GEOTECHNICAL ENGINEERING REPORT
NOLAND PROPERTY SUBDIVISION

PROPERTY LOCATION:
13400 SOUTH 3200 WEST
RIVERTON, UT

Prepared for:
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DECEMBER 6, 2016

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APPENDIX

LIST OF TABLES AND FIGURES

1. INTRODUCTION

This report presents the geotechnical investigation for the proposed Noland Property Subdivision to be located near 13400 South 3200 West in Riverton, Utah as shown on the vicinity map in the Appendix. The geotechnical investigation was performed in accordance with Wilding Engineering's proposal dated August 31, 2016.

The field investigation consisted of five (5) test pits. Test Pits (TP-1 through TP-5) were excavated to depths ranging from 12 feet to 14 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 the residential subdivision consisting of single family residences with the associated utilities, roadways, driveways, and retaining walls. 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 13 acres in area and will consist of 27 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 17 acres in area and is located near 13400 South 3200 West in Riverton, Utah. More specifically, the site is located at latitude 40.510004 degrees and Longitude -111.968273 degrees.

At the time of our field investigation, the project site was a vacant agricultural land. Based on available topographic information, the subject site slopes downward to the east. The subject site is bounded by existing vacant land on the east, Utah Lake Distribution Canal on the west, and existing single family residences on the east and north sides.

4. SURFICIAL GEOLOGY

Based on the available geologic maps, the project site is underlain by lacustrine deposits. These deposits typically consist of silt and clay deposits. The soils encountered in the soil profile consisted of predominantly clayey 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 rubber-tired backhoe to depths ranging from 12 feet to 14 feet below the existing site grades. Stratigraphy and classification of the soils were logged under the direction of a geotechnical engineer.

Disturbed and undisturbed samples were obtained at various depths and examined in the field and representative portions were stored in sealed plastic bags. 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 test pits consisted of about 1½ feet of topsoil underlain by Lean Clay (CL) with varying amounts of Sand, Silty Clay with Sand (CL-ML), Silty Sand (SM) with varying amounts of Gravel, Clayey Silty Gravel with Sand (GC-GM), and Clayey Gravel with Sand (GC) to the maximum depth of exploration of 14 feet below existing site grades. Collapsible soils were encountered in the upper 3 to 8 feet of soil profile in the test pits.

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 Ground Water

Ground water was not encountered any of the test pits to the maximum depth explored to 14 feet below existing site grades. 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 during construction Wilding Engineering must be notified to observe changing conditions and provide recommendations.

6 LABORATORY TESTING

Representative soil samples were tested to evaluate physical and engineering properties. Laboratory testing included: natural water content, unit weight, grain size analysis, Atterberg Limits, and collapse 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 geotechnical recommendations based on the information provided by the client and the soils encountered during our field investigation for the proposed development. The proposed site is suitable for construction if the recommendations of this report are adhered to. The project site is suitable for proposed development. The primary geotechnical considerations with respect to the development include over-excavation of potentially collapsible soils, moisture sensitivity of the on-site fine-grained soils, potential caving of granular soils, foundation subgrade preparation, and surface drainage. Further information is provided in the following sections of this report.

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.

In general 1½ feet of topsoil was encountered during our investigation. All topsoil or any soil containing organic or deleterious materials shall be removed from the site where structures or pavement are to be placed. Topsoil may be stockpiled on site for subsequent use in landscape areas. Any unsuitable material (loose, soft, saturated, or otherwise unstable soils where structures are to be placed), shall be replaced with structural fill according to the standards set forth in section 7.2.5 and 7.2.6 of this report.

Upon completion of site grubbing and prior to placement of any fill, the exposed subgrade should be evaluated by a representative of the Geotechnical Engineer. Proof rolling with loaded 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 truck or other suitable construction vehicle should be over-excavated down to firm undisturbed native soils and backfilled with properly placed and compacted structural fill.

Excavations should be made using an excavator equipped with a smooth edge and supported from outside the excavation. If the subgrade is disturbed during construction, disturbed soils should be over-excavated to firm, undisturbed soil and backfilled with compacted granular materials.

We recommend that site preparation, earthwork, and pavement subgrade preparation be accomplished during warmer, drier months, typically extending from mid-May to mid-October of the year. Any modifications to the grading plans should be reviewed by the Geotechnical Engineer.

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 1 to 2, 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

We understand that the proposed site will require up to twenty feet of mass grading across the site. 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 Collapsible Soils

Collapsible soil can be broadly classified as soil that is susceptible to a large and sudden reduction in volume upon wetting. These soils exhibit a physical characteristic that gives them the potential for collapsing upon the introduction of water. Collapsible soil usually has a low dry density and low moisture content. Such soils can often withstand a large applied vertical stress with a small compression, but then experience much larger settlements after wetting, with no increase in vertical pressure. Based on subsurface explorations, potentially collapsible soils as evidence by the "pinhole" structures observed were encountered in the test pits TP-1 through TP-5 to depths ranging from about 3 to 8 feet below existing site grades.

