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Geotechnical Investigation
SwimKids Riverton
2630 West 12600 South
Riverton, Utah

Prepared for:

Benefactor II, LLC

IGES Job No. 02269-001

April 21, 2016

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APPENDIX

1.0 EXECUTIVE SUMMARY

This report presents the results of a geotechnical investigation conducted for the proposed SwimKids development located at 2630 West 12600 South in Riverton, Utah. Based on the subsurface conditions encountered at the site, it is our opinion that the subject site is suitable for the proposed construction provided that the recommendations contained in this report are complied with. A brief summary of the recommendations is included below:

- Soils at the site consisted of 18 to 24 inches of Lean CLAY (CL) topsoil in TP-1, TP-2 and TP-3 and 3 feet of undocumented fill in TP-4. Native soils underlying the topsoil and undocumented fill consisted of 4 to 7 feet of Lean CLAY (CL) underlain by Silty Clayey SAND (SC-SM), Silty SAND (SM) and Silty GRAVEL (GM).
- Soils with a pinhole structure were observed in the upper 5 to 9 feet. The soils with a pinhole structure are considered potentially collapsible.
- Shallow spread or continuous wall footings constructed on suitable native soils that do not have a potential for wetting-induced collapse or on a zone of structural fill that extends to these soils as described herein may be proportioned utilizing a maximum net allowable bearing pressure of **2,000 pounds per square foot (psf)** for dead load plus live load conditions.
- Concrete slabs-on-grade should be constructed over at least 4 inches of compacted gravel over structural fill as described in the body of the report. The slab may be designed with a Modulus of Subgrade Reaction of **150 psi/inch**.
- Lateral earth pressure coefficients of 0.31, 0.47 and 3.25 are recommended for active, at-rest and passive conditions respectively; lateral earth pressure coefficients for seismic conditions are also presented in the body of the report (Section 6.5).
- Various pavement section design alternatives have been provided in Section 6.8 and are based on a CBR of 5.4 and anticipated traffic conditions. At a minimum, 12-inches of the native soil should be reworked and compacted at a moisture content that is at or above the optimum moisture content due to the presence of hydro-collapsible soils.
- The recommended rigid pavement section to be used in heavy traffic areas consists of 5 inches of Portland cement concrete over 8 inches of untreated road base over 12 inches of reworked, moisture conditioned and compacted native soils.

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

NOTE: The scope of services provided within this report is limited to the assessment of the subsurface conditions at the subject site. The executive summary is provided solely for purposes of overview and is not intended to replace the report of which it is part and should not be used separately from the report.

2.0 INTRODUCTION

2.1 PROJECT UNDERSTANDING AND DESCRIPTION

This report presents the results of a Geotechnical Investigation conducted for the proposed SwimKids development on approximately 1.26 acres located at 2630 West 12600 South in Riverton, Utah (Figure A-1, *Site Vicinity Map*). The majority of the site is vacant with no structures or development, however, a home, detached garage and shed are located in the southwest corner of the site. The project as planned will include the construction of a 1- to 2-story SwimKids facility that will include a swimming pool and classrooms as well as landscaping and a parking lot. Construction plans were not available for our review at the time this report was prepared; however, we anticipate that the structures will be metal- or wood-framed constructed on grade or partially below grade on conventional spread footings.

2.2 PURPOSE AND SCOPE OF WORK

This report presents the results of a geotechnical investigation conducted for the proposed development located at 2630 West 12600 South in Riverton, Utah. The purposes of this investigation were to assess the nature and engineering properties of the subsurface soils at the site and to provide recommendations for general site grading and design and construction of foundations, slabs-on-grade, and exterior concrete flatwork.

The scope of work completed for this study included a field investigation, infiltration testing, engineering analyses, laboratory testing and preparation of this report. Our services were performed in accordance with our proposal and signed authorization, dated March 18, 2015 and signed authorization.

The recommendations contained in this report are subject to the limitations presented in the **Limitations** section of this report (Section 7.1).

3.0 METHODS OF STUDY

3.1 FIELD INVESTIGATION

As a part of this investigation, subsurface soil conditions were explored by excavating four exploratory test pits to depths of 7 to 13 feet below existing site grade. The approximate locations of the explorations are shown on Figure A-2 (*Site Map*) in Appendix A. Exploration points were placed to provide information at pertinent locations at the site. Logs of the subsurface conditions as encountered in the test pit explorations were recorded at the time of exploration by a member of our technical staff and are included on Figures A-3 through A-6 in Appendix A. A *Key to Soil Symbols and Terminology* used in the boring logs is included as Figure A-7.

Test pits were completed using a rubber tired backhoe with a 30-inch wide bucket. Soil sampling was completed to collect representative samples of the various layers observed at the site. Disturbed samples were placed in plastic bags and relatively undisturbed soil samples were collected with the use of a 6-inch long brass tube attached to a hand sampler driven with a 2-lb sledge hammer. All samples were transported to our laboratory to evaluate the engineering properties of the various earth materials observed. The soils were classified according to the *Unified Soil Classification System* (USCS) by the Geotechnical Engineer. Classifications for the individual soil units are shown on the attached *Test Pit Logs*.

3.2 LABORATORY INVESTIGATION

Geotechnical laboratory tests were conducted on selected relatively undisturbed and bulk 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:

- In situ density and moisture content (ASTM D2216 & D2937)
- Atterberg Limits (ASTM D4318)
- No. 200 Sieve Wash (ASTM D1140)
- Particle-Size Distribution (ASTM D6913)
- One-dimensional collapse (ASTM D4546 Method B)
- Unconsolidated-Undrained Triaxial Compression (ASTM D2850)
- Modified Proctor - Maximum dry density and optimum moisture content (ASTM D698/D1557)
- California Bearing Ratio (CBR) (ASTM D1883)
- Corrosion Testing-sulfate and chloride concentrations, pH and resistivity (AASHTO T 288, T 289, ASTM D4327, and C1580)

The results of laboratory tests completed for this investigation are presented on the *Test Pit Logs* in Appendix A (Figures A-3 through A-4) and the test result figures presented in Appendix B.

3.3 ENGINEERING ANALYSIS

Engineering analyses were performed using soil data obtained from the laboratory test results and empirical correlations from material density, depositional characteristics and classifications. Analyses were performed using formulas, calculations and software that represent methods accepted by the geotechnical industry. These methods include settlement, bearing capacity, lateral earth pressures, trench stability and pavement design. Appropriate factors of safety were applied to the results consistent with industry standards and the accepted standard of care.

4.0 GENERALIZED SITE CONDITIONS

4.1 SURFACE CONDITIONS

At the time of our field work the majority of the site was covered with grass and weeds, a portion of the site was covered with the three structures mentioned previously and some of the site was covered with an asphalt paved entry and parking lot for a four-plex apartment to the east. The site is located at an elevation of approximately 4,505 feet above mean sea level and is generally flat.

4.2 SUBSURFACE CONDITIONS

Subsurface soil conditions were logged at the time of exploration; logs of the excavations are included in Appendix A as Figures A-3 through A-6. The soil and groundwater conditions encountered during our investigation are discussed below.

4.2.1 Soils

Based on our observations, the site is covered by 18 to 24 inches of topsoil that was loose, dark brown and moist in TP-1, TP-2 and TP-3 and 3 feet of undocumented fill in TP-4. Typical topsoil coverage is 24 inches; the uppermost 6 inches is rooted and very loose. Native soils underlying the topsoil and undocumented fill consisted of 4 to 7 feet of Lean CLAY (CL) underlain by Silty Clayey SAND (SC-SM), Silty SAND (SM) and Silty GRAVEL (GM). The clayey soils were typically medium stiff to stiff, moist and contained frequent fine pinholes, the granular soils were typically medium dense, dry to moist and brown in color. More detailed descriptions of these soil units and thicknesses are presented on the *Test Pit Logs* (Figures A-3 to A-6).

4.2.2 Groundwater

Groundwater was not encountered in any of the test pits completed for our investigation. Seasonal fluctuations in precipitation, surface runoff from adjacent properties, or other on or offsite sources may increase moisture conditions; groundwater conditions can be expected to rise several feet seasonally depending on the time of year. However, based on our field investigation, we anticipate that groundwater will not impact the proposed construction.

4.2.3 Strength of Earth Materials

One representative sample of the near-surface clayey soils were tested to evaluate the inherent strength properties of site soils. An unconsolidated-undrained triaxial compression test was completed on a relatively undisturbed sample from TP-4 at a depth of 3½ feet; the resulting shear strength was approximately 5,162 psf at failure; the results of this test are presented in Appendix B on Figure B-8.

4.2.4 Wetting-Induced Collapsible Soils

Wetting-induced collapse is a phenomenon where undisturbed native soils under increased loading can exhibit volumetric strain and consolidation upon wetting. Collapsible soils can cause differential settling of structures and roadways. Collapsible soils do not necessarily preclude development and can be mitigated by over-excavating porous, potentially collapsible soils and replacing with engineered fill and by controlling surface drainage and runoff. Collapsible soils are typically characterized by a pinhole structure and relatively light in-situ density.

Pinholes were prevalent in the clayey soils and shallower sandy soils at the site and observed in the all of the test pits. Laboratory testing has indicated moderately low dry densities ranging between 75 and 82 pcf; low dry densities are often a characteristic of collapsible soils.

Collapse testing was completed on three samples collected at representative locations and depths as part of this investigation. The test results indicate a collapse potential that ranged from approximately 2% to 7% indicating a moderate to moderately high potential for wetting-induced collapse across the site. Detailed results of the collapse testing are provided in Appendix B (Figures B-5 to B-7). A summary of the results of the collapse testing are shown below.

Table 4.2.4.1 – Collapse Test Results

Conditions: Collapse Stress = 2,000 psf		
TP	**Sample Depth (feet)**	**% Collapse**
1	4.5	7.0
2	3	1.7
2	6.5	4.9

IGES has provided moisture protection and surface drainage recommendations in Section 6.7 to help reduce the potential for collapse related settlement as well as foundation recommendations in Section 6.3.

4.3 INFILTRATION TESTING

An infiltration test was completed in the bottom of TP-3. It should be noted that the tests were performed using clean water. Sediment collected from runoff may reduce the actual infiltration rate to be slower than the predicted rate. This and other field conditions should be considered and an appropriate factor of safety should be applied to the rates provided. Results of the infiltration testing are summarized in the following tables.

Table 4.3.1 - Infiltration Test Summary – TP-3

Conditions: Test Depth ~ 5 Feet, Starting Head = ~8 inches, Hole Diameter = 4.5 inches, Hole Depth = 8 inches, Presoak Time = 20 minutes				
Time Difference (minutes)	**Depth Difference (inches)**	**Infiltration Rate**		**Comments**
		(min/inch)	**(in/hour)**	
1.83	0.5	3.7	16	Intermediate Reading
4.55	1	4.6	13	Intermediate Reading
2.95	0.5	5.9	10	Intermediate Reading
2.93	0.5	5.9	10	Intermediate Reading
2.97	0.5	**5.9**	**10**	**Final Reading**

5.0 GEOLOGIC CONDITIONS

5.1 GEOLOGIC SETTING

The site is located at an elevation of approximately 4,505 feet in the southern portion of Salt Lake Valley. The Salt Lake Valley is a deep, sediment-filled structural basin of Cenozoic age flanked by uplifted mountain blocks (Hintze, 1980) with the eastern foothills largely being created as a result of glacial and canyon outwash as well as Lake Bonneville processes. The Wasatch Range is the easternmost expression of pronounced Basin and Range extension in north-central Utah. The Oquirrh Range marks the westernmost expression of the Salt Lake Valley in its southern extremes and in the north is bordered to the west by the Great Salt Lake.

The near-surface geology of the Salt Lake Valley is dominated by sediments, which were deposited within the last 30,000 years by Lake Bonneville (Scott et al., 1983). The lacustrine sediments near the mountain front consist mostly of gravel and sand. As the lake receded, streams began to incise large deltas which had previously formed at the mouths of major canyons along the Wasatch Range. The eroded material was deposited in shallow lakes and marshes in the basin and in a series of recessional deltas and alluvial fans. Sediments toward the center of the valley are predominately deep-water deposits of clay, silt and fine sand. However, these deep-water deposits are in places covered by a thin post-Bonneville alluvial cover. Most surficial deposits along the northern Wasatch fault zone were deposited during the Bonneville Lake Cycle that was the last cycle of Lake Bonneville between approximately 32 to 10 ka (thousands of years ago) and in the Holocene (10 ka).

Surface sediments at the site are mapped as fine-grained lacustrine deposits (Qlf) described as transgressive and regressive, deep water sediments, brown, dark-brown, grayish brown and gray calcareous laminated silt, clayey silt and sandy silt from the late Pleistocene associated with Lake Bonneville (Davis, 2000).

