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Geotechnical Investigation
Phase I – Mountainside and Parkside Communities
The Bridges at Wolf Creek Development
Eden, Utah

GeoStrata Job No. 1311-002

February 15, 2018

Prepared for:

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APPENDICES

1.0 EXECUTIVE SUMMARY

This report presents the results of a geotechnical investigation conducted for Phase I (Mountainside and Parkside Subdivisions) of the Bridges at Wolf Creek Development to be constructed in Eden, Utah. The purposes of this investigation were to assess the nature and engineering properties of the subsurface soils at the site and to provide recommendations for general site grading and the design and construction of foundations, slab-on-grades, and exterior concrete flatwork. Portions of the subject site are mapped as being underlain by landslide deposits, and as such our report also investigated the stability of the native hillslopes under static and pseudo-static conditions. Our investigation also included information provided by the Client in the form of a previously completed geotechnical investigation report prepared by GSH as well as a previously completed engineering geological assessment report likewise prepared by GSH. Finally, in addition, GeoStrata utilized information obtained during a geologic hazards screening completed by GeoStrata for the subject site.

Based on the subsurface conditions encountered at the site, it is our opinion that the subject site is suitable for the proposed development provided that the recommendations contained in this report are incorporated into the design and construction of the project. Subsurface soils were investigated through the advancement of 4 exploratory boreholes as well as 3 exploratory test pits. The boreholes were advanced to a depth of approximately 31½ to 61½ feet, whereas the test pits were advanced to a depth of 18 to 23 feet. The soils encountered in our explorations generally consisted of ½ to 2 feet of clayey topsoil, with the exception of borehole B-3, which encountered 3 feet of granular undocumented fill soils. Based on our review of published geologic maps, the topsoil is mapped as overlying both Holocene- to upper Pleistocene-aged alluvial and colluvial deposits (Qafo) as well as Holocene- to Pleistocene-aged landslide deposits (Qms). The bedrock underlying the surficial deposits is mapped as consisting of the Proterozoic lower (green arkos) member of the Maple Canyon Formation (Zmcg). A layer of perched groundwater was encountered in boreholes B-2 and B-3 at a depth of 20 feet, and persisted to a depth of 30 feet. No other evidence of groundwater was observed in our explorations. Seasonal fluctuations in precipitation, surface runoff from adjacent properties, or other on or offsite sources may increase moisture conditions; groundwater conditions can be expected to rise several feet seasonally depending on the time of year, however, it is considered unlikely that the proposed development will be impacted by groundwater.

The foundations for the proposed structures may consist of conventional strip and/or spread footings founded on competent granular soils. If fine-grained soils are encountered in the bottom of the excavations, it is recommended that a minimum of 36-inches of this material be over-excavated and replaced with properly placed and compacted structural fill. Foundation walls may need to be reinforced in order to accommodate potential near-surface soil slumps and failures during wet times of the year. We recommend that a GeoStrata representative observe all foundation soils in footing excavations prior to placing reinforcing steel or concrete. Conventional continuous/spread footings may be proportioned using a maximum net allowable bearing pressure of **2,000 pounds per square foot (psf)** for dead plus live load conditions.

Due to the possibility of moisture reaching the foundation elements during spring runoff, it is recommended that a foundation drain be constructed around the proposed residences.

Results of our slope stability analysis completed for the subject site indicate that the mapped landslide masses identified in the GSH and GeoStrata geologic reports may experience up to 6 inches of deformation during a design-level seismic event. The footings for the proposed residences should be assessed by a structural engineer and reinforced to accommodate this level of deformation. The Client should be aware that building on a landslide mass comes with inherent risks, and the possibility exists that slope instability issues may arise as a result of construction/grading as well as precipitation/runoff events. The Client should be willing to accept these risks if construction occurs on the landslide mass identified in the GSH and GeoStrata reports. Finally, GeoStrata recommends that all moisture control recommendations be implemented as contained within the body of this report.

NOTE: The scope of services provided within this report are limited to the assessment of the subsurface conditions at the subject site. The executive summary is provided solely for purposes of overview and is not intended to replace the report of which it is part and should not be used separately from the report.

2.0 INTRODUCTION

2.1 PURPOSE AND SCOPE OF WORK

This report presents the results of a geotechnical investigation conducted for Phase 1 of the proposed Bridges at Wolf Creek development located at the end of Snowflake Drive and Fairways Drive in Eden, Utah (see Plate A-1, Site Vicinity Map). The purposes of this investigation were to assess the nature and engineering properties of the subsurface soils at the site and to provide recommendations for general site grading, slope stability, and the design and construction of foundations, slab-on-grades, and exterior concrete flatwork. GeoStrata previously completed a geologic hazards screening assessment for Lots 7N, 8N, 9N, 10N, 18N, 209N, 210N, 211N, and 212N. The results of that investigation may be found in a report dated June 19, 2017.

In addition to the reports completed as described above, a geotechnical report titled “Report, Geotechnical Study, The Bridges at Wolf Creek, Northwest of Fairway Drive, Near Eden, Weber County, Utah” completed by GSH and dated January 21, 2016 was provided to GeoStrata for our review. The 2016 GSH geotechnical report was completed for the 364-lot development located on approximately 195-acres. A total of 33 exploratory test pits were completed as part of the 2016 GSH geotechnical investigation.

In addition to the GSH geotechnical study, a report titled “Report, Engineering Geology Study, The Bridges at Wolf Creek East Phase 1, Parts of Sections 15, 16, and 22 Township 7 North, Range 1 East SLBM, Eden, Utah” completed by GSH dated July 25, 2016 for the subject property was likewise provided to GeoStrata. A total of 17 test pits, 5 exploratory boreholes, and two geological trenches were completed as part of the 2016 GSH geologic investigation. An addendum to the GSH engineering geology study dated August 5, 2016 was also provided. The addendum was completed in order to address the omission of lots 10 through 17 from the July 25, 2017 report.

The scope of work completed for this study included a site reconnaissance, subsurface exploration, soil sampling, laboratory testing, engineering analyses, and preparation of this report. Our services were performed in accordance with our proposal, dated October 26, 2017 and your signed authorization. The recommendations contained in this report are subject to the limitations presented in the "Limitations" section of this report (Section 7.1).

2.2 PROJECT DESCRIPTION

The subject site is located approximately 2½ miles north of the intersection of Highway 162 and Wolf Creek Drive in Eden, Utah (see Plate A-1, *Site Vicinity Map*). Our understanding of the proposed development is based on information provided by the client. We understand that the full Bridges at Wolf Creek development will consist of the construction of 364 single-family residential building lots located on 195-acres of property. Phase I of the development, for which this report has been prepared, consists of the Parkside and Mountainside portions of the development, which consists of 126 residential building lots. The development as currently planned will consist of single-family residential structures as well as associated landscaped and paved driveway and roadway areas. Construction plans were not available for review at the time this report was prepared; however, we anticipate that the proposed structures will consist of one to two story wood-framed building with basements founded on conventional spread footings.

3.0 METHODS OF STUDY

3.1 LITERATURE REVIEW

As noted previously, in preparation of this report, we have reviewed the geotechnical study previously completed by GSH Geotechnical for the entire Bridges at Wolf Creek development. In addition, we have reviewed the engineering geology study also previously completed by GSH for Phase 1 of the Bridges at Wolf Creek development. Pertinent information from both these reports was incorporated into our report where applicable.

Surficial geologic mapping completed for the Huntsville Quadrangle completed by Coogan and King (2016) was likewise reviewed.

GeoStrata also reviewed the information presented in our previously-completed geologic hazards screening assessment for Lots 7N, 8N, 9N, 10N, 18N, 209N, 210N, 211N, and 212N.

3.1 FIELD INVESTIGATION

As a part of this investigation, subsurface soil conditions were explored by advancing 4 exploratory boreholes as well as excavating three test pits. The boreholes ranged from 31½ to 61½ feet below the site grade and were advanced using Mobile B-80 truck-mounted drill rig equipped with an ODEX drilling system. Both bulk and relatively undisturbed samples were obtained from each borehole location. A standard Split Spoon sampler and a 2.4-inch inside diameter California Sampler were utilized to collect bulk samples. Relatively undisturbed samples were obtained using Shelby tubes. The approximate locations of the borehole explorations are shown on the *Exploration Location Map*, Plate A-2 in Appendix A.

The test pits were excavated with the aid of a Komatsu PC460 trackhoe. As with the boreholes, both bulk and relatively undisturbed soil samples were obtained from the test pit locations and transported to our laboratory for testing to evaluate the engineering properties of the various earth materials observed.

Subsurface soil conditions as encountered in the explorations were logged at the time of our investigation by qualified personnel and are presented on the enclosed borehole logs (Plates B-1 to B-10) as well as the test pit logs (Plates B-11 to B-13) in Appendix B. A *Key to Soil Symbols and Terminology* is presented on Plate B-14.

3.2 LABORATORY INVESTIGATION

Geotechnical laboratory tests were conducted on selected soil samples obtained during our field investigation. The laboratory testing program was designed to evaluate the engineering characteristics of onsite earth materials. Laboratory tests conducted during this investigation include:

- Grain Size Distribution Analysis (ASTM D422)
- Atterberg Limits Test (ASTM D4318)
- Collapse/Swell Potential Test (ASTM D5333)
- 1-D Consolidation Test (ASTM D2435)
- Direct Shear Test (ASTM D3080)
- Residual Direct Shear Test (ASTM WK3822)

The results of laboratory tests are presented on the borehole and test pit logs in Appendix B (Plates B-1 and B-13), the Lab Summary Report (Plate C-1) and on the test result plates presented in Appendix C (Plates C-2 through C-16).

3.3 ENGINEERING ANALYSIS

Engineering analyses were performed using soil data obtained from the laboratory test results and empirical correlations from material density, depositional characteristics and classification. Appropriate factors of safety were applied to the results consistent with industry standards and the accepted standard of care.

Excavation stability was evaluated based on the field conditions encountered, laboratory test results, and soil type. Occupational Safety and Health (OSHA) minimum requirements are typically prescribed unless conditions warrant further flattening of excavation walls.

4.0 GENERALIZED SITE CONDITIONS

4.1 SURFACE CONDITIONS

The site is in a relatively natural state and consists of alternating areas of heavily wooded areas and grassy fields which are heavily vegetated with sage brush and native weeds and grasses. The hillside property has a topography that largely slopes to the southwest (towards Pineview Reservoir) while being interrupted with occasional minor drainages originating from the mountains to the northeast of the site. No flowing surficial water was observed in any of the drainages at the time of our investigation. Development at the site had been initiated and consisted of roadcuts and other grading activities. No other improvements were observed at the subject site.

4.2 SUBSURFACE CONDITIONS

As previously discussed, the subsurface soil conditions were explored at the site by advancing four exploratory boreholes and excavating three test pits at representative locations across the site. The depths of our explorations are as follows;

Exploration	Total Depth (ft.)
B-1	61½
B-2	46½
B-3	31½*
B-4	41½
TP-1	18
TP-2	23
TP-3	22

*ODEX refusal on clayey materials

The soils encountered in the test pit explorations were visually classified and logged during our field investigation and are included on the borehole and test pit logs in Appendix B (Plates B-1 to B-13). The subsurface conditions encountered during our investigation are discussed below.

4.2.1 Soils

Based on our field observations, the subject site is overlain by ½ to 2 feet of thickly rooted dark brown topsoil comprised of clay, sand, gravel, cobbles, and occasional boulders. The only exception to this was borehole B-3, where approximately 3 feet of undocumented fill soils were observed to overlay the site, and no topsoil was observed. Based on our review of published geologic maps, the topsoil is mapped as overlying both Holocene- to upper Pleistocene-aged alluvial and colluvial deposits (Qafo) as well as Holocene- to Pleistocene-aged landslide deposits (Qms). The bedrock underlying the surficial deposits is mapped as consisting of the Proterozoic lower (green arkos) member of the Maple Canyon Formation (Zmcg). The landslide, slump, and creep hazards associated with the landslide deposits have been discussed in both the June 19, 2017 GeoStrata Geologic Hazards Screening Assessment as well as in the July 25, 2016 GSH Engineering Geology Study, and reference should be made to those reports for further discussion concerning these hazards. Descriptions of the soil units encountered are provided below:

<u>Topsoil:</u> Generally consists of brown to dark brown Lean CLAY (CL) with varying amounts of sand, cobbles, and occasionally boulders up to 2-feet in observed diameter. These soils typically display trace 'pinhole' structure. This unit also has an organic appearance and texture with numerous roots up to 1 inch throughout. Topsoil was encountered in three of our four boreholes and in all three of our test pits and is anticipated to overlie the majority of the site.

<u>Holocene- to Pleistocene-aged Alluvial and Colluvial Deposits (Qafo):</u> Where observed, these soils consisted of alternating seams of fine-grained and coarse-grained sediments. The fine-grained sediments consist of hard, moist, brown to red-brown Elastic SILT (MH), SILT (ML), Lean CLAY (CL), and Fat CLAY (CH), each with varying amounts of sand, gravel, cobbles, and occasional boulders. The coarse-grained sediments encountered consist of dense to very dense, moist, brown to red-brown Silty GRAVEL (GM), Clayey GRAVEL (GC), Poorly Graded GRAVEL (GP-GC) with clay, Silty SAND (SM), and Clayey SAND (SC), each with occasional cobbles and boulders. In general, the fine-grained soils had low to high plasticity, and frequently contained carbonate stringers throughout. The coarse-grained sediments were generally angular to subangular and are considered likely to be associated with debris flow events. Boulders up to 3 feet in diameter were encountered during our exploration, and the possibility exists for larger clasts to exist at depth. These soil units are in general agreement with the observations described in the 2016 GSH investigations.

Holocene-aged Landslide Deposits (Qms): Where observed, these deposits also generally consist of alternating seams of fine-grained and coarse-grained sediments. The fine-grained sediments consisted of hard, moist, brown to red-brown Elastic SILT (MH), SILT (ML), Lean CLAY (CL), and Fat CLAY (CH), each with varying amounts of sand, gravel, cobbles, and occasional boulders. The coarse-grained sediments encountered consist of dense to very dense, moist, brown to red-brown Silty GRAVEL (GM), Clayey GRAVEL (GC), Poorly Graded GRAVEL (GP-GC) with clay, Silty SAND (SM), and Clayey SAND (SC), each with occasional cobbles and boulders. The landslide deposits are described as poorly sorted to unsorted clay to boulder sized material and are mapped in areas where landslide deposits are difficult to distinguish from colluvium deposits and where landslide deposits are too thin (Coogan and King, 2016).

In order to assess the thickness of the landslide mass GeoStrata used the combined information contained in both of the GSH investigations as well as in the current GeoStrata investigation. Review of the borehole logs completed for each of these studies failed to provide evidence for a clear basal shear layer with regards to changes in lithology or other visible evidence preserved in the soil sample. It was noted, however, the blowcounts recorded during our drilling activities as well as during the GSH studies generally indicate a decrease in soil density at depths ranging from 20 to 25 feet. We have generally attributed this increase in moisture and decrease in density to be a basal failure plane. As noted, the depth of the increase in soil density as measured by our blowcounts obtained during our field investigations was relatively similar across the site (regardless of the elevation of the borehole) which suggests that the failure plane driving the landslide is planar in nature rather than a circular failure. A summary of the depths associated with this increase in strength are as follows;

Borehole	Depth to Soil Density Increase (ft.)	Elevation of Borehole (ft. above mean sea level)
B-1 (GeoStrata)	20	5547
B-2 (GeoStrata)	15	5516
B-3 (GeoStrata)	20	5383
B-4 (GeoStrata)	25	5430
B-1 (GSH)	27	5435
B-2 (GSH)	25	5507
B-3 (GSH)	17	5454
B-4 (GSH)	25	5534
B-5 (GSH)	22	5533

It was likewise noted that the soils encountered during our 2017 study at a depth of 20 feet in boreholes B-2 and B-3 existed at a saturation of 80.3 to 96.1 percent, respectively. Considering the relatively soft and saturated conditions of the soils at a depth of 20 to 25 feet, it is the opinion of GeoStrata that the landslide failures are likely shearing along the soils at this depth. An average depth of 23 feet was chosen to represent the basal shear of the landslide during our analysis. GeoStrata recommends that consideration be made to developing a plan to installing and monitoring a series of inclinometers within the hillside, particularly within the area of the proposed landslide. Additional details concerning the proposed inclinometer installation may be found in Section 6.7 of this report.

Finally, based on geologic mapping completed by Coogan and King (2016), as well as on the results of the previously completed GSH study, we understand that Lots 7N, 8N, 9N, 10N, 18N, 209N, 210N, 211N and 212N are mapped as being underlain by these landslide/slump deposits.

Upper- to Middle Proterozoic-aged Maple Canyon Formation (Zmcg): While not observed in our explorations (or possibly observed as a residual soil unit), the Maple Canyon Formation is mapped as consisting of grayish-green, fine-grained arkosic meta-sandstone and sandy argillite with local quartzite lenses (Coogan and King, 2016). This unit weathers into darker gray to brown to greenish-gray and greenish brown soils with relatively high plasticity and is prone to slope failures.

The stratification lines shown on the enclosed borehole and test pit logs represent the approximate boundary between soil types (Plates B-1 to B-13). The actual in-situ transition may be gradual. Due to the nature and depositional characteristics of the native soils, care should be taken in interpolating subsurface conditions between and beyond the exploration locations.

4.2.2 Groundwater

Groundwater was not encountered in any of the test pits advanced as part of this investigation. However, a potential perched groundwater unit was encountered in borehole B-2 at a depth extending from 20 to 30 feet (80.3 percent saturation). A similar 7-foot thick saturated zone was observed in borehole B-3 at depth of 20 feet (96.1 percent saturation). Groundwater was not encountered in boreholes B-1 or B-4.

Groundwater measurements were also recorded as part of the 2016 GSH Engineering Geology Study completed for the subject site. The findings made in that report were summarized in Section **3.7 Groundwater** of the 2016 GSH geologic report and are also summarized in the following table;

Location	Level Below Surface (ft) on 5/4/16 to 5/9/16	Level Below Surface (ft) on 7/1/16	Comments
Test Pit 1	1.0	5.6	Piezometer
Test Pit 2	2.5	7.7	Piezometer
Test Pit 3	0.0	0	Piezometer…water at surface
Test Pit 4	4.0	Pipe Damaged	Piezometer
Test Pit 5	Not Encountered	Not Encountered	Piezometer…dry to 14 feet
Test Pit 6	3.0	4.7	Piezometer
Test Pit 7		Not Encountered	Piezometer…dry to 12.5 feet
Test Pit 9	3.0	7.6	Piezometer
Test Pit 11	Not Encountered	Not Encountered	Piezometer…dry to 9 feet
Test Pit 13	Not Encountered	Not Encountered	Piezometer…dry to 11.7 feet
Test Pit 14	10.0	5/9/2016	Vadose water entering test pit
Test Pit 15	Not Encountered	Not Encountered	Piezometer…dry to 9.5 feet
Test Pit 16	Not Encountered	7.5	Piezometer
Test Pit 17	Not Encountered	Pipe Damaged	Piezometer
Test Pit 18	Not Encountered	8	Piezometer
Trench 2 STA 05	5.0	Not Encountered	Observed in Trench
Boring 1	5.0	5.6	Encountered during drilling
Boring 2	5.0	3.2	Encountered during drilling
Boring 3	5.0	5.5	Encountered during drilling
Boring 4	7.5	7.3	Encountered during drilling

Location	Level Below Surface (ft) on 5/4/16 to 5/9/16	Level Below Surface (ft) on 7/1/16	Comments
Boring 5	5.0	10.5	Encountered during drilling

As can be seen above, significantly increased moisture conditions were observed during the GSH geologic investigation than during either the GSH geotechnical investigation or the GeoStrata investigations. It was noted that GSH completed their geotechnical study in December of 2015 when groundwater elevations are likely to be near their seasonal low (prior to spring runoff). During the GSH geotechnical investigation, groundwater was not encountered in any of the test pits excavated at the subject site to depths up to 13 feet below the existing site grade. The GeoStrata geotechnical investigation was completed in November of 2017, and likewise did not encounter any standing groundwater other than the perched groundwater unit at a depth of 20 feet in our explorations which persisted to a depth of 61½ feet below the site grade. However, the GSH **geological** investigation was completed in May of 2016, when groundwater elevations are likely to be near their season high (after spring runoff), with one round of subsequent groundwater measurements being completed in July, when groundwater levels are likely to be near their seasonal average. It is considered extremely unlikely that this dramatic change can be explained by raising and lowering of groundwater levels, but is more likely the result of infiltration of surface waters into the near-surface soils creating perched groundwater tables. It is believed that the relatively shallow groundwater conditions represent a transient condition, and as such GeoStrata has elected to use a groundwater elevation of approximately 15 feet below the existing site grade in our modeling. However, GeoStrata strongly recommends that the moisture control recommendations made in Sections 6.7 and 6.9 be incorporated into the design of the project to reduce the potential for the seasonal increase in subsurface moisture to impact the stability of the slope.

4.2.3 Expansive Soils

Soils with relatively high plastic limits were encountered in the majority of our test pits and it is anticipated that these soils are present across the majority of the site. Some high plasticity soils have an elevated potential to swell when wetted. Swelling soils can potentially damage foundation elements, crack concrete slabs, and create excess stress in the residential structure. Due to the presence of these soils, swell potential tests were completed on samples obtained during our geotechnical field investigation. Results of these tests indicate the soils have a low to moderate swell potential ranging from 0.01 to 1.16 percent. Recommendations concerning the remediation of these swelling soils may be found in later sections of this report.

4.2.4 Strength of Earth Materials

GeoStrata completed three residual direct shear tests on relatively "undisturbed" decomposed mass-movement samples obtained during our field investigation. Our testing yielded the following results;

Sample I.D.	Depth (ft)	Residual Friction Angle (phi)	Residual Cohesion (psf)
B-2	45	15	70
B-3	15	18	190
TP-2	10	18	145

A value of 18 degrees and 0 psf cohesion for residual soil strength values were assigned for our engineering analysis. Results of our direct shear testing may be found in Appendix C of this report.

5.0 GEOLOGIC CONDITIONS

5.1 GEOLOGIC SETTING

As mentioned previously, a geological hazards investigation has previously been completed by GeoStrata, the results of which may be found in a report dated June 19, 2017. The geological background of the subject site as well as a summary of the geological hazards identified at the subject site may be found within that report dated June 17, 2017. Additional reference should likewise be made to the 2016 GSH Engineering Geology Assessment report.

5.2 SEISMICITY AND FAULTING

The site lies within the north-south trending belt of seismicity known as the Intermountain Seismic Belt (ISB) (Hecker, 1993). The ISB extends from northwestern Montana through southwestern Utah. An active fault is defined as a fault that has had activity within the Holocene (<11ka). No active faults are mapped through or immediately adjacent to the site (Black and others, 2003; UGS 2017). The nearest active fault is the Weber Section of the Wasatch Fault Zone which is located approximately 5½ miles east of the subject site. The most recent movement along the Weber Segment of the Wasatch Fault Zone occurred during the Quaternary Period, and there is evidence that as many as 10 to 15 earthquakes have occurred along this segment in the last 15,000 years (Hecker, 1993). A location near Kaysville Utah indicated that the Weber Segment has a measurable offset of 1.4 to 3.4 meters per event (McCalpin, et all, 1994). The Weber Segment may be capable of producing earthquakes as large as magnitude 7.5 (Ms) and has a recurrence interval of approximately 1,200 years. Analysis of ground shaking hazard along the Wasatch Front suggests that the Wasatch Fault zone is the single greatest contributor to the seismic hazard in the Salt Lake City region. Each of the faults listed above show evidence of Holocene-aged movement, and is therefore considered active.

Seismic hazard maps depicting probabilistic ground motions and spectral response have been developed for the United States by the U.S. Geological Survey as part of NEHRP/NSHMP (Frankel et al, 1996). These maps have been incorporated into both *NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures* (FEMA, 1997) and the *International Building Code* (IBC) (International Code Council, 2015). Spectral responses for the Maximum Considered Earthquake (MCE_R) are shown in the table below. These values generally correspond to a two percent probability of exceedance in 50 years (2PE50) for a "firm rock" site. To account for site effects, site coefficients which vary with the magnitude of spectral acceleration are used. Based on our field and office investigations, it is our opinion that this location is best described as a Site Class C which represents a “very dense soil and soft rock” profile. The spectral accelerations are shown in the table below. The spectral accelerations are calculated based on the site’s approximate latitude and longitude of 41.3383° N and -111.8321° W, respectively, and the United States Geological Survey U.S. Seismic Design Maps web-based application. Based on the IBC, the site coefficients are F_a=1.02 and F_v=1.48. From this procedure the peak ground acceleration (PGA) is estimated to be 0.39g.

