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GEOTECHNICAL AND GEOLOGIC HAZARD INVESTIGATION
Lot 44R of Summit Eden Phase 1C
8645 E. Copper Crest
Summit Powder Mountain Resort
Weber County, Utah

IGES Project No. 02732-001

March 19, 2018

Prepared for:

Dr. Tom Buttgenbach, PhD

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1.0 INTRODUCTION

1.1 PURPOSE AND SCOPE OF WORK

This report presents the results of a geotechnical and geologic hazards investigation conducted for Lot 44R 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 January 15, 2018, 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 project site is located within the Summit Powder Mountain Resort, illustrated on the *Site Vicinity Map*, Figure A-1 in Appendix A.

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 44R is located within Phase 1C of the Powder Mountain expansion project (Summit Eden), on the south side of Copper Crest – the street address is 8645 E. Copper Crest. The 0.58-acre residential lot has an approximate buildable area (building envelope) of 6,450 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 the architectural drawings provided by Tom Wiscombe Architecture (TWA), the new home will be a three- to four-level structure, the lowest level consisting of a partial walk-out basement, founded on piers (or conventional spread footings if practical).

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 project-wide work included twenty-two test pits and one soil boring excavated at various locations across the 200-acre development. IGES has performed a single-lot investigations for Lot 75R (IGES, 2017a), located just south of Lot 44R, as well as for Lot 71R, located approximately 150 feet southeast of the Lot 44R building envelope (IGES, 2017b). 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 44R 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 two test pits at representative locations within the property boundary. The approximate locations of the test pits 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 logs, Figure A-4 and Figure A-5 in Appendix A. A key to USCS symbols and terminology is included as Figure A-6, and a key to physical rock properties is included as Figure A-7.

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)
- Fines Content (ASTM D1140)
- In situ Moisture Content (ASTM D7263)
- Direct Shear (ASTM D3080)
- 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 logs (Figures A-3 and A-4).

3.0 GEOLOGIC CONDITIONS

3.1 GENERAL GEOLOGIC SETTING

The Lot 44R 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 almost 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 44R 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-7). 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-8). In this map, the property is not located within mapped landslide deposits, though deposits mapped as "mixed slope colluvium, shallow landslides, and talus," and a large Holocene to Late Pleistocene landslide deposit have been mapped within 500 feet of the southern margin of the property. Finally, Coogan and King (2016) updated their 2001 map, which shows the property to be entirely located within the Wasatch Formation, with young landslide deposits (Qms) mapped within 500 feet to the west and south of the property (*Regional Geology Map 3*, Figure A-9). 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 44R project area is situated on a slope, with the local topographic gradient down to the southwest towards 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. Groundwater seepage is known to occur at the base of the slope at the Lot 75R road cut in the spring (IGES, 2017a).

Baseline groundwater depths for the Lot 44R property are currently unknown, but are anticipated to fluctuate both seasonally and annually. Groundwater was not encountered in either of the test pits 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 44R 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 the west. Elliott and Harty (2010) similarly does not show the property to be located within mapped landslide deposits, though deposits mapped as “Landslide undifferentiated from talus and/or colluvial deposits” are noted both west and south of the property. As noted above, both Western Geologic (2012; Figure A-8) and Coogan and King (2016; Figure A-9) show the property to be located outside of mapped landslide deposits, though mass-movement deposits are mapped both west and south of the property.

3.4.2 Faults

Neither Christenson 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

Christenson 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 Christenson 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 on the subject property in the aerial photography.

Google Earth imagery of the property from between the years of 1993 and 2017 were also reviewed. No landslide or other geological hazard features were noted in the imagery. Preceding the installation of Copper Crest, the property was observed to be covered primarily in bushes, though a large, dense tree cluster was observed across much of the east-central part of the property, including the eastern half of the building envelope. Some scattered surficial gravel, cobbles, and boulders were observed, though the property does not contain any drainages. Between September of 2011 and October of 2014, Summit Pass, Copper Crest, and Spring Park roads were in the process of being constructed with a series of up-hill cuts and down-hill fills, and the entire building envelope area was observed to have been modified by human activity at this time. Between October of 2014 and June of 2017, disturbance of the southwestern part of the property was observed in association with the installation of off-property rockeries north of the Spring Park cul-de-sac.

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)	$S_S = 0.813$	$S_1 = 0.270$
MCE Spectral Response Acceleration Site Class C (g)	$S_{MS} = S_sF_a = 0.874$	$S_{M1} = S_1F_v = 0.413$
Design Spectral Response Acceleration (g)	$S_{DS} = S_{MS}*^2/_3 = 0.582$	$S_{D1} = S_{M1}*^2/_3 = 0.275$

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 1.075 and long-period (F_v) site coefficient is 1.530. 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 $0.4 * S_{MS}$.

3.7 GEOLOGIC HAZARDS ASSESSMENT

Geologic hazards 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 44R property.

3.7.1 Landslides/Mass-Movement

According to the several most recent geologic maps produced that cover the property, the property is not situated within mapped landslide deposits, though landslide deposits have been mapped within 500 feet both west and south of the property boundary (Coogan and King, 2016; Western Geologic, 2012; Elliott and Harty, 2010). Additionally, 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. Given the geologic data alone, the risk associated with landslides is considered low to moderate, given the proximity to mapped landslide deposits.

However, the entire building envelope is situated upon a thick undocumented fill prism, and shallow groundwater is known to be present downslope during peak runoff in the spring. These conditions (loading of the slope, steep slope, possible shallow groundwater) provide an increased risk to slope instabilities. Slope stability modeling as part of our assessment indicates that the slope is stable under current conditions for both static and seismic cases, though surficial instabilities may arise under saturated conditions (see Section 4.3). The slope stability modeling confirms the landslide hazard risk classification for the property as being low to moderate.

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 pits excavated as part of this investigation. The test pits were excavated in early February, and the groundwater level was likely to be at or near its annual low. 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 downslope at the nearby Lot 75R property (IGES, 2017a).

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 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

At the time of the subsurface investigation (February 8, 2018), the site was snow-covered with between approximately 12 to 24 inches of snow. As such, detailed site reconnaissance was not able to be performed at the time. However, Mr. Peter E. Doumit, P.G., C.P.G., of IGES had previously conducted reconnaissance of the site and the immediate adjacent properties as part of a field mapping exercise for the Spring Park property to the southwest of Lot 44R (IGES, 2017c). 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 44R property and adjacent areas, based upon surficial mapping for previous nearby projects and the results of the Lot 44R subsurface investigation.

At the time of the site reconnaissance, the property was observed to have a steep fill prism extending from the southern end of the building envelope upslope to the north to Copper Crest. Downslope of the fill prism, surface topography was observed to consistently slope downhill to the southwest. A few small aspen trees were observed near the eastern margin of the property, though most of the property was covered in low-lying bushes and grasses.

Pebbles, cobbles, and some boulders of dark yellowish orange to purple quartzite were observed on the surface. These rock clasts[1] were typically subrounded to rounded, and up to 2 feet in diameter, though generally 1 to 3 inches in diameter. The clasts were interpreted to be 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. No adverse geologic conditions were observed on the property at this time.

4.2 SUBSURFACE CONDITIONS

On February 8, 2018, two exploration test pits were excavated within and adjacent to the building envelope (see Figure A-2). The test pits were excavated to a depth of between 10 and 12 feet below existing grade with the aid of a Hitachi Zaxis 160LC tracked excavator. Upon completion of logging, the test pits were backfilled without compactive effort. Detailed logs of the test pits are displayed in Figures A-3 and A-4. Four distinct geologic units were encountered in the subsurface.

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[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)

The soil and moisture conditions encountered during our investigation are discussed in the following paragraphs.

4.2.1 Earth Materials

Undocumented Fill: This artificial fill unit was found to be at least 5 feet thick, and as much as 8 feet thick or more at the crest of the slope. The unit was found to be variable in character and composition between the two test pits. In TP-1, the unit was a dark yellowish brown to moderate reddish brown, medium dense, moist, clayey SAND with gravel (SC), with gravel and larger-sized quartzite clasts comprising between approximately 15 and 20% of the unit. In TP-1, much of the fill appeared to have been derived from weathered Wasatch Formation. In TP-2, the unit was a dark yellowish brown to brownish black, medium stiff, moist, sandy lean CLAY with gravel (CL), with gravel and larger-sized quartzite clasts comprising between 10 and 15% of the unit. In TP-2, much of the fill appeared to have been moved topsoil. In both test pits, a 5-inch corrugated plastic drain pipe was observed to have been placed through the fill, and multiple layers of geofabric were observed within the fill unit.

A/B Soil Horizon: This topsoil unit was found to be between approximately 1 and 2 feet thick. The unit was a grayish brown, medium stiff, moist, sandy lean CLAY with gravel (CL), with gravel and larger-sized quartzite clasts comprising between approximately 15 and 25% of the unit. The topsoil contained abundant plant and tree roots and was found to be forming upon the underlying colluvium unit.

Colluvium: This unit was approximately 1 to 2½ feet thick. The unit consisted of a moderate yellowish brown to pale yellowish orange, medium dense, slightly moist, silty SAND with gravel (SM) that was gradational to a clayey SAND with gravel (SC). Gravel and larger-sized subrounded to subangular quartzite clasts comprised between approximately 20 and 25% of the unit. Individual clasts were as much as 1 foot in diameter, though the mode clast size was approximately 2 to 4 inches in diameter. The unit contained common to occasional pinhole voids up to 1 mm in diameter.

Wasatch Formation: This unit was at least 6 feet thick and extended to the maximum depth of exploration within both test pit. The unit consisted of weakly consolidated conglomerate bedrock that had been largely disaggregated into a dark yellowish orange to pale reddish brown, medium-dense to dense, moist mixture of clay, sand, and gravel that collectively classifies as silty GRAVEL with sand (GM). Gravel and larger-sized subrounded to subangular quartzite clasts comprised between approximately 20 and 50% of the unit, with individual clasts up to 1½ feet in diameter, with a mode clast size of 2 to 4 inches. In TP-1, this unit was observed to be softer, sandier, and less clayey than as seen in TP-2.

4.2.2 Groundwater

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

4.3 SLOPE STABILITY

4.3.1 Global Stability

The stability of the existing fill 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 rotational-type failure occurring through surficial soils (primarily embankment fill and colluvium), just above the underlying conglomerate 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; 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).

Both Copper Crest Road and Summit Pass Road are located at the top of the slope; accordingly, a traffic surcharge of 250 psf has been modeled for static conditions. The new home is expected to be founded on deep foundations; therefore, the majority of the load from the home will be transferred to deeper stratum, hence a surcharge load from the home was not included in the analysis.

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)
Colluvium (Qc)	30	100	120
Bedrock (Tw)	40	100	130
Embankment Fill (Af)	37	75	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 0.3496g. Half of the PGA, (0.175g), 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	1.65	1.13

The results of the analysis indicate 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. A summary of the slope stability analysis is presented in Appendix D.

4.3.2 Surficial Stability

Our subsurface investigation indicates that the fill slope comprising the majority of the buildable area consists of clayey sand with gravel (SC) gradational to sandy lean clay with gravel (CL). This material, having been placed by mechanical means, was generally heterogeneous and comprised of a mixture of topsoil and weathered Wasatch Formation and colluvium.

IGES assessed the potential for the upper two feet to become mobilized under saturated parallel seepage conditions. Our assessment assumes two feet of clayey sand, fully saturated, and a 2H:1V slope (this would be a transient condition that could occur during primary spring run-off and snowmelt). Our model assumes an effective friction angle of 37 degrees and a cohesion of 75 psf, and a saturated unit weight of 135 pcf. Based on this model, a factor-of-safety of 1.50 results. If the depth of saturation increases to three feet, the factor of safety is reduced to 1.27. It should be noted that our model assumes a 2H:1V slope, as the grading plans utilized by IGES indicates a 2H:1V slope; however, IGES did not survey the lot, it is possible that some portions of the slope could be steeper than 2H:1V, thereby increasing the possibility of surficial slope instability under saturated conditions. Sample calculations are presented in Appendix D.

Based on our infinite slope model, IGES considers the potential for surficial slope instability on this site to be low to moderate on the 2H:1V fill slope within the building envelope, and low for the rest of the site. For portions of the slope that are steeper than 2H:1V (should they exist), the potential for surficial slope instability is moderate.

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 44R project area:

- **The Lot 44R project area does not appear to have geological hazards that are capable of adversely impacting the development as currently proposed under the existing conditions.**

- No evidence of landsliding was observed on the surface or subsurface of the property, though the building envelope is underlain by undocumented fill on a slope with possible shallow groundwater conditions. Slope stability modeling indicates the slope is stable under current conditions, but saturated surface conditions could potentially induce localized shallow surficial slope failures, especially where slopes steeper than 2H:1V exist. Therefore, the risk of landslide hazards is considered to be low to moderate.

- 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 pits, though groundwater seepage has been observed in test pits and springs on nearby properties; therefore, shallow groundwater hazards are considered to be 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 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 ½ 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 structural 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 up to 18 inches in diameter, 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, the proposed new home may be founded on conventional shallow foundations. However, if significant eccentric loads are anticipated or the home will be an irregular-shaped structure, a deep foundation system may be preferable. Based on our conversations with TWA, a deep foundation system would necessarily be designed to resist relatively high axial and lateral loads; accordingly, drilled piers or auger-cast piles would be an appropriate choice. Driven steel pipe piles or H-piles are an option but may not be the most economically viable considering the presence of relatively dense sand and gravel, potentially challenging driving conditions, and the potential for vibration damage to nearby improvements.

5.4.1 Conventional Spread Footings

If loading conditions allow, new home may be founded on conventional spread footings. The footings may be founded either *entirely* on competent native soils <u>or</u> *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 4 feet below existing *natural* grade, but may be deeper, or shallower, at specific locations. However, the entirety of the buildable area of the lot consists of a fill embankment associated with Copper Crest Road, hence undocumented fill will be encountered in conjunction with the existing road embankment. 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 **3,400 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.

