Geotechnical Investigation
Crimson Ridge Water Reservoir
Eden, Utah

September 22, 2020

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Geotechnical Investigation
Crimson Ridge Water Reservoir
Approximately 1155 North Whispering Pines Lane
Eden, Utah
CG Project No.: 227-002

Prepared by:

9/22/20
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1.0 INTRODUCTION

1.1 PURPOSE AND SCOPE OF WORK

This report presents the results of a geotechnical investigation that was performed for the proposed Crimson Ridge Water Reservoir which is to be located at approximately 1155 North Whispering Pines Lane in Eden, 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 slabs-on-grade 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 for the reservoir by Western Geologic, dated July 17, 2020, was reviewed to assist in our assessments.

The work performed for this report was authorized by Mr. Steve Fenton and was conducted in accordance with the Christensen Geotechnical proposal dated March 31, 2020.

1.2 PROJECT DESCRIPTION

Based on information provided by Gardner Engineering, we understand that the proposed construction at the site is to consist of a concrete water reservoir on the order of 150,000 gallons in size. The reservoir is to be partially buried and 50 feet in diameter and 16 feet in height. The footing loads for the proposed structure are anticipated to be on the order of 3 to 8 klf for walls and up to 150 kips for columns. If the actual structural loads are different from those anticipated, Christensen Geotechnical should be notified in order to reevaluate our recommendations.

2.0 METHODS OF STUDY

2.1 FIELD INVESTIGATION

The subsurface conditions at the site were explored by excavating two test pits to depths of 8 and 10 feet below the existing site grade. The approximate test pit locations are shown on the Exploration Location Map, Plate 2. The subsurface conditions as encountered in the test pits were recorded at the time of excavation and are presented on the attached 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.

The test pit excavation was accomplished with a mini tracked excavator. Disturbed and undisturbed soil samples were collected from the test pit sidewalls at the time of the excavation. The disturbed samples were collected and placed in bags and buckets. The undisturbed samples consisted of block samples which were placed in bags. The samples were visually classified in the field and portions of each sample were packaged and transported to our laboratory for testing. The classifications for the individual soil 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 in order to evaluate the pertinent engineering properties. The laboratory testing that was performed included moisture content and density determinations, Atterberg limits evaluations, partial gradation analyses, a one-dimensional consolidation test, and a direct shear test. A summary of our laboratory testing is presented in the table below:

<u>Table No. 1: Laboratory Test Results</u>

Test Hole No.	Depth (ft.)	Dry Density (pcf)	Moisture Content (%)	Atterberg Limits		Grain Size Distribution (%)			Direct Shear		Soil Type
				LL	PI	Gravel (+#4)	Sand	Silt/Clay (- #200)	Friction Angle	Cohesion (psf)	
TP-1	7	97.4	19.8	32	9			64.6			CL
TP-2	3	97.8	19.8	34	20			87.8			CL

The results of our 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 plates, Plates 6 through 8.

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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 within the foothills of the mountain which is west of the existing Crimson Ridge Subdivision Phase 1. The property generally sloped down to the west at grades of 30 to 65 percent. The vegetation at the site generally consisted of dense, mature trees.

3.2 SUBSURFACE CONDITIONS

3.2.1 Soils

Based on the two test pits completed for this investigation, the site is covered with 1½ to 2 feet of topsoil. The subsurface materials below the topsoil generally consist of Claystone and Sandstone bedrock through the maximum depth explored. In Test Pit TP-1, a 4 ½ foot thick layer of Sandy Lean CLAY (CL) was observed below the topsoil and above the bedrock.

3.2.2 Groundwater

Groundwater was not encountered in our test pits at the time of excavation. It should be understood that groundwater 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 2018 International Building Code (IBC) for seismic design. The IBC seismic design is based on seismic hazard maps which depict 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 web-based application by the Applied Technology Council (ATC) 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 B, which represents a "rock" profile. The spectral acceleration values obtained from the ATC web-based application are shown below.

Table 2: IBC Seismic Response Spectrum Values

Site Location: 41.279390° N -111.833496° W	
Name	**Response Spectral Value**
$\mathbf{S_S}$	**0.957**
$\mathbf{S_1}$	**0.342**
$\mathbf{S_{MS}}$	**0.861**
$\mathbf{S_{M1}}$	**0.273**
$\mathbf{S_{DS}}$	**0.574**
$\mathbf{S_{D1}}$	**0.182**
PGA	**0.425**
$\mathbf{PGA_M}$	**0.382**

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.

