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Geotechnical Investigation
Proposed Structure
~11800 South 4000 West
Riverton, Utah

IGES Project No. 01466-022

May 12, 2016

Prepared for:

OK

Innovative Excavation

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GEOTECHNICAL INVESTIGATION
PROPOSED STRUCTURE
~11800 SOUTH 4000 WEST
RIVERTON, UTAH

IGES Project No. 01466-022

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APPENDICES

1.0 EXECUTIVE SUMMARY

This report presents the results of our geotechnical investigation conducted for the proposed structure to be located at approximately 11800 South 4000 West, Riverton, Utah. The site is located approximately 900 feet south-southwest of the intersection of 11800 South and 4000 West streets. 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 presented in this report are incorporated into the design and construction of the project.

- Lean CLAY (CL) containing varying amounts of sand was encountered in all the test pits in the upper 9 feet; the clay was medium stiff, moist to very moist and generally contained pinholes. A 1.5-foot layer of Silty GRAVEL with sand (GM) was observed in Test Pit 1 (TP-1) at approximately 4 feet below grade and a 2-foot layer of Poorly Graded GRAVEL (GP) with sand and some cobbles was observed in TP-2 at approximately 1 foot below grade.
- Due to the low to moderate collapse potential of the near-surface native fine-grained soils (clay and silt), it is recommended that the upper 5 feet of the on-site native fine-grained soils be removed from below any foundations. All footings should bear entirely on uniform, relatively undisturbed native soils at a minimum depth 5 feet below the existing site grade; footings may also bear entirely on a zone of structural fill with uniform thickness extending to such soils.
- Shallow spread or continuous wall footings constructed on as recommended above may be proportioned utilizing a maximum net allowable bearing pressure of **1,300 pounds per square foot (psf)**. The net allowable bearing value presented above is for dead load plus live load conditions.
- In determining the frictional resistance against concrete, a coefficient of friction of 0.33 for fine-grained native soils should be used. Ultimate lateral earth pressures from on-site fine-grained backfill acting against retaining walls and buried structures may be computed from the following lateral pressure coefficients: 0.39 (active), 0.56 (at rest) and 2.56 (passive).

The recommendations presented in this report are based on the assumption that an adequate program of tests and observations will be made during the construction. IGES staff should be on site to assess compliance with these recommendations at key points.

NOTICE: 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 our geotechnical investigation conducted for the proposed structure to be located at approximately 11800 South 4000 West, Riverton, Utah. The purposes of this investigation were to assess the nature and engineering properties of the subsurface soils at the subject site and to provide recommendations for design of conventional shallow spread and continuous foundations, general site grading and design of pavement sections for construction of the proposed roadways. In addition, we have assessed the geologic hazards at 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 your signed authorization dated March 30, 2016. The recommendations contained in this report are subject to the limitations presented in the "Limitations" section of this report (Section 7.1).

2.2 PROJECT DESCRIPTION

The site is located approximately 900 feet south-southwest of the intersection of 11800 South and 4000 West streets. The site is bordered by existing residential property on the south. The land north, west and east of the property is mainly undeveloped farmlands. Midas Creek runs near the south boundary of the property. The project site is shown on the *Site Vicinity Map* included in Appendix A (Figure A-1).

Construction plans were not available for our review and our understanding of the project is based on direct communication with the client. We understand the structure will be a relatively lightly loaded single-story wood or metal framed structure without a basement and the remainder of the property will be used as a parking lot and for landscaping.

3.0 METHOD OF STUDY

3.1 SUBSURFACE INVESTIGATION

As a part of this investigation, subsurface soil conditions were explored by excavating four test pits to depths extending to 9 feet below the existing surface. The *Site Map*, Figure A-2 in Appendix A shows the approximate locations of the test pits. Exploration points were selected by the client based on the anticipated development of the site and placed to provide a representative cross section of the subsurface conditions in areas anticipated for development. Subsurface conditions as encountered in the explorations were logged at the time of our investigation by a member of our technical staff and are presented on the enclosed test pit logs, Figures A-3 through A-6 in Appendix A. A *Key to Soil Symbols and Terminology* is presented on Figure A-7.

The test pits were excavated with the aid of a mini-ex provided by the client. Both bulk and relatively "undisturbed" soil samples were obtained in the test pit explorations. Relatively "undisturbed" soil samples were obtained with the use of a hand sampler attached to a 6-inch long brass tube driven into the soil with a 2-pound sledge hammer. Disturbed bulk samples were collected and placed in either buckets or baggies. All samples were transported to our laboratory for testing to evaluate engineering properties of the various earth materials observed. The soils observed in the explorations were logged and classified in general accordance with the *Unified Soil Classification System* (USCS). Classifications for the individual soil units are shown on the attached test pit logs (Figures A-3 through A-6).

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:

- In situ moisture content (ASTM D7263 Method B and D2216)
- Atterberg Limits (ASTM D4318)
- Percent Fines (ASTM D1140)
- Maximum dry density and optimum moisture content (ASTM D698/D1557)
- CBR for pavement recommendations (ASTM D1883)
- One-Dimensional Consolidation (ASTM D2435)
- Collapse potential (ASTM D4546 Method B)
- Unconfined-Unconsolidated Triaxial Test (ASTM D2850)
- Water-soluble sulfate concentration for cement type recommendations

- Resistivity and pH to evaluate corrosion potential of ferrous metals in contact with site soils

Results of the in situ dry density, moisture content, Atterberg limits and percent fines tests are shown on the test pit logs (Appendix A). The results of remaining laboratory tests are presented on the laboratory test results figures in Appendix B.

4.0 GEOLOGIC CONDITIONS

4.1 GEOLOGIC SETTING

The site is located at an elevation of approximately 4,658 to 4,663 feet in the southwest portion of the Salt Lake Valley. The Salt Lake Valley is a deep, sediment-filled structural basin of Cenozoic age flanked by two uplifted blocks, the Wasatch Range on the east and the Oquirrh Mountains to the west (Hintze, 1980; Hintze, 1993). The northern portion of the Salt Lake Valley is bordered on the northwest by the southeast shore of the Great Salt Lake. The Wasatch Range is the easternmost expression of pronounced Basin and Range extension in north-central Utah.

The near-surface geology of the Salt Lake Valley is dominated by sediments which were deposited within the last 30,000 years by Lake Bonneville and the Great Salt Lake (Scott et al., 1983). As the lake receded, streams began to incise large deltas formed at the mouths of major canyons along the Wasatch Range, and the eroded material was deposited in shallow lakes and marshes in the basin and in a series of recessional deltas and alluvial fans. Sediments toward the center of the valley are predominately deep-water deposits of clay, silt and fine sand (Scott et al., 1983). However, these deep-water deposits are in places covered by a thin post-Bonneville alluvial cover.

Surface sediments at the site are mapped as fine-grained lacustrine deposits (Qlf) described as transgressive and regressive, deep-water sediments that are brown, dark brown, gray-brown and included laminated silt, clayey silt and sandy silt with isolated pebbles, cobbles and thin lenses of sand and gravel related to the Bonneville lake cycle from the late Pleistocene (Davis, 2000).

4.2 SEISMICITY AND FAULTING

An active fault is defined as a fault that has experienced movement with the Holocene (11,000 years before present). No active faults are mapped through or immediately adjacent to the site (Black et al., 2003). The closest mapped active fault is the Salt Lake City segment of the Wasatch Fault Zone, located about 8 miles east-southeast of the site. The most recent documented event occurred in the latest Quaternary (<15 ka). The Salt Lake Segment of the Wasatch Fault Zone has a slip rate of approximately 1-5mm/yr and an overall length of 43 km. The site is also mapped approximately 8½ miles south of the Granger Fault portion of the West Valley fault zone, a north-south trending series of faults that are mapped within the middle of the Salt Lake Valley. The last event reportedly occurred on the West Valley Fault Zone <12,000 years ago, and has a recurrence interval of 6,000 to 12,000 years. The West Valley Fault Zone trends in a north-south orientation and is located in the central portion of the Salt Lake Valley (Keaton and Curry, 1993). While the West Valley Fault Zone is reported to be active and probably seismically independent of the Wasatch Fault

Zone, sympathetic movement on the West Valley Fault Zone resulting from major earthquakes on the Wasatch Fault Zone is a possibility. Analyses of ground shaking hazard along the Wasatch Front suggests that the Wasatch fault zone is the single greatest contributor to the seismic hazard in the region. The site is also located approximately 12 miles east of the Oquirrh Fault Zone.

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 "Design Maps" 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 classification is based on the upper 100 feet of the site soil profile; based on our field exploration and our understanding of the geology in this area, the subject site can be estimated as Site Class D (*stiff soil*). Based on IBC criteria, the short-period (F_a) and long-period (F_v) site coefficients are 1.044 and 1.644, respectively. Based on the design spectral response accelerations for a *Building Risk Category* of I, II or III, the site's *Seismic Design Category* is D. The Risk-Targeted Maximum Considered Earthquake (MCE_R) *Spectral Response Accelerations* are presented in Table 4.2; a summary of the *Design Maps* analysis is presented in Appendix C. The *peak ground acceleration* (PGA) may be taken as 0.479g (ASCE 7-10, see Appendix C). It should be noted that to more accurately determine the site classification, geotechnical investigation to a minimum depth of 100 feet is needed.

4.3 OTHER GEOLOGIC HAZARDS

Geologic hazards can be defined as naturally occurring geologic conditions or processes that could present a danger to human life and property. These hazards must be considered before development of the site. There are several hazards in addition to seismicity and faulting that may be present at the site, and which should be considered in the design of roads and critical facilities such as water tanks and structures designed for human habitation. Other geologic hazards considered significant for this site include stream flooding and liquefaction.

Table 4.2
Short and 1-Second Period Spectral Accelerations

Parameter	Short Period (0.2 sec)	Long Period (1.0 sec)
MCE_R Spectral Response Acceleration Site Class B (g)	$S_S = 1.139$	$S_1 = 0.378$
MCE_R Spectral Response Acceleration Site Class D (g)	$S_{MS} = 1.189$	$S_{M1} = 0.622$
Design Spectral Response Acceleration Site Class D (g)	$S_{DS} = 0.793$	$S_{D1} = 0.414$

4.3.1 Stream Flooding

Stream flooding is a hazard related to spring snowmelt, run-off and flash-flooding from summer rainstorms. Flood hazards should be considered when planning for the development of habitable structures and essential and critical facilities located within areas having a potential flood risk. The Midas Creek runs near the south boundary of the property. The civil engineer should assess the flooding potential for the Midas Creek to impact the site property.

4.3.2 Liquefaction

Liquefaction is the loss of soil strength or stiffness due to a buildup of excess pore-water pressure during strong ground shaking. Liquefaction is associated primarily with loose (low density), granular, saturated soil. Effects of severe liquefaction can include sand boils, excessive settlement, bearing capacity failures, and lateral spreading.

The geologic hazards map titled *Liquefaction-Potential Map for a Part of Salt Lake County, Utah*, dated August 1994, indicates that the subject property is located within an area designated as having a *very low* liquefaction potential. No groundwater was encountered during our investigation in the upper 9 feet of the site; therefore, it is our opinion that the upper 9 feet of the site have a very low liquefaction potential. A liquefaction hazard study, which would include multiple borings and/or CPT soundings to depths of 50 feet, was not performed and is beyond our scope of services for this project.

5.0 GENERALIZED SITE CONDITIONS

5.1 SURFACE CONDITIONS

The site is relative flat, sloping gently down to the east. Maximum topographic relief across the site is approximately 5 feet. Midas Creek runs near the south boundary of the property. The site appears to have been primarily used for agriculture.

5.2 SUBSURFACE CONDITIONS

The subsurface soil conditions were explored at the subject property by excavating four test pits across the site. Subsurface soil conditions were logged at the time of our field investigation and are included in the test pit logs in Appendix A (Figures A-3 through A-6). The soil and moisture conditions encountered during our investigation are discussed in the following paragraphs.

5.2.1 Soils

Topsoil: The upper 1 to 1.5 feet of soil consisted of topsoil, which generally consisted of dark brown clay with occasional roots.

