Geotechnical Investigation
WAJ Enterprises Property
Weber County, Utah

Prepared by:

Christensen Geotechnical

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Prepared for:

Brandon Janis

Geotechnical Investigation
WAJ Enterprises Property
Approximately 2050 North Big Sky Drive
Weber County, Utah
CG Project No.: 162-001

Prepared by:

10/12/18
MARK I. CHRISTENSEN
No. 188039

Mark I. Christensen, P.E.
Principal

Christensen Geotechnical
8143 South 2475 East
South Weber, Utah 84405

October 12, 2018

TABLE OF CONTENTS

ATTACHED PLATES

1.0 INTRODUCTION

1.1 PURPOSE AND SCOPE OF WORK

This report presents the results of a geotechnical investigation performed for the WAJ Enterprises property located at approximately 2050 North Big Sky Drive in Weber County, Utah. The general location of the project is indicated on the Project Vicinity Map, Plate 1. In general, the purposes of this investigation were to evaluate the subsurface conditions and the nature and engineering properties of the subsurface soils; and to provide recommendations for general site grading and for the design and construction of floor slabs, pavements, and foundations. This investigation included subsurface exploration, representative soil sampling, field and laboratory testing, engineering analysis, and preparation of this report. Prior to the completion of our report, the Geologic Hazards Evaluation report for the development by Western GeoLogic, dated October 4, 2018, was reviewed to assist in our assessments.

The work performed for this report was authorized by Mr. Brandon Janis and was conducted in accordance with the Christensen Geotechnical proposal dated August 29, 2018.

1.2 PROJECT DESCRIPTION

Based on conversations with Mr. Brandon Janis, we understand that the proposed development of the property is to consist of a residential subdivision approximately 27 acres in size with 8 lots. The structures within the proposed subdivision are to consist of single-family residences. The single-family residences are to be one to two stories in height with basements. The subdivision will also include utilities and an access road. Footings loads for the proposed structures are anticipated to be on the order of 3 to 4 klf for walls and 150 psf for floors. If structural loads are different from those anticipated, Christensen Geotechnical should be notified and allowed to reevaluate our recommendations.

2.0 METHODS OF STUDY

2.1 FIELD INVESTIGATION

The subsurface conditions at the site were explored by excavating 12 test pits to depths of 5½ to 12 feet below existing site grade. Ten of the test pits were excavated on the subject property. The other two were excavated on an adjacent property to assist in assessing the subgrade conditions in the slopes above the property. The approximate location of the test pits are shown on the Exploration Location Map, Plate 2. The subsurface conditions observed are recorded on the Test Pit Logs, Plates 3 through 14. A key to the symbols and terms used on the test pit logs may be found on Plate 15.

Test pit excavation was accomplished with a tracked excavator. Disturbed and undisturbed soil samples were collected from the test pit sidewalls at the time of excavation. Disturbed samples were collected and placed in bags and buckets. Undisturbed samples consisted of block samples which were placed in bags. Samples were visually classified in the field and portions of each sample were packaged and transported to our laboratory for testing. Classifications for the individual subsurface units are shown on the attached Test Pit Logs.

2.2 LABORATORY TESTING

Of the soils collected during the field investigation, representative samples were selected for testing in the laboratory to evaluate the pertinent engineering properties. Laboratory tests included moisture content and density determinations, Atterberg limits evaluations, gradation analyses, consolidation tests, a moisture-density relationship test, a California bearing ratio test, and direct shear tests. A summary of our laboratory testing is presented in the table below:

Table No. 1: Laboratory Test Results

Test Hole No.	Depth (ft.)	Dry Density (pcf)	Moisture Content (%)	Atterberg Limits		Grain Size Distribution (%)			Direct Shear		CBR (%)
				LL	PI	Gravel (+#4)	Sand	Silt/Clay (- #200)	Friction Angle	Cohesion (psf)	
TP-1	2½		19.7	68	42			92.2			2.7
TP-1	5	103	16.9						36	305	
TP-2	4		1.8			64.9	15.8	19.3			
TP-3	5	99.2	25.4	37	18			88.2			
TP-4	3		4.4			47.0	11.3	41.7			
TP-5	4	106.6	19.2	49	30			97.1	27	125	
TP-6	4		6.4	27	12	41.3	38.0	20.7			
TP-7	1½	96.8	5.2	25	7			86.2			
TP-8	2		6.0	29	11			91.0			
TP-9	6		2.0			87.7	12.0	0.4			
TP-10	2	117.3	8.1	54	36			94.0	25	55	
TP-11	7	104.3	17.9	51	29			90.7			
TP-12	3		10.3	28	12			68.4			
TP-12	8		2.1			64.3	18.9	16.9			

The results of the laboratory tests are also presented on the Test Pit Logs (Plates 3 through 14), and more detailed laboratory results are presented on the laboratory testing Plate (Plates 16 through 27).

Samples will be retained in our laboratory for 30 days following the date of this report, at which time they will be disposed of unless a written request for additional holding time is received prior to the disposal date.

3.0 GENERAL SITE CONDITIONS

3.1 SURFACE CONDITIONS

At the time of our investigation, the subject site was undeveloped land located in the foothills above Liberty, Utah. The general area of the property sloped down to the north with the topography of the property varying with slope grades generally less than 4 to 1 (horizontal to vertical) to areas that were nearly level. The total elevation change across the lot was approximately 150 feet. Coal Hollow Creek crossed the property from southwest to northeast. Several other ephemeral drainages that connect to Coal Hollow Creek also crossed the property. Vegetation at the site generally consisted of meadows of common grasses and weeds and stands of dense oak brush with other common trees. The site was bordered by existing houses to the northwest, west and southwest and undeveloped land on all other sides.

3.2 SUBSURFACE CONDITIONS

3.2.1 Soils

Based on the 12 test pits excavated for this investigation, the site is covered with 1 to 2½ feet of clayey topsoil. Subgrade conditions below the topsoil generally consist of tuffaceous claystone and conglomerate bedrock. The bedrock was generally weathered and weak, with increasing strength with depth. In test pits TP-3, TP-4, TP-7, and TP-11, 1 to 1½ feet of colluvium soil was encountered between the topsoil and bedrock. The soil consisted of zones of Clayey GRAVEL with sand and cobbles (GC), Sandy Lean CLAY (CL), Fat CLAY (CH), and Silty CLAY (CL-ML). In test pits TP-5 and TP-8, the test pits encountered soil to the bottom of the excavations and did not encounter bedrock. Soils in these test pits consisted of zones of Lean CLAY (CL), Fat CLAY (CH), and Clayey GRAVEL with sand and cobbles (GC).

3.2.2 Groundwater

Groundwater was encountered at depths of 9½ and 8½ feet within test pits TP-3 and TP-8, respectively, at the time of excavation. No other test pit encountered groundwater. It should be understood that groundwater is likely below its seasonal high and may fluctuate in response to seasonal changes, precipitation, and irrigation.

4.0 SEISMIC CONSIDERATIONS

4.1 SEISMIC DESIGN CRITERIA

The State of Utah and Utah municipalities have adopted the 2015 International Building Code (IBC) for seismic design. The IBC seismic design is based on seismic hazard maps depicting probabilistic ground motions and spectral response; the maps, ground motions, and spectral response having been developed by the United States Geological Survey (USGS). Seismic design values, including the design spectral response, may be calculated for a specific site using the USGS Seismic Design Maps web-based application and the project site’s approximate latitude and longitude and Site Class. Based on our field exploration, it is our opinion that this location is best described as a Site Class C which represents a “very dense soil and soft rock” profile. The spectral acceleration values obtained from the USGS web-based application are shown below.

Table 2: IBC Seismic Response Spectrum Values

Site Location: **Latitude = 41.2957^{o} N** **Longitude = -111.8490^{o} W**			
Spectral Period (sec)	**Response Spectrum Spectral Acceleration (g)**		
0.2	S_S=0.975g	S_{MS}=0.985g	S_{DS}=0.657g
1.0	S_1= 0.336g	S_{M1}=0.491g	S_{D1}=0.328g

Using these values, the peak ground acceleration (PGA) is estimated to be 0.39g.

4.2 LIQUEFACTION

Certain areas in the intermountain west possess a potential for liquefaction. Liquefaction is a phenomenon in which soils lose their intergranular strength due to an increase of pore pressures during a dynamic event such as an earthquake. The potential for liquefaction is based on several factors, including 1) the grain-size distribution of the soil, 2) the plasticity of the fine fraction of the soil (material passing the No. 200 sieve), 3) the relative density of the soils, 4) earthquake strength (magnitude) and duration, 5) overburden pressures, and 6) the depth to groundwater.

The map “Special Study Areas, Wasatch Front and Nearby Areas, Utah” (Christenson et al., 2008) indicates that the subject site is located in an area designated as having a very low potential for liquefaction. Due to the shallow bedrock at the site, we also assess the liquefaction potential to be very low.

5.0 ENGINEERING ANALYSIS AND RECOMMENDATIONS

5.1 GENERAL CONLUSIONS

Based on the results of our field and laboratory investigations, it is our opinion that the subject site is suitable for the proposed construction provided that the recommendations contained in this report are incorporated into the design and construction of the project.

5.2 EARTHWORK

5.2.1 General Site Preparation and Grading

Prior to site grading operations, all vegetation, topsoil, undocumented fill soils, and loose or disturbed soils should be stripped (removed) from the building pads and flatwork concrete areas. Following the stripping operations, the exposed soils should be proof rolled to a firm, unyielding condition. Site grading may then be conducted to bring the site to design grade.

Based on the test pits excavated for our investigation, 1 to 2½ feet of topsoil cover the site. This topsoil should be removed from below footings and concrete flatwork. Where over-excavation is required, the excavation should extend at least 1 foot laterally for every foot of over-excavation. A Christensen Geotechnical representative should observe the site grading operations.

5.2.2 Soft Soil Stabilization

Due to the clayey nature of much of the subgrade soils, soft soils may be exposed in excavations at the site. Once exposed, all subgrade soils should be proof rolled with a relatively large-wheeled vehicle to a firm, unyielding condition. Localized soft areas identified during the proof rolling operation should be removed and replaced with granular structural fill. If soft areas extend to more than 18 inches deep, or where large areas are encountered, stabilization may be considered. The use of stabilization should be approved by the geotechnical engineer, and would likely consist of over-excavating the area by at least 18 inches, placing a geofabric (such as Mirafi RS280i) at the bottom of the excavation, over which a stabilizing fill consisting of angular coarse gravel with cobbles is placed to the design subgrade.

5.2.3 Temporary Construction Excavations

Based on OSHA requirements and the soil conditions encountered during our field investigation, we anticipate that temporary construction excavations at the site that have vertical walls extending to depths of up to 5 feet may be occupied without shoring; however, where groundwater or fill soils are encountered, flatter slopes may be required. Excavations which extend to more than 5 feet in depth should be sloped or shored in accordance with OSHA regulations for a type C soil. Stability of construction excavations is the contractor’s responsibility. All excavations should be evaluated by qualified personnel prior to entry to assess the need for sloping or shoring.

5.2.4 Structural Fill and Compaction

All fill placed for support of structures, concrete flatwork, and roadways should consist of structural fill. Structural fill may consist of either the native sand and gravel soils or bedrock that is composed of sand and gravel particles. The native clay soils and claystone bedrock may also be used as structural fill below roadways, but due to their potential to swell, they should not be used below structures and concrete flatwork. If the native clay and claystone are used below roadways it should be understood that they may be difficult to moisture condition and compact. Imported structural fill, if required, should consist of a relatively well-graded granular soil with a maximum particle size of 4 inches, with a maximum of 50 percent passing the No. 4 sieve and a maximum of 30 percent passing the No. 200 sieve. The liquid limit of the fines (material passing the No. 200 sieve) should not exceed 35 and the plasticity index should be less than 15. All structural fill, whether native soils or imported material, should be free of topsoil, vegetation, frozen material, particles larger than 4 inches in diameter, and any other deleterious materials. Any imported materials should be approved by the geotechnical engineer prior to importing. The engineer should also be consulted for any questions in differentiating between the different types of bedrock and their suitability for use as structural fill.

Structural fill should be placed in maximum 8-inch-thick loose lifts at a moisture content within 3 percent of optimum and compacted to at least 95 percent of the maximum density as determined by ASTM D 1557. Where fill heights exceed 5 feet, the level of compaction should be increased to 98 percent.

5.2.5 Excavatability

As indicated above, claystone and conglomerate bedrock was encountered in 8 of the 10 test pits excavated on the subject property. In general, excavation of the bedrock became increasingly difficult with depth. We anticipate that excavations within the bedrock may require the use of a heavy excavator with a ripper tooth or the use of a hoe-ram. Deeper excavations within the bedrock may require additional measures. Prior to bidding, the contractor should be made aware of the subsurface conditions and the type of equipment best suited for these conditions.

5.2.6 Permanent Cut and Fill Slopes

Existing slopes should not be over steepened by cutting or filling. We recommend that all non-retained cut and fill slopes less than 8 feet in height be graded no steeper than a 3 to 1 (horizontal to vertical) grade. If steeper grades are required, engineered retaining structures should be used. If cuts or fills more than 8 feet in height are required, additional slope stability assessments should be performed to assess stability prior to construction.

5.3 FOUNDATIONS

Foundations for the planned structures may consist of conventional continuous and/or spread footings. Due to the slight potential for swelling or collapsing of the clay soils and the claystone bedrock which were encountered at the site, we recommend that where clay soils and claystone bedrock are encountered below footings, at least 12 inches of properly placed and compacted structural fill should be placed below the footings. Where either sand and gravel soils and/or bedrock that is composed of sand and gravel particles are encountered below footings, the footings may be established on undisturbed native soil, bedrock, or on properly placed and compacted structural fill extending down to undisturbed native soil. Where footing excavations expose both soil and bedrock, we recommend that the bedrock be over-excavated to allow placement of at least 12 inches of structural to provide more uniform support. Footings for the proposed structures should be a minimum of 20 inches and 30 inches wide for continuous and spot footings, respectively. Exterior footings should be established at a minimum of 30 inches below the lowest adjacent grade to provide frost protection and confinement. Interior footings not subject to frost should be embedded a minimum of 18 inches for confinement.

Continuous and spread footings established on undisturbed native soils or structural fill may be proportioned for a maximum net allowable bearing capacity of 2,000 psf. A one-third increase may be used for transient wind or seismic loads. All footing excavations should be observed by the geotechnical engineer prior to construction of the footings.

5.4 ESTIMATED SETTLEMENT

If the foundations are designed and constructed in accordance with the recommendations presented in this report, there is a low risk that total settlement will exceed 1 inch and a low risk that differential settlement will exceed ½ inch for a 30-foot span.

5.5 LATERAL EARTH PRESSURES

Buried structures, such as basement walls, should be designed to resist the lateral loads imposed by the soils retained. The lateral earth pressures on the below-grade walls and the distribution of those pressures depends upon the type of structure, hydrostatic pressures, in-situ soils, backfill, and tolerable movements. Basement and retaining walls are usually designed with triangular stress distributions, which are based on an equivalent fluid pressure and calculated from lateral earth pressure coefficients. If soils similar to the native soils are used to backfill basement walls, then the walls may be designed using the following ultimate values:

Table No. 3: Lateral Earth Pressures

Condition	Lateral Pressure Coefficient	Equivalent Fluid Density (pcf)
Active Static	0.38	45
Active Seismic	0.15	17
At-Rest	0.55	66
Passive Static	2.66	320
Passive Seismic	-0.30	-36

We recommend that walls which are allowed little or no wall movement be designed using “at rest” conditions. Walls allowed to rotate at least 0.4 percent of the wall height may be designed with “active” pressures. The coefficients and densities presented above assume level backfill with no buildup of hydrostatic pressures. If anticipated, hydrostatic pressures and any surcharge loads should be added to the presented values. If sloping backfill is present, we recommend the geotechnical engineer be consulted to provide more accurate lateral pressure parameters once the design geometry is established.

