#1

GEOTECHNICAL INVESTIGATION
ROCKY MOUNTAIN DANCE

PROPERTY LOCATION
3657 WEST MEADOW SPRING LANE
RIVERTON, UT

Project No.: 18008

Prepared For:
JT WILSTEAD HOLDINGS LLC
ATTN: JUDD WILSTEAD

FEBRUARY 23, 2018

WILDING
ENGINEERING

14721 SOUTH HERITAGE CREST WAY
BLUFFDALE, UTAH 84065

801.553.8112

OK
AC-BKM

APPENDICES

LIST OF TABLES AND FIGURES

1 INTRODUCTION

This report presents the geotechnical investigation for the proposed Rocky Mountain Dance to be located at 3657 West Meadow Springs Lane in Riverton, Utah as shown on the Site Vicinity Map in Appendix A (Figure A-1). The geotechnical investigation was performed in accordance with Wilding Engineering's proposal dated January 2, 2018.

The filed investigation consisted of five (5) borings. The borings were advanced to depths of 6.5 to 41.4 feet below existing ground surface. Detailed Test Pit Logs can be found in Appendix B (Figures B-2 to B-7). Recommendations in this report are based upon information gathered from the field investigations, site inspections, lab testing, and from reviewing geologic map and reports of the area.

2 PURPOSE AND SCOPE

The purpose of this investigation was to assess the suitability of on-site soils for the construction of a 4,400 square foot single story dance studio with the associated utilities, landscaping, parking, and driveways. The investigation included a review of surface water and groundwater conditions. Engineering and construction recommendations presented herein are based on subsurface conditions encountered in the field.

3 SITE AND PROJECT INFORMATION

3.1 PROJECT DESCRIPTION

Based on our understanding of the project, the proposed development will consist of the construction of a 4,400 square foot single story dance studio with associated utilities, landscaping, parking, and driveways. We understand the building will be constructed with the first floor being slab-on-grade. 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 100 kips and 4 kip/ft, respectively.

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

3.2 EXISTING SITE CONDITIONS

The subject property has an area of approximately 1.1 acres and is located at 3657 South Meadow Spring Lane in Riverton, Utah. The site is located at approximately latitude and longitude of 40.524922° and 111.977980° respectively.

At the time of our field investigation, the project site consisted of a vacant property. No existing structures or evidence of previously existing structures were observed. Vegetation within the property consisted of sparse native grasses and weeds. The subject site is bound by a residential lot to the south, Meadow Springs Lane to the north and west, and commercial property to the west. The property sits at an elevation of approximately 4613 to 4605 feet above mean sea level and slopes downward toward the north–northwest towards Meadow Spring Road. The existing grade of the property is approximately 30H:1V.

4 GENERAL GEOLOGY

4.1 SURFICIAL GEOLOGY

Based on the available geologic maps[1], the project site is underlain by Late Pleistocene-aged fine-grained lacustrine deposits (Qlf). The fine-grained lacustrine deposits are mapped consist of calcareous, laminated silt, clayey silt, and sandy silt. Underlying the lacustrine fine-grained deposits we observed Late Pleistocene-aged lacustrine sand deposits (Qls). The lacustrine sand deposits are mapped to consist of tan, brown, and grey, calcareous, silty, fine-grained sand. These deposits were observed in each of our borings and persisted to the maximum depth of our explorations.

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 less than 5% of having a seismic event within a 100-year period that will be strong enough to cause liquefaction[3]. Groundwater was not encountered in the boreholes advanced

1 Davis, F.D., 2000, Geologic map of the Midvale quadrangle, Salt Lake County, Utah: Utah Geological Survey, Map 177, scale 1:24,000.

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., Spitzley, J.E., Allen, A.C., 1994, Liquefaction potential map for Salt Lake County, Utah complete technical report: Utah Geological Survey, Contract Report 94-9, p. 48.

for this investigation to the maximum depths explored; therefore, we assess the liquefaction potential for the subject property to be very low.

5 FIELD EXPLORATIONS

5.1 SUBSURFACE INVESTIGATION

Subsurface soil conditions at the project site were explored at the site by advancing five (5) borings at representative locations within the subject property. The borings were advances using a truck mounted drill rig equipped with hollow stem augers to depths of 6.5 to 41.4 feet below the existing site grades. Stratigraphy and classification of the soils were logged under the direction of a geotechnical engineer.

Soil samples were obtained by driving a standard 2-inch outside diameter split-spoon sample (SPT) into the soil a distance of 18-inches using a 140-ib hammer dropped from a height of 30-inches. The number of blows required to drive the sampler 12-inches in known as the standard penetration resistance or N-value. A liner was not used in the SPT sampler, which was not designed to accommodate a liner. The N-values provided a measure of the relative density of granular soils and the relative consistency of stiffness of cohesive soils. Undisturbed samples of fine-grained soils were collected through the use of Shelby tubes. Samples types with depths and blow counts are shown in detail in the Boring Logs found in Appendix B (Figures B-2 to B-6). A Key to Soil Symbols is presented on Figure B-1.