5.2 SEISMICITY AND FAULTING

There are no known active faults that pass under or immediately adjacent to the site (Hecker, 1993; Black et al, 2003). An active fault is defined as a fault displaying evidence of movement during Holocene time (eleven thousand years ago to the present). The site is located approximately 6.2 miles west of the Salt Lake City Segment of the Wasatch fault zone which is mapped along the western flank of the Wasatch Mountains. The Salt Lake City segment of the Wasatch Fault Zone was reportedly last active approximately 1,100 years ago and has a recurrence interval of approximately 1,300 years. The site is also mapped approximately 9.5 miles south of the Taylorsville Fault portion of the West Valley fault zone, a north-south trending series of faults that are mapped within the middle of the

Salt Lake Valley. The last event reportedly occurred on the West Valley Fault Zone <12,000 years ago, and has a recurrence interval of 6,000 to 12,000 years. The site is also located approximately 13 miles east by southeast of the Oquirrh Fault Zone. Analyses of ground shaking hazard along the Wasatch Front suggests that the Wasatch fault zone is the single greatest contributor to the seismic hazard in the region.

Seismic hazard maps depicting probabilistic ground motion and spectral response have been developed for the United States by the U.S. Geological Survey as part of NEHRP/NSHMP (Frankel et al, 1996). These maps have been incorporated into both *NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures* (FEMA, 1997) and the *International Building Code* (IBC) (International Code Council, 2012).

To account for site effects, site coefficients that vary with the magnitude of spectral acceleration and *Site Class* are used. Site Class is a parameter that accounts for site amplification effects of soft soils and is based on the average shear wave velocity of the upper 100 feet. Based on our observations of native soils at the site from the upper 10 to 13 feet, it is our judgment that the subject site is appropriately classified as Site Class D (*Stiff Soil*). The spectral accelerations are calculated based on design maps and the site's approximate latitude and longitude of 40.5226° and -111.9565° respectively. Based on IBC criteria, the short-period (F_a) and long-period (F_v) site coefficients are 1.0 and 1.574, respectively. The *Spectral Response Accelerations* are presented in Table 5.2.1; a summary of the *Design Maps* analysis is presented in Appendix C. The *peak ground acceleration* (PGA) may be taken as 0.51g.

Table 5.2.1: Short and 1-Second Period Spectral Accelerations

Parameter	**Short Period (0.2 sec)**	**Long Period (1.0 sec)**
MCE Spectral Response Acceleration Site Class B (g)	$S_S = 1.285$	$S_1 = 0.426$
MCE Spectral Response Acceleration Site Class D (g)	$S_{MS} = 1.285$	$S_{M1} = 0.671$
Design Spectral Response Acceleration (g)	$S_{DS} = 0.856$	$S_{D1} = 0.447$

5.3 OTHER GEOLOGIC HAZARDS

Geologic hazards and conditions can be defined as naturally occurring geologic conditions or processes that could present a danger to human life and property or result in impacts to conventional construction procedures. These hazards and conditions must be considered before development of the site. There are several hazards and conditions in addition to seismicity and faulting that if present at a site, should be considered in the design of critical and essential facilities. The other geologic hazard considered for this site is liquefaction.

5.3.1 Liquefaction

Liquefaction is a phenomenon whereby loose, saturated, granular soil deposits lose a significant portion of their shear strength due to excess pore water pressure buildup resulting from dynamic loading, such as that caused by an earthquake. Among other effects, liquefaction can result in densification of such deposits causing settlements of overlying layers after an earthquake as excess pore water pressures are dissipated. The primary factors affecting liquefaction potential of a soil deposit are: (1) level and duration of seismic ground motions; (2) soil type and consistency; and (3) depth to groundwater.

Referring to the *Liquefaction-Potential Map for Salt Lake County, Utah* published by the Utah Geological Survey, the site is located within an area currently designated as "very low" for liquefaction potential. The upper 13 feet are not considered liquefiable based on our field observations, laboratory testing and a depth to groundwater greater than 13 feet. A liquefaction assessment which would include several boring and/or cone penetrometer testing (CPT) explorations to a depth of at least 50 feet was not performed and is beyond the proposed scope of work for this project. However, if this scope of work is desired, IGES can complete a liquefaction assessment at your request.

6.0 ENGINEERING CONCLUSIONS AND RECOMMENDATIONS

6.1 GENERAL CONCLUSIONS

Based on the subsurface conditions encountered at the site, all collapsible soils should be removed from beneath all footings. The native Lean CLAY (CL) and Silty Clayey SAND (SC-SM) soils have a moderate to moderately high collapse potential; these soils were typically underlain by either Poorly Graded GRAVEL (GP-GM), Silty GRAVEL (GM) or Silty SAND (SM) soils that do not have a wetting induced collapse potential. The soils with a collapse potential are not suitable for support of footings, suitable soils include either Poorly Graded GRAVEL (GP-GM), Silty GRAVEL (GM) or Silty SAND (SM) soils that do not have pinholes. All footings, including footings for basements, garages and any other structure, must be constructed on suitable native soils or on a zone of structural fill that extends to suitable native soils. We recommend that IGES be on site at key points during construction to see that the recommendations in this report are implemented.

The following sub-sections present our recommendations for general site grading, pavement design, design of foundations, slabs-on-grade, lateral earth pressures, moisture protection and preliminary soil corrosion.

6.2 EARTHWORK

Prior to the placement of foundations, general site grading is recommended to provide proper support for foundations, exterior concrete flatwork, and concrete slabs-on-grade. This will include removal of all collapsible soils beneath footings and a 12-inch thick zone below all roadways, parking lots, curb and gutter sections and concrete slab on grade construction. Site grading is also recommended to provide proper drainage and moisture control on the subject property and to aid in preventing differential settlement of foundations as a result of variations in subgrade conditions.

6.2.1 General Site Preparation

As discussed previously, soils with a moderate to moderately high potential for collapse upon increased moisture and loading conditions are prevalent at the site. IGES recommends that these soils be removed beneath footings. Soils beneath pavement sections and concrete slabs should be over-excavated and replaced with structural fill as recommended in subsequent sections of this report.

An IGES representative should observe the site preparation and grading operations to assess whether the recommendations presented in this report have been complied with.

6.2.2 Excavation Stability

The contractor is responsible for site safety, including all temporary slopes and trenches excavated at the site and design of any required temporary shoring. The contractor is responsible for providing the "competent person" required by OSHA standards to evaluate soil conditions. Based on our observations and laboratory testing, the onsite native fine-grained (silt and clay) soils classify as OSHA Type A soils. Close coordination between the competent person and IGES, Inc. should be maintained to facilitate construction while providing safe excavations.

Based on Occupational Safety and Health (OSHA) guidelines for excavation safety, trenches with vertical walls up to 5 feet in depth may be occupied. Where very moist soil conditions or groundwater is encountered, or when the trench is deeper than 5 feet, we recommend a trench-shield or shoring be used as a protective system to workers in the trench. Sloping of the sides at 3/4 horizontal to one vertical (3/4H:1V) (34 degrees) may be used as an alternative to shoring or shielding. However, the presence of very moist soils, granular soils (sand or gravel) or undocumented fill soils may require the slope walls to be further flattened to increase the safety to workers on site at the time of construction.

The contractor is ultimately responsible for trench and site safety. Pertinent OSHA requirements should be met to provide a safe work environment. If site specific conditions arise that require engineering analysis in accordance with OSHA regulations, IGES can respond and provide recommendations as needed.

6.2.3 Structural Fill and Compaction

All fill placed for the support of structures, flatwork or pavements, should consist of structural fill. Structural fill may consist of on-site soils or an approved imported material. If native soils that include pinholes are used as structural fill, they must be thoroughly processed, pulverized and moisture conditioned beyond the optimum moisture content (OMC) to destroy and remove the pinhole structure prior to being used as structural fill. If needed, imported soil used as structural fill should be a relatively well-graded granular soil with a maximum of 50 percent passing the No. 4 sieve and a maximum fines content (minus No.200 mesh sieve) of 20 percent. Structural fill should be relatively free of vegetation and debris, and contain no materials larger than 4 inches in nominal size (6 inches in greatest dimension). All structural fill soils should be approved by the geotechnical engineer prior to placement.

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- to medium-duty rollers, and maximum 10-inch loose lifts if compacted by heavy duty compaction equipment that is capable of efficiently compacting the entire thickness of the lift. Thinner lifts may be necessary to achieve proper compaction. We recommend that all structural fill be compacted on a horizontal plane, unless otherwise approved by IGES. Structural fill placed beneath footings and pavements should be compacted to at least 95 percent of the maximum dry density (MDD) as determined by ASTM D-1557. The moisture content should be at or slightly above the OMC for all structural fill – compacting dry of optimum is discouraged and could result in excessive settlement. Any imported fill materials should be approved prior to importing. Also, prior to placing any fill, the excavations should be observed by IGES to confirm that unsuitable materials have been removed. In addition, proper grading should precede placement of fill, as described in the General Site Preparation and Grading subsection of this report.

Backfill around foundation walls should be placed in maximum 12-inch loose lifts and compacted to approximately 90 to 95 percent of the MDD at a moisture content that is within 2 percent of OMC as determined by ASTM D-1557. Failure to properly water-condition and compact basement wall backfill may result in settlements of up to several inches should the backfill become wetted. Only small compaction equipment should be used near basement walls.

6.2.4 Utility Trench Fill and Compaction

All utility trenches backfilled below footings, pavement sections, concrete flatwork, curb and gutter and sidewalks, should be backfilled with structural fill that is at or slightly above the OMC when placed and compacted to at least 95 percent of the MDD as determined by ASTM D-1557. All other trenches in landscape areas should be backfilled and compacted to a minimum of approximately 90 percent of the MDD (ASTM D-1557). Utility trenches should be backfilled with structural fill as discussed in Section 6.2.3 of this report. Prior to backfilling the trench, pipes should be bedded in and covered with a uniform granular material that has a Sand Equivalent (SE) of 30 or greater. Pipe bedding should *not* be water-densified in-place (jetting). Alternatively, pipe bedding and shading may consist of clean ¾-inch gravel, which generally does not require compaction.

Specifications from governing authorities having their own precedence for backfill and compaction should be followed where applicable.

6.3 FOUNDATIONS

Footings should be established *entirely* on suitable native soils or *entirely* on structural fill founded on suitable native soils. Shallow spread or continuous wall footings constructed on suitable native soils that do not have a potential for wetting-induced collapse or on structural fill that extends to these soils as described previously may be proportioned utilizing a maximum net allowable bearing pressure of **2,000 pounds per square foot (psf)** for dead load plus live load conditions. A one-third increase may be used for transient wind and seismic loads.

Suitable soils as discussed in Section 6.1 of this report include Poorly Graded GRAVEL (GP-GM), Silty GRAVEL (GM) or Silty SAND (SM) soils that do not have a wetting induced collapse potential.

All foundations exposed to the full effects of frost should be established at a minimum depth of 30 inches below the lowest adjacent final grade. Interior footings, not subjected to the full effects of frost (i.e., a continuously heated structure), may be established at higher elevations, however, a minimum depth of embedment of 12 inches is recommended for confinement purposes. The minimum recommended footing width is 20 inches for continuous wall footings and 30 inches for isolated spread footings.

IGES should observe and assess all of the footing excavations prior to footing placement to assess compliance with our recommendations. IGES recommends that an exploratory test pit be excavated on each lot, or between lots, to assess the depth to suitable soils prior to beginning the foundation excavation.

6.4 SETTLEMENT

Settlements of properly designed and constructed conventional foundations, founded as described above, are anticipated to be on the order of 1 inch or less. Differential settlement is expected to be half of total settlement over a distance of 30 feet. If soils with the potential for wetting-induced collapse are left in place below footings total and differential settlements could exceed 4 inches.

6.5 EARTH PRESSURES AND LATERAL RESISTANCE

Lateral forces acting on 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 soils. In determining the frictional resistance, a coefficient of friction of 0.41 should be used for native granular soils or structural fill.

Ultimate lateral earth pressures from natural soils and backfill acting against vertical retaining walls and buried structures may be computed from the lateral pressure coefficients or equivalent fluid densities presented in the following table.

Table 6.5.1 - Lateral Pressure Coefficients and Equivalent Fluid Densities

Earth Pressure Condition	Lateral Pressure Coefficient (symbol)	Equivalent Fluid Density (pounds per cubic foot)
Active*	0.31 (k_a)	40
At-rest**	0.47 (ko)	60
Passive*	3.25 (k_p)	15
Combined (static + seismic) Active***	0.83 (k_{AE})	105
Combined (static + seismic) Passive***	2.35 (k_{PE})	300

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

These coefficients and densities assume level, on-site native soil 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 water or sloping backfill is present, IGES 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. Values of 2.0 and 1.5 for overturning and sliding, respectively, are typically used.