Native Surficial Soils: Lean CLAY (CL) containing varying amounts of sand were encountered in all the test pits in the upper 9 feet; the clay was medium stiff, moist to very moist and generally contained pinholes. A 1.5-foot layer of Silty GRAVEL with sand (GM) was observed in Test Pit 1 (TP-1) from at approximately 4 feet below grade and a 2-foot layer of Poorly Graded GRAVEL (GP) with sand and some cobbles was observed in TP-2 at approximately 1 foot below grade.

Test pit logs of the subsurface soil profiles are presented in Appendix A (Figures A-3 through A-6). 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

Groundwater was not encountered at the maximum depth (9 feet) of the test pits. It is our experience that during snowmelt, runoff, irrigation on surrounding properties, high precipitation events and other activities, the groundwater level can fluctuate several feet. We understand the proposed structure will be on-grade structure without basement and we do not anticipate the groundwater to impact the proposed structure.

5.2.3 Collapsible Soil

Collapse (often referred to as “hydro-collapse”) is a phenomenon where undisturbed soils exhibit volumetric strain upon wetting. Collapsible soils can cause differential settling of structures and roadways. Collapsible soils do not necessarily preclude development and typically can be mitigated by over-excavating porous, potentially collapsible soils and replacing with structural fill and by controlling surface drainage and runoff.

Collapse/swell tests (ASTM D4546 Method B) were performed on two relatively undisturbed samples of native clayey soil; the results are summarized in the following table:

Table 5.2.3
Summary of Collapse Test Results

Test Specimen	Load at Inundation (psf)	Collapse (%)
TP-1 @ 3 ft	1600	2.9
TP-2 @ 5 ft		0.8

The results of the tests suggest that near-surface native clayey soils will, in general, experience low to moderate volumetric strain under increased moisture conditions. The collapsible potential decreases with increasing depth based on these test results. The results of the collapse/swell tests are presented in Appendix B, Figures 4-5.

6.0 CONCLUSIONS AND RECOMMENDATIONS

6.1 GENERAL CONCLUSIONS

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

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 presented in this report are implemented into the design and construction of the project. In general, we anticipate the development can be completed using standard construction practices. We anticipate that the foundation for the proposed residential structures will consist of conventional shallow continuous or spread footings. Due to the low to moderate collapsible potential of the near-surface native fine-grained soils (clay and silt), it is recommended that the upper 5 feet of the on-site native fine-grained soils be removed from below all foundations. All footings should bear entirely on uniform, relatively undisturbed native soils at a minimum depth 5 feet below the existing site grade; footings may also bear entirely on a zone of structural fill with uniform thickness extending to such soils.

Prior to placement of footings, an IGES representative should assess the foundation subgrade for the presence of potentially collapsible soils. If potentially collapsible or otherwise deleterious soils are identified, additional over-excavation may be necessary.

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.

6.2 EARTHWORK

Prior to site improvements, general site grading is recommended to provide proper support for foundations, exterior concrete flatwork, concrete slabs-on-grade and asphalt pavement sections. Site grading is also recommended to provide proper drainage and moisture control and to aid in minimizing the potential for differential movement in foundation soils resulting from variations in moisture conditions.

6.2.1 General Site Preparation and Grading

Below proposed structures, fills, and man-made improvements, all vegetation, topsoil, debris, and undocumented fill soils should be removed. Any existing utilities should be re-routed or protected in-place. The exposed native soils should then be proof-rolled with heavy rubber-tired equipment such as a scraper or loader. Any soft/loose areas identified during proof-rolling should be removed, compacted in place or replaced with structural fill. All excavation bottoms should be observed by an IGES representative prior to placement of structural fill to evaluate whether soft, loose, or otherwise deleterious earth materials have been removed and to assess compliance with the recommendations presented herein. In addition, IGES should evaluate the prepared subgrade for the presence of soils with a potential for moisture-induced collapse (e.g., relatively low-density silt or clay with a porous soil structure, or 'pinholes').

6.2.2 Over-Excavation

Any soft, porous, or unsuitable soils identified beneath areas to receive structural fill should be over-excavated and replaced with structural fill. If over-excavation is required, the excavations should extend 1 foot laterally for every foot of depth of over-excavation, with a minimum lateral distance of 2 feet from the footings. Excavations should extend laterally at least two feet beyond flatwork, pavements and slabs-on-grade. Structural fill should be placed and compacted in accordance with the recommendations presented in this report.

Prior to placing structural fill, loose soils in the bottom of the excavations should be removed or properly compacted.

6.2.3 Temporary Excavations

The contractor is responsible for site safety, including all temporary trenches excavated at the site and design of any required temporary shoring. The contractor is responsible for providing the "competent person" required by *Occupational Safety and Health Administration* (OSHA) standards to evaluate soil conditions. Soil types are expected to consist primarily of Type B soils. Close coordination between the competent person and IGES should be maintained to facilitate construction while providing safe excavations.

Based on OSHA guidelines for excavation safety, trenches with vertical walls up to 5 feet in depth may be occupied. Where very moist soil conditions, loose granular soils or undocumented fill soils are 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 the sides at one horizontal to one vertical (1H:1V) (45 degrees) in accordance with OSHA Type B soils may be used as an alternative to shoring or shielding if only fine-grained soils are exposed in the sidewall. The preceding recommendations are for the prevailing clayey soils; where granular soils are exposed on the trench walls, stability should be evaluated on a case-by-case basis by the "competent person".

6.2.4 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 onsite native fine-grained soils or imported granular soils (see Section 6.10.2 for information regarding use of onsite fine-grained soils for structural fill). If native fine-grained soils are used as structural fill, they should be pulverized and moisture conditioned beyond the optimum moisture content (OMC) of the modified proctor (ASTM D1557) before being used as structural fill to remove the pinhole structure. Imported structural fill (if used) should be a granular material with less than 20 percent fines having an Expansion Index less than 20. Prior to use, all structural fill should be approved by IGES. Soils not meeting the aforementioned criteria may be suitable for use as structural fill; however, such material should be evaluated on a case-by-case basis and should be approved by IGES prior to use. In all cases structural fill should be relatively free of vegetation and debris, and contain no rocks larger than 4 inches in nominal size (6 inches in greatest dimension). All structural fill should be 1-inch minus material when within 1 foot of any base coarse material.

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 lift thicknesses are maximums; the Contractor should understand that thinner lifts may be necessary to achieve the required compaction. 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. During the compaction process, the moisture content should be at, or slightly above, the OMC for all structural fill. Prior to placing any fill, the subgrade should be observed by IGES to assess whether unsuitable materials have been removed and/or the subgrade has been properly prepared. 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).

6.2.5 Foundation Backfill

Backfill around foundations should consist of native soils placed in maximum 12-inch loose lifts compacted to 90 to 95 percent of the MDD and within 2 percent of the OMC (Modified Proctor, ASTM D 1557). Compacting by means of injecting water or "jetting" is not recommended. Specifications from governing authorities having their own precedence for backfill and compaction should be followed where applicable.

6.2.6 Utility Trench Backfill

Utility trenches can be backfilled with the onsite soils that are substantially free of debris, organic and oversized material. Prior to backfilling the trench, pipes should be bedded in and covered with a uniform granular material that has a Sand Equivalent (SE) of 30 or greater. Pipe bedding should *not* be water-densified in-place (jetting). Alternatively, pipe bedding and shading may consist of clean ¾-inch gravel, which generally does not require densification. Native earth materials can be used as backfill over the pipe bedding zone. All utility trenches backfilled below pavement sections, curb and gutter, and sidewalks, should be backfilled with structural fill compacted to at least 95 percent of the MDD with the moisture content at or slightly above the OMC as determined by ASTM D-1557. All other trenches, including landscape areas, should be backfilled and compacted to approximately 90 percent of the MDD with a moisture content that is within 2 percent of the OMC (ASTM D-1557). Backfill around foundations should consist of native soils placed in maximum 12-inch loose lifts compacted to 90 to 95 percent of the maximum dry density and within 2 percent of the optimum moisture content (Modified Proctor, ASTM D 1557). Compacting by means of injecting water or "jetting" is not recommended. Specifications from governing authorities having their own precedence for backfill and compaction should be followed where applicable.

6.3 FOUNDATIONS

Due to the moderate collapsible potential of the near-surface native fine-grained soils IGES recommends that the upper 5 feet of the on-site native fine-grained soils be removed from below any foundations. All footings should bear entirely on uniform, relatively undisturbed native soils a minimum of 5 feet below the existing site grade. Footings may also bear entirely on a zone of structural fill with uniform thickness extending to such soils. Native/fill transition zones are not allowed.

If required, all fill beneath the foundations should consist of structural fill and should be placed and compacted in accordance with our recommendations presented in Section 6.2.4 of this report. Shallow spread or continuous wall footings constructed as recommended above may be proportioned utilizing a maximum net allowable bearing pressure of **1,300 pounds per square foot (psf)**. The net allowable bearing value presented above is for dead load plus live load conditions.

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.

6.4 SETTLEMENT

Static settlement of properly designed and constructed conventional foundations, founded as described in Section 6.3, 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.

6.5 LATERAL EARTH PRESSURES

Lateral forces imposed upon conventional foundations due to wind or seismic forces may be resisted by the development of passive earth pressures and friction between the base of the footing and the supporting soils. In determining the frictional resistance against concrete, a coefficient of friction of 0.33 for fine-grained native soils should be used.

Ultimate lateral earth pressures from on-site fine-grained backfill acting against retaining walls and buried structures may be computed from the lateral pressure coefficients or equivalent fluid densities presented in the following table:

Table 6.5

Lateral Earth Pressure

Condition	**Level Backfill**	
	Lateral Pressure Coefficient	**Equivalent Fluid Density (pcf)**
Active (Ka)	0.39	43
At-rest (Ko)	0.56	62
Passive (Kp)	2.56	282

These coefficients and densities assume no buildup of hydrostatic pressures. The force of the water should be added to the presented values if hydrostatic pressures are anticipated. If granular backfill will be used, the values presented in Table 6.5 can be re-evaluated by IGES upon request and subsequently modified as appropriate.

Walls and structures allowed to rotate slightly should use the active condition. If the element is constrained against rotation (i.e., a basement wall), 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.

6.6 CONCRETE SLABS-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 structural fill or competent native earth materials. The gravel should consist of free-draining gravel or road base with a ¾-inch maximum particle size and no more than 5 percent passing the No. 200 mesh sieve. The layer should be compacted to at least 95 percent of the MDD as determined by ASTM D-1557 or vibrated for densification if gravel is used. Gravel materials not meeting the aforementioned criteria may be appropriate for construction; alternate materials should be evaluated on a case-by-case basis and should be approved by IGES prior to use.

The slab may be designed with a Modulus of Subgrade Reaction of **140 psi/inch**. If disturbed native soils are present, they should be removed or compacted to at least 95% of the MDD as determined by ASTM D-1557 (modified proctor) prior to placement of gravel. 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.

Our experience indicates that use of reinforcement in slabs and foundations can generally reduce the potential for drying and shrinkage cracking. However, some cracking can be expected as the concrete cures. Minor cracking is considered normal; however, it is often aggravated by a high water/cement ratio, high concrete temperature at the time of placement, small nominal aggregate size, and rapid moisture loss due to hot, dry, and/or windy weather conditions during placement and curing. Cracking due to temperature and moisture fluctuations can also be expected. The use of low slump concrete can reduce the potential for shrinkage cracking; saw cuts in the concrete at strategic locations can help to control and reduce undesirable shrinkage cracks.

6.7 PAVEMENT SECTION DESIGN

Based on soil classifications and a laboratory obtained CBR value of 4.9 for the native soil tested, the near surface fine-grained soils are expected to provide fair pavement support. Anticipated traffic volumes were not available at the time this report was prepared; however, based on our understanding of the project development we assume traffic on the roadways would consist primarily of passenger cars with occasional heavy vehicles associated with construction, municipal waste collection and similar. The following pavement designs have been developed for a 20-year design life assuming a 0 percent annual growth rate, and our assumed equivalent single axle load (ESAL) not exceeding 100,000 ESALs. Based on the information obtained and the assumptions listed above, we recommend one of the following pavement sections be constructed on properly prepared subgrade for the interior roadways and parking lots.

