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GEOTECHNICAL AND GEOLOGIC HAZARD INVESTIGATION
Lot 71R of Summit Eden Phase 1C
8488 E. Spring Park Road
Summit Powder Mountain Resort
Weber County, Utah

IGES Project No. 02565-001

August 30, 2017

Prepared for:

Mr. Michael Eisenberg



IGES[®]

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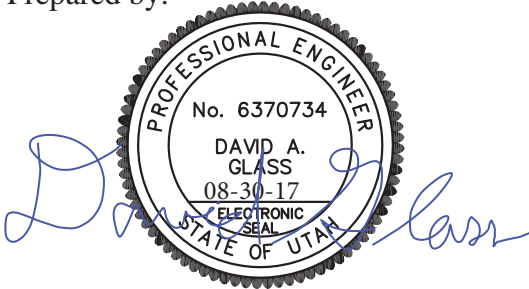
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1.0 INTRODUCTION

1.1 PURPOSE AND SCOPE OF WORK

This report presents the results of a geotechnical and geologic hazard investigation conducted for Lot 71R of Summit Eden Phase 1C, part of the currently on-going expansion at the Powder Mountain Ski Resort in Weber County. The purpose of our investigation was to assess the nature and engineering properties of the subsurface soils at the project site and to provide recommendations for the design and construction of foundations, grading, and drainage. In addition, geologic hazards have been assessed for the property. The scope of work completed for this study included literature review, site reconnaissance, subsurface exploration, engineering analyses, and preparation of this report.

Our services were performed in accordance with our proposal dated June 6, 2017, and your signed authorization. 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 Summit Powder Mountain Resort project, which included two geotechnical investigations for the greater 200-acre Powder Mountain Resort expansion project (IGES, 2012a and 2012b), as well as a number of lot-specific and site-specific geotechnical and geologic hazard investigations in various locations across the greater Powder Mountain Resort expansion area.

The Summit 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 Summit Powder Mountain project area is accessed by Powder Ridge Road. Lot 71R is located within Phase 1C of the Powder Mountain expansion project (Summit Eden), on the south side of Summit Pass – the street address is 8488 East Spring Park Road. The 0.062-acre residential lot has an approximate buildable area (building envelope) of 2,717 square feet. The proposed improvements will include a single-family home, presumably a high-end vacation home, with associated improvements such as utilities and hardscape. Construction plans were not available for our review; however, based on our experience in this area, the new home will likely be a two- to three-story structure, the lowest story consisting of a partial walk-out basement, founded on conventional spread footings. Foundation loads are expected to be on the order of 1,500 psf or less.

2.0 METHODS OF STUDY

2.1 LITERATURE REVIEW

2.1.1 Geotechnical

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.

2.1.2 Geological

Several pertinent publications were reviewed as part of this assessment. Sorensen and Crittenden, Jr. (1979) provides 1:24,000 scale geologic mapping of the Huntsville Quadrangle, and Crittenden, Jr. (1972) provides 1:24,000 scale geologic mapping of the Brown's Hole Quadrangle. Coogan and King (2001) provide more recent geologic mapping of the area, but at a 1:100,000 scale. An updated Coogan and King (2016) regional geologic map (1:62,500 scale) provides the most recent published geologic mapping that covers the project area. Western Geologic (2012) conducted a reconnaissance-level geologic hazard study for the greater 200-acre Powder Mountain expansion project, including the Lot 71R area. The Western Geologic (2012) study modified some of the potential landslide hazard boundaries that had previously been mapped at a regional scale (1:100,000) by Coogan and King (2001) and Elliott and Harty (2010). The corresponding United States Geological Survey (USGS) topographic maps for the Huntsville and Brown's Hole Quadrangles (2014) provide physiographic and hydrologic data for the project area. Regional-scale geologic hazard maps pertaining to landslides (Elliott and Harty, 2010; Colton, 1991), faults (Christenson and Shaw, 2008a; USGS and Utah Geological Survey (UGS), 2006), debris-flows (Christenson and Shaw, 2008b), and liquefaction (Christenson and Shaw, 2008c; Anderson et al., 1994) that cover the project area were also reviewed. The Quaternary Fault and Fold Database (USGS and UGS, 2006), was reviewed to identify the location of proximal faults that have had associated Quaternary-aged displacement.

Stereo-paired aerial imagery for the project site and recent and historic Google Earth imagery was also reviewed to assist in the identification of potential adverse geologic conditions. The aerial photographs reviewed are documented in the *References* section of this report.

2.2 FIELD INVESTIGATION

Subsurface soils were investigated by excavating one test pit at a central location within the property boundary. The approximate location of the test pit is illustrated on the *Geotechnical & Geology Map* (Figure A-2 in Appendix A). The soil types were visually logged at the time of our

field work in general accordance with the *Unified Soil Classification System* (USCS). Soil classifications and descriptions are included on the test pit log, Figure A-3 in Appendix A. A key to USCS symbols and terminology is included as Figure A-4, and a key to physical rock properties is included as Figure A-5.

2.3 LABORATORY TESTING

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

- Atterberg Limits (ASTM D4318)
- Grain-Size Distribution (ASTM D6913)
- In situ Moisture Content (ASTM D7263)
- Corrosion Suite (resistivity, pH, soluble sulfate, soluble chloride)

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

3.0 GEOLOGIC CONDITIONS

3.1 GENERAL GEOLOGIC SETTING

The Lot 71R property is situated in the western portion of the northern Wasatch Mountains, approximately 4 miles northeast of Ogden Valley. The Wasatch Mountains contain a broad depositional history of thick Precambrian and Paleozoic sediments that have been subsequently modified by various tectonic episodes that have included thrusting, folding, intrusion, and volcanics, as well as scouring by glacial and fluvial processes (Stokes, 1987). The uplift of the Wasatch Mountains occurred relatively recently during the Late Tertiary Period (Miocene Epoch) between 12 and 17 million years ago (Milligan, 2000). Since uplift, the Wasatch Front has seen substantial modification due to such occurrences as movement along the Wasatch Fault and associated spurs, the development of the numerous canyons that empty into the current Salt Lake Valley and Utah Valley and their associated alluvial fans, erosion and deposition from Lake Bonneville, and localized mass movement events (Hintze, 1988).

The Wasatch Mountains, as part of the Middle Rocky Mountains Province (Milligan, 2000), were uplifted as a fault block along the Wasatch Fault (Hintze, 1988). Ogden Valley itself is a fault-bounded trough that was occupied by Lake Bonneville (Sorensen and Crittenden, Jr, 1979) before being cut through by the Ogden River and subsequently dammed to form the Pineview Reservoir.

The Wasatch Fault and its associated segments are part of an approximately 230-mile long zone of active normal faulting referred to as the Wasatch Fault Zone (WFZ), which has well-documented evidence of late Pleistocene and Holocene (though not historic) movement (Lund, 1990; Hintze, 1988). The faults associated with the WFZ are all normal faults, exhibiting block movement down to the west of the fault and up to the east. The WFZ is contained within a greater area of active seismic activity known as the Intermountain Seismic Belt (ISB), which runs approximately north-south from northwestern Montana, along the Wasatch Front of Utah, through southern Nevada, and into northern Arizona. In terms of earthquake risk and potential associated damage, the ISB ranks only second in North America to the San Andreas Fault Zone in California (Stokes, 1987).

The WFZ consists of a series of ten segments of the Wasatch Fault that each display different characteristics and past movement, and are believed to have movement independent of one another (UGS, 1996). The Lot 71R property is located approximately 10.2 miles to the northeast of the Weber Segment of the Wasatch Fault, which is the closest documented Holocene-aged (active) fault to the property and trends north-south along the Wasatch Front (USGS and UGS, 2006).

3.2 SURFICIAL GEOLOGY

According to Sorensen and Crittenden, Jr. (1979), the property is entirely underlain by the undivided Tertiary/Cretaceous Wasatch and Evanston Formations (TKwe), described as

“unconsolidated pale-reddish-brown pebble, cobble, and boulder conglomerate, forms boulder-covered slopes. Clasts are mainly Precambrian quartzite and are tan, gray, or purple; matrix is mainly poorly consolidated sand and silt.” A generalized bedding attitude shows this unit striking due north and dipping 10 degrees to the east. This map forms the basemap for the *Regional Geology Map 1* (Figure A-6). Coogan and King (2001) produced a regional-scale geologic map that covered the property; this map shows the property to be near the contact between undifferentiated mass-movement deposits and the Wasatch Formation. Western Geologic (2012) identified a number of landslide deposits contained within the Powder Mountain Resort expansion area (*Regional Geology Map 2*, Figure A-7). In this map, the southern margin of the property abuts deposits mapped as “mixed slope colluvium, shallow landslides, and talus,” with a large Holocene to Late Pleistocene landslide deposit noted further south. Finally, Coogan and King (2016) updated their 2001 map, which shows the southern margin of the property to be at the contact between landslide deposits (unit Qms) and the Wasatch Formation (unit Tw) (*Regional Geology Map 3*, Figure A-8). A nearby bedding attitude shows the Wasatch Formation to be striking nearly due north and dipping at 5 degrees to the east.

3.3 HYDROLOGY

The USGS topographic maps for the Huntsville and Brown’s Hole Quadrangles (2014) show that the Lot 71R project area is situated on a slope, with the local topographic gradient down to the southeast towards a small unnamed southwest-trending dry drainage. To the southwest, this drainage eventually connects with a larger west-trending ephemeral drainage locally known as Lefty’s Canyon (see Figure A-1). No active or ephemeral stream drainages are found on or adjacent to the property, and no springs are known to occur on the property, though it is possible that springs may occur on various parts of the property during peak runoff.

Baseline groundwater depths for the Lot 71R property are currently unknown, but are anticipated to fluctuate both seasonally and annually. Groundwater was not encountered in the test pit excavated in this investigation.

3.4 GEOLOGIC HAZARDS FROM LITERATURE

Based upon the available geologic literature, regional-scale geologic hazard maps that cover the Lot 71R project area have been produced for landslide, fault, debris-flow, and liquefaction hazards. The following is a summary of the data presented in these regional geologic hazard maps.

3.4.1 Landslides

Two regional-scale landslide hazard maps have been produced that cover the project area. Colton (1991) does not show the property to be underlain by or adjacent to landslide deposits, though south-trending landslide deposits are noted nearby to west. Elliott and Harty (2010) shows deposits mapped as “Landslide undifferentiated from talus and/or colluvial deposits” near the southern margin of the property. More site-specific, Western Geologic (2012) used the Elliott and Harty

(2010) map as a base map, which reinterprets the southern half deposits to be definitive landslide deposits (see Figure A-7). As noted above, most recently Coogan and King (2016) on a regional scale show the southern margin of the property to be at the contact between the Wasatch Formation (to the north) and landslide deposits (to the south; see Figure A-8).

3.4.2 Faults

Neither Christensen and Shaw (2008a) nor the Quaternary Fault and Fold Database of the United States (USGS and UGS, 2006) show any Quaternary-aged (~2.6 million years ago to the present) faults to be present on or projecting towards the subject property. The Weber County Natural Hazards Overlay Districts defines an active fault to be “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)” (Weber County, 2015). The closest active fault to the property is the Weber Segment of the Wasatch Fault Zone, located approximately 10.2 miles southwest of the western margin of the property (USGS and UGS, 2006).

3.4.3 Debris Flows

Christensen and Shaw (2008b) do not show the project area to be located within a debris-flow hazard special study area.

3.4.4 Liquefaction

Anderson, et al. (1994) and Christensen and Shaw (2008c) both show the project area to be located in an area with very low potential for liquefaction.

3.5 REVIEW OF AERIAL IMAGERY

A series of aerial photographs that cover project area were taken from the UGS Aerial Imagery Collection and analyzed stereoscopically for the presence of adverse geologic conditions across the property. This included a review of photos collected from the years 1946, 1952, and 1963. A table displaying the details of the aerial photographs reviewed can be found in the *References* section at the end of this report.

No geologic lineaments, fault scarps, landslide headscarps, or landslide deposits were observed in the aerial photography on the subject property.

Google Earth imagery of the property from between the years of 1993 and 2014 were also reviewed. No landslide or other geological hazard features were noted in the imagery. The property was observed to be covered in bushes. Some surficial gravel, cobbles, and boulders, were observed, though the property does not contain any drainages. The only notable change to the property observed across this time frame was the installation of Spring Park Road, between September of 2011 and October of 2014. The southern approximately one-third to one-half of the lot was disturbed in the road-building process.

At the time of this report, no LiDAR data for the project area was available to be reviewed.

3.6 SEISMICITY

Following the criteria outlined in the 2015 International Building Code (IBC, 2015), 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, 2012/15); 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, 2015).

Table 3.6
Short- and Long-Period Spectral Accelerations for MCE

Parameter	Short Period (0.2 sec)	Long Period (1.0 sec)
MCE Spectral Response Acceleration (g)	$S_s = 0.811$	$S_1 = 0.269$
MCE Spectral Response Acceleration Site Class C (g)	$S_{MS} = S_s F_a = 0.872$	$S_{M1} = S_1 F_v = 0.412$
Design Spectral Response Acceleration (g)	$S_{DS} = S_{MS}^{2/3} = 0.582$	$S_{D1} = S_{M1}^{2/3} = 0.274$

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 C (*soft rock*). Based on IBC criteria, the short-period (F_a) coefficient is 1.076 and long-period (F_v) site coefficient is 1.531. 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 2.0; a summary of the *Design Maps* analysis is presented in Appendix B. The *peak ground acceleration* (PGA) may be taken as $0.4 \cdot S_{MS}$.

