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GEOTECHNICAL ENGINEERING AND GEOLOGIC STUDY

Proposed Sundown Condominiums

Parcel/Tax ID: 22-001-0014
About 6550 North Wolf Creek Drive
Eden, Weber County, Utah
CMT PROJECT NO. 17355

FOR:
Mike Brenny
4421 North Thanksgiving Way
Lehi, Utah 84043

December 9, 2021

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Mr. Brenny:

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 September 30, October 28 and November 16, 2021 CMT Engineering Laboratories (CMT) personnel were on-site and supervised drilling of one bore hole to a depth of 11 feet below the existing ground surface (bgs), and excavation of five test pits to depths of 4.9 to 15.5 feet bgs. 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) 590-0394. To schedule materials testing, please call (801) 381-5141.

Sincerely,
CMT Engineering Laboratories

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APPENDIX

1.0 INTRODUCTION

1.1 General

CMT Engineering Laboratories (CMT) was authorized by Mr. Mike Brenny to conduct a design-level geotechnical engineering and geologic study for the proposed Sundown Condominiums development (Weber County Assessor Parcel #22-001-0014, 6.97 acres), located in the NE1/4 Section 26, Township 7 North, Range 1 East (Salt Lake Base Line and Meridian). Elevation of the site ranges from about 8,065 to 8,225 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 bore hole 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 among Mr. Mike Brenny of Mackay Developments, Mr. Guy Williams of Fawkes Consultants, and Mr. Andrew Harris of CMT Engineering Laboratories (CMT), and are outlined in our proposal dated August 18, 2021.

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, drilling one bore hole, and excavating, logging and sampling five 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 site is proposed for development of five conjoined sets of six duplex condominium structures (30 total). Structures are to be of wood-framed construction and founded on spread footings with basements (if conditions allow). Maximum continuous wall and column loads are anticipated to be 1 to 4 kips per lineal foot and 10 to 75 kips, respectively.

1.4 Executive Summary

Structures can be supported upon conventional spread and continuous wall foundations established on suitable natural soils or on limited structural fill extending to suitable natural soils provided the recommendations of this report are adhered to. 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 Precambrian- and Cambrian-age sedimentary bedrock deformed by thrust and normal faulting between about 90 to 125 million years ago. The

faulting includes the upper branch of the Willard thrust (which crosses the northern part of the property), and an east-dipping normal fault that crosses the eastern part of the property and truncates the thrust fault. None of these faults are considered active from a development perspective. Bedrock in the hanging wall (east side) of the normal fault is mapped as Cambrian Langston Formation and Ute Formation, whereas the bedrock in the footwall (west fault side) is mapped as Cambrian Geertsen Canyon Quartzite. The bedrock in the hanging wall (north side) of the upper branch of the Willard thrust is mapped as Precambrian Mutual Formation. All these bedrock units are overlain by Late Pleistocene- to Holocene-age colluvium of varying thickness.

2. Slopes at the site dip to the south at an overall roughly 3:1 (horizontal:vertical; or 33.3%, 18.3 degrees) and to the southwest at an overall roughly 3.4:1 (H:V; or 29.1%, 16.2 degrees).

3. Test pits TP-1, and TP-3 through TP-5 exposed 4.5 to 13 feet of topsoil and colluvium overlying weathered quartzite bedrock. Test pit TP-2 exposed fill overlying topsoil and colluvium to its explored depth (15.5 feet). The bore hole conducted at the site encountered refusal at a depth of 11 feet bgs and exposed fill overlying weathered quartzite bedrock. No groundwater was observed in the test pits or bore hole. Groundwater at the site is estimated to be at a depth of about 250 feet based on one nearby water well, but depths may vary seasonally and locally.

4. A global stability analysis was completed along cross sections A-A and B-B (See **Figures 6A &6B**) for both static and seismic conditions. The results meet the minimum design factors of safety.

5. Bedrock was encountered as shallow as about 5 feet at test pit TP-4 and 8 to 8.5 feet at borehole B-1 and TP-3 respectively. This shallow depth to bedrock must be considered with respect to sublevel excavations.

6. We recommend that structures placed on slopes incorporate foundation walls constructed as retaining walls and/or foundations be stepped with structurally supported floors, as needed, to reduce increased loading from fill placement on the existing slopes. Some structural fills may be placed below buildings but should be limited to about 4 feet or less and be properly graded as outlined later in this report. All structural fill for roadways and structures placed on slopes shall be benched a minimum of 2.5 feet following stripping of all surface vegetation and topsoil.

7. All vegetation, topsoil and non-engineered fills shall be removed below structures.

A geotechnical engineer/geologist from CMT must be allowed to verify that all topsoil and undocumented fill materials 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 and adjoining areas were explored by drilling one bore hole and excavating five test pits as located on **Figure 4**. The test pits were excavated using a track-mounted excavator to depths of 4.9 to 15.5 feet below the ground surface (bgs) 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.

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 test pits are illustrated on **Figures 5A-E, Test Pit Logs**. 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-E**). Seismic hazards information was developed from United States Geologic Survey (USGS) databases (Peterson and others, 2008).

As shown on **Figure 3**, slopes at the site dip to the south and southwest at gradients mainly steeper than 25% (shaded in red). Vegetative cover at the site consists mainly of mature trees, grasses, and brush. The site is currently undeveloped, but an existing condominium structure is on the south-adjoining parcel. The site surface has been disturbed in several locations and buried utilities reportedly cross the site.

3.2 Subsurface Soils

Five test pits were excavated at the site and adjoining parcel on the south to evaluate subsurface soil conditions, as located on **Figure 4**. Test pits TP-1 (**Figure 5A**), TP-3 (**Figure 5C**), TP-4 (**Figure 5D**) and TP-5 (**Figure 5E**) exposed topsoil and silty to clayey gravel colluvium with cobble and boulders overlying weathered bedrock of the Cambrian Geertsen Canyon Quartzite. Test pit TP-2 (**Figure 5B**) exposed fill overlying topsoil and a coarsening-upward sequence of sandy and gravelly colluvium. No groundwater was encountered in any of the excavations to their explored depths. Detailed stratigraphic unit descriptions are shown on the logs. Text pit excavation was complicated due to the presence of steep slopes, onsite man-disturbed areas and buried utilities.

In addition, one bore hole was drilled to a depth of about 11 feet bgs in the gravel parking area south of the existing condominium structure on the south-adjoining parcel. No other locations were deemed accessible by the driller. Samples of the subsurface soils encountered in the bore hole were collected at varying depths through the hollow stem drill augers. Relatively undisturbed samples of the subsurface soils were obtained by hydraulically pushing a 3-inch diameter (Shelby) tube and 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. 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 subsurface soils encountered in the bore hole were logged and described in general accordance with ASTM D-2488. Soil samples were collected as described above, and were classified in the field based upon visual and textural examination. These field classifications were supplemented by subsequent examination and testing of select samples in our laboratory. A log of the bore hole, including a description of the soil strata encountered, is provided on **Figure 7**. Sampling information and other pertinent data and observations are also included on the log. A Key to Symbols defining the terms and symbols used on the log is provided as **Figure 8**.

3.3 Geologic Cross Sections

Figure 6A, Cross Section A-A' and **Figure 6B, Cross Section B-B'** show two geologic cross sections across the site at a scale of 1 inch equals 40 feet with no vertical exaggeration. Locations of the cross sections are shown on **Figures 3 and 4**. Units and contacts are based on subsurface data from the test pits (**Figures 5A-E**), and/or inferred from the site-specific surficial geologic mapping on **Figures 3 and 4**. The topographic profile is based on geoprocessed LIDAR data from 2016. The LIDAR data provide a snapshot of topographic conditions at the time of acquisition; past, present and future surficial topography may vary. Units and contacts should be considered approximate and inferred, and variations should be expected at depth and laterally.

3.4 Groundwater

No groundwater was encountered in the bore hole or test pit excavations conducted at the site, and no site-specific groundwater information is available. However, the Utah Division of Water Rights Well Driller Database shows one water well about 2,100 feet south of the site that has a reported static groundwater depth of 254 feet bgs. Based on the above, we infer groundwater at the site is about 250 feet deep. 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.

3.5 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 site is located slightly south of the divide between Ogden and Cache Valleys, which are to the south and north, respectively. Cache Valley is a major sediment-filled, north-south-trending intermontane valley flanked by the Bear River Range to the east and the Wellsville Mountains to the west. Ogden Valley is a roughly 40-square mile back valley within the Wasatch Range described by Gilbert (1928) as a structural trough similar to Cache and Morgan Valleys to the north and south, respectively. Both valleys are in a transition zone between the Basin and Range and Middle Rocky Mountains 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 the prominent, west-facing escarpment along 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). Yonkee and others (2019) indicate the Precambrian- and Cambrian-age sedimentary bedrock underlying the site was previously deformed by thrust and normal faulting between about 90 to 125 million years ago. The faulting includes the upper branch of the Willard thrust (which crosses the northern part of the property), and an east-dipping normal fault that crosses the eastern part of the property and truncates the upper branch of the Willard thrust.

The site is also in the central portion of the Intermountain Seismic Belt (ISB), a generally north-south trending zone of historical seismicity along the eastern margin of the Basin and Range province extending 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; the largest of these earthquakes was a M 7.5 event in 1959 near Hebgen Lake, Montana. None of these earthquakes occurred along the Wasatch fault or other known late Quaternary faults (Arabasz and others, 1992; Smith and Arabasz, 1991). The closest event was the 1934 Hansel Valley (M 6.6) event north of the Great Salt Lake. The March 18, 2020 M 5.7 earthquake[1] near Magna, Utah reportedly showed a style, location, and slip depth consistent with an earthquake on the Wasatch fault system. Despite being less than magnitude 6.0, this earthquake damaged multiple buildings and was felt from southern Idaho to south-central Utah[2]. The University of Utah Seismograph Stations[3] indicates the Magna earthquake was weakly felt in Ogden Valley, with a peak acceleration of about 0.005 g and an instrument intensity of II-III (on a Roman numeral scale of I-X).

4.2 Surficial Geology

The site is located in steep mountainous terrain in the Wasatch Range about 3.5 miles northeast of Ogden Valley near the divide between the Wellsville and Wolf Creek drainage basins. This divide marks the boundary between Weber and Cache Counties (to the south and north, respectively). The Wasatch Range is a major north-south trending mountain range that marks the eastern boundary of the Basin and Range physiographic province (Stokes; 1977, 1986); Ogden Valley is a sediment-filled intermontane valley within the Wasatch Range. Surficial geology of the site is mapped by Coogan and King (2016; **Figure 2**) as Neoproterozoic (Precambrian-age) bedrock of the Mutual Formation (unit Zm), and Cambrian-age bedrock of the Geersten Canyon Quartzite (unit Cgc), Langston Formation (unit Cl) and Ute Formation (unit Cu). Detailed surficial geologic mapping at a scale of 1 inch equals 100 feet (1:1,200) is shown on **Figures 3 and 4** based on Coogan and King (2016), air photo evidence, and site-specific subsurface evidence.

Coogan and King (2016) describe surficial geologic units in the site area on **Figure 2** as follows:

> ***Qh, Qh?*** – *Human disturbances (Historical)*. Mapped disturbances obscure original deposits or rocks by cover or removal; only larger disturbances that pre-date the 1984 aerial photographs used to map the Ogden 30 x 60-minute quadrangle are shown; includes engineered fill, particularly along Interstate Highways 80 and 84, the Union Pacific Railroad, and larger dams, as well as aggregate operations, gravel pits, sewage-treatment facilities, cement plant quarries and operations, brick plant and clay pit, Defense Depot Ogden (Browning U.S. Army Reserve Center), gas and oil field operations (for example drill pads) including gas plants, and low dams along several creeks, including a breached dam on Yellow Creek.
>
> ***Qct*** - *Colluvium and talus, undivided (Holocene and Pleistocene)*. Unsorted clay- to boulder-sized angular debris (scree) at the base of and on steep, typically partly vegetated slopes; shown mostly on steep slopes of resistant bedrock units; 6 to 30 feet (2-9 m) thick.
>
> ***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

[1] https://earthquake.usgs.gov/earthquakes/eventpage/uu60363602/executive
[2] https://www.ksl.com/article/46731630/
[3] https://earthquakes.utah.gov/magna-quake/#

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).

Wasatch Formation (Eocene and upper Paleocene) – Typically red to brownish-red sandstone, siltstone, mudstone, and conglomerate with minor gray limestone and marlstone locally (see Twl); lighter shades of red, yellow, tan, and light gray present locally and more common in uppermost part, complicating mapping of contacts with overlying similarly colored Norwood and Fowkes Formations; clasts typically rounded Neoproterozoic and Paleozoic sedimentary rocks, mainly Neoproterozoic and Cambrian quartzite; basal conglomerate more gray and less likely to be red, and containing more locally derived angular clasts of limestone, dolomite and sandstone, typically from Paleozoic strata, for example in northern Causey Dam quadrangle; sinkholes indicate karstification of limestone beds; thicknesses on Willard thrust sheet likely up to about 400 to 600 feet (120-180 m) in Sharp Mountain, Dairy Ridge, and Horse Ridge quadrangles (Coogan, 2006a-b), about 1300 feet (400 m) in Monte Cristo Peak quadrangle, about 1100 feet (335 m) in northeast Browns Hole quadrangle, about 2200 feet (670 m) in southwest Causey Dam quadrangle, about 2600 feet (800 m) at Herd Mountain in Bybee Knoll quadrangle, and about 1300 feet (400 m) in northwest Lost Creek Dam quadrangle, estimated by elevation differences between pre-Wasatch rocks exposed in drainages and the crests of gently dipping Wasatch Formation on adjacent ridges (King); thickness varies locally due to considerable relief on basal erosional surface, for example along Right Fork South Fork Ogden River, and along leading edge of Willard thrust; much thicker, about 5000 to 6000 feet (1500-1800 m), south of Willard thrust sheet near Morgan. Wasatch Formation is queried (Tw?) where poor exposures may actually be surficial deposits. The Wasatch Formation is prone to slope failures. Other information on the Wasatch Formation is in Tw descriptions under the heading "Sub-Willard Thrust - Ogden Canyon Area" since Tw strata are extensive near Morgan Valley and cover the Willard thrust, Ogden Canyon, and Durst Mountain areas.

