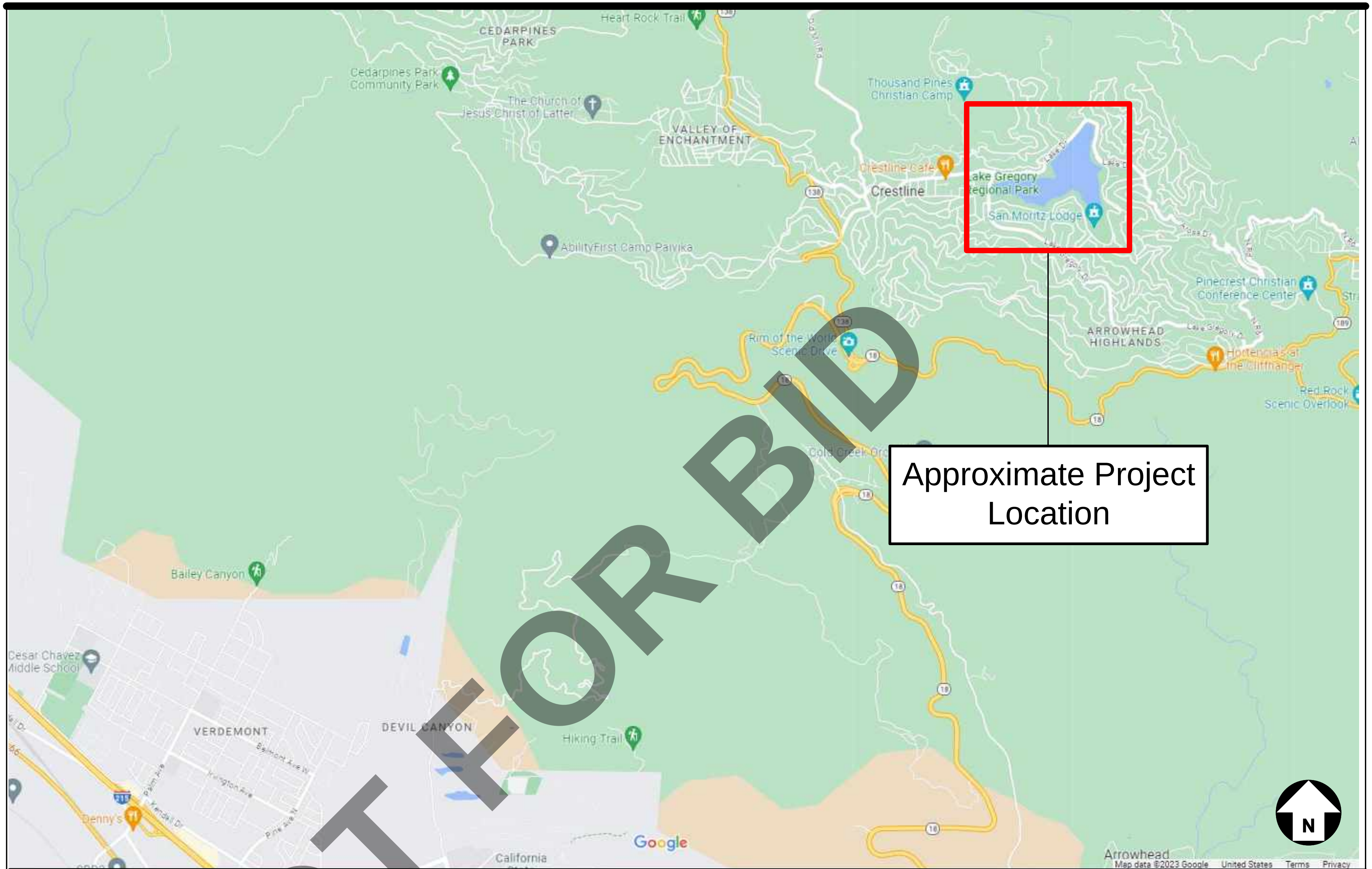
This report was prepared for the project described herein and is intended for use solely by J. Smith/T. Muli, and their authorized agents. This report may be made available to the prospective bidders for bidding purposes. This report may not contain sufficient information for use by others and/or other purposes.

2.0 PROJECT DESCRIPTION

It is our understanding that the project will include an embankment fill which will be constructed at a minimum slope ratio of 3H:1V (H = horizontal and V = Vertical). The embankment fill height will range from 5 feet to 20 feet; with the majority in the range between 10 feet and15 feet. The dirt stockpiles within the park will be used to build the embankment. The only other potential structures would be reinforced concrete pipe (RCP) storm drain, retaining walls, and rip-rap slope armoring.

3.0 SITE DESCRIPTION

Gregory Lake is an 84-acre reservoir located in the San Bernardino National Forest of the San Bernardino Mountains in the City of Crestline, San Bernardino County, California. On the western shore of the lake is a u-shaped swimming beach that is the focus of the proposed beach improvements (Photograph No. 1). The lake is bounded by Lake Drive to the north and northeast, San Moritz Drive to the southeast and south, and Lake Gregory Drive to the west. The Lake Gregory Dam is located in the northeast section of the lake with Camp Switzerland and stockpile to the north of the dam (Photograph No. 2). The library stockpile is located to the west of the swimming area. (Photograph No. 3).



Project: Lake Gregory Regional Park Sitewide Sediment Management

Location: City of Crestline, San Bernardino County, California

For: J. Smith and T. Muli

Approximate Project Location Map

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Figure No.
1

Photograph No. 1: Lake Gregory Swimming Area, facing east.

Photograph No. 2: Camp Switzerland with old stockpile in the foreground and the new stockpile in the back, facing east.

Photograph No. 3: Library with library stockpile in the foreground consisting of the steam bed, facing south.

4.0 SCOPE OF WORK

The scope of Converse's investigation is described in the following sections.

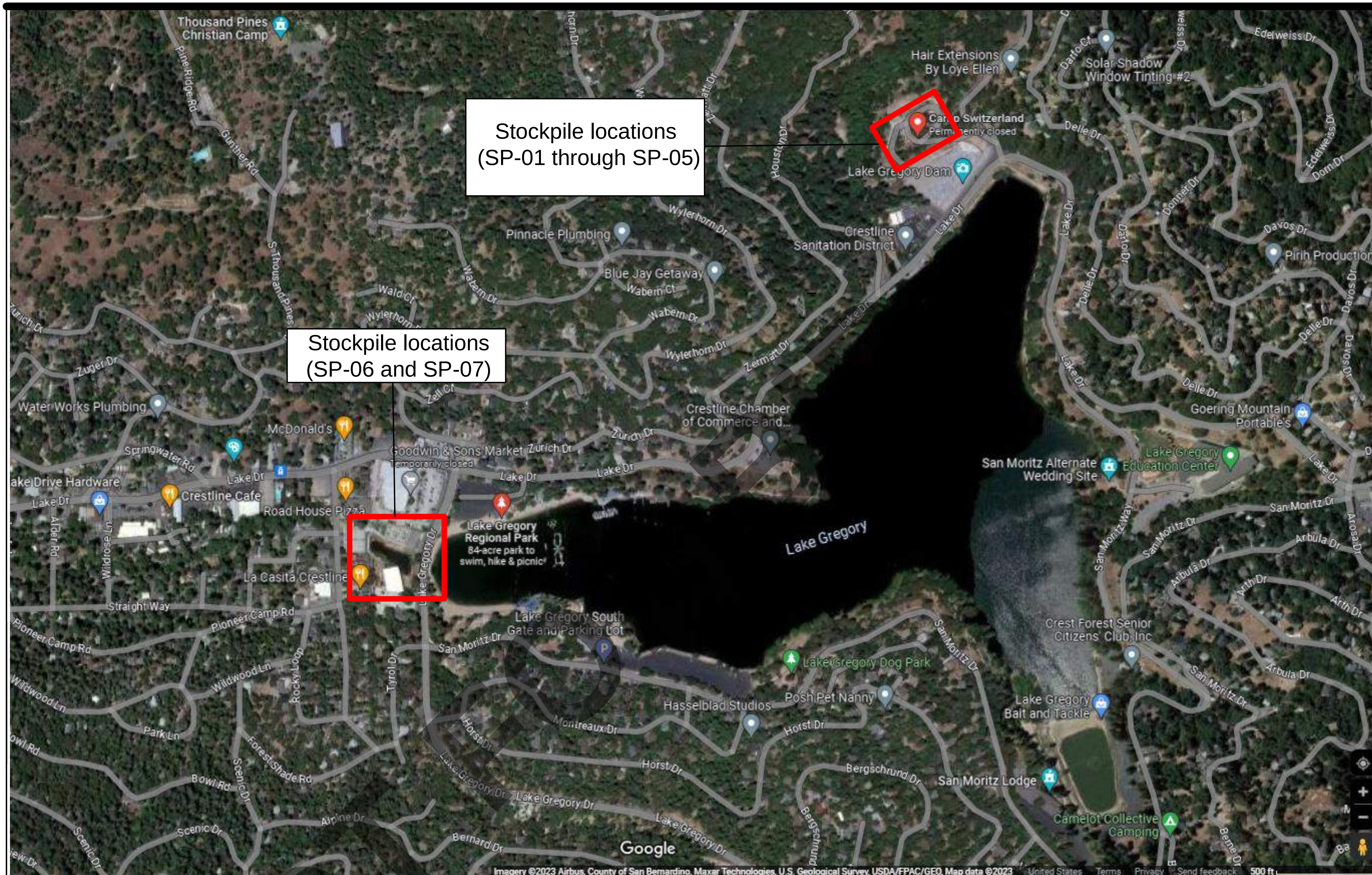
4.1 Project Set-up

The project set-up consisted of the following tasks.

