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GEOTECHNICAL INVESTIGATION
3 Tree Subdivision
13344 South 2700 West
Riverton, Utah

Prepared for:

Ridgeway Excavation

IGES Project No. 02398-001

October 19, 2016

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OK

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Attn: Todd Miller, Dusty Argyle

Geotechnical Investigation
3 Tree Subdivision
13344 South 2700 West
Riverton, Utah

IGES Project No. 02398-001

Prepared by:

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Reviewed by:

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October 19, 2016

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APPENDIX

1.0 EXECUTIVE SUMMARY

This report presents the results of a geotechnical investigation conducted for the proposed subdivision currently called "3 Tree" located at 13344 South 2700 West in Riverton, Utah. Based on the subsurface conditions encountered at the site, the subject site is suitable for the proposed construction provided that the recommendations contained in this report are complied with. A brief summary of the critical recommendations is included below:

- Soils at the site consisted primarily of 18 inches of undocumented fill comprised of Lean CLAY (CL) with some debris. The fill was underlain by Lean CLAY (CL) and Sandy Lean CLAY (CL).
- Soils with a pinhole structure (potentially collapsible) were observed in the upper 5 feet in the fine-grained soil. Test results indicate the soils are moderately collapsible (4.7%).
- Shallow spread or continuous wall footings should be established *entirely* on undisturbed native non-collapsible soils (no pinholes). Footings should be extended to a minimum of 5 feet below existing site grade to get below the potentially collapsible soils.

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

NOTE: The scope of services provided within this report is 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 3 Tree Subdivision located at 13344 South 2700 West in Riverton, Utah. The purposes of this investigation were to assess the nature and engineering properties of the subsurface soils, and to provide recommendations for general site grading and design and construction of foundations and slabs-on-grade.

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.

The recommendations presented 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 to the west of 13344 South 2700 West in Riverton, Utah (see Figure A-1, *Site Vicinity Map*). The property has a total area of approximately 1.1 acres. The development consists of 3 lots. We understand Lot 1 will remain as-is and is not a part of this investigation. It is our understanding that the proposed development will consist of 2 single-family homes, one on Lot 2 and one on Lot 3. Construction plans were not available for our review at the time this report was prepared; however, we assume that the new structures will be multi-story wood-framed residences with basements founded on conventional strip and spread footings.

3.0 METHODS OF STUDY

3.1 FIELD INVESTIGATION

As a part of this investigation, subsurface soil conditions were explored by completing 2 exploratory test pits to depths ranging from 10 to 10.5 feet below the existing site grade. The approximate locations of the explorations are shown on Figures A-2 (*Geotechnical Maps*) in Appendix A. Exploration points were placed to provide optimum coverage of the site. Logs of the subsurface conditions as encountered in the explorations were recorded at the time of excavation by a member of our technical staff and are presented as Figures A-3 through A-4 in Appendix A. A *Key to Soil Symbols and Terminology* used on the boring logs is included as Figure A-5.

The test pits were completed using a mini-ex provided by Ridgeway Excavation. Soil sampling was completed to collect representative samples of the various layers observed at the site. Disturbed samples were placed in plastic baggies and relatively undisturbed soil samples were collected with the use of a 6-inch long brass tube attached to a hand sampler driven with a 2-lb sledge hammer. All samples were transported to our laboratory to evaluate the engineering properties of the various earth materials observed. The soils were classified according to the *Unified Soil Classification System* (USCS) by the Geotechnical Engineer. Classifications for the individual soil units are shown on the attached Test Pit Logs (Figures A-3 through A-4).

3.2 LABORATORY INVESTIGATION

Geotechnical laboratory tests were conducted on selected relatively undisturbed and bulk soil samples obtained during our field investigation. The laboratory testing program was designed to evaluate the engineering characteristics of onsite earth materials. Laboratory tests conducted during this investigation include:

- Water Content (ASTM D7263)
- Unit Weight (ASTM D2216)
- Atterberg Limits (ASTM D4318)
- No. 200 Sieve Wash (ASTM D1140)
- One-dimensional collapse (ASTM D4546 & 5333)
- One-dimensional Consolidation (ASTM D2435)
- Unconsolidated-Undrained Triaxial Compression Test (ASTM D2850)
- Corrosion Testing-sulfate and chloride concentrations, pH and resistivity (ASTM D4972, D4327, D4327, C1580 and EPA 300.0)

The results of the laboratory tests are presented on the test pit logs in Appendix A (Figures A-3 through A-4) and the laboratory test results presented in Appendix B.

3.3 ENGINEERING ANALYSIS

Engineering analyses were performed using soil data obtained from the laboratory test results and empirical correlations from material density, depositional characteristics and classifications. Analyses were performed using formulas, calculations and software that represent methods currently accepted by the geotechnical industry. These methods include settlement, bearing capacity, lateral earth pressures, trench stability and pavement design. Appropriate factors of safety were applied to the results consistent with industry standards and the accepted standard of care.

4.0 GENERALIZED SITE CONDITIONS

4.1 SURFACE CONDITIONS

The subject site is located at an elevation of approximately 4,520 feet above mean sea level. At the time of our subsurface investigation the majority of the site existed as open land, there was an existing barn on Lot 2 that will be razed as part of this development. The ground surface is covered with grass, weeds, some undocumented fill and native soils. The site is generally flat.

4.2 SUBSURFACE CONDITIONS

4.2.1 Earth Materials

Based on our observations, the majority of the site is overlain by up to 18 inches of undocumented fill comprised of Lean CLAY (CL) with some debris. The undocumented fill was underlain by Lean CLAY (CL), and Sandy Lean CLAY (CL). The clay was generally stiff to medium stiff and moist. Pinholes were observed in the upper 4.5 feet in test pit 1.

The stratification lines shown on the enclosed test pit logs represent the approximate boundary between soil types (Figures A-3 to A-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. Additional descriptions of these soil units are presented on the boring logs (Figures A-3 through A-4 in Appendix A).

4.2.2 Groundwater

Groundwater was not encountered in any of the test pits completed for our 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 or fall several feet seasonally depending on the time of year. However, based on our field investigation, we anticipate that groundwater will not impact the proposed construction.

4.2.3 Collapsible Soils

Collapse is a phenomena where undisturbed native soils under increased loading can exhibit volumetric strain and consolidation upon wetting. 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. Collapsible soils are typically characterized by a pinhole structure and relatively light in-situ density. Pinholes were observed in the native fine-grained soil up to 4.5 feet in depth below existing site grade in test pit 1.

Collapse/swell tests (ASTM D4546 & D5333) were performed on one relatively undisturbed sample of native fine-grained soil in test pit 1 during this investigation.

The results of the tests suggest that the upper 4.5 feet of the native soils, in general, experience moderate volumetric strain under increased moisture conditions (about 4.7 percent strain). More detailed results of the collapse testing are provided in Appendix B.

4.3 SEISMICITY

Following the criteria outlined in the 2012 International Building Code (IBC, 2012), spectral response at the site was evaluated for the *Maximum Considered Earthquake* (MCE) which equates to a probabilistic seismic event having a two percent probability of exceedance in 50 years (2PE50). Spectral accelerations were determined based on the location of the site using the *U.S. Seismic "DesignMaps" Web Application* (USGS, 2012); this software incorporates seismic hazard maps depicting probabilistic ground motions and spectral response data developed for the United States by the U. S. Geological Survey as part of NEHRP/NSHMP (Frankel et al., 1996). These maps have been incorporated into both *NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures* (FEMA, 1997) and the *International Building Code* (IBC) (International Code Council, 2012).

To account for site effects, site coefficients that vary with the magnitude of spectral acceleration and *Site Class* are used. Site Class is a parameter that accounts for site amplification effects from soils and is based on the average shear wave velocity of the upper 100 feet; based on our field exploration and our understanding of the geology in this area, the subject site is appropriately classified as Site Class D (stiff soil). Based on IBC criteria, the short-period site coefficient (F_a) is 1.000 and long-period site coefficient (F_v) is 1.565. Based on the design spectral response accelerations for a *Building Risk Category* of I, II, III, or IV, the site's *Seismic Design Category* is D. The short- and long-period *Design Spectral Response Accelerations* are presented in Table 4.3; a summary of the *Design Maps* analysis is presented in Appendix C. The *peak ground acceleration* (PGA) may be taken as $0.4 * S_{MS}$.

