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# Geotechnical Investigation
# Approximately 4437 North 2900 East
# Liberty, Utah

GeoStrata Job No. 1348-002

July 31, 2018

Prepared for:

**Don Bingham**
**4433 West 1650 North**
**Plain City, Utah**

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**Geotechnical Investigation**
**Residential Building Lot**
**4437 North 2900 East**
**Liberty, Utah**
GeoStrata Job No. 1348-002

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## 1.0 EXECUTIVE SUMMARY

This report presents the results of a geotechnical investigation and slope stability conducted for the proposed residential development to be located at 4437 North 2900 East in Liberty, 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.

Based on the results of our geotechnical laboratory testing, it is our opinion that the 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. It should be noted that a Geologic Hazards Screening report dated December 17, 2017 has previously been completed by GeoStrata, and pertinent information from that report will be incorporated into this report.

Subsurface soil conditions were explored by advancing two test pits to a depth of 10 feet below the existing site grade. The exploration points were placed to provide a representative cross section of the subsurface soil conditions in the anticipated vicinity of the proposed structure. Based on observations made during our field investigation, grading activities have already been initiated in anticipation of development. Piles of suspected undocumented fill soils were observed within the area of the proposed residence, and should be considered undocumented fill soils unless documentation can be provided showing that the soils have been placed using engineering oversight and proper compaction techniques and testing. The soils exposed in our test pits consisted of a relatively thin veneer (about 12 inches) of clayey topsoil. Underlying the topsoil, we encountered deposits that have been identified by previously completed mapping as consisting of Pleistocene-aged landslide deposits. However, based on the results of our geologic screening report, no evidence of landslides, such as hummocky terrain and/or visible scarps are present at the subject site and it is considered possible that the site is underlain by alluvial deposits. In order to properly delineate the nature of the subsurface soils, a large-scale landslide investigation would be required. Such an investigation would require investigations such as deep borehole/trenches on properties outside of the subject property boundaries, and as such was outside of the scope of that report. Where observed, these deposits consisted of very dense, red brown, moist Clayey SAND (SC) with ½ to 4-inch thick seams of Lean CLAY (CL) with sand and cobbles up to 7 inches in diameter. These deposits persisted to the full depth of our explorations (10 feet) in test pit TP-1 and TP-2. Groundwater was not encountered in either of our explorations completed for the subject property.

Based on our literature review completed as part of our investigation of the property, there is uncertainty as to the nature of the near-surface soils at the property. Due to the uncertainty of the nature of the near-surface soils (i.e., landslide or alluvial deposits), slope stability analyses were completed for both scenarios. Results of our slope stability analyses indicate that the site meets the recommended factors of safety for static and seismic conditions if the site is underlain by alluvial deposits but is marginally stable statically and is unstable seismically if the site is underlain by landslide deposits. A study to further delineate the nature of the subsurface soils was outside of the scope of this investigation. The Client should be aware of the risks associated with developing a site that could potentially be underlain and impacted by landslide events.

If it is decided to develop the site, the foundation for the proposed structure may consist of conventional strip footings founded entirely on undisturbed native soils. 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 **1,700 pounds per square foot (psf)** for dead plus live load conditions.

Recommendations designed to reduce the potential for instability from impacting the subject site may be found within the body of this report. 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 residence.

**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 and slope stability assessment conducted for proposed residential structure to be constructed at 4437 North 2900 East in Liberty, 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.

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 July 3, 2018 and your signed authorization.

It should be noted that GeoStrata previously completed a Geologic Hazards Screening for the subject site, the results of which may be found in a report dated December 4, 2017. It is considered imperative that all recommendations made in that report be incorporated into the design of the project.

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 rectangular-shaped property is located at 4437 North 2900 East in Liberty, Utah (see Plate A-1, *Site Vicinity Map*). The lot has a total area of approximately 5.68 acres, and slopes steeply to gently towards the east at an approximate average 5H:1V grade. Our understanding of the proposed development is based on information provided by the client. We understand that the development will consist of the construction of a single-family residential structure with associated driveway and landscaped areas. Construction plans were not available for review at the time this report was prepared; however, we anticipate that the proposed structure will consist of a one- to two-story wood-framed building with a basement (if feasible) founded on conventional strip footings.

# 3.0 METHODS OF STUDY

## 3.2 FIELD INVESTIGATION

As a part of this investigation, subsurface soil conditions were explored by advancing two test pits to a depth of 10 feet below the existing site grade. Exploration points were placed to provide a representative cross section of the subsurface soil conditions in the anticipated vicinity of the proposed structure. The approximate locations of the explorations are shown on the *Exploration Location Map*, Plate A-2 in Appendix A. 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 Test Pit Logs, Plates B-1 to B-3. A *Key to Soil Symbols and Terminology* is presented on Plate B-3.

The test pits were excavated using a backhoe, and both disturbed and undisturbed soil samples were obtained from each of the exploration locations. Soil samples obtained in the test pit explorations were transported to our laboratory for testing to evaluate engineering properties of the various earth materials observed. The soils were classified according to the *Unified Soil Classification System* (USCS) by the Geotechnical Engineer. Classifications for the individual soil units are shown on the attached Test Pit Logs.

## 3.3 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)
- 1-D Swell/Collapse Potential Test (ASTM D5333)
- Direct Shear Test (ASTM D3080)

The results of laboratory tests are presented on the test pit logs in Appendix B (Plates B-1 to B-2), the Lab Summary Report (Plate C-1), on the test result plates presented in Appendix C (Plates C-2 to C-5).

## 3.4 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 property sits at an elevation ranging from 5,210 to 5,397 feet above sea level with a total topographic relief of approximately 187 feet. Slopes at the subject site are towards the east and have an average grade of 5H:1V. Based on observations made at the time of our field investigation, site grading has been initiated in anticipation of construction. A cut had been made into the native hillside near the eastern portion of the lot, and areas overlain by what is suspected to be undocumented fill were observed throughout the graded area. The areas that had not undergone site grading were covered in heavy amounts of vegetation consisting of native sage brush and scrub oak. No structures or other improvements were observed on the subject property at the time of our investigation.

## 4.2 SUBSURFACE CONDITIONS

As previously discussed, the subsurface soil conditions were explored at the site by excavating two test pits at representative locations within the lot. The test pits extended to a depth of 10 feet below the existing site grade. The soils encountered in the test pits were visually classified and logged during our field investigation and are included on the Test Pit Logs in Appendix B (Plates B-1 to B-3). The subsurface conditions encountered during our investigation are discussed below.

### 4.2.1 Soils

As mentioned previously, the site grading has been initiated at the site in anticipation of construction. As a result, several areas of suspected fill soils were observed in and near the area of the proposed residence. These soils should be considered to consist of undocumented fill soils unless documentation can be provided showing that the soils were placed with engineering oversight and testing. The thickness of these potentially undocumented fill soils could not be identified during our field investigation. The soils exposed in our field investigations consisted of a relatively thin veneer (12 inches) of clayey topsoil overlying deposits that are mapped by King and Yonkee (2016) as consisting of partially of Pleistocene-aged Bonneville related deposits, but largely of Pleistocene-aged landslide deposits.

As summarized in our December 2017 geologic hazard screening report, no available, surficial evidence such as hummocky topography or the presence of scarps were observed at the subject site. However, the results of our investigation using 2011 one-meter LiDAR data did indicate the presence of hummocky topography to the north and northwest of the subject site. A detailed investigation into that landslide was not feasible as it would require deep borehole investigations at locations outside of the subject property. As a result, it was not possible to positively identify the onsite sediments as either landslide or alluvial deposits, but the potential for slope instability to impact the subject site is present. More detailed descriptions of the soil units encountered are provided below:

Topsoil: Where observed, these soils generally consist of brown to dark brown Lean CLAY (CL) with sand, gravel, cobbles and boulders. These soils typically display a trace "pinhole" structure and have an organic appearance, with numerous up to 1-inch in diameter throughout. Topsoil was encountered in both of our test pits to a depth of 12 inches and is anticipated to overlay the majority of the site (in areas that have not undergone site grading).

Pleistocene- to Holocene-aged Landslide Deposits (Qms): Where observed, these soils typically consisted of dense to very dense, moist, red-brown to tan Clayey SAND (SC) with gravel and cobbles, as well as very stiff to hard Lean CLAY (CL) with sand. The plasticity of the fine-grained sediments ranged from low to medium, while boulders up to 7-inches in diameter were observed throughout. These deposits were observed to persist to the full depth of our investigations. Based on mapping completed by Coogan and Yonkee (2016), the landslide deposits are described as consisting of poorly sorted to unsorted clay- to boulder-sized material and are mapped in areas where landslide deposits are difficult to distinguish from colluvium deposits.

The stratification lines shown on the enclosed test pit logs represent the approximate boundary between soil types (Plates B-1 to B-2). 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 excavated as part of our investigation, and is anticipated to be relatively deep; however, due to the location of the subject property in the

Wasatch Mountain foothills, localized perched groundwater may occur during the spring months. Fluctuations in the groundwater level should be expected over time.

#### 4.2.3 Expansive Soils

Some soils having moderate plasticity can 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 swell potential of 0.31 percent. As such, it is not anticipated that swelling soils will impact the proposed development.

