GEOTECHNICAL ENGINEERING REPORT FOR:

SNOWBASIN
3925 SNOW BASIN ROAD
HUNTSVILLE, UTAH

PREPARED BY:

ANTARES, LLC
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PREPARED FOR:

BONSAI DESIGN
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DATE:
AUGUST 15, 2018

ANTARES PROJECT NO.
18109-01

ANTARES

Project No. 18109-01
August 15, 2018

Bonsai Design – Attn: Dane Wood
201 South Avenue
Grand Junction, Colorado 81501

Reference: ***Proposed Adventure Courses***
Snowbasin – 3925 Snow Basin Road
Huntsville, Utah

Subject: ***Geotechnical Engineering Report***

Dear Mr. Wood,

This report presents the results of our geotechnical engineering investigation performed at the Snowbasin Resort in Huntsville, Utah. As proposed, the project is to include development of an elevated challenge course and two ziplines, as well as associated platforms, stairways, flatwork and underground utilities.

The findings presented in this report are based on our subsurface investigation, laboratory test results, and our experience with subsurface conditions in the area. Our opinion is that the project can be completed as proposed, provided the recommendations presented in this report are implemented.

Please contact us if you have any questions regarding our observations or the recommendations presented in this report.

Sincerely,

ANTARES, LLC

Prepared by:

Chad Diez, C.E. 87093 Utah PE License Pending
Principal Engineer

copies: 4 to Dane Wood

TABLE OF CONTENTS

FIGURES

APPENDICES

1 INTRODUCTION

At the request of Bonsai Design, Antares, LLC (Antares) performed a geotechnical investigation at Snowbasin Resort in Huntsville, Utah. The geotechnical investigation was performed in general accordance with our August 2, 2018 proposal for the project, a copy of which is included as Appendix A of this report. For your review, Appendix B contains a document prepared by ASFE entitled *Important Information About Your Geotechnical Engineering Report*.

1.1 SITE DESCRIPTION

The approximate 3-acre site is located approximately 0.1 mile off of Rocklin Road in Rocklin, California. The property is bordered to the northeast by a Rocklin Road and to the northwest by Pacific Street. The remainder of the subject property is bordered by urban land.

1.2 PROPOSED IMPROVEMENTS

Based on our understanding of the project, several small structures are proposed. We understand that there will be an entry tower for the first zip line, a 3-structure challenge course, and a zip line landing for the second zipline.

1.3 PURPOSE

We performed a surface reconnaissance and limited subsurface geotechnical investigation at the site, collected soil samples for laboratory testing, and performed engineering calculations to provide geotechnical recommendations such as foundation design criteria.

1.4 SCOPE OF SERVICES

To prepare this report, we performed the following scope of services:

- We performed a site investigation, including a literature review and a limited subsurface investigation.
- We collected relatively undisturbed soil samples and bulk soil samples from three exploratory borings.
- We coordinated laboratory testing on select soil samples obtained during our subsurface investigation to determine their engineering material properties.
- Based on observations made during our subsurface investigation and the results of laboratory testing, we performed engineering calculations to provide geotechnical engineering recommendations for structural improvements.

Our scope of services did not include a groundwater flow analysis nor an evaluation of the site for the presence of hazardous materials, asbestiform minerals, mold, or corrosive subsurface conditions.

2 SITE INVESTIGATION

We performed a site investigation to characterize the existing surface conditions and shallow subsurface soil/rock conditions. Our site investigation included a literature review and field investigation as described below.

2.1 LITERATURE REVIEW

We performed a limited review of geologic literature pertaining to the project site. The following sections summarize our findings.

2.1.1 Soil Survey

As part of our investigation, we reviewed the online soil survey presented by the U.C. Davis Soil Resource Laboratory. The soil survey lists the area containing the project site as *Crooked Creek Silty Clay Loam* and *Poleline-Patio Association*.

Crooked Creek Silty Clay Loam

The crooked creek silty clay loam consists of 90% Crooked Creek series soils. The Crooked Creek series consists of very deep, poorly drained soils that formed in alluvium derived from mixed sedimentary and igneous rocks.

The typical profile of the Crooked Creek Series consists of a surface layer of approximately 24 inches of dark grayish brown clay loam. This horizon is typically underlain by clay loam to a depth of approximately 70 inches. pH is typically neutral within the Crooked Creek Series.