A review of the “Liquefaction Special Study Areas, Wasatch Front and Nearby Areas, Utah,” map (Christenson and Shaw, 2008), indicates that the subject site is located in an area designated as having a very low potential for liquefaction. A very low potential for liquefaction indicates that there is a less than 5 percent probability of liquefaction at this site within a 100-year period. Due to the shallow bedrock at the site, we assess the liquefaction potential at this site 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 footing and reservoir pad 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. 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

Once exposed through excavation, all subgrade soils should be proof rolled with a relatively large, wheeled vehicle to a firm, unyielding condition. Any localized soft soils that are encountered during the proof rolling operation should be removed and replaced with granular structural fill. If soft areas extend 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, but would likely consist of over-excavating the area by at least 18 inches and then placing a geofabric (such as Mirafi RS280i) at the bottom of the excavation. Over this, a stabilizing fill, consisting of angular coarse gravel with cobbles, would be 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 that extend 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 that extend to more than 5 feet in depth should be sloped or shored in accordance with OSHA regulations for a type C soil. The stability of construction excavations is the contractor's responsibility. If the stability of an excavation becomes questionable, the excavation should be evaluated immediately by qualified personnel.

5.2.4 Structural Fill and Compaction

All fill placed for the support of the structures and concrete flatwork should consist of structural fill. Due to their swell potential, we do not recommend that the native clay soils or the claystone bedrock be used as structural fill. 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. Additionally, 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 structural fill should be placed in loose lifts that are a maximum of 8 inches thick. The moisture content should be within 3 percent of optimum and the fill should be 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 earlier, bedrock was encountered within both of our test pits. This bedrock was generally in a weak condition, but is likely to become more competent with depth. We anticipate that the minimum equipment required for excavations within the bedrock would be the use of a heavy excavator with a ripper tooth or the use of a hoe-ram. Prior to bidding, the contractor should make themselves aware of the subsurface conditions and the type of equipment that is best suited for these conditions.

5.2.6 Permanent Cut and Fill Slopes

The existing slopes at the subject site should not be over-steepened by cutting or filling. We recommend that all non-retained cut and fill slopes be graded no steeper than a 3 to 1 (horizontal to vertical) grade. If steeper grades are required, retaining structures should be used.

5.3 FOUNDATIONS

Foundations for the planned reservoir may consist of conventional continuous and/or spread footings established on undisturbed native soil or on properly placed and compacted structural fill extending down to undisturbed native soil. Footings for the proposed reservoir 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 the 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 will depend 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	0.36	42
Active Seismic	0.15	18
At-Rest	0.53	61
Passive Static	2.77	319
Passive Seismic	-0.33	-37

We recommend that walls which are allowed little or no wall movement be designed using "at rest" conditions. Walls that are 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 that the geotechnical engineer be consulted to provide more appropriate 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 that an ultimate coefficient of friction of 0.35 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

Concrete slabs-on-grade should 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.

5.7 MOISTURE PROTECTION AND SURFACE DRAINAGE

5.7.1 Surface Drainage

Any wetting of the foundation soils will likely cause some degree of volume change within the soils and should be prevented both during and after construction. We recommend that grading be performed to prevent both ponding and the infiltration of surface water near the proposed reservoir. If necessary, diversion berms or ditches should be placed uphill of the reservoir to redirect runoff. In addition, we recommend adequate compaction of backfill around the reservoir walls. At a minimum, we recommend that the backfill around the tanks walls be compacted to at least 90 percent of the maximum density as determined by ASTM D 1557.

5.7.2 Reservoir Under-Drainage

Consideration should be given to constructing a drainage system below the reservoir. The drainage system should consist of an impermeable membrane over which at least 6 inches of free-draining gravel is placed. Perforated collection pipes should be installed within the free-draining gravel. The perforated collection pipe and the impermeable membrane should be graded to a free gravity outfall to assist in leak detection and to allow the discharge of any collected water.

5.8 SUBSURFACE DRAINAGE

Due to the alpine nature of the subject site, we recommend that all subgrade 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 and encased in at least 12 inches of free-draining gravel. The gravel should extend 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 pipe should be graded to drain to a storm drain or other free-gravity outfall unless provisions for pumped sumps are made. The gravel which extends up the wall may be replaced by a fabricated drain panel such as Mirafi G200N or equivalent.

5.9 SLOPE STABILITY

As recommended in the Western Geologic hazards evaluation (Black, 2020), a slope stability assessment was performed using the Slide computer program and the modified Bishop's method of slices. The profile assessed was based on Figure 5 of the Western Geologic report, a site plan for the project by Gardner Engineering, our observations at the site, and on the subsurface conditions that were exposed in our test pits. The location of the profile is shown on Plate 2. The strength value that was used for the claystone bedrock in our analyses was based on a direct shear test. This test indicated a strength consisting of an angle of internal friction of 32 degrees and a cohesion of 150 psf. The strength of the Maple Canyon Formation bedrock was assumed to consist of cohesion of 10,000 psf. The Holocene and late Pleistocene alluvium deposits at the site generally consisted of sand and gravel soils which were assumed to have a strength consisting of an angle of internal friction of 35 degrees and an apparent cohesion of 21 psf.

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 0.382g. 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 these safety factors were met and the site is suitable for the proposed construction. As a precaution, we recommend that footings be placed at least 15 feet laterally from the face of slopes on the property.

The slope stability analyses presented above are based on a site plan by Gardner Engineering and the subsurface investigation and laboratory testing completed for this report. If the proposed grades change significantly from that presented on the Gardner Engineering site plan, Christensen Geotechnical should be consulted and additional analyses may be required. The results of our slopes stability assessments may be found on Plates 9 and 10.

