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

IGES Project No. 01628-008

July 15, 2016

Prepared for:

Summit Holding Group

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1.0 INTRODUCTION

1.1 PURPOSE AND SCOPE OF WORK

This report presents the results of a geotechnical and geologic hazard 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. We have also evaluated the site for the presence of geologic hazards.

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.

This report is largely based on the original geotechnical report provided by IGES dated September 16, 2014. This referenced report was strictly geotechnical in nature and relied on the Western

Geologic report (2012) to assess geologic hazards; however, subsequent to submission of the report, review comments from Weber County indicated that the Western Geologic report was reconnaissance-level and did not fully assess geologic hazards in compliance with Weber County Geologic Hazard Ordinance. Therefore, in response to several rounds of comments, both geologic and geotechnical in nature, geologic hazards were fully addressed by IGES in a series of response letters, and were ultimately accepted by the Weber County reviewers. The purpose of this new report is to provide a single, complete report that addresses both the geotechnical aspects of the project and the geologic hazard aspects. To this end, the main body of the report remains effectively unchanged from the original geotechnical report, however the subsequent review response letters (which largely address geologic hazards) are included in Appendix C. Consequently, this single report meets the requirements for Weber County for both geotechnical investigations and geologic hazard evaluations.

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 reconnaissance-level report (Western Geologic, 2012). 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. In addition, IGES has geologically mapped the Ridge Nests property and adjacent areas to assess the site for the presence of potential geologic hazards. Based on the geologic evidence obtained during our site reconnaissance, and the slope stability assessment presented in Appendix C, the following conclusions are made:

1. The stability of the slopes are not adversely impacted by the geologic, stratigraphic, or hydrologic conditions observed.

2. There are no evident potential on-site or off-site geologic hazards that are likely to adversely affect the subject property, and the site is considered suitable for development from a geologic hazards standpoint.

3. The site is considered suitable for development from a geotechnical perspective, provided the recommendations presented in this report are incorporated into the design and construction of the project.

It should be noted that much of the geologic hazard field work and related analysis and slope stability was performed in response to review comments regarding our original geotechnical report for the Ridge Nests project (IGES, 2014). All subsequent geotechnical and geologic work completed after the initial 2014 report are included in Appendix C of this report, and constitute data to document and substantiate the completion of a geologic hazard study in compliance with Weber County requirements.

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. Exception: 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 metal 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.

IGES, Inc., 2014, Geotechnical Investigation, The Ridge Nests Development, Powder Mountain Resort, Weber and Cache Counties, Utah, Project No. 01628-008, dated September 16, 2008.

IGES, Inc., 2015a, Response to Review Comments, Geotechnical Investigation, The Ridge Nests Development, Powder Mountain Resort, Weber and Cache Counties, Utah, Project No. 01628-008, dated April 7, 2015.

IGES, Inc., 2015b, Response to Review Comments – Geology, Geotechnical Investigation, The Ridge Nests Development, Powder Mountain Resort, Weber and Cache Counties, Utah, Project No. 01628-008, dated September 23, 2015.

IGES, Inc., 2015c, Response to Additional Review Comments – Geology, Geotechnical Investigation, The Ridge Nests Development, Powder Mountain Resort, Weber and Cache Counties, Utah, Project No. 01628-008, dated November 4, 2015.

IGES, Inc., 2015d, Response to Review Comments – Geotechnical Engineering, Geotechnical Investigation, The Ridge Nests Development, Powder Mountain Resort, Weber and Cache Counties, Utah, Project No. 01628-008, dated December 4, 2015.

IGES, Inc., 2015e, Response to Additional Review Comments – Geology, Geotechnical Investigation, The Ridge Nests Development, Powder Mountain Resort, Weber and Cache Counties, Utah, Project No. 01628-008, dated December 11, 2015.

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

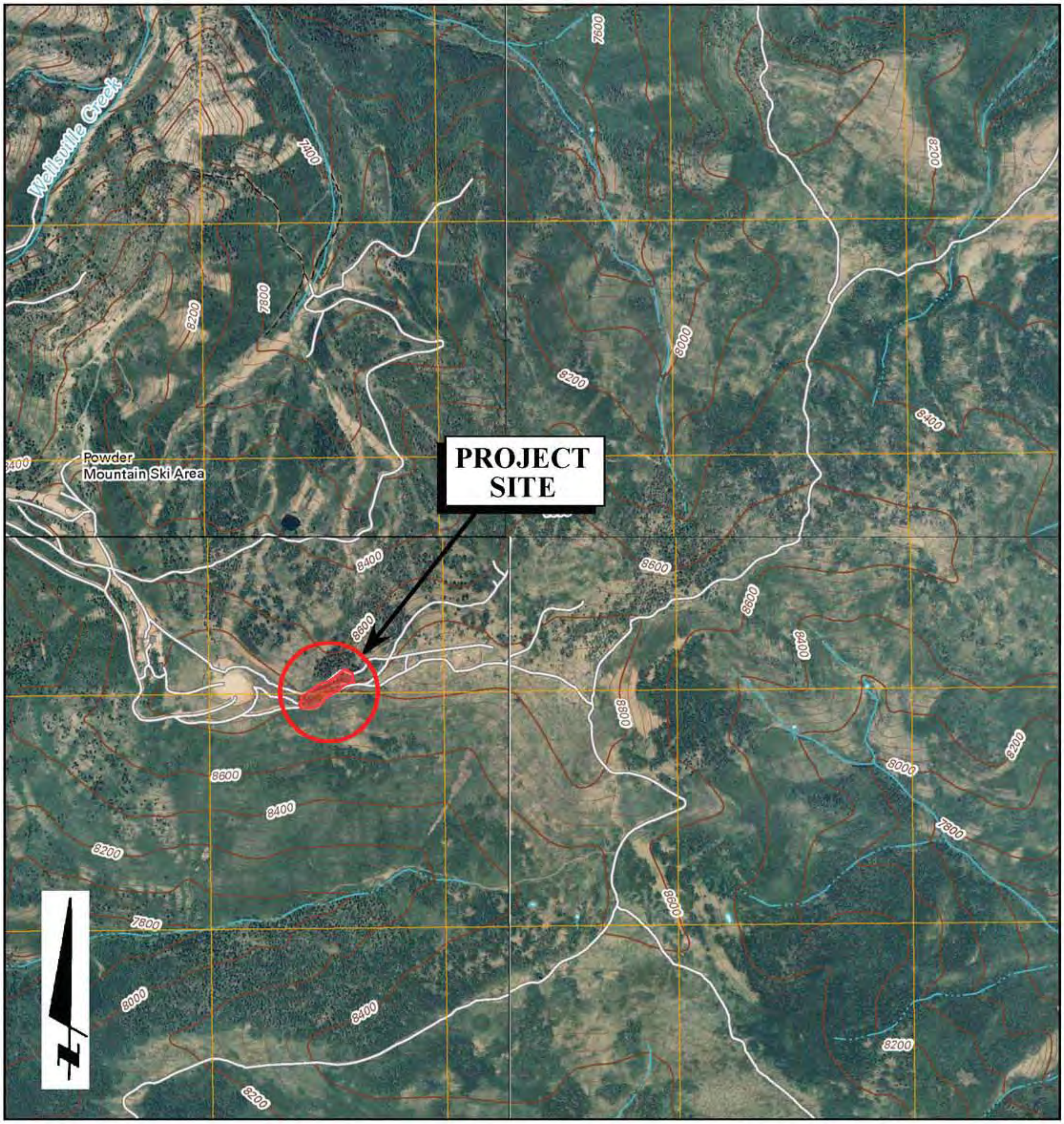
PSI, 2012, Geophysical ReMi Investigation, Powder Mountain Resort, Phase 1A, Weber County, Utah, PSI Project No. 0710375, dated September 18, 2012.

REFERENCES (CONT.)

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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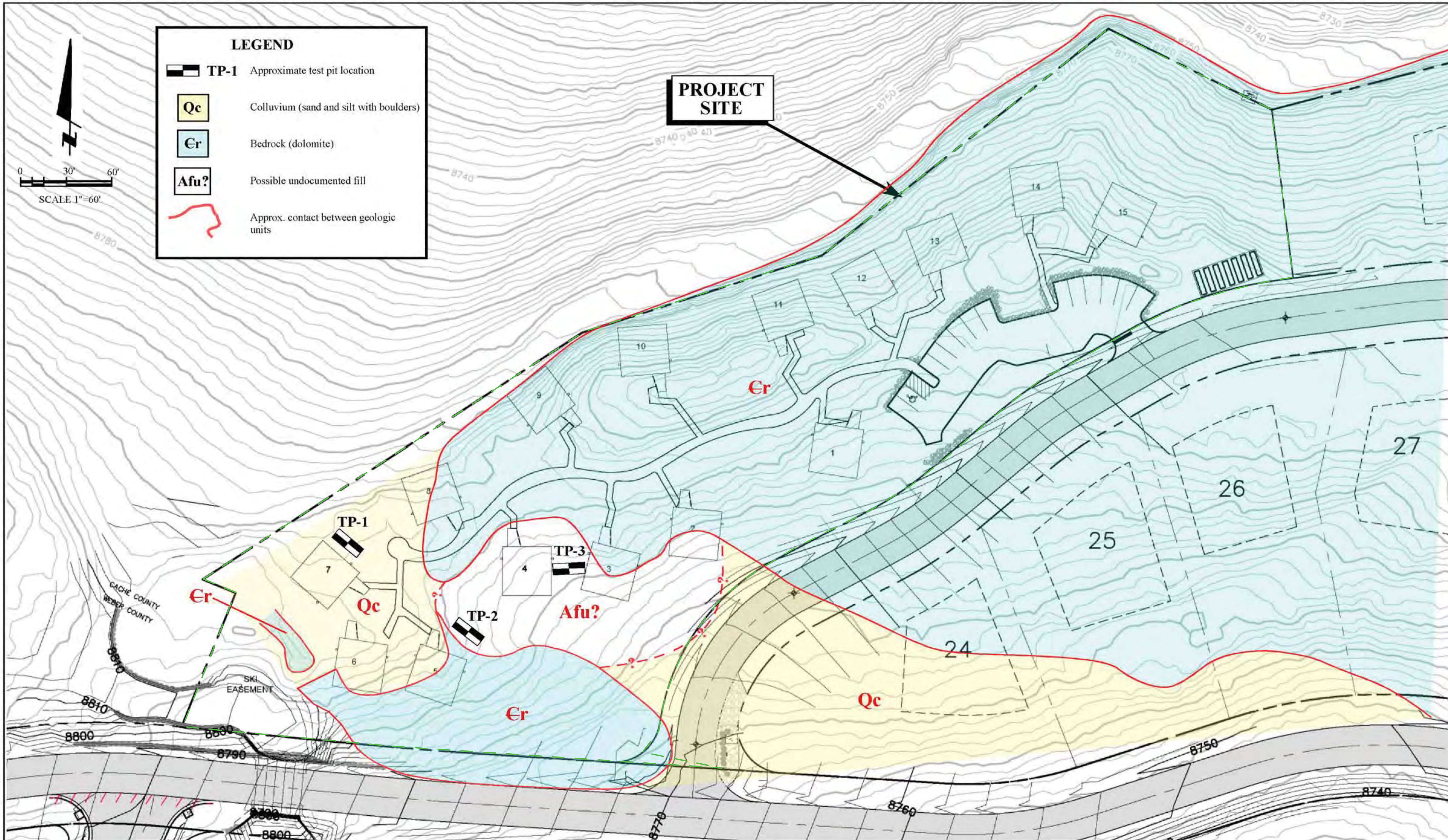
Project No. 01628-008

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						
						@ 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						
8800												
	5					Refusal at 5 feet No groundwater						
						Bottom of Test Pit @ 5 Feet						
8795												

IGES

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

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						

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

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, 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											
						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						
8780												

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

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

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.

APPARENT / RELATIVE DENSITY - COARSE-GRAINED SOIL

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

CONSISTENCY - FINE-GRAINED SOIL

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

IGES®

Key to Soil Symbols and Terminology

Figure A-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				
	$S_S \le 0.25$	$S_S = 0.50$	$S_S = 0.75$	$S_S = 1.00$	$S_S \ge 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	See Section 11.4.7 of ASCE 7				

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				
	$S_1 \le 0.10$	$S_1 = 0.20$	$S_1 = 0.30$	$S_1 = 0.40$	$S_1 \ge 0.50$
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

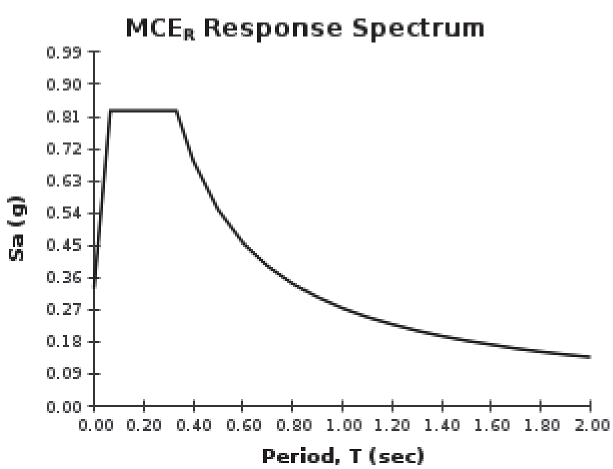
User–Specified Input

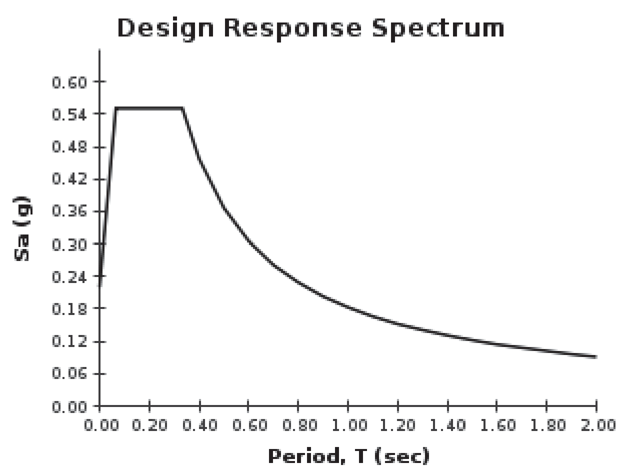
Report Title	Lot 34R Tue August 12, 2014 00:42:37 UTC
Building Code Reference Document	2012 International Building Code (which utilizes USGS hazard data available in 2008)
Site Coordinates	41.3696°N, 111.7579°W
Site Soil Classification	Site Class B – "Rock"
Risk Category	I/II/III

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.





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.

APPENDIX C

IGES®
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April 7, 2015

Summit Powder Mountain
c/o Ms. Andrea Milner
3632 North Wolf Creek Drive
Eden, Utah 84310

IGES Project No. 01628-008

Subject: Response to Review Comments
Geotechnical Investigation
The Ridge Nests Development
Powder Mountain Resort
Weber and Cache Counties, Utah

Reference: IGES, Inc., 2014, Geotechnical Investigation, The Ridge Nests Development, Powder Mountain Resort, Weber and Cache Counties, Utah Project No. 01628-008, dated September 16, 2014

Ms. Milner:

As requested, IGES has prepared the following response to recent review comments regarding the referenced geotechnical report for the Ridge Nests development, part of the larger Powder Mountain Resort expansion project in Weber County, Utah. The review comments to be addressed were prepared by Taylor Geotechnical and were posted on the Weber County website on November 19, 2014. For convenience, the review comments will be presented first, followed by our response.

Comment No. 1
"Please have IGES provide their hand calculations that substantiate the allowable bearing capacity and settlement analysis."

Response to Comment No. 1
The bearing capacity and settlement calculations are attached. For material properties, IGES has made the following conservative assumptions:

- Coarse natural colluvium: friction angle = 38 degrees, Es ~ 350 ksf
- Fine sandy structural fill soils: friction angle = 34 degrees, Es ~ 100 ksf
- In-place dolomite: intact uniaxial compressive strength ~ 1,000 ksf (lower-bound of typically reported values)

Unit weight of the colluvium and the sandy alluvial soils/structural fill has been approximated as 125 pcf.

Based on the Hoek-Brown criterion, the equivalent soil strength of the rock (Mohr-Coulomb Fit) is estimated to be: friction angle = 14 degrees, cohesion = 9.5 ksf. These values were estimated using RocLab1 software, available as a free download from Rocscience. In consideration of these estimated equivalent values, settlement of the rock is expected to be negligible under the anticipated relatively light loads of a residential structure (e.g., a small cabin or cottage).

Comment No. 2

"*Confirm that the recommendations for Lot 4 is three feet of compacted structural fill over potentially undocumented fill for foundation support.*"

Response to Comment No. 2

With a 3-foot over-excavation below the footings, the total over-excavation below existing ground will be on the order of 6½ feet. This is expected to remove most, if not all, deleterious earth materials below the foundation.

Per our recommendations in the referenced geotechnical report, IGES should observe the foundation subgrade prior to placement of structural fill, steel or concrete. The purpose of this recommendation is to allow IGES to qualitatively assess the condition of the subgrade and to identify adverse conditions that could impact the structure (e.g., soft, loose soil, undocumented fill, rock/soil transition zones, etc.). This recommendation is particularly relevant to Lot 4, as the lot is entirely within an area that may be undocumented fill, but is at the very least in an area of a natural soil deposit that is relatively loose and potentially compressible. The reviewer correctly alludes to the implications of building a structure on undocumented fill, which is risky and is considered outside of the *standard of care*.

If soft, loose, or otherwise deleterious earth materials are identified by IGES within the foundation subgrade, additional over-excavation will be required.

Closure

We appreciate the opportunity to provide you with our services. If you have any questions please contact the undersigned at your convenience (801) 748-4044.

Respectfully Submitted,
IGES, Inc.

PROFESSIONAL ENGINEER
No. 6370734
DAVID A.
GLASS
04-07-15
ELECTRONIC SEAL
STATE OF UTAH

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

Attachments:

Bearing Capacity, Settlement, and Hoek-Brown Rock Strength Calculations

Allowable Bearing Capacity Calculations

Modified Meyerhof (1963)

IGES Project No.: 01628-008
Date: 4/7/2015
Model: Coarse Colluvium

c	0	psf
ϕ	38	deg.
γ	125	pcf
B	1.67	ft.
D	3.5	ft.
L	20	ft.
β	0	deg.
FS	3	
FS_{shear}	1.5	

c cohesion
ϕ friction angle
γ wet unit weight of soil
B width of footing
D depth of footing
β inclination of the load on the foundation with respect to the vertical
L length of footing

Note[1]: if round footing, L=B=diameter of footing
Note[2]: you may want to neglect depth factors for shallow foundations

Bearing Capacity Factors

Nq	48.9	(Reissner, 1924)
Nc	61.4	(Prandtl, 1921)
Ny	78.0	(Vesic, 1973)

Modified Bearing Capacity Factors (Shear)

c_d	0	psf
ϕ_d	27.5	deg.
Nq'	14.0	
Nc'	24.9	
Ny'	15.6	

Shape Factors (De Beer, 1970)

F_{cs}	1.1
F_{qs}	1.1
F_{ys}	0.97

Depth Factors (Hansen, 1970)

F_{cd}	1.5
F_{qd}	1.3
F_{yd}	1

Incliniation Factors (Meyerhof 1963; Hanna and Meyerhof 1981)

F_{ci}	1.00
F_{qi}	1.00
F_{yi}	1.00

Bearing Capacity

gross			net		
q_u	q_{all}	$q_{all(shear)}$	q_u	q_{all}	$q_{all(shear)}$
36,602	12,201	9,766	36,164	12,055	9,328

$q_u = cNcF_{cs}F_{cd}F_{ci} + \gamma DNqF_{qs}F_{qd}F_{qi} + 0.5\gamma BNyF_{ys}F_{yd}F_{yi}$

$q_{all} = qu/FS$

$q_{all(shear)} = c_dNc'F_{cs}F_{cd}F_{ci} + \gamma DNq'F_{qs}F_{qd}F_{qi} + 0.5\gamma BNy'F_{ys}F_{yd}F_{yi}$ where $c_d = c/FS_{shear}$ and $\phi_d = \tan^{-1}(\tan(\phi/FS_{shear}))$

Note: net values do not take into account removal of existing overburden (Dγ)

Figure 1

Allowable Bearing Capacity Calculations

Modified Meyerhof (1963)

IGES Project No.: 01628-008
Date: 4/7/2015
Model: Sandy structual fill

c	0	psf
ϕ	34	deg.
γ	125	pcf
B	1.67	ft.
D	3.5	ft.
L	20	ft.
β	0	deg.
FS	3	
FS_{shear}	1.5	

c cohesion
ϕ friction angle
γ wet unit weight of soil
B width of footing
D depth of footing
β inclination of the load on the foundation with respect to the vertical
L length of footing

Note[1]: if round footing, L=B=diameter of footing
Note[2]: you may want to neglect depth factors for shallow foundations

Bearing Capacity Factors

Nq	29.4	(Reissner, 1924)
Nc	42.2	(Prandtl, 1921)
Ny	41.1	(Vesic, 1973)

Modified Bearing Capacity Factors (Shear)

c_d	0	psf
ϕ_d	24.2	deg.
Nq'	9.8	
Nc'	19.6	
Ny'	9.7	

Shape Factors (De Beer, 1970)

F_{cs}	1.1
F_{qs}	1.1
F_{ys}	0.97

Depth Factors (Hansen, 1970)

F_{cd}	1.5
F_{qd}	1.3
F_{yd}	1

Incliniation Factors (Meyerhof 1963; Hanna and Meyerhof 1981)

F_{ci}	1.00
F_{qi}	1.00
F_{yi}	1.00

Bearing Capacity

gross			net		
q_u	q_{all}	$q_{all(shear)}$	q_u	q_{all}	$q_{all(shear)}$
21,762	7,254	6,857	21,325	7,108	6,420

$q_u=cNcF_{cs}F_{cd}F_{ci}+\gamma DNqF_{qs}F_{qd}F_{qi}+0.5\gamma BNyF_{ys}F_{yd}F_{yi}$

