Geotechnical Investigation
Bybee Reservoir No. 2
Weber County, Utah

May 16, 2020

Prepared by:

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Geotechnical Investigation
Bybee Reservoir No. 2
Approximately 6384 Bybee Drive
Weber County, Utah
CG Project No.: 226-001

Prepared by:

5/16/20
LICENSED PROFESSIONAL ENGINEER
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May 16, 2020

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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 located at approximately 6384 South Bybee Drive in Weber County, Utah. The general location of the project is indicated on the Project Vicinity Map, Plate 1. In general, the purposes of this investigation were to evaluate the subsurface conditions and the nature and engineering properties of the subsurface soils, and to provide recommendations for general site grading and for the design and construction of floor slabs and foundations. This investigation included subsurface exploration, representative soil sampling, field and laboratory testing, engineering analysis, and preparation of this report. Prior to the completion of our report, the Geologic Hazards Evaluation report for the site by Western Geologic, dated April 29, 2020, was reviewed to assist in our assessments.

The work performed for this report was authorized by Mr. Matt Hartvigsen, P.E. and was conducted in accordance with the Christensen Geotechnical proposal dated February 7, 2020.

1.2 PROJECT DESCRIPTION

Based on conversations with our client, we understand that the existing water reservoir at the site is to be razed and new reservoir constructed. The new reservoir is to be a buried concrete reservoir on the order of 100 feet in diameter and will extend approximately 23 feet below the existing site grade. The structural loads for the proposed reservoir are anticipated to be on the order of 3 to 6 klf for walls and up to 120 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 completing three borings with a CME 850 tracked drill rig equipped with hollow-stem augers. The approximate locations of the borings are shown on the Exploration Location Map, Plate 2. The borings extended to depths of approximately 51½ feet below the existing site grade. The subsurface conditions as encountered in the borings were recorded at the time of drilling and are presented on the attached Boring Logs, Plates 3 through 5. A key to the symbols and terms used on the Boring Logs may be found on Plate 6.

Representative disturbed soil samples were collected from the borings through the collection of drill cuttings and through the use of standard split-spoon samplers. Due to the granular nature of most of the soils encountered, undisturbed samples were not collected. The classifications for the individual soil units are shown on the attached Boring Logs. The samples were visually classified in the field and portions of each sample were packaged and transported to our laboratory for testing.

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

Table No. 1: Laboratory Test Results

Boring No.	Depth (ft.)	Dry Density (pcf)	Moisture Content (%)	Atterberg Limits		Silt/Clay (- #200)	Minimum Resistivity (Ohm-cm)	pH	Sulfate (ppm)	Direct Shear		Soil Type
				LL	PI					Friction Angle	Cohesion (psf)	
B-1	5		11.1	NP	NP	42.7						SM
B-1	20		14.9	23	6	78.2				34	190	CL-ML
B-1	40		5.8	NP	NP	4.9						SP
B-2	10		4.6	NP	NP	29.6						SM
B-2	30		14.1	NP	NP	39.7						SM
B-2	50		24.7	NP	NP	38.9						SM
B-3	15		2.1	NP	NP	10.4	6,570	9.06	<5.1			SP-SM
B-3	30		21.4	27	11	97.6						CL
B-3	50		19.2	NP	NP	75.6						ML

The results of our laboratory tests are also presented on the Boring Logs, Plates 3 through 5. More detailed laboratory results are presented on the laboratory testing plates, Plates 7 through 11.

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 the location of an existing concrete reservoir on the north side of Bybee Drive. The existing reservoir was located on a nearly level pad which had been graded into an existing slope. The pad was approximately 45 feet above Bybee Drive with slopes above and below the pad. The slope above the pad was approximately 40 feet high with a grade of up to 50 percent. The slope below the pad was approximately 45 feet in height with grades of 30 to 65 percent. The vegetation at the site consisted of common grasses and weeds with some trees.

3.2 SUBSURFACE CONDITIONS

3.2.1 Soils

Based on the three borings completed for this investigation, the site is covered with up to 14 feet of undocumented fill. Below the fill, native soils generally consist of Silty SAND (SM) with occasional interbedded zones of SILT with sand (ML), Silty CLAY with sand (CL-ML), Lean CLAY (CL), Poorly Graded SAND (SP), and Poorly Graded SAND with silt (SP-SM) through the maximum depth explored (51½ feet).

3.2.2 Groundwater

Groundwater was encountered within borings B-1 and B-2 at depths of 45 and 50 feet below existing site grade, respectively. It should be understood that groundwater may fluctuate in response to seasonal changes, precipitation, and irrigation.

4.0 SEISMIC CONSIDERATIONS

4.1 SEISMIC DESIGN CRITERIA

The State of Utah and Utah municipalities have adopted the 2018 International Building Code (IBC) for seismic design. The IBC seismic design is based on seismic hazard maps which depict probabilistic ground motions and spectral response; the maps, ground motions, and spectral response having been developed by the United States Geological Survey (USGS). Seismic design values, including the design spectral response, may be calculated for a specific site using the web-based application by the Applied Technology Council (ATC) and the project site's approximate latitude and longitude and Site Class. Based on our field exploration, it is our opinion that this location is best described as a Site Class D, which represents a "stiff soil" profile. The spectral acceleration values obtained from the ATC web-based application are shown below.

Table 2: IBC Seismic Response Spectrum Values

Site Location: 41.38149º N -111.89300º W	
Name	**Response Spectral Value**
S_S	**1.272**
S_1	**0.474**
S_{MS}	**1.272**
S_{M1}	**See ASCE Section 11.4.8**
S_{DS}	**0.848**
S_{D1}	**See ASCE Section 11.4.8**
PGA	**0.584**
PGA_M	**0.642**

4.2 LIQUEFACTION

Certain areas in the intermountain west possess a potential for liquefaction. Liquefaction is a phenomenon in which soils lose their intergranular strength due to an increase of pore pressures during a dynamic event such as an earthquake. The potential for liquefaction is based on several factors, including 1) the grain-size distribution of the soil, 2) the plasticity of the fine fraction of the soil (material passing the No. 200 sieve), 3) the relative density of the soils, 4) earthquake strength (magnitude) and duration, 5) overburden pressures, and 6) the depth to groundwater.

A review of the “Liquefaction-Potential Map for a Part of Weber County, Utah” (Anderson, 1994), indicates that the subject site is located in an area designated as having a moderate potential for liquefaction. A moderate potential for liquefaction indicates that there is a 10 to 50 percent probability of liquefaction at this site within a 100-year period. Due to the mapped designation, a site-specific liquefaction assessment was made using the subsurface information developed for this investigation. The liquefaction assessment was conducted using the method from the 1996 and 1998 NCEER Workshops (Youd and Idriss, 2001). Our analysis indicates that the site has a low potential for liquefaction.

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 reservoir footprint and flatwork concrete areas. Following the stripping operations, the exposed soils should be proof rolled to a firm, unyielding condition. Site grading may then be conducted to bring the site to design grade.

Based on the borings completed at the site, the site is covered with up to 14 feet of undocumented fill. These fill soils should be removed from below footings and concrete flatwork. Where over-excavation is required, the excavation should extend at least 1 foot laterally for every foot of over-excavation. A Christensen Geotechnical representative should observe the site grading operations.

5.2.2 Soft Soil Stabilization

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

5.2.3 Temporary Construction Excavations

Based on OSHA requirements and the soil conditions encountered during our field investigation, we anticipate that temporary construction excavations at the site that have vertical walls that extend to depths of up to 5 feet may be occupied without shoring; however, where groundwater or fill soils are encountered, flatter slopes may be required. Excavations that extend to more than 5 feet in depth should be sloped or shored in accordance with OSHA regulations for a type C soil. The stability of construction excavations is the contractor's responsibility. If the stability of an excavation becomes questionable, the excavation should be evaluated immediately by qualified personnel.

5.2.4 Structural Fill and Compaction

All fill placed for the support of reservoir and concrete flatwork should consist of structural fill. The structural fill may consist of the native sand soils or an imported material. Imported structural fill, if required, should consist of a relatively well-graded granular soil with a maximum particle size of 4 inches, with a maximum of 50 percent passing the No. 4 sieve and a maximum of 30 percent passing the No. 200 sieve. The liquid limit of the fines (material passing the No. 200 sieve) should not exceed 35 and the plasticity index should be less than 15. Additionally, all structural fill, whether native soils or imported material, should be free of topsoil, vegetation, frozen material, particles larger than 4 inches in diameter, and any other deleterious materials. Any imported materials should be approved by the geotechnical engineer prior to importing.

