# Geotechnical Investigation
# Proposed Water Reservoir
# Legacy Mountain Development
# Huntsville, Weber County, Utah

**January 15, 2021**

**Prepared by:**

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# Geotechnical Investigation
# Proposed Water Reservoir
# Legacy Mountain Development
# Approximately 5586 Old Snow Basin Road
# Huntsville, Weber County, Utah
**CG Project No.: 133-011**

Prepared by:

1/15/21
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 a proposed water reservoir which is to be constructed for the Legacy Mountain Development and will be located at approximately 5586 Old Snow Basin Road in Huntsville, 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 concrete slabs and foundations. This investigation included subsurface exploration, representative soil sampling, field and laboratory testing, engineering analysis, and preparation of this report. The work performed for this report was authorized by Mr. John Lewis.

## 1.2 PROJECT DESCRIPTION

Based on conversations with our client, we understand that the proposed construction at the site is to consist of a concrete water reservoir. The reservoir is to be approximately 80 feet in diameter, 8 feet in height, and to be buried below grade. The footing loads for the proposed reservoir are anticipated to be on the order of 3 to 6 klf for walls and up to 200 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 one test pit at the site of the water reservoir. Two additional test pits that were excavated for the Legacy Mountain development were also used in our slope stability assessment of the reservoir site (Christensen, 2021). These test pits were excavated to depths of 7 to 9 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 to 5. A key to the symbols and terms used on the Test Pit Logs may be found on Plate 6.

The 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. 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.

During the logging of test pit TP-26, a Schmitt rebound hammer was used to estimate the compressive strength of the bedrock that was exposed. The results of the rebound hammer testing indicated a compressive strength of 100,000 psf at a depth of 6 feet.

## 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 performed included moisture content determinations, Atterberg limits evaluations, partial gradation analyses, and a direct shear test. A summary of our laboratory testing is presented in the table below:

## Table No. 1: Laboratory Test Results

| Test Pit No. | Depth (ft.) | Dry Density (pcf) | Moisture Content (%) | Atterberg Limits |  | Silt/Clay (- #200) | Direct Shear |  | Soil Type |
|---|---|---|---|---|---|---|---|---|---|
|  |  |  |  | LL | PI |  | Friction Angle | Cohesion (psf) |  |
| TP-18 | 8 |  | 19.7 | 55 | 37 | 79.6 | 26 | 120 | CH |
| TP-19 | 3 |  | 13.1 | 57 | 40 | 51.3 |  |  | CH |

The results of our laboratory tests are also presented on the Test Pit Logs, Plates 3 through 5, and more detailed laboratory results are presented on the laboratory testing plates, Plates 7 and 8.

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 undeveloped land located on a ridge that sloped down to the north, east, and south with grades of 10 to 60 percent. At the time of our investigation, all vegetation had been removed and the site had been graded to create a nearly level pad. The vegetation in the vicinity of the pad generally consisted of common grasses, weeds, and oak brush.

## 3.2 SUBSURFACE CONDITIONS

### 3.2.1 Soils

Based on the test pit excavated at the site of the water reservoir, the subsurface materials consisted of conglomerate to sandstone bedrock. The bedrock was completely weathered and weak, with the bedrock strength increasing with depth.

### 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 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 C, which represents a "very dense soil and soft 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: $41.240857^{0}$ N $-111.815606^{0}$ W** | |
|---|---|
| **Name** | **Response Spectral Value** |
| **$S_S$** | **0.892** |
| **$S_1$** | **0.316** |
| **$S_{MS}$** | **01.07** |
| **$S_{M1}$** | **0.474** |
| **$S_{DS}$** | **0.713** |
| **$S_{D1}$** | **0.316** |
| **PGA** | **0.395** |
| **$PGA_M$** | **0.474** |

## 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 at this site, we 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 the site grading operations, all vegetation, topsoil, undocumented fill soils, and loose or disturbed soils should be stripped (removed) from the building 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 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 A 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 reservoir and concrete flatwork should consist of structural fill. The structural fill may consist of the native bedrock if it is crushed to a maximum particle size of 4 inches. 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 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, 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 the fill heights exceed 5 feet, the level of compaction should be increased to 98 percent.

#### 5.2.4 Excavatability

As indicated earlier, all subsurface material exposed in our test pit at the site consisted of conglomerate to sandstone bedrock. This bedrock was generally in a weak condition, but increased in strength with depth. We anticipate that the minimum equipment required for excavations within the bedrock will be the use of a heavy excavator with a ripper tooth or a hoe-ram. Prior to bidding, the contractor should be provided with this report in order to be made aware of the subsurface conditions so that they can assess the type of equipment that will be best suited for these conditions.

#### 5.2.5 Permanent Cut and Fill Slopes

The existing slopes on the property 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, additional slope stability assessments may be required.