$q_{all}=qu/FS$

$q_{all(shear)}=c_dNc'F_{cs}F_{cd}F_{ci}+\gamma DNq'F_{qs}F_{qd}F_{qi}+0.5\gamma BNy'F_{ys}F_{yd}F_{yi}$ where $c_d=c/FS_{shear}$ and $\phi_d=\tan^{-1}(\tan(\phi/FS_{shear}))$

Note: net values do not take into account removal of existing overburden (Dγ)

Figure 2

Static Settlement Calculations
Simplified Schmertmann Method

Coarse Colluvium

For continuous footings (L/B≥10)

$$\delta = \frac{C_1 C_2 C_3 (q - \sigma'_{zD})(2I_{\epsilon p} + 0.1)B}{E_s}$$

For square and circular foundations (L/B=1)

$$\delta = \frac{C_1 C_2 C_3 (q - \sigma'_{zD})(I_{\epsilon p} + 0.025)B}{E_s}$$

$$C_1 = 1 - 0.5\left(\frac{\sigma'_{zD}}{q - \sigma'_{zD}}\right)$$

$$C_2 = 1 + 0.2 \log\left(\frac{t}{0.1}\right)$$

$$C_3 = 1.03 - \frac{0.03L}{B} \geq 0.73$$

$$I_{\epsilon p} = 0.5 + 0.1\sqrt{\frac{q - \sigma'_{zD}}{\sigma'_{zp}}}$$

where…

δ = total static settlement (inches)
q = bearing pressure (psf)
D = depth to bottom of footing measured from original grade (ft)
σ'_{zD} = vertical effective stress at depth D below the ground surface (psf)
σ'_{zp} = initial vertical effective stress at depth of peak strain influence factor*
$I_{\epsilon p}$ = peak strain influence factor (no units)
B = width of footing (ft)
L = length of footing (ft)
E_s = equivalent modulus of elasticity in soil layer (ksf)
C_1 = depth factor
C_2 = secondary creep factor
C_3 = shape factor (equals 1 for square and circular foundations)
t = time since application of load (yr, typically taken as a 50-year design life)

*(for square and circular foundations, compute at a depth of D+B/2 below the ground surface; for continuous footings (L/B>10), compute at a depth of D+B)

Input:

q =	3,500	psf
D =	3.5	ft (assume zero for engineered fill)
B =	1.7	ft
L =	50	ft
E_s =	300	ksf (conservative estimate)
t =	50	years
Y' =	125	unit weight, pcf

Calculated Values:

σ'_{zD} =	0	psf
σ'_{zp} =	650	psf
C_1 =	1	
C_2 =	1.54	
C_3 =	0.73	
$I_{\epsilon p}$ =	0.73	

δ = $\boxed{0.42}$ inches

Static Settlement Calculations
Simplified Schmertmann Method

structural fill

For continuous footings (L/B≥10)

$$\delta = \frac{C_1 C_2 C_3 (q - \sigma'_{zD})(2I_{\epsilon p} + 0.1)B}{E_s}$$

For square and circular foundations (L/B=1)

$$\delta = \frac{C_1 C_2 C_3 (q - \sigma'_{zD})(I_{\epsilon p} + 0.025)B}{E_s}$$

$$C_1 = 1 - 0.5\left(\frac{\sigma'_{zD}}{q - \sigma'_{zD}}\right)$$

$$C_2 = 1 + 0.2 \log\left(\frac{t}{0.1}\right)$$

$$C_3 = 1.03 - \frac{0.03L}{B} \geq 0.73$$

$$I_{\epsilon p} = 0.5 + 0.1\sqrt{\frac{q - \sigma'_{zD}}{\sigma'_{zp}}}$$

where…

δ = total static settlement (inches)
q = bearing pressure (psf)
D = depth to bottom of footing measured from original grade (ft)
σ'_{zD} = vertical effective stress at depth D below the ground surface (psf)
σ'_{zp} = initial vertical effective stress at depth of peak strain influence factor*
$I_{\epsilon p}$ = peak strain influence factor (no units)
B = width of footing (ft)
L = length of footing (ft)
E_s = equivalent modulus of elasticity in soil layer (ksf)
C_1 = depth factor
C_2 = secondary creep factor
C_3 = shape factor (equals 1 for square and circular foundations)
t = time since application of load (yr, typically taken as a 50-year design life)

*(for square and circular foundations, compute at a depth of D+B/2 below the ground surface; for continuous footings (L/B>10), compute at a depth of D+B)

Input:

q =	2,200	psf
D =	0	ft (assume zero for engineered fill)
B =	1.7	ft
L =	50	ft
E_s =	100	ksf (conservative estimate)
t =	50	years
Y' =	125	unit weight, pcf

Calculated Values:

σ'_{zD} =	0	psf
σ'_{zp} =	212.5	psf
C_1 =	1	
C_2 =	1.54	
C_3 =	0.73	
$I_{\epsilon p}$ =	0.82	

δ = $\boxed{0.88}$ inches

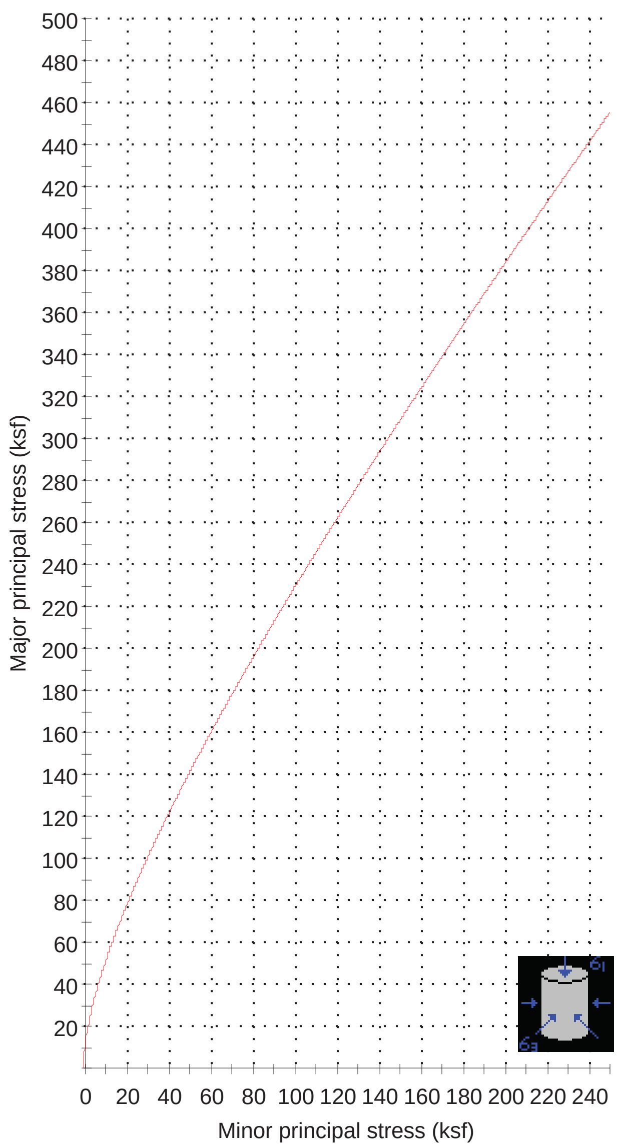
Analysis of Rock Strength using RocLab

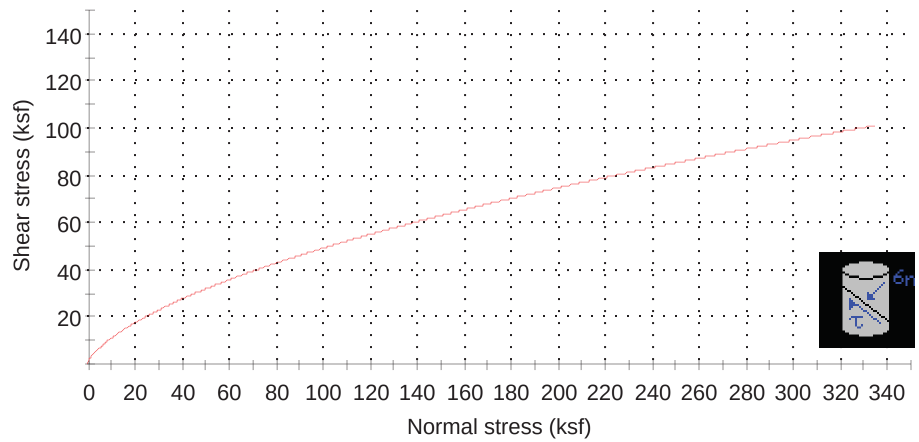
Hoek-Brown Classification
intact uniaxial comp. strength (sigci) = 1000 ksf
GSI = 45   mi = 9   Disturbance factor (D) = 1
intact modulus (Ei) = 240000 ksf

Hoek-Brown Criterion
mb = 0.177   s = 0.0001   a = 0.508

Mohr-Coulomb Fit
cohesion = 20.880 ksf   friction angle = 14.38 deg

Rock Mass Parameters
tensile strength = -0.590 ksf
uniaxial compressive strength = 9.491 ksf
global strength = 53.814 ksf
deformation modulus = 12165.97 ksf





Intermountain GeoEnvironmental Services, Inc.
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September 23, 2015

Summit Powder Mountain
c/o Ms. Andrea Milner
3632 North Wolf Creek Drive
Eden, Utah 84310

IGES Project No. 01628-008

Subject: Response to Review Comments - Geology
Geotechnical Investigation
The Ridge Nests Development
Powder Mountain Resort
Weber and Cache Counties, Utah

Ms. Milner:

As requested, IGES has prepared the following response to recent review comments regarding the referenced geotechnical report for the Ridge Nests development, part of the larger Powder Mountain Resort expansion project in Weber County, Utah. The review comments to be addressed were prepared by Simon Associates LLC (SA) in a letter dated August 18, 2015. The review letter by SA was intended to address Lot 13; however, in consideration that the comments by SA would also be applicable to several other lots, it is the intention of IGES to address the comments with respect to the entire Ridge Nests development. For convenience, the review comments will be presented first, followed by our response.

Comment No. 1

"In accordance with the recommendations provided in the Western Geologic (2012) development report, SA recommends Weber County request IGES perform a slope stability analysis as stipulated in the Geologic Hazard Study for the development (Western Geologic, 2012), since the slope at the building envelope is greater than 20%."

Response to Comment No. 1

The global stability of the slope was modeled using gSTABL7 slope stability software. Bishop's Method and Janbu's Simplified method was used to model the slope, as appropriate. For our analysis, we have assessed two representative sections, Section A-A' and Section B-B', illustrated on Plate 1 (*Geologic Map*) and the *Geologic Cross-Sections*, Figure 1, attached. Calculations for stability were developed by searching for the minimum factor-of-safety for both a circular-type failure and a block-type (translational) failure. For the circular analysis model, arcuate failure surfaces and homogenous earth materials were assumed. For the block analysis, anisotropic strength parameters in the bedrock was assumed, based on the apparent dip of bedding. A minimum static factor-of-safety of 1.5 and seismic factor-of-safety of 1.0 (global stability) was considered acceptable for this project considering the available information and design assumptions.

The prevailing earth materials on the north side of the development, which forms the steepest part of the site, consist of relatively competent, moderately weathered dolomite. The software package RocLab (V. 1.033), which is based on the Hoek-Brown failure Criterion (1997) was utilized to estimate equivalent strength parameters (friction angle and cohesion) to be used in conventional limit-equilibrium slope stability software. Input parameters utilized to estimate reasonable strength parameters were as follows:

- Uniaxial Compressive Strength: 1,500 ksf
- GSI: 45 (geologic strength index)
- Mi Value: 9 (intact rock parameter)
- D: 0.7 (disturbance factor)
- MR: 425 (Modulus Ratio, used to estimate the intact rock deformation modulus, Ei)

Based on these input parameters, RocLab indicates an equivalent cohesion of 44.844 ksf and a friction angle of 20.1 degrees for the dolomite. For our analysis, IGES has conservatively reduced the estimated equivalent cohesion by approximately 20% to 35 ksf. For our anisotropic analysis, strength along bedding and/or jointing has been estimated to have a friction angle of 42 degrees and a cohesion of zero. The output file for RocLab is attached.

The surficial unit described on the geologic map as Qc-sw is undifferentiated colluvium and slope wash. This material is generally very coarse and bouldery; constituents generally have a moderate degree of angularity. As such, the strength of this material has been modeled as having a friction angle of 42 degrees and a cohesion of zero.

For the seismic (pseudo-static) assessment of the slopes, the seismic coefficient k_h is modeled as equal to 50% of the peak ground acceleration (PGA) resulting from a MCE seismic event (2PE50). From our referenced geotechnical report, the PGA resulting from a 2PE50 seismic event is taken as 0.33g. Therefore, for seismic analysis we have adopted a seismic coefficient of 0.165g.

Based on our analysis, the global stability of the north-facing natural slope meets the minimum factors-of-safety of 1.5 and 1.0 for static and seismic conditions, respectively. The results of the global stability analyses are attached.

Comment No. 2

"*Figure A-2, Geotechnical Map, of the September 16, 2014 IGES report depicts "…the relative locations of the various geologic units…" described in the September 16, 2014, IGES report. SA recommends Weber County request IGES:*

a. Include, for a reasonable distance, geologic units of adjacent properties.

b. Evaluate whether any potential off-site geologic hazards may impact the subject property; the evaluation should be completed under the direction of an engineering geologist."

Response to Comment No. 2a

A geologic investigation of Lot 13 and the immediate area surrounding the Ridge Nests subdivision was conducted by an IGES engineering geologist between August 26 and 27, 2015. Plate 1 (*Geologic Map)* is an updated, expanded version of the original Figure A-2, *Geotechnical Map*, from the IGES geotechnical report. The geologic mapping has been extended to several hundred feet in all directions from the original map, and minor modifications to the original geologic contacts have been made based upon the findings of the investigation. Additionally, bedding and jointing attitudes, and the approximate locations and orientations of identified faults are presented on the map. Two geologic cross-sections providing a representative picture of the subsurface of the property are illustrated on Figure 1. A brief description of the findings of the geologic investigation follows.

A prominent bedrock outcrop of the Dolomite Member of the Cambrian St. Charles Limestone near the southwestern corner of Lot 27 provided an understanding of the bedrock stratigraphy. At this location, approximately 45 feet of bedrock is continuously exposed, and displays four distinct lithologic units:

1. Unit 1: The uppermost unit is a dark gray, sparry dolomite found to contain abundant round, curved, whitish-yellow shell fragments in massive blocks. The exposed thickness of this unit at this location is approximately 3 feet.
2. Unit 2: Immediately underlying Unit 1 is a dark gray to light gray sparry dolomite containing faint laminations in thickly bedded blocks. Within the unit are distinct dark gray beds that contain abundant rounded *Girvanella* nodules up to 1 centimeter in diameter. Bedding becomes more prominent with depth in this unit, and this unit is seen to be approximately 10 to 12 feet thick.
3. Unit 3: Immediately underlying Unit 2 is a dark gray, sparry dolomite that is transitional between the overlying two units, in that it contains some laminations and curved shelly material. The unit is thickly to moderately bedded, and is distinct from the overlying units in that it contains abundant thin yellow stringers of calcium carbonate. The unit is seen to be approximately 20 to 25 feet thick.
4. Unit 4: The basal unit in the exposed outcrop is a light gray to pinkish gray, finely sparry dolomite with a highly variegated, mottled coloration in irregular, elongated lobes. Distinct to this unit is the presence of small vugs up to 2 inches in diameter, commonly filled with recrystallized dolomite. The exposed thickness of this unit at this location is approximately 5 feet.

Bedding at this outcrop was found to strike at N24°W and dip at 25°NE, which was largely characteristic of the bedding found on the property as a whole. Across the property, the bedrock was found to have blocky jointing, with the two major joint sets being orthogonal to one another. One joint set was parallel to the bedding, and the other was perpendicular to the bedding, dipping steeply to the southwest.

Bedrock for the property at large was found to be largely constrained between the road that forms the northern boundary of the Ridge Nests property and North Powder Mountain Road to the south. Bounding the bedrock in all directions for at least several hundred feet laterally are Quaternary deposits that consist of undifferentiated colluvium and slopewash. Clasts were found to be exclusively Precambrian quartzite and conglomerate, rounded to subrounded, and up to 6 feet in diameter. These clasts were found to be in a variety of colors, but graded between pink and tan and gray. Total thickness of the Quaternary colluvial/slopewash deposits on and immediately surrounding the property is unknown, but is noted by Sorenson and Crittenden, Jr (1979) to be between 0 and 30 meters thick. When present on the property, these deposits appear to be a relatively thin veneer of possibly 5 feet or less, as the transitions to the bedrock are abrupt.

Response to Comment No. 2b

No landslide deposits were found either on the property or in the immediate vicinity of the property. It is therefore concluded that landslide risk is low and are not expected to adversely impact the subject property.

A semi-continuous exposure of bedrock is present along the southern side of the road that forms the northern boundary for the Ridge Nests property. Along this road, two faults were identified, near the northwest corner of Lot 10 and in between Lots 10 and 11, respectively (see Plate 1 and representative photos on Figure 2). The fault adjacent to the northwest corner of Lot 10 was found to be a subvertical normal fault that juxtaposed Unit 1 and Unit 3, with a minimum of approximately 10 feet of offset (Photo 1). Along the fault trace was a dark red silty material, possibly gouge, that was found linearly along the exposed road cut from the base of the exposed outcrop to just below a large pine tree sitting atop the outcrop (Photo 2). The west side (footwall) of the fault contained bedding that had been tilted in a manner not seen elsewhere on the property, steeply dipping (>45°) to the southeast, while the east side (hanging wall) of the fault contained bedding attitudes that were similar to the bedrock elsewhere on the property (dipping between 15 and 25° to the northeast). This fault is considered to be inactive, due to several factors:

1. The fault extends up to, but not through, the overlying soil profile.
2. Abundant vegetation is present above the fault trace, and is not offset or disturbed in any way.
3. The topographic surface has a consistent slope across the fault trace, and there is no evident associated fault scarp.
4. The bedrock is Cambrian in age, and has likely undergone much deformation since deposition, including faulting. The fact that the footwall block shows such drastic deformation not seen elsewhere on the property suggests that the displacement happened in the ancient geologic past, and subsequent geomorphic processes have returned the bedrock block back to stable topographic conditions across the fault trace.

A second possible fault was encountered approximately 60 feet east of the first fault along the road, between Lots 10 and 11. This possible fault had a much gentler dip (32°NE) than the first, though it passed through an area of disrupted, highly weathered bedrock which did not have clear-cut offset or deformation (Photo 3). However, a couple blocks west of the feature seen in the photo show abnormally tilted bedding akin to that seen in the first fault, though these may just have been artificially rotated during road excavation. A dark red to gray silty material, possibly fault gouge, was found along a linear trace from the base of the slope to the base of a highly weathered bedrock overlay, found immediately below the topsoil. It is possible that this feature is merely a joint that has been infilled with surficial materials. If it is indeed a fault, the fault is considered inactive for the same reasons specified above.

Based on the geologic evidence presented on the attached *Geologic Map* (Plate 1) and the associated geologic cross-sections, and the slope stability assessment presented herein, the following conclusions are made:

1. The stability of the slopes are not adversely impacted by the geologic, stratigraphic, or hydrologic conditions observed.

2. There are no evident potential on-site or off-site geologic hazards that can adversely affect the subject property, and the site is considered suitable for development from a geologic hazards standpoint.

3. The site is considered suitable for development from a geotechnical perspective, provided the recommendations presented in the referenced 2014 geotechnical report and subsequent addenda are incorporated into the design and construction of the project.

Closure

We appreciate the opportunity to provide you with our services. If you have any questions please contact the undersigned at your convenience (801) 748-4044.

Respectfully Submitted,
IGES, Inc.

PROFESSIONAL ENGINEER
No. 6370734
DAVID A. GLASS
09-23-15
ELECTRONIC SEAL
STATE OF UTAH

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

Reviewed by:

LICENSED PROFESSIONAL GEOLOGIST
No. 5248599
C. CHARLES PAYTON
9-23-2015
STATE OF UTAH

C. Charles Payton, P.G.
Engineering Geologist

Peter E. Doumit, P.G., C.P.G.
Senior Geologist

Attachments (next page)

Attachments:

References
Figure 1 – Geologic Cross-Sections
Figure 2 – Photos (Normal Faults)
Plate 1 – Geologic Map
Slope Stability Analysis
RocLab Output

References

Hoek, E., and Brown, E.T., 1997, Practical Estimates of Rock Mass Strength, in *International Journal of Rock Mechanics & Mining Science & Geomechanics Abstracts*, 34(8), 1165-1186.

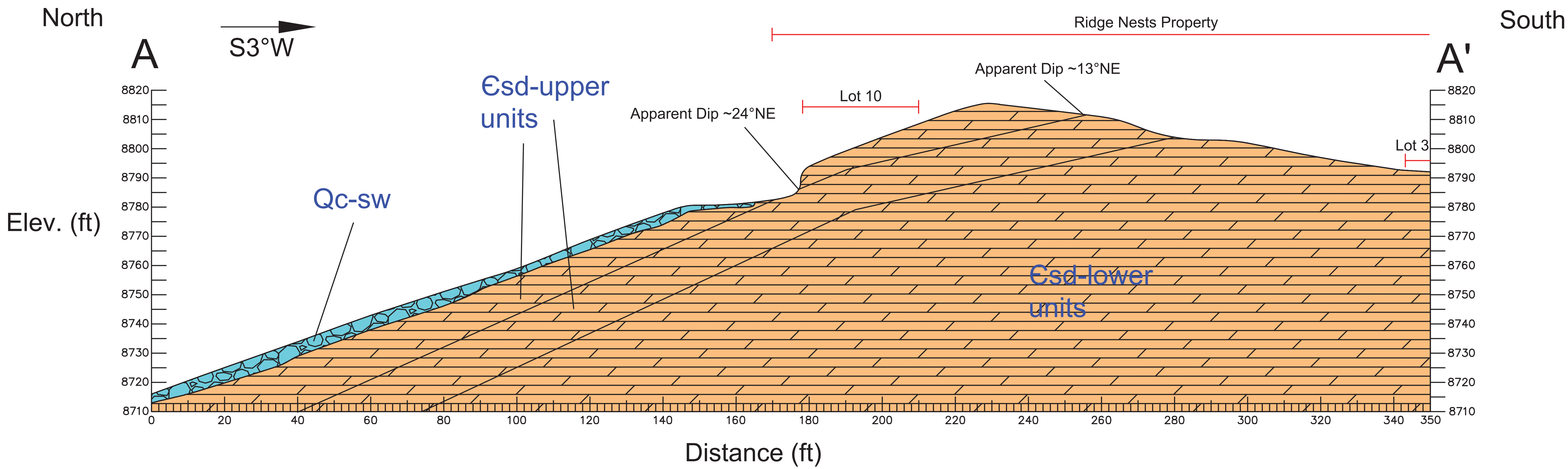
IGES, Inc., 2014, Geotechnical Investigation, The Ridge Nests Development, Powder Mountain Resort, Weber and Cache Counties, Utah Project No. 01628-008, dated September 16, 2014.

