

**GEOTECHNICAL ENGINEERING REPORT  
MCINTYRE FARMS SUBDIVISION**

**PROPERTY LOCATION:  
12350 SOUTH 3600 WST  
RIVERTON, UT**

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## **1. INTRODUCTION**

This report presents the geotechnical investigation for the proposed McIntyre Farms Subdivision to be located near the intersection of 12340 South Street and 3600 West Street in Riverton, Utah as shown on the vicinity map in the Appendix. The geotechnical investigation was performed in accordance with Wilding Engineering's proposal.

The field investigation consisted of five (5) test pits. Test Pits (TP-1 through TP-5) were excavated to depths ranging from 11½ feet to 12½ feet below the existing ground surface. Detailed test pit logs can be found in the Appendix. Recommendations in this report are based upon information gathered from the field investigation, site inspection, lab testing, and from reviewing geologic maps and reports of the area.

## **2. PURPOSE AND SCOPE**

The purpose of this investigation was to determine the suitability of on-site soils for the development of a residential subdivision consisting of single family residences with the associated utilities, roadways, and driveways. The investigation includes a review of surface water and ground water conditions and their affects. Engineering and construction recommendations are presented based on subsurface conditions encountered in the field along with the effects of both subsurface and surface waters.

## **3. SITE AND PROJECT INFORMATION**

### **3.1. Proposed Project Description**

Based on our understanding of the project, the proposed development will consist of two story, single family residential buildings with the associated utilities and roadway. We understand the buildings will be constructed with typical wood framed walls with a 6 to 8-foot basement. The proposed site is approximately 10 acres in area and will consist of 31 lots. Loading information was not available at the time of this report. Based on our experience and understanding of the proposed construction, maximum column and wall loads are assumed to be about 50 kips and 4 kip/ft, respectively. A site map is located in the Appendix of this report.

Recommendations presented in this report are based upon the current available information. If the assumed building loads or any information presented is incorrect or has changed, please inform Wilding Engineering in writing so that we may amend the recommendations presented in this report appropriately.

### **3.2 Existing Site Conditions**

The proposed residential subdivision consists of about 10 acres in area and is located near the southwest corner of the intersection of 12340 South Street and 3600 West Street in Riverton, Utah. More specifically, the site is located at latitude 40.526317 degrees and Longitude -111.978627 degrees.

At the time of our field investigation, the project site was a vacant agricultural land. We also observed an existing home and storage area located in the south-central portion of the site. This portion of the site also consists of landscaped area with trees. It is our understanding that the house will be removed. Based on available topographic information, the subject site slopes downward to the east. The subject site is bounded by existing 3600 West Street on the east, vacant land and commercial buildings on the south, and existing single family residences on the west and north sides.

#### **4. SURFICIAL GEOLOGY**

Based on the available geologic maps, the project site is underlain by silt and clay deposits. The soils encountered in the soil profile consisted of cohesive soils underlain by granular soils.

#### **5. FIELD EXPLORATIONS**

##### **5.1 Subsurface Investigation**

Subsurface conditions at the project site were evaluated with five (5) test pits designated TP-1 through TP-5 as indicated on Site Map with Test Pit Locations presented in the Appendix. The test pits were excavated using a trackhoe to depths ranging from 11½ feet to 12½ feet below the existing site grades. Stratigraphy and classification of the soils were logged under the direction of a geotechnical engineer.

Disturbed samples were obtained at various depths and examined in the field and representative portions were stored in sealed plastic bags. Undisturbed samples were not feasible due to the presence of stiff cohesive soils and granular soils. The samples were transported to our laboratory for further examination and testing. The test pits were backfilled to the ground surface with on-site soils. Sample types with depths are shown in detail in the Test Pit Logs found in the Appendix.

##### **5.2 Subsurface Conditions**

###### **5.2.1 Soils**

The soil profile encountered in the test pits consisted of about 2 to 3 feet of topsoil underlain by layers of Fat Clay (CH), Silty Clayey Sand (SC-SM), Lean Clay to Sandy Lean Clay (CL), Silt (ML) with varying amounts of Sand, Silty Sand (SM), Clayey Gravel with Sand (GC), and Poorly Graded Gravel with Silt and Sand (GP-GM) to the maximum depth of exploration or 12½ feet.

For a detailed description of the materials and conditions encountered at each test pit locations, please refer to the Test Pit Logs in the Appendix.

The subsurface profile descriptions above are a generalized interpretation provided to highlight the major subsurface stratification features and material characteristics. The test pit logs included in the Appendix should be reviewed for more specific information. The stratifications shown on the test pit logs represent the conditions only at each test pit

log location. The stratifications represent the approximate boundary between subsurface materials and the transition may be gradual.

### **5.2.2 Groundwater**

Groundwater was not encountered during any of the test pits to the maximum depth of 12½ feet. It should be noted that it is possible for the ground water levels to fluctuate during the year depending on the season and climate. Additionally, discontinuous zones of perched water may exist at various locations and depths beneath the ground surface. This could result in encountering ground water conditions during construction which may have been different than during our field investigation. If ground water is encountered at during construction, Wilding Engineering must be notified to amend our recommendations, as appropriate.

## **6 LABORATORY TESTING**

Representative soil samples were tested to evaluate physical and engineering properties. Laboratory testing included: natural water content, grain size analysis, and Atterberg Limits testing. Lab results are presented on the Test Pit Logs and Summary of Lab Results in the Appendix.

## **7 RECOMMENDATIONS AND CONCLUSIONS**

### **7.1 Geotechnical Discussion**

Wilding Engineering, Inc. has provided the following recommendations based on the information provided by the client and the soils encountered during our field investigation for the proposed construction. The proposed site is suitable for the development of the residential property if the recommendations of this report are followed.

### **7.2 Site Work**

#### **7.2.1 Site Preparation**

It is the contractor's responsibility to locate and protect all existing utility lines, whether shown on the drawings or not.

About 2 to 3 feet of topsoil was encountered during our investigation. All existing vegetation, trees, roots, and topsoil shall be grubbed and removed from the site or stockpiled for use in landscaped areas. Deeper or shallower excavations may be required locally. Topsoil stockpile shall be a safe distance away from any on-site or imported fill material to prevent contamination.

We understand that the existing structure located in the south-central portion of the site will be removed. Removal of the structure will require proper backfill of the remaining excavations. Construction debris, i.e, brick or concrete rubble, steel and wood etc, are not suitable backfill and should be removed from the site. All deleterious material should be removed from the site prior to beginning backfill operations. Loose or otherwise unstable soils at the bottom of the excavations should be stabilized prior to fill placement. The excavations should be backfilled using structural fill meeting the

requirements of section 7.2.4. The material should be placed in no greater than 8-inch loose lifts and compacted to 95 percent of the maximum dry density and within 2 percent of the optimum moisture as determined by ASTM D 1557 (Modified Proctor). All backfill operations should be monitored and documented by a representative of the Geotechnical Engineer

Upon completion of site stripping and prior to placement of any fill material, the exposed subgrade should be evaluated by a representative of the Geotechnical Engineer. Proof rolling with construction equipment may be a part of this evaluation. Soils that are observed to rut or deflect excessively (typically greater than 1-inch) under the moving load of a loaded rubber-tired loaded dump truck (typically, 9 ton/axle) or other suitable construction vehicle should be over-excavated down to firm undisturbed native soils and backfilled with properly placed and compacted structural fill.