We recommend that the on-site soils beneath foundations, floor slabs and pavement areas near the above noted test pits be excavated to depth of at least 3 to 4 feet or to the depth recommended by our field engineer during construction. Excavated on-site soils below topsoil should be moisture conditioned to within 2 percent of the optimum moisture content, placed in lifts not exceeding 9 inches thick (loose) and compacted to at least 95 percent of the maximum dry density as determined by ASTM D 1557.

Care should be taken to limit the introduction of water into these soils after during and after the construction of the proposed residences. This may include special measures to seal utilities and roofs drains or make them flexible at critical locations to reduce the potential for the introduction of water into the subgrade materials. Perimeter drains should also be installed to minimized to inflow of surface water percolation into the potentially collapsible soils under the proposed residences. Perimeter drains should be designed and constructed with care and specific feature to limit the drain's potential to collect, pool, or concentrate water along its alignment.

7.2.5 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 7.1 for gradation specification. This material shall be free from organics, garbage, frost, and other loose, compressible, or deleterious materials.

Table 7.2.5 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

Fine-grained materials (clays and silts) are not suitable for use as fill in areas that will be carrying a structural load such as roads, buildings, and utility trenches in roadways. However, they may be used as site grading fills in landscaped areas.

7.2.6 Fill Placement and Compaction

Fill under houses, 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.

7.2.7 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 7.2.5.

> **Pipe foundation:** shall consist of imported granular soils. 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, 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).

7.2.8 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 7.2.5.

7.2.9 Surface Drainage

A grading and drainage plan has been 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 native undisturbed native soils (non-collapsible) or on structural fill which is bearing on native soils and is compacted to 95% of the modified proctor (maximum dry density as determined with ASTM D1557 method of test). All undocumented fill should be over excavated and removed in areas where structures 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 ±2% of the optimum moisture content.

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 width dimensions of 24-

inches for continuous wall footings. Footings placed on slopes shall be "benched" so that all footing bases are horizontal and do not follow the natural slope.

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

Shallow foundation bearing on entirely undisturbed native soils or on properly placed and compacted granular structural fill extending down to undisturbed native soils may be designed with a maximum net allowable bearing capacity of **2,000 pounds per square foot (psf)**, or a subgrade modulus value of **100 pounds per cubic inch (pci)**. The recommended allowable bearing pressure refers to the total dead load and can be increased by 1/3 to included the sum of all loads including wind and seismic.

7.3.3 Settlement

The anticipated total settlement is not expected 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 a 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 2015). According to the IBC 2015, soils with poor drainage characteristics require that a foundation drain be installed 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". **A footing drain is required in these soil types.**

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 (μ) which is estimated to be 0.25 for sands, and the resistance of lateral sliding is 130 psf for clays and silts. [1]

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."[2] 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 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, ϕ, as 32 degrees[3].

Table 7.4.2 Static Conditions

Equivalent Fluid Pressures and Coefficients				
Conditions	**Kγ**	γ	**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] International Building Code 2006, Ch. 18, Table 1804.2
[8] International Building Code 2006, Section 1610, Table 1610.1
[3] Bowles, Joseph E., PE, PhD, Foundation Analysis and Design, fifth edition, 1996

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.

Table 7.4.3 Dynamic Conditions

Yielding Wall Dynamic Pressures and Coefficients			
Conditions:	**Component**	γ	**K**
Active	130 pcf	120	K'_a=1.39
Passive	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 2015 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 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 6½ 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 2015), and the USGS National Earthquake Hazards Reduction Program (NEHRP) interpolated probabilistic ground motion values for S_S an S_1 are 1.13g and 0.46g, respectively. Values from the NEHRP were estimated with 40.510004 degrees and longitude of -111.968273 degrees. (See table below).

USGS
science for a changing world

Table 7.6.1 USGS Earthquake Hazards Estimated Values

	2% PE in 50 year	10% PE in 50 year
Peak Ground Acceleration (g)	0.469	0.214
0.2 sec Spectral Acceleration (g)	1.130	0.509
1.0 sec Spectral Acceleration (g)	0.460	0.173

The design spectral accelerations were determined according to IBC 2015 and ASCE 07-05 and were found to be 0.79g and 0.48g 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.

