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GEOTECHNICAL INVESTIGATION
The Ridge Nests Development
Powder Mountain Resort
Weber and Cache Counties, Utah

IGES Project No. 01628-008

September 16, 2014

Prepared for:

Summit, LLC

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TABLE OF CONTENTS

APPENDICES

1.0 INTRODUCTION

1.1 PURPOSE AND SCOPE OF WORK

This report presents the results of a geotechnical investigation conducted for *The Ridge Nests* development, a part of the currently on-going expansion at the Powder Mountain Ski Resort in Weber County, Utah (the *Ridge Nests* site straddles both Weber and Cache Counties). The purposes of our investigation was to assess the nature and engineering properties of the subsurface soils at the proposed home sites 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, literature review, engineering analyses, and preparation of this report.

Our services were performed in accordance with our proposal to Summit, LLC (Client), dated August 8, 2014. The recommendations presented in this report are subject to the limitations presented in the "Limitations" section of this report (Section 6.1).

1.2 PROJECT DESCRIPTION

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) and subsequent geotechnical consulting for several other aspects of the project.

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. The *Ridge Nests* development is located north of Summit Pass and north/east of Heartwood Drive, approximately 7880 East 6075 North (see *Site Vicinity Map*, Figure A-1 in Appendix A). The approximately 3.1-acre *Ridge Nests* project will consist of fifteen single-family residences that are essentially small cottages, presumably intended to be vacation homes. The individual cottages will vary with the Owner's tastes; however, the cottages are expected to have a structural footprint on the order of 1,300 square feet and will be on-grade structures (no basement). Access to the individual units will be from a sidewalk – parking will be accommodated by a parking lot with 15 stalls – there is no provisions for parking or garages at the individual units. The concept of the development is to maintain as natural an environment as possible; as such, landscaping or other features is expected to be kept to a minimum. Some of the units may be constructed on 'stilts' to further minimize the visual impact to the natural environment.

2.0 METHOD OF STUDY

2.1 LITERATURE REVIEW

The earliest geotechnical report for the area is by AMEC (2001), which was a reconnaissance-level geotechnical and geologic hazard study. IGES later 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 *Ridge Nests* development.

2.2 FIELD INVESTIGATION

The site largely consists of bedrock outcrops; as such, the primary focus of our field investigation was to surface map the contact between bedrock and surficial soils (colluvium). Where surficial soils were identified, additional subsurface exploration was conducted. Subsurface soils were investigated by excavating three test pits at representative locations. The approximate location of the test pits are illustrated on the *Geotechnical Map* (Figure A-2 in Appendix A).

The soil and rock types were visually logged at the time of our field work in general accordance with the *Unified Soil Classification System* (USCS). Rock and soil classifications and descriptions are included on the test pit logs, Figures A-3 through A-5 in Appendix A. A key to USCS symbols and terminology is included as Figure A-6.

2.3 LABORATORY TESTING

The majority of the site consists of hard rock, with limited areas consisting of coarse colluvium and possibly undocumented fill. As such, soil samples suitable for laboratory testing could not be obtained. Therefore, engineering analysis was based largely on previously completed geotechnical investigations (IGES, 2012a & 2012b), including laboratory work completed on soil samples obtained from nearby test pits completed in 2012 and test pits recently completed for lots adjacent to *The Ridge Nests* development.

3.0 GEOLOGIC CONDITIONS

3.1 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 report for the project (IGES, 2012b). The report by Western Geologic indicates that the development is located outside of known geologically unstable areas. The Western Geologic report also includes a large-scale geologic map that shows the development is 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 in the test pits. Also, geomorphic expressions of shallow, surficial landslides were not observed within the development. 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.

3.2 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 3.2; a summary of the *Design Maps* analysis is presented in Appendix B. The *peak ground acceleration* (PGA) may be taken as $0.4*S_{MS}$.