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

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.

Black, Bill, July 17, 2020, "Geologic Hazards Evaluation, Proposed Crimson Ridge Water Reservoir, About 1155 North Whispering Pines Lane, Eden, Utah," Western Geologic, consultant's unpublished report.

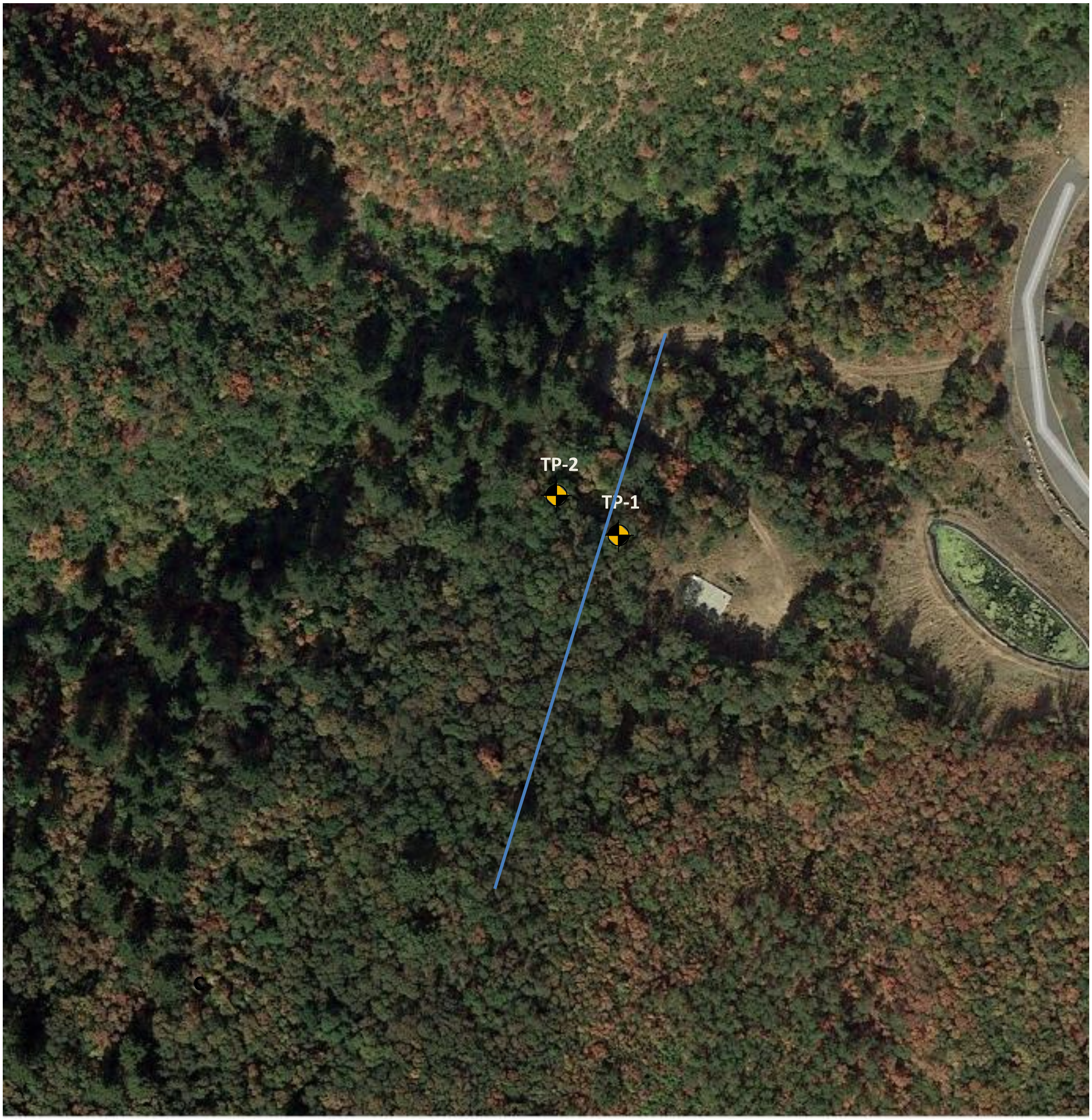
Base Photo: Utah AGRC

Drawing Not to Scale

Approximate Project Boundary

N

Christensen Geotechnical	B & H Investment Properties, LLC Crimson Ridge Water Reservoir Eden, Utah Project No. 227-002 **Vicinity Map**	Plate 1



Base Photo: Utah AGRC

Approximate Test Pit Location

Slope Stability Profile

Drawing Not to Scale

N

Christensen Geotechnical	B & H Investment Properties, LLC Crimson Ridge Water Reservoir Eden, Utah Project No. 227-002 **Exploration Location Map**	Plate 2

Date		
Started:	4/16/2020	
Completed:	4/16/2020	
Backfilled:	4/20/2020	

TEST PIT LOG

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

Test Pit No.