Poleline-Patio Association

The Poline-Patio Series consists of deep, well-drained, moderately permeable soils that formed in colluvium from sandstone. The series typically consists of gravelly to very gravelly loam to a depth of approximately 44 inches, where sandstone is typically encountered.

2.2 FIELD INVESTIGATION

We performed our field investigation on August 8, 2018. During our field investigation, we observed the local topography and surface conditions and performed a limited subsurface investigation. The following sections summarize surface and subsurface conditions observed during our field investigation.

Our subsurface investigation included the advancement of three exploratory borings in the areas of proposed structures as shown on the boring location map. We advanced our borings to depths ranging between 16.5 and 21.5 feet below the ground surface (bgs) using a Mobile B-80 drill rig equipped with a 4-inch diameter ODEX system. Blow counts were made and samples were taken via a STP sampler. An engineer from our firm logged the soil conditions revealed in the exploratory borings and collected relatively undisturbed and bulk soil samples for laboratory testing. Figure 2 shows the approximate exploratory boring locations.

2.2.1 Surface Conditions

At the time of our investigation, the ares of the proposed zip launch and landing stations consisted of low-lying brush. The area of the proposed challenge zone was within a heavily wooded area in which a small area had been cleared for drill-rig access. Topography in the areas of the zip launch tower and the challenge course was moderately sloped, ranging from zero to 30 percent. The proposed zip landing area was flat.

2.2.2 Subsurface Soil Conditions

The soil conditions described in the following paragraph are generalized, based on our observations of soil revealed in our three exploratory borings. More detailed information can be found in the trench logs in Appendix C.

Our exploratory borings generally revealed stiff gravelly clay to dense gravelly sand with boulders throughout. The borings generally revealed less fine-grained material with depth. Cobbles and boulders were encountered throughout the depth of all borings.

2.2.3 Groundwater Conditions

During our site investigation, we did not encounter groundwater seepage in our exploratory borings, nor did we observe onsite springs or seeps emanating from the ground surface. Based on the course grain content of the soils encountered in our exploratory borings, we do not anticipate that groundwater will be encountered during construction.

3 LABORATORY TESTING

We coordinated laboratory testing on selected soil samples collected from our subsurface exploratory borings to determine their engineering material properties. These engineering material properties were used to develop geotechnical engineering design recommendations for earthwork and structural improvements.

We performed the following laboratory tests:

- Moisture Content, (ASTM D2216),
- Density (unit weight), (ASTM D2937),
- Particle Size (ASTM D422),
- Atterberg Limits (ASTM D4318),
- Direct Shear Strength (ASTM D3080),

Table 3.1 summarizes moisture/density and direct shear test results.

Table 3.1 – Summary of Moisture/Density and Direct Shear Testing

Boring Number	Sample Number	Depth (feet)	Dry Density (pcf)	Moisture Content (%)	Shear Friction Angle (degrees)	Shear Cohesion (psf)
B-1	BT1-5	5.0	103.6	16	35.5	265
B-2	BT2-7.5	7.5	96.9	14	--	--
B-3	BT3-3	3.0	99.6	23	34.3	305

We performed a particle size determination on a sample collected from 2.5 feet bgs in Boring B-1. The test revealed the sample consisted of approximately 40 percent sand, 35 percent gravel, and 25 percent silt and clay.

We also performed an Atterberg limits determination on the sample. The Atterberg limits determination revealed that the portion of the sample passing the No. 40 sieve had a liquid limit of 53 and a plastic limit of 41, resulting in a plasticity index of 12. Based on the Atterberg limits determination and the particle size determination, we classified the soil as a gravelly sand with clay (GW).

4 CONCLUSIONS

The following conclusions are based on our field observations, laboratory test results, and our experience in the area.

1. Our opinion is that the site is suitable for the proposed improvements, provided that the geotechnical engineering recommendations and design criteria presented in this report are incorporated into the project plans.
2. Our primary concern is the presence of resistant cobbles and boulders at shallow depths that may affect foundation excavatability, depending on the methods utilized.
3. Based on our site observations, the geology of the region, and our experience in the area, our opinion is that the risk of seismically induced hazards such as slope instability, liquefaction, and surface rupture are remote at the project site.
4. During our site investigation, we did not observe groundwater or seepage within our exploratory borings. Our observations were made during the summer after an extended period of dry weather.
5. Fill material was encountered in exploratory boring B-3 to a depth of approximately 15 feet. The fill material appeared to be generated from onsite soils based on the gradation. Blow counts indicated that the fill was well-compacted.
6. Prior to grading and construction, if prepared by a company other than Antares, we should be retained to review the proposed grading plan and structural improvements to confirm our recommendations.
7. The project site is mapped as having a frost depth of 36 inches. This frost depth shall be accounted for in the design of the foundation elements for the project. Skin friction shall be neglected within the frost depth.

