

August 11, 2014

Mr. Grant H. Blakeslee Summit, LLC 3632 North Wolf Creek Drive Eden, Utah 84310

IGES Project No. 01628-006

RE: Geotechnical Investigation Report (Revised)
Lot 34R of Powder Mountain Resort
7958 East Heartwood Drive
Weber County, Utah

Mr. Blakeslee,

As requested, IGES has conducted a geotechnical investigation for the proposed residence to be constructed on Lot 34R of the Powder Mountain Resort located at 7958 East Heartwood Drive in Weber County, Utah. The approximate location of the property is illustrated on the *Site Vicinity Map* (Figure A-1 in Appendix A). The purposes of our investigation was to assess the nature and engineering properties of the subsurface soils at the proposed home site and to provide recommendations for the design and construction of foundations, grading, and drainage. The scope of work completed for this study included subsurface exploration, laboratory testing, engineering analyses and preparation of this letter. This report has been revised from the original report dated August 7, 2014 to further discuss the presence of bedrock at the site.

Project Understanding

Our understanding of the project is based primarily on our previous involvement with the Powder Mountain resort project, which included two geotechnical investigations for the greater 200-acre Powder Mountain Resort expansion project (IGES, 2012a and 2012b).

The Powder Mountain Resort expansion project is located southeast of SR-158 (Powder Mountain Road), south of previously developed portions of Powder Mountain Resort, in unincorporated Weber County, Utah. The project is accessed by Powder Ridge Road.

Lot 34R is a ¾-acre single-family residential lot with a buildable envelope of approximately 0.21 acres. A single-family home will be constructed at the site, presumably a high-end vacation home. Construction plans were not available for our review; however, we assume the new home will be a one- or two-story wood-framed structure, with a walk-out basement, founded on conventional spread footings. The development is expected to include improvements common for residential subdivisions such as underground utilities, curb and gutter, flatwork, landscaping, and possibly appurtenant structures.

METHOD OF STUDY

Literature Review

IGES completed a geotechnical investigation for the Powder Mountain Resort expansion in 2012 (2012a, 2012b). Our previous work included twenty-two test pits and one soil boring excavated at various locations across the 200-acre development; as a part of this current study, the logs from relevant nearby test pits and other data from our reports were reviewed. In addition, Western Geologic (2012) completed a geologic hazard study for the greater 200-acre Powder Mountain expansion project – this report was reviewed to assess the potential impact of geologic hazards on the subject lot.

Field Investigation

Subsurface soils were investigated by excavating one test pit approximately 12 feet below the existing site grade. The approximate location of the test pit is illustrated on the *Geotechnical Map* (Figure A-2 in Appendix A). The soil types and conditions were visually logged at the time of the excavation in general accordance with the Unified Soil Classification System (USCS). Subsurface soil classifications and descriptions are included on the test pit log included as Figure A-3 in Appendix A. A key to USCS symbols and terminology is included as Figure A-4.

Laboratory Testing

Samples retrieved during the subsurface investigation were transported to the laboratory for evaluation of engineering properties. Specific laboratory tests include:

- Moisture Content and Unit Weight
- Soluble Sulfate, Soluble Chloride, pH and Resistivity

Results of the laboratory testing are discussed in this report and presented in Appendix B. Some test results, including moisture content; and unit weight, have been incorporated into the test pit log (Figure A-3).

In addition to laboratory testing on samples obtained from this lot, engineering analysis was also based on previously completed laboratory work on soil samples obtained near the site (IGES, 2012a & 2012b).

Engineering Analysis

Engineering analyses were performed using soil data obtained from laboratory testing and empirical correlations based on material density, depositional characteristics and classification. Appropriate factors of safety were applied to the results consistent with industry standards and the accepted standard of care. An allowable bearing pressure value was proportioned based on estimated shear strength of bearing soils.

FINDINGS

Surface Conditions

At the time of the excavation, the lot was in a relatively natural state and was covered with a variety of vegetation including mature pine trees, native grasses and shrubs. The lot slopes relatively steeply toward north at a gradient of approximately 2.4H:1V, away from Heartwood Drive. On the southern boundary of the lot there is a 'ridge' jutting northeast into the building envelope, forming a topographic high point for the lot. This ridge is covered with a stand of mature pine trees. The ridge also represents an exposure of bedrock (dolomite). The remainder of the lot is essentially a sloped grassy field. Aside from the rocky outcrops on the ridge, several angular boulders could be observed at various locations on the surface.

Earth Materials

The earth materials exposed at the site consist of a rocky northeast-southwest-trending salient exposing dolomite bedrock, surrounded by a thick sequence of sandy colluvial cover (this is illustrated on Figure A-2). The soil at the surface of the site consists of approximately 6 inches of poorly-developed topsoil consisting of mottled silty sand characterized by an abundance of organic matter (roots, etc.). The topsoil was underlain by medium dense clayey sand extending to a depth of approximately 9 feet below existing grade. Underlying this layer, we encountered coarse colluvium consisting of medium-dense clayey gravel. The colluvium was characterized by abundant coarse angular rock fragments, which extended to the bottom of the excavation (approximately 12 feet below the existing grade). Due to the coarsness of the colluvium at 12 feet, it is postulated that bedrock could have been within a few feet of the bottom of the test pit; however, difficult excavating conditions limited the depth of the test pit.

