Geotechnical Investigation
Green Hills Estates Phase 6 Lot 107
Weber County, Utah

August 10, 2018

Prepared by:

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Geotechnical Investigation
Green Hills Estates Phase 6 Lot 107
1088 North Maple Drive
Weber County, Utah
CG Project No.: 152-001

Prepared by:

8/10/18
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August 10, 2018

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1.0 INTRODUCTION

1.1 PURPOSE AND SCOPE OF WORK

This report presents the results of a geotechnical investigation performed for Lot 107 of Green Hills Estates Phase 6 located at 1088 North Maple Drive in Weber County, Utah. The general location of the project is indicated on the Project Vicinity Map, Plate 1. In general, the purposes of this investigation were to evaluate the subsurface conditions; to assess the nature and engineering properties of the subsurface soils; and to provide recommendations for general site grading and for 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 completion of our report, the report *Professional Geologist Site Reconnaissance and Review* for the site by GCS Geoscience dated September 11, 2017 was reviewed to assist in our assessments.

The work performed for this report was authorized by Mr. Randy Aadland and was conducted in accordance with the Christensen Geotechnical proposal dated August 15, 2017.

1.2 PROJECT DESCRIPTION

Based on conversations with our client, we understand that the proposed construction is to consist of single-family residence. The proposed structure is to have a footprint on the order of 2,000 square feet and is to be one story in height with slab-on-grade floors. Footings loads for the proposed structure are anticipated to be on the order of 3 to 4 klf for walls and 100 psf for floors. If structural loads are different than those assumed, Christensen Geotechnical should be notified and allowed to reevaluate our recommendations.

2.0 METHODS OF STUDY

2.1 FIELD INVESTIGATION

The subsurface conditions at the site were explored by excavating three test pits to depths of 7, 8 and 12 feet below existing site grade. The approximate locations of the test pits are shown on the Exploration Location Map, Plate 2. Logs of the subsurface conditions as encountered in the test pits were recorded at the time of excavation and are presented on the Test Pit Logs, Plates 3 through 5. A key to the terms used on test pit logs may be found on Plate 6.

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

2.2 LABORATORY TESTING

Representative samples were tested in the laboratory to evaluate their pertinent engineering properties. Laboratory tests included moisture content and density determinations, grain size distribution analyses, material passing the No. 200 sieve analyses, Atterberg limits determinations, and a one-dimensional consolidation test. A summary of our laboratory testing is presented in the table below:

Table No. 1: Laboratory Test Results

Test Hole No.	Depth (ft.)	Dry Density (pcf)	Moisture Content (%)	Atterberg Limits		Grain Size Distribution (%)			Soil Type
				LL	PI	Gravel (+#4)	Sand	Silt/Clay (- #200)	
TP-1	6		4.4			56.1	18.4	25.4	GC
TP-2	2½		17.1	47	29			81.2	CL
TP-2	6		10.6			17.2	49.2	33.6	SC
TP-3	4	116.6	19.6	50	30			72.5	CH
TP-3	11		8.6			47.8	14.9	37.3	GC

The results of 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 6 through 9).

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 west with a grade of approximately 20 percent, with a steeper-cut slope associated with Maple Drive along the west property line. The cut slope was 20 to 30 feet in height with a grade of 35 to 50 percent. Vegetation at the site generally consisted of common grasses, weeds and sagebrush. The site was bordered by Maple Drive to the west and undeveloped land on all other sides.

3.2 SUBSURFACE CONDITIONS

3.2.1 Soils

Based on the three test pits completed for this investigation, the site is covered with approximately 1 to 1½ feet of topsoil. Below the topsoil, soils generally consist of Clayey GRAVEL with sand (GC) with occasional zones of Clayey SAND with gravel (SC), Lean CLAY with sand (CL), and Fat CLAY with sand (CH) through the maximum depths explored (10 feet). Tackhoe refusal was encountered on very dense soils at depths of 7 and 11 feet in test pits TP-2 and TP-3, respectively.

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, with a possible rise of up to several feet. However, we do not anticipate groundwater affecting the proposed project as planned.

4.0 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 the United States Geological Survey (USGS)-developed seismic hazard maps depicting probabilistic ground motions and spectral response. Seismic design values, including the design spectral response, may be calculated for a specific site using the USGS Seismic Design Maps web-based application and the project site's approximate latitude and longitude and Site Class. Based on our field exploration, it is our opinion that this location is best described as a Site Class D, which represents a "stiff soil" profile. The spectral acceleration values obtained from the USGS web-based application are shown below.

Table 2: IBC Seismic Response Spectrum Values

Site Location: **Latitude = 41.2796^0 N** **Longitude = -111.7277^0 W**			
Spectral Period (sec)	**Response Spectrum Spectral Acceleration (g)**		
0.2	S_S=0.778g	S_{MS}=0.925g	S_{DS}=0.617g
1.0	S_1= 0.258g	S_{M1}=0.487g	S_{D1}=0.324g

Using these values, the peak ground acceleration (PGA) is estimated to be 0.37g.

