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June 9, 2017

Mr. Hollis Carter
2118 15th Street
Boulder, Colorado 80301

IGES Project No. 02347-001

**RE: Geotechnical & Geologic Hazard Investigation Report (Rev 1)
Lot 75R of Summit Powder Mountain Resort
8452 E. Spring Park
Weber County, Utah**

PLAN REVIEW ACCEPTANCE
FOR COMPLIANCE WITH THE APPLICABLE CONSTRUCTION CODES IDENTIFIED BELOW.
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BY: MEM DATE: 10/11/17
WEST COAST CODE CONSULTANTS, INC.

Mr. Carter,

As requested, IGES has conducted a geotechnical investigation for the proposed residence to be constructed on Lot 75R of the Powder Mountain Resort located at 8452 East Spring Park in Weber County, Utah. The approximate location of the property is illustrated on the *Site Vicinity Map* (Figure A-1 in Appendix A). The purposes of our investigation was to assess the nature and engineering properties of the subsurface soils at the proposed home site and to provide recommendations for the design and construction of foundations, grading, and drainage. The scope of work completed for this study included an assessment of geologic hazards, subsurface exploration, laboratory testing, engineering analyses and preparation of this report. This document is a revision of the first letter-report dated August 11, 2016, and includes additional geological information as requested by Weber County.

Project Understanding

Our understanding of the project is based primarily on our previous involvement with the Summit Powder Mountain Resort project, which included two geotechnical investigations for the greater 200-acre Powder Mountain Resort expansion project (IGES, 2012a and 2012b).

The Powder Mountain Resort expansion project is located southeast of SR-158 (Powder Mountain Road), south of previously developed portions of Powder Mountain Resort, in unincorporated Weber County, Utah. The project site is accessed by Powder Ridge Road.

Lot 75R is a 0.16-acre single-family residential lot with a buildable envelope of approximately 3,750 square feet. A single-family home will be constructed at the site, presumably a high-end vacation home. Plans for the home were not available at the time of this report; however, based on our experience in the area, we anticipate the home will be a two-story wood-framed structure, possibly with a walk-out basement (three levels total), founded on conventional spread footings. The project is expected to include improvements common for residential developments such as underground utilities, curb and gutter, flatwork, landscaping, and possibly appurtenant structures.

METHODS OF STUDY

Literature Review

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

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 75R* 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.

Field Investigation

Subsurface soils were investigated by excavating one test pit approximately 11 feet below the existing site grade. The approximate location of the test pit is illustrated on the *Local Geology and Geotechnical Map* (Plate 1). The soil types and conditions were visually logged at the time of the excavation in general accordance with the *Unified Soil Classification System* (USCS). Subsurface soil classifications and descriptions are included on the test pit log included as Figures A-2 in Appendix A. A key to USCS symbols and terminology is included as Figure A-3.

Laboratory Testing

The majority of materials encountered in the test pit consisted of coarse, cemented colluvium with abundant cobbles, or relatively stiff/hard sandy clay. As such, soil samples suitable for testing in an oedometer could not be obtained. Therefore, laboratory testing and engineering analysis was based largely on previously completed geotechnical investigations (IGES, 2012a & 2012b) and laboratory testing for this project that included index testing (grain size analysis, Atterberg Limits).

Engineering Analysis

Engineering analyses were performed using soil data obtained from laboratory testing and empirical correlations based on material density, depositional characteristics and classification. Appropriate factors of safety were applied to the results consistent with industry standards and the accepted standard of care. An allowable bearing pressure value was proportioned based on estimated shear strength of bearing soils with due consideration for allowable settlement.

FINDINGS

General Geologic Setting

The *Lot 75R* property is situated in the western portion of the northern Wasatch Mountains, approximately 4 miles northeast of Ogden Valley. The Wasatch Mountains contain a broad depositional history of thick Precambrian and Paleozoic sediments that have been subsequently modified by various tectonic episodes that have included thrusting, folding, intrusion, and volcanics, as well as scouring by glacial and fluvial processes (Stokes, 1987). The uplift of the Wasatch Mountains occurred relatively recently during the Late Tertiary Period (Miocene Epoch) between 12 and 17 million years ago (Milligan, 2000). Since uplift, the Wasatch Front has seen substantial modification due to such occurrences as movement along the Wasatch Fault and associated spurs, the development of the numerous canyons that empty into the current Salt Lake Valley and Utah Valley and their associated alluvial fans, erosion and deposition from Lake Bonneville, and localized mass movement events (Hintze, 1988).

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

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

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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 subject property is located approximately 10 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).

Surficial Geologic Setting

According to Crittenden, Jr. (1972), the *Lot 75R* 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. Coogan and King (2001) produced a regional-scale geologic map that covered the property; this map also shows the property to be largely underlain by the Wasatch Formation. Western Geologic (2012) identified a number of landslide deposits contained within the Powder Mountain Resort expansion area, and deposits mapped as "mixed slope colluvium, shallow landslides, and talus" are found near the southern margin of the property, though no landslides are mapped on the property. Finally, Coogan and King (2016) updated their 2001 map, which shows the property to be entirely underlain by the Wasatch Formation, with landslide deposits located approximately 175 feet south and east of the property (*Regional Geology Map*, Figure A-4). The landslide deposits (Qms) are described as "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." Wasatch Formation bedrock in the area is shown to be striking approximately to the north-northeast, and dipping approximately 5 degrees to the east-southeast.

Geologic Hazards From Literature

Based upon the available geologic literature, regional-scale geologic hazard maps that cover the *Lot 75R* 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.

Two regional-scale landslide hazard maps have been produced that cover the project area. Neither Colton (1991) nor Elliott and Harty (2010) 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" near the southern margin 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 deposits mapped as "mixed slope colluvium, shallow landslides, and talus" near the southern portion of the property.

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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 miles southwest of the western margin of the property (USGS and UGS, 2006).

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

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.

Surface Conditions From Site Reconnaissance

At the time of the site reconnaissance and excavation, the lot was in a relatively natural state and was covered with a sparse vegetative cover including native grasses and shrubs. Several boulders (>12 inches) were observed throughout the site. The lot drains to the southwest; the gradient of the lot is roughly 3H:1V on the upslope side of the lot transitioning to 5H:1V near the proposed building footprint. On the south side of the lot, an approximate 14-foot high, 2H:1V cut slope descends to the south to daylight with the cul-de-sac. There is about 11½ feet of vertical relief across the building envelope. Site-specific geologic mapping of the property found the local geology to be consistent with that as-mapped by Coogan and King (2016), covered by the Wasatch Formation (see Plate 1). Because the southern part of the property had been disturbed by human activity involving the construction of Spring Park Road, the test pit was spotted in the northern part of the property in order to observe native soils in the subsurface.

Earth Materials

The site is overlain by approximately 18 inches of dark brown topsoil characterized by an abundance of organic matter (roots, etc.). The topsoil was underlain by coarse colluvium consisting of dense dark reddish-brown clayey gravel with sand. The colluvium was characterized by occasional roots and abundant rounded quartzite cobbles up to 8 inches in diameter. At approximately 5 to 7 feet below existing grade the colluvium transitions to the Wasatch Formation, which is a bedrock unit consisting of cemented conglomerate. The conglomerate is highly weathered and readily disaggregates into soil that classifies as Clayey GRAVEL with sand (GC). Excavation of the material became more difficult with depth. The earth materials within the Wasatch Formation were observed to be orangish red, moist, clayey gravel with well-graded sands. Some quartzite clasts were observed to be as much as 24 inches in diameter; however, the mode clast size was approximately 3 inches in diameter.

