Geotechnical Investigation
Legends at Hawkins Creek Lot 2
Weber County, Utah

November 10, 2020

Prepared by:

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Geotechnical Investigation
Legends at Hawkins Creek Lot 2
6682 East Chaparral Road
Weber County, Utah
CG Project No.: 259-001

Prepared by:

11/10/20
LICENSED PROFESSIONAL ENGINEER
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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 that was performed for Legends at Hawkins Creek Lot 2 which is located at 6682 East Chaparral Road 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 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, we reviewed the October 23, 2020 Geologic Hazards Evaluation by Western Geologic to assist in our assessments.

The work performed for this report was authorized by Mr. Joe Sadler and was conducted in accordance with the Christensen Geotechnical proposal dated August 19, 2020.

1.2 PROJECT DESCRIPTION

Based on conversations with our client, we understand that the proposed construction at the site is to consist of a single-family residence. The proposed structure is to have a footprint on the order of 3,000 square feet and is to be one to two stories in height with a basement. Up to 15 feet of structural fill is to be placed on the lot to facilitate the construction of the residence. The footing 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 the 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, one to 7½ feet and one to 9 feet, below the existing site grade. Each test pit was terminated due to trackhoe refusal on bedrock. 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 tracked excavator. Undisturbed soil samples were collected from the test pit sidewalls at the time of excavation. These 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 included a Schmitt Hammer test that was performed on a block of the bedrock which had been collected from the test pits. The results of this test indicated a compressive strength of 260,000 psf.

The 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 an undeveloped lot in an existing subdivision. The lot generally sloped down to the north with a grade of approximately 35 percent. The vegetation at the site generally consisted of common grasses and weeds with a few bushes and brush. The site was bordered by Chaparral Road to the south and undeveloped land on all other sides.

3.2 SUBSURFACE CONDITIONS

3.2.1 Soils

Based on the two test pits that were completed for this investigation, the site is covered with 1½ to 3 feet of topsoil. The subsurface materials below the topsoil consisted of sandstone bedrock which extended through the maximum depth explored. Each of our test pits was terminated due to trackhoe refusal on the bedrock.

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 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), the project site's approximate latitude and longitude, and its 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's web-based application are shown below.

Table 2: IBC Seismic Response Spectrum Values

Site Location: 41.240797º N -111.788767º W	
Name	**Response Spectral Value**
S_S	**0.835**
S_1	**0.293**
S_{MS}	**0.751**
S_{M1}	**0.234**
S_{DS}	**0.501**
S_{D1}	**0.156**
PGA	**0.368**
PGA_M	**0.331**

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.

Due to the shallow bedrock encountered within our test pits, 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, and all other soils should be stripped (removed) from the building pad, flatwork concrete areas, and any other areas where structural fill will be placed in order to exposed the underlying bedrock. Following the stripping operations, the exposed bedrock should be excavated into horizontal terraces. The excavation of terraces provides a non-uniform plain below the proposed construction which will key the overlying fill and structure into the bedrock, providing greatly increased resistance to slope failures. The vertical distance in between the terraces should be 3 to 5 feet in height. Once the bedrock has been terraced, structural fill may be placed to bring the site to design grade. A Christensen Geotechnical representative should observe the site grading operations.

5.2.2 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 into structural fill of native soils 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.

5.2.3 Structural Fill and Compaction

All fill that is placed for the support of structures and concrete flatwork should consist of structural fill. The sandstone bedrock may be used as structural fill below any exterior flatwork concrete and pavements if it is crushed to a maximum particle size of 4 inches. All structural fill placed below the proposed residence should consist of an imported material. The imported structural fill 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 with 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 should be free of topsoil, vegetation, frozen material, particles larger than 4 inches in diameter, and any other deleterious materials. All imported materials should be approved by the geotechnical engineer prior to importing.

The 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 the fill heights exceed 5 feet, the level of compaction should be increased to 98 percent.

5.2.4 Excavatability

As indicated earlier, bedrock was encountered within each of our test pits. The trackhoe experienced practical equipment refusal at 7½ and 9 feet below grade. The bedrock was in a moderately strong condition. 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. Of note, prior to bidding, this report should be provided to all contractors in order for them to be informed of the subsurface conditions and make their own assessment as to the type of equipment best suited for these conditions.