#### 4.2.4 Strength of Earth Materials

A direct shear test was performed on a relatively "undisturbed" sample of the near-surface soils that classify as a Clayey SAND (SC). The test indicated that the sample tested had a cohesion of 375 psf and an internal angle of friction (phi) of 20 degrees. As discussed previously, it was not possible to delineate the origins of these deposits as either alluvium or landslide due to the limitations in access to nearby properties. As such, we assumed residual shear values consisting of a friction angle of 15 degrees and cohesion of 0 psf for our analysis where the near-surface soils are assumed to consist of landslide deposits, and a peak shear value consisting of a friction angle of 20 degrees and a cohesion of 375 psf for our analysis in which the near-surface soils are assumed to consist of alluvial deposits.

A summary of the test results discussed above may be found on Plate C-5.

### 4.3 STABILITY OF NATURAL SLOPES

An approximate 4.7H:1V (Profile A) descending slope is present across the site (see Plate A-2). The global stability of the slope stability profile was modeled using the SLIDE computer application and the Bishop's Simplified Method of analysis. Calculations for stability were developed by searching for the minimum factor of safety for a circular-type failure. Homogenous earth materials and arcuate failure surfaces were assumed. Topographic information for the profiles was obtained using the 2011 1-meter LiDAR provided by the State of Utah AGRC. The location and steepness of cuts and fills were assumed based on the location of the test pits completed for this investigation and an assumed width of the proposed building pad.

Slope stability analysis was performed for both the static and pseudo-static (seismic) conditions. The pseudo-static assessment was completed utilizing the peak ground acceleration (PGA) associated with a 2 percent chance of exceedance in 50 years. Based on seismic design parameters for the site (IBC, 2015), a PGA value of 0.46g was utilized in our analysis (see Section 5.2).

Strength parameters for the soils located at the subject property were obtained utilizing the results of a direct shear test completed as part of this investigation as discussed in Section 4.2.4 of this report. For our slope stability modeling, GeoStrata has elected to use a soil strength value consisting of an angle of friction (phi) of 20 degrees and 375 psf cohesion as these are the minimum values returned from our laboratory testing. These values have been applied to the near-surface clayey soils. A distributed load of 1,500 psf has been incorporated into our model to account for potential loading from the proposed structure. The location of these loads has been based on the proposed building location as described by the Client. If a different location is chosen, then GeoStrata will need to be informed and updated modeling may be required.

A second series of slope stability models have also been completed for the subject site utilizing the assumption that the sediments observed in our explorations consist of landslide deposits (i.e., are at residual strength). Based on the liquid limit and the percent passing the No. 200 sieve of the soils encountered, a residual friction angle of 18 degrees for the subsurface soils was assumed using the correlation developed by Stark, et al. (2005).

Groundwater was not encountered in any of the explorations advanced as part of this investigation. Review of the in-situ moisture conditions as measured during our laboratory testing did not indicate the presence of any perched groundwater units (saturation ranging from 34 to 56 percent). As such, groundwater was not incorporated into our slope stability modeling. GeoStrata still recommends that all surficial moisture mitigation recommendations given in this report be implemented into the design of the project to reduce the potential for near-surface saturation and slope failure to occur. It is considered likely that a septic system is planned as part of the project. The proposed septic tank is to be located to the northeast of the residence. We recommend that the septic system be placed as far down slope as practicable. Moisture and seepage out of this system could increase the overall groundwater level and impact on the stability analysis completed for this structure.

Results of our slope stability investigation are as follows:

| Factor of Safety – Peak Soil Strengths | | | |
|---|---|---|---|
| Profile | Failure Type | Static | Pseudo Static |
| A | Circular | 1.62 | 1.05 |

| Factor of Safety – Residual Soil Strengths | | | |
|---|---|---|---|
| Profile | Failure Type | Static | Pseudo Static |
| A | Circular | 0.93 | 0.64 |

As can be seen from the results above, the slope stability of the site meets adequate industry standards of care **IF** the sediments at the site consist of alluvial deposits (i.e., are at peak strengths).

Based on the results of our slope stability analysis described above, the site is marginally stable under static conditions and will experience deformation during a seismic event if the soils are modeled as having residual strengths (landslide deposits). As a result, statically, the slopes on the subject property could potentially be very sensitive to alterations to the existing site grade and topography, as well as in changes in groundwater elevation. Should excessive cuts/fills be introduced as part of the proposed development, or should elevated groundwater conditions exist, the potential exists for the slopes investigated to deform/fail. During seismic events, in order to quantify the amount of deformation that can be expected during a MCE (maximum creditable earthquake) GeoStrata has completed a deformation analysis utilizing the methodology outlined by Bray and Travasarou (2007). Shear wave velocities utilized in this methodology were obtained from the Utah Geological Survey for "tertiary sedimentary and volcanic rocks". This report can be accessed at http://geology.utah.gov/ghp/consultants/geophysical_data/shear-wave_velocity.htm. Based on the results of our deformation analysis, it is anticipated that the site may experience up to 67.9 inches of displacement during an MCE event. The Client should be aware of and accept the potential for this deformation to occur should they choose to develop the subject property.

As mentioned previously, due to limitations in site access and scope of investigation, it was not possible to determine if the site was underlain by landslide deposits or by Bonneville-aged sediments. If the Client wishes to have a better understanding of the nature of the subsurface soils at the subject site, then a landslide investigation utilizing deep boreholes and trenching should be completed. All recommendations made in the 2017 GeoStrata Geologic Hazards Assessment report should likewise be incorporated into the design of the project to reduce the potential for the site to be impacted by slope stability hazards.

# 5.0 GEOLOGIC CONDITIONS

## 5.1 GEOLOGIC SETTING

As mentioned previously, a geological hazards investigation is concurrently being completed for the subject site, the results of which may be found in a report completed by GeoStrata. 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 December 4, 2017.

## 5.2 SEISMICITY AND FAULTING

During the event of an earthquake, seismic waves radiate outward from the initial point of rupture and dissipate with distance. The ground shakes as the seismic waves displace the ground both vertically and horizontally. Ground shaking can cause significant damage to and potentially collapse structures and can also trigger landslides, avalanches and liquefaction. The type of soil a seismic wave travels through can amplify or dampen the effects of ground shaking.

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 D which represents a “stiff soil” 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 41.33879° and -111.87950° respectively and the United States Geological Survey U.S. Seismic Design Maps web-based application. Based on the IBC, the site coefficients are $F_a$=1.07 and $F_v$= 1.64. From this procedure the peak ground acceleration (PGA) is estimated to be 0.46g.

| **Site Location:**<br>**Latitude = 41.33879 N**<br>**Longitude = -111.87950 W** | **Site Class D Site Coefficients:**<br>**Fa = 1.07**<br>**Fv = 1.64** |
|---|---|
| **Spectral Period (sec)** | **Response Spectrum Spectral Acceleration (g)** |
| 0.2 | $\mathbf{S_{MS}}$=($F_{a}$*$S_s$=1.07*1.078) = 1.15 |
| 1.0 | $\mathbf{S_{M1}}$=($F_{v}$*$S_1$=1.64*0.382) = 0.63 |
| [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 D[a].**

# 6.0 ENGINEERING CONCLUSIONS AND RECOMMENDATIONS

## 6.1 GENERAL CONCLUSIONS

Based on the subsurface conditions encountered at the site as well as on our geological literature review and engineering analysis, it is our opinion that the subject site is stable under static and seismic conditions in its current configuration if the site is underlain by alluvial deposits. However, the site is marginally stable in its current configuration, and likely unstable under seismic conditions, with an estimated deformation of 67.9 inches if the near-surface soils consist of landslide deposits. Results of our slope stability analyses may be found in Section 4.3 of this report. If the Client wishes to have a greater understanding of the hazard landslides pose to the development of the subject site, then an in-depth landslide investigation should be completed, and may require the investigation of the stability of off-site properties.

Regardless of the nature of the near-surface soils, construction on a property underlain by landslide deposits or within close proximity to landslide deposits poses inherent risks which the Client must be aware of and accept, due to the fact that the potential for the proposed structure to be impacted by slope instability is elevated. If it is decided to develop on the subject lot despite the presence of potentially unstable slopes at the site and if the risks associated with these slopes are accepted, the following recommendations should be adhered to in order to reduce the potential for instability to occur.

Due to the potential presence of landslide deposits on the subject property as well as on neighboring properties, site grading should be minimized and impacts to the current geometry of the property should be avoided. In addition, setbacks from the slope located on the eastern portion of the site as described in the following paragraphs should be adhered to. Finally, the introduction of moisture into the subsurface soils should be avoided, and an appropriate landscape plan should be implemented. In addition, we recommend that the septic system be placed as far down slope as practicable. Moisture and seepage out of this system could increase the overall groundwater level and impact on the stability analysis completed for this structure.

The foundation for the proposed structure may consist of conventional shallow spread footings founded entirely on competent native earth materials. To reduce the potential for adverse differential settlement of the proposed residential structure, efforts should be made to avoid building foundations over a bedrock/soil transition zone (if encountered).

## 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 (including tree roots), potentially expansive soils, loose, soft or other deleterious materials should also be removed and replaced with structural 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.

As mentioned in Section 6.1 of this report, due to the potential presence of marginally stable slopes at the site, it is recommended that cuts/fill sections at the property be limited to 4 feet or less. Cuts and/or fill sections exceeding this height could potentially impact the stability of the subject site.