5 RECOMMENDATIONS

The following engineering recommendations are based on our understanding of the project as currently proposed, our field observations, the results of laboratory testing, engineering analysis, and our experience in the area.

5.1 GRADING

Based on our understanding of the project, we assume that minimal grading will be needed to construct the proposed improvements. However, we have included the following grading recommendations as general guidelines to be followed during construction. The grading recommendations address clearing and grubbing, soil preparation, cut slope grading, fill placement, fill slope grading, erosion control, surface water drainage, underground utility trenches, soil corrosion potential, plan review, and construction monitoring.

5.1.1 Clearing and Grubbing

Areas to be graded should be cleared and grubbed to remove vegetation and other deleterious materials as described below.

1. Strip and remove debris from clearing operations and the top 2 to 3 inches of soil containing shallow vegetation, roots and other deleterious materials. The organic topsoil can be stockpiled onsite and used in landscape areas but is not suitable for use as fill. The project geotechnical engineer should approve any proposed use of the spoil generated from stripping prior to placement.
2. Overexcavate any relatively loose debris and soil that is encountered in any onsite excavations such as existing or abandoned utility trenches or exploratory trenches by others to underlying, competent material.
3. Although not observed during our investigation, if loose, untested fill is encountered during site development, overexcavate to competent native soil or weathered rock a minimum of 5 feet beyond the areas of proposed improvements.
4. Remove rocks greater than 8 inches in greatest dimension (oversized rock) from native soil by scarifying to a depth of 12 inches below finish grade in areas to support pavement, slabs-on-grade or other flatwork. Oversized rock may be used in landscape areas, rock landscape walls, or removed from the site. Oversized rock can be stockpiled onsite and used to construct fills, but must be placed at or near the bottom of deep fills and must be placed in windrows to avoid nesting. No oversized rock should be placed in the upper 3 feet of any structural fill. Unless used as rip-rap, oversized rock placed in fill should not be located within 5 feet horizontally of the finished fill slope face. The project geotechnical engineer should approve the use of oversized rock prior to constructing fill.
5. Fine grained, potentially expansive soil, as determined by Antares, that is encountered during grading should be mixed with granular soil, or overexcavated and stockpiled for removal from the project site or for later use in landscape areas. A typical mixing ratio for granular to expansive soil is 4 to 1. The actual mixing ratio should be determined by Antares.
6. Vegetation, deleterious materials, structural debris, and oversized rocks not used in landscape areas, drainage channels, or other non-structural uses should be removed from the site.

5.1.2 Cut Slope Grading

Based on our understanding of the project at this time, we anticipate that permanent cut slopes up to 4 feet in height will be created during grading of the proposed improvements. In general, permanent cut slopes should not be steeper than 2:1, horizontal to vertical (H:V). Steeper cut slopes may be feasible, depending on the soil/rock conditions encountered and should be reviewed on a case-by-case basis. The upper two feet of all cut slopes should be graded to an approximate 2:1, H:V, slope to reduce sloughing and erosion of looser surface soil.

Temporary cut slopes may be constructed to facilitate retaining wall construction. We anticipate that subsurface conditions will be favorable for construction of temporary cut slopes no steeper than ½:1, H:V, for a maximum height of approximately 8 feet. To reduce the likelihood of sloughing or failure, temporary cut slopes should not remain over the winter.

A representative of Antares must observe temporary cut slopes steeper than 2:1, H:V, during grading to confirm the soil and rock conditions encountered. We recommend that personnel not be allowed between the cut slope and the proposed retaining structure, form work, grading equipment, or parked vehicles during construction, unless the stability of the slope has been reviewed by Antares or the slope has been confirmed to meet OSHA excavation standards.

5.1.3 Soil Preparation for Fill Placement

If proposed, in areas where fill will be placed, the surface soil exposed by clearing and grubbing should be prepared as described below.

