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GEOTECHNICAL AND GEOLOGIC HAZARD INVESTIGATION
Thomas Quinn Living Trust Property
Parcel No. 20-035-0033, Old Snowbasin Road
Unincorporated Weber County, Utah

IGES Project No. 02350-001

September 16, 2016

Prepared for:

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APPENDICES

1.0 EXECUTIVE SUMMARY

This report presents the results of a geotechnical and geologic hazard investigation conducted for the *Thomas Quinn Living Trust Property*, located along Old Snowbasin Road, near the town of Huntsville, in Weber County, Utah. Based on the surface and subsurface conditions encountered at the property, it is our opinion that the proposed development is feasible provided that the recommendations presented in this report are incorporated into the design and construction of the project.

- The site is overlain with approximately 1½ to 2 feet of topsoil, although sequences of topsoil as thick as 3.5 feet were observed locally. However, the prevailing earth materials encountered consisted of bedrock (Norwood Tuff), which readily disaggregates to sandy lean clay (CL) grading to sandy fat clay (CH) within the upper 10 feet. Below about 10 feet, the bedrock becomes less weathered, and generally disaggregates to clayey sand (SC). Large rocks (boulders, cobbles) are present but are uncommon, and are not expected to pose a significant issue for the basement excavation. The bedrock present at the site was readily excavatable using conventional earth-moving equipment within the upper 10 feet. Below 10 feet, excavation became increasingly difficult, but was still possible with a large tracked excavator. Adverse geologic conditions were not encountered on the property.

- Footings for the proposed residential structure should be founded either *entirely* on bedrock <u>or</u> *entirely* on a minimum of 2 feet of structural fill. Native/fill transition zones are not allowed.

- Shallow spread or continuous wall footings constructed on competent bedrock may be proportioned utilizing a maximum net allowable bearing pressure of **4,500 pounds per square foot (psf)**. However, if the foundations are underlain by a minimum of 2 feet of structural fill, a maximum net allowable bearing pressure of **3,500 psf** should be used for design. The net allowable bearing values presented above are for dead load plus live load conditions.

- Based on soil classifications for the near-surface soils, the near-surface soils are expected to provide poor pavement support. Pavement sections should consist of *3 inches of asphalt over 6 inches of road base over 8 inches of subbase* for the driveway. We recommend that the owner give consideration to placing a separation fabric between the subgrade and the pavement aggregate section.

NOTE: The scope of services provided within this report are limited to the assessment of the subsurface conditions at the subject site. The executive summary is provided solely for purposes of overview and is not intended to replace the report of which it is part and should not be used separately from the report.

2.0 INTRODUCTION

2.1 PURPOSE AND SCOPE OF WORK

This report presents the results of a geotechnical and geologic hazard investigation conducted for the *Thomas Quinn Property*, located along Old Snowbasin Road, near the town of Huntsville, in Weber County, Utah. Based on the surface and subsurface conditions encountered at the property, it is our opinion that the proposed development is feasible provided that the recommendations presented in this report are incorporated into the design and construction of the project. The purposes of this investigation were:

- To assess the nature and engineering properties of the subsurface soils across the site;
- To provide recommendations for general site grading and design and construction of foundations, slab-on-grades, exterior concrete flatwork, and drainage; and
- To provide an assessment of the geologic hazards that may impact the property.

The scope of work completed for this study included a literature and aerial imagery review, site reconnaissance, subsurface exploration, soil sampling, laboratory testing, engineering analyses, and preparation of this report. Our services were performed in accordance with our proposal dated June 27, 2016 and your signed authorization. The recommendations contained in this report are subject to the limitations presented in the "Limitations" section of this report (Section 7.1).

2.2 PROJECT DESCRIPTION

The property is located along Old Snobasin Road (Highway 226) in Weber County, near the City of Huntsville, Utah, approximately 1 mile south of Pineview Reservoir in the southeastern quarter of Section 23, Township 6 North, Range 1 East (Figure A-1). The property is bound on all sides by largely densely vegetated, undeveloped lands, though a residence has been developed on the top of the ridge located approximately 315 feet west of the western margin of the property. We understand that the proposed development will consist of a single family residence, an accessory building with an in-law apartment on top, a driveway, and utilities over an approximately 5.8-acre site. Construction plans were not available at the time of this report; however, we anticipate the new home will be a one- to two-story conventional wood-frame structure with a basement, founded on spread footings. The accessory structure is assumed to be an on-grade structure, most likely consisting of a detached garage or work shop, with a smaller residential unit on the second story. We also understand that the home will have on-site sewage disposal, and that a percolation test has already been conducted and accepted by the County.

3.0 METHODS OF STUDY

3.1 LITERATURE REVIEW

A number of pertinent publications were reviewed as part of this assessment. King, et al. (2008) provides the most recent 1:24,000 scale geologic mapping that covers the area in which the property of interest is located, in the form of the Snow Basin Geologic Quadrangle (Figure A-2). Coogan and King (2016) provide the most recent published geologic mapping that covers the project area, but at a regional (1:62,500) scale. A United States Geological Survey (USGS) topographic map for the Snow Basin Quadrangle (2014) provides 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), liquefaction (Christenson and Shaw, 2008c; Anderson et al., 1994), and radon (Solomon, 1996) that cover the project area were also reviewed.

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 Cited* (Section 8.0) section of this report.

3.2 FIELD INVESTIGATION

The field exploration program initially involved site reconnaissance and field mapping, which was subsequently followed by subsurface exploration. Site reconnaissance and geologic mapping of the property was performed on August 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.

The subsurface component of the field investigation was performed on August 3, 2016. Four (4) exploration test pits were excavated in various locations across the property to depths generally ranging from 11 to 13 feet below existing grade. The exploration test pits were excavated with the aid of a Caterpillar 315C tracked excavator. Practical refusal on hard bedrock (Norwood Tuff) was encountered in all of the test pits. The excavations were spotted in specific areas that would:

- Address potential or suspicious geologic hazard areas, as identified in the site reconnaissance;
- Provide representative coverage of the subsurface conditions across the property, including depth to bedrock and groundwater (if present); and
- Minimize disturbance to the dense native foliage on the property.

The *Local Geology Map,* Figure A-3 in Appendix A, shows the approximate location of the exploration test pits and the surficial geologic materials across and adjacent to the property as mapped from the site reconnaissance and encountered in the test pits. Subsurface conditions as encountered in the exploration test pits were logged at the time of our investigation by a licensed geologist. The test pit logs are presented in Figures A-4 through A-7 of Appendix A. A *Key to Soil Symbols and Terminology* is presented as Figure A-8 and a *Key to Physical Rock Properties* is presented as Figure A-9.

Bulk soil samples were obtained from the test pit explorations. Due to the coarse/hard nature of the subsurface materials, no ‘undisturbed’ tube samples were able to be collected. All soil samples were transported to the IGES laboratory for testing to evaluate the engineering properties of the earth materials observed.

3.3 LABORATORY INVESTIGATION

Geotechnical laboratory tests were conducted on selected soil samples obtained during our field investigation. The laboratory testing program was designed to evaluate the engineering characteristics of onsite earth materials. Laboratory tests conducted during this investigation include:

- In situ moisture content (ASTM D7263)
- Atterberg Limits (ASTM D4318)
- Fines Content (% passing the #200 sieve) (ASTM D1140)
- Gradation (ASTM D6913)
- Direct Shear Test (ASTM D3080)
- Corrosion Suite
- Point Load Test (ASTM D5731)

Results of the laboratory testing are included with this report in Appendix B.

3.4 ENGINEERING ANALYSIS

Engineering analyses were performed using soil data obtained from the laboratory test results and empirical correlations from 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.

4.0 GEOLOGIC CONDITIONS

4.1 GEOLOGIC SETTING

4.1.1 Regional Geology

The Thomas Quinn Living Trust property is situated in the foothills of the Wasatch Mountains in the southern part of the Ogden Valley. Ogden Valley separates the western part of the Wasatch Range from the Bear River Range to the east, a subgroup of mountains that are part of the parent Wasatch Range. 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 Thomas Quinn Living Trust property is located approximately 6.5 miles to the east 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).

4.1.2 Local Geology from Literature and Aerial Imagery

According to King et al. (2008), the property is mainly located on deposits that have been mapped as possible landslide blocks of the Norwood Tuff (see Figure A-2; unit Qms?(Tn)). This map also shows the northwestern corner and southern margin of the property to be underlain by undivided landslide/slump[1] and colluvial[2] deposits (unit Qmc), and the northeastern margin and southwestern corner of the property to be underlain by landslide and slump deposits (unit Qms). Additionally, a notable north-south trending landslide headscarp[3] has been mapped approximately 500 feet east of the western margin of the property. As shown in Figure A-2, the entire mountainous area within several miles of the property is seen to be predominantly landslide deposits with pockets of in-place Norwood Tuff (Tn) scattered throughout.

The more recent, though regional, Coogan and King (2016) map displays the subject property to be entirely underlain by Holocene to middle Pleistocene-aged landslide deposits. This map is largely consistent with King et al. (2008).

No faults have been mapped within one mile of the property, and no faults, either active or inactive, have been mapped on or projecting towards the property. An active fault is defined by the Weber County Code of Ordinances as "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)

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 year 1946, which were taken prior to the development of any of the residences along this section of Old Snow Basin Road. 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 or projecting toward the subject property. However, the landslide headscarp to the west of the property as mapped by King, et al. (2008) and Coogan and King (2016) was observed, though it could not be easily delineated from the photographs.

[1] Slump: A landslide characterized by a shearing and rotary movement of a generally independent mass of rock or earth along a curved slip plane (concave upward). (AGI, 2005)
[2] Colluvium: A general term applied to any loose, heterogeneous, and incoherent mass of soil material and/or rock fragments deposited by rainwash, sheetwash, or slow continuous downslope creep, usually collecting at the base of gentle slopes or hillsides. (AGI, 2005)
[3] Headscarp: The abrupt scarp at the head, or top, of a landslide or slump. (AGI, 2005)

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 on the property in the imagery. The southeastern approximately half of the property was observed to contain dense scrub oak, while the northwestern approximately half of the property was observed to be covered in low-lying shrubs with few trees. The ground appeared to be uneven in places, but not hummocky, and no headscarps were observed. Three (3) historic test pits were observed to have been dug on and adjacent to the property sometime between October of 1997 and September of 2003. The east-central margin of the property and extending downslope to the east appeared to be most suspicious from the imagery, as it was observed to have somewhat of a bowl-like shape and had a more irregular vegetation pattern than the rest of the property.

No LiDAR data for the property was available to be reviewed at the time of this investigation.

4.1.3 Local Geology from Site Reconnaissance and Subsurface Investigation

Site reconnaissance and geologic mapping of the property was performed as part of the fieldwork for this project, and served largely as the basis upon which the test pit locations were determined. 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-3 in Appendix A is a site-specific geologic map of the property and adjacent areas.

Most of the property was observed to be in its natural state and covered in very dense scrub oak. A thin colluvial (Qc) cover was found across much of the property, evidenced by scattered quartzite cobbles up to 10 inches in diameter. Norwood Tuff clasts[4] were also found in association with the quartzite, though commonly in a much lower proportion than the quartzite. Norwood Tuff (Tn) bedrock was observed to outcrop on the knob forming the topographic high of the property, along the hillslope to the west, along the road cut to the north, and in the areas surrounding the historic test pits.

In some places, large shrinkage cracks were observed in the surficial soil that were observed to be as much as one inch wide. These cracks are indicative of the presence of swelling (fat) clays in the soil profile, and are commonly associated with weathered volcanic ash deposits.

No surface water or groundwater was observed on the property during either the site reconnaissance or subsurface investigation.

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

In order to observe the subsurface conditions across the property, four test pits were excavated. Topsoil encountered was generally a well-developed fat clay between 1.5 and 3.5 feet thick and sometimes exhibited desiccation cracks. Norwood Tuff block-and-ash and tuff bedrock was encountered in all four of the excavations, with the top of the unit encountered between 1.5 and 9 feet below existing grade. The Qc unit was only observed in the subsurface in TP-3.

Based upon the surface reconnaissance and subsurface investigation, two distinct geologic units were differentiated on the property (see Figure A-3). Both of these units are discussed in turn below.

Qc (Quaternary Colluvium)
This unit was mapped across the entire southern portion of the property, as well as much of the north and central portion of the property. The unit is characterized by the predominant presence of occasional reddish-orange subrounded to subangular quartzite cobbles up to 10 inches in diameter, though these were commonly only a couple inches in diameter or smaller. Minor angular Norwood Tuff clasts up to 3 inches in diameter were also found with the quartzite, and both types of clast were found in a silty lean clay matrix. This unit was as much as 4 feet thick.

Tn (Tertiary Norwood Tuff)
Norwood Tuff bedrock was found to be highly silty and sandy, and was commonly weathered (chemically altered) to fat clay (CH). The bedrock was found to be generally a finely (though faintly) bedded rhyolitic[5] lithic[6]-crystal tuff[7] with angular, fine to medium-grained quartz grains and crystals in a glassy matrix. Tuff clasts were typically light gray to white, angular, and moderately weathered and broken, though not crumbly to the touch. In the subsurface, the unit was found as a combination of decomposed volcanic ash, block-and-ash deposits, and friable, moderately to poorly competent tuff. The unit also displayed abundant calcium carbonate-filled subhorizontal and subvertical fracturing in the subsurface. The unit is at least 10 feet thick across the property.

4.2 GEOLOGIC HAZARD ASSESSMENT

Geologic hazards can be defined as naturally-occurring geologic conditions or processes that could present a danger to human life and property. 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 in the following

[5] Rhyolite: The fine-grained equivalent of granite. (AGI, 2005)
[6] Lithic: Said of a medium-grained sedimentary rock or of a pyroclastic deposit if either one contains abundant fragments of previously formed rocks. (AGI, 2005)
[7] Tuff: Consolidated or cemented volcanic ash and lapilli. (AGI, 2005)

paragraphs is based upon both qualitative and quantitative 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 a negative way. Areas with a low-risk determination for a particular geologic hazard generally 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 and 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 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 generally always require additional site-specific hazard investigations and associated mitigation practices. For areas with a high-risk geologic hazard, simple avoidance is often considered.

The following are the results of the geologic hazard assessment for the property.

4.2.1 Landslides/Mass Movement

Landslides and mass movement hazards pose the most risk to development on the property. The property is entirely within an area previously mapped as either having or possibly having landslide deposits (King, et al., 2008; Coogan and King, 2016), and aerial imagery displayed a landslide headscarp near the property (to the west-southwest). Additionally, the site reconnaissance observed the steepest slopes along the eastern margin of the property as a possible older landslide headscarp and identified the most irregular topography on the property to be located in the southwestern portion of the property.

Based on this information, test pits were specifically spotted to observe subsurface conditions in these areas. TP-1 was spotted near the eastern margin of the property with the intention of intersecting the basal slide plane for the possible older landslide headscarp, if present. However, no slide planes, slickensides, or evidence of shear was encountered in the test pit, and a well-developed topsoil 1.5 feet thick was observed to be developing on Norwood Tuff block-and-ash deposits (see Figure A-4). Similarly, TP-2 was spotted further south near the eastern margin of the property, and was a reopening and enlargement of a historic test pit at that location. Like TP-1, no slide planes, slickensides, or evidence of shear was encountered in the test pit, and a well-developed topsoil 1.5 to 2 feet thick was observed to be developing on weathered Norwood Tuff bedrock (see Figure A-5). TP-3 was spotted near the west-central margin of the property to address subsurface conditions near the topographic high of the property and near Norwood Tuff outcropping at the surface. No landslide evidence was observed in the test pit and near-surface bedrock was encountered, as anticipated (see Figure A-6). TP-4 was spotted near the southwestern corner of the property to observe subsurface conditions in the most irregular topography on the property. Though this test pit encountered a possible buried paleosol[8] and the deepest Norwood Tuff bedrock depth (approximately 8.5 feet below existing grade), there were no slide planes, slickensides, or evidence of shear encountered in the test pit (see Figure A-7). Additionally, the test pit had the thickest topsoil development of the four test pits (approximately 3.5 feet thick), suggesting that the slope has been stable for a significant amount of time.

Given this data, the landslide hazard risk associated with development on the property is considered to be low to moderate. Though no landslide evidence was observed in the subsurface, the Norwood Tuff is a known landslide-prone unit. As such, there is always some associated risk of a landslide hazard when developing on this unit.

4.3 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).

To account for site effects, site coefficients that vary with the magnitude of spectral acceleration and *Site Class* are used. Site Class is a parameter that accounts for site amplification effects of soft soils and is based on the average shear wave velocity of the upper 100 feet; based on our field exploration and our understanding of the geology in this area, the subject site is appropriately classified as Site Class C (*soft rock*). Based on IBC criteria, the short-period site coefficient (F_a) is 1.057, and the long-period site coefficient (F_v) is 1.511. Based on the design spectral response accelerations for a *Building Risk Category* of I, II, III, or IV, the site's *Seismic Design Category* is D. The short- and long-period *Design Spectral Response Accelerations* are presented in Table 4.3;

[8] Paleosol: A soil that formed on a landscape in the past with distinctive morphological features resulting from a soil-forming environment that no longer exists at the site.

a summary of the *Design Maps* analysis is presented in Appendix C. The *peak ground acceleration* (PGA) may be taken as $0.4*S_{MS}$.

Table 4.3
Short- and Long-Period Spectral Accelerations for MCE

Parameter	**Short Period (0.2 sec)**	**Long Period (1.0 sec)**
MCE Spectral Response Acceleration (g)	$S_S = 0.857$	$S_1 = 0.289$
MCE Spectral Response Acceleration Site Class C (g)	$S_{MS} = S_sF_a = 0.906$	$S_{M1} = S_1F_v = 0.437$
Design Spectral Response Acceleration (g)	$S_{DS} = S_{MS}*{}^2/_3 = 0.604$	$S_{D1} = S_{M1}*{}^2/_3 = 0.291$

4.4 OTHER GEOLOGIC HAZARDS

There are several hazards in addition to landslides and seismicity that, if present at the site, should be considered in the design of the proposed structures. IGES has assessed the potential for the presence of other geologic hazards, including liquefaction, rockfall, surface fault rupture, and debris flow and flooding. Based on the observed geology, hydrology, stratigraphy, and topography, the potential for these geologic hazards impacting the site is considered low. Detailed discussions about these potential hazards are presented in the following paragraphs.

4.4.1 Liquefaction

Both Anderson, et al. (1994) and Christensen and Shaw (2008c) designate the area on which the property is located as being in a very low potential liquefaction area. Additionally, shallow groundwater was not encountered in the test pits, bedrock was found to be shallow, and granular soils were largely absent. Given this data, the risk associated with earthquake-induced liquefaction is considered to be low.

4.4.2 Rockfall

IGES observed that there are no cliffs, exposed outcrops on steep slopes, or other geomorphic features that would result in a rockfall hazard at the site. Therefore, the rockfall hazard for the property is considered to be low.

4.4.3 Surface Fault Rupture

There are no active or inactive faults currently mapped on, or trending toward the site (King, et al. (2008); UGS and USGS (2006); Coogan and King (2016)). Therefore, the risk associated with surface fault rupture hazard for the property is considered to be low.

4.4.4 Debris Flow and Flooding

Debris-flows typically occur on existing alluvial fans located at the mouth of active canyons, while flooding typically occurs in drainage channels and lowland areas within a drainage basin. With the site being located near a topographic high, major debris-flow sources and active canyons are absent from the property. The property is therefore deemed to have a low potential for being impacted by debris-flows.

Similarly, being near a topographic high and the absence of drainage channels and lowland areas on the property likely preclude the property from being prone to flooding hazards. The FEMA flood map covering the property has not been printed by FEMA because there are no special flood hazard areas within the map area. Therefore, the flood hazard potential for the property is considered to be low.

4.4.5 Radon

Solomon (1996) conducted a radon investigation of the Ogden Valley, and the property is located just outside of the study area. However, the property is adjacent to a large swath of land designated to have a moderate radon hazard, and the property is underlain by the Norwood Tuff of volcanic origin. Therefore, the risk associated with the radon hazard for the property is considered to be moderate.

5.0 GEOTECHNICAL CONDITIONS

5.1 SURFACE CONDITIONS

The site is in a relatively natural state, aside from a couple four-wheeler trails and a temporary driveway. Dense scrub oak is found across most of the eastern and southern portions of the property, with native shrubs and grasses covering the rest of the property. The southern approximately ½ of the property drains to the southeast, while the northern approximately ½ of the property drains to the northeast. The elevation across the property ranges from approximately 5,423 feet (msl) along the west-central margin of the property to approximately 5,308 feet at the southeastern corner of the property. The typical gradient across the property ranges from about 19% to 27%.

5.2 SUBSURFACE CONDITIONS

The subsurface soils were investigated by excavating a total of four (4) exploration test pits across the property. Generally, the depth of the exploration test pits ranged from 11 to 13 feet, and refusal on competent Norwood Tuff bedrock was encountered in all four test pits. The locations of the test pits are illustrated on Figure A-3, *Local Geology Map*; detailed test pit logs are presented in Figures A-4 through A-7. The earth materials encountered in the exploration test pits were visually classified and logged by an IGES licensed geologist. The subsurface conditions encountered during our investigation are discussed below.

5.2.1 Earth Materials

Based on our observations, the site is generally covered by a veneer of topsoil ranging in depth from 1½ to 2 feet thick, although topsoil as thick as 3½ feet thick was observed locally. The topsoil is underlain by earth materials consisting of clayey colluvium and/or bedrock (Norwood Tuff). Descriptions of the earth materials encountered are presented in the following paragraphs.