5.2 SUBSURFACE CONDITIONS

5.2.1 Soils

The soil encountered in the borings consisted of Lean Clay (CL) with roots and vegetation underlain by Lean Clay (CL), Lean CLAY with Sand (CL), Sandy Lean CLAY (CL), Well-Graded Gravel with Silt and Sand (GW-GM), Poorly Graded GRAVEL with Silt and Sand (GP-GM), and Poorly Graded SAND with Silt and Gravel (SP-SM). The soils encountered in B-1 consisted of granular soils deposits to the maximum depth explored. The soils encountered in B-2 consisted of fine grained soils to the maximum depth explored.

The stratification lines shown on the enclosed Boring Logs (Figures B-2 to B-7) 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 in the borings advanced for this investigation. It should be noted that it is possible for the groundwater 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 groundwater conditions during construction which may differ from those encountered during our field investigation. If groundwater is encountered during construction, Wilding Engineering must be notified to observe changing conditions and provide recommendations.

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 (ASTM D422)
- Percent of Fines by Washing (ASTM D1140)
- Atterberg Limits Test (ASTM D4318)
- Moisture Content of Soil by Mass (ASTM D2216)
- 1-D Consolidation of Soil (ASTM D2435)

The results of laboratory tests are presented on the boring 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-5).

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 physical 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 in conjunction with construction, and/or if design and layout changes are initiated, Wilding Engineering must be informed so that our recommendations can be reviewed and revised as changes or conditions may require.

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

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, all topsoil, undocumented fill, or any soil containing organic or deleterious materials shall be removed where structures or concrete flatwork are to be placed. Topsoil may be stockpiled on site for subsequent use in landscape areas. Any unsuitable material (loose, soft, saturated, disturbed, or otherwise unstable soils) where structures or pavements are to be placed shall be replaced with structural fill according to Sections 7.2.4 and 7.2.5 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 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, typically extending from mid-May to mid-October. Any modifications to the grading plans should be reviewed by the Geotechnical Engineer.

7.2.2 Soft Soil Stabilization

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

In large areas of soft subgrade soils, stabilization of the subgrade may not be practical using the method outlined above. In these areas, it may be more economical to place a woven geotextile fabric against the soft soils covered by 18 inches of coarse, sub-rounded to rounded material over the woven geotextile. An inexpensive non-woven geotextile "filter" fabric should also be placed over the top of the coarse, sub-rounded to rounded fill prior to placing structural fill or pavement section soils to reduce infiltration of fines from above. The woven geotextile should be Mirafi RS280i or prior approved equivalent. The filter fabric should consist of a Mirafi 140N, or equivalent as approved by the Geotechnical Engineer.

7.2.3 *Excavation Stability*

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 maintain 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 one and one-half horizontal to one vertical (1.5H:1V), 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.

7.2.4 *Structural Fill Material*

All fill placed for support of structures, concrete flatwork, or pavements shall consist of structural fill. The native fine-grained soils (silt and clay) are unsuitable for use as structural fill but may be used as fill in landscape areas. Structural fill may consist of an imported material meeting the requirements below. 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 inert 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 a maximum of 15% passing the No.200 sieve. Fill material portion finer than the No.40 sieve shall have a liquid limit (LL) less than 35 and a plasticity index (PI) less than 25, see Table 7.1 below for gradation specification. This material shall be free from organics, debris, frozen material, and other compressible or deleterious materials.

Table 7.1 – Structural Fill Requirements

Grain Size	Percent Passing
4-inch	100
2-inch	85 to 100
No. 4	15 to 50
No. 200	< 25
Plastic Index (PI)	< 25
Liquid Limit (LL)	< 35

The contractor should anticipate testing all soils used as structural fill frequently to assess the maximum dry density, fines content, and moisture content, etc.

7.2.5 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 should be compacted to at least 95% of the maximum dry density, as determined by ASTM D1557. The moisture content should be at or slightly above the optimum moisture content at the time of placement and compaction. Also, prior to placing any fill, the excavations should be observed by the geotechnical engineer to observe that any unsuitable materials or loose soils have been removed. In addition, proper grading should precede placement of fill, as described in Section 7.2.1, Site Preparation and Grading.

Fill soils placed for subgrade below exterior flat work and pavements should be within 3% of the optimum moisture content when placed and compacted to at least 95% of the maximum dry density as determined by ASTM D1557. All utility trenches backfilled below the proposed structure, pavements, and flatwork concrete should be backfilled with structural fill that is within 3% of the optimum moisture content when placed and compacted to at least 95% of the maximum dry density as determined by ASTM D1557. All other trenches, in landscape areas, should be backfilled and compacted to at least 90% of the maximum dry density (ASTM D1557).

7.2.6 Utility Trenches

In our experience, individual municipalities will have local requirements regarding installation of utilities; however, in the absence of specified requirements the following is recommended:

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 Section 7.2.4, Structural Fill Material.

> **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 7.2.4), 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. 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.

Backfill for utility trenches located beneath structures or driveways 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 D1557).

7.2.7 Moisture Protection and Surface Drainage

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

Moisture should not be allowed to infiltrate the soils in the vicinity of, or upslope from, the structures. We recommend that roof runoff devices be installed to direct all runoff a minimum of 10 feet away from structures. The grade within 10 feet of a structure should be sloped a minimum of 5% away from the structure.

7.3 FOUNDATIONS

The foundations for the proposed structures may consist of conventional strip and/or spread footings. Strip and spread footings should be a minimum of 20 and 36 inches wide, respectively, and exterior shallow footings should be embedded at least 30 inches below final grade for frost protection and confinement. Interior shallow footings not susceptible to frost conditions should be embedded at least 18 inches for confinement.

