GeoStrata

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Geotechnical Investigation
Commercial Property
Approximately 12600 South 2300 West
Riverton, UT

GeoStrata Job No. 1012-011

June 15, 2017

Prepared for: Princeton office Building

Keystone Construction
8679 South Sandy Parkway, Suite A
Sandy, UT 84070

Attn: Mr. Grant Lefgren

OK

RC-BKM

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APPENDICES

1.0 EXECUTIVE SUMMARY

This report presents the results of a geotechnical investigation conducted for the proposed commercial structure to be constructed at approximately 12600 South 2300 West in Riverton, Utah. The purposes of this investigation were to assess the nature and engineering properties of the subsurface soils at the site and to provide recommendations for general site grading and the design and construction of foundations, slab-on-grades, and exterior concrete flatwork.

Based on the results of our analysis, it is our opinion that the site is suitable for the proposed development provided that the recommendations contained in this report are incorporated into the design and construction of the project.

As part of this investigation, subsurface soil conditions were explored by advancing four exploratory test pits at the site to depths of 11 to 13 feet below the site grade as it existed at the time of our investigation. Based on our observations, the subject site is overlain with 0 to 5 feet of undocumented fill soils or up to 1 foot of clayey topsoil. These units were in turn underlain by Pleistocene-aged silt and clay deposits which extended to the full depth of our investigations. Where observed, the lacustrine sediments consisted of stiff, brown, moist, Lean CLAY (CL) grading into a Silty SAND (SM) with depth. Groundwater was not encountered in any of the explorations advanced as part of this investigation.

The foundations for the proposed structures may consist of conventional strip and/or spread footings founded entirely on relatively undisturbed, native soils or upon a minimum of 12 inches of properly placed and compacted structural fill. It should be noted that the property was previously occupied by residential structures, and as such there is the potential for relatively thick undocumented fill sections to exist at the site. We recommend that a GeoStrata representative observe all foundation soils in footing excavations prior to placing reinforcing steel or concrete. Conventional strip/spread footings may be proportioned using a maximum net allowable bearing pressure of **1,600 pounds per square foot (psf)**. The following subsections present our recommendations for general site grading, design of foundations, slabs-on-grade, moisture protections and soil corrosivity.

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

2.0 INTRODUCTION

2.1 PURPOSE AND SCOPE OF WORK

This report presents the results of a geotechnical investigation conducted for the proposed commercial structure to be constructed at approximately 12600 South 2300 West in Riverton, Utah. The purposes of this investigation were to assess the nature and engineering properties of the subsurface soils at the site and to provide recommendations for general site grading and the design and construction of foundations, slab-on-grades, and exterior concrete flatwork.

The scope of work completed for this study included a site reconnaissance, subsurface exploration, soil sampling, laboratory testing, engineering analyses, and preparation of this report. Our services were performed in accordance with our proposal, dated May 15, 2017 and your signed authorization.

The recommendations contained in this report are subject to the limitations presented in the "Limitations" section of this report (Section 7.1).

2.2 PROJECT DESCRIPTION

The subject property is located at approximately 12600 South 2300 West in Riverton, Utah. (see Plate A-1, *Site Vicinity Map*). Our understanding of the proposed development is based on information provided by the client as well as on a drawing titled "concept – Site Plan" (undated), (preparing agency unknown). Based on this information, we understand that the structure will consist of a single story, wood or metal framed building with no basement, founded on conventional strip footings, and will have a footprint of approximately 4,000 square feet. The project will also include a paved parking area containing 19 paved parking stalls. The site has a total area of approximately ¾ of an acre. For the purposes of this report we have assumed structural loads on the order of 2 to 3 kips per lineal foot.

3.0 METHODS OF STUDY

3.1 FIELD INVESTIGATION

As part of this investigation, subsurface soil conditions were explored by advancing four exploratory test pits at the site to depths of 11 to 13 feet below the site grade as it existed at the time of our investigation. The approximate locations of the explorations are shown on the *Exploration Location Map*, Plate A-2 in Appendix A. Exploration points were selected to provide a representative cross section of the subsurface soil conditions in the anticipated vicinity of the proposed structures. Subsurface soil conditions as encountered in the explorations were logged at the time of our investigation by a qualified geotechnical engineer and are presented on the enclosed Test Pit Logs, Plates B-1 through B-4 in Appendix B. A *Key to USCS Soil Symbols and Terminology* is presented on Plate B-5.

The test pits were excavated using a backhoe. Disturbed and undisturbed samples were obtained from the test pits. Disturbed soil samples were obtained with the use of bags and buckets. Undisturbed samples were collected from blocks of soil taken from the test pit walls. All samples were transported to our laboratory for testing to evaluate engineering properties of the various earth materials observed. The soils were classified according to the *Unified Soil Classification System* (USCS). 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 soil samples obtained during our field investigation. The laboratory testing program was designed to evaluate the engineering characteristics of onsite earth materials. Laboratory tests conducted during this investigation include:

- Grain Size Distribution Analysis (ASTM D422)
- Atterberg Limits Test (ASTM D4318)
- 1-D Consolidation Test (ASTM D2435)
- Collapse Potential Test (ASTM D 5333)
- California Bering Ratio (CBR) Test (ASTM D1883)
- Sulfate Content
- Soil Electrical Resistivity and pH

4.0 GENERALIZED SITE CONDITIONS

4.1 SURFACE CONDITIONS

At the time of our subsurface investigation, the property existed as an undeveloped parcel. Vegetation at the site included native grasses and weeds. Historical aerial photography of the site shows that the subject property was previously divided into several residential properties with associated structures. Plate A-4 has been prepared to illustrate the locations of the previously existing structures. The site is relatively flat with a topographic relief of approximately 4 feet and is bound to the north by 12600 South, a paved, 4-lane roadway, on the west by 2360 West, a paved 2-lane roadway, on the east by 2295 West, a paved 2-lane roadway, and on the south by established residential development. Access to the site is gained through the use of 12600 South at 2360 West, or 2295 West.

4.2 SUBSURFACE CONDITIONS

As previously discussed, the subsurface soil conditions were explored at the site by excavating four test pits at representative locations within the subject property. The test pits extended to depths of 11 to 13 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 to B-4). The subsurface conditions encountered during our investigation are discussed below;

4.2.1 Soils

Based on our observations and geologic literature review, the subject site is overlain by 0 to 5 feet of undocumented fill soils. As mentioned in Section 4.1 of this report, the site was previously occupied by two residential structures, and it is suspected that the undocumented fill originated from the demolition of those structures. Underlying the undocumented fill (where encountered), we encountered up to one foot of clayey topsoil. Finally, underlying the fill/topoil we encountered Pleistocene-aged fine-grained lacustrine deposits associated with the regressive phase of the Bonneville lake cycle. Descriptions of the soil units encountered are provided below:

<u>Undocumented Fill:</u> Undocumented fill soils were encountered in test pits TP-1 and TP-3 where they were observed to extend to a depth of 5 and 2½ feet, respectively. Where observed, these

soils consisted of Poorly Graded GRAVEL (GP-GM) with silt and sand. In test pit TP-1, plastic and metal debris were observed at a depth of 5 feet. It should be noted that based on the fact that the site has previously been developed, the possibility exists that thicker sections of undocumented fill soils may be present at the site than was observed during our field investigation.

Topsoil: Topsoil was encountered in test pit TP-2, where it was observed to extend to a depth of 12 inches. Where observed, the topsoil generally consists of dark brown Lean CLAY (CL) with sand and gravel. This unit has an organic appearance and texture with roots throughout. Topsoil was encountered just one of the test pits excavated as part of this investigation, and is anticipated to exist in isolated areas across the site where previous development has not disturbed it.

Pleistocene-aged Lake Bonneville Silt and Clay Deposits: These deposits were encountered in each of the test pits excavated as part of this investigation. Where observed, these soils consisted of stiff, moist, brown, Lean CLAY (CL) with varying amounts of sand. These deposits graded into coarse-grained sediments at depths ranging from 10 to 11½ feet and included dense, moist, brown, Silty SAND (SM) with gravel. Each of the units contained significant iron staining throughout. These soils are mapped as having been deposited in deep and/or quiet water in the lower part of the basin, and typically grades laterally into other deposits of the Bonneville lake cycle.

The stratification lines shown on the enclosed test pit logs represent the approximate boundary between soil types (Plates B-1 to B-4). 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 our explorations completed as part of this investigation. Seasonal fluctuations in precipitation, surface runoff from adjacent properties, or other on or offsite sources may increase moisture conditions; groundwater conditions can be expected to rise several feet during wetter years and seasonally depending on the time of year. It is unlikely that groundwater will impact the proposed improvements.

4.2.3 Hydro-Collapsible Soils

Collapse (often referred to as "hydro-collapse") is a phenomena whereby undisturbed soils exhibit volumetric strain and consolidation upon wetting under increased loading conditions. Collapsible soils can cause differential settling of structures and roadways. Collapsible soils do not necessarily preclude development and can be mitigated by over-excavating porous, potentially collapsible soils and replacing with engineered fill and by controlling surface drainage and runoff. For some structures that are particularly sensitive to differential settlement, or in areas where collapsible soils are identified at great depth, a deep foundation system should be considered.

Soils that have a potential to collapse under increased loading and moisture conditions are typically characterized by a pinhole structure and relatively low unit weights. In general, potentially collapsible soils are observed in fine-grained soils that include clay and silt, although collapsible soils may include sandy soils. Results of our laboratory testing indicated that the subsurface soils have a low collapse potential, with the collapse potential ranging from 0.19 to 0.90 percent. As such, it is not anticipated that collapsible soils will impact the proposed improvements.

5.0 GEOLOGIC CONDITIONS

5.1 GEOLOGIC SETTING

The site is located in Salt Lake City, Utah at an elevation of approximately 4,493 feet above mean sea level within the central portion of Salt Lake Valley. The Salt Lake Valley is a deep, sediment-filled structural basin of Cenozoic age flanked by the Wasatch Range to the east and the Oquirrh Mountains, the Promontory Mountains, and the West Hills to the west (Hintze, 1980). A portion of western boundary of the Salt Lake Valley is bordered by the eastern shore of the Great Salt Lake. The Wasatch Range is the easternmost expression of pronounced Basin and Range extension in north-central Utah.

The near-surface geology of the Salt Lake Valley 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. However, these deep-water deposits are in places covered by a thin post-Bonneville alluvial cover. Surface sediments at the site are mapped as upper Pleistocene-aged lacustrine silt and clay deposits associated with the Provo (regressive) phase of the Bonneville Lake cycle (Davis, 2000).

