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

GEOTECHNICAL ENGINEERING AND GEOLOGIC STUDY

Sundown Condominiums Phase 3

About 6550 North Powder Mountain Road
Eden, Utah
CMT PROJECT NO. 24298

FOR:
New West Building Company
P.O. Box 13308
Jackson, Wyoming 83002

June 21, 2025

June 23, 2025

Mr. Craig Wilcox
New West Building Company
6143 South Willow Drive, Suite 300
Salt Lake City, Utah 84111

Subject: Geotechnical Engineering and Geology Study
Sundown Condominiums Phase 3
About 6550 North Powder Mountain Road
Eden, Utah
CMT Project No. 24298

Mr. Wilcox:

Submitted herewith is the report of our geotechnical engineering study for the subject site. This report contains the results of our findings and an engineering 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 May 22, 2025, a CMT Technical Services (CMT) staff professional was on-site and supervised the excavation of 4 test pits extending to depths of about 9.0 to 12.5 feet below the existing ground surface. We obtained soil samples during the field operations that we subsequently transported to our laboratory for further testing and observation.

Conventional spread and/or continuous footings may be utilized to support the proposed structures, provided the recommendations in this report are followed. This report presents detailed discussions of design and construction criteria for this site.

We appreciate the opportunity to work with you at this stage of the project. CMT offers a full range of Geotechnical Engineering, Geological, Material Testing, Special Inspection services, and Phase I and II Environmental Site Assessments. With offices throughout Utah, Idaho, Arizona, Colorado and Texas, our staff is capable of efficiently serving your project needs. If we can be of further assistance or if you have any questions regarding this project, please do not hesitate to contact us at 801-590-0394.

Sincerely,
CMT Technical Services

LICENSED PROFESSIONAL GEOLOGIST
6/23/2025
BILL DUANE BLACK
NO. 5224898-2250
STATE OF UTAH

LICENSED PROFESSIONAL ENGINEER
276476
BRYAN N. ROBERTS
6/23/2025
STATE OF UTAH

Bill D. Black, P.G.
State of Utah No. 5224898-2250
Engineering Geologist

Bryan N. Roberts, P.E.
State of Utah No. 276476
Senior Geotechnical Engineer

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APPENDIX

1.0 INTRODUCTION

1.1 General

This report presents results of a geotechnical engineering and geologic study conducted by CMT Technical Services (CMT) for Phase 3 of the proposed Sundown Condominiums development in Eden, Utah. The property is located in the N1/2, Section 1, Township 7 North, Range 1 East (Salt Lake Base Line and Meridian) and is identified as Weber County Assessor Parcels #22-001-0046 (0.7 acres) and #22-001-0045 (2.66 acres). Elevation of the property ranges from about 8,009 to 8,133 feet above sea level. Location of the property is shown on **Figure 1, Vicinity Map**. Regional geology of the property and nearby area is shown on **Figure 2, Geologic Map**. Locations of the geologic test pits conducted for our subsurface investigation are shown on **Figure 3, Site Evaluation.** A historical black and white air photo of the property and nearby area is shown on **Figure 4, 1958 Air Photo**. Slope-terrain information of the property and nearby area is shown on **Figure 5, Slope Analysis**. Site-specific surficial geology of the property and nearby area is shown on **Figure 6, Site-Specific Geology**.

1.2 Objectives, Scope and Authorization

The objectives and scope of our study were planned in discussions among Mr. Craig Wilcox of New West Building Companies, Mr. Guy Williams of Fawkes Consultants, and Mr. Bryan Roberts of CMT Technical Services (CMT), and are outlined in our proposal dated April 21, 2025.

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 excavating, logging and sampling four 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. A fifth test pit was planned in the southeast part of the property, but was not conducted due to buried utility conflicts.

Engineering geologic analyses and this report have been conducted and prepared in accordance with Bowman and Lund (2020) and current generally accepted professional engineering geologic and geotechnical principles and practice in Utah. Based on the above, recommendations are provided herein to be utilized in appropriate site development and design and construction of the proposed development.

1.3 Description of Proposed Construction

The site is proposed for expanding the existing development to the west and will incorporate 22 additional condominium units in three separate rows with extended roadway and infrastructure. Structures are anticipated 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 50 kips, respectively. New, private roadways are anticipated to be paved with asphalt concrete.

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 faulting between about 90 to 125 million years ago. The thrust faulting is associated with the upper branch of the Willard thrust, which crosses the northern part of the property. An east-dipping normal fault is also further east that truncates the thrust fault. None of these faults are considered active from a development perspective. Bedrock in the north side of the Willard thrust is mapped as Precambrian Mutual Formation, whereas bedrock in the south side is mapped as Cambrian Geertsen Canyon Quartzite. Upper Pleistocene- to Holocene-age mass wasting colluvium of varying thickness overlies these bedrock units.

2. Slopes at the site are mainly moderate to steep and dip to the southeast at an overall roughly 4.5:1 (horizontal:vertical; or 22.2%, 12.5 degrees). An area of gentle slopes is in the southeast part of the site associated with a cut and fill for the access road to Sundown Condominiums Phases 1 and 2 further east.

3. Test pits TP-1 through TP-4 all exposed colluvium to their explored depths (9 to 11 feet). A roughly 3 to 4 foot-thick zone of groundwater seepage from seasonal snowmelt was also observed in the test pit exposures at a depth of around 2 feet. This seasonal perched groundwater layer likely dries up in the summer and fall. Groundwater at the site is otherwise estimated to be at a depth of about 250 feet based on nearby water well data.

4. No evidence for active faulting, landslides, recent or ongoing slope instability, characteristic debris flow morphology, springs or seeps, bedrock outcrops that could pose a source area for rockfalls, or other geologic hazards (except for earthquake ground shaking) was identified at the site during our investigation.
5. All vegetation, topsoil and non-engineered fills shall be removed below structures.
6. Bedrock was not encountered within the upper about 11 feet explored but is assumed to be shallow based on adjacent explorations completed for Phase 2 and was assumed in the slope modeling. CMT must be allowed to review the construction excavations to confirm the conditions assumed. Where bedrock is anticipated within the deeper cuts, heavy equipment and possible blasting could become necessary to complete the excavations.

7. The overall slope generally has factors of safety for both static and pseudo-static (earthquake) conditions in excess of those typically considered acceptable *<u>with the exception of the cut sections</u>*

downslope of the proposed structures (See **Figures 9A and 9B Stability Results**). *Planned retaining walls and foundation walls will require further design to resist the lateral loads.*

8. For roadway construction we recommend that cuts and fills not be steepened more that 2:1 (H:V) without retaining structures. 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.

A geotechnical engineer/geologist from CMT must be allowed to observe the subgrade and excavations to assess if topsoil, undocumented fills, or other deleterious 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 the 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 Project were explored by excavating four test pits to depths of up to 11 feet below the ground surface (bgs) for geologic/geotechnical logging and sampling. Exploration locations are displayed on **Figures 3 through 6**. During the course of the field exploration, a continuous log of the subsurface conditions encountered was maintained. Undisturbed tube, block and/or 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[1] 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. Field classifications may therefore differ somewhat from lab data.

Test pit locations were measured using a handheld GPS unit and by trend and distance methods. Location, trend, unit descriptions, and other pertinent data and observations are provided on the logs. Geologic logs of the test pits are provided on **Figures 7A through 7D, Test Pit Logs**. Geotechnical logs (measured sections) of the test pits are provided on **Figures 7E through 7H, Geotechnical Logs**, including sampling information, location, and other pertinent geotechnical data and observations. A Key to Symbols defining the terms and symbols used on the geotechnical test pit logs is provided on **Figure 9, Key to Symbols**. Subsurface conditions encountered in the test pits are summarized in Section 3.2 below.

[1]American Society for Testing and Materials

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 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); photogeologic analysis of 1997 and 2012 orthophotography available from the Utah Geospatial Resource Center, as well as a high-resolution Google Earth™ aerial image from 2024 (**Figure 3**); examination of stereo-paired U.S. Department of Agriculture aerial photography from 1958 (frames AAJ 26V-74 and AAJ 26V-75, dated June 17, 1958, original scale 1:20,000; **Figure 4**); review of geoprocessed Light Detection and Ranging (LiDAR) bare earth digital elevation (DEM) mapping from 2016 available from the Utah Geospatial Resource Center (**Figure 5**); a field reconnaissance of the property; and interpretation of the test pits conducted at the site as part of our field program. Site-specific geology of the Project at a scale of 1 inch equals 100 feet (1:1,200) is shown on **Figure 6** based on previous mapping and site-specific data. Unit labels and descriptions on **Figure 6** correspond to those provided in Section 5.2. Seismic hazards information was developed from United States Geologic Survey (USGS) databases (Peterson and others, 2008).

The property is located adjacent to Power Mountain Ski Area at the head of South Fork Wolf Creek, which flows southwestward to Ogden Valley. Native vegetation at the property consists mainly of mature trees, grasses and brush. Slopes at the site appeared moderate to steep, except for a cut and fill for the access road to the condominiums further east. Numerous buried utilities were identified in the access road. An area of surface seepage was evident in the central part of the site, likely from recent snowmelt. No evidence for active faulting, landslides, recent or ongoing slope instability, characteristic debris flow morphology, bedrock outcrops that could pose a source area for rockfalls, or other geologic hazards was also observed at the site on air photos or during the field reconnaissance.

3.2 Subsurface Soils

Five test pits were initially planned at the property to evaluate subsurface soil conditions, but only four test pits were completed due to utility conflicts. The test pit locations are shown on **Figures 3 through 6**. All the test pits exposed mass wasting colluvium comprised mainly of silty to clayey sand and gravel with cobbles and boulders to their explored depths (9 to 11 feet). Weathered quartzite bedrock likely underlies the colluvium and was observed in test pits conducted further east in 2021 by CMT for Sundown Condominiums Phase 2.

3.3 Geologic Cross Section

Figure 8, Cross Section A-A' shows one geologic cross section across the site at a scale of 1 inch equals 40 feet with no vertical exaggeration. Location of the cross section is shown on **Figures 3 through 6**. Units and contacts are based on subsurface data from the test pits (**Figures 5A-H**) and/or inferred from the site-specific surficial geologic mapping on **Figure 6**. The existing and proposed grades are based on a Fawkes Consultants Inc. exhibit dated 6/10/2025. Units and contacts should be considered approximate and inferred, and variations should be expected at depth and laterally.

3.4 Groundwater

A roughly 3 to 4 foot-thick zone of groundwater seepage from seasonal snowmelt was also observed in the test pit exposures at a depth of around 2 feet. This seasonal perched groundwater layer likely dries up in the summer and fall. No additional site-specific groundwater information was found available, but the Utah Division of Water Rights Well Driller Database shows one water well about 1,950 feet south-southeast 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 typically more than 250 feet deep. However, groundwater depths may vary seasonally from snowmelt runoff, annually from climatic fluctuations, locally with topography and subsurface conditions, and in response to upslope surface-water infiltration (such as from snowmelt or irrigation). The seasonal perched groundwater at the site would be reflective of such variations.