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Detailed descriptions of earth materials encountered in our test pit are presented on the test pit log, Figure A-2, in Appendix A.

Groundwater

Localized seeping water was observed in the test pit excavation and observed to be at approximately 10 feet below existing grade. We identified this water source to be a localized spring or seepage point and not a phreatic surface. Water was also observed to be seeping from the South Spring Park road cut to the south of the site. Based on our observations, groundwater is not anticipated to adversely impact the proposed construction provided the basement excavation does not extend more than about eight feet below natural grade. Groundwater levels could rise at any time based on several factors including recent precipitation, on- or off-site runoff, irrigation, and time of year (e.g., spring run-off). Seeps and/or springs may be present on the foundation excavation during spring run-off. Should the groundwater become a concern during the proposed construction, IGES should be contacted so that dewatering recommendations may be provided. It is possible that some temporary dewatering could be necessary during construction, depending on the time of year and depth of excavation.

Expansive/Collapsible Soils

Expansive soils generally consist of clay soils that exhibit significant swelling when wetted. Expansive soils typically consist of Fat CLAY (CH) and have a “greasy” luster. Expansive soils can potentially damage foundation elements, crack concrete slabs, and create excess stress in the proposed structures. Although soils classifying as fat clay are often associated with expansive soils, soil classification alone cannot predict the expansive characteristics of clay soils. Based on our observations and our laboratory test results, soils classifying as fat clay were not encountered. Furthermore, soils classifying as fat clay are uncommon throughout the Powder Mountain area (IGES, 2012a, 2012b), although fat clay soils do occur locally. As such, the potential for expansive soils impacting the proposed development is considered low.

Collapse (often referred to as “hydro-collapse”) is a phenomena whereby undisturbed soils exhibit volumetric strain and consolidation upon wetting. Collapsible soils can cause differential settling of structures and roadways. Collapsible soils do not necessarily preclude development and can be mitigated by over-excavating porous, potentially collapsible soils and replacing with engineered fill and by controlling surface drainage and runoff. For some structures that are particularly sensitive to differential settlement, or in areas where collapsible soils are identified at great depth, a deep foundation system may be prescribed. Typical characteristics of collapsible soils include a) low dry unit weight (silts and fine sands), b) relatively dry soils, and c) porous soil structure (“pinholes”). These characteristics were not identified during our subsurface exploration; as such, wetting-induced collapse is not expected to significantly impact the proposed improvements.

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

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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 75R* property.

According to the most recent geologic mapping of the property, the property is not located on mapped landslide deposits (Coogan and King, 2016; Western Geologic, 2012). Additionally, landslide deposits or geomorphic features indicative of landsliding were not observed on the property in the aerial imagery, during the site reconnaissance, or in the subsurface. It should be noted, however, that landslide deposits have been mapped within approximately 175 feet to the east and south of the property. Given this information, the risk associated with landslides is considered low to moderate.

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

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 miles to the southwest 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

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design according to appropriate building code and design parameters can assist in mitigating the hazard associated with earthquake ground shaking.

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

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.

Slope Stability

The site is located on the side of a mountain, and therefore is on sloped terrain. The sloped terrain was modeled using SLIDE version 6.024 slope stability software. Spencer's Method was used to evaluate the stability of the slope. Calculations for stability were developed by searching for the minimum factor-of-safety for a circular-type failure. A minimum static factor-of-safety of 1.5 and seismic factor-of-safety of 1.0 was considered acceptable for this project considering the available information. The section analyzed is Section A-A', illustrated on Plate 1 of this report.

Considering the available geotechnical data, the soil types observed (coarse clayey sand and gravel), and our experience in the area, appropriate engineering parameters have been selected for our model; these parameters are summarized in Table 1.0.

Table 1.0
Engineering Parameters for Subsurface Model

Soil Type	Elevation (ft. below existing grade)	Unit Weight (pcf)	Friction Angle (Degrees)	Cohesion (psf)
Fill (Afc)	Variable	125	34	100
Clayey Gravel (Qal)	0-7	120	36	0
Wasatch Formation (Twe)	>7	130	39	0

Groundwater was modeled to be approximately 10 feet below existing grade.

For the seismic (pseudo-static) assessment of slope, the seismic coefficient k_h is modeled as equal to 50% of the peak ground acceleration (PGA) resulting from a MCE seismic event (2PE50). From our referenced geotechnical report, the PGA resulting from a 2PE50 seismic event is taken as 0.326g. Therefore, we have adopted a seismic coefficient of 0.17g.

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Based on our analysis, minimum factors-of-safety of 1.5 and 1.0 for static and seismic conditions, respectively, are maintained with respect to the proposed building envelope. The results of the global stability analyses are attached within Appendix C.

Stability of Saturated Slopes

IGES assessed the potential for surficial soils becoming mobilized under saturated parallel seepage conditions. Our assessment assumes coarse colluvium, fully saturated, and a 3.1H:1V slope, which is representative for the area below the building envelope, within the property boundary. Our model assumes an effective friction angle of 36 degrees with 50 psf cohesion, and a saturated unit weight of 136 pcf. The analysis indicates the slope will maintain a factor of safety against surficial failure under parallel seepage conditions of 1.68. Sample are presented in Appendix C.

Seismicity

Following the criteria outlined in the 2012 International Building Code (IBC, 2012), spectral response at the site was evaluated for the *Maximum Considered Earthquake* (MCE) which equates to a probabilistic seismic event having a two percent probability of exceedance in 50 years (2PE50). Spectral accelerations were determined based on the location of the site using the *U.S. Seismic "DesignMaps" Web Application* (USGS, 2012); this software incorporates seismic hazard maps depicting probabilistic ground motions and spectral response data developed for the United States by the U. S. Geological Survey as part of NEHRP/NSHMP (Frankel et al., 1996). These maps have been incorporated into both *NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures* (FEMA, 1997) and the *International Building Code* (IBC) (International Code Council, 2012).

Table 2.0
Short- and Long-Period Spectral Accelerations for MCE

Parameter	**Short Period (0.2 sec)**	**Long Period (1.0 sec)**
MCE Spectral Response Acceleration (g)	$S_S = 0.813$	$S_1 = 0.270$
MCE Spectral Response Acceleration Site Class B (g)	$S_{MS} = S_sF_a = 0.813$	$S_{M1} = S_1F_v = 0.270$
Design Spectral Response Acceleration (g)	$S_{DS} = S_{MS}*^2/_3 = 0.542$	$S_{D1} = S_{M1}*^2/_3 = 0.180$

To account for site effects, site coefficients that vary with the magnitude of spectral acceleration and *Site Class* are used. Site Class is a parameter that accounts for site amplification effects of soft soils and is based on the average shear wave velocity of the upper 100 feet; based on our field exploration and our understanding of the geology in this area, the subject site is appropriately classified as Site Class B (*rock*). Based on IBC criteria, the short-period (F_a) coefficient is 1.0 and long-period (F_v) site coefficient is 1.0. Based on the design spectral response accelerations for a *Building Risk Category* of I, II or

III, the site's *Seismic Design Category* is D. The short- and long-period *Design Spectral Response Accelerations* are presented in Table 2.0; a summary of the *Design Maps* analysis is presented in Appendix B. The *peak ground acceleration* (PGA) may be taken as $0.4 * S_{MS}$.

