GeoStrata

14425 South Center Point Way Bluffdale, Utah 84065
Phone (801) 501-0583 | Fax (801) 501-0584

Geotechnical Investigation
LPC – Upper System Tank
6970 North Durfee Way
Liberty, UT.

GeoStrata Job No. 746-010

June 27, 2016

Prepared for:

Gardner Engineering
5150 South 375 East
Ogden, UT 84405
Attn: Mike Durtschi, E.I.T.

Learn More

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PROFESSIONAL ENGINEER
6/27/16
8846946
J. SCOTT
SEAL
STATE OF UTAH

Daniel J. Brown, E.I.T.
Staff Engineer

J. Scott Seal, P.E.
Project Engineer

GeoStrata
14425 South Center Point Way
Bluffdale, UT 84065
(801) 501-0583

June 27, 2016

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APPENDICES

1.0 EXECUTIVE SUMMARY

This report presents the results of a Geotechnical Investigation conducted for the proposed buried 250,000-gallon water tank to be constructed at 6970 North Durfee Way in Liberty, 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 the design and construction of foundations and cut/fill slopes.

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.

Subsurface soil conditions were investigated by advancing two exploratory test pits to depths ranging from 3 to 5½ feet below the existing site grade. Based on our observations and geologic literature review, the subject site is overlain by 6 inches of gravelly topsoil. Underlying the topsoil we encountered Upper Proterozoic-aged feldspathic quartzite bedrock. This bedrock deposit persisted to the full depth of our investigation.

Conventional ring-wall or mat footings may be used to support the proposed structure. Bedrock will likely be encountered in excavations for the proposed water tank. Heavy excavation equipment and/or blasting may be necessary to remove bedrock and oversized material. Foundations may be established upon undisturbed native soils or bedrock and proportioned using a net bearing capacity of **4,000 psf.** Differential settlement of the structure if founded as described in this report should be on the order of ½ the total settlement over 30 feet.

All temporary excavations may be sloped at a maximum 1 horizontal to 1 vertical (1H:1V) slope. Permanent slopes may be designed using a 3 horizontal to 1 vertical (3H:1V) slope to maintain slope stability.

NOTE: This executive summary is not intended to replace the report of which it is part and should not be used separately from the report. The executive summary omits a number of details, any one of which could be crucial to the proper application of this report.

2.0 INTRODUCTION

2.1 PURPOSE AND SCOPE OF WORK

This report presents the results of a Geotechnical Investigation conducted for the proposed buried 250,000-gallon water tank to be constructed at 6970 North Durfee Way in Liberty, 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 the design and construction of foundations and cut/fill slopes.

The scope of work completed for this study included a site reconnaissance, subsurface exploration, soil sampling, engineering analyses, and preparation of this report. Our services were performed in accordance with our proposal 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 property is located at 6970 North Durfee Way in Liberty, Utah. An existing water tank was observed adjacent to the location of the proposed water tank. Footings for the proposed water tank are to be constructed at a depth of 16 to 19 feet below existing ground surface and the top of the tank will sit at an elevation near the current ground surface. Grading will be conducted both uphill and downhill from the proposed tank to create a level ground surface around the perimeter of the tank, which will steepen slopes on both the uphill and downhill sides of the tank.

Our understanding of the proposed development is based on information provided by the client, including a preliminary site plan. No construction drawings were available at the time this investigation was completed; however, we understand that site development will involve constructing a 250,000-gallon water tank approximately 16 feet in height. Construction of the tank will require both cut and fill slopes into the hillside. We understand that these slopes will have grades of 3H:1V (horizontal to vertical). This report does not address the construction or design of any associated pipelines.

3.0 METHODS OF STUDY

3.1 FIELD INVESTIGATION

As a part of this investigation, subsurface soil conditions were explored by completing two investigatory test pits to depths ranging from 3 to 5½ feet below the existing site grade. The approximate locations of the explorations are shown on Plate A-2, (*Exploration Location Map)* in Appendix A. Exploration points were placed to provide a representative cross section of the subsurface soil conditions. Test pit refusal was encountered at the final depth of each of the test pits on competent bedrock. Subsurface soil conditions as encountered in the explorations were logged at the time of our investigation by a geotechnical engineer and are presented on the enclosed Test Pit Logs, Plates B-1 through B-2 in Appendix B. A *Soils Symbols Description Key* used in the test pit logs is included as Plate B-3.

The test pits were excavated using a CAT 320C trackhoe. Bulk soil samples were obtained from each of the test pit locations; undisturbed soil samples were not obtained due to the granular nature of the soil profile. All samples were transported to our laboratory to evaluate the 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.

3.2 ENGINEERING ANALYSIS

Engineering analyses were performed using soil data obtained from the field observations. 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 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

At the time of our subsurface investigation the site of the proposed tank existed as a vacant, undeveloped lot with light to moderate amounts of vegetation cover. An existing water tank was present, approximately 50 feet northwest of the proposed water tank. Numerous angular cobbles and boulders were observed on the surface of the property. In general, the property slopes down moderately to steeply to the southeast and east. The site is located at an elevation of approximately 5,970 feet above mean sea level.

4.2 SUBSURFACE CONDITIONS

As previously discussed, the subsurface soil conditions were explored at the site by excavating two test pits at representative locations at the subject site. The test pits extended to depths ranging from 3 to 5½ feet below existing site grade. The soils encountered in the test pit explorations were visually classified and logged during our field investigation and are included on the test pit logs in Appendix B (Plates B-1 through B-2). The subsurface conditions encountered during our investigation are discussed below.

4.2.1 *Soils/Bedrock*

Based on our observations and geologic literature review, the subject site is overlain by 6 inches of gravelly topsoil. Underlying the topsoil we encountered Upper Proterozoic-aged feldspathic quartzite bedrock. This bedrock deposit persisted to the full depth of our investigation. The geologic units encountered are discussed below:

<u>Topsoil:</u> Generally consists of brown, moist, Silty GRAVEL (GM) with sand. These soils typically have an organic appearance and texture, and contain numerous fine roots. Topsoil and is expected to overlie the majority of the site.

<u>Upper Proterozoic-aged Feldspathic Quartzite Bedrock:</u> The bedrock encountered at the site consisted of a moderately weathered, strong, red-brown feldspathic quartzite. This unit was encountered at depths ranging from ½ to 5½ feet below the existing site grade. Trackhoe refusal was encountered in both test pits in this bedrock unit.

4.2.2 *Groundwater*

Groundwater was not encountered in any of the explorations completed for this investigation, and is not expected to impact the development. It is our experience that during snowmelt, runoff, irrigation on the property and surrounding properties, high precipitation events, and other activities, the groundwater level can rise several feet. Fluctuations in the groundwater level should be expected over time.