5.2 FAULTING AND SEISMICITY

The site lies within the north-south trending belt of seismicity known as the Intermountain Seismic Belt (ISB) (Hecker, 1993). The ISB extends from northwestern Montana through southwestern Utah. An active fault is defined as a fault that has had activity within the Holocene (<11ka). No active faults are mapped through or immediately adjacent to the site (Black and others, 2003, and Solomon and others, 2006). The site is located approximately 5½ miles west of the nearest mapped portion of the of the Salt Lake City segment of the Wasatch Fault Zone, which is mapped along the western flank of the Wasatch Mountains and the Salt Lake Salient. The Salt Lake City segment of the Wasatch Fault Zone was reportedly last active approximately 1,800 years ago and has a recurrence interval of approximately 2,400 years (Black et. al., 1996, Black et. al., 2003). The site is also located approximately 9½ miles south of the nearest mapped portion of the Granger fault. The Granger fault is one of two main splays of the West Valley fault zone (Keaton and Curry, 1993). The West Valley fault zone trends in a north-south orientation and is located in the central portion of the Salt Lake Valley. While the West Valley fault zone is reported to be active and probably seismically independent of the Wasatch fault zone, sympathetic movement on the West Valley fault zone resulting from major earthquakes on the Wasatch fault zone Salt Lake City segment of the Wasatch fault zone is a possibility. Finally, the site is also located approximately 13¼ miles east of the Oquirrh Fault Zone. The Oquirrh Fault Zone consists of a normal fault located along the western base of the Oquirrh Mountains in the eastern Tooele Valley. This fault was reportedly last active approximately 4,300 and 6,900 years ago, and appears to be seismically independent of the Wasatch Fault Zone (Black and others, 2003). 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 Salt Lake City region. Each of the faults listed above show evidence of Holocene-aged movement, and is therefore considered active.

Seismic hazard maps depicting probabilistic ground motions and spectral response have been developed for the United States by the U.S. Geological Survey as part of NEHRP/NSHMP (Frankel et al, 1996). These maps have been incorporated into both *NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures* (FEMA, 1997) and the *International Building Code* (IBC) (International Code Council, 2015). Spectral responses for the Maximum Considered Earthquake (MCE_R) are shown in the table below. These values generally correspond to a two percent probability of exceedance in 50 years (2PE50) for a "firm rock" site. To account for site effects, site coefficients which vary with the magnitude of spectral acceleration are used. Based on our field exploration, it is our opinion that this location is best described as a Site Class D. The spectral accelerations are shown in the table below. The spectral accelerations are calculated based on the site's approximate latitude and longitude of 40.5221° and -111.9506° respectively and the USGS Seismic Design Maps web based application. Based on IBC, the site coefficients are F_a=1.00 and F_v=1.57. From this procedure the peak ground acceleration (PGA) is estimated to be 0.52g.

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

Site Location: Latitude = 40.5221°N Longitude = -111.9506° W	Site Class D Site Coefficients: Fa = 1.00 Fv = 1.57
Spectral Period (sec)	**Response Spectrum Spectral Acceleration (g)**
0.2	$S_{MS}=(F_a*S_s=1.00*1.31) = 1.31$
1.0	$S_{M1}=(F_v*S_1=1.57*0.44) = 0.68$
[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.	

5.3 LIQUEFACTION

Certain areas within the intermountain region possess a potential for liquefaction during seismic events. Liquefaction is a phenomenon whereby loose, saturated, granular soil deposits lose a significant portion of their shear strength due to excess pore water pressure buildup resulting from dynamic loading, such as that caused by an earthquake. Among other effects, liquefaction can result in densification of such deposits causing settlements of overlying layers after an earthquake as excess pore water pressures are dissipated. The primary factors affecting liquefaction potential of a soil deposit are: (1) level and duration of seismic ground motions; (2) soil type and consistency; and (3) depth to groundwater.

Based on our review of the *Liquefaction-Potential Map for a part of Salt Lake County, Utah, Non-Technical Summary* (Anderson, 1994), the site is located in an area currently designated as having a “Very Low” potential for liquefaction. “Very Low” liquefaction potential indicates that there is less than a 5% probability of having an earthquake within a 100-year period that will be strong enough to cause liquefaction. Groundwater was not encountered in any of the test pits completed for this investigation. As such, the near-surface soils are not considered to be susceptible to liquefaction. A liquefaction analysis was beyond the scope of the project; however, if the owner wishes to have greater understanding of the liquefaction potential of the soils at greater depths, a liquefaction analysis should be completed at the site.

6.0 ENGINEERING CONCLUSIONS 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. We recommend that as part of the site grading that all undocumented fill currently present at the site be removed from beneath the footings. Footings should be established on relatively undisturbed native soils or on a minimum of 12 inches of properly placed and compacted structural fill. The following subsections present our recommendations for general site grading, design of foundations, slabs-on-grade, moisture protection, and corrosivity.

As mentioned previously, review of historic aerial photographs indicates that several structures historically existed at the site. It is possible that foundation walls and footings may be present and not detected during our investigation. The contractor should be aware of this potential and plan accordingly, similarly, portions of the site may have deep undocumented fill segments which will need to be removed and replaced with competent structural backfill.

6.2 EARTHWORK

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

6.2.1 General Site Preparation and Grading

Within areas to be graded (below proposed structures, fill sections, or concrete flatwork), any existing vegetation, topsoil, undocumented fill, debris, or otherwise unsuitable soils should be removed. Any soft, loose, or disturbed soils should also be removed. If the existing fill are undocumented (i.e., no record of compaction tests) they should be over excavated until approximately one foot remains. This remaining one foot of fill can be scarified and compacted in place to a minimum of 95% of the maximum dry density of the modified proctor (ASTM D-1557) if no deleterious (i.e., wood, metal, plastic debris, clasts greater than 6-inches in diameter, etc.) materials are observed in the fill. The material that was over-excavated can likewise be used as structural fill if free of deleterious material and compacted in accordance with the recommendations in this report.

If over-excavation is required, the excavation should extend a minimum of one foot laterally for every foot of depth of over-excavation. Excavations should extend laterally at least two feet beyond flatwork, pavements, and slabs-on-grade. Following the removal of vegetation, topsoil, undocumented fill, unsuitable soils, and loose or disturbed soils, as described above, site grading may be conducted to bring the site to design elevations.

6.2.2 Excavation Stability

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

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

6.2.3 Soft Soil Stabilization

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

In large areas of soft subgrade soils, stabilization of the subgrade may not be practical using the method outlined above. In these areas it may be more economical to place a woven geotextile fabric against the soft soils covered by 18 inches of coarse, sub-rounded to rounded material over the woven geotextile. An inexpensive non-woven geotextile "filter" fabric should also be placed over the top of the coarse, sub-rounded to rounded fill prior to placing structural fill or pavement section soils to reduce infiltration of fines from above. The woven geotextile should be Mirafi RS280i or prior approved equivalent. The filter fabric should consist of a Mirafi 140N, or equivalent as approved by the Geotechnical Engineer.

6.2.4 Structural Fill and Compaction

All fill placed for the support of structures, concrete flatwork, or pavements should consist of structural fill. Structural fill may consist of onsite native soils, onsite fill (as discussed in Section 6.2.1), or an approved imported granular material. If native soils are used as structural fill, the contractor should be aware that native soils will likely require time intensive moisture conditioning and may require additional effort to achieve the desired levels of compaction as recommended in this report. Additionally, if undocumented fill soils are used as structural fill, deleterious materials such as plastic, wood, pvc, and oversized material such as pieces of asphalt, concrete, etc. that are greater than 4-inches will need to be removed. Alternatively, an imported structural fill meeting the specifications below may be used. If soil is imported for use as structural fill, we recommend that it be a relatively well graded granular soil with a maximum of 50 percent passing the No. 4 mesh sieve and a maximum fines content (minus No.200 mesh sieve) of 25 percent. All structural fill soils should be approved by the Geotechnical Engineer prior to placement. Clay and silt particles in imported structural fill should have a liquid limit less than 35 and a plasticity index less than 15 based on the Atterberg Limit's test (ASTM D-4318). The contractor should anticipate testing all soils used as structural fill frequently to assess the maximum dry density, fines content, and moisture content, etc.

All structural fill should be placed in maximum 6-inch loose lifts if compacted by small hand-operated compaction equipment, maximum 8-inch loose lifts if compacted by light-duty rollers, and maximum 10-inch loose lifts if compacted by heavy duty compaction equipment that is capable of efficiently compacting the entire thickness of the lift. We recommend that all structural fill be compacted on a horizontal plane, unless otherwise approved by the geotechnical engineer. Structural fill should be compacted to at least 95% of the maximum dry density, as determined by ASTM D-1557. If the fill soils are greater than 5 feet in thickness, the percent compaction of the soils placed should be increased to 98% of the maximum dry density. The moisture content should be at or slightly above the optimum moisture content at the time of placement and compaction. Also, prior to placing any fill, the excavations should be observed by the geotechnical engineer to observe that any unsuitable materials or loose soils have been removed. In addition, proper grading should precede placement of fill, as described in the **General Site Preparation and Grading** subsection of this report (Section 6.2.1).

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

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

6.3 FOUNDATIONS

The foundations for the proposed structures may consist of conventional strip and/or spread footings founded on undisturbed native soils or on a minimum of 12 inches properly placed and compacted structural fill. Strip and spread footings should be a minimum of 20 and 30 inches wide, respectively, and exterior shallow footings should be embedded at least 30-inches below final grade for frost protection and confinement. Interior footings not subject to frost should be embedded at least 18 inches below final grade to provide confinement.

Conventional strip and spread footings founded on undisturbed native soils or on properly placed and compacted structural fill soils may be proportioned for a maximum net allowable bearing capacity of **1,600 psf**. The net allowable bearing capacity may be increased (typically by one-third) for temporary loading conditions such as transient wind and seismic loads. All footing excavations should be observed by the Geotechnical Engineer prior to footing placement.

6.4 SETTLEMENT

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

6.5 CONCRETE SLAB-ON-GRADE CONSTRUCTION

Concrete slabs-on-grade should be constructed over at least 4 inches of compacted gravel overlying a zone of structural fill that is at least 12 inches thick. Structural fill should be compacted to at least 95% of the maximum dry density as determined by ASTM D-1557 (modified proctor) prior to placement of gravel. The gravel should consist of road base or clean drain rock with a ¾-inch maximum particle size and no more than 12 percent fines passing the No. 200 mesh sieve. The gravel layer should be compacted to at least 95 percent of the maximum dry density of modified proctor or until tight and relatively unyielding if the material is non-proctorable. All concrete slabs should be designed to minimize cracking as a result of shrinkage. Consideration should be given to reinforcing the slab with welded wire, re-bar, or fiber mesh.

