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GEOTECHNICAL AND GEOLOGIC HAZARD INVESTIGATION
Lot 16 of Summit Eden Phase 1A
Summit Powder Mountain Resort
Weber County, Utah

IGES Project No. 02529-001

August 8, 2017

Prepared for:

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1.0 INTRODUCTION

1.1 PURPOSE AND SCOPE OF WORK

This report presents the results of a geotechnical and geologic hazard investigation conducted for Lot 16R of Summit Eden Phase 1A, 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, subsurface exploration, engineering analyses, and preparation of this report.

Our services were performed in accordance with our proposal dated April 20, 2017, and your signed authorization. The recommendations presented in this report are subject to the limitations presented in the "Limitations" section of this report (Section 6.1).

1.2 PROJECT DESCRIPTION

Our understanding of the project is based primarily on the floor plans prepared by MacKay-Lyons Sweetapple Architects (MLS) dated May 19, 2017, plus 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) and subsequent geotechnical consulting for several other aspects of the project.

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 16R is located within Phase 1A of the Powder Mountain expansion project (Summit Eden), on the south side of Horizon Run Road. The roughly 2-acre residential lot has an approximate buildable area (building envelope) of 12,600 square feet. The proposed improvements will include a single-family home with a structural footprint of approximately 5,200 sqft, with associated improvements such as utilities and hardscape. Based on a review of the plans provided by MLS, the new home will be a three-level structure, the lowest story consisting of a partial walk-out basement, founded on conventional spread footings. Foundation loads are expected to be on the order of 1,500 psf or less. The development will also include a rockery and assorted concrete retaining walls.

2.0 METHODS OF STUDY

2.1 LITERATURE REVIEW

2.1.1 Geotechnical

The earliest geotechnical report for the area is by AMEC (2001), which was a reconnaissance-level geotechnical and geologic hazard study. IGES later completed a geotechnical investigation for the Powder Mountain Resort expansion in 2012 (2012a, 2012b). Our previous work included twenty-two test pits and one soil boring excavated at various locations across the 200-acre development; as a part of this current study, the logs from relevant nearby test pits and other data from our reports were reviewed.

2.1.2 Geological

Several pertinent publications were reviewed as part of this assessment. Sorensen and Crittenden, Jr. (1979) provides 1:24,000 scale geologic mapping of the Huntsville Quadrangle, and Crittenden, Jr. (1972) provides 1:24,000 scale geologic mapping of the Brown's Hole Quadrangle. Coogan and King (2001) provide more recent geologic mapping of the area, but at a 1:100,000 scale. An updated Coogan and King (2016) regional geologic map (1:62,500 scale) provides the most recent published geologic mapping that covers the project area. Western Geologic (2012) conducted a reconnaissance-level geologic hazard study for the greater 200-acre Powder Mountain expansion project, including the Lot 16R 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 across the property. The approximate location of the test pits are 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, Figures A-3 and A-4 in Appendix A. A key to USCS symbols and terminology is included as Figure A-5, and a key to physical rock properties is included as Figure A-6.

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 & Dry Density (ASTM D2216 & 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 16R property is situated in the western portion of the northern Wasatch Mountains, approximately 4 miles north of Ogden Valley. The Wasatch Mountains contain a broad depositional history of thick Precambrian and Paleozoic sediments that have been subsequently modified by various tectonic episodes that have included thrusting, folding, intrusion, and volcanics, as well as scouring by glacial and fluvial processes (Stokes, 1987). The uplift of the Wasatch Mountains occurred relatively recently during the Late Tertiary Period (Miocene Epoch) between 12 and 17 million years ago (Milligan, 2000). Since uplift, the Wasatch Front has seen substantial modification due to such occurrences as movement along the Wasatch Fault and associated spurs, the development of the numerous canyons that empty into the current Salt Lake Valley and Utah Valley and their associated alluvial fans, erosion and deposition from Lake Bonneville, and localized mass movement events (Hintze, 1988).

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

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

The WFZ consists of a series of ten segments of the Wasatch Fault that each display different characteristics and past movement, and are believed to have movement independent of one another (UGS, 1996). The Lot 16R property is located approximately 9.5 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 almost entirely underlain by undifferentiated Holocene-aged colluvium, slopewash, and landslide deposits, with the northernmost portion of the property mapped as being 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.” 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 approximately northern half of the property to be underlain by undivided mass-movement deposits, with the approximately southern half of the property to be underlain by the Wasatch Formation. Western Geologic (2012) identified a number of landslide deposits contained within the Powder Mountain Resort expansion area (Figure A-8). In this map, the area denoted as Wasatch Formation by Coogan and King (2001) was reinterpreted to be Holocene to Late Pleistocene Landslides, with the northern half of the property underlain by deposits mapped as “mixed slope colluvium, shallow landslides, and talus.” Finally, Coogan and King (2016) updated their 2001 map, which shows the property to straddle the contact between landslide deposits (unit Qms) and the Wasatch Formation (unit Tw) (Figure A-9).

3.3 HYDROLOGY

The USGS topographic maps for the Huntsville and Brown’s Hole Quadrangles (2014) show that the Lot 16R project area is situated on a slope, with the topographic gradient down to the south towards a west-trending unnamed drainage locally known as Lefty’s Canyon (see Figure A-1). No active or ephemeral stream drainages are found on the property, and no springs are known to occur on the property, though it is possible that springs may occur on various parts of the property during peak runoff.

Baseline groundwater depths for the Lot 16R property are currently unknown, but are anticipated to fluctuate both seasonally and annually. Groundwater was not encountered in the two 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 16R 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) shows the property to be underlain by south-trending landslide deposits. Elliott and Harty (2010) shows the southern half of the property to be underlain by deposits mapped as “Landslide and/or landslide undifferentiated from talus, colluvial, rock-fall, glacial, and soil-creep deposits,” and the northern half of the property to be underlain by deposits mapped as “Landslide undifferentiated from talus and/or colluvial deposits.” On a site-specific basis, Western Geologic

(2012) used the Elliott and Harty (2010) map as a base map, which reinterprets the southern half deposits to be definitive landslide deposits (see Figure A-8). As noted above, most recently Coogan and King (2016) on a regional scale show the property to straddle the contact between Wasatch Formation (to the east) and landslide deposits (to the west; See Figure A-9).

3.4.2 Faults

Neither Christensen and Shaw (2008a) nor the Quaternary Fault and Fold Database of the United States (USGS and UGS, 2006) show any Quaternary-aged (~2.6 million years ago to the present) faults to be present on or projecting towards the subject property. The Weber County Natural Hazards Overlay Districts defines an active fault to be “a fault displaying evidence of greater than four inches of displacement along one or more of its traces during Holocene time (about 11,000 years ago to the present)” (Weber County, 2015). The closest active fault to the property is the Weber Segment of the Wasatch Fault Zone, located approximately 9.5 miles southwest of the western margin of the property (USGS and UGS, 2006).

3.4.3 Debris Flows

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

3.4.4 Liquefaction

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

3.5 REVIEW OF AERIAL IMAGERY

A series of aerial photographs that cover project area were taken from the UGS Aerial Imagery Collection and analyzed stereoscopically for the presence of adverse geologic conditions across the property. This included a review of photos collected from the years 1947, 1953, and 1963. A table displaying the details of the aerial photographs reviewed can be found in the *References* section at the end of this report.

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

Google Earth imagery of the property from between the years of 1993 and 2014 were also reviewed. No landslide or other geological hazard features were noted in the imagery. The property was observed to be densely covered in trees and bushes, though the southeastern part of the lot is mainly covered in bushes. Some surficial gravel, cobbles, and boulders, were observed, though the property does not contain any drainages. No notable changes to the property, either human or natural, were observed in the aerial imagery across this time frame.

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)	$S_S = 0.831$	$S_1 = 0.276$
MCE Spectral Response Acceleration Site Class C (g)	$S_{MS} = S_sF_a = 0.887$	$S_{M1} = S_1F_v = 0.421$
Design Spectral Response Acceleration (g)	$S_{DS} = S_{MS}*{}^2/_3 = 0.591$	$S_{D1} = S_{M1}*{}^2/_3 = 0.281$

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 (*very dense soil/soft rock*). Based on IBC criteria, the short-period (F_a) coefficient is 1.068 and the long-period (F_v) site coefficient is 1.524. Based on the design spectral response accelerations for a *Building Risk Category* of I, II or III, the site's *Seismic Design Category* is D. The short- and long-period *Design Spectral Response Accelerations* are presented in Table 3.6; 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 HAZARD ASSESSMENT

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

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

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

The following is a summary of the geologic hazard assessment for the Lot 16R property.

3.7.1 Landslides/Mass Movement

The landslide hazard constitutes the greatest geologic hazard risk associated with the property. According to the several most recent geologic maps produced that cover the property, the lot is either entirely or partially situated on mapped landslide deposits (Coogan and King, 2016; Western Geologic, 2012; Elliott and Harty, 2010). Additionally, multiple breaks in slope observed south and downslope of the building envelope were noted during the site reconnaissance as potentially corresponding to landslide scarps, and thick landslide deposits were observed below a colluvial cover in TP-1 (see Figure A-3). Within TP-1, Unit 3 was interpreted to be a landslide deposit due to several collaborative features: it exhibited several distinct fat clay lenses that appeared to have been derived from different parent materials which generally dipped downslope; it contained abundant pinhole voids; and it included some angular Nounan Dolomite bedrock clasts, which are absent in the Wasatch Formation.

However, other geologic evidence suggests that while landslide deposits are present on the property, the associated hazard is not currently at a high level. The ground surface within the building envelope and the roughly northern half of the property was observed to be uneven, not hummocky, and generally exhibited a consistent slope to the south. Between 1.5 and 2 feet of topsoil development was observed in the test pits, with an additional 1.5 to 2 feet of colluvial deposits overlying the noted landslide deposits and Wasatch Formation (see Figures A-3 and A-4). Within the landslide deposit in TP-1, though the major constituent observed was a fat clay, no slickensides or evidence of shearing was observed, and no slide plane was encountered. Taken collectively, this data suggests that the landslide deposits are of an older age (likely Late Pleistocene), that a relatively stable geomorphic surface exists across the building envelope, and that there is no indication of active movement associated with the deposits.

In TP-2, though clayier than normally observed on Powder Mountain, Unit 3 underlying the colluvium was interpreted to be weathered Wasatch Formation because it was significantly sandier and gravellier than TP-1, and appeared to exhibit the typical characteristics of Wasatch Formation (clayey sand, exclusively subrounded to subangular quartzite clasts, clasts comprising ~20-30% of unit, etc).

Though some evidence of slight to moderate soil creep was observed in the aspen trees found on the property, the subsurface data indicate that this is restricted to the topsoil.

Given the geologic data alone, the risk associated with landslide hazards on the property is considered to be moderate. However, slope stability analyses has indicated the slope is stable under the current conditions (see Section 4.3). As such, the corresponding landslide and slope stability hazard risk is considered to be 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 9.5 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 at least in part 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 south. 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 either of the two test pits excavated as part of this investigation. The test pits were excavated in mid-July, and the groundwater level was likely to be on its way down from its seasonal high. No springs were observed on the property, and no plants indicative of shallow groundwater conditions were observed on the property. However, shallow groundwater conditions have been observed at the nearby *Horizon Neighbourhood* property (IGES, 2016).

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

4.0 GENERALIZED SITE CONDITIONS

4.1 SITE RECONNAISSANCE

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

At the time of the site reconnaissance, the property was observed to have uneven surface topography that consistently sloped downhill to the south. Dense vegetation in the form of aspen trees and low-lying bushes was observed across much of the property. The aspens displayed evidence of slight to moderate and in some places strong soil creep, though this was later found to be restricted to the topsoil.

Variously-sized boulders and cobbles were found scattered across the property, as part of the surficial colluvial geologic unit derived from weathered Wasatch Formation. These were typically subrounded, and were found to be as large as 3 feet in diameter. The rock clasts[1] were found to be comprised entirely of massive, coarsely crystalline quartzite, which was light gray in color when unweathered, but commonly weathered to pale reddish orange. The clasts were observed to be weathering out of a sandy lean clay topsoil.

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

Two notable breaks in slope were observed on the southern part of the property. The first, and less prominent, break in slope was observed just south of the southern edge of the building envelope. TP-2 was spotted in this area to cut across the break in slope. The second, more prominent break in slope was observed to be approximately 150 feet further south of the first, where the grade becomes notably steeper and the dense aspen trees give way to an area conspicuously devoid of trees. Near the southern margin of the property the grade levels out and highly irregular, possibly hummocky topography was observed to coincide with an influx of large to very large quartzite boulders up to 5 feet in diameter.

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

4.2 SUBSURFACE CONDITIONS

On July 10, 2017, two exploration test pits were excavated at representative locations (see Figure A-2). The test pits were excavated to depths ranging between 16 and 16.5 feet below existing grade with the aid of a Caterpillar 320F tracked excavator. Upon completion of logging, the test pits were backfilled without compactive effort. Detailed logs for the test pits are displayed in Figure A-3 and Figure A-4, respectively. Four distinct geologic units were encountered in the subsurface, with two of the units being found in both of the test pits. The soil and moisture conditions encountered during our investigation are discussed in the following paragraphs.

4.2.1 Earth Materials

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

Colluvium: This unit was encountered in both of the test pits, being between approximately 1.5 and 2 feet thick. The unit consisted of a dark brown, loose to medium stiff, moist to slightly moist, sandy lean CLAY with gravel (CL). Gravel and larger-sized subrounded to subangular quartzite clasts comprised between approximately 40 and 50% of the unit, with individual clasts up to 2 feet in diameter, though the mode clast size was approximately 6 inches in diameter.