- Review of plans and data relevant to the project.
- Conducted a site reconnaissance to view the area of the proposed improvements and collect representative bulk samples from the stockpiles.

4.2 Soil Sampling

Three bulk samples (SP-01 through SP-03) from Camp Switzerland old stockpile, two bulk samples (SP-04 and SP-05) from Camp Switzerland new stockpile and two bulk samples (SP-06 and SP-07) from Library stockpile were collected to perform laboratory testing. The approximate locations of the stockpiles are shown on Figure Nos. 2a through 2c, *Approximate Stockpile Locations Map*.



Project: Lake Gregory Regional Park Sitewide Sediment Management

Location: City of Crestline, San Bernardino County, California

For: J. Smith and T. Muli

Approximate Stockpile Locations Map

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Figure No.
2a

Project: Lake Gregory Regional Park Sitewide Sediment Management

Location: City of Crestline, San Bernardino County, California

For: J. Smith and T. Muli

Approximate Stockpile Locations Map

Project No.
23-81-115-01

Converse Consultants

Figure No.
2b

Project: Lake Gregory Regional Park Sitewide Sediment Management

Location: City of Crestline, San Bernardino County, California

For: J. Smith and T. Muli

Approximate Stockpile Locations Map

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Figure No.
2c

4.3 Laboratory Testing

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

- *In-situ* moisture contents (ASTM D2216)
- Organic content (ASTM D2974, Methods A and C)
- Expansion index (ASTM D4829)
- Soils corrosivity (CTM 643, 422, 417, 532)
- Grain size analysis (ASTM D6913)
- Maximum dry density and optimum-moisture content (ASTM D1557)
- Remolded direct shear (ASTM D3080)

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

4.4 Analysis and Report Preparation

Data obtained from the laboratory testing program was assembled and evaluated. Geotechnical analyses of the compiled data were performed, followed by the preparation of this report to present our findings, conclusions, and recommendations for the project.

5.0 STOCKPILE CONDITIONS

To describe stockpile conditions, a summary of the relevant laboratory test results of each stockpile is presented in Table No. 1, *Summary of Relevant Laboratory Test Results*.

5.1 Camp Switzerland Old Stockpile

Based on the laboratory test results, the stockpile' materials primarily consist of a mixture of sand, silt and gravel with organic material. The expansion indices of the stockpile' soils were 0, corresponding to very low expansion potential. Test results indicate soils are non-corrosive.

5.2 Camp Switzerland New Stockpile

Based on the laboratory test results, the stockpile' materials primarily consist of a mixture of sand, silt and gravel. The expansion index of the stockpile' soils was 0, corresponding to very low expansion potential.

Table No. 1, Summary of Relevant Laboratory Test Results

Stockpile	Sample	USCS Soil Classification	EI	Corrosivity				Max. Dry Density (pcf)	Optimum Moisture Content (%)	Friction Angle (Deg.)	Cohesion (psf)
				pH	Soluble Sulfates (ppm)	Soluble Chlorides (ppm)	Min. Resistivity (ohm-cm)				
Camp Switzerland (Old)	SP-01	Sand with Silt and Gravel (SP-SM)	0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
	SP-02	Sand (SP)	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
	SP-03	Silty Sand (SM)	0	7.7	16	17	20,081	N/A	N/A	N/A	N/A
Camp Switzerland (New)	SP-04	Silty Sand (SM)	0					118	11	34	50
	SP-05	Silty Sand with Gravel (SM)	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Library	SP-06	Sand with Gravel (SP)	N/A	7.6	19	18	26,795	N/A	N/A	N/A	N/A
	SP-07	Sand with Gravel (SP)	N/A	N/A	N/A	N/A	N/A	120	12	33	50

Note: EI = Expansion Index; Friction Angle and Cohesion values are based on remolded direct shear.

5.3 Library

Based on the laboratory test results, the stockpile' materials primarily consist of a mixture of sand and gravel. Test results indicate soils are non-corrosive.

6.0 FAULTING AND SEISMICITY

The location of the site with respect to active faults and associated seismicity is discussed below.

6.1 Faulting

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

The project site is not located within a currently mapped State of California; however, it is located within a San Bernardino County Earthquake Fault Zone (SBC 2010b and CGS, 2007). 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 generalized site coordinates 34.2423 N, 117.2751 W, 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
Cleghorn	3.65	strike slip	25	3.0	6.80
S. San Andreas	6.48	strike slip	548	n/a	8.18
North Frontal (West)	8.31	reverse	50	1.0	7.20
San Jacinto	11.94	strike slip	241	n/a	7.88
Cucamonga	16.87	thrust	28	5.0	6.70
Helendale-So Lockhart	37	strike slip	114	0.6	7.40
San Jose	40.89	strike slip	20	0.5	6.70
North Frontal (East)	44.92	thrust	27	0.5	7.00
Sierra Madre	44.93	reverse	57	2.0	7.20
Sierra Madre Connected	44.93	reverse	76	2.0	7.30
Chino, alt 2	49.26	strike slip	29	1.0	6.80
Chino, alt 1	49.32	strike slip	24	1.0	6.70
Clamshell-Sawpit	52.65	reverse	16	0.5	6.70

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

6.2 CBC Seismic Design Parameters

Seismic parameters based on the 2022 California Building Code (CBSC, 2019) and ASCE 7-22 are provided in the following table. These parameters were determined using the generalized coordinates (34.2423N, 117.2751W) and ASCE 7 Hazard online tool.

Table No. 3, CBC Seismic Design Parameters

Seismic Parameters	
Site Coordinates	34.2423 N, 117.2751 W
Site Class	D*
Risk Category	II
Mapped Short period (0.2-sec) Spectral Response Acceleration, S_s	2.56g
Mapped 1-second Spectral Response Acceleration, S_1	0.86g
MCE 0.2-sec period Spectral Response Acceleration, S_{MS}	2.21g
MCE 1-second period Spectral Response Acceleration, SM_1	2.33g
Design Spectral Response Acceleration for short period S_{DS}	1.48g
Design Spectral Response Acceleration for 1-second period, S_{D1}	1.55g
Site Modified Maximum Peak Ground Acceleration, PGA_M	0.770g

** Stiff Soil Classification*

6.3 Secondary Effects of Seismic Activity

In general, secondary effects of seismic activity include surface fault rupture, soil liquefaction, landslides, lateral spreading, tsunamis, seiches, and earthquake-induced flooding. The site-specific potential for each of these seismic hazards is discussed in the following sections.

Surface Fault Rupture: The project site is not located within a State of California designated earthquake fault zone; However, it is located within a San Bernardino County designated earthquake fault zone (CGS, 2007; SBC, 2021b). Based on the dense nature of the underlying bedrock 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 project site is not located within an area designated as a liquefaction risk by the State of California and San Bernardino County (CGS, 2007; SBC, 2021b), therefore the risk for liquefaction is negligible.

Landslides: Seismically induced landslides and other slope failures are common occurrences during or soon after earthquakes. The proposed project is not located within a designated State of California or San Bernardino County landslide hazard zone (CGS, 2007; SBC, 2021b).

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 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 characterized by near-vertical cracks with predominantly horizontal movement of the soil mass involved. Due to the low risk of liquefaction 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 project 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 project site is located near and within Lake Gregory, an enclosed body of water and is at risk of flooding due to on and off-site seiches.

Earthquake-Induced Flooding: Dams or other water-retaining structures may fail as a result of large earthquakes, resulting in flooding. The project site is not located within a State of California or San Bernardino County designated dam inundation area (DSOD, 2021 and SBC, 2021a). The risk of earthquake-induced flooding at the project site due to failure of offsite dams is considered low. However, the Camp Switzerland stockpiles are within a State of California or San Bernardino County designated dam inundation area (DSOD, 2021 and SBC, 2021a). The risk of earthquake-induced flooding at the project Camp Switzerland stockpiles due to failure of the Lake Gregory Dam, No. 1803-3; National Dam ID:CA00224 dams is considered High.

7.0 LABORATORY TEST RESULTS

Laboratory testing was performed to determine the physical and chemical characteristics and engineering properties of the stockpiles' soils. Current physical test results are included in Appendix A, *Laboratory Testing Program*. Discussions of the various test results are presented below.

