GeoStrata

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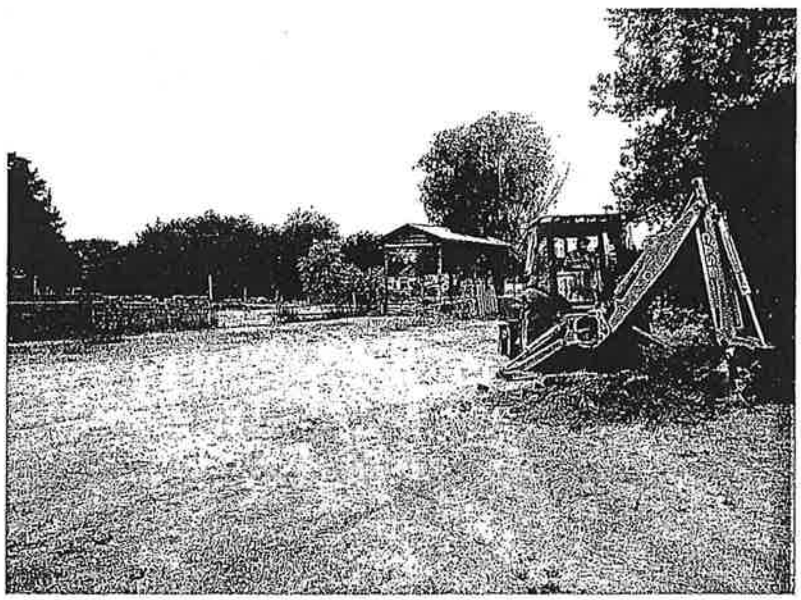
Geotechnical Investigation
Newman Construction (Riverton Peaks Subdivision)
Townhome Development -Riverton, UT
GeoStrata Job No. 178-070



July 22, 2014

Newman Construction, Inc.
13331 Redwood Road
Riverton, UT 84065
Attention: Mr. Mark Newman

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MEMORANDUM

To: **Mark Newman**
Newman Construction, Inc.

From: **J. Scott Seal, P.E.**
Mark Christensen, P.E.

Date: **July 28, 2014**

Subject: **Revised Pavement Recommendations – Townhouse Development, Riverton Utah**

The memorandum has been completed as a response to a request by Newman Construction, Inc, to re-evaluate the recommended pavement section as stated in a geotechnical report completed by GeoStrata for the proposed development. In that report, dated July 22, 2014, the following pavement section options were presented;

Flexible Pavement Section	
Asphalt Concrete (in)	Untreated Base Course (in)
3	18

Flexible Pavement Section		
Asphalt Concrete (in)	Untreated Base Course (in)	Granular Borrow (in)
3	8	14

The above pavement sections were completed assuming traffic counts consisting of 600 passenger vehicles a day, 4 small trucks a day, and one larger truck a day, resulting in anticipated ESAL count of 105,000. Since the completion of our original report, an updated anticipated traffic load was provided to GeoStrata. The updated traffic loads included 300 passenger vehicles a day and 2 small trucks a day, resulting in an ESAL count of 48,000. Based on this updated traffic loading, the following adjusted pavement section may be utilized for the proposed development;

Flexible Pavement Section	
Asphalt Concrete (in)	Untreated Base Course (in)
3	14

Alternatively, the following equivalent pavement section may be used;

Flexible Pavement Section		
Asphalt Concrete (in)	Untreated Base Course (in)	Granular Borrow (in)
3	6	12

Asphalt has been assumed to be a high stability plant mix and base course material (road base) composed of crushed stone with a minimum CBR of 70. We have further assumed that the traffic will be relatively consistent over the design life of the pavement sections. Therefore, no growth factor was applied in calculation of loading for each pavement sections' design life.

The conclusions and recommendations contained in this memorandum which include professional opinions and judgments, are based on the information available to us at the time of our evaluation, the results of our field observations, our limited subsurface exploration and our understanding of the proposed site development. This memorandum 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.

This memorandum was written for the exclusive use of Newton Construction, Inc., and only for the proposed project described herein. 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 memorandum in its entirety. We are not responsible for the technical interpretations by others of the information described or documented in this memorandum.

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Prepared for:

Newman Construction, Inc.
Attn: Mark Newman
13331 Redwood Road
Riverton, UT 84065

Geotechnical Investigation- Newman Construction
Townhome Development -Riverton, UT
GeoStrata Job No. 178-070

Prepared by:

J. Scott Seal, P.E.
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Reviewed by:

7/22/14

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July 22, 2014

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1.0 EXECUTIVE SUMMARY

This report presents the results of a geotechnical investigation conducted for the proposed townhouse residential development to be constructed at approximately 12650 South Redwood Road in Riverton, Utah. The purposes of this investigation were to assess the nature and engineering properties of the subsurface soils at the proposed site and to provide recommendations for general site grading and the design and construction of foundations, pavement sections, and slabs-on-grade.

Based on the subsurface conditions encountered at the site, it is our opinion that the subject site is suitable for the proposed construction provided that the recommendations contained in this report are complied with. The subsurface soil conditions were explored at the subject property by excavating three test pits to depths ranging from 11½ to 12½ feet below the existing site grade. Subsurface soil conditions consisted of approximately 12 inches of clayey topsoil. Based on our geologic review of the site, the topsoil is underlain by Pleistocene-aged lacustrine silt and clay deposits. Groundwater was not encountered in any of the test pits advanced as part of our investigation.

The foundation for the proposed structures may consist of conventional strip and/or spread footings founded on undisturbed native soils. Strip and spread footings should be a minimum of 20 and 36 inches wide, respectively, and exterior shallow footings should be embedded at least 30-inches below final grade for frost protection and confinement. Conventional strip and spread footings founded on undisturbed, native soils may be proportioned for a maximum net allowable bearing capacity of **1,600 psf**.

An assumed CBR of 3.0 for near-surface soils was utilized in the pavement design. Based on assumed traffic loads, a pavement section of 3 inches of asphalt over 18 inches of untreated base course is recommended. Alternatively, an equivalent pavement section of 3 inches of asphalt, 8 inches of untreated base and 14 inches of granular borrow may be utilized. A thinner pavement section may be accomplished if a woven geofabric is incorporated into the pavement design.

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

NOTE: This executive summary is not intended to replace the report of which it is part and should not be used separately from the report. The executive summary omits a number of details, any one of which could be crucial to the proper application of this report.

2.0 INTRODUCTION

2.1 PURPOSE AND SCOPE OF WORK

This report presents the results of a geotechnical investigation conducted for the proposed townhouse development to be located at approximately 12650 South Redwood Road in Riverton, Utah. The purposes of this investigation were to assess the nature and engineering properties of the subsurface soils at the proposed site and to provide recommendations for general site grading and the design and construction of foundations, pavement sections, 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, dated July 7, 2014. The recommendations contained in this report are subject to the limitations presented in the "Limitations" section of this report.

