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GEOTECHNICAL AND GEOLOGIC HAZARD INVESTIGATION
Main Street West
Summit Powder Mountain Resort
Weber County, Utah

IGES Project No. 01628-023

January 18, 2017

Prepared for:

Summit Mountain Holding Group

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1.0 INTRODUCTION

1.1 PURPOSE AND SCOPE OF WORK

This report presents the results of a geotechnical and geologic hazard investigation conducted for the *Main Street West* mixed-use development, 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 to Summit Mountain Holding Group (Client), dated October 20, 2016. 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 3-page conceptual drawings prepared by Studio MA and conceptual drawings prepared by EB5 Development, 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. The *Main Street West* development will be located within the Phase 1C area, between Summit Pass Road and Copper Crest Road (see *Site Vicinity Map*, Figure A-1 and *Geotechnical & Geologic Map*, Figure A-2, in Appendix A). The approximately 0.55-acre *Main Street West* project will consist of three main structures:

- 2A – Clubhouse, may include a community center, library, restaurants, a spa, a small grocery store, or some retail,
- 2B – Townhouses/Retail, may include townhomes, rental units, restaurants, or shops,
- 2C – Townhouses/Retail, may include townhomes, rental units, restaurants, and shops.

The structures will be multi-story, between 2 and 3 stories above-ground with perhaps as much as 5 stories for 2B and 2C; the structures will also have a basement level, most likely for underground parking and/or workshops.

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 Main Street West 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. 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.

2.3 LABORATORY TESTING

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

- Atterberg Limits (ASTM D4318)
- Grain-Size Distribution (ASTM D6913)
- Fines Content (ASTM D1140)
- In situ Moisture Content (ASTM D2216)
- Soluble Sulfate, Soluble Chloride, pH and Resistivity (AASHTO T 288, T 289; ASTM D 4327 and C1580)

Results of the laboratory testing are discussed in this report and presented in Appendix B. Some test results, including moisture content 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 Main Street West 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 Main Street West property is located approximately 10.25 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 Crittenden, Jr. (1972), the property is entirely underlain by the undivided Tertiary/Cretaceous Wasatch and Evanston Formations (TKwe), described as “unconsolidated pale-red to greenish-red pebble, cobble, and boulder conglomerate. Forms boulder-covered slopes but does not crop out anywhere. Clasts are mainly Precambrian quartzite and are tan, gray, or purple; matrix is mainly poorly consolidated sand and silt.” A generalized bedding attitude shows this unit striking due north and dipping 10 degrees to the east; this map forms the basemap for the *Regional Geology Map 1* (Figure A-6). Coogan and King (2001) produced a regional-scale geologic map that covered the property; this map shows the property to be entirely underlain by the Wasatch Formation. Western Geologic (2012) identified a number of landslide deposits contained within the Powder Mountain Resort expansion area, though none of these were shown underlying the Main Street West area (Figure A-7). Deposits mapped as “mixed slope colluvium, shallow landslides, and talus” are found southwest of the property. Finally, Coogan and King (2016) updated their 2001 map, which shows the property to be straddling the contact between the northeasternmost reach of a lobe of landslide deposits (unit Qms) and the Wasatch Formation (unit Tw) (Figure A-8). Wasatch Formation bedrock in the area is shown to be striking approximately to the north-northeast, and dipping between 3 and 6 degrees to the east-southeast; additionally, according to this map, the property is adjacent to a north-south trending concealed syncline[1].

3.3 HYDROLOGY

The USGS topographic maps for the Huntsville and Brown’s Hole Quadrangles (2014) show that the Main Street West project area is situated on a slope, with the topographic gradient down to the southwest towards Lefty’s Canyon (see Figure A-1). No active or ephemeral stream drainages are found on the property, and no gullies were observed during the site reconnaissance. 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 Main Street West property are currently unknown, but are anticipated to fluctuate both seasonally and annually. At the time of our subsurface exploration, seepage was observed at a depth of 12 feet in TP-1. This seepage is likely part of a localized perched aquifer and is not expected to represent the local piezometric groundwater surface.

3.4 GEOLOGIC HAZARDS FROM LITERATURE

Based upon the available geologic literature, regional-scale geologic hazard maps that cover the Main Street West 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.

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[1] Syncline: A fold of which the core contains the stratigraphically younger rocks; it is generally concave upward. (AGI, 2005)

3.4.1 Landslides

Two regional-scale landslide hazard maps have been produced that cover the project area. Colton (1991) does not show the property to be underlain by or adjacent to landslide deposits. Elliott and Harty (2010) shows deposits mapped as "Landslide undifferentiated from talus and/or colluvial deposits" southwest of the property. Most recently and more site-specific, Western Geologic (2012) used the Elliott and Harty (2010) map as a base map, which shows "mixed slope colluvium, shallow landslides, and talus" deposits southwest of the property (see Figure A-7).

3.4.2 Faults

Neither Christensen and Shaw (2008a) nor the Quaternary Fault and Fold Database of the United States (USGS and UGS, 2006) show any Quaternary-aged (~2.6 million years ago to the present) faults to be present on or projecting towards the subject property. The Weber County Natural Hazards Overlay Districts defines an active fault to be "a fault displaying evidence of greater than four inches of displacement along one or more of its traces during Holocene time (about 11,000 years ago to the present)" (Weber County, 2015). The closest active fault to the property is the Weber Segment of the Wasatch Fault Zone, located approximately 10.25 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 2016 were also reviewed. No landslide or other geological hazard features were noted in the imagery, though irregular topography coinciding with possible landslide deposits was observed south of Spring Park Road. The property was observed to contain some surficial gravel, cobbles, and boulders, and to be devoid of drainages. Most of the project area was found to be fairly densely covered in aspen trees, with no bedrock exposures anywhere on the property. A northeast to southwest-trending gravel road was observed to pass through the middle of the property, which was initially emplaced sometime prior to 1993.

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)	$S_S = 0.810$	$S_1 = 0.269$
MCE Spectral Response Acceleration Site Class C (g)	$S_{MS} = S_sF_a = 0.872$	$S_{M1} = S_1F_v = 0.411$
Design Spectral Response Acceleration (g)	$S_{DS} = S_{MS*}{}^2/_3 = 0.581$	$S_{D1} = S_{M1*}{}^2/_3 = 0.274$

To account for site effects, site coefficients that vary with the magnitude of spectral acceleration and *Site Class* are used. Site Class is a parameter that accounts for site amplification effects of soft soils and is based on the average shear wave velocity of the upper 100 feet; based on our field exploration and our understanding of the geology in this area, the subject site is appropriately classified as Site Class C (*very dense soil/soft rock*). Based on IBC criteria, the short-period (F_a) coefficient is 1.076 and the long-period (F_v) site coefficient is 1.531. Based on the design spectral response accelerations for a *Building Risk Category* of I, II or III, the site's *Seismic Design Category* is D. The short- and long-period *Design Spectral Response Accelerations* are presented in Table 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 affecting 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 Main Street West property.

3.7.1 Landslides/Mass Movement/Slope Stability

The property is situated at the contact between mapped landslides and Wasatch Formation bedrock, according to the most recent geologic map covering the property (Coogan and King, 2016). However, other literature sources, including the Western Geologic (2012) reconnaissance-level geologic hazard assessment for the greater Powder Mountain area and Elliott and Harty (2010), do not show the property to be underlain by landslide deposits, but near deposits mapped as landslide or colluvial deposits southwest of the property. Additionally, landslide deposits or headscarps were not observed in the aerial imagery evaluation on or upslope of the property, and no geomorphic expression of landslide deposits or headscarps were observed on or upslope of the property during the site reconnaissance. Though evidence of soil creep was observed in the aspen trees found on the property, the subsurface data indicate that this is restricted to the topsoil.

The average slope across the property is found to be approximately 6:1 (horizontal:vertical), which does not require site-specific slope stability analyses. Given this data, the risk associated with landslide and slope stability hazards on the property is considered to be low.