6.6 EARTH PRESSURES AND LATERAL RESISTANCE

Lateral forces imposed upon conventional foundations due to wind or seismic forces may be resisted by the development of passive earth pressures and friction between the base of the footing and the supporting soils. In determining the frictional resistance against concrete, a coefficient of friction of 0.39 should be used for structural fill against concrete.

Ultimate lateral earth pressures from native material acting against buried walls and structures for long term condition 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*	0.33	40
At-rest**	0.50	60
Passive*	3.00	360
Seismic Active***	0.61	74
Seismic Passive***	-0.99	-118

* Based on Coulomb's equation

** Based on Jaky

*** Based on Mononobe-Okabe Equation

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

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

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

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

6.7 MOISTURE PROTECTION AND SURFACE DRAINAGE

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

Moisture should not be allowed to infiltrate the soils in the vicinity of, or upslope from, the structures. We recommend that roof runoff devices be installed to direct all runoff a minimum of 10 feet away from structures. The grade within 10 feet of the structures should be sloped a minimum of 5% away from the structure.

6.8 ASPHALT PAVEMENT DESIGN

The laboratory determined CBR value for a bulk sample collected from the site is 1.5. No traffic information was available at the time this report was prepared. For this pavement section design we have assumed 250 passenger vehicles/day, 1 light duty trucks/day, and 1 medium duty trucks/day. The following pavement design alternatives have been developed for a 20-year design life assuming an annual growth rate of 0%, and an estimated equivalent single axle load (ESAL) of approximately 35,000 ESALs. Based on the information obtained and the above mentioned assumptions, we recommend that the following pavement section be constructed;

Flexible Pavement Section

Asphalt Concrete (in.)	Untreated Base Course (in.)
3½	**16**

As an alternative, the following equivalent pavement section may be considered;

Flexible Pavement Section

Asphalt Concrete (in.)	Untreated Base Course (in.)	Granular Borrow (in.)
3½	**8**	**12**

As an alternative pavement design, a reinforcing geosynthetic fabric can be used on the site to reduce the required thickness of the untreated base course. A woven fabric can greatly increase the effective strength of a subsurface soil by carrying a portion of the tensile load experienced from the anticipated traffic. Based on our analysis, the following flexible pavement section was created incorporating a Tencate Mirafi® RS580i woven geosynthetic fabric.

Flexible Pavement Section with Woven Geosynthetic

Asphalt Concrete (in)	Untreated Base Course (in)
3½	**8**

The woven fabric should be placed directly on the undisturbed native soils in accordance with the manufacturers recommendations.

Asphalt has been assumed to be a high stability plant mix; base course material should be composed of crushed stone with a minimum CBR of 70. Asphalt should be compacted to a minimum density of 96% of the Marshall value and base course should be compacted to at least 95% of the MDD of the modified proctor. Additionally, we have assumed that the upper 12 inches of the subgrade will be reworked and compacted to at least 95% of the modified proctor.

It is our experience that pavement in areas where trucks frequently turn around, backup, or load and unload, including the drive thru area, experience more distress. If the owner wishes to prolong the life of the pavement in these areas, consideration should be given to using a Portland cement concrete (rigid) pavement in these areas. The following rigid pavement section is recommended:

Rigid Pavement Section

Concrete (in.)	Untreated Base Course (in.)
5	**6**

Concrete should consist of a low slump, low water cement ratio mix with a minimum 28-day compressive strength of 4,000 psi. Base course should be compacted to at least 95% of the MDD as determined by ASTM D-1557. Additionally, we have assumed that the upper 12 inches of the subgrade will be reworked and compacted to at least 95% of the MDD as determined by ASTM D-1557.

If traffic conditions vary significantly from our stated assumptions, GeoStrata should be contacted so we can modify our pavement design parameters accordingly. Specifically if the traffic counts are significantly higher or lower, we should be contacted to revise the pavement section design if necessary. The pavement section thicknesses above assumes that the majority of construction traffic including cement trucks, cranes, loaded haulers, etc. has ceased. If a significant volume of construction traffic occurs after the pavement section has been constructed, the owner should anticipate maintenance or a decrease in the design life of the pavement area.

As mentioned in Section 4.2.1 of this report, undocumented fill soils were encountered within certain portions of the subject site. These fill soils persisted to a depth of approximately 5 feet as observed in our test pit TP-1. It is possible that potentially thicker sections of undocumented fill could be encountered depending on the original topography of the site. The lateral extent of the undocumented fill soils could not be exactly determined, but a site map showing the locations of the previously existing structures has been included as Plate A-4. It is anticipated that the undocumented fill sections will be thickest in these areas. Pavement sections underlain by undocumented fill soils may experience excessive and premature distress. To completely remove the potential for premature distress from occurring, the undocumented fill soils located under the proposed roadways and parking areas should be removed and the site brought back up to design grade using properly compacted structural fill. Should this prove uneconomical, the upper 24 inches of the undocumented fill soils may be over-excavated and replaced with structural fill, however, the Client should be aware that by leaving the undocumented fill soils in place below the roadways, there is the potential for excessive distress and increased maintenance of pavements in these areas.

6.9 SOIL CORROSIVITY

Chemical testing was completed as a part of this investigation. It was found that the native soils have a minimum resistivity of 1,100 OHM-cm. Based on this data we expect that the native onsite soils will be *very corrosive*. We recommend that a corrosion engineer be consulted to design cathodic protection or sacrificial thicknesses.

The sulfate content was measured at 39.8 ppm and the pH was slightly basic at 8.45. This indicates that the onsite soils have a *negligible* potential for sulfate attack on concrete. We recommend that standard Type I/II cement be used for this project.

7.0 CLOSURE

7.1 LIMITATIONS

The recommendations contained in this report are based on our limited field exploration, laboratory testing, and understanding of the proposed construction. The subsurface data used in the preparation of this report were obtained from the explorations made for this investigation. It is possible that variations in the soil and groundwater conditions could exist between the points explored. The nature and extent of variations may not be evident until construction occurs. If any conditions are encountered at this site that are different from those described in this report, we should be immediately notified so that we may make any necessary revisions to recommendations contained in this report. In addition, if the scope of the proposed construction changes from that described in this report, GeoStrata should be notified.

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

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

7.2 ADDITIONAL SERVICES

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

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

8.0 REFERENCES CITED

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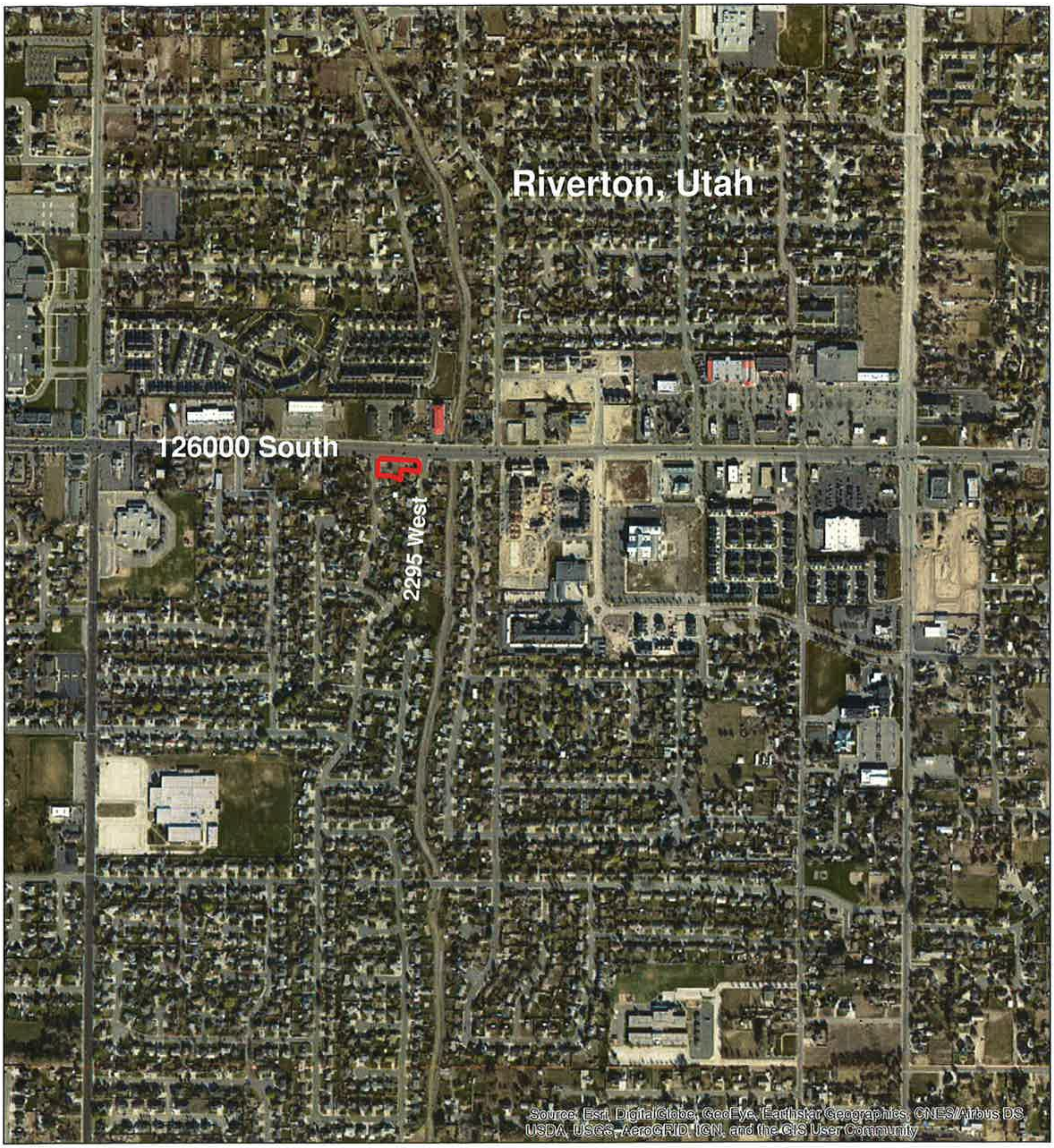

