REPORT
GEOTECHNICAL STUDY
LOT 43 SUMMIT AT SKI LAKE NO. 11
6785 EAST VIA CORTINA STREET
HUNTSVILLE, UTAH

Submitted To:

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Submitted By:

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March 31, 2016

Job No. 2063-01N-16

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Mr. Rich Zollinger
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Livermore, California 94550

Re: Report
Geotechnical Study
Lot 43 Summit at Ski Lake No. 11
6785 East Via Cortina Street
Huntsville, Utah
(41.2429 N; 111.7850 W)

1. INTRODUCTION

1.1 GENERAL

This report presents the results of our geotechnical study performed for the proposed home on Lot 43 Summit at Ski Lake No. 11 located at 6785 East Via Cortina Street in Huntsville, Utah. The general location of the site with respect to major roadways, as of 2014, is presented on Figure 1, Vicinity Map. A more detailed layout of the site showing the proposed improvements is presented on Figure 2, Site Plan. The locations of the test pits/trenches excavated in conjunction with this study are also presented on Figure 2.

1.2 OBJECTIVES AND SCOPE

The objectives and scope of our study were planned in discussions between Mr. Rich Zollinger, homeowner, and Mr. Andrew Harris of GSH Geotechnical, Inc. (GSH).

In general, the objectives of this study were to:

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

2. Provide appropriate foundation, earthwork, and slope stability recommendations as well as geoseismic information to be utilized in the design and construction of the proposed home.

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In accomplishing these objectives, our scope has included the following:

1. A field program consisting of the excavating, logging, and sampling of 4 test pits and 2 trenches.

2. A laboratory testing program.

3. An office program consisting of the correlation of available data, engineering analyses, and the preparation of this summary report.

1.3 AUTHORIZATION

Authorization was provided by returning a signed copy of our Professional Services Agreement No. 16-0123Nrev1 dated January 27, 2016.

1.4 PROFESSIONAL STATEMENTS

Supporting data upon which our recommendations are based are presented in subsequent sections of this report. Recommendations presented herein are governed by the physical properties of the soils encountered in the exploration test pits/trenches, projected groundwater conditions, and the layout and design data discussed in Section 2, Proposed Construction, of this report. If subsurface conditions other than those described in this report are encountered and/or if design and layout changes are implemented, GSH must be informed so that our recommendations can be reviewed and amended, if necessary.

Our professional services have been performed, our findings developed, and our recommendations prepared in accordance with generally accepted engineering principles and practices in this area at this time.

2. PROPOSED CONSTRUCTION

The proposed project consists of constructing a single-family residence on Lot 43 Summit at Ski Lake No. 11 in Huntsville, Utah. Construction will likely consist of reinforced concrete spread footings and basement foundation walls supporting 1 to 2 wood-framed levels above grade. Projected maximum column and wall loads are on the order of 10 to 20 kips and 1 to 3 kips per lineal foot, respectively.

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 2 to 8 feet. Larger cuts and fills may be required in isolated areas and must be planned to maintain stability of the site slopes.

3. INVESTIGATIONS

3.1 FIELD PROGRAM

In order to define and evaluate the subsurface soil and groundwater conditions at the site, 4 test pits and 2 trenches were excavated to depths of about 8.5 to 14.0 feet below existing grades. The test pits were excavated using a track-mounted excavator. The test pit and trench locations are presented on Figure 2.

The field portion of our study was under the direct control and continual supervision of an experienced member of our geotechnical staff. During the course of the excavating operations, a continuous log of the subsurface conditions encountered was maintained. In addition, samples of the typical soils encountered were obtained for subsequent laboratory testing and examination. Samples taken were placed in sealed plastic bags and containers. The soils were classified in the field based upon visual and textural examination. These classifications have been supplemented by subsequent inspection and testing in our laboratory. Detailed graphical representation of the subsurface conditions encountered is presented on Figures 3A through 3H, Test Pit Log. Soils were classified in accordance with the nomenclature described on Figure 4, Key to Test Pit Log (USCS).

Following completion of excavating and logging, each test pit/trench was backfilled. Although an effort was made to compact the backfill with the trackhoe, backfill was not placed in uniform lifts and compacted to a specific density. Consequently, settlement of the backfill with time is likely to occur.

3.2 LABORATORY TESTING

3.2.1 General

In order to provide data necessary for our engineering analyses, a laboratory testing program was performed. The program included moisture, density, Atterberg limits, partial gradations, consolidation, direct shear, and residual direct shear tests. The following paragraphs describe the tests and summarize the test data.

3.2.2 Moisture and Density

To provide index parameters and to correlate other test data, moisture and density tests were performed on selected samples. The results of these tests are presented on the test pit logs, Figures 3A through 3H.

3.2.3 Atterberg Limit Tests

To aid in classifying the soils, Atterberg limit tests were performed on samples of the fine-grained cohesive soils. Results of the test are tabulated on the following page.

Test Pit No.	Depth (feet)	Liquid Limit (percent)	Plastic Limit (percent)	Plasticity Index (percent)	Soil Classification
TP-1	7.0	59	36	23	MH
TP-2	2.5	49	19	30	CL
TR-1	8.0	50	32	18	MH

3.2.4 Partial Gradation Tests

To aid in classifying the granular soils, partial gradation tests were performed. Results of the tests are tabulated below:

Test Pit No.	Depth (feet)	Moisture Content Percent	Percent Passing No. 200 Sieve	Soil Classification
TP-1	7.0	31.2	57.9	CL
TP-2	2.5	21.4	55.5	CL
TR-1	3.5	7.2	10.1	SM

3.2.5 Consolidation Tests

To provide data necessary for our settlement analyses, consolidation tests were performed on each of 2 representative samples of the silty clay/clayey silt soils encountered at the site. Based upon data obtained from the consolidation tests, the silty clay/clayey silt soils are moderately over-consolidated and will exhibit moderate strength and compressibility characteristics under the anticipated loadings. Additionally, the silty clay/clayey silt soils exhibit a moderate expansive potential and swell pressure of about 600 to 800 psf. Detailed results of the test are maintained within our files and can be transmitted, at the client’s request.

3.2.6 Laboratory Direct Shear Test

To determine the shear strength of the soils encountered at the site, a laboratory direct shear test was performed on a sample of the site soils. The results of the test are tabulated on the following and on Figures 5 and 6, attached.

Test Pit No.	Depth (feet)	Soil Type	In-Situ Moisture Content (percent)	Dry Density (pcf)	Internal Friction Angle (degrees)	Apparent Cohesion (psf)
TP-1	7.0	MH/Bedrock	---	---	43	540
TR-1	3.5	SM*	14	89	38	25

*Remolded Sample

3.2.7 Laboratory Residual Direct Shear Test

To determine the residual shear strength of the soils encountered at the site, a laboratory residual direct shear test was performed on a sample of the site soils. The results of the test are tabulated below and on Figure 7, attached:

Trench/Test Pit No.	Depth (feet)	Soil Type	In-Situ Moisture Content (percent)	Dry Density (pcf)	Internal Friction Angle (degrees)	Apparent Cohesion (psf)
TR-1	2.5	CL	21	89	8	90

4. SITE CONDITIONS

4.1 GEOLOGIC SETTING

A geologic study[1] dated March 30, 2016 was prepared for the subject property by GSH Geotechnical, Inc., and a copy of that report is included in the attached Appendix.

4.2 SURFACE

The subject property is a vacant, rectangular-shaped lot located at 6785 East Via Cortina Street in Huntsville, Utah. The topography of the site is relatively flat with a slight slope to the south within the building area with an overall change in elevation of about 50 feet across the site. Vegetation at the site consists primarily of native weeds, grasses, and a number of mature trees, particular over the slope area. The site is bordered on the north by Via Cortina Street, on the east and south by undeveloped property, and on the east by residential development and undeveloped property.

[1] "Report, Geological Study, Proposed Residence, Lot 43 Summit at Ski Lake No. 11, Weber County, Utah," GSH Geotechnical, Inc., GSH Job No. 2063-01N-15, March 30, 2016.

4.3 SUBSURFACE SOIL

Subsurface conditions encountered at the test pit and trench locations varied slightly across the site. In a portion of Trench 1, Trench 2, and test pits TP-2 and TP-3, mass movement soil deposits were encountered below the topsoil and disturbed soils extending to about 6.0 to 9.0 feet below surrounding site grades. The mass movement deposits were comprised of a mixture of silty sand, clayey silt, silty clay, and degraded/weathered sandstone/siltstone. Natural soils were encountered beneath the mass movement deposit soils to the full depth penetrated, about 8.0 to 12.0 feet, and consisted of weathered siltstone bedrock. Natural soils were observed outside the mass movement deposits in Trench 1, Trench 2, and test pits TP-2 and TP-3, and to the full depth penetrated in test pits TP-1 and TP-4, about 10.0 to 11.0 feet below surrounding grades and consisted of silty clay, clayey silt, weathered siltstone, and fine to coarse sand with varying amounts of silt (weathered sandstone), and occasional mixture of these soils.

The natural sand soils encountered were medium dense to dense, dry to moist, light brown in color, and will generally exhibit moderately high strength and low compressibility characteristics under the anticipated loading.

The natural clay and silt soils encountered were medium stiff to hard, slightly moist to moist, light brown to gray in color, and will generally exhibit moderate strength and compressibility characteristics under the anticipated loading.

The siltstone and sandstone bedrock soils were dry to slightly moist, light brown to brown in color and weathered

For a more detailed description of the subsurface soils encountered, please refer to Figures 3A through 3H, Test Pit Log, and within the referenced geological study. The lines designating the interface between soil types on the test pit logs generally represent approximate boundaries. In-situ, the transition between soil types may be gradual.

4.4 GROUNDWATER

Groundwater was not encountered in the test pits and trenches at the time of our field exploration. Groundwater is anticipated to be at significant depths in the area. Seasonal and longer-term groundwater fluctuations on the order of 1.0 to 2.0 feet should be anticipated with the highest levels occurring during the late spring and summer months. Landscape irrigation on this and surrounding areas may also create additional seasonal groundwater fluctuations. The limitations of landscape irrigation at the site are discussed further in Section 5.9, Site Irrigation, and measures to reduce infiltration of surface water at the site are discussed further in Section 5.8, Subdrains.

5. DISCUSSIONS AND RECOMMENDATIONS

5.1 SUMMARY OF FINDINGS

The results of our analyses indicate that the proposed structure may be supported upon cast-in-place drilled piers extending a minimum of 10 feet into bedrock. Under no circumstance shall footings or structural fill be established in the existing mass movement deposit soils at the site.

The most significant geotechnical aspect of the site are the presence of mass movement deposit soils in the proposed home location and maintaining stability of the slope at the rear of the property.

The location of the home must be planned to avoid mass movement deposits at the site. If this is not feasible, all mass movement deposit soils must be removed to suitable natural soils below the structure and replaced with structural fill prior to the construction of the drilled pier foundation. Additionally, a subdrain system must be installed near the head of the mass movement deposit soils to reduce the potential for surface water infiltration, as discussed further within this report.

The on-site soils are not appropriate to be used as structural site grading fill, however, they may be used as general grading fill in landscape areas.

A geotechnical engineer from GSH will need to verify that all mass movement deposit soils, fill material (if encountered) and topsoil/disturbed soils have been completely removed and suitable natural soils encountered prior to the placement of structural site grading fills, floor slabs, drilled pier foundations, or rigid pavements. Additionally, drilled pier foundations must be observed prior to and during construction.

In the following sections, detailed discussions pertaining to earthwork, foundations, lateral pressure and resistance, floor slabs, slope stability, and the geoseismic setting of the site are provided.

5.2 EARTHWORK

5.2.1 Site Preparation

Initial site preparation will consist of the removal of surface vegetation , topsoil and any other deleterious materials from beneath an area extending out at least 3 feet from the perimeter of the proposed building and 2 feet beyond pavements and exterior flatwork areas.

All non-engineered fills such as backfill from test pits/trenches and mass movement deposit soils must be removed below all structures. In situ, non-engineered fills and mass movement deposit soils may remain below pavements if the owner accepts the risk of movement, if free of debris and deleterious materials, if less than 4 feet in thickness, and if properly prepared. Proper preparation will consist of the scarification of the upper 12 inches below asphalt concrete (flexible pavement) and 24 inches below rigid pavement followed by moisture preparation and re-compaction to the requirements of structural fill. The thicker sequence of prepared soils below rigid pavements would require the temporary removal of 12 inches of fill or mass movement deposit soils, scarifying, moisture conditioning, and recompacting the underlying 12 inches and backfilling with 12 inches of compacted suitable fills.

Even with proper preparation, pavements established overlying non-engineered fills and mass movement soil deposits may encounter some long-term movements unless the non-engineered fills and mass movement deposit soils are completely removed. Installing reinforcement in slabs over fills may help reduce potential displacement cracking.

It must be noted that from a handling and compaction standpoint, onsite soils containing high amounts of fines (silts and clays) are inherently more difficult to rework and 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. Additionally, the onsite soils are likely above optimum moisture content for compacting at present and would require some drying prior to recompacting. As an alternative, the fills may be removed and replaced with imported granular structural fill over unfrozen, proofrolled subgrade.

Subsequent to stripping and prior to the placement of structural site grading fill, pavements, driveway, and parking slabs on grade, the prepared subgrade must 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 to a maximum depth of 2 feet and replaced with structural fill. Beneath footings, all loose and disturbed soils must be totally removed. Fill soils must be handled as described above.

Surface vegetation, debris, and other deleterious materials shall generally be removed from the site. Topsoil, although unsuitable for utilization as structural fill, may be stockpiled for subsequent landscaping purposes.

A representative of GSH must verify that suitable natural soils and/or proper preparation of existing fills have been encountered/met prior to placing site grading fills, footings, slabs, and pavements.

5.2.2 Excavations

For granular (cohesionless) soils, construction excavations above the water table, not exceeding 4 feet, shall be no steeper than one-half horizontal to one vertical (0.5H:1V). For excavations up to 8 feet, in granular soils and above the water table, the slopes shall be no steeper than one horizontal to one vertical (1H:1V). Excavations encountering saturated cohesionless soils will be very difficult and will require very flat sideslopes and/or shoring, bracing and dewatering. Excavations deeper than 8 feet are not anticipated at the site.

Temporary excavations up to 8 feet deep in fine-grained cohesive soils (if encountered), above or below the water table, may be constructed with sideslopes no steeper than one-half horizontal to one vertical (0.5H:1V).

To reduce disturbance of the natural soils during excavation, it is recommended that smooth edge buckets/blades be utilized.

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.

5.2.3 Structural Fill

Structural fill will be required as site grading fill, as backfill over foundations and utilities, and possibly as replacement fill beneath some footings. All structural fill must be free of sod, rubbish, construction debris, frozen soil, and other deleterious materials.

Structural site grading fill is defined as fill placed over fairly large open areas to raise the overall site grade. The maximum particle size within structural site grading fill should generally not exceed 4 inches; although, occasional particles up to 6 to 8 inches may be incorporated provided that they do not result in "honeycombing" or preclude the obtainment of the desired degree of compaction. In confined areas, the maximum particle size should generally be restricted to 2.5 inches.

Only granular soils are recommended in confined areas such as utility trenches, below footings, etc. Generally, we recommend that all imported granular structural fill consist of a well-graded mixture of sands and gravels with no more than 20 percent fines (material passing the No. 200 sieve) and less than 30 percent retained on the 3/4 inch sieve. The plasticity index of import fine-grained soil shall not exceed 18 percent.

To stabilize soft subgrade conditions or where structural fill is required to be placed closer than 1.0 foot above the water table at the time of construction, a mixture of coarse gravels and cobbles and/or 1.5- to 2.0-inch gravel (stabilizing fill) should be utilized. It may also help to utilize a stabilization fabric, such as Mirafi 600X or equivalent, placed on the native ground if 1.5- to 2.0-inch gravel is used as stabilizing fill.

On-site soils are not recommended as structural fill but may be used as non-structural grading fill in landscape areas. Non-structural site grading fill is defined as all fill material not designated as structural fill and may consist of any cohesive or granular soils not containing excessive amounts of degradable material.

5.2.4 Fill Placement and Compaction

All structural fill shall be placed in lifts not exceeding 8 inches in loose thickness. Structural fills shall be compacted in accordance with the percent of the maximum dry density as determined by the ASTM[2] D-1557 (AASHTO[3] T-180) compaction criteria in accordance with the table on the following page.

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[2] American Society for Testing and Materials

[3] American Association of State Highway and Transportation Officials

Location	Total Fill Thickness (feet)	Minimum Percentage of Maximum Dry Density
Beneath an area extending at least 5 feet beyond the perimeter of the structure	0 to 8	95
Site Grading Fills outside area defined above	0 to 5	90
Site Grading Fills outside area defined above	5 to 8	95
Trench Backfill	--	96
Pavement granular base/subbase	--	96

Structural fills greater than 8 feet thick are not anticipated at the site.

Subsequent to stripping and prior to the placement of structural site grading fill, the subgrade shall be prepared as discussed in Section 5.2.1, Site Preparation, of this report. In confined areas, subgrade preparation shall consist of the removal of all loose or disturbed soils.

If utilized for stabilizing fill, coarse gravel and cobble mixtures should be end-dumped, spread to a maximum loose lift thickness of 15 inches, and compacted by dropping a backhoe bucket onto the surface continuously at least twice. As an alternative, the fill may be compacted by passing moderately heavy construction equipment or large self-propelled compaction equipment at least twice. Subsequent fill material placed over the coarse gravels and cobbles shall be adequately compacted so that the “fines” are “worked into” the voids in the underlying coarser gravels and cobbles.

5.2.5 Utility Trenches

All utility trench backfill material below structurally loaded facilities (flatwork, floor slabs, roads, 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 compacted prior to the construction of any exterior flatwork over a backfilled trench. Proofrolling may 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 must 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-1-a/A-1-b (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. We recommend that as the major utilities continue onto the site that these compaction specifications are followed.

The natural or imported silt/clay soils are not recommended for use as trench backfill, particularly in structurally loaded areas.

5.3 SLOPE STABILITY

5.3.1 Parameters

The properties of the soils at this site were estimated using the results of our laboratory testing, published correlations, and our experience with similar soils. Accordingly, we estimated the following parameters for use in the stability analyses:

Accordingly, we estimated the following parameters for use in the stability analyses:

Material	Internal Friction Angle (degrees)	Apparent Cohesion (psf)	Saturated Unit Weight (pcf)
Bedrock Soils	30	100	125
Silty Clay (Colluvium)	8	80	120
Mass Movement	8	0	120
Concrete	0	288,000	150

For the seismic (pseudostatic) analysis, a peak horizontal ground acceleration of 0.33*g* with a 2 percent probability of exceedance in 50 years was obtained for site (grid) locations of 41.2429 degrees latitude (north) and 111.7850 degrees longitude (west). To model sustained accelerations at the site, one-half of this value is typically employed. Accordingly, a value of 0.17g was used as the pseudostatic coefficient for the stability analysis.

5.3.2 Stability Analyses

We evaluated the global stability of the existing slope using the computer program *SLIDE*. This program uses a limit equilibrium (Simplified Bishop) method for calculating factors of safety against sliding on an assumed failure surface and evaluates numerous potential failure surfaces, with the most critical failure surface identified as the one yielding the lowest factor of safety of those evaluated. We analyzed the following configuration based on cross-sections provided in the referenced geologic study (see appendix for cross-section information and location):

- A relatively flat roadway area grading downward to the proposed home location and slope at grades ranging from about 6H:1V (Horizontal to Vertical) to 3H:1V (Horizontal:Vertical). To simulate the load imposed on the slope by the proposed home, a load of 1,500 psf was modeled over the proposed building area (outside the mass movement soil deposit). In addition, a phreatic surface was included to account for potential water from seasonal runoff and snowmelt.

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 existing slope configuration combined with the home loading will not meet these requirements. To improve the stability of the slope and reduce the potential for damage to the structure, a grid of 18-inch diameter concrete piers spaced 10 to 15 feet apart within the proposed home area were included in the model. The results of our analyses indicate that the minimum static factor of safety will be met provided our recommendations are followed. The minimum seismic factor of safety was less than 1.0, thus a deformation analysis was performed for a yield acceleration (factor of safety equal to 1.0) of 0.115g using the Newmark sliding analysis. This evaluation indicates less than 2.5 inches of slope deformation is anticipated for an earthquake generating a peak horizontal acceleration of 0.34g. The slope stability and deformation data are included as Figures 8 through 12, attached.

Slope movements or even failure can occur if the slope soils are undermined or become saturated. Groundwater was not encountered during the course of our field investigation; however saturation of the slope soils can adversely affect the stability of the slope. Measures must be implemented to reduce the potential for saturation of the soils at the site. Surface drainage at the bottom and top of the slope should be directed to prevent ponding at the toe or crest of the slope, and a cut-off drain on the slope above the homes is recommended to reduce the potential for infiltration of surface water at the site, as discussed further in Section 5.8, Subdrains. Landscape irrigation on this and surrounding areas may also create additional seasonal groundwater fluctuations. The limitations of landscape irrigation at the site are discussed further in Section 5.9, Site Irrigation. The property owner and the owner's representatives should be made aware of the risks should these or other conditions occur that could saturate or erode/undermine the slope soils.

Changes to the grading at the site and any retaining walls must be properly engineered to maintain stability of the slopes. GSH must review the final grading plans for the project prior to initiation of any construction.

5.4 DRILLED PIER FOUNDATIONS

5.4.1 Design Parameters

To minimize the impact of the proposed home on the slope, structural loads must be carried to suitable bedrock materials through a cast-in-place drilled pier system. Drilled piers must be a minimum of 18 inches in diameter and must extend a minimum of 10 feet into the bedrock soils below the proposed home. An end-bearing pressure of 1,500 psf and a skin friction of 250 psf may be utilized for design of piers with in the bedrock.

5.4.2 Pier Spacing

Pier spacing is recommended to be not less than three times the diameter of the pier or 10 feet, whichever is greater. No reduction in load carrying capacity, due to group action, should be necessary with this spacing.

5.4.3 Settlements

Static settlements of drilled piers designed with a minimum embedment depth of 10 feet are projected to be less than 1 inch.

5.4.4 Installation

The pier excavation shall be inspected to ensure it is clean of loose soil that may slough into the excavation. The pier excavation should have a straight smooth side and not be allowed to flare near the ground surface. The excavation shall be inspected for irregularities that may affect the pier performance to determine if the excavation meets the structural engineer's design tolerances. The pier should be reinforced its entire length. Concrete shall be placed immediately following drilling to reduce the safety risk of the open excavation.

Concrete shall be pumped or tremmied to the bottom of the excavation and not allowed to free-fall more than 3 feet. Placement of the concrete shall continue to be pumped until all floating water/cement paste is expelled and coarse aggregate is visible at the surface. The volume of concrete shall be compared with planned pier volume.

5.5 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 foundations and the supporting soils. In determining frictional resistance, a coefficient of 0.30 should be utilized for foundations placed over natural soils and bedrock. 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 300 pounds per cubic foot. Below the water table, this granular soil should be considered equivalent to a fluid with a density of 150 pounds per cubic foot.

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

5.6 LATERAL PRESSURES

The lateral pressure parameters, as presented within this section, are for backfills which will consist of drained granular soil placed and compacted in accordance with the recommendations presented herein. The lateral pressures imposed upon subgrade facilities will, therefore, be basically dependent upon the relative rigidity and movement of the backfilled structure. For active walls, such as retaining walls which can move outward (away from the backfill), granular backfill may be considered equivalent to a fluid with a density of 35 pounds per cubic foot in computing lateral pressures. For more rigid walls (moderately yielding), generally not exceeding 8 feet in height, granular backfill may be considered equivalent to a fluid with a density of 45 pounds per cubic foot. The above values assume that the surface of the soils slope behind the wall is no steeper than 4 horizontal to 1 vertical and that the granular fill within 3 feet of the wall will be compacted with hand-operated compacting equipment.

For seismic loading, a uniform pressure shall be added. The uniform pressures based on different wall heights are provided in the following table:

Wall Height (feet)	**Seismic Loading Active Case (psf)**	**Seismic Loading Moderately Yielding (psf)**
4	25	55
6	40	85
8	55	115

5.7 FLOOR SLABS

Floor slabs may be established upon suitable natural soils and/or upon structural fill extending to suitable natural soils. Under no circumstances shall floor slabs be established over mass movement deposit soils, non-engineered fills, loose or disturbed soils, sod, rubbish, construction debris, other deleterious materials, frozen soils, or within ponded water. In order to provide a capillary break and facilitate curing of the concrete, it is recommended that floor slabs be directly underlain by 4 inches of "free-draining" fill, such as "pea" gravel or three-quarters- to one-inch minus clean gap-graded gravel.

Settlement of lightly loaded floor slabs (average uniform pressure of 200 pounds per square foot or less) is anticipated to be less than 1/4 inch.

The tops of all floor slabs in habitable areas must be established at least 4 feet above the highest anticipated normal water level or 1.5 feet above the maximum groundwater level controlled by land drains.

5.8 SUBDRAINS

5.8.1 General

Groundwater was not encountered at the site, however we recommend that the perimeter foundation subdrains and a cutoff drain near the head of the mass movement deposit soils be installed as indicated below.

5.8.2 Foundation Subdrains

Foundation subdrains should consist of a 4-inch diameter perforated or slotted plastic or PVC pipe enclosed in clean gravel. The invert of a subdrain should be at least 2 feet below the top of the lowest adjacent 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 subdrain, a minimum 4-inch-wide zone of "free-draining" sand/gravel should be placed adjacent to the foundation walls and extend to within 2 feet of final grade. The upper 2 feet of soils should consist of a compacted clayey cap to reduce surface water infiltration into the drain. As an alternative to the zone of permeable sand/gravel, a prefabricated "drainage board," such as Miradrain or equivalent, may be placed adjacent to 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 gravel placed around the drain pipe should be clean 0.75-inch to 1.0-inch minus gap-graded gravel and/or "pea" gravel. The foundation subdrains can be discharged into the area subdrains, storm drains, or other suitable down-gradient location.
We recommend final site grading slope away from the structures at a minimum 2 percent for hard surfaces (pavement) and 5 percent for soil surfaces within the first 10 feet from the structures.

5.8.3 Cutoff Drain

To reduce potential infiltration of surface water and groundwater into the subsurface soils at the site, a cutoff drain should be installed near the head of the mass movement deposit soils. The drain should consist of a perforated 4-inch minimum diameter pipe wrapped in fabric and placed near the bottom of a minimum 24 inch wide trench excavated to a depth of at least 15 feet below existing grade or bedrock and lined in filter fabric. The pipe should daylight at one or both ends of the drain and discharge to an appropriate drainage device or area. Clean gravel up to 2 inches in maximum size, with less than 10 percent passing the No. 4 sieve and less than 5 percent passing the No. 200 sieve, should be placed around the drain pipe. A fabric, such as Mirafi 140N or equivalent, should be placed between the clean gravel and the adjacent soils. A zone of clean gravel and fabric at least 24 inches wide should also extend above the drain, to within 2 feet of the ground surface, with fabric placed over the gravel. The upper 2 feet of soils should consist of a compacted clayey cap to reduce surface water infiltration into the drain.

