GeoStrata

Geotechnical Investigation

Lot 3R Powder 11 at Powder Mountain

6599 North Powder Mountain Road
Eden, UT 84310

July 1, 2020

Prepared For:

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GeoStrata Job No. 1174-006

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APPENDICES

1.0 EXECUTIVE SUMMARY

This report presents the results of a geotechnical investigation conducted for the proposed residential development to be constructed at 6599 North Mountain Road in Eden, Utah. The subject site has an area of approximately 0.68 acres. The purposes of this investigation are to provide an assessment of the nature and engineering properties of the subsurface soils at the subject site and to provide recommendations for general site grading and the design and construction of foundations, pavement sections, and slabs-on-grade. We have also performed a slope stability assessment of the proposed cut and fill plans as part of the development of the site.

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 a final, design grade geotechnical investigation be completed prior to the initiation of construction activities.

The subsurface soil conditions were explored at the subject property by excavating two test pits to depths ranging from 15 to 16 feet below the existing site grade. Based on our field observations, the site is overlain by approximately 0- to ½-ft of undocumented fill composed of silt and gravel. The undocumented fill is underlain by approximately 1 foot of topsoil composed of silt, clay and gravel. Based on our geologic review of the site and our subsurface investigation, the topsoil is underlain by Neoproterozoic Mutual Formation (Zm) as shown on Plate A-4, Site Vicinity 30' x 60' Geologic Map.

Based on the results of our investigation, the foundations for the proposed structures may consist of conventional strip and/or spread footings. Strip and spread footings should be a minimum of 20 and 36 inches wide, respectively, and exterior shallow footings should be embedded at least 48 inches below final grade for frost protection and confinement. Interior shallow footings not susceptible to frost conditions should be embedded at least 18 inches for confinement. The foundation for the proposed structure may consist of conventional strip footings founded entirely on undisturbed native granular soils or entirely on bedrock. If footing excavations expose combination soils or a combination of soils and bedrock, the foundation excavation should be over-excavated at least 12 inches to allow placement of a minimum of 12 inches of structural fill to limit the potential for differential settlement. Conventional strip and spread footings founded as described above may be proportioned for a maximum net allowable bearing capacity of **1,500 pounds per square foot (psf).**

Recommendations for general site grading, design of foundations, slabs-on-grade, moisture protection as well as other aspects of construction are included in this report.

<u>IMPORTANT INFORMATION ABOUT THIS GEOTECHNICAL-ENGIEERING REPORT:</u>

Do <u>not</u> rely on the executive summary. The executive summary omits a number of details, any one of which could be crucial. Read and refer to the report in full. Do <u>not</u> rely on this report if this report was prepared for a different client, different project, different purpose, different site, and/or before important events occurred at the site or adjacent to it. All recommendations in this report are confirmation dependent. A two-page document prepared by GBA explains these items with greater detail is found in Appendix E (Plates E-1 and E-2).

2.0 INTRODUCTION

2.1 PURPOSE AND SCOPE OF WORK

This report presents the results of a geotechnical investigation conducted for the proposed residential development to be constructed at 6599 North Mountain Road in Eden, Utah. The subject site has an area of approximately 0.68 acres. The purposes of this investigation are to provide an assessment of the nature and engineering properties of the subsurface soils at the subject site and to provide recommendations for general site grading and the design and construction of foundations, pavement sections, and slabs-on-grade. We have also performed a slope stability assessment of the proposed cut and fill plans as part of the development of the site.

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 and signed authorization, dated June 1, 2020. The recommendations contained in this report are subject to the limitations presented in the "Limitations" section of this report.

2.2 PROJECT DESCRIPTION

The project site is located at 6599 North Mountain Road in Eden, Utah (see Plate A-1, *Site Vicinity Map* and Plate A-2, *Site Topographic Map*). Information concerning the nature of the project was provided by the Client in the form of permit plans titled “Wingate Residence” prepared by Reeve and Associates, Inc (April, 2020). The proposed residences will consist of a 2 to 3 story, wood-framed structure, with basement (if feasible) founded on standard strip and spread footings. Our investigation for the development will be used to provide geotechnical design parameters for construction of buildings, pavement, and associated infrastructure.

3.0 METHOD OF STUDY

3.1 OFFICE INVESTIGATION

To prepare for the investigation, GeoStrata reviewed pertinent literature and maps listed in the references section of this report, which provided background information on the local geologic history of the area (Elliot and Harty, 2010; Coogan and King, 2016; Black and others, 2003). A stereographic aerial photograph interpretation was performed for the subject site using a set of stereo aerial photographs obtained from the UGS as shown in Table 1.

Source	Photo Number	Date	Scale
USFS	ELK_2-202	June 25, 1963	1:15,840
USFS	ELK_2-201	June 25, 1963	1: 15,840

Table 1: Aerial Stereosets.

GeoStrata also conducted a review of 2016 0.5-meter lidar provided by the State of Utah AGRC to assess the subject site for visible lineations related to landslide geomorphology that would indicate instability off the hillside in the area of proposed development. The digital elevation models were used to create hillshade imagery that could be reviewed for assessment of geomorphic features.

3.2 SUBSURFACE INVESTIGATION

As part of this geotechnical investigation, subsurface soil conditions were explored by excavating 2 exploratory test pits within the subject property. The test pits were advanced to depths ranging from 15 to 16 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-3 in Appendix A. Our exploration points were selected to provide a representative cross-section of the subsurface soils across the site. 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 to B-2 in Appendix B. A *Key to USCS Soil Symbols and Terminology* is presented on Plate B-3.

The test pits were excavated using a trackhoe. Bulk soil samples were obtained in each of the test pit explorations through the collection of bag and tube samples. 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) by the Geotechnical Engineer. Classifications for the individual soil units are shown on the attached Test Pit Logs.

3.3 LABORATORY TESTING

Geotechnical laboratory tests were conducted on 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 4318)
- Direct Shear Test (ASTM D3080)
- 1-D Collapse/Swell Test (ASTM D4546)
- 1-D Consolidation Test (ASTM D2435)

The results of laboratory tests are presented on the Test Pit Logs in Appendix B (Plates B-1 to B-2), the Laboratory Summary Table, and the test result plates presented in Appendix C (Plates C-1 to C-5).

3.4 ENGINEERING ANALYSIS

Our preliminary 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.

4.0 GEOLOGIC CONDITIONS

4.1 GEOLOGIC SETTING

The site is located in Eden, Utah at an elevation between approximately 8,180 to 8,250 feet above mean sea level in the mountains east of Ogden Valley. Outcroppings of Proterozoic age sedimentary bedrock indicative of a coastal environment with fluctuating sea levels are visible primarily in the northern and eastern portions of Ogden Valley. After their formation, these sedimentary beds were exposed to folding and uplift related to the collision between the North America and Farallon tectonic plates during what is referred to as the Sevier Orogeny which lasted from the Cretaceous to early Tertiary. The Willard Thrust fault, one of the largest faults in the Sevier mountain belt, bounds the western side of Ogden Valley. The Maple Canyon Thrust fault is also part of the Sevier Orogeny and is located trending northeast through Maple Canyon. Volcanism during the Tertiary gave rise to the deposition of Norwood Tuff which is prevalent in the southern portion of Ogden Valley and along knolls or foothills in the central portion of the valley. Transition from thrust faulting to Basin and Range extension occurred during the Cenozoic. As a result, the Ogden Valley is a northwest trending structural basin or fault graben flanked by two uplifted blocks, the Wasatch Range on the west and unnamed flat-topped mountains to the east (King and others 2008). The Wasatch Range is the easternmost expression of pronounced Basin and Range extension in north-central Utah (Stokes, 1986).

The near-surface geology of the Ogden Valley is dominated by lake sediments which were deposited within the last 30,000 years during the high stand of the Lake Bonneville Cycle when water inundated Ogden Canyon and formed a small lake in Ogden Valley (Scott and others, 1983; Hintze, 1993; Leggette and Taylor, 1937; King and others, 2008). 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 unnamed flat-topped mountains bounding the eastern margins of Ogden Valley. The eroded material was then 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 whereas sediments closer to the mountain fronts are shallow-water deposits of coarse sand and gravel. However, these deep-water deposits are in places covered by a thin post-Bonneville alluvial cover.

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4.2 REPORTED SITE GEOLOGY

The geology mapped as overlying the subject site, as reported on available geologic maps, are shown on Plate A-4, *Site Vicinity 30' x 60' Geologic Map* (Coogan and King, 2016). As shown on Plate A-4, Coogan and King (2016) delineates the geology within the subject site as Neoproterozoic Mutual Formation (Zm) bedrock. The Mutual Formation (Zm) is described by Coogan and King (2016) as grayish-red to purplish-gray, medium to thick-bedded quartzite with pebble conglomerate lenses. Middle Cambrian Ute Formation (Cu) is mapped southeast of the subject site and Middle and Lower Cambrian or possibly Neoproterozoic Geertsen Canyon Quartzite (Cgc) is mapped southwest of the subject site. North and northeast trending thrust faults related to the Sevier Orogeny are mapped north and south of the subject site and mark the contact boundary between the above bedrock units. The Ute Formation is described as gray thin- to thick-bedded limestone with tan-,yellowish-tan-, and reddish-tan-weathering, wavy, silty layers and partings, and olive-gray to tan-gray, thin-bedded shale and micaceous argillite. The Geertsen Canyon Quartzite is described as an off-white and tan quartzite with pebble conglomerate beds.

4.3 TECTONIC SETTING AND SEISMICITY

The subject site is located in the Powder Mountain Ski Resort at 6599 North Mountain Road in Eden, Utah. The nearest active fault (Holocene age) is the north trending and west dipping Weber Section of the Wasatch Fault Zone (WFZ) which is located approximately 8½ miles east of the subject site. The Weber segment extends for about 35 miles from its southern terminus to its northern terminus (Nelson and Personius, 1993). 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). Prior paleoseismic studies report that the Weber segment of the WFZ is thought to have experienced four surface faulting seismic events since the middle Holocene. Nelson and others (2006) report four surface faulting seismic events since the middle Holocene with the most recent event being a partial segment rupture which occurred approximately 500 years ago resulting in a 1.6 feet surface rupture displacement. DuRoss and others (2009) report evidence from the 2007 Rice Creek trench site of as many as six surface faulting seismic events during the Holocene with four surface faulting events in approximately the past 5,400 years. This data from DuRoss and others (2009) supports the partial segment surface rupture timing reported by Nelson and others (2006). A location near Kaysville, Utah indicated that the Weber Segment has a measurable offset of 1.4 to 3.4 meters per event (McCalpin and others, 1994). The Weber Segment may be capable of producing earthquakes as large as magnitude 7.5 (Ms). The consensus preferred recurrence interval for the Weber segment, determined by the Utah Quaternary Fault Working Group, is approximately 1,400 years for the past four surface fault rupture earthquakes (Lund, 2005).

Spectral responses for the Risk-Targeted Maximum Considered Earthquake (MCE_R) are shown in the table below. These values generally correspond to a one percent probability of structure collapse in 50 years 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 experience and field exploration to 16 feet, it is our opinion that this location is best described as a Site Class C. Due to the size of the potential structures we have assumed that the structures have a fundamental period of vibration less than 0.5 seconds. According to the exception in ASCE 7-16 Section 20.3 the site would be classified as a Site Class C (very dense soil and soft rock). The spectral accelerations are calculated based on the site's approximate latitude and longitude of 41.3793° and -111.7853° respectively and the Seismic Design Maps web-based application at https://seismicmaps.org/.

Description	Value
S_s - MCE_R ground motion (period – 0.2s)	0.856
S_1 - MCE_R ground motion (period – 1.0s)	0.298
F_a - Site amplification factor at 1.0s	1.200
F_v - Site amplification factor at 1.0s	1.500
PGA - MCE_G peak ground acceleration	0.374
PGA_M – Site modified peak ground acceleration	0.449

It should be noted that our investigation did not include a site-specific ground motion hazards assessment. Based on geologic mapping and observations within the test pits excavated at the site, a site class C has been assigned to the near surface soils in this report. Due to the S_1 values exceeding 0.2g, the structural engineer will need to take an exception as per Section 11.4.8 of the ASCE 7-16. As an alternative, GeoStrata may be contacted to complete a ground motion hazards analysis for the subject site as per Chapter 21 of the ASCE 7-16.

5.0 GENERALIZED SITE CONDITIONS

5.1 SURFACE CONDITIONS

At the time of our subsurface investigation, the property existed as an undeveloped parcel on a steep native hillside in the Powder Mountain Ski Resort located in Ede, Utah. Minor disturbance due to placement of water utilities was observed through the central portion of the subject site. No permanent above-ground structures were observed at the subject site. The property is bordered to the north and east by established single-family residences, an undeveloped single-family residential parcel to the east and by North Mountain Road to the south. Dense mature scrub oak and large brush was observed to cover much of the northeast corner of the subject site while the remainder of the parcel contained native grasses. Mutual Formation (Zm) quartzite bedrock outcroppings were observed in the southeast corner of the subject site.

5.2 SUBSURFACE CONDITIONS

As mentioned previously, the subsurface soil conditions were explored at the subject property by excavating two test pits to depths ranging from 15 to 16 feet below the existing site grade. Subsurface soil conditions were logged during our field investigation and are included on the Test Pit Logs in Appendix B (Plates B-1 to B-2). The soil and moisture conditions encountered during our investigation are discussed below.

5.2.1 Soils

Based on our field observations, the site is overlain by approximately 0- to ½-ft of undocumented fill composed of silt and gravel. The undocumented fill is underlain by approximately 1 foot of topsoil composed of silt, clay and gravel. Based on our geologic review of the site and our subsurface investigation, the topsoil is underlain by Neoproterozoic Mutual Formation (Zm) as shown on Plate A-4, *Site Vicinity 30' x 60' Geologic Map*. Descriptions of the soil units encountered are described below:

<u>Undocumented Fill:</u> Where observed, the undocumented fill consisted of light brown, slightly moist, Silty GRAVEL (GM). Undocumented fill was encountered only in TP-2 advanced as part of this investigation and is anticipated to thinly overlie the western portions of the site and outside of the proposed building footprint.

Topsoil: Where observed, the topsoil consisted of dark brown, moist, SILT (ML) with clay and gravel. This unit also has an organic appearance and texture, with numerous roots throughout. Topsoil was encountered in each of the test pits advanced as part of this investigation and is anticipated to overlie the majority of the site.

Neoproterozoic Quartzite of the Mutual Formation (Zm): Where observed, these soils consisted of a completely weathered bedrock that weathers to coarse-grained soils. These sediments consisted of a very dense, moist, strong brown Silty Clayey GRAVEL (GC-GM) and Silty Clayey SAND (SC-SM) with varying amounts of gravel. These coarse-grained sediments consisted of subangular clasts ranging from pea sized to a maximum observed gravel diameter of approximately 11-inches.

