GSH

REPORT
GEOLOGICAL STUDY
LOTS 50R TO 54R SUMMIT AT SKI LAKE NO. 13
EAST CLAIRETINA COURT
HUNTSVILLE, UTAH

Submitted To:

Valley Enterprise Investment Company
Attention: Mr. Ray Bowden
5393 East 3850 North
Eden, Utah

Submitted By:

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

June 3, 2016

Job No. 2077-01N-16

Exhibit A

GSH

June 3, 2016
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Mr. Ray Bowden
Valley Enterprise Investment Company
5393 East 3850 North
Eden, Utah 84310

RE: Report
Geological Study
Five Residential Development Lots
Lots 50R, 51R, 52R, 53R, and 54R
The Summit at Ski Lake Phase 13
Weber County, Utah
(Parts of Section 24, Township 6 North, Range 1 East, Salt Lake base and meridian)

1. INTODUCTION

In response to your request, GSH Geotechnical, Inc (GSH) has prepared this Geological Study for proposed residential construction for Lots 50R, 51R, 52R, 53R, and 54R at the Summit at Ski Lake Phase 13. The Summit at Ski Lake Phase 13 Subdivision (Parcel) 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, Township 6 North, Range 1 East, Salt Lake base and meridian. The Summit at Ski Lake Phase 13 consists of five residential development lots roughly one-acre or greater in area with common space, comprising a total area of approximately 9.3 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 downslope of the Phase 13 parcel. The Via Cortina roadway accesses the five development lots as shown on Figure 2, Site Plan. Elevation rises approximately 110 feet from the north side of the Parcel to the south side of the Parcel. Based upon plan drawings prepared by Great Basin Engineering (2016) proposed or suggested "Building Areas" for each of the lots is shown on Figure 2. The general area of the Phase 13 Parcel and improvements includes slopes on the order of 20-pecent to 32-percent.

1.1 Weber County Natural Hazards Overlay Districts

Because the proposed residential development is located on a sloping hillside area with susceptible expansive soil and rock conditions, Weber County (Planning Commission) has recorded the five lots as Restricted (R), and requested that additional geotechnical and geological studies be conducted prior to use to evaluate conformance with development plans. At this time specific guidelines for these studies have not been specified by the County, however Weber County Code Section 104-27-2, Potential Hazards (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.

Sensitive Lands Overlay District maps addressing Landslide/Tectonic Subsidence zones for Weber County is presently not available for the site. A preliminary review of site geological mapping prepared by Utah Geological Survey (UGS) geologists (King, et al, 2008), show on Figure 3, Geologic Map has indicated that parts of the Phase 13 Parcel is within mapped Quaternary landslide deposits (Qms and Qmc). Based upon our review of the mapping, the Parcel is mapped as being mostly underlain by Tertiary age Norwood Formation (Tn) rocks, with peripheral areas that include Quaternary landslide deposits (Qms and Qmc) as mapped by King, et al., (2008).

To present the adequacy of our proposed geotechnical and geological studies for the Phase 13 development, a Work Plan for these proposed studies dated March 31, 2016 was prepared for submission to the Weber County Staff.

1.2 Work Plan

The Work Plan presented the following scope of work (work plan) for this evaluation of the Parcel in general and the five residential development lots as it pertains to the Weber County Natural Hazards Overlay District Code:

Pending any scoping meeting amendments, 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 five walk-in exploration trenches approximately 60 to 120 feet in length and as much as 14 feet in depth and the geotechnical logging and sampling of 5 walk-in test pits to a depth of as much as 20 feet, and two geotechnical hollow-stem auger borings to 30 to 50 feet in depth (or auger refusal), at the locations 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 25-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, trenches and borings 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 trenches, test pits and borings.*
- *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 and radon exposure.*
- *Cross-sections of slopes 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.*

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 on the south side of the Parcel. 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 Phase 13 Parcel. 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 on the south side of the Parcel 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 are included on Figure 4, Work Plan.

2.2 Field Program

GSH conducted field operations at the site on the dates April 11 through April 18, 2016. The field program involved the excavation and geological logging of five exploration trenches and five test pits and the advancement of two drilled boreholes on Parcel locations shown on Figure 4. The excavations and borings were logged to observe and characterize site subsurface/geologic and groundwater conditions for the site and the proposed residential development improvements. Trenches and test pits were located to evaluate the conditions for each of the proposed Building Areas, and borings were placed on slope locations in order to evaluate geologic subsurface conditions relative to slope stability conditions for the Phase 13 Parcel. The locations of our trenches, test pits and borings are included on Figure 4. Trenches were from 72 to 132 feet in length and extended to depths of 4.0 to 16.0 feet, and the test pits consisted of walk-in excavations, 15.0 to 25.0 feet in length and extending to depths of 10.0 to 15.0 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 Building Areas across the site. The trenches and test pits were excavated using a 20-ton class excavator with a 36-inch bucket and was refused at depth in most of the excavations as indicated in our field logs. 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. Feature location and elevation data were recorded using a hand-held GPS receiver device.

Our field program was conducted by Senior Engineering Geologist Dr. Greg Schlenker, PG of our geotechnical staff. Mr. Amos Allard, Staff Geologist 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. Mr. Allard also supervised drilling operations for Boring 1 and 2.

The soils and geology in the trenches, test pits and borings 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 5 through Figure 9, Log of Trenches, and Figure 10 and Figure 11, Log of Test Pits. It should be noted that no Log for Test Pit 54 is presented as the excavation at that location was refused at a depth of 1.0 feet. The soil and rock units observed in the trenches and test pits were classified in accordance with the Unified Soil Classification System (USCS), and were further classified on the basis of geological site formation processes.

Bulk and thin wall samples of representative soil layers encountered in the test pits 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 will be included in our accompanying geotechnical report. Groundwater was not observed in any of the excavations or test pits during the dates of our field program.

The logs of the two borings shown on Figure 4 that were made for our concurrent geotechnical study and included in this reporting, are included on Figure 12 and Figure 13. The borings were completed using a CME 55 truck-mounted drill rig using hollow-stem auger/rotary wash equipment and methods. Soil and rock samples were recovered at 2.5-foot intervals using driven 2.42-inch inside diameter drive Dames & Moore sampler. The borings were also logged in accordance with the Unified Soil Classification System (USCS).

2.3 LiDAR - Slope Analysis

To asses slope conditions, interpret terrain, and develop site specific geologic cross sections 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 14, LiDAR-Slope Analysis, presents the results of our slope analysis efforts. Shown on Figure 14 is the 25-percent, and greater than 30-percent slope gradients across the site. The shaded relief rendering on Figure 14 provides a visual basis for landform interpretation, and the contour elevation data shown on Figure 14 is used to develop the cross sections shown on Figures 15 and 16, 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 Sections shown on Figure 15 and 16 will be used for modeling 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 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 14, field reconnaissance of the general site area, and the interpretation of the trenches, test pits and borings excavated and drilled 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 in the Ski Lake area 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 2015 observations of the Logged Roadway Geology Cut shown on Figure 4, revealed bedded exposures of lacustrine 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

As shown on Figure 2 and Figure 4, the site consists of an area of approximately 9.3 acres in size that is currently vacant and undeveloped. At the time of our 2016 field program, general grading for the Via Cortina roadway Clairtina Court had taken place. Surface vegetation consists of open areas of grasses, weeds and sage brush with clustered wooded areas of scrub oak, alder, box elder and maple tree cover. The topography of the site consists of a north facing hillslope 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 our logs, Figures 5 to Figure 9, Logs of Trenches, and Figure 10 and Figure 11, Logs of Test Pits, generally consisted, from bottom to top of:

Bedrock sequences consisting of: 1) weathered Claystone CS light olive in color, slightly to moderately weathered, very thinly bedded, moderately fractured, weak consistency; 2) weathered Siltstone ST light olive, slightly to moderately weathered, thinly to moderately and massively bedded, moderately fractured, hard consistency; and 3) weathered Sandstone SS light brown to brown, slightly to moderately weathered, thinly to moderately and massively bedded, moderately fractured, weak consistency. The bedrock observed in the trenches and test pits at the site are believed to be localized lacustrine sediments, derived from Norwood Formation tuff deposits.

Soil sequences overlying the bedrock consist of weathered residual soils derived from underlying Norwood Formation rocks, or transported colluvial and/or landslide derived soils from the weathered residual soils. The weathered residual soils consisted of weathered siltstone (ST-ML) consisting of clayey silt, generally light yellowish brown, very stiff, slightly moist, with massive structure. The transported soils included silty clay and clays (CL), yellow-red, reddish brown and brown, stiff, slightly moist, massively bedded

Surficial pedogenic soil sequences, consisting soil A horizons of (ML) clayey silt, moist, medium stiff, dark brown, major herb roots from 6.0 to12.0 inches in thickness that were observed on the surfaces of the trench and test pit exposures. Below the surface in Trenches 50, 51 and 52, thick soil B-horizon vertisol sequences that extended as much as 6.0 to 10.0 feet in depth were observed. The vertisol soils consisted of silty clays (CL), dark brown to reddish brown, stiff, slightly moist, with vertical cracks extending the thickness of the vertisol units.

Landslide movement was observed in Trench 50 and Trench 51, and evidence of movement within the same feature was observed during our previous 2015 study in test pits GSH-2, GSH-3 and GSH-4 and on the Logged Roadway Geology Cut (GSH, 2015). This landslide feature is mapped on Figure 14, as Qms-2015. The landslide deformation and movement was observed beginning at the Logged Roadway Geology Cut on the Via Cortina roadway with movement detected in the trenches and test pits downslope from the cut in a northward direction, extending beyond the north property boundary, and joining a larger mapped landslide feature (King et al., 2008) north of the property boundary. The axial north-south length of the feature to the north property boundary is approximately 325 feet, and the east-west width is approximately 245 feet. The observed thickness of the landslide feature was from 10.0 feet in Trench 50, and possibly as thick as 15.0 feet in Test Pits GSH-2 and GSH-3 from our 2015 study (GSH 2015). The soils observed in Trench 51, appear to apical landslide flank soils on the western margin of the slide feature. The slide plane surface feature observed in the trenches displayed nominal deformation along the plane surface, with some oxidation accumulations (Fe Oxides) observed along the base of the movement in the trenches.

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 14 of this report. The engineering geology shown 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 (GSH 2015). A summary of the mapping units identified on the site vicinity are listed below in relative age sequence (youngest-top to oldest bottom):

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

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 soil A horizons, with B horizon vertisol sequences that extended in depth (thickness) as much as 10.0 feet, and consisting of to stiff silty clays derived from weathered rock and colluvial sources. At depth weathered rock sequences consisting of claystone, siltstone, and sandstone 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 6.0 to 10.0 feet below the surface in Trenches 50, 51 and 52, and Test Pit GSH-4. 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 14, LiDAR-Slope Analysis. For the Phase 13 Parcel area the slope gradient averaged approximately 30.0-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 lots and Phase 13 parcel. These issues are addressed in order importance below:

1. **Landslide and slump deposits:** Presence of mass-movement landslide and slump deposits (**Qms-2015**) is based upon developed field observations including; deformation of soils and rock beds observed in Trench 50, Trench 51, and Test Pits GSH-2, GSH-3 and GSH-4, and location of the topographic features evident on the LiDAR imagery on Figure 14 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 14 consists of a relatively shallow, approximately 10.0 to 15.0 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 13 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 site location is mapped as "Moderate" by the UGS (Solomon, 1996).

4.2 Conclusions

Based upon our geological studies herein, we believe that the Lots 50R, 51R, 52R, 53R, and 54R of the Summit at Ski Lake Phase 13 are suitable for development, provided that soils identified in our borings, trenches and test pits as subject to past landslide movement as discussed in Section 3.3 of this report, are avoided or mitigated. Although plans are at this time not finalized, we understand that deep foundation systems and soil improvement techniques are being considered as a methodology 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 Trench 50, 51 and 52, and Test Pit GSH-2 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 will be included as part of our concurrent geotechnical study.

Due to the potential for radon to penetrate lower levels of the proposed structures, indoor radon testing must be completed at the time of construction.

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

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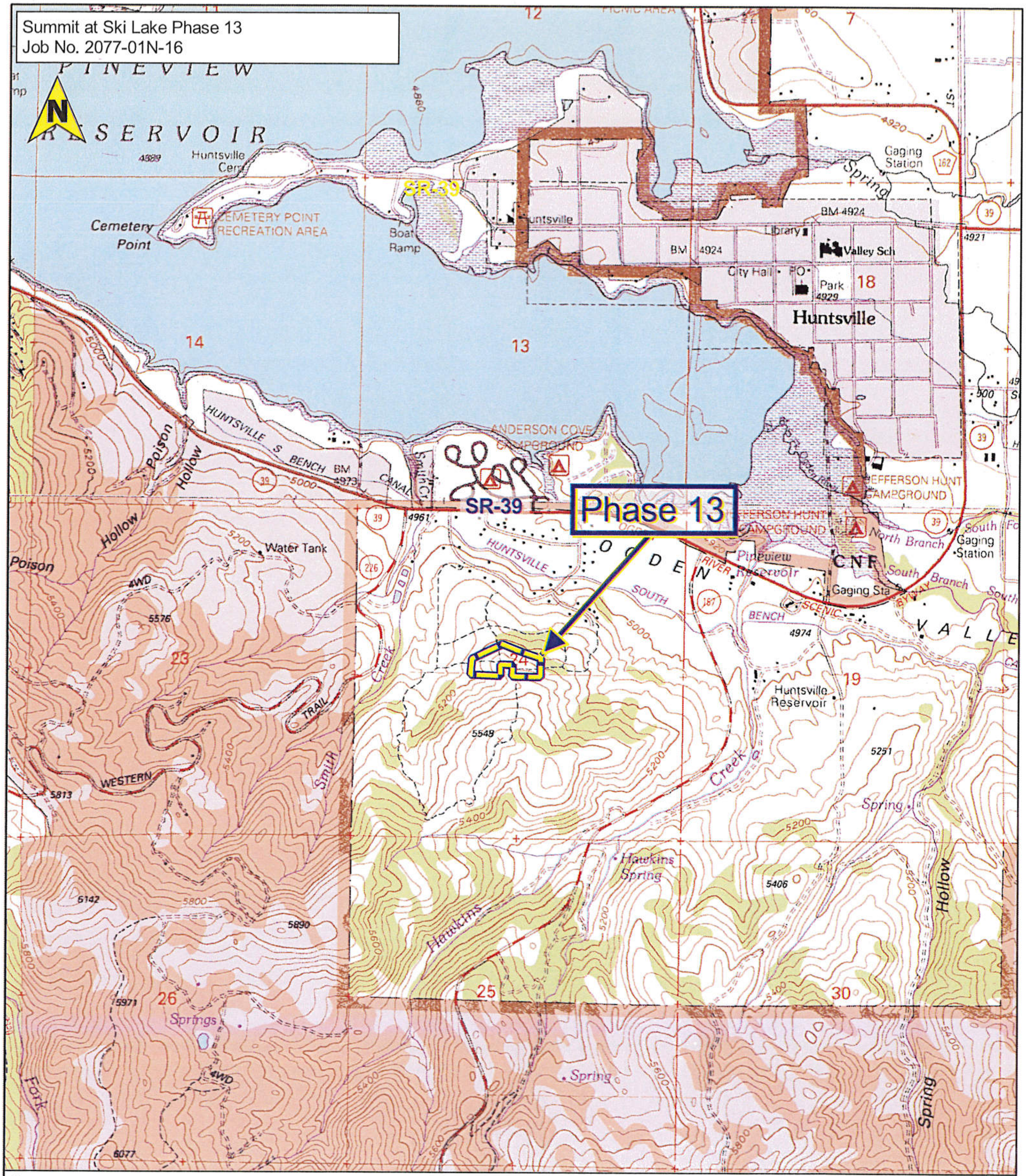
Exhibit A



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

Exhibit A

Base: 2012 6-inch Color HRO Orthoimagery, from Utah AGRC; http://gis.utah.gov/
Elevation: 2006 2.0m Geoprocessed LiDARfrom Utah AGRC; http://gis.utah.gov/