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

5.3 FOUNDATIONS

The foundations for the planned reservoir may consist of conventional continuous and/or spread footings established either on undisturbed native soil or on properly placed and compacted structural fill which extends down to undisturbed native soil. The footings for the proposed reservoir should be a minimum of 20 inches and 30 inches wide for continuous and spot footings, respectively. Exterior footings should be established 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 that are established on undisturbed native soils 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 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	0.29	35
Active Seismic	0.21	25
At-Rest	0.46	55
Passive Static	3.39	407
Passive Seismic	-0.47	-57

We recommend that walls which are allowed little or no wall movement be designed using "at rest" conditions. Walls that are allowed to rotate at least 0.4 percent of the wall height may be designed with "active" pressures. The coefficients and densities presented above assume level backfill with no buildup of hydrostatic pressures. If anticipated, hydrostatic pressures and any surcharge loads should be added to the presented values. If sloping backfill is present, we recommend that the geotechnical engineer be consulted to provide more appropriate lateral pressure parameters once the design geometry is established.

The seismic active and passive earth pressure coefficients provided in the table above are based on the Mononobe-Okabe method and only account for the dynamic horizontal force produced by a seismic event. The resulting dynamic pressure should therefore be added to the static pressure to determine the total pressure on the wall. The dynamic pressure distribution may be approximated as an inverted triangle, with stress decreasing with depth and the resultant force acting approximately 0.6 times the height of the retaining wall, measured upward from the bottom of the wall.

Lateral building loads will be resisted by frictional resistance between the footings and the foundation soils and by passive pressure developed by backfill against the wall. For footings on native soils, we recommend that an ultimate coefficient of friction of 0.42 be used. If passive resistance is used in conjunction with frictional resistance, the passive resistance should be reduced by ½. Passive earth pressure from soils subject to frost or heave should usually be neglected in design.

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

5.6 CONCRETE SLAB-ON-GRADE CONSTRUCTION

Concrete slabs-on-grade should be constructed over at least 4 inches of compacted gravel to help distribute floor loads, break the rise of capillary water, and to aid in the curing process. The gravel should consist of free-draining gravel compacted to a firm, unyielding condition. To help control normal shrinkage and stress cracking, the floor slab should have adequate reinforcement for the anticipated floor loads, with the reinforcement continuous through the interior joints. In addition, we recommend adequate crack control joints to control crack propagation.

5.7 MOISTURE PROTECTION AND SURFACE DRAINAGE

5.7.1 Surface Drainage

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

5.7.2 Reservoir Under-Drainage

Consideration should be given to the construction of 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 to assist leak detection and allow the discharge of collected water.

5.8 SLOPE STABILITY

Due to the relatively steep slopes at the site, a slope stability assessment was performed using the Slide computer program and the modified Bishop’s method of slices. Two profiles of the slopes on the lot were assessed and are shown on Plate 2. The profiles are based on a site plan by Jones & Associates and the three borings drilled for this investigation. The direct shear testing of a sample of native soil indicated a soil strength consisting of an angle of internal friction of 34 degrees and a cohesion of 190 psf, which was used in our analyses.

The profiles were 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 0.642g. 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 Profile A has safety factors greater than 1.5 and 1.0 for the static and pseudo static conditions. Analysis of profile B indicates that the slope above the proposed reservoir has an adequate static safety factor; however, the pseudo static factor of safety was assessed to be less than 1.0. Due to the low pseudo static factor of safety, a deformation analysis was performed using the Bray and Travasarau method (2007). The results of this assessment indicate approximately 6 inches of slope deformation during a strong seismic event. Based on our slope stability assessments and the deformation analysis, we recommend that the proposed reservoir be located at least 15 feet from ascending and descending slopes at the site.

The slope stability analyses presented above are based on the site plan by Jones & Associates and the subsurface investigation and laboratory testing completed for this report. If the proposed grades change significantly from that presented on the Jones & Associates site plan, Christensen Geotechnical should be consulted and additional analyses may be required. The results of our slopes stability assessments may be found on Plates 12 through 16.

5.9 SOIL CORROSION

A representative sample of the native soil collected from the site was tested for soluble sulfate content, minimum resistivity, and pH to assist in assessing the corrosive potential of the native soil to concrete and metals. The test results indicate that the native soil has a soluble sulfate content of less than 5.1 ppm, which represents a low risk of sulfate attack. Based on this result, Type I/II Portland cement may be used for the proposed reservoir. Resistivity and pH testing indicate a minimum resistivity of 6,570 ohm-cm and a pH of and 9.06, which represents a moderately corrosive environment for metals. We recommend that a qualified corrosion engineer be retained to design a corrosion protection system where metals will be in contact with the native soils.

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

Anderson, L.R., Keaton, J.R. and Bay, J.R., 1994, "Liquefaction-Potential Map for a Part of Weber County Utah: Utah Geological Survey," Public Information Series 27.

Bray, Jonathan D. and Travasarou, Thaleia, 2007, "Simplified Procedure for Estimating Earthquake-Induced Deviatoric Slope Displacements," ASCE, Journal of Geotechnical and Geoenvironmental Engineering, April 2007, pages 381-392.

Youd, T. L. and Idriss, I. M., 2001, "Liquefaction Resistance of Soils: Report from the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils," Journal of Geotechnical and Geoenvironmental Engineering, April, 2001.

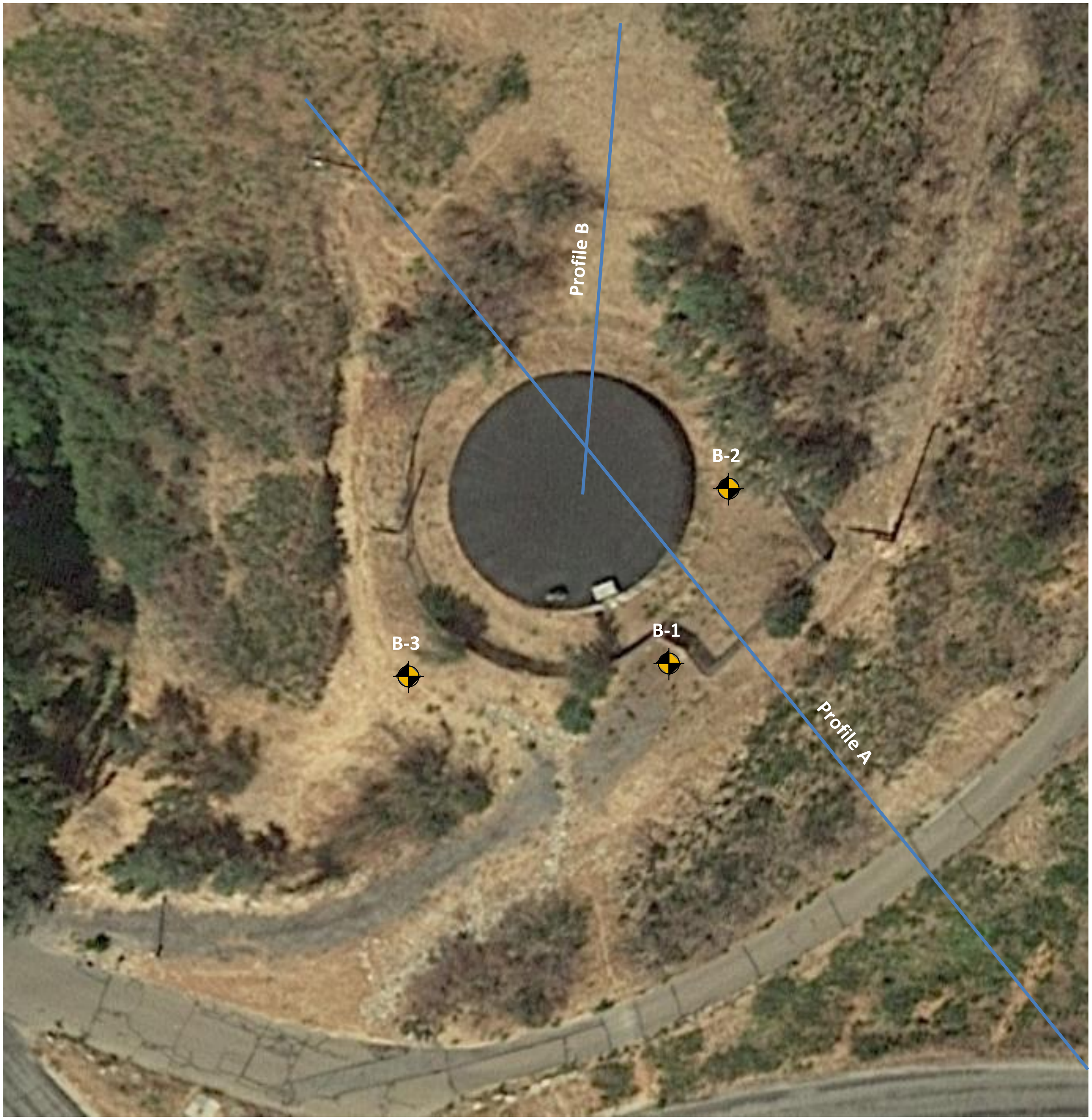
Base Photo: Utah AGRC

Drawing Not to Scale

Approximate Project Boundary

N

Christensen Geotechnical	Uintah City Bybee Reservoir No. 2 Weber County, Utah Project No. 226-001 **Vicinity Map**	Plate 1