0 395 790 1,580 2,370 3,160 Feet

1:10,000

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GeoStrata

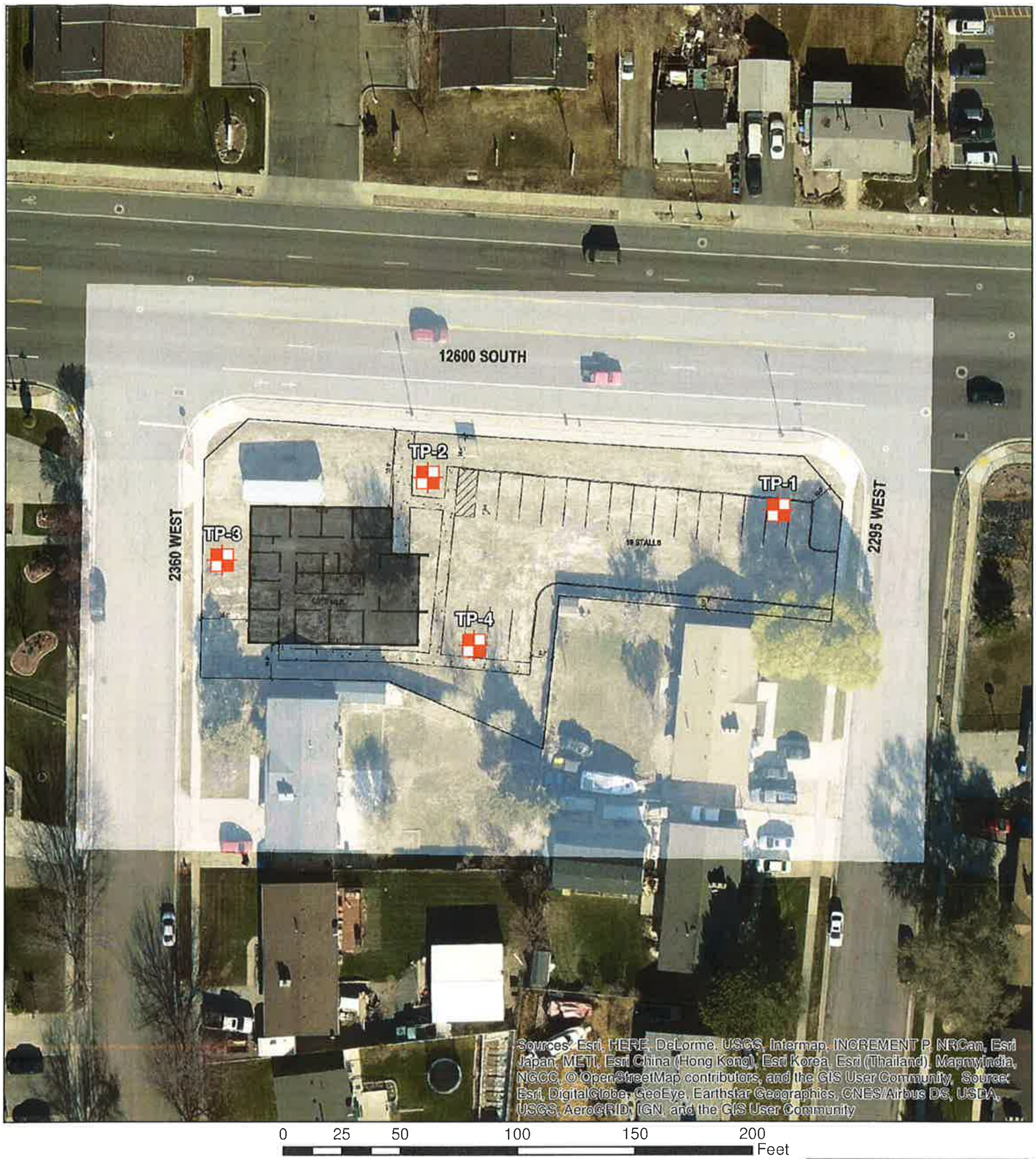
Keystone Construction
Commercial Building
Riverton, Utah
Project Number: 1012-012

Site Vicinity Map

Plate A-2

Legend

Appriximate Site Boundary



GeoStrata

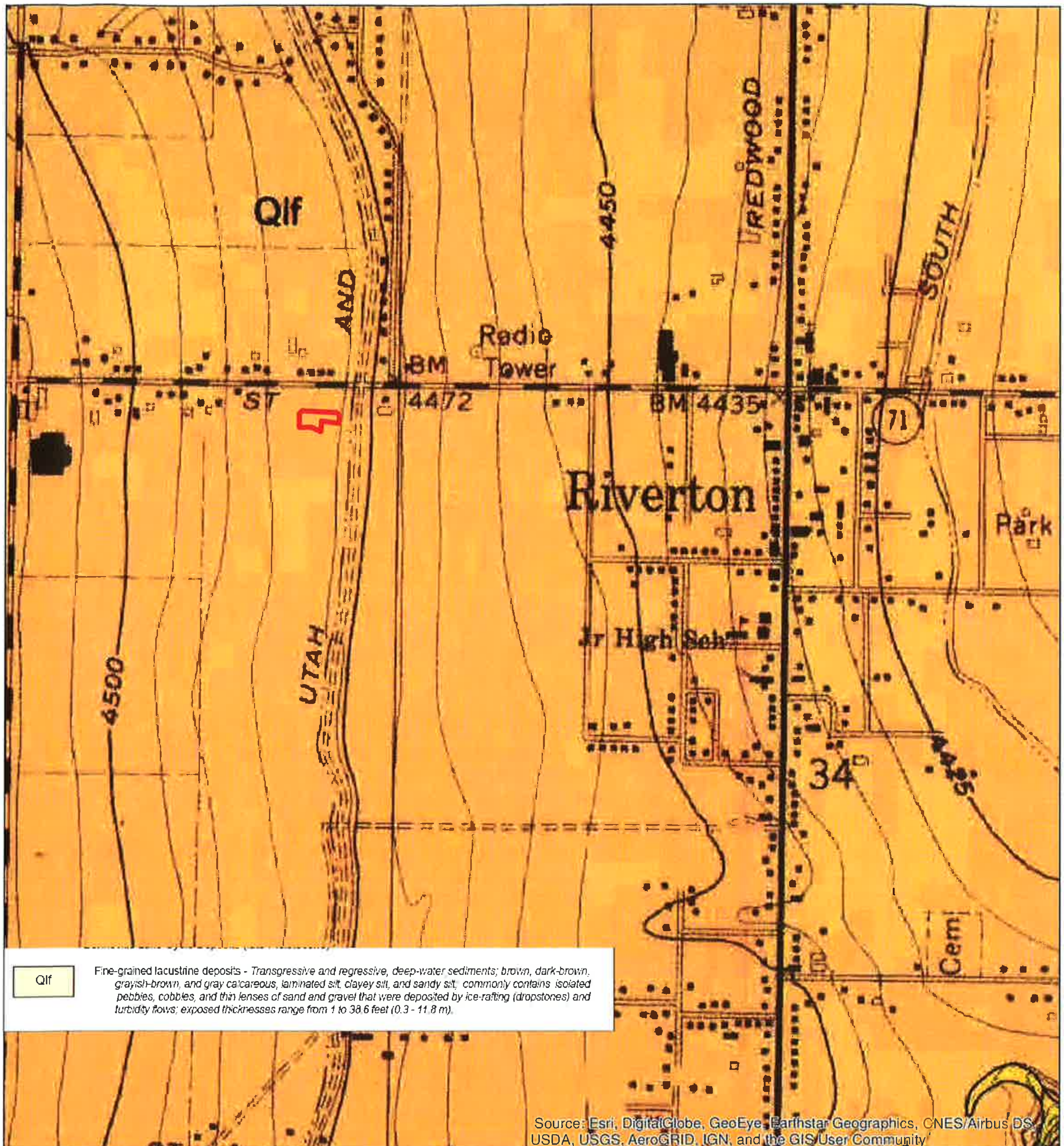
Keystone Construction
Commercial Building
Riverton, Utah
Project Number: 1012-012

Exploration Location Map

Plate A-2

Legend

Approximate Test Pit Location



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1:10,484

Basemap

USGS Geological Map of the Midvale Quadrangle, Salt Lake County, Utah

Fitzhugh D. Davis 2000

N

GeoStrata

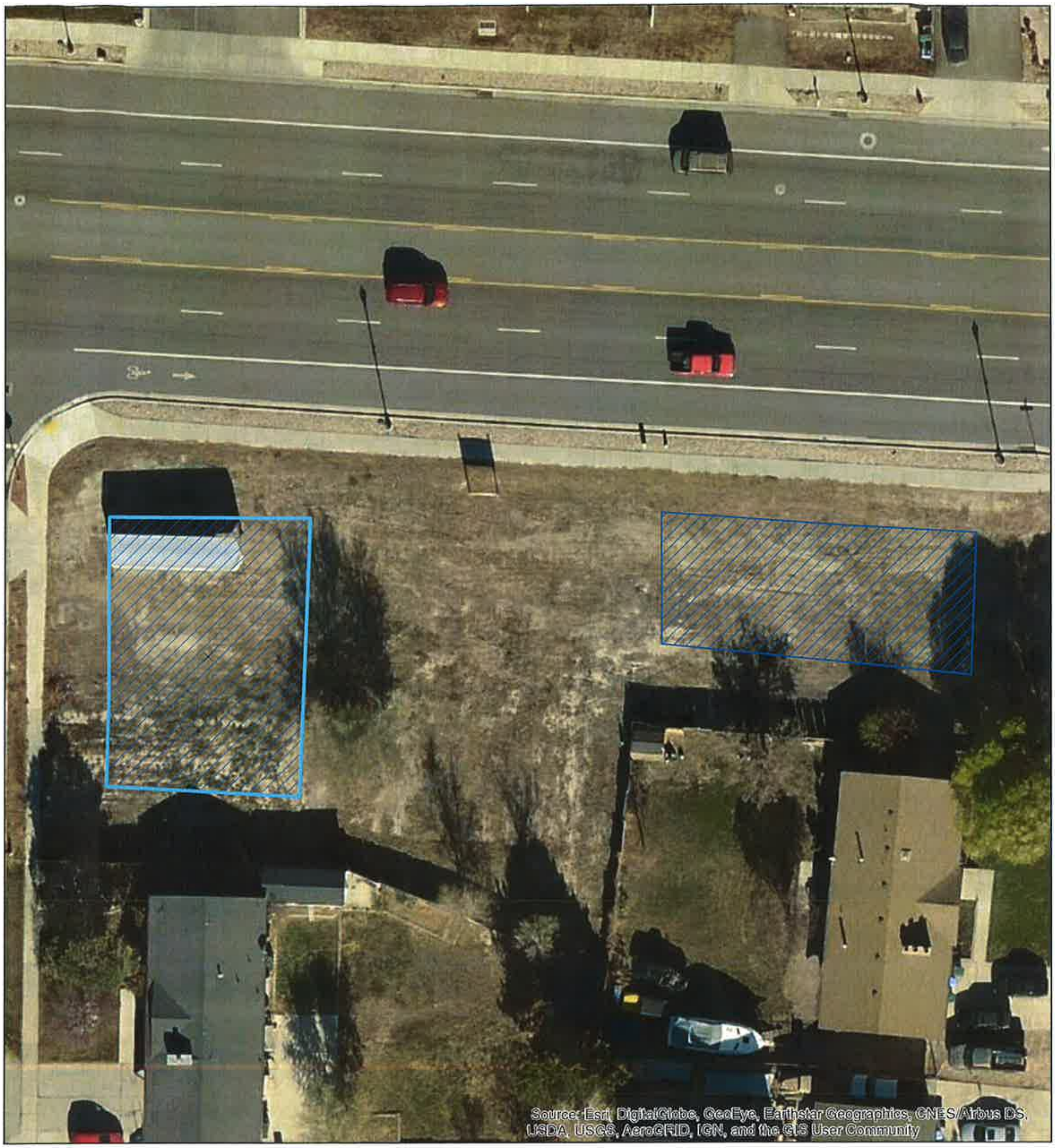
Legend

Apprìximate Site Boundary

Keystone Construction
Commercial Building
Riverton, Utah
Project Number: 1012-012