### 5.3 FOUNDATIONS

The foundations for the planned reservoir may consist of conventional continuous and/or spread footings established either on undisturbed bedrock or on properly placed and compacted structural fill which extends down to undisturbed bedrock. 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 30 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 undisturbed 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 | 0.36 | 42 |
| Active Seismic | 0.14 | 17 |
| At-Rest | 0.53 | 61 |
| Passive Static | 2.77 | 319 |
| Passive Seismic | -0.31 | -35 |

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 native bedrock, 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 12 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

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 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 tank’s 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, such as an HDPE liner, over which at least 6 inches of free-draining gravel should be placed. Perforated collection pipes should be installed within the free-draining gravel, and the perforated pipe and the impermeable membrane should be graded to facilitate drainage to a low point in order to assist in leak detection and to allow the discharge of collected water.

## 5.8 SUBSURFACE DRAINAGE

Due to the high alpine setting 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 free-gravity outfall. 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 Geologic Hazards Evaluation by Western Geologic, the stability of the slope at the site was assessed using the Slide computer program and the modified Bishop's method of slices. The location of the profile is shown on Plate 2 and is based on the cross section presented in the Western Geologic report. For our analyses, we assumed that the top 5 feet of the bedrock was highly weathered. The strength of this highly weathered portion of the bedrock that was used in our analyses was based on a direct shear test performed for the Legacy Mountain development. This test indicated a strength consisting of an angle of internal friction of 25 degrees and a cohesion of 140 psf (Christensen, 2021). The strength of the remaining, less weathered bedrock was based on Schmitt rebound hammer testing that was performed in our test pit at the time of excavation. As indicated in Section 2.1, the result of this test indicated a compressive strength value of 100,000 psf. For our analyses, we reduced these strength value to 20,000 psf (cohesion value of 10,000 psf). The strength value used for the landslide deposits was based on the Stark method (Stark et al., 2005) which indicated a residual strength of 17 degrees.

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 0.4.74g. 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 slope in the area of the proposed water reservoir has safety factors greater than 1.5 and 1.0 for the static and pseudo static conditions. It is therefore our opinion that the proposed site is suitable for construction of the reservoir. It should be noted, however, that the landslide deposits below the reservoir site have a static factor of safety less than 1.0 and therefore have a high risk of slope movement unless mitigation measures are taken. These mitigation measures are presented in the aforementioned Legacy Mountain Development report. The results of our slope stability assessments may be found on Plates 9 through 11.

The slope stability analysis presented above is 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 and increase the risk of slope failures. If cuts or fills over 15 feet are planned, 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.

The recommendations presented within this report are based on the assumption that an adequate program of tests and observations will be followed during construction to verify compliance with our recommendations. We also assume that we will review the project plans and specifications to verify that our conclusions and recommendations are incorporated and remain appropriate (based on the actual design).

# 7.0 REFERENCES

Black, Bill, December 11, 2020, “Geologic Hazards Evaluation, Proposed Water Tank, Legacy Mountain Development, Sections 14 and 23, Township 6 North, Range 1 East, Huntsville, Weber County, Utah,” Western Geologic, consultant’s unpublished report.

Christensen, Mark, January 8, 2021, “Geotechnical Investigation, Legacy Mountain Development, Huntsville, Weber County, Utah,” Christensen Geotechnical, consultant’s unpublished report.

Stark, Timothy D., Choi, Hangseok, and McCone, Sean, 2005, “Drained Shear Strength Parameters of Analysis of Landslides,” ASCE, Journal of Geotechnical and Environmental Engineering, May 2005, pages 575-588.