<u>Topsoil</u>: Generally consists of dark brown to brownish black fat clay (CH) with varying amounts of sand and gravel. This soil typically exhibits high plasticity, and is characterized by a relatively high organic component, mostly roots and decayed vegetation. The topsoil unit was encountered in all of the exploration test pits and potholes, and is expected to cover most of the site.

<u>Surficial Soils</u>: Where identified, surficial soils consisted of colluvium (slopewash), and is typically comprised of stiff to very stiff sandy lean clay and lean clay with sand (CL), often transitional to sandy fat clay (CH). The coarse fraction was typically 30 percent or less; gravel-size constituents are estimated to comprise less than 5 percent by weight.

<u>Bedrock</u>: The prevailing earth materials onsite consists of the Norwood Tuff unit, which is comprised of weathered volcanic ash and block-and-ash deposits. This unit is highly weathered within the upper 10 feet and readily disaggregates to soils classifying as sandy lean clay and lean

clay with sand (CL), although at depth the less-weathered tuff generally disaggregates to soils classifying as clayey sand (SC). The liquid limit of the fines is somewhat high, ranging from the high 40s to the high 60s; accordingly, much of the clay constituents will classify as fat clay (CH).

The lines shown on the enclosed logs represent the approximate boundary between the different earth materials. Due to differing depositional natures of natural earth materials, care should be taken in interpolating subsurface conditions between and beyond the exploration locations.

5.2.2 Groundwater

Groundwater was not encountered in any of the exploration test pits completed during this investigation, and is not expected to impact the development. Due to the season of our investigations (late summer), we anticipate groundwater levels to be below their seasonal low. It is our experience that during snowmelt, runoff, irrigation on the property and surrounding properties, high precipitation events, and other activities the groundwater level can rise several feet. Fluctuations in the groundwater level could occur over time.

5.2.3 Strength of Earth Materials

A consolidated-drained direct shear test was completed under drained conditions on a remolded sample of the prevailing Norwood Tuff. The test results indicated that the sample tested had a friction angle of 29 degrees and a cohesion of 713 psf (peak strength values).

Five samples of the Norwood Tuff were tested to assess the uniaxial compressive strength of intact, moderately weathered bedrock. A wide range of values was obtained, ranging from 782 psi to 18,032 psi, with an average of 6,951 psi and a standard deviation of 6,972. If the high and low values are neglected, an average of 5,313 psi with a standard deviation of 3,575 is obtained.

The test results suggest that the prevailing Norwood Tuff is quite variable with respect to strength, but will nonetheless provide good bearing capacity for structures.

5.2.4 Expansive Soils

Fat CLAY (CH) was described in the field descriptions for the topsoil in TP-1, TP-3, and TP-4, and was confirmed in the laboratory testing for TP-1. Soils classifying as fat clay are potentially expansive; these soils are typically stiff to hard, moist, and have a “greasy” luster. Swelling 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.

Assuming the new home will be founded on bedrock, expansive soils are not expected to impact the home. However, expansive soils could impact hardscape (e.g. driveways and patios).

5.3 SLOPE STABILITY

The stability of the existing natural slope has been assessed in accordance with methodologies set forth in Blake, et al. (2002) with respect to Section A-A', illustrated on Figure A-3. Our section is necessarily limited in length due to the limitations of available topographic data; however, the section does cover the entire buildable area and the entire property. The stability of the slope was modeled using SLIDE, a computer application incorporating (among others) Spencer's Method of analysis. Calculations for stability were developed by searching for the minimum factor of safety for a circular-type failure. Homogeneous earth materials and arcuate failure surfaces were assumed. Analysis was performed for the following cases:

a) Static analysis of existing geometry
b) Pseudo-static analysis of existing geometry

Pseudo-static (seismic screening) analysis of the proposed slope was performed in general conformance with Blake, et al. (2002). The design seismic event was taken as the ground motion with a 2 percent probability of exceedance in 50 years (2PE50). Based on information provided on the USGS website ground motion calculator, the Peak Ground Acceleration (PGA) associated with a 2PE50 event is estimated to be 0.34g. Half of the PGA, (0.17g), was taken as the horizontal seismic coefficient (k_h) (Hynes and Franklin, 1984), and used in the pseudo-static seismic screen analysis.

Groundwater was not encountered during our subsurface investigation, and accordingly was not modeled in our analysis. If the new home will have an on-site sewage absorption system (septic system), introduction of water into the subsurface could conceivably impact slope stability. Our analysis assumes that a septic system, if any, would be placed well down-hill from the proposed home.

Based on laboratory test results and or observation of the subsurface, soil strength was modeled as having a friction angle of 29 degrees and a cohesion (apparent cohesion) of 700 psf. Our model assumes the slope is comprised entirely of Norwood Tuff bedrock.

Based on our analysis, the existing slope meets the minimum factors-of-safety of 1.5 for static and 1.0 for seismic conditions. The results of the stability analyses are presented in Appendix D.

6.0 CONCLUSIONS AND RECOMMENDATIONS

6.1 GENERAL CONCLUSIONS

Supporting data upon which the following 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 soils encountered in the exploratory test pits and the anticipated design data discussed in the *Project Description* section of this report (Section 2.2). 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.

Based on the subsurface conditions encountered at the site, it is our opinion that the subject site is suitable for the proposed development provided that the recommendations presented in this report are implemented into the design and construction of the project. In general, we anticipate the home can be completed using standard construction practices. We anticipate that the foundation for the proposed residential structure will consist of conventional shallow spread footings founded entirely on competent native earth materials or entirely on a minimum of two feet of structural fill.

The following sub-sections present our recommendations for general site grading, pavement design, design of foundations, slabs-on-grade, lateral earth pressures, moisture protection and preliminary soil corrosion.

6.2 EARTHWORK

6.2.1 General Site Preparation and Grading

Below proposed structures, engineered fill, and man-made improvements, all vegetation, debris, and undocumented fill soils should be removed. Any existing utilities should be re-routed or protected in-place. The exposed native earth materials should then be proof-rolled with heavy rubber-tired equipment such as a scraper or loader (proof-rolling not required where competent bedrock is exposed). 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 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.

6.2.2 Over-Excavations

The prevailing earth materials anticipated at foundation grade are expected to consist largely of bedrock (Norwood Tuff), or coarse, dense colluvium. Regardless, unanticipated adverse soil conditions could be encountered in any excavation. Accordingly, soft, porous, or otherwise unsuitable soils beneath structural elements may need to be over-excavated and replaced with structural fill. If over-excavation is required, the excavations should extend a minimum of 1 foot laterally for every foot of depth of over-excavation. Excavations should extend laterally at least two feet beyond slabs-on-grade. Structural fill should consist of granular materials and should be placed and compacted in accordance with the recommendations presented in this report.

Prior to placing structural fill, all excavation bottoms should be scarified to at least 6 inches, moisture-conditioned as necessary to at or slightly above optimum moisture content (OMC) and compacted to at least 95 percent of the maximum dry density (MDD) as determined by ASTM D-1557 (modified Proctor). The scarification recommendation need not apply where competent bedrock is exposed.

6.2.3 Excavation Stability

The Contractor is responsible for site safety, including all temporary slopes and trenches excavated at the site and 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. Within the upper 5 to 8 feet soil types are expected to consist largely of Type C soils (sand and gravel), which will be underlain by bedrock. 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 are encountered, or where raveling sands or gravels are exposed on the trench walls, 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. Sloping the sides at 1.5H:1V (34 degrees) in accordance with OSHA Type C soils (sands and gravel) may be used as an alternative to shoring or shielding. Where bedrock is encountered, sidewall slopes can be constructed at 0.5H:1V. Vertical slopes in bedrock may be allowed for specific cases, pending written authorization by IGES upon site observation.

A qualified person should inspect all excavations frequently to evaluate stability. The Contractor is ultimately responsible for trench and site safety. Pertinent OSHA requirements should be met to provide a safe work environment.

6.2.4 Structural Fill and Compaction

All fill placed for the support of structures should consist of structural fill. Structural fill may consist of excavated onsite soils and/or bedrock, or an approved imported granular soil. The fines should have a liquid limit less than 25 and plasticity index less than 7. Structural fill should be free of vegetation and debris, and contain no rocks larger than 4 inches in nominal size (6 inches in greatest dimension). Soils not meeting the aforementioned criteria may be suitable for use as structural fill but must be approved by IGES prior to use. However, soil classifying as Fat CLAY (CH) (based on USCS classification) are generally not suitable for use as structural fill. It should be noted that soils classifying as Fat CLAY are commonly encountered on the project site.

All structural fill should be placed in maximum 8-inch loose lifts if compacted by small hand-operated compaction equipment, maximum 10-inch loose lifts if compacted by light-duty rollers, and maximum 12-inch loose lifts if compacted by heavy duty compaction equipment that is capable of efficiently compacting the entire thickness of the lift. These values are *maximums*; the Contractor should be aware that thinner lifts may be necessary to achieve the required compaction criteria. We recommend that all structural fill be compacted on a horizontal plane, unless otherwise approved by IGES. Structural fill placed beneath footings 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 – compacting dry of optimum is discouraged. Any imported fill materials should be approved by IGES prior to importing. Also, prior to placing any fill, the excavations should be observed by IGES to assess whether unsuitable materials have been removed.

6.2.5 Oversized Material

In general, the prevailing Norwood Tuff mechanically disaggregates to soils classifying as clayey sand (SC) or sandy clay (CH or CL); however, some particularly unweathered blocks of bedrock may be resistant to mechanical break-down during excavation, thereby creating *over-size materials* (cobbles and boulders, greater than 6 inches in greatest dimension) If encountered, over-size material should be segregated out of any earth materials to be utilized as structural fill. Oversize material may also be crushed and mixed with local soils to be used as structural fill.

6.3 FOUNDATIONS

Based on our field observations and considering the presence of relatively competent native earth materials (Norwood Tuff), we recommend that the footings for the proposed home be founded either *entirely* on competent bedrock or *entirely* on a minimum of 2 feet of structural fill. Native/fill transition zones are not allowed, nor are bedrock/soil transition zones allowed. If part of the foundation excavation exposes colluvium, foundations should be deepened such that the entire foundation system is placed on bedrock. Exceptions may be allowed for small areas; any exception must be approved by IGES in writing prior to placement of steel or concrete.

Shallow spread or continuous wall footings constructed on competent bedrock may be proportioned utilizing a maximum net allowable bearing pressure of **4,500 pounds per square foot (psf)**. However, if the foundations are underlain by a minimum of 2 feet of structural fill, a maximum net allowable bearing pressure of **3,500 psf** should be used for design. The net allowable bearing values presented above are 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 allowable bearing capacity may be increased by one-third for short-term loading (wind and seismic).

All 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 (e.g., *a continuously heated structure*), may be established at higher elevations; however, a minimum depth of embedment of 12 inches is recommended for confinement purposes.

6.4 SETTLEMENT

6.4.1 Static Settlement

Static settlement of properly designed and constructed conventional foundations, founded as described above, are anticipated to be on the order of ¾ inch or less. Differential settlement is expected to be half of total settlement over a distance of 30 feet.

6.4.2 Dynamic Settlement

Based on the field data collected for this site, it is our opinion that the prevailing bedrock (Norwood Tuff) encountered throughout the site will exhibit negligible seismically-induced settlement during a MCE seismic event. Similarly, properly compacted structural fill is expected to exhibit minor seismically-induced settlement during a MCE seismic event.

6.5 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 bedrock or granular fill should be used.

Ultimate lateral earth pressures from natural soils and *granular* backfill acting against retaining walls and buried structures may be computed from the lateral pressure coefficients or equivalent fluid densities presented in Table 6.5.

These coefficients and densities assume no buildup of hydrostatic pressures; the force of the water should be added to the presented values if hydrostatic pressures are anticipated. If select imported granular backfill will be used, the values presented in Table 6.5 can be re-evaluated by IGES upon request and subsequently modified as appropriate.

Clayey soils drain poorly and may swell upon wetting, thereby greatly increasing lateral pressures acting on earth retaining structures. Therefore, clayey soils, particularly soils classifying as fat clay (CH), should not be used as retaining wall backfill. Backfill should consist of either native granular soil or sandy imported material with an Expansion Index (EI) less than 25 and a fines content less than 30 percent.

Table 6.5
Recommended Lateral Earth Pressure Coefficients

Condition	Lateral Pressure Coefficient	Equivalent Fluid Density (pcf)
Active (Ka)	0.33	40
At-rest (Ko)	0.50	60
Passive (Kp)	3.0	360

Walls and structures allowed to rotate slightly should use the active condition; if the element is 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.

6.6 CONCRETE SLABS-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 structural fill or competent native earth materials. The gravel should consist of free-draining gravel or road base with a ¾-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. Gravel materials not meeting the aforementioned criteria may be appropriate for construction; alternate materials should be evaluated on a case-by-case basis and should be approved by IGES prior to use.

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"x 4" W2.9 x W2.9 welded wire mesh within the middle third of the slab. We recommend a minimum slab thickness of 4 inches. We recommend that concrete be tested to assess that the slump and/or air content is in compliance with the plans and specifications. If slump and/or air content are beyond the recommendations as specified in the plans and specifications, the concrete may not perform as desired. We recommend that concrete be placed in general accordance with the requirements of the American Concrete Institute (ACI).

Our experience indicates that use of reinforcement in slabs and foundations can generally reduce the potential for drying and shrinkage cracking. However, some cracking can be expected as the concrete cures. Minor cracking is considered normal; however, it is often aggravated by a high water/cement ratio, high concrete temperature at the time of placement, small nominal aggregate size, and rapid moisture loss due to hot, dry, and/or windy weather conditions during placement and curing. Cracking due to temperature and moisture fluctuations can also be expected. The use of low slump concrete can reduce the potential for shrinkage cracking; saw cuts in the concrete at strategic locations can help to control and reduce undesirable shrinkage cracks.

6.7 MOISTURE PROTECTION AND SURFACE DRAINAGE

<u>During Construction</u>: Over-wetting the soils prior to, during, or after construction may result in softening and pumping, causing equipment mobility problems and difficulty in achieving compaction. Every effort should be taken to ensure positive drainage away from the access road (driveway) to reduce the potential for mobility issues or difficulty with compaction. The recommended minimum slope is two percent (2%). Moisture should not be allowed to infiltrate the soils in the vicinity of, or upslope from, the access road or driveway.

<u>Residential Structure</u>: Moisture should not be allowed to infiltrate into the soils in the vicinity of the foundations. As such, the following design strategies to minimize ponding and infiltration near the home should be implemented:

- We recommend that hand watering, desert landscaping or Xeriscape be considered within 5 feet of the foundations.
- Roof runoff devices should be installed to direct all runoff a minimum of 10 feet away from structures.
- Irrigation valves shall be a minimum of five feet away from foundation walls and must not be placed within the basement backfill zone.
- The builder should be responsible for compacting the exterior backfill soils around the foundation.
- 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 surrounding pavement.

<u>Foundation Drainage</u>: The majority of soils exposed on the foundation subgrade is expected to consist of relatively poor-draining bedrock. Therefore, IGES recommends a foundation drainage system be incorporated into the design of the home. The foundation drainage system should be designed in accordance with the guidelines presented in the 2012 *International Residential Code* (IRC), Section R405, *Foundation Drainage*.

6.8 SOIL CORROSION POTENTIAL

Laboratory test results indicate that near surface native soils had a sulfate content of 178 ppm. Based on soil conditions encountered during our field investigation and results of chemical testing, the soils are classified as having a 'low' potential for deterioration of concrete due to the presence of soluble sulfate. We recommend that conventional Type I/II Portland cement 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), soluble chloride content, and pH. The test indicated that the onsite soil tested has a minimum soil resistivity of 380 OHM-cm, a soluble chloride content of 642 ppm, and a pH of 7.9. Based on this result, the onsite native soil is considered *severely* corrosive to ferrous metal. Consideration should be given to retaining the services of a qualified corrosion engineer to provide an assessment of any metal that will be in contact with native clay soils.

6.9 PAVEMENT DESIGN

Near-surface soils encountered at the site consist largely of clayey soils, and are therefore expected to provide poor pavement support. The driveway/access road for the project is expected to experience minimal traffic over its lifetime, with the exception of heavy vehicles during construction of the home and associated improvements. Based on our assessment of the subgrade soils, the following pavement sections are presented to provide a 20-year design life for the access. It should be noted that construction traffic will likely account for the majority of the loading during the life of the road.

Table 6.9
Pavement Recommendations

Asphalt (in.)	Roadbase (in.)	Subbase (in.)
3	6	8

Earth materials classifying as Fat CLAY (CH) were identified onsite. Where fat clay is identified on the pavement subgrade, IGES recommends over-excavating an additional 12 inches and replacing with relatively frost-free granular materials (subbase or a pit-run gravel will generally fulfill this requirement). Because of the potential for Fat CLAY to exist beneath the access road/driveway, it is imperative that the pavement section be constructed as recommended and that the pavement be designed to divert surface runoff to gutters and storm drains to minimize the risk of pavement distress arising from expansive soils and/or frost heave. The pavement should be constructed to divert water away from the center of the roadway with a minimum 2 percent slope towards the gutter. Our recommendation to overexcavate and remove the uppermost 12 inches of the Fat CLAY assumes that these moisture and drainage recommendations will be implemented. If these recommendations are not implemented or if poor asphalt quality allows the subgrade to become saturated, differential heave may occur which could cause distress to the pavement section.

Asphalt has been assumed to be a high stability plant mix and base course material composed of crushed stone with a minimum CBR of 70, and subbase (granular borrow) should have a minimum CBR of 30. Road base and subbase should be compacted to 95% of MDD as determined by ASTM D-1557 (Modified Proctor). Asphalt should be compacted to a minimum of 96 percent of the Marshall maximum density. Asphalt and aggregate base material should conform to local requirements. Subgrade should be scarified to a depth of 6 inches and compacted to 95% of MDD as determined by ASTM D-1557. Positive drainage away from roadways must be provided to minimize the potential for saturation of subgrade soils beneath constructed pavements.

The pavement section recommended herein assume that there is no mixing over time between the aggregate section and the underlying native subgrade. In order to prevent mixing or fines migration, and thereby prolong the life of the pavement section, we recommend that the owner give consideration to placing an inexpensive filter fabric between the native soils and the road base, such as the Propex Geotex NW-401 or an IGES-approved equivalent.

6.10 GEOLOGIC HAZARD ASSESSMENT

Based upon the geologic reconnaissance of the property and the subsurface conditions observed in the exploration test pits, geologic features indicative of adverse geologic conditions were not observed on the property. Given the geologic evidence discussed herein, the following conclusions are made:

1. The landslide/mass movement hazard for the property is considered to be low to moderate. Though no landslide evidence was observed in the subsurface, the Norwood Tuff underlies the entire property and is a known landslide-prone unit. As such, there is always some associated risk of a landslide hazard when developing on this unit.

2. Surface fault rupture, liquefaction, rockfall, debris flow, flooding, and shallow groundwater hazards are all considered to be low for the property.

3. In the absence of additional data, the radon hazard for the property is considered to be moderate.

4. Well-developed topsoil, shallow Norwood Tuff bedrock, and an absence of shear or other landslide features across the property are indicative that the property has long been geomorphically stable. As such, **the property is considered suitable for development from a geologic hazards standpoint.**

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

1. The property as a whole is largely underlain by the Norwood Tuff, which is a known landslide-prone unit. Additionally, landslide deposits have been previously mapped on and near the property. Therefore, it is recommended that an IGES engineering geologist observe the foundation excavations for the proposed structures to assess the absence (or presence) of landslide evidence or other adverse geologic conditions.

2. To adequately address the radon hazard for the property, a site-specific radon assessment is recommended.

6.11 CONSTRUCTION CONSIDERATIONS

6.11.1 Expansive Soils

Soils classifying as Fat CLAY (CH) have been identified at the site. Soils classifying as fat clay are potentially expansive; expansive soils can swell upon wetting, thereby inducing damage to foundations, pavement, and other structural elements in contact with site soils. It should be noted that soils classifying as fat clay are not necessarily expansive; however, expansive clays are typically classified as fat clay, so classification should be taken as an indication of possible expansion potential and not a definitive diagnosis.

The proposed home is expected to be founded directly on bedrock (Norwood Tuff), and therefore expansive soils are not expected to impact the proposed home. However, pavement sections (e.g., driveways, patios, etc.) may be impacted by expansive soils. If clay soils with a high degree of plasticity are noted below planned pavement sections, the Owner and/or Contractor should consider testing the subgrade for expansion potential by means of the Expansion Index test (ASTM D4829). If highly expansive soils are identified, the Owner may wish to consider steps to mitigate the effects of expansive soils, such as over-excavation, increasing the pavement section thickness (adding a section of non-expansive subbase), and taking steps to control moisture under pavement sections.

If expansive soils are identified, IGES can provide specific recommendations to help mitigate the impact of expansive soils upon request.

6.11.2 Moisture Control and Slope Stability

Introduction of water into the subsurface arising from effluent from a septic system, leaking pool, or on-site storm water detention/retention, etc., could create an unstable slope condition, although the slope instability (should it occur) would most likely be surficial. As such, the septic system and/or storm water detention/retention structures should be located well down-hill from the home.

If a pool is planned, the pool design should include a method to detect leaks and other design features intended to minimize the chance of the pool leaking, or minimize the chance of a leak going undetected for long periods of time.