5.0 ENGINEERING ANALYSIS AND RECOMMENDATIONS

5.1 GENERAL CONLUSIONS

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

5.2 EARTHWORK

5.2.1 General Site Preparation and Grading

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

Based on the test pits excavated at the site, the site is covered with approximately 1 to 1½ feet of topsoil. The topsoil should be removed from below footings, concrete flatwork, and pavements. 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

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

5.2.3 Temporary Construction Excavations

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

5.2.4 Structural Fill and Compaction

All fill placed for support of structures, concrete flatwork and pavements should consist of structural fill. Structural fill may consist of the native gravel soils with particles larger than 4 inches in diameter removed. The native clay soils should not be used. 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 fines content (material passing the No. 200 sieve) of 30 percent. The liquid limit of the fines (material passing the No. 200 sieve) should not exceed 35 and the plasticity index should be less 15. All structural fill, whether native soils or imported material, should be free of topsoil, vegetation, frozen material, particles larger than 4 inches in diameter, and any other deleterious materials. Any imported materials should be approved by the geotechnical engineer prior to importing.

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

5.3 FOUNDATIONS

Foundations for the planned structure at the site may consist of conventional continuous and/or spread footings. Due to the potential for landslides at the site, footings should either extend down through the existing landslide deposits and be founded on the underlying non-displaced soils or be founded on structural fill which extends down through the existing landslide deposits. Based on the test pits excavated for this investigation, we estimate that the landslide deposits at the planned location of the house extend 7 to 8 feet below existing site grade. If clay soils are exposed below footings, the clay should be over-excavated to allow placement of at least 36-inches of structural fill. Footings for the proposed structure should be a minimum of 20 inches and 30 inches wide for continuous and spot footings, respectively. Exterior footings should be established at a minimum of 30 inches below the lowest adjacent grade to provide frost protection and confinement. Interior footings not subject to frost should be embedded a minimum of 18 inches for confinement.

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

5.5 LATERAL EARTH PRESSURES

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

Table No. 3: Lateral Earth Pressures

Condition	Lateral Pressure Coefficient	Equivalent Fluid Density (pcf)
Active Static	0.36	42
Active Seismic	0.13	15
At-Rest	0.53	61
Passive Static	2.77	319
Passive Seismic	-0.29	-33

We recommend that walls which are allowed little or no wall movement be designed using “at rest” conditions. Walls allowed to rotate at least 0.4 percent of the wall height may be designed with “active” pressures. The coefficients and densities presented above assume level backfill with no buildup of hydrostatic pressures. Hydrostatic and any surcharge loads should be added to the presented values if anticipated. If sloping backfill is present, we recommend the geotechnical engineer be consulted to provide more precise 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 foundations soils and by passive pressure developed by backfill against the wall. For footings on native soils, we recommend an ultimate coefficient of friction of 0.38 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, or which are above the recommended minimum depths of embedment, should usually be neglected in design.

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

5.6 CONCRETE SLAB-ON-GRADE CONSTRUCTION

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

5.7 MOISTURE PROTECTION AND SURFACE DRAINAGE

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

1. The ground surface should be graded to drain away from the structures in all directions, with a minimum fall of 8 inches in the first 10 feet.
2. Roof runoff should be collected in rain gutters with down spouts. The down spouts should discharge all collected water into a solid pipe which discharges all water at the bottom of the slope at Maple Drive.
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. Irrigation of the property should be minimized and xeriscaping should be considered.
6. Snow which is plowed from the driveway should not be stockpiled on the slope, but should be distributed at the bottom of the slope on Maple Drive.

5.8 SUBSURFACE DRAINAGE

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

5.9 SLOPE STABILITY

Due to the landslide deposits identified in the “Professional Geologist Site Reconnaissance and Review” report by GCS Geoscience, the stability of the slope at the site was assessed using the Slide computer program and the modified Bishop’s method of slices. The profile used in our assessment was based on a site plan provided to us by Habitations Residential Design Group.

During our visit to the site to excavate test pits, a small landslide was identified approximately 100 feet south of Lot 107 within the road cut associated with Maple Drive. The residual strength of the landslide deposits used in our assessment was developed by back calculating strength within this landslide. Our back calculation assumed that this landslide has a safety factor of 1. The profile used in our back calculation assessment was based on measurements made with handheld equipment. The location of the landslide was situated below a small drainage in an area where plants associated with high moisture contents were visible. Given the evidence of high moisture, a relatively-high groundwater assumption was used in our back calculation. The results of this back calculation indicate a soil strength consisting of an angle of internal friction of 19 degrees with no cohesion. The results of this assessment may be found on Plate 10. The strength for the clayey gravel and clayey sand soils below the landside deposits were assumed to consist of an angle of internal friction of 35 degrees with 100 psf cohesion.

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 4.0, the peak ground acceleration at this site is estimated to be 0.37g. As is common practice, half of this value was used in our pseudo static assessment. Minimum factors of safety of 1.5 and 1.0 for static and seismic conditions, respectively, were considered acceptable. Our analyses indicate that the proposed home location has factors of safety of at least 1.5 and 1.0 for the static and pseudo static conditions, respectively. The results of our slope stability analyses may be found on Plates 11 and 12.