2.2 PROJECT DESCRIPTION

The roughly rectangular-shaped project site is located at approximately 12650 South between Redwood Road and 1630 West in Riverton, Utah (see Plate A-1, *Site Vicinity Map*) and has a total area of approximately 6 acres. We understand that the development as planned will include the demolition of the existing developments, and the construction of multi-family residential buildings as well as paved parking/roadway areas and landscaped areas. Construction plans were not available for our review prior to the preparation of this report, however we anticipated that the structures associated with this development will consist of 1 to 2 story, wood-framed structures with basements (where feasible) founded on standard strip or spread footings.

3.0 METHOD OF STUDY

3.1 SUBSURFACE INVESTIGATION

As part of this investigation, subsurface soil conditions were explored by advancing 3 test pits at representative locations across the site. The test pits were excavated to depths ranging from 11½ to 12½ feet below the site grade as it existed at the time of our investigation. The approximate locations of the explorations are shown on the *Exploration Location Map*, Plate A-2 in Appendix A. Our exploration points were selected to provide a representative cross-section of the subsurface soils across the site. Subsurface soil conditions as encountered in the explorations were logged at the time of our investigation by a qualified geotechnical engineer and are presented on the enclosed Test Pit Logs, Plates B-1 to B-3 in Appendix B. A *Key to USCS Soil Symbols and Terminology* is presented on Plate B-4.

The test pits were advanced using a Case 580 backhoe. Bulk soil samples were obtained in each of the test pit locations through the collection of bag and bucket samples. Relatively "undisturbed" samples were obtained through the collection of block samples as well as through driving 3-inch diameter brass tubes. All samples were transported to our laboratory for testing to evaluate engineering properties of the various earth materials observed. The soils were classified according to the *Unified Soil Classification System* (USCS) by the Geotechnical Engineer. Classifications for the individual soil units are shown on the attached Test Pit Logs.

3.2 LABORATORY TESTING

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

- Grain Size Distribution Analysis (ASTM D422)
- Atterberg Limits Test (ASTM 4318)
- 1-D Consolidation Test (ASTM D2435)

The results of laboratory tests are presented on the Test Pit Logs in Appendix B (Plates B-1 to B-9), the Laboratory Summary Table, and the test result plates presented in Appendix C (Plates C-1 to C-5).

3.3 ENGINEERING ANALYSIS

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

4.0 GENERALIZED SITE CONDITIONS

4.1 SURFACE CONDITIONS

At the time of our subsurface investigation, the property was occupied by a mix of residential and commercial structures, as well as several areas of vacant property. Improvements at the site include 3 relatively large commercial structures with associated paved and unpaved parking areas, as well as two residential structures with associated paved driveways and landscaped areas. Vegetation at the site was largely isolated to the vacant and residential portions of the property, and included grass, weeds, and occasional mature trees. The site is relatively flat, with a maximum topographic relief of approximately 6 feet. The properties to north and east are occupied by commercial developments, whereas the properties to the south and east are occupied by residential developments.

4.2 SUBSURFACE CONDITIONS

As mentioned previously, the subsurface soil conditions were explored at the subject property by advancing three test pits to depths ranging from 11½ to 12½ feet below the site grade as it existed at the time of our investigation. Subsurface soil conditions were logged during our field investigation and are included on the Test Pit Logs in Appendix B (Plates B-1 to B-3). The soil and moisture conditions encountered during our investigation are discussed below.

4.2.1 Soils

Based on our field observations, the site is overlain by approximately 12-inches of clayey topsoil. Considering the pre-existing developments located on the property, it is likely that large portions of the site will also be overlain by undocumented fill soils, the thicknesses of which could vary greatly depending on the original topography of the site. Underlying the topsoil we encountered Pleistocene-aged lacustrine clay and silt deposits. Descriptions of the soil units encountered are described below:

<u>Topsoil:</u> Generally consists of stiff, moist, dark brown Lean CLAY (CL) with varying amounts of sand. Typically displays trace 'pinhole' structure. This unit also has an organic appearance and texture, with roots throughout. Topsoil was encountered in each of the test pits and is expected to overlie the majority of the site.

Pleistocene-Aged Lacustrine Silt and Clay Deposits: Where observed, these soils generally consisted of stiff, moist, brown to light brown Lean CLAY (CL) with varying amounts of fine sand. Each of the test pits also contained a 12-inch thick layer of fine-grained Silty SAND (SM) at a depths ranging from 6½ to 7 feet. These soils generally had low plasticity, and occasionally contained fine pinholes.

The stratification lines shown on the enclosed Test Pit Logs represent the approximate boundary between soil types. The actual in-situ transition may be gradual. Due to the nature and depositional characteristics of the native soils, care should be taken in interpolating subsurface conditions between and beyond the exploration locations.

4.2.2 Groundwater Conditions

Groundwater was not encountered in any of the test pits excavated as part of this investigation. Seasonal fluctuations in precipitation, surface runoff from adjacent properties, or other on or offsite sources may increase moisture conditions; groundwater conditions can be expected to rise several feet during wetter years and seasonally depending on the time of year. However, it is not anticipated that groundwater will impact the proposed development.

4.2.3 Collapse Potential

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

Soils that have a potential to collapse under increased loading and moisture conditions are typically characterized by a pinhole structure and relatively low unit weights. In general, potentially collapsible soils are observed in fine-grained soils that include clay and silt, although collapsible soils may include sandy soils. Results of our laboratory testing indicated that the subsurface soils have a low collapse potential, with the collapse potential ranging from 0.00 to

0.05 percent. As such, it is anticipated that collapsible soils will not significantly impact the foundation elements within the proposed development.

5.0 GEOLOGIC CONDITIONS

5.1 GEOLOGIC SETTING

The site is located in Riverton, Utah at an elevation of approximately 4,440 feet above mean sea level within the southern portion of the Salt Lake Valley. The Salt Lake Valley is a deep, sediment-filled structural basin of Cenozoic age flanked by the Wasatch Range to the east and the Oquirrh Mountains, the Promontory Mountains, and the West Hills to the west (Hintze, 1980). A portion of western boundary of the Salt Lake Valley is bordered by the eastern shore of the Great Salt Lake. The Wasatch Range is the easternmost expression of pronounced Basin and Range extension in north-central Utah.

The near-surface geology of the Salt Lake Valley is dominated by sediments, which were deposited within the last 30,000 years by Lake Bonneville (Scott and others, 1983; Hintze, 1993). As the lake receded, streams began to incise large deltas that had formed at the mouths of major canyons along the Wasatch Range, and the eroded material was deposited in shallow lakes and marshes in the basin and in a series of recessional deltas and alluvial fans. Sediments toward the center of the valley are predominately deep-water deposits of clay, silt and fine sand. However, these deep-water deposits are in places covered by a thin post-Bonneville alluvial cover. Surface sediments at the site are mapped as Pleistocene-aged lacustrine silt and clay deposits (Personius and Scott, 1992).