1. The surface soil should be scarified to a minimum depth of 12 inches below the existing ground surface, or to resistant rock, whichever is shallower. Following scarification, the soil should be uniformly moisture conditioned to within approximately 3 percentage points of the ASTM D1557 optimum moisture content.
2. The scarified and moisture conditioned soil should then be compacted to achieve a minimum relative compaction of 90 percent based on ASTM D1557 maximum dry density. The moisture content, density, and relative percent compaction should be verified by a representative of Antares or approved alternative. The earthwork contractor should assist testing and observation representatives by excavating test pads with onsite earth moving equipment.
3. Where fill placement is proposed on native slopes steeper than approximately 5:1, H:V, a base key and routine benches must be provided. Unless otherwise recommended by the project engineer, the base key should be excavated at the toe of the fill a minimum of 2 feet into competent stratum, as determined by a representative of Antares during construction observation. The bottom of the base key should be sloped slightly into the hillside at an approximate gradient of 5 percent or greater.
4. The fill must be benched into existing side slopes as fill placement progresses. Benching must extend through loose surface soil into firm material, and at intervals such that no loose surface soil is beneath the fill.

5.1.4 Fill Placement

Soil fill placement proposed for the project should incorporate the following recommendations:

1. Soil used for fill should consist of uncontaminated, predominantly granular, non-expansive native soil or approved import soil. If encountered, rock used in fill should be broken into pieces no larger than 8 inches in diameter. Rocks larger than 8 inches are considered oversized material and should be stockpiled for offhaul or later use in landscape areas and drainage channels. If approved by the project geotechnical engineer, oversized rock may be placed at or near the bottom of deep fills. Oversized rock must be placed in windrows to avoid nesting and to facilitate the placement of compacted fill. No oversized rock should be placed in the upper 3 feet of any structural fill. The project geotechnical engineer should approve the use of oversized rock prior to constructing fill.
2. Import soil (if used) should be predominantly granular, non-expansive and free of deleterious material. Import material that is proposed for use onsite should be submitted to Antares for approval and possible laboratory testing at least 72 hours prior to transport to the site.
3. Cohesive, predominantly fine grained, or potentially expansive soil encountered during grading should be stockpiled for removal, mixed as directed by Antares, or used in landscape areas.
4. Soil used to construct fill should be uniformly moisture conditioned to within approximately 3 percentage points of the ASTM D1557 optimum moisture content. Wet soil may need to be air dried or mixed with drier material to facilitate placement and compaction, particularly during or following the wet season.
5. Fill should be constructed by placing uniformly moisture conditioned soil in maximum 8-inch-thick loose, horizontal lifts (layers) prior to compacting.
6. All fill should be compacted to a minimum relative compaction of 90 percent of the ASTM D1557 maximum dry density. The upper 12 inches of fill in paved areas, beneath proposed slabs-on-grade, and within the proposed building footprint should be compacted to a minimum of 95 percent relative compaction.

The moisture content, density and relative percent compaction of fill should be confirmed by a representative of Antares or approved alternative during construction.

5.1.5 Differential Fill Depth

The recommendations presented in this section are intended to reduce the magnitude of differential settlement-induced structural distress associated with variable fill depth beneath structures.

1. Site grading should be performed so that cut-fill transition lines do not occur directly beneath any structures. The cut portion of the cut-fill pads, if proposed, should be scarified to a minimum depth of 8 inches, and recompacted to 95 percent relative compaction.
2. Differential fill depths beneath structures should not exceed 5 feet. For example, if the maximum fill depth is 8 feet across a pad, the minimum fill depth beneath that pad should not be less than 3 feet. If a cut-fill pad is used in this example, the cut portion would need to be overexcavated 3 feet and rebuilt with compacted fill.

5.1.6 Fill Slope Grading

The following recommendations address fill slopes that are not supported by retaining walls. Although not anticipated for the project, the recommendations have been included as general guidelines for fill slope grading.

In general, permanent fill slopes created onsite should be no steeper than 2:1, H:V. Antares should review fill slope configurations greater than approximately 5 feet in height, if proposed, prior to fill placement. Compaction and fill slope grading must be confirmed by Antares or an approved alternative in the field.

Steeper fill slopes may be feasible with the use of geotextile reinforcement and/or rock facing. We can provide reinforced or buttressed fill slope design for the project, if requested.

Fill should be placed in horizontal lifts to the lines and grades shown on the project plans. Slopes should be constructed by overbuilding the slope face and then cutting it back to the design slope gradient. Fill slopes should not be constructed or extended horizontally by placing soil on an existing slope face and/or compacted by track walking.

Where placement of oversized rock in deep fill is proposed, the oversized rock should be placed a minimum of 5 feet horizontally from the finished fill slope face.