Neoproterozoic Argillite of the Mutual Formation (Zm): Where observed, these soils consisted of a completely weathered bedrock that weathers to coarse-grained soils. These sediments consisted of a very dense, moist, strong brown to light strong brown with white and black mottling Silty SAND (SM).

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.

5.2.2 Groundwater Conditions

Groundwater was encountered in each test pits advanced as part of this investigation at a depth of approximately 11 feet in TP-1 and 7½ feet in TP-2. It is our opinion that based on the elevation of the subject site and location, the observed groundwater levels are related to snowmelt and seasonal runoff. Foundation drains are recommended as part of the construction of the proposed single-family residence and will be discussed in more detail in Section 6.6 of this report.

5.2.3 Swell Potential

Swelling soils, also known as expansive soils, are undisturbed soils that exhibit volumetric strain and expansion upon wetting under increased loading conditions. Swelling soils can cause differential settling of structures and roadways. Swelling soils do not necessarily preclude development and can be mitigated by restricting the introduction of landscape water to the site, engineering proper site drainage design to remove surface water by unimpeded surface runoff to limit surface water infiltration, and over-excavation of swelling soils and replacement of the over excavated soils with properly placed and compacted structural fill approved by the project geotechnical engineer. For some structures that are particularly sensitive to differential settlement, or in areas where swelling soils are identified to great depth, a deep foundation system should be considered.

Soils that have a potential to swell under increased moisture conditions are typically characterized by a high liquid limit and plasticity index. In general, potentially expansive soils are observed in elastic silts and fat clays. Some potentially swelling soils were encountered at the site and classify as a Sandy Silty CLAY (CL). Free swell tests were performed on the Sandy Silty CLAY (CL) soils encountered at the subject site. Results indicate a low swell potential of 0.03%.

5.3 INTERPRETATION OF SUBSURFACE CONDITIONS

Based on our review of published geologic maps, aerial stereosets, hillshades derived from 2016 0.5-meter lidar, our subsurface investigation, and our field observations, GeoStrata has compiled a *Site Specific Geologic Map*, Plate A-5.

Based on the test pits excavated as part of this study, published geologic maps, field observations and review of hillshades derived from 2016 0.5-meter lidar, it is our opinion that the subject site is underlain by completely weathered quartzite and argillite of the Mutual Formation (Zm) bedrock. The Mutual Formation (Zm) is described by Coogan and King (2016) as grayish-red to purplish-gray, medium to thick-bedded quartzite with pebble conglomerate lenses and contains medial argillite in the James Peak Quadrangle. Based on our subsurface investigation, the completely weathered quartzite unit is up to 10 feet thick and was observed to overlie the completely weathered argillite unit. The completely weathered argillite unit was observed to extend the remainder of the depth of the exploratory test pits.

5.4 STRENGTH OF EARTH MATERIALS

Direct shear tests were completed on remolded samples of the weathered bedrock soils observed within in the test pits. Due to the granular nature of the soil sampled, it was not feasible to obtain a relatively “undisturbed” sample of this deposit. The size of the aggregate exceeds the limits of our direct shear equipment, so the sample was screened, and the larger aggregate were removed

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to run the remolded direct shear tests. The results of our direct shear tests are summarized in the table below.

Sample Location	Depth (ft)	Friction Angle (phi) (degrees)	Cohesion (psf)
TP-1	7	36	280
TP-1	12.5	32	30
TP-2	15.5	30	445

Based on the results of our laboratory strength testing of the weathered bedrock units, the table below summarizes the assigned strength of earth materials.

Material	Friction Angle (phi) (degrees)	Cohesion (psf)
Quartzite of the Mutual Formation (Zm)	36	140
Argillite of the Mutual Formation (Zm)	30	200

6.0 ENGINEERING ANALYSIS AND RECOMMENDATIONS

6.1 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.

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 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, concrete flatwork, or pavement sections), any existing vegetation, debris, undocumented fill, or otherwise unsuitable soils should be removed. Any soft, loose, or disturbed 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 our site investigation, there is approximately 0 to 6 inches of undocumented fill and 12 inches of organic topsoil overlying the site. This material should be removed prior to placement of structural fill, structures, concrete flatwork, and roadways. If over-excavation is required, the excavation should extend a minimum of one foot laterally for every foot of depth of over-excavation. Excavations should extend laterally at least two feet beyond flatwork, pavements, and slabs-on-grade. If materials are encountered that are not represented in the test pit logs or may present a concern, GeoStrata should be notified so observations and further recommendations as required can be made.

A GeoStrata representative should observe the site preparation and grading operations to assess that the recommendations presented in this report are complied with.

6.2.2 Soft Soil Stabilization

Soft or pumping soils may be exposed in excavations at the site. Once exposed, all subgrade surfaces beneath proposed structure, pavements, and flat work concrete should be proof rolled with a piece of 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 non-woven geotextile fabric against the soft soils covered by a geogrid and 12 inches of granular structural fill meeting requirements of <u>Section 6.2.4</u> below. The geogrid should consist of Tensar TX130S or prior approved equivalent. The filter fabric should consist of Tencate Mirafi 140N or equivalent as approved by the Geotechnical Engineer.

6.2.3 Excavation Stability

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

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

6.2.4 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 a reworked native, granular soil provided it is first screened for debris, vegetation, and clasts exceeding 4 inches in maximum diameter. Alternatively, an imported fill meeting the specifications below may be used. Imported structural fill should 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. 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). Regardless if the structural fill is imported or native, it should be free of vegetation, debris or frozen material, and should contain no inert materials larger than 4 inches nominal size. All structural fill soils should be approved by the Geotechnical Engineer prior to placement. 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 sections up to 4-feet in thickness should be compacted to at least 95% of the maximum dry density (MDD), as determined by ASTM D-1557. Structural fill in excess of 4-feet in thickness should be compacted to at least 98% of the MDD (ASTM D-1557). If structural fill is required beneath footings, for maximum fill sections of 3-ft or less, we recommend that at least 1-ft of structural fill be placed beneath all footings. If maximum structural fill sections exceed 3-ft but are less than 5-ft, we recommend that at least 2-ft of structural fill be placed beneath all footings. If maximum structural fill sections exceed 5-ft, we recommend that at least 3-ft of structural fill be placed beneath all footings. The moisture content should be at or slightly above the optimum moisture content (OMC) at the time of placement and compaction. Also, prior to placing any fill, the excavations should be observed by the geotechnical engineer to observe that any unsuitable materials or loose soils have been removed. In addition, proper grading should precede placement of fill, as described in the **General Site Preparation and Grading** subsection of this report (Section 6.2.1).

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

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

6.3 FOUNDATIONS

The foundations for the proposed structures may consist of conventional strip and/or spread footings. Strip and spread footings should be a minimum of 20 and 36 inches wide, respectively, and exterior shallow footings should be embedded at least 48 inches below final grade for frost protection and confinement. Interior shallow footings not susceptible to frost conditions should be embedded at least 18 inches for confinement.

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6.3.1 Installation and Bearing Material

The foundation for the proposed structure may consist of conventional strip footings founded entirely on undisturbed native granular soils or entirely on bedrock. If footing excavations expose combination soils or a combination of soils and bedrock, the foundation excavation should be over-excavated at least 12 inches to allow placement of a minimum of 12 inches of structural fill to limit the potential for differential settlement. Strip footings should be a minimum of 20-inches wide and exterior shallow footings should be embedded at least 48-inches below final grade for frost protection and confinement. Interior footings not subject to frost should be embedded at least 18 inches below final grade to provide confinement. To provide adequate support and confinement, we recommend that footings be place at least 15 feet, measured horizontally, from the face of existing or fill slopes at the site.

Soft or pumping soils may be exposed in foundation excavations due to presence of perched groundwater and the high fines content of some of the granular soils observed in our test pits. Where soft or pumping soils are exposed, prior to placement of foundations, the soft or pumping soils should be stabilized (See Section 6.2.2).

All organic material, soft areas, frozen material or other inappropriate material shall be removed from the footing zone to a depth determined by the Geotechnical Engineer and be replaced with structural fill where over excavation is required.

6.3.2 Bearing Pressure

Conventional strip and spread footings founded as described above may be proportioned for a maximum net allowable bearing capacity of **1,500 pounds per square foot (psf).** The recommended net allowable bearing pressure refers to the total dead load and can be increased by 1/3 to include the sum of all loads including wind and seismic.

6.3.3 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.

6.3.4 Frost Depth

All exterior footings are to be constructed at least 48 inches below the ground surface for frost protection and confinement. This includes walk-out areas and may require fill to be placed around buildings. Interior footings not susceptible to frost conditions should be embedded at least 18 inches for confinement. If foundations are constructed through the winter months, all soils on which footings will bear shall be protected from freezing.

6.3.5 Construction Observation

A geotechnical engineer shall periodically monitor excavations prior to installation of footings. Inspection of soil before placement of structural fill or concrete is required to detect any field conditions not encountered in the investigation which would alter the recommendations of this report. All structural fill material shall be tested under the direction of a geotechnical engineer for material and compaction requirements.

6.3.6 Foundation Drainage

Due to the observed perched groundwater and the possibility of moisture reaching the foundation elements during spring runoff, it is recommended that a foundation drain be constructed around any subgrade walls. The foundation drain should consist of a 4-inch perforated pipe placed at or below the footing elevation. The pipe should be covered with at least 12 inches of free draining gravel (containing less than 5 percent passing the No 4 sieve) and be graded to a free gravity out fall or to a pumped sump. A separator fabric, such as Mirafi 140N, should separate the free draining gravel and native soil (i.e. the separator fabric should be placed between the gravel and the native soils at the bottom of the gravel, the side of the gravel where the gravel does not lie against the concrete footing or foundation and at the top of the gravel). We recommend that the gravel extend up the foundation wall to within 3 feet of the final ground surface. As an alternative, the gravel extending up the foundation wall may be replaced with a prefabricated drain panel, such as Ecodrain-E.

6.4 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 subgrade. In determining the frictional resistance, a coefficient of friction of 0.42 should be used for granular native soils, structural fill or drain gravel 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*	0.28	34
At-rest**	0.47	59
Passive*	7.33	917
Seismic Active***	0.51	64
Seismic Passive***	-2.74	-343

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

These coefficients and densities assume level 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 CONCRETE SLAB-ON-GRADE CONSTRUCTION

Concrete slabs-on-grade should be constructed over at least 4 inches of compacted gravel overlying undisturbed native soil or a zone of structural fill that is at least 12 inches thick. Disturbed native soils should be compacted to at least 95% of the MDD as determined by ASTM D-1557 (modified proctor) prior to placement of gravel. The gravel should consist of roadbase or clean drain rock with a ¾-inch maximum particle size and no more than 12 percent fines passing the No. 200 mesh sieve. The gravel layer should be compacted to at least 95 percent of the MDD of modified proctor or until tight and relatively unyielding if the material is non-proctorable. The maximum load on the floor slab should not exceed 300 psf; greater loads would require additional subgrade preparation and additional structural fill. All concrete slabs should be designed to minimize cracking as a result of shrinkage. Consideration should be given to reinforcing the slab with welded wire, re-bar, or fiber mesh.

6.6 GLOBAL STABILITY OF NATURAL SLOPES

The global slope stability of the proposed construction was modeled using the SLIDE computer application and the Bishop's Simplified Method of analysis. The slope stability profiles has been identified as Section A-A' on Plate A-3. A geologic cross section of the subsurface soils was prepared by a licensed geologist and is included as Plate A-6. Calculations for stability were developed by searching for the minimum factor of safety for a circular-type failure. Homogenous earth materials and arcuate failure surfaces were assumed. Topographic information for the profile was obtained using the provided grading plan for the proposed construction prepared by Reeve & Associates (dated August 2019).

Slope stability analysis was performed for both the static and pseudo-static (seismic) conditions. The pseudo-static assessment was completed utilizing the peak ground acceleration (PGA) associated with a 2 percent chance of exceedance in 50 years. A seismic coefficient based on seismic design parameters for the site (IBC, 2018) was utilized in our analysis (see Section 4.3).

Strength parameters for the soils located at the subject property were obtained utilizing the results of laboratory direct shear testing and literature review completed as part of this investigation as discussed in Section 5.4 of this report.

Perched groundwater was encountered in both test pits excavated as part of this investigation at depths ranging from 7.5 to 11 feet below the existing site grade. As such, it is the opinion of GeoStrata that the moisture observed within the test pits are the result of seasonal perched groundwater that is perched on top of the weathered bedrock. Our model has accounted for this anticipated perched groundwater by modeling a piezometric surface that is applied to the soils above the residual bedrock-bedrock interface.

The results of our slope stability investigation are as follows:

Cross Section A-A' Factor of Safety			
Profile	Failure Type	Static	Pseudo Static
A-A'	Circular	1.658	1.154

Based on the results of our slope stability analysis described above, Cross Section A-A' meets the industry standard of care recommended factors of safety of 1.5 and 1.0 for static and pseudo-static conditions, respectively. This result indicates that the proposed building pad is anticipated to be relatively stable under both static and pseudo static conditions.

6.7 NEAR SURFACE STABILITY OF NATURAL SLOPES

Based on our field observations and experience in assessing alpine slopes, GeoStrata has completed an infinite slope analysis in order to assess the stability of the near-surface soils when saturated to the depth of the bottom of the perched groundwater. Our assessment has been completed on the steepest portion of the lot, where grades up 2H:1V have been observed. Our analysis has been completed using the methodology outlined by Das (1988), and utilizes the following parameters;

Cohesion, c (psf)	Saturated Unit Weight, γsat (pcf)	Height (thickness) of slope, H (feet)	Slope angle, β (degrees)	Internal Angle of Friction ϕ (degrees)	Calculated Factor of Safety
140	120	11	26.6	36	1.72

Based on our infinite slope modeling, the site has a factor of safety against near-surface slope stability of 1.72 and is therefore considered stable.

6.8 MOISTURE PROTECTION AND SURFACE DRAINAGE

Moisture should not be allowed to infiltrate the soils in the vicinity of the foundations. We recommend the following mitigation measures be implemented at the building location.

- The ground surface within 10 feet of the entire perimeter of the building should slope a minimum of five percent away from the structure. Alternatively, a slope of 5% is acceptable if the water is conveyed to a concrete ditch that will convey the water to a point of discharge that is at least 10 feet from the structures.
- Roof runoff devices (rain gutters) should be installed to direct all runoff a minimum of 10 feet away from the structure and preferably day-lighted to the curb where it can be transferred to the storm drain system. Rain gutters discharging roof runoff adjacent to or within the near vicinity of the structure may result in excessive differential settlement.
- We do not recommend storm drain collection sumps be used as part of this development. However, if necessary, sumps should not be located adjacent to foundations or within roadway pavements
- We recommend irrigation around foundations be minimized by selective landscaping and that irrigation valves be constructed at least 5 feet away from foundations.
- Jetting (injecting water beneath the surface) to compact backfill against foundation soils may result in excessive settlement beneath the building and is not allowed.
- Backfill against foundations walls may consist of on-site native soils and should be placed in lifts and compacted to 90% modified proctor to create a moisture barrier.