The seismic active and passive earth pressure coefficients provided in the table above are based on the Mononobe-Okabe method and only account for the dynamic horizontal force produced by a seismic event. The resulting dynamic pressure should therefore be added to the static pressure to determine the total pressure on the wall. The dynamic pressure distribution may be approximated as an inverted triangle, with stress decreasing with depth and the resultant force acting approximately 0.6 times the height of the retaining wall, measured upward from the bottom of the wall.

Lateral building loads will be resisted by frictional resistance between the footings and the foundation soils and by passive pressure developed by backfill against the wall. For footings on native soils, we recommend an ultimate coefficient of friction of 0.33 be used. If passive resistance is used in conjunction with frictional resistance, the passive resistance should be reduced by ½. Passive earth pressure from soils subject to frost or heave should usually be neglected in design.

The coefficients and equivalent fluid densities presented above are ultimate values and should be used with an appropriate factor of safety against overturning and sliding. A value of 1.5 is typically used.

5.6 CONCRETE SLAB-ON-GRADE CONSTRUCTION

The laboratory testing completed for this investigation indicates that the native clay soils and claystone bedrock at the site have some risk for expansion. Concrete slabs, including basement floor slabs and exterior flatwork, have a high risk of movement due to their light loading. To reduce the risk of expansion and slab movement, consideration should be given to placing 24 inches of structural fill below any concrete slabs where clay soils and claystone bedrock are encountered below the proposed structures. At a minimum, we recommend that concrete slabs-on-grade be constructed over at least 4 inches of compacted gravel to help distribute floor loads, break the rise of capillary water, and to aid in the curing process. The gravel should consist of free-draining gravel compacted to a firm, unyielding condition. To help control normal shrinkage and stress cracking, the floor slab should have adequate reinforcement for the anticipated floor loads with the reinforcement continuous through the interior joints. In addition, we recommend adequate crack control joints to control crack propagation. Prior to construction of slabs-on-grade, the site grading recommendations presented in Section 5.2.1 should be followed.

5.7 MOISTURE PROTECTION AND SURFACE DRAINAGE

Wetting of the foundation soils will likely cause some degree of volume change within the soil and should be prevented both during and after construction. We recommend that the following precautions be taken at this site:

1. The ground surface should be graded to drain away from the structures in all directions, with a minimum fall of 8 inches in the first 10 feet.
2. Roof runoff should be collected in rain gutters with down spouts designed to discharge well outside of the backfill limits.
3. Sprinkler heads should be aimed away from and placed at least 12 inches from foundation walls.

4. There should be adequate compaction of backfill around foundation walls, to a minimum of 90% density (ASTM D 1557). Water consolidation methods should not be used.

5.8 SUBSURFACE DRAINAGE

Due to the relative high elevation of the subject site, we recommend that all basement and retaining walls incorporate a foundation drain. The foundation drain should consist of a 4-inch-diameter slotted pipe placed at or below the bottom of footings, encased in at least 12 inches of free-draining gravel. The gravel should be extended up the foundation wall to within 2 feet of the final ground surface, and a filter fabric, such as Mirafi 140N, should separate the gravel from the native soils. The slotted pipe should transition to a solid pipe which should be graded to a free gravity outfall. The gravel extending up the wall may be replaced by a fabricated drain panel such as Mirafi G200N or equivalent.

5.9 SLOPE STABILITY

As recommended by Western Geologic in their “Geologic Hazards Evaluation,” we performed a slope stability assessment for the site using the Slide computer program and the modified Bishop’s method of slices. Three profiles of the property, profiles A, B, and C as shown on Plate 2, were assessed for slope stability. The profile subsurface conditions used in our analyses were based on cross sections of the profiles provided by Western Geologic and our field explorations. For our assessments, we assumed there is at least 5 feet of colluvium soil overlying bedrock at the site. Where the Western Geologic cross sections showed soil layers deeper than 5 feet, we used those depths indicated. Three direct shear tests were performed on samples of native clay soil and claystone bedrock. Two tests were performed on fully softened, remolded samples and one test was performed on a sample of undisturbed claystone bedrock. The results of our testing produced fully softened strengths consisting of an angle of internal friction of 25 degrees with a cohesion of 55 psf and an angle of internal friction of 27 degrees with a cohesion of 125 psf. A strength consisting of an angle of internal friction of 36 degrees with a cohesion of 305 psf was obtained for undisturbed bedrock. For our analyses, we used strength values consisting of an angle of internal friction of 27 degrees with a cohesion of 125 psf for the native colluvium soils, an angle of internal friction of 25 degrees with a cohesion of 55 psf for the landslide deposits, and an angle of internal friction of 36 degrees with a cohesion of 300 psf for the bedrock.

The profiles were assessed under static and pseudo static conditions. The pseudo static condition is used to assess the slope during a seismic event. As indicated in Section 4.1, the peak ground acceleration at this site is estimated to be 0.39g. As is common practice, half of this value was used in our pseudo static assessments. Minimum factors of safety of 1.5 and 1.0 for static and seismic conditions, respectively, were considered acceptable. Our analyses indicate that the profiles have safety factors greater than 1.5 and 1.0 for the static and pseudo static conditions and are therefore considered adequate for residential development. The results of our slope stability assessments may be found on Plates 28 through 33.

The slope stability analyses presented above are based on the assumption that no significant cuts or fills will occur during the development of the subdivision or construction of the houses. Significant changes to the site grade, such as the steepening of slopes with cuts or fills, may adversely affect the stability of the slopes at the site and increase the risk of slope failures. If cuts or fills over 8 feet are planned at the site or if retaining walls over 5 feet in height are constructed within the development, additional slope stability assessments may be necessary and Christensen Geotechnical should be contacted to provide the additional assessments.

5.10 PAVEMENT DESIGN

It is our understanding that the roadway within the development will consist of an unpaved road. As such, the roadway within the proposed development was assessed using the PAS computer program prepared by the American Concrete Pavement Association and a laboratory-obtained CBR value of 2.7 percent. No traffic information was available at the time this report was prepared; Christensen Geotechnical has therefore assumed a traffic load for the roadways based on our experience with similar projects. We have assumed that traffic will consist of 80 passenger cars per day and 1 medium truck per day. We have further assumed no increase in traffic over the life of the roadway. Based on this information, we recommend the roadways consist of 18 inches of untreated base. As an alternative, a roadway consisting of 8 inches of untreated base over 12 inches of granular borrow may be used. Untreated base should meet the material requirements for Weber County or UDOT. Granular borrow should meet the recommendations for imported structural fill presented in Section 5.2.4. Untreated base and granular borrow should be compacted to at least 95 percent of the maximum dry density as determined by ASTM D 1557.

5.11 CONSTRUCTION CONSIDERATIONS

Figure 3C of the Western GeoLogic Geologic Hazards Assessment report identifies an area of seeps on lot 8 of the subdivision and an area of landslide deposits on lot 6. In addition, a very wet area was observed at the north end of lots 3 and 4 during our field investigation. Due to the increased hazards of basement seepage, construction mobility issues, excessive settlement, and potential slope movement (in the case of the landslide deposits on lot 6), we recommend that homes not be located in these areas. Western Geologic or Christensen Geotechncial may be contacted to assist in locating these areas during construction of the homes.

6.0 LIMITATIONS

The recommendations contained in this report are based on limited field exploration, laboratory testing, and our understanding of the proposed construction. The subsurface data used in this report was obtained from the explorations that were made specifically for this investigation. It is possible that variations in the soil and groundwater conditions could exist between and beyond the points explored. The nature and extent of variations may not be evident until construction occurs. If any conditions are encountered at this site that are different from those described in this report, Christensen Geotechnical should be immediately notified so that we may make any necessary revisions to the recommendations contained in this report. In addition, if the scope of the proposed construction changes from that described in this report, Christensen Geotechnical should be notified.

This report was prepared in accordance with the generally accepted standard of practice at the time the report was written. No other warranty, expressed or implied, is made.

It is the client's responsibility to see that all parties to the project, including the designer, contractor, subcontractors, etc., are made aware of this report in its entirety. The use of information contained in this report for bidding purposes should be done at the contractor's option and risk.

7.0 REFERENCES

Black, Bill, October 4, 2018, “Geologic Hazards Evaluation, WAJ Enterprises Property, About 2050 North Big Sky Drive, Liberty, Weber County, Utah,” Western GeoLogic, consultant’s unpublished report.

Christenson, Gary E. and Shaw, Lucas M., 2008, Liquefaction Special Study Areas, Wasatch Front and Nearby Areas, Utah,” Utah Geological Survey, Supplement Map to Utah Circular 106.

Base Photo: Utah AGRC

Drawing Not to Scale

☐ **Approximate Project Boundary**

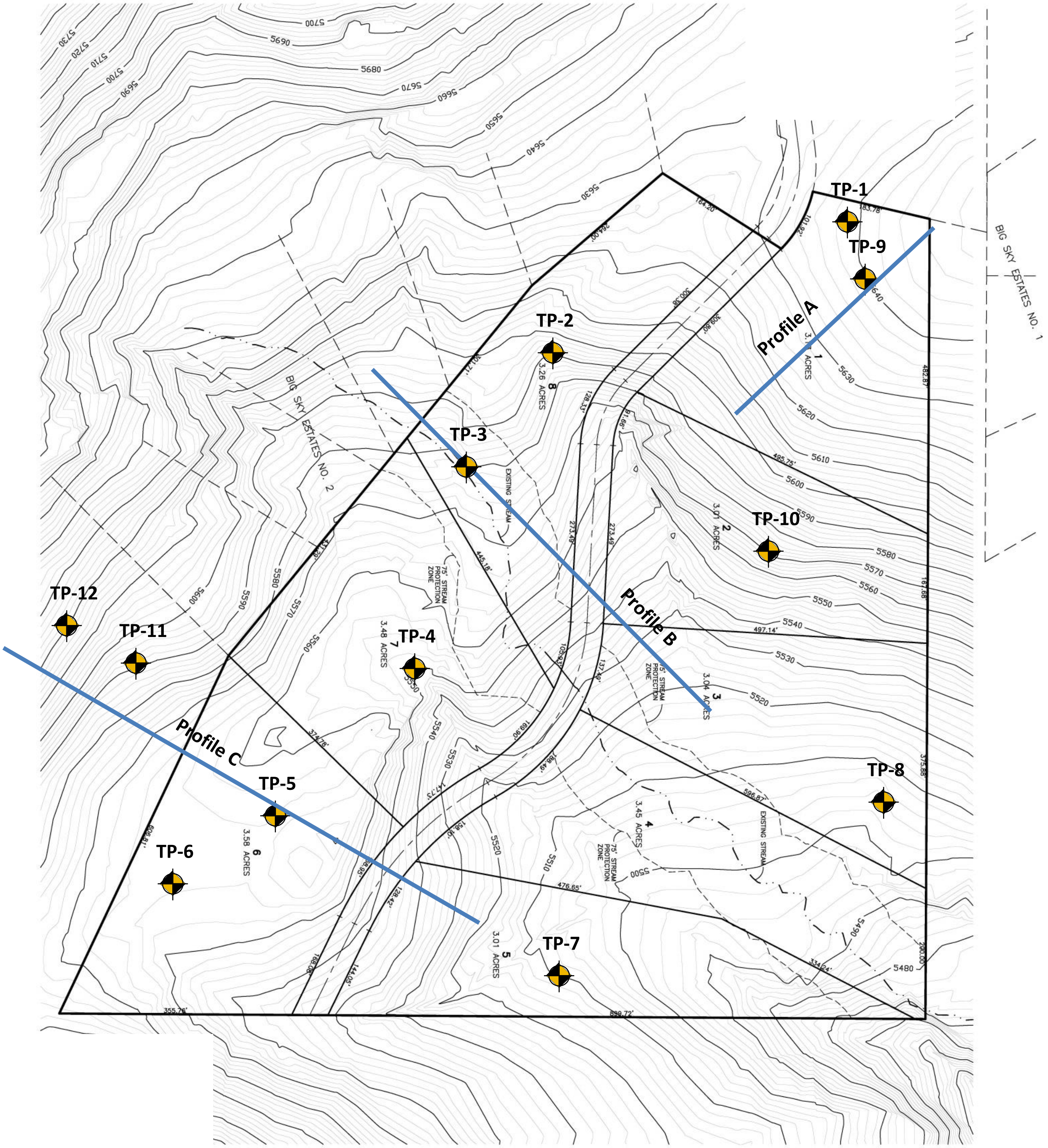
N

Christensen Geotechnical

Brandon Janis
WAJ Enterprises Property
Weber County, Utah
Project No. 162-001

Vicinity Map

Plate
1



Base Map: Reeve & Associates, Inc.

⊕ **Approximate Test Pit Location**

Drawing Not to Scale

— **Slope Stability Profile**

Christensen Geotechnical	Brandon Janis WAJ Enterprises Property Weber County, Utah Project No. 162-001 **Exploration Location Map**	Plate 2

Date				
Started:	9/18/2018	# TEST PIT LOG	Logged By: M Christensen	Test Pit No.
Completed:	9/18/2018		Equipment: Trackhoe	**TP-1**
Backfilled:	Unknown		Location: See Plate 2	Sheet 1 of 1

Depth (feet)	Sample Type	Groundwater	Graphic Log	Group Symbol	**Material Description**	Dry Density (pcf)	Moisture Content (%)	Minus #200 (%)	Liquid Limit	Plastic Limit
			▓		Topsoil; Lean CLAY - moist, dark brown					
	☒		▨		Tuffaceaous Claystone Bedrock - weathered, weak, red-brown		19.7	92.2	68	42
5	▥		▨		- moderately strong and with gravel and cobbles below 5 feet	103.0	16.9			
					Bottom of test pit at 8 feet					
10										
15										

☒ Bulk/Bag Sample
▥ Undisturbed Sample

▼ Stabllized Groundwater
▽ Groundwater At Time of Excavation

Christensen Geotechnical

Brandon Janis
WAJ Enterprises Property
Weber County, Utah
Project No.: 162-001

Plate

3

Date		
	Started:	9/18/2018
	Completed:	9/18/2018
	Backfilled:	Unknown

TEST PIT LOG

Logged By: M Christensen
Equipment: Trackhoe
Location: See Plate 2

Test Pit No.

TP-2

Sheet 1 of 1

Depth (feet)	Sample Type	Groundwater	Graphic Log	Group Symbol	**Material Description**	Dry Density (pcf)	Moisture Content (%)	Minus #200 (%)	Liquid Limit	Plastic Limit
			▒		Topsoil; Lean CLAY - moist, dark brown					
			▨		Tuffaceous Conglomerate Bedrock - weathered, weak, light brown, with cobble clasts up to 12 inches in diameter					
	☒		▨				1.8	19.3		
5			▨		- moderately strong below 5 feet					
					Bottom of test pit at 8 feet					
10										
15										

☒ Bulk/Bag Sample
▥ Undisturbed Sample

▼ StabIlized Groundwater
▽ Groundwater At Time of Excavation

Christensen Geotechnical

Brandon Janis
WAJ Enterprises Property
Weber County, Utah
Project No.: 162-001

Plate

4

Date		
	Started:	9/18/2018
	Completed:	9/18/2018
	Backfilled:	Unknown

TEST PIT LOG

Logged By: M Christensen
Equipment: Trackhoe
Location: See Plate 2

Test Pit No.