CONCLUSIONS AND RECOMMENDATIONS

Based on the results of field observations, laboratory testing for this project and previously completed geotechnical investigations (IGES 2012a, IGES 2012b, IGES 2014a, IGES 2014b, IGES 2015a, IGES 2015b), the subsurface conditions are considered suitable for construction of a single-family home 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.

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 75R* project area:

- ***The Lot 75R* project area does not appear to have geological hazards that could adversely affect the development as currently proposed.**

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

- Though no evidence of landsliding was observed on the property, given the proximity to mapped landslides the risk of landslide hazards is considered to be low to moderate.

- 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:

a) Because landslide deposits are noted near the property, an IGES geologist or geotechnical engineer must observe the foundation excavations to confirm the absence of landslide deposits.

General Site Preparation and Grading

Prior to the placement of foundations, general site grading is recommended to provide proper support for exterior concrete flatwork, concrete slabs-on-grade, and pavement

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sections. Site grading is also recommended to provide proper drainage and moisture control on the subject property and to aid in preventing differential movement in foundation soils as a result of variations in moisture conditions.

Below proposed structures, fills, and man-made improvements, all vegetation, topsoil, debris and undocumented fill soils (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 that recommendations presented in this report have been complied with.

Excavations

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

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

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 flatter grade pending evaluation of soil conditions by the geotechnical engineer. Soil

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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.

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. 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. These lift thicknesses are *maximums*; the contractor should be aware that thinner lifts may be necessary to achieve the desired compaction. 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.

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.

Utility Trench Backfill

Utility trenches should be backfilled with structural fill in accordance with the previous section. Utility trenches can be backfilled with the onsite soils free of debris, organic and oversized material. Prior to backfilling the trench, pipes should be bedded in and shaded with a uniform granular material that has a Sand Equivalent (SE) of 30 or greater. Pipe bedding may be water-densified in-place (jetting). Alternatively, pipe bedding and shading may consist of clean ¾-inch gravel, which generally does not require densification. Native earth materials can be used as backfill over the pipe bedding zone. All utility trenches backfilled below pavement sections, curb and gutter, 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.

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Foundations

Based on our field observations and considering the presence of relatively competent native earth materials, we recommend that the footings for the proposed home be founded either *entirely* on competent native earth materials or *entirely* on structural fill. Native/fill transition zones are not allowed. Furthermore, if part of the foundation excavation exposes hard/cemented colluvium and/or conglomerate bedrock, all foundations should be deepened such that the entire foundation system is placed on similarly firm earth materials.

If soft, loose, or otherwise deleterious earth materials are exposed in the footing excavations, then the footings should 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.

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 **2,500 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.

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 must be installed around below-ground foundations (e.g., basement walls) to minimize the potential for flooding from water seepage (spring-like conditions), which may be present at various times during the year, particularly during spring run-off.

Settlement

Static settlement of properly designed and constructed conventional foundations, founded as described above, 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.

Competent native earth materials and/or properly compacted structural fill is expected to exhibit negligible seismically-induced settlement during a MCE seismic event.

Earth Pressure 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

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the footing and the supporting soils. In determining the frictional resistance against concrete, a coefficient of friction of 0.45 for coarse granular 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 3.0:

Table 3.0
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	40	0.53	64
At-rest (Ko)	0.50	60	0.80	96
Passive (Kp)	3.0	360	—	—

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 or buried tank 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.

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

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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.

Moisture Protection

Moisture should not be allowed to infiltrate into the soils in the vicinity of the foundations. As such, design strategies to minimize ponding and infiltration near the home should be implemented. The new home may be subject to sheet flow during periods of heavy rain or snow melt; therefore, the Civil Engineer may also wish to consider construction of additional surface drainage to intercept surface runoff, or a curtain drain to intercept seasonal groundwater flow.

We recommend that hand watering, desert landscaping or Xeriscape be considered within 5 feet of the foundations. We further recommend roof runoff devices be installed to direct all runoff a minimum of 10 feet away from structures. The home builder should be responsible for compacting the exterior backfill soils around the foundation. Additionally, the ground surface within 10 feet of the house should be constructed so as to slope a minimum of **five** percent away from the home. Pavement sections should be constructed to divert surface water off of the pavement into storm drains. Parking strips and roadway shoulder areas should be constructed to prevent infiltration of water into the areas surrounding pavement. Landscape plans must conform to Weber County development codes.

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

Soil Corrosion Potential

Laboratory testing of a representative soil sample obtained from the test pit indicated that the soil sample tested had a sulfate content of 135 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 5,071 OHM-cm, soluble chloride content of 8.46 ppm and a pH of 5.24. Based on this result, the onsite native soil is considered to be *mildly corrosive* to ferrous metal. To address the acidic soil conditions, we recommend

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a lower water/cement ratio, ~0.4, for reinforced concrete. The lower water/cement ratio will reduce permeability of the concrete and reduce the susceptibility of the reinforcing steel to acidic corrosion.

Construction Considerations

- Excavation Difficulty: The rocky, cemented colluvium identified approximately 6 feet below existing grade was difficult to excavate. Hard, cemented gravels, or conglomerate bedrock (Wasatch Formation) may be difficult to excavate and may require heavy-duty rippers or other specialized excavation procedures.

- Over-Size Material: Rounded boulders to 24 inches were identified in the test pits and on the ground surface; larger rocks may be present locally. The site is overlain with bouldery colluvium, largely derived from the underlying Wasatch Formation, which consists of cobbly/bouldery conglomerate. Large rocks may require special handling, such as segregation from structural fill, and disposal.

- Water was observed entering the test pit at a depth of approximately 10 feet below existing grade. Furthermore, water was observed seeping from the 2H:1V cut slope that descends toward the cul-de-sac. Although this water is not considered groundwater in the conventional sense (a piezometric surface), it is very likely that this water represents a localized perched groundwater body and is in effect a localized underground spring. Water flow is expected to be maximum during spring run-off, and will likely taper off as the season progresses, although the water seepage may continue year-round. The Contractor and Civil Engineer should be aware of this seepage; the design of the structure should take into account the presence of seepage, and subterranean portions of the home should be well-drained. Temporary dewatering may be required during construction. A French Drain or a Curtain Drain may be desirable to help control water around the property. Alternatively, the Owner may wish to consider designing the home as an on-grade structure (e.g., no basement) to minimize the potential for seepage to impact the subterranean components of the home.

CLOSURE

The recommendations presented in this report are based on limited field exploration, literature review, and a general understanding of the proposed construction. The subsurface data used in the preparation of this report were obtained from the exploration(s) made for this investigation. It is possible that variations in the soil and groundwater conditions could exist beyond the point 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, IGES should be immediately notified so that any necessary revisions to recommendations contained in this report may be made. 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.

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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.

Additional Services

The recommendations presented in this report are based on the assumption that an adequate program of tests and observations will be made during the construction. IGES staff should be on site to verify compliance with these recommendations. These tests and observations should include, but not necessarily be limited to, the following:

- Observations and testing during site preparation, earthwork and structural fill placement.
- Consultation as may be required during construction.
- Quality control testing of cast-in-place concrete.
- Review of plans and specifications to assess compliance with our recommendations.