Site Geology Map

Plate A-3



0 15 30 60 90 120 Feet

1:400

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Keystone Construction
Commercial Building
Riverton, Utah
Project Number: 1012-012

Historicial Development Location Map

Plate A-4

Legend

Potential Fill Locatin Map

DATE	
STARTED:	5/25/17
COMPLETED:	5/25/17
BACKFILLED:	5/25/17

Keystone Construction
Commercial Building
Riverton, Utah
Project Number 1012-012

GeoStrata Rep: AP
Rig Type: Case 580

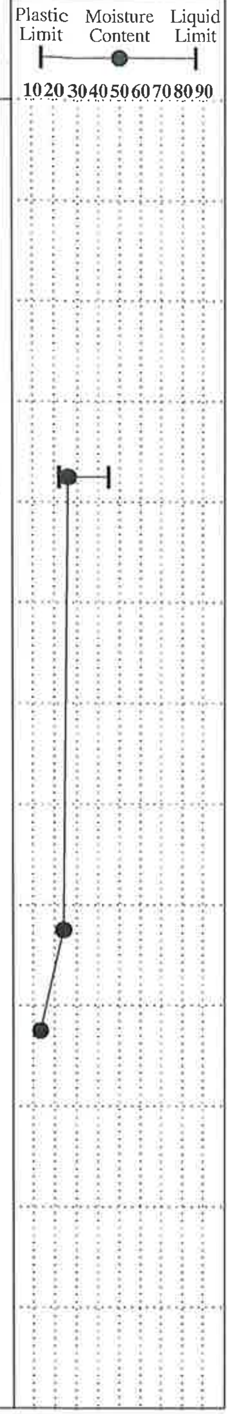
TEST PIT NO:
TP-1
Sheet 1 of 1

LOCATION

NORTHING	EASTING	ELEVATION

Depth (feet)	Unified Soil Classification	Material Description	Dry Density (pcf)	Moisture Content %	Percent minus 200	Liquid Limit	Plasticity Index
0		Fill; Lean CLAY with sand and gravel - dark brown, moist					
		Fill; Lean CLAY - stiff, brown with teal and orange mottling, moist, iron staining and pinholes throughout					
		- Torvane 1133 psf	92.0	25.7	90.0	45	23
5		- Rebar and PVC @ 5 feet					
	CL	Lean CLAY with sand and gravel - stiff, brown with teal and orange mottling, moist, iron staining and pinholes throughout					
			101.5	23.8			
	SC	Clayey SAND with gravel - dense, broen, moist					
				12.9	54.7		
10							
	ML	Sandy SILT - stiff, yellow-brown, moist					
					53.8		

Moisture Content and Atterberg Limits



Bottom of Test Pit @ 13 Feet

GeoStrata

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

WATER LEVEL
- MEASURED
- ESTIMATED

NOTES:

Plate
B-1

DATE	
STARTED:	5/25/17
COMPLETED:	5/25/17
BACKFILLED:	5/25/17

Keystone Construction
Commercial Building
Riverton, Utah
Project Number 1012-012

GeoStrata Rep: AP
Rig Type: Case 580

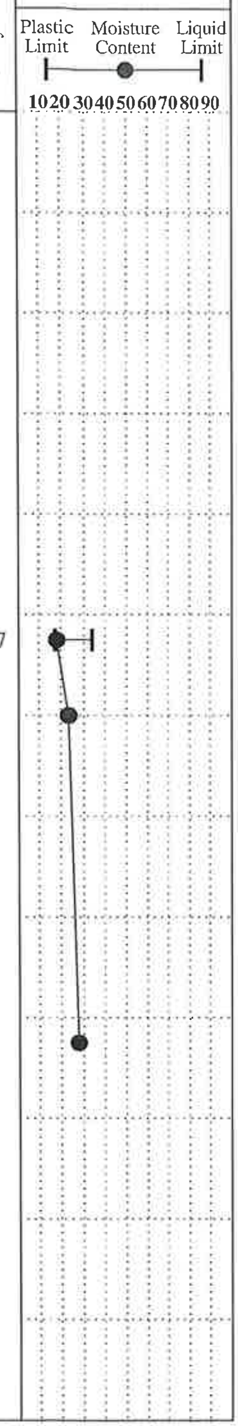
TEST PIT NO: **TP-2**
Sheet 1 of 1

LOCATION: NORTHING | EASTING | ELEVATION

MATERIAL DESCRIPTION

Depth (feet)	Samples	USCS	Material Description	Dry Density (pcf)	Moisture Content %	Percent minus 200	Liquid Limit	Plasticity Index
0			TOPSOIL; Lean CLAY with sand and gravel - dark brown, moist					
		CL	Lean CLAY with sand and gravel - stiff, brown, moist					
5	Grab				17.8	82.1	34	17
		CL	Lean CLAY - stiff, brown with teal and orange mottling, moist, iron staining throughout	93.7	23.1			
	Grab			93.8	27.5			
10								
	Grab	SM	Silty SAND with gravel - medium dense, yellow-brown, moist					
			Bottom of Test Pit @ 12 Feet					

Moisture Content and Atterberg Limits



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

WATER LEVEL
- MEASURED
- ESTIMATED

NOTES:

GeoStrata

Plate B-2

DATE	
STARTED:	5/25/17
COMPLETED:	5/25/17
BACKFILLED:	5/25/17

Keystone Construction
Commercial Building
Riverton, Utah
Project Number 1012-012

GeoStrata Rep: AP
Rig Type: Case 580

TEST PIT NO:
TP-3
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; Poorly Graded GRAVEL with sand - grey-brown, moist						
					CL	Lean CLAY - stiff, brown, moist						
1		Grab							90.0	38	19	
	5											
2												
		Grab			CL	Lean CLAY with sand and gravel - stiff, brown with teal and orange mottling, moist, iron staining and pinholes throughout						
3	10											
					SM	Silty SAND with gravel - medium dense, yellow-brown, moist, iron staining throughout						
						Bottom of Test Pit @ 11 Feet						

GeoStrata

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

WATER LEVEL
- MEASURED
- ESTIMATED

NOTES:

Plate
B-3

DATE	
STARTED:	5/25/17
COMPLETED:	5/25/17
BACKFILLED:	5/25/17

Keystone Construction
Commercial Building
Riverton, Utah
Project Number 1012-012

GeoStrata Rep: AP
Rig Type: Case 580

TEST PIT NO: **TP-4**
Sheet 1 of 1

LOCATION: NORTHING | EASTING | ELEVATION

DEPTH (METERS)	DEPTH (FEET)	SAMPLES	WATER LEVEL	UNIFIED SOIL CLASSIFICATION	MATERIAL DESCRIPTION	Dry Density (pcf)	Moisture Content %	Percent minus 200	Liquid Limit	Plasticity Index
0	0			CL	Lean CLAY - stiff, brown orange mottling, moist, iron staining throughout					
1		Grab			- Torvane 950 psf	97.7	23.2			
2	5									
				CL	Lean CLAY - stiff, brown with teal and orange mottling, moist, iron staining throughout					
3	10			SM	Silty SAND with gravel - medium dense, yellow-brown, moist, iron staining throughout					
					Bottom of Test Pit @ 11 Feet					

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

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

WATER LEVEL
- MEASURED
- ESTIMATED

NOTES:

Plate B-4

UNIFIED SOIL CLASSIFICATION SYSTEM

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

LOG KEY SYMBOLS

BORING SAMPLE LOCATION

TEST-PIT SAMPLE LOCATION

WATER LEVEL (level after completion)

WATER LEVEL (level where first encountered)

CEMENTATION

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

OTHER TESTS KEY

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

MODIFIERS

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

MOISTURE CONTENT

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

STRATIFICATION

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

GENERAL NOTES

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

APPARENT / RELATIVE DENSITY - COARSE-GRAINED SOIL

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

CONSISTENCY - FINE-GRAINED SOIL

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

GeoStrata

Soil Symbols Description Key

Keystone Construction
Commercial Building
Riverton, Utah
Project Number: 1012-012