It should be understood that the slope stability analyses presented above are based on the site plan provided to us by Habitations Residential Design Group. Significant changes to the house location or changes to the site grades such as steeping slopes with cuts or fills may adversely affect the stability of the slopes at the site and increase the risk of slope failures. In addition, our analyses assumed that the soils at the site will remain in an unsaturated condition. The addition of water into the subgrade soils may also adversely affect the stability of the slopes and increase the risk of slope failures. In order to reduce these risks, we recommend that all retaining walls constructed on the site be engineered. The engineering for all retaining walls should include an assessment for slope stability. If significant cuts and fills are planned, Christensen Geotechnical should be consulted; additional analyses may be required. In addition, we recommend that all water from roof drains and the foundation drain be collected in a non-perforated pipe which discharges at the bottom of the slope on Maple Drive. Irrigation of the property should be minimized and xeriscaping should be considered. Snow which is plowed from the driveway should not be stockpiled on the slope, but should be distributed at the bottom of the slope on Maple Drive.

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 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

Schlenker, Gregory C., September 11, 2017, “Report Professional Geologist Site Reconnaissance and Review, 4.72 Acre Parcel #21-082-0002, Lot #107, Green Hills Estates Phase 6, 1088 Maple Street, Huntsville, Weber County, Utah,” GCS Geoscience, consultant’s unpublished report.

Base Photo: Utah AGRC

Drawing Not to Scale

▢ **Approximate Project Boundary**

Christensen Geotechnical	Randy Aadland Green Hills Estates Phase 6 Lot 107 Weber County, Utah Project No. 152-001 **Vicinity Map**	Plate 1

Base Map: Ensign Engineering

⊕ **Approximate Test Pit Location**

Drawing Not to Scale

— **Slope Stability Profile**

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Randy Aadland
Green Hills Estates Phase 6 Lot 107
Weber County, Utah
Project No. 152-001

Exploration Location Map

Plate
2

Date	Started: 7/20/2018 Completed: 7/20/2018 Backfilled: 7/20/2018

TEST PIT LOG

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

Test Pit No.
TP-1
Sheet 1 of 1

Depth (feet)	Sample Type	Groundwater	Graphic Log	Group Symbol	Material Description	Dry Density (pcf)	Moisture Content (%)	Minus #200 (%)	Liquid Limit	Plastic Limit
			▨		Clayey GRAVEL with sand - dense, slightly moist, brown					
			▨	GC						
5			▨							
	☒		▨				4.4	25.4		
					Bottom of test pit at 8 feet					
10										
15										

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

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Randy Aadland
Green Hills Estates Phase 6 Lot 107
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Plate
3

Date		
Started:	7/20/2018	
Completed:	7/20/2018	
Backfilled:	7/20/2018	

TEST PIT LOG

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

Test Pit No.
TP-2
Sheet 1 of 1

Depth (feet)	Sample Type	Groundwater	Graphic Log	Group Symbol	**Material Description**	Dry Density (pcf)	Moisture Content (%)	Minus #200 (%)	Liquid Limit	Plastic Limit
			■		Topsoil; Clayey GRAVEL with sand - moist, dark brown					
	▥		▨	CL	Lean CLAY with sand - very stiff, slightly moist, brown		17.1	81.2	47	29
5	☒		▧	GC	Clayey GRAVEL with sand - very dense, slighlty moist, brown					
	☒		▒	SC	Clayey SAND with gravel - very dense, slighlty moist, brown		10.6	33.6		
					Refusal on very dense soil at 7 feet					
10										
15										

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

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Randy Aadland
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Plate
4

Date		
Started:	7/20/2018	
Completed:	7/20/2018	
Backfilled:	7/20/2018	

TEST PIT LOG

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

Test Pit No.

TP-3

Sheet 1 of 1

Depth (feet)	Sample Type	Groundwater	Graphic Log	Group Symbol	**Material Description**	Dry Density (pcf)	Moisture Content (%)	Minus #200 (%)	Liquid Limit	Plastic Limit
			▩		Topsoil; Clayey GRAVEL with sand - moist, dark brown					
			▒	GC	Clayey GRAVEL with sand - very dense, slighlty moist, brown					
5	▥		▧	CH	Fat CLAY with sand - very stiff, slightly moist, brown	116.6	19.6	72.5	50	30
	▥		▨	CL	Lean CLAY - stiff, slightly moist, brown, with calcium carbonate mottling					
10	☒		▒	GC	Clayey GRAVEL with sand - very dense, slighlty moist, brown		8.6	37.3		
15					Refusal on very dense soil at 11 feet					