**Base Photo: Utah AGRC**

**Drawing Not to Scale**

▢ **Approximate Project Boundary**

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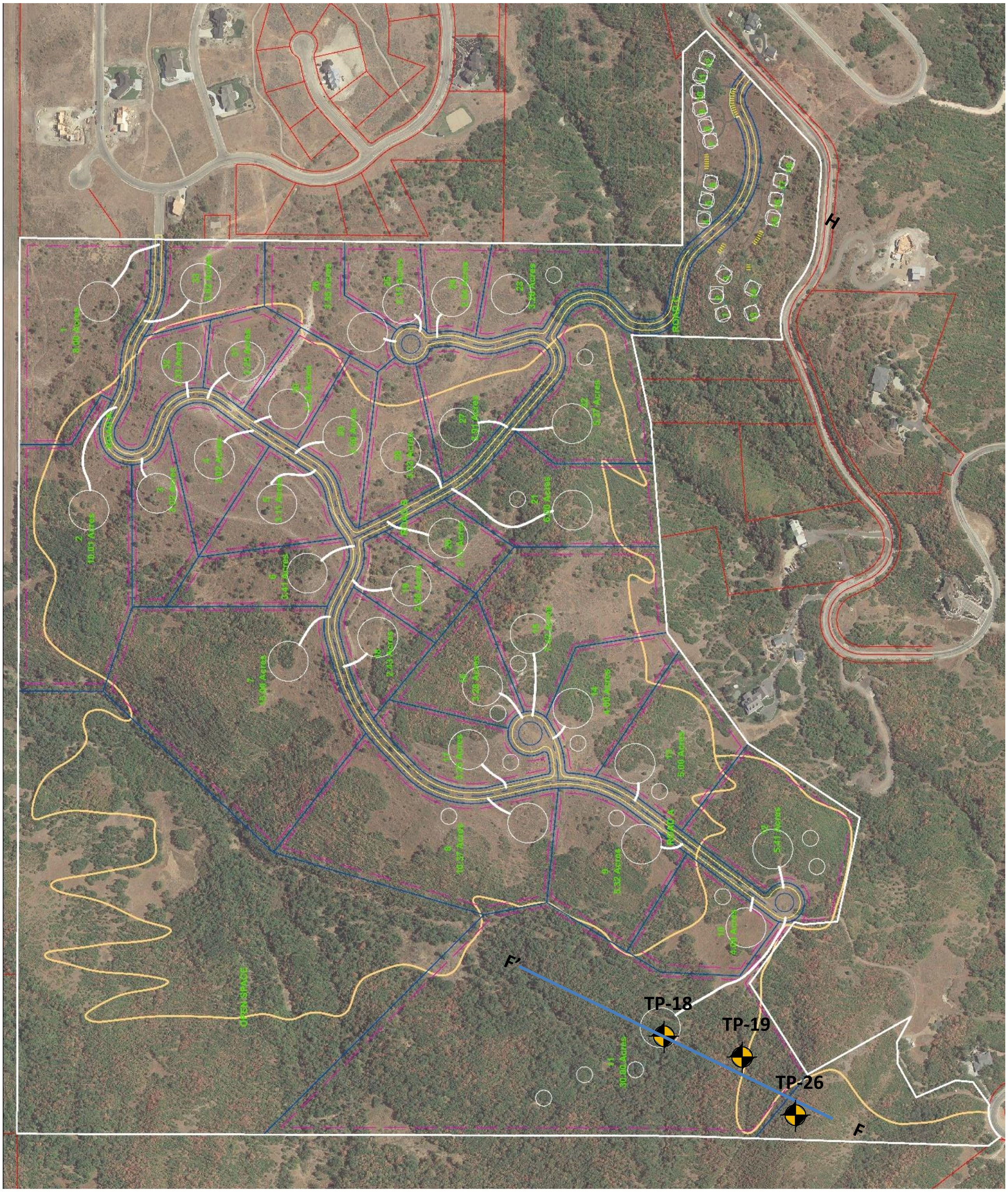
Water Reservoir Legacy - Mountain Development
Huntsville, Weber County, Utah
Project No. 133-011

**Vicinity Map**

Plate
1



⊕ Approximate Test Pit Location

╱ Slope Stability Profile

**Base Map: Great Basin Engineering**

**Drawing Not to Scale**

| | | |
|---|---|---|
| Christensen Geotechnical | Lewis Homes<br>Water Reservoir Legacy - Mountain Development<br>Huntsville, Weber County, Utah<br>Project No. 133-011<br>**Exploration Location Map** | Plate<br>2 |

| Date | | |
|---|---|---|
| | Started: | 10/14/2020 |
| | Completed: | 10/14/2020 |
| | Backfilled: | --- |

# TEST PIT LOG

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

Test Pit No.

## TP-18

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; Lean CLAY - moist, dark brown | | | | | |
| | | | ▨ | | Fat CLAY with sand - very stiff, slightly moist, brown | | | | | |
| | ▥ | | ▨ | | | | | | | |
| 5 | | | ▨ | | | | | | | |
| | | | ▨ | CH | | | | | | |
| | ▥ | | ▨ | | | | 19.7 | 79.6 | 55 | 37 |
| | | | | | Bottom of test pit at 9 feet | | | | | |
| 10 | | | | | | | | | | |
| 15 | | | | | | | | | | |

☒ Bulk/Bag Sample
▥ Undisturbed Sample

▼ StabIlized Groundwater
▽ Groundwater At Time of Excavation

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**Water Reservoir - Legacy Mountain Dev.**
**Huntsville, Weber County, Utah**
**Project No.: 133-011**

**Plate**

**3**

Date
Started: 10/14/2020
Completed: 10/14/2020
Backfilled: ---

# TEST PIT LOG

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

Test Pit No.
## TP-19
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; Lean CLAY - moist, dark brown | | | | | |
| | ▥ | | ▨ | CH | Sandy Fat CLAY - very stiff, slightly moist, brown | | 13.1 | 51.3 | 57 | 40 |
| 5 | ☒ | | ▦ | | Siltstone to Sandstone Bedrock - weathered, weak, brown | | | | | |
| 10 | | | | | Bottom of test pit at 9 feet | | | | | |
| 15 | | | | | | | | | | |

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

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**Project No.: 133-011**

**Plate**
**4**

| Date | | | | |
|---|---|---|---|---|
| Started: 11/24/2020 | # TEST PIT LOG | Logged By: M Christensen | Test Pit No. |
| Completed: 11/24/2020 | | Equipment: Trackhoe | TP-26 |
| Backfilled: --- | | Location: See Plate 2 | Sheet 1 of 1 |

| Depth (feet) | Sample Type | Groundwater | Graphic Log | Group Symbol | Material Description | Dry Density (pcf) | Moisture Content (%) | Minus #200 (%) | Liquid Limit | Plasticity Index |
|---|---|---|---|---|---|---|---|---|---|---|
| | | | ▦ | | Conglomerate to Sandstone Bedrock - completely weathered, weak, brown | | | | | |
| | ▥ | | ▦ | | | | | | | |
| 5 | | | ▦ | | | | | | | |
| | | | | | Bottom of test pit at 7 feet | | | | | |
| 10 | | | | | | | | | | |
| 15 | | | | | | | | | | |