Base Photo: Utah AGRC

Approximate Boring Location

Slope Stability Profile

Drawing Not to Scale

N

Christensen Geotechnical	Uintah City Bybee Reservoir No. 2 Weber County, Utah Project No. 226-001 **Exploration Location Map**	Plate 2

Date					
Started:	3/30/2020	**BORING LOG**	Logged By: M Christensen	Boring No.
Completed:	3/30/2020	Sheet 1 of 2	Equipment: CME 850	**B-1**
Backfilled:	3/30/2020		Location: See Plate 2	

Depth (feet)	Sample Type	Blow Counts (blows/foot)	Groundwater	Graphic Log	Symbol	Material Description	Dry Density (pcf)	Moisture Content (%)	Minus #200 (%)	Liquid Limit	Plasticity Index
0						Fill; Silty GRAVEL with sand - moist, brown					
4						Fill; Silty SAND - moist, brown					
5	SPT	15						11.1	42.7	NP	NP
10	SPT	2									
14					SM	Silty SAND - medium dense, moist, brown					
15	SPT	16									
19					CL-ML	Silty CLAY with sand - hard, moist, brown					
20	California	44						14.9	78.2	23	6
23					SM	Silty SAND - medium dense, moist, brown, with occasional clay seams and lenses					
25											

SPT Sampler
California Sampler
Shelby Tube
Bulk/Bag Sample
Stabilized Groundwater
Groundwater at time of Drilling

Christensen Geotechnical

Uintah City
Bybee Reservoir No 2
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Plate
3a

BORING LOG B-1 Continued

Depth (feet)	Sample Type	Blow Counts (blows/foot)	Groundwater	Graphic Log	Symbol	Material Description	Dry Density (pcf)	Moisture Content (%)	Minus #200 (%)	Liquid Limit	Plasticity Index
	SPT Sampler	22				(continued) Silty SAND - medium dense, moist, brown, with occasional clay seams and lenses					
30	SPT Sampler	15			SM						
35	SPT Sampler	16									
						Poorly Graded SAND - medium dense, slightly moist, brown					
40	California Sampler	34			SP			5.8	4.9	NP	NP
						Silty SAND - medium dense, moist, brown, with occasional clay seams and lenses - wet below 45 feet					
45	SPT Sampler	31	Groundwater at time of Drilling								
					SM						
50	SPT Sampler	20									
						Bottom of boring at 51½					

SPT Sampler
California Sampler
Shelby Tube
Bulk/Bag Sample
Stabllized Groundwater
Groundwater at time of Drilling

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Plate

3b

Date			BORING LOG	Logged By: M Christensen	Boring No.
	Started:	3/30/2020		Equipment: CME 850	B-2
	Completed:	3/30/2020		Location: See Plate 2	
	Backfilled:	3/30/2020	Sheet 1 of 2		

Depth (feet)	Sample Type	Blow Counts (blows/foot)	Groundwater	Graphic Log	Symbol	Material Description	Dry Density (pcf)	Moisture Content (%)	Minus #200 (%)	Liquid Limit	Plasticity Index
0						Silty SAND - medium dense, moist, brown, with occasional clay seams and lenses					
5	California Sampler	27									
10	SPT Sampler	14			SM			4.6	29.6	NP	NP
15	SPT Sampler	12									
20	California Sampler	46									
25											

SPT Sampler
California Sampler
Shelby Tube
Bulk/Bag Sample
Stabllized Groundwater
Groundwater at time of Drilling

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Uintah City
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Plate
4a

BORING LOG B-2 Continued

Depth (feet)	Sample Type	Blow Counts (blows/foot)	Groundwater	Graphic Log	Symbol	Material Description	Dry Density (pcf)	Moisture Content (%)	Minus #200 (%)	Liquid Limit	Plasticity Index
	SPT	19				(continued) Silty SAND - medium dense, moist, brown, with occasional clay seams and lenses					
30	SPT	17						14.1	39.7	NP	NP
35	SPT	18									
40	SPT	27									
45	California	37									
50	SPT	24	Groundwater at time of Drilling			- wet below 50 feet		24.7	38.9	NP	NP
						Bottom of boring at 51½					

SPT Sampler
California Sampler
Shelby Tube
Bulk/Bag Sample
Stabllized Groundwater
Groundwater at time of Drilling

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Plate

4b

Date			BORING LOG	Logged By: M Christensen	Boring No.
	Started:	3/30/2020	Sheet 1 of 2	Equipment: CME 850	B-3
	Completed:	3/31/2020		Location: See Plate 2	
	Backfilled:	3/31/2020			

Depth (feet)	Sample Type	Blow Counts (blows/foot)	Groundwater	Graphic Log	Symbol	Material Description	Dry Density (pcf)	Moisture Content (%)	Minus #200 (%)	Liquid Limit	Plasticity Index
0–9	SPT (5)	9				Fill; Silty SAND - medium dense, moist, brown, with occasional clay seams and lenses					
9–14	SPT (10)	29			SM	Silty SAND - medium dense, moist, brown, with occasional clay seams and lenses					
14–18	California (15)	34			SP-SM	Poorly Graded SAND with silt - medium dense, slightly moist, brown		2.1	10.4	NP	NP
18–25	SPT (20)	20			SM	Silty SAND - medium dense, moist, brown, with occasional clay seams and lenses					

SPT Sampler
California Sampler
Shelby Tube
Bulk/Bag Sample
Stabilized Groundwater
Groundwater at time of Drilling

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Plate

5a

BORING LOG B-3 Continued

Depth (feet)	Sample Type	Blow Counts (blows/foot)	Groundwater	Graphic Log	Symbol	Material Description	Dry Density (pcf)	Moisture Content (%)	Minus #200 (%)	Liquid Limit	Plasticity Index
	SPT	22			SM	(continued) Silty SAND - medium dense, moist, brown, with occasional clay seams and lenses					
30	California	37			CL	Lean CLAY - hard, moist, brown, with sand lenses		21.4	97.6	27	11
35	SPT	29			SM	Silty SAND - medium dense, moist, brown, with occasional clay seams and lenses					
40	SPT	21									
45	SPT	24									
50	SPT	22			ML	SILT with sand - very stiff, moist, brown		19.2	75.6	NP	NP
						Bottom of boring at 51½					

SPT Sampler
California Sampler
Shelby Tube
Bulk/Bag Sample
Stablized Groundwater
Groundwater at time of Drilling