This pavement design is based on the assumption that the upper 8 inches of the native beneath all pavement sections will be removed and/or reworked in place and compacted to at least 95 percent of the MDD with the moisture content at or above OMC as determined by ASTM D-1557. Asphalt has been assumed to be a high stability plant mix, base course material should be composed of crushed stone with a minimum CBR of 70 and granular borrow should consist of a pit-run type of material with a minimum CBR of 30. Asphalt should be compacted to a minimum density of 96% of the Marshall value; base course and granular borrow should be compacted to at least 95% of the MDD as determined by ASTM D-1557.

Table 6.7.1

Flexible (Asphalt) Pavement Section

Pavement Section Options	Asphalt Concrete (in.)	Untreated Base Course (in.)	Granular Borrow (in.)	Reworked Native Soil (in.)
Option 1	3	8	7	8
Option 2	3.5	11	-	
Option 3	4	10	-	

Alternatively, a geotextile could be incorporated to reduce the overall thickness of the pavement section by using one of the options shown below using the TenCate Mirafi ® RS380i Geosynthetic placed over the native soils; the pavement section alternatives below were developed for the information and assumptions stated previously.

Table 6.7.2

Flexible (Asphalt) Pavement Section with TenCate Mirafi ® RS380i

Alternate Pavement Sections	Asphalt Concrete (in.)	Untreated Base Course (in.)	Reworked Native Soil (in.)
Option 4	3	9	-
Option 5	3.5	8	-

If one of these options are used, the woven geotextile should be placed directly over the native soils in accordance with manufacturers recommendations.

It is our experience that pavement in areas where vehicles frequently turn around, backup, or load and unload, including round-a-bouts and exit and entrance areas, often experience more distress. If the owner wishes to prolong the life of the pavement in these areas, consideration should be given to using a Portland cement concrete (rigid) pavement in these areas. For these conditions, the following rigid pavement section is recommended:

Table 6.7.3

Rigid Pavement Section

Concrete (in.)	Untreated Base Course (in.)	Reworked Native Soil (in.)
5	8	8

Concrete should consist of a low slump, low water cement ratio mix with a minimum 28-day compressive strength of 4,000 psi. The base course should be compacted to at least 95% of the MDD as determined by ASTM D-1557.

If traffic conditions vary significantly from our stated assumptions, IGES should be contacted so we can modify our pavement design parameters accordingly. Specifically, if the traffic counts are significantly higher or lower, IGES should be contacted to revise the pavement section design if necessary. The pavement sections presented assume that the majority of construction traffic including cement trucks, cranes, loaded haulers, etc. has ceased. If a significant volume of construction traffic occurs after the pavement section has been constructed, the owner should anticipate a reduced life and increased maintenance in some areas of the property.

The pavement section thicknesses above assume that there is no mixing over time between the road base/granular borrow and the softer native subgrade below. In order to prevent mixing or fines migration, and thereby prolong the life of the pavement section, we recommend that the owner give consideration to placing a non-woven filter fabric between the native soils and the road base/granular borrow. We recommend that a product such as TenCate Mirafi 160N, or an IGES-approved equivalent be used for separation. If one of the options from Table 6.7.2 is used, filter fabric is not required since RS380i also acts to separate in addition to structural reinforcement.

6.8 MOISTURE PROTECTION AND SURFACE DRAINAGE

During Construction: Over-wetting the soils prior to, during, or after construction may result in softening and pumping, causing equipment mobility problems and difficulty in achieving compaction. Every effort should be taken to ensure positive drainage away from roadway areas to reduce the potential for water to migrate below pavements and concrete flatwork. The recommended minimum slope is two percent (2%) in pavement areas.

Moisture should not be allowed to infiltrate the soils in the vicinity of, or upslope from, the roadways.

Residential Structures: Moisture should not be allowed to infiltrate into the soils in the vicinity of the foundations. As such, the following design strategies to minimize ponding and infiltration near the home should be implemented:

- We recommend that hand watering, desert landscaping or Xeriscape be considered within five feet of the foundations.
- Roof runoff devices should be installed to direct all runoff a minimum of 10 feet away from structures or beyond the limits of excavation; whichever distance is greater.
- Irrigation valves shall be a minimum of five feet away from foundation walls and must not be placed within the basement backfill zone.
- The home builder should be responsible for compacting the exterior backfill soils around the foundation.
- The ground surface within 10 feet of the house should be constructed so as to slope a minimum of five percent away from the home.
- Pavement sections should be constructed to divert surface water off of the pavement into storm drains.
- Parking strips and roadway shoulder areas should be constructed to prevent infiltration of water into the surrounding pavement.

6.9 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 89.3 ppm. Based on this result, the onsite native soils are expected to exhibit a low potential for sulfate attack to concrete. Conventional Type I/II cement may 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 803 OHM-cm, a chloride content of 116 ppm and a pH value of 8.02. Based on these results, the onsite native soil is considered *very corrosive* to ferrous metal. Consideration should be given to retaining the services of a qualified corrosion engineer to provide an assessment of any metal that may be associated with construction of ancillary water lines and reinforcing steel, valves and similar improvements in contact with native soils.

6.10 CONSTRUCTION CONSIDERATIONS

6.10.1 Collapsible Soils

Collapsible soils are typically identified in the field by a porous, open soil structure ('pinholes'), relatively low moisture content and low in-situ dry density. Based on our laboratory tests of onsite soils and direct observation, there are potentially collapsible soils onsite near surface. All footings should bear entirely on uniform, relatively undisturbed native soils at a minimum depth of 5 feet below the existing site grade. All footings may also bear entirely on structural fill with a uniform thickness extending to such soils.

Prior to placement of footings, an IGES representative should assess the foundation subgrade for the presence of potentially collapsible soils. If particularly adverse soil conditions are identified (porous soils, low dry unit weight), additional over-excavation or recommendations may be necessary, depending on the extent and severity of the problematic soils.

6.10.2 Structural Fill

The prevailing clayey soils identified across the site are suitable for use as structural fill; however, the Contractor and Owner should be aware that properly moisture-conditioning and compacting clay soils is often challenging and time-consuming. If structural fill is needed, the Owner and/or Contractor may wish to consider importing a more suitable granular material for use as structural fill. IGES should approve any borrow source prior to importing structural fill.

7.0 CLOSURE

7.1 LIMITATIONS

The recommendations presented in this report are based on limited field exploration, laboratory testing, and our 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 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, 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 also be notified.

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

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

7.2 ADDITIONAL SERVICES

The recommendations made in this report are based on the assumption that an adequate program of tests and observations will be made during the construction. IGES staff or other qualified personnel should be on site to assess compliance with these recommendations. These tests and observations should include the following:

- Observations and testing during site preparation, earthwork and structural fill placement.
- Consultation as may be required during construction.
- Quality control on concrete placement to verify slump, air content, and strength.
- Quality control and testing during placement and compaction of asphalt.

We also recommend that project plans and specifications be reviewed by us to assess 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 (801) 748-4044.

8.0 REFERENCES

ASCE 7 Standard, 2010, Minimum Design Loads for Buildings and Other Structures

Black, B.D., Hecker, S., Hylland, M.D., Christenson, G.E., and McDonald, G.N.,2003, Quaternary Fault and Fold Database and Map of Utah, Utah Geological Survey Map 193DM.

Davis, F.D., 2000, Geologic Map of the Midvale Quadrangle, Salt Lake County, Utah, Utah Geologic Survey, Map 177.

Federal Emergency Management Agency [FEMA], 1997, *NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures,* FEMA 302, Washington, D.C.

Frankel, A., Mueller, C., Barnard, T., Perkins, D., Leyendecker, E.V., Dickman, N., Hanson, S., and Hopper, M., 1996, *National Seismic-hazard Maps: Documentation*, U.S. Geological Survey Open-File Report 96-532, June.

Hintze, L.F., 1980, Geologic map of Utah: Utah Geological and Mineral Survey Map A-1, Scale 1:500,000.

Hintze, L.F., 1993, Geologic History of Utah, Brigham Young University Studies, Special Publication 7, 202p.

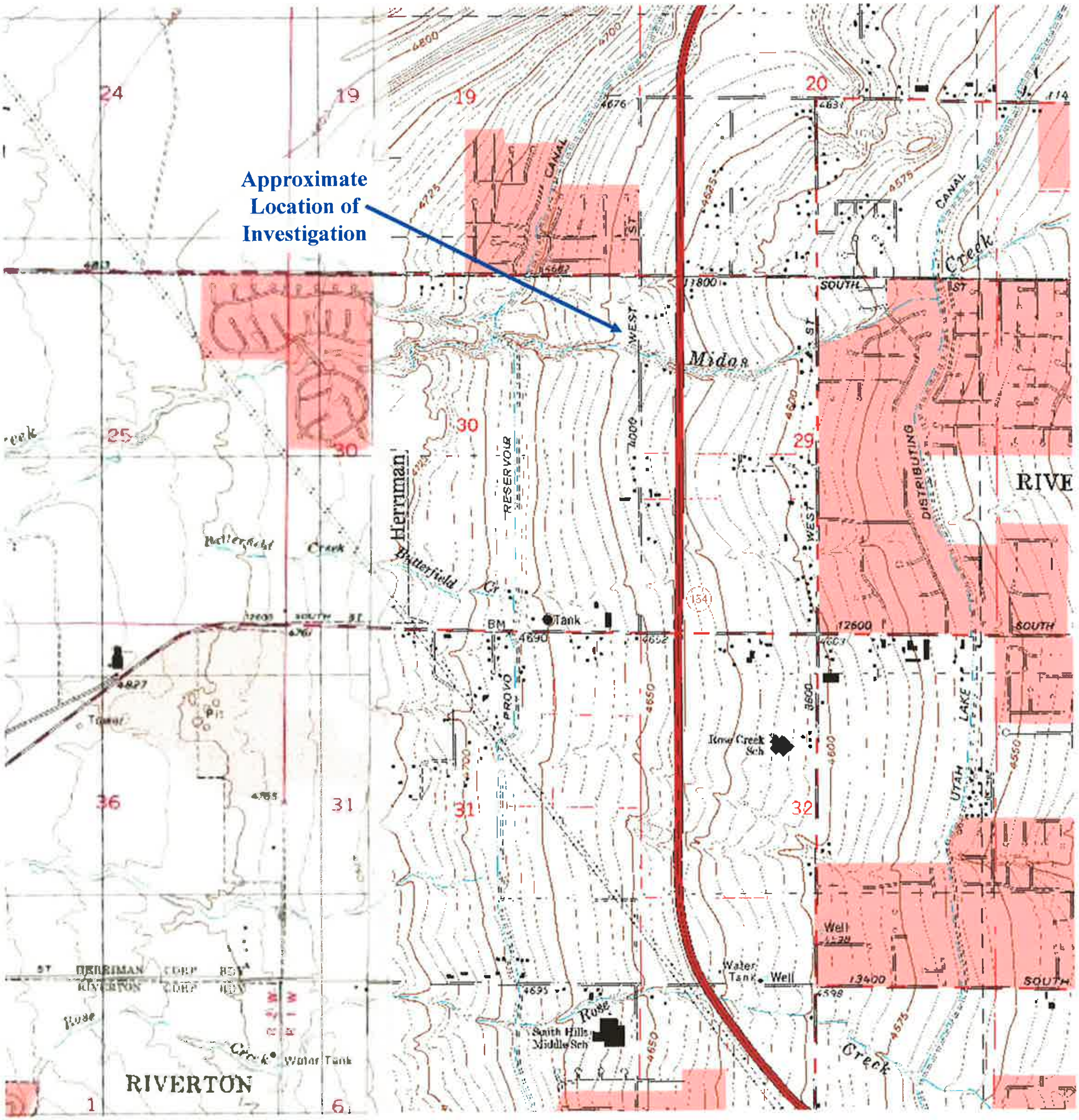
International Building Code [IBC], 2012, International Code Council, Inc.

Keaton, J.R., and Curry, D.R., 1993, Earthquake hazard evaluation of the West Valley fault zone in the Salt Lake City urban area, Utah: Utah Geological Survey, Contract Report 93-7, p. 69.

Scott, W.E., McCoy, W.D., Shorba, R.R., and Rubin, Meyer, 1983, Reinterpretation of the exposed record of the last two cycles of Lake Bonneville, western United States: Quaternary Research, v.20, p. 261-285.

Utah Geological Survey, 1994, Liquefaction-Potential Map for a Part of Salt Lake County, Utah, Public Information Series 25, August 1994.