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.

6.6 CONCRETE SLAB-ON-GRADE CONSTRUCTION

Concrete slabs-on-grade should be constructed over at least 4 inches of compacted free-draining gravel over a minimum of 12 inches of structural fill that may include reworked native soils processed as described in the body of the report (Section 6.2.3). The slab may be designed with a Modulus of Subgrade Reaction of **150 psi/inch**. The gravel should consist of road base or clean drain rock with a ¾-inch maximum particle size and no more than 12 percent fines passing the No. 200 mesh sieve. The gravel layer should be compacted to at least 95 percent of the MDD of the modified proctor if road base is used or vibrated in place for densification until tight and relatively unyielding if drain rock gravel is used. The maximum load on the floor slab should not exceed 300 psf; greater loads would require additional subgrade preparation and additional structural fill.

All concrete slabs should be designed to minimize cracking as a result of shrinkage. Consideration should be given to reinforcing the slab with a welded wire fabric, re-bar, or fiber mesh. Slab reinforcement should be designed by the structural engineer; however, as a minimum, slab reinforcement should consist of 4"x4" W4.0xW4.0 welded wire mesh within the middle third of the slab.

Our experience indicates that use of reinforcement in slabs and foundations can generally reduce the potential for drying and shrinkage cracking. However, some cracking can be expected as the concrete cures. Minor cracking is considered normal; however, it is often aggravated by a high water/cement ratio, high concrete temperature at the time of placement, small nominal aggregate size, and rapid moisture loss due to hot, dry, and/or windy weather conditions during placement and curing. Cracking due to temperature and moisture fluctuations can also be expected. The use of low slump concrete can reduce the potential for shrinkage cracking; saw cuts in the concrete at strategic locations can help to control and reduce undesirable shrinkage cracks.

We recommend a **minimum slab thickness of 4 inches** and the concrete be tested to assess that the slump and/or air content is in compliance with the plans and specifications. We recommend that concrete be placed in general accordance with the requirements of the American Concrete Institute (ACI).

A moisture barrier (vapor retarder) consisting of 10-mil thick Visqueen (or equivalent) plastic sheeting should be placed below slabs-on-grade where moisture-sensitive floor coverings or equipment is planned. Prior to placing this moisture barrier, any objects that could puncture it, such as protruding gravel or rocks, should be removed from the building pad. Alternatively, the subgrade may be covered with 2 inches of clean sand.

6.7 MOISTURE PROTECTION AND SURFACE DRAINAGE

Over wetting the soils prior to or during construction may result in increased softening and pumping, causing equipment mobility problems and difficulty in achieving compaction. Precautions should be taken during and after construction to minimize the potential for saturation of foundation soils beneath footings, exterior slab on grade construction, sidewalks and roadways. We recommend the following be implemented after construction is complete:

- If sumps or detention/retention basins are used at the site we recommend that they be placed as far away from structures, sidewalks and pavements as possible.

- We recommend that roof runoff devices be installed to direct all roof runoff a minimum of 10 feet away from structures or beyond the limits of backfill, which ever distance is greater.
- The grade within 10 feet of the structures should be sloped a minimum of 5% away from the structure. Alternatively, if the surface within 10 feet of the structure consists of a hard surface such as concrete or asphalt, the minimum recommended slope would be 2% away from the structure.
- No pressurized irrigation lines should be placed within 5 feet of the structures and we recommend the area within 5 feet of the structure be hardscaped, xeriscaped or planted with drought tolerant plants that do not require irrigation.
- Prior to backfilling trenches that have been excavated for utilities or other purposes, we recommend that a clay barrier or other relatively impermeable barrier be constructed to minimize water from flowing towards structures. The clay barrier could include flowable fill or compacted fine-grained soils such as clay with a high percentage of fines (a minimum of 85% passing the #200 sieve). The relatively impermeable barrier should be a minimum of 18 inches thick and extend 12 inches beyond the edge of the utility excavation.
- Compact backfill against the foundation walls in maximum 12-inch loose lifts to approximately 90 to 95 percent of the MDD and at a moisture content that is within 2 percent of the OMC (ASTM D1557); native soils may be used as backfill.

6.8 ASPHALT CONCRETE PAVEMENT DESIGN

Based on soil classifications and a laboratory obtained CBR value of 5.4 for the native soil tested, the near-surface soils are expected to provide fair pavement support. IGES has prepared various pavement section alternatives be used to support anticipated traffic conditions not exceeding 110,000 equivalent single axle loads (ESALs) for the parking lot based on the information contained herein.

Table 6.8.1 - Flexible (Asphalt) Pavement Section – Parking Lot

Pavement Section Options	Asphalt Concrete (in.)	Untreated Base Course (in.)	Granular Borrow (in.)	Reworked Native Soil/Undocumented Fill Soils if Present (in.)
Option 1	3	6	7	12/24
Option 2	3	11	-	
Option 3	3.5	9	-	

This pavement design is based on the assumption that the upper 12 inches (or 24-inch zone of reworked soils if undocumented fill soils are present) of the native soils beneath all pavement sections will be removed and/or reworked in place due to the presence of hydro-collapsible soils and compacted to at least 95 percent of the MDD with the moisture content at or above OMC as determined by ASTM D-1557. Asphalt has been assumed to be a high stability plant mix, base course material should be composed of crushed stone with a minimum CBR of 70 and granular borrow should consist of a pit-run type of material with a minimum CBR of 30. Asphalt should be compacted to a minimum density of 96% of the Marshall value; base course and granular borrow should be compacted to at least 95% of the MDD as determined by ASTM D-1557.

Alternatively, a geotextile could be incorporated to reduce the overall thickness of the pavement section by using one of the options shown below using the TenCate Mirafi ® RS380i Geosynthetic placed over the native soils, which have been prepared as recommended; the pavement section alternatives below were developed for the information and assumptions stated previously.

Table 6.8.2 - Flexible (Asphalt) Pavement Section with TenCate Mirafi ® RS380i

Alternate Pavement Sections	Asphalt Concrete (in.)	Untreated Base Course (in.)	Reworked Native Soil/Undocumented Fill Soils if Present (in.)
Option 4	3	7	12/24

If this option is selected, the woven geotextile should be placed over a 12-inch zone of reworked native soils (or 24-inch zone of reworked soils if undocumented fill soils are present) in accordance with manufacturers recommendations. Based on the laboratory obtained CBR value of 5.4, the reinforced pavement section is the preferred option.

It is our experience that pavement in areas where vehicles frequently turn around, backup, or load and unload, including round-a-bouts and exit and entrance areas, 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 6.8.3 - Rigid Pavement Section – Parking Lot

Concrete (in.)	Untreated Base Course (in.)	Reworked Native Soil/Undocumented Fill Soils if Present (in.)
5	8	12/24

Concrete should consist of a low slump, low water cement ratio mix with a minimum 28-day compressive strength of 4,000 psi. The base course should be compacted to at least 95% of the MDD as determined by ASTM D-1557.

If traffic conditions vary significantly from our stated assumptions, IGES should be contacted so we can modify our pavement design parameters accordingly. Specifically, if the traffic counts are significantly higher or lower, IGES 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, the owner should anticipate a reduced life and increased maintenance in some areas of the property.

The pavement section thicknesses above assume that there is no mixing over time between the road base/granular borrow and the softer native subgrade below. In order to prevent mixing or fines migration, and thereby prolong the life of the pavement section, we recommend that the owner give consideration to placing a non-woven filter fabric between the native soils and the road base/granular borrow. We recommend that a product such as TenCate Mirafi 160N, or an IGES-approved equivalent be used for separation. If the option from Table 6.8.2 is used (Option 4) filter fabric is not required since RS380i also acts to separate in addition to structural reinforcement.

6.9 Preliminary Soil Corrosion Potential

IGES did not complete any testing to evaluate the corrosion potential of concrete in contact with onsite native soil. However, we have provided the following recommendations based on our experience and laboratory testing for similar types of soils in the general vicinity of the project site. We anticipate that the potential for sulfate attack will be low and that a conventional Type I/II cement may be used for all concrete in contact with site soils. We also anticipate that the native fine-grained clayey soils will be *very corrosive* when in contact with ferrous metal. IGES recommends that a qualified corrosion engineer be consulted to provide an assessment of any metal such as water lines, reinforcing steel, valves and similar improvements in direct contact with native soils.

6.10 CONSTRUCTION CONSIDERATIONS

6.10.1 Collapsible Soils

Collapsible soils are typically identified in the field by a porous, open soil structure ('pinholes'), relatively low moisture content, and low in-situ dry density. Based on our laboratory tests of onsite soils and direct observation, clayey and sandy soils with a moderate to moderately high potential for hydro-collapse exist on site. All footings, for both the garage and basement, should bear entirely on uniform, competent native granular soils (sand and gravel) or entirely on structural fill with uniform thickness extending to suitable native granular soils.

Prior to placement of footings, an IGES representative should assess the foundation subgrade for the presence of potentially collapsible soils. If particularly adverse soil conditions are identified (porous soils, low dry unit weight), additional over-excavation may be necessary, depending on the extent and severity of the problematic soils.

6.10.2 Structural Fill

The prevailing clayey soils identified across the site are suitable for use as structural fill; however, the Contractor and Owner should be aware that properly moisture-conditioning and compacting clay soils is often challenging and time-consuming. If structural fill is needed, the Owner and/or Contractor may wish to consider importing a more suitable granular material for use as structural fill. IGES should approve any borrow source prior to import of structural fill.

7.0 CLOSURE

7.1 LIMITATIONS

The recommendations contained in this report are based on our limited field exploration, laboratory testing, and understanding of the proposed construction. The subsurface data used in the preparation of this report were obtained from the explorations made for this investigation. It is possible that variations in the soil and groundwater conditions could exist between 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, IGES should be notified.

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

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

7.2 ADDITIONAL SERVICES

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

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

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

We appreciate the opportunity to be of service on this project. Should you have any questions regarding the report or wish to discuss additional services, please do not hesitate to contact us at your convenience at (801) 748-4044.

8.0 REFERENCES CITED

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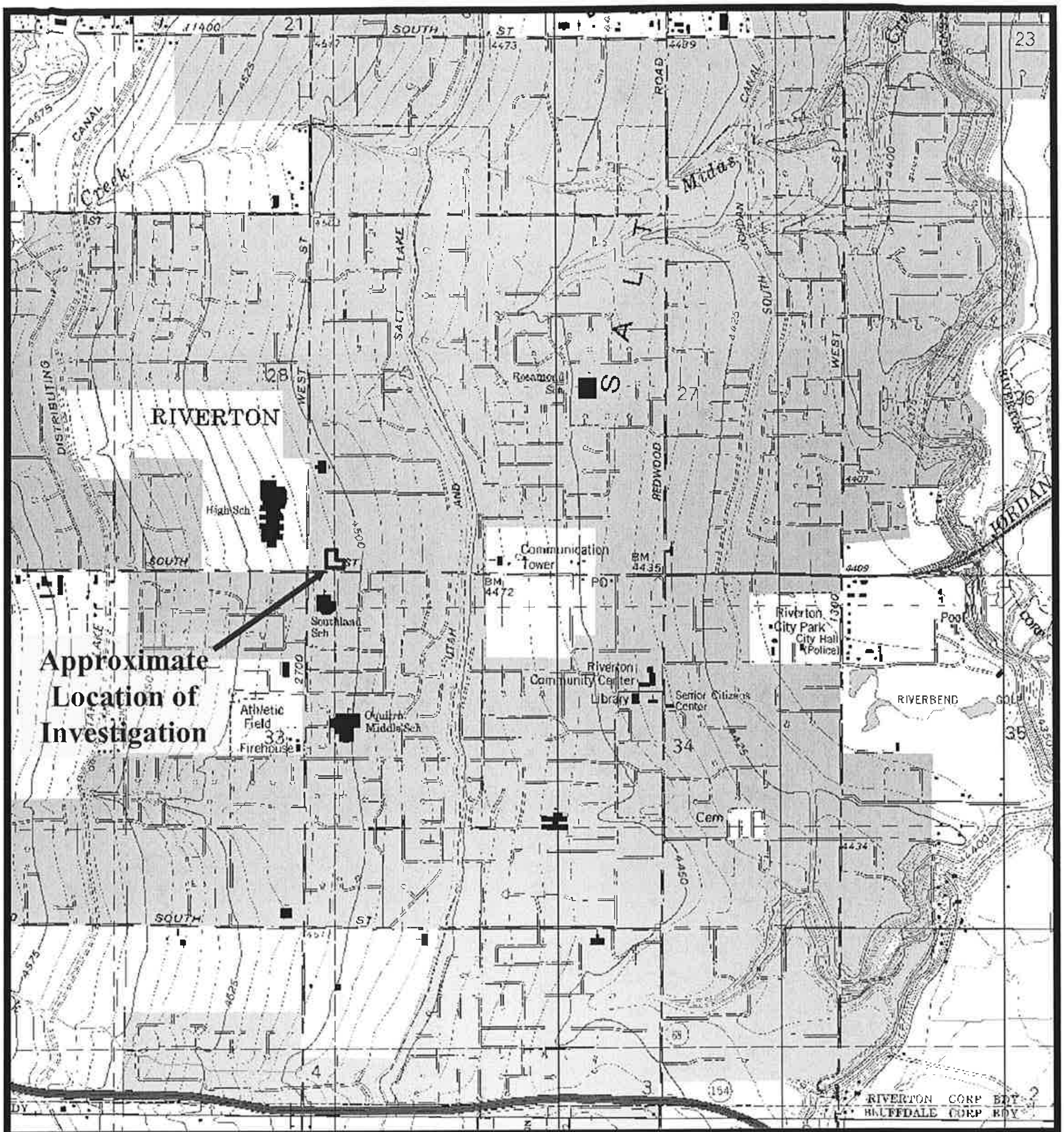
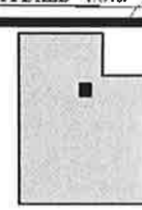
United States Geological Survey, Midvale, Utah Topographic Quadrangle Map 7.5 Minute Series.

