GEOTECHNICAL INVESTIGATION
HIDDIN PINES SUBDIVISION

PROPERTY LOCATION
3854 WEST 13800 SOUTH
RIVERTON, UT

Project No.: 18187

Prepared For:
LOVELL DEVELOPMENT
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MARCH 4, 2019

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04/10/2019

APPENDICES

1 INTRODUCTION

This report presents the geotechnical investigation for the proposed residential development at 3854 West 13800 South, Riverton, UT, as shown on the Site Vicinity Map in Appendix A (Figure A-1).

The field investigation consisted of eight (8) test pits. The test pits were excavated to depths of 8 to 12 feet below the existing ground surface. Detailed test pit logs can be found in Appendix B (Figures B-2 to B-9). Recommendations in this report are based upon information gathered from the field investigation, site observation, published geologic maps, laboratory testing, and engineering analysis.

2 PURPOSE AND SCOPE

The purpose of this investigation was to assess the suitability of on-site soils for the development with the associated utilities, landscaping, and roadways and provide geotechnical recommendations. The scope of work completed for this study included site reconnaissance, subsurface exploration, soil sampling, laboratory testing, engineering analyses, and preparation of this report.

3 SITE AND PROJECT INFORMATION

3.1 PROJECT DESCRIPTION

Based on our understanding of the project, the proposed development will consist of 42 new single family homes with the associated utilities, landscaping, and roadways. No specific structural loading information is provided at the time of this report. However, we understand the proposed structures will be one- to two-story with typical wood framed walls and a basement, constructed on traditional continuous or spread footings.

3.2 EXISTING SITE CONDITIONS

At the time of our field investigation, the property is being used for horse boarding services and construction storage for a roofing company. The site can be accessed from the south on 13800 South through an unpaved road. The site can also be accessed at the northwest corner from Deer Mountain Drive. Mature trees are located on the south and southeast portion of the site. Various buildings including stable, barns, garages and modular homes are located on the property. Open areas in the center and east portions of the property are used to store horses. A large horse manure pile is located on the north end of the property. The manure pile is approximately 150 feet wide, 300 feet long with a maximum height of 7 feet. There are various weeds and grasses on the ground surface of the site. The site is bound by a vacant field to the north, a park to the east, 13800 South Street to the south and existing residential homes to the west. The site is relatively flat and slopes generally towards the east.

4 GEOLOGY RESEARCH AND REVIEW

4.1 SURFICIAL GEOLOGY

Based on the available geologic maps[1], the project site is mapped within the Qlf zone, which is described as: *Fine-grained lacustrine deposits - Transgressive and regressive, deep-water sediments; brown, dark-brown, grayish-brown, and gray calcareous, laminated silt, clayey silt, and sandy silt; commonly contains isolated pebbles, cobbles, and thin lenses of sand and gravel that were deposited by ice-rafting (dropstones) and turbidity flows; exposed thicknesses range from 1 to 38.6 feet (0.3 - 11.8 m).*

4.2 LIQUEFACTION

Certain areas within the intermountain region possess a potential for liquefaction during seismic events. Liquefaction is a phenomenon whereby loose, saturated, non-cohesive 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. Liquefaction can result in densification of such deposits, resulting in settlement of overlying layers. 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 must be strong enough to cause liquefaction.

Based on the liquefaction hazard map, the site lies within an area designated as having a “very low” liquefaction probability[2]. A “very low” liquefaction potential indicates that there is probability of 5% or less of having a seismic event exceeding critical acceleration in 100 years[3]. A site-specific liquefaction study is not required per the Special Study Areas Map published by Salt Lake County Planning and Development Services and is beyond our proposed scope of work.

4.3 LANDSLIDES

Slope stability hazards such as landslides, slumps, and other mass movements can develop along moderate to steep slopes where a slope has been disturbed, the head of a slope is loaded, or where increased groundwater pore pressures result in driving forces within the slope

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1 Davis, F., D., 2000, Geologic Map of the Midvale Quadrangle, Salt Lake County, Utah, Plate 1 Utah Geological Survey Map 177
2 Christenson, G.E., Shaw, L.M., 2008, Liquefaction special study areas, Wasatch Front and nearby areas, Utah: Utah Geological Survey, Supplement map to Circular 106, scale 1:250,000
3 Anderson, L.R., Keaton, J.R., Bischoff, J.E., 1994, Liquefaction potential map for Utah County, Utah complete technical report: Utah Geological Survey, Contract Report 94-8, p. 22.

exceeding restraining forces. Slopes exhibiting prior failures, and also deposits from large landslides, are particularly vulnerable to instability and reactivation. The project site is not mapped within landslide special study areas[1].

4.4 DEBRIS FLOW

Debris flow hazards are typically associated with unconsolidated alluvial fan deposits at the mouths of large range-front drainages. The project site is not mapped within debris flow special study areas[2].

5 FIELD EXPLORATIONS

5.1 SUBSURFACE INVESTIGATION

Subsurface soil conditions at the project site were explored at the site by excavating eight (8) test pits at representative locations within the subject property. The test pits were excavated using a rubber-track mini-ex to depths of 8 to 12 feet below the existing site grades. Stratigraphy and classification of the soils were logged under the direction of the Geotechnical Engineer.

Disturbed and undisturbed samples were obtained at various depths. The samples were transported to our laboratory for 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 Appendix B (Figures B-2 to B-9). A Key to Soil Symbols is presented on Figure B-1.

5.2 SUBSURFACE CONDITIONS

5.2.1 Soils

The soils encountered in the test pits consisted of up to 16 inches of topsoil and/or undocumented fill underlain by native clayey soils extending to 3 to 5.5 feet below the existing ground surface. Below the clay layer was mainly gravelly soils which extended to the full depth of the test pits excavated for this investigation. The stratification lines shown on the enclosed Test Pit Logs (Figures B-2 to B-9) represent the approximate boundary between soil types. The actual in-situ transition may be gradual. Due to the nature and depositional characteristics of native soils, care should be taken in interpolating subsurface conditions between and beyond the exploration locations.

5.2.2 Groundwater

Groundwater was not encountered within the test pits excavated for our field investigation at a maximum depth of 12 feet. It should be noted that it is possible for the groundwater levels to

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1 Christenson, G.E., Shaw, L.M., 2008, Landslide special study areas, Wasatch Front and nearby areas, Utah: Utah Geological Survey, Supplement map to Circular 106, scale 1:200,000

2 Christenson, G.E., Shaw, L.M., 2008, Debris-flow/alluvial-fan special study areas, Wasatch Front and nearby areas, Utah: Utah Geological Survey, Supplement map to Circular 106, scale 1:200,000

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. Therefore, groundwater conditions encountered during and/or after construction may differ from those encountered during our field investigation.

5.2.3 Soil Collapse Potential

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 indicated by the "pinhole" structures were encountered in Test Pit 3 and Test Pit 4 to an approximate depth of 5.5 feet. One collapse test was performed on sample obtained at 3 feet below the existing site grade at Test Pit 4. The collapse test shows a collapsing of 1.5% when water was introduced to the sample at 2,000 psf vertical stress.

Since soil condition may vary across the site, Wilding Engineering should visit the site at the time of the foundation excavation to evaluate the soil conditions for individual lots and assess the soil collapse potential. Care should be taken to limit the introduction of water into these soils during and after the construction of the proposed residences according to *Section 7.2.6 Moisture Protection and Surface Drainage*.

6 LABORATORY TESTING

Geotechnical laboratory tests were conducted on selected soil samples obtained during our field investigation. The laboratory testing program was designed to evaluate the engineering characteristics of onsite earth materials. Laboratory tests conducted during this investigation include: Grain Size Distribution Analysis, Atterberg Limits Test, Moisture Content of Soil by Mass, Collapse Test, and Direct Shear Test.

The results of laboratory tests are presented on the test pit logs in Appendix B (Figures B-2 to B-9), the Summary of Laboratory Test Results table (Figure C-1), and on the test result figures presented in Appendix C (Figures C-2 through C-6).

7 RECOMMENDATIONS AND CONCLUSIONS

7.1 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 engineering properties of the earth materials encountered and tested as part of our subsurface exploration and the anticipated design data discussed in *Section 3.1, Project Description*. If subsurface conditions other than those described herein are encountered during construction, and/or if design changes are initiated, Wilding Engineering must be informed so that our recommendations can be reviewed and revised as changes or conditions may require.

7.2 EARTHWORK

7.2.1 Site Preparation and Grading

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

In general, up to 16 inches of topsoil and/undocumented fill was encountered during our investigation. All topsoil, undocumented fill, the horse manure located at the north end of the property or any soil containing organic or deleterious materials shall be removed where structures, pavements, or concrete flatwork are to be placed. Topsoil may be stockpiled on site for subsequent use in landscape areas.

Upon completion of site grubbing and prior to placement of any fill, the exposed subgrade should be evaluated by Wilding Engineering. 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 *Sections 7.2.3 and 7.2.4*.

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

For ease of construction and to increase the likelihood of favorable soil conditions, we recommend that site preparation, earthwork, and pavement subgrade preparation be accomplished during warmer, drier months.

7.2.2 Excavation Stability

All utility excavations shall be carefully supported, maintained, and protected during construction in accordance with OSHA Regulations. It is the responsibility of the contractor to maintain safe working conditions. Temporary construction excavations shall be properly sloped or shored, in compliance with current federal, state, and local requirements. Excavations are to be made to minimize subsequent filling. A trench box or shoring may be used. Wet soils, coarse-grained material and soil with low fine content (material passing the No. 200 sieve) can easily become unstable and in some areas there could be toppling, cave-ins or sliding.

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 excavations depth, exceed those specified in local, state, and federal safety regulations.

7.2.3 Structural Fill Material

All fill placed for support of structures, concrete flatwork, or pavements shall consist of structural fill. The contractor should have confidence that the anticipated method of compaction will be suitable for the type of structural fill used. All structural fill should be free of vegetation, debris or frozen material, and should contain no materials larger than 4 inches nominal size.

Imported structural fill shall consist of a well-graded, granular material with a maximum aggregate size of 4 inches, and 5% to 20% fines content (material passing the No.200 sieve). Fill material portion finer than the No.40 sieve shall have a liquid limit (LL) less than 30 and a plasticity index (PI) less than 10, see Table 7.1 below for material specifications. This material shall be free from organics, debris, frozen material, and other compressible or deleterious materials. Imported structural fill is preferred and it is usually easier for compaction. On-site native sandy and gravelly soils may be also be used as structural fill provided it meets material specifications in Table 7.1 and materials larger than 4 inches are screened. Onsite fine-grained materials (clays and silts) are not generally suitable for use as structural fill due to their inherent resistance to uniform moisture conditioning and workability to achieve desired compaction.

Table 7.1 Structural Fill Material Specifications

Grain Size	Percent Passing
4-inch	100
2-inch	85 to 100
No. 4	15 to 50
No. 200	5 to 20
Plastic Index (PI)	< 10
Liquid Limit (LL)	< 30

The contractor should anticipate testing all soils used as structural fill frequently to assess the maximum dry density, fines content, and moisture content, etc. Specifications from governing authorities such as cities and special service districts having their own precedence should be followed where applicable.