Table 4.3 - Short- and Long-Period Spectral Accelerations for MCE

Parameter	Short Period (0.2 sec)	Long Period (1.0 sec)
MCE Spectral Response Acceleration (g)	$S_S = 1.309$	$S_1 = 0.435$
MCE Spectral Response Acceleration Site Class D (g)	$S_{MS} = S_sF_a = 1.309$	$S_{M1} = S_1F_v = 0.681$
Design Spectral Response Acceleration (g)	$S_{DS} = S_{MS}*^2/_3 = 0.873$	$S_{D1} = S_{M1}*^2/_3 = 0.454$

4.4 OTHER GEOLOGIC HAZARDS

Geologic hazards and conditions can be defined as naturally occurring geologic conditions or processes that could present a danger to human life and property or result in impacts to conventional construction procedures. These hazards and conditions must be considered before development of the site. There are several hazards and conditions in addition to seismicity and faulting that if present at a site, should be considered in the design of critical and essential facilities. The hazards considered for this site include liquefaction and hydro-collapsible soils.

4.4.1 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 settlement 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.

Referring to the *Liquefaction-Potential Map for Salt Lake County, Utah* published by the Utah Geological Survey, the site is located within an area currently designated as "very low" for liquefaction potential. The upper 10 to 10.5 feet are not considered liquefiable based on our field observations and laboratory testing. Deeper deposits may be more susceptible, but a full liquefaction study is not part of the scope of work and beyond the standard of care for single family residential housing.

5.0 ENGINEERING CONCLUSIONS AND RECOMMENDATIONS

5.1 GENERAL CONCLUSIONS

Based on the subsurface conditions encountered at the site, 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 process any undocumented fill or otherwise unsuitable soils currently present at the site be removed from beneath proposed footings or footings be deepened to extend below the unsuitable soils. We also recommend that IGES be on site at key points during construction to see that the recommendations in this report are implemented. Footings may be established *entirely* on undisturbed native non-collapsible soils (no pinholes). Footings should be extended a minimum of 5 feet in depth below existing site grade to get below the potentially collapsible soils. As mentioned previously, there is a *moderate* collapse potential in the upper 5 feet in test pit TP-1; therefore, we recommend the client closely follow the moisture protection and surface drainage section (Section 5.7) in this report to minimize the potential for water to infiltrate to these soils.

The following sub-sections present our recommendations for general site grading, pavement design, design of foundations, slabs-on-grade, lateral earth pressures, moisture protection and preliminary soil corrosion.

5.2 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 minimizing the risk of differential settlement of foundations as a result of variations in subgrade conditions.

5.2.1 General Site Preparation and Grading

Within the areas to be graded (below proposed structures, fill sections, concrete flatwork, or pavement sections), any existing surface vegetation, debris, asphalt, undocumented fill and concrete should be removed and the upper 12 to 18 inches should be grubbed to remove the majority of the roots and organic matter. Any existing utilities should be re-routed or protected in-place. Although not anticipated, if tree roots are exposed they should be grubbed-out and replaced with engineered fill. The exposed native soils should then be proof-rolled with heavy rubber-tired equipment such as a loader. Any soft/loose areas identified during proof-rolling should be removed and replaced with structural fill.

An IGES representative should observe the site preparation and grading operations to assess whether the recommendations presented in this report have been complied with.

5.2.2 Excavations

Soft, porous, or otherwise unsuitable soils beneath foundations or concrete flatwork may need to be over-excavated and replaced with structural fill. The excavations should extend a minimum of 1-foot laterally for every foot of depth of over-excavation. Excavations should extend laterally at least two feet beyond slabs-on-grade. Structural fill should consist of granular materials and should be placed and compacted in accordance with the recommendations presented in this report.

5.2.3 Excavation Stability

The contractor is responsible for site safety, including all temporary slopes and trenches excavated at the site and design of any required temporary shoring. The contractor is responsible for providing the "competent person" required by OSHA standards to evaluate soil conditions. Soil types are expected to consist of *Type B* soils (cohesive soils with unconfined compressive strength greater than 0.5 tsf, but less than 1.5 tsf). Close coordination between the competent person and IGES should be maintained to facilitate construction while providing safe excavations.

Based on Occupational Safety and Health (OSHA) guidelines for excavation safety, trenches with vertical walls up to 5 feet in depth may be occupied. Where very moist soil conditions or groundwater is encountered, or 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. Sloping of the sides at 1H:1V (45 degrees) in *Type B* soils may be used as an alternative to shoring or shielding.

5.2.4 Structural Fill and Compaction

All fill placed for the support of structures, flatwork or pavements, should consist of structural fill. Structural fill may consist of the on-site native fine-grained soils or an approved imported material. The native fine-grained soils were typically observed to be moisture sensitive and it can be difficult to achieve the desired compaction and moisture content; therefore, it may be more economical to import material that will require less effort. Imported soil used as structural fill should be a relatively well-graded granular soil with a maximum of 50 percent passing the No. 4 sieve and a maximum fines content (minus No.200 mesh sieve) of 15 percent. Structural fill should be free of vegetation and debris, and contain no rocks larger than 4 inches in nominal size

(6 inches in greatest dimension). Topsoil may not be used as structural fill; this material must be kept segregated from other soils intended to be used as structural fill.

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. These values are *maximums*; the Contractor should be aware that thinner lifts may be necessary to achieve the required compaction criteria. We recommend that all structural fill be compacted on a horizontal plane, unless otherwise approved by IGES. Structural fill placed beneath footings and pavements should be compacted to at least 95 percent of the maximum dry density (MDD) as determined by ASTM D-1557. The moisture content should be at or slightly above the optimum moisture content (OMC) for all structural fill – compacting dry of optimum is discouraged. Any imported fill materials should be approved by IGES prior to importing. Also, prior to placing any fill, the excavations should be observed by IGES to confirm that unsuitable materials 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.

All utility trenches backfilled below pavement sections, curb and gutter and concrete flatwork, should be backfilled with structural fill compacted to at least 95 percent of the MDD as determined by ASTM D-1557. All other trenches, including landscape areas, should be backfilled and compacted to a minimum of 90 percent of the MDD (ASTM D-1557).

Backfill around foundation walls should be placed in 10-inch loose lifts or thinner and compacted to 90 percent of the MDD at or slightly above the OMC as determined by ASTM D1557. Failure to properly moisture-condition and compact foundation wall backfill may result in settlements of up to several inches. Only small compaction equipment should be used near basement walls such as jumping jacks and walk-behind/remote controlled compactors. If possible, backfill placement against foundation walls should not be completed until floor joists are in place or the basement walls are braced.

Specifications from governing authorities having their own precedence for backfill and compaction should be followed where applicable.

5.2.5 Soft Soil Stabilization

Due to the presence of the moist fine-grained native soils, soft and/or pumping soils may be encountered. If soft soils become problematic, stabilization of soft or pumping subgrade should be accomplished by using a clean, coarse angular material worked into the soft subgrade. We recommend the material be greater than 3 inches in nominal diameter, but less than 6 inches. Alternately, a locally available pit-run gravel may be suitable but should contain a high percentage of particles larger than 3 inches diameter and have less than 5 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 will likely require more material be placed. The stabilization material should be worked (pushed) into the soft subgrade soils until a relatively firm and unyielding surface is established. Once a relatively firm and unyielding surface is achieved, the area may be brought to final design grade using structural fill. Other earth materials not meeting aforementioned criteria may also be suitable; however, such material should be evaluated on a case-by-case basis and should be approved by IGES prior to use.

The placement of a woven geotextile and compacted structural fill may be used as an alternative or in conjunction to the procedures previously described to stabilize soft soils. The woven geotextile should consist of Mirafi 500X or approved equivalent. The geotextile should be placed to cover the entire excavation bottom where structural fill will be placed. The geotextile should be installed in accordance with the manufacturer's recommendations; seams should be overlapped a minimum of 12 inches. Following placement of the geotextile, compacted structural fill may be placed to the required grade.