APPENDIX A



Base Maps:
USGS Midvale
7.5-Minute Quadrangle Topographic Map

0' 1000' 2000'
SCALE 1:24,000



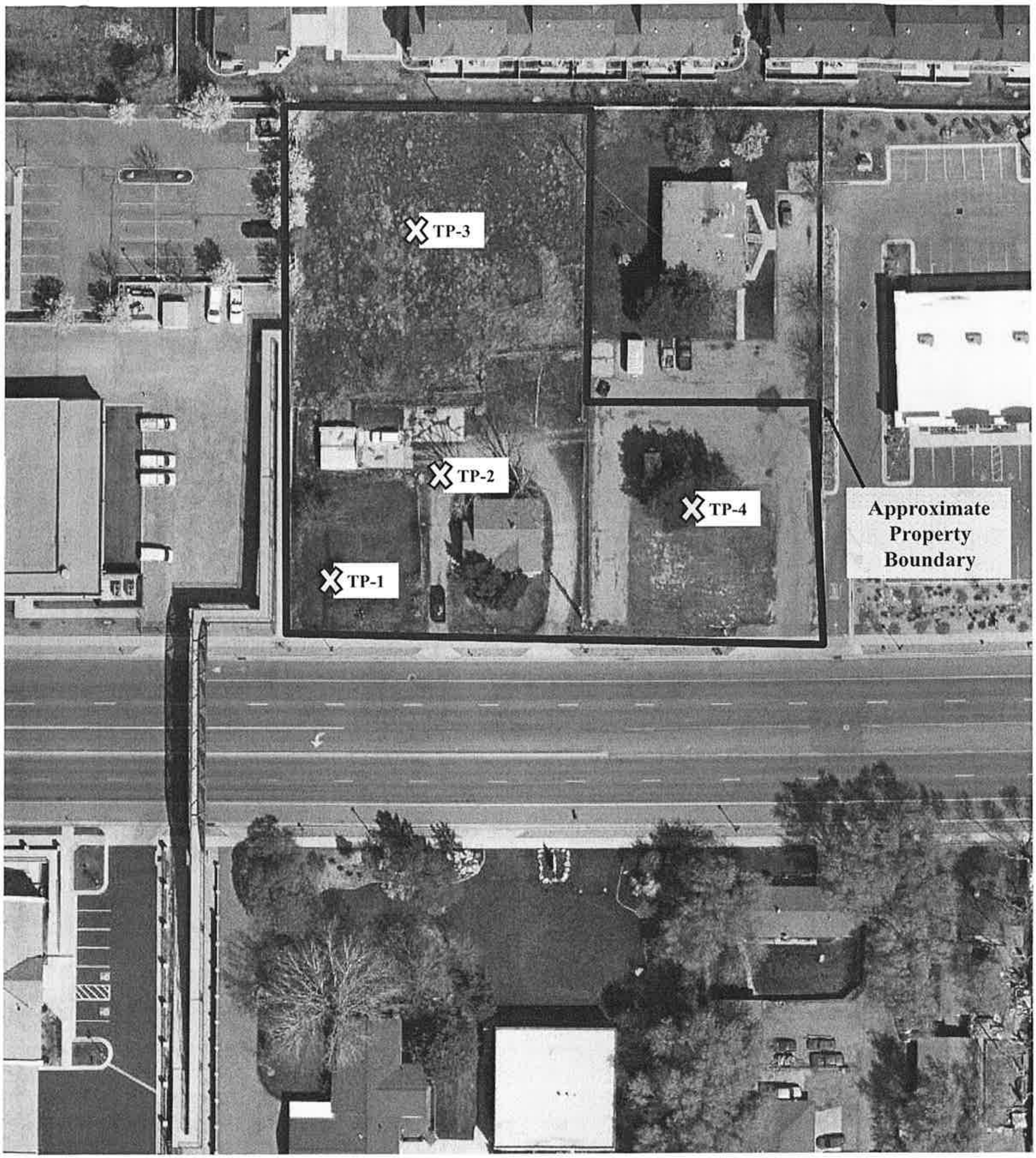
MAP LOCATION

IGES®
Project Number 02269-001

Geotechnical Investigation
SwimKids Riverton
2630 West 12600 South
Riverton, Utah

SITE VICINITY MAP

Figure
A-1



TP-3 Approximate Location of Test Pit

IGES®
Project Number – 02269-001

Geotechnical Investigation
SwimKids Riverton
2630 West 12600 South
Riverton, Utah

Aerial Photography:
July 3, 2012

SITE MAP

Figure
A-2

DATE	
STARTED:	3/29/16
COMPLETED:	3/29/16
BACKFILLED:	3/29/16

Geotechnical Investigation
SwimKids Riverton
2630 West 12600 South
Riverton, Utah

Project Number 02269-001

IGES Rep: SL
Rig Type: Backhoe (JCB 214S)

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

LOCATION
LATITUDE | LONGITUDE | ELEVATION (ft)

DEPTH METERS	DEPTH FEET	SAMPLES	WATER LEVEL	GRAPHICAL LOG	UNIFIED SOIL CLASSIFICATION	MATERIAL DESCRIPTION	Dry Density(pcf)	Moisture Content %	Percent minus 200	Liquid Limit	Plasticity Index	Moisture Content and Atterberg Limits (Plastic Limit, Moisture Content, Liquid Limit; 10 20 30 40 50 60 70 80 90)
0	0					Topsoil - Lean CLAY - medium stiff, moist, dark brown, frequent medium roots						
					CL	Native - Lean CLAY - medium stiff, dry to moist, light brown, frequent medium pinholes						
1												
	5	3" O.D. thin-walled					79.9	11.4				
		Grab				- frequent fine pinholes		12.9		35	16	
					CL	Sandy Lean CLAY - medium stiff, dry, light brown, frequent fine pinholes, slightly iron staining						
2												
		Grab						6.8	51.2			
					GM	Silty GRAVEL with sand and some cobbles - medium dense, dry to moist, brown, gravel and cobbles are sub-angular, up to 4 inches in diameter with 1/2 to 2 inches typical						
3	10											
4						No groundwater encountered Bottom of test pit @ 13 Feet						

IGES

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

WATER LEVEL
▼- MEASURED
▽- ESTIMATED

NOTES:

Figure A-3

DATE	
STARTED:	3/29/16
COMPLETED:	3/29/16
BACKFILLED:	3/29/16

Geotechnical Investigation
SwimKids Riverton
2630 West 12600 South
Riverton, Utah

Project Number 02269-001

IGES Rep: SL

Rig Type: Backhoe (JCB 214S)

TEST PIT NO: **TP-2**

Sheet 1 of 1

LOCATION: LATITUDE | LONGITUDE | ELEVATION (ft)

Depth (Meters)	Depth (Feet)	Samples	Water Level	Unified Soil Classification	Material Description	Dry Density (pcf)	Moisture Content %	Percent minus 200	Liquid Limit	Plasticity Index
0	0				Topsoil - Lean CLAY - medium stiff, moist, dark brown, frequent medium roots					
				CL	Native - Lean CLAY - medium stiff to stiff, moist, gray, frequent fine pinholes					
1		3" O.D. Thin-Walled Hand Sampler				75.4	29.5			
		Grab Sample					31.1		47	21
	5									
				SC-SM	Silty, Clayey SAND - medium dense, dry to moist, reddish brown, occasional fine pinholes, slightly iron staining					
2		3" O.D. Thin-Walled Hand Sampler				78.7	13.0			
				GP-GM	Poorly Graded GRAVEL with silt, sand and some cobbles - medium dense, dry to moist, light brown, gravel and cobbles are sub-angular, up to 4 inches in diameter with 1/2 to 3 inches typical					
3	10	Grab Sample					3.6	8.2		
				SM	Silty SAND with some gravel - medium dense, moist, reddish brown, no pinholes observed					
4										
					No groundwater encountered					
					Bottom of test pit @ 12 Feet					

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

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

WATER LEVEL
- ▼ - MEASURED
- ▽ - ESTIMATED

NOTES:

Figure A-4

IGES

DATE			
STARTED:	3/29/16		
COMPLETED:	3/29/16		
BACKFILLED:	3/29/16		

Geotechnical Investigation
SwimKids Riverton
2630 West 12600 South
Riverton, Utah

Project Number 02269-001

IGES Rep: SL
Rig Type: Backhoe (JCB 214S)

TEST PIT NO:
TP-3
Sheet 1 of 1

LOCATION

LATITUDE | LONGITUDE | ELEVATION (ft)

DEPTH METERS	DEPTH FEET	SAMPLES	WATER LEVEL	GRAPHICAL LOG	UNIFIED SOIL CLASSIFICATION	MATERIAL DESCRIPTION	Dry Density(pcf)	Moisture Content %	Percent minus 200	Liquid Limit	Plasticity Index
0	0					Topsoil - Lean CLAY - medium stiff, moist, dark brown, frequent medium roots					
					CL	Native - Lean CLAY - medium stiff, moist, gray, frequent fine pinholes					
1											
	5				SM	Silty SAND - medium dense, moist, yellowish brown, iron staining					
2								15.7			
						No groundwater encountered Bottom of test pit @ 7 Feet					
3	10										
4											

Moisture Content and Atterberg Limits

Plastic Limit | Moisture Content | Liquid Limit

10 20 30 40 50 60 70 80 90

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

WATER LEVEL
- MEASURED
- ESTIMATED

NOTES:

IGES

Figure A-5

DATE		
STARTED:	3/29/16	
COMPLETED:	3/29/16	
BACKFILLED:	3/29/16	

Geotechnical Investigation
SwimKids Riverton
2630 West 12600 South
Riverton, Utah

Project Number 02269-001

IGES Rep: SL

Rig Type: Backhoe (JCB 214S)

TEST PIT NO: **TP-4**

Sheet 1 of 1

LOCATION

LATITUDE LONGITUDE ELEVATION (ft)

MATERIAL DESCRIPTION

Depth (meters)	Depth (feet)	Samples	Water Level	USCS	Material Description	Dry Density (pcf)	Moisture Content %	Percent minus 200	Liquid Limit	Plasticity Index
0	0				Undocumented fill - 1-2 feet of medium stiff silt and clay overlaying 1.5 feet of loose 1-inch gravel, the fill is dry and light brown					
1		3" O.D. thin-walled hand sampler		CL	Native - Lean CLAY - medium stiff to stiff, dry to moist, light gray, frequent fine pinholes	81.8	15.0			
	5	Grab sample								
2					- frequent medium pinholes					
				SM	Silty SAND - medium dense, dry to moist, yellowish brown					
				GM	Silty GRAVEL with sand - medium dense, dry to moist, light brown, gravel is sub-angular to sub-rounded, 1/2 to 3 inches in diameter					
3	10									
					No groundwater encountered					
					Bottom of test pit @ 12 Feet					
4										

Moisture Content and Atterberg Limits

Plastic Limit — Moisture Content — Liquid Limit

10 20 30 40 50 60 70 80 90

IGES

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

WATER LEVEL
- MEASURED
- ESTIMATED

NOTES:

Figure A-6

UNIFIED SOIL CLASSIFICATION SYSTEM

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

LOG KEY SYMBOLS

BORING SAMPLE LOCATION

TEST-PIT SAMPLE LOCATION

WATER LEVEL (level after completion)