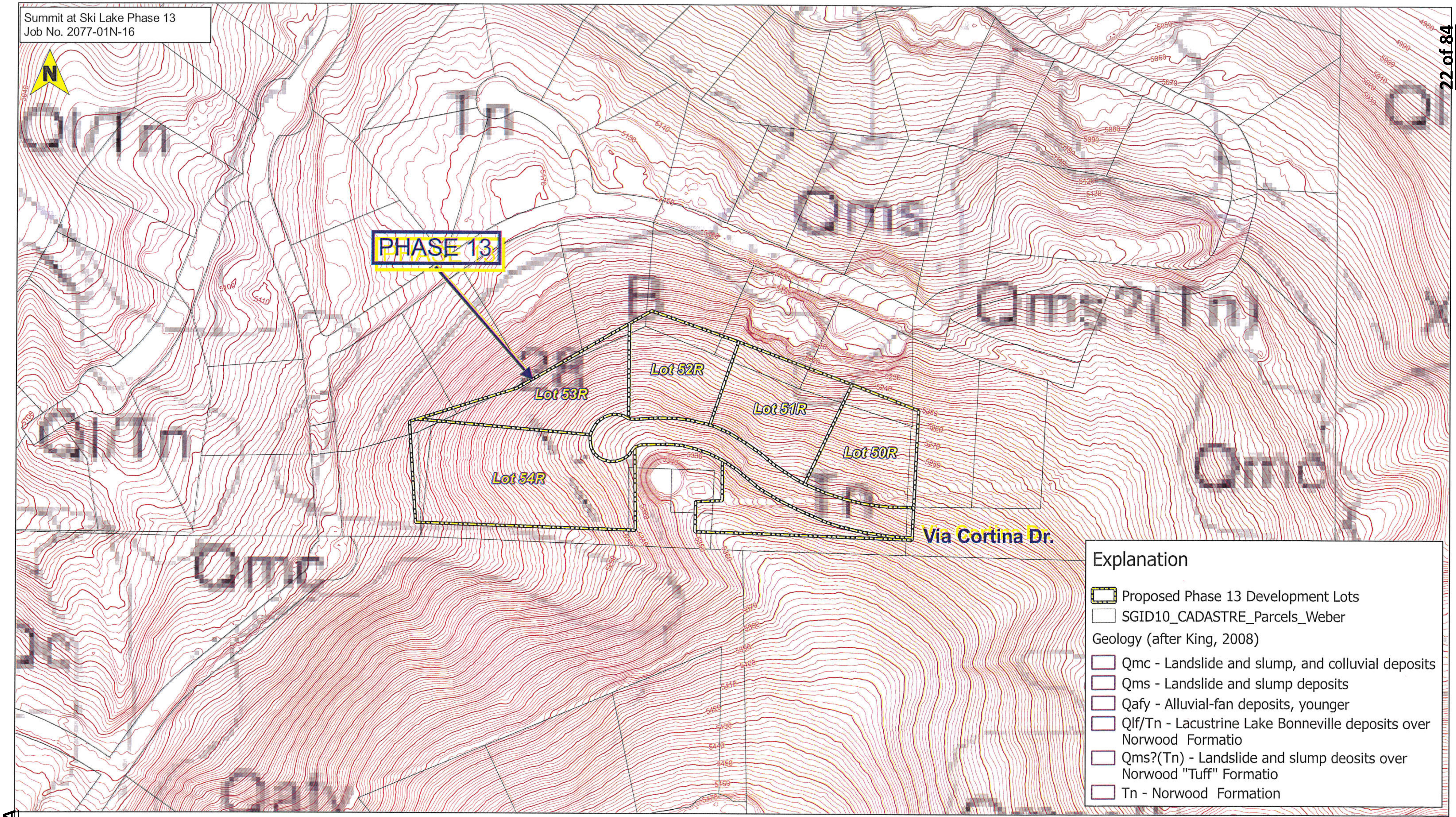
0 100 200 300 400 ft

1:960

FIGURE 2
SITE PLAN

GSH

Exhibit A



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

1:2,400

FIGURE 3
GEOLOGIC MAP

GSH

Exhibit A

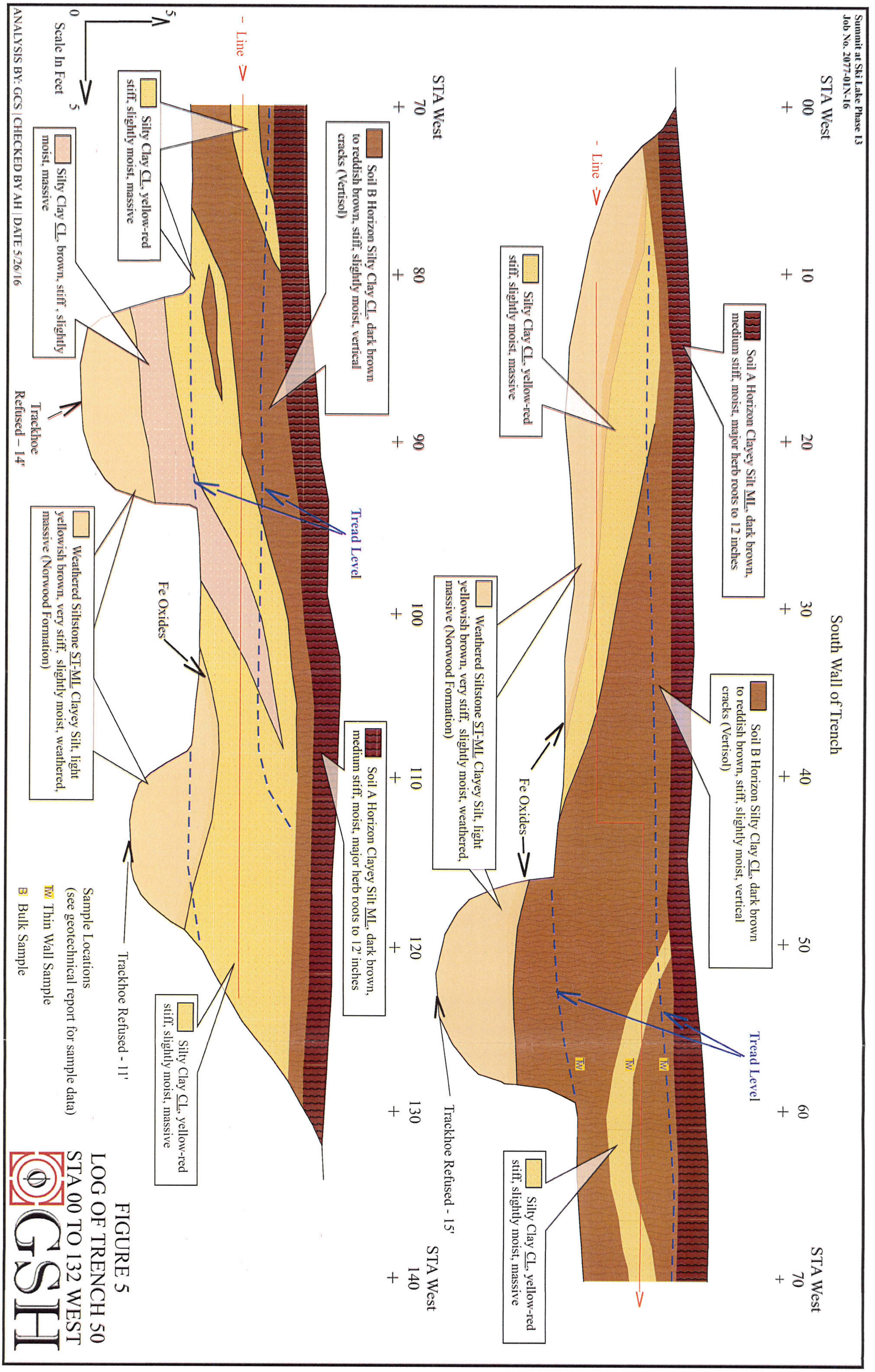
Base: 2012 6-inch Color HRO Orthoimagery, from Utah AGRC; http://gis.utah.gov/
Elevation: 2006 2.0m Geoprocessed LiDARfrom Utah AGRC; http://gis.utah.gov/

0 100 200 300 400 ft
1:960

FIGURE 4
WORK PLAN

GSH

Exhibit A



Summit at Ski Lake Phase 13
Job No. 2077-01N-16
South Wall of Trench
STA West
00
10
20
30
40
50
60
STA West
70
Soil A Horizon Clayey Silt ML, dark brown, medium stiff, moist, major herb roots to 12 inches
Soil B Horizon Silty Clay CL, dark brown to reddish brown, stiff, slightly moist, vertical cracks (Vertisol)
Tread Level
TW
TW
TW
- Line ->
Silty Clay CL, yellow-red stiff, slightly moist, massive
Fe Oxides
Silty Clay CL, yellow-red stiff, slightly moist, massive
Weathered Siltstone ST-ML Clayey Silt, light yellowish brown, very stiff, slightly moist, weathered, massive (Norwood Formation)
Trackhoe Refused - 15'
STA West
70
80
90
100
110
120
130
STA West
140
Soil B Horizon Silty Clay CL, dark brown to reddish brown, stiff, slightly moist, vertical cracks (Vertisol)
Tread Level
Soil A Horizon Clayey Silt ML, dark brown, medium stiff, moist, major herb roots to 12' inches
- Line ->
Fe Oxides
Silty Clay CL, yellow-red stiff, slightly moist, massive
Silty Clay CL, yellow-red stiff, slightly moist, massive
Trackhoe Refused - 11'
5
0
5
Scale In Feet
Silty Clay CL, brown, stiff, slightly moist, massive
Trackhoe Refused – 14'
Weathered Siltstone ST-ML Clayey Silt, light yellowish brown, very stiff, slightly moist, weathered, massive (Norwood Formation)
Sample Locations
(see geotechnical report for sample data)
TW Thin Wall Sample
B Bulk Sample
FIGURE 5
LOG OF TRENCH 50
STA 00 TO 132 WEST
GSH
ANALYSIS BY: GCS | CHECKED BY AH | DATE 5/26/16

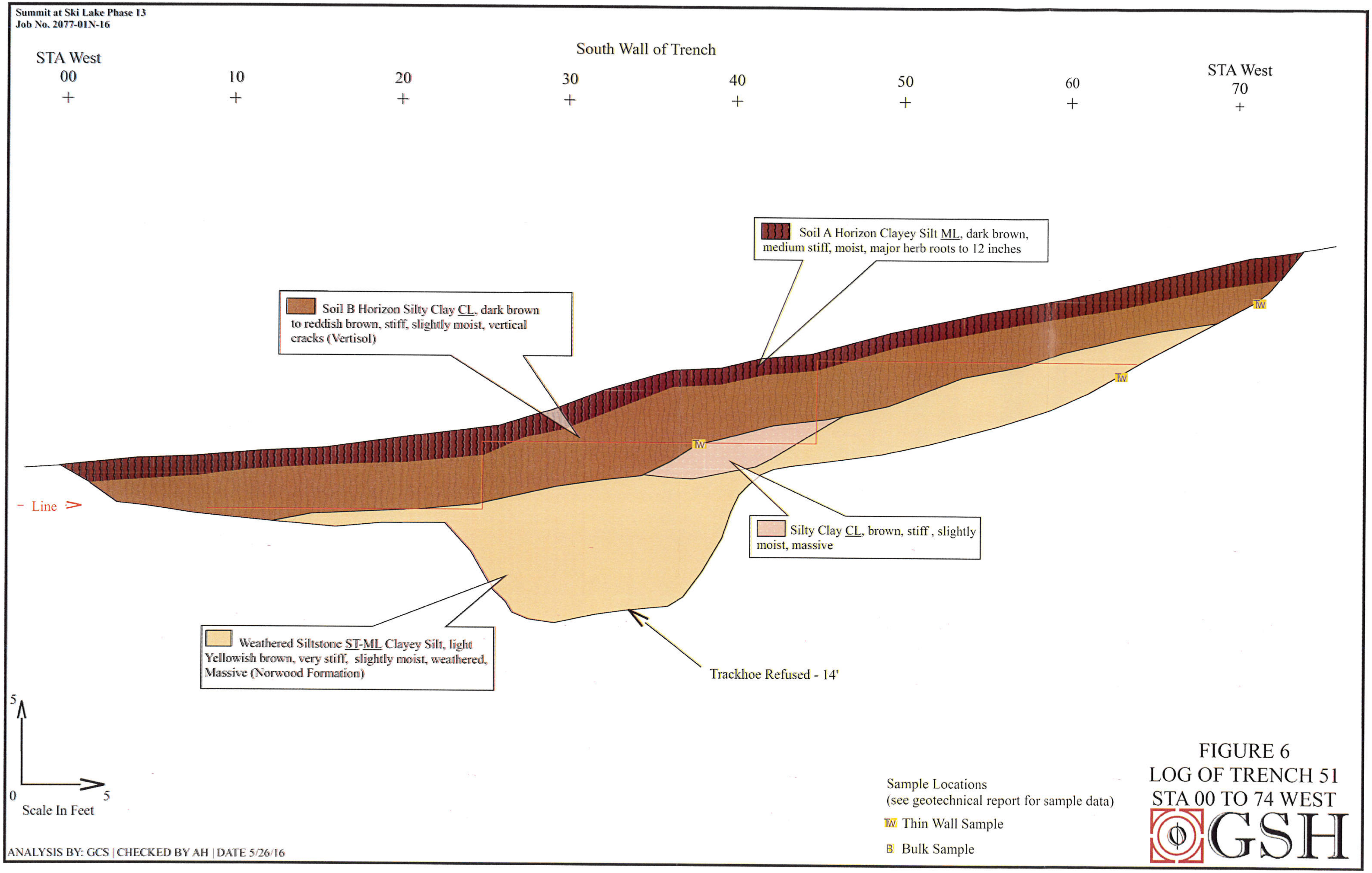


FIGURE 6
LOG OF TRENCH 51
STA 00 TO 74 WEST

Exhibit A

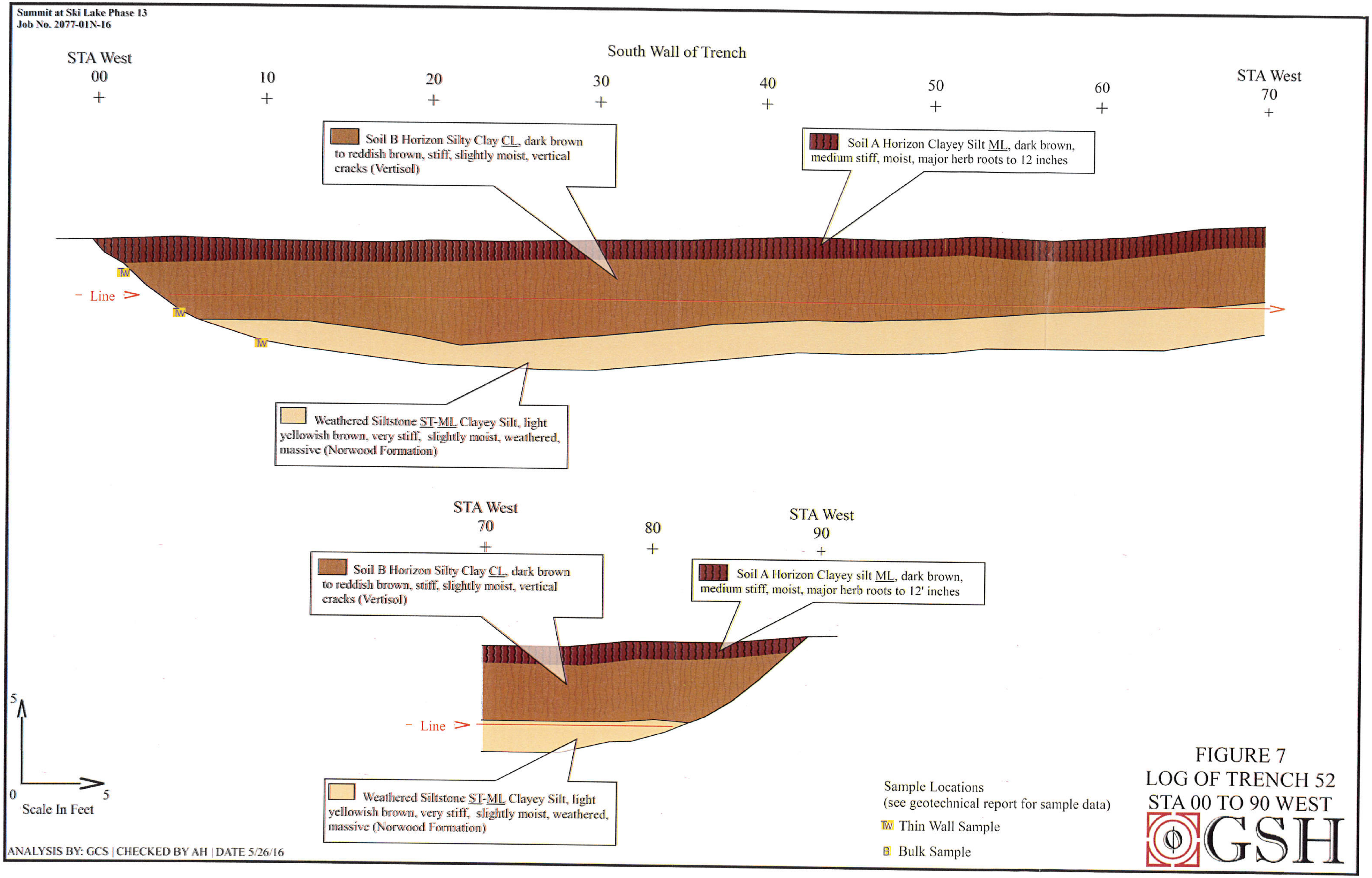

Summit at Ski Lake Phase 13
Job No. 2077-01N-16
South Wall of Trench
STA West
00
10
20
30
40
50
60
STA West
70
Soil B Horizon Silty Clay CL, dark brown to reddish brown, stiff, slightly moist, vertical cracks (Vertisol)
Soil A Horizon Clayey Silt ML, dark brown, medium stiff, moist, major herb roots to 12 inches
TW
TW
TW
- Line ->
Weathered Siltstone ST-ML Clayey Silt, light yellowish brown, very stiff, slightly moist, weathered, massive (Norwood Formation)
STA West
70
80
STA West
90
Soil B Horizon Silty Clay CL, dark brown to reddish brown, stiff, slightly moist, vertical cracks (Vertisol)
Soil A Horizon Clayey silt ML, dark brown, medium stiff, moist, major herb roots to 12' inches
- Line ->
Weathered Siltstone ST-ML Clayey Silt, light yellowish brown, very stiff, slightly moist, weathered, massive (Norwood Formation)
5
0
5
Scale In Feet
Sample Locations
(see geotechnical report for sample data)
TW Thin Wall Sample
B Bulk Sample
FIGURE 7
LOG OF TRENCH 52
STA 00 TO 90 WEST
GSH
ANALYSIS BY: GCS | CHECKED BY AH | DATE 5/26/16