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. Also, after completing the logging and sampling, the test pits were backfilled with the excavated soils but minimal to no effort was made to compact these soils. Thus, the test pit backfill is considered undocumented fill and settlement of the backfill in the test pits over time should be anticipated.

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 (WFZ) 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 further east that 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[2] 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[3]. The University of Utah Seismograph Stations[4] 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 Geertsen Canyon Quartzite (unit Cgc). Detailed surficial geologic mapping at a scale of 1 inch equals 100 feet (1:1,200) is shown on **Figures 3 through 6** 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.

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

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

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

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

Tw – *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 figures in the above descriptions are not provided herein, but are in Coogan and King (2016). Descriptions of other units on Figure 2 not provided above are also 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 WFZ 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 International Building Code (IBC) 2021, 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 2021 Section 1613.2.2 refers to Chapter 20, Site Classification Procedure for Seismic Design, of ASCE[5] 7-16, which stipulates that the average values of shear wave velocity, blow count and/or shear strength within the upper 100 feet (30 meters) be utilized to determine seismic site class.

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 2021 Table R301.2.2.1.1) are based upon the Site Class discussed in the previous section. For Site Class C at site grid coordinates of 41.3777 degrees north latitude and 111.78568 degrees west longitude, $\mathbf{S_{DS}}$ is 0.685 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 section of the WFZ 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 build up 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

[5]American Society of Civil Engineers

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, all four test pits exposed mass wasting colluvium likely from a mix of slope creep and upslope slumps and slides. The colluvium overlies weathered quartzite bedrock, though thickness of the colluvial veneer is unconfirmed. Slopes at the site are also moderate to steep and may be subject to shallow surficial failures involving the colluvial veneer. Slope stability is discussed in **Section 5.0**.

4.6 Other Geologic Hazards

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

4.6.1 Sloping Surfaces

A slope analysis based on geoprocessed LiDAR terrain data is presented on **Figure 5** that shows areas where slopes are less than 15 percent (unshaded), between 15and 30 percent (shaded in yellow), and steeper than 30 percent (shaded in red). Based on **Figure 5**, slopes at the site are mainly moderate to steep and dip to the southeast at an overall roughly 4.5:1 (horizontal:vertical; or 22.2%, 12.5 degrees). An area of gentle slopes is in the southeast part of the site associated with a cut and fill for the access road to Sundown Condominiums Phases 1 and 2 further east.

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. 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. Evaluation of the need for mitigation of alluvial-fan flooding is a planning decision that weighs the existing and future hazard potential against what will be at risk and level of exposure. Both active and passive measures are typically employed to mitigate risk. Active measures (such as debris basins) are considered optimal to attenuate flows, but such strategies are typically deployed to protect subdivision-scale developments and are not always feasible. Passive measures (such as berms and routing channels) may be deployed for smaller-scale developments, but are not always effective and tend to increase risk to adjacent properties.

The site is not in a mapped active alluvial fan, and no evidence for characteristic debris flow landforms was observed during our reconnaissance. No deposits suggestive of Holocene debris flow deposition were also observed in the test pits at the property. Based on all the above, debris flows and floods are not considered to pose a risk to the proposed development.

4.6.3 Stream Flooding Hazards

No active drainages were observed crossing the property and 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 LABORATORY TESTING

5.1 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. Atterberg Limits, ASTM D-4318, Plasticity and workability
3. Gradation Analysis, ASTM D-1140/C-117, Grain Size Analysis
4. Direct Shear Test, ASTM D-3080, Shear strength parameters

5.2 Lab Summary

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

LAB SUMMARY TABLE

TP No	Depth (feet)	Soil Class	Sample Type	Moisture Content (%)	Dry Denstiy (pcf)	Gradation			Atterberg Limits		
						Grav	Sand	Fines	LL	PL	PI
TP-1	2	SM-GM	Bag	13		29	40	31	15	13	2
	6	SM	Bag			17	57	26			
TP-2	4	SC	bag	19		2	43	55			
	8	SC-CL	Bag	20		5	35	60	33	25	8
TP-4	6	SM-SC	Bag	10							
	7	ML	Bag	11		0	33	67	26	22	4
TP-5	2.5	SM-GM	Bag	4		38	45	17			

5.3 Direct Shear Test

To determine the shear strength of the surficial colluvium at the site, a laboratory direct shear test was performed on each of two representative samples 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-1	2	SM-GM	173	34
TP-2	9	CL-SC	270	33.8

6.0 SLOPE STABILITY

6.1 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[6], and our experience with similar soils. Accordingly, we estimated the following parameters for use in the stability analyses:

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

Material	Internal Friction Angle (degrees)	Apparent Cohesion (psf)	Unit Weight (pcf)
Mass Wasting Colluvium	33.8	173	120
Cambrian Quartzite W Pebble Conglomerate (Bedrock)	25	5000	155
Concrete	0	7000 (49psi)	145

The stability analyses provided are based on **Figure 6A, Cross Section A-A'** and represent projected final grading as provided by Fawkes Consultants Inc (project Civil Engineer). If/where grade changes are made, CMT must review final grading plans. Further, bedrock was not encountered within the upper about 11 feet explored but is assumed to be shallow based on adjacent explorations completed for Phase 2 and was assumed in the slope modeling. CMT must be allowed to review the construction excavations to confirm the conditions assumed. Where bedrock is anticipated within the deeper cuts, heavy equipment and possible blasting could become necessary to complete the excavations.

The pseudostatic coefficient for the seismic analyses was obtained by taking half of the modified peak ground acceleration adjusted for site class C (0.459g) queried for the site which resulted in a value of 0.2295g.

6.2 Stability Analyses

We evaluated the global stability of the cross-sections A-A' located as shown on **Figure 4 Site Evaluation**. The analysis was completed using the computer program *SLIDE2 (version 9.0)*. This program uses a limit equilibrium (Simplified Bishop) method for calculating factors of safety against sliding on an assumed failure surface and evaluates numerous potential failure surfaces, with the most critical failure surface identified as the one yielding the lowest factor of safety of those evaluated. Typically, the required minimum factors of safety are 1.5 for static conditions and 1.0 for seismic (pseudostatic) conditions.

A projected perched water (phreatic) surface was incorporated at 2 to 5 feet in the model, based on a roughly 3 to 4-foot-thick zone of groundwater seepage from seasonal snowmelt observed in the test pit exposures at a depth of around 2 feet. This seasonal perched groundwater layer likely dries up in the summer and fall. No additional site-specific groundwater information was found available, but the Utah Division of Water Rights Well Driller Database shows one water well about 1,950 feet south-southeast of the site that has a reported static groundwater depth of 254 feet bgs.

A roughly 10-inch concrete wall was initially utilized at the proposed wall cut sections to provided an initial stability model.

- Cross-section A-A' (**Figure 6**) consists of a roughly 538-foot-long horizontal cross section with an overall elevation change of about 120 feet with a moderate to steep slope and dip to the southeast at an overall roughly 4.5:1 (horizontal:vertical; or 22.2%, 12.5 degrees) excluding building construction cuts which will require retaining structures. Based on the slope stability analysis, the overall slope generally has factors of safety for both static and pseudo-static (earthquake) conditions in excess of those typically considered acceptable *with the exception of the cut sections downslope of the proposed structures* (See

Figures 9A and 9B Stability Results). *Planned retaining walls and foundation walls will require further design to resist these lateral loads.* The failure surfaces with the lowest factors of safety are shown on the stability analysis plot, with the lowest calculated factor of safety displayed (1.057-Static: .921-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 and must incorporate drains to reduce hydrostatic pressure buildup. Final proposed grading, if different than currently understood, 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 unbraced 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) without retaining structures. 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.

6.3 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.

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 and pavements. 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 6 inches in thickness. Trees and/or brush with large root mats will required deeper removal depth.

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

Where bedrock is anticipated to be relatively shallow within the deeper cuts (anticipated to be as much as about 30 feet, heavy equipment and possible blasting could become necessary to complete the excavations.

The site should be examined by a CMT geotechnical engineer/geologist to assess site stripping and 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 shown on current civil plans could be as much as about 30 feet. Perched, believed to be seasonal, seepage water conditions was observed at about 2 feet below the ground surface at the test pit locations and was roughly about 2 to 4 feet which is assumed to be following snow melt or other similar conditions.

Bedrock was not encountered within the test pits which extended to depths of 9 to 12.5 feet but, based on an earlier study for the existing construction it may be relatively shallow in varying location and would require heavy equipment, chipping and possible blasting to remove bedrock.

The natural surficial colluvium soils encountered at this site predominantly consisted of silty/clayey sand and gravel as well as some layers of sandy silt. 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).

With some larger planned cuts, shoring/bracing must be anticipated.

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[7] T-180) in accordance with the following recommendations:

LOCATION	TOTAL FILL THICKNESS (FEET)	MINIMUM PERCENTAGE OF MAXIMUM DRY DENSITY
Beneath an area extending at least 4 feet beyond the perimeter of structures, and below flatwork and pavement (applies to structural fill and site grading fill) extending at least 2 feet beyond the perimeter	0 to 5 5 to 10	95 98
Site grading fill outside area defined above	0 to 5 5 to 10	92 95
Utility trenches within structural areas	--	96

[7] 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 about 10 feet thick are not anticipated at the site. For best compaction results, we recommend that the moisture content for structural fill/backfill be within 2% of optimum. Field density tests should be performed on each lift as necessary to verify that proper compaction is being achieved.

7.5 Utility Trenches

For the bedding zone around the utility, we recommend utilizing sand bedding fill material that meets current APWA[8] 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 7.4**.

7.6 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. This coarse material may be placed and worked into the soft soils until firm and non-yielding, or the soft soils may be removed an additional 18 inches (minimum) and backfilled with clean stabilizing angular gravels/cobbles. A test area should be implemented to achieve a proper stabilization strategy. 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] American Public Works Association

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.

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 soils or structural fill extending to suitable natural soils. Footings may then be designed using a net bearing pressure of 2,500 psf. The term “net bearing pressure” refers to the pressure imposed by the portion of the structure located above lowest adjacent final grade, thus the weight of the footing and backfill to lowest adjacent final grade need not be considered. The allowable bearing pressure may be increased by 1/3 for temporary loads such as wind and seismic forces.

We 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 12 inches below grade.
3. Continuous footing widths should be maintained at a minimum of 18 inches.
4. Spot footings should be a minimum of 24 inches wide.

8.2 Installation

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

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

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

Further, where the structural fill thickness will be greater than 3 feet, the minimum thickness of structural fill below footings should be equivalent to one-third the thickness of structural fill below any other portion of the foundations. For example, if the maximum depth of structural fill is 6 feet, all footings for the new structure should be underlain by a minimum 2 feet of structural fill.

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 natural sand/gravel soils and select 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 400 pcf. A combination of passive earth resistance and friction may be utilized if the passive resistance component of the total is divided by 1.5. Note that frictional resistance is mobilized as soon as any movement occurs, while full passive pressure is typically achieved after a small amount of movement occurs (approximately 0.5% of the footing height).

