CMT ENGINEERING LABORATORIES

ENGINEERING • GEOTECHNICAL • ENVIRONMENTAL (ESA I & II)
MATERIALS TESTING • SPECIAL INSPECTIONS •
ORGANIC CHEMISTRY • PAVEMENT
DESIGN • GEOLOGY

GEOTECHNICAL ENGINEERING AND GEOLOGIC STUDY

Proposed Paul Coles Cabin

Parcel/Tax ID: 20-035-0020
Tolliver Lane
Huntsville, Weber County, Utah
CMT PROJECT NO. 15297

FOR:
Paul Coles
675 Windsong Lane
North Salt Lake, Utah 84054

September 29, 2020

September 29, 2020

Mr. Paul Coles
675 Windsong Lane
North Salt Lake, Utah 84054

Subject: Geotechnical Engineering and Geologic Study
Proposed Paul Coles Cabin
Parcel/Tax ID: 20-035-0020
Tolliver Lane
Huntsville, Weber County, Utah
CMT Project No. 15297

Mr. Zamani:

Submitted herewith is the report of our geotechnical engineering and geologic 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.

On August 26 to 27 and September 3, 2020 CMT Engineering Laboratories (CMT) personnel were on-site and supervised the excavation of one trench and five test pits extending to depths of 8.1 to 10.4 feet below the existing ground surface. Soil samples were obtained during the field operations and subsequently transported to our laboratory for further testing and observation. 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 four 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 the Project, please do not hesitate to contact us at (801) 870-6730. To schedule materials testing, please call (801) 381-5141.

Sincerely,
CMT Engineering Laboratories

LICENSED PROFESSIONAL GEOLOGIST
9/29/2020
BILL DUANE BLACK
NO. 5224898-2250
STATE OF UTAH

LICENSED PROFESSIONAL ENGINEER
276476
BRYAN N. ROBERTS
9/29/2020
STATE OF UTAH

Bill D. Black, P.G.	Bryan N. Roberts, P.E.
State of Utah No. 5224898-2250	State of Utah No. 276476
Engineering Geologist	Senior Geotechnical Engineer

TABLE OF CONTENTS

APPENDIX

1.0 INTRODUCTION

1.1 General

CMT Engineering Laboratories (CMT) was authorized by Mr. Paul Coles to conduct a design-level geotechnical engineering and geologic study for a proposed cabin to be constructed on a 8.6-acre property (Weber County Assessor Parcel #20-035-0020), located in the SE1/4 Section 23, Township 6 North, Range 1 East (Salt Lake Base Line and Meridian). The property does not have a formal street address. Elevation of the site ranges from about 5,231 to 5,547 feet above sea level. The Project location is shown on **Figure 1, Vicinity Map**. Regional geology of the Project and nearby area is provided on **Figure 2, Geologic Map**. Slope-terrain information is provided on **Figure 3, LIDAR Analysis**. Locations of the trench and test pits excavated for our subsurface investigation are shown on **Figure 4, Site Evaluation**.

1.2 Objectives, Scope and Authorization

The objectives and scope of our study were planned in discussions between Mr. Zamani and Mr. Andrew Harris of CMT Engineering Laboratories (CMT), and are outlined in our proposal dated September 9, 2020.

Our objectives and scope of work included:

1. Performing a site-specific geologic study, in accordance with Section 108-22 Natural Hazard Areas guidelines and standards of the Weber County Code of Ordinances (October 28, 2019), to assess whether all or parts of the site are exposed to natural hazards including, but not limited to: Surface-Fault Rupture, Landslides, Tectonic Subsidence, Rock Falls, Debris Flows, Liquefaction and Flooding.

2. Defining and evaluating site conditions, including: (a) a field program consisting of surficial observation and excavation, logging, and sampling of one trench and five walk-in test pits to evaluate subsurface conditions; (b) a laboratory soils testing program; and (c) an office program consisting of data compilation and correlation, applicable engineering and geological analyses, and preparation of this report summarizing our findings.

Based on the above, recommendations are provided herein to be utilized in appropriate site development and design and construction of the proposed home.

1.3 Description of Proposed Construction

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 40 kips, respectively.

1.4 Executive Summary

The proposed structure can be supported upon conventional spread and continuous wall foundations established on suitable natural soils or on structural fill extending to suitable natural soils. The most significant geotechnical/geological aspects of the site are:

1. The site is in an area mapped by the Utah Geological Survey (UGS) as being underlain by Pleistocene- to Holocene-age landslide deposits from various failures sourced in Tertiary-age Norwood Formation bedrock. Smith Creek flows to the northeast across the western part of the property from higher slopes to the southwest. The proposed cabin location is on a northeast-trending ridge between Smith Creek and a drainage swale further east. Slopes are locally variable in aspect and steepness, but show gradients of from 3% on the ridge top to up to 50% in the eastern half of the site. Test pits TP-1 and TP-2 and the trench exposed sandy to clayey landslide colluvium containing blocks of tuffaceous sandstone to siltstone, overlain by organic-enriched mixed alluvium and colluvium; test pits TP-3 through TP-5 exposed a few feet of similar landslide colluvium overlying undeformed tuffaceous sandstone. The proposed cabin should be situated on the sandstone bedrock no further east than TP-5. Groundwater was not observed in the trench or test pits conducted for our study, but may be about 10 feet deep seasonally in the drainage swale and along Smith Creek.

2. The upper about 1 to 2 feet of surficial soil contains major roots, root holes/potential pin holes, and is considered to be topsoil.

3. Surficial clay soils/weathered bedrock are moderately to highly plastic and may exhibit swell properties under relatively light loading. These soils should be removed down to exposed suitable bedrock and incorporate a minimum thickness of replacement fill below structural elements (see section **7.0 Site Preparation and Grading**).

3. Slope stability is a critical factor in development of the property. Specific analyses and recommendations related to slope stability are subsequently presented within this report.

A geotechnical engineer from CMT must be allowed to verify that all topsoil, undocumented fill material and potentially expansive soils Arena have been completely removed from beneath proposed structures, and suitable natural soils encountered prior to the placement of structural fills, floor slabs, footings, foundations, or concrete flatwork.

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.

2.0 FIELD EXPLORATION

Subsurface soil conditions at the site were explored by excavating one trench and five walk-in test pits (short trenches) located as shown on **Figure 4, Site Evaluation**. The test pits were excavated using a track-mounted excavator to depths of 8.1 to 10.4 feet for geologic/geotechnical logging and sampling. During the course of the excavation operations, a continuous log of the subsurface conditions encountered was maintained. Undisturbed tube, block and disturbed bulk samples of representative soils encountered in the test pits were obtained for subsequent laboratory testing and examination. The representative soil samples were placed in sealed plastic bags and containers prior to transport to the laboratory. No samples were obtained from TP-5.

The collected samples were logged and described in general accordance with ASTM D-2488, packaged, and transported to our laboratory. The soils were classified in the field based upon visual and textural examination. These classifications were supplemented by subsequent inspection and testing in our laboratory. The subsurface conditions encountered in the field exploration are discussed below in **Section 3.2**. Geologic logs of the trench and test pits are illustrated on **Figures 5A-5B** (trench log) and **Figures 6A-6E** (test pit logs). The logging methodology followed McCalpin (1996). The excavation locations were measured using a handheld GPS unit and by trend and distance methods. Location, trend, and other pertinent data and observations are provided on the logs.

When backfilling the excavations, only minimal effort was made to compact the backfill and no compaction testing was performed. Thus, the backfill must be considered as non-engineered and settlement of the backfill in the test pits over time must be anticipated.

3.0 SITE CONDITIONS

3.1 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 (Coogan and King, 2016); GIS analyses of elevation and geoprocessed 2016 LiDAR terrain data as shown on **Figure 3**; photogeologic analyses of 2012 imagery shown on **Figure 4**; field reconnaissance of the general site area; and interpretation of the test pits conducted at the site as part of our field program (**Figures 5A-B** and **6A-E**). Seismic hazards information was developed from United States Geologic Survey (USGS) databases (Peterson and others, 2008).

As shown on **Figure 3**, topography of the site vicinity varies in aspect and steepness, including northwest to northeast-facing slopes that have gradients from 3% to 50%. Most of the site is on slopes steeper than 25%; slopes less than 15% are found mainly on the ridge between Smith Creek and the unnamed drainage swale further east, and on a landslide deposit in the eastern half of the site that extends northwestward into the unnamed drainage swale. The proposed cabin location is on the shallower sloped ridge. Vegetative cover at the site consists mainly of grasses and dense oak brush. The site is currently vacant land. The surrounding area is generally undeveloped, except for a residential home upslope to the south. Smith Creek was dry at the time of our on-site investigation and therefore appears to be intermittent.

3.2 Subsurface Soils

One trench and five test pits were excavated in the area of the proposed cabin location to evaluate subsurface soil conditions, as located on **Figure 4**. The trench exposed mainly clayey to sandy landslide colluvium with blocks of fractured, jumbled and degraded tuffaceous sandstone to siltstone; the landslide colluvium is overlain by organic-enriched clayey to sandy mixed alluvium and colluvium (**Figures 5A-B**). Test pits TP-1 and TP-2 (**Figures 6A-B**) exposed a similar sequence of colluvium overlying what we infer are rafted blocks of Norwood Formation bedrock based on the lack of bedrock in this area in the trench exposure. Test pits TP-3 through TP-5 (**Figures 6C-E**) exposed a few feet of landslide colluvium overlying sandstone bedrock of the Norwood Formation showing strike and dips similar to regional strike and dips shown on **Figure 2**. Given our observations, we infer the bedrock in test pits TP-3 through TP-5 is undeformed. The proposed cabin should be situated on the undeformed bedrock (i.e. no further east than TP-5). No groundwater was encountered in any of the excavations to their explored depths. Detailed stratigraphic unit descriptions are shown on the logs.

3.3 Groundwater

No groundwater was encountered in the excavations conducted at the site to their maximum explored depths of 8.1 to 10.4 feet below the existing ground surface. However, groundwater levels may vary locally, annually from climatic fluctuations, and seasonally from snow-melt runoff or from man-made sources such as landscape irrigation and septic systems. The Utah Division of Water Rights Well Driller Database indicates static groundwater is at a depth of 134 feet in the water well for the existing home about 500 feet southwest (upslope) of the site. We conservatively estimate that groundwater is seasonally at a depth of about 10 feet in the bottom of the unnamed drainage swale and along Smith Creek, but deepens in the ridge between the drainages and in the summer and fall. This projected water level was incorporated into our slope stability analysis.

3.4 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, care should be taken in interpolating or extrapolating subsurface conditions between or beyond the exploratory locations.

4.0 ENGINEERING GEOLOGY

4.1 Seismotectonic Setting

The property is located at the southern margin of Ogden Valley, a roughly 40-square mile back valley described by Gilbert (1928) as a structural trough similar to Cache and Morgan Valleys to the north and south, respectively. The back valleys of the northern Wasatch Range are in a transition zone between the Basin and Range and Middle Rocky Mountains physiographic provinces (Stokes, 1977, 1986). The Basin and Range is characterized by a series of generally north-trending elongate mountain ranges, separated by predominately alluvial and lacustrine sediment-filled valleys and typically bounded on one or both sides by major normal faults (Stewart, 1978). The boundary between the Basin and Range and Middle Rocky Mountains provinces is marked by the Wasatch fault zone at the base of the Wasatch Range. Late Cenozoic normal faulting, a characteristic of the Basin and Range, began between about 17 and 10 million years ago in the Nevada (Stewart, 1980) and Utah (Anderson, 1989) portions of the province. The faulting is a result of a roughly east-west directed, regional extensional stress regime that has continued to the present (Zoback and Zoback, 1989; Zoback, 1989). The back valleys are morphologically similar to valleys in the Basin and Range, but exhibit less structural relief (Sullivan and others 1988).

Ogden Valley occupies a structural trough created by up to 2,000 feet of vertical displacement on normal faults bounding the northeastern and southwestern margins of the valley. Coogan and King (2016) and the Utah Geological Survey Quaternary Fault Database (Black and others, 2003; January 2017 update) show these faults about 4.0 and 1.0 miles to the northeast and west, respectively. Both faults were most-recently active more than 10,000 years ago (Sullivan and others, 1986). The nearest active (Holocene-age) fault to the site is the Weber segment of the Wasatch fault zone about 6.7 miles to the west.

The site is also situated near the central portion of the Intermountain Seismic Belt (ISB). The ISB is a north-south-trending zone of historical seismicity along the eastern margin of the Basin and Range province which extends for approximately 900 miles from northern Arizona to northwestern Montana (Sbar and others, 1972; Smith and Sbar, 1974). At least 16 earthquakes of magnitude 6.0 or greater have occurred within the ISB since 1850, with the largest of these events the MS 7.5 1959 Hebgen Lake, Montana earthquake. However, none of these events have occurred along the Wasatch fault zone or other known late Quaternary faults in the region (Arabasz and others, 1992; Smith and Arabasz, 1991). The closest of these events to the site was the 1934 Hansel Valley (MS 6.6) event north of the Great Salt Lake and south of the town of Snowville.

4.2 Surficial Geology

The site is located in southern Ogden Valley in an area mapped by Coogan and King (2016) as being underlain by various landslide deposits (units Qms and Qmc) sourced in and underlain by Norwood Formation bedrock (unit Tn, **Figure 2**). Coogan and King (2016) describe surficial geologic units in the site area on **Figure 2** as follows:

> ***Qlamh*** – *Lacustrine, marsh, and alluvial deposits, undivided (Historical).* Sand, silt, and clay mapped where streams enter Pineview Reservoir, and reservoir levels fluctuate such that lacustrine, marsh, and alluvial deposits are intermixed; thickness uncertain.
>
> ***Qa2, Qa2?, Qay*** – *Younger alluvium (mostly Holocene).* Like undivided alluvium, with Qay at to slightly above present drainages, unconsolidated, and not incised by active drainages; likely mostly Holocene in age and postdates late Pleistocene Provo shoreline of Lake Bonneville; height above present drainages is low and is within certain limits, with suffix 1 (not present on this map) being the youngest and being at to slightly (<10 feet [3 m]) above drainages and suffix 2 being slightly higher and older, with y suffix where ages 1 and 2 cannot be separated; Qa2 is up to about 20 feet (6 m) above drainage on south side of Round Valley indicating unit includes slightly older post Provo-shoreline alluvium; generally 6 to 20 feet (2-6 m) thick. Mapped as Qa2 (queried) where about 20 feet (6 m) above incised stream in Stephens Canyon (Devils Slide quadrangle).