Exhibit A

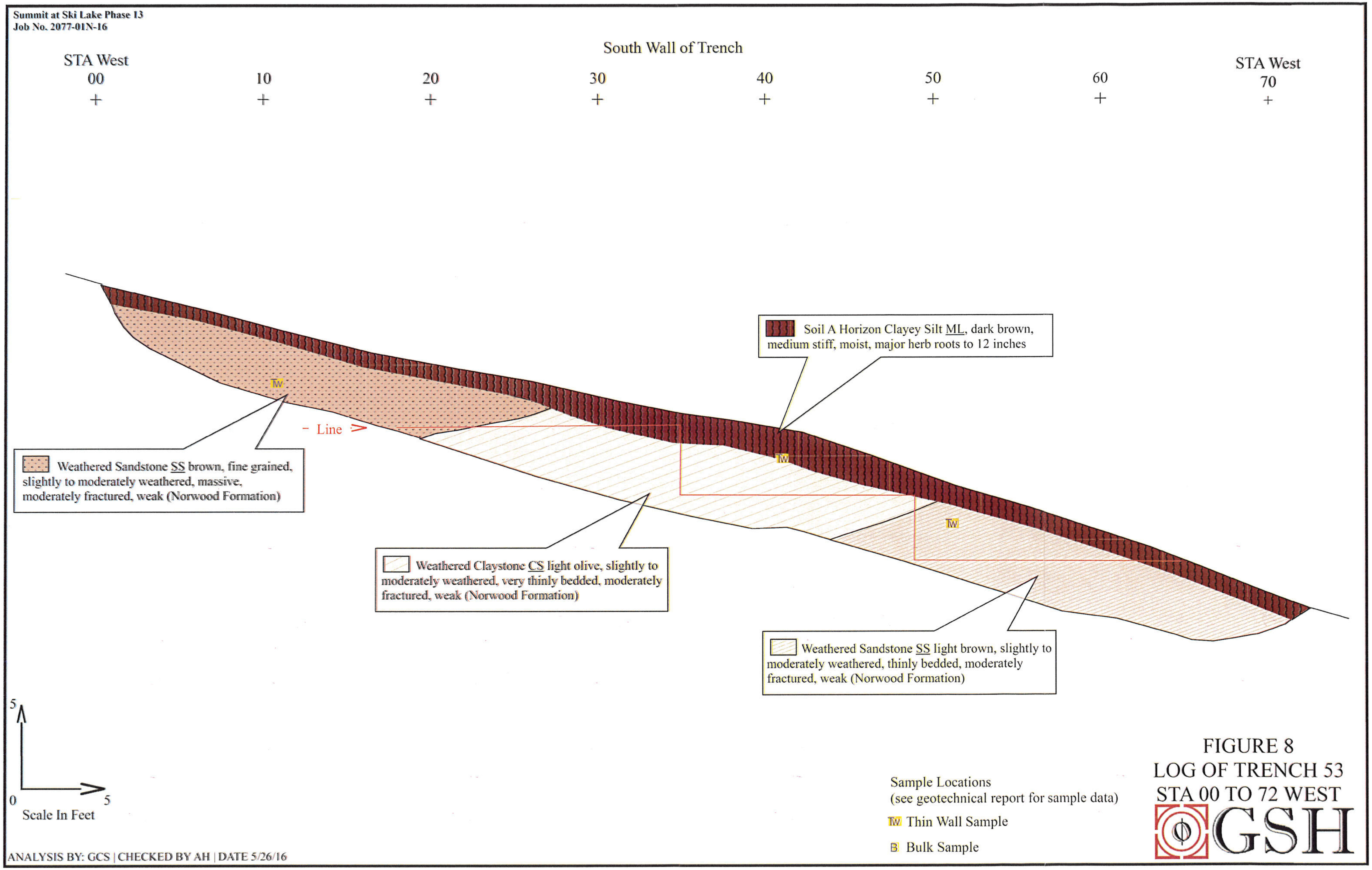
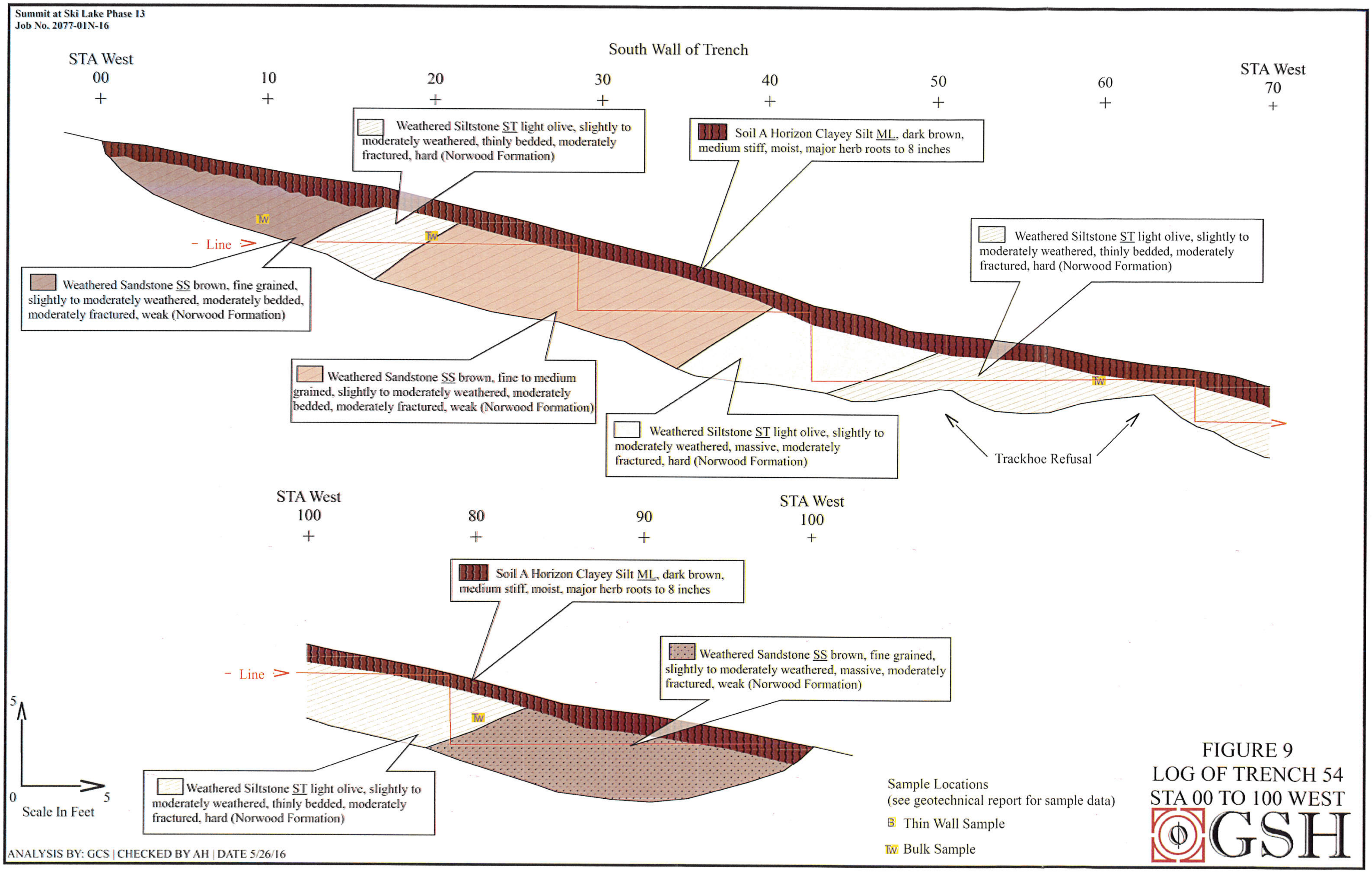

Summit at Ski Lake Phase 13
Job No. 2077-01N-16
South Wall of Trench
STA West
00
10
20
30
40
50
60
STA West
70
Soil A Horizon Clayey Silt ML, dark brown, medium stiff, moist, major herb roots to 12 inches
TW
- Line ->
Weathered Sandstone SS brown, fine grained, slightly to moderately weathered, massive, moderately fractured, weak (Norwood Formation)
Weathered Claystone CS light olive, slightly to moderately weathered, very thinly bedded, moderately fractured, weak (Norwood Formation)
Weathered Sandstone SS light brown, slightly to moderately weathered, thinly bedded, moderately fractured, weak (Norwood Formation)
5
0
5
Scale In Feet
Sample Locations
(see geotechnical report for sample data)
TW Thin Wall Sample
B Bulk Sample
FIGURE 8
LOG OF TRENCH 53
STA 00 TO 72 WEST
GSH
ANALYSIS BY: GCS | CHECKED BY AH | DATE 5/26/16

Exhibit A



Summit at Ski Lake Phase 13
Job No. 2077-01N-16
South Wall of Trench
STA West 00
10
20
30
40
50
60
STA West 70
Weathered Siltstone ST light olive, slightly to moderately weathered, thinly bedded, moderately fractured, hard (Norwood Formation)
Soil A Horizon Clayey Silt ML, dark brown, medium stiff, moist, major herb roots to 8 inches
Weathered Siltstone ST light olive, slightly to moderately weathered, thinly bedded, moderately fractured, hard (Norwood Formation)
- Line >
Weathered Sandstone SS brown, fine grained, slightly to moderately weathered, moderately bedded, moderately fractured, weak (Norwood Formation)
Weathered Sandstone SS brown, fine to medium grained, slightly to moderately weathered, moderately bedded, moderately fractured, weak (Norwood Formation)
Weathered Siltstone ST light olive, slightly to moderately weathered, massive, moderately fractured, hard (Norwood Formation)
Trackhoe Refusal
STA West 100
80
90
STA West 100
Soil A Horizon Clayey Silt ML, dark brown, medium stiff, moist, major herb roots to 8 inches
Weathered Sandstone SS brown, fine grained, slightly to moderately weathered, massive, moderately fractured, weak (Norwood Formation)
- Line >
Weathered Siltstone ST light olive, slightly to moderately weathered, thinly bedded, moderately fractured, hard (Norwood Formation)
5
0
5
Scale In Feet
Sample Locations
(see geotechnical report for sample data)
B Thin Wall Sample
TW Bulk Sample
FIGURE 9
LOG OF TRENCH 54
STA 00 TO 100 WEST
GSH
ANALYSIS BY: GCS | CHECKED BY AH | DATE 5/26/16

Summit at Ski Lake Phase 13
Job No. 2077-01N-16

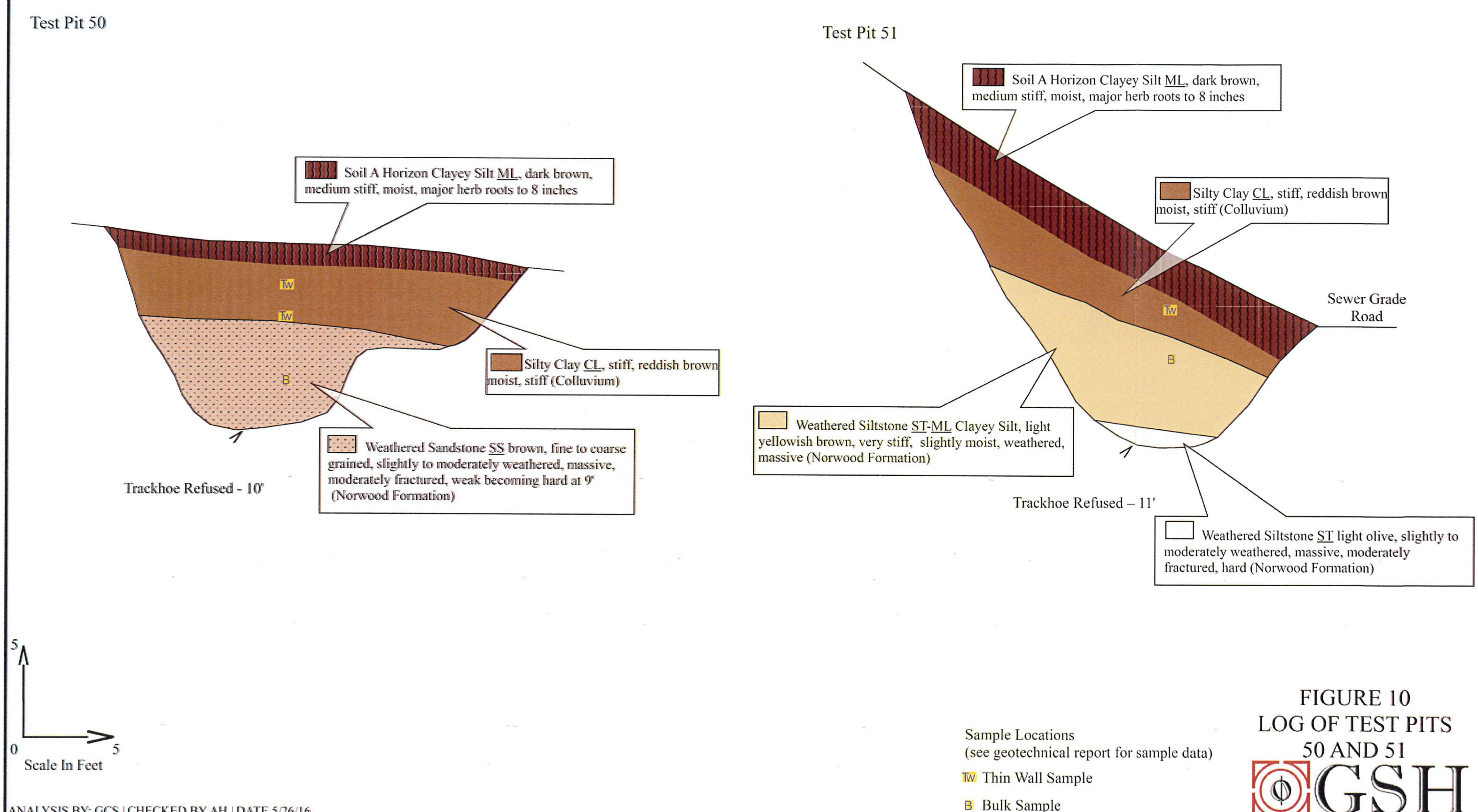
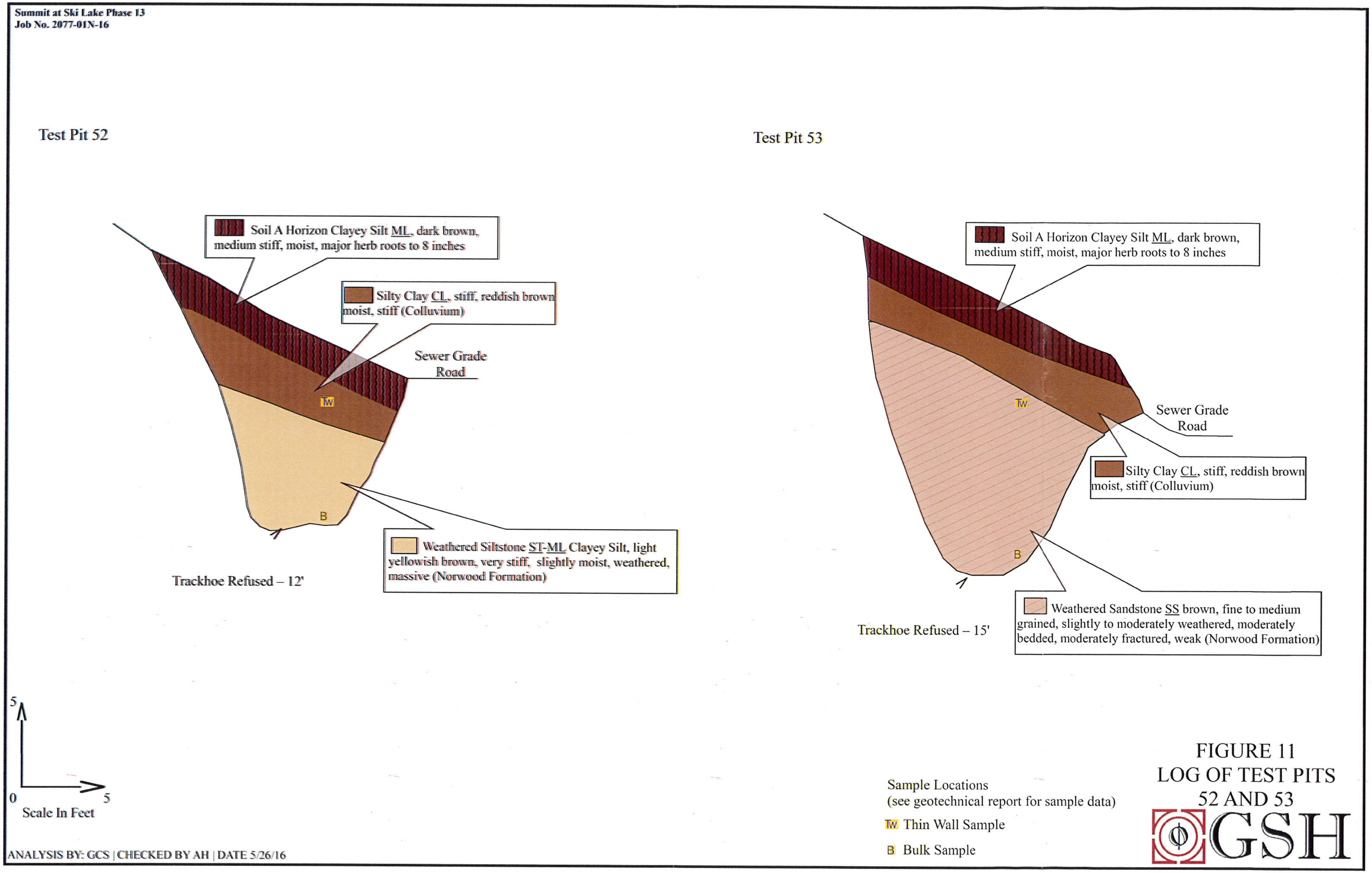


FIGURE 10
LOG OF TEST PITS
50 AND 51

GSH

ANALYSIS BY: GCS | CHECKED BY AH | DATE 5/26/16

Exhibit A



Summit at Ski Lake Phase 13
Job No. 2077-01N-16
Test Pit 52
Soil A Horizon Clayey Silt ML, dark brown, medium stiff, moist, major herb roots to 8 inches
Silty Clay CL, stiff, reddish brown moist, stiff (Colluvium)
Sewer Grade Road
TW
B
Weathered Siltstone ST-ML Clayey Silt, light yellowish brown, very stiff, slightly moist, weathered, massive (Norwood Formation)
Trackhoe Refused – 12'
Test Pit 53
Soil A Horizon Clayey Silt ML, dark brown, medium stiff, moist, major herb roots to 8 inches
Sewer Grade Road
Silty Clay CL, stiff, reddish brown moist, stiff (Colluvium)
Weathered Sandstone SS brown, fine to medium grained, slightly to moderately weathered, moderately bedded, moderately fractured, weak (Norwood Formation)
Trackhoe Refused – 15'
5
0
5
Scale In Feet
Sample Locations
(see geotechnical report for sample data)
TW Thin Wall Sample
B Bulk Sample
FIGURE 11
LOG OF TEST PITS
52 AND 53
GSH
ANALYSIS BY: GCS | CHECKED BY AH | DATE 5/26/16

GSH BORING LOG

Page: 1 of 1

BORING: B-1

CLIENT: Valley Enterprise Investment Company | PROJECT NUMBER: 2077-01N-16

PROJECT: Summit at Ski Lake No. 13 | DATE STARTED: 4/14/16 | DATE FINISHED: 4/14/16

LOCATION: East Clairetina Court, Huntsville, Utah | GSH FIELD REP.: AA

DRILLING METHOD/EQUIPMENT: 3-3/4" ID Hollow-Stem Auger | HAMMER: Automatic | WEIGHT: 140 lbs | DROP: 30"

GROUNDWATER DEPTH: Not Encountered (4/14/16) | ELEVATION: ---

WATER LEVEL	USCS	DESCRIPTION	DEPTH (FT.)	BLOW COUNT	SAMPLE SYMBOL	MOISTURE (%)	DRY DENSITY (PCF)	% PASSSING 200	LIQUID LIMIT (%)	PLASTICITY INDEX	REMARKS
		Ground Surface	0								
	CL FILL	SILTY CLAY, FILL brown									
	SM/ BR	WEATHERED/FRACTURED SANDSTONE BEDROCK/SILY SAND reddish-brown									dry very dense
			5	50+	⧗						
				50+	⧗	15	102	34			
		End of Exploration at 9.0' No groundwater encountered at time of drilling Installed 1.25" diameter slotted PVC pipe to 7.5'	10								
			15								
			20								
			25								

See Subsurface Conditions section in the report for additional information.