Along the South Fork Ogden River, Wasatch strata are mostly pebble, cobble, and boulder conglomerate with a matrix of smaller gravel, sand, and silt in the Browns Hole quadrangle, and coarse-grained sandstone to granule conglomerate as well as siltstone and mudstone to the east in the Causey Dam quadrangle; note thinning to east away from source area. The Wasatch weathers to boulder-covered dip(?) slopes north of the South Fork Ogden River, for example in Evergreen Park. Along the South Fork, the Wasatch Formation is separated from the underlying Hams Fork Member of the Evanston Formation by an angular unconformity of a few degrees, with the Hams Fork containing less siltstone and mudstone than the Wasatch and having a lighter color.

The Herd Mountain surface is developed on the Wasatch Formation at elevations of 7600 to 8600 feet (2300-2620 m) in the Bybee Knoll quadrangle and in remnants in the Huntsville, Browns Hole, and Sharp Mountain quadrangles. The origin of this boulder-strewn surface is debated (see Eardley, 1944; Hafen, 1961; Mullens, 1971). Eardley's (1944) Herd Mountain surface is flat lying or gently east dipping, about the same as the underlying Wasatch Formation, and is strewn with quartzite boulders to pebbles that King thinks are residual and colluvial deposits of uncertain age that were derived from the Wasatch Formation. The other characteristic of this surface is the presence of pimple mounds and, given the elevations of greater than about 7500 feet (2300 m), possible periglacial patterned ground. Photogrammetric dips on the Wasatch Formation under the surface are nearly flat (<3°) and an apparent angular unconformity is present in the Wasatch since dips on older Wasatch strata are greater than 3 degrees. King mapped this unconformity as a marker bed, but Coogan does not agree that this is an unconformity.

Cbk, Cbk? - *Blacksmith Formation (Middle Cambrian)*. Typically, medium-gray, very thick to thick-bedded, dolomite and dolomitic limestone with tan-weathering, irregular silty partings to layers; weathers to lighter gray cliffs and ridges; 250 to 760 feet (75-230 m) thick in our map area. The Blacksmith Formation on the leading edge of the Willard thrust sheet thickens southward from 600 feet (180 m) along Sugar Pine Creek in the Dairy Ridge quadrangle, to about 760 feet (230 m) in the northwestern Horse Ridge quadrangle (Coogan, 2006a-b). To the south and west, the Blacksmith is about 500 feet (150 m) thick near Causey Dam (Mullens, 1969), with a 530-foot (161 m) thickness reported at the Baldy Ridge section (Rigo, 1968, aided by Mullens) in the Causey Dam or Horse Ridge quadrangle. Farther west, the Blacksmith is reportedly 409 feet (125 m) thick in the Sharp Mountain area (Hafen, 1961) and is about 250 feet (75 m) thick near the South Fork Wolf Creek in the Huntsville quadrangle (Coogan this report); still farther west, this unit is reportedly about 700 to 800 feet (210-245 m) thick near Mantua (Williams, 1948; Ezell, 1953; Sorensen and Crittenden, 1976a). So the thickness of the Blacksmith Formation is low in the Huntsville quadrangle and thickens to north, west, and east, and thickens southward on leading edge of thrust sheet.

The Blacksmith to the north of our map area is about 475 feet (144 m) thick in the Porcupine Reservoir quadrangle (Rigo, 1968; Hay, 1982), about 450 feet (137 m) thick near the Blacksmith Fork River (Maxey, 1958), and 410 feet (125 m) thick in Blacksmith Fork Canyon (Hay, 1982). The Blacksmith thickness in the Browns Hole area is uncertain due to poorly exposed Cambrian strata. Laraway's (1958) Blacksmith contacts are not those of Crittenden (1972) or our mapping (see also Hodges member above); so his reported 730-foot (220 m) thickness is suspect. Laraway's (1958) report of Bolaspidella and Ehmaniella trilobite fossils in his Blacksmith is also problematic because these fossils are characteristic of the Bloomington and Ute Formations, respectively (Maxey, 1958). Also, Laraway's description of covered intervals in typically cliff-forming Blacksmith imply a fault repetition of the Ute or his measuring at least 986 feet (300 m) of Ute (see Ute description for comparison) and less than 403 feet (123 m) of Blacksmith; further, Crittenden's (1972) large thicknesses (~1300 or less likely 1150 feet [~400 or <350 m]) and mixed carbonates above Ute shale on his lithologic column imply fault repetition(s). Our Blacksmith-Bloomington contact is above a non-resistant Ute interval that overlies a resistant cliffy interval in the Ute. This makes the Ute about 700 feet (215 m) thick on Crittenden's (1972) lithologic column, and the Blacksmith and lower Bloomington about 650 feet (200 m) thick on his column. Finally, Crittenden's (1972) lithologies are not like what Laraway (1958) reported in his measured section.

Cu, Cu? - *Ute Formation (Middle Cambrian)*. Interbedded gray thin- to thick-bedded limestone with tan-, yellowish-tan-, and reddish-tan-weathering, wavy, silty layers and partings, and olive-gray to tan-gray, thin-bedded shale and micaceous argillite; and minor, medium-bedded, gray to light-gray dolomite; sand content in limestone increases upward such that calcareous sandstone is present near top of formation; mostly slope and thin ledge former; base less resistant (more argillaceous) than underlying Langston Formation; Zacanthoides, Kootenia, Bathyuriscus, and Peronopsis sp. trilobite fossils reported by Rigo (1968, USGS No. 5960-CO) in Causey Dam quadrangle; estimate 450 to 1000 feet (140-300 m) thick and thinnest on leading edge of Willard thrust sheet.

The thickness range for the Ute Formation is based on multiple studies. It is reportedly 600 to 700 feet (180-210 m) thick west of Sharp Mountain (see Ezell, 1953; Crittenden, 1972; Deputy, 1984), and though a 840-foot (256 m) thickness was reported north of our map area in the Porcupine Reservoir area (Rigo, 1968), the Ute only looks about 600 feet (180 m) thick on the Porcupine Reservoir map of Berry (1989). The Ute is reportedly 1090 and 1380 feet (330 and 420 m) thick in the Sharp Mountain area (Hafen, 1961; Rigo, 1968, respectively), but these thicknesses are suspect since the Ute is thinner to the north, east, and west. We suspect that Hafen (1961) used dips that were too steep (~30 degrees vs ~16.5 degrees) so the real Ute thickness is about 620 feet (190 m) where he measured his section; we do not know what Rigo (1968) measured. North of our map area in the Hardware Ranch quadrangle, Deputy (1984) measured 681 feet (207.6 m) of Ute. To the east, the Ute is about 450 feet (137 m) thick in the Horse Ridge and Dairy Ridge quadrangles (Coogan, 2006a-b) and 515 feet (157 m) thick at the Baldy Ridge section (Rigo, 1968) in the Horse Ridge quadrangle. The thickest Ute may be near the South Fork Wolf Creek in the Huntsville quadrangle, where Coogan estimates a 1000-foot (300 m) thickness, 1150 feet (350 m) thick if steeper dip, while King estimates the Ute is about 1100 feet (335 m) thick, based on a higher Ute-Langston contact than Coogan picked. Rigo (1968) reported 1370 feet (418 m) of Ute near the South Fork Wolf Creek, but his contacts are not used on our map. To the south in the Browns Hole quadrangle, about 700 feet (210 m) of mixed shale and limestone was shown by Crittenden (1972) and his depiction is likely derived from the 659 feet (201 m) of Ute reported by Laraway (1958) along the South Fork Ogden River; this is about what Laraway (1958) mapped. But Crittenden (1972) did not map the Ute-Blacksmith contact; further, see problems above under Blacksmith Formation.

The Ute Formation as first mapped in the James Peak, Mantua, and Huntsville quadrangles was too thick because Coogan mapped the lower shale in the Langston Formation as the entire Langston, not realizing the base of the Ute is a shale above the upper carbonate (typically dolomite) of the Langston. He did this because the upper carbonate is not distinct in these quadrangles, like it is to the west in the Mount Pisgah quadrangle and to the east in the Sharp Mountain quadrangle. The same problem exists locally in the Sharp Mountain quadrangle. Though King revised the present map to place the upper Langston carbonate in the Langston, problems with this contact and Ute and Langston Formation thicknesses may persist.

Just north of our map area in the Wellsville Mountains, Maxey (1958) reported Ehmaniella(?) sp. and Glossopleura sp. trilobites in and at the base of the Ute Formation, respectively, making it Middle Cambrian. Deiss (1938) and Berry (1989) reported Ehmaniella sp. trilobites north of our map area near the Blacksmith Fork River.

Cl, Cl? - *Langston Formation (Middle Cambrian)*. Upper part is gray, sandy dolomite and limestone that weathers to ledges and cliffs; middle part is yellowish- to reddish-brown to gray weathering, greenish-gray, fossiliferous shale and lesser interbedded gray, laminated to very thin-bedded, silty limestone (Spence Shale Member); basal part is light-brown-weathering, ledge forming gray limestone and dolomite with local poorly indurated tan, dolomitic sandstone at bottom; basal part that is less resistant (Naomi Peak Member) is present at least in northwest part of our map area; conformably overlies Geertsen Canyon Quartzite; 200 to 400 feet (60-120 m) thick. Designated “Formation” rather than “Dolomite” due to the varied lithologies.

The thickness of the Langston Formation is based on several studies. North of the map area, 410 feet (125 m) of Langston was measured along the upper Blacksmith Fork River in the Hardware Ranch quadrangle by Buterbaugh (1982). The Langston is 270 feet (80 m) thick in the Sharp Mountain area (Hafen, 1961) and to the east it is about 200 to 250 feet (60 to 75 m) thick in the Horse and Dairy Ridge quadrangles (Coogan, 2006a-b); the 85-foot (26 m) thickness reported at the Baldy Ridge section (Rigo, 1968) in the Horse Ridge quadrangle is likely incorrect. The 170 feet (50 m) of dolomite reported near Browns Hole (Crittenden, 1972) is likely only the basal dolomite of the Langston Formation; Laraway (1958) probably measured 120 feet (37 m) of this basal dolomite and 298 feet (91 m) of Langston along the South Fork Ogden River in the Browns Hole quadrangle. Laraway’s (1958) reported 398-foot (121 m) Langston thickness is likely an error, since he measured and mapped about 300 feet (90 m) of Langston. Near the South Fork Wolf Creek in the Huntsville quadrangle, the Langston is about 300 feet (90 m) thick (Coogan’s measurements), but King used a higher contact on our map making the Langston about 390 feet (120 m) thick. Farther west the Langston is about 400 to 460 feet (120-140 m) thick (see Ezell, 1953; Maxey, 1958; Rigo, 1968; Buterbaugh, 1982).

Just north of the map area near the Blacksmith Fork River, the Langston trilobite fauna (Glossopleura zone) is Middle Cambrian in age (Maxey, 1958), and near Brigham City, the fauna (Glossopleura trilobite zone in Spence Shale, Albertella trilobite zone in Naomi Peak) is earliest Middle Cambrian in age (Maxey, 1958; Jensen and King, 1996, table 2).

Cgc, Cgc? *- Geertsen Canyon Quartzite (Middle and Lower Cambrian and possibly Neoproterozoic).* In the west mostly buff (off-white and tan) quartzite, with pebble conglomerate beds; pebbles are mostly rounded light colored quartzite; contains cross bedding, and pebble layers and lenses; colors vary from tan and light to medium gray, with pinkish, orangish, reddish, and purplish hues; outcrops darker than these fresh quartzite colors; cliff forming; some brown-weathering, interbedded micaceous argillite and quartzite common at top and mappable locally; pebble to cobble conglomerate lenses more abundant in middle part of quartzite, and basal, very coarse-grained arkose locally; near Huntsville, total thickness about 4200 feet (1280 m), including upper argillite about 375 feet (114 m) thick and basal coarse-grained arkose (arkosic to feldspathic quartzite) about 300 to 400 feet (90-120 m) thick (Crittenden and others, 1971). Overall seems to be thinner near Browns Hole. Called Prospect Mountain Quartzite and Pioche Shale (argillite at top) by some previous workers.

Upper and lower parts of Crittenden and others (1971; Crittenden, 1972; Sorensen and Crittenden, 1979) are not mappable outside the Browns Hole and Huntsville quadrangles, likely because the marker cobble conglomerate and change in grain size and feldspar content reported by Crittenden and others (1971) is not at a consistent horizon; quartz-pebble conglomerate beds are present in most of the Geertsen Canyon Quartzite.

