CMT ENGINEERING LABORATORIES

Geotechnical Engineering and Geological Reconnaissance Study Proposed Single-Family Residence 21.72-Acre Parcel #20-015-0012 About 5535 East Highway 39 Huntsville, Weber County, Utah

PREPARED FOR:

Mr. Todd Merrill
1691 Martinet Lane
Ogden Utah 84403

PREPARED BY:

CMT Engineering Laboratories

CMT Project No. 12670

June 19, 2019

CMT ENGINEERING LABORATORIES

June 19, 2019

Mr. Todd Merrill
1691 Martinet Lane
Ogden Utah 84403

Subject: Geotechnical Engineering and Geological Reconnaissance Study
Proposed Single-Family Residence
21.72-Acre Parcel #20-015-0012
About 5535 East Highway 39
Huntsville, Weber County, Utah

Mr. Merrill:

Submitted herewith is the report of our geotechnical engineering and geological reconnaissance study for the subject site. This report contains the results of our findings and an interpretation of the results with respect to the available project characteristics. It also contains recommendations to aid in the design and construction of the earth related phases of this project.

CMT Engineering Laboratories (CMT) personnel supervised the excavation of an exploration trench and two test pits extending to depths of approximately 5.0 to 12.0 feet below the existing ground surface at the proposed residence location, and two geotechnical soil bore holes were advanced to auger refusal at 12.0 feet and 13.0 feet on the site. Soil samples were obtained during the field operations and subsequently transported to our laboratory for further testing. Based on the findings of the subsurface explorations, conventional spread and continuous footings may be utilized to support the proposed residence, provided the recommendations in this report are followed. A detailed discussion of design and construction criteria is presented in this report.

We appreciate the opportunity to work with you on this project. CMT offers a full range of Geotechnical Engineering, Geological, Material Testing, Special Inspection services, and Phase I and II Environmental Site Assessments. With nine offices throughout Northern Utah, and in Arizona, our staff is capable of efficiently serving your project needs. If we can be of further assistance or if you have any questions regarding this project, please do not hesitate to contact us at (801) 870-6730. To schedule materials testing please call (801) 908-5859.

Sincerely,
CMT Engineering Laboratories

LICENSED PROFESSIONAL GEOLOGIST GREGORY C. SCHLENKER 5224720-2250 STATE OF UTAH

Gregory C. Schlenker, PhD, PG
State of Utah No. 5224720
Senior Engineering Geologist

LICENSED PROFESSIONAL ENGINEER # 374995 JEFFREY J. EGBERT 6/19/19 STATE OF UTAH

Jeffrey J. Egbert, P.E., LEED A.P., M. ASCE
State of Utah No. 375995
Senior Geotechnical Engineer

Table of Contents

APPENDIX

1.0 INTRODUCTION

1.1 General

CMT Engineering Laboratories (CMT) was authorized by Mr. Todd Merrill to conduct a design level geotechnical engineering and reconnaissance level geological study for a proposed single-family residence to be constructed on a 21.72-Acre property, the Merrill Parcel (Parcel #20-015-0012), located at about 5535 East Highway 39 in the Huntsville area, Weber County, Utah. The parcel is located on the south side of Ogden Valley as shown on Figure 1, Vicinity Map, and more detailed aerial coverage of the parcel is shown on Figure 2, Site Plan. Geological mapping of the parcel is included on Figure 3, Geological Mapping, and slope-terrain information is provided on Figure 4, LiDAR Analysis. The locations of our test pits and bore holes for our subsurface evaluation is shown on Figure 5, Site Evaluation.

The Merrill Parcel is presently occupied by an existing residence. It is our understanding the owners of the property wish to subdivide the parcel, and construct an additional residence at the location shown on Figure 2 and Figure 5. The subject parcel and surrounding properties are zoned by Weber County as Forest Zone FV-3 (Forest Valley Zone - 3) land-use zone. According to the Weber County Code of Ordinances (Weber County, 2019) *the purpose of the Forest Valley Zone, FV-3 is to provide area for residential development in a forest setting at a low density, as well as to protect as much as possible the naturalistic environment of the development.* The prescribed minimum building lot area in the FV-3 Zone is 3 acres (excluding cluster type provision areas), with single-family residences included as a permitted use.

1.2 Objectives and Scope

The objectives and scope of our study were planned in discussions between Mr. Merrill and Mr. Andrew Harris of CMT Engineering Laboratories (CMT). In general, the objectives of this study were to:

1. To conduct a design level geotechnical study and reconnaissance level geologic study for the proposed design and construction in accordance with Weber County Code, Section 108-22 Natural Hazard Areas guidelines and standards (Weber County, 2019).
2. To define and evaluate the subsurface soil, groundwater, and slope stability conditions on the site.
3. To provide appropriate foundation and earthwork recommendations as well as geoseismic information to be utilized in the design and construction of the proposed residence.

To achieve these objectives our scope of work included:

1. Geological reconnaissance studies to assess whether all or parts of the site are exposed to the hazards that are included in the Weber County Code, Section 108-22 Natural Hazard Areas. These hazards include, but are not limited to: Surface-Fault Ruptures, Landslide, Tectonic Subsidence, Rock Fall, Debris Flows, Liquefaction Areas, Flood, or other Hazardous Areas.
2. A field program consisting of the excavating, logging, and sampling of one exploration trench, two geotechnical test pits, two geotechnical bore holes, which included sampling of the subsurface soils encountered.
3. A laboratory soils testing program.
4. An office program consisting of the correlation of available data, engineering and geological analyses, and the preparation of this summary report.

1.3 Authorization

Authorization was provided by Mr. Merrill by returning a signed copy of our Proposal dated April 4, 2019.

2.0 EXECUTIVE SUMMARY

The following is a brief summary of our findings and conclusions:

The results of our analyses indicate that the proposed residence may be supported upon conventional spread and/or continuous wall foundations established upon 18 inches of structural fill extending to suitable natural soils.

The most significant geotechnical/geological aspects of the site are:

1. The proposed residence location is located partly upon mapped Holocene age colluvial landslide deposits (**Qmc**). The Holocene age colluvial landslide deposits (**Qmc**) are considered presently inactive under the existing site slope conditions. The proposed residence is to be located upon a moderately steep slope, 22.7 percent slope, and our site-specific slope stability analysis found the site slopes to be stable under both presently static and future dynamic-seismic loading conditions.

2. The natural clay soils encountered have some moisture sensitivity in the form of a slight potential to experience additional settlement (collapse) when wetted. To help minimize the effect of potentially collapsible in-situ natural soils on the foundations, we recommend that 18 inches of granular structural replacement fill be placed directly below footings and a minimum of 12 inches of granular structural fill be placed below structural slabs-on-grade.

The site for the proposed residence was found to include Holocene age colluvial landslide deposits (**Qmc**), and lower Oligocene and upper Eocene age Norwood Formation (**Tn**), a tuffaceous country rock, surrounds and underlies the parcel as mapped by Utah Geological Survey (UGS) geologists (King and others, 2008; Coogan and King, 2016). The surface of the site slopes moderately (22.7 percent) to the east. Groundwater was not encountered in our subsurface explorations to the maximum depth explored of about 13 feet, and static groundwater is projected to be below project depths, on the order of about 15 to 20 feet for the site. The soils encountered in the explorations ranged from fine grained to very coarse.

A site-specific slope stability study was performed for the proposed residence location, and found that the slope in its present configuration is stable for both static and seismic loading conditions. Where the homesite construction is proposed, the site slope is moderately sloping and less than 25 percent. We recommend that CMT be provided with grading plans when available to further assess stability for proposed cuts and fills associated with construction of a residence.

A geotechnical engineer from CMT will need to verify that all non-engineered fill material and topsoil/disturbed soils have been completely removed and suitable natural soils encountered prior to the placement of structural fills, floor slabs, footings, foundations, or rigid pavements.

In the following sections, detailed discussions pertaining to proposed construction, field exploration, the geologic setting and mapped hazards, geoseismic setting of the site, earthwork, foundations, lateral pressure and resistance, floor slabs, and subdrains are provided.

3.0 DESCRIPTION OF PROPOSED CONSTRUCTION

The proposed project consists of the construction of a single-family residence on the parcel as shown on Figure 2 and Figure 5. The structure is to be of wood-framed construction and founded on spread footings with a basement (if conditions allow). Maximum continuous wall and column loads are anticipated to be 1 to 3 kips per lineal foot and 10 to 50 kips, respectively.

Site development will require a moderate amount of earthwork in the form of site grading. We estimate in general that maximum cuts and fills to achieve design grades will be on the order of 3.0 to 6.0 feet. Projected site grading is anticipated to consist primarily of cutting into the existing ground to construct the residence, with very little fill projected for the site. Larger cuts and fills may be required in isolated areas. Final cuts and fills must be designed to maintain stability of the slopes at the site and not steepen the slope greater than four horizontal to one vertical (4H:1V), and all planned retaining walls will need to be properly engineered.

4.0 FIELD EXPLORATION AND SITE CONDITIONS

The site subsurface soil conditions were explored by excavating a 100-foot long trench, two test pits, and by drilling two bore holes on the site at the locations shown on Figure 5. For our subsurface explorations the trench and test pits were excavated using an 8-ton class rubber-tired excavator with a 24-inch bucket, and the bore holes were drilled with a CME 55 truck-mounted drilling rig using hollow stem auger procedures. The trench and test pits were excavated on April, 26, 2019, and the bore holes were drilled April 24, 2019. The trench and test pits extended to depths of approximately 5.0 to 13.0 feet below the existing ground surface, at which point excavation was either stopped or refused. The two bore holes were extended to refusal at about 12.0 feet, and about 13.0 feet, for B-1 and B-2, respectively. During the course of the excavating and drilling operations, a continuous log of the subsurface conditions encountered was maintained. Within the test pits undisturbed tube, block and disturbed bulk samples of the typical soils encountered were obtained for subsequent laboratory testing and examination.

Samples of the subsurface soils encountered in the bore holes were collected at varying depths through the hollow stem drill augers. Relatively undisturbed samples of the subsurface soils were obtained by driving a split-spoon sampler with 2.5-inch outside diameter rings/liners into the undisturbed soils below the drill augers. Disturbed samples were collected utilizing a standard split spoon sampler. This standard split spoon sampler was driven 18 inches into the soils below the drill augers using a 140 pound hammer free-falling a distance of 30 inches. The number of hammer blows needed for each 6 inch interval was recorded. The sum of the hammer blows for the final 12 inches of penetration is known as a standard penetration test and this 'blow count' was recorded on the bore hole logs. Where more than 50 blows occurred before the 6 inch interval was achieved, the sampling was terminated and the number of blows and inches penetrated by the sampler were recorded. The blow count provides a reasonable approximation of the relative density of granular soils, but only a limited indication of the relative consistency of fine grained soils because the consistency of these soils is significantly influenced by the moisture content.

