Geotechnical Investigation
Beckstead Property
Weber County, Utah

Prepared by:

Christensen Geotechnical

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Prepared for:

Brandon Janis

Geotechnical Investigation
Beckstead Property
Approximately 1860 North Big Sky Drive
Weber County, Utah
CG Project No.: 162-002

Prepared by:

10/12/18
LICENSED PROFESSIONAL ENGINEER
MARK I. CHRISTENSEN
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ATTACHED PLATES

1.0 INTRODUCTION

1.1 PURPOSE AND SCOPE OF WORK

This report presents the results of a geotechnical investigation performed for the Beckstead property located at approximately 1860 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 8, 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 September 26, 2018.

1.2 PROJECT DESCRIPTION

Based on conversations with Mr. Brandon Janis, we understand that the subject is to be developed with a single-family residence. The single-family residence is to be one to two stories in height with a basement. Footings loads for the proposed structure 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 2 test pits to depths of 8 and 9 feet below existing site grade. 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 and 4. A key to the symbols and terms used on the test pit logs may be found on Plate 5.

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 (%)		
				LL	PI	Gravel (+#4)	Sand	Silt/Clay (- #200)
TP-1	7	104.3	17.9	51	29			90.7
TP-2	3		10.3	28	12			68.4
TP-2	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 and 4), and more detailed laboratory results are presented on the laboratory testing Plate (Plates 6 through 8).

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. Slope grades on the property were generally less than 4 to 1 (horizontal to vertical), with flatter grades at the north end of the property. The total elevation change across the lot was approximately 100 feet. Vegetation at the site generally consisted of dense oak brush with other common trees. The site was bordered by an existing house to the west and undeveloped land on all other sides.

3.2 SUBSURFACE CONDITIONS

3.2.1 Soils

Based on the two test pits excavated for this investigation, the site is covered with 1½ 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 pit TP-1, 1 foot of colluvium soil was encountered between the topsoil and the bedrock. The colluvium consisted of Fat CLAY (CH).

3.2.2 Groundwater

Groundwater was not encountered within our test pits at the time of excavation. 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 = 41.29363° N Longitude = -111.84889° W			
Spectral Period (sec)	**Response Spectrum Spectral Acceleration (g)**		
0.2	$\mathbf{S_S}$=0.974g	$\mathbf{S_{MS}}$=0.984g	$\mathbf{S_{DS}}$=0.656g
1.0	$\mathbf{S_1}$= 0.335g	$\mathbf{S_{M1}}$=0.491g	$\mathbf{S_{D1}}$=0.327g

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, 1½ 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 the 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 structure 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. The location of the profile assessed is shown on Plate 2. The profile subsurface conditions used in our analyses were based on a cross section of the profile 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. The soil strengths used in our analyses were based on three direct shear tests which were performed on samples of clay soil and claystone bedrock for the WAJ Enterprises property located north of the subject lot. Two of these tests were performed on fully softened, remolded samples and one of the tests was performed on a sample of undisturbed claystone bedrock. The results of this 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 profile was 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 site has safety factors greater than 1.5 and 1.0 for the static and pseudo static conditions and is therefore considered adequate for residential development. The results of our slope stability assessments may be found on Plates 9 and 10.

The slope stability analyses presented above are based on the assumption that no significant cuts or fills will occur during the development of the site. 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.

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 8, 2018, "Geologic Hazards Evaluation, Beckstead Property, About 1860 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.