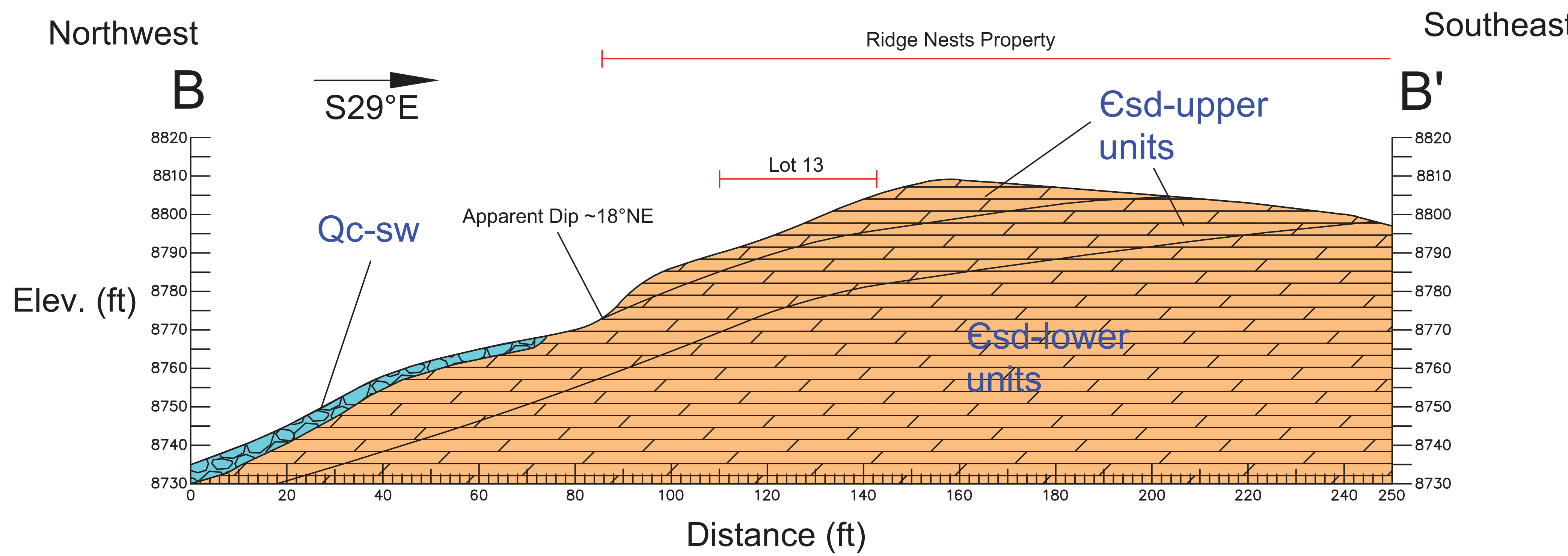
IGES, Inc., 2015a, Response to Review Comments, Geotechnical Investigation, The Ridge Nests Development, Powder Mountain Resort, Weber and Cache Counties, Utah Project No. 01628-008, dated April 7, 2015.

IGES, Inc., 2015b, Addendum to Geotechnical Report, The Ridge Nests Development, Powder Mountain Resort, Weber and Cache Counties, Utah Project No. 01628-008, dated August 18, 2015.

Sorensen, M.L., and Crittenden, M.D., Jr., 1979, Geologic map of the Huntsville quadrangle, Weber and Cache Counties, Utah: U.S. Geological Survey Geologic Quadrangle Series Map GQ-1503, scale 1:24,000.

Western Geologic, 2012, Report: Geologic Hazards Reconnaissance, Proposed Area 1 Mixed-Use Development, Powder Mountain Resort, Weber County, Utah, dated August 28, 2012.





LEGEND

Qc-sw: Colluvium and Slope Wash, Undifferentiated

Consists of rounded to subrounded clasts of pink to tan to gray quartzite and conglomerate up to 6' in diameter.

Єsd: St. Charles Limestone-Dolomite Member

Consists of light to dark gray sparry dolomite exhibiting 4 distinct lithologies (upper to lower):
1) Dark gray sparry dolomite with abundant round, curved whitish-yellow shells; massive
2) Dark gray to light gray sparry dolomite; faint laminations; thickly bedded; dark gray component has aligned peloids/colloids within unit; bedding more prominent with depth.
3) Dark gray sparry dolomite; gradational between overlying 2 units; some curved shelly material, some laminations; thickly to moderately bedded; abundant yellow CaCO3 stringers.
4) Light gray dolomite with vugs; highly variegated, mottled coloration.

0 20 40 60
FEET

FIGURE 1

CROSS-SECTIONS
GEOLOGIC INVESTIGATION
THE RIDGE NESTS DEVELOPMENT
POWDER MOUNTAIN RESORT
WEBER COUNTY, UTAH

DATE: 8/31/2015	SCALE: 1"=30'
FILE: 01628-008	

IGES


PHOTO 3


PHOTO 1


PHOTO 2

IGES®

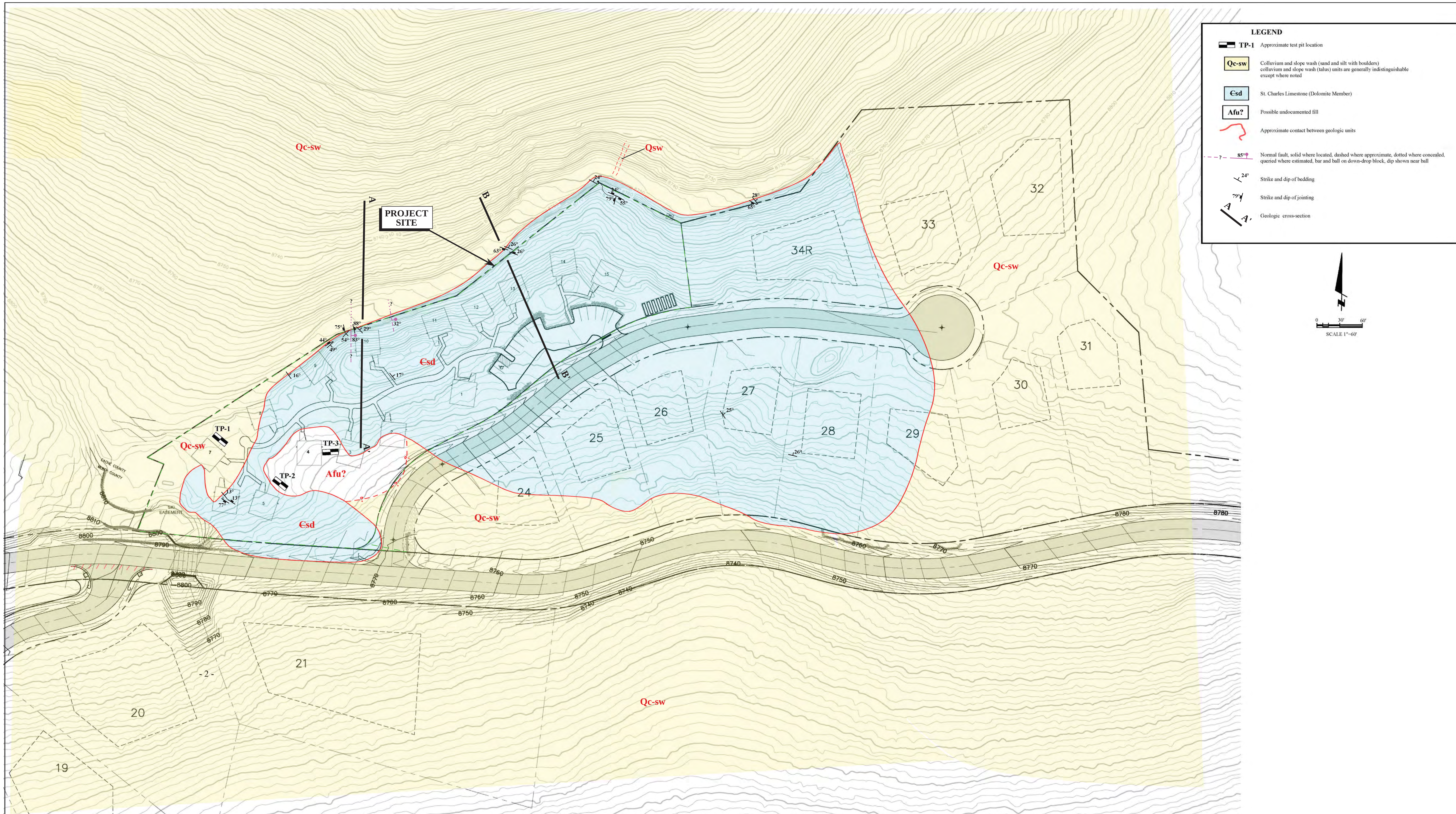
Project No. 01628-008

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

SITE PHOTOS

Figure

2



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

IGES®
Project No. 01628-008

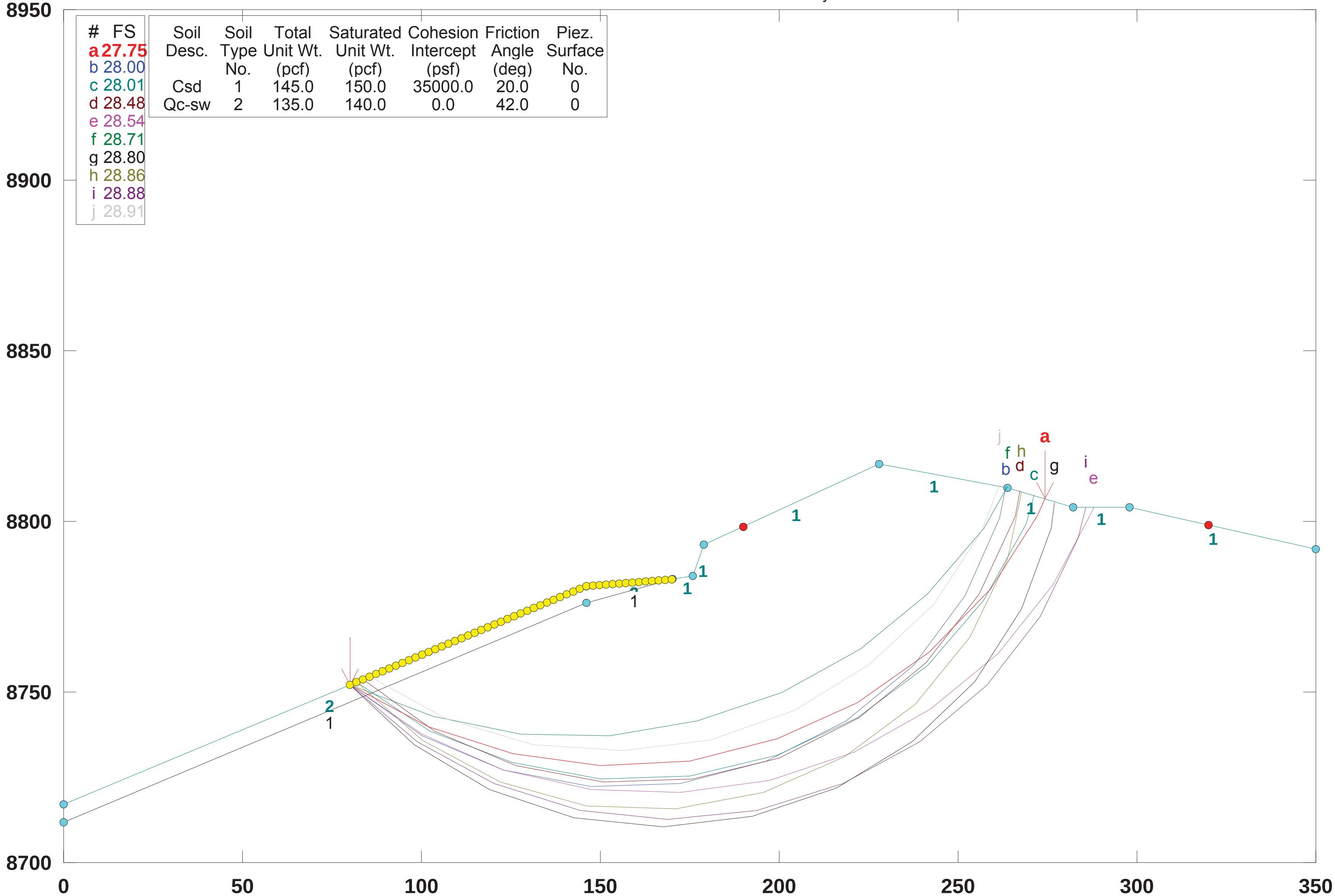
Geotechnical Investigation
The Ridge Nests Development
Powder Mountain Resort
Weber County, Utah

GEOLOGIC MAP

PLATE
1

Summit/Ridge Nests; A-A'; Static

C:\DOCUME~1\DAVIDG\DESKTOP\01628008\A1.PL2 Run By: DAG 9/1/2015 1:42PM



Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
Csd	1	145.0	150.0	35000.0	20.0	0
Qc-sw	2	135.0	140.0	0.0	42.0	0

GSTABL7 v.2 FSmin=27.75

Safety Factors Are Calculated By The Modified Bishop Method

*** GSTABL7 ***

** GSTABL7 by Garry H. Gregory, P.E. **

** Original Version 1.0, January 1996; Current Version 2.002, December 2001 ** 1

****

SLOPE STABILITY ANALYSIS SYSTEM

Modified Bishop, Simplified Janbu, or GLE Method of Slices.
(Includes Spencer & Morgenstern-Price Type Analysis)
Including Pier/Pile, Reinforcement, Soil Nail, Tieback,
Nonlinear Undrained Shear Strength, Curved Phi Envelope,
Anisotropic Soil, Fiber-Reinforced Soil, Boundary Loads, Water
Surfaces, Pseudo-Static Earthquake, and Applied Force Options.

*** 1

Analysis Run Date: 9/1/2015
Time of Run: 1:42PM
Run By: DAG
Input Data Filename: C:A1.
Output Filename: C:A1.OUT
Unit System: English

Plotted Output Filename: C:A1.PLT

PROBLEM DESCRIPTION: Summit/Ridge Nests; A-A'; Static

BOUNDARY COORDINATES

9 Top Boundaries
11 Total Boundaries

Boundary No.	X-Left (ft)	Y-Left (ft)	X-Right (ft)	Y-Right (ft)	Soil Type Below Bnd
1	0.00	8717.00	146.00	8781.00	2
2	146.00	8781.00	170.00	8783.00	2
3	170.00	8783.00	176.00	8784.00	1
4	176.00	8784.00	179.00	8793.00	1
5	179.00	8793.00	228.00	8817.00	1
6	228.00	8817.00	264.00	8810.00	1
7	264.00	8810.00	282.00	8804.00	1
8	282.00	8804.00	298.00	8804.00	1
9	298.00	8804.00	350.00	8792.00	1
10	0.00	8712.00	146.00	8776.00	1
11	146.00	8776.00	170.00	8783.00	1

User Specified Y-Origin = 8700.00(ft)

ISOTROPIC SOIL PARAMETERS

2 Type(s) of Soil

Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
1	145.0	150.0	35000.0	20.0	0.00	0.0	0
2	135.0	140.0	0.0	42.0	0.00	0.0	0

A Critical Failure Surface Searching Method, Using A Random Technique For Generating Circular Surfaces, Has Been Specified.

2500 Trial Surfaces Have Been Generated.

50 Surface(s) Initiate(s) From Each Of 50 Points Equally Spaced Along The Ground Surface Between X = 80.00(ft) and X = 170.00(ft)

Each Surface Terminates Between X = 190.00(ft) and X = 320.00(ft)

Unless Further Limitations Were Imposed, The Minimum Elevation At Which A Surface Extends Is Y = 0.00(ft)

25.00(ft) Line Segments Define Each Trial Failure Surface.

Following Is Displayed The Most Critical Of The Trial Failure Surfaces Evaluated.

* * Safety Factors Are Calculated By The Modified Bishop Method * *

Total Number of Trial Surfaces Evaluated = 2500

Statistical Data On All Valid FS Values:
FS Max = 187.868   FS Min = 27.754   FS Ave = 58.595
Standard Deviation =   26.045   Coefficient of Variation =   44.45 %

**** END OF GSTABL7 OUTPUT ****

Failure Surface Specified By 11 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	80.00	8752.07
2	101.75	8739.73
3	125.44	8731.77
4	150.22	8728.47
5	175.18	8729.95
6	199.40	8736.16
7	221.98	8746.87
8	242.12	8761.70
9	259.06	8780.08
10	272.18	8801.36
11	274.16	8806.61

Circle Center At X =   155.00 ; Y = 8858.94 ; and Radius =   130.56

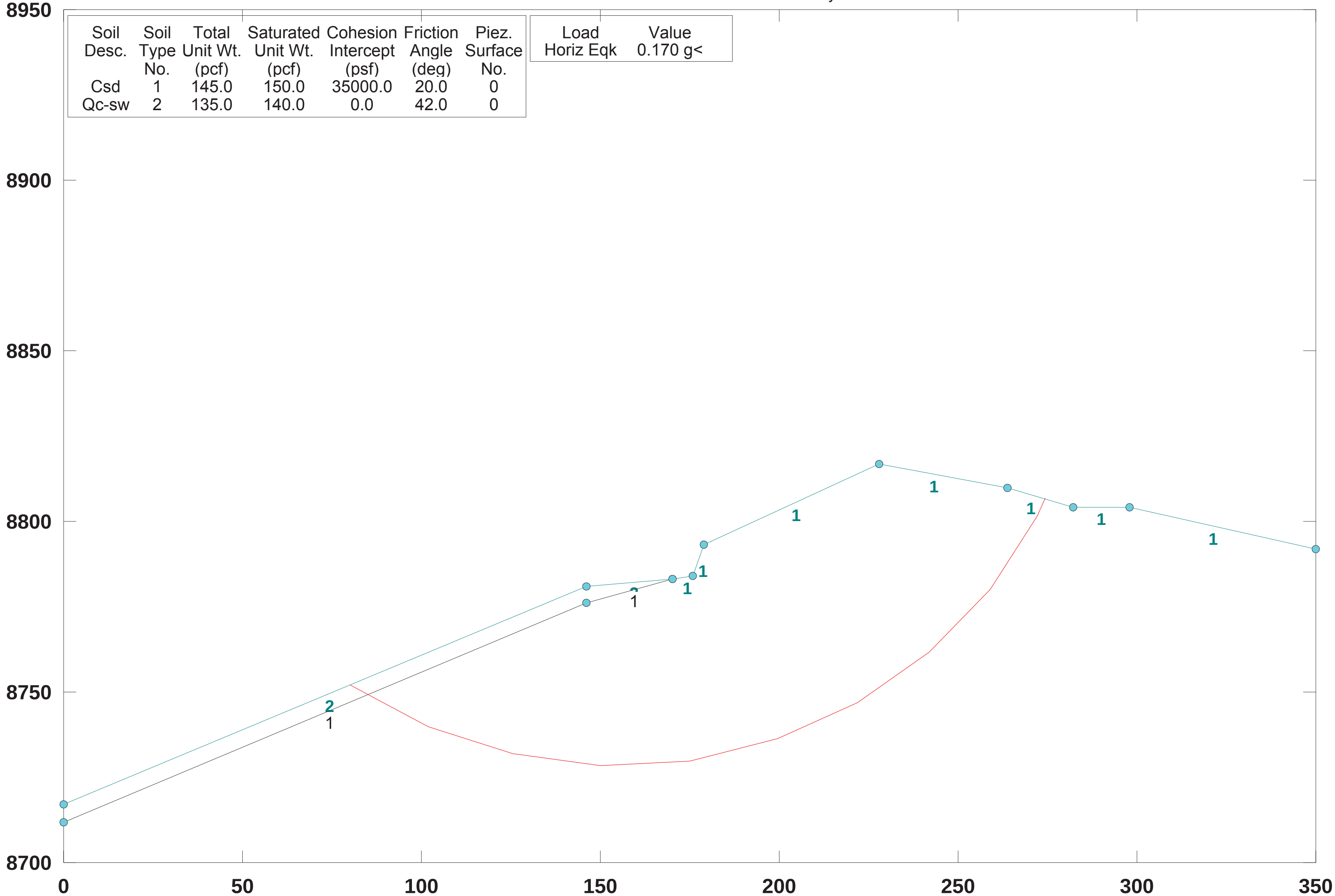
Factor of Safety
***   27.754   ***

Individual data on the   17 slices

Slice No.	Width (ft)	Weight (lbs)	Water Force Top (lbs)	Water Force Bot (lbs)	Tie Force Norm (lbs)	Tie Force Tan (lbs)	Earthquake Force Hor (lbs)	Earthquake Force Ver (lbs)	Surcharge Load (lbs)
1	5.0	1678.0	0.0	0.0	0.	0.	0.0	0.0	0.0
2	16.8	31833.5	0.0	0.0	0.	0.	0.0	0.0	0.0
3	23.7	105486.1	0.0	0.0	0.	0.	0.0	0.0	0.0
4	20.6	136369.9	0.0	0.0	0.	0.	0.0	0.0	0.0
5	4.2	31918.2	0.0	0.0	0.	0.	0.0	0.0	0.0
6	19.8	151921.4	0.0	0.0	0.	0.	0.0	0.0	0.0
7	5.2	40286.7	0.0	0.0	0.	0.	0.0	0.0	0.0
8	0.8	6405.8	0.0	0.0	0.	0.	0.0	0.0	0.0
9	3.0	25210.9	0.0	0.0	0.	0.	0.0	0.0	0.0
10	20.4	190607.4	0.0	0.0	0.	0.	0.0	0.0	0.0
11	22.6	219461.5	0.0	0.0	0.	0.	0.0	0.0	0.0
12	6.0	57951.4	0.0	0.0	0.	0.	0.0	0.0	0.0
13	14.1	121033.3	0.0	0.0	0.	0.	0.0	0.0	0.0
14	16.9	102466.0	0.0	0.0	0.	0.	0.0	0.0	0.0
15	4.9	18920.8	0.0	0.0	0.	0.	0.0	0.0	0.0
16	8.2	16489.8	0.0	0.0	0.	0.	0.0	0.0	0.0
17	2.0	849.6	0.0	0.0	0.	0.	0.0	0.0	0.0

Summit/Ridge Nests; A-A'; Pseudo-Static

C:\DOCUME~1\DAVIDG\DESKTOP\01628008\A1P.PLT Run By: DAG 9/1/2015 1:57PM



GSTABL7 v.2 FSmin=18.49

Factor Of Safety Is Calculated By The Modified Bishop Method

*** GSTABL7 ***

** GSTABL7 by Garry H. Gregory, P.E. **

** Original Version 1.0, January 1996; Current Version 2.002, December 2001 ** 1

****

SLOPE STABILITY ANALYSIS SYSTEM
Modified Bishop, Simplified Janbu, or GLE Method of Slices.
(Includes Spencer & Morgenstern-Price Type Analysis)
Including Pier/Pile, Reinforcement, Soil Nail, Tieback,
Nonlinear Undrained Shear Strength, Curved Phi Envelope,
Anisotropic Soil, Fiber-Reinforced Soil, Boundary Loads, Water
Surfaces, Pseudo-Static Earthquake, and Applied Force Options.