5.4.2 Deep Foundations

We understand that the Architect is considering some sort of cast-in-place concrete piles for this project; two common methods of construction are conventional *drilled piers/shafts* and *Augered Cast-in-Place Piles* (ACIP). Conventional drilled piers/shafts (also referred to as *caissons*) are constructed using large-diameter flight augers, cased or uncased, drilled to a specified depth and/or bearing layer. Reinforcement is placed within the open shaft and concrete is subsequently pumped into the bottom of the shaft. ACIP piles are constructed by first drilling a pile shaft with a conventional continuous flight auger. As the flight auger is withdrawn from the ground, concrete is injected under pressure at the tip of the auger. Once the auger has been removed, a steel reinforcement assembly (or “cage”) is placed within the concrete, thereby creating a reinforced concrete shaft. ACIP piles are particularly well-suited for areas with shallow groundwater or loose sands, conditions which may cause difficulties with construction of conventional drilled shafts. ACIP piles and drilled shafts can be more economical than driven piles when relatively few piles will be constructed (i.e., on the order of 100 piles or less); however, if driven piles are considered as an option, the Owner should discuss construction costs with a specialty foundation contractor as a part of assessing both cost and feasibility.

Considering that the structure will be on sloped ground, and that subsurface conditions may vary over a short distance depending on the elevation of the pile cap, design of the deep foundation system should be completed once the following is known:

- Pile foundation layout
- Elevation(s) of pile caps
- Axial loading on piles and/or pile groups (compression, tension)
- Lateral loads on piles
- Allowable lateral deflection of piles

For *planning purposes only*, Table 5.4.2 presents allowable axial capacities for piles of various lengths and diameters. These values assume the pile is constructed fully within Wasatch Formation, with the upper 5 feet neglected for skin friction. Actual capacities for the final design may be different depending on the elevation of the pile cap – for example, for the portion of a pile constructed within undocumented fill, skin friction should be neglected, and down-drag may need to be considered. For lateral capacity, variability with respect to finish grade/slope gradient and earth materials (undocumented fill, colluvium, Wasatch Formation) could have a significant impact on the design; hence, evaluation of lateral capacity should be performed as a part of the final pile design.

Table 5.4.2
Preliminary Allowable Capacity for Concrete Cast-in-Place Pile Foundations

Concrete Pile diameter (in)	Pile Length (ft)*	Allowable axial compression (kips)	Allowable axial uplift (kips)
24		179	27
30	20	270	37
36		380	48
24		296	55
30	30	440	74
36		612	94
24		429	94
30	40	630	123
36		869	154

*Length measured from bottom of pile cap to tip of shaft

For pile lengths and diameters falling between the values presented, linear interpolation may be used for design. Actual capacities may be adjusted for installation method (e.g. higher capacities can generally be obtained for ACIP, however piles greater than 24 inches dia. may be impractical for ACIP installation methods).

We recommend a minimum center-to-center spacing of three (3) pile diameters if individual concrete piles are used. At this and larger spacing the axial capacity of a group may be taken as the sum of the individual (single) pile axial capacities. If closer spacing is desired, appropriate reduction factors should be considered in the design.

5.4.2.1 Constructability

Special conditions may impact the contractor's ability to install *drilled shafts*, particularly the presence of loose sand and/or groundwater (these conditions do not appreciably impact the installation of *auger-cast piles* since there is no open excavation). The contractor should be aware that casing may be required to maintain an open hole during construction. The casing should be removed carefully during concrete placement. The bottom of the casing should be maintained approximately one foot below the level of the rising concrete to avoid caving in of the shaft side.

Consideration of subsurface conditions should be given when selecting the size of the drilling equipment for either the drilled shafts or ACIP piles. Due to the larger particle size observed in our investigation, the contractor should select a shaft size that will allow for the proper evacuation of the drilled material.

An IGES representative should be present during drilling of shafts to observe that the bottoms are relatively clean, established at the proper depth, and located within a dense sand/gravel layer or Wasatch Formation. Shafts should be straight and plumb and all shafts should be observed and documented prior to concrete placement. All shaft excavations should be cleaned of loose soil or slough that may be present at the bottom of the shaft prior to placement of concrete. All concrete should be placed using an "elephant trunk"/tremie pipe from the bottom up to avoid segregation.

The reinforcement in the concrete piers should be continuous for the entire length of the pier and should be designed by the structural engineer. We recommend water-reducing admixtures be used to increase the slump during placement and still maintain a low water cement ratio. The concrete in each drilled shaft should be placed in one continuous pour.

5.4.2.2 Evaluation of Concrete Pile Integrity

Concrete pile integrity should be evaluated by testing after the completion of the shafts. This testing could include (but is not necessarily limited to) Cross-Hole Sonic Logging (CHS), Thermal Integrity Profiler (TIP), or Pile Integrity Testing. All holes should be visually inspected when possible, however if ground water is present or sloughing soils are observed CHS or TIP testing should be performed. Any shafts that are not CHS or TIP tested but experience questionable results (such as concrete pour delays or a concrete imbalance) should be tested using a PIT. Further testing of the shafts is advisable if the owner is risk adverse. IGES can provide pier integrity evaluation services upon request. It should be noted that TIP testing may be impractical for piers that are shorter than 4 time the shaft diameter.

5.5 SETTLEMENT

5.5.1 Static Settlement

Static settlements of properly designed and constructed conventional foundations, founded as described in Section 5.4.1, 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 of conventional spread footings 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.

5.5.3 Deep Foundation Settlement

Settlement of deep foundations should be evaluated by the designer; however, considering that deep foundations are likely to be installed several feet into a bedrock unit (Wasatch Formation), settlement of deep foundation elements is not expected to significantly impact the proposed development.

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 0.47 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 **270 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. Irrigation valves should be placed a minimum of 5 feet from foundation walls and must not be placed within the basement backfill zone. Over-watering near the foundation walls is discouraged; use of Xeriscape and/or a drip irrigation system should be considered. Pavement sections should be constructed to divert surface water off the pavement into storm drains, curb/gutter, or another suitable location.

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. The foundation perimeter drain be should constructed in accordance with the latest edition of 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 516 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 9,373 OHM-cm, soluble chloride content of 72 ppm and a pH of 5.6. 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 18 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 warrantees, 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 44R. 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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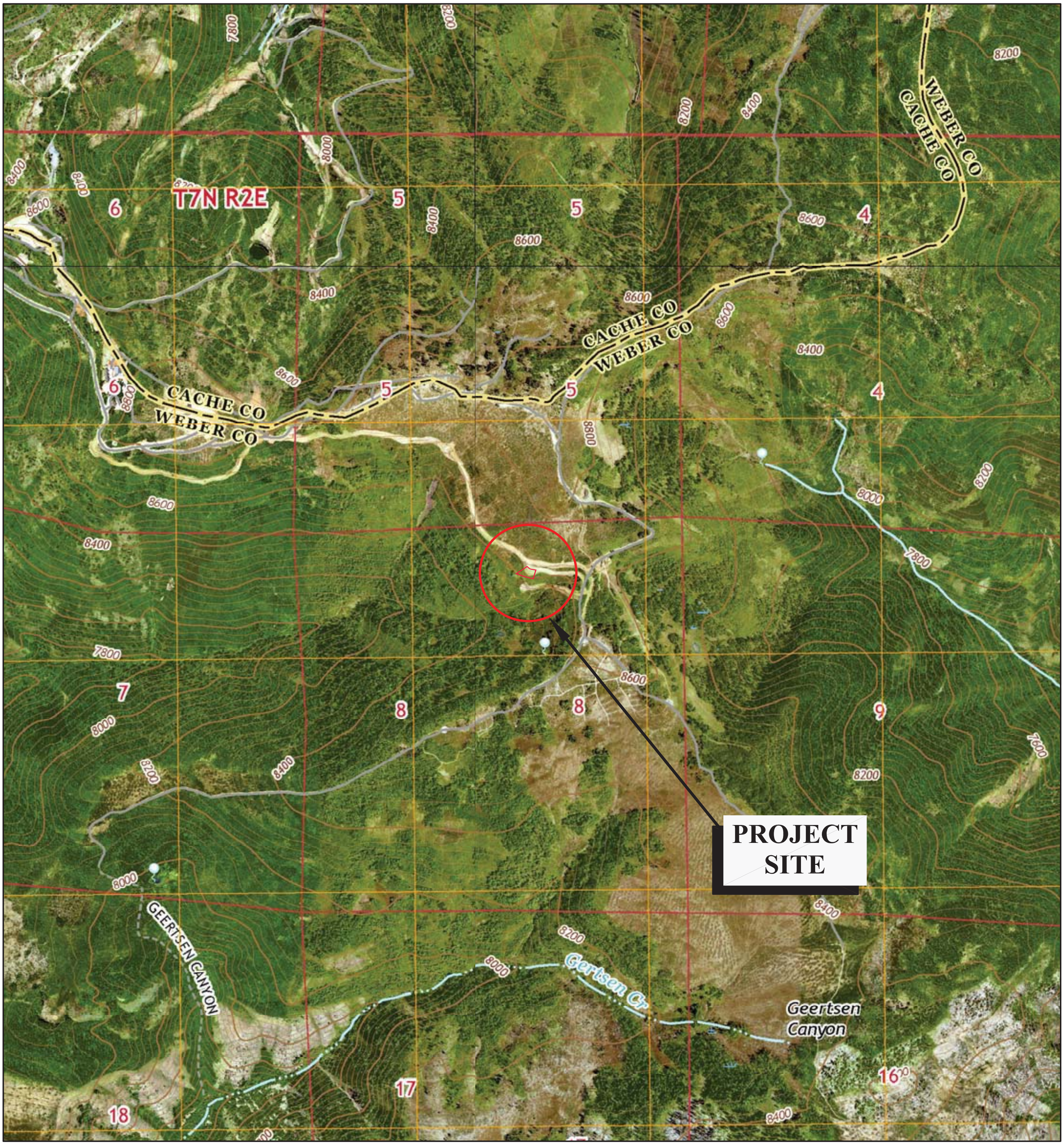
AERIAL PHOTOGRAPHS

Data Set	Date	Flight	Photographs	Scale
1947 AAJ	August 10, 1946	AAJ_1B	88, 89, 90	1:20,000
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 Maps:
USGS *Huntsville*, *Brown's Hole*, *James Peak*, and *Sharp Mountain* 7.5-Minute Quadrangles (2017)

Map Location

N

0' 1,000' 2,000' 4,000'

SCALE: 1"=2,000'

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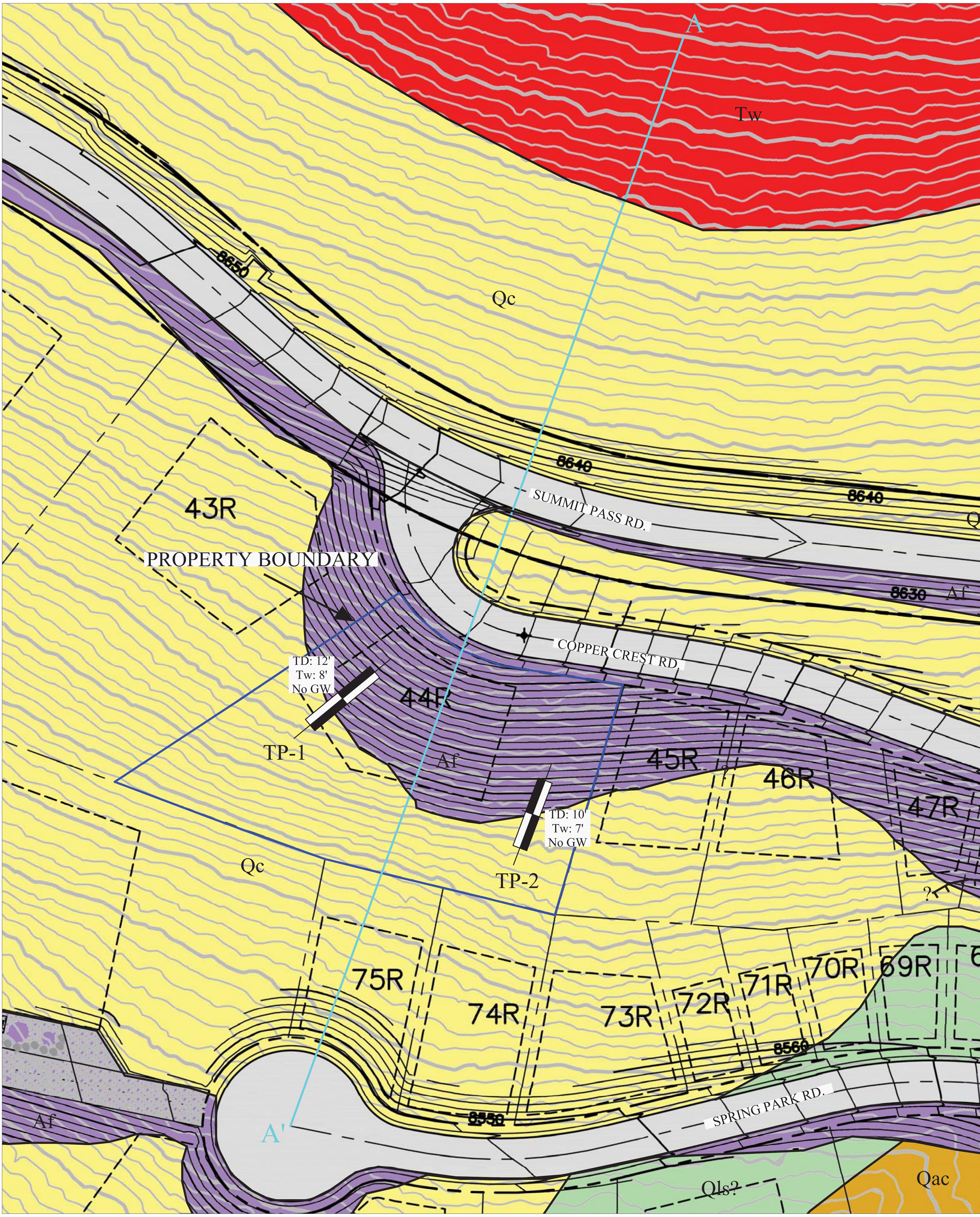
SITE VICINITY MAP

Figure

A-1



LEGEND

BASE MAP:

-UNDATED 100-SCALE MAP OF PHASE 1C-1D PREPARED BY NV5

CONTOUR INTERVAL: 2'

Af ARTIFICIAL FILL

Qls? POSSIBLE LANDSLIDE DEPOSITS

Qac ALLUVIUM & COLLUVIUM

Qc COLLUVIUM

Tw WASATCH FORMATION

APPROXIMATE GEOLOGIC CONTACTS

PROPERTY BOUNDARY

TP-1 TEST PITS

A A' CROSS SECTION

LANDSLIDE SCARP

N

0 50 100

FEET

1" = 50'