☒ Bulk/Bag Sample
▥ Undisturbed Sample

▼ StablIized Groundwater
▽ Groundwater At Time of Excavation

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**Plate**

**5**

## 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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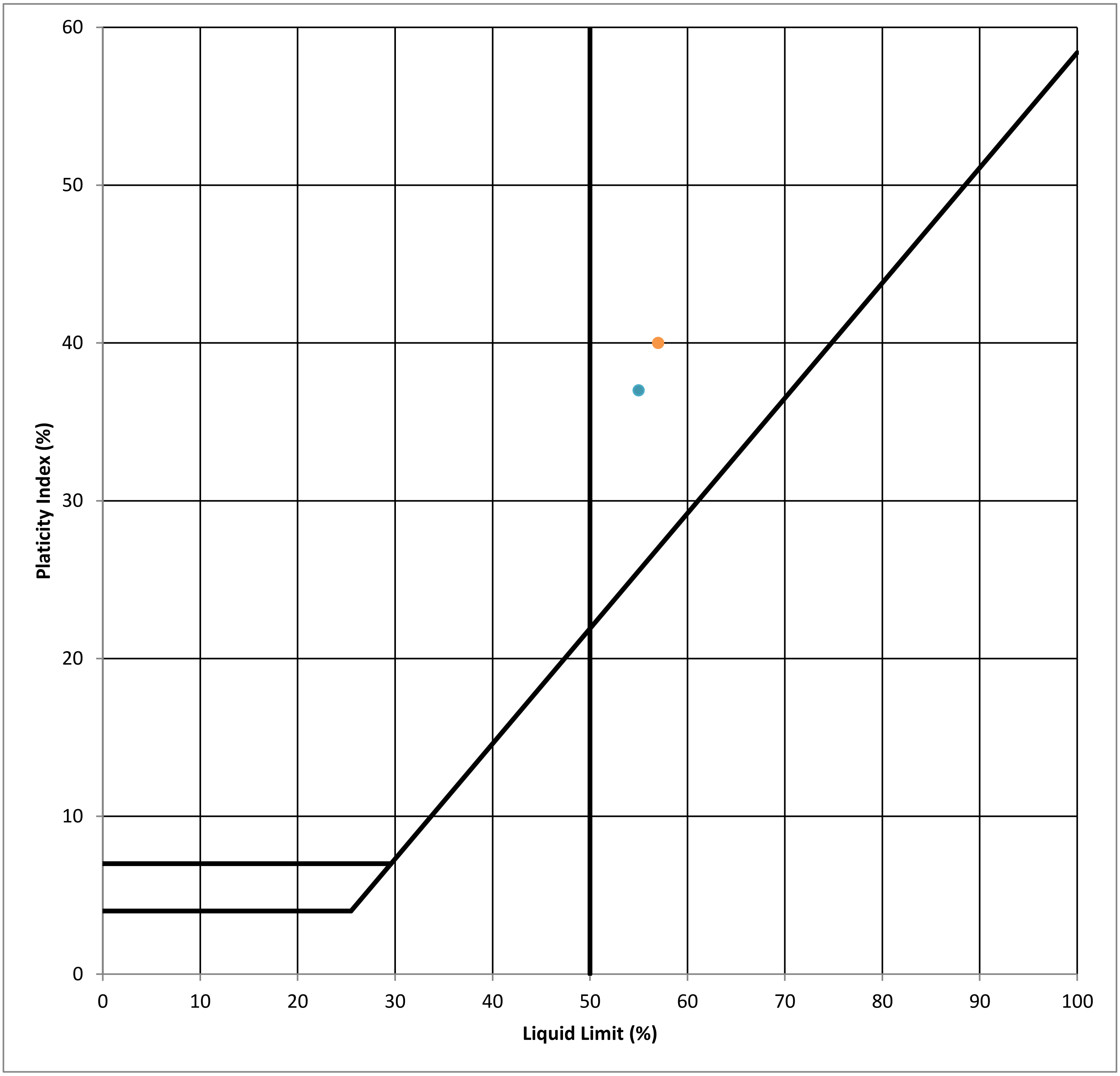
# Soil Terms Key

Plate
6

# Atterberg Limits



| Location | Depth (ft) |  | Classification | Liquid Limit | PI |
|---|---|---|---|---|---|
| TP-18 | 8 | ● | Fat CLAY with sand | 55 | 37 |
| TP-19 | 3 | ● | Sandy Fat CLAY | 57 | 40 |