3.7 GEOLOGIC HAZARD ASSESSMENT

Geologic hazard assessments are necessary to determine the potential risk associated with particular geologic hazards that are capable of adversely affecting a proposed development area.

As such, they are essential in evaluating the suitability of an area for development and provide critical data in both the planning and design stages of a proposed development. The geologic hazard assessment discussion below is based upon a qualitative assessment of the risk associated with a particular geologic hazard, based upon the data reviewed and collected as part of this investigation.

A “low” hazard rating is an indication that the hazard is either absent, is present in such a remote possibility so as to pose limited or little risk, or is not anticipated to impact the project in an adverse way. Areas with a low-risk determination for a particular geologic hazard do not require additional site-specific studies or associated mitigation practices with regard to the geologic hazard in question. A “moderate” hazard rating is an indication that the hazard has the capability of adversely affecting the project at least in part, and that the conditions necessary for the geologic hazard are present in a significant, though not abundant, manner. Areas with a moderate-risk determination for a particular geologic hazard may require additional site-specific studies, depending on location and construction specifics, as well as associated mitigation practices in the areas that have been identified as the most prone to susceptibility to the particular geologic hazard. A “high” hazard rating is an indication that the hazard is very capable of or currently does adversely affect the project, that the geologic conditions pertaining to the particular hazard are present in abundance, and/or that there is geologic evidence of the hazard having occurred at the area in the historic or geologic past. Areas with a high-risk determination always require additional site-specific hazard investigations and associated mitigation practices where the location and construction specifics are directly impacted by the hazard. For areas with a high-risk geologic hazard, simple avoidance is often considered.

The following is a summary of the geologic hazard assessment for the Lot 71R property.

3.7.1 Landslides/Mass Movement

According to the several most recent geologic maps produced that cover the property, the southern margin of the lot abuts mapped landslide deposits (Coogan and King, 2016; Western Geologic, 2012; Elliott and Harty, 2010). However, landslide deposits or geomorphic features indicative of landsliding were not observed on the property in the aerial imagery, during the site reconnaissance, or in the subsurface. Though some evidence of slight to moderate soil creep was observed in the aspen trees found on the property, the subsurface data indicate that this is restricted to the topsoil. Given this information, the risk associated with landslides is considered low to moderate, given the proximity to mapped landslide deposits.

3.7.2 Rockfall

Though the property is on a slope, no bedrock outcrops are exposed upslope of the property. As such, the rockfall hazard associated with the property is considered to be low.

3.7.3 Surface-Fault Rupture and Earthquake-Related Hazards

No faults are known to be present on or project across the property, and the closest active fault to the property is the Weber Segment of the Wasatch Fault Zone, located approximately 10.2 miles to the west of the property (USGS and UGS, 2006). Given this information, the risk associated with surface-fault-rupture on the property is considered low.

The entire property is subject to earthquake-related ground shaking from a large earthquake generated along the active Wasatch Fault. Given the distance from the Wasatch Fault, the hazard associated with ground shaking is considered to be moderate. Proper building design according to appropriate building code and design parameters can assist in mitigating the hazard associated with earthquake ground shaking.

3.7.4 Liquefaction

The site is underlain by the Wasatch Formation, a poorly consolidated sedimentary rock unit (conglomerate). Rock units such as these are not considered susceptible to liquefaction; as such, the potential for liquefaction occurring at the site is considered low.

3.7.5 Debris-Flows and Flooding Hazards

The property does not contain and is not located adjacent to any active or ephemeral drainages. Additionally, there are no debris-flow source areas upslope of the property, and the property is on a consistent slope downhill to the southeast. Given these conditions, the debris-flow and flooding hazard associated with the property is considered to be low.

3.7.6 Shallow Groundwater

Groundwater was not encountered in the test pit excavated as part of this investigation. The test pit was excavated in mid-June, and the groundwater level was likely to be on its way down from its seasonal high. No springs were observed on the property, and no plants indicative of shallow groundwater conditions were observed on the property. However, shallow groundwater conditions have been observed at the nearby *Lot 75R* property (IGES, 2017).

Given the existing data, it is expected that groundwater levels will fluctuate both seasonally and annually, and the risk associated with shallow groundwater hazards is considered low to moderate. Spring thaw and runoff are likely to significantly contribute to elevated groundwater conditions (localized perched conditions). However, shallow groundwater issues can be mitigated through appropriate grading measures and/or the avoidance of the construction of basement levels, or constructing basements with foundation drains.

4.0 GENERALIZED SITE CONDITIONS

4.1 SITE RECONNAISSANCE

Mr. Peter E. Doumit, P.G., C.P.G., of IGES conducted reconnaissance of the site and the immediate adjacent properties on June 16, 2017. The site reconnaissance was conducted with the intent to assess the general geologic conditions present across the property, with specific interest in those areas identified in the geologic literature and aerial imagery reviews as potential geologic hazard areas. Additionally, the site reconnaissance provided the opportunity to geologically map the surficial geology of the area. Figure A-2 is a site-specific geologic map of the Lot 71R property and adjacent areas.

At the time of the site reconnaissance, the property was observed to have uneven surface topography that consistently sloped downhill to the southeast. A cluster of small aspen trees (many of which were dead) were observed near the northern margin of the property, though most of the property was covered in low-lying bushes and grasses. The aspens displayed evidence of slight to moderate soil creep in places, though this was later found to be restricted to the topsoil.

Pebbles, cobbles, and some boulders of dark yellowish orange to purple quartzite were observed on the surface. These rock clasts¹ were typically subrounded to rounded, and up to 2 feet in diameter, though generally 1 inch in diameter, and part of a surficial colluvial geologic unit derived from weathered Wasatch Formation.

No springs, seeps, or running water were observed on the property at the time of the site visit.

The lot was found to be elevated at least 6 feet above Spring Park Road, and aside from the road cut, appeared to be native ground. No adverse geologic conditions were observed on the property at this time.

4.2 SUBSURFACE CONDITIONS

On June 19, 2017, a single exploration test pit was excavated in the north-central portion of the lot (see Figure A-2). The test pit was excavated to a depth of 12 feet below existing grade with the aid of a Caterpillar 315C tracked excavator. Upon completion of logging, the test pit was backfilled without compactive effort. A detailed log of the test pit is displayed in Figure A-3. Three distinct geologic units were encountered in the subsurface. The soil and moisture conditions encountered during our investigation are discussed in the following paragraphs.

¹ Clast: An individual constituent, grain, or fragment of a sediment or rock, produced by the mechanical or chemical disintegration or a larger rock mass. (AGI, 2005)

4.2.1 Earth Materials

A/B Soil Horizon: This topsoil unit was found to be between approximately 1.5 to 2 feet thick. The unit was a grayish brown, medium stiff, moist, lean CLAY with gravel (CL), with gravel and larger-sized quartzite clasts comprising between approximately 5 and 10% of the unit. The topsoil was found to be forming upon the underlying colluvium unit.

Colluvium: This unit was approximately 3 to 3.5 feet thick. The unit consisted of a dark brown, loose to medium stiff, moist, gravelly lean CLAY (CL) that was gradational to a clayey GRAVEL (GC). Gravel and larger-sized subrounded to subangular quartzite clasts comprised between approximately 25 and 30% of the unit, though in places was as much as 50% of the unit. Individual clasts were as much as 3 feet in diameter, though the mode clast size was approximately 3 to 6 inches in diameter.

Wasatch Formation: This unit was at least 7 feet thick and extended to the maximum depth of exploration within the test pit. The unit consisted of weakly consolidated conglomerate bedrock that had been largely disaggregated into a moderate reddish brown, medium-dense to dense, moist mixture of clay, sand, and gravel that collectively classifies as clayey GRAVEL with sand (GC). Gravel and larger-sized subrounded quartzite clasts comprised between approximately 40 and 50% of the unit, with individual clasts up to 1 foot in diameter, with a mode clast size of 2 to 3 inches. Pinhole voids up to 1 mm in diameter were commonly observed within the unit.

4.2.2 Groundwater

Groundwater was not encountered in the test pit excavated for this project; however, it should be noted that shallow groundwater seepage has been encountered in the nearby Lot 75R property (IGES, 2017).

4.3 SLOPE STABILITY

4.3.1 Global Stability

The stability of the existing natural slope has been assessed in accordance with methodologies set forth in Blake et al. 2002 and AASHTO LRFD for Bridge Design Specifications with respect to a representative cross-section, illustrated on Figure D-1 in Appendix D (the section is identified in plan-view on Figure A-2). The stability of the slope was modeled using SLIDE, a computer application incorporating (among others) Spencer's Method of analysis. Calculations for stability were developed by searching for the minimum factor of safety for a translational-type failure occurring through surficial soils (colluvium and Qlos), just above the underlying conglomerate bedrock. A translational-type failure has been assumed, occurring within the surficial soils overlying bedrock. Analysis was performed for both static and seismic (pseudo-static) cases.

Groundwater, e.g. a piezometric groundwater surface, was not encountered during our subsurface investigation; however, seepage was noted in test pits on nearby properties (IGES, 2017). Accordingly, groundwater was not modeled in our limit-equilibrium analysis. Saturated parallel seepage has been modeled in a separate analysis (see Section 4.3.2).

Soil strength parameters were selected based on soil types observed, local experience, correlation with index properties (Atterberg Limits, clay content), site-specific strength testing (direct shear test), and comparisons with soil strength laboratory data from a nearby sites. Based on this assessment, the following soil strength parameters were selected for this analysis:

**Table 4.3.1a
Soil Strength Parameters**

Earth Materials	Friction angle (degrees)	Cohesion (psf)	Unit Weight (pcf)
Qac	36	0	120
Bedrock (Tw)	39	0	130
Qls	33	200	125
Engineered Fill (Af)	34	100	125

Pseudo-static (seismic screening) analysis of the proposed slope was performed in general conformance with Blake et al. 2002, ASCE 7-10 and AASHTO LRFD for Bridge Design Specifications. The design seismic event was taken as the ground motion with a 2 percent probability of exceedance in 50 years (2PE50). Based on information provided on the USGS website ground motion calculator, the Peak Ground Acceleration (PGA) associated with a 2PE50 event is estimated to be 0.35g. Half of the PGA, (0.17g), was taken as the horizontal seismic coefficient (k_h) (Hynes and Franklin, 1984), and used in the pseudo-static seismic screen analysis. The results of the analyses have been summarized in Table 4.3.1b.

**Table 4.3.1b
Results of Slope Stability Analyses**

Section	Static Factor of Safety	Pseudo-Static Factor of Safety
Existing Condition	2.6	1.6

The results of the analysis indicated the existing conditions meet the minimum required factors-of-safety of 1.5 and 1.0 for both the static and seismic (pseudo-static) case, respectively. The planned improvements will include a basement level, which would tend to unload the slope and further improve the stability of the slope; significant fill placement on the slope, which would tend to load the slope and decrease stability, is not anticipated. A summary of the slope stability analysis is presented in Appendix D.

4.3.2 Surficial Stability

Our subsurface investigation indicates that the near-surface soils generally consist of sandy clay with gravel (CL). Material identified as 'topsoil' (A/B Horizon) generally ranges in thickness from 1.5 to 2 feet; the topsoil has developed on the prevailing colluvial cover, and therefore consists largely of gravelly clay, but with a higher organic component (abundant roots).

IGES assessed the potential for the upper three feet to become mobilized under saturated parallel seepage conditions. Our assessment assumes three feet of clayey colluvium or topsoil, fully saturated, and a 3.5H:1V slope (this would be a transient condition that could occur during primary spring run-off and snowmelt). Our model assumes an estimated effective friction angle of 28 degrees and a cohesion of 100 psf, and a saturated unit weight of 136 pcf. Based on this model, a factor-of-safety of 1.93 results. Sample calculations are presented in Appendix D.

Our calculations do not take into account the beneficial effects of plant roots, which were commonly observed throughout the topsoil units. Many of the existing natural slopes are thickly vegetated, which is expected to reduce the likelihood of shallow surficial slope instability.

Based on our infinite slope model, and the foregoing discussion, IGES considers the potential for surficial slope instability on this site to be low.

5.0 CONCLUSIONS AND RECOMMENDATIONS

5.1 GENERAL CONCLUSIONS

Based on the results of the field observations, literature review, and slope stability analyses, **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 conclusions and 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 GEOLOGIC CONCLUSIONS AND RECOMMENDATIONS

Based upon the data collected and reviewed as part of the geologic hazard assessment, IGES makes the following conclusions regarding the geological hazards present at the Lot 71R project area:

- **The Lot 71R project area does not appear to have geological hazards that are capable of adversely impacting the development as currently proposed under the existing conditions.**
- Though recent geologic mapping shows the site to be located adjacent to landslide deposits, no evidence of landsliding was observed on the surface or subsurface of the property. Therefore, the risk of landslide hazards is considered to be low, but should not be considered zero given the proximity to mapped landslide deposits.
- Earthquake ground shaking may potentially affect all parts of the project area and is considered to pose a moderate risk.
- Shallow groundwater conditions were not observed in the test pit, though groundwater seepage has been observed in test pits on nearby properties; therefore, shallow groundwater hazards are considered to be low to moderate for the property.
- Rockfall, surface-fault-rupture, liquefaction, debris-flow, and flooding hazards are considered to be low for the property.