Upon the topographic high point of the lot (illustrated on Figure A-2 in red, designated as geologic unit Cr), we observed bedrock outcrops consisting of highly weathered, closely fractured dark gray dolomite. The rock unit is fairly hard – samples could only be obtained with a firm blow from a rock hammer. It should be noted that the rock/colluvium contact it thought to dip steeply, since bedrock was not encountered in the test pit even though the test pit was excavated near the bedrock outcrop.

Detailed descriptions of earth materials encountered are presented on the test pit log, Figure A-3, in Appendix A.

Groundwater

Groundwater was not encountered in the test pit excavation. Based on our observations, groundwater is not anticipated to adversely impact the proposed construction. However, groundwater levels could rise at any time based on several factors including recent precipitation, on- or off-site runoff, irrigation, and time of year (e.g., spring run-off). Should the groundwater become a concern during the proposed construction, IGES should be contacted so that dewatering recommendations may be provided.

Geology and Geologic Hazards

Geology and geologic hazards have been previously addressed by Western Geologic in a separate submittal (Western Geologic, 2012). This work has also been referenced in our previous geotechnical reports for the project (IGES, 2012a and 2012b). The report by Western Geologic indicates that the lot is located outside of known geologically unstable areas. The Western Geologic report also includes a large-scale geologic map that shows the subject lot in an area mapped as "undifferentiated dolomite". Dolomite is a rock that has similar mechanical properties to limestone and is fairly hard, often forming cliffs and other near-vertical formations.

During our subsurface investigation, potentially adverse geologic structures (e.g., evidence of faulting or landslides) were not evident to the maximum depth of exploration (12 feet). Geomorphic expressions of shallow, surficial landslides were not observed on, or near the lot. Based on currently available data and our observations, the potential for geologic hazards such as landslides, liquefaction, or surface fault rupture impacting the site is considered low.

Seismicity

Following the criteria outlined in the 2012 International Building Code (IBC, 2012), spectral response at the site was evaluated for the *Maximum Considered Earthquake* (MCE) which equates to a probabilistic seismic event having a two percent probability of exceedance in 50 years (2PE50). Spectral accelerations were determined based on the location of the site using the *U.S. Seismic "DesignMaps" Web Application* (USGS, 2012); this software incorporates seismic hazard maps depicting probabilistic ground motions and spectral response data 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 field exploration and our understanding of the geology in this area, the subject site is appropriately classified as Site Class B (Rock). Based on IBC criteria, the short-period (F_a) coefficient is 1.0 and long-period (F_v) site coefficient is 1.0. Based on the design spectral response accelerations for a Building Risk Category of I, II or III, the site's Seismic Design Category is D. The short- and long-period Design Spectral Response Accelerations are presented in Table 1.0; a summary of the Design Maps analysis is presented in Appendix C. The peak ground acceleration (PGA) may be taken as 0.4*Sms.

Table 1.0
Short- and Long-Period Spectral Accelerations for MCE

Parameter	Short Period (0.2 sec)	Long Period (1.0 sec)
MCE Spectral Response Acceleration (g)	$S_S = 0.826$	$S_1 = 0.274$
MCE Spectral Response Acceleration Site Class C (g)	$S_{MS} = S_s F_a = 0.826$	$S_{M1} = S_1 F_v = 0.274$
Design Spectral Response Acceleration (g)	$S_{DS} = S_{MS}*^2/_3 = 0.551$	$S_{D1} = S_{M1}*^2/_3 = 0.183$

CONCLUSIONS AND RECOMMENDATIONS

Based on the results of the field observations, laboratory testing and previously completed geotechnical investigation (IGES, 2012a), the subsurface conditions are considered suitable for the proposed construction provided that the recommendations presented in this report are incorporated into the design and construction of the project.

General Site Preparation and Grading

Prior to the placement of foundations, general site grading is recommended to provide proper support for exterior concrete flatwork, concrete slabs-on-grade, and pavement sections. Site grading is also recommended to provide proper drainage and moisture control on the subject property and to aid in preventing differential movement in foundation soils as a result of variations in moisture conditions.

Below proposed structures, fills, and man-made improvements, all vegetation, topsoil, debris and undocumented fill soils (if any) should be removed. Any existing utilities should be re-routed or protected in place. The exposed native soils should then be proof-rolled with heavy rubber-tired equipment such as a scraper or loader. Any soft/loose areas identified during proof-rolling should be removed and replaced with structural fill. All excavation bottoms should be observed by an IGES representative during proof rolling or otherwise prior to placement of engineered fill to evaluate whether soft, loose, or otherwise deleterious earth materials have been removed and that recommendations presented in this report have been complied with.

Excavations

Soft, loose, or otherwise unsuitable soils beneath structural elements, hardscape or pavements may need to be over-excavated and replaced with structural fill. If over-excavation is required, the excavations should extend one foot laterally for every foot of depth of over-excavation. Excavations should extend laterally at least two feet beyond flatwork, pavements, and slabs-on-grade. Structural fill should consist of granular materials and should be placed and compacted in accordance with the recommendations presented in this report.

Prior to placing engineered fill, all excavation bottoms should be scarified to at least 6 inches, moisture-conditioned as necessary at or slightly above optimum moisture content (OMC), and compacted to at least 90 percent of the maximum dry density (MDD) as determined by ASTM D-1557 (Modified Proctor). Scarification is not required where bedrock is exposed.