9.0 LATERAL EARTH PRESSURES

The lateral earth pressure values given below are for a backfill material that will consist of drained sand/gravel soils placed and compacted in accordance with the recommendations presented herein. If other soil types will be used as backfill, we should be notified so that appropriate modifications to these values can be provided, as needed.

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.

Where proposed wall are less than 12 feet high, employing a seismic at-rest lateral earth pressure for design is not needed.

CONDITION	STATIC (psf/ft)*	SEISMIC (psf/ft)**
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)	37	17
At-Rest Pressure (wall is not allowed to yield)	57	17
Passive Pressure (wall moves into the soil)	400	210

*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

Properly engineered floor slabs should be established upon uniform, compacted bearing soils comprised of suitable, undisturbed uniform natural soils or on structural fill extending to suitable natural soils (similar to Under no circumstances shall floor slabs be established directly on any topsoil, undocumented fills, loose or disturbed soils, sod, rubbish, construction debris, other deleterious materials, frozen soils, or within ponded water. Floor slabs should be properly designed by a structural engineer to accommodate anticipated loads.

To facilitate curing of the concrete, we recommend that floor slabs be directly underlain by at least 4 inches of moist aggregate base or bedding material, or "free-draining" fill such as "pea" gravel or 3/4-inch to 1-inch minus, clean, gap-graded gravel. To help control normal shrinkage and stress cracking, the floor slab thickness and joint layout should be designed by a qualified engineer. Design provisions should address the following items:

1. Adequate reinforcement for the anticipated floor loads;
2. Using smooth bar reinforcement for load transfer through interior floor joints;
3. Portland cement concrete mix design selection to minimize shrinkage concerns;
4. Joint layout and spacing in accordance with ACI[9] or other local standards; and
5. Properly isolate floor slabs from foundations and other structural elements per recommendations provided by ACI 302 (Guide to Concrete Floor and Slab Construction).

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:

[9] American Concrete Institute

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 building 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 structure.

12.0 PAVEMENTS

All pavement areas must be prepared as discussed above in **Section 7.1**. Under no circumstances shall pavements be established over topsoil, unprepared fills (if encountered), loose or disturbed soils, sod, rubbish, construction debris, other deleterious materials, frozen soils, or within ponded water. We anticipate traffic to consist of primarily light automobiles and light trucks with a weekly or by weekly garbage truck and occasional emergency vehicle. Current plans are to install a private pavement section comprised of:

MATERIAL	PAVEMENT SECTION THICKNESS (inches)
Asphalt	**3**
Road-Base	**6**
Subbase	**8**
Total Thickness	**16**

This proposed section is suitable for the anticipated traffic.

Untreated base course (UTBC), typically known as road-base, should conform to APWA specifications, or to 1.5-inch-minus UDOT specifications for A–1-a/NP, and have a minimum CBR value of 70%. Subbase should be a granular material having a minimum CBR value of 40%. Road base and subbase material should be compacted as recommended above in **Section 7.4**. Asphalt material generally should conform to APWA requirements, having a ½-inch maximum aggregate size, a 75-gyration Superpave mix containing no more than 15% of recycled asphalt (RAP) and a PG58-28 binder.

For dumpster pads, we recommend a pavement section consisting of 6.5 inches of Portland cement concrete and 6 inches of aggregate base over properly prepared suitable natural subgrade or site grading structural fills extending to suitable natural soils. Dumpster pads constructed overlying undocumented fills must be avoided or heavily reinforced.

12.0 QUALITY CONTROL

We recommend that CMT be retained 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 this project, please do not hesitate to contact us at (801) 590-0394. To schedule materials testing, please call (801) 381-5141.

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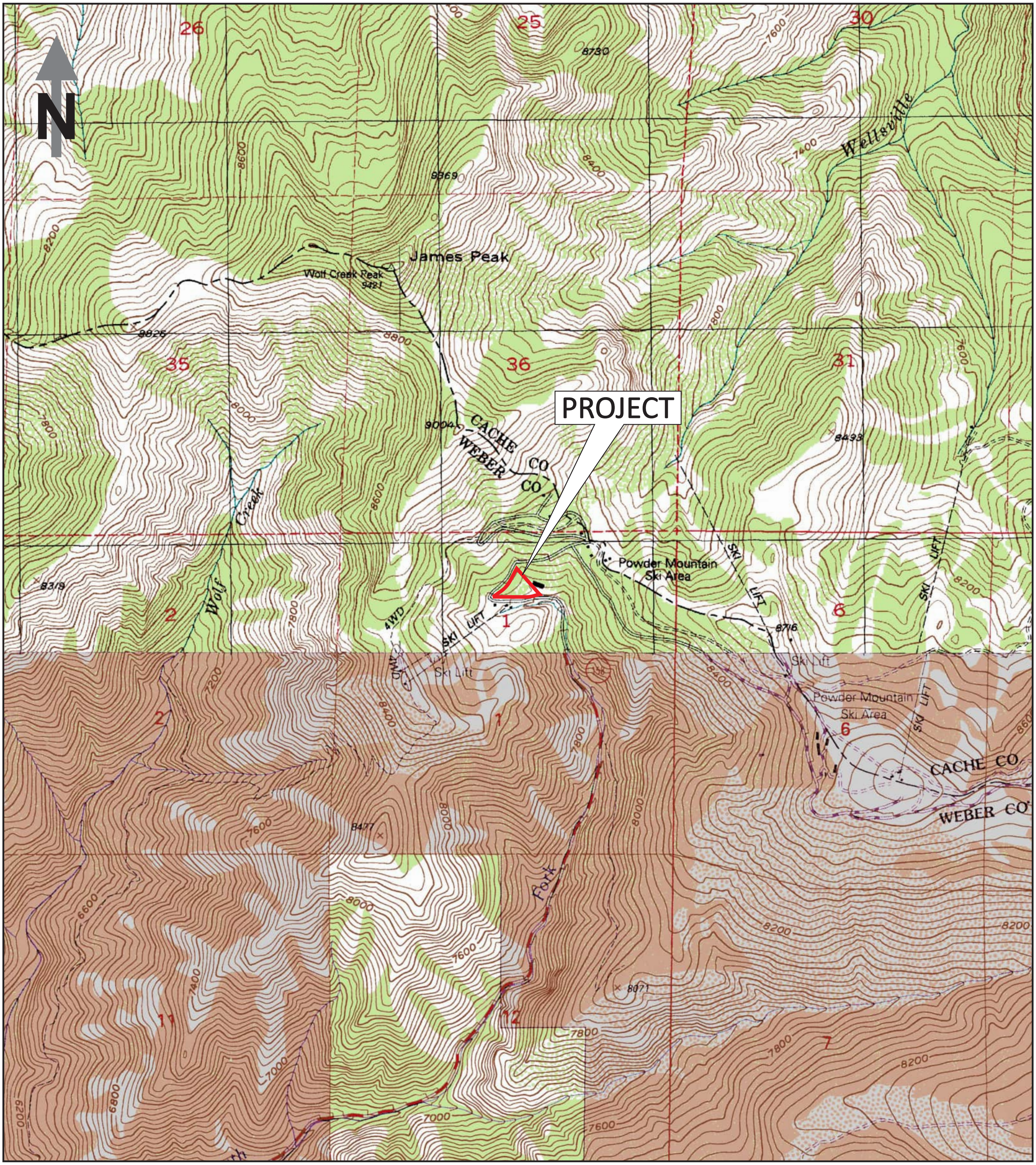
APPENDIX

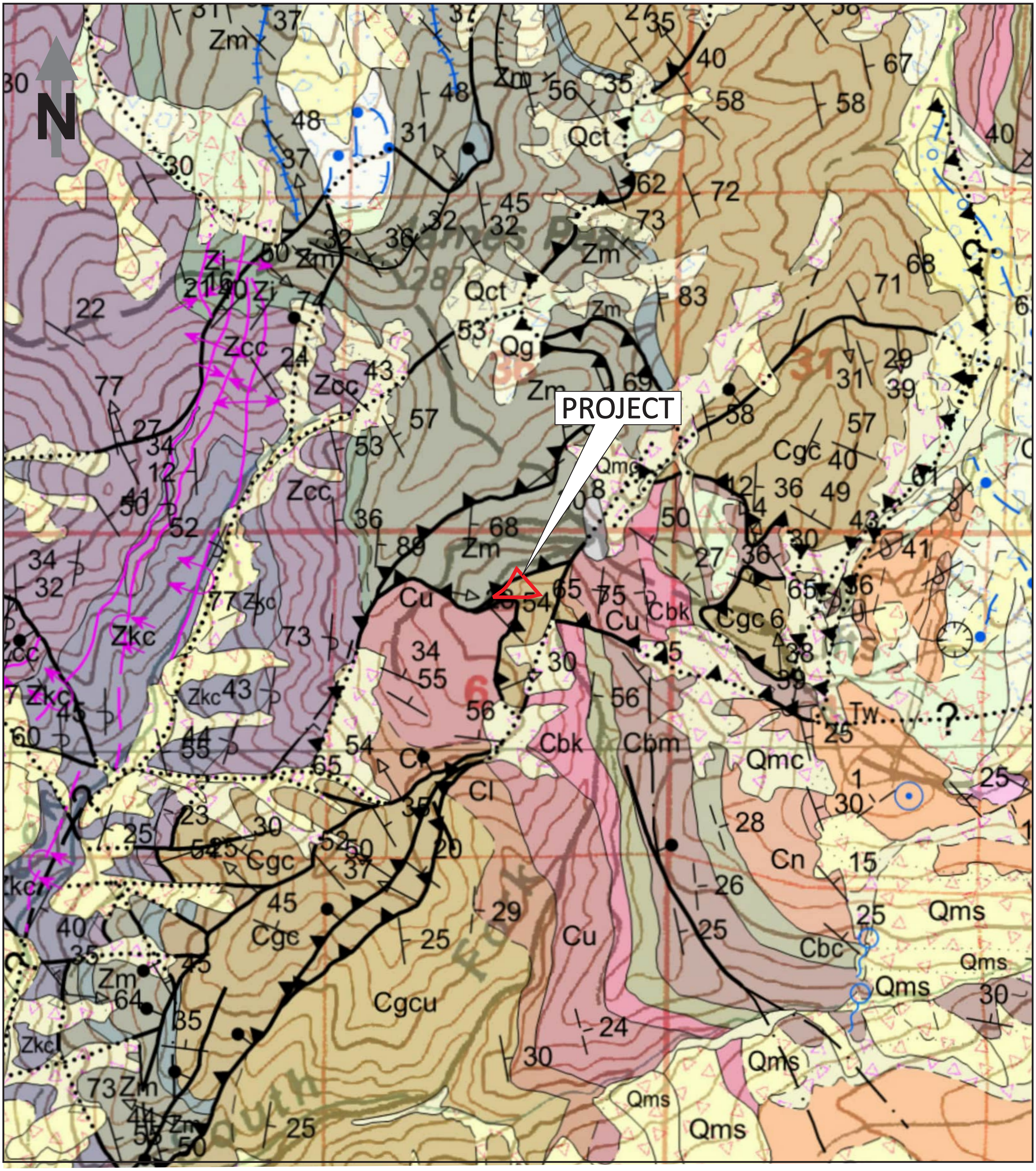
SUPPORTING
DOCUMENTATION



Base:
U.S.G.S. 7.5-Minute Series Topographic Maps, Utah - James Peak, 1991 and Huntsville, 1998;
Project location N1/2, Section 1, Township 7 North, Range 1 East (SLBM).