Landslide: This unit was observed in TP-1 only, and was found to be at least 12 feet thick, extending to the maximum depth of exploration within the test pit. The unit consisted of a moderate reddish brown to dark yellowish orange to dark reddish brown, stiff to very stiff, moist to very moist, sandy lean CLAY with gravel (CL). Gravel and larger-sized subrounded quartzite clasts and angular dolomite clasts comprised between approximately 10 and 15% of the unit (estimated), with individual clasts up to 1.5 feet in diameter, though the mode clast size was 1 to 2 inches. Pinhole voids between 1 and 2 mm diameter were abundant within the unit, and the unit exhibited intertonguing beds/lenses of fat clay, sandy clay, and some silt, denoted by different colors.

Wasatch Formation: This unit was observed in TP-2 only, being more than 13.5 feet thick and extending to the maximum depth of exploration within the test pit. The unit consisted of weakly consolidated conglomerate bedrock that had been largely disaggregated into a moderate reddish brown to dark reddish brown, dense to medium-dense, moist mixture of clay, sand, and gravel that collectively classifies as clayey SAND with gravel (SC). Gravel and larger-sized subrounded quartzite clasts comprised between approximately 20 and 30% of the unit, with individual clasts up to 2 feet in diameter, with a mode clast size of 2 to 4 inches.

4.2.2 Groundwater

Groundwater was not encountered in either of the test pits excavated for this project; however, it should be noted that groundwater has been encountered in several test pit excavations located east of the subject lot in the *Horizon Neighbourhood* property (IGES, 2016). Additionally, the landslide unit observed at depth in TP-1 was very moist in places, and it is quite possible that groundwater, or local seeps, could be encountered locally in excavations that exceed a depth of 16 feet below existing grade.

4.2.3 Strength of Earth Materials

One consolidated-drained direct shear test was completed under drained conditions on a relatively undisturbed sample (tube sample) obtained from clayey soils that are part of the Wasatch Formation. The test results indicate that the clayey soils, which classify as sandy clay (~71% fines), has a friction angle of 34 degrees and a cohesion of 180 psf. A summary of the direct shear test is presented in Appendix B.

4.3 SLOPE STABILITY

4.3.1 Global Stability

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

Groundwater, e.g. a piezometric groundwater surface, was not encountered during our subsurface investigation; however, seepage was noted in test pits on nearby properties (IGES, 2016). Accordingly, groundwater was not modeled in our limit-equilibrium analysis. Saturated parallel seepage has been modeled in a separate analysis (see Section 4.3.2).

Soil strength parameters were selected based on soil types observed, local experience, correlation with index properties (Atterberg Limits, clay content), site-specific strength testing (direct shear test), and comparisons with soil strength laboratory data from a nearby site (IGES, 2016). 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	36	150	120
Bedrock (Tw)	45	100	145
Older landslide (Qlos)	34	181	120
Engineered Fill (Af)	36	200	135

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 0.36g. Half of the PGA, (0.18g), was taken as the horizontal seismic coefficient (k_h) (Hynes and Franklin, 1984), and used in the pseudo-static seismic screen analysis. The results of the analyses have been summarized in Table 4.3.1b.

Table 4.3.1b
Results of Slope Stability Analyses

Section	Static Factor of Safety	Pseudo-Static Factor of Safety
Existing Condition	2.19	1.43

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

4.3.2 Surficial Stability

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

IGES assessed the potential for the upper three feet to become mobilized under saturated parallel seepage conditions. Our assessment assumes three feet of clayey colluvium or topsoil, fully saturated, and a 3.5H:1V slope (this would be a transient condition that could occur during primary spring run-off and snowmelt). Our model assumes an estimated effective friction angle of 28 degrees and a cohesion of 150 psf, and a saturated unit weight of 135 pcf. Based on this model, a factor-of-safety of 2.06 results. Sample calculations are presented in Appendix D.

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

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

5.0 CONCLUSIONS AND RECOMMENDATIONS

5.1 GENERAL CONCLUSIONS

Based on the results of the field observations, literature review, and slope stability analyses, **the subsurface conditions are considered suitable for the proposed development provided that the recommendations presented in this report are incorporated into the design and construction of the project.**

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

5.2 GEOLOGIC CONCLUSIONS AND RECOMMENDATIONS

Based upon the data collected and reviewed as part of the geologic hazard assessment, IGES makes the following conclusions regarding the geological hazards present at the Lot 16R project area:

- **The Lot 16R project area does not appear to have geological hazards that are capable of adversely impacting the development as currently proposed under the existing conditions.**

- Landsliding represents the greatest geologic risk to the property. The property is located on mapped landslide deposits, and older landslide deposits were observed in the subsurface in TP-1. However, geologic evidence indicative of active movement was not observed, and the slope stability analyses indicates a stable slope for the property. As such, the landslide hazard for the property is considered to be low to moderate, as there is always some inherent risk when developing on known landslide deposits.

- Earthquake ground shaking is the only other identified hazard that may potentially affect all parts of the project area and is considered to pose a moderate risk.

- Shallow groundwater conditions were not observed in either of the two test pits, though groundwater seepage has been observed in test pits on adjacent properties; therefore, shallow groundwater hazards are considered to be low to moderate for the property.

- Rockfall, surface-fault-rupture, liquefaction, debris-flow, and flooding hazards are considered to be low for the property.

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

- Because landslide deposits are noted on the property, an IGES engineering geologist or geotechnical engineer should observe the foundation excavation to assess the absence (or presence) of landslide-induced shearing.

- The contact between the landslide deposits and the Wasatch Formation is located at some point between TP-1 and TP-2, which means it is likely somewhere within the main part of the building envelope. As such, an IGES engineering geologist should be present to identify the contact, note its trend, and provide recommendations for overexcavation and the placement of structural fill, if necessary.

- Effort should be made to limit the introduction of water into the subsurface near the proposed residence. Appropriate grading and drainage away from the home and xeriscape or natural landscaping will assist in reducing the risk of landsliding.

5.3 EARTHWORK

5.3.1 General Site Preparation and Grading

Below proposed structures, fills, and man-made improvements, all vegetation, topsoil, debris and undocumented fill (if any) should be removed. Any existing utilities should be re-routed or protected in place. The exposed native soils should then be proof-rolled with heavy rubber-tired equipment such as a scraper or loader*. Any soft/loose areas identified during proof-rolling should be removed and replaced with structural fill. All excavation bottoms should be observed by an IGES representative during proof-rolling or otherwise prior to placement of engineered fill to evaluate whether soft, loose, or otherwise deleterious earth materials have been removed, and to assess compliance with the recommendations presented in this report.
*not required where bedrock is exposed in the foundation subgrade

5.3.2 Excavations

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

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

5.3.3 Excavation Stability

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

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

5.3.4 Structural Fill and Compaction

All fill placed for the support of structures, flatwork or pavements should consist of structural fill. Structural fill should consist of granular native soils, which may be defined as soils with less than 25% fines, 10-60% sand, and contain no rock larger than 4 inches in nominal size (6 inches in greatest dimension). Structural fill should also be free of vegetation and debris. All structural fill should be 1 inch minus material when within 1 foot of any base coarse material. Soils not meeting these criteria may be suitable for use as structural fill; however, such soils should be evaluated on a case by case basis and should be approved by IGES prior to use.

All structural fill should be placed in maximum 4-inch loose lifts if compacted by small hand-operated compaction equipment, maximum 6-inch loose lifts if compacted by light-duty rollers, and maximum 8-inch loose lifts if compacted by heavy duty compaction equipment that is capable of efficiently compacting the entire thickness of the lift. Additional lift thickness may be allowed by IGES provided the Contractor can demonstrate sufficient compaction can be achieved with a given lift thickness with the equipment in use. We recommend that all structural fill be compacted on a horizontal plane, unless otherwise approved by IGES. Structural fill underlying all shallow footings and pavements should be compacted to at least 95 percent of the MDD as determined by ASTM D-1557. **The moisture content should be at, or slightly above, the OMC for all structural fill**. Any imported fill materials should be approved prior to importing. Also, prior to placing any fill, the excavations should be observed by IGES to confirm that unsuitable materials have been removed. In addition, proper grading should precede placement of fill, as described in the General Site Preparation and Grading subsection of this report.

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

5.3.5 Oversize Material

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

5.3.6 Utility Trench Backfill

Utility trenches should be backfilled with structural fill in accordance with Section 5.3.4 of this report. Utility trenches can be backfilled with the onsite soils free of debris, organic and oversized material. Prior to backfilling the trench, pipes should be bedded in and shaded with a uniform granular material that has a Sand Equivalent (SE) of 30 or greater. Pipe bedding may be water-densified in-place (jetting). Alternatively, pipe bedding and shading may consist of clean ¾-inch gravel. Native earth materials can be used as backfill over the pipe bedding zone. All utility trenches backfilled below pavement sections, curb and gutter, and hardscape, should be backfilled with structural fill compacted to at least 95 percent of the MDD as determined by ASTM D-1557. All other trenches should be backfilled and compacted to approximately 90 percent of the MDD (ASTM D-1557). However, in all cases the pipe bedding and shading should meet the design criteria of the pipe manufacturer. Specifications from governing authorities having their own precedence for backfill and compaction should be followed where they are more stringent.

5.4 FOUNDATION RECOMMENDATIONS

Based on our field observations and considering the presence of relatively competent native earth materials, we recommend that the footings for proposed single-family home be founded either *entirely* on competent native soils <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 3 feet below existing grade (in TP-2), but may be deeper, or shallower, at specific locations. We recommend that IGES assess the bottom of the foundation excavation prior to the placement of steel or concrete, or structural fill, to identify the competent native earth materials as well as any unsuitable soils or transition zones. Additional over-excavation may be required based on the actual subsurface conditions observed.

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

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

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

5.5 SETTLEMENT

5.5.1 Static Settlement

Static settlements of properly designed and constructed conventional foundations, founded as described in Section 5.4, are anticipated to be on the order of 1 inch or less. Differential settlement is expected to be half of total settlement over a distance of 30 feet.

5.5.2 Dynamic Settlement

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

Based on the subsurface conditions encountered, dynamic settlement arising from a MCE seismic event is expected to be low; for design purposes, settlement on the order of ½ inch over 40 feet may be assumed.

5.6 EARTH PRESSURES AND LATERAL RESISTANCE

Lateral forces imposed upon conventional foundations due to wind or seismic forces may be resisted by the development of passive earth pressures and friction between the base of the footing and the supporting soils. In determining the frictional resistance against concrete, a coefficient of friction of 0.45 for sandy/gravelly native soils or structural fill should be used.

Ultimate lateral earth pressures from *granular* backfill acting against retaining walls, temporary shoring, or buried structures may be computed from the lateral pressure coefficients or equivalent fluid densities presented in Table 5.6. These lateral pressures should be assumed even if the backfill is placed in a relatively narrow gap between a vertical bedrock cut and the foundation wall. These coefficients and densities assume no buildup of hydrostatic pressures. The force of water should be added to the presented values if hydrostatic pressures are anticipated.

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

Walls and structures allowed to rotate slightly should use the active condition. If the element is to be constrained against rotation (i.e., a basement wall), the at-rest condition should be used. These values should be used with an appropriate factor of safety against overturning and sliding. A value of 1.5 is typically used. Additionally, if passive resistance is calculated in conjunction with frictional resistance, the passive resistance should be reduced by ½.

Table 5.6
Lateral Earth Pressure Coefficients

Condition	Level Backfill		2H:1V Backfill	
	Lateral Pressure Coefficient	Equivalent Fluid Density (pcf)	Lateral Pressure Coefficient	Equivalent Fluid Density (pcf)
Active (Ka)	0.33	41.7	0.53	66.5
At-rest (Ko)	0.50	55	0.80	85
Passive (Kp)	3.0	375	—	—
Seismic Active	0.12	15.1	0.38	47.4
Seismic Passive	-0.33	-40.8	—	—
Seismic At-rest	0.18	22.5	0.57	71.7

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

5.7 CONCRETE SLAB-ON-GRADE CONSTRUCTION

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

All concrete slabs should be designed to minimize cracking as a result of shrinkage. Consideration should be given to reinforcing the slab with a welded wire fabric, re-bar, or fibermesh. Slab reinforcement should be designed by the structural engineer; however, as a minimum, slab reinforcement should consist of 4''×4'' W2.9×W2.9 welded wire mesh within the middle third of the slab. We recommend that concrete be tested to assess that the slump and/or air content is in compliance with the plans and specifications. We recommend that concrete be placed in general accordance with the requirements of the American Concrete Institute (ACI). A Modulus of Subgrade Reaction of **250 psi/inch** may be used for design.

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

5.8 MOISTURE PROTECTION AND SURFACE DRAINAGE

Surface moisture should not be allowed to infiltrate into the soils in the vicinity of the foundations. As such, design strategies to minimize ponding and infiltration near the structures should be implemented.

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

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

5.9 SOIL CORROSION POTENTIAL

Laboratory testing of a representative soil sample obtained during our subsurface exploration indicated that the soil sample tested had a sulfate content of 5.7 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 5,311 OHM-cm, soluble chloride content of 5.7 ppm and a pH of 6.6. 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 48 inches in diameter) were observed on the surface and within the test pits; as such, excavation of the basement may generate an abundance of over-size material that may require special handling, processing, or disposal.

6.0 CLOSURE

6.1 LIMITATIONS

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

The information contained in this report is based on limited field testing and understanding of the project. The subsurface data used in the preparation of this report were obtained largely from the explorations made for the Building 4 project. It is very likely that variations in the soil, rock, and groundwater conditions exist between and beyond the points 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.