FIGURE 12

Exhibit A

GSH	BORING LOG Page: 1 of 2	BORING: B-2

CLIENT: Valley Enterprise Investment Company	PROJECT NUMBER: 2077-01N-16
PROJECT: Summit at Ski Lake No. 13	DATE STARTED: 4/14/16 DATE FINISHED: 4/14/16
LOCATION: East Clairetina Court, Huntsville, Utah	GSH FIELD REP.: AA
DRILLING METHOD/EQUIPMENT: 3-3/4" ID Hollow-Stem Auger	HAMMER: Automatic WEIGHT: 140 lbs DROP: 30"
GROUNDWATER DEPTH: Not Encountered (4/14/16)	ELEVATION: ---

WATER LEVEL	USCS	DESCRIPTION	DEPTH (FT.)	BLOW COUNT	SAMPLE SYMBOL	MOISTURE (%)	DRY DENSITY (PCF)	% PASSSING 200	LIQUID LIMIT (%)	PLASTICITY INDEX	REMARKS
		Ground Surface	0								
	CL FILL	SILTY CLAY, FILL with trace fine to coarse sand; light brown									
	CL	SILTY CLAY with trace fine sand; gray with oxidation									slightly moist hard
				95+							
	SM	SILTY FINE SAND/FINE SANDY SILT highly weathered siltstone/sandstone; light brown	5	82		19	105				slightly moist very dense
	ML	SILT with trace fine sand; gray with oxidation									slightly moist hard
				80+		35	72		46	13	
	SM/ BR	SILTY FINE SAND/WEATHER SANDSTONE BEDROCK reddish-brown									dry very dense
			10	98+							
				86+		17		19			
			15	50+							
				50+							
			20	82+		27	93	16			
				50+							
			25								

See Subsurface Conditions section in the report for additional information.

FIGURE 13

Exhibit A

GSH

BORING LOG

Page: 2 of 2

BORING: B-2

CLIENT: Valley Enterprise Investment Company — PROJECT NUMBER: 2077-01N-16

PROJECT: Summit at Ski Lake No. 13 — DATE STARTED: 4/14/16 — DATE FINISHED: 4/14/16

WATER LEVEL	USCS	DESCRIPTION	DEPTH (FT.)	BLOW COUNT	SAMPLE SYMBOL	MOISTURE (%)	DRY DENSITY (PCF)	% PASSSING 200	LIQUID LIMIT (%)	PLASTICITY INDEX	REMARKS
			25	100+							slightly moist
				50+							
		End of Exploration at 29.0' No groundwater encountered at time of drilling Installed 1.25" diameter slotted PVC pipe to 27.5'	30								
			35								
			40								
			45								
			50								

See Subsurface Conditions section in the report for additional information.

FIGURE 13
(continued)

Exhibit A

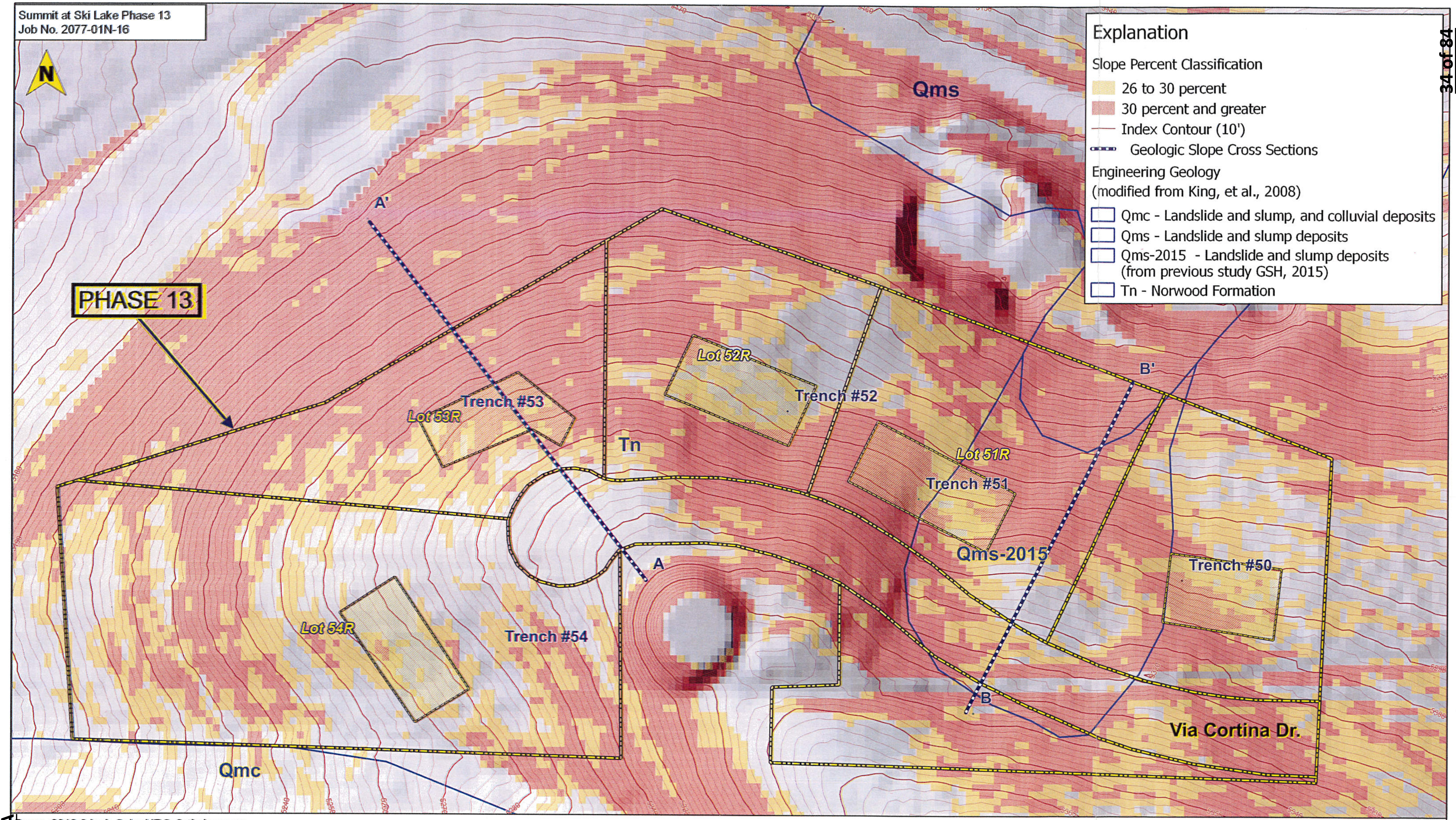
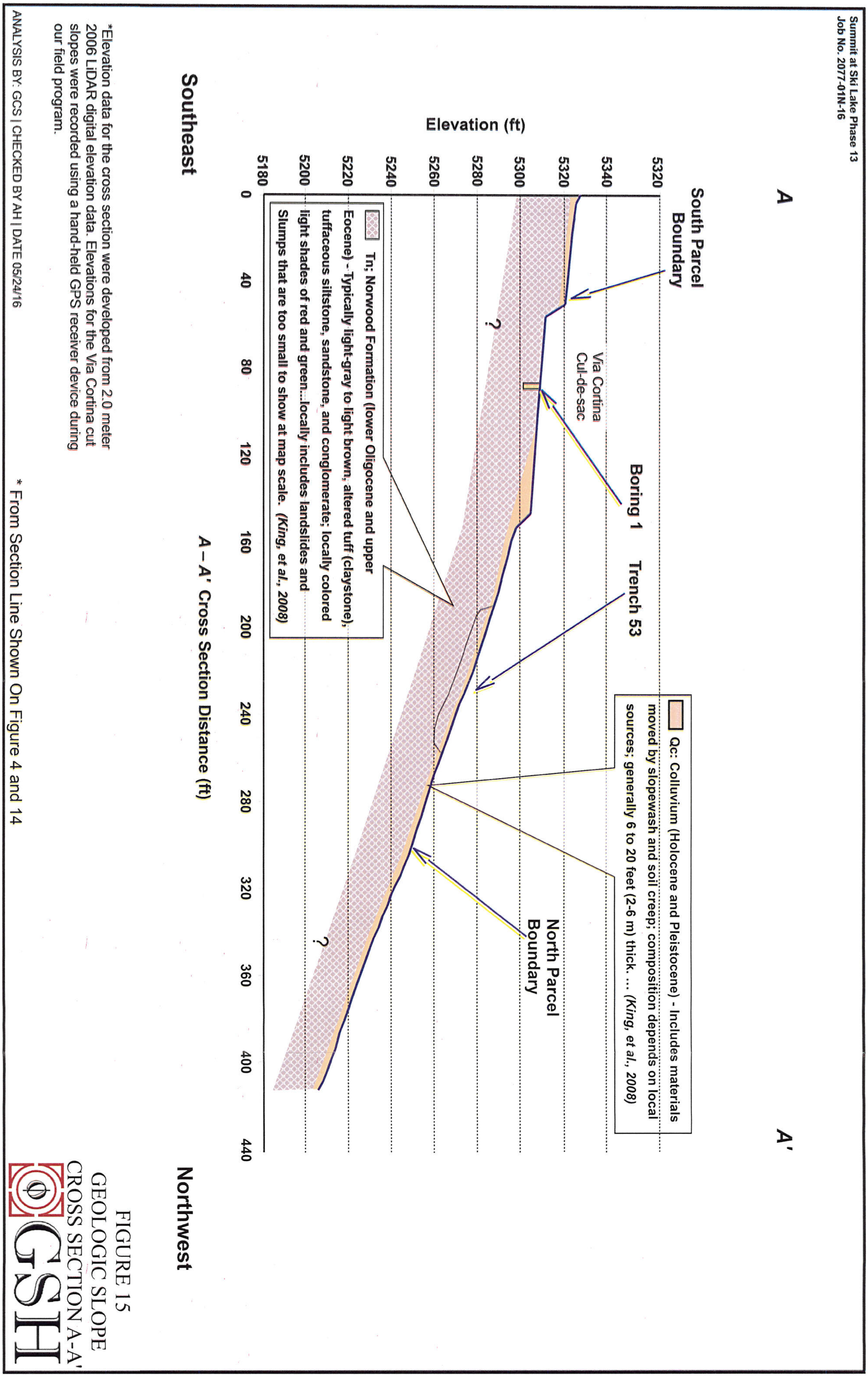


FIGURE 14
LiDAR-SLOPE ANALYSIS

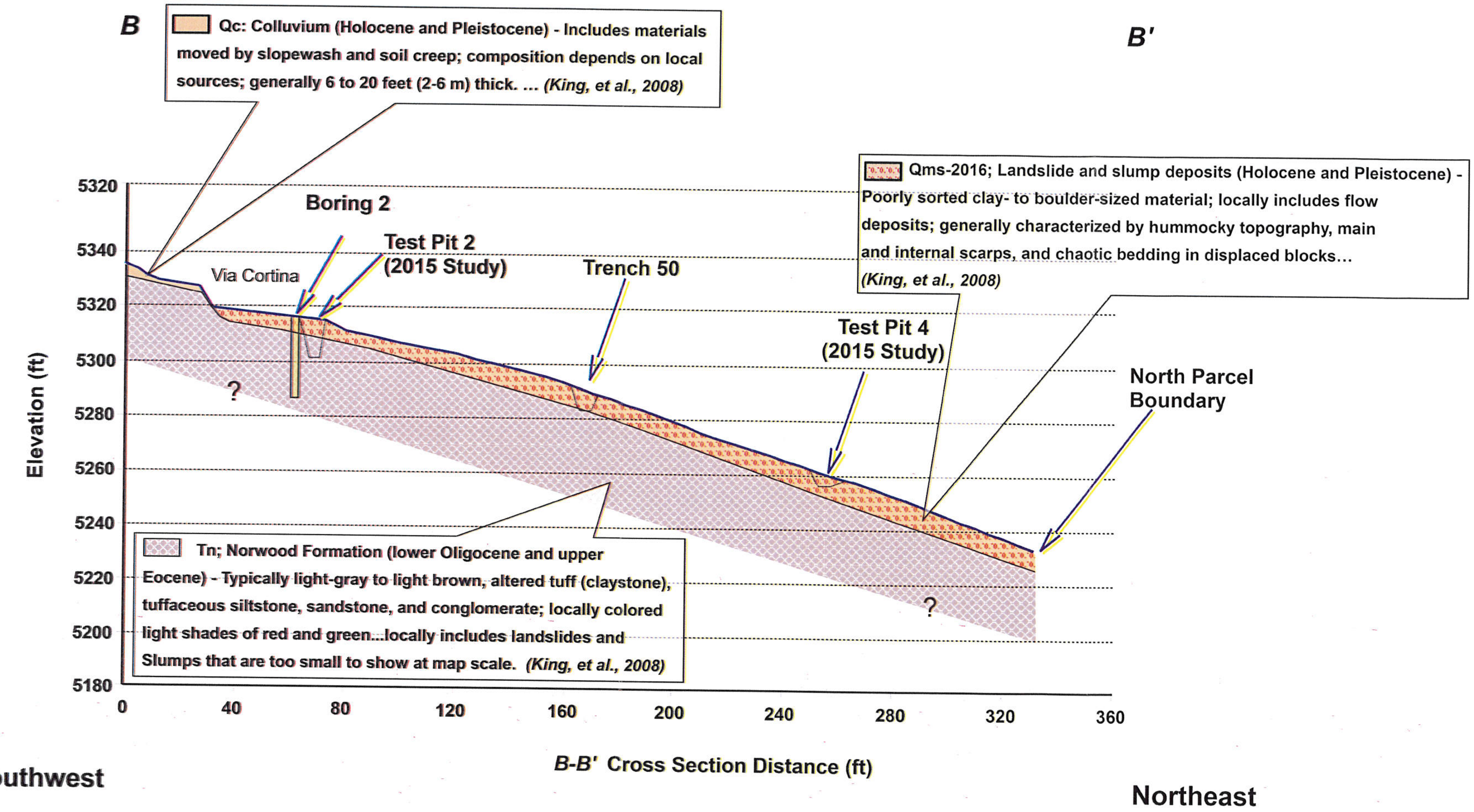
GSH

Exhibit A



Summit at Ski Lake Phase 13
Job No. 2077-01N-16
A
A'
Elevation (ft)
5320
5340
5320
5300
5280
5260
5240
5220
5200
5180
0
40
80
120
160
200
240
280
320
360
400
440
South Parcel Boundary
Via Cortina Cul-de-sac
Boring 1
Trench 53
North Parcel Boundary
?
?
Qc: Colluvium (Holocene and Pleistocene) - Includes materials moved by slopewash and soil creep; composition depends on local sources; generally 6 to 20 feet (2-6 m) thick. ... (King, et al., 2008)
Tn; Norwood Formation (lower Oligocene and upper Eocene) - Typically light-gray to light brown, altered tuff (claystone), tuffaceous siltstone, sandstone, and conglomerate; locally colored light shades of red and green...locally includes landslides and Slumps that are too small to show at map scale. (King, et al., 2008)
Southeast
Northwest
A – A' Cross Section Distance (ft)
*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.
ANALYSIS BY: GCS | CHECKED BY AH | DATE 05/24/16
* From Section Line Shown On Figure 4 and 14
FIGURE 15
GEOLOGIC SLOPE
CROSS SECTION A-A'
GSH

Exhibit A



*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.