To the east on leading margin of Willard thrust sheet, the Geertsen Canyon is thinner, an estimated 3200 feet (975 m) total thickness (Coogan, 2006a-b), and may be divided into different members, though informal members to west and east are based on conglomerate lenses near member contact and feldspathic lower member (see Crittenden and others, 1971; Coogan, 2006a-b).

Lower part in west (Cgcl, Cgcl?) is typically conglomeratic and feldspathic quartzite (only up to 20% feldspar reported by Crittenden and Sorensen, 1985a, so not an arkosic), with 300- to 400-foot (90-120 m), basal, very coarse-grained, more feldspathic or arkosic quartzite; 1175 to 1700 feet (360-520 m) thick (Crittenden and others, 1971; Crittenden, 1972; Sorensen and Crittenden, 1979) and at least 200 to 400 feet (60-120 m) thinner near Browns Hole (compare Crittenden, 1972 to Sorensen and Crittenden, 1979). Unit queried where poor exposures may actually be surficial deposits.

Zm, Zm? *- Mutual Formation (Neoproterozoic).* Grayish-red to purplish-gray, medium to thick-bedded quartzite with pebble conglomerate lenses; also reddish-gray, pink, tan, and light-gray in color and typically weathering to darker shades than, but at least locally indistinguishable from, Geertsen Canyon Quartzite; commonly cross-bedded and locally feldspathic; contains argillite beds and, in the James Peak quadrangle, a locally mappable medial argillite unit; 435 to 1200 feet (130-370 m) thick in Browns Hole quadrangle (Crittenden, 1972) and thinnest near South Fork Ogden River (W. Adolph Yonkee, Weber State University, verbal communication, 2006); thicker to northwest, up to 2600 feet (800 m) thick in Huntsville quadrangle
(Crittenden and others, 1971) and 2556 feet (780 m) thick in James Peak quadrangle (Blau, 1975); may be as little as 300 feet (90 m) thick south of the South Fork Ogden River (King this report); absent or thin on leading edge of Willard thrust sheet (see unit Zm?c); thins to south and east.

Zi, Zi? *- Inkom Formation (Neoproterozoic).* Overall gray to reddish-gray weathering, poorly resistant, psammite and argillite, with gray-weathering meta-tuff lenses in lower part; upper half dominantly dark green, very fine-grained meta-sandstone (psammite) with lower half olive gray to lighter green-gray, greenish gray-weathering, laminated, micaceous meta-siltstone (argillite); lower greenish-weathering part missing near South Fork Ogden River and the Inkom is less than 200 feet (60 m) thick; in Mantua quadrangle, Inkom typically 300 feet (90 m) thick, and is only less than 200 feet (60 m) thick where faulted (King this report); 360 to 450 feet (110-140 m) thick northeast of Huntsville (Crittenden and others, 1971), and absent on leading edge of Willard thrust sheet (Coogan, 2006a); location of “pinch-out” not exposed.

Zcc, Zcc? - *Caddy Canyon Quartzite (Neoproterozoic).* Mostly vitreous, almost white, cliff-forming quartzite; colors vary and are tan, light-gray, pinkish-gray, greenish-gray, and purplish-gray, that are typically lighter shades than the Geertsen Canyon Quartzite; 1000 to 2500 feet (305-760 m) thick in west part of our map area, thickest near Geertsen Canyon in Huntsville quadrangle (Crittenden and others, 1971; Crittenden, 1972); 1500 feet (460 m) thick near South Fork Ogden River (Coogan and King, 2006); thinner, 725 to 1300 feet (220-400 m) thick, and less vitreous on leading edge of Willard thrust sheet. Lower contact with Kelley Canyon Formation is gradational with brownish-gray quartzite and argillite beds over a few tens to more than 200 feet (3-60 m) (see Crittenden and others, 1971). Where thick, this gradational-transitional zone is what is mapped as the Papoose Creek Formation. Near Geertsen Canyon, this transition zone is 600 feet (180 m) thick and was mapped with and included in the Caddy Canyon Quartzite by Crittenden and others (1971, figure 7), and in the Caddy Canyon and Kelley Canyon Formations by Crittenden (1972, see lithologic column).

Zkc, Zkc? - *Kelley Canyon Formation (Neoproterozoic).* Dark-gray to black, gray to olive-gray-weathering argillite to phyllite, with rare metacarbonate (for example basal meta-dolomite); grades into overlying Caddy Canyon quartzite with increasing quartzite; gradational interval mapped as Papoose Creek Formation (Zpc); 1000 feet (300 m) thick in Mantua quadrangle (this report), where Papoose Creek Formation is mapped separately, and reportedly 2000 feet (600 m) thick near Huntsville (Crittenden and others, 1971, figure 7), but only shown as about 1600 feet (500 m) thick to Papoose Creek transition zone by Crittenden (1972). The Kelley Canyon Formation is prone to slope failures.

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

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) 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[4] 7-16. Given the subsurface soils exposed in the test pits at the site and the anticipated shallow depth to bedrock, it is our opinion the site best fits Site Class C – Very Dense Soil and Soft Rock, 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 R301.2.2.1.1) are based upon the Site Class discussed in the previous section.

[4] American Society of Civil Engineers

For Site Class C at site grid coordinates of 41.3785 degrees north latitude and 111.7839 degrees west longitude, $\mathbf{S_{DS}}$ is 0.684 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 8.5 miles to the southwest. 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 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.

No landslides are mapped at the site and no evidence for recent or ongoing landsliding or slope instability was observed during our reconnaissance. However, slopes at the site are steep and may be subject to shallow surficial failures involving colluvial veneers overlying bedrock. Slope stability is discussed 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% (shaded in red). Slopes at the site dip to the south at an overall roughly 3:1 (horizontal:vertical; or 33.3%, 18.3 degrees) and to the southwest at an overall roughly 3.4:1 (H:V; or 29.1%, 16.2 degrees).

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 property is not in a mapped alluvial fan; no evidence of debris-flow channels, levees, or other debris-flow features were observed at the site; and no inferred debris-flow deposits were exposed in the test pits. Given this, debris flows and floods are not considered to pose a risk to the site.

4.6.3 Stream Flooding Hazards

No perennial or intermittent drainage courses are mapped or were observed crossing the site, but an ephemeral drainage is in the eastern part of the site that may be seasonally active following spring snowmelt. Federal Emergency Management Agency flood insurance rate mapping (Map Number 49057C0050E, effective December 2005, unprinted) classifies the Project in "Zone D - Areas of Undetermined Flood Hazard". Surface drainage and hydrology should be addressed in the civil engineering design for the development.

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 and bedrock encountered in the test pits and bore hole were estimated using laboratory testing, published correlations[5], 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)
Mass Wasting Colluvium	38.2	100	120
Cambrian Quartzite W Pebble Conglomerate (Bedrock)	25	5000	155

[5] Geoengineer.org/education/laboratory -testing… and finessoftware.eu/geo5/en/unit-weight-of rocks-01.

Limestone (bedrock)	25	4000	155
Weathered Rock, Shale, Argillite	25	3000	155
Colluvium FILL	35	75	120

The stability analyses provided are based on **Figure 6A, Cross Section A-A'** and **Figure 6B, Cross Section B-B'** and represent the existing slope conditions and do not include final grading. CMT must review final grading plans.

The pseudostatic coefficient for the seismic analyses was obtained by taking half of the modified peak ground acceleration adjusted for site class C (0.448g) queried for the site which resulted in a value of 0.224g.

5.2 Stability Analyses

We evaluated the global stability of the cross-sections A-A' and B-B' located as shown on **Figure 4 Site Evaluation**. The analysis was completed using the computer program *SLIDE2*. 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.

A projected water (phreatic) surface was not incorporated in the model based on nearby water well data placing the groundwater at an elevation of about 4950 feet which is deeper than the cross section analyzed.

- Cross-section A-A' (**Figure 6**) consists of a 577-foot long horizontal cross section with an overall elevation change of about 192 feet with an overall slope gradient of about 3.0:1 (H:V) downward to the south. Based on the slope stability analysis, the current slope generally has factors of safety for both static and pseudo-static (earthquake) conditions in excess of those typically considered acceptable (See **Figures 9A and 9B Stability Results**). The failure surfaces with the lowest factors of safety are shown on the stability analysis plot, with the lowest calculated factor of safety displayed (1.674-Static: 1.335-Seismic).
- Cross-section B-B' (**Figure 6**) consists of a 577-foot long horizontal cross section with an overall elevation change of about 169 feet with an overall slope gradient of about 3.4:1 (H:V) with isolated areas as steep as about 1.6:1 (H:V) downward to the southwest. Based on the slope stability analysis, the current slope generally has factors of safety for both static and pseudo-static (earthquake) conditions in excess of those typically considered acceptable (See **Figures 10A and 10B Stability Results**). The failure surfaces with the lowest factors of safety are shown on the stability analysis plot, with the lowest calculated factor of safety displayed (1.789-Static: 1.171-Seismic).

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. Following grading at the site, we recommend the slope surface must be re-vegetated as soon as possible to limit erosion.

For roadway construction we recommend that cuts and fills not be steepened more that 2:1 (H:V). Fills placed on slopes for structures should not be steepened more than 2.5:1 (H:V) and extend beyond the footing such that an imaginary line drawn from the footing edge to the slope surface at maximum steepness of 1.5:1V (H:V) does not daylight at the slope surface. Further, all structural fill for roadways and structures placed on slopes shall be benched a minimum of 2.5 feet following stripping of all surface vegetation and topsoil.

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. Direct Shear Test, ASTM D-3080, Shear strength parameters

6.2 Lab Summary

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

LAB SUMMARY TABLE

BH/TP No	Depth (feet)	Soil Class	Sample Type	Moisture Content (%)	Dry Denstiy (pcf)	Gradation			Atterberg Limits		
						Grav	Sand	Fines	LL	PL	PI
B-1	2.5	FILL-SC	SPT	4.4				26.1	42	30	12
	5	SM	SPT	5.9		31	23	46			
TP-1	4	GM-GC	Bag	6.5		61	16	23.2			
TP-2	9	GP	Bag	2.8		65	31	3.7			NP
TP-3	1.5	SM-SC	Bag	21		28	34	38			
TP-4	3	GM-GC	Bag	9.5		50	39	21			
TP-5	5	GM-GC	Bag	3.3		69	16	14.9			

6.3 Direct Shear Test

To determine the shear strength of the surficial colluvium at the site, a laboratory direct shear test was performed on a representative sample recovered.

During the direct shear test, the samples were evenly consolidated within the test ring, loaded, and saturated immediately after the load was applied. Loading was conducted at a slower rate to simulate saturated-drained condition. The results of the direct shear tests are presented in the following table below:

Direct Shear Results

Sample Location	Sample Depth (feet)	Unified Soils Classification	Apparent Cohesion (psf)	Measured Internal Friction Angle (degrees)
TP-3	4.0	GC-GM	100	38.2

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 topsoil or deleterious materials shall extend out at least 4 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 up to about 2 feet in thickness.

Similarly, any non-engineered fill must be removed below structures down to suitable natural soil.

The site should be examined by a CMT geotechnical engineer/geologist to assess that site stripping, site grading cuts, meet the requirement of this report, prior to placing site grading/structural fills, footings, slabs, and flatwork/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 and minimum horizontal depth 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.

Bedrock was encountered as shallow as about 5 feet at test pit TP-4 and 8.0 to 8.5 feet at borehole B-1 and TP-3, respectively. This shallow depth to bedrock must be considered with respect to sublevel excavations. Heavy equipment, chipping and possible blasting may be required to remove bedrock.

The natural surficial colluvium soils encountered at this site predominantly consisted of silty/clayey sand and gravel. 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.

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).

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
Select 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, and less than 30% passing the No. 200 sieve, and a maximum Plasticity Index of 12.
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**).

On site granular soils may be utilized as structural fill/site grading fill if processed to meet the requirements given above, and may also be used in non-structural fill situations.

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[6] 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 2 feet beyond below flatwork and pavement (applies to structural fill and site grading fill)	0 to 5 5 to 10	95 97
Site grading fill outside area defined above	0 to 5 5 to 10	92 95
Utility trenches within structural areas	--	96

[6] American Association of State Highway and Transportation Officials

Location	Total Fill Thickness (feet)	Minimum Percentage of Maximum Dry Density
Roadbase and subbase	-	96
Non-structural fill	0 to 5 5 to 10	90 92

Structural fills greater than 10 feet thick are not anticipated at the site. Onsite soils may be utilized as structural site grading fill if free of deleterious materials and if the appropriate compaction can be achieved. 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. Please note that fine grained soils (silts/clays) are inherently difficult to adequately moisture prepare and compact to the requirements for structural fill and near impossible to do so during wet and cold periods of the year.

7.5 Utility Trenches

For the bedding zone around the utility, we recommend utilizing sand bedding fill material that meets current APWA[7] 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.

[7] 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 suitable natural granular soils or structural fill extending to suitable natural soils. Footings may then be designed using a net bearing pressure of 3,000 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 1/2 for temporary loads such as wind and seismic forces.

We recommend that structures placed on slopes incorporate foundation walls constructed as retaining walls and/or foundations be stepped with structurally supported floors, as needed, to reduce increased loading from fill placement on the existing slopes. Some structural fills may be placed below buildings but we recommend they be limited to about 4 feet and be properly graded as outlined in this report.