Site Location: **Latitude = 41.3383° N** **Longitude = -111.8321° W**	**Site Class C Site Coefficients:** **Fa = 1.02** **Fv = 1.48**
Spectral Period (sec)	**Response Spectrum Spectral Acceleration (g)**
0.2	$\mathbf{S_{MS}}$=(F_a*S_s=1.02*0.94) = 0.96
1.0	$\mathbf{S_{M1}}$=(F_v*S_1=1.48*0.32) = 0.48
[a] IBC 1613.3.4 recommends scaling the MCE_R values by 2/3 to obtain the design spectral response acceleration values; values reported in the table above have not been reduced.	

Table 3: MCE_R Seismic Response Spectrum Spectral Acceleration Values for IBC Site Class C[a].

6.0 ENGINEERING CONCLUSIONS AND RECOMMENDATIONS

6.1 GENERAL CONCLUSIONS

Based on the subsurface conditions encountered at the site, it is our opinion that the subject site is suitable for the proposed construction provided that the recommendations contained in this report are complied with. The recommendations presented in this report are based on our understanding of the proposed project, the subsurface conditions observed during field exploration, the results of laboratory testing, and our engineering analyses. If subsurface conditions other than those described herein are encountered in conjunction with construction, and/or if design and layout changes are initiated, we must be informed so that the recommendations herein can be reviewed and revised as changes or conditions may require.

6.2 EARTHWORK

Prior to the placement of foundations, general site grading is recommended to provide proper support for foundations, exterior concrete flatwork, and concrete slab-on-grade. Site grading is also recommended to provide proper drainage and moisture control on the subject property and to aid in preventing differential settlement of foundations as a result of variations in subgrade moisture conditions.

6.2.1 General Site Preparation and Grading

In areas beneath footings and concrete flat work, topsoil should be stripped and stockpiled for use in landscape areas or disposal. Debris, undocumented fill, vegetation, roots, loose, soft or other deleterious materials should also be removed and replaced with structural fill. Tree roots are anticipated and should be grubbed-out and replaced with engineered fill. If over-excavation is required, the excavation should extend a minimum of one foot laterally for every foot of depth of over-excavation. Excavations should extend laterally at least two feet beyond flatwork, pavements, and slabs-on-grade. If materials are encountered that are not represented in the test pit logs or may present a concern, GeoStrata should be notified so observations and further recommendations as required can be made. The exposed native soils should then be proof-rolled with heavy rubber-tired equipment. If soft soils are observed, they should be stabilized in accordance with our recommendations in the Soft Soil Stabilization Section (Section 6.2.3); if loose soils are observed, they should be compacted as recommended in Section 6.2.4.

Landslides have historically occurred in the area of the subject site. In order to minimize the potential for contributing to landslide issues, care should be taken in planning and designing the site grading. We recommend that cut slopes be limited to no greater than 3 feet in height and should be created at no steeper than a 3H:1V (horizontal to vertical) slope. Additionally, we recommend that fill placed on the site have a maximum height of 3 feet, especially near or on any existing slopes. The final slope of the fill sections should also be no greater than 3H:1V. We further recommend that the geotechnical engineer be given the opportunity to review the proposed grading plan. Additional slope stability assessment will need to be completed to assess how the proposed grading plan will affect the native slopes.

6.2.2 Excavation Stability

Based on Occupational Safety and Health Administration (OSHA) guidelines for excavation safety, trenches with vertical walls up to 5 feet in depth may be occupied, however, the presence of fill soils, loose soils, or wet soils may require that the walls be flattened to maintain safe working conditions. When the trench is deeper than 5 feet, we recommend a trench-shield or shoring be used as a protective system to workers in the trench. Based on our soil observations, laboratory testing, and OSHA guidelines, native soils at the site classify as Type C soils. Deeper excavations, if required, should be constructed with side slopes no steeper than one and one-half horizontal to one vertical (1.5H:1V). If wet conditions are encountered, side slopes should be further flattened to maintain slope stability. Alternatively, shoring or trench boxes may be used to improve safe work conditions in trenches. The contractor is ultimately responsible for trench and site safety. Pertinent OSHA requirements should be met to provide a safe work environment. If site specific conditions arise that require engineering analysis in accordance with OSHA regulations, GeoStrata can respond and provide recommendations as needed.

We recommend that a GeoStrata representative be on-site during all excavations to assess the exposed foundation soils. We also recommend that the Geotechnical Engineer be allowed to review the grading plans when they are prepared in order to evaluate their compatibility with these recommendations.

6.2.3 Soft Soil Stabilization

Soft or pumping soils may be exposed in excavations at the site. Once exposed, all subgrade surfaces beneath proposed structure, pavements, and flat work concrete should be proof rolled with heavy wheeled-construction equipment. If soft or pumping soils are encountered, these soils should be stabilized prior to construction of footings. Stabilization of the subgrade soils can be accomplished using a clean, coarse angular material worked into the soft subgrade. We recommend the material be greater than 2 inches in diameter, but less than 6 inches. A locally available pit-run gravel may be suitable but should contain a high percentage of particles larger than 2 inches and have less than 7 percent fines (material passing the No. 200 sieve). A pit-run gravel may not be as effective as a coarse, angular material in stabilizing the soft soils and may require more material and greater effort. The stabilization material should be worked (pushed) into the soft subgrade soils until a firm relatively unyielding surface is established. Once a firm, relatively unyielding surface is achieved, the area may be brought to final design grade using structural fill.

In large areas of soft subgrade soils, stabilization of the subgrade may not be practical using the method outlined above. In these areas it may be more economical to place a woven geotextile fabric against the soft soils covered by 18 inches of coarse, sub-rounded to rounded material over the woven geotextile. An inexpensive non-woven geotextile "filter" fabric should also be placed over the top of the coarse, sub-rounded to rounded fill prior to placing structural fill or pavement section soils to reduce infiltration of fines from above. The woven geotextile should be Amoco 2004 or prior approved equivalent. The filter fabric should consist of an Amoco 4506, Amoco 4508, or equivalent as approved by the Geotechnical Engineer.

6.2.4 Structural Fill and Compaction

All fill placed for the support of the structures, pavements, or flatwork concrete should consist of structural fill. Due to the high plastic nature of the native soils we do not recommend that they be used as structural fill. Structural fill should consist of an imported granular soil with a maximum fines content (minus No.200 mesh sieve) of 30 percent. All structural fill should be free of vegetation and debris and contain no materials larger than 3-inches in nominal size. All structural fill soils should be approved by the Geotechnical Engineer prior to placement. Clay and silt particles in imported structural fill should have a liquid limit less than 35 and a plasticity index less than 15 based on the Atterberg Limit's test (ASTM D-4318). The contractor should have confidence that the anticipated method of compaction will be suitable for the type of structural fill used. The contractor should anticipate testing all soils used as structural fill frequently to assess the maximum dry density, fines content, and moisture content, etc.

All structural fill should be placed in maximum 6-inch loose lifts if compacted by small hand-operated compaction equipment, maximum 8-inch loose lifts if compacted by light-duty rollers, and maximum 12-inch loose lifts if compacted by heavy duty compaction equipment that is capable of efficiently compacting the entire thickness of the lift. We recommend that all structural fill be compacted on a horizontal plane, unless otherwise approved by the geotechnical engineer. Structural fill with an overall thickness of 5 feet or less should be compacted to at least 95% of the maximum dry density (MDD), as determined by ASTM D-1557 (modified proctor). The moisture content should be within 3% of the optimum moisture content (OMC) at the time of placement and compaction. Also, prior to placing any fill, the excavations should be observed by the geotechnical engineer to observe that any unsuitable materials or loose soils have been removed. In addition, proper grading should precede placement of fill, as described in the *General Site Preparation and Grading* subsection of this report (Section 6.2.1).

Fill soils placed for subgrade below exterior flat work should be within 3% of the OMC when placed and compacted to at least 95% of the MDD as determined by ASTM D-1557. All utility trenches backfilled below the proposed structure, pavements, and flatwork concrete, should be backfilled with structural fill that is within 3% of the OMC when placed and compacted to at least 95% of the MDD as determined by ASTM D-1557. All other trenches, in landscape areas, should be backfilled and compacted to at least 90% of the MDD (ASTM D-1557).

The gradation, placement, moisture, and compaction recommendations contained in this section meet our minimum requirements but may not meet the requirements of other governing agencies such as city, county, or state entities. If their requirements exceed our recommendations, their specifications should override those presented in this report.

6.3 FOUNDATIONS

Due to the presence of potentially swelling soils as well as the presence of potentially unstable near-surface sediments at the site (see Section 6.7 of this report) the foundations for the proposed structures may consist of conventional strip and/or spread footings founded entirely on competent **granular** (sand and gravel) soils. Foundation elements should not be established on the near-surface clay soils unless approved by the Geotechnical Engineer. If fine-grained soils are encountered in the bottom of the foundation excavation, then the fine-grained soils should be over-excavated a minimum of 36 inches or until granular soils are encountered. The site may then be brought back up to design grade using properly placed and compacted structural fill. Foundation walls will likely need to be reinforced in order to aid in retaining the upslope, near-surface soils. Exterior shallow footings should be embedded at least 40-inches below final grade for frost protection and confinement. Interior footings not subject to frost should be embedded at least 18 inches below final grade to provide confinement. To provide adequate support and confinement, we recommend that footings be placed at least 15 feet, measured horizontally, from the face of existing or fill slopes at the site.

Conventional strip footings founded entirely on granular soils or on structural fill founded entirely on undisturbed, native granular soils or structural fill as described above may be proportioned for a maximum net allowable bearing capacity of **2,000 psf**. The net allowable bearing capacity may be increased (typically by one-third) for temporary loading conditions such as transient wind and seismic loads. All footing excavations should be observed by the Geotechnical Engineer prior to footing placement.

As noted earlier in the report, the onsite clay soils had a moderate potential for swelling under increased moisture conditions. To minimize the potential damage associated with swelling soils, the foundation elements should be designed to have a minimum load of 1,500 psf.

Settlements of properly designed and constructed conventional footings, founded as described above, are anticipated to be less than 1 inch. Differential settlements should be on the order of half the total settlement over 30 feet.

6.4 FOUNDATION DRAINAGE

Due to the possibility of moisture reaching the foundation elements during spring runoff, it is recommended that a foundation drain be constructed if basements are incorporated into the proposed construction. The foundation drain should consist of a 4-inch perforated pipe placed at or below the footing elevation. The pipe should be covered with at least 12 inches of free draining gravel (containing less than 5 percent passing the No 4 sieve) and be graded to a free gravity out fall or to a pumped sump. A separator fabric, such as Mirafi 140N, should separate the free draining gravel and native soil (i.e. the separator fabric should be placed between the gravel and the native soils at the bottom of the gravel, the side of the gravel where the gravel does not lie against the concrete footing or foundation, and at the top of the gravel). We recommend that the gravel extend up the foundation wall to within 3 feet of the final ground surface. As an alternative, the gravel extending up the foundation wall may be replaced with a prefabricated drain panel, such as Ecodrain-E. The Owner should discuss and obtain any required permit for the discharge of the drains with the City. The discharge may need to be piped into a storm sewer system or be collected on site in an appropriately designed and lined pond.

6.5 CONCRETE SLAB-ON-GRADE CONSTRUCTION

Laboratory testing indicated that native soils at the site have some potential for expansion. Due to the light loads associated with concrete slabs, it is possible that both exterior and interior slab could experience movement due to changing moisture conditions. If it is desired to reduce this risk, consideration should be given to placing concrete slabs on 24 inches of structural fill. As a minimum, concrete slabs-on-grade should be constructed over at least 4 inches of compacted gravel overlying native soils or a zone of structural fill that is at least 12 inches thick. Disturbed native soils should be compacted to at least 95% of the MDD as determined by ASTM D-1557 (modified proctor) prior to placement of gravel. The gravel should consist of road base or clean drain rock with a ¾-inch maximum particle size and no more than 12 percent fines passing the No. 200 mesh sieve. The gravel layer should be compacted to at least 95% of the MDD of modified proctor or until tight and relatively unyielding if the material is non-proctorable. All concrete slabs should be designed to minimize cracking as a result of shrinkage. Consideration should be given to reinforcing the slab with welded wire, re-bar, or fiber mesh. Loading on any concrete slabs should not exceed 300 psf.

6.6 EARTH PRESSURES AND LATERAL RESISTANCE

Lateral forces imposed upon conventional foundations due to wind or seismic forces may be resisted by the development of passive earth pressures and friction between the base of the footing and the supporting soils. In determining the frictional resistance against concrete, a coefficient of friction of 0.43 should be used for native granular soils or structural fill.

Ultimate lateral earth pressures from *granular* backfill acting against retaining walls and buried structures may be computed from the lateral pressure coefficients or equivalent fluid densities presented in the following table:

Condition	Lateral Pressure Coefficient	Equivalent Fluid Density (pounds per cubic foot)
Active*	0.30	33
At-rest**	0.50	55
Passive*	6.11	672
Seismic Active***	0.42	46
Seismic Passive***	-2.04	-225

* Based on Coulomb's equation
** Based on Jaky
*** Based on Mononobe-Okabe Equation

If native fine-grained soils are to be utilized as backfill, then the following values should be utilized;

Condition	Lateral Pressure Coefficient	Equivalent Fluid Density (pounds per cubic foot)
Active*	0.35	38
At-rest**	0.56	62
Passive*	4.39	483
Seismic Active***	0.50	55
Seismic Passive***	-1.66	-183

* Based on Coulomb's equation
** Based on Jaky
*** Based on Mononobe-Okabe Equation

These coefficients and densities assume level, granular backfill with no buildup of hydrostatic pressures. The force of the water should be added to the presented values if hydrostatic pressures are anticipated. If sloping backfill is present, we recommend the geotechnical engineer be consulted to provide more accurate lateral pressure parameters once the design geometry is established.

Walls and structures allowed to rotate slightly should use the active condition. If the element is constrained against rotation, the at-rest condition should be used. These values should be used with an appropriate factor of safety against overturning and sliding. A value of 1.5 is typically used. Additionally, if passive resistance is calculated in conjunction with frictional resistance, the passive resistance should be reduced by ½.

For seismic analyses, the *active* and *passive* earth pressure coefficient provided in the table is based on the Mononobe-Okabe pseudo-static approach and only accounts for the dynamic horizontal thrust produced by ground motion. Hence, the resulting dynamic thrust pressure *should be added* to the static pressure to determine the total pressure on the wall. The pressure distribution of the dynamic horizontal thrust may be closely approximated as an inverted triangle with stress decreasing with depth and the resultant acting at a distance approximately 0.6 times the loaded height of the structure, measured upward from the bottom of the structure.

The coefficients shown assume a vertical wall face. Hydrostatic and surcharge loadings, if any, should be added. Over-compaction behind walls should be avoided. Resisting passive earth pressure from soils subject to frost or heave, or otherwise above prescribed minimum depths of embedment, should usually be neglected in design.

6.7 SLOPE STABILITY

As mentioned previously, and as discussed within the geological hazards report completed by GeoStrata for the referenced project, the subject site is located in an area where slope failures have occurred. In order to assess the stability of the native slopes at the subject site, two slope stability profiles were analyzed as part of this investigation. These slope stability profiles have been identified on Plate A-3 as Profile A, and Profile B. Each of these profiles are located along the maximum longitudinal axis of the mapped landslide deposits identified on the property and have been analyzed for slope stability under both static and pseudo-static (seismic) conditions.

The global stability of the slope stability profiles was modeled using Slide, a computer application which incorporates, among others, Bishop's Simplified Method of analysis. Calculations for stability were developed by searching for the minimum factor of safety for a block-type failure. Homogenous earth materials and arcuate failure surfaces were assumed. Topographic information for the profiles was obtained using the 2006 2-meter LiDAR provided by the State of Utah AGRC.

Slope stability analysis was performed for both static and pseudo-static (seismic) conditions. The pseudo-static assessment was completed considering the peak ground acceleration (PGA) associated with a 2 percent probability of exceedance in 50 years. A PGA value of 0.39g was previously calculated as presented in Section 5.2 of this report.

Strength parameters for the soils located at the subject property were obtained utilizing the results of residual direct shear tests completed as part of this investigation. Residual shear test results were utilized due to the presence of several landslide deposits across the site. In summary, the results of our residual shear tests are as follows;

Residual Shear Strength – Residual Soils	
Residual Friction Angle (phi) (degrees)	Cohesion (psf)
15-18	70-190

Due to the potentially residual nature of these soils, and in accordance with the standard of care, the cohesive aspect of the soil strength has been reduced to 0 psf. Although a single laboratory test gave a phi angle of 15 degrees, upon review it was found that the resultant curves were irregular and may not have accurately conveyed the final soil strength for this material. As discussed in Section 4.2.1 of this report, the depth of the landslide deposits is estimated to be approximately 23 feet based on the relatively lower blowcounts and relatively higher moisture content of the soils encountered at this depth. As such, these residual strength values have been applied to the near-surface clayey soils (upper 23 feet) in order to match the description of the landslide deposits given in Section 4.2.1 of this report as well as in the 2016 GSH report.

For greater depths, GeoStrata utilized the blowcounts to estimate the strength of the subsurface soils/residual bedrock. Using the correlation between soil strength and blowcounts proposed by Schmertmann (1975), the resulting soil strength parameters for the soils at depth exceed 40 degrees. These values have been reduced to a friction angle of 26 degrees and 250 psf cohesion in order to account for potential variations across the site.

Layers of perched groundwater were encountered in boreholes B-2 and B-3 at depths ranging from 20 to 30 feet below the site grade. In addition, as summarized in Section 4.2.2 of this report, groundwater was observed at relatively shallow depths across the subject site during the fieldwork completed for the 2016 GSH geologic study, with groundwater observed at the surface in one trench. Based on the lack of water in our test pit and borehole explorations, we are considering surficial groundwater to be a transient condition. As such, groundwater has been modeled as existing at a depth of 15 feet throughout the length of the profile in order to account for potential variations across the site.

Results of our slope stability investigation are as follows:

Factors of Safety			
Profile	Failure Depth (ft.)	Static	Pseudo Static
A	23 ft.	2.07	0.89
B	23 ft.	1.49	0.84

Based on these results, it is anticipated that the slopes investigated may experience near-surface slope creep and shallow failures, particularly in response to seasonal fluctuations in moisture content. In order to reduce the potential of these instabilities from impacting the proposed residences, the recommendations given in Section 6.9 of this report should be implemented. The owner should be aware that maintenance of these slopes may be required on a regular basis.

The landslide does not meet the minimum factor of safety of 1.0 for seismic conditions, indicating that both of the landslide masses will experience recurrent movement during a maximum credible earthquake event. As such a deformation analysis was completed following the methodology outlined by Bray and Travasarou (2007). Based on the results of our deformation analysis, it is anticipated that Profile A may experience up to 10 cm (3.9 inches) of deformation as a result of a seismic event, whereas Profile B may experience up to 14.1 cm (5.6 inches) of displacement. Results of our slope stability modeling and deformation analysis can be found in Appendix D.

Based on the results of our slope stability modeling and deformation analysis, it is likely that 3 to 6 inches of lateral movement may occur on each of the landslide bodies as a whole during a magnitude 7.1 Mw event. Although actual movement may vary from lot to lot and will likely be less than that stated, we recommend that the structural elements for the proposed structures be designed to withstand the 3 to 6 inches noted. This deformation should be modeled as occurring along a single discrete plane of weakness.

GeoStrata likewise strongly recommends that consideration be made to the construction of an intercept trench located on the upslope limits of the development across the head of the mapped landslides (perpendicular to landslide direction). This trench should consist of a 20-foot deep excavation backfilled with ¾-inch minus crushed gravel. A 6-inch diameter perforated pipe should be installed at the bottom of the trench (inside the gravel), and allowed to daylight in an approved location that will not cause further instability. The construction of such a trench should proceed in stages so as to not destabilize the landslide during installation. If the Client wishes to have a more in-depth design of such a system, GeoStrata may be contacted to provide such a design.

The limits of the mapped landslide extend beyond the property boundaries, and as such it not feasible to complete a more in-depth investigation into the stability of both mapped landslides. The Client should be aware that building on mapped landslide deposits has an inherent risk, and they should be willing to accept these risks if development is to occur.

6.7.1 Drainage Lots

During our review of the proposed site plan, it was noted that several proposed residences are planned on lots located adjacent to several drainages oriented in a northeast-southwest direction. Based on geologic mapping completed for the subject site, it appears that these drainages have a greater potential to be impacted by slope stability concerns. It is recommended that a lot specific slope stability assessment be completed on each lot located adjacent to a drainage once a proposed development plan for the lot is available.

6.7.2 Inclinometer and Piezometer Installation and Monitoring

Due to the marginal factors of safety obtained during our slope stability analysis as well as due to the significant variation in the groundwater measurements, GeoStrata strongly recommends that consideration be made to installing a series of inclinometers within the areas underlain by mapped landslide deposits. GeoStrata recommends that a minimum of one inclinometer be installed near the head and the toe of the mapped landslides, as well as single inclinometer within the central portion of the slide mass to monitor the potential movement of the different portions of the slide in response to varying moisture conditions. We recommend that the two inclinometers located on the distal portions of the landslide mass be advanced to a minimum depth of 35 feet, and the third inclinometer located in the central portion of the slide mass be advanced to a depth of 60 feet. These inclinometers should be read on a quarterly basis during construction, semiannually for three years after construction is completed, then on an annual basis for five years after that.

We likewise recommend that piezometers be installed at the same time as the inclinometers described above, and that a similar monitoring plan be implemented. And indication of slope instability or unusually high groundwater elevations should be reported to GeoStrata upon detection.

6.8 PERMANENT CUT AND FILL SLOPES

As mentioned previously in the report, due to the presence of mapped landslide deposits within the subject property, as well as the elevated potential for slope instability, it is recommended that cut/fill sections at the subject site be limited to 3 feet or less. Larger cut/fill sections may be feasible at the subject site; however, it is recommended that a location specific stability investigation be completed within the area of any proposed cut/fill sections that exceed 3 feet. The cut/fill slopes should be created no steeper than 3:1 horizontal to vertical.

6.9 MOISTURE PROTECTION AND SURFACE DRAINAGE

Precautions should be taken during and after construction to minimize the potential for saturation of foundation soils. Over wetting the soils prior to or during construction may result in increased softening and pumping, causing equipment mobility problems and difficulty in achieving compaction.

As discussed previously, portions of the proposed development are mapped as being underlain by landslide deposits. As such, it is strongly recommended that irrigation at the site be limited to drip-line systems and that xeriscaping using native vegetation be implemented as much as is practicable. The Client should be aware that excessive irrigation may cause the local destabilization of slope, which may damage the proposed infrastructure. In addition, it is recommended that all utility pipelines be tested periodically to assess the system for leaks that could also lead to slope instability.

Infiltration of moisture in the vicinity of structures should be minimized. We recommend that roof runoff devices be installed to direct all runoff a minimum of 10 feet away from structures. The grade within 10 feet of the structures should be sloped a minimum of 5% away from the structure in accordance with the IBC, 2015.

During spring months, melt water from the slope to the northeast of the property may impact the proposed residences if strategic site grading is not completed. Catchment basins and diversionary berms should be installed upgradient from the properties and should direct all moisture toward the storm drains on the nearest roadways. Any areas where standing water is identified upgradient from the proposed development should likewise be regraded to discharge water into a storm drain system.

Based on the irregular nature of the groundwater from the different investigations, interception trenches and drains should be constructed near the head of drainages in an effort to intercept groundwater and minimize saturating the soils at the depths of the assumed failure plane. We recommend that the drainage trenches be constructed to the details presented on Plate D-1 in Appendix D. All seepage collected from the drains should be piped and conveyed further downslope.

7.0 CLOSURE

7.1 LIMITATIONS

The recommendations contained in this report are based on our limited field exploration, laboratory testing, and understanding of the proposed construction. The subsurface data used in the preparation of this report were obtained from the explorations made for this investigation. It is possible that variations in the soil and groundwater conditions could exist between 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, we 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, GeoStrata should be notified.

It should be noted that building on mapped landslide deposits has an inherent risk, and possibility exists that movement beyond the limits stated in this report may occur as a response to construction or heavy precipitation and runoff. The Client should be willing to accept this potential if development is completed on the landslide masses identified by GSH and GeoStrata. Additionally, GeoStrata should be notified if movement is detected and additional investigations may be warranted.