Christensen Geotechnical

Uintah City
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Plate

5b

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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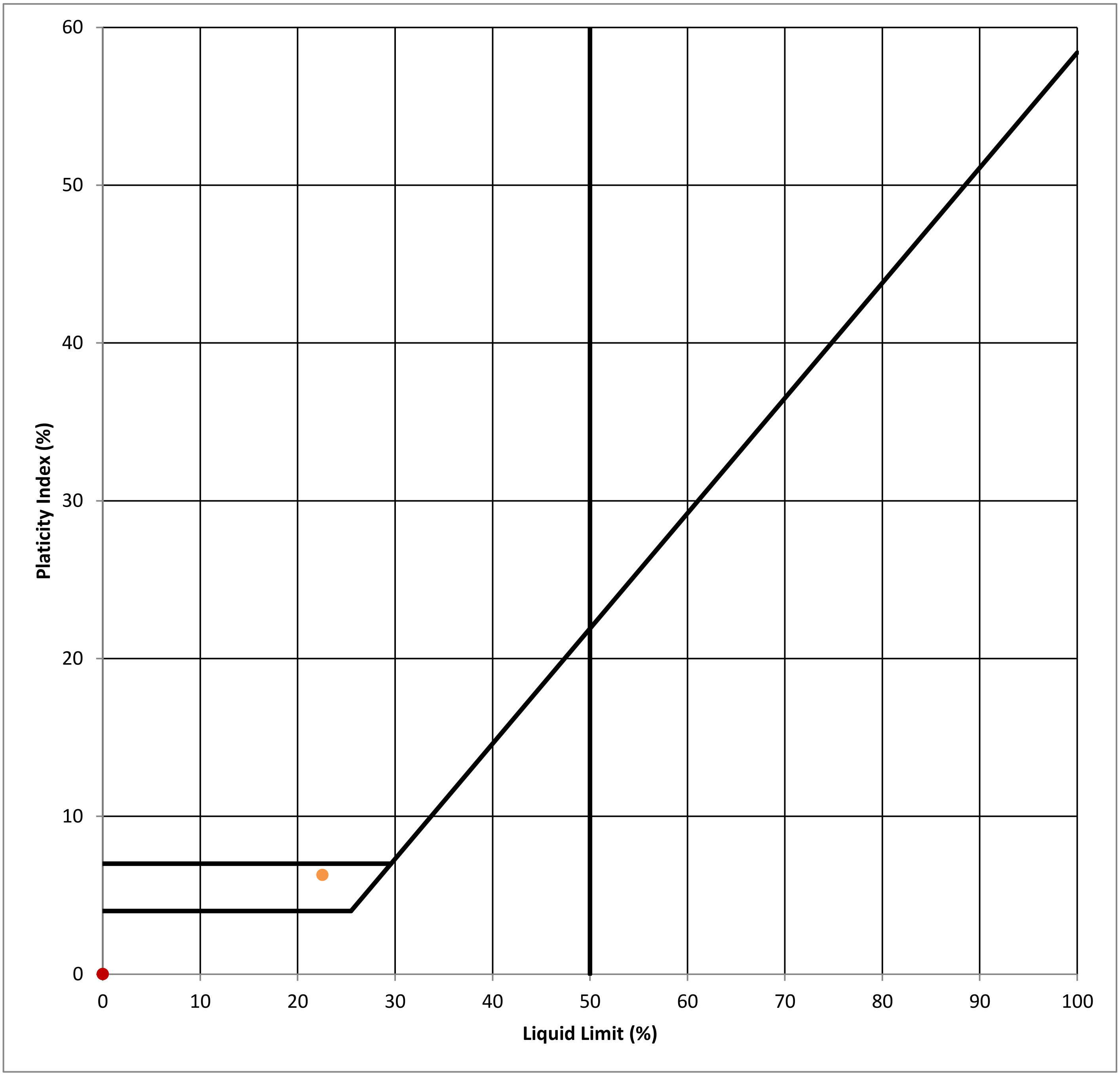
Soil Terms Key

Plate 6

Atterberg Limits



Location	Depth (ft)		Classification	Liquid Limit	PI
B-1	5		Silty SAND	NP	NP
B-1	20		Silty CLAY with sand	23	6
B-1	40		Poorly Graded SAND	NP	NP
B-2	10		Silty SAND	NP	NP
B-2	30		Silty SAND	NP	NP

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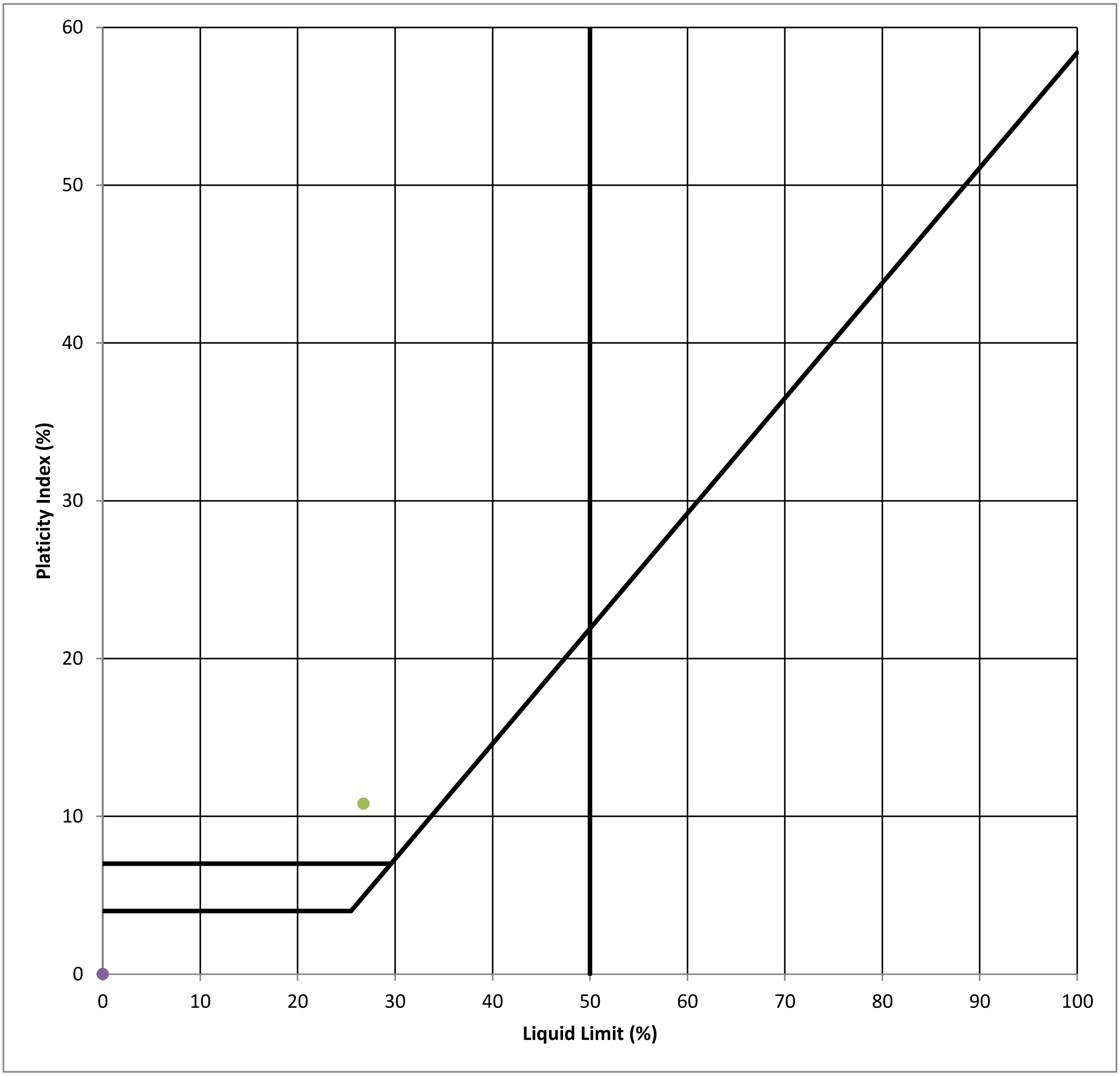
Atterberg Limits



Location	Depth (ft)		Classification	Liquid Limit	PI
B-2	50	●	Silty SAND	NP	NP
B-3	15	●	Poorly Graded SAND with silt	NP	NP
B-3	30	●	Lean CLAY	27	11
B-3	50	●	SILT with sand	NP	NP