5.2 SEISMICITY AND FAULTING

The site lies within the north-south trending belt of seismicity known as the Intermountain Seismic Belt (ISB) (Hecker, 1993). The ISB extends from northwestern Montana through southwestern Utah. An active fault is defined as a fault that has had activity within the Holocene (<11ka). No active faults are mapped through or immediately adjacent to the site (Black et. al, 2003, Hecker, 1993). The site is located approximately 5¼ miles west of the nearest mapped portion of the of the Salt Lake City segment of the Wasatch Fault Zone, which is mapped along the western flank of the Wasatch Mountains and the Salt Lake Salient. The Salt Lake City segment of the Wasatch Fault Zone was reportedly last active approximately 1,800 years ago and has a recurrence interval of approximately 2,400 years (Black et. al., 1996, Black et. al., 2003). The site is also located approximately 9½ miles south of the mapped Taylorsville fault. The Taylorsville fault is one of two main splays of the West Valley fault zone (Keaton and Curry, 1993). The West Valley fault zone trends in a north-south orientation and is located in the central portion of the Salt Lake Valley. While the West Valley fault zone is reported to be active and probably seismically independent of the Wasatch fault zone, sympathetic movement on the West Valley fault zone resulting from major earthquakes on the Wasatch fault zone Salt Lake City segment of the Wasatch fault zone is a possibility. Finally, the site is also located approximately 14 miles east of the Oquirrh Fault Zone. The Oquirrh Fault Zone consists of a normal fault located along the western base of the Oquirrh Mountains in the eastern Tooele Valley. This fault was reportedly last active approximately 4,300 and 6,900 years ago, and appears to be seismically independent of the Wasatch Fault Zone (Black and others, 2004). Analyses of ground shaking hazard along the Wasatch Front suggests that the Wasatch Fault Zone is the single greatest contributor to the seismic hazard in the Salt Lake City region. Each of the faults listed above show evidence of Holocene-aged movement, and is therefore considered active.

Seismic hazard maps depicting probabilistic ground motions and spectral response have been developed for the United States by the U.S. Geological Survey as part of NEHRP/NSHMP (Frankel et al, 1996). These maps have been incorporated into both *NEHRP Recommended Provisions for Seismic* regulations for new buildings and other structures (FEMA, 1997) and the international building code (IBC) (International Code Council, 2009). Spectral responses for the maximum considered earthquake (MCE) are shown in the table below. These values generally correspond to a two percent probability of exceedance in 50 years (2PE50) for a "firm rock" site. To account for site effects, site coefficients which vary with the magnitude of spectral acceleration are used. Based on our field exploration, it is our opinion that this location is best described as a Site Class D. The spectral accelerations are shown in the table below. The spectral accelerations are calculated based on the site's approximate latitude and longitude of 40.5206 and -111.9378° respectively and the United States Geological Survey 2009 ground motion calculator version 5.1.0 (USGS, 2011). Based on the IBC, the site coefficients are F_a=1.01 and F_v= 1.50. From this procedure the peak ground acceleration (PGA) is estimated to be 0.49g. The MCE PGA and design response spectrum are presented in Appendix D on Plate D-1.

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

Site Location: **Latitude = 40.5206 N** **Longitude = -111.9378 W**	**Site Class D Site Coefficients:** **Fa = 1.01** **Fv = 1.50**
Spectral Period (sec)	**Response Spectrum Spectral Acceleration (g)**
0.2	$S_{MS}=(F_a*S_s=1.00=1*1.22) = 1.24$
1.0	$S_{M1}=(F_v*S_1=1.50*0.51) = 0.76$

[a] IBC 1615.1.3 recommends scaling the MCE values by 2/3 to obtain the design spectral response acceleration values; values reported in the table above have not been reduced.

5.3 OTHER GEOLOGIC HAZARDS

Geologic hazards can be defined as naturally occurring geologic conditions or processes that could present a danger to human life or property or result in increased construction costs. These hazards must be considered before development of the site. There are several hazards in addition to seismicity and faulting that if present at the site, should be considered in the design of critical and essential facilities such as communication towers. The other identified geologic hazards considered for this site are liquefaction and lateral spread. A complete list of potential geologic hazards is included in the *Summary of Geologic Hazards Table* in Appendix D (Plate D-2).

5.3.1 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 settlements of overlying layers after an earthquake as excess pore water pressures are dissipated. The primary factors affecting liquefaction potential of a soil deposit are: (1) level and duration of seismic ground motions; (2) soil type and consistency; and (3) depth to groundwater.

Based on our review of the *Surface Rupture Liquefaction Potential Special Study Areas, Salt Lake County, Utah* prepared by the Salt Lake County Planning and Development Services Division, the site is located in an area currently designated as having a "Very Low" potential for liquefaction. "Very Low" liquefaction potential indicates that there is less than a 5% probability of having an earthquake within a 100-year period that will be strong enough to cause liquefaction. The majority of the soils observed within our test pits consisted of fine-grained sediment, which is typically not considered susceptible to liquefaction. As such, the near-surface soils are not considered to be susceptible to liquefaction and the "Very Low" designation appears to be appropriate. A liquefaction analysis was beyond the scope of the project; however, if the owner wishes to have greater understanding of the liquefaction potential of the soils at greater depths, a liquefaction analysis should be completed at the site.

5.3.2 Lateral Spread

Another hazard associated with seismic events is lateral spread. The areas most commonly affected by lateral spread are areas that have an appreciable slope or open channels, cut slopes, streams banks, etc. Mapping completed by the Utah Liquefaction Advisory Group indicates that the subject property is located within an area expected to experience 0 to 0.1 meters of deformation during a magnitude 7 earthquake. It is anticipated that this deformation would be towards the west (towards the Jordan River). A lateral spread analysis was not completed for this site. If the owner would like to have a greater understanding of the lateral spread potential of the site, a lateral spread analysis should be completed.

6.0 ENGINEERING ANALYSIS AND RECOMMENDATIONS

6.1 GENERAL CONCLUSIONS

Supporting data upon which the following recommendations are based have been presented in the previous sections of this report. The recommendations presented herein are governed by the physical properties of the earth materials encountered and tested as part of our subsurface exploration and the anticipated design data discussed in the **PROJECT DESCRIPTION** section. If subsurface conditions other than those described herein are encountered in conjunction with construction, and/or if design and layout changes are initiated, GeoStrata must be informed so that our recommendations can be reviewed and revised as changes or conditions may require.

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

6.2 EARTHWORK

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

6.2.1 General Site Preparation and Grading

Within areas to be graded (below proposed structures, fill sections, concrete flatwork, or pavement sections), any existing vegetation, debris, topsoil, undocumented fill, or otherwise unsuitable soils should be removed. Any soft, loose, or disturbed soils should also be removed. Following the removal of vegetation, unsuitable soils, and loose or disturbed soils, as described above, site grading may be conducted to bring the site to design elevations.