7.2.4 Structural Fill Placement and Compaction

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 12-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 placed for subgrade below load bearing areas including footings, concrete slabs and pavements should be compacted to at least 95% of the maximum dry density as determined by ASTM D1557. Structural fill placed in non-load bearing areas including landscape areas should be compacted to at least 90% of the maximum dry density (ASTM D1557). The moisture content should be at or slightly above the optimum moisture content at the time of placement and compaction. Wilding Engineering should be notified if structural fill thickness exceeds 4 feet so the percentage compaction requirement can be adjusted accordingly. Also, prior to placing any fill, the contractor should request Wilding Engineering to observe the excavations and evaluate if any unsuitable materials or loose soils have been removed. Proper grading should precede placement of fill, as described in *Section 7.2.1, Site Preparation and Grading*.

Specifications from governing authorities such as cities and special service districts having their own precedence should be followed where applicable.

7.2.5 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.

The soils in the utility pipe trenches are to meet the specified structural fill requirements in *Sections 7.2.3 and 7.2.4*.

> **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 shall be filled with compacted well-graded, granular soil (*Sections 7.2.3 and 7.2.4*).
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> **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. Placement and compaction of bedding materials shall be performed 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.

Specifications from governing authorities such as cities and special service districts having their own precedence should be followed where applicable.

7.2.6 Moisture Protection and Surface Drainage

Precautions should be taken during and after construction to eliminate saturation of foundation soils. Over wetting the soils prior to or during construction may result in increased softening and pumping, causing equipment mobility problems and difficulties in achieving compaction.

Moisture should not be allowed to infiltrate the soils in the vicinity of, or upslope from, the structures. It should be noted that there will be an increased risk of settlement if foundation soils become over-wetted. After the footings were constructed, the following recommendations for foundation moisture protection and drainage should be considered:

- Backfill around foundation walls should consist of fine-grained soils with low-permeability. Free-draining sandy and gravelly soils should not be used. The backfill should be placed in 12-inch lifts and compacted to at least 90% of the maximum dry density of the modified Proctor (ASTM D1557).
- The ground surface within 10 feet of the foundation walls should be sloped to drain away from structure with a minimum slope of 5% (2% if hardscaped).
- Roof runoff devices and downspouts should be installed around the entire perimeter of the structure to collect and discharge all roof runoff a minimum of 10 feet from the foundation walls. The runoff should always be allowed to flow away as designed and not back flow against the foundation; pop-ups, direct drainage or other options may be considered. Rain gutters, downspouts, discharge pipes and pop-ups (if used) should be inspected and cleared frequently so they remain unclogged.
- Only hand watering or drip irrigation should be used within 5 feet of the foundation walls but xeriscaping or desert landscaping is preferred. Irrigation and/or water lines near the foundation walls should be maintained in good working order.

7.3 FOUNDATION RECOMMENDATIONS

The foundations for the proposed structures may consist of conventional strip and/or spread footings. Strip and spread footing 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. Interior shallow footings not susceptible to frost conditions should be embedded at least 12 inches for confinement.

7.3.1 *Installation and Bearing Material*

Footings may be placed on undisturbed native soils below any potentially collapsible soils or on structural fill which is bearing on undisturbed native soils below any potentially collapsible soils. Footings should not be placed partially on native soils and partially on structural fill unless approval from Wilding Engineering is obtained. Structural fill should meet material recommendations and be placed and compacted as recommended in *Sections 7.2.3 and 7.2.4*. As mentioned in *Section 5.2.3 Soil Collapse Potential*, potentially collapsible soils were encountered in Test Pit 3 and Test Pit 4 at relatively shallow depth. Based on the findings from the test pits, we anticipate most foundations of homes with basement will extend below potentially collapse soils. Since soil condition may vary across the site, the actual soil conditions should be evaluate by Wilding Engineering during excavation for individual homes.

If encountered, all topsoil, undocumented fill, soft areas, frozen material or other inappropriate material shall be removed from the footing zone to a depth recommended by Wilding Engineering. Footings placed on slopes shall be benched so that all footing bases are horizontal.

Footing excavations shall be observed by us prior to placement of structural fill, concrete, or reinforcement steel to assess their suitability for placement of footings.

7.3.2 *Bearing Pressure*

Conventional strip and spread footings constructed as described above may be proportioned for a maximum net allowable bearing pressure of **1,800 pounds per square foot (psf)**. The recommend net allowable bearing pressure refers to the total dead load and can be increased by 20% to include the sum of all loads including wind and seismic.

7.3.3 *Settlement*

Assuming no additional surcharge is applied, 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 or ½ inch over 30 feet.

7.3.4 *Frost Protection*

All exterior footings are to be constructed at least 30 inches below the ground surface for frost protection and confinement. This includes walk-out areas and may require fill to be placed around buildings. Interior footings not susceptible to frost conditions should be embedded at least 12 inches for confinement. 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*

Wilding Engineering shall periodically monitor excavations prior to installation of footings. Observation of soil before placement of structural fill or concrete is required to evaluate any field conditions not encountered in the investigation which would alter the recommendations or this report. **All structural fill material shall be tested under the direction of the Geotechnical Engineer for material and compaction requirements.**

7.3.6 Foundation Drainage

Soils encountered in the subsurface explorations at elevations of proposed foundations consisted of both Group I soils and Group II soils according to 2018 International Residential Code (IRC) Section R405. We anticipate the majority of the foundation soils for homes with basement will consist of Group I soils. A drainage system is not required where the foundation is installed on Group I soils per IRC 2018. However, a drainage system is required where the foundation is installed on Group II soils per IRC 2018 if the foundations retain earth and enclose habitable or usable spaces located below grade. Due to the soil type variation at the subject site, we should be on site for the foundation excavation for each individual lot to evaluate if a drainage system is required. If required, the drainage system should designed according to IRC 2018 Section R405, which can be accessed at https://codes.iccsafe.org/public/document/IRC2018/chapter-4-foundations.

7.4 LATERAL FORCES

7.4.1 Resistance for Footings

Lateral forces imposed upon conventional foundations due to wind or seismic forces may be resisted by the development of passive earth pressures and frictional resistance between the base of the footing and the supporting subgrade. In determining the frictional resistance, a coefficient of friction of 0.32 may be used for native fine-grained soils (clay and silt) against concrete, and a coefficient of friction of 0.43 may be used for native granular soils (sand and gravel) against concrete.

7.4.2 Lateral Earth Pressures on Retaining/Foundation Walls

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

Table 7.2 Lateral Earth Pressures – Fine-grained Soils

Condition	Lateral Pressure Coefficient	Equivalent Fluid Density (pounds per cubic foot)
Active	0.38	38
At-rest	0.55	55
Passive	2.66	266

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

Table 7.3 Lateral Earth Pressures – Granular Soils

Condition	Lateral Pressure Coefficient	Equivalent Fluid Density (pounds per cubic foot)
Active	0.27	30
At-rest	0.43	47
Passive	3.69	406

It should be noted that the above static and seismic coefficients and densities assume horizontal backfill and vertical wall face with no buildup of hydrostatic pressures. Hydrostatic and surcharge loadings, if any, should be added to the presented values. Over-compaction behind walls should be avoided. If sloping backfill is present, we should 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. Additionally, if passive resistance is calculated in conjunction with frictional resistance, the passive resistance should be reduced by ½. Resisting passive earth pressure from soils subject to frost or heave, or otherwise above prescribed minimum depths of embedment, should be neglected in design.

A section of granular soils should be used as backfill material behind retaining walls because of their high permeability. Using granular fills along with drainage systems including weep holes in the retaining walls or perforated pipes at the bottom of the granular fill directly behind the heel of the retaining walls help minimize the accumulation of hydrostatic pressures.

7.5 CONCRETE SLABS-ON-GRADE & MODULUS OF SUBGRADE REACTION

Concrete slabs-on-grade for interior floor slabs should be constructed on 4" of free draining gravel, overlying undisturbed native soils or a zone of structural fill that is at least 12 inches thick where potentially collapsible soils with pinholes are encountered. The 4 inches of free draining gravel is recommended to provide a capillary break below the finish floor slab and underlying soils. The gravel should consist of a ¾ inch minus clean drain rock. The gravel should be compacted until tight and relatively unyielding.

Concrete slabs–on-grade for exterior flatwork should be constructed on firm undisturbed native soils or zone of structural fill that is at least 12 inches thick where potentially collapsible soils with pinholes are encountered.

For all slab-on-grade construction the structural fill shall be consistent with *Sections 7.2.3 and 7.2.4* with the additional recommendation of minimum fines content of 10% to reduce its permeability. The concrete slabs constructed on subgrade prepared in accordance with the preceding recommendations may be designed using a **modulus of subgrade reaction (k) of 120 psi/in** and should be designed with appropriately spaced, deep control joints to control the location of cracking as a result of shrinkage. Consideration should be given to reinforcing the slabs with welded wire, rebar, or fiber mesh.

7.6 SEISMIC INFORMATION

Based on the USGS Quaternary Fault and Fold Database of the United States, the project site is located approximately 7½ miles west of the Salt Lake City section of the Wasatch fault zone, 11 miles south of the West Valley fault zone, 11 miles north-northwest of the Utah Lake faults, and 12 miles east of the Oquirrh fault zone.

Seismic values were obtained for the subject property utilizing the SEAOC & OSHPD Seismic Design Maps[1] as recommended on USGS website per the 2015 International Building Code (IBC) and ASCE 7-10 code. The ground motions values produced by the web tool are presented in Table 7.4 below based on the site coordinates of 40.5027°N, 111.9827°W. Based on our geotechnical investigation, the on-site soils in the upper 12 feet meet the criteria of Stiff Soils (Site Class D) per ASCE 7-10 Table 20.3-1[2]. More Detailed information is presented in Appendix D-4.

Table 7.4 Seismic Ground Motion Parameters

Parameter	Acceleration (g)	
Mapped Acceleration - Site Class B	S_S = 1.218	S_1 = 0.404
MCE_R Spectral Response Acceleration - Site Class D	S_{MS} = 1.233	S_{M1} = 0.645
Design Spectral Response Acceleration - Site Class D	S_{DS} = 0.822	S_{D1} = 0.43
Peak Ground Acceleration, PGA - Site Class D	0.499	

7.7 PAVEMENT DESIGN AND CONSTRUCTION

Based on our field observation of on-site soils, we assumed a California Bearing Ratio (CBR) of 4 for design of pavements for the proposed development. We have prepared various pavement section options be used to support anticipated traffic loads for the subdivision

1 SEAOC & OSHPD Seismic Design Maps, https://seismicmaps.org/, accessed March 4, 2019.
2 The soils at deeper depths may have properties that meet criteria of other site classification. According to ASCE 7-10 Section 20.1, the site class shall be based on site-specific data to a depth of 100 feet. A geotechnical investigation to 100 feet is beyond our scope of work. Where the soil properties are not known in sufficient detail to determine the site class, Site Class D shall be used unless the authority having jurisdiction or geotechnical data determine Site Class E or F soils are present at the site (ASCE 7-10 Section 20.1).

interior roadways with equivalent single axle loads (ESALs) not exceeding 50,000 per year[1] and a twenty (20) year design life. The table below presents recommended pavement section thickness based on the above assumptions and the material descriptions provided in the following sections. These pavement section options are equivalent to each other and may be selected based on economic considerations.