5.9 SITE IRRIGATION

Proper site drainage is important to maintaining slope stability at the site. Saturation of soils at the site may result in slope movement or failure. Therefore, we recommend that no irrigation lines should be placed on the slope. Landscaping at the site should be planned to utilize drought resistant plants that require minimal watering. Plants or lawn may be placed on the slope, with plants watered using direct drip systems targeted only for each plant, and any lawn areas watered using sprinklers placed a minimum of 30 feet from the slope. Overwatering should be strictly avoided. The surface of the site should be graded to prevent the accumulation or ponding of surface water at the site. The property owner and the owner's representatives should be made aware of the risks should these or other conditions occur that could saturate or erode/undermine the slope soils.

To reduce the potential for saturation of the site soils, overwatering at the site should be strictly avoided. Watering at the site should be limited to a maximum equivalent rainfall of 0.5 inches per week. Irrigation at the site should be strictly avoided during periods of natural precipitation.

5.10 GEOSEISMIC SETTING

5.10.1 General

Utah municipalities have adopted the International Building Code (IBC) 2012. The IBC 2012 code determines the seismic hazard for a site based upon 2008 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).

The structure must be designed in accordance with the procedure presented in Section 1613, Earthquake Loads, of the IBC 2012 edition.

5.10.2 Faulting

Based upon our review of available literature, no active faults are known to pass through the site. The nearest active fault is the Wasatch Fault Zone Weber Section, approximately 7.4 miles west of the site.

5.10.3 Soil Class

For dynamic structural analysis, the Site Class D – Stiff Soil Profile as defined in Chapter 20 of ASCE 7 (per Section 1613.3.2, Site Class Definitions, of IBC 2012) can be utilized.

5.10.4 Ground Motions

The IBC 2012 code is based on 2008 USGS mapping, which provides values of short and long period accelerations for the Site Class B boundary for 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 following table summarizes the peak ground and short and long period accelerations for the MCE event and incorporates the appropriate soil amplification factor for a Site Class C soil profile. Based on the site latitude and longitude (41.2429 degrees north and -111.7850 degrees west, respectively), the values for this site are tabulated on the following page.

Spectral Acceleration Value, T	Site Class B Boundary [mapped values] (% g)	Site Coefficient	Site Class D [adjusted for site class effects] (% g)	Design Values (% g)
Peak Ground Acceleration	33.3	F_a = 1.167	38.9	25.9
0.2 Seconds (Short Period Acceleration)	S_S = 83.3	F_a = 1.167	S_{MS} = 97.2	S_{DS} = 64.8
1.0 Second (Long Period Acceleration)	S_1 = 28.0	F_v = 1.840	S_{M1} = 51.5	S_{D1} = 34.3

5.10.5 Liquefaction

The site is located in an area that has been identified by the Utah Geologic Survey as having "very low" liquefaction potential. Liquefaction is defined as the condition when saturated, loose, finer-grained sand-type soils lose their support capabilities because of excessive pore water pressure which develops during a seismic event. Clay soils, even if saturated, will generally not liquefy.

Liquefaction of the site soils is not anticipated during the design seismic event due to the unsaturated nature of the site soils.

5.11 SITE OBSERVATIONS

As stated previously, prior to placement of foundations, floor slabs, pavements, and site grading fills, a geotechnical engineer from GSH must verify that all mass movement deposit soils, non-engineered fill materials, topsoil, and disturbed soils have been removed and/or properly prepared and suitable subgrade conditions encountered. Also, drilled pier foundations must be observed prior to and during construction. Additionally, GSH must observe fill placement and verify in-place moisture content and density of fill materials placed at the site.

5.12 CLOSURE

If you have any questions or would like to discuss these items further, please feel free to contact us at (801) 393-2012.

Respectfully submitted,

GSH Geotechnical, Inc.

Andrew M. Harris, P.E.
State of Utah No. 740456
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Reviewed by:

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Senior Geotechnical Engineer

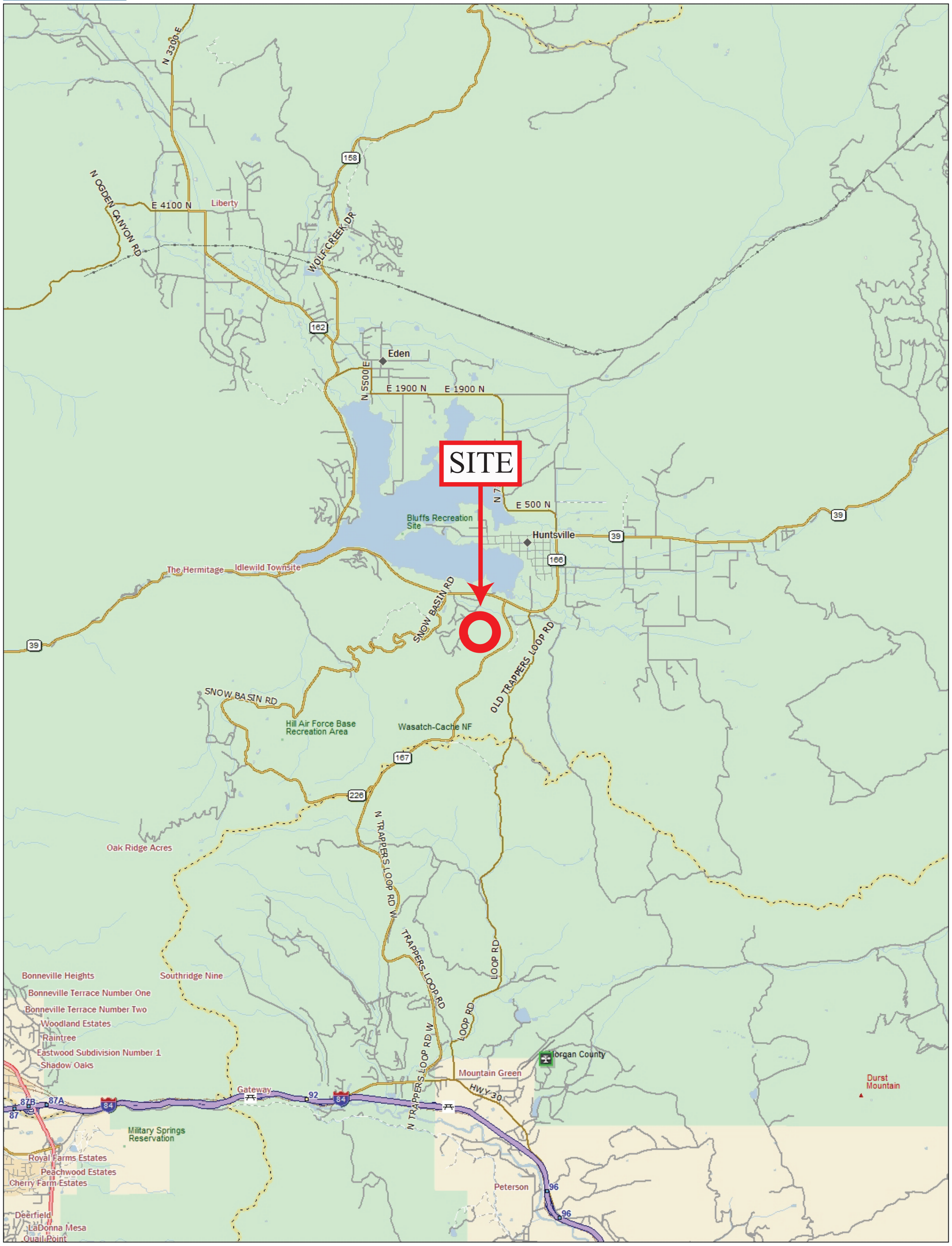
AMH/WGT:mmh

Encl. Figure 1, Vicinity Map
Figure 2, Site Plan
Figures 3A through 3H, Test Pit Logs
Figure 4, Key to Test Pit Log (USCS)
Figures 5 through 7, Direct Shear Test
Figures 8 through 11, Stability Results
Figure 12, Newmark Displacement Method – Analysis Results
Appendix

Addressee (Email)

RICH ZOLLINGER
JOB NO. 2063-01N-16

DeLorme Street Atlas USA® 2014



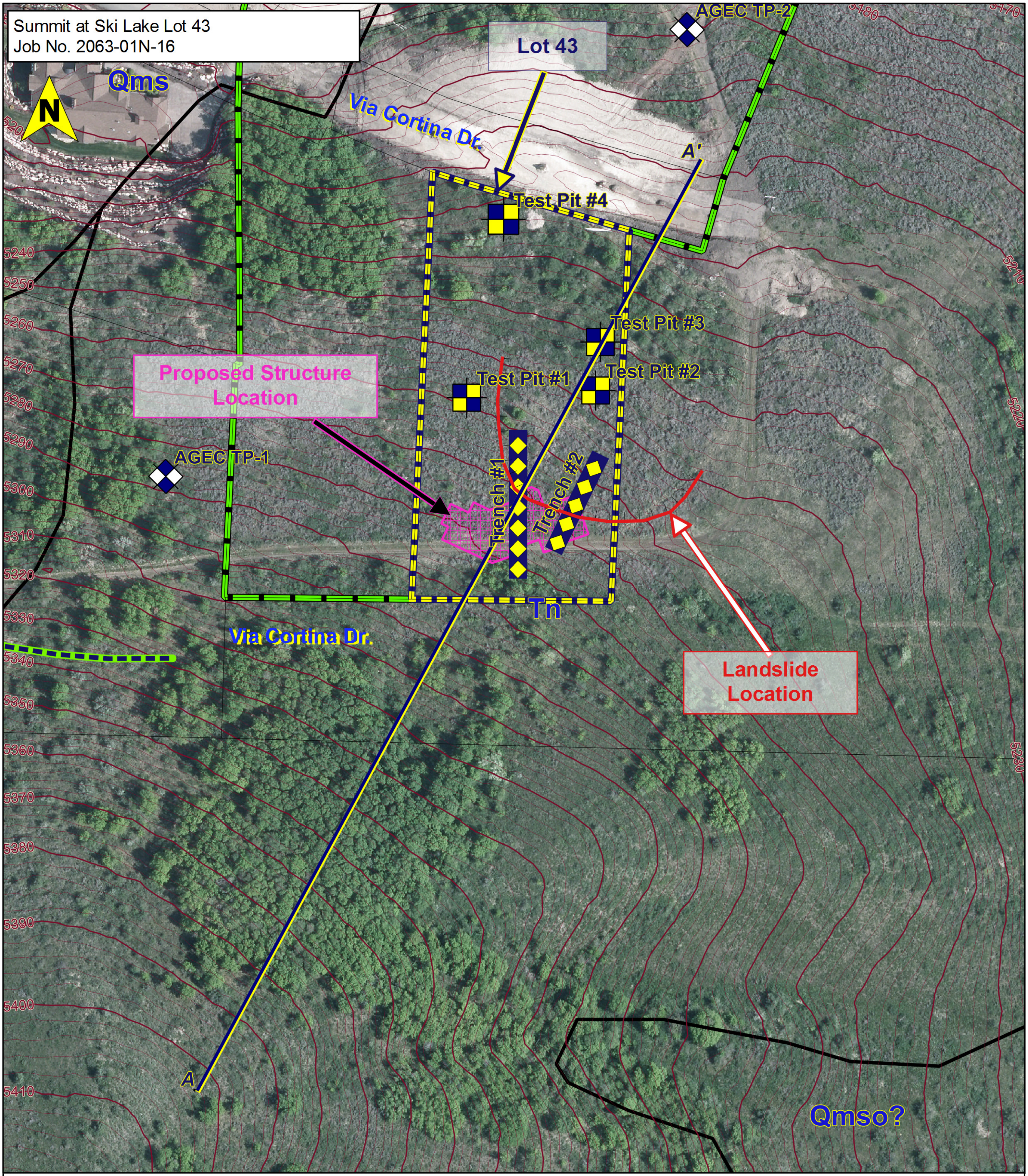
mi
0 1 2 3
Data Zoom 11-0

FIGURE 1

VICINITY MAP

REFERENCE:
DELORME STREET ATLAS

Summit at Ski Lake Lot 43
Job No. 2063-01N-16



Base:
2012 0.5ft Color HRO Orthoimagery,
from Utah AGRC. http://gis.utah.gov/

0 50 100 150 200 ft

1:1,200

FIGURE 2
SITE PLAN

GSH

TEST PIT LOG

Page: 1 of 1

TEST PIT: TP-1

CLIENT: Rich Zollinger | PROJECT NUMBER: 2063-01N-16

PROJECT: Lot 43 Summit at Ski Lake No. 11 | DATE STARTED: 2/22/16 | DATE FINISHED: 2/22/16

LOCATION: 6785 East Via Cortina Street, Huntsville, Utah | GSH FIELD REP.: AA

EXCAVATING METHOD/EQUIPMENT: KOMATSU - Trackhoe

GROUNDWATER DEPTH: Not Encountered (2/22/16) | ELEVATION: ---

WATER LEVEL	USCS	DESCRIPTION	DEPTH (FT.)	SAMPLE SYMBOL	MOISTURE (%)	DRY DENSITY (PCF)	% PASSING 200	LIQUID LIMIT (%)	PLASTICITY INDEX	REMARKS
		Ground Surface	0							
	CL	SILTY CLAY with some fine to coarse sand; major roots (topsoil) to 6"; brown								moist medium stiff stiff slightly moist
	SM	SILTY FINE TO COARSE SAND/WEATHERED SANDSTONE light brown	5							dry dense
	MH	CLAYEY SILT/WEATHERED SANDSTONE with some fine to coarse sand; brown			31		58	59	23	slightly moist very stiff
		End of Exploration at 10.0' No significant sidewall caving No groundwater encountered at time of excavation	10							
			15							
			20							
			25							

See Subsurface Conditions section in the report for additional information.

FIGURE 3A

TEST PIT LOG

GSH

Page: 1 of 1

TEST PIT: TP-2

CLIENT: Rich Zollinger — PROJECT NUMBER: 2063-01N-16

PROJECT: Lot 43 Summit at Ski Lake No. 11 — DATE STARTED: 2/22/16 — DATE FINISHED: 2/22/16

LOCATION: 6785 East Via Cortina Street, Huntsville, Utah — GSH FIELD REP.: AA

EXCAVATING METHOD/EQUIPMENT: KOMATSU - Trackhoe

GROUNDWATER DEPTH: Not Encountered (2/22/16) — ELEVATION: ---

WATER LEVEL	USCS	DESCRIPTION	DEPTH (FT.)	SAMPLE SYMBOL	MOISTURE (%)	DRY DENSITY (PCF)	% PASSING 200	LIQUID LIMIT (%)	PLASTICITY INDEX	REMARKS
		Ground Surface								
	CL	SILTY CLAY with some fine to coarse sand; major roots (topsoil) to 6"; brown	0							moist medium stiff
		light brown			21		56	49	30	slightly moist stiff
			5							
	ML/ BEDROCK	CLAYEY SILT/WEATHERED SILTSTONE light brown								slightly moist very stiff to hard
		End of Exploration at 10.0' No significant sidewall caving No groundwater encountered at time of excavation	10							
			15							
			20							
			25							

See Subsurface Conditions section in the report for additional information.

FIGURE 3B

TEST PIT LOG

Page: 1 of 1

TEST PIT: TP-3

CLIENT: Rich Zollinger — PROJECT NUMBER: 2063-01N-16

PROJECT: Lot 43 Summit at Ski Lake No. 11 — DATE STARTED: 2/22/16 — DATE FINISHED: 2/22/16

LOCATION: 6785 East Via Cortina Street, Huntsville, Utah — GSH FIELD REP.: AA

EXCAVATING METHOD/EQUIPMENT: KOMATSU - Trackhoe

GROUNDWATER DEPTH: Not Encountered (2/22/16) — ELEVATION: ---

WATER LEVEL	USCS	DESCRIPTION	DEPTH (FT.)	SAMPLE SYMBOL	MOISTURE (%)	DRY DENSITY (PCF)	% PASSING 200	LIQUID LIMIT (%)	PLASTICITY INDEX	REMARKS
		Ground Surface								
	CL	SILTY CLAY with some fine to coarse sand; major roots (topsoil) to 6"; brown light brown	0							moist medium stiff
	SM/ BEDROCK	SILTY FINE TO COARSE SAND/WEATHERED SANDSTONE light brown	5							slightly moist dense to very dense
	ML/ BEDROCK	CLAYEY SILT/WEATHERED SILTSTONE light brown	10							slightly moist very stiff
		End of Exploration at 10.5' No significant sidewall caving No groundwater encountered at time of excavation	15 20 25							

See Subsurface Conditions section in the report for additional information.

FIGURE 3C

TEST PIT LOG

Page: 1 of 1

TEST PIT: TP-4

CLIENT: Rich Zollinger — PROJECT NUMBER: 2063-01N-16

PROJECT: Lot 43 Summit at Ski Lake No. 11 — DATE STARTED: 2/22/16 — DATE FINISHED: 2/22/16

LOCATION: 6785 East Via Cortina Street, Huntsville, Utah — GSH FIELD REP.: AMH

EXCAVATING METHOD/EQUIPMENT: KOMATSU - Trackhoe

GROUNDWATER DEPTH: Not Encountered (2/22/16) — ELEVATION: ---

WATER LEVEL	USCS	DESCRIPTION	DEPTH (FT.)	SAMPLE SYMBOL	MOISTURE (%)	DRY DENSITY (PCF)	% PASSING 200	LIQUID LIMIT (%)	PLASTICITY INDEX	REMARKS
		Ground Surface								
	CL	SILTY CLAY major roots (topsoil) to 6"; brown to dark brown	0							moist medium stiff
	ML/ BEDROCK	CLAYEY SILT/WEATHERED SILTSTONE light brown to brown	5 10							slightly moist very stiff
		End of Exploration at 11.0' No significant sidewall caving No groundwater encountered at time of excavation	15 20 25							

See Subsurface Conditions section in the report for additional information.

FIGURE 3D

TEST PIT LOG

GSH

Page: 1 of 1

TEST PIT: TR-1A (South End)

CLIENT: Rich Zollinger | PROJECT NUMBER: 2063-01N-16

PROJECT: Lot 43 Summit at Ski Lake No. 11 | DATE STARTED: 2/19/16 | DATE FINISHED: 2/19/16

LOCATION: 6785 East Via Cortina Street, Huntsville, Utah | GSH FIELD REP.: AA

EXCAVATING METHOD/EQUIPMENT: KOMATSU - Trackhoe

GROUNDWATER DEPTH: Not Encountered (2/19/16) | ELEVATION: ---

WATER LEVEL	USCS	DESCRIPTION	DEPTH (FT.)	SAMPLE SYMBOL	MOISTURE (%)	DRY DENSITY (PCF)	% PASSING 200	LIQUID LIMIT (%)	PLASTICITY INDEX	REMARKS
		Ground Surface								
	CL	SILTY CLAY with some fine to coarse sand; brown	0							slightly moist medium stiff
				▌	21	89				
	ML/ BEDROCK	CLAYEY SILT/WEATHERED SILTSTONE light brown to gray	5	▌						slightly moist
		End of Exploration at 5.5' No significant sidewall caving No groundwater encountered at time of excavation	10 15 20 25							

See Subsurface Conditions section in the report for additional information.

FIGURE 3E

GSH

TEST PIT LOG

TEST PIT: TR-1B (North End)

CLIENT: Rich Zollinger	PROJECT NUMBER: 2063-01N-16
PROJECT: Lot 43 Summit at Ski Lake No. 11	DATE STARTED: 2/19/16 DATE FINISHED: 2/19/16
LOCATION: 6785 East Via Cortina Street, Huntsville, Utah	GSH FIELD REP.: AA
EXCAVATING METHOD/EQUIPMENT: KOMATSU - Trackhoe	
GROUNDWATER DEPTH: Not Encountered (2/19/16)	ELEVATION: ---

WATER LEVEL	USCS	DESCRIPTION	DEPTH (FT.)	SAMPLE SYMBOL	MOISTURE (%)	DRY DENSITY (PCF)	% PASSING 200	LIQUID LIMIT (%)	PLASTICITY INDEX	REMARKS
		Ground Surface								
	CL	SILTY CLAY with some fine to coarse sand; organics and rootholes; major roots (topsoil) to 6"; dark brown	0							moist medium stiff
	SP/SM/BEDROCK	FINE TO MEDIUM SAND/WEATHERED SANDSTONE with silt; mass movement; light brown			7		10			moist dense
	MH/BEDROCK	CLAYEY SILT/WEATHERED SILTSTONE with trace fine to coarse sand; gray	5					50	18	slightly moist very stiff to hard
			10							
		End of Exploration at 12.0' No significant sidewall caving No groundwater encountered at time of excavation								
			15							
			20							
			25							

See Subsurface Conditions section in the report for additional information.

FIGURE 3F

TEST PIT LOG

Page: 1 of 1

TEST PIT: TR-2A (South End)

CLIENT: Rich Zollinger — PROJECT NUMBER: 2063-01N-16

PROJECT: Lot 43 Summit at Ski Lake No. 11 — DATE STARTED: 2/19/16 — DATE FINISHED: 2/19/16

LOCATION: 6785 East Via Cortina Street, Huntsville, Utah — GSH FIELD REP.: AMH

EXCAVATING METHOD/EQUIPMENT: KOMATSU - Trackhoe

GROUNDWATER DEPTH: Not Encountered (2/19/16) — ELEVATION: ---

WATER LEVEL	USCS	DESCRIPTION	DEPTH (FT.)	SAMPLE SYMBOL	MOISTURE (%)	DRY DENSITY (PCF)	% PASSING 200	LIQUID LIMIT (%)	PLASTICITY INDEX	REMARKS
		Ground Surface	0							
	CL	SILTY CLAY brown to dark brown								moist medium stiff slightly moist stiff
			5							
	MH/BEDROCK	CLAYEY SILT/WEATHERED SILTSTONE light brown to brown								slightly moist very stiff
		End of Exploration at 7.0' No significant sidewall caving No groundwater encountered at time of excavation								
			10							
			15							
			20							
			25							

See Subsurface Conditions section in the report for additional information.

FIGURE 3G

TEST PIT LOG

Page: 1 of 1

TEST PIT: TR-2B (North End)

CLIENT: Rich Zollinger	PROJECT NUMBER: 2063-01N-16
PROJECT: Lot 43 Summit at Ski Lake No. 11	DATE STARTED: 2/19/16 DATE FINISHED: 2/19/16
LOCATION: 6785 East Via Cortina Street, Huntsville, Utah	GSH FIELD REP.: AMH
EXCAVATING METHOD/EQUIPMENT: KOMATSU - Trackhoe	
GROUNDWATER DEPTH: Not Encountered (2/19/16)	ELEVATION: ---

WATER LEVEL	USCS	DESCRIPTION	DEPTH (FT.)	SAMPLE SYMBOL	MOISTURE (%)	DRY DENSITY (PCF)	% PASSING 200	LIQUID LIMIT (%)	PLASTICITY INDEX	REMARKS
		Ground Surface	0							
	CL	SILTY CLAY brown								moist medium stiff slightly moist stiff
	MH/ BEDROCK	CLAYEY SILT/WEATHERED SILTSTONE light brown to brown	5							slightly moist very stiff
			10							
		End of Exploration at 11.0' No significant sidewall caving No groundwater encountered at time of excavation								
			15							
			20							
			25							

See Subsurface Conditions section in the report for additional information.

FIGURE 3H

CLIENT: Rich Zollinger PROJECT: Lot 43 Summit at Ski Lake No. 11 PROJECT NUMBER: 2063-01N-16	**KEY TO TEST PIT LOG**

WATER LEVEL	USCS	DESCRIPTION	DEPTH (FT.)	SAMPLE SYMBOL	MOISTURE (%)	DRY DENSITY (PCF)	% PASSING 200	LIQUID LIMIT (%)	PLASTICITY INDEX	REMARKS
①	②	③	④	⑤	⑥	⑦	⑧	⑨	⑩	⑪

COLUMN DESCRIPTIONS

① **Water Level:** Depth to measured groundwater table. See symbol below.

② **USCS:** (Unified Soil Classification System) Description of soils encountered; typical symbols are explained below.

③ **Description:** Description of material encountered; may include color, moisture, grain size, density/consistency,

④ **Depth (ft.):** Depth in feet below the ground surface.

⑤ **Sample Symbol:** Type of soil sample collected at depth interval shown; sampler symbols are explained below.

⑥ **Moisture (%):** Water content of soil sample measured in laboratory; expressed as percentage of dryweight of

⑦ **Dry Density (pcf):** The density of a soil measured in laboratory; expressed in pounds per cubic foot.

⑧ **% Passing 200:** Fines content of soils sample passing a No. 200 sieve; expressed as a percentage.

⑨ **Liquid Limit (%):** Water content at which a soil changes from plastic to liquid behavior.

⑩ **Plasticity Index (%):** Range of water content at which a soil exhibits plastic properties.

⑪ **Remarks:** Comments and observations regarding drilling or sampling made by driller or field personnel. May include other field and laboratory test results using the following abbreviations:

CEMENTATION:	MODIFIERS:	MOISTURE CONTENT (FIELD TEST):
Weakly: Crumbles or breaks with handling or slight finger pressure.	**Trace** <5%	**Dry:** Absence of moisture, dusty, dry to the touch.
Moderately: Crumbles or breaks with considerable finger pressure.	**Some** 5-12%	**Moist:** Damp but no visible water.
Strongly: Will not crumble or break with finger pressure.	**With** > 12%	**Saturated:** Visible water, usually soil below water table.

Descriptions and stratum lines are interpretive; field descriptions may have been modified to reflect lab test results. Descriptions on the logs apply only at the specific boring locations and at the time the borings were advanced; they are not warranted to be representative of subsurface conditions at other locations or times.

UNIFIED SOIL CLASSIFICATION SYSTEM (USCS)

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

Note: Dual Symbols are used to indicate borderline soil classifications.