5.3 FOUNDATIONS

The foundations for the planned structure may consist of conventional continuous and/or spread footings established entirely on bedrock or entirely on at least 12 inches of properly placed and compacted structural fill. The footings for the proposed structure should be a minimum of 20 inches and 30 inches wide for continuous and spot footings, respectively. The exterior footings should be established at a minimum of 36 inches below the lowest adjacent grade to provide frost protection and confinement. Interior footings that are not subject to frost should be embedded a minimum of 18 inches for confinement.

Continuous and spread footings that are established on bedrock or structural fill may be proportioned for a maximum net allowable bearing capacity of 3,000 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 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 the 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	0.27	33
Active Seismic	0.11	14
At-Rest	0.43	51
Passive Static	3.69	443
Passive Seismic	-0.31	-38

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 that are presented above assume a 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 can be represented 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 bedrock or structural fill, we recommend that an ultimate coefficient of friction of 0.45 be used. If passive resistance is used in conjunction with frictional resistance, the passive resistance should be reduced by ½. The 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

Any 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 downspouts that are 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 high alpine setting of the subject site, we recommend that all basement and retaining walls incorporate a foundation drain. The foundations 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 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 pipe should be graded to drain to the land drains, a storm drain or another free-gravity outfall unless provisions for pumped sumps are made. The gravel which is to extend up the foundation 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, our understanding of the proposed development of the site, and on the subsurface conditions that were exposed in our test pits. The location of the profile is shown on Plate 2. A Schmitt hammer test was performed on a block sample of the sandstone bedrock that had been collected from the site; this test indicated a compressive strength of 260,000 psf. For our analyses, we reduced this value to 26,000 psf (cohesion value of 13,000 psf). The near-surface Sandy Lean CLAY (CL) was assumed to have a strength consisting of an angle of internal friction of 28 degrees and a cohesion of 100 psf. All structural fill that is to be placed below the house should consist of a Silty GRAVEL with sand (GM) which has a minimum strength consisting of an angle of internal friction of 35 degrees and a cohesion of 50 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 0.331g. 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 profile has safety factors greater than 1.5 and 1.0 for the static and pseudo static conditions and is therefore considered suitable for the planned construction.

As indicated in Section 5.2.1, following the stripping operations, it is important that the exposed bedrock be excavated into horizontal terraces with each terrace being 3 to 5 feet in height. This will provide a non-uniform plain below the proposed construction which will key the overlying fill and structure into the bedrock, providing greatly increased resistance to slope failures.

The slope stability analysis presented above is based on our understanding of the proposed construction. Significant changes to the site grade, such as the steepening of slopes by way of cuts or fills, may adversely affect the stability of the slopes at the site and increase the risk of slope failures. If significant cuts of more than 15 feet of fill are planned to be placed on the lot, 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 23, 2020, "Geologic Hazards Evaluation, Legends at Hawkins Creek Lot 2, 6682 East Chaparral Road, Huntsville, Weber County, Utah," Western Geologic, consultant's unpublished report.

Base Photo: Utah AGRC

Drawing Not to Scale

Approximate Project Boundary

N

Christensen Geotechnical	Habitations Residential Design Group Legends at Hawkins Creek Lot 2 Weber County, Utah Project No. 259-001 **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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Exploration Location Map

Plate 2

TEST PIT LOG

Date		
Started:	10/8/2020	
Completed:	10/8/2020	
Backfilled:	10/8/2020	

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	Plasticity Index
0–3					Topsoil; Sandy Lean CLAY - slightly moist, dark brown					
3–9					Sandstone Bedrock - weak, weathered, light gray					
5					- moderately strong below 5 feet					
8	⊠									
9					Refusal at 9 feet					
10										
15										

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

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Plate

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Date			TEST PIT LOG	Logged By: M Christensen Equipment: Trackhoe Location: See Plate 2	Test Pit No. TP-2
	Started:	10/8/2020			
	Completed:	10/8/2020			
	Backfilled:	10/8/2020			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
					Topsoil; Sandy Lean CLAY - slightly moist, dark brown					
					Sandstone Bedrock - weak, weathered, light gray					
					- moderately strong below 3 feet					
5										
	⊠									
					Refusal at 7½ feet					
10										
15										