APPENDIX A



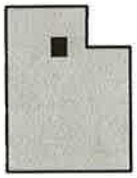
BASE MAP:
WEST: LARK, UTAH, U.S.G.S. 7.5 MINUTE
QUADRANGLE

EAST: MIDVALE, UTAH, U.S.G.S. 7.5 MINUTE
QUADRANGLE

0' 1000' 2000'

SCALE 1:24,000

Contour Interval is 20 feet



MAP LOCATION

N

IGES®

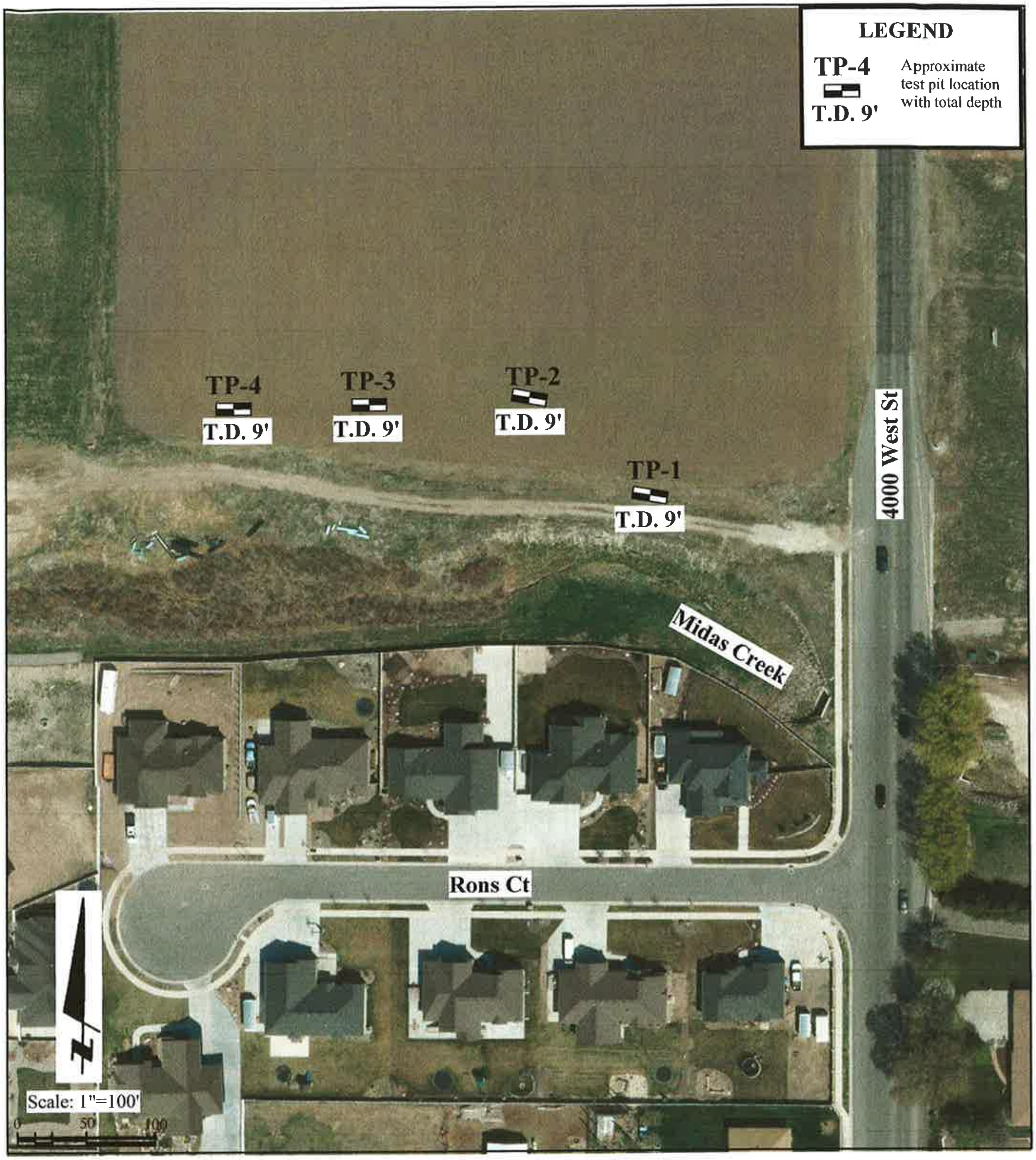
Project Number – 01466-022

Geotechnical Investigation
Proposed Structure
~11800 South 4000 West
Riverton, Utah

SITE VICINITY MAP

Figure

A-1



BASE MAP:
AGRC Utah HRO Imagery, 12TVK160860.tif, Date 7-3-2012

NOTE:
Test pit locations were specified by the client.

IGES®

Project No. 01466-022

Geotechnical Investigation
Proposed Structure
~11800 South 4000 West
Riverton, Utah

SITE MAP

Figure

A-2

DATE	
STARTED:	4/8/16
COMPLETED:	4/8/16
BACKFILLED:	4/8/16

Geotechnical Investigation
Proposed Structure
~11800 South 4000 West
Riverton, Utah

Project Number 01466-022

IGES Rep: SL
Rig Type: Mini-ex

TEST PIT NO:
TP-1
Sheet 1 of 1

LOCATION
LATITUDE
LONGITUDE
ELEVATION

MATERIAL DESCRIPTION

UNIFIED SOIL CLASSIFICATION	MATERIAL DESCRIPTION	Dry Density(pcf)	Moisture Content %	Percent minus 200	Liquid Limit	Plasticity Index
	Topsoil - Sandy Lean CLAY with gravel - medium stiff, moist, dark brown, occasional roots in upper 4 inches					
CL	Native - Sandy Lean CLAY - medium stiff, moist, brown, moderate fine pinholes					
		89.6	21.5			
GM	Silty GRAVEL with sand - medium dense, moist, brown					
			5.9	15.4		
CL	Sandy Lean CLAY - medium stiff, moist, gray, occasional fine pinholes					
			25.8			

No groundwater encountered

Bottom of Test Pit @ 9 Feet

DEPTH: FEET 0, 5, 10

Moisture Content and Atterberg Limits

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

WATER LEVEL
▼- MEASURED
▽- ESTIMATED

NOTES:

Figure
A-3

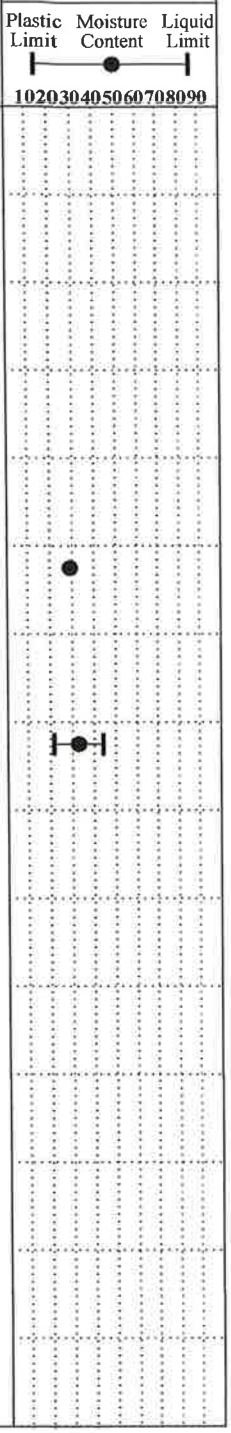
DATE		Project		TEST PIT NO:
STARTED: 4/8/16		Geotechnical Investigation Proposed Structure ~11800 South 4000 West Riverton, Utah	IGES Rep: SL	**TP-2**
COMPLETED: 4/8/16			Rig Type: Mini-ex	Sheet 1 of 1
BACKFILLED: 4/8/16		Project Number 01466-022		

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			Topsoil - Sandy Lean CLAY with gravel - medium stiff, moist, dark brown, occasional roots in upper 4 inches					
		GP	Native - Poorly Graded GRAVEL with sand and some cobbles - medium dense, moist, brown, gravel and cobbles are sub-angular to sub-rounded, up to 6 inches in diameter with 1/2 to 2 inches typical					
		CL	Sandy Lean CLAY with trace gravel - medium stiff, very moist, brown, occasional fine pinholes					
5	⧗			91.8	28.8			
			-no pinholes observed					
	▯		Lean CLAY - medium stiff, very moist, light brown, no pinholes observed		32.5		44	23
			No groundwater encountered					
			Bottom of Test Pit @ 9 Feet					
10								

Moisture Content and Atterberg Limits



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

WATER LEVEL
- ▼ - MEASURED
- ▽ - ESTIMATED

NOTES:

Figure A-4

DATE		
STARTED:	4/8/16	
COMPLETED:	4/8/16	
BACKFILLED:	4/8/16	

Geotechnical Investigation
Proposed Structure
~11800 South 4000 West
Riverton, Utah

Project Number 01466-022

IGES Rep: SL

Rig Type: Mini-ex

TEST PIT NO:

TP-3

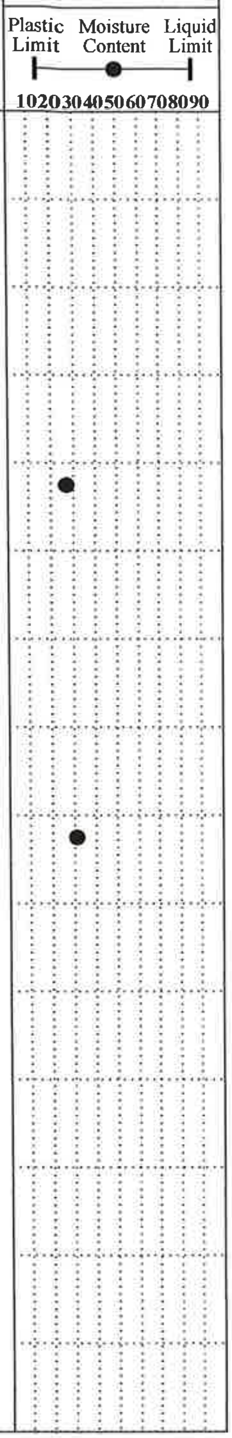
Sheet 1 of 1

LOCATION

LATITUDE LONGITUDE ELEVATION

ELEVATION	DEPTH FEET	SAMPLES	WATER LEVEL	UNIFIED SOIL CLASSIFICATION	MATERIAL DESCRIPTION	Dry Density(pcf)	Moisture Content %	Percent minus 200	Liquid Limit	Plasticity Index
	0				Topsoil - Sandy Lean CLAY with gravel - medium stiff, moist, dark brown, occasional roots in upper 4 inches					
				CL	Native - Sandy Lean CLAY - medium stiff, moist, brown, moderate fine pinholes					
		▯								
		⧗				85.8	26.4			
	5				-occasional fine pinholes					
					Lean CLAY - medium stiff, moist, light brown, moderate fine pinholes					
		▯					30.7	91.4		
					No groundwater encountered					
					Bottom of Test Pit @ 9 Feet					
	10									

Moisture Content and Atterberg Limits



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

WATER LEVEL
▼- MEASURED
▽- ESTIMATED

NOTES:

Figure

A-5

DATE	
STARTED:	4/8/16
COMPLETED:	4/8/16
BACKFILLED:	4/8/16

Geotechnical Investigation
Proposed Structure
~11800 South 4000 West
Riverton, Utah

Project Number 01466-022

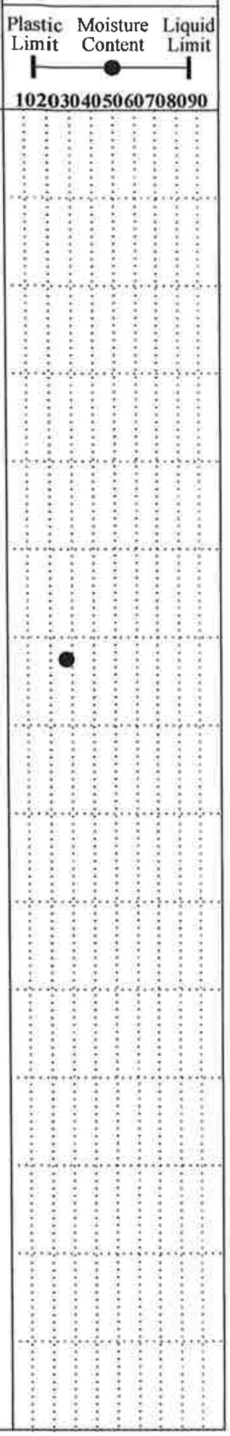
IGES Rep: SL
Rig Type: Mini-ex

TEST PIT NO:
TP-4
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					Topsoil - Sandy Lean CLAY - medium stiff, moist, dark brown, occasional roots in upper 4 inches					
				▨	CL	Native - Lean CLAY with sand, medium stiff, moist, brown, moderate fine pinholes					
		▯									
	5										
		⧗					90.3	26.9			
						-no pinholes observed					
						-light brown					
						No groundwater encountered Bottom of Test Pit @ 9 Feet					
	10										