If the subgrade is disturbed during construction, disturbed soils should be recompacted and tested or it should be over-excavated to firm, undisturbed soil and backfilled with compacted granular materials.

Footing excavations should be made using an excavator equipped with a smooth edge and supported from outside the excavation to minimize disturbance to the natural soil subgrade. If the subgrade is disturbed during construction, disturbed soils should be recompacted and tested or it should be over-excavated to firm, undisturbed soil and backfilled with compacted granular materials.

#### **7.2.2 Excavation Consideration**

All utility excavations shall be carefully supported, maintained, and protected during construction in accordance with OSHA Regulations as stated in 29 CFR Part 1926. It is the responsibility of the contractor to have safe working conditions. Temporary construction excavations shall be properly sloped or shored, in compliance with current federal, state, and local requirements.

Construction excavations up to 4 feet deep may be constructed with near-vertical side slopes. Excavations between 4 feet and 10 feet deep shall have side slopes not steeper than 1H:2V, or a trench box or shoring may be used. Excavations are to be made to minimize subsequent filling. Coarse-grained material can easily become unstable and is anticipated in localized areas to experience toppling, cave-in or sliding. Boulders and cobbles larger than six inches shall be removed from trenches.

Wilding Engineering does not assume responsibility for construction site safety or the contractor's or other parties' compliance with local, state, and federal safety or other regulations. As stated in the OSHA regulations, "a competent person shall evaluate the soil exposed in the excavations as part of his/her safety procedures". In no case should slope height, slope inclination, or excavation depth, including utility trench excavation depth, exceed those specified in local, state, and federal safety regulations.

### 7.2.3 Cut and Fill

Permanent cut and fill slopes not exceeding ten (10) feet in depths shall not be steeper than 2H:1V horizontal to vertical. If anticipated cut and fill heights exceed ten (10) feet, or slopes are required to exceed 2H:1V, we should be notified so that we can provide additional stability analysis and recommend improvements and slopes. It is recommended that permanent slopes be maintained and vegetated to prevent possible erosion.

### 7.2.4 Structural Fill Material

Structural fill shall consist of well-graded granular material, with a maximum aggregate size of 2 inches, and a maximum of 15% passing the #200 sieve. The fill material which is finer than the number 40 sieve shall have a liquid limit (LL) less than 35 and a Plastic Index (PI) less than 25, see table 7.2.4 for gradation specification. This material shall be free from organics, garbage, frost, and other loose, compressible, or deleterious materials.

**Table 7.2.4 Structural Fill Requirements**

Grain Size	Percent Passing
2-inch	100
¾-inch	85 to 100
No. 4	15 to 45
No. 200	< 15
Plastic Index (PI)	< 25
Liquid Limit (LL)	< 35

Subsurface soils profile at the site predominantly consists of granular soils. These soils can be used as fill provided they do not contain material greater than 4 inches in size and the specified compaction requirements are achieved.

### 7.2.5 Fill Placement and Compaction

Fill under footings, interior floor slabs, concrete flatwork, driveway, and utilities should be placed in nine (9) inch lifts (loose) and shall be compacted to at least 95% of the modified proctor (maximum dry density as determined by the ASTM D 1557 method of compaction). Landscaped areas are to be compacted to at least 90% of the modified proctor. Each lift shall be tested for adequate compaction (see section 7.3.1 for fill placement and compaction under foundations). Wilding Engineering can provide this service under an additional agreement.

### 7.2.6 Utility Trenches

Construction of the pipe bedding shall consist of preparing an acceptable pipe foundation, excavating the pipe groove in the prepared foundation and backfilling from



the foundation to 12 inches above the top of the pipe. All piping shall be protected from lateral displacement and possible damage resulting from impact or unbalanced loading during backfilling operations by being adequately bedded. In our experience individual municipalities will have local requirements regarding installation of utilities. However, in the absence of specified requirements the following is recommended:

The soils in the utility pipe trenches are to meet the specified structural fill requirements in Section 7.2.4.

**Pipe foundation:** shall consist of native soils if the soils are stable and undisturbed. Wherever the trench subgrade material does not afford a sufficiently solid foundation to support the pipe and superimposed load, the trench shall be excavated below the bottom of the pipe to such depth as may be necessary, to eliminate unstable soils and this additional excavation filled with compacted well-graded, granular soil (per 7.2.5), compacted to 95% of the modified proctor.

**Pipe groove:** shall be excavated in the pipe foundation to receive the bottom quadrant of the pipe so that the installed pipe will be true to line and grade. Bell holes shall be dug after the trench bottom has been graded. Bell holes shall be excavated so that only the barrel of the pipe bears on the pipe foundation.

**Pipe bedding:** (from pipe foundation to 12 inches above top of pipe) shall be deposited and compacted in layers not to exceed 9 inches in uncompacted depth. Deposition and compaction of bedding materials shall be done simultaneously and uniformly on both sides of the pipe. All bedding materials shall be placed in the trench in such a manner that they will be scattered alongside the pipe and not dropped into the trench in compact masses.

Backfill for utility trenches located beneath roadways shall be compacted to 95% of the modified proctor. In non-load bearing areas (landscape), trenches shall be compacted to 90% of the modified proctor (ASTM D 1557).

#### **7.2.7 Native Soil As Fill**

The native soils generally consist of cohesive soils in the upper 5 feet. Clayey and silty soils are generally not acceptable as fill, because of the difficulty in achieving compaction due to their moisture sensitivity. We recommend that a well-graded granular material be imported as per the gradation requirements presented in Table 7.2.4.

#### **7.2.8 Surface Drainage**

A grading and drainage plan shall be prepared for the site by a qualified engineer and shall be followed for the site drainage. Generally, each building site shall be graded in such a manner that surface water will flow away from the buildings foundations. Natural drainage is generally from west to east. Surface water should be prevented from entering trenches during construction. An embankment may be used to divert any storm water from construction areas and directed into temporary retention basin.

### **7.3 Foundations**

#### **7.3.1 Installation and Bearing Material**

Footings must be placed entirely on firm undisturbed native soils or entirely on 2 feet of structural fill which is bearing on native soils (below the existing topsoil layer) and is compacted to 95% of the modified proctor (maximum dry density as determined with ASTM D1557 method of test). Any existing topsoil shall be removed from the areas where footings are to be located. All load bearing soils which are disturbed or considered soft or loose areas are unsuitable for support for foundations and should be removed down to firm native soils and properly replaced and compacted with structural fill within  $\pm 2\%$  of the optimum moisture content. Bottom of footing must be at least 2 feet above groundwater level.