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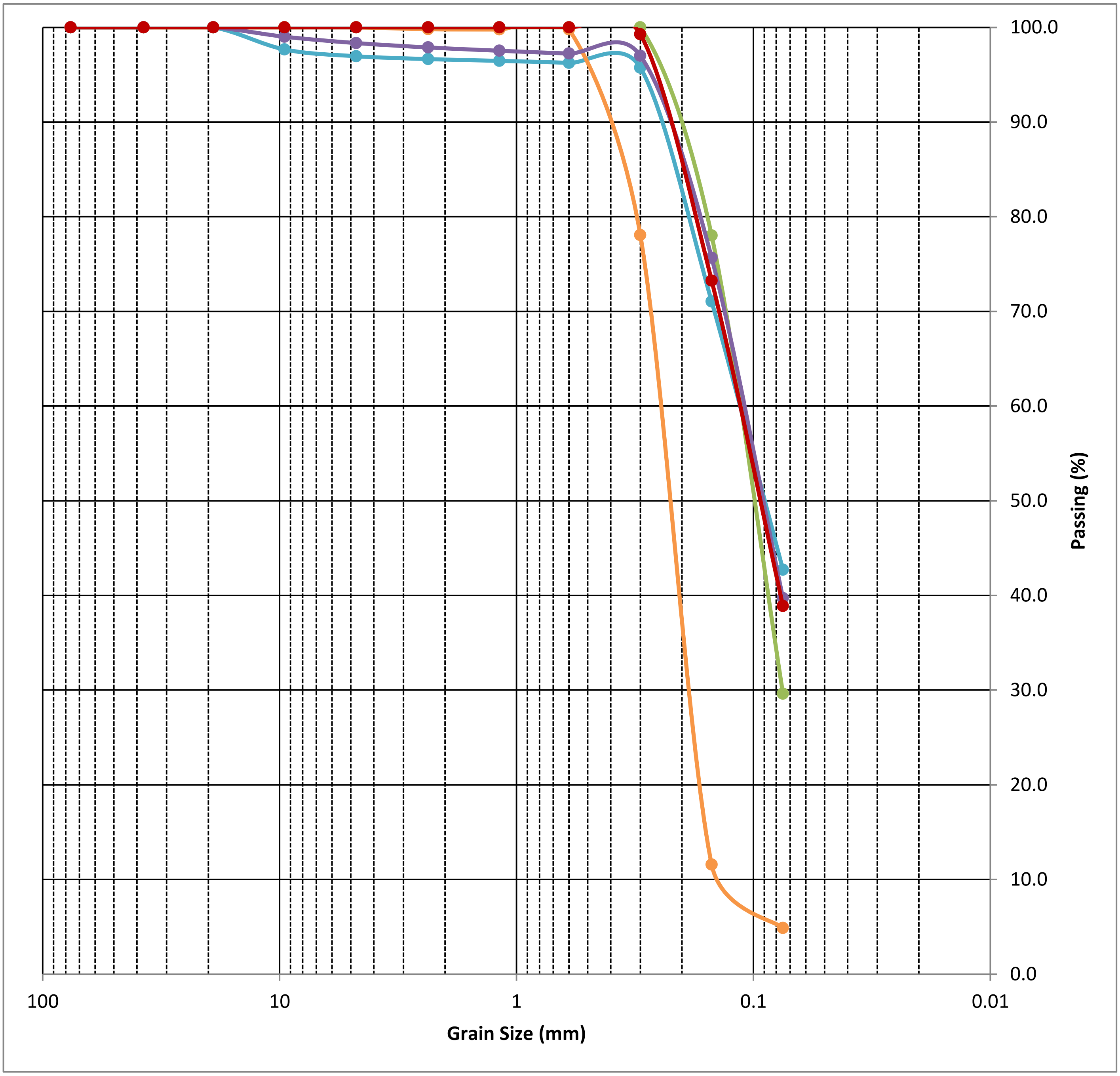
Grain Size Distribution



Location	Depth		Classification	% Gravel	% Sand	% Silt and Clay
B-1	5	●	Silty SAND	3.0	54.2	42.7
B-1	40	●	Poorly Graded SAND	0.0	95.1	4.9
B-2	10	●	Silty SAND	0.0	70.4	29.6
B-2	30	●	Silty SAND	1.7	58.6	39.7
B-2	50	●	Silty SAND	0.0	61.1	38.9

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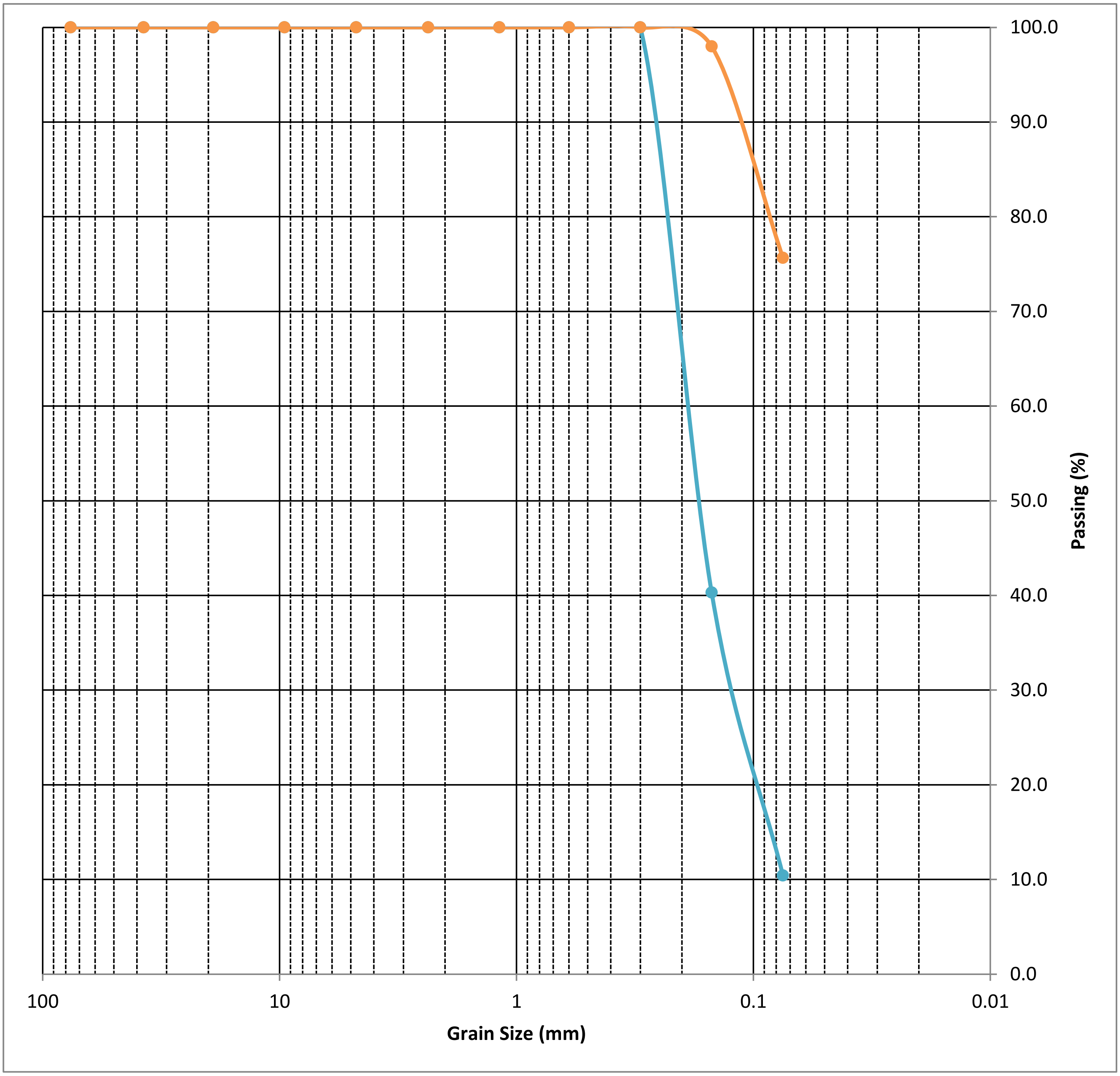
Grain Size Distribution



Location	Depth		Classification	% Gravel	% Sand	% Silt and Clay
B-3	15	●	Poorly Graded SAND with silt	0.0	89.6	10.4
B-3	50	●	SILT with sand	0.0	24.4	75.6