ANALYSIS BY: GCS | CHECKED BY AH | DATE 05/24/16

* From Section Line Shown On Figure 4 and 14

FIGURE 16
GEOLOGIC SLOPE
CROSS SECTION *B-B'*

GSH

REPORT
GEOTECHNICAL STUDY
LOTS 50R TO 54R SUMMIT AT SKI LAKE NO. 13
EAST CLAIRETINA COURT
HUNTSVILLE, UTAH

Submitted To:

Valley Enterprise Investment Company
Attention: Mr. Ray Bowden
5393 East 3850 North
Eden, Utah

Submitted By:

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

June 3, 2016

Job No. 2077-01N-16

June 3, 2016
Job No. 2077-01N-16

Mr. Ray Bowden
Valley Enterprise Investment Company
5393 East 3850 North
Eden, Utah 84310

Re: Report
Geotechnical Study
Lots 50R to 54R, Summit at Ski Lake No. 13
East Claitetina Court
Huntsville, Utah
(41.2429° N; 111.7894° W)

1. INTRODUCTION

1.1 GENERAL

This report presents the results of our geotechnical study performed for the proposed structures on lots 50R to 54R of the Summit at Ski Lake No. 13 located on East Clairetina Court in Weber County, near 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 borings/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. Ray Bowden of Valley Enterprise Investment Company 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.

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Ogden, Utah 84401
Tel: 801.393.2012

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.

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

1. A field program consisting of the drilling/excavating, logging, and sampling of 2 borings, 4 test pits and 5 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. 16-0240N dated February 16, 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 borings/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 single-family residences for lots 50R to 54R of the Summit at Ski Lake No. 13 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 25 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.

To facilitate site grading for the proposed homes, engineered retaining walls will likely be required. Retaining walls must be engineered based on the site specific site grading plans. At the time of this report, site specific grading plans are not available; however preliminary grading plans were produced for the site by Great Basin Engineering.

3. INVESTIGATIONS

3.1 FIELD PROGRAM

In order to define and evaluate the subsurface soil and groundwater conditions at the site, 2 borings, 4 test pits, and 5 trenches were explored to depths of about 5.0 to 29.0 feet below existing grades. The borings were drilled using a truck-mounted drill rig equipped with hollow-stem augers and mud-rotary, and the test pits were excavated using a 20-ton track-mounted excavator. Refusal with the excavator was encountered in the test pit and trench excavations. Locations of the borings and test pits/trenches 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 drilling and excavating operations, a log of the subsurface conditions encountered was maintained. In addition, relatively undisturbed and small disturbed samples of the typical soils encountered were obtained for subsequent laboratory testing and examination. 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 representations of the subsurface conditions encountered are presented on Figures 3A and 3B, Boring Logs, and Figures 4A through 4J, Test Pit Logs. Soils were classified in accordance with the nomenclature described on Figure 5, Key to Boring Log (USCS), and Figure 6, Key to Test Pit Log (USCS).

A 3.0-inch outside diameter, 2.42-inch inside diameter drive sampler (Dames & Moore) and a 2.0-inch outside diameter, 1.38-inch inside diameter drive sampler (SPT) were utilized in the subsurface soil sampling at select locations. The blow counts recorded on the boring logs were those required to drive the sampler 12 inches with a 140-pound hammer dropping 30 inches.

A 2.42-inch inside diameter thin-wall drive sampler was utilized in the subsurface sampling at the site.

Following completion of drilling operations, 1.25-inch diameter slotted PVC pipe was installed in borings B-1 and B-2 in order to provide a means of monitoring potential groundwater fluctuations. The borings were backfilled with auger cuttings. 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, the backfill soils must be considered as non-engineered and settlement of the backfill with time is likely to occur.

GSH

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 boring logs, Figures 3A and 3B, and test pit logs, Figures 4A through 4J.

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 below:

Boring/Test Pit/Trench No.	Depth (feet)	Liquid Limit (percent)	Plastic Limit (percent)	Plasticity Index (percent)	Soil Classification
B-2	7.5	46	33	13	ML
TP-50	2.0	31	21	10	CL
TP-50	4.0	37	29	8	ML
TP-52	13.0	56	41	15	MH/BR
TR-50A	6.0	33	25	8	ML
TR-50B	4.0	37	29	8	ML
TR-51	5.0	36	27	9	ML

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:

Boring/Test Pit/Trench No.	Depth (feet)	Moisture Content Percent	Percent Passing No. 200 Sieve	Soil Classification
B-1	7.5	15.0	34.1	SM/BR
B-2	12.5	16.7	18.5	SM/BR
B-2	20.0	26.9	16.1	SM/BR
TR-53	2.0	28.1	48.3	SM/BR

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 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 400 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 below:

Test Pit/ Trench/ Boring No.	Depth (feet)	Soil Type	In-Situ Moisture Content (percent)	Dry Density (pcf)	Internal Friction Angle (degrees)	Apparent Cohesion (psf)
B-2	7.5	ML	35	72	36	320
TP-50	4.0	ML	27	81	31	125
TP-53	5.0	CL	---	---	33	50

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:

Test Pit/ Trench/ Boring No.	Depth (feet)	Soil Type	In-Situ Moisture Content (percent)	Dry Density (pcf)	Internal Friction Angle (degrees)	Apparent Cohesion (psf)
TP-50	4.0	ML	27	81	16	555
TP-53	5.0	CL	---	---	24	25

4. SITE CONDITIONS

4.1 GEOLOGIC SETTING

A geologic study[1] dated June 3, 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 site consists of 5 residential lots within the existing Summit at Ski Lakes development located on at the end of Via Cortina street on Clairetina Court in Huntsville, Utah. The topography of the site slopes downward to the north/northwest with an overall change in elevation of about 100 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 and west by residential development, and on the south and east by undeveloped property.

4.3 SUBSURFACE SOIL

Subsurface conditions encountered at the boring, test pit, and trench locations varied slightly across the site. Fill material consisting of silty clay was encountered at both of the boring locations extending to about 1.5 to 3.0 feet below existing site grades. Topsoil and disturbed soils were observed in the upper 3 to 12 inches at the test pit and trench locations. In a portion of Trench 50A, Trench 50B, and Trench 51 and boring B-2, mass movement soil deposits were encountered below the topsoil and disturbed soils extending to about 7.5 to 10.0 feet below surrounding site grades. The mass movement deposits were comprised of a mixture of silty

[1] "Report, Geological Study, Five Residential Development Lots, Lots 50R, 51R, 52R, 53R, and 54R, The Summit at Ski Lake Phase 13, Weber County, Utah," GSH Geotechnical, Inc., GSH Job No. 2077-01N-16, June 3, 2016.

sand, clayey silt, silty clay, and degraded/weathered sandstone/siltstone. Natural soils were observed beneath the mass movement soils, fill material, and topsoil/disturbed soils to the full depth penetrated, about 5.0 to 29.0 feet below surrounding grades and consisted of silty clay, clayey silt, fine to coarse sand with varying amounts of silt, weathered bedrock (weathered sandstone/claystone/siltstone), and occasional mixture of these soils.

The natural sand soils encountered were medium dense to very dense, slightly moist to moist, light brown to reddish-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 black 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 and 3B, Boring Log, and Figures 4A through 4J, 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 borings, 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 structures on Lot 52R, 53R, and 54R upon conventional spread and/or continuous wall foundations established upon a minimum of 18 inches of granular structural fill extending to suitable natural soils.

The proposed structures on Lot 50R and 51R may be supported upon cast-in-place drilled piers extending a minimum of 10 feet into bedrock (about 25 feet below existing grades) following removal of the mass movement deposits from beneath and upslope of the building pad locations.

In addition, a series of grouted tieback anchors approximately 160 feet long will be required extending beneath the home and below the uphill ground surface in order to provide adequate stability for the home. The lengths given for the pier and grouted tieback anchors are preliminary. Final design of the grouted tieback anchor system and pier systems must be provided by the installer. 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 shallow bedrock at the site, the presence of mass movement deposit soils in the proposed home locations on Lots 50R and 51R, and maintaining stability of the slope at the rear of the property.

The location of the homes on lots 50R and 51R 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 upslope of the home and near the head of the remaining mass movement deposit soils below the home 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. Additionally, mass movement deposits must be removed upslope of the building pads. In situ, non-engineered fills and mass movement deposit soils (downslope of the building pad) 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 below:

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

2 American Society for Testing and Materials
3 American Association of State Highway and Transportation Officials

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)
Silt/Clay (Colluvium and Weathered Bedrock)	26	200	120
Mass Movement	16	200	120
Structural Fill	34	50	125
Concrete	0	288,000	150

For the seismic analysis, a peak horizontal ground acceleration of 0.238*g* using IBC 2012 guidelines and adjusted for Site Class effects (for Site Class C soils) was obtained for site (grid) locations of 41.2429 degrees latitude (north) and 111.7894 degrees longitude (west). To model sustained accelerations at the site, one-half of this value is typically employed. Accordingly, a value of 0.12g was used as the pseudostatic coefficient for the stability analysis.

5.3.2 Redi-Rock Wall Design

Using these input parameters, the internal (block-to-block) stability of the wall was evaluated considering sliding, overturning, and bearing capacity to achieve respective minimum factors of safety of 1.5, 2.0, and 3.0 for static conditions and 1.1, 1.1, and 1.5 for seismic conditions. The results of this analysis (see attached Figure 12) indicate that a maximum exposed wall height of 9 feet can be achieved for 41-inch deep Redi-Rock blocks with the top row being 28-inch deep blocks.

5.3.3 Stability Analyses

We evaluated the global stability of the existing slopes 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 configurations based on cross-sections provided in the referenced geologic study (see geological study in appendix for cross-section information and location):

- Cross-section A-A' consisting of a relatively flat roadway area grading downward to the proposed home location and slope at grades ranging from about 2.5H:1V (Horizontal to Vertical) to 3H:1V (Horizontal:Vertical). A relatively flat building pad comprised of up to 8 feet of structural fill and a Redi-Rock retaining wall was included in the model. 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. In addition, a phreatic surface was conservatively included below the extent of our drilling depth. This cross-section is representative of lots 52R, 53R and 54R.

- Cross-section B-B' consisting of a relatively flat roadway area grading downward to the proposed home location and slope at grades ranging from about 2H:1V (Horizontal to Vertical) to 4.5H: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. In addition, a phreatic surface was included near the base of the landslide deposit to account for potential water from seasonal runoff and snowmelt. This cross-section is representative of lots 50R and 51R.

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 slope configuration A-A' will meet both these requirements provided our recommendations are followed (see Figures 7 and 8).

The results of our analyses indicate that slope configuration B-B' combined with the home loading will not meet these requirements. Based on our preliminary analyses, to improve the stability of the slope and reduce the potential for damage to the structure, a series of tieback anchors roughly 160 feet long and angled at about 15 degrees from horizontal will be required below the home spaced about 10 feet apart. The grouted tieback anchors must extend laterally at a 1:1 (in the horizontal plane) away from the foundation. The results of our preliminary analyses indicate that the minimum static factor of safety will be met provided our recommendations are followed; however final design of the grouted tieback anchors based on planned grading and home location must be completed by the installer prior to construction. The slope stability data is included as Figures 9 through 11, 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.

5.3.4 Preliminary Redi-Rock Wall Recommendations

Based on the results of our preliminary analyses, Redi-Rock block retaining walls below the building pads on lots 52R, 53R, and 54R will be stable if constructed as follows (also see Figure 13, attached). Retaining walls on lots 50R and 51R will be more difficult to construct and will require planning and design beyond the scope of this study. Again, 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.

- The Redi-Rock block walls may be constructed up to a maximum total height of 8 feet using 41-inch deep blocks with the top row being 28-inch deep blocks. The walls must be embedded a minimum 6 inches below lowest adjacent grade.

- The bottom row of blocks must be placed on a minimum 12 inches of crushed ¾-inch to 1.5-inch size gravel structural fill material. This material shall be compacted until firm.

- Each row of blocks shall be set back a minimum 1½ inch from the underlying row of blocks, per the manufacturer's recommendation.

- Backfill materials behind the blocks may consist of imported soils having a maximum particle size of 3 inches, 70 percent or more passing the ¾-inch sieve and less than 30 percent fines (percent passing the No. 200 sieve). Backfill shall be moisture conditioned to within 2 percent of optimum and compacted to a minimum 95 percent of the maximum dry density per ASTM D1557 (Modified Proctor).

- Drainage behind each wall tier must be included, as shown on Figure 4. The drain shall consist of a perforated 4-inch minimum diameter pipe wrapped in fabric and placed at the bottom and behind the lowest row of blocks. The pipe shall daylight at one or both ends of the wall 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, shall be placed around the drain pipe. A fabric, such as Mirafi 140N or equivalent, must be placed between the clean gravel and the adjacent soils. A zone of clean gravel and fabric at least 12 inches wide shall also extend above the drain, upward and immediately behind the blocks to about 2 feet below the top of the wall, as shown on Figure 13.

- Irrigation lines must not be placed within the backfill or directly on top of the walls. Surface drainage at the bottom and top of the walls shall also be directed away from the walls.

5.3.5 Site Observations

A geotechnical engineer from GSH must observe construction of the block wall at the following times:

- After the excavation is complete for the lowest row of blocks (prior to the placement of any blocks, gravel, or fabric);

- After the bottom row of blocks, drain pipe, clean gravel, and fabric have been placed (prior to placing more than 3 rows of blocks);

- At the approximate midpoint of the block wall construction; and

- Upon completion of the block wall construction. GSH will then provide an as-constructed letter indicating our observations of the wall construction.

5.4 SPREAD AND CONTINUOUS WALL FOUNDATIONS

5.4.1 Design Data

The results of our analyses indicate that the proposed structures on lot 52R, 53R and 54R may be supported upon conventional spread and/or continuous wall foundations established upon suitable natural soils or granular structural fill extending to suitable natural soils. For design, with respect to the proposed construction and anticipated loading given in Section 2.0, Proposed Construction, the following parameters are recommended.

Minimum Recommended Depth of Embedment for Frost Protection	- 30 inches
Minimum Recommended Depth of Embedment for Non-frost Conditions	- 15 inches
Recommended Minimum Width for Continuous Wall Footings	- 18 inches
Minimum Recommended Width for Isolated Spread Footings	- 24 inches
Recommended Net Bearing Pressure for Real Load Conditions	- 1,500 pounds per square foot
Bearing Pressure Increase for Seismic Loading	- 50 percent

The term “net bearing pressure” refers to the pressure imposed by the portion of the structure located above lowest adjacent final grade. Therefore, the weight of the footing and backfill to lowest adjacent final grade need not be considered. Real loads are defined as the total of all dead plus frequently applied live loads. Total load includes all dead and live loads, including seismic and wind.

5.4.2 Installation

Practical refusal of excavating equipment (20-ton trackhoe with 36 inch bucket) was encountered at shallow depths within the test pit and trench excavations. Shallow bedrock at the site will require large excavating equipment, chipping, and possible light blasting to penetrate the bedrock for home and utility excavations. Under no circumstances shall the footings be established upon non-engineered fills, loose or disturbed soils, topsoil, sod, rubbish, construction debris, other deleterious materials, frozen soils, or within ponded water. If unsuitable soils are encountered, they must be completely removed and replaced with compacted structural fill.

The width of structural replacement fill below footings should be equal to the width of the footing plus one foot for each foot of fill thickness. For instance if the footing width is 2 feet and the structural fill depth beneath the footing is 1.5 feet, the fill replacement width should be 3.5 feet, centered beneath the footing.

5.4.3 Settlements

Maximum settlements of foundations designed and installed in accordance with recommendations presented herein and supporting maximum anticipated loads as discussed in Section 2, Proposed Construction, are anticipated to be 1 inch or less.

Approximately 40 percent of the quoted settlement should occur during construction.

5.5 DRILLED PIER FOUNDATIONS

5.5.1 Design Parameters

To minimize the impact of the proposed homes on Lot 50R and 51R 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. Given the current site grades, piers are likely to extend a minimum of 25 feet below current site grades, however changes in site grading will impact the required pier lengths. Final design of the drilled pier system must be provided by the installer prior to construction.

As indicated previously, all mass movement deposits must be removed from below and upslope of the building pads prior to installation of the drilled pier system.

A grouted tie-back system in addition to the drilled pier foundations is required below the home locations on Lots 50R and 51R, as discussed above.

5.5.2 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.

5.5.3 Settlements

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

5.5.4 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.

5.6 LATERAL RESISTANCE

For homes on lots 52R, 53R, and 54R 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.

For homes on Lots 50R and 51R, lateral loads imposed upon foundations due to wind or seismic forces may be resisted by the pier system.

5.7 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 table on the following page.

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

5.8 FLOOR SLABS

For lots 52R, 53R and 54R, floor slabs may be established upon a minimum of 18 inches of structural fill extending to suitable natural soils. For lots 50R and 51R, floor slabs must be supported structurally on a pier and grade beam foundation. 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 150 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.

5.9 SUBDRAINS

5.9.1 General

Groundwater was not encountered at the site, however we recommend that the perimeter foundation subdrains and a cutoff drain upslope of all the home and near the head of the mass movement deposit soils on lots 50R and 51R be installed as indicated below.

5.9.2 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.

5.9.3 Cutoff Drain

To reduce potential infiltration of surface water and groundwater into the subsurface soils at the site, a cutoff drain should be installed upslope of the home and near the head of the mass movement deposit soils below the home. 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 competent 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.

5.10 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.

5.11 GEOSEISMIC SETTING

5.11.1 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.

5.11.2 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 7.2 miles west of the site.

5.11.3 Soil Class

For dynamic structural analysis, the Site Class C – Very Dense Soil and Soft Rock Profile as defined in Chapter 20 of ASCE 7 (per Section 1613.3.2, Site Class Definitions, of IBC 2012) can be utilized.