All organic material, soft areas, frozen material or other inappropriate material shall be removed from the footing zone to a depth determined by the Geotechnical Engineer and be replaced with structural fill. Foundations shall have minimum dimensions of 18-inches wide for continuous wall footings and 24-inches square for isolated column footings correlating to the prescribed bearing pressure. Footings placed on slopes shall be "benched" so that all footing bases are horizontal and are not sloped.

**Footing excavations shall be inspected by a Geotechnical Engineer prior to placement of structural fill, concrete or reinforcement steel to verify their suitability for placement of the footings.**

#### **7.3.2 Bearing Pressure**

Based on our experience and understanding of the proposed construction, maximum column and continuous wall loads are assumed to be about 50 kips and 4 kip/ft, respectively. Footings bearing entirely on firm undisturbed native soils or entirely on structural fill may be designed with a maximum allowable bearing capacity of **2,000 pounds per square foot (psf)**, or a subgrade modulus value of **100 pounds per cubic inch (pci)**. Consolidation settlement was limited to 1 inch. The recommended allowable bearing pressure refers to the total dead load and may be increased by 1/3 to include the sum of all transient loads including wind and seismic.

#### **7.3.3 Settlement**

The total settlement is anticipated not to exceed 1-inch, which is the recommended maximum settlement for these types of structures. Differential settlement is expected to approach about 50 to 75 percent of the total settlement under static conditions.

#### **7.3.4 Frost Depth**

All exterior footings are to be at least 30 inches below the ground surface to protect against possible frost heave. This includes walk-out areas. This may require fill to be placed around buildings. Slab on grade construction, interior footings require 18 inches of cover. If foundations are constructed through the winter months, all soils on which footings will bear shall be protected from freezing.

### 7.3.5 Construction Observation

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

### 7.3.6 Foundation Drainage

Wilding Engineering recommends footings and foundations be designed according to the International Building Code (IBC 2012). According to the IBC 2012, foundation drain shall be installed if the native soils have poor drainage characteristics, to allow water to drain away from the foundation. During our field investigation, soils encountered had significant amount of fines content. These soils are "not" considered "free draining". We recommend that a footing drain be installed.

## 7.4 Lateral Forces

### 7.4.1 Resistance for Footings

Wind and seismic forces, which cause lateral loads on foundations, are resisted by friction and passive earth pressures at the foundation ground interface. In the design of spread footings against shear forces, the total dead weight is multiplied by the coefficient of friction for lateral sliding ( $\mu$ ) which is estimated to be 0.25 for sands, and the resistance of lateral sliding is 130 psf for clays and silts.<sup>1</sup>

### 7.4.2 Lateral Earth Pressures on Foundation Walls

The following equivalent fluid weights are given for the design of sub-grade walls and retaining structures. Basement, foundation and retaining walls shall be designed to resist lateral soil loads.

Basement walls and other walls in which horizontal movement is restricted at the top and bottom (non-yielding) shall be designed for at-rest lateral earth pressure based on the equivalent fluid having a unit weight of 55 pcf for horizontal backfill and 80 pcf for backfill slopes upward at 2H:1V (26.7°). At-rest equivalent fluid pressure is a product of the soil unit weight times the coefficient of earth pressure at rest for coarse grained soils (Jaky, 1944).

Retaining walls free to move and rotate at the top are permitted to be designed for active pressure (Coulombs 1776). **Exception:** Basement walls extending not more than 8 feet below grade and supporting flexible floor systems shall be permitted to be designed for active pressure."<sup>2</sup> Both active and passive earth pressure coefficients and equivalent fluid pressures are provided in Table 7.4.1. Passive earth pressures are typically neglected in design to be conservative. However, they may be used, if required, as it can be expected that they will develop as active pressure increases. The equivalent

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<sup>1</sup> International Building Code 2006, Ch. 18, Table 1804.2

fluid pressures below assume that the backfill material is fully drained where pore water pressures are not allowed to build up behind the wall. For coarse grained material correlations were used to estimate the internal angle of friction,  $\phi$ , as 32 degrees<sup>3</sup>.

**Table 7.4.2 Static Conditions**

Equivalent Fluid Pressures and Coefficients				
Conditions	$K\gamma$	$\gamma$	K	2H:1V Slope
At-rest ( $K_o\gamma$ )	55 pcf	120	$K_o=0.47$	80 pcf
Active ( $K_a\gamma$ )	35 pcf	120	$K_a=0.31$	56 pcf
Passive ( $K_p\gamma$ )	390 pcf	120	$K_p=3.25$	Not Applicable

### 7.4.3 Seismic Conditions

Under dynamic conditions, at rest earth pressure for non-yielding walls can be estimated using the procedure presented by Seed and Whitman (1970). The static component is known to act at H/3 above the base of the wall. Seed and Whitman (1970) recommended that it would be appropriate for the dynamic component be taken to act at approximately 0.6H for non-yielding walls. Non-yielding walls can be designed based on a seismic at-rest component of 27 pcf. This component shall be included in addition to the static equivalent at-rest earth pressure value from above.

The Mononobe-Okabe M-O Method (Mononobe and Matsuo (1929); Okabe (1924) and Kapila (1962)) is reused in determining active and passive, respectively, seismic earth pressure coefficients. Determining seismically induced active and passive lateral earth pressures is an extension of the Coulomb theory for static stress conditions. The method entails three fundamental assumptions:

- The driving soil wedge and the retaining structure act as rigid bodies and therefore experience uniform accelerations throughout the respective bodies.
- The driving soil wedge inducing the lateral earth pressures is formed by a planar failure surface starting at the base and extending to the free surface at the top of the wall with backfill. The maximum shear strength of the backfill is mobilized along this failure plane
- Wall movement (flexibility) is sufficient to ensure either active or passive conditions, as the case may be.

Active and passive seismic components have been estimated using the M-O method for seismic design in retaining walls. Coulomb's theory overestimates the passive resistance of walls and is generally neglected in wall design.

<sup>2</sup> International Building Code 2012, Section 1610, Table 1610.1

<sup>3</sup> Bowles, Joseph E., PE, PhD, Foundation Analysis and Design, fifth edition, 1996

**Table 7.4.3 Dynamic Conditions**

<b>Yielding Wall Dynamic Pressures and Coefficients</b>			
<b>Conditions:</b>	<b>Component</b>	<b><math>\gamma</math></b>	<b>K</b>
<b>Active</b>	130 pcf	120	$K'_a=1.39$
<b>Passive</b>	220 pcf	120	$K'_p=1.39$

The active seismic component shall be included in addition to the static equivalent active pressure value and, if relied upon, the passive seismic component shall be included as a reduction in the static passive resistance value.