3.7.2 Rockfall

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

3.7.3 Surface-Fault Rupture and Earthquake-Related Hazards

No faults are known to be present on or project across the property, and the closest active fault to the property is the Weber Segment of the Wasatch Fault Zone, located approximately 10.25 miles to the west of the property (USGS and UGS, 2006). Given this information, the risk associated with surface-fault-rupture on the property is considered low.

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

3.7.4 Liquefaction

The site is underlain by 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 southwest. 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 encountered in TP-1 at a depth of 12 feet below existing grade. The test pits were excavated in early November, and the groundwater level was likely to be on its way down towards its seasonal low. No springs were observed on the property, and no plants indicative of shallow groundwater conditions were observed on the property.

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 high. Spring thaw and runoff are likely to significantly contribute to elevated groundwater conditions, especially if groundwater levels are still within 20 feet of existing grade in November. 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 November 2, 2016. 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 Main Street West property and adjacent areas.

At the time of the site reconnaissance, the property was observed to be sloping to the southwest, with the steepness of the slope decreasing downslope to the west. The property was largely covered in aspen trees, except where the preexisting gravel road seen in the aerial imagery passed through the middle of the property.

Variously-sized boulders and cobbles were found scattered across the property, as part of a surficial geologic unit considered to be either weathered Wasatch Formation or colluvial deposits derived from weathered Wasatch Formation. These were typically subrounded, and were found to be as large as 2 feet in diameter. The rock clasts[2] were found to be comprised predominantly of banded to massive purple quartzite, though some dark gray sandy limestone and yellowish-orange micaceous sandstone clasts were also observed; these latter lithologies were considered to be possibly indicative of near-surface Cambrian bedrock units or leftover material used in the construction of Summit Pass Road.

No drainages, gullies, springs, seeps, or running water were observed on the property at the time of the site visit. Aside from shallow soil creep, most conspicuously evidenced in the aspens in the northern part of the property, no evidence of landsliding or other geologic hazards was observed on the property. Potential localized landslide features were observed to the southwest of the property (see Figure A-2).

4.2 SUBSURFACE CONDITIONS

On November 3, 2016, two exploration test pits were excavated at representative locations across the property (Figure A-2). The test pits were excavated with to depths ranging between 9½ and 12 feet below existing grade with the aid of a Caterpillar 313F tracked excavator. Detailed logs for the test pits are displayed in Figure A-3 and Figure A-4. Four distinct geologic units were encountered in the subsurface, with two of these units being found in both of the test pits. The soil

[2] 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)

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 approximately 2 to 3 feet thick in both test pits. The unit was a dark brown, loose, moist, sandy lean CLAY with gravel (CL), with gravel and larger-sized quartzite clasts comprising approximately 25% of the unit. The topsoil was found to be forming upon the underlying colluvium unit.

Loose Colluvium: This unit was only observed in TP-1, and was found to be approximately 3 to 5 feet thick. The unit consisted of a dark reddish brown, loose, moist, sandy lean CLAY with gravel (CL). Gravel and larger-sized subrounded quartzite clasts comprised approximately 35% of the unit, with individual clasts up to 7 inches in diameter.

Cemented Colluvium: This unit was only observed in TP-2, and was found to be approximately 3 to 4 feet thick. The unit consisted of a pale reddish brown, stiff to very stiff, slightly moist, silty, clayey SAND (SC-SM). Gravel and larger-sized subrounded quartzite clasts comprised approximately 30% of the unit, with individual clasts up to 6 inches in diameter. The unit was observed to contain abundant pinhole voids (1 mm diameter).

Wasatch Formation: This unit was found to underlie the colluvial unit in both test pits, being at least 7 feet thick and extending to the maximum depth of exploration in both test pits. The unit consisted of weakly consolidated conglomerate bedrock that had been largely disaggregated into a heterogeneous moderate reddish brown to dark reddish brown, dense to very dense, slightly moist to wet mixture of clay, sand, and gravel that classifies as clayey GRAVEL with sand (GC). Gravel and larger-sized subrounded quartzite clasts comprised approximately 35% of the unit, with individual clasts up to 5 inches in diameter and a mode clast size of 1 to 2 inches. In TP-1, an alluvial subunit was observed near the top of the unit, consisting of pale reddish brown clayey GRAVEL with sand (GC) gradational to clayey SAND with gravel (SC). The alluvial subunit clast composition was as much as 60% of the unit, and contained clasts up to 1.5 feet in diameter.

4.2.2 Groundwater

Groundwater was encountered in TP-1; water was observed seeping out of the lower exposed portion of the Wasatch Formation at a depth of 12 feet below existing grade, and ultimately leveled-off at a depth of approximately 11 feet below existing grade. In TP-2, no groundwater was observed, although it should be noted that the maximum depth of the excavation was 9.5 feet below existing grade in TP-2.

5.0 CONCLUSIONS AND RECOMMENDATIONS

5.1 GENERAL CONCLUSIONS

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

Supporting data upon which the following 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 Main Street West project area:

- **The Main Street West project area does not appear to have major geological hazards that would adversely affect the development as currently proposed.**

- Shallow groundwater conditions were observed on the property in TP-1, despite the excavations occurring in November; therefore, shallow groundwater hazards are considered to be high for the property.

- 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.

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

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

- Because landslide deposits are noted near the property, an IGES geologist or geotechnical engineer should observe the foundation excavations to confirm the absence of landside deposits.

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 waterdensified in-place (jetting). Alternatively, pipe bedding and shading may consist of clean ¾-inch gravel, which generally does not require densification. Native earth materials can be used as backfill over the pipe bedding zone. All utility trenches backfilled below pavement sections, curb and gutter, and hardscape, should be backfilled with structural fill compacted to at least 95 percent of the MDD as determined by ASTM D-1557. All other trenches should be backfilled and compacted to approximately 90 percent of the MDD (ASTM D-1557). However, in all cases the pipe bedding and shading should meet the design criteria of the pipe manufacturer. Specifications from governing authorities having their own precedence for backfill and compaction should be followed where they are more stringent.

5.4 FOUNDATION RECOMMENDATIONS

Based on our field observations and considering the presence of relatively competent native earth materials, we recommend that the footings for proposed mult-use structures be founded either *entirely* on competent native soils <u>or</u> *entirely* on structural fill. Native/fill transition zones are not allowed. If soft, loose, or otherwise deleterious earth materials are exposed in the footing excavations, then all footings must be deepened such that all footings bear on relatively uniform, competent native earth materials. Alternatively, the foundation excavation may be over-excavated a minimum of 2 feet below the bottom of proposed footings and replaced with structural fill, such that the footings bear entirely on a uniform fill blanket. We recommend that IGES assess the bottom of the foundation excavation prior to the placement of steel or concrete 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.

Considering the structures will have a subterranean level (parking basements), we anticipate the structural foundations will be placed at least 8 feet below existing grade upon earth materials classifying as bedrock, or Wasatch Formation. Accordingly, shallow spread or continuous wall footings constructed entirely on competent, uniform native earth materials or on a minimum of 2 feet of *structural fill* may be proportioned utilizing a maximum net allowable bearing pressure of **4,600 pounds per square foot (psf)** for dead load plus live load conditions. The net allowable bearing value presented above is for dead load plus live load conditions. The minimum recommended footing width is 20 inches for continuous wall footings and 30 inches for isolated spread footings. The maximum recommended footing width is 12 feet for continuous wall footings and 20 feet for isolated spread footings. For appurtenant structures where the footing will be placed near-grade (e.g. no basement), the allowable bearing capacity should be reduced by half.

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.

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	35	0.53	56
At-rest (Ko)	0.50	55	0.80	85
Passive (Kp)	3.0	320	—	—

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

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’’ W4.0×W4.0 welded wire mesh within the middle third of the slab. We recommend that concrete be tested to assess that the slump and/or air content is in compliance with the plans and specifications. We recommend that concrete be placed in general accordance with the requirements of the American Concrete Institute (ACI). A Modulus of Subgrade Reaction of **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 multi-use 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, particularly around basement walls. Additionally, the ground surface within 10 feet of the structure 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.