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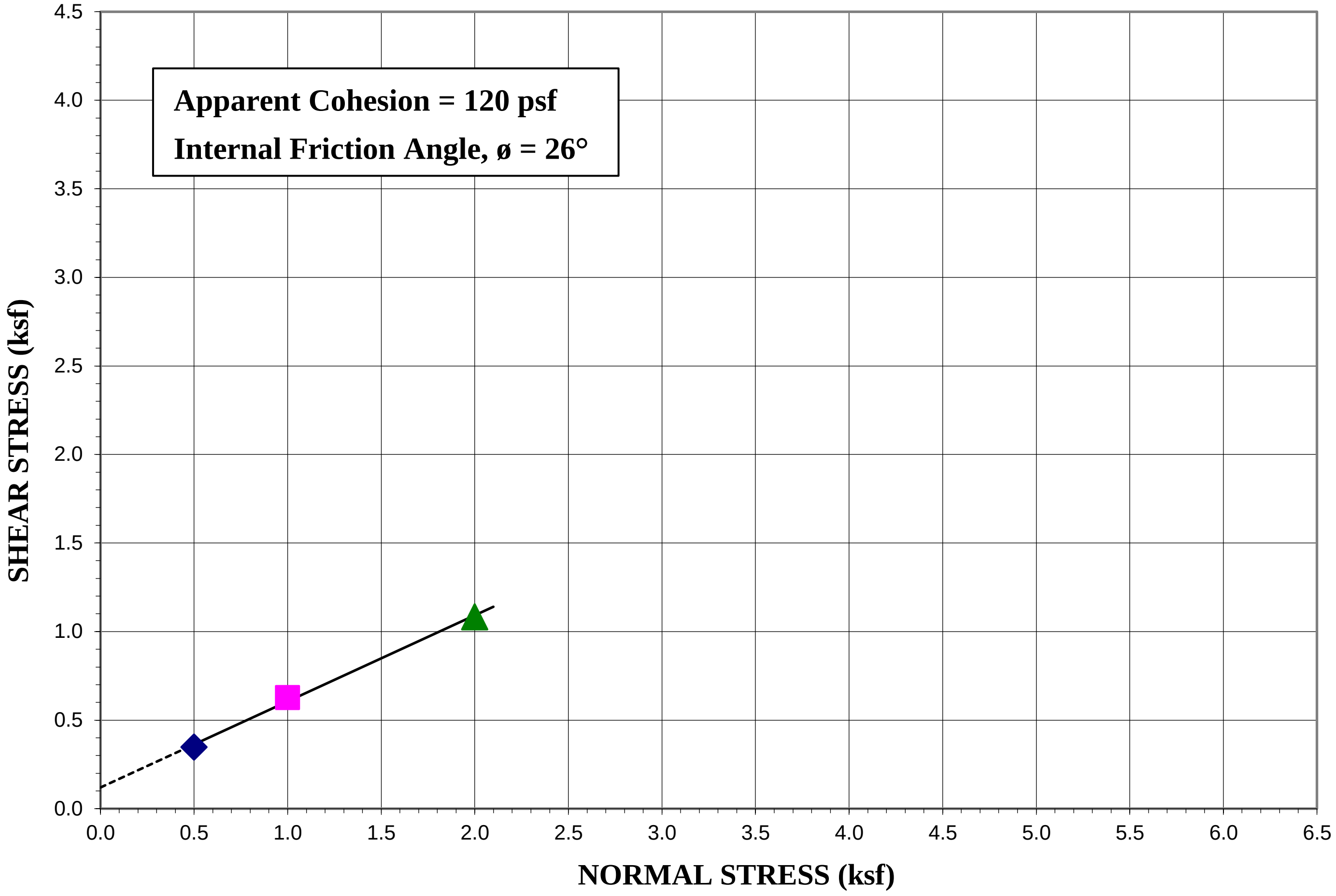
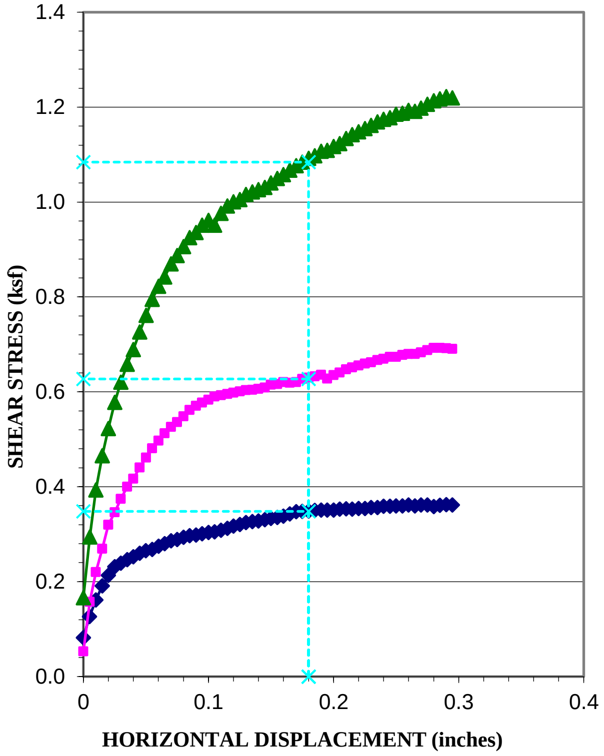
**7**