During backfill placement and compaction below grade or behind retaining walls, the contractor shall use caution. Retaining walls can experience excessive build up of lateral pressures when backfill is over-compacted. We recommend using manual compaction practices (jumping jack, etc.). Avoid unnecessary large equipment or heavy items from being placed or operated within 5 feet of any un-braced concrete foundation or basement wall. Backfill material should meet IBC 2012 requirements and should not have aggregate greater than 3 inches in size.

### **7.5 Concrete Slabs on Grade**

Floor slabs are to be entirely supported on either suitable firm native soils or on imported structural fill placed which shall be compacted to 95% of the modified proctor (maximum dry density as determined by the ASTM D 1557 method of compaction) extending to the undisturbed native soils. It is recommended that areas immediately below any exposed concrete, i.e., driveway, sidewalks and patios, be placed with six (6) inches coarse aggregate base to distribute floor loads and provide proper drainage. Floor slabs to receive tile flooring shall have a minimum of four (4) inches of coarse aggregate base placed immediately below slabs. Floor slabs shall have adequate number of joints set by the structural engineer to reduce cracking resulting from any differential movements and shrinkage.

### **7.6 Seismic Information**

#### **7.6.1 Faulting**

Based on the Salt Lake County Geologic Hazards Map the project site is located about 9¼ miles south of Granger Fault of the West Valley Fault Zone and 7½ miles west of the Salt Lake City Section of the Wasatch Fault Zone. Surface rupture has not been mapped and was not observed at the site. The International Building Code (IBC 2012), and the USGS National Earthquake Hazards Reduction Program (NEHRP) interpolated probabilistic ground motion values for  $S_s$  and  $S_1$  are 1.10g and 0.44g, respectively. Values from the NEHRP were estimated with 40.526317 degrees and longitude of -111.978627 degrees. (See table below).



**Table 7.6.1 USGS Earthquake Hazards Estimated Values**

	2% PE in 50 year	10% PE in 50 year
Peak Ground Acceleration (g)	0.453	0.215
0.2 sec Spectral Acceleration (g)	1.102	0.511
1.0 sec Spectral Acceleration (g)	0.439	0.173

The design spectral accelerations were determined according to IBC 2012 and ASCE 07-05 and were found to be 0.78g and 0.496 for  $S_{DS}$  and  $S_{D1}$  respectively. The figure below shows the spectral response parameters used to develop the design values and a code specified response spectrum for the site based upon a site class of "D" for a stiff soil profile.

## Seismic Provisions ASCE 7-05

		Mapped MCE Spectral Response Acceleration Parameters $F_a$ and $F_v$				
Site Class: D	Short Period	1.6	1.4	1.2	1.1	1.0
	1 Second	2.4	2.0	1.8	1.6	1.5

Obtained  $S_s$  and  $S_1$  from <http://eqint.cr.usgs.gov/eq-men/cgi-bin/find-II-2002-interp.cgi>

$S_s$ :	1.102	$F_a =$	1.06	$S_{MS} =$	1.167	$S_{DS} =$	0.778
$S_1$ :	0.439	$F_v =$	1.56	$S_{M1} =$	0.684	$S_{D1} =$	0.456

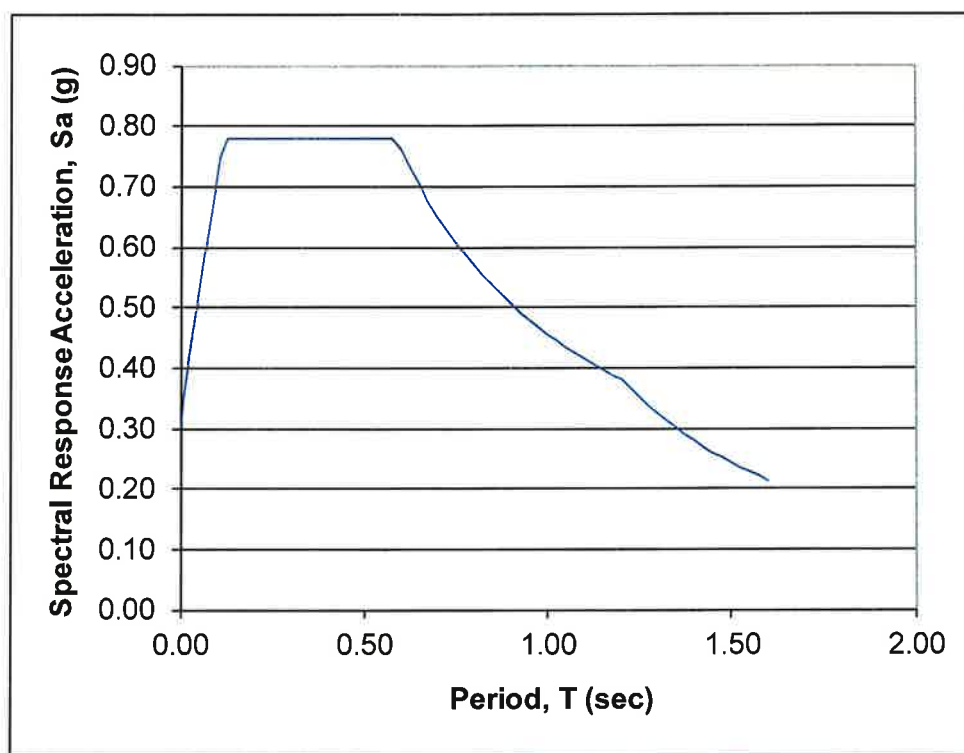


Figure 7.6 ASCE 7-05 Seismic Provisions

### 7.6.2 Liquefaction

A review of the geologic hazards maps for Salt Lake County indicate that the project site is located in an area designated as "very low" in liquefaction potential. Three conditions must be present for liquefaction to occur, in soils:

- The soil must be susceptible to liquefaction, i.e., granular layers with less than fifteen percent fines, existing below the groundwater table.
- The soil must be in a loose state.

- Ground shaking strong enough to cause liquefaction.

Test Pits were excavated to depths of about 12½ feet below existing ground surface. Groundwater was not encountered in any of the test pits. Based on our subsurface investigation, the subsurface profile encountered cohesive soils underlain by granular soils. Based on the subsurface soils encountered in the upper 12½ feet, the soils are not likely to liquefy during a seismic event.

### 7.6.3 Structures

Structures are to be designed for lateral loading as defined in the International Building Code. The site location has a design spectral response acceleration of 0.78g for short periods ( $S_{DS}$ ) and 0.46g for a one second period ( $S_{D1}$ ). Lateral loading is to be the greater of seismic loads or wind loads.

## 7.7 Pavement Design and Construction

A pavement design has been prepared for the anticipated roadway. On-site soil characteristics from the test pit samples collected were used in determining soil strength. The pavement design assumptions consist of traffic of about 50,000 Equivalent Single Axle Loads (ESALs) with a twenty (20) year design period of 80% reliability, a California Bearing Ratio CBR of 2 (from lab test result), standard deviation of 0.35, and Initial and Terminal serviceability of 4.2 and 2.5, respectively. The following sections will provide preparation and design for pavement based on AASHTO design procedures.