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 contact the undersigned at (801) 748-4044.

Respectfully submitted,
IGES, Inc.

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Attachments: Next Page

Attachments:

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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/

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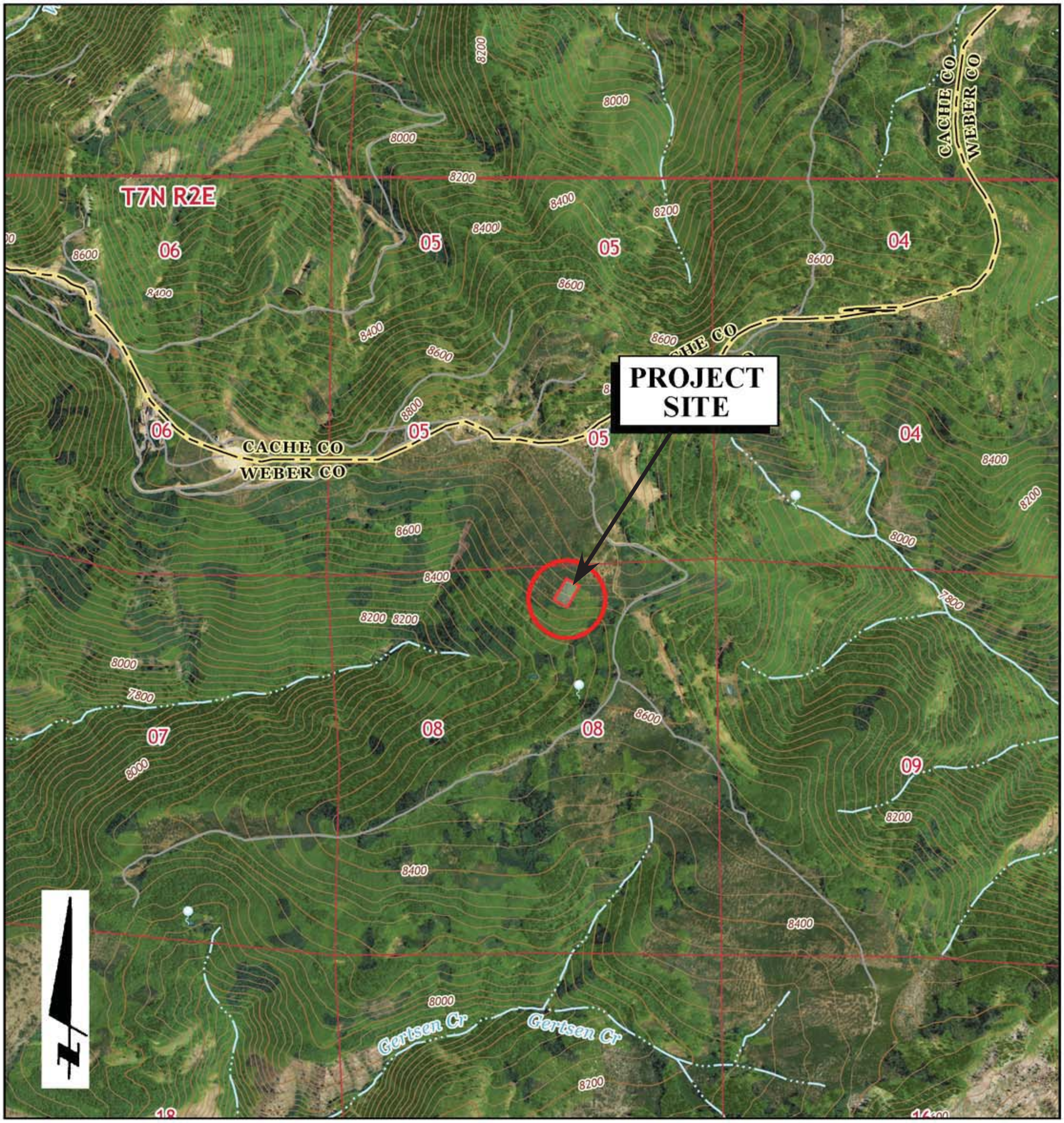
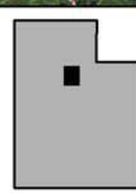
APPENDIX A



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



MAP LOCATION

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

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Project No. 02347-001

Geotechnical Investigation
Lot 75R of Summit Eden Phase 1C
8452 E. Spring Park
Weber County, Utah

SITE VICINITY MAP

Figure

A-1

DATE	
STARTED:	7/12/16
COMPLETED:	7/12/16
BACKFILLED:	7/12/16

Geotechnical Investigation
Lot 75R of Summit Eden Phase 1C
8452 East Spring Park
Weber County, Utah

Project Number 02347

IGES Rep: ET
Rig Type: CAT 315C

TEST PIT NO: **TP-1**
Sheet 1 of 1

LOCATION
LATITUDE 41.36302 LONGITUDE -111.74719 ELEVATION 8,570

ELEVATION	DEPTH FEET	SAMPLES	WATER LEVEL	UNIFIED SOIL CLASSIFICATION	MATERIAL DESCRIPTION	Dry Density(pcf)	Moisture Content %	Percent minus 200	Liquid Limit	Plasticity Index
	0			CL	@ 0' **TOPSOIL** Sandy lean CLAY with gravel - light brown to grayish brown, medium stiff, slightly moist, low pliasticity, trace subrounded quartzite clasts (up to 24-inch diameter), abundant roots,					
				SP	@ 1.5' **COLLUVIUM (Qc)** Poorly-graded SAND with gravel - light brown, medium dense, moist, fine-grained sand, subrounded quartzite clasts (up to 8-inches in diameter), occasional root					
8565	5									
		Grab sample								
					@ 7.5' **WASATCH FORMATION (Twe)** Conglomerate, highly weathered, moderatley cemented, readily dissagregates to soil classifying as Poorly Graded Clayey GRAVEL with sand (GC) - orangish red, moist, low plasticity clay, well-graded sand, large subangular to subrounded quartzite clasts (1- to 2-foot diameter maximum and 2- to 3-inch diameter typical), similar to overlying colluvium except Twe is more cemented and increasingly difficult to excavate			39.0	36	20
8560	10		▼							
					End of test pit at approximately 11 feet below existing grade					
8555										

Moisture Content and Atterberg Limits
Plastic Limit — Moisture Content — Liquid Limit
10 20 30 40 50 60 70 80 90

LOG OF TEST PITS 7/1/2016 02347-001.GPJ IGES.GDT 7/28/16

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SAMPLE TYPE
- GRAB SAMPLE
- 3" O.D. THIN-WALLED HAND SAMPLER

WATER LEVEL
▼ - MEASURED
▽ - ESTIMATED

NOTES:

Figure A-2

UNIFIED SOIL CLASSIFICATION SYSTEM

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

LOG KEY SYMBOLS

BORING SAMPLE LOCATION

TEST-PIT SAMPLE LOCATION

WATER LEVEL (level after completion)

WATER LEVEL (level where first encountered)