5.0 GEOLOGIC CONDITIONS

5.1 GEOLOGIC SETTING

The subject site is located at an elevation of approximately 5,980 feet within the foothills of the northeastern portion of Ogden Valley. The Ogden Valley is a fault trough bounded on both the east and the west by faults that dip towards the middle of the valley. This fault trough contains unconsolidated deposits of clay, sand, and gravel, whose thickness in places is more than 600 feet. These materials are stream and lake deposits and in places are well sorted and stratified. The lake sediments were laid down in a small lake that occupied Ogden Valley and that was connected with glacial Lake Bonneville at its high stage by an arm of water that occupied Ogden Canyon. More recently, sediments associated with the North, Middle, and South Forks of the Ogden River have been deposited (Leggette, R.M, and others, 1937).

5.2 TECTONIC SETTING

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 et. al, 2003). The site is located approximately 4 miles east of the of the nearest mapped portion of the Weber segment of the Wasatch fault zone. 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, and others, 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. The site is also located approximately 32 miles east of the East Great Salt Lake Fault Zone (Hecker, 1993). Evidence suggests that this fault zone has been active during the Holocene (0 to 30,000 yrs) and has segment lengths comparable to that of the Wasatch Fault Zone, indicating that it is capable of producing earthquakes of a comparable magnitude (7.5 Ms). Each of the faults listed above have shown evidence of movement during the Holocene, and are 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 exploration, it is our opinion that this location is best described as a Site Class B. The spectral accelerations are shown in the table below. The spectral accelerations are calculated based on the site's approximate latitude and longitude of 41.3853° and -111.8999° respectively and the USGS Seismic Design Maps web based application. Based on IBC, the site coefficients are F_a=1.00 and F_v= 1.00. From this procedure the peak ground acceleration (PGA) is estimated to be 0.43g.

MCE_R Seismic Response Spectrum Spectral Acceleration Values for IBC Site Class B[a]

Site Location: **Latitude = 41.3853 N** **Longitude = 111.8999 W**	**Site Class B Site Coefficients:** **Fa = 1.00** **Fv = 1.00**
Spectral Period (sec)	**Response Spectrum Spectral Acceleration (g)**
0.2	$S_{MS}=(F_a \cdot S_S=1.00*1.06) = 1.06$
1.0	$S_{M1}=(F_v \cdot S_1=1.00*0.37) = 0.37$

[a] IBC 1613.3.4 recommends scaling the MCE values by 2/3 to obtain the design spectral response acceleration values; values reported in the table above have not been reduced.

6.0 ENGINEERING ANALYSIS AND RECOMMENDATIONS

6.1 GENERAL CONCLUSIONS

Supporting data upon which the following recommendations are based have been presented in the previous sections of this report. The recommendations presented herein are governed by the physical properties of the earth materials encountered as part of our subsurface exploration and the anticipated design data discussed in the PROJECT DESCRIPTION section. If subsurface conditions other than those described herein are encountered in conjunction with construction, and/or if design and layout changes are initiated, GeoStrata must be informed so that our recommendations can be reviewed and revised as changes or conditions may require.

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.

The following sub-sections present our recommendations for general site grading, excavation, temporary cut stability, foundations and moisture protection.

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 slabs-on-grade. Site grading is also recommended to provide proper drainage away from the water tank foundation and moisture control on the subject property and to aid in preventing differential movement in foundation materials as a result of variations in subgrade moisture conditions.

6.2.1 General Site Preparation and Grading

Based on our current understanding of the project, the tank is to be buried. Although not expected, any underground obstructions or existing utilities under the proposed tank footprint should be removed and/or rerouted. Any resulting removal cavities should be properly backfilled with compacted fill.

Within areas to be graded (below proposed structures, fill sections, concrete flatwork), any existing vegetation, debris, or otherwise unsuitable soils should be removed. Any soft, loose, frozen, disturbed, or undocumented fill (if encountered) soils should also be removed. Following the removal of vegetation, unsuitable soils, and loose or disturbed soils, as described above, site grading may be conducted to bring the site to design elevations.

Based on our observations in the test pits excavated for the site investigation, there are approximately 0 to 6 inches of topsoil throughout the proposed development. This topsoil should be removed prior to placement of structural fill, structures, and concrete flatwork.

A GeoStrata representative should observe the site preparation and grading operations to assess that the recommendations presented in this report are complied with and to provide an assessment of the exposed soils

6.2.2 *Temporary Excavation Stability*

As mentioned previously, the tank is to be fully buried. As such, the worst-case scenario of a fully-buried tank (20 foot excavation) was modeled using Slide, a computer program incorporating (among others) Bishop's Simplified Method of analysis. Calculations for stability were developed by searching for the minimum factor of safety for a circular-type failure. Homogeneous earth materials (lacustrine sands and clays) and arcuate failure surfaces were assumed.

Strength parameters used in our analyses were developed based on our observations within the test pits, experience, and engineering judgment. Based on our observations, the site is underlain by approximately 0.5 feet of topsoil and 2.5 to 5 feet of weathered bedrock which overlies relatively solid, un-weathered bedrock. Strength testing was not feasible for these materials due to their granular nature. As such, the following assumed strength parameters were assigned;

Soil Type	Friction Angle (ϕ)	Cohesion (psf)
Soil (GM)	33	0
Weathered Bedrock	35	500
Fresh Bedrock	45	3000

Strength values and the modeled depth of the soil unit were based on explorations completed on the site.

Groundwater was not encountered during our investigation, nor were there any indications, such as springs or groundwater seeps, that groundwater exists near the current site grade. As such, groundwater is anticipated to be relatively deep and was not modeled as part of this slope stability analysis. If groundwater or high moisture conditions are encountered during construction, GeoStrata should be notified as updated stability modeling will need to be performed prior to continued construction.

The modeled geometry of the slope as well as the anticipated structure location were based on information obtained from the client. Based on this information, we modeled the critical state (maximum cut height of 20 feet) although the recommendations made within are applicable for slopes of lesser heights. Results of our stability analysis for temporary slopes are included in Appendix C (Plate C-1 to Plate C-2).

It is recommended that the temporary slope may be constructed at a 1 horizontal to 1 vertical slope (1H:1V). It is possible that this slope may be steepened if it is observed that the fracture sets change in direction or dip that reduce the potential for these sets to adversely impact the excavation. GeoStrata should inspect the exposed bedrock prior to steepening the slope beyond the recommended 1H:1V slope.

