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GEOTECHNICAL AND GEOLOGIC HAZARD INVESTIGATION
Matthew Toliver Property
Parcel No. 20-035-0039, Old Snowbasin Road
Unincorporated Weber County, Utah

IGES Project No. 02489-001

July 19, 2017

Prepared for:

Mr. Matthew Toliver

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1.0 EXECUTIVE SUMMARY

This report presents the results of a geotechnical and geologic hazard investigation conducted for the *Matthew Toliver Property* (Parcel #200350039), located along Old Snow Basin 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 1.5 to 2 feet of topsoil, although sequences of topsoil as thick as 5.5 feet were observed locally. However, the prevailing earth materials encountered consisted of weathered bedrock (Norwood Formation), which readily disaggregates to sandy lean clay (CL) grading to sandy fat clay (CH) within the uppermost 5 to 7 feet. Below about 7 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 uppermost 11 feet below existing grade. With increasing depth, excavation became increasingly difficult, but was still possible with a large tracked excavator. **Adverse geologic conditions were not encountered within the proposed building site.**
- Footings for the proposed residential structure should be founded either *entirely* on bedrock or *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 *Matthew Toliver Property,* Parcel #200350039, located along Old Snow Basin 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 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 January 24, 2017 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 Snow Basin Road (Highway 226) in Weber County, near the City of Huntsville, Utah, approximately 1.2 miles 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. We understand that the proposed development will consist of a single-family residence and possibly a detached garage, a driveway, and utilities over an approximately 11-acre site. Given that much of the existing topography on the property consists of steep slopes, an approximately 0.78-acre rectangular area in the south-central portion of the property has been identified as the proposed building site (limits of disturbance); this site is located on an elongated knob that is isolated between two drainages, and contains the gentlest grades on the property (see Figure A-2, *Geotechnical Map*). 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. 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-3). Coogan and King (2016) provide the most recent published geologic mapping that covers the project area, but at a regional (1:62,500) scale (Figure A-4). 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 April 14, 2017. The site reconnaissance was conducted with the intent to assess the general geologic conditions present across the property, with specific interest in those areas identified in the geologic literature and aerial imagery reviews as potential geologic hazard areas. Additionally, the site reconnaissance provided the opportunity to geologically map the surficial geology of the area. The road cuts for the proposed driveway were also assessed for potentially adverse conditions.

The subsurface component of the field investigation was performed on June 6, 2017. Four (4) exploration trenches were excavated in various locations across the property to depths generally ranging from 9 to 11 feet below existing grade. The exploration trenches were excavated with the aid of a Caterpillar 315C tracked excavator. Practical refusal on hard bedrock (Norwood Formation) was not encountered in any of the trenches, though the excavator noted harder digging with depth. 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 building site, including depth to bedrock and groundwater (if present); and

- Minimize disturbance to the dense native foliage on the property.

The *Geotechnical Map,* Figure A-2 in Appendix A, shows the approximate location of the exploration trenches, while the *Local Geology Map*, Figure A-5, shows the surficial geologic materials across and adjacent to the property as mapped from the site reconnaissance and encountered in the trenches. Subsurface conditions as encountered in the exploration trenches were logged at the time of our investigation by a licensed geologist. The trench logs are presented in Figures A-6 through A-9 of Appendix A. A *Key to Soil Symbols and Terminology* is presented as Figure A-10 and a *Key to Physical Rock Properties* is presented as Figure A-11. Upon completion of the trench logging, the trenches were backfilled without compacted effort.

Both bulk soil samples and 'undisturbed' tube soil samples were obtained from the trench explorations. 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 & dry density (ASTM D2216 and 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)

Selected test results are presented on the test pit logs; detailed test results 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 Toliver 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 Toliver property is located approximately 6.7 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 underlain by several different geologic units. The central portion of the property, including the building site, is mapped as an older block landslide deposit consisting of a largely intact block of Norwood Formation (see Figure A-3; unit Qmso(Tn)). This map shows the western portion of the property to be underlain by in-place Norwood Formation (unit Tn), with the drainages that isolate the elongated knob upon which the proposed building site sits mapped as being filled with undivided landslide/slump[1] and colluvial[2] deposits (unit Qmc) or undivided alluvial[3] and colluvial deposits (unit Qac), with the southeastern corner of the property to be underlain by landslide and slump deposits (unit Qms). Several landslide headscarps[4] have been mapped near the property, with two smaller scarps mapped near the northern and southwestern corners of the property, respectively, and a larger scarp located approximately 500 feet to the south of the southeastern margin of the property. As shown in Figure A-3, the entire mountainous area within several miles of the property is seen to be predominantly landslide or undivided landslide/colluvial deposits with pockets of in-place Norwood Formation (Tn) scattered throughout. In-place Norwood Formation bedrock in the vicinity of the property is shown to strike north-south and dip into the ground to the east at between approximately 14 and 19 degrees.

The more recent, though regional, Coogan and King (2016) map is largely consistent with King et al. (2008), though it shows more of the property to be underlain by the older block landslide deposits (unit Qmso(Tn); see Figure A-4). In this map, only the larger landslide headscarp south of the southeastern margin of the property is shown.

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

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[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] Alluvium: A general term for clay, silt, sand, gravel, or similar unconsolidated detrital material, deposited during comparatively recent geologic time by a stream or other body of running water. (AGI, 2005)
[4] Headscarp: The abrupt scarp at the head, or top, of a landslide or slump. (AGI, 2005)

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 south 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.

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 landslide scarp to the south of the property exposes the only rock in the vicinity of the property, and the property was observed to be extremely densely covered in trees. The only human-disturbed area consists of an access road that crosses the middle fork of Smith Creek in the north-central part of the property. This access road was found to be emplaced sometime between August 18, 2003 and May 26, 2004, with an additional access road to the main part of the property branching off of the aforementioned access road being put in sometime between June 6, 2015 and July 8, 2016.

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

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 trench 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-5 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. Steep slopes were observed across most of the property, though these slopes were generally found to be at a consistent grade. Two sizable drainages were observed to pass roughly north-south through the property, which isolate the large elongated knob upon which the proposed building site is located (see Figure A-2). The eastern drainage is named Smith Creek, and was observed at the time of the site reconnaissance to be weakly to intermittently flowing with water. This drainage did not appear to have had a significant amount of flowing water, and anecdotal evidence suggests this drainage rarely flows with water (Matt Toliver, 2017, personal communication). The western drainage is unnamed, wider than the Smith Creek drainage (as much as 50 feet in places), and was observed to have slowly flowing water at the time of the site reconnaissance. Approximately 90 feet south of the southwestern margin of the property, a small, localized landslide feature was observed on the eastern side of this drainage. This feature appeared to be a surficial slide that was at least 25 years old (Matt Toliver, 2017, personal communication), was approximately 25 feet wide and approximately 3 to 4 feet thick. Within the drainages, occasional angular to subangular boulders of light gray to moderate yellowish brown Norwood Formation tuffaceous sandstone and sandy tuff[5] up to 3 feet in diameter were observed.

The proposed building site on the top of the elongated knob was found to contain the gentlest topography on the property, with an average grade of approximately 7:1 (horizontal:vertical). A thin colluvial cover was found across much of the property, evidenced by rare Norwood Formation gravel clasts[6] lying on the surface. Norwood Formation (Tn) bedrock was not observed to outcrop on the property, but was observed in several places along the driveway road cut leading up to the proposed building site. No surface water was observed in the vicinity of the proposed building site.

Subsurface Investigation

In order to observe the subsurface conditions across the property, four trenches were excavated. In general, topsoil encountered was generally a brownish black to dark yellowish brown fat clay between 1.5 and 2 feet thick. Underlying the topsoil was a brownish gray fat clay between 2 and as much as 8 feet thick that was interpreted to be a colluvial unit, but could also possibly represent completely decomposed bedrock. Norwood Formation consisting of partially to highly weathered tuffaceous sandstone and sandy tuff bedrock was encountered in three of the four excavations (absent in T-2), generally between the depths of 5 and 7 feet below existing grade. Where encountered, the Norwood Formation bedrock was observed to dip between 10 and 21 degrees to the east.

Based upon the surface reconnaissance and subsurface investigation, four distinct geologic units were differentiated on the property (see Figure A-5). Each of these units are discussed in turn below.

Qac (Quaternary Alluvium and Colluvium)

This unit was mapped within the two drainages that trend roughly north-south across the property, and was not assessed with the trenches. The unit is characterized by the presence of angular to subangular cobbles and boulders of Norwood Formation tuffaceous sandstone and sandy tuff up to 3 feet in diameter. The unit has an unknown thickness, but is unlikely to exceed 10 feet.

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[5] Tuff: Consolidated or cemented volcanic ash and lapilli. (AGI, 2005)

[6] 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)

Qmc (Quaternary Undifferentiated Colluvium and Mass-Movement Deposits)
This unit was mapped on both sides of the two drainages, along the hillslopes leading into both drainages. The unit is characterized on the surface by the presence of uneven, possibly hummocky ground in some places. In the subsurface, this unit contained characteristics of both colluvium and some possible localized landslide deposits. The colluvial component was a brownish gray lean to fat clay, sometimes containing Norwood Formation gravel-sized clasts. Rare slickensides[7] and calcium carbonate deposits dipping consistent with the modern slope suggest that the unit is prone to mass-movement. The unit was found to be as much as 8 feet thick or more.

Qls (Quaternary Landslide Deposits)
This unit was mapped in the southeastern corner of the property and south of the property, both east of the Smith Creek drainage. The unit was characterized on the surface by irregular, hummocky topography, and was not observed in the subsurface. The thickness of these landslide deposits is currently unknown.

Tn (Tertiary Norwood Formation)
This unit was mapped across much of the elongated knob containing the building site, and was found on the surrounding hillslopes that exhibited consistent topographic slopes. When encountered in the subsurface, Norwood Formation bedrock was found to be highly silty and sandy, and was commonly partially to highly weathered (chemically altered) to sandy fat clay (CH) and well-graded sand (SW). Where intact, the bedrock was generally found to be a medium light gray to light gray tuffaceous sandstone grading to a sandy tuff that was fine-grained, weakly calcium carbonate and iron oxide cemented, moderately hard, medium bedded, blocky jointed, and finely laminated in places. The unit is at least 5 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 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.

[7] Slickenside: Originally, a polished fault surface formed by frictional wear during sliding, but now used to denote any of several types of lineated fault surfaces… Slickensides are also common below 50 cm in swelling clays subject to large changes in water content. (AGI, 2005)

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 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 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. Much of the property has been mapped as being underlain by some form of mass-movement deposits (King, et al., 2008; Coogan and King, 2016; see Figures A-3 and A-4), and a significant landslide headscarp south of the property was observed in the aerial imagery and during the site reconnaissance (see Figure A-5). Additionally, a small localized landslide was observed adjacent to the western drainage south of the property during the site reconnaissance (see Figure A-5). However, on the most recently published landslide map covering the property, no landslides are mapped on the property, with the southern margin of the property adjacent to an area mapped as “landslide undifferentiated from talus and/or colluvial deposits” (Elliott and Harty, 2010).

No evidence of landsliding was observed within and adjacent to the proposed building site during the site reconnaissance, and therefore the four trenches were specifically spotted to observe subsurface conditions along the slopes at the four corners of the proposed building site, where the grade significantly steepens in these areas (see Figure A-2). Shallow Norwood Formation bedrock was encountered in 3 of the 4 trenches between the depths of 5 and 7 feet below existing grade. In T-2, a colluvial unit was observed to extend to the total depth of the trench (11 feet) and showed bedding dipping consistent with the modern slope (see Figure A-7). This unit became increasingly sandy with depth, possibly indicating a transition into weathered bedrock. T-1 and T-3 exhibited a colluvium/possibly highly weathered bedrock sandy fat clay unit below the topsoil that displayed rare discontinuous slickensides and a glassy fat clay sheen, though the contact between the unit and the underlying bedrock unit did not exhibit slickensides or any evidence of shear (see Figures

A-6 and A-8). In T-4, a possible shallow landslide was postulated (Unit 3) due to the common presence of white to light brownish gray mudstone clasts, which were not observed anywhere else on the property (see Figure A-9). This unit also contained common pinholes, a significantly larger proportion of gravel and larger-sized clasts than seen in other units across the property, and a clay-rich basal subunit, though no slickensides or evidence of shear was observed within the unit or at the contact with the underlying bedrock.

Though the most recent geologic mapping shows that the proposed building site is located on a knob of older block landslide deposits, it is our conclusion that the proposed building site appears to be located on in-place Norwood Formation bedrock. This is evidenced by the fact that the bedding orientations are largely consistent with previously mapped orientations for the in-place Norwood Formation; dip of the bedrock as observed in T-1, T-3, and T-4 was between 10 and 21 degrees to the east (see Figure A-5), which is consistent with the 14 to 19 degree dips to the east as shown on the maps for the in-place Norwood Formation (see Figure A-3 and A-4). Only the colluvial unit in T-2 and also exposed in the driveway road cut were alternative dips (to the west and consistent with the modern slope) observed. Additionally, no definitive slide planes that would be associated with a large block landslide deposit were observed in the trenches.

Given this data, the landslide hazard risk associated with development on the property at large is considered to be moderate to high. However, the landslide hazard risk associated with development within the proposed building site is considered to be low to moderate. Most of the proposed building site contains gentle slopes, shallow bedrock, and there is no evidence of active movement along a distinct, continuous slide plane even when a possible shallow landslide unit was observed locally in T-4. Though no evidence of active movement was observed in the subsurface, the Norwood Formation is a known landslide-prone unit, and the clayey colluvium/highly weathered bedrock that overlies it has the potential to move under the right conditions. 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 2015 International Building Code (IBC, 2015), spectral response at the site was evaluated for the *Maximum Considered Earthquake* (MCE) which equates to a probabilistic seismic event having a two percent probability of exceedance in 50 years (2PE50). Spectral accelerations were determined based on the location of the site using the *U.S. Seismic "DesignMaps" Web Application* (USGS, 2012/15); this software incorporates seismic hazard maps depicting probabilistic ground motions and spectral response data developed for the United States by the U. S. Geological Survey as part of NEHRP/NSHMP (Frankel et al., 1996). These maps have been incorporated into both *NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures* (FEMA, 1997) and the *International Building Code* (IBC) (International Code Council, 2015).

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 1.060, and the long-period site coefficient (F_v) is 1.513. 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; 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)	$S_S = 0.850$	$S_1 = 0.287$
MCE Spectral Response Acceleration Site Class C (g)	$S_{MS} = S_s F_a = 0.901$	$S_{M1} = S_1 F_v = 0.434$
Design Spectral Response Acceleration (g)	$S_{DS} = S_{MS} * {}^2/_3 = 0.600$	$S_{D1} = S_{M1} * {}^2/_3 = 0.289$

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 trenches, 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 proposed building site being located near a topographic high, any structure located on the proposed building site would not be subjected to debris-flows, and thus the debris-flow hazard is considered to be low. Additionally, though the driveway crosses both drainages, there is little exposed material updrainage that could be incorporated into a potential debris-flow.

Similarly, any structure located within the proposed building site would not be subject to flooding hazards; accordingly, the flood hazard potential for the residence is considered to be low. However, the driveway crosses both drainages (the western drainage is crossed twice), and therefore may be subject to occasional flooding events. As such, the flood hazard potential for the driveway is considered to be moderate.

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 Formation of (at least in part) 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 temporary driveway. Dense scrub oak is found across most of the property, with native shrubs and grasses found in places within the proposed building site. In general, the property drains to the north. The elevation across the property ranges from approximately 5,452 feet (msl) along the south-central margin of the property to approximately 5,277 feet within the western drainage near the northern corner of the property. Within the proposed building site, the elevation ranges from 5,433 feet along the southern margin to 5,403 feet in the northeastern corner. The typical gradient across the proposed building site is approximately 7:1 (H:V), though much of the rest of the property is around 3:1.

5.2 SUBSURFACE CONDITIONS

The subsurface soils were investigated by excavating a total of four (4) exploration trenches across the property. Generally, the depth of the exploration trenches ranged from 9 to 11 feet, and though Norwood Formation bedrock was found in three of the four trenches, practical refusal was not experienced. The locations of the trenches are illustrated on Figure A-2, *Geotechnical Map*; detailed trench logs are presented in Figures A-6 through A-9. The earth materials encountered in the exploration trenches 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 5½ feet thick was observed locally. The topsoil is underlain by earth materials consisting of clayey colluvium and/or bedrock (Norwood Formation). Descriptions of the earth materials encountered are presented in the following paragraphs.

<u>Topsoil</u>: Generally consists of brownish black to black sandy fat clay (CH) with varying amounts of 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 trenches, 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 moderate yellowish brown, stiff to very stiff sandy fat clay (CH), often transitional to sandy lean clay (CL), and occasionally grading to clayey sand (SC). The coarse fraction was typically 50 percent or less; where measured, gravel-size constituents comprise about 12 percent by weight.

Bedrock: The prevailing earth materials onsite consists of the Norwood Formation unit, which is comprised of a combination weathered volcanic ash and block-and-ash deposits interbedded with sandstone and siltstone beds. This unit is highly weathered within the upper 10 feet and readily disaggregates to soils classifying as clayey sand with gravel (SC). Where measured, the liquid limit of the fines is 42; accordingly, much of the clay constituents will classify as lean clay (CL), although fines classifying as fat clay (CH) are also likely present.

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 trenches completed during this investigation, and is not expected to impact the development. Due to the season of our investigations (early June), we anticipate groundwater levels to be near their seasonal high. 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

Two consolidated-drained direct shear tests were completed under drained conditions on relatively ‘undisturbed’ samples of the colluvial material overlying the Norwood Formation. The test results indicated that the samples tested had friction angles on the order of 42 to 43 degrees and cohesion values of 167 psf and 682 psf (peak strength values). The test results indicate the surficial soils at the site have a relatively high friction angle (about 42 degrees); the tests also indicate the soils are cohesive, although the degree to which the soils are cohesive will be greatly dependent on the percent fines and plasticity of the fines.

Three samples of the Norwood Formation were tested to assess the uniaxial compressive strength of intact, moderately weathered bedrock. A wide range of values was obtained, ranging from 1,018 psi to 3,004 psi, with an average of 2,146 psi. The result was utilized to estimate the strength of the Norwood Formation for purposes of slope stability analysis.

5.2.4 Expansive Soils

Fat CLAY (CH) was described in the field descriptions for the colluvial soils, and was identified in our laboratory testing. 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 (Norwood Formation), expansive soils are not expected to significantly 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 Sections A-A', B-B', and C-C', illustrated on Figure A-3 and presented in Figures D-1 through D-3 in Appendix D. 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 0.36g. Half of the PGA, (0.18g), was taken as the horizontal seismic coefficient (k_h) (Hynes and Franklin, 1984), and used in the pseudo-static seismic screen analysis.

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 our observation of the subsurface, surficial soil strength was modeled as having a friction angle of 42 degrees and a cohesion (apparent cohesion) of 150 psf. The strength of the Norwood Formation was estimated using RocLab v.1.033 software combined with the uniaxial strength test results; from this exercise, the underlying Norwood Formation was modeled as having a cohesion of 2,000 psf and a friction angle of 25 degrees.

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 trenches 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 (preferably Norwood Formation) 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 Formation), or clayey 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 B soils (cohesive fine-grained soils), 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 1H:1V (45 degrees) in accordance with OSHA Type B soils (cohesive soils) 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 *that do not classify as fat clay* (CH) and/or bedrock, or an approved imported granular soil. For imported fill, 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 Formation 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 Formation), 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 Formation) 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 (2 percent is acceptable if the surface is comprised of relatively impermeable concrete flatwork).
- 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 version (or later) *International Residential Code* (IRC), Section R405, *Foundation Drainage*.

6.8 SLOPE GRADING - DRIVEWAY

The following generalized recommendations are for engineered slopes (cut slopes above the driveway). Recommendations for grading of cut slopes are intended to minimize the potential for future surficial failures. For purposes of this report, *surficial failure* includes excessive erosion, sloughing, slumping, mass wasting, rockfall, and similar relatively shallow failures.

For slope cuts in competent native earth materials, the cut slope should be no steeper than 2H:1V. In some instances, where the slope adjacent to the driveway is relatively steep, cutting a 2H:1V slope up-hill may be impractical as the distance to daylight at natural grade could be very far; in such cases, a more practical solution can be achieved by utilizing a rockery (0.5H:1V) or modular block wall (near vertical) near the roadway; in many cases, such measures can minimize or even eliminate the 2H:1V slope. Alternatively, cut slopes may be constructed at 1.5H:1V provided a turf reinforcing product is utilized that incorporates ground anchors, such as Western Excelsior PP5 Extreme Armoring System.

Slope planting/reseeding and other measures should be provided immediately following construction. Slope protection polymers, straw waddles, and/or jute mesh should also be considered to limit the amount of erosion on slopes subject to erosion until landscaping and other permanent erosion protection measures are fully in place.

At the time of this report, specific grading plans for the driveway were not available for our review; therefore, we recommend that the final driveway grading plans be reviewed by IGES to assess compliance with the following generalized recommendations and to assess local slope stability.

6.9 SOIL CORROSION POTENTIAL

Laboratory test results indicate that near surface native soils had a sulfate content of 71 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. Conventional Type I/II Portland cement may be used for all concrete in contact with site soils.

To evaluate the corrosion potential of ferrous metal in contact with onsite native soil, a 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 773 OHM-cm, a soluble chloride content of 16.5 ppm, and a pH of 6.2. Based on these results, 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.

6.10 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 6.10
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 non-woven filter fabric between the native soils and the road base, such as the Mirafi N160 or an IGES-approved equivalent.

6.11 GEOLOGIC HAZARD ASSESSMENT

Based upon the geologic reconnaissance of the property and the subsurface conditions observed in the exploration trenches, adverse geologic conditions are not anticipated to be present within the proposed building site. Given the geologic evidence discussed herein, the following conclusions are made:

1. The landslide/mass movement hazard for the proposed building site is considered to be low to moderate, though the landslide hazard is considered to be moderate to high for the rest of the property. Though no evidence of active movement was observed in the subsurface, the Norwood Formation is a known landslide-prone unit, and the clayey colluvium/highly weathered bedrock that overlies it has the potential to move under the right conditions. 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 proposed building site. The driveway, which crosses both drainages, may be subject to occasional flooding events; accordingly, the flooding risk for the driveway is considered to be moderate.

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

4. An established topsoil that is generally 1.5 to 2 feet thick, shallow Norwood Formation bedrock with a consistent dip to mapped in-place bedrock, and the general absence of shear or other landslide features across the proposed building site are indicative that the site 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 Formation, 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. Surficial soils overlying the Norwood Formation bedrock are comprised largely of soils classifying as fat clay; these soils have the potential to move under the right conditions, particularly under increased moisture conditions. Therefore, it is recommended that this material be over-excavated where identified below foundations and replaced with structural fill (if necessary). It should be noted that if any part of the structure is supported by structural fill, the entire structure must be supported on a minimum of 2 feet of structural fill, such that the home is supported on a relatively uniform fill blanket. An IGES engineering geologist should observe the foundation excavation to assess whether potentially adverse earth material has been adequately removed.

3. To reduce the risk of damage to the driveway due to flooding, the driveway should be constructed over culverts where it crosses the drainages, with the appropriate culvert sizing as designed by the Civil Engineer.

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

6.12 CONSTRUCTION CONSIDERATIONS

6.12.1 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 Formation), and therefore expansive soils are not expected to significantly 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.

6.12.2 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.

6.12.3 Grading Plan Review

The final grading plan, particularly the planned engineered cut slopes for the driveway, should be reviewed by IGES to assess compliance with the recommendations presented in this report. Cut slopes above the driveway should not be steeper than 2H:1V. Additional recommendations are presented in Section 6.8.