| | | | |
|---|---|---|---|
| **Location:** TP-18 | **Depth:** 8.0 ft | | |
| **Type of Test:** | Consolidated Drained/Saturated | | |
| **Test No. (Symbol)** | **1 (◆)** | **2 (■)** | **3 (▲)** |
| **Sample Type:** | Fully Softened | | |
| **Initial Height, in.** | 1 | 1 | 1 |
| **Diameter, in.** | 2.4 | 2.4 | 2.4 |
| **Dry Density Before, pcf** | 82.7 | 83.6 | 82.9 |
| **Moisture % Before** | 37.7 | 37.7 | 37.7 |
| **Normal Load, ksf** | 0.5 | 1.0 | 2.0 |
| **Shear Stress, ksf** | 0.35 | 0.63 | 1.08 |
| **Strain Rate** | 0.0006 in/min | | |

| Sample Properties | |
|---|---|
| **Cohesion, psf** | 120 |
| **Friction Angle, ϕ** | 26 |
| **Liquid Limit, %** | 55 |
| **Plasticity Index, %** | 37 |
| **Percent Gravel** | --- |
| **Percent Sand** | --- |
| **Percent Passing No. 200 sieve** | 79.6 |
| **Classification** | Fat CLAY with sand |

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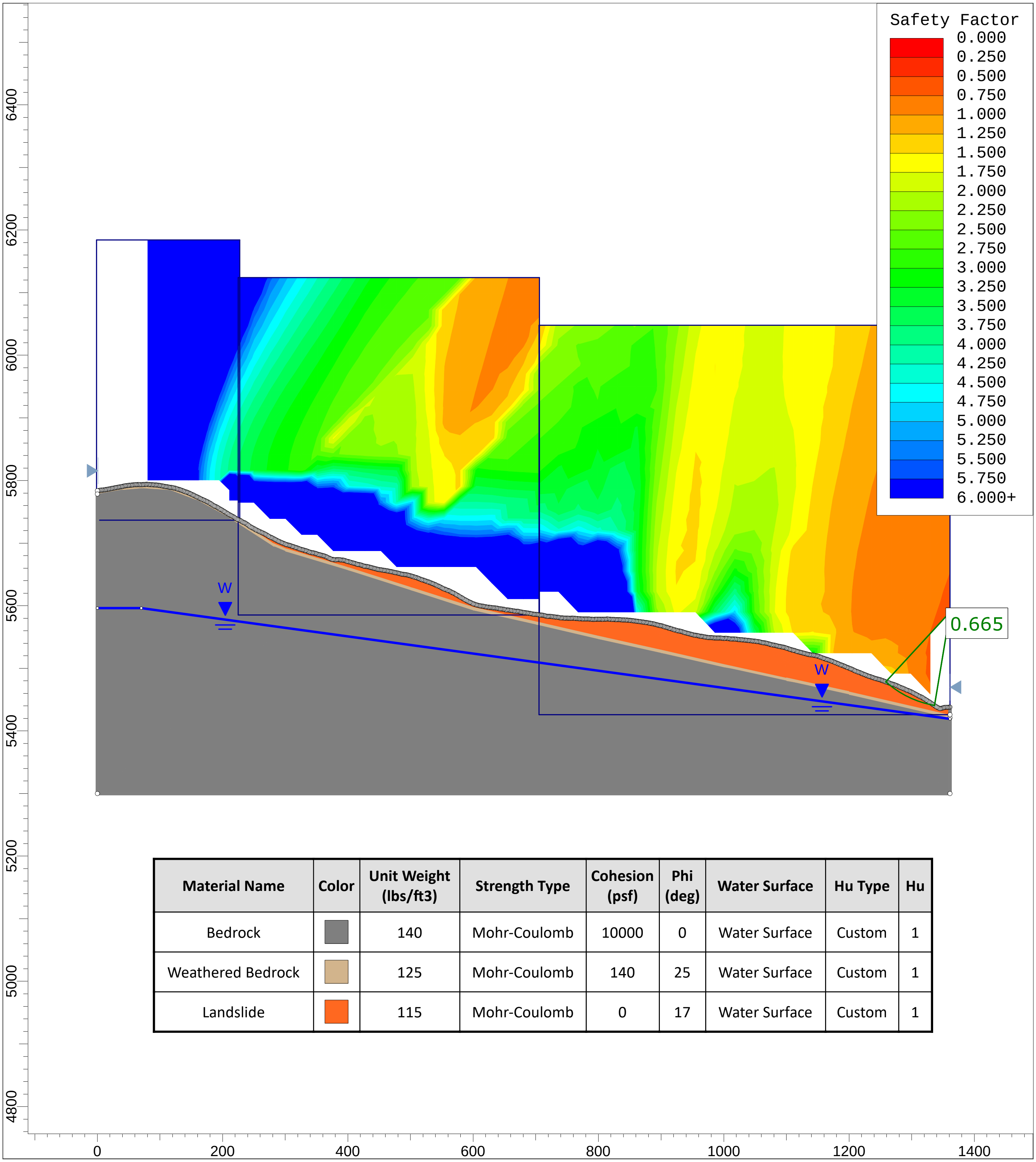