☒ Bulk/Bag Sample
▥ Undisturbed Sample

▼ StabIlized 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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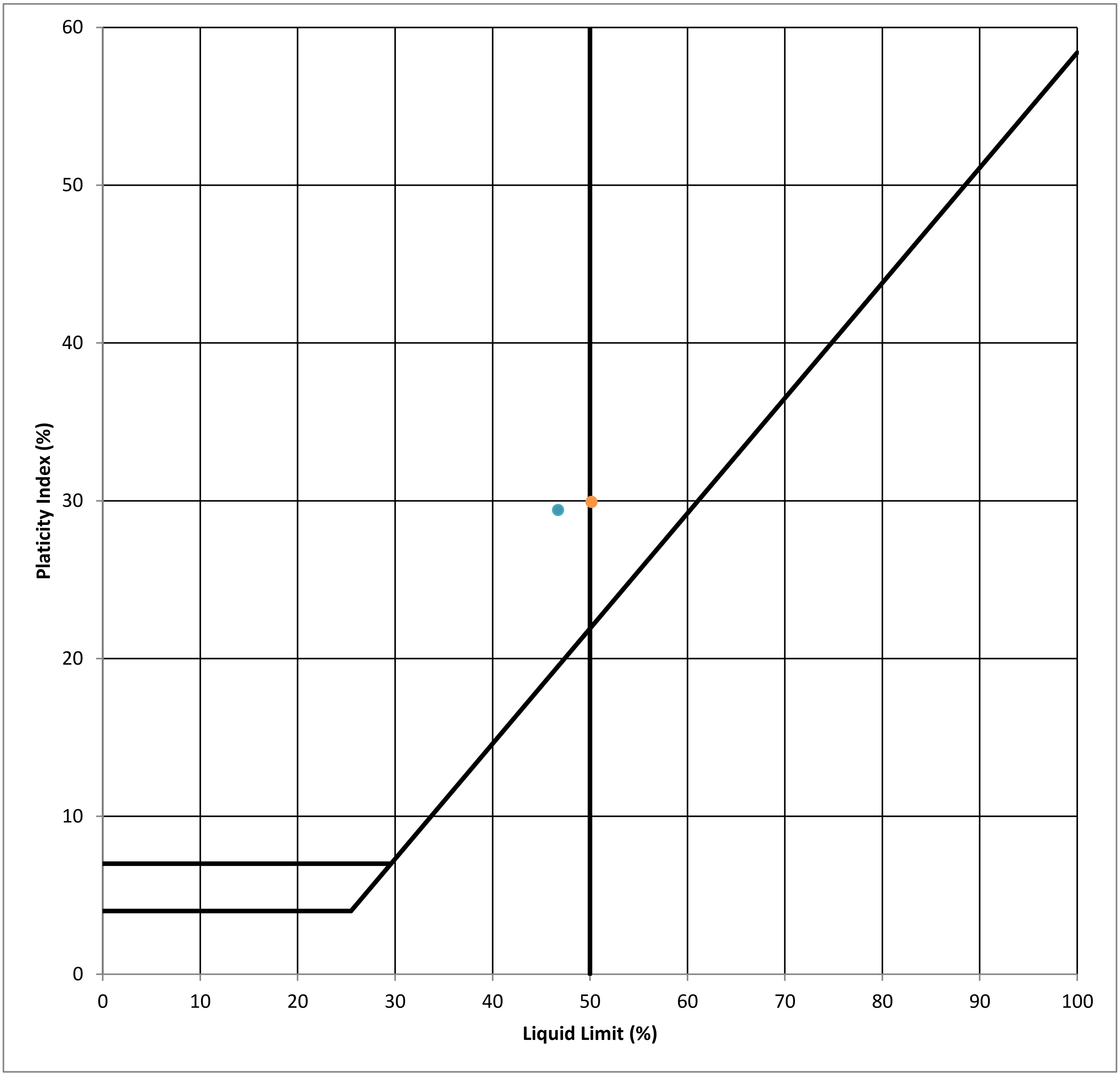
Soil Terms Key

Plate 6

Atterberg Limits



Location	Depth (ft)		Classification	Liquid Limit	PI
TP-2	2½	●	Lean CLAY with sand	47	29
TP-3	4	●	Fat CLAY with sand	50	30