WATER LEVEL (level where first encountered)

CEMENTATION

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

OTHER TESTS KEY

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

MODIFIERS

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

MOISTURE CONTENT

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

GENERAL NOTES

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

STRATIFICATION

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

APPARENT / RELATIVE DENSITY - COARSE-GRAINED SOIL

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

CONSISTENCY - FINE-GRAINED SOIL

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

IGES®

Key to Soil Symbols and Terminology

Figure A-7

IGES, Inc. Project No. 02269-001

APPENDIX B

Particle-Size Distribution (Gradation) of Soils Using Sieve Analysis

(ASTM D6913)

IGES

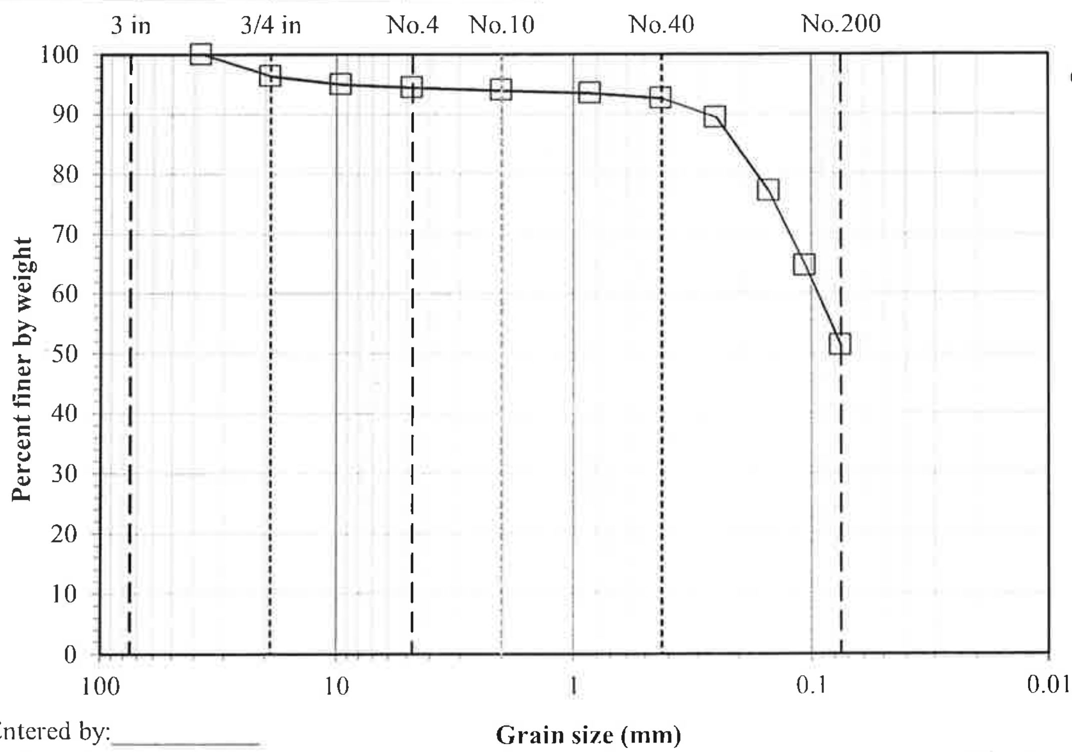
Project: SwimKids Riverton GTI
No: 02269-001
Location: Riverton, UT
Date: 4/5/2016
By: BRR

Boring No.: TP-1
Sample:
Depth: 8.0'
Description: Brown sandy silt

	Moist	Dry
Split:	Yes	
Split sieve:	3/8"	
Total sample wt. (g):	2436.31	2281.18
+3/8" Coarse fraction (g):	116.08	115.29
-3/8" Split fraction (g):	891.61	832.30
Split fraction:	0.949	

Water content data	C.F.(+3/8")	S.F.(-3/8")
Moist soil + tare (g):	271.54	1208.22
Dry soil + tare (g):	270.74	1148.91
Tare (g):	154.01	316.61
Water content (%):	0.7	7.1

Sieve	Accum. Wt. Ret. (g)	Grain Size (mm)	Percent Finer	
8"	-	200	-	
6"	-	150	-	
4"	-	100	-	
3"	-	75	-	
1.5"	-	37.5	100.0	
3/4"	83.60	19	96.3	
3/8"	115.29	9.5	94.9	←Split
No.4	4.73	4.75	94.4	
No.10	8.68	2	94.0	
No.20	13.03	0.85	93.5	
No.40	20.34	0.425	92.6	
No.60	48.72	0.25	89.4	
No.100	156.63	0.15	77.1	
No.140	266.18	0.106	64.6	
No.200	383.27	0.075	51.2	



Gravel (%): 5.6
Sand (%): 43.2
Fines (%): 51.2

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Reviewed:__________

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Particle-Size Distribution (Gradation) of Soils Using Sieve Analysis

(ASTM D6913)

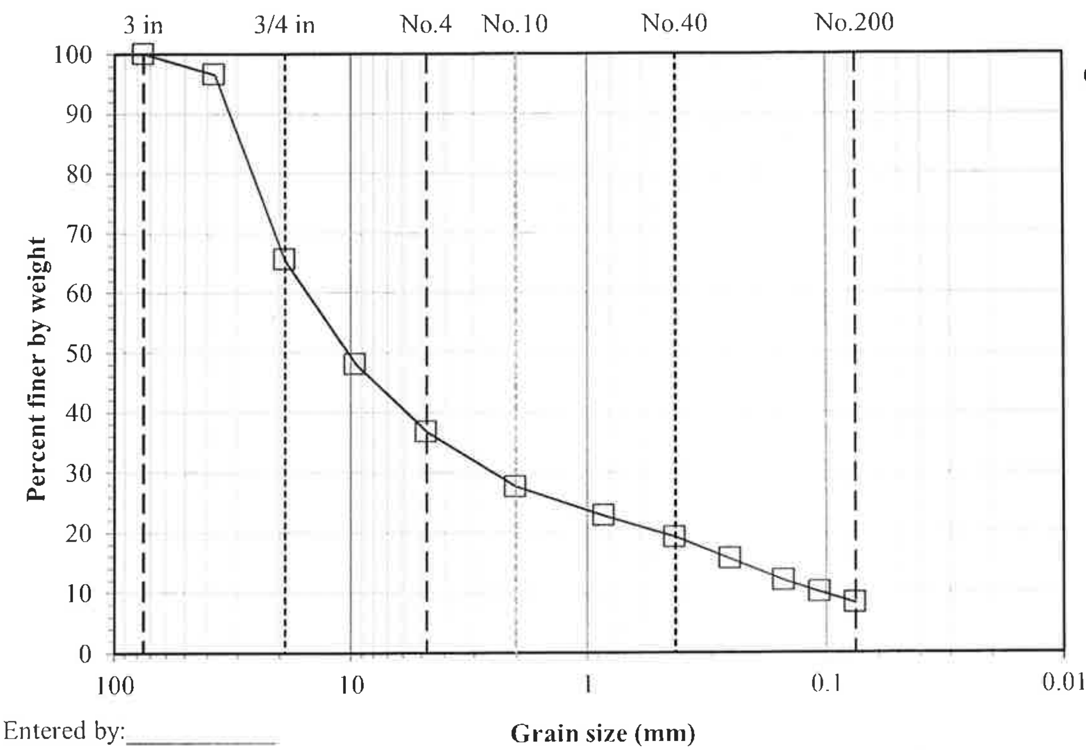
Project: SwimKids Riverton GTI
No: 02269-001
Location: Riverton, UT
Date: 4/5/2016
By: BRR

Boring No.: TP-2
Sample:
Depth: 10.0'
Description: Brown gravel with silt and sand

Split:	Yes	
Split sieve:	3/4"	
	Moist	Dry
Total sample wt. (g):	5294.92	5109.22
+3/4" Coarse fraction (g):	1786.84	1762.23
-3/4" Split fraction (g):	1409.65	1344.92
Split fraction:	0.655	

Water content data	C.F.(+3/4")	S.F.(-3/4")
Moist soil + tare (g):	2117.73	1720.08
Dry soil + tare (g):	2093.12	1655.35
Tare (g):	330.83	310.43
Water content (%):	1.4	4.8

Sieve	Accum. Wt. Ret. (g)	Grain Size (mm)	Percent Finer	
8"	-	200	-	
6"	-	150	-	
4"	-	100	-	
3"	-	75	100.0	
1.5"	177.76	37.5	96.5	
3/4"	1762.23	19	65.5	←Split
3/8"	358.73	9.5	48.0	
No.4	590.80	4.75	36.7	
No.10	780.06	2	27.5	
No.20	879.88	0.85	22.7	
No.40	953.37	0.425	19.1	
No.60	1026.08	0.25	15.5	
No.100	1100.17	0.15	11.9	
No.140	1140.16	0.106	10.0	
No.200	1176.77	0.075	8.2	



Gravel (%): 63.3
Sand (%): 28.5
Fines (%): 8.2

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Laboratory Compaction Characteristics of Soil

(ASTM D698 / D1557)

IGES

Project: SwimKids Riverton GTI
No: 02269-001
Location: Riverton, UT
Date: 4/6/2016
By: DKS

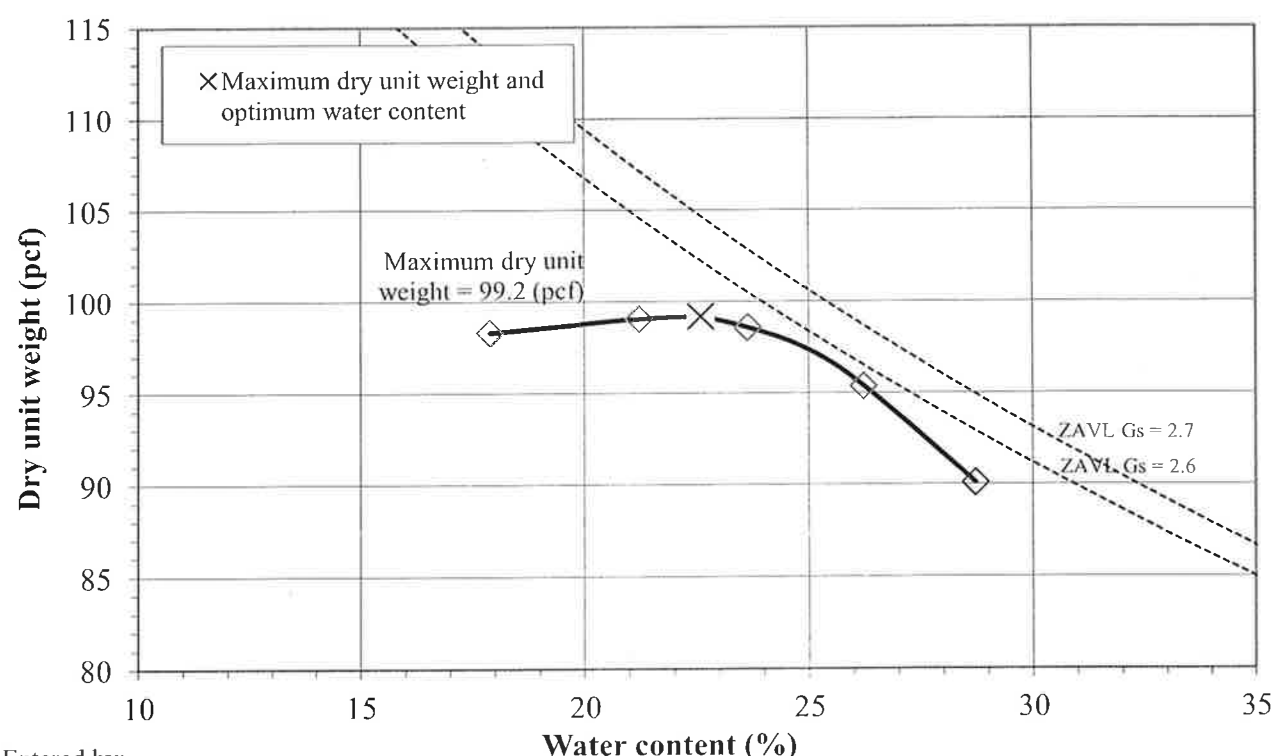
Boring No.: TP-3
Sample:
Depth: 2-3'
Sample Description: Grey clay
Engineering Classification: Not requested
As-received water content (%): Not requested
Preparation method: Moist
Rammer: Mechanical-sector face
Rock Correction: No

Method: ASTM D1557 C
Mold Id. Inc 7
Mold volume (ft^3): 0.0751

Optimum water content (%): 22.6
Maximum dry unit weight (pcf): 99.2

Point Number	As Is	-2%	-4%	-6%	-8%			
Wt. Sample + Mold (g)	10464.6	10614.7	10666.2	10604.7	10462.3			
Wt. of Mold (g)	6514.8	6514.8	6514.8	6514.8	6514.8			
Wet Unit Wt., γ_m (pcf)	116.0	120.4	121.9	120.1	115.9			
Wet Soil + Tare (g)	1031.79	1201.91	1603.47	1160.69	1476.53			
Dry Soil + Tare (g)	851.47	998.30	1337.95	986.45	1286.37			
Tare (g)	223.51	222.05	215.34	165.46	223.37			
Water Content, w (%)	**28.7**	**26.2**	23.7	21.2	17.9			
Dry Unit Wt., γ_d (pcf)	**90.1**	**95.4**	98.6	99.1	98.3			

Comments:
Due to insufficient sample, the -8% contained previously compacted material.