### 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 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-inch 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 structure or flatwork concrete should consist of structural fill. Due to the potential for the native fine-grained soils to be highly plastic, we do not recommend that they be used as structural fill. Native coarse-grained soils (sand and gravels) may be utilized as structural fill provided they are first screened for debris and material exceeding 4-inches in diameter. Alternatively, an imported structural fill may be utilized. An imported structural fill should consist of a granular soil with maximum fines content (minus No.200 mesh sieve) of 30 percent and no less than 15% fines. 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 6 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).

Backfill around basement walls should be compacted to approximately 90 percent of the MDD as determined by the ASTM D-1557. Failure to properly water-condition and compact basement wall backfill may result in settlements of up to several inches should the backfill become wetted. Only small compaction equipment should be used near basement walls.

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

Bearing capacity values were calculated using Meyerhof and others' modifications to Terzaghi's original bearing capacity formula. Strength parameters for the bearing strata were assigned based on laboratory shear strength parameters and field observations. A factor of safety of 3 is generally used in developing allowable bearing values; however, additional reduction of allowable bearing is typically warranted to account for static settlement.

It is recommended that shallow spread or continuous wall footings be constructed on competent, relatively undisturbed native soils. Foundations founded on suitable, native soils or on structural fill may be proportioned utilizing a maximum net allowable bearing pressure of **1,500 pounds per square foot (psf)** for dead load plus live load conditions.

All foundations exposed to the full effects of frost should be established at a minimum depth of 42 inches below the lowest adjacent final grade. Interior footings, not subjected to the full effects of frost, may be established at higher elevations, however, a minimum depth of embedment of 12 inches is recommended for confinement purposes. The minimum recommended footing width is 20 inches for continuous wall footings and 30 inches for isolated spread footings.

## 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 around any subgrade walls. 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.

### 6.5 CONCRETE SLAB-ON-GRADE CONSTRUCTION

Concrete slabs-on-grade should be constructed over at least 4 inches of compacted gravel overlying undisturbed native soil 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 percent of the MDD of modified proctor or until tight and relatively unyielding if the material is non-proctorable. The maximum load on the floor slab should not exceed 300 psf; greater loads would require additional subgrade preparation and additional structural fill. 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. We recommend that concrete be tested to assess that the slump and/or air content in compliance with the plans and specifications. If the slump and/or air content are beyond the recommendations as specified in the plans and specifications, the concrete may not perform as desired. We recommend that concrete be placed in general accordance with the requirements of the American Concrete Institute (ACI).

A capillary break consisting of 4 inches of sand or clean gravel or a moisture barrier (vapor retarder) consisting of 10-mil thick Visqueen (or equivalent) plastic sheeting should be placed below slabs-on-grade where moisture-sensitive floor coverings or equipment is planned. Prior to placing this moisture barrier, any objects that could puncture it, such as protruding gravel or rocks, should be removed from the building pad.

### 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 0.34 for native Clayey SAND (SC) should be used.

Ultimate lateral earth pressures from natural soils and *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* | 0.35 | 38 |
| At-rest** | 0.56 | 62 |
| Passive* | 4.40 | 484 |
| Seismic Active*** | 0.78 | 86 |
| Seismic Passive*** | -2.17 | -239 |

* 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.

Clayey soils drain poorly, and may swell upon wetting, thereby greatly increasing lateral pressures acting on earth retaining structures; therefore, clayey soils should not be used as retaining wall backfill. Backfill should consist of either native granular soils or sandy imported material with an Expansion index (EI) of less than 20.

## 6.7 MOISTURE PROTECTION AND SURFACE DRAINAGE

Moisture should not be allowed to infiltrate into the soils in the vicinity of the foundations. As such, design strategies to minimize ponding and infiltration near the home should be implemented. The natural slope of the subject lot may be subject to sheet flow during periods of heavy rain or snow melt. Therefore, the Civil Engineer may also wish to consider construction of additional surface drainage to intercept surface runoff, or a curtain drain to intercept seasonal groundwater flow, if any.

We recommend that hand watering, desert landscaping or Xeriscape be considered for the subject lot due to the potential proximity to landslide deposits. We further recommend roof runoff devices be installed to direct all runoff a minimum of 10 feet away from structures or to storm water runoff areas. The home builder should be responsible for compacting the exterior backfill soils around the foundation. Additionally, the ground surface within 10 feet of the house should be constructed so as to slope a minimum of two percent away from the home. Pavement sections should be constructed to divert surface water off of the pavement into storm drains. Parking strips and roadway shoulder areas should be constructed to prevent infiltration of water into the areas surrounding pavement.

As noted earlier, we recommend that the septic system be placed as far down slope as practicable. Moisture and seepage out of this system could increase the overall groundwater level and impact on the stability analysis completed for this structure.

# 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.

This report was prepared in accordance with the generally accepted standard of practice at the time the report was written. No other warranty, expressed or implied, is made.

It is the Client's responsibility to see that all parties to the project including the Designer, Contractor, Subcontractors, etc. are made aware of this report in its entirety. The use of information contained in this report for bidding purposes should be done at the Contractor's option and risk.

## 7.2 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. 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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500 0 500 1000 1500 2000 ft