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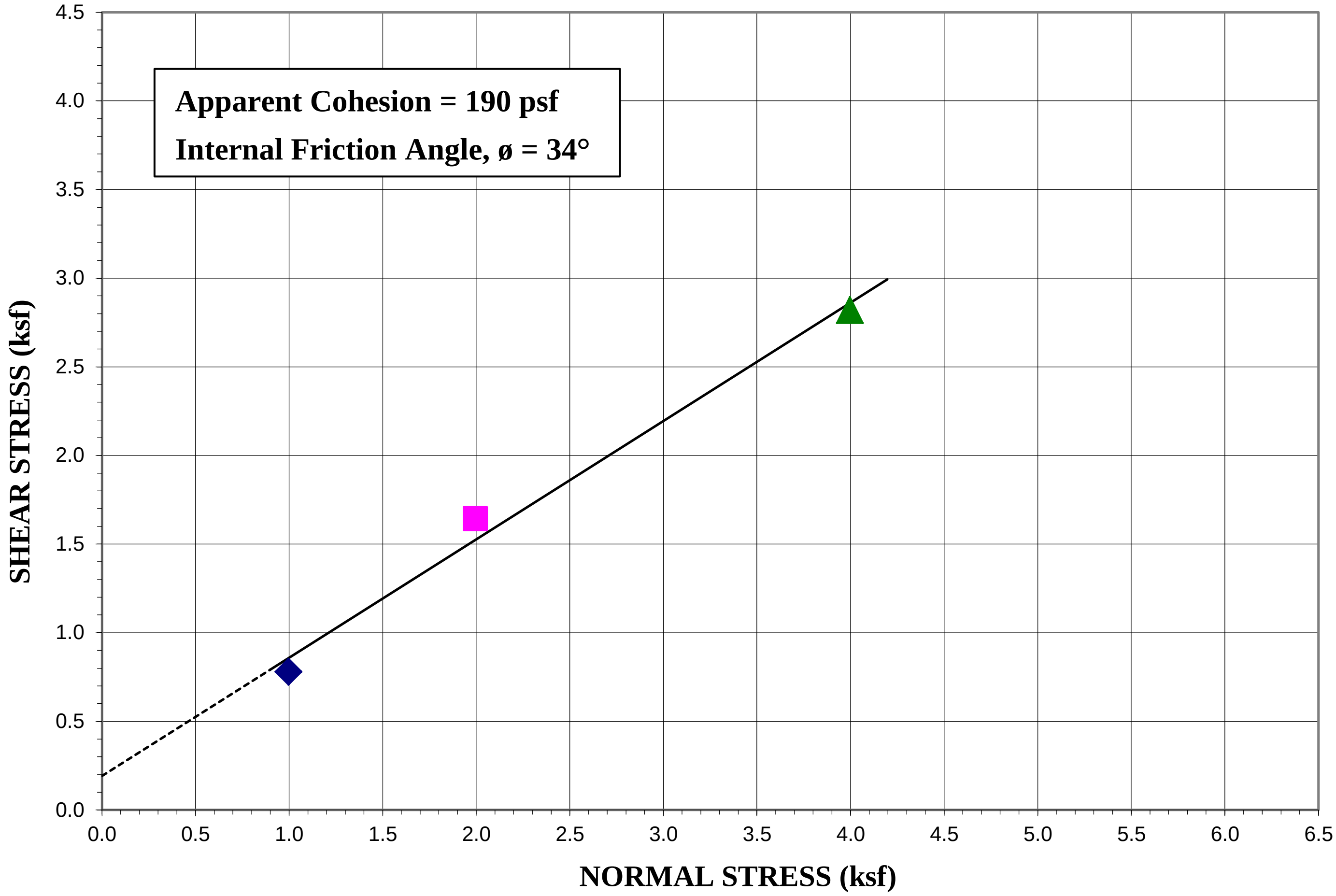

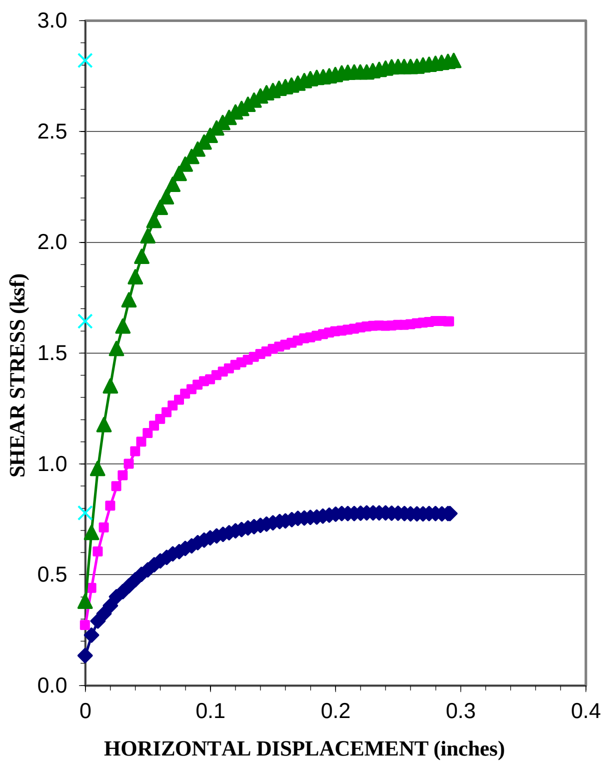

Location:	B-1	**Depth:**	20.0 ft	
Type of Test:		Consolidated Drained/Saturated		
Test No. (Symbol)		**1 (◆)**	**2 (■)**	**3 (▲)**
Sample Type:		Remolded		
Initial Height, in.		1	1	1
Diameter, in.		2.4	2.4	2.4
Dry Density Before, pcf		91.8	96.5	97.9
Moisture % Before		16.1	16.1	16.1
Normal Load, ksf		1.0	2.0	4.0
Shear Stress, ksf		0.78	1.64	2.82
Strain Rate		0.01 in/min		

Sample Properties	
Cohesion, psf	190
Friction Angle, ϕ	34
Liquid Limit, %	23
Plasticity Index, %	6
Percent Gravel	---
Percent Sand	---
Percent Passing No. 200 sieve	78.2
Classification	Silty CLAY (CL-ML)

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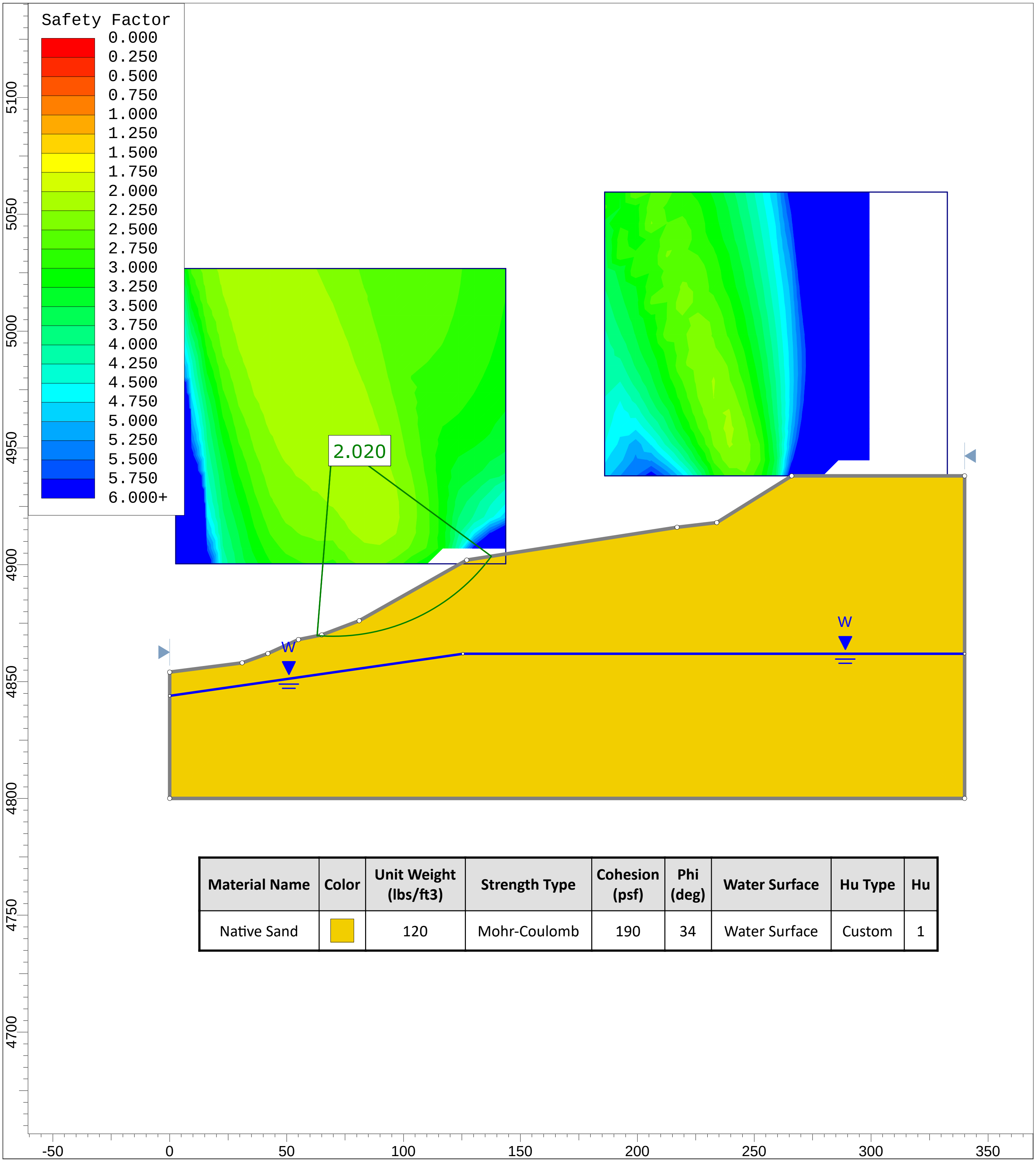

