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GEOLOGIC HAZARDS AND GEOTECHNICAL ENGINEERING STUDY

Smith Creek Subdivision

Approximately 1250 South Snow Basin Road
Huntsville, Weber County, Utah
CMT PROJECT NO. 23423

FOR:

Mr. Ray Bowden
5396 East 3850 North
Eden, Utah 84310

February 6, 2025

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Subject: Geologic Hazards and Geotechnical Engineering and Study
Smith Creek Subdivision
Approximately 1250 South Snow Basin Road
Huntsville, Weber County, Utah
CMT Project Number 23392

Mr. Bowden:

Submitted herewith is the report of our geotechnical engineering and geologic hazards study for the proposed Smith Creek Subdivision in Huntsville, Utah. This report contains the results of our findings and an interpretation of the results with respect to the project characteristics available. It also contains recommendations to aid in the design and construction of the earth related phases of this project.

CMT Technical Services (CMT) personnel supervised the excavation of four (4) test pits extending to depths of approximately 8 feet below the existing ground surface. One (1) bore hole was also advanced to a depth of 10 feet below the ground surface where auger refusal occurred. Samples of the subsurface soils encountered in the explorations were collected during the field operations and subsequently transported to our laboratory for further observation and testing of select samples. Based on the findings of the subsurface explorations, conventional spread and continuous footings may be utilized to support the proposed residences, provided the recommendations in this report are followed. A detailed discussion of design and construction criteria is presented in this report. Geologic observations and data obtained from the explorations and field reconnaissance were used to evaluate potential geologic hazards at the site.

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

Sincerely,

CMT Technical Services

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APPENDIX

1.0 INTRODUCTION

1.1 General

CMT Technical Services (CMT) was retained to conduct a design level geotechnical engineering study, and a reconnaissance level geologic hazards study for the proposed Smith Creek Subdivision, which is in the Huntsville area of Weber County, Utah. The subject site is a 6.95-acre property that is presently undeveloped and proposed for subdivision into two single-family residential lots. The proposed subdivision site is located on the southern margin of Ogden Valley, south of Pineview Reservoir, as shown in ***Figure 1, Vicinity Map***, and more detailed coverage of the site location is shown in ***Figure 2, Site Plan*** and ***Figure 3, Aerial Photo***. Geological mapping of the site and vicinity is included in ***Figure 4, Geologic Map***, and slope-terrain information for the area is provided on ***Figure 5, LiDAR DEM*** and ***Figure 6, Slope Map***. The approximate locations of the test pits and boring completed for our subsurface evaluation are shown on ***Figure 7, Site Evaluation***.

1.2 Objectives and Scope

The objectives and scope of our study were planned in discussions between Mr. Bowden and Mr. Andrew Harris, P.E. of CMT. In general, the objectives of this study were to:

1. Provide a reconnaissance-level geologic hazard study as specified by Weber County Code, Section 108-22 Natural Hazard Areas guidelines and standards (Weber County, 2024). The reconnaissance level geological study was performed to assess whether all or parts of the site are exposed to the hazards that are included in the Weber County Code, Section 108-22 Natural Hazard Areas. These hazards include, but are not limited to: Surface-Fault Ruptures, Landslide, Tectonic Subsidence, Rock Fall, Debris Flows, Liquefaction Areas, Flood, or other Hazardous Areas.

2. Define and evaluate the subsurface soil and groundwater conditions across the site.

3. Perform engineering analysis, including slope stability analysis.

4. Provide appropriate foundation and earthwork recommendations as well as geoseismic information to be utilized in the development of the site and design and construction of the proposed residences.

To achieve these objectives our scope of work included the following tasks:

1. An office program including review of published geologic information consisting of geologic, topographic, and hazards maps, current and historic aerial photos, and lidar DEM imagery.

2. A geologic reconnaissance of the subject site and adjacent areas.

3. A field program consisting of the excavating, logging, and sampling of four (4) geologic test pits and the drilling/logging/sampling of one (1) hollow-stem-auger geotechnical bore hole.

4. A laboratory testing program of select samples of the subsurface soils collected in the explorations.

5. An office program consisting of the correlation of available data, engineering and geological analyses including slope stability analysis, and the preparation of this summary report.

1.3 Authorization

Authorization was provided by Mr. Bowden by returning a signed copy of our proposal dated October 10, 2024.

2.0 EXECUTIVE SUMMARY

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

A plan of the proposed site subdivision provided to CMT delineates "buildable areas" on both proposed lots (see **Figure 2**). The results of our study indicate that the proposed residences within these buildable areas may be supported upon conventional spread and/or continuous wall foundations established on suitable, undisturbed natural soils, or upon a minimum 18 inches of granular structural replacement fill extending to suitable natural soils. For design, a net bearing pressure of 1,500 pounds per square foot may be utilized.

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

1. The proposed development is located on an east-facing slope, up-slope and west of the Smith Creek drainage channel. The slope is underlain by east-dipping beds of the Norwood Formation which are overlain at the surface by a veneer of colluvium and alluvium.
2. Landslide and slump deposits are present on the southwest portion of the site, but do not encroach the proposed building areas on the central and northern portions of the site.
3. The proposed building areas at the site are not at risk from surface-fault-rupture hazards, liquefaction, tectonic subsidence, active alluvial-fan processes and debris flow hazards, flooding, rockfall, and snow avalanche.
4. Slope stability analysis performed for the slope in the proposed building areas at the site (**Section 7.0**), utilizing a geologic cross-section prepared by the project geologist (***Figure 15***), indicates stability of the slope in relation to the proposed development meet both static and seismic minimum factors of safety requirements, provided our recommendations are followed.

In the following sections, detailed discussions pertaining to the site are provided, including subsurface descriptions, geologic setting, seismicity, earthwork, foundations, lateral resistance, lateral pressure, and floor slabs.

3.0 DESCRIPTION OF PROPOSED CONSTRUCTION

We anticipate the proposed residences will be constructed using conventional wood-framed construction supported on concrete spread footings with basements. Maximum continuous wall and column loads are anticipated to be 3,000 pounds per lineal foot and 40,000 pounds, respectively. If the structural loading conditions are different than we have projected, please notify us so that any appropriate modifications to our conclusions and recommendations contained herein can be made.

Site development will require a moderate amount of earthwork in the form of site grading. We estimate in general that maximum cuts and fills, excluding basements which may require deeper cuts, to achieve design grades will be on the order of about 3.0 to 5.0 feet. Larger cuts and fills may be required in isolated areas. In general, the projected site grading activities are anticipated to consist primarily of cutting into the existing ground to construct the residences, with very little fill projected for the site. Final cuts and fills must be designed to maintain stability of the slopes on the site and not steepened (unbraced) greater than four horizontal to one vertical (4H:1V). Any planned retaining walls considered to be structural walls will need to be properly engineered.

4.0 FIELD EXPLORATIONS

The site subsurface soil conditions were explored by excavating four (4) test pits and advancing one (1) hollow-stem-auger bore hole on November 7, 2024, at the selected locations shown on ***Figure 7***. The test pits were excavated using a track-mounted mini-excavator and extended to depths of up to approximately 8 feet below the existing ground surface. The bore hole was drilled with a walk-behind, track-mounted drill rig and advanced to a depth of 10 feet where equipment refusal was encountered. During the excavating and drilling operations, continuous logs of the subsurface conditions encountered were maintained

In the test pits, representative samples of the subsurface soils exposed were collected by obtaining disturbed "grab" samples.

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

The collected samples were sealed in plastic bags or containers prior to transport to the laboratory.

The soils exposed in the test pits and the samples retrieved from the bore hole were classified in the field based upon visual and textural examination in general accordance with ASTM[1] D-2488. These field classifications have been supplemented by subsequent inspection and testing in our laboratory. The subsurface conditions encountered in the field explorations are discussed below in **Section 5.4, Subsurface Soil Conditions**, and are presented graphically on ***Figures 8 through 11, Geologic Test Pit Logs*** and ***Figure 12, Log of Boring***. Sampling information and other pertinent data and observations are also included on the logs. In addition, a Key to Symbols defining the terms and symbols used on the bore hole is provided as ***Figure 13, Key to Symbols***.

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

5.0 ENGINEERING GEOLOGY

5.1 General Geology

The site is in the foothills on the southern margin of Ogden Valley, which is a northwest trending fault bounded graben structure, with the Wasatch Range comprising the western flank of the valley and the Bear River Range the eastern flank (Avery, 1994). Topographically the site is located on the valley margin in the foothills of Mount Ogden of the Wasatch Range. The elevation of the site ranges between approximately 5,020 feet at the Smith Creek Channel on the northern site boundary to about 5,140 feet at the southwest corner of the site. The site is located on an east-facing slope, west of Smith Creek, which crosses the site in a general south to north orientation.

The Wasatch fault, approximately 6.9 miles west of the site, generally marks the western base of the Wasatch Range and provides the basis of division between the Middle Rocky Mountain Physiographic Province on the east and the Basin and Range Physiographic Province on the west. The Basin and Range Physiographic Province is characterized by approximately north-south trending valleys and mountain ranges that have been formed by extensional tectonics and displacement along normal faults, extending from the Wasatch Range on the east to the Sierra Nevada Range on the west. The Middle Rocky Mountain province covers parts of Utah, Colorado, Wyoming, Idaho, and Montana. The geology of the province is an assemblage of sedimentary, igneous, and metamorphic rocks that have been folded, faulted, and uplifted. Mountain building (tectonic) activity commenced about 30 million years ago (Cretaceous time) and continues to the present. The province is characterized by mountainous terrain with deep canyons and broad intervening basins, with temperate semi-arid to mesic climatic conditions (Hunt, 1967).

The site is located within a setting of complex geological conditions wherein Pre-Cambrian and Paleozoic rocks were locally thrusted over the same during a series of eastward thrust extensions, the last of which is named the Willard Thrust sheet, which is believed to have moved onto the vicinity during the Cretaceous Sevier orogeny, and occurred approximately 140 million years ago (ma). Locally constrained within the

[1] American Society for Testing and Materials

valley are mid Tertiary units of the Norwood Formation that ramp along the base of the mountains to the east and west of the valley. The Norwood Formation is described as "light-gray to light brown, altered tuff (claystone), tuffaceous siltstone, sandstone, and conglomerate" derived from volcanic ash deposition (Coogan and King, 2016). The claystone, siltstone and sandstone occurrences of the formation are primarily a result of lacustrine (lake processes) redeposition of the volcanic ash.

The exposure of the present surficial geology of the site vicinity is the result of the uplift and exposure of older, Cambrian and pre-Cambrian rocks which form the crest of Mount Ogden (9,579 feet) southwest of the valley and James Peak on the east-northeast. This exposure was the result of movement along locally high-angle faults (i.e., the Wasatch fault) during late Tertiary and Quaternary time (Bryant, 1988). The present topography was finally shaped by Quaternary stream deposition and planation by Smith Creek, and similar valley-margin drainages, which have deposited range-margin coarse alluvium that has been modified by late-Pleistocene lacustrine processes (Lake Bonneville). The current geological mapping drawn from King and others (2008) of the site vicinity is shown in ***Figure 4***.

5.2 Site Surface Conditions

The site conditions and site geology were interpreted through an integrated compilation of data, including a review of literature and mapping from previous studies conducted in the area (Bryant, 1988; King and others, 2008; King and McDonald, 2014; and Coogan and King, 2016); photogeologic analyses of 2012 and 2021 orthoimagery shown in ***Figure 3***; historic stereoscopic imagery flown in 1963 (Utah Geological Survey, 2025); Analyses of elevation and LiDAR terrain data as shown in ***Figures 5 and 6***; field reconnaissance of the general site area; and the interpretation of the test pit exposures made on the site as part of our field program. Seismic hazards information was developed from United States Geologic Survey (USGS) databases.

The topography of the site vicinity consists of gentle to moderately steep valley-margin foothill and drainage slopes. Vegetation at the site is generally dense brush and trees with interspersed grasses and weeds. Slope gradients of the east-facing slope west of Smith Creek developed from our LiDAR analysis and site observations were found to range between approximately 20% and 25% as shown in ***Figure 6***. Smith Creek, flowing down-slope to the north, has incised a channel at the base of the slope that extends up to approximately 10 to 15 feet below the adjacent banks.

5.3 Surficial Geology

The surficial geology of the site is presented in ***Figure 4*** of this report and has been taken from mapping prepared by King and others (2008). A summary of the mapping units identified at and near the subject lot are paraphrased below in relative age sequence (youngest (top) to oldest):

> **Qh** – Human disturbance (Historical) - Obscures original deposits by cover or removal; mostly fill along railroad and highway grades, and some large gravel pits that predate 1986 aerial photographs.

Qms and **Qmsy** – Landslide and slump deposits (Holocene and Pleistocene) - Poorly sorted clay-to boulder-sized material; locally includes flow deposits; generally characterized by hummocky topography, main and internal scarps, and chaotic bedding in displaced blocks; composition depends on local sources; morphology becomes more subdued with time and amount of water in deposits; Qms may be in contact with Qms when two different slide/slumps abut; locally, unit involved in slide/slump is shown in parentheses where a nearly intact block is visible; Qms and Qmso queried (?) where bedrock block may be in place; thickness highly variable, boreholes in Rogers (1986) show thicknesses of about 20 to 30 feet (6-9 m) on small slides/flows.

Qms without a suffix is mapped where the age is uncertain (though likely Holocene and/or upper Pleistocene), where portions of slide/slump complexes have different ages but cannot be shown separately at map scale, or where boundaries between slides/slumps of different ages are not distinct. Estimated time of emplacement indicated by relativeage number and letter suffixes with: 1 - likely emplaced in the last 80 to 150 years, mostly historical; y - post- Lake Bonneville in age and mostly pre-historic…

Qac – Alluvium and colluvium (Holocene and Pleistocene) - Includes stream and fan alluvium, colluvium, and, locally, mass-movement deposits; 0 to 20 feet (0-6 m) thick.

Qc - Colluvium (Holocene and Pleistocene) - Includes materials moved by slopewash and soil creep; composition depends on local sources; generally 6 to 20 feet (2-6 m) thick; not mapped where less than 6 feet (2 m) thick.

Qmc – Landslide and slump, and colluvial deposits, undivided (Holocene and Pleistocene) - Mapped where landslides and slumps are difficult to distinguish from colluvium (slopewash and soil creep) and where mapping separate, small, intermingled areas of slides and slumps, and colluvial deposits is not possible at map scale; locally includes talus and debris flows; typically mapped where landslides and slumps are thin (“shallow”); also mapped where the blocky or rumpled morphology that is characteristic of landslides and slumps has been diminished (“smoothed”) by slopewash and soil creep; composition depends on local sources; 0 to 40 feet (0-12 m) thick. These deposits are as unstable as other landslides and slumps units (Qms).

Ql - Lake Bonneville deposits, undivided (upper Pleistocene) - Silt, clay, sand, and cobbly gravel; mapped where grain size is mixed or surface weathering obscures grain size and deposits are not exposed in scarps and construction cuts; thickness uncertain.

Tn - Norwood Formation (lower Oligocene and upper Eocene) - Typically light-gray to lightbrown, altered tuff (claystone), tuffaceous siltstone, sandstone, and conglomerate; locally colored light shades of red and green; variable calcareous cement and zeolitization, that is less common to south of Snow Basin quadrangle; zeolite marker beds mapped as an aid to recognizing geologic structure; locally includes landslides and slumps that are too small to show at map scale.

Much of the subject site, and particularly the designated buildable areas, are located upon east-dipping Norwood formation beds (Unit Tn) overlain at the surface by a veneer of colluvium (Unit Qc) deposited primarily by slope wash processes. King and others (2008) also mapped landslide and slump deposits (Units Qmsy, Qms, and Qmc, **Figure 4**) on portions of the east-facing slope at and adjacent to the site.

5.4 Subsurface Soil Conditions

The soils exposed in the test pits excavated at the site were generally consistent between the pit locations and consisted of between approximately 2.5 to 4 feet of colluvium (Unit 1, ***Figures 8 to 11***) comprised of Lean to Fat CLAY (CL to CH) with varying amounts of gravel. The colluvium overlies deposits of the Norwood Formation (Unit 2, ***Figures 8 to 11***) consisting of beds of Lean CLAY (CL), Fat CLAY (CH), sandstone, and welded volcanic ash. The surficial colluvial deposits are interpreted to have been deposited primarily by slope wash processes. A pedogenic soil A horizon (Unit 1sA, ***Figures 8 to 11***) has formed on the surface of the slope within the colluvium. Equipment refusal due to very hard/stiff soil or rock conditions was encountered in each of the test pits at the depth each pit was terminated. Detailed descriptions of the subsurface soil conditions exposed in the test pits are provided in ***Figures 8 through 11***.

The orientation of bedding within the Norwood Formation exposed and measured in test pits TP-1, TP-2, and TP-4 displayed strikes between N 03° E and N 05° W and dips between 7° and 17° east, with an average dip of 11° east.

The bore hole completed at the site encountered layered Sandy Lean CLAY (CL) at the surface interpreted to be colluvium as previously discussed. The colluvium was found to be underlain by beds of Gravelly Clayey SAND (SC), Fat CLAY (CH), and Lean CLAY (CL) interpreted to be beds of the Norwood Formation. Detailed descriptions of the subsurface soils encountered in the bore hole are included in ***Figure 12***. Equipment refusal was encountered at approximately 10 feet below the surface in the bore hole where a hard/dense layer was encountered.

Groundwater was not encountered in the test pits or bore hole at the time of our field program. No springs or seeps were observed on the east-facing slope at the site. Future seasonal and longer-term groundwater fluctuations should be anticipated for the site, with the highest seasonal levels generally occurring during the late spring and summer months. Numerous other factors such as heavy precipitation, rapid snow-melt, and other unforeseen factors, may also influence ground water elevations at the site. Groundwater is not anticipated to be encountered during construction.

5.5 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, caution should be taken in interpolating or extrapolating subsurface conditions beyond the exploratory locations. Seasonal fluctuations in ground water conditions may also occur.

In addition, once the subsurface explorations were completed, the bore hole was backfilled with auger cuttings and the test pits were backfilled with the excavated soils, but no effort was made to compact these soils. Test pit backfill soils must be considered non-engineered fill. Settlement of the backfill in the test pits over time should be anticipated and caution should be exercised when constructing over these locations.

5.6 Seismic Setting

5.6.1 General

Utah has adopted the IBC 2021 code 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[2] 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.

5.6.2 Active Earthquake Faults

Based upon our review of available maps and literature, no active faults are known to pass through or immediately adjacent to the subject site. The nearest active (Holocene) earthquake fault to the site is the Weber segment of the Wasatch fault zone (UT2351E) which is located 6.9 miles west of the site (Black and others, 2004). Accordingly, fault-rupture-hazards are not considered present at or adjacent to the site.

5.6.3 Soil Site Class

Considering our explorations only extended to maximum depths of approximately 8 to 10 feet where very hard/dense conditions were encountered, and projecting that these dense conditions extend to at least 100 feet below the existing ground surface, it is our opinion that site best fits Site Class C – Very Dense Soil and Soft Rock profile, which we recommend for seismic structural design.

The Seismic Design Categories in the International Residential Code (IRC 2021 Table R301.2.2.1.1) are based upon the Site Class as addressed in the previous section. For Site Class C at site grid coordinates of 41.2435 degrees north latitude and -111.7946 degrees west longitude, $\mathbf{S_{DS}}$ is 0.507 and the **Seismic Design Category** is D_0.

5.6.4 Liquefaction

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

[2] American Society of Civil Engineers

primary factors affecting liquefaction potential of a soil deposit are: (1) magnitude and duration of seismic ground motions; (2) soil type and consistency; and (3) occurrence and depth to groundwater.

