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Geotechnical Investigation
Reservoir No. 3 Rebuild
Approximately 2550 East Jaqueline Drive
Weber County, UT

GeoStrata Job No. 1065-001

March 6, 2015

Prepared for:

Uinta Highlands Improvement District
c/o Jones & Associates
1716 East 5600 South
South Ogden, Utah 84403

Attn: Mr. Matt Hartvigsen, P.E.

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March 6, 2015

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1.0 EXECUTIVE SUMMARY

This report presents the results of a Geotechnical Investigation conducted for the proposed 200,000-gallon concrete water tank to be constructed in at approximately 2550 East Jacqueline Drive in Weber County, 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, general soil characteristics, settlement analysis and liquefaction.

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 8 to 8½ feet below the existing site grade. Based on our observations, the western portion of the site is overlain by 4½ feet of locally derived fill consisting of Silty GRAVEL with sand and cobbles. Fill soils may be locally deeper depending on the original terrain. Underlying the fill soils we encountered approximately 1½ feet of topsoil. Native soil encountered below the topsoil and in the test pit in the eastern portion of the site consisted of Silty to Clayey GRAVEL with sand and cobbles (GM to GC).

All temporary excavations may be sloped at a maximum 1.5 horizontal to 1 vertical (1.5H:1V) slope. Permanent slopes may be designed using a 2 horizontal to 1 vertical slope to maintain slope stability.

Conventional ring-wall or mat footings may be used to support the proposed structure. Heavy excavation equipment may be necessary to remove oversized material. Foundations may be established upon undisturbed native soils using a net bearing capacity of **4,000 psf.** For matt foundations we recommend an ultimate modulus of subgrade reaction of **1,300 pci**. 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.

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 200,000-gallon concrete water tank to be constructed in at approximately 2550 East Jacqueline Drive in Weber County, 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, general soil characteristics, settlement analysis and liquefaction.

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 February 4, 2015 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

Based on conversations with our client and site plans provided by our client, we understanding that the proposed project is to consist of razing the existing water reservoir at the site and constructing a new concrete reservoir. The new reservoir is to have a 200,000 gallon capacity and be on the order of 50 feet in diameter. The bottom 6 feet of the tank is to be buried. A small chlorination building is also planned at the site. The new reservoir and chlorination building are to be located on the existing building pad requiring no significant modification to the existing slopes.

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 8 and 8½ feet below the existing site grade. The approximate locations of the explorations are shown on Plate A-2, (Exploration Location Map) in Appendix A. Due to accessibility issues, the exploration points were located only on the north side of the existing reservoir. 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 to 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 trackhoe. Soil sampling typically occurred at changes in the soil characteristics. 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 LABORATORY INVESTIGATION

Geotechnical laboratory tests were conducted on selected relatively undisturbed and bulk 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 (ASTM D4318)
- Moisture Content (ASTM D2216)

The results of laboratory tests are presented on the test pit logs in Appendix B (Plates B-1 to B-2) and on the test result plates presented in Appendix C (Plates C-1 to C-3).

3.3 ENGINEERING ANALYSIS

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

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

4.0 GENERALIZED SITE CONDITIONS

4.1 SURFACE CONDITIONS

The subject property is located on the south side of a drainage in the foothills above the Uintah Highlands area of Weber County, Utah. An existing retangualar shaped concrete water reservoir is currently located at the site on a small nearly level pad which appeared to have been created for the existing water reservoir. A cut slope had been excavated into the foothill slope above the existing reservoir. This cut slope was up to 15 feet high with an approximate 1H to 1V grade. The ground surface sloped down below the existing reservoir at grades of 1H to 1V to 3H to 1V. The site is located at an elevation of 5670 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 within the subject site. The test pits extended to depths of 8 and 8½ 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 and B-2). The subsurface conditions encountered during our investigation are discussed below.

4.2.1 *Soils*

Based on our observations the western portion of the site is overlain by 4½ feet of locally derived fill consisting of Silty GRAVEL with sand and cobbles. Fill soils may be locally deeper depending on the original terrain. Underlying the fill soils we encountered approximately 1½ feet of topsoil. Native soil encountered below the topsoil and in the test pit in the eastern portion of the site consisted of Silty to Clayey GRAVEL with sand and cobbles (GM to GC). The soils were generally brown and slightly moist.

The stratification lines shown on the enclosed test pit logs represent the approximate boundary between soil types. 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 explorations completed for this investigation, and is not expected to impact the development. Due to the season of our investigation (At the beginning of winter), we anticipate groundwater levels to be near their seasonal low. 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; however, we do not anticipate groundwater affecting the project as planned.

5.0 GEOLOGIC CONDITIONS

5.1 GEOLOGIC SETTING

The subject site is located at an elevation of approximately 5,340 feet in the foothills of the Wasatch Mountain Range adjacent to the northern portion of the Salt Lake Basin. The Wasatch Mountains mark the eastern edge of the Basin and Range province, and the western edge of the Middle Rocky Mountains province, and the range stretches from northern Utah to central Utah. Uplift of the Wasatch Mountains began approximately 10 to 15 million years ago as a result of extensional forces influencing the western U.S. The Wasatch Mountains were heavily glaciated during the Pleistocene, and several canyons contain glacial deposits and erosional features (Bugden, undated).

The near-surface geology of the foothills of the Wasatch Foothills is dominated by sediments, which were deposited within the last 30,000 years by Lake Bonneville (Scott and others, 1983; Hintze, 1993). As the lake receded, streams began to incise large deltas that had formed at the mouths of major canyons along the Wasatch Range, and the eroded material was deposited in shallow lakes and marshes in the basin and in a series of recessional deltas and alluvial fans. Sediments toward the center of the valley are predominately deep-water deposits of clay, silt and fine sand. These deep-water deposits are in places covered by a thin post-Bonneville alluvial cover. The sediments near the mountain front are predominately alluvial fan, colluviual and deltaic deposits (Yonke and Lowe, 2004). Near surface sediment at the site are mapped as consisting of pre-Bonneville to Bonneville transgressive landside deposits consisting of unsorted, unstratified deposits of angular boulders, sand, silt, clay, and bedrock blocks. These landslide units were deposited by multiple slides, slumps, and flows; part of these slides are covered by Lake Bonneville deposits and reworked along the Bonneville Shoreline, and parts of some slides are interlayered with Bonneville-transgressive lacustrine deposits.