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.
5. All footings on slopes shall be installed at depth such that an imaginary line drawn from the footing edge to the slope surface at maximum steepness of 1.5H:1V (Horizontal:Vertical) does not daylight at the slope surface.

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 12 feet tall might be constructed at this site. Parameters, as presented within this section, are for backfills which will consist of drained onsite granular soil placed and compacted in accordance with the recommendations presented herein.

The lateral pressures imposed upon subgrade facilities will depend upon the relative rigidity and movement of the backfilled structure. Following are the recommended lateral pressure values, which also assume that the soil surface behind the wall is horizontal and that the backfill within 3 feet of the wall will be compacted with hand-operated compacting equipment.

CONDITION	STATIC (psf/ft)*	SEISMIC (psf)*
Active Pressure (wall is allowed to yield, i.e. move away from the soil, with a minimum 0.001H movement/rotation at the top of the wall, where "H" is the total height of the wall)	33	24
At-Rest Pressure (wall is not allowed to yield)	51	N/A
Passive Pressure (wall moves into the soil)	425	60

*Equivalent Fluid Pressure (applied at 1/3 Height of Wall)
*Equivalent Fluid Pressure (added to static and applied at 1/3 Height of Wall)

10.0 FLOOR SLABS

Floor slabs may be established upon suitable, undisturbed, natural soils or structural fill extending to suitable natural soils. Under no circumstances shall floor slabs be established directly on any topsoil, 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 to develop seasonally upon the bedrock as a result of seasonal snow melt, precipitation, etc., which may occur against sublevel foundations, it is recommended that a foundation drain be installed around the home sublevel(s).

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) 590-0394. To schedule materials testing, please call (801) 381-5141.

14.0 REFERENCES

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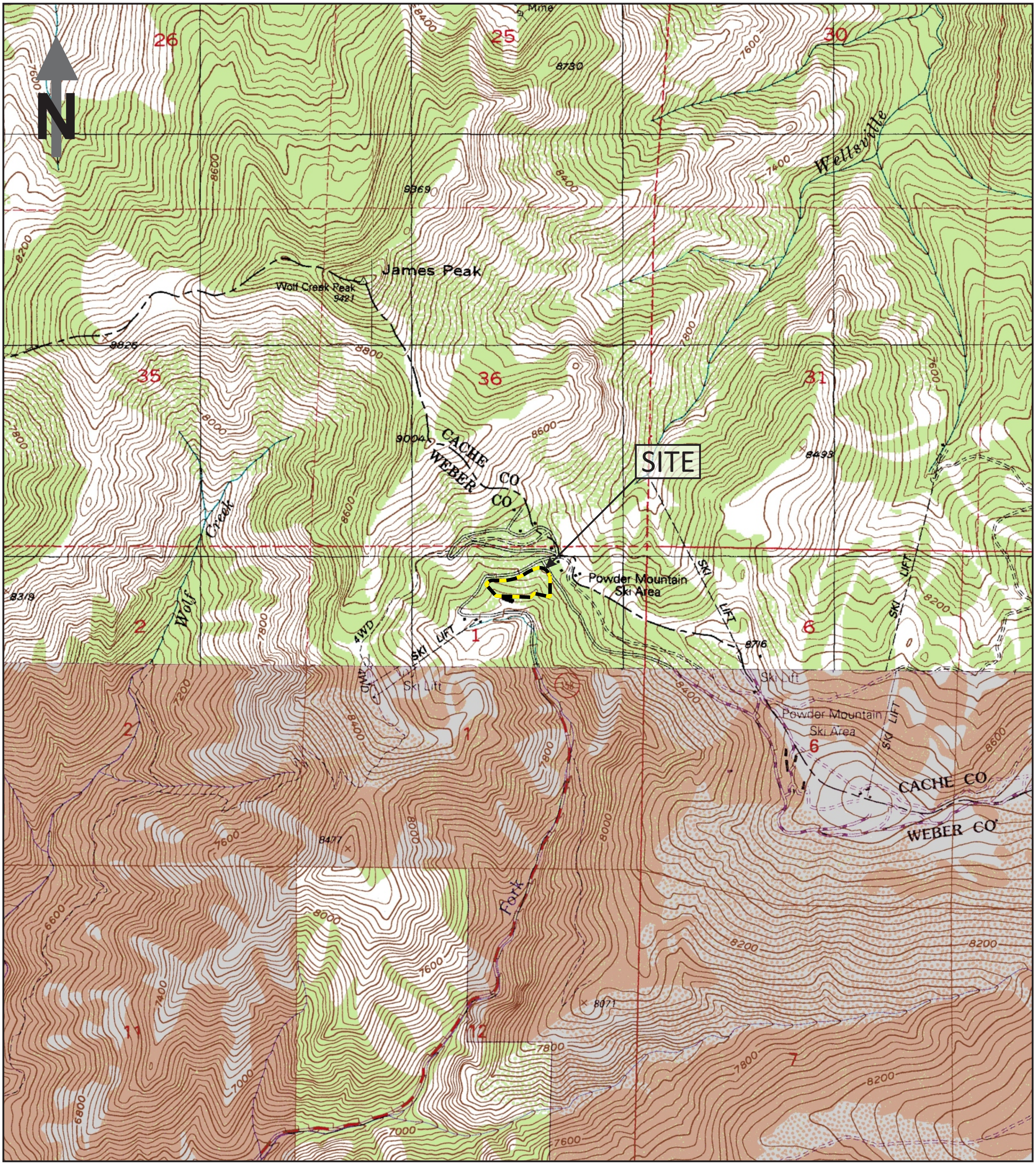
APPENDIX

SUPPORTING DOCUMENTATION

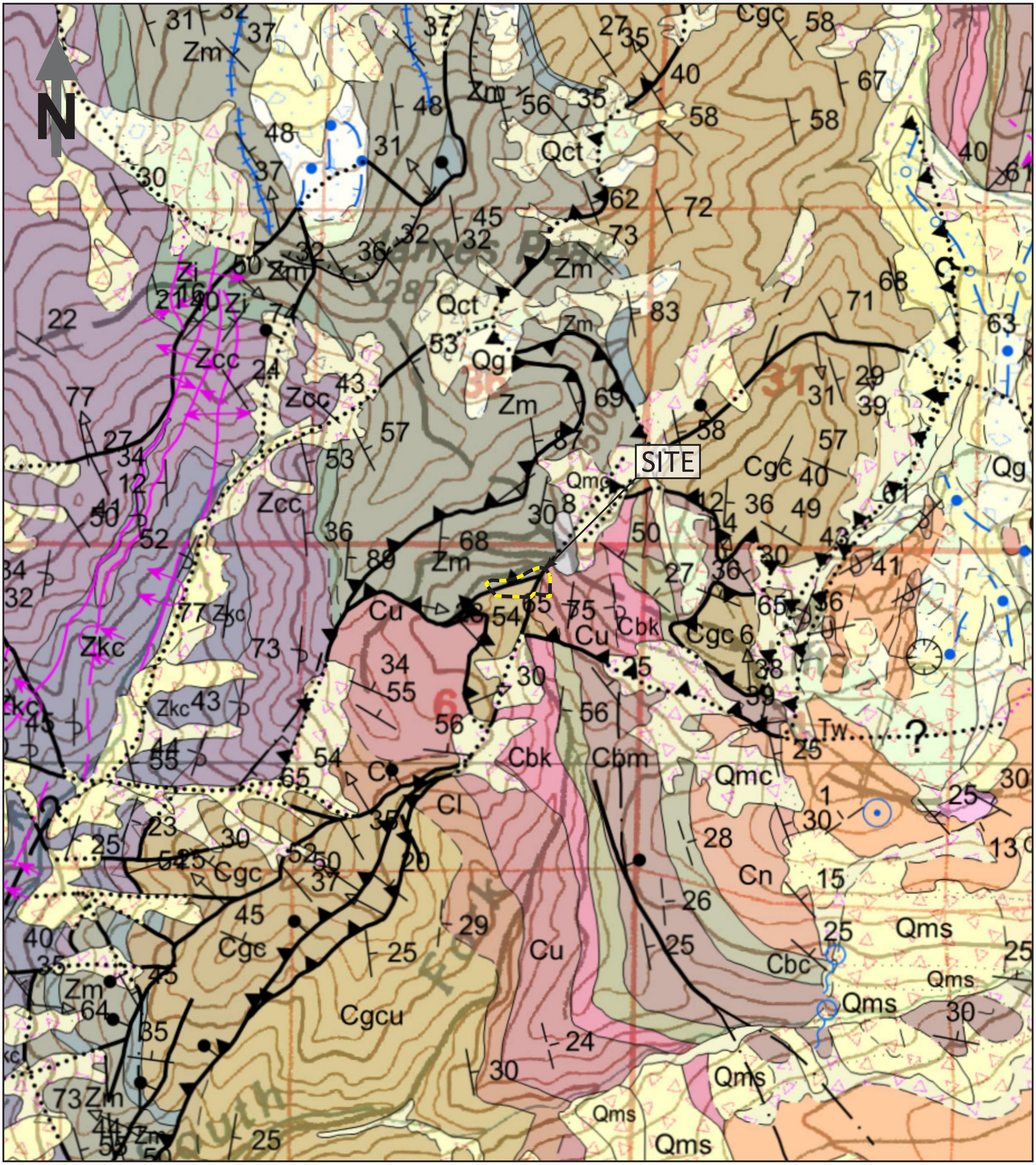


Base:
USGS 7.5-minute topographic quadrangles,
Utah; JAMES PEAK, 1991; and HUNTSVILLE, 1998.

0 2,000 ft 4,000 ft

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

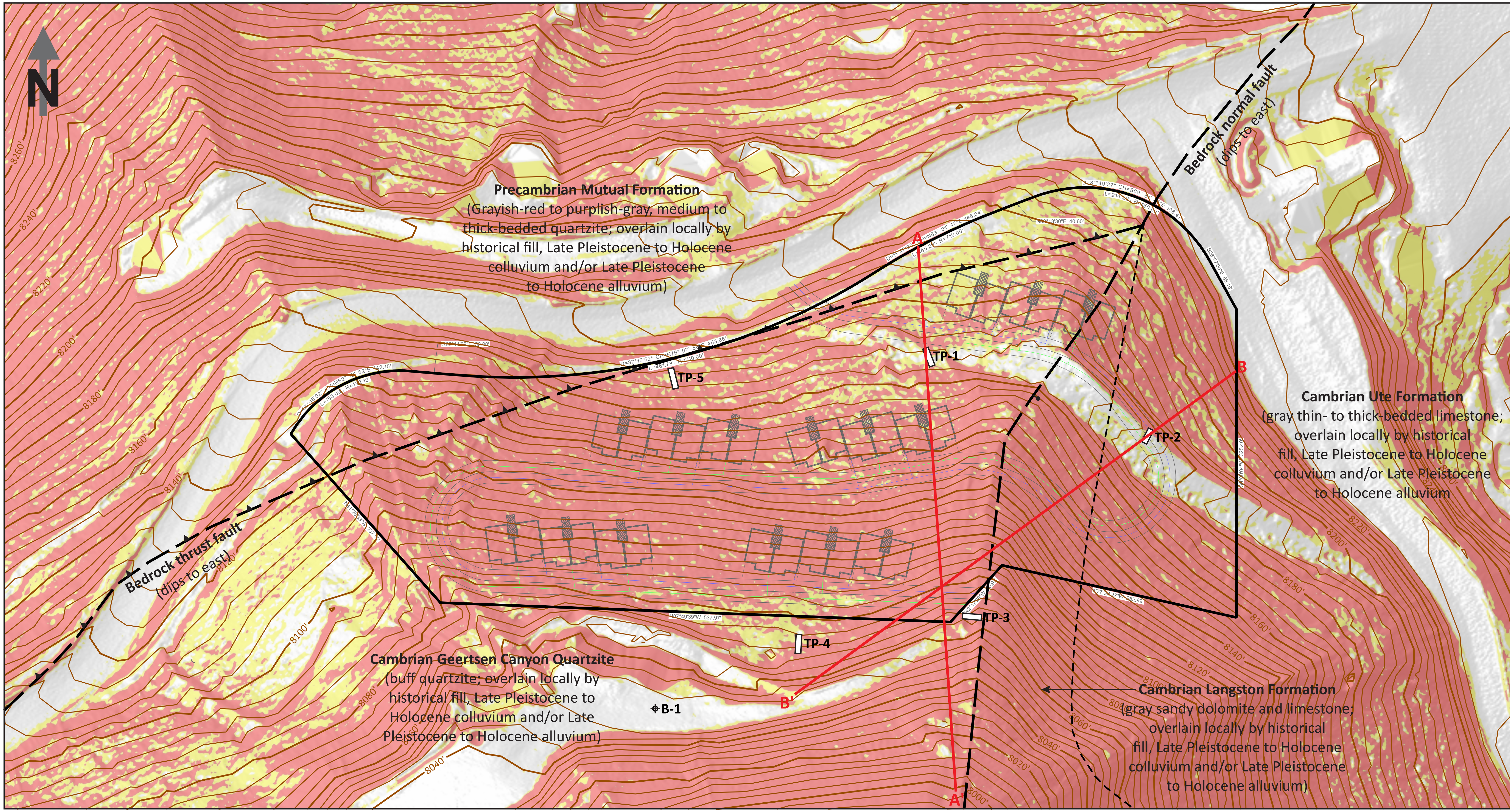
Proposed Sundown Condominiums About 6550 North Wolf Creek Drive, Eden, Utah	CMT ENGINEERING LABORATORIES Vicinity Map	Date: 30-Nov-2021 Job # 17355	Figure 1