STRATIFICATION:

DESCRIPTION	THICKNESS
Seam	up to 1/8"
Layer	1/8" to 12"

Occasional: One or less per 6" of thickness
Numerous; More than one per 6" of thickness

TYPICAL SAMPLER GRAPHIC SYMBOLS

- Bulk/Bag Sample
- Standard Penetration Split Spoon Sampler
- Rock Core
- No Recovery
- 3.25" OD, 2.42" ID D&M Sampler
- 3.0" OD, 2.42" ID D&M Sampler
- California Sampler
- Thin Wall

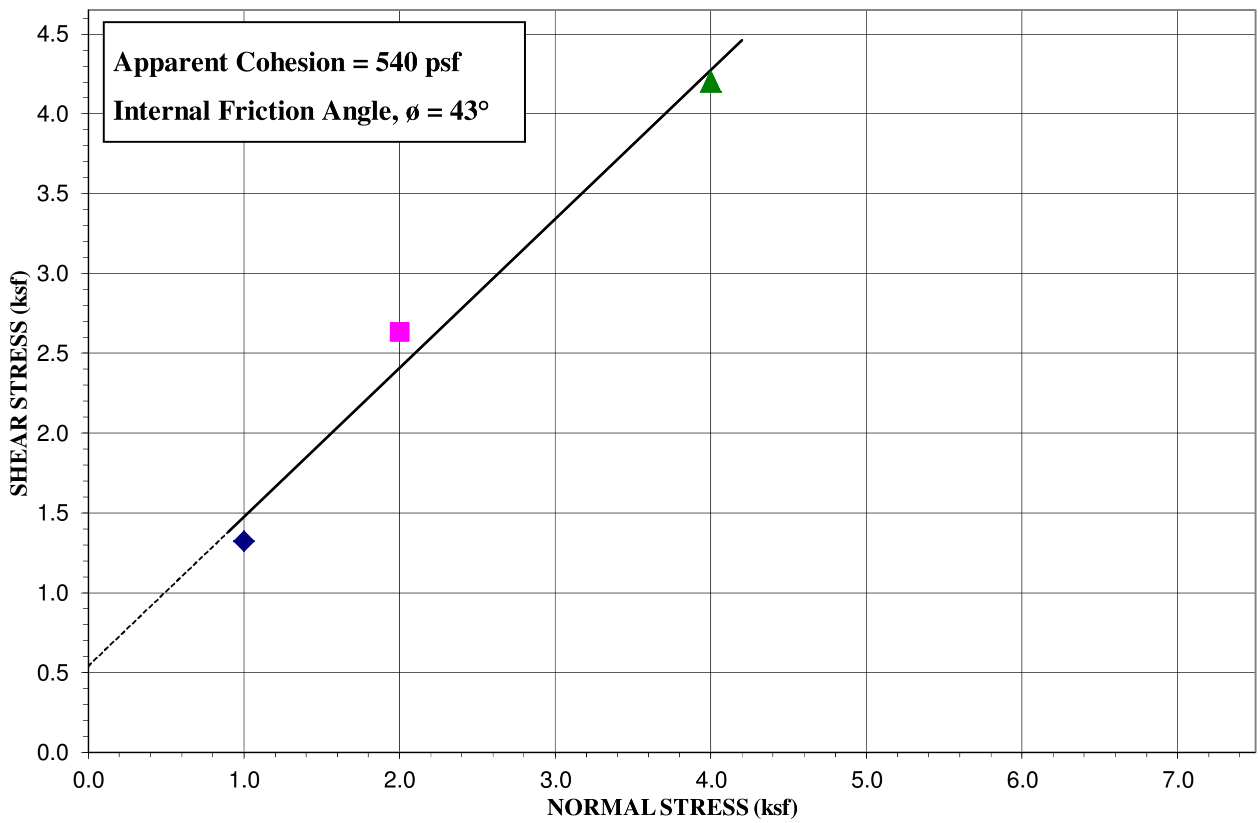
WATER SYMBOL

Water Level

FIGURE 4

GSH

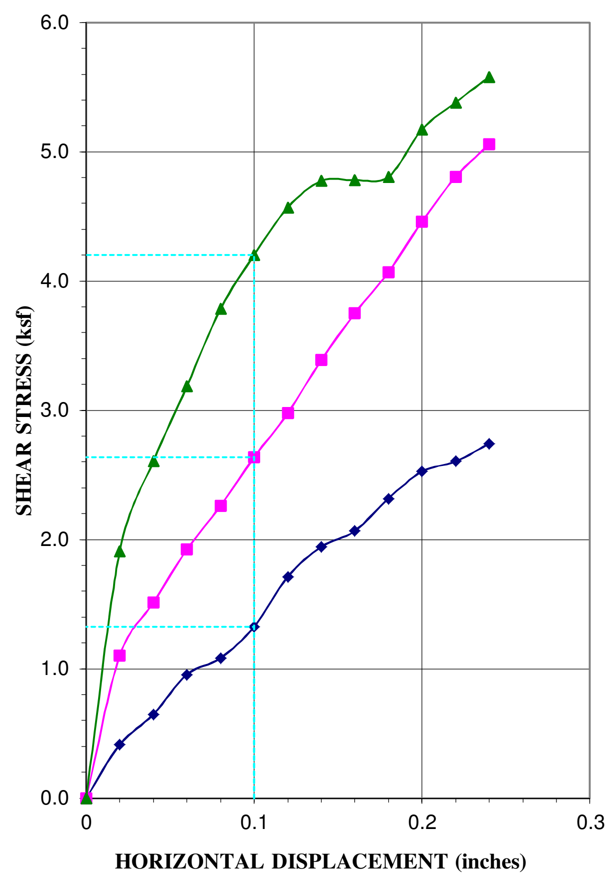
DIRECT SHEAR TEST



Source: TP-1	Depth: 7.0 ft		
Type of Test:	Consolidated Drained		
Sample Type	Thinwall		
Strain Rate	0.01 in/min		
Test No. (Symbol)	1 (◆)	2 (■)	3 (▲)
Initial Height, in.	1	1	1
Diameter, in.	2.42	2.42	2.42
Dry Density Before, pcf			
Dry Density After, pcf	-38.2	-37.9	-37.1
Moisture % Before			
Moisture % After	-415.9	-415.9	-415.9
Normal Load, ksf	1.0	2.0	4.0
Shear Stress, ksf	1.32	2.64	4.20
Vertical Deformation, in	1.0	2.0	4.0

Sample Properties	
Cohesion, psf	540
Friction Angle, ϕ	43
Liquid Limit, %	---
Plasticity Index, %	---
Percent Gravel	---
Percent Sand	---
Percent Passing #200 sieve	---
Classification	MH/Bedrock

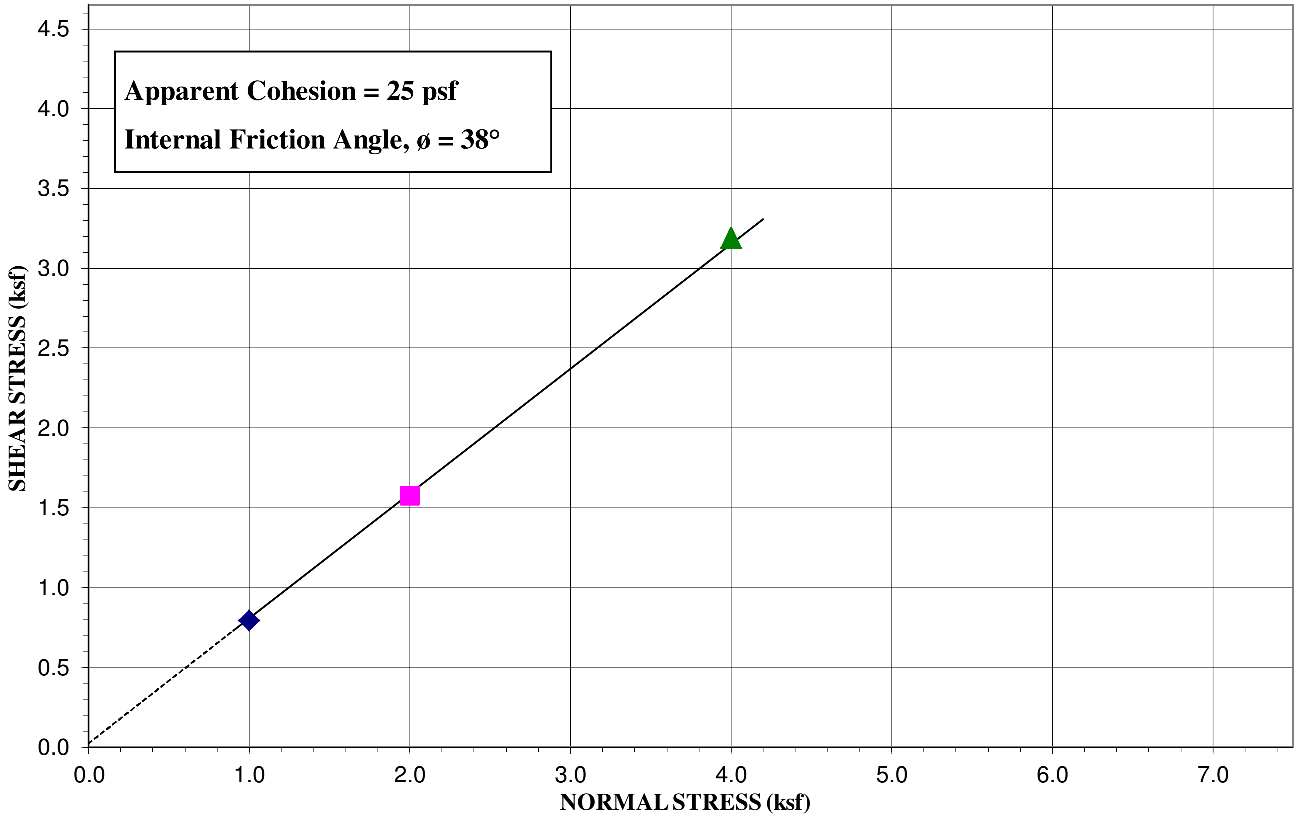
Testing Laboratory: AMEC, SLC, Utah



PROJECT: Lot 43 Summit at Ski Lake

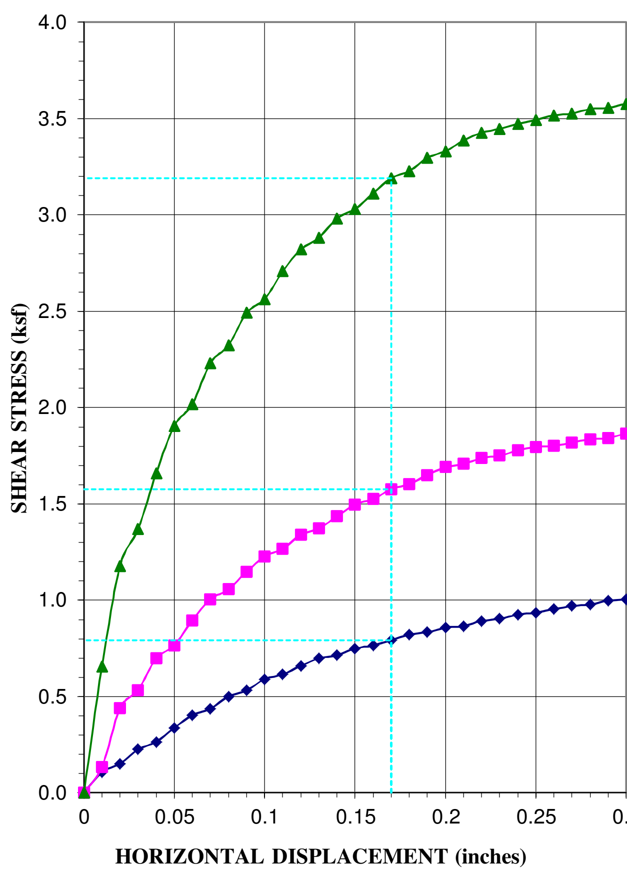
PROJECT NO.: 2063-01N-16	GSH	**FIGURE NO.:** 5

DIRECT SHEAR TEST



Source: TR-1	**Depth:** 3.5 ft			
Type of Test:	Consolidated - Drained			
Test No. (Symbol)	**1 (◆)**	**2 (■)**	**3 (▲)**	
Sample Type	Remolded			
Initial Height, in.	1.00	1.00	1.00	
Diameter, in.	2.45	2.45	2.45	
Dry Density Before, pcf	89.3	92.3	86.0	
Dry Density After, pcf	95.0	98.2	90.7	
Moisture % Before	10.7	13.2	18.1	
Moisture % After	23.1	22.1	25.2	
Saturation % Before	35.7	47.8	55.4	
Saturation % After	90.2	93.8	87.4	
Normal Load, ksf	1.0	2.0	4.0	
Shear Stress, ksf	0.79	1.58	3.19	
Strain Rate	0.005 in/min			
Sample Properties				
Cohesion, psf	25			
Friction Angle, ϕ	38			
Liquid Limit, %	---			
Plasticity Index, %	---			
Percent Gravel	---			
Percent Sand	---			
Percent Passing No. 200 sieve	---			
Classification	SM			

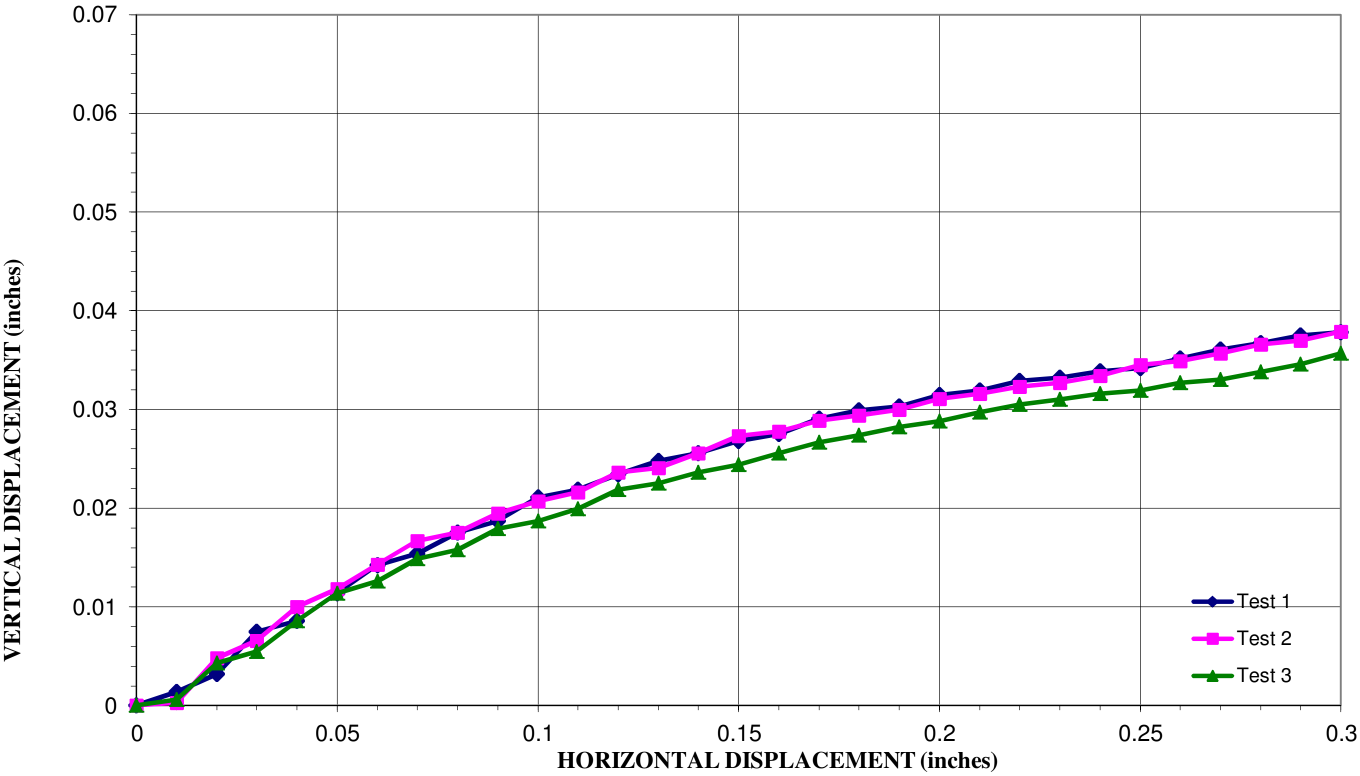
Testing Laboratory: GSH

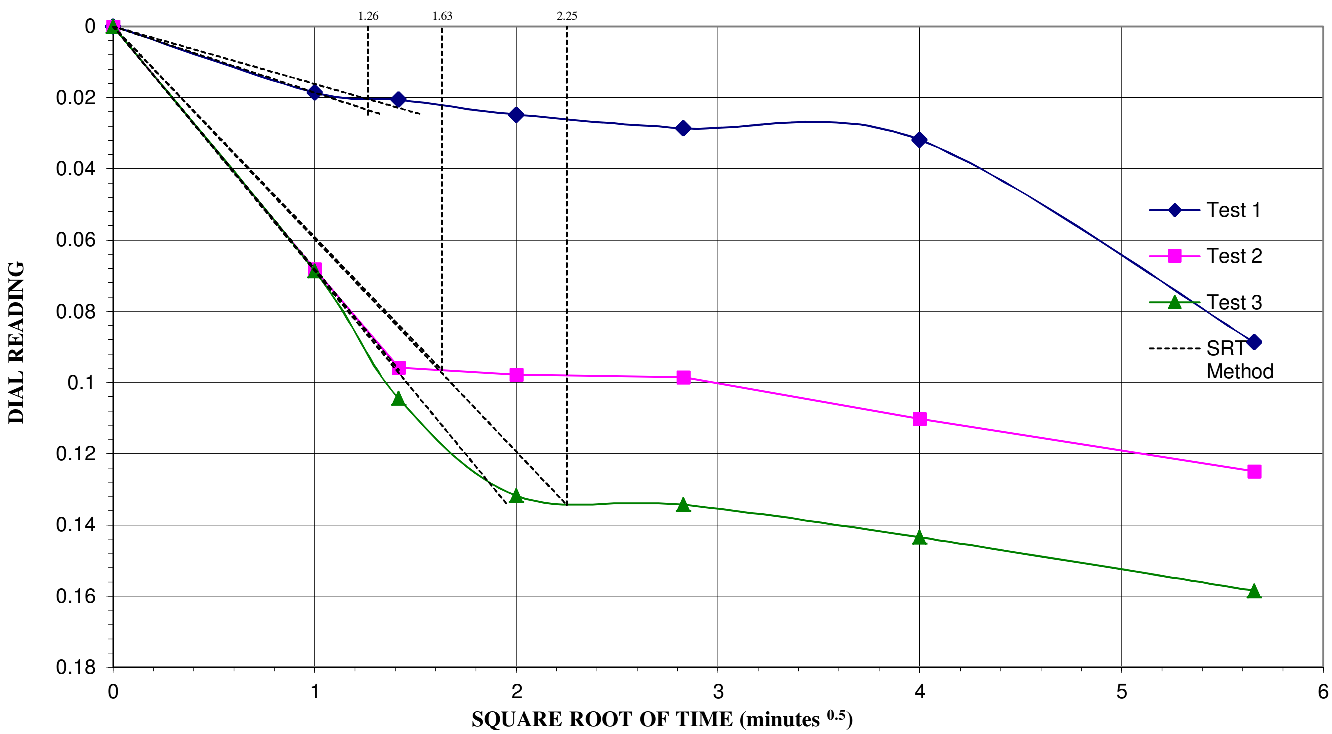


PROJECT: Lot 43 Summit at Ski Lake

PROJECT NO.: 2063-01N-16 | GSH | **FIGURE NO.:** 6A

DIRECT SHEAR TEST





Test No. (Symbol)	1 (◆)	2 (■)	3 (▲)
t_{90} (from sq-root time)	1.6	2.66	5.06
$t_f = 50\ t_{90} / 4.28$	18.7	31.1	59.1
max. $d_r = 0.5 / t_f$	0.0268	0.0161	0.0085
selected d_r (in./min)	0.0050	0.0050	0.0050

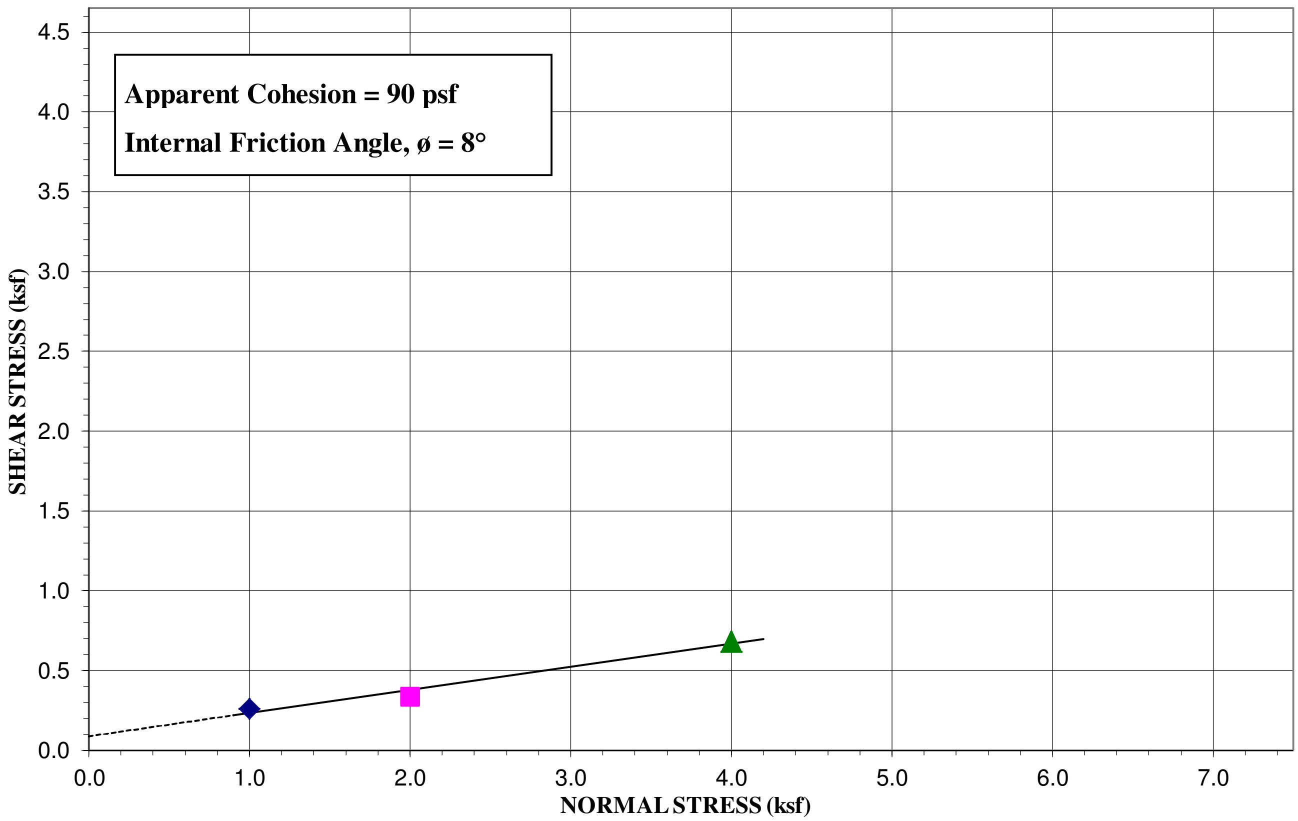
PROJECT: Lot 43 Summit at Ski Lake

PROJECT NO 2063-01N-16

GSH

FIGURE NO.: 6B

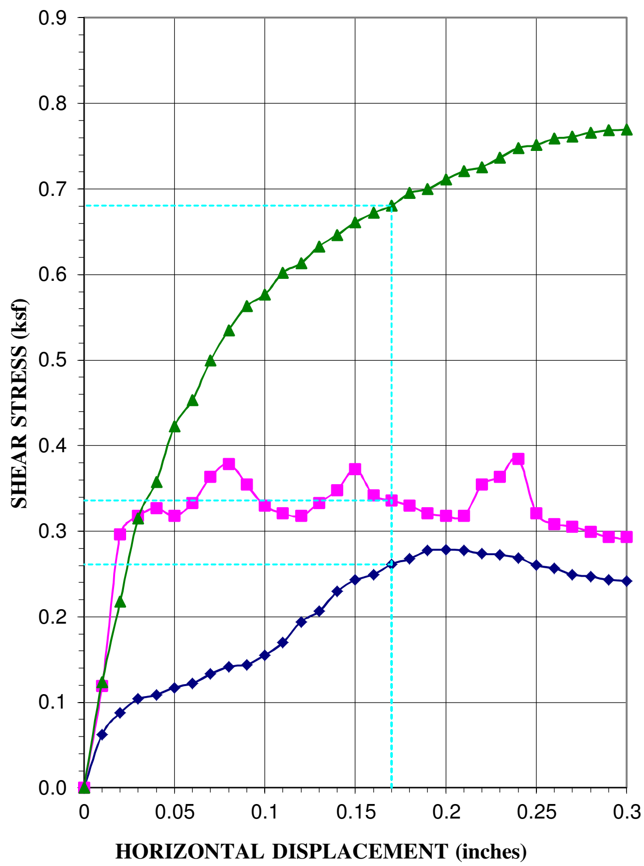
DIRECT SHEAR TEST



Source: TR-1		Depth: 2.5 ft	
Type of Test:	Consolidated - Drained		
Test No. (Symbol)	1 (◆)	2 (■)	3 (▲)
Sample Type	Thinwall		
Initial Height, in.	1.00	1.00	1.10
Diameter, in.	2.45	2.45	2.45
Dry Density Before, pcf	93.5	83.9	87.2
Dry Density After, pcf	93.5	84.5	90.6
Moisture % Before	18.5	28.6	17.2
Moisture % After	25.9	32.5	32.2
Saturation % Before	62.4	76.7	49.7
Saturation % After	87.0	88.3	101.2
Normal Load, ksf	1.0	2.0	4.0
Shear Stress, ksf	0.26	0.34	0.68
Strain Rate	0.005 in/min		

Sample Properties	
Cohesion, psf	90
Friction Angle, ϕ	8
Liquid Limit, %	---
Plasticity Index, %	---
Percent Gravel	---
Percent Sand	---
Percent Passing No. 200 sieve	---
Classification	CL