7.1 Physical Testing

- In-Place Moisture – *In-place* moisture content of the stockpiles' soils were determined in accordance with ASTM Standard D2216. Moisture of the soils content ranged from 5 to 60 (saturated sample) percent.
- *Expansion Index (EI)* - Three representative bulk soil samples were tested to evaluate the expansion potential stockpiled soils in accordance with ASTM Standard D4829. The test results indicated expansion indices of 0, corresponding to very low expansion potential.
- Grain Size Analyses (PA) – Six 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 Nos. B-1a and B-1b, *Grain Size Distribution Results* in Appendix A, *Laboratory Testing Program*.
- Maximum Dry Density and Optimum Moisture Content (CP) – Typical moisture-density relationship tests were conducted on one representative sample in accordance with ASTM D1557. The results are presented in Drawing No. B-2, *Moisture-Density Relationship Results* in Appendix A, *Laboratory Testing Program*. The laboratory maximum dry densities were 118.0 and 120 pounds per cubic feet (pcf) and the optimum moisture contents of 11.0 and 12 percent, respectively.
- Remolded Direct Shear (DS) – Two direct shear tests were performed in accordance with ASTM Standard D3080 on samples remolded to 90% of the laboratory maximum dry density. The results of the direct shear tests are presented in Drawing Nos. B-3 and B-4, Direct Shear Test Results in Appendix A, Laboratory Testing Program.

7.2 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 this test was to determine the corrosion potential of stockpiled soils when placed in contact with common construction 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 A, *Laboratory Testing Program and* are summarized below.

- The pH measurements of the samples tested were 7.6 and 7.7.
- The sulfate contents of the samples tested were 16 and 19 ppm (0.016 and 0.019 percent by weight).
- The chloride concentrations of the samples tested were 17 and 18 ppm.
- The minimum electrical resistivities when saturated were 20,081 and 26,795 ohm-cm.

8.0 EARTHWORK AND SITE GRADING RECOMMENDATIONS

Recommendations for earthwork are presented in the following subsections.

8.1 General

This section contains our general recommendations regarding earthwork for the Lake Gregory Regional Park Sitewide Sediment Management project.

These recommendations are based on the results of our site visit and laboratory testing as well as 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 observation of the actual field conditions during remedial grading.

Prior to the start of construction, all underground existing utilities and appurtenances 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. 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 existing structures (if any), debris, deleterious material and surficial soils containing roots and perishable materials should be stripped and removed from the project site. Deleterious material, including organics, organic disturbed soils, 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.

8.2 Overexcavation

The overexcavation for different improvements' areas are presented below.

Table No. 4, Overexcavation Depths

Location	Condition	Minimum Excavation Depth
12:1 Slope Areas	Cut Area	6 inches
	Fill Area	12 inches
Berm Areas (3:1 Slope Areas)	Fill Area	12 inches
Walls	Footings	18 inches below footing bottom

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

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

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

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

8.3 Engineered Fill

No fill should be placed until excavations and/or natural ground preparation has been observed by the geotechnical consultant. Stockpiled and excavated soils should be processed, including removal of 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 8 inches in largest dimension.
- Rocks larger than 4 inches 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.
- Contain less than 30 percent by weight retained in 3/4-inch sieve.
- Contain less than 40 percent fines (passing #200 sieve).

Based on laboratory testing results, on-site soils may be suitable as fill materials, considering subsection 8.4, *Organic Content*.

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 a geotechnical representative prior to delivery to the site.

8.4 Organic Content

Based on the laboratory test results, organic content of the stockpiled soils is 11.5 percent. Fill materials should contain no more than 4 percent overall organic. Therefore, these soils consisting of organic content should be blended with natural soils/clean imported soils (free of organic content) or removed from the site prior to use as fill materials.

The partially organic soils can be blended with the on-site natural soils at a ratio of 4 to 1 (natural soils/clean imported soils to partially organic soils) and placed as compacted fill, provided they are completely mixed during fill placement. The type of equipment and method of placement; blending and mixing of the partially organic materials with onsite natural soils or clean imported soils to be utilized by the grading contractor, should be reviewed and accepted by the geotechnical consultant prior to implementation. The testing frequency for verifying the percent organic content should be established by the geotechnical consultant prior to fill placement once the method of blending, placement, and mixing of the partially organic materials with onsite natural soils or clean imported soils.

8.5 Compacted Fill Placement

All surfaces to receive structural fills should be scarified to a depth of 12 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 density as determined by ASTM Standard D1557 test method unless a higher compaction is specified herein.

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.

8.6 Backfill Recommendations Behind Walls

Compaction of backfill adjacent to retaining walls, which may be proposed, can produce excessive lateral pressures. Improper types and locations of compaction equipment and/or compaction techniques may damage the walls. The use of heavy compaction equipment should not be permitted within a horizontal distance of 5 feet from the wall. Backfill behind any structural walls within the recommended 5-foot zone should be compacted using lightweight construction equipment such as handheld compactors to avoid overstressing the walls.

9.0 DESIGN RECOMMENDATIONS

Design recommendations for this project is presented below:

9.1 Soil Design Parameters

Soil design parameters are presented below.

Table No. 5, Design Parameters

Parameter	Value
Allowable bearing capacity	1500 psf
Active Earth Pressure	35 psf
Passive Resistance	250 psf

Retaining walls may be supported on continuous footings. The minimum continuous footings' width and depth of embedment should be 15 and 18 inches, respectively.

The net allowable bearing value indicated above is 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.

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

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 1,500 psf for compacted fill.

9.2 Fill Slope Stability

The proposed 3:1 (H:V) berm slope will be constructed with on-site recycled Camp Switzerland fill material which is comprised of sand, silt and gravel.

The anticipated global stability of the proposed slope (embankment and cut slopes) under static and pseudo-static conditions were evaluated using the Slide 9.023 software (RocScience, 2022). Pseudostatic analyses were performed using a seismic coeffiecient of 0.2g.

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

The relevant soil parameters for the proposed slope including unit weight, friction angle, cohesion was derived from field and laboratory test data, are presented in Table No. 6, *Soil Parameters Used for Slope Stability Analyses.*

Table No. 6, Soil Parameters Used for Slope Stability Analyses

Slope	Conditions	Unit Weight (pcf)	Internal Friction Angle (Degree)	Cohesion (psf)
Embankment	Dry (Static and Pseudo-Static)	116	33	0
	Fully Submerged (Static)	116	33	0
	Fully Submerged (Pseudo-Static)	116	34	50
Note: For fully submerged pseudo-static conditions, peak remolded direct shear test results were used. From a geotechnical perspective this is acceptable.				

The results of the analyses are presented in Table No. 7, *Factor of Safety Against Slope Failure* and in Appendix B, *Slope Stability Analysis Results*.

Table No. 7, Factors of Safety Against Slope Failure

Slope	Condition	Slope Ratio	Max. Slope Height (feet)	Static FOS	Required Min. FOS	Remarks
Embankment	Dry-Static	3H:1V	20	2.070	1.3	Stable
	Dry-PseudoStatic			1.215	1.1	Stable
	Fully Submerged-Static			2.070	1.5	Stable
	Fully Submerged-PseudoStatic			1.104	1.1	Stable
Note: H = Horizontal, V = Vertical						

9.3 Retaining Walls Drainage

The recommended lateral earth pressure values do not include lateral pressures due to hydrostatic forces. Therefore, wall backfill should be free draining and provisions should be made to collect and dispose of excess water that may accumulate behind earth retaining structures. Behind wall drainage may be provided by free-draining gravel surrounded by synthetic filter fabric or by prefabricated, synthetic drain panels or weep holes. In either case, drainage should be collected by perforated pipes and directed to a sump, storm drain, or other suitable location for disposal. We recommend drain rock should consist of durable stone having 100 percent passing the 1-inch sieve and less than 5 percent passing the No. 4 sieve. Synthetic filter fabric should have an equivalent opening size (EOS), U.S. Standard Sieve, of between 40 and 70, a minimum flow rate of 110 gallons per minute per square foot of fabric, and a minimum puncture strength of 110 pounds.

10.0 CONSTRUCTION RECOMMENDATIONS

Temporary sloped excavation recommendations are presented in the following sections.

10.1 General

Prior to the start of construction, all existing underground utilities (if any) 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.

Excavations near existing structures 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.

10.2 Permanent Fill Slopes

Berm fill slopes should be constructed with slope ratios no steeper than 3:1 (H:V). Fill slopes should be constructed on compacted fill prepared in accordance with Section 8.5, *Compacted Fill Placement*.

Fill slopes should be properly compacted out to the slope face. This may be achieved by either overbuilding then cutting back to the compacted core, frequent backrolling, or by utilizing other methods that meet the intent of the project specifications. The fill slope face should be track rolled to achieve compaction.

11.0 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.

12.0 CLOSURE

This report is prepared for the project described herein and is intended for use solely by J. Smith and T. Muli and their authorized agents, to assist in the development 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. 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 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.

13.0 REFERENCES

AMERICAN SOCIETY OF CIVIL ENGINEERS (ASCE), 2022, Minimum Design Loads and Associated Criteria for Buildings and Other Structures, SEI/ASCE Standard No. 7-22.

CALIFORNIA BUILDING STANDARDS COMMISSION (CBSC), 2022, California Building Code (CBC).

CALIFORNIA GEOLOGICAL SURVEY (CGS), 2007, Fault-Rupture Hazard Zones in California, Alquist-Priolo Earthquake Faulting Zoning Act with Index to Earthquake Fault Zone Maps, Special Publication 42, revised 2007.