TP-3

Sheet 1 of 1

Depth (feet)	Sample Type	Groundwater	Graphic Log	Group Symbol	Material Description	Dry Density (pcf)	Moisture Content (%)	Minus #200 (%)	Liquid Limit	Plastic Limit
			▓		Topsoil; Lean CLAY - moist, dark brown					
			▨		Sandy Lean CLAY - stiff, moist, yellow-brown					
			▦		Tuffaceous Claystone Bedrock - weathered, weak, yellow-brown mottled gray					
5	▥		▦		- moderately strong below 5 feet	99.2	25.4	88.2	37	18
		▼	▦		- wet below 9½ feet					
10					Bottom of test pit at 10 feet					
15										

☒ Bulk/Bag Sample
▥ Undisturbed Sample

▼ StabIlized Groundwater
▽ Groundwater At Time of Excavation

Christensen Geotechnical

Brandon Janis
WAJ Enterprises Property
Weber County, Utah
Project No.: 162-001

Plate

5

Date			
Started: 9/18/2018	# TEST PIT LOG	Logged By: M Christensen	Test Pit No.
Completed: 9/18/2018		Equipment: Trackhoe	**TP-4**
Backfilled: Unknown		Location: See Plate 2	Sheet 1 of 1

Depth (feet)	**Sample Type**	**Groundwater**	**Graphic Log**	**Group Symbol**	**Material Description**	**Dry Density (pcf)**	**Moisture Content (%)**	**Minus #200 (%)**	**Liquid Limit**	**Plastic Limit**
			▩		Topsoil; Lean CLAY - moist, dark brown					
	☒		▧		Clayey GRAVEL with sand - medium dense, slightly moist, light brown, with cobbles up to 12 inches in diameter		4.4	41.7		
5			▨		Tuffaceous Conglomerate Bedrock - weathered, moderately strong, light brown, with cobble clasts up to 12 inches in diameter					
10 15					Bottom of test pit at 7 feet					

☒ Bulk/Bag Sample
▥ Undisturbed Sample

▼ StabIlized Groundwater
▽ Groundwater At Time of Excavation

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Weber County, Utah
Project No.: 162-001

Plate
6

Date		
Started:	9/18/2018	
Completed:	9/19/2018	
Backfilled:	Unknown	

TEST PIT LOG

Logged By: M Christensen
Equipment: Trackhoe
Location: See Plate 2

Test Pit No.

TP-5

Sheet 1 of 1

Depth (feet)	Sample Type	Groundwater	Graphic Log	Group Symbol	Material Description	Dry Density (pcf)	Moisture Content (%)	Minus #200 (%)	Liquid Limit	Plastic Limit
			▩		Topsoil; Lean CLAY - moist, dark brown					
			▨		Lean CLAY - very stiff, slighlty moist, red-brown					
	▥		▨			106.6	19.2	97.1	49	30
5			▨	CL	- moist below 5 feet					
	▥		▨							
10			▨	CL	Sandy Lean CLAY - very stiff, moist, red-brown					
					Bottom of test pit at 11½ feet					
15										

☒ Bulk/Bag Sample
▥ Undisturbed Sample
▼ Stabllized Groundwater
▽ Groundwater At Time of Excavation

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Project No.: 162-001

Plate

7

Date		
Started:	9/18/2018	
Completed:	9/19/2018	
Backfilled:	Unknown	

TEST PIT LOG

Logged By: M Christensen
Equipment: Trackhoe
Location: See Plate 2

Test Pit No.

TP-6

Sheet 1 of 1

Depth (feet)	Sample Type	Groundwater	Graphic Log	Group Symbol	**Material Description**	Dry Density (pcf)	Moisture Content (%)	Minus #200 (%)	Liquid Limit	Plastic Limit
			▓		Topsoil; Lean CLAY - moist, dark brown					
			▨		Tuffaceous Conglomerate Bedrock - weathered, weak, light brown, with cobble clasts up to 12 inches in diameter					
	▥		▨				6.4	20.7	27	12
5			▨							
			▨		Tuffaceaous Claystone Bedrock - weathered, moderately strong, red-brown					
10	▥		▨							
					Bottom of test pit at 11 feet					
15										

☒ Bulk/Bag Sample
▥ Undisturbed Sample

▼ StabIlized Groundwater
▽ Groundwater At Time of Excavation

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Weber County, Utah
Project No.: 162-001

Plate

8

Date		
Started:	9/18/2018	
Completed:	9/19/2018	
Backfilled:	Unknown	

TEST PIT LOG

Logged By: M Christensen
Equipment: Trackhoe
Location: See Plate 2

Test Pit No.

TP-7

Sheet 1 of 1

Depth (feet)	Sample Type	Groundwater	Graphic Log	Group Symbol	Material Description	Dry Density (pcf)	Moisture Content (%)	Minus #200 (%)	Liquid Limit	Plastic Limit
			▓		Topsoil; Lean CLAY - moist, dark brown					
	▥		▒		Silty CLAY - stiff, slightly moist, light brown	96.8	5.2	86.2	25	7
5	▥		▨		Tuffaceaous Claystone Bedrock - weathered, weak, brown - moderately strong below 5 feet					
10			▨		Tuffaceous Conglomerate Bedrock - weathered, weak, brown, with cobble clasts up to 12 inches in diameter					
			▨		Tuffaceaous Claystone Bedrock - weathered, moderately strong, yellow-brown					
15					Bottom of test pit at 12 feet					

☒ Bulk/Bag Sample
▥ Undisturbed Sample

▼ StabIlized Groundwater
▽ Groundwater At Time of Excavation

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Project No.: 162-001

Plate

9

Date		
Started:	9/18/2018	
Completed:	9/19/2018	
Backfilled:	Unknown	

TEST PIT LOG

Logged By: M Christensen
Equipment: Trackhoe
Location: See Plate 2

Test Pit No.

TP-8

Sheet 1 of 1

Depth (feet)	Sample Type	Groundwater	Graphic Log	Group Symbol	Material Description	Dry Density (pcf)	Moisture Content (%)	Minus #200 (%)	Liquid Limit	Plastic Limit
			▩		Topsoil; Lean CLAY - moist, dark brown					
	▥		▨		Lean CLAY - very stiff, moist, brown		6.0	91.0	29	11
5			▨		- moist below 5 feet					
	▥		▨		Fat CLAY - very stiff, moist, red-brown					
		▼	▦		Clayey GRAVEL with sand - medium dense, moist, brown, with cobbles up to 12 inches in diameter - wet below 8½ feet					
10					Bottom of test pit at 9½ feet					
15										

☒ Bulk/Bag Sample
▥ Undisturbed Sample
▼ StabIlized Groundwater
▽ Groundwater At Time of Excavation

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Project No.: 162-001

Plate

10

Date				
Started:	9/18/2018	**TEST PIT LOG**	Logged By: M Christensen	Test Pit No.
Completed:	9/19/2018		Equipment: Trackhoe	**TP-9**
Backfilled:	Unknown		Location: See Plate 2	Sheet 1 of 1

Depth (feet)	**Sample Type**	**Groundwater**	**Graphic Log**	**Group Symbol**	**Material Description**	**Dry Density (pcf)**	**Moisture Content (%)**	**Minus #200 (%)**	**Liquid Limit**	**Plastic Limit**
					Topsoil; Lean CLAY - moist, dark brown					
					Tuffaceous Conglomerate Bedrock - weathered, weak, brown, with cobble clasts up to 12 inches in diameter					
5					Tuffaceaous Claystone Bedrock - weathered					
	☒				Tuffaceous Conglomerate Bedrock - weathered, moderately strong, light brown, with cobble clasts up to 12 inches in diameter		2.0	0.4		
10 15					Bottom of test pit at 7 feet					

☒ Bulk/Bag Sample

▥ Undisturbed Sample

▼ Stabllized Groundwater

▽ Groundwater At Time of Excavation

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Project No.: 162-001

Plate

11

Date
Started: 9/18/2018
Completed: 9/19/2018
Backfilled: Unknown

TEST PIT LOG

Logged By: M Christensen
Equipment: Trackhoe
Location: See Plate 2

Test Pit No.

TP-10

Sheet 1 of 1

Depth (feet)	Sample Type	Groundwater	Graphic Log	Group Symbol	Material Description	Dry Density (pcf)	Moisture Content (%)	Minus #200 (%)	Liquid Limit	Plastic Limit
			▩		Topsoil; Lean CLAY - moist, dark brown					
	▥		▨		Tuffaceaous Claystone Bedrock - weathered, weak, red-brown	117.3	8.1	94.0	54	36
5			▨		Tuffaceous Conglomerate Bedrock - weathered, moderately strong, yellow-brown					
10 15					Bottom of test pit at 5½ feet					

☒ Bulk/Bag Sample
▥ Undisturbed Sample
▼ Stabllized Groundwater
▽ Groundwater At Time of Excavation

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Project No.: 162-001

Plate

12

Date
Started: 9/18/2018
Completed: 9/19/2018
Backfilled: Unknown

TEST PIT LOG

Logged By: M Christensen
Equipment: Trackhoe
Location: See Plate 2

Test Pit No.
TP-11
Sheet 1 of 1

Depth (feet)	Sample Type	Groundwater	Graphic Log	Group Symbol	Material Description	Dry Density (pcf)	Moisture Content (%)	Minus #200 (%)	Liquid Limit	Plastic Limit
			▓		Topsoil; Lean CLAY - moist, dark brown					
	▥		▨		Fat CLAY - very stiff, slightly moist, brown					
			▩		Tuffaceaous Claystone Bedrock - weathered, weak, brown					
5			▩		- moderately strong and light brown below 5 feet					
	▥		▩			104.3	17.9	90.7	51	29
					Bottom of test pit at 8 feet					
10										
15										

☒ Bulk/Bag Sample
▥ Undisturbed Sample
▼ StabIlized Groundwater
▽ Groundwater At Time of Excavation

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Project No.: 162-001

Plate
13

Date
Started: 9/18/2018
Completed: 9/19/2018
Backfilled: Unknown

TEST PIT LOG

Logged By: M Christensen
Equipment: Trackhoe
Location: See Plate 2

Test Pit No.

TP-12

Sheet 1 of 1

Depth (feet)	Sample Type	Groundwater	Graphic Log	Group Symbol	Material Description	Dry Density (pcf)	Moisture Content (%)	Minus #200 (%)	Liquid Limit	Plastic Limit
			▓		Topsoil; Lean CLAY - moist, dark brown					
	▥		▨		Tuffaceaous Claystone Bedrock - weathered, weak, light brown		10.3	68.4	28	12
5										
	☒		▨		Tuffaceous Conglomerate Bedrock - weathered, weak, light brown, with cobble clasts up to 12 inches in diameter		2.1	16.9		
					Bottom of test pit at 9 feet					
10										
15										

☒ Bulk/Bag Sample
▥ Undisturbed Sample

▼ StablIized Groundwater
▽ Groundwater At Time of Excavation

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Project No.: 162-001

Plate

14

RELATIVE DENSITY – COURSE GRAINED SOILS

Relative Density	SPT (blows/ft.)	3 In OD California Sampler (blows/ft.)	Relative Density (%)	Field Test
Very Loose	<4	<5	0 – 15	Easily penetrated with a ½ inch steel rod pushed by hand
Loose	4 – 10	5 – 15	15 – 35	Difficult to penetrate with a ½ inch steel rod pushed by hand
Medium Dense	10 – 30	15 – 40	35 – 65	Easily penetrated 1-foot with a steel rod driven by a 5 pound hammer
Dense	30 – 50	40 – 70	65 – 85	Difficult to penetrate 1-foot with a steel rod driven by a 5 pound hammer
Very Dese	>50	>70	85 - 100	Penetrate only a few inches with a steel rod driven by a 5 pound hammer

CONSISTENCY – FINE GRAINED SOILS

Consistency	SPT (blows/ft)	Torvane Undrained Shear Strength (tsf)	Pocket Penetrometer Undrained Shear Strength (tsf)	Field Test
Very Soft	<2	<0.125	<0.25	Easily penetrated several inches with thumb
Soft	2 – 14	0.125 – 0.25	0.25 – 0.5	Easily penetrated one inch with thumb
Medium Stiff	4 – 8	0.25 – 0.5	0.5 – 1.0	Penetrated over ½ inch by thumb with moderate effort. Molded by strong finger pressure
Stiff	8 – 15	0.5 – 1.0	1.0 – 2.0	Indented ½ inch by thumb with great effort
Very Stiff	15 – 30	1.0 – 2.0	2.0 – 4.0	Readily indented with thumbnail
Hard	>30	>2.0	>4.0	Indented with difficulty with thumbnail

CEMENTATION

Weakly	Crumbles or breaks with handling or little finger pressure
Moderately	Crumbles or breaks with considerable finger pressure
Strongly	Will not crumble or break with finger pressure

MOISTURE

Dry	Absence of moisture, dusty, dry to the touch
Moist	Damp but no visible water
Wet	Visible water, usually below water table

GRAIN SIZE

Description		Sieve Size	Grain Size (in)	Approximate Size
Boulders		>12”	>12”	Larger than basketball
Cobbles		3” – 12”	3” – 12”	Fist to basketball
Gravel	Coarse	3/4” - 3”	3/4” - 3”	Thumb to fist
	Fine	#4 – 3”	0.19 – 0.75	Pea to thumb
Sand	Coarse	#10 - #4	0.079 – 0.19	Rock salt to pea
	Medium	#40 - #10	0.017 – 0.079	Sugar to rock salt
	Fine	#200 - #40	0.0029 – 0.017	Flour to sugar
Silt/Clay		<#200	<0.0029	Flour sized or smaller

STRATAFICATION

Occasional	One or less per foot of thickness
Frequent	More than one per foot of thickness

MODIFIERS

Trace	<5%
Some	5-12%
With	>12%

STRATIFICATION

Seam	1/16 to 1/2 inch
Layer	1/2 to 12 inch

NOTES

1. The logs are subject to the limitations and conclusions presented in the report.
2. Lines separating strata represent approximate boundaries only. Actual transitions may be gradual.
3. Logs represent the soil conditions at the points explored at the time of our investigation.
4. Soils classifications shown on logs are based on visual methods . Actual designations (based on laboratory testing)may vary.