This report was prepared in accordance with the generally accepted standard of practice at the time the report was written. No 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.

7.2 ADDITIONAL SERVICES

The recommendations made in this report are based on the assumption that an adequate program of tests and observations will be made during construction. GeoStrata staff should be on site to verify compliance with these recommendations. These tests and observations should include, but not necessarily be limited to, the following:

- Observations and testing during site preparation, earthwork and structural fill placement.
- Observation of foundation soils to assess their suitability for footing placement.
- Observation of soft/loose soils over-excavation.
- Observation of temporary excavations and shoring.
- Consultation as may be required during construction.
- Quality control and observation of concrete placement.

We also recommend that project plans and specifications be reviewed by us to verify compatibility with our conclusions and recommendations. This includes any final grading plans for the property, as it is recommended that final slope stability analyses be completed once such information is available. Additional information concerning the scope and cost of these services can be obtained from our office.

We appreciate the opportunity to be of service on this project. Should you have any questions regarding the report or wish to discuss additional services, please do not hesitate to contact us at your convenience at (801) 501-0583.

8.0 REFERENCES CITED

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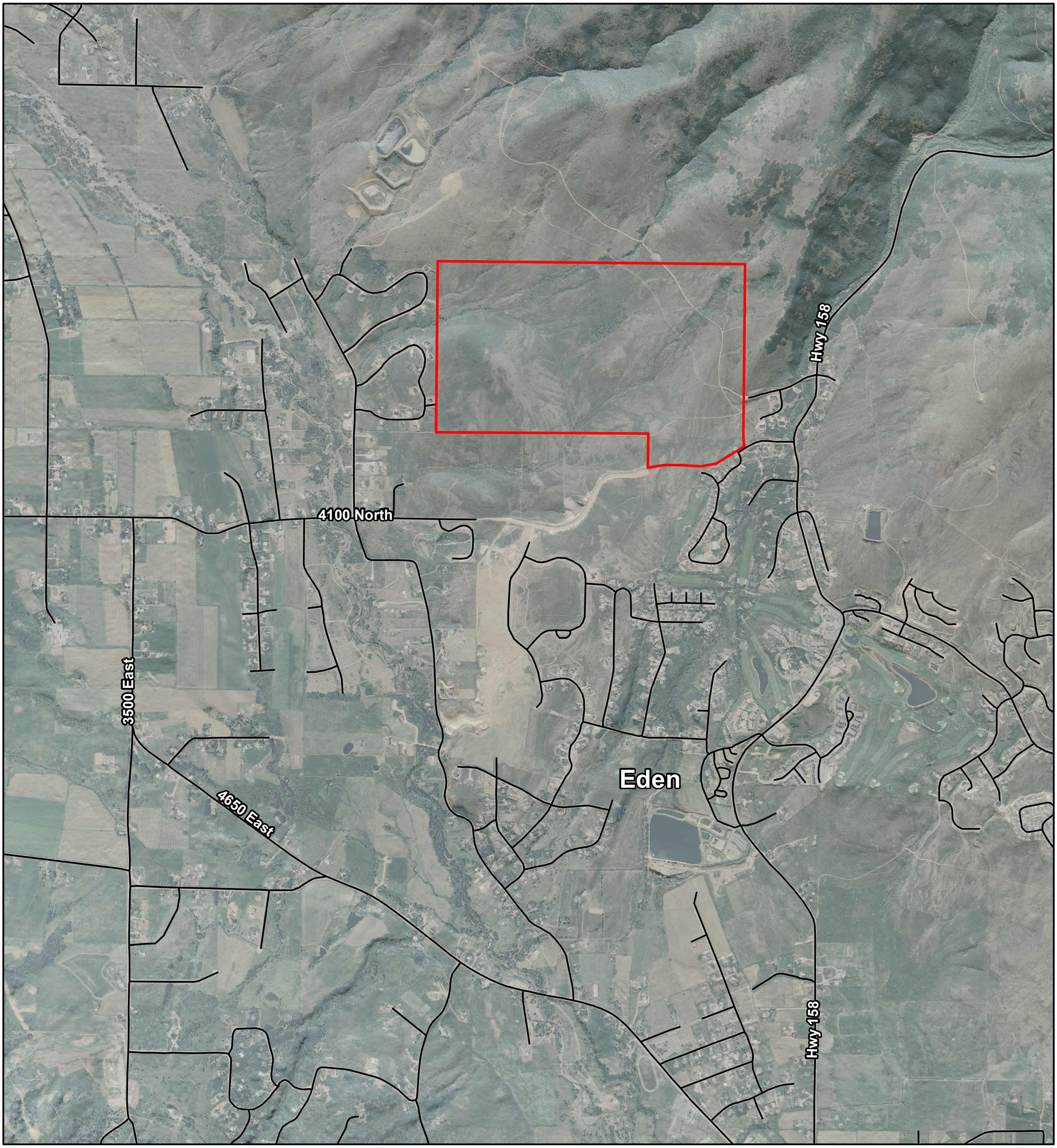


Legend

Approximate Site Boundary

0 750 1,500 3,000 4,500 6,000 Feet

1 inch = 2,000 feet

Base Maps:
2009 1 meter NAIP and hillshades derived from 1 meter LiDAR provided by the State of Utah AGRC.

N

GeoStrata

Eric Householder
The Bridges
Eden, Utah
Project Number: 1311-002

Site Vicinity Map

Plate A-1

Legend

Approximate Boring Location

Approximate Test Pit Location

Landslide (Coogan and King)

0 112.5 225 450 675 900 Feet

1 inch = 300 feet

Base Maps:
2009 1 meter NAIP and hillshades derived from 1 meter LiDAR provided by the State of Utah AGRC.

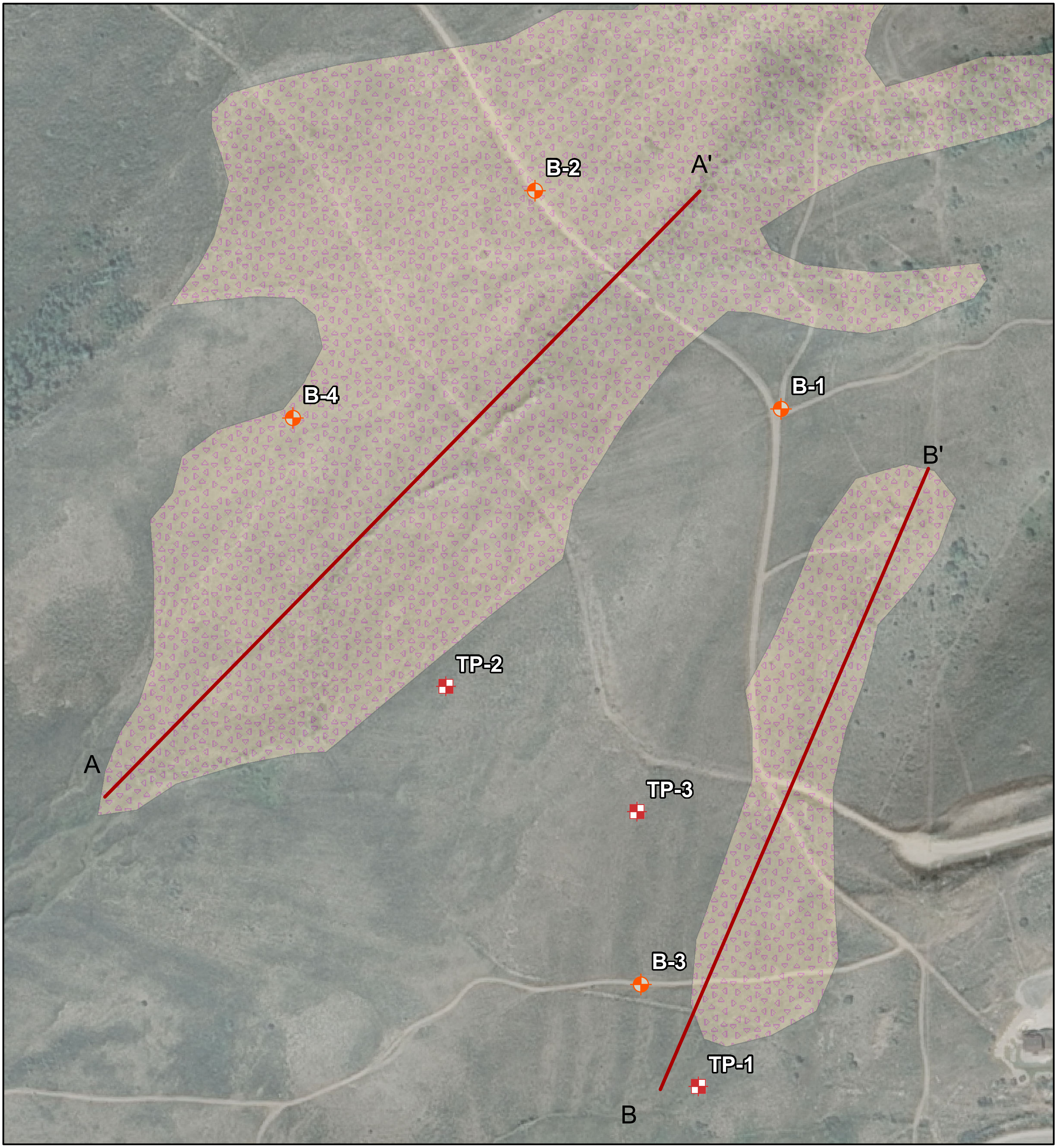
N

GeoStrata

Eric Householder
The Bridges
Eden, Utah
Project Number: 1311-002

Exploration Location Map

Plate
A-2



Legend

Approximate Boring Location

Approximate Test Pit Location

Cross Sections

Landslide (Coogan and King, 2016)

0 112.5 225 450 675 900 Feet

1 inch = 300 feet

Base Maps:
2009 1 meter NAIP and hillshades derived from 1 meter LiDAR provided by the State of Utah AGRC.

N

GeoStrata

Eric Householder
The Bridges
Eden, Utah
Project Number: 1311-002

Geologic Cross-Sections

Plate A-3

DATE	
STARTED:	11/9/17
COMPLETED:	11/9/17
BACKFILLED:	11/9/17

Eric Householder
The Bridges Development
Eden, Utah
Project Number 1311-002

GeoStrata Rep: A. Peay
Rig Type: B-80 Mobile Drill
Boring Type: Auger

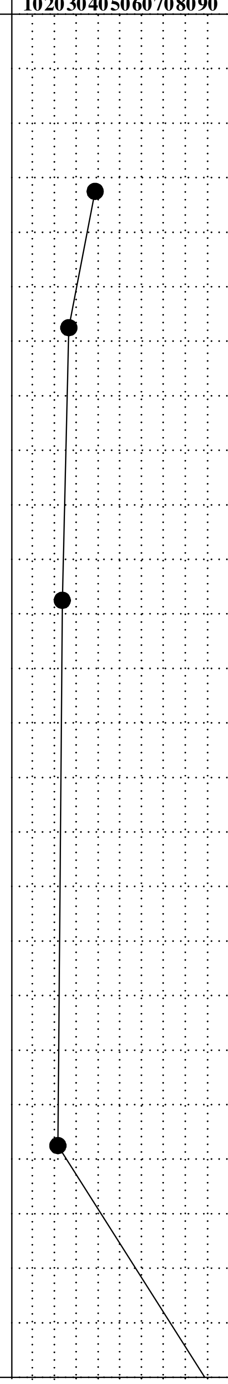
BORING NO:
B-1
Sheet 1 of 3

LOCATION
STATION OFFSET ELEVATION

UNIFIED SOIL CLASSIFICATION	MATERIAL DESCRIPTION
	TOPSOIL; Lean CLAY - dark brown, moist, organics throughout
MH	Sandy Elastic SILT - hard, red-brown with tan mottling, moist
GM	Silty GRAVEL with sand - very dense, red-brown, moist, sub-angular gravels up to 2 inches in diameter
CL	Sandy Lean CLAY - hard, red-brown with black mottling and white carbonate stringers, moist,

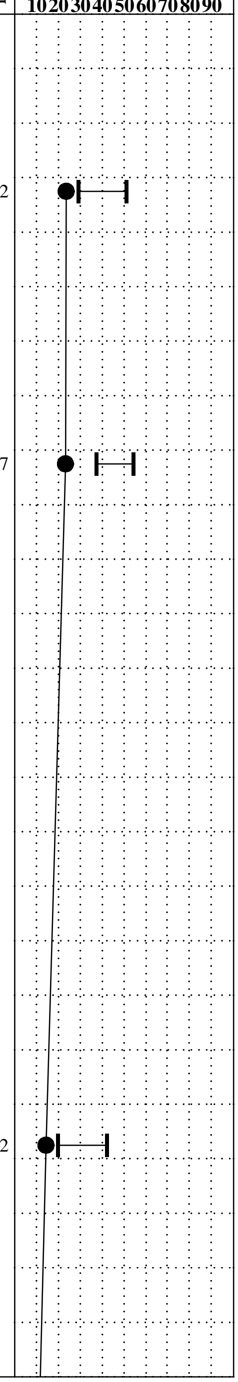
N	N*	Dry Density(pcf)	Moisture Content %	Percent minus 200	Liquid Limit	Plasticity Index
48	38		23.5	60.9	51	22
35	26					
100			23.2	31.6	54	17
34	23					
100						
30	21	119.2	14.3	69.7	42	22

SPT BLOW COUNT



Moisture Content and Atterberg Limits

Plastic Limit Moisture Content Liquid Limit



DEPTH: METERS 0 1 2 3 4 5 6 7; FEET 0 5 10 15 20

N - OBSERVED UNCORRECTED BLOW COUNT

N* - CORRECTED N1(60) EQUIVALENT SPT BLOW COUNT

GeoStrata

SAMPLE TYPE
- 2" O.D./1.38" I.D. Split Spoon Sampler
- 2.5" O.D./2" I.D. California Split Spoon Sampler
- 3" O.D. Thin-Walled Shelby Sampler
- Grab Sample
- 2" O.D./1.625" I.D. Liner Sampler

NOTES:

WATER LEVEL
- MEASURED
- ESTIMATED

Plate
B - 1

DATE	
STARTED:	11/9/17
COMPLETED:	11/9/17
BACKFILLED:	11/9/17

Eric Householder
The Bridges Development
Eden, Utah

Project Number 1311-002

GeoStrata Rep: A. Peay
Rig Type: B-80 Mobile Drill
Boring Type: Auger

BORING NO: **B-1**

Sheet 2 of 3

LOCATION — STATION / OFFSET / ELEVATION

DEPTH (FEET)	USCS	MATERIAL DESCRIPTION	N	N*	Dry Density (pcf)	Moisture Content %	Percent minus 200	Liquid Limit	Plasticity Index
25	CL	Lean CLAY with gravel - hard, red-brown with trace blue-grey mottling, moist, gravels up to 3/4 inch in diameter	100	100					
30			100	100		8.1	14.4		
35			100	100					
40	GC	Clayey GRAVEL - very dense, red-brown with blue-grey and light pink mottling, moist, sample contains approximately 60% gravels up to 2 inch in diameter	100	100					
45	GP-GC	Poorly Graded GRAVEL with clay and sand - very dense, red-brown with red-brown and yellow-brown mottling, moist, sample contains approximately 60% gravels up to 2.5 inch in diameter	100	50		7.2	9.8	41	25

Depth scale: METERS 8–15; FEET 25–45.

Moisture Content and Atterberg Limits (Plastic Limit — Moisture Content — Liquid Limit): 10 20 30 40 50 60 70 80 90

SPT BLOW COUNT: 10 20 30 40 50 60 70 80 90

N - OBSERVED UNCORRECTED BLOW COUNT
N* - CORRECTED N1(60) EQUIVALENT SPT BLOW COUNT

GeoStrata

SAMPLE TYPE
- 2" O.D./1.38" I.D. Split Spoon Sampler
- 2.5" O.D./2" I.D. California Split Spoon Sampler
- 3" O.D. Thin-Walled Shelby Sampler
- Grab Sample
- 2" O.D./1.625" I.D. Liner Sampler

NOTES:

WATER LEVEL
▼ - MEASURED ▽ - ESTIMATED

Plate
B - 2

DATE	
STARTED:	11/9/17
COMPLETED:	11/9/17
BACKFILLED:	11/9/17

Eric Householder
The Bridges Development
Eden, Utah
Project Number 1311-002

GeoStrata Rep: A. Peay
Rig Type: B-80 Mobile Drill
Boring Type: Auger

BORING NO:
B-1
Sheet 3 of 3

LOCATION
STATION OFFSET ELEVATION

DEPTH (FEET)	UNIFIED SOIL CLASSIFICATION	MATERIAL DESCRIPTION	N	N*	Dry Density(pcf)	Moisture Content %	Percent minus 200	Liquid Limit	Plasticity Index
50		- No recovery due to large rock stuck in sampler	100	100					
55	SC	Clayey SAND with gravel - very dense, red-brown with blue-grey and light pink mottling, moist, gravels up to 2 inch in diameter	100	47					
60					123.9	14.7	35.1		
61.5		Bottom of Boring @ 61.5 Feet							

DEPTH: METERS 16, 17, 18, 19, 20, 21, 22; FEET 50, 55, 60, 65, 70

SAMPLES / WATER LEVEL / GRAPHICAL LOG

SPT BLOW COUNT
10 20 30 40 50 60 70 80 90

Moisture Content and Atterberg Limits
Plastic Limit Moisture Content Liquid Limit
10 20 30 40 50 60 70 80 90

N - OBSERVED UNCORRECTED BLOW COUNT
N* - CORRECTED N1(60) EQUIVALENT SPT BLOW COUNT

GeoStrata

SAMPLE TYPE
- 2" O.D./1.38" I.D. Split Spoon Sampler
- 2.5" O.D./2" I.D. California Split Spoon Sampler
- 3" O.D. Thin-Walled Shelby Sampler
- Grab Sample
- 2" O.D./1.625" I.D. Liner Sampler

NOTES:

WATER LEVEL
- MEASURED - ESTIMATED

Plate
B - 3

DATE	
STARTED:	11/9/17
COMPLETED:	11/9/17
BACKFILLED:	11/9/17

Eric Householder
The Bridges Development
Eden, Utah
Project Number 1311-002

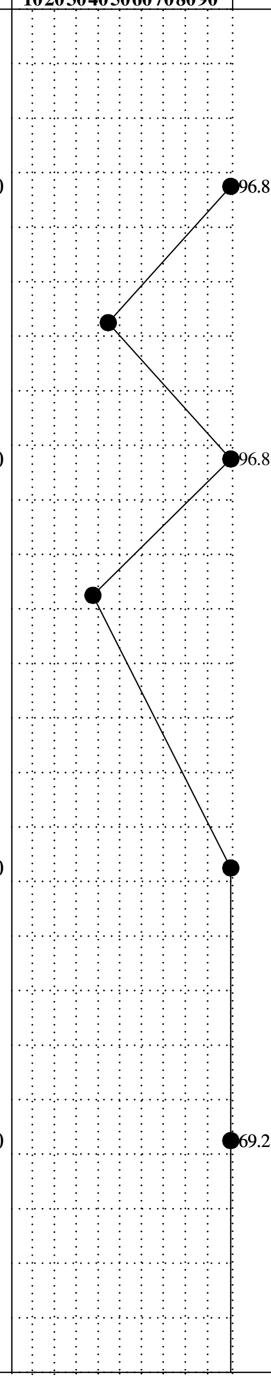
GeoStrata Rep: A. Peay
Rig Type: B-80 Mobile Drill
Boring Type: Auger

BORING NO:
B-2
Sheet 1 of 2

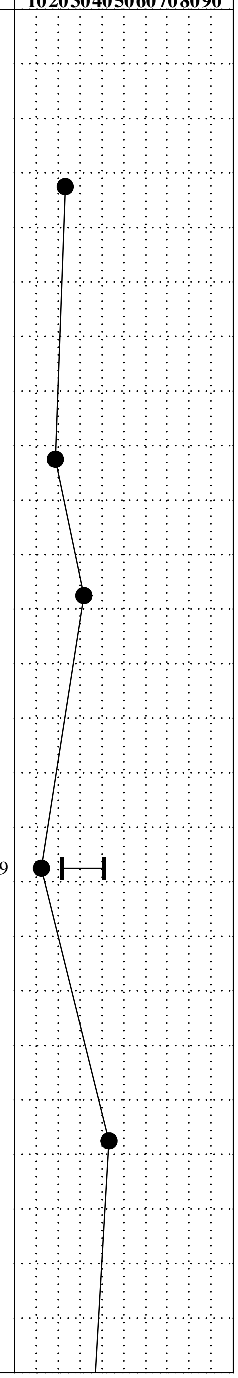
LOCATION
STATION OFFSET ELEVATION

DEPTH (Feet)	SAMPLES	UNIFIED SOIL CLASSIFICATION	MATERIAL DESCRIPTION	N	N*	Dry Density(pcf)	Moisture Content %	Percent minus 200	Liquid Limit	Plasticity Index
0			TOPSOIL; Gravelly Lean CLAY - dark brown, moist, organics throughout, surface cobbles and boulders up to 1.5 feet in diameter							
		SC	Clayey SAND with gravel - hard, medium brown moist, subangular gravels up to 2 inches in diameter - No recovery @ 2.5 Feet due to large rock stuck in sampler							
~3	2" Split Spoon			100	100	96.8	23.2			
~5.5	2" Split Spoon			36	44					
		CL	Sandy Lean CLAY with gravel - hard, medium brown moist, subangular gravels up to 2 inches in diameter - Red-brown with sand and gravel							
~8	2.5" California			100	100	96.8	18.7	45.4		
		CL	Sandy Lean CLAY with gravel - stiff, red brown moist, iron staining throughout							
~10.5	2" Split Spoon			35	37		31.7	54.5		
~15.5	2.5" California			100	100		12.3	54.6	41	19
		CL	Lean CLAY - hard, blue-grey, moist							
			- Soils very moist to wet from 20 feet to 30 feet							
~20.5	2" Split Spoon			100	100	69.2	43.2			

SPT BLOW COUNT
10 20 30 40 50 60 70 80 90



Moisture Content and Atterberg Limits
Plastic Limit Moisture Content Liquid Limit
10 20 30 40 50 60 70 80 90



N - OBSERVED UNCORRECTED BLOW COUNT N* - CORRECTED N1(60) EQUIVALENT SPT BLOW COUNT

GeoStrata

SAMPLE TYPE
- 2" O.D./1.38" I.D. Split Spoon Sampler
- 2.5" O.D./2" I.D. California Split Spoon Sampler
- 3" O.D. Thin-Walled Shelby Sampler
- Grab Sample
- 2" O.D./1.625" I.D. Liner Sampler

NOTES:

WATER LEVEL
- MEASURED - ESTIMATED

Plate
B - 4

DATE		
STARTED:	11/9/17	
COMPLETED:	11/9/17	
BACKFILLED:	11/9/17	

Eric Householder
The Bridges Development
Eden, Utah

Project Number 1311-002

GeoStrata Rep: A. Peay
Rig Type: B-80 Mobile Drill
Boring Type: Auger

BORING NO:
B-2
Sheet 2 of 2

LOCATION
STATION | OFFSET | ELEVATION

DEPTH (FEET)	UNIFIED SOIL CLASSIFICATION	MATERIAL DESCRIPTION	N	N*	Dry Density(pcf)	Moisture Content %	Percent minus 200	Liquid Limit	Plasticity Index
25			100	100					
30		- Blue- grey mottling	100	100					
	GC	Clayey GRAVEL with sand - hard, red-brown, moist							
35			100	100		21.2	42.2	33	17
40			27	42		18.9	47.6		
45	CH	Sandy Fat CLAY with gravel - hard, medium brown moist, subangular gravels up to 2 inches in diameter	100	35		19.4	54.1	57	34
		- Drilling ceased @ 48.5 feet due to extremely hard clay soils							
		Bottom of Boring @ 48.5 Feet							

METERS: 8, 9, 10, 11, 12, 13, 14, 15
FEET: 25, 30, 35, 40, 45

SAMPLES | WATER LEVEL | GRAPHICAL LOG

SPT BLOW COUNT
10 20 30 40 50 60 70 80 90

Moisture Content and Atterberg Limits
Plastic Limit | Moisture Content | Liquid Limit
10 20 30 40 50 60 70 80 90