| Material Name | Color | Unit Weight (lbs/ft3) | Strength Type | Cohesion (psf) | Phi (deg) | Water Surface | Hu Type | Hu |
|---|---|---|---|---|---|---|---|---|
| Bedrock | ⬛ | 140 | Mohr-Coulomb | 10000 | 0 | Water Surface | Custom | 1 |
| Weathered Bedrock | 🟫 | 125 | Mohr-Coulomb | 140 | 25 | Water Surface | Custom | 1 |
| Landslide | 🟧 | 115 | Mohr-Coulomb | 0 | 17 | Water Surface | Custom | 1 |

# Global Stability - Static

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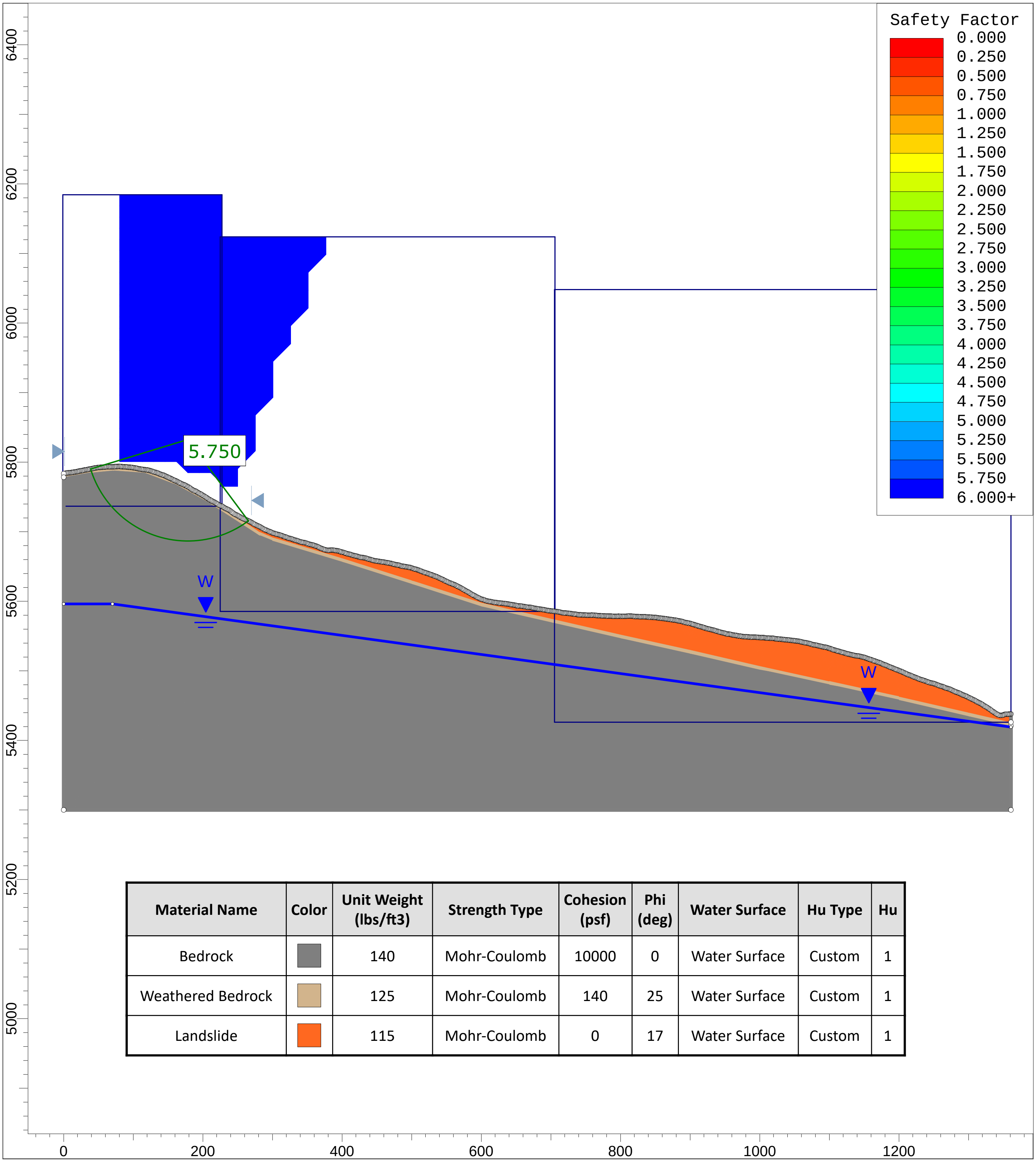

| Material Name | Color | Unit Weight (lbs/ft3) | Strength Type | Cohesion (psf) | Phi (deg) | Water Surface | Hu Type | Hu |
|---|---|---|---|---|---|---|---|---|
| Bedrock | ▨ | 140 | Mohr-Coulomb | 10000 | 0 | Water Surface | Custom | 1 |
| Weathered Bedrock | ▨ | 125 | Mohr-Coulomb | 140 | 25 | Water Surface | Custom | 1 |
| Landslide | ▨ | 115 | Mohr-Coulomb | 0 | 17 | Water Surface | Custom | 1 |