7.0 CLOSURE

7.1 LIMITATIONS

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

The information contained in this report is based on limited field testing and understanding of the project. The subsurface data used in the preparation of this report were obtained largely from the explorations made specifically for the proposed building site of the Toliver Property project. It is very likely that variations in the soil, rock, and groundwater conditions exist between and beyond the points explored. The nature and extent of the variations may not be evident until construction occurs and additional explorations are completed. If any conditions are encountered at this site that are different from those described in this report, IGES must be immediately notified so that we may make any necessary revisions to recommendations presented in this report. In addition, if the scope of the proposed construction or grading changes from those described in this report, our firm must also be notified.

This report was prepared for our client's exclusive use on the project identified in the foregoing. Use of the data, recommendations or design information contained herein for any other project or development of the site not as specifically described in this report is at the user's sole risk and without the approval of IGES, Inc. It is the client's responsibility to see that all parties to the project including the designer, contractor, subcontractors, etc. are made aware of this report in its entirety. The use of information contained in this report for bidding purposes should be done at the contractor's option and risk.

We recommend that IGES be retained to review the final design plans, grading plans and specifications to determine if our engineering recommendations have been properly incorporated in the project development documents. We also recommend that IGES be retained to evaluate construction performance and other geotechnical aspects of the project as construction initiates and progresses through its completion.

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

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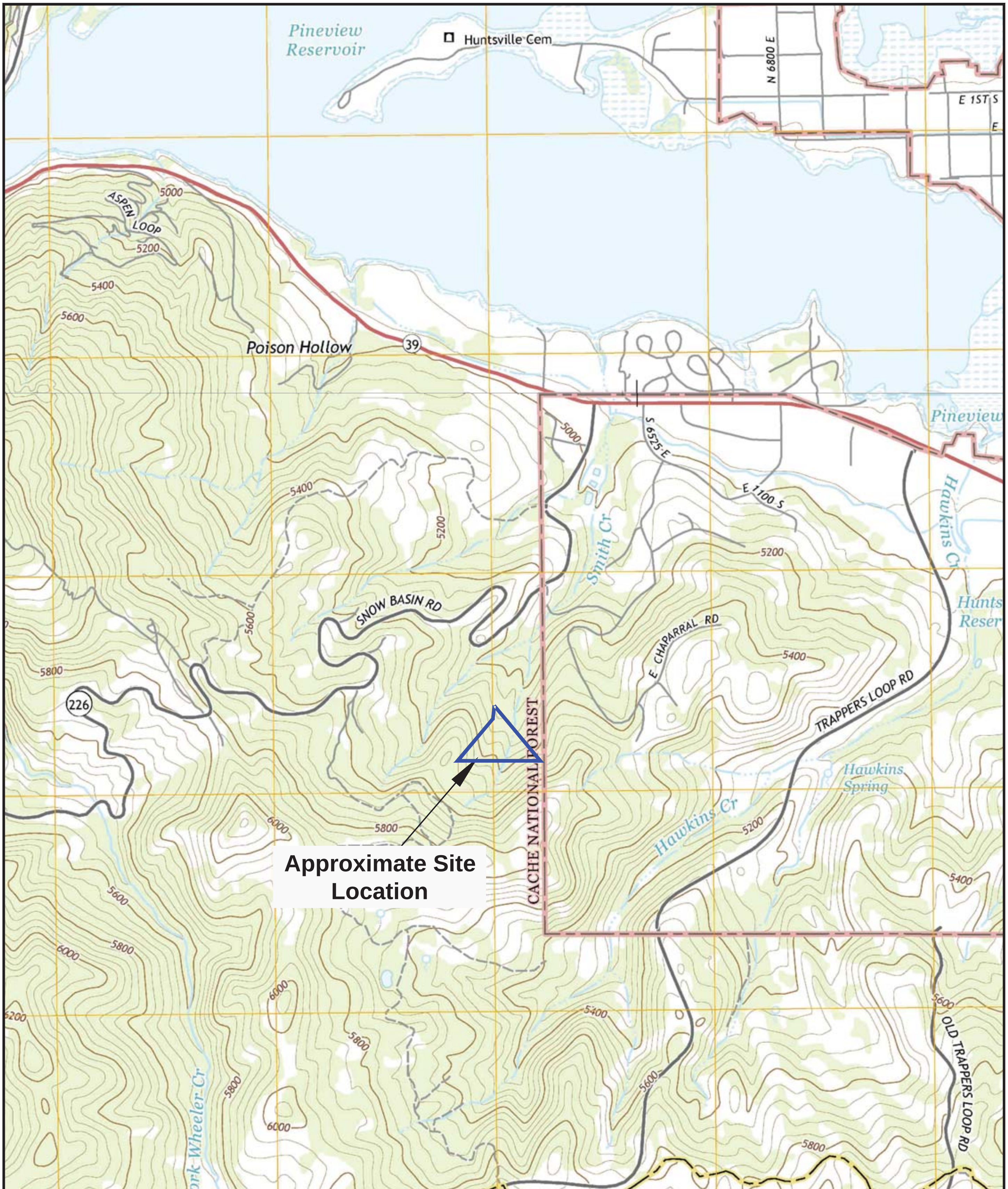
AERIAL PHOTOGRAPHS

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

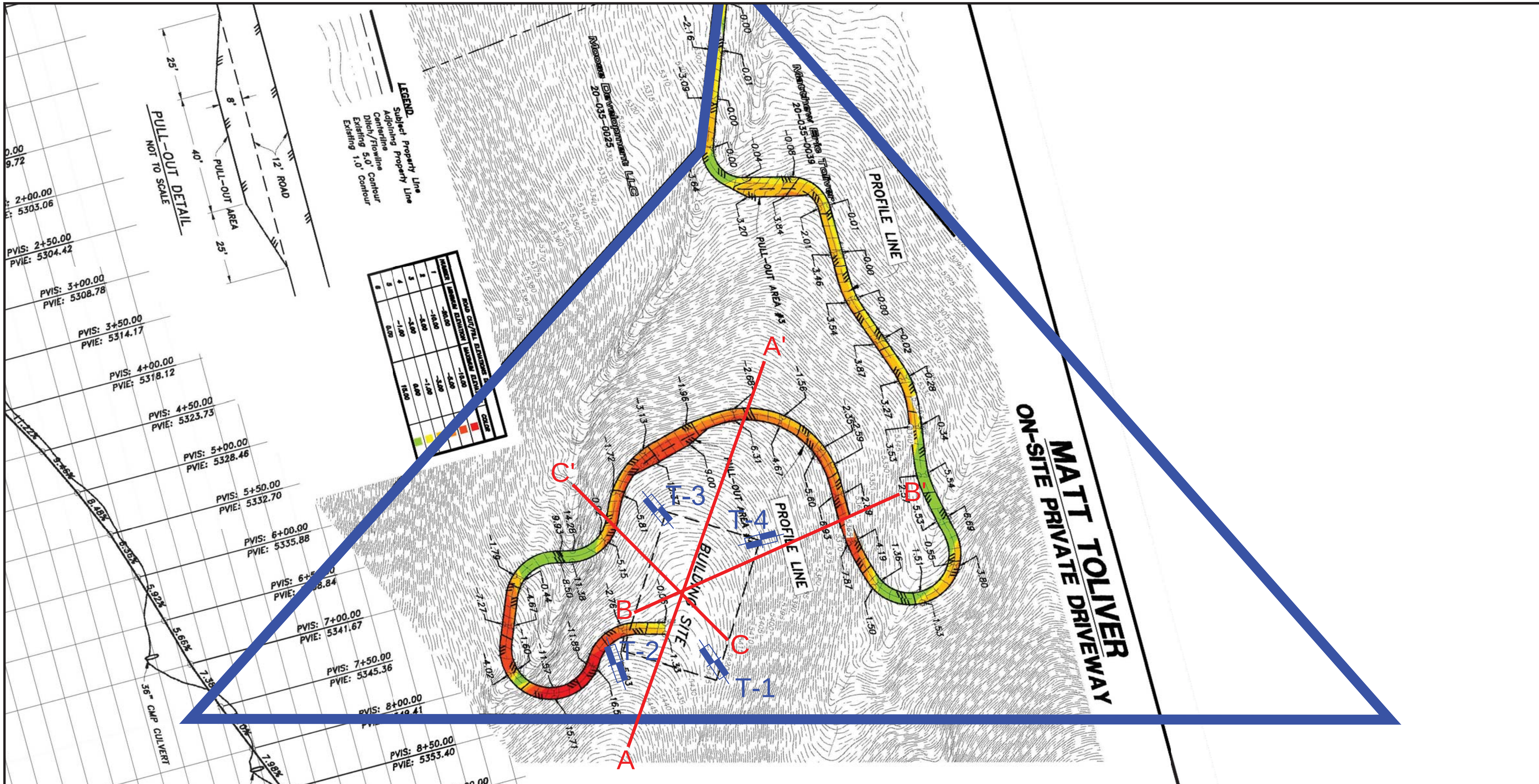
MATTHEW TOLIVER PROPERTY

GEOTECHNICAL AND GEOLOGIC HAZARDS INVESTIGATION

OLD SNOW BASIN ROAD

WEBER COUNTY, UTAH

DATE: 06/30/2017	SCALE: 1"=2,000'	IGES
PROJECT:02489-001		



LEGEND:

T-4 — **Trench Location**

B — B' — **Cross-Section Line**

Base map by Hansen & Associates, Inc., titled "Plan and Profile for Matt Toliver" dated 12/12/16.

N

0 100 200 300
FEET

FIGURE A-2

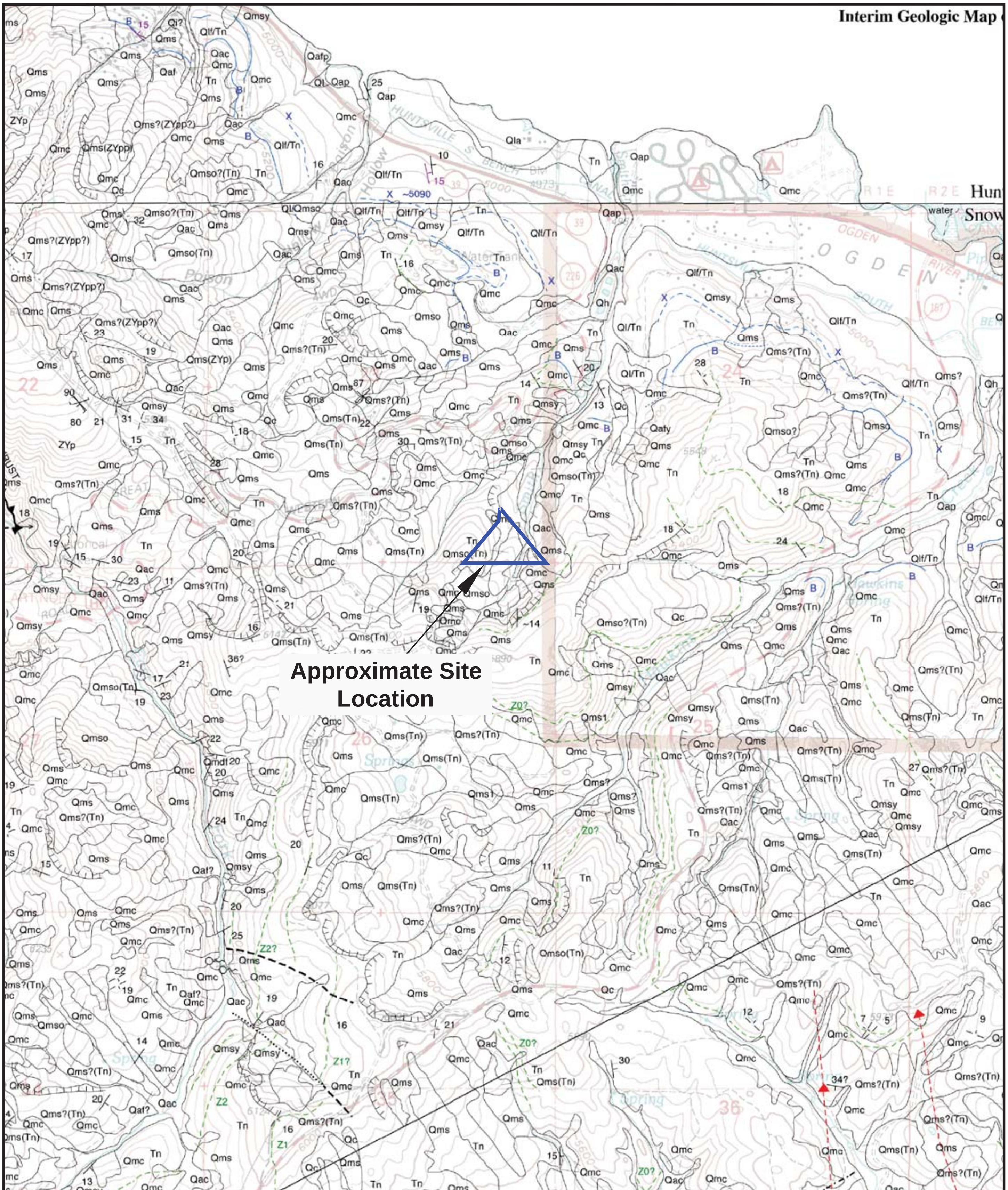
GEOTECHNICAL MAP

MATTHEW TOLIVER PROPERTY

GEOTECHNICAL AND GEOLOGIC HAZARDS INVESTIGATION

OLD SNOW BASIN ROAD
WEBER COUNTY, UTAH

DATE: 07/17/2017	SCALE: 1"=100'	IGES
PROJECT:02489-001		



BASE MAP:

UGS 7.5-Minute Geologic Quadrangle Map
Interim Geologic Map of the Snow Basin and Part of the Huntsville Quadrangle
OFR-638, King, Yonkee, and Coogan (2008)

QUADRANGLE LOCATION

N

0 1000 2000 3000

FEET

FIGURE A-3a

REGIONAL GEOLOGY MAP 1

MATTHEW TOLIVER PROPERTY

GEOTECHNICAL AND GEOLOGIC HAZARDS INVESTIGATION

OLD SNOW BASIN ROAD

WEBER COUNTY, UTAH

DATE: 07/13/2017 | SCALE: 1"=2,000'

PROJECT:02489-001

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, mostl 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_).

Mixed Deposits

Qac Alluvium and colluvium (Holocene and Pleistocene) - Includes stream and fan alluvium, 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-638, King, Yonkee, and Coogan (2008)

QUADRANGLE LOCATION

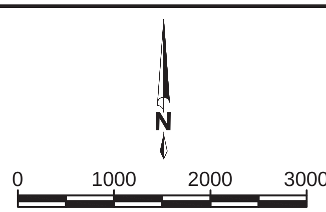


FIGURE A-3b

REGIONAL GEOLOGY MAP 1

MATTHEW TOLIVER PROPERTY

GEOTECHNICAL AND GEOLOGIC HAZARDS INVESTIGATION

OLD SNOW BASIN ROAD

WEBER COUNTY, UTAH

DATE: 07/13/2017 | SCALE: 1"=2,000' | IGES

PROJECT:02489-001

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

Strike and dip of bedding (red from Pavlis, 1979; purple from Sorensen and Crittenden, 1979)

43 Upright

78 Overturned

Vertical

Horizontal

50 750t Determined photogrammetrically, upright on left, overturned on right

Approximate (queried where dip uncertain)

60 Strike and dip of high-grade foliation (upright)

45 Strike and dip of low-grade cleavage (upright)

Fractures (near vertical)

Sinkhole

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-638, King, Yonkee, and Coogan (2008)

QUADRANGLE LOCATION

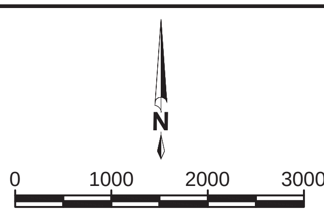


FIGURE A-3c

REGIONAL GEOLOGY MAP 1

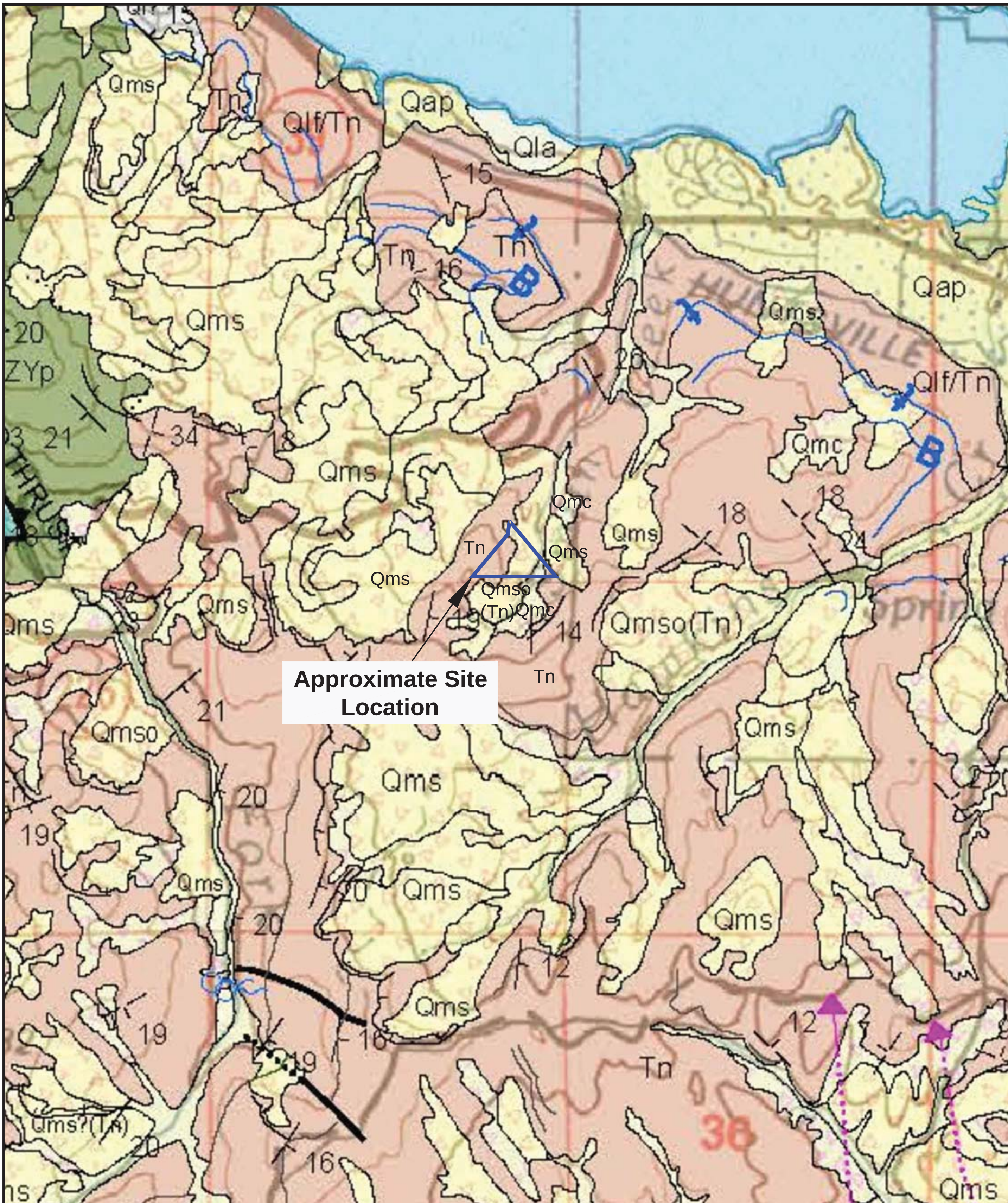
MATTHEW TOLIVER PROPERTY

GEOTECHNICAL AND GEOLOGIC HAZARDS INVESTIGATION

OLD SNOW BASIN ROAD

WEBER COUNTY, UTAH

DATE: 07/13/2017	SCALE: 1"=2,000'	IGES
PROJECT:02489-001		



BASE MAP:

UGS Ogden 30' x 60' Geologic Quadrangle Map OFR-635DM, Coogan and King (2016)

UTAH

QUADRANGLE LOCATION

N

0 1000 2000 3000

FEET

FIGURE A-4a

REGIONAL GEOLOGY MAP 2

MATTHEW TOLIVER PROPERTY

GEOTECHNICAL AND GEOLOGIC HAZARDS INVESTIGATION

OLD SNOW BASIN ROAD

WEBER COUNTY, UTAH

DATE: 07/13/2017	SCALE: 1"=2,000'	IGES
PROJECT: 02489-001		

MAP LEGEND

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

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

Qms without a suffix is mapped where the age is uncertain (though likely Holocene and/or late Pleistocene), where portions of slide complexes have different ages but cannot be shown separately at map scale, or where boundaries between slides of different ages are not distinct. Estimated time of emplacement is indicated by relative-age letter suffixes with: Qmsy mapped where landslides deflect streams or failures are in Lake Bonneville deposits, and scarps are variably vegetated; Qmso typically mapped where deposits are "perched" above present drainages, rumpled morphology typical of mass movements has been diminished, and/or younger surficial deposits cover or cut Qmso. Lower perched Qmso deposits are at Qao heights above drainages (95 ka and older) and the higher perched deposits may correlate with high level alluvium (QTa_) (likely older than 780 ka) (see table 1). Suffixes y and o indicate probable Holocene and Pleistocene ages, respectively, with all Qmso likely emplaced before Lake Bonneville transgression. These older deposits are as unstable as other slides, and are easily reactivated with the addition of water, be it irrigation or septic tank drain fields.

Qmso?(Qafoe), Qmso?(QTcg?), Qmso?(Ts), Qmso?(Tcg), Qmso?(Tn), Qmso?(Tf), Qmso?(Xfc)

Block landslide and possible block landslide deposits (Holocene and upper and middle? Pleistocene) – Mapped where nearly intact block is visible in landslide (mostly block slide) with stratal strikes and dips that are different from nearby in-place bedrock; unit involved in landslide shown in parentheses, for example Qms(Tw) and composition depends bedrock unit; rx shown where bedrock unit in block not known or multiple units are in the block, with Zrx shown where the units are Neoproterozoic; see surficial deposits or rock unit in parentheses for descriptions of blocks; thickness highly variable, up to about 20 to 30 feet (6-9 m) for small slides, and cross sections show larger blocks are about 150 feet (45 m) thick. Relative ages are like those for other landslide deposits (Qms, Qmso).

Qms and Qmso queried (Qms?, Qmso?) where bedrock block may be in place, that is stratal strikes and dips in queried block are about the same as nearby in-place bedrock.

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

Tn, Tn? **Norwood Formation (lower Oligocene and upper Eocene)** – Typically light-gray to light-brown altered tuff (claystone), altered tuffaceous siltstone and sandstone, and conglomerate; unaltered tuff, present in type section south of Morgan, is rare; locally colored light shades of red and green; variable calcareous cement and zeolitization; involved in numerous landslides of various sizes; estimate 2000-foot (600 m) thick in exposures on west side of Ogden Valley (based on bedding dip, outcrop width, and topography). Norwood Formation queried where poor exposures may actually be surficial deposits. For detailed Norwood Formation information see description under heading "Sub-Willard Thrust - Ogden Canyon Area" since most of this unit is in and near Morgan Valley and covers the Willard thrust, Ogden Canyon, and Durst Mountain areas.