Based on our observations in the test pits excavated for the site investigation, there is approximately 12 inches of clayey topsoil overlying the subject site. This material should be removed prior to placement of structural fill, structures, concrete flatwork and roadways. Considering the presence of existing structures located on the property, as well as the potential demolition of these structures, it is likely that at least some undocumented fill will be encountered on the property. These soils should also be removed prior to placement of structural fill, structures, concrete flatwork and roadways. If over-excavation is required, the excavation should extend a minimum of one foot laterally for every foot of depth of over-excavation. Excavations should extend laterally at least two feet beyond flatwork, pavements, and slabs-on-grade. If materials are encountered that are not represented in the test pit logs or may present a concern, GeoStrata should be notified so observations and further recommendations as required can be made.

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

6.2.2 Soft Soil Stabilization

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

In large areas of soft subgrade soils, stabilization of the subgrade may not be practical using the method outlined above. In these areas it may be more economical to place a woven geotextile fabric against the soft soils covered by 18 inches of coarse, sub-rounded to rounded material over the woven geotextile. An inexpensive non-woven geotextile "filter" fabric should also be placed over the top of the coarse, sub-rounded to rounded fill prior to placing structural fill or pavement section soils to reduce infiltration of fines from above. The woven geotextile should be Amoco 2004 or prior approved equivalent. The filter fabric should consist of an Amoco 4506, Amoco 4508, or equivalent as approved by the Geotechnical Engineer.

6.2.3 Excavation Stability

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

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

6.2.4 Structural Fill and Compaction

All fill placed for the support of structures, concrete flatwork or pavements should consist of structural fill. Structural fill may consist of a reworked, native soil, although the contractor should be aware that it can be difficult to moisture condition and compact the fine grained soils to the specified maximum density. Alternatively, an imported fill meeting the specifications below may be used. Imported structural fill should be a relatively well graded granular soil with a maximum of 50 percent passing the No. 4 mesh sieve and a maximum fines content (minus No.200 mesh sieve) of 25 percent. Clay and silt particles in imported structural fill should have a liquid limit less than 35 and a plasticity index less than 15 based on the Atterberg Limit's test (ASTM D-4318). Regardless if the structural fill is imported or native, it fill should be free of vegetation, debris or frozen material, and should contain no inert materials larger than 4 inches nominal size. All structural fill soils should be approved by the Geotechnical Engineer prior to placement. The contractor should anticipate testing all soils used as structural fill frequently to assess the maximum dry density, fines content, and moisture content, etc.

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

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

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

6.3 FOUNDATIONS

The foundation for the proposed structure may consist of conventional strip and/or spread footings founded on undisturbed native soils. Strip and spread footings should be a minimum of 20 and 36 inches wide, respectively, and exterior shallow footings should be embedded at least 30-inches below final grade for frost protection and confinement.

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

6.4 SETTLEMENT

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

6.5 EARTH PRESSURES AND LATERAL RESISTANCE

Lateral forces imposed upon conventional foundations due to wind or seismic forces may be resisted by the development of passive earth pressures and friction between the base of the footing and the supporting subgrade. In determining the frictional resistance, a coefficient of friction of 0.41 should be used for structural fill or drain gravel against concrete. A coefficient of friction of 0.28 should be used for clayey native soils against concrete.

Ultimate lateral earth pressures from *granular* backfill acting against buried walls and structures may be computed from the lateral pressure coefficients or equivalent fluid densities presented in the following table:

Condition	Lateral Pressure Coefficient	Equivalent Fluid Density (pounds per cubic foot)
Active*	0.31	38
At-rest**	0.52	63
Passive*	5.60	670
Seismic Active***	0.23	28
Seismic Passive***	-1.25	-150

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

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

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

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

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

6.6 CONCRETE SLAB-ON-GRADE CONSTRUCTION

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

6.7 MOISTURE PROTECTION AND SURFACE DRAINAGE

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

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

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

6.9 PAVEMENT SECTION

An assumed CBR value for the near surface subgrade soils of 3 was used in our analysis, as it is anticipated that the near-surface soils will provide relatively poor pavement support. No traffic information was available at the time this report was prepared, therefore, GeoStrata has assumed traffic counts for access roads and parking areas. We assumed that vehicle traffic in and out of paved area would consist of approximately 600 passenger car trips per day, 4 small trucks per day, and 1 large truck per day with a 20 year design life. Based on these assumptions our analysis used 105,000 ESAL's for the traffic over the life of the pavement. Asphalt has been assumed to be a high stability plant mix and base course material (road base) composed of crushed stone with a minimum CBR of 70. We have further assumed that the traffic will be relatively consistent over the design life of the pavement sections. Therefore, no growth factor was applied in calculation of loading for each pavement sections' design life. Based on this information we recommend the following pavement section;

Flexible Pavement Section	
Asphalt Concrete (in)	Untreated Base Course (in)
3	18

Alternatively, the following equivalent pavement section may be used;

Flexible Pavement Section		
Asphalt Concrete (in)	Untreated Base Course (in)	Granular Borrow (in)
3	8	14

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

Flexible Pavement Section With Geosynthetic fabric	
Asphalt Concrete (in)	Untreated Base Course (in)
3	11

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

7.0 CLOSURE

7.1 LIMITATIONS

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

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

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

7.2 ADDITIONAL SERVICES

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

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

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

We appreciate the opportunity to be of service on this project. Should you have any questions regarding the report or wish to discuss additional services, please do not hesitate to contact us at your convenience at (801) 501-0583.

8.0 REFERENCES CITED

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Appendix A

GeoStrata

Newman Construction
Townhome Development
Riverton, UT
Project Number: 178-070

Site Vicinity Map

Plate
A-1

GeoStrata

Legend

Approximate Test Pit Location

Approximate Site Boundary

Newman Construction
Townhome Development
Riverton, UT
Project Number: 178-070

Exploration Location Map

Plate A-2

Appendix B

DATE	
STARTED:	7/9/14
COMPLETED:	7/9/14
BACKFILLED:	7/9/14

Newman Construction
Townhome Development
Riverton, UT
Project Number 178-070

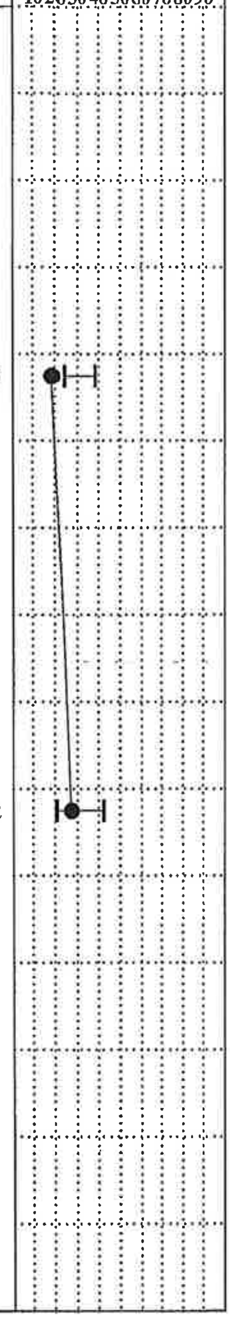
GeoStrata Rep: D. Brown
Rig Type: Case 580