Given the conclusions listed above, IGES makes the following recommendations:

- Because landslide deposits are noted near the property, an IGES engineering geologist or geotechnical engineer should observe the foundation excavation to assess the absence (or presence) of landslide-induced shearing.
- Effort should be made to limit the introduction of water into the subsurface near the proposed residence. Appropriate grading and drainage away from the home and xeriscape or natural landscaping will assist in reducing the risk of landsliding.

5.3 EARTHWORK

5.3.1 General Site Preparation and Grading

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

*not required where bedrock is exposed in the foundation subgrade

5.3.2 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 hard bedrock is exposed.

5.3.3 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. For planning purposes, Soil Type C is expected to predominate at the site (sands

and gravels). Close coordination between the competent person and IGES should be maintained to facilitate construction while providing safe excavations.

Based on OSHA guidelines for excavation safety, trenches with vertical walls up to 5 feet in depth may be occupied. Where very moist soil conditions or groundwater is encountered, or when the trench is deeper than 5 feet, we recommend a trench-shield or shoring be used as a protective system to workers in the trench. As an alternative to shoring or shielding, trench walls may be laid back at one and one-half horizontal to one vertical (1½H:1V) (34 degrees) in accordance with OSHA Type C soils. Trench walls may need to be laid back at a steeper grade pending evaluation of soil conditions by the geotechnical engineer. Soil conditions should be evaluated in the field on a case-by-case basis. Large rocks exposed on excavation walls should be removed (scaled) to minimize rock fall hazards.

5.3.4 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 and/or special service districts having their own precedence for backfill and compaction should be followed where more stringent.

5.3.5 Oversize Material

Based on our observations, there is a significant potential for the presence of oversize materials (larger than 6 inches in greatest dimension). Large rocks, particularly boulders (>12 inches), may require special handling, such as segregation from structural fill, and disposal.

5.3.6 Utility Trench Backfill

Utility trenches should be backfilled with structural fill in accordance with Section 5.3.4 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. 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.

5.4 FOUNDATION RECOMMENDATIONS

Based on our field observations and considering the presence of relatively competent native earth materials, we recommend that the footings for proposed single-family home be founded either *entirely* on competent native soils or *entirely* on structural fill. Native/fill transition zones are not allowed. Where soft, loose, or otherwise deleterious earth materials are exposed on the foundation subgrade, IGES recommends a minimum over-excavation of 2 feet and replacement with structural fill. Alternatively, the foundations may be extended such that the foundations bear directly on competent earth materials (Wasatch Formation, e.g. conglomerate bedrock). It should be noted that Wasatch Formation was encountered at a depth of approximately 5 feet below existing grade, but may be deeper, or shallower, at specific locations. We recommend that IGES assess the bottom of the foundation excavation prior to the placement of steel or concrete, or structural fill, to identify the competent native earth materials as well as any unsuitable soils or transition zones. Additional over-excavation may be required based on the actual subsurface conditions observed.

Shallow spread or continuous wall footings constructed entirely on structural fill, or entirely on competent, uniform native earth materials (Wasatch Formation conglomerate) may be proportioned utilizing a maximum net allowable bearing pressure of **2,900 pounds per square foot (psf)** for dead load plus live load conditions. The net allowable bearing values presented above are for dead load plus live load conditions. The allowable bearing capacity may be increased by one-third for short-term loading (wind and seismic). The minimum recommended footing width is 20 inches for continuous wall footings and 30 inches for isolated spread footings.

All conventional foundations exposed to the full effects of frost should be established at a minimum depth of 42 inches below the lowest adjacent final grade. Interior footings, not subjected to the full effects of frost (i.e., *a continuously heated structure*), may be established at higher elevations, however, a minimum depth of embedment of 12 inches is recommended for confinement purposes.

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

5.5 SETTLEMENT

5.5.1 Static Settlement

Static settlements of properly designed and constructed conventional foundations, founded as described in Section 5.4, 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.

5.5.2 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 low; for design purposes, settlement on the order of ½ inch over 40 feet may be assumed.

5.6 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/gravelly native soils or structural fill should be used.

Ultimate lateral earth pressures from *granular* backfill acting against retaining walls, temporary shoring, or buried structures may be computed from the lateral pressure coefficients or equivalent fluid densities presented in Table 5.6. 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.

**Table 5.6
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	41.7	0.53	66.5
At-rest (Ko)	0.50	55	0.80	85
Passive (Kp)	3.0	375	—	—
Seismic Active	0.12	15.1	0.38	47.4
Seismic Passive	-0.33	-40.8	—	—
Seismic At-rest	0.18	22.5	0.57	71.7

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

5.7 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'' W2.9×W2.9 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 **250 psi/inch** may be used for design.

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

5.8 MOISTURE PROTECTION AND SURFACE DRAINAGE

Surface 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 structures should be implemented.

We recommend roof runoff devices be installed to direct all runoff a minimum of 10 feet away from foundations. The builder should be responsible for compacting the exterior backfill soils around the foundation; failure to properly compact the basement backfill can result in excessive settlement and damage to exterior improvements such as pavement or other flatwork. Additionally, the ground surface within 10 feet of the structures should be constructed so as to slope a minimum of **five** percent away from the structure. Pavement sections should be constructed to divert surface water off the pavement into storm drains, curb/gutter, or another suitable location.

Where basements are planned, IGES recommends a perimeter foundation drain be constructed in accordance with the International Residential Code (IRC).

5.9 SOIL CORROSION POTENTIAL

Laboratory testing of a representative soil sample obtained during our subsurface exploration indicated that the soil sample tested had a sulfate content of 16.5 ppm. Accordingly, the soils are classified as having a 'low potential' for deterioration of concrete due to the presence of soluble sulfate. As such, conventional Type II Portland cement may be used for all concrete in contact with site soils.

To evaluate the corrosion potential of ferrous metal in contact with onsite native soil a sample was tested for soil resistivity, soluble chloride and pH. The test indicated that the onsite soil tested has

a minimum soil resistivity of 15,283 OHM-cm, soluble chloride content of 5.3 ppm and a pH of 7.1. Based on this result, the onsite native soil is considered to be *mildly corrosive* to ferrous metal.

5.10 CONSTRUCTION CONSIDERATIONS

5.10.1 Over-Size Material

Large boulders (up to 48 inches in diameter) were observed on the surface and within the test pits; as such, excavation of the basement may generate an abundance of over-size material that may require special handling, processing, or disposal.

6.0 CLOSURE

6.1 LIMITATIONS

The concept of risk is a significant consideration of geotechnical analyses. The analytical means and methods used in performing geotechnical analyses and development of resulting recommendations do not constitute an exact science. Analytical tools used by geotechnical engineers are based on limited data, empirical correlations, engineering judgment and experience. As such the solutions and resulting recommendations presented in this report cannot be considered risk-free and constitute IGES's best professional opinions and recommendations based on the available data and other design information available at the time they were developed. IGES has developed the preceding analyses, recommendations and designs, at a minimum, in accordance with generally accepted professional geotechnical engineering practices and care being exercised in the project area at the time our services were performed. No warranties, guarantees or other representations are made.

The information contained in this report is based on limited field testing and our understanding of the project. The subsurface data used in the preparation of this report were obtained largely from the exploration made on Lot 71R. It is very likely that variations in the soil, rock, and groundwater conditions exist between and beyond the point explored. The nature and extent of the variations may not be evident until construction occurs and additional explorations are completed. If any conditions are encountered at this site that are different from those described in this report, IGES must be immediately notified so that we may make any necessary revisions to recommendations presented in this report. In addition, if the scope of the proposed construction or grading changes from those described in this report, our firm must also be notified.

This report was prepared for our client's exclusive use on the project identified in the foregoing. Use of the data, recommendations or design information contained herein for any other project or development of the site not as specifically described in this report is at the user's sole risk and without the approval of IGES, Inc. It is the client's responsibility to see that all parties to the project including the designer, contractor, subcontractors, etc. are made aware of this report in its entirety. The use of information contained in this report for bidding purposes should be done at the contractor's option and risk.

We recommend that IGES be retained to review the final design plans, grading plans and specifications to determine if our engineering recommendations have been properly incorporated in the project development documents. We also recommend that IGES be retained to evaluate construction performance and other geotechnical aspects of the project as construction initiates and progresses through its completion.

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.

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

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

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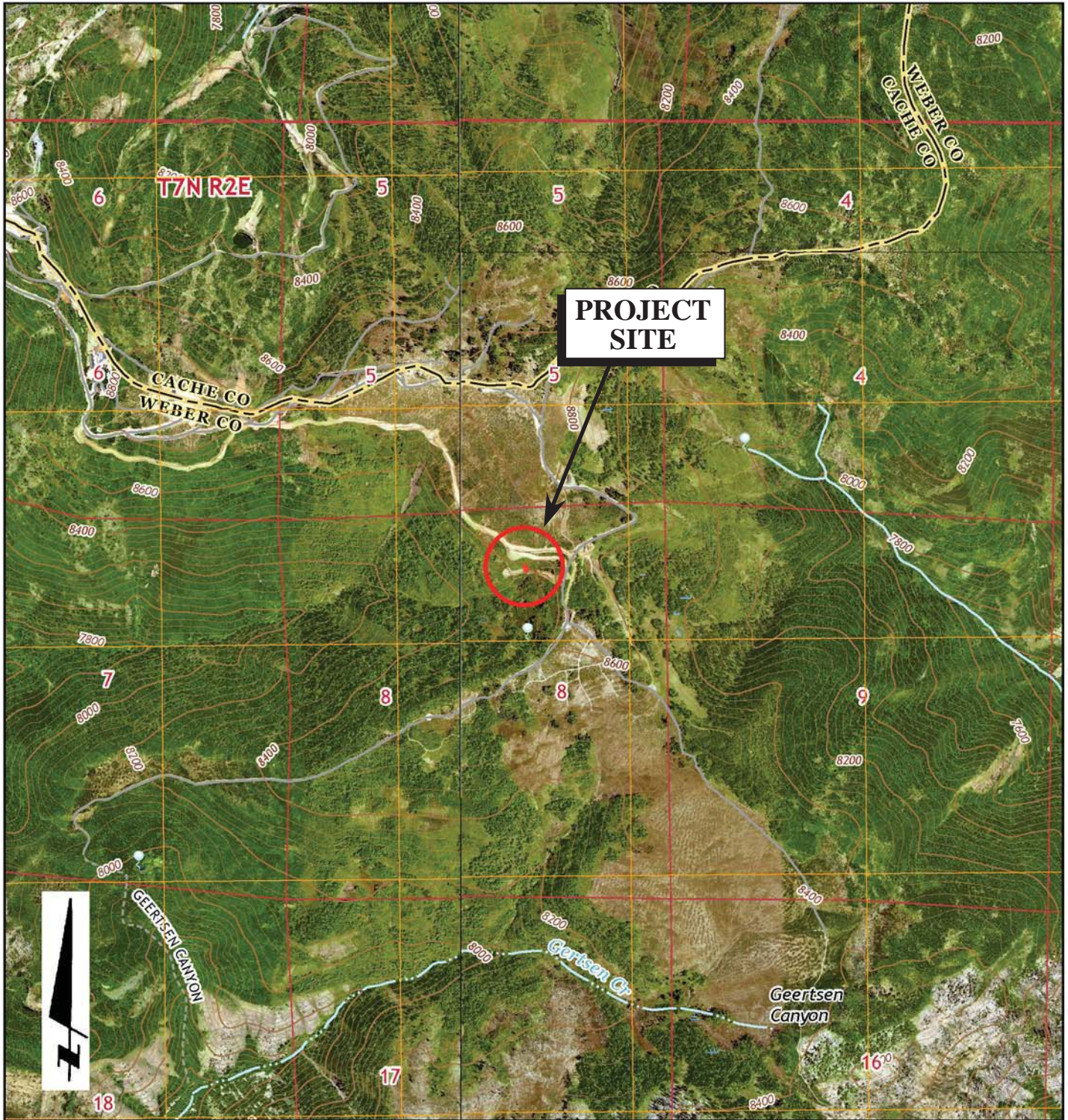
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AERIAL PHOTOGRAPHS

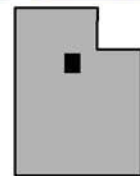
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1953 AAI	September 14, 1952	AAI_4K	34, 35, 36	1:20,000
1963 ELK	June 25, 1963	ELK_3	57, 58, 59	1:15,840

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

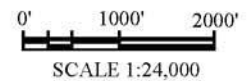
APPENDIX A



BASE MAP:
 USGS Huntsville, Browns Hole, James Peak and Sharp Mountain
 7.5-Minute Quadrangle Topographic Maps (2017)