Failure to comply with these recommendations could result in excessive total and differential settlements causing structural damage.

7.0 CLOSURE

7.1 LIMITATIONS

The conclusions and recommendations contained in this report, which include professional opinions and judgments, are based on are based on the information available to us at the time of our evaluation, our limited field exploration, laboratory testing, and understanding of the proposed site development. 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 and beyond 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, GeoStrata 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 warranty, expressed or implied, is made.

All services were performed for the exclusive use and benefit of the above addressee. No other person or entity is entitled to rely on GeoStrata's services or use the information contained in this letter without the express written consent of GeoStrata. The above addressee is not entitled to transfer their rights to use this report to any other person or entity without the express written consent of GeoStrata. We are not responsible for the technical interpretations by others of the information described or documented in this report. It is the Client's responsibility to see that all parties to the project including the Designer, Contractor, Subcontractors, etc. are made aware of this report in its entirety. The use of information contained in this report for bidding purposes should be done at the Contractor's option and risk.

7.2 ADDITIONAL SERVICES

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

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

We also recommend that project plans and specifications be reviewed by GeoStrata 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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Appendix A

Legend

Approximate Site Boundary

0 1,500 3,000 6,000 9,000 12,000 Feet

1 inch = 4,000 feet

Base Map:
2009 1 meter NAIP aerial imagery and hillshades derived from digital elevation model (DEM) provided by the State of Utah AGRC.

N

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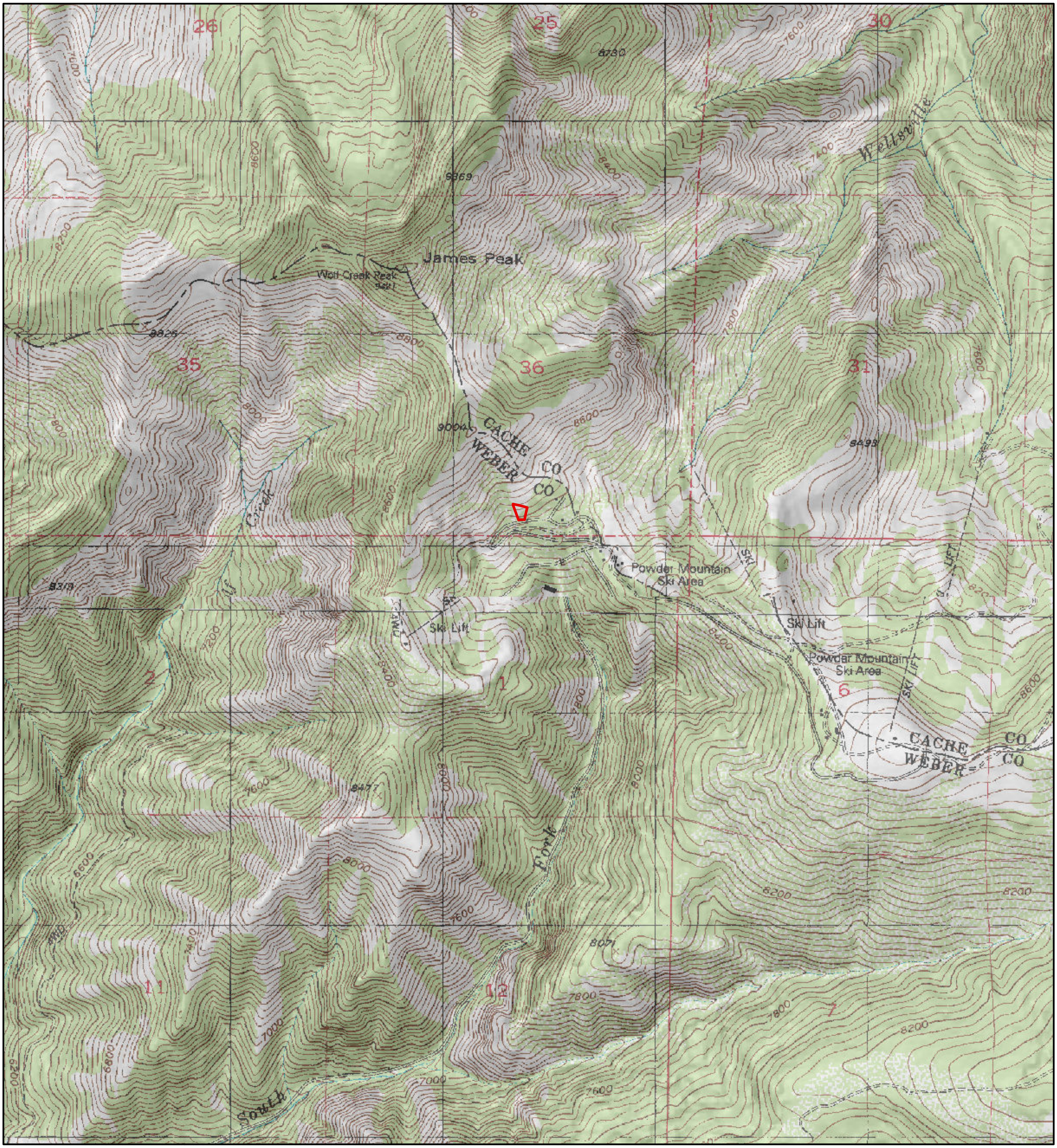
Geotechnical Investigation
Lot 3R Powder 11 at Powder Mountain
6599 North Powder Mountain Road
Eden, Utah
Project Number: 1174-006

Site Vicinity Map

Plate A-1



Legend

Approximate Site Boundary

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

N

1 inch = 2,000 feet

Base Map:
Browns Hole, Huntsville, James Peak and Sharp Mountain Quadrangles, Utah 7.5 Minute Series (Topographic). USGS and hillshades derived from 5 meter DEM provided by the State of Utah AGRC.

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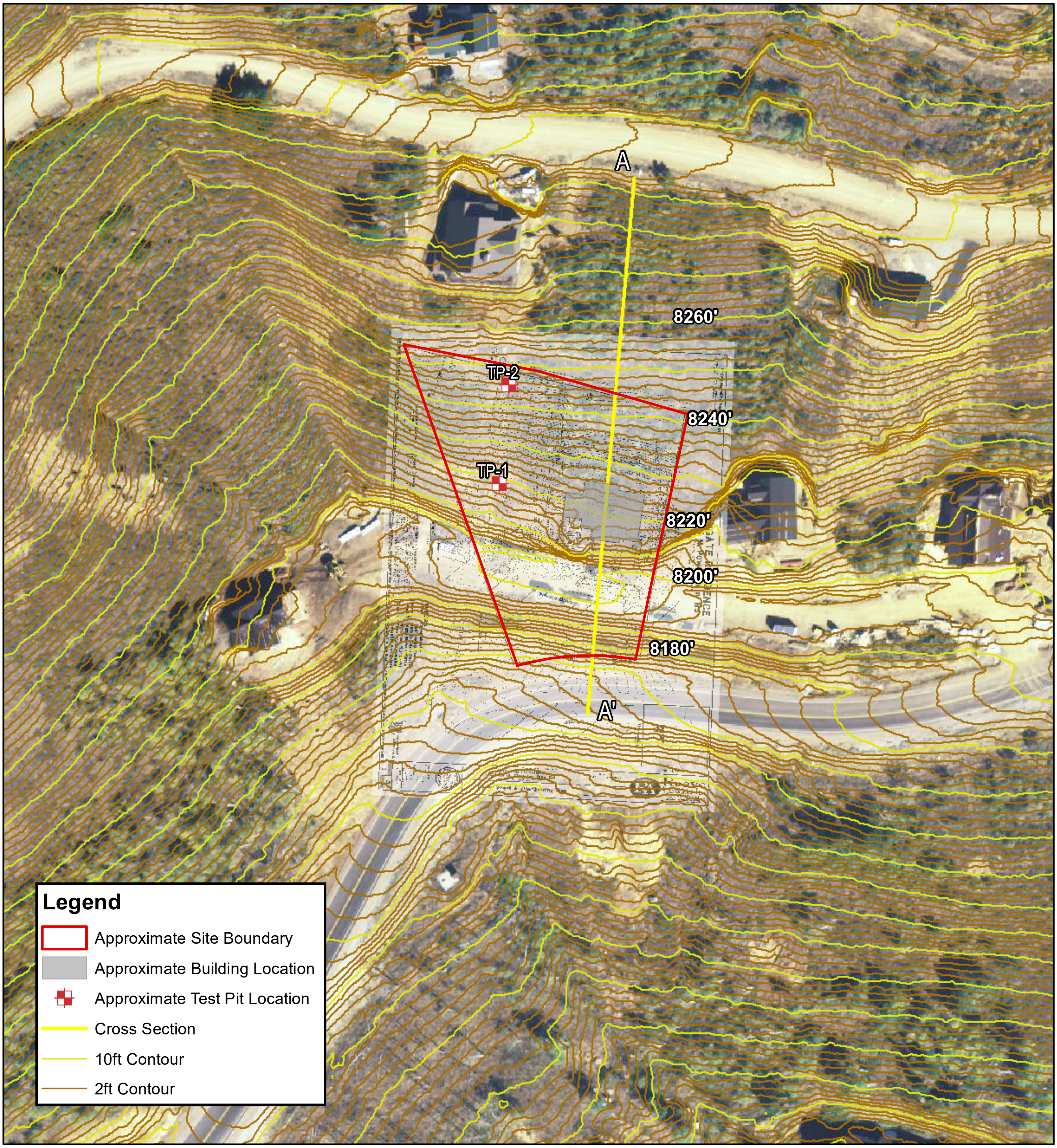
Eden, Utah
Project Number: 1174-006

Site Vicinity Map

Plate A-2



0 35 70 140 210 280 Feet

1 inch = 100 feet

Base Map:
Hillshades derived from 2016 and 2018 0.5 meter LiDAR
provided by the State of Utah AGRC.

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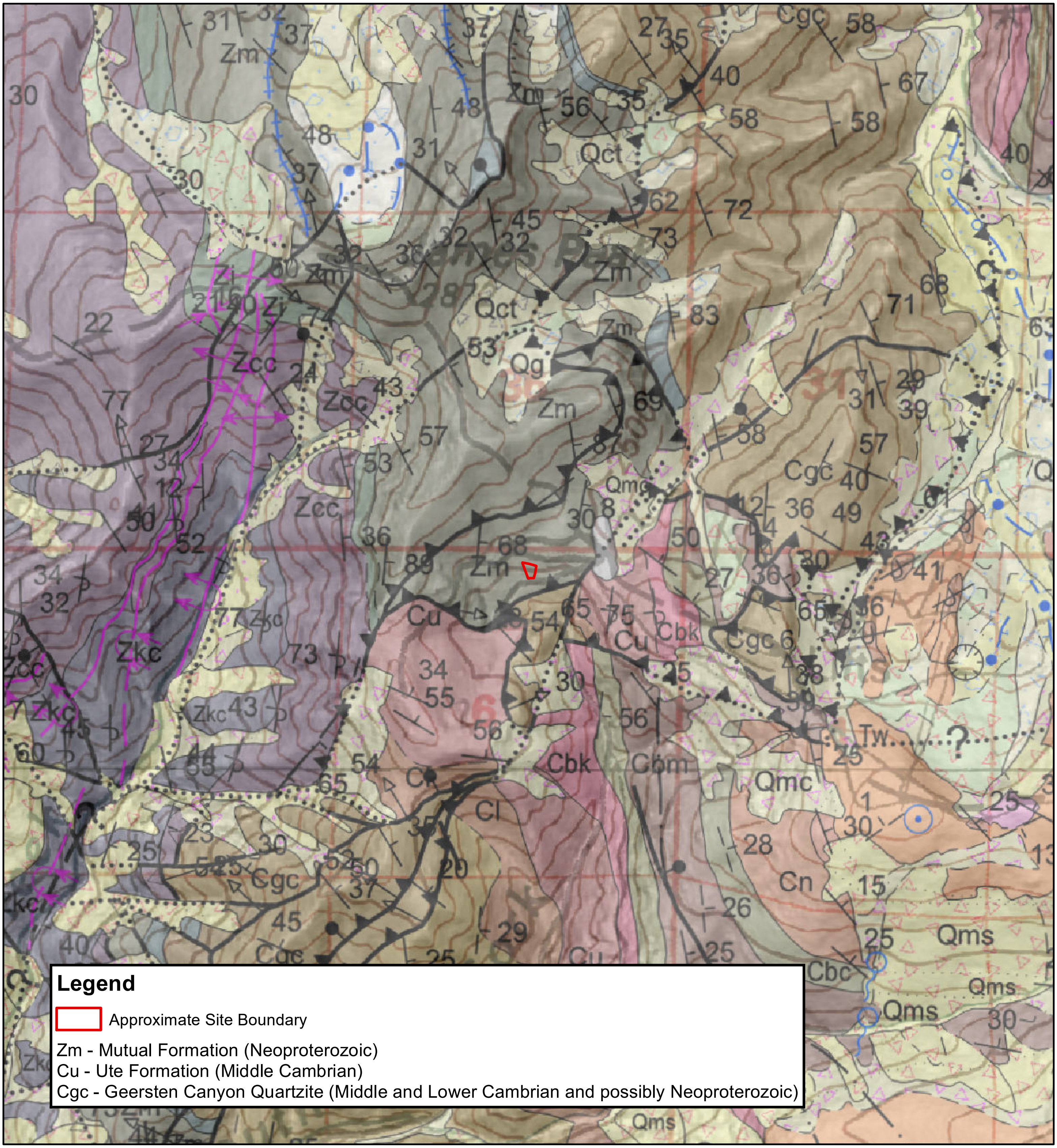
Eden, Utah
Project Number: 1174-006

Exploration Location Map

Plate A-3

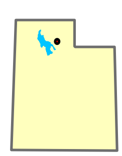


0 750 1,500 3,000 4,500 6,000 Feet

1 inch = 2,000 feet

Base Map:
Interim Geologic Map of the Ogden 30' x 60' Quadrangle, Box Elder, Cache, Davis, Morgan, Rich, and Summit Counties, Utah, and Uinta County, Wyoming, Coogan and King, 2016. Hillshades derived from digital elevation models (DEM) provided by the State of Utah AGRC.