7.3.1 *Installation and Bearing Material*

Primarily granular soils were encountered in B-1, while fine grained soils were encountered in B-2. To limit differential settlement we recommend the footings be placed entirely on the clay soils or on a minimum of 2 feet of structural fill which is bearing on undisturbed native soils. Structural fill should meet material recommendations and be placed and compacted as recommended in Sections 7.2.4 and 7.2.5.

If soils are saturated during construction, soft or pumping soils may be exposed in foundation excavations due to the fine-grained nature of some of the soils observed in our test pits. Where soft or pumping soils are exposed, prior to placement of foundations, the soft or pumping soils should be stabilized as recommended in Section 7.2.2 of this report.

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 where over excavation is required. Footings placed on slopes shall be benched so that all footing bases are horizontal.

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 footings.

7.3.2 *Bearing Pressure*

Conventional strip and spread footings founded entirely on undisturbed native sand soils or entirely on 2 feet of properly placed and compacted structural fill extending down to undisturbed native soils as described above may be proportioned for a maximum net allowable bearing capacity of **2,000 pounds per square foot (psf).** The recommended net allowable bearing pressure refers to the total dead load and can be increased by 1/3 to include the sum of all loads including wind and seismic.

7.3.3 *Settlement*

Settlements of properly designed and constructed conventional footings, founded as described above, are anticipated to be less than 1 inch. Differential settlements should be on the order of half (½) the total settlement over 30 feet.

7.3.4 *Frost Depth*

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 18 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*

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 the direction of a geotechnical engineer for material and compaction requirements.

7.3.6 Foundation Drainage

Groundwater was not encountered in the borings advanced for this investigation. Soils encountered in the subsurface explorations at elevations of proposed foundations consisted of clay and silt (fine-grained) soils.

Wilding Engineering recommends footings and foundations be designed according to the International Building Code (IBC 2015). Soils with poor drainage characteristics require that a foundation drain be installed to allow water to drain away from the foundation and to reduce the risk of flooding of enclosed interior subgrade spaces. The soils encountered in the test pits excavated for this investigation are considered to have poor drainage characteristics. If a basement is incorporated into the design of the proposed structure, a foundation drain is required in these soil types according to the IBC.

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 friction between the base of the footing and the supporting subgrade. In determining the frictional resistance, a coefficient of friction of 0.34 should be used for native soils against concrete.

7.4.2 Lateral Earth Pressures on Foundation Walls

Ultimate lateral earth pressures from native soil backfill 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

Condition	Lateral Pressure Coefficient	Equivalent Fluid Density (pounds per cubic foot)
Active*	0.36	45
At-rest**	0.53	66
Passive*	2.77	346
Seismic Active***	0.69	86
Seismic Passive***	-1.04	-130

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

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

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

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

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

7.5 CONCRETE SLABS-ON-GRADE

Concrete slabs-on-grade should be constructed over at least 4 inches of compacted gravel overlying native soils or a zone of structural fill that is at least 6 inches thick. Disturbed native soils should be compacted to at least 95% of the maximum dry density as determined by ASTM D1557 (modified proctor) prior to placement of gravel. The gravel should consist of road base or clean drain rock with a ¾-inch maximum particle size and no more than 12 percent fines passing the No. 200 mesh sieve. The gravel layer should be compacted to at least 95 percent of the maximum dry density of the modified proctor or until tight and relatively unyielding if the material is non-proctorable. All concrete slabs should be designed to minimize cracking as a result of shrinkage. Consideration should be given to reinforcing the slab with welded wire, rebar, or fiber mesh.

7.6 Seismic Information

7.6.1 Faulting and Seismicity

Based on the USGS Quaternary Fault and Fold Database of the United States, the project site is located approximately 7.2 miles west of the closest portion of the Salt Lake City section of the Wasatch fault zone and approximately 9.5 miles south of the Granger fault within the West Valley fault zone. Surface rupture has not been mapped within and was not observed at the site.

Seismic screening was conducted for the subject property utilizing the USGS U.S. Seismic Design Maps tool[4] with a probability of exceedance of 2% in 50 years (return period of ~2400 years). Probabilistic ground motion values for S_S and S_1 are 1.193g and 0.396g, respectively. Values were estimated with latitude and longitude of 40.52492° and -111.97798° respectively (See Figure 7.1 below).

The design spectral accelerations were determined according to IBC 2015 and ASCE 07-10 and were found to be 0.813g and 0.425g 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.