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

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SAN BERNARDINO COUNTY, 2010a, San Bernardino County General Plan Hazard Overlays, Map Sheet FH22B, scale 1:14,400, dated March 9, 2010.

SAN BERNARDINO COUNTY, 2010b, San Bernardino County General Plan Geologic Hazard Overlays, Map Sheet FH22C, scale 1:14,400, dated March 9, 2010.

U.S. GEOLOGICAL SURVEY (USGS), 2008, 2008 National Seismic Hazard Maps (https://earthquake.usgs.gov/cfusion/hazfaults_2008_search), accessed July 2023.

Appendix A

Laboratory Testing Program

APPENDIX A

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. The following is a summary of the various laboratory tests conducted for this project.

In-Place Moisture Content

The stockpiles' soils were tested to evaluate the *In-place* moisture content in accordance with ASTM Standard D2216. The test result is presented in the following table.

Table No. A-1, In-place Moisture Content Results

Sample No.	Depth (ft)	Soil Classification	Moisture Content (%)
SP-01	0.0-2.0	Sand with Silt and Gravel (SP-SM)	10.0
SP-02	0.0-2.0	Sand (SP)	5.0
SP-03	0.0-2.0	Silty Sand (SM)	19.0
SP-04	0.0-2.0	Silty Sand (SM)	9.0
SP-05	0.0-2.0	Silty Sand with Gravel (SM)	14.0
SP-06	0.0-2.0	Sand with Gravel (SP)	*60.0
SP-07	0.0-2.0	Sand with Gravel (SP)	6.0

**Please note SP-06 has a high moisture content due to saturated sample.*

Organic Content

Test was performed on one select sample in accordance with the ASTM Standard D2974 test, Methods A and C. Test result is summarized in the table below.

Table No. A-2, Summary of Organic Content Test Result

Sample No.	Depth (feet)	Soil Description	Total Organic Content (%)
SP-03	0.0-1.3	Silty Sand (SM)	11.5

Expansion Index (EI)

Three representative bulk samples were tested to evaluate the expansion potential of materials collected from the stockpiled soils in accordance with ASTM D4829 Standard. The test result is presented in the following table.

Table No. A-3, Expansion Index Test Results

Sample No.	Depth feet)	Soil Description	Expansion Index	Expansion Potential
SP-01	0.0-2.0	Sand with Silt and Gravel (SP-SM)	0	Very Low
SP-03	0.0-2.0	Silty Sand (SM)	0	Very Low
SP-04	0.0-2.0	Silty Sand (SM)	0	Very Low

Soil Corrosivity (CR)

Two representative soil samples were tested to determine minimum electrical resistivity, pH, and chemical content, including soluble sulfate and chloride concentrations. The purpose of this test was to determine the corrosion potential of stockpiled soils when placed in contact with common construction materials. The test was performed by AP Engineering and Testing, Inc. (Pomona, CA) in accordance with Caltrans Test Methods 643, 422 and 417. Test results are presented in the following table.

Table No. A-4, Soil Corrosivity Test Results

Sample No.	Depth (feet)	pH	Soluble Sulfates (CA 417) (ppm)	Soluble Chlorides (CA 422) (ppm)	Min. Resistivity (CA 643) (Ohm-cm)
SP-03	0.0-2.0	7.7	16	17	20,081
SP-06	0.0-2.0	7.6	19	18	26,795

Grain-Size Analyses (PA)

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

Table No. A-5, Grain Size Distribution Test Results

Sample No.	Depth (ft)	Soil Classification	% Gravel	% Sand	%Silt	%Clay
SP-01	0.0-2.0	Sand with Silt and Gravel (SP-SM)	18.0	72.9	9.1	
SP-02	0.0-2.0	Sand (SP)	14.0	82.0	4.0	
SP-04	0.0-2.0	Silty Sand (SM)	11.0	68.9	20.1	
SP-05	0.0-2.0	Silty Sand with Gravel (SM)	19.0	62.7	18.3	
SP-06	0.0-2.0	Sand with Gravel (SP)	19.0	62.7	18.3	
SP-07	0.0-2.0	Sand with Gravel (SP)	18.0	81.1	0.9	

Maximum Dry Density and Optimum Moisture Content (CP)

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

Table No A-6, Summary of Moisture-Density Relationship Results

Sample No.	Depth (feet)	Soil Description	Optimum Moisture (%)	Maximum Density (lb./cft)
SP-04	0.0-2.0	Silty Sand (SM), Dark Brown	11.0	118.0
SP-07	0.0-2.0	Sand with Gravel (SP), Brown	12.0	120.0

Direct Shear (DS)

Two direct shear tests were performed on soil samples (remolded to 90 percent of the maximum dry density and optimum moisture content) under soaked moisture conditions, in accordance with the ASTM D3080 method. For each test, three samples contained in a brass sampler ring 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 results, see Drawing Nos. A-3 and A-4, *Direct Shear Test Results,* and in the following table.

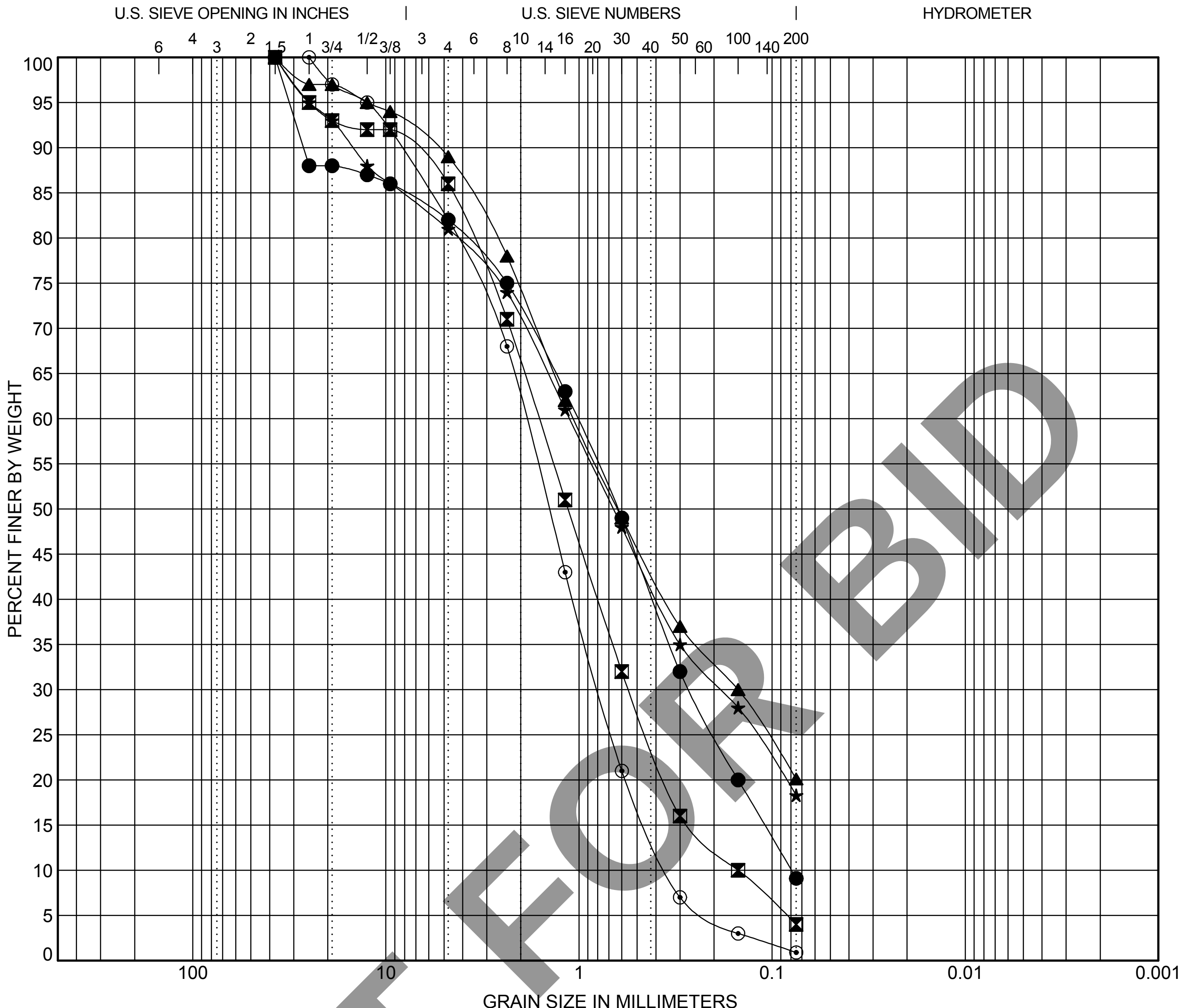
Table No. A-7, Direct Shear Test Results

Sample No.	Depth (feet)	Soil Description	Ultimate Strength Parameters	
			Friction Angle (degrees)	Cohesion (psf)
*SP-04	0.0-2.0	Silty Sand (SM)	34	50
*SP-07	0.0-2.0	Sand with Gravel (SP)	33	50

(*Remolded to 90% of laboratory maximum dry density.)