N



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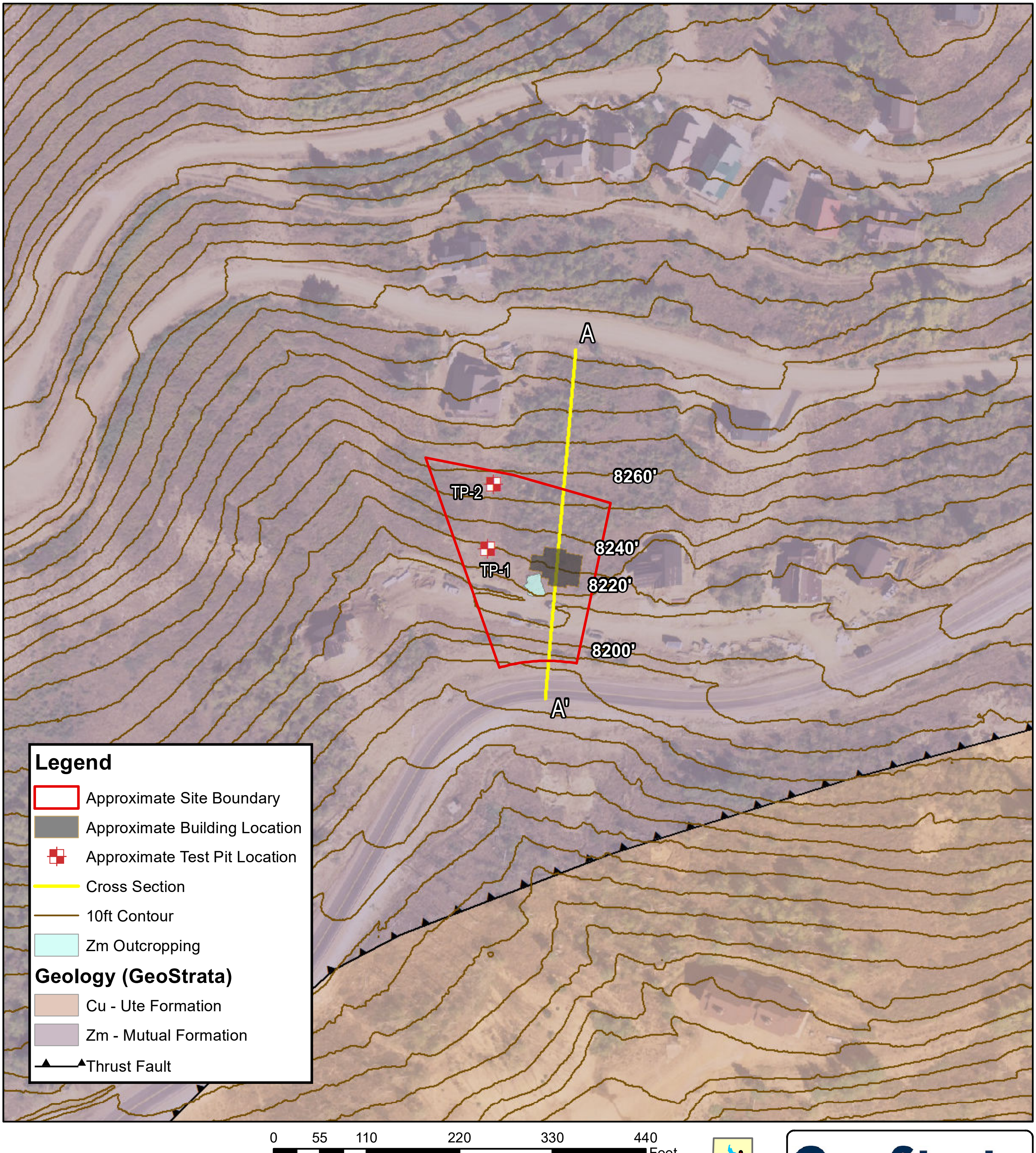
Lot 3R Powder 11 at Powder Mountain
6599 North Powder Mountain Road
Eden, Utah
Project Number: 1174-006

Site Vicinity 30x60 Geologic Map

Plate
A-4



1 inch = 150 feet

Base Map:
Interim Geologic Map of the Ogden 30' x 60' Quadrangle, Box Elder, Cache, Davis, Morgan, Rich, and Summit Counties, Utah, and Uinta County, Wyoming, Coogan and King, 2016. Hillshades derived from digital elevation models (DEM) provided by the State of Utah AGRC.

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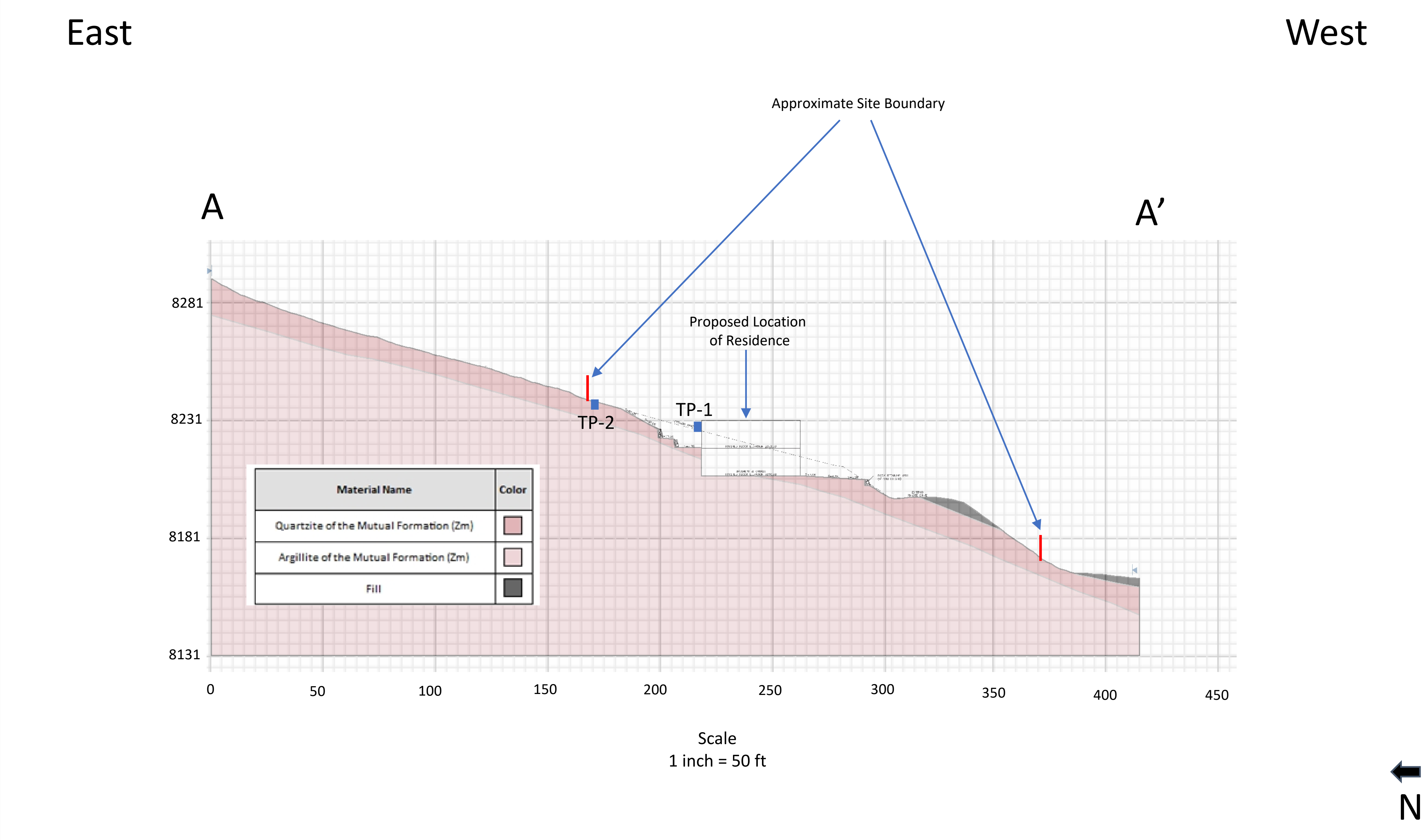
Geotechnical Investigation
Lot 3R Powder 11 at Powder Mountain
3599 North Mountain Road
Eden, Utah
Project Number: 1174-006

Site Specific Geologic Map

Plate A-5



Geotechnical Investigation
Lot 3R Powder Mountain
6599 North Powder Mountain Road
Eden, Utah
Project Number: 1174-006

Geologic Cross-Section A-A’

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Plate
A-6

Appendix B

DATE			
STARTED:	6/5/20	Lot 3R Powder Mountain 6599 North Powder Mountain Road Eden, Utah	GeoStrata Rep: S. Agopian
COMPLETED:	6/5/20		Rig Type: trackhoe
BACKFILLED:	6/5/20	Project Number 1174-006	

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
0	0					TOPSOIL; clay, silt, gravel, moist, dark brown, numerous fine roots					
					GC-GM	Quartzite of the Mutual Formation Bedrock (Zm) - completely weathers to a Silty Clayey GRAVEL with sand - moist, very dense, yellow-brown, subangular cobbles up to 11 inches in diameter					
1											
	5										
2									20.7	25	5
3	10					- large fractured bedrock block in a matrix of clay					
			▼		SM	Argillite of the Mutual Formation (Zm) - completely weather to a Silty SAND - moist, very dense, light strong brown with white mottling					
4								12.5	46.3	34	5
	15					Bottom of Test Pit @ 15 Feet					
5											
6											

Moisture Content and Atterberg Limits

Plastic Limit | Moisture Content | Liquid Limit

10 20 30 40 50 60 70 80 90

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

WATER LEVEL
▼- MEASURED
▽- ESTIMATED

NOTES:

Plate

B-1

DATE		
STARTED:	6/5/20	
COMPLETED:	6/5/20	
BACKFILLED:	6/5/20	

Lot 3R Powder Mountain
6599 North Powder Mountain Road
Eden, Utah
Project Number 1174-006

GeoStrata Rep: S. Agopian
Rig Type: trackhoe

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

LOCATION
NORTHING EASTING ELEVATION

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

DEPTH (METERS)	DEPTH (FEET)	UNIFIED SOIL CLASSIFICATION	MATERIAL DESCRIPTION	Dry Density (pcf)	Moisture Content %	Percent minus 200	Liquid Limit	Plasticity Index
0	0		FILL; clay, silt and gravel					
			TOPSOIL; clay, silt, gravel, moist, dark brown, numerous fine roots					
		GC-GM	Quartzite of the Mutual Formation (Zm) - completely weathers to a Silty Clayey GRAVEL - moist, very dense, strong brown					
		SC-SM	Quartzite of the Mutual Formation (Zm) - completely weathers to a Silty Clayey SAND - moist, very dense, strong brown, some gravel					
1	5							
2				123.7 124.2	13.7 12.0	31.9	24	4
		SM	Argillite of the Mutual Formation (Zm) - completely weathers to a Silty SAND - moist, very dense, strong brown, water observed to be running along this contact					
3	10							
					19.0	42.8	33	7
4	15							
			- sandy, strong brown to orange brown with black mottling		17.3	40.0	31	4
5			Bottom of Test Pit @ 16 Feet					
6								

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

WATER LEVEL
- MEASURED
- ESTIMATED

NOTES:

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Plate B-2

UNIFIED SOIL CLASSIFICATION SYSTEM

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

LOG KEY SYMBOLS

BORING SAMPLE LOCATION

TEST-PIT SAMPLE LOCATION

WATER LEVEL (level after completion)

WATER LEVEL (level where first encountered)

CEMENTATION

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

OTHER TESTS KEY

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

MODIFIERS

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

MOISTURE CONTENT

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

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.

STRATIFICATION

DESCRIPTION	THICKNESS
SEAM	1/16 - 1/2"
LAYER	1/2 - 12"

DESCRIPTION	THICKNESS
OCCASIONAL	ONE OR LESS PER FOOT OF THICKNESS
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

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

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Soil Symbols Description Key

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Plate B-3

Bedding of Sedimentary Rocks

Splitting Property	Thickness	Stratification
Massive	>4.0 ft	Very thick bedded
Blocky	2.0-4.0 ft	Thick-bedded
Slabby	2 ½-24 in	Thin-bedded
Flaggy	½-2 ½ in	Very thin-bedded
Shaly or platy	⅛ – ½ in	Laminated
Papery	< ⅛ in	Thinly laminated

Rock Classification Should Include:
1. Rock name (or classification) 2. Color 3. Weathering 4. Fracturing 5. Competency 6. Additional comments indicating rock characteristics which might affect engineering properties

Weathering

Weathering	Field Test
Fresh	No visible sign of decomposition or discoloration. Rings under hammer impact.
Slightly Weathered	Slight discoloration inwards from open fractures, otherwise similar to Fresh.
Moderately Weathered	Discoloration throughout. Weaker minerals such as feldspar are decomposed. Strength somewhat less than fresh rock but cores cannot be broken by hand or scraped with a knife. Texture preserved.
Highly Weathered	Most minerals somewhat decomposed. Specimens can be broken by hand with effort or shaved with a knife. Core stones present in rock mass. Texture becoming indistinct but fabric preserved.
Completely Weathered	Minerals decomposed to soil but fabric and structure preserved. Specimens easily crumble or penetrated.