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Lot 44R of Summit Eden Phase 1C
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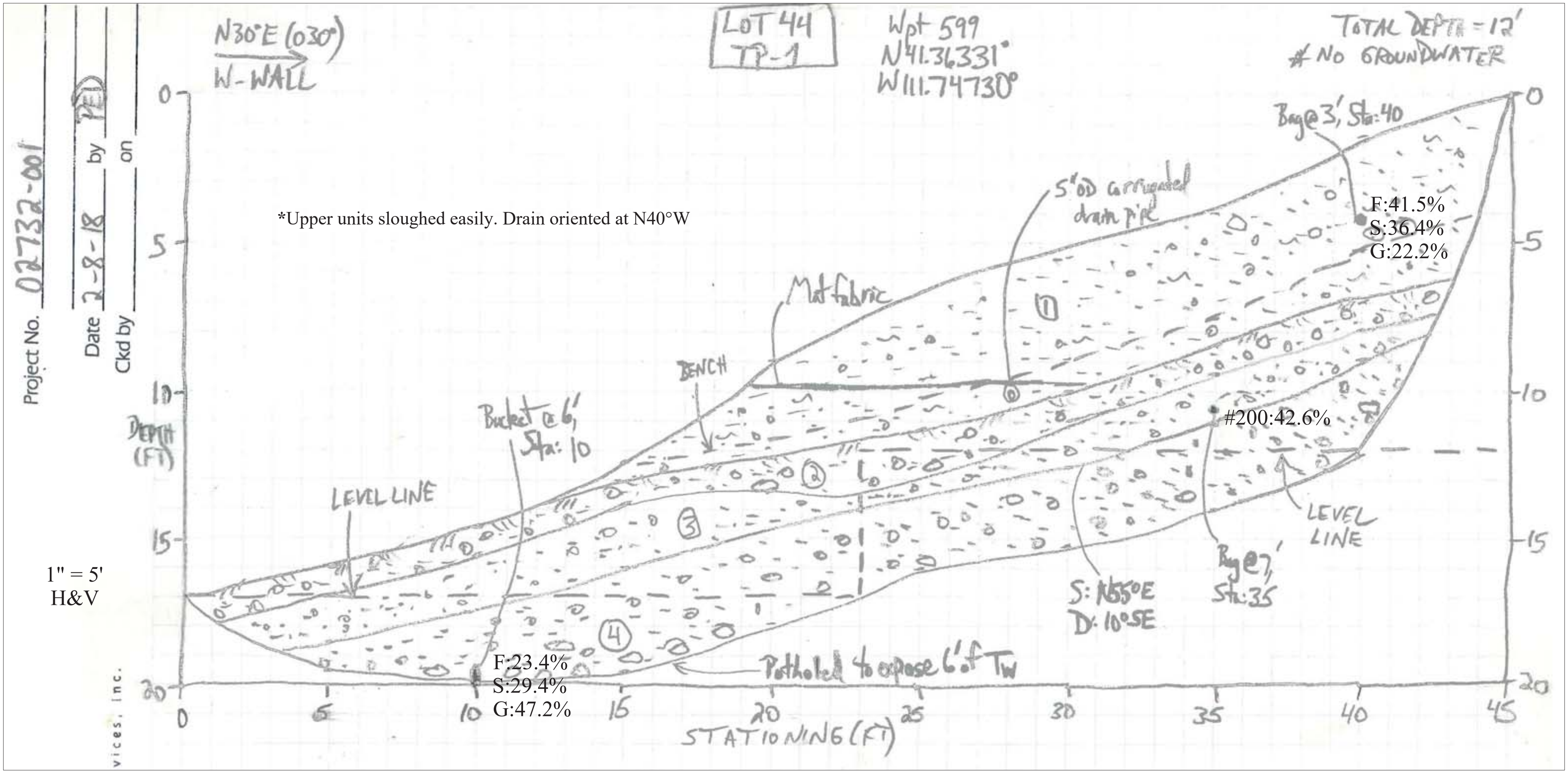
Weber County, Utah

LOCAL GEOLOGY AND GEOTECH MAP

FIGURE

A-2



LITHOLOGIC UNIT DESCRIPTIONS

1. Undocumented Fill:~ up to 8' thick (as seen on north wall); dark yellowish brown (10YR $\frac{4}{2}$) to moderate reddish brown (10R $\frac{4}{6}$), clayey SAND with gravel (SC), medium dense, moist, low to moderate plasticity fines, massive; gravel and larger sized clasts comprise ~15-20% of unit; clasts all medium light gray (N6) to pale yellowish orange (10YR $\frac{8}{6}$) quartzite, subrounded to subangular, up to 1.5' in diameter, though mode size is ~2-4"; appears as mainly modified Wasatch Formation (Tw); sharp, irregular basal contact.

2. A/B Soil Horizon: ~1-1.5' thick; grayish brown (5Y $\frac{3}{2}$) sandy lean CLAY with gravel (CL), medium stiff, moist, low plasticity, massive; gravel and larger sized clasts comprise ~25% of unit, clasts are rounded medium light gray (N6) quartzite up to 1' in diameter, though mode size is ~3-4"; abundant plant and tree roots; sharp, irregular basal contact.

3. Colluvium: ~1-2' thick; light brown (5YR $\frac{6}{4}$) to pale yellowish orange (10YR $\frac{8}{6}$) silty SAND with gravel (SM), medium dense to dense, slightly moist, low plasticity fines, massive; gravel and larger sized clasts comprise 20-25% of the unit, all subrounded to subangular quartzite as above, up to 1' in diameter, though mode size is ~3-4"; occasional pinhole voids up to 1 mm in diameter; contains some small irregular pockets of topsoil inclusions, possible burrows; few plant and tree roots; gradational, irregular basal contact.

4. Wasatch Formation: >6' thick; dark yellowish orange (10YR $\frac{6}{6}$) to pale reddish brown (10R $\frac{5}{4}$) well-graded gravelly SAND (SW), medium dense to dense, slightly moist to moist, minor low plasticity fines, massive to faintly thinly bedded; sand is medium-grained to coarse-grained; gravel and larger sized clasts comprise ~30-40% of the unit; clasts are subrounded to subangular quartzite as above, up to 1.5' in diameter, though mode size is ~3-4"; clast size increases with depth (normally graded); softer, sandier, and less clayey than TP-2.

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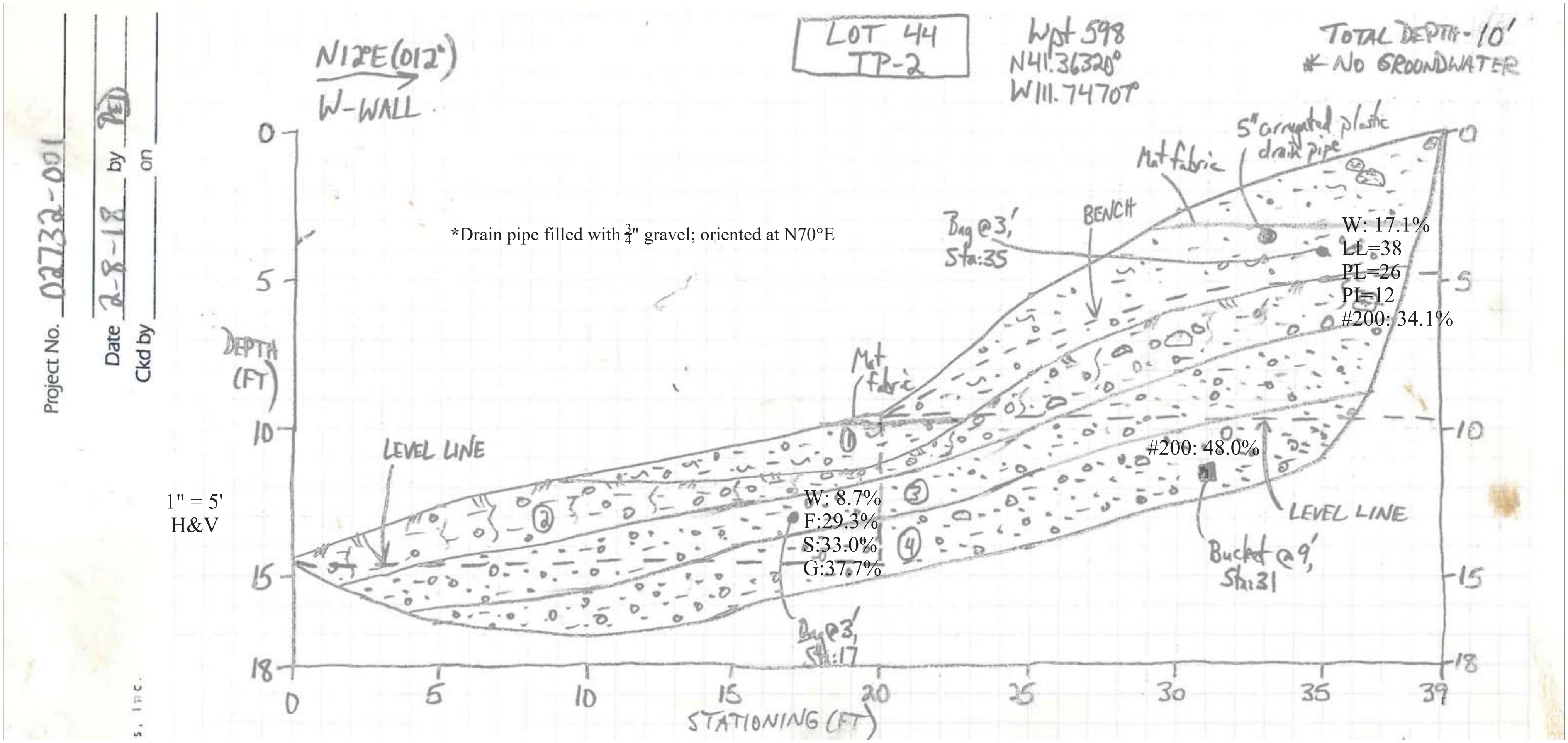
TEST PIT 1 LOG

FIGURE

A-3



LITHOLOGIC UNIT DESCRIPTIONS

1. Undocumented Fill: Up to ~5' thick; dark yellowish brown (10YR $\frac{4}{2}$) to brownish black (5YR $\frac{2}{1}$) sandy lean CLAY with gravel (CL), medium stiff, moist, low plasticity, massive; gravel and larger sized clasts comprise ~10-15% of the unit; clasts entirely subangular to subrounded medium light gray (N6) quartzite up to 1' in diameter, though mode size is ~1"; organic-rich, and common plant and tree roots; contains multiple layers of geofabric; thicker mat fabric immediately above drain pipe, and $\frac{3}{4}$" square mesh at existing ground level; thins downslope; gradational, irregular contact.

2. A/B Soil Horizon: Up to 2' thick; dark yellowish brown (10YR $\frac{4}{2}$) to grayish brown (5Y $\frac{3}{2}$) sandy lean CLAY with gravel (CL), medium stiff, moist, low plasticity, massive; gravel and larger sized clasts comprise ~15-20% of the unit; clasts subrounded to subangular quartzite as above, up to 1' in diameter, though mode size is ~4-6"; abundant plant and tree roots; sharp, irregular basal contact.

3. Colluvium: ~2-2.5' thick; moderate yellowish brown (10YR $\frac{5}{4}$) to light brown (5YR $\frac{6}{4}$) to pale yellowish orange (10YR $\frac{8}{6}$) clayey SAND with gravel (SC), medium dense, moist, low plasticity fines, massive to faintly bedded; gravel and larger sized clasts comprise ~20-25% of the unit; clasts entirely quartzite as above, subangular to subrounded, up to 6" in diameter, though mode size is ~2"; minor silt component, common to abundant 1 mm diameter pinholes throughout; few plant and tree roots; sharp, irregular basal contact; possibly weathered Wasatch Formation (Tw).

4. Wasatch Formation: > 3' thick; dark yellowish orange (10YR $\frac{6}{6}$) to dark reddish brown (10R $\frac{3}{4}$), clayey SAND with gravel (SC), dense, moist, low to moderate plasticity fines, massive to faintly bedded; gravel and larger sized clasts comprise ~20-25% of the unit, all subrounded to subangular quartzite as above, up to 8" in diameter, though mode size is ~2-4"; occasional <1 mm diamter pinholes where clayey; minor silt component; becomes harder with depth.

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TEST PIT 2 LOG

FIGURE
A-4

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	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

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.

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

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

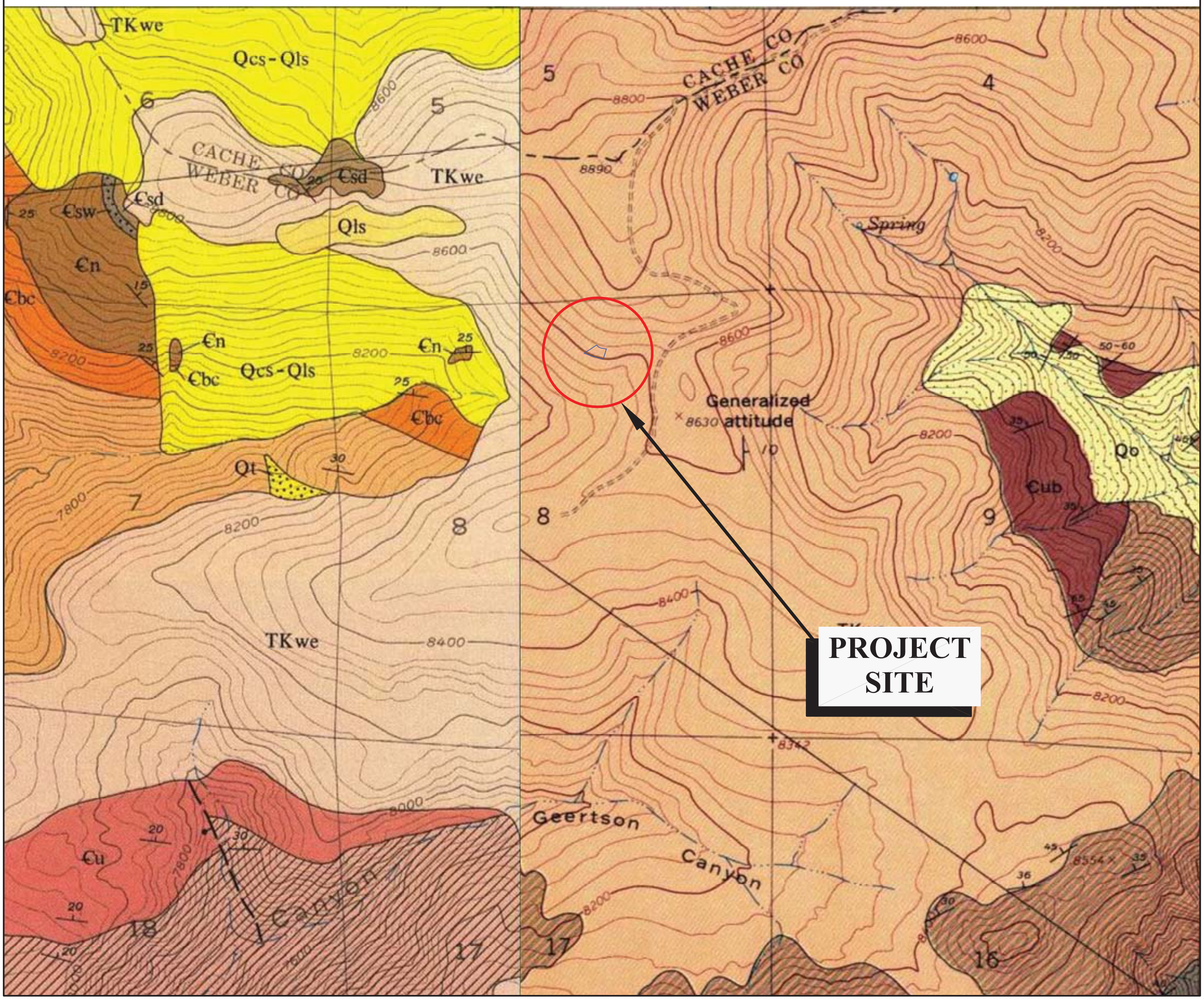
Splitting Property	Thickness	Stratification
Massive	>4.0 ft	Very thick bedded
Blocky	2.0-4.0 ft	Thick-bedded
Slabby	2 ½-24 in	Thin-bedded
Flaggy	½-2 ½ in	Very thin-bedded
Shaly or platy	⅛ – ½ in	Laminated
Papery	< ⅛ in	Thinly laminated

