Geotechnical Investigation
KHR Holdings Warehouse
Ogden, Utah

July 28, 2022

Prepared by:

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Prepared for:

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Geotechnical Investigation
KHR Holdings Warehouse
Approximately 9550 West 900 South
Ogden, Utah
CG Project No.: 333-001

Prepared by:

7/28/22
LICENSED PROFESSIONAL ENGINEER
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STATE OF UTAH

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July 28, 2022

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1.0 INTRODUCTION

1.1 PURPOSE AND SCOPE OF WORK

This report presents the results of a geotechnical investigation that was performed for a proposed warehouse building which is to be located at 9550 West 900 South in Ogden, 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, pavements, 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. Chad Spencer and was conducted in accordance with the Christensen Geotechnical proposal dated May 17, 2022.

1.2 PROJECT DESCRIPTION

Based on conversations with our client, we understand that the proposed development is to consist of one warehouse building with a footprint of approximately 50,000 square feet. The proposed building is to be a one to two story structure with slab-on-grade floors at or near existing grades. The development of the site will also include parking and access drives. The structural loads for the proposed building 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 six borings with a Mobile B-80 truck-mounted drill rig equipped with hollow-stem augers. The approximate locations of the borings are shown on the Exploration Location Map, Plate 2. The depths of the borings ranged from 21½ to 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 8. A key to the symbols and terms used on the Boring Logs may be found on Plate 9.

Representative disturbed soil samples were collected from the borings through the collection of drill cuttings and through the use of standard split-spoon samplers. Undisturbed samples were obtained through the use of Shelby tubes. 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 included natural moisture content and dry density determinations, Atterberg limits evaluations, gradation analyses, one-dimensional consolidation tests, a moisture-density relationship test, and a California bearing ratio (CBR) test. 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		Grain Size Distribution (%)			CBR	Soil Type
				LL	PI	Gravel (+#4)	Sand	Silt/Clay (- #200)		
B-1	5		27.3	NP	NP	0.0	49.2	50.8		ML
B-1	30		23.8	NP	NP	0.0	56.9	43.1		SM
B-2	2.5		19.1	NP	NP	2.3	82.1	15.6		SM
B-3	5		27.5	NP	NP	0.0	28.1	71.9		ML
B-3	10	44.7	98.7	99	65			95.4		CH
B-4	5		27.2	NP	NP	0.0	71.5	28.5		SM
B-5	2.5		28.0	NP	NP	0.0	37.3	62.7		ML
B-6	7.5	70.2	45.7	127	89			85.6		CH

The results of our laboratory tests are also presented on the Boring Logs, Plates 3 through 8. More detailed laboratory results are presented on the laboratory testing plates, Plates 10 through 16.

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. Portions of the property were used for truck and equipment parking. The property was nearly level. The vegetation at the site consisted of sparse grass and weeds. Properties adjacent to the site consisted of an industrial complex to the north, undeveloped land to the east and west, and 900 South Street to the south.

3.2 SUBSURFACE CONDITIONS

3.2.1 Soils

Based on the six borings that were completed for this investigation, the site is covered with ½ to 1 foot of fill and topsoil. The native soils below the fill and topsoil generally consist of interbedded zones of medium dense/stiff Silty SAND (SM), Sandy SILT (ML), and SILT with sand (ML) to a depth of 6½ to 9 feet below existing site grade. Below 6 ½ to 9 feet, the soils consist of soft Fat CLAY (CL) to a depth of approximately 23 feet. Below 23 feet, the soils generally consist of medium dense Silty SAND (SM) to a depth of 48 feet, where Lean CLAY (CL) soils are present through the maximum depth explored (51½ feet).

3.2.2 Groundwater

Groundwater was encountered within each of our borings at depths of 3 to 6 feet below existing site grade. 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 SURFACE GEOLOGY

The subject site is located within a large valley basin in Weber County, Utah. Geologic mapping of this area indicates that the near-surface geology of the subject site consists of Provo Formation and younger lake bottom sediments. These deposits generally consist of clays, silts, and sands (Fitzhugh, 1985).

4.2 FAULTING

Based upon published data, no active faults are known to traverse the site. The nearest known active fault is associated with the Great Salt Lake Fault, which lies approximately 11.6 miles west of the subject property (UGS).

4.3 SEISMIC DESIGN CRITERIA

The State of Utah and Utah municipalities have adopted the 2018 International Building Code (IBC) for seismic design. The IBC seismic design is based on seismic hazard maps which depict probabilistic ground motions and spectral response; the maps, ground motions, and spectral response having been developed by the United States Geological Survey (USGS). Seismic design values, including the design spectral response, may be calculated for a specific site using the web-based application by the Applied Technology Council (ATC), the project site’s approximate latitude and longitude, and its Site Class. Based on our field exploration, it is our opinion that this location is best described as a Site Class E. However, if rammed aggregate piers are used to support the foundations for the proposed structure, Site Class D (Stiff Soil) may be used for design. The spectral acceleration values obtained from the ATC’s web-based application are shown below.

Table 2: IBC Seismic Response Spectrum Values Site Class D

Site Location: 41.250574^{0} N -112.208235^{0} W	
Name	**Response Spectral Value**
S_S	**0.85**
S_1	**0.305**
S_{MS}	**0.986**
S_{M1}	**See ASCE Section 11.4.8**
S_{DS}	**0.657**
S_{D1}	**See ASCE Section 11.4.8**
PGA	**0.367**
PGA_M	**0.453**

4.4 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 high potential for liquefaction. A high potential for liquefaction indicates that there is a 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 CONCLUSIONS

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, pavement, 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, ½ to 1 foot of fill and topsoil cover the subject site. These soils 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

Once exposed through excavation, all subgrade soils should be proof rolled with a relatively large, wheeled vehicle to a firm, unyielding condition. Due to the high groundwater at the site, soft soils are likely to be encountered. Where encountered, these localized soft areas 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 that is placed for the support of structures, concrete flatwork and pavements should consist of structural fill. The structural fill may consist of the native sand and soils. Due to their high plasticity, 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 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.

5.3 FOUNDATIONS

The foundations for the planned structures may consist of conventional continuous and/or spread footings. Due to the soft, compressible nature of the clay soils encountered at the site, we recommend that the spread footings be founded on rammed aggregate piers. The footings for the proposed structures should be a minimum of 20 inches and 30 inches wide for continuous and spot footings, respectively. The exterior footings should be established 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.

The rammed aggregate piers should be designed by experienced personnel and constructed by a qualified contractor. The bearing capacity of the rammed aggregate piers should be assessed by the pier designer.

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	0.30	34
Active Seismic	0.18	21
At Rest	0.50	58
Passive Static	3.00	345
Passive Seismic	-1.16	-134

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 soils, we recommend that 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. The passive earth pressure from soils subject to frost or heave should usually be neglected in design.