Table 7.5 Pavement Design Recommended Thickness

Pavement Section Options	Asphalt Concrete (in.)	Untreated Base Course (in.)	Granular Borrow (in.)
Option 1	**3**	**6**	**6**
Option 2	**3.5**	**9**	**-**
Option 3	**4**	**8**	**-**

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

Table 7.6 - Rigid Pavement Section

Concrete (in.)	Untreated Base Course
5	**8**

Concrete should consist of a low slump, low water cement ratio mix with a minimum 28-day compressive strength of 4,000 psi.

7.7.1 Sub-grade Preparation

All topsoil, undocumented fill or other unsuitable materials must be removed below pavements. The sub-grade shall then be proof rolled with a loaded dump truck or other compaction equipment. Any unsuitable soils shall be removed and replaced with structural fill according to *Sections 7.2.3 and 7.2.4*.

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

7.7.2 *Material Recommendations*

All subgrade preparation and pavement section materials (asphalt concrete, untreated base course and granular borrow) should conform to the recommendations presented in this document and all applicable specifications from governing authorities such as cities and counties. Additionally, untreated base course should possess a minimum CBR value of 70, and the granular borrow should have a minimum CBR value of 30. The untreated base course and granular borrow should be placed and compacted in accordance with Sections 7.2.3 and 7.2.4 of this report. The asphalt should be compacted to a minimum of 96% of the Marshall (50 blow) maximum density.

7.7.3 *Drainage and Maintenance*

Drainage shall be designed to direct 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. IBC 2015 recommends that a minimum of five percent gradient for a ten feet distance away from any structures.

7.8 PRELIMINARY SOIL CORROSIVITY

Two soil samples were tested for soil chemical reactivity by American West Analytical Laboratories. Chemical reactivity tests were performed to evaluate soil pH, resistivity, and concentrations of sulfate. Results from these tests are summarized below in Table 7.7. More detailed results are presented in Appendix C (Figures C-7 and C-8).

Table 7.7 Summary of Results from Chemical Reactivity Tests

Location	Depth (ft)	Sulfate (ppm)	Resistivity (Ω-cm)	Soil pH @ 25° C
TP-1	6	63.9	2,470	6.88
TP-8	3.5	106	1,290	7.07

Based on soluble sulfate concentrations results and the American Concrete Institute (ACI) Building Code, there is a "*negligible*" degree of sulfate attack on concrete. Therefore, there is no special requirements on the concrete type selection for sulfate resistance.

Laboratory soil resistivity has a direct impact on the degree of corrosion in underground metals. A decrease in resistivity indicates an increase in corrosion activity. Based on the resistivity test results, the onsite native soils are considered to be "Highly *Corrosive*"[1]. A qualified corrosion

1 Roberge, P.R., 2000, Handbook of corrosion engineering: McGraw-Hill, p. 150.

engineer should be consulted to provide a corrosion assessment and recommendations for any underground metals including water lines, reinforcing steel, valves, etc.

8 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 or below the maximum depths of exploration. The nature and extent of variations may not be evident until construction occurs or after. If any conditions are encountered at this site that are different from those described in this report, we should be immediately notified so that we may make any necessary revisions to recommendations contained in this report. In addition, if the scope of the proposed construction changes from that described in this report, Wilding Engineering should be notified.

This report was prepared in accordance with the generally accepted standard of practice in the project area at the time the report was written. No other warranty, expressed or implied, is made. The concept of risk is a significant consideration of geotechnical analyses. The analytical means and methods used in performing geotechnical analyses and development of resulting recommendations do not constitute an exact science. Analytical tools used by geotechnical engineers are based on limited data, empirical correlations, engineering judgment and experience. As such the solutions and resulting recommendations presented in this report cannot be considered risk-free and constitute our best professional opinions and recommendations based on the available data and other design information available at the time they were developed.

This report was prepared for our client's exclusive use on the project. 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.

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.

PROFESSIONAL ENGINEER
10394605–2202
SHUN LI
3-4-19
ELECTRONIC SEAL
STATE OF UTAH

Shun Li, P.E.
Geotechnical Department Manager

Jeremy G. Wright, P.E.I.
Staff Engineer

APPENDIX A

Image Reference:
Google Earth Imagery Dated 6/15/2017

600 0 600 1200 1800 2400 ft

1:10,000

WILDING ENGINEERING

Legend

Approximate Site Boundary

Lovell Development
3854 West 13800 South
Riverton, UT
Project Number: 18187

Site Vicinity Map

Figure A-1

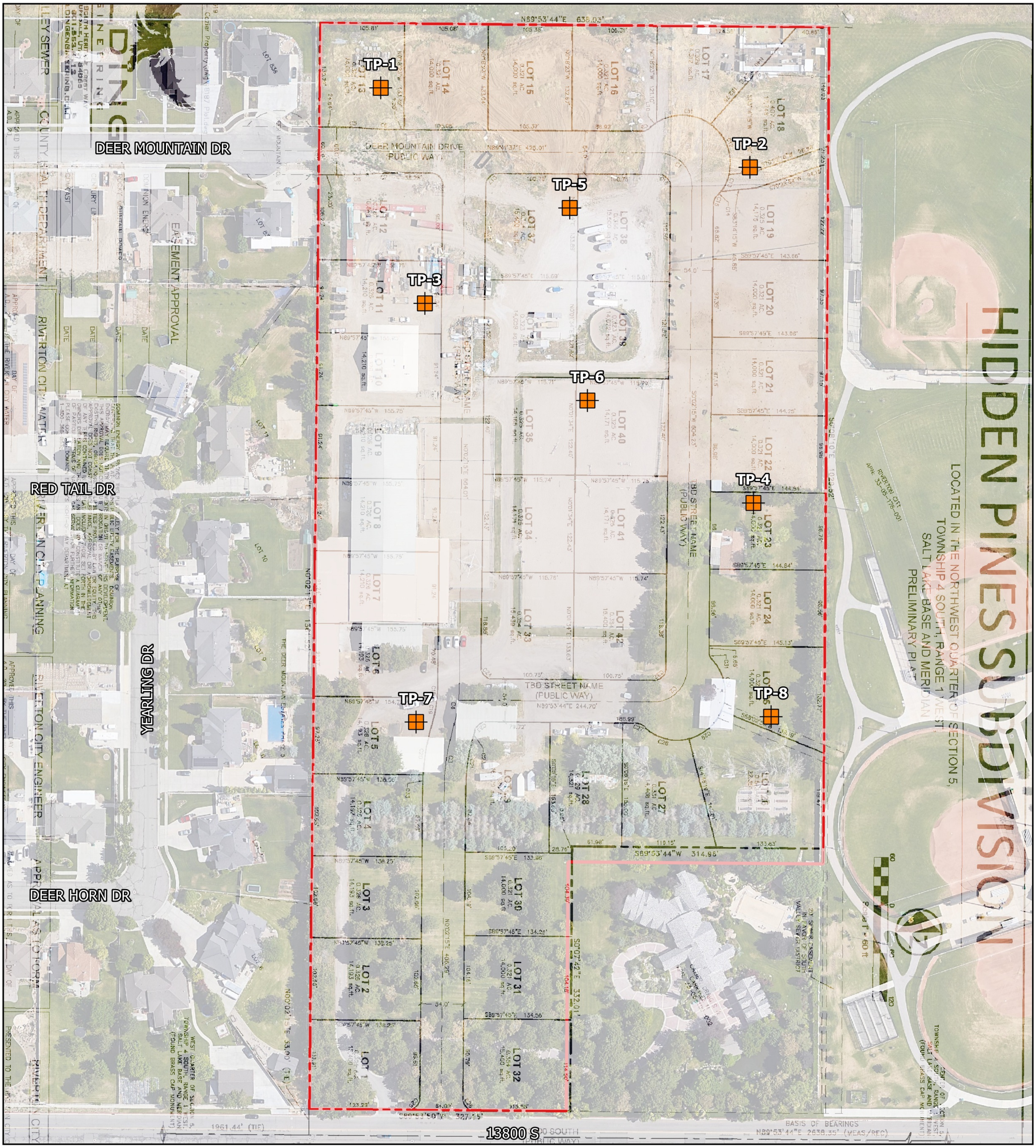

Image Reference:
Google Earth Imagery Dated 6/15/2017

100 0 100 200 300 400 ft

1:1,900

WILDING ENGINEERING

Legend

- Approximate Site Boundary
- Approximate Test Pit Location

Lovell Development
3854 West 13800 South
Riverton, UT
Project Number: 18187

Exploration Location Map

Figure A-2

APPENDIX B

KEY TO SYMBOLS

Wilding Engineering

CLIENT Lovell Development

PROJECT NAME Hidden Pines Subdivision

PROJECT NUMBER 18187

PROJECT LOCATION Riverton, Utah

LITHOLOGIC SYMBOLS (Unified Soil Classification System)

- CL: USCS Low Plasticity Clay
- FILL: Fill (made ground)
- GM: USCS Silty Gravel
- GP-GM: USCS Poorly-graded Gravel with Silt
- GW-GM: USCS Well-graded Gravel with Silt
- SM: USCS Silty Sand
- SP-SM: USCS Poorly-graded Sand with Silt
- TOPSOIL: Topsoil

SAMPLER SYMBOLS

- Hand Sample
- 3" O.D. Thin Walled Shelby Tube

WELL CONSTRUCTION SYMBOLS

ABBREVIATIONS

LL - LIQUID LIMIT (%)
PI - PLASTIC INDEX (%)
W - MOISTURE CONTENT (%)
DD - DRY DENSITY (PCF)
NP - NON PLASTIC
-200 - PERCENT PASSING NO. 200 SIEVE
PP - POCKET PENETROMETER (TSF)

TV - TORVANE
PID - PHOTOIONIZATION DETECTOR
UC - UNCONFINED COMPRESSION
ppm - PARTS PER MILLION

Water Level at Time Drilling, or as Shown
Water Level at End of Drilling, or as Shown
Water Level After 24 Hours, or as Shown

KEY TO SYMBOLS - GINT STD US LAB.GDT - 2/27/19 16:40 - G:\DATA\18187 LOVELL - CAZIER PROPERTY\SOILS\GINT\TEST PIT LOG.GPJ

TEST PIT NUMBER TP-1

Wilding Engineering

CLIENT Lovell Development	**PROJECT NAME** Hidden Pines Subdivision
PROJECT NUMBER 18187	**PROJECT LOCATION** Riverton, Utah
DATE STARTED 2/18/19 **COMPLETED** 2/18/19	**GROUND ELEVATION** **TEST PIT SIZE** N/A inches
EXCAVATION CONTRACTOR G & R Lancaster	**GROUND WATER LEVELS:**
EXCAVATION METHOD Test Pit	**AT TIME OF EXCAVATION** ---
LOGGED BY JGW **CHECKED BY** SL	**AT END OF EXCAVATION** ---
NOTES	**AFTER EXCAVATION** ---

DEPTH (ft)	SAMPLE TYPE NUMBER	TESTS AND REMARKS	U.S.C.S.	GRAPHIC LOG	MATERIAL DESCRIPTION
0.0			CL	1.0	**TOPSOIL - Lean CLAY:** soft, moist, dark brown
2.5	H 1		CL	3.0	**NATIVE - Lean CLAY:** medium stiff, moist, brown
5.0	H 2 H 3		GP-GM	8.0	**Poorly Graded GRAVEL with Silt and Sand:** very dense, moist, light brown, with few rounded cobbles, subrounded and rounded gravel --trace boulders, difficult to excavate
7.5 10.0	H 4 H 5	MC = 7% LL = NP PL = NP Fines = 17%	GM	10.5	**Silty GRAVEL with Sand:** very dense, moist, light brown, rounded gravel, rounded and subroudned cobbles
		Groundwater was not encountered during excavation			Bottom of test pit at 10.5 feet.