4 "U.S. Seismic Design Maps" USGS Earthquake Hazards Program, https://earthquake.usgs.gov/designmaps/us/application.php

Figure 7.1 – Seismic Ground Motion Parameters

Site Coordinates:

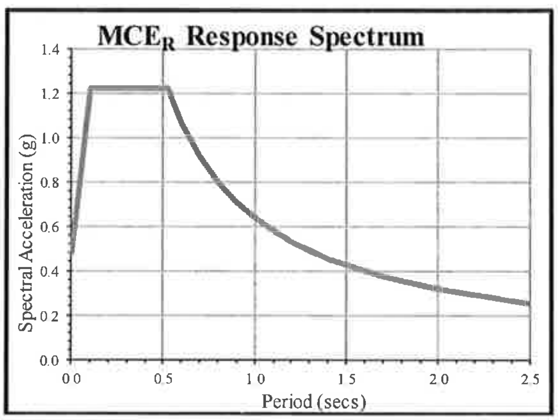
Latitude:	**40.52492**	degrees
Longitude:	**-111.97798**	degrees
Exceedance Probability:	**2**	%
Exposure Time:	**50**	years
S_s =	**1.193**	From USGS Seismic Design Maps
S_1 =	**0.396**	
Site Soil Class:	**D**	(Stiff soil)
F_a =	1.02	
F_v =	1.61	

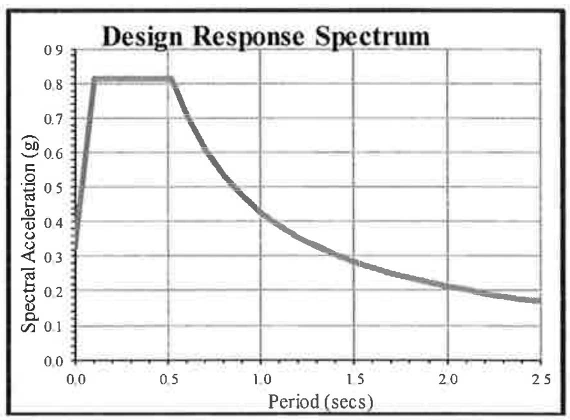
Adjusted for Site Conditions:

$S_{MS} = F_a \times S_S$ =	(1.02 x 1.193) =	1.220	g
$S_{M1} = F_v \times S_1$ =	(1.61 x 0.396) =	0.637	g
$S_{DS} = 2/3 \times S_{MS}$ =	(2/3 x 1.220) =	0.813	g
$S_{D1} = 2/3 \times S_{M1}$ =	(2/3 x 0.637) =	0.425	g
MCE_R PGA = 0.4 x S_{MS} =	(0.4 x 1.22) =	**0.49**	g
MCE_R $T_0 = 0.2 \times (S_{M1}/S_{MS})$ =	(0.2 x [0.64/1.22]) =	0.10	secs
MCE_R $T_S = (S_{M1}/S_{MS})$ =	(0.64/1.22) =	0.52	secs

Site Class	Values of Site Factor, F_a, for Short-Period Range of Spectral Acceleration				
	$S_S \le 0.25$	$S_S = 0.5$	$S_S = 0.75$	$S_S = 1.0$	$S_S \ge 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	See Section 11.4.7 of ASCE 7				

Site Class	Values of Site Factor, F_v, for Long-Period Range of Spectral Acceleration				
	$S_1 \le 0.1$	$S_1 = 0.2$	$S_1 = 0.3$	$S_1 = 0.4$	$S_1 \ge 0.5$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	2.4
F	See Section 11.4.7 of ASCE 7				





7.6.2 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 0.813g for short periods (S_{DS}) and 0.425g 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

On-site soil characteristics from the soil samples collected were used in determining soil strength for design of pavements for the proposed development. Based on the fines content of the soils observed in the test pits excavated for this investigation, an assumed CBR of 4 was used in our analysis. We assumed that vehicle traffic in and out of paved areas would consist of a moderate volume of automobiles and light trucks with occasional medium and heavy trucks. Based on these assumptions, our analysis used 32,000 ESALs with a twenty (20) year design period of 90% reliability, standard deviation of 0.45, and initial and terminal serviceability of 4.2 and 2.5, respectively.

The following sections provide preparation and design requirements for pavement based on AASHTO design procedures. The table below (Table 7.3) presents recommended pavement section thickness based on the above assumptions and the material descriptions provided in the following sections. The three pavement sections given below are equivalent options that may be selected based on economic considerations.

Table 7.3 – Pavement Design Recommended Thickness

Pavement Materials	Recommended Minimum Thickness (inches)		
	Pavement 1	Pavement 2	Pavement 3
Asphaltic Concrete	3	3	3.5
Untreated Base Course	12	9	8
Granular Borrow	---	6	---

For dumpster pads, we recommend a pavement section consisting of 6 inches of Portland cement concrete, 4 inches of structural fill, over properly prepared suitable natural subgrade or site grading structural fills extending to suitable natural soils. Dumpster pads shall not be constructed overlying non-engineered fills unless heavily reinforced.

7.7.1 Subgrade Preparation

All topsoil, undocumented fill, 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. Any unsuitable soils shall be removed and replaced with structural fill according to Sections 7.2.4 and 7.2.5. Any areas of fill or disturbed areas shall be compacted to 95% of the ASTM D1557 modified proctor. A geotechnical engineer shall observe unsuitable subgrade remediation.

7.7.2 Granular Borrow

If economical, granular subbase may be placed to reduce the required thickness of untreated base course (road base) material. Subbase material should consist of a granular borrow material as defined in APWA Standard Specifications, Section 31 05 13, "Common Fill", and should have a CBR of 30. See Table 7.3 above for recommended granular subbase section thickness. Granular borrow shall be placed and compacted in accordance with Sections 7.2.4 and 7.2.5 of this report.

7.7.3 Untreated Base Course

A thickness of eight (8) to twelve (12) inches of untreated base course is required for paved areas per table 7.3. The base course shall comply with Aggregate Class A, ¾-inch mix per APWA Standard Specifications, Section 32 11 23, "Aggregate Base Courses". Untreated base course shall be placed and compacted in accordance with Sections 7.2.4 and 7.2.5 of this report.