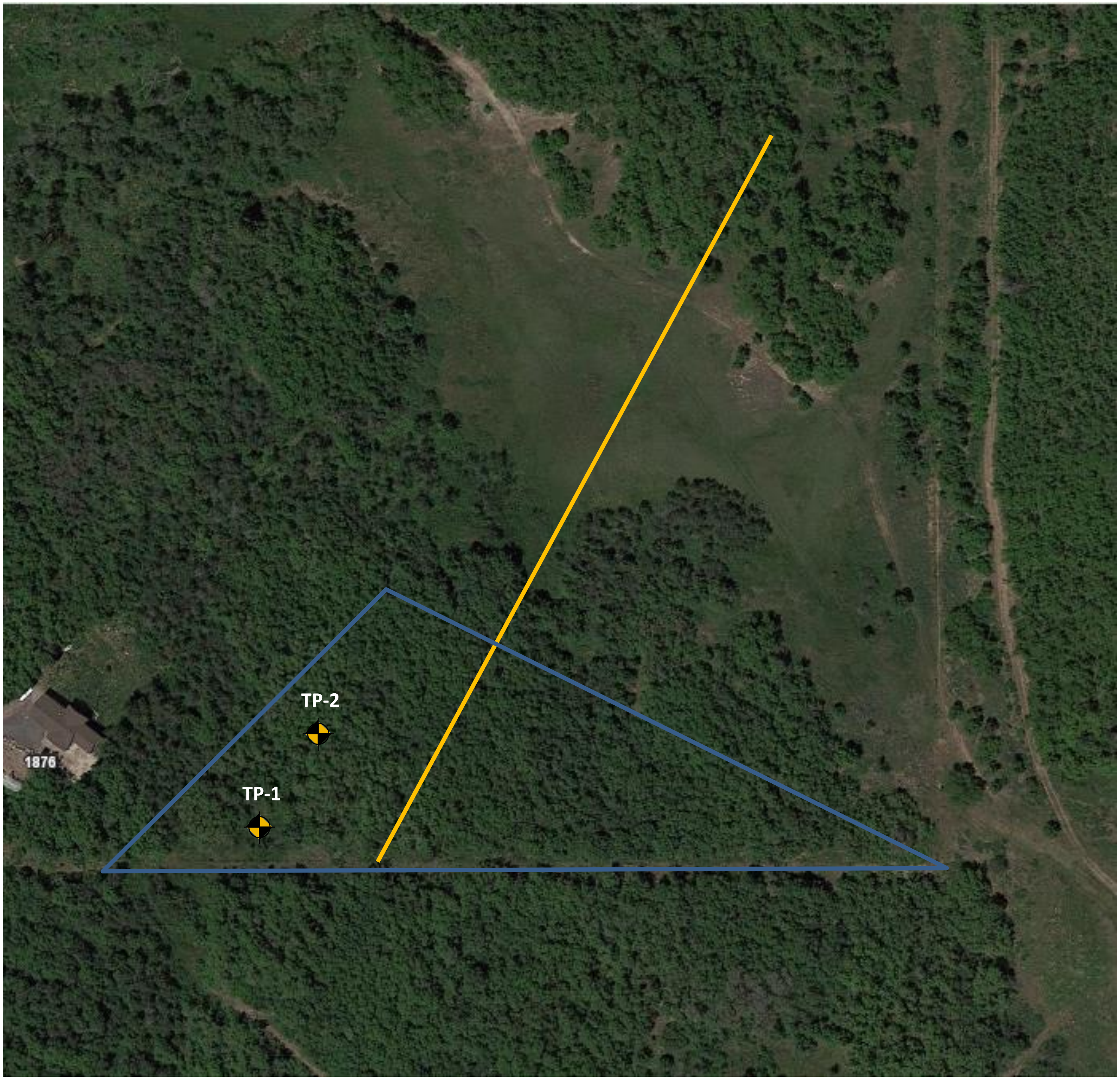
Base Photo: Utah AGRC

Drawing Not to Scale

Approximate Project Boundary

N

Christensen Geotechnical	Brandon Janis Beckstead Property Weber County, Utah Project No. 162-002 **Vicinity Map**	Plate 1



Approximate Test Pit Location

Slope Stability Profile

Approximate Project Boundary

Base Photo: Utah AGRC

Drawing Not to Scale

N

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Brandon Janis
Beckstead Property
Weber County, Utah
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Exploration Location Map

Plate 2

TEST PIT LOG

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

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

Test Pit No. **TP-1**

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					
	Undisturbed				Fat CLAY - very stiff, slightly moist, brown					
					Tuffaceaous Claystone Bedrock - weathered, weak, brown					
5					- moderately strong and light brown below 5 feet					
	Undisturbed					104.3	17.9	90.7	51	29
					Bottom of test pit at 8 feet					
10										
15										

☒ Bulk/Bag Sample
▥ Undisturbed Sample
Stablllized Groundwater
Groundwater At Time of Excavation