TEST PIT NO:
TP-1
Sheet 1 of 1

LOCATION
NORTHING EASTING ELEVATION

DEPTH (FEET)	UNIFIED SOIL CLASSIFICATION	MATERIAL DESCRIPTION	Dry Density(pcf)	Moisture Content %	Percent minus 200	Liquid Limit	Plasticity Index
0		TOPSOIL; Lean CLAY with sand - dark brown, slightly moist					
	CL	Lean CLAY - stiff, brown, slightly moist to moist, some pinholes					
~4 (sample)			89.8	18.2	96.3	38	15
	SM	Silty SAND - dense, light brown, moist					
	CL	Lean CLAY - stiff, brown, moist					
~9 (sample)			95.4	27.1	99.3	42	22
12.5		Bottom of Test Pit @ 12.5 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:

GeoStrata

Plate B-1

DATE	
STARTED:	7/9/14
COMPLETED:	7/9/14
BACKFILLED:	7/9/14

Newman Construction
Townhome Development
Riverton, UT
Project Number 178-070

GeoStrata Rep: D. Brown
Rig Type: Case 580

TEST PIT NO:
TP-2
Sheet 1 of 1

LOCATION: NORTHING | EASTING | ELEVATION

Moisture Content and Atterberg Limits: Plastic Limit | Moisture Content | Liquid Limit (10 20 30 40 50 60 70 80 90)

DEPTH (METERS)	DEPTH (FEET)	SAMPLES	WATER LEVEL	GRAPHICAL LOG	UNIFIED SOIL CLASSIFICATION	MATERIAL DESCRIPTION	Dry Density(pcf)	Moisture Content %	Percent minus 200	Liquid Limit	Plasticity Index
0	0					TOPSOIL; Lean CLAY with sand - dark brown, slightly moist					
					CL	Lean CLAY - stiff, brown, slightly moist to moist, some pinholes					
1		Grab									
	5										
2					SM	Silty SAND - dense, light brown, moist					
					CL	Lean CLAY - stiff, brown, moist					
3	10										
		Grab									
						Bottom of Test Pit @ 12 Feet					
4											

GeoStrata

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

WATER LEVEL
- MEASURED
- ESTIMATED

NOTES:

Plate B-2

DATE	
STARTED:	7/9/14
COMPLETED:	7/9/14
BACKFILLED:	7/9/14

Newman Construction
Townhome Development
Riverton, UT
Project Number 178-070

GeoStrata Rep: D. Brown
Rig Type: Case 580

TEST PIT NO: **TP-3**
Sheet 1 of 1

LOCATION
NORTHING EASTING ELEVATION

MATERIAL DESCRIPTION

Depth (Meters)	Depth (Feet)	Samples	Water Level	Unified Soil Classification	Material Description	Dry Density (pcf)	Moisture Content %	Percent minus 200	Liquid Limit	Plasticity Index
0	0				TOPSOIL; Lean CLAY with sand - dark brown, slightly moist					
				CL	Lean CLAY - stiff, brown, slightly moist to moist, some pinholes					
		Grab sample				97.4	9.8	90.6	29	10
2		Grab sample		SM	Silty SAND - dense, light brown, moist					
				CL	Lean CLAY - stiff, brown, moist					
3	10	Grab sample								
					Bottom of Test Pit @ 11.5 Feet					

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

GeoStrata

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

WATER LEVEL
- MEASURED
- ESTIMATED

NOTES:

Plate B-3

UNIFIED SOIL CLASSIFICATION SYSTEM

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

LOG KEY SYMBOLS

BORING SAMPLE LOCATION

TEST-PIT SAMPLE LOCATION

WATER LEVEL (level after completion)

WATER LEVEL (level where first encountered)

CEMENTATION

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

OTHER TESTS KEY

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

MODIFIERS

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

MOISTURE CONTENT

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

STRATIFICATION

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

GENERAL NOTES

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

APPARENT / RELATIVE DENSITY - COARSE-GRAINED SOIL

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

CONSISTENCY - FINE-GRAINED SOIL

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

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Soil Symbols Description Key

Newman Construction
Townhome Development
Riverton, UT
Project Number: 178-070

Plate B-4

Appendix C

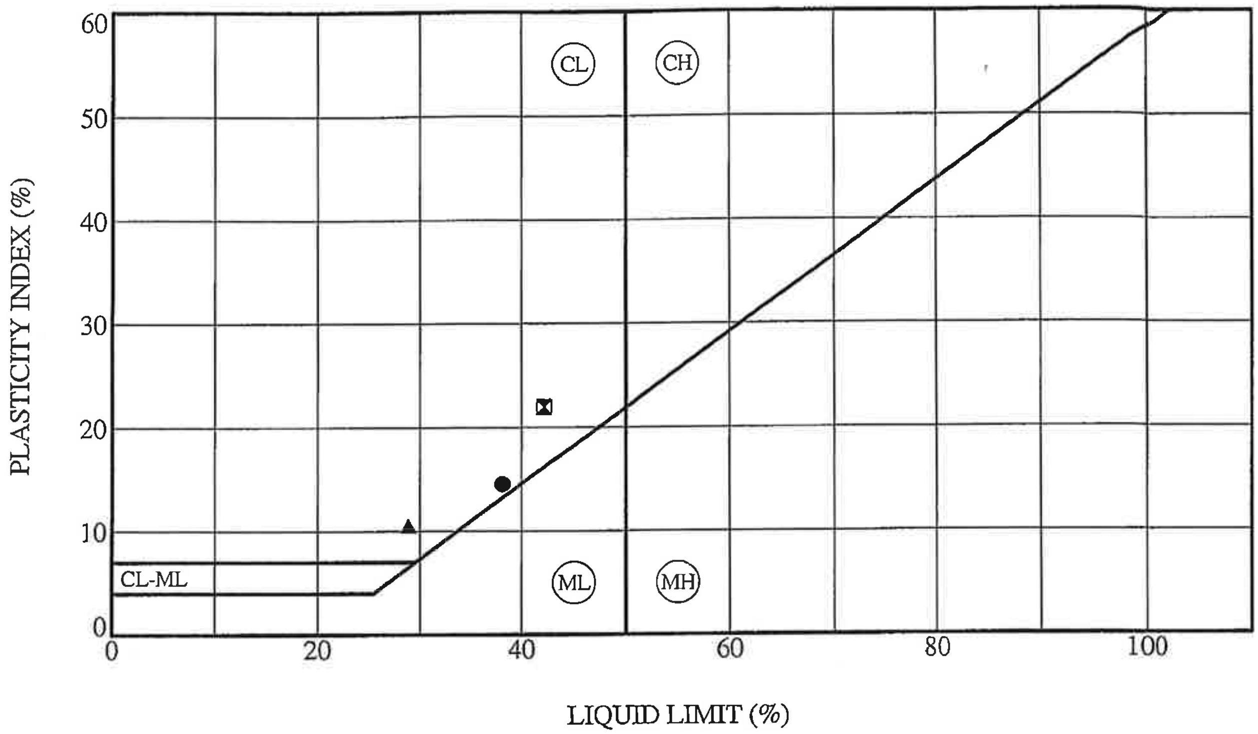
Test Pit No.	Sample Depth (feet)	USCS Soil Classification	Natural Moisture Content (%)	Natural Dry Density (pcf)	Gradation			Atterberg Limits		Consolidation			
					Gravel (%)	Sand (%)	Fines (%)	LL	PI	Cc	Cr	OCR	Collapse (%)
TP-1	4.0	CL	18.2	89.8	3.7		96.3	38	14	0.136	0.023	5.0	0.00
TP-1	9.0	CL	27.1	95.4	0.7		99.3	42	22	0.075	0.028	5.0	0.02
TP-3	2.0	CL	9.8	97.4	9.4		90.6	29	11	0.133	0.018	5.0	0.05