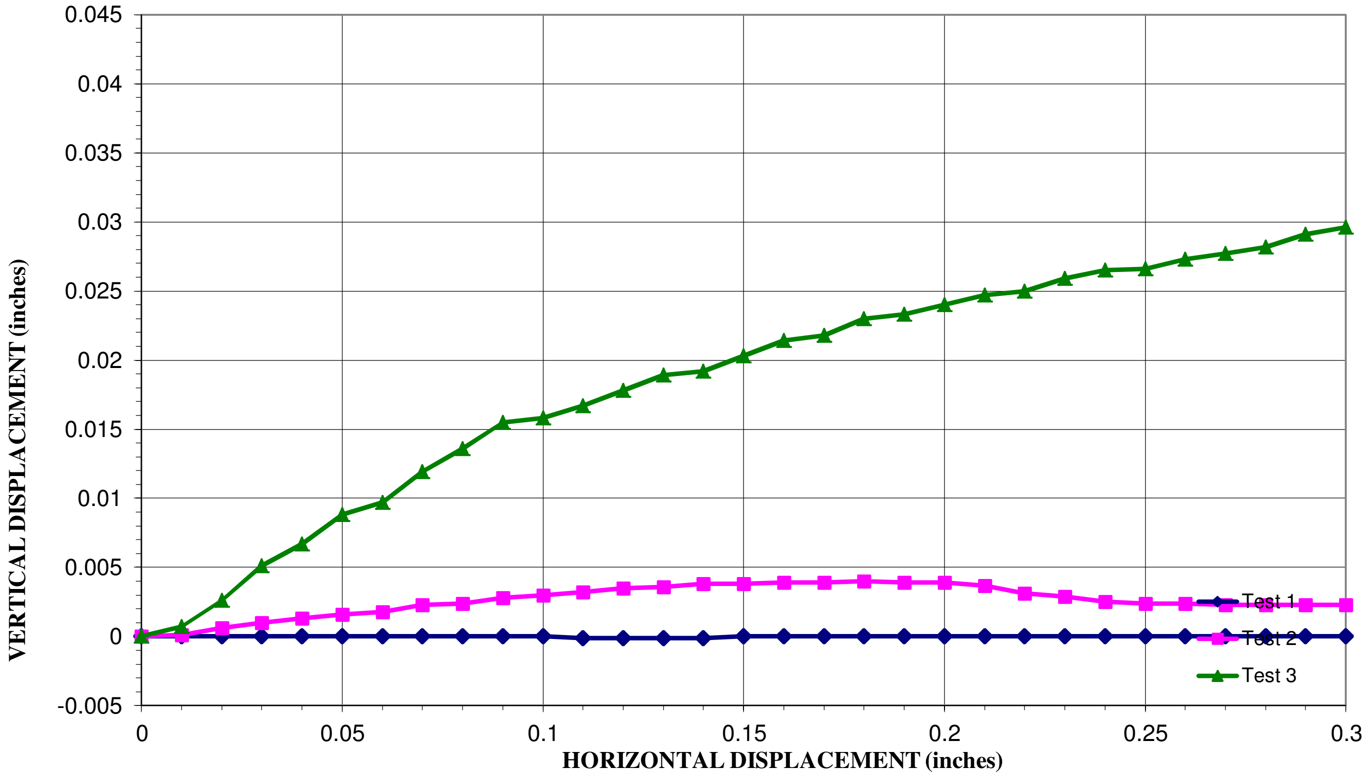
Testing Laboratory: GSH

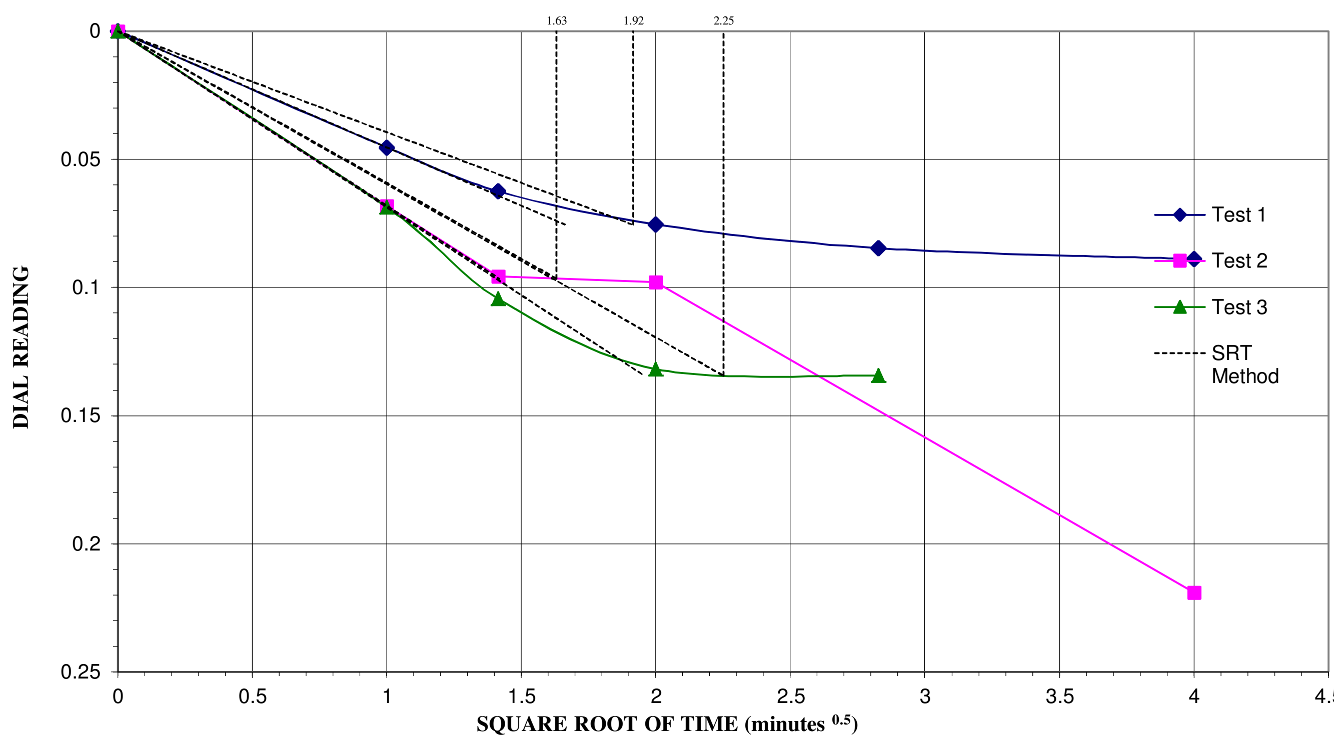


PROJECT: Lot 43 Summit at Ski Lake

PROJECT NO.: 2063-01N-16	GSH	**FIGURE NO.:** 7A

DIRECT SHEAR TEST





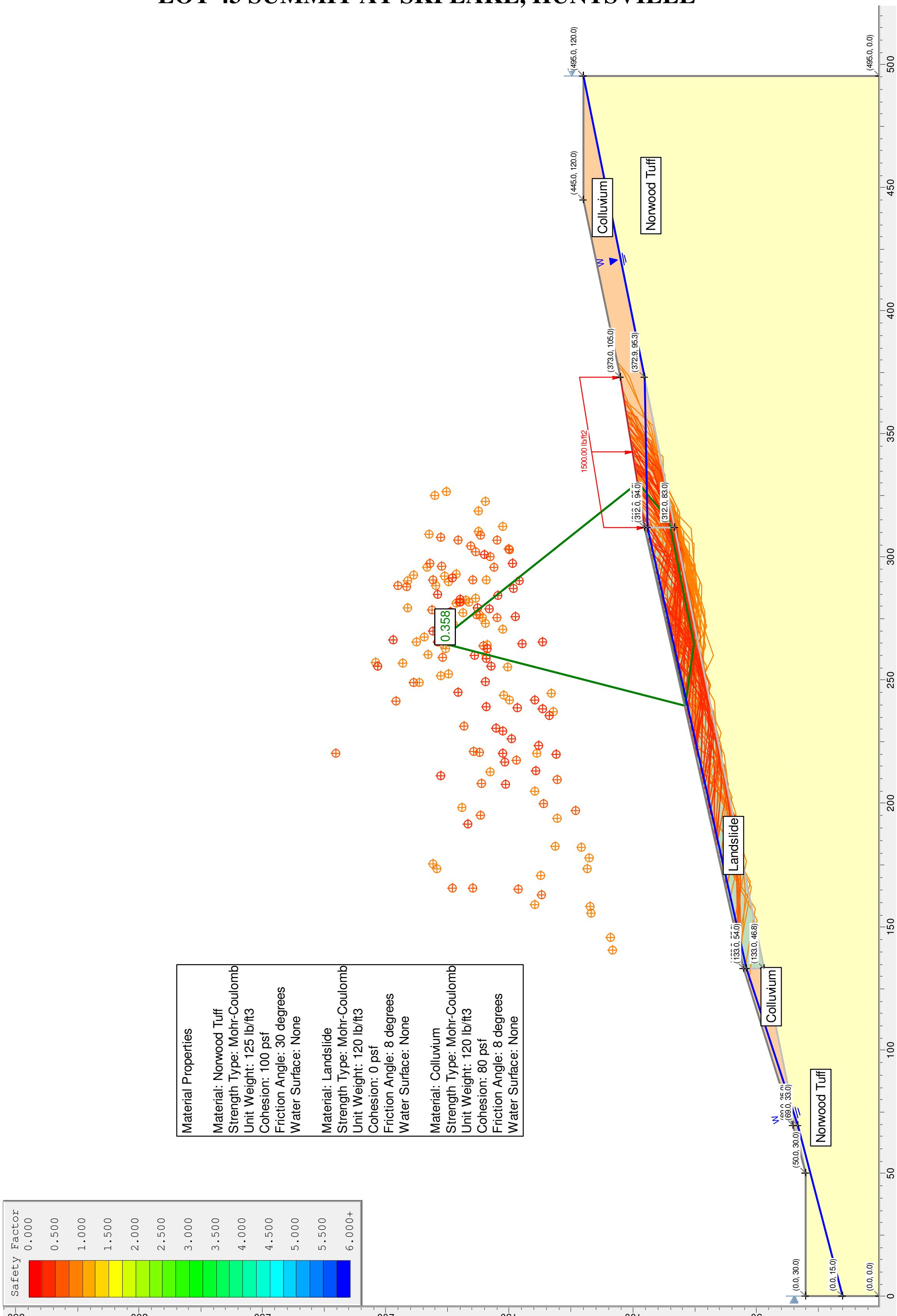
Test No. (Symbol)	1◆( )	2■( )	3 (▲)
t_{90} (from sq-root time)	3.67	2.66	5.06
$t_r = 50\ t_{90} / 4.28$	42.9	31.1	59.1
max. $d_r = 0.5 / t_f$	0.0117	0.0161	0.0085
selected d_r (in./min)	0.0050	0.0050	0.0050

PROJECT: Lot 43 Summit at Ski Lake

ROJECT NO 2063-01N-16 GSH **FIGURE NO.:** 7B

STABILITY RESULTS

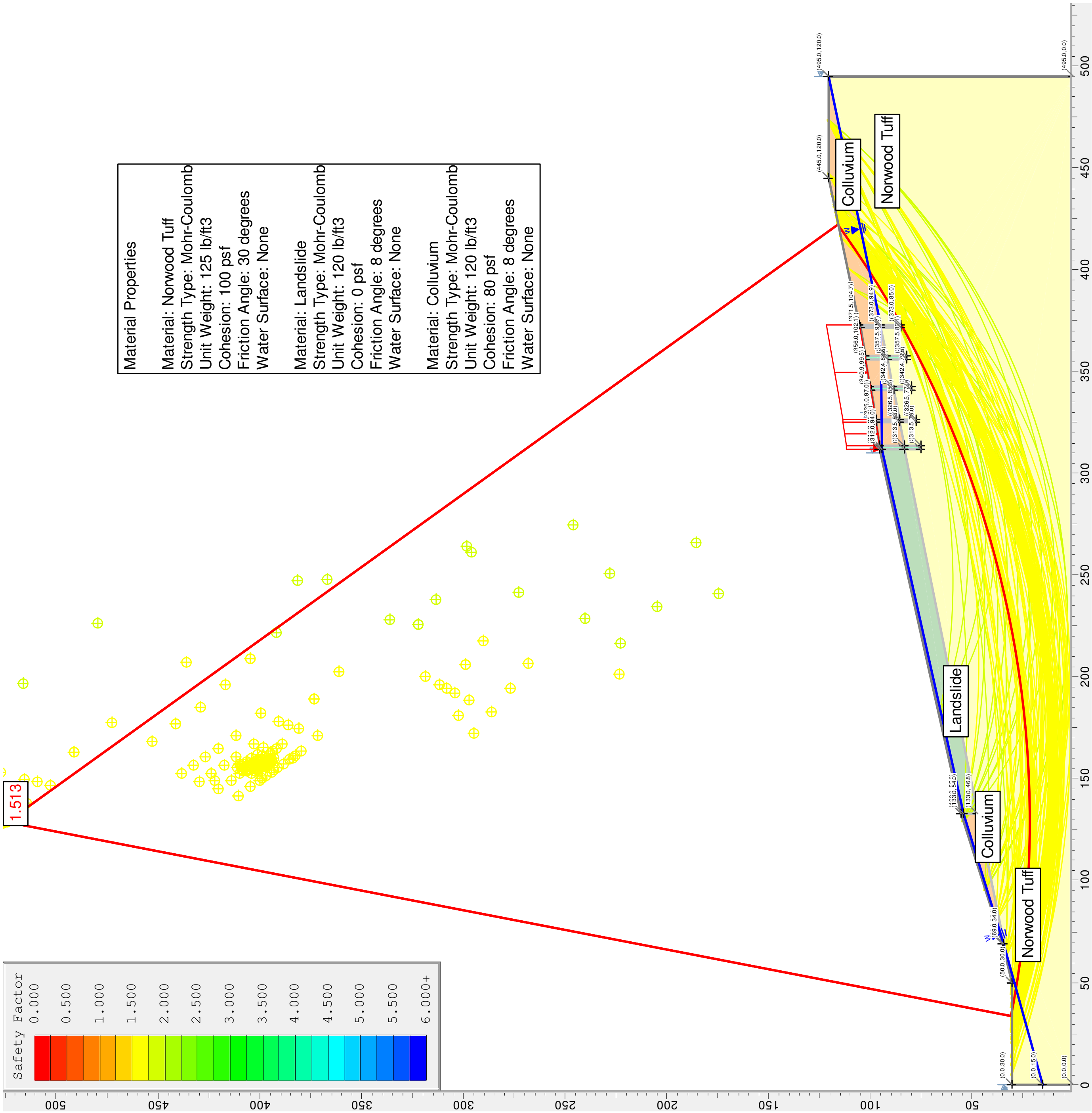
LOT 43 SUMMIT AT SKI LAKE, HUNTSVILLE



PROJECT NO.: 2063-01N-16	GSH	**FIGURE NO.:** 8

STABILITY RESULTS

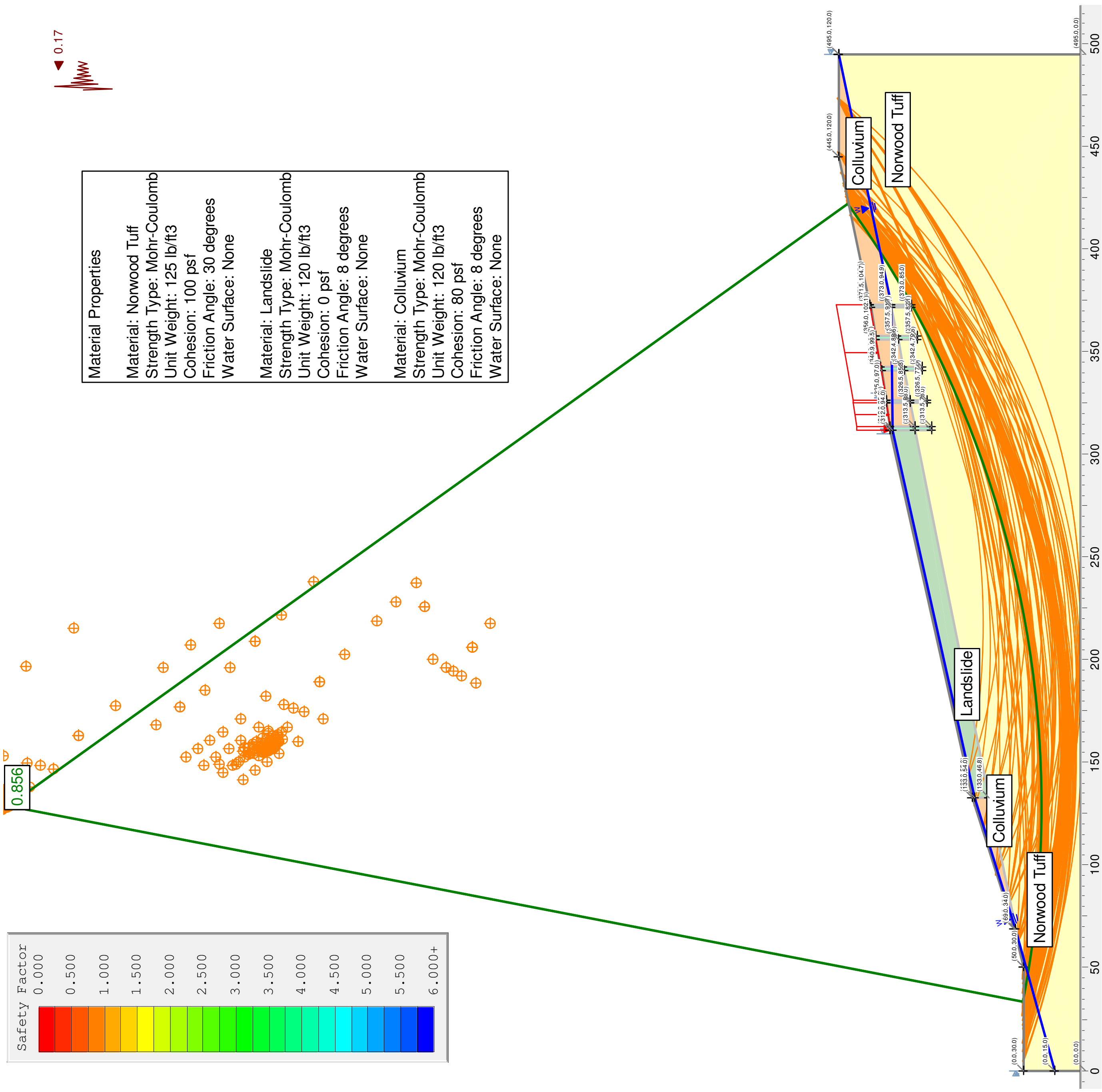
LOT 43 SUMMIT AT SKI LAKE, HUNTSVILLE



PROJECT NO.: 2063-01N-16	GSH	**FIGURE NO.:** 9

STABILITY RESULTS

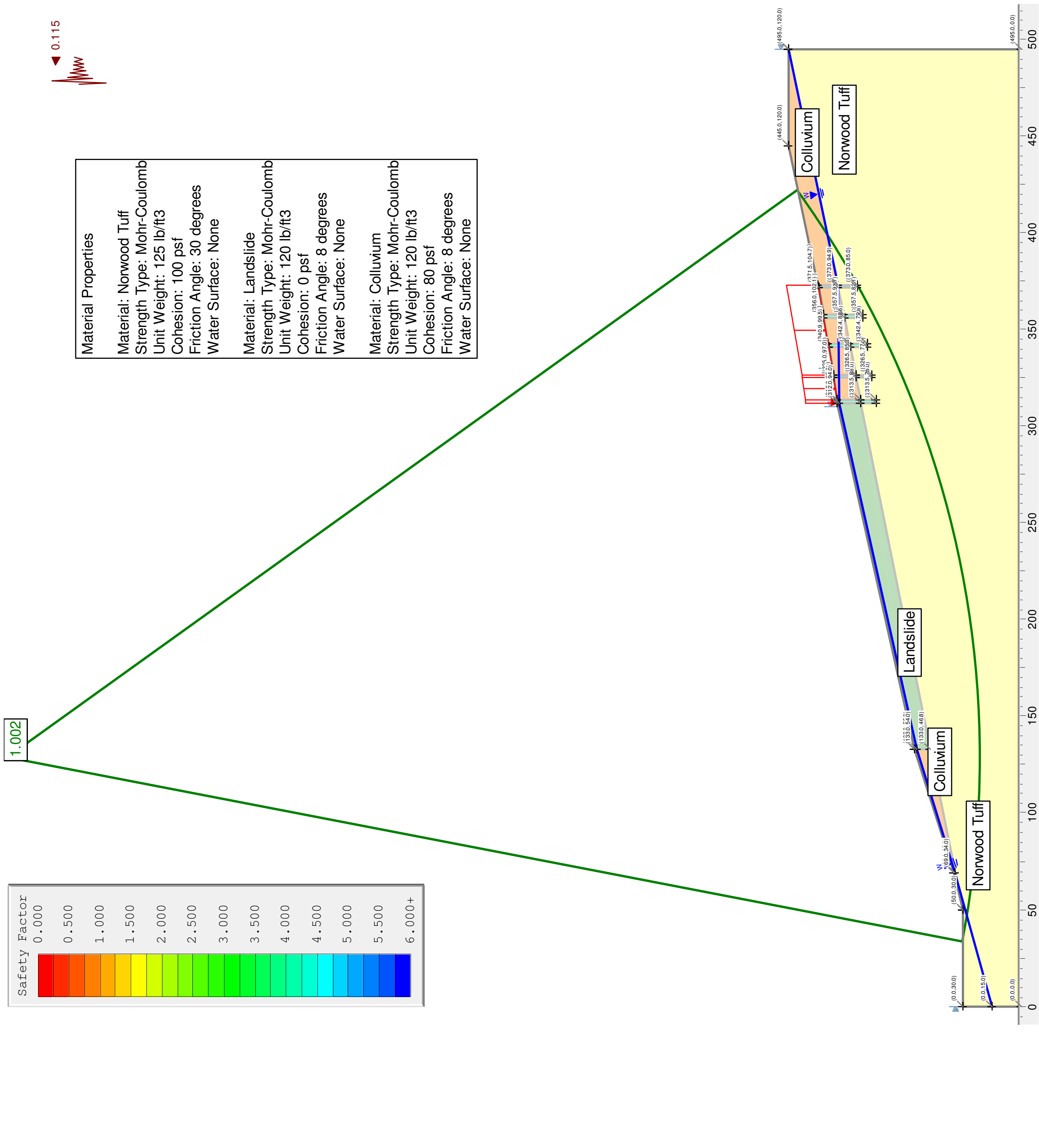
LOT 43 SUMMIT AT SKI LAKE, HUNTSVILLE



PROJECT NO.: 2063-01N-16	GSH	**FIGURE NO.:** 10

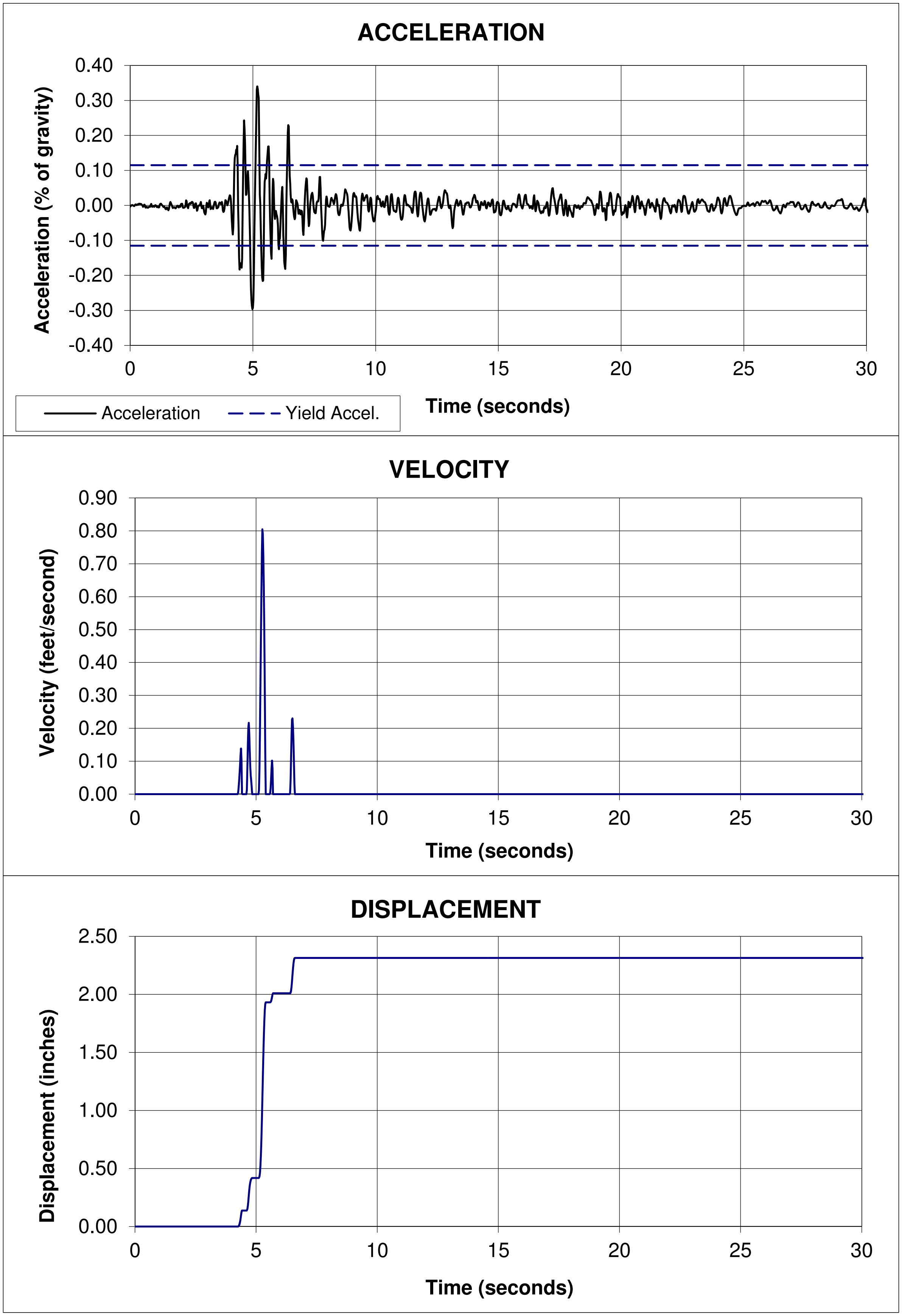
STABILITY RESULTS

LOT 43 SUMMIT AT SKI LAKE, HUNTSVILLE



PROJECT NO.: 2063-01N-16	GSH	**FIGURE NO.:** 11

NEWMARK DISPLACEMENT METHOD - ANALYSIS RESULTS



Reference Earthquake: Cape Mendocino 04/25/92 1806, Rio Dell Overpass FF, 360

APPENDIX

REPORT
GEOLOGICAL STUDY
LOT 43 THE SUMMIT AT SKI LAKE NO. 11
6785 EAST VIA CORTINA STREET
HUNTSVILLE, UTAH

Submitted To:

Rich Zollinger
2379 Sheffield Drive
Livermore, California

Submitted By:

GSH Geotechnical, Inc.
1596 West 2650 South
Ogden, Utah 84401

March 30, 2016

Job No. 2063-01N-16

March 30, 2016
Job No. 2063-01N-16

Mr. Rich Zollinger
2379 Sheffield Drive
Livermore, California 94550

Attn: Mr. Zollinger

RE: Report
Geological Study
Proposed Residence
Lot 43 The Summit at Ski Lake No. 11,
Weber County, Utah
(Parts of Section 24, Township 6 North, Range 1 East, Salt Lake base and meridian)

1. INTRODUCTION

In response to your request, GSH Geotechnical, Inc (GSH) has prepared this Geological Study for the proposed residential construction referenced above. The Summit at Ski Lake Phase 11 Subdivision is located in the vicinity of Huntsville Town, Weber County, Utah (41.2429, -111.7884). The general Ski Lake development area is located on the south side of Utah SR-39 between MP-16.6 and -17.4, and entirely within Section 24, T6N-R1E SLBM. The Summit at Ski Lake Phase 11 consists of four residential development lots roughly one-acre or greater in area, comprising a total area of approximately 5.7 acres as shown on Figure 1, Site Vicinity Map. Previous phases of the Summit at Ski Lake development are established to the north and generally down slope of the Phase 11 parcel. The Via Cortina access roadway loops around the lot 43 as shown on Figure 2, Site Plan, thus allowing frontage on either the north or south sides of the property. Elevation rises approximately 70 feet from the north side of the lot to the south side of the lot. The recorded address for Lot 43 is 6785 East Via Cortina, and it is listed as comprising 1.17 acres. Architectural drawings prepared by Creative Line LLC. (2015) show plans for an approximately 2,900 square-foot structural foot print for a residential structure on the south, upslope side, of the property as Shown on Figure 2. The Creative Line LLC drawings indicate the structure is to include a partial basement which lower level will daylight on the north, down-slope, side of the structure. The general area of the Phase 11 development includes slopes on the order of 20-pecent to 32-percent.