Moisture Content and Atterberg Limits



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

WATER LEVEL
▼- MEASURED
▽- ESTIMATED

NOTES:

IGES

Figure
A-6

UNIFIED SOIL CLASSIFICATION SYSTEM

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

LOG KEY SYMBOLS

⊕ BORING SAMPLE LOCATION

⧗ TEST-PIT SAMPLE LOCATION

▼ WATER LEVEL (level after completion)

▽ WATER LEVEL (level where first encountered)

CEMENTATION

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

OTHER TESTS KEY

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

MODIFIERS

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

MOISTURE CONTENT

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

STRATIFICATION

DESCRIPTION	THICKNESS
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

GENERAL NOTES

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

APPARENT / RELATIVE DENSITY - COARSE-GRAINED SOIL

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

CONSISTENCY - FINE-GRAINED SOIL

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

IGES®

Key to Soil Symbols and Terminology

Figure A-7

APPENDIX B

Laboratory Compaction Characteristics of Soil

(ASTM D698 / D1557)

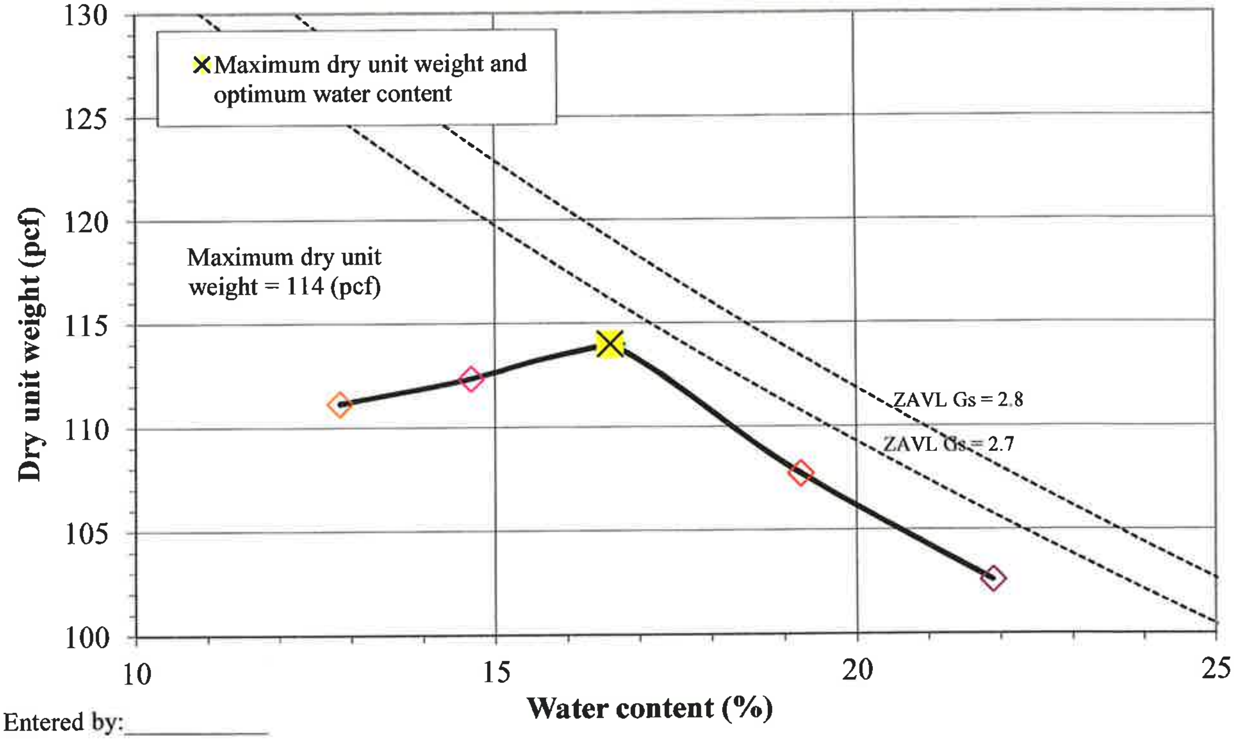
Project: **11800 S 4000 W GTI**
No: **01466-022**
Location: Riverton, UT
Date: 4/20/2016
By: IM / ET

Method: ASTM D1557 C
Mold Id. Inc 7
Mold volume (ft^3): 0.0751

Boring No.: **TP-4**
Sample:
Depth: **2.0-3.0'**
Sample Description: Light brown sandy clay
Engineering Classification: Not requested
As-received water content (%): Not requested
Preparation method: Moist
Rammer: Mechanical-sector face
Rock Correction: No

Optimum water content (%): **16.6**
Maximum dry unit weight (pcf): **114**

Point Number	-4%	-6%	-8%	-10%	-12%			
Wt. Sample + Mold (g)	10772.4	10887.6	11041.6	10901.1	10785.2			
Wt. of Mold (g)	6514.4	6514.4	6514.4	6514.4	6514.4			
Wet Unit Wt., γ_m (pcf)	125.0	128.4	132.9	128.8	125.4			
Wet Soil + Tare (g)	1585.92	1713.05	1428.27	1600.83	1525.42			
Dry Soil + Tare (g)	1357.75	1502.92	1271.52	1455.51	1389.77			
Tare (g)	315.81	410.36	328.96	464.60	333.18			
Water Content, w (%)	**21.9**	**19.2**	**16.6**	**14.7**	**12.8**			
Dry Unit Wt., γ_d (pcf)	**102.6**	**107.7**	**114.0**	**112.3**	**111.1**			



Entered by:____________

Reviewed:____________

Figure B-1

California Bearing Ratio

(ASTM D 1883)

Project: 11800 S 4000 W GTI
Number: 01466-022
Location: Riverton, UT
Date: 4/26/2016
By: ET

Boring No.: TP-4
Sample:
Depth: 2.0-3.0'
Original Method: ASTM D1557 C
Engineering Classification: Not requested
Condition of Sample: Soaked
Scalp and Replace: No

Maximum Dry Unit Weight (pcf):	114
Optimum Water Content (%):	16.6
Relative Compaction (%):	95.8
0.1 in. CBR (%):	**4.9**
0.2 in. CBR (%):	**5.0**

As Compacted Data			Before	After
Mold Id.	CBR-1	Wet Soil + Tare (g)	809.35	882.89
Wt. of Mold + Sample (g)	11610.8	Dry Soil + Tare (g)	713.56	776.53
Wt. of Mold (g)	7299.5	Tare (g)	115.75	123.24
Dry Unit Weight (pcf)	109.2	Water Content (%)	16.0	16.3

After Soaking Data			Average	Top 1 in.
Wt. of Mold + Sample (g)	11774.5	Wet Soil + Tare (g)	1554.16	642.46
Dry Unit Weight (pcf)	106.8	Dry Soil + Tare (g)	1360.82	540.7
		Tare (g)	409.8	112.2
		Water Content (%)	20.3	23.7

Swell Data

Date	Time	Dial
4/21/2016	10:10	0.492
4/25/2016	09:45	0.597

Surcharge (psf) 50
Swell (%) 2.29
Soaking Period (hr) 96

Penetration Data	Piston ID	CBR T1

Zero load (lb) = 0
Area of Piston (in^2) = 3.0

Penetration (in.)	Raw Load (lb)	Piston Stress (psi)	Std. Stress (psi)
0.000	0	0	
0.025	49	16	
0.050	86	29	
0.075	118	39	
0.100	146	49	1000
0.125	170	57	1125
0.150	190	63	1250
0.175	209	70	1375
0.200	225	75	1500
0.300	284	95	1900
0.400	341	114	2300
0.500	395	132	2600

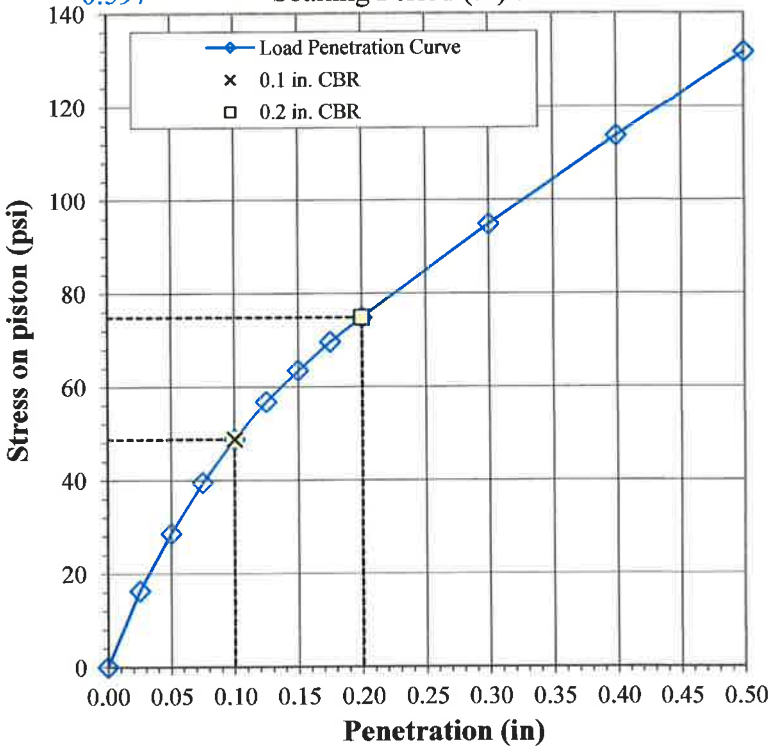


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Figure B-2

One-Dimensional Consolidation Properties of Soils

(ASTM D2435)

Project: 11800 S 4000 W GTI
No: 01466-022
Location: Riverton, UT
Date: 4/18/2016
By: BRR

Boring No.: TP-4
Sample:
Depth: 6.0'
Sample Description: Brown clay
Engineering Classification: Not requested
Sample type: Undisturbed-trimmed from thin-wall

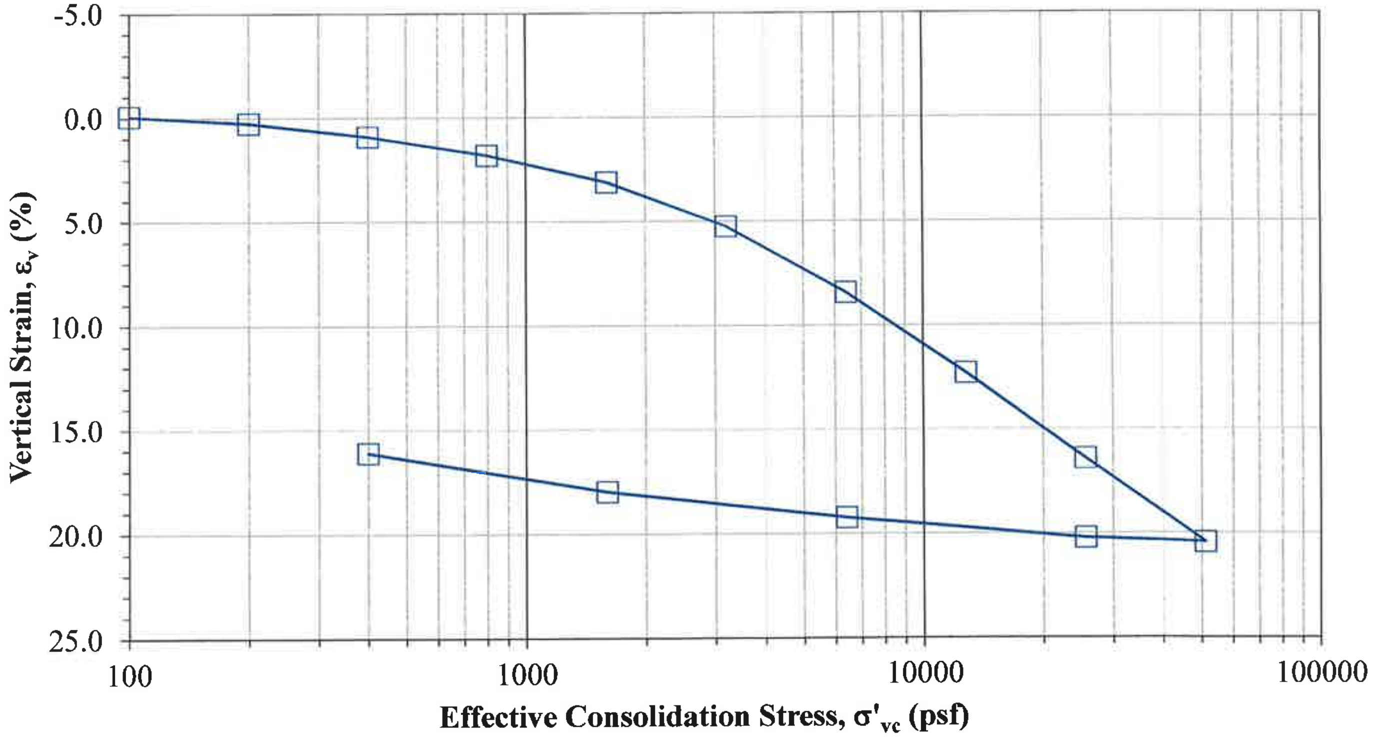
Test method:	A	
Inundation stress (psf), timing:	Seating	Beginning
Specific gravity, G_s	2.70	Assumed