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

5.6 CONCRETE SLAB-ON-GRADE CONSTRUCTION

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

5.7 MOISTURE PROTECTION AND SURFACE DRAINAGE

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

1. The ground surface should be graded to drain away from the structures in all directions, with a minimum fall of 8 inches in the first 10 feet.
2. Roof runoff should be collected in rain gutters with downspouts that are designed to discharge well outside of the backfill limits.

3. Sprinkler heads should be aimed away from and placed at least 12 inches from foundation walls.
4. There should be adequate compaction of backfill around foundation walls, to a minimum of 90% density (ASTM D 1557). Water consolidation methods should not be used.

5.8 PAVEMENT DESIGN

Pavement sections for the proposed warehouse were assessed using the PAS computer program (prepared by the American Concrete Pavement Association) and a laboratory obtained CBR value of 4.9 percent. It is our understanding that traffic will consist of 50 passenger cars per day, 4 medium trucks per day and 2 heavy trucks per day. We have assumed no increase in traffic over the life of the pavement. Based on this information, we recommend a pavement section consisting of 3 inches of asphalt over 10 inches of untreated base. As an alternative, a pavement section of 3 inches of asphalt, 6 inches of untreated base, and 6 inches of granular borrow may be used. The asphalt should consist of a high-stability plant mix and should be compacted to at least 96 percent of the Marshall maximum density. The untreated base should meet the material requirements for Plain City or UDOT. The granular borrow should meet the recommendations for imported structural fill as presented in Section 5.2.4 of this report. The untreated base and granular borrow should be compacted to at least 95 percent of the maximum dry density as determined by ASTM D 1557.

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

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.

Fitzhugh, Davis D., 1985, “Geologic Map of the Northern Wasatch Front, Utah,” Utah Geological Survey, Map 53-A.

UGS, Utah Quaternary Fault and Fold Database, interactive web-based map.

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

☐ **Approximate Project Boundary**

Drawing Not to Scale

N

Christensen Geotechnical	KHR Holdings, LLC KHR Holdings Warehouse Ogden, Utah Project No. 333-001 **Vicinity Map**	Plate 1

Base Photo: Utah AGRC

Drawing Not to Scale

Approximate Test Pit Location

Approximate Project Boundary

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KHR Holdings, LLC
KHR Holdings Warehouse
Ogden, Utah
Project No. 333-001

Exploration Location Map

Plate
2

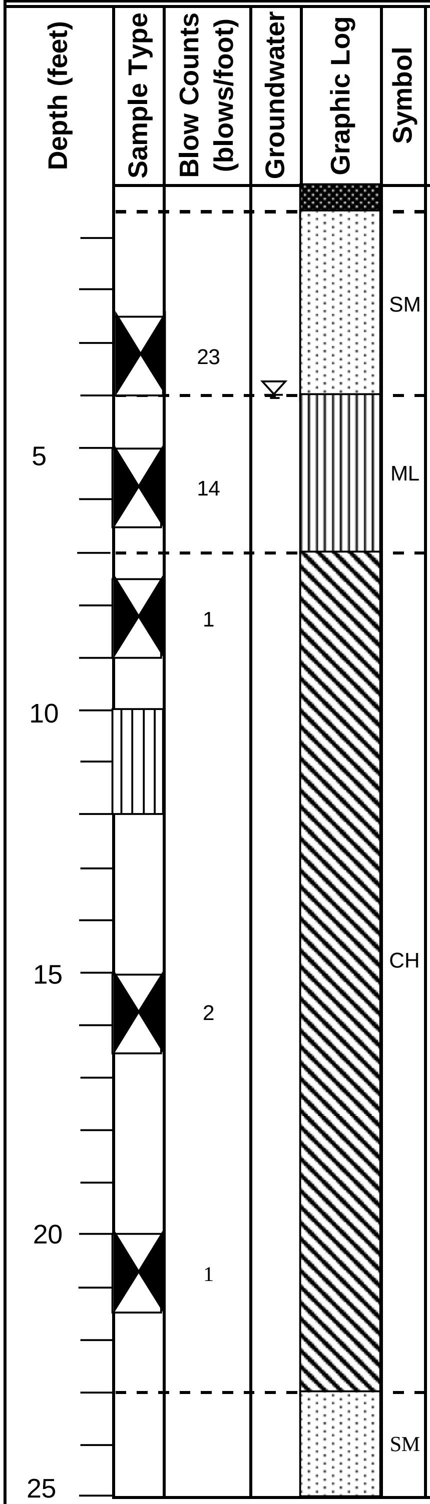
Date		
Started:	6/21/2022	
Completed:	6/21/2022	
Backfilled:	6/21/2022	

BORING LOG

Sheet 1 of 2

Logged By: M Christensen
Equipment: Mobile B-80
Location: See Plate 2

Boring No.
B-1



Symbol	Material Description	Dry Density (pcf)	Moisture Content (%)	Minus #200 (%)	Liquid Limit	Plasticity Index
	Topsoil; Silty SAND - moist, dark brown					
SM	Silty SAND - medium dense, moist, brown					
ML	Sandy SILT - stiff, wet, brown		27.3	50.8	NP	NP
CH	Fat CLAY - soft, wet, gray					
SM	Silty SAND - medium dense, wet, gray					

☒ SPT Sampler
◩ California Sampler
▥ Shelby Tube
☒ Bulk/Bag Sample
▼ Stablized Groundwater
▽ Groundwater at time of Drilling

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Ogden, Utah
Project No.: 333-001

Plate
3a

BORING LOG B-1 Continued

Sheet 2 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
	⧗	24		░		(continued) Silty SAND - medium dense, wet, gray					
30	⧗	21		░				23.8	43.1	NP	NP
35	⧗	22		░	SM						
40	⧗	24		░							
45	⧗	33		░							
50	⧗	9		▨	CL						
						Bottom of boring at 51½ feet					

⧗ SPT Sampler

◪ California Sampler

▥ helby Tube

☒ Bulk/Bag Sample

▼ Stabllized Groundwater

▽ Groundwater at time of Drilling

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Ogden, Utah
Project No.: 333-001