Qal, Qal1, Qal2, Qal2? – *Stream alluvium and flood-plain deposits (Holocene and uppermost Pleistocene).* Sand, silt, clay, and gravel in channels, flood plains, and terraces typically less than 16 feet (5 m) above river and stream level; moderately sorted; unconsolidated; along the same drainage Qal2 is lower than Qat2 and has likely been subject to flooding, at least prior to dam building; present in broad plains along the Bear, Ogden, and Weber Rivers and larger tributaries like Deep, Cottonwood, East Canyon, Lost, and Saleratus Creeks, along Box Elder, Heiners, and Yellow Creeks, and in narrower plains of larger tributary streams; locally includes muddy, organic overbank and oxbow lake deposits; composition depends on source area, so in back valleys typically contains many quartzite cobbles recycled from the Wasatch Formation; mostly Holocene, but deposited after regression of Lake Bonneville from the late Pleistocene Provo shoreline; width in Morgan Valley is combined flood plain of Weber River and East Canyon and Deep Creeks; 6 to 20 feet (2-6 m) thick and possibly as much as 50 feet (15 m) along Weber River and thinner in the Kaysville quadrangle; greater thicknesses (>50 feet [15 m]) are reported in Morgan Valley (Utah Division of Water Rights, well drilling database), but likely include Lake Bonneville and older Pleistocene deposits.

Suffixes 1 and 2 indicate ages where they can be separated, with 1 including active channels and 2 including low terraces 10 to 20 feet (3-6 m) above the Weber and Ogden Rivers, and the South Fork Ogden River that may have been in the flood plain prior to damming of these waterways. Qal2 queried in low terraces above Bear River, Saleratus Creek, and Dry Creek where deposits may not be in the flood plain.

Qat, Qat2, Qaty, Qatp, Qatp?, Qatpb, Qato – *Stream-terrace alluvium (Holocene and Pleistocene).* Sand, silt, clay, and gravel in terraces above floodplains near late Pleistocene Lake Bonneville and are geographically in the Ogden and Weber River, and lower Bear River drainages; moderately sorted; variably consolidated; upper surfaces slope gently downstream; locally includes thin and small mass-movement and alluvial-fan deposits; where possible, subdivided into relative ages, indicated by number and letter suffixes, with 2 being the lowest/youngest terraces, typically about 10 to 20 feet (3-6 m) above adjacent flood plains; Qat with no suffix used where age unknown or age subdivisions of terraces cannot be shown separately at map scale; 6 to at least 20 feet (2-6+ m) thick, with Qatp 50 to 80 feet (15-24 m) thick in Mantua Valley.

Relative ages are largely from heights above adjacent drainages in Morgan and Round Valleys. This subdivision apparently works in and is applied in Ogden, Henefer, and Lost Creek Valleys and above the North, Middle, and South Forks of Ogden River (see tables 1 and 2). Despite the proximity to Lake Bonneville, terraces along and near Box Elder Creek in the northwest corner of the Ogden map area (Mantua quadrangle) seem to be slightly higher than comparable terraces in Morgan Valley. Terraces labeled Qat2 are post-Lake Bonneville and are likely mostly Holocene in age. A terrace labeled Qaty is up to 20 feet (6 m) above the South Fork Ogden River, but may be related to the Provo or regressional shorelines. Terraces labeled Qatp are likely related to the Provo and slightly lower shorelines of Lake Bonneville (at and less than ~4820 feet [1470 m] in area), and with Qap form "benches" at about 4900 feet (1494 m) along the Weber River and South Fork Ogden River. Qato terraces pre-date Lake Bonneville. Relative age queried (Qatp?) where age is uncertain, generally due to height not fitting into ranges in table 1 and/or typical order of surfaces contradicts height-derived age.

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.

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 (slope wash 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 slope wash 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).

Qap, Qap?, Qab, Qab?, Qapb – *Lake Bonneville-age alluvium (upper Pleistocene).* Like undivided alluvium but height above present drainages appears to be related to shorelines of Lake Bonneville and is within certain limits, and unconsolidated to weakly consolidated; alluvium labeled Qap and Qab is related to Provo (and slightly lower) and Bonneville shorelines of Lake Bonneville (at ~4800 to 4840 feet [1463-1475 m] and 5180 feet [1580 m] in Morgan Valley), respectively; suffixes partly based on heights above adjacent drainages near Morgan Valley (see tables 1 and 2); Qap is typically about 15 to 40 feet (5-12 m) above present adjacent drainages, but is locally 45 feet (12 m) above; Qapb is used where more exact age cannot be determined, typically away from Lake Bonneville, or where alluvium of different ages cannot be shown separately at map scale; Qap is up to about 50 feet (15 m) thick, with Qapb and Qab, at least locally up to 40 and 90 feet (12 and 27 m) thick, respectively. Queried where classification or relative age uncertain (see Qa).

A prominent surface ("bench") is present on Qap and Qatp at about 4900 feet (1494 m) elevation and about 25 to 40 feet (8-12 m) above the Weber River in Morgan Valley and along the South Fork Ogden River.

In the Devils Slide quadrangle, the Qab that is mapped about 80 to 95 feet (24-29 m) above Round Valley and 40 to 50 feet (12-15 m) above adjacent drainages at the mouth of Geary Hollow appears unique. Based on heights above adjacent drainages, these deposits would be Qao (see table 1), but similar alluvial deposits to the east near Phil Shop Hollow have a Bonneville shoreline cut in them and are much thinner than 40 feet (12 m). The lack of a Bonneville shoreline, and small thickness and heights above drainages indicate the deposits could be a Bonneville shoreline fan-delta.

Ql, Ql? – *Lake Bonneville deposits, undivided (upper Pleistocene)*. Silt, clay, sand, and cobbly gravel in variable proportions; mapped where grain size is mixed, deposits of different materials cannot be shown separately at map scale, or surface weathering obscures grain size and deposits are not exposed in scarps or construction cuts; thickness uncertain.

Qlf, Qlf?, Qlfb, Qlfb? – *Fine-grained lacustrine deposits (Holocene and upper Pleistocene).* Mostly silt, clay, and fine-grained sand deposited near- and off-shore in Lake Bonneville; typically mapped as Qlf below the Provo shoreline (P) because older transgressive (Qlfb) deposits are indistinguishable from younger regressive deposits; mapped as Qlfb above the Provo shoreline because these deposits can only be related to the Bonneville shoreline (B) and transgression; grades upslope with more sand into Qls or Qlsp; typically eroded from shallow Norwood Formation in Ogden and Morgan Valleys and at least 12 feet (4 m) thick near Mountain Green. Qlf and Qlfb queried where grain size is uncertain.

In the Kaysville quadrangle, Qlf deposits that are below the Gilbert (G) shoreline are at least partly the same age as this shoreline (Holocene-latest Pleistocene) and post-date late Pleistocene Lake Bonneville. Qlf deposits below the Holocene (H) highstand shoreline are Holocene. Both ages of deposits are generally less than 15 feet (5 m) thick.

Deeper water fine-grained deposits overlie older shoreline and delta gravels (Qlf/Qdlb) at the mouths of several drainages along the Weber River. These gravels were deposited above the Provo shoreline during transgression of Lake Bonneville to the Bonneville shoreline (see unit Qdlb).

Qls, Qls?, Qlsp, Qlsb, Qlsb? – *Lake Bonneville sand (upper Pleistocene).* Mostly sand with some silt and gravel deposited nearshore below and near the Provo shoreline (Qlsp) and between the Provo and Bonneville shorelines (Qlsb); Qls mapped downslope from slope break below Provo shoreline beach deposits where thin Lake Bonneville regressional sand may overlie transgressional sand; grades downslope into unit Qlf with decreasing sand content and laterally with more gravel into units Qdlp, Qdlb, and upslope with more gravel into unit Qlgb; Qls and Qlsb queried where grain size or unit identification uncertain; may be as much as 75 feet (25 m) thick, and thickest near Ogden; typically less than 20 feet (6 m) thick in Morgan Valley; may include small deltas and deltas that lack typical delta shape.

Qao, Qao? *– Older alluvium (mostly upper Pleistocene).* Sand, silt, clay, and gravel above and likely older than the Bonneville shoreline; mapped on surfaces above Lake Bonneville-age alluvium (Qap, Qab, Qapb); deposits lack fan shape (Qaf) and are distinguished from terraces (Qat) based on upper surface sloping toward adjacent streams from sides of drainage; also shown where areas of fans and terraces are too small to show separately at map scale; composition depends on source area; at least locally up to 110 feet (34 m) thick. Queried where classification or relative age is uncertain (see Qa for details); for example near head of Saleratus Creek.

Older alluvium is likely older than Lake Bonneville and the same age as Qafo, so likely Bull Lake age, 95,000 to 130,000 years old (see Chadwick and others, 1997, and Phillips and others, 1997); see table 1 and note revision from Coogan and King (2006) and King and others (2008). From our work in the Henefer (Coogan, 2010b) and Devils Slide quadrangles and ages in Sullivan and Nelson (1992) and Sullivan and others (1988), older alluvium (Qao, Qafo, Qato) may encompass an upper (pre-Bull Lake) and lower (Bull Lake) alluvial surface that is not easily recognized in Morgan Valley (see tables 1 and 2).

Tcg, Tcg? *– Unnamed Tertiary conglomeratic rocks (Oligocene?).* Characterized by rounded, cobble- to boulder-sized, quartzite-clast conglomerate with pebbles and less than 10 percent to more than 50 percent gray, tan, or reddish-gray to reddish-tan matrix; conglomerate clasts locally angular to subangular Tintic Quartzite and angular to rounded lower Paleozoic carbonate rocks; interbedded with tan, gray, and reddish-brown, pebble-bearing mudstone to sandstone and some claystone (altered tuff); most beds poorly indurated and poorly exposed; mudstone likely constitutes matrix of conglomeratic beds; in Morgan and Durst Mountain quadrangles, about 500 to 700 feet (150-210 m) thick and thickening northward to possibly 3000 feet (900 m), though faulting may make this estimate too large.

Reddish-hued Tcg strata mostly contain recycled Wasatch Formation clasts (quartzite and carbonate) with a distinct reddish patina in a reddish matrix. Some non-conglomeratic beds in Tcg look like gray upper Norwood Formation (Tn) and are locally tuffaceous, indicating the units are interbedded. Further, some Tcg pebble beds have carbonate and chert clasts (like the Norwood) and lesser quartzite clasts, and Tcg conglomerate includes rare altered tuff clasts from the Norwood Formation. Despite tuffaceous matrix, unit Tcg seems to be less prone to mass movements than Norwood strata.

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.

ZYp, ZYp? *– Formation of Perry Canyon (Neoproterozoic and possibly Mesoproterozoic)*. Argillite to metagraywacke upper unit, middle meta-diamictite, and basal slate, argillite, and meta-sandstone; phyllitic at least south of Pineview Reservoir; due to overturned folding, only one diamictite unit (Adolph

Yonkee, Weber State University, February 2, 2011, email communication) rather than two (see Crittenden and others, 1983); total thickness likely less than 2000 feet (600 m) (this report). Queried in knob west of North Fork Ogden River in North Ogden quadrangle because rock is quartzite that may be in this unit or the Papoose Creek Formation. The formation of Perry Canyon is prone to slope failures.

Balgord's (2011; Balgord and others, 2013) detrital zircon uranium-lead and lead-lead maximum depositional ages (~950-1030 Ma) on the basal mudstone unit straddle the Upper and Middle Proterozoic boundary, but other maximum ages (925 Ma) on this mudstone unit are Upper Proterozoic; her maximum ages on the upper unit are about 640, 660, and 690 Ma.

Lower part of formation not measured where thick in the Wasatch Range and stratigraphy not worked out, because upper and lower parts incompletely measured and at least locally the upper and lower parts in the Wasatch Range are lithologically indistinguishable. Unit ("member") thicknesses vary due to syndepositional faulting (see Balgord and others, 2013). The best stratigraphic section of the lower unit (ZYpm), volcanic unit (Zpb), and diamictite (Zpd) is 30 miles (50 km) to the southwest on Fremont Island in Great Salt Lake, but the base of ZYpm is not exposed (see Balgord, 2011, figure 14, p. 51; Balgord and others, 2013, figure 5). The Fremont Island section is likely in a different Proterozoic faulted basin; compare thicknesses and lithologies between Fremont Island and Willard Peak shown by Balgord (2011, Balgord and others (2013). Also, although both localities are shown on the Willard thrust sheet by Yonkee and Weil (2011), they may be on different thrust sheets. Therefore, the formal term Perry Canyon Formation is not used. Where possible divided into several lithosomes which have been called members.

Citations, tables, and/or figures referenced above are not provided herein but are in Coogan and King (2016).

4.3 Lake Bonneville History

Lakes occupied nearly 100 basins in the western United States during late-Quaternary time, the largest of which was Lake Bonneville in northwestern Utah. The Bonneville basin consists of several topographically closed basins created by regional extension in the Basin and Range (Gwynn, 1980; Miller, 1990), and has been an area of internal drainage for much of the past 15 million years. Lake Bonneville consisted of numerous topographically closed basins, including the Salt Lake and Cache Valleys (Oviatt and others, 1992). Portions of Ogden Valley were inundated by Lake Bonneville at its highstand and sediments from Lake Bonneville are mapped on Figure 2 north of the site.

Timing of events related to the transgression and regression of Lake Bonneville is indicated by calendar age estimates of significant radiocarbon dates in the Bonneville Basin (Oviatt, 2015). Approximately 30,000 years ago, Lake Bonneville began a slow transgression (rise) to its highest level of 5,160 to 5,200 feet above mean sea level. The lake rise eventually slowed as water levels approached an external basin threshold in northern Cache Valley at Red Rock Pass near Zenda, Idaho. Lake Bonneville reached the Red Rock Pass threshold and occupied its highest shoreline, termed the Bonneville beach, around 18,000 years ago. During the transgression and highstand, major drainages that emanate from within the Wasatch Range (such as the Weber River) formed large deltaic complexes in the lake at their canyon mouths. Headward erosion of the Snake River-Bonneville basin drainage divide then caused a catastrophic incision of the threshold and the lake level lowered by roughly

360 feet in fewer than two months (Jarrett and Malde, 1987; O'Conner, 1993). The Project is situated above the highest (Bonneville) shoreline (Figure 2, blue line and B).