N - OBSERVED UNCORRECTED BLOW COUNT
N* - CORRECTED N1(60) EQUIVALENT SPT BLOW COUNT

GeoStrata

SAMPLE TYPE
- 2" O.D./1.38" I.D. Split Spoon Sampler
- 2.5" O.D./2" I.D. California Split Spoon Sampler
- 3" O.D. Thin-Walled Shelby Sampler
- Grab Sample
- 2" O.D./1.625" I.D. Liner Sampler

NOTES:

WATER LEVEL
- MEASURED
- ESTIMATED

Plate
B - 5

DATE		
STARTED:	11/9/17	
COMPLETED:	11/9/17	
BACKFILLED:	11/9/17	

Eric Householder
The Bridges Development
Eden, Utah
Project Number 1311-002

GeoStrata Rep: A. Peay
Rig Type: B-80 Mobile Drill
Boring Type: Auger

BORING NO:
B-3
Sheet 1 of 2

LOCATION

STATION | OFFSET | ELEVATION

DEPTH (FEET)	UNIFIED SOIL CLASSIFICATION	MATERIAL DESCRIPTION	N	N*	Dry Density(pcf)	Moisture Content %	Percent minus 200	Liquid Limit	Plasticity Index
0		FILL; Lean CLAY with gravel - medium brown, moist							
~3	GP-GC	Poorly Graded GRAVEL with clay and sand - very dense, white gravel with red-brown sand and clay with occasional blue-grey mottling, moist							
5		- No recovery	100	100					
			100	100					
10			100	100					
~13	SM	Silty SAND with gravel - dense, yellow-brown to red-brown with black mottling, moist							
15			22	25			12.4	NP	NP
20		- Water @ 20 feet			80.0	40.0			

SPT BLOW COUNT: 10 20 30 40 50 60 70 80 90

Moisture Content and Atterberg Limits: Plastic Limit, Moisture Content, Liquid Limit — 10 20 30 40 50 60 70 80 90

N - OBSERVED UNCORRECTED BLOW COUNT

N* - CORRECTED N1(60) EQUIVALENT SPT BLOW COUNT

SAMPLE TYPE
- 2" O.D./1.38" I.D. Split Spoon Sampler
- 2.5" O.D./2" I.D. California Split Spoon Sampler
- 3" O.D. Thin-Walled Shelby Sampler
- Grab Sample
- 2" O.D./1.625" I.D. Liner Sampler

NOTES:

WATER LEVEL
▼ - MEASURED ▽ - ESTIMATED

Plate
B - 6

DATE	
STARTED:	11/9/17
COMPLETED:	11/9/17
BACKFILLED:	11/9/17

Eric Householder
The Bridges Development
Eden, Utah

Project Number 1311-002

GeoStrata Rep: A. Peay
Rig Type: B-80 Mobile Drill
Boring Type: Auger

BORING NO:
B-3
Sheet 2 of 2

LOCATION

STATION OFFSET ELEVATION

DEPTH METERS	DEPTH FEET	SAMPLES	WATER LEVEL	GRAPHICAL LOG	UNIFIED SOIL CLASSIFICATION	MATERIAL DESCRIPTION	N	N*	SPT BLOW COUNT 10 20 30 40 50 60 70 80 90	Dry Density(pcf)	Moisture Content %	Percent minus 200	Liquid Limit	Plasticity Index	Moisture Content and Atterberg Limits (Plastic Limit, Moisture Content, Liquid Limit) 10 20 30 40 50 60 70 80 90
	25				ML	Gravelly SILT with sand - very stiff, yellow-brown to red-brown with black mottling, moist					19.8	56.2	44	15	
8															
9	30					- No recovery									
						Bottom of Boring @ 31.5 Feet									
10															
11	35														
12	40														
13															
14	45														
15															

N - OBSERVED UNCORRECTED BLOW COUNT

N* - CORRECTED N1(60) EQUIVALENT SPT BLOW COUNT

GeoStrata

SAMPLE TYPE
- 2" O.D./1.38" I.D. Split Spoon Sampler
- 2.5" O.D./2" I.D. California Split Spoon Sampler
- 3" O.D. Thin-Walled Shelby Sampler
- Grab Sample
- 2" O.D./1.625" I.D. Liner Sampler

NOTES:

WATER LEVEL
▼ - MEASURED ▽ - ESTIMATED

Plate
B - 7

DATE	
STARTED:	11/9/17
COMPLETED:	11/9/17
BACKFILLED:	11/9/17

Eric Householder
The Bridges Development
Eden, Utah
Project Number 1311-002

GeoStrata Rep: A. Peay
Rig Type: B-80 Mobile Drill
Boring Type: Auger

BORING NO: **B-4**
Sheet 1 of 2

LOCATION: STATION / OFFSET / ELEVATION

DEPTH (Meters)	DEPTH (Feet)	UNIFIED SOIL CLASSIFICATION	MATERIAL DESCRIPTION	N	N*	Dry Density (pcf)	Moisture Content %	Percent minus 200	Liquid Limit	Plasticity Index
0	0		TOPSOIL; Lean CLAY with gravel - dark brown, moist, organics throughout							
		ML	Sandy SILT - hard, red-brown with white carbonate stringers, moist, gravels up to 1/2 inch in diameter							
1				100	100		7.5	57.4	NP	NP
	5	CH	Sandy Fat CLAY with gravel - stiff to hard, red-brown with blue grey and black mottling, moist, sub-angular gravels up to 2.5 inches, driller encountered occcasional boulders up to 3 feet in diameter	35	26	98.5	23.8		50	27
2				22	15		19.5	50.9	55	32
3			- Yellow mottling							
	10			31	21					
4										
	15		- No recovery - Changed from ODEX to Air-Rotary System							
5										
6		GC	Clayey GRAVEL with sand - very dense, red-brown with yellow mottling, moist, subangular gravel up to 4 inches in diameter							
	20			30	20		22.8	30.7	40	19
7										

SPT BLOW COUNT: 10 20 30 40 50 60 70 80 90

Moisture Content and Atterberg Limits: Plastic Limit, Moisture Content, Liquid Limit — 10 20 30 40 50 60 70 80 90

N - OBSERVED UNCORRECTED BLOW COUNT
N* - CORRECTED N1(60) EQUIVALENT SPT BLOW COUNT

SAMPLE TYPE
- 2" O.D./1.38" I.D. Split Spoon Sampler
- 2.5" O.D./2" I.D. California Split Spoon Sampler
- 3" O.D. Thin-Walled Shelby Sampler
- Grab Sample
- 2" O.D./1.625" I.D. Liner Sampler

NOTES:

WATER LEVEL
- MEASURED
- ESTIMATED

GeoStrata

Plate
B - 8

DATE		Eric Householder The Bridges Development Eden, Utah	GeoStrata Rep: A. Peay Rig Type: B-80 Mobile Drill Boring Type: Auger	BORING NO: **B-4**
STARTED:	11/9/17	Project Number 1311-002		Sheet 2 of 2
COMPLETED:	11/9/17			
BACKFILLED:	11/9/17			

LOCATION: STATION / OFFSET / ELEVATION

Depth (feet)	USCS	MATERIAL DESCRIPTION	N	N*	Moisture Content %	Percent minus 200	Liquid Limit	Plasticity Index
25		- No recovery, blow count 50/0	100	100				
30		- No recovery, blow count 50/0	100	100				
35	CH	Sandy Fat CLAY with gravel - stiff to hard, red-brown with blue grey and black mottling, moist	50	31	15.1			
40			37	22	24.1	65.4	58	32
		Bottom of Boring @ 41.5 Feet						

Columns: DEPTH (METERS, FEET), SAMPLES, WATER LEVEL, GRAPHICAL LOG, UNIFIED SOIL CLASSIFICATION, MATERIAL DESCRIPTION, N, N*, SPT BLOW COUNT (10 20 30 40 50 60 70 80 90), Dry Density(pcf), Moisture Content %, Percent minus 200, Liquid Limit, Plasticity Index, Moisture Content and Atterberg Limits (Plastic Limit, Moisture Content, Liquid Limit; 10 20 30 40 50 60 70 80 90)

N - OBSERVED UNCORRECTED BLOW COUNT

N* - CORRECTED N1(60) EQUIVALENT SPT BLOW COUNT

GeoStrata

SAMPLE TYPE
- 2" O.D./1.38" I.D. Split Spoon Sampler
- 2.5" O.D./2" I.D. California Split Spoon Sampler
- 3" O.D. Thin-Walled Shelby Sampler
- Grab Sample
- 2" O.D./1.625" I.D. Liner Sampler

NOTES:

WATER LEVEL
▼ - MEASURED ▽ - ESTIMATED

Plate
B - 9

DATE		
STARTED:	11/9/17	
COMPLETED:	11/9/17	
BACKFILLED:	11/9/17	

Eric Householder
The Bridges Development
Eden, Utah
Project Number 1311-002

GeoStrata Rep: A. Peay
Rig Type: Trackhoe PC460

TEST PIT NO: **TP-1**
Sheet 1 of 1

LOCATION
NORTHING | EASTING | ELEVATION

Depth (feet)	USCS	MATERIAL DESCRIPTION	Dry Density(pcf)	Moisture Content %	Percent minus 200	Liquid Limit	Plasticity Index
0		TOPSOIL; Gravelly Lean CLAY - dark brown, moist, organics throughout, surface cobbles and boulders up to 3 feet in diameter					
	CL	Gravelly Lean CLAY with cobbles and boulders - medium brown to red-brown with occasional blue-grey mottling, moist, iron staining throughout					
	CH	Sandy Fat CLAY - very dense, blue-grey, moist		27.4	50.4	50	25
	CL	Lean CLAY with occasional cobbles, red-brown with black mottling and white carbonate stringers, moist, iron staining throughout, cobbles up to 8 inches in diameter					
18		Bottom of Test Pit @ 18 Feet					

Moisture Content and Atterberg Limits
Plastic Limit — Moisture Content — Liquid Limit
10 20 30 40 50 60 70 80 90

DEPTH: METERS 0 1 2 3 4 5 6 7; FEET 0 5 10 15 20

SAMPLE TYPE
- GRAB SAMPLE
- 3" O.D. THIN-WALLED HAND SAMPLER

WATER LEVEL
- MEASURED
- ESTIMATED

NOTES:

Plate B-10

DATE	
STARTED:	11/9/17
COMPLETED:	11/9/17
BACKFILLED:	11/9/17

Eric Householder
The Bridges Development
Eden, Utah
Project Number 1311-002

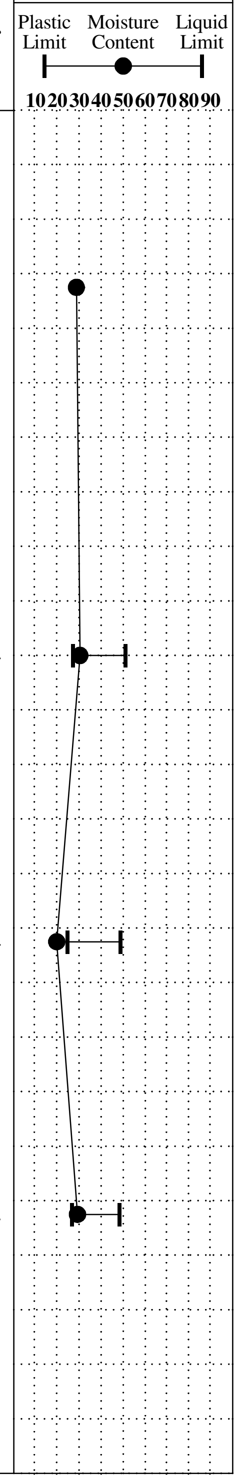
GeoStrata Rep: A. Peay
Rig Type: Trackhoe PC460

TEST PIT NO: **TP-2**
Sheet 1 of 1

LOCATION
NORTHING EASTING ELEVATION

DEPTH (FEET)	SAMPLES	UNIFIED SOIL CLASSIFICATION	MATERIAL DESCRIPTION	Dry Density(pcf)	Moisture Content %	Percent minus 200	Liquid Limit	Plasticity Index
0			TOPSOIL; Gravelly Lean CLAY - dark brown, moist, organics throughout, surface cobbles and boulders up to 3 feet in diameter					
		CL	Lean CLAY with gravel, cobbles and boulders - medium brown to red-brown with occasional blue-grey mottling, moist, iron staining throughout					
	Grab sample			89.7	28.6			
		CL	Gravelly Lean CLAY with cobbles and boulders - medium brown to red-brown, moist, iron staining throughout					
		CL	Lean CLAY with gravel, cobbles and boulders - medium brown to red-brown with occasional blue-grey mottling, moist, iron staining throughout					
10		CH	Sandy Fat CLAY - stiff, light brown with white mottling, moist		30.2	55.0	51	24
15	Grab sample	CL	Sandy Lean CLAY - dense, red-brown with black mottling, moist		19.6		49	24
			- white carbonate stringers					
20	Grab sample	CL	Sandy Lean CLAY - very stiff, red-brown with black mottling, moist		29.1	60.3	48	22
			Bottom of Test Pit @ 23 Feet					

Moisture Content and Atterberg Limits



SAMPLE TYPE
- GRAB SAMPLE
- 3" O.D. THIN-WALLED HAND SAMPLER

WATER LEVEL
- MEASURED
- ESTIMATED

NOTES:

GeoStrata

Plate B-11

DATE		
STARTED:	11/9/17	
COMPLETED:	11/9/17	
BACKFILLED:	11/9/17	

Eric Householder
The Bridges Development
Eden, Utah
Project Number 1311-002

GeoStrata Rep: A. Peay
Rig Type: Trackhoe PC460

TEST PIT NO: **TP-3**
Sheet 1 of 1

LOCATION
NORTHING EASTING ELEVATION

MATERIAL DESCRIPTION

Depth (feet)	Unified Soil Classification	Material Description	Dry Density (pcf)	Moisture Content %	Percent minus 200	Liquid Limit	Plasticity Index
0		TOPSOIL; Gravelly Lean CLAY - dark brown, moist, organics throughout, surface cobbles and boulders up to 3 feet in diameter					
	CL	Lean CLAY with sand and gravel - stiff, red-brown, moist, trace cobbles up to 6 inches in diameter					
	SC	Clayey SAND with gravel - very dense, red-brown, moist, cobbles up to 5 inches in diameter	94.9	23.3	26.1	44	23
10	SC	Clayey SAND - dense, red-brown with white mottling, moist, trace cobbles up to 3 inches in diameter		21.5	29.3	48	25
		- Blue-grey mottling					
15					12.4		
	CL	Sandy Lean CLAY with gravel - hard, red-brown with white mottling, moist, trace cobbles up to 4 inches in diameter			56.2		
22		Bottom of Test Pit @ 22 Feet					

Moisture Content and Atterberg Limits
Plastic Limit — Moisture Content — Liquid Limit
10 20 30 40 50 60 70 80 90

SAMPLE TYPE
- GRAB SAMPLE
- 3" O.D. THIN-WALLED HAND SAMPLER

WATER LEVEL
- MEASURED
- ESTIMATED

NOTES:

GeoStrata

Plate B-12

UNIFIED SOIL CLASSIFICATION SYSTEM

MAJOR DIVISIONS			USCS SYMBOL	TYPICAL DESCRIPTIONS
COARSE GRAINED SOILS (More than half of material is larger than the #200 sieve)	GRAVELS (More than half of coarse fraction is larger than the #4 sieve)	CLEAN GRAVELS WITH LITTLE OR NO FINES	GW	WELL-GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE OR NO FINES
			GP	POORLY-GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE OR NO FINES
		GRAVELS WITH OVER 12% FINES	GM	SILTY GRAVELS, GRAVEL-SILT-SAND MIXTURES
			GC	CLAYEY GRAVELS, GRAVEL-SAND-CLAY MIXTURES
	SANDS (More than half of coarse fraction is smaller than the #4 sieve)	CLEAN SANDS WITH LITTLE OR NO FINES	SW	WELL-GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE OR NO FINES
			SP	POORLY-GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE OR NO FINES
		SANDS WITH OVER 12% FINES	SM	SILTY SANDS, SAND-GRAVEL-SILT MIXTURES
			SC	CLAYEY SANDS SAND-GRAVEL-CLAY MIXTURES
FINE GRAINED SOILS (More than half of material is smaller than the #200 sieve)	SILTS AND CLAYS (Liquid limit less than 50)		ML	INORGANIC SILTS & VERY FINE SANDS, SILTY OR CLAYEY FINE SANDS, CLAYEY SILTS WITH SLIGHT PLASTICITY
			CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
			OL	ORGANIC SILTS & ORGANIC SILTY CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS (Liquid limit greater than 50)		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SAND OR SILT
			CH	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS
			OH	ORGANIC CLAYS & ORGANIC SILTS OF MEDIUM-TO-HIGH PLASTICITY
HIGHLY ORGANIC SOILS			PT	PEAT, HUMUS, SWAMP SOILS WITH HIGH ORGANIC CONTENTS

LOG KEY SYMBOLS

BORING SAMPLE LOCATION

TEST-PIT SAMPLE LOCATION

WATER LEVEL (level after completion)

WATER LEVEL (level where first encountered)

CEMENTATION

DESCRIPTION	DESCRIPTION
WEAKELY	CRUMBLES OR BREAKS WITH HANDLING OR SLIGHT FINGER PRESSURE
MODERATELY	CRUMBLES OR BREAKS WITH CONSIDERABLE FINGER PRESSURE
STRONGLY	WILL NOT CRUMBLE OR BREAK WITH FINGER PRESSURE

OTHER TESTS KEY

C	CONSOLIDATION	SA	SIEVE ANALYSIS
AL	ATTERBERG LIMITS	DS	DIRECT SHEAR
UC	UNCONFINED COMPRESSION	T	TRIAXIAL
S	SOLUBILITY	R	RESISTIVITY
O	ORGANIC CONTENT	RV	R-VALUE
CBR	CALIFORNIA BEARING RATIO	SU	SOLUBLE SULFATES
COMP	MOISTURE/DENSITY RELATIONSHIP	PM	PERMEABILITY
CI	CALIFORNIA IMPACT	-200	% FINER THAN #200
COL	COLLAPSE POTENTIAL	Gs	SPECIFIC GRAVITY
SS	SHRINK SWELL	SL	SWELL LOAD

MODIFIERS

DESCRIPTION	%
TRACE	<5
SOME	5 - 12
WITH	>12

MOISTURE CONTENT

DESCRIPTION	FIELD TEST
DRY	ABSENCE OF MOISTURE, DUSTY, DRY TO THE TOUCH
MOIST	DAMP BUT NO VISIBLE WATER
WET	VISIBLE FREE WATER, USUALLY SOIL BELOW WATER TABLE

STRATIFICATION

DESCRIPTION	THICKNESS	DESCRIPTION	THICKNESS
SEAM	1/16 - 1/2"	OCCASIONAL	ONE OR LESS PER FOOT OF THICKNESS
LAYER	1/2 - 12"	FREQUENT	MORE THAN ONE PER FOOT OF THICKNESS

GENERAL NOTES

1. Lines separating strata on the logs represent approximate boundaries only. Actual transitions may be gradual.
2. No warranty is provided as to the continuity of soil conditions between individual sample locations.
3. Logs represent general soil conditions observed at the point of exploration on the date indicated.
4. In general, Unified Soil Classification designations presented on the logs were evaluated by visual methods only. Therefore, actual designations (based on laboratory tests) may vary.

APPARENT / RELATIVE DENSITY - COARSE-GRAINED SOIL

APPARENT DENSITY	SPT (blows/ft)	MODIFIED CA. SAMPLER (blows/ft)	CALIFORNIA SAMPLER (blows/ft)	RELATIVE DENSITY (%)	FIELD TEST
VERY LOOSE	<4	<4	<5	0 - 15	EASILY PENETRATED WITH 1/2-INCH REINFORCING ROD PUSHED BY HAND
LOOSE	4 - 10	5 - 12	5 - 15	15 - 35	DIFFICULT TO PENETRATE WITH 1/2-INCH REINFORCING ROD PUSHED BY HAND
MEDIUM DENSE	10 - 30	12 - 35	15 - 40	35 - 65	EASILY PENETRATED A FOOT WITH 1/2-INCH REINFORCING ROD DRIVEN WITH 5-LB HAMMER
DENSE	30 - 50	35 - 60	40 - 70	65 - 85	DIFFICULT TO PENETRATED A FOOT WITH 1/2-INCH REINFORCING ROD DRIVEN WITH 5-LB HAMMER
VERY DENSE	>50	>60	>70	85 - 100	PENETRATED ONLY A FEW INCHES WITH 1/2-INCH REINFORCING ROD DRIVEN WITH 5-LB HAMMER

CONSISTENCY - FINE-GRAINED SOIL

CONSISTENCY	SPT (blows/ft)	TORVANE UNTRAINED SHEAR STRENGTH (tsf)	POCKET PENETROMETER UNCONFINED COMPRESSIVE STRENGTH (tsf)	FIELD TEST
VERY SOFT	<2	<0.125	<0.25	EASILY PENETRATED SEVERAL INCHES BY THUMB. EXUDES BETWEEN THUMB AND FINGERS WHEN SQUEEZED BY HAND.
SOFT	2 - 4	0.125 - 0.25	0.25 - 0.5	EASILY PENETRATED ONE INCH BY THUMB. MOLDED BY LIGHT FINGER PRESSURE.
MEDIUM STIFF	4 - 8	0.25 - 0.5	0.5 - 1.0	PENETRATED OVER 1/2 INCH BY THUMB WITH MODERATE EFFORT. MOLDED BY STRONG FINGER PRESSURE.
STIFF	8 - 15	0.5 - 1.0	1.0 - 2.0	INDENTED ABOUT 1/2 INCH BY THUMB BUT PENETRATED ONLY WITH GREAT EFFORT.
VERY STIFF	15 - 30	1.0 - 2.0	2.0 - 4.0	READILY INDENTED BY THUMBNAIL.
HARD	>30	>2.0	>4.0	INDENTED WITH DIFFICULTY BY THUMBNAIL.