MAP LOCATION

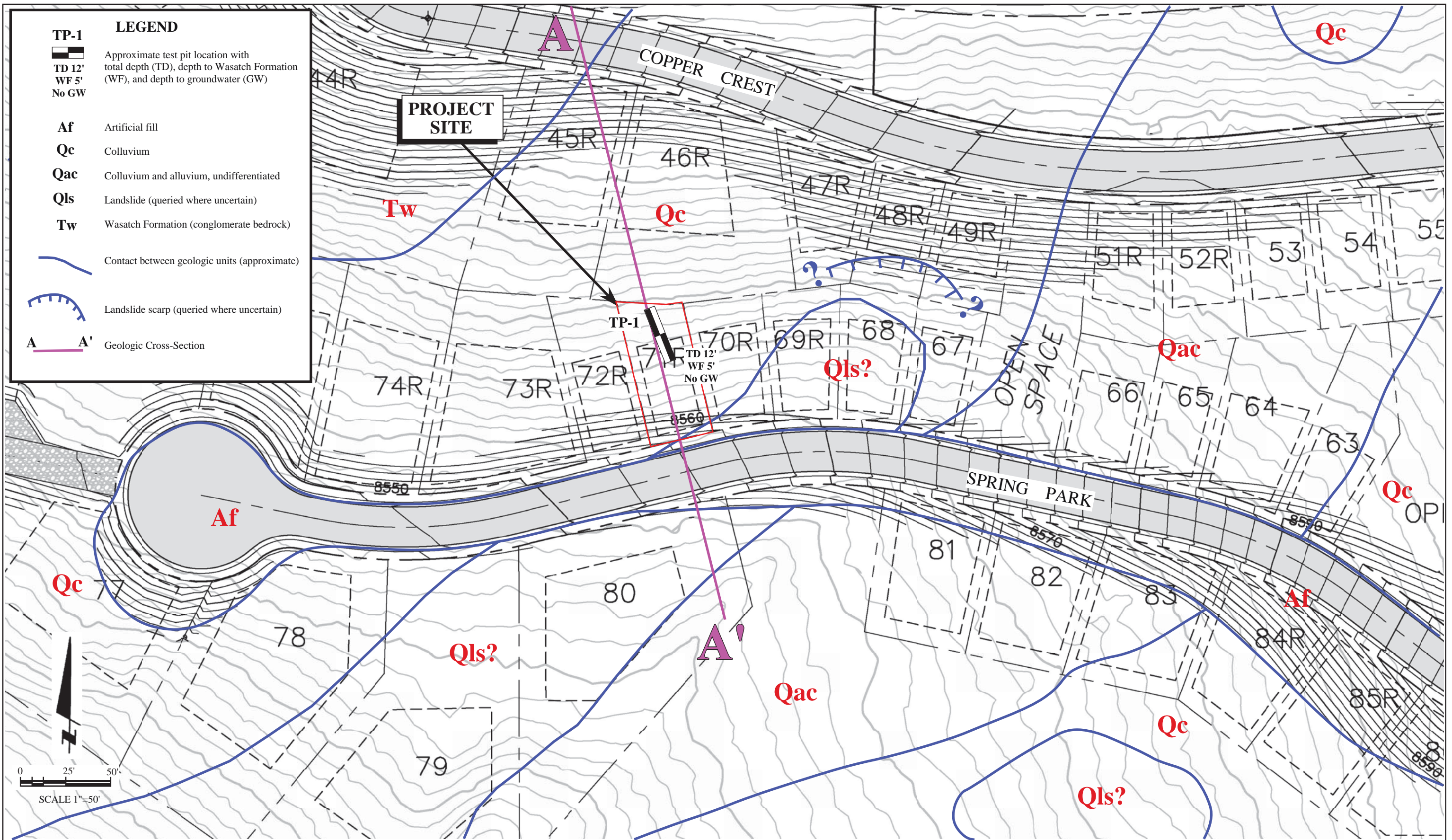


Project No. 02565-001

Geotechnical & Geologic Hazard Investigation
 Lot 71R of Summit Eden Phase 1C
 8488 E. Spring Park Road
 Weber County, Utah

SITE VICINITY MAP

Figure
A-1

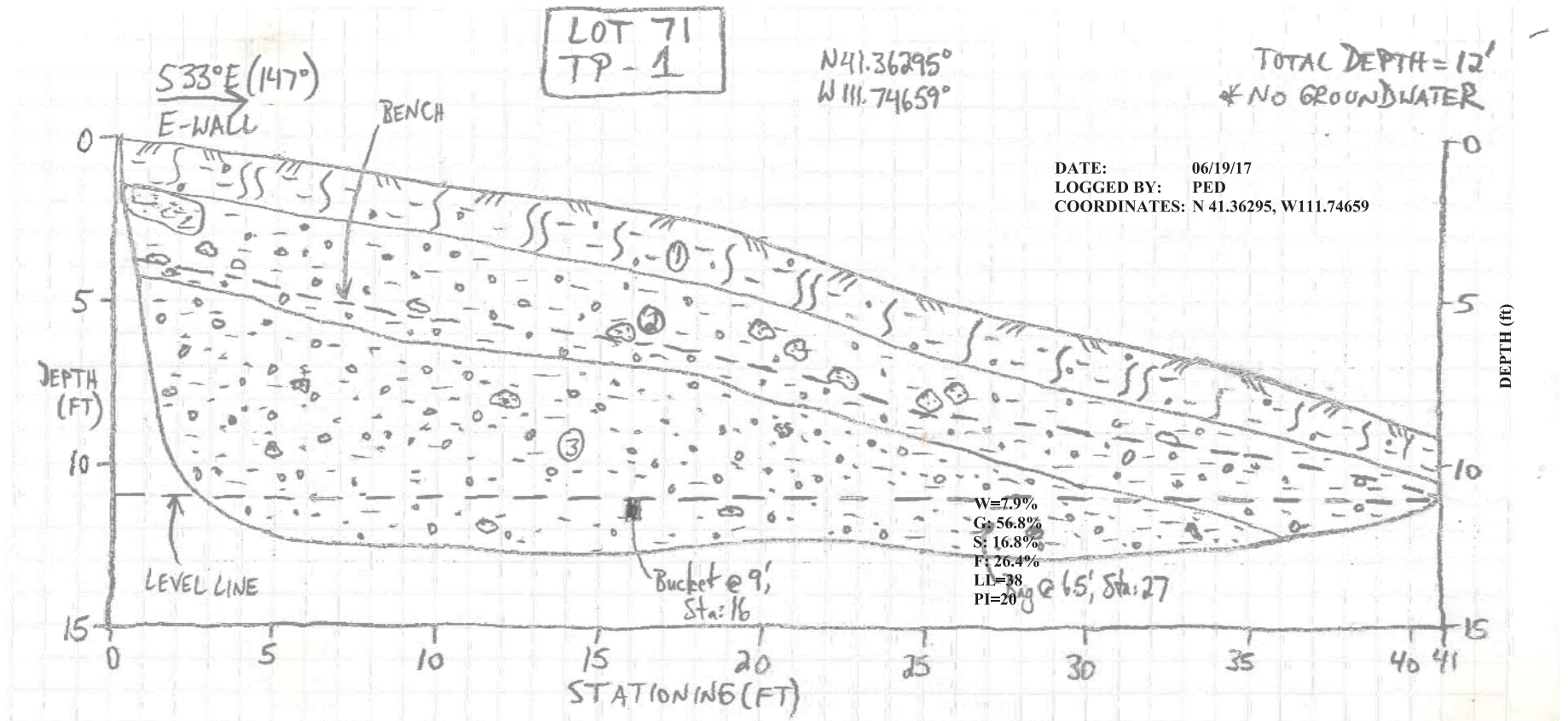


Basemap: Undated 50-scale map of Phase 1C prepared by NV5



Geotechnical & Geologic Hazard Investigation
 Lot 71R of Summit Eden Phase 1C
 Summit Powder Mountain Resort
 Weber County, Utah **GEOTECH & GEOLOGY MAP**

Figure
A-2



LITHOLOGIC UNIT DESCRIPTIONS:

- 1) **A/B Soil Horizon:** ~1.5-2' thick; grayish brown (5Y 3/2) lean CLAY with gravel (CL), medium stiff, moist, low plasticity, massive; gravel and larger sized clasts comprise ~5-10% of unit; clasts entirely subrounded pale yellowish orange (10YR 8/6) to medium gray (N5) quartzite up to 6" in diameter, though mode size ~1-2"; abundant plant and tree roots; gradational, irregular basal contact.
- 2) **Colluvium:** ~3-3.5' thick; dark yellowish brown (10YR 4/2) gravelly lean CLAY (CL), loose to medium stiff, moist, low plasticity, massive; gravel and larger sized clasts comprise ~25-30% of unit, though can be as much as 50% in places; clasts entirely subrounded to subangular quartzite as above up to 3' in diameter, though mode size ~3-6"; likely contains an alluvial component; common to abundant plant and tree roots; possibly reversely graded; gradational, irregular basal contact.

- 3) **Wasatch Formation:** At least ~7' thick; poorly consolidated conglomerate bedrock, largely disaggregated to moderate reddish brown (10R 4/6) clayey GRAVEL with sand (GC), medium dense to dense, moist, low to moderate plasticity fines, massive; gravel and larger sized clasts comprise ~40-50% of unit; clasts entirely subrounded quartzite as above, though some clasts exhibit hydrothermal alteration; clasts up to 1' in diameter, though mode size ~2-3"; clast distribution appears fairly homogeneous; matrix-supported; common 1 mm pinholes throughout; grades to sandy lean clay with gravel in south side of test pit; occasional plant and tree roots.

SCALE: 1"=5' H&V



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 Summit Powder Mountain Resort
 Weber County, Utah

TEST PIT LOG TP-1

Figure

A-3

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

LOG KEY SYMBOLS

	BORING SAMPLE LOCATION		TEST-PIT SAMPLE LOCATION
	WATER LEVEL (level after completion)		WATER LEVEL (level where first encountered)

CEMENTATION

DESCRIPTION	DESCRIPTION
WEAKLY	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

TEST	DESCRIPTION	TEST	DESCRIPTION
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 RELATIONS-IP	PM	PERMEABILITY
CI	CALIFORNIA IMPACT	-200	% FINER THAN #200
COL	COLLAPSE POTENTIAL	Gs	SPECIFIC GRAVITY
SS	SHRINK SWELL	SL	SWELL LOAD

MODIFIERS

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

GENERAL NOTES

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

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

APPARENT / RELATIVE DENSITY - COARSE-GRAINED SOIL

APPARENT DENSITY	SPT (blows/ft)	MODIFIED CA. SAMPLER (blows/ft)	CALIFORNIA SAMPLER (blows/ft)	RELATIVE DENSITY (%)	FIELD TEST
VERY LOOSE	<4	<4	<5	0 - 15	EASILY PENETRATED WITH 1/2-INCH REINFORCING ROD PUSHED BY HAND
LOOSE	4 - 10	5 - 12	5 - 15	15 - 35	DIFFICULT TO PENETRATE WITH 1/2-INCH REINFORCING ROD PUSHED BY HAND
MEDIUM DENSE	10 - 30	12 - 35	15 - 40	35 - 65	EASILY PENETRATED A FOOT WITH 1/2-INCH REINFORCING ROD DRIVEN WITH 5-LB HAMMER
DENSE	30 - 50	35 - 60	40 - 70	65 - 85	DIFFICULT TO PENETRATE 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		FIELD TEST
		UNTRAINED SHEAR STRENGTH (tsf)	POCKET PENETROMETER UNCONFINED COMPRESSIVE STRENGTH (tsf)	
VERY SOFT	<2	<0.125	<0.25	EASILY PENETRATED SEVERAL INCHES BY THUMB. EXUDES BETWEEN THUMB AND FINGERS WHEN SQUEEZED BY HAND.
SOFT	2 - 4	0.125 - 0.25	0.25 - 0.5	EASILY PENETRATED ONE INCH BY THUMB. MOLDED BY LIGHT FINGER PRESSURE.
MEDIUM STIFF	4 - 8	0.25 - 0.5	0.5 - 1.0	PENETRATED OVER 1/2 INCH BY THUMB WITH MODERATE EFFORT. MOLDED BY STRONG FINGER PRESSURE.
STIFF	8 - 15	0.5 - 1.0	1.0 - 2.0	INDENTED ABOUT 1/2 INCH BY THUMB BUT PENETRATED ONLY WITH GREAT EFFORT.
VERY STIFF	15 - 30	1.0 - 2.0	2.0 - 4.0	READILY INDENTED BY THUMB/NAIL.
HARD	>30	>2.0	>4.0	INDENTED WITH DIFFICULTY BY THUMB/NAIL.

KEY TO SOIL SYMBOLS AND TERMINOLOGY

Project No. 02565-001
 Engr. DAG
 Drafted By DAG
 Date August 2017



Figure A-4

Weathering

Rock Classification Should Include:	
1.	Rock name (or classification)
2.	Color
3.	Weathering
4.	Fracturing
5.	Competency
6.	Additional comments indicating rock characteristics which might affect engineering properties

Weathering	Field Test
Fresh	No visible sign of decomposition or discoloration. Rings under hammer impact.
Slightly Weathered	Slight discoloration inwards from open fractures, otherwise similar to Fresh.
Moderately Weathered	Discoloration throughout. Weaker minerals such as feldspar are decomposed. Strength somewhat less than fresh rock but cores cannot be broken by hand or scraped with a knife. Texture preserved.
Highly Weathered	Most minerals somewhat decomposed. Specimens can be broken by hand with effort or shaved with a knife. Core stones present in rock mass. Texture becoming indistinct but fabric preserved.
Completely Weathered	Minerals decomposed to soil but fabric and structure preserved. Specimens easily crumble or penetrated.