1.1 Weber County Natural Hazards Overlay Districts

Because the proposed residential is located on a sloping hill side area with susceptible expansive soil and rock conditions, Weber County (Planning Commission) has recorded lot 43 as Restricted (R), and requested that additional geotechnical and geological studies be conducted to evaluate conformance with development plans.

At this time specific guidelines for these studies have not been specified by the County, however Weber County Chapter 27 Natural Hazards Overlay Districts, Section 27-2B (Weber County Code, 2015), pertaining to Landslide/Tectonic Subsidence provides the following requirements:

> *... any development proposed within a designated landslide hazard area, as delineated on the Sensitive Lands Overlay District maps, shall require the submittal, review and approval by the planning commission, of specific site studies, including grading plans, cut/fill, and plans produced by a qualified engineering geologist and a Utah licensed geotechnical engineer. The site specific study shall address slope stability (including natural or proposed cut slopes), evaluate slope-failure potential, effects of development and recommendations for mitigative measures. Slope stability analysis shall include potential for movement under static, development-induced and earthquake-induced conditions as well as likely groundwater conditions.*

A review of site geological mapping prepared by Utah Geological Survey (UGS) geologists (King, et al, 2008), shown on Figure 3, Geologic Map, has indicated that parts of the Phase 11 subdivision is within mapped Quaternary landslide deposits (Qms and Qmc). Based upon our review of the mapping, the subject Lot 43 is mapped as being entirely underlain by Tertiary age Norwood Formation (Tn) rocks (King, et al., 2008).

To address the concerns and expectations of the Weber County Planning and Engineering Staff a scoping meeting was held on February 11, 2016 between the Lot 43 Summit at Ski Lake applicant proponents and Weber County Staff. Based upon our experience with Weber County the purpose of the scoping meeting was to accomplish the following:

> *Scoping Meeting: The developer or consultant should schedule a scoping meeting with the Weber County to evaluate the engineering geologist's/geotechnical engineer's investigative approach. At this meeting, the consultant should present a work plan that includes locations of anticipated geologic hazards and locations of proposed exploratory excavations, such as trenches, borings, CPT soundings, etc., which meet the minimum standard of practice. The investigation approach should allow for flexibility due to unexpected site conditions. Field findings may require modifications to the work plan*

1.2 Scoping Meeting and Revised Work Plan

The following individuals were present for the February 11 scoping meeting with Weber County Planning and Engineering Staff:

Ben Hatfield (Weber County Engineering)
Dana Schuler PE (Weber County Engineering)
David Simon PG, (Simon and Associates), Weber County Geological Consultant
Alan Taylor PE, (Taylor Geotechnical), Weber County Geotechnical Consultant
Greg Schlenker, PG, GSH Geotechnical Inc., Applicant Geological Consultant. (by teleconference)
Andrew Harris, PE, GSH Geotechnical Inc., Applicant Geotechnical Engineering Consultant

Chad Roberts, Applicant Building Contractor

During this meeting GSH consultants presented the following scope of work (work plan) for the evaluation of the Lot 43 Summit at Ski Lake site relevant to the Weber County Natural Hazards Overlay District Code:

GSH proposes to conduct Geotechnical/Geological Study to include; 1) Work Plan and scope of work development and plan implementation and meetings with Weber County Staff, 2) a search and review of previous relevant documentation of site engineering and geologic studies and including UGS mapping (King, et al, 2008), and reports and studies prepared by our staff and others (GSH Geotechnical Inc., 2015; Applied GeoTech, 2013; KPS and Associates, Inc., 2001); 3) a field reconnaissance study including the geologic/geotechnical logging and geotechnical sampling of a single walk-in test pit (trench) approximately 75 feet in length and as much as 14 feet in depth and the geotechnical logging and sampling of 3 pits to a depth of as much as 20 feet as shown on Figure 2, 4) site specific geological mapping and classification to identify critical geological units and exposure to proposed site improvements, 5) slope analysis from LiDAR DEM geoprocessing identifying critical areas 30-percent or greater across the site and/or surficial features potentially affecting the proposed site improvements, 6) A laboratory geotechnical soils testing program of samples recovered from the test pits and trenches for typical and critical geological units explored and identified in our subsurface evaluation. Laboratory testing program to include but not be limited to the moisture, density, gradation, Atterberg limits, consolidation, vane shear, and direct shear tests of representative soil samples, and 7) preparation of summary report presenting results of our analysis and findings including:

- *A vicinity map showing the location of the property relative to site vicinity and topographic features.*
- *A geologic map showing the site specific surficial geology of the property and surrounding area.*
- *Aerial photography showing the site and nearby surficial geologic features.*
- *Logs of test pits and trenches.*
- *An assessment of potential geologic hazards in the vicinity of the site and the exposure of the site and proposed site improvements to hazards named in the ordinance including but not limited to: landsliding and slope stability; alluvial fan processes including debris-flow; surface fault rupture hazards, strong earthquake ground motion, and liquefaction hazards; rockfall and avalanche hazards, and flood hazards.*
- *Cross-section of slope depicting encountered geological conditions.*
- *Site development recommendations based upon our findings and professional experience.*
- *Following completion of the geologic study, a geotechnical study will be prepared for the subject property based on the findings of the geologic study and concurrent/subsequent geotechnical evaluations.*

During the course of our field operations, areas of concern, including landslide movement, were identified in the trench, and the originally proposed trench (Trench 1) was extended to a length of 113 feet, and an additional 72 foot trench (Trench 2) was excavated on the east side of the site.

2. INVESTIGATIONS

2.1 Literature Review

During the Work Plan development, existing previous reports and geological literature sources were reviewed. Specific to the site and immediate surrounding area, reports and mapping by KPS and Associates, Inc., 2001; King, et al., 2008; Applied GeoTech, 2013; and GSH Geotechnical Inc., 2015, were reviewed. The KPS and Associates study involved a geotechnical evaluation and test pit excavations for a water tank constructed approximately 650 feet west of the Lot 43 site. The King, et al , 2008 document is an Open-file UGS geological mapping project of the Snow Basin and Huntsville, Utah quadrangles, which includes the location of the Lot 43 site. The 2013 Applied GeoTech study was a geotechnical evaluation conducted for surrounding Phases 12 and 13 of the Ski Lake development that included four test pit explorations. The 2015 GSH Geotechnical, Inc. study was a geological investigation conducted for the extension of the Via Cortina roadway, beginning approximately 200 feet west of the site. The GSH Study included the geological logging of approximately 700 feet of vertical cut exposure made for the roadway extension, and four "walk-in" test pits. The locations of the test pits and cut exposure walls investigated in the previous studies is included with areal geologic mapping of King et al., (2008) on Figure 4, Site Geology.

2.2 Field Program

GSH conducted field operations at the site on the dates February 18, 19 and 23, 2016. The field program involved the excavation and geological logging of two exploration trenches and four walk-in test pits. The excavations were logged to observe and characterize site subsurface/geologic and groundwater conditions for the site and the proposed residence construction. Trench 1 was located to evaluate the conditions beneath the proposed residence structure location, and the Test pits were located to observe conditions within the building lot, but away from the structure location. The locations of our trenches and test pits are included on Figure 4. Trench 1 was 113.0 feet in length and extended to depths of 6.0 to 11.0 feet, and Trench 2 extended 72.0 feet, and extended to depths of 6.0 to 13.5 feet. The test pits consisted of walk-in excavations, 30.0 to 35.0 feet in length and extending to depths of 8.0 to 11.5 feet. The trenches and test pits were logged so as to illustrate the vertical and lateral characteristics and variations of soil and rock conditions underlying the proposed residence and across the site. The trenches and test pits were excavated using a 20-ton class excavator with a 36-inch bucket. In addition to the observations in the trenches and test pits, the general surface of the site and surrounding area was reconnoitered to assess geological and slope conditions, and feature location and elevation data were recorded using a hand-held GPS receiver device.

Our field program was conducted by Dr. Greg Schlenker PG of our geotechnical staff and Mr. Amos Allard also of our geotechnical staff visited the site to assist Dr. Schlenker and to collect soil samples from the trenches test pits for laboratory geotechnical testing.

The soils and geology in the test pits and trenches were classified in the field based upon visual and textural examination, and interpretation of geologic site formation processes. These classifications have been supplemented by subsequent inspection and testing in our laboratory. Detailed graphical representations of the subsurface conditions encountered are presented on Figure 6, Log of Trench 1, and Figure 7, Log of Trench 2, the test pit logs are included on Figures 8 and 9, Log of Test Pits. The soil and rock units observed on the cut and in the test pits and trenches were classified in accordance with the Unified Soil Classification System (USCS), and were further classified on the basis of geological site formation processes. Photography and field logs of the trenches and test pits drawn during our field program are provided in the Appendices of this report.

Bulk and thin wall samples of representative soil layers encountered in the test pits and trenches were obtained and placed in sealable bags and/or were recovered undisturbed using driven sample tubes. The locations of the sample recovery locations are included on our trench and test pit logs. The results of our laboratory analysis and testing of the soils recovered from the test pits and trenches will be included in forthcoming geotechnical reports. Groundwater was not observed in any of the excavations or test pits during the dates of our field program.

Photographic documentation of the test pits and trenches are presented in the attached Appendices.

2.3 LiDAR - Slope Analysis

To asses slope conditions, interpret terrain, and develop site specific geologic cross section for the site, a LiDAR - Slope Analysis was performed for the site. Elevation data consisting of 2.0 meter LiDAR digital elevation data (DEM), for the site was obtained from Utah Automated Geographic Reference Center (AGRC). These data were geo-processed using the QGIS® GIS platform, and using the r.slope, r.shaded.relief and r.contour.level GRASS® (Geographic Resources Analysis Support System) modules, slope percentages, relief renderings and elevation contours for the site area were processed.

Figure 5, LiDAR-Slope Analysis, presents the results of our slope analysis efforts. Shown on Figure 5 is the 25-percent, and greater than 30-percent slope gradients across the site. The shaded relief rendering on Figure 5 provides a visual basis for landform interpretation, and the contour elevation data shown on Figure 5 was used to develop the cross section shown on Figure 10, Geologic Slope Cross Section. The critical gradient for slope development considerations according to the Weber County Section 108-14-3. (Weber County Code, 2015), includes slopes greater that 25-percent. The Geologic Slope Cross Section shown on Figure 10 will be used for slope stability analysis in our geotechnical reporting.

3. SITE 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 (Sorensen and Crittenden, 1979; Currey and Oviatt, 1985; Bryant, 1988; Coogan and King, 2001; and King et al., 2008) including a review of previous evaluations discussed previously in the Literature Review Section of this report, photogeologic analyses of 2014 and 2012 imagery shown on Figure 2 and Figure 4, and historical stereoscopic imagery flown in 1946. GIS analyses of elevation and geoprocessed DEM terrain data as discussed in the previous section (LiDAR-Slope Analysis) and shown on Figure 5, field reconnaissance of the general site area, and the interpretation of the trench and test pits excavated on the site as part of our field program. Seismic hazards information was developed from United States Geologic Survey (USGS) databases (Peterson, et al., 2008).

3.1 Geologic Setting

The site is located on the eastern flank of Mount Ogden which western flank comprises the Wasatch Front. The Wasatch Front is marked by the Wasatch fault, which is 7.0 miles west of the site, and provides the basis of division between the Middle Rocky Mountain Physiographic 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 surficial geology of the site vicinity is the result of the uplift and exposure of older pre-Cambrian rocks which forms the crest of Mount Ogden east of the site. This exposure was the result of movement along high-angle faults during late Tertiary and Quaternary age (Bryant, 1988).

Bounding the east foothill flank of Mount Ogden are mid Teritary units of the Norwood Formation that ramp along the base of the mountains south and west of the Ogden Valley floor. The Norwood Formation is described as "light-gray to light brown, altered tuff (claystone), tuffaceous siltstone, sandstone, and conglomerate" derived from volcanic ash deposition (King, et al., 2008), and has been measured to be as much as 7000 feet thick in the vicinity of the site. The claystone, siltstone and sandstone occurrences of the formation are primarily a result of lacustrine (lake processes) redeposition of the volcanic ash. The site location is largely underlain by Norwood Formation lacustrine rock units which beds appear to slope gently down to the northeast across the site (King et. al, 2008). Our previous investigation for the Via Cortina roadway extension immediately west of the site, revealed bedded exposures of lucustrine rock sequences generally consisting of moderate to thick bed units, (one to two feet in thickness) typically fining upward (sandstone-siltstone-claystone), colored light shades of buff, tan red and green and gray, and ranged from *weak* to *strong* in field test competency (GSH Geotechnical Inc., 2015). The existing surface of the site and vicinity appears to have been modified by Quaternary age erosion, and localized late-Quaternary stream, lacustrine (Currey and Oviatt, 1985), residual soil weathering and development, and mass movement processes (King, et al., 2008).

3.2 Surface Conditions

Although the site was covered with approximately one-and-one half feet of snow at the time of our field program, previous work in the vicinity of the site has provided an understanding of surface conditions without snow cover. As shown on Figure 2 and Figure 4, the site consists of an area of approximately one acre in size that is currently vacant and undeveloped. Surface vegetation consists of open areas of grasses, weeds and sage brush with clustered wooded areas of scrub oak, alder and maple tree cover. The topography of the site consists of a north facing hill slope with slopes on the property generally facing downward toward the north and northwest toward Ogden Valley.

Topographically the site is located on base foothills on the northeast side of Mount Ogden, and overlooks Ogden Valley and the South Fork of the Ogden River floodplain, which is inundated by Pineview Reservoir waters, to the north of the site. The site, as shown on Figure 2 and Figure 4 is bordered on the south, and east by vacant undeveloped lands, and on the north and west and by residential estate property land uses.

3.3 Subsurface Conditions

The natural rock and soils observed in the trenches and test pits and illustrated on Figure 6, Figure 7, Figure 8 and Figure 9, generally consisted, from bottom to top of:

1. Weathered Norwood Formation siltstone (ML-ST), weathering to clayey silt, slightly moist, very stiff to weak, light olive-buff and light yellowish brown color, showing massive to weak tabular structures, with observed thickness extending as much as 8.0 feet.
2. Weathered Norwood Formation sandstone (SM-SS,) weathering to fine to coarse sand with traces of silt, slightly moist to dry, very dense-hard, buff colored, massive structure, with observed thickness ranging from 2.5 to 5.5 feet (Observed only in Trench 1, Test Pit 1 and Test Pit 3).
3. Surficial pedogenic A-B soil vertisol sequences that extended in depth as much as 1.5 to 6.0 feet, consisting on the surface of <u>(ML)</u> Clayey silt, moist, medium stiff, dark brown, major herb roots to 6' inches, becoming with depth stiff, dark to reddish brown silty clay (CL), slightly moist, with deep vertical (vertisol) cracking. These soils are believed to be locally derived from weathered rock and colluvial sources.

Landslide movement was observed in Trench 1, Trench 2, Test Pit 2 and possibly Test Pit 3. The Landslide movement feature was observed beginning in Trench 1 at STA 62-feet north, and in Trench 2 at STA 28-feet north, with movement detected to be down slope in a northward direction. The landslide movement feature was observed to extend northward through Test Pit 2 and possibly through Test Pit 3, with a down slope axial length of approximately 150 feet and a width of approximately 195 feet. The observed thickness of the landslide feature was from 6.0 feet to 9.0 feet in Trench 1, 6.0 feet in Trench 2, 5.5 feet in Test Pit 2, and possibly 6.5 feet in Test Pit 3. The slide plane surface feature observed in the trenches displayed nominal deformation along the plane surface, with secondary "dark olive' clay accumulations observed on the failure surface in Trench 2, and oxidation accumulations observed in Trench 2 and Test Pit 2.

Groundwater was not observed in any of the trenches or test pits during our field program.

3.4 Site Engineering Geology

Our interpretation of the site engineering geology is presented on Figure 4, Figure 5 and Figure 10 of this report. The engineering geology shown on the three figures is largely based on previous mapping prepared by King, et al., (2008), with amendments to the mapping drawn on the basis of the findings of this and previous studies. A summary of the mapping units identified on the site vicinity are listed below in relative age sequence (youngest-top to oldest-bottom):

Qms-2016; Landslide and slump deposits (exposed during this study).
Qms-2015; Landslide and slump deposits (exposed during previous study).
Qmc; Landslide and slump, and colluvial deposits.
Qms; Landslide and slump deposits.
Tn; Norwood "Tuff" Formation.

In addition to the areal distribution of the geological deposits shown on Figures 4 and 5, a wave-cut shoreline attributed to the "Bonneville" highstand of ancient Lake Bonneville that was cut approximately 15,000 years ago (Currey and Oviatt, 1985), is shown to cross on the northwest and north sides of the site vicinity.

4. DISCUSSIONS AND RECOMMENDATIONS

4.1 Summary of Findings

4.1.1 Subsurface Observations: The geology exposed by trenches and test pits were generally found to consists of surficial, upper 1.0 to 1.5 feet of pedogenic soil A horizons, B horizon vertisol sequences that extended in depth (thickness) as much as 6.0 feet, and consisting of stiff silty clays derived from weathered rock and colluvial sources. At depth, weathered rock sequences consisting of sandstone and siltstone were observed extending to the depths penetrated by our test pits and trenches.

4.1.2 Expansive Soils. Vertical cracking associated with vertisol development was observed to extend from 1.0 to 6.0 feet below the surface in all of the trenches and test pits excavated for this study. The vertical cracking demonstrated by these soils is a result of naturally high expansive clay content within these soils (Graham and Southard, 1982). The presence or absence of the vertisol soils should be evaluated where structural loads are to be placed during future development.

4.1.3 Sloping Surfaces. The surface of site slopes developed from our LiDAR analysis range from level to over 55-percent as shown on Figure 5, LiDAR-Slope Analysis. For the Lot 43 site area the slope gradient averaged 23.5-percent, for the general vicinity of the Phase 11 parcel area the slope gradient averaged 24.5-percent. As previously discussed in the LiDAR-Slope Analysis section of this report, the critical gradient for slope development considerations according to the Weber County Code is 25-percent.

4.1.4 Site Engineering Geology And Mapping. The engineering geology mapping of the site presented on Figure 4 and Figure 5 reveals two issues pertinent to site development planning. These issues include: (1) **Landslide and slump deposits** (**Qms-2016)** - the presence of Landslide and slump deposits Qms-2106 deposits on the northeast side of the Lot 43 property; (2) **Norwood "Tuff" Formation (Tn)** - the presence of Norwood Tuff Formation **Tn** underlying much of the area comprising the development lot and Phase 11 parcel. These issues are addressed in order importance below:

> **1. Landslide and slump deposits:** Presence of mass-movement landslide and slump deposits (**Qms-2016,** this study) is based upon developed field observations including; deformation of soils and rock beds observed in Trench 1, Trench 2, Test Pit 2 and Test Pit 3, and location of the topographic features evident on the LiDAR imagery on Figure 5 indicating the planform area of movement observed in the trenches and test pits.
>
> Based on our observations, the area of movement, Qms-2016 shown on Figures 4 and 5 consists of a relatively shallow, approximately 9.0-feet in thickness, block of soil that appears to have moved or "creeped" downslope in response to inherent weak and expansive soil characteristics, and the moderately steep slope conditions in this area. Based upon our observations of evident topographic surface expression of this feature, we believe that this movement is presently active.
>
> **2. Norwood Tuff Formation (Tn):** The Norwood Formation has a notoriety of poor stability performance and geotechnically challenging soils throughout Northern Utah (Mulvey, 1992). Furthermore, we have observed an apparent genetic relationship with the occurrence of the Norwood Formation (and Norwood "Tuff") and surficial vertisol soils, which are subject seasonal shrink-swell processes (Graham and Southard, 1982). Based upon our past experience with areas underlain by Norwood Formation rock and soil, we believe that appropriate geological/geotechnical studies should be conducted before structural improvements are made in those areas.

4.1.5 Geoseismic Setting: Utah municipalities have adopted the International Building Code (IBC) 2012. The IBC 2012 code determines the seismic hazard for a site based upon 2008 mapping of bedrock accelerations prepared by the United States Geologic Survey (USGS) and the soil site class (Peterson, et al., 2008). The USGS values are presented on maps incorporated into the IBC code and are also available based on latitude and longitude coordinates (grid points).

Based on probabilistic estimates (Peterson, et al., 2008) queried for the site , the expected peak horizontal ground acceleration on rock from a large earthquake with a ten-percent probability of exceedance in 50 years is as high as 0.16*g,* and for a two-percent probability of exceedance in 50 years is as high as 0.33*g* for the site. Ground accelerations greater than these are possible but will have a lower probability of occurrence.

4.1.6 Active Earthqauke Faults: Based upon our review of available literature, no active faults are known to pass through or immediately adjacent to the site. The nearest active (Holocene) fault is the Weber Segment of the Wasatch fault, located 7.0 miles west of the site (Black et al., 2004). The Wasatch Fault Zone is considered capable of generating earthquakes as large as magnitude 7.3 (Arabasz, et al., 1992).

4.1.7 Liquefaction Potential Hazards: In conjunction with the ground shaking potential of large magnitude seismic events as discussed previously, 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 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 commonly occurs in saturated non-cohesive soils such as alluvium, thus no areas of the Phase 11 site appears to be susceptible to liquefaction processes.

4.1.8 Alluvial Fan Deposits: Alluvial fan deposits indicative of processes including flash flooding and debris flow hazard do not occur on the site: The nearest active alluvial fan deposits to the site, mapped as Qafy by king, et al., (2008), are located on a small fan surface (<4.0 acres in area) approximately 2,000 feet southwest of the site, and do not appear to represent a potential impact the site.

4.1.9 Flooding Hazards: No significant water ways pass in the vicinity of the site and flood insurance rate mapping by Federal Emergency Management Agency for the site vicinity has not been prepared at this time.

4.1.10 Rockfall and Avalanche Hazards: The site is over two miles from steep slope areas where such hazards may originate.

4.1.10 Radon Exposure: Radon is a naturally occurring radioactive gas that has no smell, taste, or color, and comes from the natural decay of uranium that is found in nearly all rock and soil.

Radon and has been found occur in the Ogden Valley area, and can be a hazard in buildings because the gas collects in enclosed spaces. Indoor testing following construction to detect and determine radon hazard exposure should be conducted to determine if radon reduction measures are necessary for new construction. The radon-hazard potential for the lot 43 location is mapped as "Moderate" by the UGS (Solomon, 1996).

4.2 Conclusions

Based upon our geological studies herein, we believe that the Lot 43 Summit at Ski Lake is suitable for development, provided that soils identified in our trenches and test pits as subject to past landslide movement as discussed in Section 3.3 of this report, are avoided or mitigated. Further study of the slope stability of the site is required as part of the geotechnical engineering study. A cross-section of the site slopes and geology is provided on Figure 10 and must be incorporated into the slope stability analyses by the geotechnical engineer. Although plans are at this time not finalized, we understand that deep foundation systems are being considered as a methodology to reduce exposure to the landslide soil movement observed on the site.

The site has been shown to be underlain by Norwood Formation deposits, and expansive vertisol soils were observed in all of the excavations made for this study. Areas where these soils are present should be evaluated prior to the placement of structural loads. Further study of the expansive potential of the near surface soils is required as part of the lot specific geotechnical study.

Due to the “moderate” radon potential for the site, radon testing of the home following construction is recommended.

Test pits and trenches were excavated in the proposed home area. The backfill soils for these explorations is likely unsuitable for bearing structures. The trench/test pit backfill soils within the structure must be removed and replaced with compacted structural meeting the requirements of the lot specific geotechnical study.

Due to the potential for landslide deposits at the site, observation of the home excavation during construction is required.

CLOSURE

If you have any questions or would like to discuss the results of this study further, please feel free to contact us at (801) 393 2012.

Respectfully submitted,

GSH Geotechnical, Inc.

LICENSED PROFESSIONAL GEOLOGIST
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GS/AMH:mmh

Encl.	Figure	1,	Vicinity Map
	Figure	2,	Site Plan
	Figures	3,	Geologic Map
	Figure	4,	Site Geology
	Figure	5,	LiDAR-Slope Analysis
	Figure	6,	Log of Trench 1
	Figure	7,	Log of Trench 2
	Figure	8,	Log of Test Pit 1 and Test Pit 2
	Figure	9,	Log of Test Pit 3 and Test Pit 4
	Figure	10,	Geologic Cross Section A-A'

Appendix A-1, Field Trench Logs
Appendix A-2, Field Test Pit Logs
Appendix B-1 to B-6 Trench and Test Pit Photographs.