Table 3.2
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 B (g)	$S_{MS} = S_sF_a = 0.826$	$S_{M1} = S_1F_v = 0.274$
Design Spectral Response Acceleration (g)	$S_{DS} = S_{MS}{*}^2/_3 = 0.551$	$S_{D1} = S_{M1}{*}^2/_3 = 0.183$

4.0 GENERALIZED SITE CONDITIONS

4.1 SURFACE CONDITIONS

At the time of our field work the site was in a relatively natural state and was covered with a variety of vegetation including mature pine trees, native grasses and shrubs. A rough dirt road transects the site roughly east-west. The site runs along a ridge formed by an outcrop of dolomite bedrock.

4.2 SUBSURFACE CONDITIONS

The subsurface soil conditions were explored at the subject property by excavating three test pits where surficial soil was observed (the majority of the site is underlain by hard bedrock). Subsurface soil conditions were logged during our field investigation and are included in the exploration logs in Appendix A at the end of this report (Figures A-3 through A-5). The relative locations of the various geologic units described herein are illustrated on the *Geotechnical Map*, Figure A-2. The soil and moisture conditions encountered during our investigation are discussed below.

4.2.1 Earth Materials

Topsoil: Topsoil was encountered in limited areas; where encountered, the topsoil is generally thin, poorly developed, and rocky. Where encountered, topsoil cover was generally less than six inches. Areas of deeper topsoil deposits may exist within localized topographic depressions; however, the presences of topsoil is expected to have a negligible impact to the development.

Colluvium: Where encountered, the majority of surficial soils consist of rocky colluvium, likely derived from nearby bedrock outcrops of dolomite and/or conglomerate. The colluvium generally consisted of silty sand with gravel, cobbles, and boulders.

Bedrock: Based on our review of geologic literature and field observations, the majority of the site is underlain by bedrock consisting of undifferentiated Cambrian-age dolomite (Cr). This rock unit is fairly hard – samples could only be obtained with a firm blow from a rock hammer. Where exposed, the bedrock was moderately weathered, closely fractured, and dark gray, and reacted weakly to dilute HCl. At the time of our field work Geneva was excavating a utility line just off-site to the northeast – the trench exposed dolomite from the surface to the bottom of the trench (about nine feet). Geneva personnel indicated that excavation of the dolomite was very difficult, requiring a ram-hoe (a large jack-hammer on the end of an excavator arm). In addition to the dolomite, in Test Pit 1 at a depth of about 3½ feet we encountered very hard stratum that is believed to be representative of the Tertiary-age Wasatch Formation (Tw), which generally consists of well-cemented conglomerate.

Undocumented Fill: Earth materials suspected as being undocumented fill (Afu) were encountered in limited areas; these areas are delineated on Figure A-2. These soils generally consist of fine-grained sand with occasional to frequent rocks, particularly angular dolomite rock fragments.

Within this area, excavation was relatively easy, which is uncharacteristic for the area surrounding *The Ridge Nests* development. Also, the topography in the suspect area is relatively planar and appears out of place – it is postulated that the suspected undocumented fill area may consist of an in-filled natural drainage channel, possibly used as a place to deposit excess spoils during construction of dirt roads in the past.

Detailed descriptions of earth materials encountered are presented on the test pit logs, Figures A-3 through A-5, in Appendix A. Due to the nature and depositional characteristics of the native earth materials, care should be taken in interpolating subsurface conditions between and beyond the exploration locations.

4.2.2 Groundwater

Groundwater was not encountered in the test pit excavations. In addition, groundwater was not observed in the nearby utility excavation that was on-going during our field work. Based on our observations, groundwater is not anticipated to adversely impact the proposed development. 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.

5.0 CONCLUSIONS AND RECOMMENDATIONS

5.1 GENERAL CONCLUSIONS

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

Supporting data upon which the following recommendations are based have been presented in the previous sections of this report. The recommendations presented herein are governed by the physical properties of the earth materials encountered in the subsurface explorations. If subsurface conditions other than those described herein are encountered in conjunction with construction, and/or if design and layout changes are initiated, IGES must be informed so that our recommendations can be reviewed and revised as deemed necessary.

5.2 EARTHWORK

5.2.1 General Site Preparation and Grading

Below proposed structures, fills, and man-made improvements, all vegetation, topsoil, debris and known undocumented fill soils 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 contained in this report have been complied with.

*not required where bedrock is exposed in the foundation subgrade

5.2.2 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.