Boring / Test Pit No.	Sample Depth (feet)	USCS Soil Classification	Natural Moisture Content (%)	Natural Dry Density (pcf)	Gradation			Atterberg		Consolidation			Swell (%)	Residual Shear	
					Gravel (%)	Sand (%)	Fines (%)	LL	PI	Cc	Cr	OCR		Cohseion (psf)	Friction Angle (°)
B-1	2.5	MH	23.5		9.8	29.3	60.9	51	22						
B-1	7.5	GM	23.2		36.7	31.7	31.6	54	17						
B-1	20	CL	14.3	119.2	1.1	29.2	69.7	42	22						
B-1	30	GP-GC	8.1		51.3	34.3	14.4								
B-1	45	GP-GC	7.2		68.0	22.2	9.8	41	25						
B-1	60	SC	14.7	123.9	29.4	35.5	35.1								
B-2	2.5	CL	23.2	96.8											
B-2	7.5	ML	18.7	96.8	18.9	35.7	45.4						1.16		
B-2	10	CL	31.7		0.0	45.5	54.5								
B-2	15	CL	12.3		19.8	25.6	54.6	41	19						
B-2	20	CL	43.2	69.2											
B-2	35	GC	21.2		35.2	22.6	42.2	33	17						
B-2	40	CL	18.9		25.9	26.5	47.6								
B-3	15	SM	8.7		40.4	47.2	12.4								
B-3	25	CL	19.3		25.7	18.1	56.2								
B-2	45	CH	19.4		20.3	25.6	54.1	57	34					70	15
B-3	15	SM			40.4	47.2	12.4	NP	NP					190	18
B-3	20	SM	40	80											
B-3	25	ML	19.8		25.7	18.1	56.2	44	15						
B-4	2.5	CL	7.5		0.0	42.6	57.4	NP	NP						
B-4	5	CL	23.8	98.5				50	27				0.73		
B-4	7.5	CH	19.5		18.3	30.8	50.9	55	32						
B-4	20	GC	22.8		36.4	32.9	30.7	40	19						
B-4	35	CH	15.1												
B-4	40	CH	24.1				65.4	58	32						
TP-1	7	CH	27.4		10.1	39.5	50.4	50	25						
TP-2	3	CL	28.6	89.7									0.01		
TP-2	10	CH	30.2		0.0	45.0	55.0	51	24					145	18
TP-2	15	CL	19.6					49	25						
TP-2	20	CL	29.1		0.0	39.7	60.3	48	21						
TP-3	7	SC	23.3	94.9	18.8	55.1	26.1	44	23	0.087	0.012	5			
TP-3	10	SC	21.5		13.3	57.4	29.3	48	25						
TP-3	15	SC			40.4	47.2	12.4								
TP-3	21	CL			25.7	18.1	56.2								

Lab Summary

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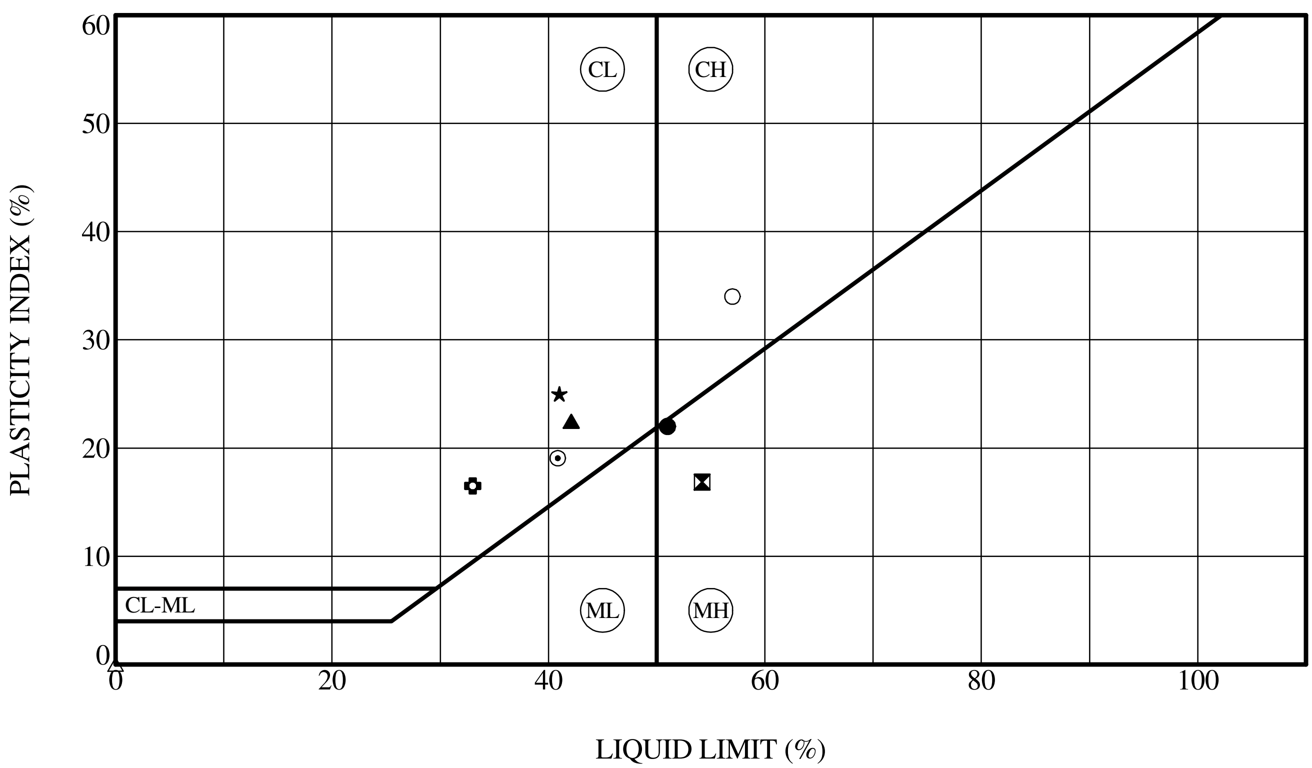
Project Number: 1311-002

Plate C - 1



	Sample Location	Depth (ft)	LL (%)	PL (%)	PI (%)	Fines (%)	Classification
●	**B-1**	**2.5**	**51**	**29**	**22**	**60.9**	**Sandy Elastic SILT**
⊠	**B-1**	**7.5**	**54**	**37**	**17**	**31.6**	**Silty GRAVEL with sand**
▲	**B-1**	**20.0**	**42**	**20**	**22**	**69.7**	**Sandy Lean CLAY**
★	**B-1**	**45.0**	**41**	**16**	**25**	**9.8**	**Poorly Graded GRAVEL with clay and sand**
⊙	**B-2**	**15.0**	**41**	**22**	**19**	**54.6**	**Sandy Lean CLAY with gravel**
✿	**B-2**	**35.0**	**33**	**17**	**17**	**42.2**	**Clayey GRAVEL with sand**
○	**B-2**	**45.0**	**57**	**23**	**34**	**54.1**	**Sandy Fat CLAY with gravel**
△	**B-3**	**15.0**	**NP**	**NP**	**NP**	**12.4**	**Silty SAND with gravel**

ATTERBERG LIMITS' RESULTS - ASTM D 4318

GeoStrata

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Project Number: 1311-002

Plate

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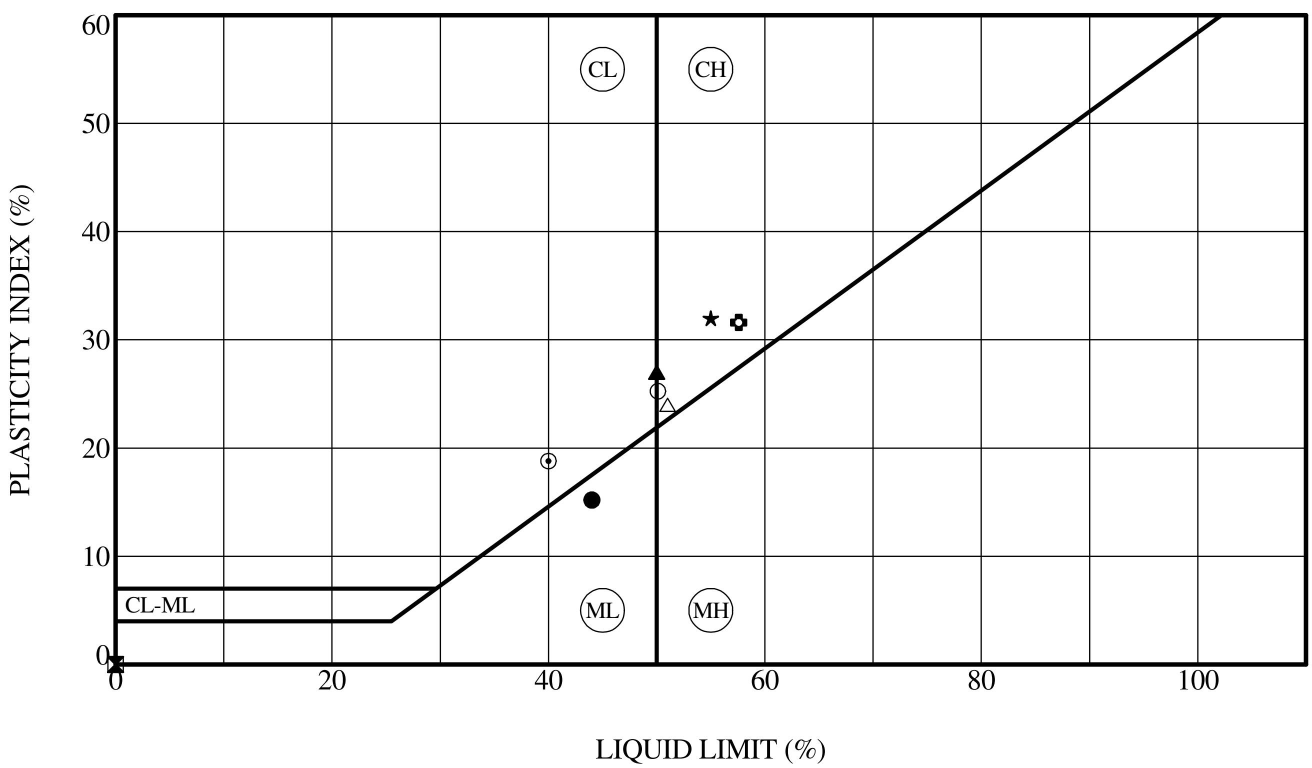

	Sample Location	Depth (ft)	LL (%)	PL (%)	PI (%)	Fines (%)	Classification
●	**B-3**	**25.0**	**44**	**29**	**15**	**56.2**	**Gravelly SILT with sand**
⊠	**B-4**	**2.5**	**NP**	**NP**	**NP**	**57.4**	**Sandy SILT**
▲	**B-4**	**5.0**	**50**	**23**	**27**		**Sandy Fat CLAY with gravel**
★	**B-4**	**7.5**	**55**	**23**	**32**	**50.9**	**Sandy Fat CLAY with gravel**
⊙	**B-4**	**20.0**	**40**	**21**	**19**	**30.7**	**Clayey GRAVEL with sand**
✿	**B-4**	**40.0**	**58**	**26**	**32**	**65.4**	**Sandy Fat CLAY with gravel**
○	**TP-1**	**7.0**	**50**	**25**	**25**	**50.4**	**Sandy Fat CLAY**
△	**TP-2**	**10.0**	**51**	**27**	**24**	**55.0**	**Sandy Fat CLAY**

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ATTERBERG LIMITS' RESULTS - ASTM D 4318

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Project Number: 1311-002

Plate

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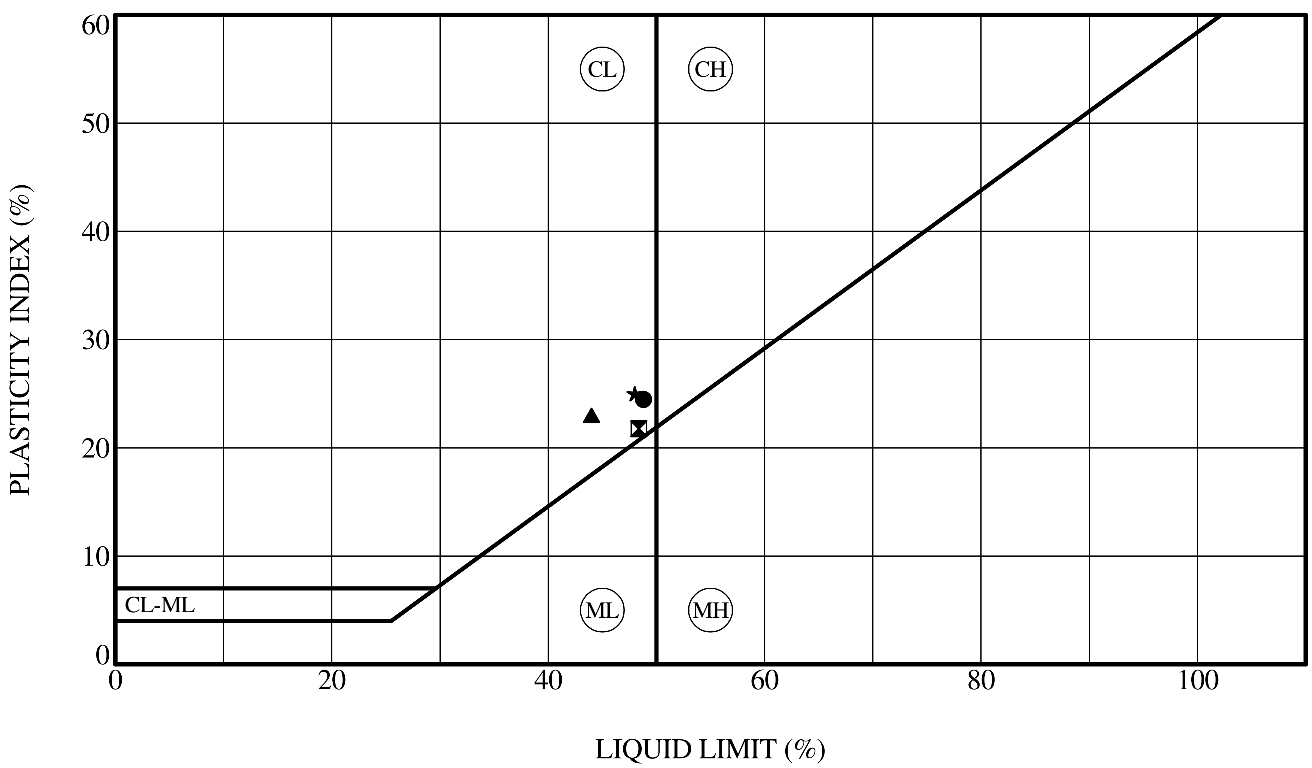

	Sample Location	Depth (ft)	LL (%)	PL (%)	PI (%)	Fines (%)	Classification
●	**TP-2**	**15.0**	**49**	**24**	**25**		**Sandy Lean CLAY**
⊠	**TP-2**	**20.0**	**48**	**27**	**21**	**60.3**	**Sandy Lean CLAY**
▲	**TP-3**	**7.0**	**44**	**21**	**23**	**26.1**	**Clayey SAND with gravel**
★	**TP-3**	**10.0**	**48**	**23**	**25**	**29.3**	**Clayey SAND**

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ATTERBERG LIMITS' RESULTS - ASTM D 4318

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Project Number: 1311-002

Plate
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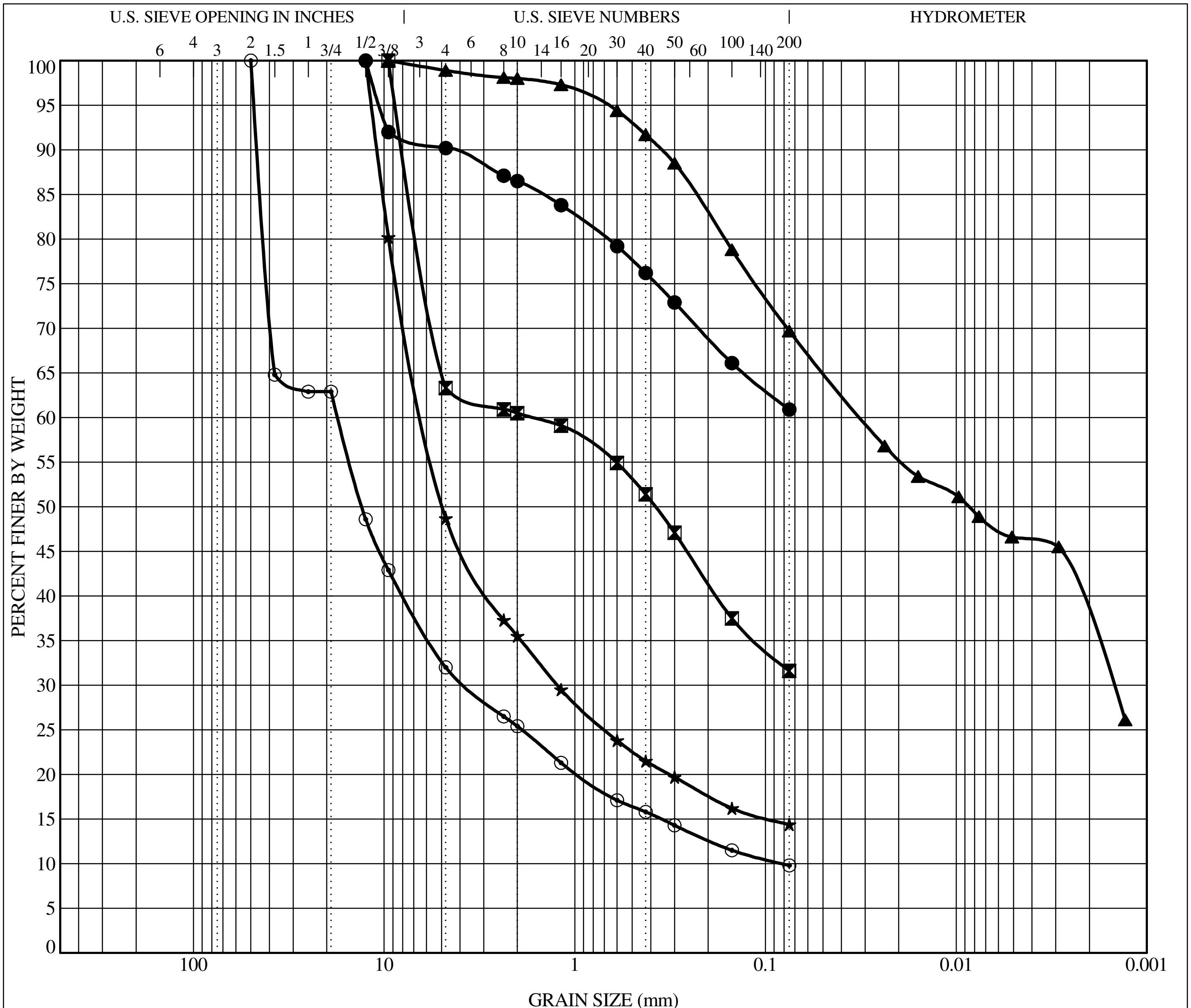
C_ATTERBERG BORING AND TP LOGS.GPJ GEOSTRATA.GDT 2/2/18



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	Sample Location	Depth	Classification	LL	PL	PI	Cc	Cu
●	**B-1**	**2.5**	**Sandy Elastic SILT**	**51**	**29**	**22**		
⧗	**B-1**	**7.5**	**Silty GRAVEL with sand**	**54**	**37**	**17**		
▲	**B-1**	**20.0**	**Sandy Lean CLAY**	**42**	**20**	**22**		
★	**B-1**	**30.0**	**Poorly Graded GRAVEL with clay and sand**					
⊙	**B-1**	**45.0**	**Poorly Graded GRAVEL with clay and sand**	**41**	**16**	**25**	**9.55**	**214.49**

	Sample Loctaion	Depth	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**B-1**	**2.5**	**12.5**				**9.8**	**29.3**	**60.9**	
⧗	**B-1**	**7.5**	**9.5**	**1.657**			**36.7**	**31.7**	**31.6**	
▲	**B-1**	**20.0**	**9.5**	**0.032**	**0.002**		**1.1**	**29.2**	**33.2**	**36.5**
★	**B-1**	**30.0**	**12.5**	**6.091**	**1.233**		**51.3**	**34.3**	**14.4**	
⊙	**B-1**	**45.0**	**50**	**17.453**	**3.683**	**0.081**	**68.0**	**22.2**	**9.8**	

GeoStrata

GRAIN SIZE DISTRIBUTION - ASTM D422

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Plate

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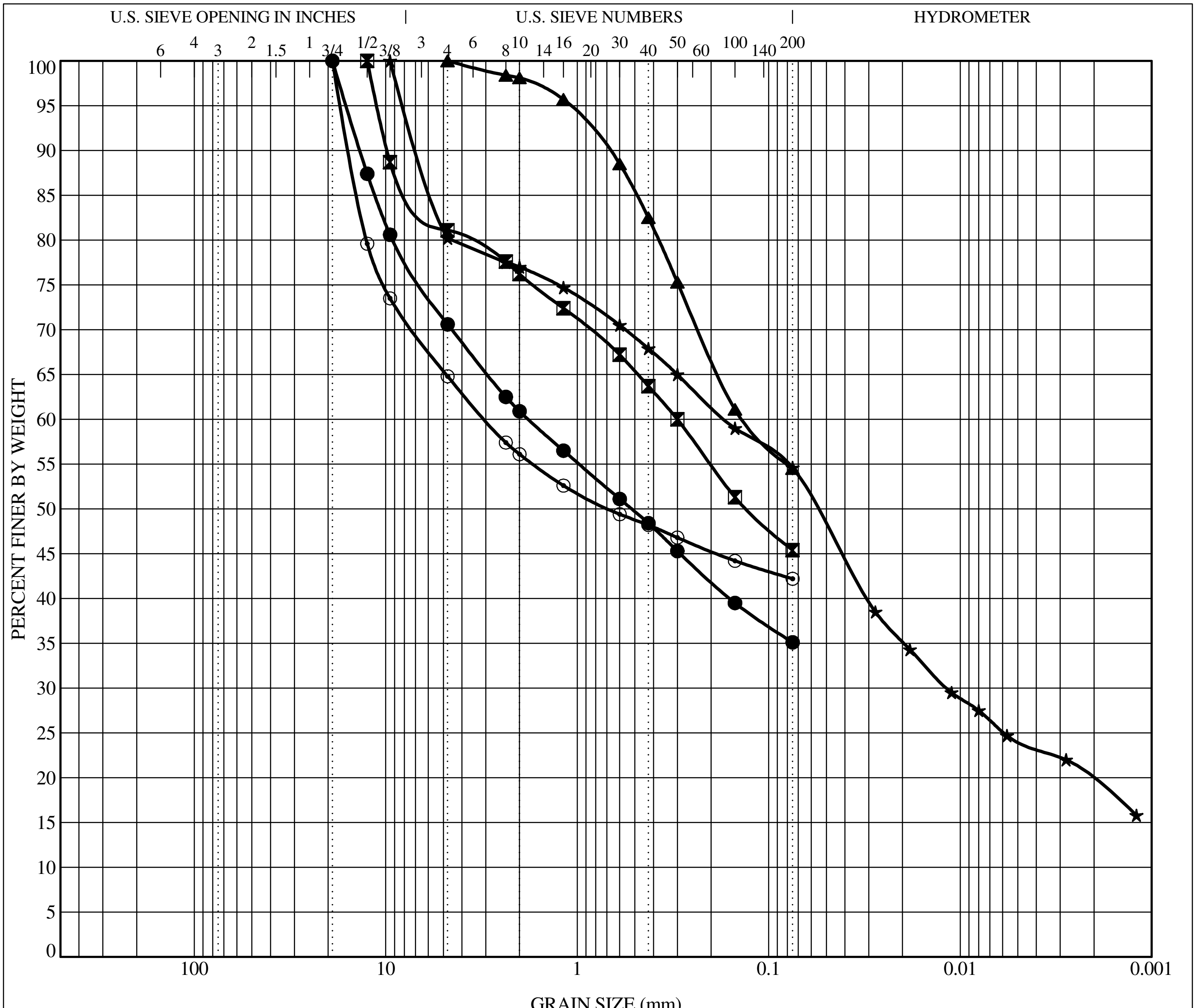

COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	Sample Location	Depth	Classification	LL	PL	PI	Cc	Cu
●	**B-1**	**60.0**	**Clayey SAND with gravel**					
⊠	**B-2**	**7.5**	**Clayey SAND with gravel**					
▲	**B-2**	**10.0**	**Sandy Lean CLAY**					
★	**B-2**	**15.0**	**Sandy Lean CLAY with gravel**	**41**	**22**	**19**		
⊙	**B-2**	**35.0**	**Clayey GRAVEL with sand**	**33**	**17**	**17**		

	Sample Loctaion	Depth	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**B-1**	**60.0**	**19**	**1.795**			**29.4**	**35.5**	**35.1**	
⊠	**B-2**	**7.5**	**12.5**	**0.3**			**18.9**	**35.7**	**45.4**	
▲	**B-2**	**10.0**	**4.75**	**0.134**			**0.0**	**45.5**	**54.5**	
★	**B-2**	**15.0**	**9.5**	**0.168**	**0.012**		**19.8**	**25.6**	**35.1**	**19.5**
⊙	**B-2**	**35.0**	**19**	**3.017**			**35.2**	**22.6**	**42.2**	