Landslide deposits were observed in the subsurface within the building envelope of the property. Though subsurface data indicate that the landslide is older and appears to be inactive, there is always inherent risk in developing on landslide deposits as changes to current conditions (through increased moisture or excessive grading) may induce the reactivation of a currently inactive landslide. It should be noted that while the slope stability assessment as performed as part of this investigation indicate that the slope is currently stable, the landslide risk cannot be assumed to be zero.

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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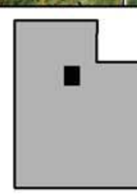
AERIAL PHOTOGRAPHS

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 MAP:
USGS Huntsville (2017), Browns Hole (2014), James Peak (2017) and Sharp Mountain (2014)
7.5-Minute Quadrangle Topographic Maps



MAP LOCATION

0' 1000' 2000'
SCALE 1:24,000

IGES®

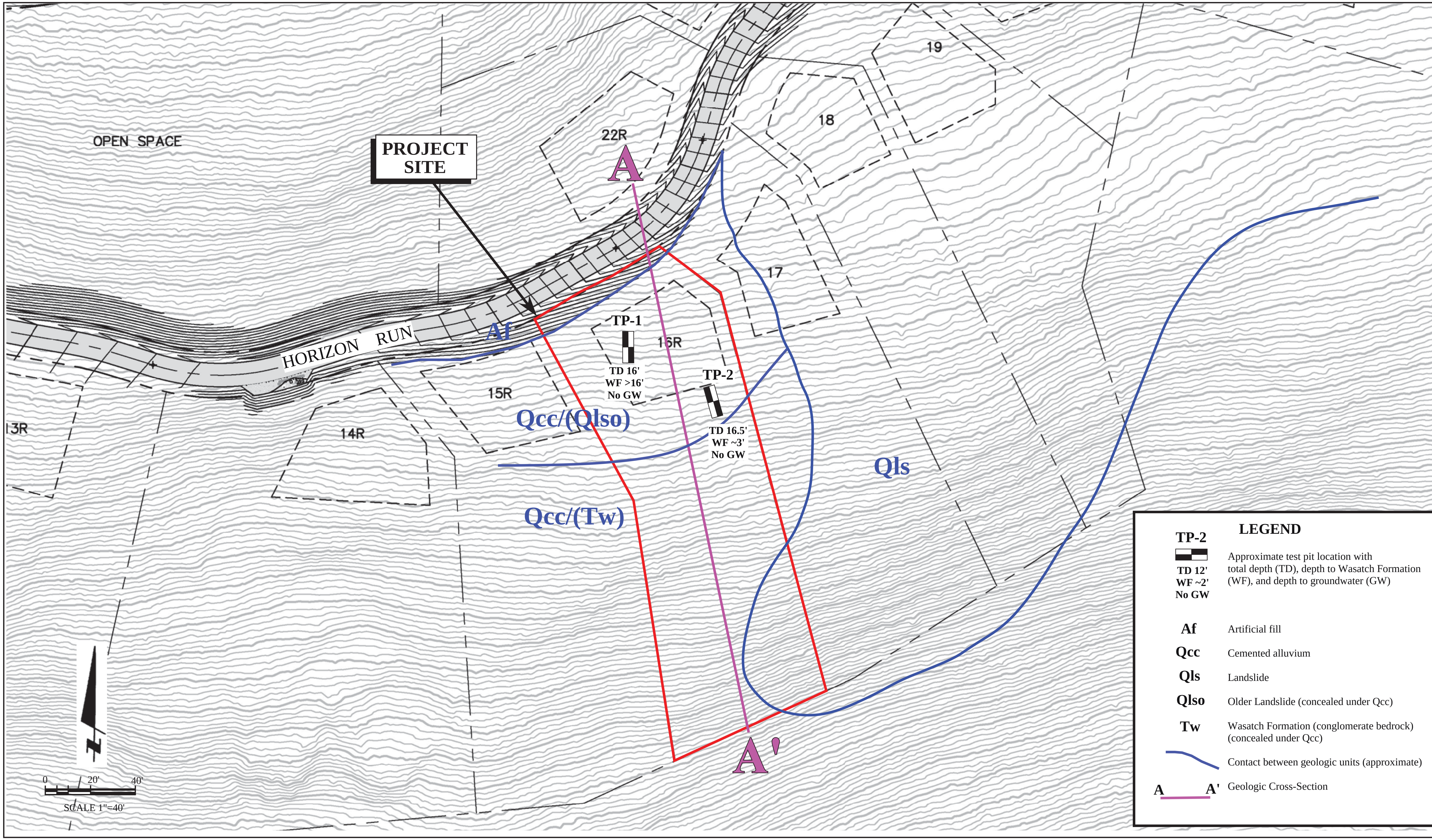
Project No. 02529-001

Geotechnical & Geologic Hazard Investigation
Lot 16 of Summit Eden Phase 1A
Summit Powder Mountain Resort
Weber County, Utah

SITE VICINITY MAP

Figure

A-1



Basemap: Undated 100-scale map of Phase 1A prepared by NV5

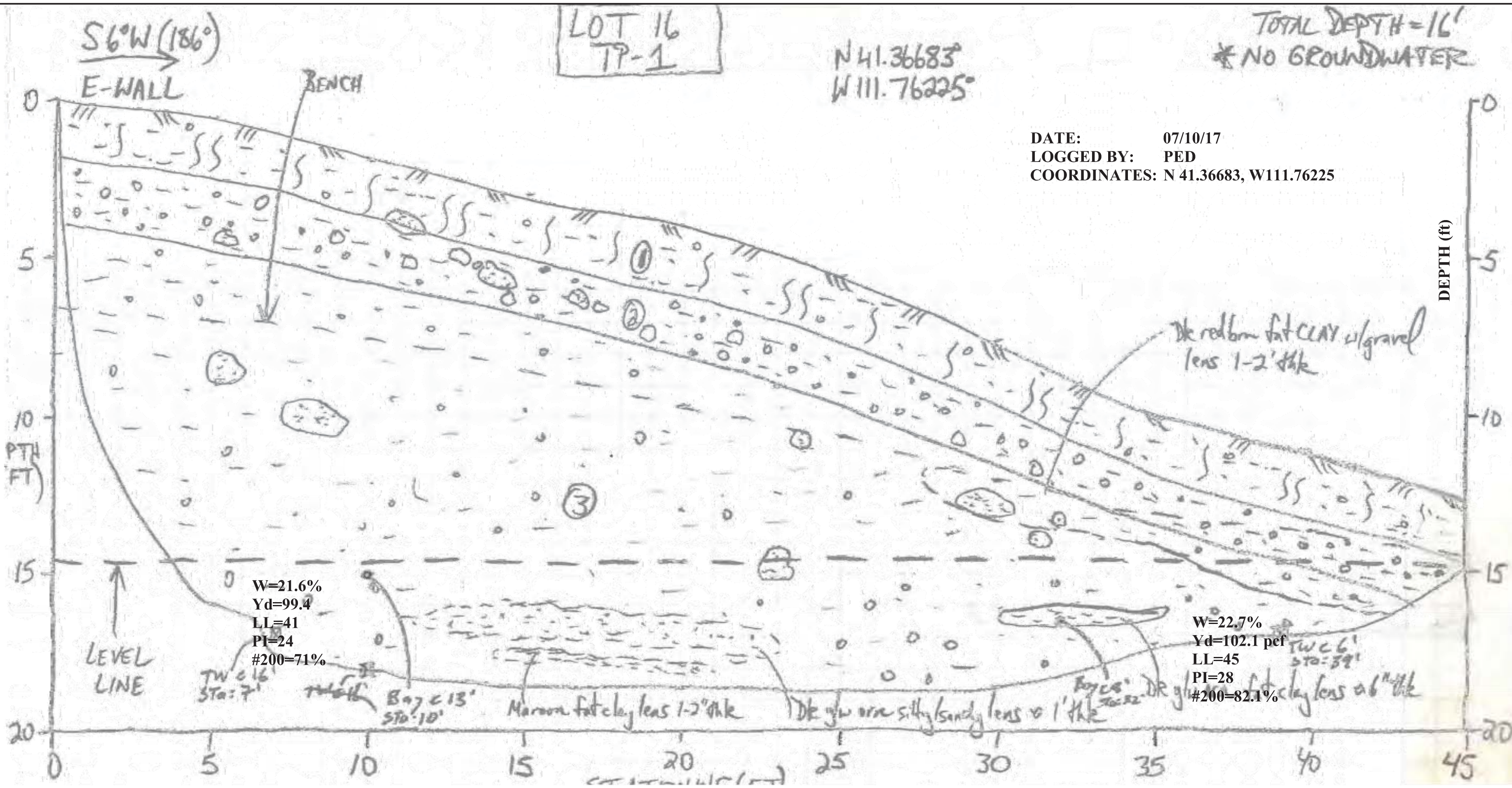
IGES®

Project No. 02529-001

Geotechnical & Geologic Hazard Investigation
Lot 16R of Summit Eden Phase 1A
Summit Powder Mountain Resort
Weber County, Utah GEOTECH & GEOLOGY MAP

Figure

A-2



LITHOLOGIC UNIT DESCRIPTIONS:

1) **A/B Soil Horizon**: ~2' thick; dark yellowish brown (10YR 4/2) sandy lean CLAY with gravel (CL), loose, slightly moist, low plasticity, massive; gravel and larger sized clasts comprise ~5-10% of unit; clasts entirely subrounded to subangular medium light gray (N6) to pale yellowish orange (10YR 8/6) quartzite up to 1.5' in diameter, though mode size ~1-2"; only uppermost ~3" is heavily vegetated with abundant plant and tree roots, though roots do extend to base of unit; minor silt component; gradational, planar basal contact.

2) **Colluvium**: ~2' thick; dark yellowish brown (10YR 4/2) sandy lean CLAY with gravel (CL) gradational to gravelly lean CLAY with sand (CL), loose to medium stiff, slightly moist, low plasticity, massive; gravel and larger sized clasts comprise ~40% of unit; clasts entirely subrounded to subangular quartzite as above up to 1.5' in diameter, though mode size ~3-6"; likely contains an alluvial component; common to abundant plant and tree roots; sharp, planar basal contact.

3) **Old Landslide**: At least ~12' thick; moderate reddish brown (10R 4/6) to dark yellowish orange (10YR 6/6) to dark reddish brown (10R ¾) sandy lean CLAY with gravel (CL) predominantly, though gradational to fat CLAY with gravel (CH) in places; stiff to very stiff, moist to very moist, moderate to high plasticity, massive; gravel and larger sized clasts comprise ~10-15% of unit; clasts predominantly quartzite as above up to 1.5' in diameter and mode size ~1-2"; some dark yellowish orange, highly oxidized, angular sandy dolomite clasts up to 3" in diameter; abundant 1-2 mm diameter pinholes throughout; appears to have intertonguing beds of clay and sandy clay and some silt, denoted by different colors; maroon/purple fat clay appears ~1' off bottom of test pit; no observed slickensides or evidence of shearing.

SCALE: 1"=5' H&V

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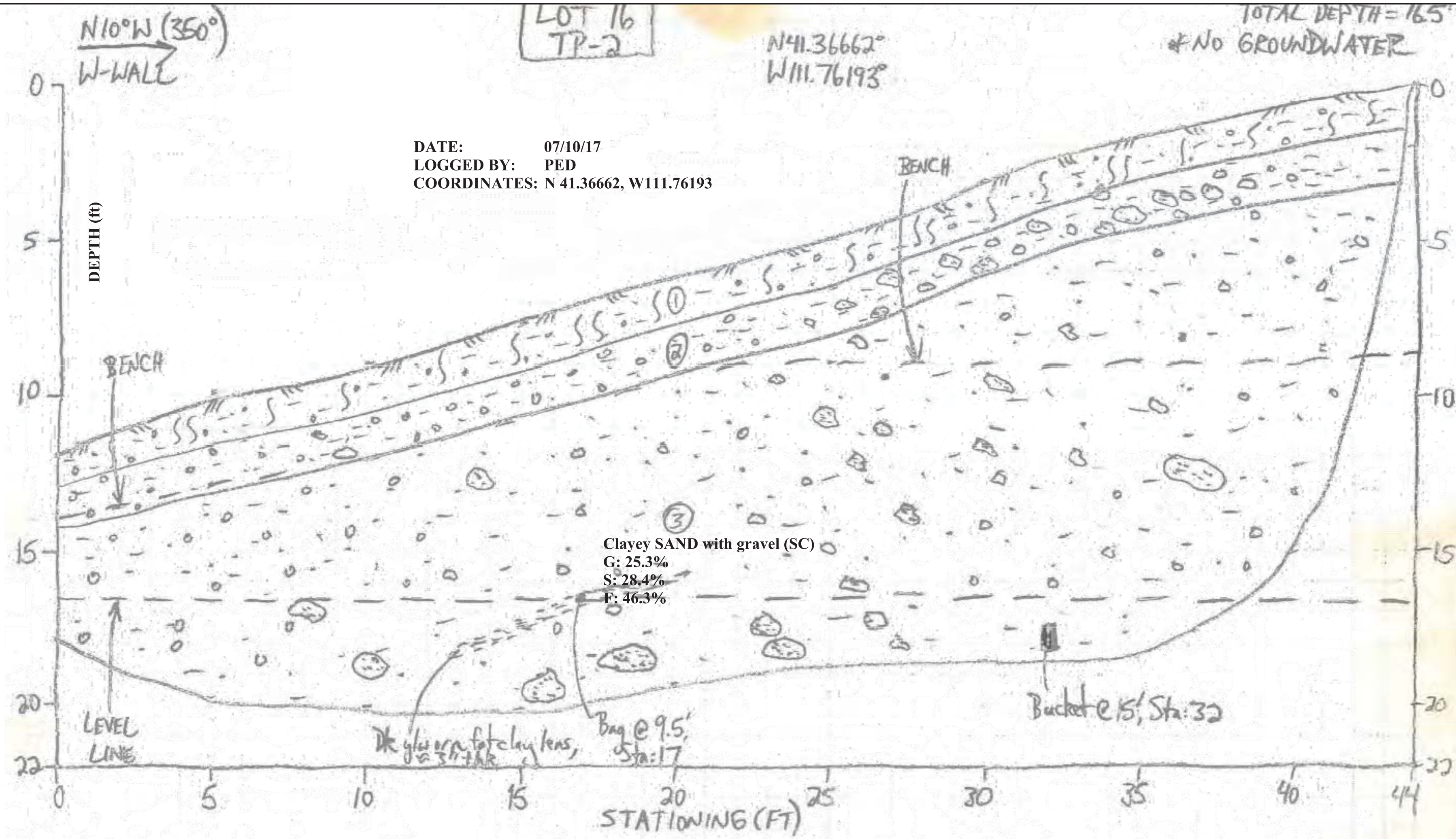
Geotechnical & Geologic Hazard Investigation
Lot 16R of Summit Eden Phase 1A
Summit Powder Mountain Resort
Weber County, Utah

TEST PIT LOG TP-1

Figure

A-3



LITHOLOGIC UNIT DESCRIPTIONS:

1) **A/B Soil Horizon:** ~1.5' thick; dark yellowish brown (10YR 4/2) sandy lean CLAY with gravel (CL), loose to medium stiff, slightly moist, low plasticity, massive; gravel and larger sized clasts comprise ~10% of unit; clasts entirely subangular medium light gray (N6) to pale yellowish orange (10YR 8/6) quartzite up to 4" in diameter, though mode size <1"; abundant plant and tree roots; gradational, planar basal contact.