### 7.7.1 Sub-grade Preparation

All topsoil, or any soil containing organic materials, must be removed from locations where structural loads will be applied. To evaluate its stability, the sub-grade shall be "proof rolled" with a loaded dump truck or tested with a nuclear density gauge. Any unsuitable soils shall be removed and replaced with structural fill according to Section 7.2.4. Any areas of fill or disturbed areas shall be compacted to 95% of the ASTM D1557 modified proctor. A geotechnical engineer shall observe unsuitable sub-grade remediation.

### 7.7.2 Base Course

A minimum of eight (8) inches of untreated base course is required for the roadway. The base course shall comply with a ¾-inch mix per UDOT Standard Specifications, Section 02721, "Untreated Base Course."

**Table 7.7.2 Pavement Design Recommended Thickness**

Pavement Materials	Recommended Minimum Thickness (inches)
	Drive Areas
Asphaltic Concrete	3
Granular Base Course	8



### 7.7.3 Surface Course

A minimum of three (3) inches of asphalt concrete pavement is required for the roadway. This asphalt concrete pavement is to comply with UDOT Standard Specifications, Section 02741, and "Hot Mix Asphalt (HMA)."

### 7.7.4 Drainage and Maintenance

Drainage shall be designed to ensure direct positive surface water away from proposed buildings and into proper discharge locations. Water shall not be allowed to puddle in low areas of the pavement. Pooling areas could decrease the design life of the asphalt and cause cracking or uplift. Periodic seasonal maintenance should be anticipated by sealing cracks and joints. A storm drainage plan is suggested to detain and convey storm water. IBC 2012 recommends that a minimum of five percent gradient for a ten feet distance away from any structures.

Soils encountered in the test pits in the proposed residential subdivision had significant amount of fines content at varying depths. These soils are "not" considered "free draining". We recommend that a basement footing drain be installed.

## 8 LIMITATIONS AND PROFESSIONAL STATEMENT

This report has been prepared in accordance with generally accepted geologic and geotechnical engineering practices in the area for the use of the client for design purposes. The conclusions and recommendations included within the report are based on the information obtained from the borings and test pits advanced at the locations indicated on the site map, laboratory test results, and our engineering analysis. Variations in the subsurface conditions may not become evident until additional exploration or excavation is conducted. If the subsurface soil or ground water conditions are found to be significantly different than that which is described in this report, we should be notified so that we can re-evaluate recommendations.

We appreciate the opportunity of providing this service for you. If you have any questions concerning this report or require additional information or services, please contact us at 801-553-8112.

Report prepared by:  
WILDING ENGINEERING, INC.



**Chad P. Bhongir, PE**  
Geotechnical Engineer

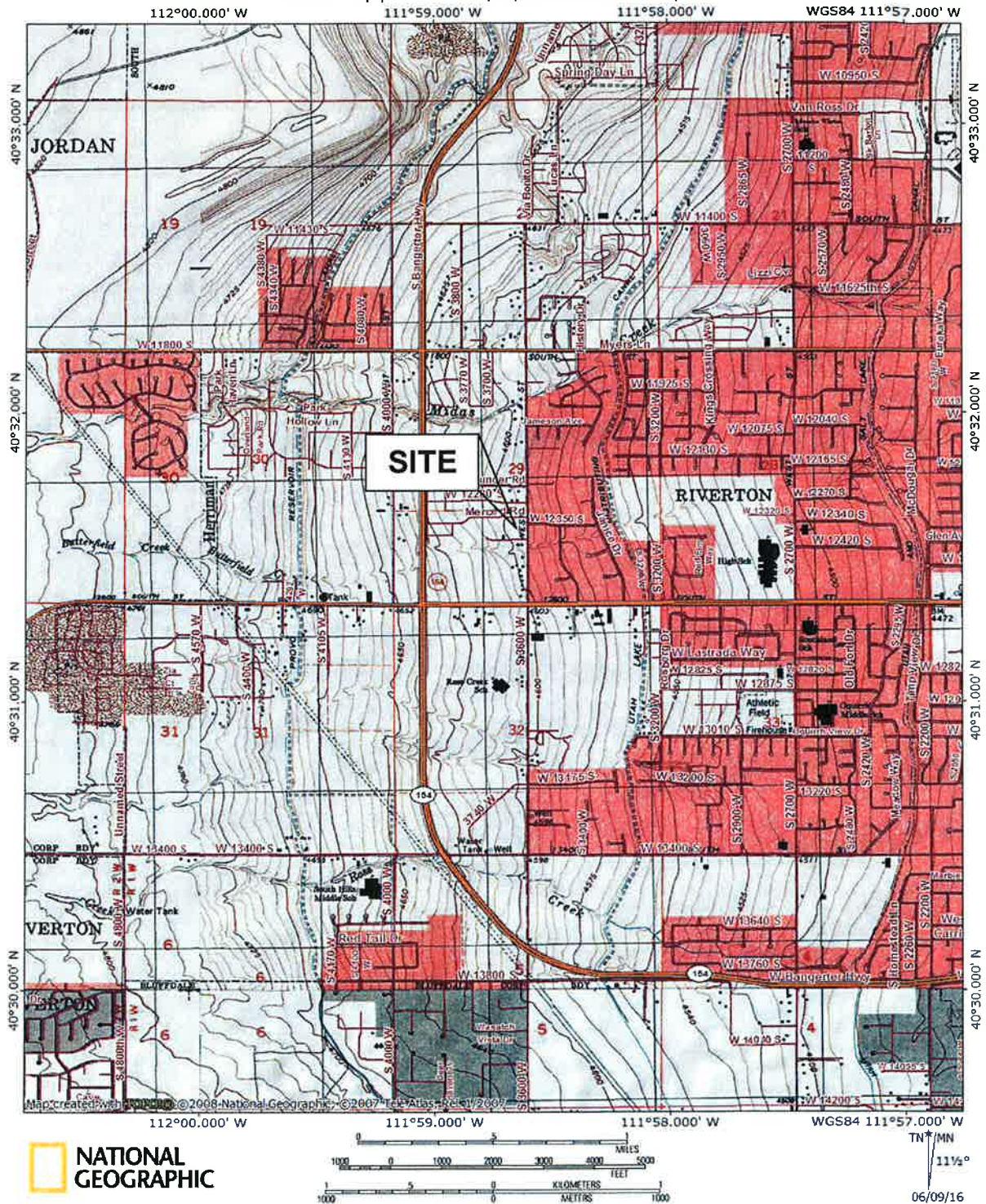


# **APPENDIX**



# VICINITY MAP

TOPO! map printed on 06/09/16 from "Untitled.tpo"



Project:

**McIntyre Farms Subdivision**  
**12350 South 3600 West**  
**Riverton, Utah**

Project No: **16090**

Date: June 2016

Drawn By: CPB

Figure: A-1



# SITE MAP WITH TEST PIT LOCATIONS



 Approximate Test Pit Location



PROJECT NAME  
MCINTYRE FARMS SUBDIVISION

DATE  
6/10/16

SCALE  
As Referenced

DRAWN  
CPB

CHECKED  
MEC

PROJECT #  
16090

SHEET

A-2

FILE NAME:  
G:\Data\16090\dwg\Site Map.dwg





Photograph 1 – Looking West from Eastern End of the Site



Photograph 2 – Looking West from South-Central Portion of the Site

Figure A-3A







Photograph 3 – View of Existing House in the South-Central Portion of the Site



Photograph 4 – View of Storage Area in the South-Central Portion of the Site

Figure A-3B

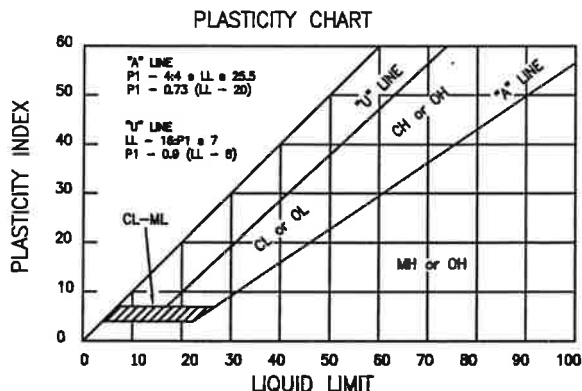


# UNIFIED SOIL CLASSIFICATION SYSTEM

Soils are visually classified for engineering purposes by the Unified Soil Classification System. Grain-sized analyses and Atterberg Limits tests often are performed on selected samples to aid in classification. The classification system is briefly outlined on this chart. Graphic symbols are used on boring logs presented on this report. For a more detailed description of the system, see "Standard Practice for Description and Identification of Soils (Visual-Manual Procedure)" ASTM Designation: 2488-84 and "Standard Test Method for Classification of Soils for Engineering Purposes" ASTM Designation: 2487-85.

MAJOR DIVISIONS			GRAPHIC SYMBOL	GROUP SYMBOL	TYPICAL NAMES
COARSE-GRAINED SOILS (Less than 50% passes No. 200 sieve)	GRAVELS (50% or less of coarse fraction passes No. 4 sieve)	CLEAN GRAVELS (Less than 5% passes No. 200 sieve)		GW	WELL GRADED GRAVELS, GRAVEL-SAND MIXTURES, OR SAND-GRAVEL-COBBLE MIXTURES
				GP	POORLY GRADED GRAVELS, GRAVEL-SAND MIXTURES, OR SAND-GRAVEL-COBBLE MIXTURES
		GRAVELS WITH FINES (More than 12% passes No. 200 sieve)		GM	SILTY GRAVELS, GRAVEL-SAND-SILT MIXTURES
				GC	CLAYEY GRAVELS, GRAVEL-SAND-CLAY MIXTURES
	SANDS (50% or more of coarse fraction passes No. 4 sieve)	CLEAN SANDS (Less than 5% passes No. 200 sieve)		SW	WELL GRADED SANDS, GRAVELLY SANDS
				SP	POORLY GRADED SANDS, GRAVELLY SANDS
		SANDS WITH FINES (More than 12% passes No. 200 sieve)		SM	SILTY SANDS, SAND-SILT MIXTURES
				SC	CLAYEY SANDS, SAND-CLAY MIXTURES
FINE-GRAINED SOILS (50% or more passes No. 200 sieve)	SILTS Limited plot below "A" line & hatched zone on plasticity chart	SILTS OF LOW PLASTICITY (Liquid limit less than 50)		ML	INORGANIC SILTS, CLAYEY SILTS OF LOW TO MEDIUM PLASTICITY
		SILTS OF HIGH PLASTICITY (Liquid limit 50 or more)		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS SILTY SOILS, ELASTIC SILTS
	CLAYS Limited plot above "A" line & hatched zone on plasticity chart	CLAYS OF LOW PLASTICITY (Liquid limit less than 50)		CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY, SANDY, AND SILTY CLAYS
		CLAYS OF HIGH PLASTICITY (Liquid limit 50 or more)		CH	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS, SANDY CLAYS OF HIGH PLASTICITY
	ORGANIC SILTS AND CLAYS	ORGANIC SILTS AND CLAYS OF LOW PLASTICITY (Liquid limit less than 50)		OL	ORGANIC SILTS AND CLAYS OF LOW TO MEDIUM PLASTICITY, SANDY ORGANIC SILTS AND CLAYS
		ORGANIC SILTS AND CLAYS OF HIGH PLASTICITY (Liquid limit 50 or more)		OH	ORGANIC SILTS AND CLAYS OF HIGH PLASTICITY, SANDY ORGANIC SILTS AND CLAYS
ORGANIC SOILS		PRIMARILY ORGANIC MATTER (dark in color and organic odor)		PT	PEAT

NOTE: Coarse-grained soils with between 5% and 12% passing thru No. 200 sieve and fine-grained soils with limit plotting in the hatched zone on the plasticity chart have dual classifications.



## DEFINITION OF SOIL FRACTIONS

SOIL COMPONENT	PARTICLE SIZE RANGE
Boulders	Above 12 in.
Cobbles	12 in. to 3 in.
Gravel	3 in. to No. 4 sieve
Coarse Gravel	3 in. to 3/4 in.
Fine Gravel	3/4 in. to No. 4 sieve
Sand	No. 4 to No. 200 sieve
Coarse sand	No. 4 to No. 10 sieve
Medium sand	No. 10 to No. 40 sieve
Fine sand	No. 40 to No. 200 sieve
Fines(silt and clay)	Less than No. 200 sieve



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**TEST PIT NUMBER TP-1**

PAGE 1 OF 1

**WILDING**  
ENGINEERINGCLIENT DR HortonPROJECT NAME McIntyre Farms SubdivisionPROJECT NUMBER 16090PROJECT LOCATION Riverton, UtahDATE STARTED 5/3/16 COMPLETED 5/3/16GROUND ELEVATION 4622 ft TEST PIT SIZE 60 inchesEXCAVATION CONTRACTOR Gary Lancaster

GROUND WATER LEVELS:

EXCAVATION METHOD Test PitAT TIME OF EXCAVATION ---LOGGED BY JGW CHECKED BY CPBAT END OF EXCAVATION ---

NOTES

AFTER EXCAVATION ---

GENERAL BH / TP / WELL - GINT STD US LAB.GDT - 6/10/16 10:58 - G:\DATA\16090 DR HORTON MCINTYRE FARMS\SOILS\TEST PIT LOGS\16090 MCINTYRE FARMS SUBDIVISION.GPJ

DEPTH (ft)	SAMPLE TYPE NUMBER	TESTS	U.S.C.S.	GRAPHIC LOG	MATERIAL DESCRIPTION
0.0					
					<b>TOPSOIL:</b> Lean Clay, moist, dark brown.
	GB 1				
2.5					
	GB 2	MC = 38% LL = 50 PL = 21 Fines = 97%	CH		<b>FAT CLAY:</b> moist, light brown.
5.0					
	GB 3				
			SC-SM		<b>SILTY CLAYEY SAND:</b> moist, light brown.
7.5					
	GB 4		CL		<b>LEAN CLAY WITH SAND:</b> moist, reddish brown.
	GB 5	MC = 19% LL = 43 PL = 17 Fines = 57%			<b>SANDY LEAN CLAY:</b> moist, reddish brown, with trace Gravel.
10.0			CL		
	GB 6				

Bottom of test pit at 11.5 feet.