7.0 CLOSURE

7.1 LIMITATIONS

The conclusions and recommendations presented in this report are based on our limited geologic literature review, field exploration, laboratory testing and 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 likely that variations in the soil and groundwater conditions 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 presented in this report. In addition, if the scope of the proposed construction changes from that described in this report, IGES should be notified.

It should be noted that these conclusions are based solely upon the geological hazards investigated for this report, and do not pertain to other potential geologic hazards that may be present on the property. Additional geologic hazards and/or geologic hazards initially concluded to pose low risk may be present that may not be identified until construction activities expose adverse geologic conditions. Therefore, the geologic hazard classifications as denoted in this report are potentially subject to change with data collected from additional site-specific observations, particularly the foundation observation.

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.

7.2 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 construction. IGES staff should be on site to assess 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.
- Observation of foundation soils to assess their suitability for footing placement.
- Observation of soft/loose soils overexcavation.
- Observation of temporary excavations and shoring.
- Consultation as may be required during construction.
- Quality control and observation of concrete placement.

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.

8.0 REFERENCES CITED

American Geological Institute (AGI), 2005, Glossary of Geology, Fifth Edition, revised, Neuendorf, K.K.E., Mehl, Jr. J.P., and Jackson, J.A., editors: American Geological Institute, Alexandria, Virginia, 783 p.

Anderson, L.R., Keaton, J.R., and Bay, J.A., 1994, Liquefaction Potential Map for the Northern Wasatch Front, Utah, Complete Technical Report: Utah Geological Survey Contract Report 94-6, 169 p.

Blake, T.F., Hollingsworth, R.A. and Stewart, J.P., Editors (2002), Recommended Procedures for Implementation of DMG Special Publication 117, Guidelines for analyzing and mitigating landslide hazards in California: organized by the Southern California Earthquake Center.

Christenson, G.E., and Shaw, L.M., 2008a, Surface Fault Rupture Special Study Areas, Wasatch Front and Nearby Areas, Utah: Utah Geological Survey Supplement Map to Utah Geological Survey Circular 106, 1 Plate, Scale 1:200,000.

Christenson, G.E., and Shaw, L.M., 2008b, Debris-Flow/Alluvial Fan Special Study Areas, Wasatch Front and Nearby Areas, Utah: Utah Geological Survey Supplement Map to Utah Geological Survey Circular 106, 1 Plate, Scale 1:200,000.

Christenson, G.E., and Shaw, L.M., 2008c, Liquefaction Special Study Areas, Wasatch Front and Nearby Areas, Utah: Utah Geological Survey Supplement Map to Utah Geological Survey Circular 106, 1 Plate, Scale 1:200,000.

Colton, R.B., 1991, Landslide Deposits in the Ogden 30’ x 60’ Quadrangle, Utah and Wyoming: U.S. Geological Survey Open-File Report 91-297, 1 Plate, 8 p., Scale 1:100,000.

Coogan, J.C., and King, J.K., 2001, Progress Report Geologic Map of the Ogden 30’ x 60’ Quadrangle, Utah and Wyoming – Year 3 of 3: Utah Geological Survey Open-File Report 380, 1 Plate, 33 p., Scale 1:100,000.

Coogan, J.C., and King, J.K., 2016, Interim Geologic Map of the Ogden 30’ x 60’ Quadrangle, Box Elder, Cache, Davis, Morgan, Rich, and Summit Counties, Utah, and Uinta County, Wyoming: Utah Geological Survey Open-File Report 653DM, 1 Plate, 151 p., Scale 1:62,500.

Elliott, A.H., and Harty, K.M., 2010, Landslide Maps of Utah, Ogden 30’ X 60’Quadrangle: Utah Geological Survey Map 246DM, Plate 6 of 46, Scale 1:100,000.

Federal Emergency Management Agency [FEMA], 1997, *NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures,* FEMA 302, Washington, D.C.

REFERENCES CITED (Cont.)

Frankel, A., Mueller, C., Barnard, T., Perkins, D., Leyendecker, E.V., Dickman, N., Hanson, S., and Hopper, M., 1996, *National Seismic-hazard Maps: Documentation*, U.S. Geological Survey Open-File Report 96-532, June.

Hintze, L.F., 1988, Geologic History of Utah: Brigham Young University Geology Studies Special Publication 7, Provo, Utah, 202 p.

Hynes, M.E. and A. G. Franklin (1984). “Rationalizing the Seismic Coefficient Method” Miscellaneous Paper GL-84-13, U.S. Army Waterways Experiment Station, Vicksburg, Miss.

International Building Code [IBC], 2012, International Code Council, Inc.

King, J.K., Yonkee, W.A., and Coogan, J.C., 2008, Interim Geologic Map of the Snow Basin Quadrangle and Part of the Huntsville Quadrangle, Davis, Morgan, and Weber Counties, Utah: Utah Geological Survey Open-File Report 536, 1 Plate, 31 p., Scale 1:24,000.

Lund, W.R., 1990, editor, Engineering geology of the Salt Lake City metropolitan area, Utah: Utah Geological Survey Bulletin 126, 66 p.

Milligan, M.R., 2000, How was Utah’s topography formed? Utah Geological Survey, Survey Notes, v. 32, no.1, pp. 10-11.

Solomon, B.J., 1996, Radon-Hazard Potential in Ogden Valley, Weber County, Utah: Utah Geological Survey Public Information Series 36, 2 p.

Sorensen, M.L., and Crittenden, Jr., M.D., 1979, Geologic Map of the Huntsville Quadrangle, Weber and Cache Counties, Utah: U.S. Geological Survey GQ-1503, 1 Plate, Scale 1:24,000.

Stokes, W.L., 1987, Geology of Utah: Utah Museum of Natural History and Utah Geological and Mineral Survey Department of Natural Resources, Salt Lake City, UT, Utah Museum of Natural History Occasional Paper 6, 280 p.

U.S. Geological Survey and Utah Geological Survey, 2006, Quaternary fault and fold database for the United States, accessed 8-1-16, from USGS website: http://earthquakes.usgs.gov/regional/qfaults

U.S. Geological Survey, 2012, U.S. *Seismic “Design Maps” Web Application*, site: https://geohazards.usgs.gov/secure/designmaps/us/application.php.

Utah Geological Survey, 1996, The Wasatch Fault: UGS Public Information Series 40, 17 p.

Weber County, 2015, Natural Hazards Overlay Districts, Chapter 27 of Title 104 of the Weber County Code of Ordinances, adopted on December 22, 2015.

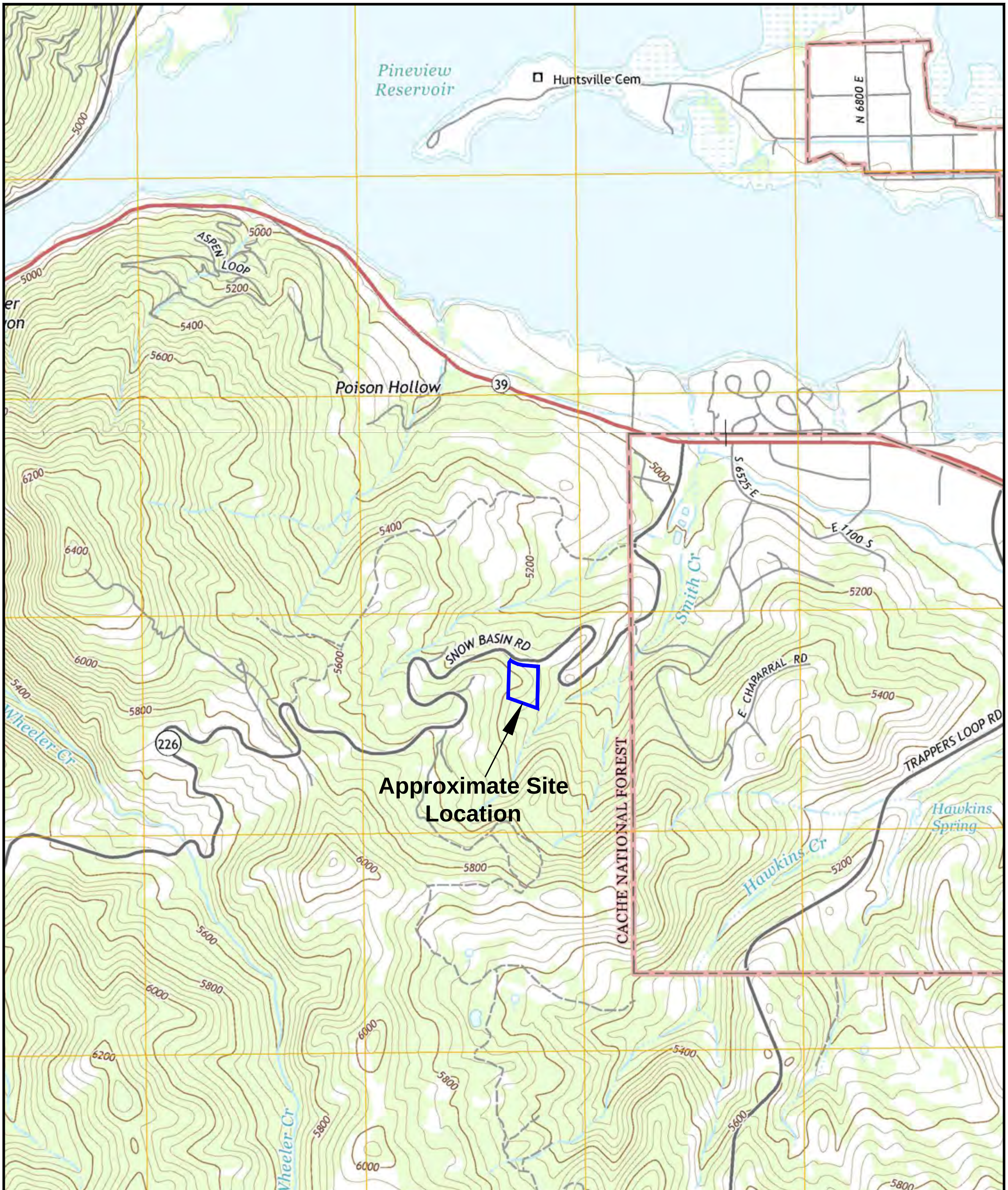
REFERENCES CITED (Cont.)

AERIAL PHOTOGRAPHS

Data Set	Date	Flight	Photographs	Scale
1947 AAJ	August 10, 1946	AAJ_1B	28, 29, 51, 52	1:20,000

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

APPENDIX A



BASE MAPS:

USGS Huntsville and Snow Basin 7.5-Minute Topographic Quadrangle Maps (2014)

N

0 1000 2000 3000

FEET

FIGURE A-1

SITE VICINITY MAP

THOMAS QUINN PROPERTY

GEOTECHNICAL AND GEOLOGIC HAZARDS INVESTIGATION

OLD SNOW BASIN ROAD

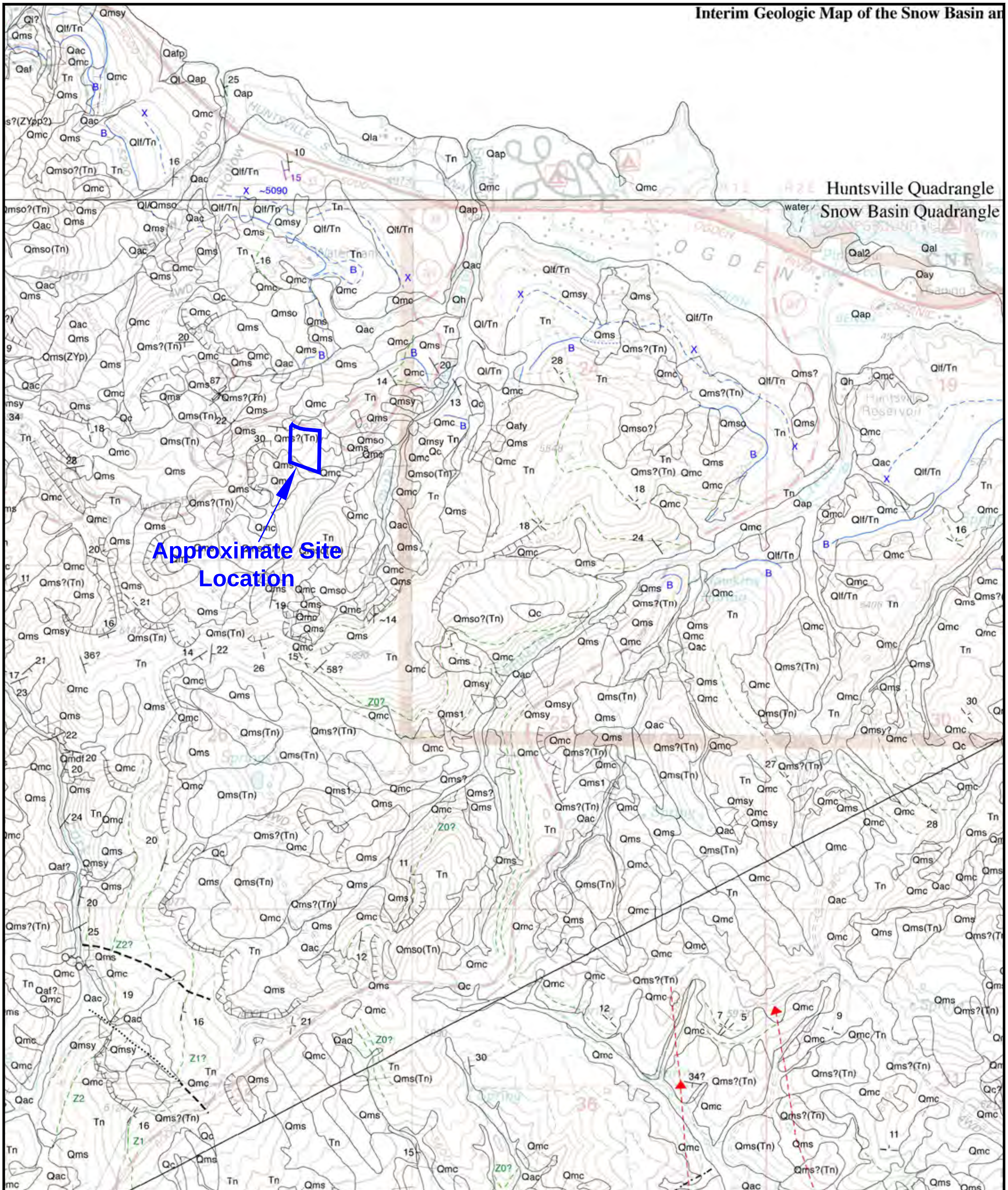
WEBER COUNTY, UTAH

DATE: 09/09/2016

PROJECT:02350-001

SCALE: 1"=2,000'

IGES



BASE MAP:

UGS 7.5-Minute Geologic Quadrangle Map
Interim Geologic Map of the Snow Basin and
Part of the Huntsville Quadrangle
OFR-536, King, Yonkee, and Coogan (2008)

QUADRANGLE
LOCATION

N

0 1000 2000 3000

FEET

FIGURE A-2a

REGIONAL GEOLOGY MAP

THOMAS QUINN PROPERTY

GEOTECHNICAL AND GEOLOGIC
HAZARDS INVESTIGATION

OLD SNOW BASIN ROAD

WEBER COUNTY, UTAH

DATE: 09/09/2016	SCALE:
PROJECT:02350-001	1"=2,000'

IGES

MAP LEGEND

Qms, Qms1, Qmsy, Qmso

Landslide and slump deposits (Holocene and Pleistocene) - Poorly sorted clay- to boulder-sized material; locally includes flow deposits; 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 deposits; Qms may be in contact with Qms when two different slide/slumps abut; locally, unit involved in slide/slump is shown in parentheses where a nearly intact block is visible; Qms and Qmso queried (?) where bedrock block may be in place; thickness highly variable, boreholes in Rogers (1986) show thicknesses of about 20 to 30 feet (6-9 m) on small slides/flows.

Qms without suffix is mapped where age uncertain (though likely Holocene and/or upper Pleistocene), where portions of slide/slump complexes have different ages but cannot be shown separately at map scale, or where boundaries between slides/slumps of different ages are not distinct. Estimated time of emplacement indicated by relative-age number and letter suffixes with: 1 - likely emplaced in the last 80 to 150 years, mostly historical; y - post- Lake Bonneville in age and mostly pre-historic; and o - likely emplaced before Lake Bonneville transgression. Suffixes y (as well as 1) and o indicate probable Holocene and Pleistocene ages, respectively. Qmso typically mapped where rumpled morphology typical of mass movements has been diminished and/or younger surficial deposits cover or cut Qmso. These older deposits are as unstable as other landslides and slumps, and are easily reactivated with the addition of water, be it irrigation or septic tank drain fields.

Qmc Landslide and slump, and colluvial deposits, undivided (Holocene and Pleistocene) - Mapped where landslides and slumps are difficult to distinguish from colluvium (slopewash and soil creep) and where mapping separate, small, intermingled areas of slides and slumps, and colluvial deposits is not possible at map scale; locally includes talus and debris flows; typically mapped where landslides and slumps are thin ("shallow"); also mapped where the blocky or rumpled morphology that is characteristic of landslides and slumps has been diminished ("smoothed") by slopewash and soil creep; composition depends on local sources; 0 to 40 feet (0-12 m) thick. These deposits are as unstable as other landslides and slumps units (Qms_).

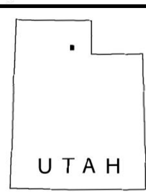
Qc Colluvium (Holocene and Pleistocene) - Includes materials moved by slopewash and soil creep; composition depends on local sources; generally 6 to 20 feet (2-6 m) thick; not mapped where less than 6 feet (2 m) thick.

Mixed Deposits

Qac Alluvium and colluvium (Holocene and Pleistocene) - Includes stream and fan alluvi colluvium, and, locally, mass-movement deposits; 0 to 20 feet (0-6 m) thick.

BASE MAP:

UGS 7.5-Minute Geologic Quadrangle Map
Interim Geologic Map of the Snow Basin and Part of the Huntsville Quadrangle
OFR-536, King, Yonkee, and Coogan (2008)



QUADRANGLE LOCATION

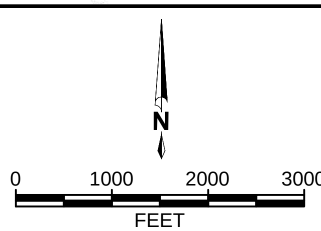


FIGURE A-2b

REGIONAL GEOLOGY MAP

THOMAS QUINN PROPERTY

GEOTECHNICAL AND GEOLOGIC HAZARDS ASSESSMENT

WEBER COUNTY, UTAH

DATE: 09/09/2016 | SCALE: 1"=2,000'

PROJECT:02350-001

IGES

MAP LEGEND

Tn Norwood Formation (lower Oligocene and upper Eocene) - Typically light-gray to light-brown, altered tuff (claystone), tuffaceous siltstone, sandstone, and conglomerate; locally colored light shades of red and green; variable calcareous cement and zeolitization, that is less common to south of Snow Basin quadrangle; zeolite marker beds mapped as an aid to recognizing geologic structure; locally includes landslides and slumps that are too small to show at map scale.

Mass-movement scarp

Moraine crest or ice-carved bedrock ridge, lowest number, m1, is youngest; queried where correlation uncertain; m1 and m2 are likely Holocene and are conspicuous to west, upslope in Ogden 7.5' quadrangle; m3, and m4 are Pinedale-age features; m5 may be Pinedale- or Bull Lake-age features; BL is Bull Lake-age features. Possible age-equivalent end moraine in Cottonwood Canyon near Salt Lake City (Madsen and Currey, 1979) in parentheses.
m1 and m2 (upper and lower Devils Castle, respectively)
m3 (Hogum Fork, double crested)
m4 (Bells Canyon)
m5 (uncertain) Correlation problem because next moraine in Cottonwood Canyon is Dry Creek, but Dry Creek moraine has well developed soil and is pre-Wisconsin (Bull Lake)

Strike and dip of bedding (red from Pavlis, 1979; purple from Sorensen and Crittenden, 1979)
43 Upright
78 Overturned
Vertical
Horizontal
50 150+ Determined photogrammetrically, upright on left, overturned on right
Approximate (queried where dip uncertain)

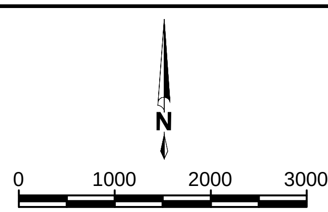
Ql/Tn Thin Quaternary deposits underlain by other units (bedrock in this case)

Qms(Tn) Landslide with nearly intact rotated blocks of unit in parentheses; for example Qms(Tcg), Qms(Tn), Qms(Tw), Qms(Xfc); queried (Qms?, Qmso?) where blocks may be in place.

