UPDATED GEOTECHNICAL ENGINEERING AND GEOLOGICAL RECONNAISSANCE STUDY

Proposed Jones Subdivision

About 2600 East North Ogden Canyon Road
Weber County, Utah
CMT PROJECT NO. 12290

FOR:
Mr. Doug Jones
245 West 200 North
Willard, Utah 84340

March 31, 2020

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

CMT ENGINEERING LABORATORIES

March 31, 2020

Mr. Doug Jones
245 West 200 North
Willard, Utah 84340

Subject: Updated Geotechnical Engineering and Geological Reconnaissance Study
Proposed Jones Subdivision
Weber County Tax ID Nos. 220090055; 220090081; 220090044
About 2600 East North Ogden Canyon Road
Weber County, Utah
CMT Project No. 12290

Mr. Jones:

Submitted herewith is the updated report of our geotechnical engineering and geological reconnaissance study for the subject site. This report contains the results of our findings and an interpretation of the results with respect to the available project characteristics. It also contains recommendations to aid in the design and construction of the earth related phases of this project, and has been updated to include stability evaluations for the three lots (instead of one).

On January 9, 2019, CMT Engineering Laboratories (CMT) personnel were on-site and supervised the excavation of 5 test pits extending to depths of about 5.5 to 15 feet below the existing ground surface. Soil samples were obtained during the field operations and subsequently transported to our laboratory for further testing and observation.

Conventional spread and/or continuous footings may be utilized to support the proposed structures, provided the recommendations in this report are followed. A detailed discussion of design and construction criteria is presented in this report.

We appreciate the opportunity to work with you at this stage of the project. CMT offers a full range of Geotechnical Engineering, Geological, Material Testing, Special Inspection services, and Phase I and II Environmental Site Assessments. With 9 offices throughout Utah, Idaho, 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) 870-6730.

Sincerely,
CMT Engineering Laboratories

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CMT ENGINEERING LABORATORIES

TABLE OF CONTENTS

APPENDIX

1.0 INTRODUCTION

1.1 General

CMT Engineering Laboratories (CMT) was retained by Mr. Doug Jones to conduct a design level geotechnical engineering and reconnaissance level geological study for a proposed single-family home to be constructed on the property located at about 2600 North Ogden Canyon Road. This site is located just west of the Liberty area of Weber County, Utah, as shown on **Figure 1, Vicinity Map**; a more detailed aerial coverage of the site is shown on **Figure 2, Site Plan**. The site is located where the North Ogden Divide Canyon Road descends into the Ogden Valley on the west site of the valley. Geological mapping of the site is included on **Figure 3, Geological Mapping**, and slope-terrain information is provided on **Figure 4, LiDAR Analysis**. The locations of the proposed single-family dwelling and our test pits (excavated for our subsurface evaluation) are shown on **Figure 5, Site Evaluation**.

1.2 Objectives, Scope and Authorization

The objectives and scope of our study were planned in discussions between Mr. Jones and Mr. Andrew Harris of CMT Engineering Laboratories (CMT). In general, the objectives of this study were to conduct a design level geotechnical study and reconnaissance level geologic study for the proposed design and construction.

In accomplishing these objectives, our scope of work included the following:

1. Provide geological reconnaissance studies as specified by Weber County Code, Section 108-22 Natural Hazard Areas guidelines and standards (Weber County, 2018). 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 at the site.

3. Provide appropriate foundation and earthwork recommendations as well as geoseismic information to be utilized in the design and construction of the proposed home site including; a field program consisting of the excavating, logging, and sampling of five geotechnical test pits; a laboratory soils testing program; and an office program consisting of the correlation of available data, engineering and geological analyses, and the preparation of this summary report.

1.3 Description of Proposed Construction

The Jones Subdivision site consists of three property parcels comprising an area of 18.17 acres. At this time we understand that Mr. Jones intends to construct a single-family residence on the 5-acre easternmost parcel of the three properties (Tax ID #220090055). The subject parcels and surrounding properties are zoned by Weber County as Forest Zone FV-3 (Forest Valley Zone - 3) land-use zone. According to the Weber County

Code of Ordinances, "*the purpose of the Forest Valley Zone, FV-3 is to provide area for residential development in a forest setting at a low density, as well as to protect as much as possible the naturalistic environment of the development.*" The prescribed minimum building lot area in the FV-3 Zone is 3 acres (excluding cluster type provision areas), with single-family residences included as a permitted use.

The proposed project consists of constructing a single-family residence on the site as shown on Figure 2. The structure is likely to be constructed with a concrete basement level supported on conventional spread and strip footings. Above grade levels will consist of wood framed construction, one to three levels in height. Projected maximum column and wall loads will be on the order of 50,000 pounds and 4,000 pounds per lineal foot, respectively. The approximate location of the septic system drain field for the residence is also shown on Figure 2.

Site development will require a moderate amount of earthwork in the form of site grading. We estimate in general that maximum cuts and fills to achieve design grades will be on the order of 6 to 10 feet. Projected site grading is anticipated to consist primarily of cutting into the existing ground to construct the residence, with very little fill projected for the site. Larger cuts and fills may be required in isolated areas. Final cuts and fills must be designed to maintain stability of the slopes at the site and not steepen the slope greater than 4H:1V (Horizontal:Vertical), and all planned retaining walls will need to be properly engineered.

1.4 Executive Summary

Proposed structures can be supported upon conventional spread and continuous wall foundations established on suitable natural soils or on structural fill extending to suitable natural soils, provided they are constructed outside the "Qms" areas shown on **Figure 5, Site Evaluation** provided in the appendix. The most significant geotechnical/geological aspects of the site are:

1. The site is found to be upon both Holocene age landslide deposits (Qms) and Pleistocene and/or Pliocene age mega-landslide deposits (QTms(ZYp)) as mapped by Utah Geological Survey (UGS) geologists (Coogan and King, 2016). The surface of the site slopes moderately to steeply (31.1 percent) to the south. Groundwater was not observed in the test pits at the time of our study but can be expected during wet seasons. The soils encountered in the test pits were generally fine grained (clay), with bedrock encountered at depth.

2. The natural clayey to sandy soils encountered are suitable for bearing foundations and slabs; collapse and expansive potentials are considered low to negligible for this site.

3. A site-specific slope stability study was performed for each parcel and through the recommended building envelope(s). The stability results indicate that the three lots will be stable for both static and seismic conditions if the homes are sited at or near the cross-section locations shown on **Figure 5**.

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

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

2.0 FIELD EXPLORATION

The site subsurface soil conditions were explored by excavating 5 test pits on the site at the locations shown on **Figure 5, Site Evaluation**. The test pits were excavated using an 8-ton rubber-tired excavator, and extended to depths of approximately 5.5 to 15 feet below the existing ground surface, at which point excavation was either stopped or refused due to encountering bedrock. During the course of the excavating operations, a continuous log of the subsurface conditions encountered was maintained. Within the test pits, bulk samples of the typical soils encountered were obtained for subsequent laboratory testing and examination. The representative soil samples were placed in sealed plastic bags and containers prior to transport to the laboratory.

The collected samples were logged and described in general accordance with ASTM D-2488, packaged, and transported to our laboratory. The soils were classified in the field based upon visual and textural examination. These classifications were supplemented by subsequent inspection and testing in our laboratory. The subsurface conditions encountered in the field exploration are discussed below in **Section 5.4**, and are illustrated on **Figure 6, Log of Test Pit 1, 2 and 3**, and **Figure 7, Log of Test Pit 4 and 5**. Sampling information and other pertinent data and observations are also included on the logs.

When backfilling the test pits, only minimal effort was made to compact the backfill and no compaction testing was performed. Thus, settlement of the backfill in the test pits over time should be anticipated.

3.0 LABORATORY TESTING

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

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

Laboratory test results are presented on **Figures 8 and 9, Direct Shear Test**, and in the following Lab Summary table:

LAB SUMMARY TABLE

Bore Hole	Depth (feet)	Soil Class	Sample Type	Moisture Content (%)	Gradation			Atterberg Limits			Internal Friction Angle (ϕ) and Apparent Cohesion, c
					Grav	Sand	Fines	LL	PL	PI	
TP-1	3	SC-SM	Bag	6	25	50	25	25	19	6	
TP-2	3	SC	Bag	18							ϕ = 31.2°, c = 237 psf
TP-2	5	Bedrock	Bag	10				29	20	9	
TP-3	2	SP-SC	Bag	10	37	52	11				
TP-3	4	SC-SM	Bag	11			21				
TP-4	14.5	SC	Bag	16							ϕ = 33.1°, c = 317 psf
TP-5	2	GC-GM	Bag	10	47	35	18	22	17	5	

4.0 GEOLOGIC & SEISMIC CONDITIONS

4.1 Geologic Setting

The site is located on the western margin of Ogden Valley, on the east side of the Wasatch Range which is marked by the Wasatch fault along its western side. The Wasatch fault is approximately 2.5 miles west of the site, and provides the basis of division between the Middle Rocky Mountain Physiographic Province on the east and the Basin and Range Physiographic Province on the west. The Basin and Range Physiographic Province is characterized by approximately north-south trending valleys and mountain ranges that have been formed by extensional tectonics and displacement along normal faults, and extends from the Wasatch Range on the east to the Sierra Nevada Range on the west (Hunt, 1967).

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

The site is located within a setting of complex geological conditions wherein Pre-Cambrian and Paleozoic rocks were rafted over the same during a series of eastward thrust extensions, the last of which is named the Willard Thrust sheet, that is believed to have moved onto the vicinity during the Cretaceous Sevier orogeny, and occurred approximately 140 million years ago (ma). The thrusting was followed by subsequent uplift and exposure of older pre-Cambrian rocks which now form the crest of Chilly Peak a few miles northwest of the site. This exposure was the result of movement along high-angle faults along the Wasatch fault during the late Tertiary and Quaternary age (Bryant, 1988). The controlling geologic unit for the site consists of the Pleistocene and/or Pliocene age mega-landslide deposits (QTms(ZYp)), that are comprised of Neoproterozoic rocks that moved as a block on the present site location during Pleistocene and/or Pliocene age. Finally, Quaternary stream incision by Chicken Creek to the south of the site has steepened local slopes resulting in Holocene age slope movement (Qms) on parts of the site. The current geological mapping drawn from Coogan and King (2016) of the site is shown on **Figure 3, Geologic Mapping**.

4.2 Surficial Geology

The surficial geology of the site is presented on Figure 3 of this report and has been taken from mapping prepared by Coogan and King (2016). A summary of the mapping units identified on the site vicinity and described by Coogan and King (2016) are paraphrased below in relative age sequence (youngest-top to oldest bottom):

Qafy, Qaf - Alluvial-fan deposits (Holocene and Pleistocene) – Mostly sand, silt, and gravel that is poorly bedded and poorly...variably consolidated; includes debris flows, particularly in drainages and at drainage mouths (fan heads)... with unit **Qafy** being the lowest (youngest) fans and **Qaf** being undivided in age determination...

Qal - Stream alluvium and flood-plain deposits (Holocene and uppermost Pleistocene) – Sand, silt, clay, and gravel in channels, flood plains, and terraces…

Qmdf - Debris- and mud-flow deposits (Holocene and upper and middle? Pleistocene) – Very poorly sorted, clay- to boulder-sized material in unstratified deposits characterized by rubbly surface and debris-flow levees with channels, lobes, and mounding...

Qac - Alluvium and colluvium (Holocene and Pleistocene) – Unsorted to variably sorted gravel, sand, silt, and clay in variable proportions; includes stream and fan alluvium, colluvium, and, locally, mass-movement deposits…

Qms - Landslide deposits (Holocene and upper and middle? Pleistocene) – Poorly sorted clay- to boulder sized material; includes slides, slumps, and locally flows and floods...

Qms?(QTms) - Block landslide and possible block landslide deposits (Holocene and upper and middle? Pleistocene) – Mapped where nearly intact block is visible in landslide (mostly block slide) with stratal strikes and dips that are different from nearby in-place bedrock...comprised of Quaternary and/or Tertiary mega-landslide (Pleistocene and/or Pliocene) – Jumbled mass of formation of Perry Canyon (**ZYp**) with blocks of rock from North Ogden divide…

QTms(ZYp) - Quaternary and/or Tertiary mega-landslide (Pleistocene and/or Pliocene) – Jumbled mass of formation of Perry Canyon (**ZYp**) with blocks of rock from North Ogden divide...

The site is located upon **QTms(ZYp)** Quaternary and/or Tertiary mega-landslide, that have been deposited during Pleistocene and/or Pliocene, within which are surficial swales that contain **Qms** landslide deposits of Holocene and upper and middle(?) Pleistocene age; the **Qms** deposits should be considered presently active.

Thrust faulting associated with the Cretaceous Willard thrust is shown to the south of the site on Figure 3, however this faulting is ancient and is not associated with presently active movement.

4.3 Faulting

Based upon our review of available maps and literature, no active faults are known to pass through or immediately adjacent to the site. The nearest active (Holocene) earthquake fault to the site is the Weber segment of the Wasatch fault zone (UT2351E) which is located 2.5 miles west of the site (Black and others, 2004). Accordingly, fault rupture hazards are not considered present on the site. The northern extent of the Ogden Valley southwestern margin faults (UT2375) terminates 1560 feet to the west of the site, however the most recent movement along this fault is estimated to be pre-Holocene (>15,000 ybp), and presently is not considered an active risk (Black, and others, 1999).

4.4 Seismicity

4.4.1 Site Class

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

4.4.2 Seismic Design Category

The 2014 USGS mapping utilized by the IBC provides values of peak ground, short period and long period accelerations for the Site Class B/C boundary and the Maximum Considered Earthquake (MCE). This Site Class B boundary represents average bedrock values for the Western United States and must be corrected for local soil conditions. The Seismic Design Categories in the International Residential Code (IRC 2018 Table R301.2.2.1.1) are based upon the Site Class as addressed in the previous section. For Site Class C at site grid coordinates of 41.3352 degrees north latitude and -111.8886 degrees west longitude, $\mathbf{S_{DS}}$ is 0.937 and the **Seismic Design Category** is D_2.

4.4.3 Liquefaction

Liquefaction is defined as the condition when saturated, loose, sandy soils lose their support capabilities because of excessive pore water pressure which develops during a seismic event. Clayey soils, even if saturated, will generally not liquefy during a major seismic event.

Liquefaction potential hazards have not been studied or mapped for the Ogden Valley area, but has occurred in other parts of northern Utah (Anderson and others 1994). Liquefaction commonly occurs in saturated non-cohesive soils such as sandy alluvium, which conditions are not found on the site. These conditions indicate that liquefaction of these soils is not likely to occur.