GeoStrata

Lab Summary Report

Newman Construction
Townhome Development
Riverton, UT
Project Number: 178-070

Plate
C-1



	Sample Location	Depth (ft)	LL (%)	PL (%)	PI (%)	Fines (%)	Classification
●	**TP-1**	**4.0**	**38**	**24**	**14**	**96.3**	**Lean CLAY**
⊠	**TP-1**	**9.0**	**42**	**20**	**22**	**99.3**	**Lean CLAY**
▲	**TP-3**	**2.0**	**29**	**18**	**11**	**90.6**	**Lean CLAY**

C_ATTERBERG TEST PIT LOGS.GPJ GEOSTRATA.GDT 7/21/14

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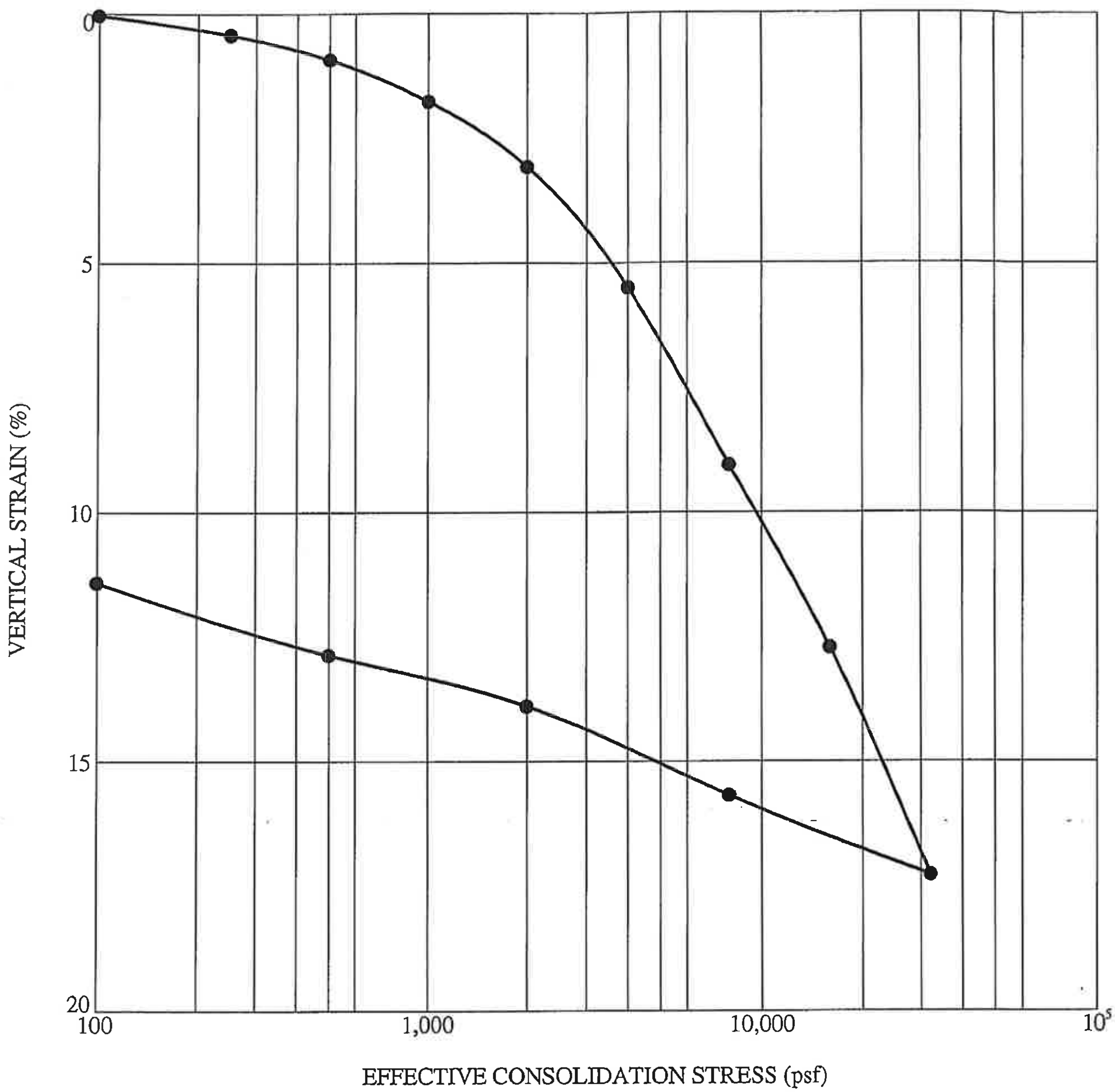
ATTERBERG LIMITS' RESULTS - ASTM D 4318

Newman Construction
Townhome Development
Riverton, UT
Project Number: 178-070

Plate C - 2



	Sample Location	Depth (ft)	Classification	γ_d (pcf)	MC (%)	C'_c	C'_r	OCR	Inundation Load (psf)	Swell (%)	Collapse (%)
●	**TP-1**	**4.0**	**Lean CLAY**	**90**	**18**	**0.136**	**0.023**	**5.0**	**1000**	----	**0.00**
⊠											
▲											
★											
⊙											