BASE MAP:

UGS Ogden 30' x 60' Geologic Quadrangle Map
OFR-635DM, Coogan and King (2016)

N

0 1000 2000 3000
FEET

FIGURE A-4b

REGIONAL GEOLOGY MAP 2

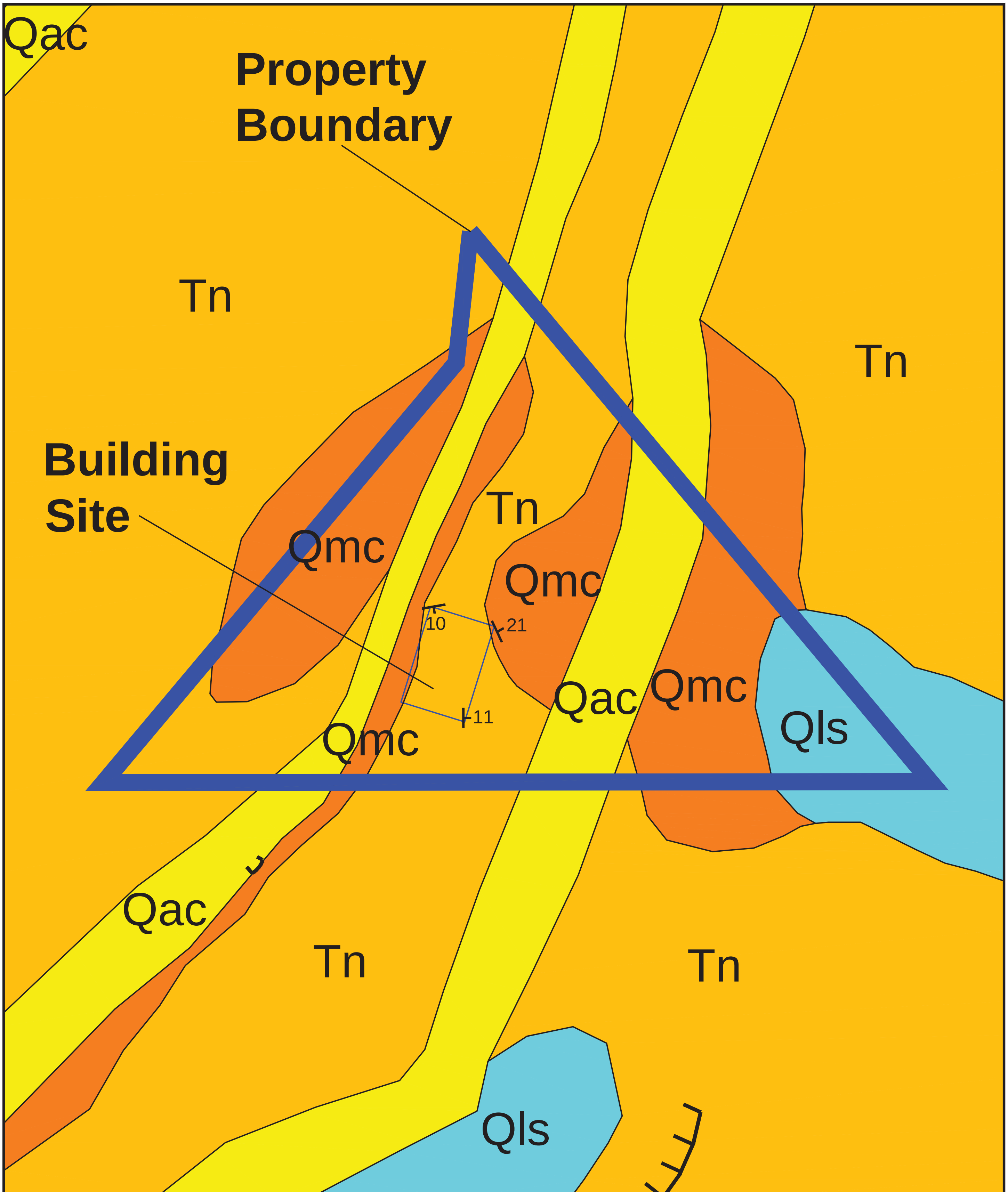
MATTHEW TOLIVER PROPERTY

GEOTECHNICAL AND GEOLOGIC HAZARDS INVESTIGATION

OLD SNOW BASIN ROAD

WEBER COUNTY, UTAH

DATE: 07/13/2017	SCALE: 1"=2,000'	IGES
PROJECT:02489-001		



LEGEND:

- Qac: Alluvium and colluvium
- Qmc: Undifferentiated colluvium and mass-movement deposits
- Qls: Landslide deposits
- Tn: Norwood Formation
- Landslide Headscarp
- 11 Bedrock Orientation

N

0 100 200 300
FEET

FIGURE A-5

LOCAL GEOLOGY MAP

MATTHEW TOLIVER PROPERTY

GEOTECHNICAL AND GEOLOGIC HAZARDS INVESTIGATION

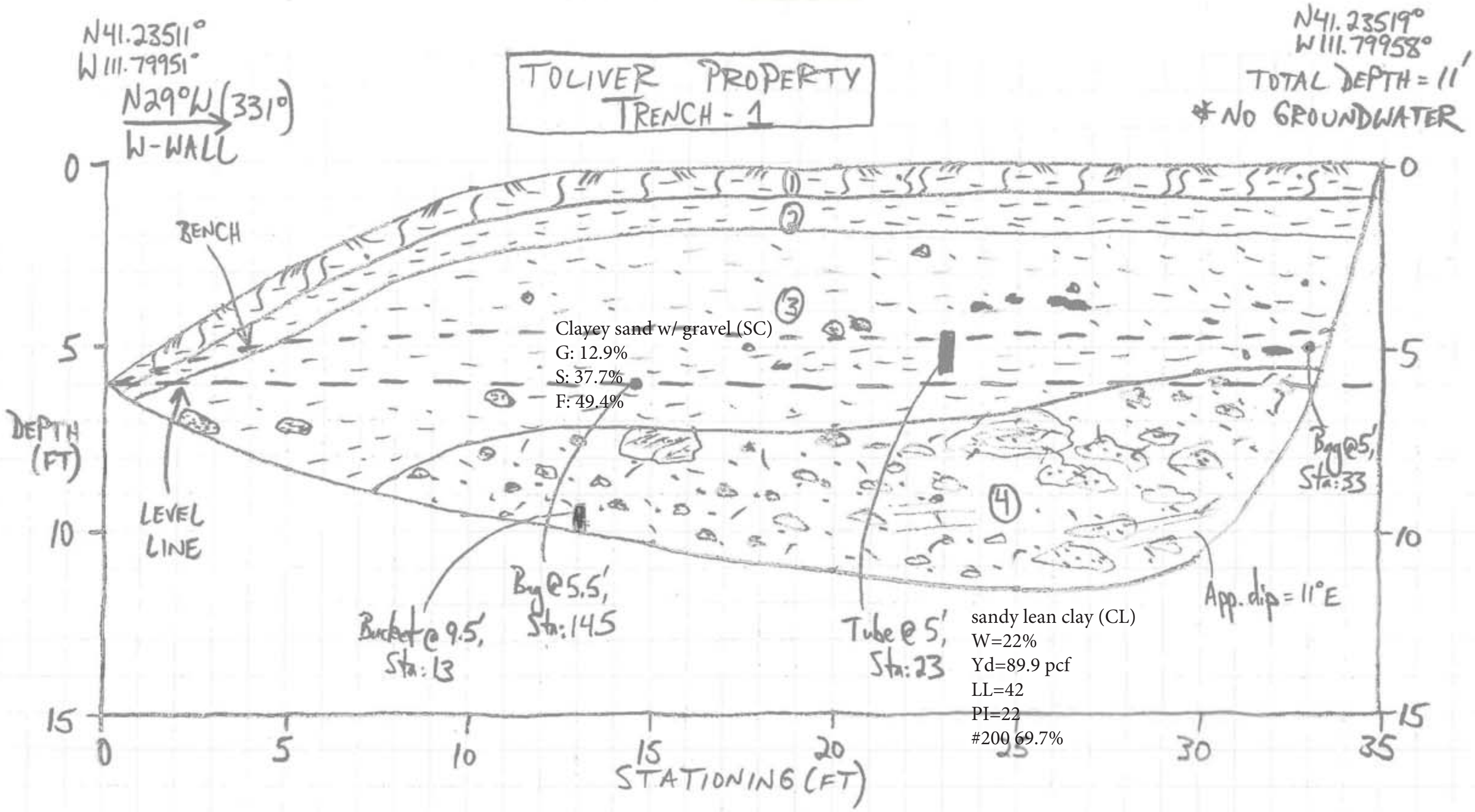
OLD SNOW BASIN ROAD

WEBER COUNTY, UTAH

DATE: 07/17/2017 | SCALE: 1"=200' | IGES

PROJECT:02489–001

Project No. 02489-001
Date 6-6-17 by PEJ
Ckd by ______ on ______



LITHOLOGIC UNIT DESCRIPTIONS:

1. A Soil Horizon: ~6"-1' thick; brownish black (5YR $\frac{2}{1}$) fat CLAY (CH), medium stiff to stiff, moist, moderate plasticity, massive; rare (<1%) gravel, all tuff clasts up to 5" diameter; abundant plant and tree roots; sharp, largely planar basal contact.

2. B Soil Horizon: ~1' thick; dark yellowish brown (10YR $\frac{4}{2}$) to brownish gray (5YR $\frac{4}{1}$) fat CLAY (CH), stiff to medium stiff, moist, high plasticity, massive; rare (~1%) gravel, all light gray (N7), platy, subangular to angular, finely-bedded tuffaceous sandstone up up to 5" diameter; common to abundant plant and tree roots; minor fat clay sheen observed in places; gradational, irregular basal contact.

3. Colluvium: ~5-5.5' thick; moderate yellowish brown (10YR $\frac{5}{4}$) to dark yellowish brown (10YR $\frac{4}{2}$) lean CLAY (CL), stiff to medium stiff, slightly moist, moderate plasticity, massive; gravel and larger sized clasts comprise <5% of unit; clasts consist entirely of light gray (N7) to medium dark gray (N4) tuffaceous sandstone gradational to sandy tuff, up to 6" in diameter; clasts are subangular to angular, and predominantly quartz-rich, though minor (<5%) biotite and lithics; unit contains common topsoil inclusions (possibly burrows/krotovina), though some irregular); occasional lenses of fat clay with a glassy sheen, though these are not continuous (<1" in length) and rarely exhibit slickensides; common to abundant plant and tree roots; basal contact is oxidized to dark yellowish orange (10YR $\frac{6}{6}$); basal contact is sharp, though gradational in places, and slightly wavy; no shear surfaces observed along basal contact.

4. Weathered Norwood Formation: >5.5' thick; medium light gray (N6) to light brown (5YR $\frac{6}{4}$) partially weathered Norwood Formation bedrock; consists largely of blocks of tuffaceous sandstone, moderately hard, finely bedded, with common dark yellowish orange (10YR $\frac{6}{6}$) oxidized bands along bedding planes and also liesegang bands; sandstone is quartzose, very fine-grained, with a weak calcium carbonate and iron oxide cement, subangular, well sorted; fine laminations, some cross-bedded; in places, highly disaggregated to clayey SAND (SC), medium dense, slightly moist, low plasticity fines; common plant and tree roots, especially along fracture surfaces; bedrock exhibits blocky jointing with ~2-6" spacing; unit appears in situ, though some blocks may have rotated during the weathering process.

FIGURE A-6
TR-1 LOG

MATTHEW TOLIVER PROPERTY

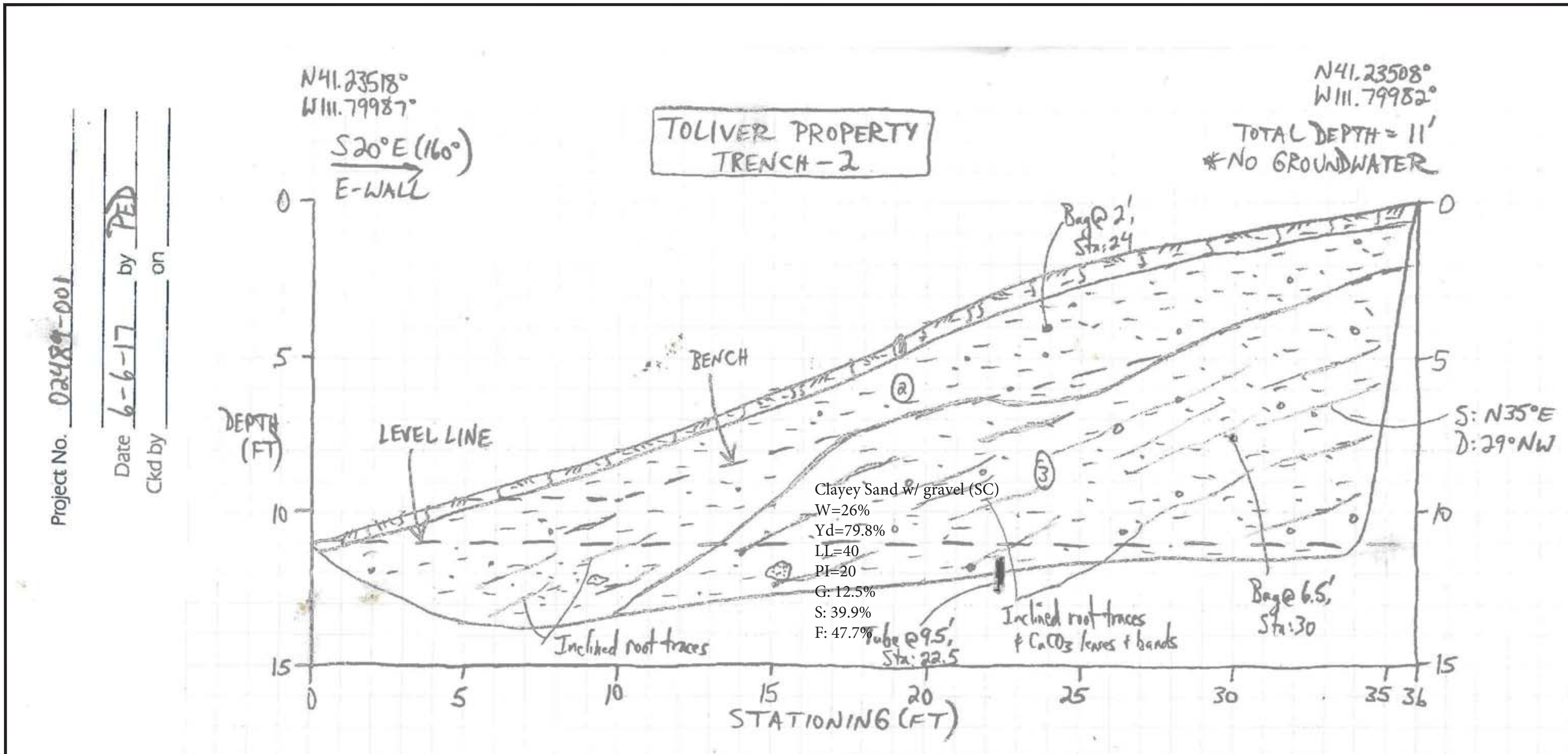
GEOTECHNICAL AND GEOLOGIC HAZARDS INVESTIGATION

OLD SNOW BASIN ROAD

WEBER COUNTY, UTAH

DATE: 07/13/2017	SCALE: 1"=5'
PROJECT: 02489-001	

IGES



LITHOLOGIC UNIT DESCRIPTIONS:

1. A Soil Horizon: ~4-6" thick; brownish black (5YR $\frac{2}{1}$) to black (N1) fat CLAY (CH), loose to medium stiff, moist, moderate plasticity, massive; rare (<1%) gravel, all tuff clasts up to 5" diameter; abundant plant and tree roots; thins downslope; sharp, irregular basal contact.

2. B Soil Horizon: ~2-5' thick; moderate yellowish brown (10YR $\frac{5}{4}$) to light brownish gray (5YR $\frac{6}{1}$) to dark yellowish orange (10YR $\frac{6}{6}$) fat CLAY with gravel (CH), stiff to medium stiff, moist, moderate to high plasticity, massive, though small blocky texture throughout; gravel and larger sized clasts comprise ~5-10% of unit; clasts are entirely angular light gray (N7) to light brownish gray very fine-grained, weathered sandy tuff and tuffaceous sandstone, up to 2" in diameter, though mode size ~2-5 mm; occasionally mottled with calcium carbonate; common plant and tree roots and some krotovina; some plant roots and calcium carbonate inclined parallel to modern slope; sandy in places; minor oxidation throughout to dark yellowish orange; thickens notably downslope; gradational, highly irregular basal contact.

3. Colluvium: ~>8' thick; light gray (N7) to medium light gray (N6) to light brown (5YR $\frac{6}{4}$), though highly mottled with white (N9) calcium carbonate and dark yellowish orange iron oxide; sandy lean CLAY with gravel (CL) gradational to clayey SAND with gravel (SC), stiff, slightly moist, moderate plasticity, medium bedded; gravel and larger sized clasts comprise ~5-10% of unit; clasts consist entirely of highly oxidized dark yellowish orange (10YR $\frac{6}{6}$) tuffaceous sandstone up to 8" diameter, though mode size <1"; abundant calcium carbonate throughout, especially along inclined bedding planes which parallel modern topography; more brownish in color where sandier; calcium carbonate lenses may have a tuffaceous component; no fat clay sheen or slickensides observed; common to abundant plant and tree roots, some inclined parallel to modern topography; becomes denser, sandier, and more clast-rich with depth, possibly grading into weathered bedrock.

FIGURE A-7
TR-2 LOG

MATTHEW TOLIVER PROPERTY

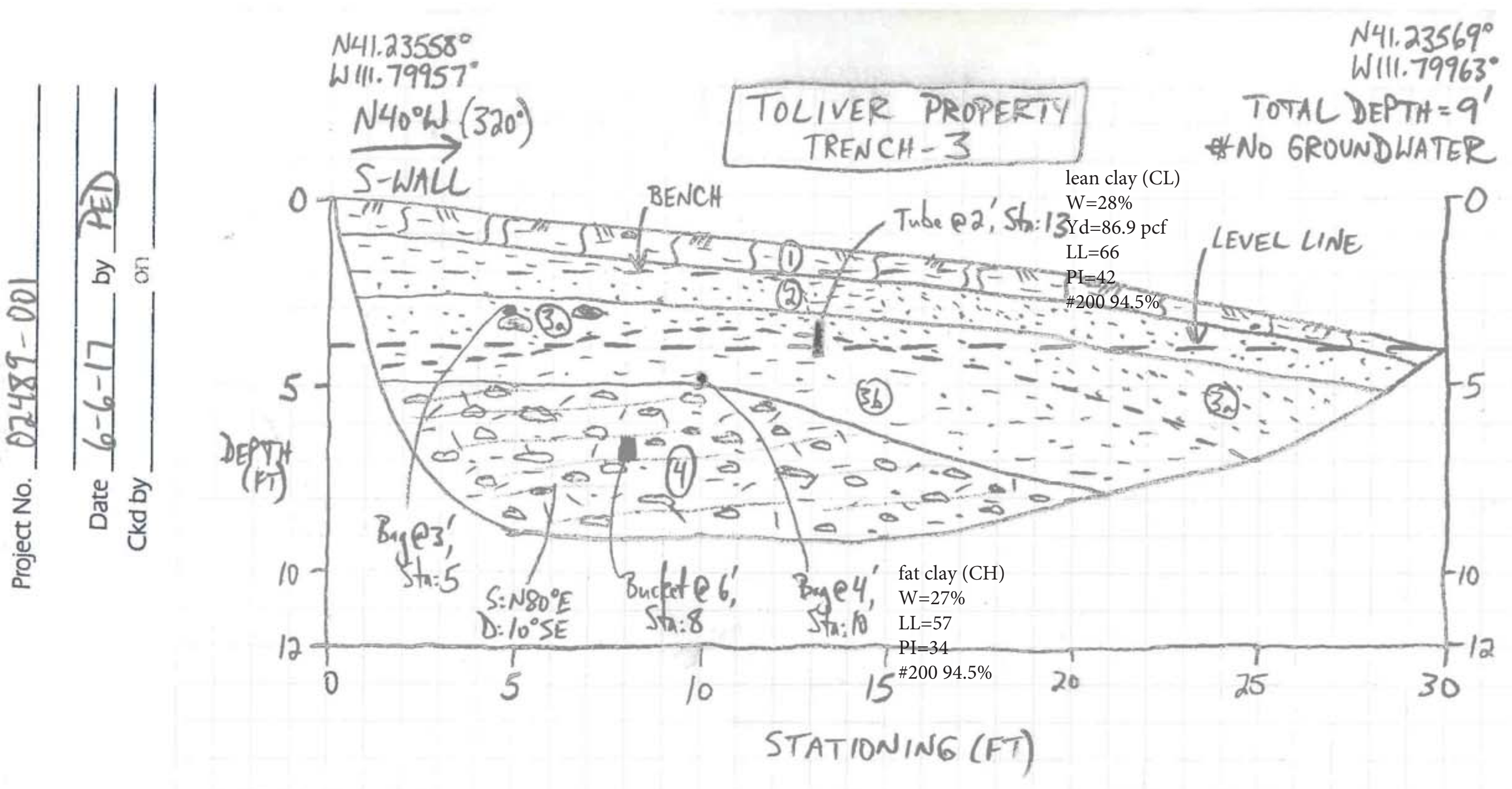
GEOTECHNICAL AND GEOLOGIC HAZARDS INVESTIGATION

OLD SNOW BASIN ROAD

WEBER COUNTY, UTAH

DATE: 07/13/2017	SCALE:
PROJECT: 02489-001	1"=5'

IGES



LITHOLOGIC UNIT DESCRIPTIONS:

1. A Soil Horizon: ~6"-1' thick; brownish black (5YR $\frac{2}{1}$) to black (N1) sandy fat CLAY (CH), medium stiff, moist, moderate plasticity, massive; gravel and larger sized clasts comprise <5% of unit, all platy tuffaceous sandstone clasts aligned along modern slope and up to 4" in diameter; abundant plant and tree roots; sharp, irregular basal contact.

2. B Soil Horizon: ~1-1.5' thick; dark yellowish brown (10YR $\frac{4}{2}$) to brownish gray (5YR $\frac{4}{1}$) clayey SAND (SC) gradational to sandy fat CLAY (CH), medium dense, moist, high plasticity, massive; rare (<1%) gravel, all tuffaceous sandstone as above; clayey above the bench and in a small lens between Stations 13 and 15; clay exhibits glassy sheen in places, though no observed slickensides; sand is fine-grained to medium-grained, and largely devoid of clay; abundant plant and tree roots; gradational, irregular basal contact.

3. Highly Weathered Bedrock: ~2-4+' thick; unit is comprised to two subunits, consisting of decomposed sandstone and tuff interbeds of the Norwood Formation:

3a. Sand Seam: ~1-2.5' thick; dark yellowish brown (10YR $\frac{4}{2}$) clayey SAND (SC), medium dense, moist, high plasticity fines, finely bedded in places where relict bedrock blocks are still in place; in those areas, sand is largely devoid of clay; sand is fine-grained to medium grained, with a weak iron oxide cement; minor oxidation; abundant plant and tree roots, especially along laminae; sharp, irregular basal contact.

3b. Clay Seam: ~1-2.5' thick; moderate reddish brown (10R $\frac{4}{6}$) fat CLAY (CH), stiff to very stiff, moist, high plasticity, finely bedded with relict bedding; highly blocky texture, with $\frac{1}{2}$-1" blocks; common glassy sheen, though no slickensides observed; sandy in places, especially downslope where subunit thickens; common plant and tree roots along edges of blocks; gradational, largely planar basal contact.

4. Partially Weathered Bedrock: >4' thick; brownish gray (5YR $\frac{4}{1}$) to medium gray (N5) partially weathered Norwood Formation bedrock; consists of blocks of tuffaceous sandstone and sandy tuff, moderately hard, poorly competent, medium bedded, blocky; disaggregates to sandy fat CLAY (CH) and well graded gravelly SAND (SW), depending on parent material; very dense, slightly moist, moderate to high plasticity fines, relict medium to fine bedding; abundant roots along blocky fractures; common calcium carbonate fracture infilling; minor to moderate oxidation in places.