IGES recommends a perimeter foundation drain be constructed in accordance with the International Residential Code (IRC).

5.9 SOIL CORROSION POTENTIAL

To evaluate the corrosion potential of concrete in contact with onsite native soil, a representative soil sample was tested in our soils laboratory for soluble sulfate content. Laboratory test results indicate that the sample tested had a sulfate content of 25 ppm. Based on this result, the onsite native soils are expected to exhibit a low potential for sulfate attack to concrete. Conventional Type I/II 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 representative soil sample was tested in our soils laboratory for soil resistivity (AASHTO T288), chloride content, and pH. The tests indicated that the onsite soil tested has minimum soil resistivity of 42,424 OHM-cm, a chloride content of 5.3 ppm, and a pH value of 7.8. Based on these results, the onsite native soil is considered *mildly corrosive* to ferrous metal.

5.10 CONSTRUCTION CONSIDERATIONS

5.10.1 Temporary Shoring

Temporary shoring may be required during excavation of the basement/parking levels to protect existing improvements, particularly if utilities have been installed in adjacent streets that preclude the possibility of laying-back the slope.

5.10.2 Over-Size Material

Large boulders (up to 24 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.

5.10.3 Groundwater

Water was encountered seeping into TP-1 at a depth of 12 feet below existing grade; this water most likely represents a localized perched aquifer and constitutes local seepage rather than a piezometric groundwater surface. Nevertheless, water seepage could conceivably impact the proposed development, particularly during construction of the foundations; seepage could cause equipment mobility problems, and could cause localized excavation instability. The Contractor should be aware that shoring and/or localized dewatering may be necessary during construction of the foundations, particularly during spring and early summer.

6.0 CLOSURE

6.1 LIMITATIONS

The recommendations presented in this report are based on limited field exploration, review of existing hazard studies and other geotechnical data, and our understanding of the proposed construction. The subsurface data used in the preparation of this report were obtained from the explorations made for this investigation. It is possible that variations in the soil and groundwater conditions could exist between and beyond the points explored. The nature and extent of variations may not be evident until construction occurs. If any conditions are encountered at this site that are different from those described in this report, we should be immediately notified so that we may make any necessary revisions to recommendations contained in this report. In addition, if the scope of the proposed construction changes from that described in this report, IGES should also be notified.

This report was prepared in accordance with the generally accepted standard of practice at the time the report was written. No warranty, expressed or implied, is made.

It is the Client's responsibility to see that all parties to the project including the Designer, Contractor, Subcontractors, etc. are made aware of this report in its entirety. The use of information contained in this report for bidding purposes should be done at the Contractor's option and risk.

6.2 ADDITIONAL SERVICES

The recommendations made in this report are based on the assumption that an adequate program of tests and observations will be made during the construction. IGES staff or other qualified personnel should be on site to verify compliance with these recommendations. These tests and observations should include at a minimum the following:

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

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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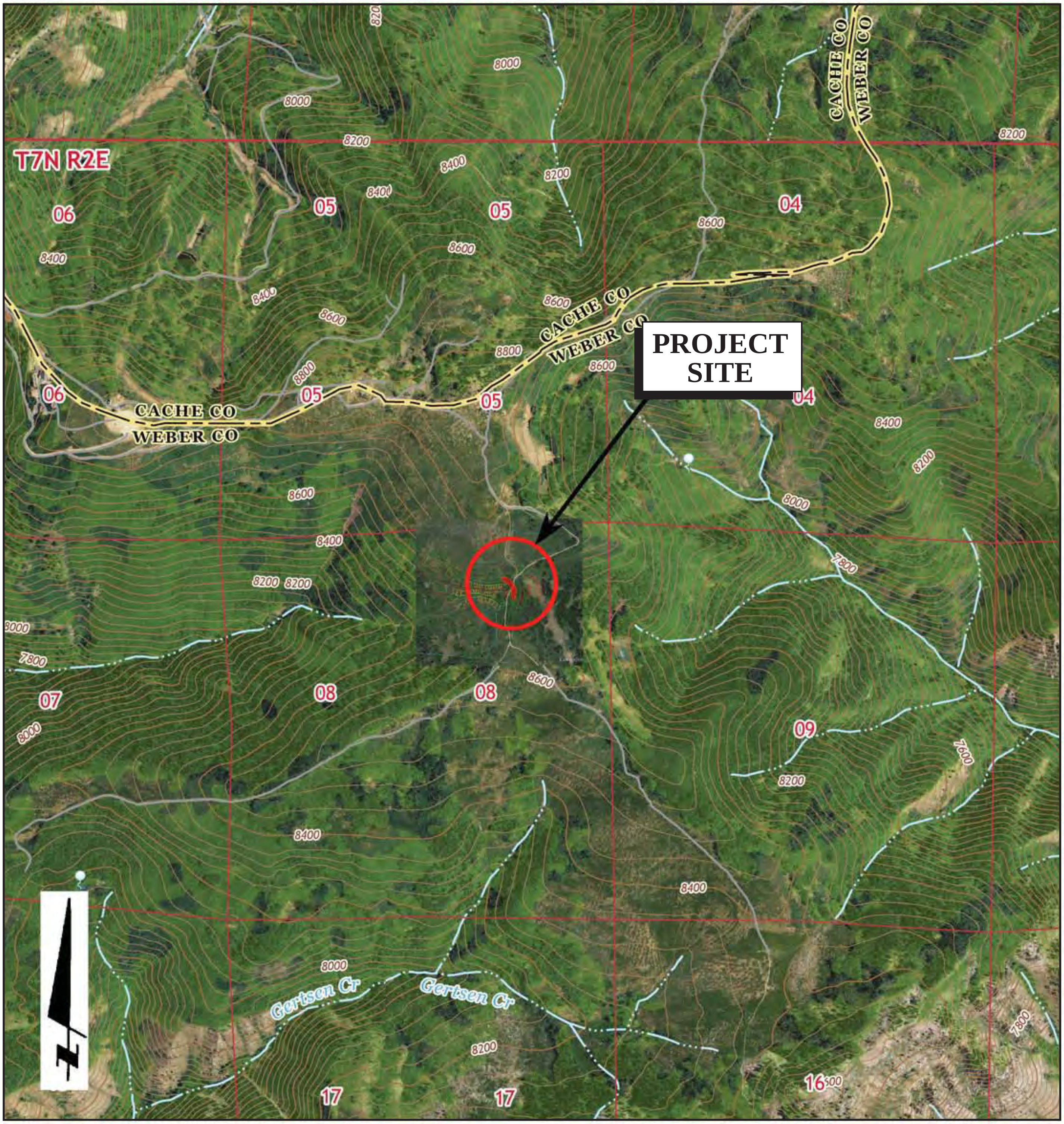
AERIAL PHOTOGRAPHS

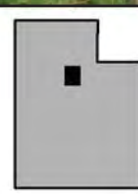
Data Set	Date	Flight	Photographs	Scale
1947 AAJ	August 10, 1946	AAJ_1B	88, 89, 90	1:20,000
1953 AAI	September 14, 1952	AAI_4K	34, 35, 36	1:20,000
1963 ELK	June 25, 1963	ELK_3	57, 58, 59	1:15,840

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

APPENDIX A



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



MAP LOCATION

0 1000' 2000'