Following the Bonneville flood, the lake stabilized and formed a lower shoreline referred to as the Provo shoreline between about 16,500 and 15,000 years ago. Climatic factors then caused the lake to regress rapidly from the Provo shoreline, and by about 13,000 years ago the lake had eventually dropped below historic levels of Great Salt Lake. Drainages that fed Lake Bonneville began downcutting through stranded deltaic complexes and near-shore deposits as the lake receded from the Provo shoreline. Oviatt and others (1992) deem this low stage the end of the Bonneville lake cycle. Great Salt Lake then experienced a brief transgression around 11,600 years ago to the Gilbert level at about 4,250 feet before receding to and remaining within about 20 feet of its historic average level (Lund, 1990).

4.4 Seismic Hazards

4.4.1 Strong Ground Motions

Strong ground motion is likely to present a significant risk during moderate to large earthquakes located within a 60-mile radius of the Project area (Boore and others, 1993). Seismic sources include mapped active faults, as well as a random or "floating" earthquake source on faults not evident at the surface. The Utah Geological Survey Quaternary Fault Database (Black and others, 2003; January 2017 update) shows numerous class A faults within 60 miles of the Project that may pose potential seismic sources. Strong ground motions originating from the Wasatch fault or other near-by seismic sources are 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).

4.4.2 Site Class

Utah has adopted the 2018 International Building Code (IBC), which determines the seismic hazard for a site based upon 2014 mapping of bedrock accelerations prepared by the United States Geologic Survey (USGS) and the soil site class. The USGS values are presented on maps incorporated into the IBC code and are also available based on latitude and longitude coordinates (grid points). For site class definitions, IBC 2018 Section 1613.2.2 refers to Chapter 20, Site Classification Procedure for Seismic Design, of ASCE[1] 7-16. Given the surficial soils encountered and the relatively weathered and soft bedrock in the area of the proposed cabin, it is our opinion the site best fits Site Class D – Stiff Soil, which we recommend for seismic structural design.

4.4.3 Seismic Design Category

The 2014 USGS mapping utilized by the IBC provides values of peak ground, short period and long period accelerations for the Site Class B/C boundary and the Maximum Considered Earthquake (MCE). This Site Class B/C boundary represents average bedrock values for the Western United States and must be corrected for local soil conditions. The Seismic Design Categories in the International Residential Code (IRC 2018 Table

[1] American Society of Civil Engineers

R301.2.2.1.1) are based upon the Site Class discussed in the previous section. For Site Class B at site grid coordinates of 41.236883 degrees north latitude and 111.798644 degrees west longitude, $\mathbf{S_{DS}}$ is 0.68 and the **Seismic Design Category** is D_1.

4.4.4 Surface Faulting

Movement along faults at depth generates earthquakes. During earthquakes larger than Richter magnitude 6.5, ruptures along normal faults in the intermountain region generally propagate to the surface (Smith and Arabasz, 1991) as one side of the fault is uplifted and the other side down dropped. The resulting fault scarp has a near-vertical slope. The surface rupture may be expressed as a large singular rupture or several smaller ruptures in a broad zone. Ground displacement from surface fault rupture can cause significant damage or even collapse to structures located on an active fault.

No evidence of active surface faulting is mapped or was evident at the site. The nearest active (Holocene-age) fault to the site is the Weber segment of the Wasatch fault zone about 6.7 miles to the west. Surface faulting is not therefore considered to pose a risk to the site.

4.4.5 Liquefaction

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 has not been studied or mapped for the Project area, but subsurface data from the trench and test pits suggest the risk from liquefaction is likely low. Weber County hazard mapping shows the site is in an area of very low liquefaction potential (Code 1).

4.4.6 Tectonic Subsidence

Tectonic subsidence is surface tilting subsidence that occurs along the boundaries of normal faults in response to surface-faulting earthquakes (Keaton, 1986). The site is not located on the downthrown side of and near any active earthquake faults, and tectonic subsidence is not therefore considered to pose a risk.

4.5 Landslide and Slump Deposits

Landslides, slumps, and other mass movements are gravity-induced downslope movements of rock or soil. Such failures may be both deep and shallow seated. Deep-seated failures include rotational and translational slides and associated earthflows where the failure plane is more than 10 feet deep (Varnes, 1978; Cruden and Varnes, 1996). Landslides can develop in moderate to steep slopes where a slope has been disturbed, the head of a slope loaded, or where increased groundwater pore pressures result in driving forces within the slope exceeding restraining forces.

Figure 2 shows most of the site is underlain by landslide deposits, except for near the northeast site corner. Evidence for landslides was also exposed in the trench and test pits conducted at the site, but undeformed bedrock was exposed in test pits TP-3, TP-4 and TP-5 beneath 1.8 feet to 3.8 feet of colluvium. **Figures 3 and 4** show site-specific geologic mapping based on the subsurface data, Coogan and King (2016) and air photo evidence. A landslide that may be contemporaneous with or post-date late Pleistocene Lake Bonneville is in the slopes in the southeast half of the site; the toe of this landslide extends into the unnamed drainage swale east of Smith Creek and was exposed in the trench and test pits TP-1 and TP-2. The ridge between Smith Creek and the drainage swale is underlain by undeformed bedrock with a thin colluvial veneer. Given the above, landslides pose a risk to the site. No evidence for recent or ongoing slope instability was observed during the field investigation. Specific analyses and recommendations regarding slope stability are provided in **Section 5.0**.

4.6 Other Geologic Hazards

Other potential geologic hazards at the site are addressed in the following subsections.

4.6.1 Sloping Surfaces

Surface slopes at the Project developed from our LiDAR analysis, as shown on **Figure 3**, are mainly steeper than 25% (in red). Slopes gentler than 15% on **Figure 3** (unshaded) are found along the ridge between Smith Creek and the unnamed drainage swale further east, and in the landslide underlying the southeast half of the site.

4.6.2 Alluvial Fan Flooding

Alluvial-fan flooding refers to a continuum of processes that includes debris slides, debris flows, debris floods, and flash flooding on alluvial fans (National Research Council, 1996). Debris flows and related sediment-enriched floods and flows are fast moving flow-type landslides comprised of a slurry of rock, mud, organic matter, and water that move down drainage-basin channels onto alluvial fans (Giraud, 2005). Debris flow hazards are commonly associated with areas underlain by Holocene alluvial-fan deposits at the mouths of range-front drainages, such as those along the Wasatch Range.

The Project is not in an area subject to alluvial-fan flooding and no debris-flow channels, levees, or other debris-flow features were observed. Debris flows and floods are not therefore considered to pose a risk to the site.

4.6.3 Stream Flooding Hazards

Smith Creek crosses the western part of the site and an unnamed drainage heads at the site further east. No drainage course was observed in the swale at the location of the trench (**Figure 4, Site Evaluation**) and Smith Creek was dry at the time of our evaluation. Federal Emergency Management Agency flood insurance rate mapping (Map Number 49057C0475F, effective June 2015, unprinted) classifies the Project in "Zone X - Area of Minimal Flood Hazard". Given the above, stream flooding is not considered to pose a significant risk to the site.

4.6.4 Rockfall and Avalanche Hazards

The site is not located downslope from steep slopes with source areas where rockfalls and avalanches may originate.

5.0 SLOPE STABILITY

5.1 Input Parameters

The properties of the natural soils encountered in the test pits and bore holes were estimated using laboratory testing, published correlations[2], and our experience with similar soils. Accordingly, we estimated the following parameters for use in the stability analyses:

Material	Internal Friction Angle (degrees)	Apparent Cohesion (psf)	Unit Weight (pcf)
Topsoil (CLAY)	24	100	115
CLAY with some Gravel and Sand	24	100	120
Norwood Formation Bedrock	30	500	130

The stability analyses provided are based on **Figure 7, Cross Section A-A'** and represent the existing slope conditions and do not include any future grading, which plans we have not been provided for. CMT must review future grading plans.

The pseudostatic coefficient for the seismic analyses was obtained by taking half of the modified peak ground acceleration (0.46g) queried for the site which resulted in a value of 0.23g.

5.2 Stability Analyses

We evaluated the global stability of the cross-section A-A' located as shown on **Figures 7**. The analysis was completed using the computer program *SLIDE* version 7.0. This program uses a limit equilibrium (Simplified Bishop) method for calculating factors of safety against sliding on an assumed failure surface and evaluates numerous potential failure surfaces, with the most critical failure surface identified as the one yielding the lowest factor of safety of those evaluated. Typically, the required minimum factors of safety are 1.5 for static conditions and 1.0 for seismic (pseudostatic) conditions.

[2] U.S. Bureau of Reclamation, 1987, "Design Standards No. 13, Embankment Dams," Denver, Colorado.

A projected water (phreatic) surface was incorporated in the analysis based on potential seasonal conditions at a depth of about 10 feet in the bottom of the unnamed drainage swale and along Smith Creek, and deepening in the ridge between the drainages (see **Figure 7 Cross Section**). Following is the cross section we analyzed and our evaluation results:

- Cross-section A-A' consists of a 495-foot long horizontal cross section with an overall elevation change of about 88 feet and slope gradient of about 5.6:1 (horizontal:vertical) downward to the north-northeast (See **Figure 7, Cross Section**). Based on the slope stability analysis, the current slope has factors of safety for both static and pseudo-static (earthquake) conditions in excess of those typically considered acceptable. The failure surfaces with the lowest factors of safety are shown on the stability analysis plot, with the lowest calculated factor of safety displayed. See **Figures 8 and 9**.

Slope movements or even failure can occur if the slope soils are undermined or become saturated. Any planned retaining walls must be properly engineered, including stability analyses. Proposed 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, we recommend the slope surface must be re-vegetated as soon as possible to limit erosion. 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.

5.3 Site Drainage and Irrigation

Proper site drainage is important to maintaining slope stability at the site. The surface of the site should be graded to prevent the accumulation or ponding of surface water at the site. It is anticipated that little to no landscape watering will occur. Landscaping if/as incorporated at the site should be planned to utilize native, drought resistant plants that require minimal watering.

6.0 LABORATORY TESTING

6.1 General

Selected samples of the subsurface soils were subjected to various laboratory tests to assess pertinent engineering properties, as follows:

1. Moisture Content, ASTM D-2216, Percent moisture representative of field conditions
2. Dry Density, ASTM D-2937, Dry unit weight representing field conditions
3. Atterberg Limits, ASTM D-4318, Plasticity and workability
4. Gradation Analysis, ASTM D-1140/C-117, Grain Size Analysis
5. One Dimension Consolidation, ASTM D-2435, Consolidation properties

6.2 Lab Summary

Laboratory test results are presented in the following Lab Summary Table:

LAB SUMMARY TABLE

Test Pit No	Depth (feet)	Soil Class	Sample Type	Moisture Content (%)	Dry Denstiy (pcf)	Gradation			Atterberg Limits			Collapse (-) or Expansion (+)
						Grav	Sand	Fines	LL	PL	PI	
TP-4	2	CL-CH	Block	18.9	99			83	50	28	22	5.6

6.3 One-Dimensional Consolidation Tests

To provide data necessary for our settlement analyses, a consolidation test was completed on the clay soil encountered in test pit TP-4 at a depth of about 2 feet below the ground surface. The data obtained from the tests indicated that the clay soil could swell when wetted up to about 5.6 percent under light loading conditions (floor slab loading range). Further the clay soil tested is moderately over-consolidated and will exhibit moderate strength and compressibility characteristics under the anticipated foundation loadings. Detailed results of the tests are maintained within our files and can be transmitted to you, upon your request.

7.0 SITE PREPARATION AND GRADING

7.1 General

All deleterious materials should be stripped from the site prior to commencement of construction activities. This includes loose and disturbed soils, topsoil, vegetation, etc. The removal of any surface vegetation, topsoil, and any other deleterious materials shall extend out at least 3 feet beyond new structures and 2 feet beyond flatwork. Based upon the conditions observed in the test pits there is topsoil on the surface of the site which we estimated to be about one to two feet in thickness. Where scrub oak exists and is removed, larger roots (greater than about ½ inch) will likely extend deeper and should be removed beneath the residence in those localized areas. Also, any existing undocumented fill shall be removed from beneath the structure.

Surficial clay soils and highly weathered claystone bedrock was encountered at the test pits locations extending to depths of about 6.5 to 9.0 feet at the test pit locations. Some of these soils exhibit high plasticity and potential swell characteristics when wetted which may induce unwanted movement of structural elements. As discussed previously, test results indicate that under light loading conditions, these soils could swell up to about 5.6 percent. Therefore, where floor slabs will bear within this upper clay soil sequence, it is recommended that a minimum of 30 inches of low plasticity, non-expansive granular structural replacement fill, be placed directly below floor slabs. It is recommended that the foundation extend to suitable bedrock where it appears to be relatively shallow. However, if foundation locations necessitate, they be placed within the upper clay soil sequence, a minimum of 18 inches of low plasticity, non-expansive granular structural replacement fill, shall be placed directly below foundations. This structural fill is recommended to have a minimum of 15 percent low plastic fines (lean clay/silt) to reduce permeability. Further, care must be taken in order to minimize drying of the exposed clay soil prior to installed the replacement structural fill.

The site should be examined by a CMT geotechnical engineer to assess that suitable natural soils have been exposed and any deleterious materials, loose and/or disturbed soils have been removed, prior to placing site grading/structural fills, footings, slabs, and pavements.

Site grading fill should be placed on relatively level surfaces and against relatively vertical surfaces. Thus, where the existing slope is steeper than about 5H:1V (Horizontal:Vertical), the existing ground should be benched to create horizontal and vertical surfaces for receiving the fill. We recommend maximum bench heights of about 30 inches.

7.2 Temporary Excavations

Excavations deeper than about 10 feet are not anticipated at the site. Groundwater was not encountered within the depths explored, up to about 10 feet at the time of our field explorations, and thus is not anticipated to affect excavations.

The natural soils encountered at this site predominantly consisted of silty and fine sandy CLAY(CL-CH) overlying SAND with varying fines (SC/SM), SANDSTONE, and CLAYSTONE to the full depth penetrated, about 10 feet. In clayey (cohesive) soils, temporary construction excavations not exceeding 4 feet in depth may be constructed with near-vertical side slopes. Temporary excavations up to 10 feet deep, above or below groundwater, may be constructed with side slopes no steeper than one-half horizontal to one vertical (0.5H:1V).