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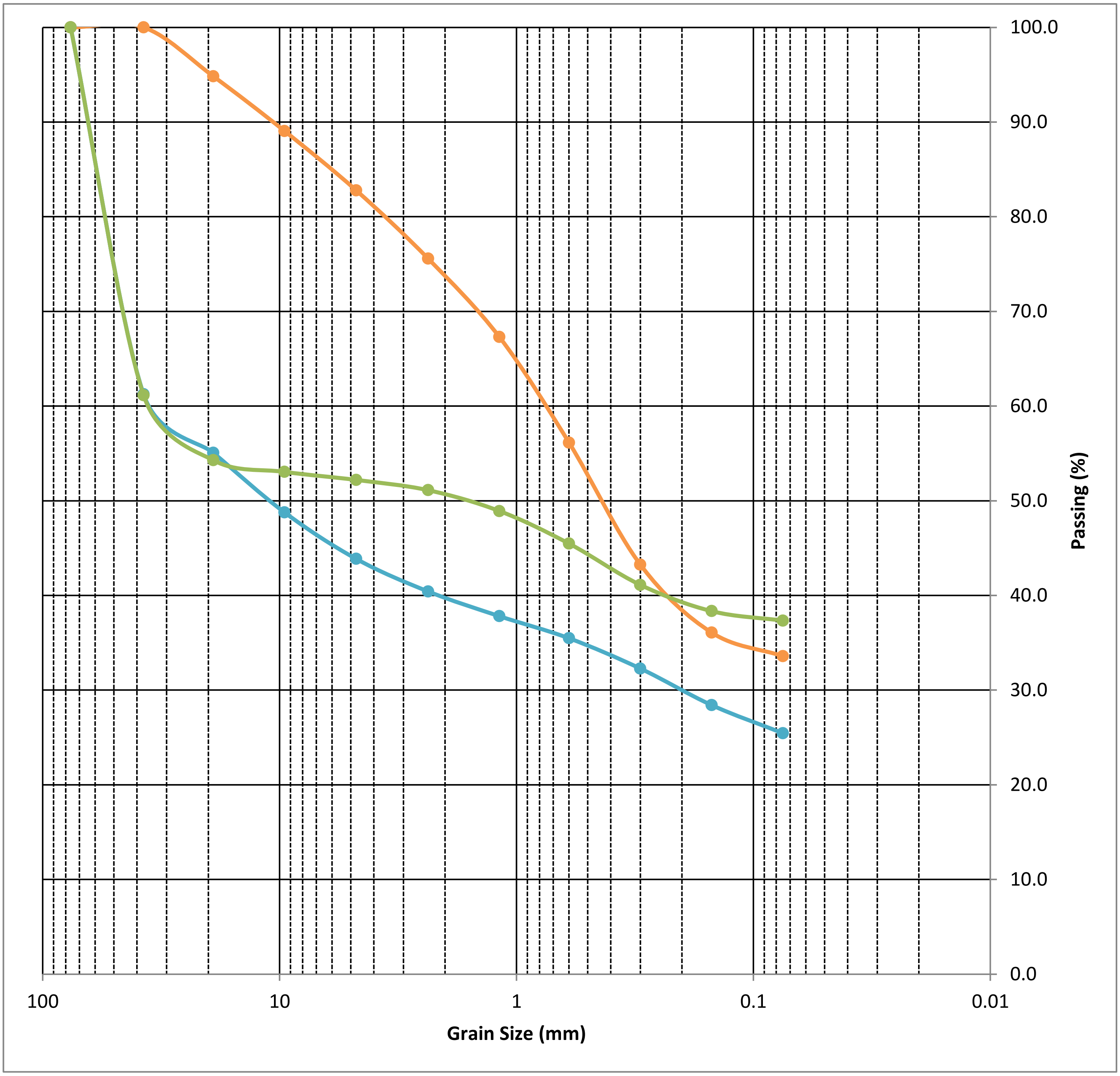
Grain Size Distribution



Location	Depth		Classification	% Gravel	% Sand	% Silt and Clay
TP-1	6	●	Clayey GRAVEL with sand	56.1	18.4	25.4
TP-2	6	●	Clayey SAND with gravel	17.2	49.2	33.6
TP-3	11	●	Clayey GRAVEL with sand	47.8	14.9	37.3