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[1]American Society of Civil Engineers

4.4.4 Tectonic Subsidence

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

4.5 Landslide and Slump Deposits

The surface of the site is located in part on mapped Holocene age landslide deposits (**Qms**), but is primarily underlain by Pleistocene and/or Pliocene age mega-landslide deposits (**QTms(ZYp)**). The Holocene age landslide deposits (**Qms**) should be considered potentially active, and should be avoided for the placement of dwellings. The Pliocene age mega-landslide deposits (**QTms(ZYp)**) are not considered active, but are weakened from the past ancient movement, and should be treated with caution when designing/planning site grading and cuts/fills placement. The location of the proposed dwellings as shown on Figure 5 appear to adequately avoid the **Qms** landslide deposits, and should be suitable for the placement of the proposed residences.

4.6 Other Geologic Hazards

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

4.6.1 Sloping Surfaces

The surface slopes of the site vicinity developed from our LiDAR analysis and shown on Figure 4 range from near-level to over 100-percent. For the area within the three subject parcels, the slope gradients averaged 46.1 percent, with the slope gradient for the easternmost lot averaging 42.2 percent. The limiting steep slope gradients for development considerations according to the Weber County Code is 25 percent (Weber County Code, 2018). Site grading of the two road cuts shown on Figure 2 and Figure 5 traversing the site has taken place since the LiDAR imagery (Figure 4) was flown in 2011, and as such it appears the surface grade for the proposed dwelling location of the east lot has been modified to accommodate the 25-percent restriction.

4.6.2 Alluvial Fan - Debris Flow Processes

Alluvial fan deposits indicative of processes including flash flooding and debris flow hazard do not appear to occur on the site. Debris flow (Qmdf), and alluvial fan (Qafy, Qaf) deposits indicative of processes including flash flooding and debris flow hazard are mapped on lower ground immediately south of the site, and are thus not anticipated to be a hazard to the proposed dwelling location.

4.6.3 Flooding Hazards

Mapping by Federal Emergency Management Agency (FEMA, 2015) is shown on Figure 3. The Zone AE shown on Figure 3, includes the 100-year flood hazard zone as delimited by recent FEMA (2015) studies conducted in the Ogden Valley area. On the basis of the FEMA determination *...mandatory flood insurance purchase requirements and floodplain management standards apply...*for improvements made in the Zone AE area shown on Figure 3. The entirety of the proposed subdivision is shown to be outside the flood zone areas shown on Figure 3.

4.6.4 Rockfall and Avalanche Hazards

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

5.0 SITE CONDITIONS

5.1 Surface Conditions

The site conditions and site geology were interpreted through an integrated compilation of data, including a review of literature and mapping from previous studies conducted in the area (Bryant, 1988; King and McDonald, 2014; and Coogan and King, 2016); photo-geologic analyses of 2012 and 2014 imagery shown on Figure 2; historical stereoscopic imagery flown in 1963; GIS analyses of elevation and geoprocessed LiDAR terrain data as shown on Figure 4; field reconnaissance of the general site area; and the interpretation of the test pits made on the site as part of our field program. Seismic hazards information was developed from United States Geologic Survey (USGS) databases (Peterson and others, 2008).

As shown on Figure 2, the site consists of an area of 18.17 acres in size that is presently vacant and undeveloped, with two road cuts (an upper and lower) traversing the site. The site and surrounding area consist of bedrock-controlled slopes and swales occupied with thick colluvial and slide mass deposits. Alluvial fan and debris flow deposits occupy the lower ground to the south of the site along the Chicken Creek drainage. The topography of the site vicinity consists of moderately steep bedrock-controlled slopes. Vegetative cover at the site consists primarily of dense to clustered scrub oak and maple trees and brush, with few open areas covered with grass, weeds and sage brush.

The site slopes developed from our LiDAR analysis were found to range from level to over 100 percent as shown on Figure 4. For the area within the three parcels the slope gradients averaged 46.1 percent, with the slope gradient for the easternmost lot averaging 42.2 percent.

5.2 Subsurface Soils

At the locations of the test pits, we encountered approximately 12 inches of Soil A-B horizons (topsoil) at the surface, except in Test Pit 4 where the topsoil appears to have been removed. In Test Pit 2, we encountered approximately 1.5 feet of fill soils at the surface (overlying the topsoil), which consisted of clayey gravel; the fill is considered undocumented/untested and should be removed from beneath the proposed residence.

Subsurface conditions encountered below the fill/topsoil at the test pit locations were relatively consistent, and exposed primarily cohesive soils mantled upon weathered rock surfaces. The soils in the test pits consisted of a surficial layer of medium dense colluvial silty clayey Sand or Gravel (SC-SM or GC-CM) with angular cobbles, yellowish brown in color, and generally slightly moist to moist. Beneath the surficial colluvium in Test Pits 1, 2, 3, and 5, weathered rock/bedrock deposits of closely fractured, strong Greywacke Rock, dark olive in color, was encountered resulting in equipment refusal within a few feet of exposure. Beneath the surficial colluvium in Test Pit 4, a thick layer of medium dense, clayey Sand with angular gravel (SC), reddish brown in color, moist, and believed to be colluvial or landslide deposits, was observed to the maximum depth explored of 15 feet.

For a more descriptive interpretation of subsurface conditions, please refer to the test pit logs, **Figures 6 and 7**.

5.3 Groundwater

Groundwater was not encountered at the time of our field explorations within the maximum depth explored of about 15 feet below the existing ground surface. Groundwater levels can fluctuate as much as 1.5 to 2 feet seasonally. Numerous other factors such as heavy precipitation, or other unforeseen factors, may also influence ground water elevations at the site. The detailed evaluation of these and other factors, which may be responsible for ground water fluctuations, is beyond the scope of this study.

5.4 Site Subsurface Variations

Based on the results of the subsurface explorations and our experience, variations in the continuity and nature of subsurface conditions should be anticipated. Due to the heterogeneous characteristics of natural soils, care should be taken in interpolating or extrapolating subsurface conditions between or beyond the exploratory locations.

Also, when logging and sampling of the test pits was completed, the test pits were backfilled with the excavated soils but minimal to no effort was made to compact these soils. Test pit backfill soils must be considered non-engineered/undocumented. Thus, settlement of the backfill in the test pits over time should be anticipated, and footings should not be placed over such areas without removal/replacement or other approved mitigation.

6.0 SITE PREPARATION AND GRADING

6.1 General

All deleterious materials should be stripped from the site prior to commencement of construction activities. This includes loose and disturbed soils, topsoil, vegetation, etc. Based upon the conditions observed in the test pits there is topsoil on the surface of the site which we estimated to be about 12 inches (plus or minus) in thickness. When stripping and grubbing, topsoil should be distinguished by the apparent organic content and not solely by color; thus we estimate that topsoil stripping will need to include at least the upper 6 inches. Where scrub oak exists and is removed, larger roots (greater than about ½ inch) will likely extend deeper and should be removed beneath the residence in those localized areas. Also, any existing undocumented fill shall be removed from beneath the structure.

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

Any fill should be placed on relatively level surfaces and against relatively vertical surfaces. Thus, where the existing slope is steeper than about 5H:1V (Horizontal:Vertical), the existing ground should be benched to create horizontal and vertical surfaces for receiving the fill. We recommend maximum bench heights of about 3 feet.

6.2 Temporary Excavations

Excavations deeper than about 10 feet are not anticipated at the site. Groundwater was not encountered within the depths explored, up to about 15 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 sandy clay to clayey sand. In clayey (cohesive) soils, temporary construction excavations not exceeding 4 feet in depth may be constructed with near-vertical side slopes. Temporary excavations up to 10 feet deep may be constructed with side slopes no steeper than one-half horizontal to one vertical (0.5H:1V).

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

6.3 Fill Material

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. Well-graded sand/gravel mixture, with maximum particle size of 4 inches, a minimum 70% passing 3/4-inch sieve, a maximum 20% passing the No. 200 sieve, and a maximum Plasticity Index of 10.
Site Grading Fill	Placed over larger areas to raise the site grade. Sandy to gravelly soil, with a maximum particle size of 6 inches, a minimum 70% passing 3/4-inch sieve, and a maximum 50% 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 (see discussion below).
Stabilization Fill	Placed to stabilize soft areas prior to placing structural fill and/or site grading fill. Coarse angular gravels and cobbles 1 inch to 8 inches in size. May also use 1.5-inch to 2.0-inch gravel placed on stabilization fabric, such as Mirafi RS280i, or equivalent (see **Section 6.6**).

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

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

6.4 Fill Placement and Compaction

The various types of compaction equipment available have their limitations as to the maximum lift thickness that can be compacted. For example, hand operated equipment is limited to lifts of about 4 inches and most "trench compactors" have a maximum, consistent compaction depth of about 6 inches. Large rollers, depending on soil and moisture conditions, can achieve compaction at 8 to 12 inches. The full thickness of each lift should be compacted to at least the following percentages of the maximum dry density as determined by ASTM D-1557 (or AASHTO[2] T-180) in accordance with the following recommendations:

LOCATION	TOTAL FILL THICKNESS (FEET)	MINIMUM PERCENTAGE OF MAXIMUM DRY DENSITY
Beneath an area extending at least 4 feet beyond the perimeter of structures, and below flatwork and pavement (applies to structural fill and site grading fill) extending at least 2 feet beyond the perimeter	0 to 5 5 to 10	95 98
Site grading fill outside area defined above	0 to 5 5 to 10	92 95
Utility trenches within structural areas	--	96
Roadbase and subbase	-	96
Non-structural fill	0 to 5 5 to 10	90 92

Structural fills greater than 10 feet thick are not anticipated at the site. 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.

6.5 Utility Trenches

For the bedding zone around the utility, we recommend utilizing sand bedding fill material that meets current APWA[3] requirements.

[2] American Association of State Highway and Transportation Officials
[3] American Public Works Association

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

Most utility companies and local governments are requiring Type A-1a or A-1b (AASHTO Designation) soils (sand/gravel soils with limited fines) be used as backfill over utilities within public rights of way, and the backfill be compacted over the full depth above the bedding zone to at least 96% of the maximum dry density as determined by AASHTO T-180 (ASTM D-1557). The natural sandy soils at this site will not likely meet these specifications.

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

6.6 Slope Stability

We evaluated the stability along three cross sections, including proposed/recommended building areas at each of the three lots. The location of the cross-sections we evaluated are shown as line A-A'; B-B'; and C-C' on **Figure 5, Site Evaluation**. The properties of the clayey sand soils encountered at the site were obtained from the direct shear testing, as shown on **Figures 8 and 9**, attached. We estimated that the bedrock materials have similar properties, except they typically have much higher cohesion. Accordingly, we used the following estimated parameters in the stability analyses:

MATERIAL	INTERNAL FRICTION ANGLE (DEGREES)	APPARENT COHESION (PSF)	SATURATED UNIT WEIGHT (PCF)
Clayey Sand	32	150	130
Bedrock (Graywacke)	32	500	140

For the seismic (pseudostatic) analysis, a peak horizontal ground acceleration (PGA_M) of 0.635*g* after adjusting for Site Class C was obtained for site (grid) locations of 41.3352 degrees north latitude and -111.8886 degrees west longitude. The pseudostatic coefficient for the seismic stability analysis was obtained using the method presented by Bray and Travasarou[4], which resulted in a pseudostatic coefficient of 0.18 based on the PGA_M and a slope height (within the effectual cross-section areas) of about 120 feet.

Using these input parameters, we evaluated the global stability of the site with the proposed residences using limit equilibrium (Simplified Bishop) methods via the computer program *SLIDE* (version 7.0). The configurations we analyzed consisted of the existing ground surface and subsurface soils/bedrock along cross-sections A-A'; B-B'; and C-C' as shown on **Figure 5**. Given that septic systems will be constructed on the three lots, we included a phreatic surface in each cross-section to account for water seepage into the ground from those systems. We also estimated the locations of the proposed residences, which will likely be mostly cut

[4] 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.

into the existing hillside. 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 indicate that the slopes on the three lots will meet both these requirements, provided the structures are not placed on the Qms material shown in Figure 5. The analyses are graphically shown on **Figures 10 through 15, Stability Results**, provided in the Appendix, and are summarized in the following table.

CONDITION	SEISMIC COEFFICIENT	SLOPE CROSS-SECTION	*SLIDE* LOWEST FACTOR OF SAFETY	MINIMUM ALLOWABLE FACTOR OF SAFETY
Static	---	A-A’	1.51	1.5
Seismic	0.18	A-A’	1.04	1.0
Static	---	B-B’	1.44*	1.5
Seismic	0.18	B-B’	1.05	1.0
Static	---	C-C’	1.55	1.5
Seismic	0.18	C-C’	1.04	1.0

*Does not meet minimum factor of safety

If cut and fill slopes are required to facilitate development plans, retaining walls or shoring must be planned. Further we recommend that all fills placed along the current slope for site grading must be benched into the slope a minimum of 2 feet following removal of surface vegetation. Any retaining walls must be properly engineered to maintain stability of the slopes. We strongly recommend that re-vegetation of the cut/fill soils be completed as soon as possible to reduce surface erosion. CMT must review the final grading plans for the project prior to initiation of any construction.

6.7 Soil Stabilization

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

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

7.0 FOUNDATION RECOMMENDATIONS

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

7.1 Foundation Recommendations

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

We also recommend the following:

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

7.2 Installation

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

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

All structural fill should meet the requirements for such, and should be placed and compacted in accordance with **Section 6** 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.

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

7.3 Estimated Settlement

Foundations designed and constructed in accordance with our recommendations could experience some settlement, but we anticipate that total settlements of footings founded as recommended above will not exceed 1 inch, with differential settlements on the order of 0.5 inches over a distance of 25 feet. We expect approximately 50% of the total settlement to initially take place during construction.

7.4 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 for structural fill, may be utilized for design. Passive resistance provided by properly placed and compacted structural fill may be considered equivalent to a fluid with a density of 350 pcf. A combination of passive earth resistance and friction may be utilized if the friction component of the total is divided by 1.5.

8.0 LATERAL EARTH PRESSURES

We project that basement walls up to 8 feet tall might be constructed at this site. The lateral earth pressure values given below are for a backfill material that will consist of drained sand/gravel soils (less than 10% passing No. 200 sieve) placed and compacted in accordance with the recommendations presented herein. If other soil types will be used as backfill (including on-site soils), 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. For rigid basement walls that are not more than 10 inches thick, sand/gravel backfill may be designed using an at-rest equivalent fluid pressure of 55 pcf (psf/ft). This value assumes 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 seismic loading of rigid basement walls up to 8 feet tall, we recommend using a uniform (rectangular) at-rest lateral pressure of 125 psf for design.