SCALE 1:24,000

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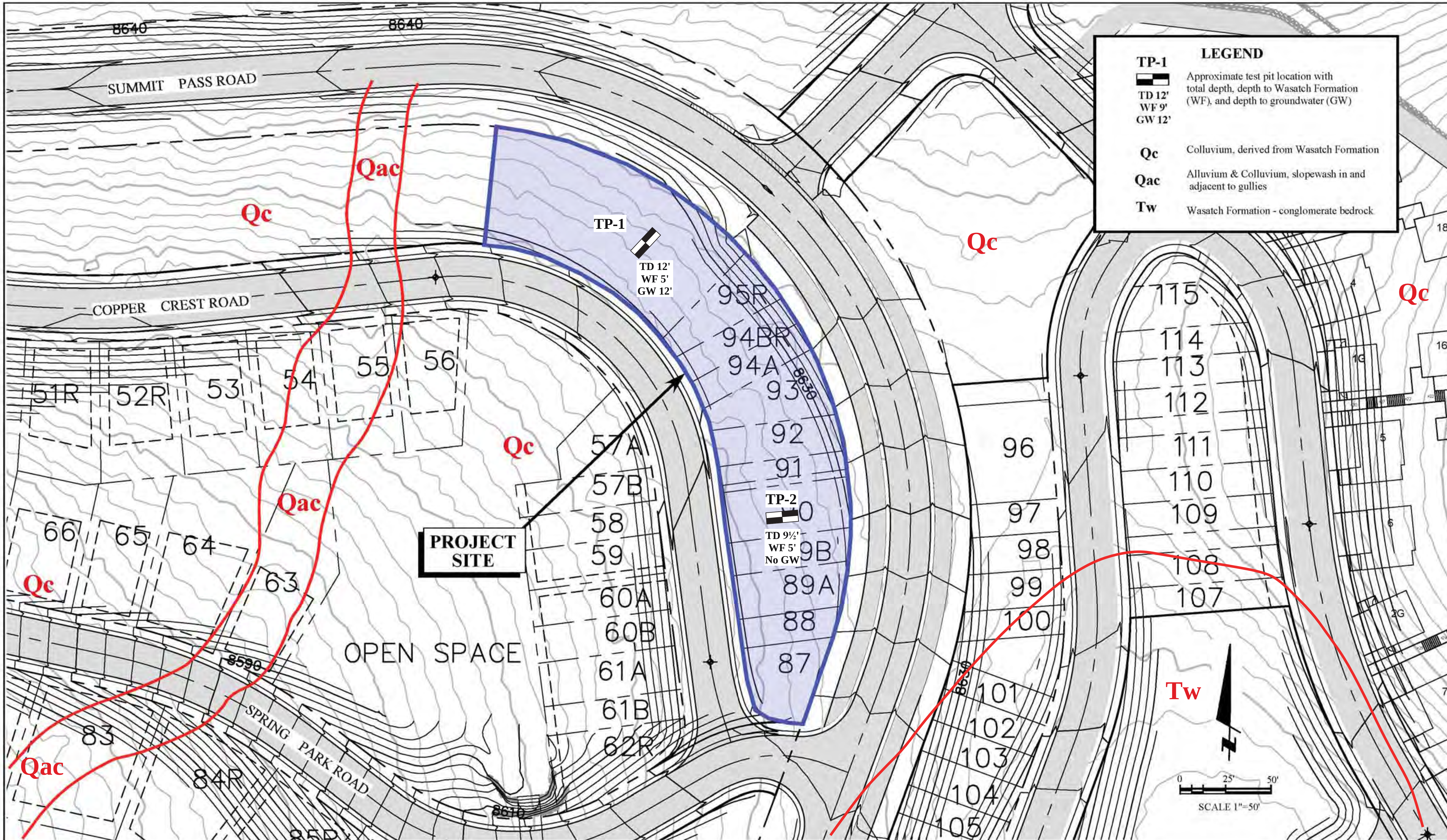
Summit Powder Mountain Resort
Weber County, Utah

SITE VICINITY MAP

Figure

A-1



Basemap: Undated 50-scale site plan prepared by NV-5

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GEOTECH & GEOLOGY MAP

Figure

A-2

IGES

Intermountain GeoEnvironmental Services, Inc

Project No. 01628-023

Date 11/3/16 by ET

Ckd by PD on 11/7/16

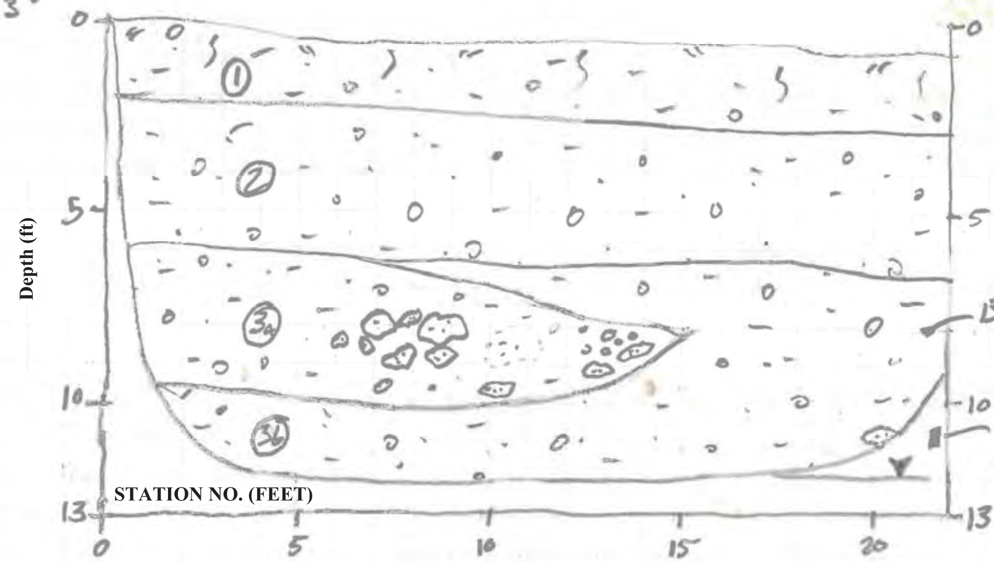
Main ST West
TP-1

S Wall →
223°

WPT 645 1505
N41.36327
W111.74448

Max depth: 12'

GW encountered
@ base of Pit.
(12')



Bag @ 7'

Bucket Taken from spoils

*Groundwater filled up test pit to a depth of 11 feet below existing grade while logging on 11/03/16

0-12'
CLAYEY GRAVEL W/ SAND (GC)
G: 35.2%
S: 34.2%
F: 30.6%

TOTAL DEPTH 12 FEET
GW AT 11 FEET

LITHOLOGIC UNIT DESCRIPTIONS:

1. A/B Soil Horizon: ~2' thick topsoil; dark yellowish brown (10YR 4/2) sandy lean CLAY with gravel (CL), loose, moist, low plasticity, massive; gravel and larger sized clasts comprise ~25% of unit; clasts entirely subangular quartzite up to 6" in diameter, though mode size ~1"; abundant plant and tree roots; gradational, planar basal contact.

2. Loose Colluvium: ~3-5' thick; dark reddish brown (10R 3/4) sandy lean CLAY with gravel (CL), loose, moist, low plasticity, massive; gravel and larger sized clasts comprise ~35% of unit; clasts entirely subangular quartzite up to 7" in diameter, though mode size is ~1-1.5"; sharp, irregular basal contact.

3. Wasatch Fm: Comprised of 2 subunits:
3a: Alluvial: ~3' thick; pale reddish brown (10R 5/4) clayey GRAVEL with sand (GC) gradational to clayey SAND with gravel (SC), medium-dense to dense, slightly moist, massive; gravel and larger sized clasts comprise up to 60% of unit; clasts entirely subangular quartzite up to 1.5' in diameter, though mode size ~2.5"; low plasticity fines; irregular basal contact with Unit 3b.

3b: Bedrock: At least 4' thick; moderate reddish brown (10R 4/6) to dark reddish brown (10R 3/4) conglomerate bedrock largely disaggregated to silty GRAVEL with sand (GM-GC), dense, slightly moist, massive; gravel and larger sized clasts comprise ~35% of unit; clasts entirely subrounded to subangular quartzite up to 5' in diameter, though mode size ~1-2"; low to medium plasticity fines; consistent Wasatch Formation character as seen in other test pits on Powder Mountain.