2) **Colluvium:** ~1.5' thick; dark yellowish brown (10YR 4/2) sandy lean CLAY with gravel (CL) gradational to clayey GRAVEL (GC), loose to medium stiff, moist, low plasticity, massive; gravel and larger sized clasts comprise ~40% of unit; clasts entirely subangular to subrounded quartzite as above up to 2' in diameter, though mode size ~6-8"; likely contains an alluvial component; minor silt component; common plant and tree roots; sharp, planar basal contact.

3) **Wasatch Formation:** At least 13.5' thick; conglomerate bedrock largely disaggregated to moderate reddish brown (10R 4/6) to dark reddish brown (10R ¾) clayey SAND with gravel (SC), dense to medium dense, moist, moderate plasticity fines, massive; gravel and larger sized clasts comprise ~20-30% of unit; clasts entirely subrounded to subangular quartzite as above up to 2' in diameter, though mode size ~2-4"; common 1 mm pinholes throughout and especially where clayey; occasional thin dark reddish brown fat clay lenses, though no slickensides or shear observed; appears more clay-rich than typical Wasatch Formation, though exhibits the rest of typical Wasatch Formation characteristics; poorly sorted; common sloughing.

SCALE: 1"=5' H&V

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Lot 16R of Summit Eden Phase 1A
Summit Powder Mountain Resort
Weber County, Utah

TEST PIT LOG TP-2

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.

KEY TO SOIL SYMBOLS AND TERMINOLOGY

Project No. 02529-001

Engr. DAG

Drafted By DAG

Date July 2017

IGES

Intermountain Geo-Environmental Services, Inc.

Figure A-5

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

KEY TO PHYSICAL ROCK PROPERTIES

Project No. 02529-001
Engr. DAG
Drafted By DAG
Date July 2017

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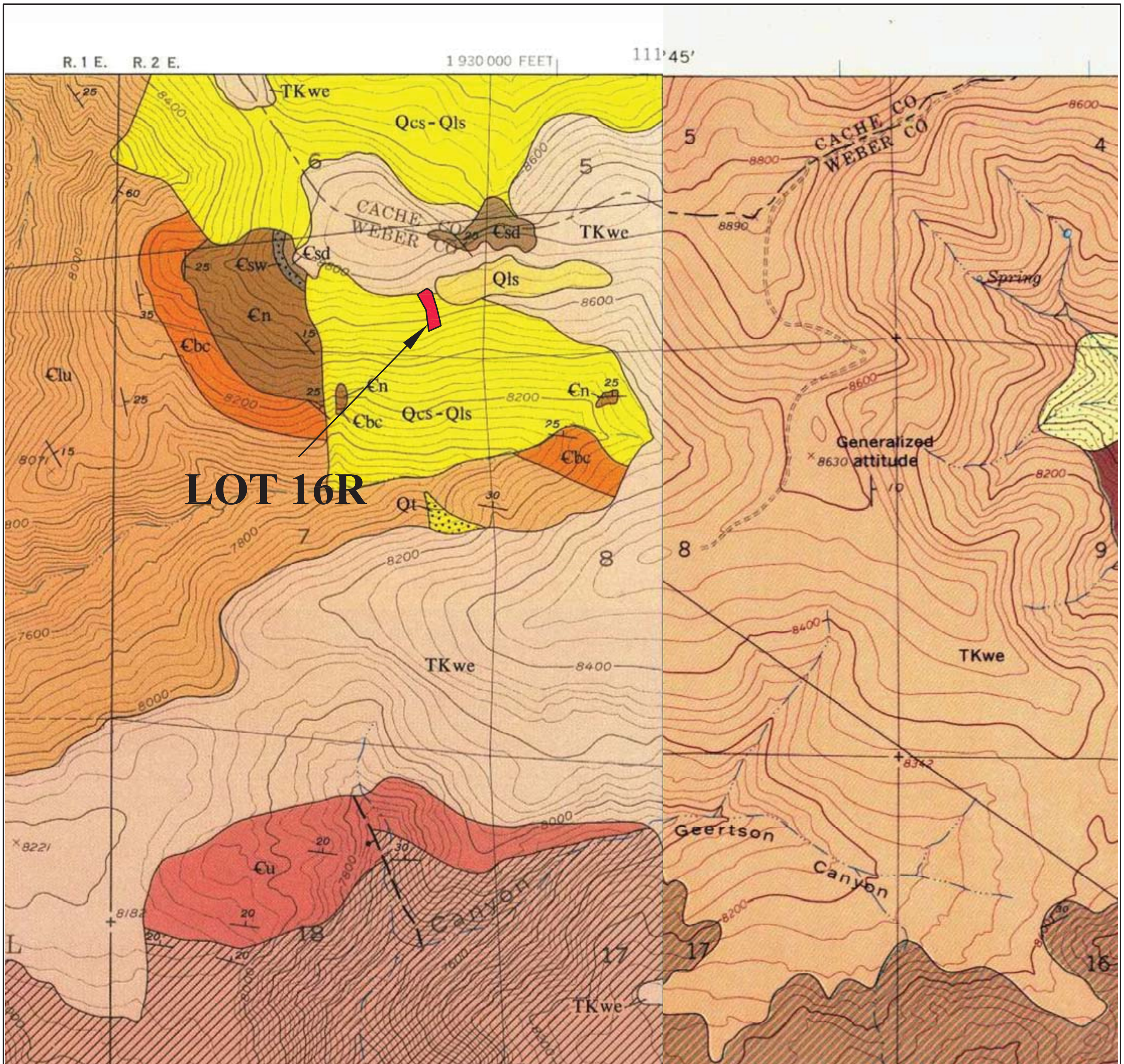
Figure A-6



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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Geotechnical & Geologic Hazard Investigation
Lot 16R of Summit Eden Phase 1A
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:

Єsd — 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

Єsw — 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

Єn — 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

Єbc — 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

Єlu — CAMBRIAN LIMESTONES, UNDIVIDED (Middle Cambrian) – Includes limestone and Hodges Shale Members of Bloomington Formation, and Blacksmith and Ute Limestones

Єb — 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

Ꞓu

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

Ꞓgcu

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

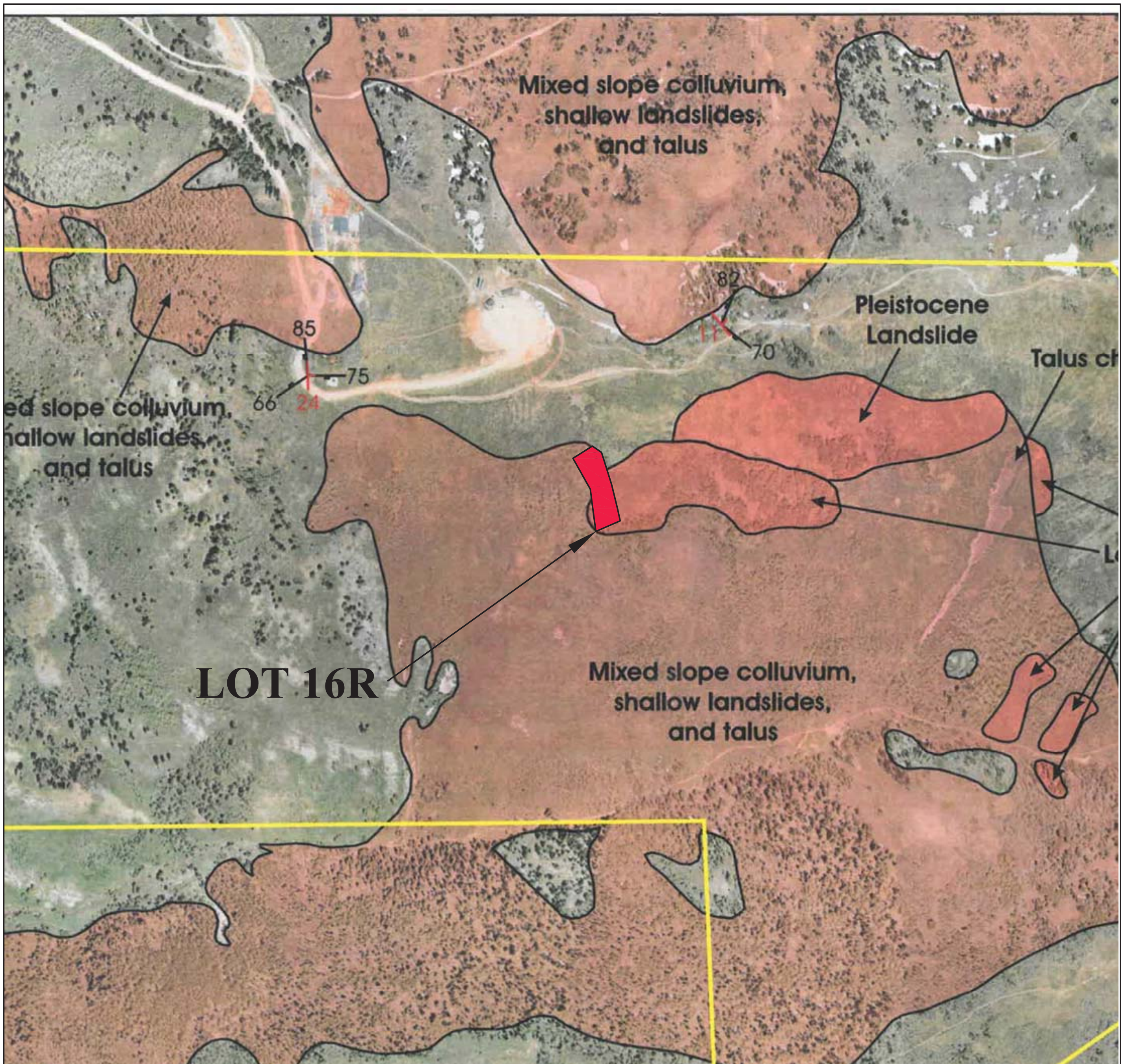
Ꞓgcl

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
Reconaissance Report, Figure 3

N

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

SCALE: 1"=1,000'