9.0 FLOOR SLABS

Floor slabs may be established upon suitable, undisturbed, natural soils/weathered bedrock and/or on structural fill extending to suitable natural soils/weathered bedrock (same as for foundations). Under no circumstances shall floor slabs be established directly on any topsoil, non-engineered fills, loose or disturbed soils, sod, rubbish, construction debris, other deleterious materials, frozen soils, or within ponded water.

In order to facilitate curing of the concrete, we recommend that floor slabs be directly underlain by at least 4 inches of “free-draining” fill, such as “pea” gravel or 3/4-inch to 1-inch minus, clean, gap-graded gravel. To help control normal shrinkage and stress cracking, the floor slabs should have the following features:

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

10.0 DRAINAGE RECOMMENDATIONS

10.1 Surface Drainage

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

1. All areas around the structure should be sloped to provide drainage around and away from the foundations. We recommend a minimum slope of 4 inches in the first 10 feet away from the structure. This slope should be maintained throughout the lifetime of the structure.

2. All roof drainage should be collected in rain gutters with downspouts designed to discharge at least 10 feet from the foundation walls or well beyond the backfill limits, whichever is greater.

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

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

5. Other precautions that may become evident during construction.

10.2 Foundation Subdrains

Groundwater was not encountered at this site, but bedrock was encountered on which water could accumulate or travel. We recommend that a foundation drain be installed for all habitable structures.

Foundation subdrains should consist of a 4-inch diameter perforated or slotted plastic or PVC pipe surrounded by clean gravel. The invert of the subdrain should be at least 2 feet below the top of the lowest adjacent floor slab. The gravel portion of the drain should extend a minimum 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. Prior to the installation of the footing subdrain, the below-grade walls should be dampproofed. The slope of the subdrain should be at least 0.5%. The gravel placed around the drain pipe should be clean 3/4-inch to 1-inch minus gap-graded gravel and/or “pea” gravel. The foundation subdrains can be discharged to a suitable down-gradient location.

11.0 QUALITY CONTROL

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

11.1 Field Observations

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

11.2 Fill Compaction

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

11.3 Excavations

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

12.0 LIMITATIONS

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

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

We appreciate the opportunity to be of service to you on this project. If we can be of further assistance or if you have any questions regarding this project, please do not hesitate to contact us at (801) 870-6730. To schedule materials testing, please call (801) 381-5141.

13.0 REFERENCES

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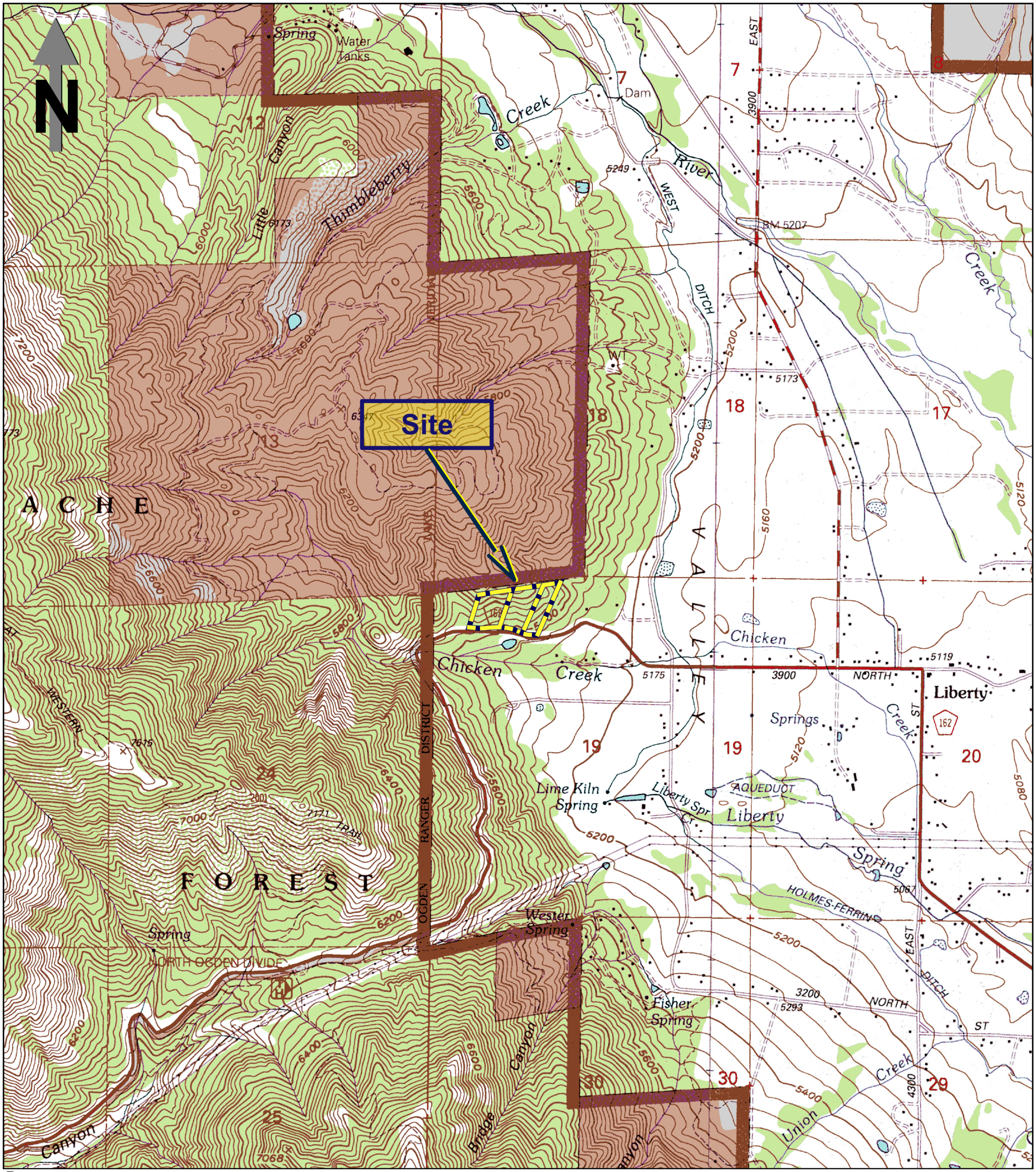
APPENDIX

SUPPORTING
DOCUMENTATION



Base:
USGS 7.5 Minute topographic maps titled"North Ogden, Utah 1998; ; and Huntsville, Utah 1998" from Utah AGRC: http://gis.utah.gov/

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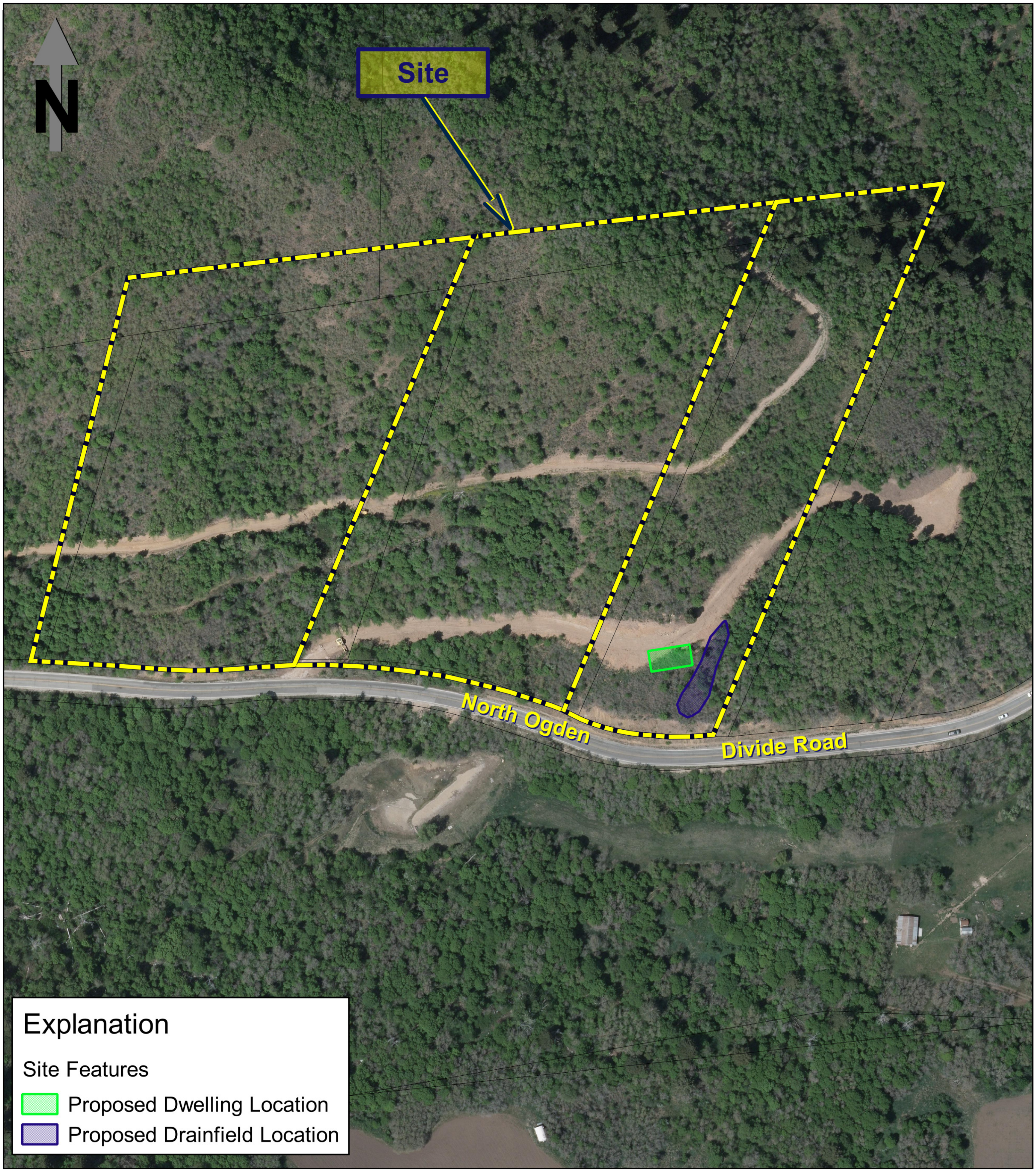
Jones Subdivision About 2600 E. North Ogden Canyon Rd. Liberty, Utah	CMT ENGINEERING LABORATORIES Vicinity Map	Date: Feb. 22-19 Job # 12290	Figure 1



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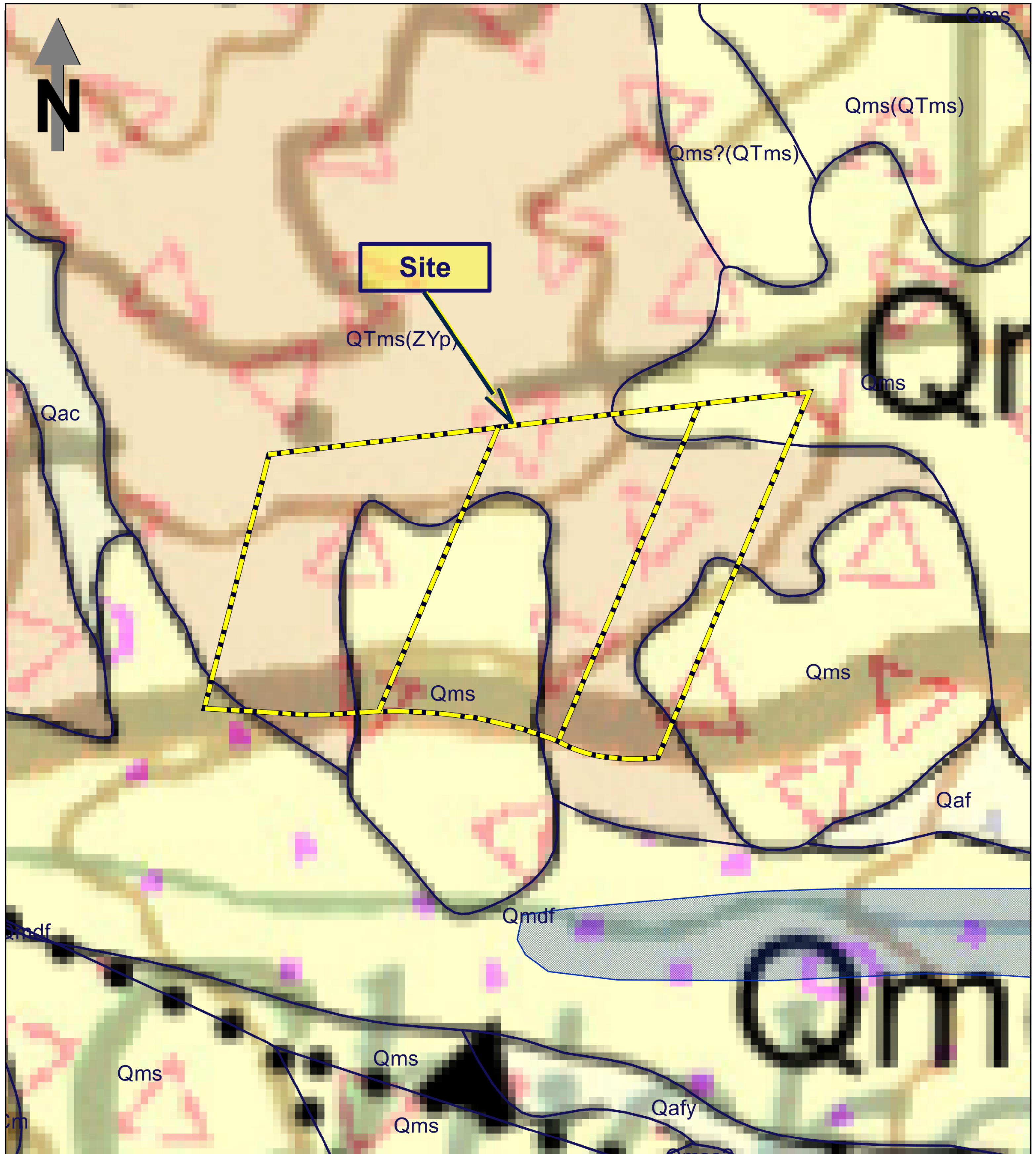
Jones Subdivision About 2600 E. North Ogden Canyon Rd. Liberty, Utah	CMT ENGINEERING LABORATORIES Site Plan	Date: Feb. 22-19 Job # 12290	Figure 2



Base:
Coogan and King, 2016.