TP-1

Depth (feet)	Sample Type	Groundwater	Graphic Log	Group Symbol	Material Description	Dry Density (pcf)	Moisture Content (%)	Minus #200 (%)	Liquid Limit	Plasticity Index
					Topsoil; Lean CLAY - moist, dark brown					
	Undisturbed			CL	Sandy Lean CLAY - stiff, moist, brown					
5										
	Undisturbed				Claystone Bedrock - weathered, weak, brown	97.4	19.8	64.6	32	9
					Bottom of test pit at 8 feet					
10										
15										

☒ Bulk/Bag Sample
▥ Undisturbed Sample

Stablllized Groundwater
Groundwater At Time of Excavation

Christensen Geotechnical

B & H Investment Properties, LLC
Crimson Ridge Water Reservoir
Eden, Utah
Project No.: 227-002

Plate

3

Date		
Started:	4/16/2020	
Completed:	4/16/2020	
Backfilled:	4/20/2020	

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	Plasticity Index
0–2					Topsoil; Lean CLAY - moist, dark brown					
2–9				CL	Claystone Bedrock - weathered, weak, brown					
3	Undisturbed					97.8	19.8	87.8	34	20
8	Undisturbed									
9–10					Sandstone Bedrock - fractured, weathered, weak					
10					Bottom of test pit at 10 feet					

Depth scale: 5, 10, 15

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

Christensen Geotechnical

B & H Investment Properties, LLC
Crimson Ridge Water Reservoir
Eden, Utah
Project No.: 227-002

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

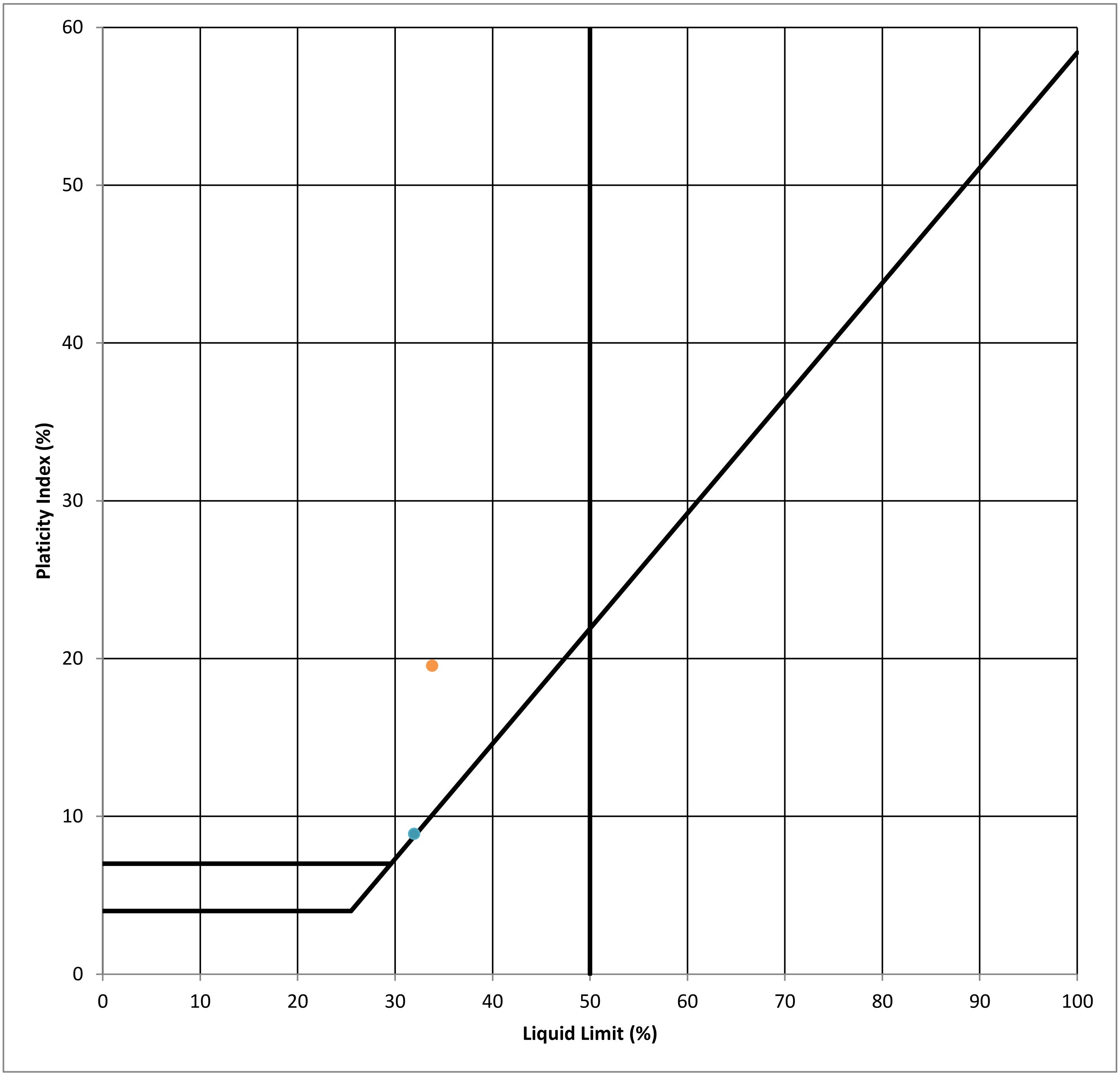
Seam	1/16 to 1/2 inch
Layer	1/2 to 12 inch