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Plate
3

Date			Logged By: M Christensen	Test Pit No.
Started:	9/18/2018	**TEST PIT LOG**	Equipment: Trackhoe	**TP-2**
Completed:	9/19/2018		Location: See Plate 2	
Backfilled:	Unknown			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
0					Topsoil; Lean CLAY - moist, dark brown					
2.5					Tuffaceaous Claystone Bedrock - weathered, weak, light brown					
	Undisturbed						10.3	68.4	28	12
6					Tuffaceous Conglomerate Bedrock - weathered, weak, light brown, with cobble clasts up to 12 inches in diameter					
	Bulk/Bag						2.1	16.9		
9					Bottom of test pit at 9 feet					

Depth scale: 5, 10, 15

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

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Plate
4

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

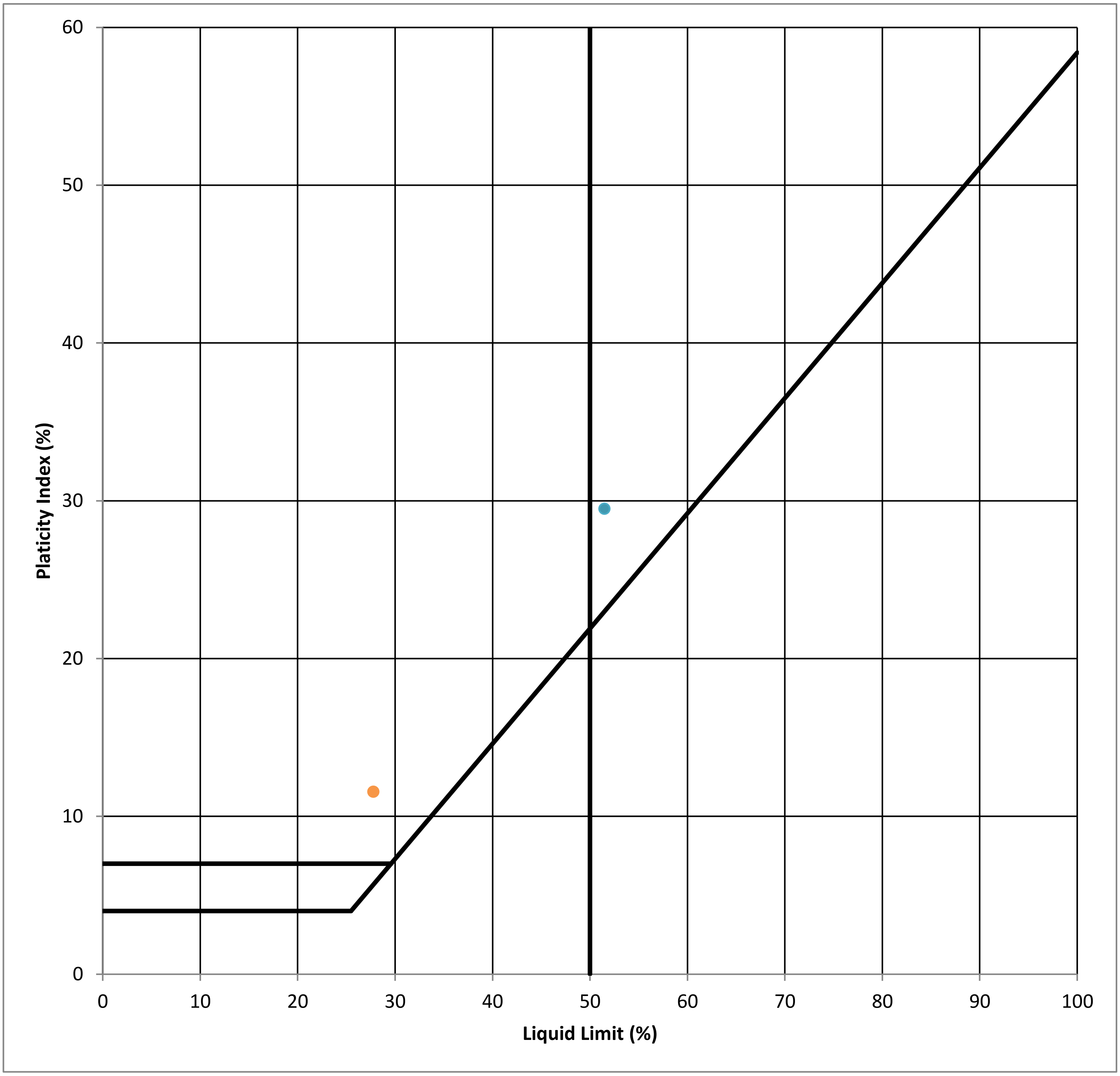
Seam	1/16 to 1/2 inch
Layer	1/2 to 12 inch

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.