5.2 SEISMICITY AND FAULTING

The site lies within the north-south trending belt of seismicity known as the Intermountain Seismic Belt (ISB) (Hecker, 1993). The ISB extends from northwestern Montana through southwestern Utah. An active fault is defined as a fault that has had activity within the Holocene (<11ka). No active faults are mapped through or immediately adjacent to the site (Yonke and Lowe, 2004). The site is located approximately 1200 feet east of the nearest mapped portion of the Weber segment of the Wasatch Fault zone. The Weber segment of the Wasatch fault is thought to have most recently experienced a seismic event during the Quaternary Period, and there is evidence that as many as 10 to 15 events 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 measureable 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 southern terminus of the Weber Segment occurs at the Salt Lake Salient, a ridge of Paleozoic and Tertiary bedrock that extends west of the Wasatch Front at the northern end of the Salt Lake rupture segment. The geometry of linkage between the main rupture zones in the Weber segment and faults in the interior of the Salt Lake salient is not clear. Surface scarps at the southern margin of the salient are discontinuous but apparently extend into the large normal fault along the eastern boundary of the segment. There is no reported evidence for Quaternary movement on this fault in the interior of the salient, so presumably the Quaternary ruptures have not reactivated most of this fault. The Pleasant View Salient marks the boundary between the Weber Segment and the Brigham City Segment to the north (Personius, 1986, Zoback, 1983). Analyses of ground shaking hazard along the Wasatch Front suggests that the Wasatch Fault Zone is the single greatest contributor to the seismic hazard in the Wasatch Front region. Each of the faults listed above show evidence of Holocene-aged movement, and is therefore considered active.

Seismic hazard maps depicting probabilistic ground motions and spectral response have been developed for the United States by the U.S. Geological Survey as part of NEHRP/NSHMP (Frankel et al, 1996). These maps have been incorporated into both *NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures* (FEMA, 1997) and the *International Building Code* (IBC) (International Code Council, 2012). Spectral responses for the Maximum Considered Earthquake (MCE) 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 C. The spectral accelerations are shown in the table below. The spectral accelerations are calculated based on the site’s approximate latitude and longitude of 41.1634˚ and -111.9157˚ respectively. Based on IBC, the site coefficients are F_a=1.00 and F_v= 1.34. From this procedure the peak ground acceleration (PGA) is estimated to be 0.50g. The MCE PGA and design response spectrum are presented in Appendix D on Plate D-1.

MCE Seismic Response Spectrum Spectral Acceleration Values for IBC Site Class C[a]

Site Location: **Latitude = 41.1634 N** **Longitude = -111.9157 W**	**Site Class C Site Coefficients:** **Fa = 1.00** **Fv = 1.34**
Spectral Period (sec)	**Response Spectrum Spectral Acceleration (g)**
0.2	**S_{MS}**=($F_{a*}S_s$=1.00*1.25)= 1.25
1.0	**S_{M1}**=($F_{v*}S_1$=1.34*0.47) = 0.63

[a] IBC 1615.1.3 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 and tested 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 reservoir 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 approximately 6 feet. Any underground obstructions, footings from the existing reservoir or existing utilities under the proposed new reservoir 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, or pavement sections), any existing vegetation, debris, or otherwise unsuitable soils should be removed. Any soft, loose, 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 is approximately 4½ feet of undocumented fill and topsoil on the western portion of the site. This fill and topsoil should be removed prior to placement of structural fill, structures, concrete flatwork and roadways.

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 Excavations*

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 our analyses. If groundwater or high moisture conditions are encountered during construction, GeoStrata should be notified to reassess temporary slope recommendations.

The temporary slope may be constructed at a 1.5 horizontal to 1 vertical slope (1.5H:1V). If saturated conditions are encountered, or if adverse bedding, jointing, or fractures are identified during the excavation, slopes will likely require flattening or shoring to maintain stability.

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 may need to be placed on the slope to protect workers from raveling of near surface soils and potentially 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 *Permanent Cut and Fill Slopes*

Existing slopes should not be over steepened by cutting or filling. We recommend that all non-retained cut and fill slopes be graded no steeper than a 2 to 1 (horizontal to vertical) grade. If steeper grades are required retaining structures should be used. We would be happy to provide retaining walls recommendations if they are desired.

6.2.4 *Rippability and Oversize Material*

It is possible that competent bedrock may be present at depths greater than our investigations. In addition, it is possible that large boulders not detected by our limited subsurface investigation may be present. It is possible that these earth materials may require large ripping equipment, a hoe ram and/or blasting.

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 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 GeoStrata. 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 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 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 on native granular soils or bedrock may be used to support the proposed tank. The proposed well house may be founded on spread footings bearing on undisturbed native granular soils. Conventional spread and strip footings may be proportioned for a maximum net allowable bearing capacity of **4,000 pounds per square foot (psf)**. For matt foundations we recommend an ultimate modulus of subgrade reaction of **1,300 pci**.

Strip footings for the water tank should be a minimum of 20 inches wide and exterior shallow footings should be embedded at least 30 inches below final grade for frost protection and confinement purposes. Isolated interior footings should be a minimum of 24 inches wide and also be embedded a minimum of 18 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. Strip footings for the chlorination building should be a minimum of 20 inches wide.

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 0.49 should be used for structural fill, drain gravel, or coarse-grained native soils against concrete.

Ultimate lateral earth pressures from *granular* backfill acting against buried walls and 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*	0.26	31
At-rest**	0.41	49
Passive*	3.85	462
Seismic Active***	0.43	52
Seismic Passive***	-0.84	-101

* 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.

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.

6.6 SOIL CORROSION

Based on our experience within the area of the project site as well as with similar soils, the near-surface soils are expected to exhibit a negligible potential for sulfate attack when in contact with concrete elements. We further anticipate that conventional Type I/II cement can be used for all concrete associated with the project.

6.7-SLOPE STABILITY

As indicated in Section 5.1, the reservoir site is underlain by pre-Bonneville to Bonneville transgressive landside deposits. No evidence of post-Bonneville movement of this landslide deposit was observed along the contact with Bonneville-transgressive lacustrine deposits observed to overlie the toe of the landslide deposit. This would mean the landslide has not experienced recurrent movement in at least 15,000 years while experiencing several M.7 earthquakes in that time. It is likely that the risk associated with this landside is low due to its age; however, a geologic assessment of the landslide has not been performed by GeoStrata. Due to the size and terrain, assessment of this landside would be very difficult and costly and was outside the scope of this investigation. If desired, GeoStrata would be happy to provide a proposal for assessment of the landside.

The global stability of the existing slopes in the vicinity of the landslide above and below the proposed reservoir 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. Homogeneous earth materials and arcuate failure surfaces were assumed. The profiles used in our analyses were based on topographic maps of the site provided to us by our client. Slope stability was performed for the static and pseudo static conditions. The pseudo static assessment used one half of the peak ground acceleration presented in Section 5.2 of this report.