Fracturing

Spacing	Description
>6 ft	Very Widely
2-6 ft	Widely
8-24 in	Moderately
2 ½-8 in	Closely
¾-2 ½ in	Very Closely

Bedding of Sedimentary Rocks

Splitting Property	Thickness	Stratification
Massive	>4.0 ft	Very thick bedded
Blocky	2.0-4.0 ft	Thick-bedded
Slabby	2 ½-24 in	Thin-bedded
Flaggy	½-2 ½ in	Very thin-bedded
Shaly or platy	¼ – ½ in	Laminated
Papery	< ¼ in	Thinly laminated

RQD

RQD (%)	Rock Quality
90-100	Excellent
75-90	Good
50-75	Fair
25-50	Poor
0-25	Very Poor

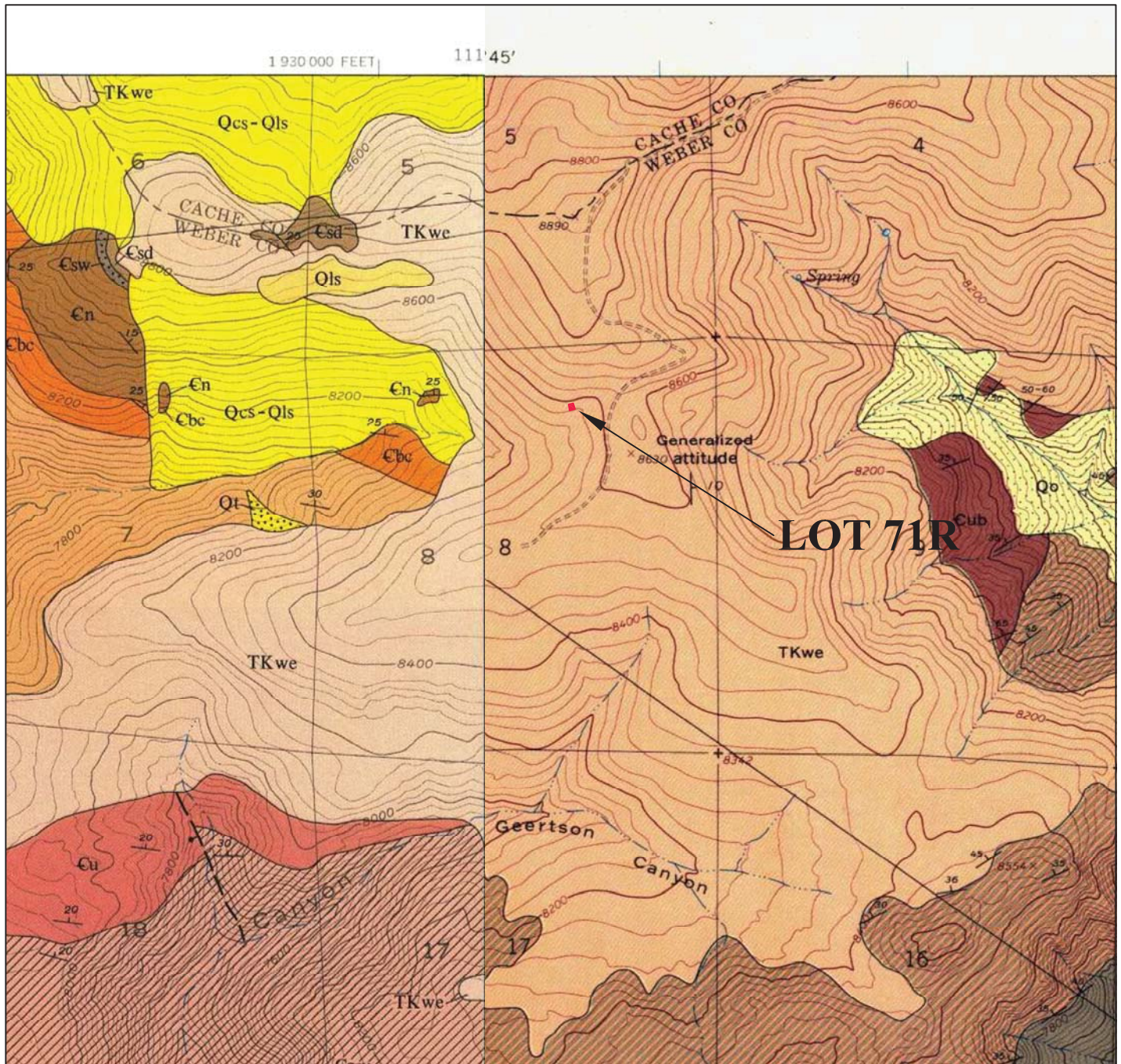
Competency

Class	Strength	Field Test	Approximate Range of Unconfined Compressive Strength (tsf)
I	Extremely Strong	Many blows with geologic hammer required to break intact specimen.	>2000
II	Very Strong	Hand-held specimen breaks with pick end of hammer under more than one blow.	2000-1000
III	Strong	Cannot be scraped or peeled with knife, hand-held specimen can be broken with single moderate blow with pick end of hammer	1000-500
IV	Moderately Strong	Can just be scraped or peeled with knife. Indentations 1-3 mm show in specimen with moderate blow with pick end of hammer.	500-250
V	Weak	Material crumbles under moderate blow with pick end of hammer and can be peeled with a knife, but is hard to hand-trim for triaxial test specimen.	250-10
VI	Friable	Material crumbles in hand.	N/A

KEY TO PHYSICAL ROCK PROPERTIES

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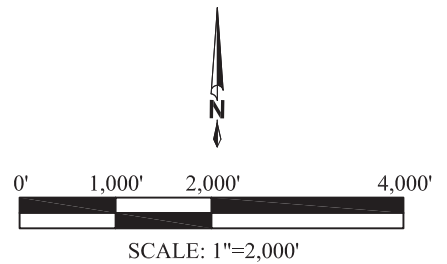




BASE MAPS

-USGS Huntsville 7.5-Minute Geologic Quadrangle Map (GQ-1503), Sorensen and Crittenden, Jr. (1979)

-USGS Brown's Hole 7.5-Minute Geologic Quadrangle Map (GQ-968), Crittenden, Jr. (1972)



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 Weber County, Utah REGIONAL GEOLOGY MAP 1

Figure

A-6a

MAP LEGEND

Qal	ALLUVIAL DEPOSITS, UNDIFFERENTIATED (Holocene) – Unconsolidated gravel, sand, and silt deposits in presently active stream channels and floodplains; thickness 0-6 m
Qcs	COLLUVIUM AND SLOPEWASH (Holocene) – Bouldery colluvium and slopewash chiefly along eastern margin of Ogden Valley; in part, lag from Tertiary units; thickness 0-30 m
Qt	ALLUVIAL FAN DEPOSITS (Holocene) – Alluvial fan deposits; postdate, at least in part, time of highest stand of former Lake Bonneville; thickness 0-30 m
Qls	LANDSLIDE DEPOSITS (Holocene) – thickness 0-6 m
Qt	TALUS DEPOSITS (Holocene) – thickness 0-6 m
TKwe	WASATCH AND EVANSTON(?) FORMATIONS, UNDIVIDED (Eocene, Paleocene, and Upper Cretaceous?) – Unconsolidated pale-reddish-brown pebble, cobble, and boulder conglomerate; forms boulder-covered slopes. Clasts are mainly Precambrian quartzite and are tan, gray, or purple; matrix is mainly poorly consolidated sand and silt; thickness 0-150 m
€sd	ST. CHARLES LIMESTONE (Upper Cambrian) – Includes: Dolomite member – Thin- to thick-bedded, finely to medium crystalline, light- to medium-gray, white- to light-gray-weathering, cliff-forming dolomite; linguloid brachiopods common in basal 15 m; thickness 150-245 m
€sw	Worm Creek Quartzite Member – Thin-bedded, fine- to medium-grained, medium- to dark-gray, tan- to brown-weathering calcareous quartzitic sandstone; detrital grains well-sorted and well-rounded; thickness 6 m
€n	NOUNAN DOLOMITE (Upper and Middle Cambrian) – Thin- to thick-bedded, finely crystalline, medium-gray, light- to medium-gray-weathering, cliff-forming dolomite; white twiggy structures common throughout unit; thickness 150-230 m
€bc	CALLS FORT SHALE MEMBER OF BLOOMINGTON FORMATION (Middle Cambrian) – Olive-drab to light-brown shale and light- to dark-blue-gray limestone with intercalated orange to rusty-brown silty limestone; intraformational conglomerate common throughout unit; thickness 23-90 m
€lu	CAMBRIAN LIMESTONES, UNDIVIDED (Middle Cambrian) – Includes limestone and Hodges Shale Members of Bloomington Formation, and Blacksmith and Ute Limestones
€b	BLACKSMITH LIMESTONE (Middle Cambrian) – Medium- to thin-bedded, light-gray to dark-blue-gray limestone; thin-bedded, flaggy-weathering, gray to tan silty limestone and interbedded siltstone; light- to dark-gray dolomite, with some reddish siliceous partings; thickness 400? m



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Geotechnical & Geologic Hazard Investigation
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 Summit Powder Mountain Resort
 Weber County, Utah REGIONAL GEOLOGY MAP 1

Figure

A-6b

MAP LEGEND

Eu




UTE LIMESTONE (Middle Cambrian) – Medium- to thin-bedded, finely crystalline, light- to dark-gray silty limestone with irregular wavy partings, mottled and streaked surfaces, worm tracks, and twiggy structures common throughout unit; oolites and *Girvanella* in many beds; olive-drab fissile shale interbedded throughout unit. Includes thin-bedded, gray-weathering, pale-tan to brown dolomite exposed at base of unit, 18-24 m at head of Geertsen Canyon and 0-3 m elsewhere; thickness 245? m

€gcu

GEERTSEN CANYON QUARTZITE (Lower Cambrian) – Includes:
Upper member – Pale-buff to white or flesh-pink quartzite, locally streaked with pale red or purple. Coarse-grained; small pebbles occur throughout unit and increase in abundance downward. Base marked by zone 30-60 m thick of cobble conglomerate in beds 30 cm to 2 m thick; clasts, 5-10 cm in diameter, are mainly reddish vein quartz or quartzite, sparse gray quartzite, or red jasper; thickness 730-820 m

€gcl

Lower member – Pale-buff to white and tan quartzite with irregular streaks and lenses of cobble conglomerate decreasing in abundance downward. Lower 90-120 m strongly arkosic, streaked greenish or pinkish. Feldspar clasts increase in size to 0.6-1.3 cm in lower part of unit; thickness 490-520 m

-  Recently active normal fault – Dashed where inferred. Ticks on downthrown side
-  Pre-Tertiary normal fault – Dotted where concealed
Bar and ball on downthrown side
-  Thrust fault – Dashed where inferred
Sawteeth on upper plate

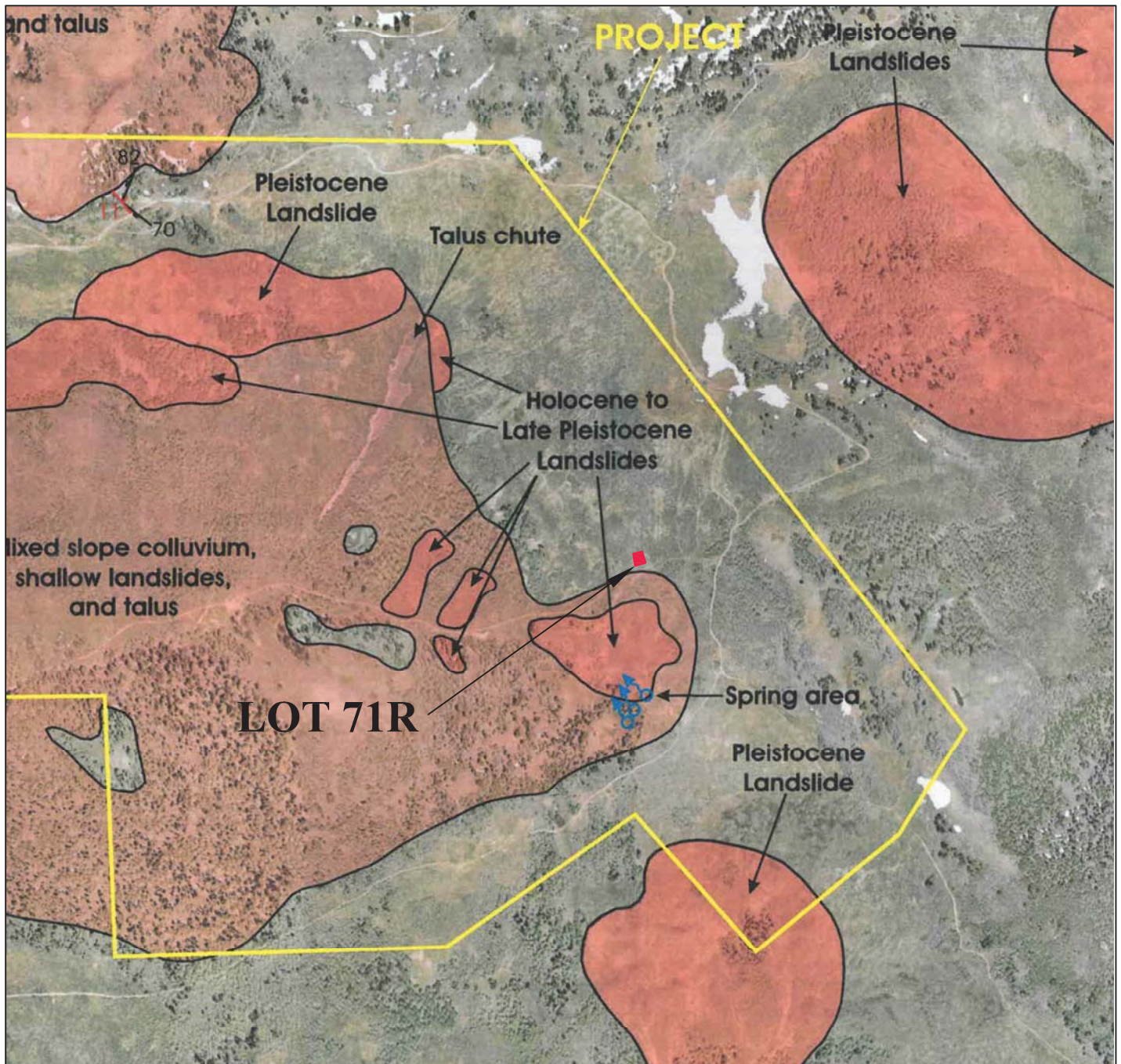


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Weber County, Utah REGIONAL GEOLOGY MAP 1