5.11.4 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 -111.7894 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 C [adjusted for site class effects] (% g)	Design Values (% g)
Peak Ground Acceleration	33.6	F_a = 1.064	35.7	23.8
0.2 Seconds (Short Period Acceleration)	S_S = 83.9	F_a = 1.064	S_{MS} = 89.3	S_{DS} = 59.5
1.0 Second (Long Period Acceleration)	S_1 = 28.2	F_v = 1.518	S_{M1} = 42.8	S_{D1} = 28.5

5.11.5 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.

5.12 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.

5.13 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.

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AMH/MSH:mmh

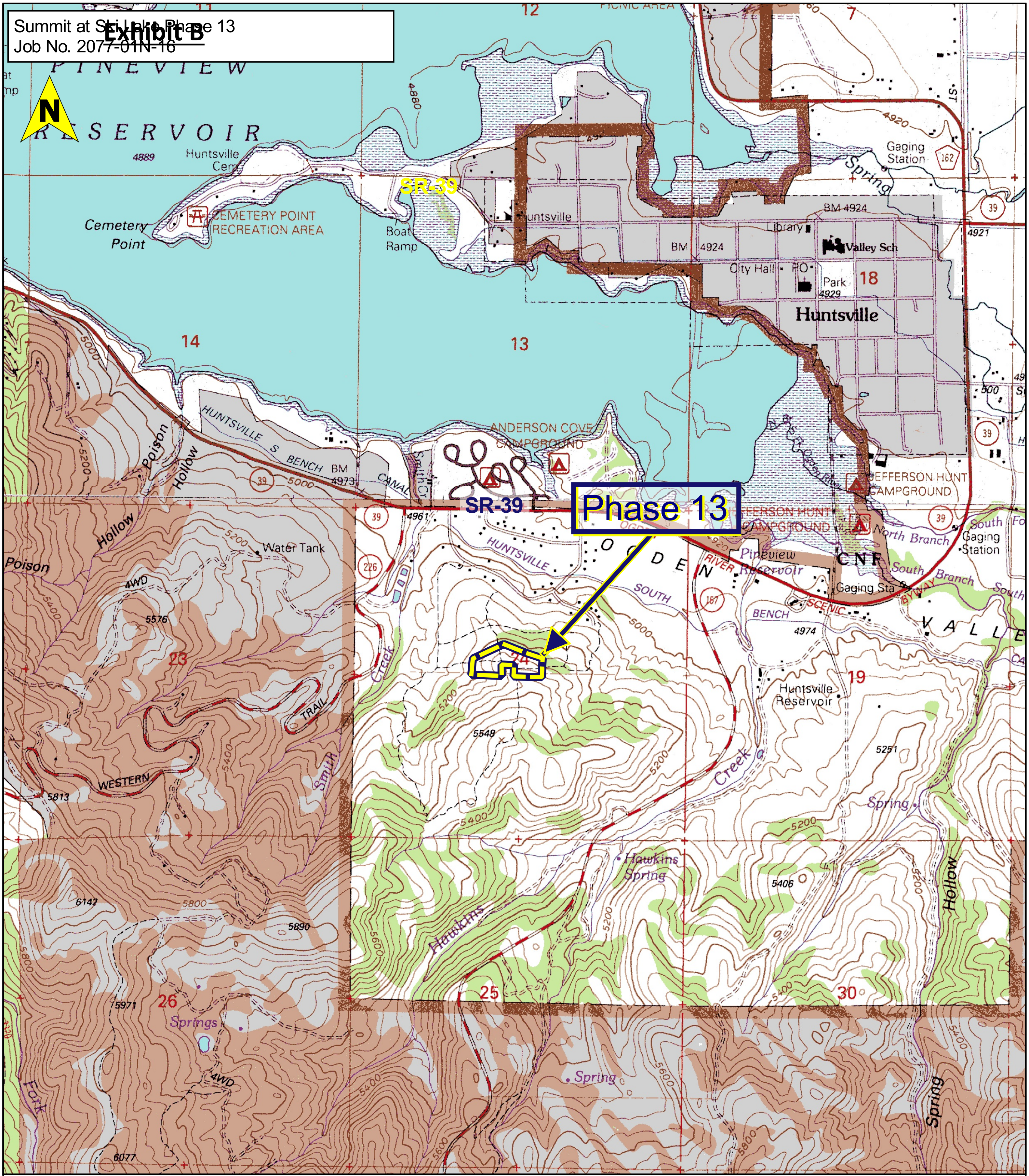
Encl.	Figure	1,	Vicinity Map
	Figure	2,	Site Plan
	Figures	3A	and 3B, Boring Logs
	Figures	4A	through 4J, Test Pit Logs
	Figure	5	Key to Boring Log (USCS)
	Figure	6,	Key to Test Pit Log (USCS)
	Figures	7	through 11, Stability Results
	Figure	12,	Redi-Rock Wall Stability Evaluation
	Figure	13,	Redi-Rock Wall Detail

Appendix

Addressee (Email)

Exhibit B

Summit at Ski Lake Phase 13
Job No. 2077-01N-16



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

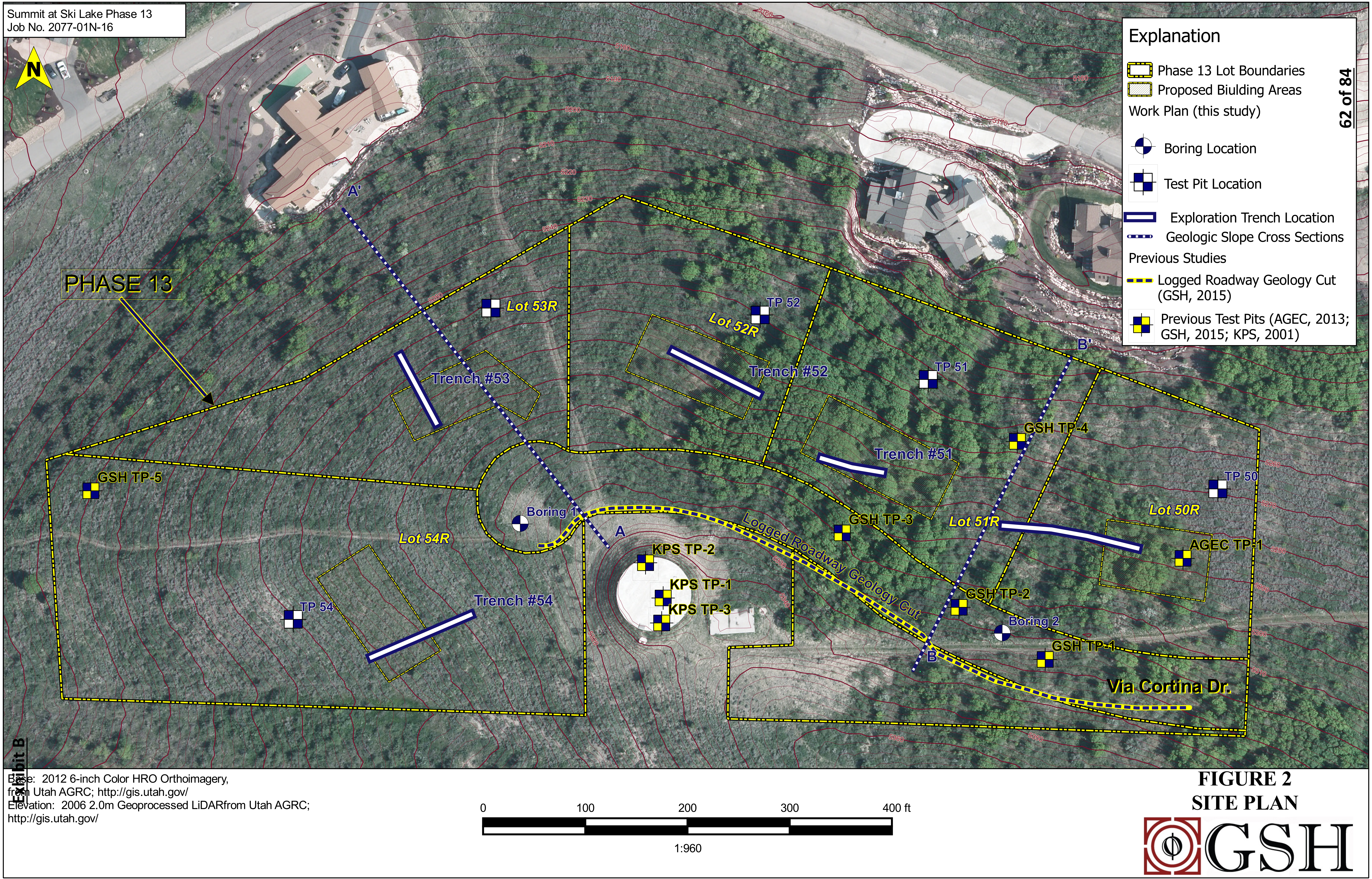


Exhibit B

Base: 2012 6-inch Color HRO Orthoimagery, from Utah AGRC; http://gis.utah.gov/
Elevation: 2006 2.0m Geoprocessed LiDARfrom Utah AGRC; http://gis.utah.gov/

FIGURE 2
SITE PLAN

GSH

Exhibit B

BORING LOG

Page: 1 of 1

BORING: B-1

CLIENT: Valley Enterprise Investment Company	PROJECT NUMBER: 2077-01N-16
PROJECT: Summit at Ski Lake No. 13	DATE STARTED: 4/14/16 DATE FINISHED: 4/14/16
LOCATION: East Clairetina Court, Huntsville, Utah	GSH FIELD REP.: AA
DRILLING METHOD/EQUIPMENT: 3-3/4" ID Hollow-Stem Auger	HAMMER: Automatic WEIGHT: 140 lbs DROP: 30"
GROUNDWATER DEPTH: Not Encountered (4/14/16)	ELEVATION: ---

WATER LEVEL	USCS	DESCRIPTION	DEPTH (FT.)	BLOW COUNT	SAMPLE SYMBOL	MOISTURE (%)	DRY DENSITY (PCF)	% PASSSING 200	LIQUID LIMIT (%)	PLASTICITY INDEX	REMARKS
		Ground Surface	0								
	CL FILL	SILTY CLAY, FILL brown									
	SM/ BR	WEATHERED/FRACTURED SANDSTONE BEDROCK/SILTY SAND reddish-brown									dry very dense
			5	50+	X						
				50+	X	15	102	34			
		End of Exploration at 9.0' No groundwater encountered at time of drilling Installed 1.25" diameter slotted PVC pipe to 7.5'	10								
			15								
			20								
			25								

See Subsurface Conditions section in the report for additional information.

FIGURE 3A

Exhibit B

GSH BORING LOG

Page: 1 of 2

BORING: B-2

CLIENT: Valley Enterprise Investment Company	PROJECT NUMBER: 2077-01N-16
PROJECT: Summit at Ski Lake No. 13	DATE STARTED: 4/14/16 DATE FINISHED: 4/14/16
LOCATION: East Clairetina Court, Huntsville, Utah	GSH FIELD REP.: AA
DRILLING METHOD/EQUIPMENT: 3-3/4" ID Hollow-Stem Auger	HAMMER: Automatic WEIGHT: 140 lbs DROP: 30"
GROUNDWATER DEPTH: Not Encountered (4/14/16)	ELEVATION: ---

WATER LEVEL	USCS	DESCRIPTION	DEPTH (FT.)	BLOW COUNT	SAMPLE SYMBOL	MOISTURE (%)	DRY DENSITY (PCF)	% PASSSING 200	LIQUID LIMIT (%)	PLASTICITY INDEX	REMARKS
		Ground Surface	0								
	CL FILL	SILTY CLAY, FILL with trace fine to coarse sand; light brown									
	CL	SILTY CLAY with trace fine sand; gray with oxidation									slightly moist hard
				95+	X						
	SM	SILTY FINE SAND/FINE SANDY SILT highly weathered siltstone/sandstone; light brown	5	82	X	19	105				slightly moist very dense
	ML	SILT with trace fine sand; gray with oxidation		80+	X	35	72		46	13	slightly moist hard
	SM/ BR	SILTY FINE SAND/WEATHER SANDSTONE BEDROCK reddish-brown									dry very dense
			10	98+	X						
				86+	X	17		19			
			15	50+	X						
				50+	X						
			20	82+	X	27	93	16			
				50+	X						
			25		X						

See Subsurface Conditions section in the report for additional information.

FIGURE 3B

Exhibit B

GSH

BORING LOG

Page: 2 of 2

BORING: B-2

CLIENT: Valley Enterprise Investment Company | PROJECT NUMBER: 2077-01N-16

PROJECT: Summit at Ski Lake No. 13 | DATE STARTED: 4/14/16 | DATE FINISHED: 4/14/16

WATER LEVEL	USCS	DESCRIPTION	DEPTH (FT.)	BLOW COUNT	SAMPLE SYMBOL	MOISTURE (%)	DRY DENSITY (PCF)	% PASSING 200	LIQUID LIMIT (%)	PLASTICITY INDEX	REMARKS
			25	100+							slightly moist
				50+							
		End of Exploration at 29.0' No groundwater encountered at time of drilling Installed 1.25" diameter slotted PVC pipe to 27.5'	30								
			35								
			40								
			45								
			50								

See Subsurface Conditions section in the report for additional information.

FIGURE 3B
(continued)

Exhibit B

TEST PIT LOG

Page: 1 of 1

TEST PIT: TP-50

CLIENT: Valley Enterprise Investment Company | PROJECT NUMBER: 2077-01N-16

PROJECT: Summit at Ski Lake No. 13 | DATE STARTED: 4/13/16 | DATE FINISHED: 4/13/16

LOCATION: East Clairetina Court, Huntsville, Utah | GSH FIELD REP.: AA

EXCAVATING METHOD/EQUIPMENT: KOMATSU - Trackhoe

GROUNDWATER DEPTH: Not Encountered (4/13/16) | 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 sand; major roots (topsoil) to 12"; brown to reddish-								slightly moist medium stiff
	ML	SILT brown								slightly moist medium stiff
	SM/BR	SILTY FINE TO MEDIUM SAND reddish-brown grades weathered sandstone bedrock	5							slightly moist medium dense
		End of Exploration at 10.0' due to excavator refusal 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 4A

TEST PIT LOG

Page: 1 of 1

TEST PIT: TP-52

CLIENT: Valley Enterprise Investment Company	PROJECT NUMBER: 2077-01N-16
PROJECT: Summit at Ski Lake No. 13	DATE STARTED: 4/12/16 DATE FINISHED: 4/12/16
LOCATION: East Clairetina Court, Huntsville, Utah	GSH FIELD REP.: AA
EXCAVATING METHOD/EQUIPMENT: KOMATSU - Trackhoe	
GROUNDWATER DEPTH: Not Encountered (4/12/16)	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 12"; brown to reddish-brown	0							moist medium stiff
	MH/ BR	WEATHERED SILTSTONE dark gray to black	5							
		End of Exploration at 12.5' No significant sidewall caving No groundwater encountered at time of excavation	12.5							

See Subsurface Conditions section in the report for additional information.

FIGURE 4B

GSH

TEST PIT LOG

Page: 1 of 1

TEST PIT: TP-53

CLIENT: Valley Enterprise Investment Company — PROJECT NUMBER: 2077-01N-16

PROJECT: Summit at Ski Lake No. 13 — DATE STARTED: 4/12/16 — DATE FINISHED: 4/12/16

LOCATION: East Clairetina Court, Huntsville, Utah — GSH FIELD REP.: AA

EXCAVATING METHOD/EQUIPMENT: KOMATSU - Trackhoe

GROUNDWATER DEPTH: Not Encountered (4/12/16) — 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 sand; major roots (topsoil) to 12"; brown to reddish-	0							moist medium stiff
			5	▮						
	SM/BR	WEATHERED SANDSTONE BEDROCK light brown								moist dense
			10							
			15	◢						
		End of Exploration at 15.5' No significant sidewall caving No groundwater encountered at time of excavation								
			20							
			25							

See Subsurface Conditions section in the report for additional information.