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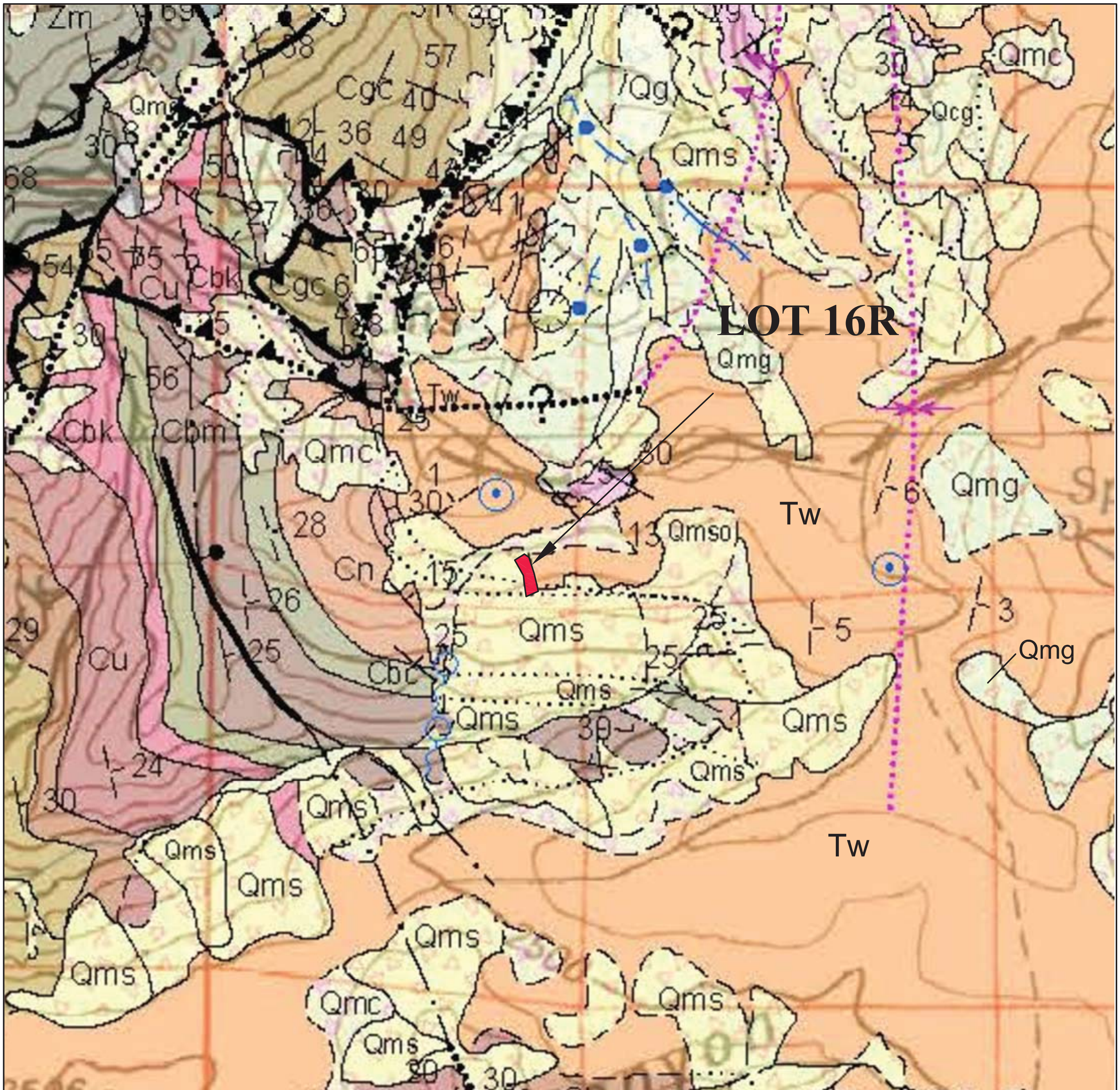
Lot 16R of Summit Eden Phase 1A
Summit Powder Mountain Resort
Weber County, Utah

REGIONAL GEOLOGY MAP 2

Figure

A-8



BASE MAP

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

N

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

SCALE: 1"=2,000'

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Lot 16R of Summit Eden Phase 1A
Summit Powder Mountain Resort
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)

Project: **Kimmelman/ May Lot 16**
No: **02529-001**
Location: Summit Powder Mtn.
Date: 7/31/2017
By: BSS

Sample Info.	Boring No.	T-1							
	Sample:								
	Depth:	6.0'							
Unit Weight Info.	Sample height, H (in)	5.187							
	Sample diameter, D (in)	2.400							
	Sample volume, V (ft^3)	0.0136							
	Mass rings + wet soil (g)	1017.71							
	Mass rings/tare (g)	246.14							
	Moist soil, Ws (g)	771.57							
	Moist unit wt., γ_m (pcf)	125.26							
Water Content	Wet soil + tare (g)	481.53							
	Dry soil + tare (g)	416.09							
	Tare (g)	127.45							
Water Content, w (%)		**22.7**							
Dry Unit Wt., γ_d (pcf)		**102.1**							

Entered by:___________

Reviewed:___________

Liquid Limit, Plastic Limit, and Plasticity Index of Soils

(ASTM D4318)

IGES

Project: **Kimmelman/ May Lot 16**
No: **02529-001**
Location: Summit Powder Mtn.
Date: 8/2/2017
By: BRR
Grooving tool type: Plastic
Liquid limit device: Mechanical
Rolling method: Hand

Boring No.: **TP-1**
Sample:
Depth: **6.0'**
Description: Reddish brown lean clay

Preparation method: Wet
Liquid limit test method: Multipoint
Screened over No.40: Yes
Larger particles removed: Wet sieved
Approximate maximum grain size: 3/8"
Estimated percent retained on No.40: See Particle Size Distribution
As-received water content (%): 22.7

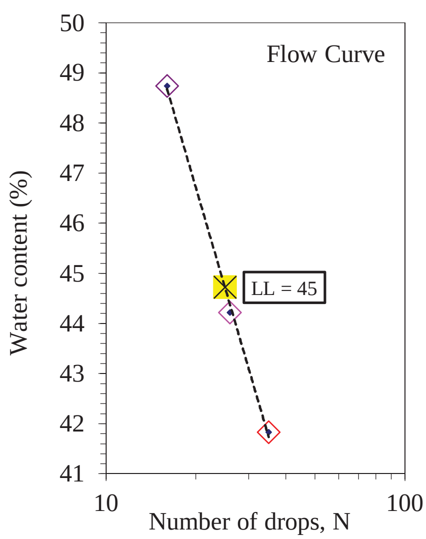
Plastic Limit

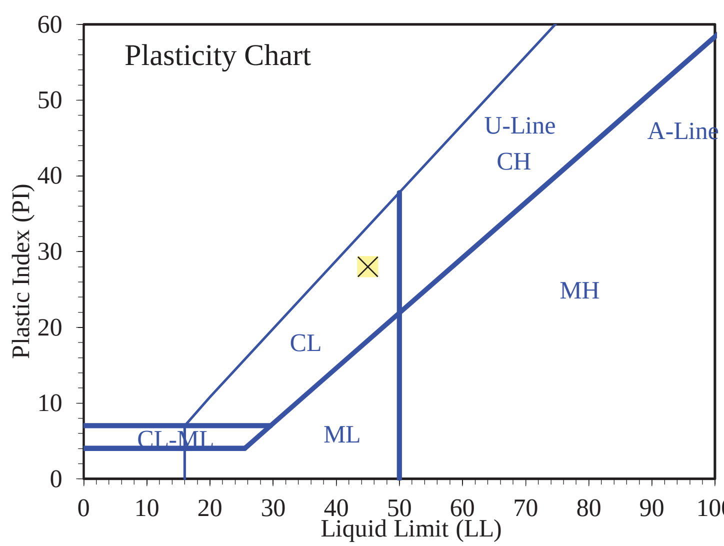
Determination No	1	2				
Wet Soil + Tare (g)	27.74	28.33				
Dry Soil + Tare (g)	26.83	27.41				
Water Loss (g)	0.91	0.92				
Tare (g)	21.50	22.03				
Dry Soil (g)	5.33	5.38				
Water Content, w (%)	17.07	17.10				

Liquid Limit

Determination No	1	2	3			
Number of Drops, N	35	26	16			
Wet Soil + Tare (g)	29.33	27.79	28.49			
Dry Soil + Tare (g)	27.13	25.84	26.37			
Water Loss (g)	2.20	1.95	2.12			
Tare (g)	21.87	21.43	22.02			
Dry Soil (g)	5.26	4.41	4.35			
Water Content, w (%)	41.83	44.22	48.74			
One-Point LL (%)		44				

Liquid Limit, LL (%)	**45**
Plastic Limit, PL (%)	**17**
Plasticity Index, PI (%)	**28**





Entered by:____________
Reviewed:____________

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Liquid Limit, Plastic Limit, and Plasticity Index of Soils

(ASTM D4318)

Project: Kimmelman/ May Lot 16
No: 02529-001
Location: Summit Powder Mtn.
Date: 8/2/2017
By: BRR

Boring No.: TP-1
Sample:
Depth: 16.0'
Description: Reddish brown lean clay

Grooving tool type: Plastic
Liquid limit device: Mechanical
Rolling method: Hand

Preparation method: Wet
Liquid limit test method: Multipoint
Screened over No.40: Yes
Larger particles removed: Wet sieved
Approximate maximum grain size: 3/8"
Estimated percent retained on No.40: Not requested
As-received water content (%): Not requested

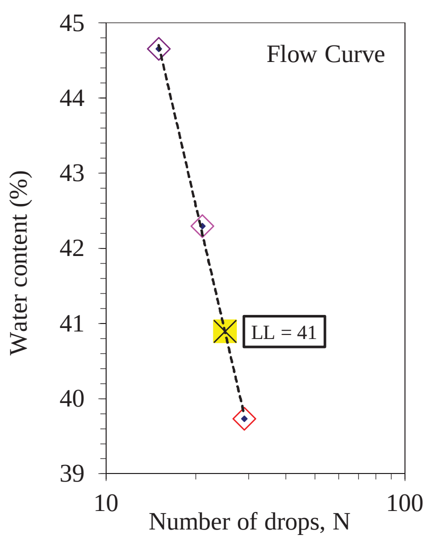
Plastic Limit

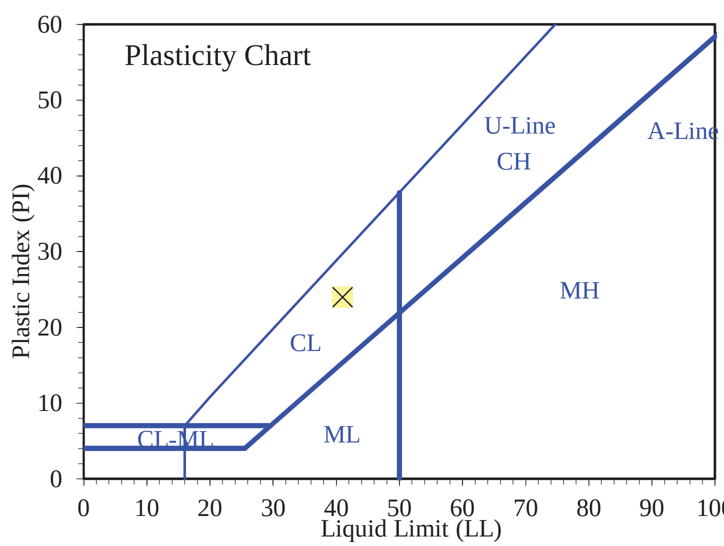
Determination No	1	2				
Wet Soil + Tare (g)	28.95	28.69				
Dry Soil + Tare (g)	27.88	27.67				
Water Loss (g)	1.07	1.02				
Tare (g)	21.83	21.72				
Dry Soil (g)	6.05	5.95				
Water Content, w (%)	17.69	17.14				

Liquid Limit

Determination No	1	2	3			
Number of Drops, N	29	21	15			
Wet Soil + Tare (g)	28.00	28.88	29.42			
Dry Soil + Tare (g)	26.22	26.85	27.04			
Water Loss (g)	1.78	2.03	2.38			
Tare (g)	21.74	22.05	21.71			
Dry Soil (g)	4.48	4.80	5.33			
Water Content, w (%)	39.73	42.29	44.65			
One-Point LL (%)	40	41				

Liquid Limit, LL (%)	**41**
Plastic Limit, PL (%)	**17**
Plasticity Index, PI (%)	**24**





Entered by:____________
Reviewed:____________

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

(ASTM D6913)

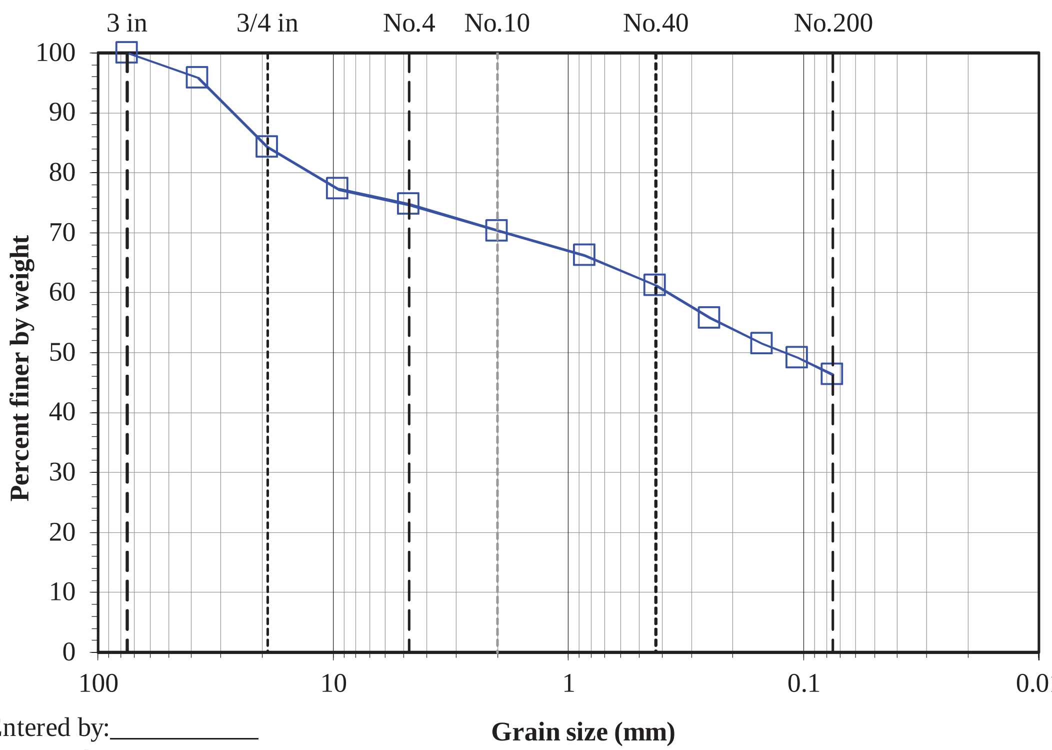
Project: Kimmelman/ May Lot 16
No: 02529-001
Location: Summit Powder Mtn.
Date: 8/1/2017
By: BSS