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Hu Type	Hu
Native Sand		120	Mohr-Coulomb	190	34	Water Surface	Custom	1

Profile A - Static

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Plate

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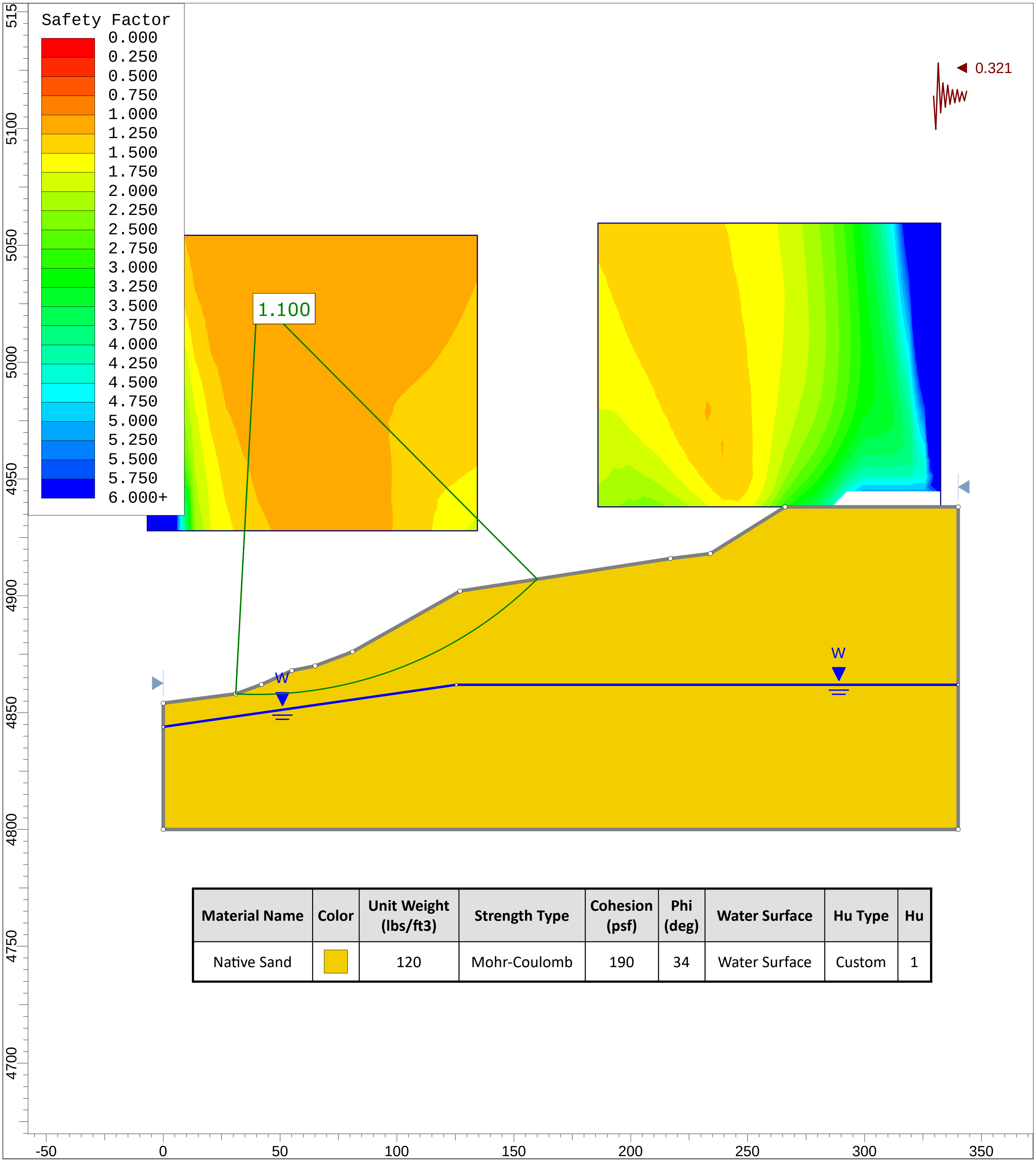

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Hu Type	Hu
Native Sand		120	Mohr-Coulomb	190	34	Water Surface	Custom	1

Profile A - Pseudo Static

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Plate

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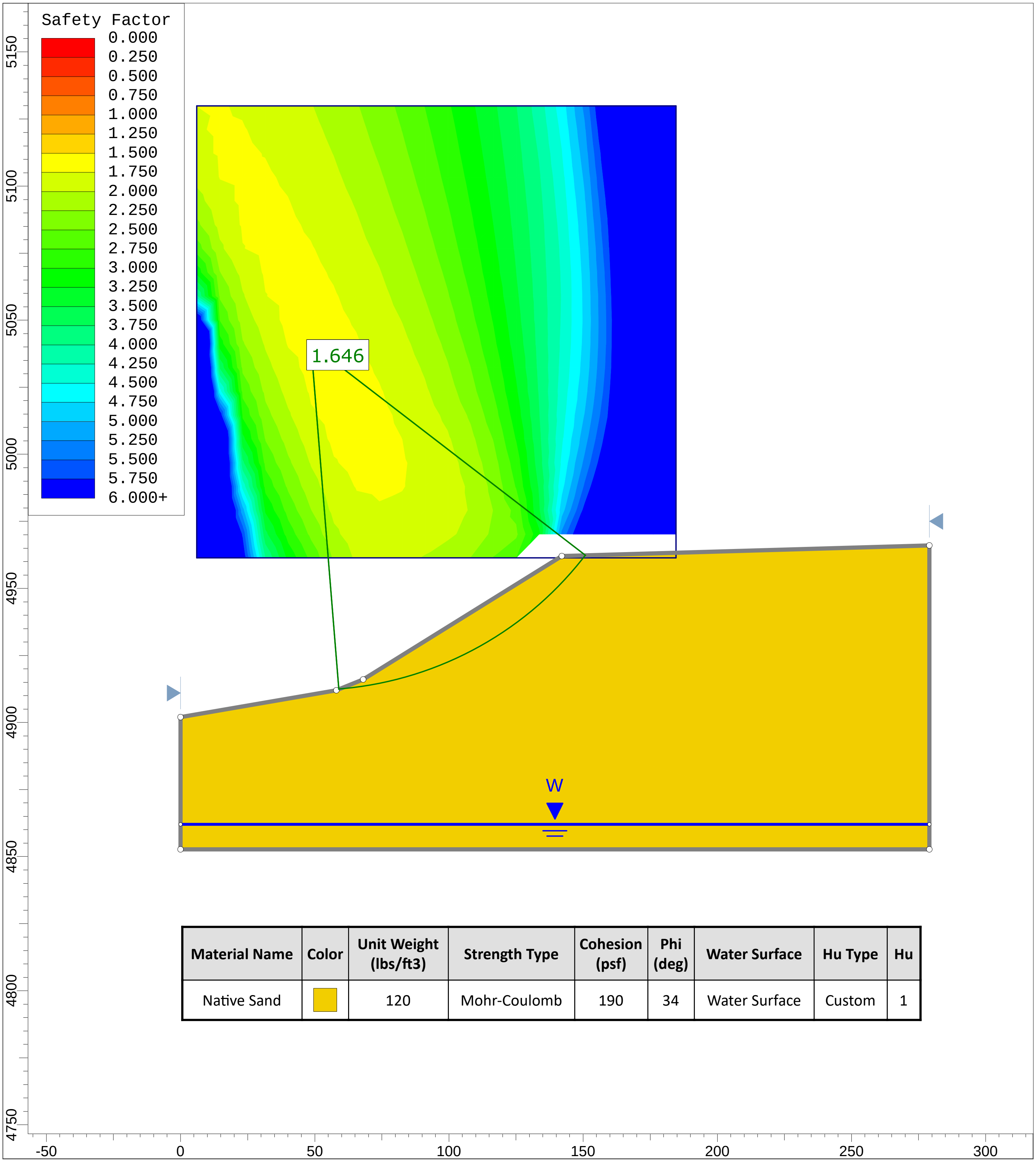