0 300 600 ft
1:3,600

Geologic Classification

Geology after Coogan and King, 2016

Qafy Qaf - Alluvial-fan deposits (Holocene and Pleistocene) – Mostly sand, silt, and gravel that is poorly bedded and poorly...variably consolidated; includes debris flows, particularly in drainages and at drainage mouths (fan heads)...with unit Qafy being the lowest (youngest) fans and Qaf being undivided in age determination...<

Qal - Stream alluvium and flood-plain deposits (Holocene and uppermost Pleistocene) – Sand, silt, clay, and gravel in channels, flood plains, and terraces...

Qmdf, - Debris- and mud-flow deposits (Holocene and upper and middle? Pleistocene) – Very poorly sorted, clay- to boulder-sized material in unstratified deposits characterized by rubbly surface and debris-flow levees with channels, lobes, and mounding...<

Qac - Alluvium and colluvium (Holocene and Pleistocene) – Unsorted to variably sorted gravel, sand, silt, and clay in variable proportions; includes stream and fan alluvium, colluvium, and, locally, mass-movement deposits...

Qms - Landslide deposits (Holocene and upper and middle? Pleistocene) – Poorly sorted clay- to boulder sized material; includes slides, slumps, and locally flows and floods...

Qms?(QTms) - Block landslide and possible block landslide deposits (Holocene and upper and middle? Pleistocene) – Mapped where nearly intact block is visible in landslide (mostly block slide) with stratal strikes and dips that are different from nearby in-place bedrock...comprised of Quaternary and/or Tertiary mega-landslide (Pleistocene and/or Pliocene) – Jumbled mass of formation of Perry Canyon (ZYp) with blocks of rock from North Ogden divide

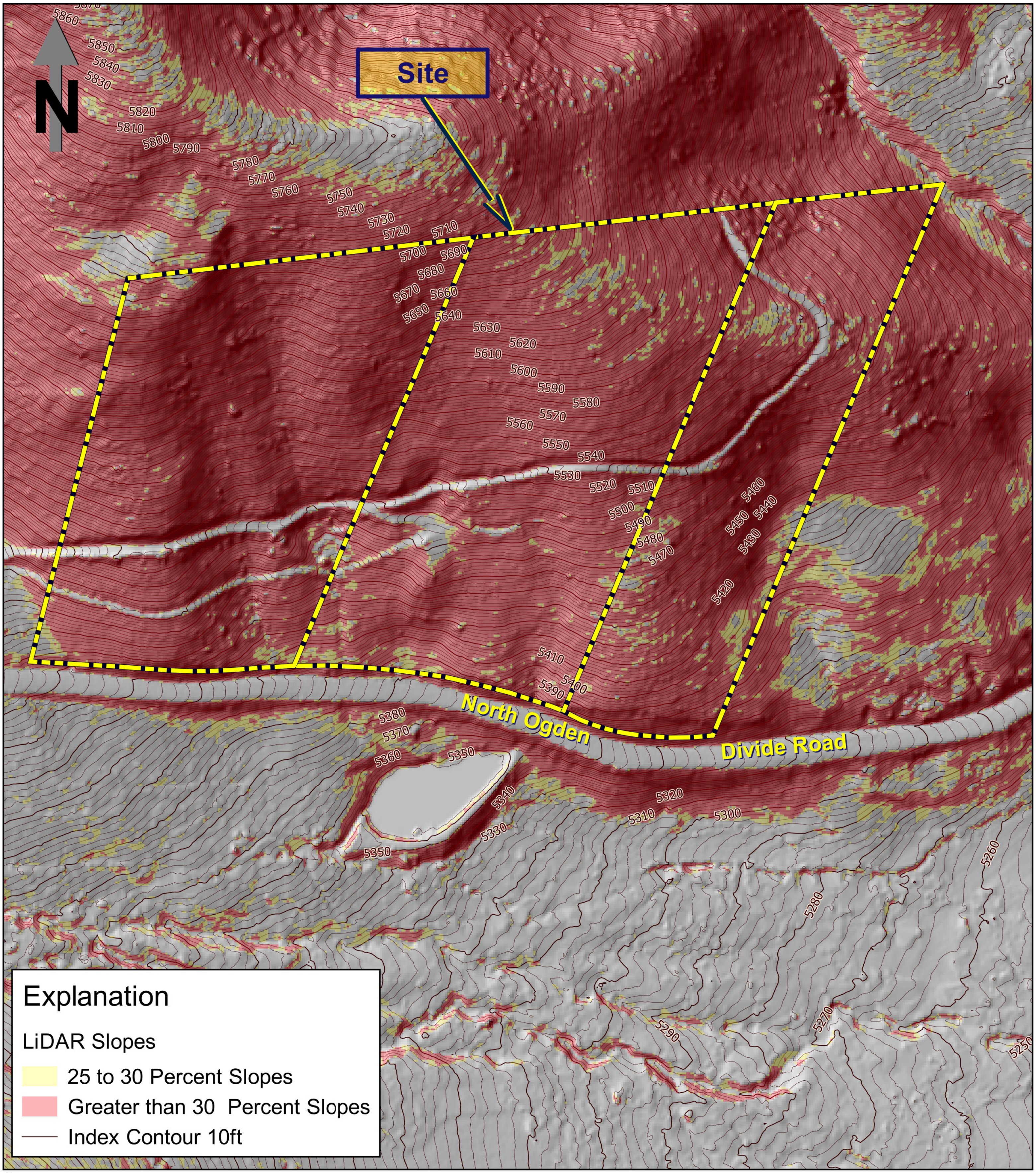
QTms(ZYp) - Quaternary and/or Tertiary mega-landslide (Pleistocene and/or Pliocene) – Jumbled mass of formation of Perry Canyon (ZYp) with blocks of rock from North Ogden divide...

Thrust Fault Concealed

FEMA - Flood Insurance Rating Zones (2015)

Zone A and AE

CMT ENGINEERING LABORATORIES	Jones Subdivision About 2600 E. North Ogden Canyon Rd. Liberty, Utah	GEOLOGIC MAPPING	Date: Feb. 22-19 Job # 12290	3



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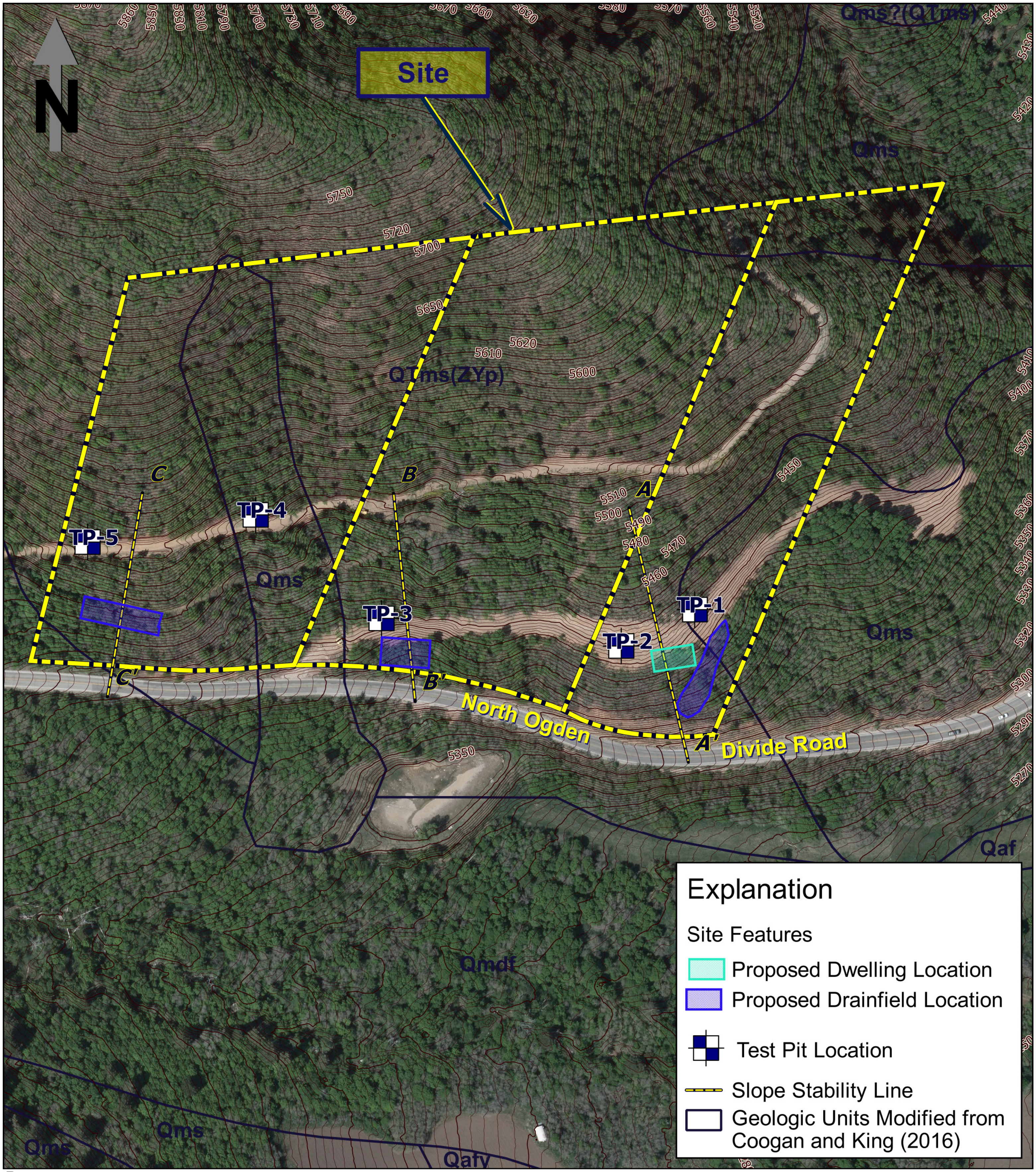
Jones Subdivision About 2600 E. North Ogden Canyon Rd. Liberty, Utah	CMT ENGINEERING LABORATORIES		Figure 4
	LiDAR Analysis	Date: Feb. 18-19 Job # 12290	



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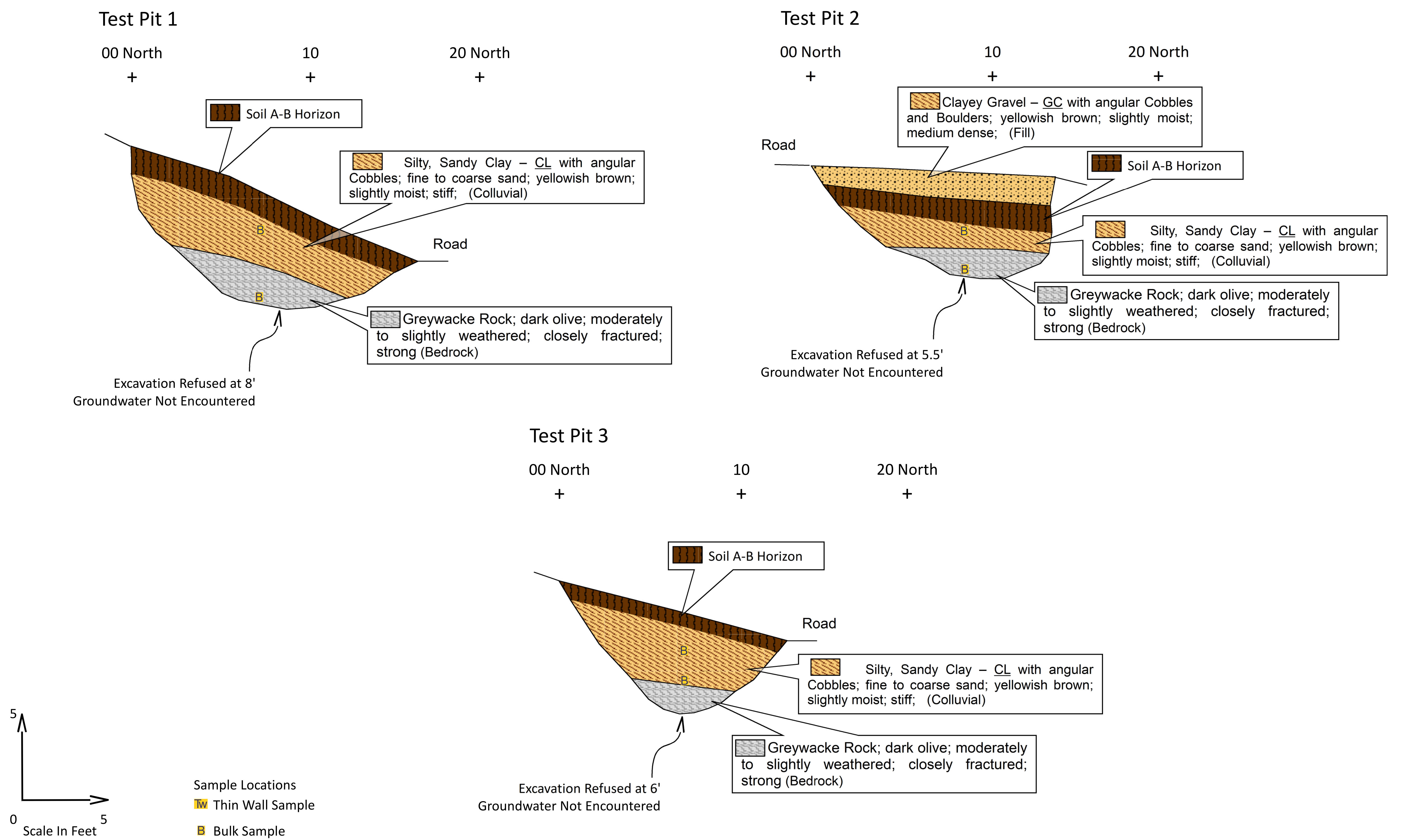
0 200 400 ft