5.2.3 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. Where surficial soil is encountered (expected largely on the western quarter of the project), Soil Type C is expected to predominate (loose sands and gravels). However, the majority of the site is expected to be underlain by shallow dolomite (hard rock). 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. Where dolomite is exposed, lay-back or shoring of the trench probably will not be required, except where adverse jointing/bedding patterns or other hazardous geologic conditions prevail. 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.

5.2.4 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. All structural fill should be 1 inch minus material when within 1 foot of any base coarse material. 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. In addition, proper grading should precede placement of fill, as described in the General Site Preparation and Grading subsection of this report.

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

5.2.5 Oversize Material

The majority of the 3.1-acre site consists of bedrock outcrops of dolomite. In addition, large boulders up to 24 inches are known to occur on the surface in the vicinity of the development; larger boulders may also be present within the colluvial soil. As such, development of the individual lots could 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 foundations.

5.2.6 Utility Trench Backfill

Utility trenches should be backfilled with structural fill in accordance with Section 6.2.4 of this report. 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, and 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.

5.3 FOUNDATION RECOMMENDATION

Subsurface conditions across the site vary, and may consist of bedrock, coarse colluvium, undocumented fill, or in limited cases more than one soil type may underlie a building footprint. The following sections are intended to address specific conditions that are anticipated for specific lots.

5.3.1 Bedrock Foundations

Lots 1 and 9 through 15 are expected to be founded entirely on dolomite bedrock. As such, we recommend that the footings for the proposed homes be founded *entirely* on competent bedrock. Bedrock/soil transition zones are not allowed. 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.

It should be noted that the bedrock at the site is expected to be very difficult to excavate (see Section 5.10, *Construction Considerations*).

5.3.2 Colluvium Foundations

Lots 6 and 7 are expected to be founded entirely on coarse natural colluvium deposits. As such, we recommend that the footings for the proposed homes be founded *entirely* on competent granular colluvium. It is possible that bedrock (e.g., Wasatch Formation conglomerate) may be encountered at depth; if encountered, the foundation excavation should be deepened such that all foundations bear on competent bedrock – bedrock/soil transition zones are not allowed. Shallow spread or continuous wall footings constructed entirely on competent colluvial soils may be proportioned utilizing a maximum net allowable bearing pressure of **3,500 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.

5.3.3 Undocumented Fill

Lot 4 is mapped within an area designated as potentially undocumented fill; regardless of whether these soils consist of a natural deposit or man-made, by observation the soils are generally loose and easy to excavate. As such, IGES recommends that the foundations for Lot 4 be underlain by a minimum of three feet of structural fill. Shallow spread or continuous wall footings constructed entirely on properly prepared structural fill may be proportioned utilizing a maximum net allowable bearing pressure of **2,200 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.

5.3.4 Transitions Zones

Lots 2, 3, 5, and 8 are mapped as being in a *transition zone*, e.g. part of the foundation will be on rock and part of the foundation will be on surficial soils. Founding a structure partly on bedrock and partly on soil will greatly increase the likelihood of long-term differential settlement damaging the home; therefore, IGES recommends that the homes be founded entirely on bedrock. If the footings are deepened such that they bear entirely on bedrock, then the recommendations presented in Section 5.3.1 apply.

Founding the home on bedrock may necessitate significant over-excavation, depending on the depth of surficial soils. For Lots 5 and 8, the depth of surficial soil is not expected to present a significant challenge to development; however, for Lots 2 and 3, the depth of surficial soil could be up to several feet deep. Therefore, for Lots 2 and 3, it may be more cost-effective to support that portion of the home *not supported by bedrock* with micropiles extending to bedrock.

As an alternative to deepening foundations or underpinning, the homes may be moved such that there is no bedrock underlying the footprint (this alternative is considered most applicable to Lots 2 and 3). If a home is moved away from bedrock, the recommendations presented in Section 5.3.3 may be followed. A second alternative would be to over-excavate both the bedrock and soils a minimum of three feet and replace with structural fill, such that *the entire structure is underlain by a uniform 3-foot thick fill blanket*, in which case the recommendations presented in Section 5.3.3 would apply.