5.3 FOUNDATIONS

Based on our field observations and considering the presence of relatively competent native earth materials, we recommend that footings be established *entirely* on undisturbed native non-collapsible soils (no pinholes) or *entirely* on structural fill extending to undisturbed native non-collapsible soils. Footings should be extended a minimum of 5 feet in depth below existing site grade to get below the potentially collapsible soils. Native/fill transition zones must be avoided. If soft, loose, porous, potentially collapsible, or otherwise deleterious earth materials are exposed in the footing excavations, then the footings should be deepened further such that all footings bear on relatively uniform, competent native earth materials. All footing excavations should be observed by IGES or other qualified geotechnical engineer prior to constructing footings.

Shallow spread or continuous wall footings constructed as described above may be proportioned utilizing a maximum net allowable bearing pressure of **1,200 pounds per square foot (psf)** for dead load plus live load conditions. A one-third increase may be used for transient wind and seismic loads. If required, all fill beneath the foundations should consist of structural fill/reworked native soils and should be placed and compacted in accordance with our recommendations contained in Section 5.2.3 of this report.

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

5.4 SETTLEMENT

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

5.5 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 0.40 for *granular structural fill* should be used.

Ultimate lateral earth pressures from *granular structural fill* backfill acting against retaining walls and buried structures may be computed from the lateral pressure coefficients or equivalent fluid densities presented in Table 5.5. The coefficients and densities presented in Table 5.5 assume no buildup of hydrostatic pressures. The force of the water should be added to the presented values if hydrostatic pressures are anticipated.

Table 5.5
Recommended Lateral Earth Pressure Coefficients

Condition	Level Backfill	
	Lateral Pressure Coefficient	**Equivalent Fluid Density (pcf)**
Active (Ka)	0.33	42
At-rest (Ko)	0.50	63
Passive (Kp)	3.0	375

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

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.

5.6 CONCRETE SLAB-ON-GRADE CONSTRUCTION

To minimize settlement and cracking of slabs, and to aid in drainage beneath the concrete floor slabs, all concrete slabs should be founded on a minimum 4-inch layer of compacted gravel overlying undisturbed suitable native subgrade soils. The gravel should consist of free draining gravel with a 3/4-inch maximum particle size and no more than 5 percent passing the No. 200 mesh sieve.

All concrete slabs should be designed to minimize cracking as a result of shrinkage. Consideration should be given to reinforcing the slab with a welded wire fabric, re-bar, or fibermesh. Slab reinforcement should be designed by the structural engineer. We recommend that concrete be tested to assess that the slump and/or air content is in compliance with the plans and specifications. If slump and/or air content are measured above the recommendations contained in the plans and specifications, the concrete may not perform as desired. We recommend that concrete be placed in general accordance with the requirements of the American Concrete Institute (ACI).

5.7 MOISTURE PROTECTION AND SURFACE DRAINAGE

As part of good construction practices, moisture should not be allowed to infiltrate into the soils in the vicinity of the foundations. As such, design strategies to minimize ponding and infiltration near the structure should be implemented as follows:

- Hand watering, desert or Xeriscape landscaping should be completed within 5 feet of the foundations.
- Rain gutters should be installed around the entire perimeter of the homes and discharge a minimum of 10 feet away from the structure.
- Irrigation valves should be placed a minimum of 5 feet from foundations and must be placed beyond the limits of foundation backfill.
- The ground surface within 10 feet of the structure should be constructed so as to slope a minimum of five percent away.
- Pavement sections should be constructed to divert surface water off of the pavement into storm drains.

5.8 PRELIMINARY SOIL CORROSION POTENTIAL

To evaluate the corrosion potential of concrete in contact with onsite native soil, a representative soil sample was tested in our soils laboratory for soluble sulfate content. Laboratory test results indicate that the sample tested had a sulfate content of 35.7 ppm. Based on this result, the onsite native soils are expected to exhibit a *moderate* potential for sulfate attack on concrete. A conventional Type I/II cement should be used for all concrete in contact with site soils.

To evaluate the corrosion potential of ferrous metal in contact with onsite native soil, a representative soil sample was tested in our soils laboratory for soil resistivity (AASHTO T288), chloride content, and pH. The tests indicated that the onsite soil tested has minimum soil resistivity of 1601 OHM-cm, a chloride content of 49.9 ppm, and a pH value of 7.32. Based on these results, the onsite native soil is considered to be *very corrosive* when in contact with ferrous metal. Consideration should be given to retaining the services of a qualified corrosion engineer to provide an assessment of any metal such as ancillary water lines, reinforcing steel, valves, and similar improvements in contact with native soils.

6.0 CLOSURE

6.1 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, IGES 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.

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.

6.2 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. IGES staff should be on site to verify compliance with these recommendations. These tests and observations should include, but not necessarily be limited to, the following:

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

We also recommend that project plans and specifications be reviewed by us to verify compatibility with our conclusions and recommendations. Additional information concerning the scope and cost of these services can be obtained from our office.

We appreciate the opportunity to be of service on this project. Should you have any questions regarding the report or wish to discuss additional services, please do not hesitate to contact us at your convenience at (801) 748-4044.

7.0 REFERENCES CITED

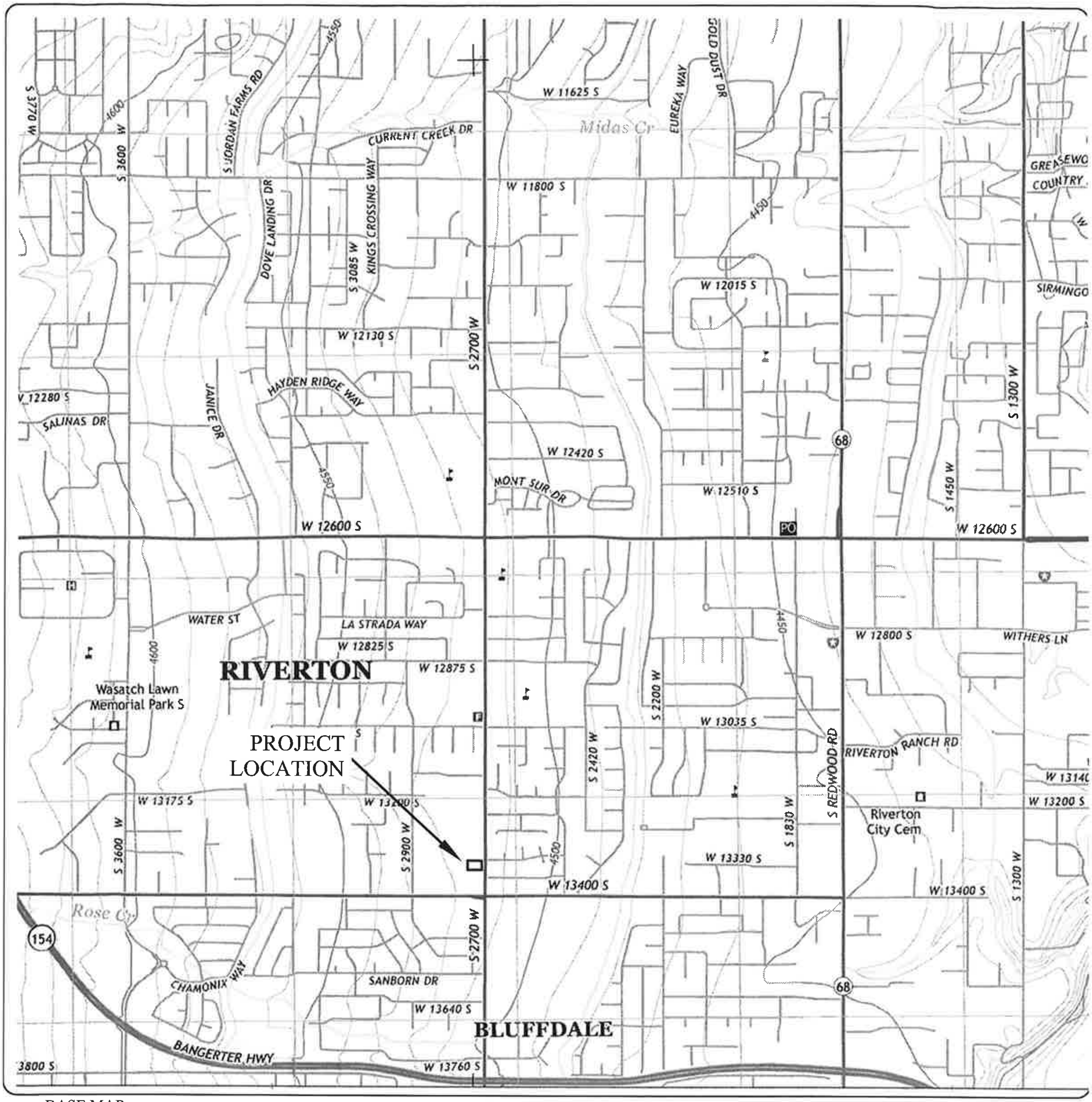
International Building Code [IBC], 2012, International Code Council, Inc.