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

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Plate
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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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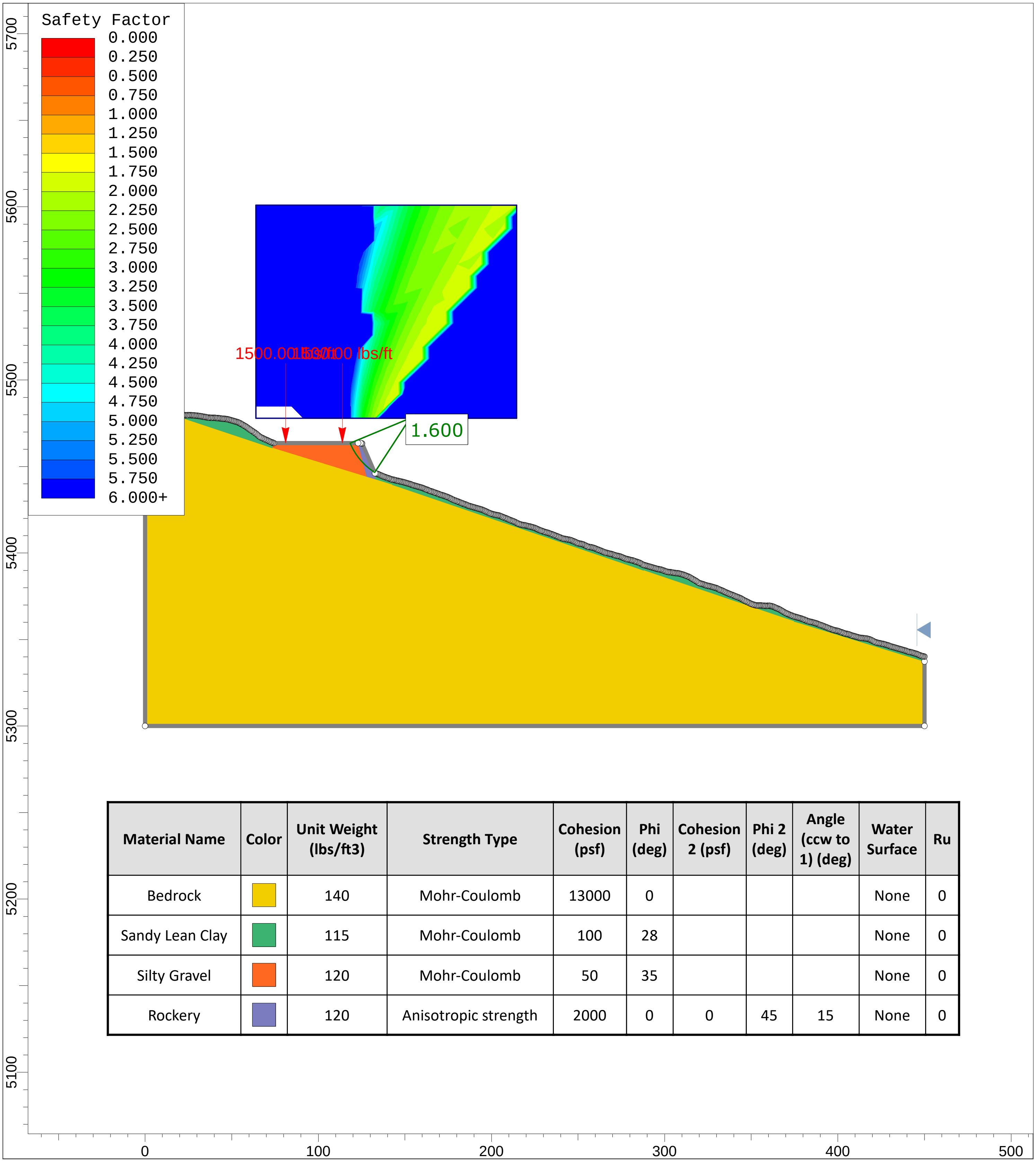
Soil Terms Key

Plate 5



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Cohesion 2 (psf)	Phi 2 (deg)	Angle (ccw to 1) (deg)	Water Surface	Ru
Bedrock		140	Mohr-Coulomb	13000	0				None	0
Sandy Lean Clay		115	Mohr-Coulomb	100	28				None	0
Silty Gravel		120	Mohr-Coulomb	50	35				None	0
Rockery		120	Anisotropic strength	2000	0	0	45	15	None	0