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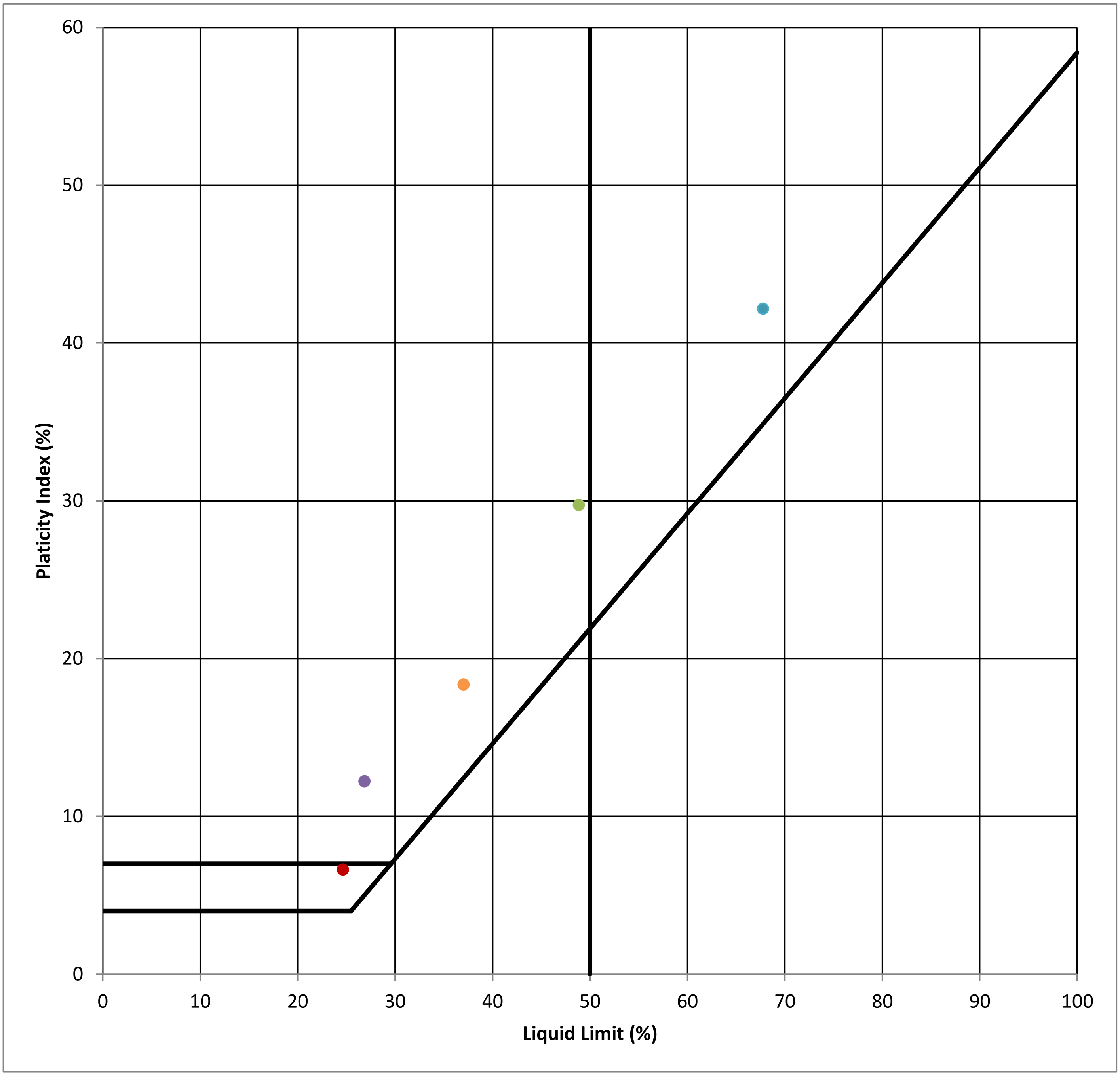
Soil Terms Key

Atterberg Limits



Location	Depth (ft)		Classification	Liquid Limit	PI
TP-1	2½	●	Fat CLAY	68	42
TP-3	5	●	Lean CLAY	37	18
TP-5	4	●	Lean CLAY	49	30
TP-6	4	●	Clayey GRAVEL with sand	27	12
TP-7	1½	●	Silty CLAY	25	7

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Project No.: 162-001

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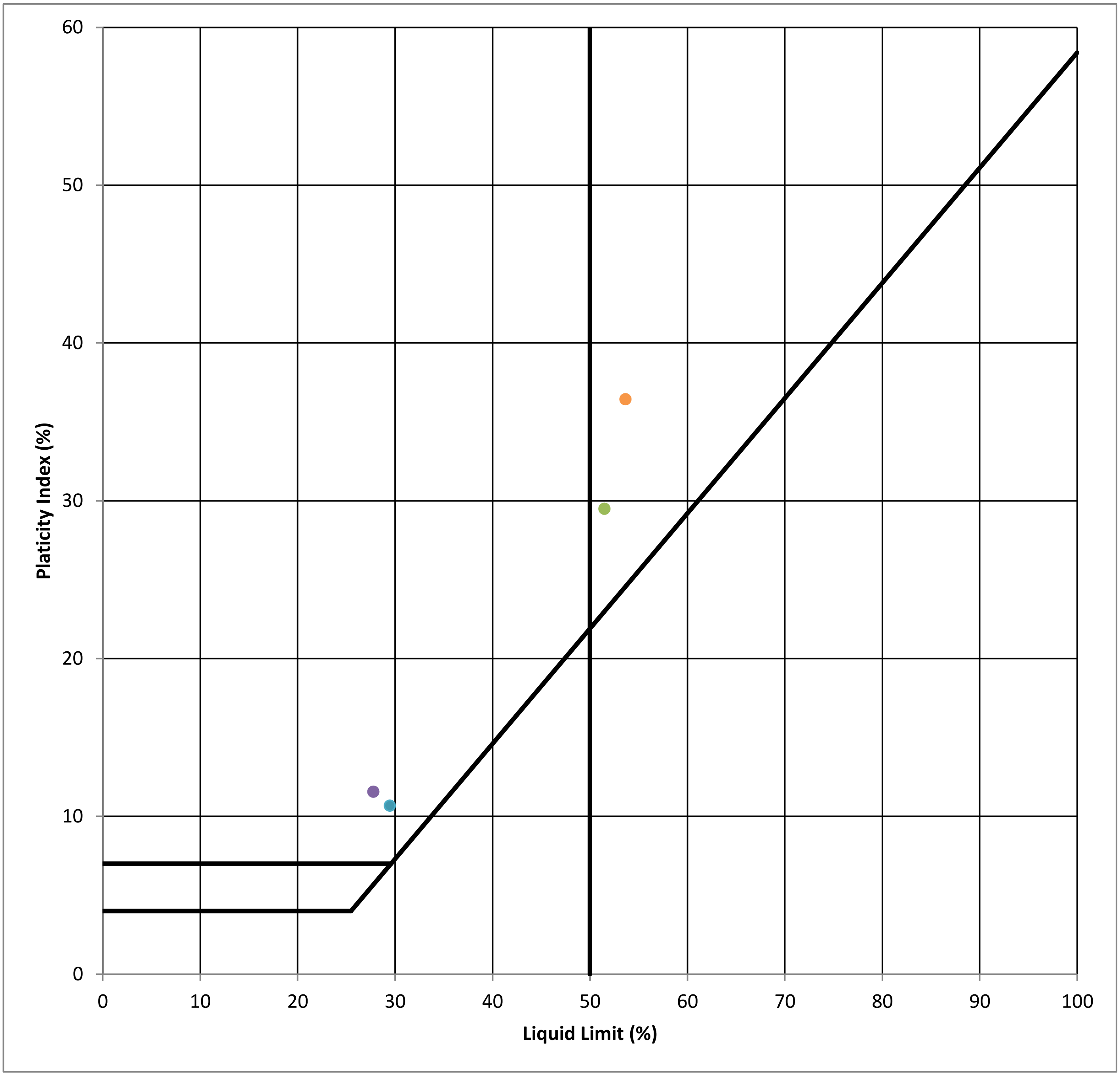
16

Atterberg Limits



Location	Depth (ft)		Classification	Liquid Limit	PI
TP-8	2	●	Lean CLAY	29	11
TP-10	2	●	Fat CLAY	54	36
TP-11	7	●	Fat CLAY	51	29
TP-12	3	●	Sandy Lean CLAY	28	12

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Project No.: 162-001

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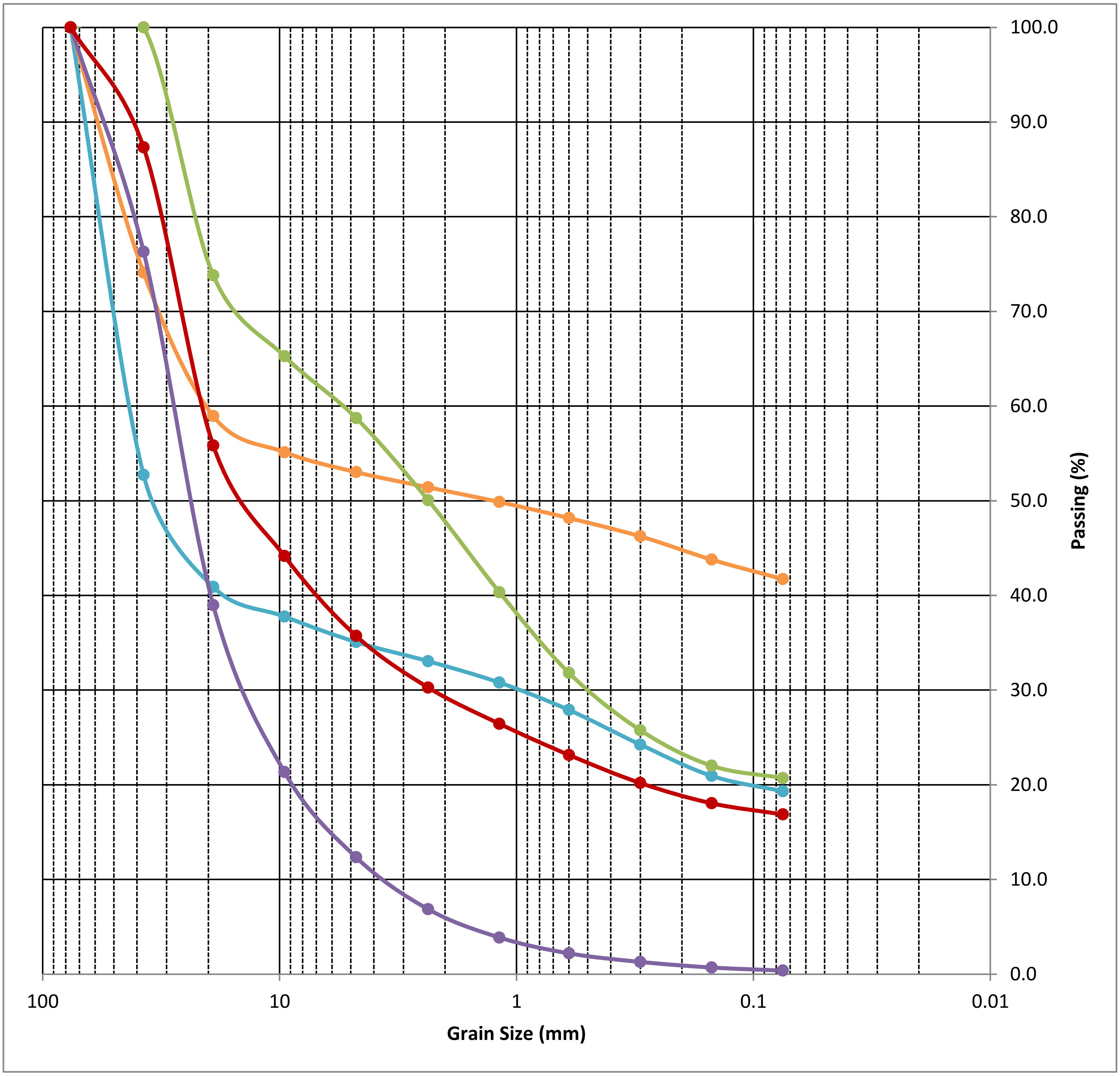
17

Grain Size Distribution



Location	Depth		Classification	% Gravel	% Sand	% Silt and Clay
TP-2	4	●	Clayey GRAVEL with sand	64.9	15.8	19.3
TP-4	3	●	Clayey GRAVEL with sand	47.0	11.3	41.7
TP-6	4	●	Clayey GRAVEL with sand	41.3	38.0	20.7
TP-9	6	●	Poorly Graded GRAVEL with sand	87.7	12.0	0.4
TP-12	8	●	Clayey GRAVEL with sand	64.3	18.9	16.9

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Project No.: 162-001

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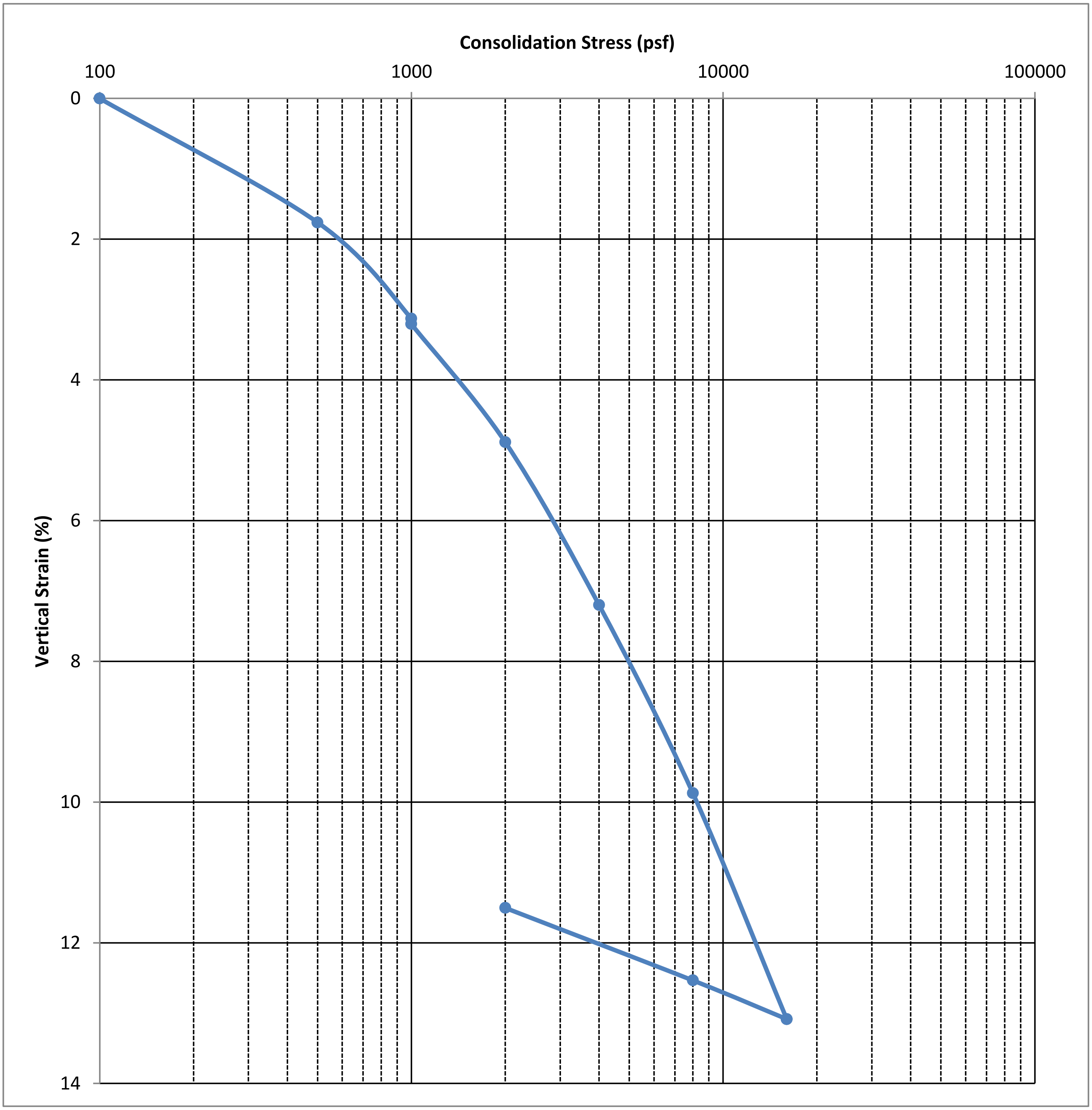
18

1-D Consolidation



Location	Depth (ft)	Dry Density (pcf)	Moisture Content (%)	σ_o (psf)	σ_p (psf)	C_c	C_r	OCR
TP-3	5	99.2	25.4	600	1,700	0.098	0.017	2.8