GENERAL BH / TP / WELL - GINT STD US LAB.GDT - 2/27/19 16:47 - G:\DATA\18187 LOVELL - CAZIER PROPERTY\SOILS\GINT\TEST PIT LOG.GPJ

TEST PIT NUMBER TP-2

CLIENT Lovell Development
PROJECT NAME Hidden Pines Subdivision
PROJECT NUMBER 18187
PROJECT LOCATION Riverton, Utah
DATE STARTED 2/18/19 **COMPLETED** 2/18/19
GROUND ELEVATION **TEST PIT SIZE** N/A inches
EXCAVATION CONTRACTOR G & R Lancaster
GROUND WATER LEVELS:
EXCAVATION METHOD Test Pit
AT TIME OF EXCAVATION ---
LOGGED BY JGW **CHECKED BY** SL
AT END OF EXCAVATION ---
NOTES
AFTER EXCAVATION ---

DEPTH (ft)	SAMPLE TYPE NUMBER	REMARKS	U.S.C.S.	GRAPHIC LOG	MATERIAL DESCRIPTION
0.0			CL	1.0	**TOPSOIL - Lean CLAY:** soft, moist, dark brown
2.5	H 1		CL	3.0	**NATIVE - Lean CLAY:** medium stiff, moist, brown
	H 2		SM	4.0	**Silty SAND with Gravel:** medium dense, moist, light brown, trace rounded cobbles
5.0			SP-SM	5.0	**Poorly Graded SAND with Silt and Gravel:** dense, moist, light brown, rounded and subrounded gravel
7.5	H 3		GP-GM		**Poorly Graded GRAVEL with Silt and Sand:** very dense, moist, light brown, with few rounded cobbles, subrounded and rounded gravel
					-- decreasing cobbles
10.0	H 4	Groundwater was not encountered during excavation		10.0	Bottom of test pit at 10.0 feet.

Figure No.: B-3

TEST PIT NUMBER TP-3

Wilding Engineering

CLIENT Lovell Development

PROJECT NAME Hidden Pines Subdivision

PROJECT NUMBER 18187

PROJECT LOCATION Riverton, Utah

DATE STARTED 2/18/19 **COMPLETED** 2/18/19

GROUND ELEVATION **TEST PIT SIZE** N/A inches

EXCAVATION CONTRACTOR G & R Lancaster

GROUND WATER LEVELS:

EXCAVATION METHOD Test Pit

AT TIME OF EXCAVATION ---

LOGGED BY JGW **CHECKED BY** SL

AT END OF EXCAVATION ---

NOTES

AFTER EXCAVATION ---

DEPTH (ft)	SAMPLE TYPE NUMBER	REMARKS	U.S.C.S.	GRAPHIC LOG	MATERIAL DESCRIPTION
0.0			CL		**UNDOCUMENTED FILL - Lean CLAY:** medium stiff, moist, dark brown
1.0			CL		**NATIVE - Lean CLAY:** medium stiff, moist, brown
	H 1				-- dry, with some pinholes to 5.5 feet
2.5					
	S 2				
5.0					
5.5			GP-GM		**Poorly Graded GRAVEL with Silt and Sand:** very dense, moist, light brown, with few rounded cobbles, subrounded and rounded gravel. Layer of cobble at soil transition zone
					--decreasing cobbles
7.5					
	H 3				
10.0	H 4				
12.0		Groundwater was not encountered during excavation			Bottom of test pit at 12.0 feet.

GENERAL BH / TP / WELL - GINT STD US LAB.GDT - 2/27/19 16:47 - G:\DATA\18187 LOVELL - CAZIER PROPERTY\SOILS\GINT\TEST PIT LOG.GPJ

TEST PIT NUMBER TP-4

WILDING ENGINEERING

Wilding Engineering

CLIENT Lovell Development

PROJECT NAME Hidden Pines Subdivision

PROJECT NUMBER 18187

PROJECT LOCATION Riverton, Utah

DATE STARTED 2/18/19 **COMPLETED** 2/18/19

GROUND ELEVATION **TEST PIT SIZE** N/A inches

EXCAVATION CONTRACTOR G & R Lancaster

GROUND WATER LEVELS:

EXCAVATION METHOD Test Pit

AT TIME OF EXCAVATION ---

LOGGED BY JGW **CHECKED BY** SL

AT END OF EXCAVATION ---

NOTES

AFTER EXCAVATION ---

DEPTH (ft)	SAMPLE TYPE NUMBER	TESTS AND REMARKS	U.S.C.S.	GRAPHIC LOG	MATERIAL DESCRIPTION
0.0			CL		**TOPSOIL - Lean CLAY:** soft to medium stiff, moist, dark brown
1.0	H 1		CL		**NATIVE - Lean CLAY with Sand:** medium stiff to stiff, moist, brown, white calcuim mottling
2.5					-- dry, with some pinholes to 5.5 feet
	S 2	MC = 17% DD = 86 pcf LL = 33 PL = 19 Fines = 83%			
5.0					
5.5	H 3	MC = 8% LL = NP PL = NP Fines = 14%	GP-GM		**Silty GRAVEL with Sand:** very dense, moist, light brown, with few rounded cobbles, trace subrounded cobbles, subrounded and rounded gravel
7.5	H 4				--trace boulders, difficult to excavate
8.0		Groundwater was not encountered during excavation			Refusal at 8.0 feet. Bottom of test pit at 8.0 feet.

GENERAL BH / TP / WELL - GINT STD US LAB.GDT - 2/27/19 16:47 - G:\DATA\18187 LOVELL - CAZIER PROPERTY\SOILS\GINT\TEST PIT LOG.GPJ

TEST PIT NUMBER TP-5

Wilding Engineering

CLIENT Lovell Development		**PROJECT NAME** Hidden Pines Subdivision	
PROJECT NUMBER 18187		**PROJECT LOCATION** Riverton, Utah	
DATE STARTED 2/18/19	**COMPLETED** 2/18/19	**GROUND ELEVATION**	**TEST PIT SIZE** N/A inches
EXCAVATION CONTRACTOR G & R Lancaster		**GROUND WATER LEVELS:**	
EXCAVATION METHOD Test Pit		**AT TIME OF EXCAVATION** ---	
LOGGED BY JGW	**CHECKED BY** SL	**AT END OF EXCAVATION** ---	
NOTES		**AFTER EXCAVATION** ---	

DEPTH (ft)	SAMPLE TYPE NUMBER	TESTS AND REMARKS	U.S.C.S.	GRAPHIC LOG	MATERIAL DESCRIPTION
0.0			CL	0.4	**UNDOCUMENTED FILL - Poorly Graded Sand with Silt and Gravel:** medium dense, moist, light brown
			CL	1.3	**TOPSOIL - Lean CLAY:** medium stiff, moist, dark brown
2.5	S 1	MC = 34% DD = 75 pcf LL = 39 PL = 20 Fines = 95%	CL	5.0	**NATIVE - Lean CLAY:** soft to medium stiff, very moist, brown
5.0 7.5 10.0	H 2 H 3	Groundwater was not encountered during excavation	GP-GM	10.5	**Poorly Graded GRAVEL with Silt and Sand:** very dense, moist, light brown, with some rounded cobbles, trace subrounded boulders, subrounded and rounded gravel

Bottom of test pit at 10.5 feet.

GENERAL BH / TP / WELL - GINT STD US LAB.GDT - 2/27/19 16:47 - G:\DATA\18187 LOVELL - CAZIER PROPERTY\SOILS\GINT\TEST PIT LOG.GPJ

TEST PIT NUMBER TP-6

Wilding Engineering

CLIENT Lovell Development	**PROJECT NAME** Hidden Pines Subdivision
PROJECT NUMBER 18187	**PROJECT LOCATION** Riverton, Utah
DATE STARTED 2/18/19 **COMPLETED** 2/18/19	**GROUND ELEVATION** **TEST PIT SIZE** N/A inches
EXCAVATION CONTRACTOR G & R Lancaster	**GROUND WATER LEVELS:**
EXCAVATION METHOD Test Pit	**AT TIME OF EXCAVATION** ---
LOGGED BY JGW **CHECKED BY** SL	**AT END OF EXCAVATION** ---
NOTES	**AFTER EXCAVATION** ---

DEPTH (ft)	SAMPLE TYPE NUMBER	TESTS AND REMARKS	U.S.C.S.	GRAPHIC LOG	MATERIAL DESCRIPTION
0.0			CL		**UNDOCUMENTED FILL - Lean CLAY with Sand:** medium stiff, very moist, brown, imported soil mixed with native soils to prepare horse arena
1.0			CL		**NATIVE - Lean CLAY:** medium stiff, moist, brown
2.5					
	H 1				
4.0			GP-GM		**Poorly Graded GRAVEL with Silt and Sand:** very dense, moist, light brown, with few subrounded cobbles, trace subrounded boulders, subrounded and rounded gravel
5.0	H 2				
7.5					
8.0	H 3	MC = 6% LL = NP PL = NP Fines = 6%	GW-GM		**Well-Graded GRAVEL with Silt and Sand:** very dense, moist, light brown, with few subrounded cobbles, trace subrounded boulders, subrounded and rounded gravel
9.5			CL		**Lean CLAY:** very stiff, moist, light brown, blocky clay structure
10.0	H 4				
11.5	H 5	Groundwater was not encountered during excavation			Bottom of test pit at 11.5 feet.

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TEST PIT NUMBER TP-7

CLIENT Lovell Development
PROJECT NAME Hidden Pines Subdivision
PROJECT NUMBER 18187
PROJECT LOCATION Riverton, Utah
DATE STARTED 2/18/19 **COMPLETED** 2/18/19
GROUND ELEVATION **TEST PIT SIZE** N/A inches
EXCAVATION CONTRACTOR G & R Lancaster
GROUND WATER LEVELS:
EXCAVATION METHOD Test Pit
AT TIME OF EXCAVATION ---
LOGGED BY JGW **CHECKED BY** SL
AT END OF EXCAVATION ---
NOTES
AFTER EXCAVATION ---

DEPTH (ft)	SAMPLE TYPE NUMBER	REMARKS	U.S.C.S.	GRAPHIC LOG	MATERIAL DESCRIPTION
0.0			CL	0.5	**UNDOCUMENTED FILL - Silty SAND with Gravel:** very dense, moist, frozen
	H 1		CL	3.0	**NATIVE - Lean CLAY:** stiff, moist, brown, frozed to 1.5 feet
2.5					
	H 2				**Poorly Graded GRAVEL with Silt and Sand:** very dense, moist, light brown, with few subrounded cobbles, subrounded and rounded gravel
5.0			GP-GM		
7.5	H 3				
	H 4				
10.0		Groundwater was not encountered during excavation		10.0	Bottom of test pit at 10.0 feet.