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Hu Type	Hu
Native Sand		120	Mohr-Coulomb	190	34	Water Surface	Custom	1

Profile B - Static

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Plate

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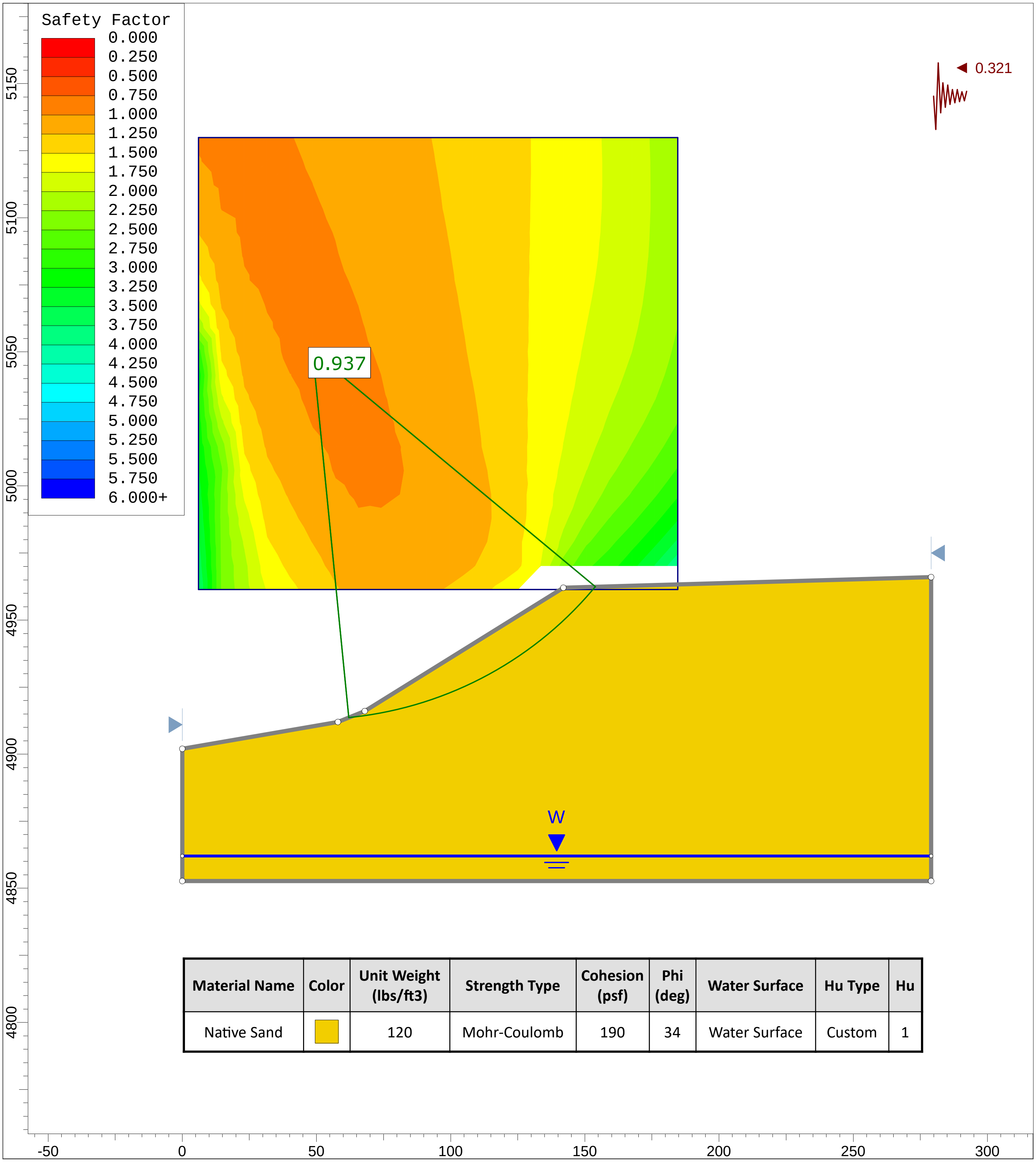

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Hu Type	Hu
Native Sand		120	Mohr-Coulomb	190	34	Water Surface	Custom	1

Profile B - Pseudo Static

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Plate

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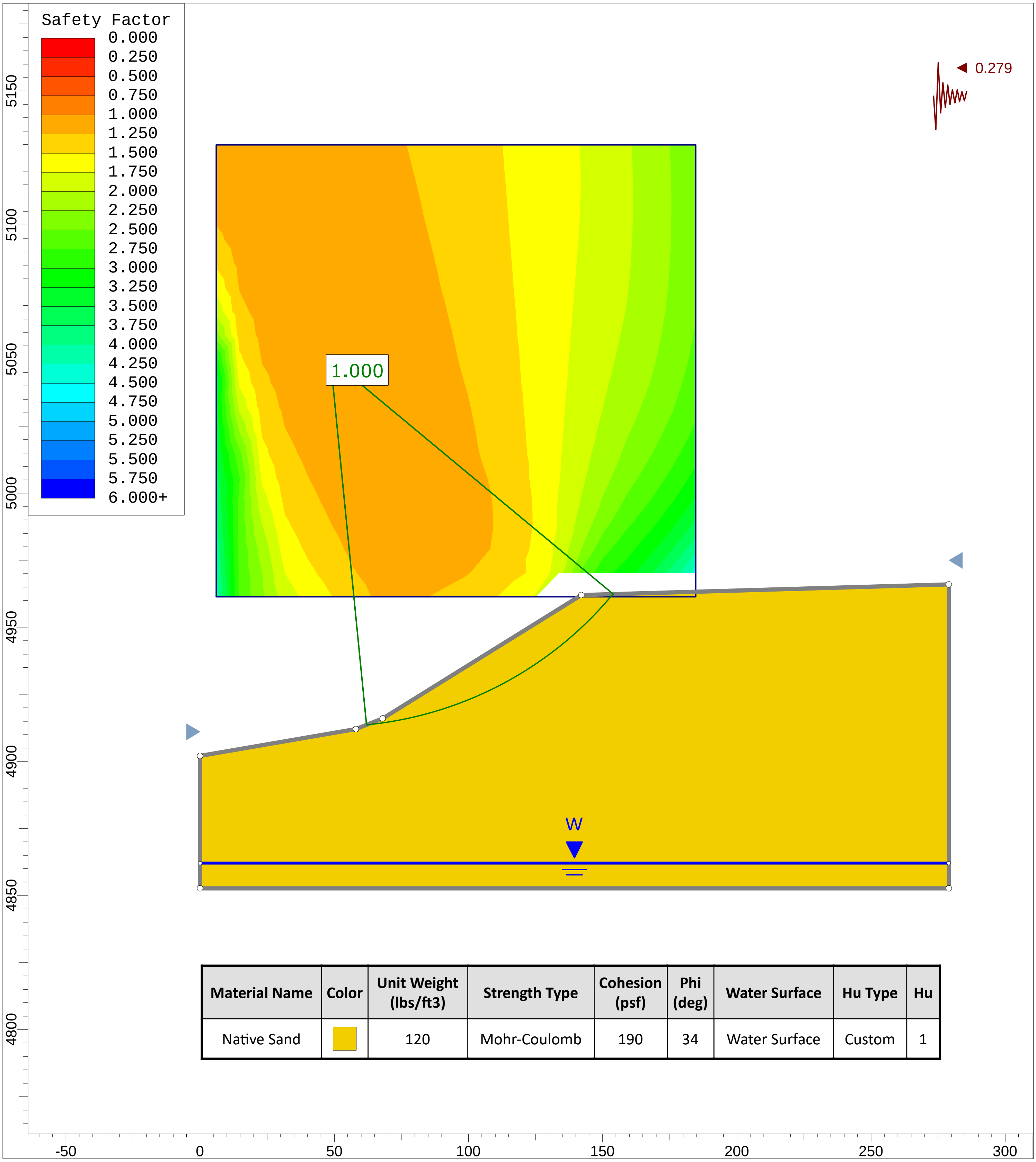

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Hu Type	Hu
Native Sand		120	Mohr-Coulomb	190	34	Water Surface	Custom	1

Profile B - Yield Acceleration

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