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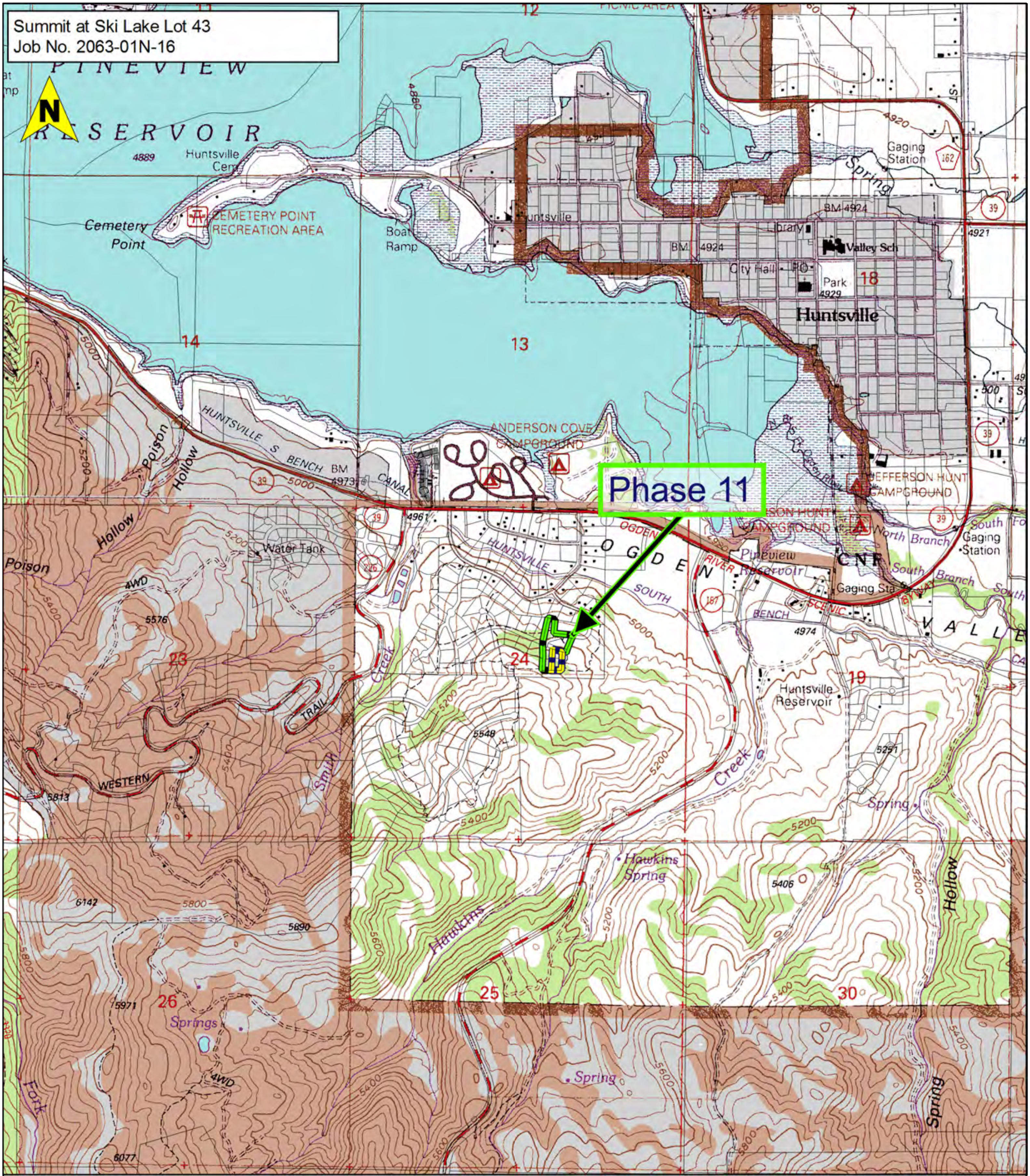


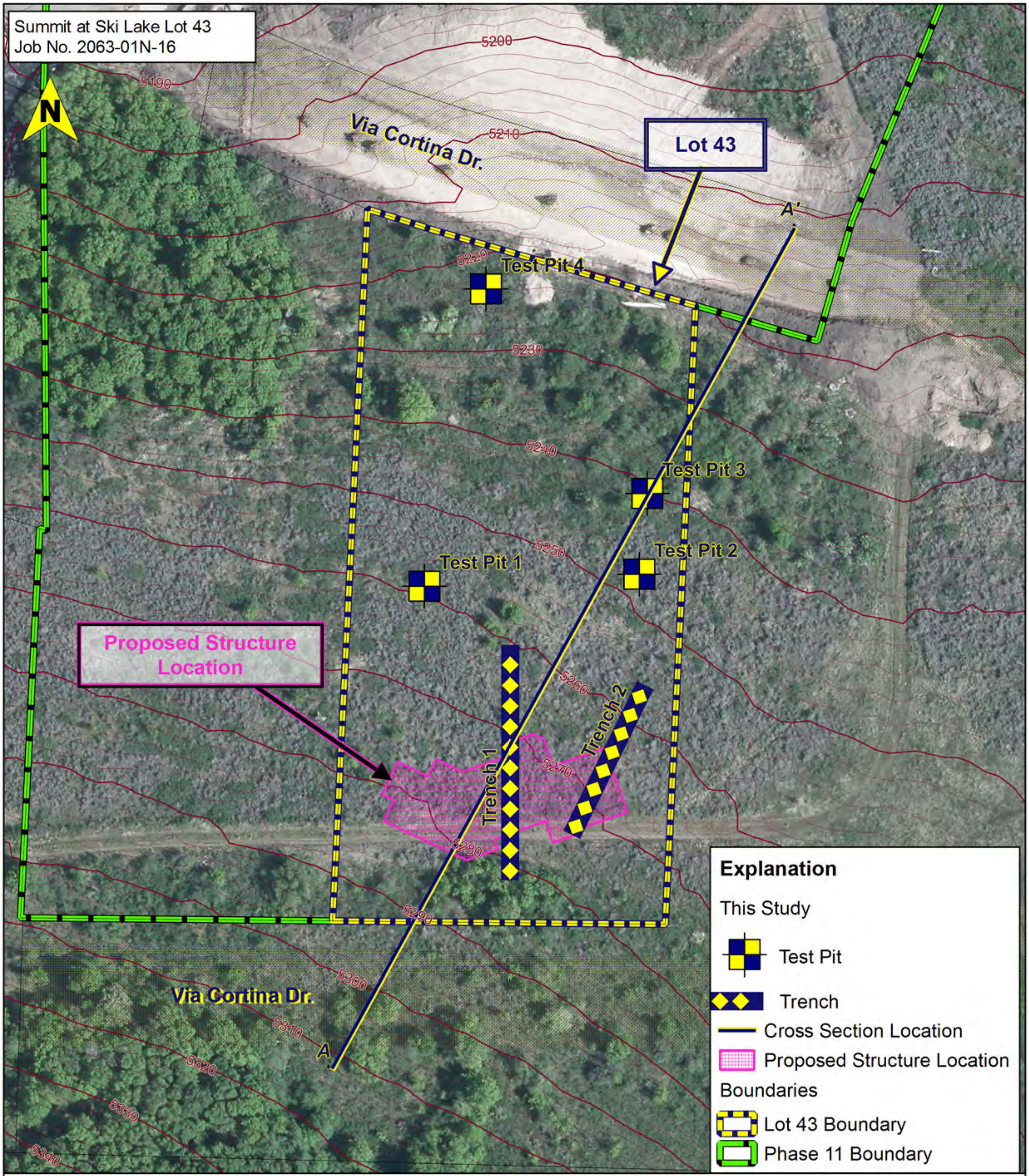
Base:
1998 7.5 Minute USGS Toporgraphic Maps Titled
Snowbasin, Utah, and Huntsville, Utah.

0 1000 2000 3000 4000 ft

1:24,000

FIGURE 1
VICINITY MAP

GSH

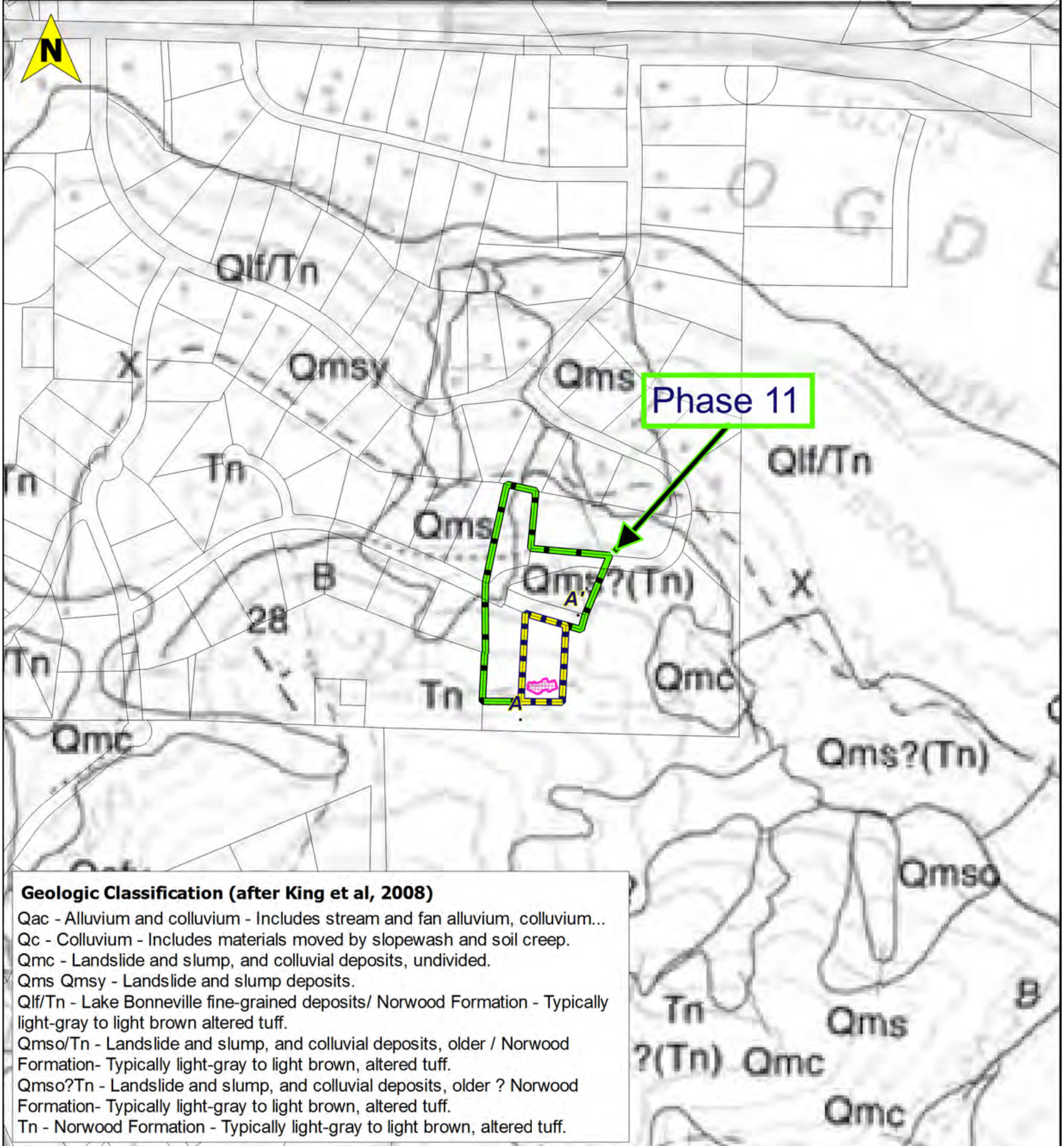


Base:
2012 0.5ft Color HRO Orthoimagery,
from Utah AGRC. http://gis.utah.gov/

FIGURE 2
SITE PLAN

GSH

Summit at Ski Lake Lot 43
Job No. 2063-01N-16



Geologic Classification (after King et al, 2008)

Qac - Alluvium and colluvium - Includes stream and fan alluvium, colluvium...
Qc - Colluvium - Includes materials moved by slopewash and soil creep.
Qmc - Landslide and slump, and colluvial deposits, undivided.
Qms Qmsy - Landslide and slump deposits.
Qlf/Tn - Lake Bonneville fine-grained deposits/ Norwood Formation - Typically light-gray to light brown altered tuff.
Qmso/Tn - Landslide and slump, and colluvial deposits, older / Norwood Formation- Typically light-gray to light brown, altered tuff.
Qmso?Tn - Landslide and slump, and colluvial deposits, older ? Norwood Formation- Typically light-gray to light brown, altered tuff.
Tn - Norwood Formation - Typically light-gray to light brown, altered tuff.

Base:
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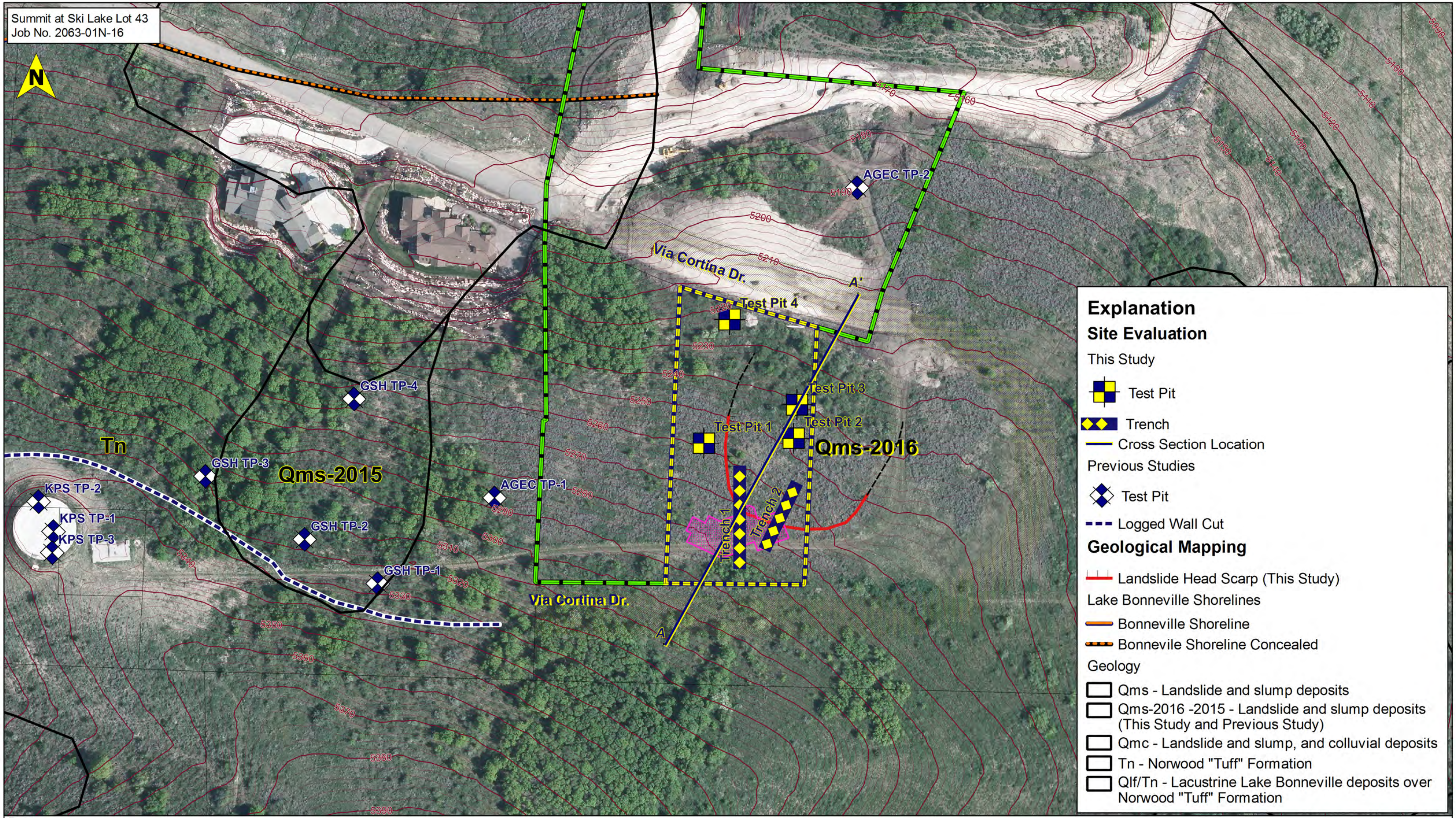
0 500 1000 ft

1:6,000

FIGURE 3
GEOLOGIC MAP

GSH



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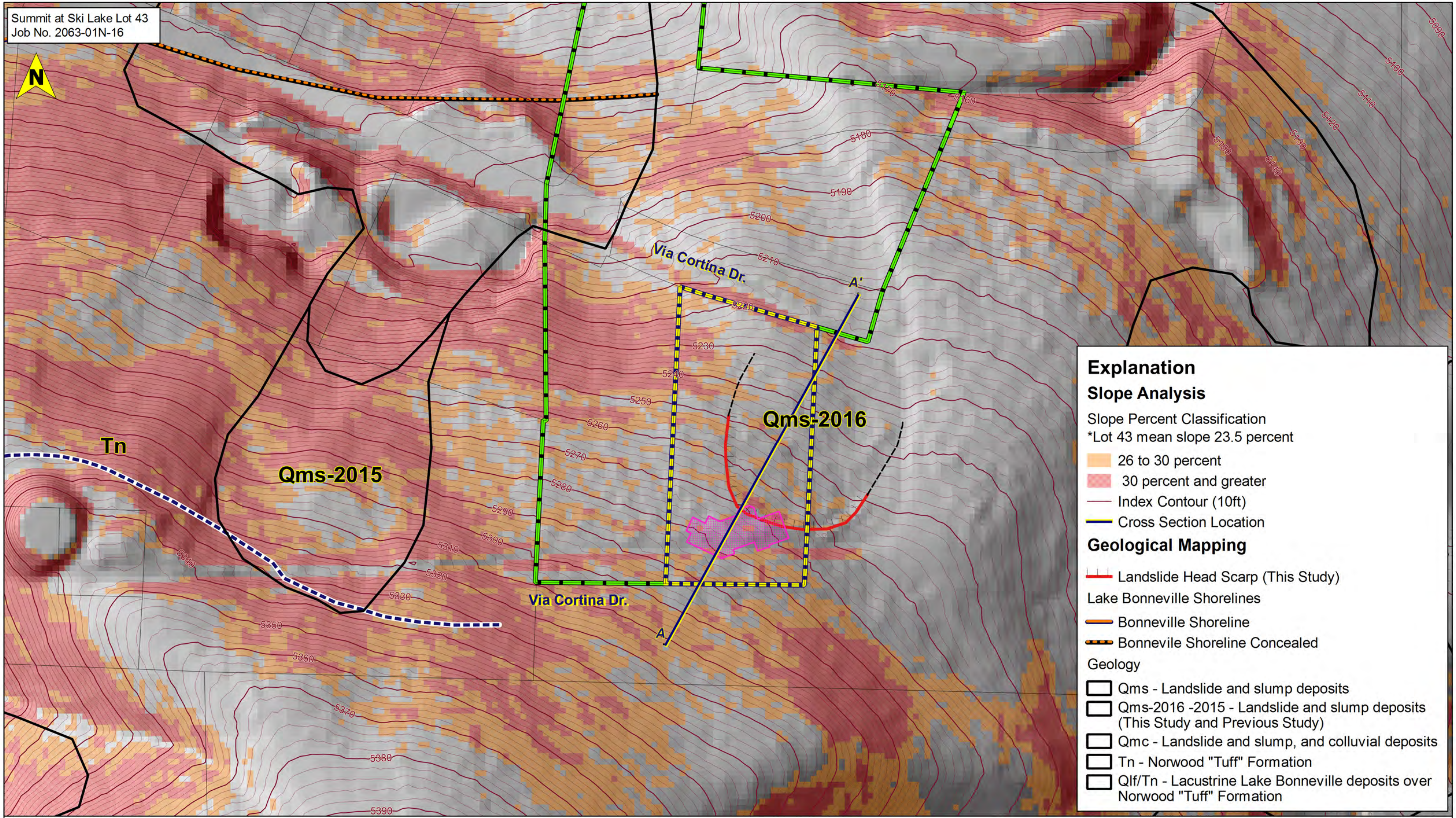
2012 0.5ft Color HRO Orthoimagery,
from Utah AGRC. http://gis.utah.gov/
Geology:
King, J.K., Yonkee, W.A., and Coogan, J.C., 2008, Interim geologic map of the Snow Basin and part of the Huntsville quadrangle, Davis, Morgan, and Weber Counties, Utah: Utah Geological Survey Open-File Report 536, scale 1:24,000.

0 100 200 300 400 ft

1:1,200

FIGURE 4
SITE GEOLOGY

GSH



Base Coverage-Elevation:
2006 2.0m Geoprocessed LiDAR from
Utah AGRC; http://gis.utah.gov/
Geology:
King, J.K., Yonkee, W.A., and 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, scale 1:24,000.