Excavation Stability

The contractor is responsible for site safety, including all temporary trenches excavated at the site and the design of any required temporary shoring. The contractor is responsible for providing the "competent person" required by Occupational Safety and Health (OSHA) standards to evaluate soil conditions. For planning purposes, Soil Type C is expected to predominate at the site (sands and gravels). Close coordination between the competent person and IGES should be maintained to facilitate construction while providing safe excavations.

Based on 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. As an alternative to shoring or shielding, trench walls may be laid back at one and one half horizontal to one vertical (1½H:1V) (34 degrees) in accordance with OSHA Type C soils. Trench walls may need to be laid back at a steeper grade pending evaluation of soil conditions by the geotechnical engineer. Soil conditions should be evaluated in the field on a case-by-case basis. Large rocks exposed on excavation walls should be removed (scaled) to minimize rock fall hazards. Where dolomite bedrock is exposed, near-vertical walls (0.25H:1V) may be permitted provided adverse jointing or bedding patterns are absent and the excavation is assessed by the OSHA 'competent person' prior to occupancy.

Structural Fill and Compaction

All fill placed for the support of structures, flatwork or pavements should consist of structural fill. Structural fill should consist of granular native soils, which may be defined as soils with less than 25% fines, 10-60% sand, and contain no rock larger than 4 inches in nominal size (6 inches in greatest dimension). Structural fill should also be free of vegetation and debris. Soils not meeting these criteria may be suitable for use as structural fill; however, such soils should be evaluated on a case by case basis and should be approved by IGES prior to use.

All structural fill should be placed in maximum 4-inch loose lifts if compacted by small hand-operated compaction equipment, maximum 6-inch loose lifts if compacted by light-duty rollers, and maximum 8-inch loose lifts if compacted by heavy duty compaction equipment that is capable of efficiently compacting the entire thickness of the lift. Additional lift thickness may be allowed by IGES provided the Contractor can demonstrate sufficient compaction can be achieved with a given lift thickness with the equipment in use. We recommend that all structural fill be compacted on a horizontal plane, unless otherwise approved by IGES. Structural fill underlying all shallow footings and pavements should be compacted to at least 95 percent of the MDD as determined by ASTM D-1557.

The moisture content should be at, or slightly above, the OMC for all structural fill. 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.

Specifications from governing authorities such as Weber County and/or special service districts having their own precedence for backfill and compaction should be followed where more stringent.

Utility Trench Backfill

Utility trenches should be backfilled with structural fill in accordance with the previous section. Utility trenches can be backfilled with the onsite soils free of debris, organic and oversized material. Prior to backfilling the trench, pipes should be bedded in and shaded with a uniform granular material that has a Sand Equivalent (SE) of 30 or greater. Pipe bedding may be water-densified in-place (jetting). Alternatively, pipe bedding and shading may consist of clean ¾-inch gravel, which generally does not require densification. Native earth materials can be used as backfill over the pipe bedding zone. All utility trenches backfilled below pavement sections, curb and gutter, hardscape, should be backfilled with structural fill compacted to at least 95 percent of the MDD as determined by ASTM D-1557. All other trenches should be backfilled and compacted to approximately 90 percent of the MDD (ASTM D-1557). However, in all cases the pipe bedding and shading should meet the design criteria of the pipe manufacturer. Specifications from governing authorities having their own precedence for backfill and compaction should be followed where they are more stringent.

Foundations

Based on our field observations and considering the presence of bedrock exposures within the building envelope, we recommend that the footings for proposed home be founded *entirely* on bedrock. Bedrock/soil transition zones are not allowed. However, it is possible, and even likely, that deep colluvial deposits located on the north side of the building envelope may preclude the practical construction of all foundation on bedrock; as such, as an alternative to extending all foundations to bedrock, foundations constructed over colluvium may be underpinned with micropiles or a similar underpinning technology. This is conceptually illustrated on Figure D-1 in Appendix D.

Since the bedrock/colluvium contact cannot be known with certainty, and since the design of the new home is currently in the planning stages, the extent to which micropiles will be necessary (or perhaps not required) will not be evident until the basement is excavated. We recommend that IGES inspect the bottom of the foundation excavation prior to the placement of steel or concrete to identify any unsuitable soils or transition zones. If bedrock/soil transitions zones are identified, the Contractor may wish to pot-hole to assess the depth to bedrock and thus determine if deepening the foundations is practical, or if underpinning the foundations is the preferred option.

It should be noted that the bedrock at the site is expected to be very difficult to excavate (see *Construction Considerations* on page 11 of this report).

Shallow spread or continuous wall footings constructed entirely on competent bedrock may be proportioned utilizing a maximum net allowable bearing pressure of **5,000 pounds per square foot (psf)** for dead load plus live load conditions. The net allowable bearing value presented above is for dead load plus live load conditions. The minimum recommended footing width is 20 inches for continuous wall footings and 30 inches for isolated spread footings.

All conventional foundations exposed to the full effects of frost should be established at a minimum depth of 42 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.

Foundation drains should be installed around below-ground foundations (e.g., basement walls) to minimize the potential for flooding from shallow groundwater, which may be present at various times during the year, particularly spring run-off.

Underpinning

Underpinning, if used, should be designed by IGES or an engineer experienced in deep foundation design. *For planning purposes*, underpinning may consist of micropiles conforming to the following criteria:

- Injection Bore micropile, R38N hollow bar, uncased.
- 6-inch grouted diameter.
- Socket a minimum of three feet into bedrock or 20 feet into colluvium, whichever is shorter.
- A single micropile, as described above, may be assumed to have an allowable axial capacity of 35 kips.
- Lateral resistance, if required by the Structural Engineer, will require a cased micropile and must be designed for specific project requirements.