****

Analysis Run Date: 9/1/2015
Time of Run: 1:57PM
Run By: DAG
Input Data Filename: C:A1P.
Output Filename: C:A1P.OUT
Unit System: English

Plotted Output Filename: C:A1P.PLT 1

PROBLEM DESCRIPTION: Summit/Ridge Nests; A-A'; Pseudo-Static

BOUNDARY COORDINATES

9 Top Boundaries
11 Total Boundaries

Boundary No.	X-Left (ft)	Y-Left (ft)	X-Right (ft)	Y-Right (ft)	Soil Type Below Bnd
1	0.00	8717.00	146.00	8781.00	2
2	146.00	8781.00	170.00	8783.00	2
3	170.00	8783.00	176.00	8784.00	1
4	176.00	8784.00	179.00	8793.00	1
5	179.00	8793.00	228.00	8817.00	1
6	228.00	8817.00	264.00	8810.00	1
7	264.00	8810.00	282.00	8804.00	1
8	282.00	8804.00	298.00	8804.00	1
9	298.00	8804.00	350.00	8792.00	1
10	0.00	8712.00	146.00	8776.00	1
11	146.00	8776.00	170.00	8783.00	1

User Specified Y-Origin = 8700.00(ft)

ISOTROPIC SOIL PARAMETERS

2 Type(s) of Soil

Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
1	145.0	150.0	35000.0	20.0	0.00	0.0	0
2	135.0	140.0	0.0	42.0	0.00	0.0	0

A Horizontal Earthquake Loading Coefficient
Of0.170 Has Been Assigned

A Vertical Earthquake Loading Coefficient
Of0.000 Has Been Assigned

Cavitation Pressure = 0.0(psf)

Trial Failure Surface Specified By 11 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	80.00	8752.07
2	101.75	8739.73
3	125.44	8731.77
4	150.22	8728.47
5	175.18	8729.95
6	199.40	8736.16
7	221.98	8746.87
8	242.12	8761.70
9	259.06	8780.08
10	272.18	8801.36
11	274.16	8806.61

Circle Center At X = 155.00 ; Y = 8858.95; and Radius = 130.57

* * Factor Of Safety Is Calculated By The Modified Bishop Method * *

Factor Of Safety For The Preceding Specified Surface = 18.492

Table 1 - Individual Data on the 17 Slices

Slice No.	Width (ft)	Weight (lbs)	Water Force Top (lbs)	Water Force Bot (lbs)	Tie Force Norm (lbs)	Tie Force Tan (lbs)	Earthquake Force Hor (lbs)	Earthquake Force Ver (lbs)	Surcharge Load (lbs)
1	5.0	1678.0	0.0	0.0	0.0	0.0	285.3	0.0	0.0
2	16.8	31849.7	0.0	0.0	0.0	0.0	5414.5	0.0	0.0
3	23.7	105456.4	0.0	0.0	0.0	0.0	17927.6	0.0	0.0
4	20.6	136383.8	0.0	0.0	0.0	0.0	23185.2	0.0	0.0
5	4.2	31885.7	0.0	0.0	0.0	0.0	5420.6	0.0	0.0
6	19.8	151943.4	0.0	0.0	0.0	0.0	25830.4	0.0	0.0
7	5.2	40285.1	0.0	0.0	0.0	0.0	6848.5	0.0	0.0
8	0.8	6405.8	0.0	0.0	0.0	0.0	1089.0	0.0	0.0
9	3.0	25210.5	0.0	0.0	0.0	0.0	4285.8	0.0	0.0
10	20.4	190649.4	0.0	0.0	0.0	0.0	32410.4	0.0	0.0
11	22.6	219387.1	0.0	0.0	0.0	0.0	37295.8	0.0	0.0
12	6.0	57994.2	0.0	0.0	0.0	0.0	9859.0	0.0	0.0
13	14.1	121052.6	0.0	0.0	0.0	0.0	20578.9	0.0	0.0
14	16.9	102471.3	0.0	0.0	0.0	0.0	17420.1	0.0	0.0
15	4.9	18906.4	0.0	0.0	0.0	0.0	3214.1	0.0	0.0
16	8.2	16500.0	0.0	0.0	0.0	0.0	2805.0	0.0	0.0
17	2.0	849.0	0.0	0.0	0.0	0.0	144.3	0.0	0.0

Table 2 - Base Stress Data on the 17 Slices

Slice No. *	Alpha (deg)	X-Coord. Slice Cntr (ft)	Base Leng. (ft)	Available Shear Strength (psf)	Mobilized Shear Stress (psf)
1	-29.56	82.49	5.72	312.52	-136.22
2	-29.56	93.36	19.29	36093.93	-812.12
3	-18.57	113.60	24.99	36864.02	-1342.02
4	-7.59	135.72	20.74	37512.71	-865.57
5	-7.59	148.11	4.26	37849.31	-977.06
6	3.39	160.11	19.81	37751.86	456.22
7	3.39	172.59	5.19	37786.53	468.86
8	14.38	175.59	0.85	37653.34	1937.37
9	14.38	177.50	3.10	37867.54	2037.36
10	14.38	189.20	21.06	38208.71	2250.41
11	25.38	210.69	24.99	38179.86	3764.24
12	36.37	224.99	7.48	37956.21	4606.27
13	36.37	235.06	17.54	37575.75	4096.16
14	47.33	250.59	25.00	36423.81	3016.47
15	58.34	261.53	9.41	35267.12	1714.98
16	58.34	268.09	15.59	34628.68	904.28
17	69.35	273.17	5.61	33411.05	150.27

Sum of the Resisting Forces (including Pier/Pile, Tieback, Reinforcing
Soil Nail, and Applied Forces if applicable) = 8322990.00 (lbs)

Average Available Shear Strength (including Tieback, Pier/Pile, Reinforcing,
Soil Nail, and Applied Forces if applicable) = 36090.45(psf)

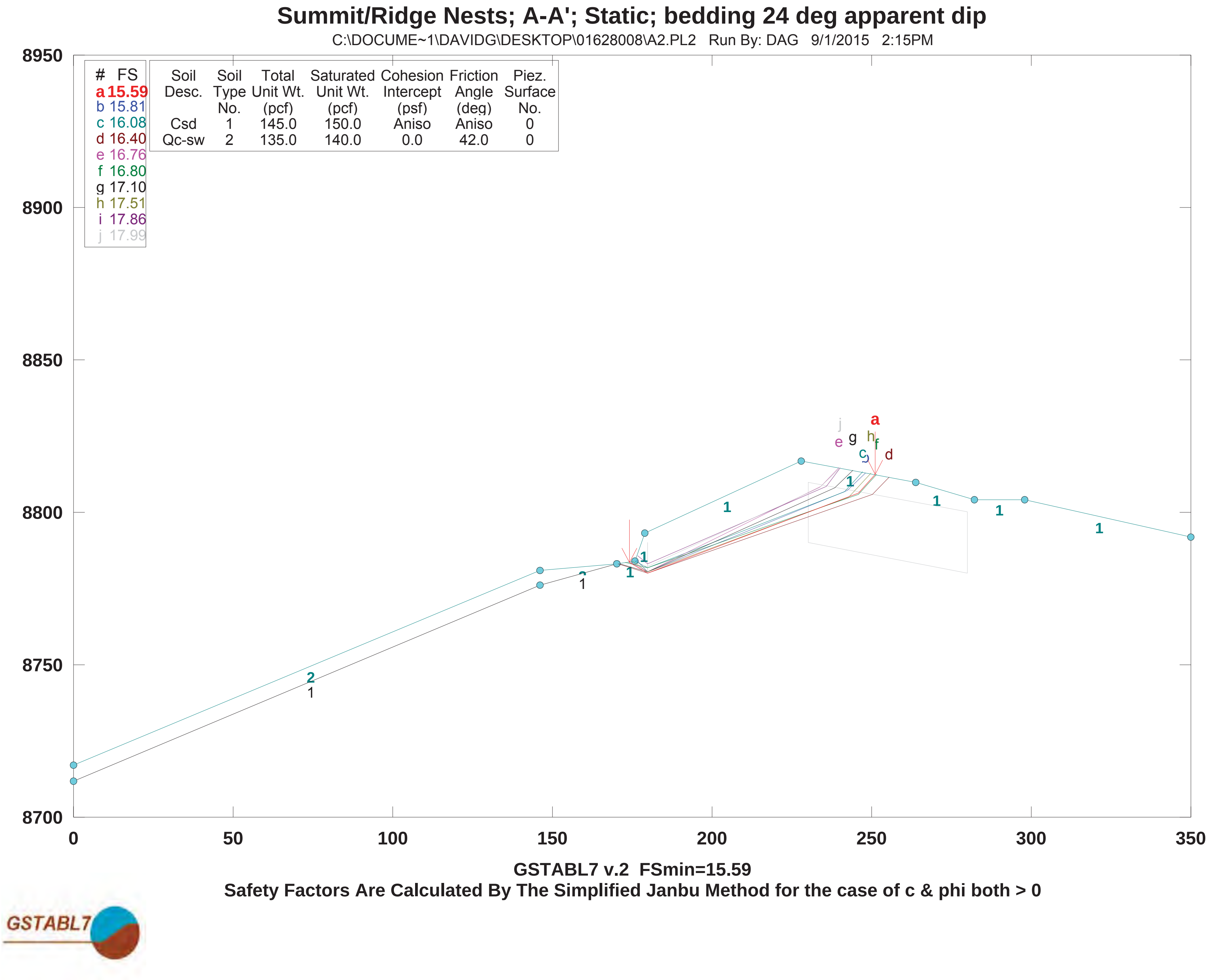
Sum of the Driving Forces = 450097.31 (lbs)

Average Mobilized Shear Stress = 1951.73(psf)

Total length of the failure surface = 230.61(ft)

CAUTION - Factor Of Safety Is Calculated By The Modified Bishop
Method. This Method Is Valid Only If The Failure Surface
Approximates A Circular Arc.

**** END OF GSTABL7 OUTPUT ****



Summit/Ridge Nests; A-A'; Static; bedding 24 deg apparent dip
C:\DOCUME~1\DAVIDG\DESKTOP\01628008\A2.PL2 Run By: DAG 9/1/2015 2:15PM
FS
a 15.59
b 15.81
c 16.08
d 16.40
e 16.76
f 16.80
g 17.10
h 17.51
i 17.86
j 17.99
Soil Desc. | Soil Type No. | Total Unit Wt. (pcf) | Saturated Unit Wt. (pcf) | Cohesion Intercept (psf) | Friction Angle (deg) | Piez. Surface No.
Csd | 1 | 145.0 | 150.0 | Aniso | Aniso | 0
Qc-sw | 2 | 135.0 | 140.0 | 0.0 | 42.0 | 0
8950
8900
8850
8800
8750
8700
0
50
100
150
200
250
300
350
GSTABL7 v.2 FSmin=15.59
Safety Factors Are Calculated By The Simplified Janbu Method for the case of c & phi both > 0
GSTABL7

*** GSTABL7 ***

** GSTABL7 by Garry H. Gregory, P.E. **

** Original Version 1.0, January 1996; Current Version 2.002, 1
December 2001 **

****

SLOPE STABILITY ANALYSIS SYSTEM
Modified Bishop, Simplified Janbu, or GLE Method of Slices.
(Includes Spencer & Morgenstern-Price Type Analysis)
Including Pier/Pile, Reinforcement, Soil Nail, Tieback,
Nonlinear Undrained Shear Strength, Curved Phi Envelope,
Anisotropic Soil, Fiber-Reinforced Soil, Boundary Loads, Water
Surfaces, Pseudo-Static Earthquake, and Applied Force Options.

****

Analysis Run Date: 9/1/2015
Time of Run: 2:15PM
Run By: DAG
Input Data Filename: C:a2.
Output Filename: C:a2.OUT
Unit System: English

Plotted Output Filename: C:a2.PLT

PROBLEM DESCRIPTION: Summit/Ridge Nests; A-A'; Static; beddin
g 24 deg apparent dip

BOUNDARY COORDINATES

9 Top Boundaries
11 Total Boundaries

Boundary No.	X-Left (ft)	Y-Left (ft)	X-Right (ft)	Y-Right (ft)	Soil Type Below Bnd
1	0.00	8717.00	146.00	8781.00	2
2	146.00	8781.00	170.00	8783.00	2
3	170.00	8783.00	176.00	8784.00	1
4	176.00	8784.00	179.00	8793.00	1
5	179.00	8793.00	228.00	8817.00	1
6	228.00	8817.00	264.00	8810.00	1
7	264.00	8810.00	282.00	8804.00	1
8	282.00	8804.00	298.00	8804.00	1
9	298.00	8804.00	350.00	8792.00	1
10	0.00	8712.00	146.00	8776.00	1
11	146.00	8776.00	170.00	8783.00	1

1

User Specified Y-Origin = 8700.00(ft)

ISOTROPIC SOIL PARAMETERS

2 Type(s) of Soil

Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
1	145.0	150.0	35000.0	20.0	0.00	0.0	0
2	135.0	140.0	0.0	42.0	0.00	0.0	0

ANISOTROPIC STRENGTH PARAMETERS
1 soil type(s)

Soil Type 1 Is Anisotropic

Number Of Direction Ranges Specified = 3

Direction Range No.	Counterclockwise Direction Limit (deg)	Cohesion Intercept (psf)	Friction Angle (deg)
1	20.0	35000.00	20.00
2	30.0	0.00	42.00
3	90.0	35000.00	20.00

ANISOTROPIC SOIL NOTES:
(1) An input value of 0.01 for C and/or Phi will cause Aniso
C and/or Phi to be ignored in that range.
(2) An input value of 0.02 for Phi will set both Phi and
C equal to zero, with no water weight in the tension crack.
(3) An input value of 0.03 for Phi will set both Phi and
C equal to zero, with water weight in the tension crack.

Janbus Empirical Coef is being used for the case of c & phi both > 0

A Critical Failure Surface Searching Method, Using A Random
Technique For Generating Sliding Block Surfaces, Has Been
Specified.

2000 Trial Surfaces Have Been Generated.

2 Boxes Specified For Generation Of Central Block Base

Length Of Line Segments For Active And Passive Portions Of Sliding Block Is 35.0

Box No.	X-Left (ft)	Y-Left (ft)	X-Right (ft)	Y-Right (ft)	Height (ft)
1	180.00	8785.00	180.00	8785.00	10.00
2	230.00	8800.00	280.00	8790.00	20.00

Following Is Displayed The Most Critical Of The Trial Failure Surfaces Evaluated.

* * Safety Factors Are Calculated By The Simplified Janbu Method * *

Total Number of Trial Surfaces Evaluated = 2000

Statistical Data On All Valid FS Values:
FS Max = 494.331 FS Min = 15.586 FS Ave = 123.714
Standard Deviation = 69.420 Coefficient of Variation = 56.11 %

Failure Surface Specified By 4 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	174.31	8783.72
2	180.00	8780.10
3	245.90	8806.51
4	250.98	8812.53

Factor of Safety
*** 15.586 ***

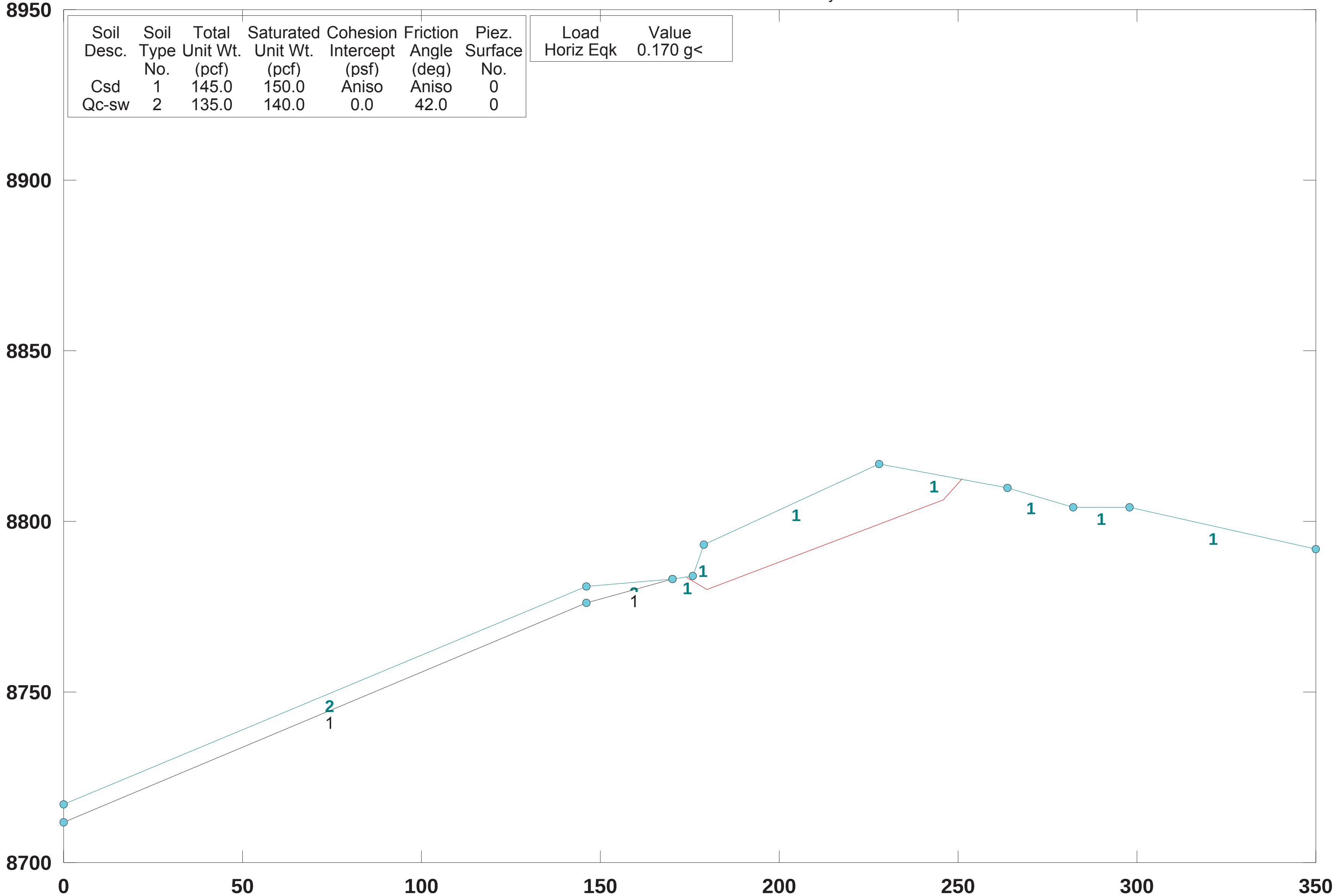
Individual data on the 6 slices

Slice No.	Width (ft)	Weight (lbs)	Water Force Top (lbs)	Water Force Bot (lbs)	Tie Force Norm (lbs)	Tie Force Tan (lbs)	Earthquake Force Hor (lbs)	Earthquake Force Ver (lbs)	Surcharge Load (lbs)
1	1.7	166.5	0.0	0.0	0.	0.	0.0	0.0	0.0
2	3.0	2962.6	0.0	0.0	0.	0.	0.0	0.0	0.0
3	1.0	1859.8	0.0	0.0	0.	0.	0.0	0.0	0.0
4	48.0	108070.3	0.0	0.0	0.	0.	0.0	0.0	0.0
5	17.9	32032.9	0.0	0.0	0.	0.	0.0	0.0	0.0
6	5.1	2580.8	0.0	0.0	0.	0.	0.0	0.0	0.0

**** END OF GSTABL7 OUTPUT ****

Summit/Ridge Nests; A-A'; Pseudo-Static;bedding 24 deg apparent dip

C:\DOCUME~1\DAVIDG\DESKTOP\01628008\A2P.PLT Run By: DAG 9/1/2015 2:16PM



GSTABL7 v.2 FSmin=10.70

Factor Of Safety Is Calculated By The Simplified Janbu Method

*** GSTABL7 ***

** GSTABL7 by Garry H. Gregory, P.E. **

** Original Version 1.0, January 1996; Current Version 2.002, 1
December 2001 **

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SLOPE STABILITY ANALYSIS SYSTEM
Modified Bishop, Simplified Janbu, or GLE Method of Slices.
(Includes Spencer & Morgenstern-Price Type Analysis)
Including Pier/Pile, Reinforcement, Soil Nail, Tieback,
Nonlinear Undrained Shear Strength, Curved Phi Envelope,
Anisotropic Soil, Fiber-Reinforced Soil, Boundary Loads, Water
Surfaces, Pseudo-Static Earthquake, and Applied Force Options.