Fracturing

Spacing	Description
>6 ft	Very Widely
2-6 ft	Widely
8-24 in	Moderately
2 ½-8 in	Closely
¾-2 ½ in	Very Closely

Competency

Class	Strength	Field Test	Approximate Range of Unconfined Compressive Strength (tsf)
I	Extremely Strong	Many blows with geologic hammer required to break intact specimen.	>2000
II	Very Strong	Hand-held specimen breaks with pick end of hammer under more than one blow.	2000-1000
III	Strong	Cannot by scraped or peeled with knife, hand-held specimen can be broken with single moderate blow with pick end of hammer	1000-500
IV	Moderately Strong	Can just be scraped or peeled with knife. Indentations 1-3 mm show in specimen with moderate blow with pick end of hammer.	500-250
V	Weak	Material crumbles under moderate blow with pick end of hammer and can be peeled with a knife, but is hard to hand-trim for triaxial test specimen.	250-10
VI	Friable	Material crumbles in hand.	N/A

RQD

RQD (%)	Rock Quality
90-100	Excellent
75-90	Good
50-75	Fair
25-50	Poor
0-25	Very Poor

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Physical Rock Properties Key

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Plate B-4

Appendix C

Test Pit No.	Sample Depth (feet)	USCS Soil Classification	Natural Moisture Content (%)	Natural Dry Density (pcf)	Gradation			Atterberg		Consolidation Test			Collapse (%)	Direct Shear	
					Gravel (%)	Sand (%)	Fines (%)	LL	PI	C_c	C_r	OCR		Friction Angle (°)	Apparent Cohesion (psf)
TP-1	7	GC-GM	9.9		59.8	19.5	20.7	25	5					36	280
TP-1	12.5	SM	12.5		53.7		46.3	34	5					32	30
TP-2	6	SC-SM	13	124.2	68.1		31.9	24	4	0.300	0.005	2.4	0.03		
TP-2	12	SM	19		57.2		42.8	33	7						
TP-2	15.5	SM	17		60.0		40.0	31	4					30	445

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Lab Summary Report

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6599 North Powder Mountain Road
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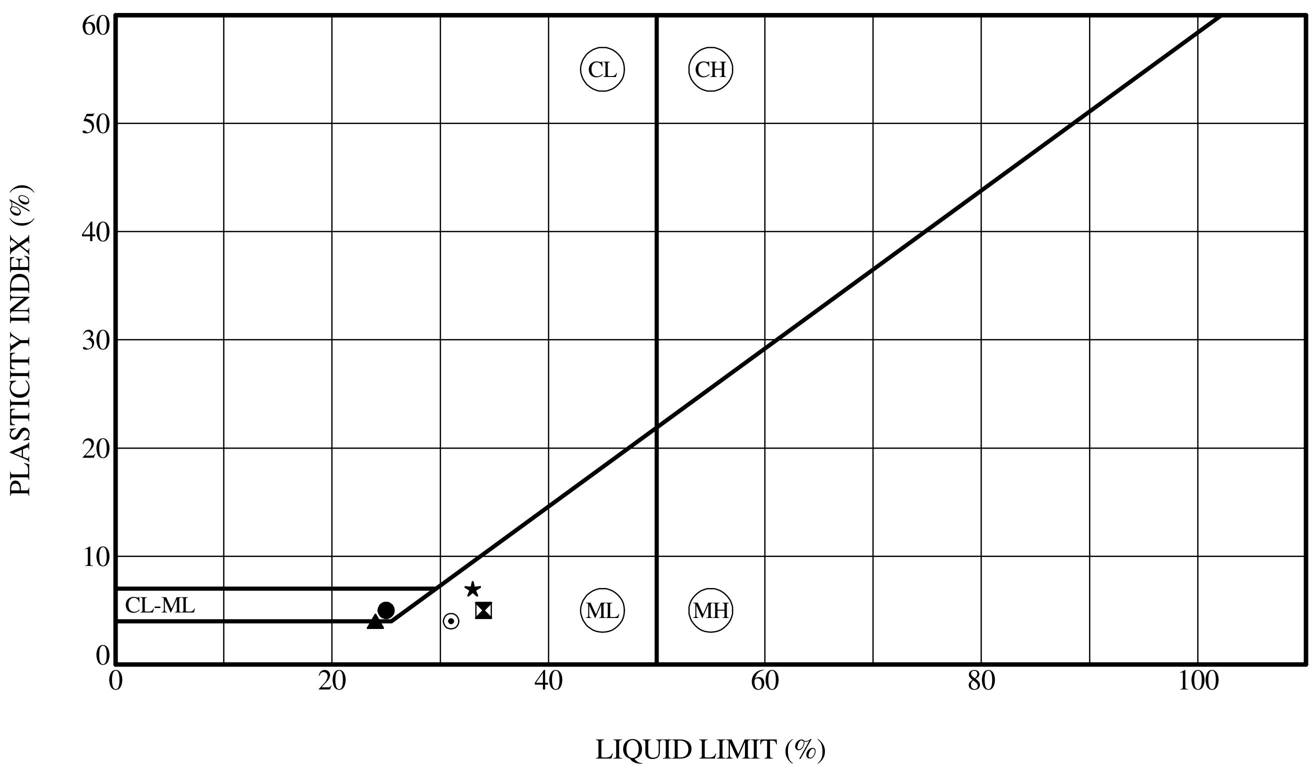
Plate C - 1



	Sample Location	Depth (ft)	LL (%)	PL (%)	PI (%)	Fines (%)	Classification
●	**TP-1**	**7.0**	**25**	**20**	**5**	**20.7**	**Silty Clayey GRAVEL with sand**
⊠	**TP-1**	**12.5**	**34**	**29**	**5**	**46.3**	**Silty SAND**
▲	**TP-2**	**6.0**	**24**	**20**	**4**	**31.9**	**Silty Clayey SAND**
★	**TP-2**	**12.0**	**33**	**26**	**7**	**42.8**	**Silty SAND**
⊙	**TP-2**	**15.5**	**31**	**27**	**4**	**40.0**	**Silty SAND**

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ATTERBERG LIMITS' RESULTS - ASTM D 4318

Lot 3R Powder Mountain
6599 North Powder Mountain Road
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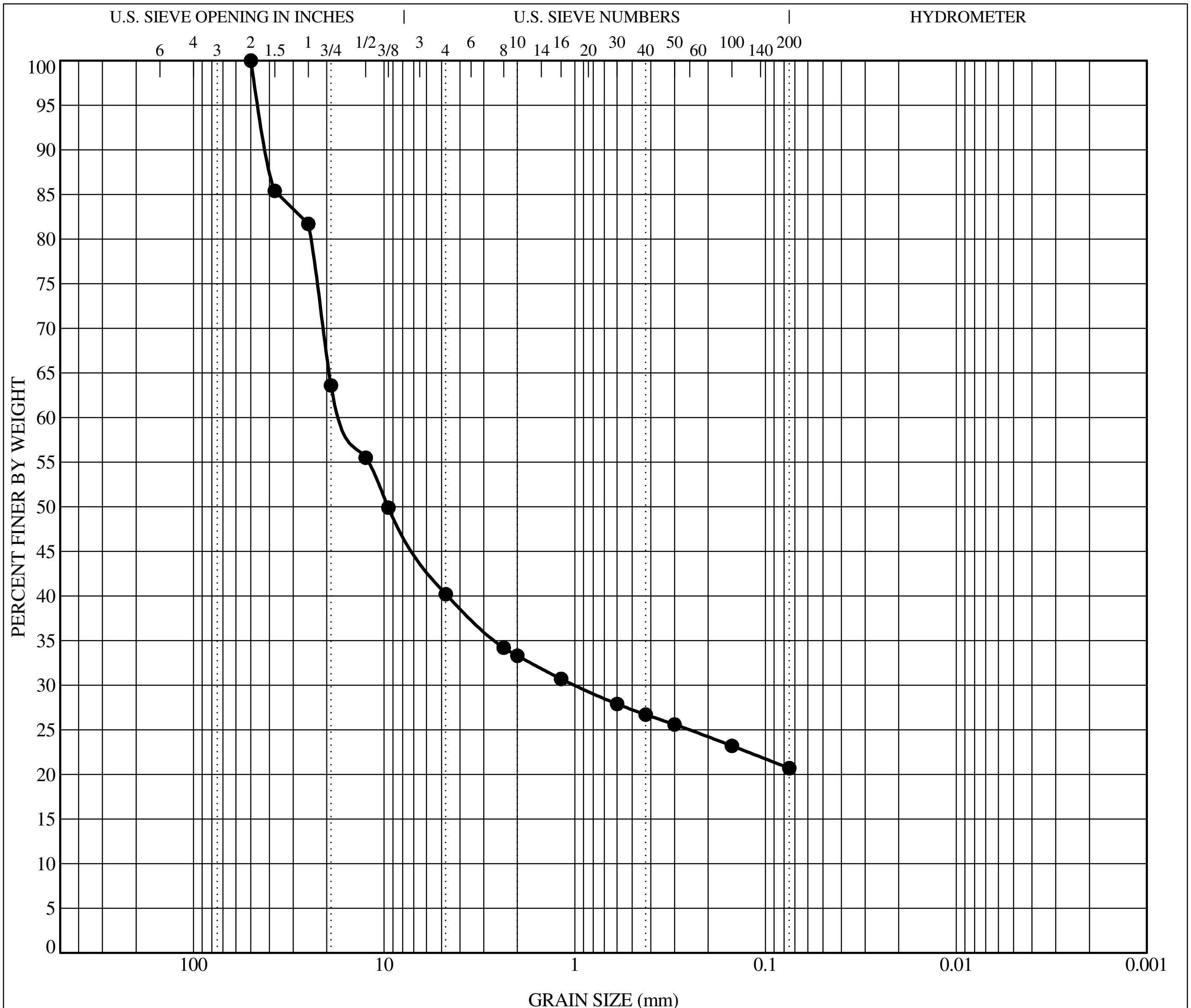
Project Number: 1174-006

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**	**7.0**	**Silty Clayey GRAVEL with sand**	**25**	**20**	**5**		

Sample Loctaion	Depth	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
● **TP-1**	**7.0**	**50**	**15.774**	**0.996**		**59.8**	**19.5**	**20.7**	

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GRAIN SIZE DISTRIBUTION - ASTM D422

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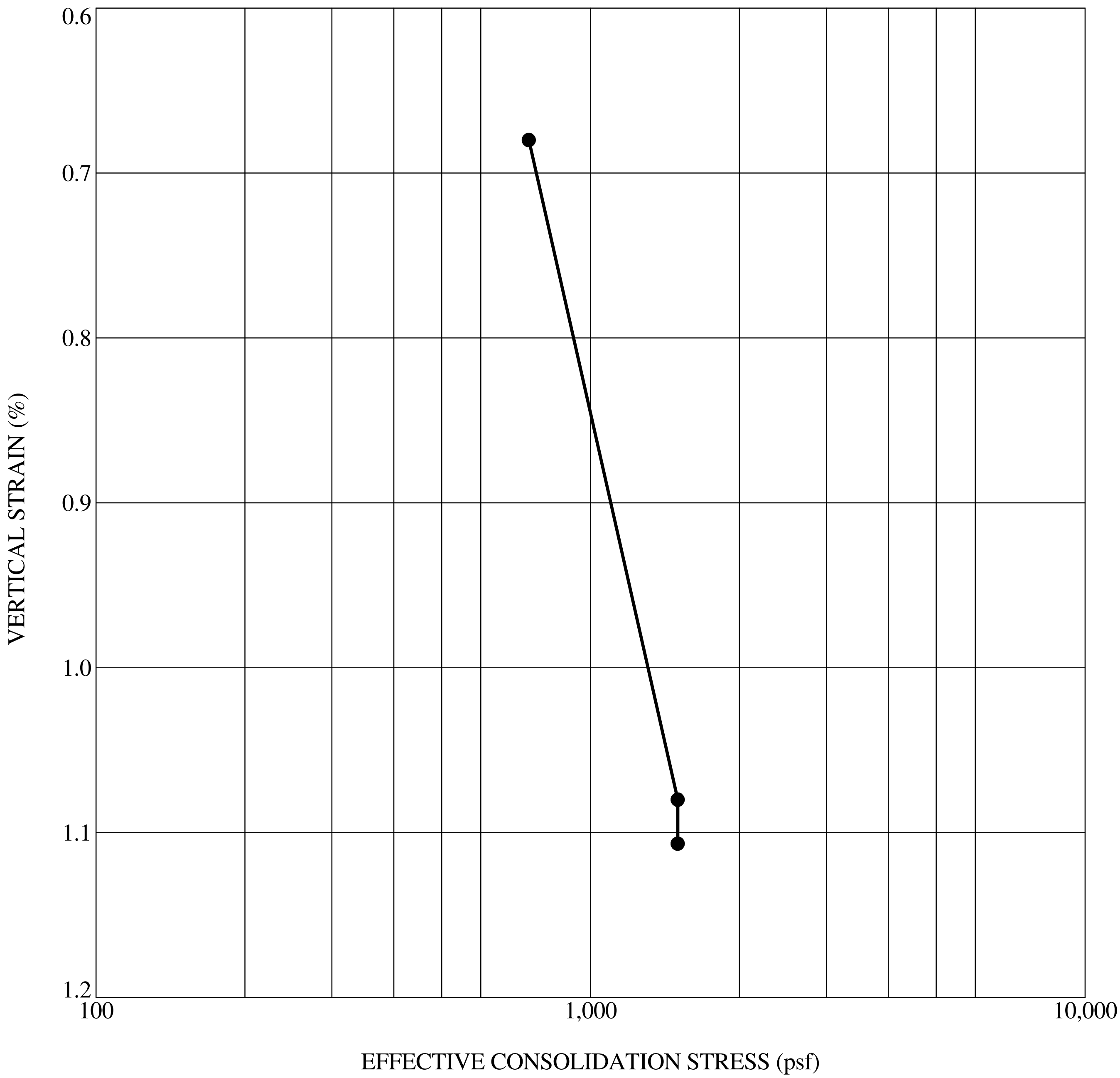
6599 North Powder Mountain Road
Eden, Utah
Project Number: 1174-006

Plate

C - 3



	Sample Location	Depth (ft)	Classification	γ_d (pcf)	MC (%)	Inundation Load (psf)	Swell (%)	Collapse (%)
●	**TP-2**	**6.1**	**Silty Clayey SAND**	**6**	**124**	**1500**		**0.03**