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

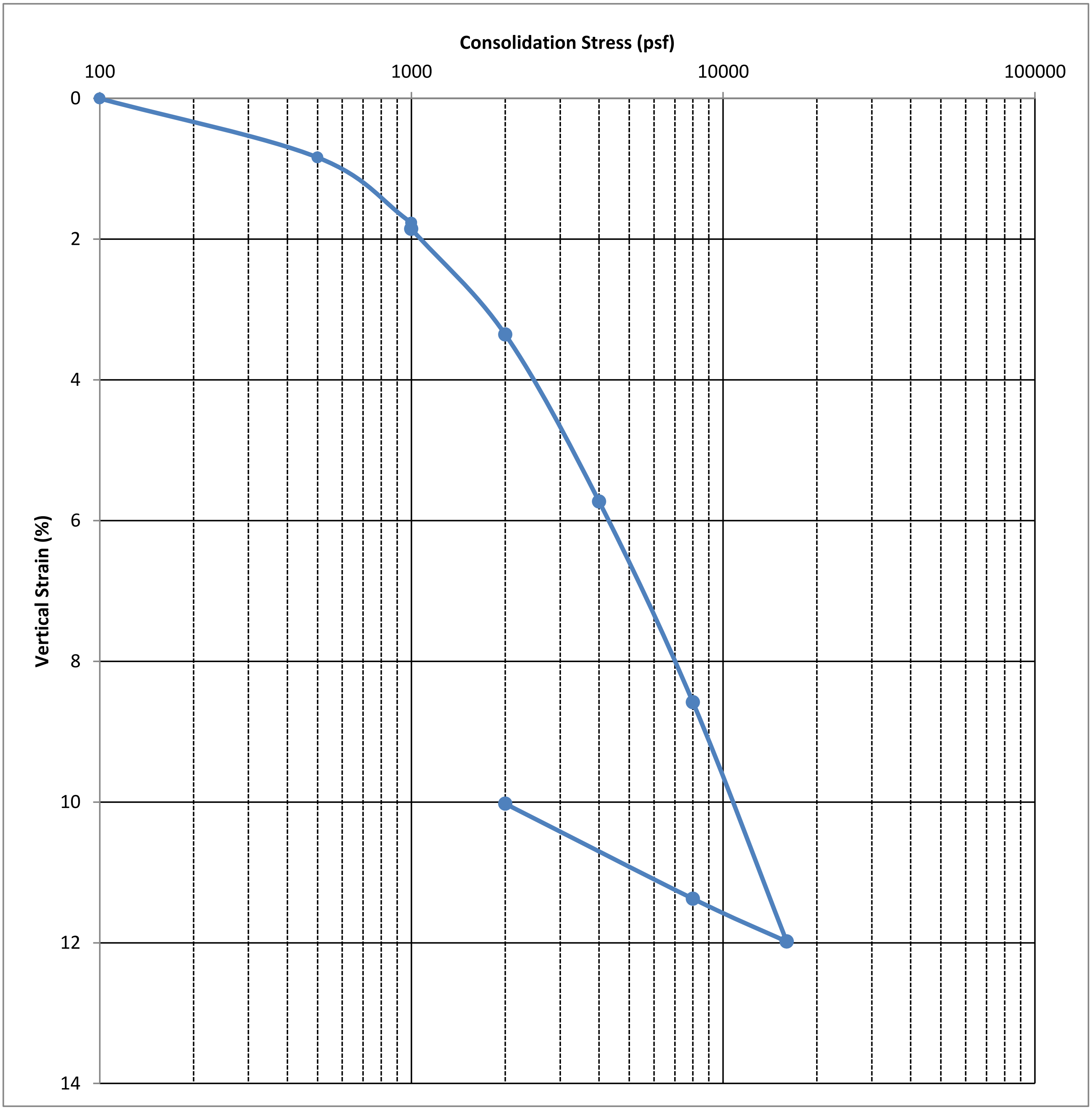
Atterberg Limits



Location	Depth (ft)		Classification	Liquid Limit	PI
TP-1	7	●	Sandy Lean CLAY	32	9
TP-2	3	●	Lean CLAY	34	20