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

Based on the lack of groundwater and the predominantly clayey soils encountered, liquefaction of the natural soils encountered within the maximum depth explored, approximately 10 feet, are not susceptible to liquefaction.

5.6.5 Tectonic Subsidence

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

5.7 Landslide and Slump Deposits

King and others (2008) mapped landside and slump deposits on the slope on and adjacent to the subject site (Units Qmsy, Qms, and Qmc, ***Figure 4***). Based on our surface and subsurface observations, these deposits and associated landslide processes have not impacted the proposed and delineated buildable areas on the lots. No surficial or subsurface evidence of past or ongoing slope movement (e.g. hummocky and stair-step terrain, grabens, head-scarps, pressure ridges, displaced landforms, lobate deposits, convex/concave surface morphology, disturbed or deformed bedding, zones of shearing in subsurface deposits, etc.) was observed in the proposed building areas. No evidence of landslide deposits (Unit Qms, Figure 4) mapped by King and others (2008) on the northwest portion of the site was observed at the surface or in the subsurface.

Landslide and slump deposits (Units Qmsy and Qmc, ***Figure 4***) mapped by King and others (2008) on the southwest portion of the site were not investigated by our subsurface explorations. Visual inspection of the slope surface on this portion of the site was found to be inconclusive due to dense vegetation cover on the slope. The lidar DEM analysis presented in ***Figure 5***, as well as our stereoscopic analysis of the 1963 aerial photos (UGS, 2025), revealed the presence of hummocky terrain and a possible subdued head scarp that are indicative of past slope movement on this portion of the site. Based on our observations and analysis, this slope movement has not impacted the proposed and delineated building areas at the site.

Based on the findings of this study, and in conjunction with the geologic mapping completed by King and others (2008), a site geologic map was produced and is included as ***Figure 14, Site Geologic Map***. No structures designed for human occupancy or critical infrastructure should be sited on the southwest portion of the subject site, where units Qmc and Qmsy are mapped, without additional investigation.

A geologic cross-section was completed along line A-A' shown in ***Figures 7 and 14***. The geologic cross-section is included as ***Figure 15, Geologic Cross-Section A-A'***, and was used by the project geotechnical engineer to model and assess the stability of the slope at the proposed building locations at the site. The stability analysis (**Section 7.0**) of the slope indicates that the existing slope in its present configuration has factors of safety against instability typically considered acceptable for both static and seismic conditions, provided our recommendations are followed. We conclude the proposed building areas at the subject site are not at risk from landslide hazards.

5.8 Sloping Surfaces

The surface slopes of the site vicinity developed from our LiDAR analysis and on-site observations and measurements are shown on ***Figure 6***. The slope of the site was found to range between approximately 20% and 25%. The limiting steep slope gradients for development considerations according to the Weber County Code is 25-percent (Weber County Code, 2023).

5.9 Alluvial Fan - Debris Flow Processes

The subject site is not mapped (King and others, 2008) on or adjacent to any alluvial-fans and no evidence of active alluvial-fan or debris flow deposits or processes (e.g. flow levees, lobate deposits, convex surface morphology, mud coatings on boulders and vegetation, damage to vegetation, etc.) was observed on the surface or in the subsurface of the subject lot. The nearest deposits associated with potential debris flow origin and activity are mapped as Units Qla and Qap (King and others, 2008) approximately 1,300 feet to the north of the site.

Additionally, a 20+ foot high berm has been placed across Smith Creek as part of the construction of Chaparral Road up-stream of the subject site. A culvert in the berm allows ephemeral[3] stream flow in the Smith Creek drainage channel to pass below the road, however, the berm would act as a barrier to debris flows in the drainage.

Based on the referenced geologic mapping and our site observations, the proposed and delineated building areas at the subject site are not at risk from debris flow or other active alluvial-fan processes.

5.10 Flooding Hazards

Mapping by Federal Emergency Management Agency (FEMA, 2015) indicates the subject site is not within or adjacent to any FEMA designated flood hazard zones.

Local sheet flow, slope wash, and seasonally perched soil water typical of sloping areas should be anticipated for the site, and site improvements.

[3] Ephemeral Stream: An ephemeral stream has flowing water only during, and for a short duration after, precipitation events in a typical year. Ephemeral stream beds are located above the water table year-round. Groundwater is not a source of water for the stream. U.S. EPA: https://www.epa.gov/sites/default/files/2016-02/documents/realestate_glossary.pdf

5.11 Rockfall and Avalanche Hazards

Based on site observations and reviewed aerial photography, no potential rockfall source areas (e.g. cliffs, bedrock outcrops, boulders on slope surfaces, etc.) are located up-slope of the subject site. No rockfall clasts are observed on or adjacent to the site. The site is not located in an observed rockfall travel path or runout zone and is not at risk from rockfall hazards.

The subject site is not located down-slope from steep, alpine slope areas, particularly north-facing slopes, where snow avalanche hazards typically originate. According to the Utah Division of Emergency Management (DEM)[4], snow avalanches are typically triggered on slopes between 30 and 50 degrees. Such slope conditions do not exist at or in the vicinity of the site, particularly on up-slope areas to the west and southwest.

6.0 LABORATORY TESTING

Selected samples of the subsurface soils collected in the explorations were subjected to various laboratory tests to assess the following pertinent engineering properties:

1. Moisture Content, ASTM D-2216, Percent moisture representative of field conditions
2. Dry Density, ASTM D-2937, Dry unit weight representing field conditions
3. Atterberg Limits, ASTM D-4318, Plasticity and workability
4. Gradation Analysis, ASTM D-1140/C-117, Grain Size Analysis
5. Direct Shear Test, ASTM D-3080, Shear strength parameters

A direct shear test was performed on a sample (B-1 at 5 feet) of the subsurface natural clay soil. Because the sample was disturbed, it was remolded into the test rings, thus the direct shear test results will likely be lower in strength than the in-situ undisturbed soil. For the direct shear test, the sample was evenly loaded/consolidated within the test ring and saturated immediately after the load was applied; after consolidation was completed, the sample was sheared horizontally while measuring the shearing force and the horizontal and vertical deformations of the sample. This process was repeated twice while increasing the normal load imposed on the sample. Detailed results of the test are included at the back of this report.
Laboratory test results are presented on the bore hole log (***Figure 12***), on ***Figure 16***, and in the **Lab Summary Table** on the following page:

[4] Utah Division of Emergency Management Hazard Mitigation: https://hazards.utah.gov/avalanche/

LAB SUMMARY TABLE

EXPLOR. #	DEPTH (feet)	SOIL CLASS	SAMPLE TYPE	MOISTURE CONTENT(%)	DRY DENSITY (pcf)	GRADATION			ATTERBERG LIMITS			FRICTION ANGLE & COHESION
						GRAV.	SAND	FINES	LL	PL	PI	
TP-1	3	CL	Grab	14		11	27	62	48	15	33	
TP-1	6	CL	Grab	21		0	16	84				
TP-2	4	CL	Grab	19		0	42	58				
TP-2	5.5	CL	Grab	24				88				
TP-2	7	CH	Grab	26				97	62	21	41	
TP-3	3	CH	Grab	23		1	19	80	64	17	47	
TP-3	5	GM	Grab	9		74	14	12				
TP-4	6	CL	Grab	32		0	31	69				
B-1	2.5	SC	Rings	6	94	41	43	16	40	16	24	
B-1	5	CH	SPT	31		0	27	73	76	30	46	40.5°, 140 psf
B-1	7.5	CL	SPT	21		12	15	73	39	14	25	

7.0 SLOPE STABILITY

7.1 Input Parameters

A general global slope stability analysis was completed for the east facing slope with our understanding that the proposed homes will be situated near the crest of this slope, approximately 100 feet from the road per the site plan provided to CMT. Site topography was based upon cross-section *A-A'*, located as shown on ***Figures 7 and 14*** and illustrated in ***Figure 15***. Soil stratigraphy was based upon the data obtained as described in section **5.4 Subsurface Soil Conditions** provided above. The properties of the slope soils were based upon the subsurface explorations, laboratory testing, and the results of previous investigations and testing CMT has performed in the vicinity of the site. Accordingly, the following were utilized in our analysis:

STABILITY INPUT PARAMETERS

Material	Internal Friction Angle (degrees)	Apparent Cohesion (psf)	Unit Weight (pcf)
FILL (Colluvium from Road Cut)	30	50	120
Colluvium (CL, CH)	30	50	120
Stream Alluvium (CL, CH)	30	50	120
Norwood Formation	33	600	130

To evaluate slope stability under seismic (pseudostatic) conditions, the peak horizontal acceleration was queried for the site and methods provide by Bray and Travasarou having a maximum allowable deformation of 2 inches were utilized. The pseudostatic coefficient for the stability analysis utilized was *0.153g* (see attached calculations).

Groundwater was not encountered in our explorations, however, there is a potential for seasonal groundwater to "perch" on the underlying very dense/hard subsurface layers. Based upon this condition the surficial colluvium layer was modeled as saturated.

To model potential structural loading a distributed load of 1,500 psf was applied at the approximate residence locations.

7.2 Stability Analyses

We evaluated the global stability of the existing slope along cross-section *A-A'* using the computer program *SLIDE2*. The *SLIDE2* 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. The results of our analyses utilizing the estimated soil properties described previously indicate factors of safety greater than the required minimums for the slope in its present configuration, and with an assumed cut for a residence. The results of the stability analyses are graphically shown on ***Figures 17 and 18 Stability Results***, provided in the Appendix (along with the input/output data), and are summarized in the following table.

STABILITY ANALYSIS RESULTS

Condition	Seismic Coefficient	Lowest Factor of Safety (F.S.)	Minimum Allowable F.S.
Static	---	4.014	1.5
Seismic	0.153	2.406	1.0

Slope movements or even failure can occur if the slope soils are undermined, steepened, or become saturated. <u>Any changes to the grading at the site</u>, <u>particularly where steepening and additional loading occurs,</u> <u>must be reviewed by CMT prior to the initiation of any construction in order to assess if our findings and recommendations remain applicable</u>.

During construction, a CMT geotechnical engineer and/or geologist must observe grading and exposed soil conditions to assess that suitable conditions are encountered. Following any grading at the site, CMT recommends that the surface of permanent slopes be revegetated as soon as possible to limit erosion and potential undermining of the slopes. The property owners and/or the owner's representatives should be made aware of the risks involved should the site soils become saturated, erode, or undermined.

8.0 SITE PREPARATION AND GRADING

8.1 Site Preparation

All deleterious materials should be stripped from the site prior to commencement of construction activities. This includes vegetation, topsoil, loose and disturbed soils, etc. Based upon the conditions observed at the time of our subsurface exploration, there is topsoil on the surface as well as fill soils near the road, likely derived from the road, cut, which should be expected to vary in depth and lateral extent.

When stripping and grubbing, topsoil should be distinguished by the apparent organic content and not solely by color.

Existing fill from the road cut, regardless of the length of time the fill has been in place, should be considered undocumented/non-engineered fill. All undocumented fill shall be removed from beneath structures, but may remain beneath exterior flatwork and pavements, provided they are properly prepared and the owner understands that these soils still have the potential to consolidate/settle over time and additional maintenance of surfaces constructed over them may be required. Outside of building footprints, proper preparation of undocumented fill and disturbed soils shall consist of removing the upper 12 inches, scarifying the exposed surface to a minimum depth of 8 inches, moisture conditioning, and recompacting the soils in place to the requirements specified in section **8.5 Fill Placement and Compaction**. The removed 12 inches, if free of debris, organics, or other deleterious materials, may then be replaced in similarly compacted lifts. In driveway areas CMT recommends the subgrade be proofrolled by passing moderate-weight rubber tire-mounted construction equipment over the surface at least twice. If excessively soft or loose soils are encountered, they must be removed (up to a maximum depth of 2 feet) and replaced with structural fill.

Following clearing, grubbing, and prior to other subgrade preparation, the exposed soils should be observed by a CMT geotechnical engineer to assess that suitable soils have been exposed and any deleterious materials, loose and/or disturbed soils have been removed, prior to placing site grading fills, footings, slabs, or pavements.

Fill placed over large areas to raise overall site grades can induce settlements in the underlying natural soils. If more than 3 feet of site grading fill is anticipated over the natural ground surface, we should be notified to assess potential settlements and provide additional recommendations as needed. These recommendations may include placement of the site grading fill far in advance to allow potential settlements to occur prior to construction.

8.2 Temporary Excavations

Excavations deeper than 8 feet are not anticipated at the site. Groundwater was not encountered within the depths explored, about 10 feet at the time of our field explorations, and thus is not anticipated to affect excavations.

The natural soils encountered at this site predominantly consisted of clay. 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 8 feet deep, above or below groundwater, may be constructed with side slopes no steeper than one-half horizontal to one vertical (0.5H:1V).

For sandy/gravelly (cohesionless) soils, temporary construction excavations not exceeding 4 feet in depth should be no steeper than one-half horizontal to one vertical (0.5H:1V). For excavations up to 8 feet and above groundwater, side slopes should be no steeper than one horizontal to one vertical (1H:1V). Excavations encountering saturated cohesionless soils will be very difficult to maintain and will require very flat side slopes and/or shoring, bracing and dewatering.

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

8.3 Structural 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. All structural fill must be free of sod, rubbish, topsoil, frozen soil, and other deleterious materials.

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

Fill Material Type	Description/Recommended Specification
Structural Fill	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, less than 25 percent passing the No. 200 sieve and a maximum Plasticity Index of 10 percent.
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 gravel soils may be suitable for use as structural fill, if found to meet the specifications in the table above. These soils may also be utilized as site grading fill.

On-site clay soils are not suitable for structural fill or site grading fill below exterior flatwork or pavements, but may be utilized as fill in landscape areas.

8.4 Utility Trenches

All utility trench backfill material below structurally loaded facilities (flatwork, floor slabs, driveways, etc.) shall be placed at the same density requirements established for structural fill. If the surface of the backfill becomes disturbed during the course of construction, the backfill shall be proofrolled and/or properly recompacted prior to the construction of any exterior flatwork over a backfilled trench. Proofrolling shall be performed by passing moderately loaded rubber tire-mounted construction equipment uniformly over the surface at least twice. If excessively loose or soft areas are encountered during proofrolling, they shall be removed to a maximum depth of 2 feet below design finish grade and replaced with structural fill.

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. These organizations are also requiring that in public roadways the backfill over major utilities be compacted over the full depth of fill to at least 96 percent of the maximum dry density as determined by the AASHTO T-180 (ASTM D-1557) method of compaction.

In private utility areas, natural soils may be re-utilized as trench backfill over the bedding layer provided that they are properly moisture prepared and compacted to the minimum requirements stated in section **8.5 Fill Placement and Compaction** below.

8.5 Fill Placement and Compaction

The various types of compaction equipment available have their limitations as to the maximum lift thickness that can be compacted. For example, hand operated equipment is limited to lifts of about 4 inches and most "trench compactors" have a maximum, consistent compaction depth of about 6 inches. Large rollers, depending on soil and moisture conditions, can achieve compaction at 8 to 12 inches. The full thickness of each lift should be compacted to at least the following percentages of the maximum dry density as determined by ASTM D-1557 (or AASHTO[5] 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)	0 to 5 5 to 8	95 98
Site grading fill outside area defined above	0 to 5 5 to 8	92 95
Utility trenches within structural areas	--	96
Roadbase and subbase below pavements	-	96
Non-structural fill	0 to 5 5 to 8	90 92

[5] American Association of State Highway and Transportation Officials

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

Field density tests should be performed on each lift as necessary to verify that compaction is being achieved.

8.6 Stabilization

The natural clay soils at this site will likely be susceptible to rutting and pumping. The likelihood of disturbance or rutting and/or pumping of the existing natural soils is a function of the moisture content, the load applied to the surface, as well as the frequency of the load. Consequently, rutting and pumping can be reduced by avoiding concentrated traffic, reducing the load applied to the surface by using lighter equipment and/or partial loads, by working in drier times of the year, or by providing a working surface for the equipment. Rubber-tired equipment particularly, because of high pressures, promotes instability in moist/wet, soft soils.

If rutting or pumping occurs, traffic should be stopped, and the disturbed soils should be removed and replaced with stabilization material. Typically, a minimum of 18 inches of the disturbed soils must be removed to be effective. However, deeper removal is sometimes required.

To stabilize soft subgrade conditions (if encountered), a mixture of coarse, clean, angular gravels and cobbles and/or 1.5- to 2.0-inch clean gravel should be utilized. This coarse material may be placed and worked into the soft soils until firm and non-yielding or the soft soils removed an additional, minimum of 18 inches, and backfilled with the clean stabilizing fill. 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.

9.0 FOUNDATION RECOMMENDATIONS

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

9.1 Foundation Recommendations

Based on our geotechnical engineering analyses, proposed residences may be supported upon conventional spread and/or continuous wall foundations established upon suitable, undisturbed soil or 12 inches of granular structural replacement fill extending to suitable natural soils utilizing a design, net bearing pressure of up to 1,500 pounds per square foot.

The term "net bearing pressure" refers to the pressure imposed by the portion of the structure located above lowest adjacent final grade, thus the weight of the footing and backfill to lowest adjacent final grade need not be considered. The allowable bearing pressure may be increased by 1/2 for temporary loads such as wind and seismic forces.

We 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.

9.2 Installation

Under no circumstances shall the footings be established upon non-engineered fills, loose or disturbed soils, topsoil, sod, rubbish, construction debris, other deleterious materials, frozen soils, or within ponded water. If unsuitable soils are encountered, they must be completely removed and replaced with compacted structural fill.

The base of footing excavations should be observed by a CMT geotechnical engineer to assess that suitable bearing soils have been exposed.

All structural fill should meet the requirements for such, and should be placed and compacted in accordance with **Section 8.0** 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 2 feet, the fill replacement width should be 4 feet, centered beneath the footing.

If the granular structural fill upon which the footings are to be established becomes disturbed, it shall be recompacted to the requirements for structural fill or be removed and replaced with new structural fill.

9.3 Estimated Settlement

Foundations designed and constructed in accordance with our recommendations could experience some settlement, but we anticipate that settlement of footings founded as recommended above will be approximately 1 inch or less.

9.4 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 clay soils or 0.40 should be utilized for natural gravel soils or structural fill. Passive resistance provided by properly placed and compacted granular structural fill above the water table may be considered equivalent to a fluid with a density of 350 pounds per cubic foot.

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

10.0 LATERAL EARTH PRESSURES

We anticipate that below-grade walls up to 8 feet high might be constructed at this site. The lateral earth pressure values given below are for a backfill material that will consist of drained natural 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. For subgrade walls 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)	40	15
At-Rest Pressure (wall is not allowed to yield)	60	N/A
Passive Pressure (wall moves into the soil)	350	85

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

11.0 FLOOR SLABS

Properly engineered floor slabs should be established upon a minimum of 12 inches of structural fill extending to suitable natural soils. 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 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 structural engineer. Design provisions should address the following features:

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[6] or other local standards recommendations; 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).

For exterior concrete slabs on grade and driveways overlying clay soils, we recommend a minimum 8 inches of roadbase be installed directly below the exterior slab on grade.

12.0 DRAINAGE RECOMMENDATIONS

12.1 Surface Drainage

It is important to the long-term performance of foundations and floor slabs that water is not allowed to collect near the foundation walls and infiltrate into the underlying soils. We recommend the following:

1. All areas around each residence should be sloped to provide drainage away from the foundations. Where possible we recommend a minimum slope of 6 inches in the first 10 feet away from 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. CMT recommends landscaping that does not require supplemental irrigation beyond establishment. If used, sprinklers should be aimed away from the foundation walls. Sprinkling systems should be designed with proper drainage, well-maintained, and checked for leaks frequently. Overwatering should be avoided.