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1-D SWELL/COLLAPSE TEST

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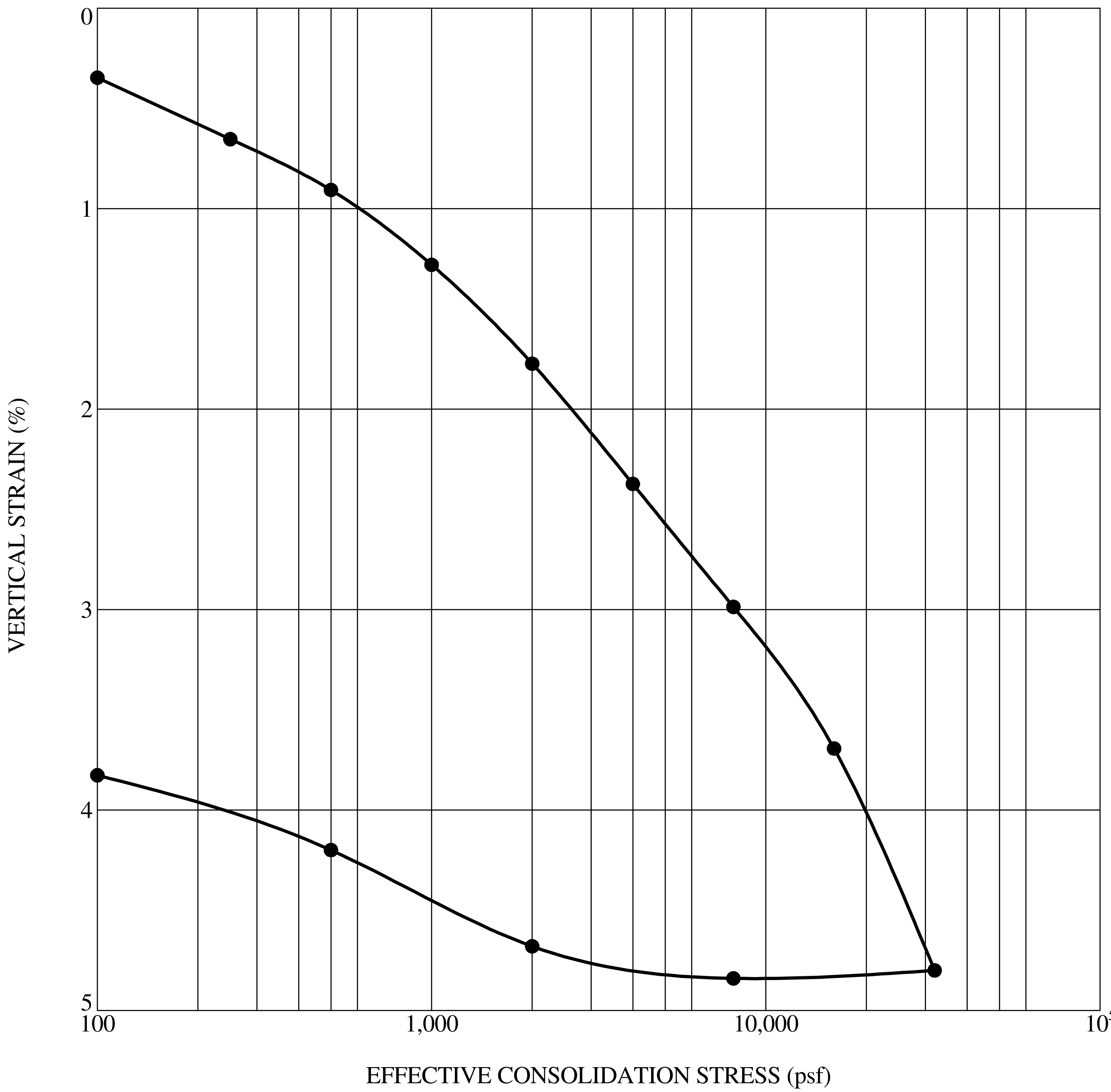
Project Number: 1174-006

Plate

C - 4

C_SWELL/COLLAPSE EXPLORATION LOGS.GPJ GEOSTRATA.GDT 6/25/20



	Sample Location	Depth (ft)	Classification	γ_d (pcf)	MC (%)	C'_c	C'_r	OCR
●	**TP-2**	**6.0**	**Silty Clayey SAND**	**124.2**	**13.0**	**0.300**	**0.005**	**2.4**

1-D CONSOLIDATION TEST - ASTM D 2435

Lot 3R Powder Mountain
6599 North Powder Mountain Road
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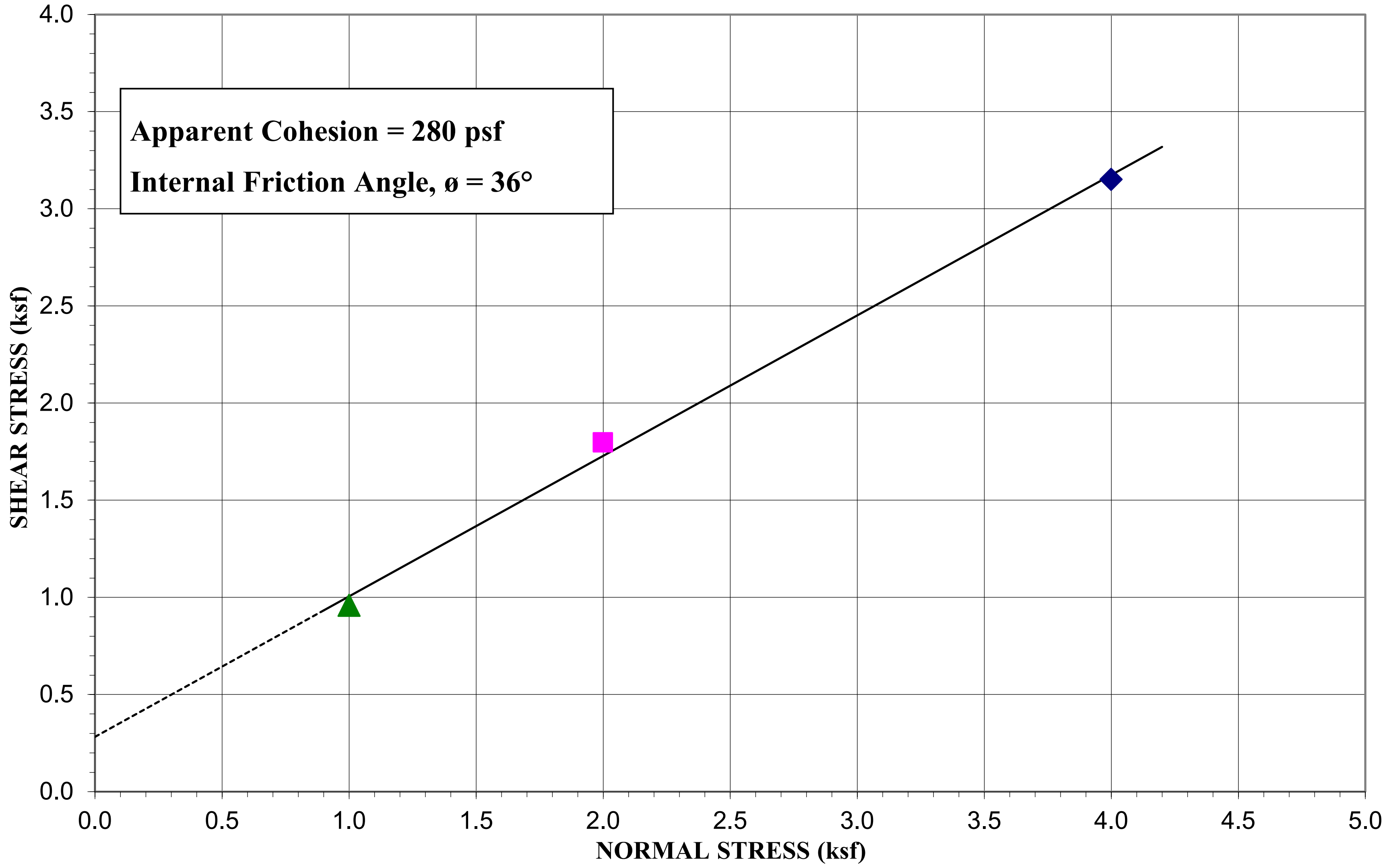
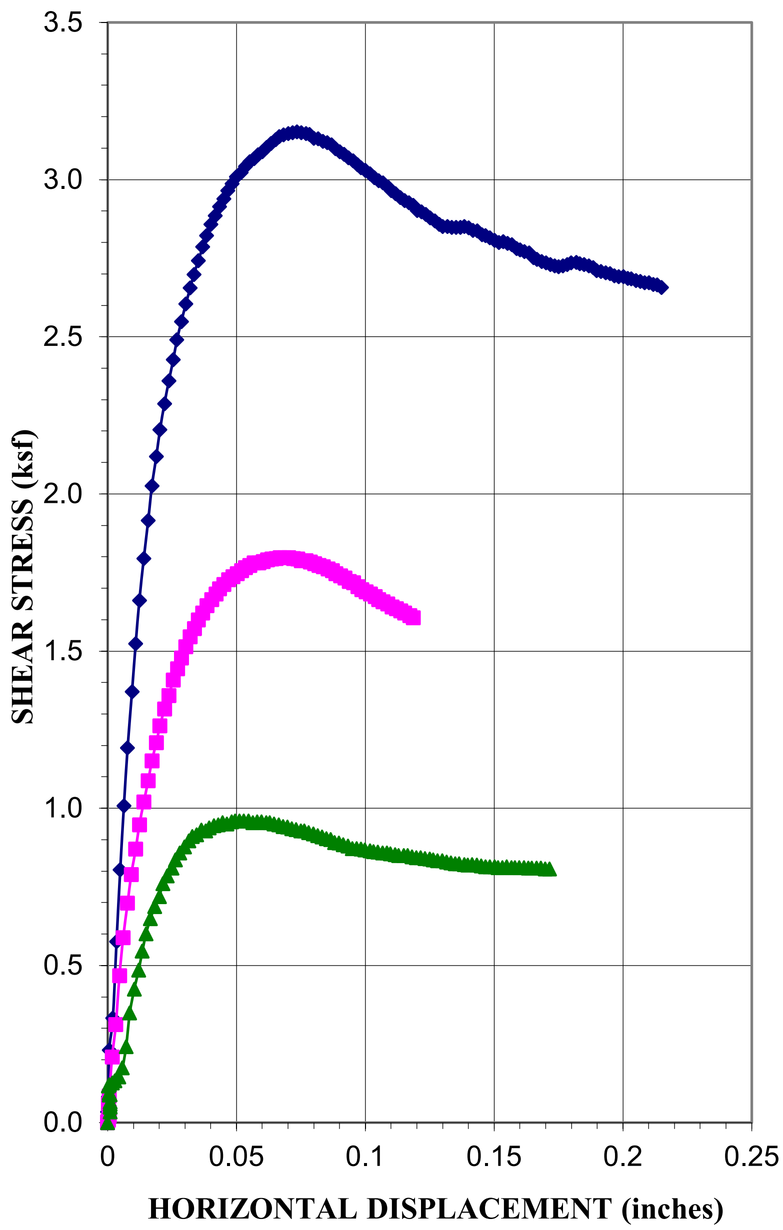
Project Number: 1174-006

Plate

C - 5

DIRECT SHEAR TEST





Sample Location:	TP-1 @ 7 feet
Type of Test:	Consolidated Drained/Saturated

Test No. (Symbol)	**1 (◆)**	**2 (■)**	**3 (▲)**
Sample Type	Remolded		
Initial Height, in.	0.862	0.862	0.86
Diameter, in.	2.5	2.5	2.5
Dry Density Before, pcf	121.0	120.3	122.0
Dry Density After, pcf	123.2	122.5	124.2
Moisture % Before	13.8	15.6	14.4
Moisture % After	17.1	17.5	18.5
Saturation, % Before	99.7	110.2	107.6
Saturation, % After	132.2	132.6	148.0
Normal Load, ksf	4.0	2.0	1.0
Shear Stress, ksf	3.15	1.80	0.96
Strain Rate	0.003333 IN/MIN		

Sample Properties	
Cohesion, psf	280
Friction Angle, ϕ	36
Liquid Limit, %	25
Plasticity Index, %	5
Percent Gravel	59.8
Percent Sand	19.5
Percent Passing No. 200 sieve	20.7
Classification	GC-GM

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PROJECT NO.: **1174-006**

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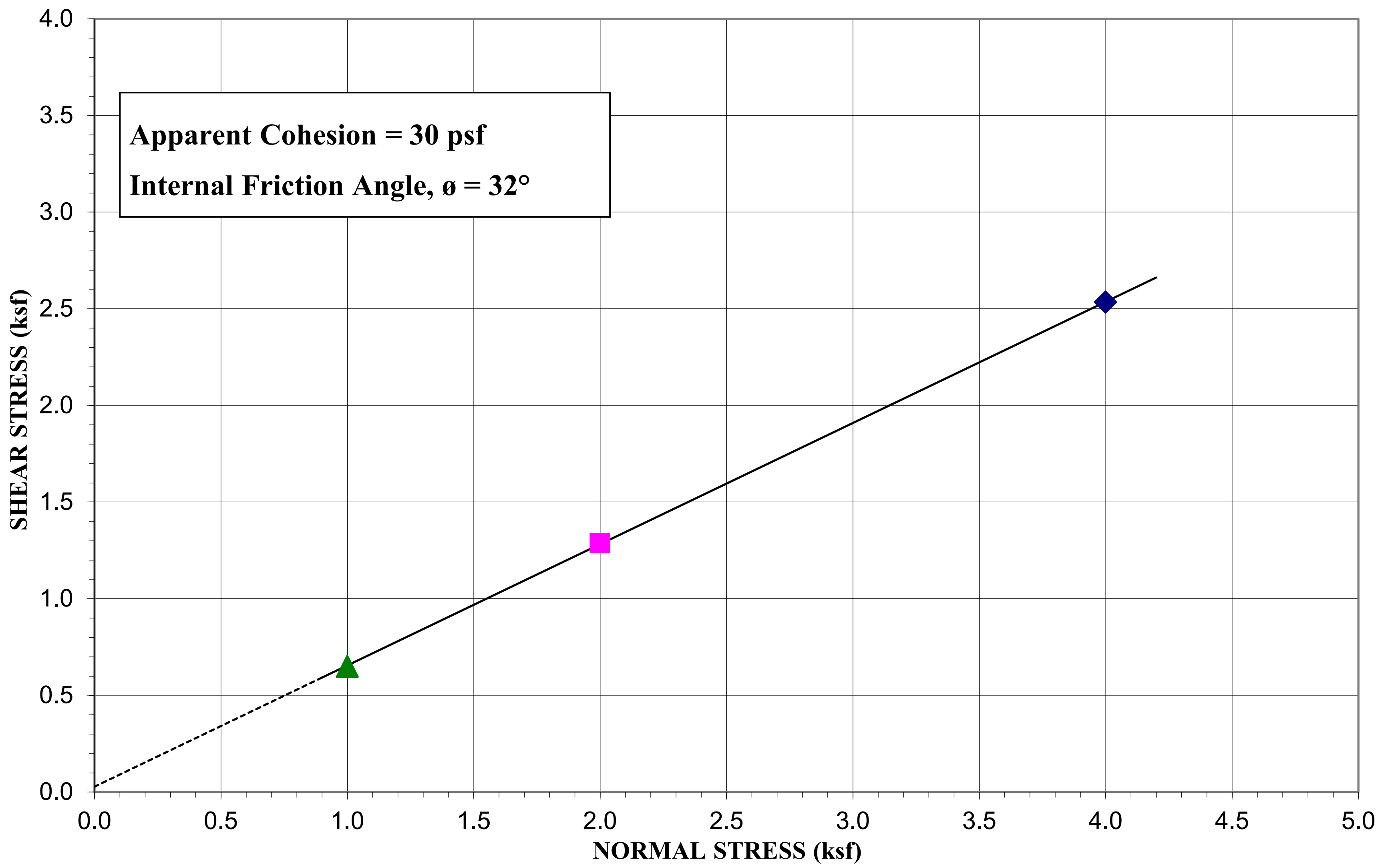
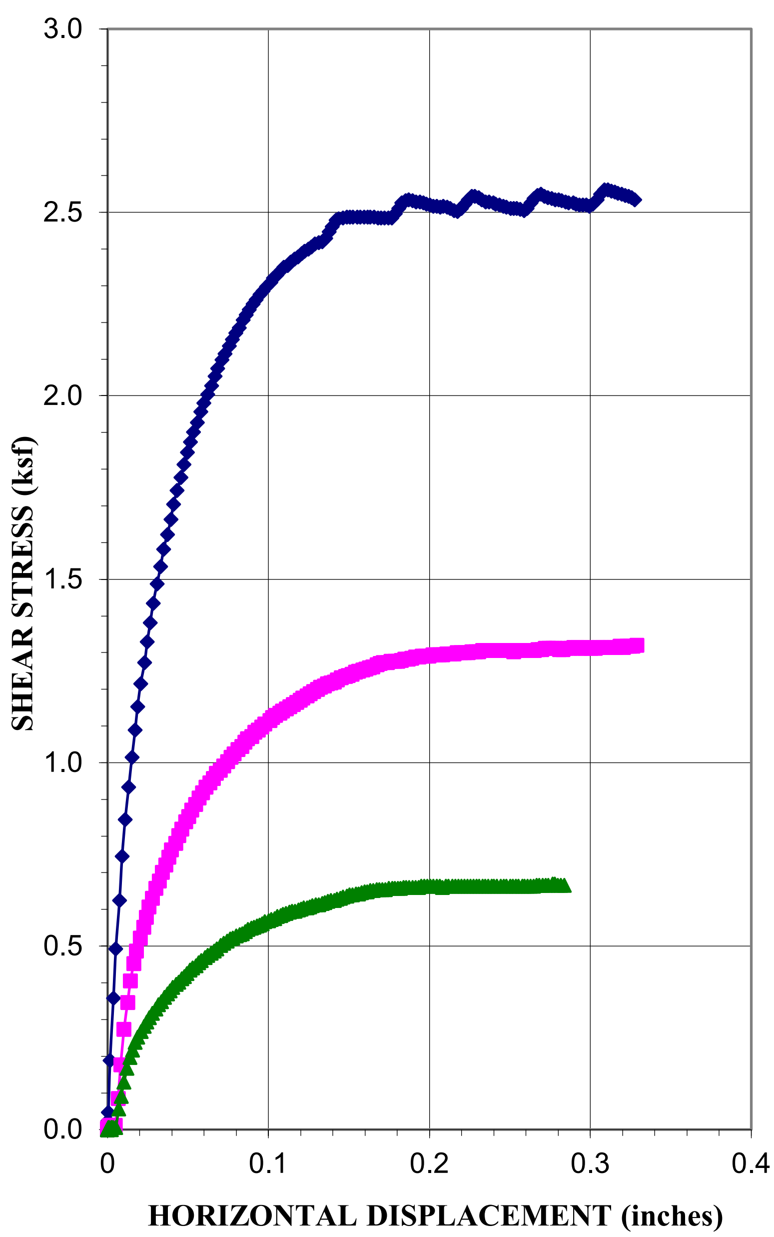
Plate C-6