United States Geological Survey, Midvale, Utah, Quadrangle Map 7.5 Minute Series.

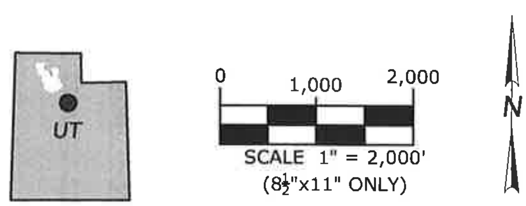
U.S. Geological Survey, 2012, U.S. *Seismic "Design Maps" Web Application*, site: https://geohazards.usgs.gov/secure/designmaps/us/application.php.

Utah Geological Survey, 1994, "Liquefaction-Potential Map for a Part of Salt Lake County, Utah", Public Information Series 28.

APPENDIX A



BASE MAP:
USGS MIDVALE, UTAH
7.5-MINUTE QUADRANGLE TOPOGRAPHIC MAP (2014)



IGES®

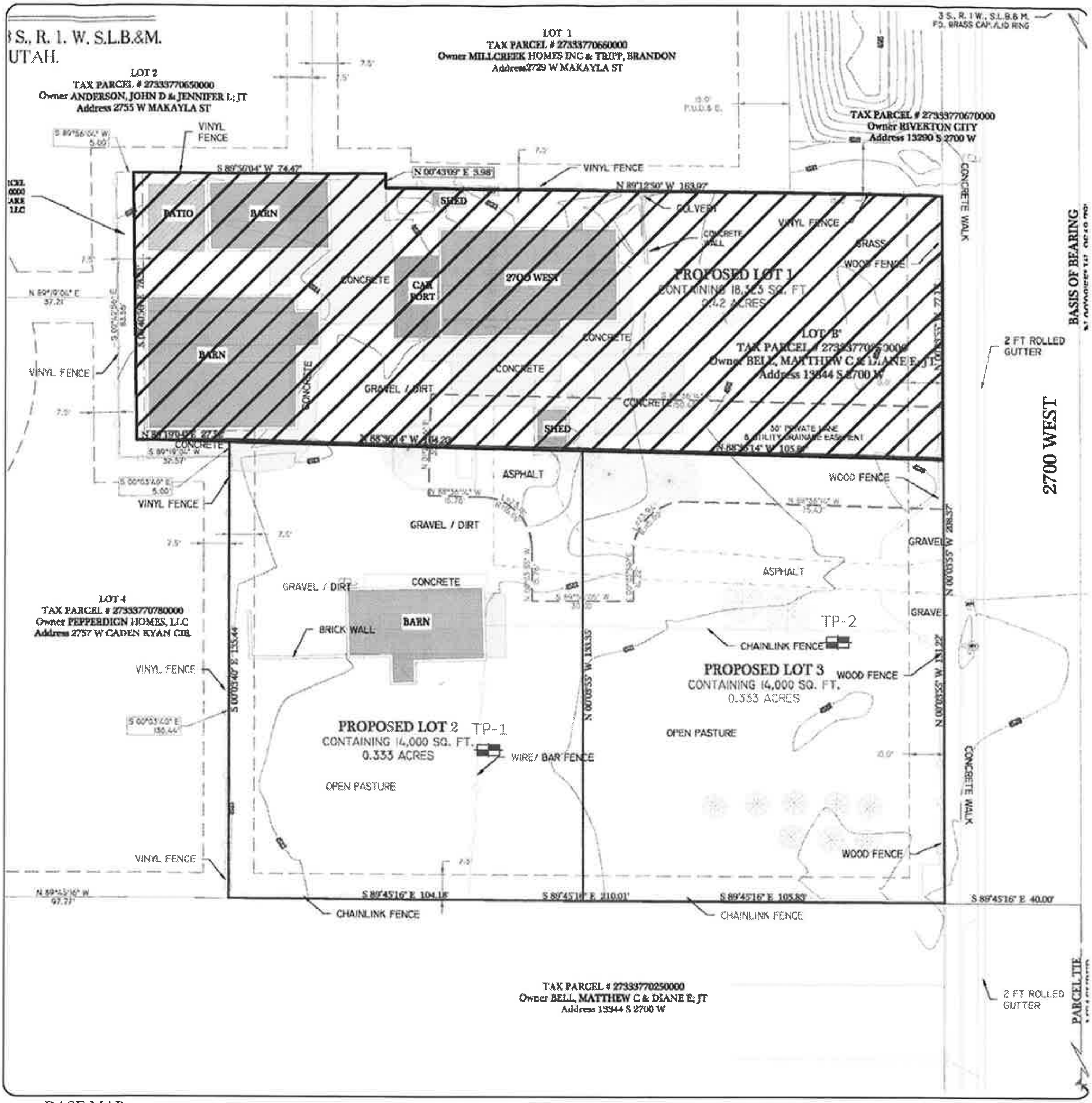
PROJECT NO: 02398-001

GEOTECHNICAL INVESTIGATION
3 TREE SUBDIVISION
13344 SOUTH 2700 WEST
RIVERTON, UTAH

SITE VICINITY MAP

FIGURE

A-1

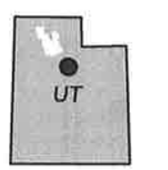


BASE MAP:
PROVIDED BY CLIENT

LEGEND:

APPROXIMATE TEST PIT LOCATION

LOT 1 NOT PART OF INVESTIGATION



0 20 40
SCALE 1" = 40'
(8½"x11" ONLY)

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PROJECT NO: 02398-001

GEOTECHNICAL INVESTIGATION
3 TREE SUBDIVISION
13344 SOUTH 2700 WEST
RIVERTON, UTAH

GEOTECHNICAL MAP

FIGURE

A-2

DATE	
STARTED:	9/19/16
COMPLETED:	9/19/16
BACKFILLED:	9/19/16

Geotechnical Investigation
3 Tree Subdivision
13344 South 2700 West
Riverton, Utah

Project Number 02389-001

IGES Rep: JWW

Rig Type: Kubota KX 121-3

TEST PIT NO: **TP-1**

Sheet 1 of 1

LOCATION

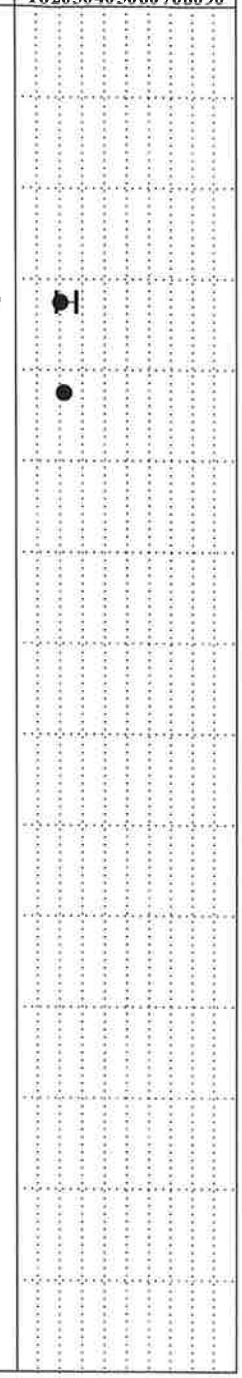
LATITUDE | LONGITUDE | ELEVATION

ELEVATION	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					Undocumented Fill - Lean CLAY - medium stiff, slightly moist, dark brown - occasional debris					
					CL	Lean CLAY - stiff, moist, medium brown - trace fine pinholes					
		Grab sample						19.6		27	9
		3" O.D. thin-walled hand sampler					83.4	21.3			
	5					- no pinholes below 5 feet					
						- stiff to medium stiff, moist, medium brown - no pinholes					
		Grab sample							94.2		
	10					No groundwater encountered Bottom of test pit @ 10 Feet					

Moisture Content and Atterberg Limits

Plastic Limit — Moisture Content — Liquid Limit

10 20 30 40 50 60 70 80 90



IGES®

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

WATER LEVEL
- ▼- MEASURED
- ▽- ESTIMATED

NOTES:

Figure A-3

DATE	
STARTED:	9/19/16
COMPLETED:	9/19/16
BACKFILLED:	9/19/16

Geotechnical Investigation
3 Tree Subdivision
13344 South 2700 West
Riverton, Utah

Project Number 02389-001

IGES Rep: JWW

Rig Type: Kubota KX 121-3

TEST PIT NO: **TP-2**

Sheet 1 of 1

LOCATION: LATITUDE | LONGITUDE | ELEVATION

DEPTH (FEET)	SAMPLES	UNIFIED SOIL CLASSIFICATION	MATERIAL DESCRIPTION	Dry Density(pcf)	Moisture Content %	Percent minus 200	Liquid Limit	Plasticity Index
0			Undocumented Fill - Lean CLAY - medium stiff, moist, dark brown - occasional debris					
		CL	Lean CLAY - medium stiff, moist, medium brown - no pinholes					
			Sandy Lean CLAY - medium stiff, moist, medium brown					
5	Grab sample					76.5		
	3" O.D. thin-walled hand sampler			85.2	33.6			
10	Grab sample				32.6		30	10
10.5			No groundwater encountered					
			Bottom of test pit @ 10.5 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:

Figure A-4

IGES

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 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 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 PENETRATE 12" WITH 1/2-INCH REINFORCING ROD DRIVEN WITH 5-LB HAMMER
VERY DENSE	>50	>60	>70	85 - 100	PENETRATED ONLY 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.

APPENDIX B

Liquid Limit, Plastic Limit, and Plasticity Index of Soils
(ASTM D4318)

IGES

Project: Riverton 3-Lot Subdivision
No: 02398-001
Location: Riverton, UT
Date: 10/4/2016
By: BRR

Boring No.: TP-1
Sample:
Depth: 3.0'
Description: Brown lean clay

Preparation method: Wet
Liquid limit test method: Multipoint

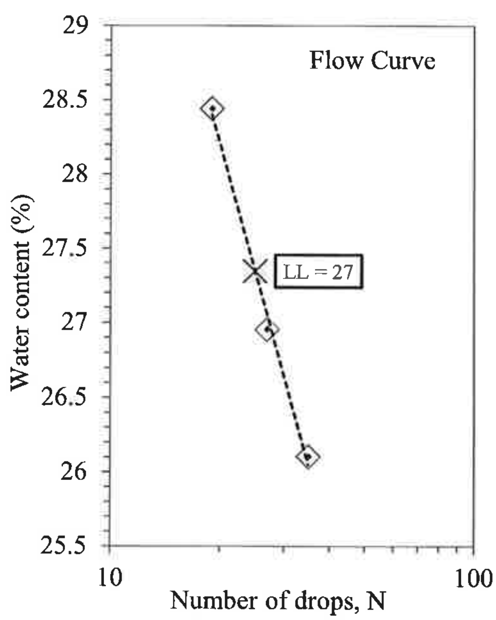
Plastic Limit

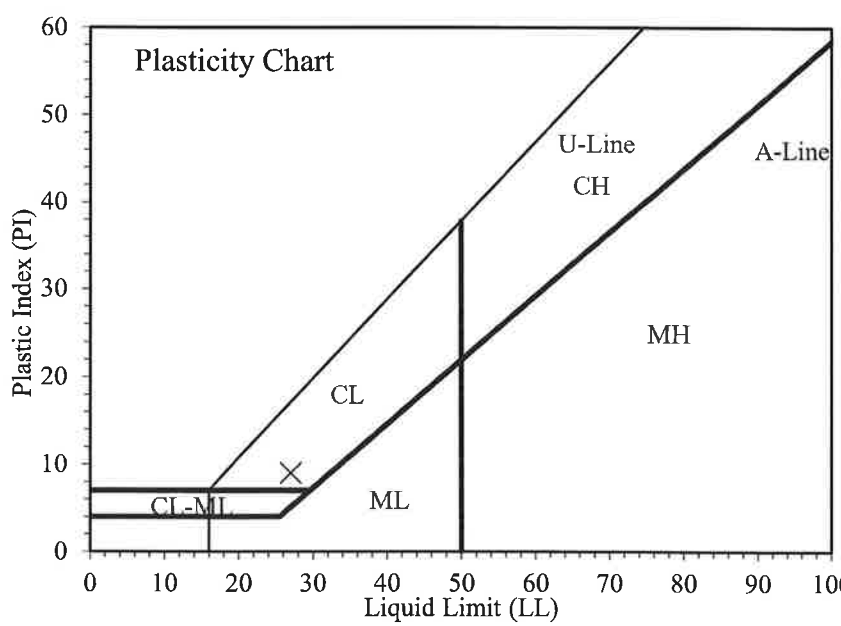
Determination No	1	2				
Wet Soil + Tare (g)	29.82	29.05				
Dry Soil + Tare (g)	28.65	27.98				
Water Loss (g)	1.17	1.07				
Tare (g)	21.99	22.01				
Dry Soil (g)	6.66	5.97				
Water Content, w (%)	17.57	17.92				

Liquid Limit

Determination No	1	2	3			
Number of Drops, N	35	27	19			
Wet Soil + Tare (g)	31.30	30.98	30.25			
Dry Soil + Tare (g)	29.35	29.05	28.35			
Water Loss (g)	1.95	1.93	1.90			
Tare (g)	21.88	21.89	21.67			
Dry Soil (g)	7.47	7.16	6.68			
Water Content, w (%)	26.10	26.96	28.44			
One-Point LL (%)		27				

Liquid Limit, LL (%)	**27**
Plastic Limit, PL (%)	**18**
Plasticity Index, PI (%)	**9**





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Reviewed:____________

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Liquid Limit, Plastic Limit, and Plasticity Index of Soils

(ASTM D4318)

IGES

Project: Riverton 3-Lot Subdivision
No: 02398-001
Location: Riverton, UT
Date: 10/4/2016
By: BRR

Boring No.: TP-2
Sample:
Depth: 10.0'
Description: Brown lean clay

Preparation method: Wet
Liquid limit test method: Multipoint

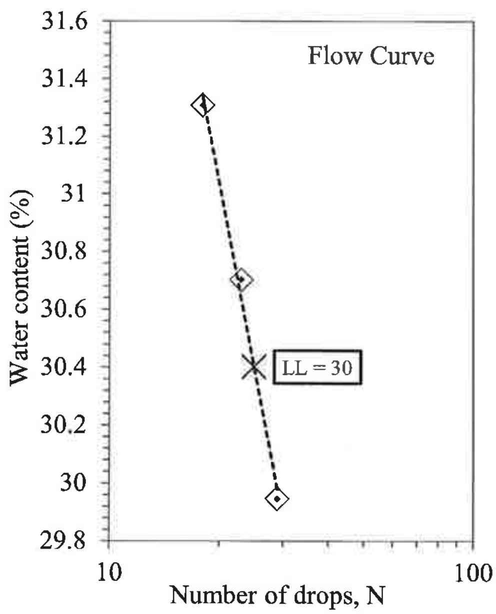
Plastic Limit

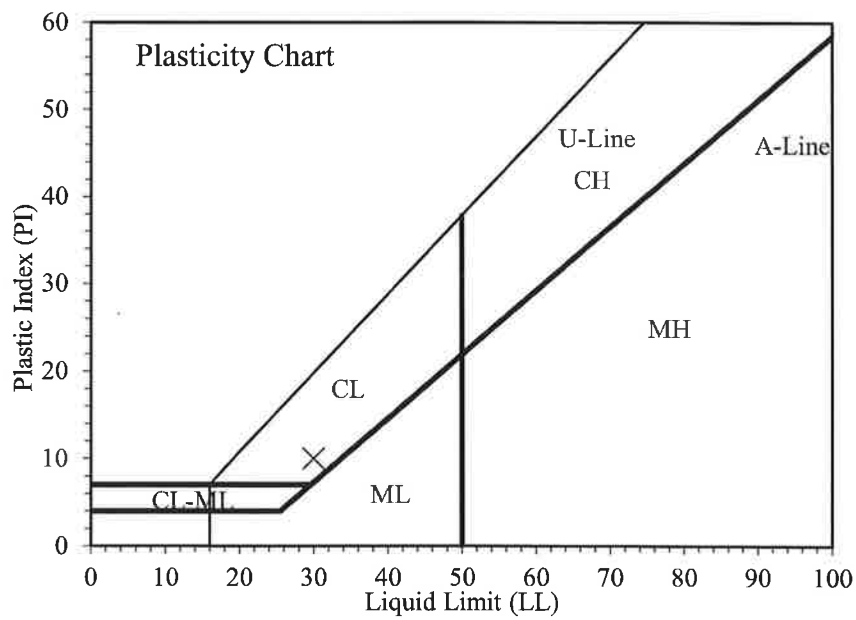
Determination No	1	2				
Wet Soil + Tare (g)	28.61	27.71				
Dry Soil + Tare (g)	27.50	26.70				
Water Loss (g)	1.11	1.01				
Tare (g)	21.90	21.59				
Dry Soil (g)	5.60	5.11				
Water Content, w (%)	19.82	19.77				