5. Other precautions may become evident during construction.

[6] American Concrete Institute

12.2 Subdrains

12.2.1 General

Due to the potential for random perched groundwater conditions within the predominantly clay soils sequence it is recommended that a foundation drain be installed around residences.

12.2.2 Foundation Subdrains

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 a 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 subdrain, the below-grade walls should be dampproofed. The slope of the subdrain should be at least 0.3 percent. The foundation subdrains shall be discharged to a down-gradient location well away from the homes.

13.0 QUALITY CONTROL

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

13.1 Field Observations

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

13.2 Fill Compaction

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

14.0 LIMITATIONS

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

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

We appreciate the opportunity to be of service to you on this project. CMT offers a full range of Geotechnical Engineering, Geological, Material Testing, Special Inspection services, and Phase I and II Environmental Site Assessments. With offices throughout Utah, as well as in 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. To schedule materials testing please call 801-908-5859.

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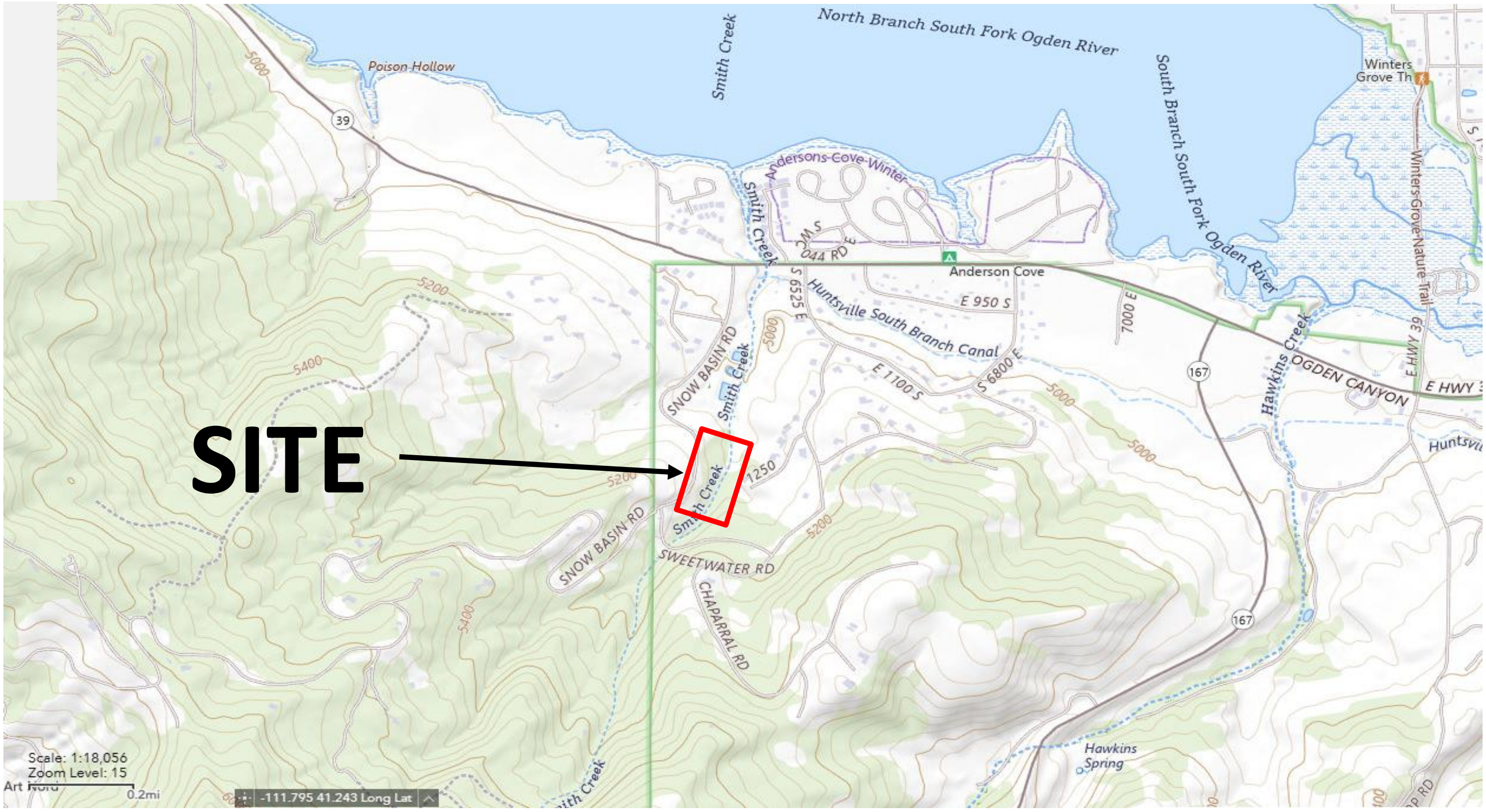
APPENDIX

SUPPORTING
DOCUMENTATION



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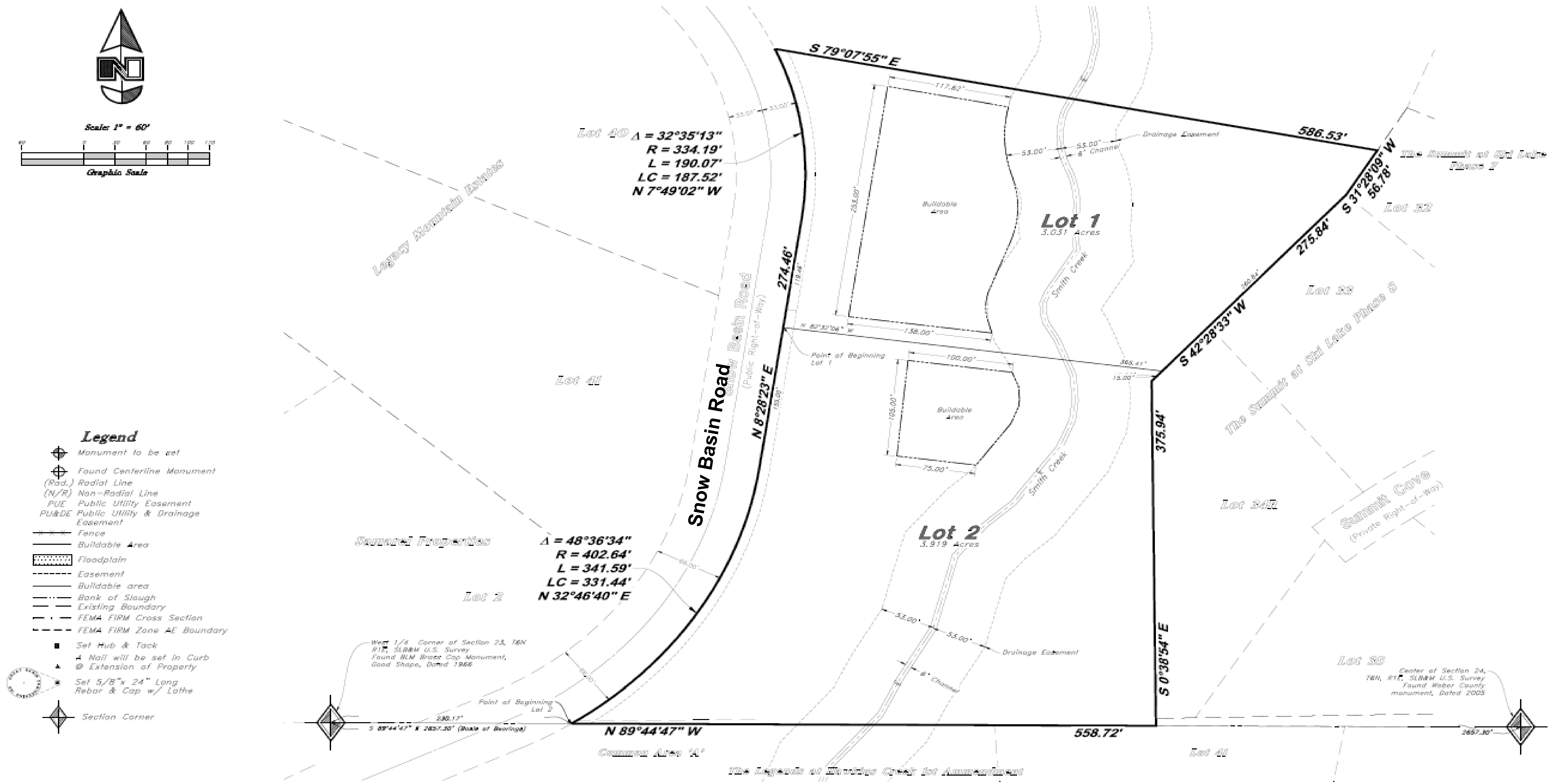
Contour Interval: Major - 200 feet

Minor - 40 feet

Base map from: USGS National Map Viewer - https://apps.nationalmap.gov/viewer/

Smith Creek Subdivision	CMT ENGINEERING LABORATORIES			Figure: 1
Approx. 1250 S. Snow Basin Rd., Huntsville, UT	Vicinity Map	Date:	4-Feb-25	
		Job #	23423	



Site plan provided by Client

CMT TECHNICAL SERVICES	**Smith Creek Subdivision** Approx. 1250 S. Snow Basin Rd., Huntsville, UT	**Site Plan**	Date: 4-Feb-2025 CMT No.: 23423	Figure: 2

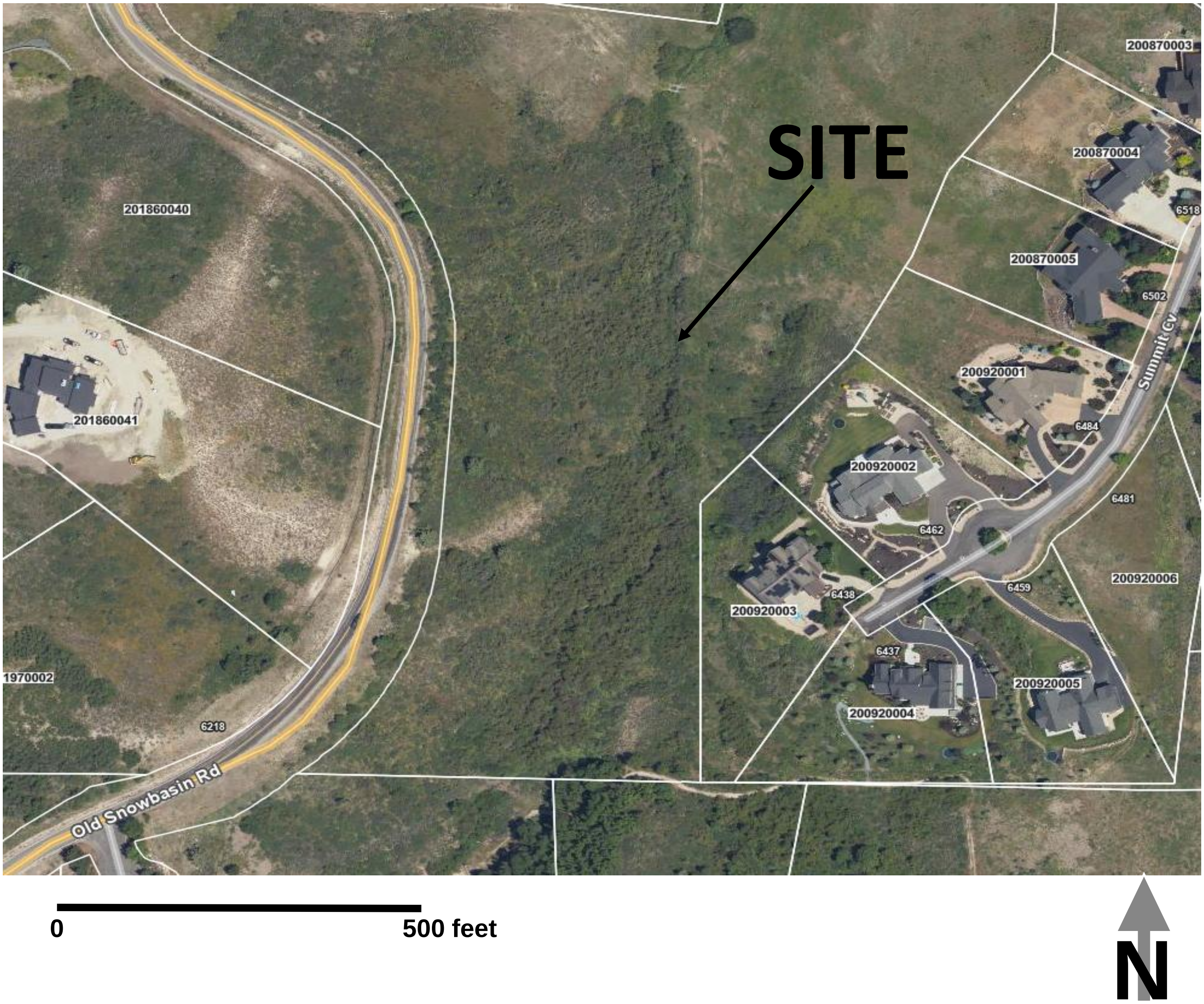
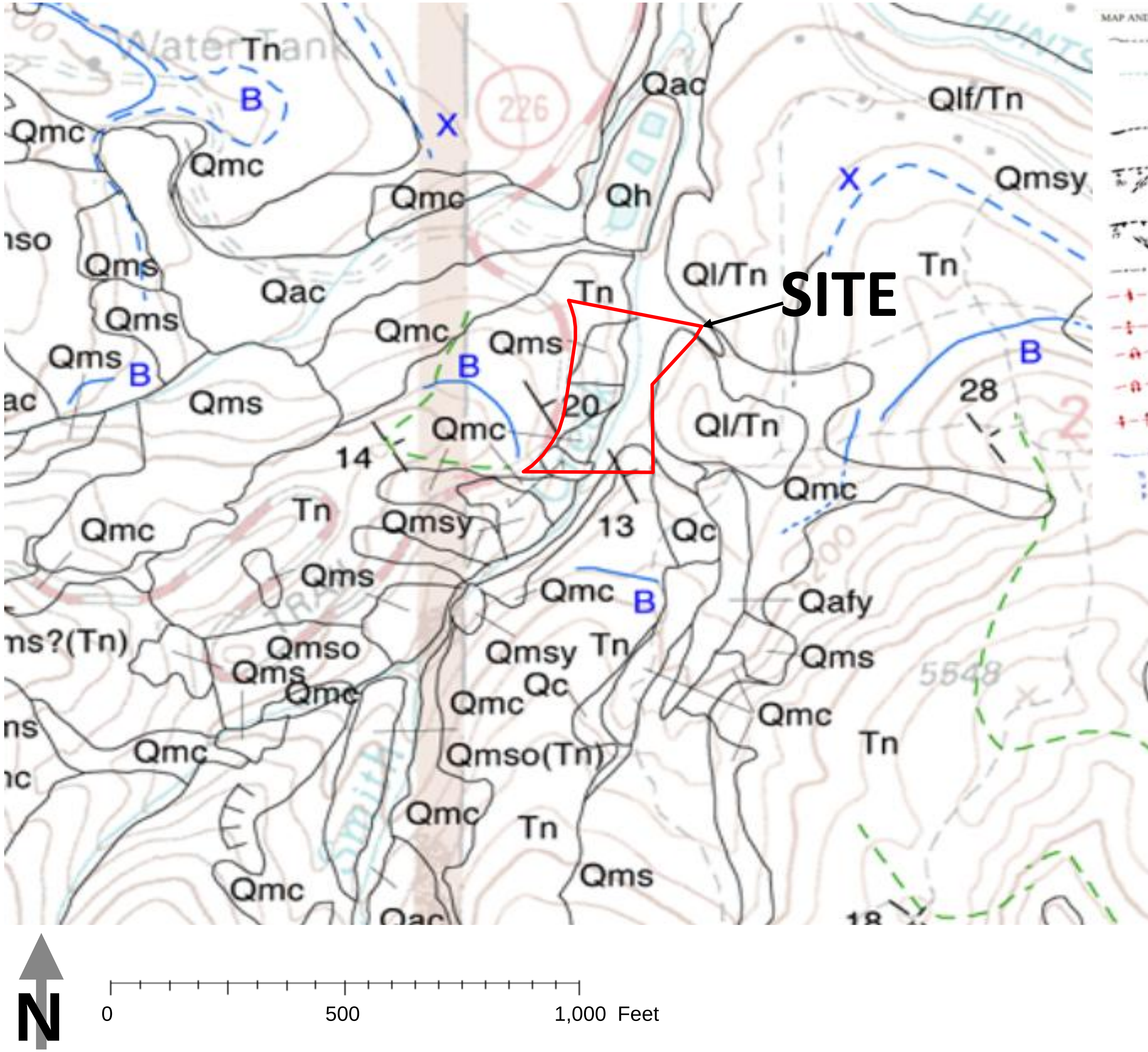


Photo: 2021 0.6m NAIP Othoimagery

Base photo from: Utah Geospacial Resource Center

https://parcels.utah.gov/#/location/-12442477.352974465,5129905.01908787,9027.977411

Smith Creek Subdivision	CMT ENGINEERING LABORATORIES		Figure: 3
Approx. 1250 S. Snow Basin Rd., Huntsville, UT	Aerial Photo	Date: 4-Feb-25 Job # 23423	



<u>Geology at Site and Vicinity</u>

Qh - Human disturbance (Historical)

Qmsy - Landslide and slump deposits (Holocene)

Qac - Alluvium and colluvium (Holocene and Pleistocene)

Qc - Colluvium (Holocene and Pleistocene)

Qms - Landslide and slump deposits (likely Holocene and/or upper Pleistocene)

Qmc - Landslide and slump, and colluvial deposits, undivided (Holocene and Pleistocene)

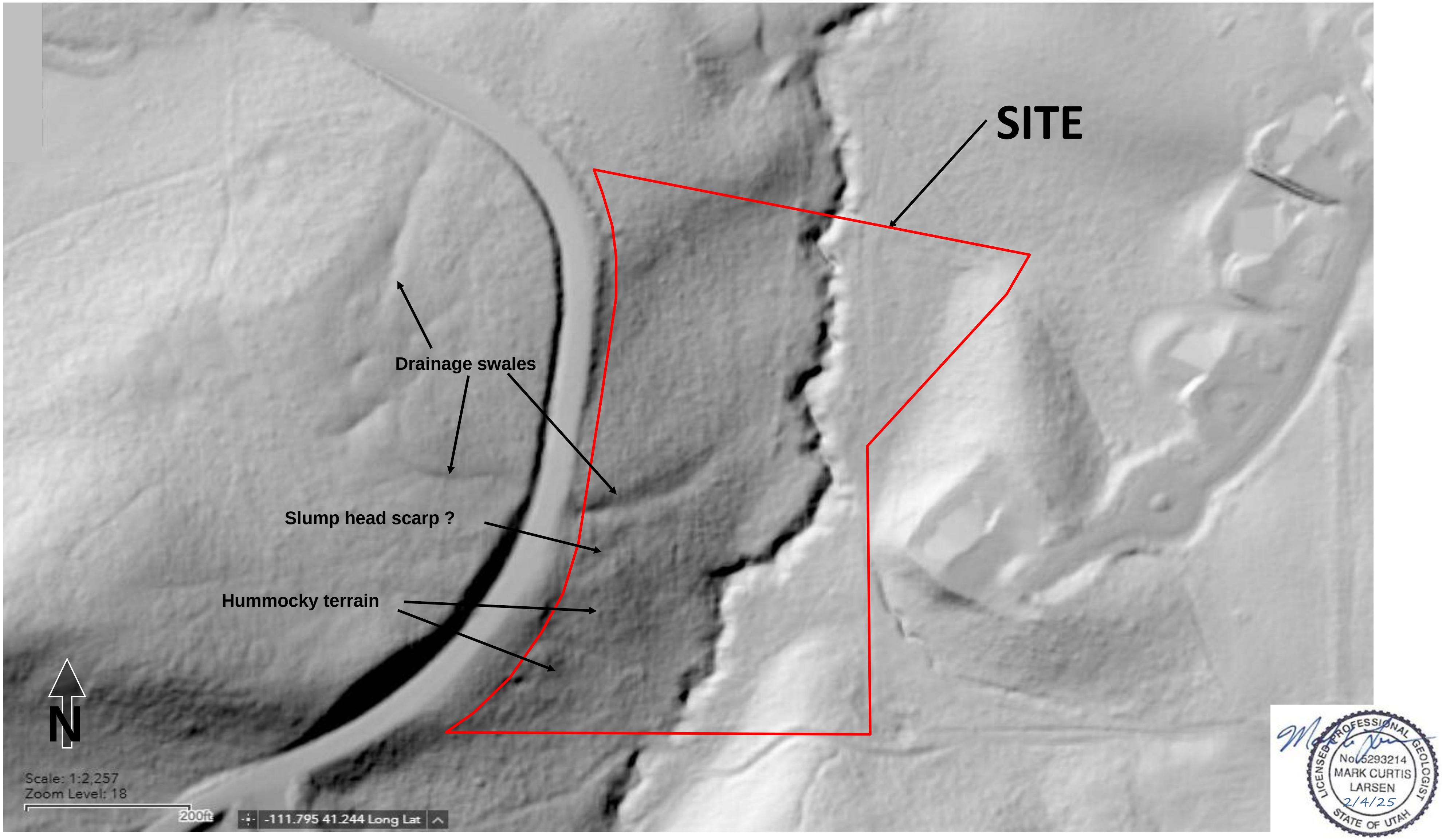
Ql - Lake Bonneville deposits, undivided (upper Pleistocene)