For sandy/gravelly (cohesionless) soils, temporary construction excavations not exceeding 4 feet in depth and above the groundwater should be no steeper than one-half horizontal to one vertical (0.5H:1V). For excavations up to 10 feet and above groundwater, side slopes should be no steeper than one horizontal to one vertical (1H:1V). Excavations encountering saturated cohesionless soils will be very difficult to maintain, and will require very flat side slopes and/or shoring, bracing and dewatering.

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.

7.3 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 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 anticipate will be used at this site:

Fill Material Type	Description/Recommended Specification
Structural Fill/Replacement Fill	Placed below structures, flatwork and pavement. Imported structural fill should consist of well-graded sand/gravel mixture, with maximum particle size of 4 inches, a minimum 70% passing 3/4-inch sieve, between 15 and 30% passing the No. 200 sieve, and a maximum Plasticity Index of 10.
Site Grading Fill	Placed over larger areas to raise the site grade. Sandy to gravelly soil, with a maximum particle size of 6 inches, a minimum 70% passing 3/4-inch sieve, and a maximum 40% passing No. 200 sieve.
Non-Structural Fill	Placed below non-structural areas, such as landscaping. On-site soils or imported soils, with a maximum particle size of 8 inches, including silt/clay soils not containing excessive amounts of degradable/organic material.
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 (see **Section 7.6**).

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

7.4 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[3] 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 3 feet beyond the perimeter of structures, and 2 feet beyond below flatwork and pavement (applies to structural fill and site grading fill)	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

[3] American Association of State Highway and Transportation Officials

Structural fills 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.

7.5 Utility Trenches

For the bedding zone around the utility, we recommend utilizing sand bedding fill material that meets current APWA[4] requirements.

All utility trench backfill material below structurally loaded facilities (foundations, floor slabs, flatwork, parking lots/drive areas, etc.) should be placed at the same density requirements established for structural fill in the previous section.

Most utility companies and City-County governments are now requiring that Type A-1a or A-1b (AASHTO Designation – basically granular soils with limited fines) soils be used as backfill over utilities. Processed natural on-site soil may meet these requirements.

Where the utility does not underlie structurally loaded facilities and public rights of way, on-site 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**.

7.6 Soil Stabilization

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 7.3**. 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.

8.0 FOUNDATION RECOMMENDATIONS

The following recommendations have been developed on the basis of the previously described Project characteristics, including the maximum loads discussed in **Section 1.3**, the subsurface conditions observed in the field and the laboratory test data, and standard geotechnical engineering practice.

[4] American Public Works Association

8.1 Foundation Recommendations

Based on our geotechnical engineering analyses, the proposed residential structure may be supported upon conventional spread and/or continuous wall foundations placed on a minimum of 18 inches of properly placed and compacted structural fill extending to suitable natural soils. This structural fill requirement may be reduced to 12 inches for foundations excavation extending entirely on suitable bedrock. Footings may then be designed using a net bearing pressure of 2,500 psf. The term “net bearing pressure” refers to the pressure imposed by the portion of the structure located above lowest adjacent final grade, thus the weight of the footing and backfill to lowest adjacent final grade need not be considered. The allowable bearing pressure may be increased by 1/3 for temporary loads such as wind and seismic forces.

We also recommend the following:

1. Exterior footings subject to frost should be placed at least 36 inches below final grade.
2. Interior footings not subject to frost should be placed at least 16 inches below grade.
3. Continuous footing widths should be maintained at a minimum of 18 inches.
4. Spot footings should be a minimum of 24 inches wide.

8.2 Installation

Under no circumstances shall foundations be placed on undocumented fill, topsoil with organics, sod, rubbish, construction debris, other deleterious materials, frozen soils, or within ponded water.

Deep, large roots may be encountered where trees and larger bushes are located or were previously located at the site; such large roots should be removed. If unsuitable soils are encountered, they must be completely removed and replaced with properly compacted structural fill. Excavation bottoms should be examined by a qualified geotechnical engineer to confirm that suitable bearing materials soils have been exposed.

All structural fill should meet the requirements for such, and should be placed and compacted in accordance with **Section 7** above. The width of structural replacement fill below footings should be equal to the width of the footing plus 1 foot for each foot of fill thickness. For instance, if the footing width is 2 feet and the structural fill depth beneath the footing is 1.5 feet, the fill replacement width should be 3.5 feet, centered beneath the footing.

8.3 Estimated Settlement

Foundations designed and constructed in accordance with our recommendations could experience some settlement, but we anticipate that total settlements of footings founded as recommended above will not exceed 1 inch. We expect approximately 50% of the total settlement to initially take place during construction.

8.4 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 0.30 for natural silt/clay soils or 0.40 for structural fill, may be utilized for design. Passive resistance provided by properly placed and compacted structural fill above the water table may be considered equivalent to a fluid with a density of 250 pcf. A combination of passive earth resistance and friction may be utilized if the friction component of the total is divided by 1.5.

9.0 LATERAL EARTH PRESSURES

We project that basement walls up to 8 feet tall might be constructed at this site. Parameters, as presented within this section, are for backfills which will consist of drained 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 40 pounds per cubic foot in computing lateral pressures. For more rigid walls (moderately yielding), backfill may be considered equivalent to a fluid with a density of 50 pounds per cubic foot. For very rigid non-yielding walls, granular backfill should be considered equivalent to a fluid with a density of at least 60 pounds per cubic foot. The above values assume that the surface of the soils slope behind the wall is horizontal and that the fill within 3 feet of the wall will be compacted with hand-operated compacting equipment.

For seismic loading of retaining/below-grade walls, the following uniform lateral pressures, in pounds per square foot (psf), should be added based on wall depth and wall case.

Uniform Lateral Pressures			
Wall Height (Feet)	**Active Pressure Case (psf)**	**Moderately Yielding Case (psf)**	**At Rest/Non-Yielding Case (psf)**
4	22	47	71
6	34	70	107
8	45	94	143

10.0 FLOOR SLABS

Floor slabs may be established upon a minimum of 30 inches of properly placed and compacted structural replacement fill extending to suitable, undisturbed, natural soils. This structural fill requirement may be reduced to 12 inches for excavations extending entirely to suitable bedrock. Under no circumstances shall floor slabs be established directly on any topsoil, potentially expansive soil, 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, we recommend that floor slabs be directly underlain by at least 4 inches of “free-draining” fill, such as “pea” gravel or 3/4-inch to 1-inch minus, clean, gap-graded gravel or granular structural fill as outlined in this report. To help control normal shrinkage and stress cracking, the floor slabs may include 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.

11.0 DRAINAGE RECOMMENDATIONS

11.1 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. 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 4 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.

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 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.

11.2 Subdrains

Due to the potential for random perched groundwater conditions within the predominantly fine grained soils and bedrock, which may occur against sublevel foundations, it is recommended that a foundation drain be installed around the home.

Foundation subdrains should consist of a 4-inch diameter perforated or slotted 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 12-inch-wide zone of “free-draining” clean sand or gravel (chimney) should be placed adjacent to the foundation walls and extend to within 2 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 2 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 or 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 sub drain, the below-grade walls should be damp proofed. The slope of the sub drain should be at least 0.3 percent. The foundation sub drains shall be discharged to down-gradient location well away from the home.

12.0 QUALITY CONTROL

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

12.1 Field Observations

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

12.2 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 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.

12.3 Excavations

All excavation procedures and processes should be observed by a geotechnical engineer from CMT or their representative. In addition, for the recommendations in this report to be valid, all backfill and structural fill placed in trenches and all pavements should be density tested by CMT. We recommend that freshly mixed concrete be tested by CMT in accordance with ASTM designations.

13.0 LIMITATIONS

The recommendations provided herein were developed by evaluating the information obtained from the subsurface explorations and soils encountered therein. The exploration logs reflect the subsurface conditions only at the specific location at the particular time designated on the 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. If we can be of further assistance or if you have any questions regarding the Project, please do not hesitate to contact us at (801) 870-6730. To schedule materials testing, please call (801) 381-5141.

14.0 REFERENCES

Anderson, R.E., 1989, Tectonic evolution of the intermontane system--Basin and Range, Colorado Plateau, and High Lava Plains, in Pakiser, L.C., and Mooney, W.D., editors, Geophysical framework of the continental United States: Geological Society of America Memoir 172, p. 163-176.

Arabasz, W.J., Pechmann, J.C., and Brown, E.D., 1992, Observational seismology and evaluation of earthquake hazards and risk in the Wasatch Front area, Utah, in Gori, P.L. and Hays, W.W., editors, Assessment of Regional Earthquake Hazards and Risk along the Wasatch Front, Utah: Washington, D.C, U.S. Geological Survey Professional Paper 1500-D, Government Printing Office, p. D1-D36.

Black, B.D., Hecker, Suzanne, Hylland, M.D., Christenson, G.E., and McDonald, G.N., 2003, Quaternary fault and fold database and map of Utah: Utah Geological Survey Map 193DM, CD-ROM.

Boore, D.M., Joyner, W.B., and Fumal, T.E., 1993, Estimation of Response Spectra and Peak Acceleration from Western North America Earthquakes--An interim report: U.S. Geological Survey Open-File Report 93-509.

Bowman, S.D., and Lund, W.R., 2016, Guidelines for conducting engineering-geology investigations and preparing engineering-geology reports in Utah, in Bowman, S.D., and Lund, W.R., editors, Guidelines for investigating geologic hazards and preparing engineering-geology reports, with a suggested approach to geologic-hazard ordinances in Utah: Utah Geological Survey Circular 122, p. 15–30.

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, scale 1:100,000, 141 p. with appendices.

Cruden, D.M. and Varnes, D.J., 1996, Landslide Types and Processes, in Turner, A.K. and Shuster, R.L., editors, Landslides--Investigation and Mitigation: Washington D.C., Transportation Research Board Special Report No. 247, 36-75.

Gilbert, G.K., 1928, Studies of Basin and Range Structure: U.S. Geological Survey Professional Paper 153, 89 p.

Gwynn, J.W. (Editor), 1980, Great Salt Lake--A scientific, historical, and economic overview: Utah Geological Survey Bulletin 166, 400 p.

Jarrett, R.D., and Malde, H.E., 1987, Paleodischarge of the late Pleistocene Bonneville flood, Snake River, Idaho, computed from new evidence: Geological Society of America Bulletin, v. 99, p. 127-134.

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.

Lund, W.R. (Editor), 1990. Engineering geology of the Salt Lake City metropolitan area, Utah: Utah Geological and Mineral Survey Bulletin 126, 66 p.

McCalpin, J.P., 1996, Paleoseismology: San Diego, California, Academic Press Inc., Volume 62 of the International Geophysical Series, 588 p.

Miller, D.M., 1990, Mesozoic and Cenozoic tectonic evolution of the northeastern Great Basin, in Shaddrick, D.R., Kizis, J.R., and Hunsaker, E.L. III, editors, Geology and Ore Deposits of the Northeastern Great Basin: Geological Society of Nevada Field Trip No. 5, p. 43-73.

O'Connor, J.E., 1993, Hydrology, hydraulics, and geomorphology of the Bonneville flood: Geological Society of America Special Paper 274, 83 p.

Oviatt, C.G., 2015, Chronology of Lake Bonneville, 30,000 to 10,000 yr B.P.: Quaternary Science Reviews, v. 110 (2015), p. 166-171.

Oviatt, C.G., Currey, D.R., and Sack, Dorothy, 1992, Radiocarbon chronology of Lake Bonneville, Eastern Great Basin, USA: Paleogeography, Paleoclimatology, Paleoecology, v. 99, p. 225-241.

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.

Sbar, M.L., Barazangi, M., Dorman, J., Scholz, C.H., and Smith, R.B., 1972, Tectonics of the Intermountain Seismic Belt, western United States--Microearthquake seismicity and composite fault plane solutions: Geological Society of America Bulletin, v. 83, p. 13-28.

Smith, R.B., and Arabasz, W.J., 1991, Seismicity of the Intermountain Seismic Belt, in Slemmons, D.B., Engdahl, E.R., Zoback, M.D., and Blackwell, D.D., editors, Neotectonics of North America: Geological Society of America, Decade of North American Geology Map v. 1, p. 185-228.

Smith, R.B. and Sbar, M.L., 1974, Contemporary tectonics and seismicity of the western United States with emphasis on the Intermountain Seismic Belt: Geological Society of America Bulletin, v. 85, p. 1205-1218.

Stewart, J.H., 1978, Basin-range structure in western North America, a review, in Smith, R.B., and Eaton, G.P., editors, Cenozoic tectonics and regional geophysics of the western Cordillera: Geological Society of America Memoir 152, p. 341-367.

____, 1980, Geology of Nevada: Nevada Bureau of Mines and Geology Special Publication 4.

Stokes, W.L., 1977, Physiographic subdivisions of Utah: Utah Geological and Mineral Survey Map 43, scale 1:2,400,000.

____, 1986, Geology of Utah: Salt Lake City, University of Utah Museum of Natural History and Utah Geological and Mineral Survey, 280 p.

Sullivan, J.T., Nelson, A.R., LaForge, R.C., Wood, C.K., and Hansen, R.A., 1986, Regional seismotectonic study for the back valleys of the Wasatch Mountains in northeastern Utah: Denver, Colorado, U.S. Bureau of Reclamation, Seismotectonic Section, Division of Geology, Engineering and Research Center, unpublished report, 317 p.