SCALE: 1"=5' H&V

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Geotechnical & Geologic Hazard Investigation
Main Street West
Summit Powder Mountain Resort
Weber County, Utah

TEST PIT LOG TP-1

Figure

A-3

IGES

Intermountain GeoEnvironmental Services, Inc.

Project No. 01628-023

Date 11/3/16 by ET

Ckd by DD on 11/7/16

Main Street West
TP-2

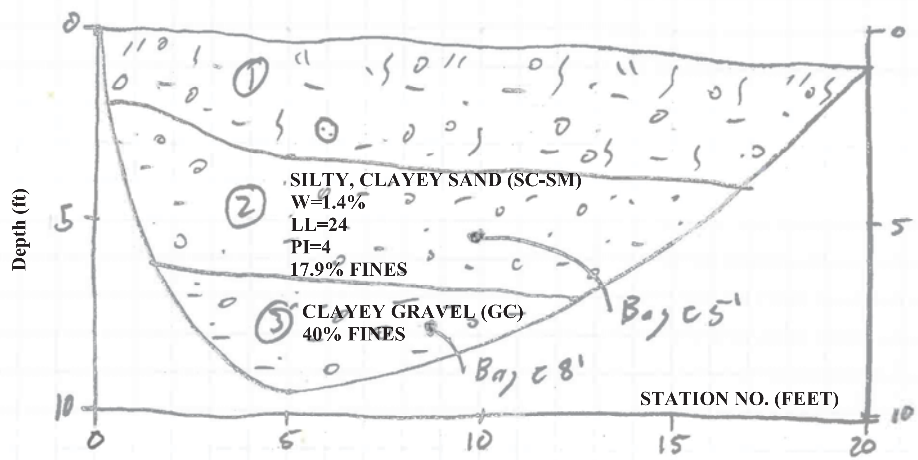
S Wall → 275°

wpt 646 1507
N41.36287
W111.74425

Max depth 9.5'

No GW encountered

Excavator noted hard digging



TOTAL DEPTH 9.5 FEET
NO GW

LITHOLOGIC UNIT DESCRIPTIONS:

1. A/B Soil Horizon: ~2-3' thick topsoil; dark yellowish brown (10YR 4/2) sandy lean CLAY with gravel (CL), loose, moist, low plasticity, massive; gravel and larger sized clasts comprise ~25% of unit; clasts entirely subangular quartzite up to 6" in diameter, though mode size ~1"; abundant plant and tree roots; sharp, planar basal contact.

2. Cemented Colluvium: ~3-4' thick; pale red (10R 6/2) to pale reddish brown (10R 5/4) silty, clayey SAND with gravel (SC-SM), medium dense, slightly moist, low plasticity, massive; gravel and larger sized clasts comprise ~30% of unit; clasts entirely subangular to subrounded quartzite up to 6" in diameter, though mode size is ~1"; occasional to abundant pinhole voids (1 mm diameter); gradational, irregular basal contact.

3. Wasatch Fm: At least 3' thick; moderate reddish brown (10R 4/6); moderately consolidated conglomerate bedrock, disaggregated to clayey GRAVEL with sand (GC) gradational to clayey SAND with gravel (SC), dense to very dense, moist, massive; gravel and larger sized clasts comprise ~25-35% of unit; clasts entirely subrounded to subangular quartzite up to 5" in diameter, though mode size 1-2"; lone sandstone clast 4" in diameter observed; low to medium plasticity fines; consistent Wasatch Formation character as seen in other test pits on Powder Mountain.

SCALE: 1"=5' H&V

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Geotechnical & Geologic Hazard Investigation
Main Street West
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

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.

STRATIFICATION

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

DESCRIPTION	THICKNESS
OCCASIONAL	ONE OR LESS PER FOOT OF THICKNESS
FREQUENT	MORE THAN ONE PER FOOT OF THICKNESS

APPARENT / RELATIVE DENSITY - COARSE-GRAINED SOIL

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

CONSISTENCY - FINE-GRAINED SOIL

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

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Key to Soil Symbols and Terminology

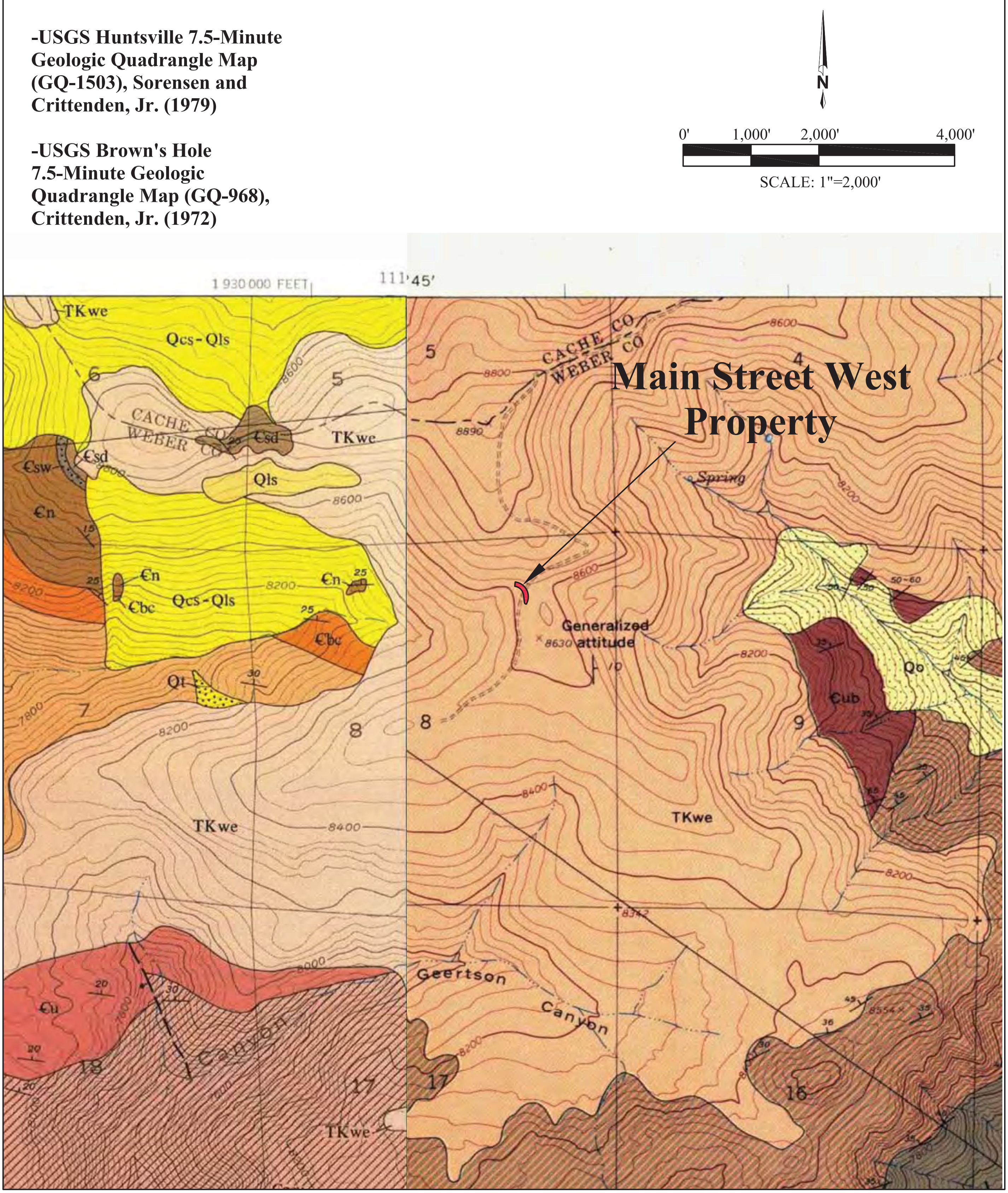
Figure A-5

IGES, Inc. Project No.:01628-023

BASE MAPS

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

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



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Geotechnical & Geologic Hazard Investigation
Main Street West
Summit Powder Mountain Resort
Weber County, Utah