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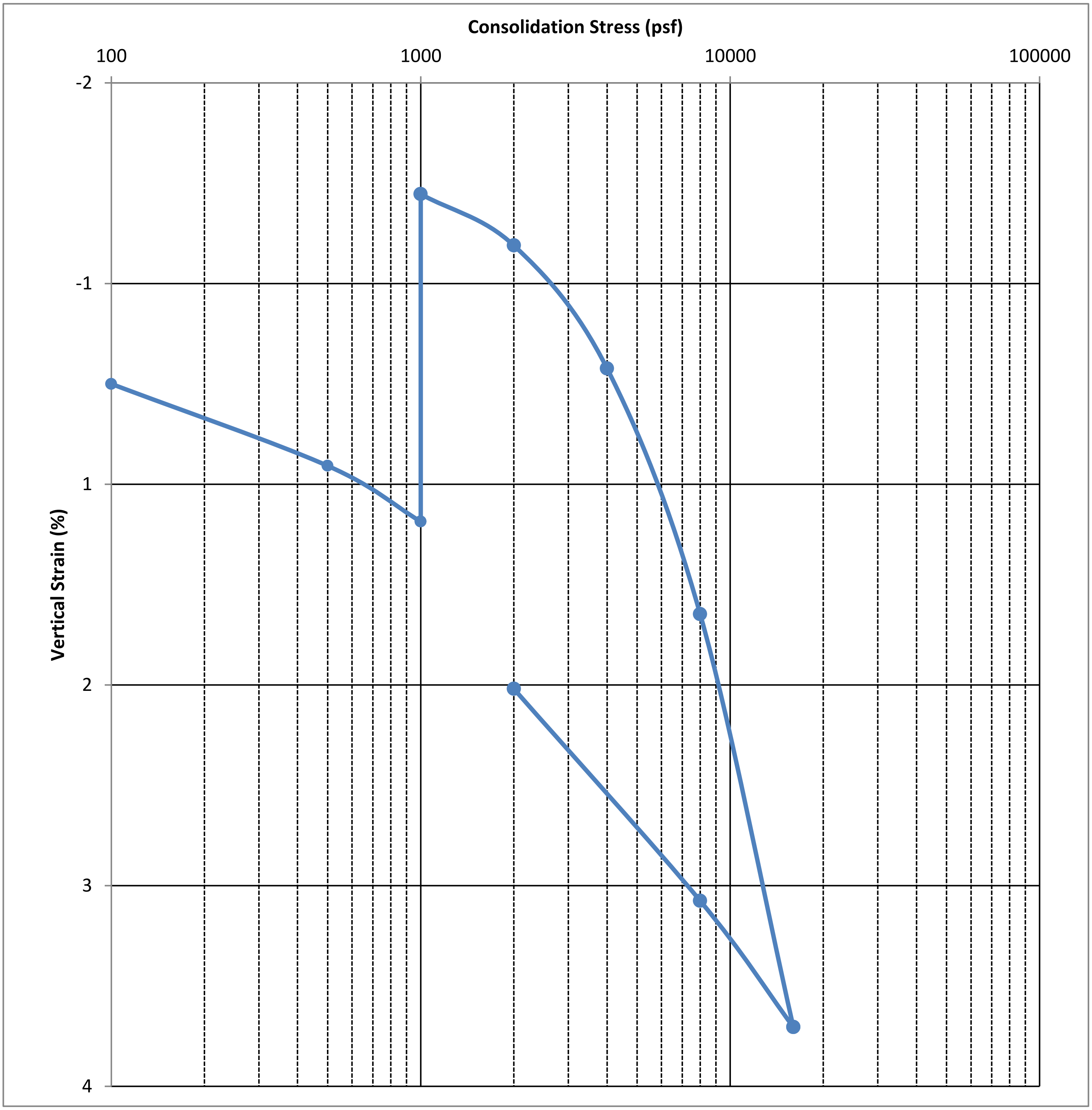
Weber County, Utah
Project No.: 162-001

Plate

19

1-D Consolidation



Location	Depth (ft)	Dry Density (pcf)	Moisture Content (%)	σ_o (psf)	σ_p (psf)	C_c	C_r	OCR
TP-5	4	106.6	19.2	500	5,000	0.055	0.019	10.0

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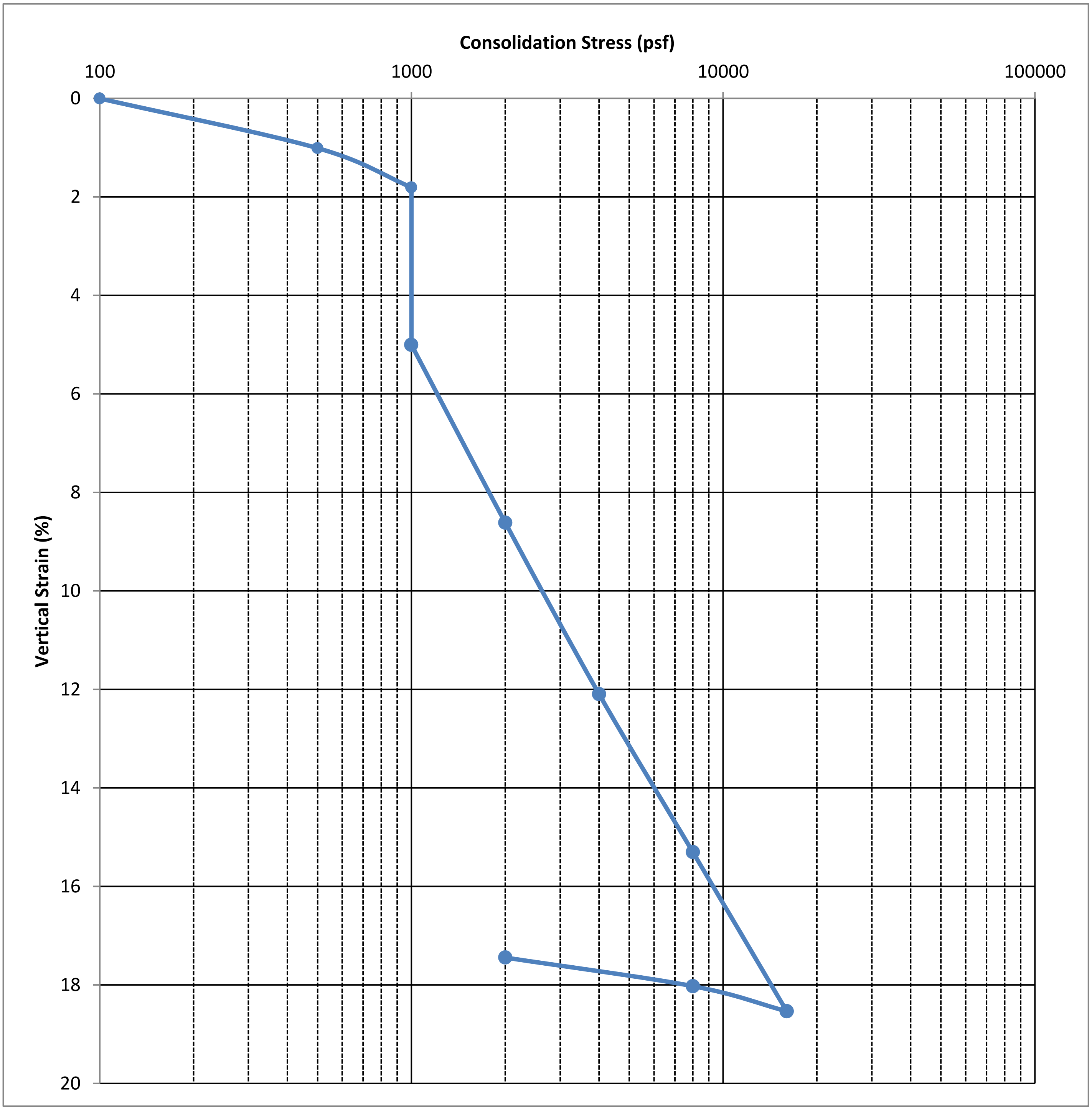
Weber County, Utah
Project No.: 162-001

Plate

20

1-D Consolidation



Location	Depth (ft)	Dry Density (pcf)	Moisture Content (%)	σ_o (psf)	σ_p (psf)	C_c	C_r	OCR
TP-7	1.5	96.8	5.2	200	1,000	0.107	0.012	5.0

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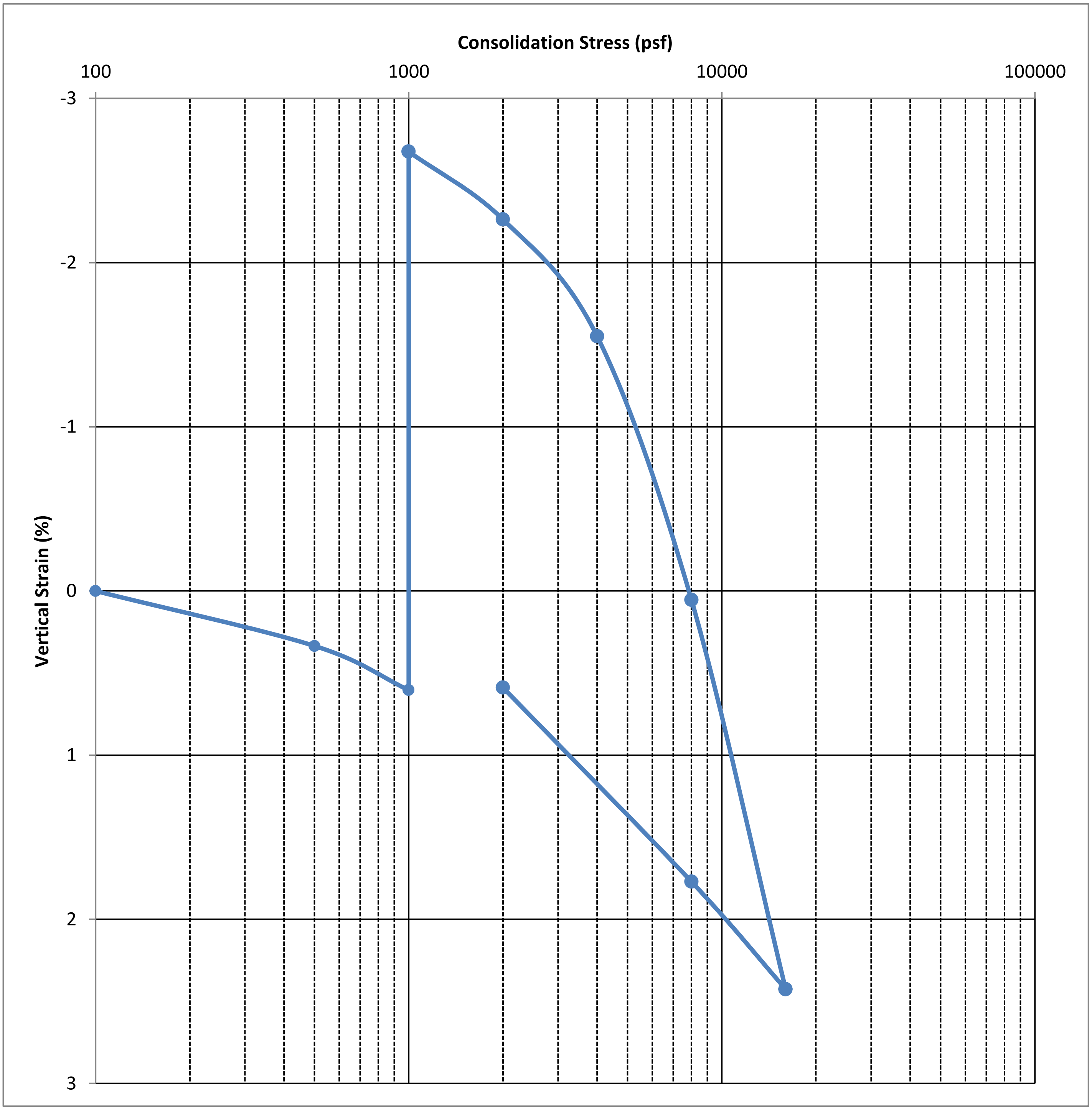
1-D Consolidation



Location	Depth (ft)	Dry Density (pcf)	Moisture Content (%)	σ_o (psf)	σ_p (psf)	C_c	C_r	OCR
TP-10	2	117.3	8.1	300	5,000	0.066	0.020	16.7

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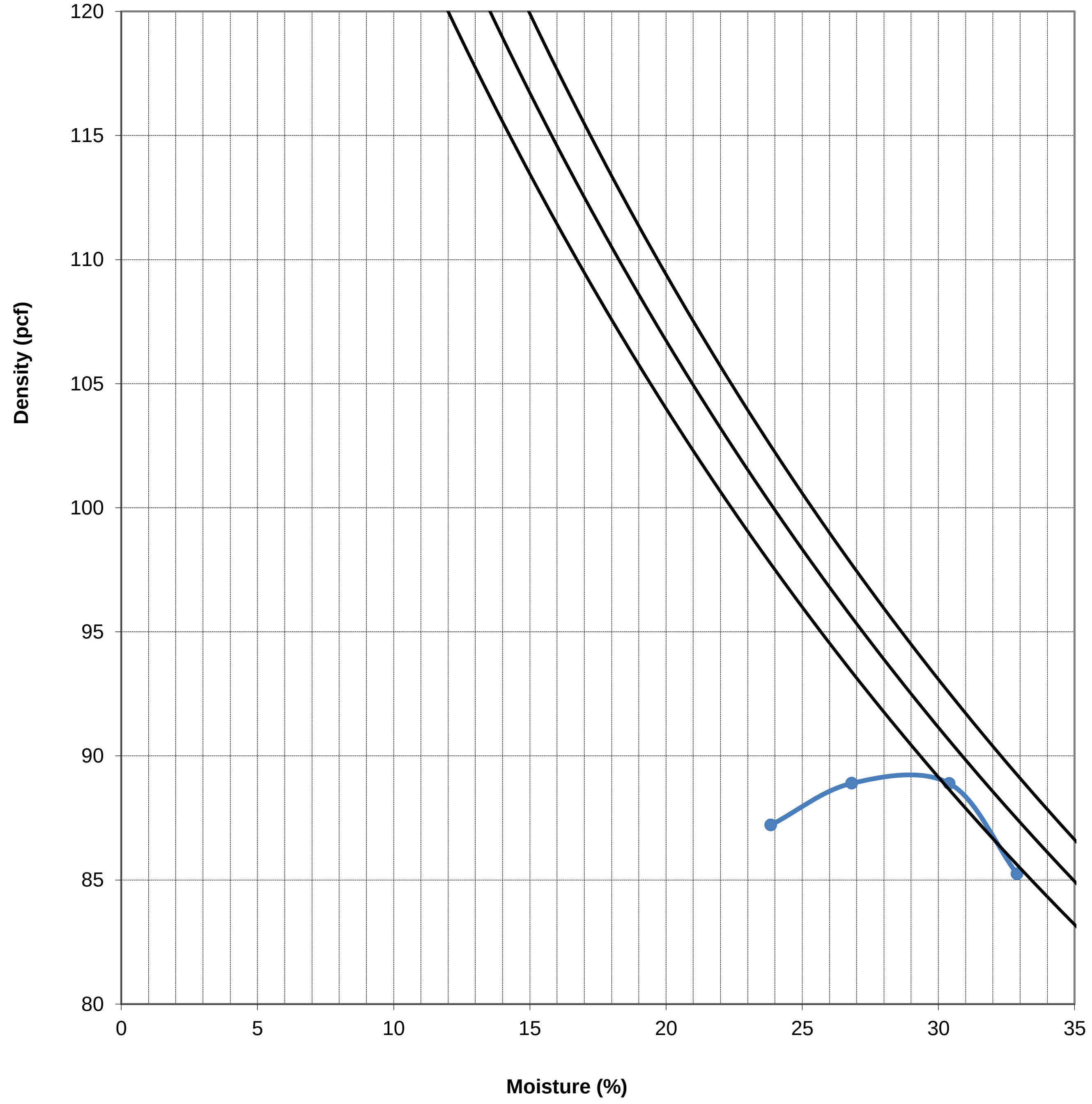
Project No.: 162-001

Plate

22

Moisture-Density Relationship



Location	Depth (ft)	Method	Maximum Density (psf)	Optimum Moisture (%)	Soil Type
TP-1	2½	ASTM D698	89	29	CH