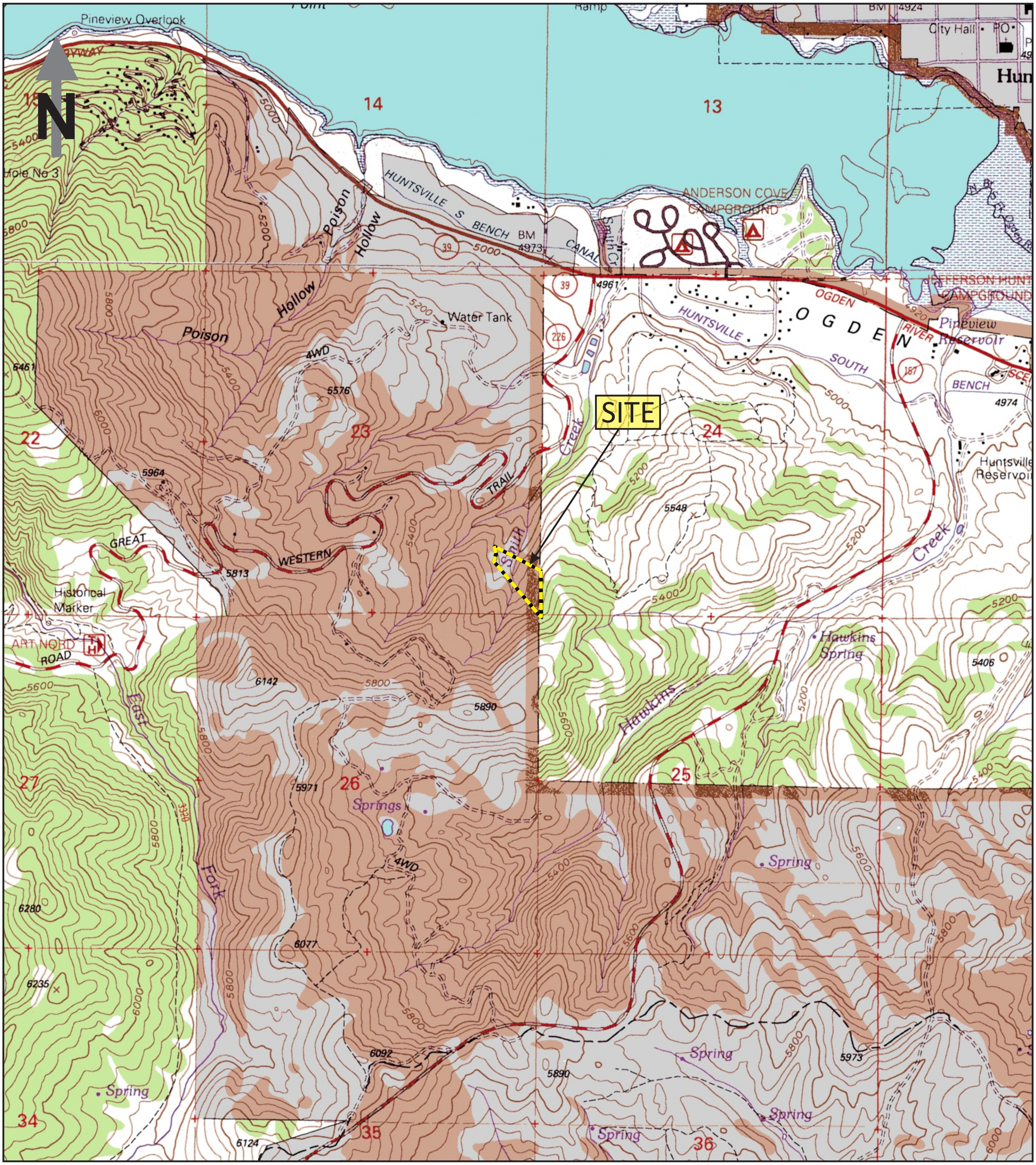
Varnes, D. J., 1978, Slope movement types and processes, in Schuster, R.L and Krizek, R.J, editors, Landslides Analysis and control: Washington D.C., Transportation Research Board Special Report 176, p. 11-33.

Zoback, M.L., 1989. State of stress and modern deformation of the northern Basin and Range province: Journal of Geophysical Research, v. 94, p. 7105-7128.

Zoback, M.L. and Zoback, M.D., 1989. Tectonic stress field of the conterminous United States: Boulder, Colorado, Geological Society of America Memoir, v. 172, p. 523-539.

APPENDIX

SUPPORTING DOCUMENTATION



Base:
USGS 7.5-minute topographic quadrangles,
Utah; HUNTSVILLE and SNOW BASIN, 1998.

0 2,000 ft 4,000 ft

1:24,000 (1 inch equals 2,000 feet)

Proposed Paul Coles Cabin
Tolliver Lane, Huntsville, Weber County, Utah

CMT ENGINEERING LABORATORIES

Vicinity Map

Date:	20-Sep-2020
Job #	15297

Figure
1

Base:
Interim Geologic Map of the Ogden 30' x 60' Quadrangle (Coogan and King, 2016).

0 2,000 ft 4,000 ft

1:24,000 (1 inch equals 2,000 feet)

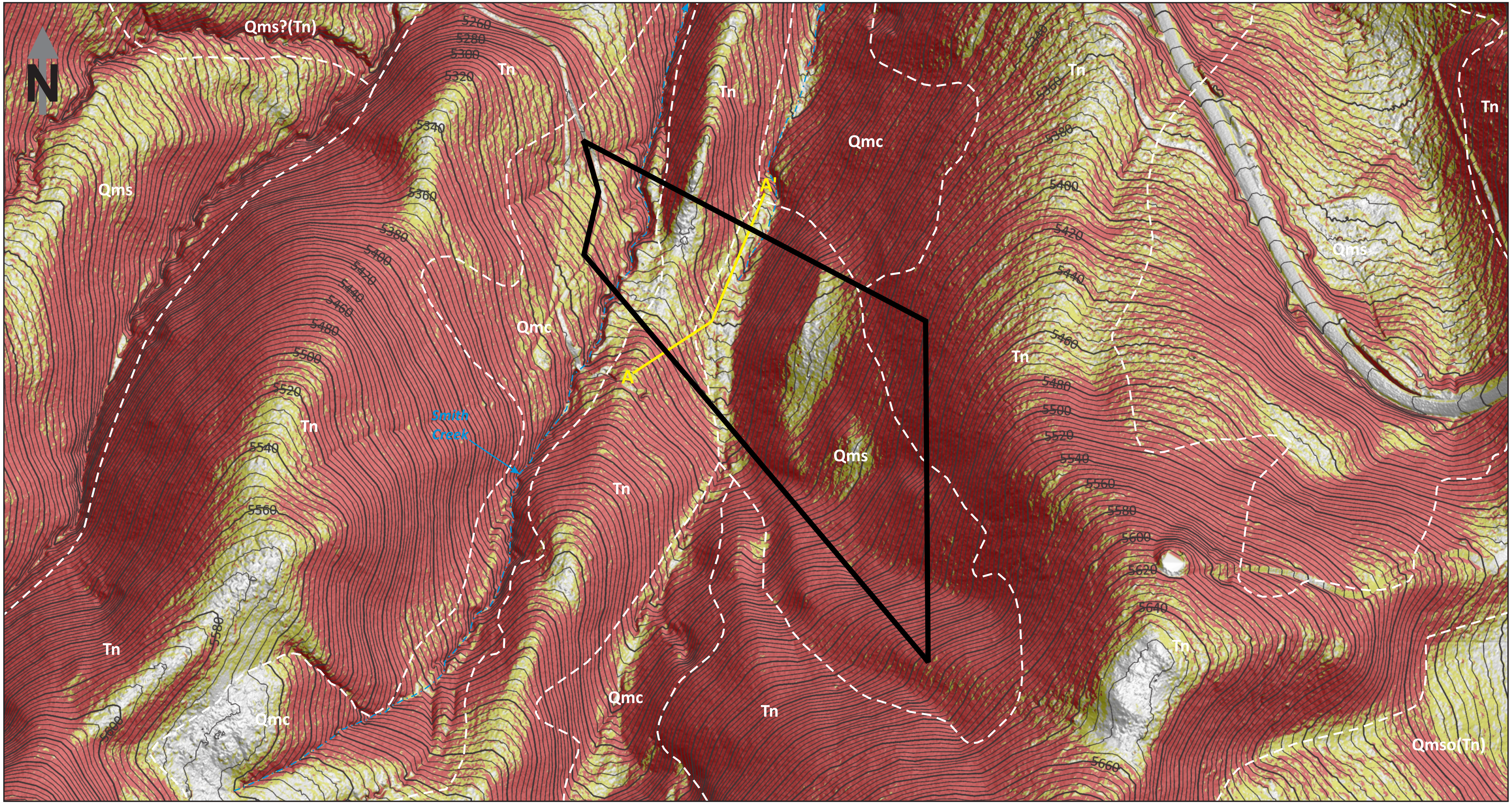
Proposed Paul Coles Cabin
Tolliver Lane, Huntsville, Weber County, Utah

CMT ENGINEERING LABORATORIES

Geologic Map

Date:	20-Sep-2020
Job #	15297

Figure
2

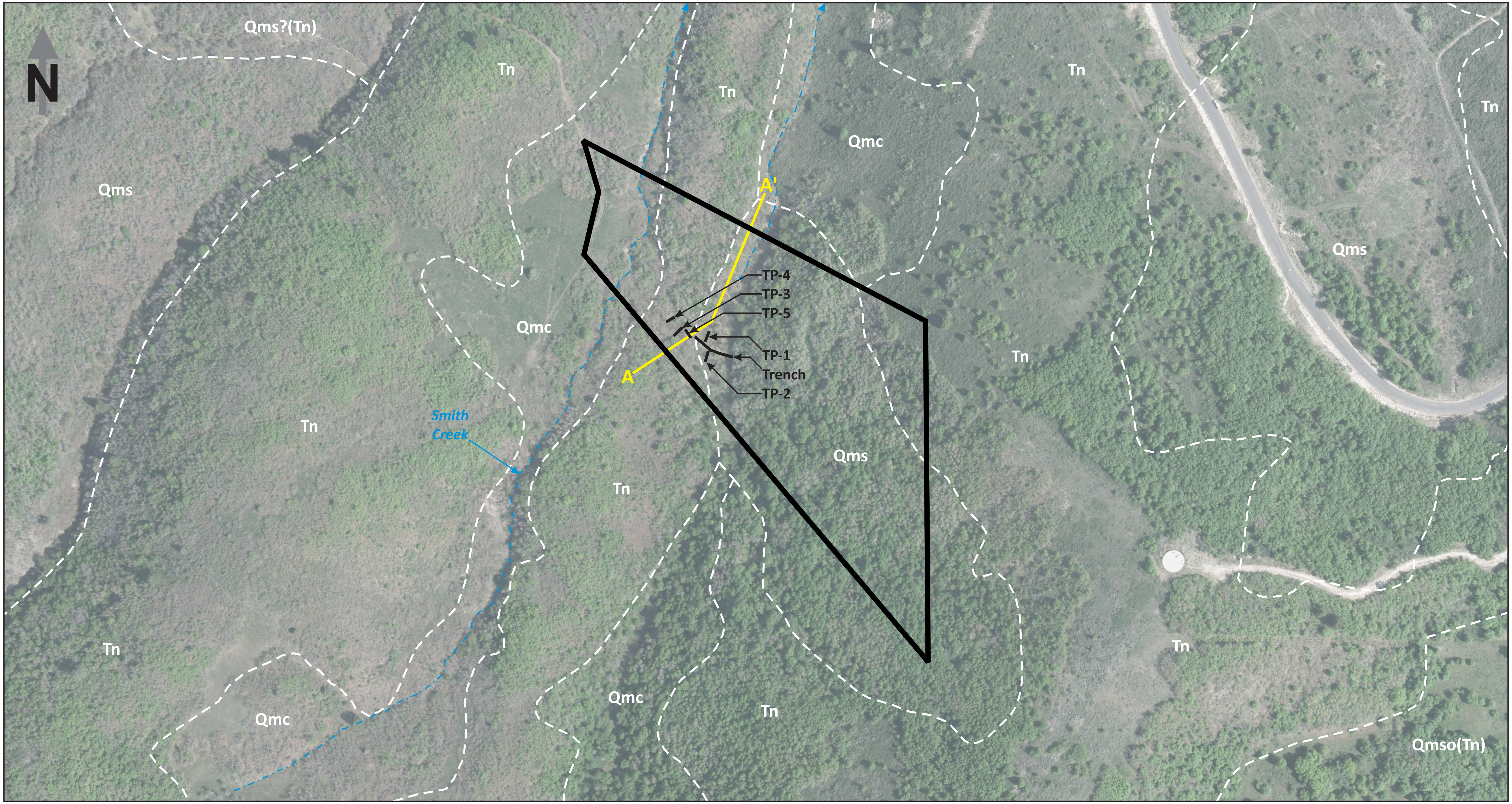


Base:
2016 LIDAR imagery available from Utah AGRC, 0.5 m resolution; slope steepness >25% shaded in red, 15-25% in yellow, and <15% unshaded. Contours generated by Global Mapper at 4-foot intervals. Cross section location shown in yellow. Surficial geologic mapping modified from Coogan and King (2016).