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**TEST PIT NUMBER TP-2**

PAGE 1 OF 1

**WILDING**  
ENGINEERINGCLIENT DR HortonPROJECT NAME McIntyre Farms SubdivisionPROJECT NUMBER 16090PROJECT LOCATION Riverton, UtahDATE STARTED 5/3/16 COMPLETED 5/3/16GROUND ELEVATION 4623 ft TEST PIT SIZE 60 inchesEXCAVATION CONTRACTOR Gary Lancaster

GROUND WATER LEVELS:

EXCAVATION METHOD Test PitAT TIME OF EXCAVATION ---LOGGED BY JGW CHECKED BY CPBAT END OF EXCAVATION ---

NOTES

AFTER EXCAVATION ---

GENERAL BH / TP / WELL - GINT STD US LAB GDT - 6/10/16 10:58 - G:\DATA\16090 DR HORTON MCINTYRE FARMS\SOILS\TEST PIT LOGS\16090 MCINTYRE FARMS SUBDIVISION.GPJ

DEPTH (ft)	SAMPLE TYPE NUMBER	TESTS	U.S.C.S.	GRAPHIC LOG	MATERIAL DESCRIPTION
0.0					
	GB 1	MC = 19%			<u>TOPSOIL</u> : Lean Clay, moist, dark brown.
2.0					4621.0
2.5	GB 2		CL		<u>LEAN CLAY WITH SAND</u> : moist, light brown.
4.0					4619.0
5.0	SH 3	MC = 27% LL = NP PL = NP Fines = 75%	ML		<u>SILT WITH SAND</u> : moist, light brown.
6.0					4617.0
7.5	GB 4		SM		<u>SILTY SAND</u> : moist, light brown.
10.0					-- with Gravel.
	GB 5	MC = 12%			
11.5					4611.5

Bottom of test pit at 11.5 feet.



Wilding Engineering, Inc

**TEST PIT NUMBER TP-3**

PAGE 1 OF 1

**WILDING**  
ENGINEERINGCLIENT DR HortonPROJECT NAME McIntyre Farms SubdivisionPROJECT NUMBER 16090PROJECT LOCATION Riverton, UtahDATE STARTED 5/3/16 COMPLETED 5/3/16GROUND ELEVATION 4611 ft TEST PIT SIZE 60 inchesEXCAVATION CONTRACTOR Gary Lancaster

GROUND WATER LEVELS:

EXCAVATION METHOD Test PitAT TIME OF EXCAVATION ---LOGGED BY JGWCHECKED BY CPBAT END OF EXCAVATION ---

NOTES

AFTER EXCAVATION ---

GENERAL BH / TP / WELL - GINT STD US LAB GDT - 6/10/16 10:58 - G:\DATA\16090 DR HORTON MCINTYRE FARMS\SOILS\TEST PIT LOGS\16090 MCINTYRE FARMS SUBDIVISION.GPJ

DEPTH (ft)	SAMPLE TYPE NUMBER	TESTS	U.S.C.S.	GRAPHIC LOG	MATERIAL DESCRIPTION
0.0					<b>TOPSOIL:</b> Lean Clay, moist, dark brown.
1.0	GB 1	MC = 16%			
2.0					4609.0
2.5	GB 2		CL		<b>LEAN CLAY WITH SAND:</b> moist, light brown.
5.0	GB 3	MC = 14% LL = NP PL = NP Fines = 97%			4606.0
7.5	GB 4		ML		
9.0	GB 5	MC = 6% LL = 28 PL = 14 Fines = 49%			4602.0
10.0			GC		<b>CLAYEY GRAVEL WITH SAND:</b> moist, light brown.
12.5	GB 6				4598.5

Bottom of test pit at 12.5 feet.



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**WILDING**  
ENGINEERING**TEST PIT NUMBER TP-4**

PAGE 1 OF 1

CLIENT DR HortonPROJECT NAME McIntyre Farms SubdivisionPROJECT NUMBER 16090PROJECT LOCATION Riverton, UtahDATE STARTED 5/3/16 COMPLETED 5/3/16GROUND ELEVATION 4600 ft TEST PIT SIZE 60 inchesEXCAVATION CONTRACTOR Gary Lancaster

GROUND WATER LEVELS:

EXCAVATION METHOD Test PitAT TIME OF EXCAVATION ---LOGGED BY JGWCHECKED BY CPBAT END OF EXCAVATION ---

NOTES

AFTER EXCAVATION ---

GENERAL BH / TP / WELL - GINT STD US LAB.GDT - 6/10/16 10:58 - G:\DATA\16090 DR HORTON MCINTYRE FARMS\SOILS\TEST PIT LOGS\16090 MCINTYRE FARMS SUBDIVISION.GPJ

DEPTH (ft)	SAMPLE TYPE NUMBER	TESTS	U.S.C.S.	GRAPHIC LOG	MATERIAL DESCRIPTION
0.0					<b>TOPSOIL:</b> Lean Clay, moist, dark brown.
2.5	GB 1				
3.0	GB 2	MC = 23%			<b>LEAN CLAY:</b> moist, light brown.
5.0	GB 3				
7.5	SH 4	MC = 38% LL = 49 PL = 22 Fines = 94%	CL		
9.0	GB 5				<b>SILTY SAND:</b> moist, light brown.
10.0			SM		-- with Gravel.
11.5	GB 6				

Bottom of test pit at 11.5 feet.