The collected samples were logged and described in general accordance with ASTM standard 2488, seal in plastic bags and containers, and transported to our laboratory. The soils were classified in the field based upon visual and textural examination. These classifications have been supplemented by subsequent inspection and testing in our laboratory. The subsurface conditions encountered during the field explorations are discussed below in Section 5.4. The exposures encountered in the trench were logged in the field at a scale of 1-inch equal to 5-feet and is presented on **Figure 6 Log of Trench**. The trench was excavated and logged to evaluate the vertical and lateral soils and rock conditions underling the proposed residence location, and to evaluate the presence or absence of evidence of past movement or deformation of the soils and rock underlying the residence location. Vertical stratigraphic logs of the subsurface soil conditions encountered in the test pits and bore holes and are illustrated on **Figures 7 and 8**, **Test Pit Logs**, and **Figures 9 and 10**, **Bore Hole Logs**. Sampling information and other pertinent data and observations are also included on the logs. In addition, a **Key to Symbols** defining the terms and symbols used on the stratigraphic logs is provided as **Figure 11** in this report.

Following completion of the excavating and logging, the trench and test pits were backfilled with the excavated soils. The backfill was not placed in uniform lifts and compacted to a specific density and therefore must be considered as non-engineered backfill. Settlement of the backfill with time is likely to occur.

5.0 ENGINEERING GEOLOGY

5.1 General Geology

The site is located in Ogden Valley which is a northwest trending fault bounded graben structure, with the Wasatch Range comprising the western flank of the valley and the Bear River Range the eastern flank (Avery, 1994). The site is located on the southern margin of Ogden Valley, on the east side of the Wasatch Range, which western side the Wasatch Front is marked by the Wasatch fault. The Wasatch fault is approximately 5.9 miles west of the site, and provides the basis of division between the Middle Rocky Mountain Physiographic Province on the east and the Basin and Range Physiographic Province on the west. The Basin and Range Physiographic Province is characterized by approximately north-south trending valleys and mountain ranges that have been formed by extensional tectonics and displacement along normal faults and extends from the Wasatch Range on the east to the Sierra Nevada Range on the west (Hunt, 1967).

The Middle Rocky Mountain province covers parts of Utah, Colorado, Wyoming, Idaho, and Montana. The geology of the province is an assemblage of sedimentary, igneous, and metamorphic rocks that have been folded, faulted, and uplifted. Mountain building (tectonic) activity commenced about 30 million years ago (Cretaceous time) and continues to the present. The province is characterized by mountainous terrain with deep canyons and broad intervening basins, with temperate semi-arid to mesic climatic conditions (Hunt, 1967).

The site is located on the eastern flank of Mount Ogden which western flank comprises the Wasatch Front. The surficial geology of the site vicinity is the result of the uplift and exposure of older pre-Cambrian rocks which forms the crest of Mount Ogden east of the site. This exposure was the result of movement along high-angle faults during late Tertiary and Quaternary age (Bryant, 1988). Bounding the east foothill flank of Mount Ogden are mid Tertiary units of the Norwood Formation that ramp along the base of the mountains south and west of the Ogden Valley floor. The Norwood Formation is described as "light-gray to light brown, altered tuff (claystone), tuffaceous siltstone, sandstone, and conglomerate" derived from volcanic ash deposition, and has been measured to be as much as 7000 feet thick in the vicinity of the site (King and others, 2008). The existing surface of the site and vicinity appears to have been modified by Quaternary age erosion, and localized late-Quaternary stream, lacustrine (Currey and Oviatt, 1985), residual soil weathering and development, and mass movement processes (King and others, 2008).

Topographically the site is located on base foothills on the northeast side of Mount Ogden, and overlooks Ogden Valley and the South Fork of the Ogden River floodplain, which is inundated by Pineview Reservoir waters, to the north of the site. The current geological mapping drawn from King and others (2008) of the site is shown on Figure 3.

5.2 Site Surface Conditions

The site conditions and site geology were interpreted through an integrated compilation of data, including a review of literature and mapping from previous studies conducted in the area (Bryant, 1988; Sorensen and Crittenden, 1979; King and others, 2008; and Coogan and King, 2016); photogeologic analyses of 2012, 2014 and 2018 imagery shown on **Figure 2**; historical 1:20,000 stereoscopic imagery flown in 1946; GIS analyses of elevation and geoprocessed LiDAR terrain data as shown on **Figure 4**; field reconnaissance of the general site area; and the interpretation of the test pits made on the site as part of our field program. Geologic Slope Cross Section *A-A'* located on **Figure 5** and illustrated on **Figure 12**, was developed from integrated LiDAR slope data and plan geologic mapping data from **Figure 3**. Seismic hazards information was developed from United States Geologic Survey (USGS) databases (Peterson and others, 2008).

As shown on **Figure 2**, the topography of the site vicinity consists of gentle to moderately steep valley-margin foothill slopes. Vegetative cover at the site is densely wooded with scrub oak and maple tree, with open areas covered with grass, weeds and sage brush. Elevations on the parcel range from 4980 (msl) feet on the north side of the site, to 5330 feet on the south side of the site, with the proposed residence at approximately 5178 feet. The site slopes developed from our LiDAR analysis were found to range from near-level to over 100-percent as shown on **Figure 4**. For the proposed residence location, the slope gradients averaged 22.7 percent.

5.3 Surficial Geology

The surficial geology of the site is presented on **Figure 3**, of this report and has been taken from mapping prepared by King and others (2008). A summary of the mapping units identified on the site vicinity and described by King and others (2008) are paraphrased below in relative age sequence (youngest-top to oldest bottom):

> **Qac -** Alluvium and colluvium (Holocene and Pleistocene) - Includes stream and fan alluvium, colluvium, and locally mass-movement deposits...
>
> **Qmc -** Landslide and slump, and colluvial deposits, undivided (Holocene and Pleistocene)... (slopewash and soil creep)...

Qmsy - Younger landslide and slump deposits (Holocene) - Poorly sorted clay- to boulder-sized material...

Qms - Landslide and slump deposits (Holocene and Pleistocene) - Poorly sorted clay- to boulder-sized material...

Qms?(ZYpp?) - Landslide and slump deposits (Holocene and Pleistocene) - over Formation of Perry Canyon bedrock (Neoproterozoic)...

Ql? - Lacustrine silt, sand and gravel deposits (Pleistocene)...

Qlf/Tn - Lacustrine fine grained deposits (Pleistocene) - over Norwood Formation rocks...

Tn- Norwood Formation (lower Oligocene and upper Eocene) - Typically light-gray to light brown, altered tuff (claystone), tuffaceous siltstone, sandstone, and conglomerate...

The approximate proposed residence location shown on **Figure 3** indicates the site to be on **Qmc -** Landslide and slump, and colluvial deposits, undivided (Holocene and Pleistocene)... (slopewash and soil creep)... and, **Qlf/Tn -** Lacustrine fine grained deposits (Pleistocene) - over Norwood Formation rocks... Based upon our findings from this evaluation, these deposits (**Qmc** and **Qlf/Tn**) are considered presently inactive under the existing site slope conditions.

5.4 Subsurface Soil Conditions

Subsurface conditions encountered in the trench and two test pit locations were relatively consistent, and exposed native coarse and cohesive materials. The generalized bottom to top sequence of the soils observed in the excavations consisted of: 1) clays with coarse sand and gravel CL, with mottled olive-FeOx coloration, correlated to weathered Norwood Formation (**Tn**) rocks; 2) sandy silts ML and silty sands SM, with gravels and cobbles, reddish brown in color, attributed to the lacustrine deposition (**Qlf/Tn**); and with 3) silty clay with fine sand CL, dark gray brown in color near the surface of the excavations, and corresponding to colluvial slope wash deposits of the **Qmc** mapping unit.

Surficial topsoil, Soil A-B horizons, approximately 12.0 inches thick were observed on the surfaces of the trench and test pits.

In Bore Hole B-1, the drill rig was refused within light brown dense silty sand with gravel SM, at a depth of 12.0 feet. Above the silty sand SM, brown stiff clay (CL), 5.0 to 9.0 feet, above which loose clayey sand SC extended to the surface. In Bore Hole B-2, the drill rig was refused within dense gray silty sand (SM) at a depth 13.0 feet, above which was overlain by dense brown clayey gravel GC, 5.5 to 9.5 feet; and with brown stiff clay with sand and gravel

CL, extending from 5.5 feet to the surface. Refusal appears to have occurred upon apparent Norwood Formation (Tn) bedrock.

For a detailed graphical description of the subsurface soils encountered, please refer to Figures 6, 7, 8, 9 and 10 of this report.

5.5 Groundwater

Static groundwater was not observed in the trench, test pits or bore holes during our field program. The local static groundwater elevation is projected to be below project depths by about 15 to 20 feet for the site.

Future seasonal and longer-term groundwater fluctuations should be anticipated for the site, with the highest seasonal levels generally occurring during the late spring and summer months. Numerous other factors such as heavy precipitation, rapid snow-melt, and other unforeseen factors, may also influence ground water elevations at the site.

5.6 Site Subsurface Variations

Based on the results of the subsurface explorations and our experience, variations in the continuity and nature of subsurface conditions should be anticipated. Due to the heterogeneous characteristics of natural soils, caution should be taken in interpolating or extrapolating subsurface conditions beyond the exploratory locations. Seasonal fluctuations in ground water conditions may also occur.

In addition, once the subsurface explorations were completed the trench and test pits were backfilled with the excavated soils, but little effort was made to compact these soils. The backfill soils must be considered non-engineered. Settlement of the backfill in the test pits over time should be anticipated and caution should be exercised when constructing over these locations.

5.7 Seismic Setting

5.7.1 General

Utah municipalities have adopted the International Building Code (IBC) 2015. The IBC 2015 code determines the seismic hazard for a site based upon 2008 mapping of bedrock accelerations prepared by the United States Geologic Survey (USGS) and the soil site class (Peterson and others 2008). The USGS values are presented on maps incorporated into the IBC code and are also available based on latitude and longitude coordinates (grid points).

5.7.2 Active Earthquake Faults

Based upon our review of available maps and literature, no active faults are known to pass through or immediately adjacent to the site. The nearest active (Holocene) earthquake fault to the site is the Weber segment of the Wasatch fault zone (UT2351E) which is located 5.9 miles west of the site (Black and others, 2004). Consequently, fault rupture hazards are not considered present on the site. The Ogden Valley southwestern margin faults (UT2375) is located much closer to the site, approximately 0.5 miles to the west, however the most recent movement along this fault is estimated to be pre-Holocene (<750,000 ybp), and is not considered an active risk to the site (Black and others, 1999).

5.7.3 Soil Class

Due to potential foundation construction being within the upper clay soils sequence we recommend that Site Class C – Dense Soft Rock Soil Profile as defined in Chapter 20 of ASCE 7 (per Section 1613.3.2, Site Class Definitions, of IBC 2015) be utilized.

5.7.4 Strong Ground Motion

Strong ground motion originating from the Wasatch fault or other near-by seismic sources is capable of impacting the site. The Wasatch fault zone is considered active and capable of generating earthquakes as large as magnitude 7.3 (Arabasz and others, 1992). Based on probabilistic estimates (Peterson and others, 2008) queried for the site (41.2549º N., 111.8142º E.) the expected peak horizontal ground acceleration (PGA) on rock from a large earthquake with a ten-percent probability of exceedance in 50 years is as high as 0.16*g*. For a two-percent probability of exceedance in 50 years, the PGA is as high as 0.35*g* for the site.