0 200 ft 400 ft

1:2,400 (1 inch equals 200 feet)

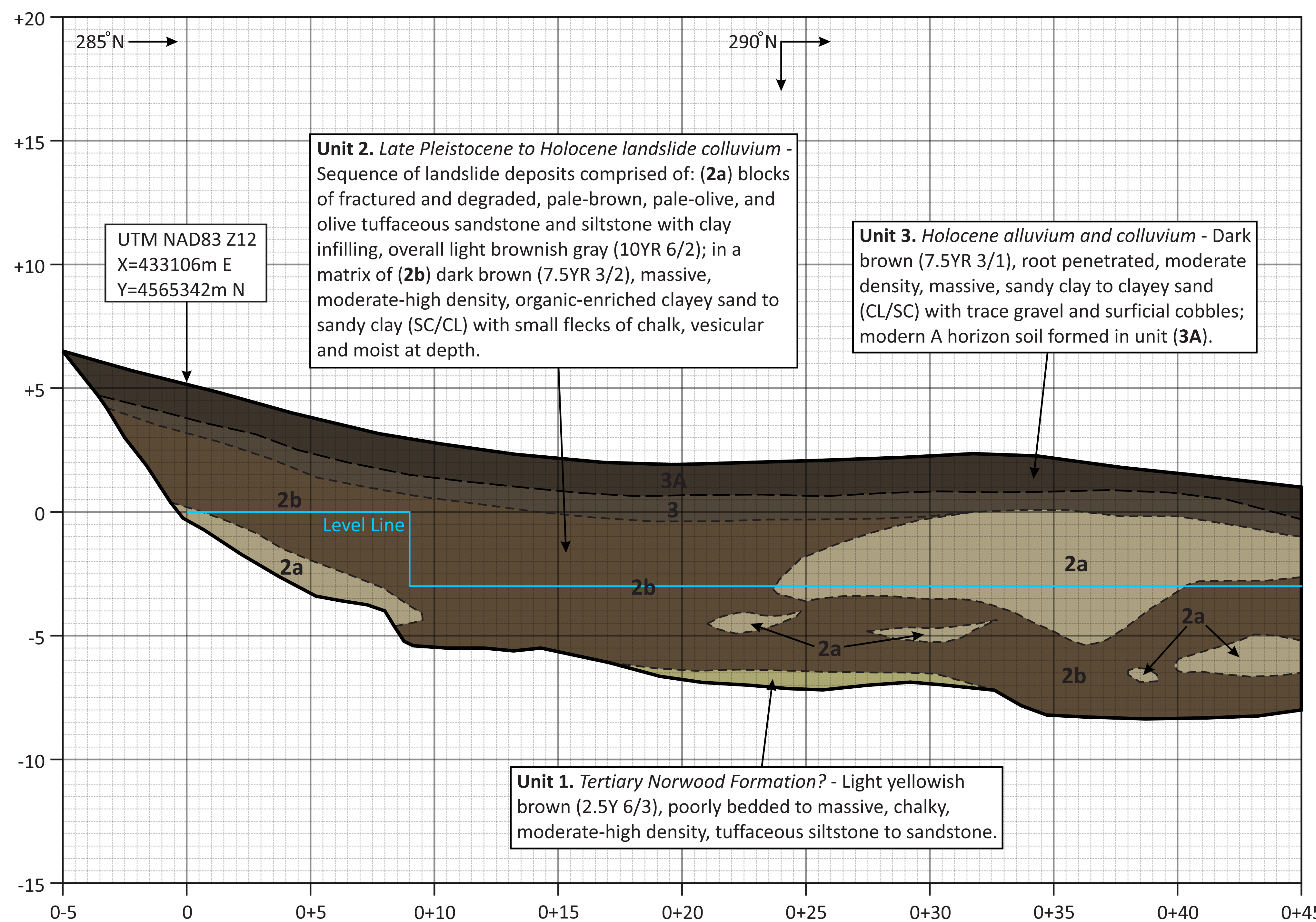
CMT ENGINEERING LABORATORIES	Proposed Paul Coles Cabin Tolliver Lane, Huntsville, Weber County, Utah	LIDAR Analysis	Date:	20-Sep-2020	Figure 3
			CMT No.:	15297	



Base:
2012 high-resolution orthophoto available from Utah AGRC, 12.5 cm resolution. Trench and test pit locations shown in black. Cross section location shown in yellow. Surficial geologic mapping modified from Coogan and King (2016).

0 200 ft 400 ft
1:2,400 (1 inch equals 200 feet)

CMT ENGINEERING LABORATORIES	**Proposed Paul Coles Cabin** Tolliver Lane, Huntsville, Weber County, Utah	**Site Evaluation**	Date:	20-Sep-2020	Figure 4
			CMT No.:	15297	

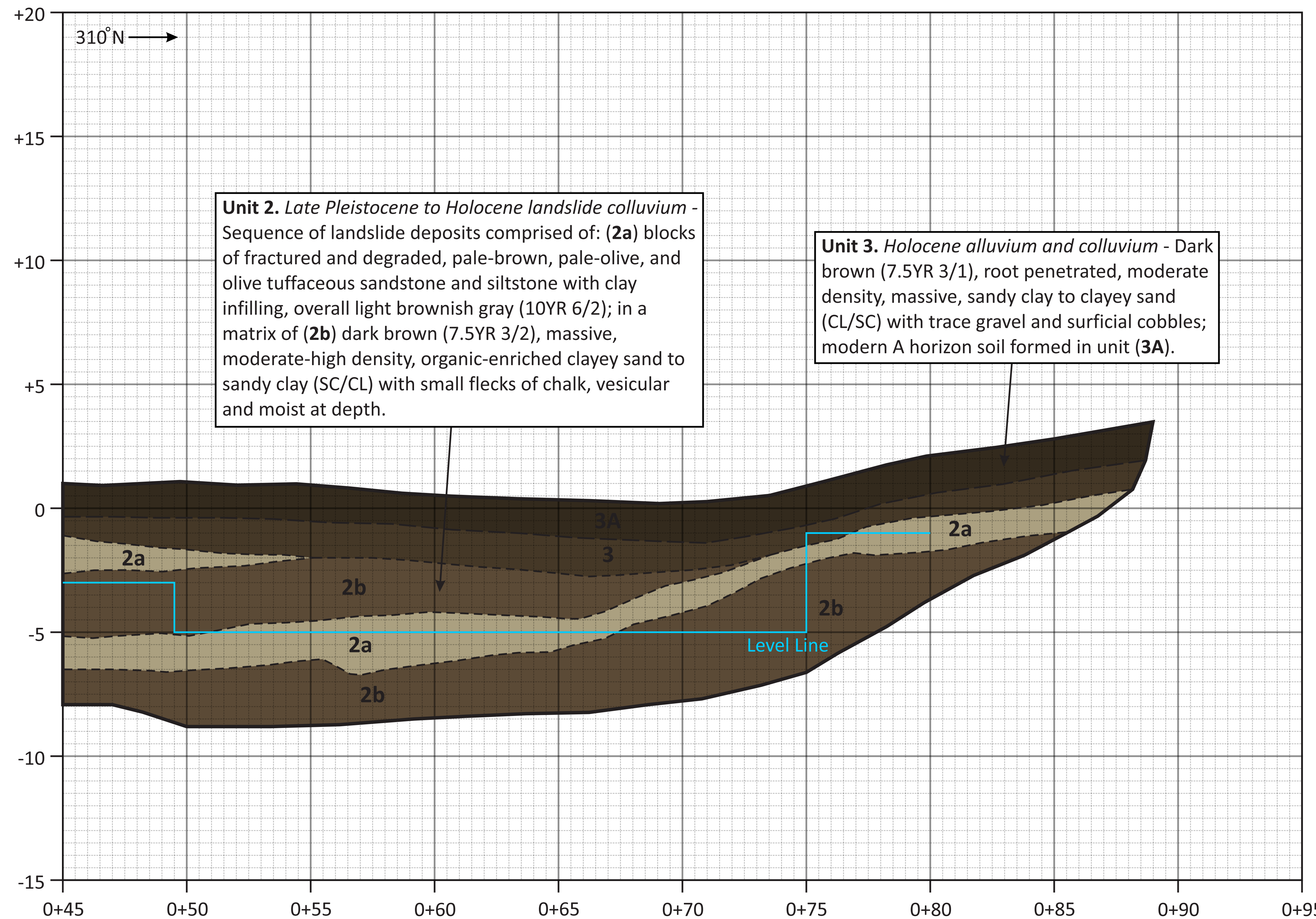


Logged by Bill D. Black, P.G. on August 27, 2020.
South wall logged, east to west. Scale 1 inch equals 5 feet (1:60).

CMT ENGINEERING LABORATORIES

Proposed Paul Coles Cabin
Tolliver Lane, Huntsville, Weber County, Utah

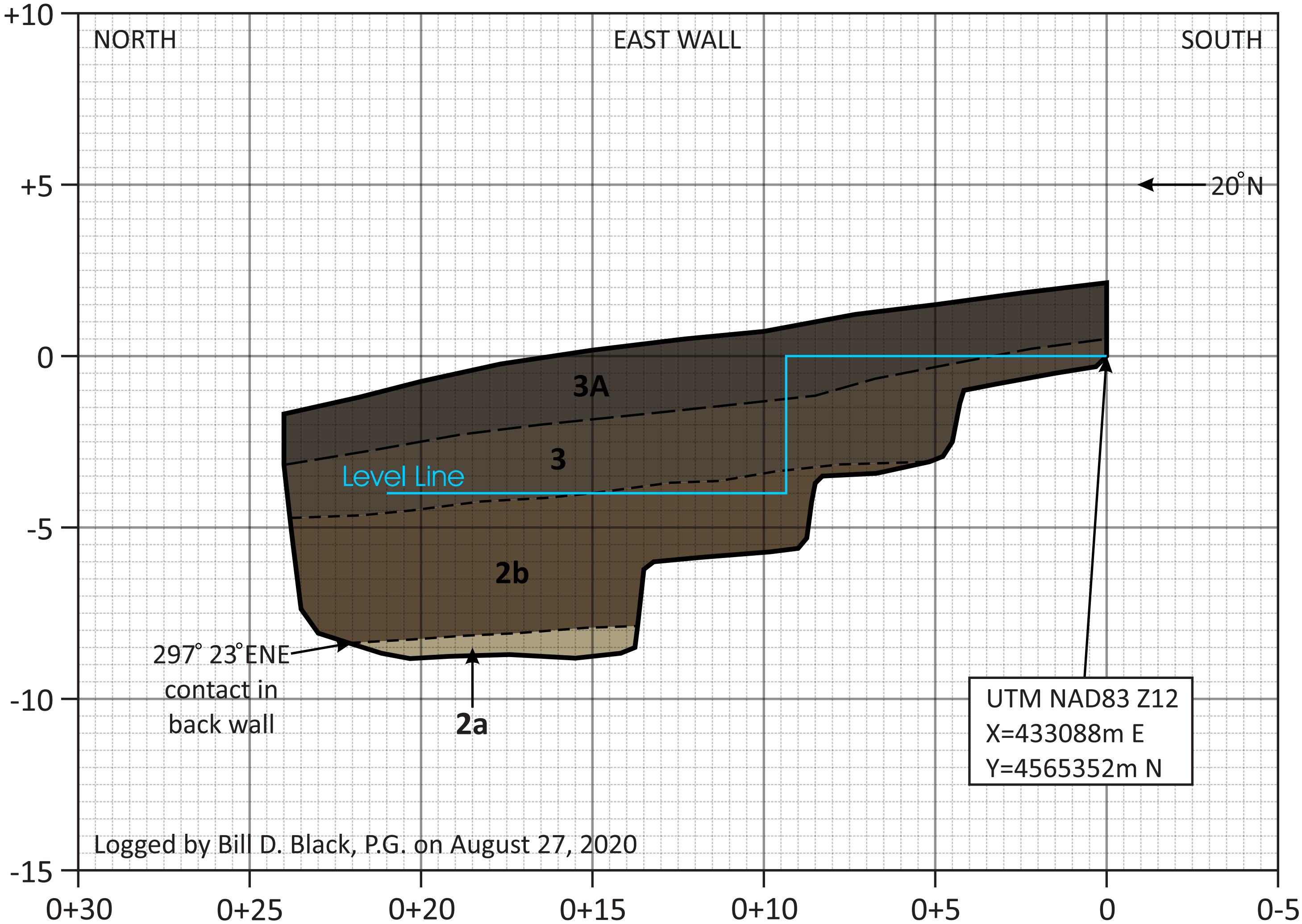
Trench Log, Sheet 1	Date:	20-Sep-2020	Figure 5A
	CMT No.:	15297	



Logged by Bill D. Black, P.G. on August 27, 2020.
South wall logged, east to west. Scale 1 inch equals 5 feet (1:60).

CMT ENGINEERING LABORATORIES	Proposed Paul Coles Cabin Tolliver Lane, Huntsville, Weber County, Utah	Trench Log, Sheet 2	Date:	20-Sep-2020	Figure 5B
			CMT No.:	15297	

TEST PIT 1



Scale in feet

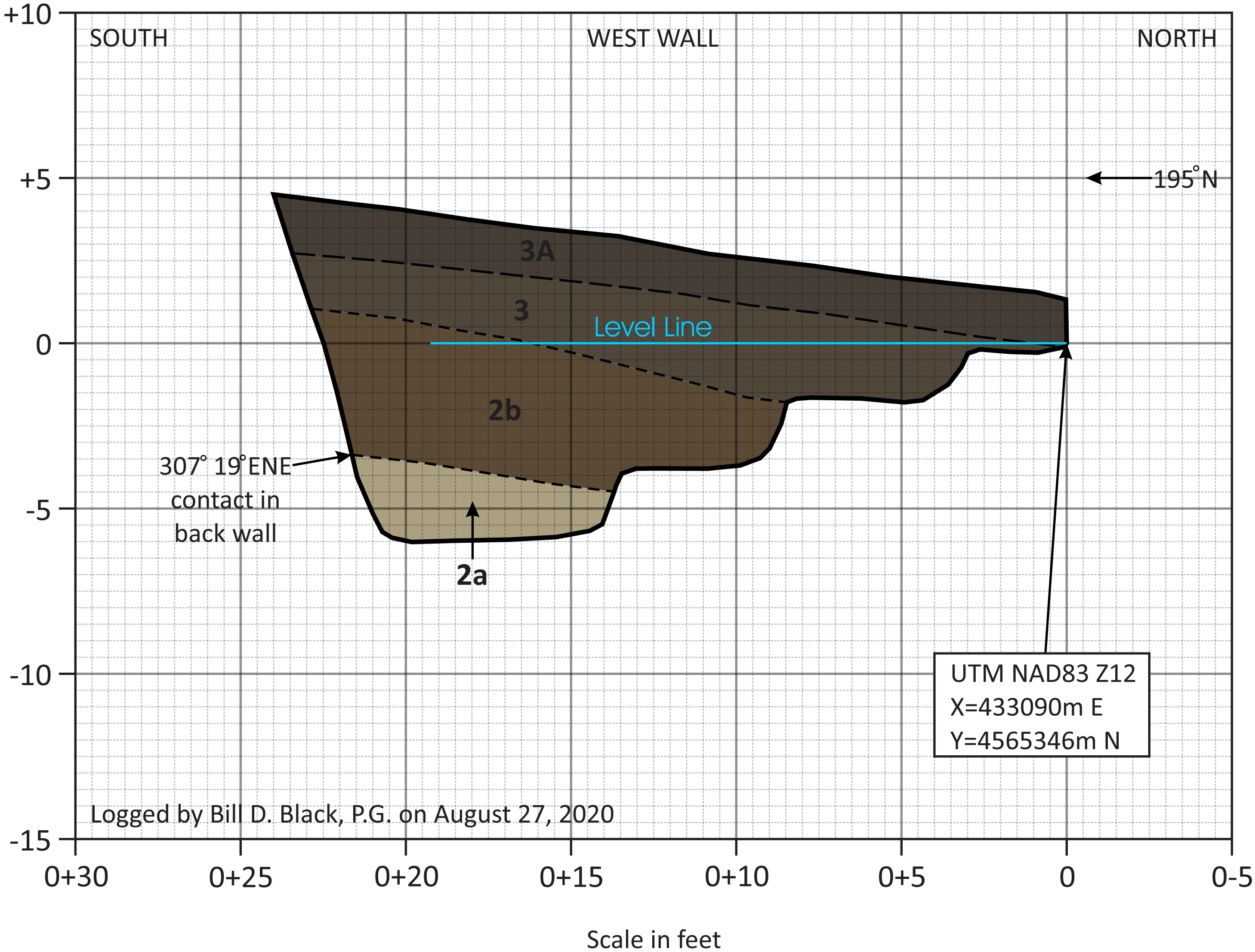
UNIT DESCRIPTIONS

Unit 2. *Late Pleistocene to Holocene landslide colluvium* - Sequence comprised of a lower (**2a**) light brownish gray (10YR 6/2), moderate-high density, poorly bedded, sandy clay to clayey sand (CL/SC), possibly rafted block of Tertiary Norwood Formation; an upper (**2b**) dark brown (7.5YR 3/2), moderate-high density, massive, sandy clay (CL) with gravel and topset cobbles of tuffaceous sandstone.