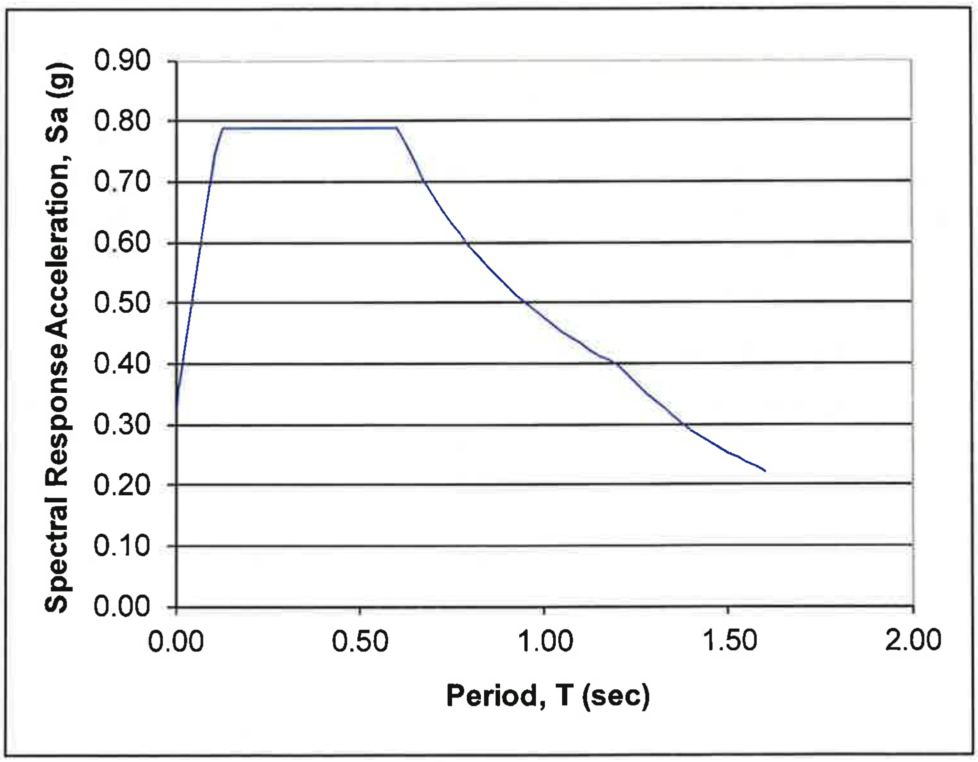
Figure 7.6 ASCE 7-05 Seismic Provisions

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-ll-2002-interp.cgi

S_S:	1.130	F_a=	1.05	S_{MS}=	1.184	S_{DS}=	0.790
S_1:	0.460	F_v=	1.55	S_{M1}=	0.712	S_{D1}=	0.475



7.6.2 Liquefaction

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 14 feet below existing ground surface. Based on our subsurface investigation, the subsurface profiles encountered consisted of predominantly fine grained soils with coarse grained soils at varied depths. Groundwater was not encountered in the test pits to the maximum depth of exploration to 14 feet below existing site grades. Based on the subsurface soils encountered in the upper 14 feet, the soils 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 0.79g for short periods (S_{DS}) and 0.48g 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 drive and parking areas to be located in front and around the proposed building. 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 Ration CBR of 3 (assumed), 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 roadways and parking lot. 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 if three (3) inches of asphalt concrete pavement is required for all roadways and parking surfaces. This asphalt concrete pavement is to comply with UDOT Standard Specifications, Section 02741, and "Hot Mix Asphalt (HMA)."

7.7.4 Concrete Pavement

Concrete pavement is anticipated for the driveway. It is recommended that concrete be used rather than asphalt to aid against excessive future maintenance. We recommend that concrete pavement be designed for a modulus of subgrade reaction, *k*, of 150 pci.

Table 7.7.4 Concrete Design Thicknesses

Pavement Materials	Recommended Minimum Pavement Thickness (inches)
Concrete (4,000 psi)	5
Granular Base Course	6

Sub-grade should meet structural fill requirements and be compacted using typical compaction methods with 95 percent compaction of the maximum dry density within +/- 2% of the optimum moisture determined by ASTM D1557. Prior to placement of concrete the sub-grade should be inspected by the Geotechnical Engineer.

Concrete for exposed conditions should meet IBC 2015 requirements with six (6) to five (5) percent air content; maximum temperature of ninety degrees, maximum allowable slump shall not exceed four (4) inches. Joints shall be in a rectangular pattern and spacing shall not exceed thirty (30) times the thickness of the slab. This will allow for expansion and contraction of the concrete with the change in seasons.

7.7.5 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 2015 recommends that a minimum of five percent gradient for a ten feet distance away from any structures.

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 test pits excavated at the locations indicated on the site plan, laboratory results, data obtained from the U.S.G.S. Library, and previous reports and studies. 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 have correlated soil types and properties such as bearing pressure and equivalent fluid lateral pressure with U.S.G.S. surveys, the International Building Code, and surrounding investigations. Any assumptions made, based on these correlations, are conservative.

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.