1:12,000

GeoStrata

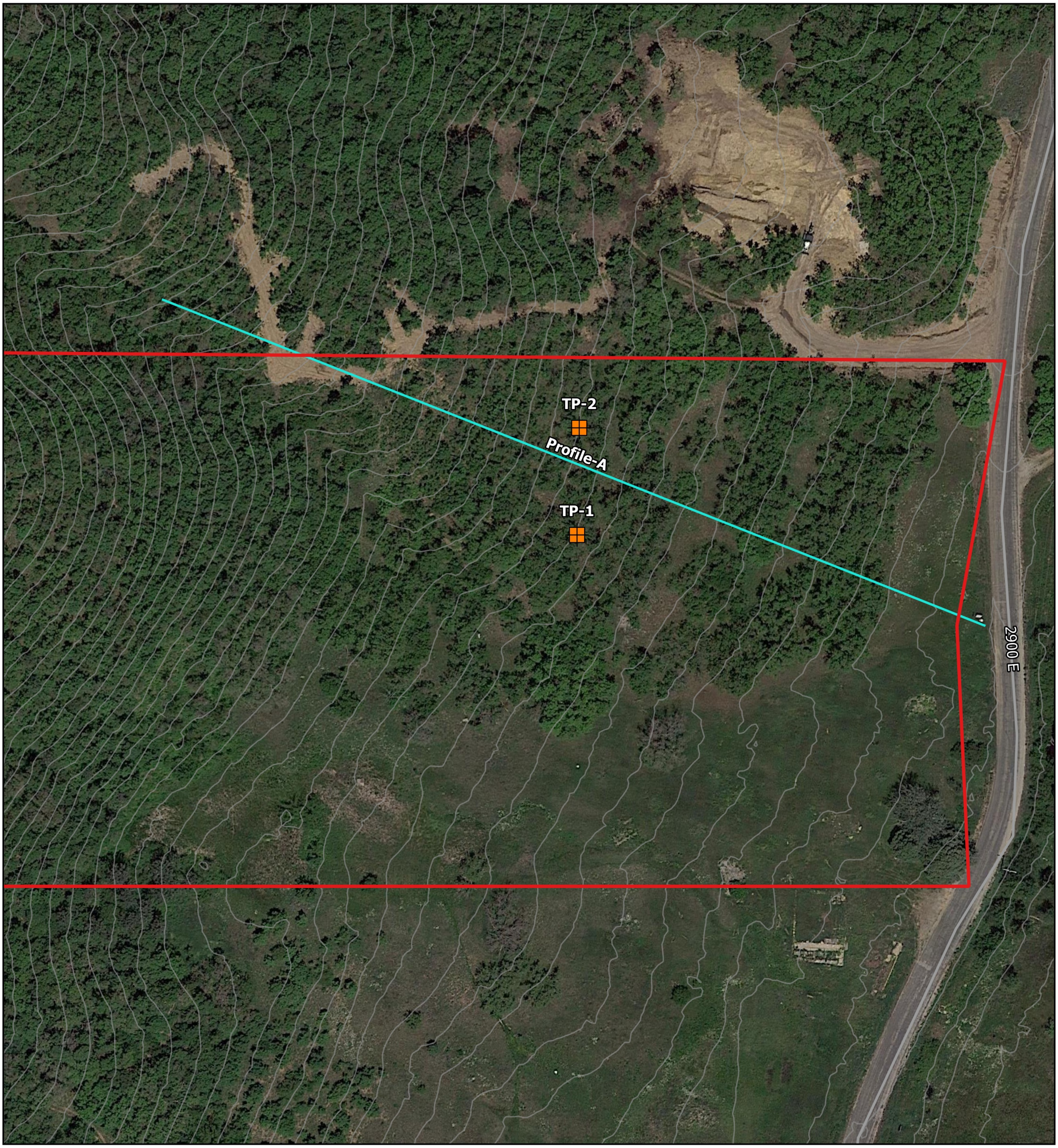

## Legend

Approximate Site Boundary

Donald Bingham
4437 N 2900 E
Liberty, Utah
Project Number: 1348-002

**Site Vicinity Map**

**Plate A-1**




50 0 50 100 150 200 ft

1:1,200

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## Legend

- Approximate Site Boundary
- Approximate Test Pit Location2
- Slope Profile

Donald Bingham
4437 N 2900 E
Liberty, Utah
Project Number: 1348-002

**Exploration Location Map**

**Plate A-2**

| DATE | | |
|---|---|---|
| STARTED: | 7/6/18 | |
| COMPLETED: | 7/6/18 | |
| BACKFILLED: | 7/6/18 | |

Don Bingham
4437 N 2900 E
Liberty, Utah
Project Number 1348-002

GeoStrata Rep: A. Peay
Rig Type: Backhoe

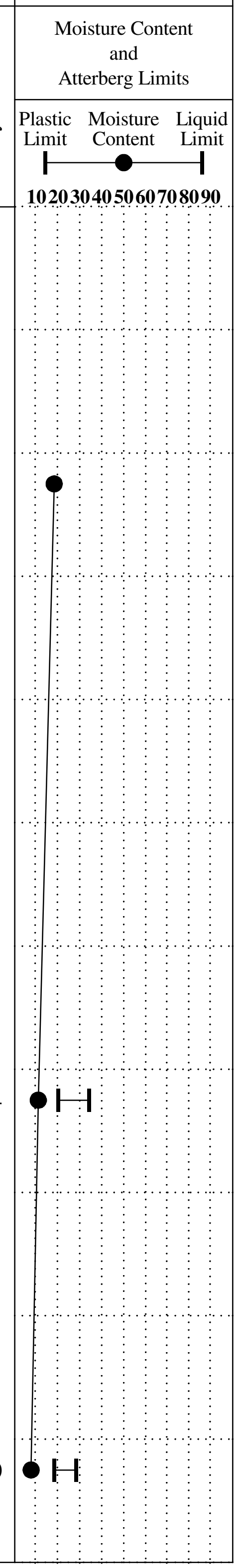
TEST PIT NO:
# TP-1
Sheet 1 of 1

LOCATION

NORTHING | EASTING | ELEVATION

## MATERIAL DESCRIPTION

| DEPTH (Meters) | DEPTH (Feet) | SAMPLES | WATER LEVEL | GRAPHICAL LOG | UNIFIED SOIL CLASSIFICATION | MATERIAL DESCRIPTION | Dry Density(pcf) | Moisture Content % | Percent minus 200 | Liquid Limit | Plasticity Index |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 0 | 0 | | | | | TOPSOIL; Lean CLAY with sand and gravel - red brown, moist | | | | | |
| | | | | | SC | Clayey SAND - very dense, red brown with tan and yellow mottling, moist, ocassional ½ to 4 inch thick seams of Lean CLAY with sand and cobbles up to 7 inches in diameter | | | | | |
| | | Grab | | | | | 102.5 | 18.1 | | | |
| 1 | | | | | | | | | | | |
| | 5 | | | | | | | | | | |
| 2 | | Grab | | | | | | 10.8 | 27.3 | 34 | 14 |
| 3 | 10 | Grab | | | | Bottom of Test Pit @ 10 Feet | | 7.5 | 39.1 | 28 | 10 |






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

WATER LEVEL
- MEASURED
- ESTIMATED

NOTES:

**Plate**
**B-1**

| DATE | | |
|---|---|---|
| STARTED: | 7/6/18 | |
| COMPLETED: | 7/6/18 | |
| BACKFILLED: | 7/6/18 | |

Don Bingham
4437 N 2900 E
Liberty, Utah
Project Number 1348-002

GeoStrata Rep: A. Peay
Rig Type: Backhoe

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

LOCATION

NORTHING EASTING ELEVATION

## MATERIAL DESCRIPTION

| DEPTH (METERS) | DEPTH (FEET) | SAMPLES | WATER LEVEL | UNIFIED SOIL CLASSIFICATION | MATERIAL DESCRIPTION | Dry Density(pcf) | Moisture Content % | Percent minus 200 | Liquid Limit | Plasticity Index |
|---|---|---|---|---|---|---|---|---|---|---|
| 0 | 0 | | | | TOPSOIL; Lean CLAY with sand and gravel - red brown, moist | | | | | |
| | | | | SC | Clayey SAND - very dense, red brown with tan and yellow mottling, moist, ocassional ½ to 4 inch thick seams of Lean CLAY with sand and cobbles up to 7 inches in diameter | | | | | |
| | 5 | Grab | | CL | Lean CLAY with sand - hard, red brown, moist | | 7.5 | 74.2 | 33 | 11 |
| 3 | 10 | | | | Bottom of Test Pit @ 10 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:

Plate
B-2



LOG OF TEST PITS (B) EXPLORATOIN LOGS.GPJ GEOSTRATA.GDT 7/20/18

## 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. |

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**Soil Symbols Description Key**

Don Bingham
4437 North 2900 East
Liberty, Utah
Project Number 1348-002

**Plate B-3**

| Test Pit No. | Sample Depth (feet) | USCS Soil Classification | Natural Moisture Content (%) | Natural Dry Density (pcf) | Gradation | | | Atterberg | | Swell (%) | Direct Shear | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | Gravel (%) | Sand (%) | Fines (%) | LL | PI | | Apparent Cohesion (psf) | Friction Angle (°) |
| TP-1 | 2 | SC | 18.1 | 102.5 | | | | | | 0.31 | | |
| TP-1 | 7 | SC | 10.8 | | 72.7 | | 27.3 | 34 | 14 | | | |
| TP-1 | 10 | SC | 7.5 | | 60.9 | | 39.1 | 28 | 10 | | 375 | 20 |
| TP-2 | 5 | Cl | 7.5 | | 25.8 | | 74.2 | 33 | 11 | | | |
| TP-3 | 1 | ML | 6 | | 38.0 | | 62.0 | 29 | NP | | | |

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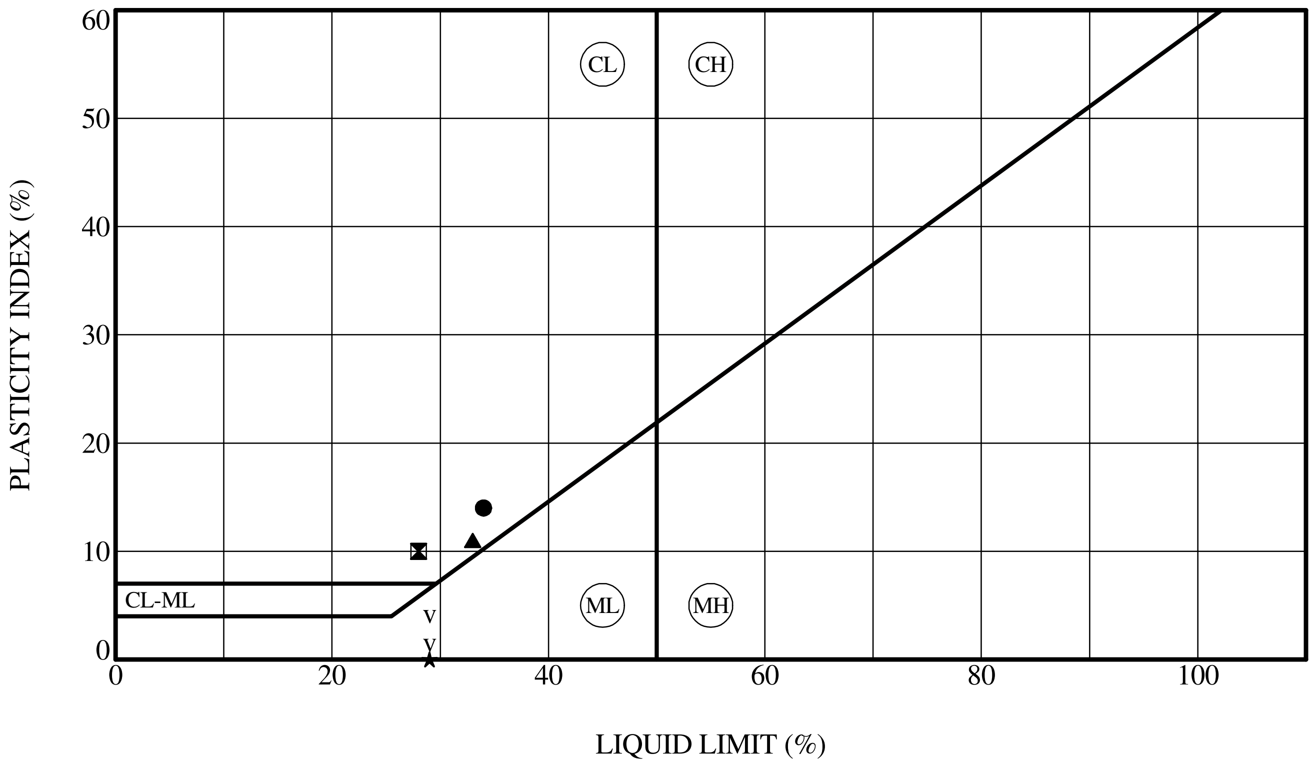
**Lab Summary Report**

Don Bingham
4437 N 2900 E
Liberty, Utah
Project Number: 1348-002

**Plate C - 1**




| | Sample Location | Depth (ft) | LL (%) | PL (%) | PI (%) | Fines (%) | Classification |
|---|---|---|---|---|---|---|---|
| ● | **TP-1** | **7.0** | **34** | **20** | **14** | **27.3** | **Clayey SAND** |
| ⊠ | **TP-1** | **10.0** | **28** | **18** | **10** | **39.1** | **Clayey SAND** |
| ▲ | **TP-2** | **5.0** | **33** | **22** | **11** | **74.2** | **Lean CLAY with sand** |
| ★ | **TP-3** | **1.0** | **29** | **36** | **NP** | **62.0** | **Sandy SILT** |