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California Bearing Ratio

(ASTM D 1883)

IGES

Project: SwimKids Riverton GTI
Number: 02269-001
Location: Riverton, UT
Date: 4/12/2016
By: DKS

Boring No.: TP-3
Sample:
Depth: 2-3'
Original Method: ASTM D1557 C
Engineering Classification: Not requested
Condition of Sample: Soaked
Scalp and Replace: No

Maximum Dry Unit Weight (pcf):	99.2
Optimum Water Content (%):	22.6
Relative Compaction (%):	94.4
0.1 in. Corrected CBR (%):	**5.4**
0.2 in. Corrected CBR (%):	**6.8**

As Compacted Data			Before	After
Mold Id.	CBR-7	Wet Soil + Tare (g)	1068.56	1645.02
Wt. of Mold + Sample (g)	10616.5	Dry Soil + Tare (g)	905.94	1375.04
Wt. of Mold (g)	6678.8	Tare (g)	222.26	221.86
Dry Unit Weight (pcf)	93.6	Water Content (%)	23.8	23.4

After Soaking Data			Average	Top 1 in.
Wt. of Mold + Sample (g)	10814.2	Wet Soil + Tare (g)	1451.59	484.24
Dry Unit Weight (pcf)	91.6	Dry Soil + Tare (g)	1203.05	399.5
		Tare (g)	330.72	124.39
		Water Content (%)	28.5	30.8

Swell Data

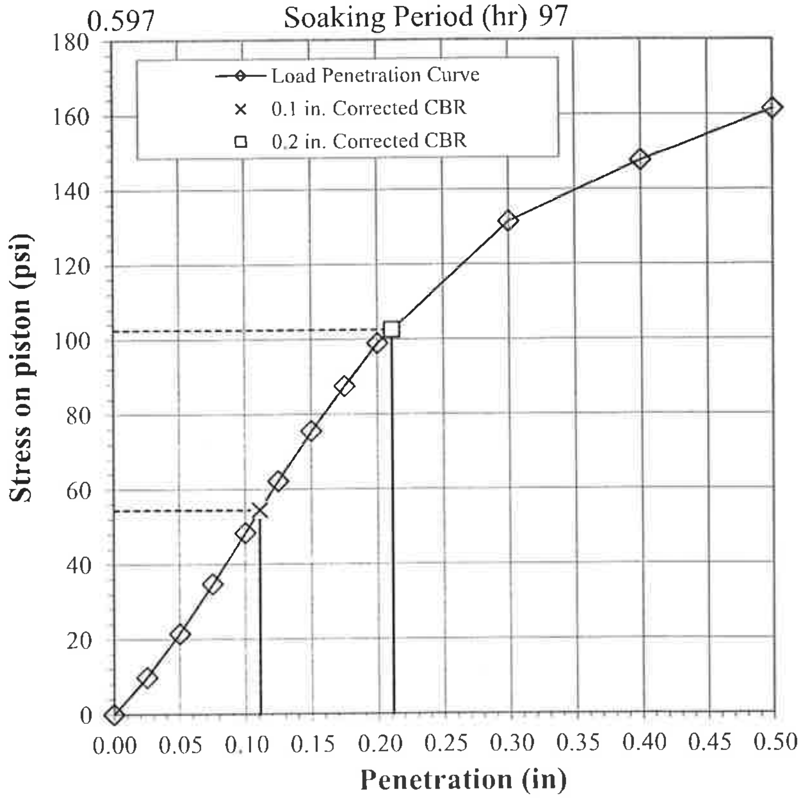
Date	Time	Dial
4/7/2016	10:35	0.494
4/11/2016	11:20	0.597

Surcharge (psf) 50
Swell (%) 2.25
Soaking Period (hr) 97

Penetration Data	Piston ID	CBR T1

Zero load (lb) = 2
Area of Piston (in^2) = 3.0

Penetration (in.)	Raw Load (lb)	Piston Stress (psi)	Std. Stress (psi)
0.000	0	0	
0.025	29	10	
0.050	64	22	
0.075	104	35	
0.100	145	48	1000
0.125	186	62	1125
0.150	226	75	1250
0.175	263	88	1375
0.200	297	99	1500
0.300	394	131	1900
0.400	443	148	2300
0.500	484	161	2600



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Collapse/Swell Potential of Soils
(ASTM D4546 Method B)

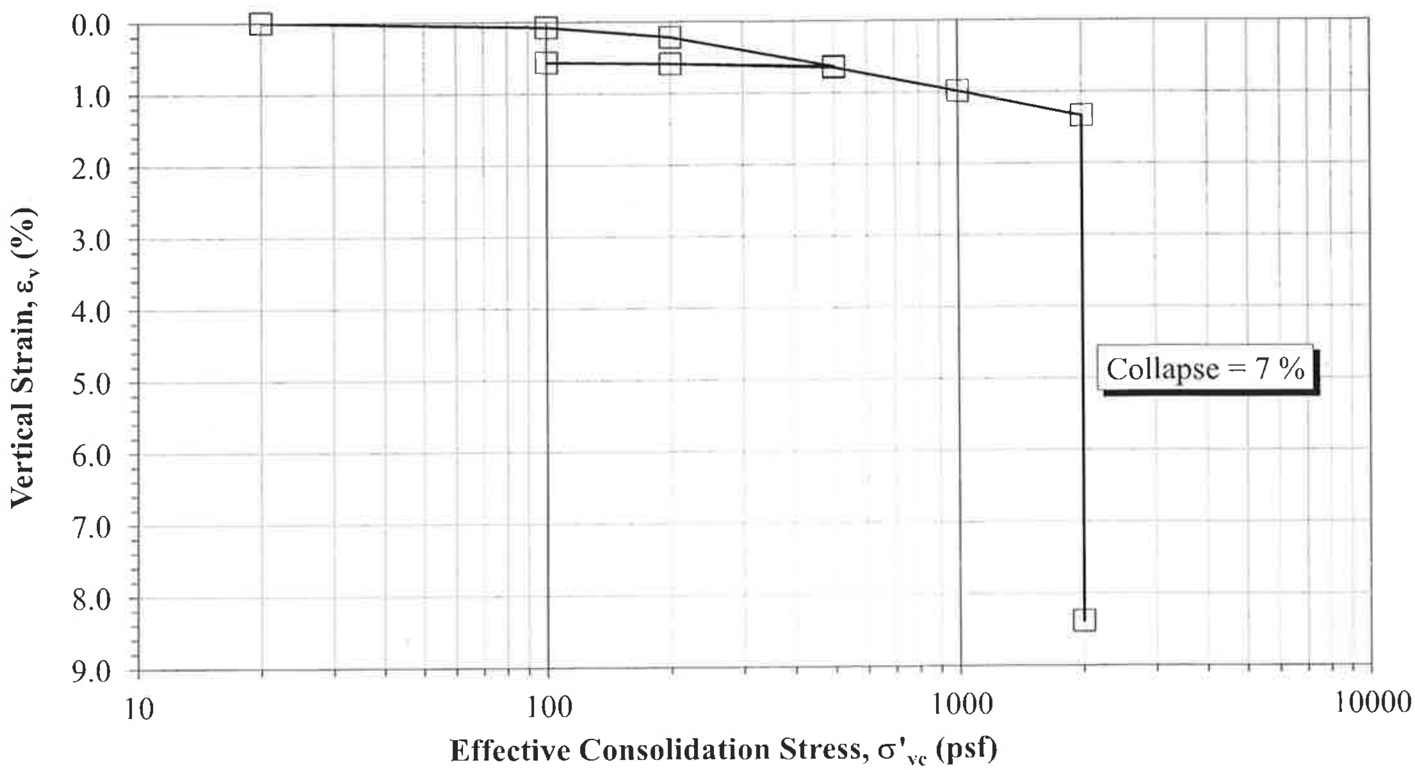
Project: SwimKids Riverton GTI
No: 02269-001
Location: Riverton, UT
Date: 4/5/2016
By: BRR

Boring No.: TP-1
Sample:
Depth: 4.5'
Sample Description: Brown clay
Engineering Classification: Not requested
Sample type: Undisturbed-trimmed from thin-wall

Consolidometer No.:	2	
Specific gravity, G_s	2.70	Assumed
Collapse (%)	7.0	
Collapse stress (psf)	2000	
Water type used for inundation	Tap	

	Initial (o)	Final (f)
Sample height, H (in.)	0.920	0.8429
Sample diameter, D (in.)	2.416	2.416
Mass rings + wet soil (g)	144.15	160.97
Mass rings/tare (g)	45.52	45.52
Moist unit wt., γ_m (pcf)	89.09	113.82
Wet soil + tare (g)	369.69	241.58
Dry soil + tare (g)	344.34	214.78
Tare (g)	122.78	126.76
Water content, w (%)	11.4	30.4
Dry unit wt., γ_d (pcf)	79.94	87.25
Saturation	27.87	88.23

Stress (psf)	Dial (in.)	1-D ε_v (%)	H_c (in.)	e
Seating	0.2181	0.00	0.9200	1.109
20	0.2181	0.00	0.9200	1.109
100	0.2188	0.08	0.9193	1.107
200	0.2201	0.22	0.9180	1.104
500	0.2239	0.63	0.9142	1.095
200	0.2235	0.59	0.9146	1.096
100	0.2233	0.57	0.9148	1.097
200	0.2235	0.59	0.9146	1.096
500	0.2241	0.65	0.9140	1.095
1000	0.2272	0.99	0.9109	1.088
2000	0.2304	1.34	0.9077	1.080
2000	0.2952	8.38	0.8429	0.932



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Collapse/Swell Potential of Soils

(ASTM D4546 Method B)

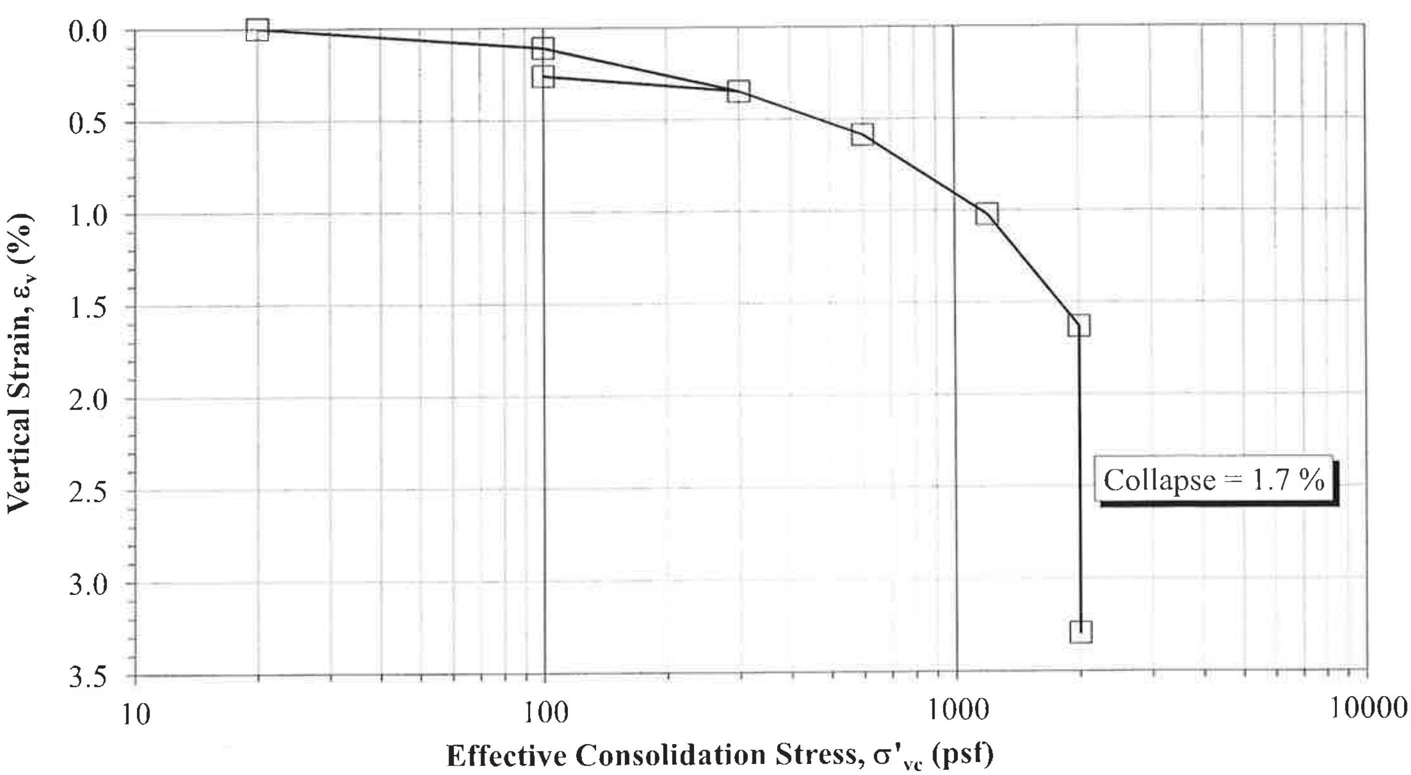
Project: **SwimKids Riverton GTI**
No: **02269-001**
Location: Riverton, UT
Date: 4/5/2016
By: BRR