GRAIN SIZE DISTRIBUTION - ASTM D422

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Project Number: 1311-002

Plate

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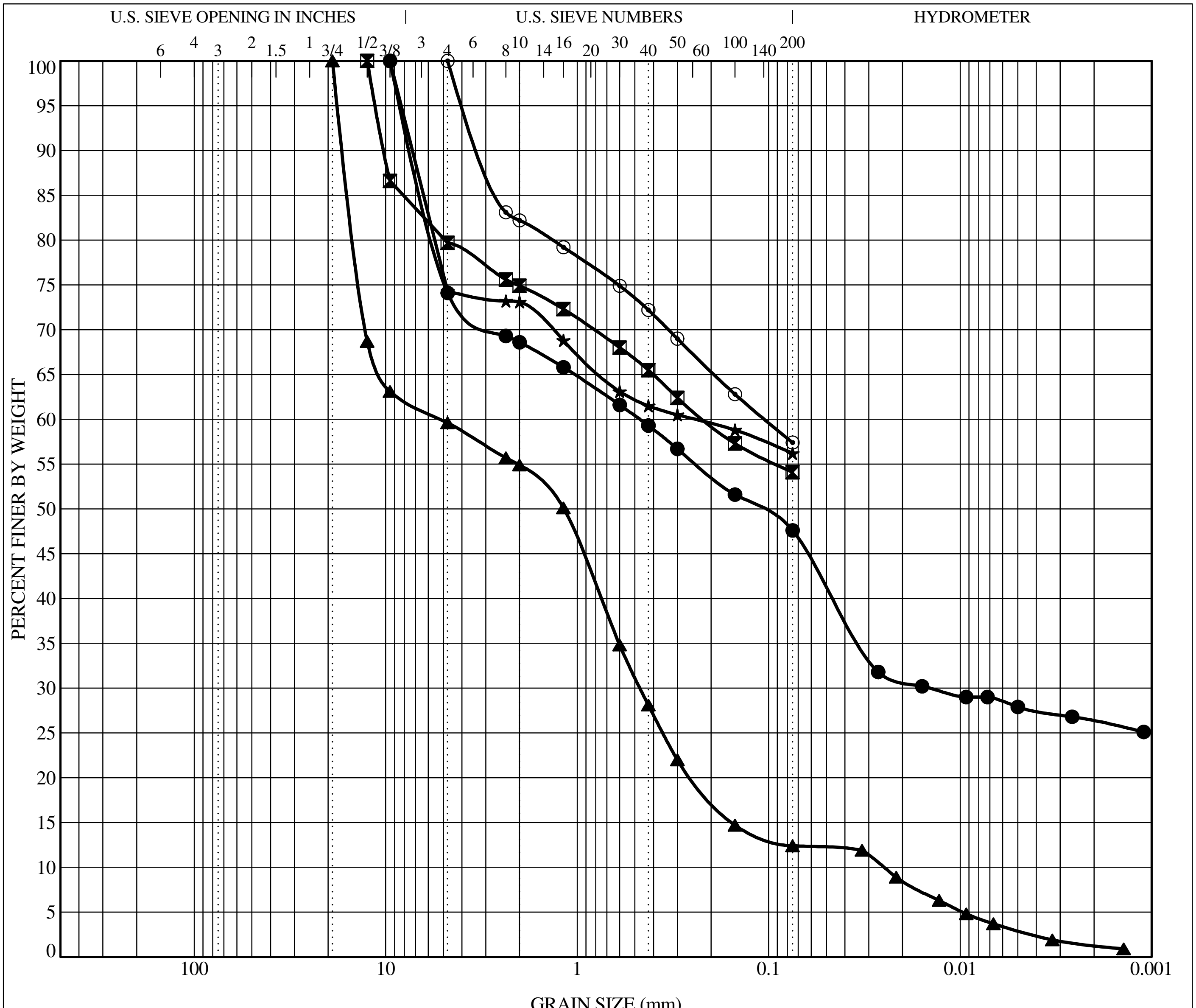
C_GSD BORING AND TP LOGS.GPJ GEOSTRATA.GDT 2/2/18



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

Sample Location		Depth	Classification	LL	PL	PI	Cc	Cu
●	**B-2**	**40.0**	**Sandy Lean CLAY with gravel**					
⊠	**B-2**	**45.0**	**Sandy Fat CLAY with gravel**	**57**	**23**	**34**		
▲	**B-3**	**15.0**	**Silty SAND with gravel**	**NP**	**NP**	**NP**	**1.70**	**204.69**
★	**B-3**	**25.0**	**Gravelly SILT with sand**	**44**	**29**	**15**		
⊙	**B-4**	**2.5**	**Sandy SILT**	**NP**	**NP**	**NP**		

Sample Loctaion		Depth	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**B-2**	**40.0**	**9.5**	**0.472**	**0.014**		**25.9**	**26.5**	**21.3**	**26.3**
⊠	**B-2**	**45.0**	**12.5**	**0.217**			**20.3**	**25.6**	**54.1**	
▲	**B-3**	**15.0**	**19**	**5.142**	**0.469**	**0.025**	**40.4**	**47.2**	**11.1**	**1.3**
★	**B-3**	**25.0**	**9.5**	**0.245**			**25.7**	**18.1**	**56.2**	
⊙	**B-4**	**2.5**	**4.75**	**0.105**			**0.0**	**42.6**	**57.4**	

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GRAIN SIZE DISTRIBUTION - ASTM D422

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Plate

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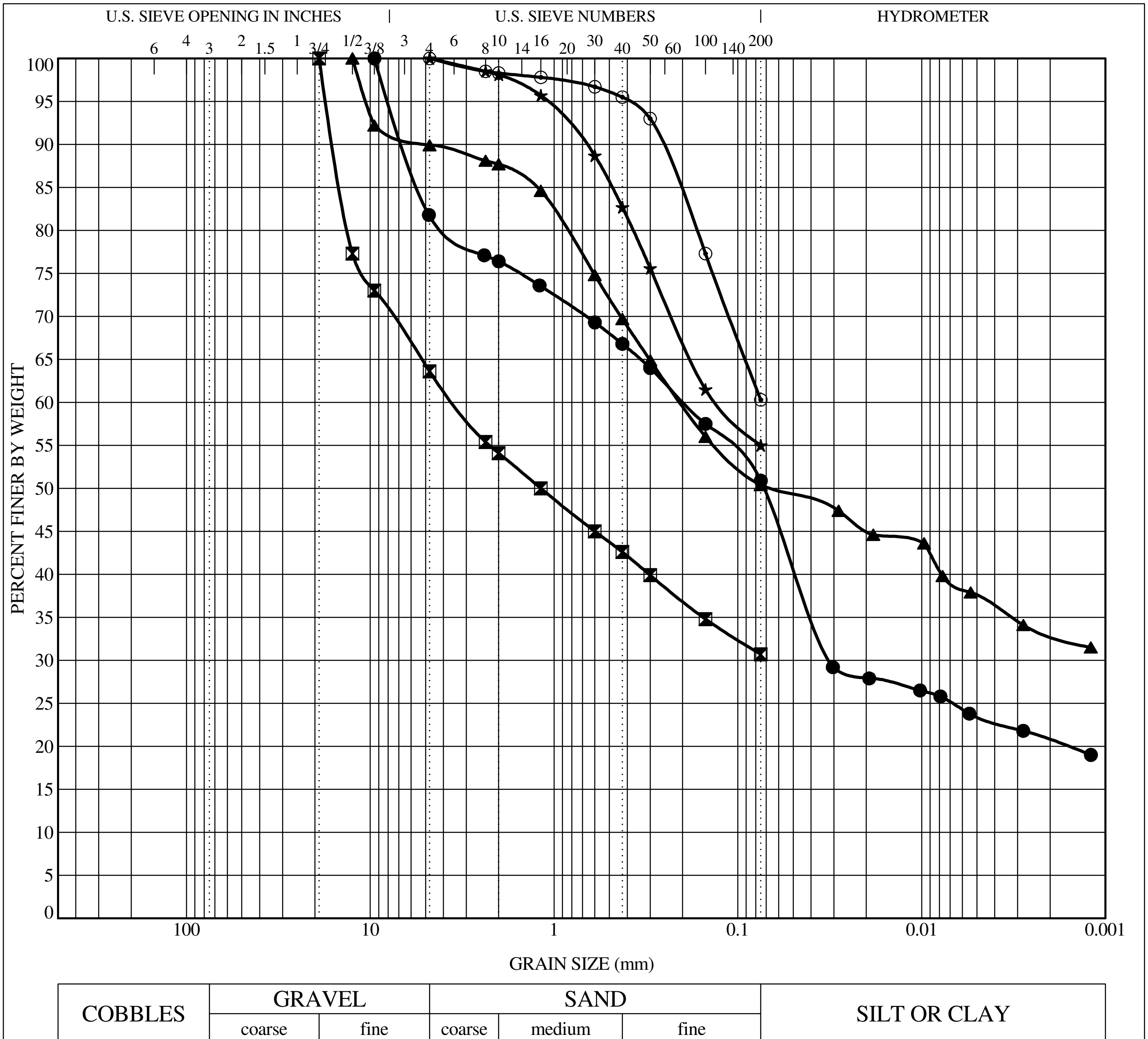

	Sample Location	Depth	Classification	LL	PL	PI	Cc	Cu
●	**B-4**	**7.5**	**Sandy Fat CLAY with gravel**	**55**	**23**	**32**		
⊠	**B-4**	**20.0**	**Clayey GRAVEL with sand**	**40**	**21**	**19**		
▲	**TP-1**	**7.0**	**Sandy Fat CLAY**	**50**	**25**	**25**		
★	**TP-2**	**10.0**	**Sandy Fat CLAY**	**51**	**27**	**24**		
⊙	**TP-2**	**20.0**	**Sandy Lean CLAY**	**48**	**27**	**21**		

	Sample Loctaion	Depth	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**B-4**	**7.5**	**9.5**	**0.196**	**0.031**		**18.3**	**30.8**	**30.2**	**20.7**
⊠	**B-4**	**20.0**	**19**	**3.494**			**36.4**	**32.9**	**30.7**	
▲	**TP-1**	**7.0**	**12.5**	**0.205**			**10.1**	**39.5**	**17.3**	**33.1**
★	**TP-2**	**10.0**	**4.75**	**0.128**			**0.0**	**45.0**	**55.0**	
⊙	**TP-2**	**20.0**	**4.75**				**0.0**	**39.7**	**60.3**	

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GRAIN SIZE DISTRIBUTION - ASTM D422

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Plate

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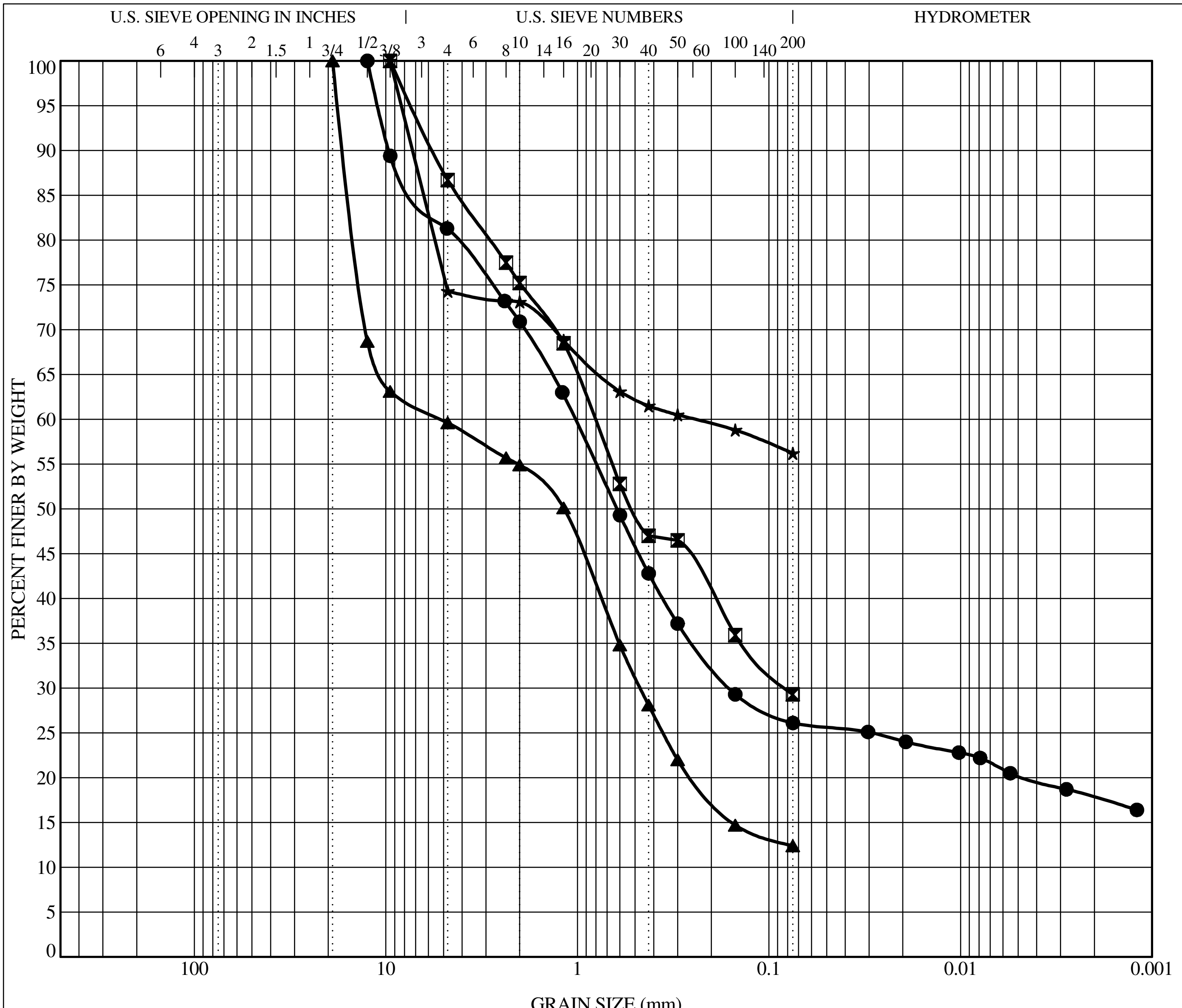
C_GSD BORING AND TP LOGS.GPJ GEOSTRATA.GDT 2/2/18



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	Sample Location	Depth	Classification	LL	PL	PI	Cc	Cu
●	TP-3	7.0	**Clayey SAND with gravel**	**44**	**21**	**23**		
⊠	TP-3	10.0	**Clayey SAND**	**48**	**23**	**25**		
▲	TP-3	15.0	**Clayey SAND with gravel**				**1.17**	**141.30**
★	TP-3	21.0	**Gravelly Lean CLAY with sand**					

	Sample Loctaion	Depth	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	TP-3	7.0	**12.5**	**1.031**	**0.16**		**18.8**	**55.1**	**8.3**	**17.8**
⊠	TP-3	10.0	**9.5**	**0.818**	**0.081**		**13.3**	**57.4**	**29.3**	
▲	TP-3	15.0	**19**	**5.142**	**0.469**		**40.4**	**47.2**	**12.4**	
★	TP-3	21.0	**9.5**	**0.245**			**25.7**	**18.1**	**56.2**	

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GRAIN SIZE DISTRIBUTION - ASTM D422

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Plate
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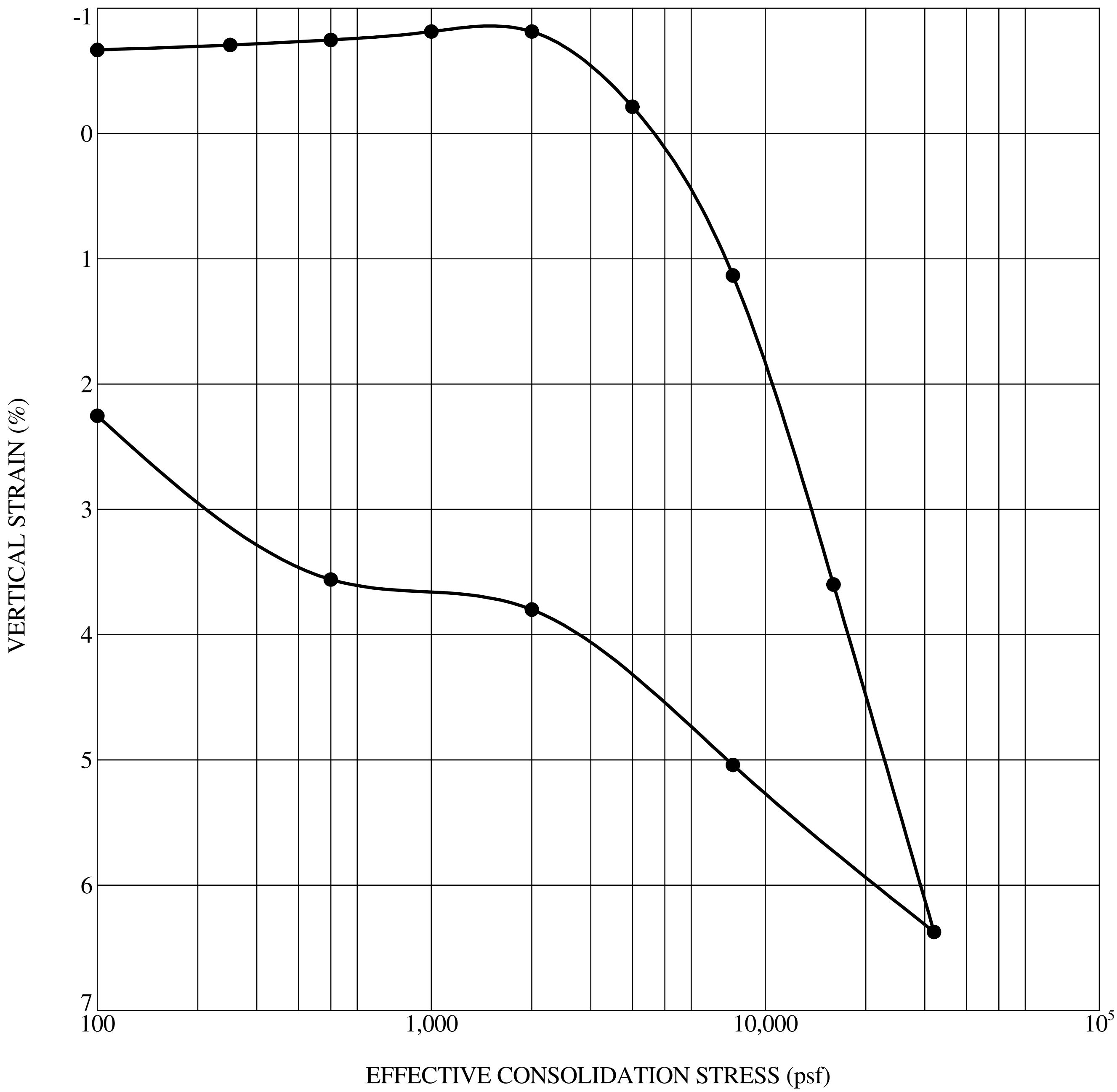

	Sample Location	Depth (ft)	Classification	γ_d (pcf)	MC (%)	C'_c	C'_r	OCR
●	**TP-3**	**7.0**	**Clayey SAND with gravel**	**95**	**24**	**0.087**	**0.012**	**5.0**

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1-D CONSOLIDATION TEST - ASTM D 2435

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Project Number: 1311-002

Plate

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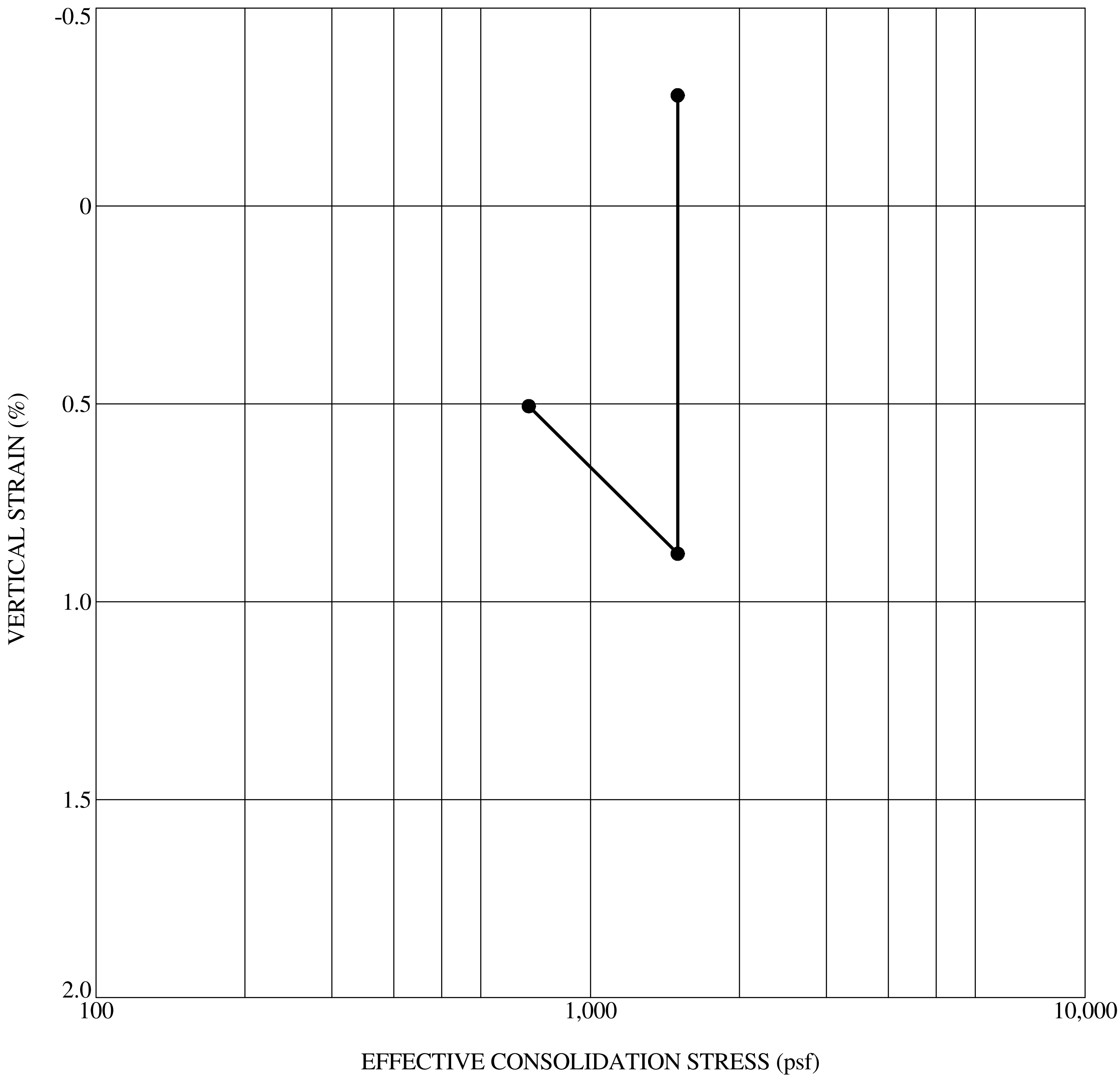

	Sample Location	Depth (ft)	Classification	γ_d (pcf)	MC (%)	Inundation Load (psf)	Swell (%)	Collapse (%)
●	**B-2**	**7.5**	**Clayey SAND with gravel**	**97**	**23**	**1500**	**1.16**	

GeoStrata

1-D SWELL/COLLAPSE TEST

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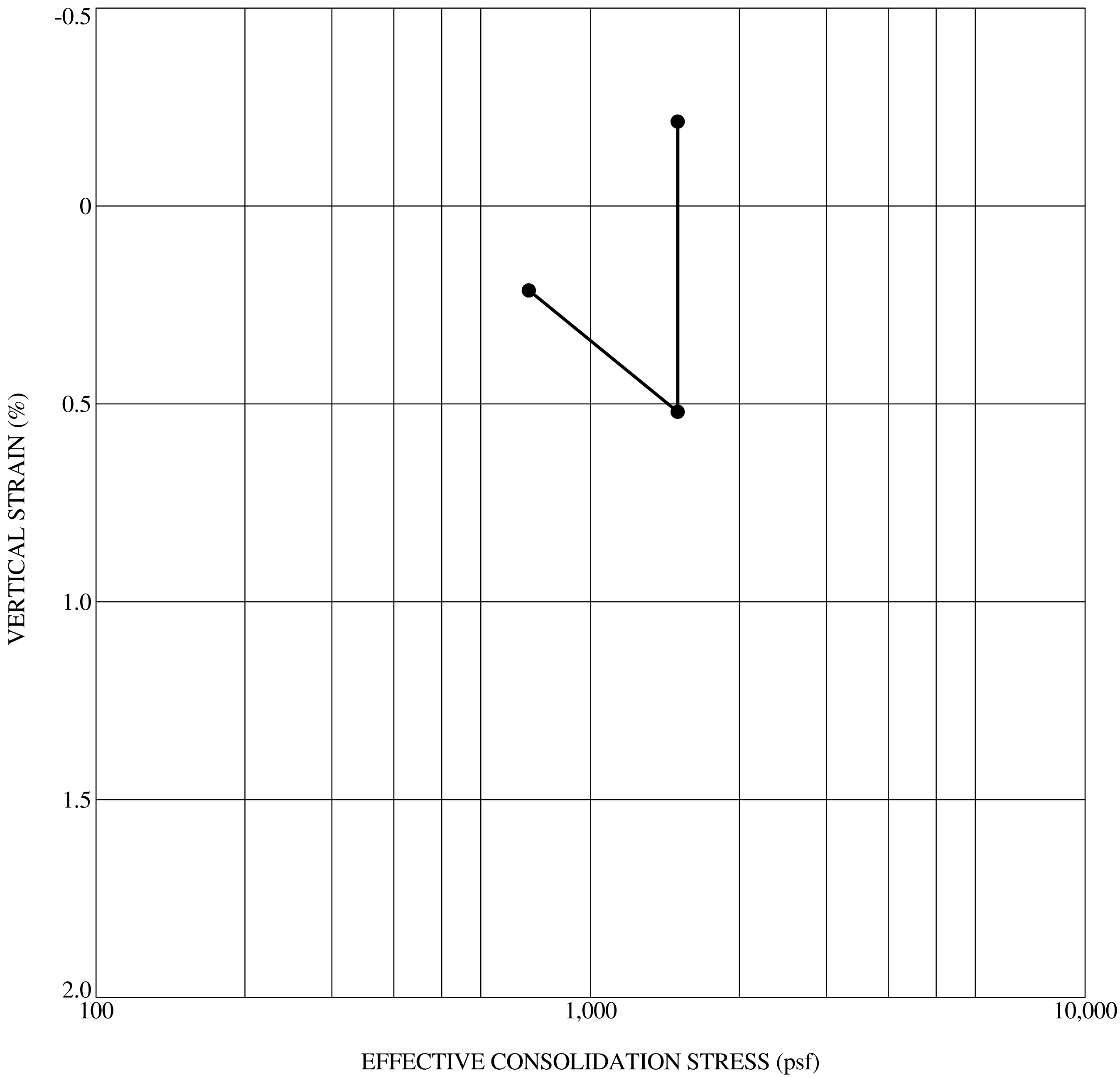
Project Number: 1311-002