Sample Storage

Soil samples currently stored in our laboratory will be discarded thirty 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
●	**SP-01**	**0-2**	**SAND WITH SILT AND GRAVEL (SP-SM)**				**0.88**	**12.85**
⊠	**SP-02**	**0-2**	**SAND (SP)**				**1.25**	**10.75**
▲	**SP-04**	**0-2**	**SILTY SAND (SM)**					
★	**SP-05**	**0-2**	**SILTY SAND WITH GRAVEL (SM)**					
⊙	**SP-06**	**0-2**	**SAND WITH GRAVEL (SP)**				**0.95**	**5.43**

	Boring No.	Depth (ft)	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**SP-01**	**0-2**	**37.5**	**1.021**	**0.267**	**0.079**	**18.0**	**72.9**	**9.1**	
⊠	**SP-02**	**0-2**	**37.5**	**1.612**	**0.55**	**0.15**	**14.0**	**82.0**	**4.0**	
▲	**SP-04**	**0-2**	**37.5**	**1.063**	**0.15**		**11.0**	**68.9**	**20.1**	
★	**SP-05**	**0-2**	**37.5**	**1.12**	**0.183**		**19.0**	**62.7**	**18.3**	
⊙	**SP-06**	**0-2**	**25**	**1.891**	**0.791**	**0.348**	**18.0**	**81.1**	**0.9**	

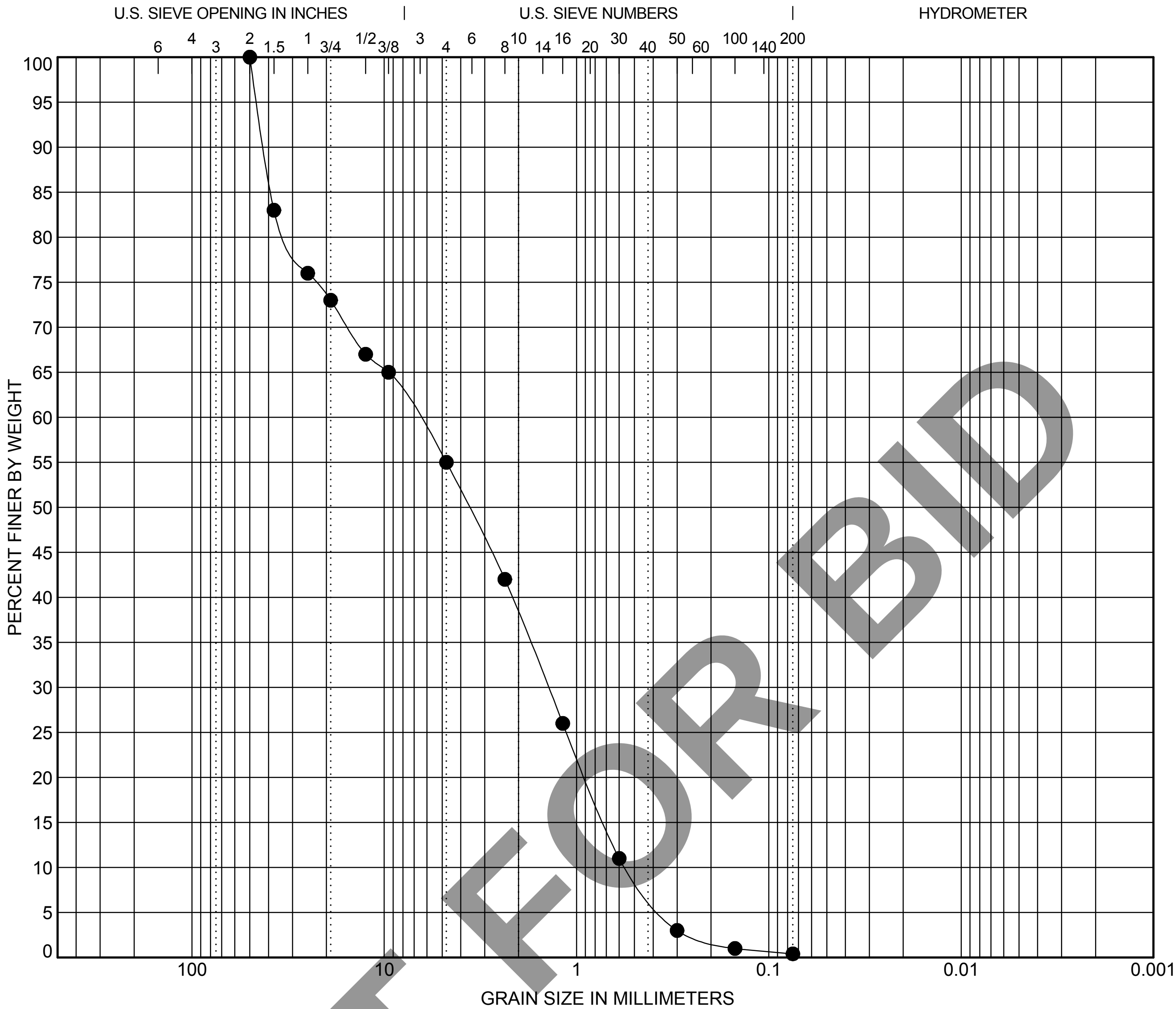
GRAIN SIZE DISTRIBUTION RESULTS

Converse Consultants

Lake Gregory Regional Park Sitewide Sediment Management
City of Crestline, San Bernadino County, California
For: J. Smith and T. Muli

Project No. **23-81-115-01**

Drawing No. **A-1a**



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	Boring No.	Depth (ft)	Description	LL	PL	PI	Cc	Cu
●	**SP-07**	**0-2**	**SAND WITH GRAVEL (SP)**				**0.53**	**12.21**

	Boring No.	Depth (ft)	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**SP-07**	**0-2**	**50**	**6.718**	**1.403**	**0.55**	**45.0**	**54.6**	**0.4**	

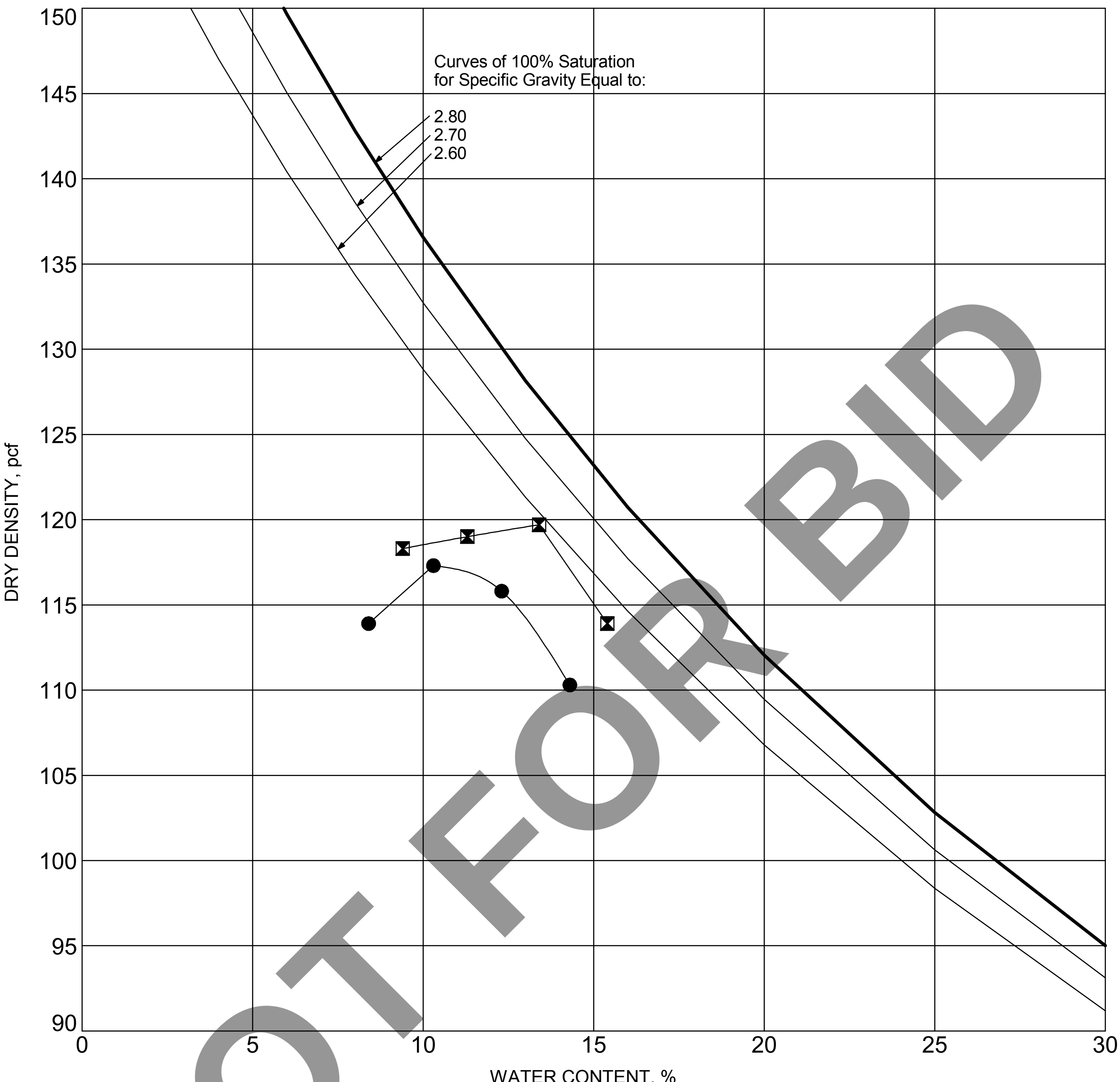
GRAIN SIZE DISTRIBUTION RESULTS

Converse Consultants

Lake Gregory Regional Park Sitewide Sediment Management
City of Crestline, San Bernadino County, California
For: J. Smith and T. Muli