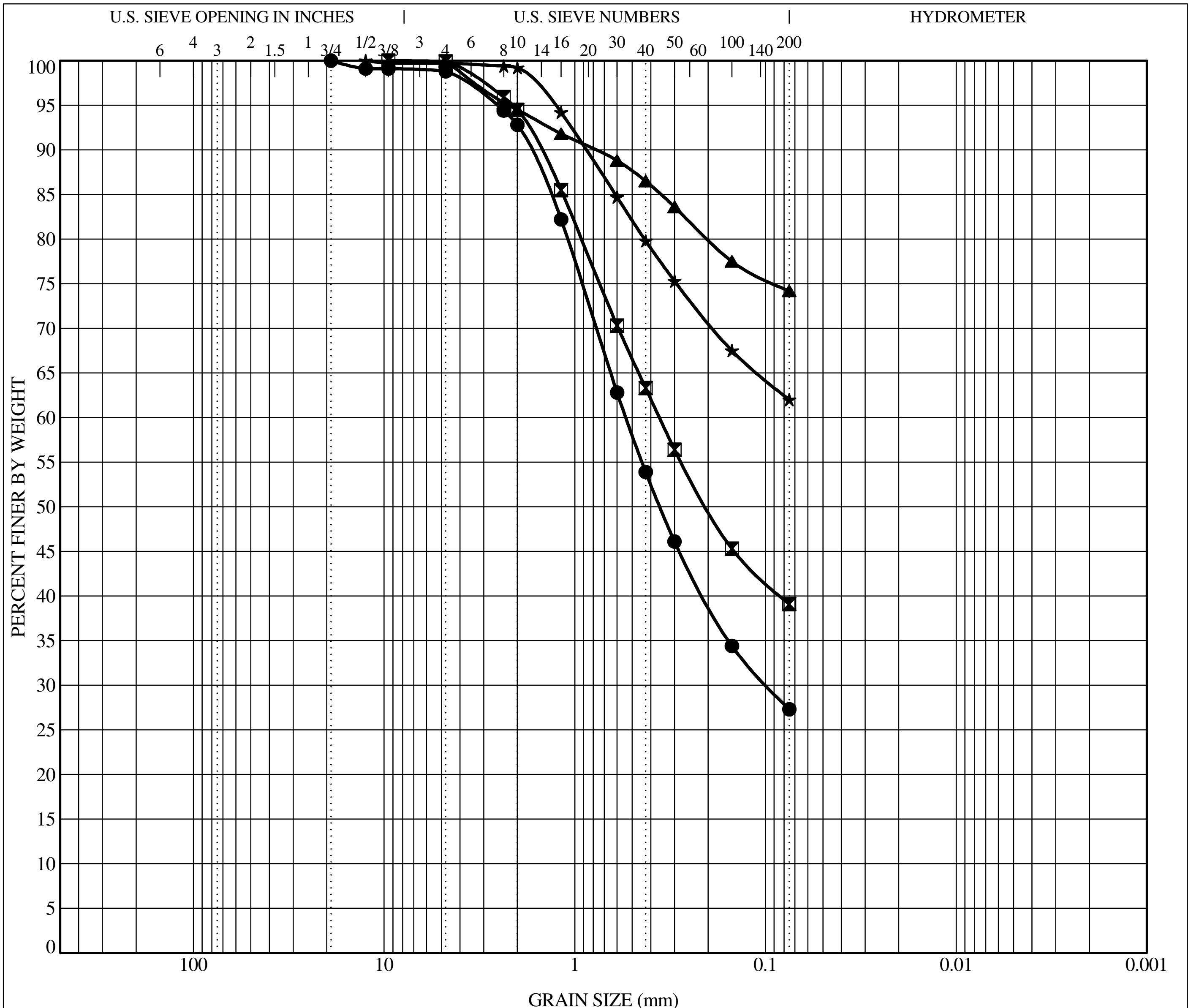
GeoStrata

**ATTERBERG LIMITS' RESULTS - ASTM D 4318**

Don Bingham
4437 N 2900 E
Liberty, Utah
Project Number: 1348-002

**Plate**
**C - 2**

C_ATTERBERG EXPLORATOIN LOGS.GPJ GEOSTRATA.GDT 7/20/18




| COBBLES | GRAVEL | | SAND | | | SILT OR CLAY |
|---|---|---|---|---|---|---|
| | coarse | fine | coarse | medium | fine | |

| | Sample Location | Depth | Classification | LL | PL | PI | Cc | Cu |
|---|---|---|---|---|---|---|---|---|
| ● | **TP-1** | **7.0** | **Clayey SAND** | **34** | **20** | **14** | | |
| ⊠ | **TP-1** | **10.0** | **Clayey SAND** | **28** | **18** | **10** | | |
| ▲ | **TP-2** | **5.0** | **Lean CLAY with sand** | **33** | **22** | **11** | | |
| ★ | **TP-3** | **1.0** | **Sandy SILT** | **29** | **36** | **NP** | | |

| | Sample Loctaion | Depth | D100 | D60 | D30 | D10 | %Gravel | %Sand | %Silt | %Clay |
|---|---|---|---|---|---|---|---|---|---|---|
| ● | **TP-1** | **7.0** | **19** | **0.538** | **0.098** | | **1.2** | **71.5** | **27.3** | |
| ⊠ | **TP-1** | **10.0** | **9.5** | **0.36** | | | **0.1** | **60.8** | **39.1** | |
| ▲ | **TP-2** | **5.0** | **4.75** | | | | **0.0** | **25.8** | **74.2** | |
| ★ | **TP-3** | **1.0** | **12.5** | | | | **0.3** | **37.7** | **62.0** | |

GeoStrata

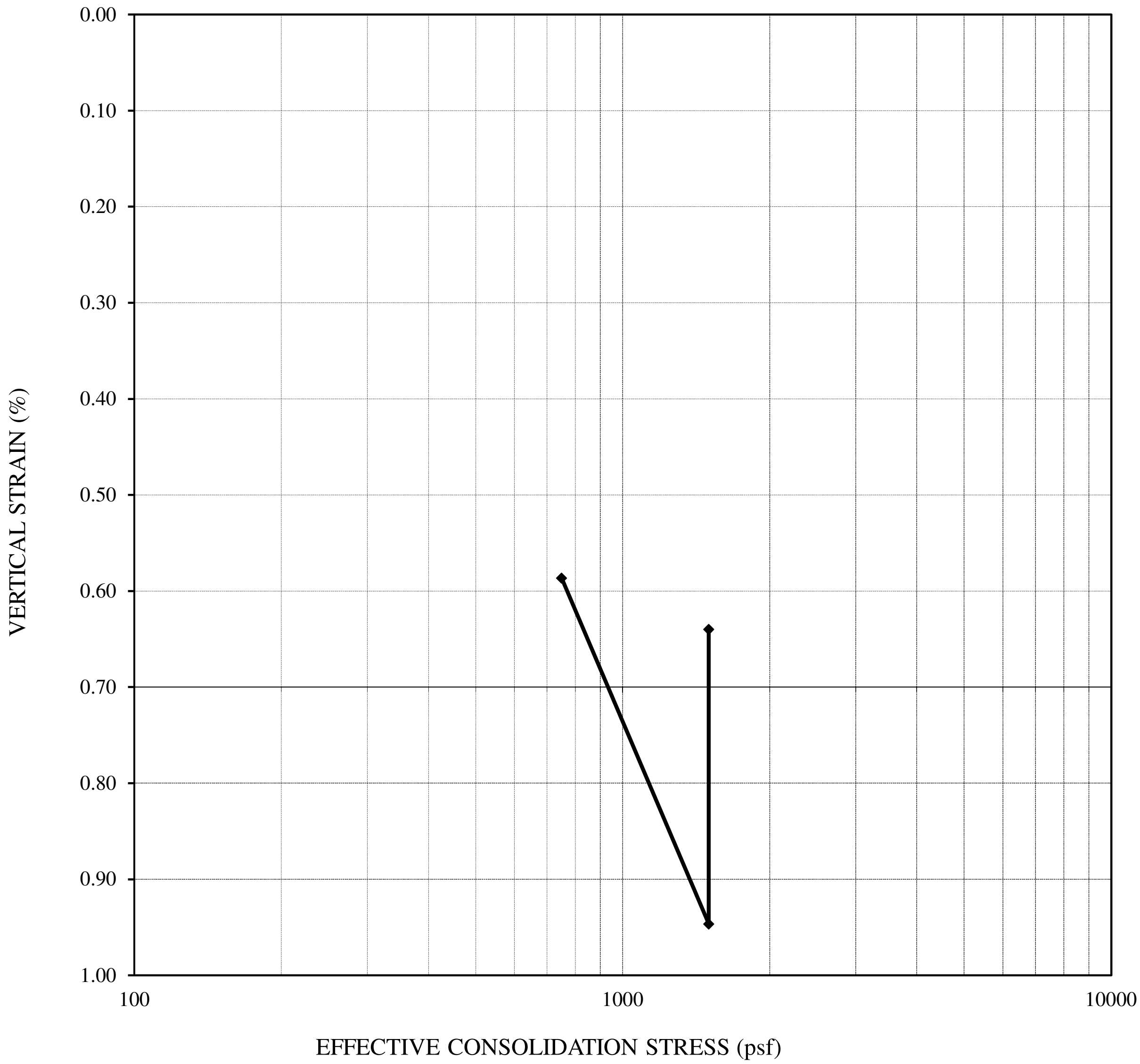
## GRAIN SIZE DISTRIBUTION - ASTM D422

Don Bingham
4437 N 2900 E
Liberty, Utah
Project Number: 1348-002

**Plate**

**C - 3**

C_GSD EXPLORATOIN LOGS.GPJ GEOSTRATA.GDT 7/20/18




| Sample Location | Depth (ft) | Classification | $\gamma_d$ (pcf) | MC (%) | Inundation Load (psf) | Swell (%) | Collapse (%) |
|---|---|---|---|---|---|---|---|
| **TP-1** | **2.0** | **Clayey SAND** | **102.5** | **18.1** | **1500** | **0.31** | **---** |
| | | | | | | | |
| | | | | | | | |
| | | | | | | | |
| | | | | | | | |