Settlement

Static settlement 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.

Competent native earth materials and/or properly compacted structural fill is expected to exhibit negligible seismically-induced settlement during a MCE seismic event.

Earth Pressure and Lateral Resistance

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 soils. In determining the frictional resistance against concrete, a coefficient of friction of 0.45 for sandy native soils or structural fill should be used.

Ultimate lateral earth pressures from *granular* backfill acting against retaining walls, temporary shoring, or buried structures may be computed from the lateral pressure coefficients or equivalent fluid densities presented in Table 2.0. These lateral pressures should be assumed even if the backfill is placed in a relatively narrow gap between a vertical bedrock cut and the foundation wall.

Table 2.0

Lateral Earth Pressure Coefficients

	Level	Backfill	2H:1V Backfill		
Condition	Lateral Pressure Coefficient	Equivalent Fluid Density (pcf)	Lateral Pressure Coefficient	Equivalent Fluid Density (pcf)	
Active (Ka)	0.33	35	0.53	56	
At-rest (Ko)	0.50	55	0.80	85	
Passive (Kp)	3.0	320	-		

These coefficients and densities assume no buildup of hydrostatic pressures. The force of water should be added to the presented values if hydrostatic pressures are anticipated.

Clayey soils drain poorly and may swell upon wetting, thereby greatly increasing lateral pressures acting on earth retaining structures; therefore, clayey soils should not be used as retaining wall backfill. Backfill should consist of native granular soil with an Expansion Index (EI) less than 20.

Walls and structures allowed to rotate slightly should use the active condition. If the element is to be constrained against rotation (i.e., a basement or buried tank wall), the atrest 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.

Concrete Slab-on-Grade Construction

To minimize settlement and cracking of slabs, and to aid in drainage beneath the concrete floor slabs, all concrete slabs should be founded on a minimum 4-inch layer of compacted gravel overlying properly prepared subgrade. The gravel should consist of free-draining gravel or road base with a 3/4-inch maximum particle size and no more than 5 percent passing the No. 200 mesh sieve. The layer should be compacted to at least 95 percent of the MDD as determined by ASTM D-1557.

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 fibermesh. Slab reinforcement should be designed by the structural engineer; however, as

a minimum, slab reinforcement should consist of 4"×4" W4.0×W4.0 welded wire mesh within the middle third of the slab. We recommend that 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 Modulus of Subgrade Reaction of 400 psi/inch may be used for design.

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.

Moisture Protection

Moisture should not be allowed to infiltrate into the soils in the vicinity of the foundations. As such, design strategies to minimize ponding and infiltration near the home should be implemented. The new home may be subject to sheet flow during periods of heavy rain or snow melt; therefore, the Civil Engineer may also wish to consider construction of additional surface drainage to intercept surface runoff, or a curtain drain to intercept seasonal groundwater flow, if any.

We recommend that hand watering, desert landscaping or Xeriscape be considered within 5 feet of the foundations. We further recommend roof runoff devices be installed to direct all runoff a minimum of 10 feet away from structures. The home builder should be responsible for compacting the exterior backfill soils around the foundation. Additionally, the ground surface within 10 feet of the house should be constructed so as to slope a minimum of **five** percent away from the home. Pavement sections should be constructed to divert surface water off of the pavement into storm drains. Parking strips and roadway shoulder areas should be constructed to prevent infiltration of water into the areas surrounding pavement. Landscape plans must conform to Weber County development codes.

IGES recommends a perimeter foundation drain be constructed for the proposed residential structure in accordance with the International Residential Code (IRC).

Soil Corrosion Potential

Laboratory testing of a representative soil sample obtained from the test pit indicated that the soil sample tested had a sulfate content of 8 ppm. Accordingly, the soils are classified as having a 'low' potential for deterioration of concrete due to the presence of soluble sulfate. As such, conventional Type I/II Portland cement may be used for all concrete in contact with site soils.

To evaluate the corrosion potential of ferrous metal in contact with onsite native soil a sample was tested for soil resistivity, soluble chloride and pH. The test indicated that the onsite soil tested has a minimum soil resistivity of 3,156 OHM-cm, soluble chloride content of 3.8 ppm and a pH of 8.2. Based on this result, the onsite native soil is considered to be

moderately corrosive to ferrous metal. Consideration should be given to retaining the services of a qualified corrosion engineer to provide an assessment of any metal that may be associated with construction of ancillary water lines and reinforcing steel, valves etc.

Construction Considerations

- Excavation Difficulty: bedrock consisting of relatively hard dolomite is exposed at the surface within the building envelope. Based on conversations with contractors currently working in the vicinity, this rock is expected to be relatively difficult to remove. Special heavy-duty excavation equipment will likely be required, such as a hammer hoe.
- Over-Size Material: A bedrock outcrop was observed within the building footprint of this lot. In addition, large boulders up to 12 inches were observed on the surface; larger boulders may be present within the colluvial soil. As such, development of the lot is expected to generate a substantial amount of over-size material (rocks larger than 6 inches in greatest dimension). Large rocks, particularly boulders, may require special handling, such as segregation from structural fill, and disposal. Bedrock is expected to require specialized equipment for removal during excavation of the basement.