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)

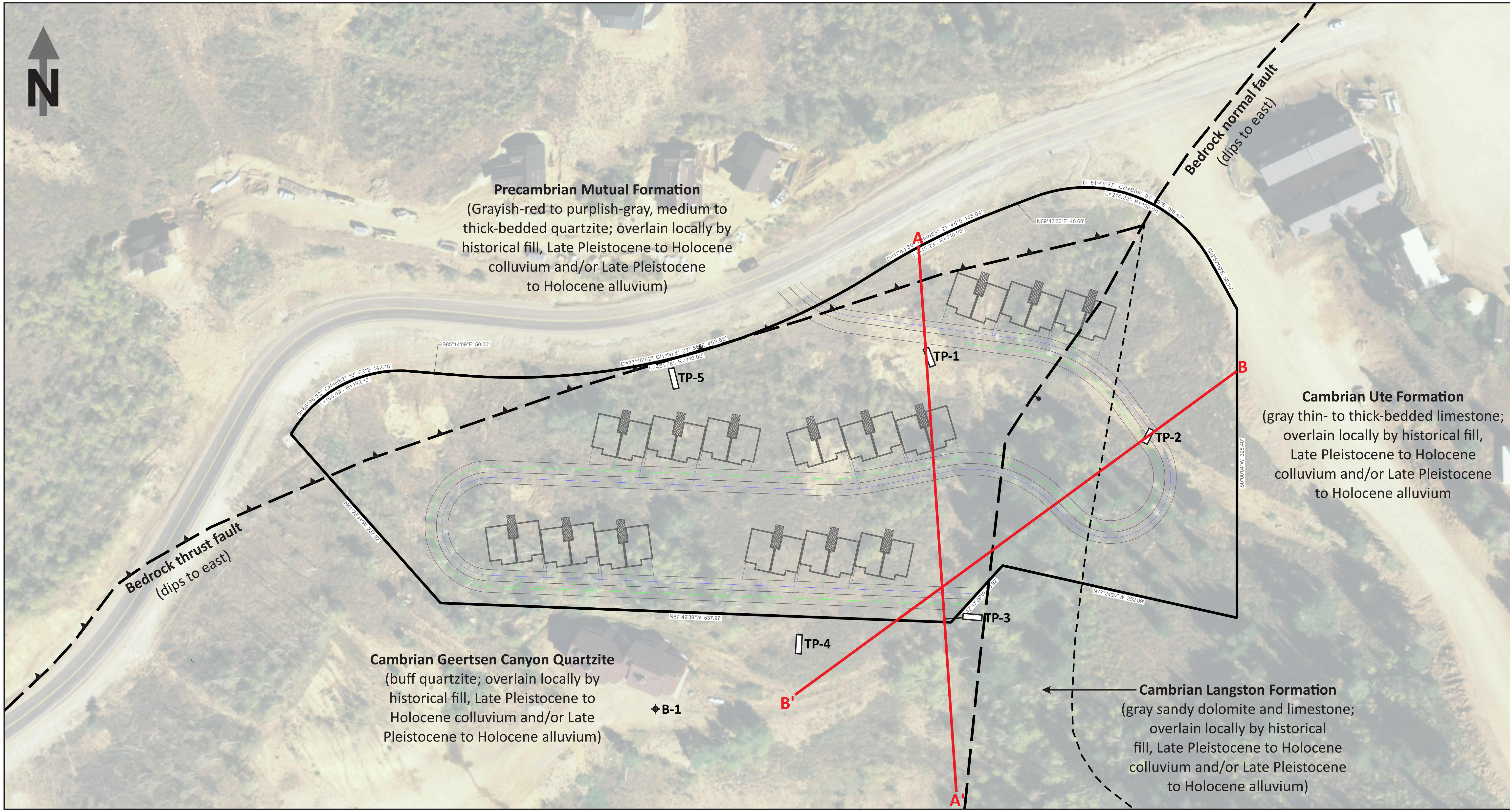
Proposed Sundown Condominiums About 6550 North Wolf Creek Drive, Eden, Utah	CMT ENGINEERING LABORATORIES		Figure 2
	Geologic Map	Date: 30-Nov-2021 Job # 17355	



Base:
Utah Geographic Resource Center, 2016 LIDAR Bare Earth DEM, 50 cm resolution; slopes <15% unshaded, 15-25% shaded in yellow, and >25% shaded in red. Contour interval 4 feet.

0 100 ft 200 ft
1:1,200 (1 inch equals 100 feet)

CMT ENGINEERING LABORATORIES	Proposed Sundown Condominiums About 6550 North Wolf Creek Drive, Eden, Utah	LIDAR Analysis	Date:	30-Nov-2021	Figure 3
			CMT No.:	17355	



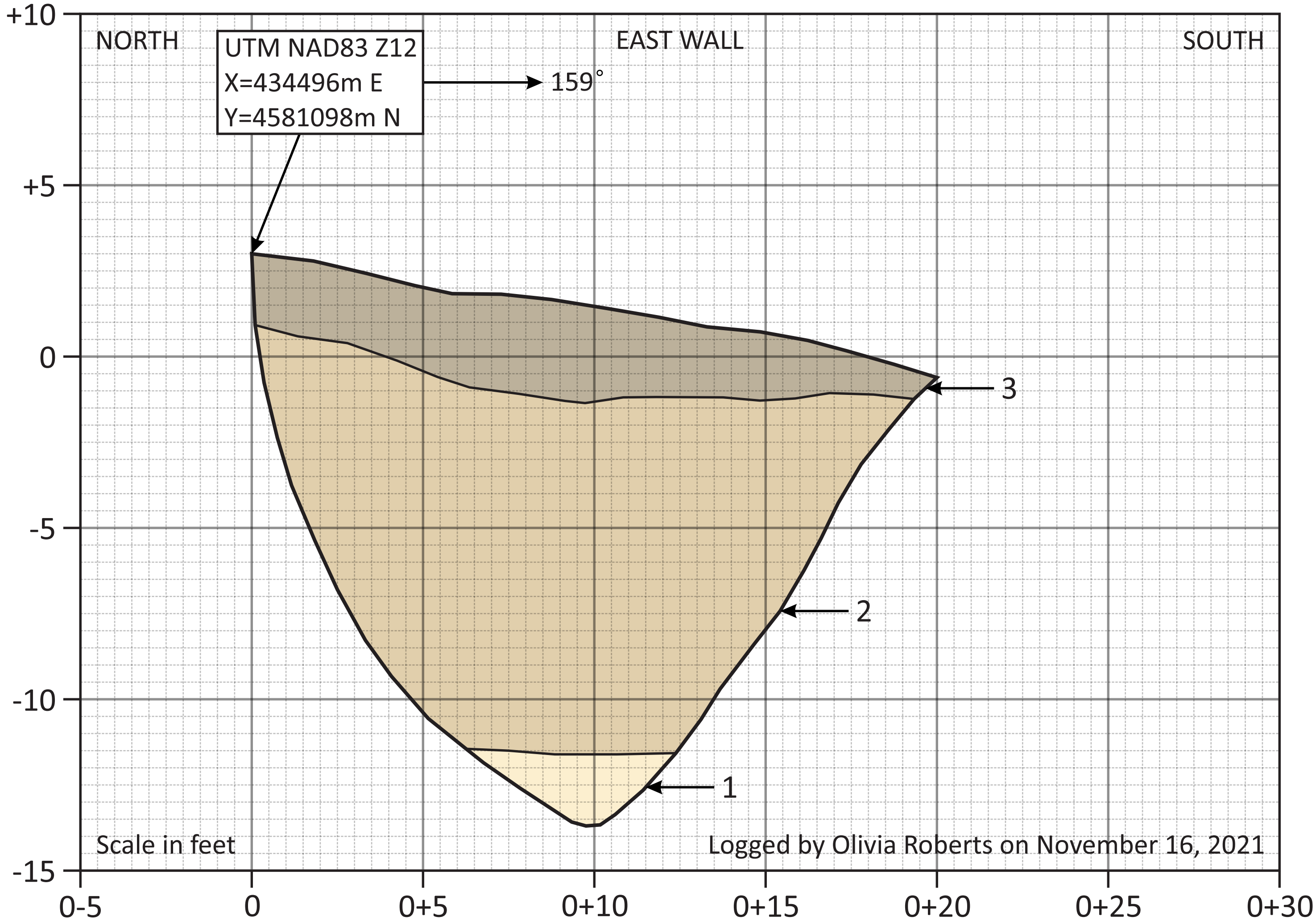
Base:
2012 Utah Geographic Resource Center high-resolution orthophoto, 12.5 cm resolution.

0 100 ft 200 ft

1:1,200 (1 inch equals 100 feet)

CMT ENGINEERING LABORATORIES	Proposed Sundown Condominiums About 6550 North Wolf Creek Drive, Eden, Utah	Site Evaluation	Date: 30-Nov-2021 CMT No.: 17355	Figure 4

TEST PIT 1



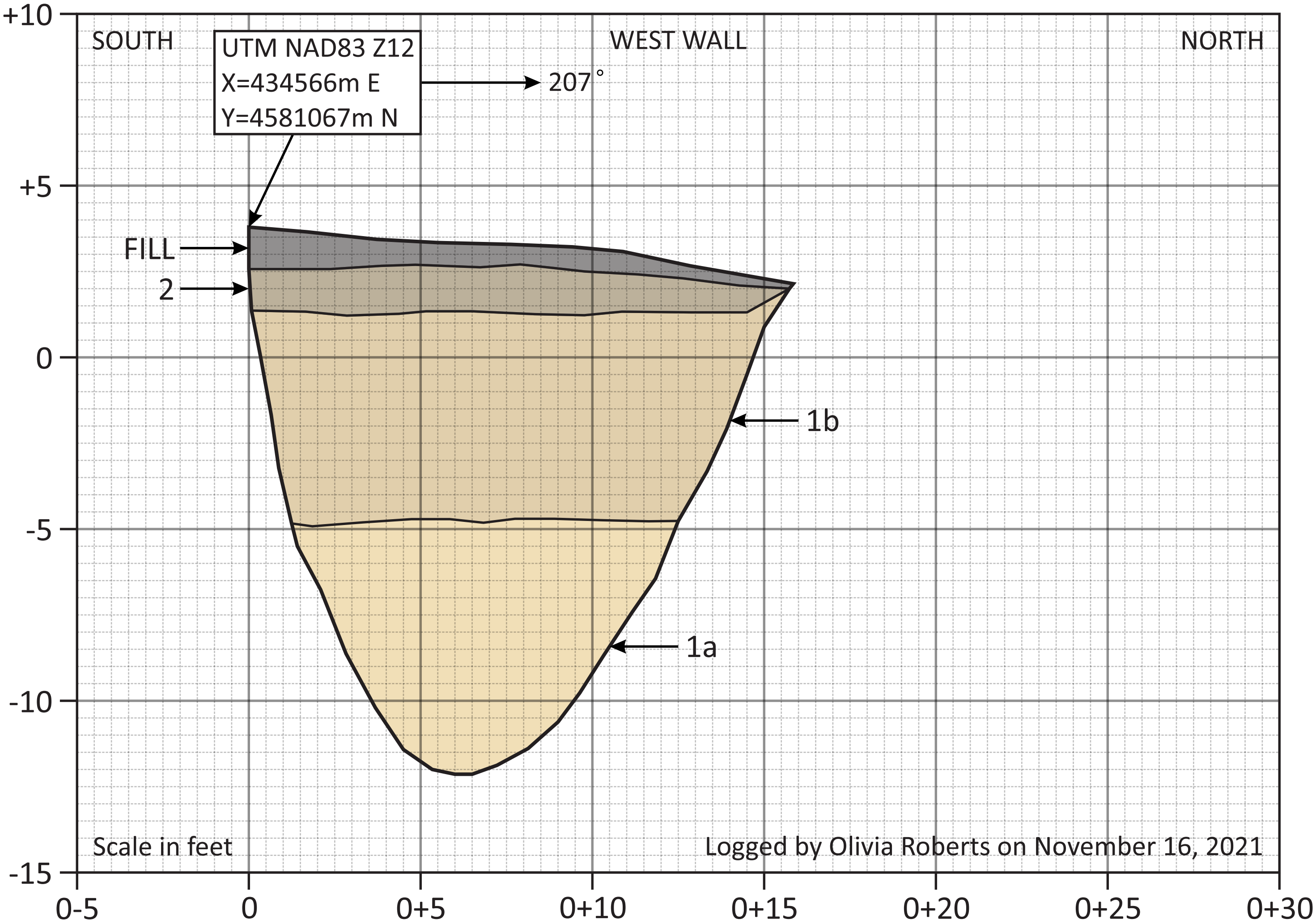
UNIT DESCRIPTIONS

Unit 1. *Cambrian Geertsen Canyon Quartzite* - yellow-brown, strong to very strong, massive, weathered quartzite bedrock; refusal at test pit floor.

Unit 2. *Late Pleistocene to Holocene mass wasting colluvium* - yellow-brown, medium dense, very poorly sorted, moist, silty to clayey gravel (GM/GC) with cobbles and sand; up to about 10.5 feet thick.

Unit 3. *Topsoil/Disturbed* - dark brown, loose, moist, silty to sandy clay (CL) with organics, some large roots, disturbed in upper 12"; about 1 to 2.5 feet thick.