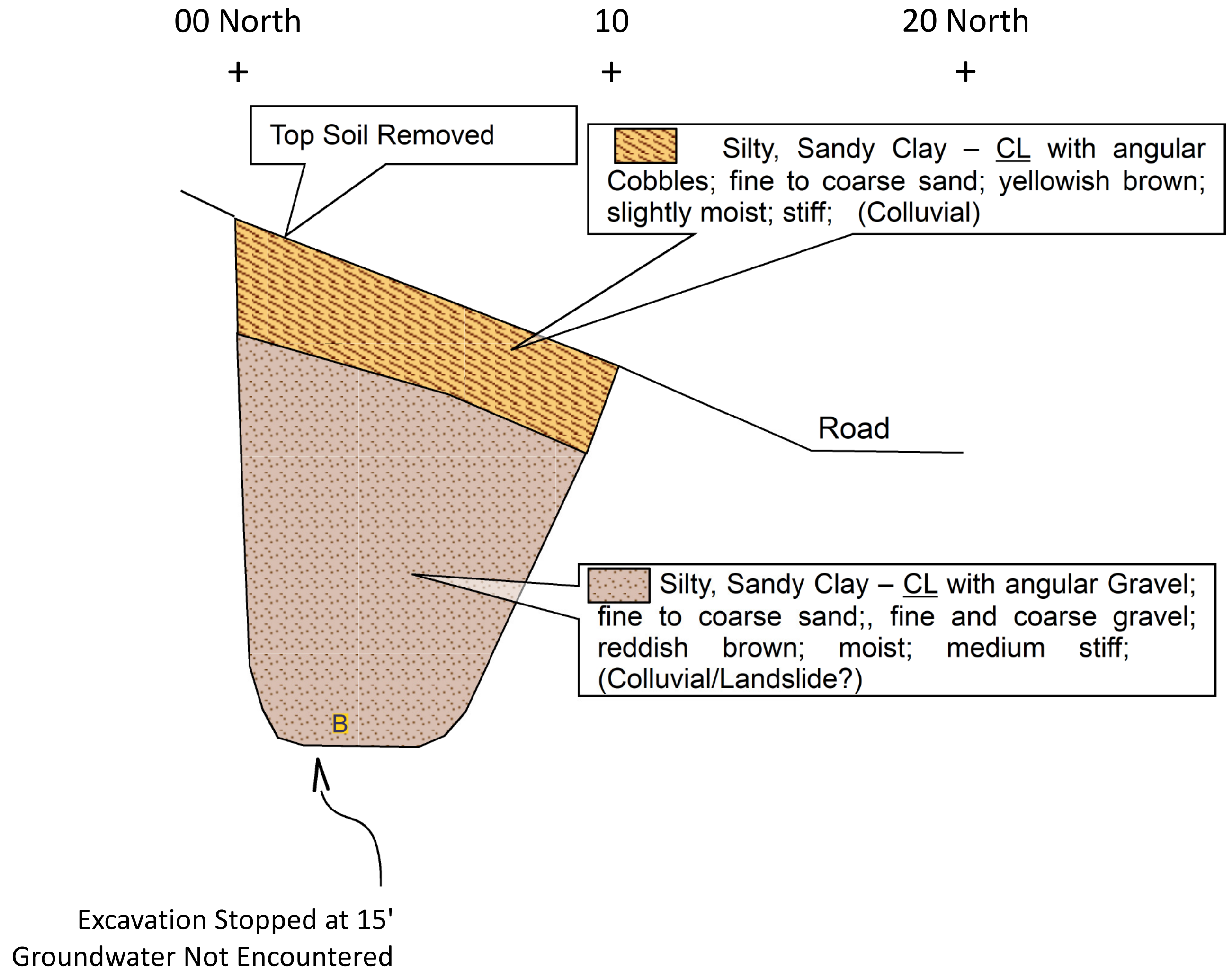
1:2,400

Jones Subdivision About 2600 E. North Ogden Canyon Rd. Liberty, Utah	CMT ENGINEERING LABORATORIES Site Evaluation	Date: Feb. 22-20 Job # 12290	Figure 5

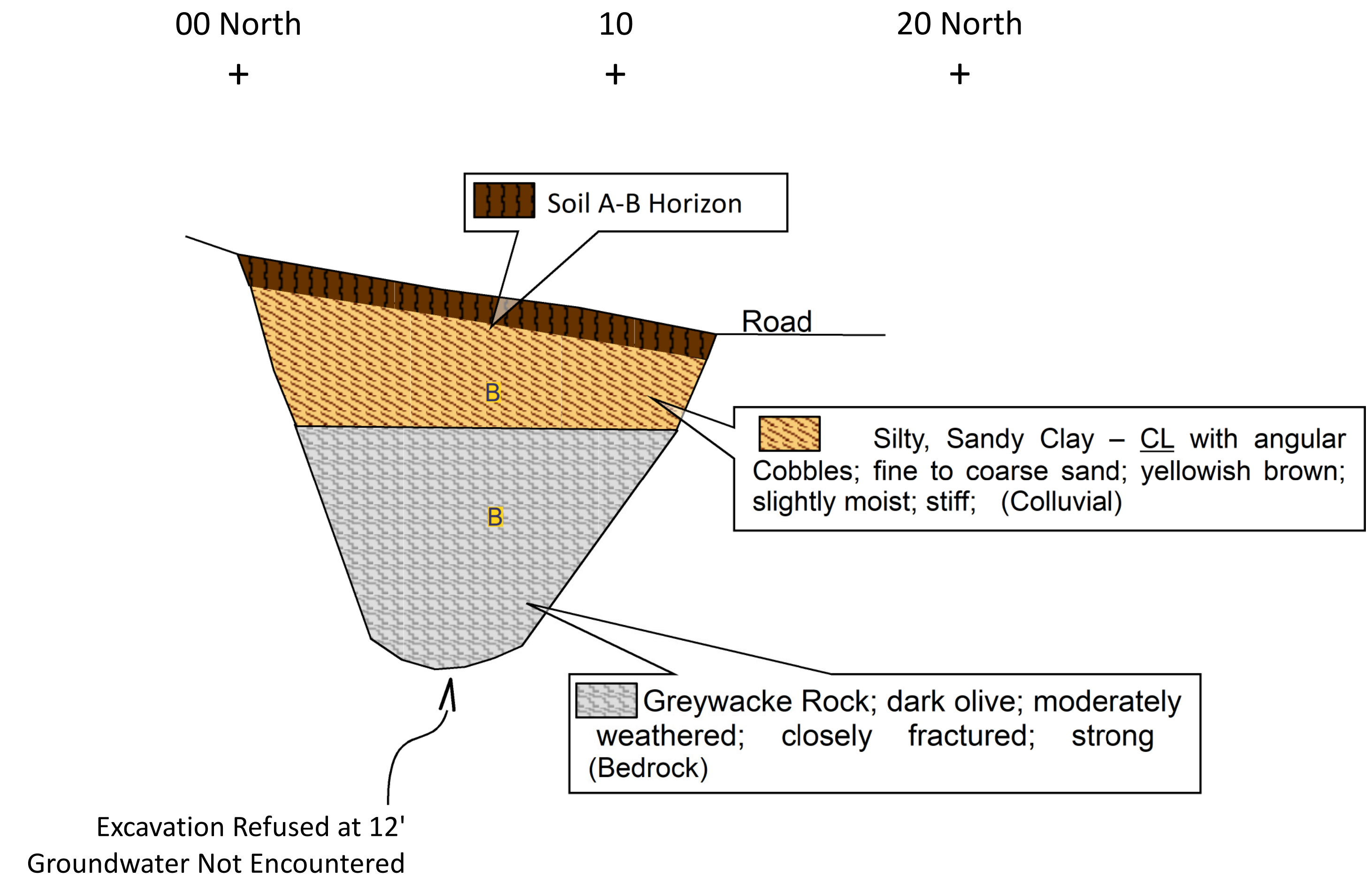


CMT ENGINEERING LABORATORIES	**Jones Subdivision** About 2600 E. North Ogden Canyon Rd. Liberty, Utah	**Log of Test Pit 1, 2 and 3**	Date:	Feb. 22-19	Figure 6
			Job #	12290	

Test Pit 4



Test Pit 5



5

0 5

Scale In Feet

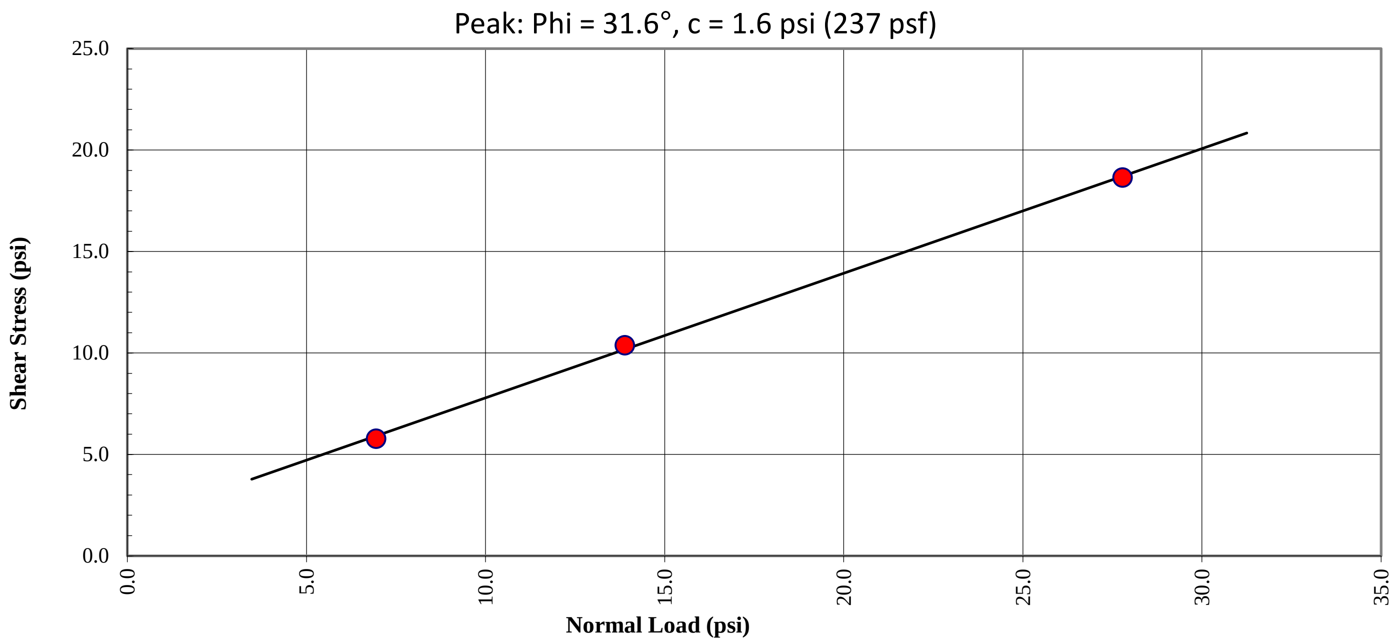
Sample Locations

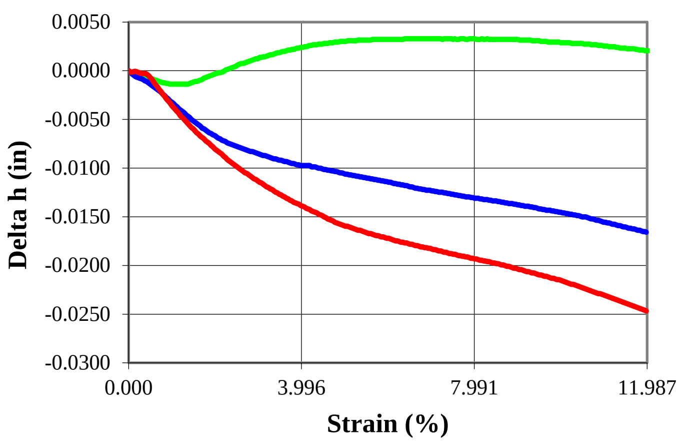
Tw Thin Wall Sample

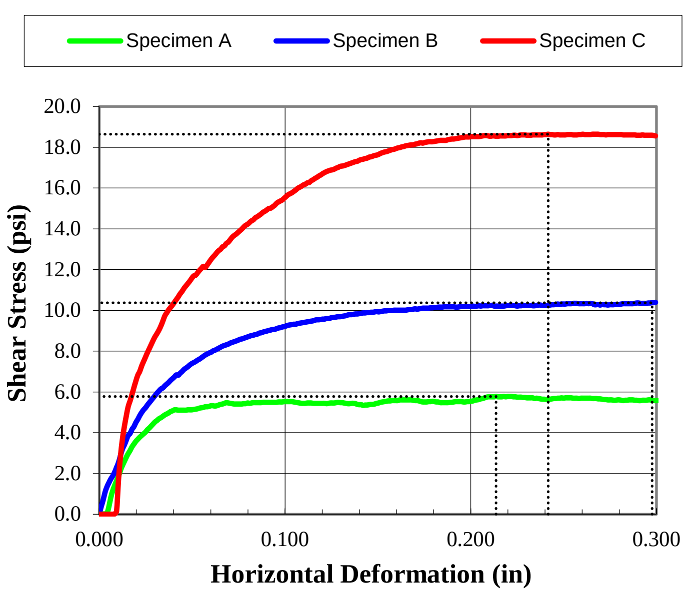
B Bulk Sample

CMT ENGINEERING LABORATORIES	**Jones Subdivision** About 2600 E. North Ogden Canyon Rd. Liberty, Utah	**Log of Test Pit 4 and 5**	Date: Feb. 22-19 Job # 12290	Figure 7

Direct Shear Test (ASTM D3080)







	Specimen		
Initial	**A**	**B**	**C**
Moisture (%)	18.34	18.34	18.34
Dry Density (pcf)	103.23	99.42	99.03
Void Ratio	0.60	0.66	0.67
Saturation (%)	80.74	73.28	72.55
Diameter (in)	2.50	2.50	2.50
Height (in)	1.00	1.00	1.00

Final	**A**	**B**	**C**
Moisture (%)	22.62	22.62	22.62
Dry Density (pcf)	104.18	101.71	100.44
Void Ratio	0.59	0.63	0.65
Saturation (%)	100.0	95.8	92.8
Diameter (in)	2.50	2.50	2.50
Height (in)	0.98	0.97	0.90
Normal Stress (psi)	6.94	13.89	27.78
Shear Stress (psi)(@Peak)	5.78	10.38	18.64
Peak Strain (%)	8.54	11.89	9.66
Rate (in/min)	0.0033	0.0033	0.0033
Peak Deformation (in.)	0.213	0.297	0.241

Sample Information	
Test Pit Number:	TP-2
Sample Number:	
Depth:	3 ft
Sample Type:	Remolded
Description:	Clayey Sand (SC)
Test Type:	Consolidated - Drained