Boring No.: TP-2
Sample:
Depth: 9.5'
Description: Red clayey sand with gravel

Split:	Yes	
Split sieve:	3/8"	
	Moist	Dry
Total sample wt. (g):	3757.42	3395.22
+3/8" Coarse fraction (g):	786.02	771.19
-3/8" Split fraction (g):	633.42	559.37
Split fraction:	0.773	

Water content data	C.F.(+3/8")	S.F.(-3/8")
Moist soil + tare (g):	1010.44	848.74
Dry soil + tare (g):	994.26	774.69
Tare (g):	152.69	215.32
Water content (%):	1.9	13.2

Sieve	Accum. Wt. Ret. (g)	Grain Size (mm)	Percent Finer	
8"	-	200	-	
6"	-	150	-	
4"	-	100	-	
3"	-	75	100.0	
1.5"	142.91	37.5	95.8	
3/4"	533.80	19	84.3	
3/8"	771.19	9.5	77.3	←Split
No.4	18.76	4.75	74.7	
No.10	50.60	2	70.3	
No.20	80.70	0.85	66.1	
No.40	116.68	0.425	61.2	
No.60	155.85	0.25	55.8	
No.100	186.88	0.15	51.5	
No.140	203.73	0.106	49.1	
No.200	224.19	0.075	46.3	



Gravel (%): 25.3
Sand (%): 28.4
Fines (%): 46.3

Entered by:___________
Reviewed:___________

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

(ASTM D1140)

IGES

Project: Kimmelman/ May Lot 16
No: 02529-001
Location: Summit Powder Mtn.
Date: 8/1/2017
By: BSS

Sample Info.	Boring No.	TP-1	TP-1						
	Sample								
	Depth	6.0'	16.0'						
	Split	No	No						
	Split Sieve*								
	Method	B	B						
	Specimen soak time (min)	320	320						
	Moist total sample wt. (g)	354.08	227.53						
	Moist coarse fraction (g)								
	Moist split fraction + tare (g)								
	Split fraction tare (g)								
	Dry split fraction (g)								
	Dry retained No. 200 + tare (g)	179.06	180.85						
	Wash tare (g)	127.45	126.61						
	No. 200 Dry wt. retained (g)	51.61	54.24						
	Split sieve* Dry wt. retained (g)								
	Dry total sample wt. (g)	288.64	187.18						
Coarse Fraction	Moist soil + tare (g)								
	Dry soil + tare (g)								
	Tare (g)								
	Water content (%)								
Split Fraction	Moist soil + tare (g)	481.53	354.14						
	Dry soil + tare (g)	416.09	313.79						
	Tare (g)	127.45	126.61						
	Water content (%)	22.67	21.56						
	Percent passing split sieve* (%)								
	Percent passing No. 200 sieve (%)	**82.1**	**71.0**						

Entered by:___________

Reviewed:___________

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Direct Shear Test for Soils Under Drained Conditions

(ASTM D3080)

Project: Kimmelman/ May Lot 16
No: 02529-001
Location: Summit Powder Mtn.
Date: 8/1/2017
By: NB

Boring No.: TP-1
Sample:
Depth: 16.0'
Sample Description: Reddish brown clay with sand
Sample type: Undisturbed-trimmed from thin-wall

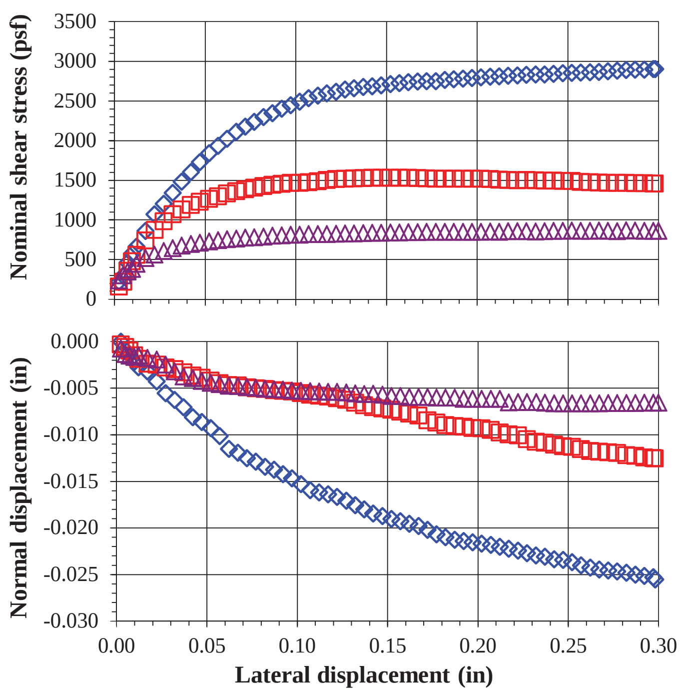
Test type: Inundated
Lateral displacement (in.): 0.3
Shear rate (in./min): 0.0022
Specific gravity, Gs: 2.70 Assumed

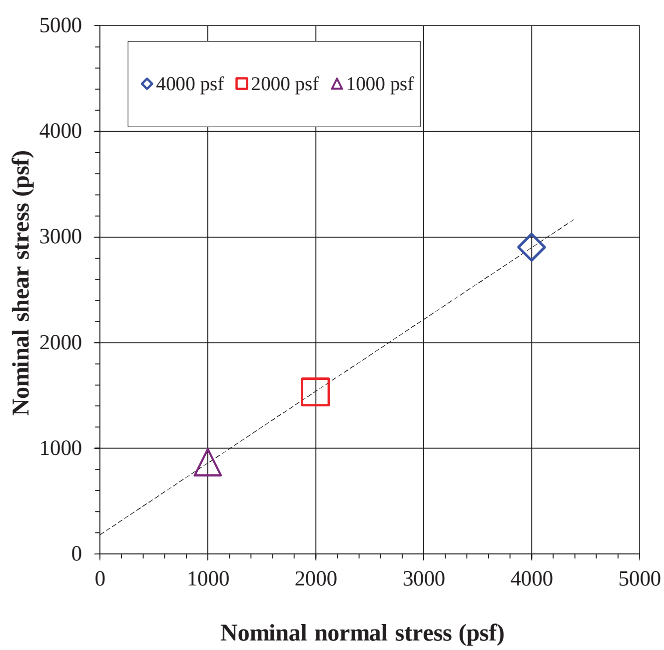
	Sample 1		Sample 2		Sample 3	
Nominal normal stress (psf)	4000		2000		1000	
Peak shear stress (psf)	2903		1531		867	
Lateral displacement at peak (in)	0.297		0.147		0.282	
Load Duration (min)	771		779		6	
	Initial	Pre-shear	Initial	Pre-shear	Initial	Pre-shear
Sample height (in)	0.9970	0.9459	1.0090	0.9882	1.0080	0.9897
Sample diameter (in)	2.412	2.412	2.396	2.396	2.394	2.394
Wt. rings + wet soil (g)	189.35	191.33	193.58	194.64	185.94	191.17
Wt. rings (g)	46.53	46.53	44.30	44.30	45.34	45.34
Wet soil + tare (g)	354.14		354.14		354.14	
Dry soil + tare (g)	313.79		313.79		313.79	
Tare (g)	126.61		126.61		126.61	
Water content (%)	21.6	23.2	21.6	22.4	21.6	26.1
Dry unit weight (pcf)	98.3	103.5	102.8	105.0	97.1	98.9
Void ratio, e, for assumed Gs	0.72	0.63	0.64	0.61	0.74	0.70
Saturation (%)*	81.3	100.0	91.1	100.0	79.1	100.0

ϕ' (deg)	34
c' (psf)	181

Average of 3 samples	Initial	Pre-shear
Water content (%)	21.6	23.9
Dry unit weight (pcf)	99.4	102.4

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





Entered by:___________
Reviewed:___________

Direct Shear Test for Soils Under Drained Conditions

(ASTM D3080)

Project: Kimmelman/ May Lot 16 **Boring No.:** TP-1

No: 02529-001 **Sample:**

Location: Summit Powder Mtn. **Depth:** 16.0'

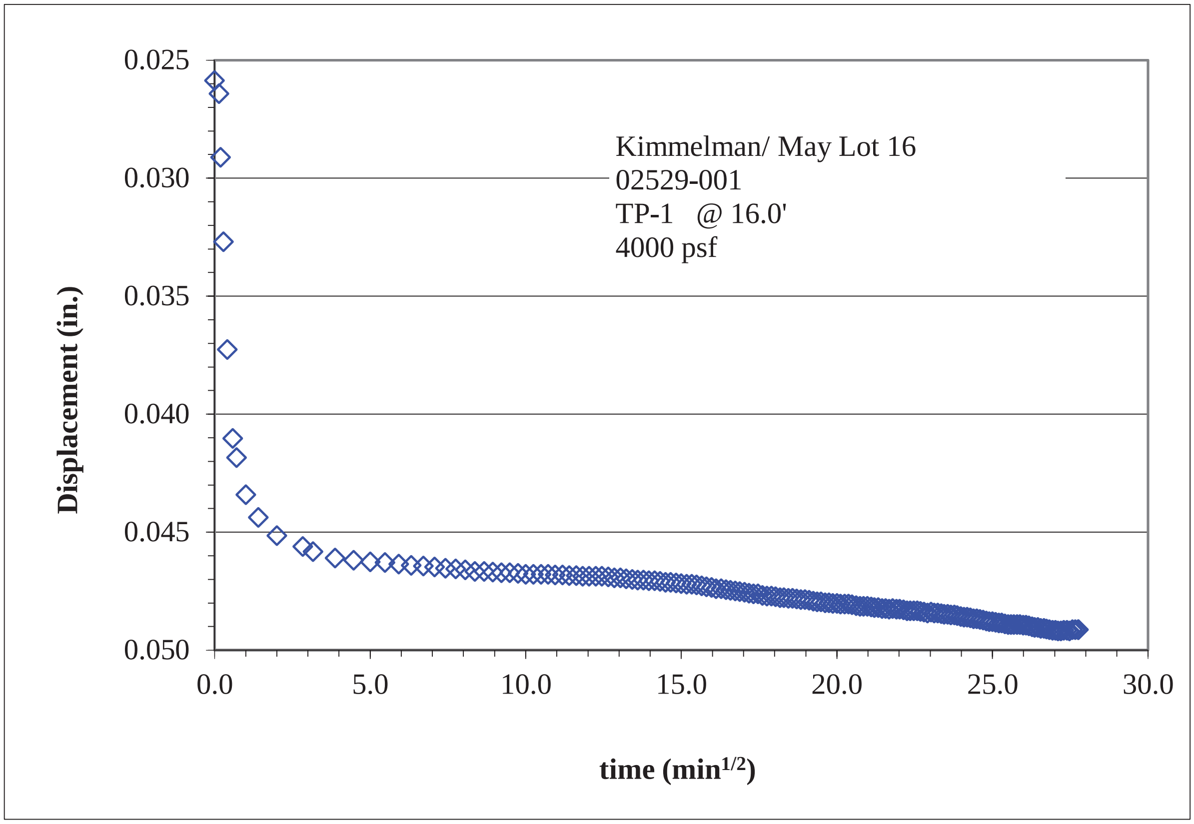
Nominal normal stress = 4000 psf			Nominal normal stress = 2000 psf			Nominal normal stress = 1000 psf		
Lateral Displacement (in.)	Nominal Shear Stress (psf)	Normal Displacement (in.)	Lateral Displacement (in.)	Nominal Shear Stress (psf)	Normal Displacement (in.)	Lateral Displacement (in.)	Nominal Shear Stress (psf)	Normal Displacement (in.)
0.002	219	0.000	0.002	158	0.000	0.002	234	-0.001
0.005	295	-0.001	0.005	219	-0.001	0.005	300	-0.001
0.007	463	-0.001	0.007	357	-0.001	0.007	347	-0.002
0.010	575	-0.002	0.010	478	-0.001	0.010	382	-0.002
0.012	663	-0.003	0.012	559	-0.002	0.012	446	-0.001
0.017	859	-0.003	0.017	738	-0.002	0.017	517	-0.002
0.022	1071	-0.004	0.022	873	-0.002	0.022	561	-0.002
0.027	1202	-0.006	0.027	981	-0.003	0.027	608	-0.002
0.032	1341	-0.006	0.032	1071	-0.003	0.032	643	-0.003
0.037	1482	-0.007	0.037	1131	-0.003	0.037	673	-0.004
0.042	1603	-0.008	0.042	1186	-0.004	0.042	697	-0.004
0.047	1728	-0.009	0.047	1228	-0.004	0.047	712	-0.004
0.052	1843	-0.009	0.052	1266	-0.004	0.052	727	-0.004
0.057	1936	-0.010	0.057	1298	-0.004	0.057	742	-0.005
0.062	2023	-0.012	0.062	1332	-0.005	0.062	757	-0.005
0.067	2110	-0.012	0.067	1361	-0.005	0.067	765	-0.005
0.072	2175	-0.013	0.072	1387	-0.005	0.072	774	-0.005
0.077	2237	-0.013	0.077	1405	-0.005	0.077	779	-0.005
0.082	2299	-0.013	0.082	1424	-0.005	0.082	786	-0.005
0.087	2347	-0.014	0.087	1442	-0.005	0.087	799	-0.005
0.092	2404	-0.014	0.092	1454	-0.005	0.092	805	-0.005
0.097	2448	-0.015	0.097	1464	-0.005	0.097	813	-0.005
0.102	2493	-0.015	0.102	1468	-0.006	0.102	813	-0.005
0.107	2534	-0.016	0.107	1474	-0.006	0.107	819	-0.005
0.112	2568	-0.016	0.112	1488	-0.006	0.112	819	-0.005
0.117	2595	-0.016	0.117	1503	-0.006	0.117	822	-0.005
0.122	2617	-0.017	0.122	1514	-0.006	0.122	825	-0.005
0.127	2643	-0.017	0.127	1520	-0.006	0.127	828	-0.005
0.132	2659	-0.018	0.132	1524	-0.007	0.132	829	-0.006
0.137	2677	-0.018	0.137	1527	-0.007	0.137	832	-0.006
0.142	2686	-0.018	0.142	1529	-0.007	0.142	838	-0.006
0.147	2698	-0.019	0.147	1531	-0.007	0.147	837	-0.006
0.152	2712	-0.019	0.152	1530	-0.007	0.152	839	-0.006
0.157	2724	-0.019	0.157	1531	-0.008	0.157	842	-0.006
0.162	2738	-0.020	0.162	1529	-0.008	0.162	845	-0.006
0.167	2745	-0.020	0.167	1527	-0.008	0.167	845	-0.006
0.172	2748	-0.020	0.172	1522	-0.008	0.172	848	-0.006
0.177	2751	-0.021	0.177	1519	-0.009	0.177	849	-0.006
0.182	2767	-0.021	0.182	1519	-0.009	0.182	849	-0.006
0.187	2772	-0.021	0.187	1518	-0.009	0.187	849	-0.006
0.192	2780	-0.021	0.192	1517	-0.009	0.192	848	-0.006
0.197	2791	-0.022	0.197	1517	-0.009	0.197	849	-0.006
0.202	2799	-0.022	0.202	1517	-0.009	0.202	849	-0.006
0.207	2806	-0.022	0.207	1514	-0.009	0.207	851	-0.006
0.212	2809	-0.022	0.212	1507	-0.010	0.212	850	-0.006
0.217	2818	-0.022	0.217	1503	-0.010	0.217	855	-0.007
0.222	2821	-0.022	0.222	1500	-0.010	0.222	855	-0.006
0.227	2830	-0.023	0.227	1501	-0.010	0.227	857	-0.006
0.232	2836	-0.023	0.232	1497	-0.011	0.232	853	-0.006
0.237	2836	-0.023	0.237	1496	-0.011	0.237	858	-0.007
0.242	2843	-0.023	0.242	1495	-0.011	0.242	860	-0.007
0.247	2852	-0.023	0.247	1492	-0.011	0.247	862	-0.007
0.252	2854	-0.024	0.252	1489	-0.011	0.252	864	-0.007
0.257	2860	-0.024	0.257	1480	-0.012	0.257	861	-0.007
0.262	2861	-0.024	0.262	1476	-0.012	0.262	861	-0.007
0.267	2866	-0.024	0.267	1471	-0.012	0.267	864	-0.007
0.272	2876	-0.025	0.272	1468	-0.012	0.272	861	-0.007
0.277	2882	-0.025	0.277	1468	-0.012	0.277	858	-0.007
0.282	2889	-0.025	0.282	1464	-0.012	0.282	867	-0.007
0.287	2893	-0.025	0.287	1462	-0.012	0.287	865	-0.007
0.292	2894	-0.025	0.292	1460	-0.012	0.292	860	-0.007
0.297	2903	-0.025	0.297	1457	-0.013	0.297	859	-0.007
0.298	2901	-0.026	0.298	1456	-0.013	0.300	861	-0.007