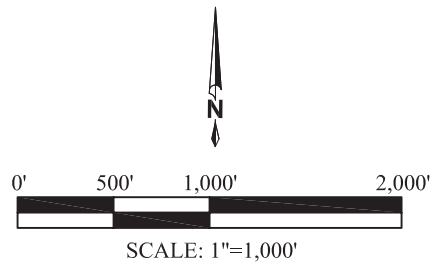
Figure

A-6c



BASE MAP

-Western Geologic (2012)
 Geologic Hazards
 Reconnaissance Report, Figure 3

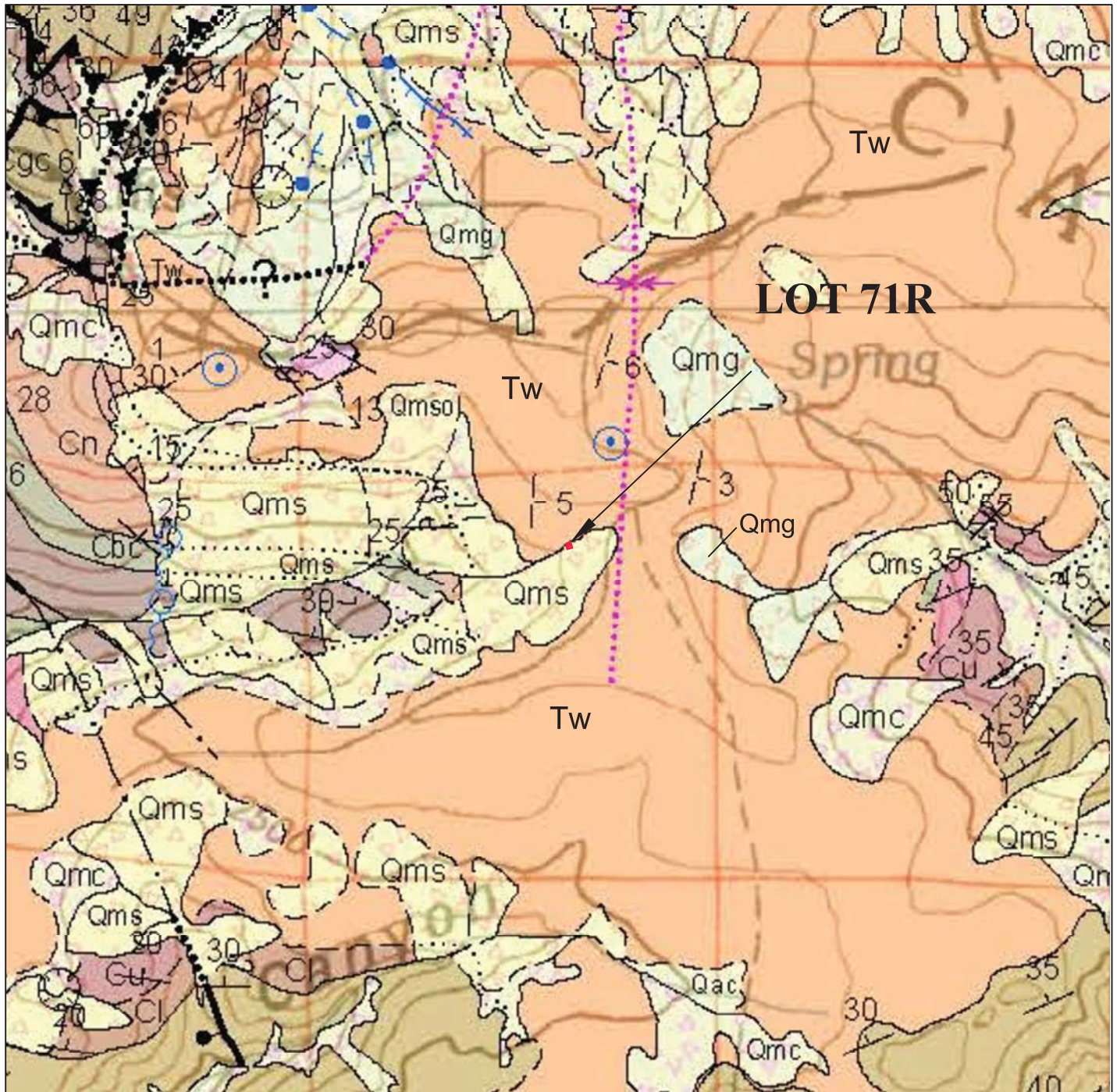


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 Summit Powder Mountain Resort
 Weber County, Utah REGIONAL GEOLOGY MAP 2

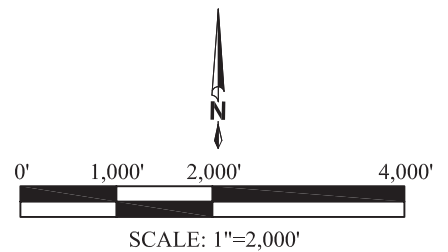
Figure

A-7



BASE MAP

-Coogan and King (2016)
 UGS Ogden 30'x60' Geologic
 Quadrangle Map, OFR-635DM
 Plate 1



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Geotechnical & Geologic Hazard Investigation
 Lot 71R of Summit Eden Phase 1C
 Summit Powder Mountain Resort
 Weber County, Utah REGIONAL GEOLOGY MAP 3

Figure

A-8a

MAP LEGEND

Qmc **Landslide and colluvial deposits, undivided (Holocene and Pleistocene)** – Poorly sorted to unsorted clay- to boulder-sized material; mapped where landslide deposits are difficult to distinguish from colluvium (slopewash and soil creep) and where mapping separate, small, intermingled areas of landslide and colluvial deposits is not possible at map scale; locally includes talus and debris flow and flood deposits; typically mapped where landslides are thin (“shallow”); also mapped where the blocky or rumpled morphology that is characteristic of landslides has been diminished (“smoothed”) by slopewash and soil creep; composition depends on local sources; 6 to 40 feet (2-12 m) thick. These deposits are as unstable as other landslide units (Qms, Qmsy, Qmso).

Human disturbances

Qh, Qh? **Human disturbances (Historical)** - Mapped disturbances obscure original deposits or rocks by cover or removal; only larger disturbances that pre-date the 1984 aerial photographs used to map the Ogden 30 x 60-minute quadrangle are shown; includes engineered fill, particularly along Interstate Highways 80 and 84, the Union Pacific Railroad, and larger dams, as well as aggregate operations, gravel pits, sewage-treatment facilities, cement plant quarries and operations, brick plant and clay pit, Defense Depot Ogden (Browning U.S. Army Reserve Center), gas and oil field operations (for example drill pads) including gas plants, and low dams along several creeks, including a breached dam on Yellow Creek.

Qms, Qms?, Qmsy, Qmsy?, Qmso, Qmso?

Landslide deposits (Holocene and upper and middle? Pleistocene) – Poorly sorted clay- to boulder-sized material; includes slides, slumps, and locally flows and floods; generally characterized by hummocky topography, main and internal scarps, and chaotic bedding in displaced blocks; composition depends on local sources; morphology becomes more subdued with time and amount of water in material during emplacement; Qms may be in contact with Qms when landslides are different/distinct; thickness highly variable, up to about 20 to 30 feet (6-9 m) for small slides, and 80 to 100 feet (25-30 m) thick for larger landslides. Qmsy and Qmso queried where relative age uncertain; Qms queried where classification uncertain. Numerous landslides are too small to show at map scale and more detailed maps shown in the index to geologic mapping should be examined.

Qmg, Qmg?

Mass-movement and glacial deposits, undivided (Holocene and Pleistocene) – Unsorted and unstratified clay, silt, sand, and gravel; mapped where glacial deposits lack typical moraine morphology, and appear to have failed or moved down slope; also mapped in upper Strawberry Bowl (Snow Basin quadrangle) where glacial deposits have lost their distinct morphology and the contacts between them and colluvium and talus in the cirques cannot be mapped; likely less than 30 feet (9 m) thick, but may be thicker in Mantua, James Peak, North Ogden, Huntsville, and Peterson quadrangles.

Tw, Tw?

Wasatch Formation (Eocene and upper Paleocene) – Typically red to brownish-red sandstone, siltstone, mudstone, and conglomerate with minor gray limestone and marlstone locally (see Tw1); lighter shades of red, yellow, tan, and light gray present locally and more common in uppermost part, complicating mapping of contacts with overlying similarly colored Norwood and Fowkes Formations; clasts typically rounded Neoproterozoic and Paleozoic sedimentary rocks, mainly Neoproterozoic and Cambrian quartzite; basal conglomerate more gray and less likely to be red, and containing more locally derived angular clasts of limestone, dolomite and sandstone, typically from Paleozoic strata, for example in northern Causey Dam



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Geotechnical & Geologic Hazard Investigation
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Summit Powder Mountain Resort
Weber County, Utah REGIONAL GEOLOGY MAP 3

Figure

A-8b

APPENDIX B

Water Content and Unit Weight of Soil

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

Project: Eisenburg/Lot 71

No: 02565-001

Location: **Summit Powder Mtn.**

Date: **7/27/2017**

By: **BSS**

Sample Info.	Boring No.	TP-1						
	Sample							
	Depth	6.5'						
	Split	Yes						
	Split sieve	3/8"						
Total sample (g)		4602.00						
Moist coarse fraction (g)		2341.80						
Moist split fraction (g)		2260.20						
	Sample height, H (in)							
	Sample diameter, D (in)							
	Mass rings + wet soil (g)							
	Mass rings/tare (g)							
	Moist unit wt., γ_m (pcf)							
Coarse Fraction	Wet soil + tare (g)	2652.02						
	Dry soil + tare (g)	2628.15						
	Tare (g)	310.51						
	Water content (%)	1.0						
Split Fraction	Wet soil + tare (g)	840.29						
	Dry soil + tare (g)	769.74						
	Tare (g)	327.86						
	Water content (%)	16.0						
Water Content, w (%)		7.9						
Dry Unit Wt., γ_d (pcf)								

Entered by: _____

Reviewed: _____

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

(ASTM D6913)

Project: Eisenberg/Lot 71

No: 02565-001

Location: Summit Powder Mtn.