7.7.4 Surface Course

A thickness of three (3) to three-and-a-half (3.5) inches of asphalt concrete pavement is required for all paved areas per table 7.3. This asphalt concrete pavement is to comply with UDOT Standard Specifications, Section 02741, and "Hot Mix Asphalt (HMA)".

7.7.5 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. The construction should not change the original storm drainage plan unless a new storm drainage plan is made 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

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

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

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

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.

Mike E. Carlton, P.E.
Professional Engineer

Jeremy G. Wright, PEI
Wilding Engineering Inc.

APPENDIX A

600 0 600 1200 1800 2400 ft

1:9,600

WILDING ENGINEERING

Wilstead Dance Studio
3657 W. Meadow Spring Lane
Riverton, UT
Project Number: 18008

Site Vicinity Map

Figure A-1

Legend

Approximate Site Boundary

WILDING ENGINEERING

Legend

- Approximate Site Boundary
- Approximate Boring Location

Wilstead Dance Studio
3657 W. Meadow Spring Lane
Riverton, UT
Project Number: 18008

Exploration Location Map

Figure A-2

APPENDIX B

KEY TO SYMBOLS

Wilding Engineering

CLIENT Judd Wilstead

PROJECT NUMBER 18008

PROJECT NAME Wilstead Dance Studio

PROJECT LOCATION Riverton, UT

LITHOLOGIC SYMBOLS
(Unified Soil Classification System)

CL: USCS Low Plasticity Clay

FILL: Fill (made ground)

GP-GM: USCS Poorly-graded Gravel with Silt

GW-GM: USCS Well-graded Gravel with Silt

SP-SM: USCS Poorly-graded Sand with Silt

SAMPLER SYMBOLS

3" O.D. Thin Walled Shelby Tube

2" O.D. Split Spoon

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 - 2/20/18 15:46 - G:\DATA\18008 WILSTEAD STUDIO\SOILS\GINT\BORING LOGS.GPJ

Wilding Engineering

BORING NUMBER B-1

CLIENT Judd Wilstead

PROJECT NAME Wilstead Dance Studio

PROJECT NUMBER 18008

PROJECT LOCATION Riverton, UT

DATE STARTED 1/9/18 COMPLETED 1/9/18

GROUND ELEVATION 4613 ft HOLE SIZE 4.25 inches

DRILLING CONTRACTOR Earthcore

GROUND WATER LEVELS:

DRILLING METHOD Hollow Stem Auger

AT TIME OF DRILLING ---

LOGGED BY JGW CHECKED BY DPW

AT END OF DRILLING ---

NOTES

AFTER DRILLING ---

DEPTH (ft)	SAMPLE TYPE NUMBER	RECOVERY %	BLOW COUNTS (N VALUE)	TESTS	U.S.C.S.	GRAPHIC LOG	MATERIAL DESCRIPTION
0	T 1	39	3-4-7 (11)		CL		**Lean CLAY:** medium stiff, dry to moist, light brown, glass in sampler
2.5	T 2	39	5-7-6 (13)		CL		4610.5 **Lean CLAY:** stiff, moist, light brown
5	T 3	39	8-11-16 (27)		CL		-- very stiff
7.5	T 4	56	12-18-25 (43)	MC = 2% LL = NP PL = NP Fines = 7%	GW-GM		4605.5 **Well-Graded GRAVEL with Silt and Sand:** dense, moist, light brown
10.0	T 5	0	50		GP-GM		4603.0 **Well-Graded SAND with Silt and Gravel:** very dense, moist, light brown
15	T 6	100	15-50/5"	MC = 5% LL = NP PL = NP Fines = 9%	GP-GM		
20.0	T 7	0	17-25-30 (55)		SP-SM		4593.0 **Poorly Graded SAND with Silt and Gravel:** very dense, moist, light brown
25							

(Continued Next Page)

BORING NUMBER B-1

WILDING ENGINEERING

Wilding Engineering

CLIENT Judd Wilstead

PROJECT NAME Wilstead Dance Studio

PROJECT NUMBER 18008

PROJECT LOCATION Riverton, UT

DEPTH (ft)	SAMPLE TYPE NUMBER	RECOVERY %	BLOW COUNTS (N VALUE)	TESTS	U.S.C.S.	GRAPHIC LOG	MATERIAL DESCRIPTION
25	T 8	80	20-49-50/3"		SP-SM		**Poorly Graded SAND with Silt and Gravel:** very dense, moist, light brown *(continued)*
30	T 9	67	16-25-30 (55)				
35	T 10	72	11-29-30 (59)				
40	T 11	94	13-21-17 (38)				-- dense
41.5							4571.5

Bottom of borehole at 41.5 feet.