Tn - Norwood Formation (lower Oligocene and upper Eocene)

(Ql/Tn - Lake Bonneville deposits overlying Norwood Formation)

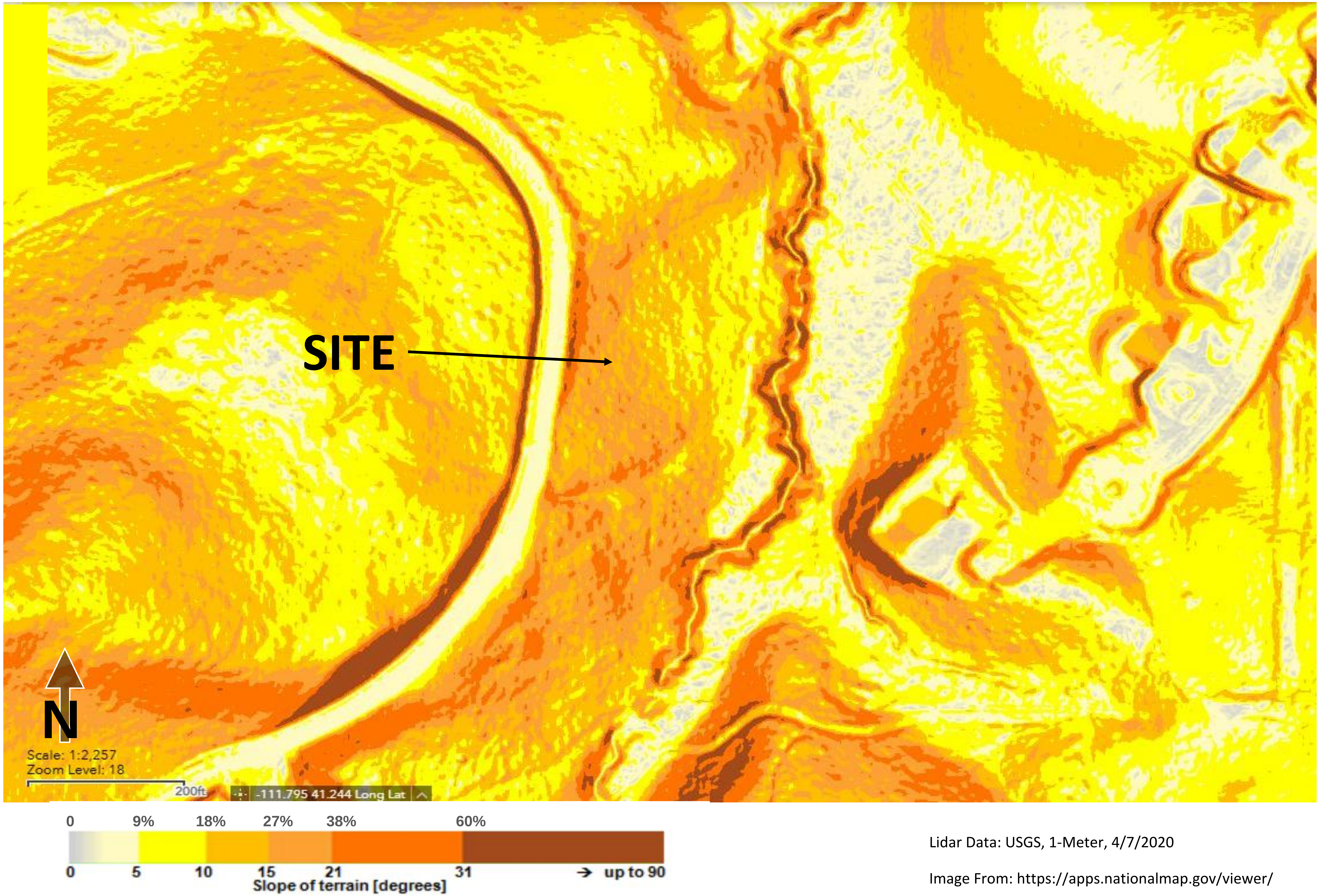
Map From:
King, J.K., Yonkee, W.A., Coogan, J.C., 2008, Interim Geologic Map of the Snow Basin and Part of the Huntsville Quadrangle, Davis, Morgan, and Weber Counties, Utah: Utah Geological Survey Open-File Report 536, for use at 1:24,000 scale.

CMT TECHNICAL SERVICES	Smith Creek Subdivision Approx. 1250 S. Snow Basin Rd., Huntsville, UT	Geologic Map	Date:	4-Feb-2025	Figure: 4
			CMT No.:	23423	



Lidar Data: USGS, 1-Meter, 4/7/2020 Image From: https://apps.nationalmap.gov/viewer/

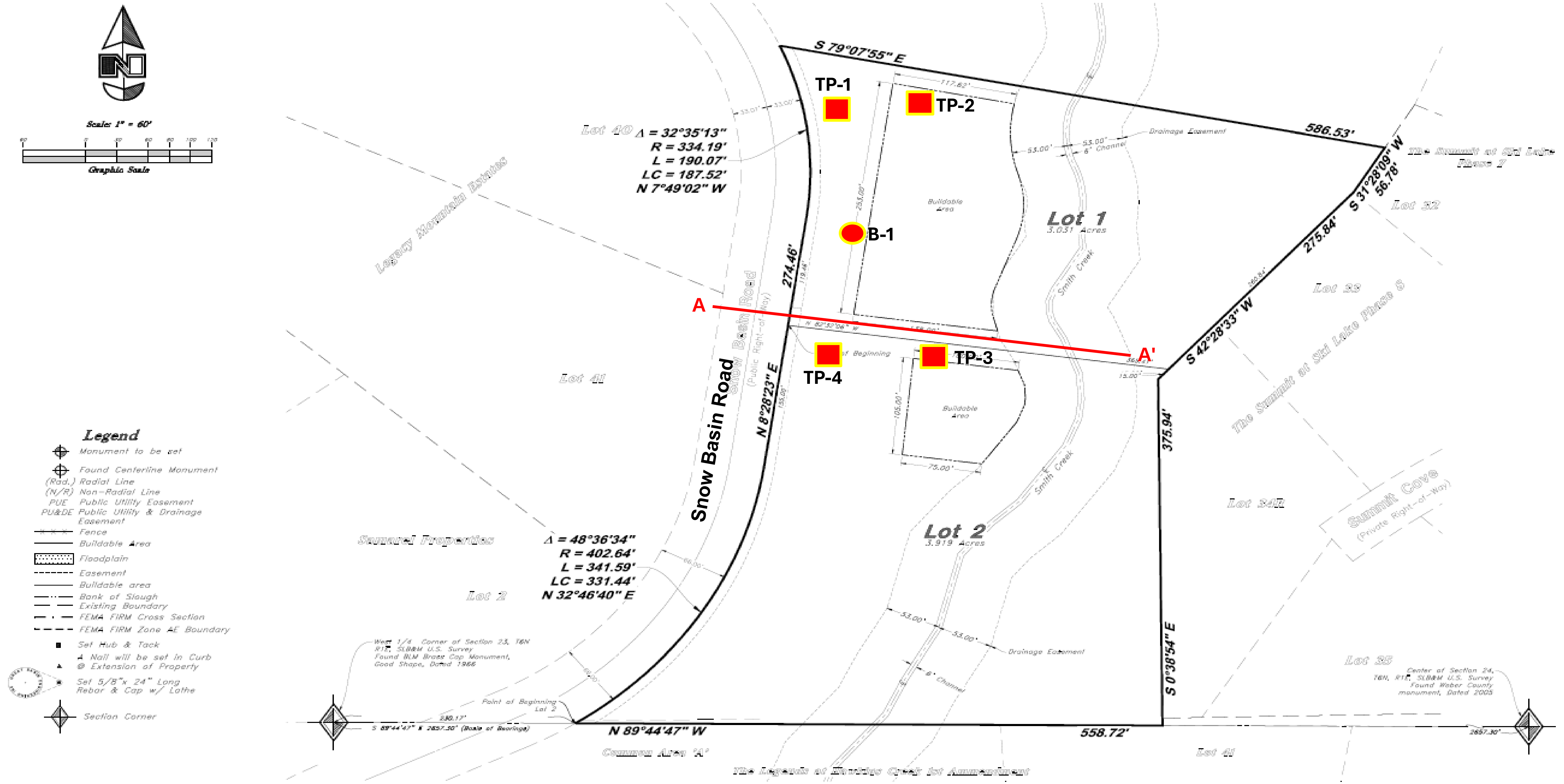
CMT TECHNICAL SERVICES	Smith Creek Subdivision Approx. 1250 S. Snow Basin Rd., Huntsville, UT	Lidar DEM	Date:	4-Feb-2025	Figure: 5
			CMT No.:	23423	



Lidar Data: USGS, 1-Meter, 4/7/2020

Image From: https://apps.nationalmap.gov/viewer/

CMT TECHNICAL SERVICES	Smith Creek Subdivision Approx. 1250 S. Snow Basin Rd., Huntsville, UT	Slope Map	Date:	4-Feb-2025	Figure: 6
			CMT No.:	23423	



A —— A' Line of Geologic Cross-Section

TP-4 Test Pit

B-1 Boring

Site plan provided by Client

CMT TECHNICAL SERVICES	Smith Creek Subdivision Approx. 1250 S. Snow Basin Rd., Huntsville, UT	Site Evaluation	Date:	4-Feb-2025	Figure: 7
			CMT No.:	23423	

Log Scale: 1:60

5 feet

Disturbed Bag Sample ⊕

Geologic Test Pit Log, TP-1

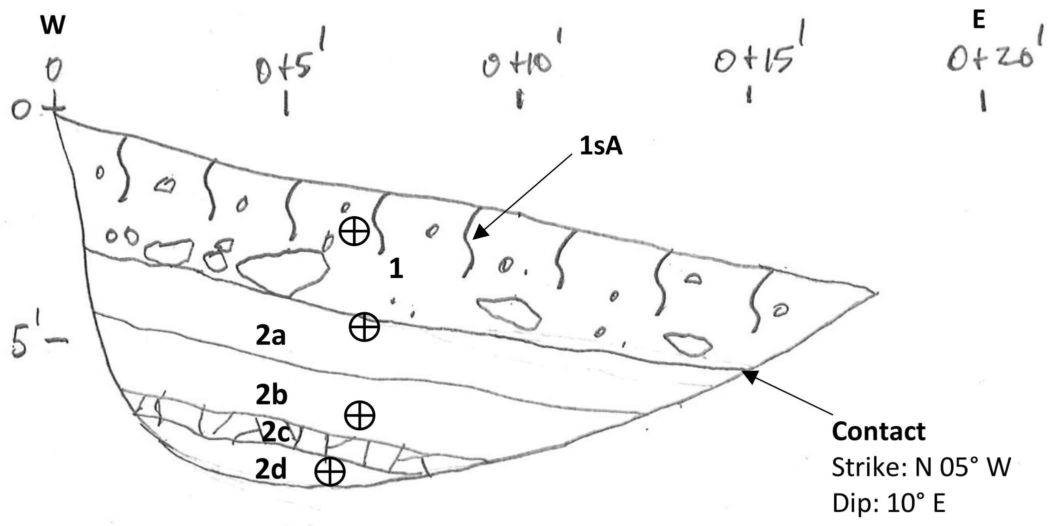
Smith Creek Subdivision, Huntsville, Weber County, UT

Logged by: Mark Larsen, P.G., CMT Senior Geologist

November 7, 2024

LICENSED PROFESSIONAL GEOLOGIST
No. 5293214
MARK CURTIS LARSEN
STATE OF UTAH
1/22/25

TP-1 (North Wall)
Orientation: N 85° E



Soil/Bedrock Units, TP-1

1. **Colluvium** – Likely deposited as slope wash. Lean clay (CL) with silt, gravel (pebbles to a few boulders up to 18± inches in longest dimension), matrix supported, stiff, slightly moist, grayish-brown. Holocene based on stratigraphic relationships.

With:

1sA. **Pedogenic Soil A horizon** formed on Unit 1, lean clay (CL) with silt and gravel, roots/organics, moist, dark brown.

Soil/Bedrock Units, TP-1 (continued)

2. **Norwood Formation**. Correlates with Unit Tn (King and Others, 2008). Lower Oligocene and upper Eocene.

2a. Lean clay (CL), trace gravel, massive, stiff, slightly moist, greenish-gray, iron oxide mottling.

2b. Lean clay (CL), thinly bedded and blocky, stiff, slightly moist, grayish-brown.

2c. Sandy welded ash bed, hard, dry, light gray, fractured with no apparent dominant fracture set.

2d. Lean clay (CL) with sand, thinly bedded and blocky, stiff, slightly moist, grayish-brown.

No groundwater encountered.

Equipment refusal at 7 feet below ground surface.

CMT TECHNICAL SERVICES

Figure 8

CMT Project No.: 23423

Log Scale: 1:60

5 feet

Disturbed Bag Sample ⊕

Geologic Test Pit Log, TP-2

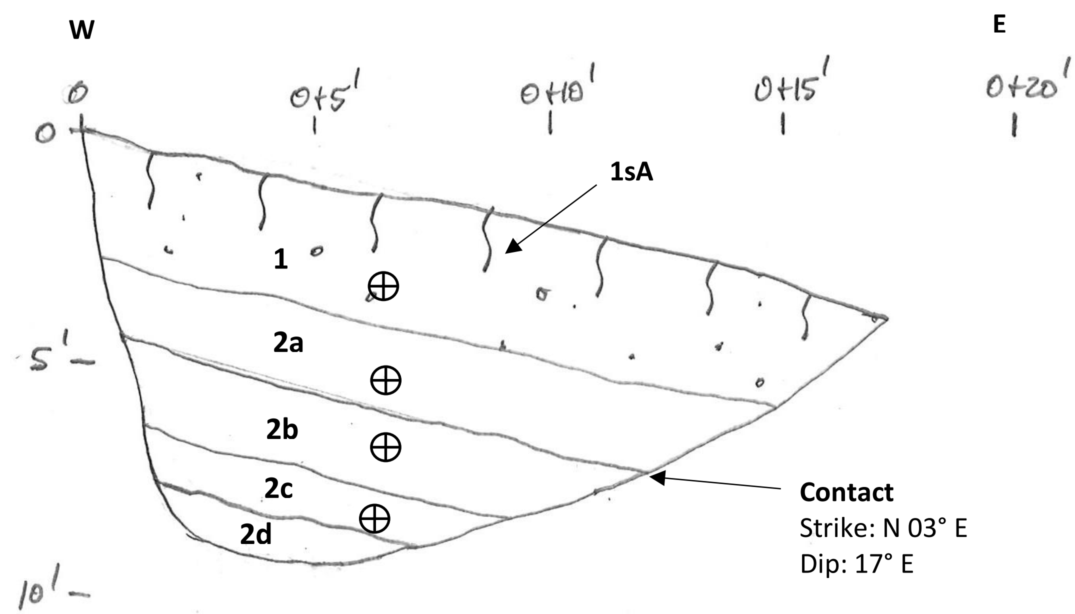
Smith Creek Subdivision, Huntsville, Weber County, UT

Logged by: Mark Larsen, P.G., CMT Senior Geologist

November 7, 2024

LICENSED PROFESSIONAL GEOLOGIST No. 5293214 MARK CURTIS LARSEN 1/22/25 STATE OF UTAH

TP-2 (North Wall)
Orientation: N 80° E



Soil/Bedrock Units, TP-2

1. **Colluvium** – Likely deposited as slope wash. Lean clay (CL) with minor gravel, stiff, slightly moist, grayish-brown.
   Holocene based on stratigraphic relationships.

With:

1sA. **Pedogenic Soil A horizon** formed on Unit 1, lean clay (CL) with silt and minor gravel, roots/organics, moist, dark brown.

Soil/Bedrock Units, TP-2 (continued)

2. **Norwood Formation**. Correlates with Unit Tn (King and Others, 2008). Lower Oligocene and upper Eocene.

2a. Lean clay (CL) with sand, thinly bedded and blocky, stiff, slightly moist, greenish-gray with iron oxide mottling. Uppermost portion of unit may be a buried soil.

2b. Lean clay (CL), some sand, thinly bedded and blocky, stiff, slightly moist, grayish-brown.

2c. Fat clay (CH), stiff, slightly moist, gray.

2d. Fat clay (CH) with sand, stiff, slightly moist, gray.

No groundwater encountered.

Equipment refusal at 8 feet below ground surface.

Figure 9

CMT Project No.: 23423

CMT TECHNICAL SERVICES

Log Scale: 1:60

5 feet

Disturbed Bag Sample ⊕

Geologic Test Pit Log, TP-3

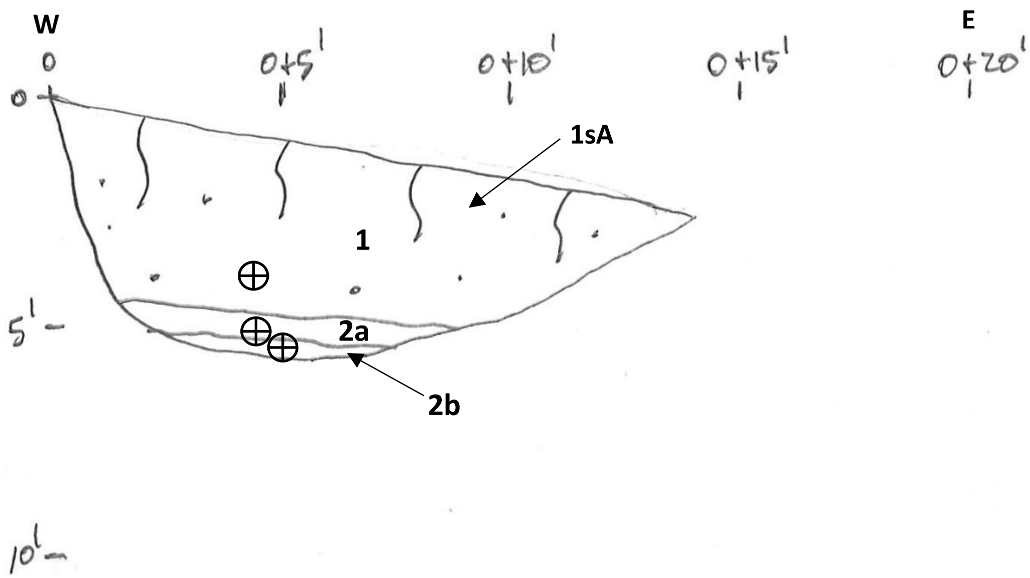
Smith Creek Subdivision, Huntsville, Weber County, UT

Logged by: Mark Larsen, P.G., CMT Senior Geologist

November 7, 2024

No. 5293214 MARK CURTIS LARSEN 1/22/25 LICENSED PROFESSIONAL GEOLOGIST STATE OF UTAH

TP-3 (North Wall)
Orientation: East-West (090°)



<u>**Soil/Bedrock Units, TP-3**</u>

1. **Colluvium** – Likely deposited as slope wash. Fat clay (CH) with sand, trace gravel, stiff, slightly moist, dark brown. Holocene based on stratigraphic relationships.