CEMENTATION

DESCRIPTION	DESCRIPTION
WEAKELY	CRUMBLES OR BREAKS WITH HANDLING OR SLIGHT FINGER PRESSURE
MODERATELY	CRUMBLES OR BREAKS WITH CONSIDERABLE FINGER PRESSURE
STRONGLY	WILL NOT CRUMBLE OR BREAK WITH FINGER PRESSURE

OTHER TESTS KEY

C	CONSOLIDATION	SA	SIEVE ANALYSIS
AL	ATTERBERG LIMITS	DS	DIRECT SHEAR
UC	UNCONFINED COMPRESSION	T	TRIAXIAL
S	SOLUBILITY	R	RESISTIVITY
O	ORGANIC CONTENT	RV	R-VALUE
CBR	CALIFORNIA BEARING RATIO	SU	SOLUBLE SULFATES
COMP	MOISTURE/DENSITY RELATIONSHIP	PM	PERMEABILITY
CI	CALIFORNIA IMPACT	-200	% FINER THAN #200
COL	COLLAPSE POTENTIAL	Gs	SPECIFIC GRAVITY
SS	SHRINK SWELL	SL	SWELL LOAD

MODIFIERS

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

MOISTURE CONTENT

DESCRIPTION	FIELD TEST
DRY	ABSENCE OF MOISTURE, DUSTY, DRY TO THE TOUCH
MOIST	DAMP BUT NO VISIBLE WATER
WET	VISIBLE FREE WATER, USUALLY SOIL BELOW WATER TABLE

STRATIFICATION

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

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

GENERAL NOTES

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

APPARENT / RELATIVE DENSITY - COARSE-GRAINED SOIL

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

CONSISTENCY - FINE-GRAINED SOIL

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

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Key to Soil Symbols and Terminology

Figure A-3

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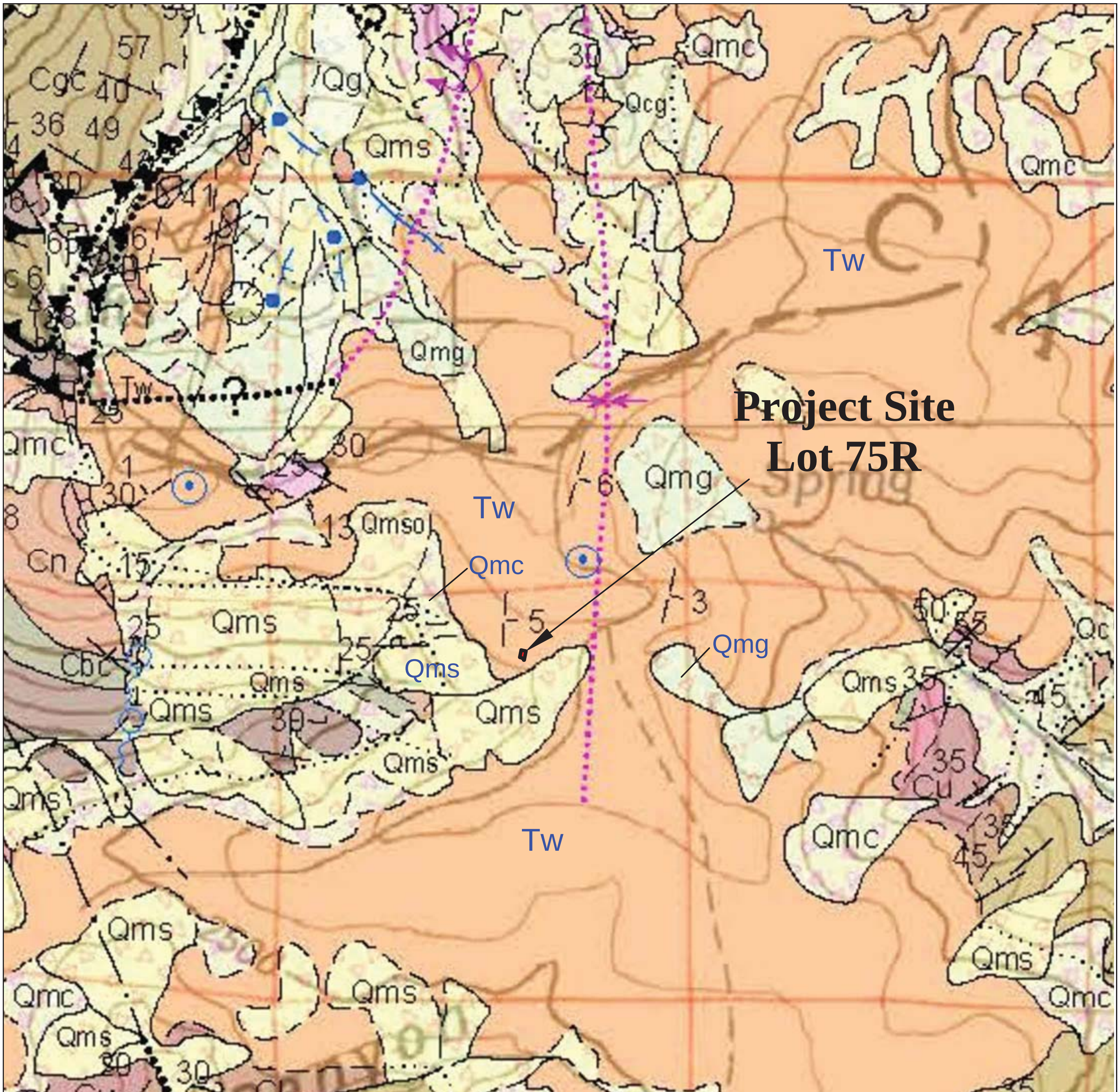
IGES, Inc. Project No.: 02347-001



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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Project No. 02347-001

Geotechnical & Geologic Hazard Investigation
Lot 75R of Summit Eden Phase 1C
8452 E. Spring Park
Weber County, Utah

REGIONAL GEOLOGY MAP

Figure

A-4a

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

Liquid Limit, Plastic Limit, and Plasticity Index of Soils

(ASTM D4318)

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Project: Lot 75R - Powder Mountain
No: 02347-001
Location: Eden, UT
Date: 7/20/2016
By: BRR