FIGURE A-8
TR-3 LOG

MATTHEW TOLIVER PROPERTY

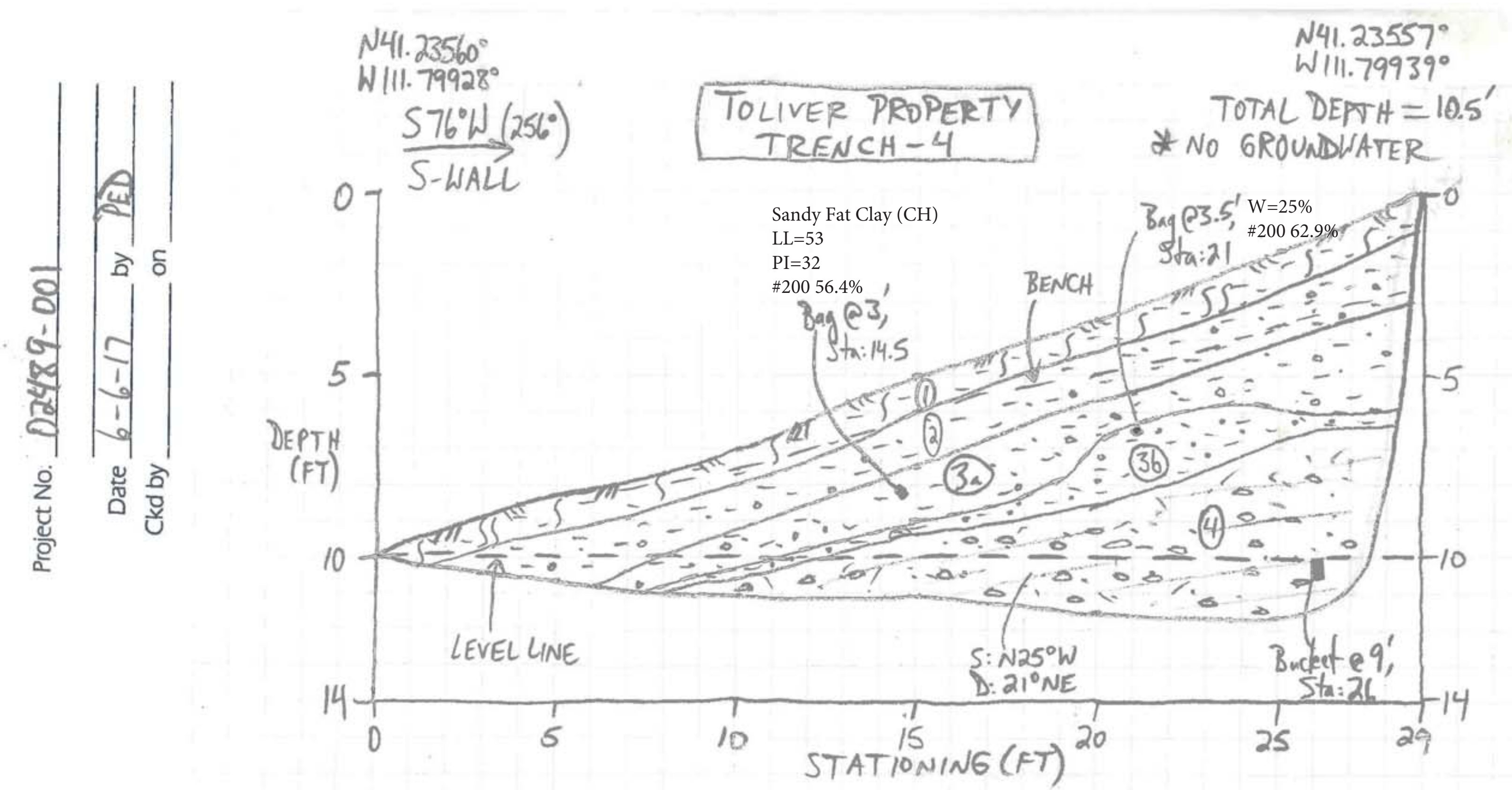
GEOTECHNICAL AND GEOLOGIC HAZARDS INVESTIGATION

OLD SNOW BASIN ROAD

WEBER COUNTY, UTAH

DATE: 07/13/2017	SCALE:
PROJECT: 02489-001	1"=5'

IGES



LITHOLOGIC UNIT DESCRIPTIONS:

1. A Soil Horizon: ~1' thick; brownish black (5YR $\frac{2}{1}$) fat CLAY (CH), medium stiff to stiff, moist, high plasticity, massive; rare (<1%) gravel, all tuff clasts up to 5" diameter; abundant plant and tree roots; sharp, irregular basal contact.

2. B Soil Horizon: ~1.5' thick; dark yellowish brown (10YR $\frac{4}{2}$) to brownish gray (5YR $\frac{4}{1}$) fat CLAY with gravel (CH), stiff, moist, high plasticity, massive; gravel and larger sized clasts comprise ~5-10% of unit; clasts are comprised of a combination of light gray (N7) tuffaceous sandstone and white (N9) to light brownish gray (5YR $\frac{6}{1}$) mudstone; all clasts are angular and up to 1.5' in diameter, though mode size ~1-3"; mudstone clasts not seen in any of the other trenches; contains irregular inclusions of topsoil, possibly krotovina; minor sand component; common to abundant plant and tree roots; gradational, irregular basal contact.

3. Shallow Landslide?: ~3' thick; possibly colluvium; comprised of two subunits:

3a. Slide: ~1-1.5' thick; dark yellowish brown (10YR $\frac{4}{2}$) to moderate yellowish brown (10YR $\frac{5}{4}$) sandy fat CLAY with gravel (CH), stiff, moist, moderate to high plasticity, massive; gravel and larger sized clasts comprise ~5-10% of unit; clasts as seen in B-Horizon, with roughly equal proportions of tuffaceous sandstone and mudstone; common pinholes up to 1 mm diameter; occasional plant and tree roots; sharp, irregular basal contact.

3b. Slide Plane?: ~1-1.5' thick; dark reddish brown (10R $\frac{3}{4}$) fat CLAY with gravel (CH), stiff to very stiff, moist, high plasticity, massive; gravel and larger sized clasts comprise ~20-25% of unit; clasts all tuffaceous sandstone and mudstone as above, all angular, and up to 4" in diameter, though mode size ~5 mm; common 1 mm diameter pinholes and glassy sheen, though no slickensides observed; occasional plant and tree roots; sharp, irregular basal contact.

4. Weathered Norwood Formation: >5' thick; light gray (N7), finely bedded tuffaceous sandstone bedrock, quartzose, fine-grained to very fine-grained moderately hard, blocky weathering; abundant white (N9) calcium carbonate flour throughout, in matrix and along laminations; minor oxidation and common root traces along laminations; disaggregates to clayey SAND with gravel (SC), dense to medium dense, slightly moist, moderate plasticity fines, finely bedded; silty in part; occasional plant and tree roots; upper contact is gravelly, moderate reddish brown (10R $\frac{4}{6}$), and oxidized.

FIGURE A-9
TR-4 LOG

MATTHEW TOLIVER PROPERTY

GEOTECHNICAL AND GEOLOGIC HAZARDS INVESTIGATION

OLD SNOW BASIN ROAD

WEBER COUNTY, UTAH

DATE: 07/14/2017
PROJECT: 02489-001
SCALE: 1"=5'
IGES

UNIFIED SOIL CLASSIFICATION SYSTEM

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

LOG KEY SYMBOLS

BORING SAMPLE LOCATION

TEST-PIT SAMPLE LOCATION

WATER LEVEL (level after completion)

WATER LEVEL (level where first encountered)

CEMENTATION

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

OTHER TESTS KEY

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

MODIFIERS

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

MOISTURE CONTENT

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

GENERAL NOTES

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

STRATIFICATION

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

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

APPARENT / RELATIVE DENSITY - COARSE-GRAINED SOIL

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

CONSISTENCY - FINE-GRAINED SOIL

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. **02489-001**

Engr. **DAG**

Drafted By **DAG**

Date **July 2017**

IGES

Intermountain Geo-Environmental Services, Inc.

Figure A-10

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. 02489-001
Engr. DAG
Drafted By DAG
Date July 2017

IGES
Intermountain Geo-Environmental Services, Inc.

Figure A-11

APPENDIX B

Water Content and Unit Weight of Soil

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

Project: **Toliver Property**
No: **02489-001**
Location: Huntsville, UT
Date: 6/30/2017
By: BSS/JDF

Sample Info.	Boring No.	T-3	T-3	T-4						
	Sample:	Sta 13	Sta 10	Sta 21						
	Depth:	2.0'	4.0'	3.5'						
Unit Weight Info.	Sample height, H (in)	4.544								
	Sample diameter, D (in)	2.410								
	Sample volume, V (ft^3)	0.0120								
	Mass rings + wet soil (g)	606.38								
	Mass rings/tare (g)	0.00								
	Moist soil, Ws (g)	606.38								
	Moist unit wt., γ_m (pcf)	111.44								
Water Content	Wet soil + tare (g)	474.36	301.01	454.65						
	Dry soil + tare (g)	397.79	264.22	394.71						
	Tare (g)	126.16	127.74	152.68						
Water Content, w (%)		**28.2**	**27.0**	**24.8**						
Dry Unit Wt., γ_d (pcf)		**86.9**								

Entered by:___________

Reviewed:___________

Liquid Limit, Plastic Limit, and Plasticity Index of Soils

(ASTM D4318)

Project: Toliver Property
No: 02489-001
Location: Huntsville, UT
Date: 7/1/2017
By: BRR

Boring No.: T-1
Sample: Sta 23
Depth: 5.0'
Description: Brown lean clay

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

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

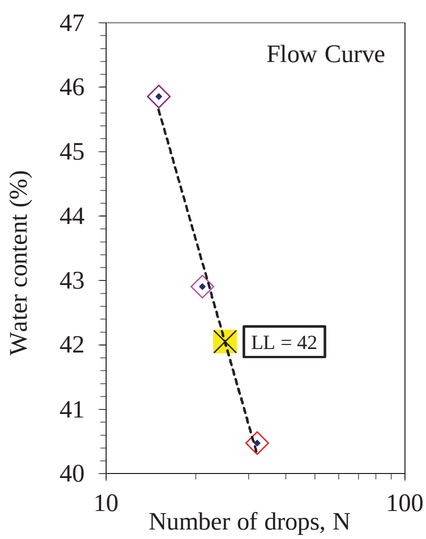
Plastic Limit

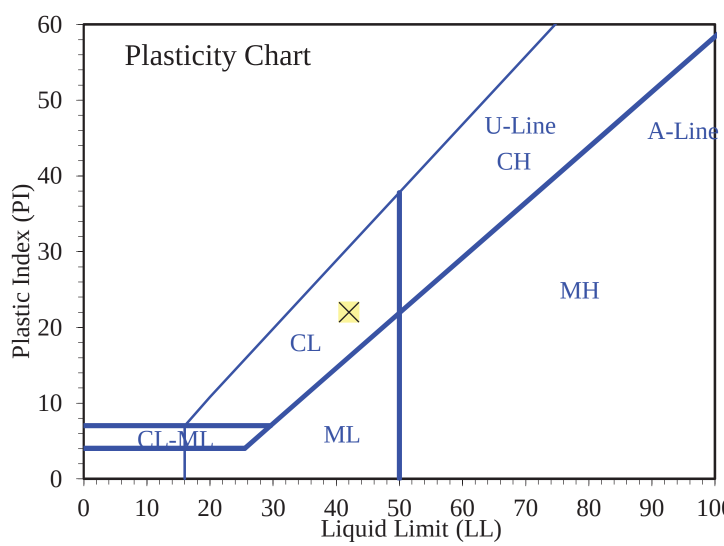
Determination No	1	2				
Wet Soil + Tare (g)	29.39	28.02				
Dry Soil + Tare (g)	28.21	26.99				
Water Loss (g)	1.18	1.03				
Tare (g)	22.16	21.80				
Dry Soil (g)	6.05	5.19				
Water Content, w (%)	19.50	19.85				

Liquid Limit

Determination No	1	2	3			
Number of Drops, N	32	21	15			
Wet Soil + Tare (g)	30.35	30.24	29.82			
Dry Soil + Tare (g)	27.95	27.73	27.33			
Water Loss (g)	2.40	2.51	2.49			
Tare (g)	22.02	21.88	21.90			
Dry Soil (g)	5.93	5.85	5.43			
Water Content, w (%)	40.47	42.91	45.86			
One-Point LL (%)		42				

Liquid Limit, LL (%)	**42**
Plastic Limit, PL (%)	**20**
Plasticity Index, PI (%)	**22**





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Reviewed:____________

Liquid Limit, Plastic Limit, and Plasticity Index of Soils

(ASTM D4318)

Project: Toliver Property
No: 02489-001
Location: Huntsville, UT
Date: 7/1/2017
By: BRR

Boring No.: T-2
Sample: Sta 22.5
Depth: 9.5'
Description: Brown lean clay

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

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

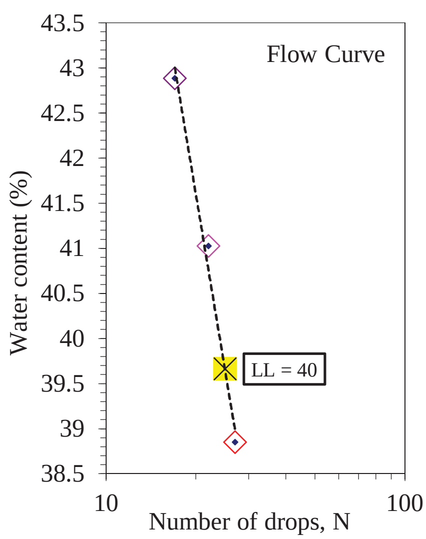
Plastic Limit

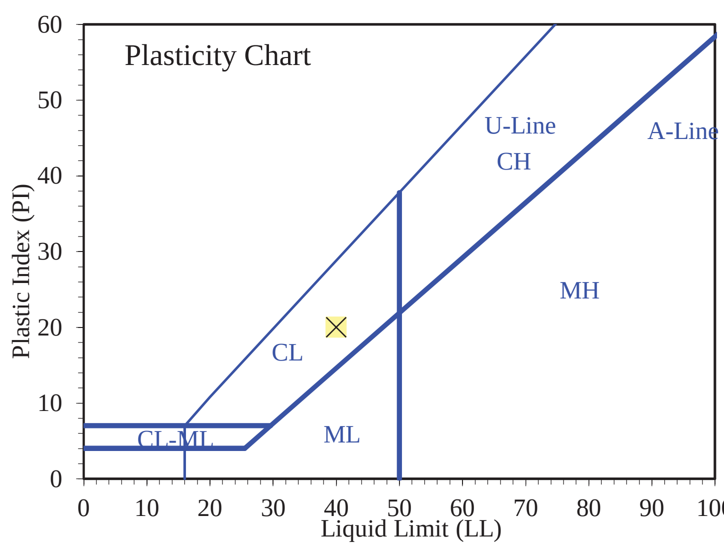
Determination No	1	2				
Wet Soil + Tare (g)	29.03	28.38				
Dry Soil + Tare (g)	27.82	27.20				
Water Loss (g)	1.21	1.18				
Tare (g)	21.88	21.48				
Dry Soil (g)	5.94	5.72				
Water Content, w (%)	20.37	20.63				

Liquid Limit

Determination No	1	2	3			
Number of Drops, N	27	22	17			
Wet Soil + Tare (g)	29.83	29.71	29.69			
Dry Soil + Tare (g)	27.60	27.31	27.40			
Water Loss (g)	2.23	2.40	2.29			
Tare (g)	21.86	21.46	22.06			
Dry Soil (g)	5.74	5.85	5.34			
Water Content, w (%)	38.85	41.03	42.88			
One-Point LL (%)	39	40				

Liquid Limit, LL (%)	**40**
Plastic Limit, PL (%)	**20**
Plasticity Index, PI (%)	**20**





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Reviewed:____________

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Liquid Limit, Plastic Limit, and Plasticity Index of Soils

(ASTM D4318)

Project: Toliver Property
No: 02489-001
Location: Huntsville, UT
Date: 7/1/2017
By: BRR
Grooving tool type: Plastic
Liquid limit device: Mechanical
Rolling method: Hand

Boring No.: T-3
Sample: Sta 13
Depth: 2.0'
Description: Brown fat clay

Preparation method: Wet
Liquid limit test method: Multipoint
Screened over No.40: No
Larger particles removed: Mixed on glass plate
Approximate maximum grain size: No.40
Estimated percent retained on No.40: Not requested
As-received water content (%): 28.2

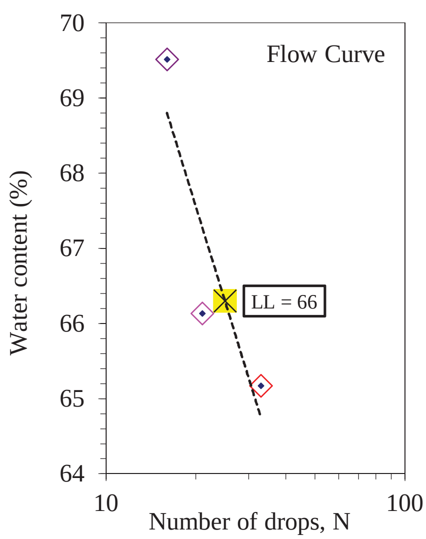
Plastic Limit

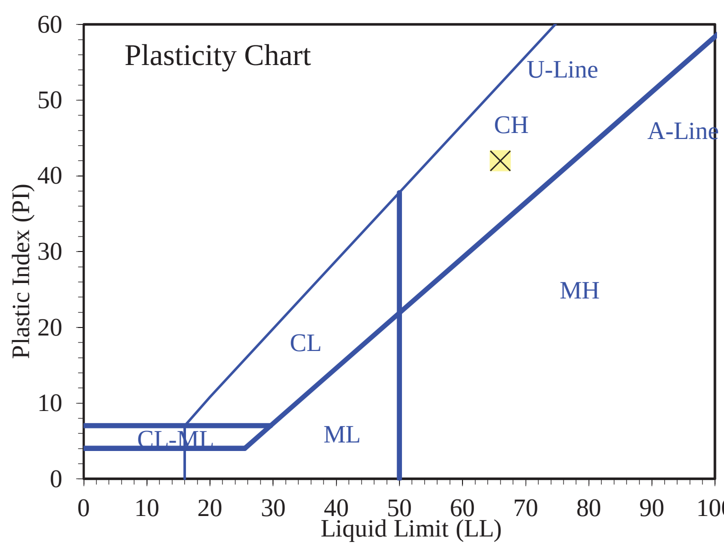
Determination No	1	2				
Wet Soil + Tare (g)	27.54	28.23				
Dry Soil + Tare (g)	26.35	27.04				
Water Loss (g)	1.19	1.19				
Tare (g)	21.32	21.98				
Dry Soil (g)	5.03	5.06				
Water Content, w (%)	23.66	23.52				

Liquid Limit

Determination No	1	2	3			
Number of Drops, N	33	21	16			
Wet Soil + Tare (g)	29.65	31.30	30.81			
Dry Soil + Tare (g)	26.60	27.59	27.39			
Water Loss (g)	3.05	3.71	3.42			
Tare (g)	21.92	21.98	22.47			
Dry Soil (g)	4.68	5.61	4.92			
Water Content, w (%)	65.17	66.13	69.51			
One-Point LL (%)		65				

Liquid Limit, LL (%)	**66**
Plastic Limit, PL (%)	**24**
Plasticity Index, PI (%)	**42**





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Reviewed:____________

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Liquid Limit, Plastic Limit, and Plasticity Index of Soils

(ASTM D4318)

IGES

Project: Toliver Property
No: 02489-001
Location: Huntsville, UT
Date: 7/1/2017
By: BRR

Boring No.: T-3
Sample: Sta 10
Depth: 4.0'
Description: Brown fat clay

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

Preparation method: Air Dry
Liquid limit test method: Multipoint
Screened over No.40: Yes
Larger particles removed: Dry sieved
Approximate maximum grain size: No.10
Estimated percent retained on No.40: Not requested
As-received water content (%): 27.0

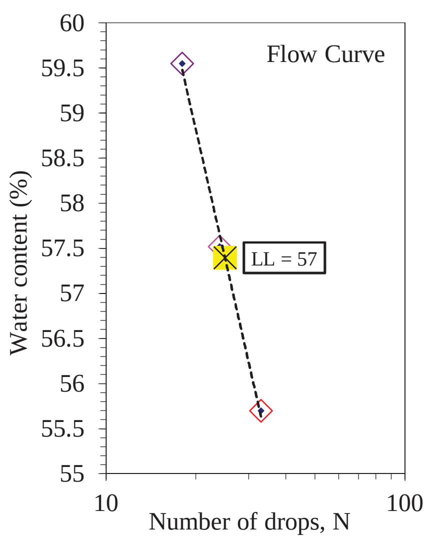
Plastic Limit

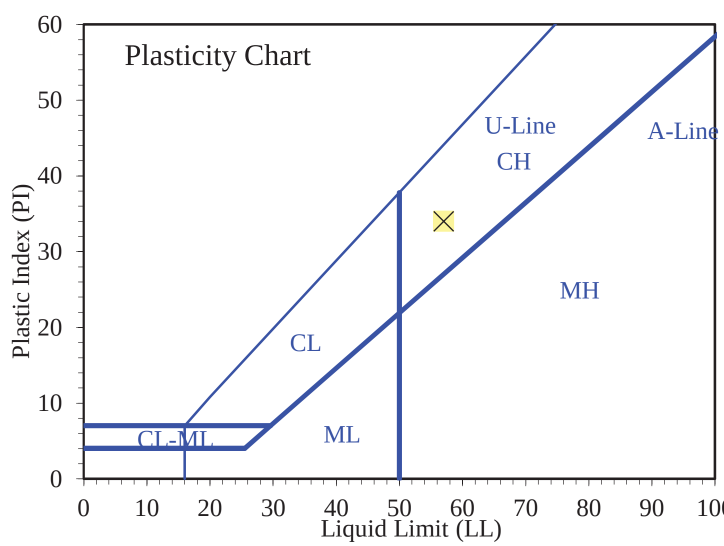
Determination No	1	2				
Wet Soil + Tare (g)	28.02	27.91				
Dry Soil + Tare (g)	26.88	26.77				
Water Loss (g)	1.14	1.14				
Tare (g)	21.86	21.81				
Dry Soil (g)	5.02	4.96				
Water Content, w (%)	22.71	22.98				

Liquid Limit

Determination No	1	2	3			
Number of Drops, N	33	24	18			
Wet Soil + Tare (g)	31.77	29.76	30.15			
Dry Soil + Tare (g)	28.30	26.70	27.00			
Water Loss (g)	3.47	3.06	3.15			
Tare (g)	22.07	21.38	21.71			
Dry Soil (g)	6.23	5.32	5.29			
Water Content, w (%)	55.70	57.52	59.55			
One-Point LL (%)		57				

Liquid Limit, LL (%)	**57**
Plastic Limit, PL (%)	**23**
Plasticity Index, PI (%)	**34**





Entered by:____________
Reviewed:____________

Liquid Limit, Plastic Limit, and Plasticity Index of Soils

(ASTM D4318)

IGES

Project: Toliver Property
No: 02489-001
Location: Huntsville, UT
Date: 7/1/2017
By: BRR

Boring No.: T-4
Sample: Sta 21
Depth: 3.5'
Description: Brown fat clay

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

Preparation method: Air Dry
Liquid limit test method: Multipoint
Screened over No.40: Yes
Larger particles removed: Dry sieved
Approximate maximum grain size: No.10
Estimated percent retained on No.40: Not requested
As-received water content (%): 24.8