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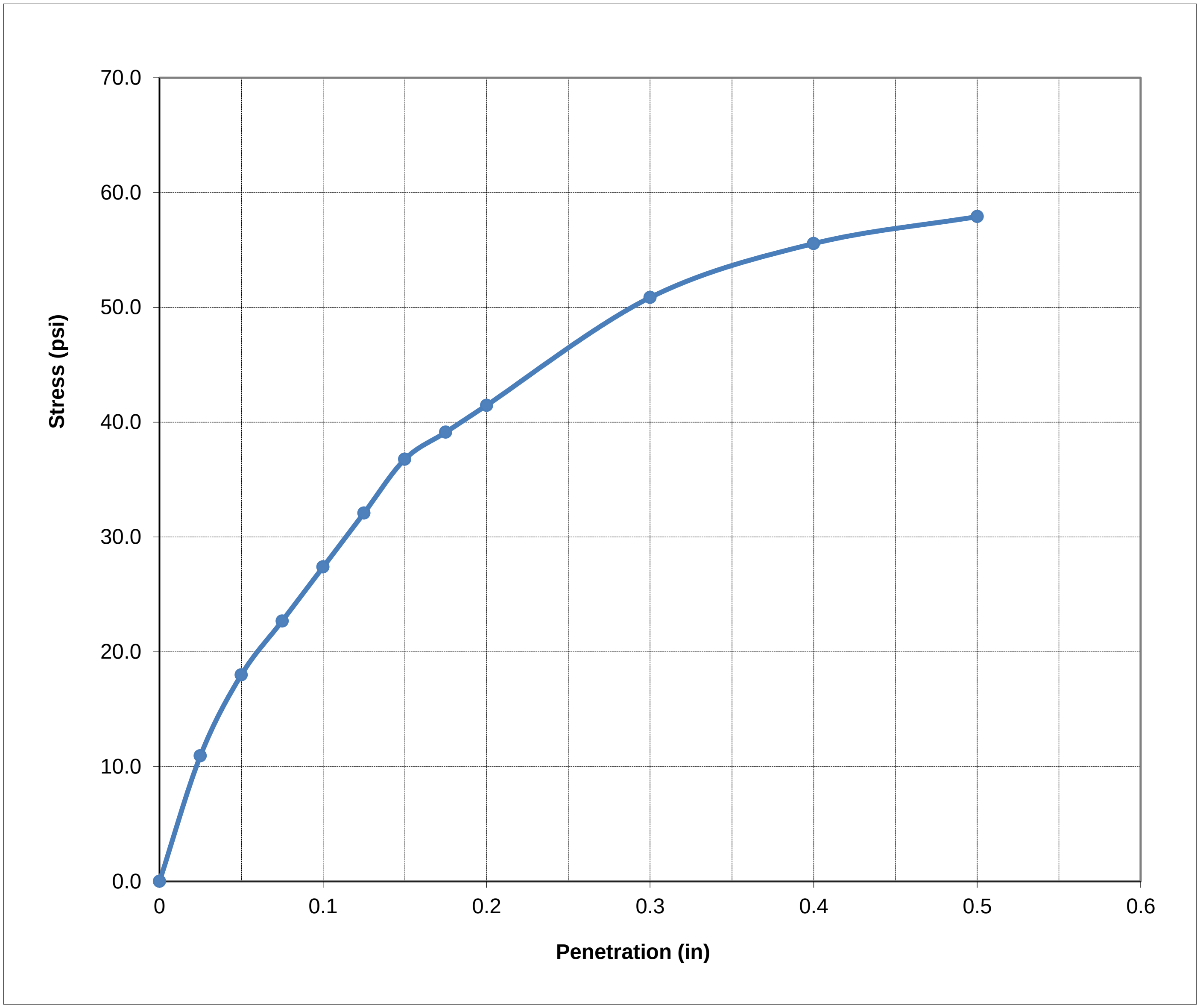
Weber County, Utah
Project No.: 162-001

Plate

23

California Bearing Ratio



Location	TP-1
Depth (ft)	2½
Method used for Preparation and Compaction	ASTM D698 D
Maximum Dry Density (pcf)	89.0
Optimum Moisture Content (%)	29.0
Dry Density of sample before soaking (pcf)	88.9
Dry Density of sample after soaking (pcf)	88.8
Moisture Content as compacted (%)	28.9
Moisture content top 1 inch after soaking (%)	35.4
Average Moisture Content after soaking (%)	31.0
Sucharge (psf)	10
Swell (%)	3.1
Bearing Ratio of Sample (%)	2.7

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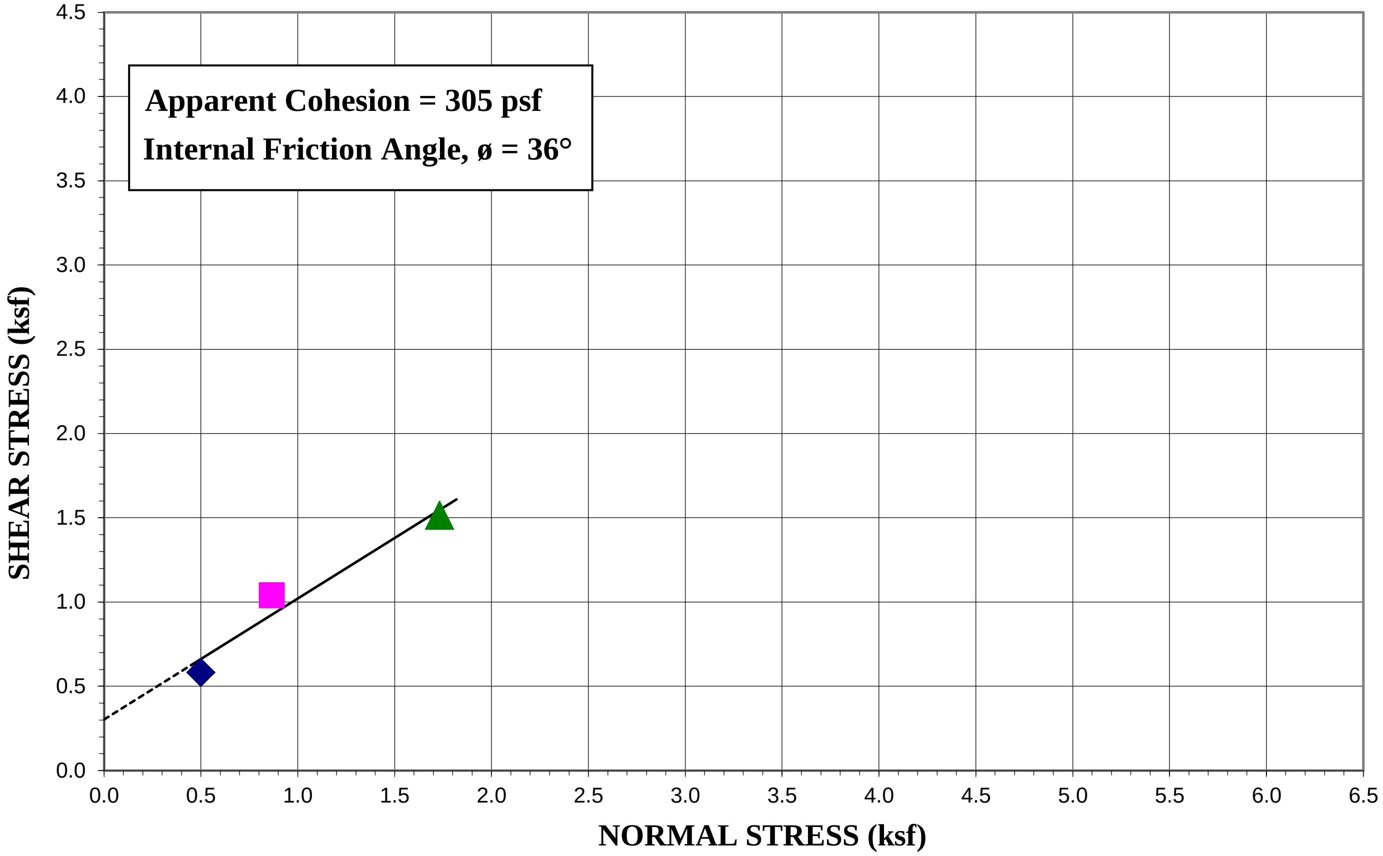
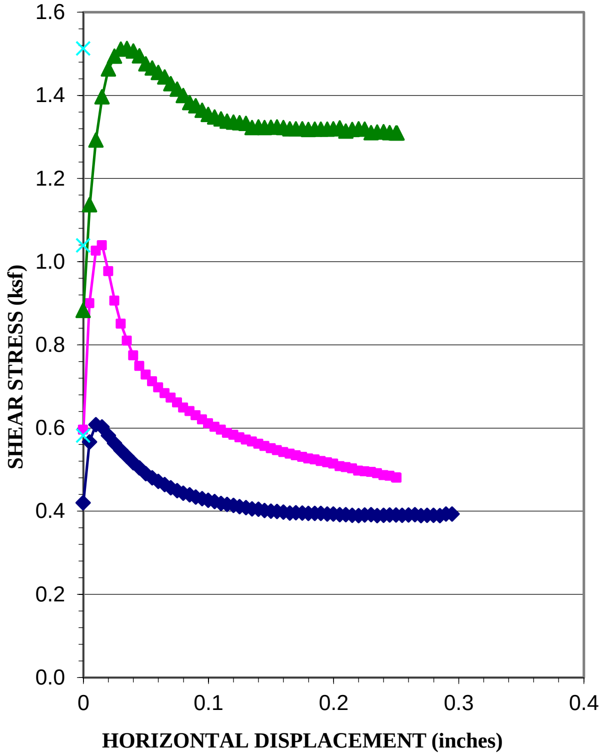
DIRECT SHEAR TEST





Location: TP-1	**Depth:** 5.0 ft		
Type of Test:	Consolidated Drained/Saturated		
Test No. (Symbol)	**1 (◆)**	**2 (■)**	**3 (▲)**
Sample Type:	Undisturbed		
Initial Height, in.	1	1	1
Diameter, in.	2.4	2.4	2.4
Dry Density Before, pcf	103.5	103.0	103.8
Moisture % Before	16.9	16.9	16.9
Normal Load, ksf	0.5	0.9	1.7
Shear Stress, ksf	0.58	1.04	1.51
Strain Rate	0.005 in/min		

Sample Properties	
Cohesion, psf	305
Friction Angle, ϕ	36
Liquid Limit, %	---
Plasticity Index, %	---
Percent Gravel	---
Percent Sand	---
Percent Passing No. 200 sieve	---
Classification	Claystone Bedrock

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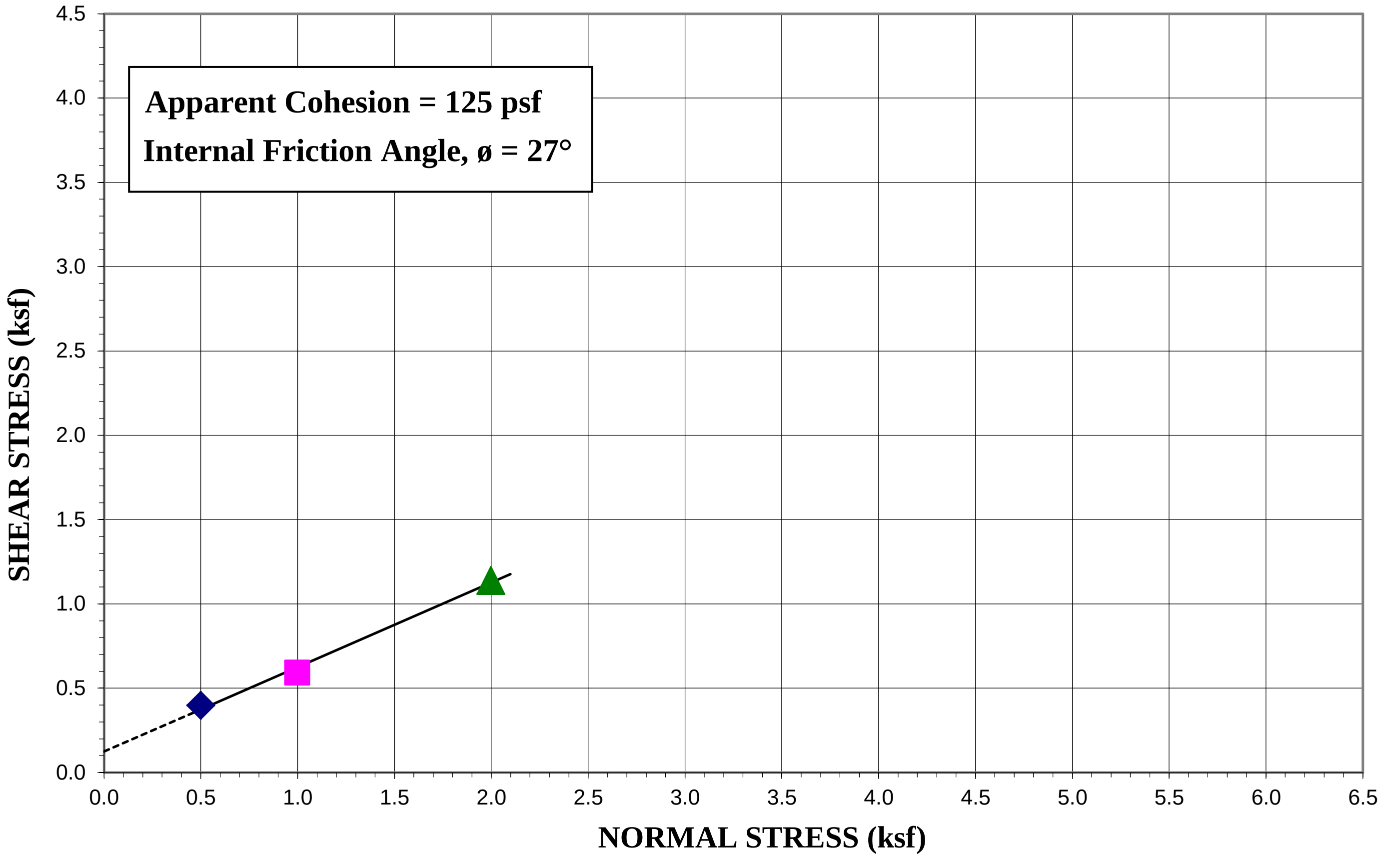
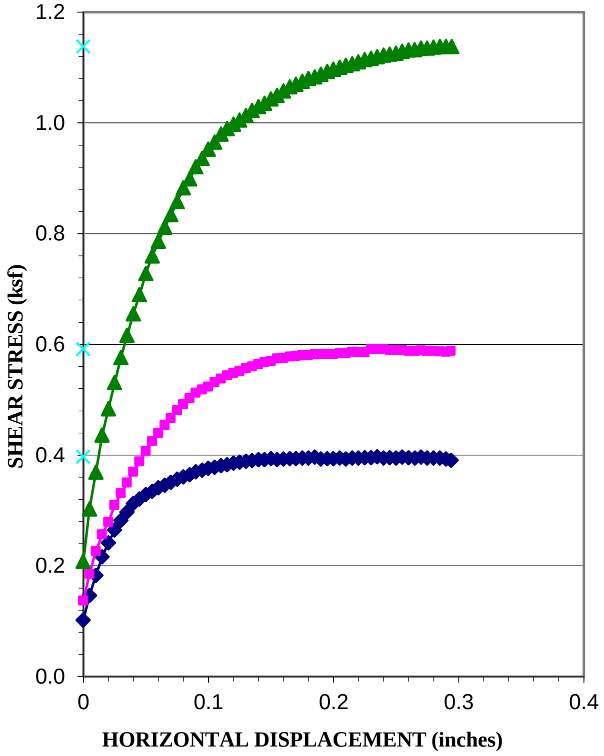
Weber County, Utah
Project No.: 162-001

Plate

25

DIRECT SHEAR TEST





Location: TP-5	**Depth:**	4.0 ft	
Type of Test:	Consolidated Drained/Saturated		
Test No. (Symbol)	**1 (◆)**	**2 (■)**	**3 (▲)**
Sample Type:	Fully Softened Remolded		
Initial Height, in.	1	1	1
Diameter, in.	2.4	2.4	2.4
Dry Density Before, pcf	82.1	84.6	82.7
Moisture % Before	37.3	37.3	37.3
Normal Load, ksf	0.5	1.0	2.0
Shear Stress, ksf	0.40	0.59	1.14
Strain Rate	0.005 in/min		