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 by 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



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)

N

0' 1,000' 2,000' 4,000'

SCALE: 1"=2,000'

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Lot 44R of Summit Eden Phase 1C
Summit Powder Mountain Resort
Weber County, Utah

REGIONAL GEOLOGY MAP 1

Figure

A-7a

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

Qf — **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

ST. CHARLES LIMESTONE (Upper Cambrian) – Includes:

€sd — 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

MAP LEGEND

€u

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

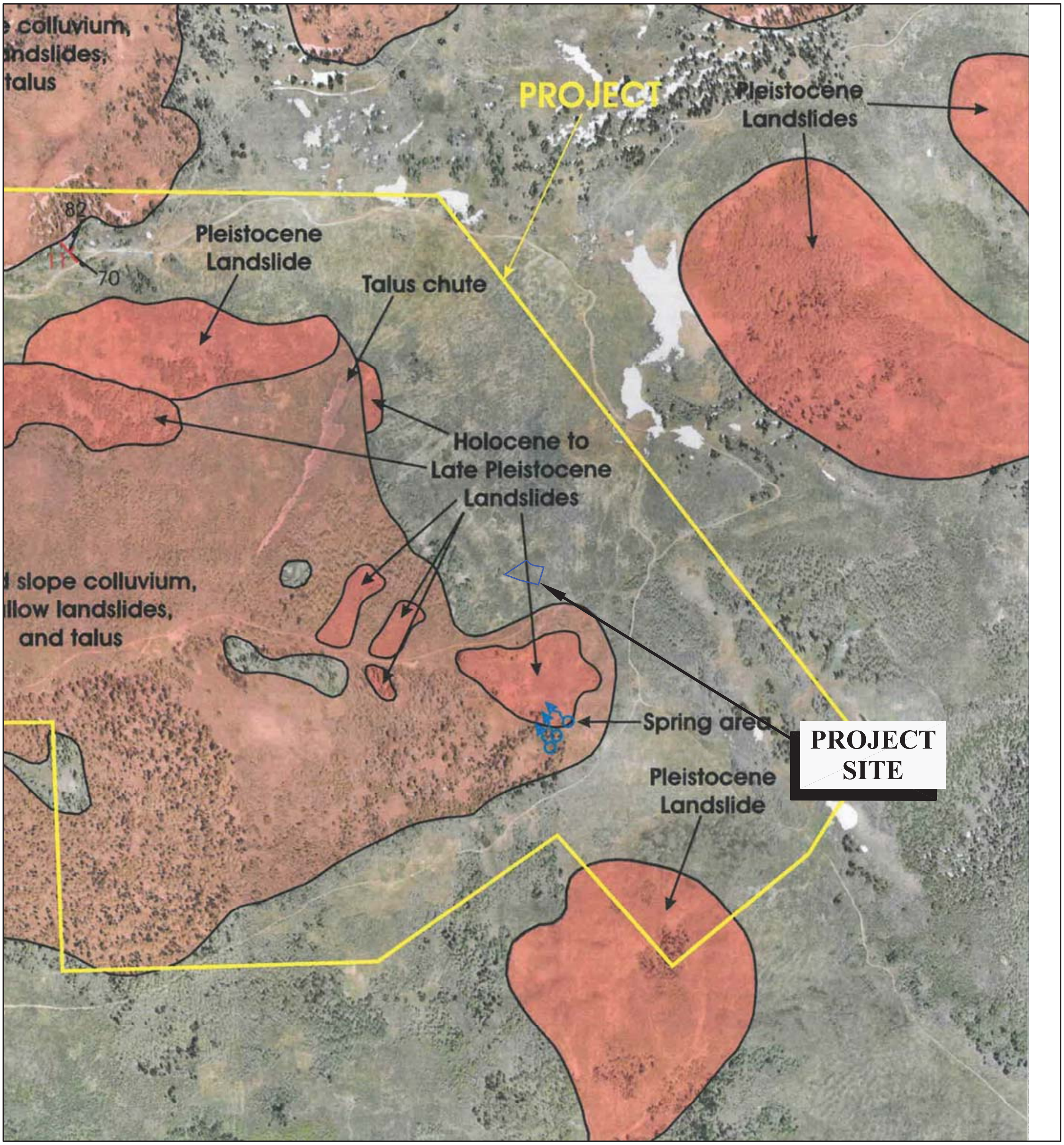
€gcl

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



Base Map:
Western Geologic (2012) Geologic Hazards
Reconnaissance Report, Figure 3

N

0' 500' 1,000' 2,000'

SCALE: 1"=1,000'

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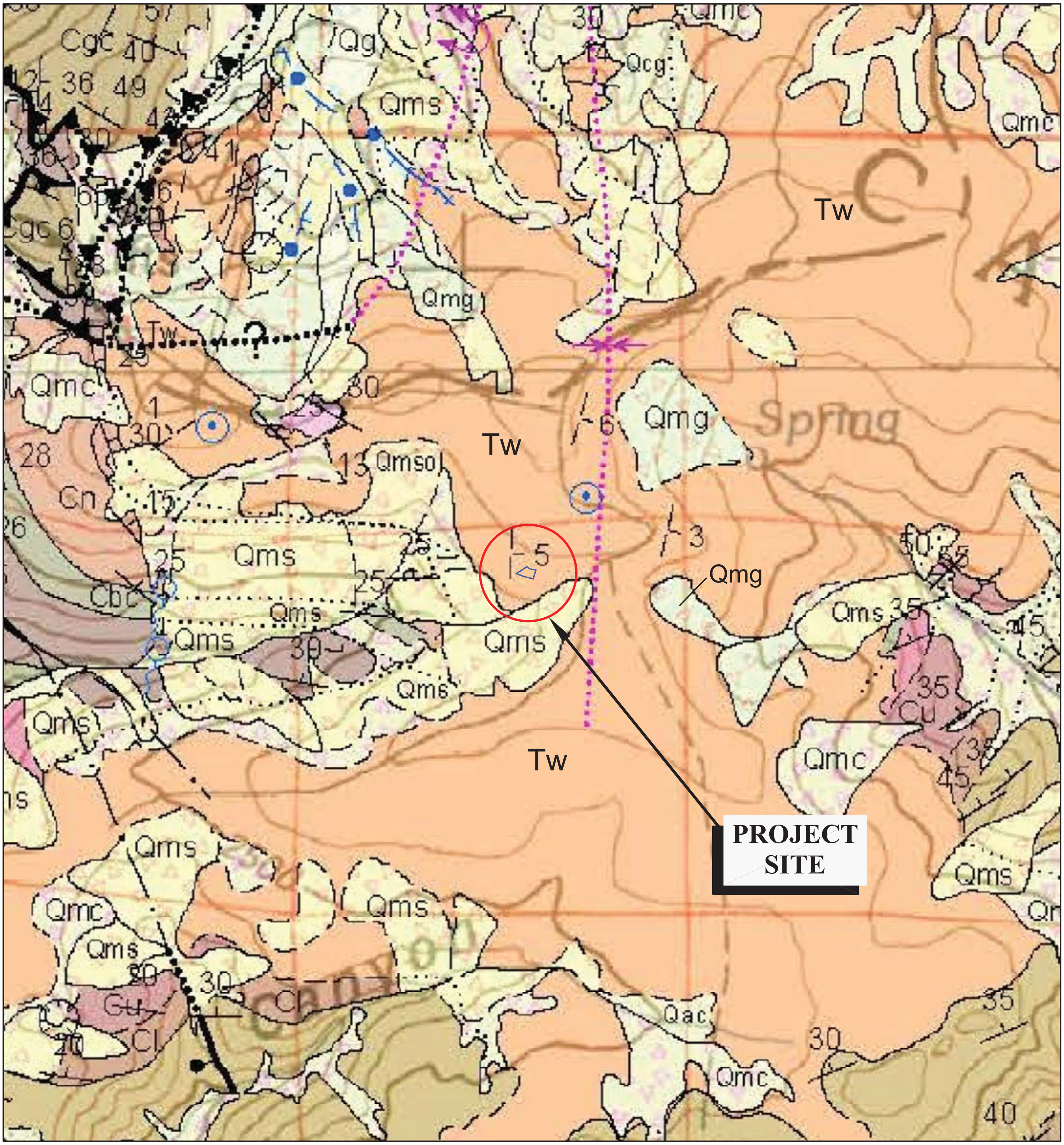
Summit Powder Mountain Resort
Weber County, Utah

REGIONAL GEOLOGY MAP 2

Figure

A-8



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

N

0' 1,000' 2,000' 4,000'

SCALE: 1"=2,000'

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Geotechnical & Geologic Hazard Investigation
Lot 44R of Summit Eden Phase 1C
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Weber County, Utah

REGIONAL GEOLOGY MAP 3

Figure

A-9a

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 Twl); 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

APPENDIX B

Water Content and Unit Weight of Soil

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

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Project: **Buttgenbach Lot 44**
No: **02732-001**
Location: Powder Mountain
Date: 2/20/2018
By: BRR

Sample Info.	Boring No.	TP-2	TP-2						
	Station	17	35						
	Depth	3.0'	3.0'						
	Split	Yes	Yes						
	Split sieve	3/8"	3/8"						
	Total sample (g)	3761.39	3150.89						
	Moist coarse fraction (g)	942.91	893.15						
	Moist split fraction (g)	2818.48	2257.74						
	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)	1205.27	1152.10						
	Dry soil + tare (g)	1183.77	1121.13						
	Tare (g)	214.18	215.41						
	Water content (%)	2.2	3.4						
Split Fraction	Wet soil + tare (g)	406.92	393.91						
	Dry soil + tare (g)	381.67	343.07						
	Tare (g)	152.33	127.67						
	Water content (%)	11.0	23.6						
	Water Content, w (%)	**8.7**	**17.1**						
	Dry Unit Wt., γ_d (pcf)								

Entered by:___________

Reviewed:___________

Liquid Limit, Plastic Limit, and Plasticity Index of Soils

(ASTM D4318)

IGES

Project: Buttgenbach Lot 44
No: 02732-001
Location: Powder Mountain
Date: 2/20/2018
By: BRR
Grooving tool type: Plastic
Liquid limit device: Mechanical
Rolling method: Hand

Boring No.: TP-2
Station: 35
Depth: 3.0'
Description: Dark brown silt

Preparation method: Air Dry
Liquid limit test method: Multipoint
Screened over No.40: Yes
Larger particles removed: Dry sieved
Approximate maximum grain size: 3/4"
Estimated percent retained on No.40: Not requested
As-received water content (%): 17.1

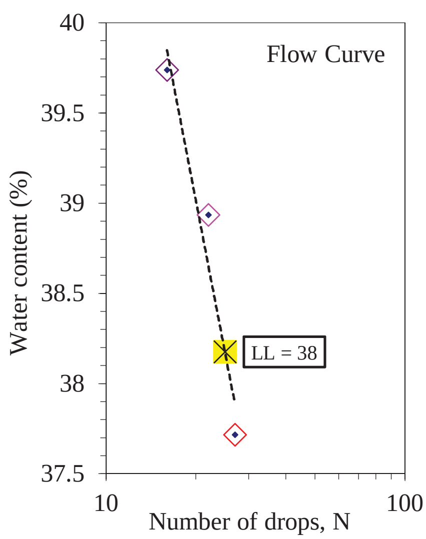
Plastic Limit

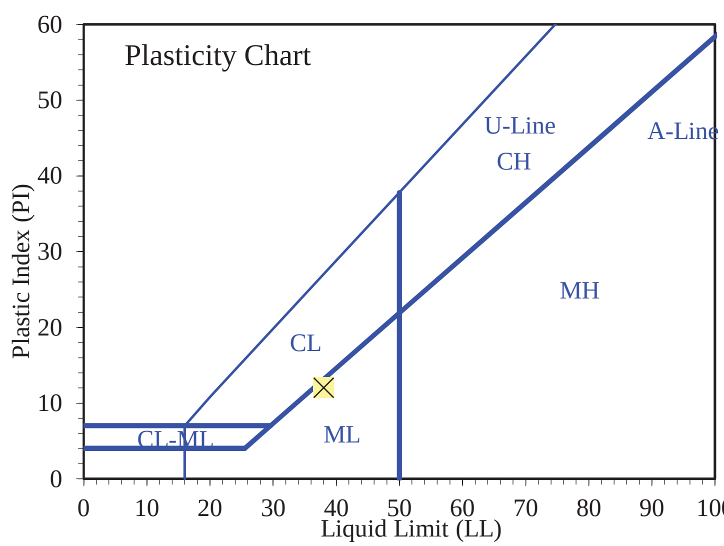
Determination No	1	2				
Wet Soil + Tare (g)	27.64	27.70				
Dry Soil + Tare (g)	26.36	26.37				
Water Loss (g)	1.28	1.33				
Tare (g)	21.48	21.31				
Dry Soil (g)	4.88	5.06				
Water Content, w (%)	26.23	26.28				