TEST PIT NUMBER TP-8

Wilding Engineering

CLIENT Lovell Development		**PROJECT NAME** Hidden Pines Subdivision	
PROJECT NUMBER 18187		**PROJECT LOCATION** Riverton, Utah	
DATE STARTED 2/18/19	**COMPLETED** 2/18/19	**GROUND ELEVATION**	**TEST PIT SIZE** N/A inches
EXCAVATION CONTRACTOR G & R Lancaster		**GROUND WATER LEVELS:**	
EXCAVATION METHOD Test Pit		**AT TIME OF EXCAVATION** ---	
LOGGED BY JGW	**CHECKED BY** SL	**AT END OF EXCAVATION** ---	
NOTES		**AFTER EXCAVATION** ---	

DEPTH (ft)	SAMPLE TYPE NUMBER	TESTS AND REMARKS	U.S.C.S.	GRAPHIC LOG	MATERIAL DESCRIPTION
0.0			CL	0.5	**TOPSOIL - Lean CLAY:** soft to medium stiff, very moist, dark brown
	H 1		CL		**NATIVE - Lean CLAY:** medium stiff, moist, brown
2.5	H 2			4.0	
			GW-GM		**Well-Graded GRAVEL with Silt and Sand:** very dense, moist, light brown, with few subrounded cobbles, subrounded and rounded gravel
5.0	H 3	MC = 11% LL = NP PL = NP Fines = 9%			-- decreasing moisture
7.5				9.0	
	H 4		GP-GM		**Poorly Graded GRAVEL with Silt and Sand:** very dense, moist, light brown, with few subrounded cobbles, subrounded and rounded gravel
10.0	H 5	Groundwater was not encountered during excavation		10.5	Refusal at 10.5 feet. Bottom of test pit at 10.5 feet.

GENERAL BH / TP / WELL - GINT STD US LAB.GDT - 2/27/19 16:47 - G:\DATA\18187 LOVELL - CAZIER PROPERTY\SOILS\GINT\TEST PIT LOG.GPJ

APPENDIX C

SUMMARY OF LABORATORY TEST RESULTS

Wilding Engineering

CLIENT Lovell Development

PROJECT NAME Hidden Pines Subdivision

PROJECT NUMBER 18187

PROJECT LOCATION Riverton, Utah

Test Pit	Depth (ft)	Moisture (%)	Dry Density (pcf)	Liquid Limit	Plastic Limit	Plasticity Index	Gravel (%)	Sand (%)	Fines (%<#200 Sieve)	Classification
TP-1	8.0	7.4		NP	NP	NP	55	28	17	GM
TP-4	3.0	16.7	85.6	33	19	14			83	CL
TP-4	5.5	7.9		NP	NP	NP	63	23	14	GM
TP-5	3.0	34.1	75.5	39	20	19			95	CL
TP-6	8.0	6.1		NP	NP	NP	65	29	6	GW-GM
TP-8	5.0	10.6		NP	NP	NP	64	27	9	GW-GM

LAB SUMMARY WILDING - GINT STD US LAB.GDT - 2/27/19 16:25 - G:\DATA\18187 LOVELL - CAZIER PROPERTY\SOILS\GINT\TEST PIT LOG.GPJ

ATTERBERG LIMITS' RESULTS

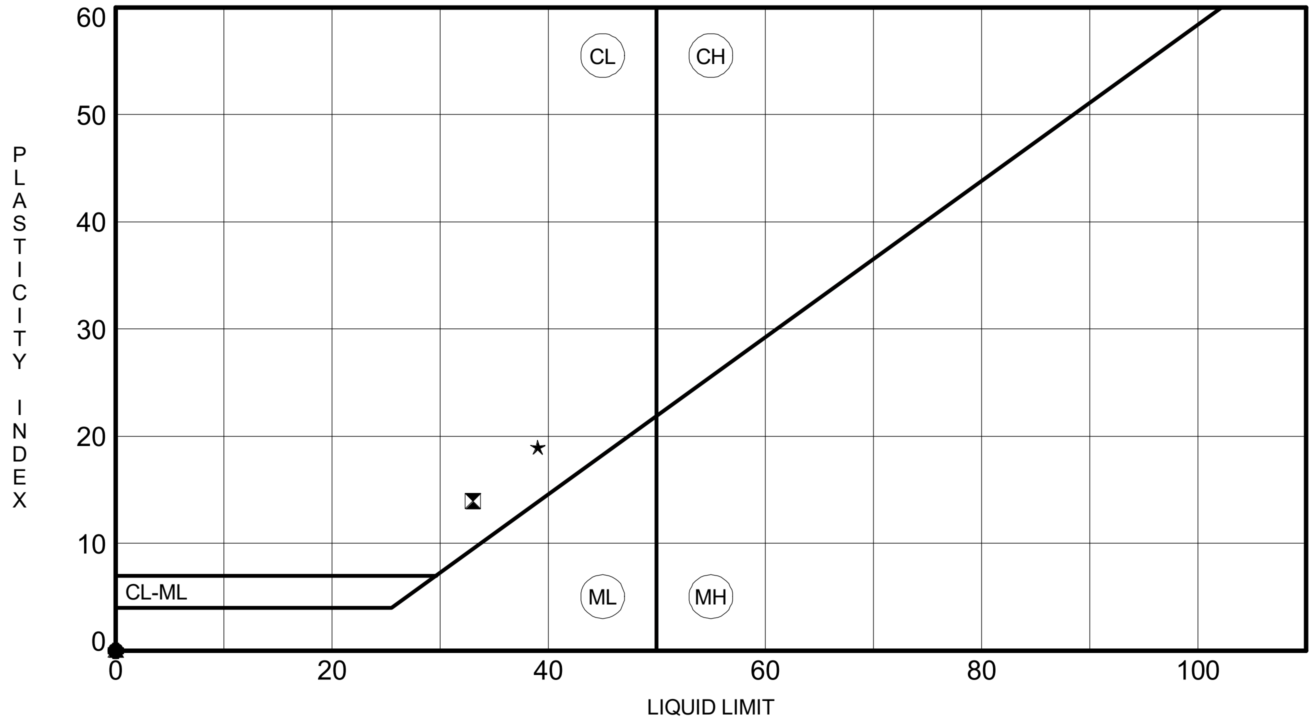
Wilding Engineering

CLIENT Lovell Development

PROJECT NAME Hidden Pines Subdivision

PROJECT NUMBER 18187

PROJECT LOCATION Riverton, Utah



	TEST PIT	DEPTH	LL	PL	PI	Fines	Classification
●	**TP-1**	**8.0**	**NP**	**NP**	**NP**	**17**	**SILTY GRAVEL with SAND(GM)**
⧗	**TP-4**	**3.0**	**33**	**19**	**14**	**83**	**LEAN CLAY with SAND(CL)**
▲	**TP-4**	**5.5**	**NP**	**NP**	**NP**	**14**	**SILTY GRAVEL with SAND(GM)**
★	**TP-5**	**3.0**	**39**	**20**	**19**	**95**	**LEAN CLAY(CL)**
⊙	**TP-6**	**8.0**	**NP**	**NP**	**NP**	**6**	**WELL-GRADED GRAVEL with SILT and SAND(GW-GM)**
✪	**TP-8**	**5.0**	**NP**	**NP**	**NP**	**9**	**WELL-GRADED GRAVEL with SILT and SAND(GW-GM)**

ATTERBERG LIMITS - GINT STD US LAB.GDT - 2/27/19 16:25 - G:\DATA\18187 LOVELL - CAZIER PROPERTY\SOILS\GINT\TEST PIT LOG.GPJ