DIRECT SHEAR TEST





Source:	TP-1 at 12.5 feet
Type of Test:	Consolidated Drained/Saturated

Test No. (Symbol)	1 (◆)	2 (■)	3 (▲)
Sample Type	Remolded		
Initial Height, in.	0.844	0.895	0.887
Diameter, in.	2.5	2.5	2.5
Dry Density Before, pcf	109.7	104.3	105.4
Dry Density After, pcf	112.1	106.5	107.6
Moisture % Before	9.5	10.9	11.4
Moisture % After	18.6	20.2	20.4
Saturation, % Before	49.5	49.4	52.9
Saturation, % After	103.8	96.8	100.6
Normal Load, ksf	4.0	2.0	1.0
Shear Stress, ksf	2.53	1.29	0.65
Strain Rate	0.003333 IN/MIN		

Sample Properties	
Cohesion, psf	30
Friction Angle, ϕ	32
Liquid Limit, %	34
Plasticity Index, %	5
Percent Gravel **Percent Sand**	53.7
Percent Passing No. 200 sieve	46.3
Classification	SM

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PROJECT NO.: 1174-006

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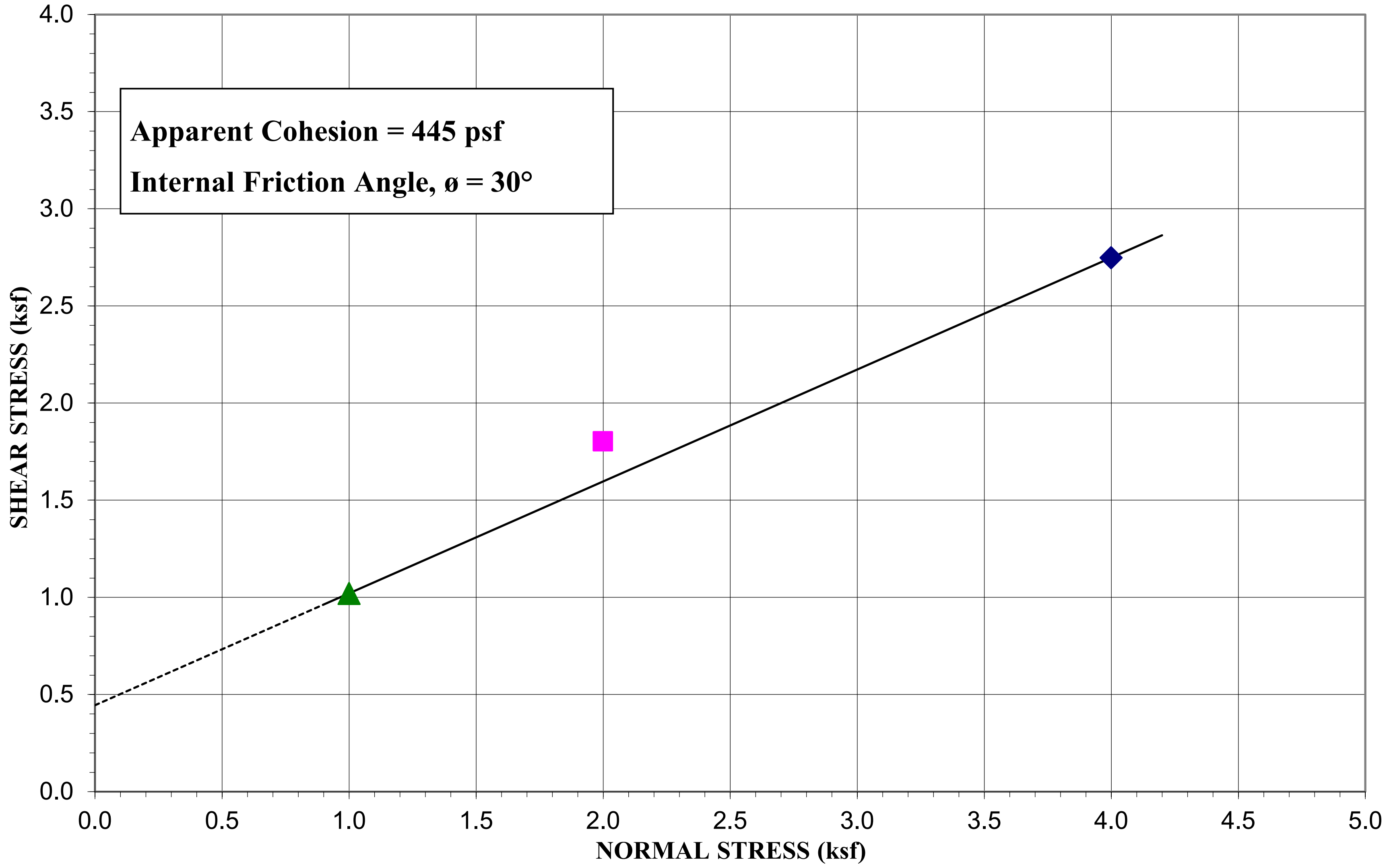
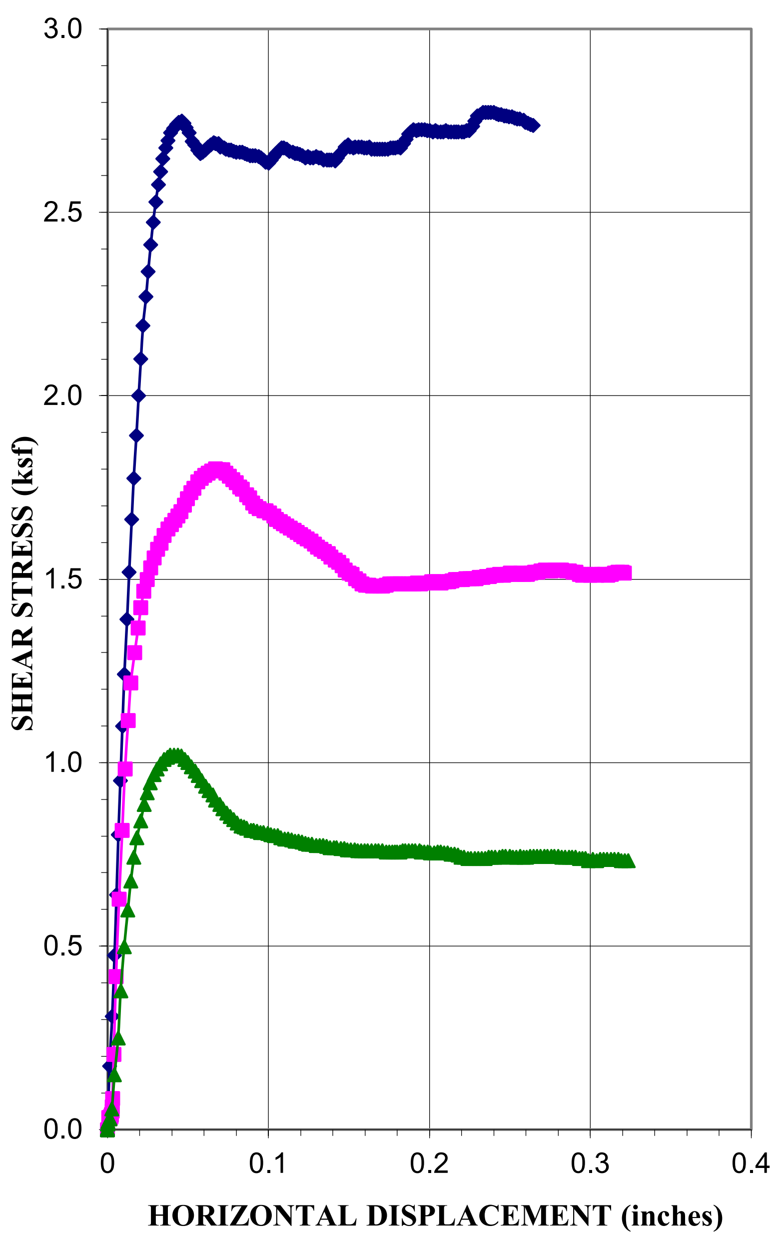
Plate C-7

DIRECT SHEAR TEST





Source:	TP-2 at 15.5 feet
Type of Test:	Consolidated Drained/Saturated

Test No. (Symbol)	1 (◆)	2 (■)	3 (▲)
Sample Type	Remolded		
Initial Height, in.	0.844	0.895	0.887
Diameter, in.	2.5	2.5	2.5
Dry Density Before, pcf	109.7	104.3	105.4
Dry Density After, pcf	112.1	106.5	107.6
Moisture % Before	9.5	10.9	11.4
Moisture % After	18.6	20.2	20.4
Saturation, % Before	49.5	49.4	52.9
Saturation, % After	103.8	96.8	100.6
Normal Load, ksf	4.0	2.0	1.0
Shear Stress, ksf	2.75	1.80	1.02
Strain Rate	0.003333 IN/MIN		

Sample Properties	
Cohesion, psf	445
Friction Angle, ϕ	30
Liquid Limit, %	31
Plasticity Index, %	4
Percent Gravel	60.0
Percent Sand	
Percent Passing No. 200 sieve	40.0
Classification	SM

PROJECT: Lot 3R Powder Mountain

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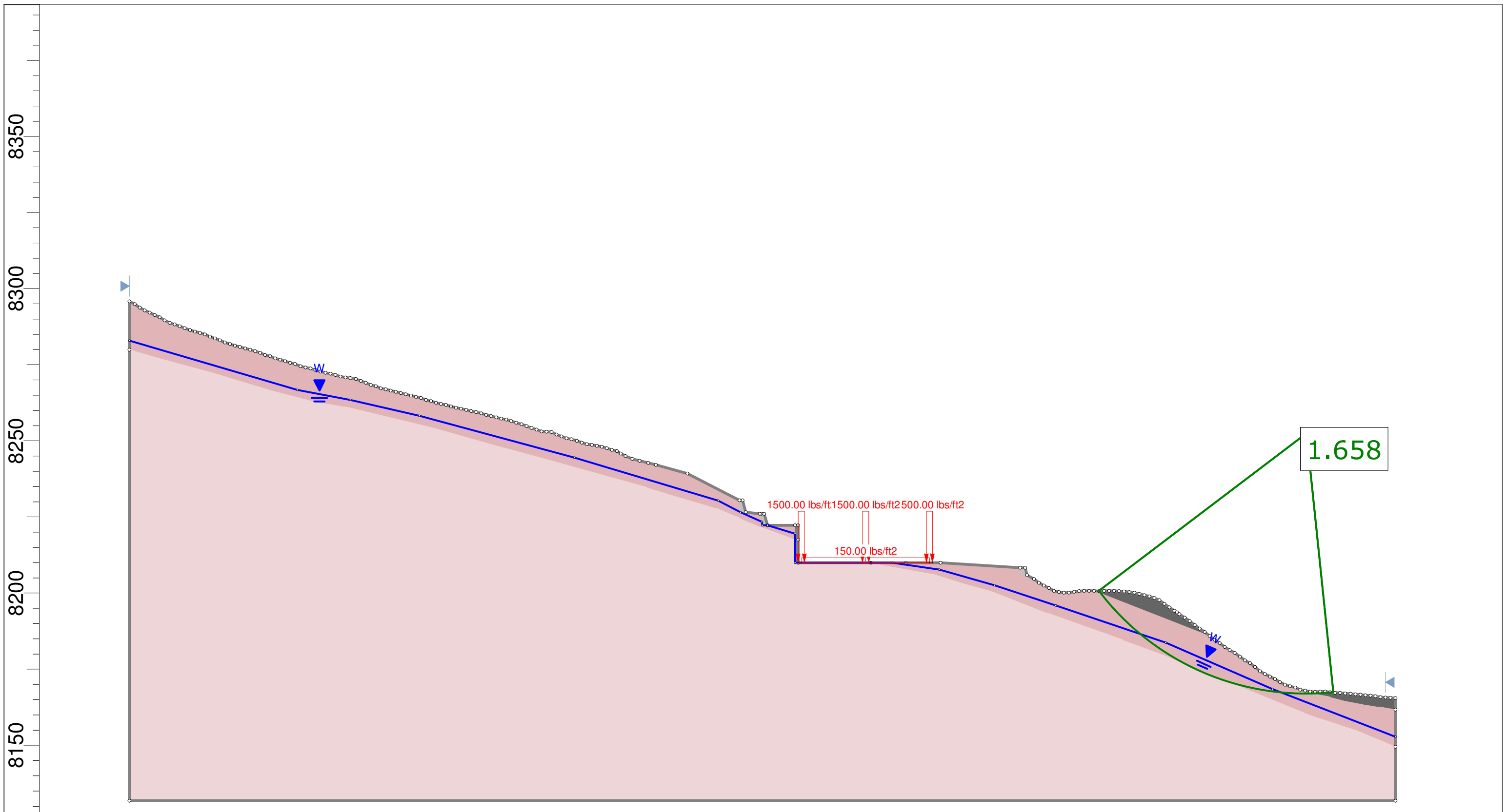
Plate C-8