Liquid Limit

Determination No	1	2	3			
Number of Drops, N	27	22	16			
Wet Soil + Tare (g)	29.95	28.30	30.03			
Dry Soil + Tare (g)	27.77	26.40	27.61			
Water Loss (g)	2.18	1.90	2.42			
Tare (g)	21.99	21.52	21.52			
Dry Soil (g)	5.78	4.88	6.09			
Water Content, w (%)	37.72	38.93	39.74			
One-Point LL (%)	38	38				

Liquid Limit, LL (%)	**38**
Plastic Limit, PL (%)	**26**
Plasticity Index, PI (%)	**12**





Entered by:____________
Reviewed:____________

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

(ASTM D6913)

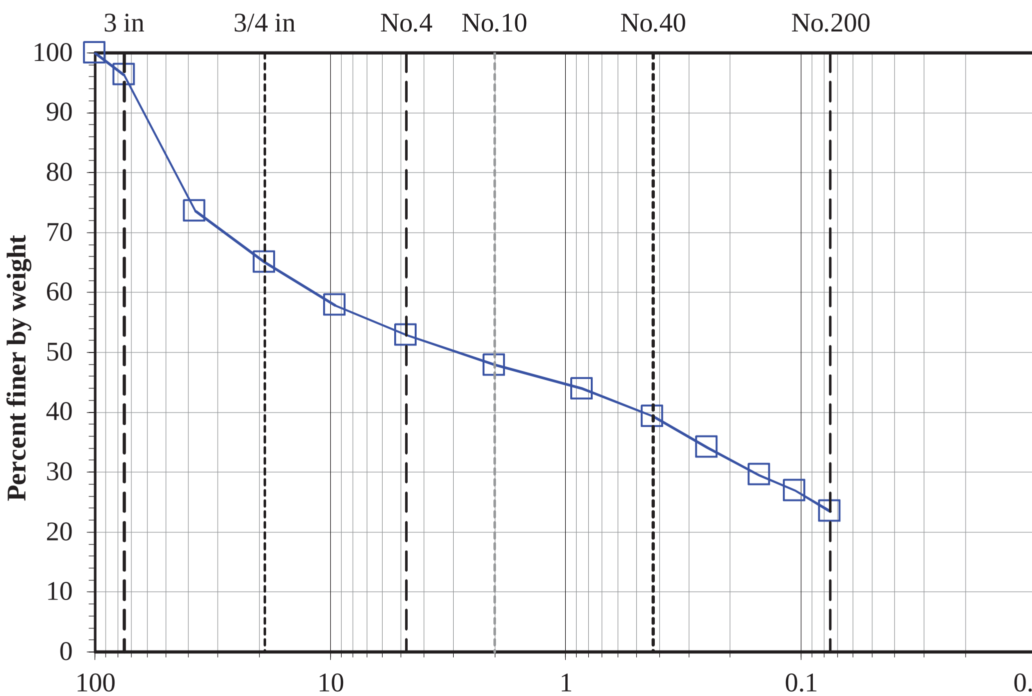
Project: Buttgenbach Lot 44
No: 02732-001
Location: Powder Mountain
Date: 2/16/2018
By: JWB

Boring No.: TP-1
Station: 10
Depth: 6.0'
Description: Red silty gravel with sand

	Moist	Dry
Split:	Yes	
Split sieve:	3/8"	
Total sample wt. (g):	27556.40	26201.83
+3/8" Coarse fraction (g):	11216.00	11046.50
-3/8" Split fraction (g):	238.68	221.37
Split fraction:	0.578	

Water content data	C.F.(+3/8")	S.F.(-3/8")
Moist soil + tare (g):	3761.62	356.64
Dry soil + tare (g):	3709.77	339.33
Tare (g):	330.74	117.96
Water content (%):	1.5	7.8

Sieve	Accum. Wt. Ret. (g)	Grain Size (mm)	Percent Finer	
8"	-	200	-	
6"	-	150	-	
4"	-	100	100.0	
3"	968.93	75	96.3	
1.5"	6918.93	37.5	73.6	
3/4"	9155.31	19	65.1	
3/8"	11046.50	9.5	57.8	←Split
No.4	19.13	4.75	52.8	
No.10	38.30	2	47.8	
No.20	53.42	0.85	43.9	
No.40	70.90	0.425	39.3	
No.60	90.79	0.25	34.1	
No.100	108.48	0.15	29.5	
No.140	118.50	0.106	26.9	
No.200	131.67	0.075	23.4	



Gravel (%): 47.2
Sand (%): 29.4
Fines (%): 23.4

Grain size (mm)

Entered by:___________
Reviewed:___________

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

(ASTM D6913)

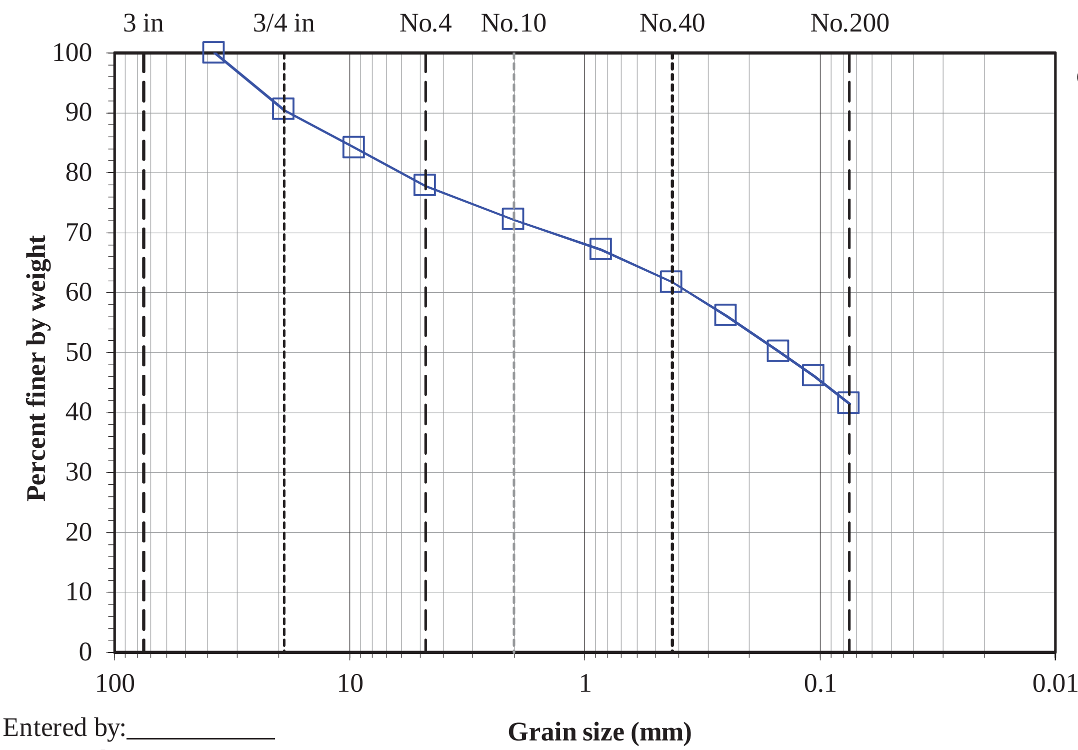
Project: Buttgenbach Lot 44
No: 02732-001
Location: Powder Mountain
Date: 2/19/2018
By: EH

Boring No.: TP-1
Station: 40
Depth: 3.0'
Description: Reddish brown clayey sand with gravel

		Moist	Dry
Split:	Yes		
Split sieve:	3/8"		
Total sample wt. (g):		3617.95	3419.22
+3/8" Coarse fraction (g):		545.95	542.41
-3/8" Split fraction (g):		243.79	228.30
Split fraction:		0.841	

Water content data	C.F.(+3/8")	S.F.(-3/8")
Moist soil + tare (g):	956.36	373.24
Dry soil + tare (g):	952.82	357.75
Tare (g):	410.41	129.45
Water content (%):	0.7	6.8

Sieve	Accum. Wt. Ret. (g)	Grain Size (mm)	Percent Finer	
8"	-	200	-	
6"	-	150	-	
4"	-	100	-	
3"	-	75	-	
1.5"	-	37.5	100.0	
3/4"	324.88	19	90.5	
3/8"	542.41	9.5	84.1	←Split
No.4	17.11	4.75	77.8	
No.10	32.55	2	72.1	
No.20	46.18	0.85	67.1	
No.40	60.82	0.425	61.7	
No.60	76.10	0.25	56.1	
No.100	92.15	0.15	50.2	
No.140	103.23	0.106	46.1	
No.200	115.80	0.075	41.5	



Gravel (%): 22.2
Sand (%): 36.4
Fines (%): 41.5

Entered by:___________
Reviewed:___________

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

(ASTM D6913)

IGES

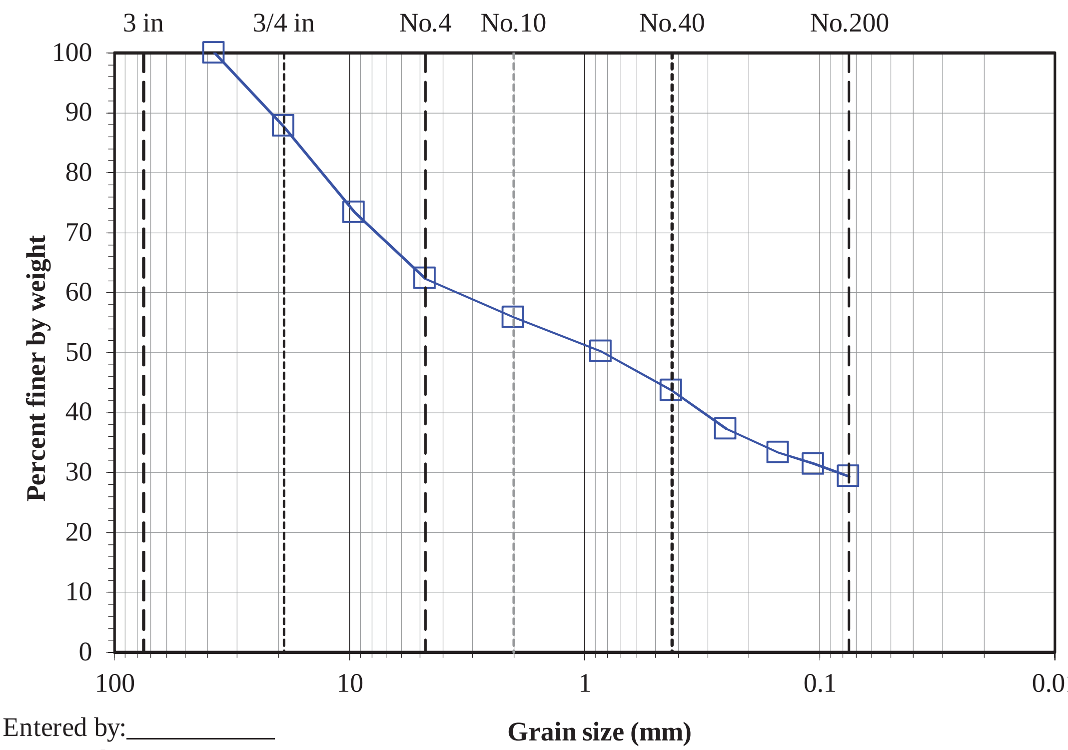
Project: Buttgenbach Lot 44
No: 02732-001
Location: Powder Mountain
Date: 2/20/2018
By: JWB

Boring No.: TP-2
Station: 17
Depth: 3.0'
Description: Brown clayey gravel with sand

Split:	Yes	
Split sieve:	3/8"	
	Moist	Dry
Total sample wt. (g):	3761.39	3461.40
+3/8" Coarse fraction (g):	942.91	922.46
-3/8" Split fraction (g):	254.59	229.34
Split fraction:	0.734	

Water content data	C.F.(+3/8")	S.F.(-3/8")
Moist soil + tare (g):	1205.27	406.92
Dry soil + tare (g):	1183.77	381.67
Tare (g):	214.18	152.33
Water content (%):	2.2	11.0

Sieve	Accum. Wt. Ret. (g)	Grain Size (mm)	Percent Finer	
8"	-	200	-	
6"	-	150	-	
4"	-	100	-	
3"	-	75	-	
1.5"	-	37.5	100.0	
3/4"	423.81	19	87.8	
3/8"	922.46	9.5	73.4	←Split
No.4	34.59	4.75	62.3	
No.10	54.66	2	55.9	
No.20	72.36	0.85	50.2	
No.40	92.77	0.425	43.7	
No.60	112.81	0.25	37.3	
No.100	125.20	0.15	33.3	
No.140	131.27	0.106	31.4	
No.200	137.77	0.075	29.3	



Gravel (%): 37.7
Sand (%): 33.0
Fines (%): 29.3

Entered by:___________
Reviewed:___________

Amount of Material in Soil Finer than the No. 200 (75μm) Sieve

(ASTM D1140)

Project: **Buttgenbach Lot 44**
No: **02732-001**
Location: Powder Mountain
Date: 2/20/2018
By: EH/DKS/BRR

Sample Info.	Boring No.	TP-1	TP-2	TP-2					
	Station	35	31	35					
	Depth	7.0'	9.0'	3.0'					
	Split	Yes	Yes	Yes					
	Split Sieve*	3/8"	3/8"	3/8"					
	Method	A	B	B					
	Specimen soak time (min)	210	200	200					
	Moist total sample wt. (g)	3956.80	20610.84	3150.89					
	Moist coarse fraction (g)	836.59	2030.64	893.15					
	Moist split fraction + tare (g)	367.22	382.27	393.91					
	Split fraction tare (g)	128.76	128.22	127.67					
	Dry split fraction (g)	215.60	228.72	215.40					
	Dry retained No. 200 + tare (g)	225.81	234.06	234.91					
	Wash tare (g)	128.76	128.22	127.67					
	No. 200 Dry wt. retained (g)	97.05	105.84	107.24					
	Split sieve* Dry wt. retained (g)	824.43	2008.48	863.62					
	Dry total sample wt. (g)	3645.52	18736.15	2690.23					
Coarse Fraction	Moist soil + tare (g)	957.56	2358.67	1152.10					
	Dry soil + tare (g)	945.40	2336.51	1121.13					
	Tare (g)	120.97	328.03	215.41					
	Water content (%)	1.47	1.10	3.42					
Split Fraction	Moist soil + tare (g)	367.22	382.27	393.91					
	Dry soil + tare (g)	344.36	356.94	343.07					
	Tare (g)	128.76	128.22	127.67					
	Water content (%)	10.60	11.07	23.60					
	Percent passing split sieve* (%)	**77.4**	**89.3**	**67.9**					
	Percent passing No. 200 sieve (%)	**42.6**	**48.0**	**34.1**					