Plate B-5

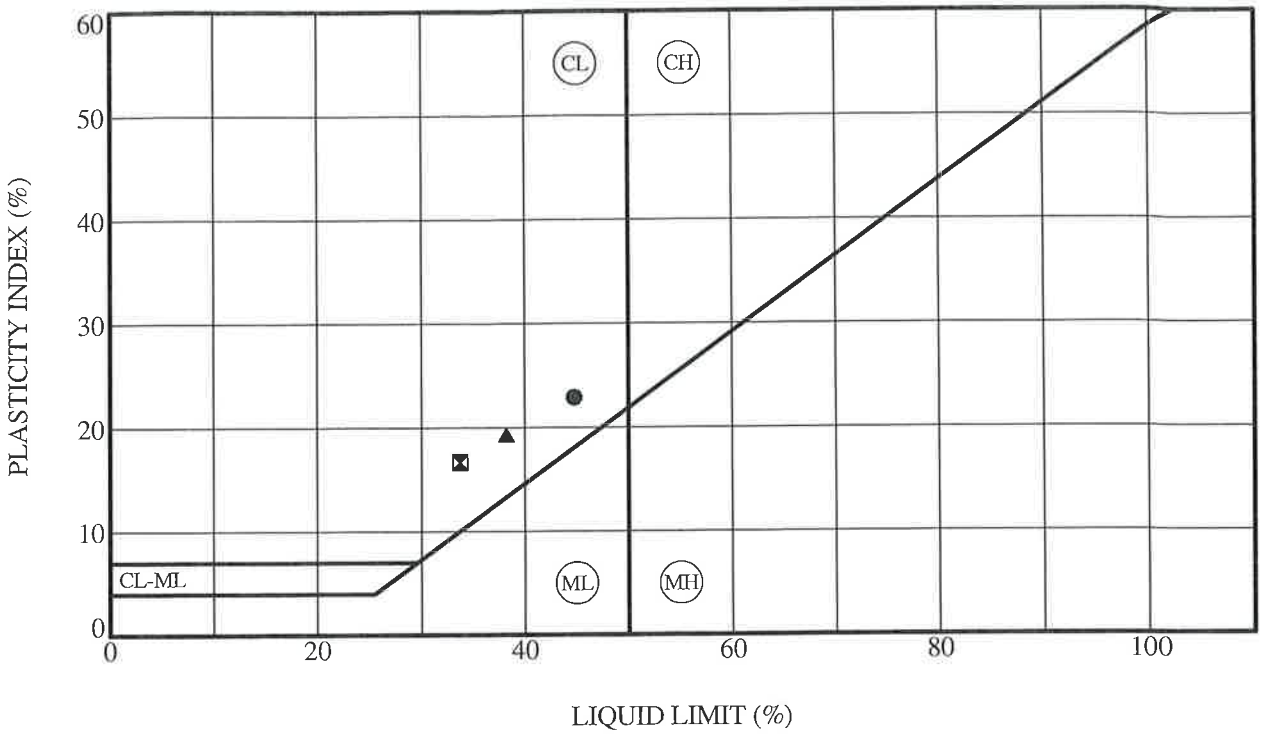
Test Pit No.	Sample Depth (feet)	USCS Soil Classification	Natural Moisture Content (%)	Natural Dry Density (pcf)	Optimum Moisture Content (%)	Maximum Dry Density (pcf)	Gradation			Atterberg		Consolidation			Collapse (%)	CBR (%)	Sulfate Content (ppm)	Resistivity (Ω-cm)	pH
							Gravel (%)	Sand (%)	Fines (%)	LL	PI	Cc	Cr	OCR					
TP-1	3.5	CL	25.7	92			10.0		90.0	45	23	0.113	0.029	3.7	0.19				
TP-1	8	CL	23.8	101.5															
TP-1	9	SC	12.9				45.3		54.7										
TP-1	12	ML					0.5	45.7	53.8										
TP-2	5	CL	17.8				17.9		82.1	34	17	0.104	0.021	3.0	0.9				
TP-2	6	CL	23.1	93.7															
TP-2	9	CL	27.5	93.8															
TP-3	3	CL		99.6	14.5	98.6	2.3	7.7	90.0	38	19					1.5	39.8	1100	8.45
TP-4	3		23.2	97.7															

GeoStrata

Lab Summary Report

Keystone Construction
Commercial Building
Riverton, Utah
Project Number: 1012-012

Plate C - 1



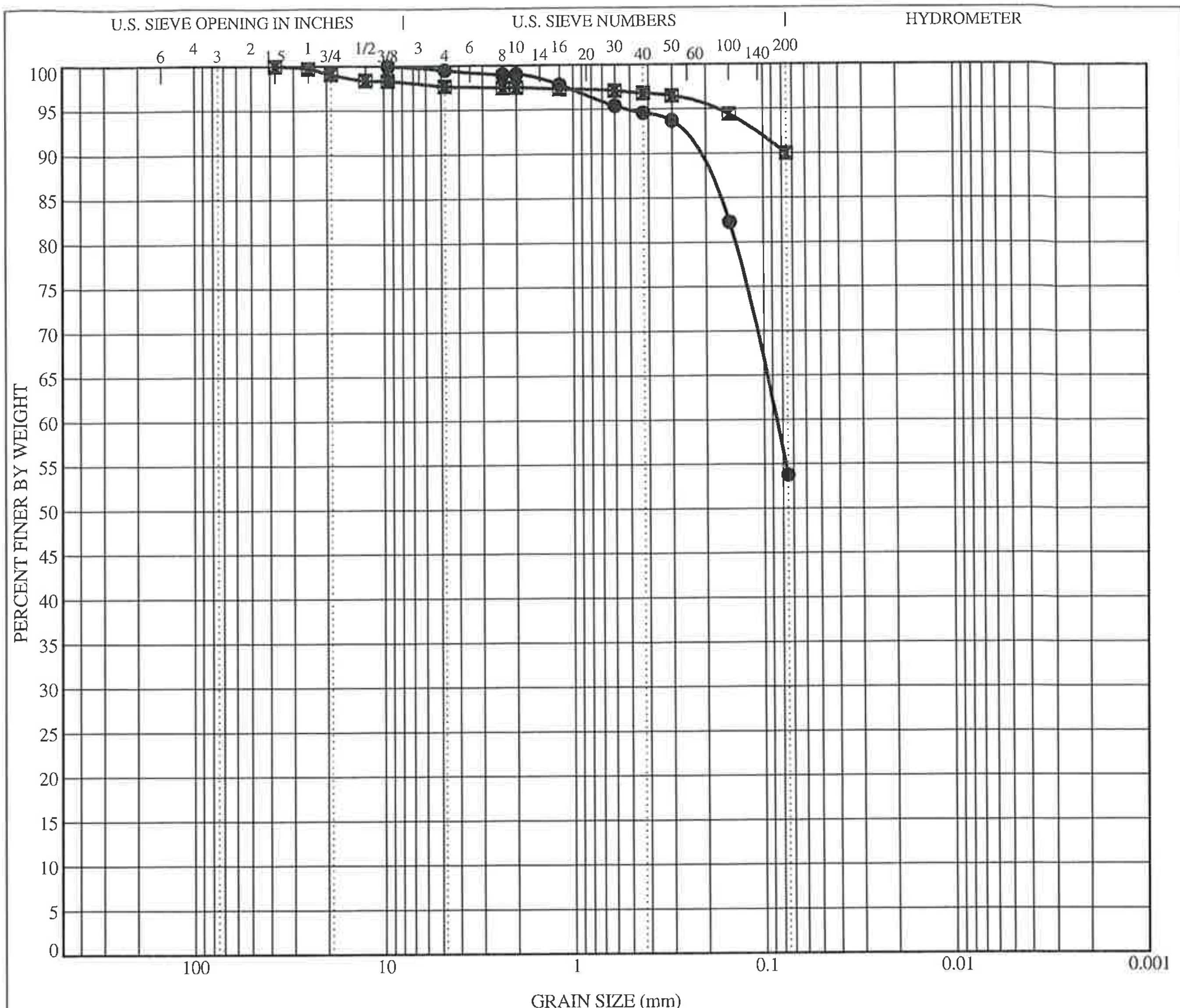
	Sample Location	Depth (ft)	LL (%)	PL (%)	PI (%)	Fines (%)	Classification
●	**TP-1**	**3.5**	**45**	**22**	**23**	**90.0**	**Lean CLAY**
⊠	**TP-2**	**5.0**	**34**	**17**	**17**	**82.1**	**Lean CLAY with sand and gravel**
▲	**TP-3**	**3.0**	**38**	**19**	**19**	**90.0**	**Lean CLAY**

GeoStrata

ATTERBERG LIMITS' RESULTS - ASTM D 4318

Keystone Construction
Commercial Building
Riverton, Utah
Project Number: 1012-012

Plate C - 2



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	Sample Location	Depth	Classification	LL	PL	PI	Cc	Cu
●	**TP-1**	**12.0**	**Sandy SILT**					
⊠	**TP-3**	**3.0**	**Lean CLAY**	**38**	**19**	**19**		

	Sample Loctaion	Depth	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**TP-1**	**12.0**	**9.5**	**0.087**			**0.5**	**45.7**	**53.8**	
⊠	**TP-3**	**3.0**	**37.5**				**2.3**	**7.7**	**90.0**	

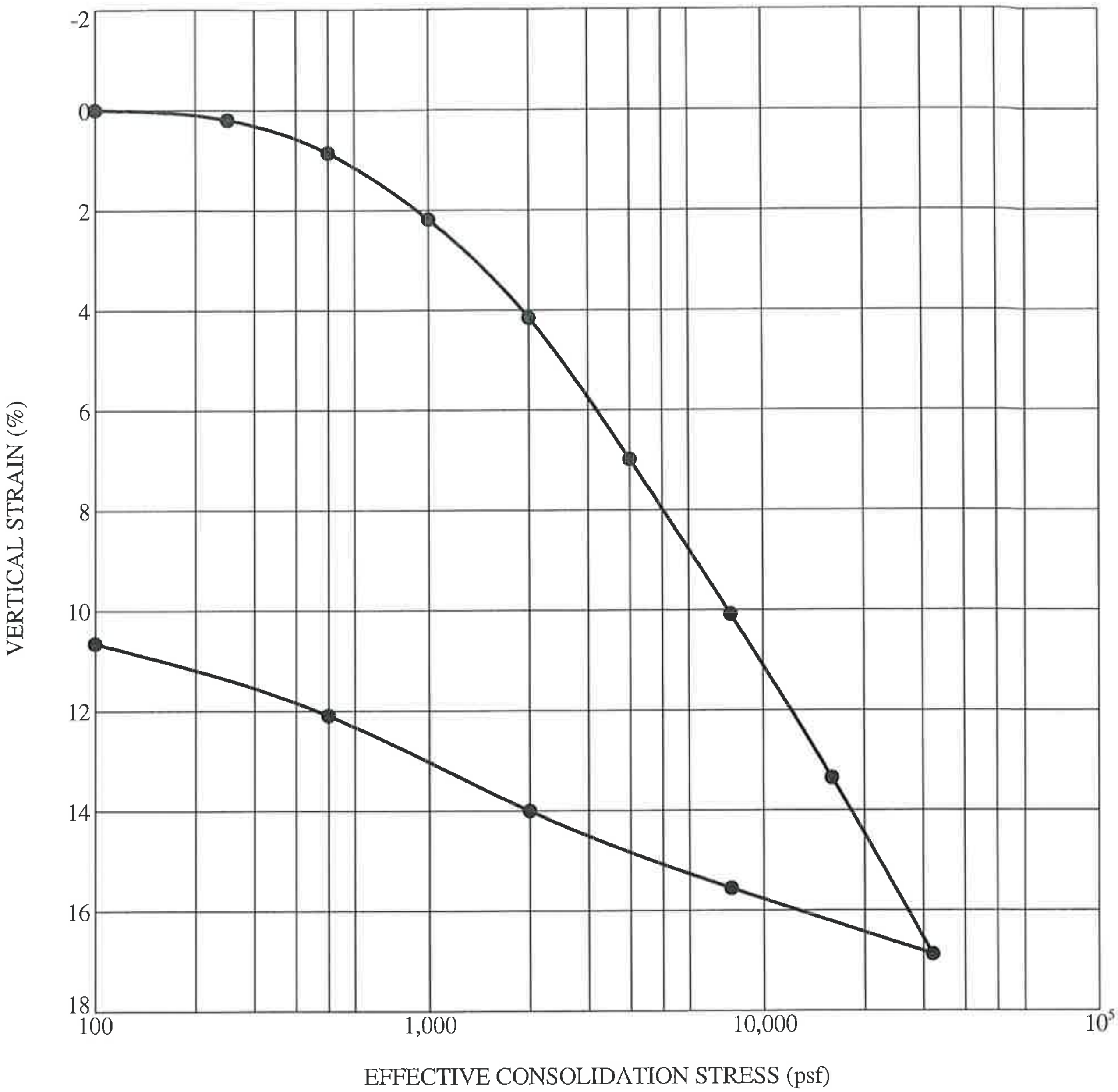
GeoStrata

GRAIN SIZE DISTRIBUTION - ASTM D422

Keystone Construction
Commercial Building
Riverton, Utah
Project Number: 1012-012