Christensen Geotechnical	**Soil Terms Key**	Plate 5

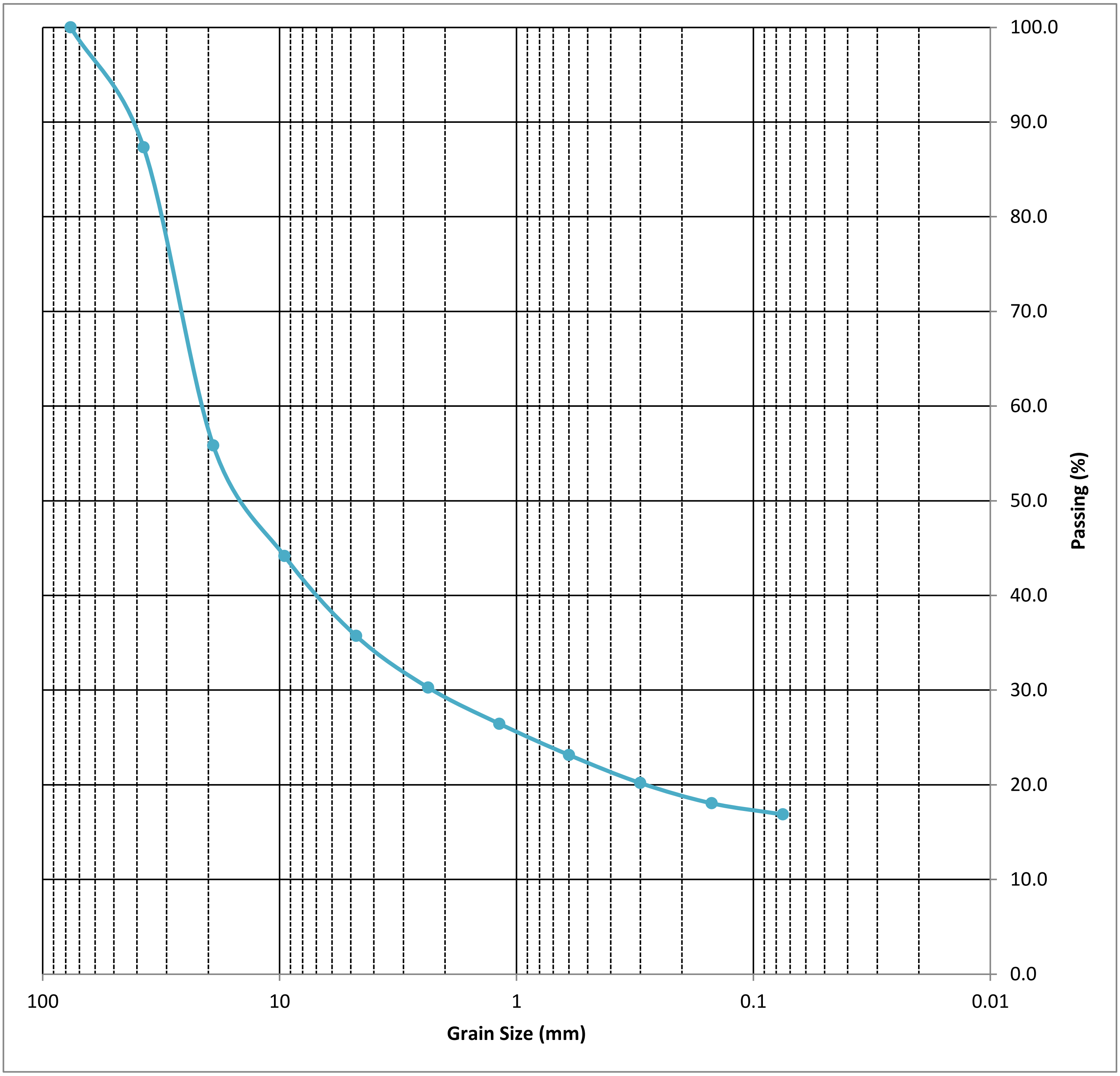
Atterberg Limits



Location	Depth (ft)		Classification	Liquid Limit	PI
TP-1	7	●	Fat CLAY	51	29
TP-2	3	●	Sandy Lean CLAY	28	12