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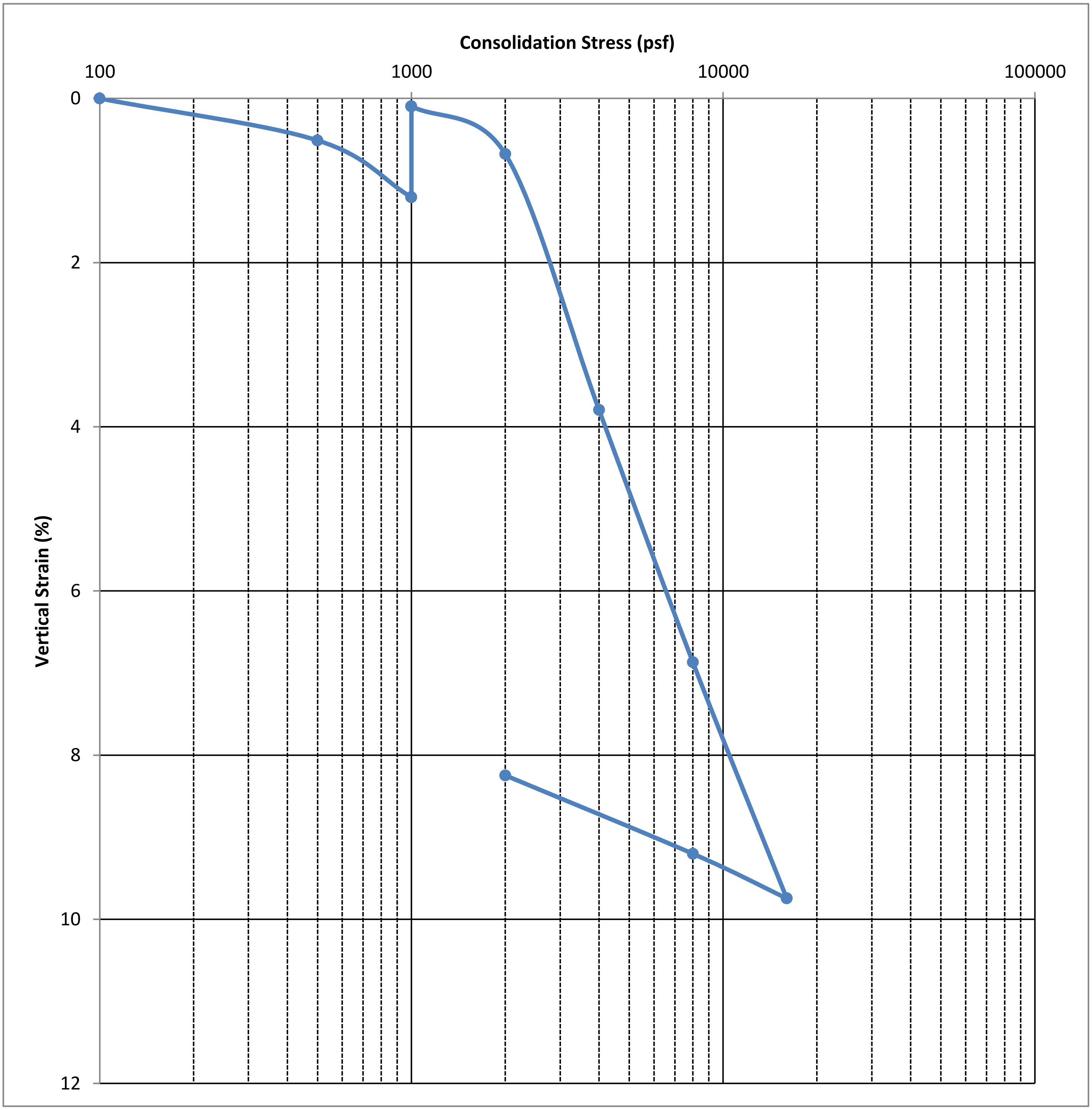
1-D Consolidation



Location	Depth (ft)	Dry Density (pcf)	Moisture Content (%)	σ_o (psf)	σ_p (psf)	C_c	C_r	OCR
TP-3	4	116.6	19.6	600	2,000	0.099	0.017	3.3

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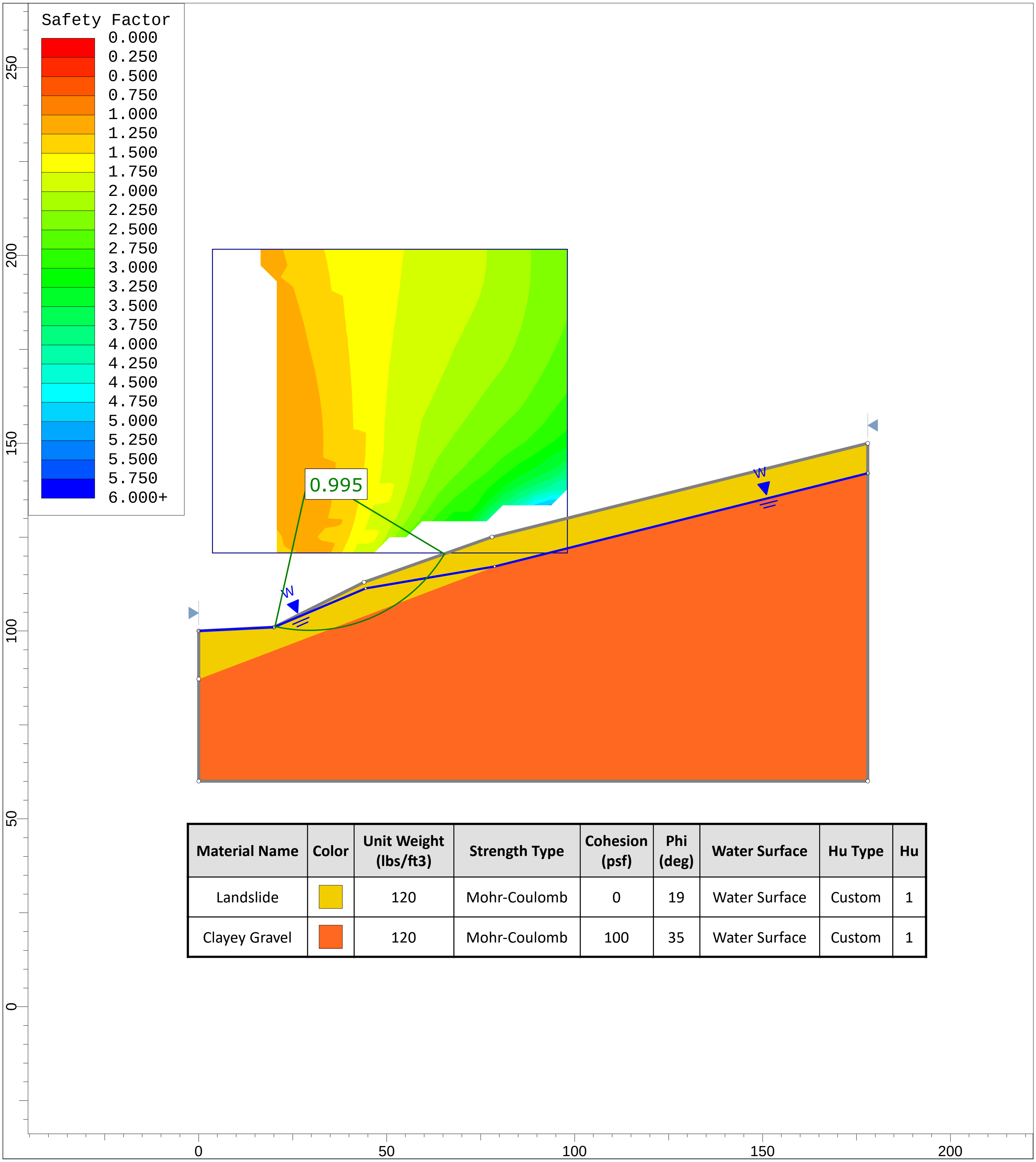