0 1000 ft 2000 ft
1:24,000 (1 inch equals 2000 feet)

Sundown Condominiums Phase 3 About 6550 North Powder Mountain Road Eden, Utah	CMT TECHNICAL SERVICES		Figure
	Location Map	Date: 17-Jun-2025 Job # 24298	1

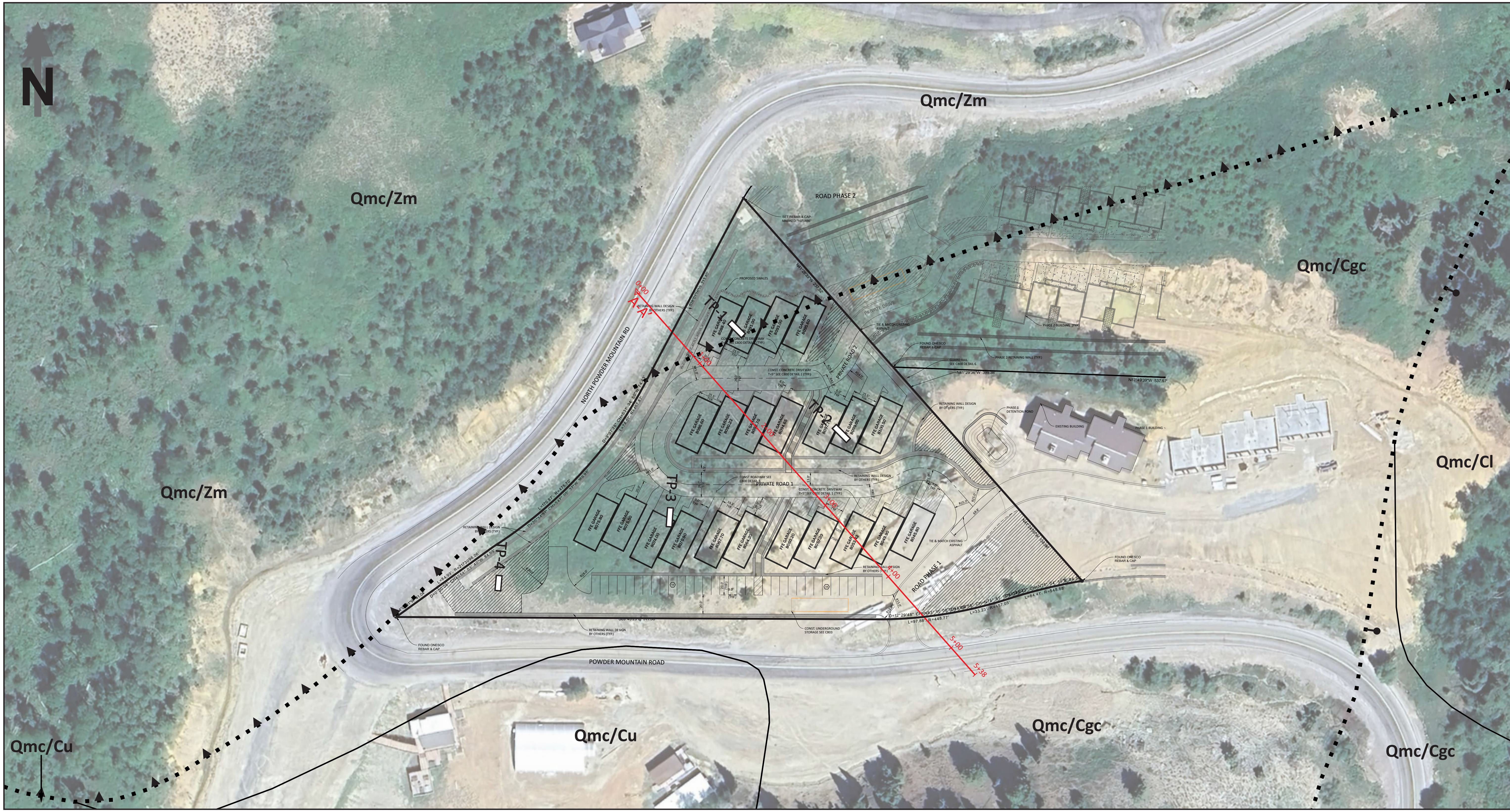


Base:
Coogan and King (2016). See report for descriptions of nearby surficial geologic units.

0 1000 ft 2000 ft

1:24,000 (1 inch equals 2000 feet)

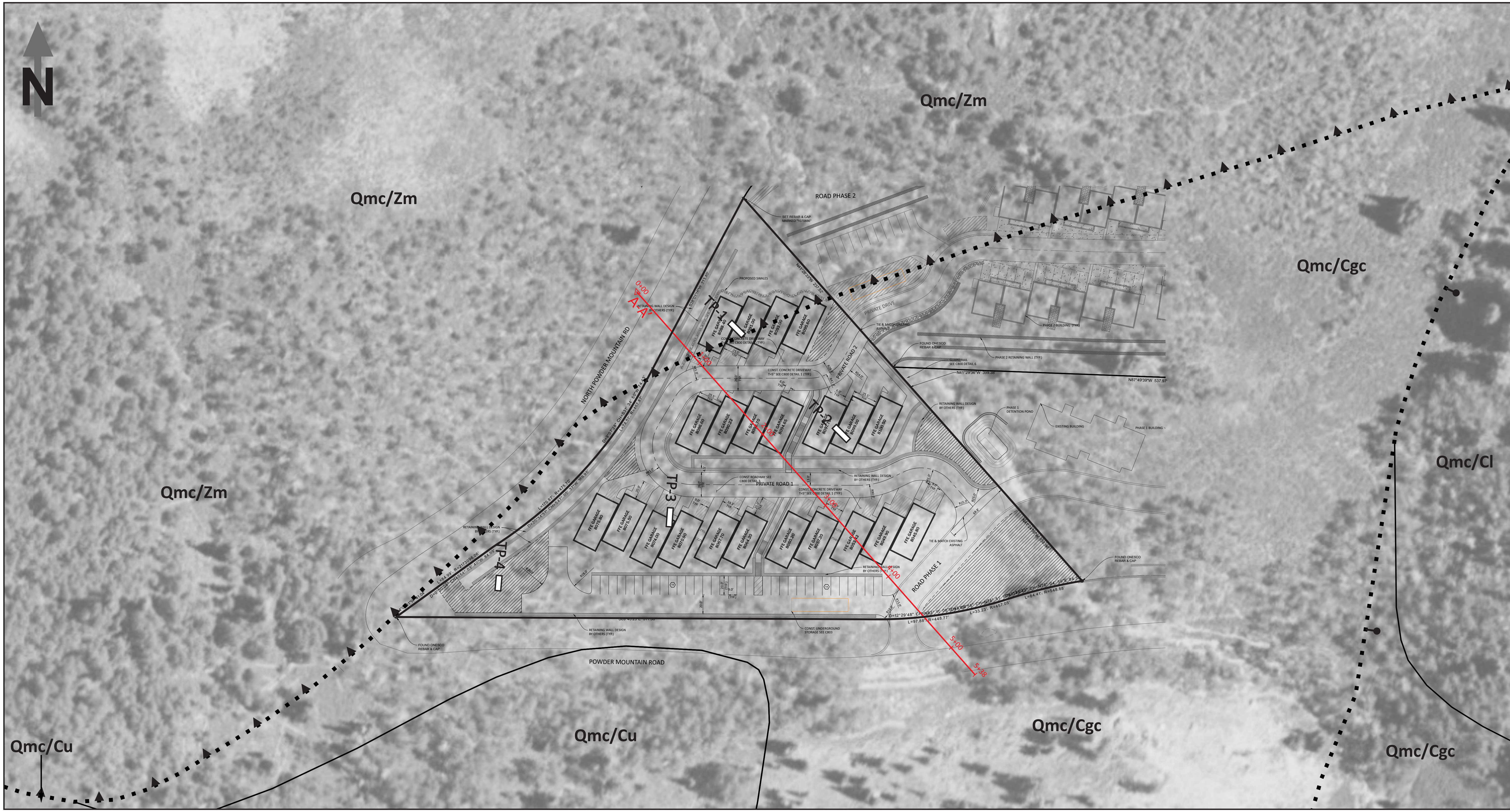
Sundown Condominiums Phase 3 About 6550 North Powder Mountain Road Eden, Utah	CMT TECHNICAL SERVICES		Figure 2
	Geologic Map	Date: 17-Jun-2025 Job #: 24298	



Base:
Google Earth, June 2024. Site plan from Fawkes Consultants Preliminary Design Site Plan sheet C200 dated May 15, 2025.

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

CMT TECHNICAL SERVICES	Sundown Condominiums Phase 3 About 6550 North Powder Mountain Road, Eden, Utah	Site Evaluation	Date:	17-Jun-2025	Figure 3
			CMT No.:	24298	