Christensen Geotechnical	Brandon Janis Beckstead Property Weber County, Utah Project No.: 162-002	Plate 6

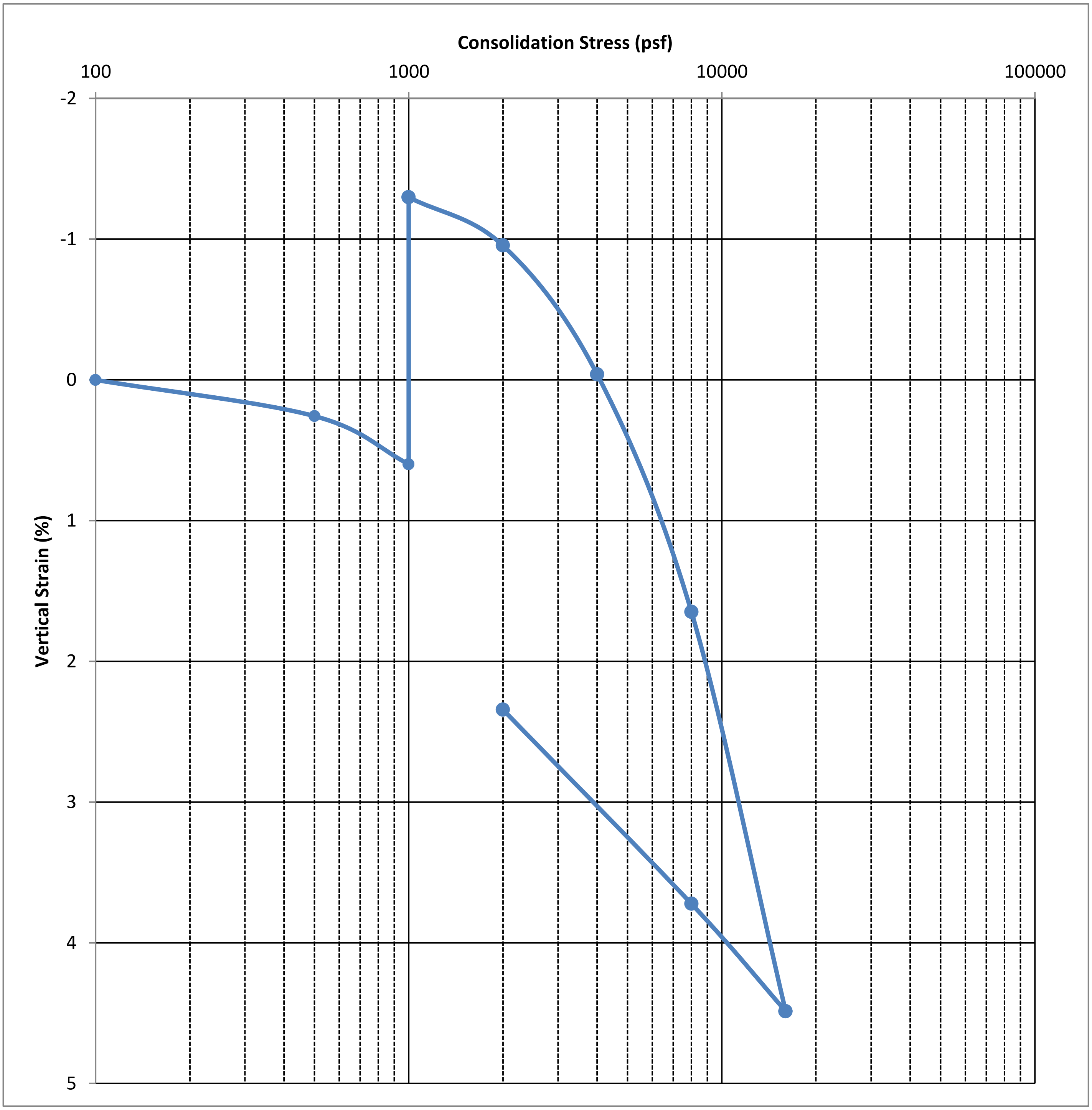
Grain Size Distribution



Location	Depth		Classification	% Gravel	% Sand	% Silt and Clay
TP-2	8	●	Clayey Gravel with sand	64.3	18.9	16.9

Christensen Geotechnical	Brandon Janis Beckstead Property Weber County, Utah Project No.: 162-002	Plate 7