Unit 3. *Holocene alluvium and colluvium* - Dark brown (7.5YR 3/1), root penetrated, moderate density, massive, sandy clay to clayey sand (CL/SC) with trace gravel and surficial cobbles; modern A horizon soil formed in unit (**3A**).

Scale 1 inch equals 5 feet (1:60) with no vertical exaggeration.

TEST PIT 2



UNIT DESCRIPTIONS

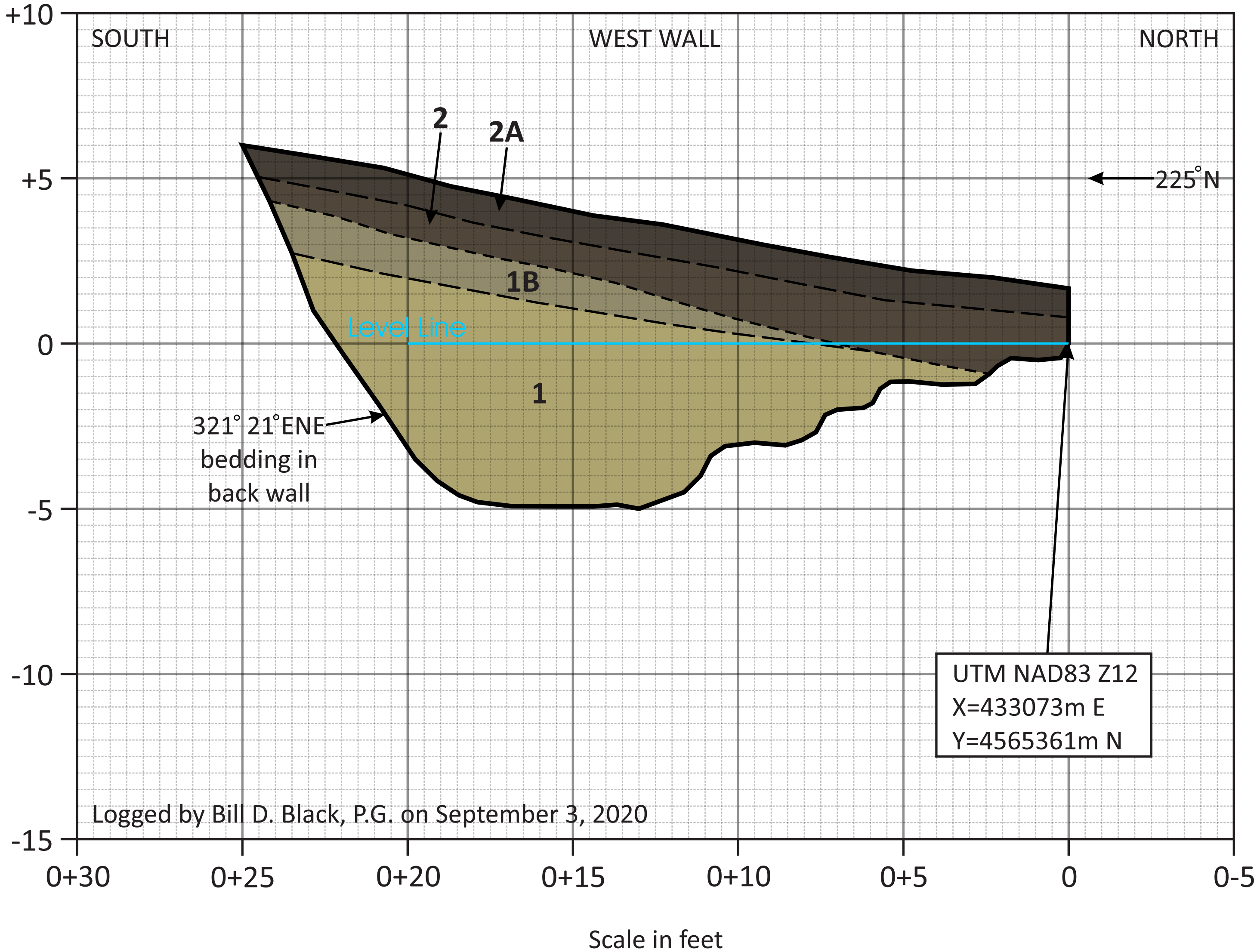
Unit 2. *Late Pleistocene to Holocene landslide colluvium* - Sequence comprised of a lower (**2a**) light brownish gray (10YR 6/2), moderate-high density, poorly bedded, sandy clay to clayey sand (CL/SC), possibly rafted block of Tertiary Norwood Formation; and an upper (**2b**) dark brown (7.5YR 3/2), moderate-high density, massive, sandy clay (CL) with gravel and cobbles of tuffaceous sandstone.

Unit 3. *Holocene alluvium and colluvium* - Dark brown (7.5YR 3/1), root penetrated, moderate density, massive, sandy clay to clayey sand (CL/SC) with trace gravel; modern A horizon soil formed in unit (**3A**).

Scale 1 inch equals 5 feet (1:60) with no vertical exaggeration.

Proposed Paul Coles Cabin Tolliver Lane, Huntsville, Weber County, Utah	CMT ENGINEERING LABORATORIES		Figure
	Test Pit 2 Log	Date: 20-Sep-2020 Job # 15297	6B

TEST PIT 3



UNIT DESCRIPTIONS

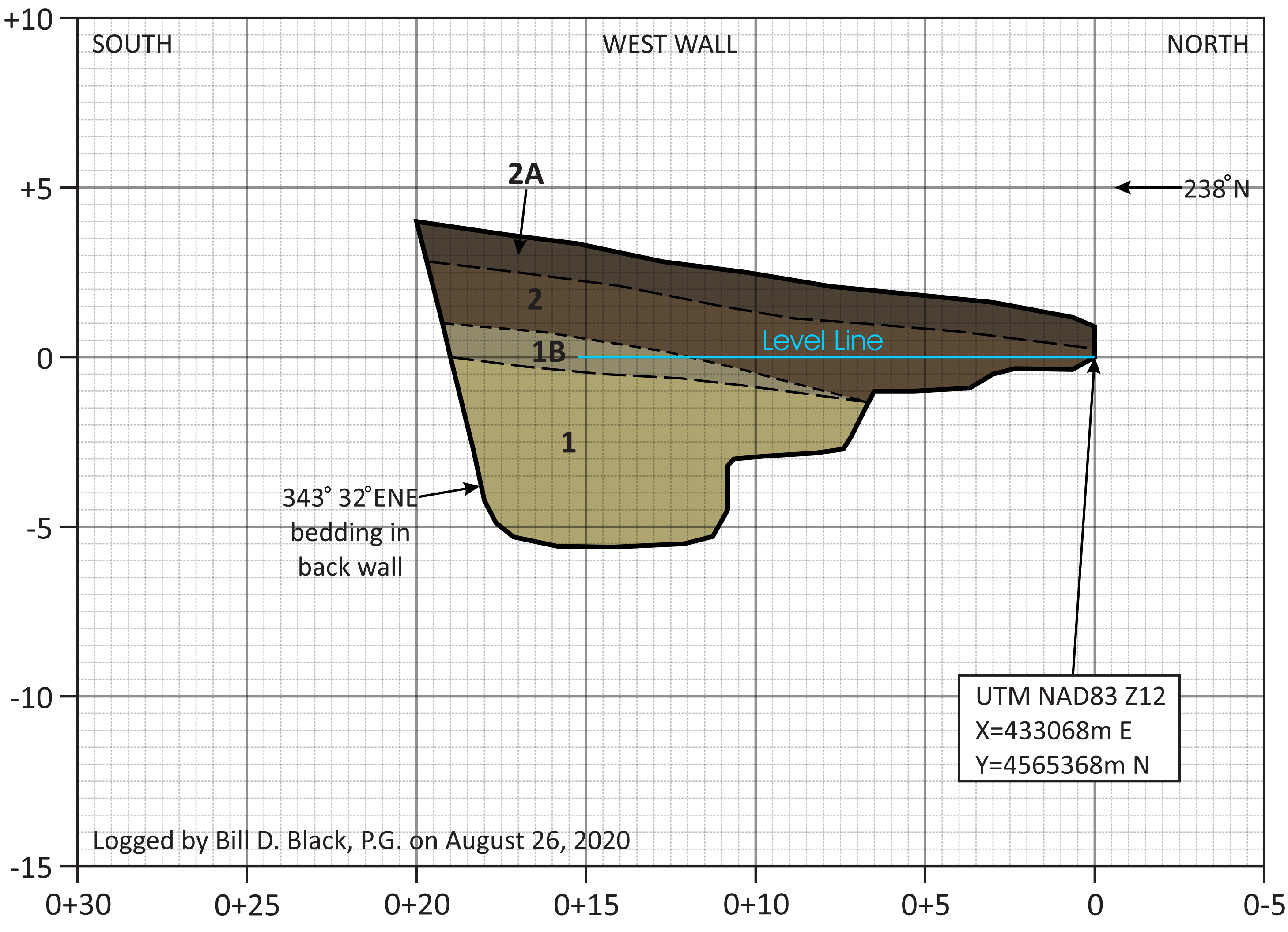
Unit 1. *Tertiary Norwood Formation* - Light yellow brown (2.5Y 6/3), dense, well bedded, tuffaceous sandstone; weathered in upper part, refusal at test pit floor; Bw/Bt horzion formed in unit (**1B**).

Unit 2. *Late Pleistocene to Holocene landslide colluvium* - Dark brown (7.5YR 3/2), moderate-high density, massive, slightly root penetrated, sandy clay (CL) with gravel and trace cobbles of pale-brown sandstone; modern A-horizon soil formed in unit at surface (**2A**).

Scale 1 inch equals 5 feet (1:60) with no vertical exaggeration.

Proposed Paul Coles Cabin Tolliver Lane, Huntsville, Weber County, Utah	CMT ENGINEERING LABORATORIES **Test Pit 3 Log**	Date: 20-Sep-2020 Job # 15297	Figure 6C

TEST PIT 4



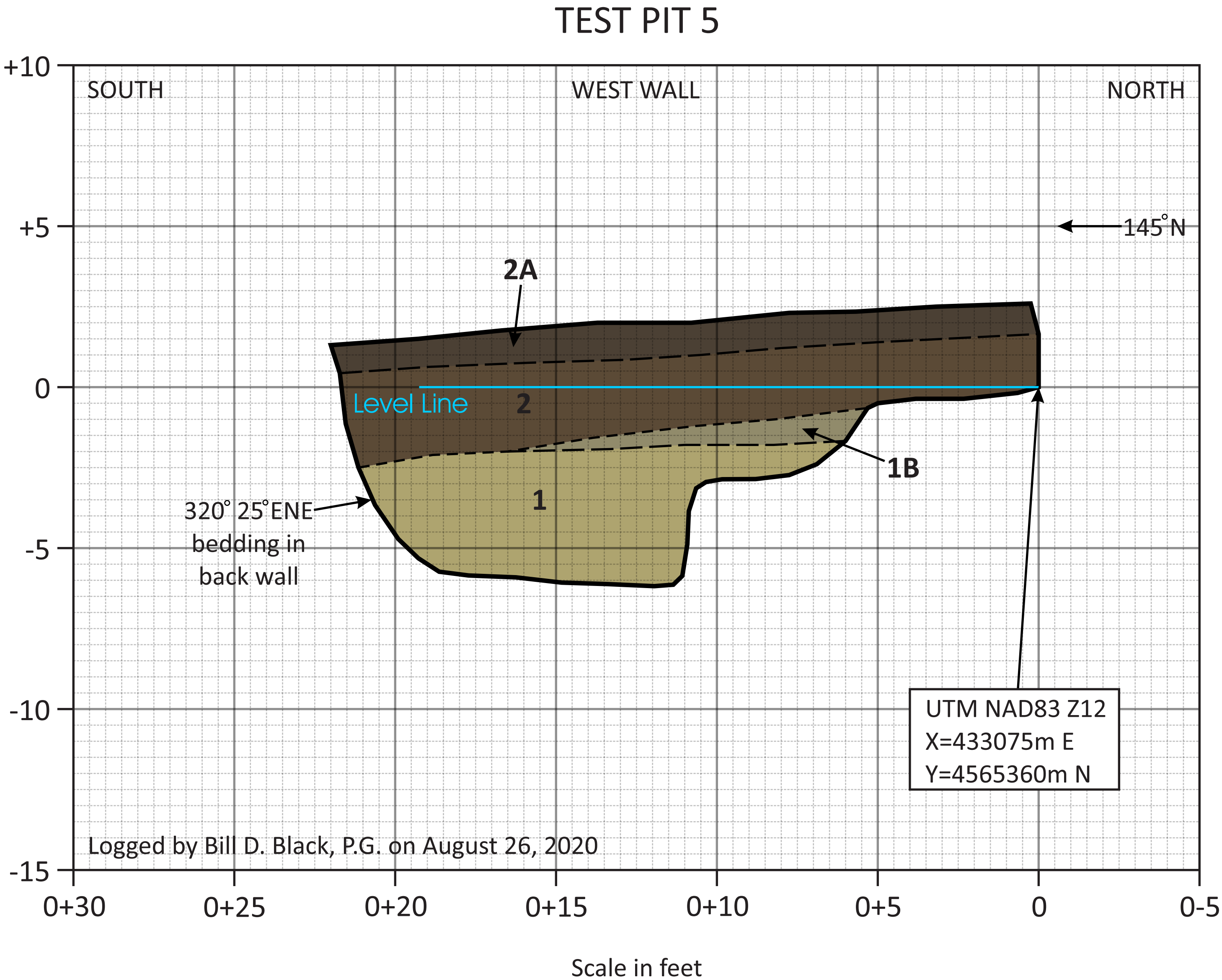
UNIT DESCRIPTIONS

Unit 1. *Tertiary Norwood Formation* - Light yellow brown (2.5Y 6/3), dense, well bedded, tuffaceous sandstone; weathered in upper part, refusal at test pit floor; Bw/Bt horzion formed in unit (**1B**).