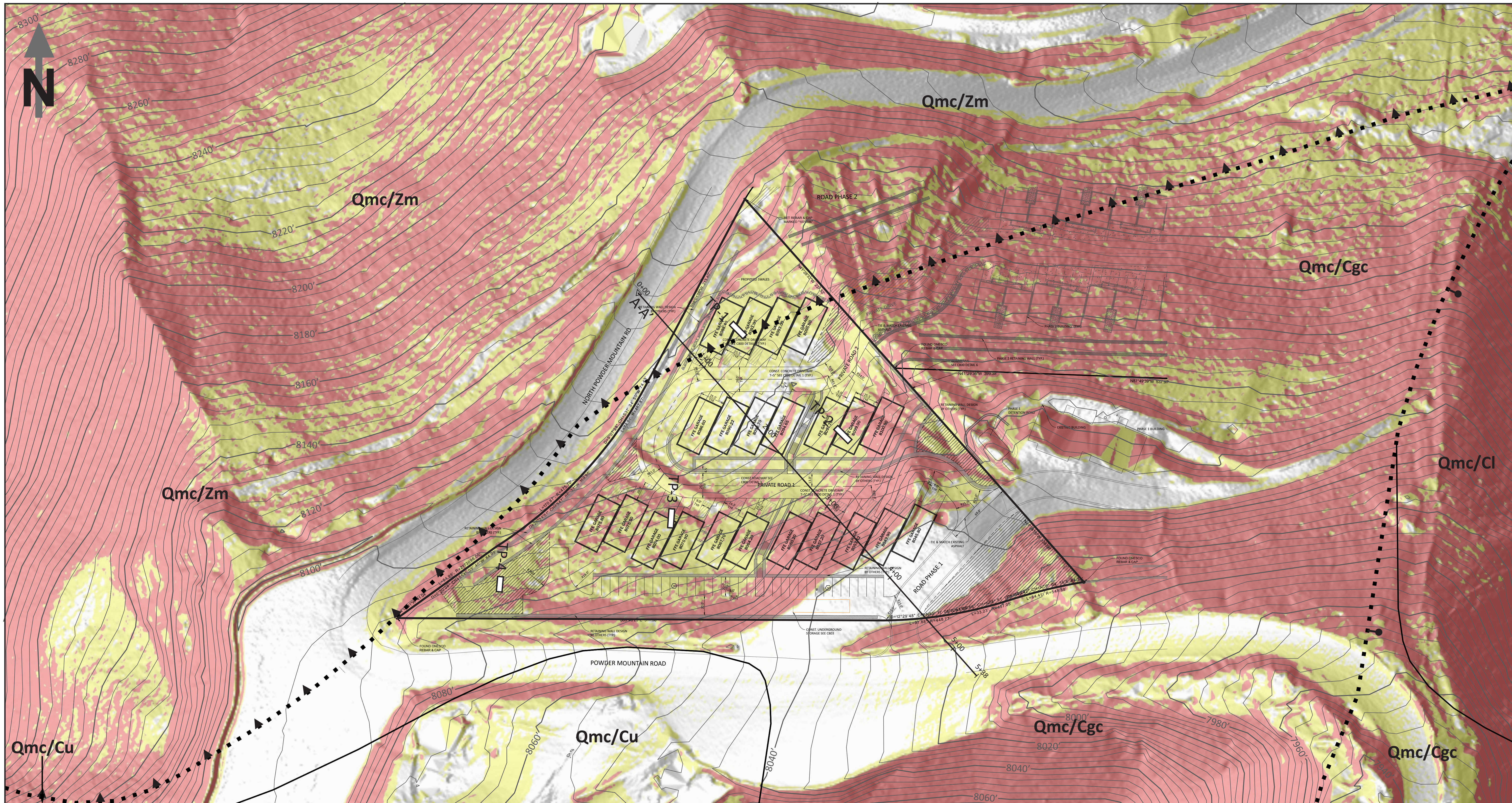


Base:
U.S. Department of Agriculture, frames AAJ 26V-74 and AAJ 26V-75, dated June 17, 1958, original scale 1:20,000.

0 100 ft 200 ft

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

CMT TECHNICAL SERVICES	Sundown Condominiums Phase 3 About 6550 North Powder Mountain Road, Eden, Utah	1958 Air Photo	Date:	17-Jun-2025	Figure 4
			CMT No.:	24298	

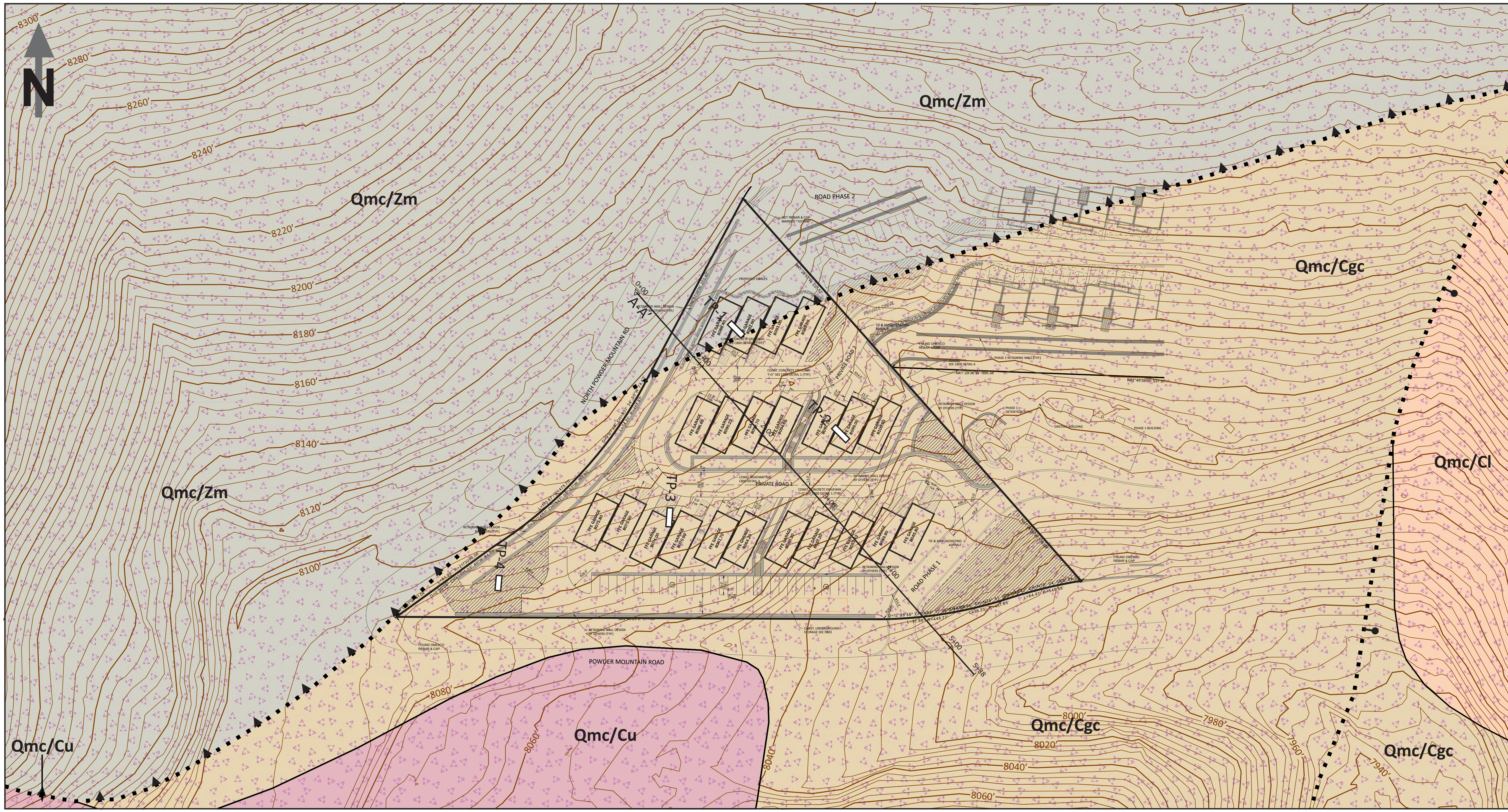


Base:
Utah Geospatial Resource Center 2016 LIDAR bare earth DEM, 50 cm resolution. Slopes <15% unshaded, 15-30% shaded in yellow, and >30% shaded in red. Contours at 4 foot intervals.

0 100 ft 200 ft

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

CMT TECHNICAL SERVICES	Sundown Condominiums Phase 3 About 6550 North Powder Mountain Road, Eden, Utah	Slope Analysis	Date: 17-Jun-2025 CMT No.: 24298	Figure 5



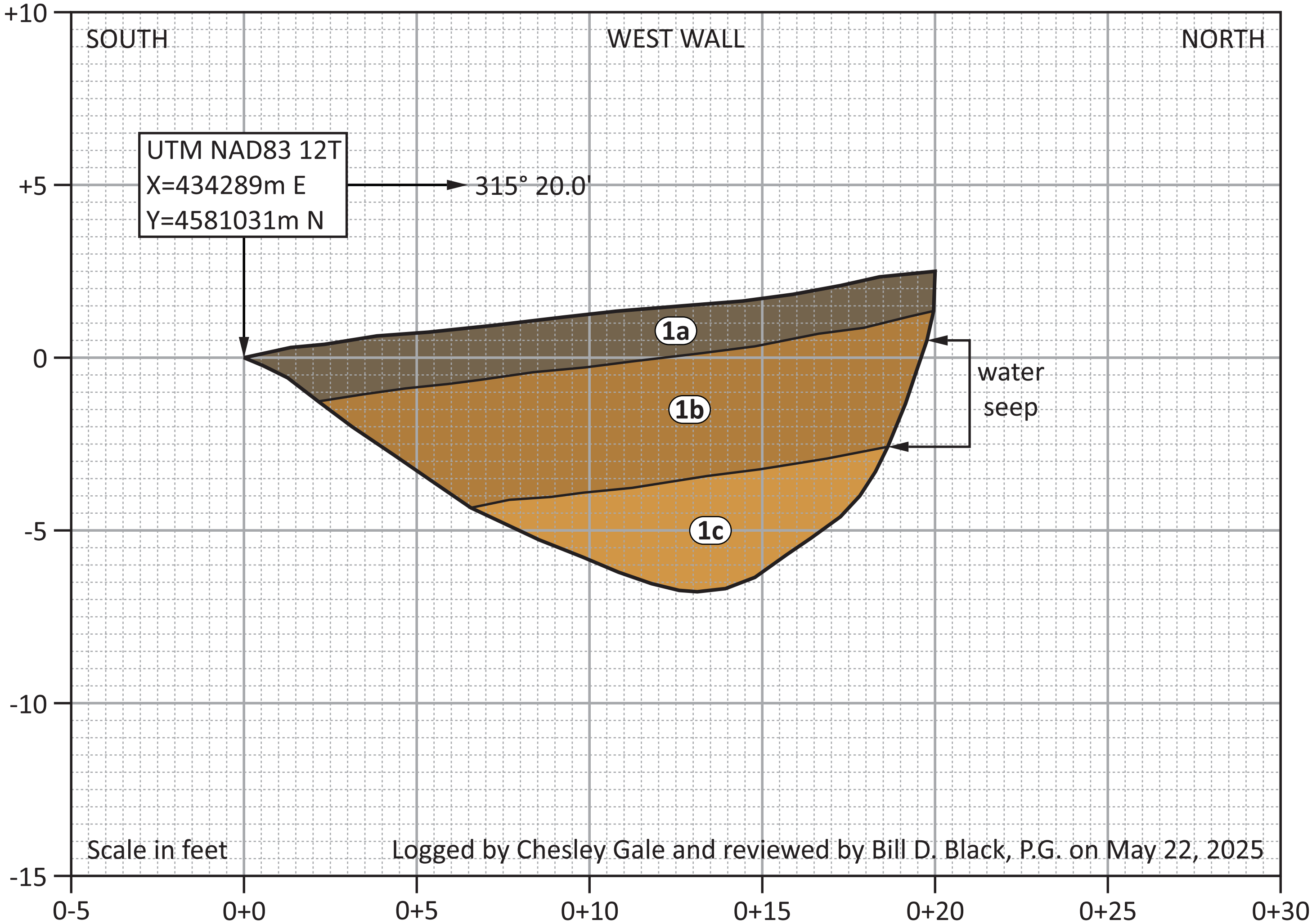
Source:
Modified from Coogan and King (2016). Contours at 4 foot intervals. Site plan from Fawkes Consultants Preliminary Design Site Plan sheet C200 dated May 15, 2025.