With:

1sA. **Pedogenic Soil A horizon** formed on Unit 1, lean clay (CL) with trace gravel, roots/organics, moist, dark brown.

2. **Norwood Formation**. Correlates with Unit Tn (King and Others, 2008). Lower Oligocene and upper Eocene.

2a. Fat clay (CH) with trace gravel, very stiff, slightly moist, greenish-gray with iron oxide mottling.

2b. Sandstone (possible reworked ash bed), very hard, slightly moist, light grayish-tan.

No groundwater encountered.

Equipment refusal at 4.5 feet below ground surface.

Figure 10

CMT Project No.: 23423

CMT TECHNICAL SERVICES

Log Scale: 1:60

5 feet

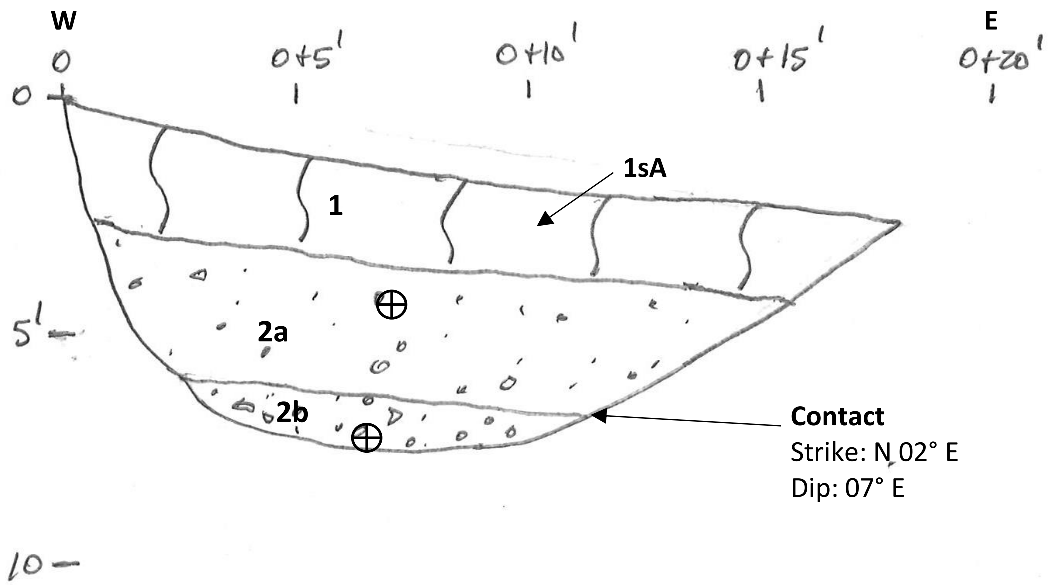
Disturbed Bag Sample ⊕

Geologic Test Pit Log, TP-4

Smith Creek Subdivision, Huntsville, Weber County, UT
Logged by: Mark Larsen, P.G., CMT Senior Geologist
November 7, 2024

No. 5293214 MARK CURTIS LARSEN
LICENSED PROFESSIONAL GEOLOGIST
STATE OF UTAH
1/22/25

TP-4 (North Wall)
Orientation: East-West (090°)



Soil/Bedrock Units, TP-4

1. **Colluvium** – Likely deposited as slope wash. Lean clay (CL), stiff, slightly moist, dark brown.
   Holocene based on stratigraphic relationships.

With:

1sA. **Pedogenic Soil A horizon** enveloping most of Unit 1, lean clay (CL), roots/organics, slightly moist, dark brown.

2. **Norwood Formation**. Correlates with Unit Tn (King and Others, 2008). Lower Oligocene and upper Eocene.

2a. Fat clay (CH) with gravel, very stiff, moist, greenish-gray with iron oxide mottling.

2b. Clayey gravel (GC), matrix supported, clasts up to 12± inches in longest dimension, very dense, moist, greenish-gray.

No groundwater encountered.

Equipment refusal at 6 feet below ground surface.

CMT TECHNICAL SERVICES

Figure 11

CMT Project No.: 23423

Smith Creek Subdivision

Old Snowbasin Road, Huntsville, Utah

Bore Hole Log B-1

Total Depth: 10'
Water Depth: (see Remarks)
Date: 11/7/24
Job #: 23423

Depth (ft)	Soil Description	Sample Type	Sample #	Blows (N)	Blows (N) Total	Moisture (%)	Dry Density(pcf)	Gradation Gravel %	Gradation Sand %	Gradation Fines %	Atterberg LL	Atterberg PL	Atterberg PI
0	Dark Brown Silty Sandy CLAY (CL) with gravel, cobbles and boulders		1	5 5 6	11								
	Olive Green Gravelly Clayey SAND (SC), siltstone, slightly moist; dense		2	12 30 34	64	6	94	41	43	16	40	16	24
4													
	Oilve Green Fat CLAY (CH) with sand, moist; very stiff		3	8 10 12	22	31		0	27	73	76	30	46
8	Olive Green Lean CLAY (CL) with sand and gravel, moist; hard		4	12 24 50/3"	50+			12	15	73	39	14	25
			5	50/4"									
	REFUSAL AT 10.0' ON BEDROCK												
12													
16													
20													
24													
28													

Remarks: Groundwater not encountered during drilling.

Coordinates: 41.243728°, -111.79509°
Surface Elev. (approx): Not Given

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

CMT TECHNICAL SERVICES

Smith Creek Subdivision

Old Snowbasin Road, Huntsville, Utah

Key to Symbols

Date: 11/7/24

Job #: 23423

① Depth (ft)	② GRAPHIC LOG	③ Soil Description	④ Sample Type	⑤ Sample #	Blows(N) ⑥	⑦ Total	⑧ Moisture (%)	⑨ Dry Density(pcf)	Gradation ⑩ Gravel %	Sand %	Fines %	Atterberg ⑪ LL	PL	PI

COLUMN DESCRIPTIONS

① **Depth (ft.):** Depth (feet) below the ground surface (including groundwater depth - see below right).

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

③ **Soil Description:** Description of soils, including Unified Soil Classification Symbol (see below).

④ **Sample Type:** Type of soil sample collected; sampler symbols are explained below-right.

⑤ **Sample #:** Consecutive numbering of soil samples collected during field exploration.

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

⑦ **Total Blows:** Number of blows to advance sampler the 2nd and 3rd 6" increments.

⑧ **Moisture (%):** Water content of soil sample measured in laboratory (percentage of dry weight).

⑨ **Dry Density (pcf):** The dry density of a soil measured in laboratory (pounds per cubic foot).

⑩ **Gradation:** Percentages of Gravel, Sand and Fines (Silt/Clay), from lab test results of soil passing No. 4 and No. 200 sieves.

⑪ **Atterberg:** Individual descriptions of Atterberg Tests are as follows:

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

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

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

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

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

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

UNIFIED SOIL CLASSIFICATION SYSTEM (USCS)

MAJOR DIVISIONS			USCS SYMBOLS	②	TYPICAL DESCRIPTIONS
COARSE-GRAINED SOILS More than 50% of material is larger than No. 200 sieve size.	**GRAVELS** The coarse fraction retained on No. 4 sieve.	**CLEAN GRAVELS** (< 5% fines)	GW		Well-Graded Gravels, Gravel-Sand Mixtures, Little or No Fines
			GP		Poorly-Graded Gravels, Gravel-Sand Mixtures, Little or No Fines
		GRAVELS WITH FINES (≥ 12% fines)	GM		Silty Gravels, Gravel-Sand-Silt Mixtures
			GC		Clayey Gravels, Gravel-Sand-Clay Mixtures
	SANDS The coarse fraction passing through No. 4 sieve.	**CLEAN SANDS** (< 5% fines)	SW		Well-Graded Sands, Gravelly Sands, Little or No Fines
			SP		Poorly-Graded Sands, Gravelly Sands, Little or No Fines
		SANDS WITH FINES (≥ 12% fines)	SM		Silty Sands, Sand-Silt Mixtures
			SC		Clayey Sands, Sand-Clay Mixtures
FINE-GRAINED SOILS More than 50% of material is smaller than No. 200 sieve size.	**SILTS AND CLAYS** Liquid Limit less than 50%		ML		Inorganic Silts and Very Fine Sands, Silty or Clayey Fine Sands or Clayey Silts with Slight
			CL		Inorganic Clays of Low to Medium Plasticity, Gravelly Clays, Sandy Clays, Silty Clays, Lean
			OL		Organic Silts and Organic Silty Clays o f Low Plasticity
	SILTS AND CLAYS Liquid Limit greater than 50%		MH		Inorganic Silts, Micacious or Diatomacious Fine Sand or Silty Soils with Plasticity (Elastic Silts)
			CH		Inorganic Clays of High Plasticity, Fat Clays
			OH		Organic Silts and Organic Clays of Medium to High Plasticity
HIGHLY ORGANIC SOILS			PT		Peat, Humus, Swamp Soils with High Organic Contents

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

SAMPLER SYMBOLS

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

WATER SYMBOL

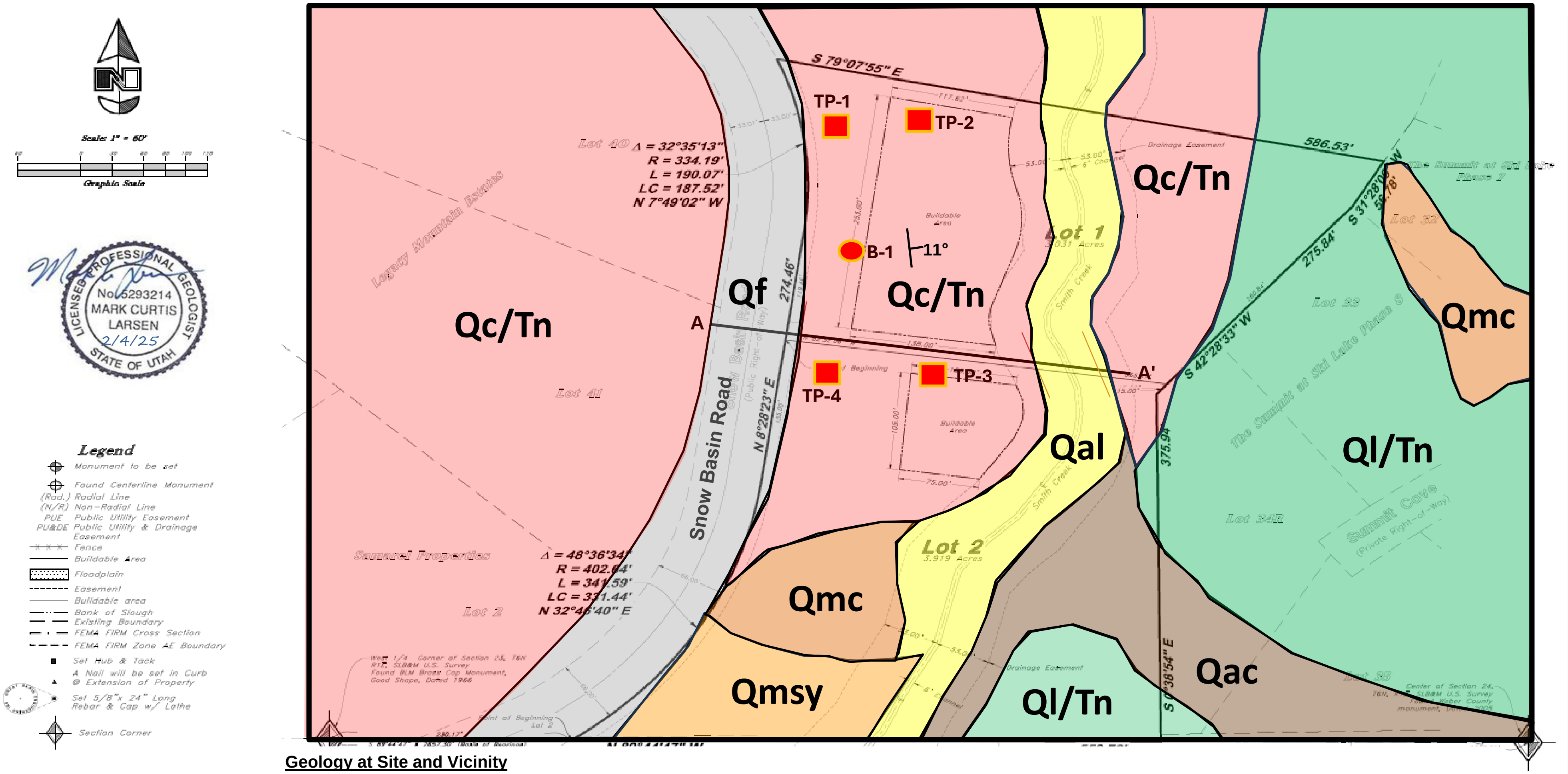
- Encountered Water Level
- Measured Water Level

(see Remarks on Logs)

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

CMT TECHNICAL SERVICES

Figure: 13



Geology at Site and Vicinity

Qf - Fill (Historical)
Qmsy - Landslide and slump deposits (Holocene)
Qal - Stream Alluvium (Holocene)
Qac - Alluvium and colluvium (Holocene and Pleistocene)
Qc - Colluvium (Holocene and Pleistocene)
Qms - Landslide and slump deposits (likely Holocene and/or upper Pleistocene)
Qmc - Landslide and slump, and colluvial deposits, undivided (Holocene and Pleistocene)
Ql - Lake Bonneville deposits, undivided (upper Pleistocene)

Ql/Tn - Lake Bonneville deposits overlying Norwood Formation

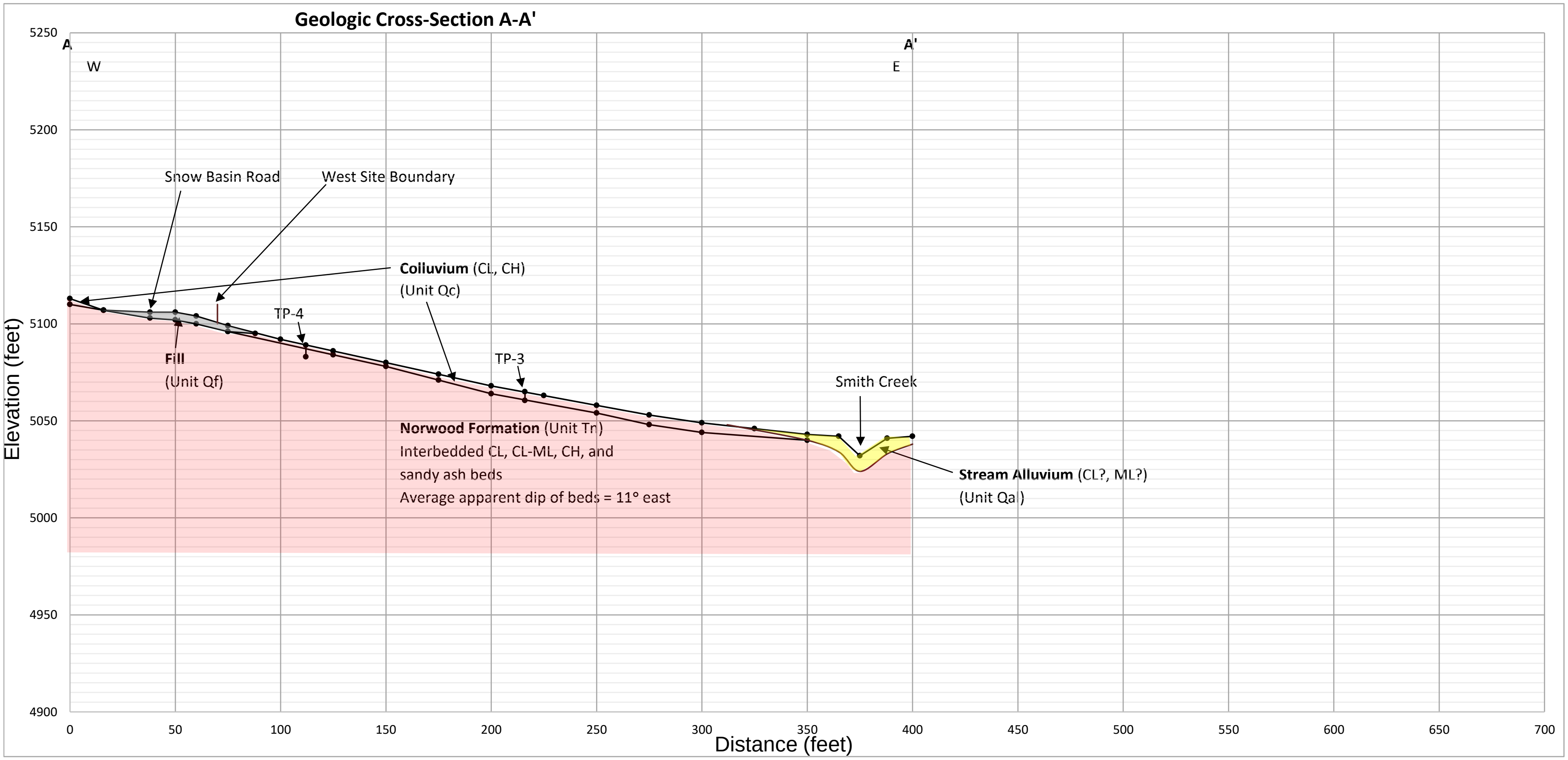
Qc/Tn - Colluvium overlying Norwood Formation