REGIONAL GEOLOGY MAP 1

Figure

A-6a

MAP LEGEND

Qal ALLUVIAL DEPOSITS, UNDIFFERENTIATED (Holocene) – Unconsolidated gravel, sand, and silt deposits in presently active stream channels and floodplains; thickness 0-6 m

Qcs COLLUVIUM AND SLOPEWASH (Holocene) – Bouldery colluvium and slopewash chiefly along eastern margin of Ogden Valley; in part, lag from Tertiary units; thickness 0-30 m

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

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

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

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

Recently active normal fault – Dashed where inferred. Ticks on downthrown side

Pre-Tertiary normal fault – Dotted where concealed Bar and ball on downthrown side

Thrust fault – Dashed where inferred Sawteeth on upper plate

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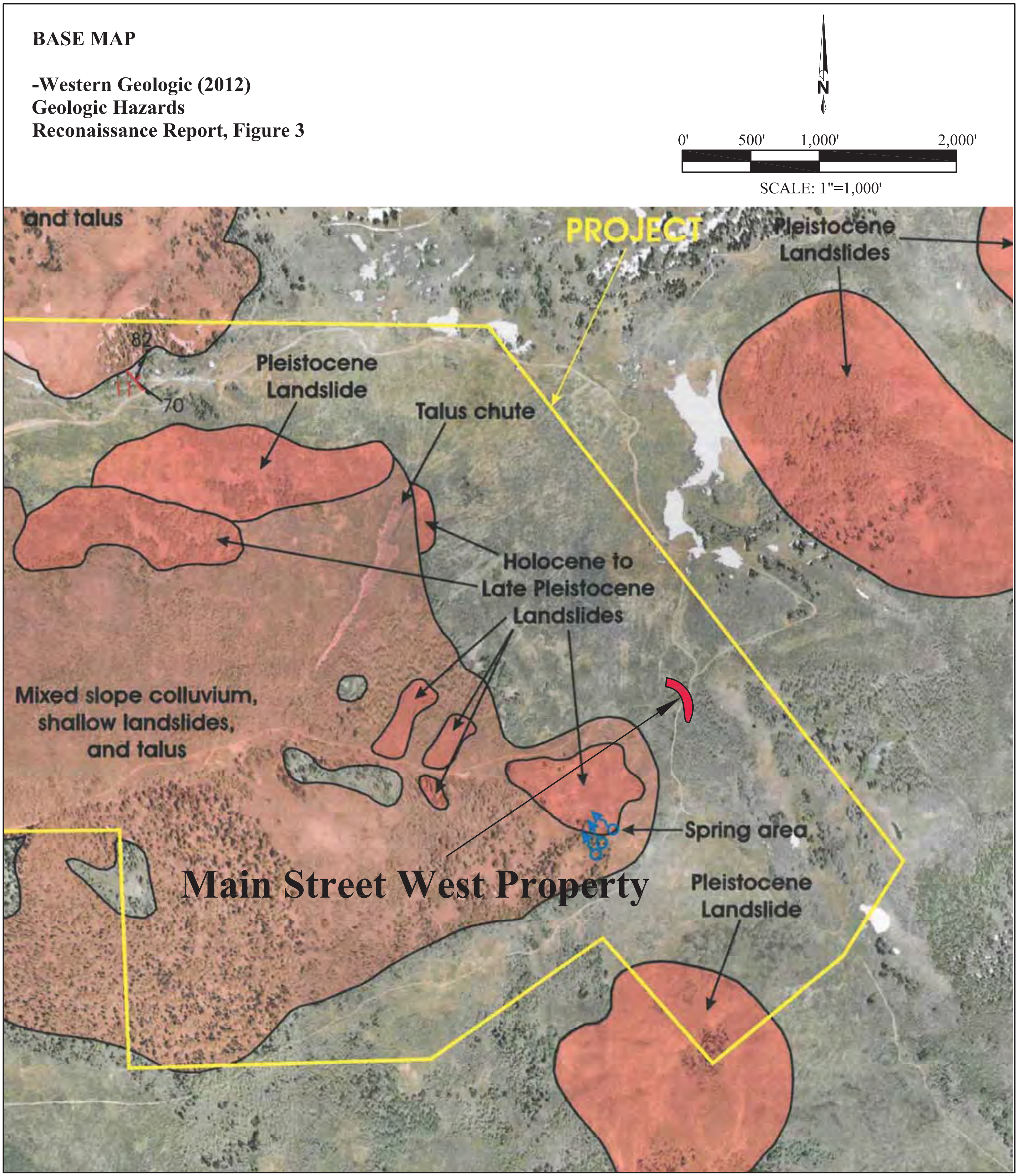
REGIONAL GEOLOGY MAP 1

Figure

A-6c

BASE MAP

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



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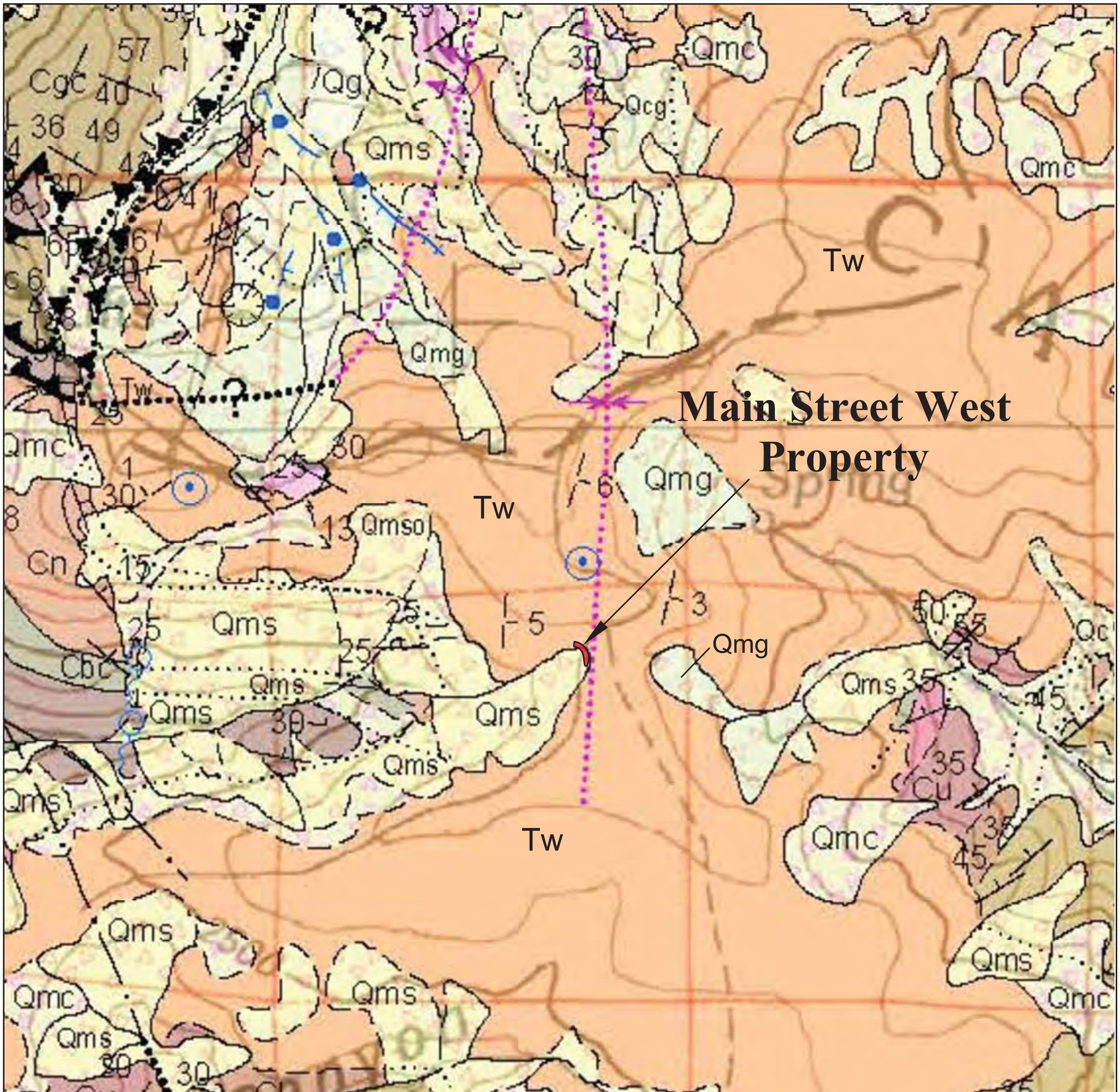
REGIONAL GEOLOGY MAP 2

Figure

A-7



BASE MAP

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

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Main Street West
Summit Powder Mountain Resort
Weber County, Utah

REGIONAL GEOLOGY MAP 3

Figure

A-8a

MAP LEGEND

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

Human disturbances

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

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

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

Qmg, Qmg?