Boring No.:
Sample: 75R
Depth:
Description: Reddish brown lean clay

Preparation method: Wet
Liquid limit test method: Multipoint

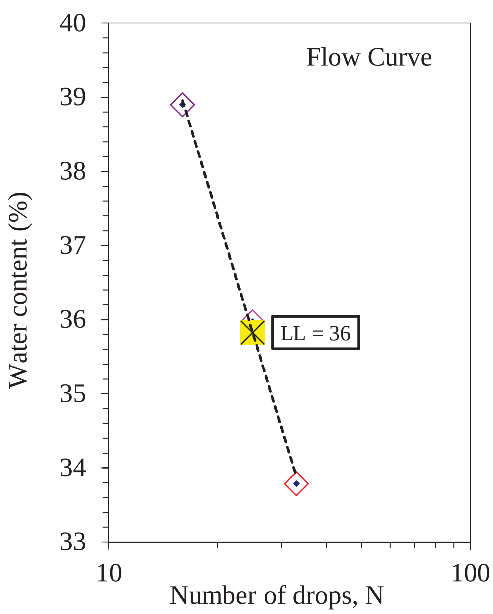
Plastic Limit

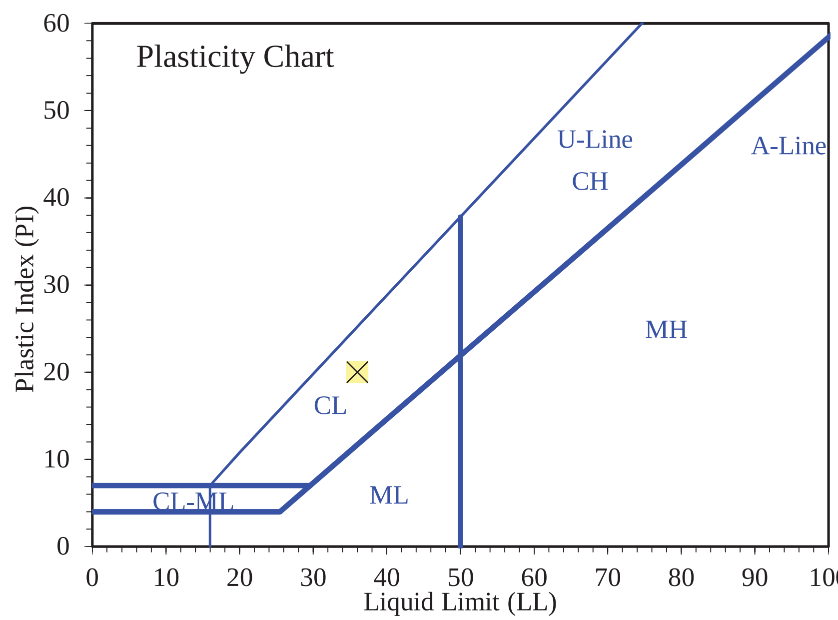
Determination No	1	2				
Wet Soil + Tare (g)	28.76	28.74				
Dry Soil + Tare (g)	27.83	27.74				
Water Loss (g)	0.93	1.00				
Tare (g)	22.11	21.64				
Dry Soil (g)	5.72	6.10				
Water Content, w (%)	16.26	16.39				

Liquid Limit

Determination No	1	2	3			
Number of Drops, N	33	25	16			
Wet Soil + Tare (g)	29.70	29.76	30.48			
Dry Soil + Tare (g)	27.73	27.67	28.15			
Water Loss (g)	1.97	2.09	2.33			
Tare (g)	21.90	21.86	22.16			
Dry Soil (g)	5.83	5.81	5.99			
Water Content, w (%)	33.79	35.97	38.90			
One-Point LL (%)		36				

Liquid Limit, LL (%)	**36**
Plastic Limit, PL (%)	**16**
Plasticity Index, PI (%)	**20**





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Reviewed:____________

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

(ASTM D6913)

© IGES 2004, 2016

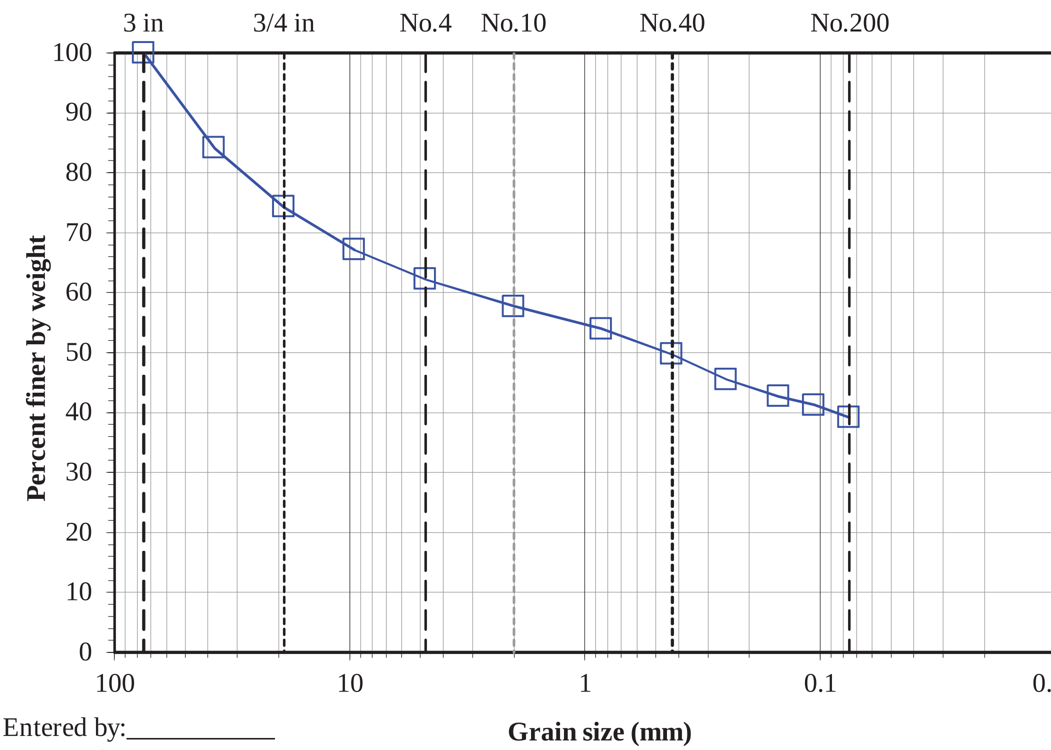
Project: Lot 75R - Powder Mountain
No: 02347-001
Location: Eden, UT
Date: 7/20/2016
By: ET

Boring No.:
Sample: 75R
Depth:
Description: Reddish brown clayey gravel with sand

Split:	Yes	
Split sieve:	3/8"	
	Moist	Dry
Total sample wt. (g):	19948.00	18522.77
+3/8" Coarse fraction (g):	6153.24	6093.90
-3/8" Split fraction (g):	827.55	745.61
Split fraction:	0.671	

Water content data	C.F.(+3/8")	S.F.(-3/8")
Moist soil + tare (g):	7081.20	995.67
Dry soil + tare (g):	7020.30	913.73
Tare (g):	766.00	168.12
Water content (%):	1.0	11.0

Sieve	Accum. Wt. Ret. (g)	Grain Size (mm)	Percent Finer	
8"	-	200	-	
6"	-	150	-	
4"	-	100	-	
3"	-	75	100.0	
1.5"	2941.70	37.5	84.1	
3/4"	4769.00	19	74.3	
3/8"	6093.90	9.5	67.1	←Split
No.4	54.59	4.75	62.2	
No.10	105.00	2	57.7	
No.20	146.61	0.85	53.9	
No.40	193.49	0.425	49.7	
No.60	239.97	0.25	45.5	
No.100	271.85	0.15	42.6	
No.140	287.78	0.106	41.2	
No.200	310.21	0.075	39.2	



Gravel (%): 37.8
Sand (%): 23.0
Fines (%): 39.2

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)*

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Project: **Lot 75R - Powder Mountain**
No: **02347-001**
Location: Eden, UT
Date: 7/27/2016
By: ET

Sample info.	Boring No.								
	Sample	75R							
	Depth								
Water content data	Wet soil + tare (g)	106.77							
	Dry soil + tare (g)	96.75							
	Tare (g)	37.33							
	Water content (%)	16.9							
Chem. data	pH	5.24							
	Soluble chloride* (ppm)	8.46							
	Soluble sulfate** (ppm)	73.8							
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	9850	0.67	6600				
		+3	7770	0.67	5206				
		+6	7568	0.67	5071				
		+9	8378	0.67	5613				
	Minimum resistivity (Ω-cm)	**5071**							