Boring No.: **TP-2**
Sample:
Depth: **3.0'**
Sample Description: Brown clay
Engineering Classification: Not requested
Sample type: Undisturbed-trimmed from thin-wall

Consolidometer No.:	6	
Specific gravity, G_s	2.70	Assumed
Collapse (%)	1.7	
Collapse stress (psf)	2000	
Water type used for inundation	Tap	

	Initial (o)	Final (f)
Sample height, H (in.)	0.920	0.8897
Sample diameter, D (in.)	2.416	2.416
Mass rings + wet soil (g)	153.62	162.73
Mass rings/tare (g)	45.53	45.53
Moist unit wt., γ_m (pcf)	97.63	109.46
Wet soil + tare (g)	382.68	234.90
Dry soil + tare (g)	324.80	201.08
Tare (g)	128.76	117.44
Water content, w (%)	29.5	40.4
Dry unit wt., γ_d (pcf)	75.38	77.94
Saturation	64.49	93.91

Stress (psf)	Dial (in.)	1-D ε_v (%)	H_c (in.)	e
Seating	0.1007	0.00	0.9200	1.236
20	0.1007	0.00	0.9200	1.236
100	0.1017	0.11	0.9190	1.234
300	0.1039	0.35	0.9168	1.228
100	0.1031	0.26	0.9176	1.230
300	0.1039	0.35	0.9168	1.228
600	0.1061	0.59	0.9146	1.223
1200	0.1101	1.02	0.9106	1.213
2000	0.1157	1.63	0.9050	1.200
2000	0.1310	3.29	0.8897	1.163



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Collapse/Swell Potential of Soils

(ASTM D4546 Method B)

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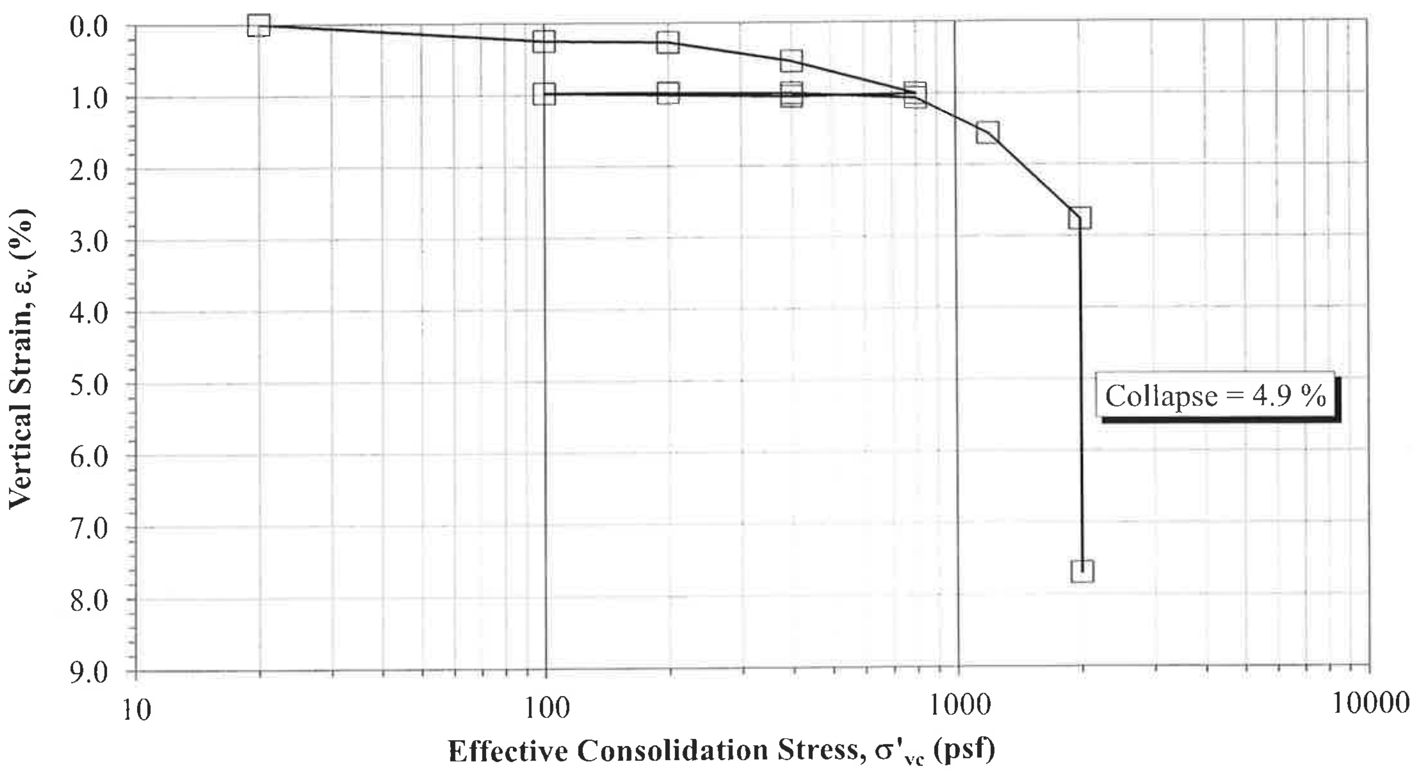
Project: SwimKids Riverton GTI
No: 02269-001
Location: Riverton, UT
Date: 4/5/2016
By: BRR

Boring No.: TP-2
Sample:
Depth: 6.5'
Sample Description: Brown sandy silt
Engineering Classification: Not requested
Sample type: Undisturbed-trimmed from thin-wall

Consolidometer No.: 5
Specific gravity, G_s: 2.70 Assumed
Collapse (%): 4.9
Collapse stress (psf): 2000
Water type used for inundation: Tap

	Initial (o)	Final (f)
Sample height, H (in.)	0.920	0.8492
Sample diameter, D (in.)	2.416	2.416
Mass rings + wet soil (g)	139.94	156.48
Mass rings/tare (g)	41.54	41.54
Moist unit wt., γ_m (pcf)	88.88	112.47
Wet soil + tare (g)	331.01	238.17
Dry soil + tare (g)	309.15	210.69
Tare (g)	140.48	124.68
Water content, w (%)	13.0	31.9
Dry unit wt., γ_d (pcf)	78.68	85.24
Saturation	30.64	88.26

Stress (psf)	Dial (in.)	1-D ε_v (%)	H_c (in.)	e
Seating	0.2516	0.00	0.9200	1.142
20	0.2516	0.00	0.9200	1.142
100	0.2539	0.25	0.9177	1.137
200	0.2541	0.27	0.9175	1.136
400	0.2566	0.54	0.9150	1.131
800	0.2608	1.00	0.9108	1.121
400	0.2611	1.03	0.9105	1.120
100	0.2606	0.98	0.9110	1.121
200	0.2606	0.98	0.9110	1.121
400	0.2607	0.99	0.9109	1.121
800	0.2614	1.07	0.9102	1.119
1200	0.2660	1.57	0.9056	1.109
2000	0.2770	2.76	0.8946	1.083
2000	0.3224	7.70	0.8492	0.977



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Unconsolidated-Undrained Triaxial Compression Test on Cohesive Soils

IGES

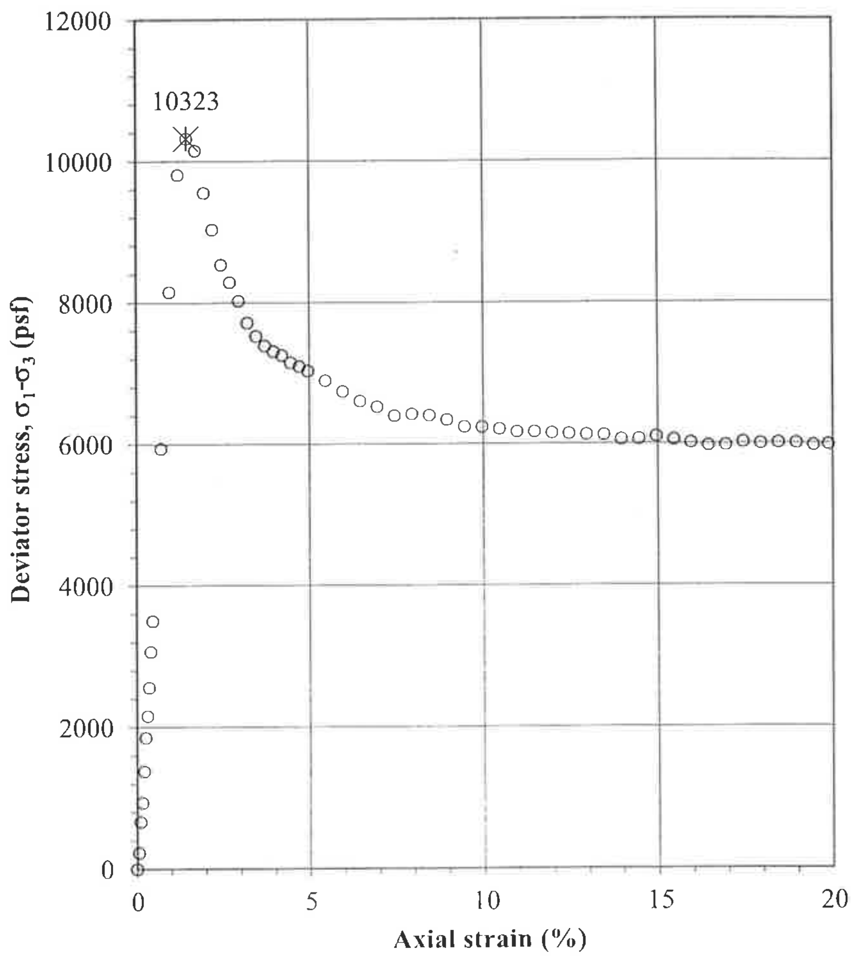
(ASTM D2850)

Project:	**SwimKids Riverton GTI**	**Boring No.:**	**TP-4**
No:	**02269-001**	**Sample:**	
Location:	Riverton, UT	**Depth:**	**3.5'**
Date:	4/4/2016	Sample Description:	Brown clay
By:	BRR	Sample type:	Undisturbed

Specific gravity, Gs	2.70	Assumed
Sample height, H (in.)	5.594	
Sample diameter, D (in.)	2.402	
Sample volume, V (ft^3)	0.0147	
Wt. rings + wet soil (g)	626.13	
Wt. rings/tare (g)	0.00	
Moist soil, Ws (g)	626.13	
Moist unit wt., γ_m (pcf)	94.1	
Dry unit wt., γ_d (pcf)	**81.8**	
Saturation (%)	38.0	
Void ratio, *e*	1.06	

Wet soil + tare (g)	1028.76
Dry soil + tare (g)	947.97
Tare (g)	408.20
Water content, w (%)	**15.0**
Confining stress, σ_3 (psf)	300
Shear rate (in/min)	0.0168
Strain at failure, ε_f (%)	1.45
Deviator stress at failure, $(\sigma_1-\sigma_3)_f$ (psf)	10323
Shear stress at failure, $q_f = (\sigma_1-\sigma_3)_f/2$ (psf)	5162