5.3.5 Micropiles

Micropiles, if used for underpinning, should be designed by IGES or an engineer experienced in deep foundation design. *For planning purposes*, micropiles should conform 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.

5.3.6 Additional Recommendations

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. <u>Exception</u>: where the foundations will be poured directly on rock (dolomite), the minimum depth below nearest adjacent grade may be reduced to 24 inches.

5.4 SETTLEMENT

5.4.1 Static Settlement

Static settlements of properly designed and constructed conventional foundations, founded as described in Section 5.3, 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.

5.4.2 Dynamic Settlement

Dynamic settlement (or seismically-induced settlement) consists of dry dynamic settlement of unsaturated soils (above groundwater) and liquefaction-induced settlement (below groundwater). During a strong seismic event, seismically-induced settlement can occur within loose to moderately dense sandy soil due to reduction in volume during, and shortly after, an earthquake event. Settlement caused by ground shaking is often non-uniformly distributed, which can result in differential settlement.

Based on the subsurface conditions encountered, dynamic settlement arising from a MCE seismic event is expected to be negligible.

5.5 EARTH PRESSURES 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.

Table 5.5
Lateral Earth Pressure Coefficients

Condition	Level Backfill		2H:1V Backfill	
	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	—	—

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 5.5. 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. 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 wall), the at-rest condition should be used. These values should be used with an appropriate factor of safety against overturning and sliding. A value of 1.5 is typically used. Additionally, if passive resistance is calculated in conjunction with frictional resistance, the passive resistance should be reduced by ½.

5.6 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 (bedrock)** or **260 psi/inch (soil)** 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.

5.7 MOISTURE PROTECTION AND SURFACE DRAINAGE

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. Some home sites 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.

We 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 (if any) should be constructed to divert surface water off the pavement into storm drains, curb/gutter, or another suitable location.

The new homes are expected to be on-grade structures; however, for any subterranean components such as storage space or a mechanical room, IGES recommends a perimeter foundation drain be constructed in accordance with the International Residential Code (IRC).

5.8 PAVEMENT SECTION DESIGN

Based on our field reconnaissance, the parking lot is expected to expose bedrock at, or very near the pavement subgrade; this earth material will provide substantial support for the pavement section. Therefore, IGES recommends that the minimum pavement section per Weber County be used for the parking lot:

Table 5.8
Recommended Pavement Section – Parking Lot

Asphalt (in.)	Untreated Road Base (in.)	Sub Base (Granular Borrow) (in.)
3	6	8

The pavement section should be constructed on properly prepared subgrade or exposed competent bedrock. Alternative pavement section(s) may also be acceptable if they can provide equal or greater structural capacity to the section presented in Table 5.8, pending acceptance by Weber County (in particular, reduction or elimination of the granular borrow section with the use of geosynthetics).

Asphalt has been assumed to be a high stability plant mix and base course material composed of crushed stone with a minimum CBR of 70, granular borrow should have a minimum CBR of 30. Road base and granular borrow should be compacted to 95% of MDD as determined by ASTM D-1557 (Modified Proctor). Asphalt should be compacted to a minimum of 96 percent of the Marshall maximum density. Asphalt and aggregate base material should conform to local requirements. Subgrade should be scarified to a depth of 8 inches and compacted to 95% of MDD as determined by ASTM D-1557 (not required where bedrock is exposed). Positive drainage away from parking lot must be provided to minimize the potential for saturation of subgrade soils beneath constructed pavements.

Where Portland Cement Concrete (PCC) pavements are planned, such as near trash enclosures or other areas expected to support heavy truck traffic, we recommend a minimum of 6 inches PCC underlain by a minimum 6 inches of aggregate base course.

If conditions vary significantly from our stated assumptions (including stated traffic assumptions) IGES should be contacted so we can modify our pavement design parameters accordingly.

5.9 SOIL CORROSION POTENTIAL

Laboratory testing of soil samples obtained from nearby explorations during previously completed geotechnical work in 2012 (IGES, 2012b) indicated that the near-surface soil sample tested had a sulfate content of 127 ppm or less. Based on the subsurface conditions observed during our field work and the results of chemical testing in 2012, the prevailing earth materials 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.