TEST PIT 2

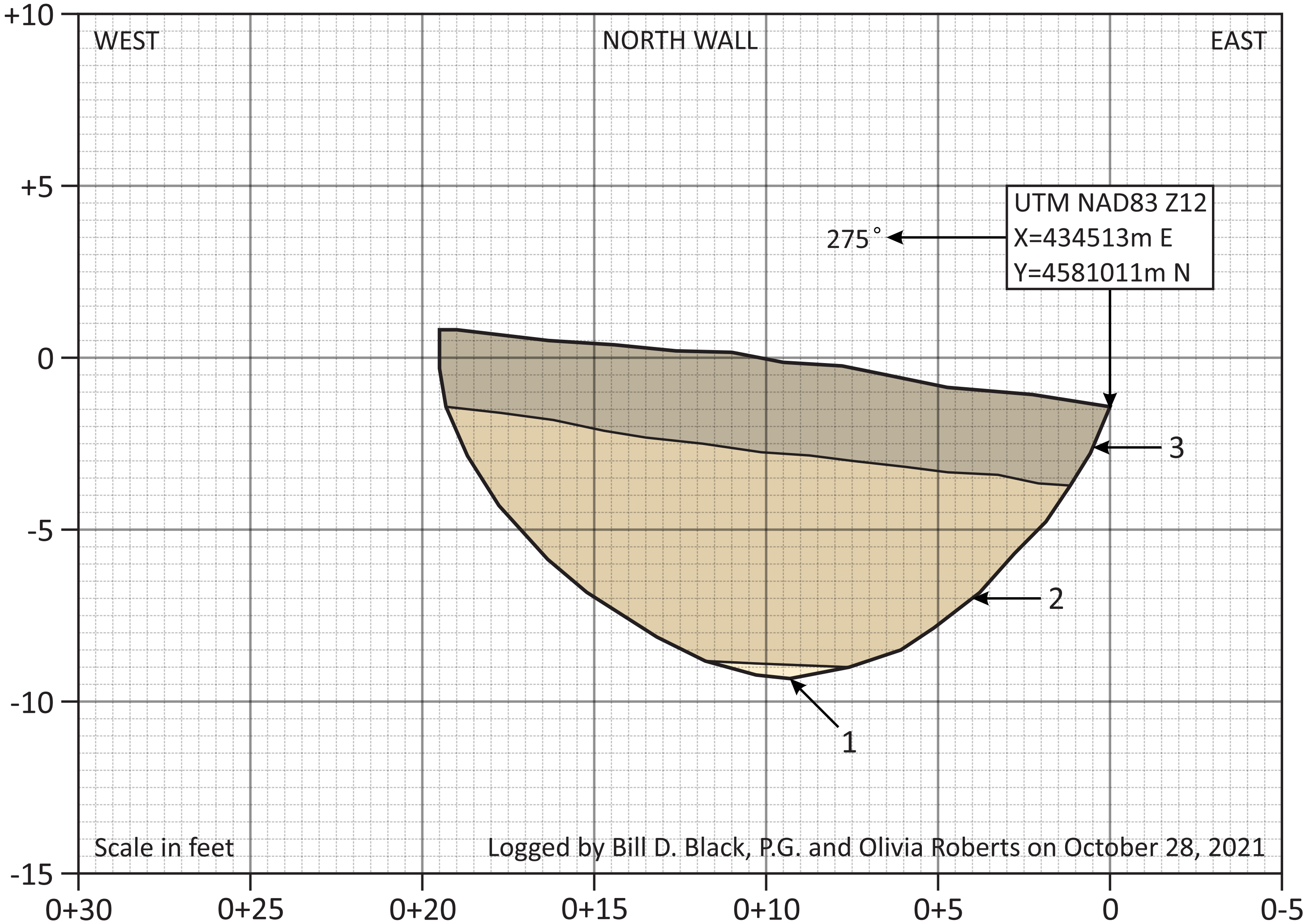


UNIT DESCRIPTIONS

Unit 1. *Late Pleistocene to Holocene mass wasting colluvium* - sequence of yellow-brown, sand to gravel comprised of a lower (1a) loose to medium dense, moderately sorted, moist, poorly bedded, sand and gravel (SW/GW); and an upper (1b) medium dense to dense, very poorly sorted, dry to slightly moist, massive, silty gravel (GM) with cobbles; overall more than 13.5 feet thick.

Unit 2. *Topsoil* - brown, loose, slightly moist, silty to sandy clay (CL) with roots, very organic rich; about 0 to 1 feet thick; overlain by fill from prior site disturbance.

TEST PIT 3

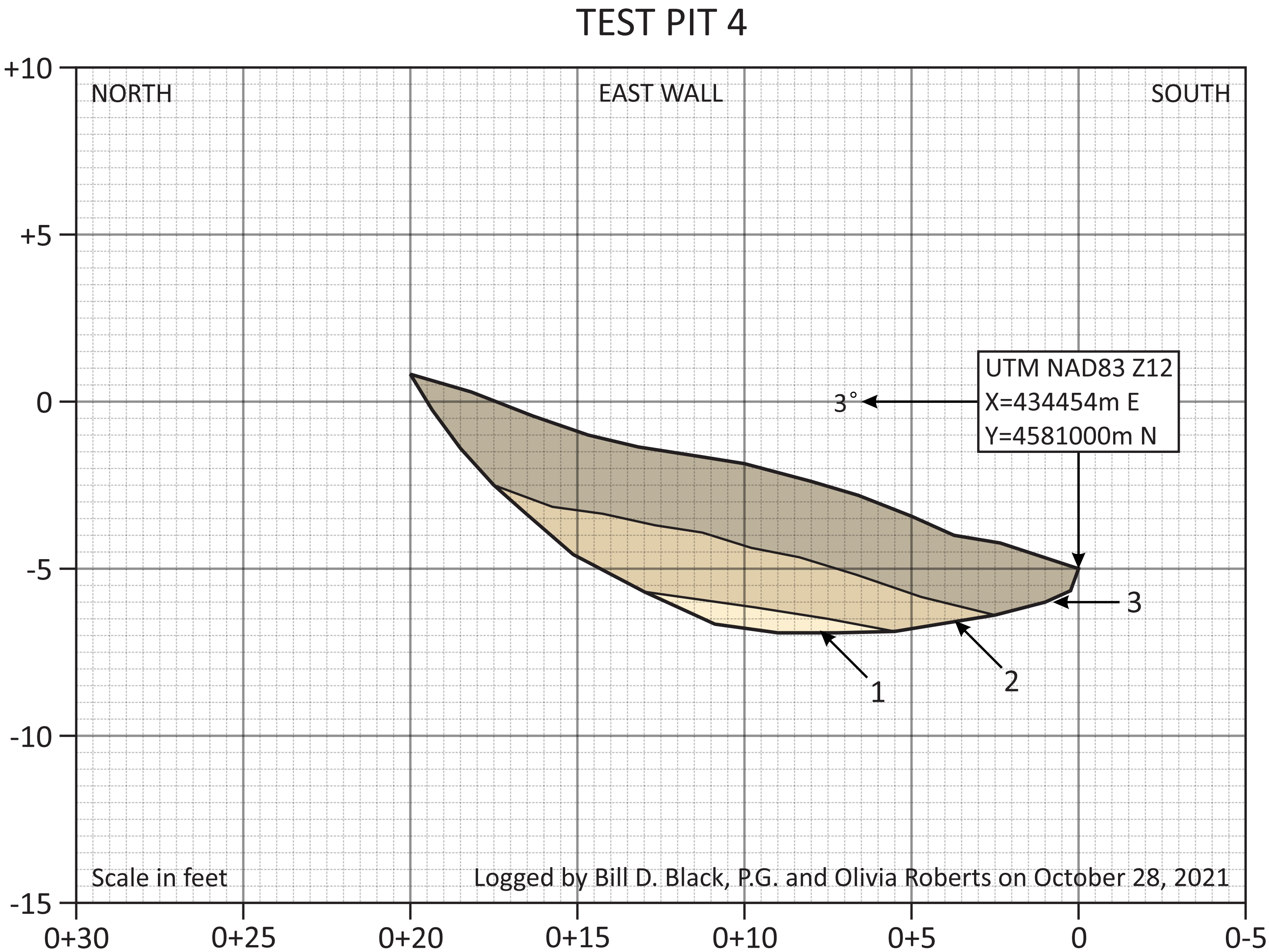


UNIT DESCRIPTIONS

Unit 1. *Cambrian Geertsen Canyon Quartzite* - yellow-brown, very strong, massive, dry, weathered quartzite bedrock.

Unit 2. *Late Pleistocene to Holocene mass wasting colluvium* - orange-brown, dense, slightly moist, clayey to silty gravel (GC/GM) with sand, cobbles and boulders; poorly bedded in lower part, coarser and massive in upper part; about 6.5 feet thick.

Unit 3. *Topsoil* - brown, medium dense, moist, silty to clayey sand (SM/SC) with gravel; A and B soil horizons formed in unit; twisted cable line at surface a few feet from west test pit end; about 2.5 feet thick.



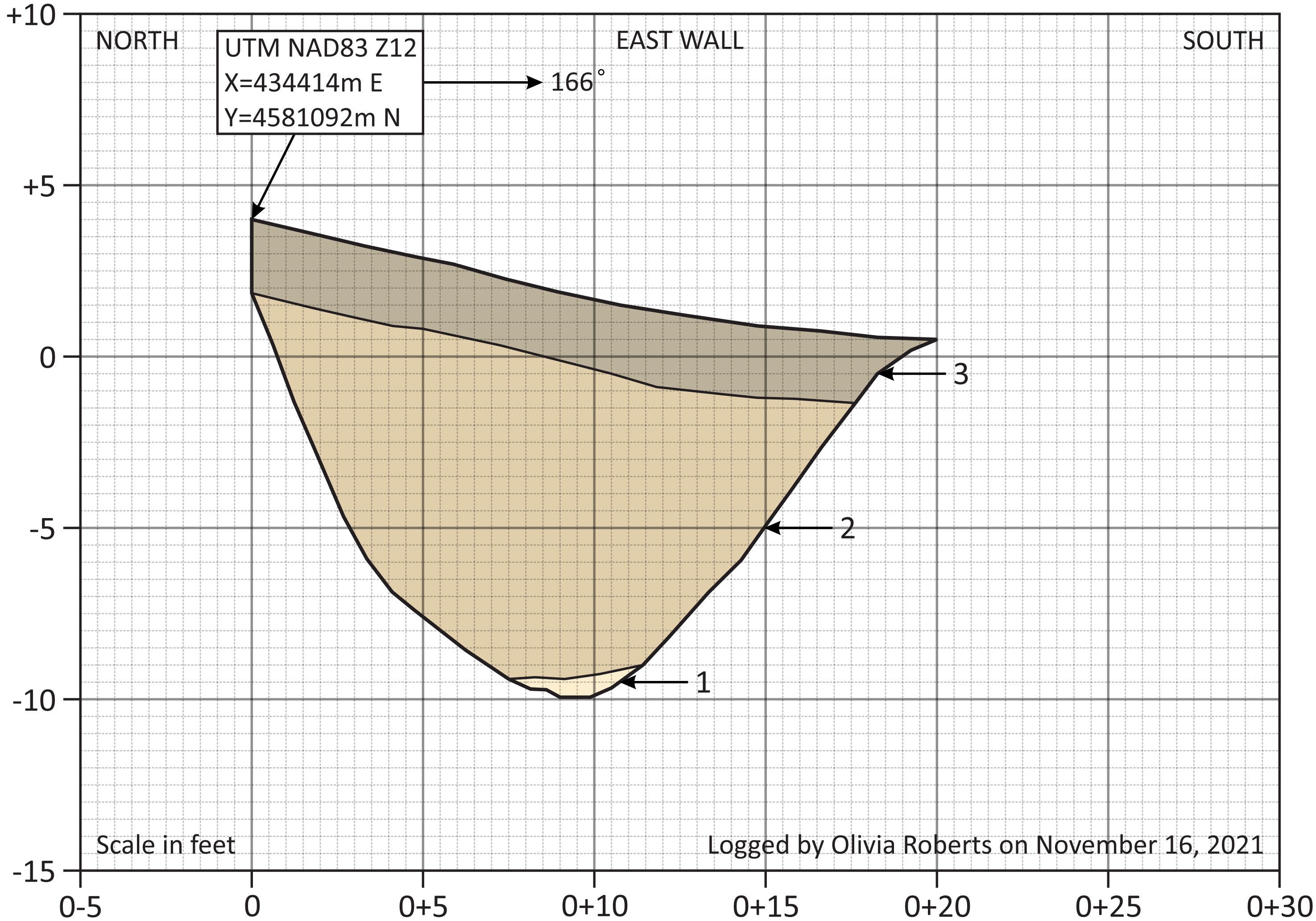
UNIT DESCRIPTIONS

Unit 1. *Cambrian Geertsen Canyon Quartzite* - yellow-brown, very strong, massive, dry, weathered quartzite bedrock; refusal at test pit floor.

Unit 2. *Late Pleistocene to Holocene mass wasting colluvium* - orange-brown, dense, massive, slightly moist, clayey to silty gravel (GC/GM) with sand, cobbles and boulders; about 2 feet thick.