The a ten-percent probability of exceedance in 50 years event has a return period of 475 years, and the 0.16*g* acceleration for this event corresponds to "strong" perceived shaking with "light" potential damage based on instrument intensity correlations. The two-percent probability of exceedance in 50 years event has a return period of 2475 years, and the 0.35*g* acceleration for this event corresponds to "severe" perceived shaking with "moderate to heavy" potential damage based on instrument intensity correlations (Wald and others, 1999).

Future ground accelerations greater than these are possible at the site but will have a lower probability of occurrence.

5.7.5 Liquefaction

In conjunction with the ground shaking potential of large magnitude seismic events as discussed previously, certain soil units may also possess a potential for liquefaction during a large magnitude event. Liquefaction is a phenomenon whereby loose, saturated, granular soil units lose a significant portion of their shear strength due to excess pore water pressure buildup resulting from dynamic loading, such as that caused by an earthquake. Among other effects, liquefaction can result in densification of such deposits causing settlements of overlying layers after an earthquake as excess pore water pressures are dissipated. Horizontally continuous liquefied layers may also have a potential to spread laterally where sufficient slope or free-face conditions exist. The primary factors affecting liquefaction potential of a soil deposit are: (1) magnitude and duration of seismic ground motions; (2) soil type and consistency; and (3) occurrence and depth to groundwater.

Liquefaction potential hazards have not been studied or mapped for the Ogden Valley area, as has occurred in other parts of northern Utah (Anderson and others 1994). Liquefaction commonly occurs in saturated non-cohesive soils such as alluvium, which conditions are not found on the site, consequently the conditions susceptible to liquefaction do not appear to be present at the site within the depths penetrated.

5.7.6 Tectonic Subsidence

Tectonic Subsidence is surface tilting subsidence that occurs along the boundaries of normal faults in response to surface-faulting earthquakes (Keaton, 1986). Because the site is not located in near proximity to active earthquake faults, tectonic subsidence hazards are not considered a risk to the site.

5.8 Landslide and Slump Deposits

The nearest active landslide units are mapped as **Qms** deposits by King and others (2008), and are located approximately 485 feet to the west of the proposed residence as shown on Figure 3, and should not potentially impact the proposed improvements.

5.9 Sloping Surfaces

The surface slopes of the site vicinity developed from our LiDAR analysis and shown on **Figure 4** range from near-level to over 100-percent. For the proposed residence location, the slope gradients averaged 22.7 percent. The limiting steep slope gradients for development considerations according to the Weber County Code is 25-percent (Weber County Code, 2019). No instances of slope movement on the site were observed during our site evaluation.

5.10 Alluvial Fan - Debris Flow Processes

Alluvial fan deposits indicative of processes including flash flooding and debris flow hazard do not appear to occur on the site. The nearest debris flow process deposits are mapped as **Qafy** by King and others (2008), and occur over a mile southeast of the site. These deposits and processes are not shown on Figure 3, do not appear to be a potential impact to the site location.

5.11 Flooding Hazards

No significant water ways pass in the vicinity of the site and flood insurance rate mapping by Federal Emergency Management Agency for the site vicinity has not been prepared for this area at this time (FEMA, 2015). Local sheet flow, slope wash, and seasonally perched soil water typical of sloping areas should be anticipated for the site, and site improvements.

5.12 Rockfall and Avalanche Hazards

The site is not located down-slope from steep slope areas where such hazards may originate.

6.0 LABORATORY TESTING

6.1 Laboratory Examination

In order to provide data necessary for our engineering analyses, a laboratory testing program was completed. The program included performing moisture, partial gradation, Atterberg limits, and consolidation tests on representative subsurface soil samples. The following paragraphs describe the tests and summarize the test data.

6.1.1 Partial Gradation Test

To aid in classifying the granular soils, a partial gradation test was performed. Results of the test are tabulated below.

Exploration No.	Depth (feet)	Percent Passing		Moisture Content (%)	Soil Classification
		#4	#200		
TP-1	6.0	---	18	14.5	SM
TP-2	9.0	60	13	11.4	SM
TP-2	12.0	74	31	16.1	SC
B-1	2.5	99	35	16.6	SC
B-1	10.0	68	13	11.1	SM
B-2	10.0	90	17	8.5	SM

6.1.2 Atterberg Limits Test

To aid in classifying the soils, Atterberg limit tests were performed on samples of the subsurface soils. Results of the tests are tabulated below:

Exploration No.	Depth (feet)	Liquid Limit (percent)	Plastic Limit (percent)	Plasticity Index (percent)	Soil Classification
TP-2	11.5	63	30	33	SC
B-1	2.5	35	31	19	SC

6.1.3 Consolidation Tests

To provide data necessary for our settlement analyses, a consolidation test was performed on a representative sample of the fine-grained clay soils encountered in TP-1. The results indicate that the tested clay soil has a slight collapse potential (about 1.5%) at its current moisture content when wetted, and will exhibit slightly moderate strength when loaded below the pre-consolidation pressure.

6.1.4 Direct Shear Testing

To provide data necessary for our slope stability analysis, direct shear tests were performed on samples of the subsurface soils collected in the bore holes and test pits. Test results are tabulated below:

Exploration No.	Depth (feet)	Dry Density (lbs/ft^3))	Friction Angle (ϕ)	Apparent Cohesion (lbs/ft^2)	Soil Classification
TP-1 (remolded)	8.0	85.0	35.7	50	SM
TP-2 (undisturbed)	2.0	140.0	27.4	517	CL
B-2	5.0	112.3	31.2	153	CL
B-2	10.0	106.0	34.8	55	SM

7.0 SLOPE STABILITY

The stability of the site was evaluated using limit equilibrium (Simplified Bishop) methods via the computer program *SLIDE* (version 7.0). The configuration we analyzed consisted of the Slope Cross Section *A-A'* shown on **Figure 12.**

The properties of the clay and sand soils observed at the site in the subsurface explorations were based upon direct shear testing of samples collected in the test pits. Accordingly, we estimated the following parameters for use in the stability analyses:

Soil	Unit Weight (lbs/ft^3)	Friction Angle (ϕ)	Apparent Cohesion (lbs/ft^2)
CLAY (CL)	130	27	150
SAND (SC)	130	35	0
Norwood Formation	140	40	50

For the seismic (pseudostatic) analysis, a peak horizontal ground acceleration of 0.35*g* (2% in 50 years) as discussed in Section 5.7.4 was used. To model sustained accelerations at the site, about one-half of this value is typically employed (0.175) as the pseudostatic coefficient for global stability analysis. We also incorporated a water table in the slope model at the transition from the surficial cohesive soils to the underlying granular soils above the presumed bedrock elevation from the bore holes. Typically, the required minimum factors of safety are 1.5 for static conditions and 1.1 for seismic (pseudostatic) conditions. The results of our analyses indicate that the slope in its present configuration meets both these requirements provided our recommendations are followed. The slope stability data are included as Figures 13 and 14, attached. We recommend that CMT be retained to re-evaluate the stability based upon proposed cuts and fills at the site once this information has been developed.

Slope movements or even failure can occur if the slope soils are undermined or become saturated. Any retaining walls must be properly engineered to maintain stability of the slopes. Any changes to the grading at the site must be reviewed by CMT prior to initiation of any construction in order to assess if our findings and recommendations remain applicable. During construction, CMT must observe grading to ensure suitable soil conditions are encountered. Following grading at the site, the slope surface must be revegetated as soon as possible to limit erosion and potential undermining of the slope. The property owner and the owner's representatives should be made aware of the risks involved should these or other conditions occur that could saturate or erode/undermine the slope soils.

8.0 SITE PREPARATION AND GRADING

8.1 Site Preparation

Initial site preparation will consist of the removal of surface vegetation, topsoil, and other deleterious materials and non-engineered fills, if encountered, from beneath an area extending out at least 4 feet from the perimeter of the proposed residence, and 2 feet beyond and exterior flatwork areas.

Based on the soils conditions encountered and the geologic history it is recommended that site cuts be limited such that un-braced site grading slopes remain similar to existing slope (roughly 4H:1V or less) unless further stability evaluation is performed based on proposed cuts and fills.

Subsequent to stripping and prior to the placement of structural site grading fill, driveway, and garage slabs on grade, the prepared subgrade must be proof rolled by passing moderate-weight rubber tire-mounted construction equipment over the surface at least twice. If excessively soft or loose soils are encountered, they must be removed to a maximum depth of 2 feet and replaced with structural fill. Beneath footings, all soft, loose, and disturbed soils must be totally removed. If removal depth required is greater than 2 feet, CMT must be informed to provide further recommendations.

Structures may be constructed directly on bedrock, if encountered. However, where structures will be partly on soil and partly on bedrock it is recommended that site grading be adjusted or additional excavation of bedrock be completed such that a minimum of 12 inches of suitable natural undisturbed soil or structural fill may be placed directly over the portion of exposed bedrock to provide a uniform bearing condition for the structure.

Surface vegetation and other deleterious materials should be removed from the site. Topsoil, although unsuitable for utilization as structural fill, may be stockpiled for subsequent landscaping purposes.

8.2 Temporary Excavations

Temporary excavations up to 8 feet deep in fine-grained cohesive soils, above or below the water table, may be constructed with sideslopes no steeper than one-half horizontal to one vertical (0.5H:1V).

For granular (cohesionless) soils, construction excavations above the water table, not exceeding 4 feet, should be no steeper than one-half horizontal to one vertical (0.5H:1V). For excavations up to 8 feet, in granular soils and above the water table, the slopes should be no steeper than one horizontal to one vertical (1H:1V). Excavations encountering saturated cohesionless soils will be very difficult and will require very flat sideslopes and/or shoring, bracing, and dewatering. Excavations deeper than 8 feet are not anticipated at the site.

The earthwork contractor must be made aware of the bedrock conditions as bedrock excavation will require heavy machinery, chipping, and/or blasting.

All excavations must be inspected periodically by qualified personnel. If any signs of instability or excessive sloughing are noted, immediate remedial action must be initiated. All excavations should be made following OSHA safety guidelines.

8.3 Permanent Cut/Fill Slopes

Based on our slope stability modeling, permanent cut and fill slope must be graded no steeper than four horizontal to one vertical (4H:1V) to be considered stable. If steeper cut and fill slopes are required to facilitate development plans, retaining walls or shoring must be planned. It is anticipated that some retaining walls will be required along the east boundary of the site adjacent to the existing development. If bedrock is encountered the permanent slope restrictions may be reevaluated.

8.4 Fill Material

Structural fill is defined as all fill which will ultimately be subjected to structural loadings, such as imposed by footings, floor slabs, pavements, etc. Structural fill will be required as backfill over foundations and utilities, as site grading fill, and possibly as replacement fill below footings. All structural fill must be free of sod, rubbish, topsoil, frozen soil, and other deleterious materials.