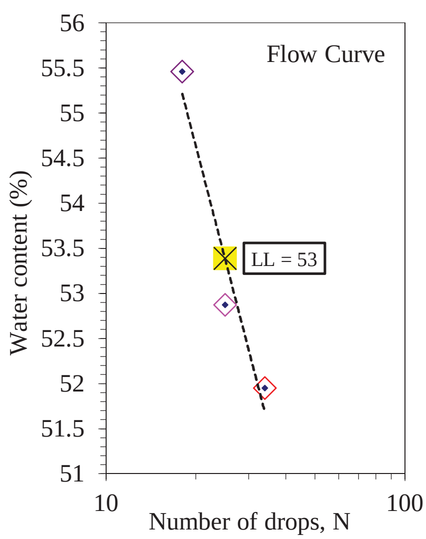
Plastic Limit

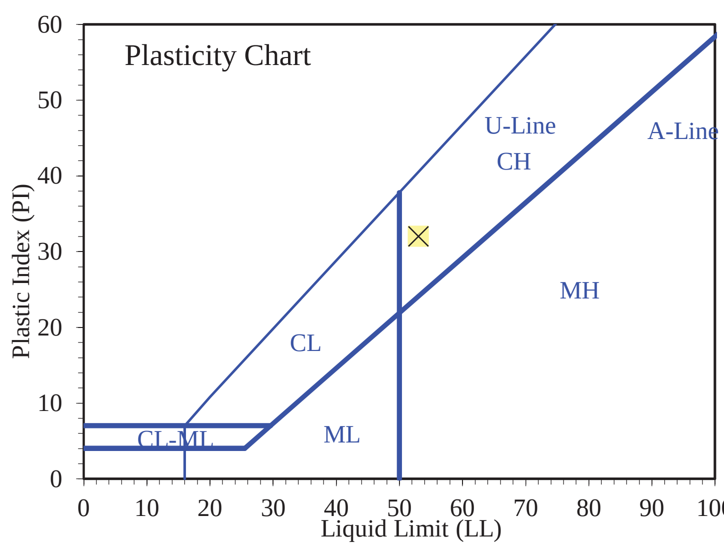
Determination No	1	2				
Wet Soil + Tare (g)	28.31	27.81				
Dry Soil + Tare (g)	27.23	26.71				
Water Loss (g)	1.08	1.10				
Tare (g)	22.07	21.53				
Dry Soil (g)	5.16	5.18				
Water Content, w (%)	20.93	21.24				

Liquid Limit

Determination No	1	2	3			
Number of Drops, N	34	25	18			
Wet Soil + Tare (g)	30.33	31.31	31.26			
Dry Soil + Tare (g)	27.53	28.18	28.06			
Water Loss (g)	2.80	3.13	3.20			
Tare (g)	22.14	22.26	22.29			
Dry Soil (g)	5.39	5.92	5.77			
Water Content, w (%)	51.95	52.87	55.46			
One-Point LL (%)		53				

Liquid Limit, LL (%)	**53**
Plastic Limit, PL (%)	**21**
Plasticity Index, PI (%)	**32**





Entered by:____________
Reviewed:____________

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

(ASTM D6913)

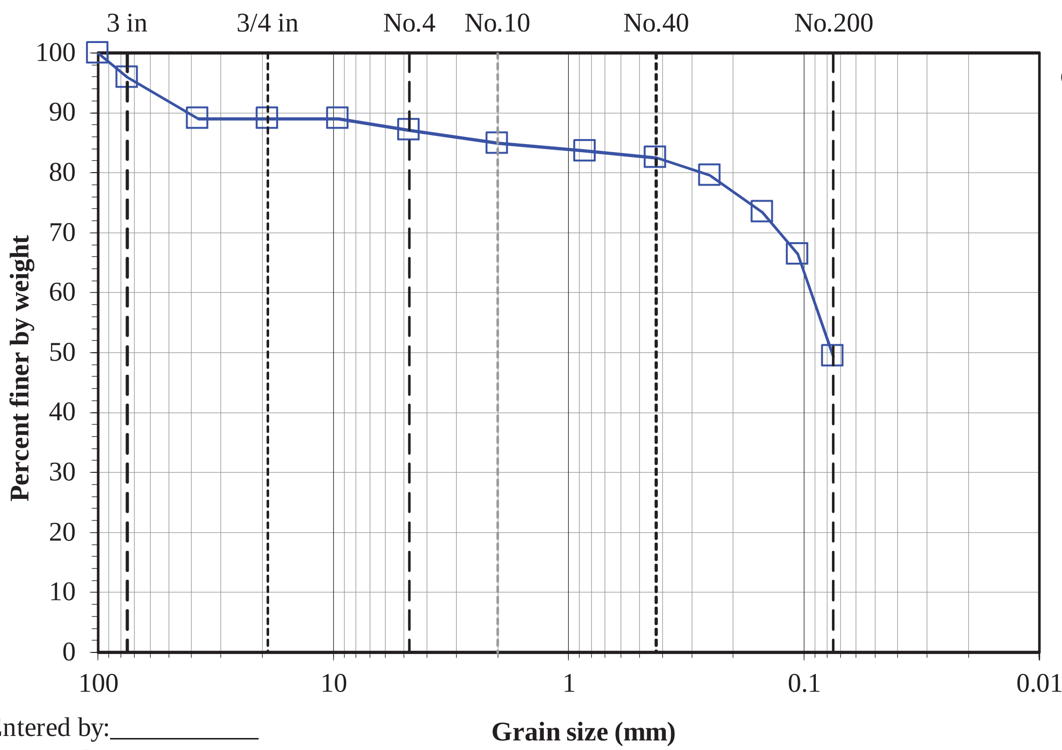
Project: Toliver Property
No: 02489-001
Location: Huntsville, UT
Date: 7/3/2017
By: BSS

Boring No.: T-1
Sample: Sta. 14.5
Depth: 9.5'
Description: Light brown clayey sand

Split:	Yes	
Split sieve:	3/8"	
	Moist	Dry
Total sample wt. (g):	28157.10	25017.64
+3/8" Coarse fraction (g):	2956.30	2750.37
-3/8" Split fraction (g):	493.96	436.46
Split fraction:	0.890	

Water content data	C.F.(+3/8")	S.F.(-3/8")
Moist soil + tare (g):	3266.21	902.44
Dry soil + tare (g):	3060.30	844.94
Tare (g):	310.23	408.48
Water content (%):	7.5	13.2

Sieve	Accum. Wt. Ret. (g)	Grain Size (mm)	Percent Finer	
8"	-	200	-	
6"	-	150	-	
4"	-	100	100.0	
3"	1021.98	75	95.9	
1.5"	2750.37	37.5	89.0	
3/4"	2750.37	19	89.0	
3/8"	2750.37	9.5	89.0	←Split
No.4	9.31	4.75	87.1	
No.10	20.08	2	84.9	
No.20	26.30	0.85	83.6	
No.40	31.85	0.425	82.5	
No.60	46.64	0.25	79.5	
No.100	76.29	0.15	73.4	
No.140	111.10	0.106	66.3	
No.200	194.24	0.075	49.4	



Gravel (%): 12.9
Sand (%): 37.7
Fines (%): 49.4

Entered by:___________
Reviewed:___________

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

(ASTM D6913)

IGES

Project: Toliver Property
No: 02489-001
Location: Huntsville, UT
Date: 7/3/2017
By: JDF

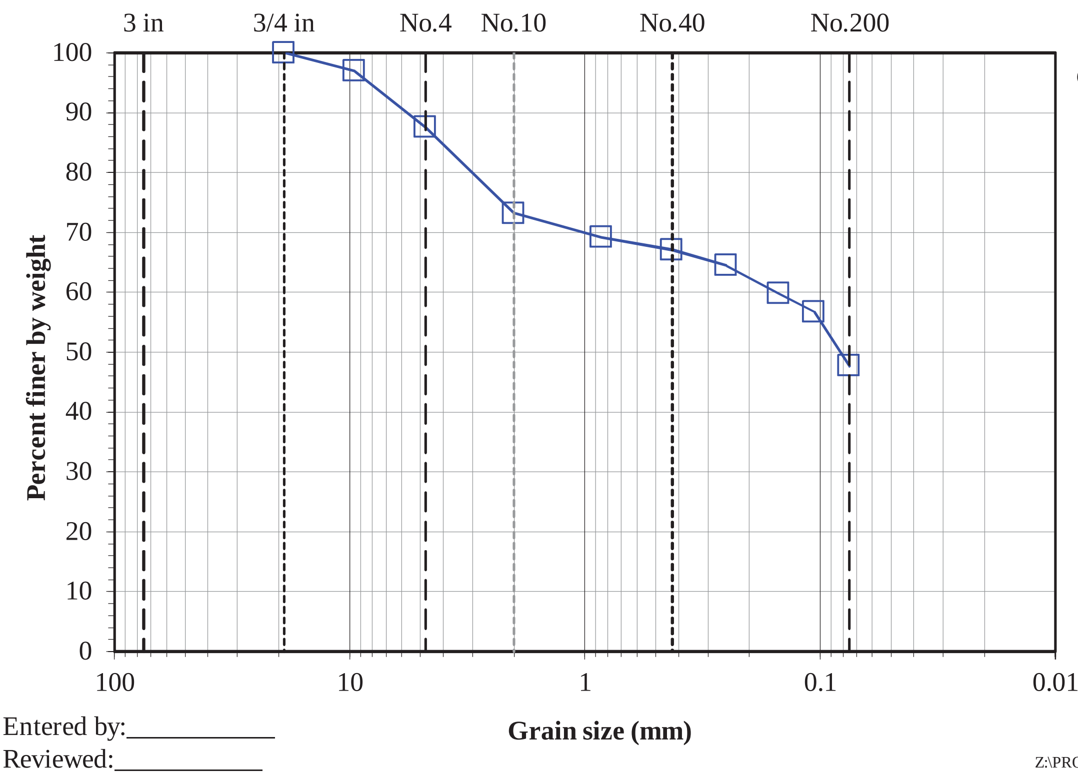
Boring No.: T-2
Sample: Sta. 22.5
Depth: 9.5'
Description: Brown clayey sand

Split:	No	
	-	
	Moist	Dry
Total sample wt. (g):	167.66	133.54
Split fraction:	1.000	

Water content data

Moist soil + tare (g):	-	289.20
Dry soil + tare (g):	-	255.08
Tare (g):	-	121.54
Water content (%):	0.0	25.6

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"	4.11	9.5	96.9
No.4	16.63	4.75	87.5
No.10	35.85	2	73.2
No.20	41.21	0.85	69.1
No.40	44.00	0.425	67.1
No.60	47.46	0.25	64.5
No.100	53.71	0.15	59.8
No.140	57.80	0.106	56.7
No.200	69.90	0.075	47.7



Gravel (%): 12.5
Sand (%): 39.9
Fines (%): 47.7

Entered by:___________
Reviewed:___________

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

(ASTM D1140)

IGES

Project: **Toliver Property**
No: **02489-001**
Location: Huntsville, UT
Date: 7/3/2017
By: JDF/BSS

Sample Info.	Boring No.	T-1	T-3	T-3	T-4	T-4			
	Sample	Sta. 23	Sta. 13	Sta. 10	Sta. 21	Sta. 26			
	Depth	5.0'	2.0'	4.0'	3.5'	9.0'			
	Split	No	No	No	No	Yes			
	Split Sieve*					3/8"			
	Method	B	B	B	B	B			
	Specimen soak time (min)	290	270	290	250	280			
	Moist total sample wt. (g)	200.83	348.20	173.27	301.97	20787.50			
	Moist coarse fraction (g)					77.30			
	Moist split fraction + tare (g)					798.27			
	Split fraction tare (g)					310.37			
	Dry split fraction (g)					420.06			
	Dry retained No. 200 + tare (g)	171.42	141.20	135.27	242.36	492.47			
	Wash tare (g)	121.54	126.16	127.74	152.68	310.37			
	No. 200 Dry wt. retained (g)	49.88	15.04	7.53	89.68	182.10			
	Split sieve* Dry wt. retained (g)					68.56			
	Dry total sample wt. (g)	164.41	271.63	136.48	242.03	17899.11			
Coarse Fraction	Moist soil + tare (g)					248.56			
	Dry soil + tare (g)					239.82			
	Tare (g)					171.29			
	Water content (%)					12.75			
Split Fraction	Moist soil + tare (g)	322.37	474.36	301.01	454.65	798.27			
	Dry soil + tare (g)	285.95	397.79	264.22	394.71	730.43			
	Tare (g)	121.54	126.16	127.74	152.68	310.37			
	Water content (%)	22.15	28.19	26.96	24.77	16.15			
	Percent passing split sieve* (%)					**99.6**			
	Percent passing No. 200 sieve (%)	**69.7**	**94.5**	**94.5**	**62.9**	**56.4**			

Entered by:___________

Reviewed:___________

Determination of the Point Load Strength Index of Rock

(ASTM D5731)

Project: Toliver Property
No: 02489-001
Location: Huntsville, UT
Date: 6/30/2017
By: JDF
Test Device: Humboldt H-1342
Test Frame: GEOTAC Sigma-1 10K   Calibration Date: 8/19/2016

Boring No.	T-1	T-1	T-3			
Sample:						
Depth:	6.0'	10.0'	8.0'			
Sample type	Block	Block	Block			
Core test type						
Distance between platen points, D (in.)	3.436	2.459	1.715			
D (mm)	87.274	62.459	43.561			
Smallest specimen width, W (in.)	3.549	3.725	2.561			
W (mm)	90.1	94.6	65.0			
Equivalent core area, D_e^2 (mm^2)	10017.0	7524.2	3607.9			
Failure load, P (lbf)	1875	1208	214			
P (N)	8340	5373	952			
Point load strength index, I_s (MPa)	0.83	0.71	0.26			
Size correction factor, F	1.367	1.281	1.086			
PLSI 50mm equivalent, $I_{s(50)}$ (MPa)	1.14	0.92	0.29			
Site specific correlation, C	18.2	18.2	24.5			
Uniaxial compressive strength, δ_{uc} (MPa)	20.71	16.65	7.02			
Uniaxial compressive strength, δ_{uc} (psi)	**3004**	**2416**	**1018**			
Comments:	Point load applied perpendicular to layering.		A small layer broke early in test before entire block broke.			

Entered by:___________

Reviewed:___________

Direct Shear Test for Soils Under Drained Conditions

(ASTM D3080)

Project: Toliver Property
No: 02489-001
Location: Huntsville, UT
Date: 6/30/2017
By: JDF

Boring No.: T-1
Sample: Station 23
Depth: 5.0'
Sample Description: Brown sandy clay
Sample type: Undisturbed-trimmed from thin-wall

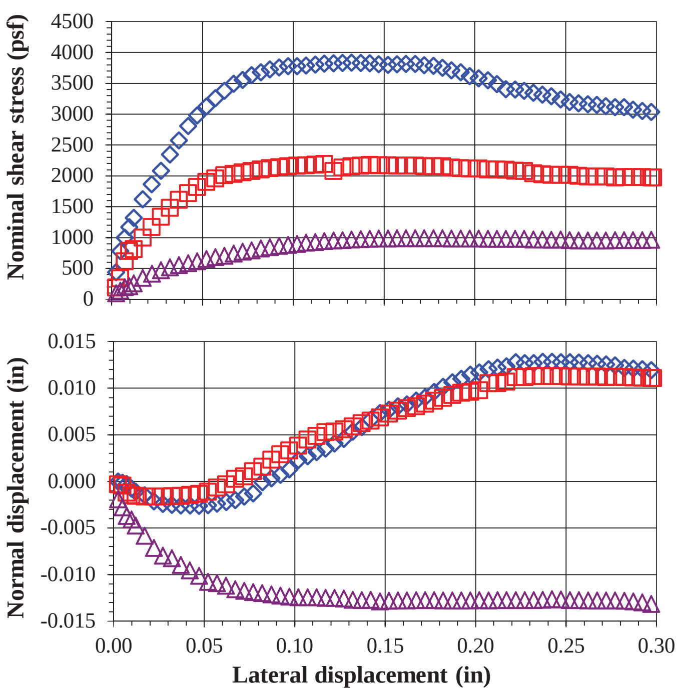
Test type: Inundated
Lateral displacement (in.): 0.3
Shear rate (in./min): 0.0007
Specific gravity, Gs: 2.70 Assumed

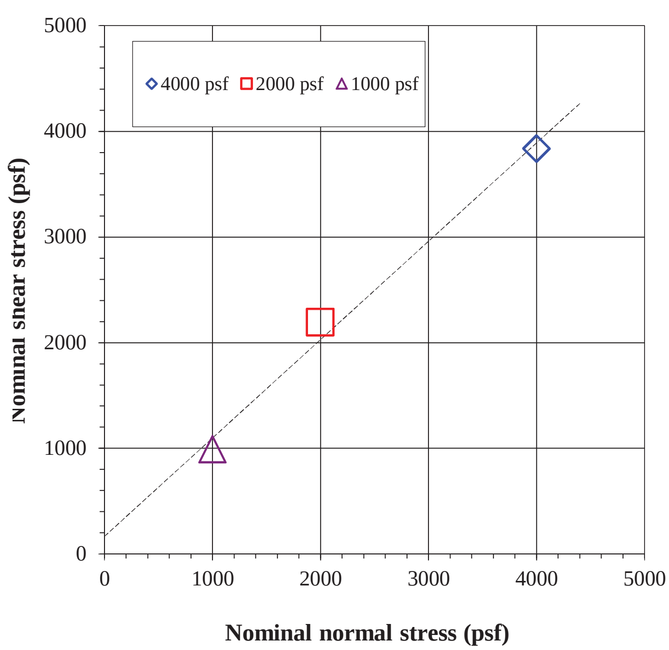
	Sample 1		Sample 2		Sample 3	
Nominal normal stress (psf)	4000		2000		1000	
Peak shear stress (psf)	3837		2192		990	
Lateral displacement at peak (in)	0.132		0.117		0.162	
Load Duration (min)	155		180		184	
	Initial	Pre-shear	Initial	Pre-shear	Initial	Pre-shear
Sample height (in)	0.9950	0.9637	0.9990	0.9800	0.9970	0.9772
Sample diameter (in)	2.414	2.414	2.419	2.419	2.415	2.415
Wt. rings + wet soil (g)	179.87	186.72	176.85	185.49	171.50	183.58
Wt. rings (g)	44.84	44.84	42.36	42.36	45.06	45.06
Wet soil + tare (g)	322.37		322.37		322.37	
Dry soil + tare (g)	285.95		285.95		285.95	
Tare (g)	121.54		121.54		121.54	
Water content (%)	22.2	28.3	22.2	30.0	22.2	33.8
Dry unit weight (pcf)	92.0	95.4	91.3	93.1	86.1	88.1
Void ratio, e, for assumed Gs	0.82	0.77	0.85	0.81	0.95	0.91
Saturation (%)*	72.7	100.0	70.8	100.0	62.8	100.0

ϕ' (deg)	43
c' (psf)	167

Average of 3 samples	Initial	Pre-shear
Water content (%)	22.2	30.7
Dry unit weight (pcf)	89.8	92.2

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





Entered by:___________
Reviewed:___________

Direct Shear Test for Soils Under Drained Conditions

(ASTM D3080)

Project: Toliver Property **Boring No.:** T-1
No: 02489-001 **Sample:** Station 23
Location: Huntsville, UT **Depth:** 5.0'

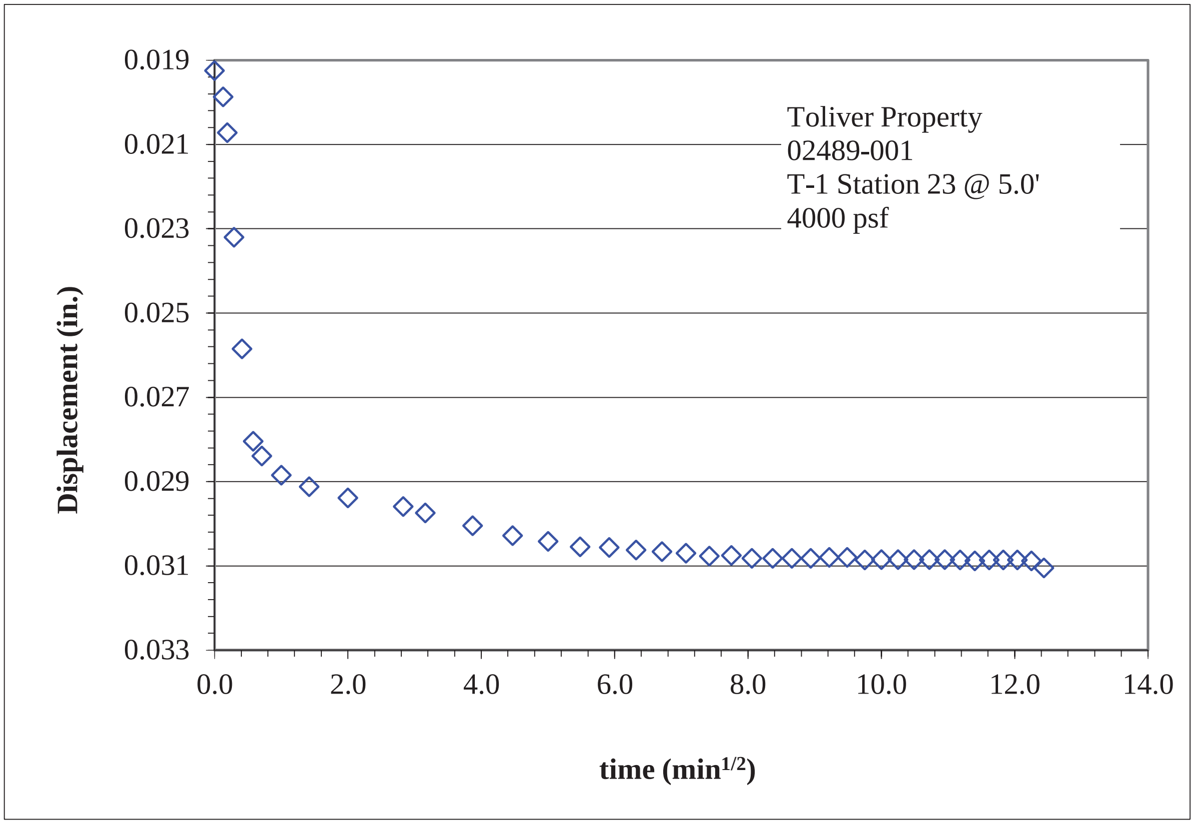
Nominal normal stress = 4000 psf			Nominal normal stress = 2000 psf			Nominal normal stress = 1000 psf		
Lateral Displacement (in.)	Nominal Shear Stress (psf)	Normal Displacement (in.)	Lateral Displacement (in.)	Nominal Shear Stress (psf)	Normal Displacement (in.)	Lateral Displacement (in.)	Nominal Shear Stress (psf)	Normal Displacement (in.)
0.002	437	0.000	0.002	188	0.000	0.002	96	-0.002
0.005	778	0.000	0.005	339	0.000	0.005	134	-0.003
0.007	1000	0.000	0.007	616	-0.001	0.007	195	-0.004
0.010	1171	-0.001	0.010	779	-0.001	0.010	211	-0.004
0.012	1320	-0.001	0.012	809	-0.002	0.012	255	-0.005
0.017	1618	-0.001	0.017	1000	-0.002	0.017	343	-0.006
0.022	1863	-0.002	0.022	1168	-0.002	0.022	414	-0.007
0.027	2080	-0.002	0.027	1337	-0.002	0.027	468	-0.008
0.032	2349	-0.003	0.032	1481	-0.002	0.032	514	-0.008
0.037	2576	-0.003	0.037	1612	-0.002	0.037	551	-0.009
0.042	2806	-0.003	0.042	1719	-0.001	0.042	581	-0.010
0.047	2979	-0.003	0.047	1818	-0.001	0.047	614	-0.010
0.052	3127	-0.003	0.052	1903	-0.001	0.052	652	-0.011
0.057	3261	-0.002	0.057	1955	-0.001	0.057	680	-0.011
0.062	3380	-0.002	0.062	2007	0.000	0.062	704	-0.011
0.067	3491	-0.002	0.067	2031	0.000	0.067	740	-0.012
0.072	3558	-0.002	0.072	2058	0.001	0.072	767	-0.012
0.077	3633	-0.001	0.077	2075	0.001	0.077	792	-0.012
0.082	3680	0.000	0.082	2101	0.002	0.082	819	-0.012
0.087	3729	0.000	0.087	2123	0.002	0.087	841	-0.012
0.092	3760	0.001	0.092	2141	0.003	0.092	860	-0.012
0.097	3780	0.001	0.097	2155	0.003	0.097	880	-0.012
0.102	3778	0.002	0.102	2165	0.004	0.102	895	-0.012
0.107	3788	0.003	0.107	2169	0.004	0.107	912	-0.012
0.112	3804	0.003	0.112	2180	0.005	0.112	928	-0.012
0.117	3819	0.004	0.117	2192	0.005	0.117	938	-0.013
0.122	3827	0.004	0.122	2079	0.005	0.122	950	-0.013
0.127	3830	0.005	0.127	2136	0.006	0.127	958	-0.013
0.132	3837	0.005	0.132	2155	0.006	0.132	965	-0.013
0.137	3830	0.006	0.137	2167	0.006	0.137	972	-0.013
0.142	3824	0.007	0.142	2175	0.007	0.142	979	-0.013
0.147	3811	0.007	0.147	2168	0.007	0.147	982	-0.013
0.152	3799	0.008	0.152	2168	0.007	0.152	985	-0.013
0.157	3811	0.008	0.157	2166	0.008	0.157	989	-0.013
0.162	3817	0.008	0.162	2166	0.008	0.162	990	-0.013
0.167	3814	0.009	0.167	2167	0.008	0.167	990	-0.013
0.172	3793	0.009	0.172	2157	0.008	0.172	987	-0.013
0.177	3783	0.010	0.177	2155	0.009	0.177	986	-0.013
0.182	3752	0.010	0.182	2157	0.009	0.182	987	-0.013
0.187	3713	0.011	0.187	2136	0.009	0.187	985	-0.013
0.192	3680	0.011	0.192	2125	0.010	0.192	984	-0.013
0.197	3620	0.011	0.197	2118	0.010	0.197	983	-0.013
0.202	3584	0.012	0.202	2118	0.010	0.202	980	-0.013
0.207	3548	0.012	0.207	2102	0.011	0.207	979	-0.013
0.212	3486	0.012	0.212	2102	0.011	0.212	977	-0.013
0.217	3403	0.012	0.217	2096	0.011	0.217	978	-0.013
0.222	3401	0.013	0.222	2082	0.011	0.222	978	-0.013
0.227	3380	0.013	0.227	2077	0.011	0.227	977	-0.013
0.232	3349	0.013	0.232	2042	0.011	0.232	967	-0.013
0.237	3315	0.013	0.237	2026	0.011	0.237	965	-0.013
0.242	3289	0.013	0.242	2017	0.011	0.242	962	-0.013
0.247	3240	0.013	0.247	2012	0.011	0.247	966	-0.013
0.252	3196	0.013	0.252	2013	0.011	0.252	959	-0.013
0.257	3176	0.013	0.257	2001	0.011	0.257	953	-0.013
0.262	3163	0.013	0.262	1990	0.011	0.262	951	-0.013
0.267	3153	0.013	0.267	1988	0.011	0.267	953	-0.013
0.272	3129	0.013	0.272	1989	0.011	0.272	953	-0.013
0.277	3114	0.012	0.277	1974	0.011	0.277	954	-0.013
0.282	3114	0.012	0.282	1978	0.011	0.282	954	-0.013
0.287	3075	0.012	0.287	1980	0.011	0.287	956	-0.013
0.292	3054	0.012	0.292	1978	0.011	0.292	958	-0.013
0.297	3039	0.012	0.297	1973	0.011	0.297	959	-0.013
0.302	3036	0.012	0.298	1972	0.011	0.302	962	-0.013