Plate

3b

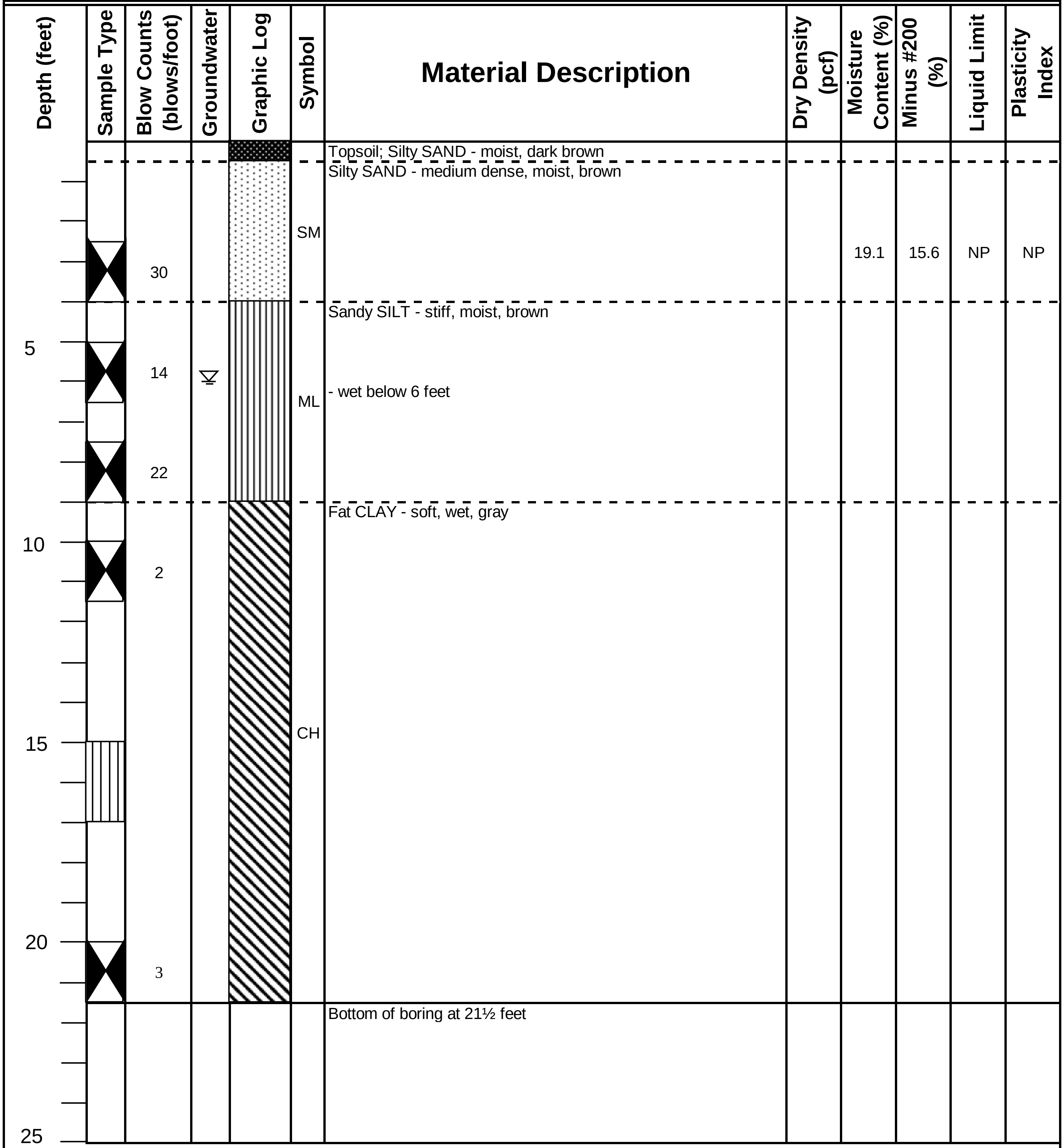
Date		
Started:	6/21/2022	
Completed:	6/21/2022	
Backfilled:	6/21/2022	

BORING LOG

Sheet 1 of 1

Logged By: M Christensen
Equipment: Mobile B-80
Location: See Plate 2

Boring No.
B-2



Sample Type	Blow Counts (blows/foot)	Groundwater	Symbol	Material Description	Dry Density (pcf)	Moisture Content (%)	Minus #200 (%)	Liquid Limit	Plasticity Index
				Topsoil; Silty SAND - moist, dark brown					
			SM	Silty SAND - medium dense, moist, brown					
⧗	30					19.1	15.6	NP	NP
			ML	Sandy SILT - stiff, moist, brown					
⧗	14	▽		- wet below 6 feet					
⧗	22								
			CH	Fat CLAY - soft, wet, gray					
⧗	2								
▥									
⧗	3								
				Bottom of boring at 21½ feet					

⧗ SPT Sampler
◩ California Sampler
▥helby Tube
☒Bulk/Bag Sample
▼ Stabllized Groundwater
▽ Groundwater at time of Drilling

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Plate

4

Date		
	Started:	6/21/2022
	Completed:	6/21/2022
	Backfilled:	6/21/2022

BORING LOG

Sheet 1 of 1

Logged By: M Christensen
Equipment: Mobile B-80
Location: See Plate 2

Boring No.
B-3

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
				▓		Topsoil; Silty SAND - moist, dark brown					
				░	SM	Silty SAND - medium dense, moist, brown					
	⧗	22	▽	░		- wet below 3 feet					
				▥	ML	SILT with sand - stiff, moist, brown		27.5	71.9	NP	NP
5	⧗	15		▥		- gray below 6 feet					
	⧗	2		▨		Fat CLAY - soft, wet, gray					
10	▥			▨			44.7	98.7	95.4	99	65
				▨	CH						
15	⧗	2		▨							
20	⧗	10		▨							
						Bottom of boring at 21½ feet					
25											

⧗ SPT Sampler
◩ California Sampler
▥helby Tube
⊠Bulk/Bag Sample
▼ Stabllized Groundwater
▽ Groundwater at time of Drilling

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Plate

5

Date		
Started:	6/21/2022	
Completed:	6/21/2022	
Backfilled:	6/21/2022	

BORING LOG

Sheet 1 of 1

Logged By: M Christensen
Equipment: Mobile B-80
Location: See Plate 2

Boring No.

B-4

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
				▩		Fill; Silty SAND - moist, brown					
				░		Silty SAND - medium dense, moist, brown					
	⧗	15		░							
			▽	░	SM	- wet below 4 feet					
5	⧗	20		░				27.2	28.5	NP	NP
				▧		Fat CLAY - soft, wet, gray					
	⧗	2		▧							
10	⧗	3		▧							
				▧	CH						
15	⧗	2		▧							
20	▥			▧							
						Bottom of boring at 22 feet					
25											

⧗ SPT Sampler
◩ California Sampler
▥helby Tube
☒Bulk/Bag Sample
▼ StabIlized Groundwater
▽ Groundwater at time of Drilling

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Ogden, Utah
Project No.: 333-001

Plate

6

Date					
Started:	6/21/2022	BORING LOG	Logged By: M Christensen	Boring No.	
Completed:	6/21/2022		Equipment: Mobile B-80	B-5	
Backfilled:	6/21/2022	Sheet 1 of 1	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
				▩		Topsoil; Silty SAND - moist, brown					
	⊠			▥		Sandy SILT - stiff, moist, brown					
	⧗	15	▽	▥	ML	- wet below 3 feet		28.0	62.7	NP	NP
5	⧗	16		▥							
	▥			▧		Fat CLAY - soft, wet, gray					
10	⧗	2		▧							
15	⧗	3		▧	CH						
20	⧗	2		▧							
						Bottom of boring at 21½ feet					
25											