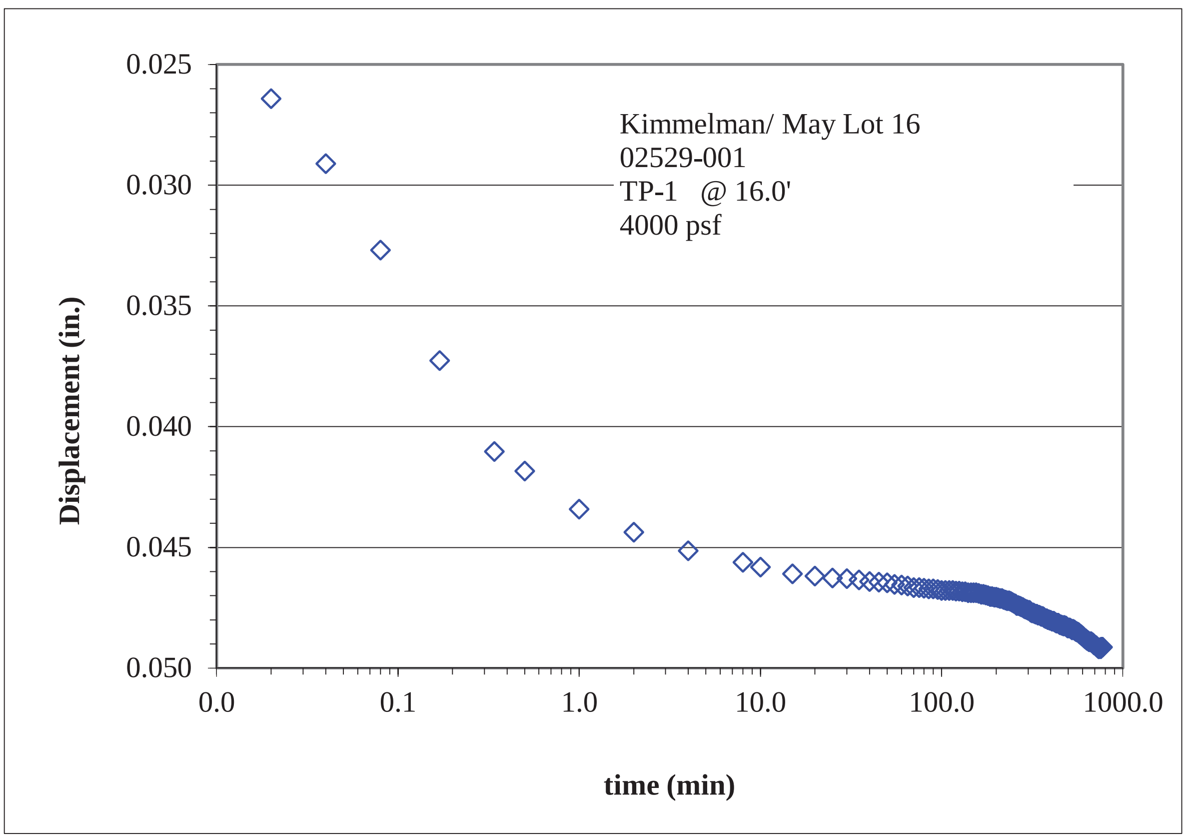
Direct Shear Test for Soils Under Drained Conditions

(ASTM D3080)

Project: Kimmelman/ May Lot 16
No: 02529-001
Location: Summit Powder Mtn.

Boring No.: TP-1
Sample:
Depth: 16.0'





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)*

Project: Kimmelman/ May Lot 16
No: 02529-001
Location: Summit Powder Mtn.
Date: 8/3/2017
By: DKS

Sample info.	Boring No.	TP-1							
	Sample								
	Depth	8.0'							
Water content data	Wet soil + tare (g)	75.07							
	Dry soil + tare (g)	71.30							
	Tare (g)	37.82							
	Water content (%)	11.3							
Chem. data	pH	6.59							
	Soluble chloride* (ppm)	<5.69							
	Soluble sulfate** (ppm)	<5.69							
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	107200	0.67	71824				
		+3	21100	0.67	14137				
		+6	10010	0.67	6707				
		+9	9170	0.67	6144				
		+12	8304	0.67	5564				
		+15	7927	0.67	5311				
		+18	8317	0.67	5572				
	Minimum resistivity (Ω-cm)	**5311**							

* Performed by AWAL using EPA 300.0

** Performed by AWAL using ASTM C1580

Entered by:___________
Reviewed:___________

APPENDIX C

USGS Design Maps Detailed Report

2012/2015 International Building Code (41.3668°N, 111.7623°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 = 0.831 g
From Figure 1613.3.1(2) [2]	S_1 = 0.276 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				
	$S_S \le 0.25$	$S_S = 0.50$	$S_S = 0.75$	$S_S = 1.00$	$S_S \ge 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	See Section 11.4.7 of ASCE 7				

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

For Site Class = C and S_S = 0.831 g, F_a = 1.068

TABLE 1613.3.3(2)
VALUES OF SITE COEFFICIENT F_v

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

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

For Site Class = C and S_1 = 0.276 g, F_v = 1.524

Equation (16-37):	$S_{MS} = F_aS_S = 1.068 \times 0.831 = 0.887$ g
Equation (16-38):	$S_{M1} = F_vS_1 = 1.524 \times 0.276 = 0.421$ g

Section 1613.3.4 — Design spectral response acceleration parameters

Equation (16-39):	$S_{DS} = ⅔\, S_{MS} = ⅔ \times 0.887 = 0.591$ g
Equation (16-40):	$S_{D1} = ⅔\, S_{M1} = ⅔ \times 0.421 = 0.281$ 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} = 0.591 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} = 0.281 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

USGS Design Maps Summary Report

User–Specified Input

Report Title Lot 16R Summit Powder Mtn.
Mon July 17, 2017 21:01:02 UTC

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

Site Coordinates 41.3668°N, 111.7623°W

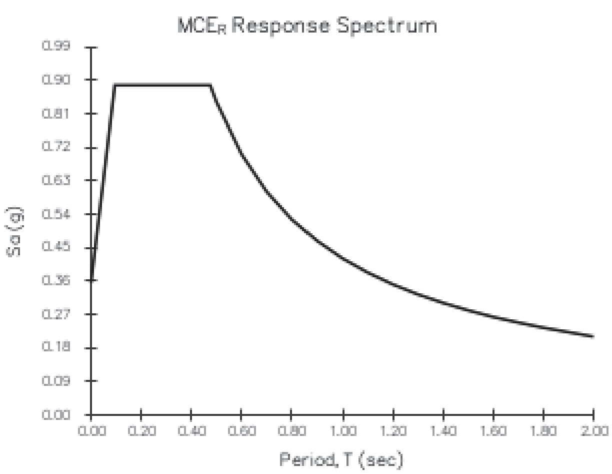
Site Soil Classification Site Class C – "Very Dense Soil and Soft Rock"

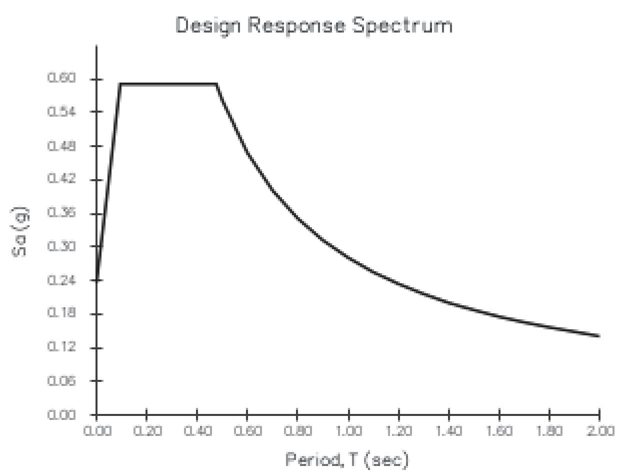
Risk Category I/II/III

USGS–Provided Output

S_S =	0.831 g	S_{MS} =	0.887 g	S_{DS} =	0.591 g
S_1 =	0.276 g	S_{M1} =	0.421 g	S_{D1} =	0.281 g

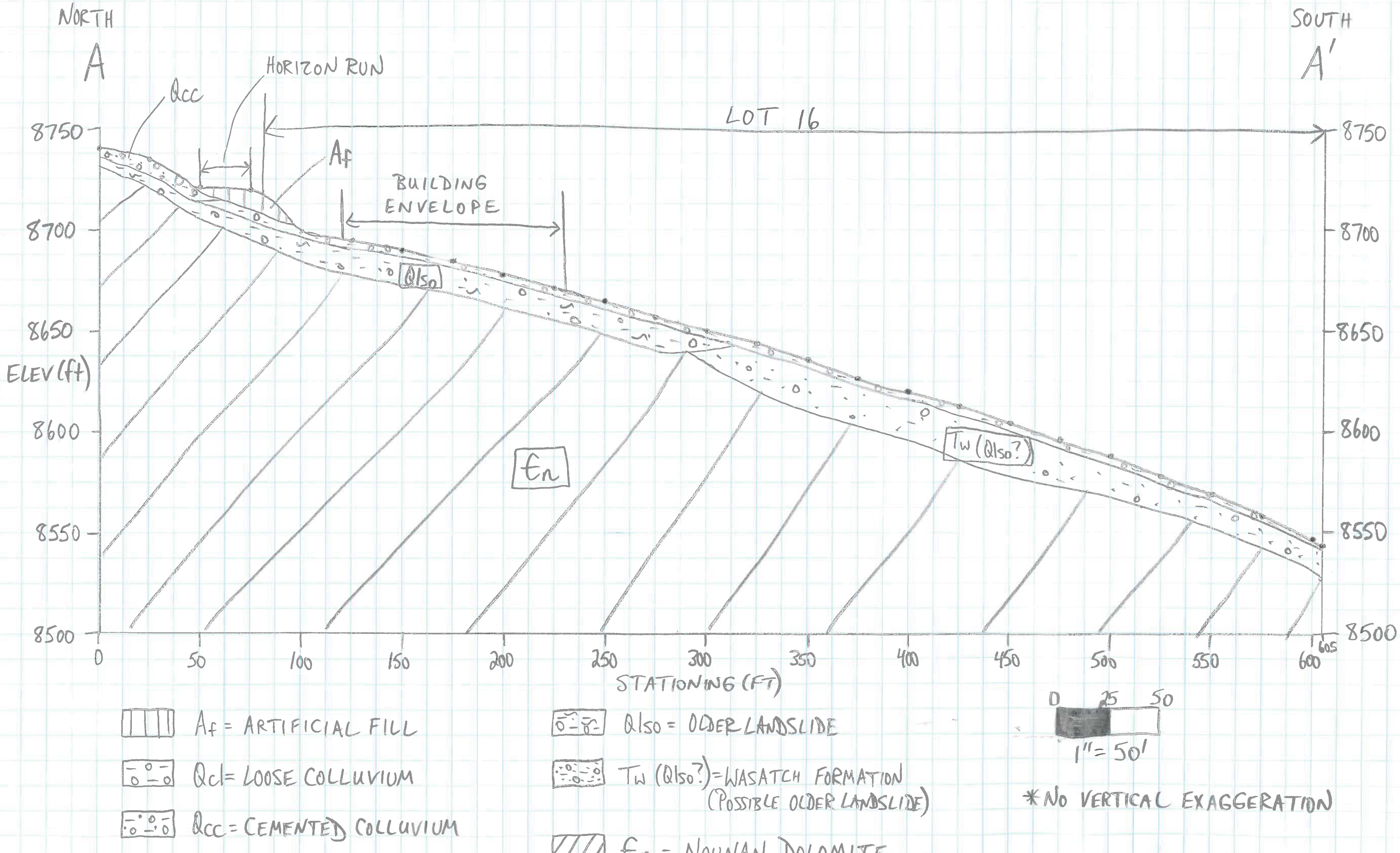
For information on how the SS and S1 values above have been calculated from probabilistic (risk-targeted) and deterministic ground motions in the direction of maximum horizontal response, please return to the application and select the "2009 NEHRP" building code reference document.