GRAIN SIZE DISTRIBUTION

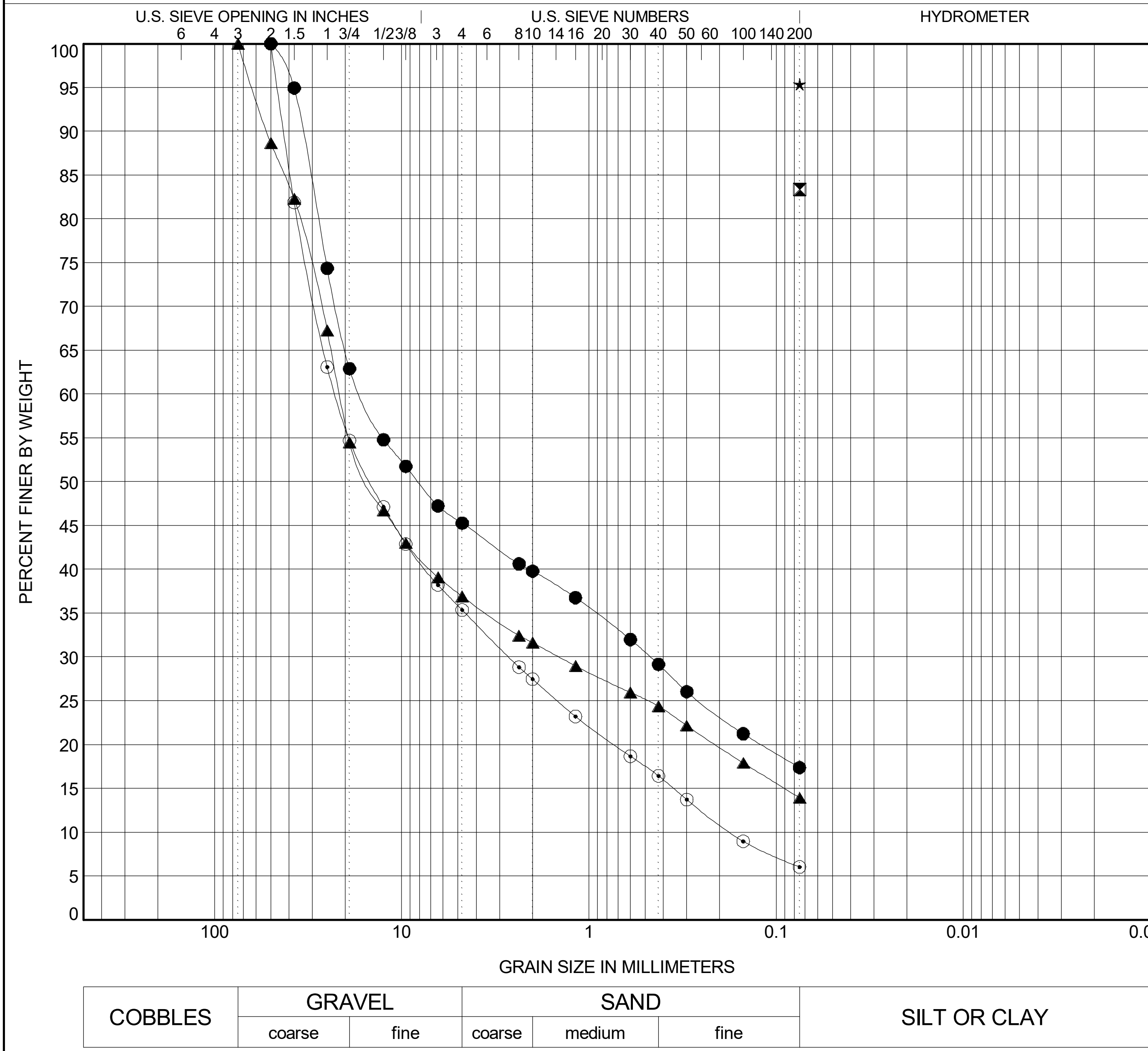
Wilding Engineering

CLIENT Lovell Development

PROJECT NAME Hidden Pines Subdivision

PROJECT NUMBER 18187

PROJECT LOCATION Riverton, Utah



	TEST PIT	DEPTH	Classification	LL	PL	PI	Cc	Cu
●	**TP-1**	**8.0**	**SILTY GRAVEL with SAND(GM)**	**NP**	**NP**	**NP**		
⧗	**TP-4**	**3.0**	**LEAN CLAY with SAND(CL)**	**33**	**19**	**14**		
▲	**TP-4**	**5.5**	**SILTY GRAVEL with SAND(GM)**	**NP**	**NP**	**NP**		
★	**TP-5**	**3.0**	**LEAN CLAY(CL)**	**39**	**20**	**19**		
⊙	**TP-6**	**8.0**	**WELL-GRADED GRAVEL with SILT and SAND(GW-GM)**	**NP**	**NP**	**NP**	**1.81**	**129.16**

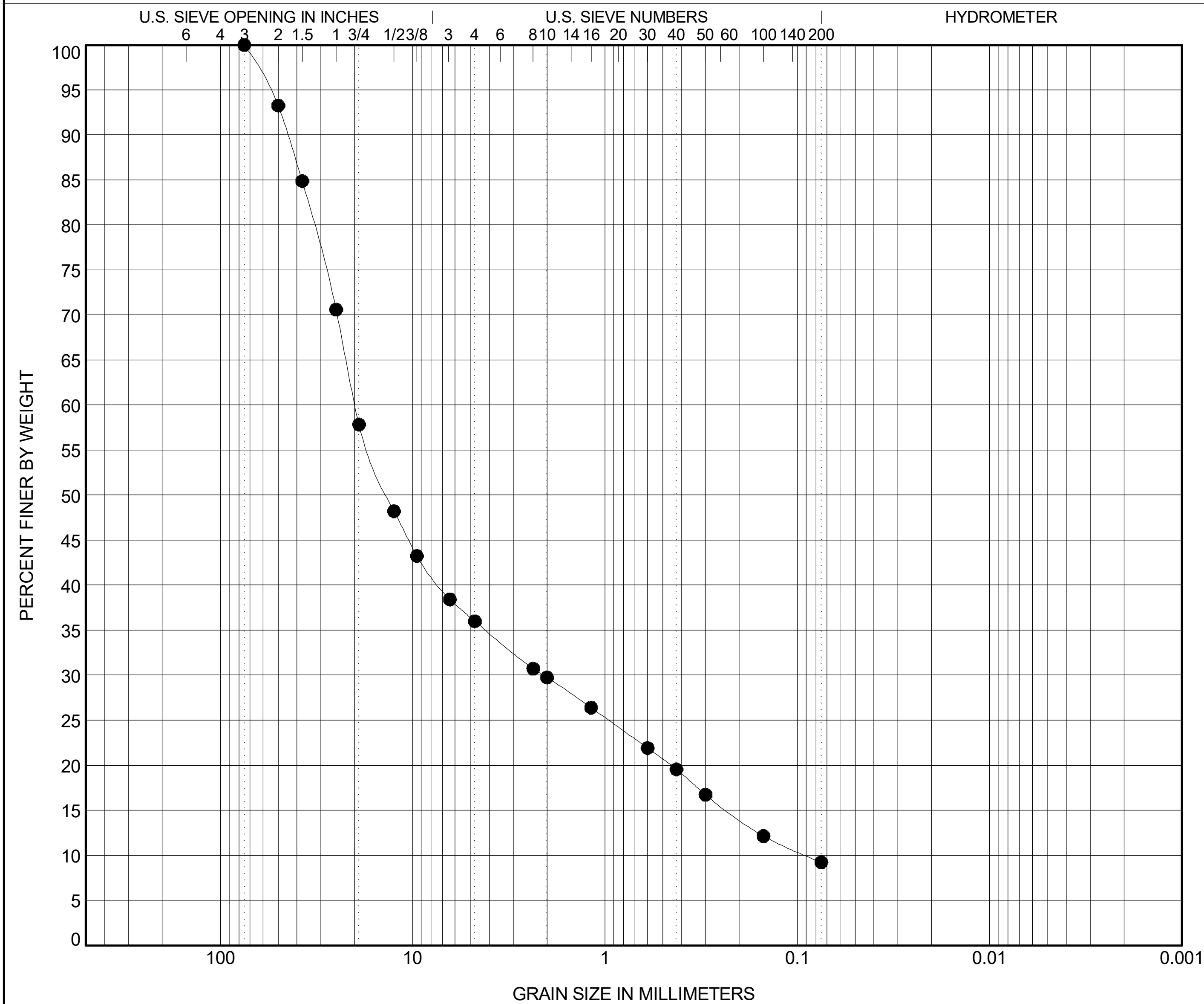
	TEST PIT	DEPTH	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**TP-1**	**8.0**	**50**	**16.356**	**0.471**		**55**	**28**	**17.4**	
⧗	**TP-4**	**3.0**	**0.075**				**0**	**0**	**83.3**	
▲	**TP-4**	**5.5**	**75**	**21.386**	**1.449**		**63**	**23**	**13.9**	
★	**TP-5**	**3.0**	**0.075**				**0**	**0**	**95.4**	
⊙	**TP-6**	**8.0**	**50**	**22.599**	**2.676**	**0.175**	**65**	**29**	**6.0**	

GRAIN SIZE DISTRIBUTION

Wilding Engineering

CLIENT Lovell Development **PROJECT NAME** Hidden Pines Subdivision

PROJECT NUMBER 18187 **PROJECT LOCATION** Riverton, Utah



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	TEST PIT	DEPTH	Classification	LL	PL	PI	Cc	Cu
●	**TP-8**	**5.0**	**WELL-GRADED GRAVEL with SILT and SAND(GW-GM)**	**NP**	**NP**	**NP**	**2.42**	**220.77**

	TEST PIT	DEPTH	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**TP-8**	**5.0**	**75**	**19.903**	**2.086**	**0.09**	**64**	**27**	**9.2**	

GRAIN SIZE - GINT STD US LAB.GDT - 2/27/19 16:26 - G:\DATA\18187 LOVELL - CAZIER PROPERTY\SOILS\GINT\TEST PIT LOG.GPJ

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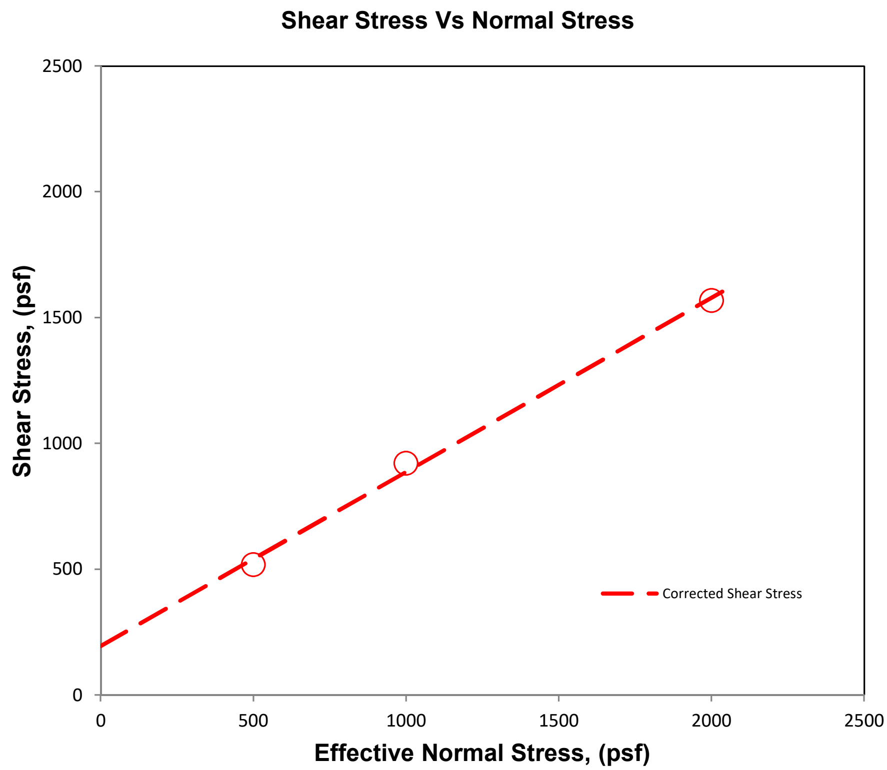
ASTM D3080-14



Mohr-Coulomb Stress Envelope			Parameter	Specimen A	Specimen B	Specimen C	Specimen D
Φ' and c' from Best-Fit Straight Line	Φ' (deg)	35	Initial Water Content (%)	6.1	6.1	6.1	N/A
	c' (psf)	190	Final Water Content (%)	23.2	22.1	22.8	N/A
	R^2	0.9968	Dry Density (psf)	103.7	104.3	104.3	N/A
	SSE	N/A	Diameter (in)	2.42			
Peak Φ' for c' = 0	Normal Stress	Ø' (deg)	Height (in)	1.00			
	500	46.0	Strain Rate (in/min)	0.0010			
	1000	42.6	LL/PL/PI	N/A			
	2000	38.1	Average T_{50} (min)	0.02			

Project:	Hidden Pines Subdivision	**Data Points**		
Project Number:	18187	**Normal Stress**	**Corrected Shear Stress**	
Boring Number:	TP-6			
Sample Number:	N/A	500	518	
Depth:	8 feet	1000	921	
Sample Type:	Reconstituted	2000	1568	
Rel. Compaction:	100%			
Description:	Well-Graded Gravel with Silt and Sand (GW-GM)			
Remarks:	Sample remolded to 110 pcf at approximate in situ moisture.			