Project No. **23-81-115-01**

Drawing No. **A-1b**



SYMBOL	BORING NO.	DEPTH (ft)	DESCRIPTION	ASTM TEST METHOD	OPTIMUM WATER, %	MAXIMUM DRY DENSITY, pcf
●	SP-04	0-2	SIILTY SAND (SM), DARK BROWN	D1557 A	11	118
⧗	SP-07	0-2	SAND WITH GRAVEL (SP), BROWN	D1557 B	12	120

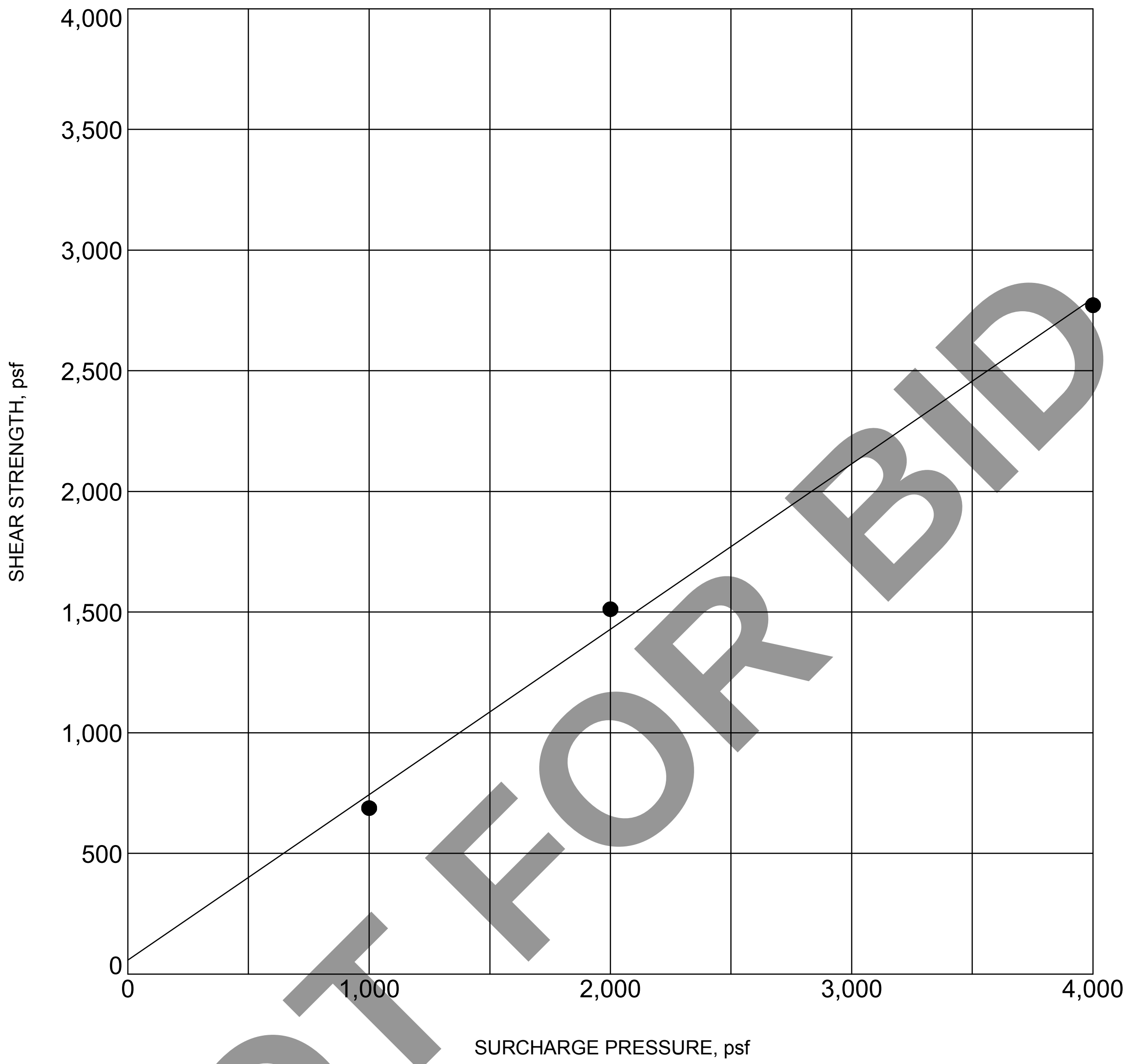
MOISTURE-DENSITY RELATIONSHIP RESULTS

Converse Consultants

Lake Gregory Regional Park Sitewide Sediment Management
City of Crestline, San Bernadino County, California
For: J. Smith and T. Muli

Project No.
23-81-115-01

Drawing No.
A-2



BORING NO. :	SP-04	DEPTH (ft) :	0-2
DESCRIPTION :	SILTY SAND (SM)		
COHESION (psf) :	60	FRICTION ANGLE (degrees):	34
MOISTURE CONTENT (%) :		DRY DENSITY (pcf) :	

NOTE: Ultimate Strength. Samples remolded to 90% of the maximum dry density.

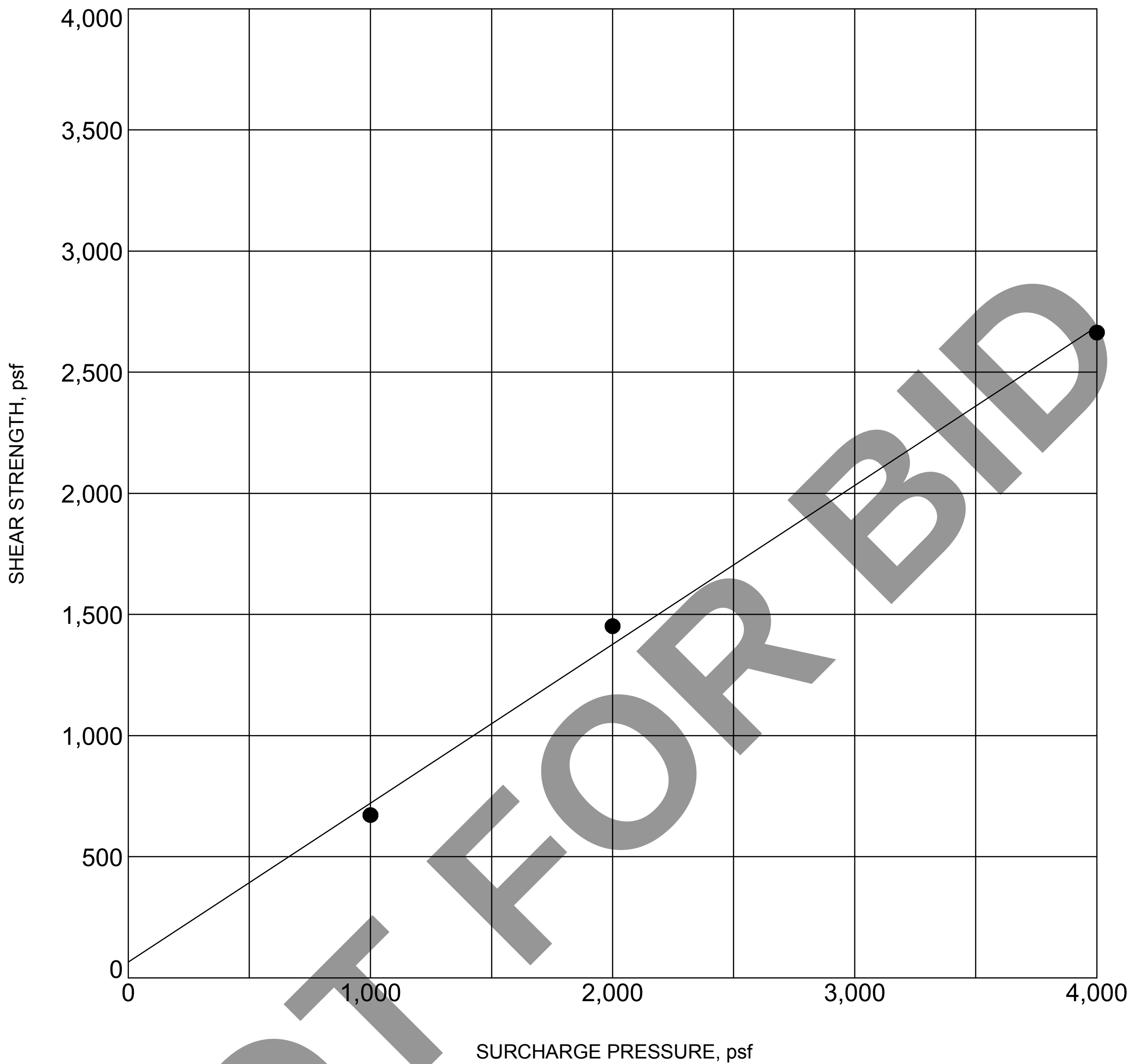
DIRECT SHEAR TEST RESULTS

Converse Consultants

Lake Gregory Regional Park Sitewide Sediment Management
City of Crestline, San Bernadino County, California
For: J. Smith and T. Muli

Project No. **23-81-115-01**

Drawing No. **A-3**



BORING NO. :	SP-07	DEPTH (ft) :	0-2
DESCRIPTION :	SAND WITH GRAVEL (SP)		
COHESION (psf) :	70	FRICTION ANGLE (degrees):	33
MOISTURE CONTENT (%) :		DRY DENSITY (pcf) :	

NOTE: Ultimate Strength. Samples remomded to 90% of the maximum dry density.