Loose soil and rock near the top of excavations should be benched back to minimize raveling problems. Suspect rocks and material near the top of the excavation should be knocked loose by equipment or by hand to avoid a rock fall hazard to workers. We further recommend that a minimum of 4 feet be provided around the outside diameter of the tank to allow a fall zone for loose material that may fall into the excavation. Additionally, netting, fencing or material will likely need to be placed on the slope to protect workers from raveling of near surface soils and potential rock fall hazards. The contractor is ultimately responsible for site safety and 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. Qualified personnel should inspect all excavations frequently to evaluate stability. We recommend that a GeoStrata representative be on-site during all excavations to assess the exposed foundation soils.

6.2.3 *Stability of Resulting Permanent Slopes*

Using the strength values described above, the stability of the resulting permanent slopes located to the northeast and southwest of the tank were modeled for both static and pseudo-static conditions. Using the assumptions for soil strength parameters described previously, the resulting permanent slopes as shown on the provided site plan with a maximum slope steepness of

2.5H:1V meet the minimum factor-of-safety of 1.5 and 1.1 for static and seismic conditions, respectively. The results of the stability analyses are presented in Appendix C (Plate C-3 to Plate C-6).

6.2.4 *Rippability and Oversize Material*

As mentioned previously, the results of our field investigation indicate that the site is underlain by 0.5 feet of soil which in turn overlies 2½ to 5 feet of moderately weathered bedrock. Based on the seismic velocities measured during our survey, it is likely that the weathered bedrock will require large excavation equipment to rip. The relatively fresh bedrock contains horizons that do not appear to be rippable, and will likely require blasting to excavate.

6.2.5 *Structural Fill*

All fill placed for the support of the proposed water tank, appurtenant structures, or concrete flatwork should consist of structural fill. We anticipate that the majority of the on-site coarse-grained soils and residual bedrock will be suitable for use as structural fill provided that they are free of vegetation, frozen material, and debris, and contain no inert materials larger than 6 inches in nominal size. Alternatively, structural fill may consist of an imported granular soil with a maximum of 50 percent passing the No. 4 mesh sieve, a maximum fines content (minus No.200 mesh sieve) of 15 percent. The fines should have a liquid limit less than 25 and plasticity index less than 10. Soil not meeting the aforementioned criteria may be suitable for use as structural fill. These soils should be evaluated on a case-by-case basis and should be approved by the Geotechnical Engineer prior to use.

Structural fill should be placed in maximum 8-inch loose lifts and compacted by equipment capable of compacting an 8-inch left on a horizontal plane, unless otherwise approved by the Geotechnical Engineer. Structural fill beneath the tank base should be compacted to at least 95 percent of the maximum dry density, as determined by ASTM D-1557. The moisture content should be slightly above optimum at the time of compaction. Also, prior to placing fill, the excavations should be observed by GeoStrata to confirm that unsuitable materials have been removed. In addition, proper grading should precede placement of structural fill, as described in the General Site Preparation and Grading subsection (6.2.1) of this report.

Utility trenches backfilled below pavement sections or structures should be backfilled with structural fill compacted to at least 95 percent of ASTM D-1557. Trenches in non-structural areas should be backfilled and compacted to approximately 90 percent of the maximum density.

6.3 FOUNDATIONS

Conventional ring-wall or mat foundation bearing entirely on either native granular soils, a minimum of 1½ feet of structural fill, or bedrock may be used to support the proposed tank. Conventional spread and strip footings may be proportioned for a maximum net allowable bearing capacity of **4,000 pounds per square foot (psf)**.

Based on our field observations and considering the presence of shallow bedrock, we recommend that the footings for the proposed tank be established entirely on bedrock or entirely on soil. If a bedrock/soil transition zone is encountered during excavation, then the footings should be deepened such that all footings bear on competent bedrock. Alternatively, the building pad may be over-excavated 18 inches below the bottom of the proposed footings and replaced with structural fill, such that the footings bear entirely on a uniform fill blanket. Footings should be a minimum of 3 feet wide and exterior shallow footings should be embedded at least 42 inches below final grade for frost protection and confinement purposes. Isolated interior footings should be a minimum of 4 feet wide and also be embedded a minimum of 42 inches below final grade for confinement purposes. Finally, it is recommended that a drainage system be constructed under the interior of the tank. Recommendations for a drainage system can be found in Section 6.5.2 of this report. All footing excavations should be observed by the Geotechnical Engineer prior to footing placement.

Settlements of properly designed and constructed conventional footings, founded as described above, are anticipated to be less than 1 inch. Differential settlements are expected to be on the order of ½ the total settlement over a distance of 30 feet.

6.4 LATERAL RESISTANCE AND EARTH PRESSURES

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 subgrade. In determining the frictional resistance, a coefficient of friction of 0.43 for structural fill against concrete should be used.

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*	0.29	37
At-rest**	0.46	57
Passive*	3.39	424
Seismic Active***	0.37	46
Seismic Passive***	-0.71	-89

* 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.

6.5 MOISTURE PROTECTION AND DRAINAGE

Over-wetting of the soils by natural or man-made means prior to or during construction may result in softening and pumping, causing equipment mobility problems and difficulty in achieving uniform compaction. Every effort should be taken to ensure positive drainage away from the tank. The recommended minimum slope is five percent (5%) away from the tank. Moisture should not be allowed to infiltrate the subgrade in the vicinity of, or upslope from, the tank.

6.5.1 Surface Drainage

Final design grades around the tank should direct runoff away from foundation elements. Design strategies to minimize ponding and infiltration near the tank should be implemented. Diversion berms or ditches should be placed uphill of the tank, if applicable, to direct runoff away from the tank area. Additionally, since the tank is to be partially or completely buried, a drainage system should be considered on the uphill portion of the tank wall to prevent the buildup of hydrostatic pressures.

6.5.2 Tank Under-Drainage

Consideration should be given to installing a subdrainage system under the tank. This system should consist of a 40-mil thick polyethylene or high-density polyethylene (HDPE) membrane placed under the tank, sloped for drainage. This liner should then be covered with at least a 6-inch thick layer of either crushed aggregate base or pervious backfill. Perforated Schedule 80 polyvinyl chloride (PVC) pipe should be embedded in the gravel, spaced no more than 15-feet on center under the tank, and wrapped in a non-woven filter fabric such as a Mirafi 140N or equivalent, with perforations facing down. This perforated pipe drainage system should collect any leakage under the tank, above the membrane, at the low collection point of the membrane. This drainage pipe can then be manifolded together for leak monitoring, and discharge by gravity to a low-lying suitable discharge, or to a sump with a pump.