Proposed Jones Subdivision

About 2600 North Ogden Canyon Road, Ogden, Utah

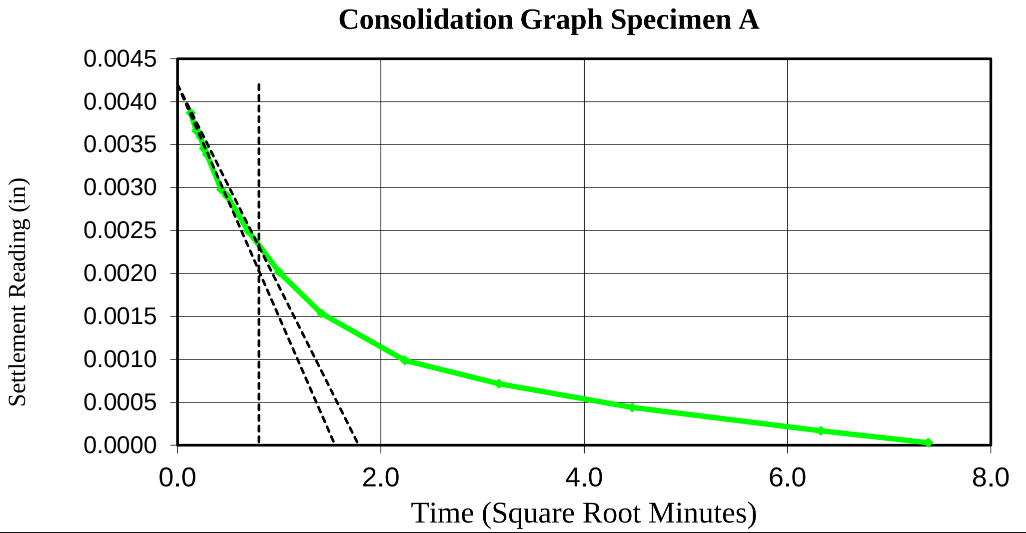
CMT ENGINEERING LABORATORIES

Lab Data

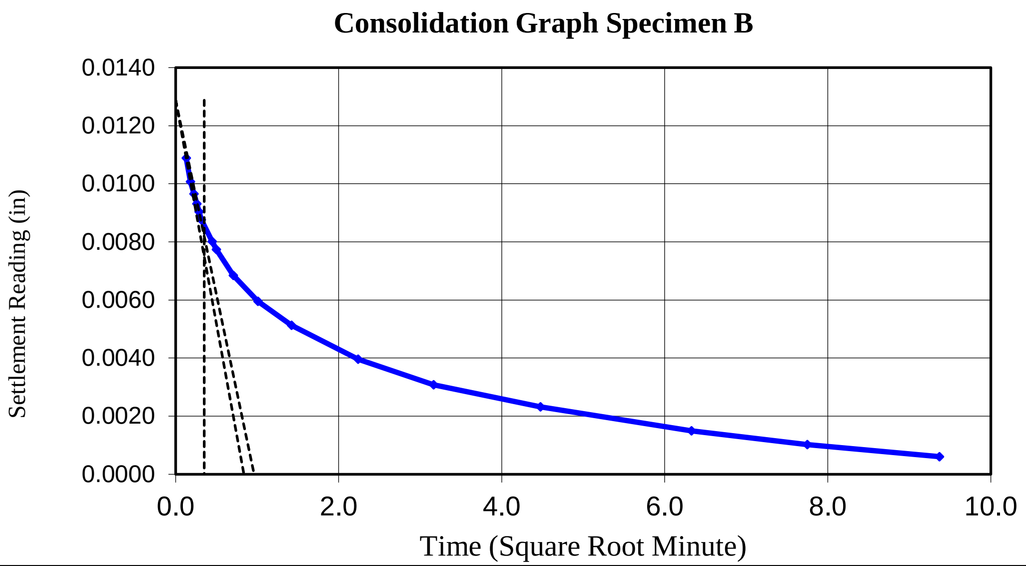
Date: 19-Mar-19
Job # 12290

Figure: 8A

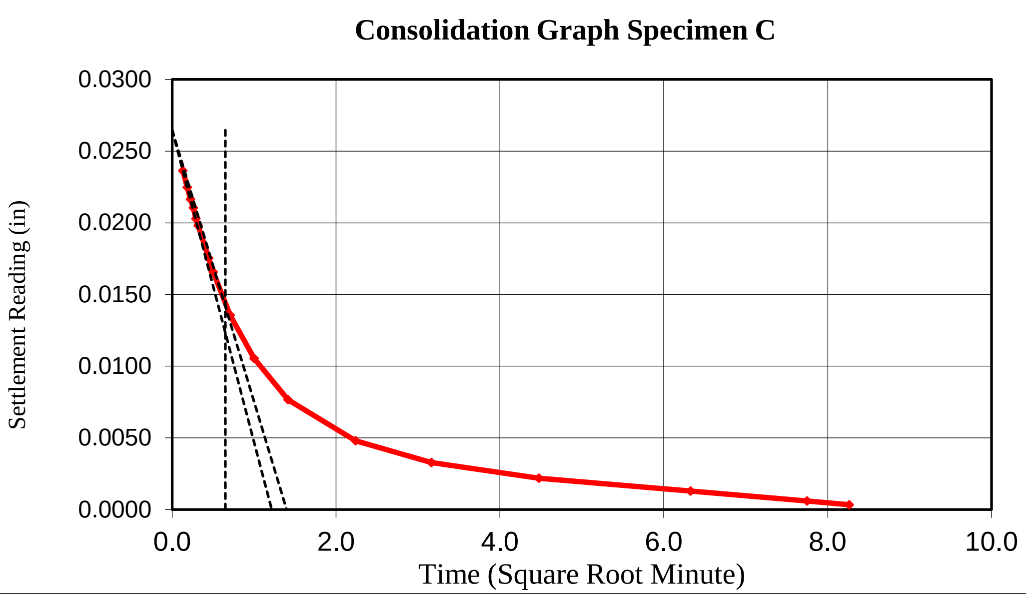
Direct Shear Test (ASTM D3080)



$\mathrm{sqrt}(t_{90})$ = 0.8
t_{90} = 0.6 min.
$t_f = 50\ t_{90} / 4.28$ = 7.5 min.
max $d_r = 0.5 / t_f$ = 0.0669 in./min.
selected d_r = 0.0033 in./min.

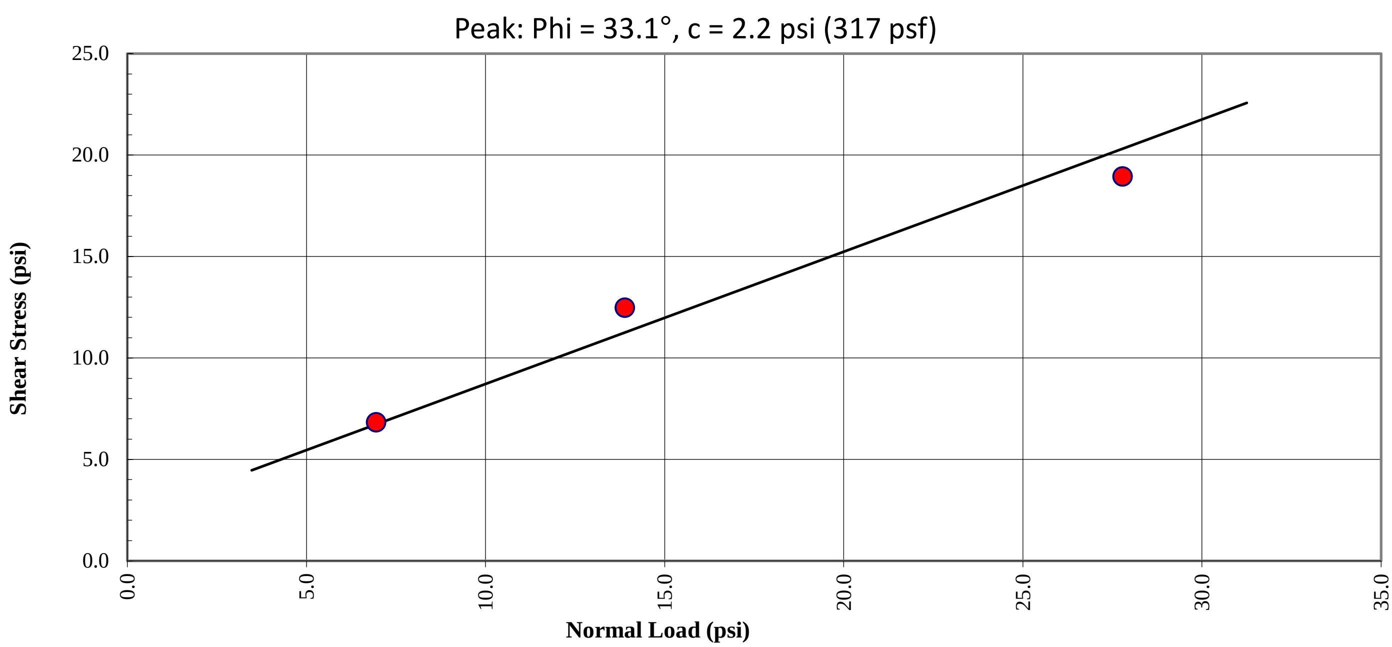


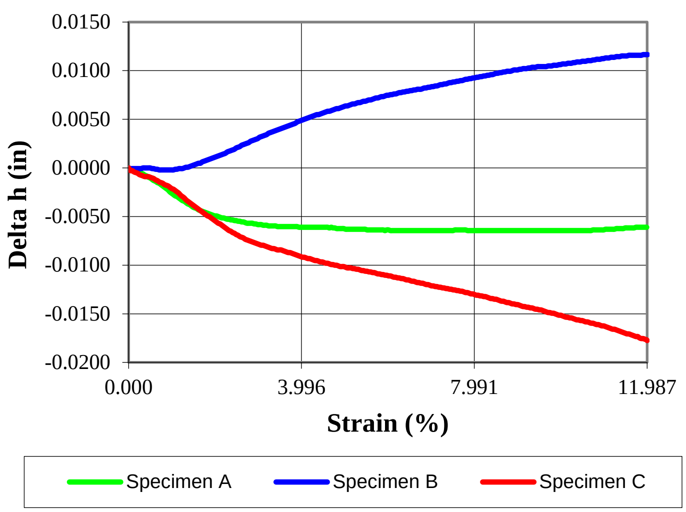
$\mathrm{sqrt}(t_{90})$ = 0.35
t_{90} = 0.1 min.
$t_f = 50\ t_{90} / 4.28$ = 1.4 min.
max $d_r = 0.5 / t_f$ = 0.3494 in./min.
selected d_r = 0.0033 in./min.

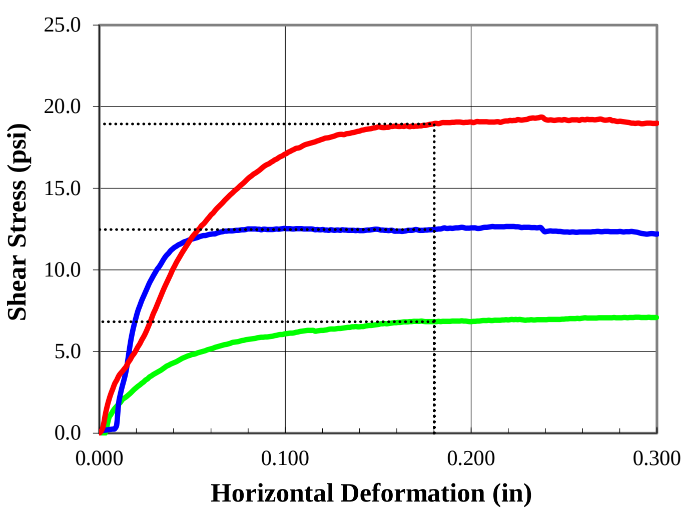


$\mathrm{sqrt}(t_{90})$ = 0.65
t_{90} = 0.4 min.
$t_f = 50\ t_{90} / 4.28$ = 4.9 min.
max $d_r = 0.5 / t_f$ = 0.1013 in./min.
selected d_r = 0.0033 in./min.

Direct Shear Test (ASTM D3080)







	Specimen		
Initial	**A**	**B**	**C**
Moisture (%)	15.83	15.83	15.83
Dry Density (pcf)	110.62	109.75	110.49
Void Ratio	0.49	0.51	0.50
Saturation (%)	84.77	82.78	84.46
Diameter (in)	2.50	2.50	2.50
Height (in)	1.00	1.00	1.00

Final	**A**	**B**	**C**
Moisture (%)	17.49	17.49	17.49
Dry Density (pcf)	113.09	112.89	115.14
Void Ratio	0.46	0.46	0.44
Saturation (%)	100.3	99.7	100.0
Diameter (in)	2.50	2.50	2.50
Height (in)	0.97	0.96	0.96
Normal Stress (psi)	6.94	13.89	27.78
Shear Stress (psi)(@0.18")	6.83	12.48	18.94
Strain at 0.18 in. (%)	6.67	3.15	7.17
Rate (in/min)	0.0033	0.0033	0.0033
Deformation (in.)	0.180	0.180	0.180

Sample Information	
Test Pit Number:	TP-4
Sample Number:	
Depth:	14.5 ft
Sample Type:	Remolded
Description:	Clayey Sand w/Gravel (SC)
Test Type:	Consolidated - Drained