DIRECT SHEAR TEST RESULTS

Converse Consultants

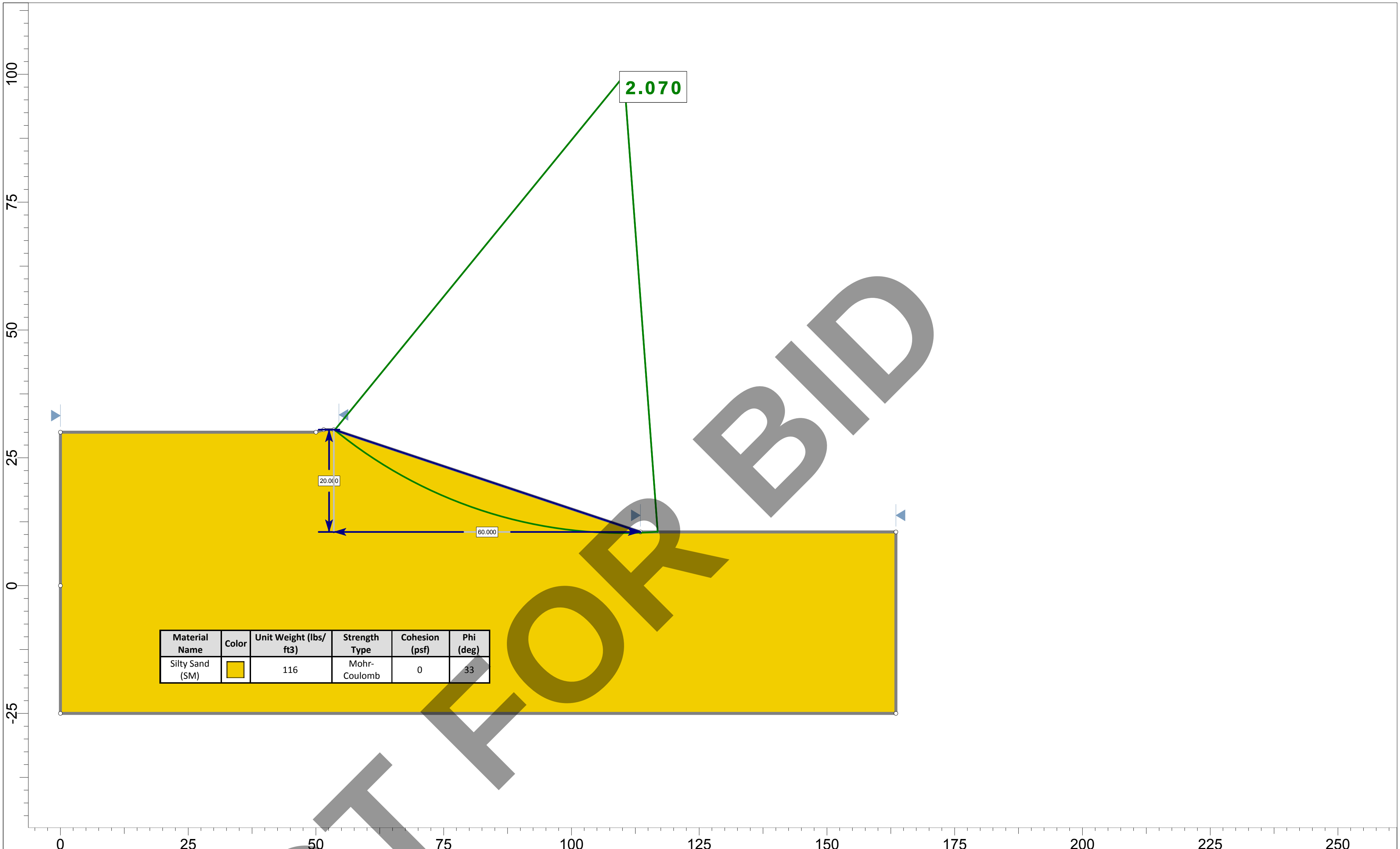
Lake Gregory Regional Park Sitewide Sediment Management
City of Crestline, San Bernadino County, California
For: J. Smith and T. Muli

Project No. **23-81-115-01**

Drawing No. **A-4**

Appendix B

Slope Stability Analysis Results

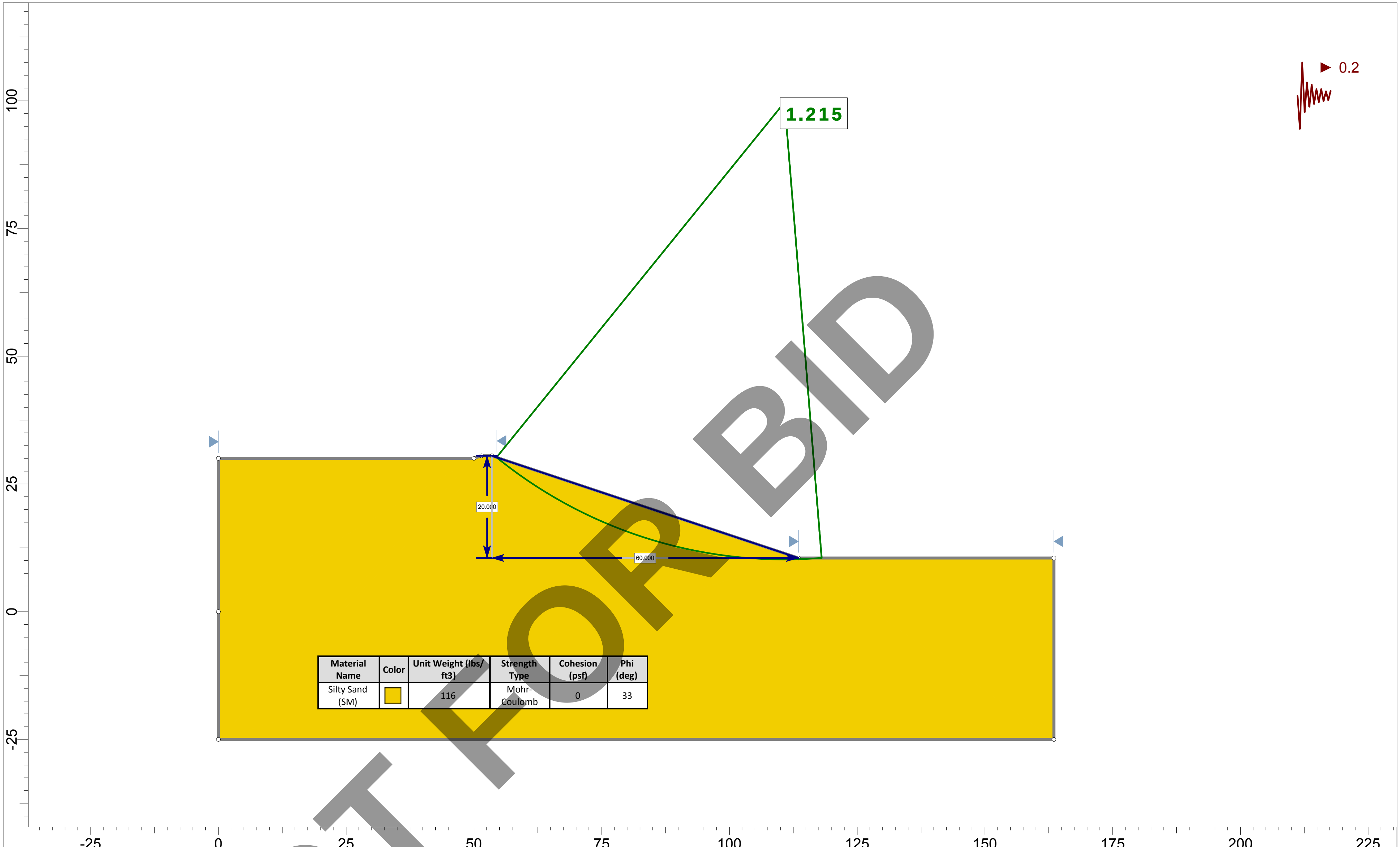


Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Silty Sand (SM)		116	Mohr-Coulomb	0	33

rocscience

SLIDEINTERPRET 9.023

Project	Lake Gregory		
Group	Dry	*Scenario*	Static
Drawn By	Jack	*Company*	Converse
Date	8/10/2023, 12:18:51 PM	*File Name*	Dry.slmd