Date: 8/1/2017

By: BSS

Boring No.: TP-1

Sample:

Depth: 6.5'

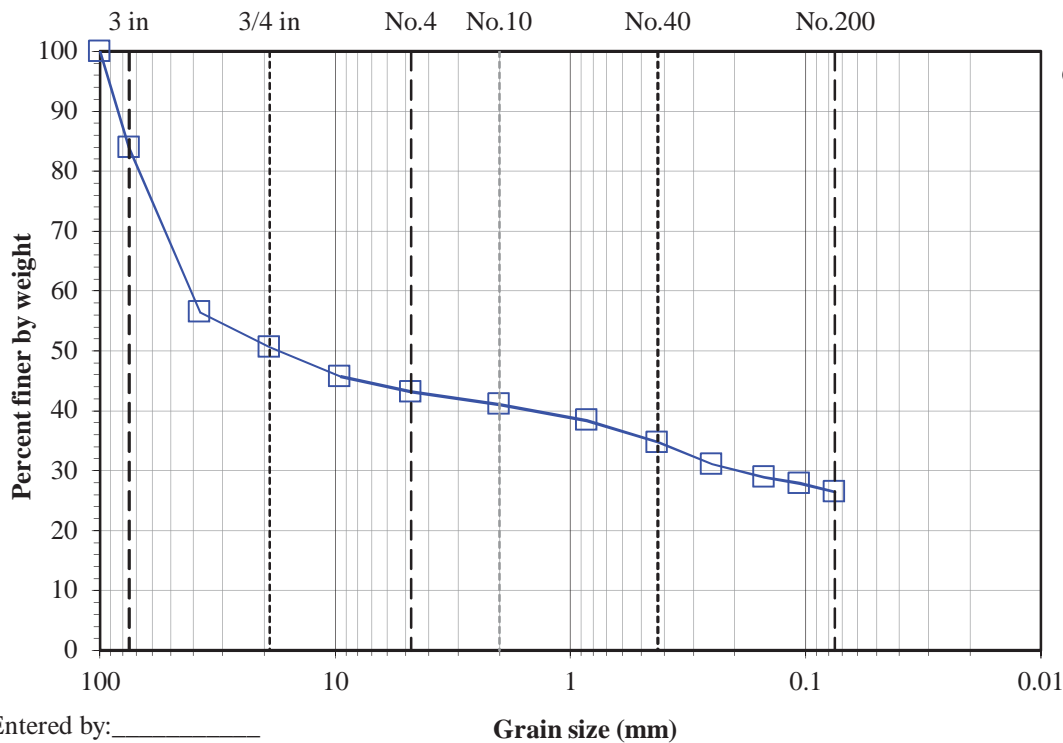
Description: Reddish brown clayey gravel with sand

Split: Yes		Moist		Dry	
Split sieve: 3/8"					
Total sample wt. (g):		4602.00	4266.95		
+3/8" Coarse fraction (g):		2341.80	2317.93		
-3/8" Split fraction (g):		512.43	441.88		
Split fraction:		0.457			

Water content data		C.F.(+3/8")	S.F.(-3/8")
Moist soil + tare (g):	2652.02	840.29	
Dry soil + tare (g):	2628.15	769.74	
Tare (g):	310.51	327.86	
Water content (%):	1.0	16.0	

Sieve	Accum. Wt. Ret. (g)	Grain Size (mm)	Percent Finer
8"	-	200	-
6"	-	150	-
4"	-	100	100.0
3"	684.05	75	84.0
1.5"	1859.65	37.5	56.4
3/4"	2108.29	19	50.6
3/8"	2317.93	9.5	45.7
No.4	24.34	4.75	43.2
No.10	44.74	2	41.1
No.20	70.65	0.85	38.4
No.40	106.36	0.425	34.7
No.60	141.28	0.25	31.1
No.100	162.40	0.15	28.9
No.140	172.21	0.106	27.9
No.200	186.43	0.075	26.4

←Split



Entered by: _____
Reviewed: _____

Liquid Limit, Plastic Limit, and Plasticity Index of Soils

(ASTM D4318)

Project: Eisenberg/Lot 71

No: 02565-001

Location: **Summit Powder Mtn.**

Date: **8/1/2017**

By: **BRR**

Grooving tool type: **Plastic**

Liquid limit device: **Mechanical**

Rolling method: **Hand**

Boring No.: TP-1

Sample:

Depth: 6.5'

Description: **Reddish brown lean clay**

Preparation method: **Wet**

Liquid limit test method: **Multipoint**

Screened over No.40: **Yes**

Larger particles removed: **Wet sieved**

Approximate maximum grain size: **3"**

Estimated percent retained on No.40: **See Particle Size Distribution**

Plastic Limit

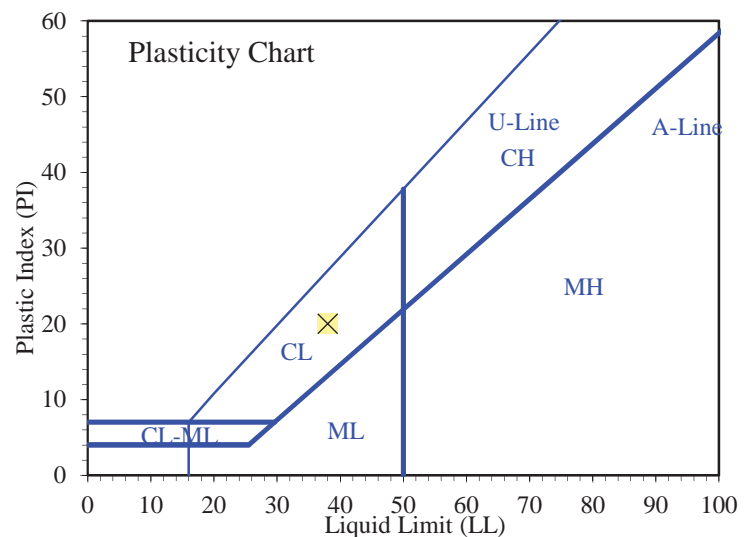
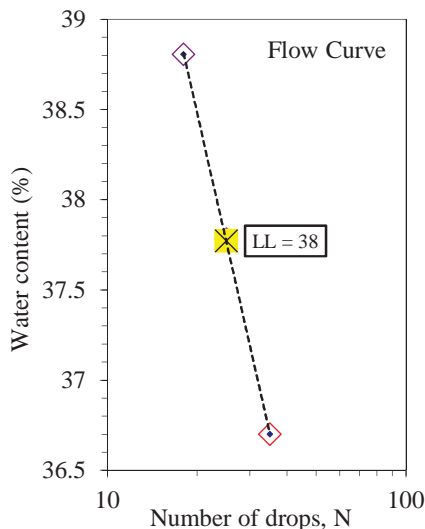
As-received water content (%): **7.9**

Determination No	1	2				
Wet Soil + Tare (g)	28.05	28.27				
Dry Soil + Tare (g)	27.06	27.32				
Water Loss (g)	0.99	0.95				
Tare (g)	21.51	22.03				
Dry Soil (g)	5.55	5.29				
Water Content, w (%)	17.84	17.96				

Liquid Limit

Determination No	1	2	3			
Number of Drops, N	35	25	18			
Wet Soil + Tare (g)	28.47	29.21	29.55			
Dry Soil + Tare (g)	26.58	27.20	27.47			
Water Loss (g)	1.89	2.01	2.08			
Tare (g)	21.43	21.88	22.11			
Dry Soil (g)	5.15	5.32	5.36			
Water Content, w (%)	36.70	37.78	38.81			
One-Point LL (%)		38				

Liquid Limit, LL (%)	38
Plastic Limit, PL (%)	18
Plasticity Index, PI (%)	20



Entered by: _____
Reviewed: _____

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



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

Project: Eisenberg/Lot 71

No: 02565-001

Location: **Summit Powder Mtn.**

Date: **8/1/2017**

By: **DKS**

Sample info.	Boring No.	TP-1							
	Sample								
	Depth	9.0'							
Water content data	Wet soil + tare (g)	90.47							
	Dry soil + tare (g)	84.66							
	Tare (g)	37.88							
	Water content (%)	12.4							
Chem. data	pH	7.12							
	Soluble chloride* (ppm)	<5.34							
	Soluble sulfate** (ppm)	16.5							
Resistivity data	Pin method	2							
	Soil box	Miller Small							
		Approximate Soil condition (%)	Resistance Reading (Ω)	Soil Box Multiplier (cm)	Resistivity (Ω-cm)	Approximate Soil condition (%)	Resistance Reading (Ω)	Soil Box Multiplier (cm)	Resistivity (Ω-cm)
		As Is	53790	0.67	36039				
		+3	25100	0.67	16817				
		+6	22810	0.67	15283				
		+9	23880	0.67	16000				
	Minimum resistivity (Ω-cm)	15283							

* Performed by AWAL using EPA 300.0

** Performed by AWAL using ASTM C1580

Entered by: _____

Reviewed: _____

APPENDIX C


Design Maps Detailed Report

2012/2015 International Building Code (41.3628°N, 111.7455°W)

Site Class C – “Very Dense Soil and Soft 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 2012/2015 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] $S_S = 0.811 \text{ g}$ **From [Figure 1613.3.1\(2\)](#) ^[2]** $S_1 = 0.269 \text{ g}$ **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 C, 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:			
<ul style="list-style-type: none"> • Plasticity index $PI > 20$, • Moisture content $w \geq 40\%$, and • Undrained shear strength $\bar{s}_u < 500 \text{ psf}$ 			
F. Soils requiring site response analysis in accordance with Section 21.1	See Section 20.3.1		

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

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

TABLE 1613.3.3(1)
VALUES OF SITE COEFFICIENT F_a

Site Class	Mapped Spectral Response Acceleration at Short Period				
	$S_s \leq 0.25$	$S_s = 0.50$	$S_s = 0.75$	$S_s = 1.00$	$S_s \geq 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	See Section 11.4.7 of ASCE 7				

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

For Site Class = C and $S_s = 0.811$ g, $F_a = 1.076$

TABLE 1613.3.3(2)
VALUES OF SITE COEFFICIENT F_v

Site Class	Mapped Spectral Response Acceleration at 1-s Period				
	$S_1 \leq 0.10$	$S_1 = 0.20$	$S_1 = 0.30$	$S_1 = 0.40$	$S_1 \geq 0.50$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	2.4
F	See Section 11.4.7 of ASCE 7				

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

For Site Class = C and $S_1 = 0.269$ g, $F_v = 1.531$

Equation (16-37):

$$S_{MS} = F_a S_s = 1.076 \times 0.811 = 0.872 \text{ g}$$

Equation (16-38):

$$S_{M1} = F_v S_1 = 1.531 \times 0.269 = 0.412 \text{ g}$$

Section 1613.3.4 — Design spectral response acceleration parameters

Equation (16-39):

$$S_{DS} = \frac{2}{3} S_{MS} = \frac{2}{3} \times 0.872 = 0.582 \text{ g}$$

Equation (16-40):

$$S_{D1} = \frac{2}{3} S_{M1} = \frac{2}{3} \times 0.412 = 0.274 \text{ g}$$

Section 1613.3.5 — Determination of seismic design category

TABLE 1613.3.5(1)

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

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

For Risk Category = I and $S_{DS} = 0.582 g$, Seismic Design Category = D

TABLE 1613.3.5(2)

SEISMIC DESIGN CATEGORY BASED ON 1-SECOND PERIOD RESPONSE ACCELERATION

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

For Risk Category = I and $S_{D1} = 0.274 g$, Seismic Design Category = D

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

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

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

References

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

USGS Design Maps Summary Report

User-Specified Input

Report Title Lot 71
Tue August 29, 2017 17:01:38 UTC

Building Code Reference Document 2012/2015 International Building Code
(which utilizes USGS hazard data available in 2008)

Site Coordinates 41.3628°N, 111.7455°W

Site Soil Classification Site Class C – “Very Dense Soil and Soft Rock”

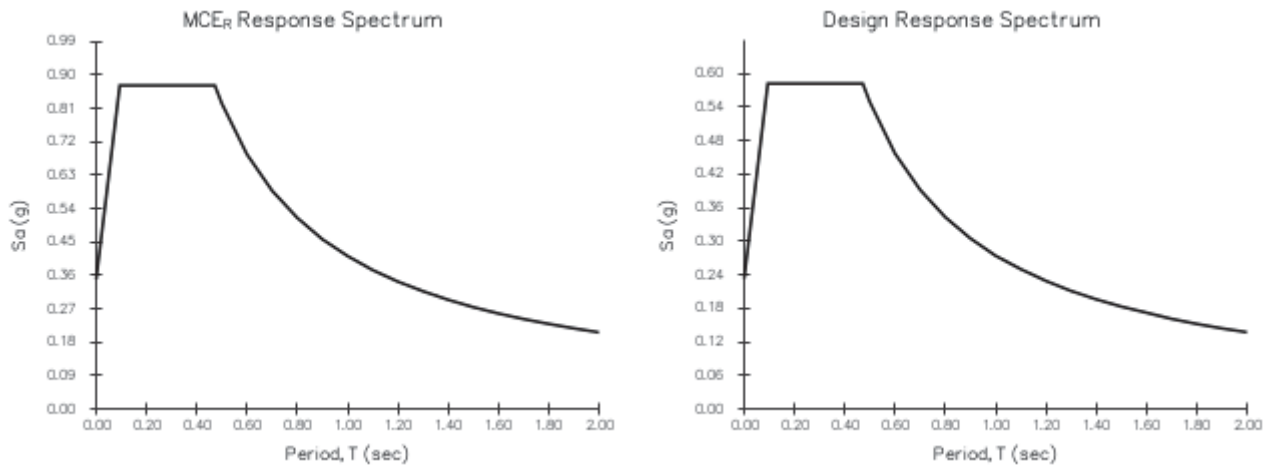
Risk Category I/II/III



USGS-Provided Output

$S_S = 0.811 \text{ g}$	$S_{MS} = 0.872 \text{ g}$	$S_{DS} = 0.582 \text{ g}$
$S_1 = 0.269 \text{ g}$	$S_{M1} = 0.412 \text{ g}$	$S_{D1} = 0.274 \text{ g}$

For information on how the S_S and S_1 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 D