Unit 3. *Topsoil* - brown, medium dense, massive, slightly moist, silty to clayey sand (SM/SC) with gravel and roots; A and B soil horizons formed in unit; about 2.5 feet thick.

TEST PIT 5

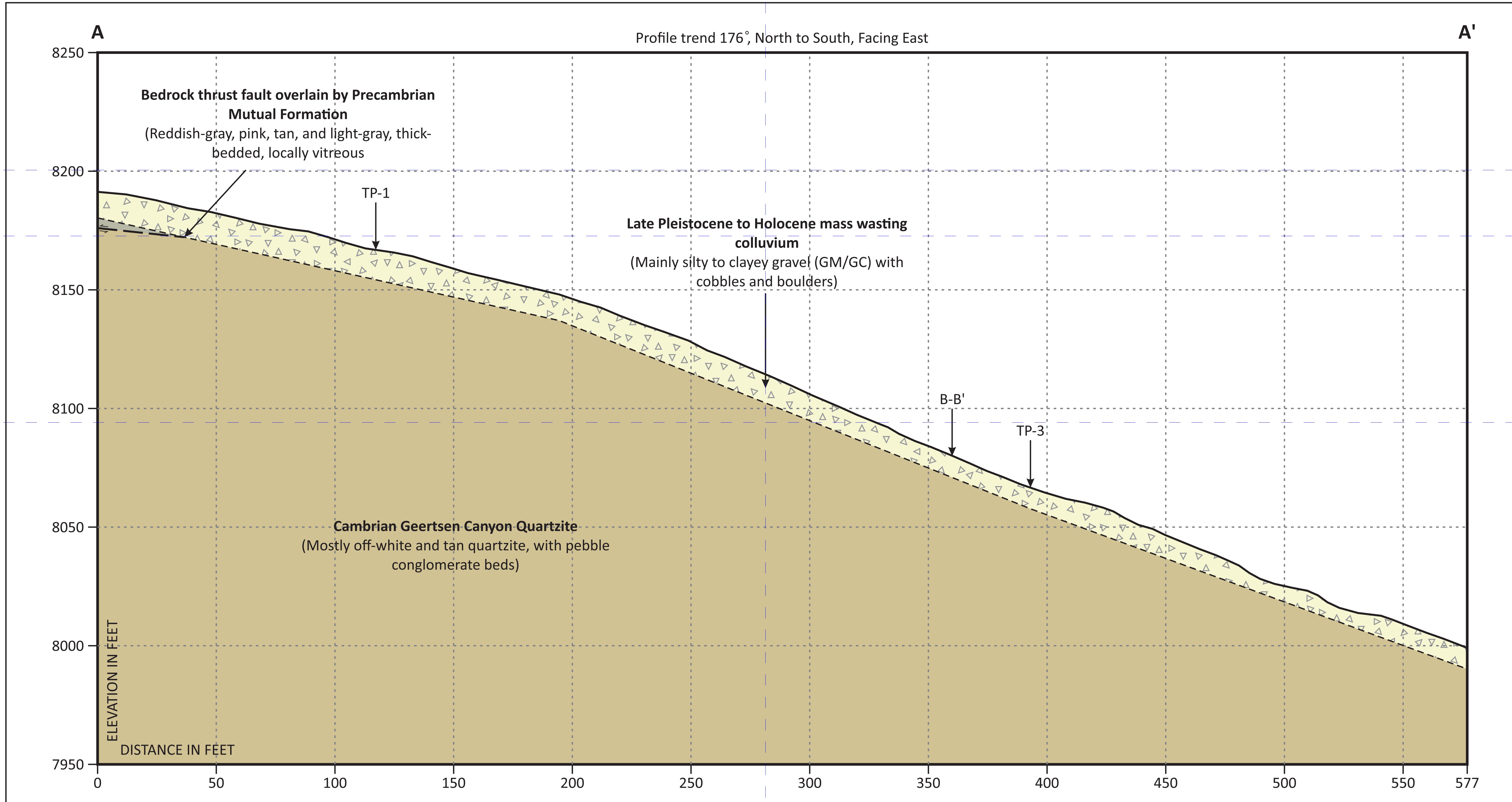


UNIT DESCRIPTIONS

Unit 1. *Cambrian Geertsen Canyon Quartzite* - yellow-brown, very strong, massive, weathered quartzite bedrock; refusal at test pit floor.

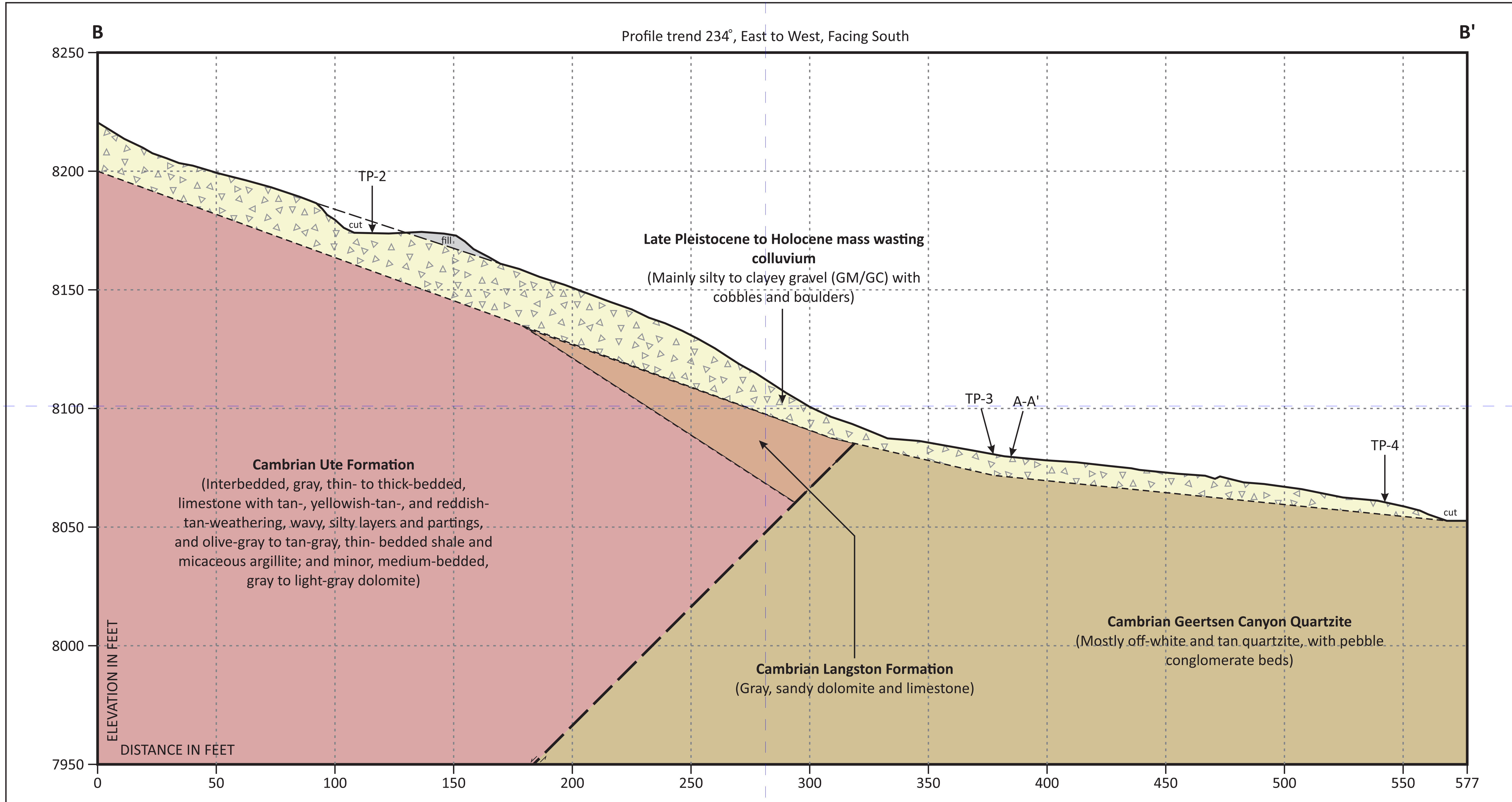
Unit 2. *Late Pleistocene to Holocene mass wasting colluvium* - yellow-brown, medium dense, angular, silty gravel (GM) with cobbles and sand; about 9.5 feet thick.

Unit 3. *Topsoil* - brown, loose, moist, silty clay (CL) with sand, organic rich; about 2 feet thick.



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

CMT ENGINEERING LABORATORIES	Proposed Sundown Condominiums About 6550 North Wolf Creek Drive, Eden, Utah	Cross Section A-A'	Date:	30-Nov-2021	Figure 6A
			CMT No.:	17355	



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

CMT ENGINEERING LABORATORIES

Proposed Sundown Condominiums
About 6550 North Wolf Creek Drive, Eden, Utah

Cross Section B-B'

Date:	30-Nov-2021
CMT No.:	17355

Figure 6B

Sundown Condominiums

About 6550 North Wolf Creek Drive, Eden, Utah

Bore Hole Log B-1A

Total Depth: 1'
Water Depth: (see Remarks)
Date: 9/30/21
Job #: 17355

Depth (ft)	GRAPHIC LOG	Soil Description	Sample Type	Sample #	Blows (N)	Blows (N) Total	Moisture (%)	Dry Density(pcf)	Gradation Gravel %	Gradation Sand %	Gradation Fines %	Atterberg LL	Atterberg PL	Atterberg PI
0		Fill; brown silty to sandy gravel with cobble; dry, very dense												
		REFUSAL AT 1.0'												
4														
8														
12														
16														
20														
24														
28														

Remarks: Groundwater not encountered during drilling.

Coordinates: 41.3777457°, -111.7837761°
Surface Elev. (approx): Not Given

Equipment: Hollow-Stem Auger
Automatic Hammer, Wt=140 lbs, Drop=30"
Excavated By: Direct Push
Logged By: Olivia Roberts

CMT ENGINEERING LABORATORIES

Figure: 7

Sundown Condominiums

About 6550 North Wolf Creek Drive, Eden, Utah

Bore Hole Log B-1

Total Depth: 11.5'
Water Depth: (see Remarks)

Date: 9/30/21
Job #: 17355

Depth (ft)	Soil Description	Sample Type	Sample #	Blows (N)	Total	Moisture (%)	Dry Density(pcf)	Gravel %	Sand %	Fines %	LL	PL	PI
0	Fill; brown clayey fine sand with gravel and roots dry, medium dense												
	grades with more gravel		1	7 10 10	20	4				26	42	30	12
4													
	Brown Silty SAND (SM) with gravel slightly moist, medium dense		2	10 15 16	31	6		31	23	46			
8	Light Reddish Brown Weathered BEDROCK (B) dry, dense		3	17 33 50/3"									
			4	15 16 35	51								
	REFUSAL AT 11.5'												
12													
16													
20													
24													
28													

Remarks: Groundwater not encountered during drilling.

Coordinates: 41.3776924°, -111.7840584°
Surface Elev. (approx): Not Given

Equipment: Hollow-Stem Auger
Automatic Hammer, Wt=140 lbs, Drop=30"
Excavated By: Direct Push
Logged By: Olivia Roberts

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Figure: 8

Sundown Condominiums

Key to Symbols

About 6550 North Wolf Creek Drive, Eden, Utah

Date: 9/30/21

Job #: 17355

① Depth (ft)	② GRAPHIC LOG	③ Soil Description	④ Sample Type	⑤ Sample #	⑥ Blows(N)	⑦ Total	⑧ Moisture (%)	⑨ Dry Density(pcf)	⑩ Gradation 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 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, Silty or Clayey Fine Sands or Clayey Silts with Slight
			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 with Plasticity (Elastic Silts)
			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

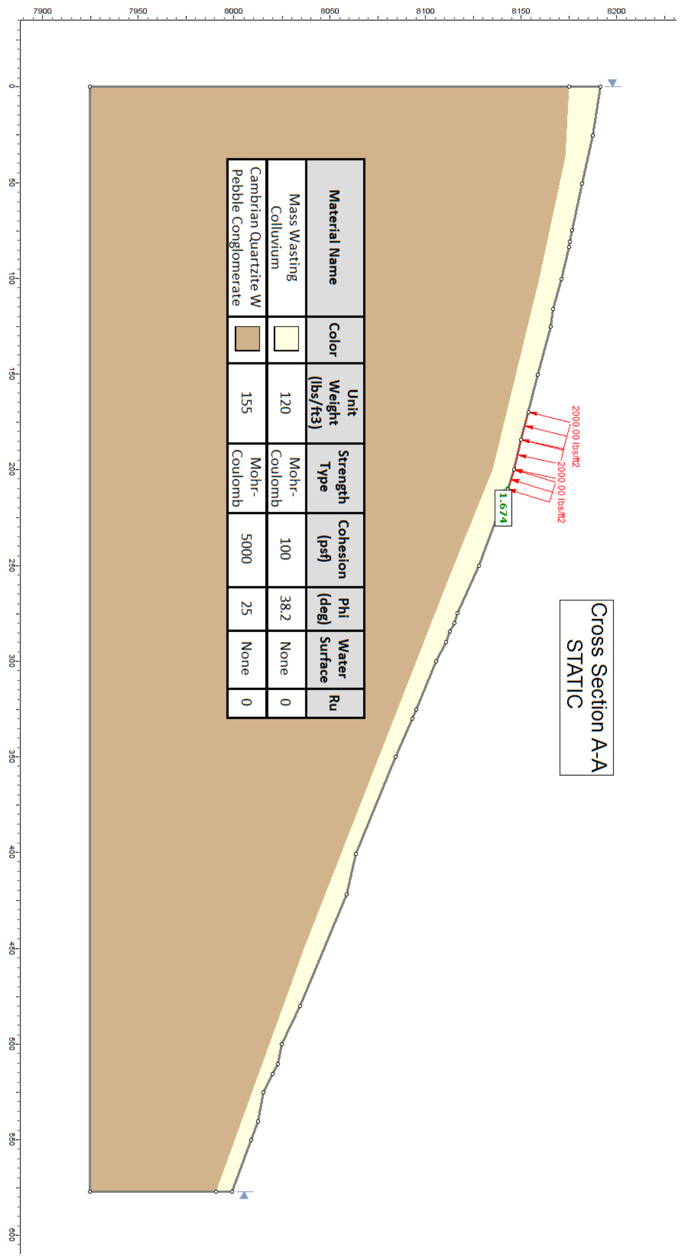
(see Remarks on Logs)