7.0 CLOSURE

7.1 LIMITATIONS

The recommendations contained in this report are based on limited field exploration and our understanding of the proposed construction. This investigation was completed for the proposed water tank and should not be used for other projects. The subsurface data used in the preparation of this report were obtained from the explorations made for this investigation. It is likely that variations in the soil and groundwater conditions will exist. 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, our firm 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, our firm should also 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 the construction. GeoStrata staff should be on site to document compliance with these recommendations and to verify geologic conditions are as anticipated. Our services should include, but not necessarily be limited to, the following:

- Observations and testing during site preparation, earthwork and structural fill placement.
- Consultation as may be required during construction, including verification that the geologic conditions are as anticipated during excavation and design of shoring if deemed necessary based on actual geologic conditions encountered during construction.

We also recommend that project plans and specifications be reviewed by us as they are developed 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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Base Map:
Utah AGRC Hybrid Base Map

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Project Number: 746-010

Site Vicinity Map

PLATE
A-1

1:24,000

Base Map:
Utah AGRC Hybrid Base Map

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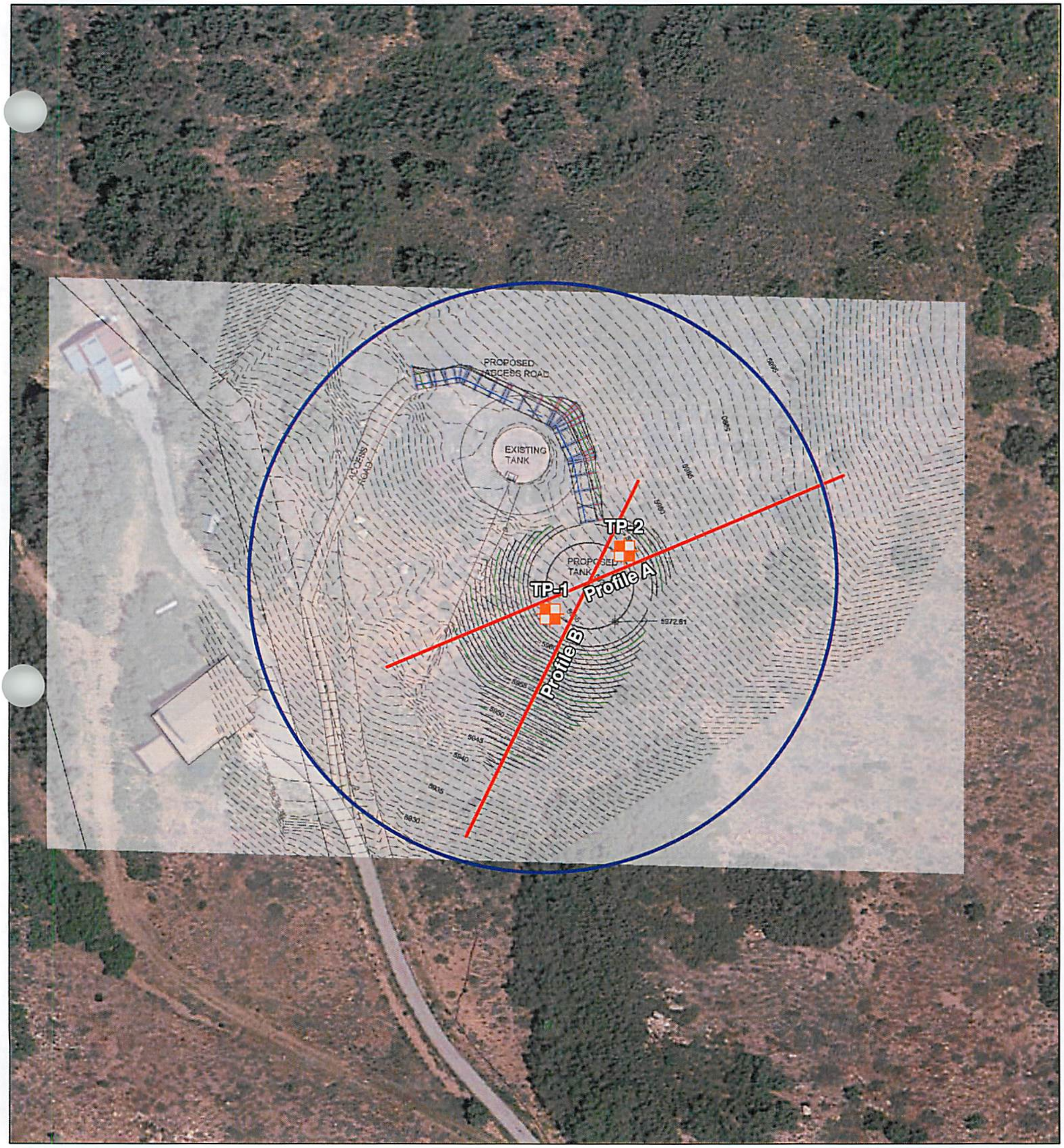
Project Number: 746-010

Site Vicinity Map

Plate A-1



0 30 60 120 180 240 Feet

1:1,000

Base Map:
Utah AGRC Hybrid Base Map
Site Plan, Gardner Engineering, 2016

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Legend

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

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Project Number: 746-010

Exploration Location Map

Plate A-2

DATE		
STARTED:	6/6/16	
COMPLETED:	6/6/16	
BACKFILLED:	6/6/16	

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Liberty, UT
Project Number 746-010

GeoStrata Rep: D. Brown
Rig Type: CAT 320 C

TEST PIT NO:
TP-1
Sheet 1 of 1

LOCATION

NORTHING | EASTING | ELEVATION

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	Moisture Content and Atterberg Limits (Plastic Limit, Moisture Content, Liquid Limit) 10 20 30 40 50 60 70 80 90
0	0					TOPSOIL; Silty GRAVEL with sand - brown, moist						
						BEDROCK; Feldspathic Quartzite - moderately weathered, hard, red-brown, white, light purple						
1												
	5					- test pit refusal @ 5.5-ft						
						Bottom of Test Pit @ 5.5 Feet						
2												
3												

GeoStrata

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

WATER LEVEL
- MEASURED
- ESTIMATED

NOTES:

Plate
B-1

DATE			
STARTED:	6/6/16	Gardner Engineering LPT - Upper System Tank Liberty, UT Project Number 746-010	GeoStrata Rep: D. Brown Rig Type: CAT 320 C
COMPLETED:	6/6/16		TEST PIT NO: TP-2 Sheet 1 of 1
BACKFILLED:	6/6/16		

LOCATION

NORTHING EASTING ELEVATION

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	Moisture Content and Atterberg Limits (Plastic Limit, Moisture Content, Liquid Limit) 10 20 30 40 50 60 70 80 90
0	0					TOPSOIL; Silty GRAVEL with sand - brown, moist						
						BEDROCK; Feldspathic Quartzite - moderately weathered, hard, red-brown, white, light purple						
						- test pit refusal @ 3-ft						
1						Bottom of Test Pit @ 3 Feet						
	5											
2												
3												