****

Analysis Run Date: 9/1/2015
Time of Run: 2:16PM
Run By: DAG
Input Data Filename: C:a2p.
Output Filename: C:a2p.OUT
Unit System: English

Plotted Output Filename: C:a2p.PLT

PROBLEM DESCRIPTION: Summit/Ridge Nests; A-A'; Pseudo-Static;
bedding 24 deg apparent dip

BOUNDARY COORDINATES

9 Top Boundaries
11 Total Boundaries

Boundary No.	X-Left (ft)	Y-Left (ft)	X-Right (ft)	Y-Right (ft)	Soil Type Below Bnd
1	0.00	8717.00	146.00	8781.00	2
2	146.00	8781.00	170.00	8783.00	2
3	170.00	8783.00	176.00	8784.00	1
4	176.00	8784.00	179.00	8793.00	1
5	179.00	8793.00	228.00	8817.00	1
6	228.00	8817.00	264.00	8810.00	1
7	264.00	8810.00	282.00	8804.00	1
8	282.00	8804.00	298.00	8804.00	1
9	298.00	8804.00	350.00	8792.00	1
10	0.00	8712.00	146.00	8776.00	1
11	146.00	8776.00	170.00	8783.00	1

User Specified Y-Origin = 8700.00(ft)

ISOTROPIC SOIL PARAMETERS

2 Type(s) of Soil

Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
1	145.0	150.0	35000.0	20.0	0.00	0.0	0
2	135.0	140.0	0.0	42.0	0.00	0.0	0

ANISOTROPIC STRENGTH PARAMETERS
1 soil type(s)

Soil Type 1 Is Anisotropic

Number Of Direction Ranges Specified = 3

Direction Range No.	Counterclockwise Direction Limit (deg)	Cohesion Intercept (psf)	Friction Angle (deg)
1	20.0	35000.00	20.00
2	30.0	0.00	42.00
3	90.0	35000.00	20.00

ANISOTROPIC SOIL NOTES:
(1) An input value of 0.01 for C and/or Phi will cause Aniso
C and/or Phi to be ignored in that range.
(2) An input value of 0.02 for Phi will set both Phi and
C equal to zero, with no water weight in the tension crack.
(3) An input value of 0.03 for Phi will set both Phi and
C equal to zero, with water weight in the tension crack.

A Horizontal Earthquake Loading Coefficient
Of0.170 Has Been Assigned

A Vertical Earthquake Loading Coefficient
Of0.000 Has Been Assigned

Cavitation Pressure = 0.0(psf)

Janbu's Empirical Coef. is being used for the case of c & phi both > 0

1

Trial Failure Surface Specified By 4 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	174.31	8783.72
2	180.00	8780.10
3	245.90	8806.51
4	250.98	8812.53

Janbu's Empirical Coefficient (fo) = 1.030

* * Factor Of Safety Is Calculated By The Simplified Janbu Method * *

Factor Of Safety For The Preceding Specified Surface = 10.700

Table 1 - Individual Data on the 6 Slices

Slice No.	Width (ft)	Weight (lbs)	Water Force Top (lbs)	Water Force Bot (lbs)	Tie Force Norm (lbs)	Tie Force Tan (lbs)	Earthquake Force Hor (lbs)	Earthquake Force Ver (lbs)	Surcharge Load (lbs)
1	1.7	166.1	0.0	0.0	0.0	0.0	28.2	0.0	0.0
2	3.0	2962.6	0.0	0.0	0.0	0.0	503.6	0.0	0.0
3	1.0	1859.9	0.0	0.0	0.0	0.0	316.2	0.0	0.0
4	48.0	108070.3	0.0	0.0	0.0	0.0	18372.0	0.0	0.0
5	17.9	32020.4	0.0	0.0	0.0	0.0	5443.5	0.0	0.0
6	5.1	2581.7	0.0	0.0	0.0	0.0	438.9	0.0	0.0

Table 2 - Base Stress Data on the 6 Slices

Slice No. *	Alpha (deg)	X-Coord. Slice Cntr (ft)	Base Leng. (ft)	Available Shear Strength (psf)	Mobilized Shear Stress (psf)
1	-32.46	175.15	2.00	42440.65	-38.64
2	-32.46	177.50	3.56	42832.72	-388.34
3	-32.46	179.50	1.19	43217.36	-731.41
4	21.84	204.00	51.71	2112.71	1192.82
5	21.84	236.95	19.28	1678.61	947.73
6	49.85	248.44	7.88	52450.32	444.15

Sum of the Resisting Forces (including Pier/Pile, Tieback, Reinforcing
Soil Nail, and Applied Forces if applicable) = 842917.00 (lbs)

Average Available Shear Strength (including Tieback, Pier/Pile, Reinforcing,
Soil Nail, and Applied Forces if applicable) = 9845.24(psf)

Sum of the Driving Forces = 81132.47 (lbs)

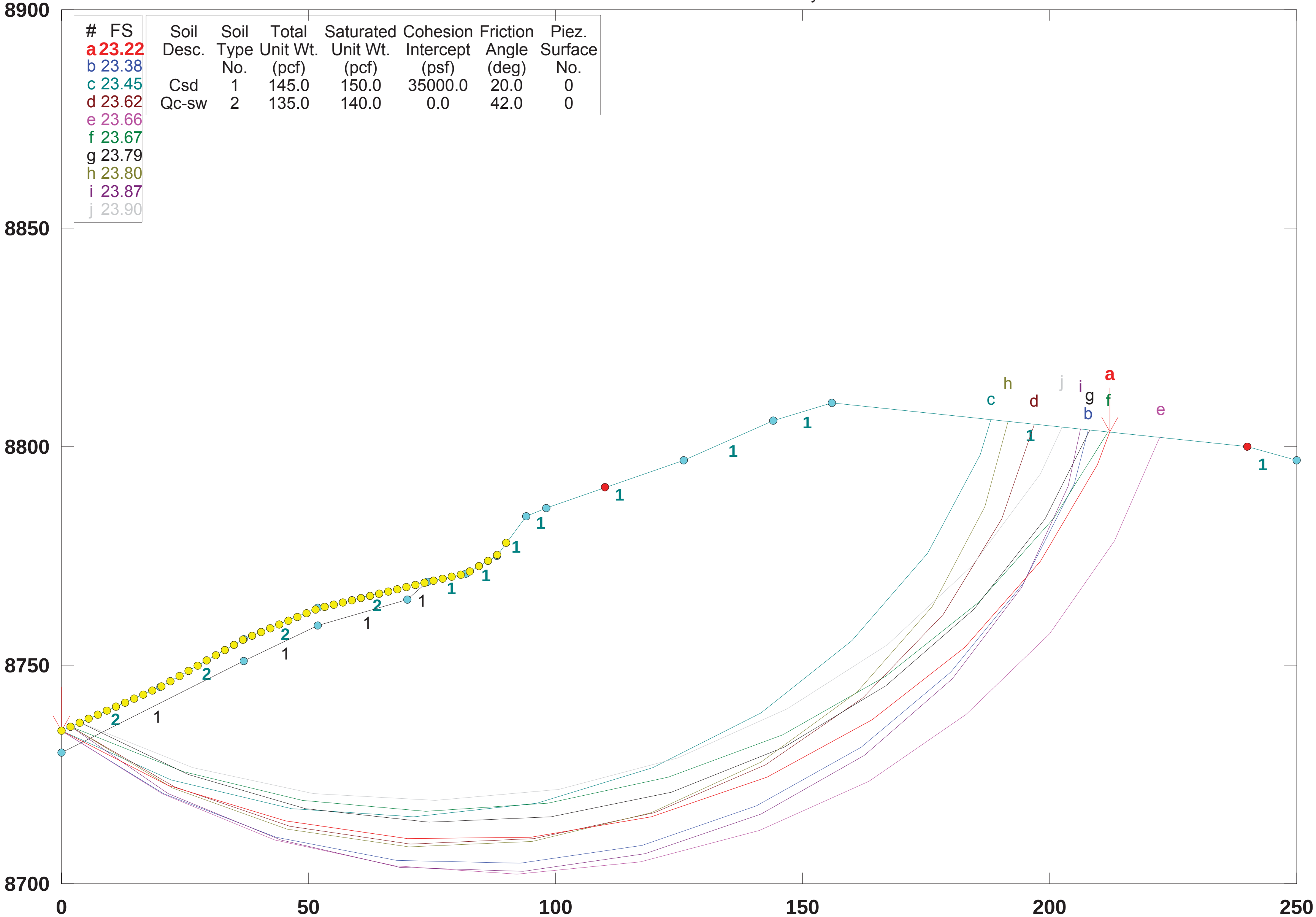
Average Mobilized Shear Stress = 947.62(psf)

Total length of the failure surface = 85.62(ft)

**** END OF GSTABL7 OUTPUT ****

Summit/Ridge Nests; B-B'; Static

C:\DOCUME~1\DAVIDG\DESKTOP\01628008\B1.PL2   Run By: DAG   9/1/2015   2:04PM



Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
Csd	1	145.0	150.0	35000.0	20.0	0
Qc-sw	2	135.0	140.0	0.0	42.0	0

GSTABL7 v.2 FSmin=23.22

Safety Factors Are Calculated By The Modified Bishop Method

GSTABL7

*** GSTABL7 ***

** GSTABL7 by Garry H. Gregory, P.E. **

** Original Version 1.0, January 1996; Current Version 2.002, December 2001 **

*** 1

SLOPE STABILITY ANALYSIS SYSTEM
Modified Bishop, Simplified Janbu, or GLE Method of Slices.
(Includes Spencer & Morgenstern-Price Type Analysis)
Including Pier/Pile, Reinforcement, Soil Nail, Tieback,
Nonlinear Undrained Shear Strength, Curved Phi Envelope,
Anisotropic Soil, Fiber-Reinforced Soil, Boundary Loads, Water
Surfaces, Pseudo-Static Earthquake, and Applied Force Options.

****

Analysis Run Date: 9/1/2015
Time of Run: 2:04PM
Run By: DAG
Input Data Filename: C:B1. 1
Output Filename: C:B1.OUT
Unit System: English

Plotted Output Filename: C:B1.PLT

PROBLEM DESCRIPTION: Summit/Ridge Nests; B-B'; Static

BOUNDARY COORDINATES

13 Top Boundaries
17 Total Boundaries

Boundary No.	X-Left (ft)	Y-Left (ft)	X-Right (ft)	Y-Right (ft)	Soil Type Below Bnd
1	0.00	8735.00	20.00	8745.00	2
2	20.00	8745.00	37.00	8756.00	2
3	37.00	8756.00	52.00	8763.00	2
4	52.00	8763.00	74.00	8769.00	2
5	74.00	8769.00	82.00	8771.00	1
6	82.00	8771.00	88.00	8775.00	1
7	88.00	8775.00	94.00	8784.00	1
8	94.00	8784.00	98.00	8786.00	1
9	98.00	8786.00	126.00	8797.00	1
10	126.00	8797.00	144.00	8806.00	1
11	144.00	8806.00	156.00	8810.00	1
12	156.00	8810.00	240.00	8800.00	1
13	240.00	8800.00	250.00	8797.00	1
14	0.00	8730.00	37.00	8751.00	1
15	37.00	8751.00	52.00	8759.00	1
16	52.00	8759.00	70.00	8765.00	1
17	70.00	8765.00	74.00	8769.00	1

User Specified Y-Origin = 8700.00(ft)

ISOTROPIC SOIL PARAMETERS

2 Type(s) of Soil

Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
1	145.0	150.0	35000.0	20.0	0.00	0.0	0
2	135.0	140.0	0.0	42.0	0.00	0.0	0

A Critical Failure Surface Searching Method, Using A Random Technique For Generating Circular Surfaces, Has Been Specified.

2500 Trial Surfaces Have Been Generated.

50 Surface(s) Initiate(s) From Each Of 50 Points Equally Spaced
Along The Ground Surface Between X = 0.00(ft)
and X = 90.00(ft)

Each Surface Terminates Between X = 110.00(ft)
and X = 240.00(ft)

Unless Further Limitations Were Imposed, The Minimum Elevation
At Which A Surface Extends Is Y = 0.00(ft)

25.00(ft) Line Segments Define Each Trial Failure Surface.

Following Is Displayed The Most Critical Of The Trial
Failure Surfaces Evaluated.

* * Safety Factors Are Calculated By The Modified Bishop Method * *

Total Number of Trial Surfaces Evaluated = 2500

Statistical Data On All Valid FS Values:
FS Max = 200.988   FS Min = 23.224   FS Ave = 46.199
Standard Deviation = 19.596   Coefficient of Variation = 42.42 %

Failure Surface Specified By 12 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	0.00	8735.00
2	21.71	8722.60
3	45.27	8714.24
4	69.94	8710.18
5	94.93	8710.57
6	119.47	8715.38
7	142.76	8724.46
8	164.07	8737.52
9	182.74	8754.16
10	198.15	8773.83
11	209.84	8795.94
12	212.19	8803.31

Circle Center At X = 80.18 ; Y = 8849.65 ; and Radius = 139.90

Factor of Safety
*** 23.224 ***

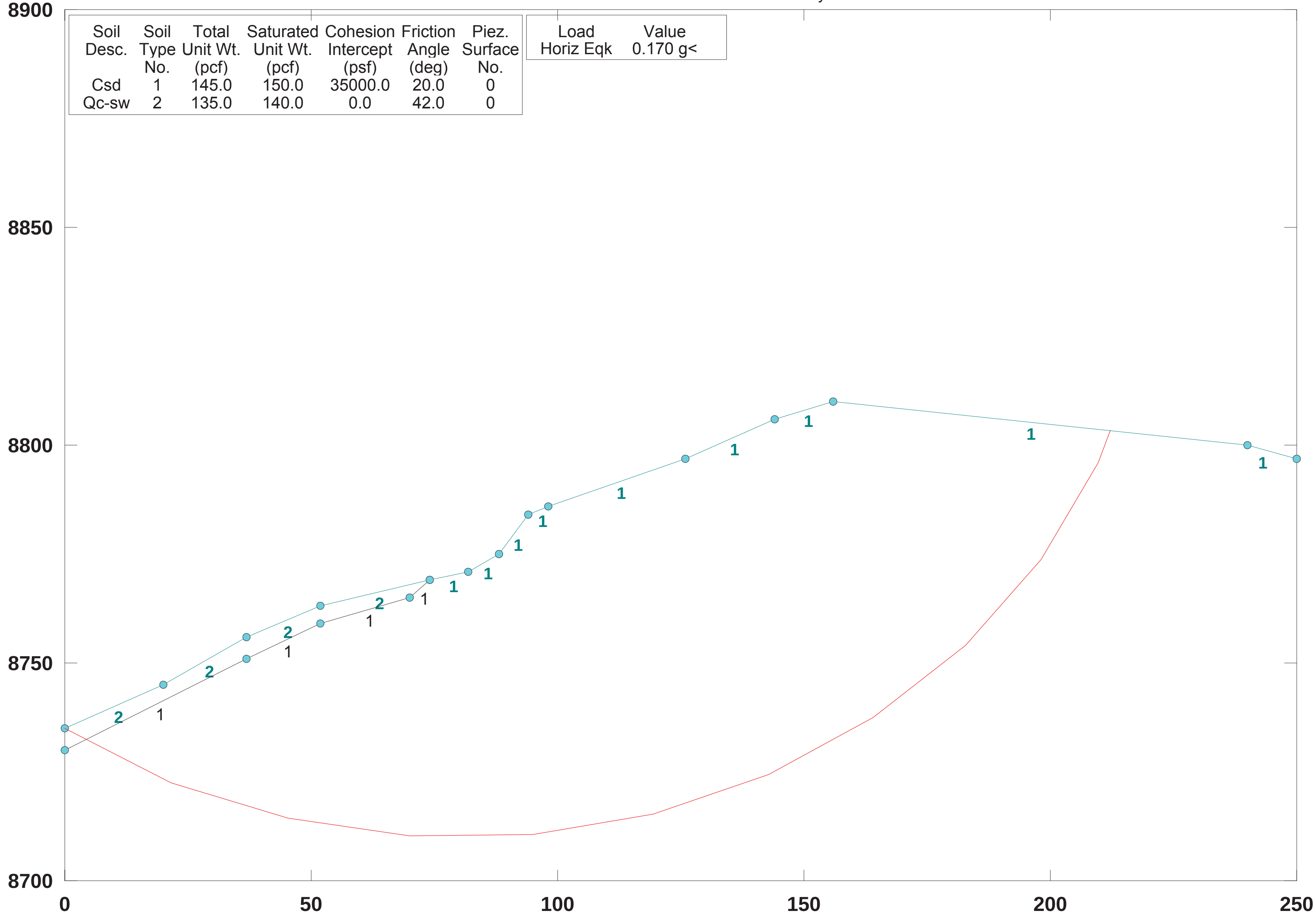
Individual data on the 24 slices

Slice No.	Width (ft)	Weight (lbs)	Water Force Top (lbs)	Water Force Bot (lbs)	Tie Force Norm (lbs)	Tie Force Tan (lbs)	Earthquake Force Hor (lbs)	Earthquake Force Ver (lbs)	Surcharge Load (lbs)
1	4.4	1393.5	0.0	0.0	0.	0.	0.0	0.0	0.0
2	15.6	28923.5	0.0	0.0	0.	0.	0.0	0.0	0.0
3	1.7	5495.5	0.0	0.0	0.	0.	0.0	0.0	0.0
4	15.3	68450.6	0.0	0.0	0.	0.	0.0	0.0	0.0
5	8.3	50225.7	0.0	0.0	0.	0.	0.0	0.0	0.0
6	6.7	46330.9	0.0	0.0	0.	0.	0.0	0.0	0.0
7	17.9	139273.8	0.0	0.0	0.	0.	0.0	0.0	0.0
8	0.1	532.7	0.0	0.0	0.	0.	0.0	0.0	0.0
9	4.0	33721.2	0.0	0.0	0.	0.	0.0	0.0	0.0
10	8.0	69244.3	0.0	0.0	0.	0.	0.0	0.0	0.0
11	6.0	54449.8	0.0	0.0	0.	0.	0.0	0.0	0.0
12	6.0	60024.9	0.0	0.0	0.	0.	0.0	0.0	0.0
13	0.9	9968.9	0.0	0.0	0.	0.	0.0	0.0	0.0
14	3.1	33069.8	0.0	0.0	0.	0.	0.0	0.0	0.0
15	21.5	239502.8	0.0	0.0	0.	0.	0.0	0.0	0.0
16	6.5	74906.4	0.0	0.0	0.	0.	0.0	0.0	0.0
17	16.8	194408.9	0.0	0.0	0.	0.	0.0	0.0	0.0
18	1.2	14547.2	0.0	0.0	0.	0.	0.0	0.0	0.0
19	12.0	137645.2	0.0	0.0	0.	0.	0.0	0.0	0.0
20	8.1	87187.7	0.0	0.0	0.	0.	0.0	0.0	0.0
21	18.7	168011.6	0.0	0.0	0.	0.	0.0	0.0	0.0
22	15.4	93680.7	0.0	0.0	0.	0.	0.0	0.0	0.0
23	11.7	32874.5	0.0	0.0	0.	0.	0.0	0.0	0.0
24	2.3	1303.3	0.0	0.0	0.	0.	0.0	0.0	0.0

**** END OF GSTABL7 OUTPUT ****

Summit/Ridge Nests; B-B'; Pseudo-Static

C:\DOCUME~1\DAVIDG\DESKTOP\01628008\B1P.PLT Run By: DAG 9/1/2015 2:17PM



Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
Csd	1	145.0	150.0	35000.0	20.0	0
Qc-sw	2	135.0	140.0	0.0	42.0	0

Load	Value
Horiz Eqk	0.170 g<

GSTABL7 v.2 FSmin=15.85

Factor Of Safety Is Calculated By The Modified Bishop Method

*** GSTABL7 ***

** GSTABL7 by Garry H. Gregory, P.E. **

** Original Version 1.0, January 1996; Current Version 2.002, December 2001 **

** 1

SLOPE STABILITY ANALYSIS SYSTEM
Modified Bishop, Simplified Janbu, or GLE Method of Slices.
(Includes Spencer & Morgenstern-Price Type Analysis)
Including Pier/Pile, Reinforcement, Soil Nail, Tieback,
Nonlinear Undrained Shear Strength, Curved Phi Envelope,
Anisotropic Soil, Fiber-Reinforced Soil, Boundary Loads, Water
Surfaces, Pseudo-Static Earthquake, and Applied Force Options.