Plate

C - 3



	Sample Location	Depth (ft)	Classification	γ_d (pcf)	MC (%)	C'_c	C'_r	OCR
●	**TP-1**	**3.5**	**Lean CLAY**			**0.113**	**0.029**	**3.7**
⊠								
▲								
★								
⊙								

1-D CONSOLIDATION TEST - ASTM D 2435

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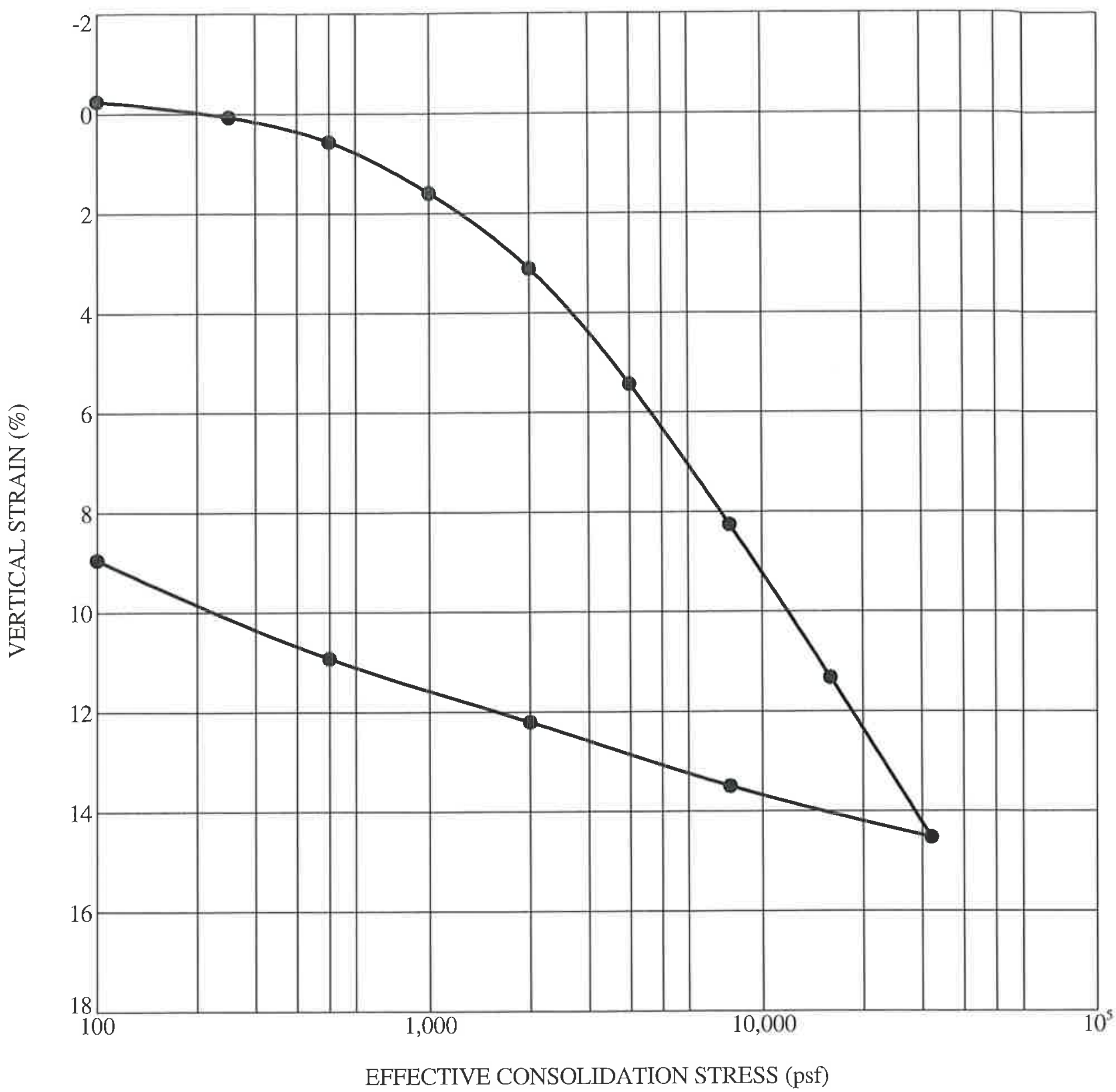
Keystone Construction
Commercial Building
Riverton, Utah
Project Number: 1012-012

Plate

C - 4



	Sample Location	Depth (ft)	Classification	γ_d (pcf)	MC (%)	C'_c	C'_r	OCR
●	**TP-2**	**5.0**	**Lean CLAY with sand and gravel**			**0.104**	**0.021**	**3.0**
⊠								
▲								
★								
⊙								

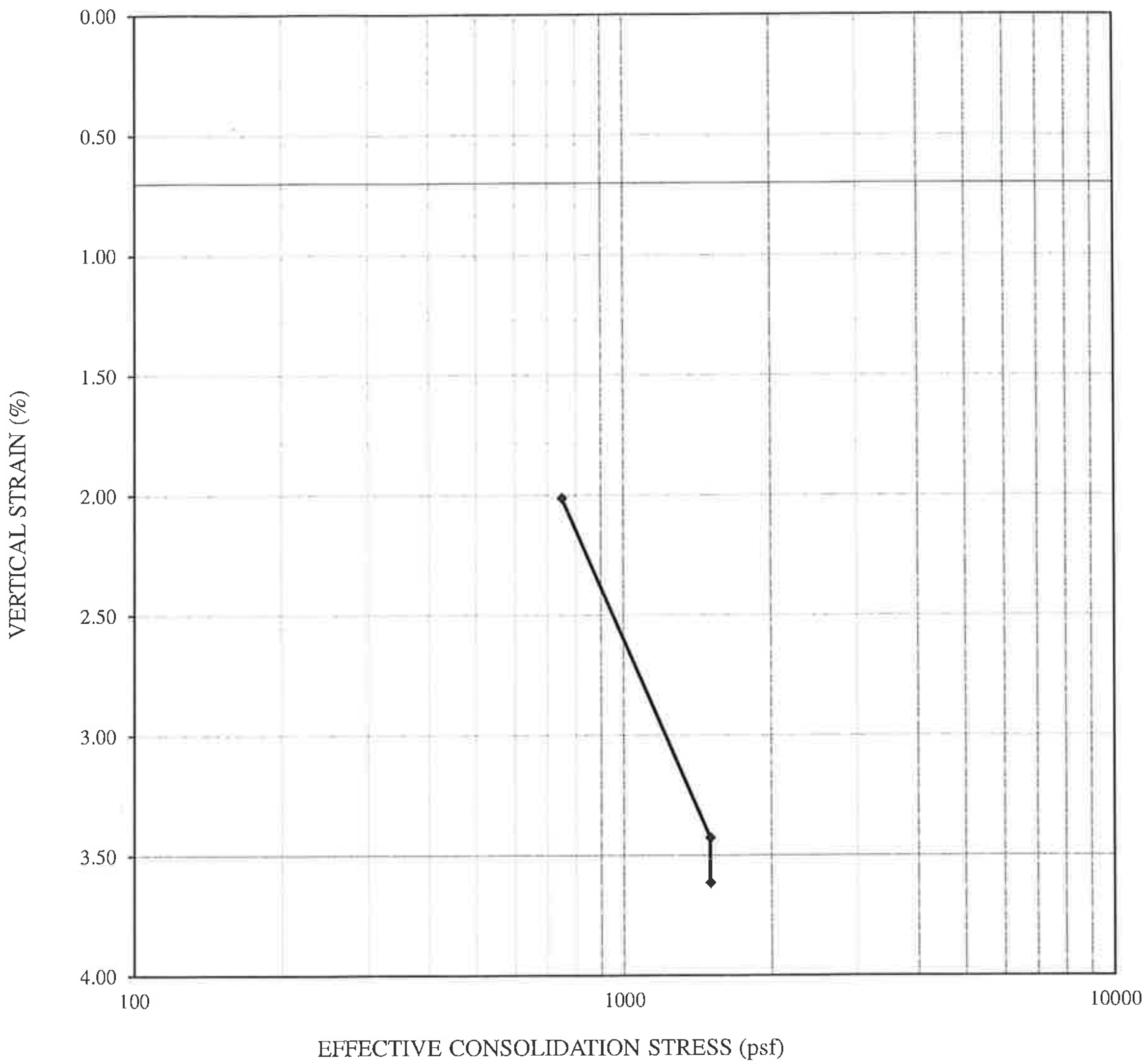
GeoStrata

1-D CONSOLIDATION TEST - ASTM D 2435

Keystone Construction
Commercial Building
Riverton, Utah
Project Number: 1012-012