⧗ SPT Sampler

◩ California Sampler

▥helby Tube

⊠Bulk/Bag Sample

▼ Stabllized Groundwater

▽ Groundwater at time of Drilling

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Date		
Started:	6/21/2022	
Completed:	6/21/2022	
Backfilled:	6/21/2022	

BORING LOG

Sheet 1 of 1

Logged By: M Christensen
Equipment: Mobile B-80
Location: See Plate 2

Boring No.
B-6

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
				░	SM	Silty SAND - medium dense, moist, brown					
	☒	15	▽	░		- wet below 3 feet					
5	☒	14		░		- gray below 6 feet					
	▥			▧	CH	Fat CLAY - soft, wet, gray	70.2	45.7	85.6	127	89
10	☒	2		▧							
15	☒	3		▧							
20	☒	2		▧							
						Bottom of boring at 21½ feet					
25											

☒ SPT Sampler
◩ California Sampler
▥ Shelby Tube
☒ Bulk/Bag Sample
▼ Stablilized Groundwater
▽ Groundwater at time of Drilling

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RELATIVE DENSITY – COURSE GRAINED SOILS

Relative Density	SPT (blows/ft.)	3 In OD California Sampler (blows/ft.)	Relative Density (%)	Field Test
Very Loose	<4	<5	0 – 15	Easily penetrated with a ½ inch steel rod pushed by hand
Loose	4 – 10	5 – 15	15 – 35	Difficult to penetrate with a ½ inch steel rod pushed by hand
Medium Dense	10 – 30	15 – 40	35 – 65	Easily penetrated 1-foot with a steel rod driven by a 5 pound hammer
Dense	30 – 50	40 – 70	65 – 85	Difficult to penetrate 1-foot with a steel rod driven by a 5 pound hammer
Very Dese	>50	>70	85 - 100	Penetrate only a few inches with a steel rod driven by a 5 pound hammer

CONSISTENCY – FINE GRAINED SOILS

Consistency	SPT (blows/ft)	Torvane Undrained Shear Strength (tsf)	Pocket Penetrometer Undrained Shear Strength (tsf)	Field Test
Very Soft	<2	<0.125	<0.25	Easily penetrated several inches with thumb
Soft	2 – 14	0.125 – 0.25	0.25 – 0.5	Easily penetrated one inch with thumb
Medium Stiff	4 – 8	0.25 – 0.5	0.5 – 1.0	Penetrated over ½ inch by thumb with moderate effort. Molded by strong finger pressure
Stiff	8 – 15	0.5 – 1.0	1.0 – 2.0	Indented ½ inch by thumb with great effort
Very Stiff	15 – 30	1.0 – 2.0	2.0 – 4.0	Readily indented with thumbnail
Hard	>30	>2.0	>4.0	Indented with difficulty with thumbnail

CEMENTATION

Weakly	Crumbles or breaks with handling or little finger pressure
Moderately	Crumbles or breaks with considerable finger pressure
Strongly	Will not crumble or break with finger pressure

MOISTURE

Dry	Absence of moisture, dusty, dry to the touch
Moist	Damp but no visible water
Wet	Visible water, usually below water table

GRAIN SIZE

Description		Sieve Size	Grain Size (in)	Approximate Size
Boulders		>12"	>12"	Larger than basketball
Cobbles		3" – 12"	3" – 12"	Fist to basketball
Gravel	Coarse	3/4" - 3"	3/4" - 3"	Thumb to fist
	Fine	#4 – 3"	0.19 – 0.75	Pea to thumb
Sand	Coarse	#10 - #4	0.079 – 0.19	Rock salt to pea
	Medium	#40 - #10	0.017 – 0.079	Sugar to rock salt
	Fine	#200 - #40	0.0029 – 0.017	Flour to sugar
Silt/Clay		<#200	<0.0029	Flour sized or smaller

STRATAFICATION

Occasional	One or less per foot of thickness
Frequent	More than one per foot of thickness

MODIFIERS

Trace	<5%
Some	5-12%
With	>12%

STRATIFICATION

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

NOTES

1. The logs are subject to the limitations and conclusions presented in the report.
2. Lines separating strata represent approximate boundaries only. Actual transitions may be gradual.
3. Logs represent the soil conditions at the points explored at the time of our investigation.
4. Soils classifications shown on logs are based on visual methods . Actual designations (based on laboratory testing)may vary.

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Soil Terms Key

Plate
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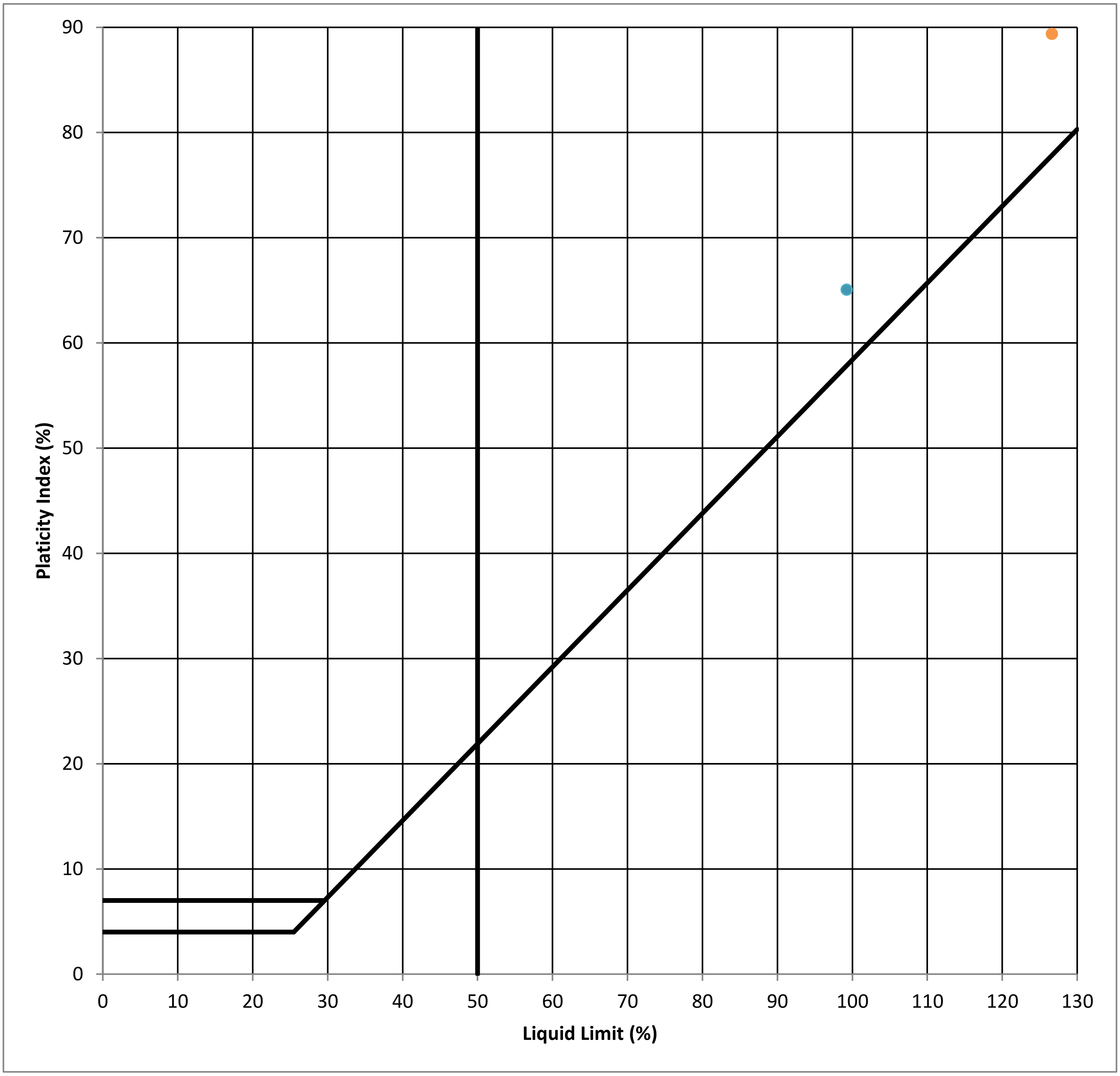
Atterberg Limits