BORING NUMBER B-2

Wilding Engineering

WILDING ENGINEERING

CLIENT Judd Wilstead	**PROJECT NAME** Wilstead Dance Studio
PROJECT NUMBER 18008	**PROJECT LOCATION** Riverton, UT
DATE STARTED 1/9/18 **COMPLETED** 1/9/18	**GROUND ELEVATION** 4611 ft **HOLE SIZE** 4.25 inches
DRILLING CONTRACTOR Earthcore	**GROUND WATER LEVELS:**
DRILLING METHOD Hollow Stem Auger	**AT TIME OF DRILLING** ---
LOGGED BY JGW **CHECKED BY** DPW	**AT END OF DRILLING** ---
NOTES	**AFTER DRILLING** ---

DEPTH (ft)	SAMPLE TYPE NUMBER	RECOVERY %	BLOW COUNTS (N VALUE)	TESTS	U.S.C.S.	GRAPHIC LOG	MATERIAL DESCRIPTION
0	T 1	33	3-4-5 (9)		CL		**Lean CLAY:** stiff, moist, light brown
	T 2	44	8-10-15 (25)				-- very stiff
5	S 3	75		MC = 21% DD = 101 pcf LL = 46 PL = 21 Fines = 98%			-- medium stiff
7.5							4603.5
	T 4	44	12-27-19 (46)		SP-SM		**Poorly Graded SAND:** dense, moist, light brown
10.0							4601.0
10	T 5	67	7-6-8 (14)	MC = 16% LL = 27 PL = 14 Fines = 81%	CL		**Lean CLAY with Sand:** stiff, moist, light brown
15	T 6	22	6-7-12 (19)	MC = 11% LL = 25 PL = 17 Fines = 57%			-- Sandy LEAN Clay: very stiff
20	T 7	71	18-32-50/5"				
21.4							4589.6

Bottom of borehole at 21.4 feet.

GENERAL BH / TP / WELL - GINT STD US LAB.GDT - 2/20/18 15:12 - G:\DATA\18008 WILSTEAD STUDIO\SOILS\GINT\BORING LOGS.GPJ

BORING NUMBER B-3

WILDING ENGINEERING

Wilding Engineering

CLIENT Judd Wilstead

PROJECT NAME Wilstead Dance Studio

PROJECT NUMBER 18008

PROJECT LOCATION Riverton, UT

DATE STARTED 1/9/18 **COMPLETED** 1/9/18

GROUND ELEVATION 4610 ft **HOLE SIZE** 4.25 inches

DRILLING CONTRACTOR Earthcore

GROUND WATER LEVELS:

DRILLING METHOD Hollow Stem Auger

AT TIME OF DRILLING ---

LOGGED BY JGW **CHECKED BY** DPW

AT END OF DRILLING ---

NOTES

AFTER DRILLING ---

DEPTH (ft)	SAMPLE TYPE NUMBER	RECOVERY %	BLOW COUNTS (N VALUE)	TESTS	U.S.C.S.	GRAPHIC LOG	MATERIAL DESCRIPTION
0	T 1	33	6-7-7 (14)		CL		**Lean CLAY:** stiff, moist, light brown
	T 2	67	14-18-15 (33)	MC = 16% LL = 38 PL = 19 Fines = 91%			-- very stiff
5	T 3	11	4-8-7 (15)			6.5	-- stiff 4603.5

Bottom of borehole at 6.5 feet.

GENERAL BH / TP / WELL - GINT STD US LAB.GDT - 2/20/18 15:12 - G:\DATA\18008 WILSTEAD STUDIO\SOILS\GINT\BORING LOGS.GPJ

BORING NUMBER B-4

WILDING ENGINEERING

Wilding Engineering

CLIENT Judd Wilstead
PROJECT NAME Wilstead Dance Studio
PROJECT NUMBER 18008
PROJECT LOCATION Riverton, UT
DATE STARTED 1/9/18 **COMPLETED** 1/9/18
GROUND ELEVATION 4613 ft **HOLE SIZE** 4.25 inches
DRILLING CONTRACTOR Earthcore
GROUND WATER LEVELS:
DRILLING METHOD Hollow Stem Auger
AT TIME OF DRILLING ---
LOGGED BY JGW **CHECKED BY** DPW
AT END OF DRILLING ---
NOTES
AFTER DRILLING ---

DEPTH (ft)	SAMPLE TYPE NUMBER	RECOVERY %	BLOW COUNTS (N VALUE)	U.S.C.S.	GRAPHIC LOG	MATERIAL DESCRIPTION
0	T 1	56	6-11-13 (24)	CL		**Lean CLAY:** very stiff, moist, light brown
	T 2	17	10-10-12 (22)			
5	T 3	17	5-7-10 (17)		6.5	4606.5

Bottom of borehole at 6.5 feet.

GENERAL BH / TP / WELL - GINT STD US LAB.GDT - 2/20/18 15:12 - G:\DATA\18008 WILSTEAD STUDIO\SOILS\GINT\BORING LOGS.GPJ

BORING NUMBER B-5

Wilding Engineering

CLIENT Judd Wilstead	**PROJECT NAME** Wilstead Dance Studio
PROJECT NUMBER 18008	**PROJECT LOCATION** Riverton, UT
DATE STARTED 1/9/18 **COMPLETED** 1/9/18	**GROUND ELEVATION** 4615 ft **HOLE SIZE** 4.25 inches
DRILLING CONTRACTOR Earthcore	**GROUND WATER LEVELS:**
DRILLING METHOD Hollow Stem Auger	**AT TIME OF DRILLING** ---
LOGGED BY JGW **CHECKED BY** DPW	**AT END OF DRILLING** ---
NOTES	**AFTER DRILLING** ---

DEPTH (ft)	SAMPLE TYPE NUMBER	RECOVERY %	BLOW COUNTS (N VALUE)	U.S.C.S.	GRAPHIC LOG	MATERIAL DESCRIPTION
0	T 1	17	8-7-8 (15)	CL		**Lean CLAY:** stiff, moist, light brown
	T 2	17	6-7-11 (18)			-- very stiff
5	T 3	33	8-8-7 (15)		6.5	-- stiff (4608.5)

Bottom of borehole at 6.5 feet.