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

WATER LEVEL
- MEASURED
- ESTIMATED

NOTES:

Plate B-2

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

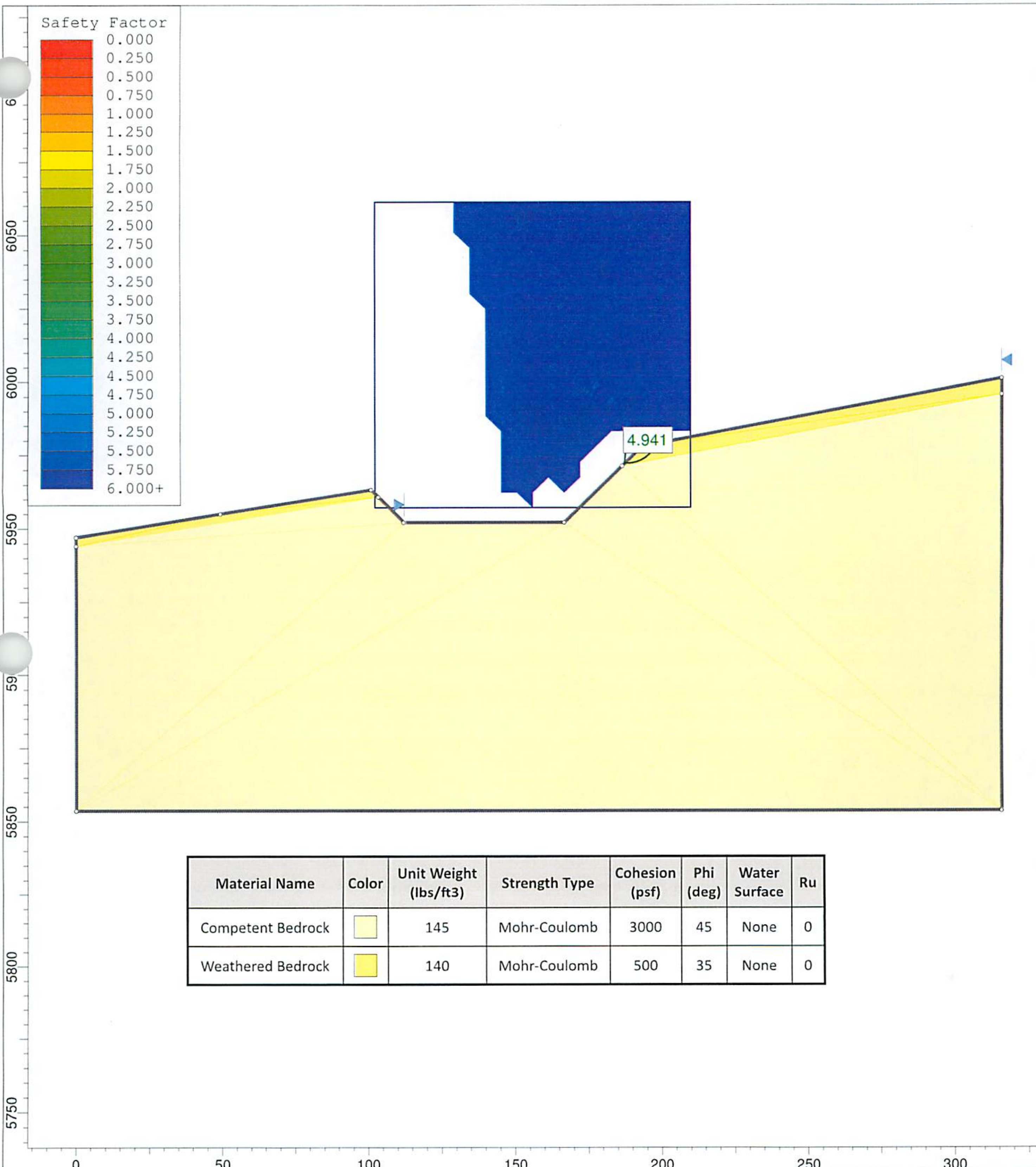
1. Lines separating strata on the logs represent approximate boundaries only. Actual transitions may be gradual.
2. No warranty is provided as to the continuity of soil conditions between individual sample locations.
3. Logs represent general soil conditions observed at the point of exploration on the 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

		TORVANE	POCKET PENETROMETER	FIELD TEST
CONSISTENCY	SPT (blows/ft)	UNTRAINED SHEAR STRENGTH (tsf)	UNCONFINED COMPRESSIVE STRENGTH (tsf)	
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.



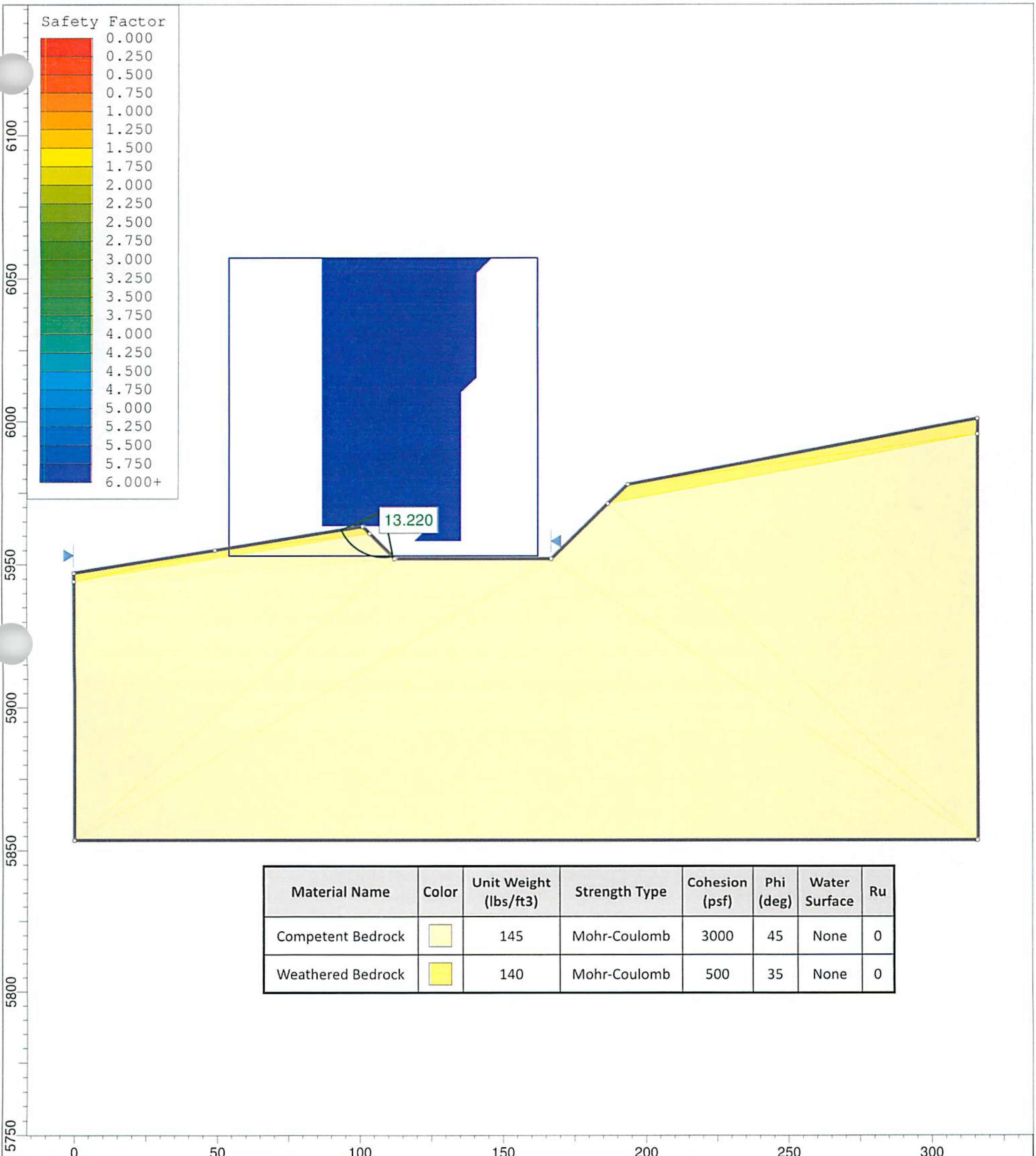
Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Ru
Competent Bedrock		145	Mohr-Coulomb	3000	45	None	0
Weathered Bedrock		140	Mohr-Coulomb	500	35	None	0