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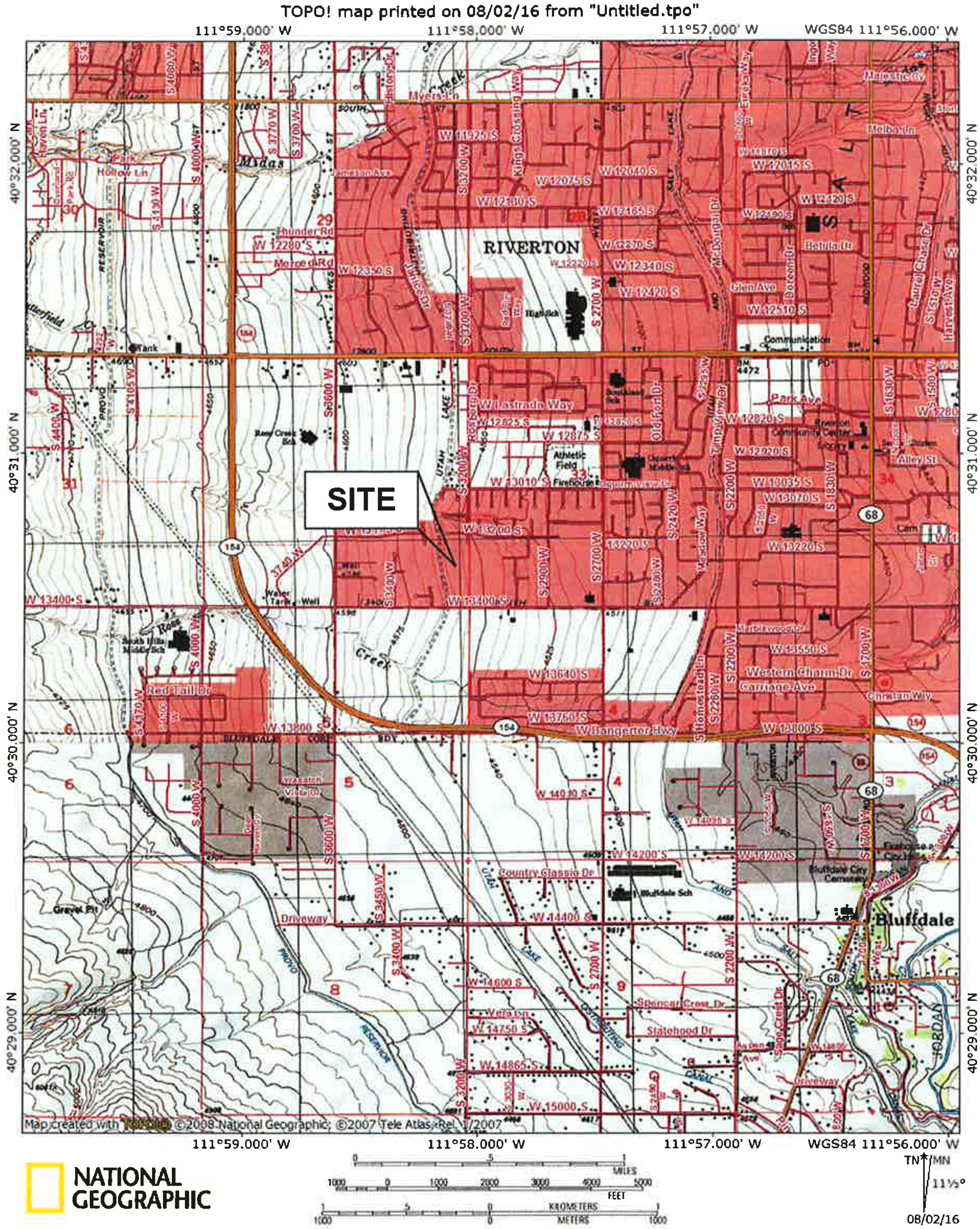
APPENDIX

VICINITY MAP

TOPO! map printed on 08/02/16 from "Untitled.tpo"



Project: **Noland Property Subdivision 13400 South 3200 West Riverton, Utah** Project No: **16215**	Date: December 2016 Drawn By: CPB Figure: A-1	WILDING ENGINEERING WWW.WILDINGENGINEERING.COM

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SITE MAP WITH TEST PIT LOCATIONS

Approximate Test Pit Location

WILDING ENGINEERING, INC

NO.	REVISION	DATE

DRAWING TITLE: SITE MAP WITH TEST PIT LOCATIONS	PROJECT NAME: NOLAND PROPERTY SUBDIVISION		DATE: 12/5/16
			SCALE: NTS
LOCATION: 13400 SOUTH 3200 WEST	DRAWN: CPB	CHECKED: CPB	SHEET: A-2
RIVERTON, UTAH	FILE NAME: G:\DATA\18215...\dwg\Test Pit Locations.dwg		

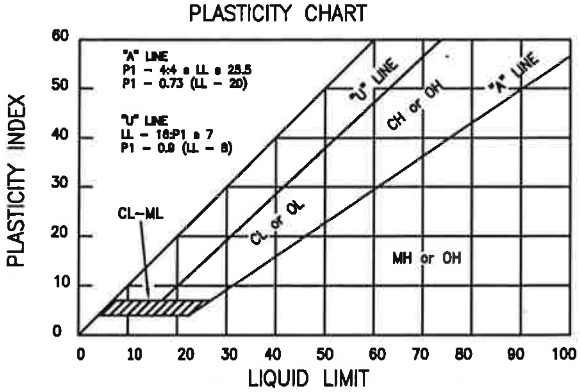
UNIFIED SOIL CLASSIFICATION SYSTEM

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 Pratice 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)	Lines plot below "A" line & hatched zone on plasticity chart		GM	SILTY GRAVELS, GRAVEL-SAND-SILT MIXTURES
			Lines plot above "A" line & hatched zone on plasticity chart		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)	Lines plot below "A" line & hatched zone on plasticity chart		SM	SILTY SANDS, SAND-SILT MIXTURES
			Lines plot above "A" line & hatched zone on plasticity chart		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.