Proposed Jones Subdivision

About 2600 North Ogden Canyon Road, Ogden, Utah

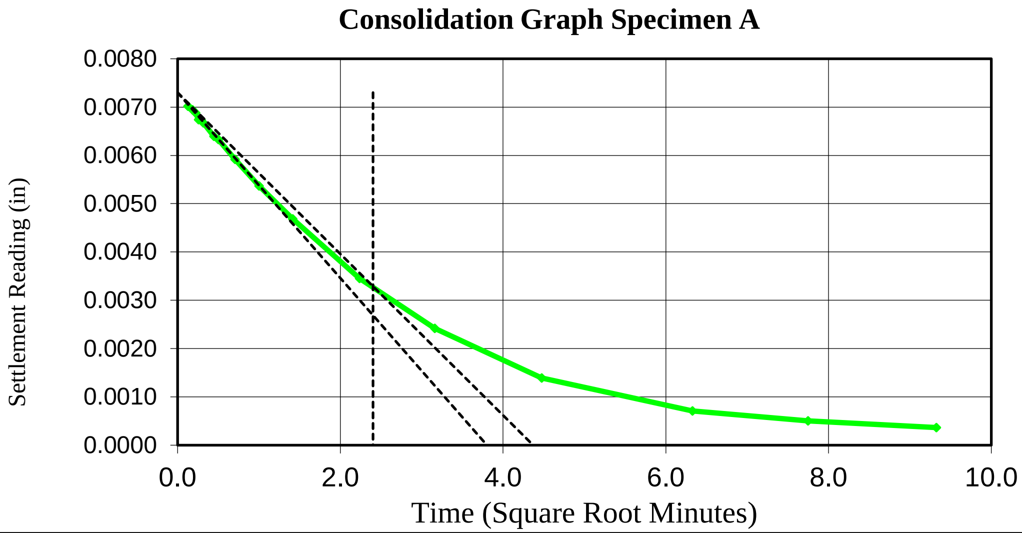
CMT ENGINEERING LABORATORIES

Lab Data

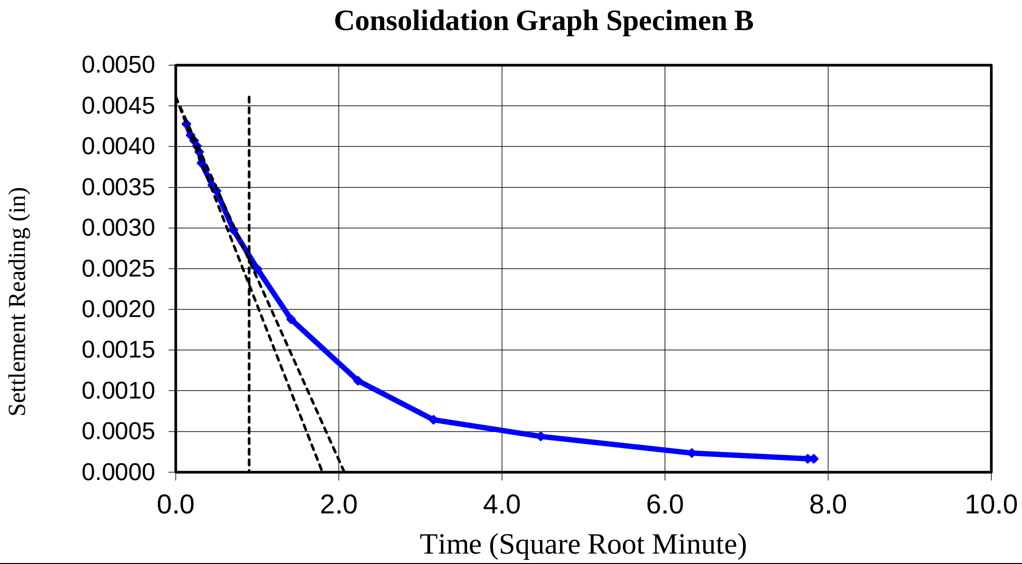
Date: 19-Mar-19
Job # 12290

Figure: 9A

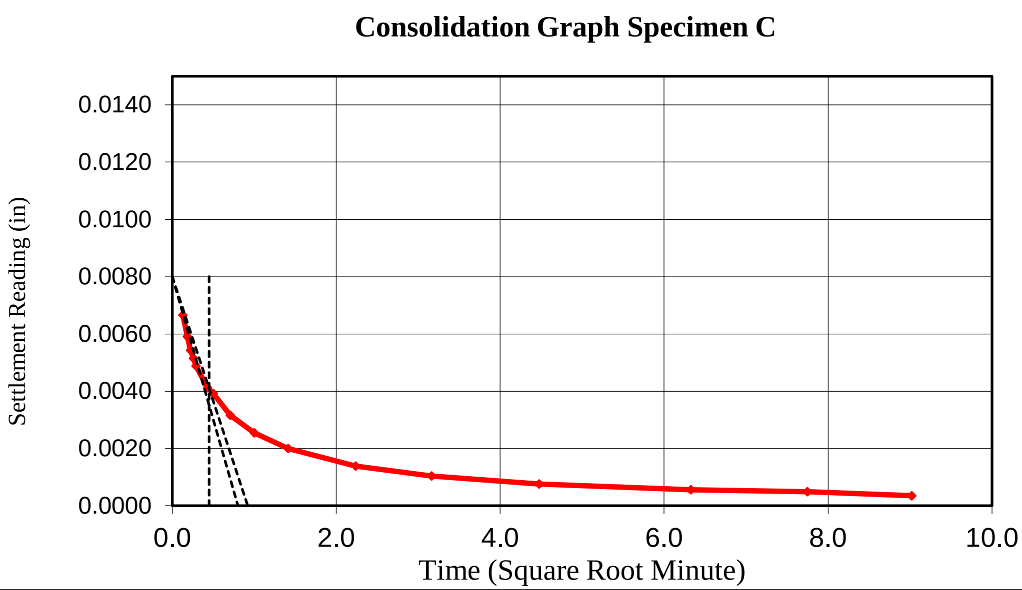
Direct Shear Test (ASTM D3080)



sqrt(t_{90}) =	2.4	
t_{90} =	5.8	min.
$t_f = 50\ t_{90} / 4.28$ =	67.3	min.
max $d_r = 0.5 / t_f$ =	0.0074	in./min.
selected d_r =	0.0033	in./min.



sqrt(t_{90}) =	0.9	
t_{90} =	0.8	min.
$t_f = 50\ t_{90} / 4.28$ =	9.5	min.
max $d_r = 0.5 / t_f$ =	0.0528	in./min.
selected d_r =	0.0033	in./min.



sqrt(t_{90}) =	0.45	
t_{90} =	0.2	min.
$t_f = 50\ t_{90} / 4.28$ =	2.4	min.
max $d_r = 0.5 / t_f$ =	0.2114	in./min.
selected d_r =	0.0033	in./min.

Proposed Jones Subdivision

About 2600 North Ogden Canyon Road, Ogden, Utah

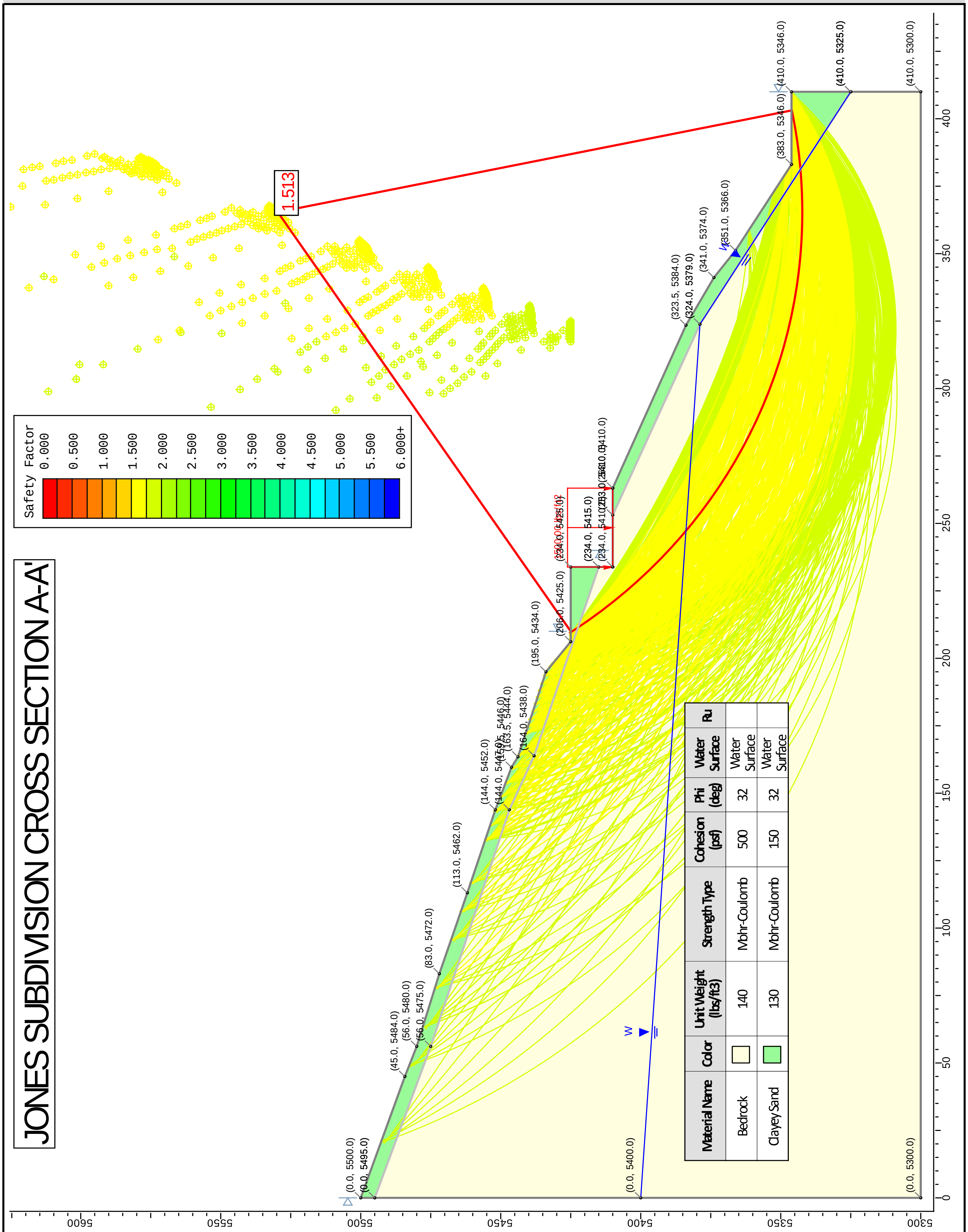
CMT ENGINEERING LABORATORIES

Lab Data

Date: 19-Mar-19
Job # 12290

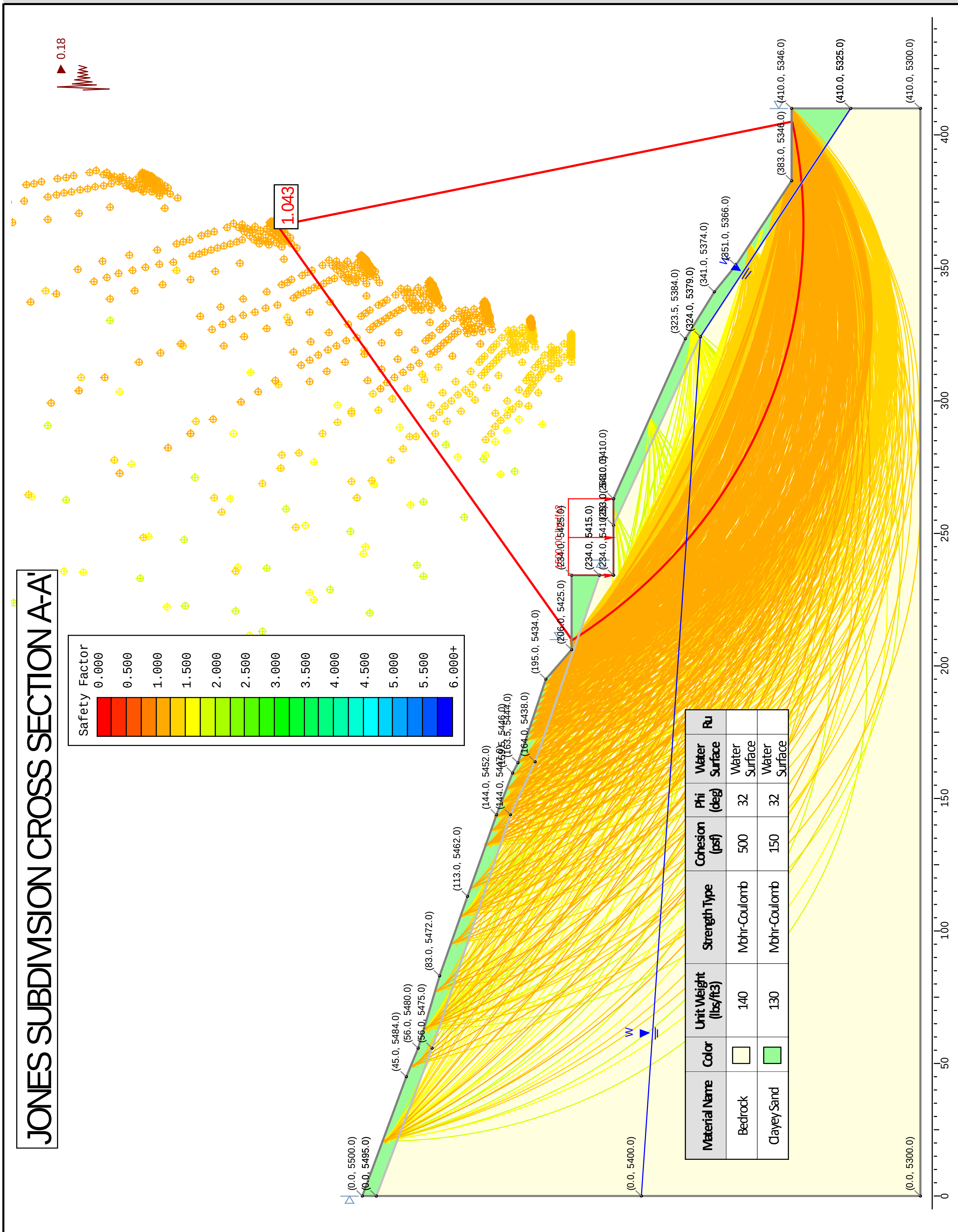
Figure:
9B

Stability Results - Static



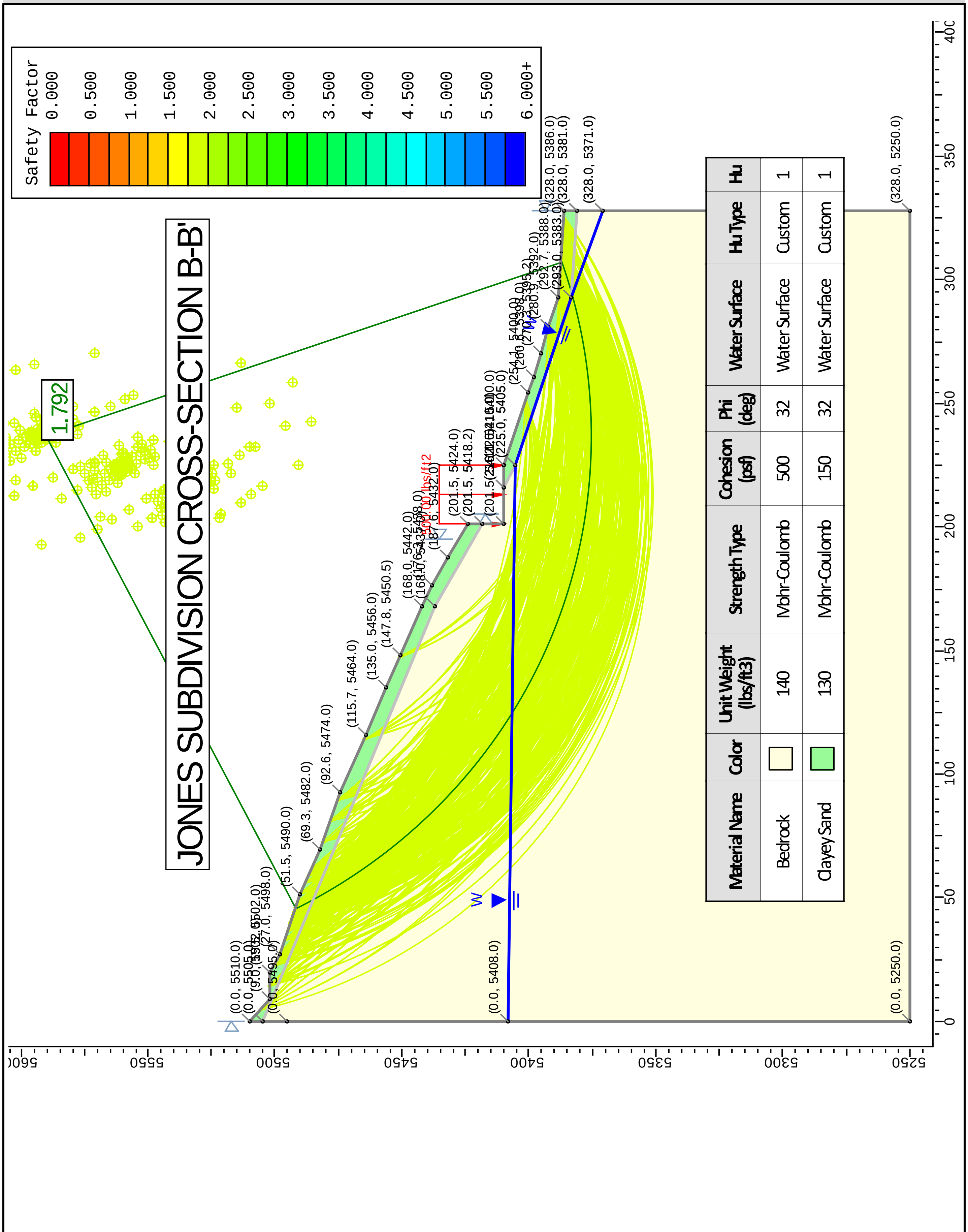
Jones Subdivision	CMT ENGINEERING LABORATORIES		Figure 10
About 2600 East North Ogden Canyon Road, Liberty, Utah	**Results**	**Date** 31-Mar-20	
		Job No. 12290	