Based on the subsurface conditions observed during our field work and the results of chemical testing in 2012, the on-site soils are considered *moderately corrosive* to ferrous metal. In addition, due to low soil pH (acidic soil chemistry) identified in soils throughout the project area, a corrosion engineer should also provide an assessment of any concrete that will be in contact with native soils.

5.10 CONSTRUCTION CONSIDERATIONS

5.10.1 Excavation Difficulty

Bedrock consisting of relatively hard dolomite is exposed over most of the surface within the project site. 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 hoe ram.

5.10.2 Over-Size Material

Most of the site consists of bedrock outcrop (surface exposures of dolomite); as such, development of most of the lots is expected to generate a substantial amount of over-size material (rocks larger than 6 inches in greatest dimension). Large rocks 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. Please refer to Figure A-2 for a map of bedrock exposures.

6.0 CLOSURE

6.1 LIMITATIONS

The recommendations presented in this report are based on limited field exploration, review of existing hazard studies and other geotechnical data, and our understanding of the proposed construction. The subsurface data used in the preparation of this report were obtained from the explorations made for this investigation. It is possible that variations in the soil and groundwater conditions could exist between and beyond the points explored. 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 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 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.

6.2 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 the construction. IGES staff or other qualified personnel should be on site to verify compliance with these recommendations. These tests and observations should include at a minimum the following:

- Observations and testing during site preparation, earthwork and structural fill placement.
- Consultation as may be required during construction.
- Quality control on concrete placement to verify slump, air content, and strength.
- Quality control and testing during placement and compaction of asphalt.

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 (801) 748-4044.

7.0 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, Preliminary Geotechnical Investigation, Powder Mountain Resort, Weber County, Utah, Project No. 01628-001, dated July 26, 2012.

IGES, Inc., 2012b, Design Geotechnical Investigation, Powder Mountain Resort, Weber County, Utah, Project No. 01628-003, dated November 9, 2012.

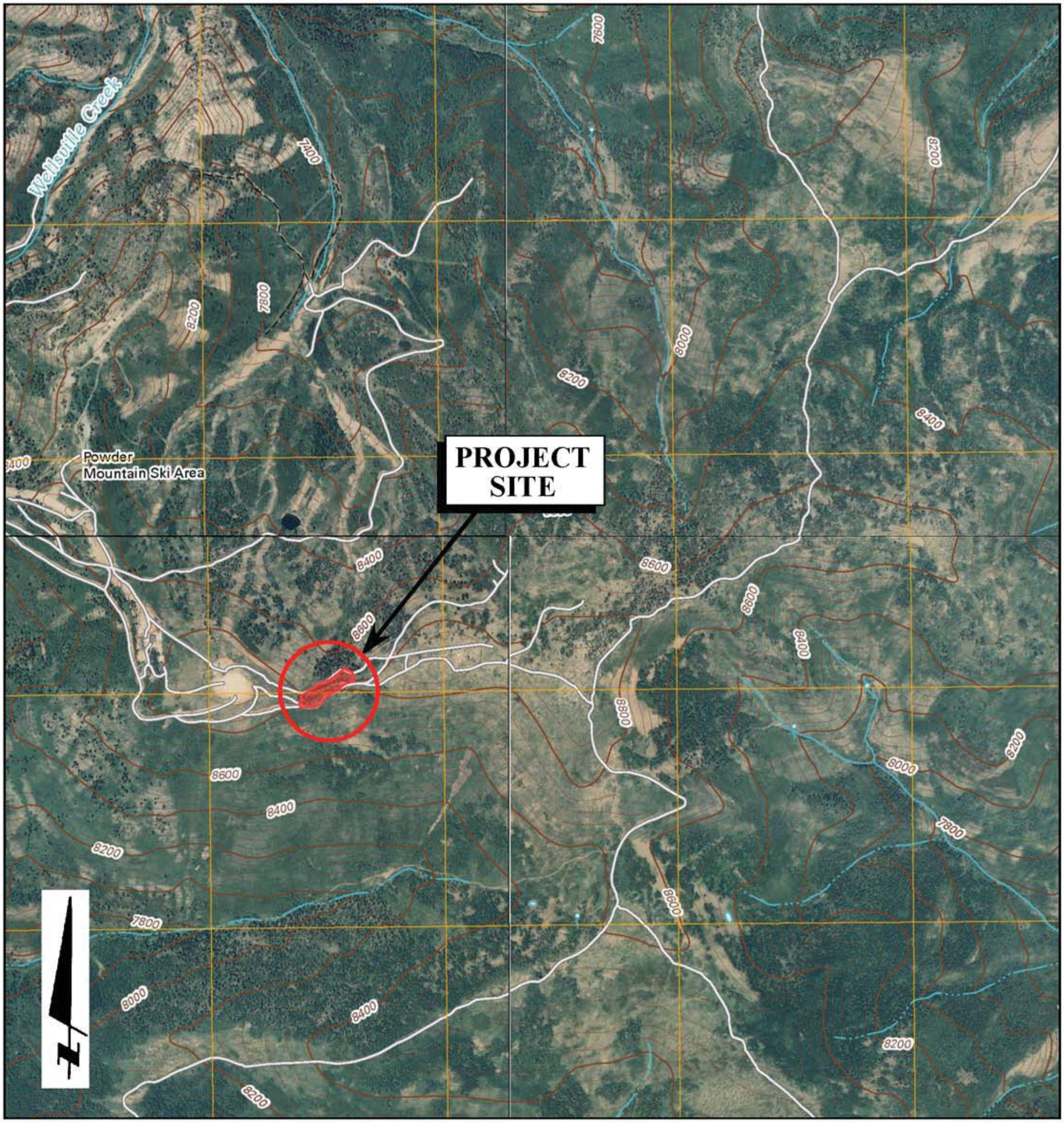
International Building Code [IBC], 2012, International Code Council, Inc.