FIGURE 4C

GSH — TEST PIT LOG

Page: 1 of 1

TEST PIT: TR-50A

CLIENT: Valley Enterprise Investment Company — PROJECT NUMBER: 2077-01N-16

PROJECT: Summit at Ski Lake No. 13 — DATE STARTED: 4/13/16 — DATE FINISHED: 4/13/16

LOCATION: East Clairetina Court, Huntsville, Utah — GSH FIELD REP.: AA

EXCAVATING METHOD/EQUIPMENT: KOMATSU - Trackhoe

GROUNDWATER DEPTH: Not Encountered (4/13/16) — ELEVATION: ---

WATER LEVEL	USCS	DESCRIPTION	DEPTH (FT.)	SAMPLE SYMBOL	MOISTURE (%)	DRY DENSITY (PCF)	% PASSING 200	LIQUID LIMIT (%)	PLASTICITY INDEX	REMARKS
		Ground Surface								
	CL/ML	SILTY CLAY/CLAYEY SILT with trace fine sand; major roots (topsoil) to 12"; light brown to brown	0							slightly moist medium stiff
				▮						
			5							
				▮						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 4D

Exhibit B

TEST PIT LOG

Page: 1 of 1

TEST PIT: TR-50B

CLIENT: Valley Enterprise Investment Company — PROJECT NUMBER: 2077-01N-16

PROJECT: Summit at Ski Lake No. 13 — DATE STARTED: 4/13/16 — DATE FINISHED: 4/13/16

LOCATION: East Clairetina Court, Huntsville, Utah — GSH FIELD REP.: AA

EXCAVATING METHOD/EQUIPMENT: KOMATSU - Trackhoe

GROUNDWATER DEPTH: Not Encountered (4/13/16) — ELEVATION: ---

WATER LEVEL	USCS	DESCRIPTION	DEPTH (FT.)	SAMPLE SYMBOL	MOISTURE (%)	DRY DENSITY (PCF)	% PASSING 200	LIQUID LIMIT (%)	PLASTICITY INDEX	REMARKS
		Ground Surface	0							
	CL/ML	SILTY CLAY/CLAYEY SILT with trace fine sand; major roots (topsoil) to 12"; light brown to brown								moist medium stiff
				▌						slightly moist stiff
				▌						medium stiff
			5							
				▌						stiff
		End of Exploration at 7.5' 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 4E

TEST PIT LOG

Page: 1 of 1

TEST PIT: TR-51

CLIENT: Valley Enterprise Investment Company — PROJECT NUMBER: 2077-01N-16

PROJECT: Summit at Ski Lake No. 13 — DATE STARTED: 4/12/16 — DATE FINISHED: 4/12/16

LOCATION: East Clairetina Court, Huntsville, Utah — GSH FIELD REP.: AA

EXCAVATING METHOD/EQUIPMENT: KOMATSU - Trackhoe

GROUNDWATER DEPTH: Not Encountered (4/12/16) — ELEVATION: ---

WATER LEVEL	USCS	DESCRIPTION	DEPTH (FT.)	SAMPLE SYMBOL	MOISTURE (%)	DRY DENSITY (PCF)	% PASSING 200	LIQUID LIMIT (%)	PLASTICITY INDEX	REMARKS
		Ground Surface	0							
	CL/ML	SILTY CLAY/CLAYEY SILT with trace fine sand; major roots (topsoil) to 12"; light brown to brown								moist stiff
			~2	■						slightly moist
			5	■						
		End of Exploration at 7.0' No significant sidewall caving No groundwater encountered at time of excavation	7							
			10							
			15							
			20							
			25							

See Subsurface Conditions section in the report for additional information.

FIGURE 4F

TEST PIT LOG

Page: 1 of 1

TEST PIT: TR-52

CLIENT: Valley Enterprise Investment Company — PROJECT NUMBER: 2077-01N-16

PROJECT: Summit at Ski Lake No. 13 — DATE STARTED: 4/13/16 — DATE FINISHED: 4/13/16

LOCATION: East Clairetina Court, Huntsville, Utah — GSH FIELD REP.: AA

EXCAVATING METHOD/EQUIPMENT: KOMATSU - Trackhoe

GROUNDWATER DEPTH: Not Encountered (4/13/16) — 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 trace fine to coarse sand; major roots (topsoil) to 12"; brown	0							moist medium stiff
				■						
		sand grades out; light brown	5	■						slightly moist stiff
				■						
		End of Exploration at 7.5' 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 4G

Exhibit B

TEST PIT LOG

Page: 1 of 1

TEST PIT: TR-53

CLIENT: Valley Enterprise Investment Company | PROJECT NUMBER: 2077-01N-16

PROJECT: Summit at Ski Lake No. 13 | DATE STARTED: 4/12/16 | DATE FINISHED: 4/12/16

LOCATION: East Clairetina Court, Huntsville, Utah | GSH FIELD REP.: AA

EXCAVATING METHOD/EQUIPMENT: KOMATSU - Trackhoe

GROUNDWATER DEPTH: Not Encountered (4/12/16) | ELEVATION: ---

WATER LEVEL	USCS	DESCRIPTION	DEPTH (FT.)	SAMPLE SYMBOL	MOISTURE (%)	DRY DENSITY (PCF)	% PASSING 200	LIQUID LIMIT (%)	PLASTICITY INDEX	REMARKS
		Ground Surface								
	SM/BR	WEATHERED SANDSTONE major roots (topsoil) to 12"; brown	0							
				◣						
				◣						
			5							
		End of Exploration at 6.0' 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 4H

TEST PIT LOG

Page: 1 of 1

TEST PIT: TR-54

CLIENT: Valley Enterprise Investment Company	PROJECT NUMBER: 2077-01N-16
PROJECT: Summit at Ski Lake No. 13	DATE STARTED: 4/12/16 DATE FINISHED: 4/12/16
LOCATION: East Clairetina Court, Huntsville, Utah	GSH FIELD REP.: AA
EXCAVATING METHOD/EQUIPMENT: KOMATSU - Trackhoe	
GROUNDWATER DEPTH: Not Encountered (4/12/16)	ELEVATION: ---

WATER LEVEL	USCS	DESCRIPTION	DEPTH (FT.)	SAMPLE SYMBOL	MOISTURE (%)	DRY DENSITY (PCF)	% PASSING 200	LIQUID LIMIT (%)	PLASTICITY INDEX	REMARKS
		Ground Surface								
	CL/ML/BR	SILTSTONE/CLAYSTONE major roots (topsoil) to 12"; light gray to gray	0							
				◣						
				◣						
		End of Exploration at 5.0' No significant sidewall caving No groundwater encountered at time of excavation	5							
			10							
			15							
			20							
			25							

See Subsurface Conditions section in the report for additional information.

FIGURE 4I

TEST PIT LOG

GSH

Page: 1 of 1

TEST PIT: TP-51

CLIENT: Valley Enterprise Investment Company — PROJECT NUMBER: 2077-01N-16

PROJECT: Summit at Ski Lake No. 13 — DATE STARTED: 4/13/16 — DATE FINISHED: 4/13/16

LOCATION: East Clairetina Court, Huntsville, Utah — GSH FIELD REP.: AA

EXCAVATING METHOD/EQUIPMENT: KOMATSU - Trackhoe

GROUNDWATER DEPTH: Not Encountered (4/13/16) — 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 reddish-brown	0	◤ (at ~2.0')						moist stiff
	ML	SILT/WEATHERED SILTSTONE with some fine sand; light brown	5							slightly moist very stiff
		End of Exploration at 11.0' 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 4J

CLIENT: Valley Enterprise Investment Company PROJECT: Summit at Ski Lake No. 13 PROJECT NUMBER: 2077-01N-16	KEY TO BORING LOG

WATER LEVEL	USCS	DESCRIPTION	DEPTH (FT.)	BLOW COUNT	SAMPLE SYMBOL	MOISTURE (%)	DRY DENSITY (PCF)	% PASSSING 200	LIQUID LIMIT (%)	PLASTICITY INDEX	REMARKS
①	②	③	④	⑤	⑥	⑦	⑧	⑨	⑩	⑪	⑫

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 **Blow Count:** Number of blows to advance sampler 12" beyond first 6", using a 140-lb hammer with 30" drop.

⑥ **Sample Symbol:** Type of soil sample collected at depth interval shown; sampler symbols are explained below.

⑦ **Moisture (%):** Water content of soil sample measured in laboratory; expressed as percentage of dryweight of

⑧ **Dry Density (pcf):** The density of a soil measured in laboratory; expressed in pounds per cubic foot.

⑨ **% Passing 200:** Fines content of soils sample passing a No. 200 sieve; expressed as a percentage.

⑩ **Liquid Limit (%):** Water content at which a soil changes from plastic to liquid behavior.

⑪ **Plasticity Index (%):** Range of water content at which a soil exhibits plastic properties.

⑫ **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 5

GSH

CLIENT: Valley Enterprise Investment Company
PROJECT: Summit at Ski Lake No. 13
PROJECT NUMBER: 2077-01N-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
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HIGHLY ORGANIC SOILS			PT	Peat, Humus, Swamp Soils with High Organic Contents

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TYPICAL SAMPLER GRAPHIC SYMBOLS

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- 3.0" OD, 2.42" ID D&M Sampler
- California Sampler
- Thin Wall

WATER SYMBOL

Water Level

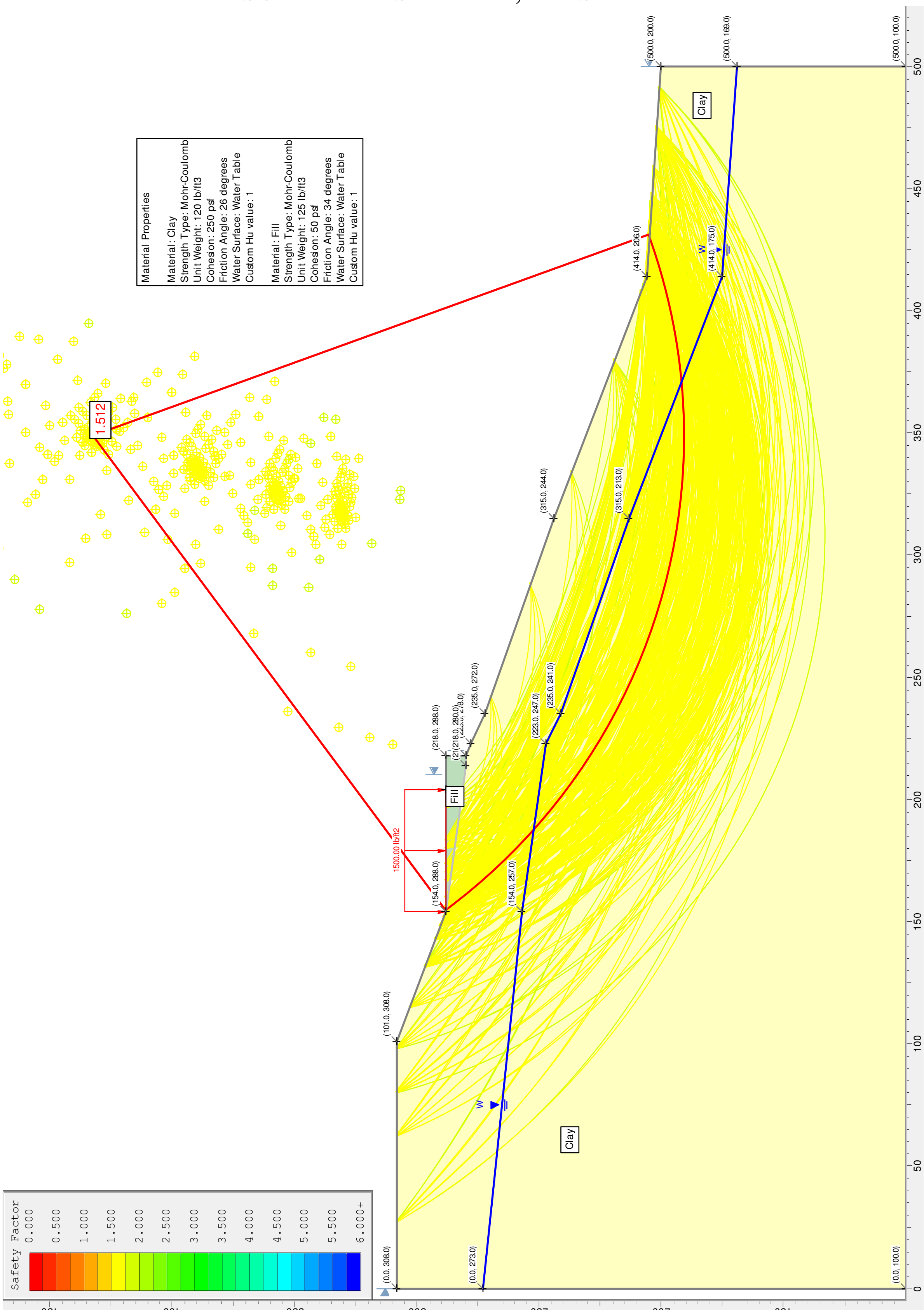
FIGURE 6

GSH

Exhibit B

STABILITY RESULTS

SUMMIT AT SKI LAKE, PHASE 13

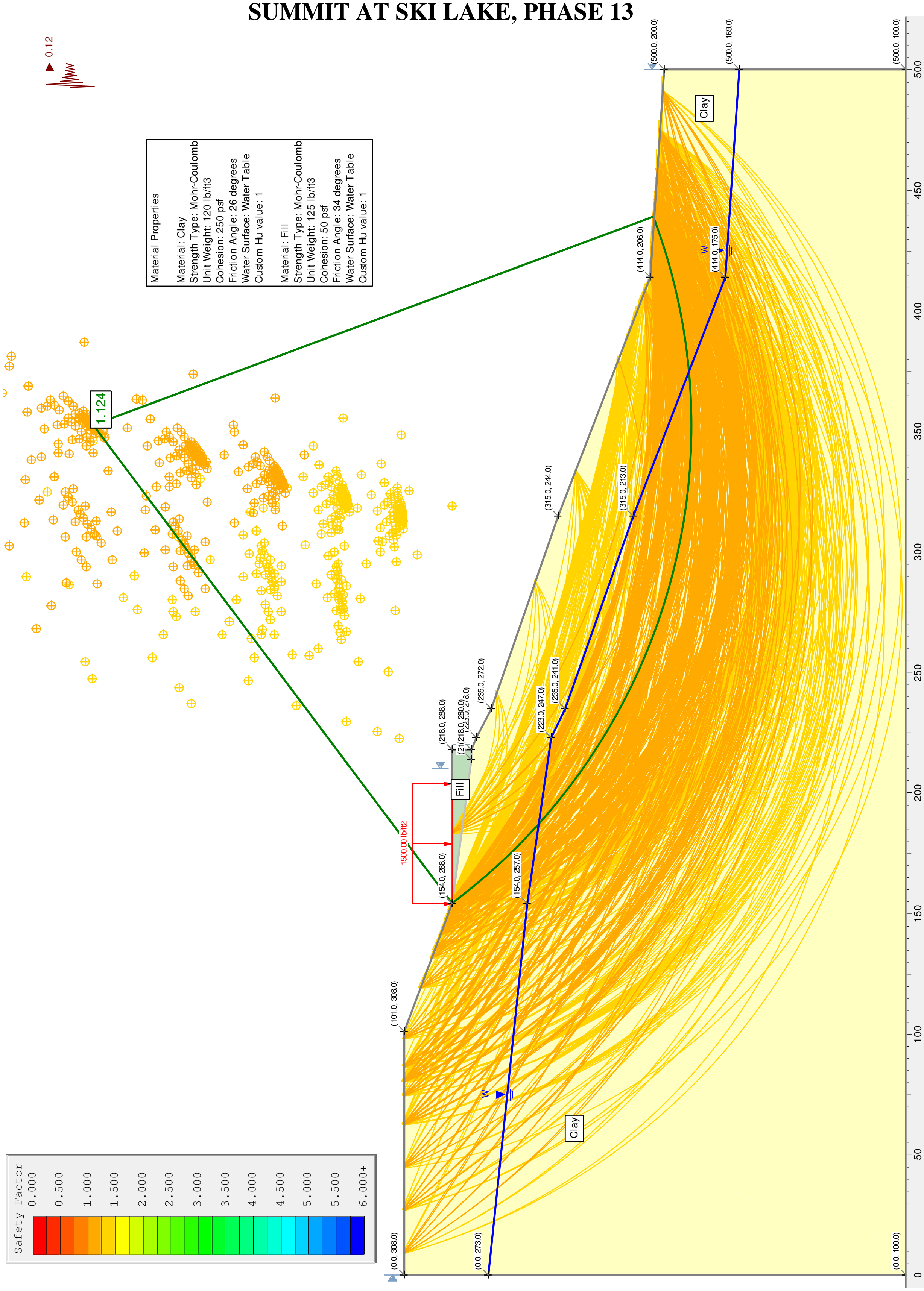


PROJECT NO.: 2077-01N-16	GSH	**FIGURE NO.:** 7

Exhibit B

STABILITY RESULTS

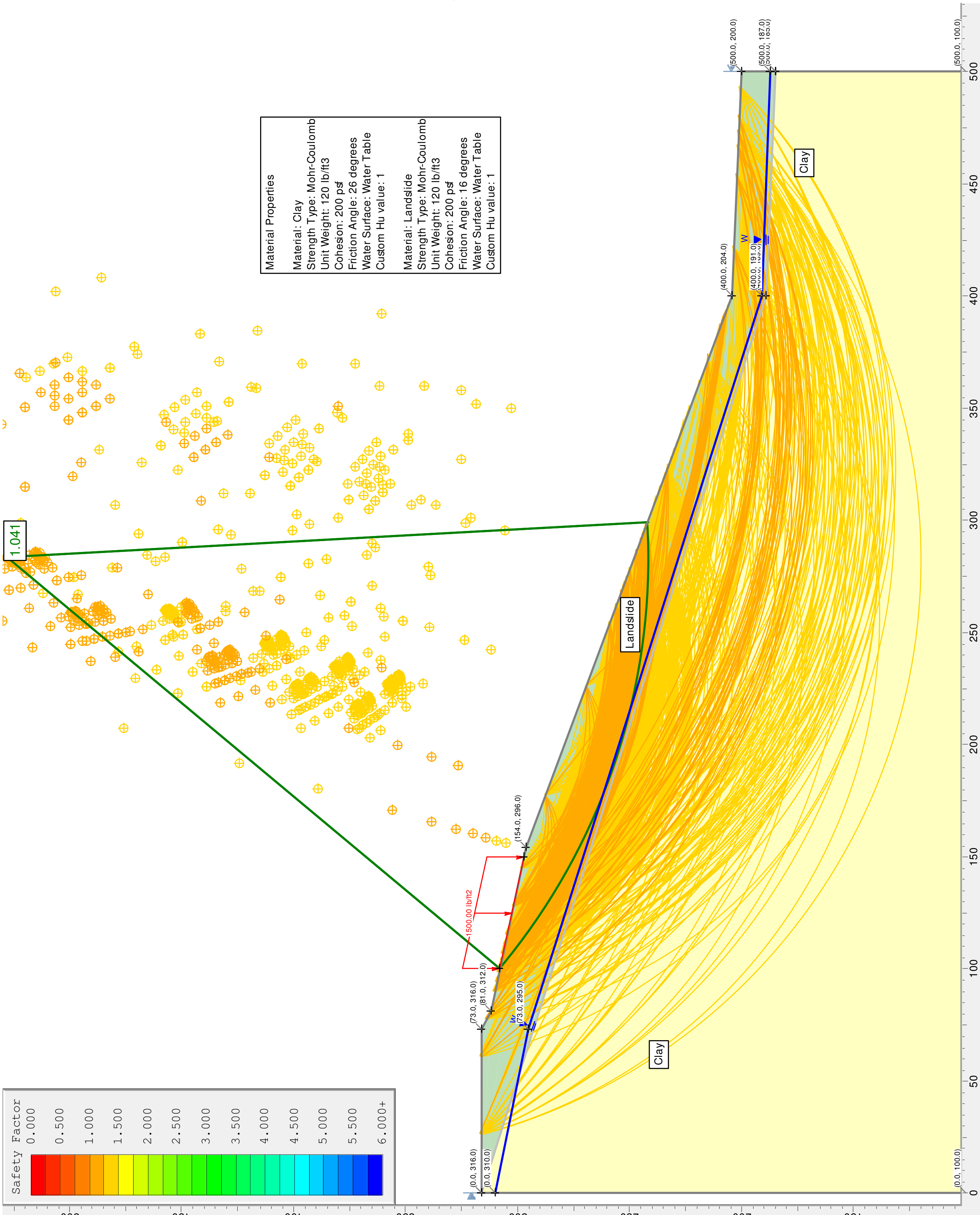
SUMMIT AT SKI LAKE, PHASE 13



PROJECT NO.: 2077-01N-16	GSH	FIGURE NO.: 8

STABILITY RESULTS

SUMMIT AT SKI LAKE, PHASE 13 (SECTION B-B')