PLASTICITY CHART



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

TEST PIT NUMBER TP-1

Wilding Engineering, Inc

CLIENT Weldon Noland	PROJECT NAME Noland Property Subdivision
PROJECT NUMBER 16215	PROJECT LOCATION Riverton, Utah
DATE STARTED 10/28/16 COMPLETED 10/28/16	GROUND ELEVATION 4569 ft TEST PIT SIZE 60 inches
EXCAVATION CONTRACTOR Weldon Noland	GROUND WATER LEVELS:
EXCAVATION METHOD Test Pit	AT TIME OF EXCAVATION ---
LOGGED BY JGW CHECKED BY CPB	AT END OF EXCAVATION ---
NOTES	AFTER EXCAVATION ---

DEPTH (ft)	SAMPLE TYPE NUMBER	TESTS	U.S.C.S.	GRAPHIC LOG	MATERIAL DESCRIPTION	Elevation
0.0					**TOPSOIL:** Lean Clay wih Sand, dark brown.	
1.5						4567.5
	GB 1	MC = 6%	CL		**LEAN CLAY WITH SAND:** dry, moderate brown. -- light brown, with pinholes from 3 to 6 feet.	
2.5						
	GB 2					
5.0						
6.0						4563.0
7.5					**SILTY CLAY WITH SAND:** light brown, dry. -- with pinholes from 6 to 8 feet.	
	H 3	MC = 6% DD = 84 pcf LL = 22 PL = 18 Fines = 85%	CL-ML			
10.0						4559.0
			CL		**LEAN CLAY WITH SAND:** dry, light brown.	
12.5						
	GB 4					
14.0						4555.0

Bottom of test pit at 14.0 feet.

TEST PIT NUMBER TP-2

Wilding Engineering, Inc

CLIENT Weldon Noland
PROJECT NUMBER 16215
DATE STARTED 10/28/16 COMPLETED 10/28/16
EXCAVATION CONTRACTOR Weldon Noland
EXCAVATION METHOD Test Pit
LOGGED BY JGW CHECKED BY CPB
NOTES

PROJECT NAME Noland Property Subdivision
PROJECT LOCATION Riverton, Utah
GROUND ELEVATION 4568 ft TEST PIT SIZE 60 inches
GROUND WATER LEVELS:
AT TIME OF EXCAVATION ---
AT END OF EXCAVATION ---
AFTER EXCAVATION ---

DEPTH (ft)	SAMPLE TYPE NUMBER	TESTS	U.S.C.S.	GRAPHIC LOG	MATERIAL DESCRIPTION
0.0	GB 1				**TOPSOIL:** Lean Clay wih Sand, dark brown.
1.5			CL		**LEAN CLAY WITH SAND:** dry, moderate brown. (4566.5)
2.5					-- light brown, with pinholes from 3 to 6 feet.
5.0	GB 2	MC = 9% LL = 33 PL = 18 Fines = 84%			
7.5	GB 3				
10.0	GB 4	MC = 14%			
12.5					
13.0			SM		**SILTY SAND WITH GRAVEL:** dry, light brown, with cobbles. (4555.0)
14.0					(4554.0)

Bottom of test pit at 14.0 feet.

GENERAL BH / TP / WELL - GINT STD US LAB.GDT - 12/5/16 17:08 - G:\DATA\16215 NOLAND SUBDIVISION\SOILS\TEST PIT LOGS\16215 NOLAND PROPERTY SUBDIVISION.GPJ

TEST PIT NUMBER TP-3

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Wilding Engineering, Inc

CLIENT Weldon Noland		**PROJECT NAME** Noland Property Subdivision	
PROJECT NUMBER 16215		**PROJECT LOCATION** Riverton, Utah	
DATE STARTED 10/28/16	**COMPLETED** 10/28/16	**GROUND ELEVATION** 4569 ft	**TEST PIT SIZE** 60 inches
EXCAVATION CONTRACTOR Weldon Noland		**GROUND WATER LEVELS:**	
EXCAVATION METHOD Test Pit		**AT TIME OF EXCAVATION** ---	
LOGGED BY JGW	**CHECKED BY** CPB	**AT END OF EXCAVATION** ---	
NOTES		**AFTER EXCAVATION** ---	

DEPTH (ft)	SAMPLE TYPE NUMBER	TESTS	U.S.C.S.	DEPTH / ELEVATION	MATERIAL DESCRIPTION
0.0	GB 1			1.5 / 4567.5	**TOPSOIL:** Lean Clay wih Sand, dark brown.
2.5	GB 2	MC = 9% LL = 28 PL = 19 Fines = 72%	CL	3.0 / 4566.0	**LEAN CLAY WITH SAND**: dry, light brown, with pinholes.
5.0	GB 3	MC = 3%	GC	13.0 / 4556.0	**CLAYEY SILTY GRAVEL WITH SAND**: dry, moderate brown.
7.5	GB 4	MC = 4% LL = 22 PL = 15 Fines = 26%			
10.0					
12.5	GB 5				

Bottom of test pit at 13.0 feet.