BASE MAP:

UGS 7.5-Minute Geologic Quadrangle Map
Interim Geologic Map of the Snow Basin and Part of the Huntsville Quadrangle
OFR-536, King, Yonkee, and Coogan (2008)

QUADRANGLE LOCATION

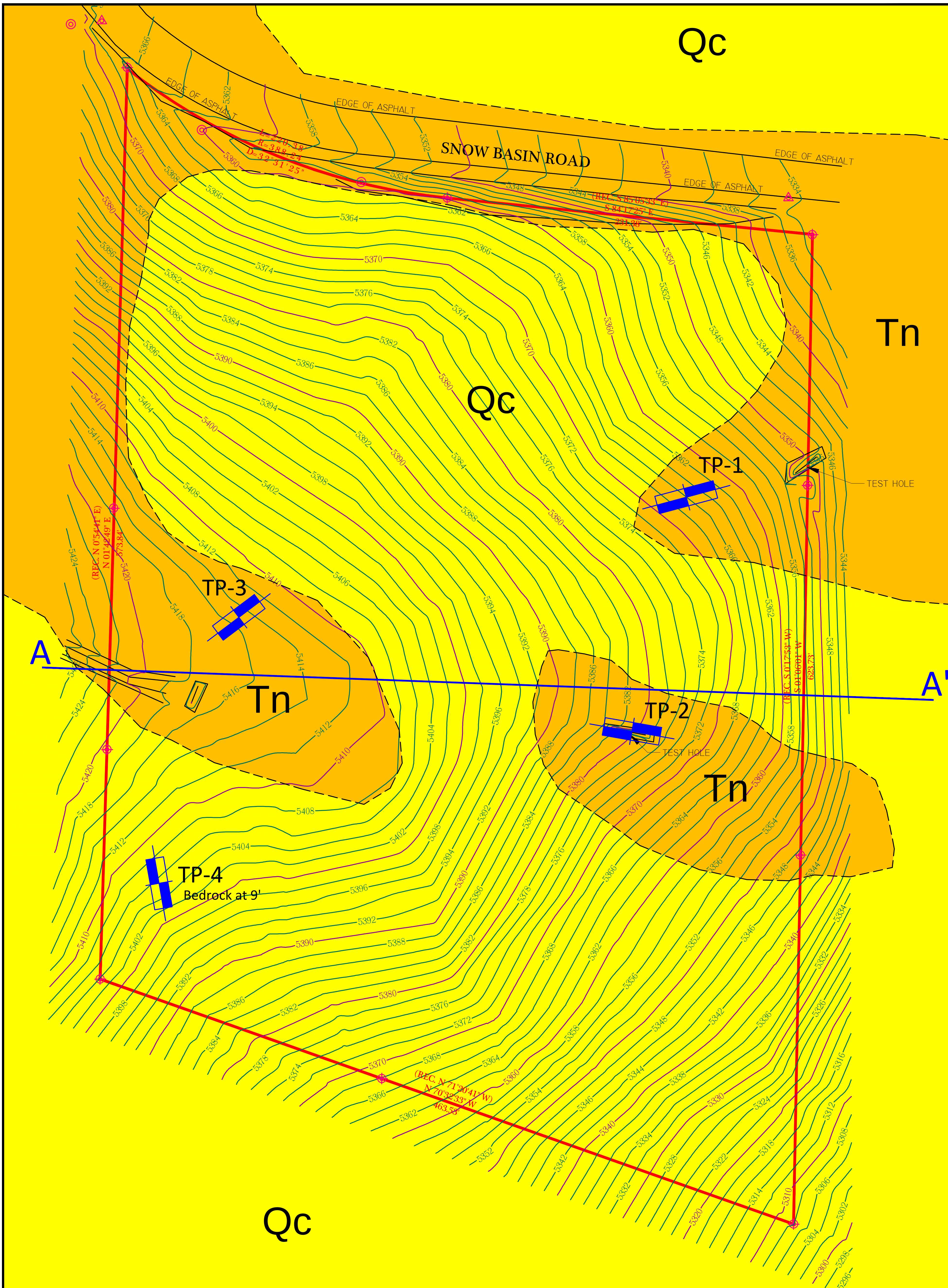


FEET

FIGURE A-2c

REGIONAL GEOLOGY MAP
THOMAS QUINN PROPERTY
GEOTECHNICAL AND GEOLOGIC HAZARDS ASSESSMENT
WEBER COUNTY, UTAH

DATE: 09/09/2016
PROJECT:02350-001
SCALE: 1"=2,000'
IGES



BASEMAP FROM JOHANSON ENGINEERING

LEGEND

Qc COLLUVIUM

Tn NORWOOD TUFF

TP-3 TEST PIT LOCATION

THOMAS QUINN LIVING TRUST PROPERTY BOUNDARY

HISTORIC TEST PIT LOCATION

TEST HOLE

ALL CONTACTS APPROXIMATE

A A'

CROSS-SECTION A-A'

N

0 120

FEET

FIGURE A-3

LOCAL GEOLOGY MAP

THOMAS QUINN PROPERTY

GEOTECHNICAL AND GEOLOGIC HAZARDS INVESTIGATION

OLD SNOW BASIN ROAD

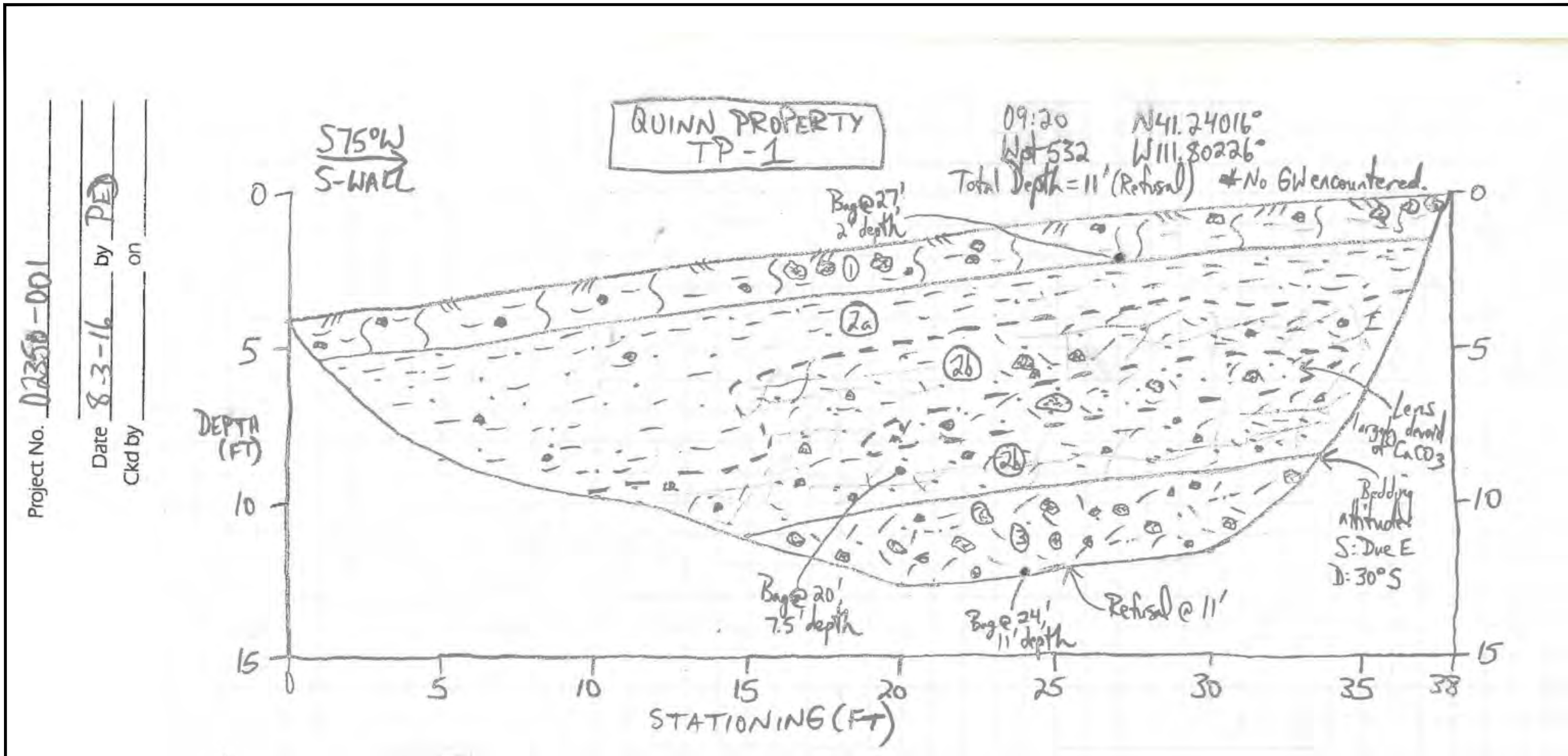
WEBER COUNTY, UTAH

DATE: 09/12/2016

FILE: 02350-001

SCALE: 1"=60'

IGES



LITHOLOGIC UNIT DESCRIPTIONS:

1.A/B Soil Horizon: ~1.5' thick; brownish black (5YR 2/1) fat CLAY with gravel (CH); very stiff to stiff, dry, medium to high plasticity, massive; gravel and larger sized clasts comprise ~5-10% of unit, all moderately weathered/oxidized dark yellowish brown (10YR 4/2) quartzite up to 5" in diameter; clasts normally found in uppermost ~1' and on surface; possible surficial colluvial unit; abundant plant and tree roots; sharp, planar basal contact.

2. Norwood Tuff Block-and-Ash: ~7.5' thick; moderate yellowish brown (10YR 5/4) to dark yellowish brown (10YR 4/2), but highly mottled due to abundant white calcium carbonate stringers and subhorizontal and subvertical fracture infilling; two distinct subunits:

2a: ~2-5' thick, dark yellowish brown to moderate yellowish brown sandy lean CLAY with gravel (CL) gradational to clayey SAND with gravel (SC); stiff, slightly moist, low plasticity, massive, though blocky texture in places; uppermost~1.5' is clayey and possibly a B-horizon; gravel and larger sized clasts comprise <5% of subunit, all quartzite up to 2"; common plant and tree roots; becomes sandier with depth.

2b: ~2-5' thick; mottled white (N9) and moderate yellowish brown clayey SAND with gravel (SC); dense, slightly moist, low plasticity, massive, though subhorizontal $CaCO_3$ fracture infilling consistent with basal contact; abundant calcite stringers and fracture infilling; gravel and larger sized clasts comprise ~10-15% of subunit; clasts are ~80% purple to dark yellowish orange (10YR 6/6) quartzite and ~20% white Norwood tuff; clasts are angular to subrounded, up to 5" in diameter, though mode average <1"; contains lens of material that is largely devoid of calcite fractures / stringers; occasional plant and tree roots; sharp, planar basal contact.

3. Norwood Tuff Bedrock: >3' thick; very light gray (N8) to white (N9); Norwood Tuff bedrock moderately to highly weathered to sandy SILT with gravel (ML), very stiff to hard, slightly moist, massive, though clasts show faint fine bedding; gravel and larger sized clasts are entirely moderately weathered to largely unaltered and competent tuff; clasts comprise ~50-60% of unit, all angular tuff up to 4" in diameter; light gray (N7) calcite flour (caliche) throughout; competent clasts are fine-grained to very fine-grained.

FIGURE A-4
TP-1 LOG

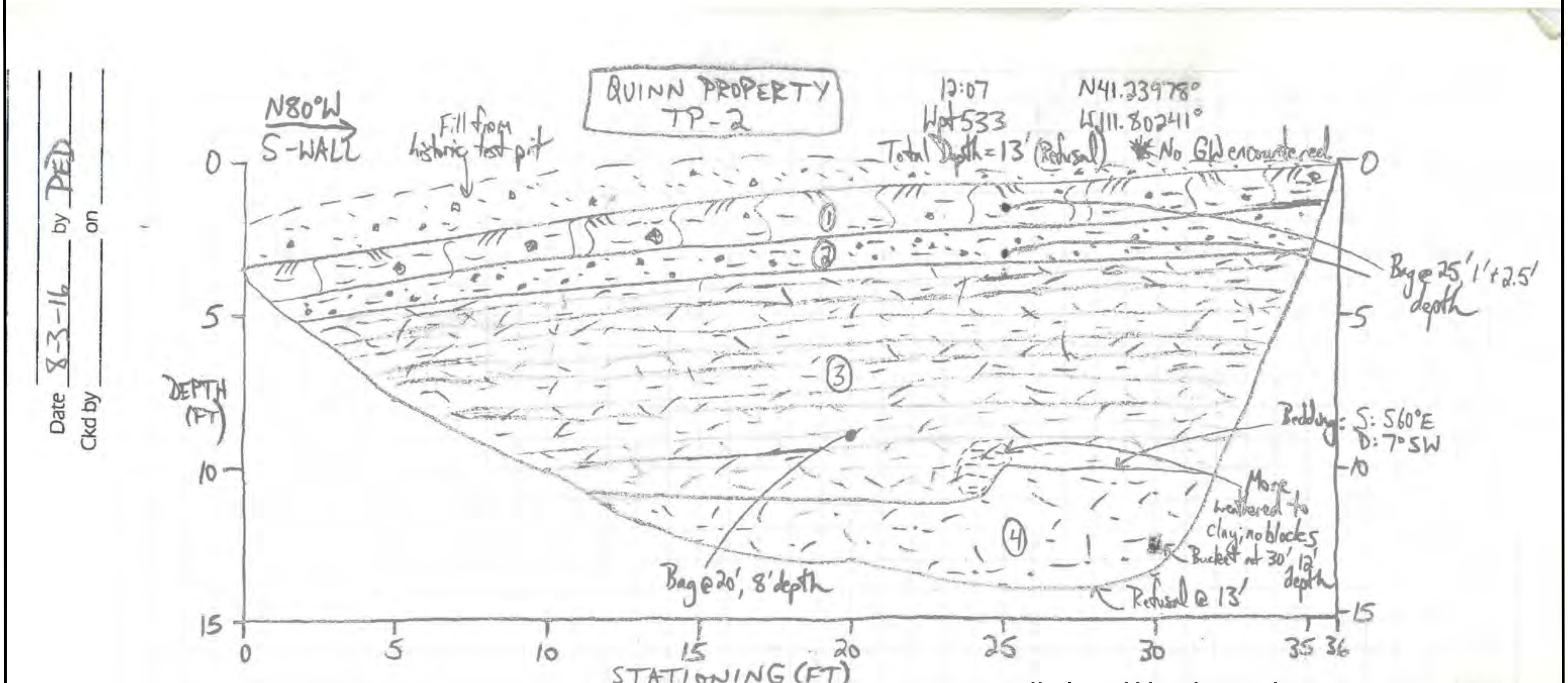
THOMAS QUINN PROPERTY

GEOTECHNICAL AND GEOLOGIC HAZARDS INVESTIGATION

OLD SNOW BASIN ROAD

WEBER COUNTY, UTAH

DATE: 08/18/2016	SCALE: 1"=5'	IGES
PROJECT: 02350-001		



***Reopened/enlarged historic test pit.**

LITHOLOGIC UNIT DESCRIPTIONS:

1. A/B Soil Horizon: ~1.5-2' thick; black (N1) to brownish black (5YR 2/1) lean CLAY (CL); medium stiff to loose, dry, moderate plasticity, massive; silty; gravel and larger sized clasts comprise <5% of unit and include predominantly Norwood Tuff clasts, though rare subrounded quartzite; clasts up to 7" in diameter, though mode size <1/2"; highly organic-rich; abundant plant and tree roots; sharp, irregular basal contact.

2. Highly Weathered Norwood Tuff: ~1-1.5' thick; mottled moderate yellowish brown (10YR 5/4) and dark yellowish orange (10YR 6/6) lean CLAY (CL); medium stiff, dry, moderate plasticity, massive; silty; mottling is due to abundant small (<1") angular Norwood Tuff clasts; largely weathered and oxidized; clasts have been largely weathered to sand and silt; abundant plant and tree roots; sharp, planar basal contact.

3. Partially Weathered Norwood Tuff: ~7' thick; light gray (N7) to light brown (5YR 6/4) to dark yellowish orange (10YR 6/6); interbedded blocky, partially weathered tuff largely disaggregated into sandy lean CLAY (CL); medium to thickly bedded (~2-5"), though fine laminae seen in some beds; dense to very dense, dry; abundant white calcite flour throughout; common calcite flour infilling of bedding plane fractures; oxidized vertical joints in blocks common; common plant and tree roots; gradational, largely planar basal contact; some thin (<1") white bands weathering to what appears to be bentonite.

4. Norwood Tuff Bedrock: >4' thick; light gray (N7) to medium light gray (N6) weakly to moderately weathered tuff bedrock; blocky appearance; rock is medium hard (harder than in units above, though easily broken with hammer and some scratched / broken with hand); thickly bedded (~5"+); becomes harder with depth; weathering to a silt; largely very fine to fine-grained; oxidized vertical jointing common.

FIGURE A-5
TP-2 LOG

THOMAS QUINN PROPERTY

GEOTECHNICAL AND GEOLOGIC HAZARDS INVESTIGATION

OLD SNOW BASIN ROAD

WEBER COUNTY, UTAH

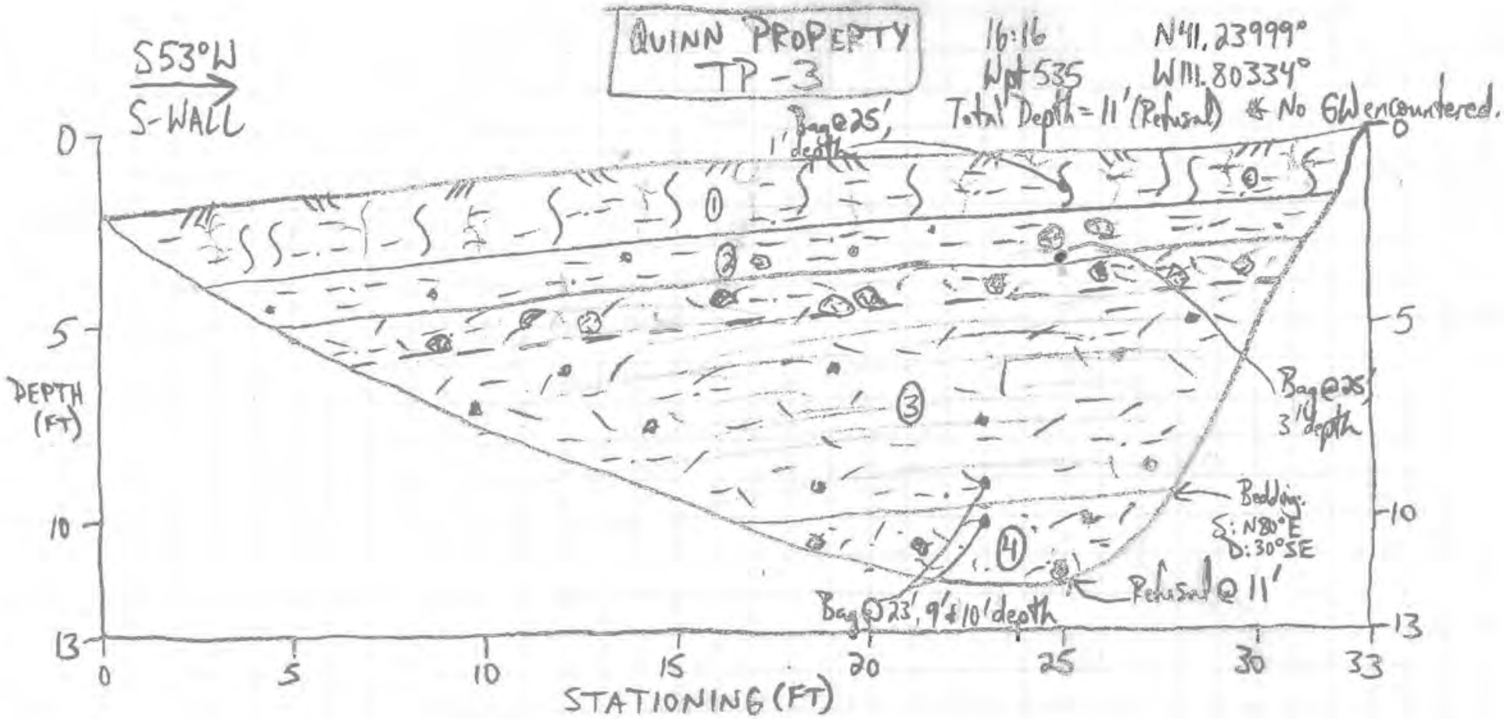
DATE: 08/18/2016	SCALE: 1"=5'
PROJECT: 02350-001	

IGES

Project No. 02350-001

Date 8-3-16 by PEJ

Ckd by ____ on ____



LITHOLOGIC UNIT DESCRIPTIONS:

1. A/B Soil Horizon: ~1.5-2' thick; brownish black (5YR 2/1) sandy fat CLAY (CH); very stiff to stiff, dry, moderate plasticity, blocky from desiccation cracks; rare (<1%) quartzite clasts up to 3" in diameter; not as fat or desiccated as seen in TP-4; abundant plant and tree roots; sharp, irregular basal contact.

2. Highly Weathered Norwood Tuff: ~1.5-2' thick; mottled light gray (N7) and dark yellowish brown (10YR 4/2) sandy lean CLAY with gravel (CL); very stiff to stiff, dry, moderate plasticity, blocky in places; gravel and larger sized clasts comprise ~5% of unit, and are predominantly medium-grained Norwood Tuff, though some purple quartzite; clasts up to 6" in diameter; tuff is highly weathered and crumbly; occasional to common plant and tree roots; sharp, planar basal contact; unit is possibly a B-horizon.

3. Norwood Tuff Block-and-Ash 1: ~6' thick; mottled light gray (N7) and dark yellowish brown (10YR 4/2); uppermost 1-1.5' is very light gray (N8), partially weathered Norwood Tuff that is fine-grained, silty, and dense; tuff clasts have irregular orientations, though are generally hard and competent; basal ~4.5-5' of unit is block-and-ash; disaggregates to sandy lean CLAY with gravel (CL); very stiff to stiff, slightly moist to dry, low plasticity, thinly bedded (<0.5"); gravel and larger sized clasts comprise ~10-15% of unit; clasts are ~75% angular Norwood Tuff and ~25% subrounded purple quartzite; clasts are up to 2" in diameter, though largely <1"; common white calcite flour along bedding plane fractures; ashy matrix; upper half of unit has occasional plant and tree roots, though rare below; gradational to clayey sand; sharp, planar basal contact.