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

STABILITY RESULTS

Sundown Condominiums – Eden, Utah
Cross Section A-A
(STATIC)

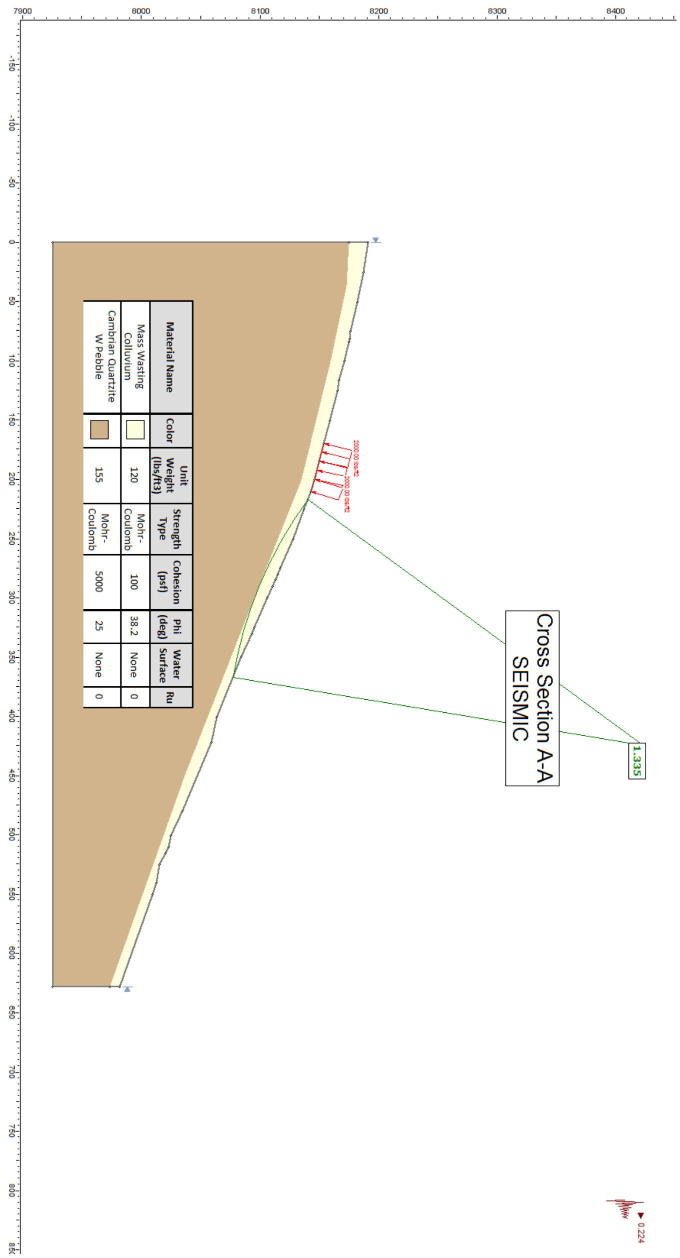


Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Ru
Mass Wasting Colluvium		120	Mohr-Coulomb	100	38.2	None	0
Cambrian Quartzite W Pebble Conglomerate		155	Mohr-Coulomb	5000	25	None	0

PROJECT NO.: 17355	Sundown Condos	**FIGURE NO.:** 9A

STABILITY RESULTS

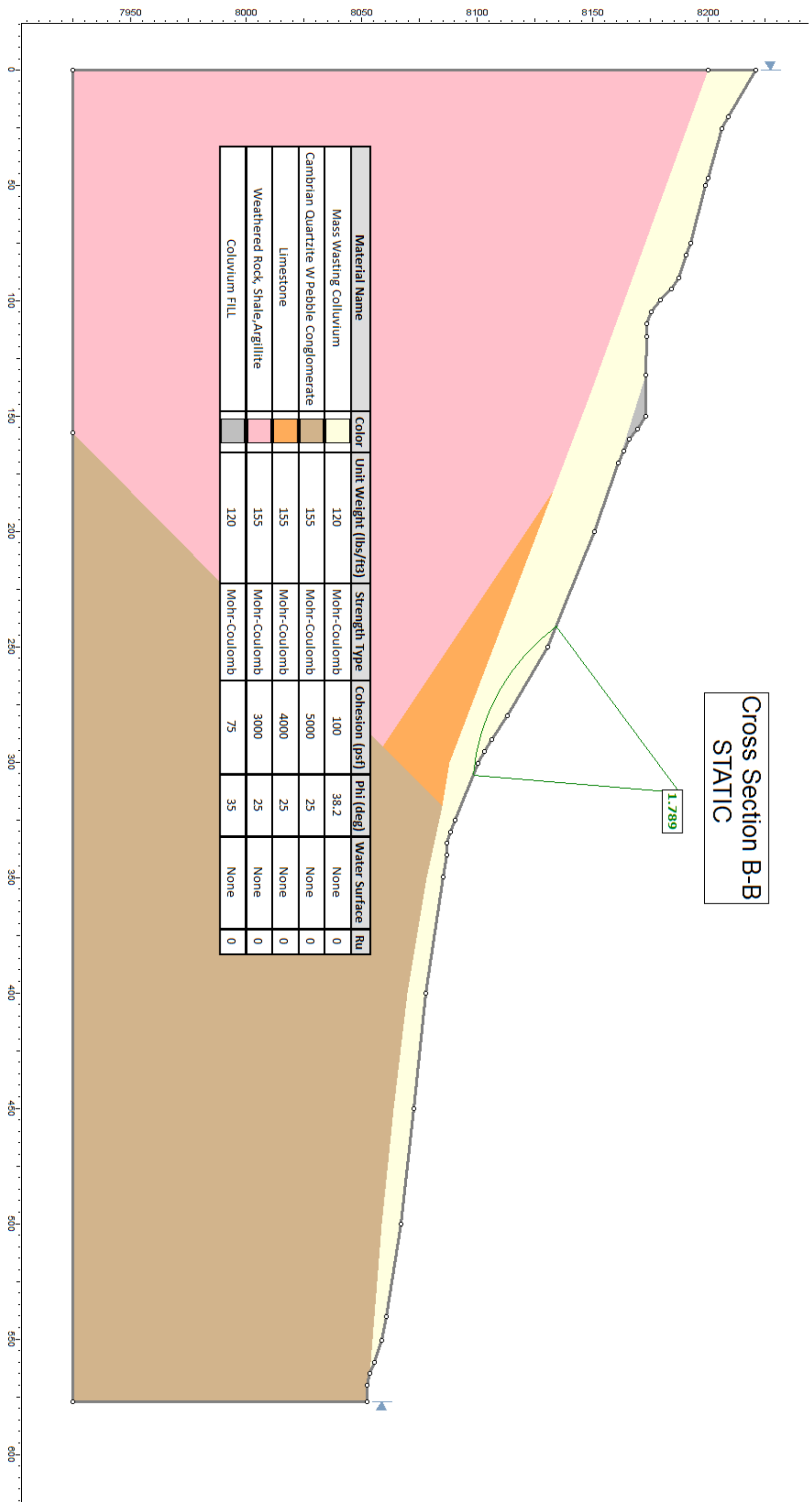
Sundown Condominiums – Eden, Utah
Cross Section A-A
(Seismic)



PROJECT NO.: 17355	Sundown Condos	**FIGURE NO.:** 9B

STABILITY RESULTS

Sundown Condominiums – Eden, Utah
Cross Section B-B
(Static)

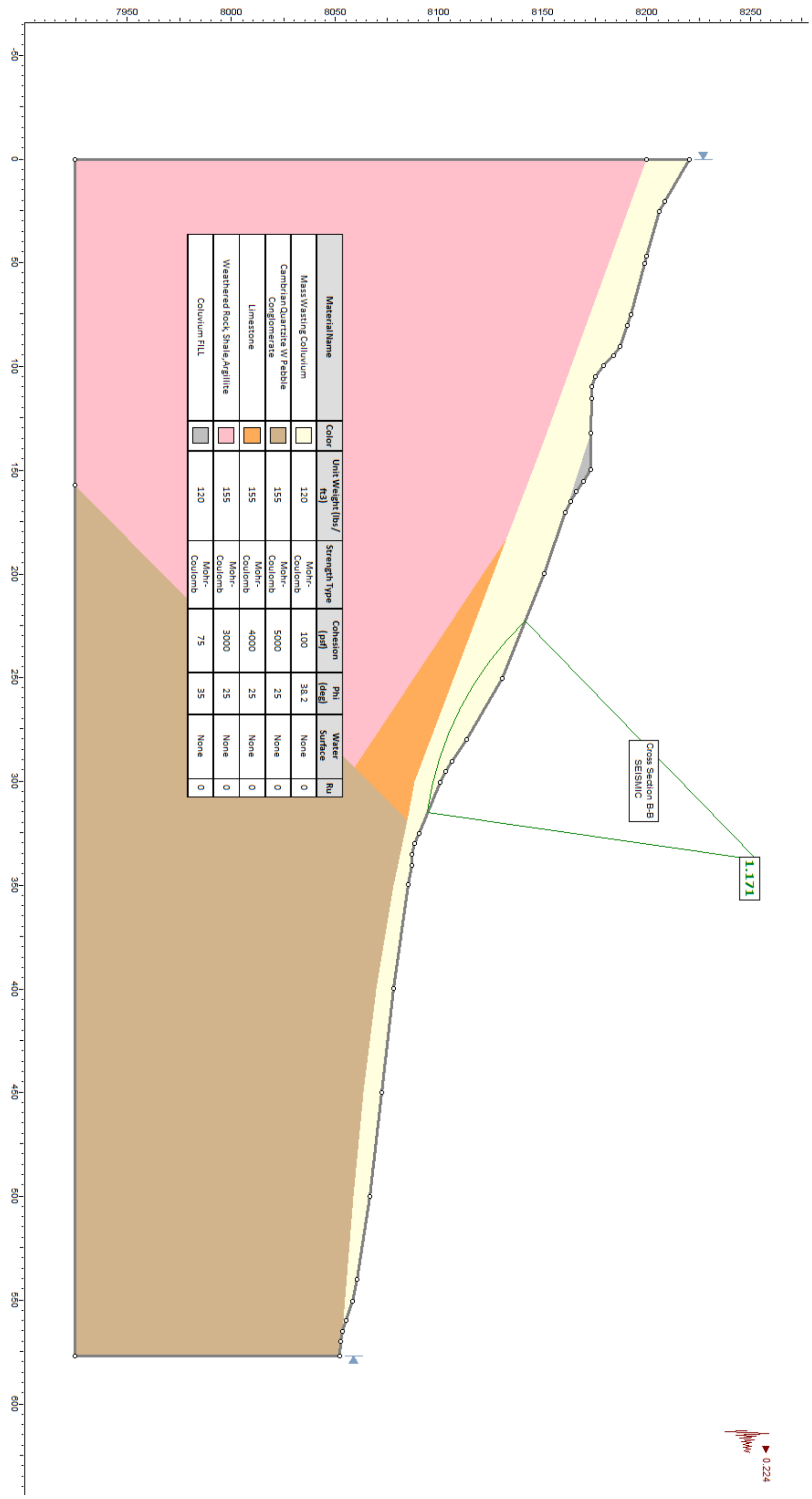


Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Ru
Mass Wasting Colluvium		120	Mohr-Coulomb	100	38.2	None	0
Cambrian Quartzite W Pebble Conglomerate		155	Mohr-Coulomb	5000	25	None	0
Limestone		155	Mohr-Coulomb	4000	25	None	0
Weathered Rock, Shale,Argillite		155	Mohr-Coulomb	3000	25	None	0
Colluvium FILL		120	Mohr-Coulomb	75	35	None	0

PROJECT NO.: 17355	Sundown Condos	**FIGURE NO.:** 10A

STABILITY RESULTS

Sundown Condominiums – Eden, Utah
Cross Section B-B
(Seismic)



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Ru
Mass Wasting Colluvium		120	Mohr-Coulomb	100	38.2	None	0
Cambrian Quartzite W Pebble Conglomerate		155	Mohr-Coulomb	5000	25	None	0
Limestone		155	Mohr-Coulomb	4000	25	None	0
Weathered Rock, Shale, Argillite		155	Mohr-Coulomb	3000	25	None	0
Coluvium FILL		120	Mohr-Coulomb	75	35	None	0

PROJECT NO.: 17355	Sundown Condos	**FIGURE NO.:** 10B