****

Analysis Run Date:	9/1/2015
Time of Run:	2:17PM
Run By:	DAG
Input Data Filename:	C:b1p.
Output Filename:	C:b1p.OUT
Unit System:	English
Plotted Output Filename:	C:b1p.PLT

PROBLEM DESCRIPTION: Summit/Ridge Nests; B-B'; Pseudo-Static

1

BOUNDARY COORDINATES

13 Top Boundaries
17 Total Boundaries

Boundary No.	X-Left (ft)	Y-Left (ft)	X-Right (ft)	Y-Right (ft)	Soil Type Below Bnd
1	0.00	8735.00	20.00	8745.00	2
2	20.00	8745.00	37.00	8756.00	2
3	37.00	8756.00	52.00	8763.00	2
4	52.00	8763.00	74.00	8769.00	2
5	74.00	8769.00	82.00	8771.00	1
6	82.00	8771.00	88.00	8775.00	1
7	88.00	8775.00	94.00	8784.00	1
8	94.00	8784.00	98.00	8786.00	1
9	98.00	8786.00	126.00	8797.00	1
10	126.00	8797.00	144.00	8806.00	1
11	144.00	8806.00	156.00	8810.00	1
12	156.00	8810.00	240.00	8800.00	1
13	240.00	8800.00	250.00	8797.00	1
14	0.00	8730.00	37.00	8751.00	1
15	37.00	8751.00	52.00	8759.00	1
16	52.00	8759.00	70.00	8765.00	1
17	70.00	8765.00	74.00	8769.00	1

User Specified Y-Origin = 8700.00(ft)

ISOTROPIC SOIL PARAMETERS

2 Type(s) of Soil

Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
1	145.0	150.0	35000.0	20.0	0.00	0.0	0
2	135.0	140.0	0.0	42.0	0.00	0.0	0

A Horizontal Earthquake Loading Coefficient
Of0.170 Has Been Assigned

A Vertical Earthquake Loading Coefficient
Of0.000 Has Been Assigned

Cavitation Pressure = 0.0(psf)

Trial Failure Surface Specified By 12 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	0.00	8735.00
2	21.71	8722.60
3	45.27	8714.24
4	69.94	8710.18
5	94.93	8710.57
6	119.47	8715.38
7	142.76	8724.46
8	164.07	8737.52
9	182.74	8754.16
10	198.15	8773.83
11	209.84	8795.94
12	212.19	8803.31

Circle Center At X = 80.18 ; Y = 8849.66; and Radius = 139.91

* * Factor Of Safety Is Calculated By The Modified Bishop Method * *

Factor Of Safety For The Preceding Specified Surface = 15.853

Table 1 - Individual Data on the 24 Slices

Slice No.	Width (ft)	Weight (lbs)	Water Force Top (lbs)	Water Force Bot (lbs)	Tie Force Norm (lbs)	Tie Force Tan (lbs)	Earthquake Force Hor (lbs)	Earthquake Force Ver (lbs)	Surcharge Load (lbs)
1	4.4	1393.9	0.0	0.0	0.0	0.0	237.0	0.0	0.0
2	15.6	28915.2	0.0	0.0	0.0	0.0	4915.6	0.0	0.0
3	1.7	5506.8	0.0	0.0	0.0	0.0	936.2	0.0	0.0
4	15.3	68427.1	0.0	0.0	0.0	0.0	11632.6	0.0	0.0
5	8.3	50240.6	0.0	0.0	0.0	0.0	8540.9	0.0	0.0
6	6.7	46305.8	0.0	0.0	0.0	0.0	7872.0	0.0	0.0
7	17.9	139304.0	0.0	0.0	0.0	0.0	23681.7	0.0	0.0
8	0.1	500.4	0.0	0.0	0.0	0.0	85.1	0.0	0.0
9	4.0	33722.3	0.0	0.0	0.0	0.0	5732.8	0.0	0.0
10	8.0	69245.4	0.0	0.0	0.0	0.0	11771.7	0.0	0.0
11	6.0	54448.9	0.0	0.0	0.0	0.0	9256.3	0.0	0.0
12	6.0	60022.4	0.0	0.0	0.0	0.0	10203.8	0.0	0.0
13	0.9	9934.3	0.0	0.0	0.0	0.0	1688.8	0.0	0.0
14	3.1	33101.9	0.0	0.0	0.0	0.0	5627.3	0.0	0.0
15	21.5	239530.2	0.0	0.0	0.0	0.0	40720.1	0.0	0.0
16	6.5	74863.1	0.0	0.0	0.0	0.0	12726.7	0.0	0.0
17	16.8	194411.2	0.0	0.0	0.0	0.0	33049.9	0.0	0.0
18	1.2	14537.0	0.0	0.0	0.0	0.0	2471.3	0.0	0.0
19	12.0	137640.1	0.0	0.0	0.0	0.0	23398.8	0.0	0.0
20	8.1	87144.3	0.0	0.0	0.0	0.0	14814.5	0.0	0.0
21	18.7	168081.1	0.0	0.0	0.0	0.0	28573.8	0.0	0.0
22	15.4	93633.2	0.0	0.0	0.0	0.0	15917.6	0.0	0.0
23	11.7	32886.2	0.0	0.0	0.0	0.0	5590.7	0.0	0.0
24	2.4	1303.4	0.0	0.0	0.0	0.0	221.6	0.0	0.0

Table 2 - Base Stress Data on the 24 Slices

Slice No. *	Alpha (deg)	X-Coord. Slice Cntr (ft)	Base Leng. (ft)	Available Shear Strength (psf)	Mobilized Shear Stress (psf)
1	-29.73	2.20	5.06	295.43	-124.01
2	-29.73	12.20	17.98	36148.28	-794.23
3	-29.73	20.85	1.97	36652.78	-1354.29
4	-19.54	29.35	16.22	36929.70	-1406.37
5	-19.54	41.14	8.78	37516.75	-1907.15
6	-9.35	48.64	6.82	37646.56	-1093.20
7	-9.35	60.97	18.18	37969.71	-1240.83
8	0.90	69.97	0.06	38022.06	1201.68
9	0.90	72.00	4.00	38054.82	147.82
10	0.90	78.00	8.00	38136.72	143.30
11	0.90	85.00	6.00	38289.22	152.53
12	0.90	91.00	6.00	38627.20	167.05
13	0.90	94.46	0.93	38873.98	236.05
14	11.09	96.46	3.13	38750.11	2055.61
15	11.09	108.74	21.88	38885.66	2108.61
16	21.30	122.74	7.01	38825.21	3889.09
17	21.30	134.38	17.99	38873.99	3929.26
18	31.50	143.38	1.45	38722.12	5267.13
19	31.50	150.00	14.07	38631.20	5114.63
20	31.50	160.04	9.46	38390.19	4817.70
21	41.71	173.40	25.01	37509.21	4474.27
22	51.93	190.45	24.99	36152.02	2952.30
23	62.13	203.99	25.01	34524.71	1165.00
24	72.31	211.01	7.74	32837.45	168.84

Sum of the Resisting Forces (including Pier/Pile, Tieback, Reinforcing Soil Nail, and Applied Forces if applicable) = 9422019.00 (lbs)

Average Available Shear Strength (including Tieback, Pier/Pile, Reinforcing, Soil Nail, and Applied Forces if applicable) = 36556.73(psf)

Sum of the Driving Forces = 594333.94 (lbs)

Average Mobilized Shear Stress = 2305.97(psf)

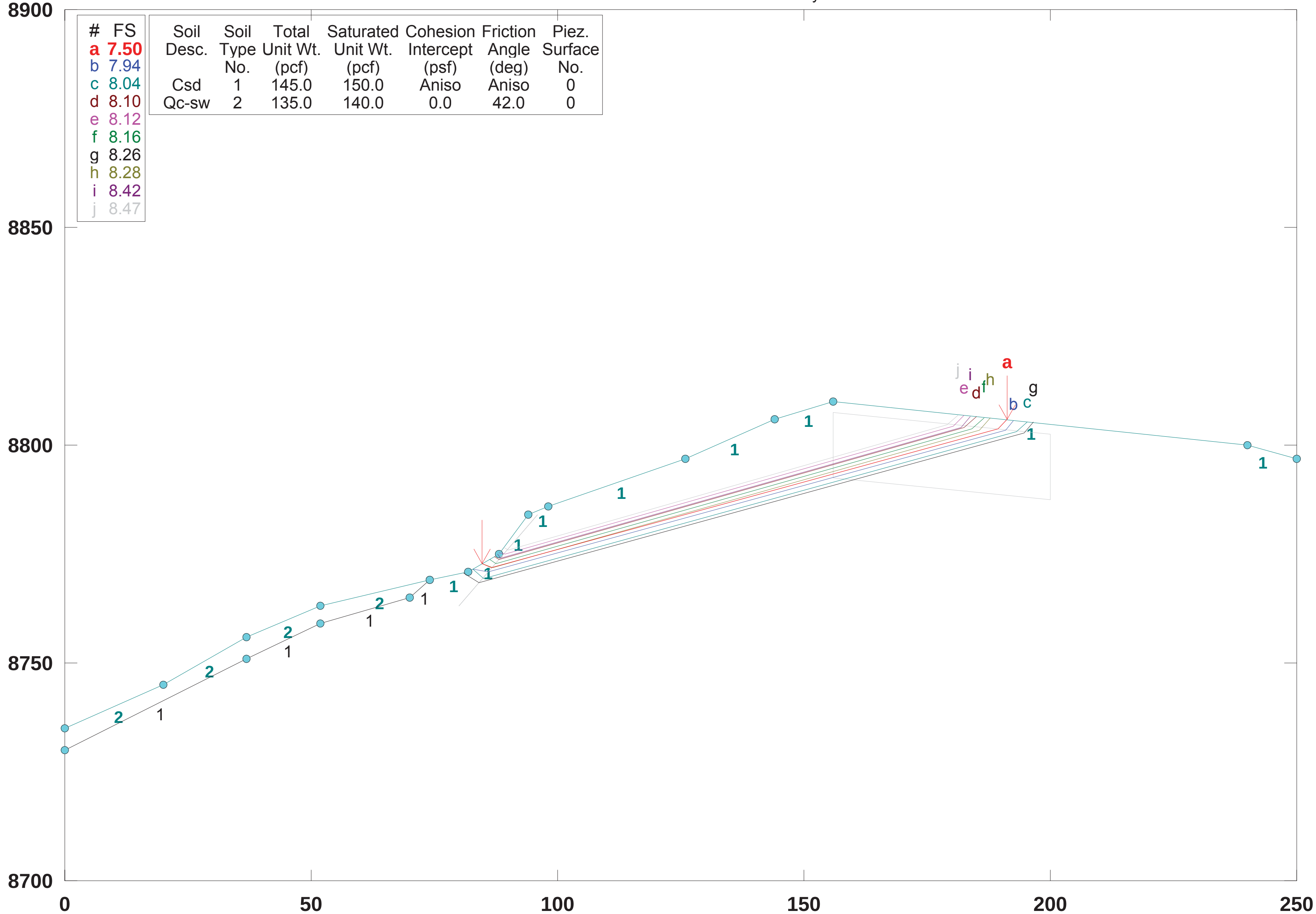
Total length of the failure surface = 257.74(ft)

CAUTION - Factor Of Safety Is Calculated By The Modified Bishop Method. This Method Is Valid Only If The Failure Surface Approximates A Circular Arc.

**** END OF GSTABL7 OUTPUT ****

Summit/Ridge Nests; B-B'; Static; bedding 17 deg apparent dip

C:\DOCUME~1\DAVIDG\DESKTOP\01628008\B2.PL2 Run By: DAG 9/1/2015 2:17PM



GSTABL7 v.2 FSmin=7.50

Safety Factors Are Calculated By The Simplified Janbu Method for the case of c & phi both > 0

*** GSTABL7 ***

** GSTABL7 by Garry H. Gregory, P.E. **

** Original Version 1.0, January 1996; Current Version 2.002, December 2001 **

*** 1

SLOPE STABILITY ANALYSIS SYSTEM

Modified Bishop, Simplified Janbu, or GLE Method of Slices.
(Includes Spencer & Morgenstern-Price Type Analysis)
Including Pier/Pile, Reinforcement, Soil Nail, Tieback,
Nonlinear Undrained Shear Strength, Curved Phi Envelope,
Anisotropic Soil, Fiber-Reinforced Soil, Boundary Loads, Water
Surfaces, Pseudo-Static Earthquake, and Applied Force Options.

****

Analysis Run Date: 9/1/2015
Time of Run: 2:17PM
Run By: DAG
Input Data Filename: C:b2.
Output Filename: C:b2.OUT
Unit System: English

Plotted Output Filename: C:b2.PLT

PROBLEM DESCRIPTION: Summit/Ridge Nests; B-B'; Static; beddin
g 17 deg apparent dip

BOUNDARY COORDINATES

13 Top Boundaries
17 Total Boundaries

Boundary No.	X-Left (ft)	Y-Left (ft)	X-Right (ft)	Y-Right (ft)	Soil Type Below Bnd
1	0.00	8735.00	20.00	8745.00	2
2	20.00	8745.00	37.00	8756.00	2
3	37.00	8756.00	52.00	8763.00	2
4	52.00	8763.00	74.00	8769.00	2
5	74.00	8769.00	82.00	8771.00	1
6	82.00	8771.00	88.00	8775.00	1
7	88.00	8775.00	94.00	8784.00	1
8	94.00	8784.00	98.00	8786.00	1
9	98.00	8786.00	126.00	8797.00	1
10	126.00	8797.00	144.00	8806.00	1
11	144.00	8806.00	156.00	8810.00	1
12	156.00	8810.00	240.00	8800.00	1
13	240.00	8800.00	250.00	8797.00	1
14	0.00	8730.00	37.00	8751.00	1
15	37.00	8751.00	52.00	8759.00	1
16	52.00	8759.00	70.00	8765.00	1
17	70.00	8765.00	74.00	8769.00	1

1

User Specified Y-Origin = 8700.00(ft)

ISOTROPIC SOIL PARAMETERS

2 Type(s) of Soil

Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
1	145.0	150.0	35000.0	20.0	0.00	0.0	0
2	135.0	140.0	0.0	42.0	0.00	0.0	0

ANISOTROPIC STRENGTH PARAMETERS
1 soil type(s)

Soil Type 1 Is Anisotropic

Number Of Direction Ranges Specified = 3

Direction Range No.	Counterclockwise Direction Limit (deg)	Cohesion Intercept (psf)	Friction Angle (deg)
1	12.5	35000.00	20.00
2	22.5	0.00	42.00
3	90.0	35000.00	20.00

ANISOTROPIC SOIL NOTES:
(1) An input value of 0.01 for C and/or Phi will cause Aniso C and/or Phi to be ignored in that range.
(2) An input value of 0.02 for Phi will set both Phi and C equal to zero, with no water weight in the tension crack.
(3) An input value of 0.03 for Phi will set both Phi and C equal to zero, with water weight in the tension crack.

Janbus Empirical Coef is being used for the case of c & phi both > 0

A Critical Failure Surface Searching Method, Using A Random Technique For Generating Sliding Block Surfaces, Has Been Specified.

2000 Trial Surfaces Have Been Generated.

2 Boxes Specified For Generation Of Central Block Base

Length Of Line Segments For Active And Passive Portions Of Sliding Block Is 30.0

Box No.	X-Left (ft)	Y-Left (ft)	X-Right (ft)	Y-Right (ft)	Height (ft)
1	80.00	8763.00	96.00	8784.00	0.00
2	156.00	8800.00	200.00	8795.00	15.00

Following Is Displayed The Ten Most Critical Of The Trial Failure Surfaces Evaluated.

* * Safety Factors Are Calculated By The Simplified Janbu Method * *

Total Number of Trial Surfaces Evaluated = 2000

Statistical Data On All Valid FS Values:
FS Max = 460.234   FS Min = 7.500   FS Ave = 60.740
Standard Deviation = 57.325   Coefficient of Variation = 94.38 %

Failure Surface Specified By 4 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	84.70	8772.80
2	86.67	8771.75
3	189.44	8803.62
4	191.25	8805.80

Factor of Safety
*** 7.500 ***

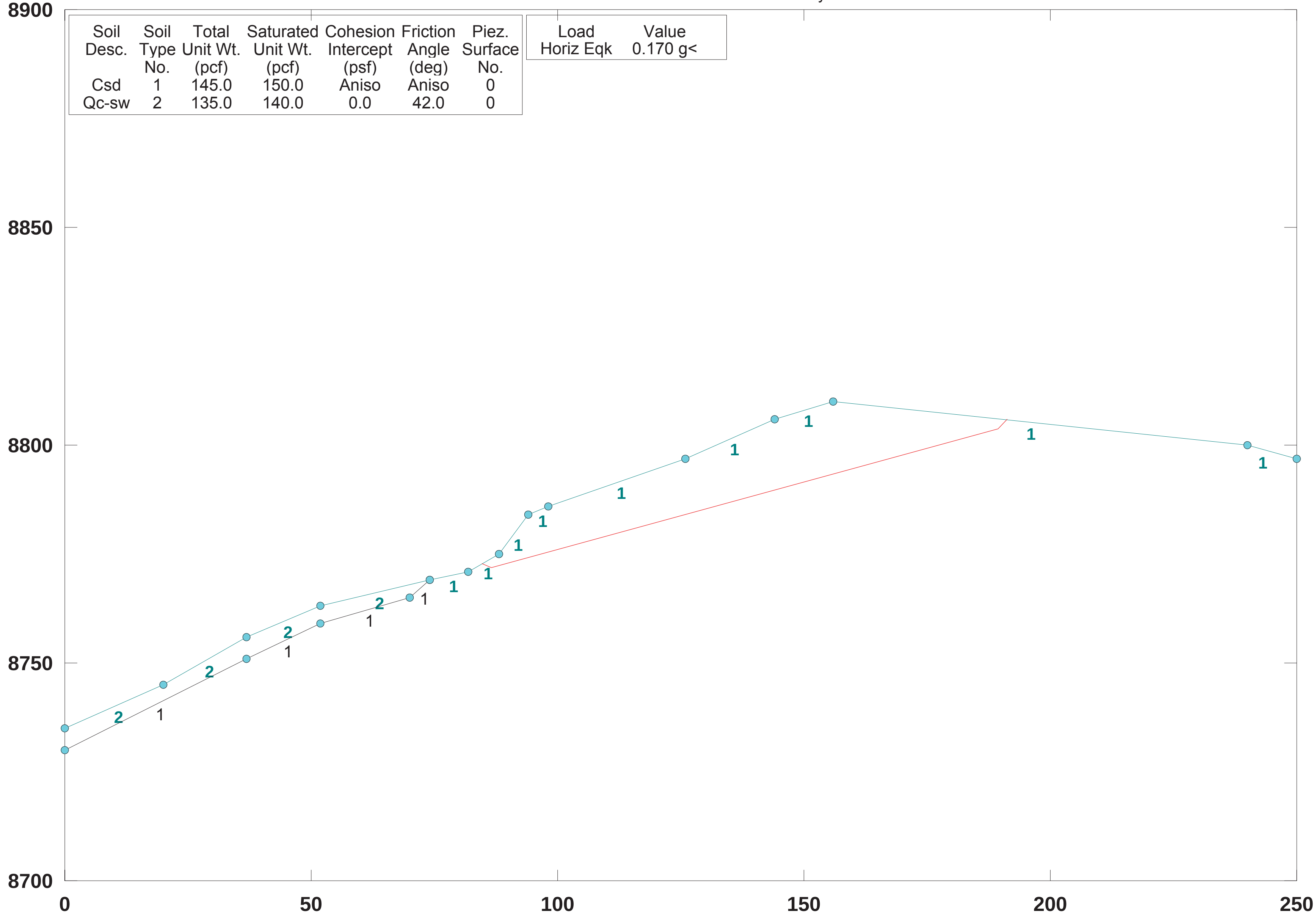
Individual data on the 9 slices

Slice No.	Width (ft)	Weight (lbs)	Water Force Top (lbs)	Water Force Bot (lbs)	Tie Force Norm (lbs)	Tie Force Tan (lbs)	Earthquake Force Hor (lbs)	Earthquake Force Ver (lbs)	Surcharge Load (lbs)
1	2.0	337.4	0.0	0.0	0.	0.	0.0	0.0	0.0
2	1.3	502.3	0.0	0.0	0.	0.	0.0	0.0	0.0
3	6.0	5573.4	0.0	0.0	0.	0.	0.0	0.0	0.0
4	4.0	6006.2	0.0	0.0	0.	0.	0.0	0.0	0.0
5	28.0	48291.8	0.0	0.0	0.	0.	0.0	0.0	0.0
6	18.0	38528.1	0.0	0.0	0.	0.	0.0	0.0	0.0
7	12.0	28902.0	0.0	0.0	0.	0.	0.0	0.0	0.0
8	33.4	46425.1	0.0	0.0	0.	0.	0.0	0.0	0.0
9	1.8	315.2	0.0	0.0	0.	0.	0.0	0.0	0.0

**** END OF GSTABL7 OUTPUT ****

Summit/Ridge Nests; B-B'; Pseudo-Static;bedding 17 deg apparent dip

C:\DOCUME~1\DAVIDG\DESKTOP\01628008\B2P.PLT Run By: DAG 9/1/2015 2:18PM



GSTABL7 v.2 FSmin=4.77

Factor Of Safety Is Calculated By The Simplified Janbu Method

*** GSTABL7 ***

** GSTABL7 by Garry H. Gregory, P.E. **

** Original Version 1.0, January 1996; Current Version 2.002, December 2001 **

****

1

SLOPE STABILITY ANALYSIS SYSTEM
Modified Bishop, Simplified Janbu, or GLE Method of Slices.
(Includes Spencer & Morgenstern-Price Type Analysis)
Including Pier/Pile, Reinforcement, Soil Nail, Tieback,
Nonlinear Undrained Shear Strength, Curved Phi Envelope,
Anisotropic Soil, Fiber-Reinforced Soil, Boundary Loads, Water
Surfaces, Pseudo-Static Earthquake, and Applied Force Options.

****

Analysis Run Date: 9/1/2015
Time of Run: 2:18PM
Run By: DAG
Input Data Filename: C:b2p.
Output Filename: C:b2p.OUT
Unit System: English

Plotted Output Filename: C:b2p.PLT

PROBLEM DESCRIPTION: Summit/Ridge Nests; B-B'; Pseudo-Static;
bedding 17 deg apparent dip

BOUNDARY COORDINATES

13 Top Boundaries
17 Total Boundaries

Boundary No.	X-Left (ft)	Y-Left (ft)	X-Right (ft)	Y-Right (ft)	Soil Type Below Bnd
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6	82.00	8771.00	88.00	8775.00	1
7	88.00	8775.00	94.00	8784.00	1
8	94.00	8784.00	98.00	8786.00	1
9	98.00	8786.00	126.00	8797.00	1
10	126.00	8797.00	144.00	8806.00	1
11	144.00	8806.00	156.00	8810.00	1
12	156.00	8810.00	240.00	8800.00	1
13	240.00	8800.00	250.00	8797.00	1
14	0.00	8730.00	37.00	8751.00	1
15	37.00	8751.00	52.00	8759.00	1
16	52.00	8759.00	70.00	8765.00	1
17	70.00	8765.00	74.00	8769.00	1

User Specified Y-Origin = 8700.00(ft)

ISOTROPIC SOIL PARAMETERS

2 Type(s) of Soil

Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
1	145.0	150.0	35000.0	20.0	0.00	0.0	0
2	135.0	140.0	0.0	42.0	0.00	0.0	0

ANISOTROPIC STRENGTH PARAMETERS
1 soil type(s)

Soil Type 1 Is Anisotropic

Number Of Direction Ranges Specified = 3

Direction Range No.	Counterclockwise Direction Limit (deg)	Cohesion Intercept (psf)	Friction Angle (deg)
1	12.5	35000.00	20.00
2	22.5	0.00	42.00
3	90.0	35000.00	20.00

ANISOTROPIC SOIL NOTES:
(1) An input value of 0.01 for C and/or Phi will cause Aniso C and/or Phi to be ignored in that range.
(2) An input value of 0.02 for Phi will set both Phi and C equal to zero, with no water weight in the tension crack.
(3) An input value of 0.03 for Phi will set both Phi and C equal to zero, with water weight in the tension crack.