PSI, 2012, Geophysical ReMi Investigation, Powder Mountain Resort, Phase 1A, Weber County, Utah, PSI Project No. 0710375, dated September 18, 2012.

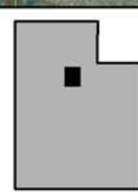
U.S. Geological Survey, 2012, U.S. *Seismic "Design Maps" Web Application*, site: https://geohazards.usgs.gov/secure/designmaps/us/application.php, site accessed on July 20, 2012.

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



BASE MAP:
USGS Huntsville, Browns Hole, James Peak and Sharp Mountain
7.5-Minute Quadrangle Topographic Maps (2011)



0 1000' 2000'
SCALE 1:24,000

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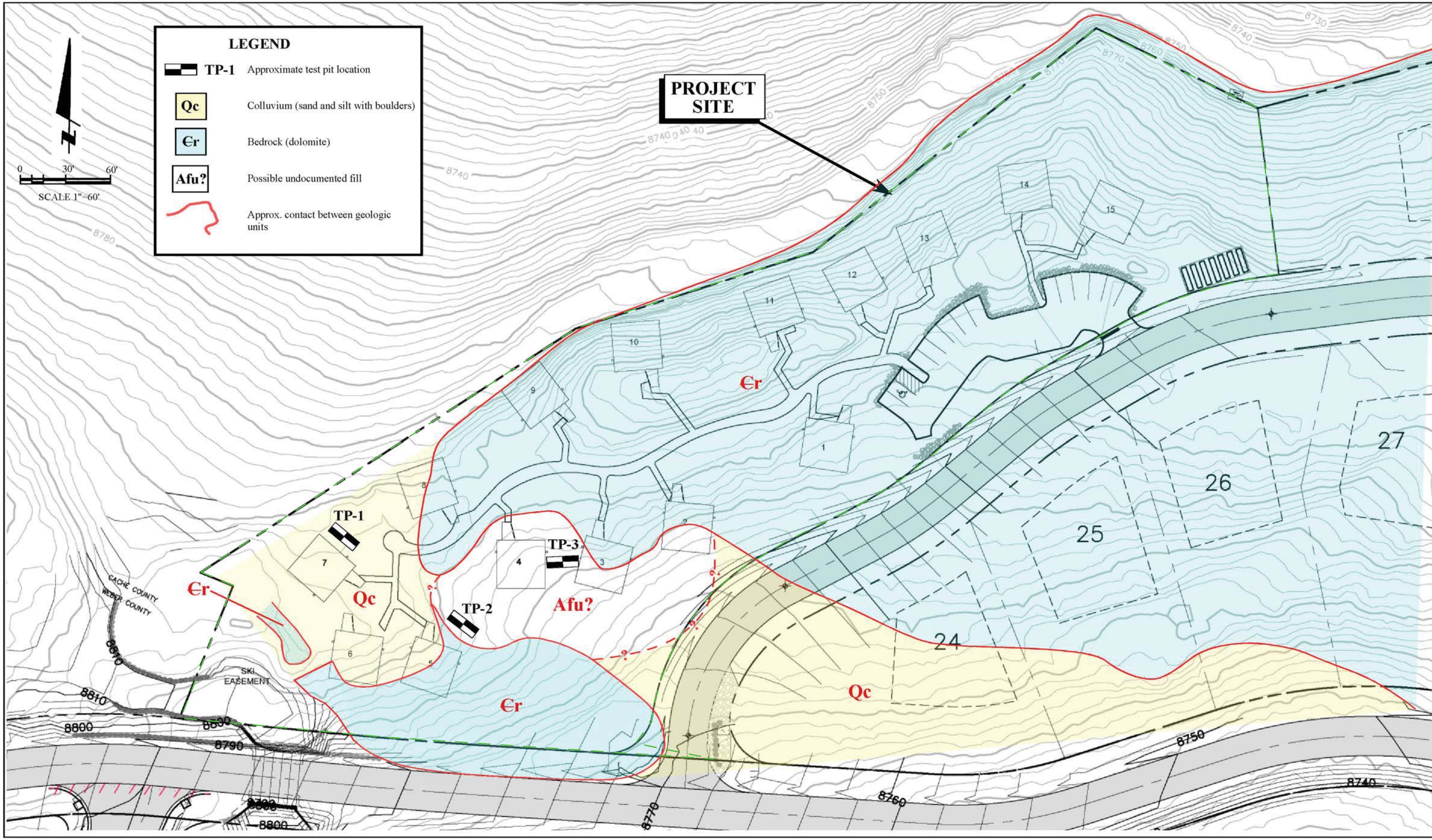
Geotechnical Investigation
The Ridge Nests Development
Powder Mountain Resort
Weber County, Utah

SITE VICINITY MAP

Figure

A-1



Basemap: Undated/uncredited 50-scale topgraphic map provided by Summit LLC

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Geotechnical Investigation
The Ridge Nests Development
Powder Mountain Resort
Weber County, Utah

GEOTECHNICAL MAP

Figure A-2

DATE		
STARTED:	9/5/14	
COMPLETED:	9/5/14	
BACKFILLED:	9/5/14	

Geotechnical Investigation
The Ridge Nests