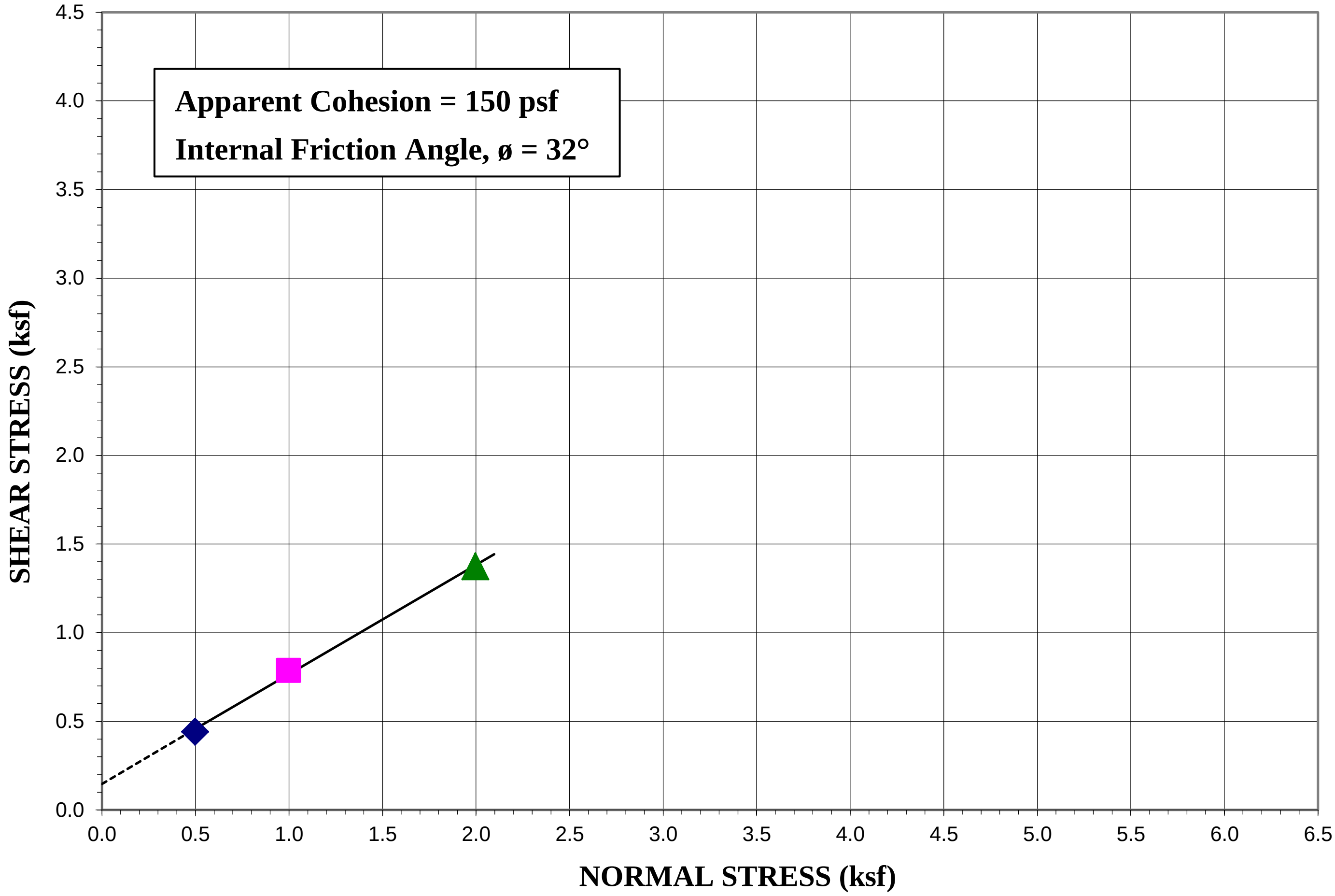
Christensen Geotechnical	B & H Investment Properties, LLC Crimson Ridge Water Reservoir Eden, Utah Project No.: 227-002	Plate 6

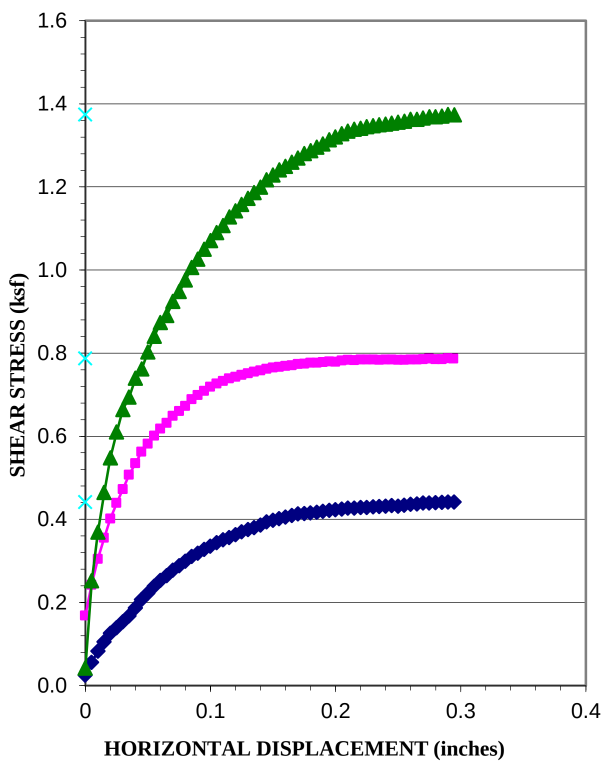
1-D Consolidation



Location	Depth (ft)	Dry Density (pcf)	Moisture Content (%)	σ_o (psf)	σ_p (psf)	C_c	C_r	OCR
TP-1	7	97.4	19.8	800	2,000	0.104	0.022	2.5