Location	Depth (ft)		Classification	Liquid Limit	PI
B-3	10	●	Fat CLAY	99	65
B-6	7.5	●	Fat CLAY	127	89

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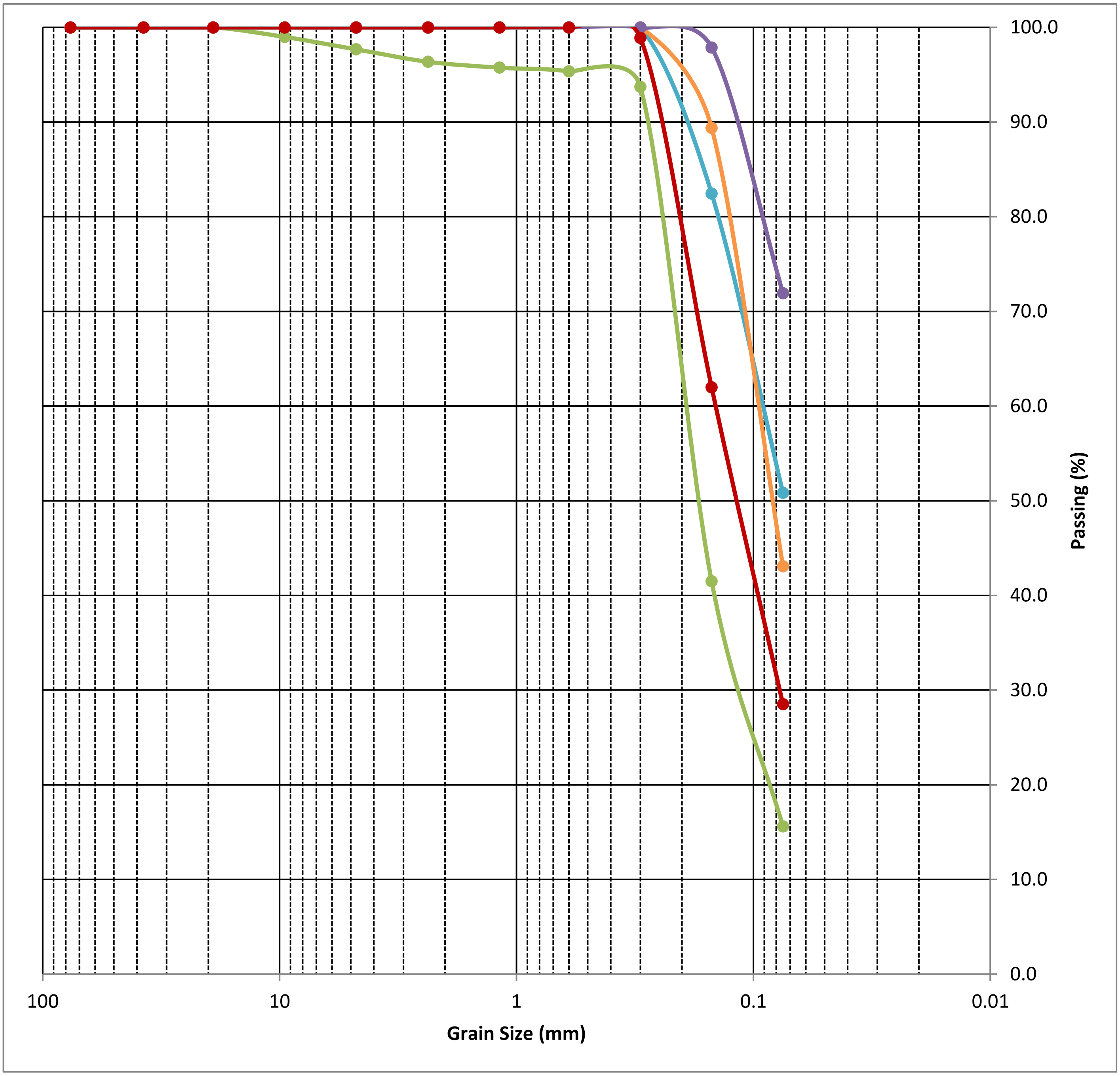
Grain Size Distribution



Location	Depth		Classification	% Gravel	% Sand	% Silt and Clay
B-1	5	●	Sandy SILT	0.0	49.2	50.8
B-1	30	●	Silty SAND	0.0	56.9	43.1
B-2	2.5	●	Silty SAND	2.3	82.1	15.6
B-3	5	●	SILT with sand	0.0	28.1	71.9
B-4	5	●	Silty SAND	0.0	71.5	28.5