GeoStrata



**1-D SWELL/COLLAPSE TEST**

Don Bigham
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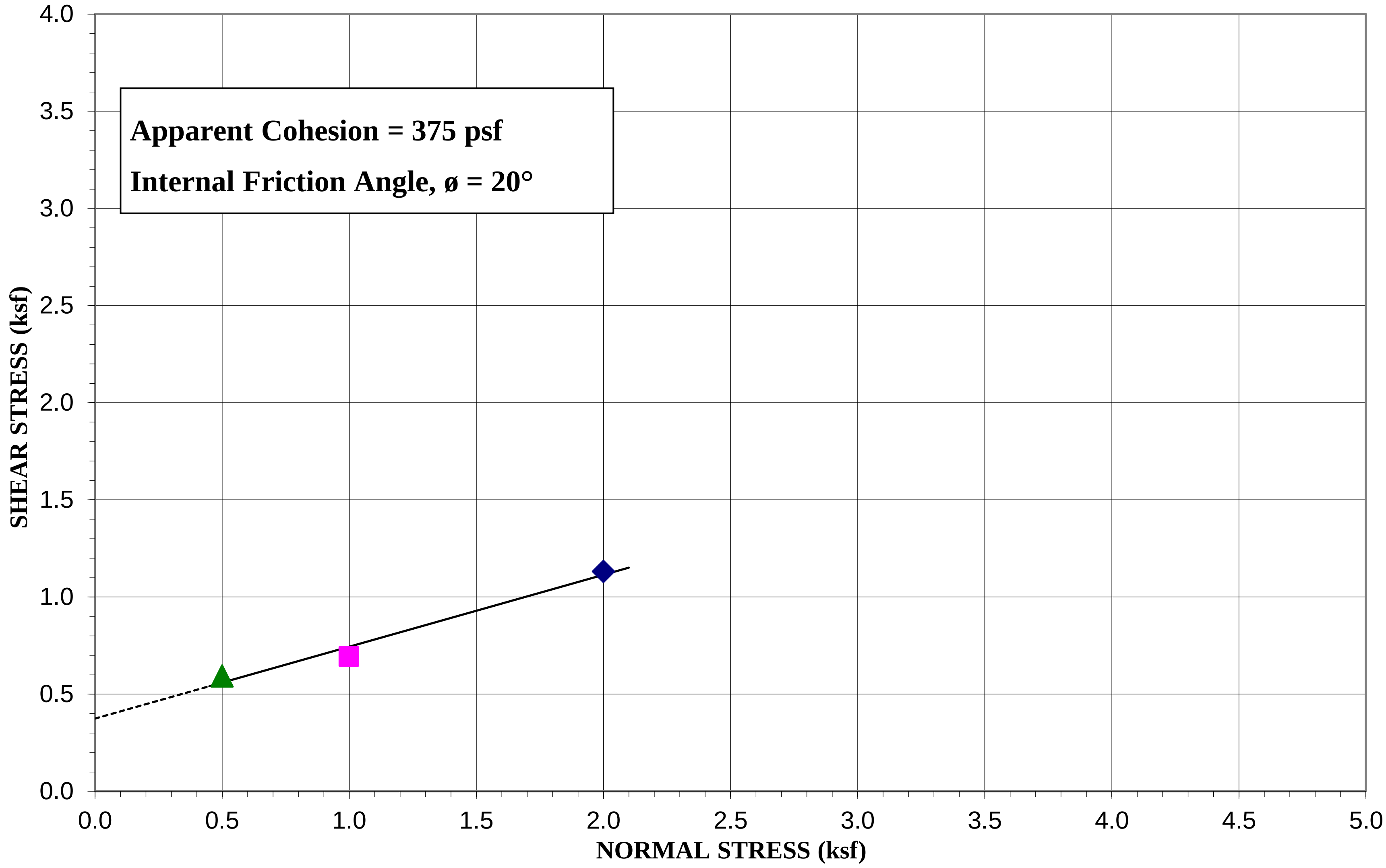
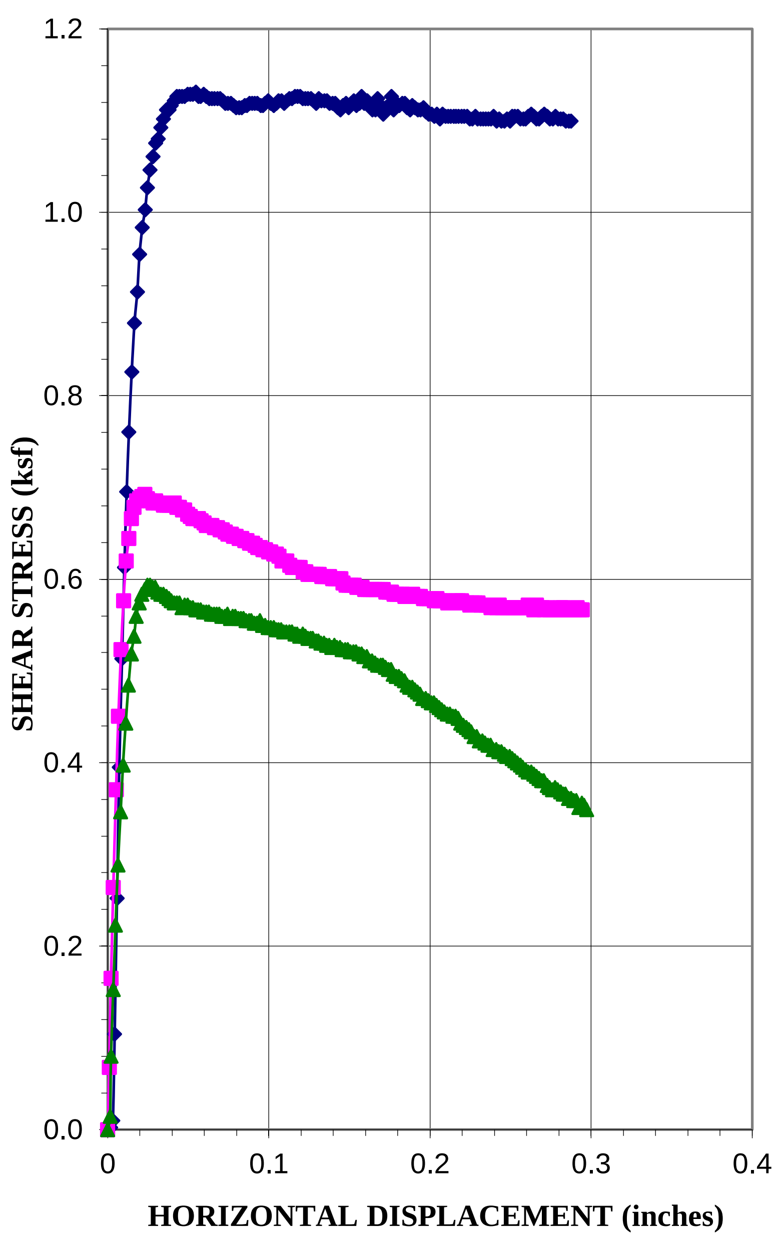
Liberty, Utah
Project Number: 1348-002

**Plate C-4**

# DIRECT SHEAR TEST




| | |
|---|---|
| **Sample Location:** | TP-1 @ 10 ft |
| **Type of Test:** | Consolidated Drained/Saturated |

| **Test No. (Symbol)** | **1 (◆)** | **2 (■)** | **3 (▲)** |
|---|---|---|---|
| **Sample Type** | Remolded | | |
| **Initial Height, in.** | 0.845 | 0.871 | 0.875 |
| **Diameter, in.** | 2.5 | 2.5 | 2.5 |
| **Dry Density Before, pcf** | 115.7 | 111.5 | 111.2 |
| **Dry Density After, pcf** | 117.9 | 113.7 | 113.3 |
| **Moisture % Before** | 9.5 | 10.7 | 10.0 |
| **Moisture % After** | 19.1 | 20.1 | 19.7 |
| **Saturation, % Before** | 58.9 | 58.9 | 54.3 |
| **Saturation, % After** | 125.7 | 117.0 | 113.9 |
| **Normal Load, ksf** | 2.0 | 1.0 | 0.5 |
| **Shear Stress, ksf** | 1.13 | 0.69 | 0.59 |
| **Strain Rate** | 0.00333 IN/MIN | | |

| **Sample Properties** | |
|---|---|
| **Cohesion, psf** | 375 |
| **Friction Angle, $\phi$** | 20 |
| **Liquid Limit, %** | 28 |
| **Plasticity Index, %** | 10 |
| **Percent Gravel** | 0.1 |
| **Percent Sand** | 60.8 |
| **Percent Passing No. 200 sieve** | 39.1 |
| **Classification** | SC |