Due to the significant gravel and cobble content of the native soil, laboratory strength testing was not practical. Strength parameters used in our analyses consisted of an angle of internal friction of 40 degrees and an apparent cohesion of 50 psf. This strength was based on the soil conditions observed in our test pits, experience, and engineering judgment. The results of our stability analyses as described above produce factors of safety as listed in the following Table:

Stability Analysis Factors of Safety

Analysis	**Minimum Factor of Safety**
Upslope - Static	2.2
Upslope - Pseudo Static	1.4
Downslope - Static	1.5
Downslope - Pseudo Static	0.9

Slopes with factors of safety of 1.5 and 1.0 for static and pseudo-static conditions, respectively, are considered stable. As indicated in the table above, the pseudo static factor of safety for the slope below the reservoir is less than 1.0. Due to the low factor of safety, a deformation analysis was performed using the method outlined by Bray and Travasarou. Using this method we estimate 7 inches of vertical movement of the slope below the reservoir during a large seismic event. We recommend that foundations for the reservoir consider this potential movement. The results of our stability analyses and deformation assessment are presented on Plates E-1 through E-5.

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 CMU building 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 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

1:26,000

N

GeoStrata

Uintah Highlands Improvement District
Reservoir No. 3 Rebuild
Uintah Highlands, UT
Project Number: 1065-001

Site Vicinity Map

Plate A-1

Base map:
Reservoir #3 Rebuild
SITE PLAN
Jones & Associates, Undated

1:280

GeoStrata

Legend

🟧 Approximate Test Pit Location

▭ Approximate Site Boundary

Uintah Highlands Improvement District
Reservoir No. 3 Rebuild
Uintah Highlands, UT
Project Number: 1065-001

Exploration Location Map

Plate
A-2

DATE	
STARTED:	2/12/15
COMPLETED:	2/12/15
BACKFILLED:	2/12/15

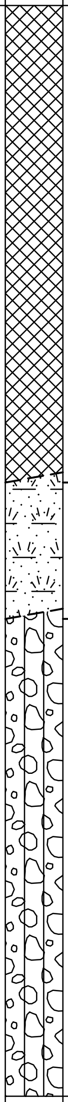
Uintah Highlands Improvement District
Reservoir No. 3 Rebuild
Uintah Highlands, UT
Project Number 1065-001

GeoStrata Rep: M. Christensen
Rig Type: Trackhoe

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					FILL; Silty GRAVEL with sand and cobbles - moist, brown, cobbles observd up to 12" in diameter						
1												
						TOPSOIL; Silty GRAVEL with sand - moist, dark brown, with roots ,weakly developed						
					GM	Silty GRAVEL with sand and cobbles - dense to very dense, moist, brown						
	5											
2												
						- trackhoe refusal @ 8 ft						
						Bottom of Test Pit @ 8 Feet						
3												

GeoStrata

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

WATER LEVEL
▼- MEASURED
▽- ESTIMATED

NOTES:

Plate
B-1

DATE		
STARTED:	2/12/15	
COMPLETED:	2/12/15	
BACKFILLED:	2/12/15	

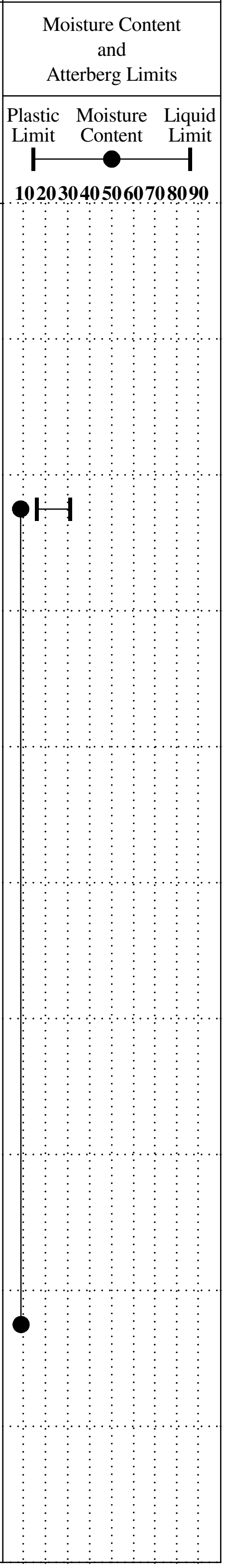
Uintah Highlands Improvement District
Reservoir No. 3 Rebuild
Uintah Highlands, UT
Project Number 1065-001

GeoStrata Rep: M. Christensen
Rig Type: Trackhoe

TEST PIT NO:
TP-2
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
0	0				GC	Clayey GRAVEL with sand and cobbles - dense, moist, brown, cobbles observed upt o 18" in diameter					
		▯						8.2	21.2	31	15
1											
						- less clay					
	5										
2											
						- trackhoe refusal @ 8.5 ft					
		▯						8.4	18.6		
						Bottom of Test Pit @ 8.5 Feet					
3											