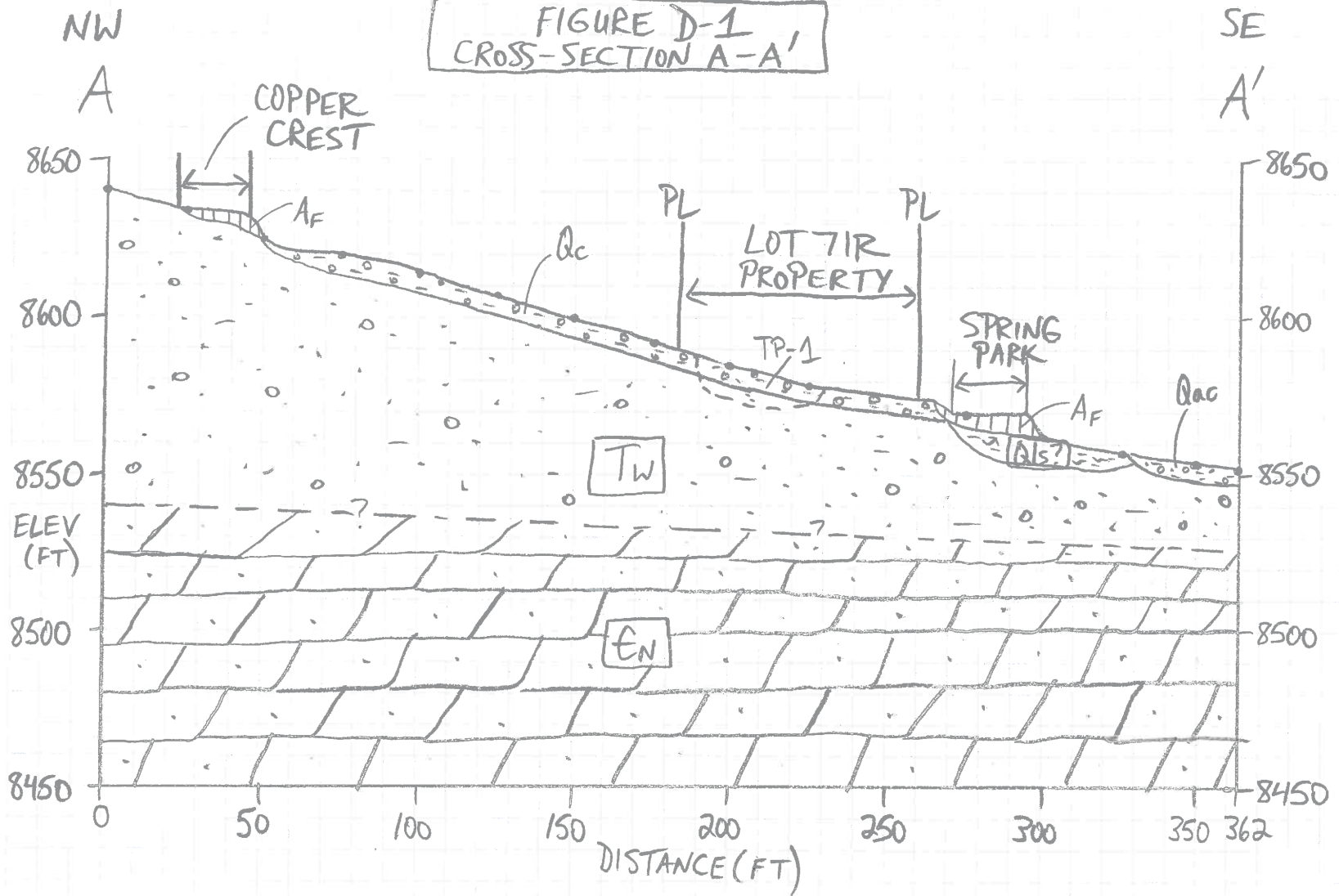


Intermountain GeoEnvironmental Services, Inc.

Project No. 02565-001


Date 8-23-17 by PED
Ckd by _____ on _____

FIGURE D-1
CROSS-SECTION A-A'



 AF = ARTIFICIAL FILL

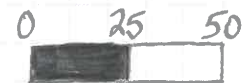
 Qac = ALLUVIUM AND COLLUVIUM

 Qc = COLLUVIUM

 ALS? = POSSIBLE LANDSLIDE

 Tw = WASATCH FORMATION

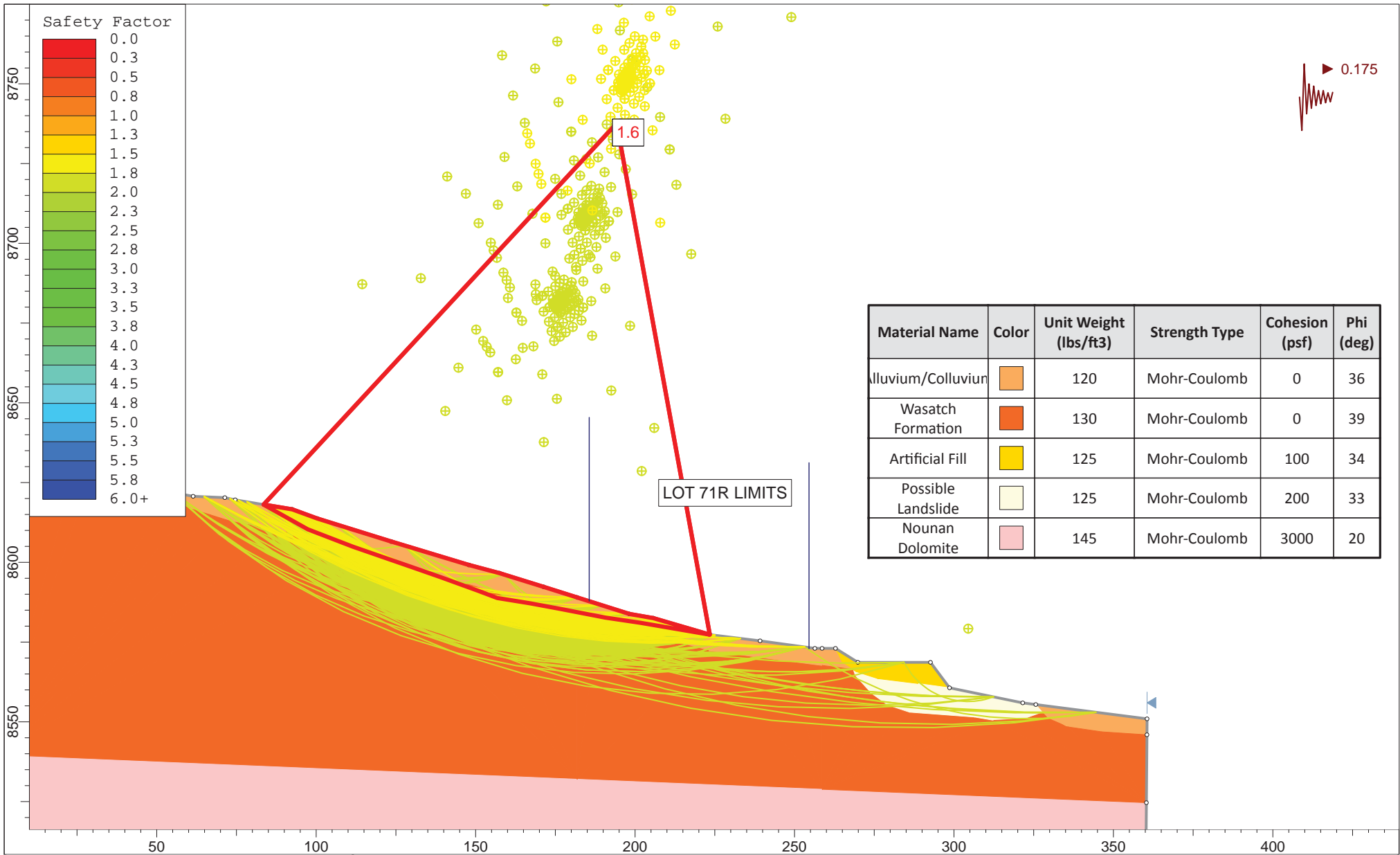
 En = NOUNAN DOLOMITE









FEET

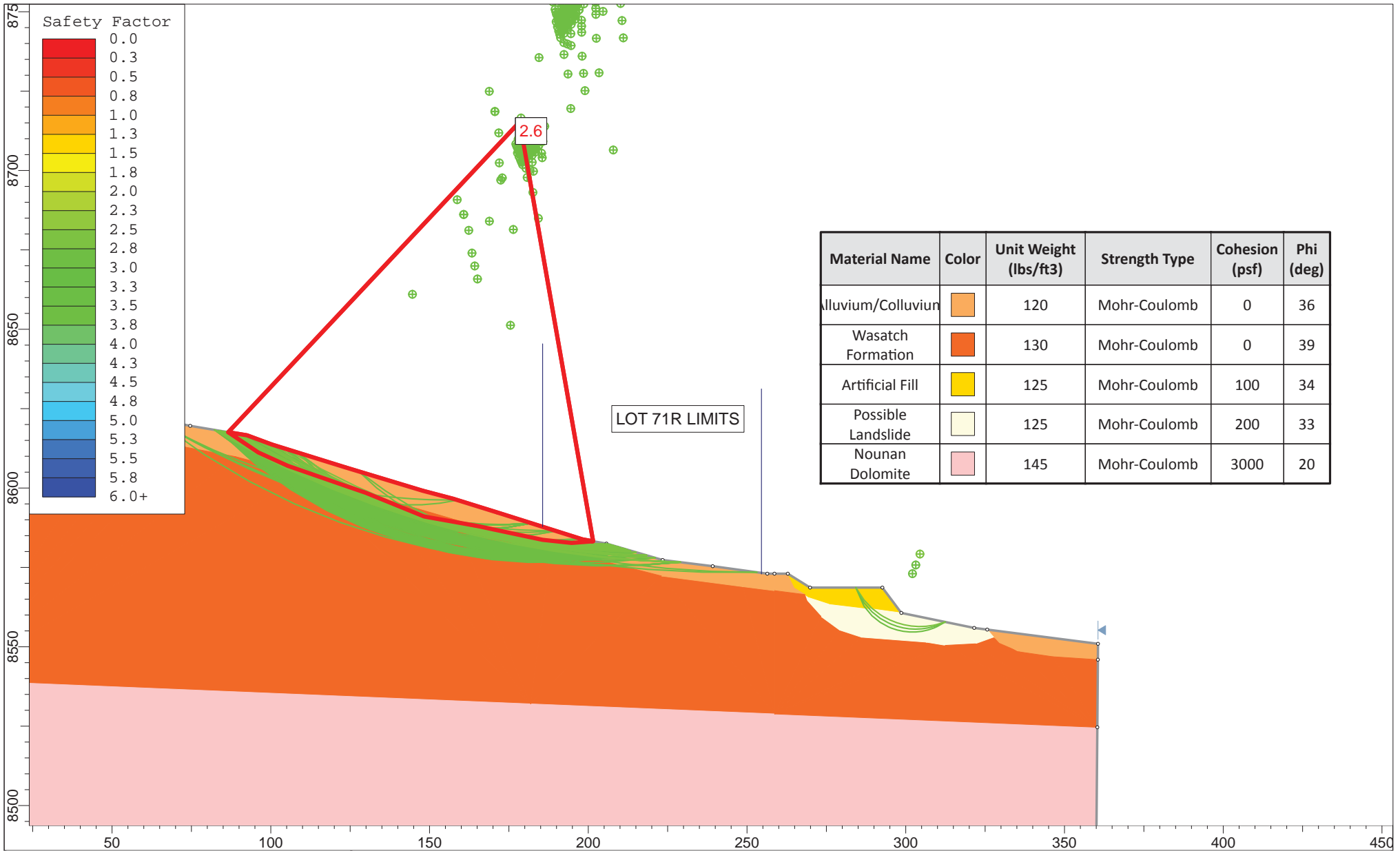
1" = 50'

*NO VERTICAL EXAGGERATION



Material Name	Color	Unit Weight (lbs/ft ³)	Strength Type	Cohesion (psf)	Phi (deg)
Illuvium/Colluvium		120	Mohr-Coulomb	0	36
Wasatch Formation		130	Mohr-Coulomb	0	39
Artificial Fill		125	Mohr-Coulomb	100	34
Possible Landslide		125	Mohr-Coulomb	200	33
Nounan Dolomite		145	Mohr-Coulomb	3000	20

	Project				Powder Mountain Lot 71R	
	Analysis Description				Seismic	
	Drawn By	TQH	Scale	1:500	Company	IGES Inc.
	Date	8/29/2017		File Name	Lot 71.slim	



SLIDEINTERPRET 7.025

Project		Powder Mountain Lot 71R			
Analysis Description		Static			
Drawn By	TQH	Scale	1:500	Company	IGES Inc.
Date	8/29/2017	File Name	Lot 71.slim		

Lot 71R
 02565-001
 8/29/2017

c'	100	psf	Effective Cohesion (including apparent cohesion for coarse, angular soils)
ϕ'	28	deg	Effective Friction Angle
γ_{sat}	136	pcf	Saturated Unit Weight of Soil
γ_w	62.4	pcf	Unit weight of water
h	3	ft	Depth to shear surface
β	16.0	deg	Slope Gradient (3.5H:1V)

FS 1.93

Input Variable
 Calculated Value

This model assumes $c > 0$ and the face of the slope is saturated to depth h