A Horizontal Earthquake Loading Coefficient
Of0.170 Has Been Assigned

A Vertical Earthquake Loading Coefficient
Of0.000 Has Been Assigned

Cavitation Pressure = 0.0(psf)

Janbu's Empirical Coef. is being used for the case of c & phi both > 0

1

Trial Failure Surface Specified By 4 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	84.70	8772.80
2	86.67	8771.75
3	189.44	8803.62
4	191.25	8805.80

Janbu's Empirical Coefficient (fo) = 1.007

* * Factor Of Safety Is Calculated By The Simplified Janbu Method * *

Factor Of Safety For The Preceding Specified Surface = 4.768

Table 1 - Individual Data on the 9 Slices

Slice No.	Width (ft)	Weight (lbs)	Water Force Top (lbs)	Water Force Bot (lbs)	Tie Force Norm (lbs)	Tie Force Tan (lbs)	Earthquake Force Hor (lbs)	Earthquake Force Ver (lbs)	Surcharge Load (lbs)
1	2.0	337.5	0.0	0.0	0.0	0.0	57.4	0.0	0.0
2	1.3	501.5	0.0	0.0	0.0	0.0	85.3	0.0	0.0
3	6.0	5574.3	0.0	0.0	0.0	0.0	947.6	0.0	0.0
4	4.0	6006.7	0.0	0.0	0.0	0.0	1021.1	0.0	0.0
5	28.0	48295.8	0.0	0.0	0.0	0.0	8210.3	0.0	0.0
6	18.0	38530.6	0.0	0.0	0.0	0.0	6550.2	0.0	0.0
7	12.0	28903.7	0.0	0.0	0.0	0.0	4913.6	0.0	0.0
8	33.4	46428.2	0.0	0.0	0.0	0.0	7892.8	0.0	0.0
9	1.8	314.7	0.0	0.0	0.0	0.0	53.5	0.0	0.0

Table 2 - Base Stress Data on the 9 Slices

Slice No. *	Alpha (deg)	X-Coord. Slice Cntr (ft)	Base Leng. (ft)	Available Shear Strength (psf)	Mobilized Shear Stress (psf)
1	-28.05	85.68	2.23	41414.95	-54.87
2	17.23	87.33	1.39	335.81	172.92
3	17.23	91.00	6.28	827.36	426.02
4	17.23	96.00	4.19	1337.32	688.61
5	17.23	112.00	29.32	1536.06	790.95
6	17.23	135.00	18.85	1906.30	981.59
7	17.23	150.00	12.56	2145.01	1104.51
8	17.23	172.72	35.01	1236.44	636.67
9	50.36	190.35	2.84	50321.15	152.76

Sum of the Resisting Forces (including Pier/Pile, Tieback, Reinforcing
Soil Nail, and Applied Forces if applicable) = 397547.47 (lbs)

Average Available Shear Strength (including Tieback, Pier/Pile, Reinforcing,
Soil Nail, and Applied Forces if applicable) = 3528.51(psf)

Sum of the Driving Forces = 83964.20 (lbs)

Average Mobilized Shear Stress = 745.24(psf)

Total length of the failure surface = 112.67(ft)

**** END OF GSTABL7 OUTPUT ****

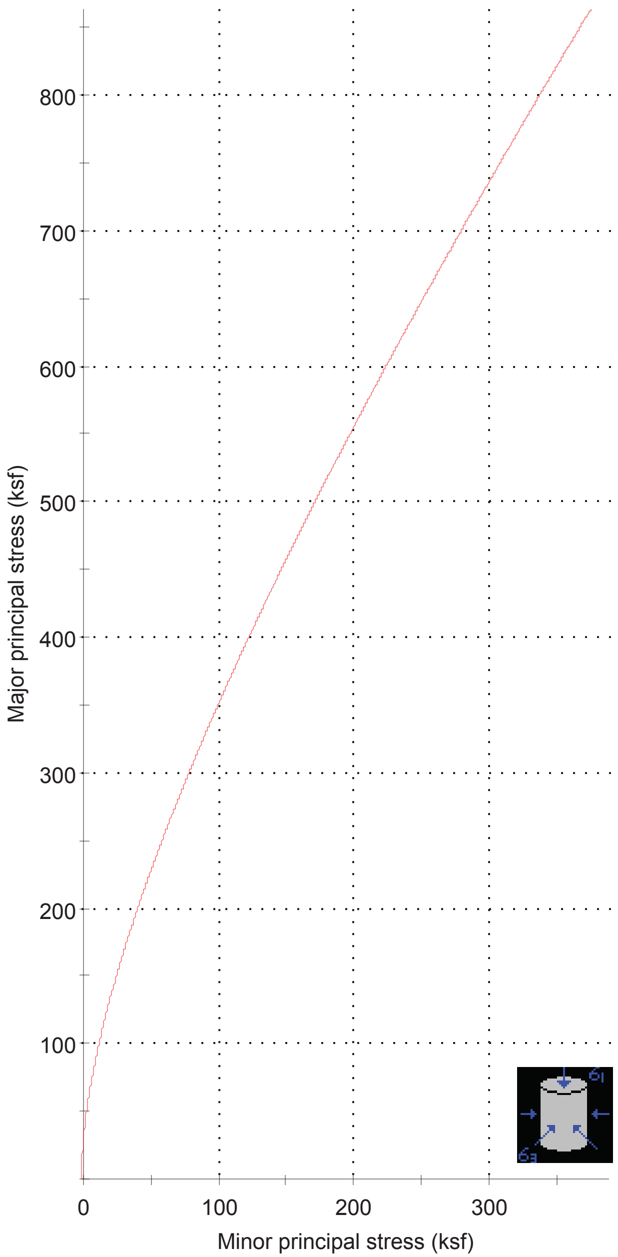
Analysis of Rock Strength using RocLab

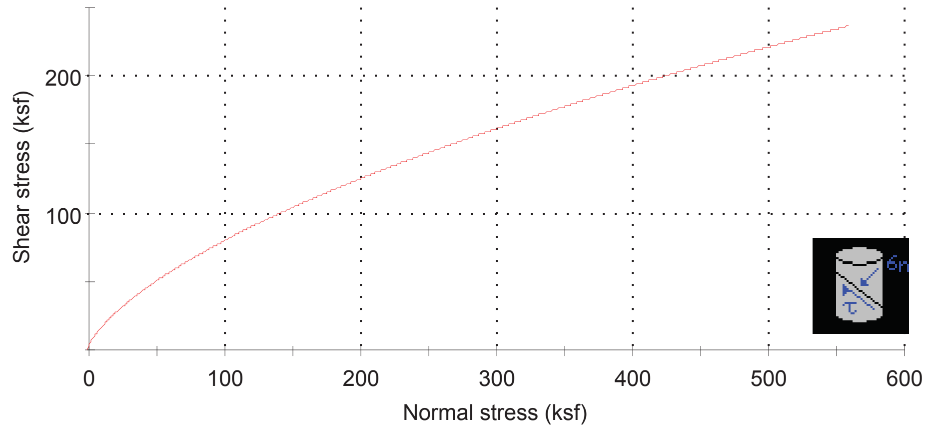
Hoek-Brown Classification
intact uniaxial comp. strength (sigci) = 1500 ksf
GSI = 45   mi = 9   Disturbance factor (D) = 0.7
intact modulus (Ei) = 637500 ksf
modulus ratio (MR) = 425

Hoek-Brown Criterion
mb = 0.438   s = 0.0003   a = 0.508

Mohr-Coulomb Fit
cohesion = 44.844 ksf   friction angle = 20.09 deg

Rock Mass Parameters
tensile strength = -1.182 ksf
uniaxial compressive strength = 26.135 ksf
global strength = 128.309 ksf
deformation modulus = 49889.73 ksf





IGES®
Intermountain GeoEnvironmental Services, Inc.
12429 South 300 East, Suite 100, Draper, Utah 84020 ~ T: (801) 748-4044 ~ F: (801) 748-4045

November 4, 2015

Summit Powder Mountain
c/o Ms. Andrea Milner
3632 North Wolf Creek Drive
Eden, Utah 84310

IGES Project No. 01628-008

Subject: Response to Additional Review Comments - Geology
Geotechnical Investigation
The Ridge Nests Development
Powder Mountain Resort
Weber and Cache Counties, Utah

Ms. Milner:

As requested, IGES has prepared the following response to additional review comments regarding the referenced geotechnical report and first review response dated September 23, 2015 for the Ridge Nests development, part of the larger Powder Mountain Resort expansion project in Weber County, Utah. The review comments to be addressed were prepared by Simon Associates LLC (SA) in a letter dated October 14, 2015; the latest comments by SA are in regard to the review response by IGES (2015c), which was prepared in response to SA's first geologic review letter (SA, 2015a) that was regarding the original geotechnical report by IGES (2015a).

The review letter by SA was intended to address Lot 13; however, in consideration that the comments by SA could also be applicable to several other lots, it is the intention of IGES to address the comments with respect to the entire Ridge Nests development. For convenience, the review comments will be presented first, followed by our response.

Comment No. 1

"*The September 23, 2015, IGES response letter did not describe the properties of the bedding and/or jointing for incorporation into the slope stability analyses, e.g., properties such as, strike and dip, degree of fracturing (generally controlled by the number of joints in a given direction), persistence of jointing, spacing of jointing, roughness of joint surface, open and/or closed joints, joint coatings and infillings, etc.*

Should the Weber County Consulting Geotechnical Engineer consider the properties of bedding, joints, and/or fractures pertinent in regards to slope stability analyses presented in the September 23, 2015 IGES response letter, SA recommends Weber County request documentation of the bedding, joint, and/or fracture properties, and incorporation of the geologic data in the slope stability analyses."

Response to Comment No. 1

IGES did describe the strike and dip of the bedding and jointing in the September 23, 2015 response letter; IGES noted bedding near the subject site was oriented (strike) about N24°W and dip (inclination from the horizontal) at 25°NE. The bedrock was found to have blocky jointing, with the two major sets being orthogonal to one another. One joint set was parallel to the bedding, and the other was perpendicular to the bedding, dipping steeply to the southwest. The joint set parallel to the bedding has the same strike and dip orientation as the bedding, while the other major joint set perpendicular to the first has a strike of approximately N24°W and a dip of approximately 65°SW.

In response to the comment, the following additional details are provided: bedrock was found to be largely moderately fractured (distance between fractures ~0.5-1.0 feet) to little fractured (distance between fractures ~1.0-4.0 feet), with localized areas of intense fracturing (distance between fractures ~0.05-0.1 feet). Joint spacing was largely found to be a product of the lithology. The finer-grained dolomite lithologies were more thinly bedded, and therefore had a smaller distance (approximately 1 to 4 inches) between bedding plane joints. These lithologies also tended to fracture into rectangular blocks generally between 4 and 18 inches in length and width, and contained both bedding-confined and through-going fractures (Photo 1). Coarser-grained dolomite lithologies were more thickly bedded to massive, with bedding plane joints separated by between 6 inches to as much as several feet. These lithologies tended to fracture into rectangular blocks with highly variable dimensions, ranging in width and length from between a couple inches to several feet, though larger blocks (with dimensions of several feet x several feet x several feet) were most common (Photo 2). Most fracturing associated with the coarser-grained dolomite lithologies consisted of large through-going fractures.

Nearly all of the joints encountered in the field investigation were open, had slightly rough to rough surfaces, and did not contain a secondary mineralization, except rare calcite infilling in places. No slickensides were observed on any joint surface. Joint apertures varied from between a few millimeters to a couple inches in width. Joints with smaller apertures tended to be devoid of any sort of fill, while the larger aperture joints were often filled with soil. In the cases of the two identified faults, reddish gray silty gouge was found to be the fill material.

Photo 1. Finer-grained dolomite lithology, exhibiting thinner beds and blocky jointing.

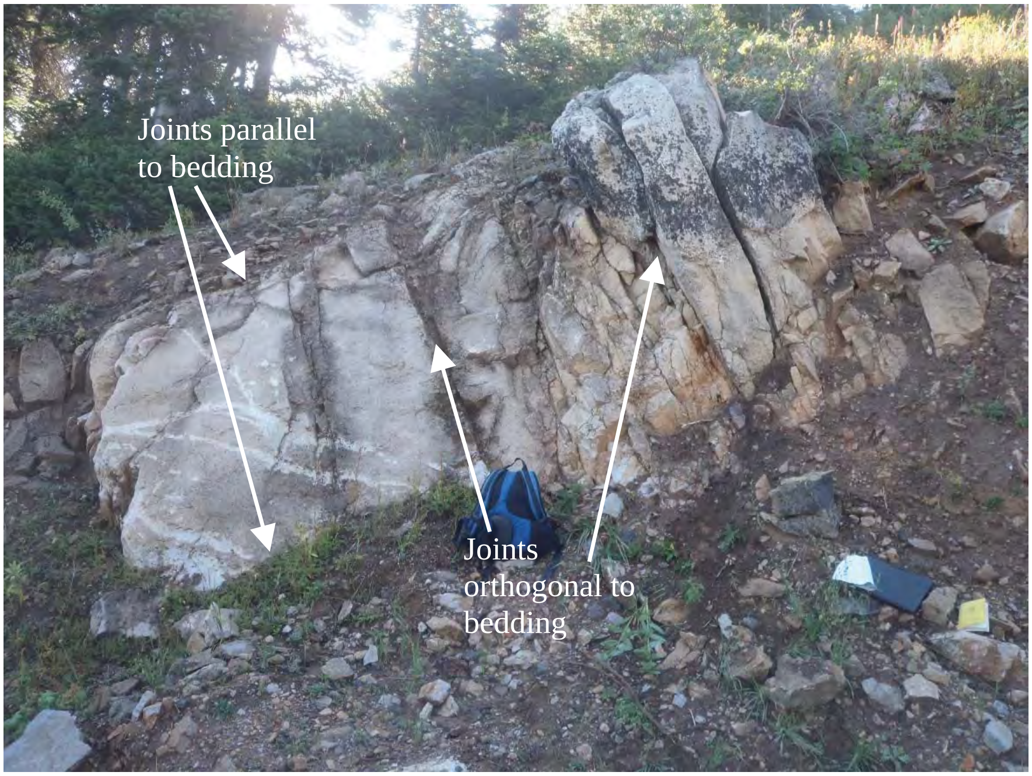


Photo 2. Coarser-grained dolomite lithology, exhibiting thicker beds and wider jointing.

The preceding bedrock characteristics were discussed between the engineering geologist and the geotechnical engineer and were taken into consideration in development of the subsurface model, geologic cross section, and subsequent slope stability analysis.

Comment No. 2

"*SA recommends Weber County request IGES provide definitions for "inactive" fault, "drastic deformation," and "ancient geologic past". Without definitions, applicability of the above factors to determine timing of surface-fault-rupture are difficult to evaluate. However, regardless of the definitions, SA considers several of the factors not to be applicable in regards to timing of surface-fault-rupture. For instance:*

a. "The fault extends up to, but not through, the overlying profile*." Without the age of the overlying soil profile, the statement is unsubstantiated.*

b. "Abundant vegetation is present above the fault trace, and is not offset or disturbed in any way*." Without an age of the vegetation, the statement is unsubstantiated.*

c. "The fact that the footwall block shows such drastic deformation not seen elsewhere on the property suggests that the displacement happened in the ancient geologic past, and subsequent geomorphic processes have returned the bedrock block back to stable topographic conditions across the fault trace*." In regards to determining timing of surface-fault rupture, SA is not aware of any paleoseismic studies correlating:*

i. "…*drastic deformation*" to displacement occurring in the "ancient geologic past."
ii. *The use of* "…subsequent geomorphic processes…[returning] bedrock blocks back to stable topographic conditions across a fault trace.""

Additionally, SA recommends Weber County suggest IGES consider the following, long established standard of practice, methods for evaluating the potential for surface-fault-rupture along the documented faults:

a. "Review of aerial photographs and surface observations to identify any fault-related geomorphic features indicative of past surface faulting at or near the property (e.g., fault scarps, vegetation lineaments, gullies, vegetation/soil contrasts, aligned springs and seeps, sag ponds, aligned or disrupted drainages, faceted spurs, grabens, and/or displaced landforms such as terraces, shorelines, geologic units, etc.).

b. "The USGS Quaternary Fault and Fold Database of the United States. (http://earthquake.usgs.gov/hazards/qfaults)."

Response to Comment No. 2

In the context of the IGES submitted letter on September 23, 2015, the following definitions are to be used in association with the terms or phrases in question:

- *Inactive fault*: a fault in which displacement of greater than 4 inches has not been observed to have occurred along one or more of its traces within Holocene time (~11,000 years ago-present). (Weber County Wiki for Natural Hazards Overlay Districts, Chapter 38-3).
- *Drastic deformation*: deformation that is anomalous to the existing geologic framework.
- *Ancient geologic past*: relating to the past in terms of millions of years, as opposed to thousands of years.

With regards to the timing of surface-fault-rupture, it should be noted that three of the four factors identified by IGES to demonstrate that the faults are inactive faults are to be taken as individual pieces of evidence that collectively indicate fault inactivity. Each piece of data provides geologic support for the cumulative conclusion that the faults are inactive, and are discussed individually below.

SA comments: "*Without the age of the overlying soil profile, the statement is unsubstantiated*."

Though the age of the soil profile overlying the faults is unknown, the presence of undisturbed soil provides a lower limit for most recent displacement along the fault traces. Soil formation can take hundreds to thousands of years to develop. Taking the conservative estimate of 100 years per inch of topsoil development (NRCS)[1], and the fact that 3.5 feet of soil were encountered in TP-1, provides a lower limit of at least 3,600 years since last displacement along the faults.

SA comments: "*Without the age of the vegetation, the statement is unsubstantiated*."

IGES concedes to the reviewer that offset of individual trees or other flora is generally not applicable for timing of fault movement. However, no alignment, pattern, or offset of vegetation was observed either in the site visit or apparent in Google Earth imagery. This suggests a lack of surficial expression of the fault traces.

SA comments: "*In regards to determining timing of surface-fault-rupture, SA is not aware of any paleoseismic studies correlating*:

iii. "*…drastic deformation" to displacement occurring in the* "ancient geologic past."
iv. *The use o*f "…subsequent geomorphic processes…[returning] bedrock blocks back to stable topographic conditions across a fault trace.""

The drastic deformation identified in this specific instance is such that there is no synchronous relationship between the event that caused the deformation and the current geologic setting for this particular area. In other words, the deformation noted on the footwall block of one of the faults, in steeply dipping to the southeast, is completely out of place from any other geologic data present at the location and is localized (e.g., restricted to the fault block). Because this deformation has no apparent relationship with any of the other geologic data present, the logical conclusion is that the event that caused the deformation (movement along the fault) occurred in

[1] http://www.nrcs.usda.gov/wps/portal/nrcs/detail/wa/soils/?cid=nrcs144p2_036333

the "ancient geologic past," otherwise similar geologic data would be present in the area, e.g. geomorphic expression of the fault.

Given that the deformation is associated with a fault trace, it is therefore to be understood that the deformation was the product of at least one but likely multiple major seismic events. Such seismic events are likely to have resulted in the production of a fault scarp exposed at the surface, but currently no such scarp is present and there are gentle topographic conditions across the fault trace. These stable topographic conditions would have subsequently been produced by geomorphic processes that would have slowly eroded away the fault scarp, leaving the existing gentle topography encountered today.

Additionally, it should be noted that the faults in question are passing through very hard bedrock comprised of dolomite, and not unconsolidated sediment. Had these faults been active during Holocene time (with a minimum of 4 inches of displacement) the activity would have produced a bedrock fault scarp exhibiting at least 4 inches of displacement. This dolomitic bedrock is very resistant to weathering and erosion as evidenced by its cliff-forming character, and its presence at the top of the ridges found in the surrounding areas. Whereas it may be likely that 4+ inches of unconsolidated material offset by a fault may be removed by weathering and erosion processes during Holocene time, it is conversely highly unlikely that 4+ inches of hard bedrock fault scarp would be removed over this same time interval, especially given the climatic conditions at the site compared to weathering rates found in industrial environments (Gauri et al., 1992). The absence of a fault scarp under these conditions, therefore, is evidence that there has not been surface-fault-rupture with greater than 4 inches of displacement during Holocene time.

SA Comment regarding *"Review of aerial photographs and surface observations to identify any fault-related geomorphic features indicative of past surface faulting at or near the property (e.g., fault scarps, vegetation lineaments, gullies, vegetation/soil contrasts, aligned springs and seeps, sag ponds, aligned or disrupted drainages, faceted spurs, grabens, and/or displaced landforms such as terraces, shorelines, geologic units, etc.)."*

IGES is unaware of any paleoseismic studies that pertain to similar geologic conditions as found in this investigation, but rather the conclusion of fault inactivity is by way of taking all of the geologic data collectively through the application of the geological principles of cross-cutting relationships and uniformitarianism.