APPENDIX D

FIGURE D-1
CROSS-SECTION A-A'



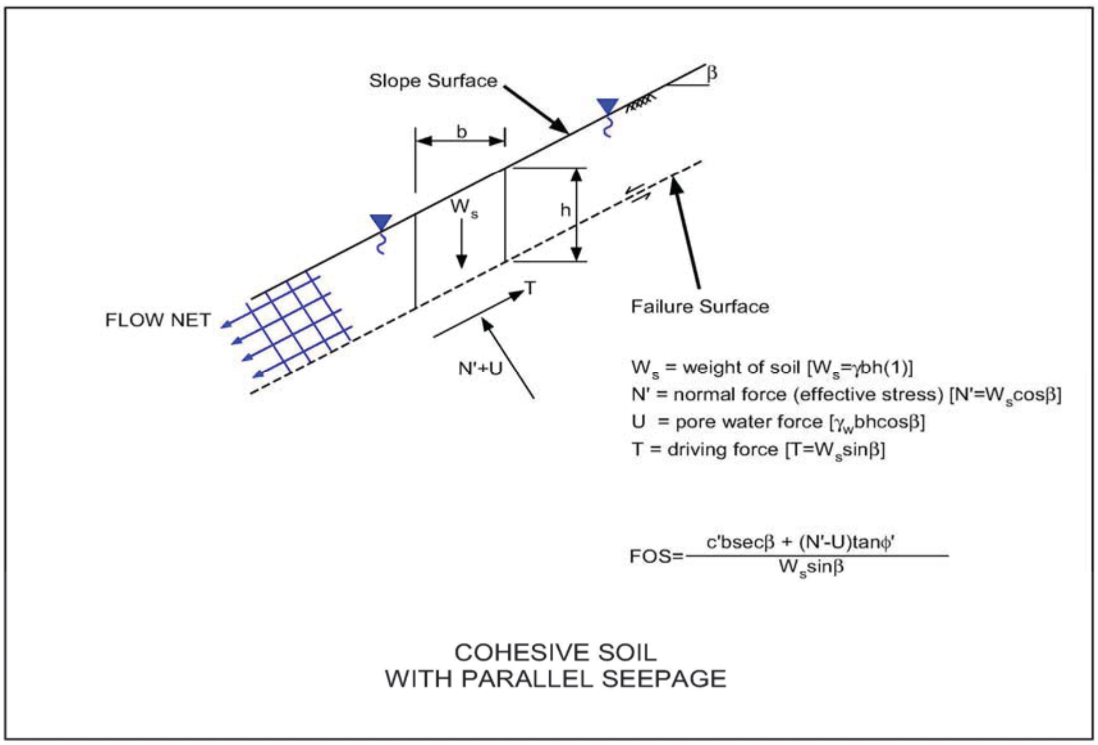
Kimmelman/Lot 16R
02529-001
8/8/2017

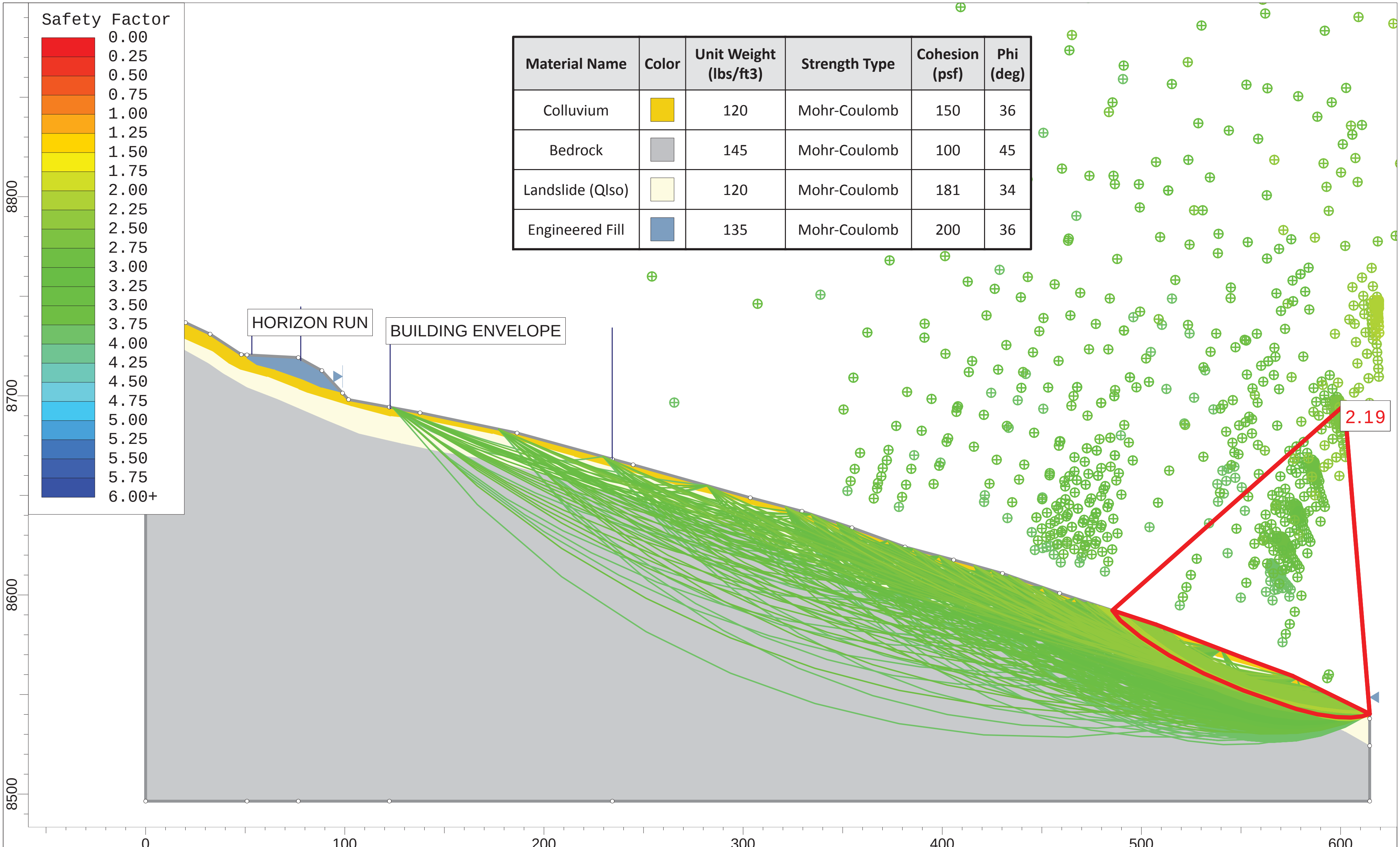
c'	150	psf	Effective Cohesion
ϕ'	28	deg	Effective Friction Angle
Y_{sat}	135	pcf	Saturated Unit Weight of Soil
Y_w	62.4	pcf	Unit weight of water
h	4	ft	Depth to shear surface
β	15.9	deg	Slope Gradient (3.5H:1V)

FS	2.06	

(yellow)	Input Variable
(green)	Calculated Value

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





Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Colluvium		120	Mohr-Coulomb	150	36
Bedrock		145	Mohr-Coulomb	100	45
Landslide (Qlso)		120	Mohr-Coulomb	181	34
Engineered Fill		135	Mohr-Coulomb	200	36

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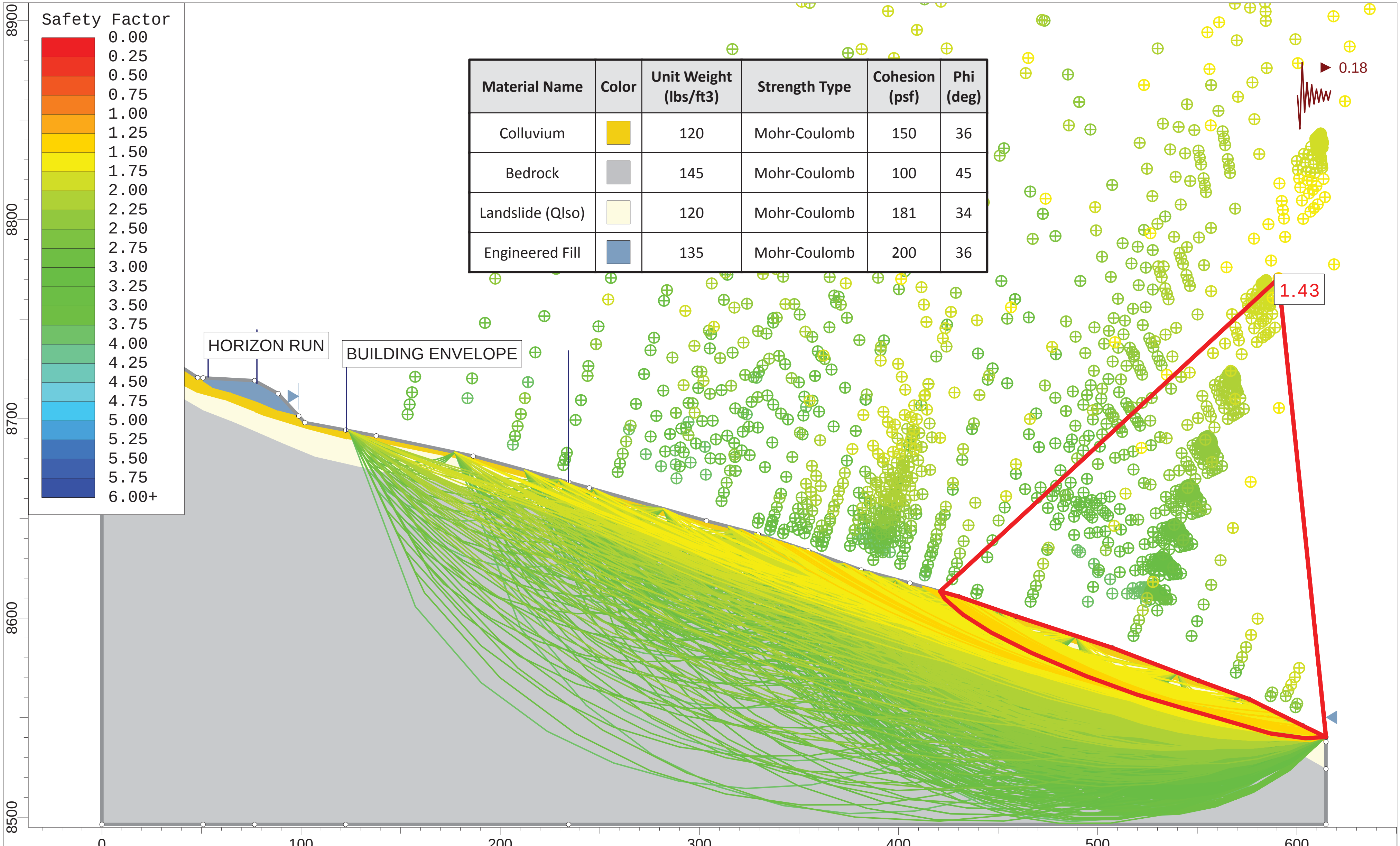
SLIDEINTERPRET 7.025

Project	Lot 16R of Summit Eden Phase 1A		
Analysis Description	Section A-A' - Static		
Drawn By TQH	*Scale* 1:800	*Company*	IGES, Inc.
Date 8-8-2017		*File Name*	Section A-A' Static.slim



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Colluvium		120	Mohr-Coulomb	150	36
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Landslide (Qlso)		120	Mohr-Coulomb	181	34
Engineered Fill		135	Mohr-Coulomb	200	36

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Project	Lot 16R of Summit Eden Phase 1A		
Analysis Description	Section A-A' - Seismic		
Drawn By TQH	*Scale* 1:800	*Company*	IGES, Inc.
Date	8-8-2017	*File Name*	Section A-A' Static.slim

SLIDEINTERPRET 7.025