0 100 ft 200 ft

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

CMT TECHNICAL SERVICES	Sundown Condominiums Phase 3 About 6550 North Powder Mountain Road, Eden, Utah	Site-Specific Geology	Date:	17-Jun-2025	Figure 6
			CMT No.:	24298	

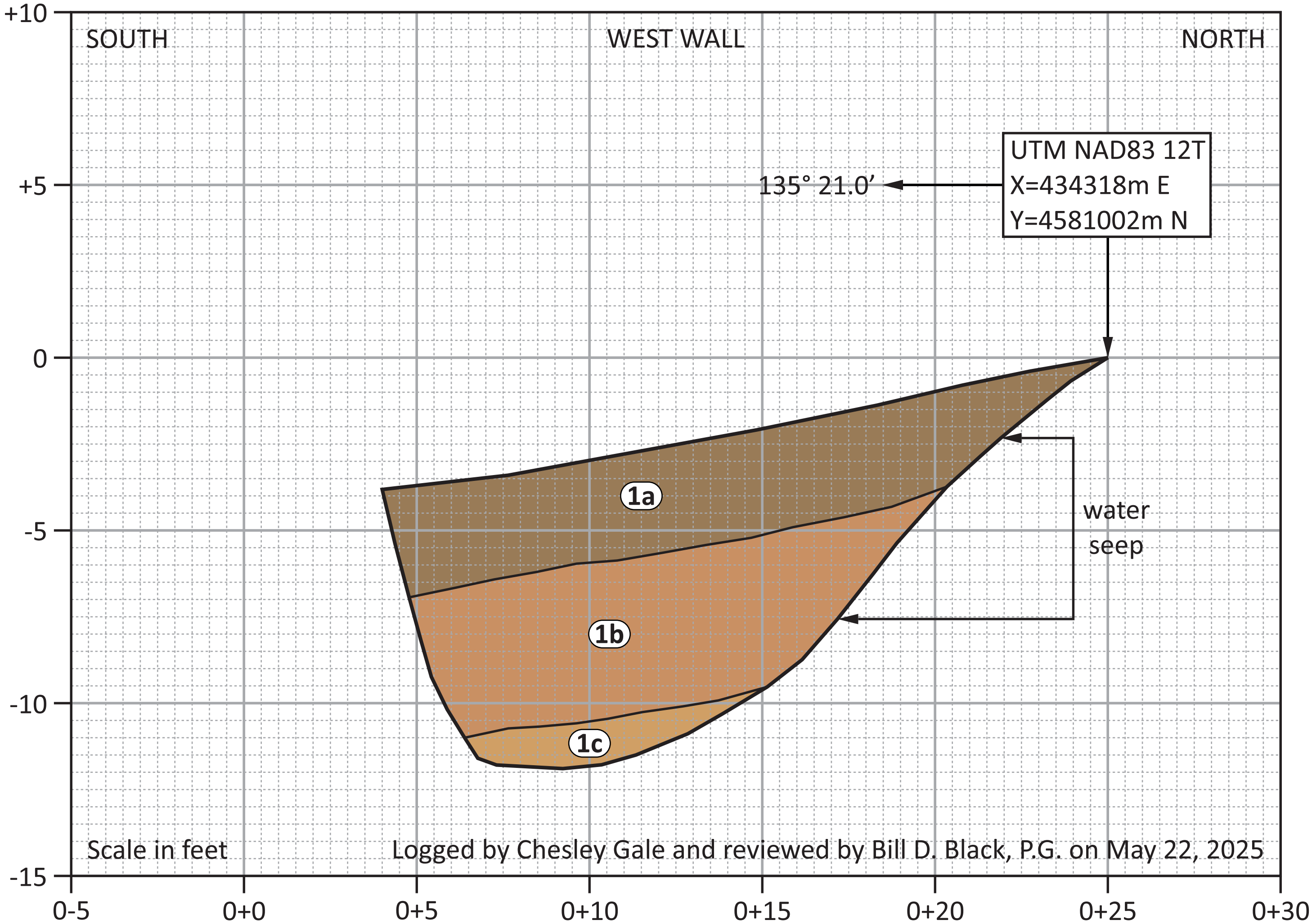
TEST PIT 1



UNIT DESCRIPTIONS

Unit 1. *Upper Pleistocene to Holocene mass wasting colluvium* - sequence comprised of an upper (**1a**) dark brown, moist, medium stiff, silt (ML) with gravel, subangular to angular cobbles and roots; a middle (**1b**) red brown, very moist to wet, medium dense, massive, silty sand and gravel (SM-GM) with subround to subangular cobbles; and a lower (**1c**) light red tan, slightly moist, very dense, massive, silty sand and gravel (SM-GM) with subangular to subround cobbles.

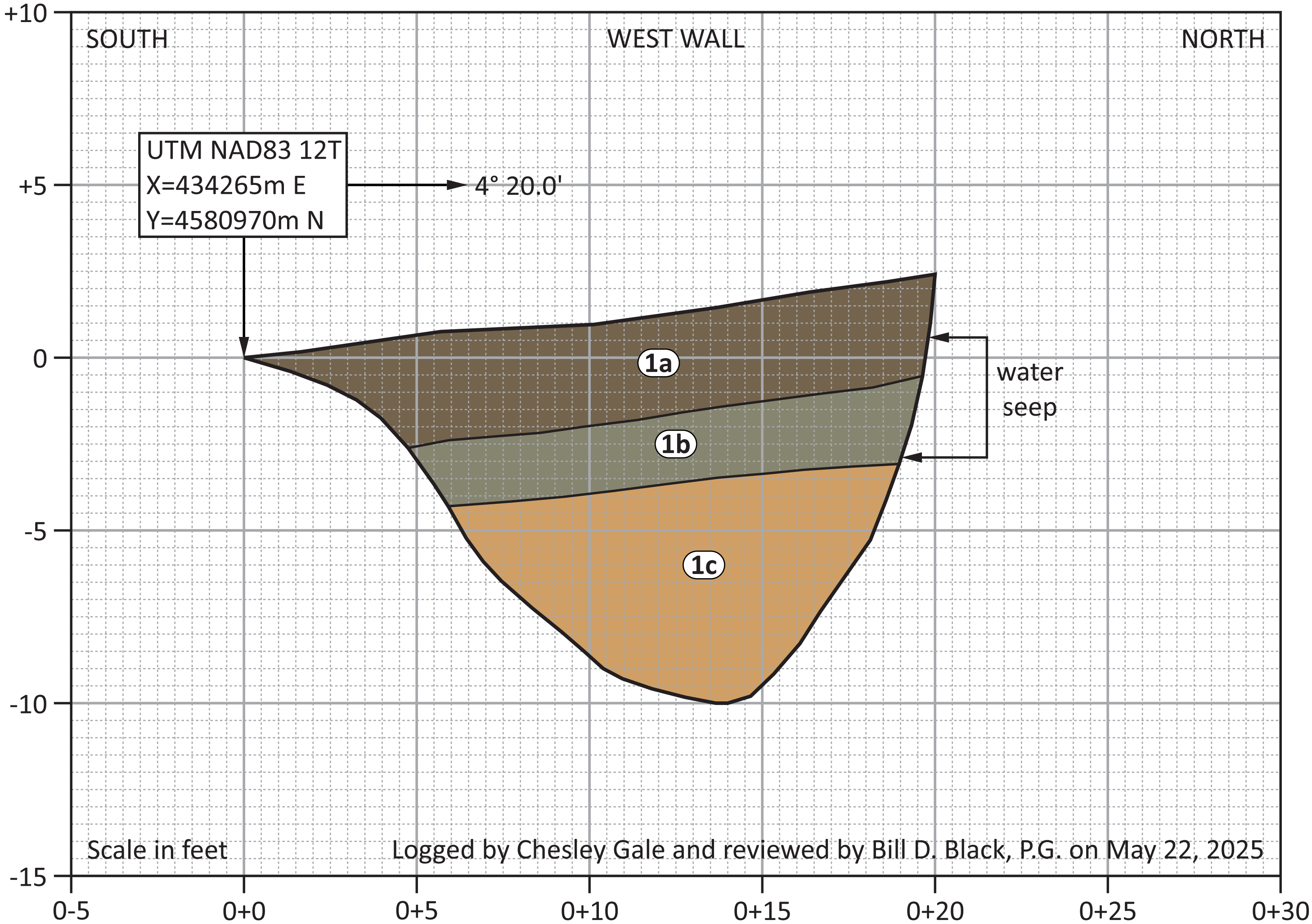
TEST PIT 2



UNIT DESCRIPTIONS

Unit 1. *Upper Pleistocene to Holocene mass wasting colluvium* - sequence comprised of an upper (**1a**) light red brown, moist, medium dense to loose, massive, silty sandy gravel (GP-GM) with cobbles and boulders; a middle (**1b**) red brown, very moist to wet, medium dense to loose, massive, clayey sand (SC) with cobbles and some gravel; and a lower (**1c**) poorly bedded clayey sand (SC) similar to **1b**, but with alternating 6-inch thick layers of stiff clay and sand.