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Hu Type	Hu
Landslide	🟨	120	Mohr-Coulomb	0	19	Water Surface	Custom	1
Clayey Gravel	🟧	120	Mohr-Coulomb	100	35	Water Surface	Custom	1

Back Calculation

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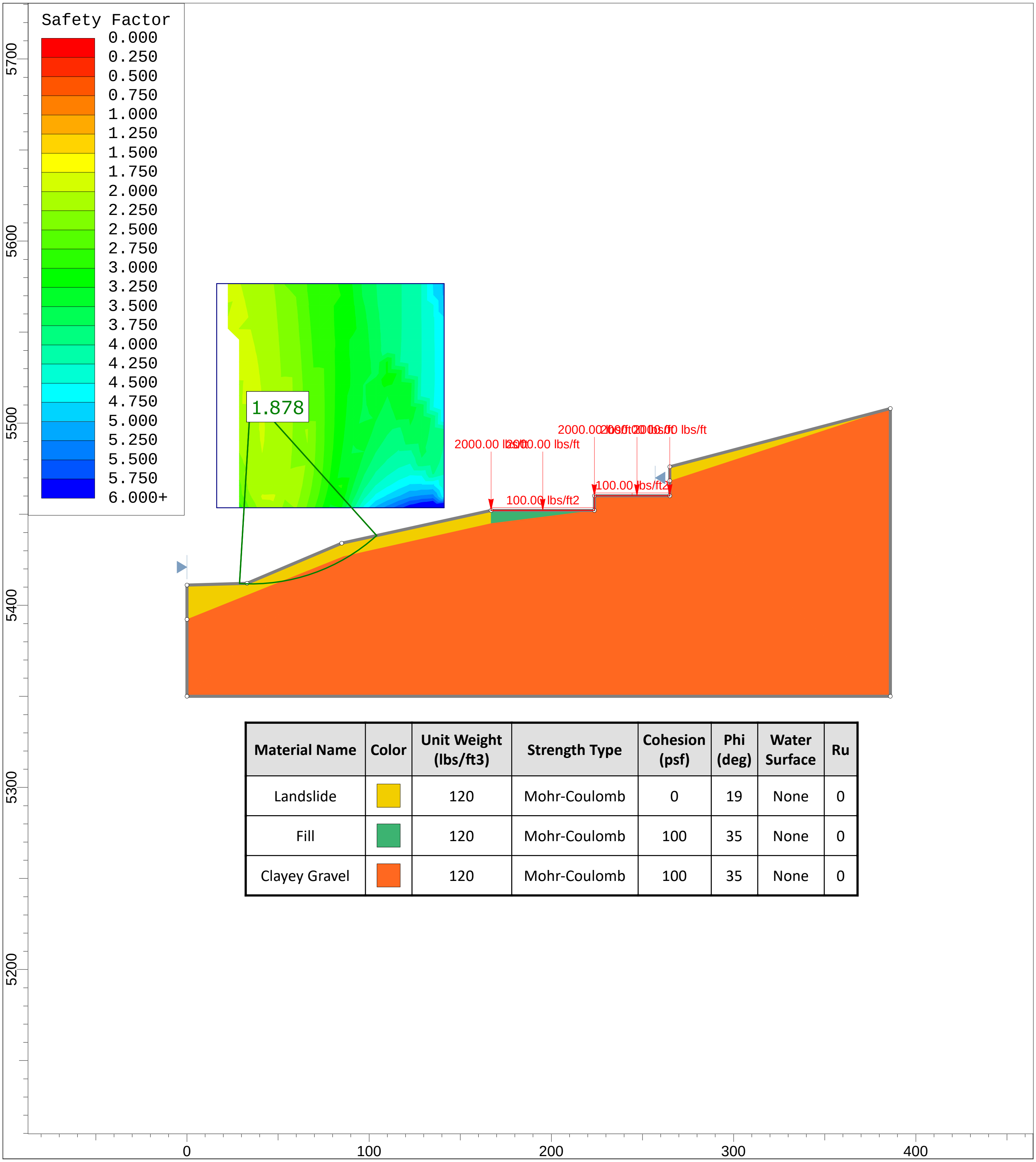