Following are our recommendations for the various fill types we project will be used at this site:

Fill Material Type	Description/Recommended Specification
Structural Fill	Placed below structures, flatwork and pavement. Import structural fill shall consist of a Well-graded sand/gravel mixture, with maximum particle size of 4 inches, a minimum 70% passing 3/4-inch sieve, a maximum 30% passing No. 200 sieve, and a maximum Plasticity Index of 10.
General Site Grading Fill	Placed over larger areas to raise the site grade, with a maximum particle size of 6- inches, a minimum 70% passing 3/4-inch sieve, a maximum 50% passing No. 200 sieve and Plastic Index less than 18 percent.
Non-Structural Fill	Placed below non-structural areas, such as landscaping. On-site soils, including silt/clay soils not containing excessive amounts of degradable/organic material (see discussion below).
Stabilization Fill	Placed to stabilize soft areas prior to placing structural fill and/or site grading fill. Coarse angular gravels and cobbles 1-inch to 8-inches in size. May also use 1.5- to 2.0-inch gravel placed on stabilization fabric, such as Mirafi RS280i or equivalent.

On-site natural granular soils or existing granular fills may be utilized as structural site grading fill provided, they meet with the requirements as stated within this report.

On-site silt/clay soils may be used as site grading fill and non-structural fill, but are also moisture-sensitive. Note that such moisture-sensitive soils are inherently more difficult to work with in proper moisture conditioning (they are very sensitive to changes in moisture content), requiring very close moisture control during placement and compaction. This will be very difficult, if not impossible, during wet and cold periods of the year.

All fill material should be approved by a CMT geotechnical engineer prior to placement.

8.5 Fill Placement and Compaction

The various types of compaction equipment available have their limitations as to the maximum lift thickness that can be compacted. For example, hand operated equipment is limited to lifts of about 4 inches and most "trench compactors" have a maximum, consistent compaction depth of about 6 inches. Large rollers, depending on soil and moisture conditions, can achieve compaction at 8 to 12 inches. The full thickness of each lift should be compacted to at least the following percentages of the maximum dry density as determined by ASTM D-1557 (or AASHTO[1] T-180) in accordance with the following recommendations:

Location	Total Fill Thickness (feet)	Minimum Percentage Of Maximum Dry Density
Beneath an area extending at least 4 feet beyond the perimeter of structures, and below flatwork and pavement (applies to structural fill and site grading fill) extending at least 2 feet beyond the perimeter	0 to 5 5 to 8	95 98
Site grading fill outside area defined above	0 to 5 5 to 8	92 95
Utility trenches within structural areas	--	96
Roadbase and subbase	-	96
Non-structural fill	0 to 5 5 to 8	90 92

Structural fills below buildings greater than 8 feet thick are not anticipated at the site. For best compaction results, we recommend that the moisture content for structural fill/backfill be within 2% of optimum. Field density tests should be performed on each lift as necessary to verify that proper compaction is being achieved.

Fill materials must be placed on horizontal benches a minimum of 5 feet wide and 2 feet high. Benching of existing slope will be required prior to the placement of site grading fills.

Embankment fills for roadways greater than 8 feet must be moisture conditioned to optimum or above and compacted to 98 percent with respect to the above criteria. Also, settlement monitoring will likely be required and should be evaluated on a case by case basis.

[1] American Association of State Highway and Transportation Officials

8.6 Utility Trenches

For the bedding zone around the utility, we recommend utilizing sand bedding fill material that meets current APWA[2] requirements.

Most utility companies and local governments are requiring Type A-1a or A-1b (AASHTO Designation) soils (sand/gravel soils with limited fines) be used as backfill over utilities within public rights of way, and the backfill be compacted over the full depth above the bedding zone to at least 96% of the maximum dry density as determined by AASHTO T-180 (ASTM D-1557). Existing sand and gravel soils at this site may meet these specifications with some processing.

Where the utility does not underlie structurally loaded facilities and public rights of way, on-site fill and natural soils may be utilized as trench backfill above the bedding layer, provided they are properly moisture conditioned and compacted to the minimum requirements stated above in **Section 6.4**.

8.7 Stabilization

The natural silt/clay soils and some existing fill soils with high fines portions at this site will likely be susceptible to rutting and pumping. The likelihood of disturbance or rutting and/or pumping of the existing natural soils is a function of the load applied to the surface, as well as the frequency of the load. Consequently, rutting and pumping can be minimized by avoiding concentrated traffic, minimizing the load applied to the surface by using lighter equipment and/or partial loads, by working in drier times of the year, or by providing a working surface for the equipment. Rubber-tired equipment particularly, because of high pressures, promotes instability in moist/wet, soft soils. If rutting or pumping occurs, traffic should be stopped and the disturbed soils should be removed and replaced with stabilization material. Typically, a minimum of 18 inches of the disturbed soils must be removed to be effective. However, deeper removal is sometimes required.

To stabilize soft subgrade conditions (if encountered), a mixture of coarse, clean, angular gravels and cobbles and/or 1.5- to 2.0-inch clean gravel should be utilized, as indicated above in **Section 6.3**. Often the amount of gravelly material can be reduced with the use of a geotextile fabric such as Mirafi RS280i or equivalent. Its use will also help avoid mixing of the subgrade soils with the gravelly material. After excavating the soft/disturbed soils, the fabric should be spread across the bottom of the excavation and up the sides a minimum of 18 inches. Otherwise, it should be placed in accordance with the manufacturer's recommendation, including proper overlaps. The gravel material can then be placed over the fabric in compacted lifts as described above.

---

[2] American Public Works Association

9.0 LATERAL EARTH PRESSURES

The lateral pressure parameters, as presented within this section, are for backfills which will consist of clean, drained on site or imported granular soil placed and compacted in accordance with the recommendations presented herein. The lateral pressures imposed upon subgrade facilities will, therefore, be basically dependent upon the relative rigidity and movement of the backfilled structure. For active walls, such as retaining walls which can move outward (away from the backfill), backfill may be considered equivalent to a fluid with a density of 45 pounds per cubic foot in computing lateral pressures. For more rigid walls (moderately yielding), generally not exceeding 8 feet in height, backfill may be considered equivalent to a fluid with a density of 55 pounds per cubic foot. The above values assume that the surface of the soils slope behind the wall is no steeper than 5 horizontal to 1 vertical and that the fill within 3 feet of the wall will be compacted with hand-operated compacting equipment.

For seismic loading, a uniform pressure should be added. The uniform pressures based on different wall heights are provided in the following table:

Wall Height (feet)	Seismic Loading Active Case (psf)	Seismic Loading Moderately Yielding (psf)
4	15	30
6	17	45
8	25	60

10.0 FOUNDATION RECOMMENDATIONS

The following recommendations have been developed on the basis of the previously described project characteristics, the subsurface conditions observed in the field, the laboratory test data, as well as common engineering practice.

10.1 Foundation Recommendations

The proposed residence may be supported upon conventional spread and continuous wall foundations established upon 18 inches of granular structural fill extending to suitable natural soil. For design, the following parameters are provided:

Minimum Recommended Depth of Embedment for
Frost Protection - 36 inches

Minimum Recommended Depth of Embedment for Non-frost Conditions	-	15 inches
Recommended Minimum Width for Continuous Wall Footings	-	16 inches
Minimum Recommended Width for Isolated Spread Footings	-	24 inches
Recommended Net Bearing Pressure for Real Load Conditions Established on 18 inches of Structural Fill Extending To Suitable Natural Soil	-	1,500 pounds per square foot
Bearing Pressure Increase for Seismic Loading	-	30 percent

The term "net bearing pressure" refers to the pressure imposed by the portion of the structure located above lowest adjacent final grade. Therefore, the weight of the footing and backfill to lowest adjacent final grade need not be considered. Real loads are defined as the total of all dead plus frequently applied live loads. Total load includes all dead and live loads, including seismic and wind.

10.2 Installation

Footings shall not be installed upon soft or disturbed soils, non-engineered fill, construction debris, frozen soil, or within ponded water. Additionally, footing shall be underlain by a minimum 18 inches of granular structural fill meeting the requirements as stated in this report. If the granular structural fill upon which the footings are to be established becomes disturbed, it shall be recompacted to the requirements for structural fill or be removed and replaced with new structural fill.

The width of structural fill, where placed below footings, shall extend laterally at least 6 inches beyond the edges of the footings in all directions for each foot of fill thickness beneath the footings.

10.3 Estimated Settlement

Foundations designed and constructed in accordance with our recommendations could experience some settlement, but we anticipate that settlement of footings founded as recommended above will be 1 inch or less. We expect approximately 50 percent of initial settlement to take place during construction.

10.4 Lateral Resistance

Lateral loads imposed upon foundations due to wind or seismic forces may be resisted by the development of passive earth pressures and friction between the base of the footings and the supporting soils. In determining frictional resistance, a coefficient of 0.40 should be utilized for structural fill. Passive resistance provided by properly placed and compacted granular structural fill above the water table may be considered equivalent to a fluid with a density of 250 pounds per cubic foot.

A combination of passive earth resistance and friction may be utilized provided that the friction component of the total is divided by 1.5.

11.0 FLOOR SLABS

Floor slabs may be established upon a minimum of 12 inches of granular structural fill extending to suitable natural soils. Under no circumstances shall floor slabs be established directly over native clay soils, non-engineered fills, loose or disturbed soils, sod, rubbish, construction debris, other deleterious materials, frozen soils, or within ponded water.

In order to facilitate curing of the concrete, it is recommended that floor slabs be directly underlain by at least 4 inches of "free-draining" fill, such as "pea" gravel or three-quarters to one-inch minus clean gap-graded gravel. This 4 inches may be incorporated as part of the 12 inches of granular structural fill requirement. To help control normal shrinkage and stress cracking, the floor slabs should have the following features:

1. Adequate reinforcement for the anticipated floor loads with the reinforcement continuous through interior floor joints;
2. Frequent crack control joints; and
3. Non-rigid attachment of the slabs to foundation walls and bearing slabs.

12.0 DRAINAGE RECOMMENDATIONS

12.1 Surface Drainage

It is important to the long-term performance of foundations and floor slabs that water not be allowed to collect near the foundation walls and infiltrate into the underlying soils. Additionally, some of the on-site near-surface silty/clayey soils may potentially be collapsible or expansive when subjected to water, thus it is very important to the long-term performance of foundations and floor slabs that water not be allowed to collect near the foundation walls and infiltrate into the underlying soils. We recommend the following:

1. All areas around the structure should be sloped to provide drainage away from the foundations. We recommend a minimum slope of 6 inches in the first 10 feet away from the structure. This slope should be maintained throughout the lifetime of the structure.

2. All roof drainage should be collected in rain gutters with downspouts designed to discharge at least 10 feet from the foundation walls or well beyond the backfill limits, whichever is greater. Further drainage shall be controlled such that it does not affect adjacent properties.

3. Adequate compaction of the foundation backfill should be provided. We suggest a minimum of 90% of the maximum laboratory density as determined by ASTM D-1557. Water consolidation methods should not be used under any circumstances.

4. Landscape sprinklers should be aimed away and maintained a distance of at least 4 feet from the foundation walls. The sprinkling systems should be designed with proper drainage and be well-maintained. Over watering should be avoided.

5. Other precautions that may become evident during construction.

12.2 Foundation Subdrains

Due to the potential for localized “perched” groundwater conditions, especially during the spring and early summer months against foundations, we recommend the installation of foundation subdrains around structures with subgrade levels. The subdrain must gravity daylight well beyond and down gradient of the home and retaining walls.