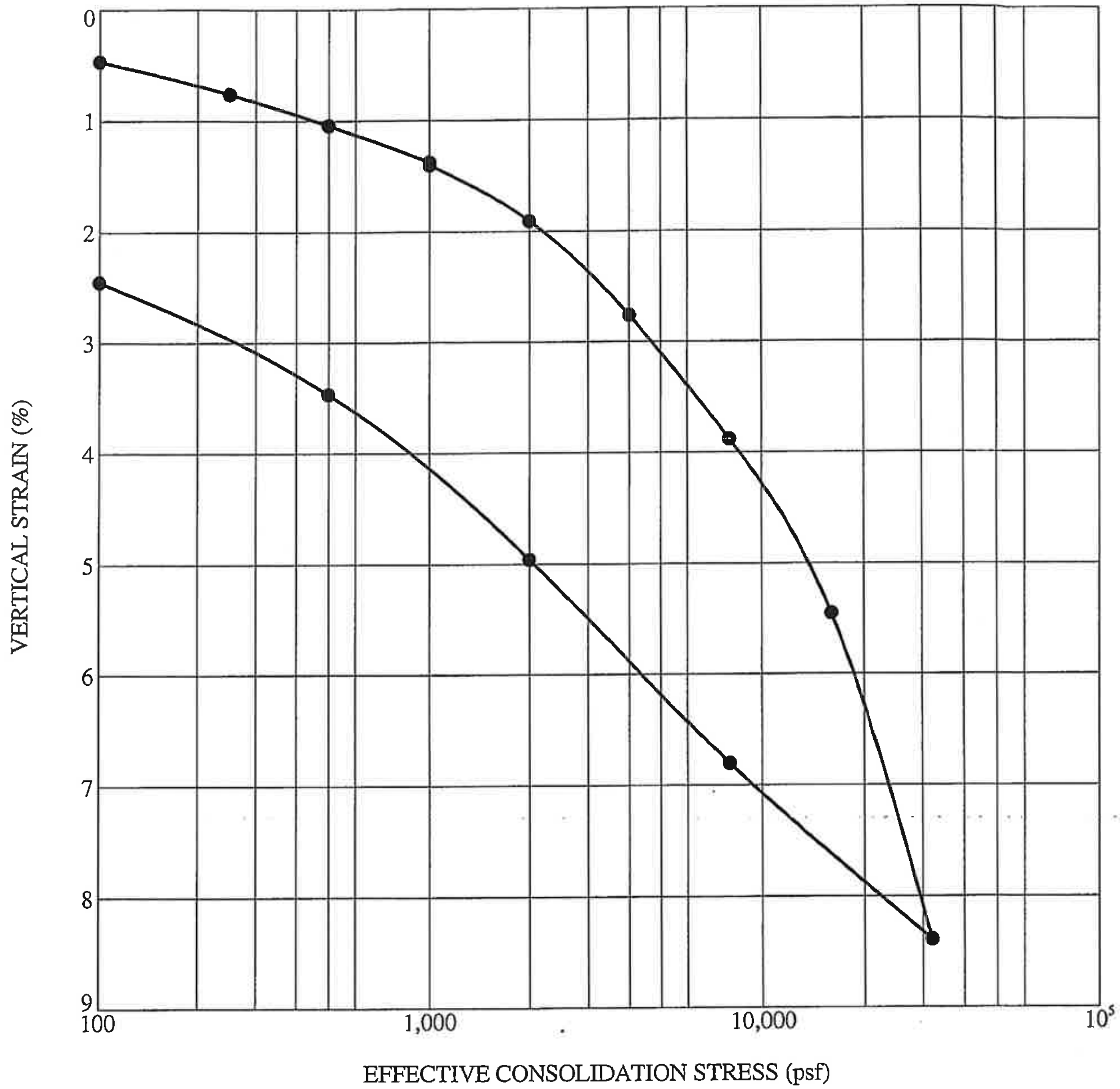
GeoStrata

1-D CONSOLIDATION/SWELL/COLLAPSE TEST

Newman Construction
Townhome Development
Riverton, UT
Project Number: 178-070

Plate

C - 3



	Sample Location	Depth (ft)	Classification	γ_d (pcf)	MC (%)	C'_c	C'_r	OCR	Inundation Load (psf)	Swell (%)	Collapse (%)
●	**TP-1**	**9.0**	**Lean CLAY**	**95**	**27**	**0.075**	**0.028**	**5.0**	**1000**	**---**	**0.02**
⊠											
▲											
★											
⊙											

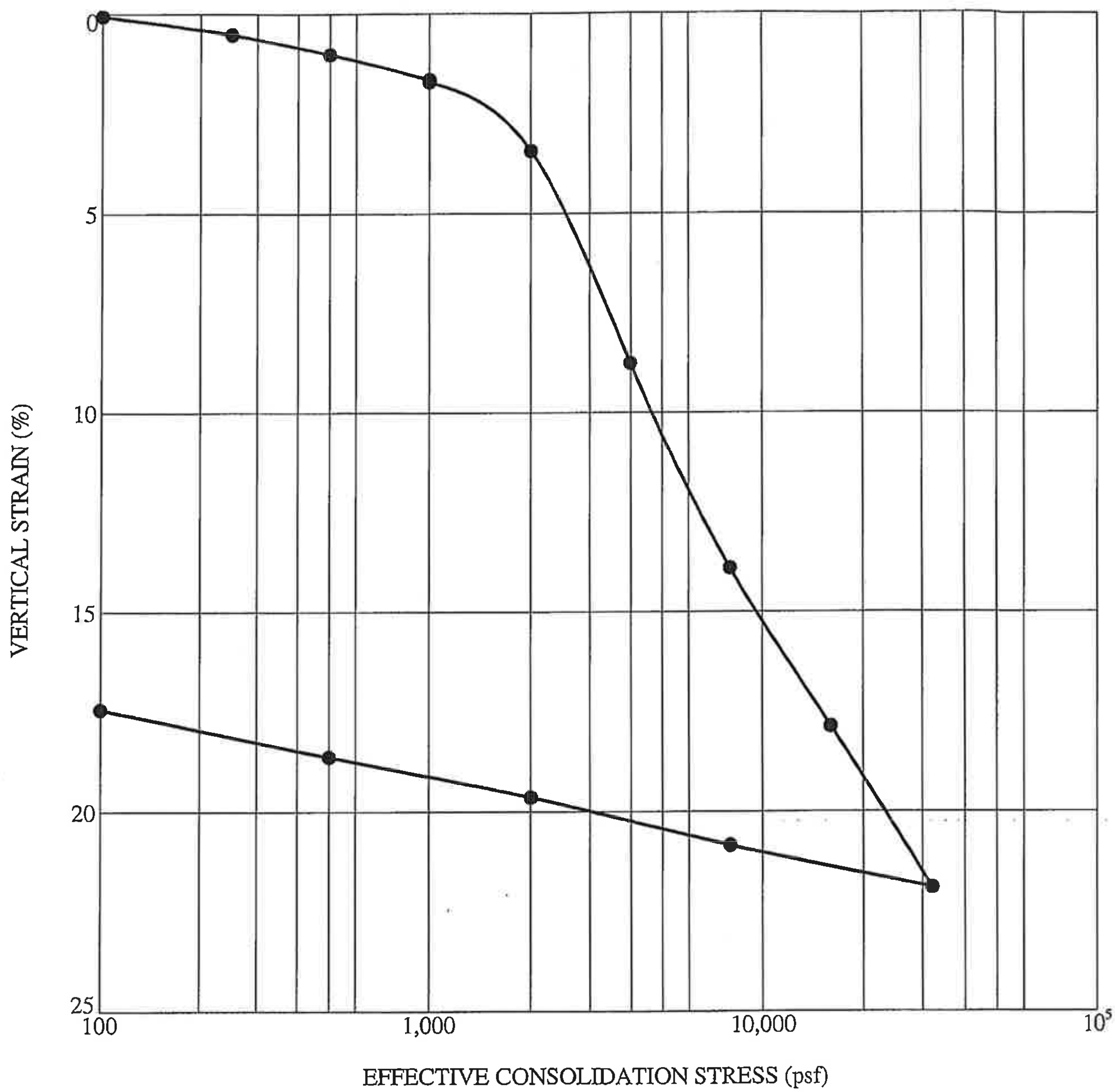
GeoStrata

1-D CONSOLIDATION/SWELL/COLLAPSE TEST

Newman Construction
Townhome Development
Riverton, UT
Project Number: 178-070

Plate C - 4

C_CONSOL SWELL/COLLAPSE TEST PIT LOGS.GPJ GEOSTRATA.GDT 7/21/14



	Sample Location	Depth (ft)	Classification	γ_d (pcf)	MC (%)	C'_c	C'_r	OCR	Inundation Load (psf)	Swell (%)	Collapse (%)
●	**TP-3**	**2.0**	**Lean CLAY**	**97**	**10**	**0.133**	**0.018**	**5.0**	**1000**	**---**	**0.05**
⊠											
▲											
★											
⊙											