Plate

C - 5



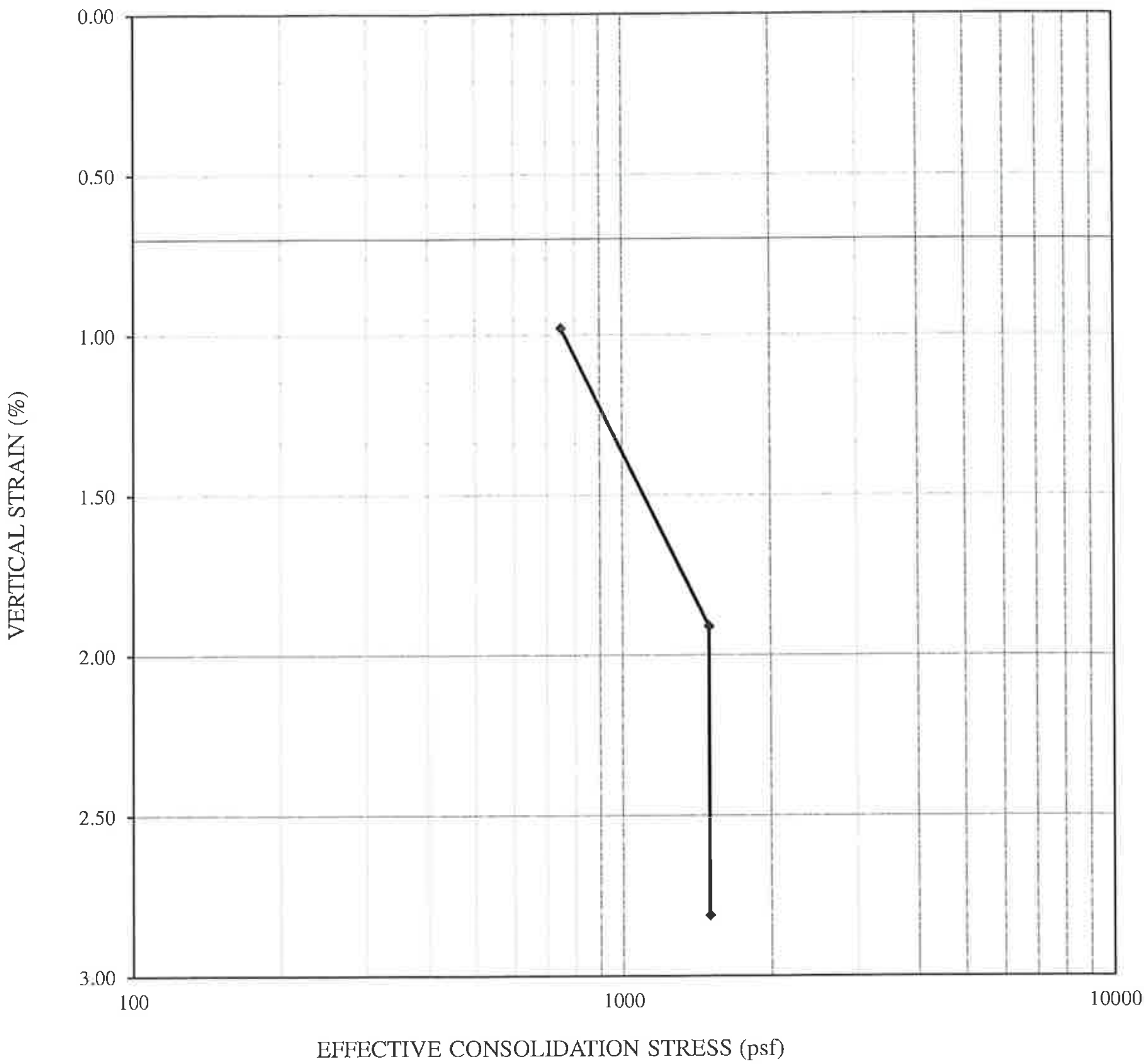
Sample Location	Depth (ft)	Classification	γ_d (pcf)	MC (%)	Inundation Load (psf)	Swell (%)	Collapse (%)
TP-1	**3.5**	**Lean CLAY**	**91.5**	**24.7**	**1500**	**---**	**0.19**

GeoStrata

1-D SWELL/COLLAPSE TEST

Keystone
Commercial Building
Riverton, Uath
Project Number: 1012-012

Plate C-6



Sample Location	Depth (ft)	Classification	γ_d (pcf)	MC (%)	Inundation Load (psf)	Swell (%)	Collapse (%)
TP-2	**5.0**	**Lean CLAY with sand and gravel**	**95.4**	**20.3**	**1500**	**---**	**0.90**

GeoStrata

1-D SWELL/COLLAPSE TEST

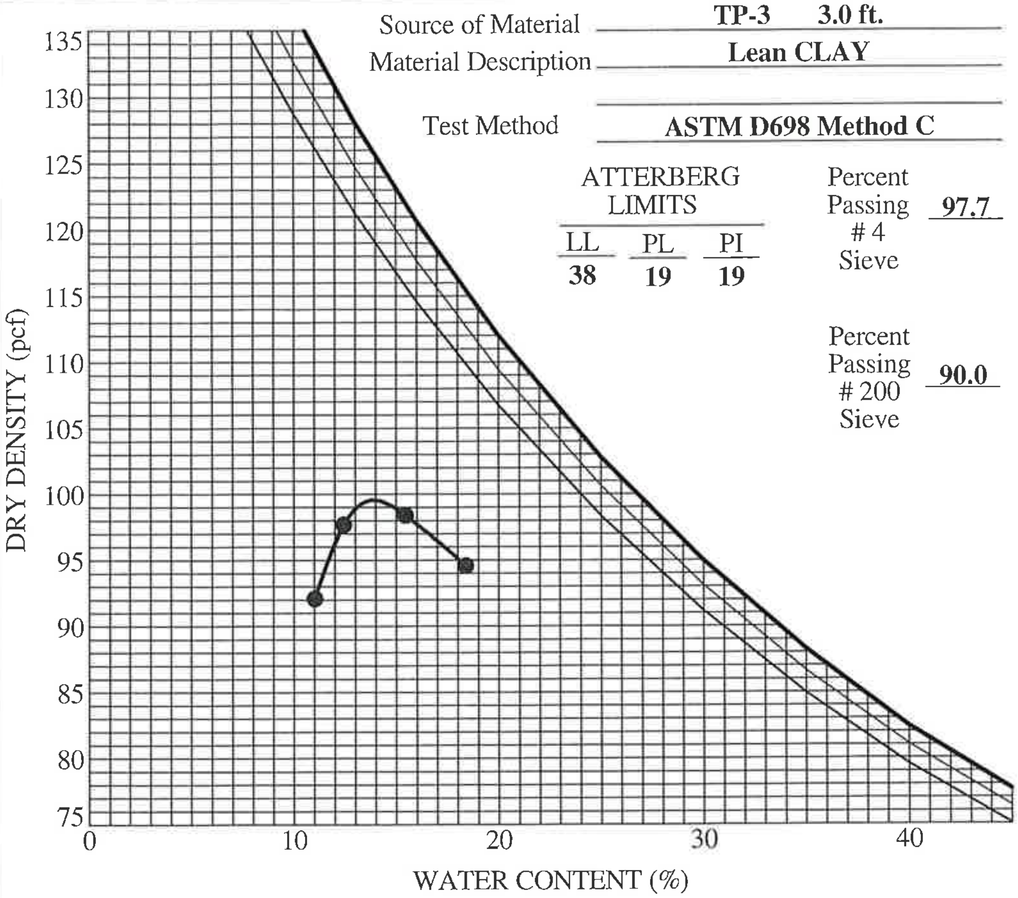
Keystone
Commercial Building
Riverton, Utah
Project Number: 1012-012

Plate C-7

Source of Material: TP-3 3.0 ft.

Material Description: Lean CLAY

Test Method: ASTM D698 Method C



ATTERBERG LIMITS

LL	PL	PI
38	19	19

Percent Passing # 4 Sieve: 97.7

Percent Passing # 200 Sieve: 90.0

TEST RESULTS

Maximum Dry Density: 98.6 (pcf)

Optimum Water Content: 14.5 (%)

Percent Rock: ____ (%)

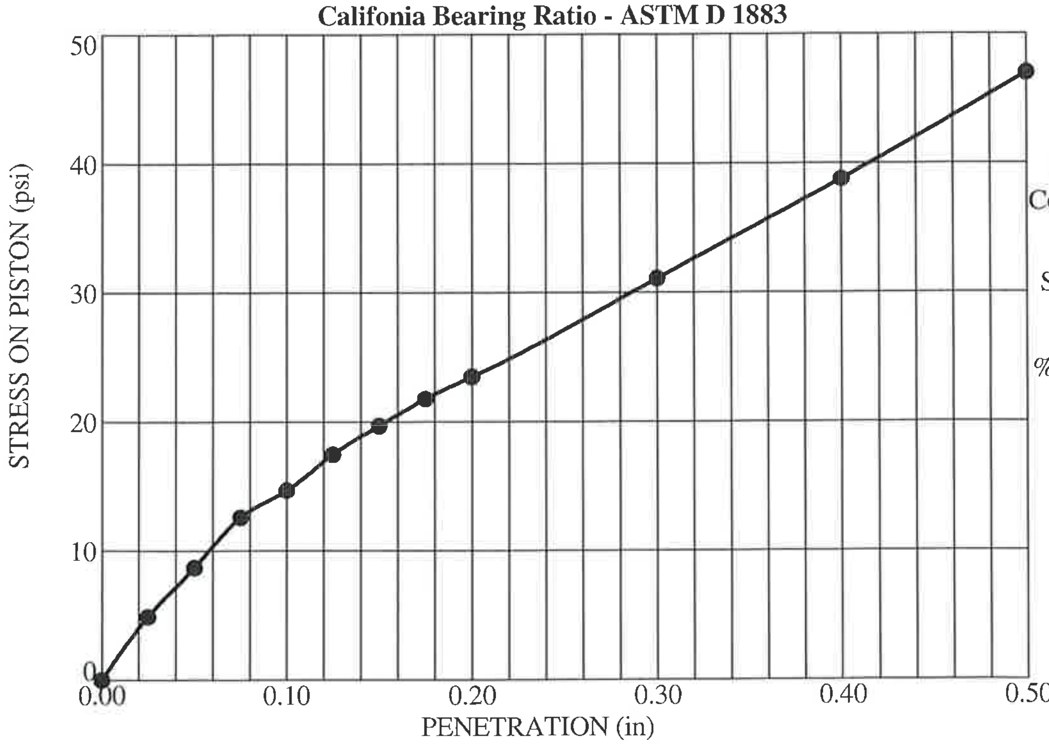
Corrected Maximum Dry Density: ____ (pcf)

Corrected Optimum Water Content: ____ (%)

Curves of 100% Saturation for Specific Gravity Equal to:

2.60, 2.70, 2.80

Califonia Bearing Ratio - ASTM D 1883



Dry Density: 99.6 (pcf)

Relative Compaction: 102 (%)

Surcharge: 50 (psf)

% Standard CBR: 1.50

Swell: 3.01 (%)

C_COMPACTION SPLIT EXPLORATION LOGS.GPJ GEOSTRATA.GDT 6/8/17

GeoStrata

COMPACTION AND CBR TEST

Keystone Construction
Commercial Building
Riverton, Utah
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Plate C - 8