Regarding the additional recommendations from SA, IGES reviewed aerial photographs, conducting surface observations, and reviewing the USGS Quaternary Fault and Fold Database of the United States prior to the submittal of the September 23, 2015 letter; regrettably, this information was not incorporated into our response. Prior to undertaking the fieldwork for this investigation, IGES reviewed the Western GeoLogic report for the area (Western GeoLogic, 2012), in which aerial photographs were analyzed and no faults were identified. Additionally, the USGS Quaternary Fault and Fold Database of the United States was reviewed, with the closest fault to the area of investigation being approximately 2.5 miles to the southwest. IGES also analyzed current and historic Google Earth imagery for the area, and did not identify any surficial features relating to faulting in the area. Finally, surface observations were made during the field investigation, and no surficial expression of the faults were found except in the road cut north of the planned development.

Closure

We appreciate the opportunity to provide you with our services. If you have any questions please contact the undersigned at your convenience (801) 748-4044.

Respectfully Submitted,
IGES, Inc.

Reviewed by:

LICENSED PROFESSIONAL GEOLGIST
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Attachments:

References

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December 4, 2015

Summit Powder Mountain
c/o Ms. Andrea Milner
3632 North Wolf Creek Drive
Eden, Utah 84310

IGES Project No. 01628-008

Subject: Response to Review Comments – Geotechnical Engineering
Geotechnical Investigation
The Ridge Nests Development
Powder Mountain Resort
Weber and Cache Counties, Utah

Ms. Milner:

As requested, IGES has prepared the following response to a review comment regarding the referenced geotechnical report for the Ridge Nests development, part of the larger Powder Mountain Resort expansion project in Weber County, Utah. The review comments to be addressed were prepared by Taylor Geotechnical (TG) in notes uploaded on Miradi (Weber County on-line application) on October 15, 2015.

The review comments by TG was intended to address Lot 13; however, in consideration that the comments by TG could also be applicable to several other lots, it is the intention of IGES to address the comments with respect to the entire Ridge Nests development. For convenience, the review comments will be presented first, followed by our response.

Comment No. 1

"*Respond to geological comments in the Simon Associates, LLC (SA) "Geologic Review, Lot 13, The Ridge Crest Subdivision, 7914 East Heartwood Drive, Eden, Utah," (SA Project No 15-160), dated October 14, 2015.*"

Response to Comment No. 1

IGES has submitted a response to the referenced comments on November 4. IGES has subsequently received additional review comments by SA in a letter dated November 29, 2015; IGES is currently preparing a response to the new review comments, which will be responded to in a separate submittal.

Comment No. 2

"*Substantiate the 42 degrees friction angle for the shear strength of dolomite bedding planes, undifferentiated colluvium, and slope wash. The physical characteristics of bedding planes can affect bedding plane shear strength depending on degree of fracturing (generally controlled by the number of joints in a given direction), persistence of jointing, spacing of jointing, roughness of joint surface, open and/or closed joints, joint coatings and infillings, etc. Similarly, the shear* *strength of the undifferentiated colluvium and slope wash could vary depending on its gradation and clay content. "*

Response to Comment No. 2

IGES concurs with the Review's comment that the "*…physical characteristics of bedding planes can affect bedding plane shear strength depending on degree of fracturing (generally controlled by the number of joints in a given direction), persistence of jointing, spacing of jointing, roughness of joint surface, open and/or closed joints, joint coatings and infillings, etc."*. The following paragraphs are intended to provide a rational basis for selection of a friction angle of 42 degrees to model the strength along bedding/jointing planes in the dolomite.

The shear strength along a planar feature is often described using the familiar Coulomb's linear relation,

$$\tau = C + \sigma_n \tan\phi \qquad (1)$$

Where,

C = cohesion, or cohesion intercept

σ_n = normal stress on the sliding plane

ϕ = friction angle

τ = Peak Shear Strength

The friction angle for unconsolidated sediments can generally be determined by direct methods (e.g., direct shear test) or estimated based on index properties, insitu testing, or other suitable data. For rock joints, bedding, e.g. planes of weakness in rock, the Reviewer correctly points out that a representative friction angle will be dependent on a number of variables (e.g., physical rock properties) that should be assessed to determine a reasonable friction angle for analysis. An empirical relationship that is often utilized in rock mechanics to predict the mean peak strength along rock joints is taken as (after Barton and Choubey, 1977):

$$\tau = \sigma_n \tan[JRC\ log_{10}\left(\frac{JCS}{\sigma_n}\right) + \phi_b] \qquad (2)$$

Where,

τ = Peak shear strength

σ_n = effective normal stress

JRC = joint roughness coefficient

JCS = joint wall compressive strength

ϕ_b = basic friction angle (obtained from residual shear tests on flat, unweathered rock surfaces)

The Joint Roughness Coefficient (JRC) is a function of the texture along the rock joint or bedding. This value can be estimated a number of ways, including back-calculation, or estimating based on visual assessment using a typical roughness profile. Based on Figure 8 in Barton and Choubey (1977), a lower-bound estimate for JRC for the dolomite can reasonably be assessed as 5.

The Joint Wall Compressive Strength (JCS) is a function of the deformation properties of the rock. Where formation of the joint is within intact, unweathered rock, the JCS value is the same as the uniaxial compressive strength of the rock (σ_c). If the joint walls are weathered, the JCS may be a fraction of σ_c. The uniaxial compressive strength of the rock can be obtained using insitu means (Schmidt hammer) or conventional laboratory means. Empirical data is not available for this particular rock unit; however, the uniaxial strength of limestone (has very similar mechanical properties to dolomite) ranges from 5,120 psi to 54,100 psi (Johnson & Degraff, 1988). The exposed dolomite generally has a moderate degree of weathering, and the observed joints are close with openings generally less than 5mm, with little or no in-filling. Therefore, a conservative estimate of the JCS can be taken as 5,000 psi.

The basic friction angle (ϕ_b) for limestone ranges from 31-37 degrees (dry surface) to 27-35 degrees (wet surface) (Table 1 from Barton and Choubey, 1977). This friction angle is based on a diamond-saw cut smooth surface. For demonstrative purposes, we have estimated a representative basic friction angle of 31 degrees for this dolomite.

The effective normal stress varies along the failure plane; to estimate a reasonable, representative value for this demonstration, IGES has assessed the slope stability analysis for Section B-B'. By observation, the average depth from the ground surface to the shear surface is approximately 15 feet. The unit weight of dolomite is likely on the order of 180 pcf (Deer et al., 1966). Therefore, a representative effective normal stress can be estimated as 2,700 psf.

Inserting the foregoing estimated values into the referenced equation,

$$\phi_r = JRC\ log_{10}\left(\frac{JCS}{\sigma_n}\right) + \phi_b \qquad (3)$$

$$\phi_r = 5\ log_{10}\left(\frac{720\ ksi}{2700\ psf}\right) + 31$$

$$\phi_r = 43.1\ degrees$$

Where ϕ_r is the representative equivalent friction angle to be used in a conventional limit-equilibrium slope stability analysis to model the friction angle along rock bedding or jointing.

Based on this assessment, the estimate of 42 degrees is considered reasonable and conservative for use in a limit-equilibrium slope stability program. The estimated values of JRC and JCS are considered fairly conservative – it is likely that actual measured JCS and JRC values would be higher, but barring new empirical data IGES considers the lower-bound estimates based on published data reasonable, particularly in light of the fact that the static factor-of-safety determined in our slope stability analysis is greater than 5. It is interesting to note that Barton and Choubey (1977) recommends that the equivalent friction angle along rock joints should be limited to no more than 70 degrees in practice.

With respect to the strength of undifferentiated colluvium, the value of 42 degrees is considered a conservative and reasonable estimate based on soil types typically encountered in the vicinity, e.g., typically clast-supported gravel and cobbles, often with angular to sub-angular constituents. Quantifying the strength of particularly coarse, angular earth materials is often difficult or impractical. Nevertheless, in our slope stability analysis, the colluvium appears to have little or no bearing on the results of the slope stability analysis, as the strength and anisotropic properties (apparent dip of jointing/bedding) of the dolomite controls the analysis.

Although the strength of the colluvium has little or no bearing on this particular slope stability analysis, IGES concedes that assessing the strength of the colluvium may be critical for other nearby lots outside of the Ridge Nests project area. To that end, IGES has recently acquired a large-diameter shear box, which will allow testing of remolded soil samples with material up to 1 inch diameter. IGES anticipates testing representative samples of the prevailing coarse colluvium at selected locations in the spring, as the need arises. As this data is developed, at the Reviewer's request IGES will share this information with the Reviewer and discuss the implications for future slope stability analysis for upcoming Powder Mountain projects, or past projects if re-assessment is warranted based on this new data.

Closure

We appreciate the opportunity to provide you with our services. If you have any questions please contact the undersigned at your convenience (801) 748-4044.

Respectfully Submitted,
IGES, Inc.

PROFESSIONAL ENGINEER
No. 6370734
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Attachments:

References

References

Barton, N., and Choubey, V., 1977, The Shear Strength of Rock Joints in Theory and Practice, in Rock Mechanics, December 1977, Volume 10, Issue 1, pp 1-54.

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IGES, Inc., 2015c, Response to Additional Review Comments – Geology, Geotechnical Investigation, The Ridge Nests Development, Powder Mountain Resort, Weber and Cache Counties, Utah, Project No. 01628-008, dated September 23, 2015.

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December 11, 2015

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c/o Ms. Andrea Milner
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IGES Project No. 01628-008

Subject: Response to Additional Review Comments - Geology
Geotechnical Investigation
The Ridge Nests Development
Powder Mountain Resort
Weber and Cache Counties, Utah

Ms. Milner:

As requested, IGES has prepared the following response to additional review comments regarding the referenced geotechnical report and second review response dated November 4, 2015 for the Ridge Nests development, part of the larger Powder Mountain Resort expansion project in Weber County, Utah. The review comments to be addressed were prepared by Simon Associates LLC (SA) in a letter dated November 29, 2015; the latest comments by SA are in regard to the review response by IGES (2015d), which was prepared in response to SA's second geologic review letter (SA, 2015b) that was regarding the original review response by IGES (2015c).

The review letter by SA was intended to address Lot 13; however, in consideration that the comments by SA could also be applicable to several other lots, it is the intention of IGES to address the comments with respect to the entire Ridge Nests development. For convenience, the review comments will be presented first, followed by our response.

Comment No. 1

"*Item 1 of the October 14, 2015, SA review letter, recommended Weber County request documentation of the bedding, joint, and/or fracture properties, and incorporation of the geologic data in the slope stability analyses.*

*On page 2 (second paragraph) of the November 4, 2015, IGES response letter, IGES states: "…*These lithologies tended to fracture into rectangular blocks with highly variable dimensions, ranging in width and length from between a couple inches to several feet, though larger blocks (with dimensions of several feet *x several feet x several feet*) were most common (Photo 2). *(italics added for emphasis).*

It appears the preceding sentence from the November 4, 2015, IGES response letter is incomplete. SA recommends Weber County request IGES clarify the seeming discrepancy."

Response to Comment No. 1

There is no discrepancy in the statement. To clarify, the larger blocks had the dimensions of several feet in length by several feet in width by several feet in height, as shown in the referenced Photo 2 of our November 4, 2015 response letter, and it was these large-dimensioned blocks that were most commonly observed on the property.

Comment No. 2

"On page 5 (first bullet) of the November 4, 2015, IGES response letter, IGES provides a definition for inactive fault, referencing Chapter 38-3 of the Weber County Natural Hazards Overlay Districts. Chapter 38-3 of the Weber County Natural Hazards Overlay Districts is obsolete (see Weber County, 2015). SA is unaware of the Weber County Code of Ordinances providing a definition for "inactive fault." SA recommends Weber County request IGES provide definitions from current references."

Response to Comment No. 2

An active fault is defined in Section 104-27-3 *Supplementary Hazard Definitions* of Chapter 27 of the Weber County Natural Hazards Overlay Districts (Weber County, accessed 12-08-15) as "a fault displaying evidence of greater than four inches of displacement along one or more of its traces during Holocene time (about 11,000 years ago to the present)."

http://webercounty-ut.elaws.us/code/coor_ptii_title104_ch27

Regardless, it is deemed appropriate that a usable definition for an inactive fault, based upon the accepted definition for an active fault, is "a fault displaying evidence of equal to or less than four inches of displacement along one or more of its traces during Holocene time (approximately 11,000 years ago to the present)," or "… a fault in which the most recent displacement along one or more of its traces has occurred prior to Holocene time." This is consistent with other geologic hazard codes in common use, e.g. the Alquist-Priolo Act of 1972 and the Draper City Geologic Hazard Ordinance, among others. For this project, the definition presented in Section 104-27-3 is considered appropriate and reasonable.

Comment No. 3

"On page 5 of the October 14, 2015, SA review letter, SA stated:

"*However, regardless of the definitions, SA considers several of the factors not to be applicable in regards to timing of surface-fault-rupture, for instance: … 'The fault extends up to, but not through, the overlying soil profile.' Without the age of the overlying soil profile, the statement is unsubstantiated*."

On page 5 (second paragraph) of the November 4, 2015, IGES letter, IGES responded:

"Though the age of the soil profile overlying the faults is unknown, the presence of undisturbed soil provides a lower limit for most recent displacement along the fault traces. Soil formation can take hundreds to thousands of years to develop. Taking the conservative estimate of 100 years per inch of topsoil development (NRCS)2, and the fact that 3.5 feet of soil were encountered in TP-1, provides a lower limit of at least 3,600 years since last displacement along the faults."

The USDA National Resource Conservation Service (NRCS) referenced in the November 4, 2015, IGES letter states the following in regards to soil formation:

"*One of the first processes to occur during soil formation is the movement of organic matter into the surface of a soil giving it a characteristic dark color. An often asked question is, "How long does it take to form an inch of topsoil?" This question has many different answers but most soil scientists agree that it takes at least 100 years and it varies depending on climate, vegetation, and other factors."*

"In a wet, hot climate soil horizons will form fairly quickly compared to those in cold, dry environments. Therefore, soils in cold, dry climates develop rather slowly in comparison. It is not just the amount of time that determines the degree of soil development but also the parent material, climate, vegetation, and intensity of soil- forming factors during that time that ultimately determine soil development."

Consistent with long-established, geologic standards-of-practice (Birkeland, 1999; McCalpin, 2009), when using pedogenic development (i.e., "soil genesis") to estimate fault activity, it is appropriate to document soil-stratigraphic development by providing at least one, representative, standard soil-profile (at times supplemented by radiocarbon ages for the pedogenic horizons) (i.e., Birkeland, 1999).

Should IGES decide to pursue pedogenic development as an "individual piece of evidence that collectively indicates fault activity,*" SA recommends Weber County request IGES:*

a. Provide at least one, representative, standard soil-profile measurement and description, including the location of the profile on the site-specific geologic map.

b. Provide the climatic, vegetation, and other factors unique, to the subject site, supporting the applicability of the NRCS generality that it takes at least 100 years to form an inch of topsoil (which can vary depending on climate, vegetation, and other factors).

c. Clarify how the 3.5 feet of soil documented by IGES in TP-1 translates to 3,600 years.

d. Clarify how a lower limit of 3,600 years for the soil profile precludes Holocene displacement."

Response to Comment No. 3

IGES will not pursue pedogenic development as "an individual piece of evidence that collectively indicates fault activity," and retracts the statements concerning an estimate of the lower limit of fault displacement based upon pedogenic development.

Comment No. 4

"*On page 6 (first paragraph), of the October 14, 2015, SA review letter, SA states*:

"*Additionally, SA recommends Weber County suggest IGES consider the following, long established standard of practice, methods for evaluating the potential for surface-fault-rupture along the documented faults… Review of aerial photographs and surface observations to identify any fault-related geomorphic features indicative of past surface faulting at or near the property (e.g., fault scarps, vegetation lineaments, gullies, vegetation/soil contrasts, aligned springs and seeps, sag ponds, aligned or disrupted drainages, faceted spurs, grabens, and/or displaced landforms such as terraces, shorelines, geologic units, etc.).*"

On page 6, fourth paragraph, of the November 4, 2015, IGES letter, IGES responded:

"IGES is unaware of any paleoseismic studies that pertain to similar geologic conditions as found in this investigation, but rather the conclusion of fault inactivity is by way of taking all of the geologic data collectively through the application of the geological principles of cross-cutting relationships and uniformitarianism."

SA recommends Weber County request IGES:

a. *Clarify the relevance of the preceding response by IGES regarding SA's suggestion that IGES review of aerial photographs and surface observations to identify fault-related geomorphic features is indicative of past surface faulting at or near the property.*

b. *Provide a summary with site specific examples of IGES'* "…application of the geological principles of cross-cutting relationships and uniformitarianism.*"*"

Response to Comment No. 4a

The paragraph in question was mistakenly placed below the stated SA paragraph regarding review of aerial photographs and surface observations. The paragraph was supposed to be placed above the SA paragraph regarding review of aerial photographs and surface observations, and was to be the conclusion paragraph for the response to SA Comment 2, not the initial paragraph of the response to the "additional recommendations."

Response to Comment No. 4b

The principle of cross-cutting relationships is generally stated as "the geologic feature which cuts another geologic feature is the younger of the two features," and is used as a means of the relative dating of features in geology (Vreeken, 1984). In the specific case for the faults on the Ridge Nests property, the faults do not cut across the soil and have not produced any notable fault-related geomorphic features on the surface. Application of the principle of cross-cutting relationships displays that the fault and movement along the fault are older than the soil, vegetation, and the present geomorphic surfaces extant at the site (note: the fact that the faults do not displace the soil or vegetation is an observation intended to respond to the reviewer's question and is not intended herein to present evidence to preclude Holocene-age fault activity).

The principle of uniformitarianism is defined as "the fundamental principle that geological processes and natural laws now operating to modify the earth's crust have acted in much the same manner and with essentially the intensity throughout geologic time, and that past geologic events can be explained by forces observable today; the classical concept that 'the present is the key to the past.' The doctrine does not imply that all change is at a uniform rate, and does not exclude minor local catastrophes." (AGI, 1984).

Application of the principle of uniformitarianism to the Ridge Nests site shows that the slow rate of weathering seen in dolomite in modern environments (see Gauri et al., 1992) is likely to have been slow in the geologic past. Because an active fault would induce 4+ inches of displacement of the dolomite bedrock during Holocene time, and given the known weathering rate of dolomite, an active Holocene-aged fault would still show some surficial geomorphic expression of the fault scarp. Since there is no such fault scarp observed, it can be reasonably concluded that the faults are inactive based upon the definition of an inactive fault provided in the response to Comment 2.

Comment No. 5

"In regards to SA's recommendation that Weber County suggest IGES review aerial photographs to identify fault-related geomorphic features indicative of past surface faulting at or near the property, the November 4, 2015, IGES letter stated (page 6):

"Regarding the additional recommendations from SA, IGES reviewed aerial photographs, conducting surface observations, and reviewing the USGS Quaternary Fault and Fold Database of the United States prior to the submittal of the September 23, 2015 letter; regrettably, this information was not incorporated into our response. Prior to undertaking the fieldwork for this investigation, IGES reviewed the Western GeoLogic report for the area (Western GeoLogic, 2012), in which aerial photographs were analyzed and no faults were identified. Additionally, the USGS Quaternary Fault and Fold Database of the United States was reviewed, with the closest fault to the area of investigation being approximately 2.5 miles to the southwest. IGES also analyzed current and historic Google Earth imagery for the area, and did not identify any surficial features relating to faulting in the area. Finally, surface observations were made during the field investigation, and no surficial expression of the faults were found except in the road cut north of the planned development."

SA recommends Weber County request IGES:

a. *Clarify if IGES actually reviewed aerial photographs or is deferring to Western GeoLogic (2012) report.*

b. *Provide the source, date, flightline number, and scale of the stereoscopic aerial photographs reviewed, if any.*

c. *Provide site specific data to support* "…no surficial expression of the faults were found except in the road cut north of the planned development.*"*

Response to Comment No. 5

IGES reviewed three stereo pairs of aerial photographs that cover the Ridge Nests property and adjacent areas. The aerial photographs reviewed for this exercise are listed in Table 1. The aerial photographs were examined stereoscopically for the presence of photo-lineaments which might be indicative of faulting, as well as other additional geomorphic features. No photo-lineaments were observed either crossing or projecting toward the subject property. Additionally, no fault-related geomorphic features indicative of past surface faulting at or near the property, including fault scarps, vegetation lineaments, gullies, vegetation/soil contrasts, aligned springs or seeps, sag ponds, aligned or disrupted drainages, faceted spurs, grabens, or displaced landforms were observed in either the aerial photograph reviewed or the site reconnaissance (surface observations detailed in the IGES response letter dated September 1, 2015).

Table 1
Stereoscopic Aerial Photographs Reviewed

SOURCE*	DATE	FLIGHT	PHOTOGRAPHS	SCALE
1947 AAJ	August 10, 1946	AAJ_1B	88-90	1:20,000
1953 AAI	September 14, 1952	AAI_4K	34-36	1:20,000
1963 ELK	June 25, 1963	ELK_3	57-59	1:15,840

*https://geodata.geology.utah.gov/imagery/

IGES reviewed the USGS Quaternary Fold and Fault Database of the United States, and no faults have been mapped on the property. The closest fault to the area of investigation is located approximately 2.5 miles to the southwest.

The absence of lineaments and fault-related geomorphic evidence in the aerial photograph and surface observation investigations constitutes reasonable geologic evidence that the faults observed in the road cut are pre-Holocene age and are to be considered inactive. As a result, from the standpoint of surface-fault-rupture, the area investigated is suitable for the proposed development.

Closure

We appreciate the opportunity to provide you with our services. If you have any questions please contact the undersigned at your convenience (801) 748-4044.

Respectfully Submitted,
IGES, Inc.

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Attachments:

References

References

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IGES, Inc., 2015a, Response to Review Comments, Geotechnical Investigation, The Ridge Nests Development, Powder Mountain Resort, Weber and Cache Counties, Utah Project No. 01628-008, dated April 7, 2015.

IGES, Inc., 2015b, Addendum to Geotechnical Report, The Ridge Nests Development, Powder Mountain Resort, Weber and Cache Counties, Utah Project No. 01628-008, dated August 18, 2015.

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Simon Associates, LLC, 2015b, Geologic Review, Lot 13, Ridge Nests Subdivision, 7914 East Heartwood Drive, Eden, Utah, SA Project No. 15-160, dated October 14, 2015.

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