CLOSURE

The recommendations presented in this letter are based on limited field exploration, literature review, and a general understanding of the proposed construction. The subsurface data used in the preparation of this letter were obtained from the exploration(s) made for this investigation. It is possible that variations in the soil and groundwater conditions could exist beyond the point 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 letter, IGES should be immediately notified so that any necessary revisions to recommendations contained in this letter may be made. In addition, if the scope of the proposed construction changes from that described in this letter, IGES should also 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 letter in its entirety. The use of information contained in this report for bidding purposes should be done at the Contractor's option and risk.

Additional Services

The recommendations presented in this report are based on the assumption that an adequate program of tests and observations will be made during the 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.
- Consultation as may be required during construction.
- Quality control testing of cast-in-place concrete.
- Review of plans and specifications to assess compliance with our recommendations.

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 contact the undersigned at (801) 748-4044.

Respectfully submitted, IGES, Inc.

Shun Li, P.E.I. Staff Engineer Reviewed by:

David A. Glass, P.E. Senior Geotechnical Engineer

Attachments:

References

Appendix A

Figure A-1 – Site Vicinity Map

Figure A-2 – Geotechnical Map

Figure A-3 – Test Pit Log

Figure A-4 – Key to Soil Symbols and Terminology

Appendix B – Laboratory Results

Appendix C – 2012 IBC MCE and Design Response Acceleration

Appendix B – Laboratory Results

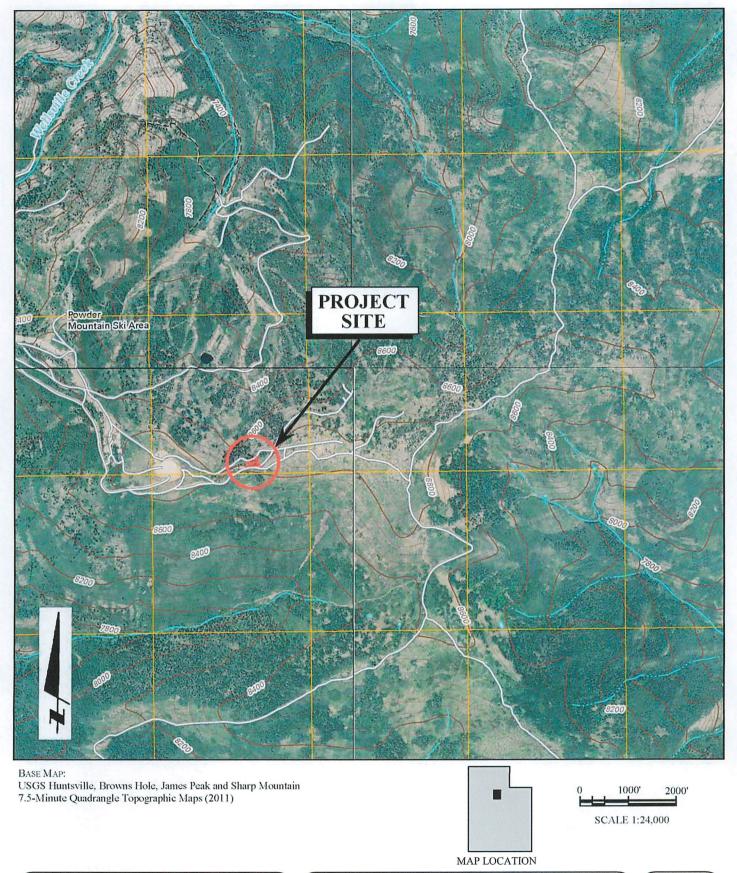
Figure D-1 – Conceptual Cross-Section – Foundation Underpinning

Figure D-2 – Conceptual Cross-Section – Source Plan-View

References

- AMEC, 2001, Report Engineering Geologic Reconnaissance/Geotechnical Study Powder Mountain Resort.
- Federal Emergency Management Agency [FEMA], 1997, NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures, FEMA 302, Washington, D.C.
- Frankel, A., Mueller, C., Barnard, T., Perkins, D., Leyendecker, E.V., Dickman, N., Hanson, S., and Hopper, M., 1996, *National Seismic-hazard Maps: Documentation*, U.S. Geological Survey Open-File Report 96-532, June.
- IGES, Inc., 2012a, Design Geotechnical Investigation, Powder Mountain Resort, Weber County, Utah, Project No. 01628-003, dated November 9, 2012.
- IGES, Inc., 2012b, Preliminary Geotechnical Investigation, Powder Mountain Resort, Weber County, Utah, Project No. 01628-001, dated July 26, 2012.
- International Building Code [IBC], 2012, International Code Council, Inc.
- PSI, 2012, Geophysical ReMi Investigation, Powder Mountain Resort, Phase 1A, Weber County, Utah, PSI Project No. 0710375, dated September 18, 2012.
- U.S. Geological Survey, 2012, U.S. Seismic "Design Maps" Web Application, site: https://geohazards.usgs.gov/secure/designmaps/us/application.php.
- Western Geologic, 2012, Report: Geologic Hazards Reconnaissance, Proposed Area 1 Mixed-Use Development, Powder Mountain Resort, Weber County, Utah, dated August 28, 2012.