Powder Mountain Resort
Weber & Cache Counties, Utah

Project Number 01628-008

IGES Rep: DAG
Rig Type: 315C

TEST PIT NO:
TP-1

Sheet 1 of 1

LOCATION

LATITUDE LONGITUDE ELEVATION 8,804

near Lot 7

ELEVATION	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					Topsoil - Clayey SAND, dark brown, loamy appearance, abundant roots, about 6 inches thick, rocky						
					SC	@ ½' **Colluvium (Qc)** Clayey SAND with abundant cobbles and boulders, hard/dense, mottled, moist, grayish brown, difficult to excavate						
8800						@ 3½' **Wasatch Formation (Tw)** Conglomerate, well-cemented, hard, highly weathered, rounded boulders and cobbles in a reddish-brown clayey matrix, very difficult to excavate, boulders to 2½ feet						
	5					Refusal at 5 feet No groundwater						
8795						Bottom of Test Pit @ 5 Feet						

LOG OF TEST PITS (A) -(4 LINE HEADER W ELEV) 01628-008.GPJ IGES.GDT 9/14/14

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

WATER LEVEL
- MEASURED
- ESTIMATED

NOTES:

FIGURE
A - 3

DATE		
STARTED:	9/5/14	
COMPLETED:	9/5/14	
BACKFILLED:	9/5/14	

Geotechnical Investigation
The Ridge Nests
Powder Mountain Resort
Weber & Cache Counties, Utah