Foundation subdrains should consist of a 4-inch diameter perforated or slotted solid-plastic or PVC pipe enclosed in clean gravel comprised of three-quarter- to one-inch minus gap graded gravel and/or “pea” gravel. The invert of a subdrain should be at least 18 inches below the top of the lowest adjacent habitable floor slab. The gravel portion of the drain should extend 2 inches laterally and below the perforated pipe and at least 1 foot above the top of the lowest adjacent floor slab. The gravel zone must be installed immediately adjacent to the perimeter footings and the foundation walls. To reduce the possibility of plugging, the gravel must be wrapped with a geotextile, such as Mirafi 140N or equivalent.

Above the foundation subdrain, a minimum 4-inch-wide zone of “free-draining” sand or gravel (chimney) should be placed adjacent to the foundation walls and extend to within 1.5 feet of final grade. The sand/gravel fill must be separated from adjacent native or backfill soils with geotextile fabric (Mirafi 140N or equivalent). The upper 1.5 feet of soils should consist of a compacted clayey soil cap to reduce surface water infiltration into the drain. As an alternative to the zone of permeable sand and gravel, a prefabricated “drainage board,” such as Miradrain or equivalent, may be placed against the exterior below-grade walls. Prior to the installation of the footing subdrain, the below-grade walls should be adequately dampproofed. The slope of the subdrain should be at least 0.3 percent. The foundation subdrains shall be discharged to a suitable down gradient location (I.E. area subdrains, storm drains, etc.).

13.0 QUALITY CONTROL

We recommend that CMT be retained to as part of a comprehensive quality control testing and observation program to help facilitate implementation of our recommendations and to address any subsurface conditions encountered which vary from those described in this report saving both time and expense. Without such a program CMT cannot be responsible for application of our recommendations to subsurface conditions which may vary from those described herein. This may include but not necessarily be limited to the following:

13.1 Field Observations

Observations should be completed during all phases of construction such as site preparation, foundation excavation, structural fill placement and concrete placement.

13.2 Fill Compaction

Compaction testing by CMT is required for all structural supporting fill materials. Maximum Dry Density (Modified Proctor/ASTM D-1557) tests should be requested by the contractor immediately after delivery of any granular fill materials. The maximum density information should then be used for field density tests on each lift as necessary to ensure that the required compaction is being achieved.

13.3 Concrete Quality

We recommend that freshly mixed concrete be tested by CMT in accordance with all applicable standards.

14.0 LIMITATIONS

The recommendations provided herein were developed by evaluating the information obtained from the test pits and site exploration. The exploration data reflects the subsurface conditions only at the specific locations at the particular time designated on the test pit logs. Soil and ground water conditions may differ from conditions encountered at the actual exploration locations. The nature and extent of any variation in the explorations may not become evident until during the course of construction. If variations do appear, it may become necessary to re-evaluate the recommendations of this report after we have observed the variation.

Our professional services have been performed, our findings obtained, and our recommendations prepared in accordance with generally accepted geotechnical engineering principles and practices. This warranty is in lieu of all other warranties, either expressed or implied.

We appreciate the opportunity to be of service to you on this project. CMT offers a full range of Geotechnical Engineering, Geological, Material Testing, Special Inspection services, and Phase I and II Environmental Site Assessments. With 4 offices throughout Northern Utah, and in Arizona, our staff is capable of efficiently serving your project needs. If we can be of further assistance or if you have any questions regarding this project, please do not hesitate to contact us at (801) 870-6730. To schedule materials testing please call (801) 908-5859.

15.0 REFERENCES

Anderson, L.R., Keaton, J.R., and Bay, J.A., 1994, Liquefaction potential map for the northern Wasatch Front, Utah, complete technical report: Utah Geological Survey Contract Report 94-6, 150 p., 6 plates, scale 1:48,000.

Arabasz, W.J., Pechmann, J.C., and Brown, E.D., 1992, Observational seismology and the evaluation of earthquake hazards and risk in the Wasatch Front area, Utah, in Gori, P.L., and Hays, W.W., eds., Assessment of regional earthquake hazards and risk along the Wasatch Front, Utah: U.S. Geological Survey Professional Paper 1500-D, 36 p.

Avery, C., 1994, Ground-water hydrology of Ogden Valley and surrounding area, eastern Weber County, Utah, and simulation of ground-water flor in the valley-fill aquifer system; Utah Department of Natural Resources, Technical Publication no. 99, 84 p.

Black, B.D., and DuRoss, C.B., and Hylland, M.D., and McDonald, G.N., and Hecker, S., compilers, 2004, Fault number 2351e, Wasatch fault zone, Weber section, in Quaternary fault and fold database of the United States: U.S. Geological Survey website, http://earthquakes.usgs.gov/hazards/qfaults

Black, B.D., Hylland, M.D., and Hecker, S., compilers, 1999, Fault number 2375, Ogden Valley southwestern margin faults, in Quaternary fault and fold database of the United States: U.S. Geological Survey website, http://earthquakes.usgs.gov/hazards/qfaults.

Bryant, B.B., 1988, Geology of the Farmington Canyon Complex, Wasatch Mountains, Utah: USGS Professional Paper 1476, 54 p., scale 1:50,000
Coogan, J.C., and King, J.K., 2016, Interim geologic map of the Ogden 30' x 60' quadrangle, Box Elder, Cache, Davis, Morgan, Rich, and Summit Counties, Utah, and Uinta County, Wyoming: Utah Geological Survey Open File Report 653DM, for use at 1:62,500 scale, 3 plates, 147 p.

Currey, D.R., and Oviatt, C.G., 1985, Durations, average rates, and probable causes of Lake Bonneville expansion, still-stands, and contractions during the last deep-lake cycle, 32,000 to 10,000 years ago, in Kay, P.A., and Diaz, H.F., (eds.), Problems of and prospects for predicting Great Salt Lake levels - Processing of a NOAA Conference, March 26-28, 1985: Salt Lake City, Utah.

FEMA, 2015, Flood Insurance Rate Map, 2015 Weber County, Utah, Panel 49057C, Scale 1 inch equals 1000 feet.

Hunt, C.B., 1967, Physiography of the United States. San Francisco, W.H. Freeman, 480 p.

Keaton, J.R., 1986, Potential consequences of tectonic deformation along the Wasatch fault: Utah State University, Final Technical Report to the U.S. Geological Survey for the National Earthquake Hazards Reduction Program, Grant 14-08-0001-G0074, 23 p.

King, J.K., Yonkee, W.A., and Coogan, J.C., 2008, Interim geologic map of the Snow Basin and part of the Huntsville quadrangle, Davis, Morgan, and Weber Counties, Utah: Utah Geological Survey Open-File Report 536, scale 1:24,000.

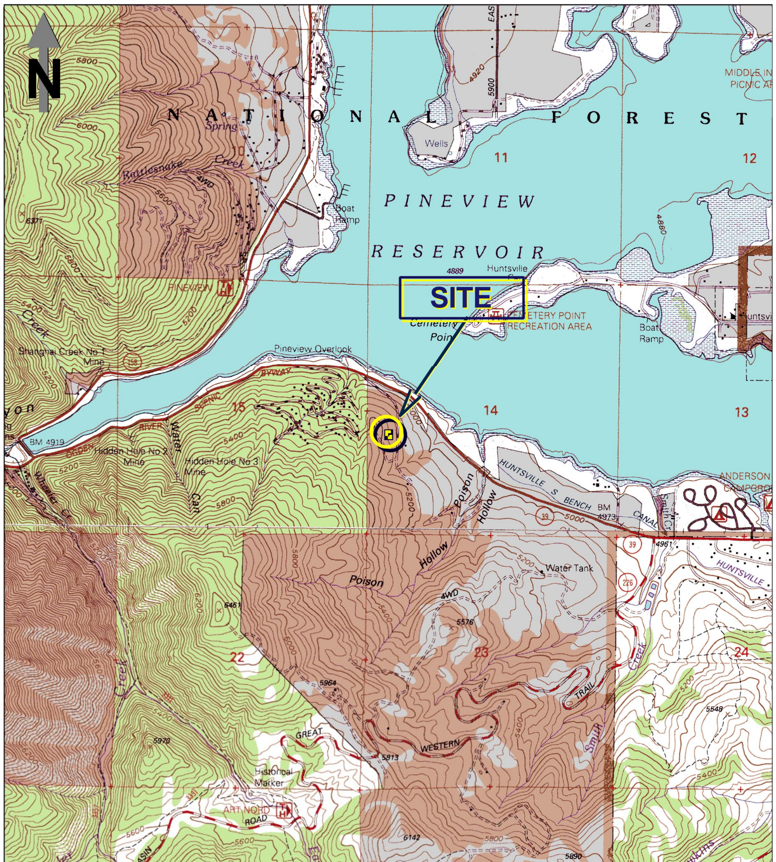
Petersen, M.D., Frankel, A.D., Harmsen, S.C., Mueller, S.C., Haller, K.M., Wheeler, R.L., Wesson, R.L., Zeng, Y., Boyd, O.S., Perkins, D.M., Luco, N., Field, E.H., Wills, C.J., and Rukstales, K.S., 2008, Documentation for the 2008 Update of the United States National Seismic Hazard Maps: USGS Open-File Report 2008-1128, 128p.

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

Wald, D.J., Quitoriano, V., Heaton, T.H., and Kanamori, H., 1999, Relationship between Peak Ground Acceleration, Peak Ground Velocity, and Modified Mercalli Intensity in California: Earthquake Spectra, v. 15, no. 3, p. 557-564

Weber County Code (2019), retrieved from: https://www.municode.com/library/ut/weber_county/codes/code_of_ordinances?nodeId=14935

Appendix

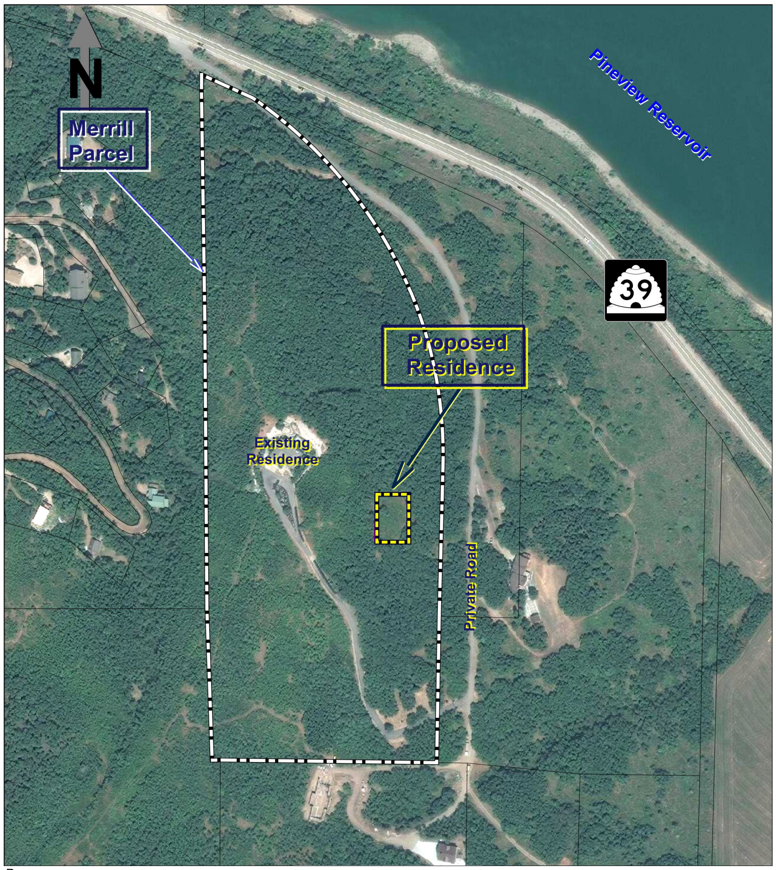


Base:
1998 7.5 Minute USGS Toporgraphic Maps Titled Snowbasin, Utah, and Huntsville, Utah.