4. Norwood Tuff Block-and-Ash 2: >2' thick; light brown (5YR 6/4) to moderate yellowish brown (10YR 5/4) sandy lean CLAY with gravel (CL) gradational to clayey SAND with gravel (SC); stiff, slightly moist to dry, low plasticity, massive; gravel and larger sized clasts comprise ~10% of unit and include ~50% tuff and quartzite up to 2" in diameter; ashy matrix; oxidized bands at top,~2" thick; becomes very hard at base.

FIGURE A-6
TP-3 LOG

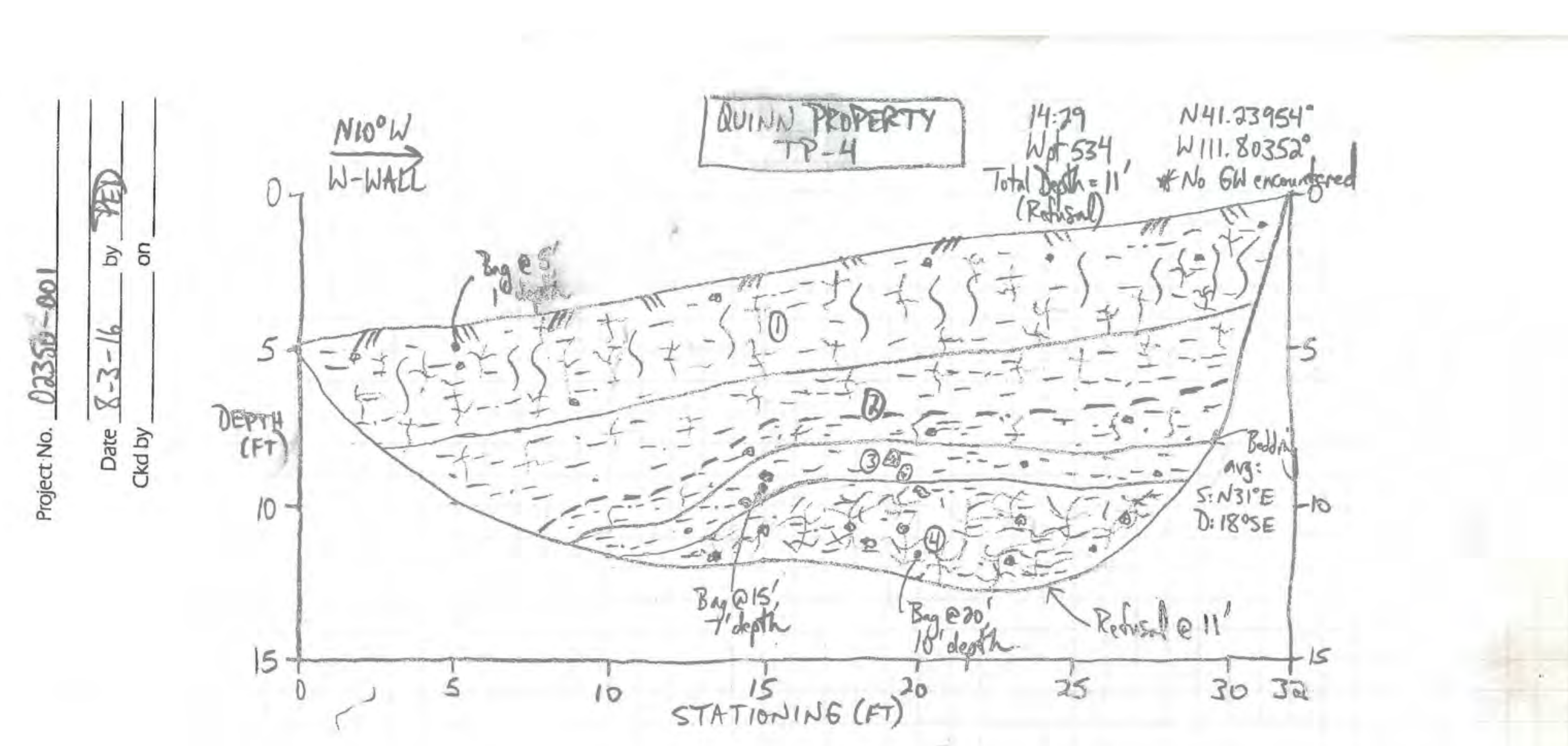
THOMAS QUINN PROPERTY

GEOTECHNICAL AND GEOLOGIC HAZARDS INVESTIGATION

OLD SNOW BASIN ROAD

WEBER COUNTY, UTAH

DATE: 08/18/2016	SCALE:	IGES
PROJECT: 02350-001	1"=5'	



LITHOLOGIC UNIT DESCRIPTIONS:

1. A/B Soil Horizon: ~3.5' thick; black (N1) to brownish black (5YR 2/1) sandy fat CLAY with gravel (CH); very stiff to stiff, dry, medium to high plasticity, blocky; gravel and larger-sized clasts comprise ~5% of the unit; clasts are entirely medium gray (N5) subrounded to rounded quartzite up to 3" in diameter, though some up to 10" diameter seen on surface; blocky structure a product of abundant desiccation cracks, which extend to base of unit; organic-rich, common plant and tree roots.

2. Slopewash?: ~2-4' thick; grayish brown (5YR 3/2) to moderate yellowish brown (10YR 5/4) to mottled with white (N9) and light gray (N7); uppermost ~2-2.5' is sandy lean CLAY (CL), stiff to very stiff, dry to slightly moist, moderate plasticity, blocky due to desiccation cracks, and may be a B-horizon; sand and silt component increases with depth; common plant and tree roots; basal ~1-1.5' is lean CLAY with gravel (CL), very stiff to stiff, dry, low plasticity, massive; silty; mottled appearance due to abundant small calcite stringers and matrix flour throughout; clasts comprise ~5% of unit, with approximately equal proportions of quartzite and tuff; abundant plant and tree roots; sharp, wavy basal contact; occasional pinhole voids (1 mm); ashy matrix, though decomposed to clay.

3. Paleosol?: ~1.5' thick; dark reddish brown (10R 3/4) lean CLAY with gravel (CL); stiff, slightly moist, low plasticity, massive; silty; gravel and larger sized clasts comprise ~5% of unit and are predominantly quartzite up to 6" in diameter, though some tuff; common plant and tree roots, ashy matrix, though decomposed to clay; unit is wavy, though consistent with modern slope; possible paleosol; no evidence of shear.

4. Weathered Norwood Tuff: >4' thick; mottled white (N9) and dark yellowish brown (10YR 4/2) due to abundant subvertical and subhorizontal white calcite fracture infilling; largely volcanic ash (possibly block-and-ash, though few blocks); disaggregates to lean CLAY with sand (CL) grading to sandy lean CLAY (CL); stiff, slightly moist, low plasticity; occasional plant and tree roots; some fine-grained, competent tuff blocks observed at base of trench.

FIGURE A-7
TP-4 LOG

THOMAS QUINN PROPERTY

GEOTECHNICAL AND GEOLOGIC HAZARDS INVESTIGATION

OLD SNOW BASIN ROAD

WEBER COUNTY, UTAH

DATE: 08/18/2016	SCALE:	IGES
PROJECT: 02350-001	1"=5'	

UNIFIED SOIL CLASSIFICATION SYSTEM

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

LOG KEY SYMBOLS

BORING SAMPLE LOCATION

TEST-PIT SAMPLE LOCATION

WATER LEVEL (level after completion)

WATER LEVEL (level where first encountered)

CEMENTATION

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

OTHER TESTS KEY

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

MODIFIERS

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

MOISTURE CONTENT

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

STRATIFICATION

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

GENERAL NOTES

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

APPARENT / RELATIVE DENSITY - COARSE-GRAINED SOIL

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

CONSISTENCY - FINE-GRAINED SOIL

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

KEY TO SOIL SYMBOLS AND TERMINOLOGY

Project No. **02350-001**

Engr. **DAG**

Drafted By **DAG**

Date **September 2016**

IGES

Intermountain Geo-Environmental Services, Inc.

Figure A-8

Rock Classification Should Include:

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

Weathering

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

Fracturing

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

Bedding of Sedimentary Rocks

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

RQD

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

Competency

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

KEY TO PHYSICAL ROCK PROPERTIES

Project No. 02350-001
Engr. DAG
Drafted By DAG
Date September 2016

IGES
Intermountain Geo-Environmental Services, Inc.

Figure A-9

APPENDIX B

Water Content and Unit Weight of Soil

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

IGES

Project: **Quinn Property**
No: **02350-001**
Location: Eden, UT
Date: 8/17/2016
By: BRR/IM

Sample Info.	Boring No.	TP-1	TP-1	TP-2	TP-3	TP-3	TP-4	TP-4	
	Sample								
	Depth	2.0'	7.5'	2.5'	3.0'	10.0'	1.0'	10.0'	
	Split	No	Yes	No	No	No	No	Yes	
	Split sieve		3/8"					3/4"	
	Total sample (g)		2732.93					3027.01	
	Moist coarse fraction (g)		41.50					330.95	
	Moist split fraction (g)		2691.43					2696.06	
	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)		165.94					454.51	
	Dry soil + tare (g)		165.66					450.81	
	Tare (g)		124.43					123.56	
	Water content (%)		0.7					1.1	
Split Fraction	Wet soil + tare (g)	681.63	812.26	730.54	764.17	1249.23	1021.29	1637.08	
	Dry soil + tare (g)	625.94	747.68	661.08	709.33	1047.24	919.47	1400.04	
	Tare (g)	273.26	393.04	288.42	294.24	214.14	221.74	326.65	
	Water content (%)	15.8	18.2	18.6	13.2	24.2	14.6	22.1	
	Water Content, w (%)	**15.8**	**17.9**	**18.6**	**13.2**	**24.2**	**14.6**	**19.4**	
	Dry Unit Wt., γ_d (pcf)								

Entered by:___________

Reviewed:___________

Liquid Limit, Plastic Limit, and Plasticity Index of Soils

(ASTM D4318)

IGES

Project: Quinn Property
No: 02350-001
Location: Eden, UT
Date: 8/18/2016
By: BRR

Boring No.: TP-1
Sample:
Depth: 2.0'
Description: Brown fat clay

Preparation method: Wet
Liquid limit test method: Multipoint

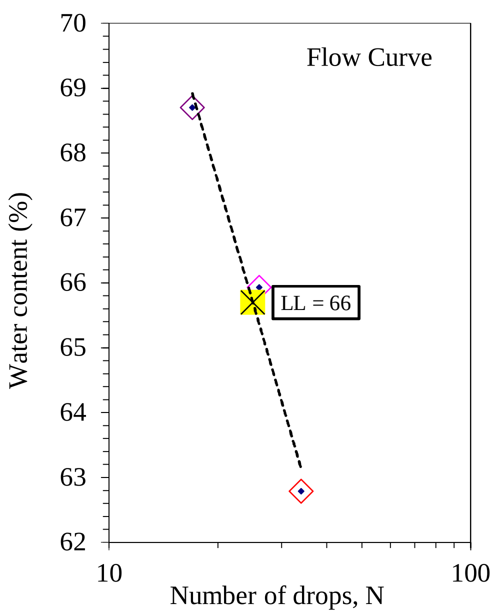
Plastic Limit

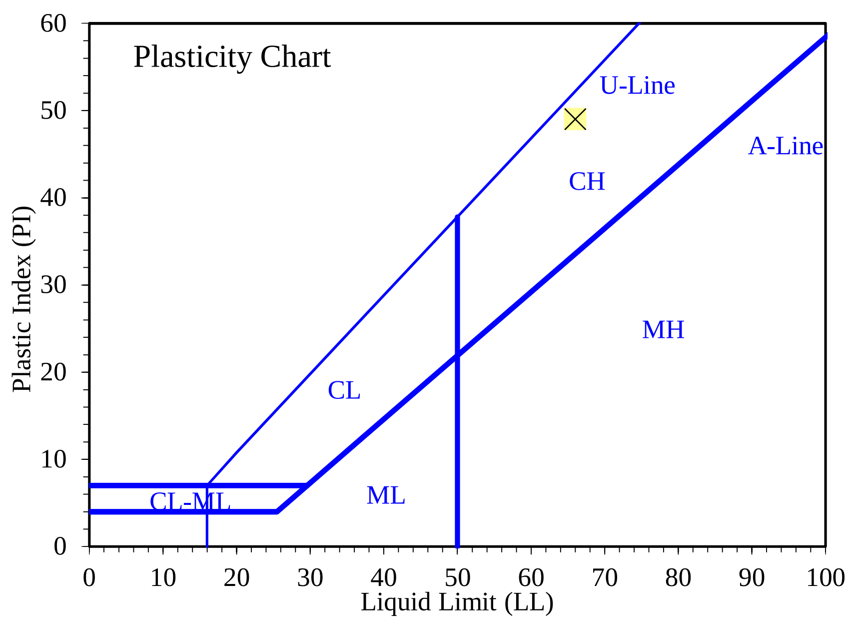
Determination No	1	2				
Wet Soil + Tare (g)	28.18	29.01				
Dry Soil + Tare (g)	27.24	27.94				
Water Loss (g)	0.94	1.07				
Tare (g)	21.74	21.81				
Dry Soil (g)	5.50	6.13				
Water Content, w (%)	17.09	17.46				

Liquid Limit

Determination No	1	2	3			
Number of Drops, N	34	26	17			
Wet Soil + Tare (g)	29.79	27.71	28.72			
Dry Soil + Tare (g)	26.77	25.33	26.02			
Water Loss (g)	3.02	2.38	2.70			
Tare (g)	21.96	21.72	22.09			
Dry Soil (g)	4.81	3.61	3.93			
Water Content, w (%)	62.79	65.93	68.70			
One-Point LL (%)		66				

Liquid Limit, LL (%)	**66**
Plastic Limit, PL (%)	**17**
Plasticity Index, PI (%)	**49**





Entered by:____________
Reviewed:____________

Liquid Limit, Plastic Limit, and Plasticity Index of Soils

(ASTM D4318)

IGES

Project: Quinn Property
No: 02350-001
Location: Eden, UT
Date: 8/18/2016
By: BRR

Boring No.: TP-1
Sample:
Depth: 7.5'
Description: Light brown lean clay

Preparation method: Wet
Liquid limit test method: Multipoint

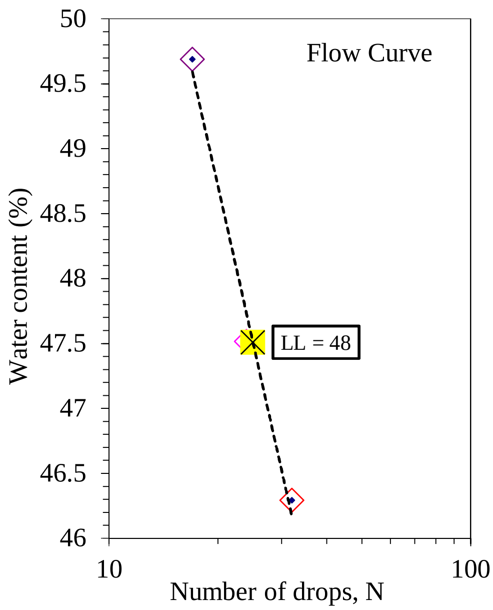
Plastic Limit

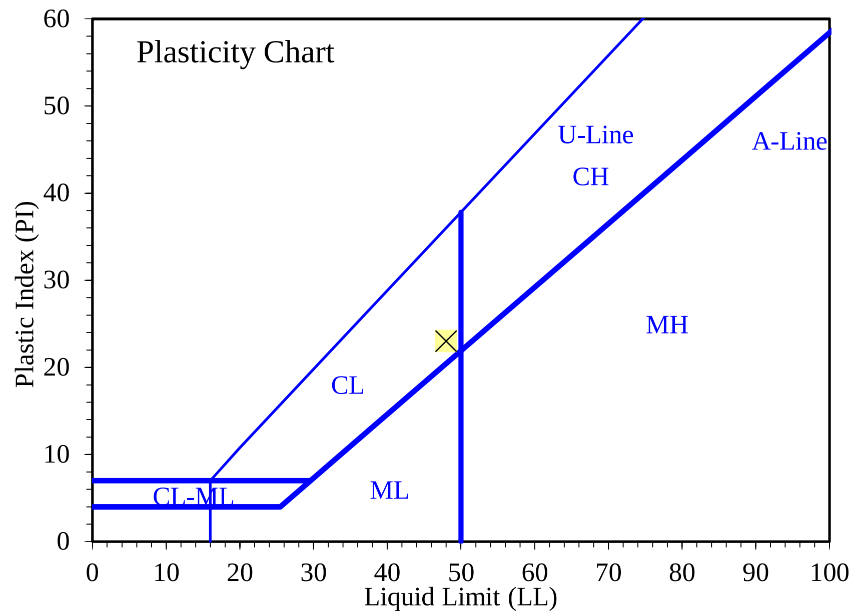
Determination No	1	2				
Wet Soil + Tare (g)	28.44	28.65				
Dry Soil + Tare (g)	27.18	27.26				
Water Loss (g)	1.26	1.39				
Tare (g)	22.13	21.63				
Dry Soil (g)	5.05	5.63				
Water Content, w (%)	24.95	24.69				

Liquid Limit

Determination No	1	2	3			
Number of Drops, N	32	24	17			
Wet Soil + Tare (g)	30.13	29.32	29.12			
Dry Soil + Tare (g)	27.57	26.93	26.72			
Water Loss (g)	2.56	2.39	2.40			
Tare (g)	22.04	21.90	21.89			
Dry Soil (g)	5.53	5.03	4.83			
Water Content, w (%)	46.29	47.51	49.69			
One-Point LL (%)		47				

Liquid Limit, LL (%)	**48**
Plastic Limit, PL (%)	**25**
Plasticity Index, PI (%)	**23**





Entered by:___________
Reviewed:___________

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

(ASTM D6913)

Project: Quinn Property
No: 02350-001
Location: Eden, UT
Date: 8/18/2016
By: IM

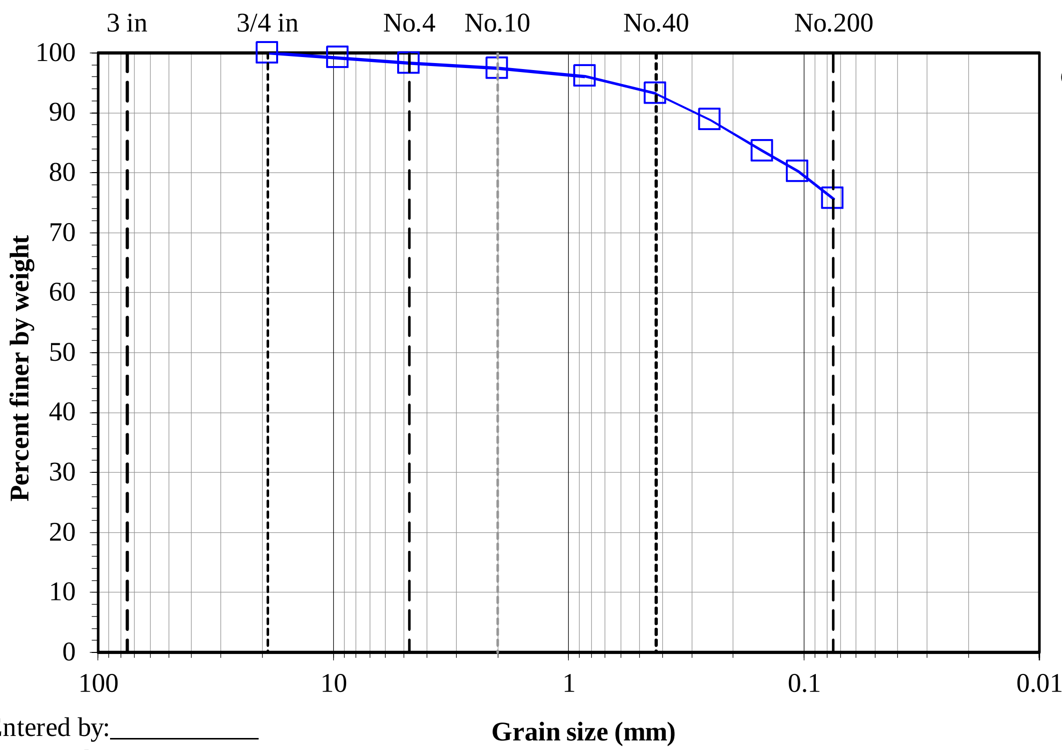
Boring No.: TP-4
Sample:
Depth: 7.0'
Description: Brown clay with sand

Split:	No	
	3/4"	
	Moist	Dry
Total sample wt. (g):	832.75	705.83
Split fraction:	1.000	

Water content data

Moist soil + tare (g):	-	1054.79
Dry soil + tare (g):	-	927.87
Tare (g):	-	222.04
Water content (%):	0.0	18.0

Sieve	Accum. Wt. Ret. (g)	Grain Size (mm)	Percent Finer
8"	-	200	-
6"	-	150	-
4"	-	100	-
3"	-	75	-
1.5"	-	37.5	-
3/4"	-	19	100.0
3/8"	5.81	9.5	99.2
No.4	12.42	4.75	98.2
No.10	18.47	2	97.4
No.20	27.90	0.85	96.0
No.40	48.10	0.425	93.2
No.60	79.03	0.25	88.8
No.100	115.58	0.15	83.6
No.140	139.55	0.106	80.2
No.200	171.89	0.075	75.6