40° Strike and dip of bedding

A —— **A'** Line of Geologic Cross-Section

Site plan provided by Client

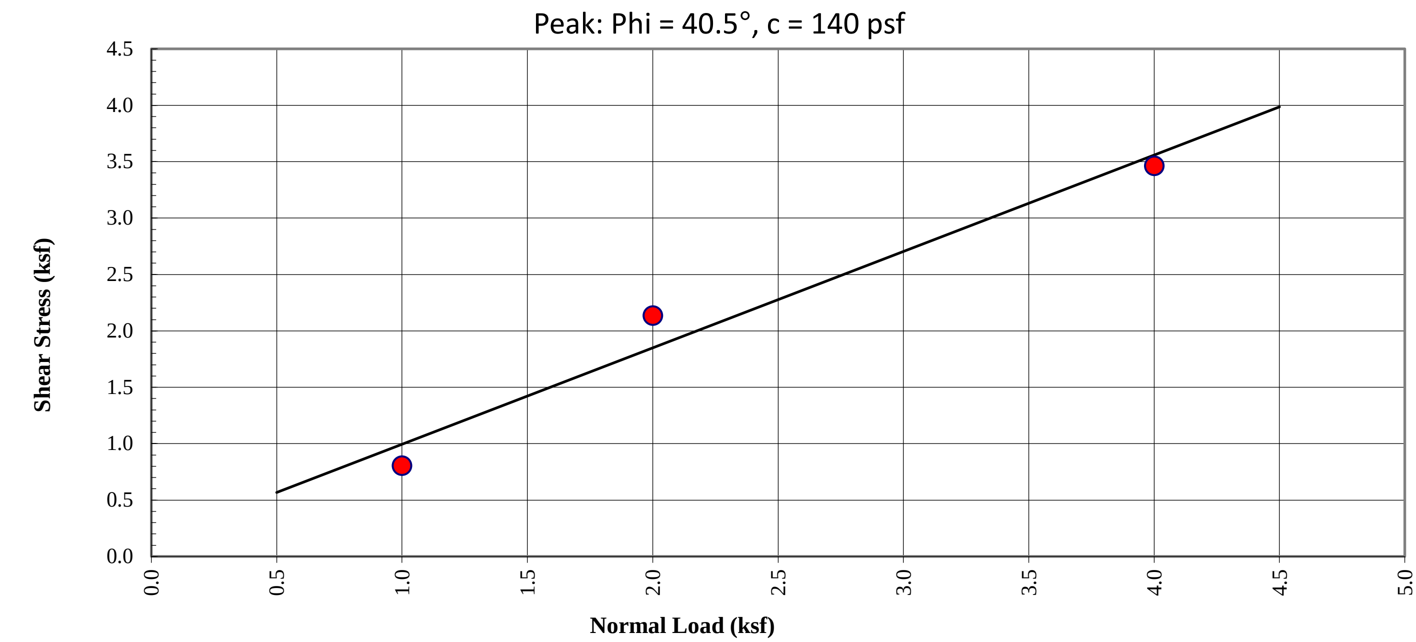
CMT TECHNICAL SERVICES	Smith Creek Subdivision Approx. 1250 S. Snow Basin Rd., Huntsville, UT	Site Geologic Map	Date: 4-Feb-2025 CMT No.: 23423	Figure: 14

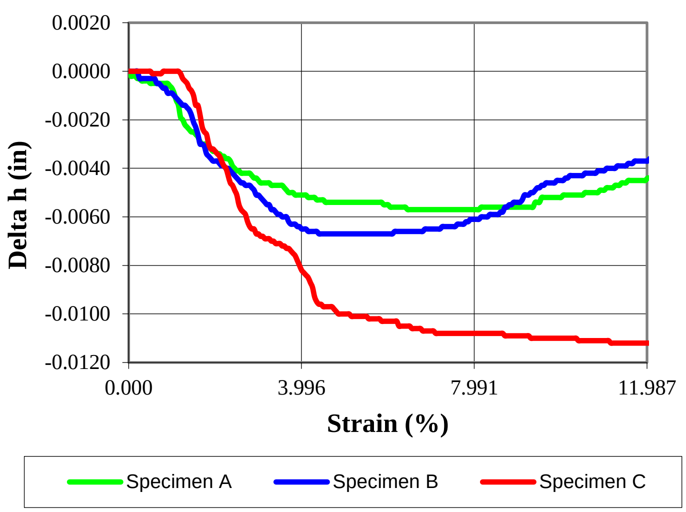


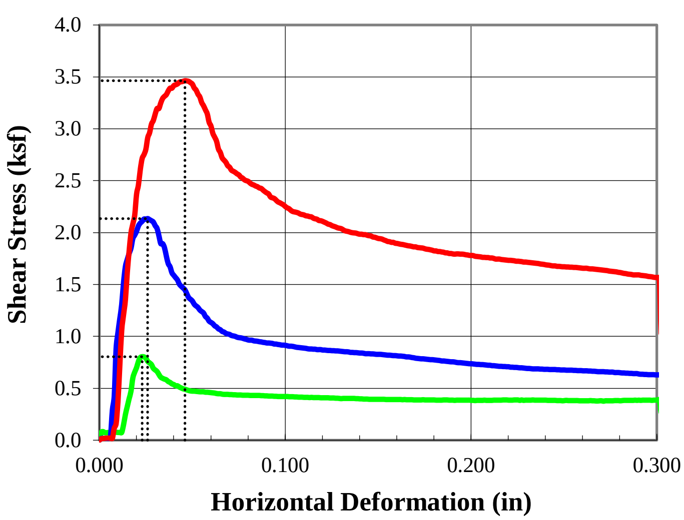
LICENSED PROFESSIONAL GEOLOGIST
No. 5293214
MARK CURTIS
LARSEN
2/4/25
STATE OF UTAH

CMT TECHNICAL SERVICES	Smith Creek Subdivision Approx. 1250 S. Snow Basin Rd., Huntsville, UT	Geologic Cross-Section A-A'	Date:	4-Feb-2025	Figure:
			CMT No.:	23423	15

Direct Shear Test (ASTM D3080)







	Specimen		
Initial	**A**	**B**	**C**
Moisture (%)	31.64	31.64	31.64
Dry Density (pcf)	90.00	90.00	90.00
Void Ratio	0.84	0.84	0.84
Saturation (%)	100.13	100.13	100.13
Diameter (in)	2.00	2.00	2.00
Height (in)	1.00	1.00	1.00

Final	**A**	**B**	**C**
Moisture (%)	38.46	38.46	38.46
Dry Density (pcf)	91.96	92.39	88.19
Void Ratio	0.80	0.79	0.88
Saturation (%)	100.0	100.0	100.0
Diameter (in)	2.00	2.00	2.00
Height (in)	1.00	1.00	1.00
Normal Load (ksf)	1.00	2.00	4.00
Shear Stress (ksf)(@Peak)	0.80	2.13	3.46
Peak Strain (%)	1.15	1.30	2.30
Rate (in/min)	0.0024	0.0024	0.0024
Peak Deformation (in.)	0.023	0.026	0.046

Sample Information	
Test Pit Number:	B-1 @ 5
Sample Number:	
Depth:	5 ft
Sample Type:	Remolded
Description:	Fat CLAY (CH) with sand
Test Type:	Consolidated - Drained

Smith Creek Subdivision

-

CMT TECHNICAL SERVICES

Lab Data

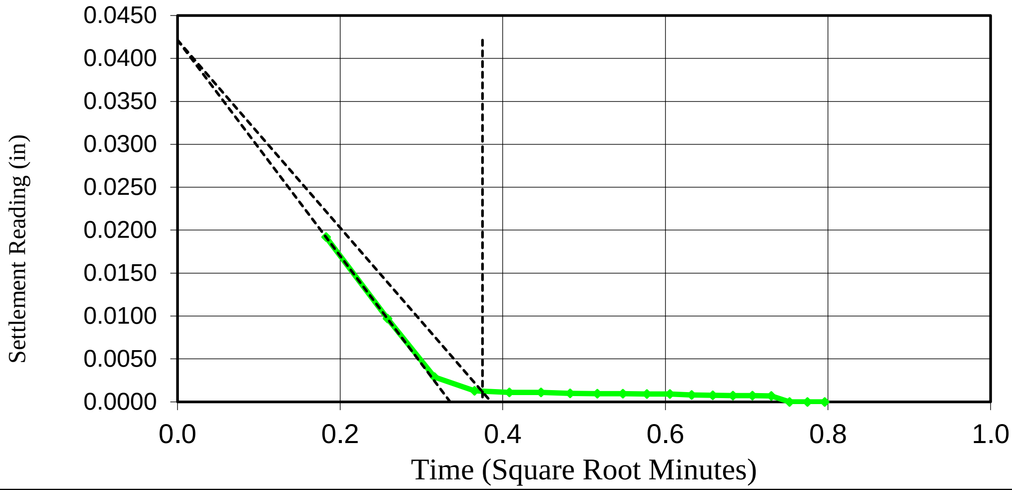
Date: 6-Feb-25

Job # 23423

Figure: 16A

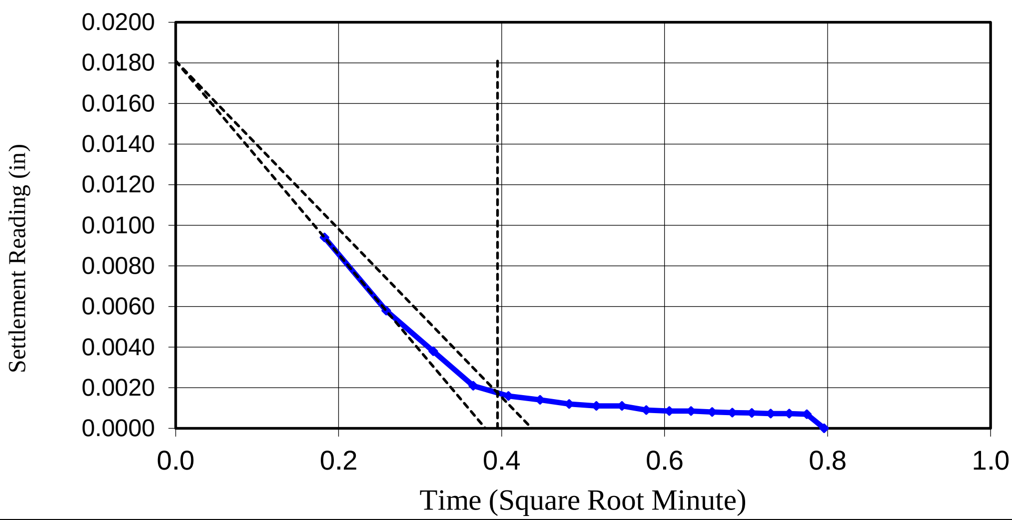
Direct Shear Test (ASTM D3080)

Consolidation Graph Specimen A



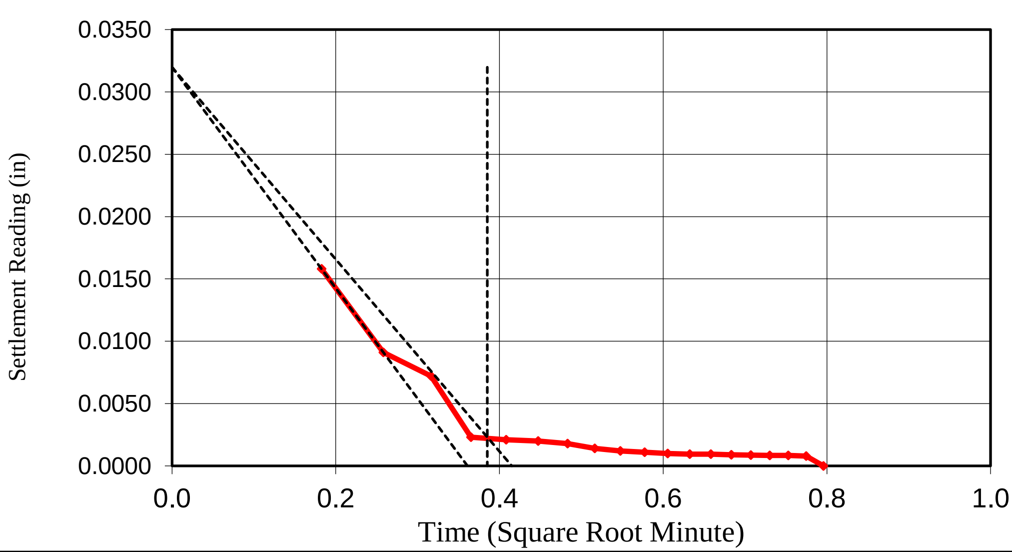
sqrt(t_{90}) =	0.375	
t_{90} =	0.1	min.
$t_f = 50\ t_{90} / 4.28$ =	1.6	min.
max $d_r = 0.5 / t_f$ =	0.3044	in./min.
selected d_r =	0.0024	in./min.
condition =	Drained	

Consolidation Graph Specimen B



sqrt(t_{90}) =	0.395	
t_{90} =	0.2	min.
$t_f = 50\ t_{90} / 4.28$ =	1.8	min.
max $d_r = 0.5 / t_f$ =	0.2743	in./min.
selected d_r =	0.0024	in./min.
condition =	Drained	

Consolidation Graph Specimen C



sqrt(t_{90}) =	0.385	
t_{90} =	0.1	min.
$t_f = 50\ t_{90} / 4.28$ =	1.7	min.
max $d_r = 0.5 / t_f$ =	0.2888	in./min.
selected d_r =	0.0024	in./min.
condition =	Drained	

Smith Creek Subdivision

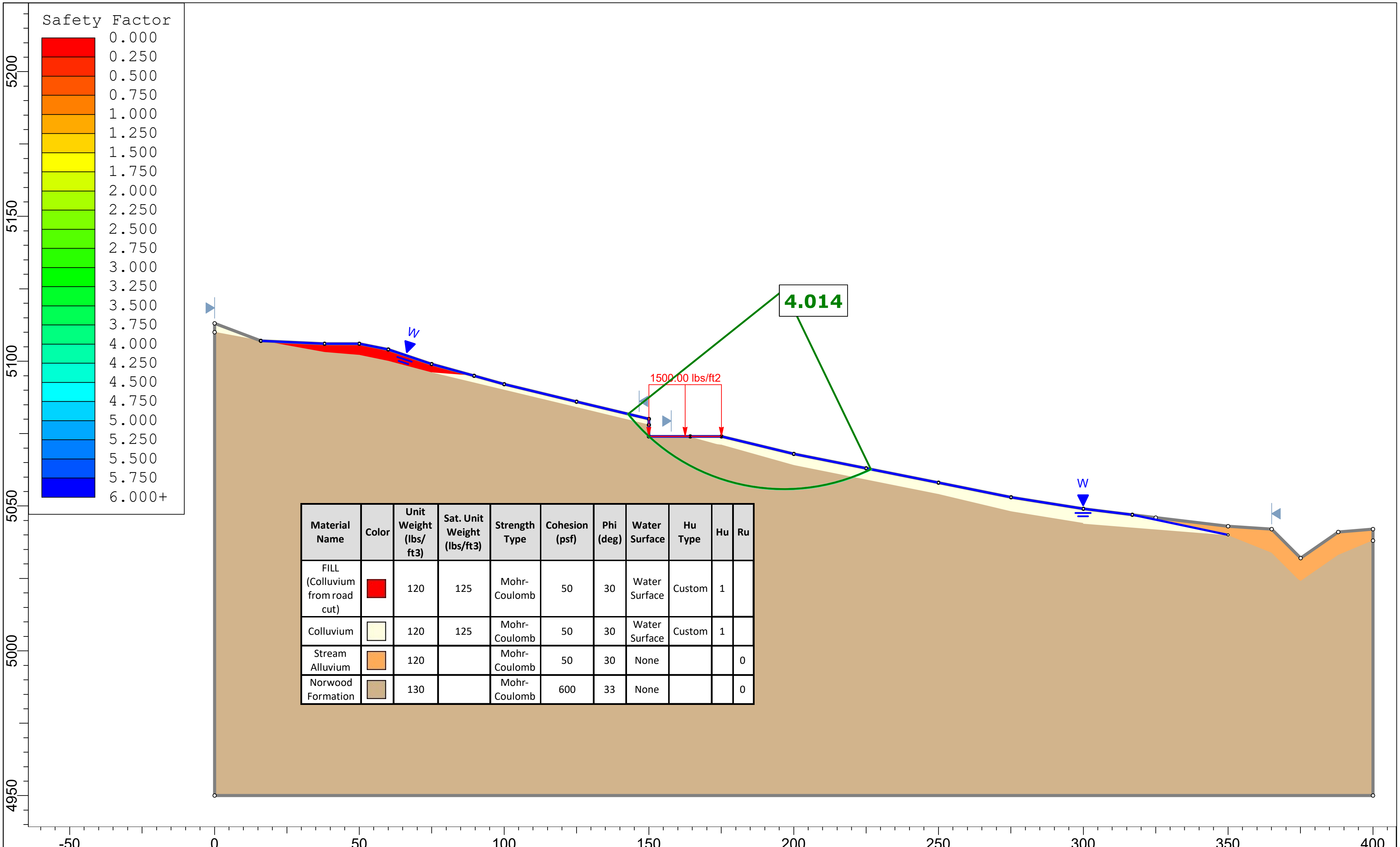
-

CMT TECHNICAL SERVICES

Lab Data

Date: 6-Feb-25
Job # 23423

Figure:
16B



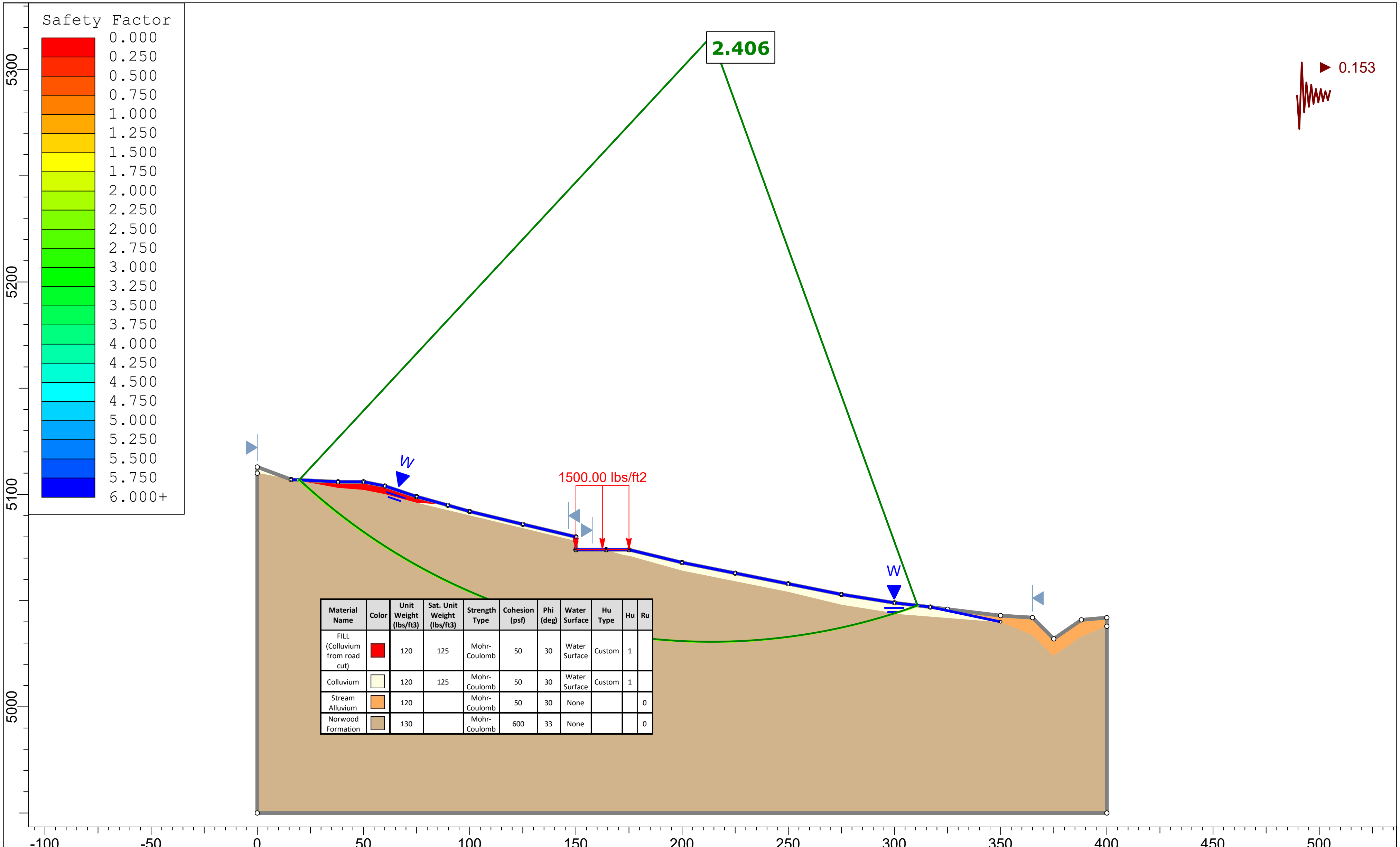
Material Name	Color	Unit Weight (lbs/ft3)	Sat. Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Hu Type	Hu	Ru
FILL (Colluvium from road cut)		120	125	Mohr-Coulomb	50	30	Water Surface	Custom	1	
Colluvium		120	125	Mohr-Coulomb	50	30	Water Surface	Custom	1	
Stream Alluvium		120		Mohr-Coulomb	50	30	None			0
Norwood Formation		130		Mohr-Coulomb	600	33	None			0