0 2000 4000 ft

1:24,000

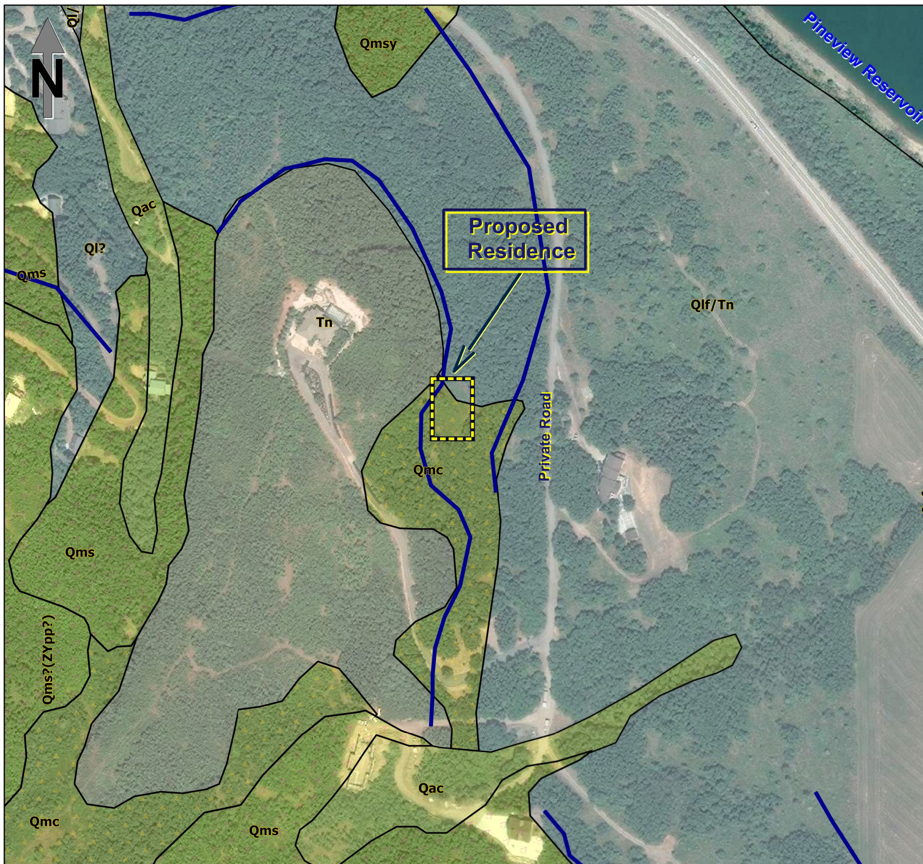
Merrill Residence Huntsville, Utah	CMT ENGINEERING LABORATORIES Vicinity Map	Date: 13 June-19 Job # 12670	Figure 1



Base:
2018 0.4m Color Orthoimagery,
from Terraserver; https://www.terraserver.com/

0 200 400 ft
1:3,000

Merrill Residence Huntsville, Utah	CMT ENGINEERING LABORATORIES		Figure 2
	Site Plan	Date: 13-June-19	
		Job # 12670	



Explanation

Geology after King and Others, 2008

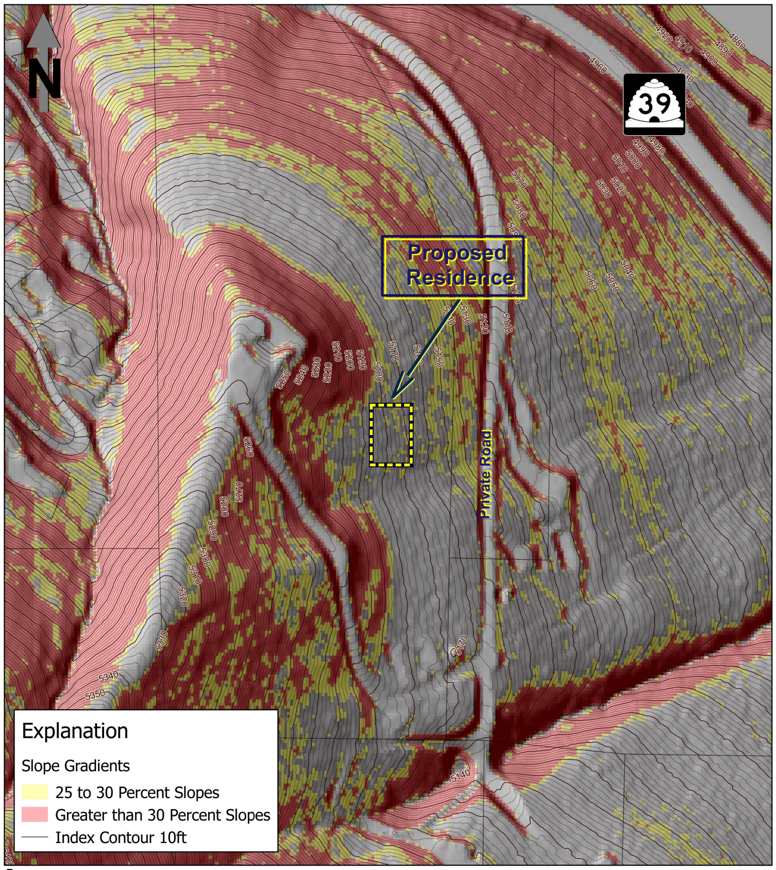
- Qac - Alluvium and colluvium (Holocene and Pleistocene) - Includes stream and fan alluvium, colluvium, and locally mass-movement deposits...
- Qmc - Landslide and slump, and colluvial deposits, undivided (Holocene and Pleistocene)... (slopewash and soil creep)...
- Qmsy - Younger landslide and slump deposits (Holocene) - Poorly sorted clay- to boulder-sized material...
- Qms - Landslide and slump deposits (Holocene and Pleistocene) - Poorly sorted clay- to boulder-sized material...
- Qms?(ZYpp?) - Landslide and slump deposits (Holocene and Pleistocene) - over Formation of Perry Canyon bedrock (Neoproterozoic)...
- Ql? - Lacustrine silt, sand and gravel deposits (Pleistocene)...
- Qlf/Tn - Lacustrine fine grained deposits (Pleistocene) - over Norwood Formation rocks...
- Tn- Norwood Formation (lower Oligocene and upper Eocene) - Typically light-gray to light brown, altered tuff (claystone), tuffaceous siltstone, sandstone, and conglomerate...
- Lake Bonneville Shoreline

Base:
2018 0.4m Color Orthoimagery,
from Terraserver; https://www.terraserver.com/

0 200 400 ft

1:2,400

CMT ENGINEERING LABORATORIES	Merrill Residence Huntsville, Utah	Geologic Mapping	Date:	13-June-19	Figure 3
			Job #	12670	

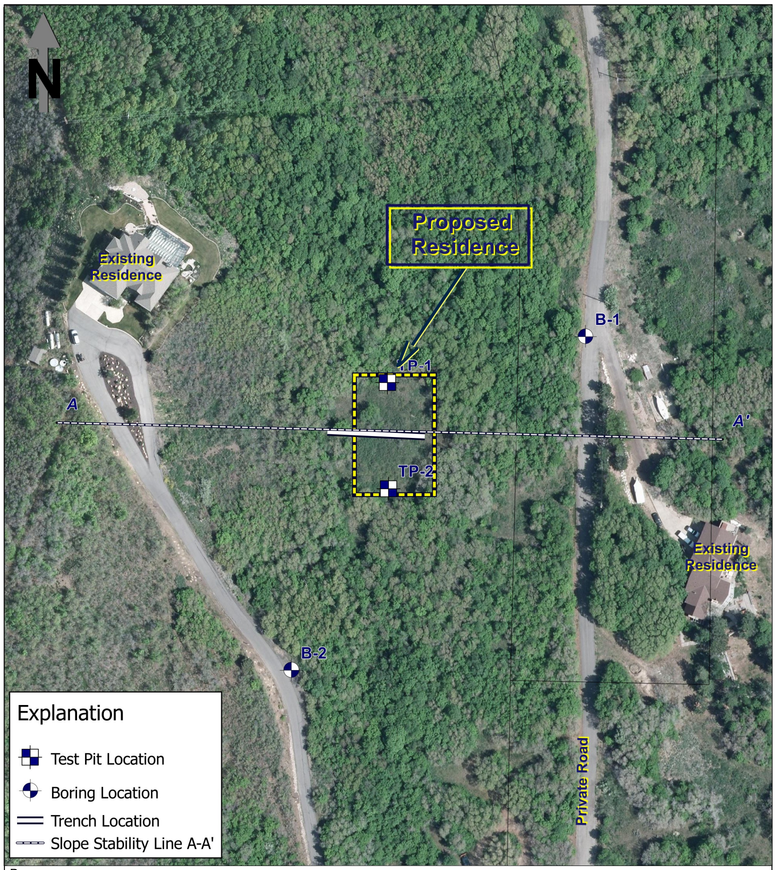


Base:
2011 1.0m LiDAR Imagery
from Utah AGRC; http://gis.utah.gov/

0 200 400 ft

1:2,400

Merrill Residence Huntsville, Utah	CMT ENGINEERING LABORATORIES LiDAR Analysis	Date: 13-June-19 Job # 12670	Figure 4