Water type used for inundation Tap

	Initial (o)	Final (f)
Sample height, H (in.)	0.920	0.7718
Sample diameter, D (in.)	2.416	2.416
Wt. rings + wet soil (g)	172.24	167.45
Wt. rings/tare (g)	45.30	45.30
Moist unit wt., γ_m (pcf)	114.7	131.51
Wet soil + tare (g)	473.97	241.80
Dry soil + tare (g)	406.06	219.87
Tare (g)	154.00	120.86
Water content, w (%)	26.9	22.1
Dry unit wt., γ_d (pcf)	90.3	107.7
Saturation	0.84	1.00

*Note: C_v, C_c, C_r, and σ_p' to be determined by Geotechnical Engineer.

Stress (psf)	Dial (in.)	1-D ε_v (%)	H_c (in.)	e
Seating	0.0000	0.00	0.9200	0.8661
100	-0.0006	-0.06	0.9206	0.8673
200	0.0023	0.25	0.9177	0.8614
400	0.0084	0.91	0.9116	0.8491
800	0.0166	1.80	0.9034	0.8325
1600	0.0286	3.11	0.8914	0.8081
3200	0.0483	5.25	0.8717	0.7682
6400	0.0775	8.43	0.8425	0.7089
12800	0.1132	12.30	0.8068	0.6365
25600	0.1511	16.42	0.7689	0.5596
51200	0.1881	20.45	0.7319	0.4846
25600	0.1861	20.23	0.7339	0.4886
6400	0.1769	19.23	0.7431	0.5073
1600	0.1654	17.98	0.7546	0.5306
400	0.1482	16.11	0.7718	0.5655



Comments: Specimen swelled upon inundation and at the 100 psf loading.

Entered: ________
Reviewed: ________

Figure B-3

Collapse/Swell Potential of Soils

(ASTM D4546 Method B)

Project: 11800 S 4000 W GTI
No: 01466-022
Location: Riverton, UT
Date: 4/18/2016
By: BRR

Boring No.: TP-1
Sample:
Depth: 3.0'
Sample Description: Brown clay
Engineering Classification: Not requested
Sample type: Undisturbed-trimmed from thin-wall

Consolidometer No.: 4
Specific gravity, G_s: 2.70 Assumed
Collapse (%): 2.9
Collapse stress (psf): 1600
Water type used for inundation: Tap

	Initial (o)	Final (f)
Sample height, H (in.)	0.880	0.8130
Sample diameter, D (in.)	2.416	2.416
Mass rings + wet soil (g)	157.80	162.28
Mass rings/tare (g)	42.55	42.55
Moist unit wt., γ_m (pcf)	108.83	122.38
Wet soil + tare (g)	391.18	246.13
Dry soil + tare (g)	344.60	221.70
Tare (g)	127.75	128.46
Water content, w (%)	21.5	26.2
Dry unit wt., γ_d (pcf)	89.59	96.97
Saturation	65.80	95.83

Stress (psf)	Dial (in.)	1-D ε_v (%)	H_c (in.)	e
Seating	0.1334	0.00	0.8800	0.881
20	0.1334	0.00	0.8800	0.881
100	0.1346	0.14	0.8788	0.879
300	0.1405	0.81	0.8729	0.866
100	0.1405	0.81	0.8729	0.866
300	0.1409	0.85	0.8725	0.865
800	0.1500	1.89	0.8634	0.846
1600	0.1752	4.75	0.8382	0.792
1600	0.2004	7.61	0.8130	0.738

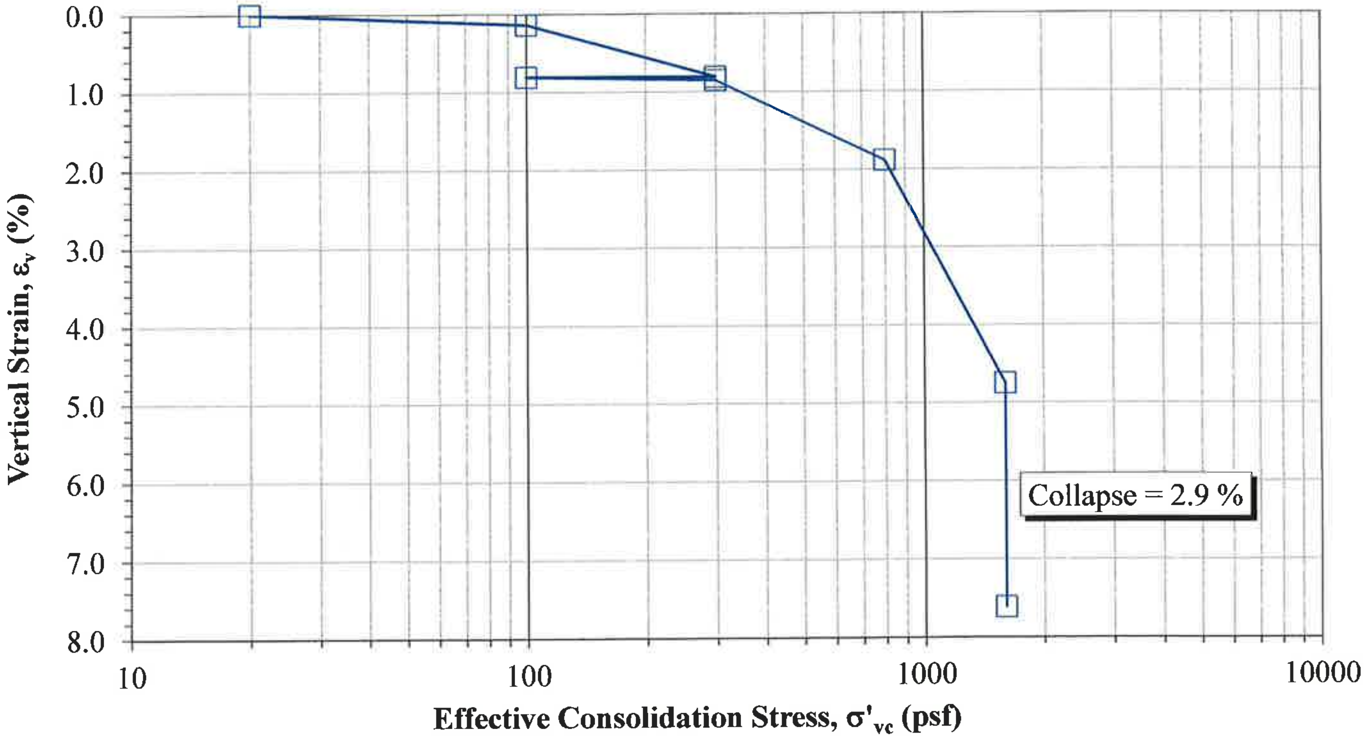


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Figure B-4

Collapse/Swell Potential of Soils

(ASTM D4546 Method B)

Project: 11800 S 4000 W GTI
No: 01466-022
Location: Riverton, UT
Date: 4/18/2016
By: BRR

Boring No.: TP-2
Sample:
Depth: 5.0'
Sample Description: Brown clay
Engineering Classification: Not requested
Sample type: Undisturbed-trimmed from thin-wall

Consolidometer No.: 1
Specific gravity, G_s 2.70 Assumed
Collapse (%) 0.8
Collapse stress (psf) 1600
Water type used for inundation Tap

	Initial (o)	Final (f)
Sample height, H (in.)	0.880	0.8464
Sample diameter, D (in.)	2.416	2.416
Mass rings + wet soil (g)	170.84	170.76
Mass rings/tare (g)	45.59	45.59
Moist unit wt., γ_m (pcf)	118.27	122.89
Wet soil + tare (g)	380.95	247.49
Dry soil + tare (g)	330.05	219.90
Tare (g)	153.31	123.82
Water content, w (%)	28.8	28.7
Dry unit wt., γ_d (pcf)	91.83	95.47
Saturation	93.06	100.00

Stress (psf)	Dial (in.)	1-D ε_v (%)	H_c (in.)	e
Seating	0.0919	0.00	0.8800	0.836
20	0.0919	0.00	0.8800	0.836
100	0.0929	0.11	0.8790	0.833
200	0.0950	0.35	0.8769	0.829
500	0.1011	1.05	0.8708	0.816
200	0.1005	0.98	0.8714	0.818
100	0.1000	0.92	0.8719	0.819
200	0.1003	0.95	0.8716	0.818
500	0.1022	1.17	0.8697	0.814
1000	0.1100	2.06	0.8619	0.798
1600	0.1186	3.03	0.8533	0.780
1600	0.1255	3.82	0.8464	0.765

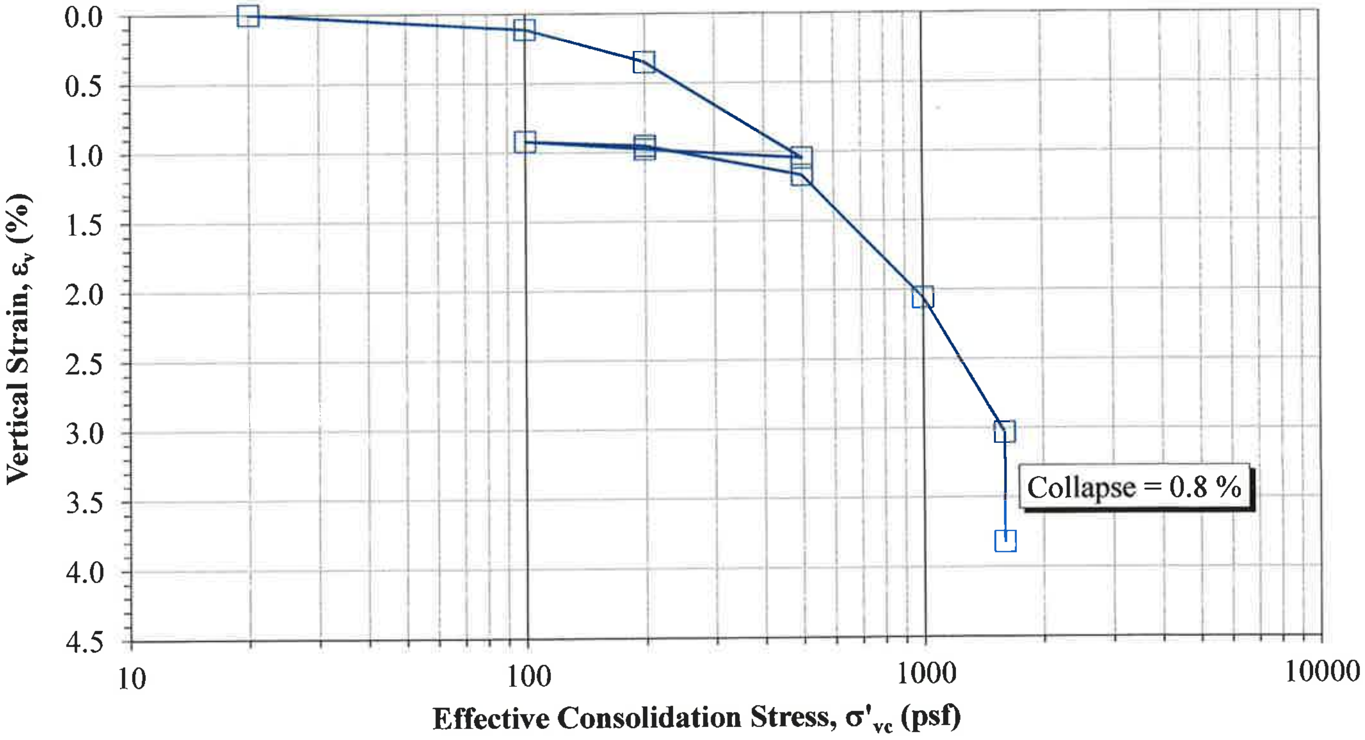

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Figure B-5

Unconsolidated-Undrained Triaxial Compression Test on Cohesive Soils

(ASTM D2850)