Christensen Geotechnical	B & H Investment Properties, LLC Crimson Ridge Water Reservoir Eden, Utah Project No.: 227-002	Plate 7

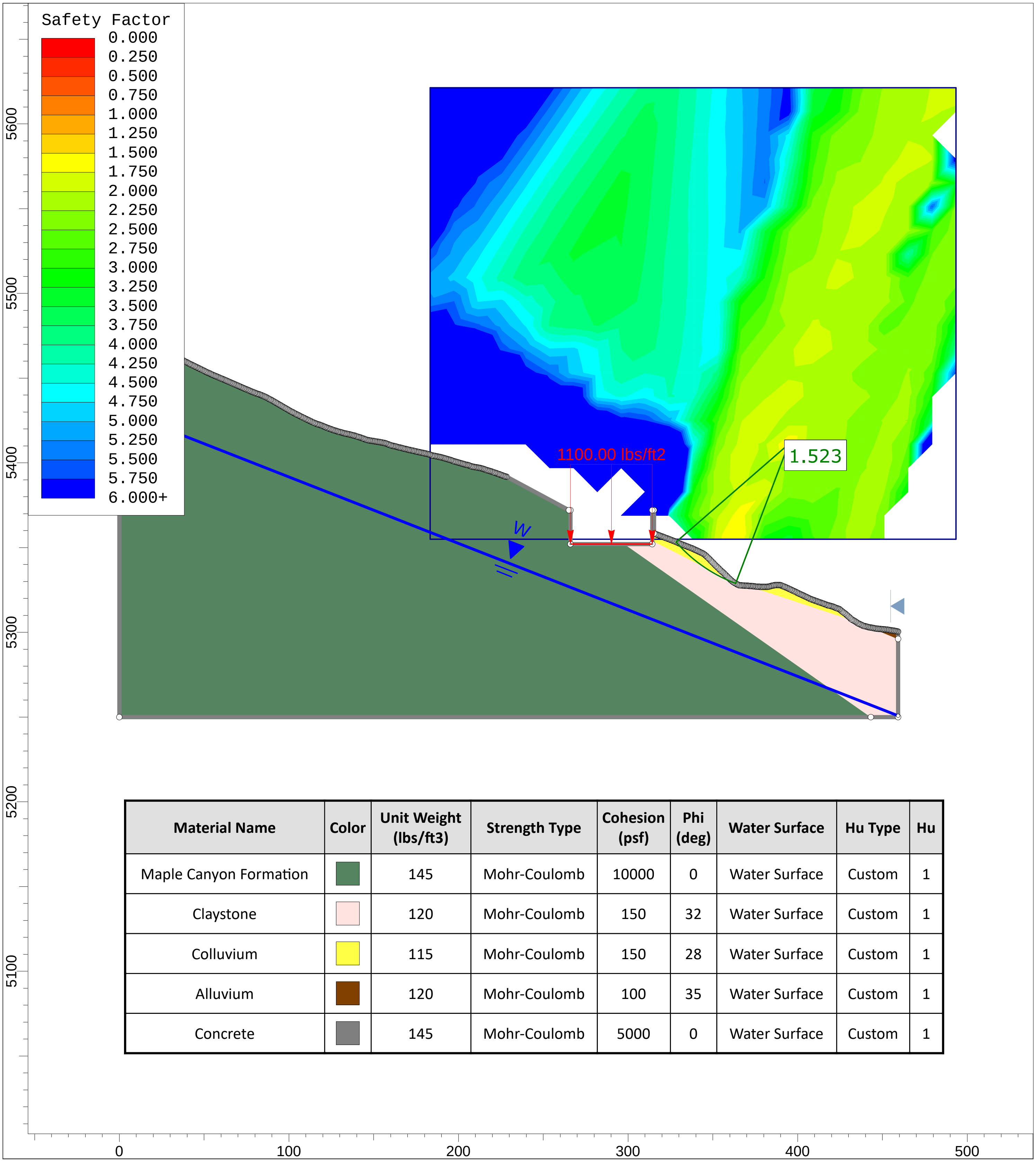




Location: TP-2	**Depth:** 3.0 ft		
Type of Test:	Consolidated Drained/Saturated		
Test No. (Symbol)	**1 (◆)**	**2 (■)**	**3 (▲)**
Sample Type:	Undisturbed		
Initial Height, in.	1	1	1
Diameter, in.	2.4	2.4	2.4
Dry Density Before, pcf	94.3	98.1	97.8
Moisture % Before	19.8	19.8	19.8
Normal Load, ksf	0.5	1.0	2.0
Shear Stress, ksf	0.44	0.79	1.37
Strain Rate	0.0025		