0 100 200 300 400 ft

1:1,200

FIGURE 5

LiDAR-SLOPE ANALYSIS

GSH

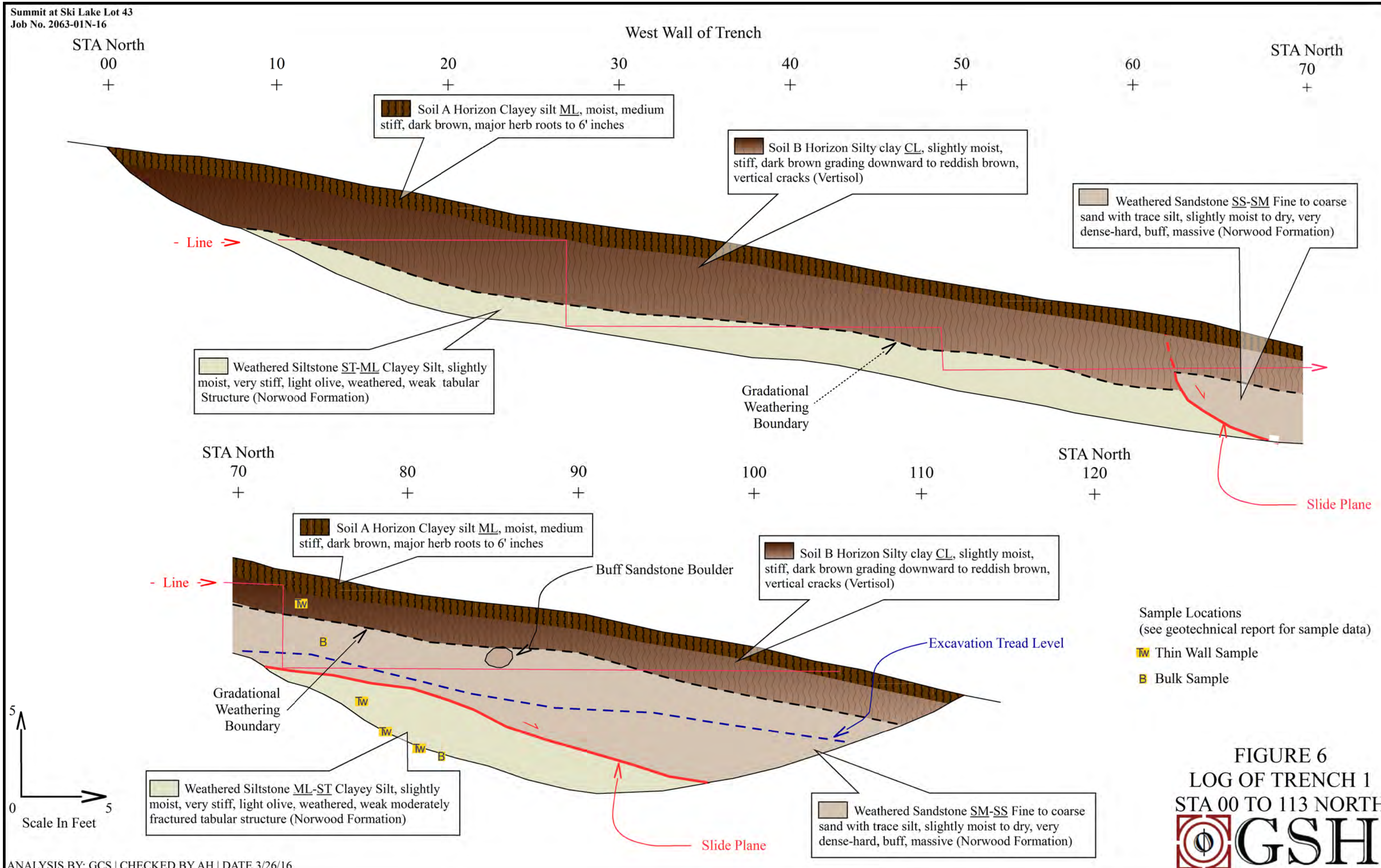


FIGURE 6
LOG OF TRENCH 1
STA 00 TO 113 NORTH

Summit at Ski Lake Lot 43
Job No. 2063-01N-16

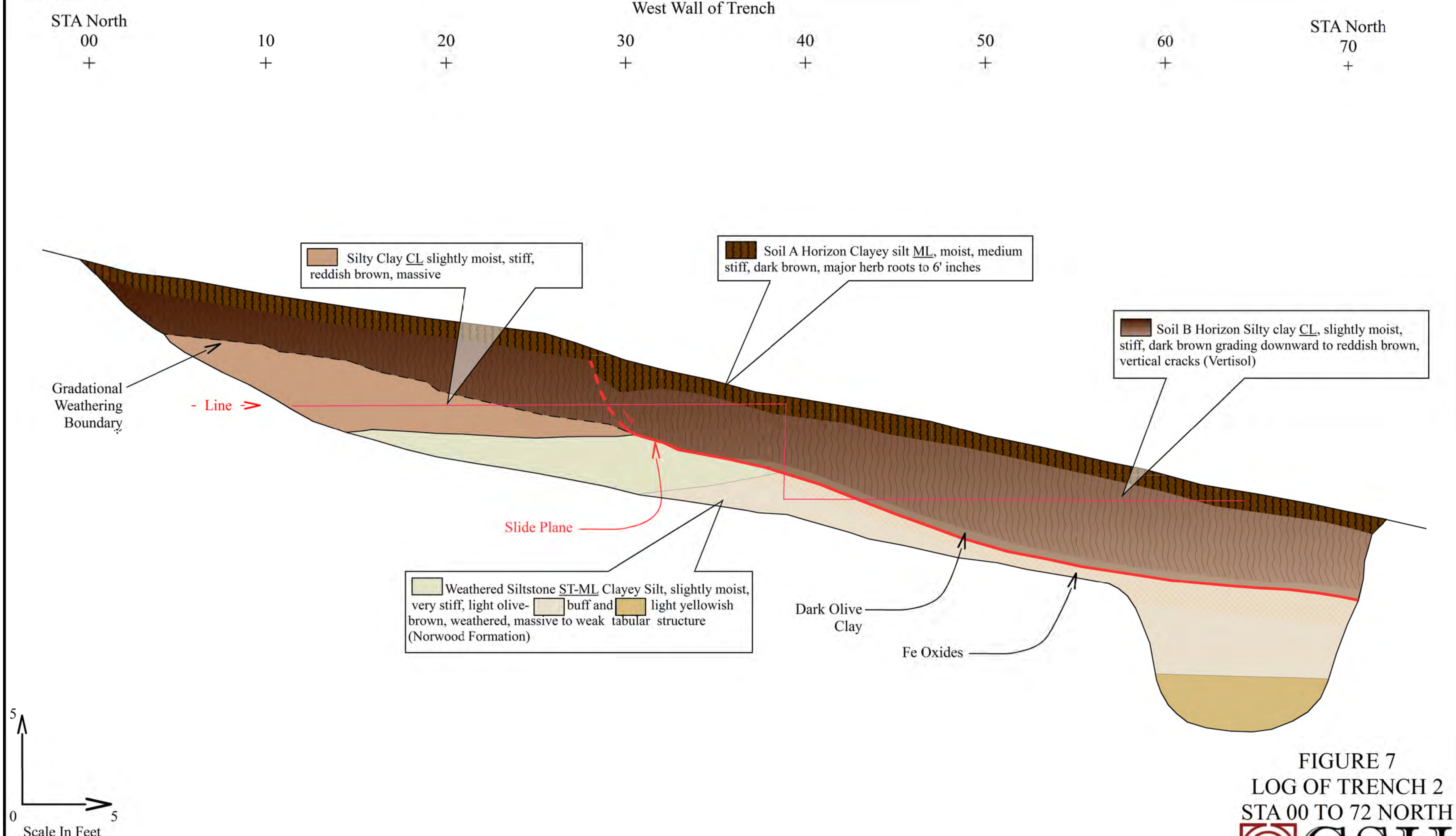


FIGURE 7
LOG OF TRENCH 2
STA 00 TO 72 NORTH

GSH

ANALYSIS BY: GCS | CHECKED BY AH | DATE 3/26/16

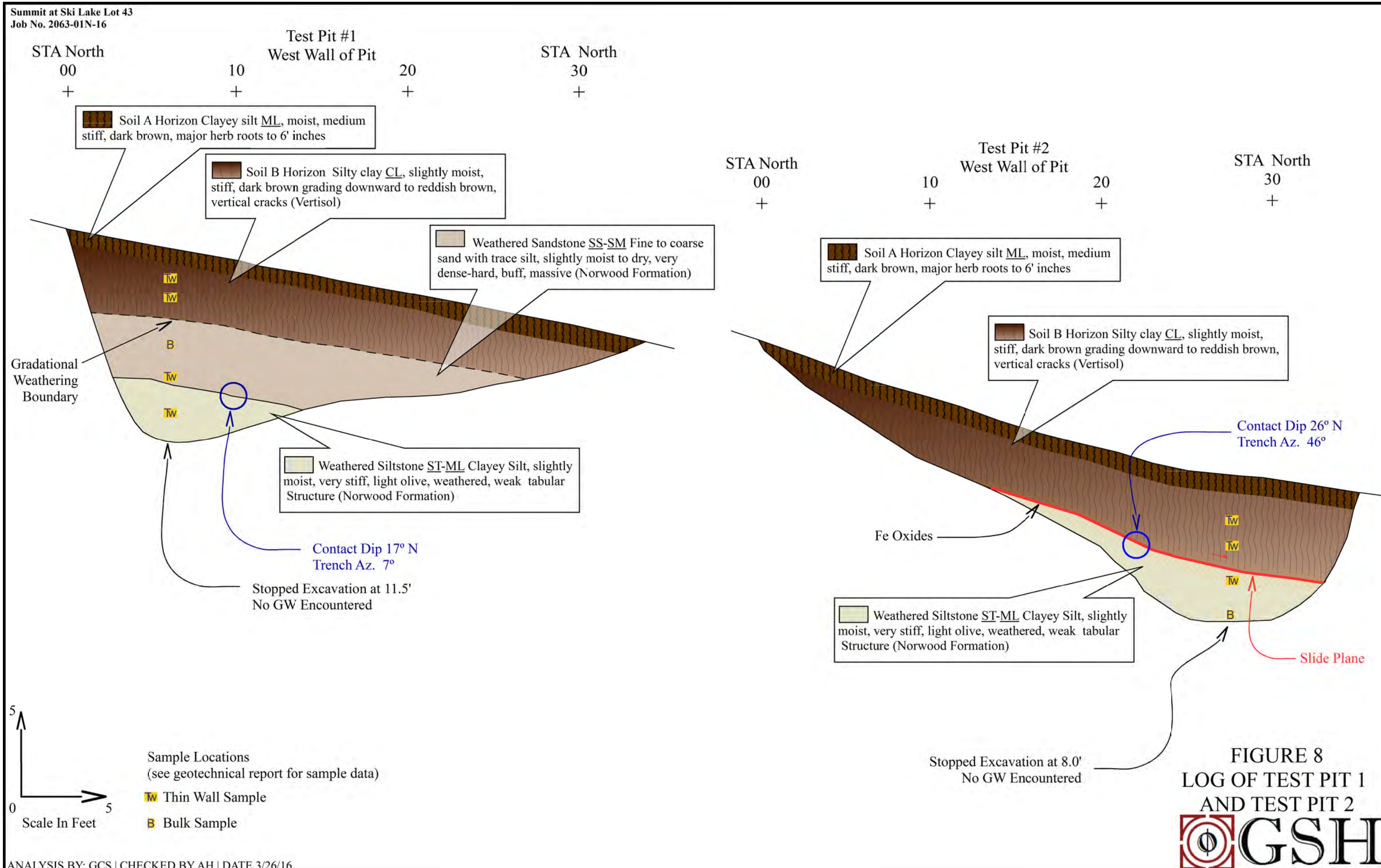


FIGURE 8
LOG OF TEST PIT 1
AND TEST PIT 2

Summit at Ski Lake Lot 43
Job No. 2063-01N-16

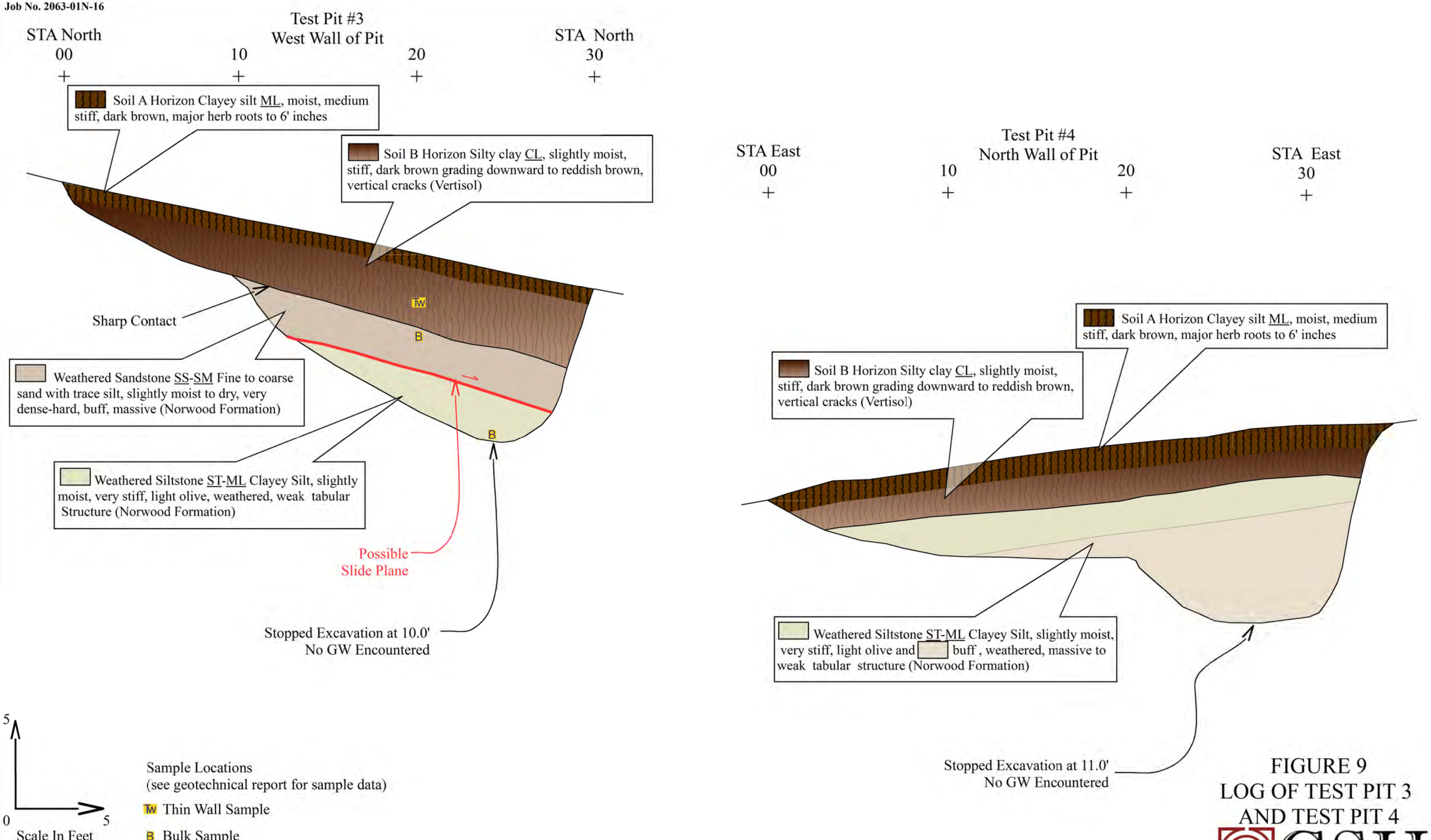
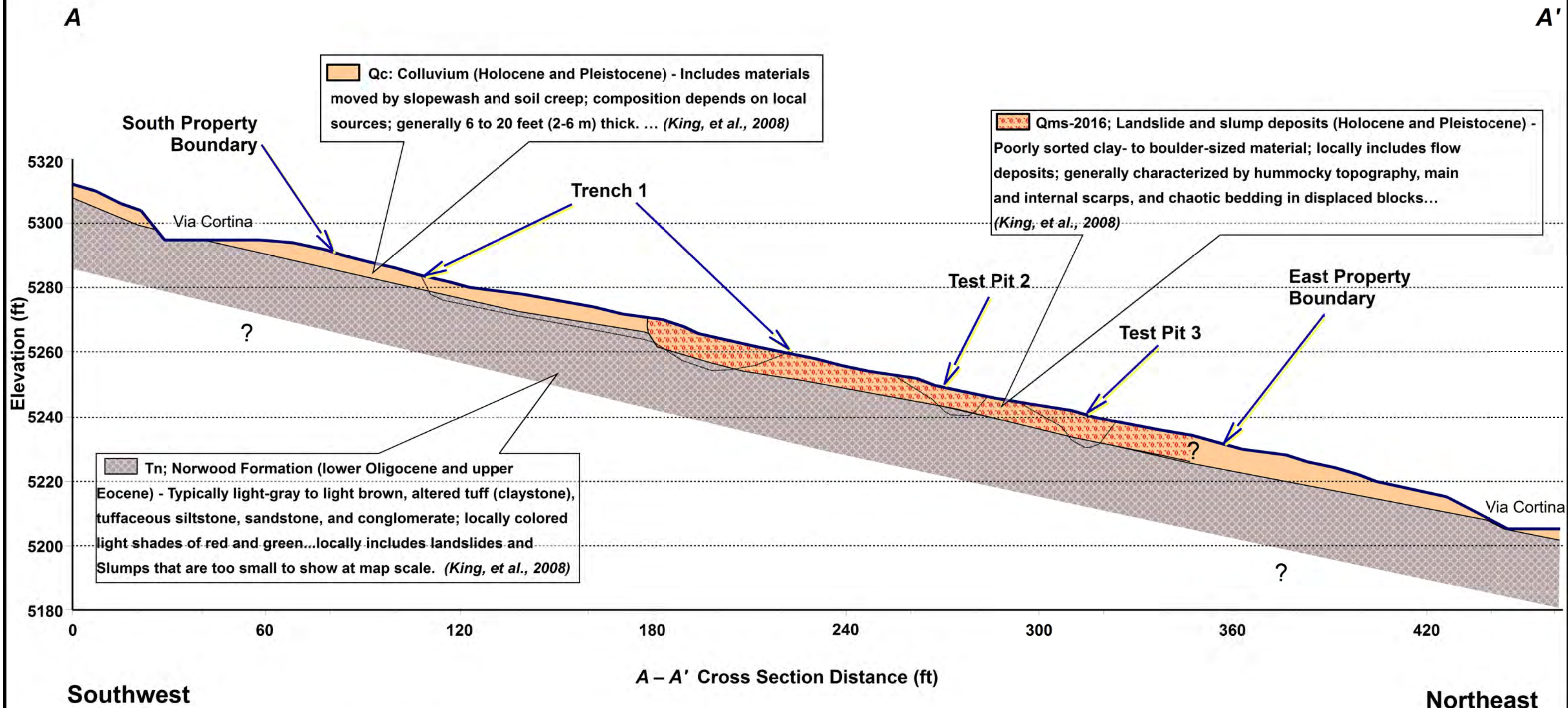


FIGURE 9
LOG OF TEST PIT 3
AND TEST PIT 4

GSH

ANALYSIS BY: GCS | CHECKED BY AH | DATE 3/26/16

Lot 43 Summit at Ski Lake
Job No. 2063-01N-16



*Elevation data for the cross section were developed from 2.0 meter 2006 LiDAR digital elevation data. Elevations for the Via Cortina cut slopes were recorded using a hand-held GPS receiver device during our field program.

FIGURE 10
GEOLOGIC SLOPE
CROSS SECTION A-A'

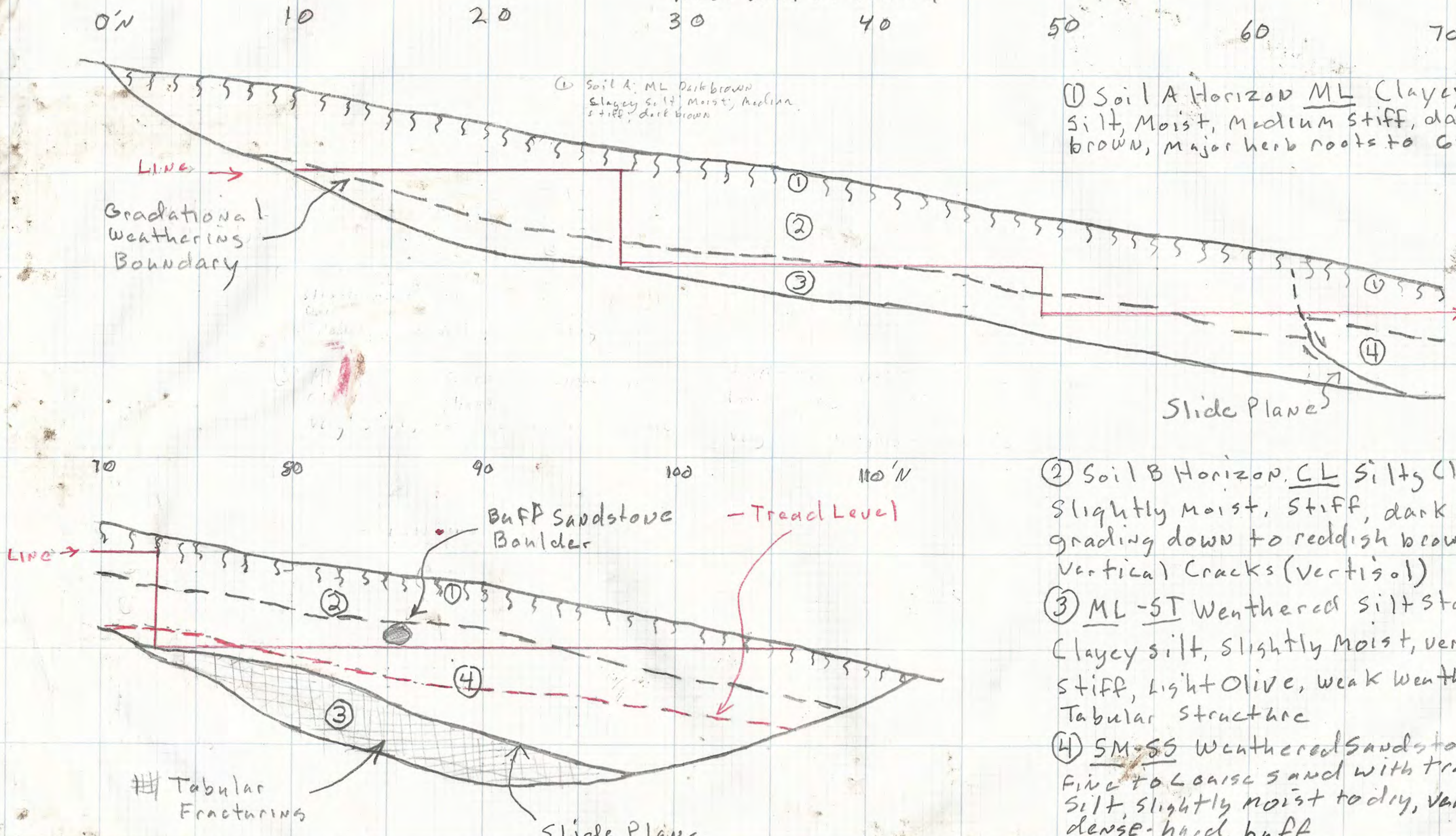
GSH

ANALYSIS BY: GCS | CHECKED BY AH | DATE 03/21/16

* From Section Line Shown On Figure 4 and 5

Lot 43 Ski Lake
2063-01N-16

Trench 1 0-113'
West Wall of Trench



① Soil A Horizon ML Clayey silt, Moist, Medium stiff, dark brown, Major herb roots to 6"

② Soil B Horizon, CL Silty Clay, Slightly moist, Stiff, dark brown grading down to reddish brown, Vertical Cracks (vertisol)

③ ML-ST Weathered siltstone, Clayey silt, slightly moist, very stiff, light olive, weak weathered Tabular structure

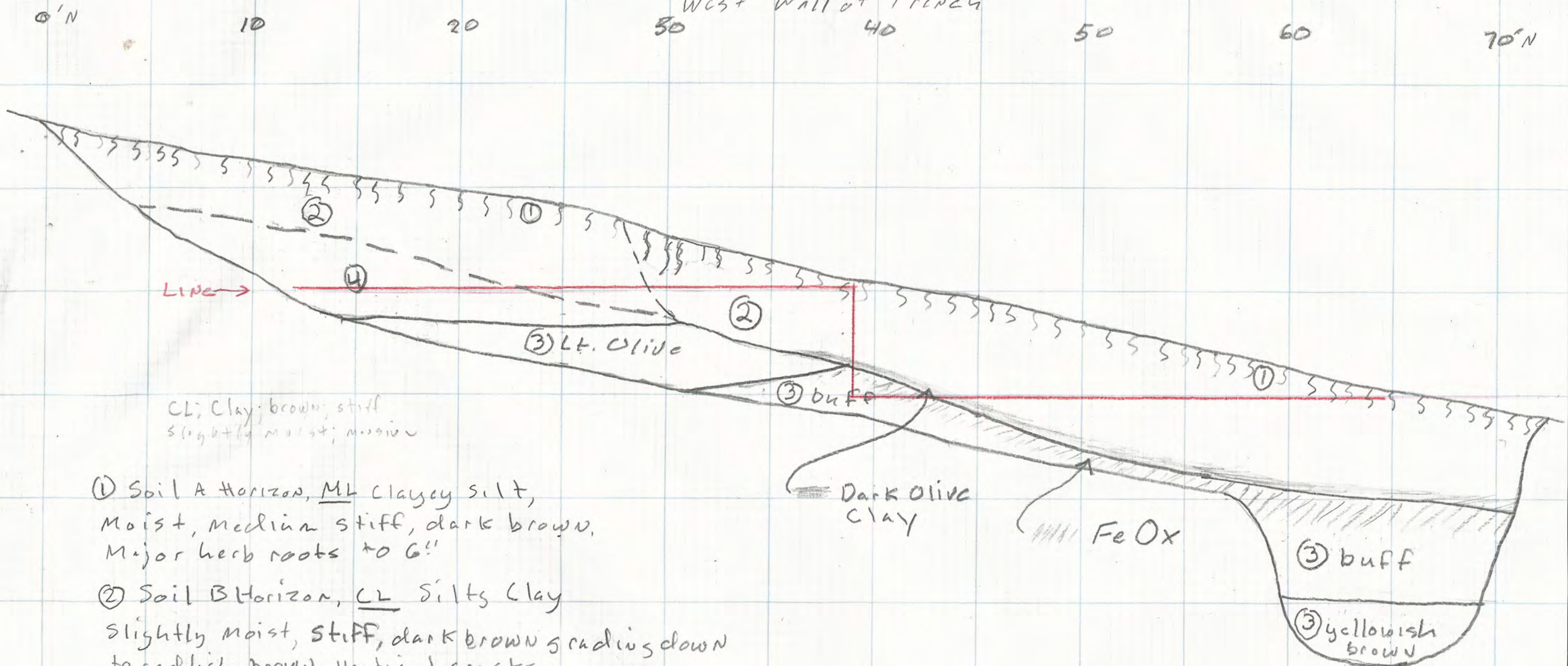
④ SM-SS Weathered Sandstone Fine to coarse sand with trace silt, slightly moist to dry, very dense-hard, buff

GSH

Project 2063-01N-16
By GCS
Date 2/23/16

Lot 43 Ski Lake
2063-01N-16

Trench 2 0-72'
West Wall of Trench



① Soil A Horizon, ML Clayey silt,
Moist, medium stiff, dark brown,
Major herb roots to 6"

② Soil B Horizon, CL Silty Clay
Slightly moist, stiff, dark brown grading down
to reddish brown, vertical cracks
(Vertisol)

③ ML-ST weathered siltstone,
Clayey silt, slightly moist, very stiff,
light Olive - buff and yellowish brown,
weak weathered tabular structure

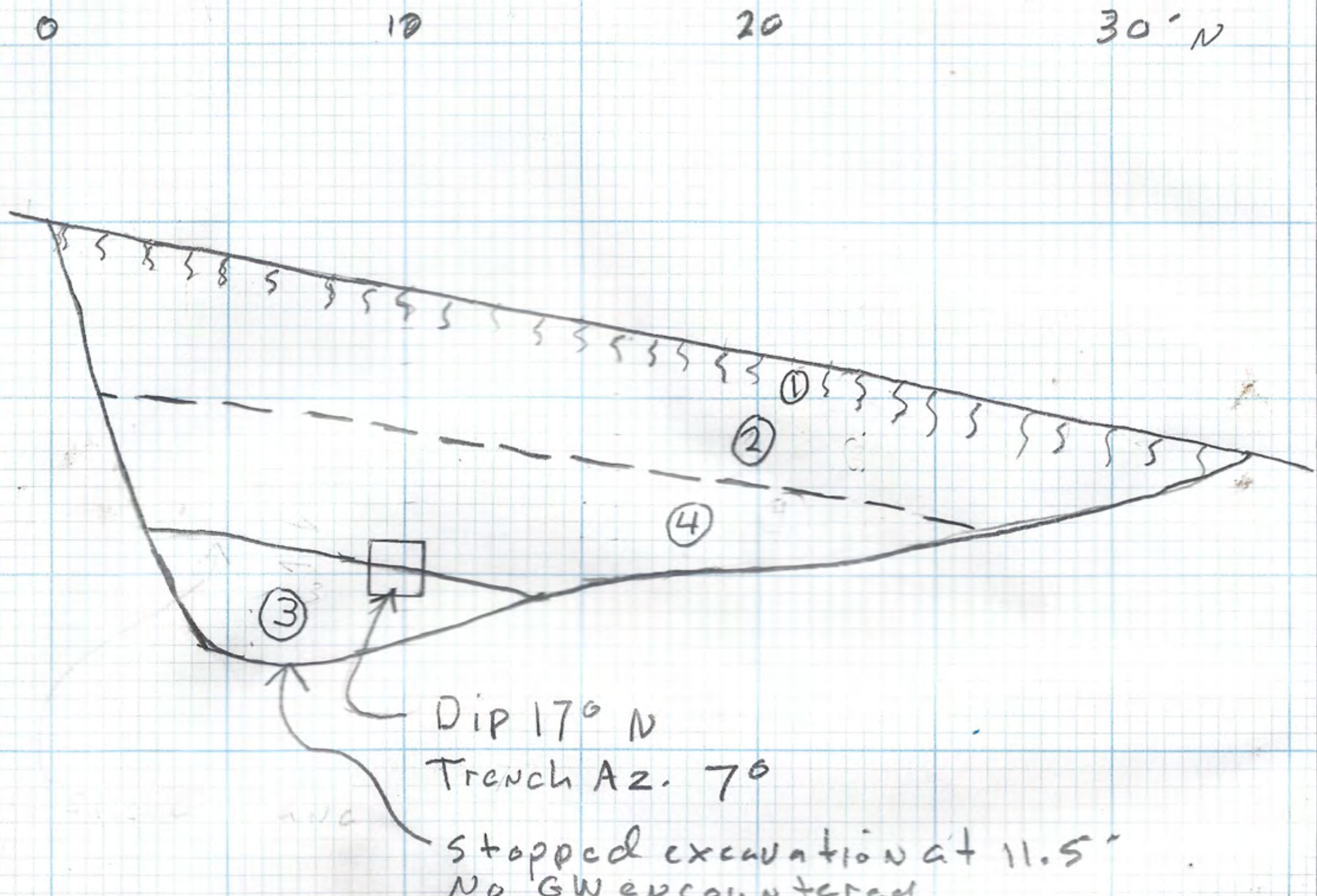
④ CL. Silty Clay, slightly moist, stiff
reddish brown, massive.