5.1.7 Erosion Controls

Graded portions of the site should be seeded as soon as possible to allow vegetation to become established prior to and during the rainy season. In addition, grading that results in greater than one acre of soil disturbance or in sensitive areas may require the preparation of a site-specific storm water pollution prevention plan. As a minimum, the following controls should be installed prior to and during grading to reduce erosion.

1. Prior to commencement of site work, fiber rolls should be installed down slope of the proposed area of disturbance to reduce migration of sediment from the site. Fiber rolls on slopes are intended to reduce sediment discharge from disturbed areas, reduce the velocity of water flow, and aid in the overall revegetation of slopes. The fiber rolls should remain in place until construction activity is complete and vegetation becomes established.
2. All soil exposed in permanent slope faces should be hydroseeded or hand seeded/strawed with an appropriate seed mixture compatible with the soil and climate conditions of the site as recommended by the local Resource Conservation District.
3. Following seeding, jute netting or erosion control blankets should be placed and secured over the slopes steeper than 2:1, H.V.
4. Surface water drainage ditches should be established as necessary to intercept and redirect concentrated surface water away from cut and fill slope faces. Under no circumstances should concentrated surface water be directed over slope faces. The intercepted water should be discharged into natural drainage courses or into other collection and disposal structures.

5.1.8 Underground Utility Trenches

Underground utility trenches should be excavated and backfilled as described below.

1. Based on subsurface conditions observed in our exploratory trenches, we anticipate that resistant rock at shallow depths may limit utility trench excavations. Pre-ripping of the trench alignment, blasting, or splitting may be required, particularly if utility trench excavations are deeper than 3 feet. Additionally, cobble to boulder sized pieces of resistant rock may need to be removed for utility trench construction.
2. The California Occupational Safety and Health Administration (OSHA) requires all utility trenches deeper than 4 feet bgs be shored with bracing equipment prior to being entered by any individuals, whether or not they are associated with the project.
3. Trench backfill used within the bedding and shading zones should consist of ¾-inch minus crushed rock, granular material with a sand equivalent greater than 30, or similar material approved by the project engineer.
4. Soil used as trench backfill should consist of non-expansive soil with a plasticity index (PI) less than or equal to 15 and should not contain rocks greater than 3 inches in greatest dimension unless otherwise approved by the geotechnical engineer.
5. Where utility trenches will intersect perimeter footings or pass within the proposed structure footprints, we recommend that a low permeability backfill plug be placed to reduce water migration and infiltration. In general, a low permeability, predominantly

fine-grained soil backfill, sand-cement slurry, or other approved material should be placed within five feet of the building exterior.

6. Trench backfill should be constructed by placing uniformly moisture conditioned soil in maximum 12-inch-thick loose lifts prior to compacting.

7. Trench backfill should be compacted to a minimum relative compaction of 90 percent of the ASTM D1557 maximum dry density. In areas of proposed pavement or concrete flatwork, the upper 12 inches of backfill should be compacted to a minimum relative compaction of 95 percent of the ASTM D1557 maximum dry density. Jetting is not an acceptable method of compacting trench backfill or bedding sand.

8. The loose lift thickness, moisture, density and relative compaction of the trench backfill soil should be observed by a representative of Antares or approved alternative during placement.

5.1.9 Soil Corrosion Potential

Laboratory testing was performed for soil corrosivity parameters (minimum resistivity, pH, chloride, and sulfate) on a selected soil sample. Based on the test results of minimum resistivity (65,550 ohm-cm), the soil is not considered to have high corrosive potential for buried metallic improvements. Additionally, the results of the pH (6.80), chloride (3.7ppm), and sulfate (6.1ppm) tests do not indicate significant corrosive potential to buried concrete structures.

To reduce the likelihood of corrosion problems, materials used for underground utilities, permanent subsurface drainage improvements, and foundation systems should be selected based on local experience and practice. If alternative or new construction methods or materials are being proposed, it may be appropriate to have the selected materials evaluated by a corrosion engineer for compatibility with the onsite soil and groundwater conditions.

The laboratory test results should only be considered as indicator parameters of potential soil corrosivity for the sample which was tested. Other soils found on the project site may be more, less, or of similar corrosive nature.

5.1.10 Surface Water Drainage

Proper surface water drainage is important to the successful development of the project.