Plate C - 11

C_SWELL/COLLAPSE BORING AND TP LOGS.GPJ GEOSTRATA.GDT 2/2/18



	Sample Location	Depth (ft)	Classification	γ_d (pcf)	MC (%)	Inundation Load (psf)	Swell (%)	Collapse (%)
●	**B-4**	**5.0**	**Sandy Fat CLAY with gravel**	**98**	**24**	**1500**	**0.73**	

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1-D SWELL/COLLAPSE TEST

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Project Number: 1311-002

Plate

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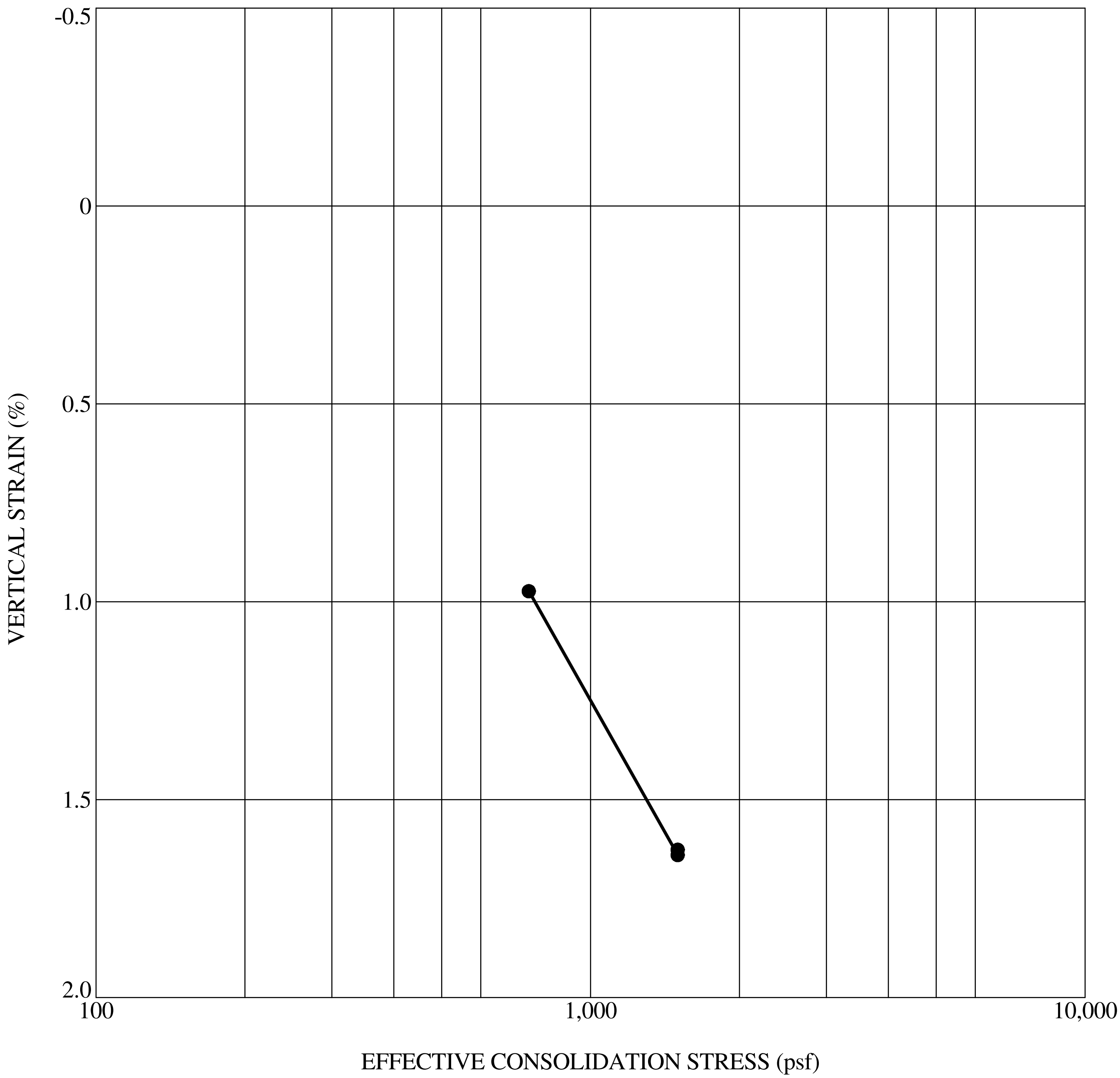

	Sample Location	Depth (ft)	Classification	γ_d (pcf)	MC (%)	Inundation Load (psf)	Swell (%)	Collapse (%)
●	**TP-2**	**3.0**	**Lean CLAY with gravel**	**90**	**29**	**15**	**0.01**	

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1-D SWELL/COLLAPSE TEST

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Plate

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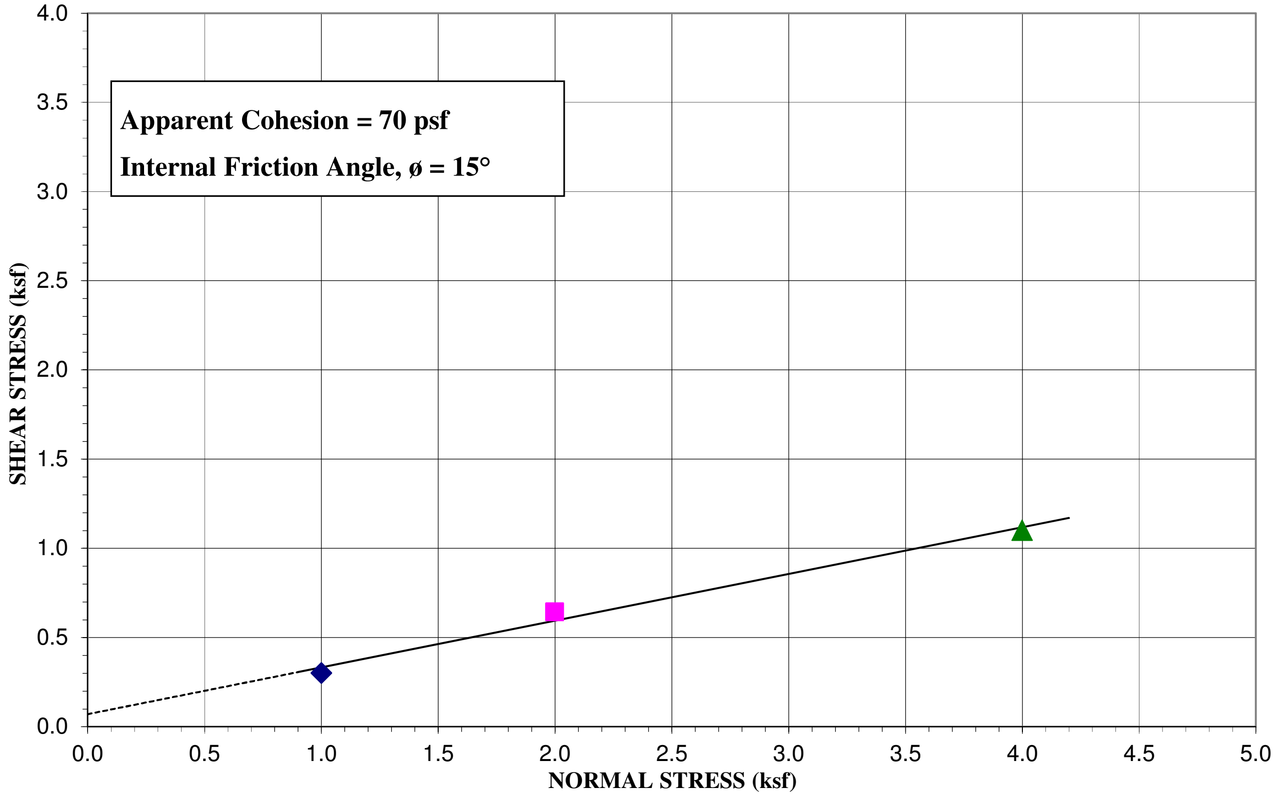
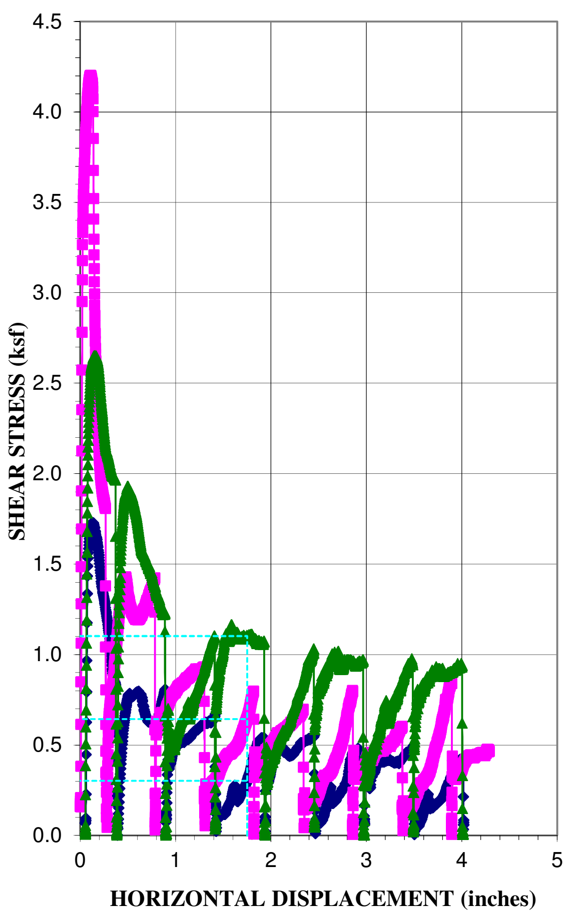
DIRECT SHEAR TEST





Source:	B-2 @ 45
Type of Test:	Consolidated Drained/Saturated

Test No. (Symbol)	**1 (◆)**	**2 (■)**	**3 (▲)**
Sample Type	Remolded		
Initial Height, in.	1.063	1.12	1.056
Diameter, in.	2.5	2.5	2.5
Dry Density Before, pcf	85.3	87.0	87.9
Dry Density After, pcf	94.7	94.1	89.3
Moisture % Before	25.0	24.1	24.9
Moisture % After	30.3	30.2	31.8
Saturation, % Before	70.5	71.0	75.0
Saturation, % After	107.5	105.7	99.1
Normal Load, ksf	1.0	2.0	4.0
Shear Stress, ksf	0.30	0.64	1.10
Strain Rate	0.0033 IN/MIN		

Sample Properties	
Cohesion, psf	70
Friction Angle, ϕ	15
Liquid Limit, %	57
Plasticity Index, %	34
Percent Gravel	20.3
Percent Sand	25.6
Percent Passing No. 200 sieve	54.1
Classification	CH

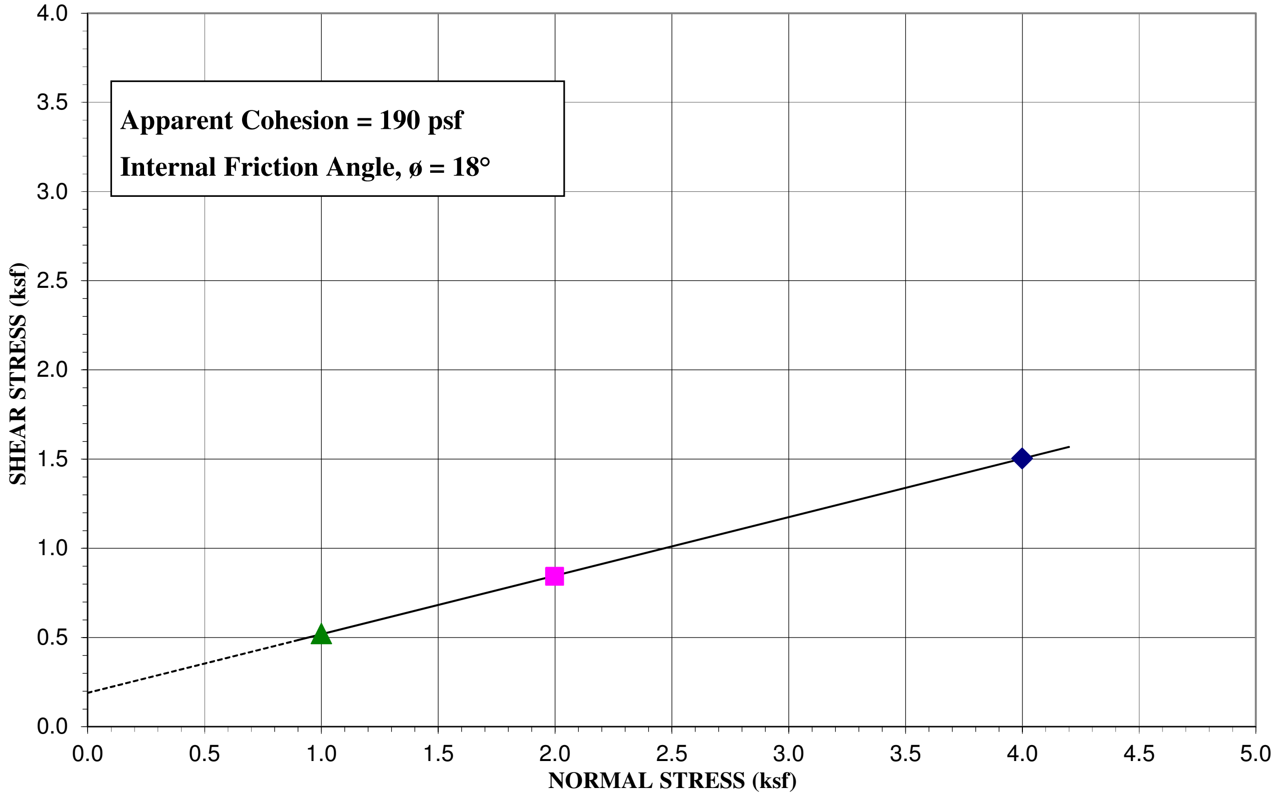
PROJECT: The Bridges Development

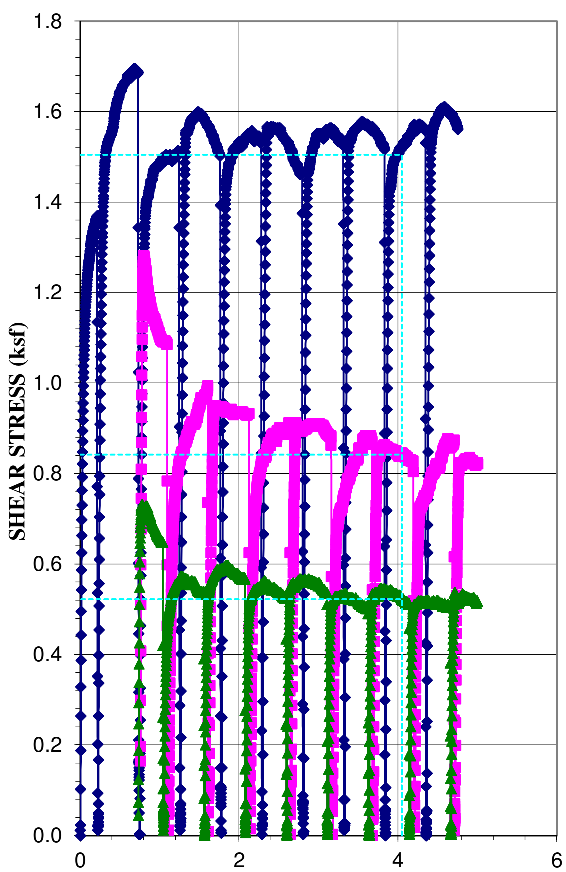
PROJECT NO.: **1311-002**

GeoStrata

Plate C-14

DIRECT SHEAR TEST





Source:	B-3 @ 15 ft
Type of Test:	Consolidated Drained/Saturated

Test No. (Symbol)	**1 (◆)**	**2 (■)**	**3 (▲)**
Sample Type	Remolded		
Initial Height, in.	1.061	1.051	1.132
Diameter, in.	2.5	2.5	2.5
Dry Density Before, pcf	80.1	83.2	84.1
Dry Density After, pcf	81.7	84.8	85.6
Moisture % Before	39.6	37.7	35.2
Moisture % After	36.8	38.6	38.0
Saturation, % Before	98.6	101.1	96.6
Saturation, % After	95.2	107.6	108.1
Normal Load, ksf	4.0	2.0	1.0
Shear Stress, ksf	1.50	0.84	0.52
Strain Rate	0.0033 IN/MIN		

Sample Properties	
Cohesion, psf	190
Friction Angle, ϕ	18
Liquid Limit, %	NP
Plasticity Index, %	NP
Percent Gravel	40.4
Percent Sand	47.2
Percent Passing No. 200 sieve	12.4
Classification	SM

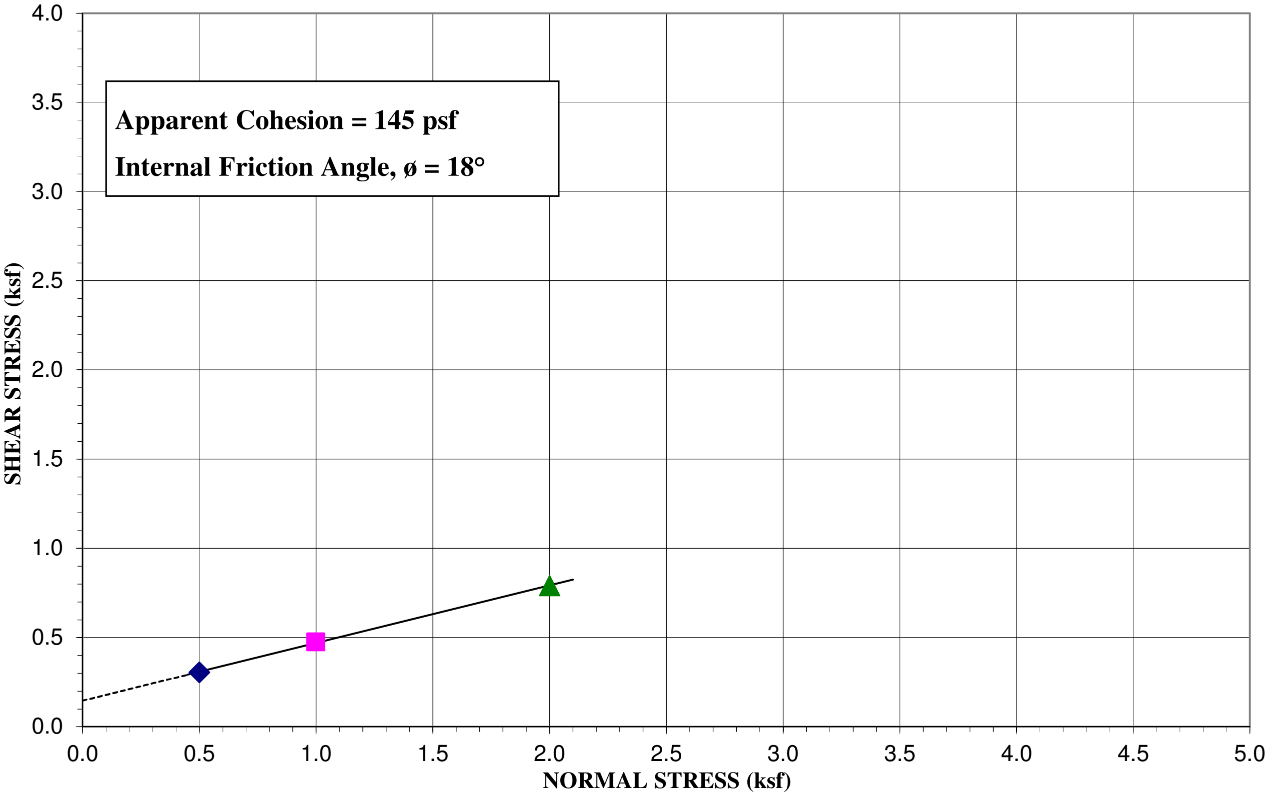
PROJECT: The Bridges Development

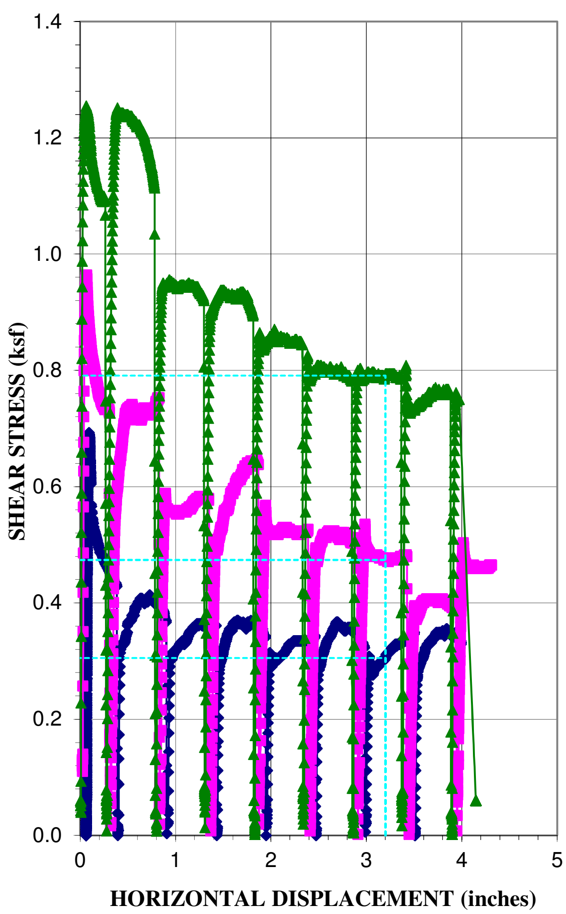
PROJECT NO.: **1311-002**

GeoStrata

Plate C-15

DIRECT SHEAR TEST





Source:	TP-2 @ 10
Type of Test:	Consolidated Drained/Saturated

Test No. (Symbol)	**1 (◆)**	**2 (■)**	**3 (▲)**
Sample Type	Remolded		
Initial Height, in.	1.016	0.97	1.101
Diameter, in.	2.5	2.5	2.5
Dry Density Before, pcf	91.0	93.8	90.8
Dry Density After, pcf	92.7	92.9	95.2
Moisture % Before	28.2	28.6	27.2
Moisture % After	30.6	30.6	30.1
Saturation, % Before	91.4	99.2	87.7
Saturation, % After	103.3	104.1	108.2
Normal Load, ksf	0.5	1.0	2.0
Shear Stress, ksf	0.31	0.47	0.79
Strain Rate	0.0033 IN/MIN		