APPENDIX A



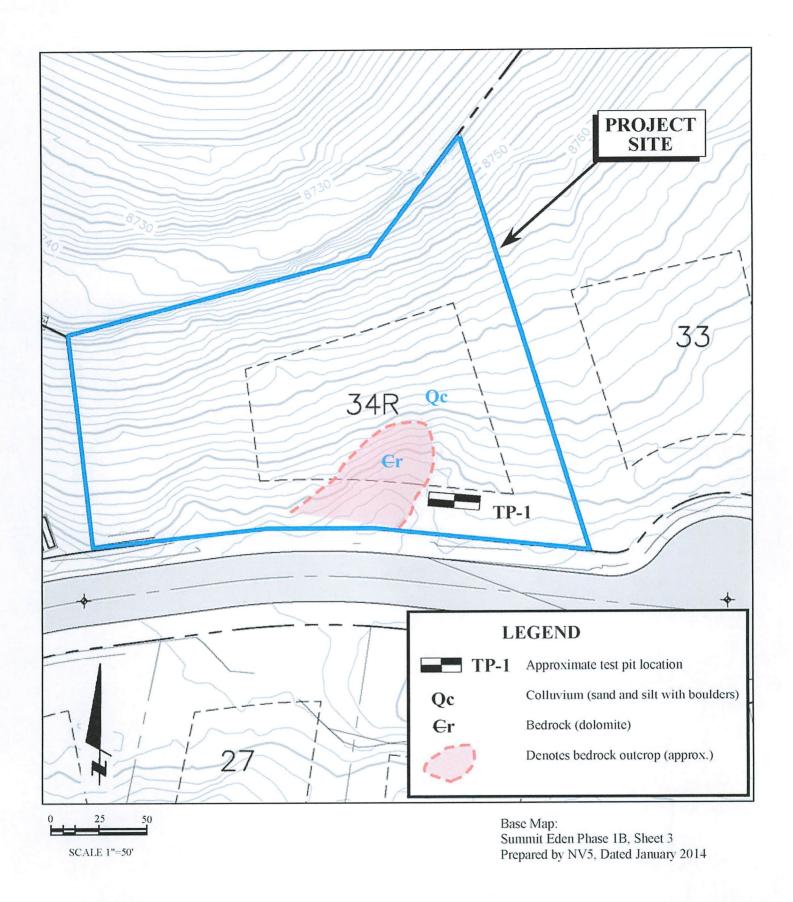


Project No. 01628-006

Geotechnical Investigation
Lot 34R of Powder Mountain Resort
7958 East Heartwood Drive
Weber County, Utah
SITE VICINITY MAP

Figure

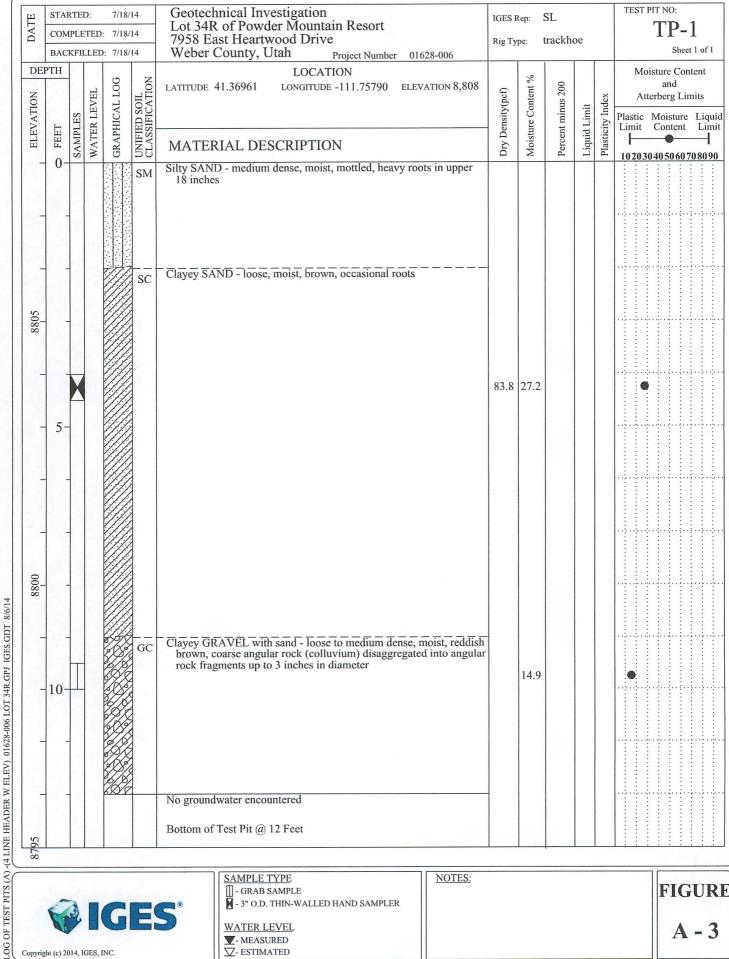
A-1





Geotechnical Investigation
Lot 34R of Powder Mountain Resort
7958 East Heartwood Drive
Weber County, Utah
GEOTECHNICAL MAP

Figure A-2





SAMPLE TYPE GRAB SAMPLE

- 3" O.D. THIN-W

- 3" O.D. THIN-WALLED HAND SAMPLER

WATER LEVEL ▼- MEASURED

✓- ESTIMATED

NOTES:

FIGURE

GRAVELS

More than half of

coarse fraction is larger than the #4 sieve)

SANDS

coarse fraction is smaller than the #4 sieve)

COARSE

GRAINED

SOILS

(More than half

is larger than

the #200 sieve)

FINE GRAINED

SOILS

(More than half of material is smaller than

the #200 sieve)