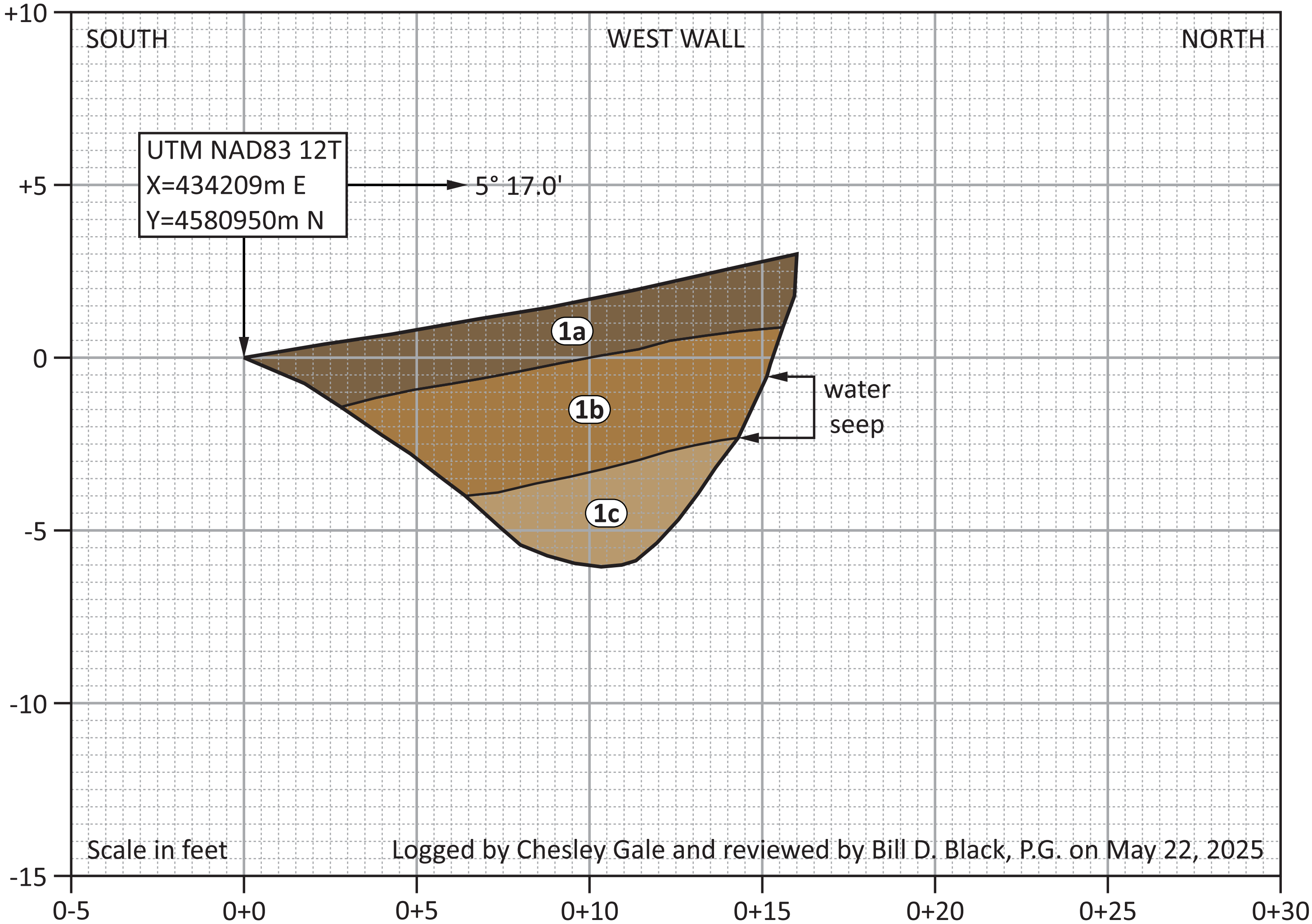
TEST PIT 3



UNIT DESCRIPTIONS

Unit 1. *Upper Pleistocene to Holocene mass wasting colluvium* - sequence comprised of an upper (**1a**) dark brown, moist, soft to medium stiff, massive, sandy silt (ML) with some gravel and trace boulders; a middle (**1b**) dark gray, very moist to wet, loose to medium dense, massive, silty clayey gravel (GM-GC) with cobbles and sand; and a lower (**1c**) moist, very stiff, poorly bedded, sandy silt (ML) with fine layers of gravel, cobbles and oxidation.

TEST PIT 4



UNIT DESCRIPTIONS

Unit 1. *Upper Pleistocene to Holocene mass wasting colluvium* - sequence comprised of an upper (**1a**) dark brown to brown, moist, medium dense, massive, silty sandy gravel (GM-GP) with cobbles, boulders and roots; a middle (**1b**) red brown, very moist to wet, medium dense, massive, silty sand and gravel (SM-GM) with cobbles; and a lower (**1c**) light red tan to brown, moist, dense, massive, silty sand and gravel (SM-GM) with cobbles.

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Sundown Condos Phase 3

About 6550 North Powder Mountain Road, Eden, Utah

Test Pit Log — TP-1

Total Depth: 9'
Water Depth: (see Remarks)
Date: 5/22/25
Job #: 24298

Depth (ft)	GRAPHIC LOG	Soil Description	Sample Type	Sample #	Moisture (%)	Dry Density(pcf)	Gradation: Gravel %	Gradation: Sand %	Gradation: Fines %	Atterberg: LL	Atterberg: PL	Atterberg: PI
0		Dark Brown SILT (ML) with gravel, cobbles and roots; moist, medium stiff (estimated)										
				1								
2		Red Brown Silty Sand and Gravel (SM-GM) with cobbles; very moist to wet, medium dense (estimated)		2	13		29	40	31	15	13	2
		* * Perched water condition between about 2-5' * *										
5		grades light red tan; slightly moist, very dense (estimated)										
6				3	4		17	57	26			
9		END AT 9'										
10												
11												
12												
13												
14												

Remarks: * * Perched water condition between about 2-5' * *

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

Equipment: Excavator
Excavated By: Blaine Hone
Logged By: Chesley Gale

CMT TECHNICAL SERVICES

Figure: 7E

Sundown Condos Phase 3

About 6550 North Powder Mountain Road, Eden, Utah

Test Pit Log — TP-2

Total Depth: 11'
Water Depth: (see Remarks)

Date: 5/22/25
Job #: 24298

Depth (ft)	Soil Description	Sample Type	Sample #	Moisture (%)	Dry Density(pcf)	Gravel %	Sand %	Fines %	LL	PL	PI
0	Light Red Brown Silty Sandy GRAVEL (GP-GM) with cobbles and boulders — moist, medium dense to loose (estimated)										
1		◣	4								
2	* * Perched water condition between about 2-6' * *										
4	Red Brown Clayey SAND (SC) with cobbles and some gravel — very moist to wet, medium dense to loose (estimated)	◣	5	19		2	43	55			
8		◣	6	20		5	35	60	33	25	8
9	grades with alternating layers of clay and clayey sand with some gravel										
11	END AT 11'										

Remarks: Groundwater not encountered during excavation.

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

Equipment: Excavator
Excavated By: Blaine Hone
Logged By: Chesley Gale

CMT TECHNICAL SERVICES

Figure: 7F

Sundown Condos Phase 3

About 6550 North Powder Mountain Road, Eden, Utah

Test Pit Log — TP-3

Total Depth: 12.5'
Water Depth: (see Remarks)
Date: 5/22/25
Job #: 24298

Depth (ft)	Soil Description	Sample Type	Sample #	Moisture (%)	Dry Density(pcf)	Gravel %	Sand %	Fines %	LL	PL	PI
0–3	Dark Brown Sandy SILT (ML) with some gravel and trace boulders moist, soft to medium stiff (estimated) * * Perched water condition between about 1.8-5.2' * *										
3–7	Dark Gray Silty Clayey GRAVEL (GM-GC) with cobbles and sand very moist to wet, loose to medium dense (estimated)										
7–12.5	Sandy SILT (ML) with fine layers of gravel, cobbles and oxidation moist, very stiff (estimated)										
12.5	END AT 12.5'										

Remarks: * * Perched water condition between about 1.8-5.2' * *

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

Equipment: Excavator
Excavated By: Blaine Hone
Logged By: Chesley Gale

CMT TECHNICAL SERVICES

Figure: 7G

Sundown Condos Phase 3

About 6550 North Powder Mountain Road, Eden, Utah

Test Pit Log — TP-4

Total Depth: 9'
Water Depth: (see Remarks)
Date: 5/22/25
Job #: 24298

Depth (ft)	Soil Description	Sample Type	Sample #	Moisture (%)	Dry Density(pcf)	Gravel %	Sand %	Fines %	LL	PL	PI
0	Dark Brown to Brown Silty Sandy GRAVEL (GM-GP) with cobbles, boulders and roots moist, medium dense (estimated)										
1			7								
2	Red Brown Silty Sand and Gravel (SM-GM) with cobbles very moist to wet, medium dense (estimated)										
3	* * Perched water condition between about 3.4-5' * *		8								
5	grades light red tan to brown moist, dense (estimated)										
6			9	10		0	60	40	28	22	6
9	END AT 9'		10	11		0	33	67	26	22	4

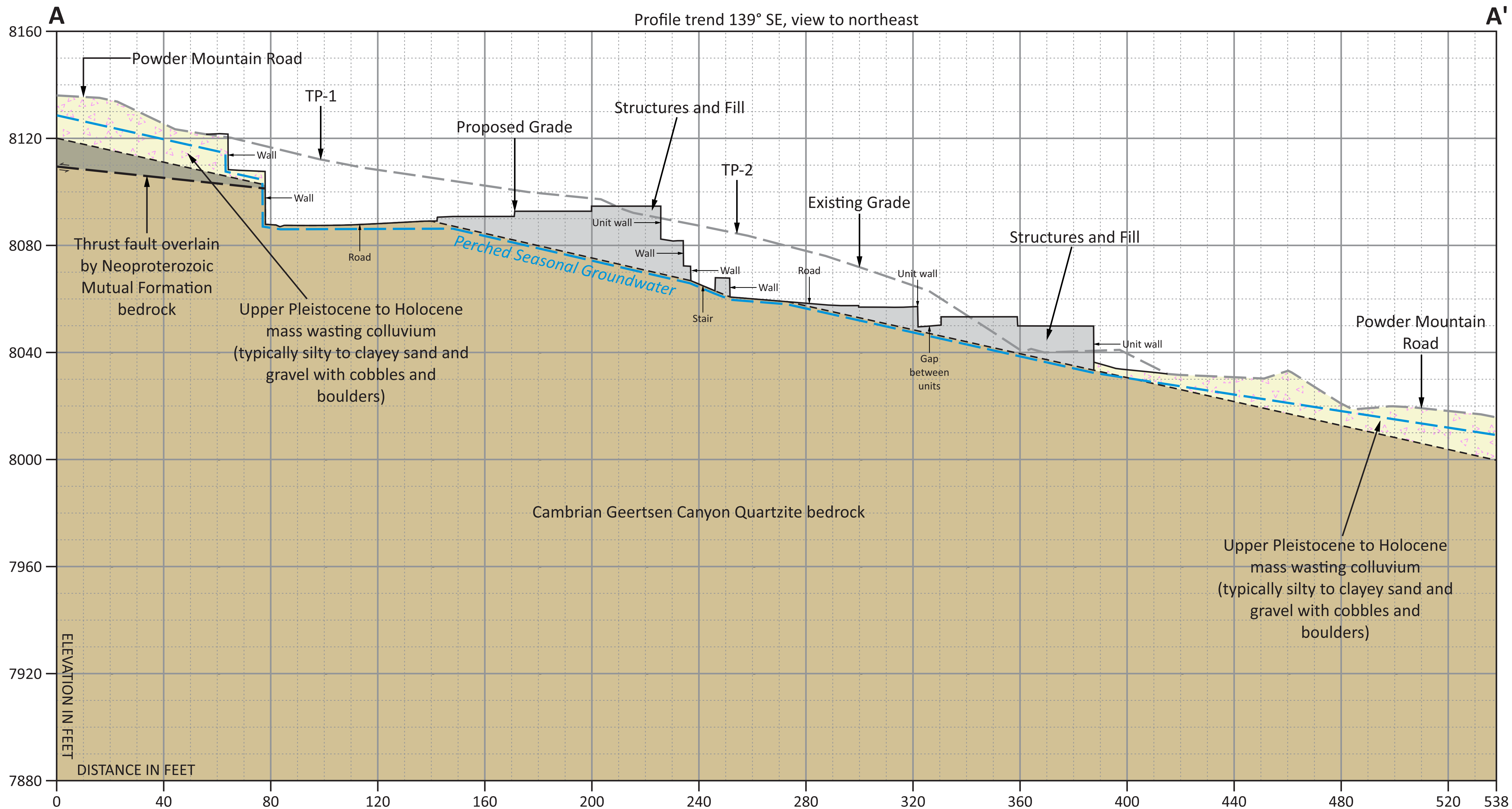
Remarks: * * Perched water condition between about 3.4-5' * *

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

Equipment: Excavator
Excavated By: Blaine Hone
Logged By: Chesley Gale

CMT TECHNICAL SERVICES

Figure: 7H



Scale 1 inch equals 40 feet (1:480) with no vertical exaggeration. Existing and proposed grades based on Fawkes Consultants Inc. exhibit dated 6/10/2025.