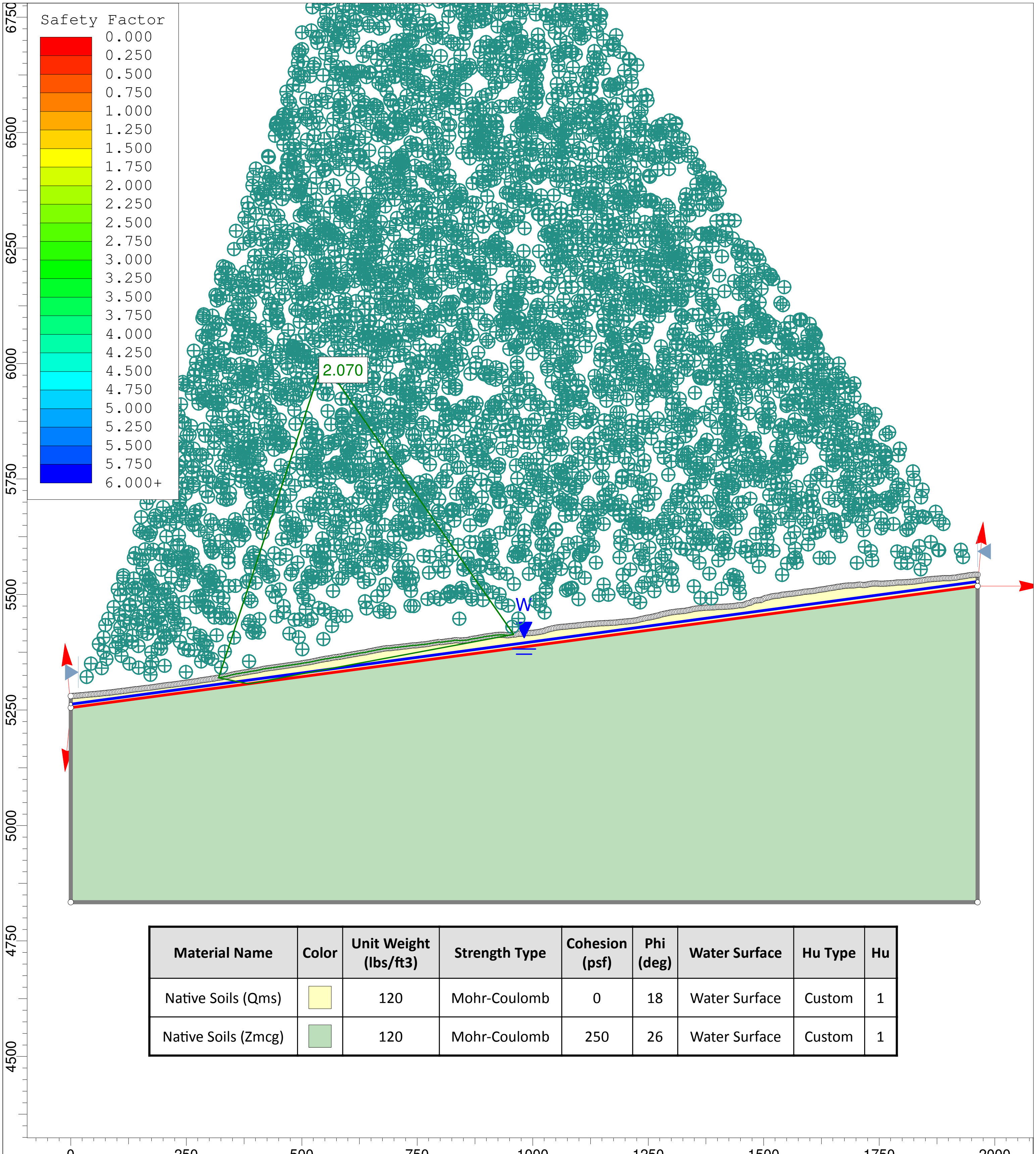
Sample Properties	
Cohesion, psf	145
Friction Angle, ϕ	18
Liquid Limit, %	51
Plasticity Index, %	24
Percent Gravel	0
Percent Sand	45
Percent Passing No. 200 sieve	55.0
Classification	CH

PROJECT: The Bridges Development

PROJECT NO.: **1311-002**

GeoStrata

Plate C-16



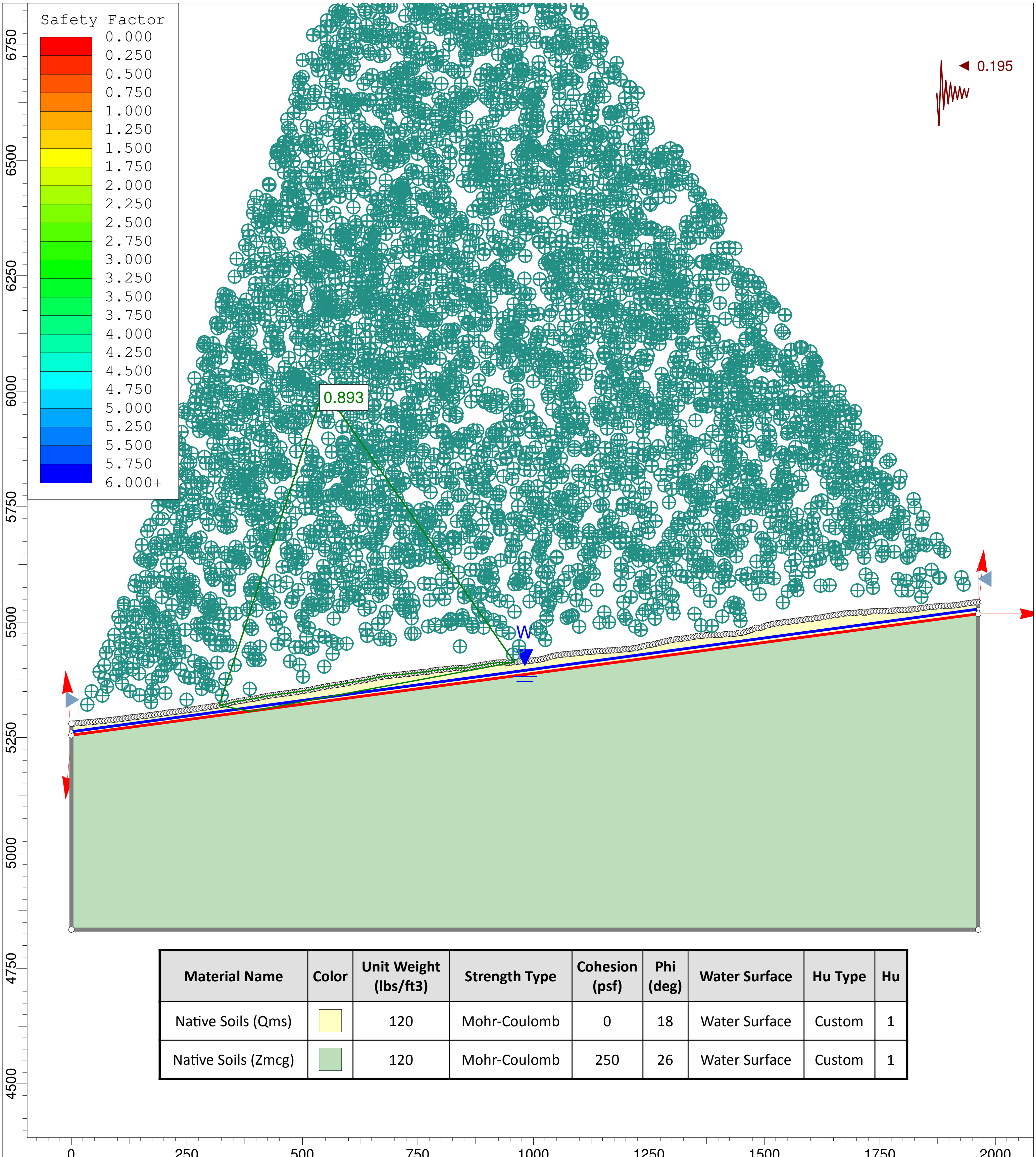
Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Hu Type	Hu
Native Soils (Qms)		120	Mohr-Coulomb	0	18	Water Surface	Custom	1
Native Soils (Zmcg)		120	Mohr-Coulomb	250	26	Water Surface	Custom	1

GeoStrata

Profile A Static Defined Failure

The Bridges at Wolf Creek
Eric Householder
Eden, Utah
GeoStrata Project No. 1311-002

Plate D-1



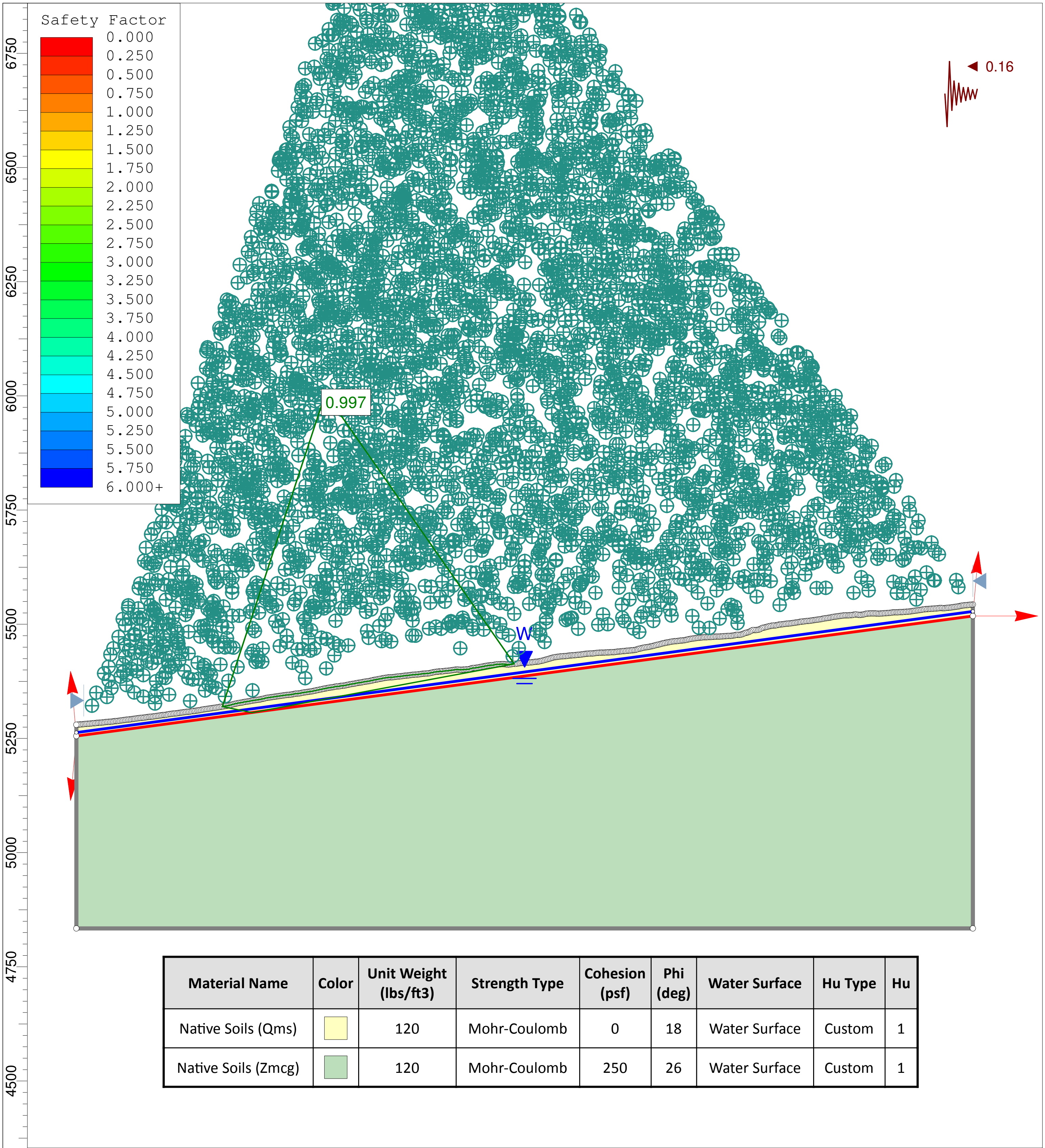
Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Hu Type	Hu
Native Soils (Qms)		120	Mohr-Coulomb	0	18	Water Surface	Custom	1
Native Soils (Zmcg)		120	Mohr-Coulomb	250	26	Water Surface	Custom	1

GeoStrata

Profile A PStatic Defined Failure

The Bridges at Wolf Creek
Eric Householder
Eden, Utah
GeoStrata Project No. 1311-002

Plate D-2



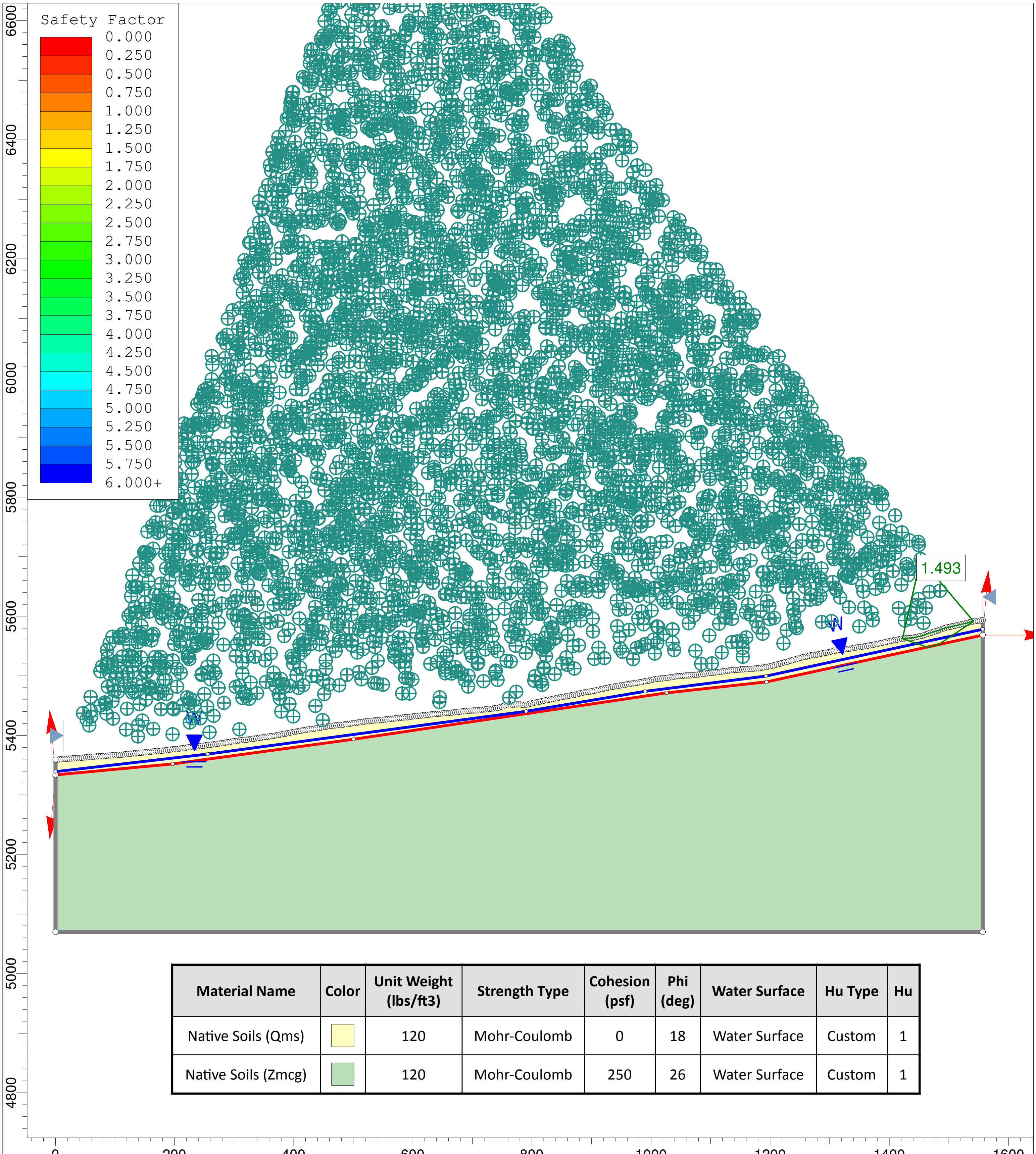
Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Hu Type	Hu
Native Soils (Qms)		120	Mohr-Coulomb	0	18	Water Surface	Custom	1
Native Soils (Zmcg)		120	Mohr-Coulomb	250	26	Water Surface	Custom	1

GeoStrata

Profile A Yeild Defined Failure

The Bridges at Wolf Creek
Eric Householder
Eden, Utah
GeoStrata Project No. 1311-002

Plate D-3



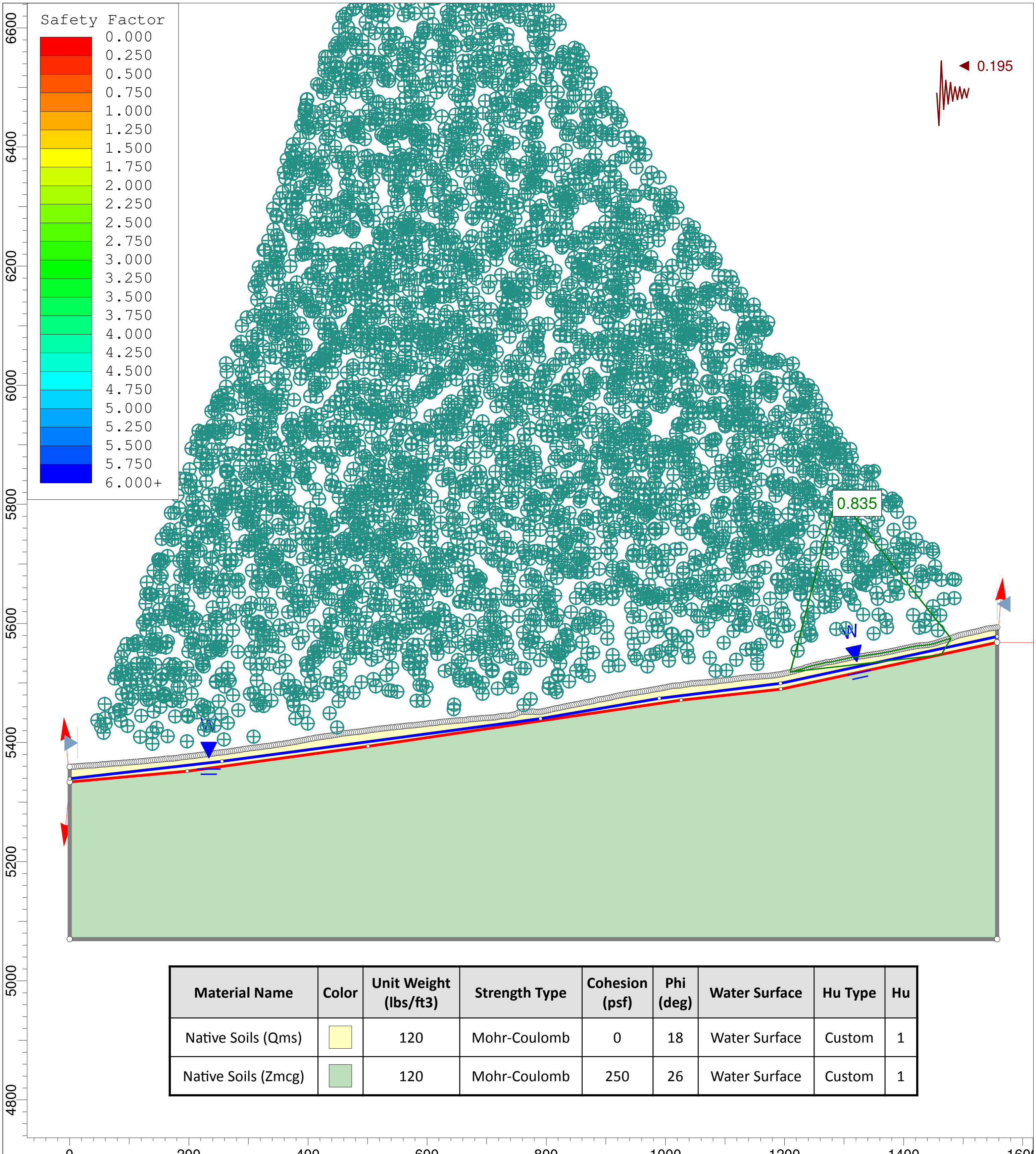
Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Hu Type	Hu
Native Soils (Qms)		120	Mohr-Coulomb	0	18	Water Surface	Custom	1
Native Soils (Zmcg)		120	Mohr-Coulomb	250	26	Water Surface	Custom	1

GeoStrata

Profile B Static 15' GW Predefined Surface

The Bridges at Wolf Creek
Eric Householder
Eden, Utah
GeoStrata Project No. 1311-002

Plate D-4



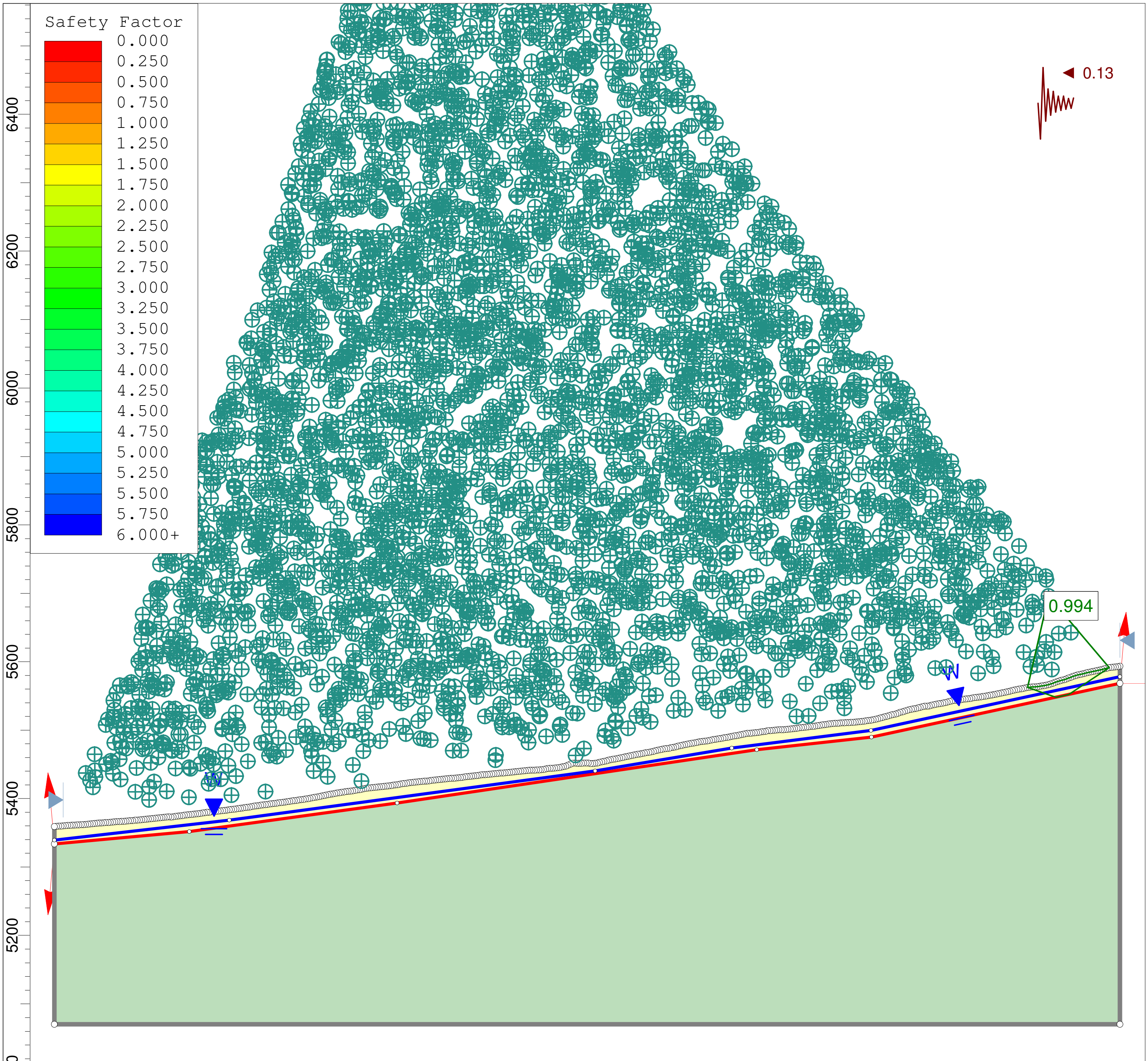
Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Hu Type	Hu
Native Soils (Qms)		120	Mohr-Coulomb	0	18	Water Surface	Custom	1
Native Soils (Zmcg)		120	Mohr-Coulomb	250	26	Water Surface	Custom	1

GeoStrata

Profile B PStatic 15' GW Predefined Surface

The Bridges at Wolf Creek
Eric Householder
Eden, Utah
GeoStrata Project No. 1311-002

Plate D-5



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Hu Type	Hu
Native Soils (Qms)		120	Mohr-Coulomb	0	18	Water Surface	Custom	1
Native Soils (Zmcg)		120	Mohr-Coulomb	250	26	Water Surface	Custom	1

GeoStrata

Profile B Yield 15' GW Predefined Surface

The Bridges at Wolf Creek
Eric Householder
Eden, Utah
GeoStrata Project No. 1311-002

Plate D-6