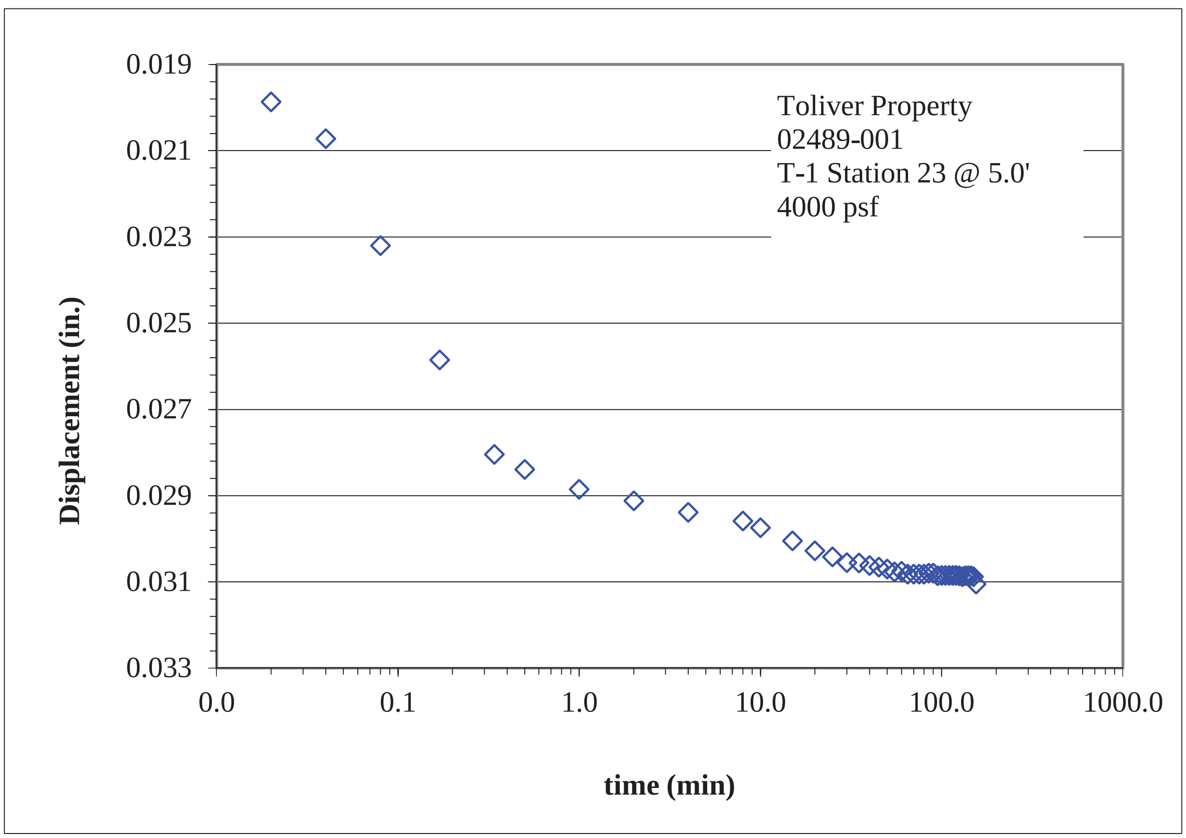
Direct Shear Test for Soils Under Drained Conditions

(ASTM D3080)

Project: Toliver Property
No: 02489-001
Location: Huntsville, UT

Boring No.: T-1
Sample: Station 23
Depth: 5.0'





Direct Shear Test for Soils Under Drained Conditions

(ASTM D3080)

Project: Toliver Property
No: 02489-001
Location: Huntsville, UT
Date: 7/3/2017
By: JDF

Boring No.: T-2
Sample: Station 22.5
Depth: 9.5'
Sample Description: Brown clayey sand
Sample type: Undisturbed-trimmed from thin-wall

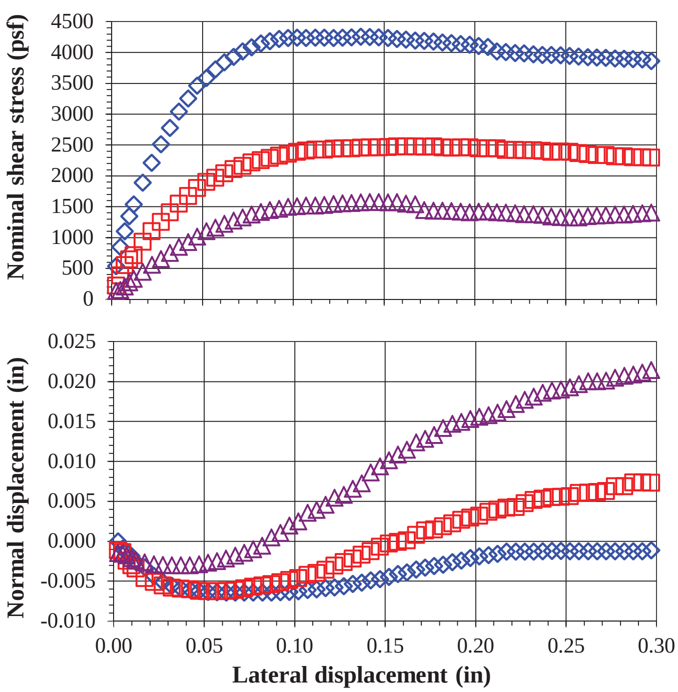
Test type: Inundated
Lateral displacement (in.): 0.3
Shear rate (in./min): 0.0009
Specific gravity, Gs: 2.70 Assumed

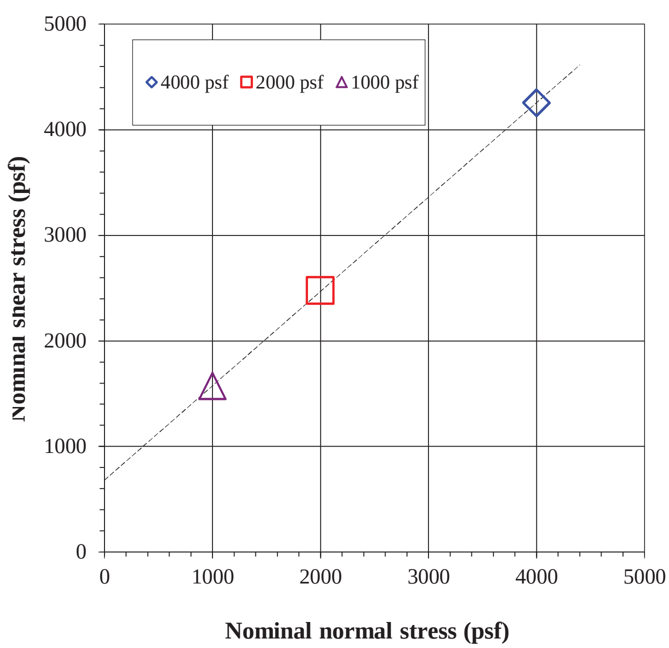
	Sample 1		Sample 2		Sample 3	
Nominal normal stress (psf)	4000		2000		1000	
Peak shear stress (psf)	4255		2475		1572	
Lateral displacement at peak (in)	0.137		0.167		0.147	
Load Duration (min)	141		175		186	
	Initial	Pre-shear	Initial	Pre-shear	Initial	Pre-shear
Sample height (in)	0.9950	0.9556	0.9930	0.9641	1.0020	0.9754
Sample diameter (in)	2.418	2.418	2.413	2.413	2.415	2.415
Wt. rings + wet soil (g)	168.03	177.30	162.17	176.15	163.91	177.83
Wt. rings (g)	42.39	42.39	45.28	45.28	44.96	44.96
Wet soil + tare (g)	289.20		289.20		289.20	
Dry soil + tare (g)	255.08		255.08		255.08	
Tare (g)	121.54		121.54		121.54	
Water content (%)	25.6	34.8	25.6	40.6	25.6	40.2
Dry unit weight (pcf)	83.0	86.8	77.6	80.4	78.8	80.7
Void ratio, e, for assumed Gs	1.02	0.94	1.16	1.10	1.14	1.09
Saturation (%)*	67.6	100.0	59.6	100.0	60.3	100.0

ϕ' (deg)	42
c' (psf)	682

Average of 3 samples	Initial	Pre-shear
Water content (%)	25.6	38.5
Dry unit weight (pcf)	79.8	82.7

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





Entered by:___________
Reviewed:___________

Direct Shear Test for Soils Under Drained Conditions

(ASTM D3080)

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Project: Toliver Property
No: 02489-001
Location: Huntsville, UT

Boring No.: T-2
Sample: Station 22.5
Depth: 9.5'

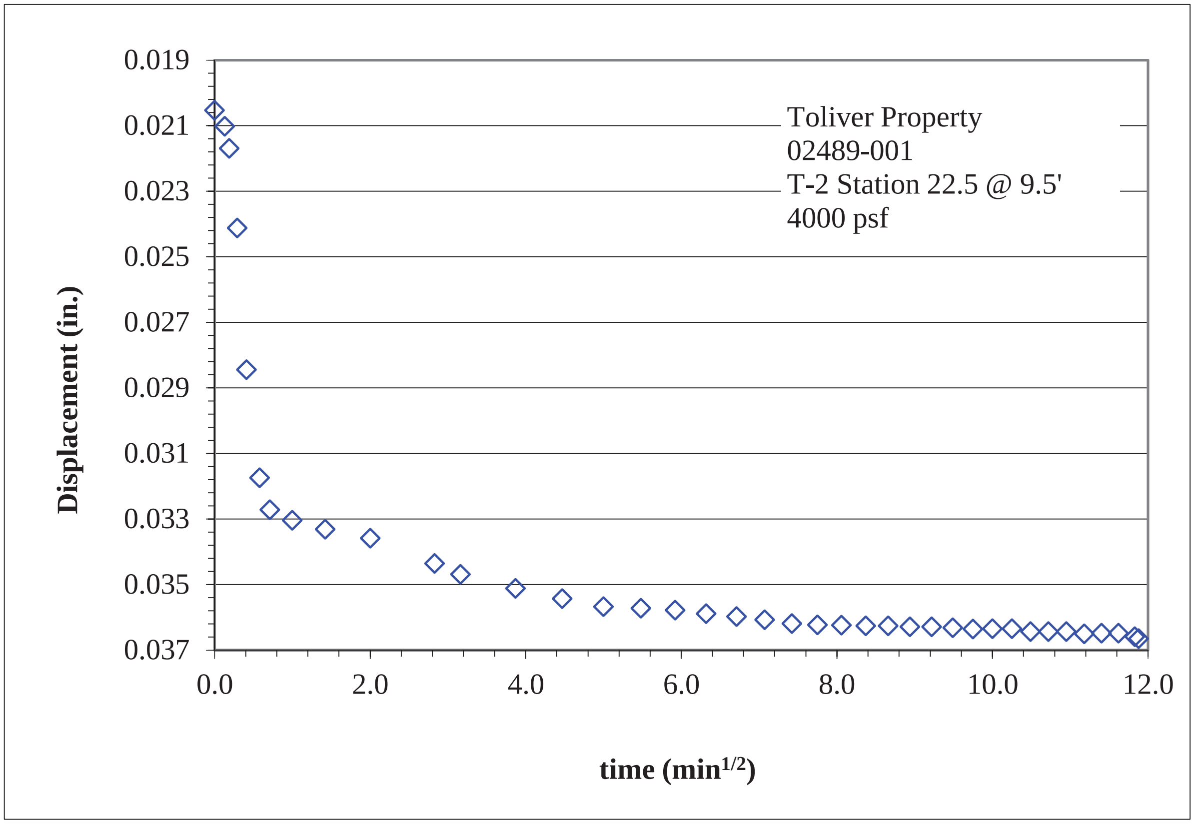
Nominal normal stress = 4000 psf			Nominal normal stress = 2000 psf			Nominal normal stress = 1000 psf		
Lateral Displacement (in.)	Nominal Shear Stress (psf)	Normal Displacement (in.)	Lateral Displacement (in.)	Nominal Shear Stress (psf)	Normal Displacement (in.)	Lateral Displacement (in.)	Nominal Shear Stress (psf)	Normal Displacement (in.)
0.002	536	0.000	0.002	221	-0.001	0.002	131	-0.001
0.005	847	-0.001	0.005	385	-0.001	0.005	146	-0.002
0.007	1100	-0.001	0.007	549	-0.002	0.007	202	-0.002
0.010	1344	-0.002	0.010	646	-0.003	0.010	271	-0.002
0.012	1538	-0.003	0.012	714	-0.003	0.012	327	-0.002
0.017	1888	-0.003	0.017	929	-0.005	0.017	443	-0.003
0.022	2210	-0.004	0.022	1102	-0.005	0.022	556	-0.003
0.027	2511	-0.005	0.027	1254	-0.006	0.027	651	-0.003
0.032	2776	-0.006	0.032	1406	-0.006	0.032	750	-0.003
0.037	3042	-0.006	0.037	1549	-0.006	0.037	848	-0.003
0.042	3253	-0.006	0.042	1670	-0.006	0.042	927	-0.003
0.047	3464	-0.006	0.047	1801	-0.006	0.047	1013	-0.003
0.052	3588	-0.006	0.052	1898	-0.006	0.052	1100	-0.003
0.057	3727	-0.006	0.057	1968	-0.006	0.057	1160	-0.003
0.062	3840	-0.006	0.062	2038	-0.006	0.062	1222	-0.002
0.067	3928	-0.006	0.067	2107	-0.006	0.067	1273	-0.002
0.072	4018	-0.006	0.072	2160	-0.006	0.072	1322	-0.001
0.077	4087	-0.006	0.077	2217	-0.006	0.077	1369	-0.001
0.082	4149	-0.006	0.082	2252	-0.006	0.082	1412	-0.001
0.087	4183	-0.006	0.087	2292	-0.005	0.087	1446	0.000
0.092	4221	-0.006	0.092	2326	-0.005	0.092	1466	0.001
0.097	4237	-0.006	0.097	2359	-0.005	0.097	1495	0.002
0.102	4239	-0.006	0.102	2386	-0.005	0.102	1508	0.002
0.107	4237	-0.006	0.107	2409	-0.004	0.107	1515	0.004
0.112	4242	-0.006	0.112	2422	-0.004	0.112	1518	0.004
0.117	4242	-0.006	0.117	2425	-0.004	0.117	1529	0.005
0.122	4237	-0.006	0.122	2438	-0.003	0.122	1544	0.005
0.127	4242	-0.006	0.127	2445	-0.003	0.127	1552	0.006
0.132	4247	-0.005	0.132	2446	-0.002	0.132	1559	0.007
0.137	4255	-0.005	0.137	2454	-0.002	0.137	1568	0.007
0.142	4250	-0.005	0.142	2459	-0.001	0.142	1571	0.009
0.147	4244	-0.005	0.147	2463	-0.001	0.147	1572	0.009
0.152	4234	-0.004	0.152	2465	0.000	0.152	1569	0.010
0.157	4219	-0.004	0.157	2474	0.000	0.157	1571	0.011
0.162	4208	-0.004	0.162	2474	0.000	0.162	1549	0.011
0.167	4195	-0.004	0.167	2475	0.001	0.167	1539	0.012
0.172	4188	-0.003	0.172	2473	0.001	0.172	1445	0.013
0.177	4177	-0.003	0.177	2475	0.001	0.177	1435	0.013
0.182	4165	-0.003	0.182	2460	0.002	0.182	1435	0.014
0.187	4154	-0.003	0.187	2457	0.002	0.187	1429	0.015
0.192	4139	-0.002	0.192	2462	0.003	0.192	1419	0.015
0.197	4126	-0.002	0.197	2462	0.003	0.197	1410	0.015
0.202	4105	-0.002	0.202	2447	0.003	0.202	1416	0.016
0.207	4092	-0.002	0.207	2445	0.004	0.207	1424	0.016
0.212	4018	-0.002	0.212	2445	0.004	0.212	1406	0.016
0.217	4007	-0.001	0.217	2420	0.004	0.217	1403	0.017
0.222	3992	-0.001	0.222	2421	0.004	0.222	1391	0.017
0.227	3987	-0.001	0.227	2414	0.005	0.227	1377	0.018
0.232	3969	-0.001	0.232	2415	0.005	0.232	1377	0.018
0.237	3959	-0.001	0.237	2404	0.005	0.237	1364	0.019
0.242	3961	-0.001	0.242	2386	0.006	0.242	1340	0.019
0.247	3953	-0.001	0.247	2393	0.006	0.247	1332	0.019
0.252	3946	-0.001	0.252	2389	0.006	0.252	1322	0.019
0.257	3935	-0.001	0.257	2365	0.006	0.257	1324	0.020
0.262	3925	-0.001	0.262	2350	0.006	0.262	1346	0.020
0.267	3917	-0.001	0.267	2338	0.006	0.267	1354	0.020
0.272	3912	-0.001	0.272	2337	0.006	0.272	1363	0.020
0.277	3904	-0.001	0.277	2315	0.007	0.277	1373	0.020
0.282	3899	-0.001	0.282	2317	0.007	0.282	1373	0.021
0.287	3892	-0.001	0.287	2301	0.007	0.287	1381	0.021
0.292	3889	-0.001	0.292	2301	0.007	0.292	1393	0.021
0.297	3861	-0.001	0.297	2295	0.007	0.297	1403	0.021
0.302	3863	-0.001	0.301	2279	0.008	0.302	1411	0.022

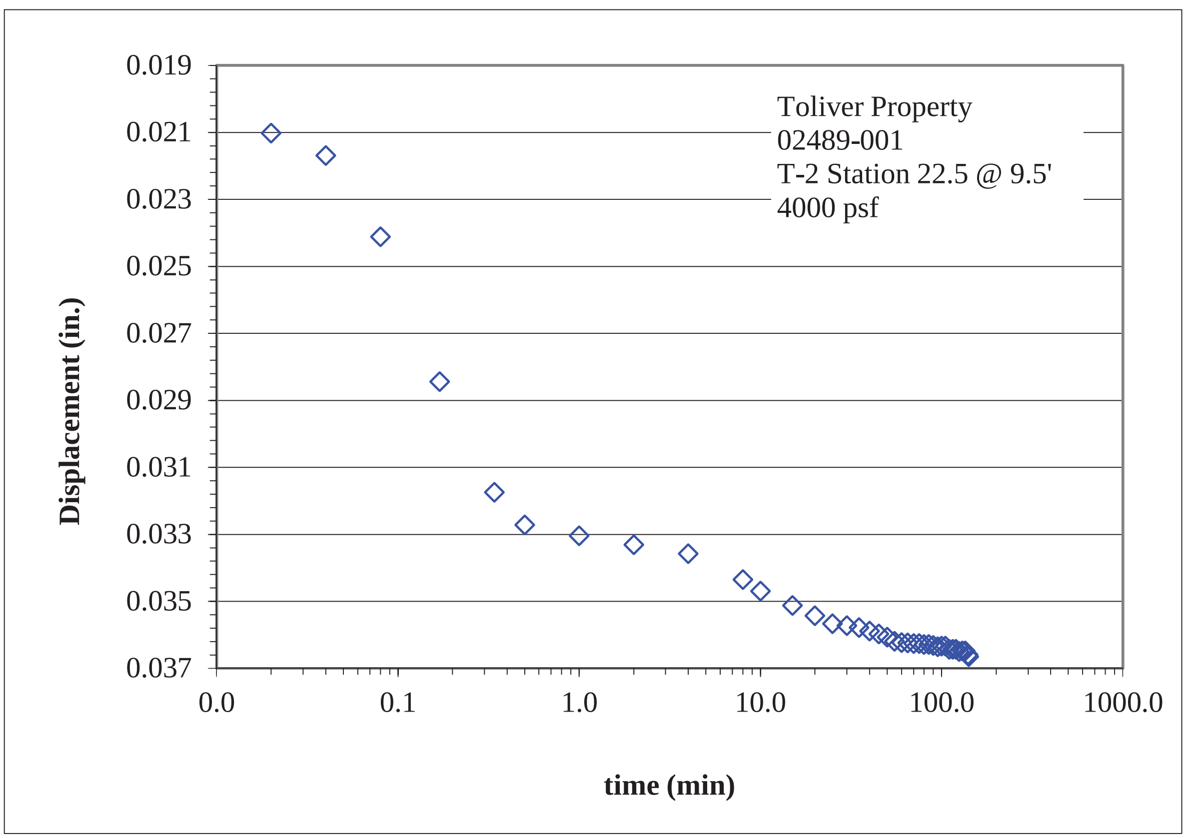
Direct Shear Test for Soils Under Drained Conditions

(ASTM D3080)

Project: Toliver Property
No: 02489-001
Location: Huntsville, UT

Boring No.: T-2
Sample: Station 22.5
Depth: 9.5'





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

IGES

Project: Toliver Property
No: 02489-001
Location: Huntsville, UT
Date: 7/3/2017
By: BSS