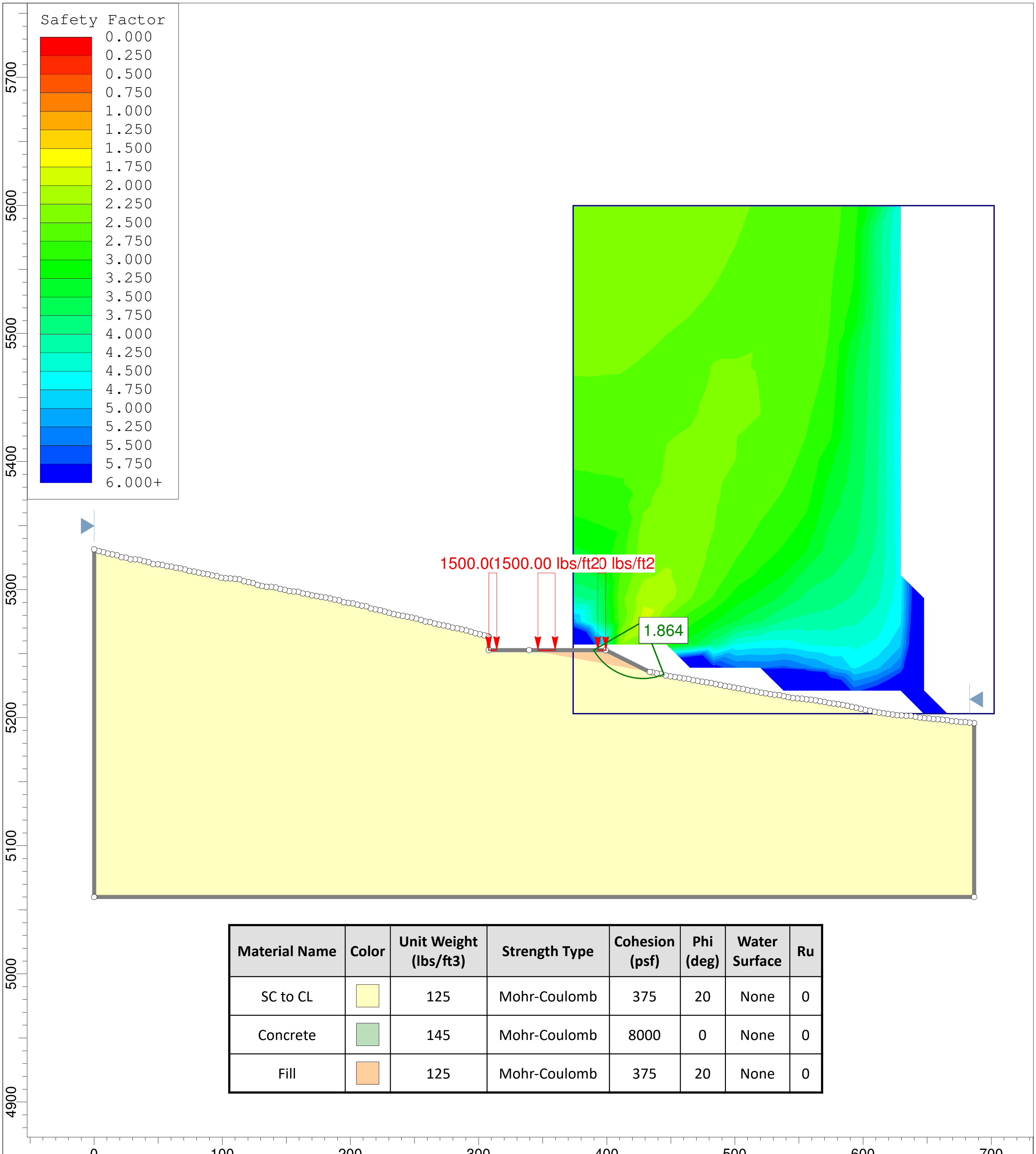
**PROJECT:** 4437 N 2900 E, Liberty, UT

**PROJECT NO.:** 1348-002

GeoStrata



**Plate C-5**




| Material Name | Color | Unit Weight (lbs/ft3) | Strength Type | Cohesion (psf) | Phi (deg) | Water Surface | Ru |
|---|---|---|---|---|---|---|---|
| SC to CL | | 125 | Mohr-Coulomb | 375 | 20 | None | 0 |
| Concrete | | 145 | Mohr-Coulomb | 8000 | 0 | None | 0 |
| Fill | | 125 | Mohr-Coulomb | 375 | 20 | None | 0 |

GeoStrata



**Profile-A - Peak Strength Static**

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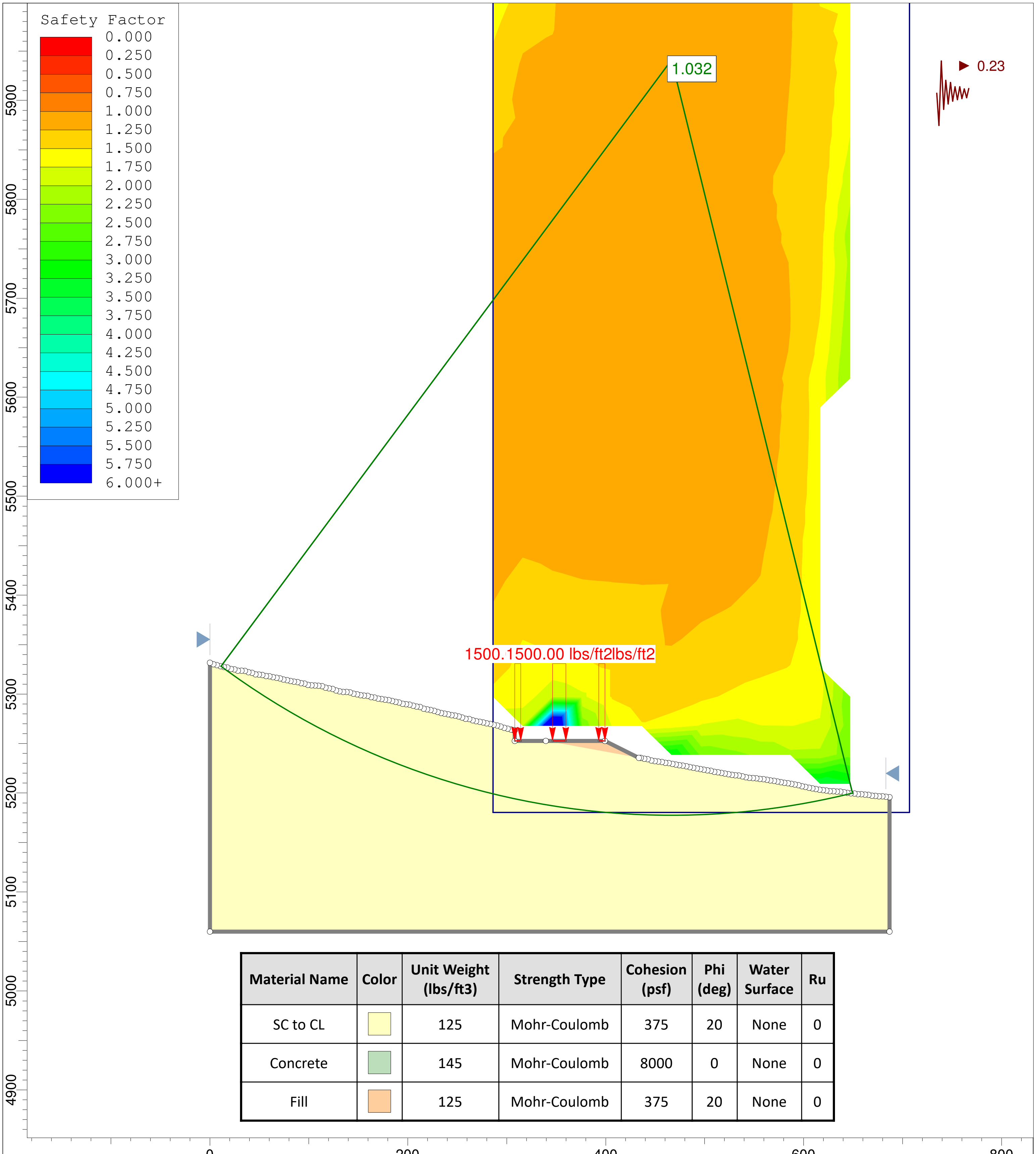
Liberty, UT
Project Number: 1348-002

**Plate D-1**




| Material Name | Color | Unit Weight (lbs/ft3) | Strength Type | Cohesion (psf) | Phi (deg) | Water Surface | Ru |
|---|---|---|---|---|---|---|---|
| SC to CL | | 125 | Mohr-Coulomb | 375 | 20 | None | 0 |
| Concrete | | 145 | Mohr-Coulomb | 8000 | 0 | None | 0 |
| Fill | | 125 | Mohr-Coulomb | 375 | 20 | None | 0 |