Liquid Limit

Determination No	1	2	3			
Number of Drops, N	29	23	18			
Wet Soil + Tare (g)	31.20	31.31	31.67			
Dry Soil + Tare (g)	28.99	29.17	29.35			
Water Loss (g)	2.21	2.14	2.32			
Tare (g)	21.61	22.20	21.94			
Dry Soil (g)	7.38	6.97	7.41			
Water Content, w (%)	29.95	30.70	31.31			
One-Point LL (%)	30	30				

Liquid Limit, LL (%)	**30**
Plastic Limit, PL (%)	**20**
Plasticity Index, PI (%)	**10**





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Collapse/Swell Potential of Soils

(ASTM D4546 Method B)

IGES

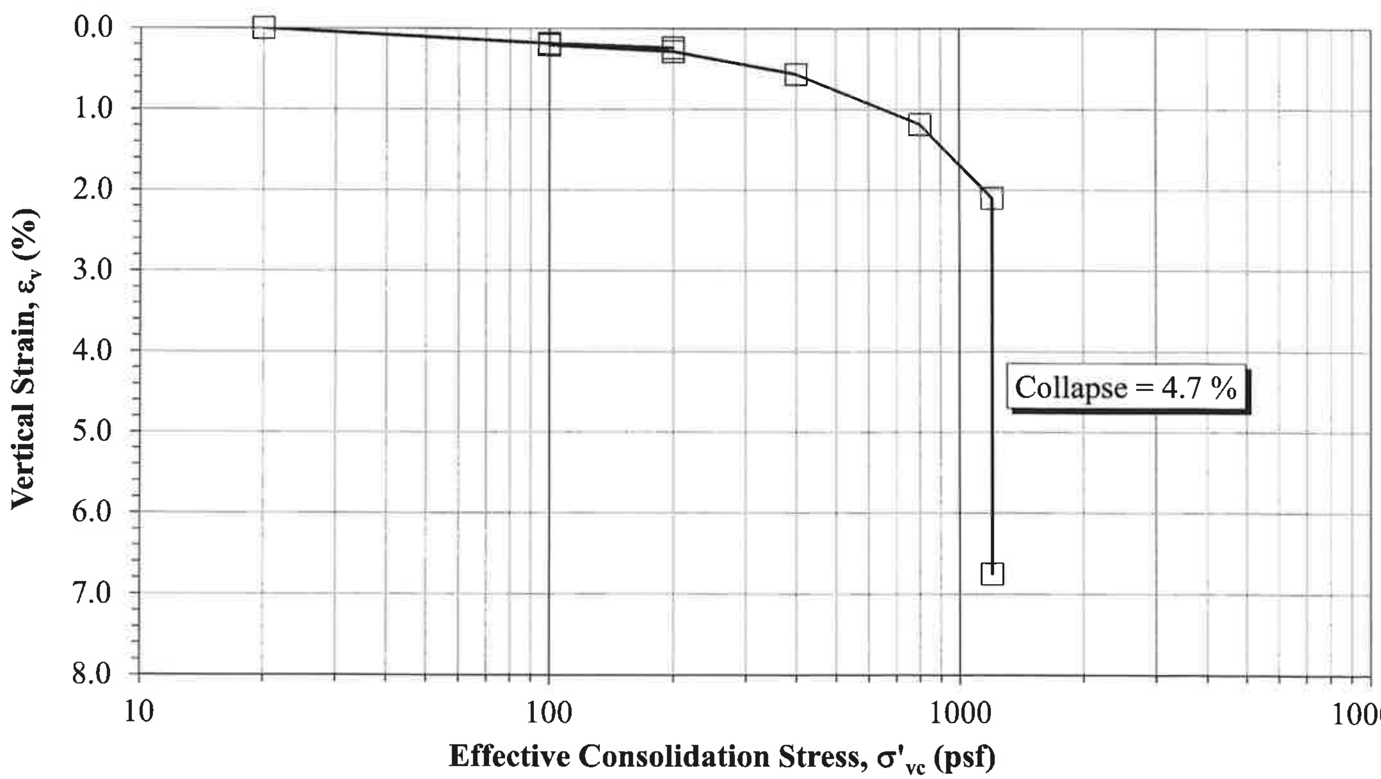
Project: **Riverton 3-Lot Subdivision**
No: **02398-001**
Location: Riverton, UT
Date: 10/5/2016
By: JDF

Boring No.: **TP-1**
Sample:
Depth: **4.0'**
Sample Description: Brown clay
Engineering Classification: Not requested
Sample type: Undisturbed-trimmed from thin-wall

Consolidometer No.:	2	
Specific gravity, G_s	2.70	Assumed
Collapse (%)	4.7	
Collapse stress (psf)	1200	
Water type used for inundation	Tap	

	Initial (o)	Final (f)
Sample height, H (in.)	0.920	0.8579
Sample diameter, D (in.)	2.416	2.416
Mass rings + wet soil (g)	154.14	164.68
Mass rings/tare (g)	42.17	42.17
Moist unit wt., γ_m (pcf)	101.13	118.67
Wet soil + tare (g)	245.79	264.75
Dry soil + tare (g)	223.70	236.82
Tare (g)	120.01	151.48
Water content, w (%)	21.3	32.7
Dry unit wt., γ_d (pcf)	83.37	89.41
Saturation	56.30	99.82

Stress (psf)	Dial (in.)	1-D ε_v (%)	H_c (in.)	e
Seating	0.2276	0.00	0.9200	1.022
20	0.2276	0.00	0.9200	1.022
100	0.2293	0.18	0.9183	1.018
200	0.2298	0.24	0.9178	1.017
100	0.2295	0.21	0.9181	1.017
200	0.2302	0.28	0.9174	1.016
400	0.2328	0.57	0.9148	1.010
800	0.2385	1.18	0.9091	0.998
1200	0.2469	2.10	0.9007	0.979
1200	0.2897	6.75	0.8579	0.885



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Reviewed: ________

Unconsolidated-Undrained Triaxial Compression Test on Cohesive Soils

(ASTM D2850)