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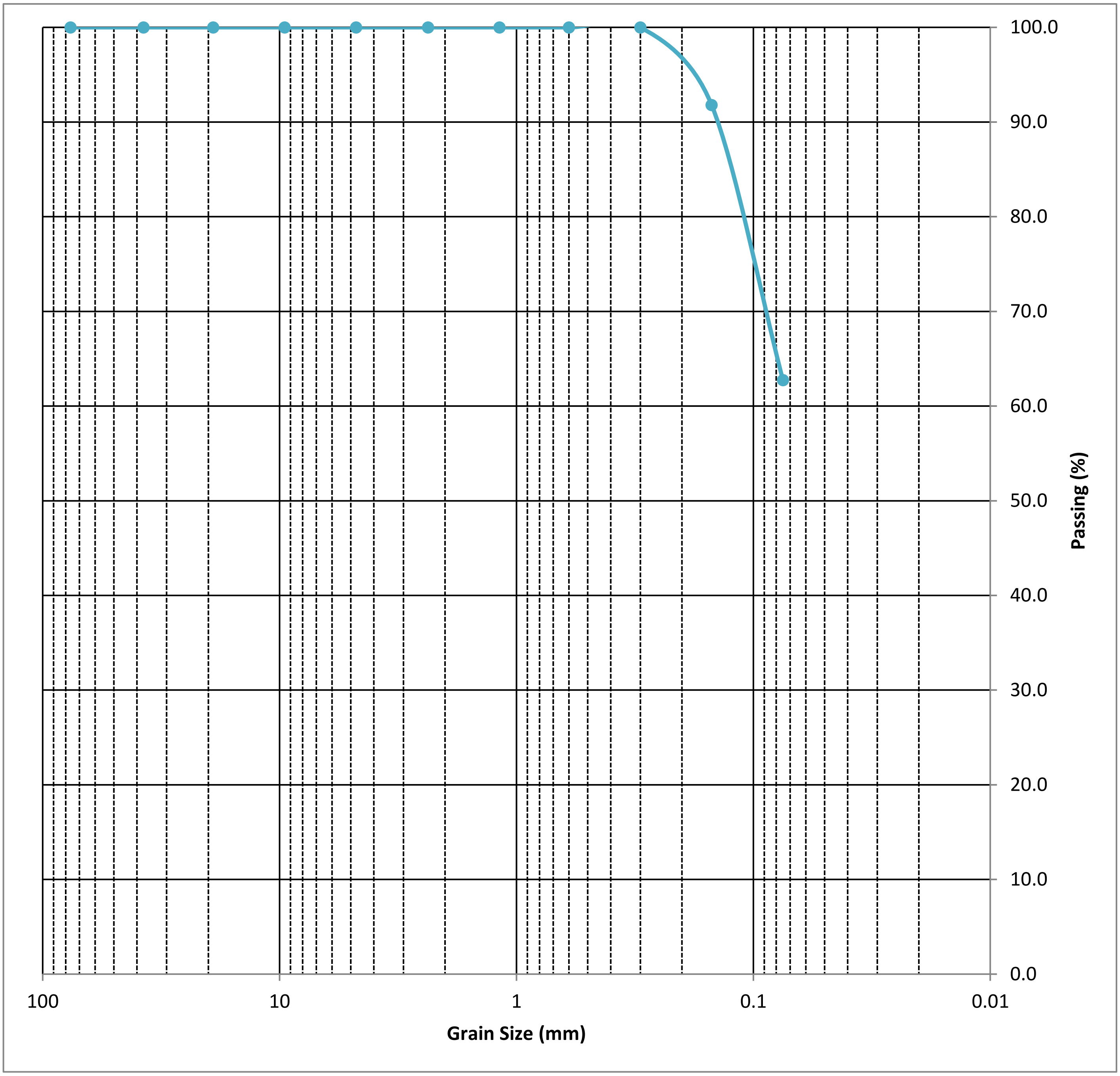
Grain Size Distribution



Location	Depth		Classification	% Gravel	% Sand	% Silt and Clay
B-5	2.5	●	Sandy SILT	0.0	37.3	62.7

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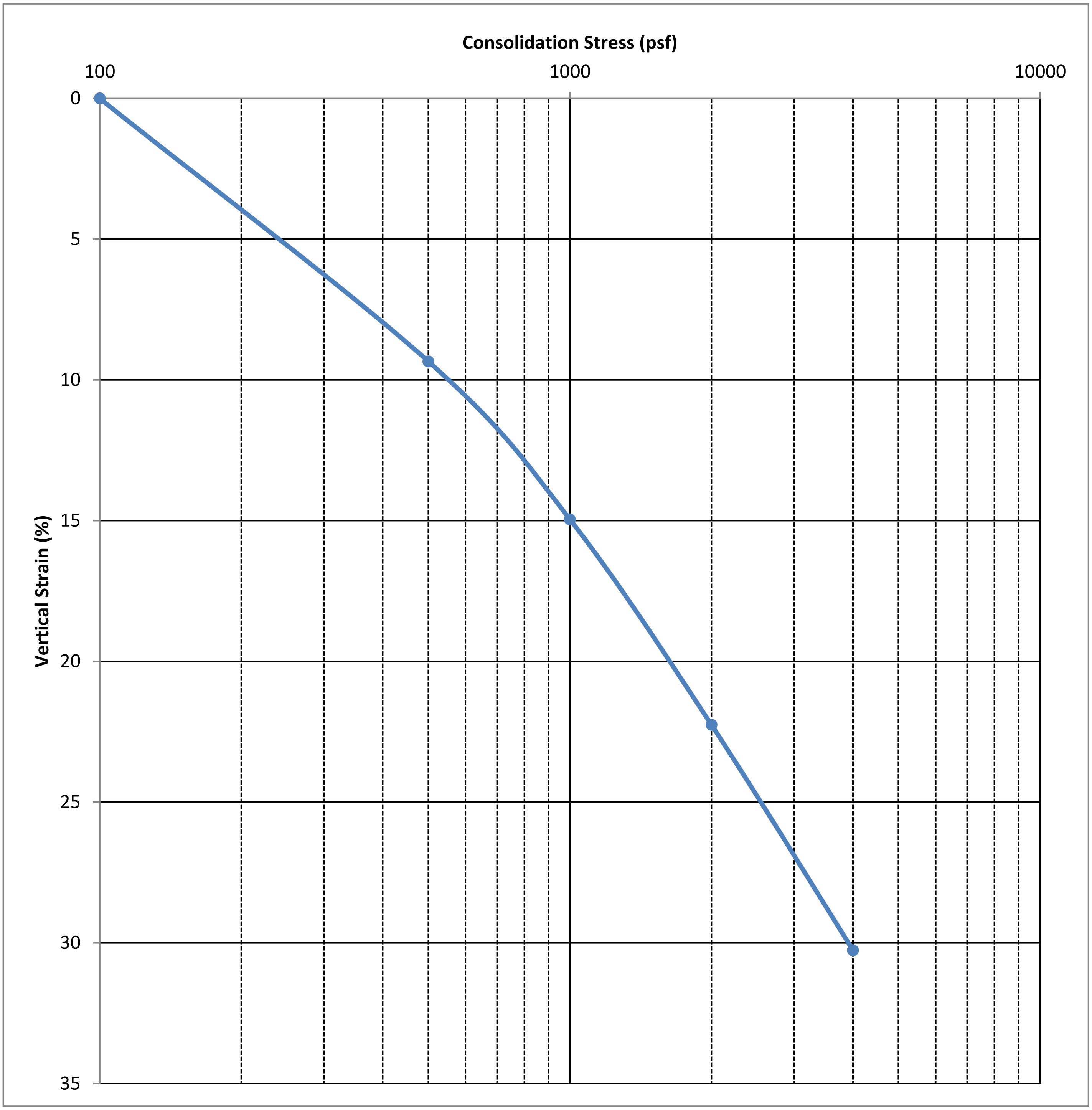
1-D Consolidation