SUMMARY OF LABORATORY TEST RESULTS

Wilding Engineering, Inc

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CLIENT Weldon Noland

PROJECT NAME Noland Property Subdivision

PROJECT NUMBER 16215

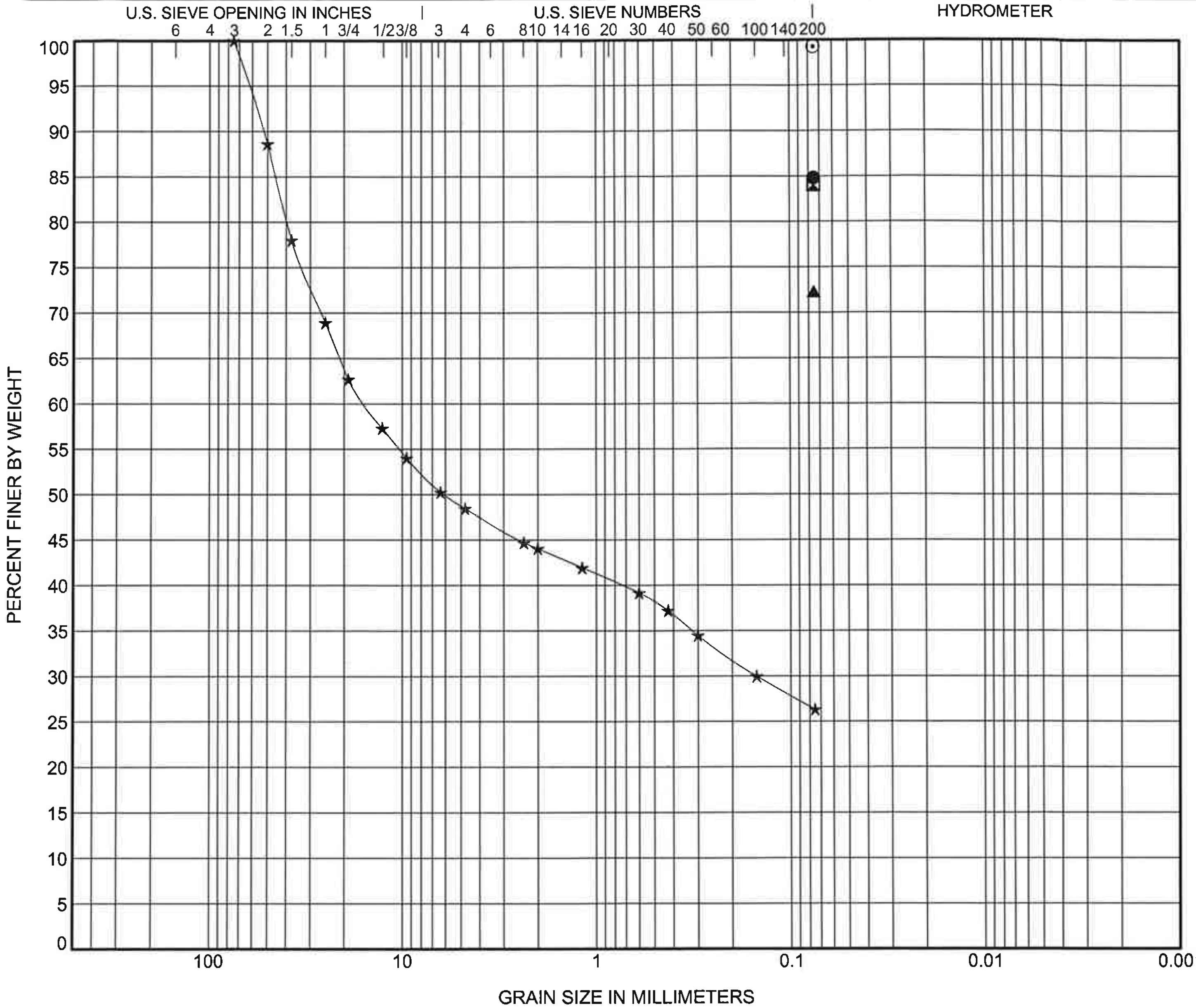
PROJECT LOCATION Riverton, Utah

Borehole	Depth (ft)	Moisture (%)	Dry Density (pcf)	Liquid Limit	Plastic Limit	Plasticity Index	Gravel (%)	Sand (%)	Fines (%<#200 Sieve)	Classification
TP-1	2.0	6.0								
TP-1	8.0	6.4	84.1	22	18	4			85	CL-ML
TP-2	4.0	8.9		33	18	15			84	CL
TP-2	10.0	14.2								
TP-3	2.0	8.6		28	19	9			72	CL
TP-3	5.0	3.1								
TP-3	8.0	4.0		22	15	7	52	22	26	GC-GM
TP-4	0.5	11.5								
TP-4	5.0	9.4		43	19	24			99	CL
TP-4	11.0	14.0								
TP-5	5.0	3.5		26	16	10	66	16	18	GC
TP-5	9.0	12.3		42	19	23			92	CL
TP-5	12.0	6.0								

GRAIN SIZE DISTRIBUTION

Wilding Engineering, Inc

CLIENT Weldon Noland
PROJECT NAME Noland Property Subdivision
PROJECT NUMBER 16215
PROJECT LOCATION Riverton, Utah