Tested By	Jason Wright
Date	2/27/19
Computed By	Jeremy Wright PEI
Date	2/28/19
Checked By	Shun Li PE
Date	2/28/19

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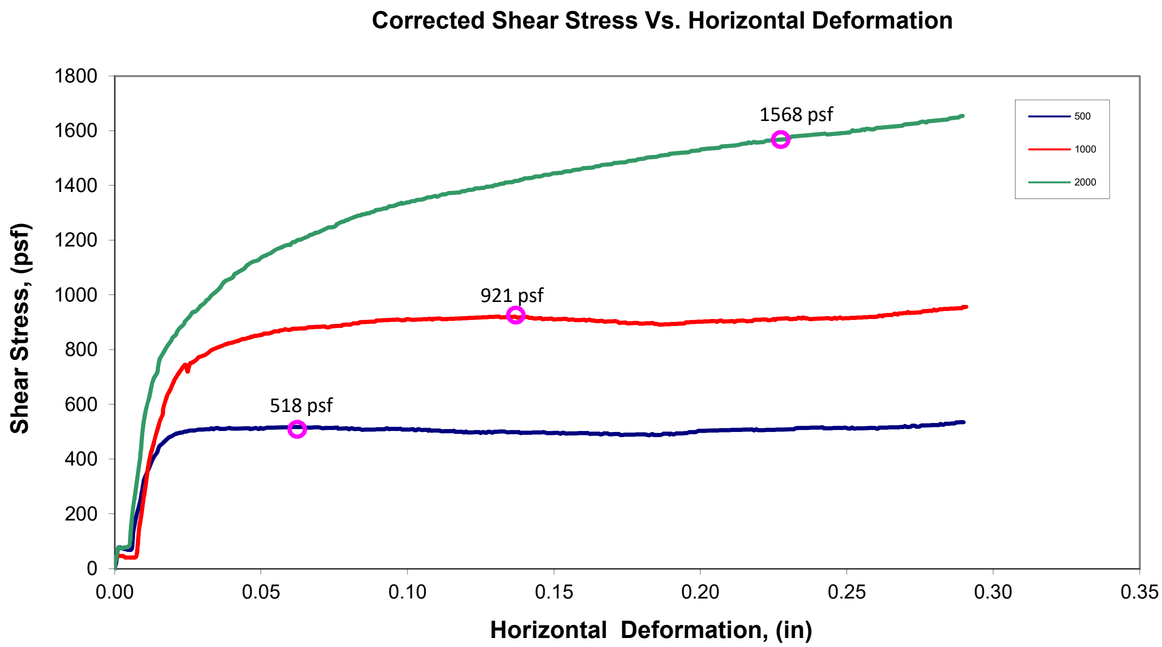
Figure No.: C-5a

Wilding Engineering, Inc.

ASTM D3080-14

Tested By	Jason Wright	Date	2/27/19	Computed By	Jeremy Wright PEI	Date	2/28/19	Checked By	Shun Li PE	Date	2/28/19



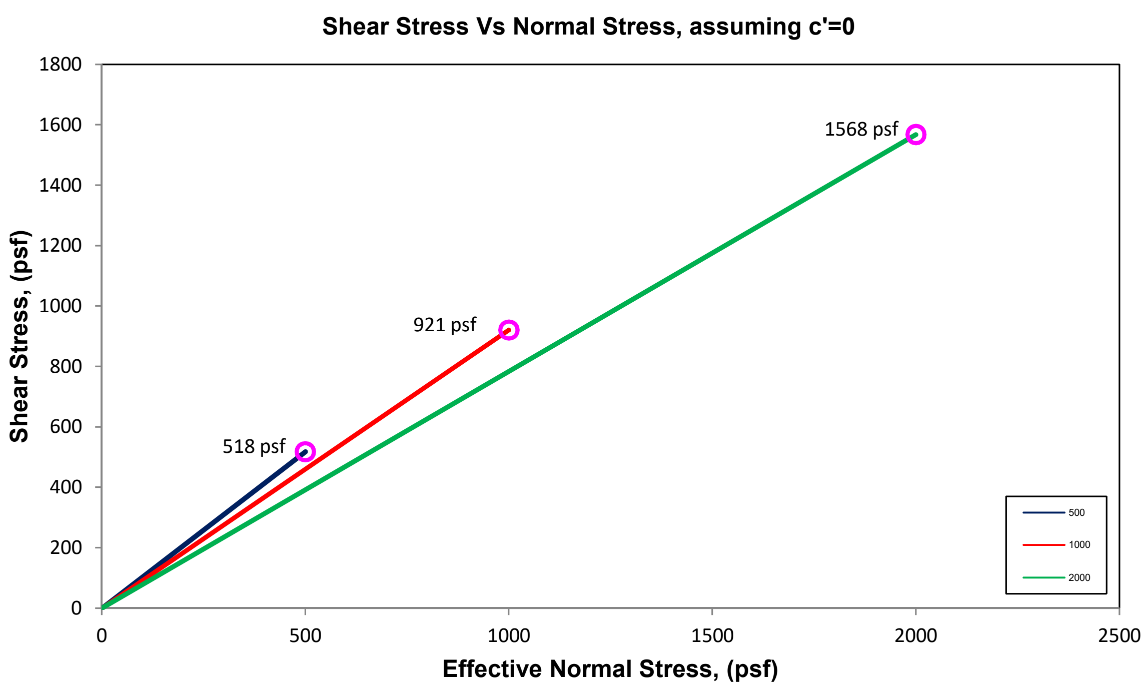


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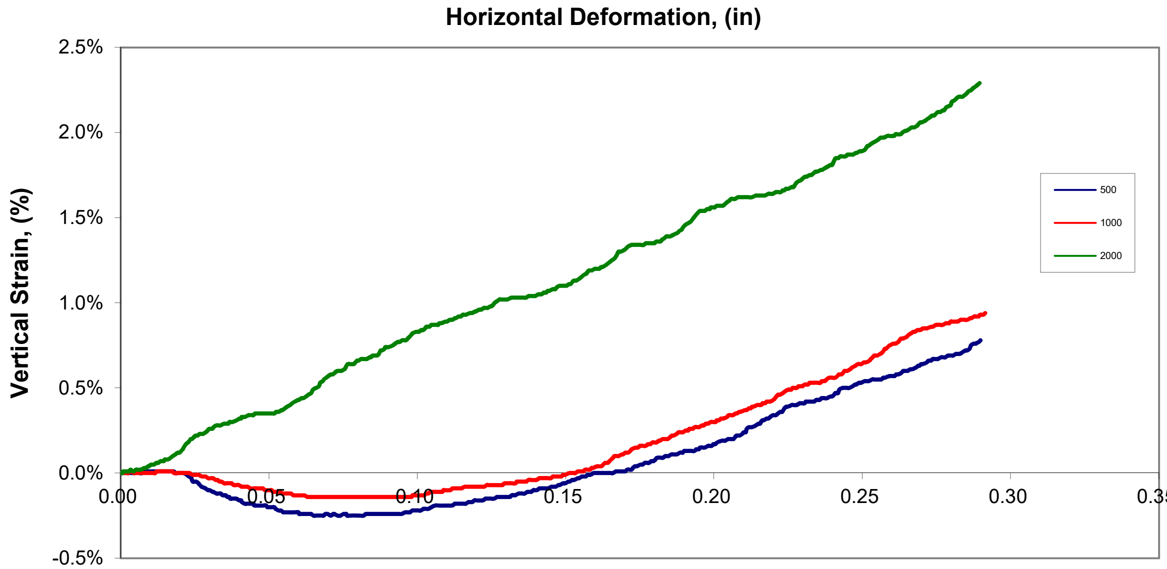
Figure No.: C-5b

Wilding Engineering, Inc.

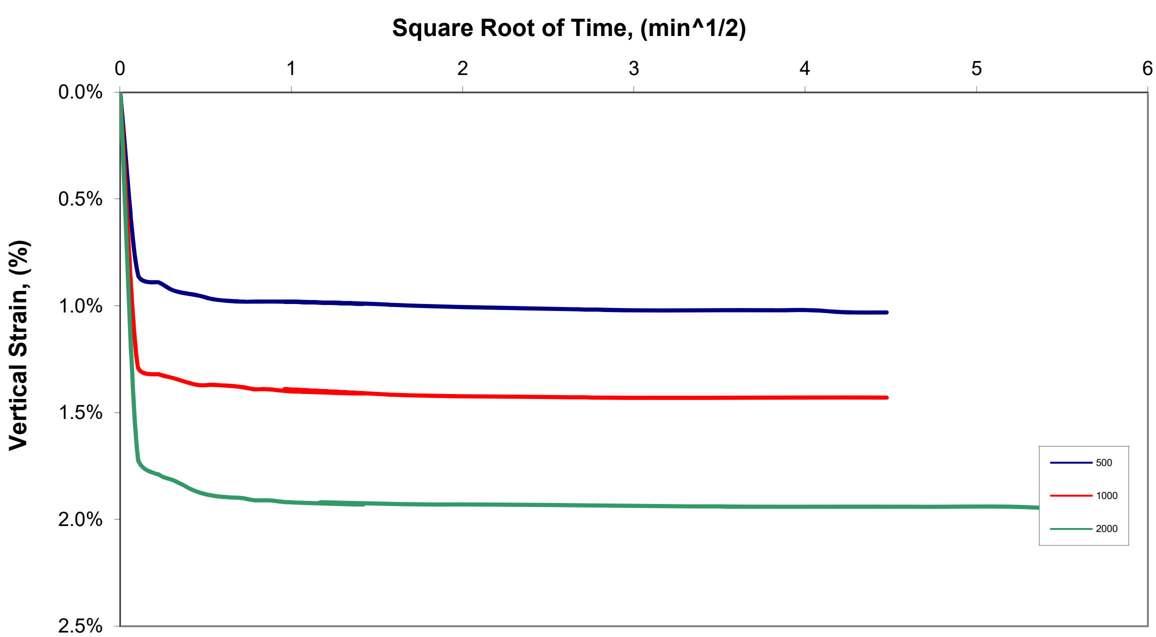
ASTM D3080-14

Tested By	Jason Wright	Date	2/27/19	Computed By	Jeremy Wright PEI	Date	2/28/19	Checked By	Shun Li PE	Date	2/28/19