* Performed by AWAL using EPA 300.0

** Performed by AWAL using ASTM C1580

Entered by:___________

Reviewed:___________

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APPENDIX C

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Qal		120	Mohr-Coulomb	0	36
Twe		130	Mohr-Coulomb	0	39
Fill		125	Mohr-Coulomb	100	34

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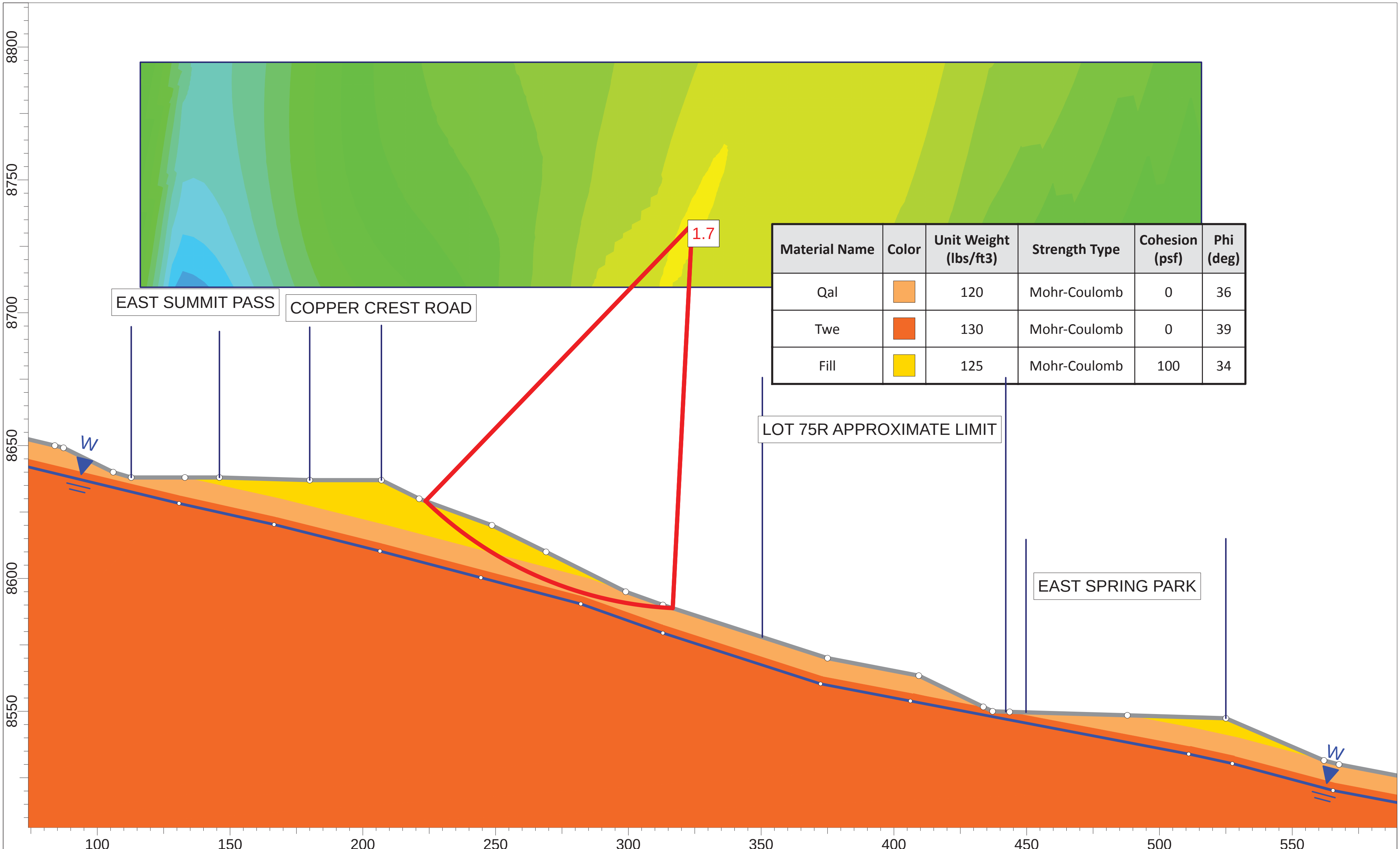
SLIDEINTERPRET 6.024

Project	Powder Mountain Lot 75R		
Analysis Description	Pseudo-Static		
Drawn By	TQH	*Company*	IGES Inc.
Date	7/15/2016	*File Name*	Lot75R Seismic.slim



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Qal		120	Mohr-Coulomb	0	36
Twe		130	Mohr-Coulomb	0	39
Fill		125	Mohr-Coulomb	100	34

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SLIDEINTERPRET 6.024

Project	Powder Mountain Lot 75R		
Analysis Description	Static		
Drawn By	TQH	*Company*	IGES Inc.
Date	7/15/2016	*File Name*	Lot 75R.slim

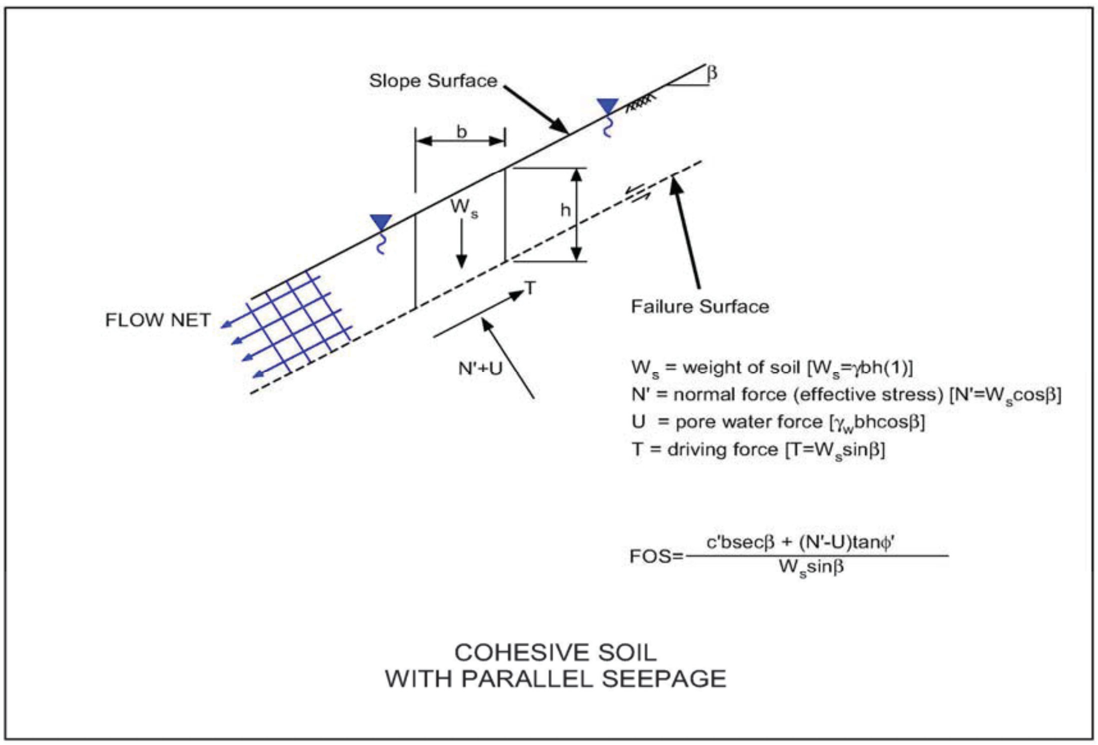
Lot 75R
02347-001
7/15/2016

c'	50	psf	Effective Cohesion (including apparent cohesion for coarse, angular soils)
ϕ'	36	deg	Effective Friction Angle
Y_{sat}	136	pcf	Saturated Unit Weight of Soil
Y_w	62.4	pcf	Unit weight of water
h	4	ft	Depth to shear surface
β	17.9	deg	Slope Gradient (3.1H:1V)