GeoStrata

Excavation Stability - Profile A Uphill

Gardner Engineering
LPT - Upper System Tank
Liberty, UT
Project Number: 746-001

Plate C-1



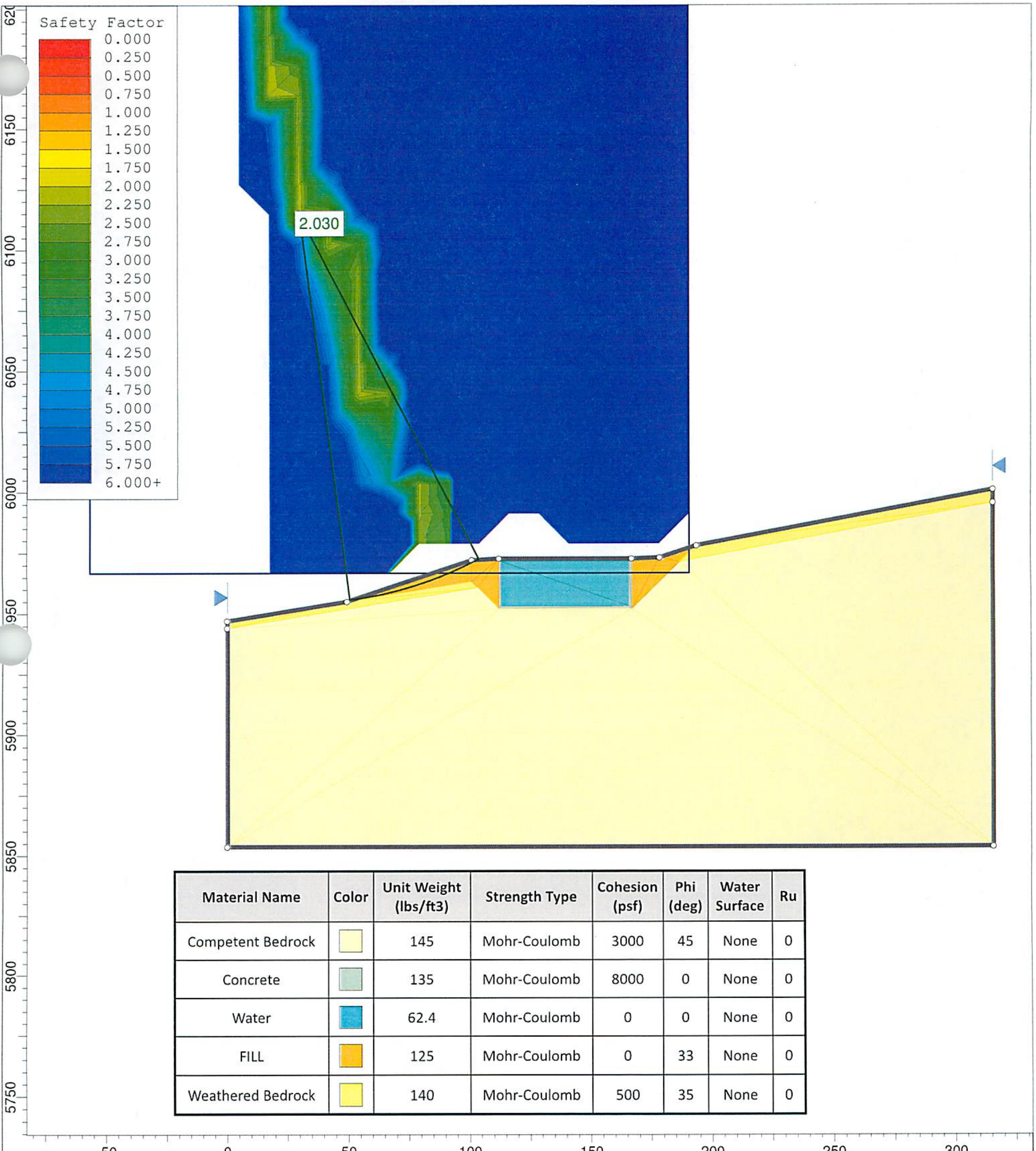
Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Ru
Competent Bedrock		145	Mohr-Coulomb	3000	45	None	0
Weathered Bedrock		140	Mohr-Coulomb	500	35	None	0