GENERAL BH / TP / WELL - GINT STD US LAB.GDT - 2/20/18 15:13 - G:\DATA\18008 WILSTEAD STUDIO\SOILS\GINT\BORING LOGS.GPJ

APPENDIX C

GRAIN SIZE DISTRIBUTION

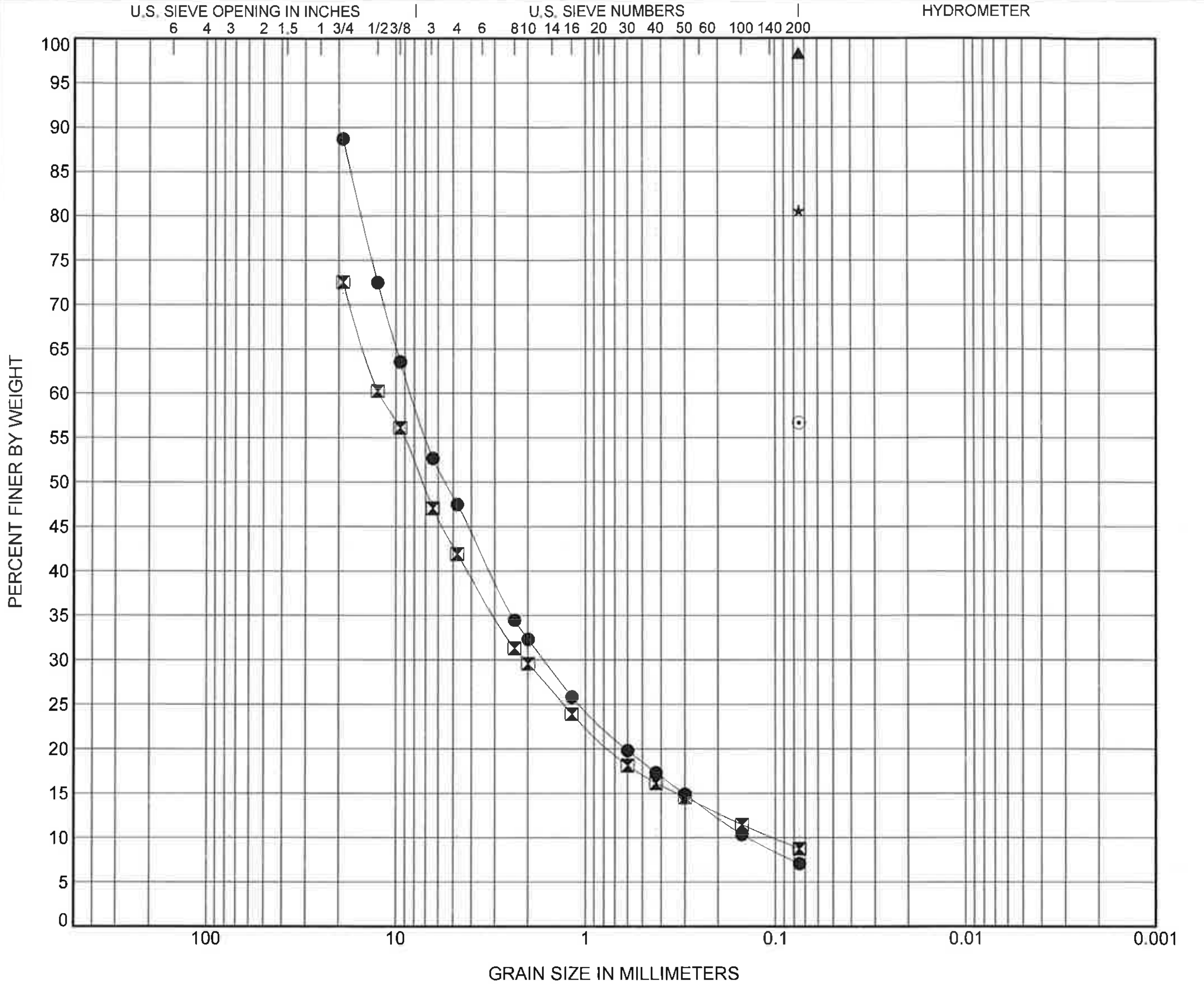
Wilding Engineering

CLIENT Judd Wilstead

PROJECT NAME Wilstead Dance Studio

PROJECT NUMBER 18008

PROJECT LOCATION Riverton, UT



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	BOREHOLE	DEPTH	Classification	LL	PL	PI	Cc	Cu
●	B-1	7.5	**WELL-GRADED GRAVEL with SILT and SAND(GW-GM)**	NP	NP	NP	2.36	60.05
⌧	B-1	15.0	**POORLY GRADED GRAVEL with SILT and SAND(GP-GM)**	NP	NP	NP	3.42	119.19
▲	B-2	5.0	**LEAN CLAY(CL)**	46	21	25		
★	B-2	10.0	**LEAN CLAY with SAND(CL)**	27	14	13		
⊙	B-2	15.0	**SANDY LEAN CLAY(CL)**	25	17	8		