Sample Properties	
Cohesion, psf	150
Friction Angle, ϕ	32
Liquid Limit, %	34
Plasticity Index, %	20
Percent Gravel	---
Percent Sand	---
Percent Passing No. 200 sieve	87.8
Classification	Lean CLAY (CL)

Christensen Geotechnical	**B & H Investment Properties** **Crimson Ridge Water Reservoir** **Eden, Utah** **Project No.: 227-002**	**Plate** **8**



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Hu Type	Hu
Maple Canyon Formation		145	Mohr-Coulomb	10000	0	Water Surface	Custom	1
Claystone		120	Mohr-Coulomb	150	32	Water Surface	Custom	1
Colluvium		115	Mohr-Coulomb	150	28	Water Surface	Custom	1
Alluvium		120	Mohr-Coulomb	100	35	Water Surface	Custom	1
Concrete		145	Mohr-Coulomb	5000	0	Water Surface	Custom	1

Global Stability - Static

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Crimson Ridge Water Reservoir
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227-002

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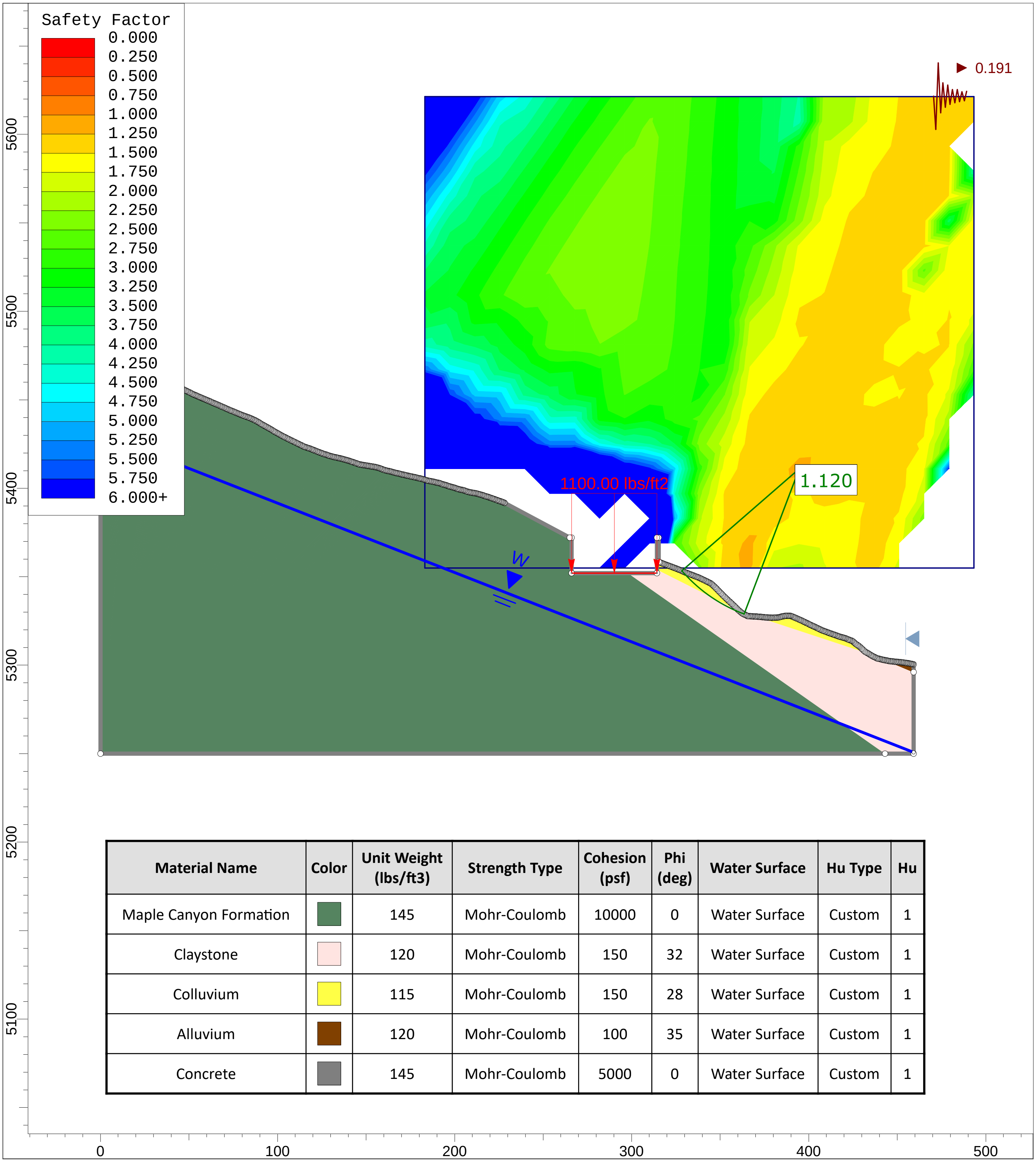

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Hu Type	Hu
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Alluvium		120	Mohr-Coulomb	100	35	Water Surface	Custom	1
Concrete		145	Mohr-Coulomb	5000	0	Water Surface	Custom	1

Global Stability - Pseudo Static

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Plate

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