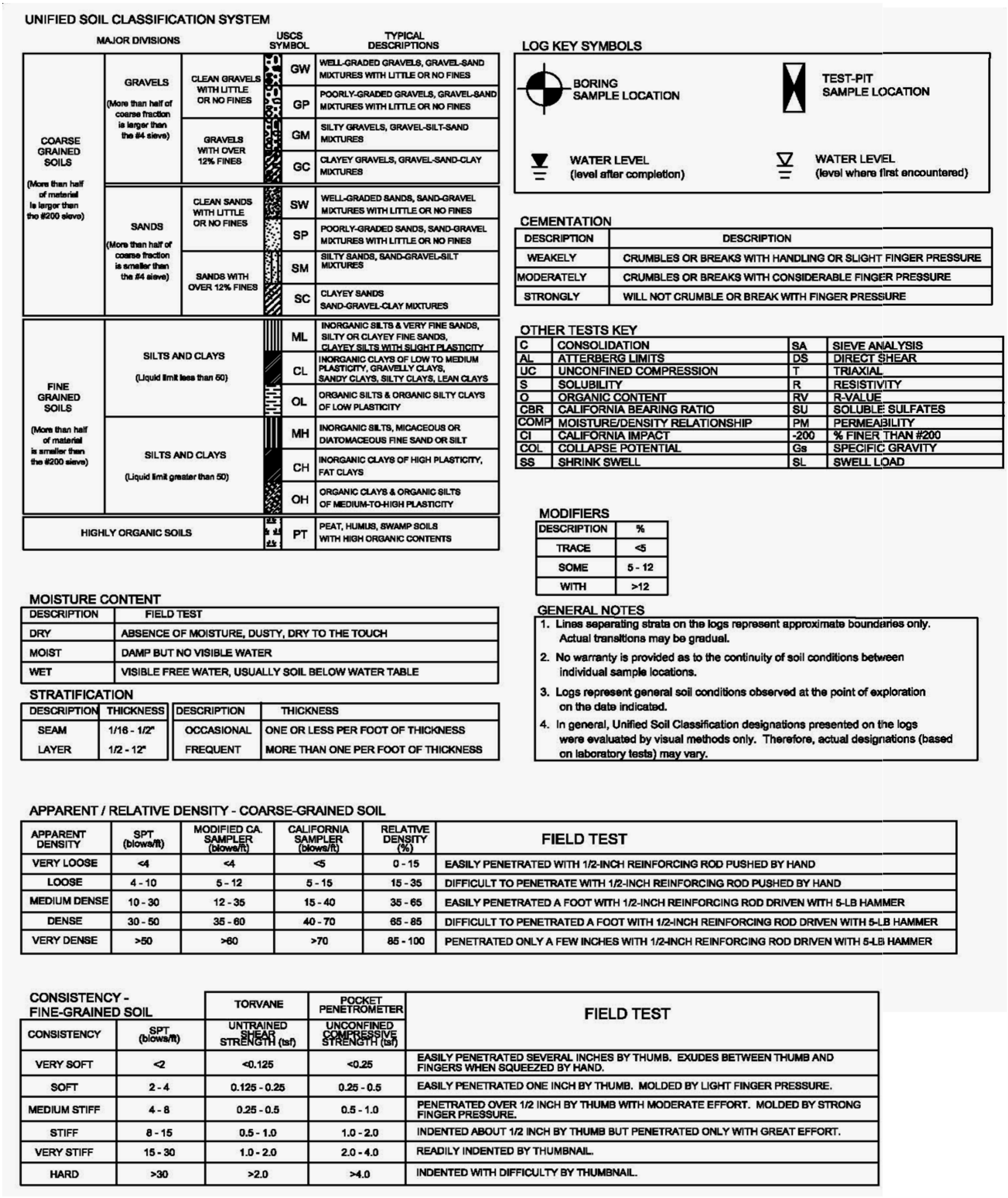


SAMPLE TYPE
▯ - GRAB SAMPLE
☒ - 3" O.D. THIN-WALLED HAND SAMPLER

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

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.

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

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.

GeoStrata

Soil Symbols Description Key

Uintah Highlands Improvement District
Reservoir No. 3 Rebuild
Uintah Highlands, UT
Project Number: 1065-001

Plate
B-3

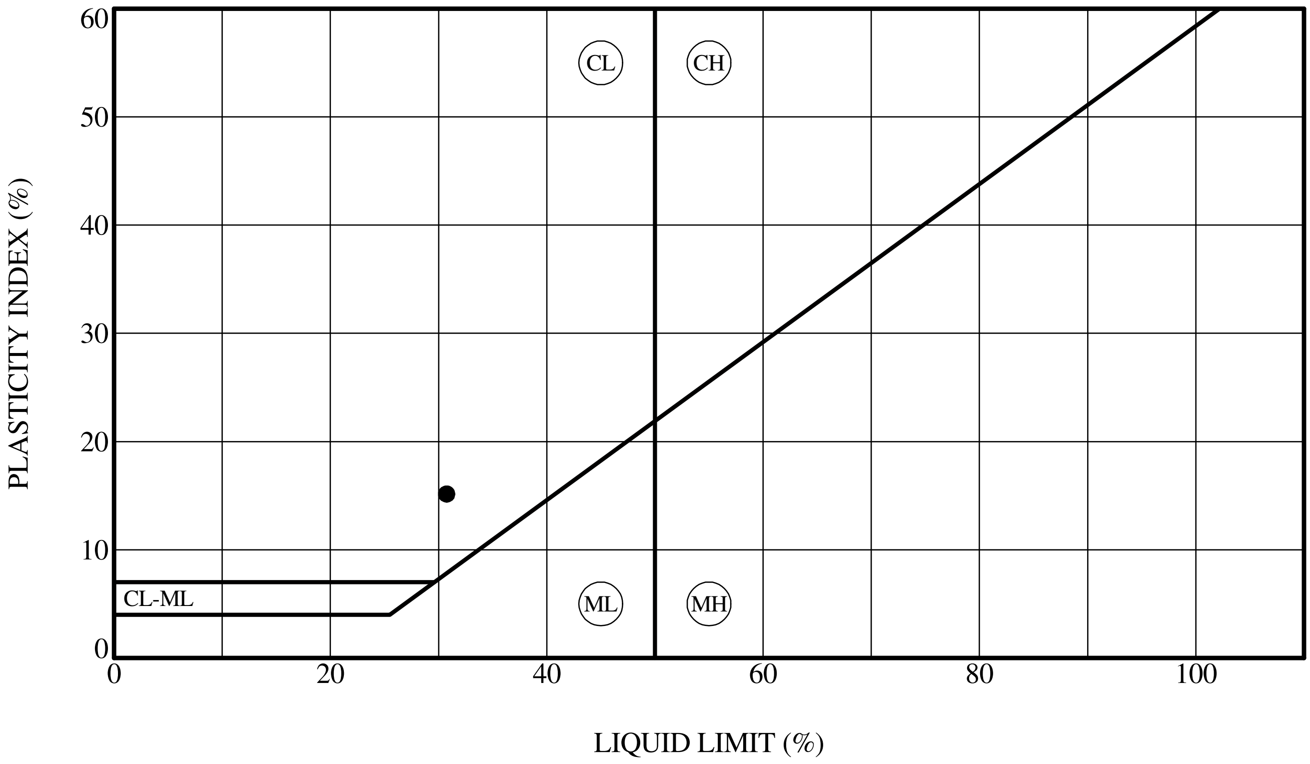
Test Pit No.	Sample Depth (feet)	USCS Soil Classification	Natural Moisture Content (%)	Gradation			Atterberg	
				Gravel (%)	Sand (%)	Fines (%)	LL	PI
TP-2	2	GC	8.2	52.6	26.2	21.2	31	15
TP-2	8	GC	8.4	41.2	40.2	18.6		

GeoStrata

Lab Summary Report

Uintah Highlands Improvement District
Reservoir No. 3 Rebuild
Uintah Highlands, UT
Project Number: 1065-001

Plate
C - 1



	Sample Location	Depth (ft)	LL (%)	PL (%)	PI (%)	Fines (%)	Classification
●	**TP-2**	**2.0**	**31**	**16**	**15**	**21.2**	**Clayey GRAVEL with sand**