# Global Stability at Reservoir Location - Static

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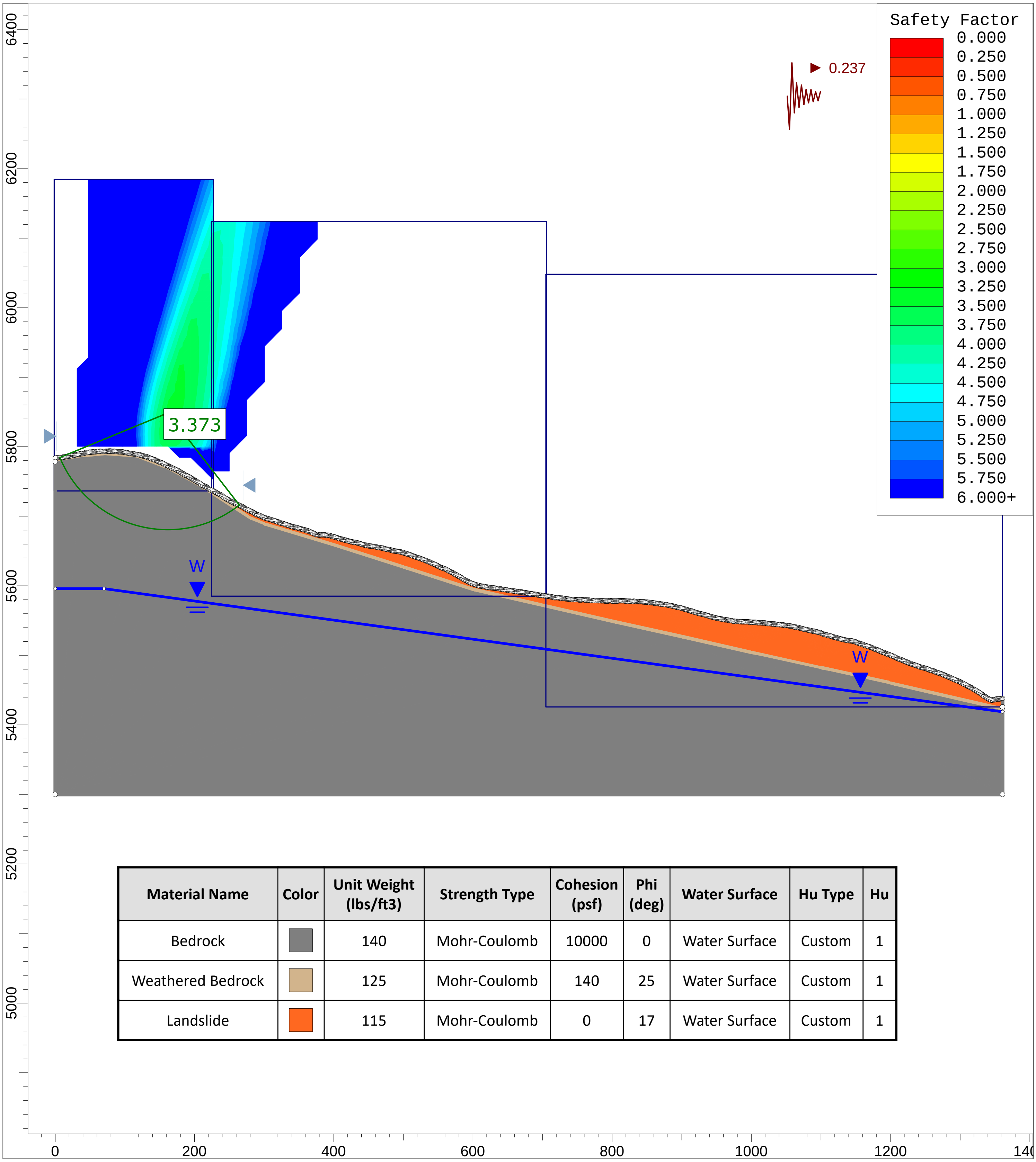

| Material Name | Color | Unit Weight (lbs/ft3) | Strength Type | Cohesion (psf) | Phi (deg) | Water Surface | Hu Type | Hu |
|---|---|---|---|---|---|---|---|---|
| Bedrock | ⬛ | 140 | Mohr-Coulomb | 10000 | 0 | Water Surface | Custom | 1 |
| Weathered Bedrock | 🟫 | 125 | Mohr-Coulomb | 140 | 25 | Water Surface | Custom | 1 |
| Landslide | 🟧 | 115 | Mohr-Coulomb | 0 | 17 | Water Surface | Custom | 1 |

# Global Stability at Reservoir Location - Pseudo Static

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