We recommend the following measures to help mitigate surface water drainage problems:

1. Slope final grades in structural areas so that surface water drains away from pad finish subgrades at a minimum 5 percent slope for a minimum distance of 10 feet.

2. To reduce surface water infiltration, compact and slope all soil placed adjacent to structure foundations such that water is not allowed to pond. Backfill should be free of deleterious materials.
3. Construct V-ditches at the top of cut and fill slopes where necessary to reduce concentrated surface water flow over slope faces. Typically, V-ditches should be 3 feet wide and at least 6 inches deep. Surface water collected in V-ditches should be directed away and downslope from proposed building pads, driveways, and sidewalks into drainage channels.

5.1.11 Grading Plan Review and Construction Monitoring

Construction quality assurance includes review of plans and specifications and performing construction monitoring as described below.

1. If grading plans and construction documents are provided by a company other than Antares, Antares should be retained to review the final grading plans prior to construction to confirm our understanding of the project at the time of our investigation, to determine whether our recommendations have been implemented, and to provide additional and/or modified recommendations, if necessary.
2. Antares should be retained to perform or coordinate construction quality assurance (CQA) monitoring of all earthwork grading performed by the contractor to determine whether our recommendations have been implemented, and if necessary, provide additional and/or modified recommendations.

5.2 STRUCTURAL IMPROVEMENT DESIGN CRITERIA

The following sections present our structural improvement design criteria and recommendations. The recommendations address foundations, seismic parameters, concrete slabs-on-grade, rock anchors, and retaining walls.

5.2.1 Seismic Design Criteria

Our classification of on-site soil conditions is based on field observations and laboratory tests. The on-site soil primarily consists of gravelly sand. Based on the soil encountered during our investigation, we classified the soil as gravelly sad (GW) for design purposes.

Table 5.2.1.1 below summarizes seismic design criteria based on ASCE 7-10, the 2016 California Building Code, and the United States Geological Survey (USGS), *U.S. Seismic Design Maps Tool* to develop the following seismic design parameters:

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Table 5.2.1.1 - Seismic Design Parameters					
Description	Value	Reference	Description	Value	Reference
Latitude Longitude	41.216 -111.859	1	Site Class	C	2
Site Coefficient, F_A	1.013	5	Site Coefficient, F_V	1.460	6
Mapped Acceleration Parameter, S_S	0.968*g*	3	Mapped Acceleration Parameter, S_1	0.340*g*	4
Maximum Considered Short (0.2 sec) Spectral Response, S_{MS}	0.981g	5	Maximum Considered Long (1.0 sec) Spectral Response, S_{M1}	0.496*g*	6
Design Spectral Response Acceleration, S_{DS}	0.654*g*	7	Design Spectral Response Acceleration, S_{D1}	0.331*g*	8

References:

1. *USGS 7.5 min*
2. *ASCE 7-10 Table 20.3-1*
3. *ASCE 7-10 Figure 22-1*
4. *ASCE 7-10 Figure 22-2*
5. *ASCE 7-10 Equation 11.4-1*
6. *ASCE 7-10 Equation 11.4-2*
7. *ASCE 7-10 Equation 11.4-3*
8. *ASCE 7-10 Equation 11.4-4*

5.2.2 Slab-on-Grade Systems

Our opinion is that exterior concrete slabs-on-grade may be used in conjunction with perimeter concrete foundations for flatwork areas, if proposed. The project structural engineer should design slabs-on-grade with regard to the anticipated loading. This section presents typical slab sections and reinforcement schedules used in the region and presents construction recommendations. We can provide project specific slab-on-grade design for the proposed improvements once anticipated loading and serviceability criteria have been established.