1-D Consolidation



Location	Depth (ft)	Dry Density (pcf)	Moisture Content (%)	σ_o (psf)	σ_p (psf)	C_c	C_r	OCR
TP-1	8	104.3	17.9	1,000	4,800	0.075	0.024	4.8

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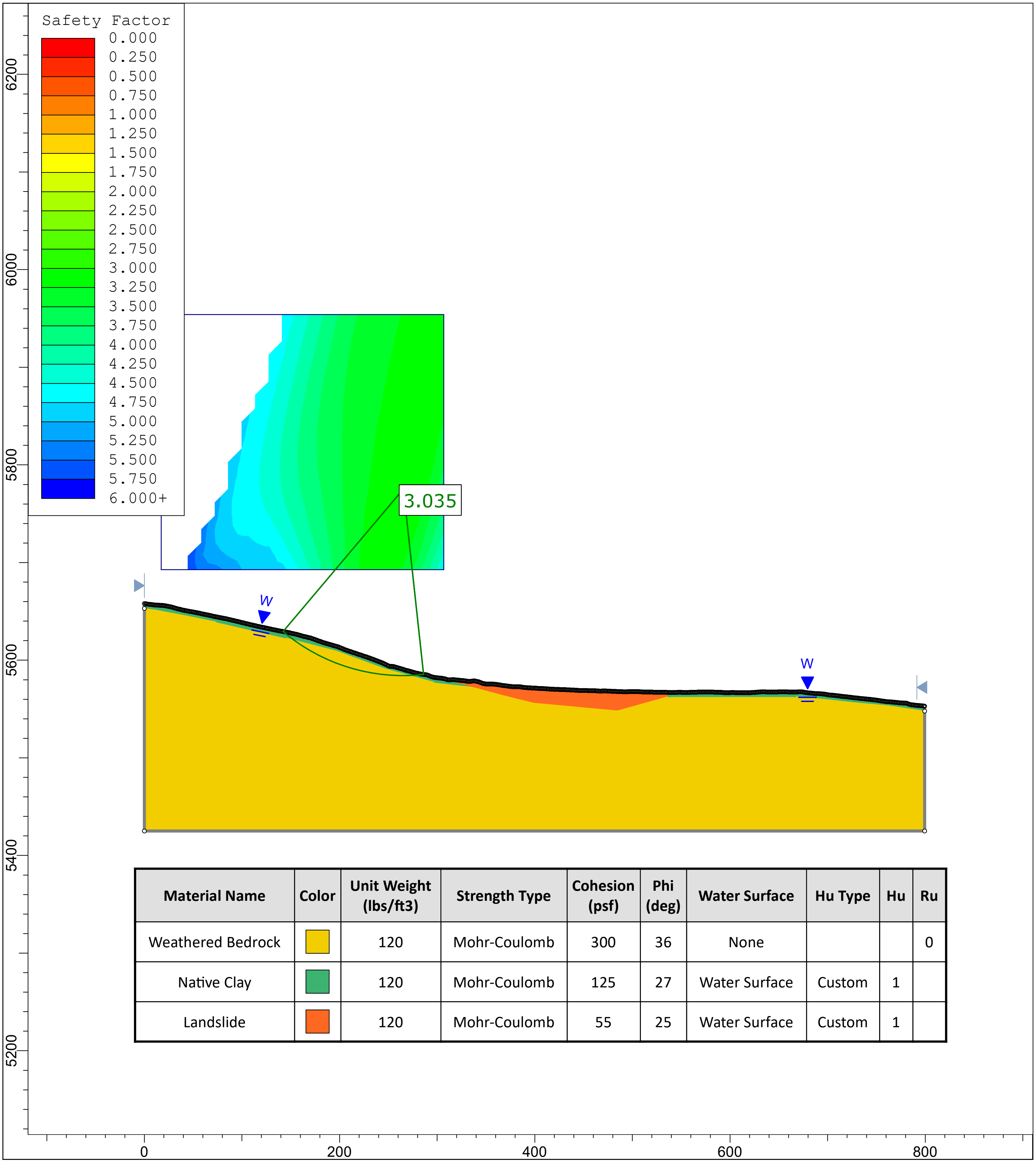

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Hu Type	Hu	Ru
Weathered Bedrock		120	Mohr-Coulomb	300	36	None			0
Native Clay		120	Mohr-Coulomb	125	27	Water Surface	Custom	1	
Landslide		120	Mohr-Coulomb	55	25	Water Surface	Custom	1	