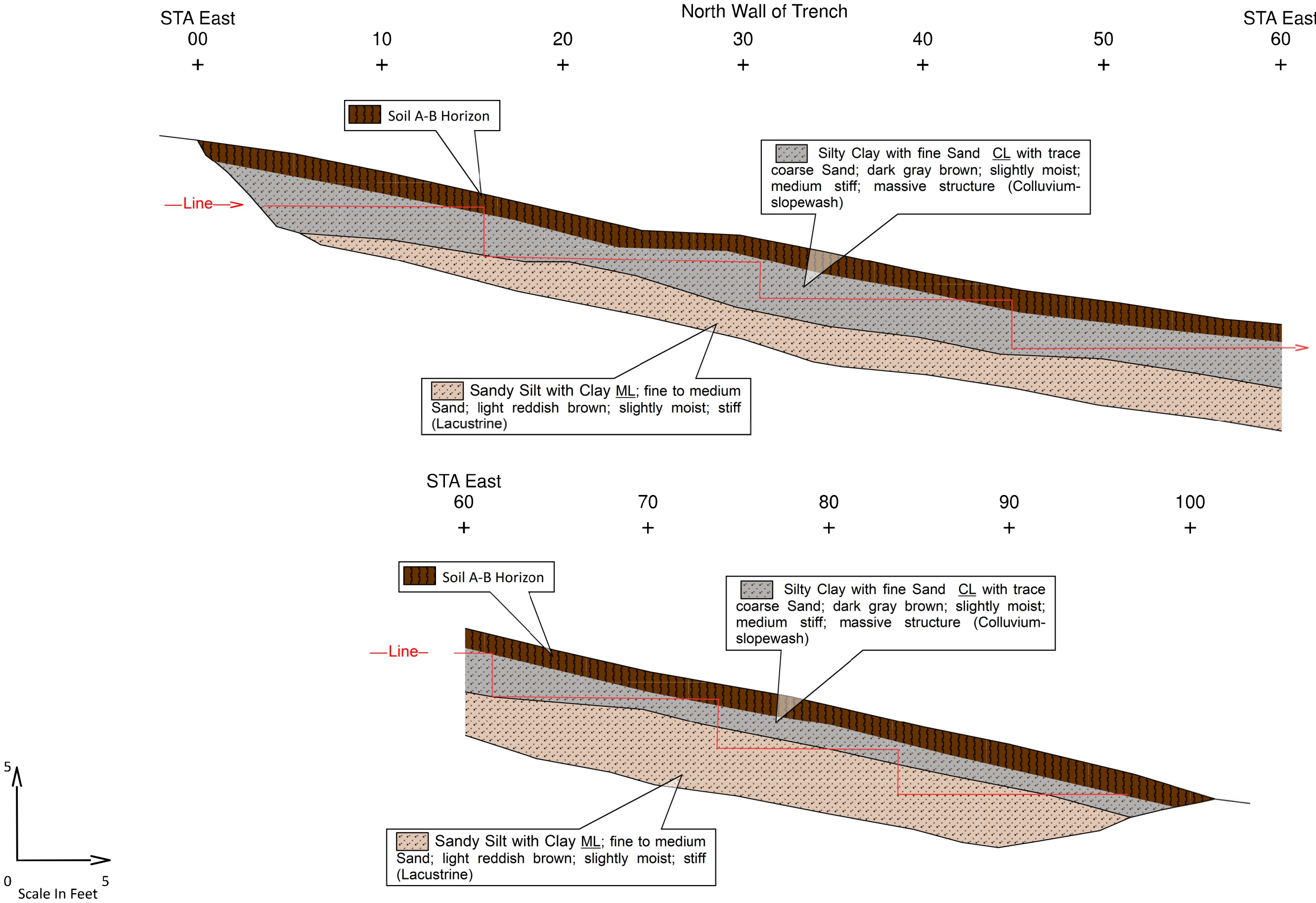


Base:
2012 5.0-Inch Color HRO Orthoimagery,
from Utah AGRC; http://gis.utah.gov/

0 100 200 ft

1:1,200

Merrill Residence Huntsville, Utah	CMT ENGINEERING LABORATORIES **Site Evaluation**	Date: 13-June-19 Job # 12670	Figure 5



CMT ENGINEERING LABORATORIES	Merrill Residence Huntsville, Utah	Log of Trench STA 00 to 101 East	Date:	13-June-19	Figure 6
			Job #	12670	

Merrill Residence

Test Pit Log — TP-1

About 5535 East Highway 39, Huntsville, Utah

Equipment: Rubber Tire Backhoe
Surface Elev. (approx):

Total Depth: 12'
Water Depth: (see Remarks)

Date: 4/24/19
Job #: 12670

Depth (ft)	Soil Description	Sample Type	Sample #	Moisture (%)	Dry Density(pcf)	Gravel %	Sand %	Fines %	LL	PL	PI
0	TOPSOIL: Clay, major roots 0"-6", moist, dark brown										
	Dark Brown CLAY (CL), some fine to coarse gravel, moist — soft (estimated)										
2	grades red-brown — medium stiff (estimated)										
			1	12.9	108						
4	Light Reddish Brown Fine to Medium Silty SAND (SM) with clay, occasional cobbles, slightly moist — medium dense (estimated)										
6			2	14.5				18			
8			3								
10	grades with fine to medium sand and subangular gravel and cobbles		4								
12	END AT 12'										

Remarks: Groundwater not encountered during excavation.

CMT ENGINEERING LABORATORIES

Excavated By: Farrer Excavation
Logged By: Michelle Bostrom
Page: 1 of 1

Figure: 7

Merrill Residence

Test Pit Log — TP-2

About 5535 East Highway 39, Huntsville, Utah

Equipment: Rubber Tire Backhoe
Surface Elev. (approx):
Total Depth: 12'
Water Depth: (see Remarks)
Date: 4/24/19
Job #: 12670

Depth (ft)	Soil Description	Sample Type	Sample #	Moisture (%)	Dry Density (pcf)	Gravel %	Sand %	Fines %	LL	PL	PI
0	Dark Brown CLAY (CL), some fine and coarse gravel, slightly moist to moist — soft to stiff (estimated)										
2	grades light brown — medium stiff (estimated)		5								
3	Light Reddish Brown Fine to Coarse Silty SAND (SM) with clay, occasional cobbles, slightly moist — medium dense (estimated)										
4			6								
9	grades with coarse gravels and cobbles		7	11.4		40	47	13			
10	grades with boulders										
11	Olive, Mottled Rust and Brown, Fine to Coarser Clayey SAND (SC) and fine to coarse gravel, moist — medium dense (estimated)										
12	END AT 12'		8	16.1		26	43	31	63	30	33

Remarks: Groundwater not encountered during excavation.

CMT ENGINEERING LABORATORIES

Excavated By: Farrer Excavation
Logged By: Michelle Bostrom
Page: 1 of 1

Figure: 8

Merrill Residence

Bore Hole Log B-1

About 5535 East Highway 39, Huntsville, Utah

Boring Type: Hollow-Stem Auger
Surface Elev. (approx.):
Total Depth: 12'
Water Depth: (see Remarks)
Date: 4/24/19
Job #: 12670

Depth (ft)	Soil Description		Sample Type	Sample #	Blows (N)	Total	Moisture (%)	Dry Density (pcf)	Gravel %	Sand %	Fines %	LL	PL	PI
0	Brown Clayey SAND (SC), clay layers up to 3", moist													
4		loose		1	2 2 4	6	16.6		1	64	35	31	19	12
	Brown CLAY (CL)	stiff		2	3 5 6	11								
8				3	4 3 5	8								
	Light Brown Silty SAND (SM) with gravel, moist	medium dense		4	7 11 13	24	11.1		32	55	13			
12	REFUSAL AT 12'			5										
16														
20														
24														
28														

Remarks: Groundwater not encountered during drilling.

CMT ENGINEERING LABORATORIES

Drilled By: Great Basin Drilling
Logged By: Sterling Howell

Figure: 9

Merrill Residence

About 5535 East Highway 39, Huntsville, Utah

Bore Hole Log

B-2

Boring Type: Hollow-Stem Auger	Total Depth: 13'	Date: 4/24/19
Surface Elev. (approx):	Water Depth: (see Remarks)	Job #: 12670

Depth (ft)	Soil Description		Sample Type	Sample #	Blows (N)	Total	Moisture (%)	Dry Density(pcf)	Gravel %	Sand %	Fines %	LL	PL	PI
0	Brown CLAY (CL) with sand and gravel, moist													
		stiff		6	3 6 7	13								
4														
	Brown Clayey GRAVEL (GC), moist	medium dense		7	6 8 23	31								
8		dense		8	12 18 14	32								
	Gray Silty SAND (SM), trace gravel, moist	dense		9	16 23 24	47	8.5		10	73	17			
12														
	REFUSAL AT 13'			10	33 50/5"									
16														
20														
24														
28														

Remarks: Groundwater not encountered during drilling.

CMT ENGINEERING LABORATORIES

Drilled By: Great Basin Drilling

Logged By: Sterling Howell

Figure: 10

Merrill Residence

About 5535 East Highway 39, Huntsville, Utah

Key to Symbols

Date: 4/24/19

Job #: 12670

① Depth (ft)	GRAPHIC LOG	Soil Description ③	④ Sample Type	⑤ Sample #	Blows(N) ⑥	⑦ Total	⑧ Moisture (%)	⑨ Dry Density(pcf)	Gradation ⑩ Gravel %	Sand %	Fines %	Atterberg LL	PL	PI

COLUMN DESCRIPTIONS

Depth (ft.): Depth (feet) below the ground surface (including groundwater depth - see water symbol below).

Graphic Log: Graphic depicting type of soil encountered (see ② below).

Soil Description: Description of soils encountered, including Unified Soil Classification Symbol (see below).

Sample Type: Type of soil sample collected at depth interval shown; sampler symbols are explained below-right.

Sample #: Consecutive numbering of soil samples collected during field exploration.

Blows: Number of blows to advance sampler in 6" increments, using a 140-lb hammer with 30" drop.

Total Blows: Number of blows to advance sampler the 2nd and 3rd 6" increments.

Moisture (%): Water content of soil sample measured in laboratory (percentage of dry weight of sample).

Dry Density (pcf): The dry density of a soil measured in laboratory (pounds per cubic foot).

Gradation: Percentages of Gravel, Sand and Fines (Silt/Clay), obtained from lab test results of soil passing the No. 4 and No. 200 sieves.

Atterberg: Individual descriptions of Atterberg Tests are as follows:

LL = Liquid Limit (%): Water content at which a soil changes from plastic to liquid behavior.

PL = Plastic Limit (%): Water content at which a soil changes from liquid to plastic behavior.

PI = Plasticity Index (%): Range of water content at which a soil exhibits plastic properties (= Liquid Limit - Plastic Limit).

STRATIFICATION	
Description	Thickness
Seam	Up to ½ inch
Lense	Up to 12 inches
Layer	Greater than 12 in.
Occasional	1 or less per foot
Frequent	More than 1 per foot

MODIFIERS	
Trace	<5%
Some	5-12%
With	> 12%

MOISTURE CONTENT
Dry: Absence of moisture, dusty, dry to the touch.
Moist: Damp / moist to the touch, but no visible water.
Saturated: Visible water, usually soil below groundwater.