Gravel (%): 1.8
Sand (%): 22.6
Fines (%): 75.6

Entered by:___________
Reviewed:___________

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

(ASTM D1140)

Project: **Quinn Property**
No: **02350-001**
Location: Eden, UT
Date: 8/18/2016
By: BSS/IM

Sample Info.	Boring No.	TP-2	TP-2	TP-3	TP-3	TP-3	TP-4		
	Sample								
	Depth	1.0'	2.5'	3.0'	9.0'	10.0'	10.0'		
	Split	No	No	No	Yes	No	Yes		
	Split Sieve*				3/8"		3/4"		
	Method	B	B	B	B	B	B		
	Specimen soak time (min)	180	250	150	210	240	230		
	Moist total sample wt. (g)	560.60	442.12	469.93	1192.46	1035.09	3027.01		
	Moist coarse fraction (g)				115.48		330.95		
	Moist split fraction + tare (g)				929.64		1637.08		
	Split fraction tare (g)				408.99		326.65		
	Dry split fraction (g)				439.15		1073.39		
	Dry retained No. 200 + tare (g)	317.36	349.18	490.29	597.31	766.69	627.28		
	Wash tare (g)	215.31	288.42	294.24	408.99	214.14	326.65		
	No. 200 Dry wt. retained (g)	102.05	60.76	196.05	188.32	552.55	300.63		
	Split sieve* Dry wt. retained (g)				113.96		327.25		
	Dry total sample wt. (g)	475.78	372.66	415.09	1022.35	833.10	2535.63		
Coarse Fraction	Moist soil + tare (g)				238.26		454.51		
	Dry soil + tare (g)				236.74		450.81		
	Tare (g)				122.78		123.56		
	Water content (%)				1.33		1.13		
Split Fraction	Moist soil + tare (g)	775.91	730.54	764.17	929.64	1249.23	1637.08		
	Dry soil + tare (g)	691.09	661.08	709.33	848.14	1047.24	1400.04		
	Tare (g)	215.31	288.42	294.24	408.99	214.14	326.65		
	Water content (%)	17.83	18.64	13.21	18.56	24.25	22.08		
	Percent passing split sieve* (%)				**88.9**		**87.1**		
	Percent passing No. 200 sieve (%)	**78.6**	**83.7**	**52.8**	**50.8**	**33.7**	**62.7**		

Entered by:___________
Reviewed:___________

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Direct Shear Test for Soils Under Drained Conditions

(ASTM D3080)

Project: Quinn Property
No: 02350-001
Location: Eden, UT
Date: 8/23/2016
By: JDF

Boring No.: TP-4
Sample:
Depth: 10.0'
Sample Description: Brown clay
Sample type: Arbitrary remold

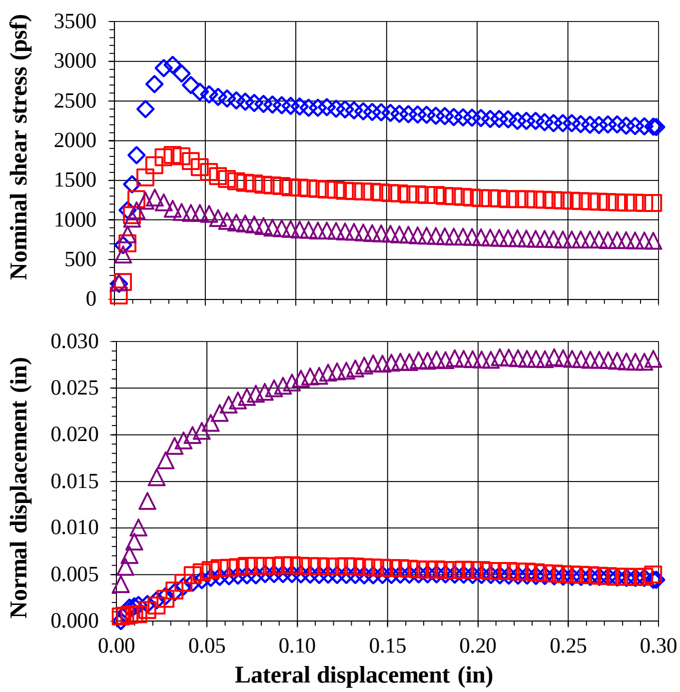
Test type: Inundated
Lateral displacement (in.): 0.3
Shear rate (in./min): 0.0003
Specific gravity, Gs: 2.70 Assumed

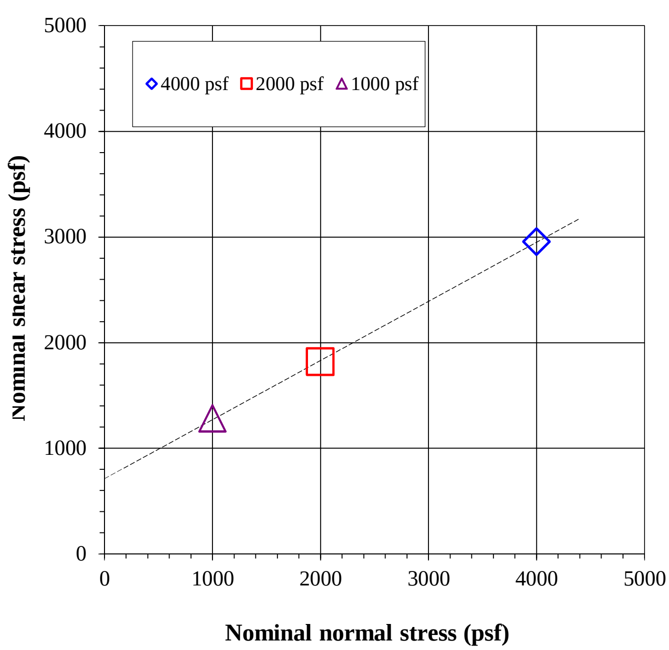
	Sample 1		Sample 2		Sample 3	
Nominal normal stress (psf)	4000		2000		1000	
Peak shear stress (psf)	2956		1818		1282	
Lateral displacement at peak (in)	0.032		0.032		0.022	
Load Duration (min)	82		93		109	
	Initial	Pre-shear	Initial	Pre-shear	Initial	Pre-shear
Sample height (in)	1.0000	1.0238	1.0000	1.0208	1.0000	1.0494
Sample diameter (in)	2.416	2.416	2.416	2.416	2.416	2.416
Wt. rings + wet soil (g)	189.62	196.56	189.26	195.98	186.16	195.02
Wt. rings (g)	45.81	45.81	45.45	45.45	42.35	42.35
Wet soil + tare (g)	347.30		347.30		347.30	
Dry soil + tare (g)	305.67		305.67		305.67	
Tare (g)	121.73		121.73		121.73	
Water content (%)	22.6	28.6	22.6	28.4	22.6	30.2
Dry unit weight (pcf)	97.4	95.1	97.4	95.4	97.4	92.8
Void ratio, e, for assumed Gs	0.73	0.77	0.73	0.77	0.73	0.82
Saturation (%)*	83.7	100.0	83.7	100.0	83.7	100.0

ϕ' (deg)	29
c' (psf)	713

Average of 3 samples	Initial	Pre-shear
Water content (%)	22.6	29.0
Dry unit weight (pcf)	97.4	94.5

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





Comments:
Specimens swelled upon inundation, and at the 125, 250, 500, 1000, 2000, and 4000 psf loadings.

Entered by:___________
Reviewed:___________

Direct Shear Test for Soils Under Drained Conditions

(ASTM D3080)

IGES

Project: Quinn Property
No: 02350-001
Location: Eden, UT

Boring No.: TP-4
Sample:
Depth: 10.0'

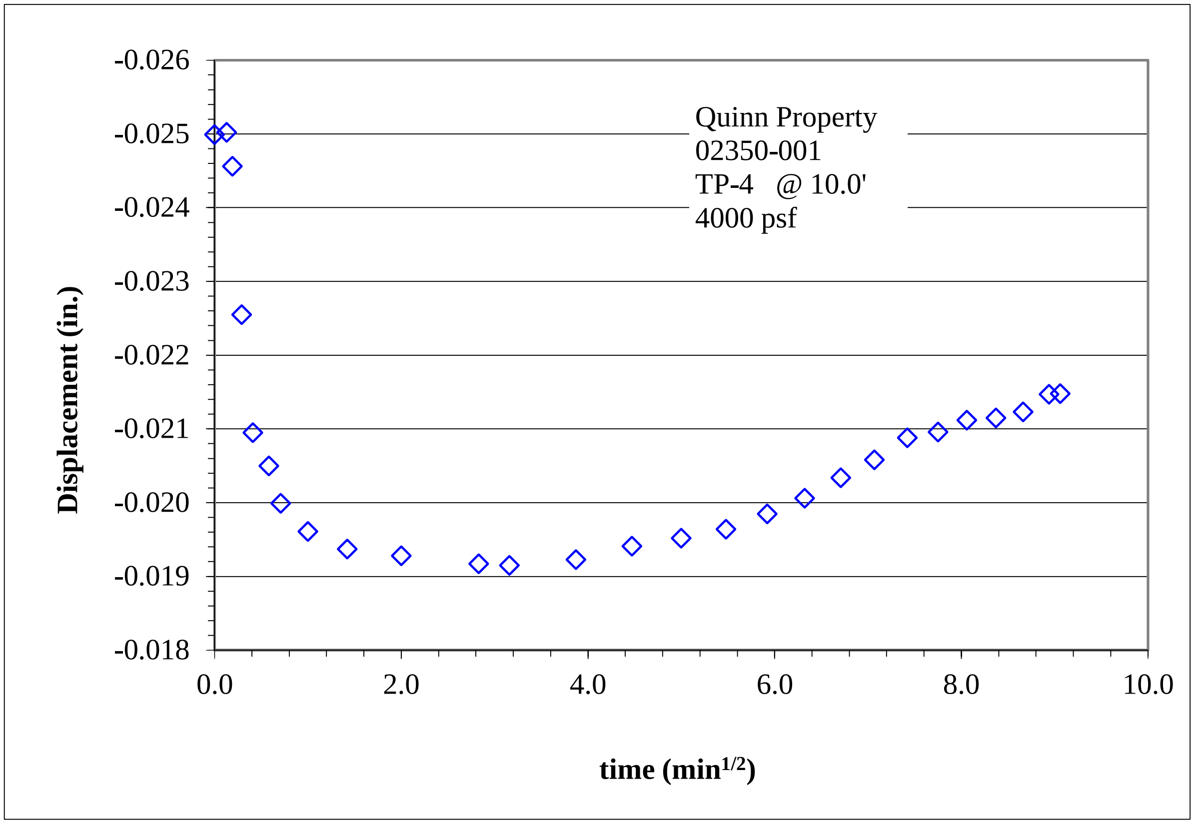
Nominal normal stress = 4000 psf			Nominal normal stress = 2000 psf			Nominal normal stress = 1000 psf		
Lateral Displacement (in.)	Nominal Shear Stress (psf)	Normal Displacement (in.)	Lateral Displacement (in.)	Nominal Shear Stress (psf)	Normal Displacement (in.)	Lateral Displacement (in.)	Nominal Shear Stress (psf)	Normal Displacement (in.)
0.002	196	0.000	0.002	46	0.000	0.002	210	0.004
0.005	681	0.001	0.005	216	0.001	0.005	560	0.006
0.007	1122	0.001	0.007	705	0.001	0.007	817	0.007
0.010	1450	0.002	0.010	1058	0.001	0.010	1016	0.009
0.012	1816	0.002	0.012	1261	0.001	0.012	1118	0.010
0.017	2399	0.002	0.017	1534	0.001	0.017	1241	0.013
0.022	2714	0.002	0.022	1689	0.002	0.022	1282	0.015
0.027	2913	0.003	0.027	1788	0.002	0.027	1220	0.017
0.032	2956	0.003	0.032	1818	0.003	0.032	1140	0.019
0.037	2848	0.004	0.037	1801	0.004	0.037	1100	0.019
0.042	2701	0.004	0.042	1741	0.005	0.042	1087	0.020
0.047	2618	0.004	0.047	1665	0.005	0.047	1085	0.020
0.052	2582	0.005	0.052	1605	0.005	0.052	1075	0.021
0.057	2551	0.005	0.057	1552	0.006	0.057	1024	0.022
0.062	2531	0.005	0.062	1513	0.006	0.062	986	0.023
0.067	2508	0.005	0.067	1485	0.006	0.067	965	0.024
0.072	2492	0.005	0.072	1468	0.006	0.072	954	0.024
0.077	2474	0.005	0.077	1456	0.006	0.077	941	0.024
0.082	2464	0.005	0.082	1441	0.006	0.082	920	0.025
0.087	2456	0.005	0.087	1432	0.006	0.087	902	0.025
0.092	2446	0.005	0.092	1424	0.006	0.092	892	0.025
0.097	2438	0.005	0.097	1411	0.006	0.097	886	0.026
0.102	2430	0.005	0.102	1406	0.006	0.102	876	0.026
0.107	2415	0.005	0.107	1396	0.006	0.107	871	0.026
0.112	2415	0.005	0.112	1391	0.006	0.112	866	0.026
0.117	2422	0.005	0.117	1382	0.006	0.117	863	0.027
0.122	2402	0.005	0.122	1376	0.006	0.122	859	0.027
0.127	2394	0.005	0.127	1366	0.006	0.127	852	0.027
0.132	2381	0.005	0.132	1362	0.006	0.132	849	0.027
0.137	2371	0.005	0.137	1357	0.006	0.137	841	0.027
0.142	2360	0.005	0.142	1351	0.006	0.142	834	0.028
0.147	2358	0.005	0.147	1345	0.006	0.147	830	0.028
0.152	2350	0.005	0.152	1336	0.006	0.152	823	0.028
0.157	2337	0.005	0.157	1331	0.006	0.157	817	0.028
0.162	2335	0.005	0.162	1322	0.006	0.162	811	0.028
0.167	2330	0.005	0.167	1320	0.006	0.167	804	0.028
0.172	2324	0.005	0.172	1313	0.006	0.172	801	0.028
0.177	2314	0.005	0.177	1311	0.006	0.177	797	0.028
0.182	2311	0.005	0.182	1301	0.005	0.182	793	0.028
0.187	2296	0.005	0.187	1297	0.005	0.187	789	0.028
0.192	2293	0.005	0.192	1288	0.006	0.192	788	0.028
0.197	2288	0.005	0.197	1281	0.005	0.197	783	0.028
0.202	2286	0.005	0.202	1273	0.005	0.202	779	0.028
0.207	2273	0.005	0.207	1272	0.005	0.207	777	0.028
0.212	2275	0.005	0.212	1269	0.005	0.212	772	0.028
0.217	2268	0.005	0.217	1262	0.005	0.217	767	0.028
0.222	2250	0.005	0.222	1261	0.005	0.222	767	0.028
0.227	2250	0.005	0.227	1261	0.005	0.227	763	0.028
0.232	2247	0.005	0.232	1258	0.005	0.232	760	0.028
0.237	2234	0.005	0.237	1251	0.005	0.237	758	0.028
0.242	2219	0.005	0.242	1247	0.005	0.242	757	0.028
0.247	2221	0.005	0.247	1244	0.005	0.247	752	0.028
0.252	2219	0.005	0.252	1242	0.005	0.252	752	0.028
0.257	2208	0.005	0.257	1237	0.005	0.257	752	0.028
0.262	2201	0.005	0.262	1233	0.005	0.262	751	0.028
0.267	2195	0.005	0.267	1228	0.005	0.267	750	0.028
0.272	2203	0.005	0.272	1222	0.005	0.272	745	0.028
0.277	2206	0.005	0.277	1221	0.005	0.277	745	0.028
0.282	2190	0.005	0.282	1216	0.005	0.282	744	0.028
0.287	2185	0.005	0.287	1215	0.005	0.287	741	0.028
0.292	2182	0.005	0.292	1212	0.005	0.292	738	0.028
0.297	2175	0.004	0.297	1210	0.005	0.297	735	0.028
0.299	2172	0.004	0.301	1206	0.005	0.301	731	0.028

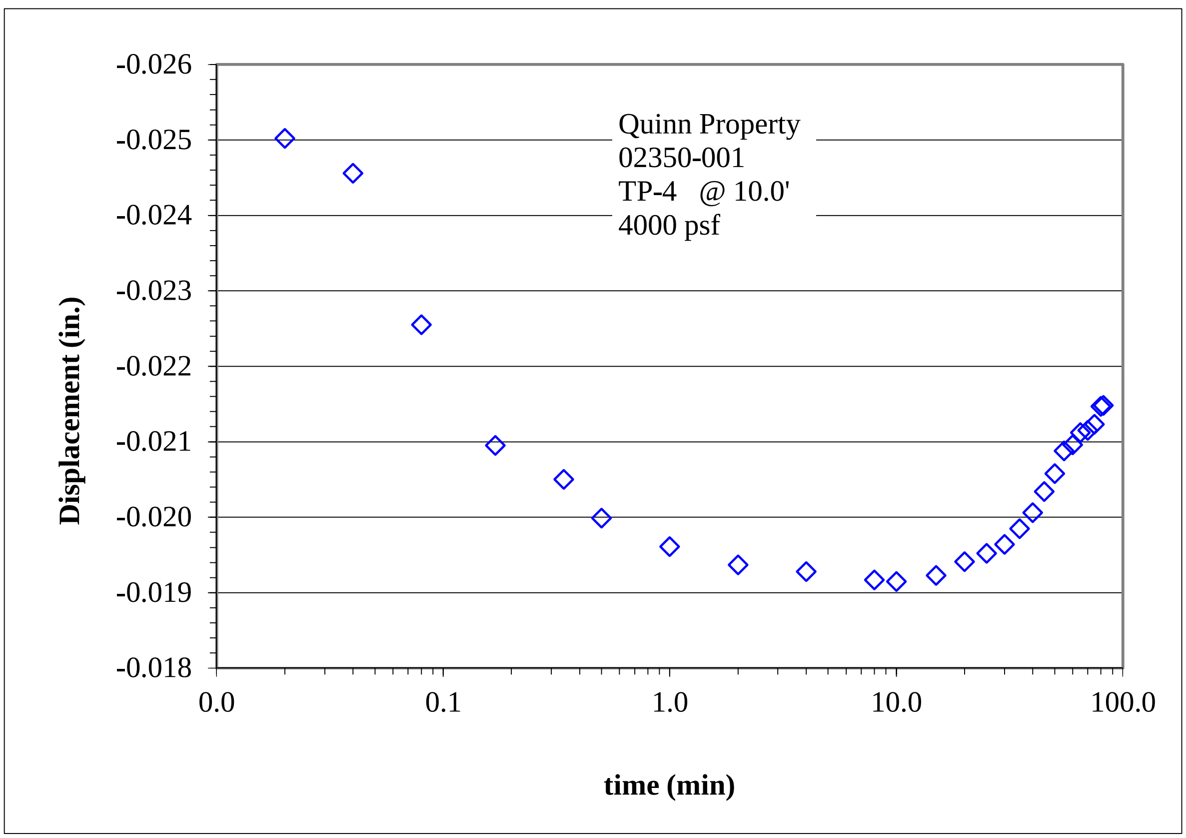
Direct Shear Test for Soils Under Drained Conditions

(ASTM D3080)

Project: Quinn Property
No: 02350-001
Location: Eden, UT

Boring No.: TP-4
Sample:
Depth: 10.0'





Determination of the Point Load Strength Index of Rock

(ASTM D5731)

Project: **Quinn Property**
No: **02350-001**
Location: Eden, UT
Date: 8/22/2016
By: JDF/ET
Test Device: Humboldt H-1342
Test Frame: GEOTAC Sigma-1 10K   Calibration Date: 8/1/2016

Boring No.	TP-1	TP-1	TP-1	TP-2	TP-2	
Sample:	1	2	3	1	2	
Depth:	11.0'	11.0'	11.0'	12.0'	12.0'	
Sample type	Block	Block	Block	Block	Block	
Core test type						
Distance between platen points, D (in.)	1.435	1.483	1.552	1.628	1.558	
D (mm)	36.449	37.668	39.421	41.351	39.573	
Smallest specimen width, W (in.)	2.566	3.259	3.080	1.864	2.297	
W (mm)	65.2	82.8	78.2	47.3	58.3	
Equivalent core area, D_e^2 (mm^2)	3024.7	3970.1	3926.6	2492.7	2939.7	
Failure load, P (lbf)	1666	4225	1276	269	145	
P (N)	7411	18794	5676	1197	645	
Point load strength index, I_s (MPa)	2.45	4.73	1.45	0.48	0.22	
Size correction factor, F	1.044	1.110	1.107	0.999	1.037	
PLSI 50mm equivalent, $I_{s(50)}$ (MPa)	2.56	5.25	1.60	0.48	0.23	
Site specific correlation, C	23.7	23.7	23.7	23.7	23.7	
Uniaxial compressive strength, δ_{uc} (MPa)	60.61	124.33	37.92	11.37	5.39	
Uniaxial compressive strength, δ_{uc} (psi)	**8791**	**18032**	**5500**	**1649**	**782**	

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)*

Project: **Quinn Property**
No: **02350-001**
Location: Eden, Utah
Date: 8/17/2016
By: BSS

Sample info.	Boring No.	TP-1							
	Sample								
	Depth	7.5'							
Water content data	Wet soil + tare (g)	69.74							
	Dry soil + tare (g)	64.76							
	Tare (g)	37.45							
	Water content (%)	18.2							
Chem. data	pH	7.89							
	Soluble chloride* (ppm)	642							
	Soluble sulfate** (ppm)	178							
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	4540	0.67	3042				
		+3	2751	0.67	1843				
		+6	1666	0.67	1116				
		+9	1181	0.67	791				
		+12	695	0.67	466				
		+15	577	0.67	387				
		+18	568	0.67	380				
		+21	578	0.67	387				
	Minimum resistivity (Ω-cm)	**380**							