	BOREHOLE	DEPTH	Classification	LL	PL	PI	Cc	Cu
●	TP-1	8.0	SILTY CLAY with SAND(CL-ML)	22	18	4		
⊠	TP-2	4.0	LEAN CLAY with SAND(CL)	33	18	15		
▲	TP-3	2.0	LEAN CLAY with SAND(CL)	28	19	9		
★	TP-3	8.0	SILTY, CLAYEY GRAVEL with SAND(GC-GM)	22	15	7		
⊙	TP-4	5.0	LEAN CLAY(CL)	43	19	24		

	BOREHOLE	DEPTH	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	TP-1	8.0	0.075						84.8	
⊠	TP-2	4.0	0.075						84.1	
▲	TP-3	2.0	0.075						72.3	
★	TP-3	8.0	75	15.501	0.151		52	22	26.3	
⊙	TP-4	5.0	0.075						99.2	

GRAIN SIZE - GINT STD US LAB.GDT - 12/5/16 12:13 - G:\DATA\16215 NOLAND SUBDIVISION\SOILS\TEST PIT LOGS\16215 NOLAND PROPERTY SUBDIVISION.GPJ

GRAIN SIZE DISTRIBUTION

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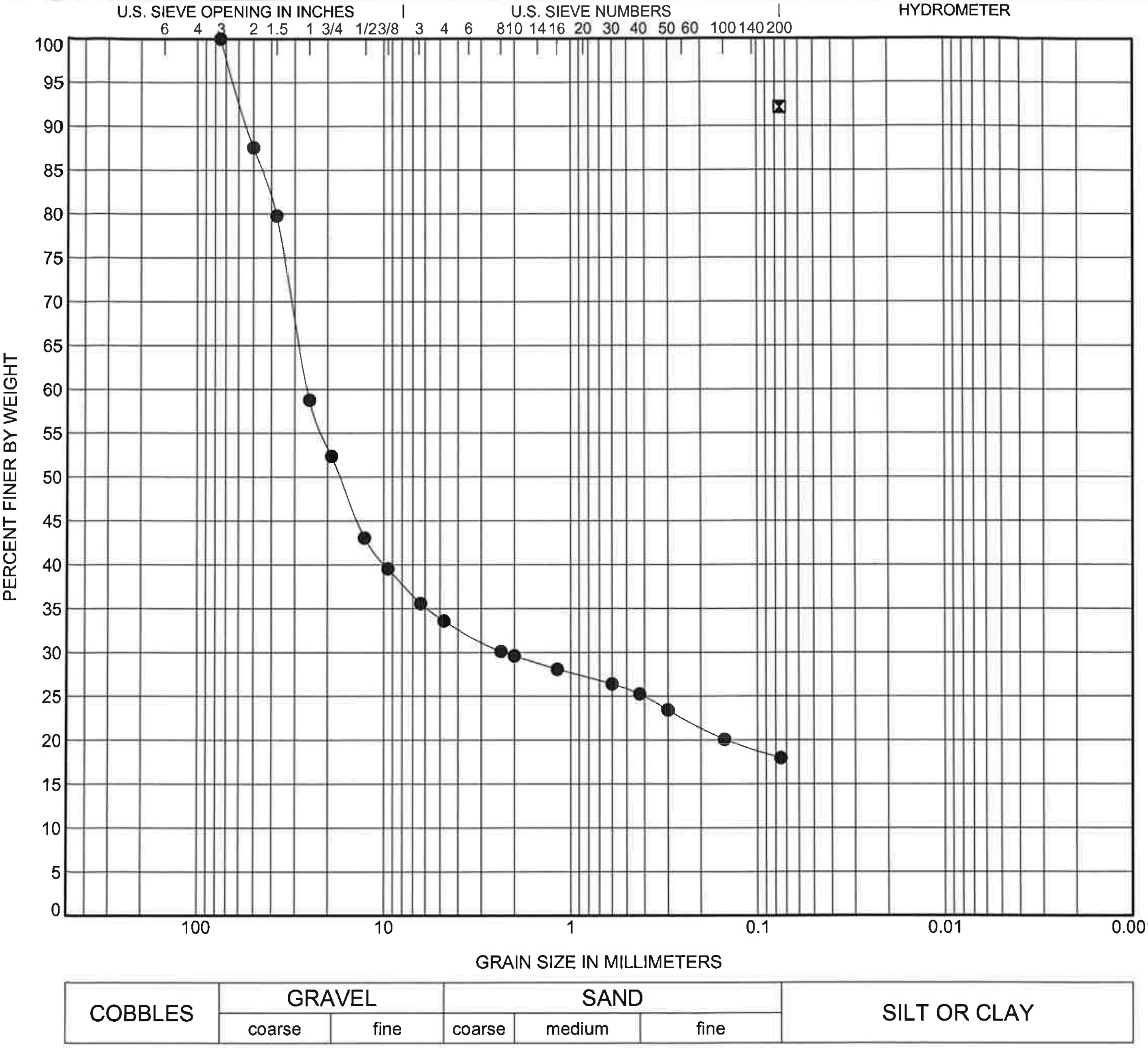
CLIENT Weldon Noland | PROJECT NAME Noland Property Subdivision