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Ru
Landslide	🟨	120	Mohr-Coulomb	0	19	None	0
Fill	🟩	120	Mohr-Coulomb	100	35	None	0
Clayey Gravel	🟧	120	Mohr-Coulomb	100	35	None	0

Global Stability - Static

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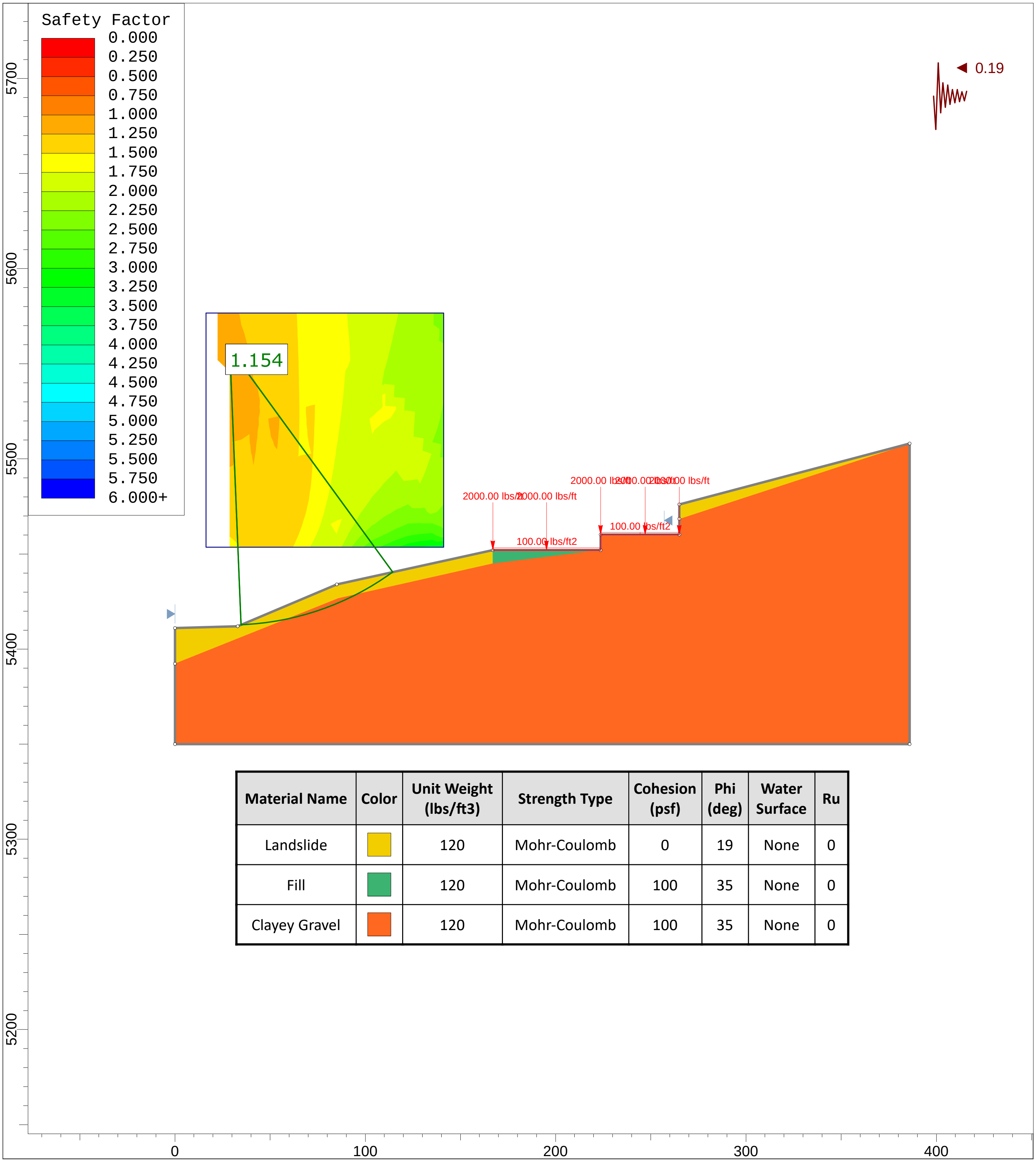

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Ru
Landslide	🟨	120	Mohr-Coulomb	0	19	None	0
Fill	🟩	120	Mohr-Coulomb	100	35	None	0
Clayey Gravel	🟧	120	Mohr-Coulomb	100	35	None	0

Global Stability - Pseudo Static

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