Vertical Strain Vs. Horizontal Deformation



Consolidation Curves



WILDING ENGINEERING

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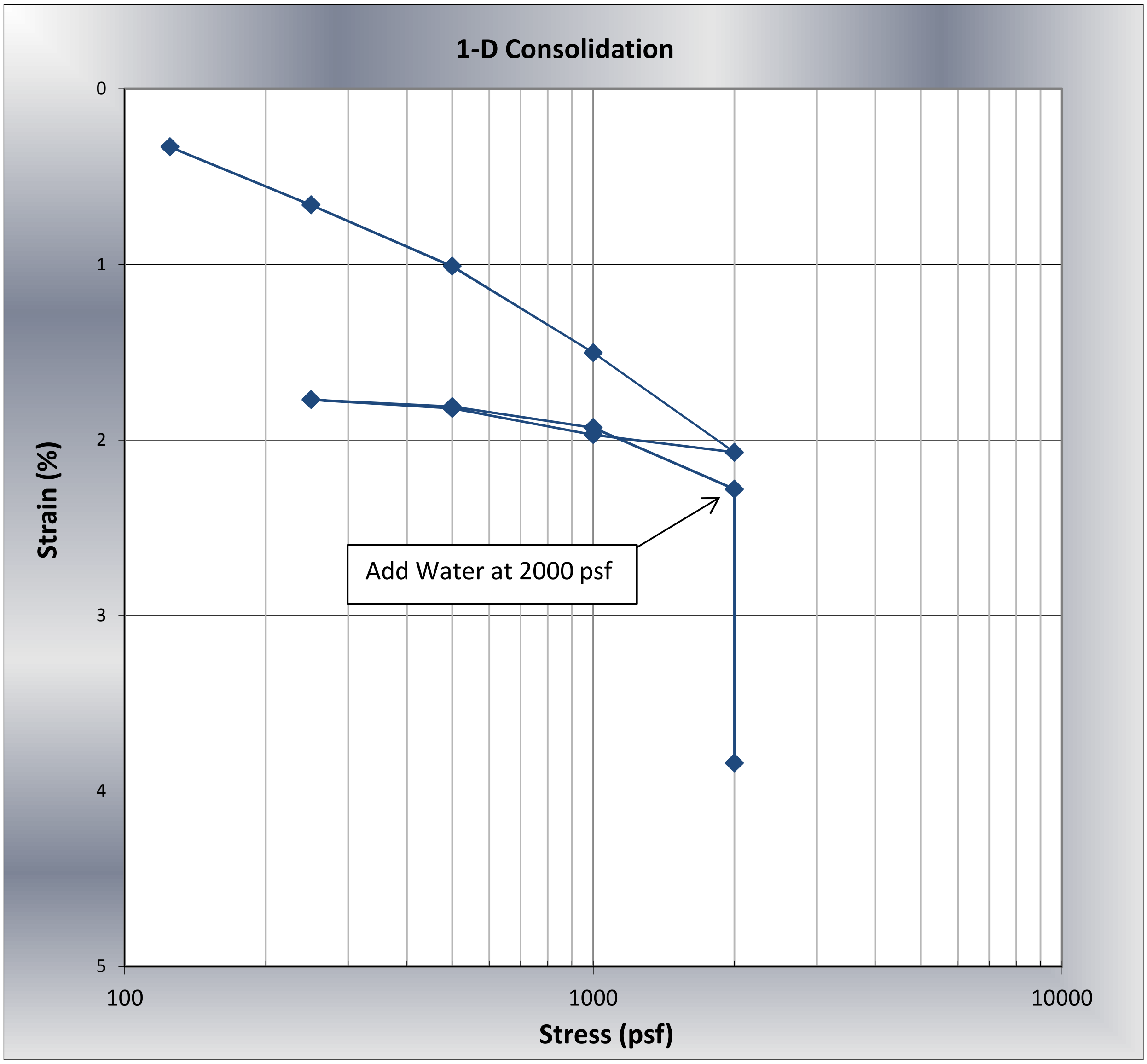
Figure No.: C-5c

1-D SWELL OR COLLAPSE (ASTM D4546)

Client: Lovell Development
Project Number: 18187

Project Name: Hidden Pines Subdivision
Project Location: Riverton, Utah



Sample Location:	TP-4	Dry Density (pcf):	85.6
Sample Depth (ft):	3	Moisture Content (%):	16.7
Sample Description:	Lean CLAY with Sand	Liquid Limit:	33
USCS Classification:	CL	Plastic Limit:	19
Collapse (%):	1.5%	Fines Content (%):	83

SOIL CORROSIVITY

Wilding Engineering, Inc

WILDING
ENGINEERING

CLIENT Lovell Development
PROJECT NAME Hidden Pines
PROJECT NUMBER 18187
PROJECT LOCATION Riverton, Utah

American West
ANALYTICAL LABORATORIES

INORGANIC ANALYTICAL REPORT

Client: Wilding Engineering, Inc. **Contact:** Shun Li
Project: Lovell-Cazier Property / 18187
Lab Sample ID: 1902438-001
Client Sample ID: TP-1 @ 6'
Collection Date: 2/18/2019
Received Date: 2/21/2019 1546h

Analytical Results

Compound	Units	Date Prepared	Date Analyzed	Method Used	Reporting Limit	Analytical Result	Qual
pH @ 25° C	pH Units		2/21/2019 1845h	SW9045D	1.00	**6.88**	H
Resistivity	ohm-cm		2/22/2019 545h	SM2510B	10.0	**2,470**	&
Sulfate	mg/kg-dry		2/22/2019 700h	SM4500-SO4-E	10.8	**63.9**	&

& - Analysis is performed on a 1:1 DI water extract for soils.
H - Sample was received outside of the holding time.

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Salt Lake City, UT 84119

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e-mail: awal@awal-labs.com

web: www.awal-labs.com

Kyle F. Gross
Laboratory Director

Jose Rocha
QA Officer

INORGANIC ANALYTICAL REPORT

Client: Wilding Engineering, Inc.
Contact: Shun Li
Project: Lovell-Cazier Property / 18187
Lab Sample ID: 1902438-002
Client Sample ID: TP-8 @ 3.5'
Collection Date: 2/18/2019
Received Date: 2/21/2019 1546h

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web: www.awal-labs.com

Kyle F. Gross
Laboratory Director

Jose Rocha
QA Officer

Analytical Results

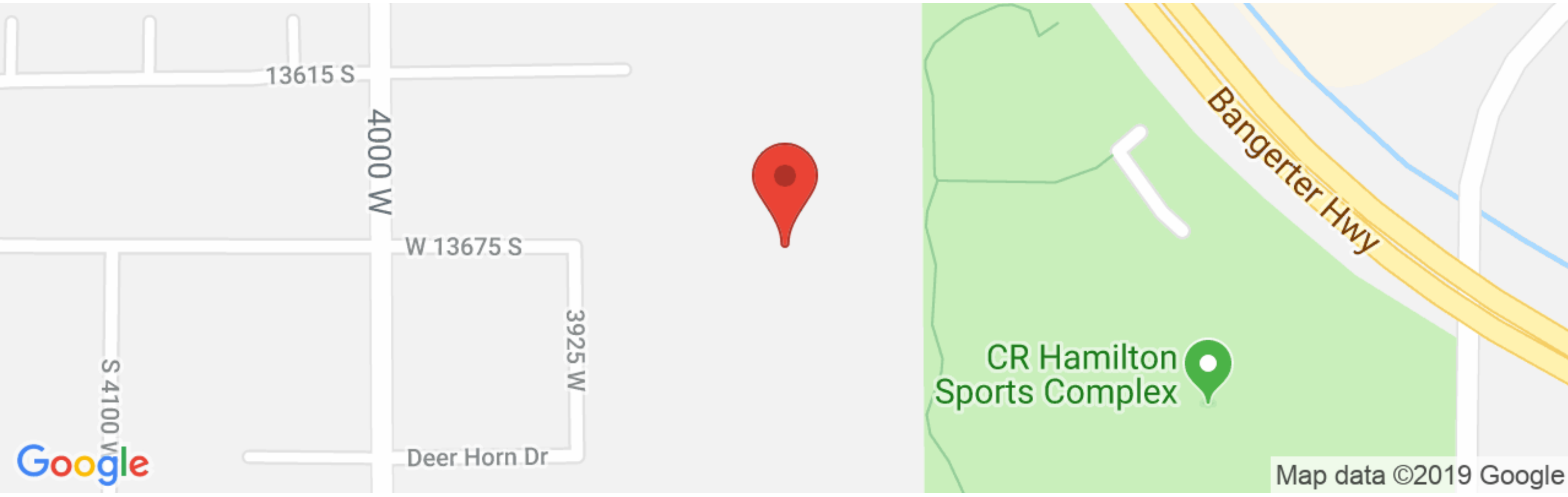
Compound	Units	Date Prepared	Date Analyzed	Method Used	Reporting Limit	Analytical Result	Qual
pH @ 25° C	pH Units		2/21/2019 1845h	SW9045D	1.00	**7.07**	H
Resistivity	ohm-cm		2/22/2019 545h	SM2510B	10.0	**1,290**	&
Sulfate	mg/kg-dry		2/22/2019 700h	SM4500-SO4-E	12.5	**106**	&

& - Analysis is performed on a 1:1 DI water extract for soils.
H - Sample was received outside of the holding time.

APPENDIX D

SEAOC / OSHPD

Latitude, Longitude: 40.502688, -111.982717

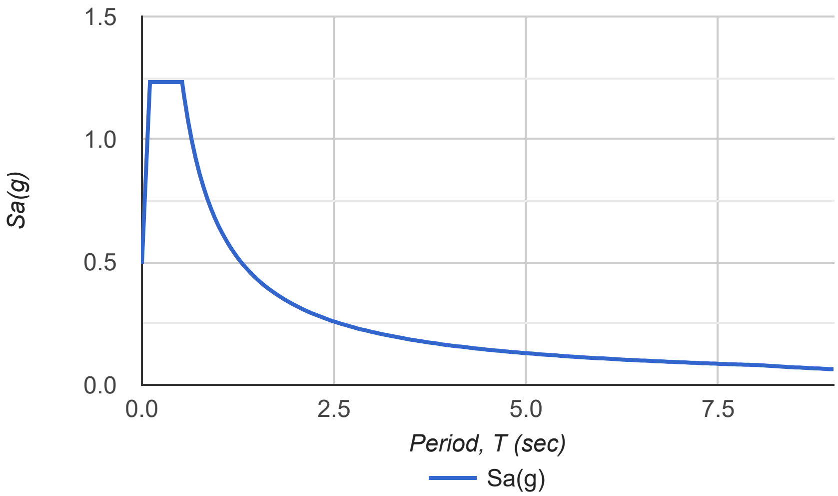


Date	3/4/2019, 2:35:32 PM
Design Code Reference Document	ASCE7-10
Risk Category	II
Site Class	D - Stiff Soil

Type	Value	Description
S_S	1.218	MCE_R ground motion. (for 0.2 second period)
S_1	0.404	MCE_R ground motion. (for 1.0s period)
S_{MS}	1.233	Site-modified spectral acceleration value
S_{M1}	0.645	Site-modified spectral acceleration value
S_{DS}	0.822	Numeric seismic design value at 0.2 second SA
S_{D1}	0.43	Numeric seismic design value at 1.0 second SA

Type	Value	Description
SDC	D	Seismic design category
F_a	1.013	Site amplification factor at 0.2 second
F_v	1.596	Site amplification factor at 1.0 second
PGA	0.499	MCE_G peak ground acceleration
F_{PGA}	1.001	Site amplification factor at PGA
PGA_M	0.499	Site modified peak ground acceleration
T_L	8	Long-period transition period in seconds
SsRT	1.218	Probabilistic risk-targeted ground motion. (0.2 second)
SsUH	1.469	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration
SsD	2.162	Factored deterministic acceleration value. (0.2 second)
S1RT	0.404	Probabilistic risk-targeted ground motion. (1.0 second)
S1UH	0.488	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration.
S1D	0.712	Factored deterministic acceleration value. (1.0 second)
PGAd	0.738	Factored deterministic acceleration value. (Peak Ground Acceleration)
C_{RS}	0.829	Mapped value of the risk coefficient at short periods
C_{R1}	0.828	Mapped value of the risk coefficient at a period of 1 s

MCER Response Spectrum



Design Response Spectrum