1. In general, slabs-on-grade should be a minimum of 4 inches thick. If loads higher than 250 psf or intermittent live loads are anticipated, a structural engineer should determine the slab thickness and steel reinforcing schedule.
2. The subgrade soil around the slabs-on-grade should be sloped away from the proposed slab subgrade a minimum of 5 percent for a distance of 10 feet as discussed in the Surface Water Drainage section of this report. A representative from Antares should observe pad and subgrade elevations prior to forming the slab footings.
3. As a minimum, No. 3 rebar on 18-inch centers or flat sheets of 6x6, W4.0xW4.0 welded wire mesh (WWM) should be used as slab reinforcement. We do not recommend using rolls of WWM because vertically centered placement of rolled mesh within the slab is difficult to achieve. All rebar and sheets of WWM should be placed in the center of the slab and supported on concrete "dobies". We do not recommend "hooking and pulling" of steel during concrete placement.
4. Prior to placing concrete, slab subgrade soil must be moisture conditioned to between 75 and 90 percent saturation to a depth of 24 inches. Moisture conditioning should be performed for a minimum of 24 hours prior to concrete placement. If the soil is not moisture conditioned prior to placing concrete, moisture will be wicked out of the concrete, possibly contributing to shrinkage cracks. To facilitate slab-on-grade construction, we recommend that the slab subgrade soil be moisture conditioned following rock placement.
5. Slabs should be underlain by 4 inches of washed rock. The rock should be uniformly graded so that 100% passes the 1-inch sieve, with 0% to 5% passing the No. 4 sieve. Following rock placement, the subgrade soil should be moisture conditioned for 24 hours.
6. Expansion joints should be provided between the slab and perimeter footings, where applicable. Control joints should bisect the length and width of the slab at intervals specified by the American Concrete Institute (ACI) or Portland Concrete Association (PCA).
7. Sidewalks, may be placed directly on compacted fill without the use of a baserock section. For exterior slabs, the native soil should be ripped, moisture conditioned and recompacted to an 8-inch depth per the grading recommendations presented in this report.
8. All deleterious material must be removed prior to placing concrete.
9. We recommend that concrete have a water/cement ratio no greater than 0.45. Pozzolans or other additives may be added to increase workability.

10. Concrete slabs should be moisture cured for at least seven days after placement. Excessive curling of the slab may occur if moisture conditioning is not performed. This is especially critical for slabs that are cast during the warm summer months.
11. Concrete slabs impart a relatively small load on the subgrade (approximately 50 psf). Therefore, some vertical movement should be anticipated from possible expansion or differential loading.

5.2.3 Rock Anchors

1. Rock anchors or doweling may be used to provide lateral and uplift resistance where shallow, competent rock limits excavation. Although bedrock was not encountered in our exploratory borings, very large boulders were encountered which would provide sufficient support for rock anchors. Rock anchors should only be installed in competent rock, to be determined in the field by a representative of Antares. The design of rock anchors should include the following criteria.
2. Pull-out resistance for rock anchors will generally be limited by the shear resistance between the grout and the native rock. For design purposes, a pull-out resistance of 50 pounds per square inch of grout/competent rock contact may be used. Because of the strain in the anchor steel during pull-out, we recommend that the upper 6 inches of grout/competent rock contact be neglected when sizing for uplift.
3. For grouted anchors, we recommend that the drilled hole have a minimum ½-inch annular clearance between the steel and surrounding rock. Thus, grouting a No. 4 rebar would require a 1½-inch diameter hole.
4. Lateral shear resistance for rock anchors should be designed using Vs=0.45 Fy, where Fy equals the tensile strength of the steel. To develop this shear resistance, a minimum steel embedment of 24 inches into undisturbed, competent rock should be used.
5. Prior to anchor placement, loose debris, dust, and standing water in the hole must be removed by blowing with oil-free compressed air, cleaning the hole with a nylon brush, and then blowing out the remaining dust. Dust and debris left in the hole will significantly reduce anchor capacity.
6. We recommend using a cement grout that has a water/cement ratio of less than 0.6 to construct rock anchors. If high strength epoxy or other adhesives are proposed, Antares should review the proposed rock anchor detail prior to construction. If epoxy will be used for anchors, anchors shall be installed per manufacturer’s recommendations and submitted to Antares for review.

7. If rock anchors are used on more than 10 percent of the foundation system of any given structure, a representative of Antares should observe grout/epoxy placement, and perform pull tests on select anchors.

5.2.1 Bearing Criteria

Based on observations of our exploratory borings, we recommend that foundation elements be designed based on an allowable bearing capacity of 4,000psf at a depth of 36 inches below the ground surface. This value can be increased by 200psf for each additional foot of embedment up to a limiting value of 5,000psf. Allowable bearing may be increased by 33 percent for additional transient loading such as wind or seismic. If foundations land on competent bedrock or boulders assumed to be a minimum of 6 feet in diameter, they may be designed based on a bearing capacity of 12,000psf.

A triangularly-distributed lateral resistance (passive soil resistance) of 350d psf, where d is footing depth, may be used for footings. This value may be increased by 33 percent for wind and seismic. As an alternative to the passive soil resistance described above, a coefficient of friction for resistance to sliding of 0.35 may be used

5.2.2 Retaining Wall Design Criteria

The following active and passive pressures are for retaining walls in cut native soil or backfilled with granular onsite soil. If import soil is used, a representative from our firm should be retained to observe and test the soil to determine its strength properties. The pressures exerted against retaining walls may be assumed to be equal to a fluid of equivalent unit weight.