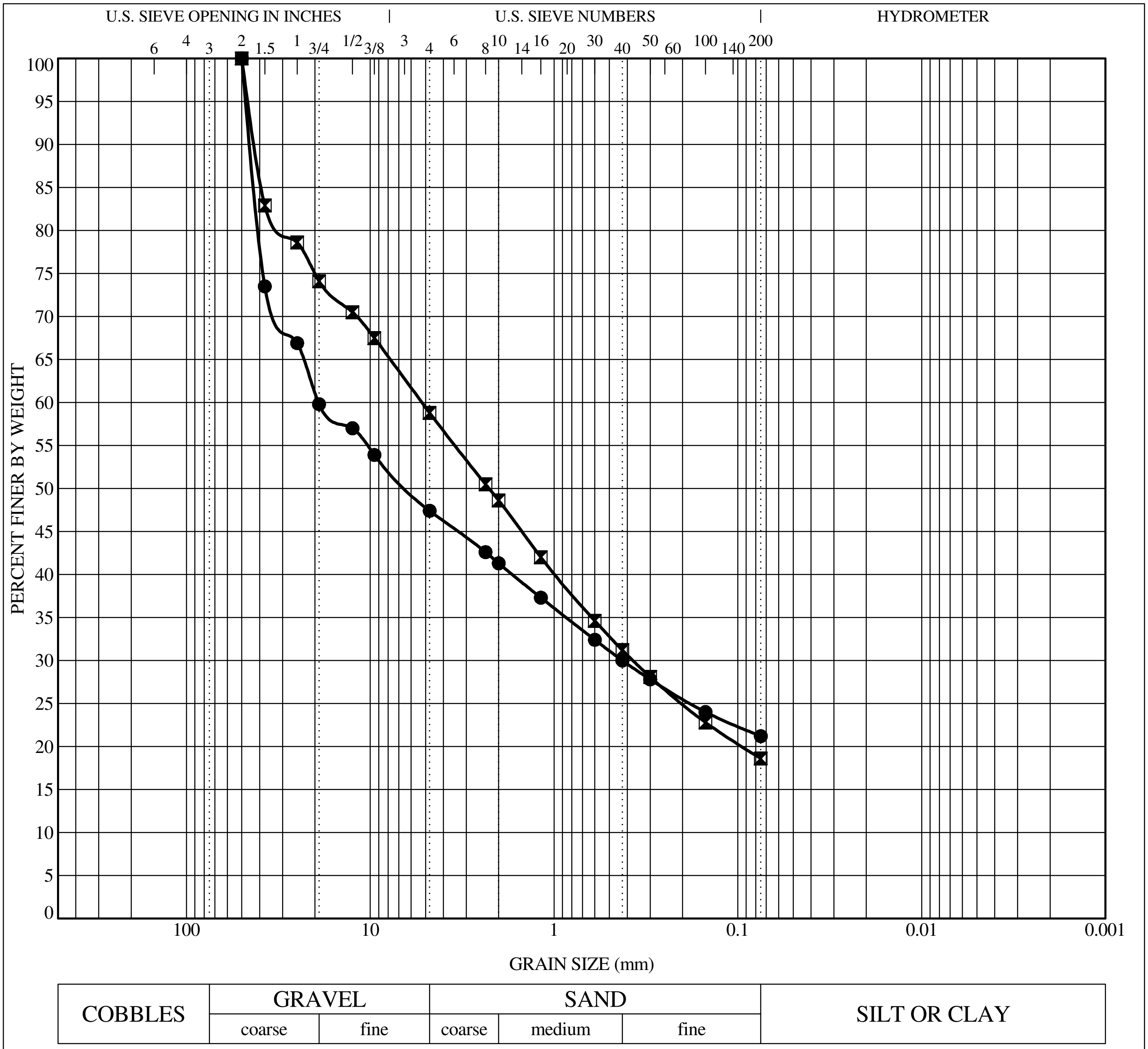
ATTERBERG LIMITS' RESULTS - ASTM D 4318

GeoStrata

Uintah Highlands Improvement District
Reservoir No. 3 Rebuild
Uintah Highlands, UT
Project Number: 1065-001

Plate

C - 2



	Sample Location	Depth	Classification	LL	PL	PI	Cc	Cu
●	**TP-2**	**2.0**	**Clayey GRAVEL with sand**	**31**	**16**	**15**		
☒	**TP-2**	**8.0**	**Clayey GRAVEL with sand**					

	Sample Loctaion	Depth	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**TP-2**	**2.0**	**50**	**19.147**	**0.425**		**52.6**	**26.2**	**21.2**	
☒	**TP-2**	**8.0**	**50**	**5.227**	**0.371**		**41.2**	**40.2**	**18.6**	

GeoStrata

GRAIN SIZE DISTRIBUTION - ASTM D422

Uintah Highlands Improvement District
Reservoir No. 3 Rebuild
Uintah Highlands, UT
Project Number: 1065-001

Plate

C - 3

C_GSD TEST PIT LOGS.GPJ GEOSTRATA.GDT 3/3/15

Seismic Ground Motion Values: USGS, 2009; Dobry and others, 2000

Project: Reservoir No. 3
Geotechnical Investigation
Project No.: 1065-001
Project Location: Uintah Highlands
Date: Wednesday, February 25, 2015
Engineer: MIC

Site Coordinates:
Latitude: **41.1634** degrees
Longitude: **-111.9157** degrees

Exceedance Probability: **2** %
Exposure Time: **50** years
S_s = **1.251** From USGS 2002 Probabilistic Seismic
S_1 = **0.469** Hazard Maps for 2475-year Return Period

Site Soil Class: **C** (Very dense soil and soft rock)
F_a = 1.00
F_v = 1.34

Site Class	Values of Site Factor, F_a, for Short-Period Range of Spectral Acceleration				
	$S_S \le 0.25$	$S_S = 0.5$	$S_S = 0.75$	$S_S = 1.0$	$S_S \ge 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	*	*	*	*	*

(*)Site-specific geotechnical investigation and dynamic site response analyses shall be performed

Site Class	Values of Site Factor, F_v, for Long-Period Range of Spectral Acceleration				
	$S_1 \le 0.1$	$S_1 = 0.2$	$S_1 = 0.3$	$S_1 = 0.4$	$S_1 \ge 0.5$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	2.4
F	*	*	*	*	*