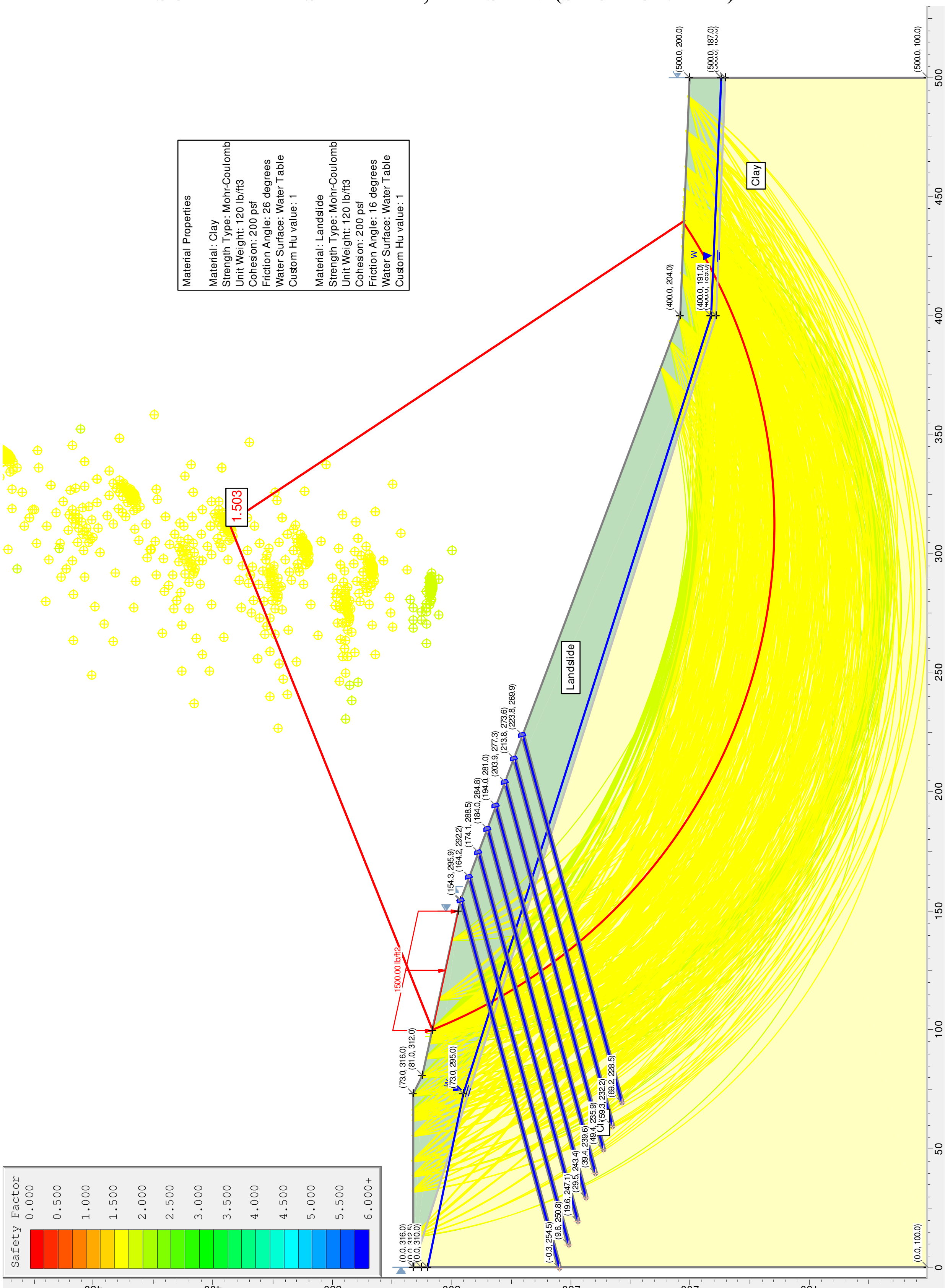


PROJECT NO.: 2077-01N-16	GSH	FIGURE NO.: 9

Exhibit B

STABILITY RESULTS

SUMMIT AT SKI LAKE, PHASE 13 (SECTION B-B')

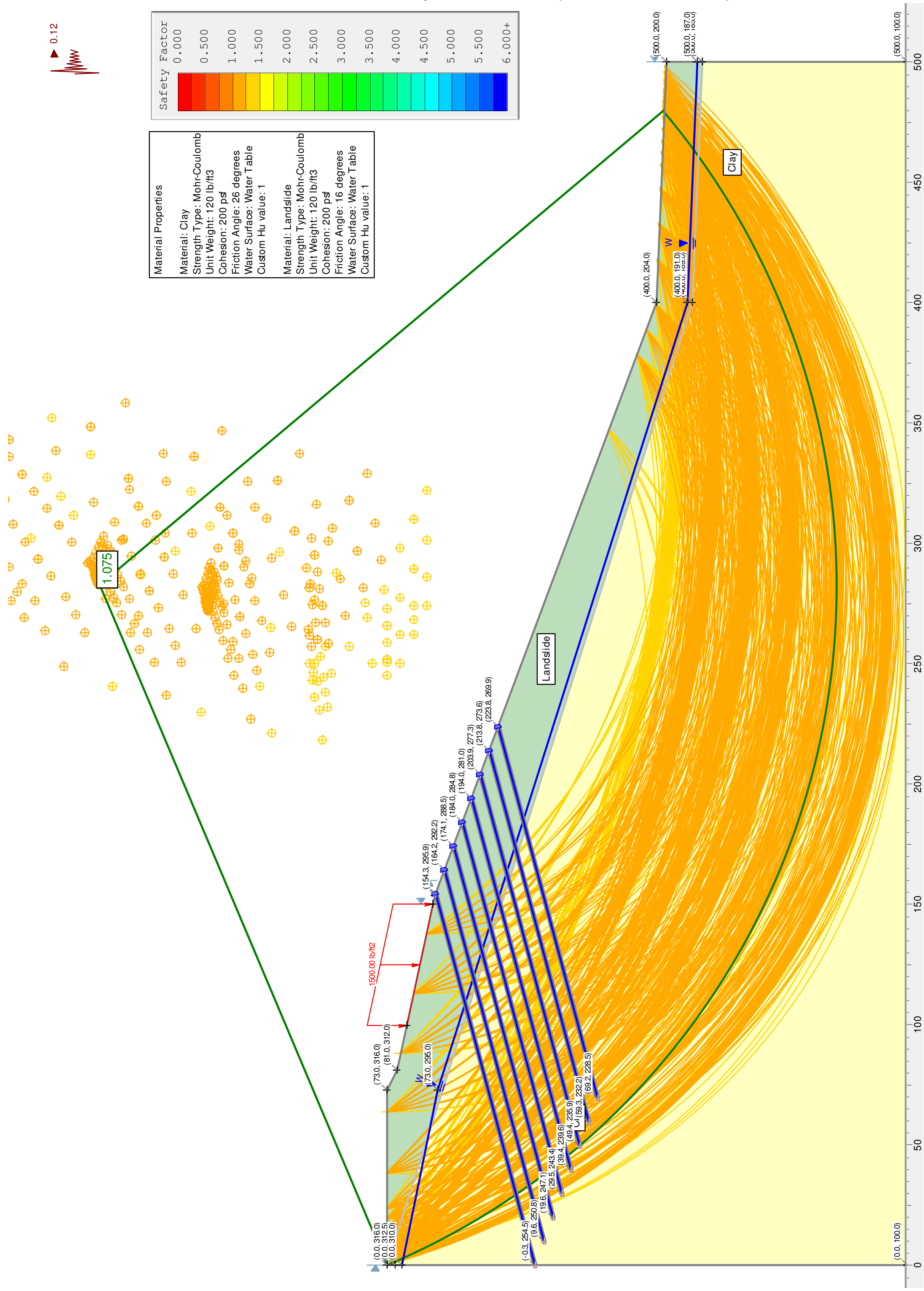


PROJECT NO.: 2077-01N-16	GSH	FIGURE NO.: 11

Exhibit B

STABILITY RESULTS

SUMMIT AT SKI LAKE, PHASE 13 (SECTION B-B')



PROJECT NO.: 2077-01N-16	GSH	FIGURE NO.: 11

Exhibit B

REDI-ROCK WALL STABILITY EVALUATION

Project:	The Summit at Ski Lake, Phase 13			Date:	6/3/16
Location:	Huntsville, Utah			By:	AMH
Backfill slope angle, β:	0	degrees (β)	Foundation soil γ :	120	pcf
Front batter angle (from vert.):	5.16	degrees (α)	Foundation soil ϕ :	26	degrees
Soil/wall interface friction:	17	degrees (δ)	Found. soil cohesion:	200	psf
Surcharge pressure:	0	psf	Retained soil γ :	120	pcf
	static	seismic	Retained soil ϕ :	26	degrees
FS against sliding:	1.5	1.1	Retain. soil cohesion:	200	psf
FS against overturning:	2.0	1.1	Block γ :	145	psf
FS for bearing:	3.0	1.5	Block ϕ :	45	degrees
Horizontal seismic coef., k_h:	0.16	(typically ½ of PGA)	Embedment depth:	0.5	feet
Vertical seismic coef., k_v:	0	(typically 0)	Block Width:	41	inches
Mononobe-Okabe theta, θ =	0.1587		Soil Bearing Capacity =	10069	psf (Meyerhoff)

STATIC

Wall Ht, H (ft)	**1.5**	**3.0**	**4.5**	**6.0**	**7.5**	**9.0**	**10.5**	**12.0**	**13.5**	**15.0**	**16.5**
Block Width (in)	**41.0**	**41.0**	**41.0**	**41.0**	**41.0**	**28.0**	**28.0**	**28.0**	**28.0**	**28.0**	**28.0**
Block Width (ft)	3.4167	3.4167	3.4167	3.4167	3.4167	2.3333	2.3333	2.3333	2.3333	2.33333	2.33333
Back batter angle, ψ:	0	5.1586	5.1586	5.1586	5.1586	5.1586	5.1586	5.1586	5.1586	5.15855	5.15855
Coulomb K_a	0.3475	0.3124	0.3124	0.3124	0.3124	0.3124	0.3124	0.3124	0.3124	0.3124	0.3124
F_a (lbs/ft)	0	0	0	4	211	501	874	1329	1867	2487	3190
Wall Wt, W (lbs/ft)	743	1486	2229	2973	3716	4223	4731	5238	5746	6253	6761
Wall $x_{centroid}$ (ft)	1.71	1.78	1.84	1.91	1.98	1.96	1.96	1.98	2.00	2.03	2.07
Wall $y_{centroid}$ (ft)	0.75	1.50	2.25	3.00	3.75	4.29	4.88	5.49	6.13	6.79	7.47
$F_{sliding}$ (lbs/ft)	0	0	0	4	207	491	855	1301	1827	2434	3122
$F_{resisting}$ (lbs/ft)	362	725	1087	1450	1833	2110	2395	2688	2989	3299	3617
$FS_{base\ sliding}$	**>100**	**>100**	**>100**	**>100**	**8.9**	**4.3**	**2.8**	**2.1**	**1.6**	**1.4**	**1.2**
$FS_{interface\ shear}$	**>100**	**>100**	**>100**	**>100**	**8.3**	**4.0**	**2.6**	**1.9**	**1.5**	**1.2**	**1.0**
$M_{overturn}$ (ft-lbs/ft)	0	0	0	8	517	1472	2994	5203	8221	12170	17170
$M_{resisting}$ (ft-lbs/ft)	1270	2640	4110	5685	7512	8650	9908	11287	12789	14417	16173
$FS_{overturn}$	**>100**	**>100**	**>100**	**>100**	**14.5**	**5.9**	**3.3**	**2.2**	**1.6**	**1.2**	**0.9**
Eccentricity, e (ft)	0.00	0.00	0.00	0.00	0.12	0.30	0.56	0.88	1.26	1.70	2.20
Bearing Pressure	218	435	653	874	1329	1941	2841	4092	5756	7896	10574
$FS_{bearing}$	**46.3**	**23.1**	**15.4**	**11.5**	**7.6**	**5.2**	**3.5**	**2.5**	**1.7**	**1.3**	**1.0**

SEISMIC

Mononobe-Okabe K_{ae}	0.4777	0.4393	0.4393	0.4393	0.4393	0.4393	0.4393	0.4393	0.4393	0.4393	0.4393
F_{ae} (lbs/ft)	0	0	0	154	489	942	1514	2205	3015	3943	4989
$F_{sliding}$ (lbs/ft)	119	238	357	626	1073	1598	2239	2996	3870	4859	5965
$F_{resisting}$ (lbs/ft)	362	725	1087	1465	1861	2154	2459	2776	3104	3444	3797
$FS_{base\ sliding}$	**3.0**	**3.0**	**3.0**	**2.3**	**1.7**	**1.3**	**1.1**	**0.9**	**0.8**	**0.7**	**0.6**
$FS_{interface\ shear}$	**2.9**	**2.9**	**2.9**	**2.2**	**1.6**	**1.2**	**1.0**	**0.8**	**0.7**	**0.6**	**0.5**
$M_{overturn}$ (ft-lbs/ft)	89	357	803	1962	3967	6701	10633	15981	22962	31790	42684
$M_{resisting}$ (ft-lbs/ft)	1270	2640	4110	5800	7729	9003	10432	12018	13766	15681	17765
$FS_{overturn}$	**14.2**	**7.4**	**5.1**	**3.0**	**1.9**	**1.3**	**1.0**	**0.8**	**0.6**	**0.5**	**0.4**
Eccentricity (ft)	0.12	0.24	0.36	0.63	0.99	1.44	2.00	2.68	3.45	4.32	5.27
Bearing Pressure	263	618	1065	1858	3065	4565	6670	9492	13141	17731	23371
$FS_{bearing}$	**38.2**	**16.3**	**9.5**	**5.4**	**3.3**	**2.2**	**1.5**	**1.1**	**0.8**	**0.6**	**0.4**

Max. Recommended Wall Height: 9 feet

PROJECT NO.: 2077-01N-16	GSH	**FIGURE NO.:** 12

REDI-ROCK WALL DETAIL

THE SUMMIT AT SKI LAKE, PHASE 13, HUNTSVILLE

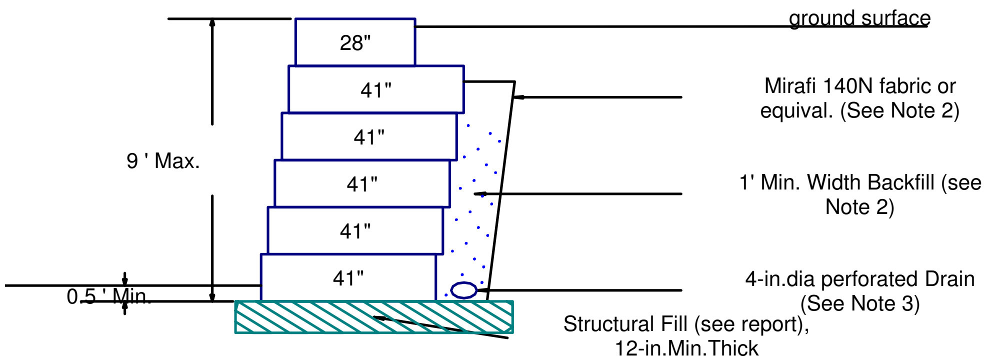
NOTES:

1. BACKFILL SOILS SHOULD BE PLACED IN LOOSE LIFTS NOT EXCEEDING A THICKNESS OF 12 INCHES, MOISTURE CONDITIONED TO WITHIN 2% OF OPTIMUM, AND COMPACTED TO A MINIMUM 95% OF THE MAXIMUM DRY DENSITY AS DETERMINED BY ASTM D1557.

2. FREE-DRAINING BACKFILL SHALL CONSIST OF GRAVEL HAVING LESS THAN 5% PASSING No. 200 SIEVE.

3. PERFORATED DRAIN SHALL BE WRAPPED WITH FABRIC, SLOPED A MINIMUM 2% TO SIDE OF WALL, AND DISCHARGED TO APPROPRIATE DRAINAGE DEVICE OR AREA.

4. BLOCK DEPTHS SHOWN FOR INDIVIDUAL BLOCKS.



NOT TO SCALE

PROJECT NO.: 2077-01N-16	GSH	**FIGURE NO.:** 13