CMT TECHNICAL SERVICES	**Sundown Condominiums Phase 3** About 6550 North Powder Mountain Road, Eden, Utah	**Cross Section A-A'**	Date: 17-Jun-2025 CMT No.: 24298	Figure 8

Sundown Condos Phase 3

Key to Symbols

About 6550 North Powder Mountain Road, Eden, Utah

Date: 5/22/25
Job #: 24298

① Depth (ft)	② GRAPHIC LOG	③ Soil Description	④ Sample Type	⑤ Sample #	⑥ Moisture (%)	⑦ Dry Density(pcf)	⑧ Gradation Gravel %	Sand %	Fines %	Atterberg LL	PL	PI

COLUMN DESCRIPTIONS

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

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

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

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

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

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

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.
Wet: 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 Sandy Silts with No Plasticity or Clayey Silts with Slight Plasticity
			CL	Inorganic Clays of Low to Medium Plasticity, Gravelly Clays, Sandy Clays, Silty Clays, Lean Clays
			OL	Organic Silts and Organic Silty Clays of Low Plasticity
	SILTS AND CLAYS Liquid Limit greater than 50%		MH	Inorganic Silts, Micacious or Diatomacious Fine Sand or Silty Soils
			CH	Inorganic Clays of High Plasticity, Fat Clays
			OH	Organic Silts and Organic Clays of Medium to High Plasticity
HIGHLY ORGANIC SOILS			PT	Peat, Soils with High Organic Contents

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

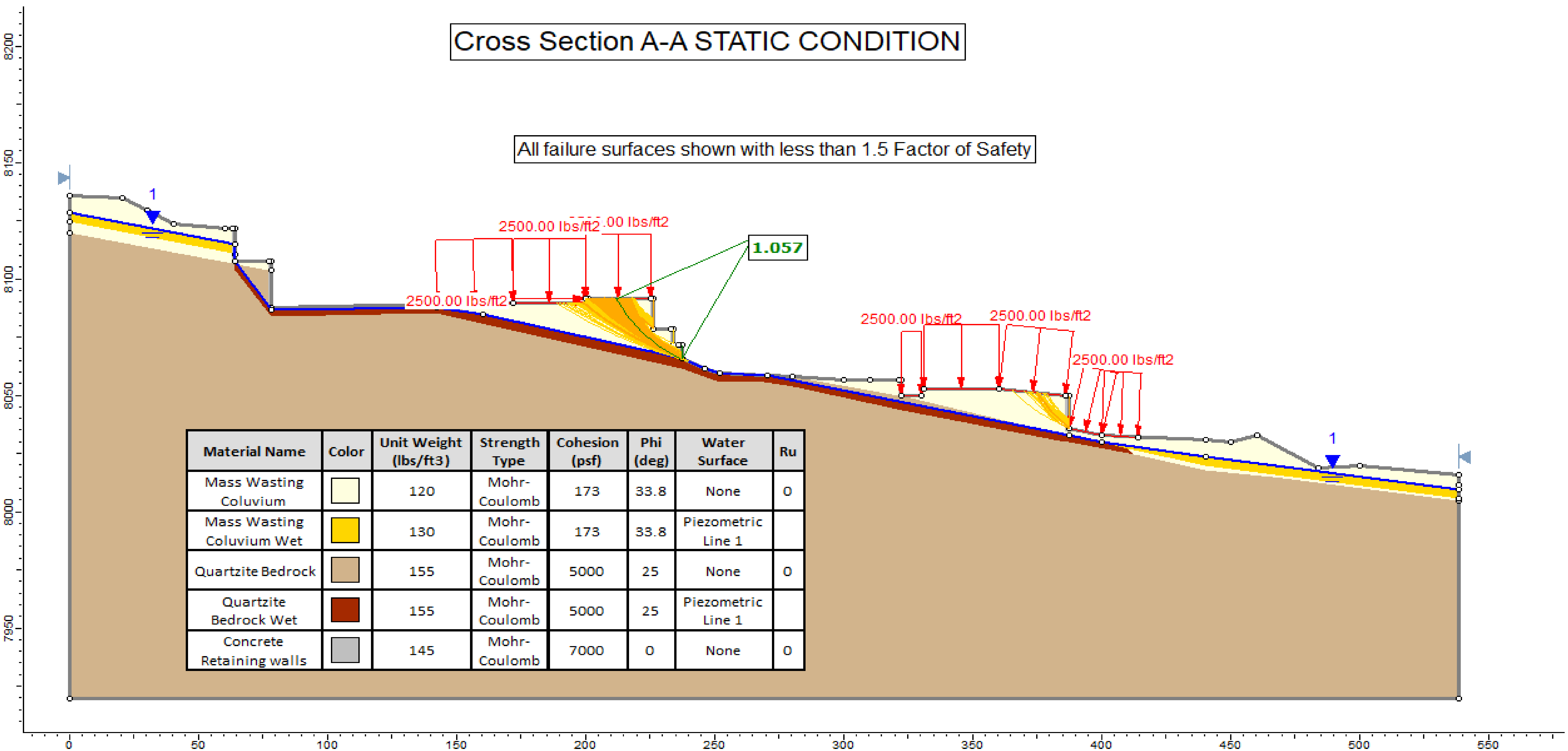
Encountered Water Level
Measured Water Level
(see Remarks on Logs)

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

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 TECHNICAL SERVICES

Figure:
9



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Ru
Mass Wasting Coluvium		120	Mohr-Coulomb	173	33.8	None	0
Mass Wasting Coluvium Wet		130	Mohr-Coulomb	173	33.8	Piezometric Line 1	
Quartzite Bedrock		155	Mohr-Coulomb	5000	25	None	0
Quartzite Bedrock Wet		155	Mohr-Coulomb	5000	25	Piezometric Line 1	
Concrete Retaining walls		145	Mohr-Coulomb	7000	0	None	0

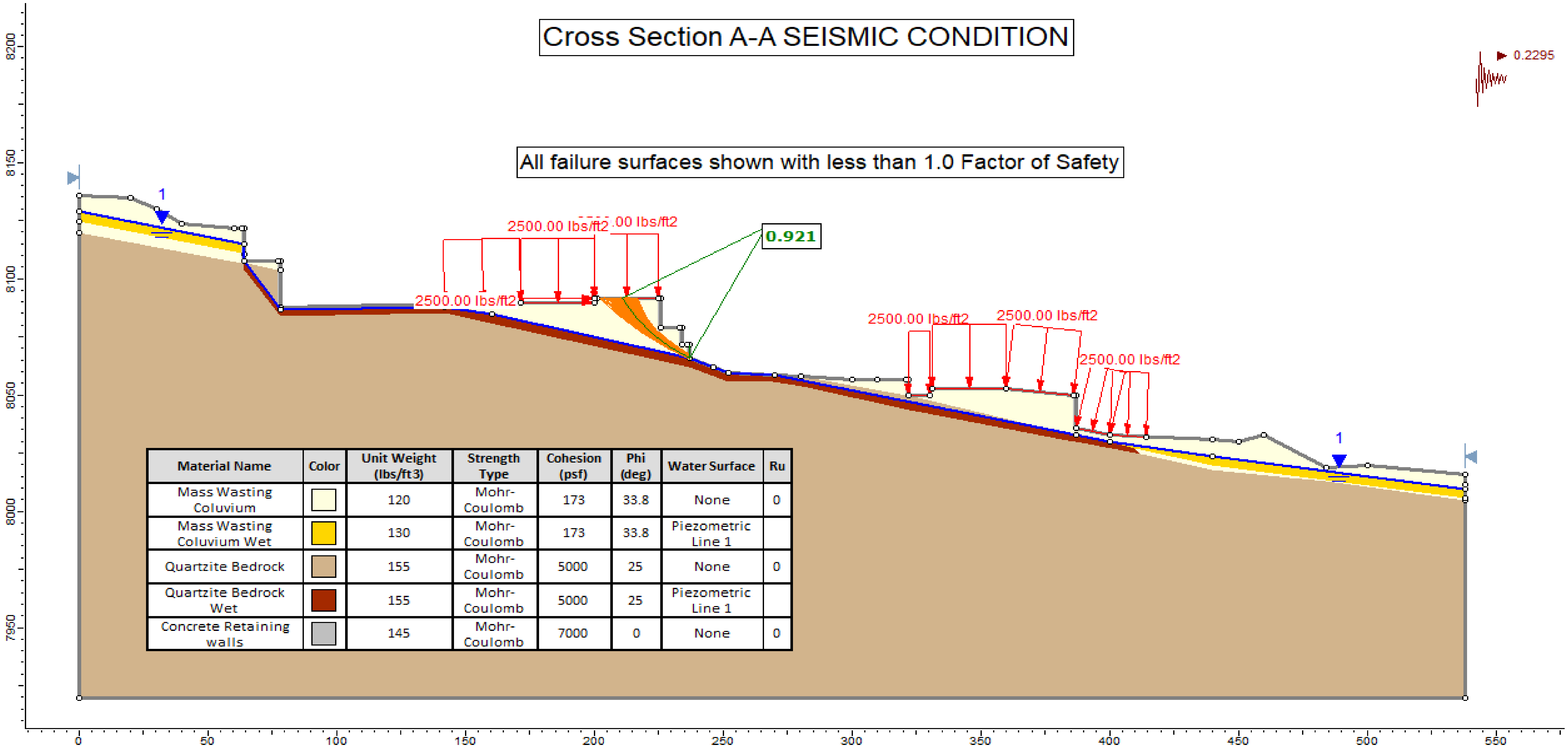
CMT TECHNICAL SERVICES

Sundown Condominiums Phase 3
About 6550 North Powder Mountain Road, Eden, Utah

Static

Date:	23-Jun-2025
CMT No.:	24298

Figure: 10A



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Ru
Mass Wasting Coluvium		120	Mohr-Coulomb	173	33.8	None	0
Mass Wasting Coluvium Wet		130	Mohr-Coulomb	173	33.8	Piezometric Line 1	
Quartzite Bedrock		155	Mohr-Coulomb	5000	25	None	0
Quartzite Bedrock Wet		155	Mohr-Coulomb	5000	25	Piezometric Line 1	
Concrete Retaining walls		145	Mohr-Coulomb	7000	0	None	0

CMT TECHNICAL SERVICES

Sundown Condominiums Phase 3
About 6550 North Powder Mountain Road, Eden, Utah

Seismic

Date: 23-Jun-2025
CMT No.: 24298
Figure: 10B