 				-			_		
	M	AJOR	DI	/ISI	SNC	3			

CS TYPICAL DESCRIPTIONS

MIXTURES

MIXTURES

CLAYEY SANDS

OF LOW PLASTICITY

FAT CLAYS

GW

GM

GC

SP

SC

CL

OL

CH

OH

CLEAN GRAVELS

OR NO FINES

GRAVELS

WITH OVER

CLEAN SANDS

WITH LITTLE OR NO FINES

SANDS WITH OVER 12% FINES

SILTS AND CLAYS

(Liquid limit less than 50)

SILTS AND CLAYS

(Liquid limit greater than 50)

HIGHLY ORGANIC SOILS

12% FINES

WELL-GRADED GRAVELS, GRAVEL-SAND

POORLY-GRADED GRAVELS, GRAVEL-SAND

MIXTURES WITH LITTLE OR NO FINES

MIXTURES WITH LITTLE OR NO FINES

SILTY GRAVELS, GRAVEL-SILT-SAND

CLAYEY GRAVELS, GRAVEL-SAND-CLAY

WELL-GRADED SANDS, SAND-GRAVEL

POORLY-GRADED SANDS, SAND-GRAVEL

MIXTURES WITH LITTLE OR NO FINES

MIXTURES WITH LITTLE OR NO FINES SILTY SANDS, SAND-GRAVEL-SILT MIXTURES

SAND-GRAVEL-CLAY MIXTURES

INORGANIC SILTS & VERY FINE SANDS.

CLAYEY SILTS WITH SLIGHT PLASTICITY

INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS

ORGANIC SILTS & ORGANIC SILTY CLAYS

INORGANIC SILTS, MICACEOUS OR

DIATOMACEOUS FINE SAND OR SILT

ORGANIC CLAYS & ORGANIC SILTS

OF MEDIUM-TO-HIGH PLASTICITY

WITH HIGH ORGANIC CONTENTS

PEAT, HUMUS, SWAMP SOILS

INORGANIC CLAYS OF HIGH PLASTICITY,

BORING SAMPLE LOCATION

LOG KEY SYMBOLS



TEST-PIT SAMPLE LOCATION



WATER LEVEL (level after completion)

 $\bar{\triangle}$

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

OTTIL	IN ILOIO NLI		
С	CONSOLIDATION	SA	SIEVE ANALYSIS
AL	ATTERBERG LIMITS	DS	DIRECT SHEAR
UC	UNCONFINED COMPRESSION	T	TRIAXIAL
S	SOLUBILITY	R	RESISTIVITY
0	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

GENERAL NOTES

- Lines separating strata on the logs represent approximate boundaries only. Actual transitions may be gradual.
- No warranty is provided as to the continuity of soil conditions between individual sample locations.
- Logs represent general soil conditions observed at the point of exploration on the date indicated.
- 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.

DESCRIPTION	FIELD 1E31	
DRY	ABSENCE OF MOISTURE, DUSTY, DRY TO THE TOUCH	
MOIST	DAMP BUT NO VISIBLE WATER	
WET	VISIBLE FREE WATER, USUALLY SOIL BELOW WATER TABLE	

STRATIFICATION

MOISTURE CONTENT

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		TORVANE	POCKET PENETROMETER	FIELD TEST
CONSISTENCY	SPT (blows/ft)	UNTRAINED SHEAR STRENGTH (tsf)	UNCONFINED COMPRESSIVE STRENGTH (tsf)	
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.



Key to Soil Symbols and Terminology

Figure A-4

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IGES, Inc. Project No.:01628-006

APPENDIX B

Water Content and Unit Weight of Soil

(In General Accordance with ASTM D7263 Method B and D2216)



Project: GTI - Powder Mountain Resort

No: 01628-006

Location: Weber County, Utah

Date: 7/29/2014

By: MP

e	Boring No.					
Sample Info.		Lot34TP1		7		
Sa	Depth:	4.0'				
	Sample height, H (in)	5.446				
nfo.	Sample diameter, D (in)	2.416				
ht I	Sample volume, V (ft ³)	0.0144				
/eig	Mass rings + wet soil (g)	948.80				4
Unit Weight Info.	Mass rings/tare (g)	250.66				1,1/2
Un	Moist soil, Ws (g)	698.14				2
	Moist unit wt., γ _m (pcf)	106.53				
or ant	Wet soil + tare (g)	819.67				
Water Content	Dry soil + tare (g)	670.76				
2 2	Tare (g)	122.36				
,	Water Content, w (%)					
	Dry Unit Wt., γ _d (pcf)	83.8		2		

Entered by:	
Reviewed:	

Minimum Laboratory Soil Resistivity, pH of Soil for Use in Corrosion Testing, and



Ions in Water by Chemically Suppressed Ion Chromatography (AASHTO T 288, T 289, ASTM D4327, and C1580)

Project: GTI - Powder Mountain Resort

No: 01628-006

Location: Weber County, Utah

Date: 8/5/2014

By: ET

-					
ble .	Boring No.				
Sample info.	Sample		Lot 34		
S	Depth		9.5		
ata	Wet soil + tare (g)		140.		
Water content data	Dry soil + tare (g)		127.	24	
Wante	Tare (g)		37.8		100
ဒ	Water content (%)		14.	9	
ıta	рН		8.1	6	
Chem. data	Soluble chloride* (ppm)		3.8	3	
neu	Soluble sulfate** (ppm)		8		
D					
	Pin method		2		
	Soil box		Miller	Small	
		Approximate		G II D	
		Soil	Resistance		Dogiativ
		condition (%)	Reading (Ω)	Multiplier (cm)	(Ω-cm
		As Is	8550	0.67	5729
		+3	6570	0.67	4402
	}	+6	4710	0.67	3156
ta		+9	4760	0.67	3189
/ da	J.C	+9	4/00	0.07	3109
Resistivity data					
sisti					
Re	d				
-	Minimum resistivity				
	Winimum resistivity (Ω-cm)		315	56	
	(32-011)				Marie Company of the