Global Stability - Static

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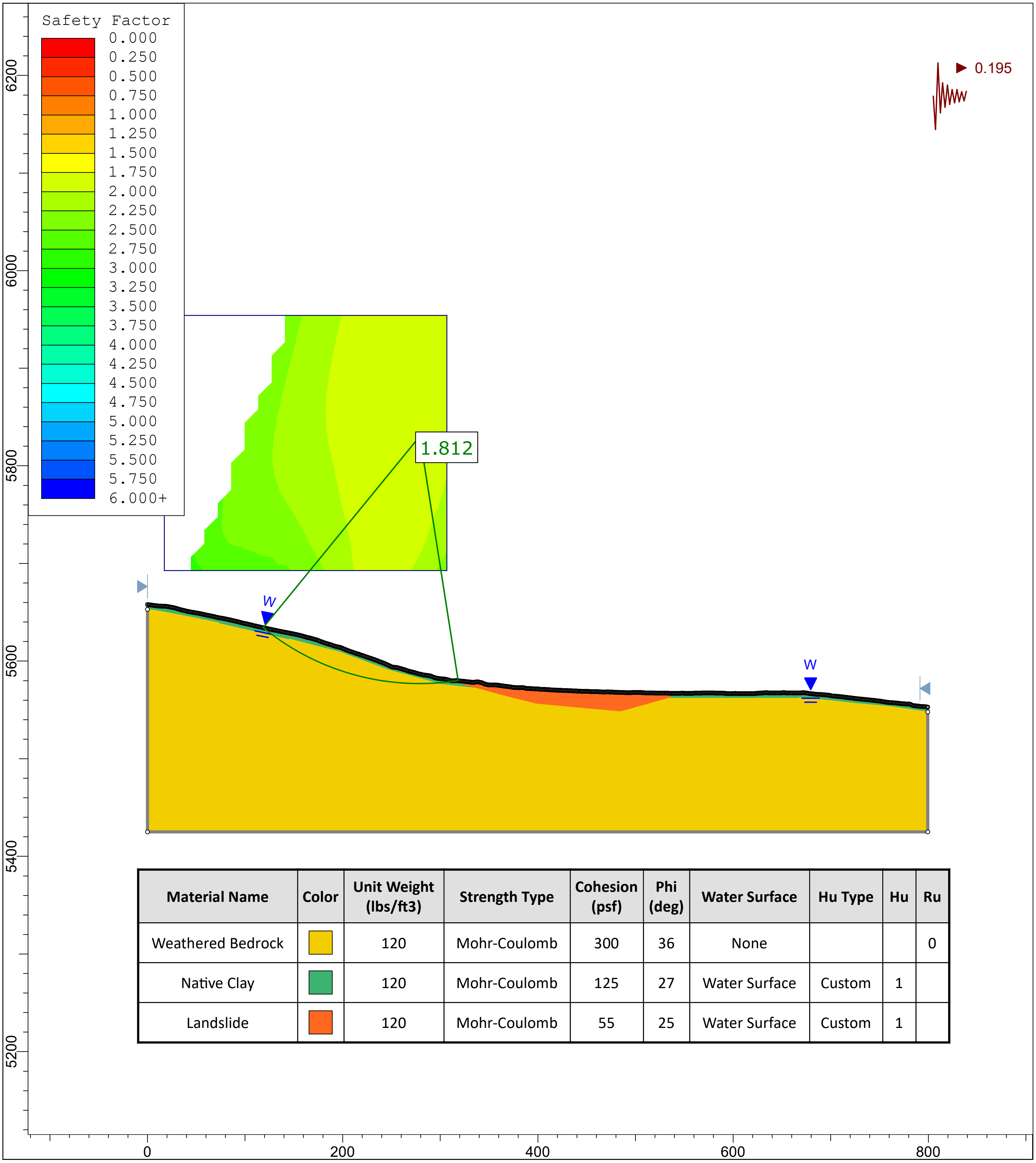

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Hu Type	Hu	Ru
Weathered Bedrock		120	Mohr-Coulomb	300	36	None			0
Native Clay		120	Mohr-Coulomb	125	27	Water Surface	Custom	1	
Landslide		120	Mohr-Coulomb	55	25	Water Surface	Custom	1	

Global Stability - Pseudo Static

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