UNIFIED SOIL CLASSIFICATION SYSTEM (USCS)

MAJOR DIVISIONS			USCS SYMBOLS		TYPICAL DESCRIPTIONS
COARSE-GRAINED SOILS More than 50% of material is larger than No. 200 sieve size.	GRAVELS The coarse fraction retained on No. 4 sieve.	CLEAN GRAVELS (< 5% fines)	GW		Well-Graded Gravels, Gravel-Sand Mixtures, Little or No Fines
			GP		Poorly-Graded Gravels, Gravel-Sand Mixtures, Little or No Fines
		GRAVELS WITH FINES (≥ 12% fines)	GM		Silty Gravels, Gravel-Sand-Silt Mixtures
			GC		Clayey Gravels, Gravel-Sand-Clay Mixtures
	SANDS The coarse fraction passing through No. 4 sieve.	CLEAN SANDS (< 5% fines)	SW		Well-Graded Sands, Gravelly Sands, Little or No Fines
			SP		Poorly-Graded Sands, Gravelly Sands, Little or No Fines
		SANDS WITH FINES (≥ 12% fines)	SM		Silty Sands, Sand-Silt Mixtures
			SC		Clayey Sands, Sand-Clay Mixtures
FINE-GRAINED SOILS More than 50% of material is smaller than No. 200 sieve size.	SILTS AND CLAYS Liquid Limit less than 50%		ML		Inorganic Silts and Very Fine Sands, Rock Flour, Silty or Clayey Fine Sands or Clayey Silts with
			CL		Inorganic Clays of Low to Medium Plasticity, Gravelly Clays, Sandy Clays, Silty Clays, Lean
			OL		Organic Silts and Organic Silty Clays o f Low Plasticity
	SILTS AND CLAYS Liquid Limit greater than 50%		MH		Inorganic Silts, Micacious or Diatomacious Fine Sand or Silty Soils
			CH		Inorganic Clays of High Plasticity, Fat Clays
			OH		Organic Silts and Organic Clays of Medium to High Plasticity
HIGHLY ORGANIC SOILS			PT		Peat, Humus, Swamp Soils with High Organic Contents

Note: Dual Symbols are used to indicate borderline soil classifications (i.e. GP-GM, SC-SM, etc.).

SAMPLER SYMBOLS

- Block Sample
- Bulk/Bag Sample
- Modified California Sampler
- 3.5" OD, 2.42" ID D&M Sampler
- Rock Core
- Standard Penetration Split Spoon Sampler
- Thin Wall (Shelby Tube)

WATER SYMBOL

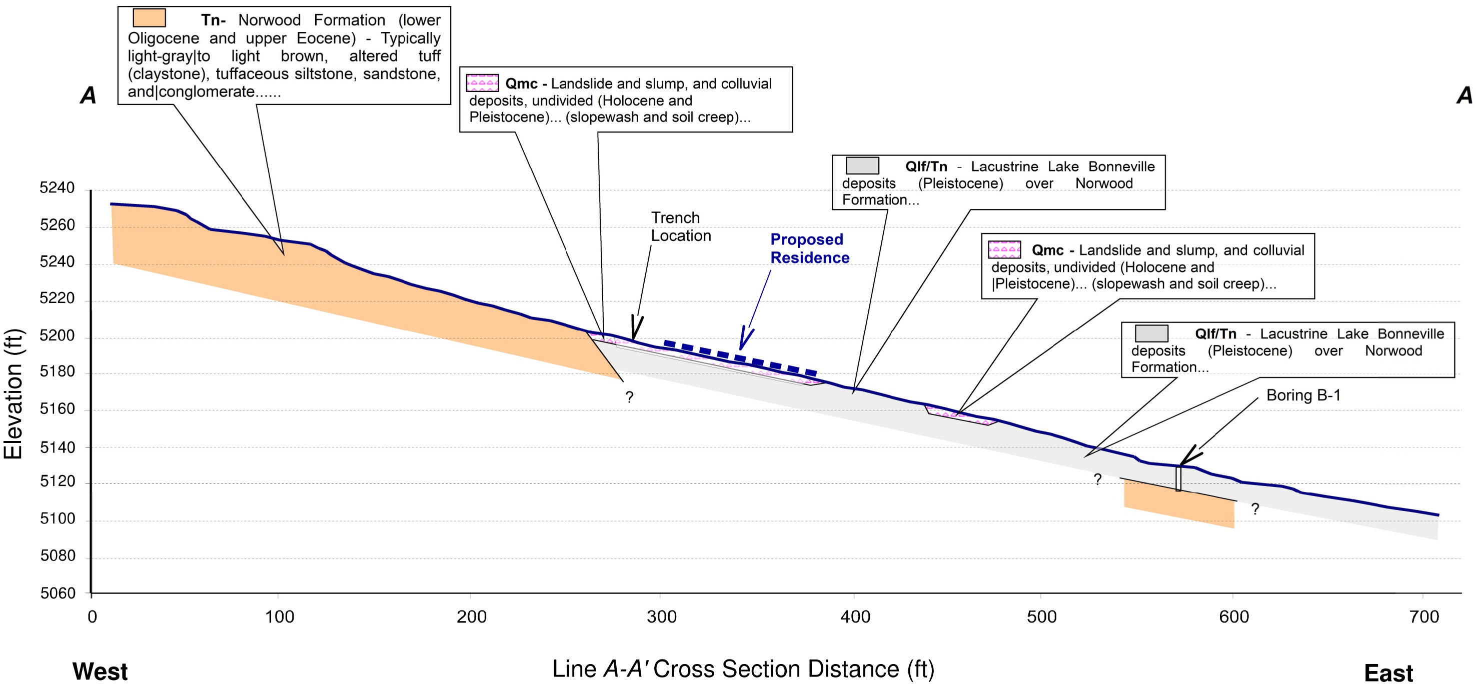
- Encountered Water Level
- Measured Water Level

(see Remarks on Logs)

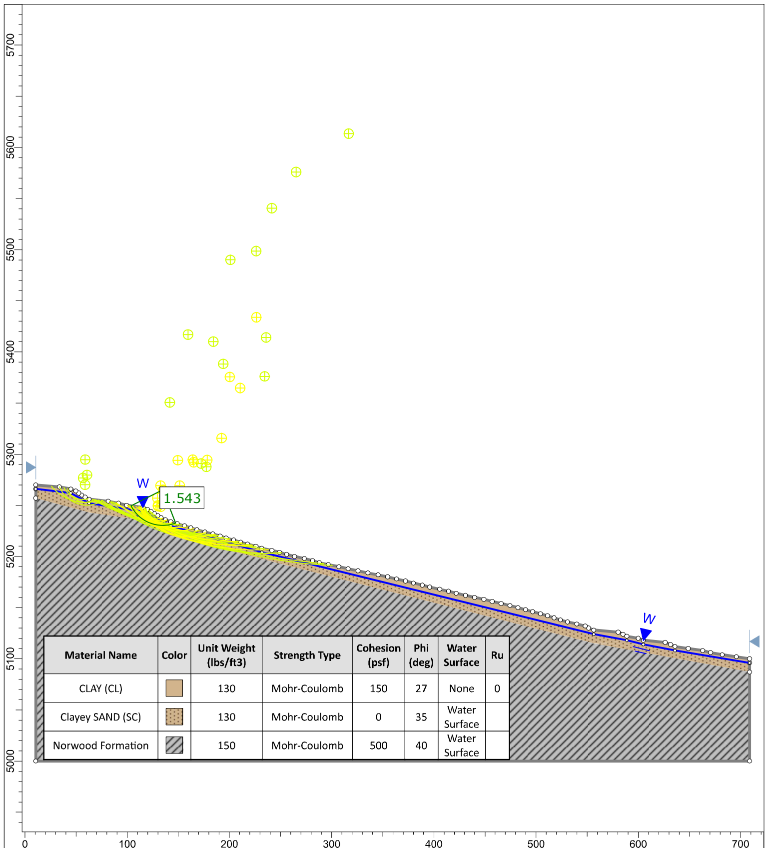
1. The results of laboratory tests on the samples collected are shown on the logs at the respective sample depths.
2. The subsurface conditions represented on the logs are for the locations specified. Caution should be exercised if interpolating between or extrapolating beyond the exploration locations.
3. The information presented on each log is subject to the limitations, conclusions, and recommendations presented in this report.

CMT ENGINEERING LABORATORIES

Figure: 11



CMT ENGINEERING LABORATORIES	Merrill Residence Huntsville, Utah	Slope Cross Section A-A'	Date: 13-June-19 Job # 12670	Figure 12



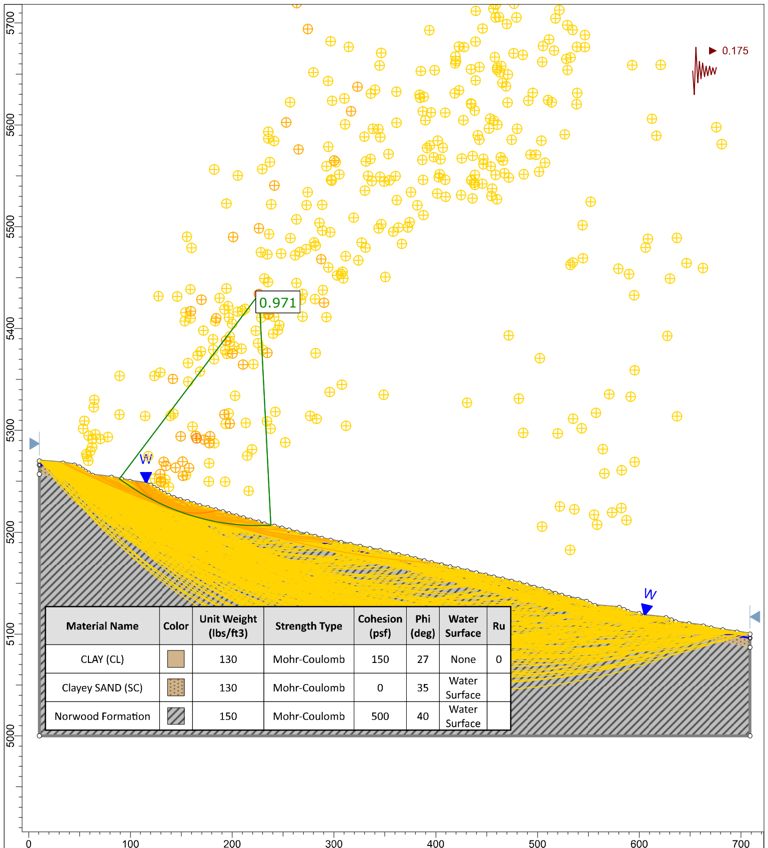
Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Ru
CLAY (CL)		130	Mohr-Coulomb	150	27	None	0
Clayey SAND (SC)		130	Mohr-Coulomb	0	35	Water Surface	
Norwood Formation		150	Mohr-Coulomb	500	40	Water Surface	

CMT ENGINEERING LABORATORIES

SLIDEINTERPRET 7.038

Project	Merrill Residence - Huntsville, UT
Analysis Description	Slope Stability - Static
Drawn By	J. Egbert
Scale	1:1115
Company	CMT Engineering
Date	6/17/2019, 12:48:47 PM
File Name	Stability Analysis - Static.slim

Figure:
13



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Ru
CLAY (CL)		130	Mohr-Coulomb	150	27	None	0
Clayey SAND (SC)		130	Mohr-Coulomb	0	35	Water Surface	
Norwood Formation		150	Mohr-Coulomb	500	40	Water Surface	

CMT ENGINEERING LABORATORIES

Project	Merrill Residence - Huntsville, UT		
Analysis Description	Slope Stability - Psuedo Static		
Drawn By J. Egbert	*Scale* 1:1120	*Company*	CMT Engineering
Date 6/17/2019, 12:48:47 PM		*File Name*	Stability Analysis - Psuedo Static.slim

SLIDEINTERPRET 7.038

Figure:
14