Global Stability - Static

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Plate

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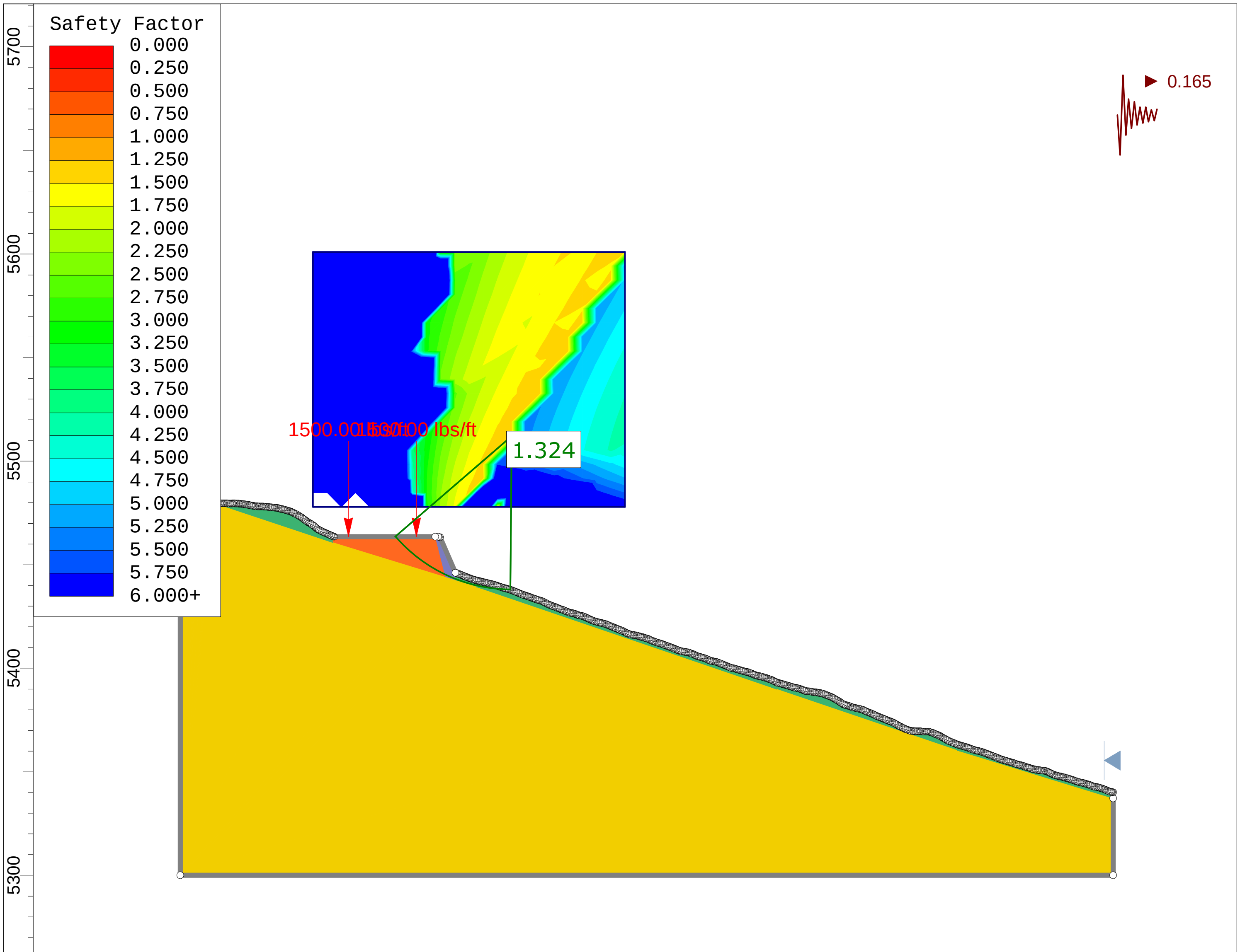

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Cohesion 2 (psf)	Phi 2 (deg)	Angle (ccw to 1) (deg)	Water Surface	Ru
Bedrock		140	Mohr-Coulomb	13000	0				None	0
Sandy Lean Clay		115	Mohr-Coulomb	100	28				None	0
Silty Gravel		120	Mohr-Coulomb	50	35				None	0
Rockery		120	Anisotropic strength	2000	0	0	45	15	None	0

Global Stability - Pseudo Static

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