PROJECT NUMBER 16215 | PROJECT LOCATION Riverton, Utah



BOREHOLE		DEPTH	Classification	LL	PL	PI	Cc	Cu
●	TP-5	5.0	CLAYEY GRAVEL with SAND(GC)	26	16	10		
⊠	TP-5	9.0	LEAN CLAY(CL)	42	19	23		

BOREHOLE		DEPTH	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	TP-5	5.0	75	25.59	2.257		66	16	18.0	
⊠	TP-5	9.0	0.075						92.1	

GRAIN SIZE - GINT STD US LAB.GDT - 12/5/16 12:13 - G:\DATA\16215 NOLAND SUBDIVISION\SOILS\TEST PIT LOGS\16215 NOLAND PROPERTY SUBDIVISION.GPJ

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DATE:	Tuesday, November 22, 2016
LABORATORY TECH:	Jeremy Wright
PROJECT:	Noland Property Subdivision
WILDING PROJECT #:	16215

Collapse Test

Sample Location:	TP-1	Dry Density:	84.1 pcf
Sample Depth (ft):	8	Moisture Content:	6.4 %
Sample Description:	Silty Clay with Sand	Liquid Limit:	22
USCS Classification:	CL-ML	Plastic Limit:	18
Percent Collapse:	2.32	Fine Content:	85 %

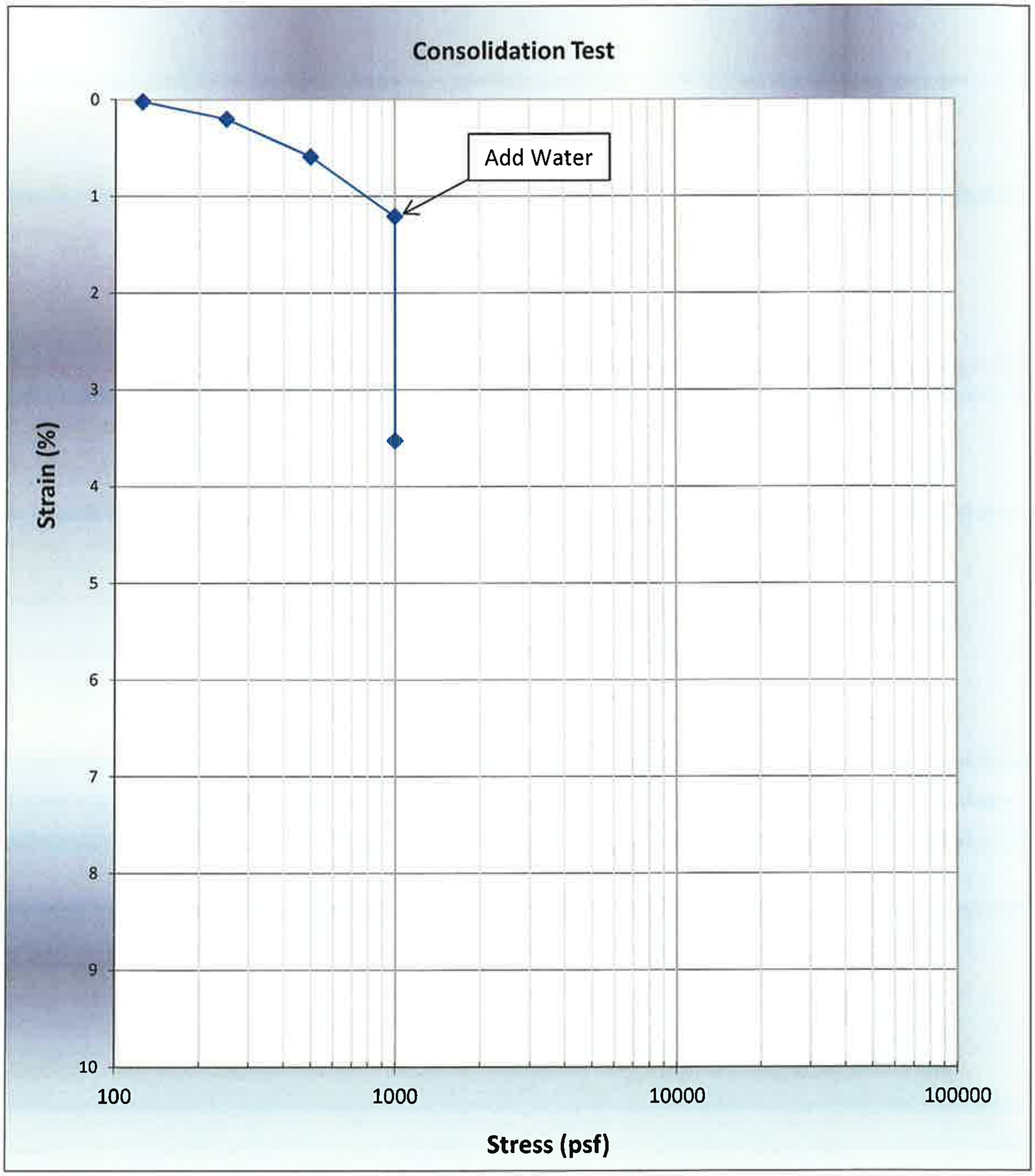

Figure No. C-1

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