Appendix D



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Cohesion 2 (psf)	Phi 2 (deg)	Angle (ccw to 1) (deg)	Water Surface	Ru
Quartzite of the Mutual Formation (Zm)		120	Mohr-Coulomb	140	36				Water Surface	
Concrete Foundation Wall		145	Mohr-Coulomb	8000	0				None	0
Argillite of the Mutual Formation (Zm)		120	Mohr-Coulomb	200	30				None	0
Fill		120	Mohr-Coulomb	50	36				None	0
Rockery		145	Anisotropic strength	0	45	2000	0	15	None	0

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A-A' Static

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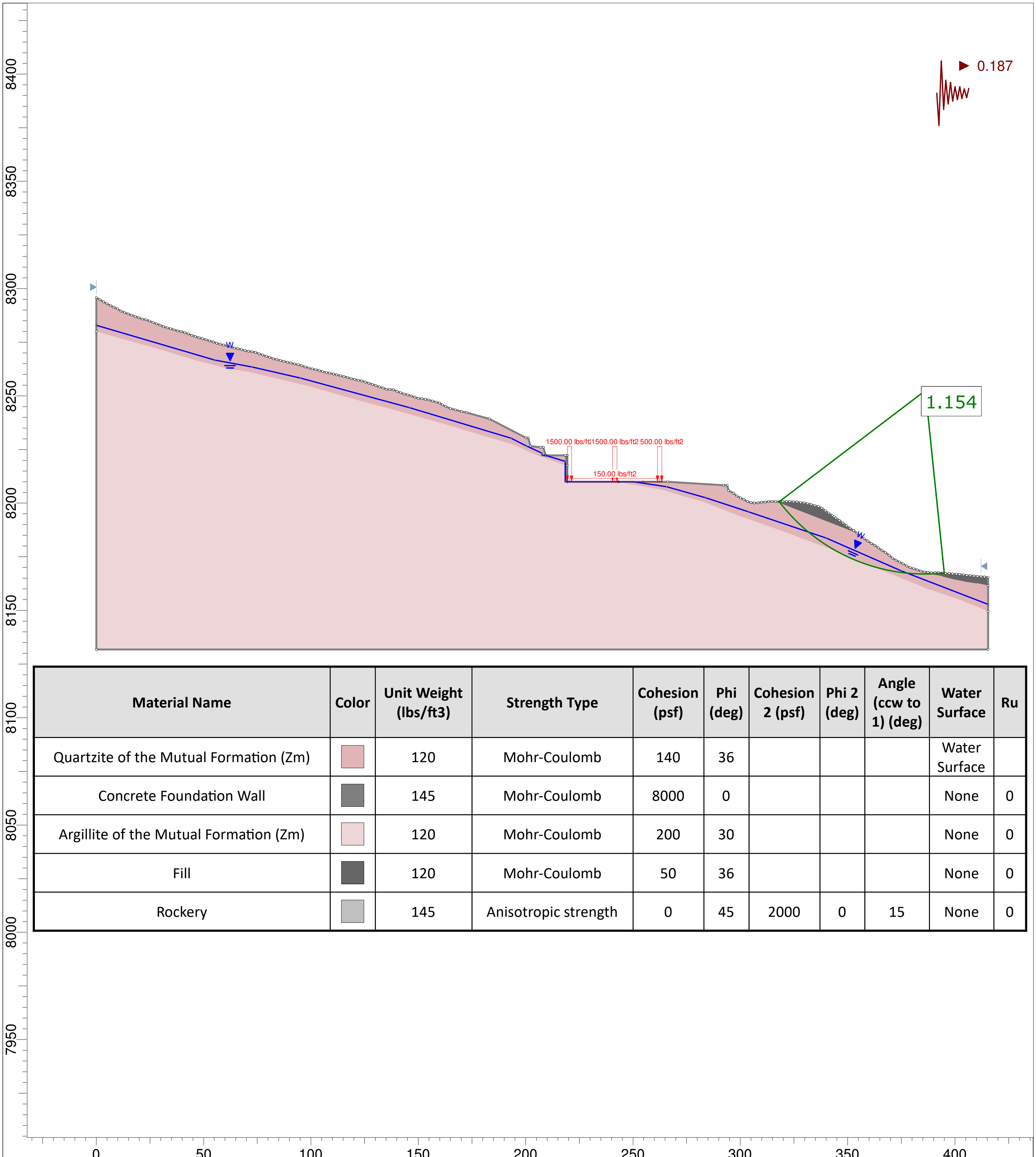
Eden, UT
Project Number: 1174-006

Plate D-1



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Cohesion 2 (psf)	Phi 2 (deg)	Angle (ccw to 1) (deg)	Water Surface	Ru
Quartzite of the Mutual Formation (Zm)		120	Mohr-Coulomb	140	36				Water Surface	
Concrete Foundation Wall		145	Mohr-Coulomb	8000	0				None	0
Argillite of the Mutual Formation (Zm)		120	Mohr-Coulomb	200	30				None	0
Fill		120	Mohr-Coulomb	50	36				None	0
Rockery		145	Anisotropic strength	0	45	2000	0	15	None	0

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A-A' Pseudo Static

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Plate D-2

Appendix E

Important Information about This

Geotechnical-Engineering Report

Subsurface problems are a principal cause of construction delays, cost overruns, claims, and disputes.

While you cannot eliminate all such risks, you can manage them. The following information is provided to help.

The Geoprofessional Business Association (GBA) has prepared this advisory to help you – assumedly a client representative – interpret and apply this geotechnical-engineering report as effectively as possible. In that way, you can benefit from a lowered exposure to problems associated with subsurface conditions at project sites and development of them that, for decades, have been a principal cause of construction delays, cost overruns, claims, and disputes. If you have questions or want more information about any of the issues discussed herein, contact your GBA-member geotechnical engineer. Active engagement in GBA exposes geotechnical engineers to a wide array of risk-confrontation techniques that can be of genuine benefit for everyone involved with a construction project.

Understand the Geotechnical-Engineering Services Provided for this Report

Geotechnical-engineering services typically include the planning, collection, interpretation, and analysis of exploratory data from widely spaced borings and/or test pits. Field data are combined with results from laboratory tests of soil and rock samples obtained from field exploration (if applicable), observations made during site reconnaissance, and historical information to form one or more models of the expected subsurface conditions beneath the site. Local geology and alterations of the site surface and subsurface by previous and proposed construction are also important considerations. Geotechnical engineers apply their engineering training, experience, and judgment to adapt the requirements of the prospective project to the subsurface model(s). Estimates are made of the subsurface conditions that will likely be exposed during construction as well as the expected performance of foundations and other structures being planned and/or affected by construction activities.

The culmination of these geotechnical-engineering services is typically a geotechnical-engineering report providing the data obtained, a discussion of the subsurface model(s), the engineering and geologic engineering assessments and analyses made, and the recommendations developed to satisfy the given requirements of the project. These reports may be titled investigations, explorations, studies, assessments, or evaluations. Regardless of the title used, the geotechnical-engineering report is an engineering interpretation of the subsurface conditions within the context of the project and does not represent a close examination, systematic inquiry, or thorough investigation of all site and subsurface conditions.

Geotechnical-Engineering Services are Performed for Specific Purposes, Persons, and Projects, and At Specific Times

Geotechnical engineers structure their services to meet the specific needs, goals, and risk management preferences of their clients. A geotechnical-engineering study conducted for a given civil engineer will not likely meet the needs of a civil-works constructor or even a different civil engineer. Because each geotechnical-engineering study is unique, each geotechnical-engineering report is unique, prepared *solely* for the client.

Likewise, geotechnical-engineering services are performed for a specific project and purpose. For example, it is unlikely that a geotechnical-engineering study for a refrigerated warehouse will be the same as one prepared for a parking garage; and a few borings drilled during a preliminary study to evaluate site feasibility will not be adequate to develop geotechnical design recommendations for the project.

Do not rely on this report if your geotechnical engineer prepared it:
- for a different client;
- for a different project or purpose;
- for a different site (that may or may not include all or a portion of the original site); or
- before important events occurred at the site or adjacent to it; e.g., man-made events like construction or environmental remediation, or natural events like floods, droughts, earthquakes, or groundwater fluctuations.

Note, too, the reliability of a geotechnical-engineering report can be affected by the passage of time, because of factors like changed subsurface conditions; new or modified codes, standards, or regulations; or new techniques or tools. *If you are the least bit uncertain* about the continued reliability of this report, contact your geotechnical engineer before applying the recommendations in it. A minor amount of additional testing or analysis after the passage of time – if any is required at all – could prevent major problems.

Read this Report in Full

Costly problems have occurred because those relying on a geotechnical-engineering report did not read the report in its entirety. Do not rely on an executive summary. Do not read selective elements only. *Read and refer to the report in full.*

You Need to Inform Your Geotechnical Engineer About Change

Your geotechnical engineer considered unique, project-specific factors when developing the scope of study behind this report and developing the confirmation-dependent recommendations the report conveys. Typical changes that could erode the reliability of this report include those that affect:
- the site's size or shape;
- the elevation, configuration, location, orientation, function or weight of the proposed structure and the desired performance criteria;
- the composition of the design team; or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project or site changes – even minor ones – and request an assessment of their impact. *The geotechnical engineer who prepared this report cannot accept*

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responsibility or liability for problems that arise because the geotechnical engineer was not informed about developments the engineer otherwise would have considered.

Most of the "Findings" Related in This Report Are Professional Opinions

Before construction begins, geotechnical engineers explore a site's subsurface using various sampling and testing procedures. *Geotechnical engineers can observe actual subsurface conditions only at those specific locations where sampling and testing is performed.* The data derived from that sampling and testing were reviewed by your geotechnical engineer, who then applied professional judgement to form opinions about subsurface conditions throughout the site. Actual sitewide-subsurface conditions may differ – maybe significantly – from those indicated in this report. Confront that risk by retaining your geotechnical engineer to serve on the design team through project completion to obtain informed guidance quickly, whenever needed.

This Report's Recommendations Are Confirmation-Dependent

The recommendations included in this report – including any options or alternatives – are confirmation-dependent. In other words, they are not final, because the geotechnical engineer who developed them relied heavily on judgement and opinion to do so. Your geotechnical engineer can finalize the recommendations *only after observing actual subsurface conditions* exposed during construction. If through observation your geotechnical engineer confirms that the conditions assumed to exist actually do exist, the recommendations can be relied upon, assuming no other changes have occurred. *The geotechnical engineer who prepared this report cannot assume responsibility or liability for confirmation-dependent recommendations if you fail to retain that engineer to perform construction observation.*

This Report Could Be Misinterpreted

Other design professionals' misinterpretation of geotechnical-engineering reports has resulted in costly problems. Confront that risk by having your geotechnical engineer serve as a continuing member of the design team, to:

- confer with other design-team members;
- help develop specifications;
- review pertinent elements of other design professionals' plans and specifications; and
- be available whenever geotechnical-engineering guidance is needed.

You should also confront the risk of constructors misinterpreting this report. Do so by retaining your geotechnical engineer to participate in prebid and preconstruction conferences and to perform construction-phase observations.

Give Constructors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can shift unanticipated-subsurface-conditions liability to constructors by limiting the information they provide for bid preparation. To help prevent the costly, contentious problems this practice has caused, include the complete geotechnical-engineering report, along with any attachments or appendices, with your contract documents, *but be certain to note conspicuously that you've included the material for information purposes only.* To avoid misunderstanding, you may also want to note that "informational purposes" means constructors have no right to rely on the interpretations, opinions, conclusions, or recommendations in the report. Be certain that constructors know they may learn about specific project requirements, including options selected from the report, *only* from the design drawings and specifications. Remind constructors that they may perform their own studies if they want to, and *be sure to allow enough time* to permit them to do so. Only then might you be in a position to give constructors the information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions. Conducting prebid and preconstruction conferences can also be valuable in this respect.

Read Responsibility Provisions Closely

Some client representatives, design professionals, and constructors do not realize that geotechnical engineering is far less exact than other engineering disciplines. This happens in part because soil and rock on project sites are typically heterogeneous and not manufactured materials with well-defined engineering properties like steel and concrete. That lack of understanding has nurtured unrealistic expectations that have resulted in disappointments, delays, cost overruns, claims, and disputes. To confront that risk, geotechnical engineers commonly include explanatory provisions in their reports. Sometimes labeled "limitations," many of these provisions indicate where geotechnical engineers' responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely.* Ask questions. Your geotechnical engineer should respond fully and frankly.

Geoenvironmental Concerns Are Not Covered

The personnel, equipment, and techniques used to perform an environmental study – e.g., a "phase-one" or "phase-two" environmental site assessment – differ significantly from those used to perform a geotechnical-engineering study. For that reason, a geotechnical-engineering report does not usually provide environmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated subsurface environmental problems have led to project failures.* If you have not obtained your own environmental information about the project site, ask your geotechnical consultant for a recommendation on how to find environmental risk-management guidance.

Obtain Professional Assistance to Deal with Moisture Infiltration and Mold

While your geotechnical engineer may have addressed groundwater, water infiltration, or similar issues in this report, the engineer's services were not designed, conducted, or intended to prevent migration of moisture – including water vapor – from the soil through building slabs and walls and into the building interior, where it can cause mold growth and material-performance deficiencies. Accordingly, *proper implementation of the geotechnical engineer's recommendations will not of itself be sufficient to prevent moisture infiltration. Confront the risk of moisture infiltration* by including building-envelope or mold specialists on the design team. *Geotechnical engineers are not building-envelope or mold specialists.*

GEOPROFESSIONAL BUSINESS ASSOCIATION

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