Sample Properties	
Cohesion, psf	125
Friction Angle, ϕ	27
Liquid Limit, %	49
Plasticity Index, %	30
Percent Gravel	---
Percent Sand	---
Percent Passing No. 200 sieve	97.1
Classification	Lean CLAY (CL)

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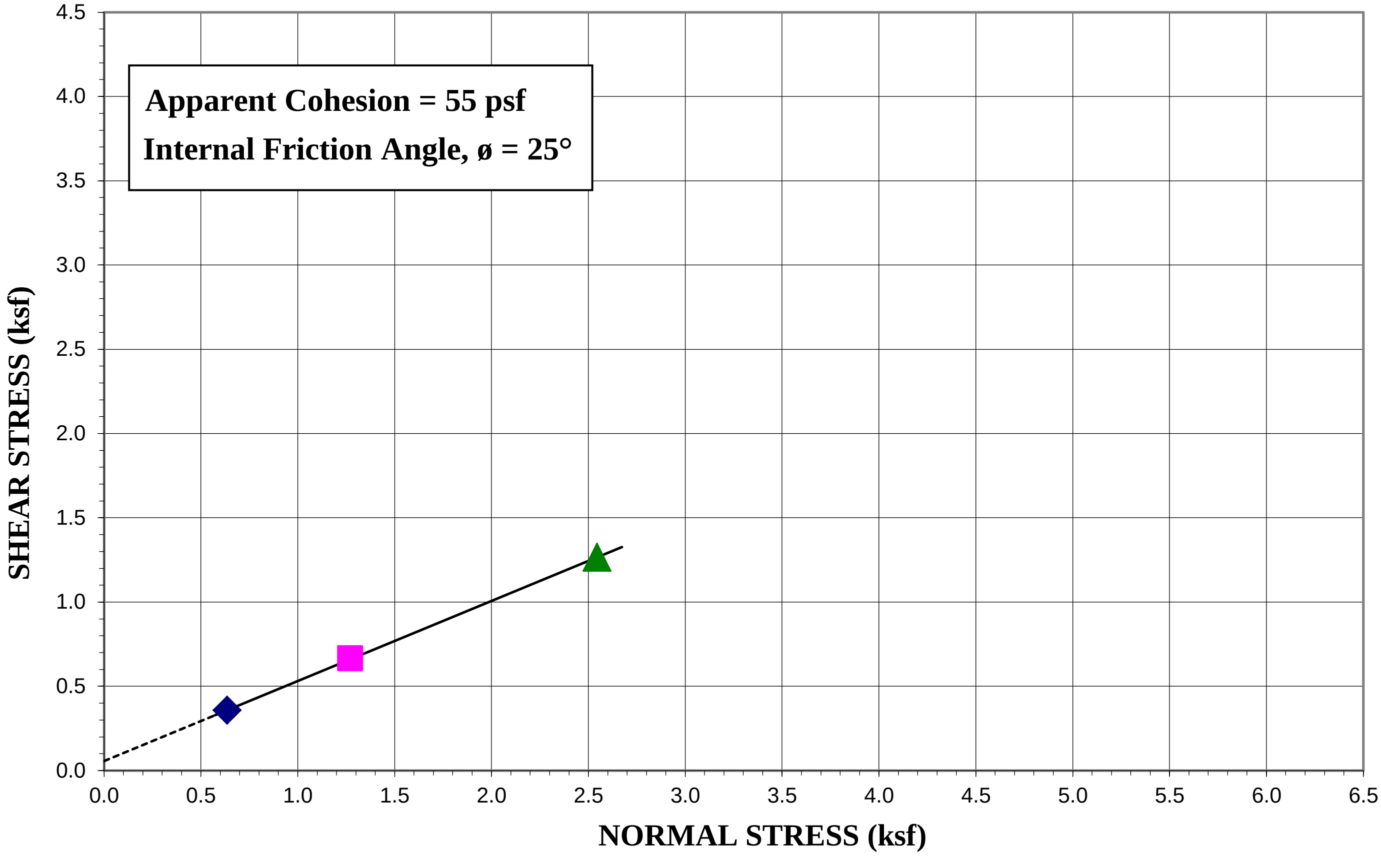
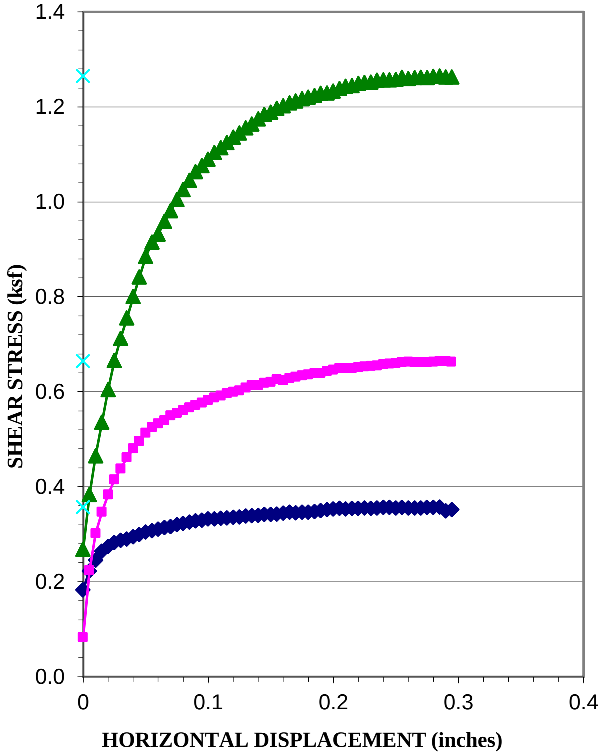
Weber County, Utah
Project No.: 162-001

Plate

26

DIRECT SHEAR TEST





Location: TP-10	**Depth:** 2.0 ft		
Type of Test:	Consolidated Drained/Saturated		
Test No. (Symbol)	**1 (◆)**	**2 (■)**	**3 (▲)**
Sample Type:	Undisturbed		
Initial Height, in.	1	1	1
Diameter, in.	2.4	2.4	2.4
Dry Density Before, pcf	87.0	87.8	88.2
Moisture % Before	31.4	31.4	31.4
Normal Load, ksf	0.6	1.3	2.5
Shear Stress, ksf	0.36	0.66	1.26
Strain Rate	0.01 in/min		

Sample Properties	
Cohesion, psf	55
Friction Angle, ϕ	25
Liquid Limit, %	54
Plasticity Index, %	36
Percent Gravel	---
Percent Sand	---
Percent Passing No. 200 sieve	94
Classification	Lean CLAY (CL)

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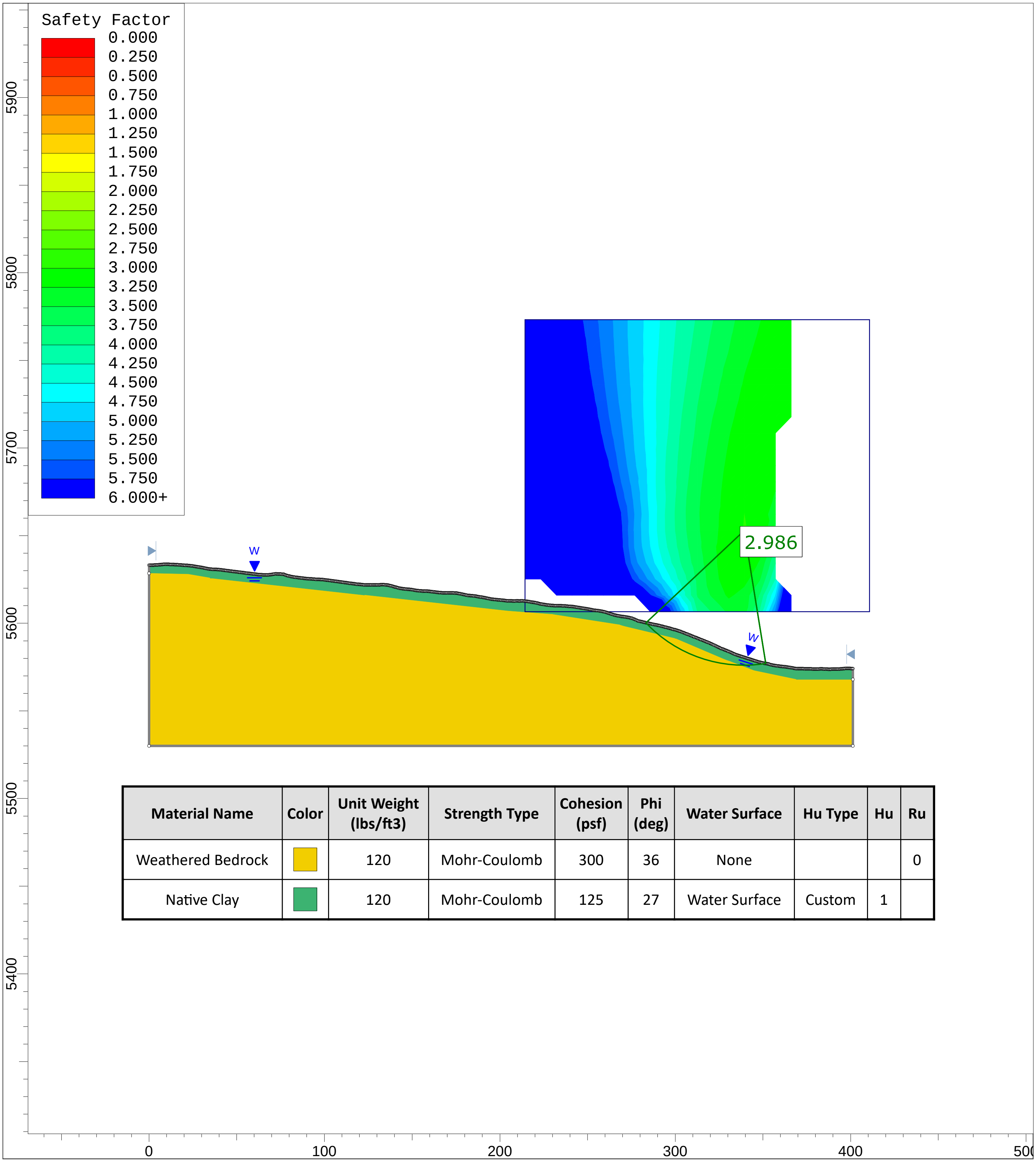
Project No.: 162-001

Plate

27



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Hu Type	Hu	Ru
Weathered Bedrock	🟨	120	Mohr-Coulomb	300	36	None			0
Native Clay	🟩	120	Mohr-Coulomb	125	27	Water Surface	Custom	1	

Profile A - Static

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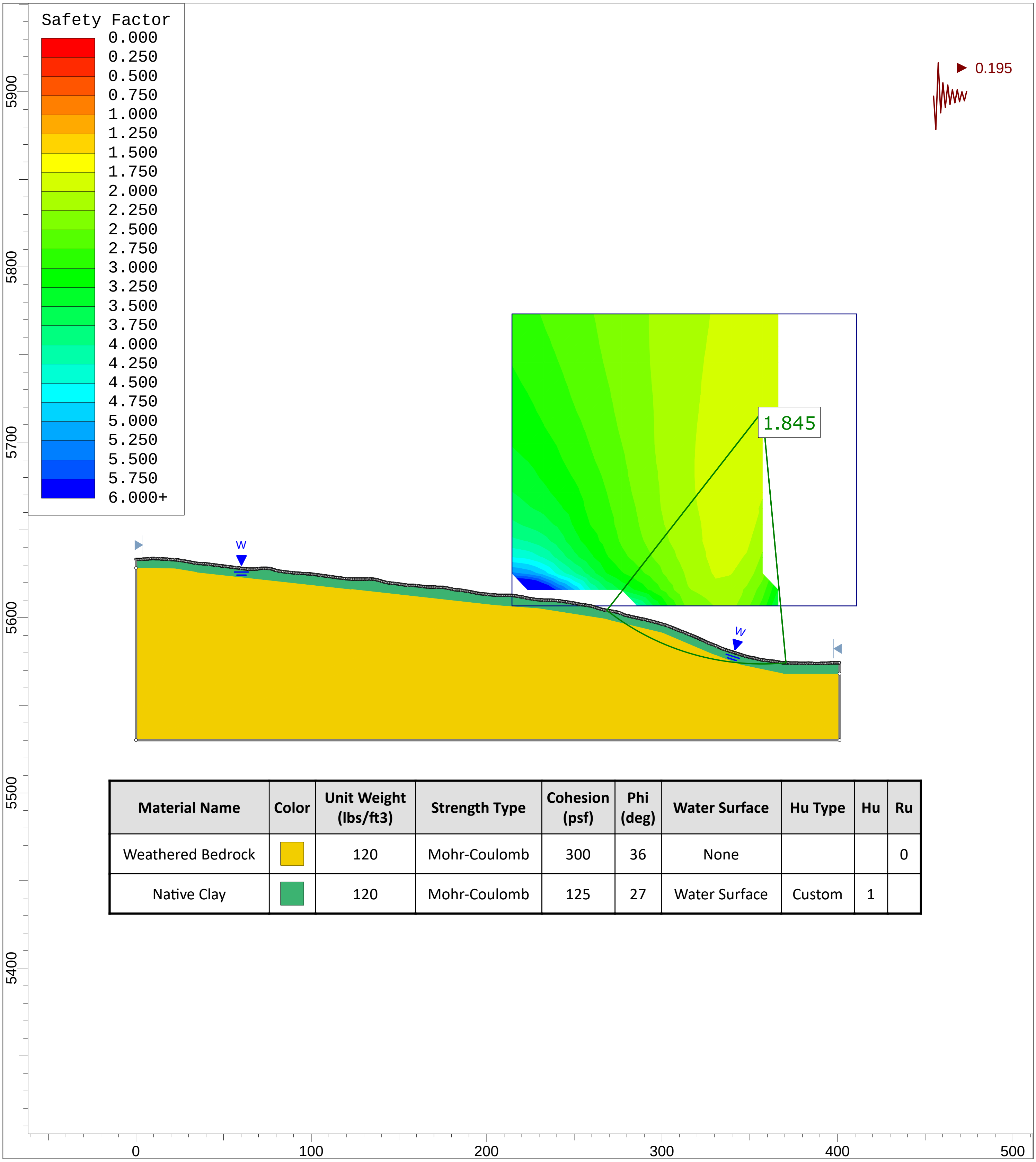
Weber County, Utah
162-001

Plate

28



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Hu Type	Hu	Ru
Weathered Bedrock	🟨	120	Mohr-Coulomb	300	36	None			0
Native Clay	🟩	120	Mohr-Coulomb	125	27	Water Surface	Custom	1	