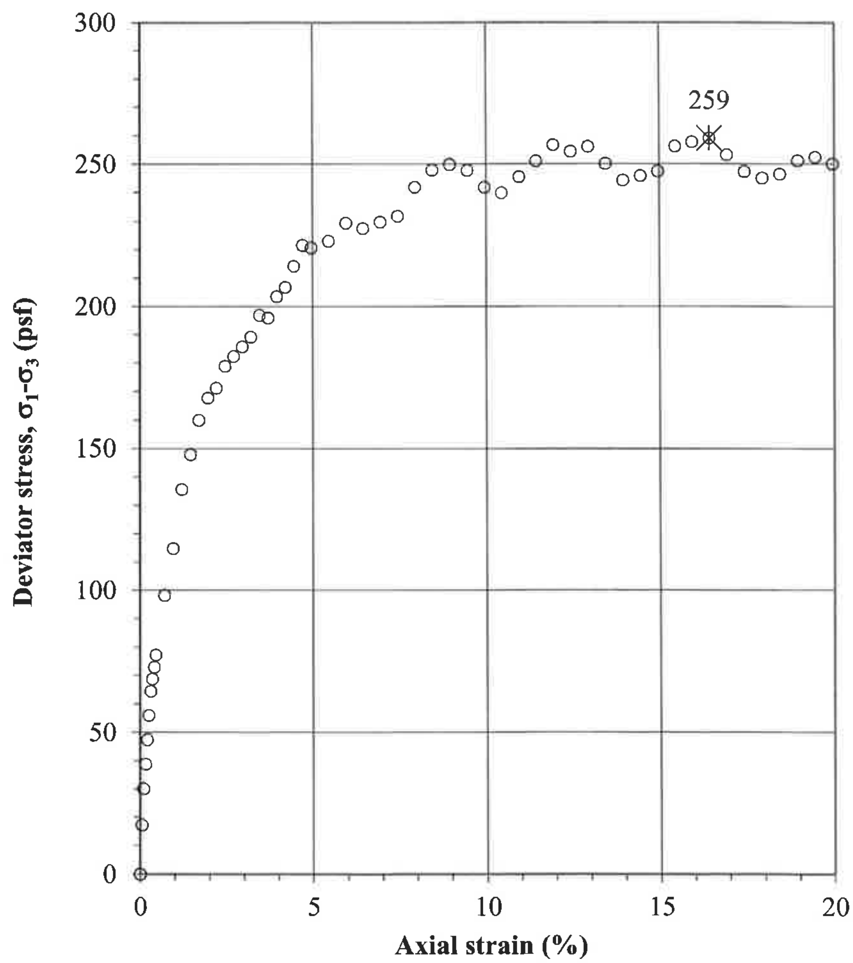
IGES

Project:	**Riverton 3-Lot Subdivision**	**Boring No.:**	**TP-2**
No:	**02398-001**	**Sample:**	
Location:	Riverton, UT	**Depth:**	**6.0'**
Date:	9/28/2016	Sample Description:	Brown sandy clay
By:	JDF	Sample type:	Undisturbed

Specific gravity, Gs	2.70	Assumed
Sample height, H (in.)	5.670	
Sample diameter, D (in.)	2.374	
Sample volume, V (ft^3)	0.0145	
Wt. rings + wet soil (g)	1508.41	
Wt. rings/tare (g)	758.28	
Moist soil, Ws (g)	750.13	
Moist unit wt., γ_m (pcf)	113.9	
Dry unit wt., γ_d (pcf)	**85.2**	
Saturation (%)	92.4	
Void ratio, e	0.98	

Wet soil + tare (g)	873.51
Dry soil + tare (g)	686.15
Tare (g)	128.01
Water content, w (%)	**33.6**
Confining stress, σ_3 (psf)	300
Shear rate (in/min)	0.0170
Strain at failure, ε_f (%)	16.45
Deviator stress at failure, $(\sigma_1-\sigma_3)_f$ (psf)	259
Shear stress at failure, $q_f = (\sigma_1-\sigma_3)_f/2$ (psf)	130

Axial Strain (%)	σ_d $\sigma_1-\sigma_3$ (psf)	Q 1/2 σ_d (psf)
0.00	0.0	0.0
0.05	17.3	8.6
0.10	30.2	15.1
0.15	38.8	19.4
0.20	47.4	23.7
0.25	56.0	28.0
0.30	64.5	32.3
0.35	68.7	34.4
0.40	73.0	36.5
0.45	77.2	38.6
0.70	98.2	49.1
0.95	114.9	57.4
1.20	135.7	67.8
1.45	147.9	73.9
1.70	160.0	80.0
1.95	167.8	83.9
2.20	171.3	85.6
2.45	179.0	89.5
2.70	182.4	91.2
2.95	185.9	92.9
3.20	189.3	94.6
3.45	196.8	98.4
3.70	196.0	98.0
3.95	203.5	101.8
4.20	206.8	103.4
4.45	214.3	107.1
4.70	221.7	110.8
4.95	220.8	110.4
5.45	223.1	111.5
5.95	229.4	114.7
6.45	227.5	113.8
6.95	229.7	114.9
7.45	231.8	115.9
7.95	241.9	120.9
8.45	247.9	123.9
8.95	249.8	124.9
9.45	247.8	123.9
9.95	241.8	120.9
10.45	239.8	119.9
10.95	245.5	122.8
11.45	251.1	125.6
11.95	256.7	128.3
12.45	254.5	127.3
12.95	256.1	128.1
13.45	250.2	125.1
13.95	244.3	122.2
14.45	245.9	123.0
14.95	247.5	123.7
15.45	256.3	128.2
15.95	257.7	128.9
16.45	259.1	129.5
16.95	253.2	126.6
17.45	247.4	123.7
17.95	245.1	122.6
18.45	246.4	123.2
18.95	251.2	125.6
19.45	252.4	126.2
19.95	250.1	125.0



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Minimum Laboratory Soil Resistivity, pH of Soil for Use in Corrosion Testing, and Ions in Water by Chemically Suppressed Ion Chromatography *(AASHTO T 288, T 289, ASTM D4327, and C1580)*

IGES

Project: Riverton 3-Lot Subdivision
No: 02398-001
Location: Riverton, UT
Date: 10/5/2016
By: ET

Sample info.	Boring No.	TP-2							
	Sample								
	Depth	5.0'							
Water content data	Wet soil + tare (g)	88.18							
	Dry soil + tare (g)	82.61							
	Tare (g)	38.21							
	Water content (%)	12.5							
Chem. data	pH	7.32							
	Soluble chloride* (ppm)	49.9							
	Soluble sulfate** (ppm)	35.7							
Resistivity data	Pin method	2							
	Soil box	Miller Small							
		Approximate Soil condition (%)	Resistance Reading (Ω)	Soil Box Multiplier (cm)	Resistivity (Ω-cm)	Approximate Soil condition (%)	Resistance Reading (Ω)	Soil Box Multiplier (cm)	Resistivity (Ω-cm)
		As Is	16470	0.67	11035				
		+3	4520	0.67	3028				
		+6	2540	0.67	1702				
		+9	2475	0.67	1658				
		+12	2390	0.67	1601				
		+15	2497	0.67	1673				
	Minimum resistivity (Ω-cm)	**1601**							

* Performed by AWAL using EPA 300.0

** Performed by AWAL using ASTM C1580

Entered by:___________

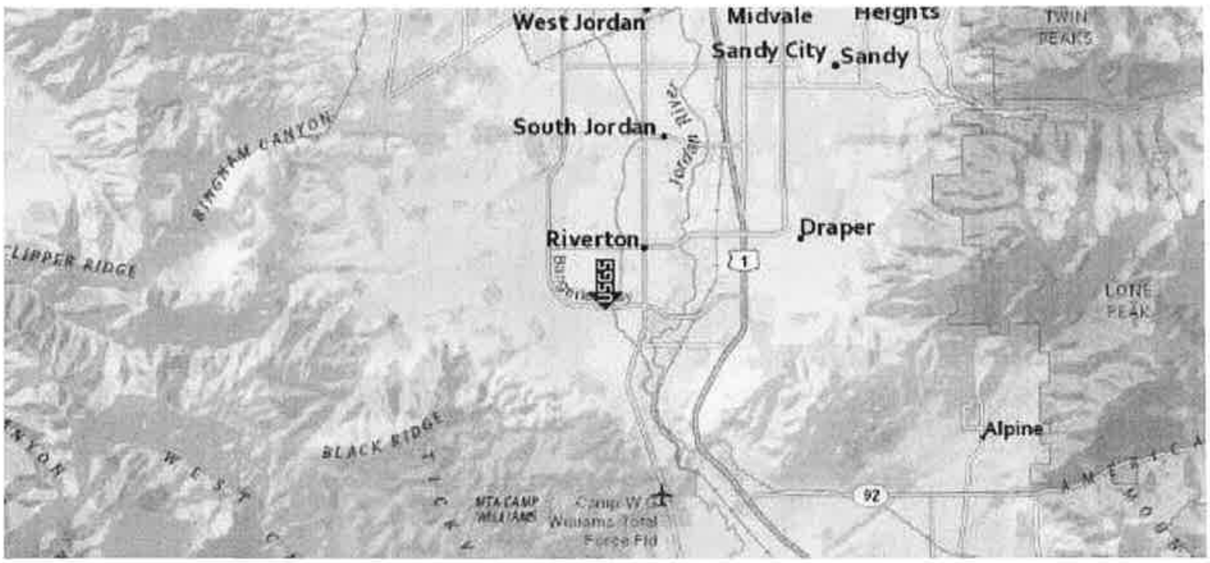
Reviewed:___________

APPENDIX C

USGS Design Maps Summary Report

User–Specified Input

Report Title	3 Tree Subdivision Mon October 17, 2016 20:44:29 UTC
Building Code Reference Document	2012/2015 International Building Code (which utilizes USGS hazard data available in 2008)
Site Coordinates	40.50904°N, 111.95805°W
Site Soil Classification	Site Class D – "Stiff Soil"
Risk Category	I/II/III