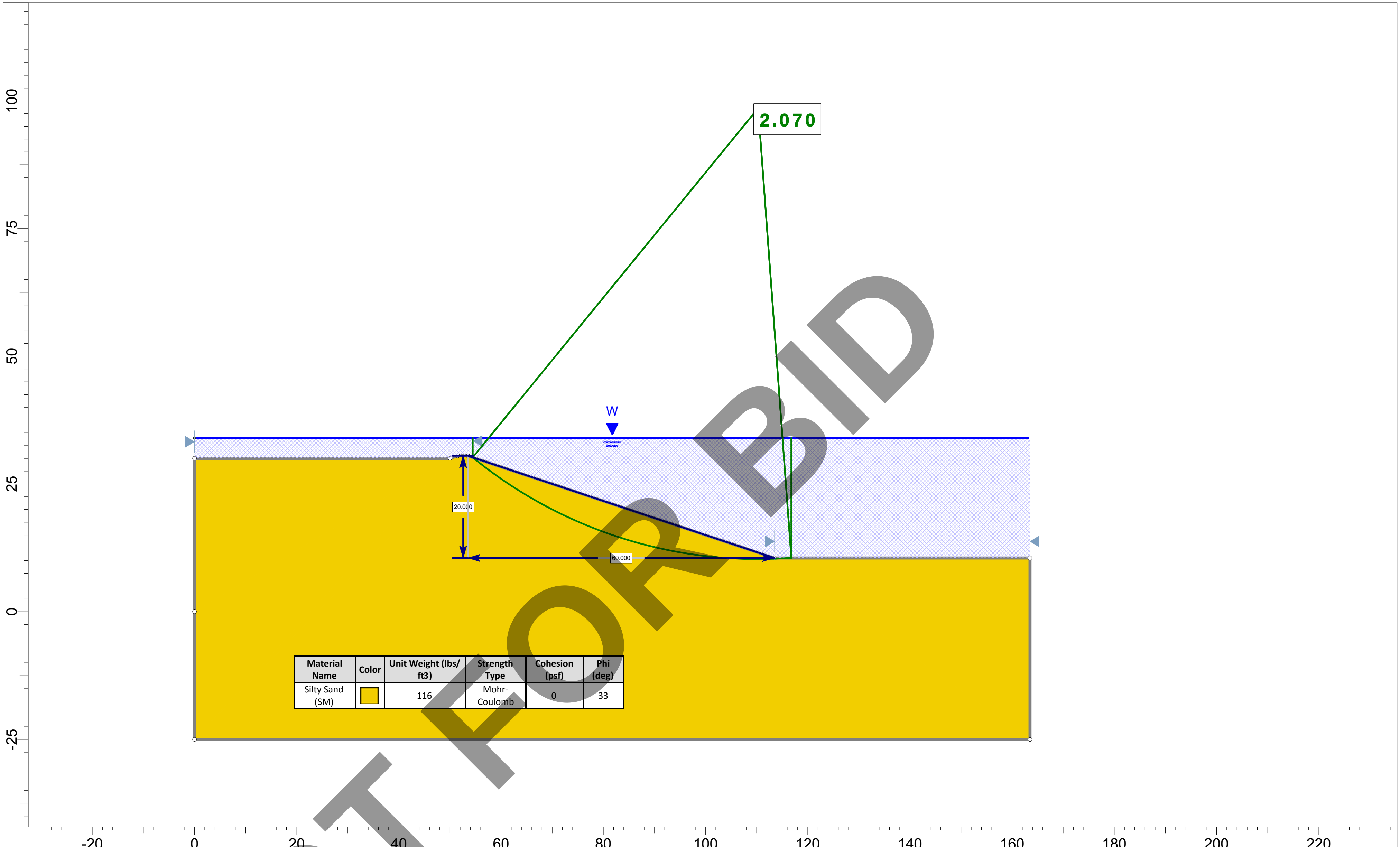


Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Silty Sand (SM)		116	Mohr-Coulomb	0	33

rocscience

SLIDEINTERPRET 9.023

Project	Lake Gregory		
Group	Dry	Scenario	Pseudo-Static
Drawn By	Jack	Company	Converse
Date	8/10/2023, 12:18:51 PM	File Name	Dry.slmd

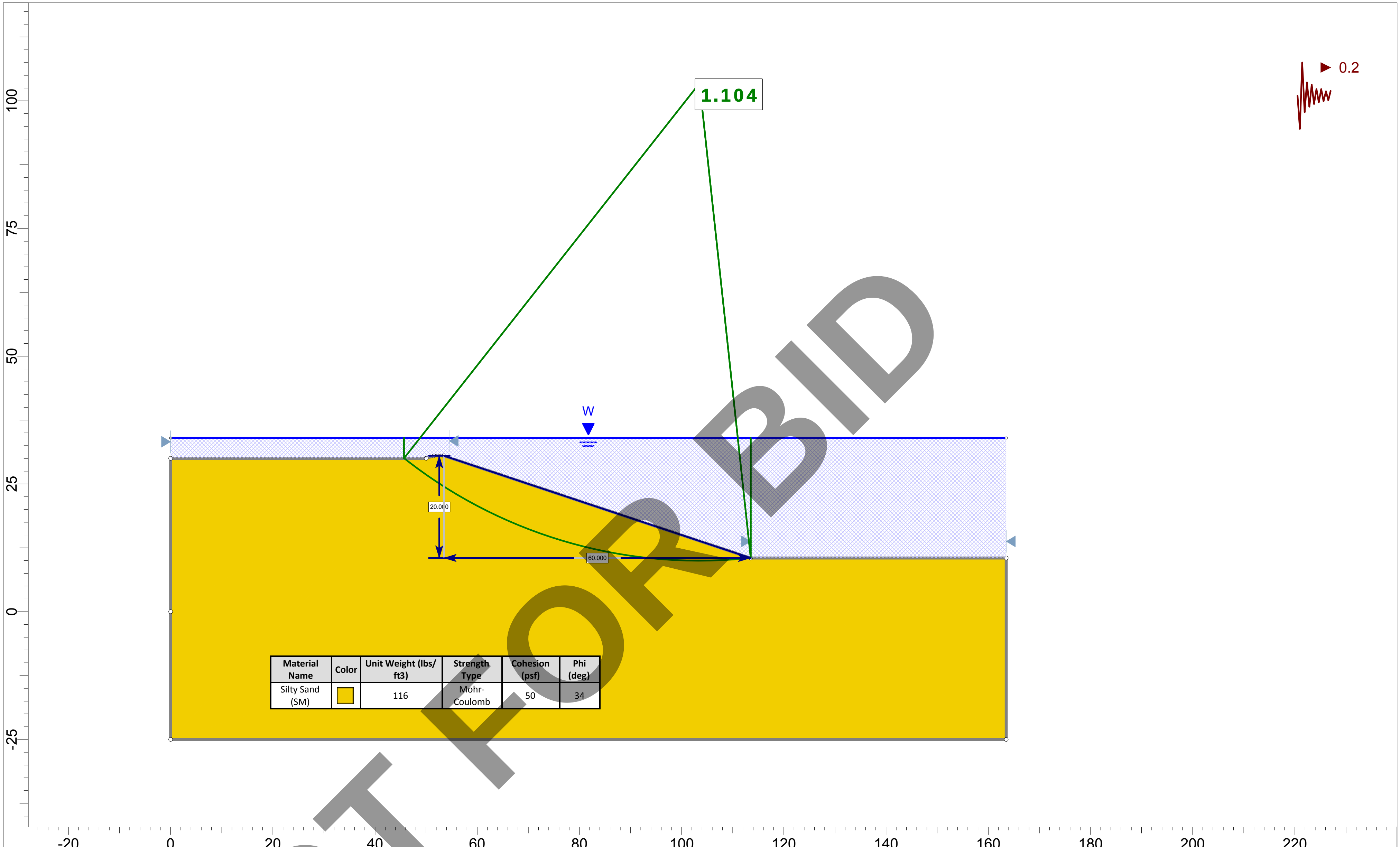


Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Silty Sand (SM)		116	Mohr-Coulomb	0	33

rocscience

SLIDEINTERPRET 9.023

Project	Lake Gregory		
Group	Fully Submerged	*Scenario*	Static
Drawn By	Jack	*Company*	Converse
Date	8/10/2023, 12:18:51 PM	*File Name*	Dry.slmd



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Silty Sand (SM)		116	Mohr-Coulomb	50	34

rocscience

SLIDEINTERPRET 9.023

Project	Lake Gregory		
Group	Fully Submerged	*Scenario*	Pseudo-Static
Drawn By	Jack	*Company*	Converse
Date	8/10/2023, 12:18:51 PM	*File Name*	Dry.slmd