Unit 2. *Late Pleistocene to Holocene landslide colluvium* - Dark brown (7.5YR 3/2), moderate-high density, massive, slightly root penetrated, sandy clay (CL) with gravel and trace cobbles of pale-brown sandstone; modern A-horizon soil formed in unit at surface (**2A**).

Scale 1 inch equals 5 feet (1:60) with no vertical exaggeration.

Proposed Paul Coles Cabin Tolliver Lane, Huntsville, Weber County, Utah	CMT ENGINEERING LABORATORIES **Test Pit 4 Log**	Date: 20-Sep-2020 Job # 15297	Figure 6D



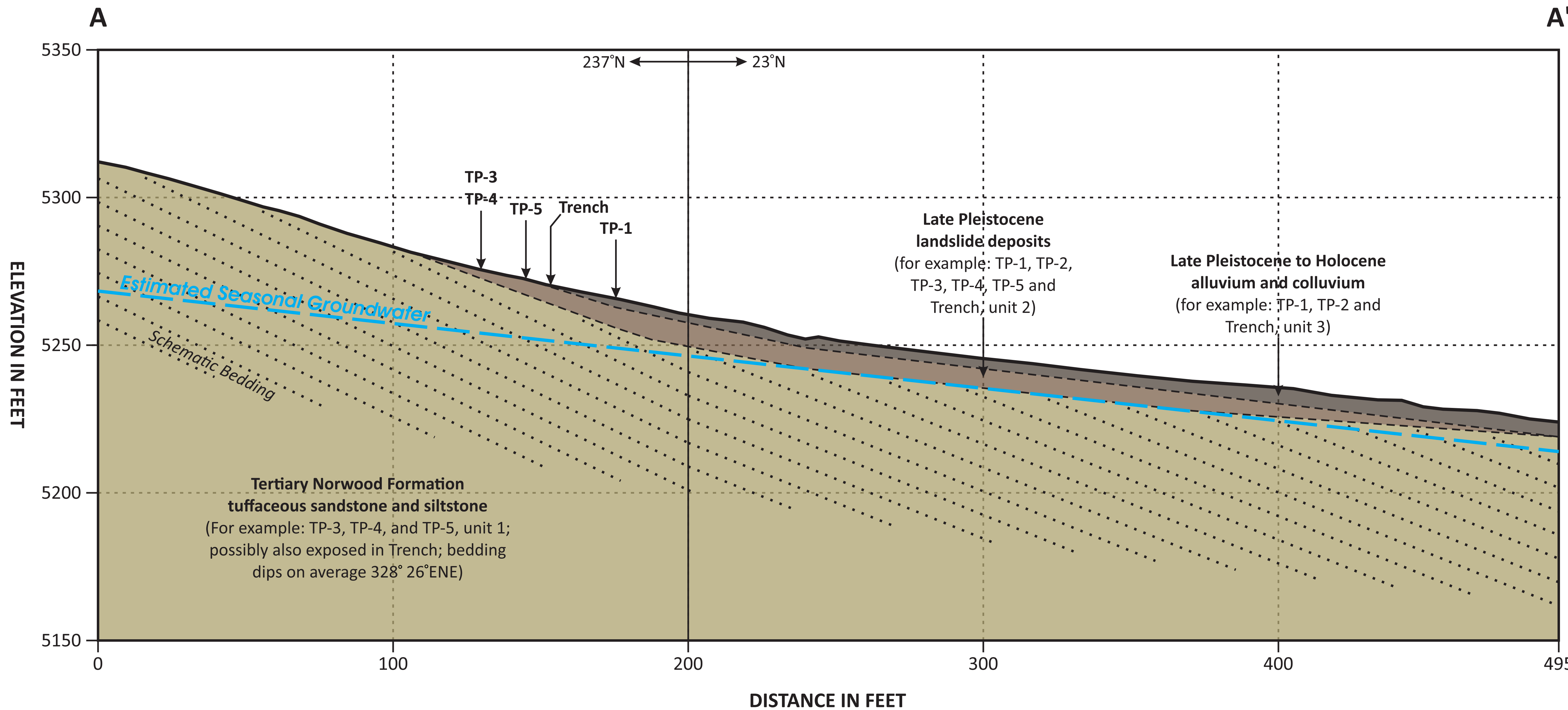
UNIT DESCRIPTIONS

Unit 1. *Tertiary Norwood Formation* - Light yellow brown (2.5Y 6/3), dense, well bedded, tuffaceous sandstone; weathered in upper part, refusal at test pit floor; Bw/Bt horzion formed in unit (**1B**).

Unit 2. *Late Pleistocene to Holocene landslide colluvium* - Dark brown (7.5YR 3/2), moderate-high density, massive, slightly root penetrated, sandy clay (CL) with gravel and trace cobbles of pale-brown sandstone; modern A-horizon soil formed in unit at surface (**2A**).

Scale 1 inch equals 5 feet (1:60) with no vertical exaggeration.

Proposed Paul Coles Cabin Tolliver Lane, Huntsville, Weber County, Utah	CMT ENGINEERING LABORATORIES		Figure 6E
	Test Pit 5 Log	Date: 20-Sep-2020 Job # 15297	

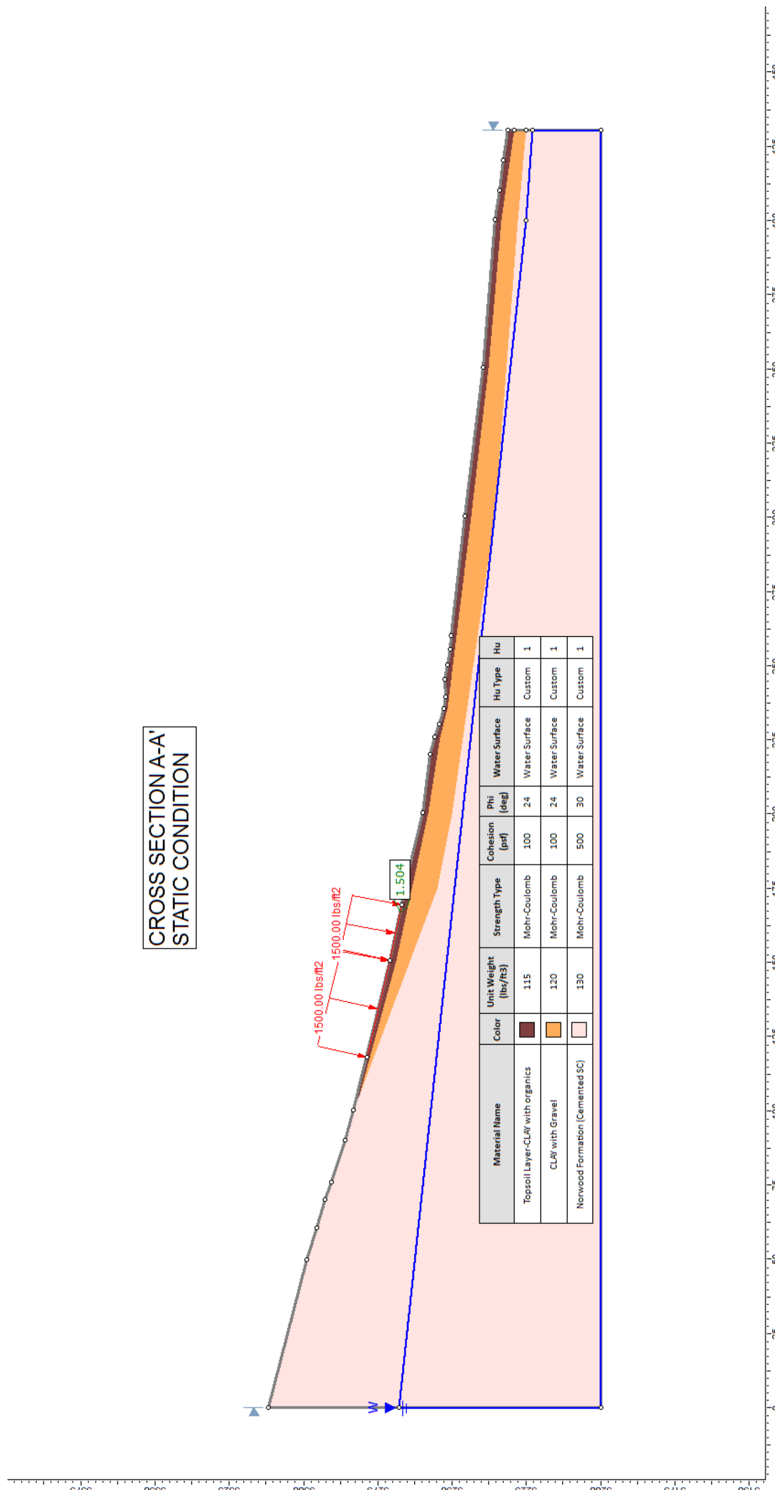


Scale 1 inch equals 40 feet (1:480) with no vertical exaggeration. Topography based on 2016 LIDAR data. Contacts and units are inferred and approximate.

CMT ENGINEERING LABORATORIES	Proposed Paul Coles Cabin Tolliver Lane, Huntsville, Weber County, Utah	Cross Section	Date:	20-Sep-2020	Figure 7
			CMT No.:	15297	

STABILITY RESULTS

Paul Coles Cabin
Cross Section A-A
STATIC

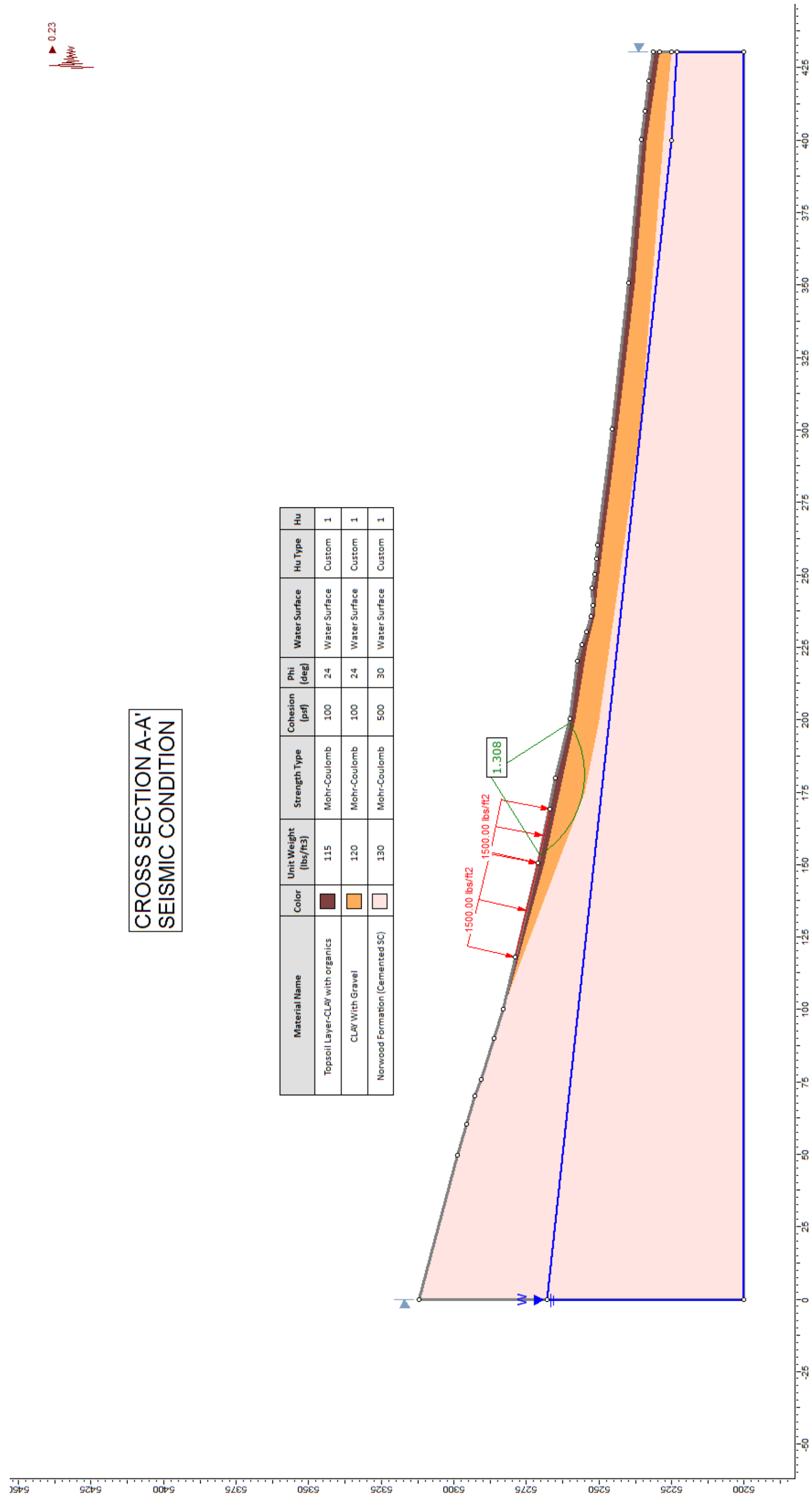


Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Hu Type	Hu
Topsoil Layer-CLAY with organics		115	Mohr-Coulomb	100	24	Water Surface	Custom	1
CLAY with Gravel		120	Mohr-Coulomb	100	24	Water Surface	Custom	1
Norwood Formation (Cemented SC)		130	Mohr-Coulomb	500	30	Water Surface	Custom	1

PROJECT NO.: 15297	Paul Coles Cabin	**FIGURE NO.:** 8

STABILITY RESULTS

Paul Coles Cabin
Cross Section A-A
SEISMIC



PROJECT NO.: 15297	Paul Coles Cabin	**FIGURE NO.:** 9