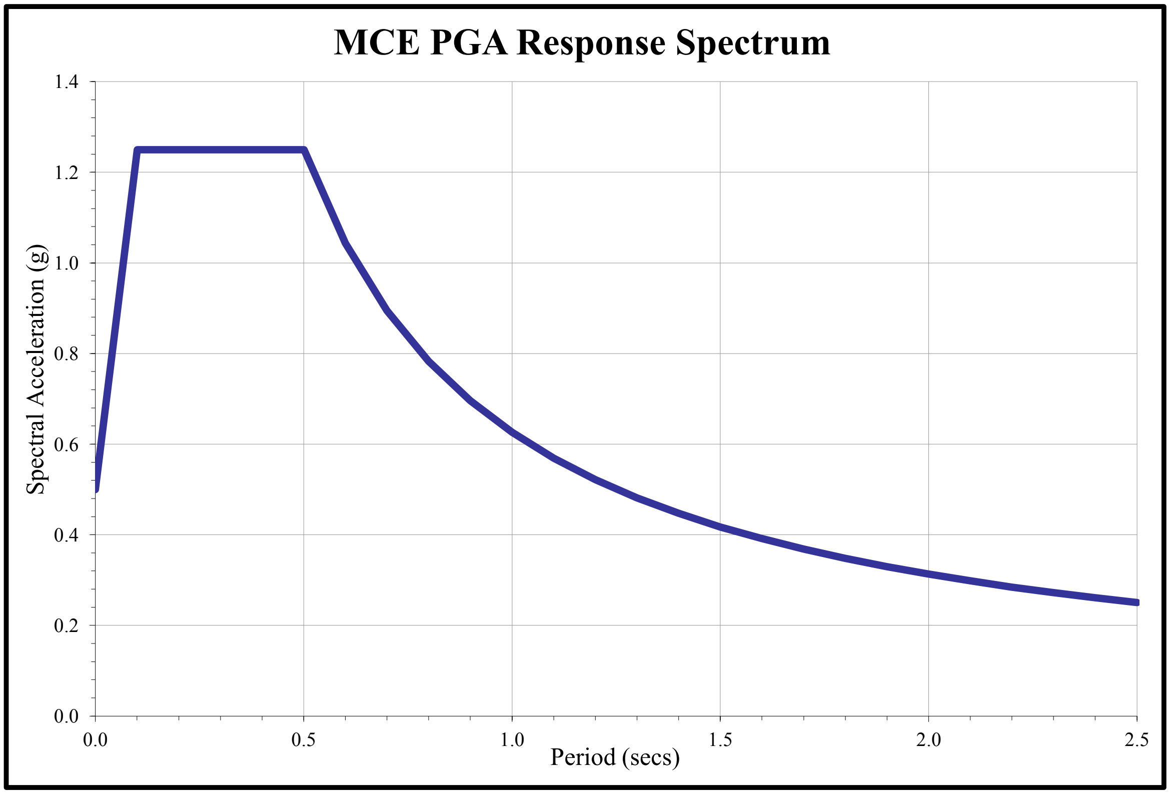
(*)Site-specific geotechnical investigation and dynamic site response analyses shall be performed

Adjusted for Site Conditions:
$S_{MS} = F_a \times S_S$ = (1.00 x 1.25) = 1.25 g
$S_{M1} = F_v \times S_1$ = (1.34 x 0.47) = 0.63 g

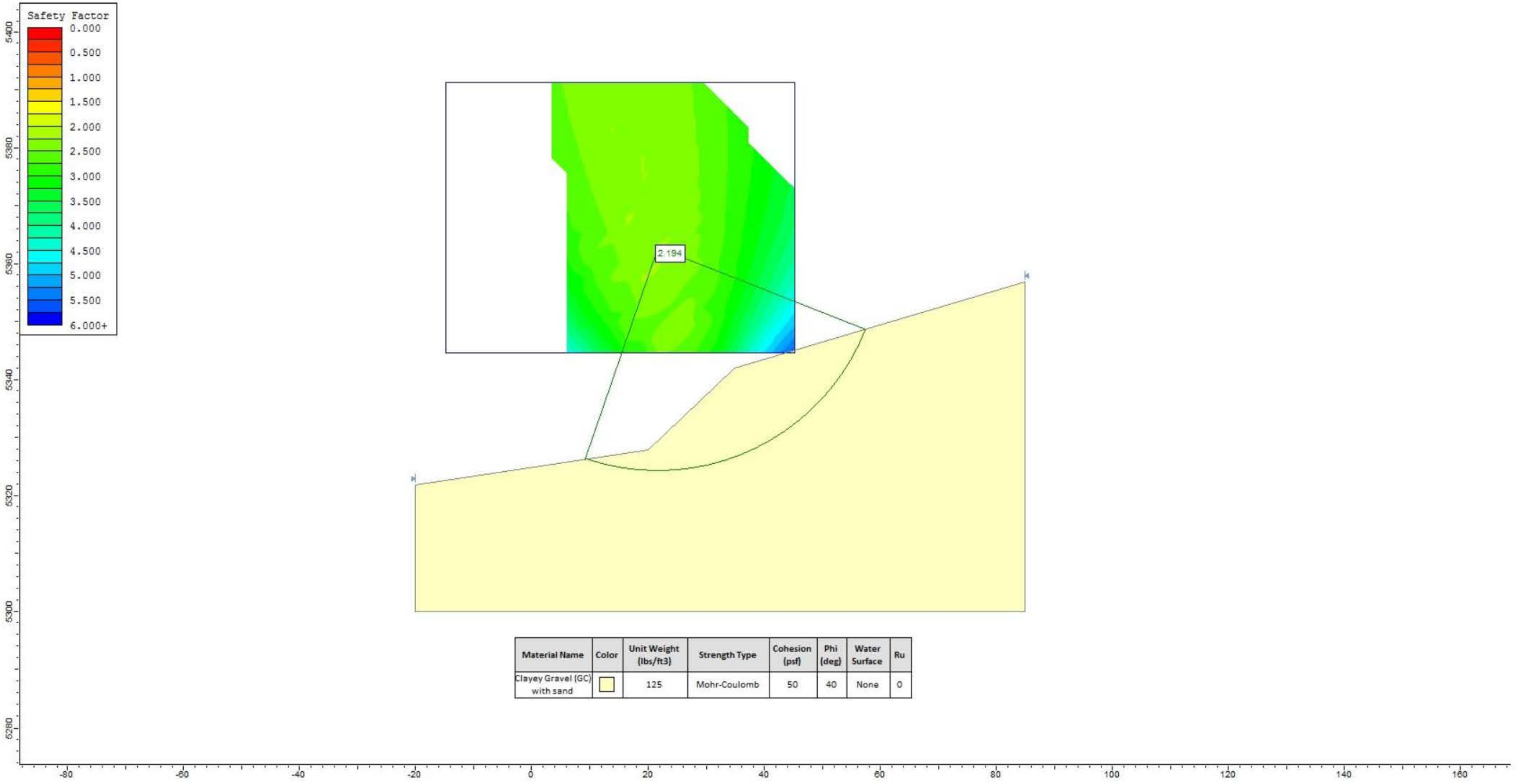
MCE PGA = 0.4 x S_{MS} = (0.4 x 1.25) = **0.50 g**
MCE $T_0 = 0.2 \times (S_{M1}/S_{MS})$ = (0.2 x [0.63/1.25]) = 0.10 secs
MCE $T_S = (S_{M1}/S_{MS})$ = (0.63/1.25) = 0.50 secs

Response Time Step, ΔT = **0.1**

Period (sec)	MCE Spectral Acceleration (g)
0.00	0.50
0.10	1.25
0.50	1.25
0.60	1.04
0.70	0.89
0.80	0.78
0.90	0.70
1.00	0.63
1.10	0.57
1.20	0.52
1.30	0.48
1.40	0.45
1.50	0.42
1.60	0.39
1.70	0.37
1.80	0.35
1.90	0.33
2.00	0.31
2.10	0.30
2.20	0.28
2.30	0.27
2.40	0.26
2.50	0.25