Entered by:___________

Reviewed:___________

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Direct Shear Test for Soils Under Drained Conditions

(ASTM D3080)

Project: Buttgenbach Lot 44
No: 02732-001
Location: Powder Mountain
Date: 2/19/2018
By: EH

Boring No.: TP-1
Station: 40
Depth: 3.0'
Sample Description: Reddish brown clayey sand
Sample type: Arbitrary remold

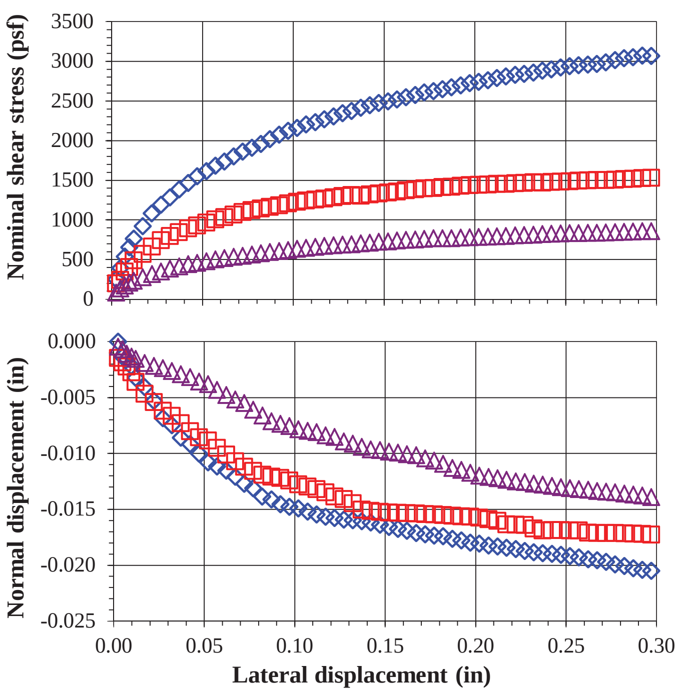
Test type: Inundated
Lateral displacement (in.): 0.3
Shear rate (in./min): 0.0033
Specific gravity, Gs: 2.65 Assumed

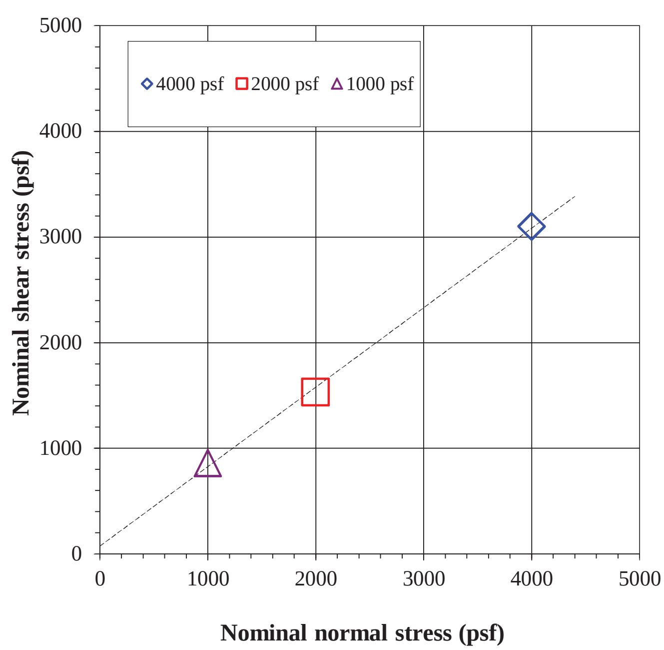
	Sample 1		Sample 2		Sample 3	
Nominal normal stress (psf)	4000		2000		1000	
Peak shear stress (psf)	3100		1530		860	
Lateral displacement at peak (in)	0.300		0.297		0.300	
Load Duration (min)	64		66		80	
	Initial	Pre-shear	Initial	Pre-shear	Initial	Pre-shear
Sample height (in)	0.994	0.929	0.998	0.964	1.003	0.985
Sample diameter (in)	2.419	2.419	2.418	2.418	2.417	2.417
Wt. rings + wet soil (g)	196.27	198.22	199.03	203.36	198.60	204.23
Wt. rings (g)	44.33	44.33	46.52	46.52	45.64	45.64
Wet soil + tare (g)	266.52		266.52		266.52	
Dry soil + tare (g)	253.87		253.87		253.87	
Tare (g)	154.02		154.02		154.02	
Water content (%)	12.7	14.1	12.7	15.9	12.7	16.8
Dry unit weight (pcf)	112.5	120.3	112.5	116.4	112.4	114.4
Void ratio, e, for assumed Gs	0.47	0.37	0.47	0.42	0.47	0.45
Saturation (%)*	71.3	100.0	71.4	100.0	71.1	100.0

φ' (deg)	37
c' (psf)	75

Average of 3 samples	Initial	Pre-shear
Water content (%)	12.7	15.6
Dry unit weight (pcf)	112.5	117.0

*Pre-shear saturation set to 100% for phase calculations





Entered by:___________
Reviewed:___________

Direct Shear Test for Soils Under Drained Conditions

(ASTM D3080)

Project: Buttgenbach Lot 44
No: 02732-001
Location: Powder Mountain

Boring No.: TP-1
Station: 40
Depth: 3.0'

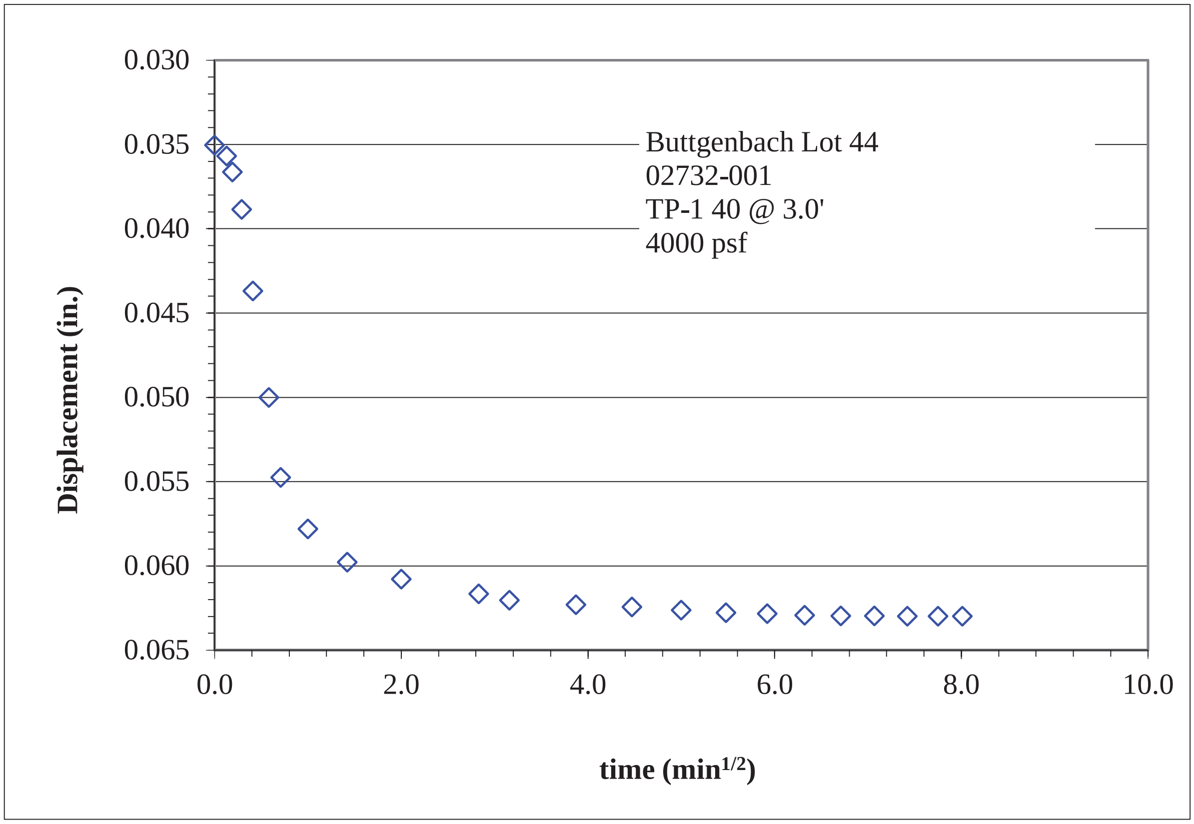
Nominal normal stress = 4000 psf			Nominal normal stress = 2000 psf			Nominal normal stress = 1000 psf		
Lateral Displacement (in.)	Nominal Shear Stress (psf)	Normal Displacement (in.)	Lateral Displacement (in.)	Nominal Shear Stress (psf)	Normal Displacement (in.)	Lateral Displacement (in.)	Nominal Shear Stress (psf)	Normal Displacement (in.)
0.002	251	0.000	0.002	200	-0.001	0.002	81	0.000
0.005	391	-0.001	0.005	245	-0.002	0.005	133	-0.001
0.007	539	-0.002	0.007	348	-0.002	0.007	172	-0.001
0.010	655	-0.002	0.010	405	-0.003	0.010	206	-0.001
0.012	764	-0.003	0.012	487	-0.004	0.012	231	-0.002
0.017	922	-0.004	0.017	569	-0.005	0.017	276	-0.002
0.022	1082	-0.005	0.022	663	-0.005	0.022	317	-0.002
0.027	1191	-0.007	0.027	742	-0.006	0.027	349	-0.002
0.032	1284	-0.007	0.032	794	-0.007	0.032	384	-0.003
0.037	1385	-0.009	0.037	844	-0.007	0.037	414	-0.003
0.042	1466	-0.009	0.042	893	-0.008	0.042	440	-0.003
0.047	1549	-0.010	0.047	931	-0.009	0.047	459	-0.004
0.052	1616	-0.011	0.052	967	-0.009	0.052	479	-0.004
0.057	1683	-0.011	0.057	1005	-0.010	0.057	501	-0.004
0.062	1735	-0.012	0.062	1035	-0.010	0.062	519	-0.005
0.067	1802	-0.012	0.067	1066	-0.011	0.067	534	-0.005
0.072	1862	-0.013	0.072	1100	-0.011	0.072	548	-0.005
0.077	1911	-0.013	0.077	1122	-0.012	0.077	562	-0.006
0.082	1960	-0.014	0.082	1145	-0.012	0.082	578	-0.007
0.087	2020	-0.014	0.087	1165	-0.012	0.087	593	-0.007
0.092	2074	-0.015	0.092	1183	-0.012	0.092	607	-0.007
0.097	2124	-0.015	0.097	1208	-0.012	0.097	619	-0.008
0.102	2162	-0.015	0.102	1229	-0.013	0.102	633	-0.008
0.107	2204	-0.015	0.107	1244	-0.013	0.107	646	-0.008
0.112	2232	-0.015	0.112	1255	-0.013	0.112	657	-0.008
0.117	2269	-0.016	0.117	1269	-0.013	0.117	671	-0.008
0.122	2307	-0.016	0.122	1286	-0.014	0.122	679	-0.009
0.127	2344	-0.016	0.127	1299	-0.014	0.127	686	-0.009
0.132	2372	-0.016	0.132	1312	-0.014	0.132	692	-0.009
0.137	2414	-0.016	0.137	1303	-0.015	0.137	703	-0.009
0.142	2447	-0.016	0.142	1322	-0.015	0.142	710	-0.010
0.147	2473	-0.016	0.147	1334	-0.015	0.147	722	-0.010
0.152	2496	-0.017	0.152	1345	-0.015	0.152	729	-0.010
0.157	2520	-0.017	0.157	1358	-0.015	0.157	740	-0.010
0.162	2548	-0.017	0.162	1373	-0.015	0.162	747	-0.010
0.167	2577	-0.017	0.167	1386	-0.015	0.167	753	-0.010
0.172	2608	-0.017	0.172	1396	-0.015	0.172	761	-0.010
0.177	2626	-0.017	0.177	1403	-0.015	0.177	766	-0.011
0.182	2647	-0.017	0.182	1412	-0.016	0.182	768	-0.011
0.187	2673	-0.018	0.187	1418	-0.016	0.187	768	-0.011
0.192	2698	-0.018	0.192	1428	-0.016	0.192	775	-0.011
0.197	2727	-0.018	0.197	1436	-0.016	0.197	780	-0.012
0.202	2740	-0.018	0.202	1441	-0.016	0.202	782	-0.012
0.207	2761	-0.018	0.207	1446	-0.016	0.207	786	-0.012
0.212	2789	-0.018	0.212	1455	-0.016	0.212	792	-0.012
0.217	2810	-0.018	0.217	1455	-0.016	0.217	798	-0.012
0.222	2828	-0.019	0.222	1463	-0.016	0.222	802	-0.013
0.227	2843	-0.019	0.227	1464	-0.016	0.227	807	-0.013
0.232	2859	-0.019	0.232	1473	-0.017	0.232	814	-0.013
0.237	2882	-0.019	0.237	1470	-0.017	0.237	820	-0.013
0.242	2900	-0.019	0.242	1480	-0.017	0.242	824	-0.013
0.247	2924	-0.019	0.247	1482	-0.017	0.247	828	-0.013
0.252	2939	-0.019	0.252	1486	-0.017	0.252	831	-0.013
0.257	2952	-0.019	0.257	1495	-0.017	0.257	834	-0.013
0.262	2960	-0.019	0.262	1500	-0.017	0.262	836	-0.013
0.267	2968	-0.020	0.267	1503	-0.017	0.267	836	-0.013
0.272	2988	-0.020	0.272	1502	-0.017	0.272	841	-0.013
0.277	3014	-0.020	0.277	1507	-0.017	0.277	844	-0.013
0.282	3038	-0.020	0.282	1515	-0.017	0.282	848	-0.014
0.287	3053	-0.020	0.287	1519	-0.017	0.287	853	-0.014
0.292	3074	-0.020	0.292	1525	-0.017	0.292	855	-0.014
0.297	3069	-0.021	0.297	1530	-0.017	0.297	857	-0.014
0.300	3100	-0.021	0.300	1525	-0.017	0.300	860	-0.014