Project Number 01628-008

IGES Rep: DAG

Rig Type: 315C

TEST PIT NO: **TP-2**

Sheet 1 of 1

LOCATION

LATITUDE | LONGITUDE | ELEVATION 8,798

east of Lot 5

DEPTH: ELEVATION	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, clayey, dark brown, well-rounded gravel and cobble, moist, poorly developed						
						@ ½' **Colluvium (Qc)** Sandy Lean Clay, stiff, low plasticity, reddish-brown, moist, rounded cobbles to 6 inches, easy to excavate, exposed electrical wires at bottom of unit						
8795												
						@ 4' Silty SAND, medium dense, about 20% non-plastic fines, fine-grained, moderate yellowish brown, moist, occasional rounded gravel and cobble to 4 inches, iron staining						
	5											
8790						Total depth 7 feet No groundwater Possible undocumented fill (Afu?), but not substantiated Bedrock outcrop 10 feet away from test pit Bottom of Test Pit @ 7 Feet						

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

WATER LEVEL
- MEASURED
- ESTIMATED

NOTES:

FIGURE

A - 4

DATE	
STARTED:	9/5/14
COMPLETED:	9/5/14
BACKFILLED:	9/5/14

Geotechnical Investigation
The Ridge Nests
Powder Mountain Resort
Weber & Cache Counties, Utah

Project Number 01628-008

IGES Rep: DAG
Rig Type: 315C

TEST PIT NO:
TP-3
Sheet 1 of 1

LOCATION

LATITUDE LONGITUDE ELEVATION 8,790

west of Lot 3

ELEVATION	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, thin (3" to 6"), poorly developed, abundant rootlets, sandy						
						@ ½' **Colluvium (Qc)** Silty SAND, loose to medium dense, fine-grained, moderate brown, moist, occasional rounded cobbles, easy to excavate						
						@ 4' abundant dolomite fragments, angular, appears as possible rubble, within a sandy matrix, undocumented fill?, easy to excavate						
8785	5											
8780												

Total depth 7½ feet
No groundwater
Possible undocumented fill (Afu?)
Bedrock exposure 15 feet away from test pit

Bottom of Test Pit @ 7.5 Feet

LOG OF TEST PITS (A) -(4 LINE HEADER W ELEV) 01628-008.GPJ IGES.GDT 9/14/14

IGES®

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

WATER LEVEL
- MEASURED
- ESTIMATED

NOTES:

FIGURE

A - 5

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

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.

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

STRATIFICATION

DESCRIPTION	THICKNESS
SEAM	1/16 - 1/2"
LAYER	1/2 - 12"

DESCRIPTION	THICKNESS
OCCASIONAL	ONE OR LESS PER FOOT OF THICKNESS
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.

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Key to Soil Symbols and Terminology

Figure A-6

IGES, Inc. Project No.:01628-008

APPENDIX B

USGS 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	$\bar{v}_s$	$\bar{N}$ 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* ≥ 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 = B and S_S = 0.826 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 = B and S_1 = 0.274 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): $S_{M1} = F_v S_1 = 1.000 \times 0.274 = 0.274$ g

Section 1613.3.4 — Design spectral response acceleration parameters

Equation (16-39): $S_{DS} = ⅔ S_{MS} = ⅔ \times 0.826 = 0.551$ g

Equation (16-40): $S_{D1} = ⅔ S_{M1} = ⅔ \times 0.274 = 0.183$ 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_{DS}	RISK CATEGORY		
	I or II	III	IV
$S_{DS} < 0.167g$	A	A	A
$0.167g \le S_{DS} < 0.33g$	B	B	C
$0.33g \le S_{DS} < 0.50g$	C	C	D
$0.50g \le S_{DS}$	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_{D1}	RISK CATEGORY		
	I or II	III	IV
$S_{D1} < 0.067g$	A	A	A
$0.067g \le S_{D1} < 0.133g$	B	B	C
$0.133g \le S_{D1} < 0.20g$	C	C	D
$0.20g \le S_{D1}$	D	D	D

For Risk Category = I and S_{D1} = 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 ≡ “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

USGS 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_1 = 0.274 g	S_{M1} = 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.