* 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 (41.2403°N, 111.8027°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 = 0.857 g
From Figure 1613.3.1(2) [2]	S_1 = 0.289 g

Section 1613.3.2 — Site class definitions

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

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

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

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

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

TABLE 1613.3.3(1)
VALUES OF SITE COEFFICIENT F_a

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

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

For Site Class = C and S_S = 0.857 g, F_a = 1.057

TABLE 1613.3.3(2)
VALUES OF SITE COEFFICIENT F_v

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

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

For Site Class = C and S_1 = 0.289 g, F_v = 1.511

Equation (16-37):	$S_{MS} = F_aS_S = 1.057 \times 0.857 = 0.906$ g
Equation (16-38):	$S_{M1} = F_vS_1 = 1.511 \times 0.289 = 0.437$ g

Section 1613.3.4 — Design spectral response acceleration parameters

Equation (16-39):	$S_{DS} = ⅔\ S_{MS} = ⅔ \times 0.906 = 0.604$ g
Equation (16-40):	$S_{D1} = ⅔\ S_{M1} = ⅔ \times 0.437 = 0.291$ g

Section 1613.3.5 — Determination of seismic design category

TABLE 1613.3.5(1)
SEISMIC DESIGN CATEGORY BASED ON SHORT-PERIOD (0.2 second) RESPONSE ACCELERATION

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

For Risk Category = I and S_{DS} = 0.604 g, Seismic Design Category = D

TABLE 1613.3.5(2)
SEISMIC DESIGN CATEGORY BASED ON 1-SECOND PERIOD RESPONSE ACCELERATION

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

For Risk Category = I and S_{D1} = 0.291 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

Design Maps Summary Report

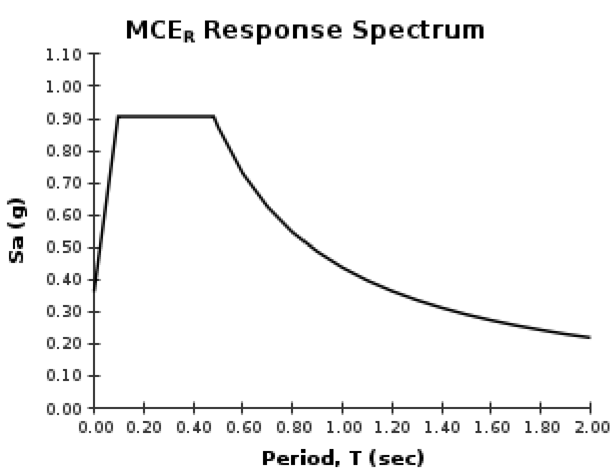
User–Specified Input

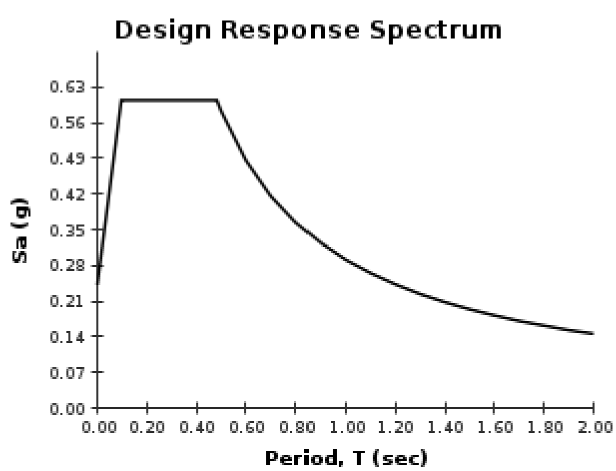
Report Title	Quinn Wed September 14, 2016 23:33:51 UTC
Building Code Reference Document	2012/2015 International Building Code (which utilizes USGS hazard data available in 2008)
Site Coordinates	41.2403°N, 111.8027°W
Site Soil Classification	Site Class C – “Very Dense Soil and Soft Rock”
Risk Category	I/II/III

USGS–Provided Output

S_S =	0.857 g	S_{MS} =	0.906 g	S_{DS} =	0.604 g
S_1 =	0.289 g	S_{M1} =	0.437 g	S_{D1} =	0.291 g

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

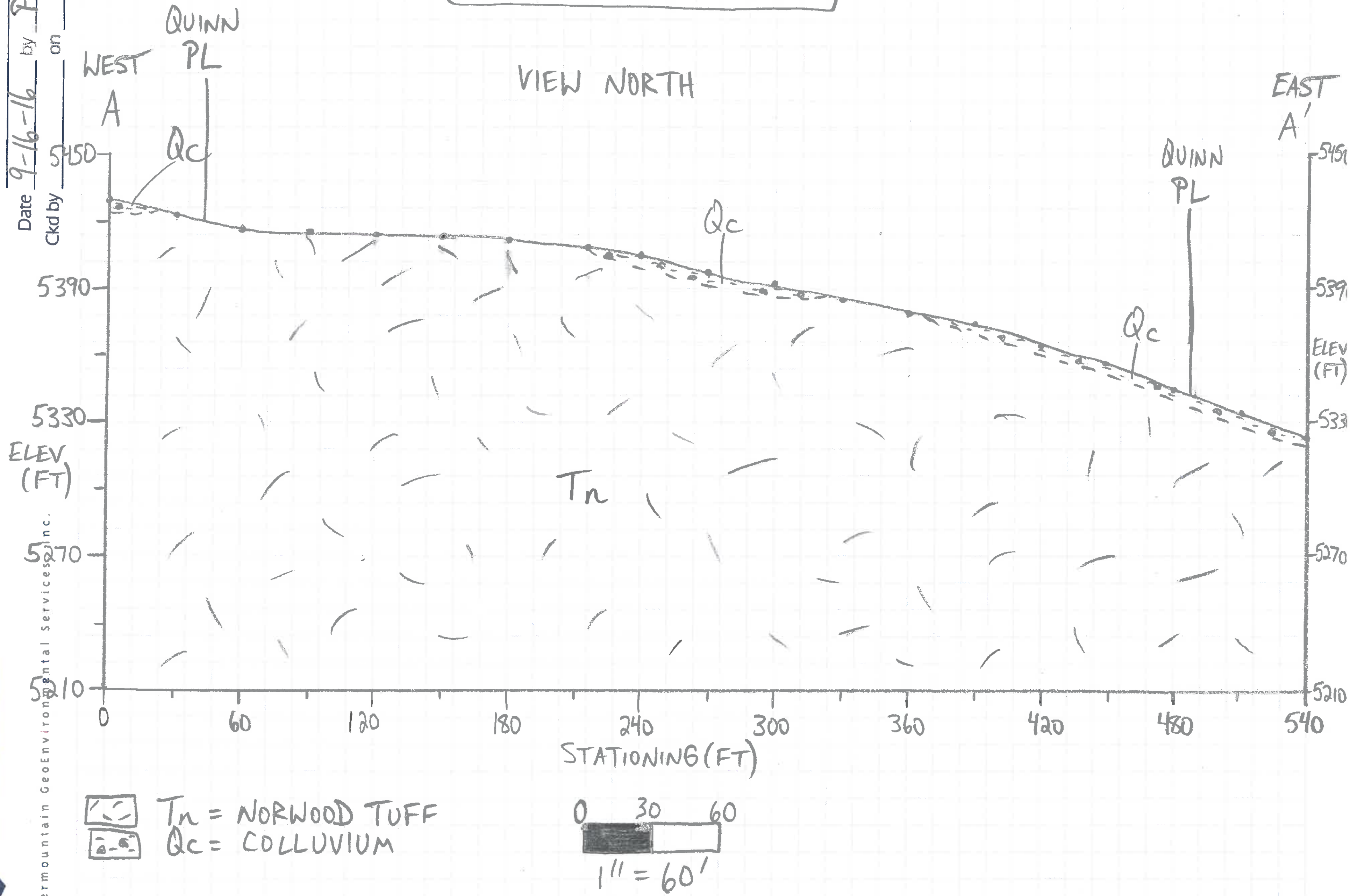




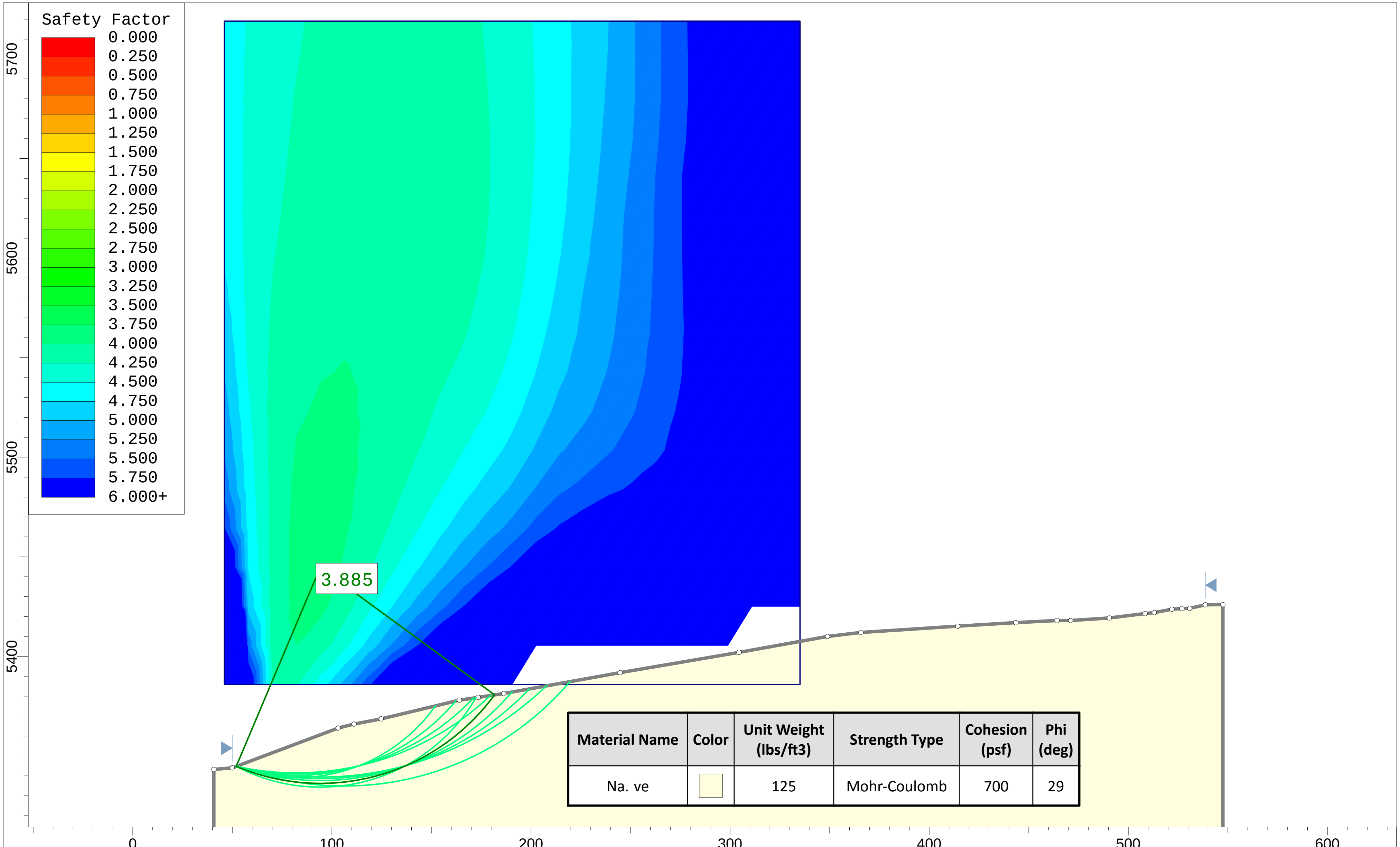
APPENDIX D

Project No. 02350-001
Date 9-16-16 by PED
Ckd by ____ on ____

FIGURE D-1
CROSS-SECTION A-A'



IGES
Intermountain GeoEnvironmental Services, Inc.



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Na. ve		125	Mohr-Coulomb	700	29

IGES®

SLIDEINTERPRET 7.018

Project	Thomas Quinn Property		
Analysis Description	Section A' : A Global Stability		
Drawn By JKW	*Scale* 1:800	*Company* IGES, Inc.	
Date 9-15-2016		*File Name* Section A_A.slim	

Slide Analysis Information

Thomas Quinn Property

Project Summary

File Name:	Section A_A (Static)
Slide Modeler Version:	7.018
Project Title:	Thomas Quinn Property
Analysis:	Section A' : A Global Stability
Author:	JKW
Company:	IGES, Inc.
Date Created:	9-15-2016

General Settings

Units of Measurement:	Imperial Units
Time Units:	days
Permeability Units:	feet/second
Failure Direction:	Right to Left
Data Output:	Standard
Maximum Material Properties:	20
Maximum Support Properties:	20

Analysis Options

Slices Type:	Vertical

Analysis Methods Used
Spencer

Number of slices:	50
Tolerance:	0.005
Maximum number of iterations:	75
Check malpha < 0.2:	Yes
Create Interslice boundaries at intersections with water tables and piezos:	Yes
Initial trial value of FS:	1
Steffensen Iteration:	Yes

Groundwater Analysis

Groundwater Method:	Water Surfaces
Pore Fluid Unit Weight [lbs/ft3]:	62.4
Use negative pore pressure cutoff:	Yes
Maximum negative pore pressure [psf]:	0
Advanced Groundwater Method:	None

Random Numbers

Pseudo-random Seed:	10116
Random Number Generation Method:	Park and Miller v.3

Surface Options

Surface Type:	Circular
Search Method:	Grid Search
Radius Increment:	10
Composite Surfaces:	Disabled

Reverse Curvature:	Invalid Surfaces
Minimum Elevation:	Not Defined
Minimum Depth:	Not Defined
Minimum Area:	Not Defined
Minimum Weight:	Not Defined

Seismic

Advanced seismic analysis:	No
Staged pseudostatic analysis:	No

Material Properties

Property	Native
Color	
Strength Type	Mohr-Coulomb
Unit Weight [lbs/ft3]	125
Cohesion [psf]	700
Friction Angle [deg]	29
Water Surface	None
Ru Value	0

Global Minimums

Method: spencer

FS	3.884840
Center:	94.182, 5444.656
Radius:	108.453
Left Slip Surface Endpoint:	51.988, 5344.748
Right Slip Surface Endpoint:	181.748, 5380.671
Resisting Moment:	3.01992e+007 lb-ft
Driving Moment:	7.7736e+006 lb-ft
Resisting Horizontal Force:	256321 lb
Driving Horizontal Force:	65979.9 lb
Total Slice Area:	2515.4 ft2
Surface Horizontal Width:	129.761 ft
Surface Average Height:	19.3849 ft

Valid / Invalid Surfaces

Method: spencer

Number of Valid Surfaces:	4647
Number of Invalid Surfaces:	303

Error Codes:

- Error Code -103 reported for 10 surfaces
- Error Code -108 reported for 14 surfaces
- Error Code -111 reported for 9 surfaces
- Error Code -112 reported for 10 surfaces
- Error Code -114 reported for 260 surfaces

Error Codes

The following errors were encountered during the computation:

- -103 = Two surface / slope intersections, but one or more surface / nonslope external polygon intersections lie between them. This usually occurs when the slip surface extends past the bottom of the soil region, but may also occur on a benched slope model with two sets of Slope Limits.

- -108 = Total driving moment or total driving force < 0.1. This is to limit the calculation of extremely high safety factors if the driving force is very small (0.1 is an arbitrary number).
- -111 = safety factor equation did not converge
- -112 = The coefficient M-Alpha = cos(alpha)(1+tan(alpha)tan(phi)/F) < 0.2 for the final iteration of the safety factor calculation. This screens out some slip surfaces which may not be valid in the context of the analysis, in particular, deep seated slip surfaces with many high negative base angle slices in the passive zone.
- -114 = Surface with Reverse Curvature.

Slice Data

- **Global Minimum Query (spencer) - Safety Factor: 3.88484**

Slice Number	Width [ft]	Weight [lbs]	Angle of Slice Base [degrees]	Base Material	Base Cohesion [psf]	Base Friction Angle [degrees]	Shear Stress [psf]	Shear Strength [psf]	Base Normal Stress [psf]	Pore Pressure [psf]	Effective Normal Stress [psf]
1	2.59522	329.722	-22.1553	Native	700	29	221.306	859.737	288.173	0	288.173
2	2.59522	976.679	-20.6823	Native	700	29	262.378	1019.3	576.026	0	576.026
3	2.59522	1599.02	-19.2235	Native	700	29	300.921	1169.03	846.148	0	846.148
4	2.59522	2197.41	-17.7776	Native	700	29	337.093	1309.55	1099.67	0	1099.67
5	2.59522	2772.46	-16.3432	Native	700	29	371.037	1441.42	1337.55	0	1337.55
6	2.59522	3324.7	-14.9194	Native	700	29	402.874	1565.1	1560.68	0	1560.68
7	2.59522	3854.59	-13.5049	Native	700	29	432.712	1681.02	1769.8	0	1769.8
8	2.59522	4362.56	-12.0987	Native	700	29	460.65	1789.55	1965.59	0	1965.59
9	2.59522	4848.98	-10.6999	Native	700	29	486.767	1891.01	2148.64	0	2148.64
10	2.59522	5314.15	-9.30756	Native	700	29	511.143	1985.71	2319.49	0	2319.49
11	2.59522	5758.34	-7.92073	Native	700	29	533.847	2073.91	2478.6	0	2478.6
12	2.59522	6181.8	-6.53855	Native	700	29	554.934	2155.83	2626.39	0	2626.39
13	2.59522	6584.7	-5.16019	Native	700	29	574.461	2231.69	2763.23	0	2763.23
14	2.59522	6967.2	-3.78481	Native	700	29	592.472	2301.66	2889.47	0	2889.47
15	2.59522	7329.42	-2.41162	Native	700	29	609.013	2365.92	3005.39	0	3005.39
16	2.59522	7671.43	-1.03981	Native	700	29	624.118	2424.6	3111.26	0	3111.26
17	2.59522	7993.28	0.331398	Native	700	29	637.825	2477.85	3207.32	0	3207.32
18	2.59522	8294.97	1.7028	Native	700	29	650.158	2525.76	3293.76	0	3293.76
19	2.59522	8576.48	3.07518	Native	700	29	661.147	2568.45	3370.77	0	3370.77
20	2.59522	8833.72	4.44932	Native	700	29	670.589	2605.13	3436.96	0	3436.96
21	2.59522	8996.46	5.82604	Native	700	29	674.679	2621.02	3465.61	0	3465.61
22	2.59522	9110.93	7.20615	Native	700	29	676.046	2626.33	3475.19	0	3475.19
23	2.59522	9203.82	8.59047	Native	700	29	676.185	2626.87	3476.18	0	3476.18

24	2.59522	9243.49	9.97987	Native	700	29	673.436	2616.19	3456.89	0	3456.89
25	2.59522	9246.13	11.3752	Native	700	29	668.715	2597.85	3423.81	0	3423.81
26	2.59522	9227.38	12.7775	Native	700	29	662.884	2575.2	3382.94	0	3382.94
27	2.59522	9186.9	14.1875	Native	700	29	655.947	2548.25	3334.33	0	3334.33
28	2.59522	9124.3	15.6064	Native	700	29	647.906	2517.01	3277.96	0	3277.96
29	2.59522	9055.8	17.0352	Native	700	29	639.607	2484.77	3219.8	0	3219.8
30	2.59522	8986.97	18.475	Native	700	29	631.331	2452.62	3161.81	0	3161.81
31	2.59522	8894.64	19.927	Native	700	29	621.912	2416.03	3095.79	0	3095.79
32	2.59522	8778.05	21.3925	Native	700	29	611.333	2374.93	3021.66	0	3021.66
33	2.59522	8636.48	22.8728	Native	700	29	599.59	2329.31	2939.35	0	2939.35
34	2.59522	8469.14	24.3695	Native	700	29	586.663	2279.09	2848.75	0	2848.75
35	2.59522	8275.12	25.884	Native	700	29	572.541	2224.23	2749.79	0	2749.79
36	2.59522	8053.42	27.4183	Native	700	29	557.207	2164.66	2642.33	0	2642.33
37	2.59522	7802.88	28.9743	Native	700	29	540.643	2100.31	2526.22	0	2526.22
38	2.59522	7522.22	30.554	Native	700	29	522.819	2031.07	2401.31	0	2401.31
39	2.59522	7209.97	32.1599	Native	700	29	503.712	1956.84	2267.4	0	2267.4
40	2.59522	6864.48	33.7946	Native	700	29	483.291	1877.51	2124.29	0	2124.29
41	2.59522	6483.83	35.4612	Native	700	29	461.525	1792.95	1971.73	0	1971.73
42	2.59522	6065.83	37.1631	Native	700	29	438.368	1702.99	1809.45	0	1809.45
43	2.59522	5607.94	38.9043	Native	700	29	413.783	1607.48	1637.14	0	1637.14
44	2.59522	5083.85	40.6894	Native	700	29	386.706	1502.29	1447.37	0	1447.37
45	2.59522	4465.28	42.5237	Native	700	29	356.074	1383.29	1232.69	0	1232.69
46	2.59522	3794.76	44.4137	Native	700	29	323.882	1258.23	1007.07	0	1007.07
47	2.59522	3068.77	46.367	Native	700	29	290.131	1127.11	770.53	0	770.53
48	2.59522	2281.19	48.3929	Native	700	29	254.754	989.677	522.592	0	522.592
49	2.59522	1424.43	50.5033	Native	700	29	217.675	845.634	262.73	0	262.73
50	2.59522	488.85	52.7128	Native	700	29	178.867	694.869	-9.25733	0	-9.25733