D-1

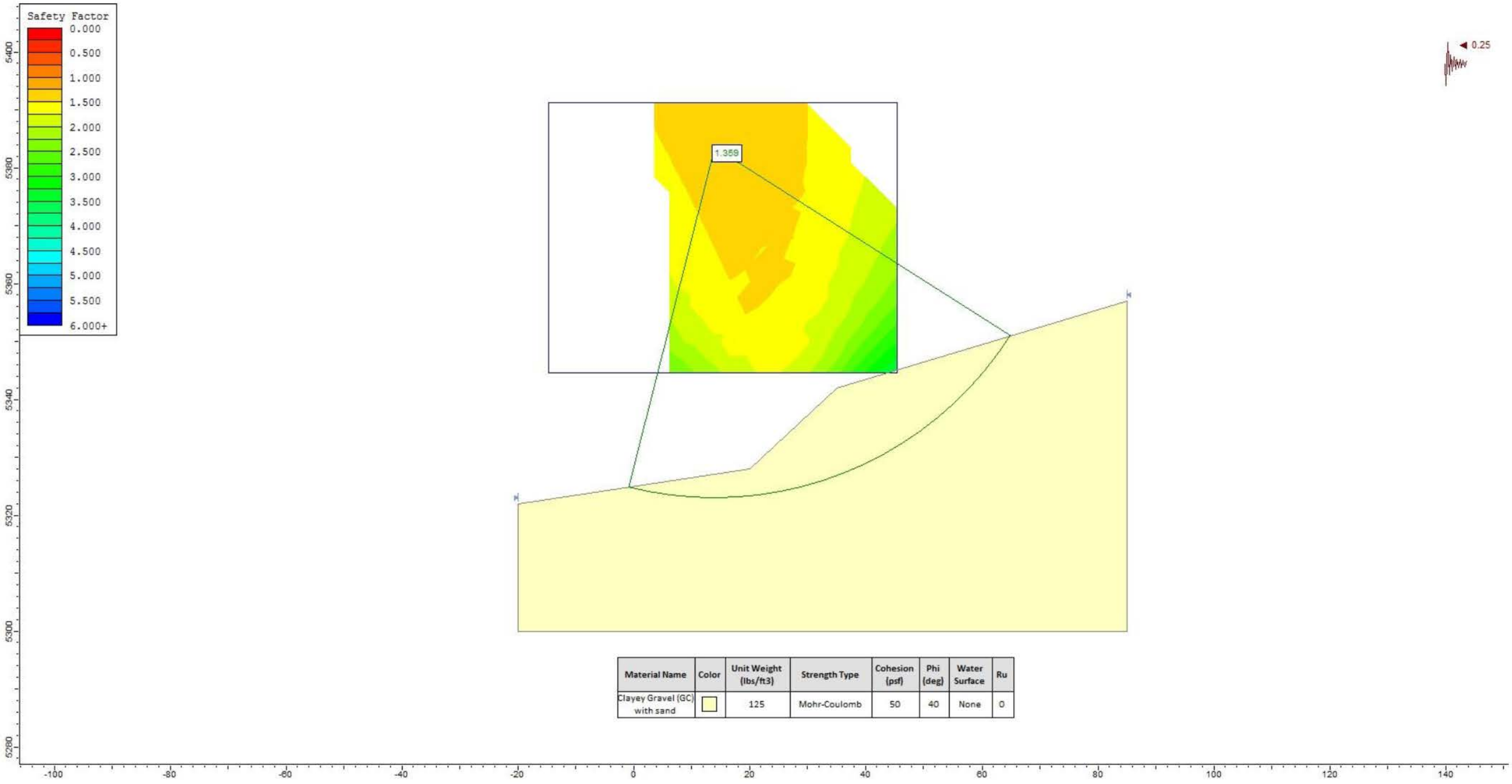


Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Ru
Clayey Gravel (GC) with sand	🟨	125	Mohr-Coulomb	50	40	None	0

Upslope – Static

Uintah Highlands Improvement District
Reservoir No. 3 Rebuild
Uintah Highlands, UT
Project Number: 1065-001

Plate
E - 1



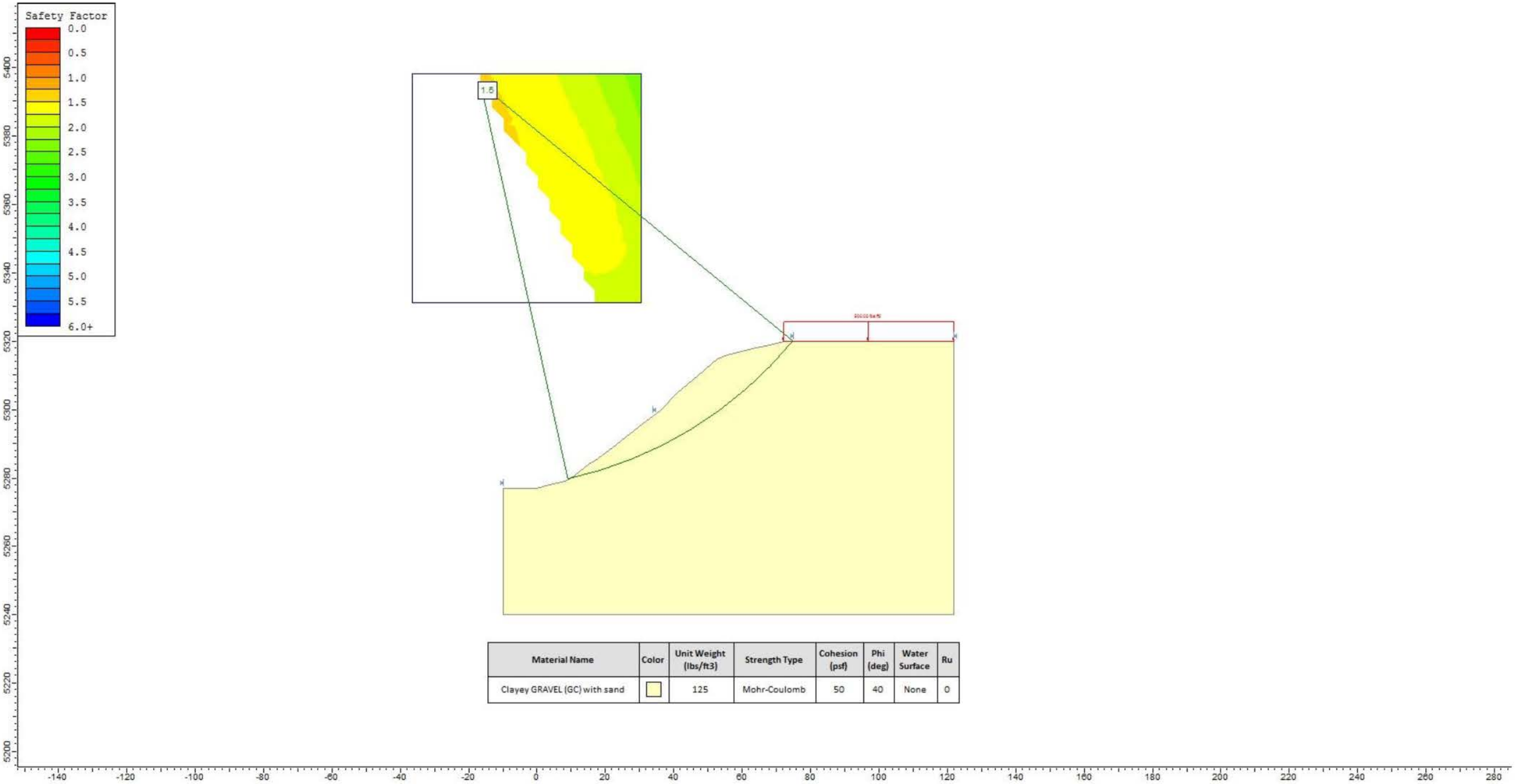
Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Ru
Clayey Gravel (GC) with sand	🟨	125	Mohr-Coulomb	50	40	None	0