FS	1.53	

(yellow)	Input Variable
(green)	Calculated Value

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



APPENDIX D

Design Maps Detailed Report

2012/2015 International Building Code (41.36298°N, 111.74751°W)

Site Class B – "Rock", Risk Category I/II/III

Section 1613.3.1 — Mapped acceleration parameters

Note: Ground motion values provided below are for the direction of maximum horizontal spectral response acceleration. They have been converted from corresponding geometric mean ground motions computed by the USGS by applying factors of 1.1 (to obtain S_S) and 1.3 (to obtain S_1). Maps in the 2012/2015 International Building Code are provided for Site Class B. Adjustments for other Site Classes are made, as needed, in Section 1613.3.3.

From Figure 1613.3.1(1) [1] S_S = 0.813 g

From Figure 1613.3.1(2) [2] S_1 = 0.270 g

Section 1613.3.2 — Site class definitions

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

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

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

analysis in accordance with Section
21.1

For SI: 1ft/s = 0.3048 m/s 1lb/ft^2 = 0.0479 kN/m^2

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 = B and S_S = 0.813 g, F_a = 1.000

TABLE 1613.3.3(2)
VALUES OF SITE COEFFICIENT F_v

Site Class	Mapped Spectral Response Acceleration at 1–s Period				
	$S_1 \leq 0.10$	$S_1 = 0.20$	$S_1 = 0.30$	$S_1 = 0.40$	$S_1 \geq 0.50$
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 = B and S_1 = 0.270 g, F_v = 1.000

Equation (16-37):	$S_{MS} = F_aS_S = 1.000 \times 0.813 = 0.813$ g
Equation (16-38):	$S_{M1} = F_vS_1 = 1.000 \times 0.270 = 0.270$ g

Section 1613.3.4 — Design spectral response acceleration parameters

Equation (16-39):	$S_{DS} = \frac{2}{3} S_{MS} = \frac{2}{3} \times 0.813 = 0.542$ g
Equation (16-40):	$S_{D1} = \frac{2}{3} S_{M1} = \frac{2}{3} \times 0.270 = 0.180$ 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} = 0.542 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} = 0.180 g, Seismic Design Category = C

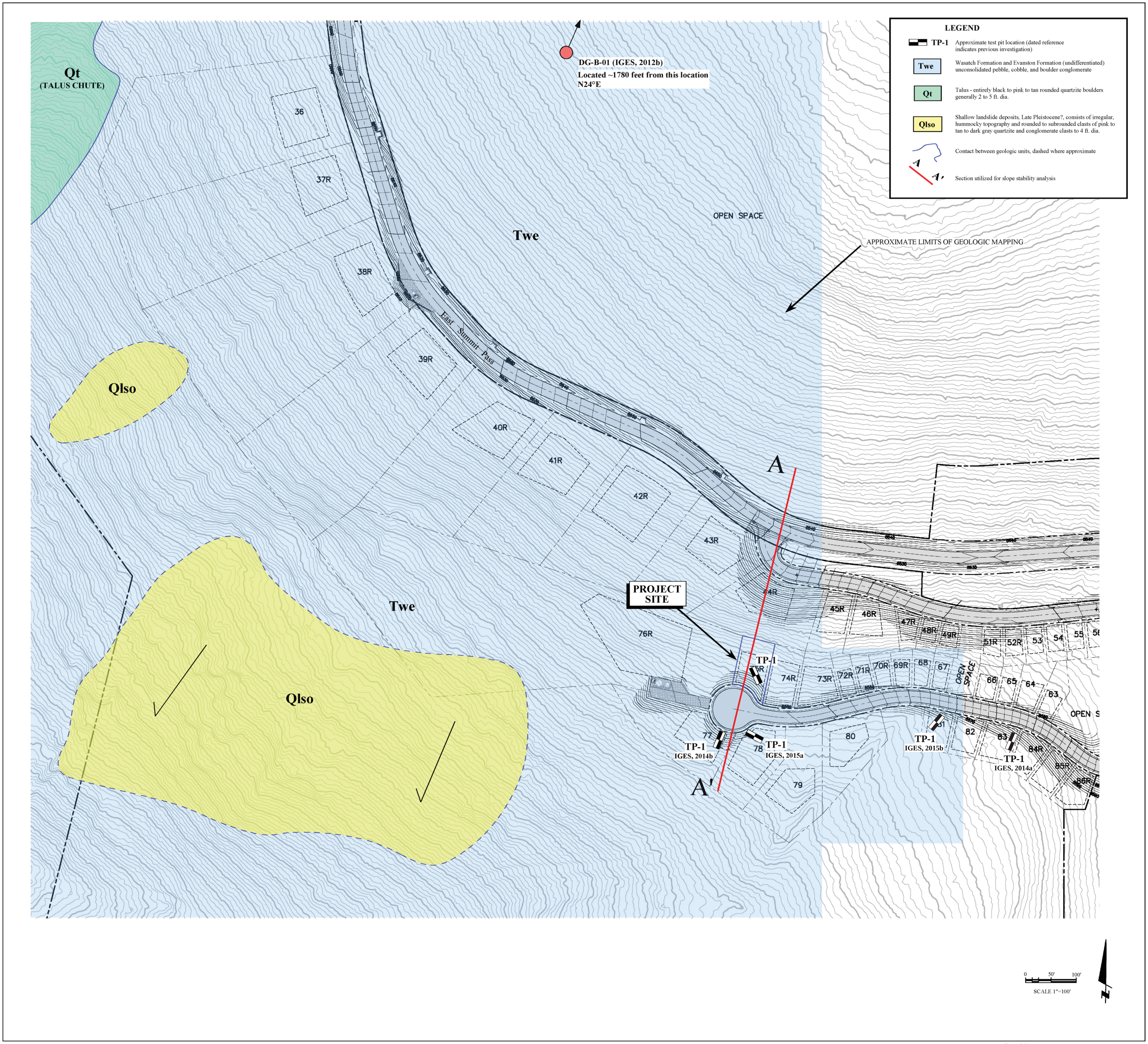
Note: When S_1 is greater than or equal to 0.75g, the Seismic Design Category is **E** for buildings in Risk Categories I, II, and III, and **F** for those in Risk Category IV, irrespective of the above.

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

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

References

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



LOCAL GEOLOGY AND GEOTECHNICAL MAP

Base Map:
Topographic map prepared by NV5, undated

IGES®
Project No. 02347-001

Geotechnical Investigation
Lot 75R of Summit Eden Phase 1C
8452 East Spring Park
Weber County, Utah

Plate
1