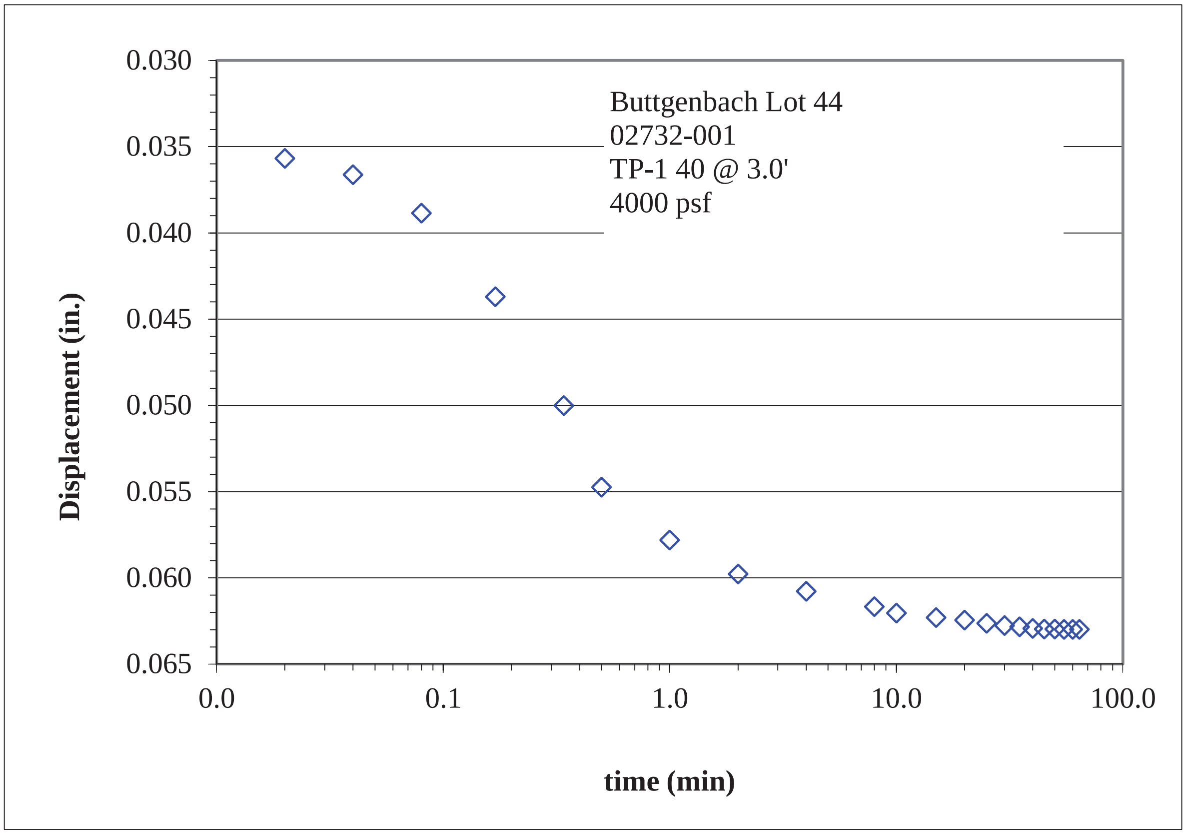
Direct Shear Test for Soils Under Drained Conditions

(ASTM D3080)

Project: Buttgenbach Lot 44
No: 02732-001
Location: Powder Mountain

Boring No.: TP-1
Station: 40
Depth: 3.0'





Minimum Laboratory Soil Resistivity, pH of Soil for Use in Corrosion Testing, and Ions in Water by Chemically Suppressed Ion Chromatography *(AASHTO T 288, T 289, ASTM D4327, and C1580)*

IGES

Project: Buttgenbach Lot 44
No: 02732-001
Location: Powder Mountain
Date: 3/1/2018
By: BSS

Section	Parameter	Sample 1	Sample 2
Sample info.	Boring No.	TP-2	
	Sample		
	Depth	9.0'	
Water content data	Wet soil + tare (g)	144.41	
	Dry soil + tare (g)	134.32	
	Tare (g)	37.95	
	Water content (%)	10.5	
Chem. data	pH	5.57	
	Soluble chloride* (ppm)	72.0	
	Soluble sulfate** (ppm)	516	
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	53480	0.67	35832				
+3	24660	0.67	16522				
+6	16500	0.67	11055				
+9	13990	0.67	9373				
+12	14240	0.67	9541				

	Sample 1	Sample 2
Minimum resistivity (Ω-cm)	**9373**	

* Performed by AWAL using EPA 300.0

** Performed by AWAL using ASTM C1580

Entered by:___________

Reviewed:___________

APPENDIX C

USGS Design Maps Summary Report

User–Specified Input

Report Title Lot 44R
Mon March 5, 2018 23:49:06 UTC

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

Site Coordinates 41.36331°N, 111.7473°W

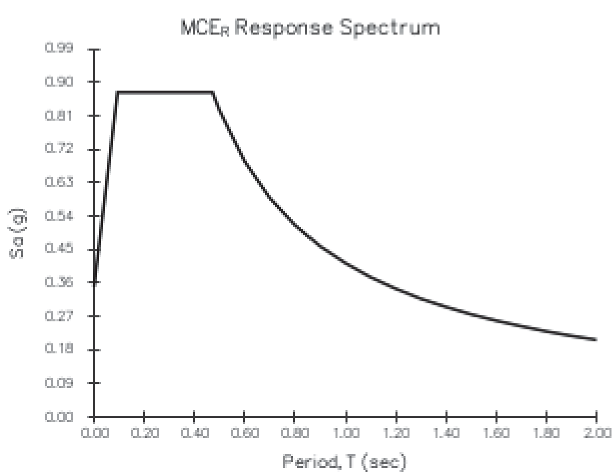
Site Soil Classification Site Class C – "Very Dense Soil and Soft Rock"

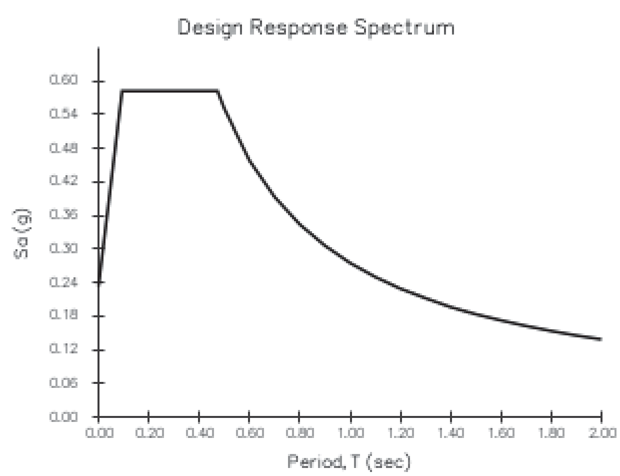
Risk Category I/II/III

USGS–Provided Output

S_S =	0.813 g	S_{MS} =	0.874 g	S_{DS} =	0.582 g
S_1 =	0.270 g	S_{M1} =	0.413 g	S_{D1} =	0.275 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.





USGS Design Maps Detailed Report

2012/2015 International Building Code (41.36331°N, 111.7473°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 = 0.813 g
From Figure 1613.3.1(2) [2]	S_1 = 0.270 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: • Plasticity index *PI* > 20, • Moisture content *w* ≥ 40%, and • Undrained shear strength $\bar{s}_u$ < 500 psf		
F. Soils requiring site response analysis in accordance with Section 21.1	See Section 20.3.1		

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

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

TABLE 1613.3.3(1)
VALUES OF SITE COEFFICIENT F_a

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

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

For Site Class = C and S_S = 0.813 g, F_a = 1.075

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 = 0.270 g, F_v = 1.530

Equation (16-37):	$S_{MS} = F_aS_S = 1.075 \times 0.813 = 0.874$ g
Equation (16-38):	$S_{M1} = F_vS_1 = 1.530 \times 0.270 = 0.413$ g

Section 1613.3.4 — Design spectral response acceleration parameters

Equation (16-39):	$S_{DS} = ⅔\ S_{MS} = ⅔ \times 0.874 = 0.582$ g
Equation (16-40):	$S_{D1} = ⅔\ S_{M1} = ⅔ \times 0.413 = 0.275$ 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} = 0.582 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
$0.067g \le S_{D1} < 0.133g$	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} = 0.275 g, Seismic Design Category = D

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

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

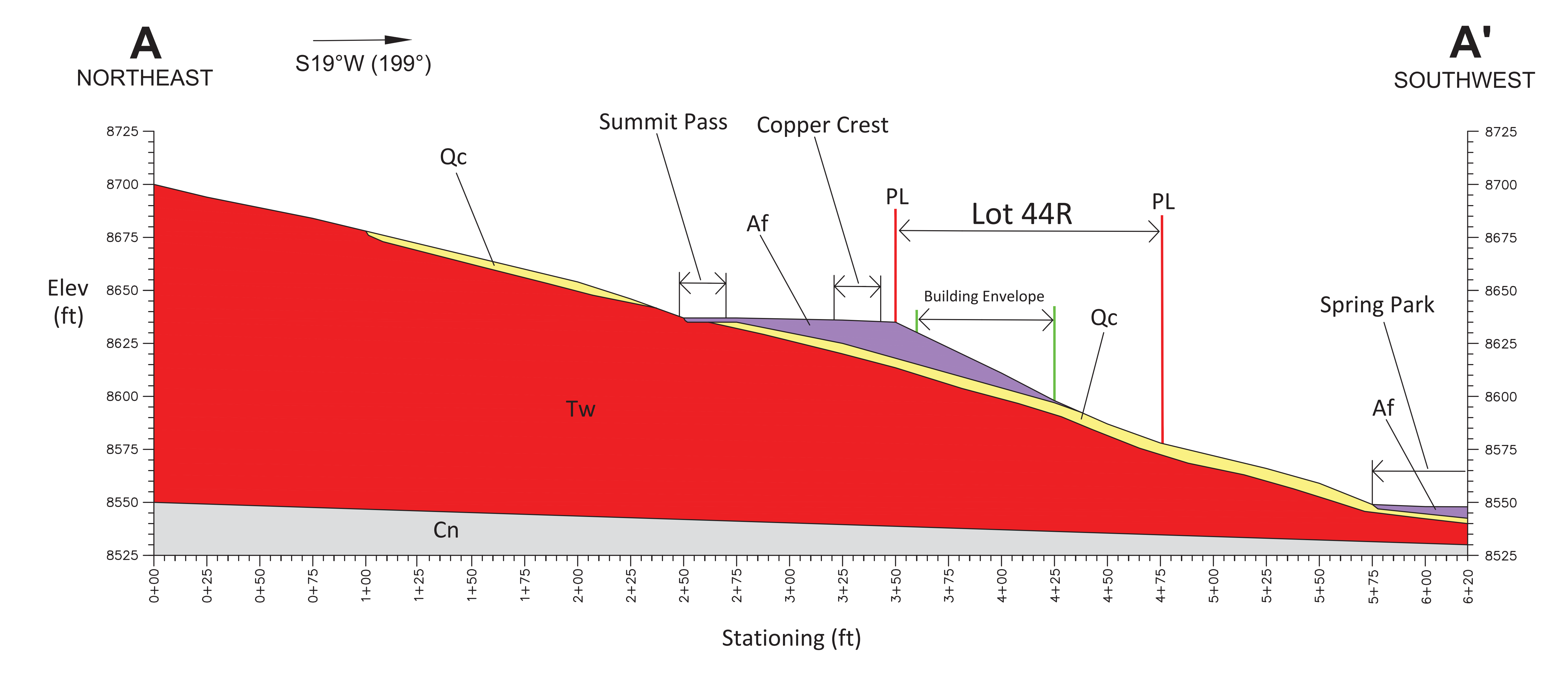
Note: See Section 1613.3.5.1 for alternative approaches to calculating Seismic Design Category.

References

1. *Figure 1613.3.1(1)*: 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

APPENDIX D

CROSS-SECTION A - A'



LEGEND

Af = Artificial Fill

Qc = Colluvium

Tw = Wasatch Formation

Cn = Nounan Dolomite

NO VERTICAL EXAGGERATION

VIEW EAST

0 25 50

FEET

1" = 50' (H&V) (11" x 17" Only)

FIGURE D-1

CROSS-SECTION A - A'

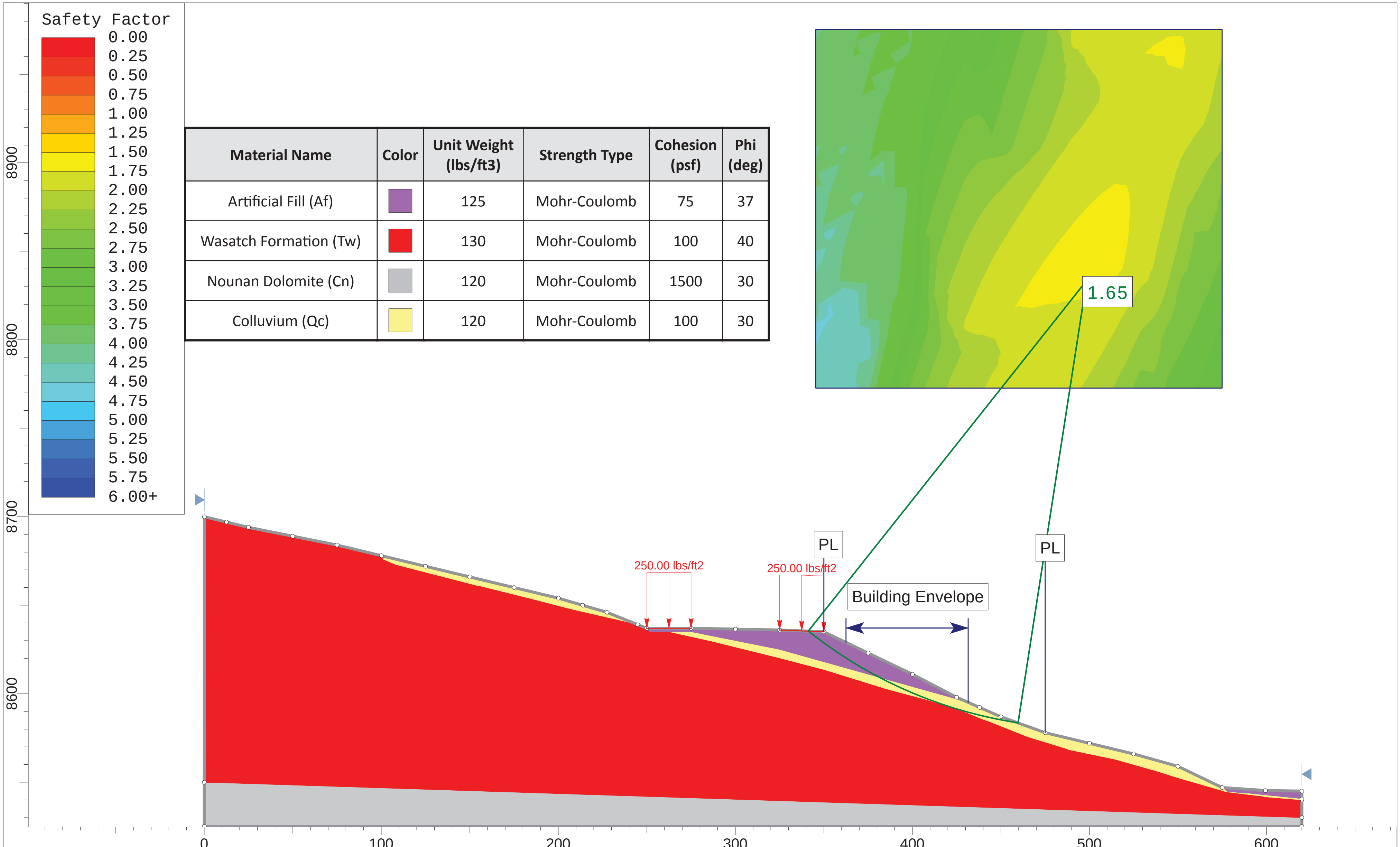
GEOTECHNICAL AND GEOLOGIC HAZARDS INVESTIGATION
LOT 44R OF THE SUMMIT EDEN PHASE 1C
POWDER MOUNTAIN RESORT
WEBER COUNTY, UTAH