Interslice Data

- **Global Minimum Query (spencer) - Safety Factor: 3.88484**

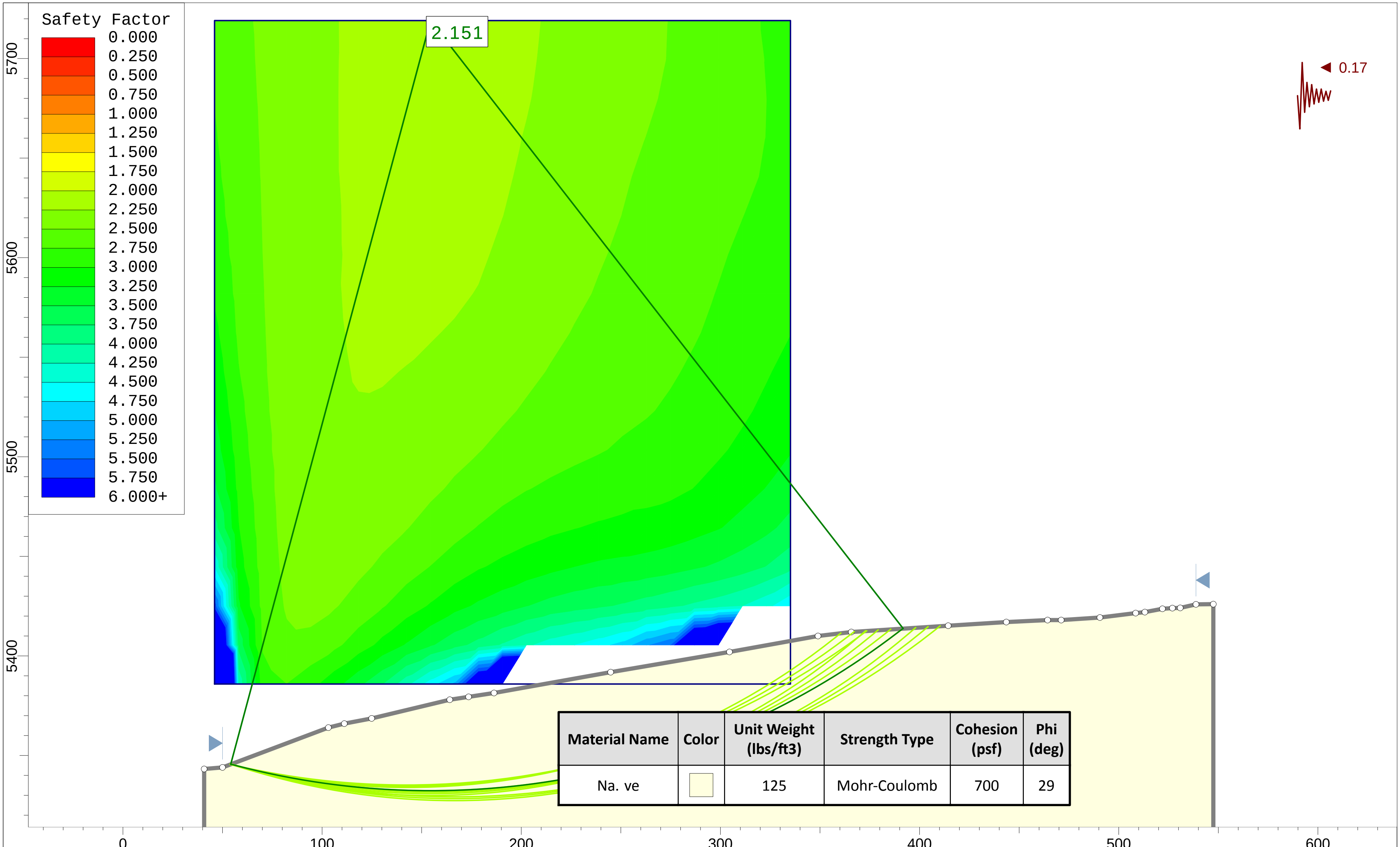
Slice Number	X coordinate [ft]	Y coordinate - Bottom [ft]	Interslice Normal Force [lbs]	Interslice Shear Force [lbs]	Interslice Force Angle [degrees]
1	51.9877	5344.75	0	0	0
2	54.5829	5343.69	878.428	185.73	11.9385
3	57.1781	5342.71	2123.2	448.918	11.9385
4	59.7733	5341.81	3669.29	775.814	11.9385
5	62.3686	5340.97	5458.51	1154.12	11.9385
6	64.9638	5340.21	7438.62	1572.78	11.9385
7	67.559	5339.52	9562.56	2021.85	11.9384
8	70.1542	5338.9	11787.8	2492.35	11.9385
9	72.7494	5338.34	14075.9	2976.12	11.9384
10	75.3446	5337.85	16391.8	3465.79	11.9385
11	77.9398	5337.43	18703.9	3954.65	11.9385
12	80.5351	5337.07	20983.3	4436.59	11.9385
13	83.1303	5336.77	23203.6	4906.04	11.9385
14	85.7255	5336.53	25341	5357.95	11.9385
15	88.3207	5336.36	27373.5	5787.69	11.9385
16	90.9159	5336.25	29281.3	6191.07	11.9385

17	93.5111	5336.21	31046.4	6564.27	11.9385
18	96.1064	5336.22	32652.3	6903.81	11.9385
19	98.7016	5336.3	34084.2	7206.57	11.9385
20	101.297	5336.44	35328.8	7469.72	11.9385
21	103.892	5336.64	36373.8	7690.66	11.9385
22	106.487	5336.9	37205.7	7866.56	11.9385
23	109.082	5337.23	37818.5	7996.13	11.9385
24	111.678	5337.62	38209.2	8078.74	11.9385
25	114.273	5338.08	38377	8114.21	11.9385
26	116.868	5338.6	38323.5	8102.91	11.9385
27	119.463	5339.19	38051.6	8045.4	11.9384
28	122.059	5339.85	37565	7942.53	11.9385
29	124.654	5340.57	36869	7795.36	11.9384
30	127.249	5341.37	35967.3	7604.72	11.9385
31	129.844	5342.23	34863	7371.23	11.9385
32	132.439	5343.18	33563.1	7096.39	11.9385
33	135.035	5344.19	32076.5	6782.07	11.9385
34	137.63	5345.29	30413.3	6430.42	11.9385
35	140.225	5346.46	28585.8	6044.02	11.9385
36	142.82	5347.72	26607.8	5625.81	11.9385
37	145.415	5349.07	24495.5	5179.19	11.9385
38	148.011	5350.5	22267.3	4708.07	11.9385
39	150.606	5352.04	19944.3	4216.91	11.9385
40	153.201	5353.67	17550.7	3710.83	11.9385
41	155.796	5355.41	15114.2	3195.65	11.9384
42	158.392	5357.25	12666.3	2678.09	11.9385
43	160.987	5359.22	10243.5	2165.82	11.9384
44	163.582	5361.32	7887.7	1667.73	11.9385
45	166.177	5363.55	5660.87	1196.9	11.9384
46	168.772	5365.93	3650.4	771.821	11.9385
47	171.368	5368.47	1929.71	408.006	11.9384
48	173.963	5371.19	584.637	123.612	11.9384
49	176.558	5374.11	-281.901	-59.6036	11.9385
50	179.153	5377.26	-544.643	-115.156	11.9384
51	181.748	5380.67	0	0	0

List Of Coordinates

External Boundary

X	Y
40.7443	5343.22
40.7443	5183.4
547.509	5183.4
547.509	5426
538.752	5425.88
530.871	5424.11
526.934	5424
521.902	5423.66
513.076	5422
508.459	5421.49
490.541	5419.3
471.073	5418.01
464.235	5418
443.519	5416.97
414.346	5415.18
365.72	5412
348.986	5410
304.533	5402
244.872	5391.84
186.34	5381.37
173.513	5379.42
164.1	5377.99
124.866	5368.63
111.17	5366
103.176	5364
50	5344



IGES®	Project	Thomas Quinn Property	
	Analysis Description	Section A' : A Global Stability	
	Drawn By: JKW	Scale: 1:800	Company: IGES, Inc.
SLIDEINTERPRET 7.018	Date: 9-15-2016		File Name: Section A_A.slim

Slide Analysis Information

Thomas Quinn Property

Project Summary

File Name:	Section A_A (Seismic)
Slide Modeler Version:	7.018
Project Title:	Thomas Quinn Property
Analysis:	Section A' : A Global Stability
Author:	JKW
Company:	IGES, Inc.
Date Created:	9-15-2016

General Settings

Units of Measurement:	Imperial Units
Time Units:	days
Permeability Units:	feet/second
Failure Direction:	Right to Left
Data Output:	Standard
Maximum Material Properties:	20
Maximum Support Properties:	20

Analysis Options

Slices Type:	Vertical

Analysis Methods Used	
	Spencer

Number of slices:	50
Tolerance:	0.005
Maximum number of iterations:	75
Check malpha < 0.2:	Yes
Create Interslice boundaries at intersections with water tables and piezos:	Yes
Initial trial value of FS:	1
Steffensen Iteration:	Yes

Groundwater Analysis

Groundwater Method:	Water Surfaces
Pore Fluid Unit Weight [lbs/ft3]:	62.4
Use negative pore pressure cutoff:	Yes
Maximum negative pore pressure [psf]:	0
Advanced Groundwater Method:	None

Random Numbers

Pseudo-random Seed:	10116
Random Number Generation Method:	Park and Miller v.3

Surface Options

Surface Type:	Circular
Search Method:	Grid Search
Radius Increment:	10
Composite Surfaces:	Disabled

Reverse Curvature:	Invalid Surfaces
Minimum Elevation:	Not Defined
Minimum Depth:	Not Defined
Minimum Area:	Not Defined
Minimum Weight:	Not Defined

Seismic

Advanced seismic analysis:	No
Staged pseudostatic analysis:	No

Loading

Seismic Load Coefficient (Horizontal): 0.17

Material Properties

Property	Native
Color	
Strength Type	Mohr-Coulomb
Unit Weight [lbs/ft3]	125
Cohesion [psf]	700
Friction Angle [deg]	29
Water Surface	None
Ru Value	0

Global Minimums

Method: spencer

FS	2.150510
Center:	154.420, 5719.034
Radius:	386.735
Left Slip Surface Endpoint:	54.093, 5345.539
Right Slip Surface Endpoint:	391.777, 5413.706
Resisting Moment:	4.01679e+008 lb-ft
Driving Moment:	1.86784e+008 lb-ft
Resisting Horizontal Force:	999874 lb
Driving Horizontal Force:	464948 lb
Total Slice Area:	11646.3 ft2
Surface Horizontal Width:	337.684 ft
Surface Average Height:	34.4888 ft

Valid / Invalid Surfaces

Method: spencer

Number of Valid Surfaces:	4630
Number of Invalid Surfaces:	320

Error Codes:

- Error Code -103 reported for 10 surfaces
- Error Code -108 reported for 7 surfaces
- Error Code -111 reported for 28 surfaces
- Error Code -112 reported for 15 surfaces
- Error Code -114 reported for 260 surfaces

Error Codes

The following errors were encountered during the computation:

- -103 = Two surface / slope intersections, but one or more surface / nonslope external polygon intersections lie between them. This usually occurs when the slip surface extends past the bottom of the soil region, but may also occur on a benched slope model with two sets of Slope Limits.
- -108 = Total driving moment or total driving force < 0.1. This is to limit the calculation of extremely high safety factors if the driving force is very small (0.1 is an arbitrary number).
- -111 = safety factor equation did not converge
- -112 = The coefficient M-Alpha = cos(alpha)(1+tan(alpha)tan(phi)/F) < 0.2 for the final iteration of the safety factor calculation. This screens out some slip surfaces which may not be valid in the context of the analysis, in particular, deep seated slip surfaces with many high negative base angle slices in the passive zone.
- -114 = Surface with Reverse Curvature.

Slice Data

- **Global Minimum Query (spencer) - Safety Factor: 2.15051**

Slice Number	Width [ft]	Weight [lbs]	Angle of Slice Base [degrees]	Base Material	Base Cohesion [psf]	Base Friction Angle [degrees]	Shear Stress [psf]	Shear Strength [psf]	Base Normal Stress [psf]	Pore Pressure [psf]	Effective Normal Stress [psf]
1	6.75368	1810.45	-14.5189	Native	700	29	477.702	1027.3	590.468	0	590.468
2	6.75368	5376.84	-13.4876	Native	700	29	640.258	1376.88	1221.13	0	1221.13
3	6.75368	8834.92	-12.4608	Native	700	29	793.268	1705.93	1814.75	0	1814.75
4	6.75368	12186	-11.438	Native	700	29	937.243	2015.55	2373.32	0	2373.32
5	6.75368	15431.4	-10.4188	Native	700	29	1072.64	2306.73	2898.63	0	2898.63
6	6.75368	18572	-9.40306	Native	700	29	1199.89	2580.38	3392.28	0	3392.28
7	6.75368	21609	-8.39024	Native	700	29	1319.36	2837.3	3855.79	0	3855.79
8	6.75368	24350.6	-7.38007	Native	700	29	1423.11	3060.42	4258.31	0	4258.31
9	6.75368	26440.2	-6.37219	Native	700	29	1496.56	3218.36	4543.25	0	4543.25
10	6.75368	28153.4	-5.36629	Native	700	29	1552.41	3338.47	4759.92	0	4759.92
11	6.75368	29768	-4.36204	Native	700	29	1602.85	3446.94	4955.61	0	4955.61
12	6.75368	31482.7	-3.35914	Native	700	29	1656.21	3561.7	5162.65	0	5162.65
13	6.75368	33128	-2.35727	Native	700	29	1705.52	3667.74	5353.94	0	5353.94
14	6.75368	34673.6	-1.35612	Native	700	29	1749.67	3762.69	5525.25	0	5525.25
15	6.75368	36119.4	-0.355379	Native	700	29	1788.85	3846.94	5677.23	0	5677.23
16	6.75368	37465.6	0.64525	Native	700	29	1823.2	3920.82	5810.51	0	5810.51
17	6.75368	38587.2	1.64608	Native	700	29	1848.04	3974.23	5906.87	0	5906.87
18	6.75368	39260.6	2.64741	Native	700	29	1855.11	3989.44	5934.3	0	5934.3
19	6.75368	39813.5	3.64954	Native	700	29	1857.4	3994.35	5943.17	0	5943.17
20	6.75368	40279.6	4.6528	Native	700	29	1856.3	3992	5938.92	0	5938.92
21	6.75368	40759.3	5.65749	Native	700	29	1855.64	3990.57	5936.34	0	5936.34
22	6.75368	41163.8	6.66393	Native	700	29	1852.13	3983.02	5922.72	0	5922.72

23	6.75368	41466.7	7.67244	Native	700	29	1844.89	3967.45	5894.64	0	5894.64
24	6.75368	41667.3	8.68335	Native	700	29	1834	3944.04	5852.41	0	5852.41
25	6.75368	41764.8	9.69699	Native	700	29	1819.55	3912.96	5796.32	0	5796.32
26	6.75368	41758.4	10.7137	Native	700	29	1801.6	3874.35	5726.68	0	5726.68
27	6.75368	41646.9	11.7339	Native	700	29	1780.21	3828.36	5643.71	0	5643.71
28	6.75368	41429.3	12.7578	Native	700	29	1755.46	3775.13	5547.69	0	5547.69
29	6.75368	41090.5	13.7859	Native	700	29	1726.93	3713.77	5436.97	0	5436.97
30	6.75368	40609.3	14.8185	Native	700	29	1694.01	3642.98	5309.28	0	5309.28
31	6.75368	40016.3	15.8561	Native	700	29	1657.87	3565.27	5169.07	0	5169.07
32	6.75368	39311.4	16.8991	Native	700	29	1618.61	3480.83	5016.75	0	5016.75
33	6.75368	38492.8	17.9478	Native	700	29	1576.26	3389.76	4852.45	0	4852.45
34	6.75368	37558.5	19.0028	Native	700	29	1530.86	3292.14	4676.34	0	4676.34
35	6.75368	36506.4	20.0646	Native	700	29	1482.46	3188.05	4488.57	0	4488.57
36	6.75368	35334.1	21.1336	Native	700	29	1431.09	3077.57	4289.26	0	4289.26
37	6.75368	34039.1	22.2103	Native	700	29	1376.78	2960.77	4078.54	0	4078.54
38	6.75368	32641.8	23.2954	Native	700	29	1320.26	2839.23	3859.27	0	3859.27
39	6.75368	31147.7	24.3894	Native	700	29	1261.8	2713.52	3632.48	0	3632.48
40	6.75368	29521.9	25.493	Native	700	29	1200.45	2581.58	3394.46	0	3394.46
41	6.75368	27760.7	26.6068	Native	700	29	1136.22	2443.45	3145.27	0	3145.27
42	6.75368	25860.1	27.7316	Native	700	29	1069.13	2299.17	2884.98	0	2884.98
43	6.75368	23815.9	28.8681	Native	700	29	999.2	2148.79	2613.7	0	2613.7
44	6.75368	21603.8	30.0172	Native	700	29	925.897	1991.15	2329.31	0	2329.31
45	6.75368	18989.1	31.1797	Native	700	29	842.786	1812.42	2006.86	0	2006.86
46	6.75368	16139.3	32.3568	Native	700	29	754.967	1623.57	1666.16	0	1666.16
47	6.75368	13010.7	33.5493	Native	700	29	661.515	1422.59	1303.6	0	1303.6
48	6.75368	9518.33	34.7586	Native	700	29	560.457	1205.27	911.527	0	911.527
49	6.75368	5843.13	35.9858	Native	700	29	457.06	982.912	510.386	0	510.386
50	6.75368	1979.8	37.2325	Native	700	29	359.888	773.943	133.395	0	133.395

Interslice Data

- **Global Minimum Query (spencer) - Safety Factor: 2.15051**

Slice Number	X coordinate [ft]	Y coordinate - Bottom [ft]	Interslice Normal Force [lbs]	Interslice Shear Force [lbs]	Interslice Force Angle [degrees]
1	54.0927	5345.54	0	0	0
2	60.8463	5343.79	3958.09	1375.31	19.1607
3	67.6	5342.17	9355.44	3250.72	19.1607
4	74.3537	5340.68	15930.8	5535.45	19.1607
5	81.1074	5339.31	23445.6	8146.59	19.1607
6	87.8611	5338.07	31681.6	11008.4	19.1608
7	94.6147	5336.95	40439.4	14051.4	19.1607
8	101.368	5335.96	49536.4	17212.3	19.1607
9	108.122	5335.08	58753.6	20415	19.1607
10	114.876	5334.33	67814.3	23563.3	19.1607
11	121.629	5333.69	76554.8	26600.4	19.1607
12	128.383	5333.18	84895.5	29498.5	19.1607
13	135.137	5332.78	92799.4	32244.8	19.1607
14	141.89	5332.5	100199	34816.1	19.1608
15	148.644	5332.34	107030	37189.6	19.1608

16	155.398	5332.3	113235	39345.5	19.1607
17	162.152	5332.38	118763	41266.5	19.1608
18	168.905	5332.57	123565	42934.9	19.1607
19	175.659	5332.88	127593	44334.6	19.1607
20	182.413	5333.31	130836	45461.3	19.1607
21	189.166	5333.86	133288	46313.2	19.1607
22	195.92	5334.53	134946	46889.5	19.1607
23	202.674	5335.32	135810	47189.7	19.1607
24	209.427	5336.23	135884	47215.5	19.1608
25	216.181	5337.26	135177	46969.7	19.1607
26	222.935	5338.42	133703	46457.4	19.1607
27	229.688	5339.69	131480	45685	19.1607
28	236.442	5341.1	128532	44660.6	19.1606
29	243.196	5342.63	124886	43394	19.1607
30	249.949	5344.28	120579	41897.5	19.1608
31	256.703	5346.07	115655	40186.4	19.1607
32	263.457	5347.99	110157	38276.1	19.1607
33	270.21	5350.04	104136	36183.9	19.1607
34	276.964	5352.23	97645.1	33928.6	19.1607
35	283.718	5354.55	90744.8	31530.9	19.1607
36	290.471	5357.02	83500	29013.6	19.1607
37	297.225	5359.63	75981.5	26401.2	19.1607
38	303.979	5362.39	68266.3	23720.4	19.1607
39	310.732	5365.3	60430.3	20997.6	19.1607
40	317.486	5368.36	52552.2	18260.2	19.1607
41	324.24	5371.58	44727	15541.2	19.1607
42	330.993	5374.96	37057.3	12876.2	19.1607
43	337.747	5378.51	29653.9	10303.8	19.1607
44	344.501	5382.24	22636.3	7865.4	19.1607
45	351.254	5386.14	16141.5	5608.64	19.1607
46	358.008	5390.22	10415.5	3619.06	19.1607
47	364.762	5394.5	5652.3	1963.99	19.1607
48	371.516	5398.98	2079.51	722.563	19.1607
49	378.269	5403.67	-17.413	-6.05047	19.1607
50	385.023	5408.57	-420.384	-146.07	19.1607
51	391.777	5413.71	0	0	0

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