Location	Depth (ft)	Dry Density (pcf)	Moisture Content (%)	σ_o (psf)	σ_p (psf)	C_c	C_r	OCR
B-3	10	44.7	98.7	500	500	0.266	---	1.0

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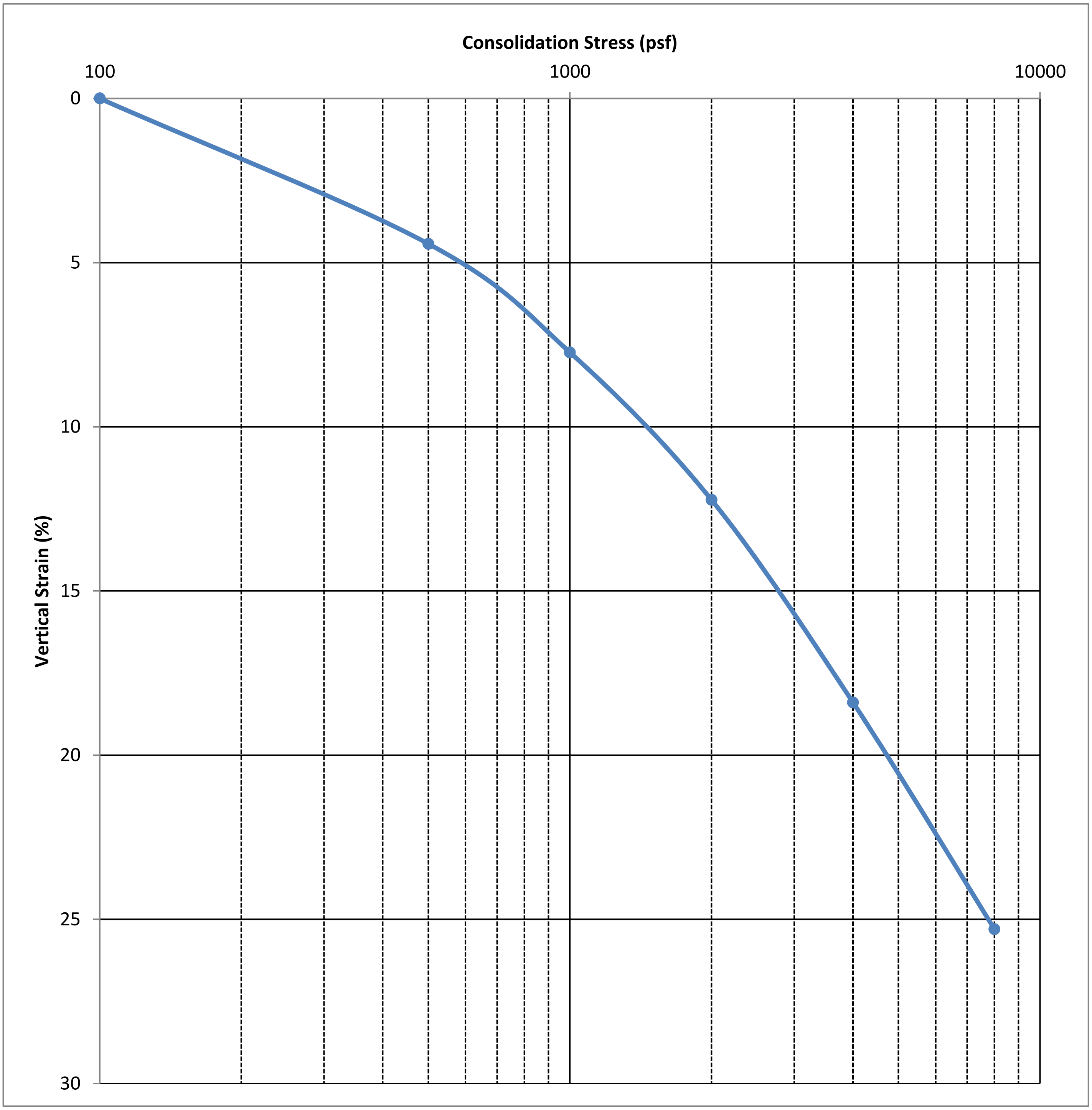
1-D Consolidation



Location	Depth (ft)	Dry Density (pcf)	Moisture Content (%)	σ_o (psf)	σ_p (psf)	C_c	C_r	OCR
B-6	7.5	70.2	45.7	500	500	0.230	---	1.0