Axial Strain (%)	σ_d σ_1-σ_3 (psf)	Q 1/2 σ_d (psf)
0.00	0.0	0.0
0.05	232.6	116.3
0.10	668.7	334.4
0.15	938.2	469.1
0.20	1381.8	690.9
0.25	1854.1	927.1
0.30	2164.1	1082.1
0.35	2564.9	1282.5
0.40	3073.2	1536.6
0.45	3506.4	1753.2
0.70	5938.4	2969.2
0.95	8156.2	4078.1
1.20	9803.0	4901.5
1.45	10323.0	5161.5
1.70	10152.0	5076.0
1.95	9557.0	4778.5
2.20	9034.8	4517.4
2.45	8539.2	4269.6
2.70	8293.0	4146.5
2.95	8027.9	4014.0
3.20	7715.7	3857.9
3.45	7529.6	3764.8
3.70	7392.2	3696.1
3.95	7311.5	3655.8
4.20	7255.1	3627.6
4.45	7155.1	3577.6
4.70	7099.2	3549.6
4.95	7035.5	3517.8
5.45	6897.0	3448.5
5.95	6743.9	3372.0
6.45	6607.6	3303.8
6.95	6526.4	3263.2
7.45	6399.6	3199.8
7.95	6423.0	3211.5
8.45	6404.0	3202.0
8.95	6339.3	3169.7
9.45	6241.0	3120.5
9.95	6236.8	3118.4
10.45	6210.3	3105.2
10.95	6168.7	3084.4
11.45	6167.5	3083.8
11.95	6151.3	3075.7
12.45	6142.1	3071.1
12.95	6129.1	3064.6
13.45	6122.9	3061.5
13.95	6059.1	3029.6
14.45	6066.8	3033.4
14.95	6098.6	3049.3
15.45	6055.6	3027.8
15.95	6009.5	3004.8
16.45	5970.2	2985.1
16.95	5972.4	2986.2
17.45	6018.8	3009.4
17.95	5999.2	2999.6
18.45	6006.6	3003.3
18.95	5999.9	3000.0
19.45	5969.1	2984.6
19.88	5984.5	2992.3



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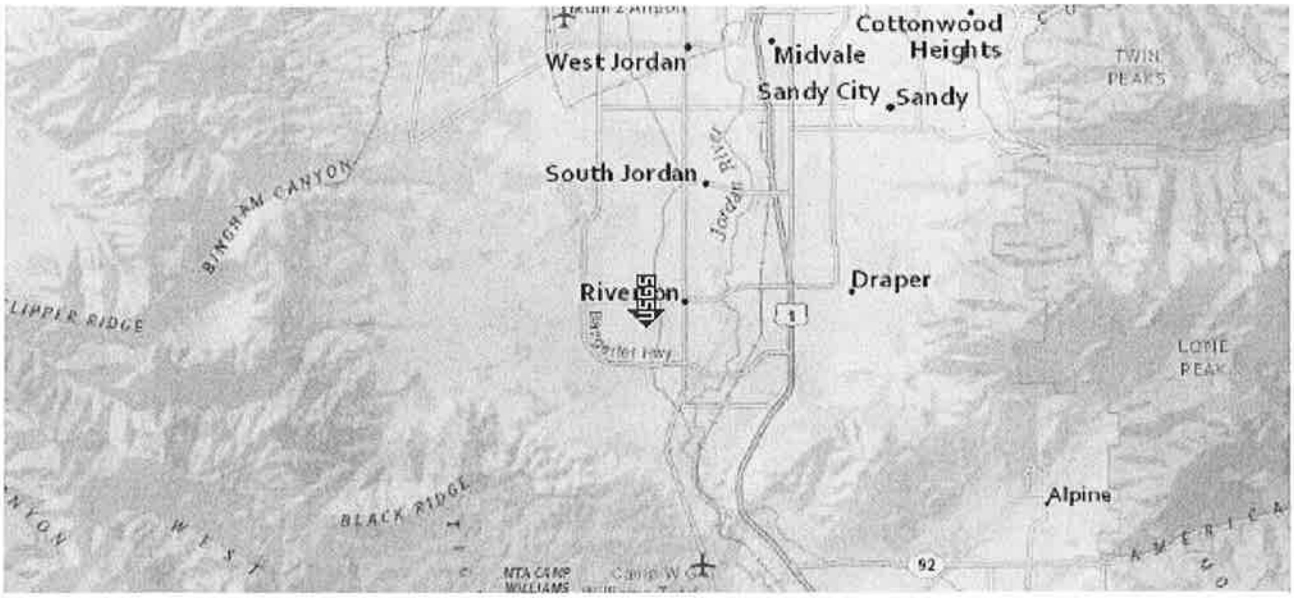
APPENDIX C

USGS Design Maps Summary Report

User–Specified Input

Building Code Reference Document	2012 International Building Code (which utilizes USGS hazard data available in 2008)
Site Coordinates	40.5226°N, 111.95651°W
Site Soil Classification	Site Class D – "Stiff Soil"
Risk Category	I/II/III

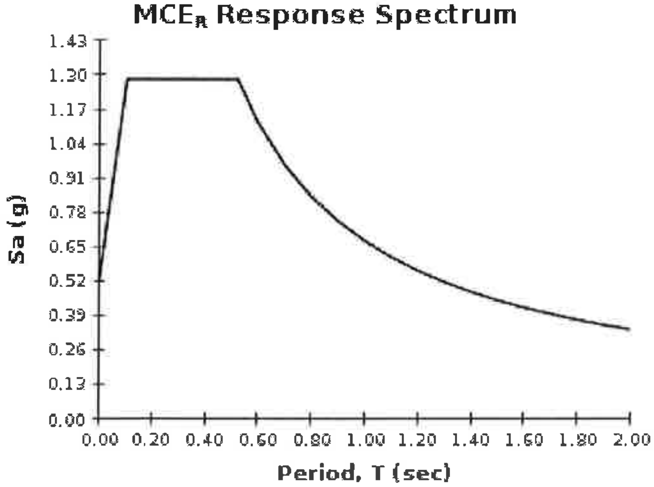


USGS–Provided Output

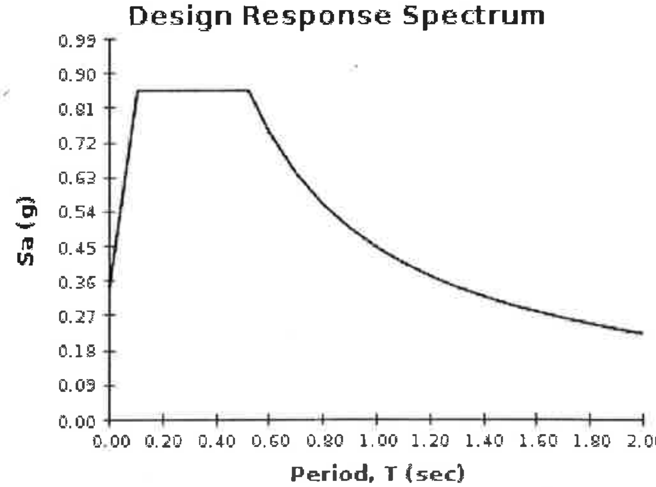
$S_S =$	1.284 g	$S_{MS} =$	1.284 g	$S_{DS} =$	0.856 g
$S_1 =$	0.426 g	$S_{M1} =$	0.671 g	$S_{D1} =$	0.447 g

For information on how the SS and S1 values above have been calculated from probabilistic (risk-targeted) and deterministic ground motions in the direction of maximum horizontal response, please return to the application and select the "2009 NEHRP" building code reference document.

MCE$_R$ Response Spectrum



Design Response Spectrum



USGS Design Maps Detailed Report

2012 International Building Code (40.5226°N, 111.95651°W)

Site Class D – "Stiff Soil", Risk Category I/II/III

Section 1613.3.1 — Mapped acceleration parameters

Note: Ground motion values provided below are for the direction of maximum horizontal spectral response acceleration. They have been converted from corresponding geometric mean ground motions computed by the USGS by applying factors of 1.1 (to obtain S_S) and 1.3 (to obtain S_1). Maps in the 2012 International Building Code are provided for Site Class B. Adjustments for other Site Classes are made, as needed, in Section 1613.3.3.

From Figure 1613.3.1(1) [1] $S_S = 1.284$ g

From Figure 1613.3.1(2) [2] $S_1 = 0.426$ g

Section 1613.3.2 — Site class definitions

The authority having jurisdiction (not the USGS), site-specific geotechnical data, and/or the default has classified the site as Site Class D, based on the site soil properties in accordance with Section 1613.

2010 ASCE-7 Standard – Table 20.3-1
SITE CLASS DEFINITIONS

Site Class	$\bar{v}_s$	$\bar{N}$ or $\bar{N}_{ch}$	$\bar{s}_u$
A. Hard Rock	>5,000 ft/s	N/A	N/A
B. Rock	2,500 to 5,000 ft/s	N/A	N/A
C. Very dense soil and soft rock	1,200 to 2,500 ft/s	>50	>2,000 psf
D. Stiff Soil	600 to 1,200 ft/s	15 to 50	1,000 to 2,000 psf
E. Soft clay soil	<600 ft/s	<15	<1,000 psf
	Any profile with more than 10 ft of soil having the characteristics: • Plasticity index $PI > 20$, • Moisture content $w \geq 40\%$, and • Undrained shear strength $\bar{s}_u < 500$ psf		
F. Soils requiring site response analysis in accordance with Section 21.1	See Section 20.3.1		

For SI: 1ft/s = 0.3048 m/s 1lb/ft² = 0.0479 kN/m²

Section 1613.3.3 — Site coefficients and adjusted maximum considered earthquake spectral response acceleration parameters

TABLE 1613.3.3(1)
VALUES OF SITE COEFFICIENT F_a

Site Class	Mapped Spectral Response Acceleration at Short Period				
	$S_S \leq 0.25$	$S_S = 0.50$	$S_S = 0.75$	$S_S = 1.00$	$S_S \geq 1.25$
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				

Note: Use straight-line interpolation for intermediate values of S_S

For Site Class = D and S_S = 1.284 g, F_a = 1.000

TABLE 1613.3.3(2)
VALUES OF SITE COEFFICIENT F_v

Site Class	Mapped Spectral Response Acceleration at 1-s Period				
	$S_1 \leq 0.10$	$S_1 = 0.20$	$S_1 = 0.30$	$S_1 = 0.40$	$S_1 \geq 0.50$
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				

Note: Use straight-line interpolation for intermediate values of S_1

For Site Class = D and S_1 = 0.426 g, F_v = 1.574

Equation (16-37): $S_{MS} = F_aS_S = 1.000 \times 1.284 = 1.284$ g

Equation (16-38): $S_{M1} = F_vS_1 = 1.574 \times 0.426 = 0.671$ g

Section 1613.3.4 — Design spectral response acceleration parameters

Equation (16-39): $S_{DS} = ⅔\, S_{MS} = ⅔ \times 1.284 = 0.856$ g

Equation (16-40): $S_{D1} = ⅔\, S_{M1} = ⅔ \times 0.671 = 0.447$ g

Section 1613.3.5 — Determination of seismic design category

TABLE 1613.3.5(1)
SEISMIC DESIGN CATEGORY BASED ON SHORT-PERIOD (0.2 second) RESPONSE ACCELERATION

VALUE OF S_{DS}	RISK CATEGORY		
	I or II	III	IV
$S_{DS} < 0.167g$	A	A	A
$0.167g \leq S_{DS} < 0.33g$	B	B	C
$0.33g \leq S_{DS} < 0.50g$	C	C	D
$0.50g \leq S_{DS}$	D	D	D

For Risk Category = I and S_{DS} = 0.856 g, Seismic Design Category = D

TABLE 1613.3.5(2)
SEISMIC DESIGN CATEGORY BASED ON 1-SECOND PERIOD RESPONSE ACCELERATION

VALUE OF S_{D1}	RISK CATEGORY		
	I or II	III	IV
$S_{D1} < 0.067g$	A	A	A
$0.067g \leq S_{D1} < 0.133g$	B	B	C
$0.133g \leq S_{D1} < 0.20g$	C	C	D
$0.20g \leq S_{D1}$	D	D	D

For Risk Category = I and S_{D1} = 0.447 g, Seismic Design Category = D

Note: When S_1 is greater than or equal to 0.75g, the Seismic Design Category is **E** for buildings in Risk Categories I, II, and III, and **F** for those in Risk Category IV, irrespective of the above.

Seismic Design Category ≡ "the more severe design category in accordance with Table 1613.3.5(1) or 1613.3.5(2)" = D

Note: See Section 1613.3.5.1 for alternative approaches to calculating Seismic Design Category.

References

1. *Figure 1613.3.1(1)*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/IBC-2012-Fig1613p3p1(1).pdf
2. *Figure 1613.3.1(2)*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/IBC-2012-Fig1613p3p1(2).pdf