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

Tw, Tw?

Wasatch Formation (Eocene and upper Paleocene) – Typically red to brownish-red sandstone, siltstone, mudstone, and conglomerate with minor gray limestone and marlstone locally (see 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)

IGES

Project: **Summit - Main Street West**
No: **01628-023**
Location: Powder Mountain, UT
Date: 12/20/2016
By: ET

Sample Info.	Boring No.	TP-2							
	Sample								
	Depth	5.0'							
	Split	Yes							
	Split sieve	3/8"							
	Total sample (g)	3885.61							
	Moist coarse fraction (g)	1540.53							
	Moist split fraction (g)	2345.08							
	Sample height, H (in)								
	Sample diameter, D (in)								
	Mass rings + wet soil (g)								
	Mass rings/tare (g)								
	Moist unit wt., γ_m (pcf)								
Coarse Fraction	Wet soil + tare (g)	938.60							
	Dry soil + tare (g)	933.00							
	Tare (g)	121.53							
	Water content (%)	0.7							
Split Fraction	Wet soil + tare (g)	705.41							
	Dry soil + tare (g)	695.23							
	Tare (g)	139.80							
	Water content (%)	1.8							
	Water Content, w (%)	**1.4**							
	Dry Unit Wt., γ_d (pcf)								

Entered by:___________

Reviewed:___________

Liquid Limit, Plastic Limit, and Plasticity Index of Soils

(ASTM D4318)

Project: Summit - Main Street West
No: 01628-023
Location: Powder Mountain, UT
Date: 12/22/2016
By: DKS

Boring No.: TP-2
Sample:
Depth: 5.0'
Description: Reddish brown silty clay

Preparation method: Wet
Liquid limit test method: Multipoint

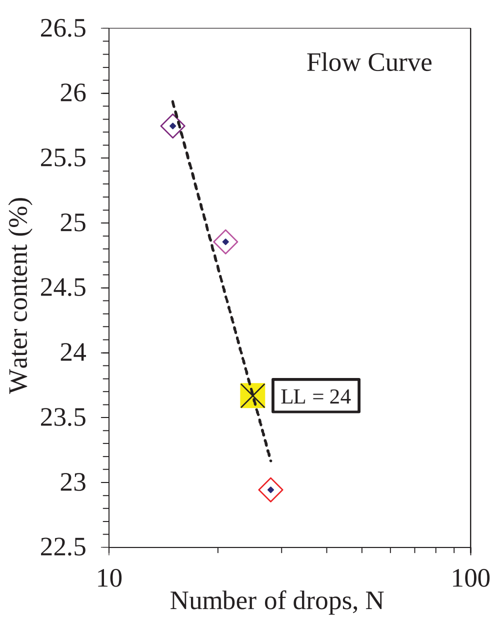
Plastic Limit

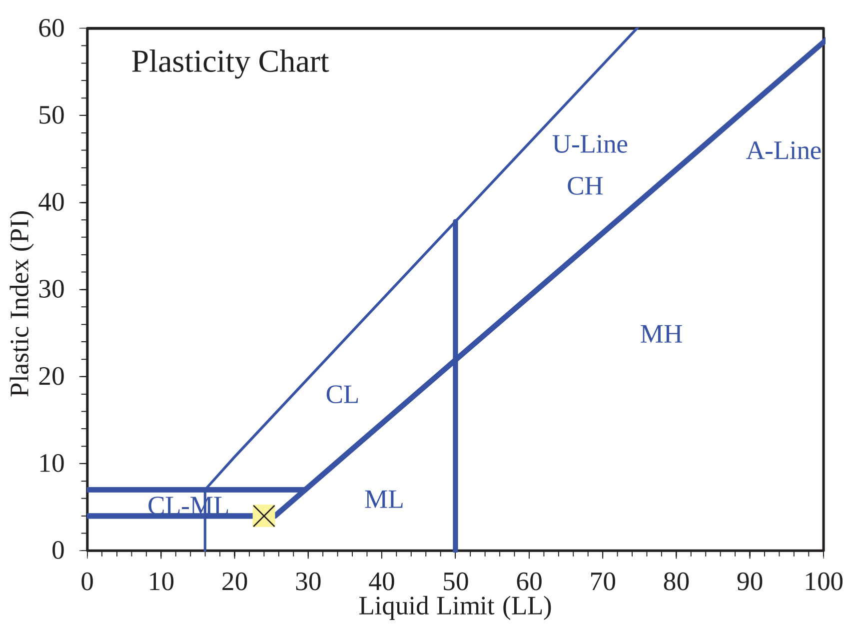
Determination No	1	2				
Wet Soil + Tare (g)	35.68	37.86				
Dry Soil + Tare (g)	33.38	35.16				
Water Loss (g)	2.30	2.70				
Tare (g)	21.94	21.95				
Dry Soil (g)	11.44	13.21				
Water Content, w (%)	20.10	20.44				

Liquid Limit

Determination No	1	2	3			
Number of Drops, N	28	21	15			
Wet Soil + Tare (g)	34.29	34.99	32.56			
Dry Soil + Tare (g)	32.03	32.44	30.32			
Water Loss (g)	2.26	2.55	2.24			
Tare (g)	22.18	22.18	21.62			
Dry Soil (g)	9.85	10.26	8.70			
Water Content, w (%)	22.94	24.85	25.75			
One-Point LL (%)	23	24				

Liquid Limit, LL (%)	**24**
Plastic Limit, PL (%)	**20**
Plasticity Index, PI (%)	**4**





Entered by:____________
Reviewed:____________

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

(ASTM D6913)

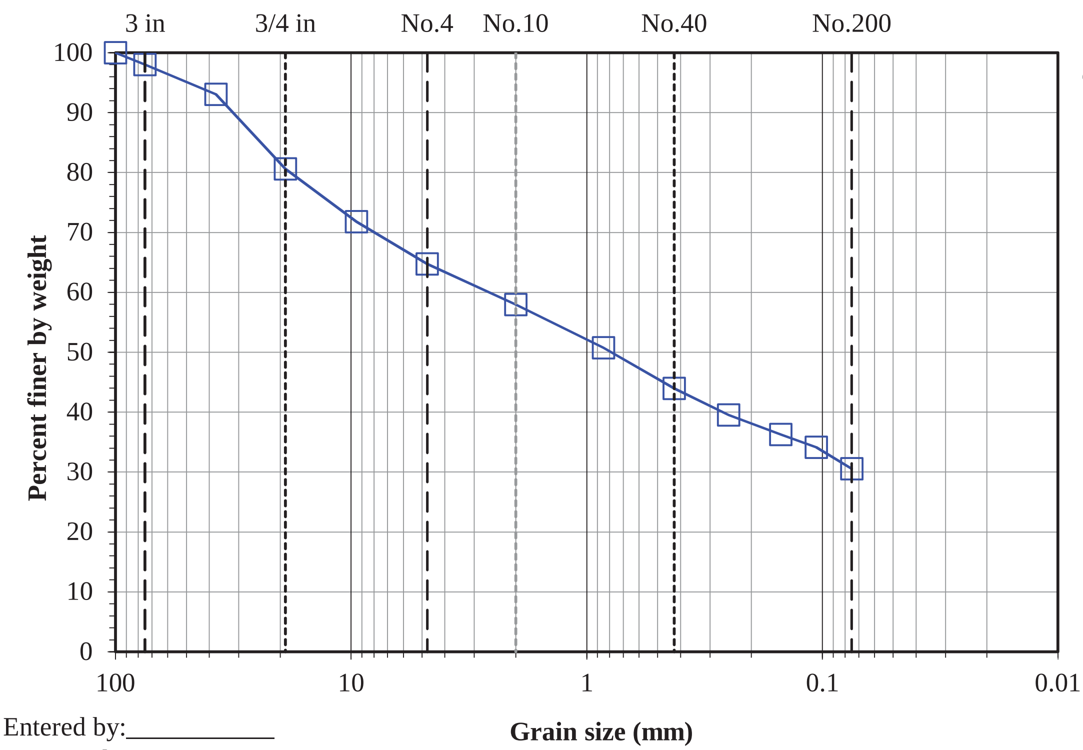
Project: Summit - Main Street West
No: 01628-023
Location: Powder Mountain, UT
Date: 12/22/2016
By: ET