^{*} Performed by AWAL using EPA 300.0

Entered by:	
Reviewed:	

^{**} Performed by AWAL using ASTM C1580

APPENDIX C

Design Maps Summary Report

User-Specified Input

Report Title Lot 34R

Tue August 12, 2014 00:42:37 UTC

Building Code Reference Document 2012 International Building Code

(which utilizes USGS hazard data available in 2008)

Site Coordinates 41.3696°N, 111.7579°W

Site Soil Classification Site Class B - "Rock"

Risk Category I/II/III



USGS-Provided Output

$$S_s = 0.826 g$$
 $S_{MS} = 0.826 g$ $S_{DS} = 0.551 g$

$$S_{MS} = 0.826 \, g$$

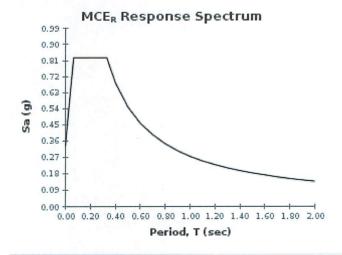
$$S = 0.551 \, g$$

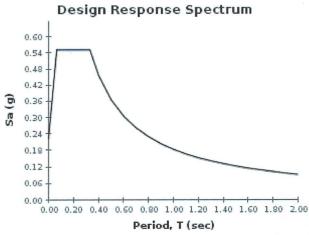
$$S_1 = 0.274 g$$
 $S_{M1} = 0.274 g$

$$S_{...} = 0.274 g$$

$$S_{D1} = 0.183 g$$

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.





Although this information is a product of the U.S. Geological Survey, we provide no warranty, expressed or implied, as to the accuracy of the data contained therein. This tool is not a substitute for technical subject-matter knowledge.

ZUSGS Design Maps Detailed Report

2012 International Building Code (41.3696°N, 111.7579°W)

Site Class B - "Rock", 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]

 $S_{s} = 0.826 g$

From Figure 1613.3.1(2) [2]

 $S_1 = 0.274 g$

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 B, based on the site soil properties in accordance with Section 1613.

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

Site Class	\overline{v}_{s}	\overline{N} or \overline{N}_{ch}	- 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 $w \ge 40\%$, and
- \bullet Undrained shear strength $s_{\parallel} < 500 \text{ psf}$

F. Soils requiring site response analysis in accordance with Section 21.1

See Section 20.3.1

For SI: $1ft/s = 0.3048 \text{ m/s} 1 \text{lb/ft}^2 = 0.0479 \text{ kN/m}^2$

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

Site Class	Mapped Spectral Response Acceleration at Short Period					
	S _s ≤ 0.25	S _s = 0.50	S _s = 0.75	S _s = 1.00	S _s ≥ 1.25	
Α	0.8	0.8	0.8	0.8	0.8	
В	1.0	1.0	1.0	1.0	1.0	
С	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_ς

For Site Class = B and $S_s = 0.826 g$, $F_a = 1.000$

TABLE 1613.3.3(2) VALUES OF SITE COEFFICIENT $F_{_{\rm V}}$

Site Class	Mapped Spectral Response Acceleration at 1-s Period					
	S ₁ ≤ 0.10	$S_{1} = 0.20$	$S_{1} = 0.30$	$S_{1} = 0.40$	$S_i \ge 0.50$	
Α	0.8	0.8	0.8	0.8	0.8	
В	1.0	1.0	1.0	1.0	1.0	
С	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 = B and $S_1 = 0.274 \text{ g}, F_v = 1.000$

Equation (16-37):
$$S_{MS} = F_a S_S = 1.000 \times 0.826 = 0.826 g$$

Equation (16-38):
$$S_{M1} = F_{V_1} = 1.000 \times 0.274 = 0.274 g$$

Section 1613.3.4 — Design spectral response acceleration parameters

Equation (16-39):
$$S_{DS} = \frac{2}{3} S_{MS} = \frac{2}{3} \times 0.826 = 0.551 g$$

Equation (16-40):
$$S_{D1} = \frac{2}{3} S_{M1} = \frac{2}{3} \times 0.274 = 0.183 g$$

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	RISK CATEGORY				
VALUE OF S _{DS}	I or II	III	IV		
S _{DS} < 0.167g	Α	А	Α		
0.167g ≤ S _{DS} < 0.33g	В	В	С		
0.33g ≤ S _{DS} < 0.50g	С	С	D		
0.50g ≤ S _{DS}	D	D	D		

For Risk Category = I and $S_{DS} = 0.551$ g, Seismic Design Category = D

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

VALUE OF S_	RISK CATEGORY				
VALUE OF S	I or II	III	IV		
S _{D1} < 0.067g	Α	А	А		
$0.067g \le S_{_{D1}} < 0.133g$	В	В	С		
$0.133g \le S_{D1} < 0.20g$	С	С	D		
0.20g ≤ S _{D1}	D	D	D		

For Risk Category = I and $S_{p_1} = 0.183$ g, Seismic Design Category = C

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 \equiv "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

...o..ru -r ----

- 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

APPENDIX D