GeoStrata



**Profile-A - Peak Strength Pseudo Static**

Donald Bingham
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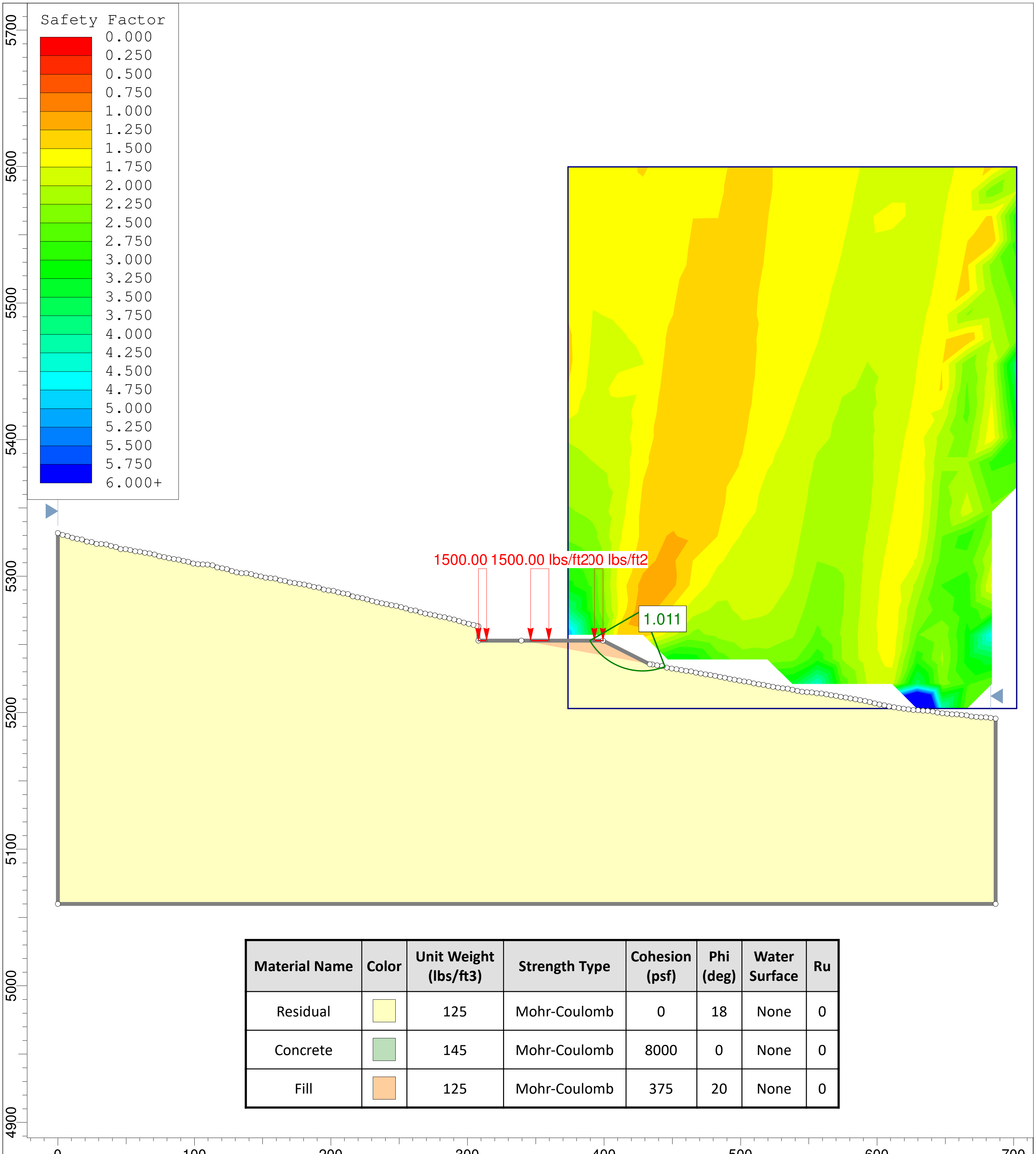
Liberty, UT
Project Number: 1348-002

**Plate D-2**




| Material Name | Color | Unit Weight (lbs/ft3) | Strength Type | Cohesion (psf) | Phi (deg) | Water Surface | Ru |
|---|---|---|---|---|---|---|---|
| Residual | | 125 | Mohr-Coulomb | 0 | 18 | None | 0 |
| Concrete | | 145 | Mohr-Coulomb | 8000 | 0 | None | 0 |
| Fill | | 125 | Mohr-Coulomb | 375 | 20 | None | 0 |

GeoStrata



**Profile-A - Residual Strength Static**

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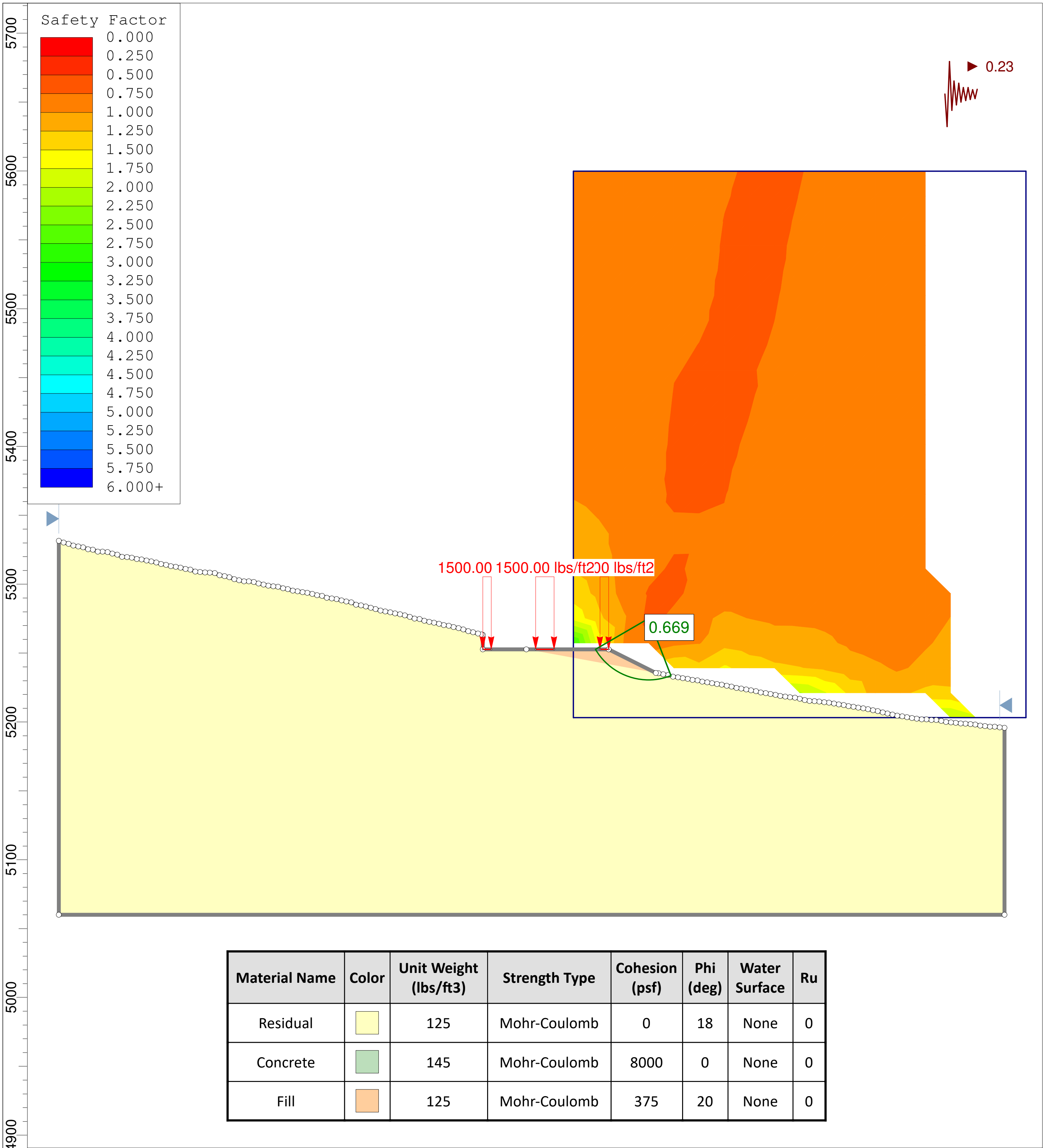
Liberty, UT
Project Number: 1348-002

**Plate D-3**




| Material Name | Color | Unit Weight (lbs/ft3) | Strength Type | Cohesion (psf) | Phi (deg) | Water Surface | Ru |
|---|---|---|---|---|---|---|---|
| Residual | | 125 | Mohr-Coulomb | 0 | 18 | None | 0 |
| Concrete | | 145 | Mohr-Coulomb | 8000 | 0 | None | 0 |
| Fill | | 125 | Mohr-Coulomb | 375 | 20 | None | 0 |

GeoStrata



**Profile-A - Residual Strength Pseudo Static**

Donald Bingham
4437 N 2900 E
Liberty, UT
Project Number: 1348-002

**Plate D-4**