Boring No.: TP-1
Sample:
Depth: 0 to 12'
Description: Reddish brown silty gravel with sand

	Moist	Dry
Split:	Yes	
Split sieve:	3/4"	
Total sample wt. (g):	26574.30	25151.12
+3/4" Coarse fraction (g):	4934.60	4878.56
-3/4" Split fraction (g):	1318.67	1235.36
Split fraction:	0.806	

Water content data	C.F.(+3/4")	S.F.(-3/4")
Moist soil + tare (g):	3306.68	1649.40
Dry soil + tare (g):	3272.89	1566.09
Tare (g):	331.43	330.73
Water content (%):	1.1	6.7

Sieve	Accum. Wt. Ret. (g)	Grain Size (mm)	Percent Finer	
8"	-	200	-	
6"	-	150	-	
4"	-	100	100.0	
3"	501.15	75	98.0	
1.5"	1741.89	37.5	93.1	
3/4"	4878.56	19	80.6	←Split
3/8"	134.59	9.5	71.8	
No.4	242.94	4.75	64.8	
No.10	346.74	2	58.0	
No.20	457.52	0.85	50.8	
No.40	561.38	0.425	44.0	
No.60	629.66	0.25	39.5	
No.100	679.34	0.15	36.3	
No.140	712.52	0.106	34.1	
No.200	766.94	0.075	30.6	



Gravel (%): 35.2
Sand (%): 34.2
Fines (%): 30.6

Entered by:___________
Reviewed:___________

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

(ASTM D1140)

Project: **Summit - Main Street West**
No: **01628-023**
Location: Powder Mountain, UT
Date: 12/22/2016
By: ET

Sample Info.	Boring No.	TP-2	TP-2						
	Sample								
	Depth	5.0'	8.0'						
	Split	Yes	Yes						
	Split Sieve*	3/8"	3/8"						
	Method	B	B						
	Specimen soak time (min)	410	240						
	Moist total sample wt. (g)	3885.61	3571.70						
	Moist coarse fraction (g)	1540.53	884.58						
	Moist split fraction + tare (g)	705.41	662.54						
	Split fraction tare (g)	139.80	124.64						
	Dry split fraction (g)	555.43	515.88						
	Dry retained No. 200 + tare (g)	529.75	363.77						
	Wash tare (g)	139.80	124.64						
	No. 200 Dry wt. retained (g)	389.95	239.13						
	Split sieve* Dry wt. retained (g)	1529.97	881.76						
	Dry total sample wt. (g)	3832.84	3458.88						
Coarse Fraction	Moist soil + tare (g)	938.60	1157.81						
	Dry soil + tare (g)	933.00	1154.99						
	Tare (g)	121.53	273.23						
	Water content (%)	0.69	0.32						
Split Fraction	Moist soil + tare (g)	705.41	662.54						
	Dry soil + tare (g)	695.23	640.52						
	Tare (g)	139.80	124.64						
	Water content (%)	1.83	4.27						
	Percent passing split sieve* (%)	**60.1**	**74.5**						
	Percent passing No. 200 sieve (%)	**17.9**	**40.0**						

Entered by:___________

Reviewed:___________

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

IGES

Project: Summit - Main Street West
No: 01628-023
Location: Powder Mountain, UT
Date: 12/22/2016
By: DKS

Sample info.	Boring No.	TP-1							
	Sample								
	Depth	7.0'							
Water content data	Wet soil + tare (g)	105.22							
	Dry soil + tare (g)	98.98							
	Tare (g)	37.57							
	Water content (%)	10.2							
Chem. data	pH	7.82							
	Soluble chloride* (ppm)	<5.31							
	Soluble sulfate** (ppm)	24.9							
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	85400	0.67	57218				
		+3	65710	0.67	44026				
		+6	63320	0.67	42424				
		+9	63500	0.67	42545				
	Minimum resistivity (Ω-cm)	**42424**							

* 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 (41.363°N, 111.7443°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 = 0.810 g
From Figure 1613.3.1(2) [2]	S_1 = 0.269 g

Section 1613.3.2 — Site class definitions

The authority having jurisdiction (not the USGS), site-specific geotechnical data, and/or the default has classified the site as Site Class C, based on the site soil properties in accordance with Section 1613.

2010 ASCE-7 Standard – Table 20.3-1
SITE CLASS DEFINITIONS

Site Class	$\bar{v}_s$	$\bar{N}$ or $\bar{N}_{ch}$	$\bar{s}_u$
A. Hard Rock	>5,000 ft/s	N/A	N/A
B. Rock	2,500 to 5,000 ft/s	N/A	N/A
C. Very dense soil and soft rock	1,200 to 2,500 ft/s	>50	>2,000 psf
D. Stiff Soil	600 to 1,200 ft/s	15 to 50	1,000 to 2,000 psf
E. Soft clay soil	<600 ft/s	<15	<1,000 psf
	Any profile with more than 10 ft of soil having the characteristics: • Plasticity index PI > 20, • Moisture content w ≥ 40%, and • Undrained shear strength $\bar{s}_u$ < 500 psf		
F. Soils requiring site response analysis in accordance with Section 21.1	See Section 20.3.1		

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

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

TABLE 1613.3.3(1)
VALUES OF SITE COEFFICIENT F_a

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

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

For Site Class = C and S_S = 0.810 g, F_a = 1.076

TABLE 1613.3.3(2)
VALUES OF SITE COEFFICIENT F_v

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

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

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

Equation (16-37):	$S_{MS} = F_aS_S = 1.076 \times 0.810 = 0.872$ g
Equation (16-38):	$S_{M1} = F_vS_1 = 1.531 \times 0.269 = 0.411$ g

Section 1613.3.4 — Design spectral response acceleration parameters

Equation (16-39):	$S_{DS} = ⅔\ S_{MS} = ⅔ \times 0.872 = 0.581$ g
Equation (16-40):	$S_{D1} = ⅔\ S_{M1} = ⅔ \times 0.411 = 0.274$ 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} = 0.581 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} = 0.274 g, Seismic Design Category = D

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

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

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

References

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

USGS Design Maps Summary Report

User–Specified Input

Report Title	Main Street West Wed January 18, 2017 20:38:10 UTC
Building Code Reference Document	2012/2015 International Building Code (which utilizes USGS hazard data available in 2008)
Site Coordinates	41.363°N, 111.7443°W
Site Soil Classification	Site Class C – "Very Dense Soil and Soft Rock"
Risk Category	I/II/III

USGS–Provided Output

S_S = 0.810 g	S_{MS} = 0.872 g	S_{DS} = 0.581 g
S_1 = 0.269 g	S_{M1} = 0.411 g	S_{D1} = 0.274 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.