CMT TECHNICAL SERVICES

SLIDEINTERPRET 9.020

Project	23423-Smith Creek Subdivision		
Analysis Description	Cross Section A-A', Global Slope Stability		
Drawn By	J. Egbert	*Company*	CMT Technical Services
Date	1/21/2025, 10:13:22 AM	*File Name*	Cross Section A-A' Slope Stability.slmd

Figure
17



Material Name	Color	Unit Weight (lbs/ft3)	Sat. Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Hu Type	Hu	Ru
FILL (Colluvium from road cut)		120	125	Mohr-Coulomb	50	30	Water Surface	Custom	1	
Colluvium		120	125	Mohr-Coulomb	50	30	Water Surface	Custom	1	
Stream Alluvium		120		Mohr-Coulomb	50	30	None			0
Norwood Formation		130		Mohr-Coulomb	600	33	None			0

CMT TECHNICAL SERVICES

SLIDEINTERPRET 9.020

Project
23423-Smith Creek Subdivision

Analysis Description
Cross Section A-A', Global Slope Stability

Drawn By
J. Egbert

Company
CMT Technical Services

Date
1/21/2025, 10:13:22 AM

File Name
Cross Section A-A' Slope Stability.slmd

Figure
18

Slide Analysis Information

Cross Section A-A' Slope Stability

Project Summary

File Name:	Cross Section A-A' Slope Stability.slmd
Slide Modeler Version:	9.02
Compute Time:	00h:00m:01.780s
Project Title:	23423-Smith Creek Subdivision
Analysis:	Cross Section A-A', Global Slope Stability
Author:	J. Egbert
Company:	CMT Technical Services
Date Created:	1/21/2025, 10:13:22 AM

General Settings

Units of Measurement:	Imperial Units
Time Units:	days
Permeability Units:	feet/second
Data Output:	Standard
Failure Direction:	Left to Right

Analysis Options

Slices Type:	Vertical
Analysis Methods Used	
	Bishop simplified
	Janbu simplified
Number of slices:	50
Tolerance:	0.005
Maximum number of iterations:	75
Check malpha < 0.2:	Yes
Create Interslice boundaries at intersections with water tables and piezos:	Yes
Initial trial value of FS:	1
Steffensen Iteration:	Yes

Groundwater Analysis

Groundwater Method:	Water Surfaces
Pore Fluid Unit Weight [lbs/ft3]:	62.4
Use negative pore pressure cutoff:	Yes
Maximum negative pore pressure [psf]:	0
Advanced Groundwater Method:	None

Surface Options

Surface Type:	Circular
Search Method:	Auto Refine Search
Divisions along slope:	20
Circles per division:	10
Number of iterations:	10
Divisions to use in next iteration:	50%
Composite Surfaces:	Disabled
Minimum Elevation:	Not Defined
Minimum Depth:	Not Defined
Minimum Area:	Not Defined
Minimum Weight:	Not Defined

Seismic Loading

Advanced seismic analysis:	No
Staged pseudostatic analysis:	No
Seismic Load Coefficient (Horizontal):	0.153

Loading

1 Distributed Load present

Distributed Load 1	
Distribution:	Constant
Magnitude [psf]:	1500
Orientation:	Normal to boundary

Materials

FILL (Colluvium from road cut)	
Color	
Strength Type	Mohr-Coulomb
Unsaturated Unit Weight [lbs/ft3]	120
Saturated Unit Weight [lbs/ft3]	125
Cohesion [psf]	50
Friction Angle [deg]	30
Water Surface	Water Table
Hu Value	1
Colluvium	
Color	
Strength Type	Mohr-Coulomb
Unsaturated Unit Weight [lbs/ft3]	120
Saturated Unit Weight [lbs/ft3]	125
Cohesion [psf]	50
Friction Angle [deg]	30
Water Surface	Water Table
Hu Value	1
Stream Alluvium	
Color	
Strength Type	Mohr-Coulomb
Unit Weight [lbs/ft3]	120
Cohesion [psf]	50
Friction Angle [deg]	30
Water Surface	None
Ru Value	0
Norwood Formation	
Color	
Strength Type	Mohr-Coulomb
Unit Weight [lbs/ft3]	130
Cohesion [psf]	600
Friction Angle [deg]	33
Water Surface	None
Ru Value	0

Global Minimums

Method: bishop simplified

FS	2.406230
Center:	213.751, 5315.683
Radius:	285.057
Left Slip Surface Endpoint:	19.748, 5106.830
Right Slip Surface Endpoint:	310.900, 5047.692
Resisting Moment:	2.54406e+08 lb-ft
Driving Moment:	1.05728e+08 lb-ft
Total Slice Area:	8341.97 ft2
Surface Horizontal Width:	291.152 ft
Surface Average Height:	28.6516 ft

Method: janbu simplified

FS	2.255910
Center:	203.766, 5250.095
Radius:	227.583
Left Slip Surface Endpoint:	27.211, 5106.490
Right Slip Surface Endpoint:	308.400, 5047.992
Resisting Horizontal Force:	958561 lb
Driving Horizontal Force:	424911 lb
Total Slice Area:	9811.95 ft2
Surface Horizontal Width:	281.189 ft
Surface Average Height:	34.8945 ft

Slice Data

Global Minimum Query (bishop simplified) - Safety Factor: 2.40623

Slice Number	Width [ft]	Weight [lbs]	Angle of Slice Base [deg]	Base Material	Base Cohesion [psf]	Base Friction Angle [deg]	Shear Stress [psf]	Shear Strength [psf]	Base Normal Stress [psf]	Pore Pressure [psf]	Effective Normal Stress [psf]	Base Vertical Stress [psf]	Effective Vertical Stress [psf]
1	0.686904	25.9625	-42.7948	FILL (Colluvium from road cut)	50	30	20.7286	49.8778	18.6083	18.8201	-0.211781	37.7997	18.9796
2	5.94884	2396.8	-41.8975	Norwood Formation	600	33	288.296	693.707	144.296	0	144.296	402.947	402.947
3	5.94884	6178.76	-40.3103	Norwood Formation	600	33	431.003	1037.09	673.064	0	673.064	1038.71	1038.71
4	5.94884	9744.03	-38.7595	Norwood Formation	600	33	568.3	1367.46	1181.79	0	1181.79	1638.05	1638.05
5	5.94884	13225.5	-37.2418	Norwood Formation	600	33	704.795	1695.9	1687.53	0	1687.53	2223.31	2223.31
6	5.94884	16615.2	-35.7541	Norwood Formation	600	33	839.957	2021.13	2188.36	0	2188.36	2793.13	2793.13
7	5.94884	19345.9	-34.2937	Norwood Formation	600	33	951.867	2290.41	2603	0	2603	3252.16	3252.16
8	5.94884	21443.7	-32.8583	Norwood Formation	600	33	1040.81	2504.42	2932.55	0	2932.55	3604.8	3604.8
9	5.94884	22941.8	-31.4457	Norwood Formation	600	33	1107.44	2664.75	3179.43	0	3179.43	3856.62	3856.62
10	5.94884	24157.6	-30.0542	Norwood Formation	600	33	1163.65	2800.01	3387.72	0	3387.72	4061.02	4061.02
11	5.94884	25307.7	-28.682	Norwood Formation	600	33	1217.74	2930.17	3588.14	0	3588.14	4254.33	4254.33
12	5.94884	26470.1	-27.3275	Norwood Formation	600	33	1272.77	3062.58	3792.04	0	3792.04	4449.74	4449.74
13	5.94884	27499.1	-25.9894	Norwood Formation	600	33	1322.9	3183.21	3977.79	0	3977.79	4622.71	4622.71
14	5.94884	28395.8	-24.6664	Norwood Formation	600	33	1368.07	3291.9	4145.16	0	4145.16	4773.44	4773.44
15	5.94884	29199	-23.3573	Norwood Formation	600	33	1409.77	3392.22	4299.64	0	4299.64	4908.46	4908.46
16	5.94884	30008.1	-22.061	Norwood Formation	600	33	1451.97	3493.78	4456.02	0	4456.02	5044.46	5044.46
17	5.94884	30708.9	-20.7764	Norwood Formation	600	33	1490.01	3585.3	4596.96	0	4596.96	5162.26	5162.26
18	5.94884	31292.1	-19.5027	Norwood Formation	600	33	1523.42	3665.69	4720.75	0	4720.75	5260.3	5260.3
19	5.94884	31760.7	-18.239	Norwood Formation	600	33	1552.24	3735.04	4827.54	0	4827.54	5339.06	5339.06
20	5.94884	32117.2	-16.9843	Norwood Formation	600	33	1576.51	3793.44	4917.47	0	4917.47	5398.98	5398.98
21	5.94884	32363.8	-15.7381	Norwood Formation	600	33	1596.25	3840.94	4990.62	0	4990.62	5440.45	5440.45

22	5.94884	32502.8	-14.4994	Norwood Formation	600	33	1611.48	3877.6	5047.05	0	5047.05	5463.79	5463.79
23	5.94884	31554.5	-13.2676	Norwood Formation	600	33	1664.13	4004.28	5242.13	0	5242.13	5634.52	5634.52
24	5.94884	28679.4	-12.042	Norwood Formation	600	33	1848.88	4448.82	5926.65	0	5926.65	6321.06	6321.06
25	5.94884	29609.8	-10.822	Norwood Formation	600	33	1899.53	4570.71	6114.35	0	6114.35	6477.46	6477.46
26	5.94884	30422.2	-9.60695	Norwood Formation	600	33	1945.51	4681.35	6284.74	0	6284.74	6614.04	6614.04
27	5.94884	31102.2	-8.39624	Norwood Formation	600	33	1979.27	4762.57	6409.8	0	6409.8	6701.94	6701.94
28	5.94884	31133	-7.1893	Norwood Formation	600	33	1607.09	3867.02	5030.77	0	5030.77	5233.49	5233.49
29	5.94884	30553.1	-5.98556	Norwood Formation	600	33	1590.48	3827.07	4969.25	0	4969.25	5136.01	5136.01
30	5.94884	29875.6	-4.78447	Norwood Formation	600	33	1569.3	3776.1	4890.76	0	4890.76	5022.11	5022.11
31	5.94884	29101.1	-3.58548	Norwood Formation	600	33	1543.51	3714.03	4795.19	0	4795.19	4891.91	4891.91
32	5.94884	28293.4	-2.38806	Norwood Formation	600	33	1515.91	3647.62	4692.92	0	4692.92	4756.14	4756.14
33	5.94884	27513.8	-1.19168	Norwood Formation	600	33	1489.23	3583.44	4594.1	0	4594.1	4625.08	4625.08
34	5.94884	26641.4	0.00417253	Norwood Formation	600	33	1458.04	3508.39	4478.52	0	4478.52	4478.41	4478.41
35	5.94884	25672.9	1.20003	Norwood Formation	600	33	1422.12	3421.94	4345.41	0	4345.41	4315.62	4315.62
36	5.94884	24608.4	2.39641	Norwood Formation	600	33	1381.38	3323.92	4194.47	0	4194.47	4136.66	4136.66
37	5.94884	23447.5	3.59384	Norwood Formation	600	33	1335.75	3214.13	4025.41	0	4025.41	3941.51	3941.51
38	5.94884	22190.1	4.79284	Norwood Formation	600	33	1285.15	3092.36	3837.89	0	3837.89	3730.13	3730.13
39	5.94884	20835.6	5.99395	Norwood Formation	600	33	1229.45	2958.34	3631.54	0	3631.54	3502.45	3502.45
40	5.94884	19382.9	7.19771	Norwood Formation	600	33	1168.53	2811.76	3405.8	0	3405.8	3258.23	3258.23
41	5.94884	17826.6	8.40468	Norwood Formation	600	33	1102.04	2651.77	3159.45	0	3159.45	2996.62	2996.62
42	5.94884	16169.8	9.61542	Norwood Formation	600	33	1030.03	2478.49	2892.61	0	2892.61	2718.11	2718.11
43	5.94884	14412.9	10.8305	Norwood Formation	600	33	952.399	2291.69	2604.97	0	2604.97	2422.77	2422.77
44	5.94884	12558.7	12.0505	Norwood Formation	600	33	869.181	2091.45	2296.62	0	2296.62	2111.07	2111.07
45	5.94884	10728.8	13.2762	Norwood Formation	600	33	786.147	1891.65	1988.96	0	1988.96	1803.47	1803.47
46	5.94884	8854.71	14.508	Norwood Formation	600	33	699.946	1684.23	1669.56	0	1669.56	1488.44	1488.44
47	5.94884	6874.79	15.7467	Norwood Formation	600	33	607.466	1461.7	1326.91	0	1326.91	1155.62	1155.62
48	5.94884	4787.15	16.9931	Norwood Formation	600	33	508.464	1223.48	960.079	0	960.079	804.693	804.693
49	5.43484	2567.7	18.1932	Colluvium	50	30	84.2002	202.605	500.119	235.8	264.319	472.446	236.646
50	5.43484	869.723	19.347	Colluvium	50	30	43.7011	105.155	175.37	79.8379	95.5319	160.026	80.1878

Global Minimum Query (janbu simplified) - Safety Factor: 2.25591

Slice Number	Width [ft]	Weight [lbs]	Angle of Slice Base [deg]	Base Material	Base Cohesion [psf]	Base Friction Angle [deg]	Shear Stress [psf]	Shear Strength [psf]	Base Normal Stress [psf]	Pore Pressure [psf]	Effective Normal Stress [psf]	Base Vertical Stress [psf]	Effective Vertical Stress [psf]
1	1.47726	159.749	-50.5832	FILL (Colluvium from road cut)	50	30	27.4715	61.9732	74.6729	53.935	20.7379	108.097	54.1623
2	5.65766	3528.55	-49.2003	Norwood Formation	600	33	333.984	753.437	236.272	0	236.272	623.2	623.2
3	5.65766	7976.61	-47.0648	Norwood Formation	600	33	512.931	1157.13	857.901	0	857.901	1409.2	1409.2
4	5.65766	12239.7	-45.0118	Norwood Formation	600	33	689.834	1556.2	1472.43	0	1472.43	2162.54	2162.54

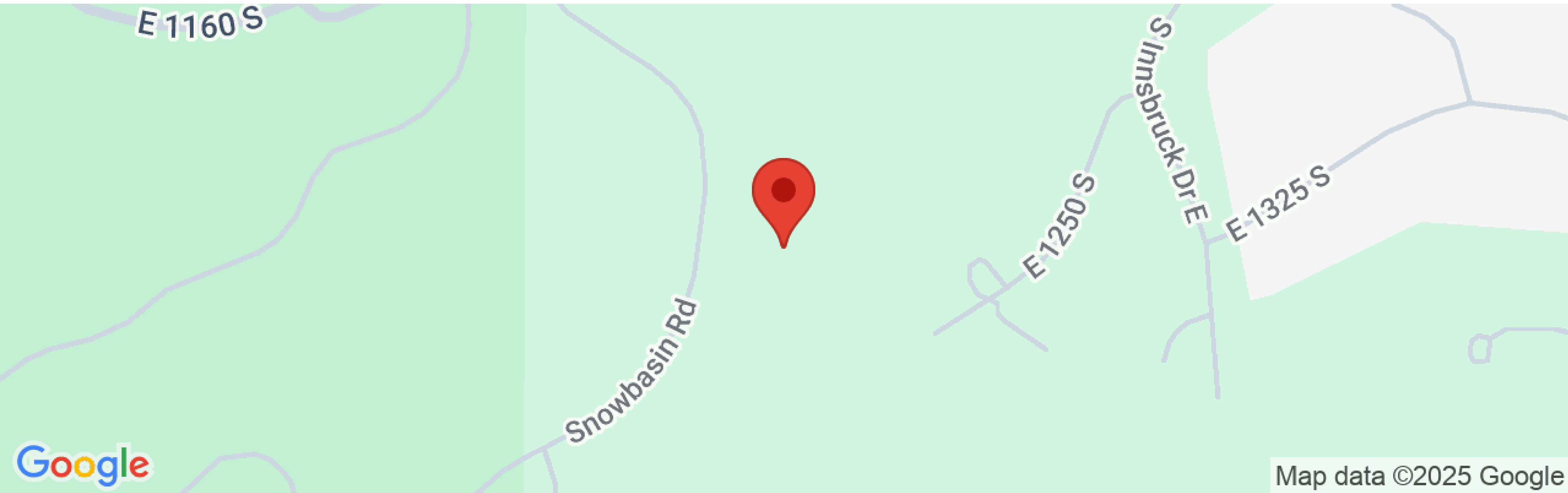
5	5.65766	16227.8	-43.03	Norwood Formation	600	33	860.216	1940.57	2064.29	0	2064.29	2867.3	2867.3
6	5.65766	19394.3	-41.1104	Norwood Formation	600	33	1001	2258.16	2553.34	0	2553.34	3426.89	3426.89
7	5.65766	22018.4	-39.2455	Norwood Formation	600	33	1122.09	2531.34	2974.01	0	2974.01	3890.65	3890.65
8	5.65766	24011.7	-37.429	Norwood Formation	600	33	1218.85	2749.61	3310.1	0	3310.1	4242.96	4242.96
9	5.65766	25720.3	-35.6556	Norwood Formation	600	33	1304.85	2943.62	3608.87	0	3608.87	4544.96	4544.96
10	5.65766	27307.9	-33.9208	Norwood Formation	600	33	1386.66	3128.17	3893.04	0	3893.04	4825.56	4825.56
11	5.65766	28855.9	-32.2207	Norwood Formation	600	33	1467.61	3310.79	4174.25	0	4174.25	5099.19	5099.19
12	5.65766	30236.5	-30.5518	Norwood Formation	600	33	1542.09	3478.82	4433	0	4433	5343.24	5343.24
13	5.65766	31455	-28.9112	Norwood Formation	600	33	1610.14	3632.33	4669.37	0	4669.37	5558.63	5558.63
14	5.65766	32531.5	-27.2961	Norwood Formation	600	33	1672.46	3772.92	4885.87	0	4885.87	5748.95	5748.95
15	5.65766	33578.8	-25.7043	Norwood Formation	600	33	1733.94	3911.61	5099.43	0	5099.43	5934.08	5934.08
16	5.65766	34513.8	-24.1334	Norwood Formation	600	33	1790.83	4039.95	5297.06	0	5297.06	6099.39	6099.39
17	5.65766	35312.6	-22.5817	Norwood Formation	600	33	1841.93	4155.23	5474.58	0	5474.58	6240.61	6240.61
18	5.65766	35979.8	-21.0472	Norwood Formation	600	33	1887.34	4257.67	5632.33	0	5632.33	6358.6	6358.6
19	5.65766	36519.7	-19.5285	Norwood Formation	600	33	1927.14	4347.45	5770.56	0	5770.56	6454.07	6454.07
20	5.65766	36935.9	-18.0238	Norwood Formation	600	33	1961.38	4424.69	5889.51	0	5889.51	6527.7	6527.7
21	5.65766	37231.8	-16.532	Norwood Formation	600	33	1990.12	4489.53	5989.34	0	5989.34	6580.04	6580.04
22	5.65766	37410.2	-15.0516	Norwood Formation	600	33	2013.4	4542.04	6070.2	0	6070.2	6611.63	6611.63
23	5.65766	35198.4	-13.5814	Norwood Formation	600	33	2148.26	4846.27	6538.68	0	6538.68	7057.66	7057.66
24	5.65766	34124.5	-12.1202	Norwood Formation	600	33	2292.2	5170.99	7038.71	0	7038.71	7530.96	7530.96
25	5.65766	34963.2	-10.6671	Norwood Formation	600	33	2349.21	5299.61	7236.77	0	7236.77	7679.26	7679.26
26	5.65766	35669	-9.22084	Norwood Formation	600	33	2400.35	5414.97	7414.41	0	7414.41	7804.07	7804.07
27	5.65766	36236.9	-7.7805	Norwood Formation	600	33	2387.45	5385.87	7369.59	0	7369.59	7695.8	7695.8
28	5.65766	36103.3	-6.3451	Norwood Formation	600	33	2037.65	4596.75	6154.46	0	6154.46	6381.04	6381.04
29	5.65766	35508.4	-4.91368	Norwood Formation	600	33	2022.57	4562.74	6102.08	0	6102.08	6275.96	6275.96
30	5.65766	34808.9	-3.48534	Norwood Formation	600	33	2001.95	4516.23	6030.46	0	6030.46	6152.39	6152.39
31	5.65766	34005.4	-2.05916	Norwood Formation	600	33	1975.73	4457.08	5939.38	0	5939.38	6010.42	6010.42
32	5.65766	33143	-0.634263	Norwood Formation	600	33	1946.12	4390.27	5836.51	0	5836.51	5858.06	5858.06
33	5.65766	32298.3	0.790245	Norwood Formation	600	33	1916.97	4324.52	5735.26	0	5735.26	5708.82	5708.82
34	5.65766	31356.9	2.21524	Norwood Formation	600	33	1882.44	4246.61	5615.3	0	5615.3	5542.48	5542.48
35	5.65766	30311.8	3.64161	Norwood Formation	600	33	1842.06	4155.53	5475.03	0	5475.03	5357.79	5357.79
36	5.65766	29162.5	5.07025	Norwood Formation	600	33	1795.72	4050.98	5314.04	0	5314.04	5154.71	5154.71
37	5.65766	27908.6	6.50205	Norwood Formation	600	33	1743.26	3932.64	5131.81	0	5131.81	4933.13	4933.13
38	5.65766	26549.1	7.93795	Norwood Formation	600	33	1684.52	3800.13	4927.76	0	4927.76	4692.88	4692.88
39	5.65766	25083.1	9.37889	Norwood Formation	600	33	1619.32	3653.04	4701.26	0	4701.26	4433.8	4433.8
40	5.65766	23509.3	10.8258	Norwood Formation	600	33	1547.45	3490.9	4451.59	0	4451.59	4155.67	4155.67
41	5.65766	21823.9	12.2798	Norwood Formation	600	33	1468.53	3312.87	4177.45	0	4177.45	3857.8	3857.8