GeoStrata

1-D CONSOLIDATION/SWELL/COLLAPSE TEST

Newman Construction
Townhome Development
Riverton, UT
Project Number: 178-070

Plate C - 5

C_CONSOL SWELL/COLLAPSE TEST PIT LOGS.GPJ GEOSTRATA.GDT 7/21/14

Appendix D

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

Project: Riverton Residential Development
Geotechnical Investigation
Project No.: 173-070
Project Location: Riverton, Utah
Date: Friday, July 18, 2014
Engineer: JSS

Site Coordinates:
Latitude: **40.5206** degrees
Longitude: **-111.9378** degrees

Exceedance Probability: **2** %
Exposure Time: **50** years
S_s = **1.224** From USGS 2002 Probabilistic Seismic
S_1 = **0.509** Hazard Maps for 2475-year Return Period

Site Soil Class: **D** (Stiff soil)
F_a = 1.01
F_v = 1.50

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

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

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

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

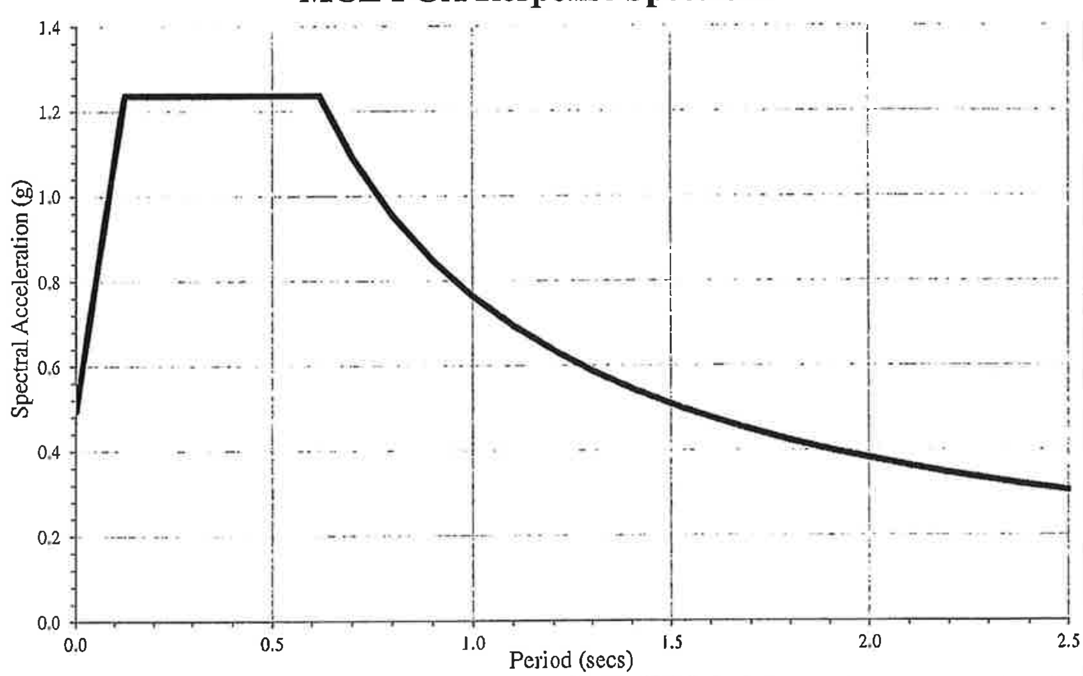
Adjusted for Site Conditions:
$S_{MS} = F_a \times S_S =$ (1.01 x 1.22) = 1.24 g
$S_{M1} = F_v \times S_1 =$ (1.50 x 0.51) = 0.76 g

MCE PGA = 0.4 x S_{MS} = (0.4 x 1.24) = **0.49** g
MCE $T_0 = 0.2 \times (S_{M1}/S_{MS})$ = (0.2 x [0.76/1.24]) = 0.12 secs
MCE $T_S = (S_{M1}/S_{MS})$ = (0.76/1.24) = 0.62 secs

Response Time Step, ΔT = **0.1**

Period (sec)	MCE Spectral Acceleration (g)
0.00	0.49
0.12	1.24
0.62	1.24
0.70	1.09
0.80	0.95
0.90	0.85
1.00	0.76
1.10	0.69
1.20	0.64
1.30	0.59
1.40	0.55
1.50	0.51
1.60	0.48
1.70	0.45
1.80	0.42
1.90	0.40
2.00	0.38
2.10	0.36
2.20	0.35
2.30	0.33
2.40	0.32
2.50	0.31

MCE PGA Response Spectrum



D-1

SUMMARY OF GEOLOGIC HAZARDS

Riverton Residential Development **Project Number 178-070**

Hazard	Hazard Rating*				Further Study Recommended**
	Not Assessed	Probable	Possible	Unlikely	
Earthquake					
Ground Shaking		X			See Geotechnical Report
Surface Faulting				X	
Tectonic Subsidence				X	
Liquefaction				X	See Geotechnical Report
Slope Stability				X	
Flooding (Including Seiche)				X	
Slope Failure					
Rock Fall				X	
Landslide				X	
Debris Flow				X	
Avalanche	X				
Problem Soils					
Collapsible				X	See Geotechnical Report
Soluble				X	
Expansive				X	
Organic				X	
Piping				X	
Non-Engineered Fill		X			See Geotechnical Report
Erosion				X	
Wind Blown Sand				X	
Mine Subsidence				X	
Shallow Bedrock				X	
Shallow Groundwater				X	See Geotechnical Report
Flooding					
Streams				X	
Alluvial Fans				X	
Lakes				X	
Dam Failure				X	
Canals/Ditches				X	
Radon	X				

* Hazard Rating :
Not assessed - report does not consider this hazard and no inference is made as to the presence or absence of the hazard at the site
Probable -Evidence is strong that the hazard exists and mitigation measures should be taken
Possible - hazard may exist, but the evidence is equivocal, based only on theoretical studies, or was not observed and furthes study is necessary as noted
Unlikely - no evidence was found to indicate that the hazard is present, hazard not known or suspected to be present

Further Study :
E - geotechnical/engineering, H - hydrologic, A - Avalanche, G - Additional detailed geologic hazard study out of the scope of this study