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**TEST PIT NUMBER TP-5**

PAGE 1 OF 1

**WILDING**  
ENGINEERINGCLIENT DR HortonPROJECT NAME McIntyre Farms SubdivisionPROJECT NUMBER 16090PROJECT LOCATION Riverton, UtahDATE STARTED 5/3/16 COMPLETED 5/3/16GROUND ELEVATION 4601 ft TEST PIT SIZE 60 inchesEXCAVATION CONTRACTOR Gary Lancaster

GROUND WATER LEVELS:

EXCAVATION METHOD Test PitAT TIME OF EXCAVATION ---LOGGED BY JGWCHECKED BY CPBAT END OF EXCAVATION ---

NOTES

AFTER EXCAVATION ---

GENERAL BH / TP / WELL - GINT STD US LAB.GDT - 6/10/16 10:58 - G:\DATA\16090 DR HORTON MCINTYRE FARMS\SOILS\TEST PIT LOGS\16090 MCINTYRE FARMS SUBDIVISION GPJ

DEPTH (ft)	SAMPLE TYPE NUMBER	TESTS	U.S.C.S.	GRAPHIC LOG	MATERIAL DESCRIPTION
0.0					<b>TOPSOIL:</b> Lean Clay, moist, dark brown.
2.5	GB 1	MC = 20%			
3.0	GB 2		CL		<b>LEAN CLAY:</b> moist, light brown.
5.0					
7.5					
8.0					<b>POORLY GRADED GRAVEL WITH SILT AND SAND</b> moist, light brown.
10.0	GB 3	MC = 7% LL = NP PL = NP Fines = 8%	GP- GM		
11.5	GB 4				

Bottom of test pit at 11.5 feet.



**WILDING**  
ENGINEERING

Wilding Engineering, Inc

# SUMMARY OF LABORATORY TEST RESULTS

PAGE 1 OF 1

CLIENT DR Horton

PROJECT NAME McIntyre Farms Subdivision

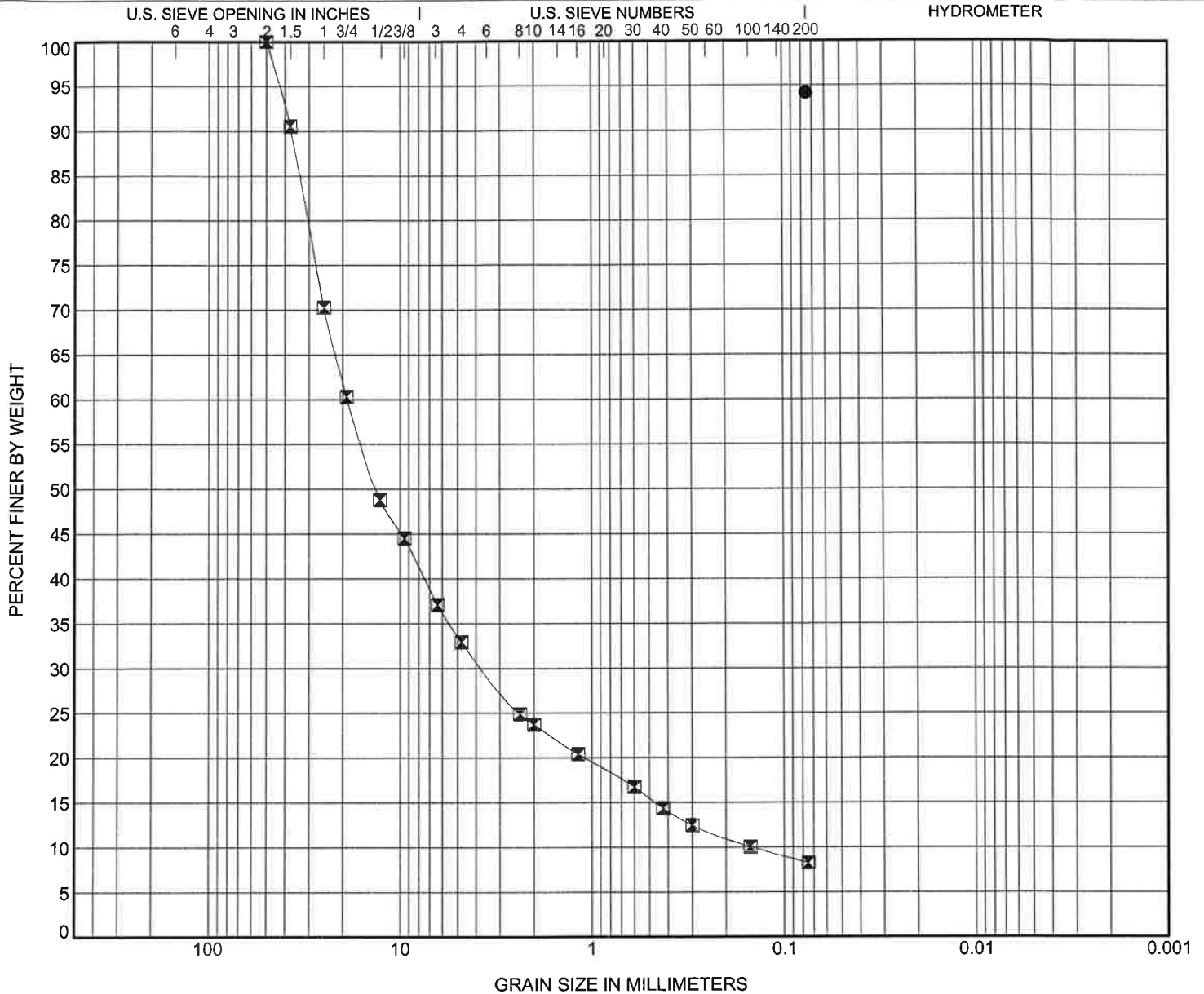
PROJECT NUMBER 16090

PROJECT LOCATION Riverton, Utah

Borehole	Depth (ft)	Moisture (%)	Dry Density (pcf)	Liquid Limit	Plastic Limit	Plasticity Index	Gravel (%)	Sand (%)	Fines (%<#200 Sieve)	Classification
TP-1	3.0	38.4		50	21	29			97	CH
TP-1	8.0	19.0		43	17	26	14	29	57	CL
TP-2	1.0	18.9								
TP-2	5.0	26.5		NP	NP	NP			75	ML
TP-2	11.0	12.0								
TP-3	1.0	16.4								
TP-3	5.0	14.0		NP	NP	NP	0	3	97	ML
TP-3	9.0	6.5		28	14	14	28	23	49	GC
TP-4	3.0	23.4								
TP-4	6.0	37.9		49	22	27			94	CL
TP-5	1.0	20.1								
TP-5	9.0	6.5		NP	NP	NP	67	25	8	GP-GM



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**GRAIN SIZE DISTRIBUTION****WILDING**  
ENGINEERINGCLIENT DR HortonPROJECT NAME McIntyre Farms SubdivisionPROJECT NUMBER 16090PROJECT LOCATION Riverton, Utah

COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

BOREHOLE	DEPTH	Classification	LL	PL	PI	Cc	Cu
● TP-4	6.0	LEAN CLAY(CL)	49	22	27		
☒ TP-5	9.0	POORLY GRADED GRAVEL with SILT and SAND(GP-GM)	NP	NP	NP	4.91	128.25

BOREHOLE	DEPTH	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
● TP-4	6.0	0.075						94.2	
☒ TP-5	9.0	50	18.801	3.68	0.147	67	25	8.3	