Project:	11800 S 4000 W GTI	**Boring No.:**	TP-3
No:	01466-022	**Sample:**	
Location:	Riverton, UT	**Depth:**	4.0'
Date:	4/15/2016	Sample Description:	Brown clay
By:	BRR	Sample type:	Undisturbed

Specific gravity, Gs	2.70	Assumed
Sample height, H (in.)	5.571	
Sample diameter, D (in.)	2.403	
Sample volume, V (ft^3)	0.0146	
Wt. rings + wet soil (g)	719.40	
Wt. rings/tare (g)	0.00	
Moist soil, Ws (g)	719.40	
Moist unit wt., γ_m (pcf)	108.5	
Dry unit wt., γ_d (pcf)	**85.8**	
Saturation (%)	73.6	
Void ratio, *e*	0.97	

Wet soil + tare (g)	842.78
Dry soil + tare (g)	692.84
Tare (g)	124.60
Water content, w (%)	**26.4**
Confining stress, σ_3 (psf)	400
Shear rate (in/min)	0.0167
Strain at failure, ε_f (%)	19.93
Deviator stress at failure, $(\sigma_1-\sigma_3)_f$ (psf)	881
Shear stress at failure, $q_f = (\sigma_1-\sigma_3)_f/2$ (psf)	440

Axial Strain (%)	σ_d σ_1-σ_3 (psf)	Q 1/2 σ_d (psf)
0.00	0.0	0.0
0.05	87.2	43.6
0.10	136.8	68.4
0.15	169.8	84.9
0.20	194.5	97.3
0.25	215.0	107.5
0.30	235.5	117.8
0.35	247.7	123.9
0.40	264.0	132.0
0.45	276.2	138.1
0.70	345.0	172.5
0.95	405.3	202.7
1.20	465.3	232.6
1.45	504.4	252.2
1.70	547.4	273.7
1.95	565.8	282.9
2.20	588.1	294.0
2.45	598.1	299.0
2.70	595.9	297.9
2.95	601.8	300.9
3.20	615.6	307.8
3.45	629.5	314.7
3.70	631.1	315.6
3.95	644.8	322.4
4.20	666.4	333.2
4.45	683.8	341.9
4.70	709.1	354.6
4.95	722.4	361.2
5.45	752.6	376.3
5.95	759.0	379.5
6.45	741.9	371.0
6.95	736.6	368.3
7.45	746.6	373.3
7.95	752.7	376.3
8.45	766.2	383.1
8.95	779.6	389.8
9.45	781.4	390.7
9.95	790.6	395.3
10.45	799.7	399.8
10.95	797.5	398.7
11.45	798.9	399.4
11.95	804.0	402.0
12.45	819.8	409.9
12.95	828.1	414.0
13.45	814.7	407.3
13.95	819.2	409.6
14.45	823.6	411.8
14.95	824.4	412.2
15.45	832.1	416.0
15.95	853.6	426.8
16.45	871.3	435.6
16.95	864.6	432.3
17.45	840.6	420.3
17.95	851.0	425.5
18.45	854.4	427.2
18.95	864.5	432.2
19.45	871.0	435.5
19.93	880.8	440.4

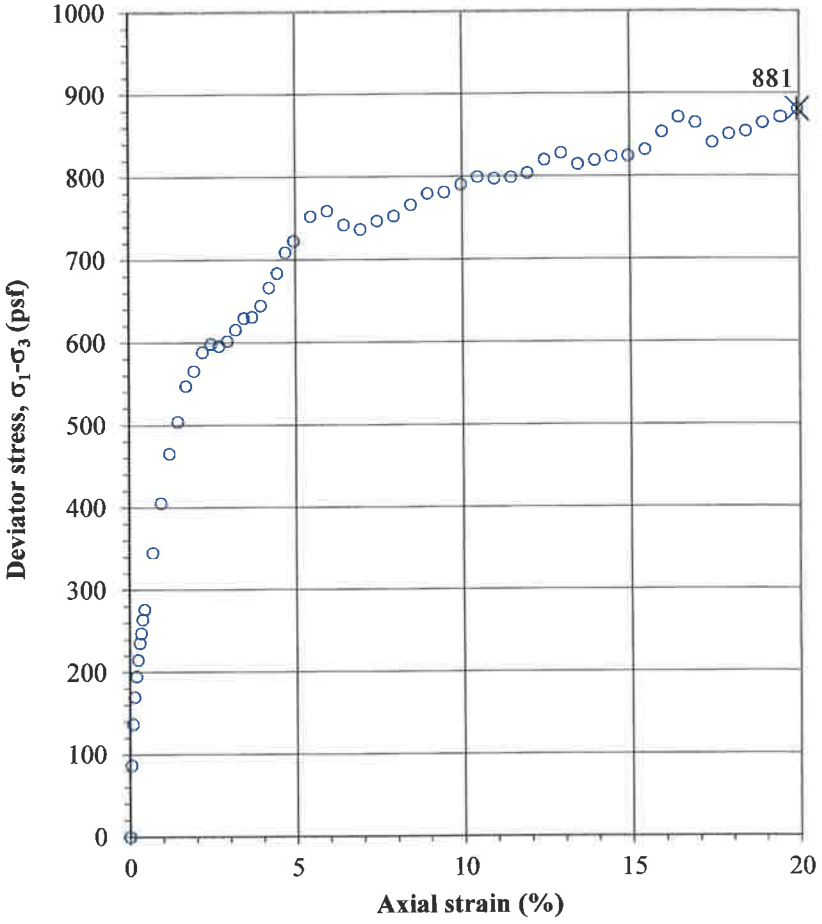

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Figure B-6

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)*

Project: 11800 S 4000 W GTI
No: 01466-022
Location: Riverton, UT
Date: 4/20/2016
By: BRR

Sample info.	Boring No.	TP-3							
	Sample								
	Depth	2.5'							
Water content data	Wet soil + tare (g)	126.79							
	Dry soil + tare (g)	116.68							
	Tare (g)	37.75							
	Water content (%)	12.8							
Chem. data	pH	8.02							
	Soluble chloride* (ppm)	116							
	Soluble sulfate** (ppm)	89.3							
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	6868	0.67	4602				
		+3	3413	0.67	2287				
		+6	2037	0.67	1365				
		+9	1288	0.67	863				
		+12	1233	0.67	826				
		+15	1198	0.67	803				
		+18	1213	0.67	813				
	Minimum resistivity (Ω-cm)	**803**							

* Performed by AWAL using EPA 300.0

** Performed by AWAL using ASTM C1580

Entered by:____________

Reviewed:____________

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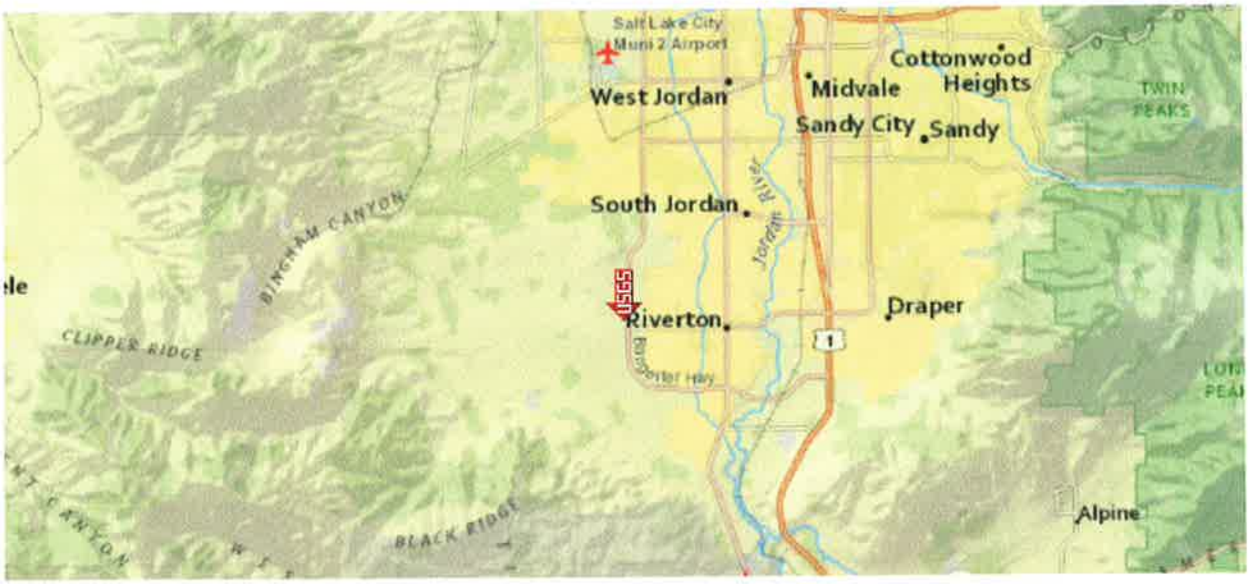
Figure B-7

APPENDIX C

USGS Design Maps Summary Report

User–Specified Input

Building Code Reference Document	2012 International Building Code (which utilizes USGS hazard data available in 2008)
Site Coordinates	40.5343°N, 111.9868°W
Site Soil Classification	Site Class D – "Stiff Soil"
Risk Category	I/II/III

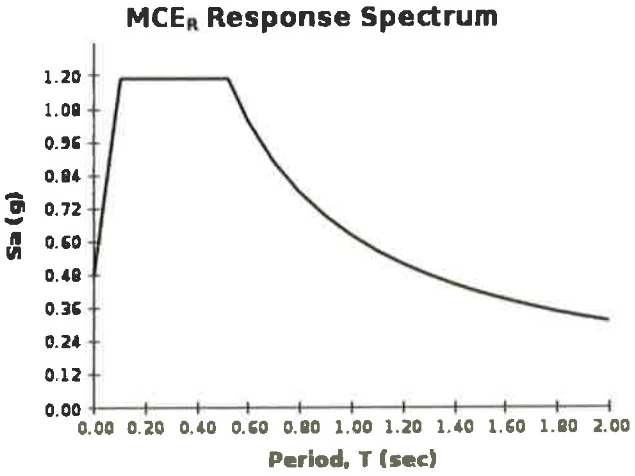


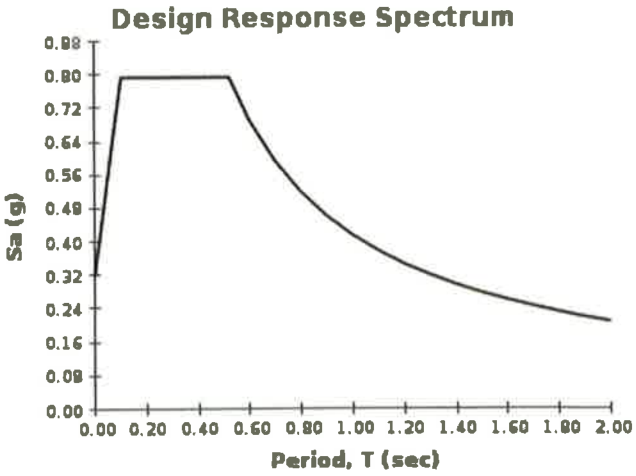
USGS–Provided Output

$S_S =$ 1.139 g $S_{MS} =$ 1.189 g $S_{DS} =$ 0.793 g

$S_1 =$ 0.378 g $S_{M1} =$ 0.622 g $S_{D1} =$ 0.414 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.