	BOREHOLE	DEPTH	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	B-1	7.5	19	8.351	1.657	0.139	41	40	7.1	
⌧	B-1	15.0	19	12.299	2.084	0.103	31	33	8.8	
▲	B-2	5.0	0.075				0	0	98.3	
★	B-2	10.0	0.075				0	0	80.6	
⊙	B-2	15.0	0.075				0	0	56.7	

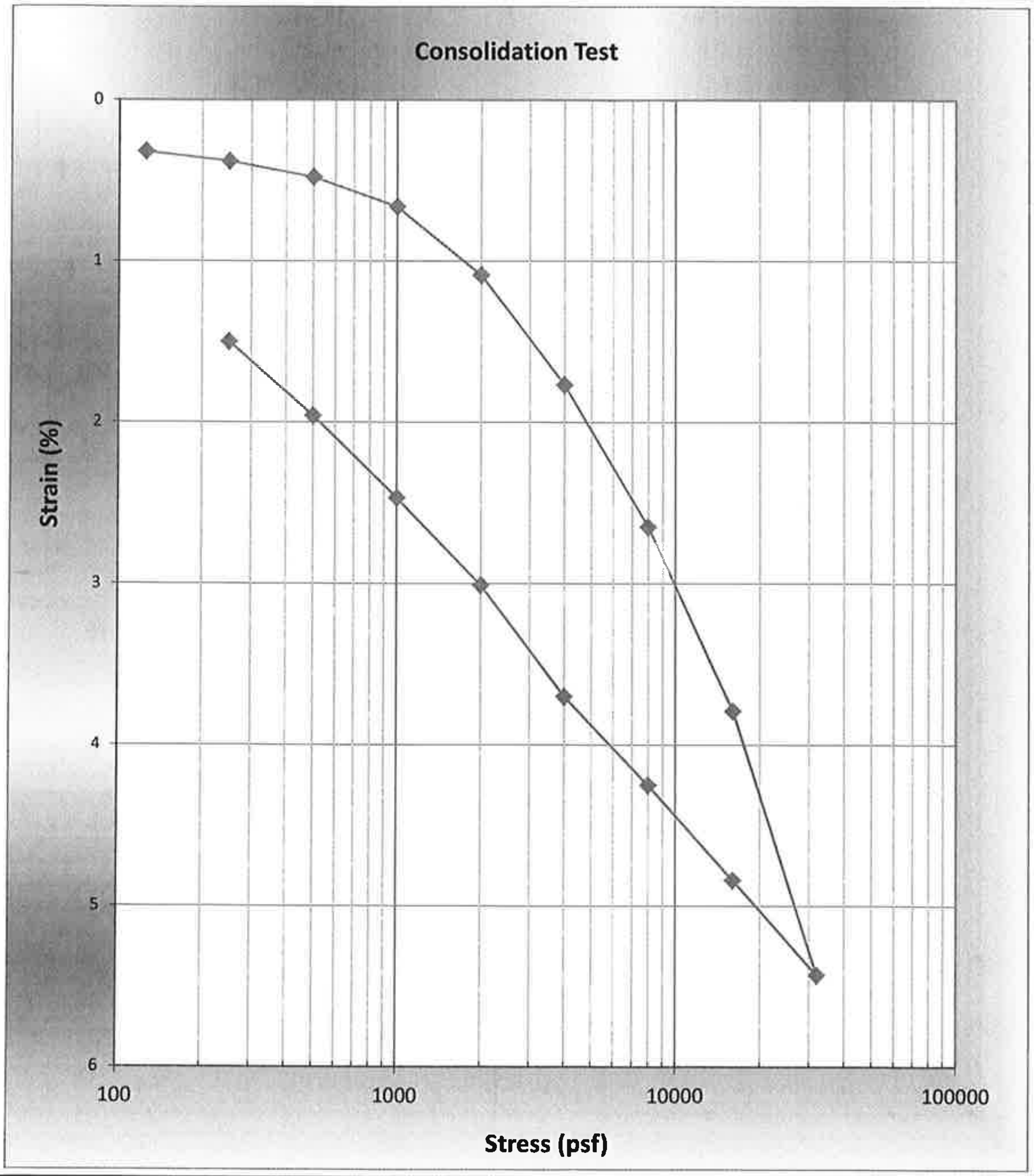
GRAIN SIZE - GINT STD US LAB.GDT - 1/22/18 12:46 - G:\DATA\18008 WILSTEAD STUDIO\SOILS\GINT\BORING LOGS.GPJ

Figure No.: C - 3

1-D CONSOLIDATION (ASTM D2435)

WILDING ENGINEERING
WWW.WILDINGENGINEERING.COM

Client: Judd Wilstead
Project Number: 18008
Project Name: Wilstead Studio
Project Location: Riverton, Utah



Sample Location:	B-2	Dry Density (pcf):	101.3	C_{re}:	0.020
Sample Depth (ft):	5	Moisture Content (%):	21.2	σ_p (psf):	5,000
Sample Description:	Lean CLAY	Liquid Limit:	46	C_c:	0.054
USCS Classification:	CL	Plastic Limit:	21		
Percent Collapse:	N/A	Percent Fines:	25		

Figure No.: C-4