Stability Results - Seismic



Jones Subdivision	CMT ENGINEERING LABORATORIES		Figure 11
About 2600 East North Ogden Canyon Road, Liberty, Utah	Results	Date 31-Mar-20	
		Job No. 12290	

Stability Results - Static



Jones Subdivision

About 2600 East North Ogden Canyon Road, Liberty, Utah

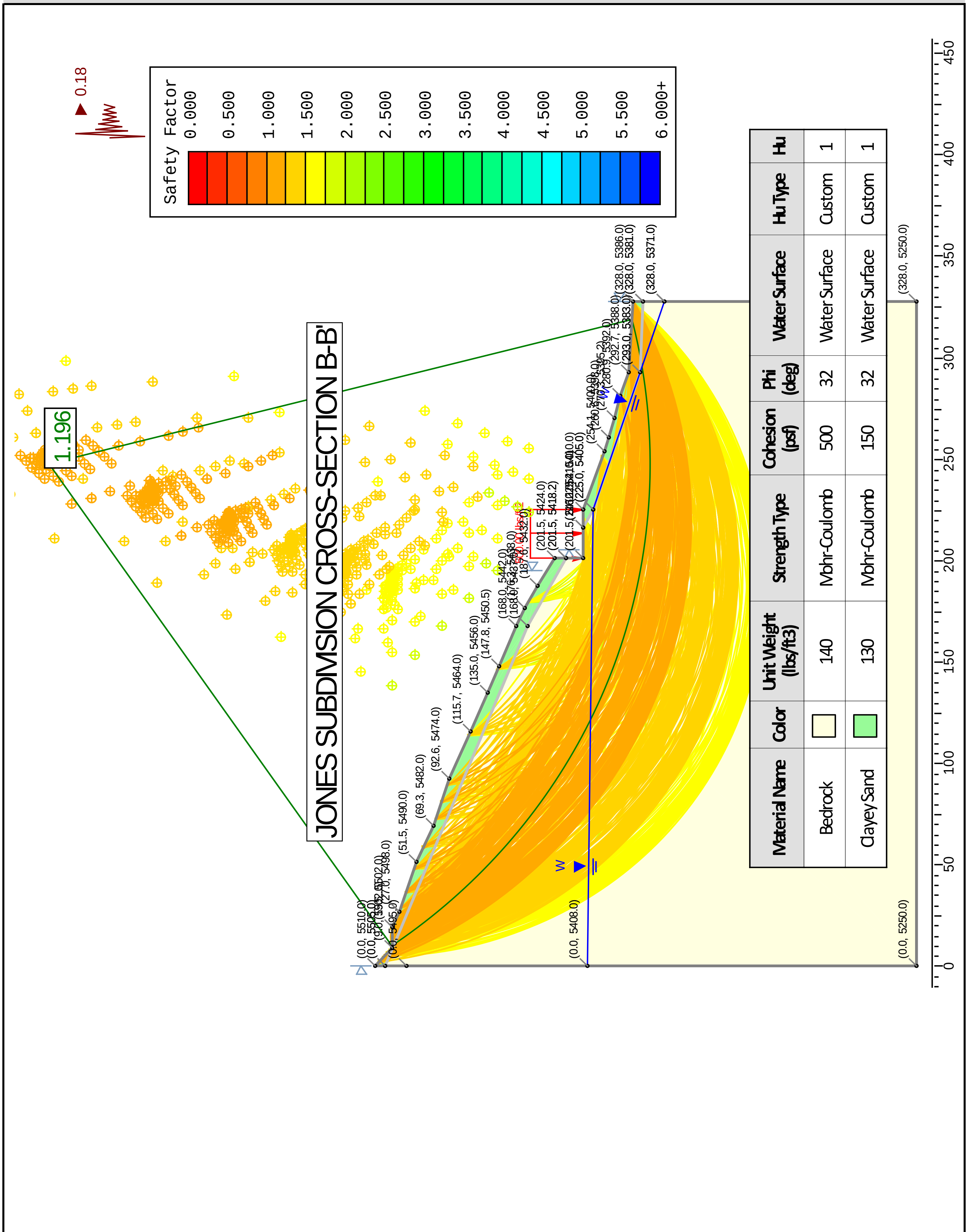
CMT ENGINEERING LABORATORIES

Results

Date	31-Mar-20
Job No.	12290

Figure 12

Stability Results - Seismic



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Hu Type	Hu
Bedrock		140	Mohr-Coulomb	500	32	Water Surface	Custom	1
Clayey Sand		130	Mohr-Coulomb	150	32	Water Surface	Custom	1

Jones Subdivision

About 2600 East North Ogden Canyon Road, Liberty, Utah

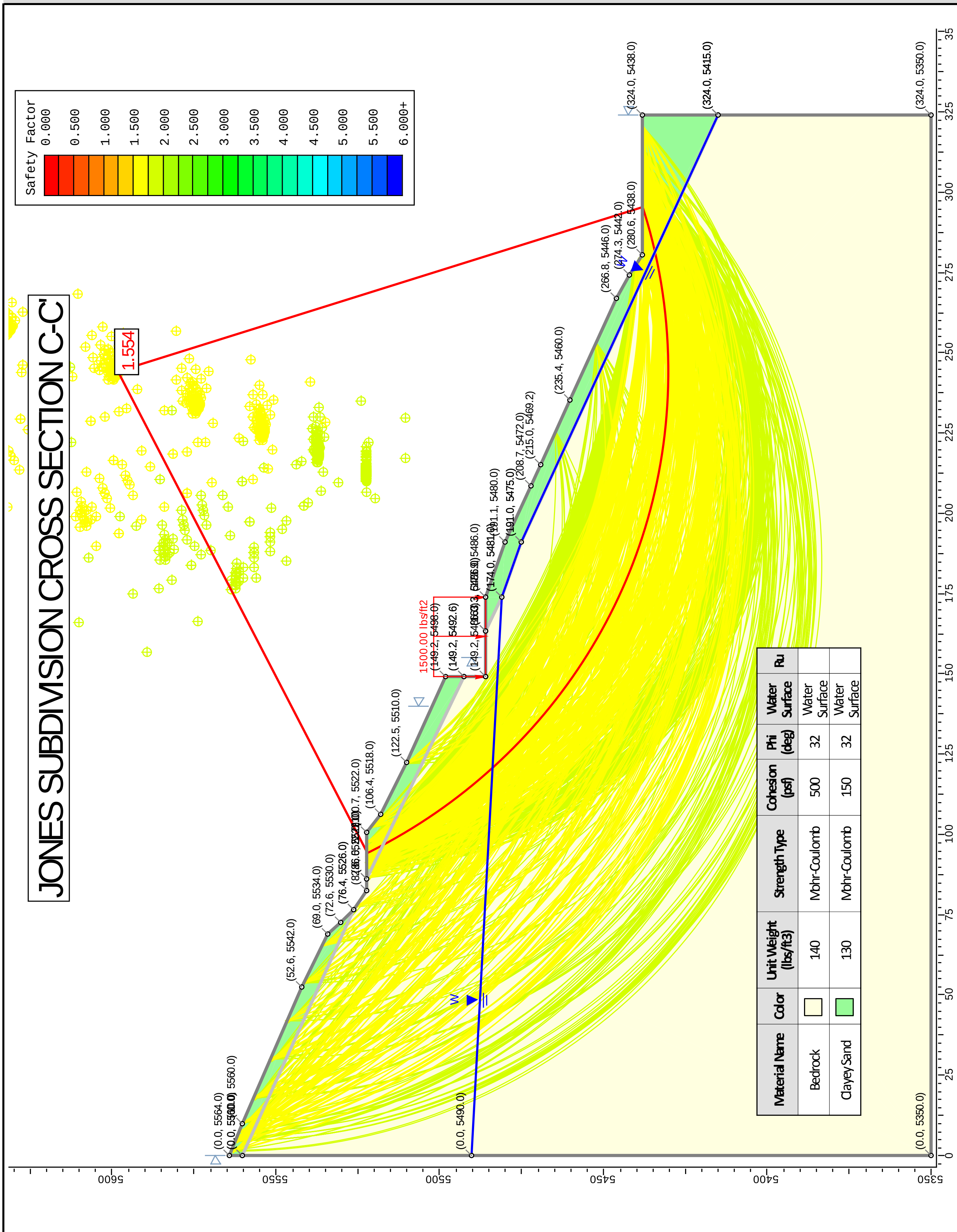
CMT ENGINEERING LABORATORIES

Results

Date	31-Mar-20
Job No.	12290

Figure 13

Stability Results - Static



Jones Subdivision

About 2600 East North Ogden Canyon Road, Liberty, Utah

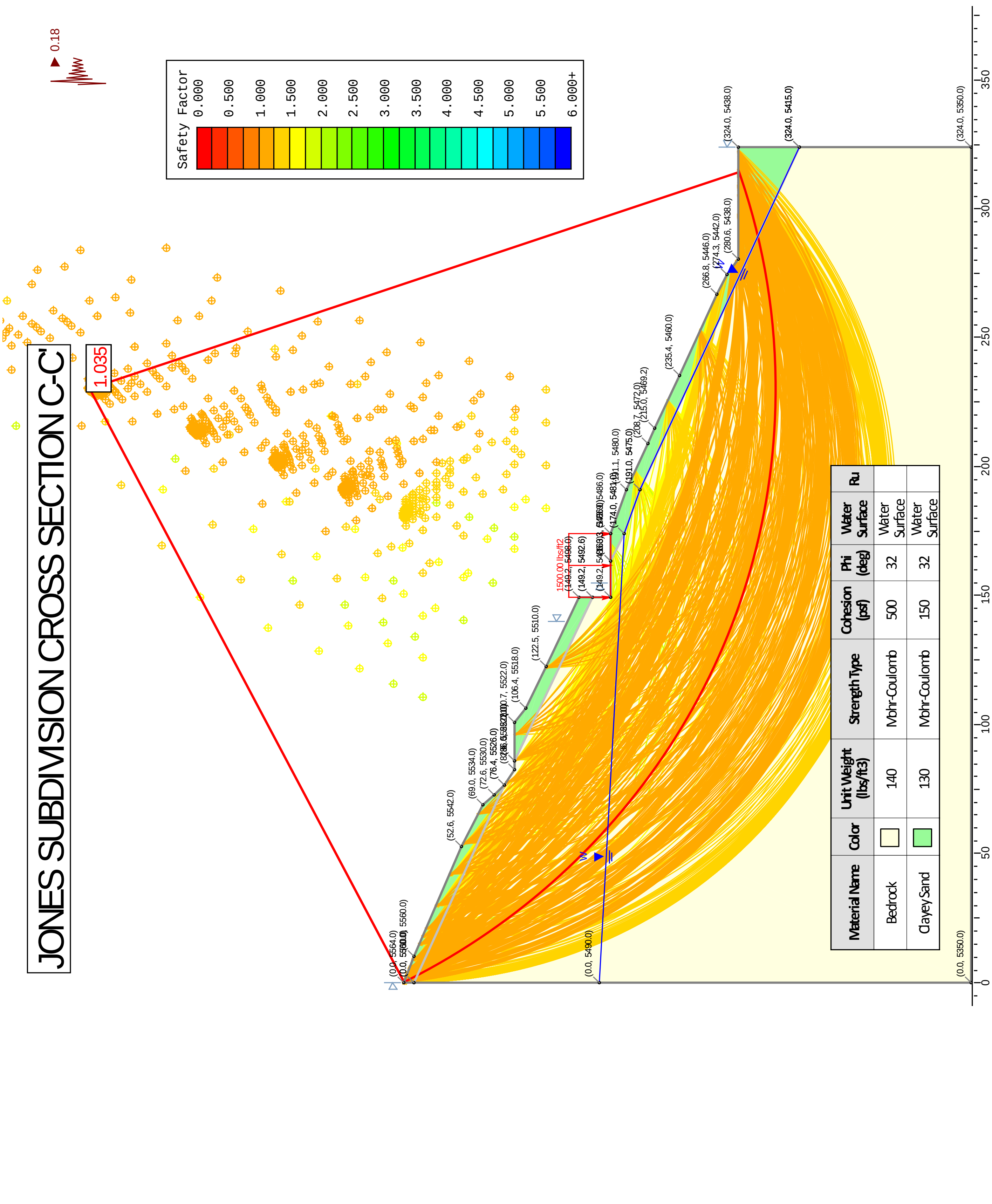
CMT ENGINEERING LABORATORIES

Results

Date	31-Mar-20
Job No.	12290

Figure 14

Stability Results - Seismic



Jones Subdivision

About 2600 East North Ogden Canyon Road, Liberty, Utah

CMT ENGINEERING LABORATORIES

Results	**Date**	31-Mar-20
	Job No.	12290

Figure 15