Sample info.	Boring No.	T-3							
	Sample								
	Depth	4.0'							
Water content data	Wet soil + tare (g)	109.05							
	Dry soil + tare (g)	94.36							
	Tare (g)	37.47							
	Water content (%)	25.8							
Chem. data	pH	6.17							
	Soluble chloride* (ppm)	16.5							
	Soluble sulfate** (ppm)	71.2							
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	3487	0.67	2336				
		+3	1955	0.67	1310				
		+6	1481	0.67	992				
		+9	1154	0.67	773				
		+12	1239	0.67	830				
	Minimum resistivity (Ω-cm)	**773**							

* 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 (41.2357°N, 111.7996°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 = 0.850 g

From Figure 1613.3.1(2) [2] S_1 = 0.287 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 $w \geq 40\%$, and • Undrained shear strength $\bar{s}_u$ < 500 psf		
F. Soils requiring site response analysis in accordance with Section 21.1	See Section 20.3.1		

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

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

TABLE 1613.3.3(1)
VALUES OF SITE COEFFICIENT F_a

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

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

For Site Class = C and S_S = 0.850 g, F_a = 1.060

TABLE 1613.3.3(2)
VALUES OF SITE COEFFICIENT F_v

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

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

For Site Class = C and S_1 = 0.287 g, F_v = 1.513

Equation (16-37): $S_{MS} = F_a S_S = 1.060 \times 0.850 = 0.901$ g

Equation (16-38): $S_{M1} = F_v S_1 = 1.513 \times 0.287 = 0.434$ g

Section 1613.3.4 — Design spectral response acceleration parameters

Equation (16-39): $S_{DS} = ⅔\ S_{MS} = ⅔ \times 0.901 = 0.600$ g

Equation (16-40): $S_{D1} = ⅔\ S_{M1} = ⅔ \times 0.434 = 0.289$ g

Section 1613.3.5 — Determination of seismic design category

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

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

For Risk Category = I and S_{DS} = 0.600 g, Seismic Design Category = D

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

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

For Risk Category = I and S_{D1} = 0.289 g, Seismic Design Category = D

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

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

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

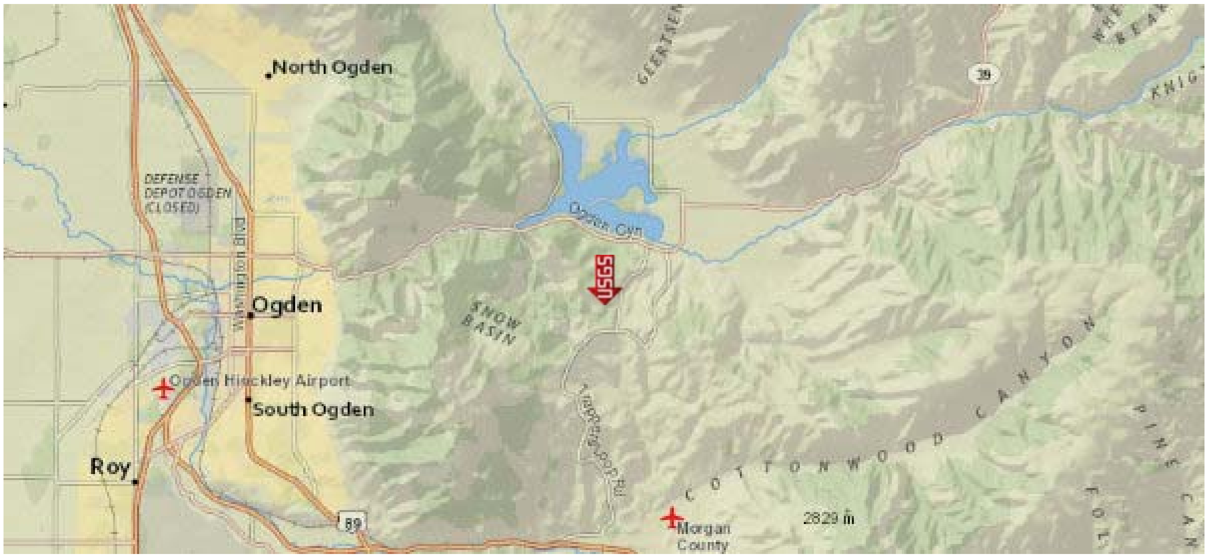
References

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

USGS Design Maps Summary Report

User–Specified Input

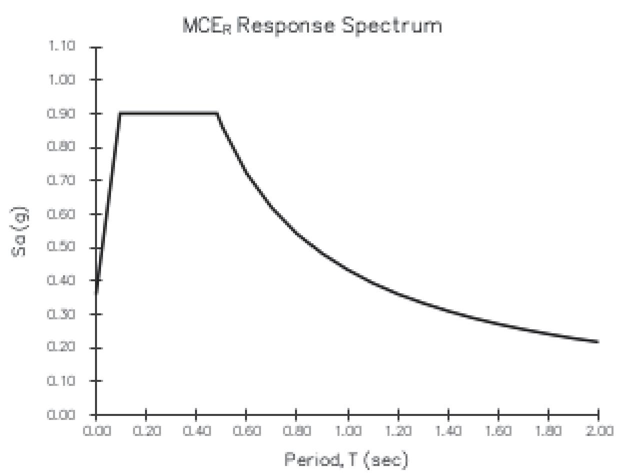
Report Title	Toliver Tue July 18, 2017 20:20:28 UTC
Building Code Reference Document	2012/2015 International Building Code (which utilizes USGS hazard data available in 2008)
Site Coordinates	41.2357°N, 111.7996°W
Site Soil Classification	Site Class C – "Very Dense Soil and Soft Rock"
Risk Category	I/II/III

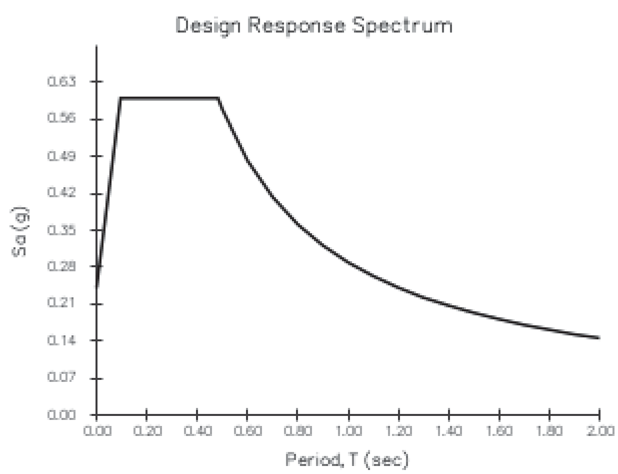


USGS–Provided Output

S_S =	0.850 g	S_{MS} =	0.901 g	S_{DS} =	0.600 g
S_1 =	0.287 g	S_{M1} =	0.434 g	S_{D1} =	0.289 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.



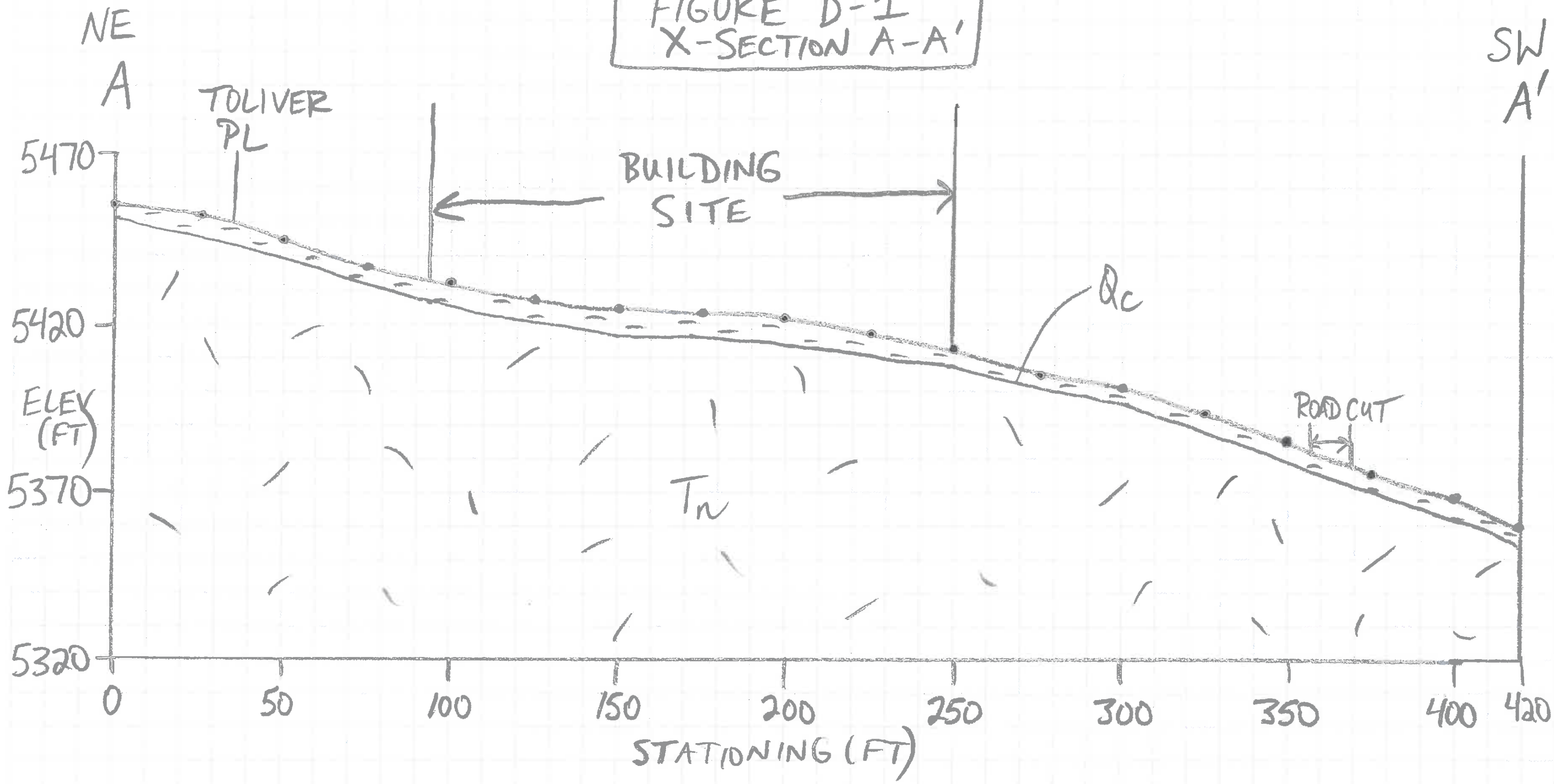


Although this information is a product of the U.S. Geological Survey, we provide no warranty, expressed or implied, as to the accuracy of the data contained therein. This tool is not a substitute for technical subject-matter knowledge.

APPENDIX D

Project No. 02489-001
Date 7-14-17 by PE
Ckd by ______ on ______

FIGURE D-1
X-SECTION A-A'



Qc = COLLUVIUM

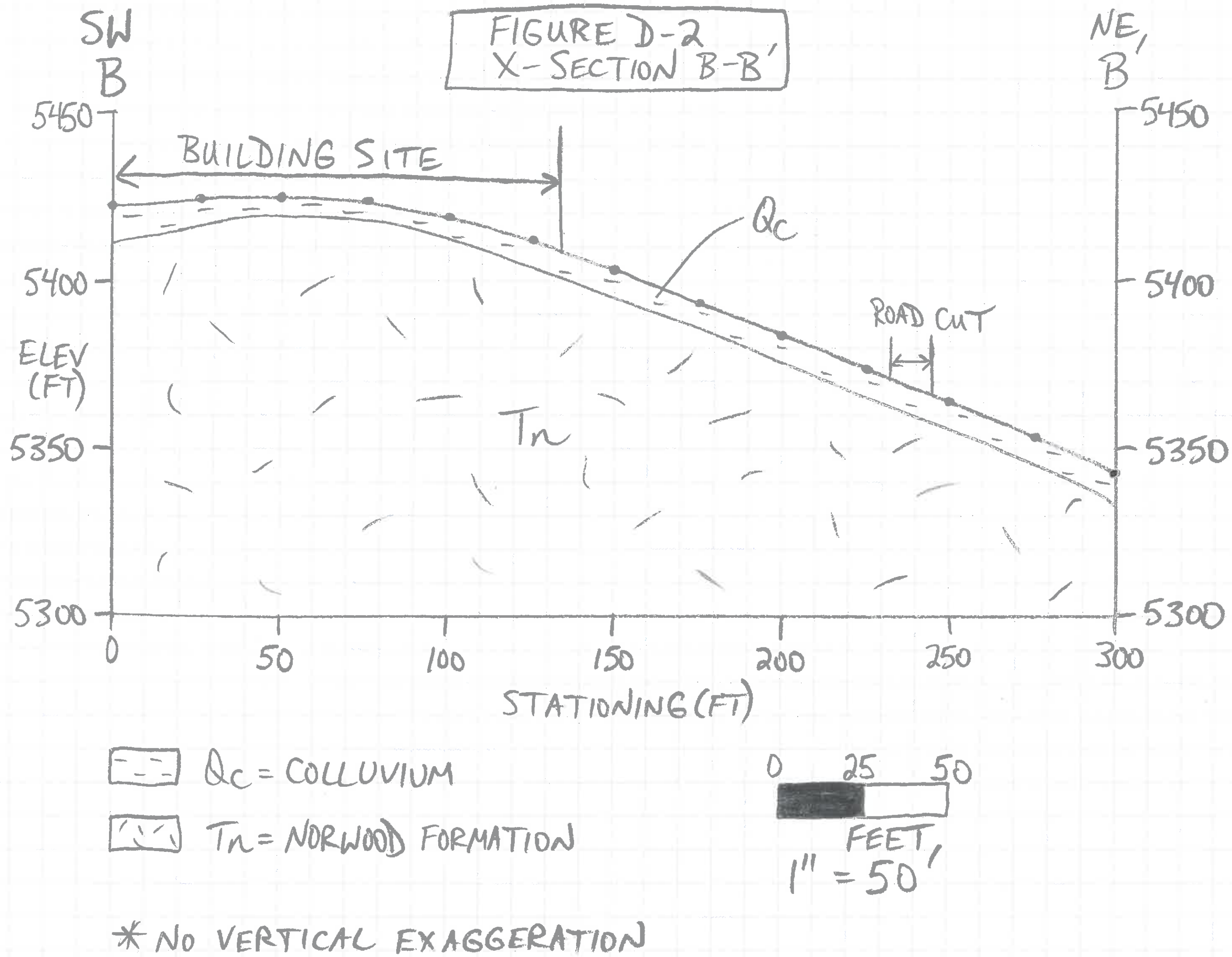
Tn = NORWOOD FORMATION

0 25 50
FEET
1" = 50'

* NO VERTICAL EXAGGERATION

IGES
Intermountain GeoEnvironmental Services, Inc.

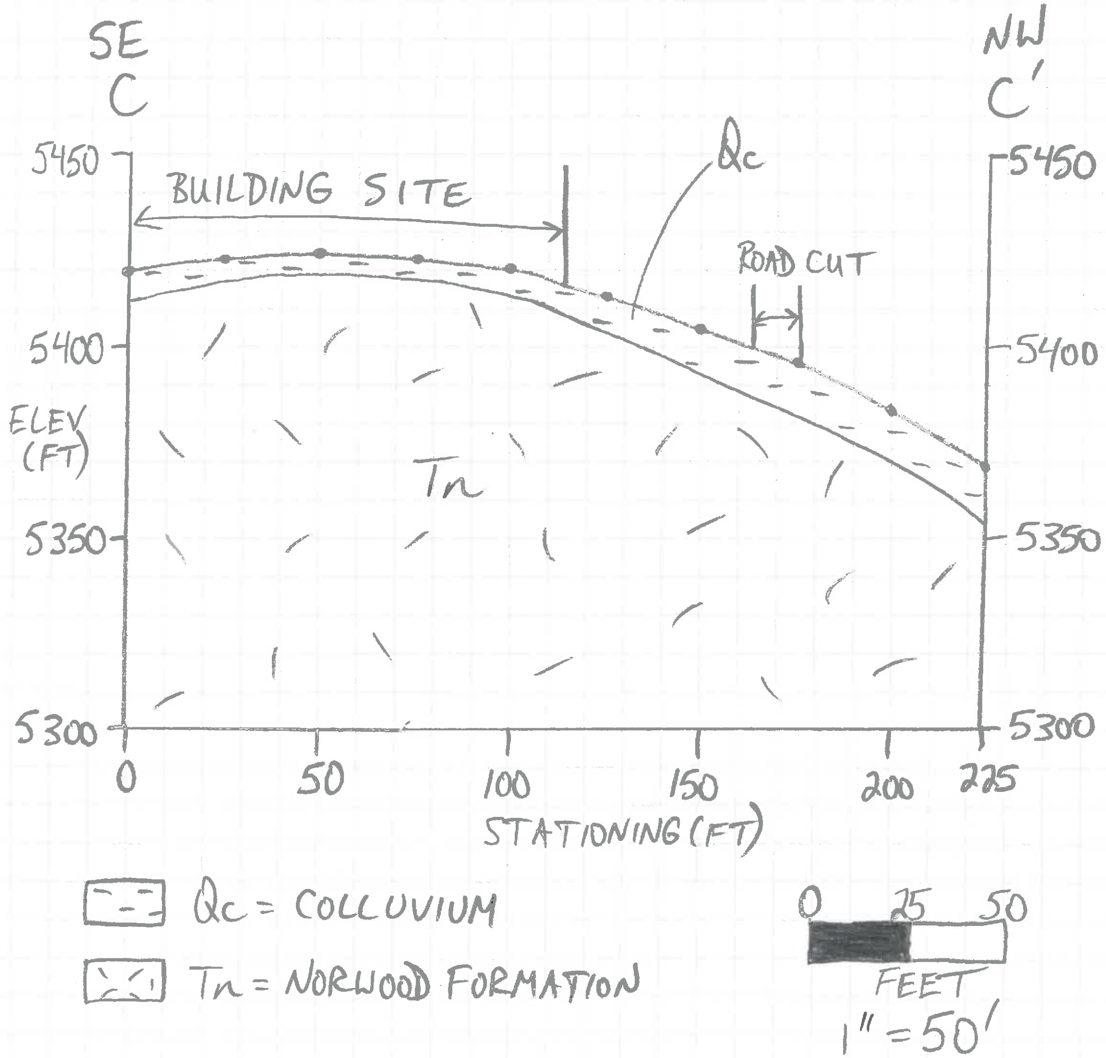
Project No. 02489-001
Date 7-17-17 by PEJ
Ckd by ___ on ___



IGES
Intermountain GeoEnvironmental Services, Inc.

Project No. 02489-001
Date 7-17-17 by PED
Ckd by ______ on ______

FIGURE D-3
X-SECTION C-C'



*NO VERTICAL EXAGGERATION

IGES
Intermountain GeoEnvironmental Services, Inc.

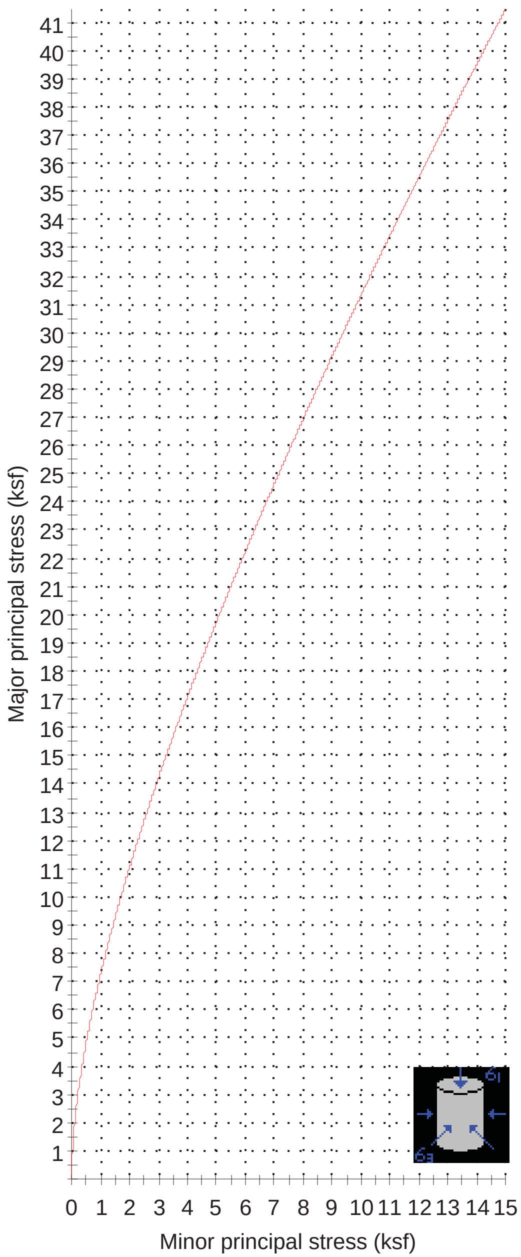
Analysis of Rock Strength using RocLab

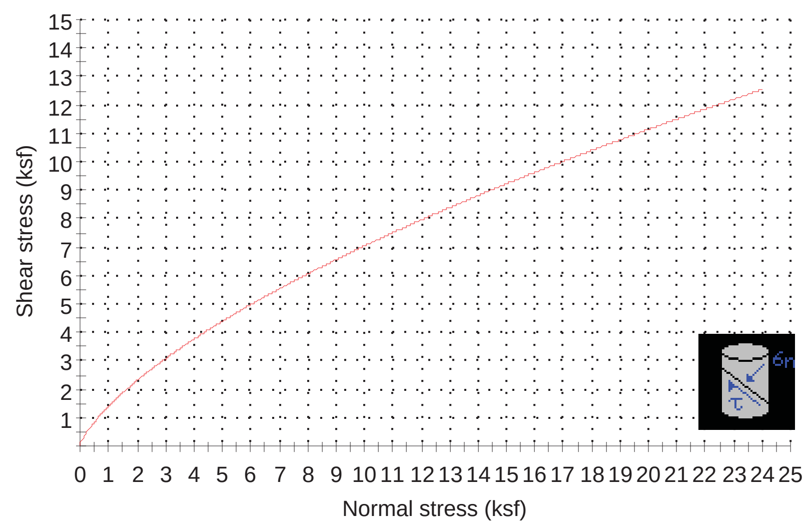
Hoek-Brown Classification
intact uniaxial comp. strength (sigci) = 60 ksf
GSI = 24   mi = 13   Disturbance factor (D) = 0
intact modulus (Ei) = 240000 ksf