DATE: 3/12/2018

PROJECT:02732-001

SCALE: 1"=50'

IGES

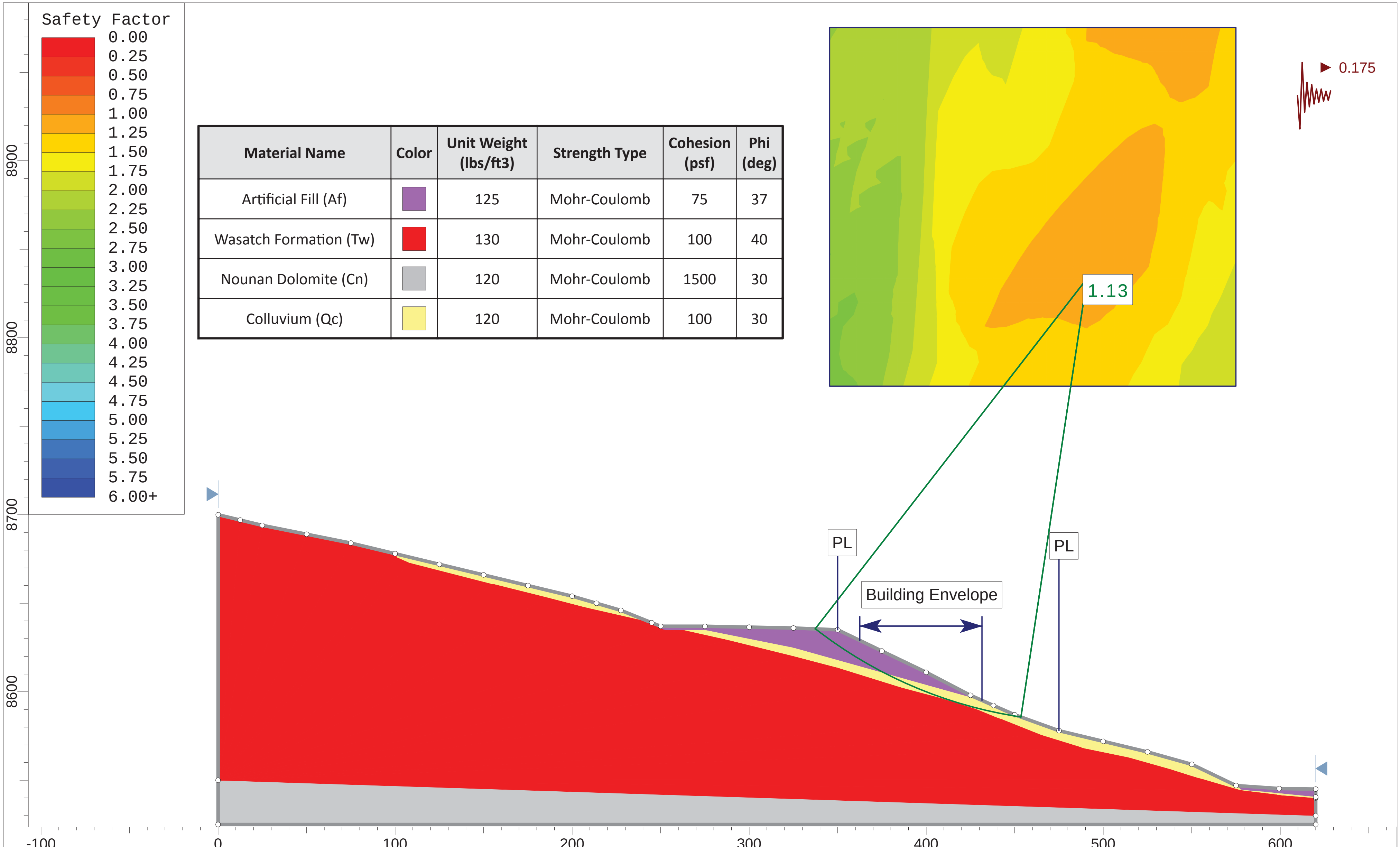


Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Artificial Fill (Af)		125	Mohr-Coulomb	75	37
Wasatch Formation (Tw)		130	Mohr-Coulomb	100	40
Nounan Dolomite (Cn)		120	Mohr-Coulomb	1500	30
Colluvium (Qc)		120	Mohr-Coulomb	100	30

IGES®

SLIDEINTERPRET 7.029

Project	Lot 14R		
Analysis Description	Slope Stability		
Drawn By EBF	*Scale* 1:900	*Company*	IGES Inc.
Date	3/6/2018, 7:57:47 AM	*File Name*	02732-001 Xsec A-A'.slim



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Artificial Fill (Af)		125	Mohr-Coulomb	75	37
Wasatch Formation (Tw)		130	Mohr-Coulomb	100	40
Nounan Dolomite (Cn)		120	Mohr-Coulomb	1500	30
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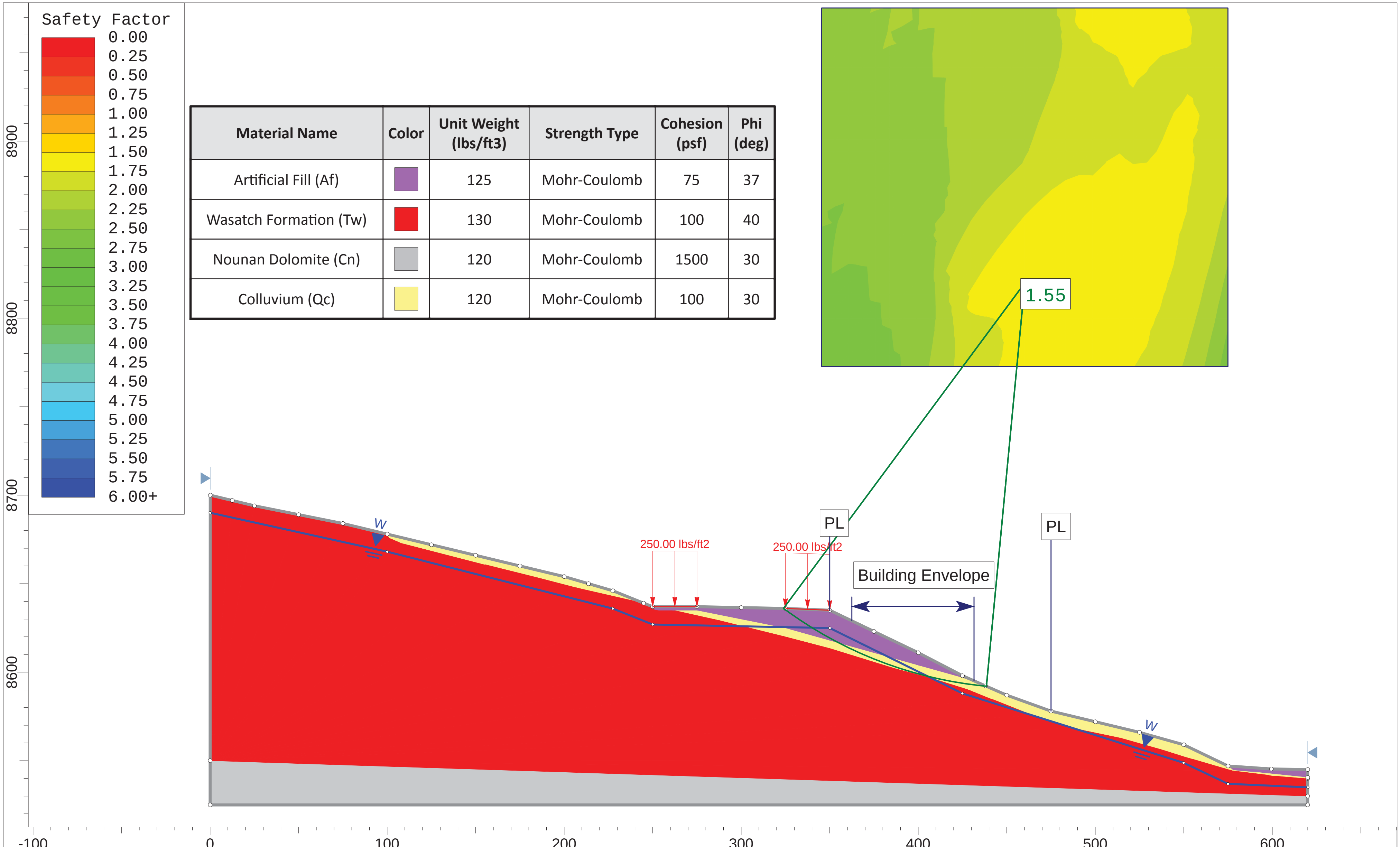
SLIDEINTERPRET 7.029

Project	Lot 14R		
Analysis Description	Slope Stability		
Drawn By	EBF	*Scale* 1:900	*Company* IGES Inc.
Date	3/6/2018, 7:57:47 AM		*File Name* 02732-001 Xsec A-A'.slim



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Artificial Fill (Af)		125	Mohr-Coulomb	75	37
Wasatch Formation (Tw)		130	Mohr-Coulomb	100	40
Nounan Dolomite (Cn)		120	Mohr-Coulomb	1500	30
Colluvium (Qc)		120	Mohr-Coulomb	100	30

IGES®

SLIDEINTERPRET 7.029

Project	Lot 14R		
Analysis Description	Slope Stability		
Drawn By EBF	*Scale* 1:900	*Company*	IGES Inc.
Date	3/6/2018, 7:57:47 AM	*File Name*	02732-001 Xsec A-A'.slim

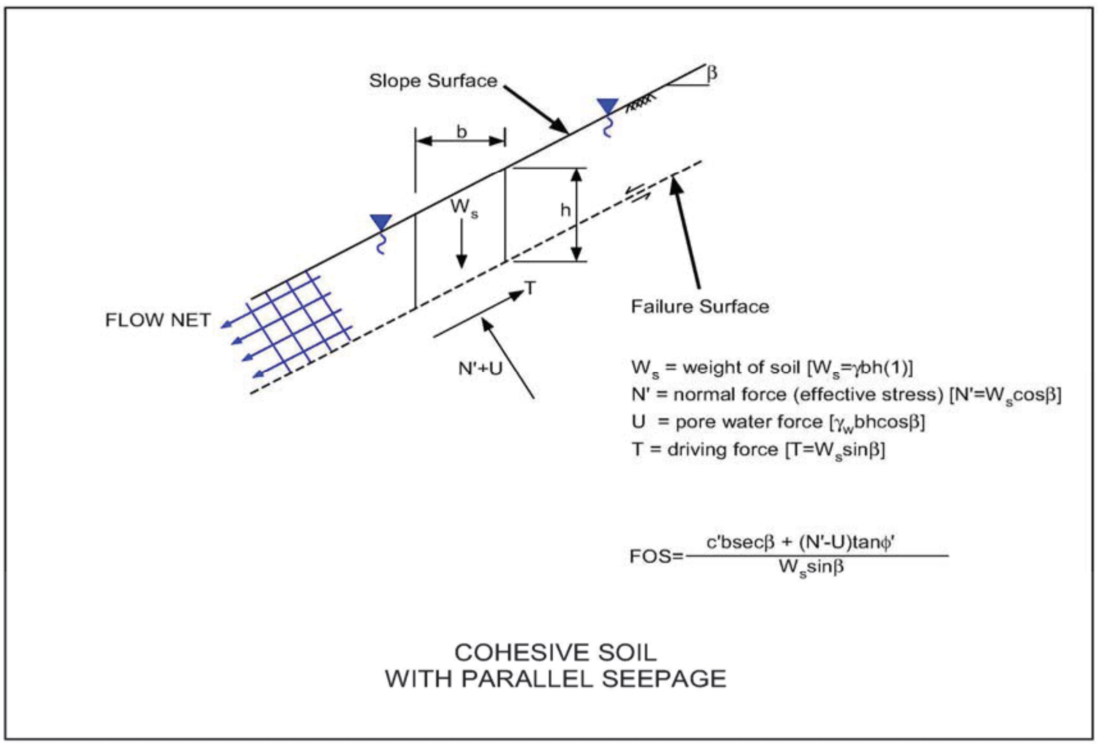
Buttgenback/Lot 44R
02732-001
3/13/2018

c'	75	psf	Effective Cohesion
ϕ'	37	deg	Effective Friction Angle
Y_{sat}	135	pcf	Saturated Unit Weight of Soil
Y_w	62.4	pcf	Unit weight of water
h	2	ft	Depth to shear surface
β	26.6	deg	Slope Gradient (2H:1V)

FS	1.50

(yellow)	Input Variable
(green)	Calculated Value

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



Buttgenback/Lot 44R
02732-001
3/13/2018

c'	75	psf	Effective Cohesion
ϕ'	37	deg	Effective Friction Angle
Y_{sat}	135	pcf	Saturated Unit Weight of Soil
Y_w	62.4	pcf	Unit weight of water
h	3	ft	Depth to shear surface
β	26.6	deg	Slope Gradient (2H:1V)
FS	1.27		
		Input Variable	
		Calculated Value	

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