GeoStrata

Excavation Stability - Profile A Downhill

Gardner Engineering
LPT - Upper System Tank
Liberty, UT
Project Number: 746-001

Plate C-2



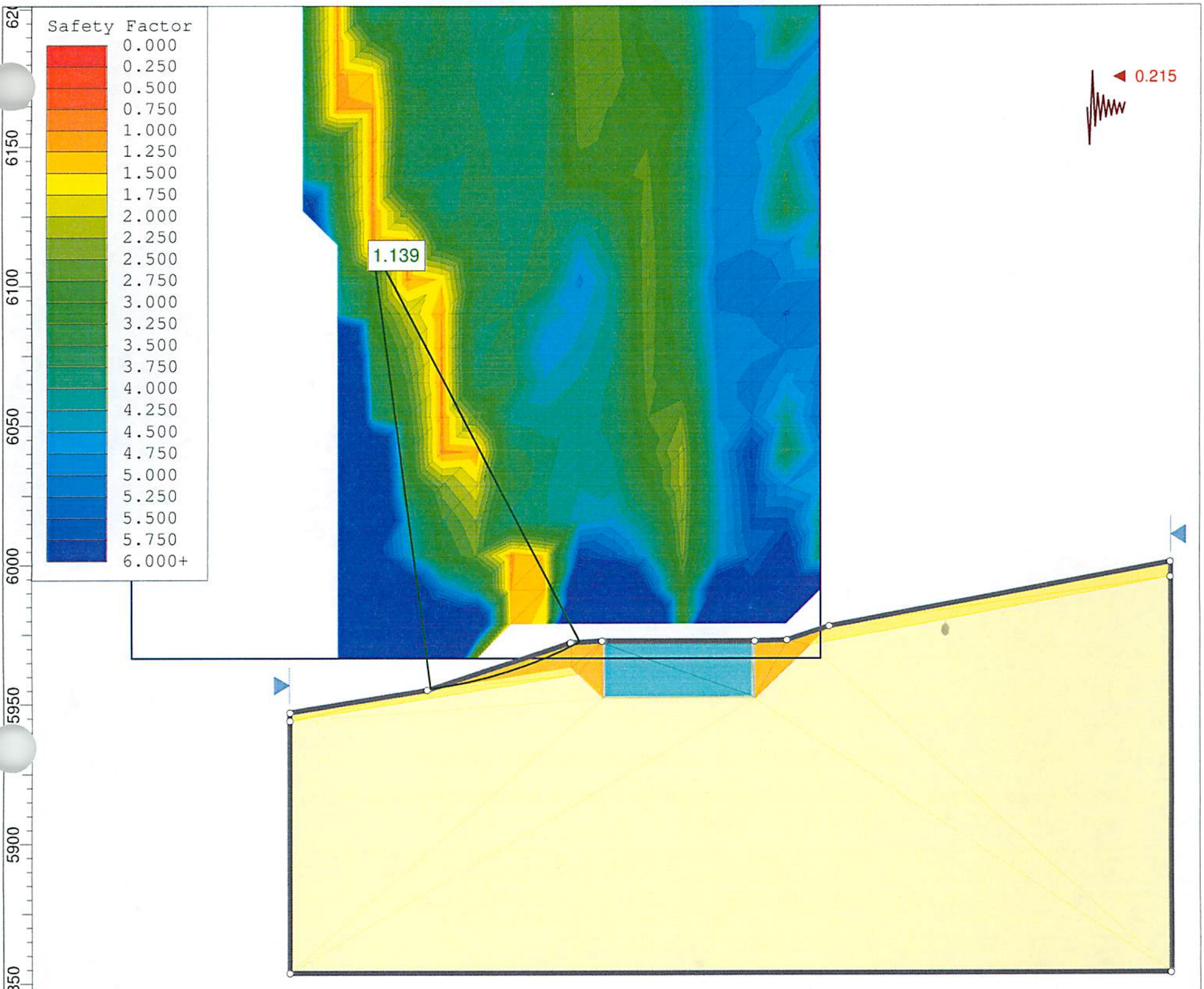
Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Ru
Competent Bedrock		145	Mohr-Coulomb	3000	45	None	0
Concrete		135	Mohr-Coulomb	8000	0	None	0
Water		62.4	Mohr-Coulomb	0	0	None	0
FILL		125	Mohr-Coulomb	0	33	None	0
Weathered Bedrock		140	Mohr-Coulomb	500	35	None	0

GeoStrata

Global Stability - Profile A Static

Gardner Engineering
LPT - Upper System Tank
Liberty, UT
Project Number: 746-001

Plate C-3



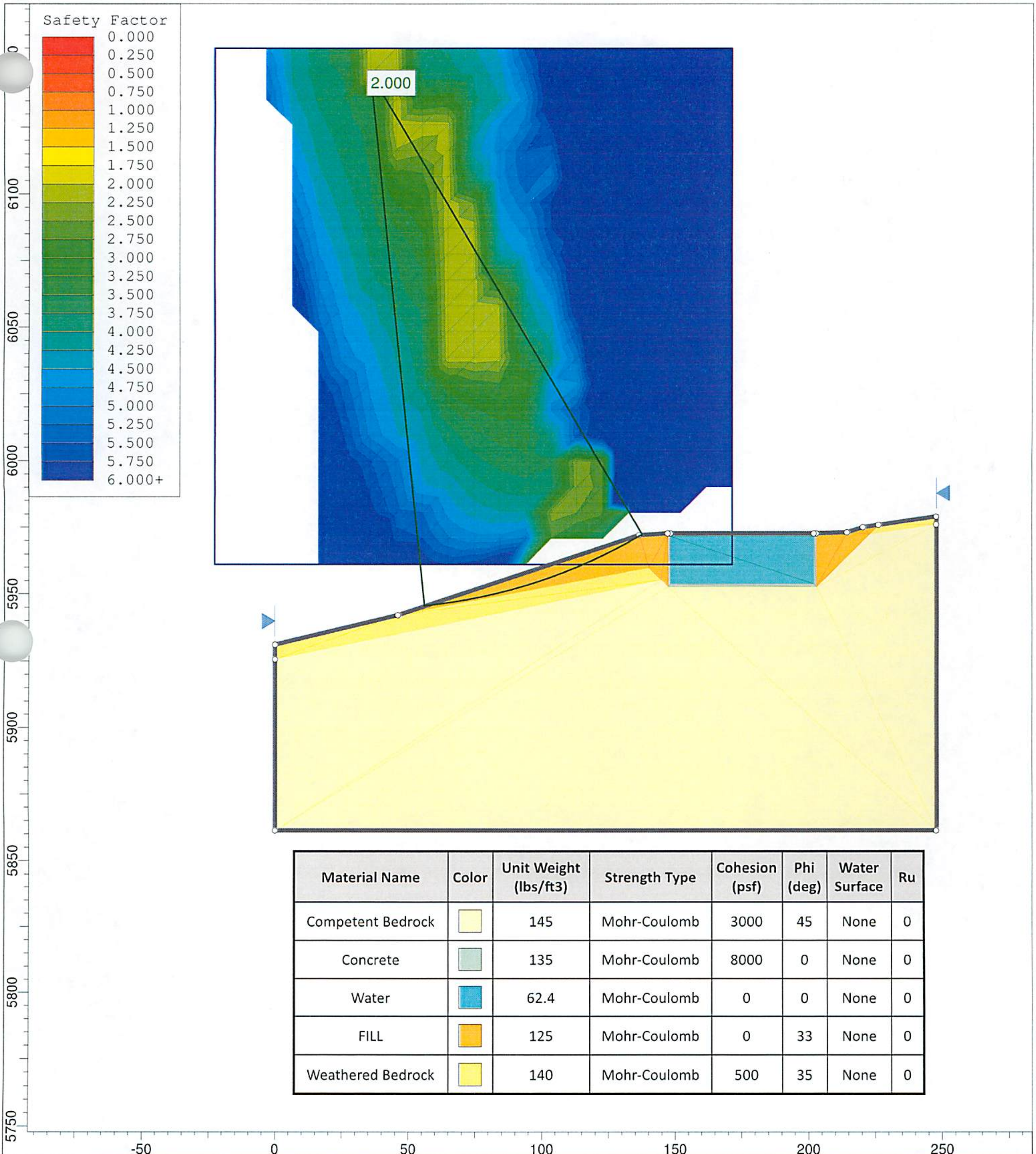
Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Ru
Competent Bedrock		145	Mohr-Coulomb	3000	45	None	0
Concrete		135	Mohr-Coulomb	8000	0	None	0
Water		62.4	Mohr-Coulomb	0	0	None	0
FILL		125	Mohr-Coulomb	0	33	None	0
Weathered Bedrock		140	Mohr-Coulomb	500	35	None	0