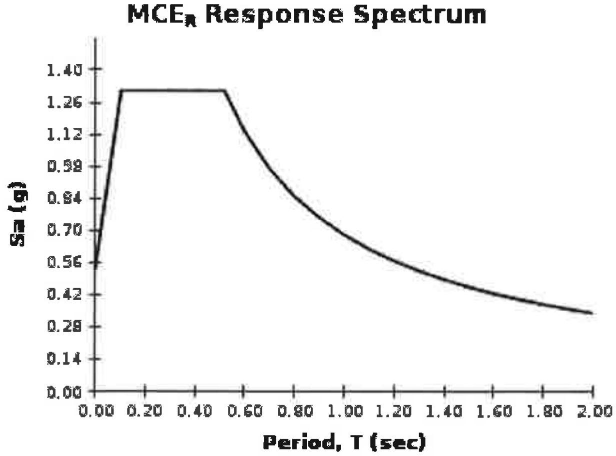


USGS–Provided Output

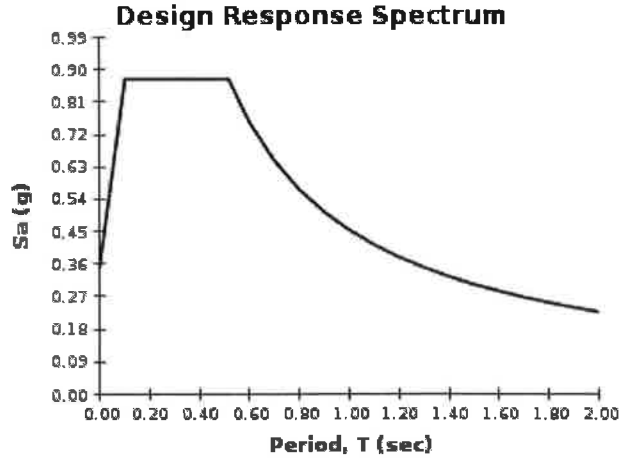
$S_S =$	1.309 g	$S_{MS} =$	1.309 g	$S_{DS} =$	0.873 g
$S_1 =$	0.435 g	$S_{M1} =$	0.681 g	$S_{D1} =$	0.454 g

For information on how the SS and S1 values above have been calculated from probabilistic (risk-targeted) and deterministic ground motions in the direction of maximum horizontal response, please return to the application and select the "2009 NEHRP" building code reference document.

MCE_R Response Spectrum



Design Response Spectrum



USGS Design Maps Detailed Report

2012/2015 International Building Code (40.50904°N, 111.95805°W)

Site Class D – "Stiff Soil", Risk Category I/II/III

Section 1613.3.1 — Mapped acceleration parameters

Note: Ground motion values provided below are for the direction of maximum horizontal spectral response acceleration. They have been converted from corresponding geometric mean ground motions computed by the USGS by applying factors of 1.1 (to obtain S_S) and 1.3 (to obtain S_1). Maps in the 2012/2015 International Building Code are provided for Site Class B. Adjustments for other Site Classes are made, as needed, in Section 1613.3.3.

From Figure 1613.3.1(1) [1]	S_S = 1.309 g
From Figure 1613.3.1(2) [2]	S_1 = 0.435 g

Section 1613.3.2 — Site class definitions

The authority having jurisdiction (not the USGS), site-specific geotechnical data, and/or the default has classified the site as Site Class D, based on the site soil properties in accordance with Section 1613.

2010 ASCE-7 Standard – Table 20.3-1
SITE CLASS DEFINITIONS

Site Class	$\bar{v}_s$	$\bar{N}$ or $\bar{N}_{ch}$	$\bar{s}_u$
A. Hard Rock	>5,000 ft/s	N/A	N/A
B. Rock	2,500 to 5,000 ft/s	N/A	N/A
C. Very dense soil and soft rock	1,200 to 2,500 ft/s	>50	>2,000 psf
D. Stiff Soil	600 to 1,200 ft/s	15 to 50	1,000 to 2,000 psf
E. Soft clay soil	<600 ft/s	<15	<1,000 psf
	Any profile with more than 10 ft of soil having the characteristics: • Plasticity index *PI* > 20, • Moisture content *w* ≥ 40%, and • Undrained shear strength $\bar{s}_u$ < 500 psf		
F. Soils requiring site response analysis in accordance with Section 21.1	See Section 20.3.1		

For SI: 1ft/s = 0.3048 m/s 1lb/ft² = 0.0479 kN/m²

Section 1613.3.3 — Site coefficients and adjusted maximum considered earthquake spectral response acceleration parameters

TABLE 1613.3.3(1)
VALUES OF SITE COEFFICIENT F_a

Site Class	Mapped Spectral Response Acceleration at Short Period				
	$S_S \leq 0.25$	$S_S = 0.50$	$S_S = 0.75$	$S_S = 1.00$	$S_S \geq 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	See Section 11.4.7 of ASCE 7				

Note: Use straight-line interpolation for intermediate values of S_S

For Site Class = D and S_S = 1.309 g, F_a = 1.000

TABLE 1613.3.3(2)
VALUES OF SITE COEFFICIENT F_v

Site Class	Mapped Spectral Response Acceleration at 1-s Period				
	$S_1 \leq 0.10$	$S_1 = 0.20$	$S_1 = 0.30$	$S_1 = 0.40$	$S_1 \geq 0.50$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	2.4
F	See Section 11.4.7 of ASCE 7				

Note: Use straight-line interpolation for intermediate values of S_1

For Site Class = D and S_1 = 0.435 g, F_v = 1.565

Equation (16-37): $S_{MS} = F_a S_S = 1.000 \text{ x } 1.309 = 1.309 \text{ g}$

Equation (16-38): $S_{M1} = F_v S_1 = 1.565 \text{ x } 0.435 = 0.681 \text{ g}$

Section 1613.3.4 — Design spectral response acceleration parameters

Equation (16-39): $S_{DS} = ⅔\, S_{MS} = ⅔ \text{ x } 1.309 = 0.873 \text{ g}$

Equation (16-40): $S_{D1} = ⅔\, S_{M1} = ⅔ \text{ x } 0.681 = 0.454 \text{ g}$

Section 1613.3.5 — Determination of seismic design category

TABLE 1613.3.5(1)
SEISMIC DESIGN CATEGORY BASED ON SHORT-PERIOD (0.2 second) RESPONSE ACCELERATION

VALUE OF S_{DS}	RISK CATEGORY		
	I or II	III	IV
$S_{DS} < 0.167g$	A	A	A
$0.167g \le S_{DS} < 0.33g$	B	B	C
$0.33g \le S_{DS} < 0.50g$	C	C	D
$0.50g \le S_{DS}$	D	D	D

For Risk Category = I and S_{DS} = 0.873 g, Seismic Design Category = D

TABLE 1613.3.5(2)
SEISMIC DESIGN CATEGORY BASED ON 1-SECOND PERIOD RESPONSE ACCELERATION

VALUE OF S_{D1}	RISK CATEGORY		
	I or II	III	IV
$S_{D1} < 0.067g$	A	A	A
$0.067g \le S_{D1} < 0.133g$	B	B	C
$0.133g \le S_{D1} < 0.20g$	C	C	D
$0.20g \le S_{D1}$	D	D	D

For Risk Category = I and S_{D1} = 0.454 g, Seismic Design Category = D

Note: When S_1 is greater than or equal to 0.75g, the Seismic Design Category is **E** for buildings in Risk Categories I, II, and III, and **F** for those in Risk Category IV, irrespective of the above.

Seismic Design Category ≡ "the more severe design category in accordance with Table 1613.3.5(1) or 1613.3.5(2)" = D

Note: See Section 1613.3.5.1 for alternative approaches to calculating Seismic Design Category.

References

1. *Figure 1613.3.1(1)*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/IBC-2012-Fig1613p3p1(1).pdf
2. *Figure 1613.3.1(2)*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/IBC-2012-Fig1613p3p1(2).pdf