Trench 2
0'-72' north

Project 2063-01N-16
By CGS
Date 2/23/16

Sky Lake Lot 43
2063-01N-16

Test Pit 1
West Wall of Pit



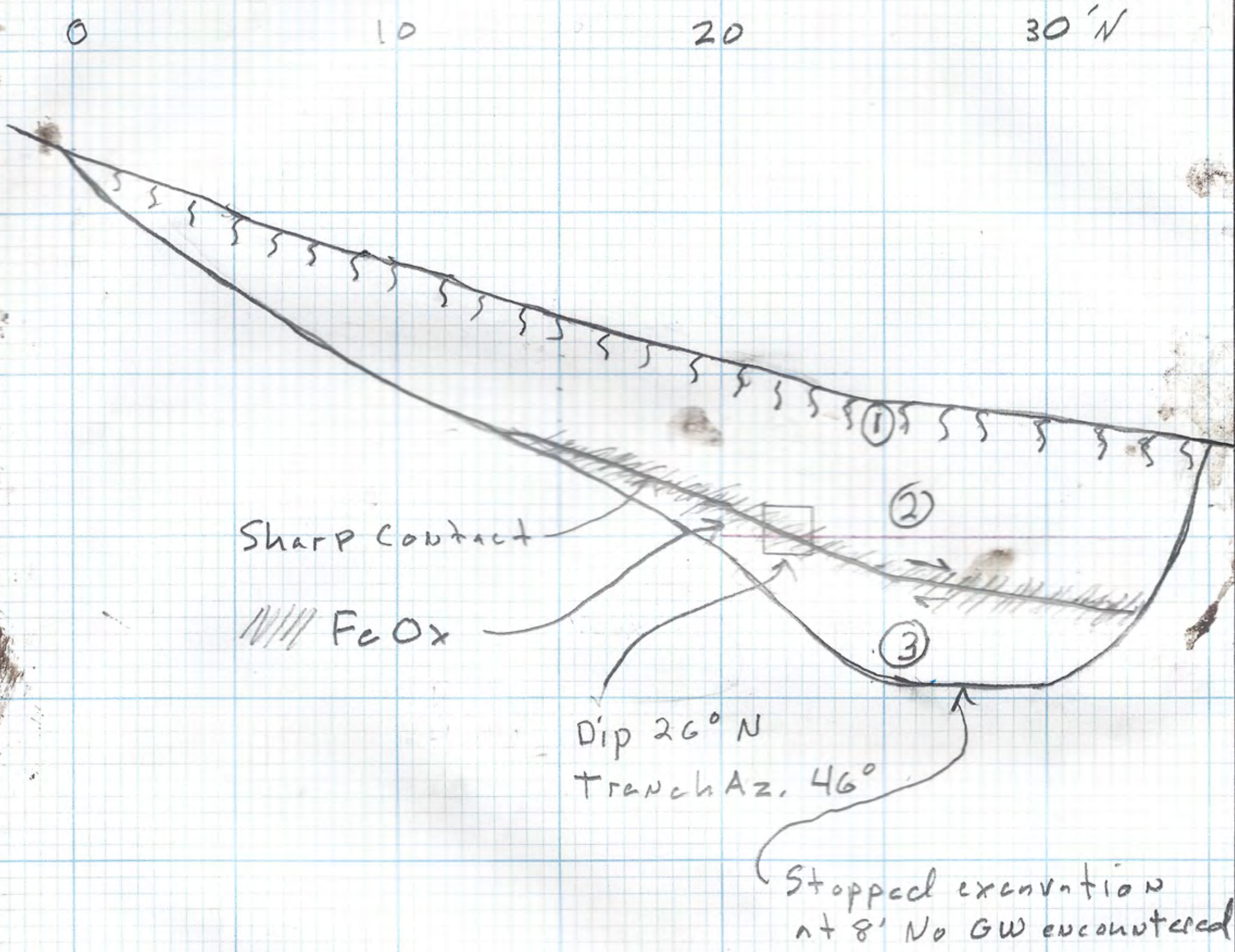
* See Trench 1 for unit descriptions

Project 2063-01N-16
By GCS
Date 2/23/16

GSH

Ski Lake Lot 43
2063-01N-16

Test Pit 2
West Wall of Pit



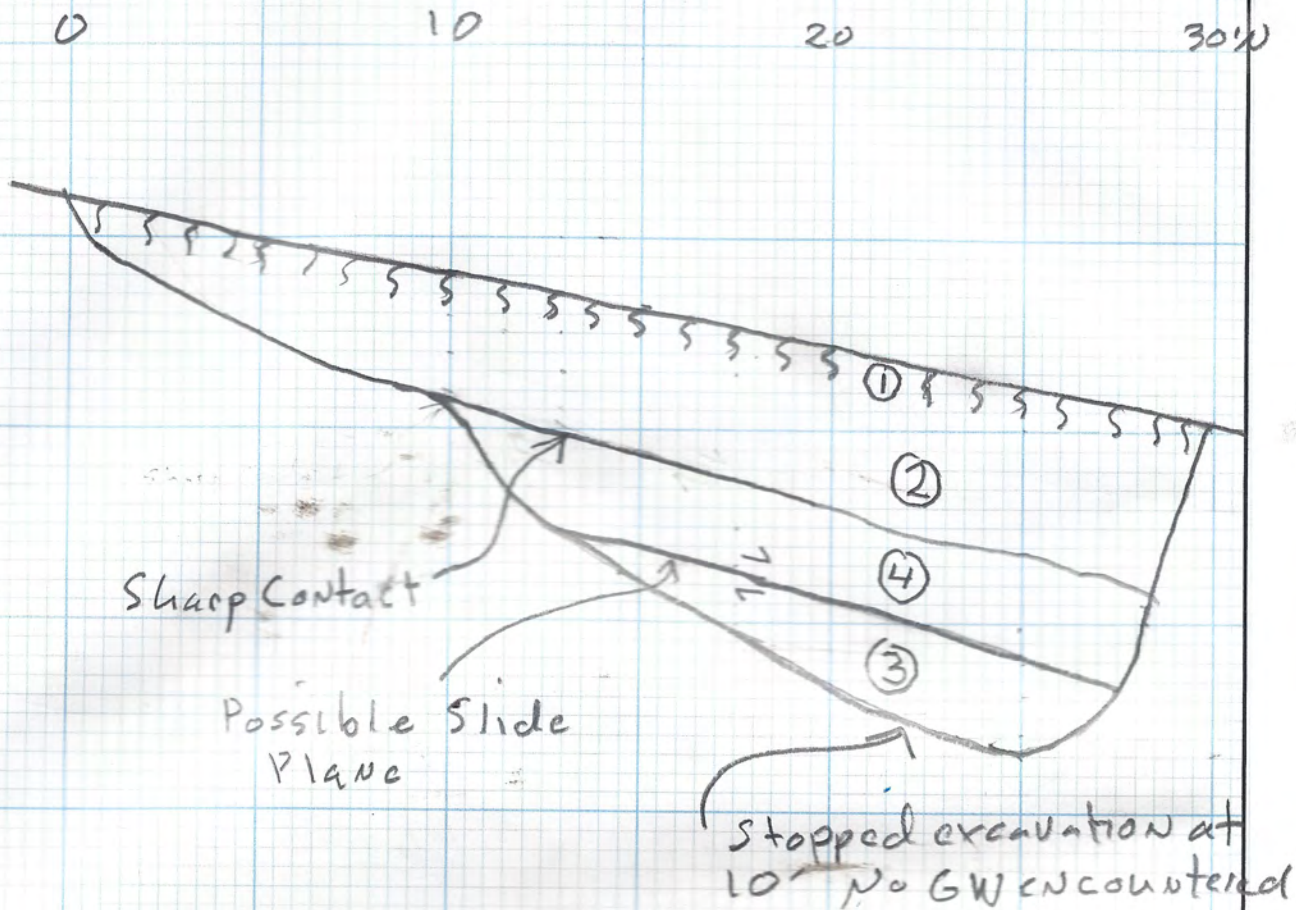
* See Trench 1 for unit descriptions

Project 2063-01N-16
By GCS
Date 2/23/16

GSH

Ski Lake Lot 43
2063-01N-16

Test Pit 3
West Wall of Pit



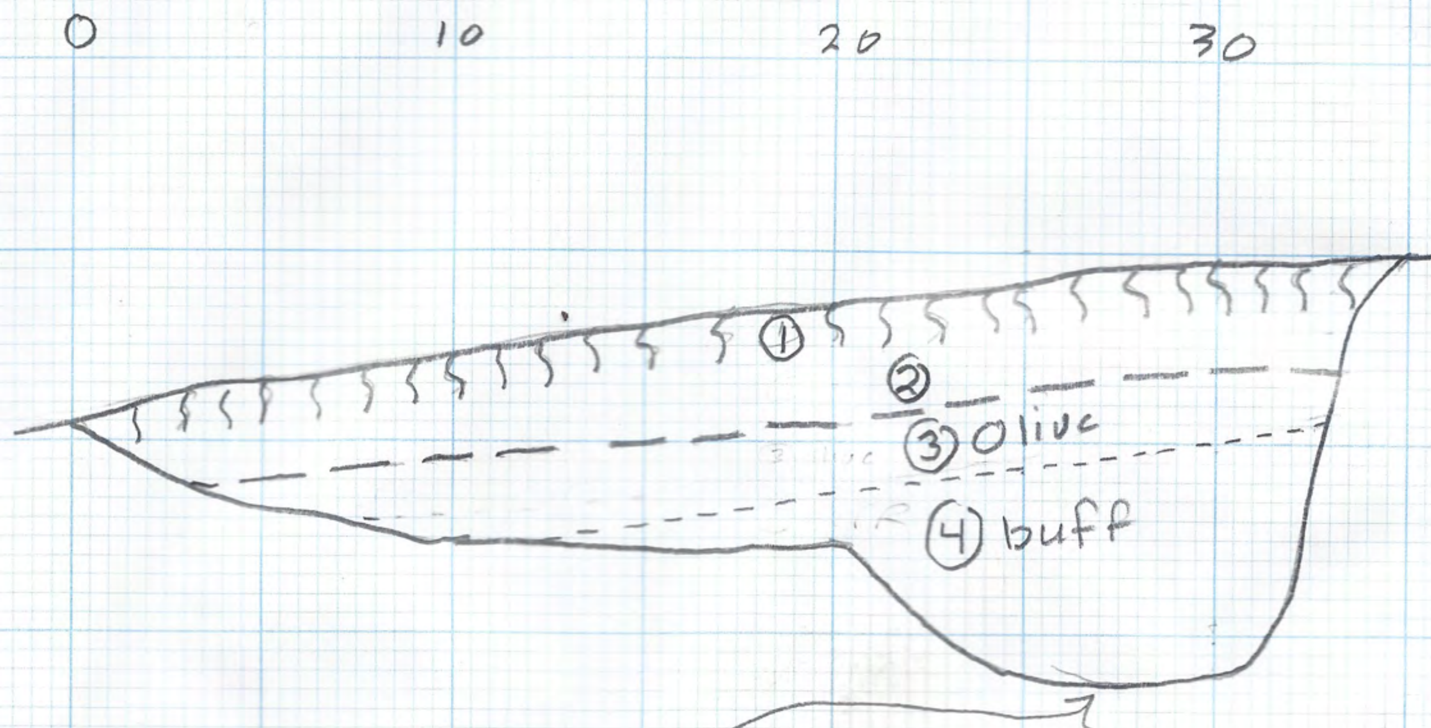
*See Trench 1 for unit descriptions

Project 2063-01N-16
By GCS
Date 2/23/16

GSH

Ski Lake Lot 43
2063-01N-16

Test Pit 4
North Wall of Pit



Stopped excavation at 11'
No GW encountered

* See Trench 2 for Unit descriptions

Project 2063-01N-16
By GCS
Date 2/23/16

GSH

Summit at Ski Lake Lot 43
Job No. 2063-01N-16


Trench 1 STA 15'


Trench 1 STA 25'

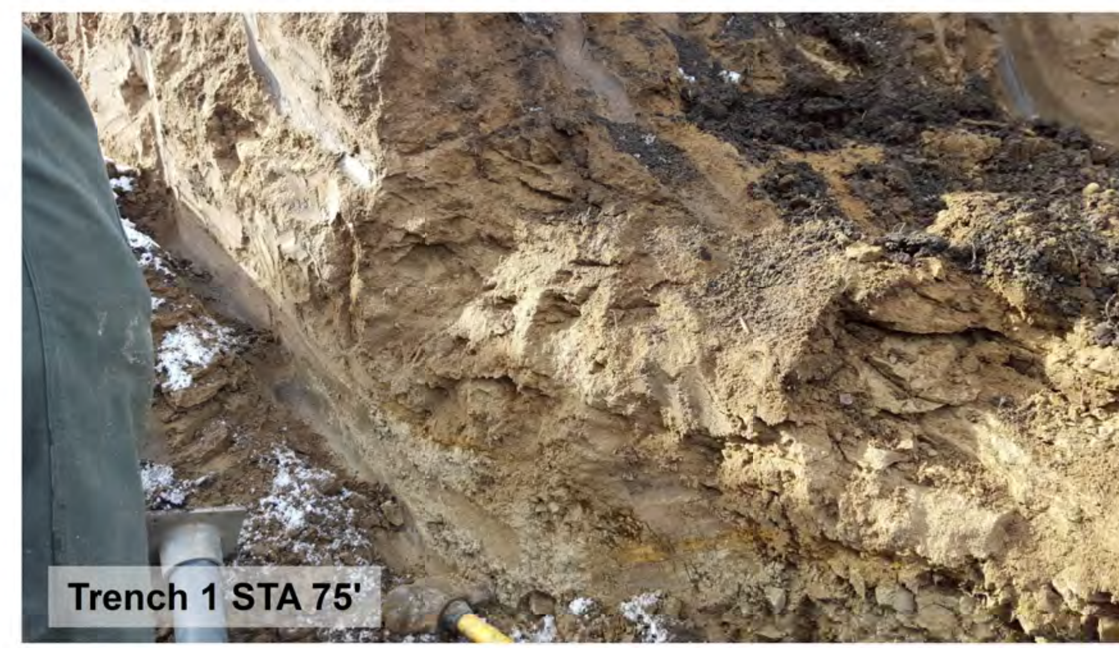

Trench 1 STA 35'


Trench 1 STA 45'


Trench 1 STA 55'


Trench 1 STA 65'


Trench 1 STA 70'

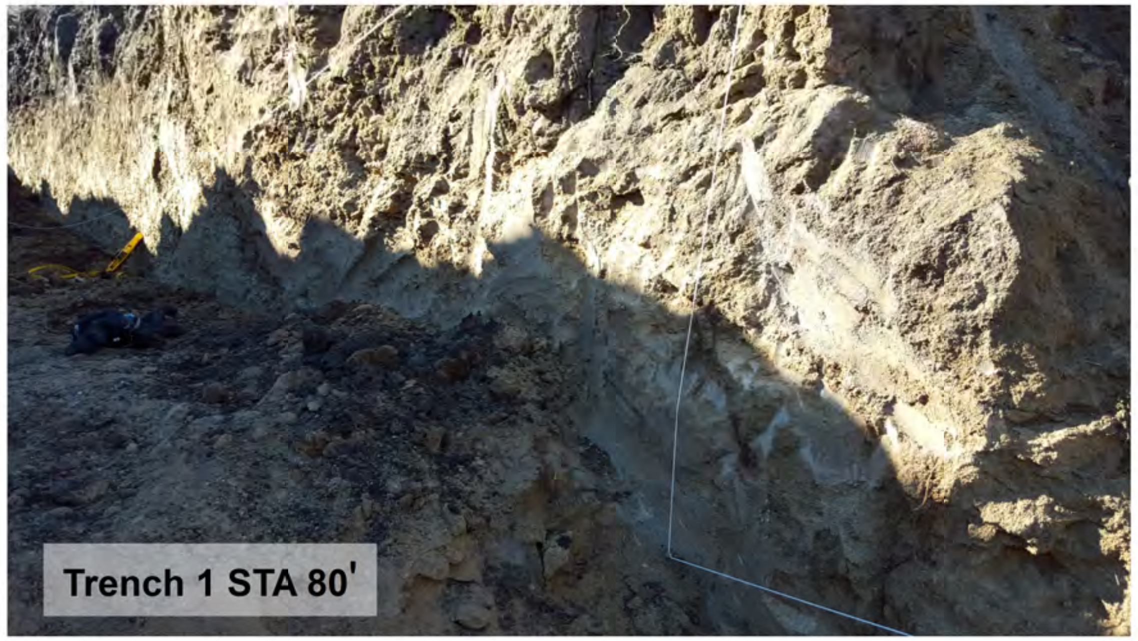

Trench 1 STA 75'

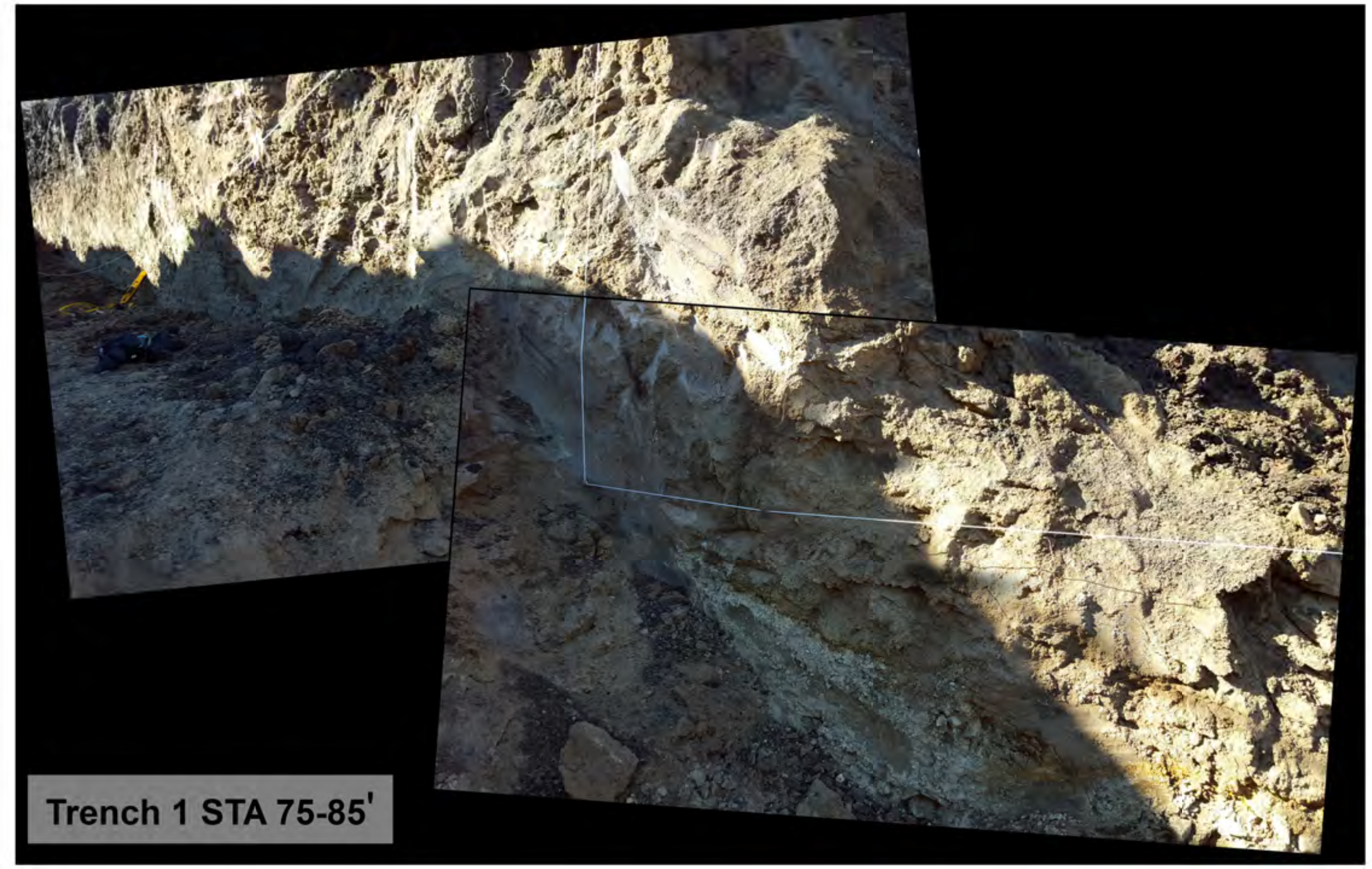
View Electronically:

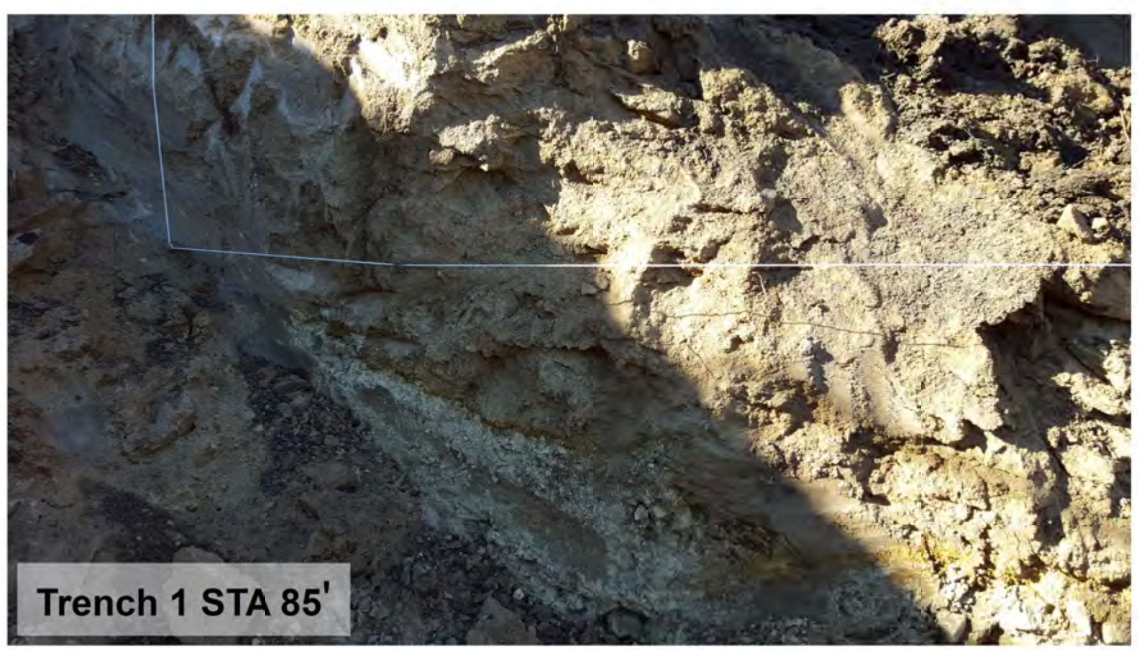
https://drive.google.com/folderview?id=0B5H7rPIYywl-WFpfUFNMMmpLVlk&usp=sharing

APPENDIX B-1
TRENCH AND TEST PIT
PHOTOGRAPHS

GSH







View Electronically:

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APPENDIX B-2
TRENCH AND TEST PIT PHOTOGRAPHS

GSH

APPENDIX B-3
TRENCH AND TEST PIT
PHOTOGRAPHS

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View Electronically:

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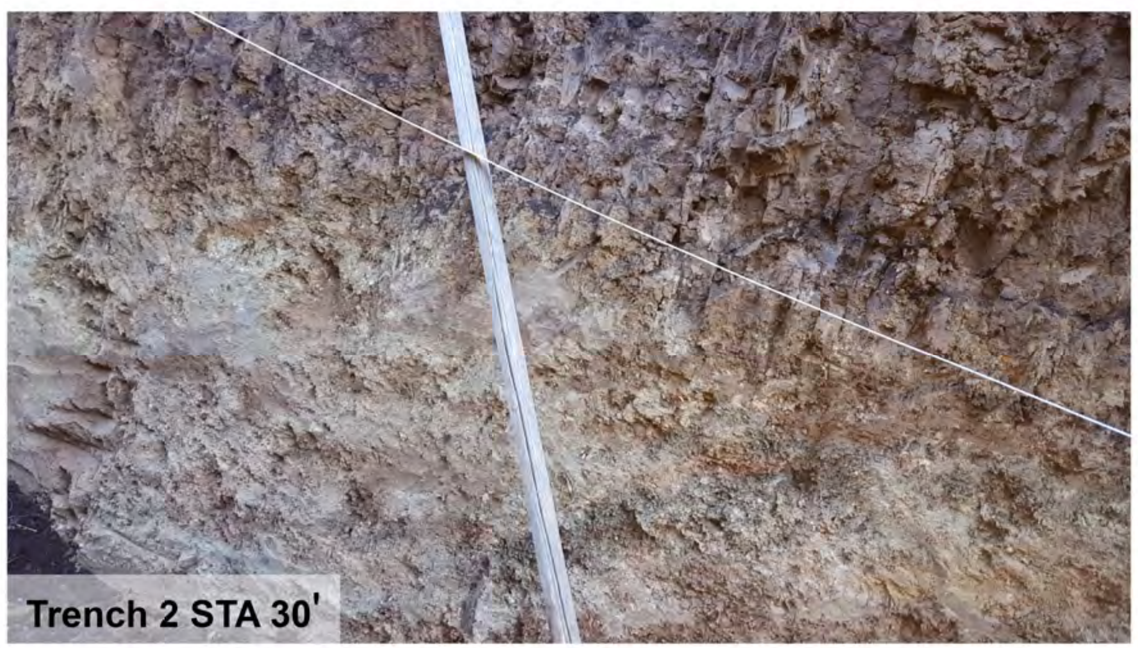
APPENDIX B-4
TRENCH AND TEST PIT PHOTOGRAPHS

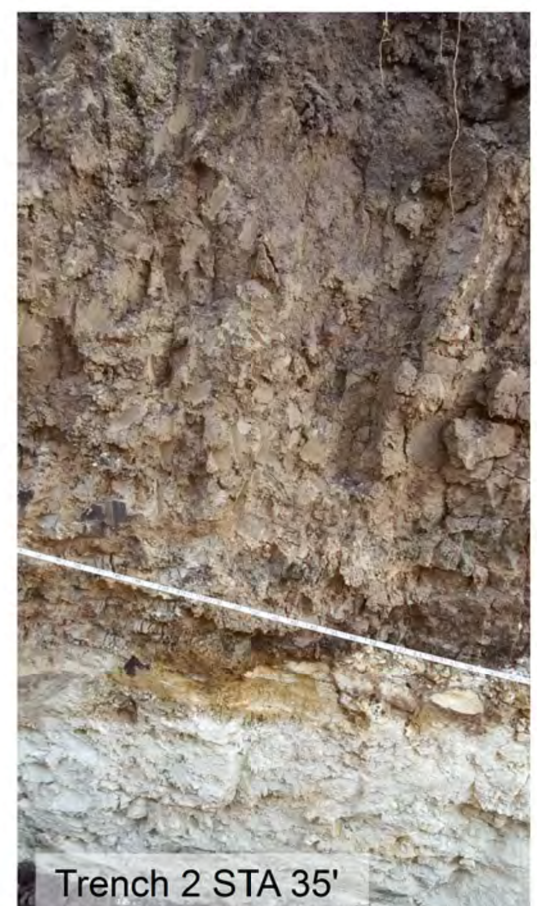
GSH


Test Pit 3


Trench 2 STA 15'


Trench 2 STA 25'


Trench 2 STA 30'


Trench 2 STA 35'


Trench 2 STA 45'

View Electronically:

https://drive.google.com/folderview?id=0B5H7rPIYywl-WFpfUFNMMmpLVlk&usp=sharing

APPENDIX B-5
TRENCH AND TEST PIT PHOTOGRAPHS

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APPENDIX B-6
TRENCH AND TEST PIT
PHOTOGRAPHS

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View Electronically:
https://drive.google.com/folderview?id=0B5H7rPIYywl-WFpfUFNMMmpLVlk&usp=sharing