USGS Design Maps Detailed Report

2012 International Building Code (40.5343°N, 111.9868°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 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 = 1.139 g
From Figure 1613.3.1(2) [2]	S_1 = 0.378 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				
	$S_S \le 0.25$	$S_S = 0.50$	$S_S = 0.75$	$S_S = 1.00$	$S_S \ge 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	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 = 1.139 g, F_a = 1.044

TABLE 1613.3.3(2)
VALUES OF SITE COEFFICIENT F_v

Site Class	Mapped Spectral Response Acceleration at 1-s Period				
	$S_1 \le 0.10$	$S_1 = 0.20$	$S_1 = 0.30$	$S_1 = 0.40$	$S_1 \ge 0.50$
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 = 0.378 g, F_v = 1.644

Equation (16-37): $S_{MS} = F_aS_S = 1.044 \times 1.139 = 1.189$ g

Equation (16-38): $S_{M1} = F_vS_1 = 1.644 \times 0.378 = 0.622$ g

Section 1613.3.4 — Design spectral response acceleration parameters

Equation (16-39): $S_{DS} = ⅔\, S_{MS} = ⅔ \times 1.189 = 0.793$ g

Equation (16-40): $S_{D1} = ⅔\, S_{M1} = ⅔ \times 0.622 = 0.414$ 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
$0.167g \leq S_{DS} < 0.33g$	B	B	C
$0.33g \leq S_{DS} < 0.50g$	C	C	D
$0.50g \leq S_{DS}$	D	D	D

For Risk Category = I and S_{DS} = 0.793 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
$0.067g \leq S_{D1} < 0.133g$	B	B	C
$0.133g \leq S_{D1} < 0.20g$	C	C	D
$0.20g \leq S_{D1}$	D	D	D

For Risk Category = I and S_{D1} = 0.414 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

USGS Design Maps Detailed Report

ASCE 7-10 Standard (40.5343°N, 111.9868°W)

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

Section 11.4.1 — 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 2010 ASCE-7 Standard are provided for Site Class B. Adjustments for other Site Classes are made, as needed, in Section 11.4.3.

From Figure 22-1 [1]	S_S = 1.139 g
From Figure 22-2 [2]	S_1 = 0.378 g

Section 11.4.2 — Site Class

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 Chapter 20.

Table 20.3–1 Site Classification

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 $w \geq 40\%$, 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 11.4.3 — Site Coefficients and Risk–Targeted Maximum Considered Earthquake (MCE_R) Spectral Response Acceleration Parameters

Table 11.4–1: Site Coefficient F_a

Site Class	Mapped MCE $_R$ Spectral Response Acceleration Parameter at Short Period				
	$S_S \le 0.25$	$S_S = 0.50$	$S_S = 0.75$	$S_S = 1.00$	$S_S \ge 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	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 = 1.139 g, F_a = 1.044

Table 11.4–2: Site Coefficient F_v

Site Class	Mapped MCE $_R$ Spectral Response Acceleration Parameter at 1–s Period				
	$S_1 \le 0.10$	$S_1 = 0.20$	$S_1 = 0.30$	$S_1 = 0.40$	$S_1 \ge 0.50$
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 = 0.378 g, F_v = 1.644

Equation (11.4–1): $S_{MS} = F_aS_S = 1.044 \times 1.139 = 1.189$ g

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Equation (11.4–2): $S_{M1} = F_vS_1 = 1.644 \times 0.378 = 0.622$ g

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Section 11.4.4 — Design Spectral Acceleration Parameters

Equation (11.4–3): $S_{DS} = ⅔\, S_{MS} = ⅔ \times 1.189 = 0.793$ g

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Equation (11.4–4): $S_{D1} = ⅔\, S_{M1} = ⅔ \times 0.622 = 0.414$ g

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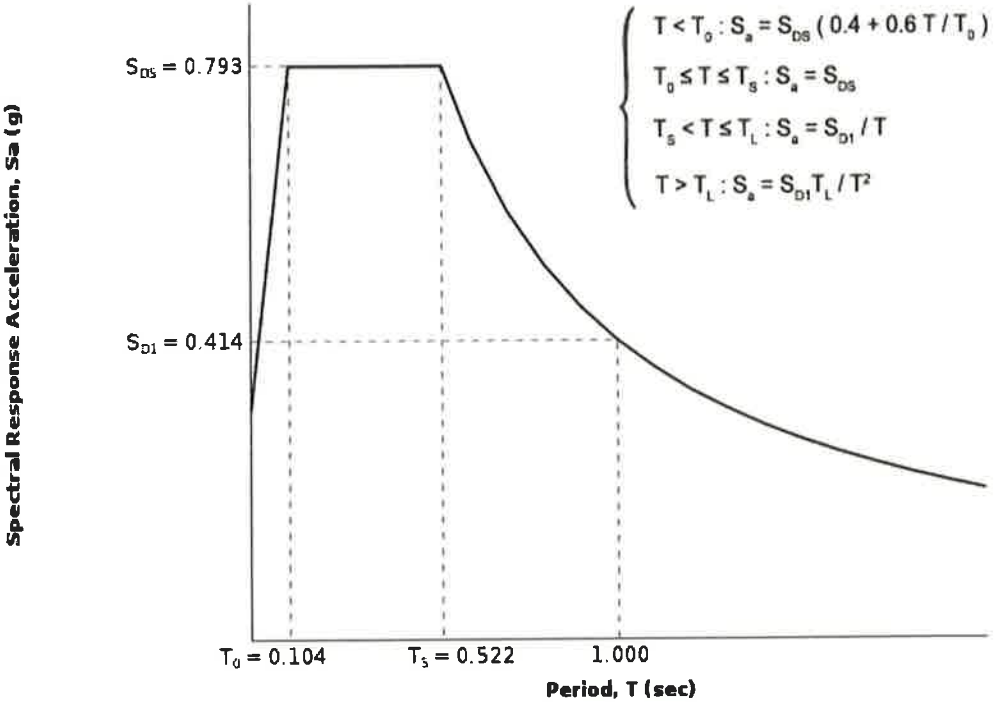
Section 11.4.5 — Design Response Spectrum

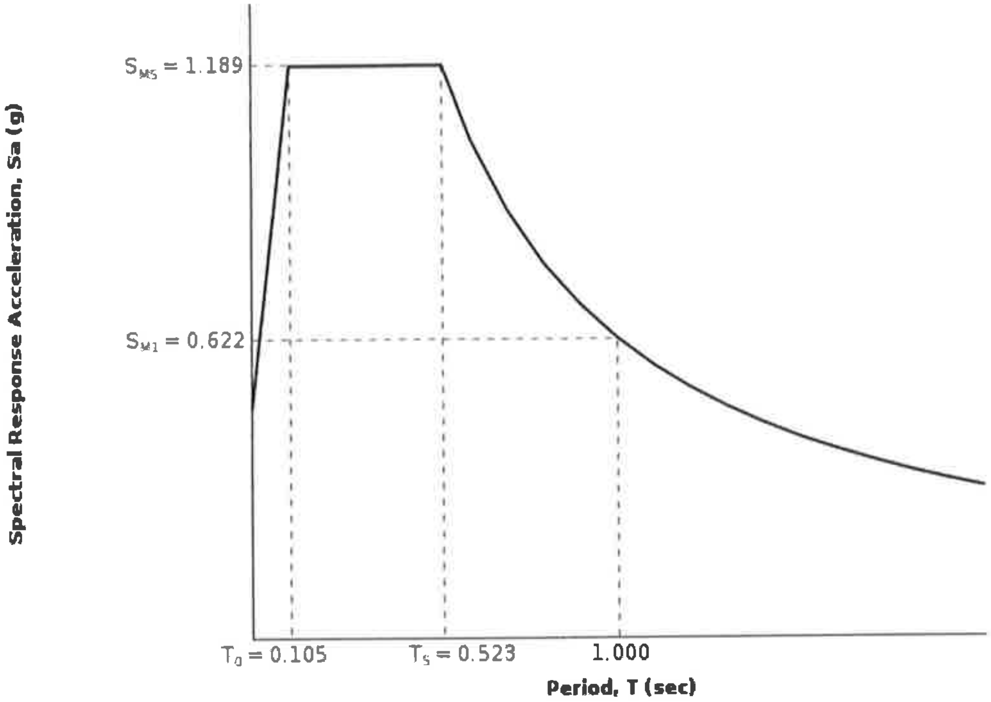
From Figure 22-12 [3] $T_L = 8$ seconds

Figure 11.4–1: Design Response Spectrum



Section 11.4.6 — Risk-Targeted Maximum Considered Earthquake (MCE_R) Response Spectrum

The MCE_R Response Spectrum is determined by multiplying the design response spectrum above by 1.5.



Section 11.8.3 — Additional Geotechnical Investigation Report Requirements for Seismic Design Categories D through F

From Figure 22-7 [4] PGA = 0.460

Equation (11.8–1): $PGA_M = F_{PGA}PGA = 1.040 \times 0.460 = 0.479$ g

Table 11.8–1: Site Coefficient F_{PGA}

Site Class	Mapped MCE Geometric Mean Peak Ground Acceleration, PGA				
	PGA ≤ 0.10	PGA = 0.20	PGA = 0.30	PGA = 0.40	PGA ≥ 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	1.2	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 PGA

For Site Class = D and PGA = 0.460 g, F_{PGA} = 1.040

Section 21.2.1.1 — Method 1 (from Chapter 21 – Site-Specific Ground Motion Procedures for Seismic Design)

From Figure 22-17 [5] C_{RS} = 0.844

From Figure 22-18 [6] C_{R1} = 0.839

Section 11.6 — Seismic Design Category

Table 11.6-1 Seismic Design Category Based on Short Period Response Acceleration Parameter

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} = 0.793 g, Seismic Design Category = D

Table 11.6-2 Seismic Design Category Based on 1-S Period Response Acceleration Parameter

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} = 0.414 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 11.6-1 or 11.6-2" = D

Note: See Section 11.6 for alternative approaches to calculating Seismic Design Category.

References

1. *Figure 22-1*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-1.pdf
2. *Figure 22-2*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-2.pdf
3. *Figure 22-12*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-12.pdf
4. *Figure 22-7*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-7.pdf
5. *Figure 22-17*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-17.pdf
6. *Figure 22-18*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-18.pdf

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 (801) 748-4044.

8.0 REFERENCES

ASCE 7 Standard, 2010, Minimum Design Loads for Buildings and Other Structures

Black, B.D., Hecker, S., Hylland, M.D., Christenson, G.E., and McDonald, G.N.,2003, Quaternary Fault and Fold Database and Map of Utah, Utah Geological Survey Map 193DM.

Davis, F.D., 2000, Geologic Map of the Midvale Quadrangle, Salt Lake County, Utah, Utah Geologic Survey, Map 177.

Federal Emergency Management Agency [FEMA], 1997, *NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures,* FEMA 302, Washington, D.C.

Frankel, A., Mueller, C., Barnard, T., Perkins, D., Leyendecker, E.V., Dickman, N., Hanson, S., and Hopper, M., 1996, *National Seismic-hazard Maps: Documentation*, U.S. Geological Survey Open-File Report 96-532, June.

Hintze, L.F., 1980, Geologic map of Utah: Utah Geological and Mineral Survey Map A-1, Scale 1:500,000.

Hintze, L.F., 1993, Geologic History of Utah, Brigham Young University Studies, Special Publication 7, 202p.

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

Keaton, J.R., and Curry, D.R., 1993, Earthquake hazard evaluation of the West Valley fault zone in the Salt Lake City urban area, Utah: Utah Geological Survey, Contract Report 93-7, p. 69.

Scott, W.E., McCoy, W.D., Shorba, R.R., and Rubin, Meyer, 1983, Reinterpretation of the exposed record of the last two cycles of Lake Bonneville, western United States: Quaternary Research, v.20, p. 261-285.

Utah Geological Survey, 1994, Liquefaction-Potential Map for a Part of Salt Lake County, Utah, Public Information Series 25, August 1994.