42	5.65766	20023.6	13.7419	Norwood Formation	600	33	1382.22	3118.17	3877.64	0	3877.64	3539.62	3539.62
43	5.65766	18110.4	15.2131	Norwood Formation	600	33	1288.44	2906.6	3551.85	0	3551.85	3201.47	3201.47
44	5.65766	16081.9	16.6947	Norwood Formation	600	33	1186.83	2677.39	3198.9	0	3198.9	2842.95	2842.95
45	5.65766	13954.3	18.1879	Norwood Formation	600	33	1078.07	2432.03	2821.09	0	2821.09	2466.89	2466.89
46	5.65766	11835.4	19.694	Norwood Formation	600	33	968.04	2183.81	2438.84	0	2438.84	2092.35	2092.35
47	5.65766	9617.29	21.2145	Norwood Formation	600	33	850.459	1918.56	2030.39	0	2030.39	1700.27	1700.27
48	5.65766	7271.4	22.7507	Norwood Formation	600	33	723.38	1631.88	1588.96	0	1588.96	1285.61	1285.61
49	5.65766	4793.68	24.3045	Norwood Formation	600	33	586.177	1322.36	1112.34	0	1112.34	847.616	847.616
50	8.14408	2539.82	26.2289	Colluvium	50	30	71.1265	160.455	346.947	155.633	191.314	311.904	156.271

USGS web services were down for some period of time and as a result this tool wasn't operational, resulting in *timeout* error.
USGS web services are now operational so this tool should work as expected.

Smith Creek

Latitude, Longitude: 41.243469, -111.794608



Date	1/21/2025, 12:23:07 PM
Design Code Reference Document	ASCE7-16
Risk Category	II
Site Class	B - Rock

Type	Value	Description
S_S	0.845	MCE_R ground motion. (for 0.2 second period)
S_1	0.297	MCE_R ground motion. (for 1.0s period)
S_{MS}	0.76	Site-modified spectral acceleration value
S_{M1}	0.238	Site-modified spectral acceleration value
S_{DS}	0.507	Numeric seismic design value at 0.2 second SA
S_{D1}	0.158	Numeric seismic design value at 1.0 second SA

Type	Value	Description
SDC	D	Seismic design category
F_a	0.9	Site amplification factor at 0.2 second
F_v	0.8	Site amplification factor at 1.0 second
PGA	0.373	MCE_G peak ground acceleration
F_{PGA}	0.9	Site amplification factor at PGA
PGA_M	0.336	Site modified peak ground acceleration
T_L	8	Long-period transition period in seconds
SsRT	0.845	Probabilistic risk-targeted ground motion. (0.2 second)
SsUH	0.953	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration
SsD	1.5	Factored deterministic acceleration value. (0.2 second)
S1RT	0.297	Probabilistic risk-targeted ground motion. (1.0 second)
S1UH	0.334	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration.
S1D	0.6	Factored deterministic acceleration value. (1.0 second)
PGAd	0.5	Factored deterministic acceleration value. (Peak Ground Acceleration)
PGA_{UH}	0.373	Uniform-hazard (2% probability of exceedance in 50 years) Peak Ground Acceleration
C_{RS}	0.886	Mapped value of the risk coefficient at short periods
C_{R1}	0.889	Mapped value of the risk coefficient at a period of 1 s
C_V	0.9	Vertical coefficient

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Determination of Pseudostatic Coefficient

Project: Summit at Ski Lakes Lot 15R

Inputs

V_s (fps):	750	(average shear velocity in upper 100')
H (ft):	80	(height of slope)
Mw:	7.09	(Moment Magnitude)
D_a (in.):	2	(allowable deformation)
ε:	0	(median displacment level, usually 0)

$$k = \exp\left[\frac{-a+\sqrt{b}}{0.665}\right] \qquad (2a)$$

where

$$a = 2.83 - 0.566\ \ln(S_a) \qquad (2b)$$

$$b = a^2 - 1.33 \cdot \{\ln(D_a) + 1.10 - 3.04\ \ln(S_a) + 0.244[\ln(S_a)]^2 - 1.5T_s - 0.278(M-7) - \varepsilon\} \qquad (2c)$$

where k_y=yield coefficient; T_s=initial fundamental period of the sliding mass in seconds (in most cases, T_s=4H/V_s, where H =representative height and V_s=average shear wave velocity of the sliding mass); S_a=5% damped elastic spectral acceleration of the site's design ground motion at a period of 1.5T_s in the unit of g (the site is defined by the earth materials below the sliding mass, and if shallow sliding is being evaluated, topographic amplification may need to be included; Bray 2007); M=earthquake's moment magnitude; and ε=normally distributed random variable with zero mean and standard deviation (σ) of 0.66. To eliminate the bias in the model, when T_s<0.05 s, the first term of Eq. (1) should be replaced with −0.22.

For Global Stability:

S_a (g):	0.30969	(spectral acc. @ 1.5*Ts=0.64 sec)
D_a (cm) =	5	
T_s (sec) =	0.42667	(=4H/Vs)
a =	3.49345	
b =	4.2999	
k =	0.118	(pseudostatic coefficient)
½ of PGA_M=	0.187	
⅓ of PGA_M=	0.124	

For Displacement:

Enter k_y :	0.25	g (from stability analysis for FS=1.0)
ln (D) =	-0.13066	
D =	0.87751	cm
D =	0.35101	inches

$$\ln(D) = -1.10 - 2.83\ \ln(k_y) - 0.333[\ln(k_y)]^2 + 0.566\ \ln(k_y)\ln(S_a) + 3.04\ \ln(S_a) - 0.244[\ln(S_a)]^2 + 1.5T_s + 0.278(M-7) \pm \varepsilon \qquad (1)$$

where k_y=yield coefficient; T_s=initial fundamental period of the sliding mass in seconds (in most cases, T_s=4H/V_s, where H =representative height and V_s=average shear wave velocity of the sliding mass); S_a=5% damped elastic spectral acceleration of the site's design ground motion at a period of 1.5T_s in the unit of g (the site is defined by the earth materials below the sliding mass, and if shallow sliding is being evaluated, topographic amplification may need to be included; Bray 2007); M=earthquake's moment magnitude; and ε=normally distributed random variable with zero mean and standard deviation (σ) of 0.66. To eliminate the bias in the model, when T_s<0.05 s, the first term of Eq. (1) should be replaced with −0.22.

Reference: Bray, J.D., & Travasarou, T., "Pseudostatic Coefficient for Use in Simplified Seismic Slope Stability Evaluation," Journal of Geotechnical & Geoenvironmental Engineering, ASCE, September 2009, p 1336-1340.

For Internal Stability (Rockery Walls - Richards & Elms Alternative Procedure):

B (ft):	3	(base width of rockery wall)
PGA =	0.373	(2% PE in 50 years)
F_{pga} =	1	
a_{peak} =	0.373	(PGA * F_{pga})
v_{peak} =	45.506	cm/sec (=122*a_{peak})
α_A =	2.12	(pg. 142)
α_V =	1.65	(pg. 142)
Δ =	3.6	in. (10% of base width)
A_a =	0.3163	(=α_A * a_{peak} / 2.5)
A_v =	0.39415	(=[α_V * v_{peak} / 2.5] / 76.2)
k_h =	0.12856	(seismic coefficient)

$$k_h = A_a\left(\frac{0.2A_v^{\ 2}}{A_a \cdot \Delta}\right)^{0.25}$$

Figure 77. Equation. Determination of horizontal seismic coefficient, k_h (Δ in inches).

Reference: Rockery Design and Construction Guidelines, Publication No. FHWA-CFL/TD-06-006, November 2006

Meyerhof (1963) General Bearing Capacity Equation

$$q_{all} = (cN_cF_{cs}F_{cd}F_{ci} + qN_qF_{qs}F_{qd}F_{qi} + 0.5\gamma BN_\gamma F_{\gamma s}F_{\gamma d}F_{\gamma i})/FS$$

where: *c* = cohesion

q = effective stress at the level of the bottom of the foundation

γ = unit weight of soil

B = width of foundation (= diameter for circular foundation)

Fcs, Fqs, Fγs = shape factors

Fcd, Fqd, Fγd = depth factors

Fci, Fqi, Fγi = load inclination factors

Nc, Nq, Nγ = bearing capacity factors

Friction Angle, ϕ =	30	degrees	*Nq* =	18.40	$= e^{\pi \tan \emptyset} tan^2\left(45 + \frac{\emptyset}{2}\right)$
Cohesion, c =	0	psf			
Effective Unit Weight, γ =	120	pcf	*Nc* =	30.14	$= (N_q - 1)\cot \emptyset$
Longest Wall Footing Length, L =	25	ft			
Load Inclination (from veritcal), β =	0	degrees	*N γ* =	22.40	$= 2(N_q + 1)\tan \emptyset$
Factor of Safety, FS =	3				

Summary Tables

Wall Footing Allowable Bearing Capacity, q_{all} (ksf)

Footing Depth, D (ft)	Structural Fill Depth, z (ft)	Footing Width, B (ft)								
		1.67	2	2.5	3	3.5	4	4.5	5	5.5
2.5	0	2.89	2.86	2.94	2.87	2.82	2.79	2.77	2.76	2.75
4	0	4.53	4.50	4.45	4.42	4.39	4.56	4.50	4.45	4.42
6	0	6.74	6.72	6.69	6.66	6.63	6.61	6.60	6.58	6.58
8	0	8.96	8.95	8.94	8.93	8.92	8.91	8.90	8.90	8.90
2.5	1.5	5.48	5.00	4.70	4.30	4.03	3.84	3.70	3.59	3.51
4	1.5	8.60	7.87	7.13	6.63	6.27	6.28	6.00	5.79	5.63
6	1.5	12.80	11.76	10.70	9.99	9.48	9.09	8.79	8.56	8.37
8	1.5	17.01	15.67	14.30	13.39	12.74	12.25	11.87	11.57	11.33

Square Footing Allowable Bearing Capacity, q_{all} (ksf)

Footing Depth, D (ft)	Structural Fill Depth, z (ft)	Footing Width, B (ft)								
		2.5	3	3.5	4	4.5	5	5.5	6	6.5
2.5	0	4.01	3.87	3.77	3.70	3.64	3.59	3.55	3.52	3.49
4	0	6.27	6.16	6.05	6.25	6.10	5.99	5.89	5.81	5.74
6	0	9.60	9.46	9.33	9.21	9.10	9.00	8.90	9.25	9.09
8	0	12.96	12.81	12.66	12.53	12.39	12.27	12.15	12.04	11.94
2.5	1.5	10.26	8.71	7.69	6.99	6.47	6.07	5.75	5.50	5.29
4	1.5	16.05	13.85	12.36	11.82	10.85	10.12	9.54	9.07	8.69
6	1.5	24.58	21.29	19.04	17.41	16.18	15.20	14.42	14.45	13.77
8	1.5	33.17	28.81	25.84	23.68	22.03	20.74	19.69	18.82	18.09

Summary Tables of Shape, Depth, and Inclination Factors

Wall Footings

Fotting Width, B =	1.67	2	2.5	3	3.5	4	4.5	5	5.5
Fcs =	1.04	1.05	1.06	1.07	1.09	1.10	1.11	1.12	1.13
Fqs =	1.04	1.05	1.06	1.07	1.08	1.09	1.10	1.12	1.13
F γs =	0.97	0.97	0.96	0.95	0.94	0.94	0.93	0.92	0.91
Footing Depth, Df =	2.5								
Fcd =	1.39	1.36	1.40	1.33	1.29	1.25	1.22	1.20	1.18
Fqd =	1.28	1.26	1.29	1.24	1.21	1.18	1.16	1.14	1.13
F γd =	1	1	1	1	1	1	1	1	1
Footing Depth, Df =	4								
Fcd =	1.47	1.44	1.40	1.37	1.34	1.40	1.36	1.32	1.29
Fqd =	1.34	1.32	1.29	1.27	1.25	1.29	1.26	1.23	1.21
F γd =	1	1	1	1	1	1	1	1	1
Footing Depth, Df =	6								
Fcd =	1.52	1.50	1.47	1.44	1.42	1.39	1.37	1.35	1.33
Fqd =	1.38	1.36	1.34	1.32	1.30	1.28	1.27	1.25	1.24
F γd =	1	1	1	1	1	1	1	1	1
Footing Depth, Df =	8								
Fcd =	1.55	1.53	1.51	1.48	1.46	1.44	1.42	1.40	1.39
Fqd =	1.39	1.38	1.37	1.35	1.33	1.32	1.31	1.29	1.28
F γd =	1	1	1	1	1	1	1	1	1

Column Footings

Fotting Width, B =	2.5	3	3.5	4	4.5	5	5.5	6	6.5
Fcs =	1.61	1.61	1.61	1.61	1.61	1.61	1.61	1.61	1.61
Fqs =	1.58	1.58	1.58	1.58	1.58	1.58	1.58	1.58	1.58
F γs =	0.60	0.60	0.60	0.60	0.60	0.60	0.60	0.60	0.60
Footing Depth, Df =	2.5								
Fcd =	1.40	1.33	1.29	1.25	1.22	1.20	1.18	1.17	1.15
Fqd =	1.29	1.24	1.21	1.18	1.16	1.14	1.13	1.12	1.11
F γd =	1	1	1	1	1	1	1	1	1
Footing Depth, Df =	4								
Fcd =	1.40	1.37	1.34	1.40	1.36	1.32	1.29	1.27	1.25
Fqd =	1.29	1.27	1.25	1.29	1.26	1.23	1.21	1.19	1.18
F γd =	1	1	1	1	1	1	1	1	1
Footing Depth, Df =	6								
Fcd =	1.47	1.44	1.42	1.39	1.37	1.35	1.33	1.40	1.37
Fqd =	1.34	1.32	1.30	1.28	1.27	1.25	1.24	1.29	1.27
F γd =	1	1	1	1	1	1	1	1	1
Footing Depth, Df =	8								
Fcd =	1.51	1.48	1.46	1.44	1.42	1.40	1.39	1.37	1.36
Fqd =	1.37	1.35	1.33	1.32	1.31	1.29	1.28	1.27	1.26
F γd =	1	1	1	1	1	1	1	1	1

All Footings

Fci = Fqi =	1
F γi =	1

LATERAL EARTH PRESSURES

	ENTER					
Project:	23423					
Density of water:	62.4	pcf				
Internal Friction Angle of Soil:	30	deg.	=	0.523598776	rad.	0.384900179
Angle of Soil Backfill (from horiz.):	0	deg.	=	0	rad.	
Friction angle of soil/wall interface:	15	deg.	=	0.261799388	rad.	
Angle of back of wall (from VERT.):	0	deg.	=	0	rad.	1.570796327
Angle of front of wall (from VERT.):	0	deg.	=	0	rad.	
Density of soil (above water):	120	pcf				
Horizontal Acceleration:	0.34	g	=>	0.324148631	(theta, radians, for k_v = 0)	
Height of Wall, H:	8	feet				
Non-Rigid (0) or Rigid (1) base of wall:	1					
Wood's Thrust Force, F_p :	1	(~ 1.0 for Poisson's Ratio of 0.3 to 0.4 & L/H>4)				

CALCULATIONS

AT REST K_o =	0.500				
At Rest Pressure =	60	psf/ft above water	=	91	psf/ft below water
Coulomb K_a =	0.301	(Accounts for wall friction)			
Coulomb K_p =	4.977	(Accounts for wall friction)			
Coulomb Active Pressure =	36	psf/ft above water	=	80	psf/ft below water
Coulomb Passive Pressure =	597	psf/ft above water	=	349	psf/ft below water
Rankine K_a =	0.333				
Rankine K_p =	3.000				
Rankine Active Pressure =	40	psf/ft above water	=	82	psf/ft below water
Rankine Passive Pressure =	360	psf/ft above water	=	235	psf/ft below water
Mononobe-Okabe Seismic K_{ae} =	0.61				
Mononobe-Okabe Seismic K_{pe} =	3.50				
M.-O. Seismic Active Pressure =	73	psf/ft above water	=	98	psf/ft below water
M.-O. Seismic Passive Pressure=	419	psf/ft above water	=	264	psf/ft below water
Dynamic Active Pressure ONLY =	33	psf/ft above water	=	18	psf/ft below water
Dynamic Passive Press. ONLY =	59	psf/ft above water	=	85	psf/ft below water
Wood's Seismic At-Rest Dynamic Pressure =	81	psf/ft	$\Delta P_{eq} = \gamma H^2(a_h/g)F_p$		
Yi's Seismic At-Rest Dynamic Pressure =	20.16	psf/ft			
Atik & Sitar Seismic At-Rest Dynamic Pressure =	41.0326	psf/ft			
Ultimate Coefficient of Friction =	0.57735				
Allowable Coefficient of Friction (FS=1.5) =	0.3849				