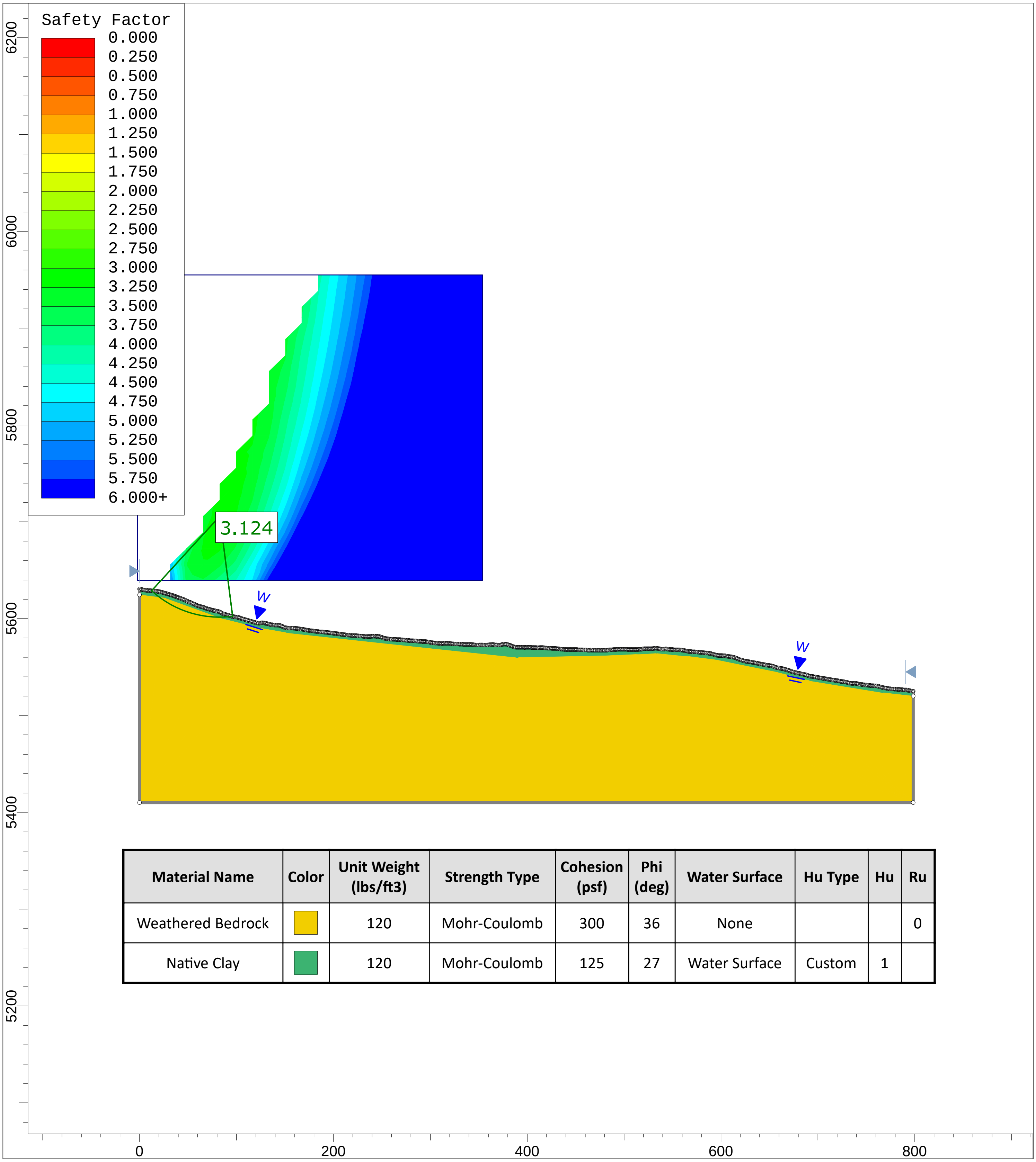
Profile A - Pseudo Static

Christensen Geotechnical

Brandon Janis
WAJ Enterprises Property
Weber County, Utah
162-001

Plate

29



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Hu Type	Hu	Ru
Weathered Bedrock	🟨	120	Mohr-Coulomb	300	36	None			0
Native Clay	🟩	120	Mohr-Coulomb	125	27	Water Surface	Custom	1	

Profile B - Static

Christensen
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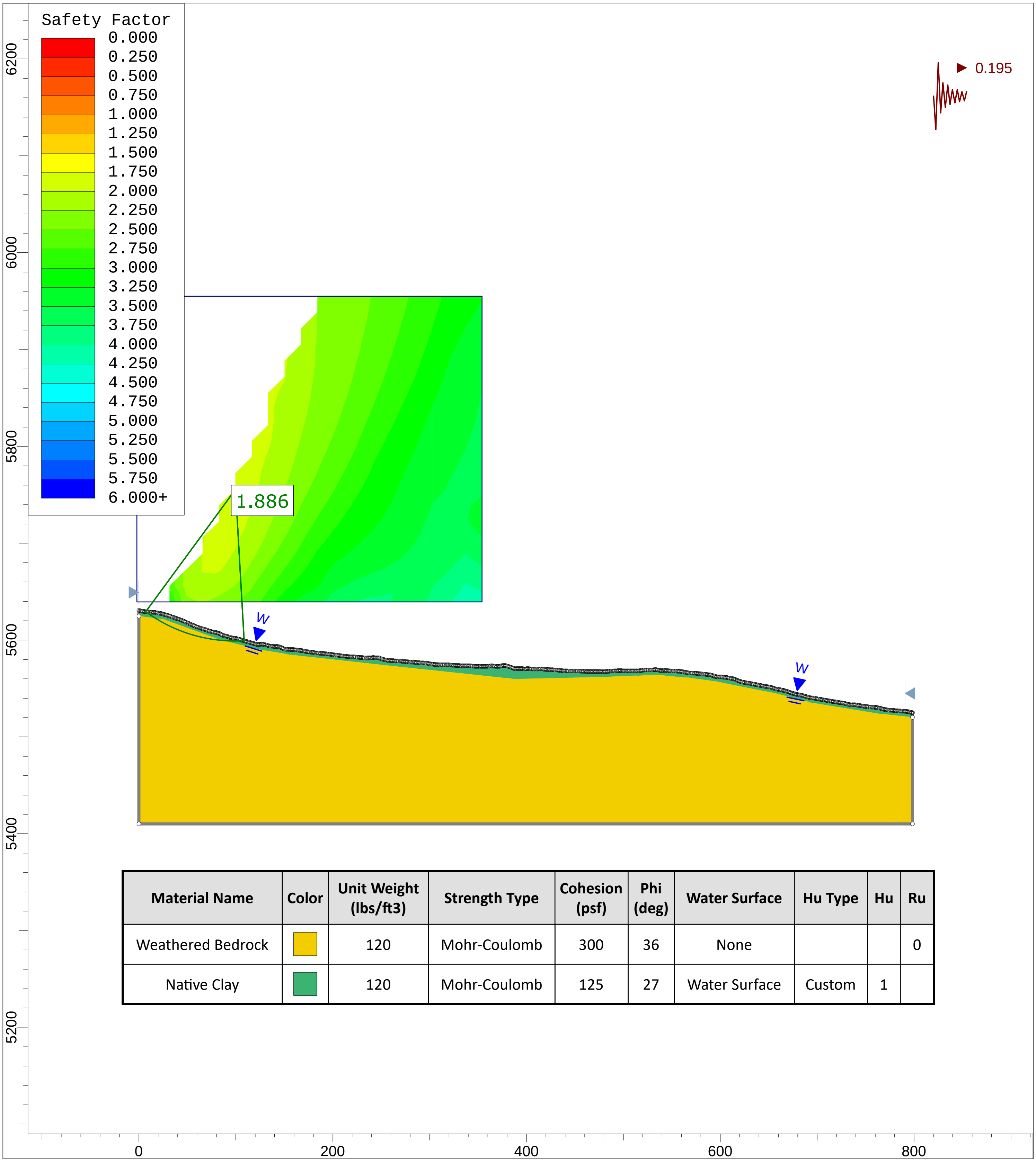
Brandon Janis
WAJ Enterprises Property
Weber County, Utah
162-001

Plate
30



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Hu Type	Hu	Ru
Weathered Bedrock	🟨	120	Mohr-Coulomb	300	36	None			0
Native Clay	🟩	120	Mohr-Coulomb	125	27	Water Surface	Custom	1	

Profile B - Pseudo Static

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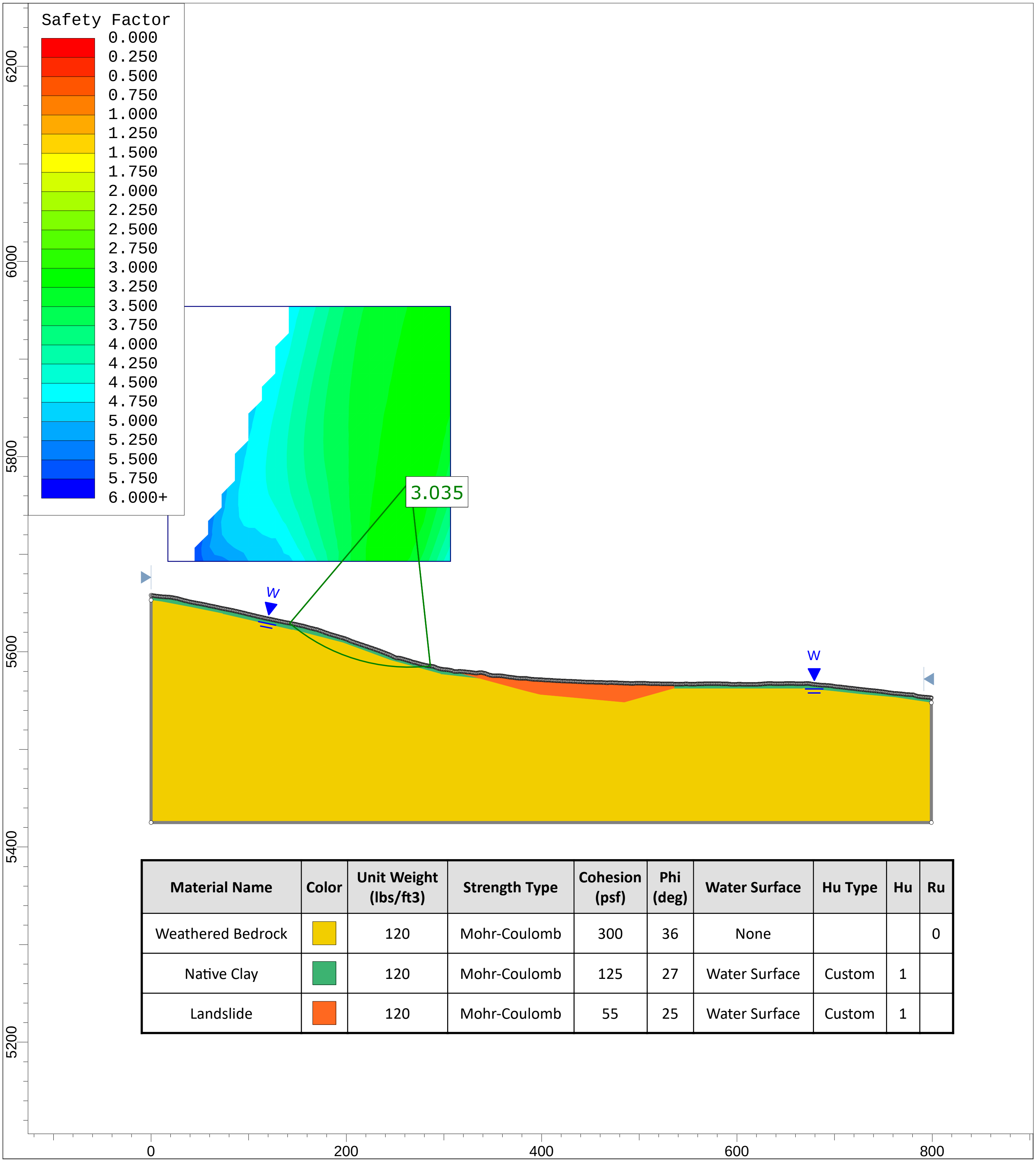
Brandon Janis
WAJ Enterprises Property
Weber County, Utah
162-001

Plate

31



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Hu Type	Hu	Ru
Weathered Bedrock	🟨	120	Mohr-Coulomb	300	36	None			0
Native Clay	🟩	120	Mohr-Coulomb	125	27	Water Surface	Custom	1	
Landslide	🟧	120	Mohr-Coulomb	55	25	Water Surface	Custom	1	

Profile C - Static

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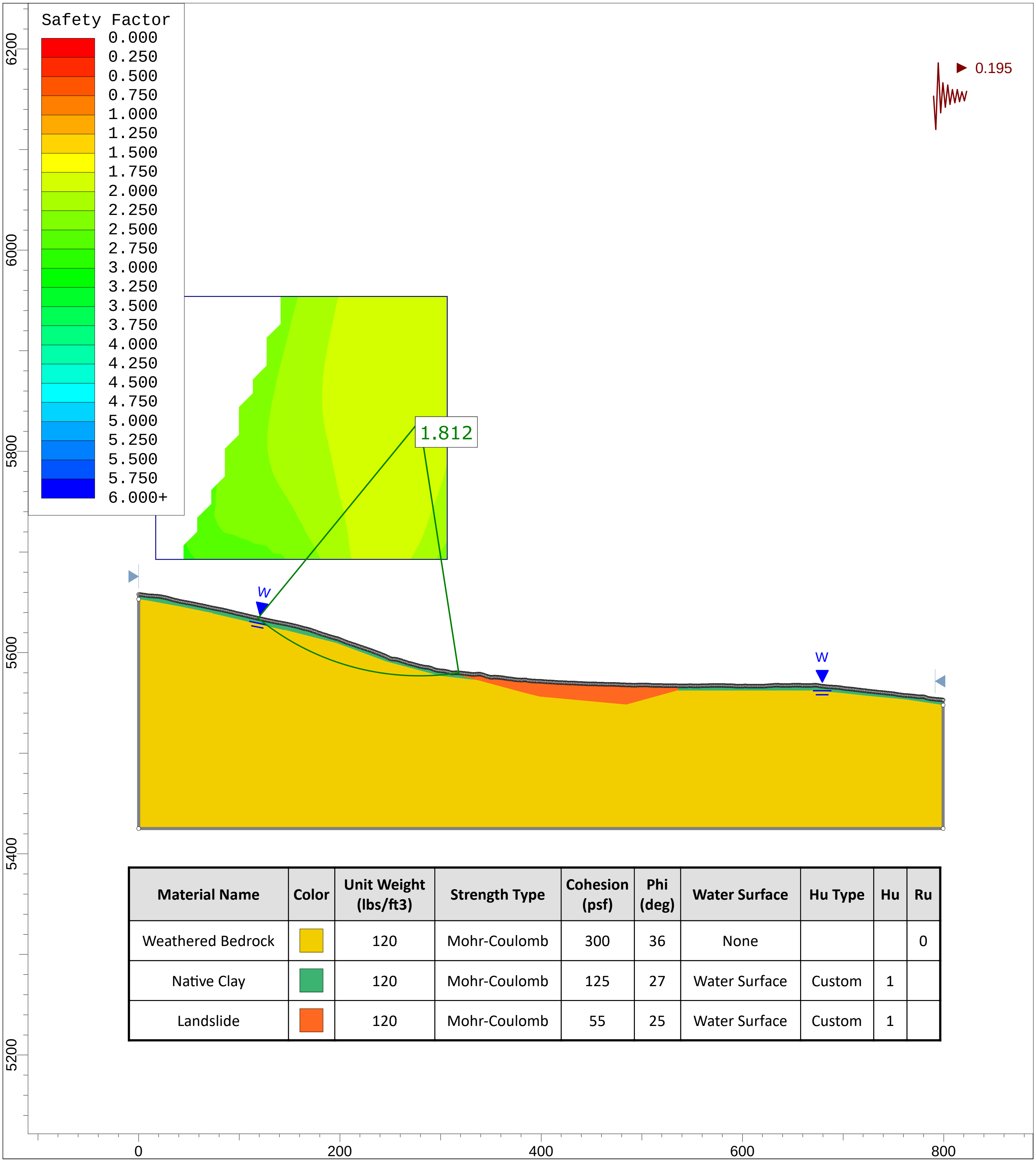
WAJ Enterprises Property
Weber County, Utah
162-001

Plate
32



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Hu Type	Hu	Ru
Weathered Bedrock	🟨	120	Mohr-Coulomb	300	36	None			0
Native Clay	🟩	120	Mohr-Coulomb	125	27	Water Surface	Custom	1	
Landslide	🟧	120	Mohr-Coulomb	55	25	Water Surface	Custom	1	

Profile C - Pseudo Static

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162-001

Plate

33