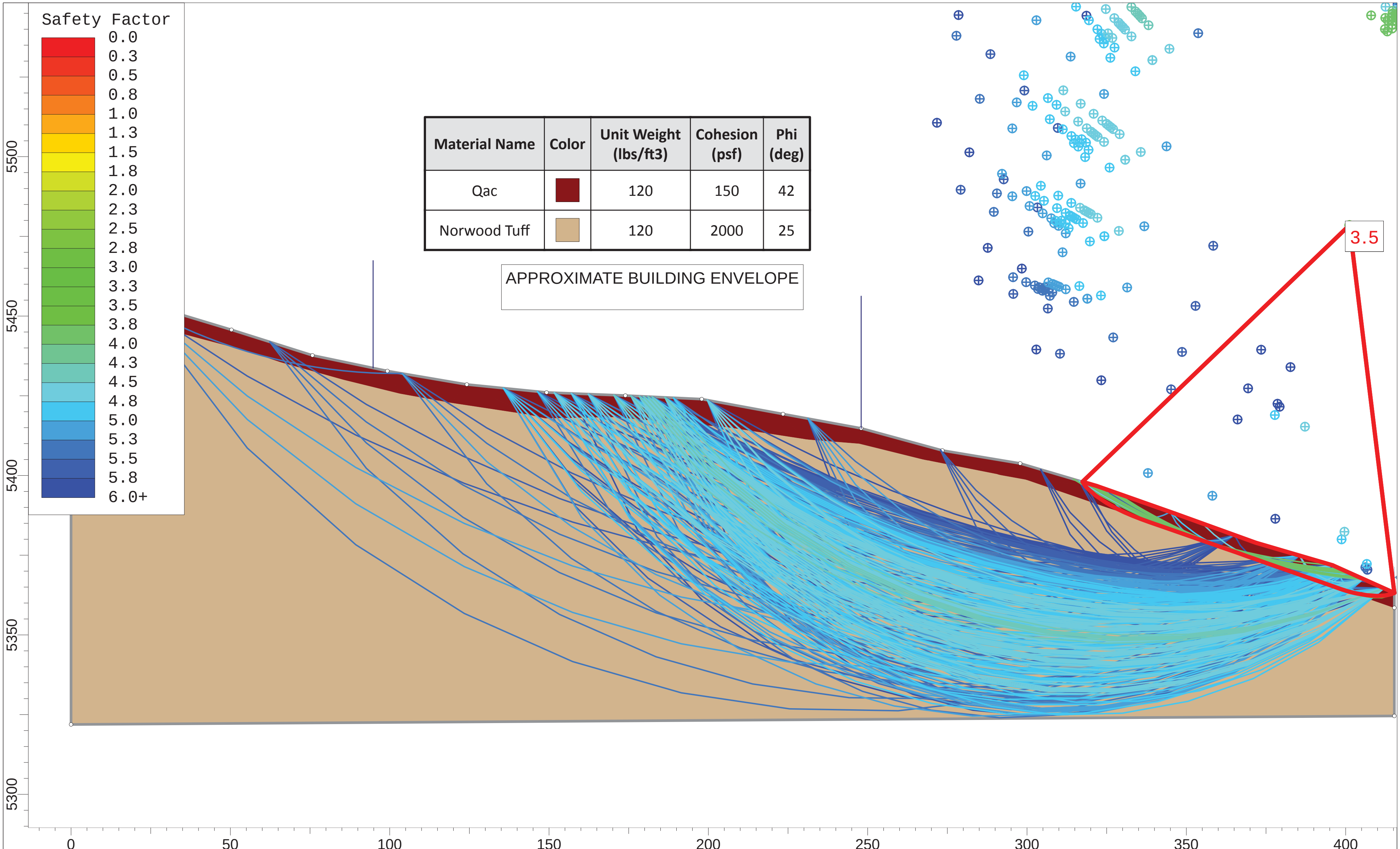
Hoek-Brown Criterion
mb = 0.861   s = 0.0002   a = 0.533

Mohr-Coulomb Fit
cohesion = 2.037 ksf   friction angle = 24.87 deg

Rock Mass Parameters
tensile strength = -0.015 ksf
uniaxial compressive strength = 0.663 ksf
global strength = 6.378 ksf
deformation modulus = 13564.50 ksf





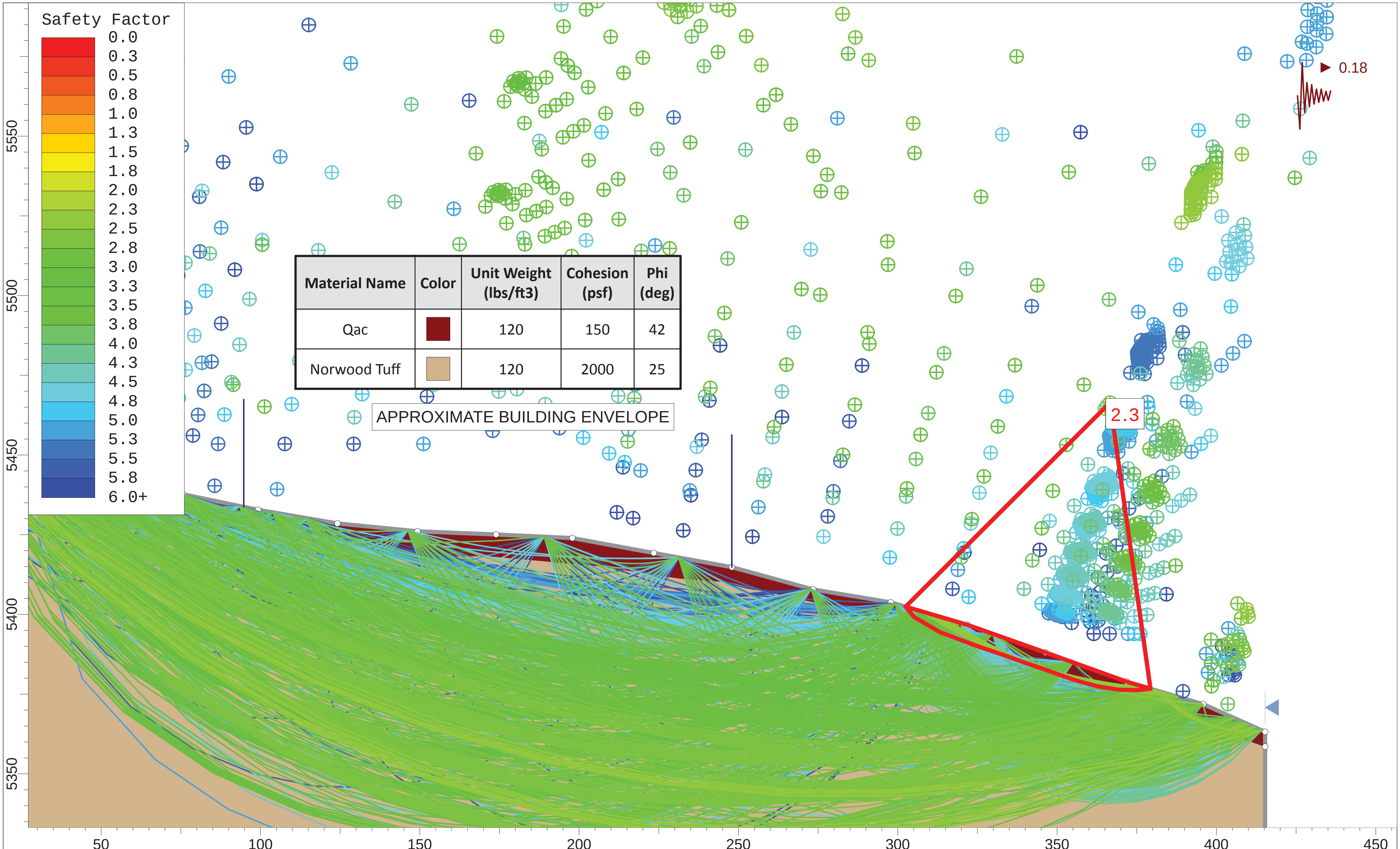


Material Name	Color	Unit Weight (lbs/ft3)	Cohesion (psf)	Phi (deg)
Qac		120	150	42
Norwood Tuff		120	2000	25

IGES®

SLIDEINTERPRET 7.025

Project	Parcel 200350039		
Analysis Description	Section A-A' - Static		
Drawn By TQH	*Scale* 1:500	*Company* IGES, Inc.	
Date 7/18/2017		*File Name* Section A-A' Static.slim	

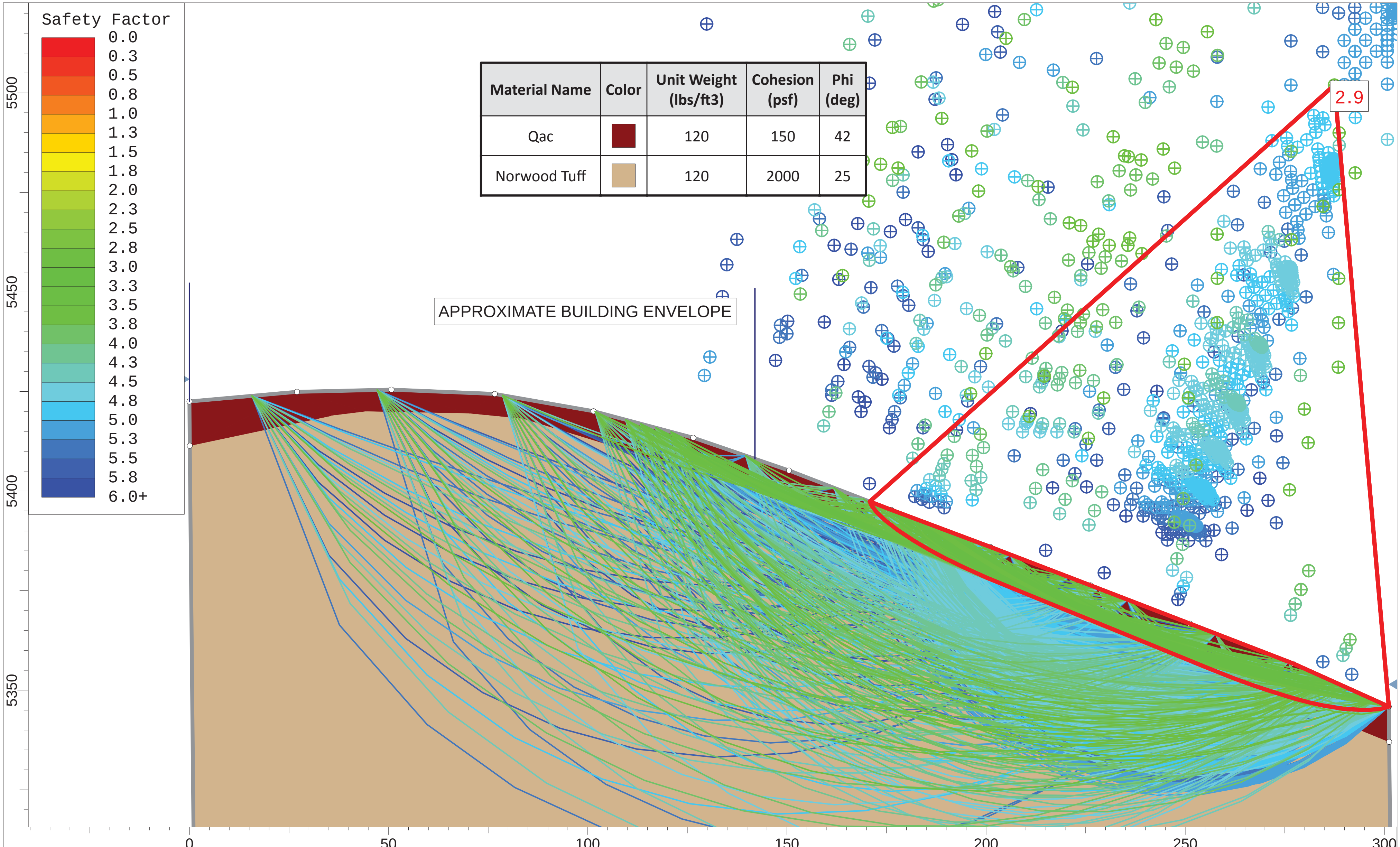


Material Name	Color	Unit Weight (lbs/ft3)	Cohesion (psf)	Phi (deg)
Qac		120	150	42
Norwood Tuff		120	2000	25

IGES®

SLIDEINTERPRET 7.025

Project	Parcel 200350039
Analysis Description	Section A-A' - Seismic
Drawn By	TQH
Scale	1:500
Company	IGES, Inc.
Date	7/18/2017
File Name	Section A-A' Static.slim

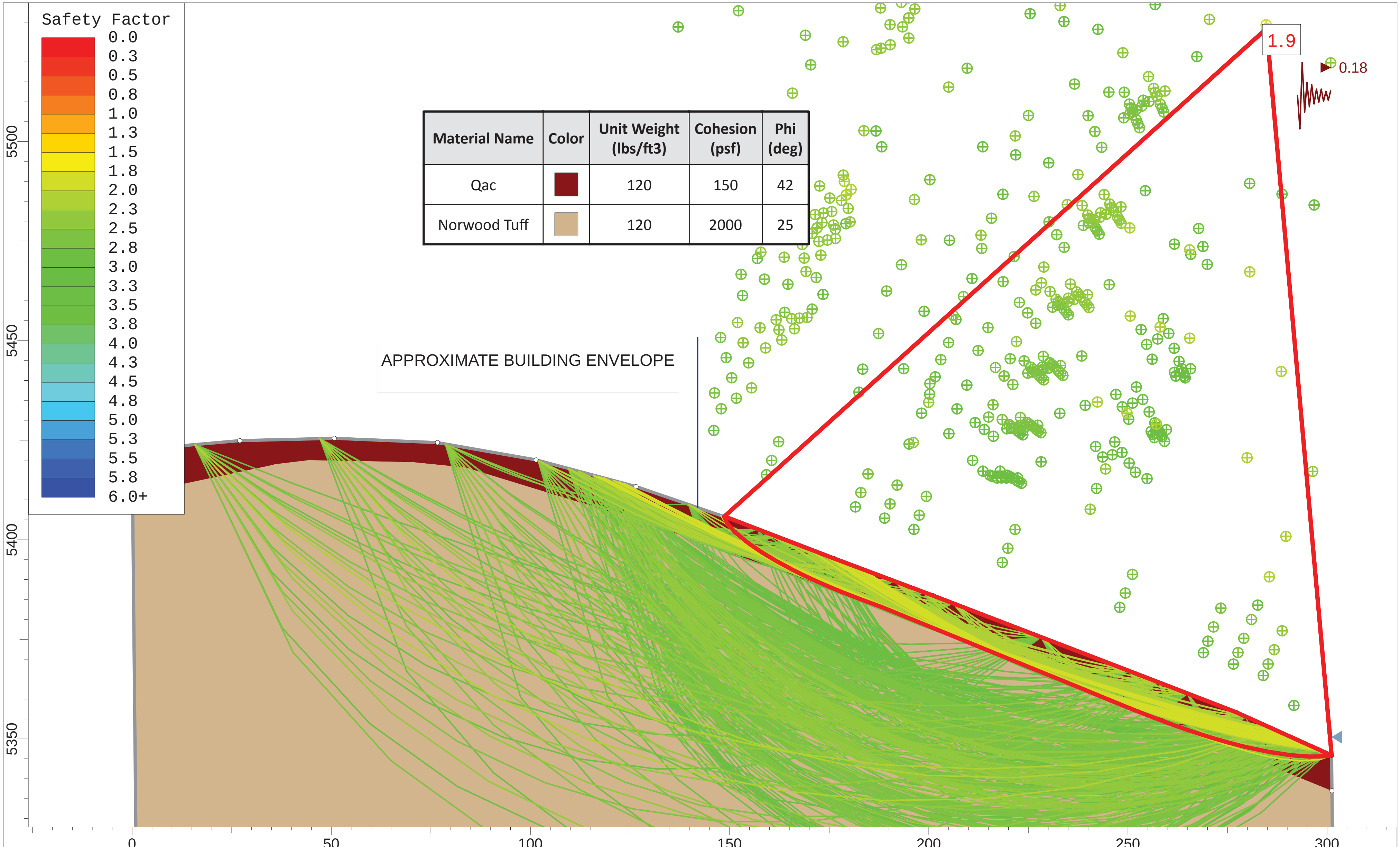


Material Name	Color	Unit Weight (lbs/ft3)	Cohesion (psf)	Phi (deg)
Qac		120	150	42
Norwood Tuff		120	2000	25

IGES®

SLIDEINTERPRET 7.025

Project	Parcel 200350039		
Analysis Description	Section B-B' - Static		
Drawn By TQH	*Scale* 1:400	*Company*	IGES, Inc.
Date	7/18/2017	*File Name*	Section B-B'.slim

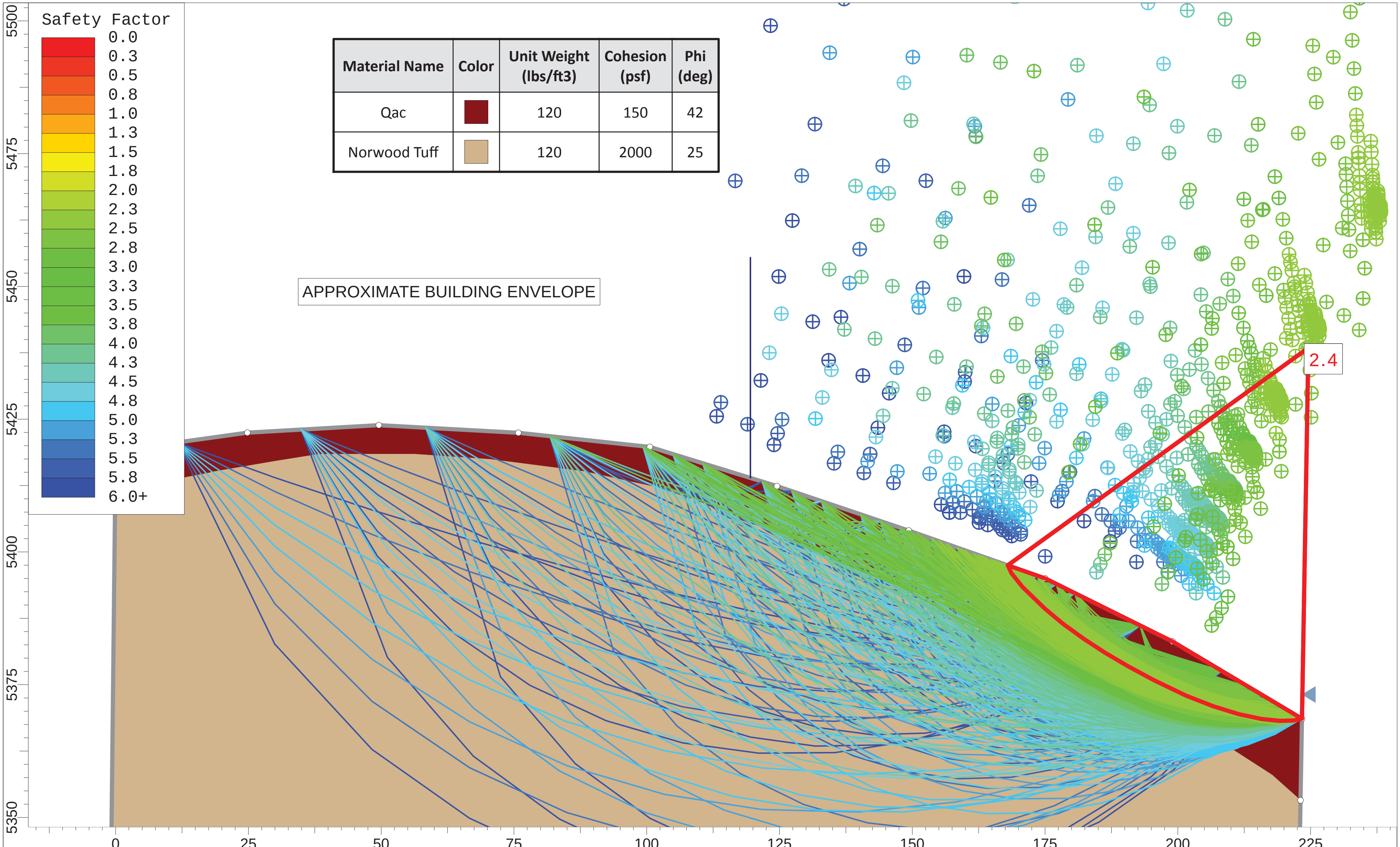


Material Name	Color	Unit Weight (lbs/ft3)	Cohesion (psf)	Phi (deg)
Qac		120	150	42
Norwood Tuff		120	2000	25

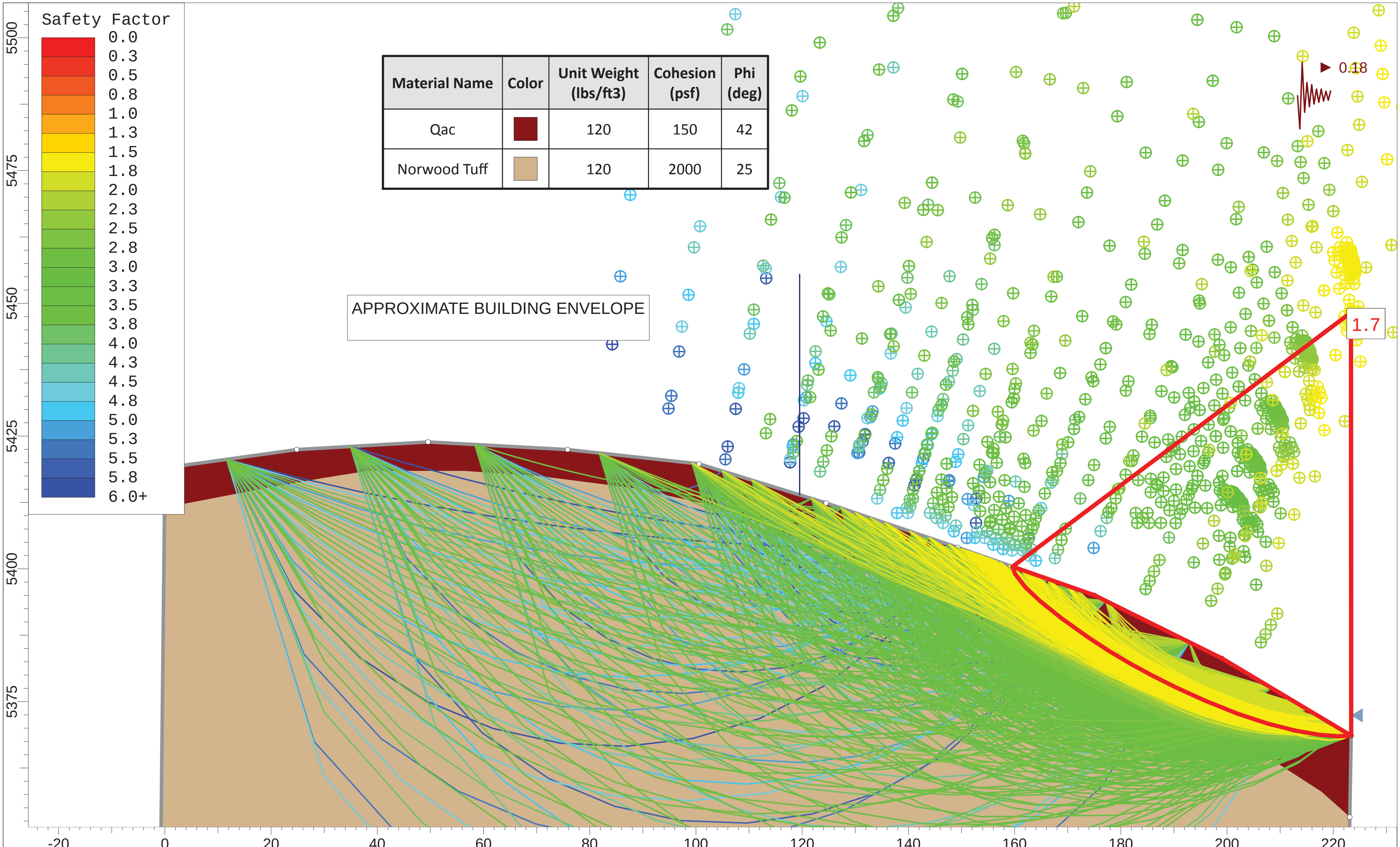
IGES®

SLIDEINTERPRET 7.025

Project	Parcel 200350039		
Analysis Description	Section B-B' - Seismic		
Drawn By TQH	*Scale* 1:400	*Company* IGES, Inc.	
Date 7/18/2017		*File Name* Section B-B'.slim	



IGES®	*Project* Parcel 200350039		
	Analysis Description Section C-C' - Static		
	Drawn By TQH	*Scale* 1:300	*Company* IGES, Inc.
SLIDEINTERPRET 7.025	*Date* 7/18/2017		*File Name* Section C-C'.slim



Material Name	Color	Unit Weight (lbs/ft3)	Cohesion (psf)	Phi (deg)
Qac		120	150	42
Norwood Tuff		120	2000	25

IGES®

SLIDEINTERPRET 7.025

Project	Parcel 200350039		
Analysis Description	Section C-C' - Seismic		
Drawn By TQH	*Scale* 1:300	*Company*	IGES, Inc.
Date	7/18/2017	*File Name*	Section C-C'.slim