GeoStrata

Global Stability - Profile A Pseudo Static

Gardner Engineering
LPT - Upper System Tank
Liberty, UT
Project Number: 746-001

Plate C-4



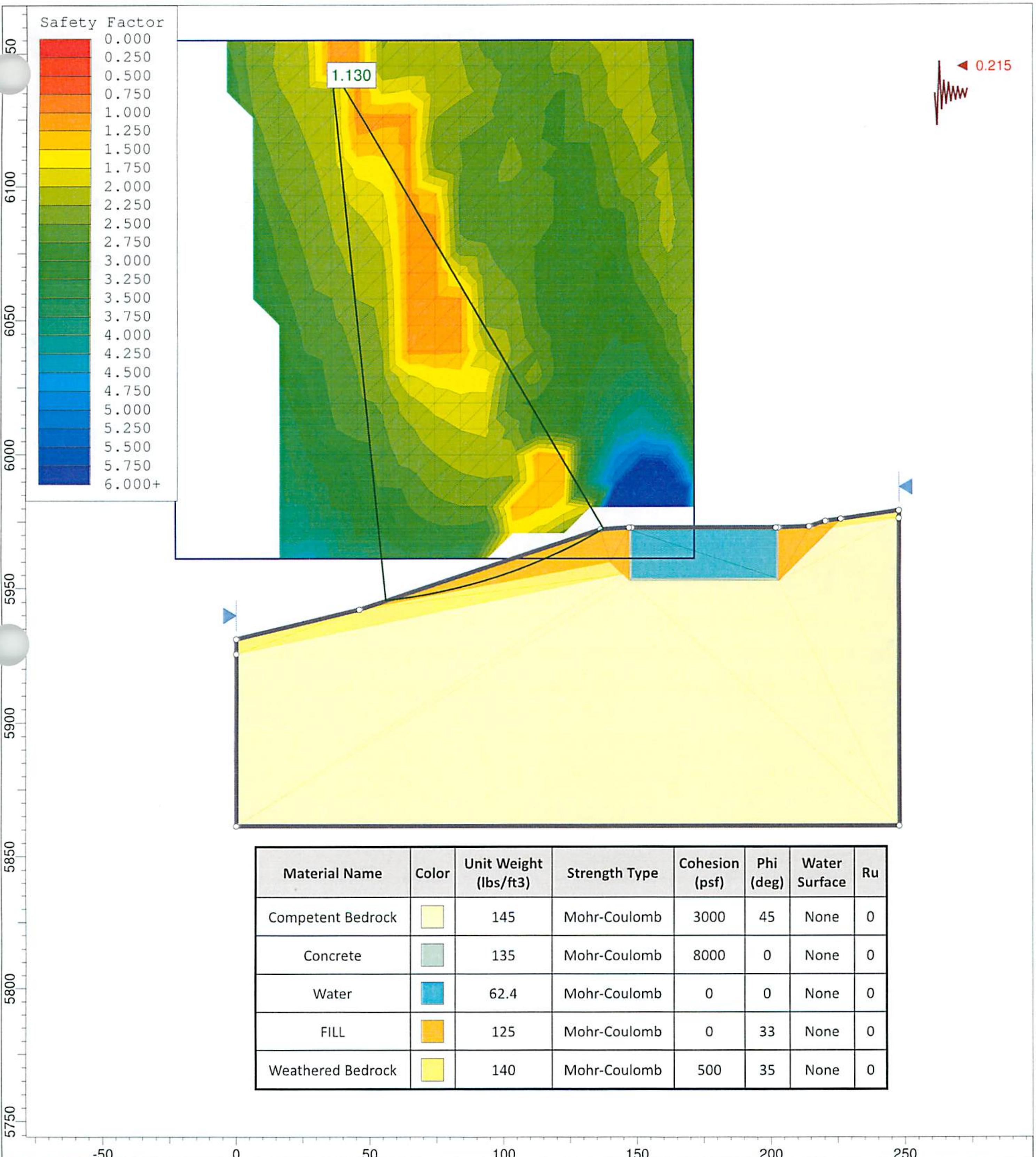
Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Ru
Competent Bedrock		145	Mohr-Coulomb	3000	45	None	0
Concrete		135	Mohr-Coulomb	8000	0	None	0
Water		62.4	Mohr-Coulomb	0	0	None	0
FILL		125	Mohr-Coulomb	0	33	None	0
Weathered Bedrock		140	Mohr-Coulomb	500	35	None	0

GeoStrata

Global Stability - Profile B Static

Gardner Engineering
LPT - Upper System Tank
Liberty, UT
Project Number: 746-001

Plate C-5



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Ru
Competent Bedrock		145	Mohr-Coulomb	3000	45	None	0
Concrete		135	Mohr-Coulomb	8000	0	None	0
Water		62.4	Mohr-Coulomb	0	0	None	0
FILL		125	Mohr-Coulomb	0	33	None	0
Weathered Bedrock		140	Mohr-Coulomb	500	35	None	0

GeoStrata

Global Stability - Profile B Pseudo Static

Gardner Engineering
LPT - Upper System Tank
Liberty, UT
Project Number: 746-001

Plate C-6