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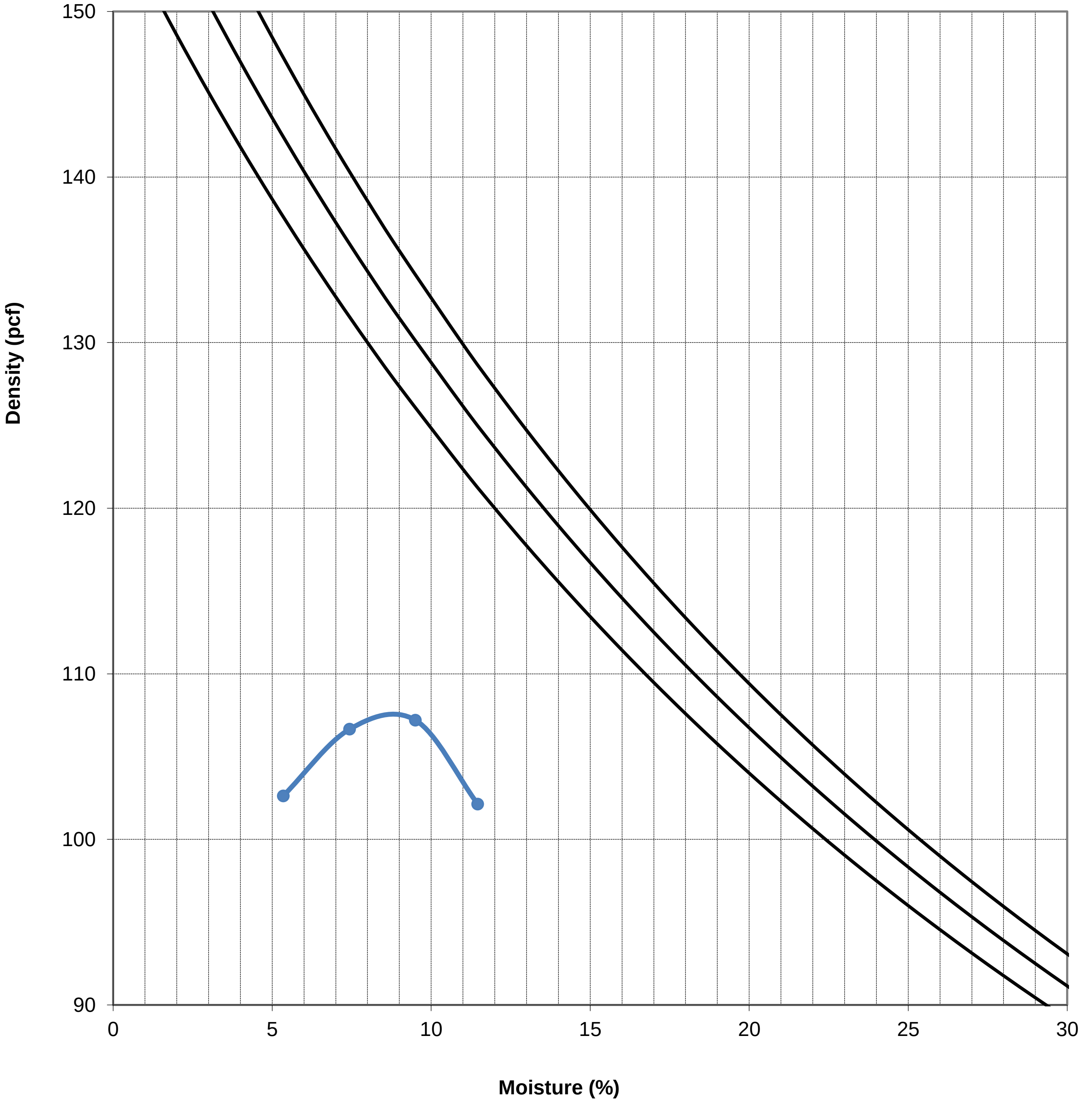
Moisture-Density Relationship



Location	Depth (ft)	Method	Maximum Density (psf)	Optimum Moisture (%)	Soil Type
B-5	0.5	ASTM D698	107.6	9	ML

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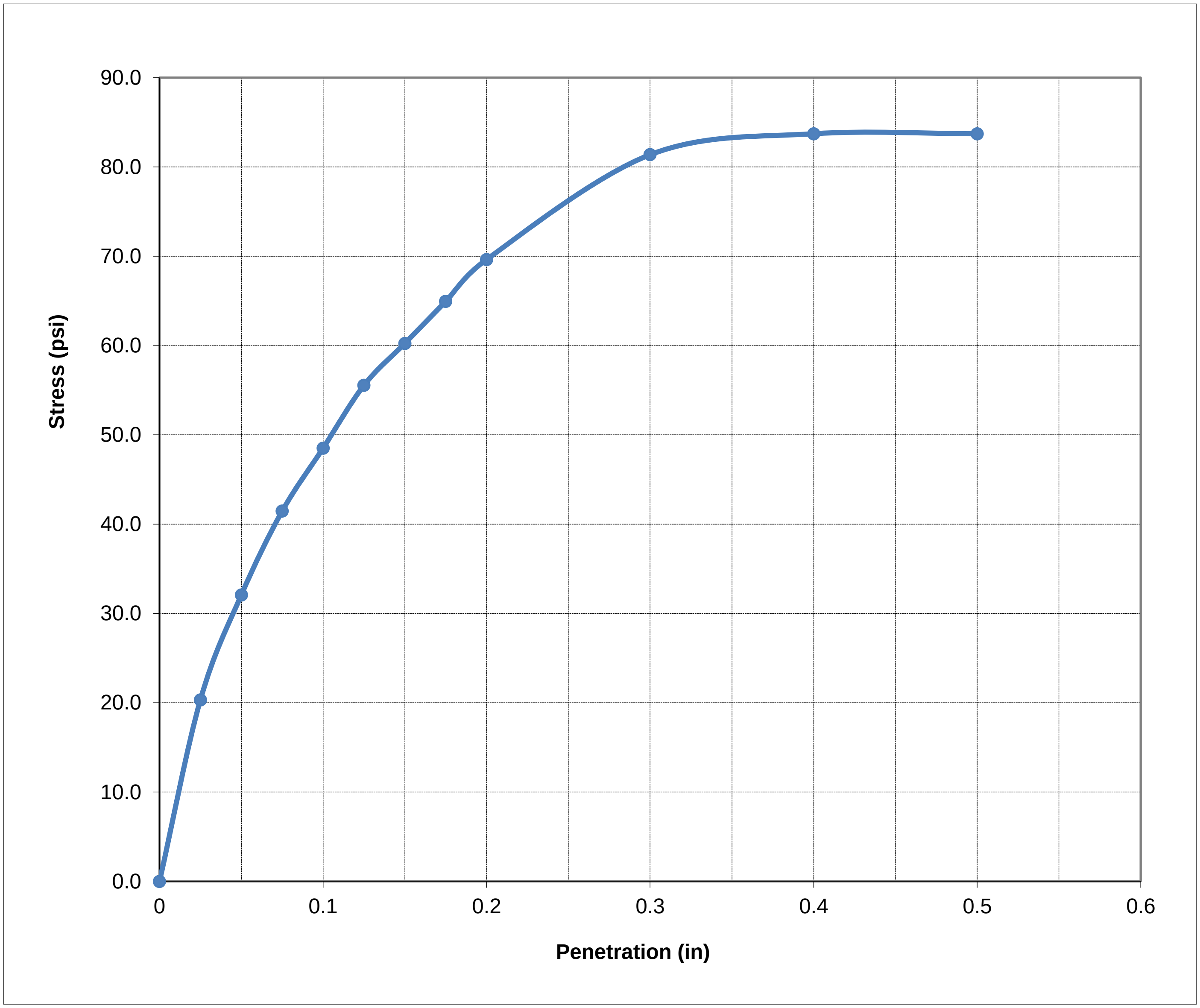
California Bearing Ratio



Location	**B-5**
Depth (ft)	**0.5**
Method used for Preparation and Compaction	**ASTM D698**
Maximum Dry Density (pcf)	**107.6**
Optimum Moisture Content (%)	**9.0**
Dry Density of sample before soaking (pcf)	**107.3**
Dry Density of sample after soaking (pcf)	**107.5**
Moisture Content as compacted (%)	**8.7**
Moisture content top 1 inch after soaking (%)	**14.5**
Average Moisture Content after soaking (%)	**12.6**
Sucharge (psf)	**10**
Swell (%)	**0.2**
Bearing Ratio of Sample (%)	**4.9**

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