GeoStrata

Upslope – Pseudo Static

Uintah Highlands Improvement District
Reservoir No. 3 Rebuild
Uintah Highlands, UT
Project Number: 1065-001

Plate
E - 2

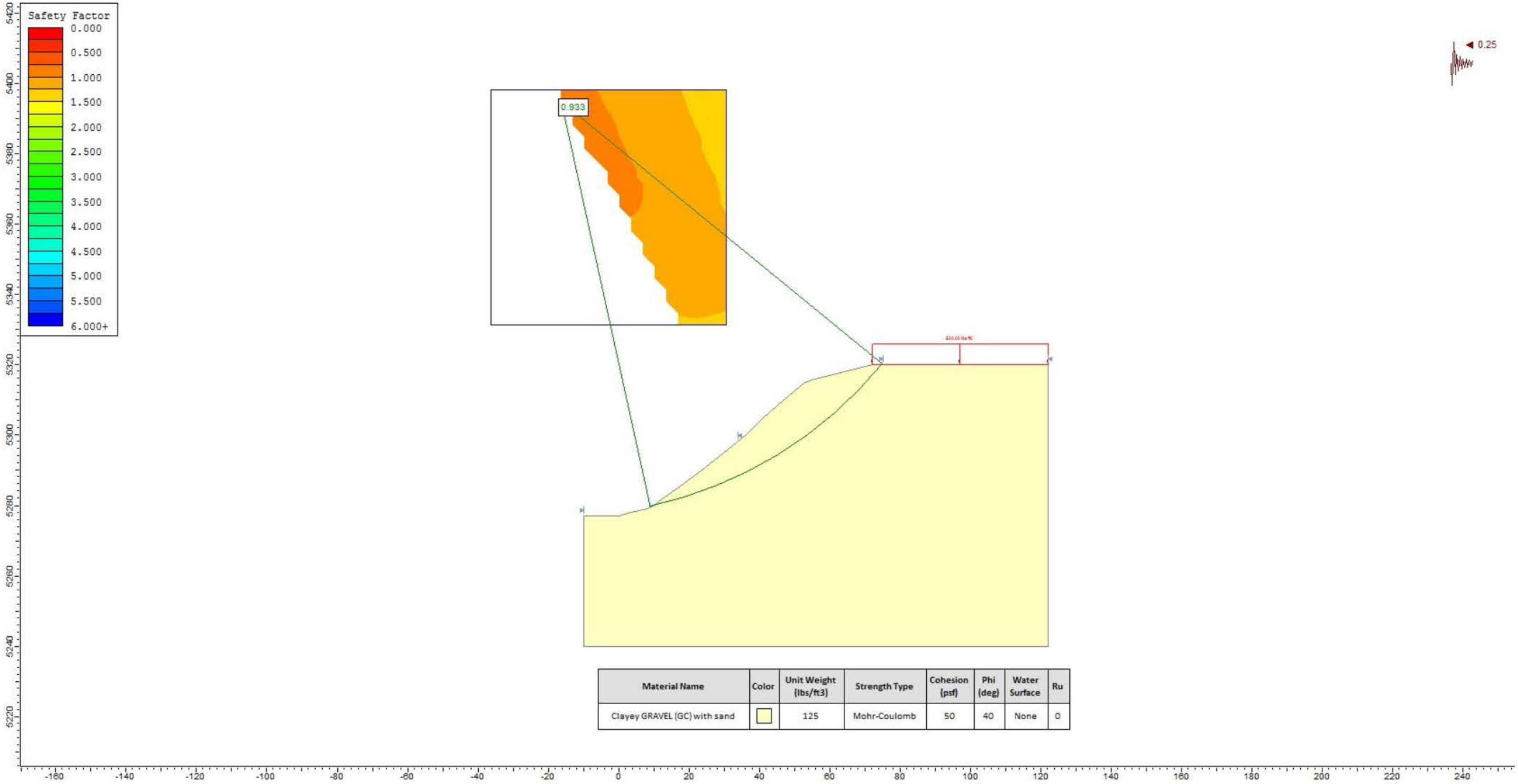


Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Ru
Clayey GRAVEL (GC) with sand	🟨	125	Mohr-Coulomb	50	40	None	0

Downslope - Static

Uintah Highlands Improvement District
Reservoir No. 3 Rebuild
Uintah Highlands, UT
Project Number: 1065-001

Plate
E - 3



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Ru
Clayey GRAVEL (GC) with sand	🟨	125	Mohr-Coulomb	50	40	None	0

GeoStrata

Downslope – Pseudo Static

Uintah Highlands Improvement District
Reservoir No. 3 Rebuild
Uintah Highlands, UT
Project Number: 1065-001

Plate
E - 4

ESTIMATION OF PERMANENT SEISMIC DISPLACMENT USING THE BRAY AND TRAVASAROU (2007) PROCEDURE

INPUT PARAMETERS:	Ky	0.21

Yeild Acceleration, Ky (g):	0.21
Vertical Thickness, h (m):	4.6
Shear Wave Vel., Vs (m/s):	500
Earthquake maginitude, M:	7.3
Earthquake Acc., (g):	0.62
Sa (Ts), (g):	1.02
Sa(1.5Ts), (g):	1.1
Threshhold, d (cm):	15

RESULTS:

Estimated Displacement, D (cm):	17.2
Estimated Displacement, D (in):	6.8
Probability of Zero Displacement (%):	0.01
Probability of threshold Exceedance (%):	58.4