The table below presents equivalent fluid unit weights for cut native soil and onsite fill compacted per the grading recommendations presented in this report. For approximately horizontal backfill we assume that the retained fill surface will be no steeper than 10% for a minimum distance of the wall height from the back of the retaining wall. If surcharge loads (such as adjacent building foundations) or live loads will be applied within a distance of the wall height from the back of the wall, we should be retained to review the loading conditions and revise our recommendations, if necessary.

Table 5.2.4.1 - Equivalent Fluid Unit Weights [(1)]		
Loading Condition	Retained Cut or Compacted Fill (approximately horizontal backfill)	Retained Cut or Compacted Fill (retained slope up to 1:1, H:V)
Active Pressure (pcf)	30	40
Passive Pressure (pcf)	350	350
At-Rest Pressure (pcf)	50	60
Coefficient of Friction	0.35	0.35

Note: (1) The equivalent fluid unit weights presented are ultimate values and do not include a factor of safety. The passive pressures provided assume footings are founded in competent native soil or engineered fill.

Please note that the use of the tabulated active pressure unit weight requires that the wall design accommodate sufficient deflection for mobilization of the retained soil to occur. Typically, a wall yield of less than 1 percent of the wall height is sufficient to mobilize active conditions in granular soil. However, if the walls are rigid or restrained to prevent rotation, at-rest conditions should be used for design.

Recommendations for design and construction of retaining walls are listed below:

1. Compaction equipment should not be used directly adjacent to retaining walls unless the wall is designed or braced to resist the additional lateral pressures.
2. If any surface loads are closer to the top of the retaining wall than its height, Antares should review the loads and loading configuration. We should be retained to review wall details and plans for any wall over 4 feet in height.
3. Retaining walls over 4 feet in height are required to be engineered. We can provide these services if requested.
4. All retaining walls must be well drained to reduce hydrostatic pressures. Walls should be provided with a drainage blanket to reduce additional lateral forces and minimize saturation of the backfill soil. Drainage blankets may consist of graded rock drains or geosynthetic blankets.
5. Rock drains should consist of a minimum 12-inch wide, Caltrans Class II, permeable drainage blanket, placed directly behind the wall; or crushed washed rock enveloped in a non-woven geotextile filter fabric such as Amoco 4546™ or equivalent. Drains should have a minimum 4-inch diameter, perforated, schedule 40, PVC pipe placed at the base of the wall, inside the drainrock, with the perforations placed down. The PVC pipe

should be sloped so that water is directed away from the wall by gravity. A geosynthetic drainage blanket such as Enkadrain™ or equivalent may be substituted for the rock drain, provided the collected water is channeled away from the wall. If a geosynthetic blanket is used, backfill must be compacted carefully so that equipment or soil does not tear or crush the drainage blanket.

6. Adequate drainage and waterproofing for retaining walls associated with finished interior spaces are essential to reduce the likelihood of seepage and vapor transmission into the living space. We recommend that an appropriate waterproofing sealant be applied to the exterior surface of such retaining walls. A waterproofing consultant may be contacted to further review seepage and vapor transmission.

7. Additional lateral loading on retaining structures due to seismic accelerations may be considered at the designer's option. For an earthquake producing a design horizontal acceleration of 0.2g, we recommend that the resulting additional lateral force applied to unrestrained (cantilevered) retaining structures with drained level backfill onsite be estimated as $P_{ae}=9H^2$ pounds, where H is the height of the wall in feet. The additional seismic force may be assumed to be applied at a height of 0.6H above the base of the wall. This seismic loading is for a drained, level backfill condition only; Antares should be consulted for values of seismic loading due to non-level or non-drained backfill conditions. The use of reduced factors of safety is often appropriate when reviewing overturning and sliding resistance during seismic events.

6 LIMITATIONS

The following limitations apply to the findings, conclusions and recommendations presented in this report:

1. Our professional services were performed consistent with the generally accepted geotechnical engineering principles and practices employed in southern California. No warranty is expressed or implied.
2. These services were performed consistent with our agreement with our client. We are not responsible for the impacts of any changes in environmental standards, practices, or regulations subsequent to performance of our services. We do not warrant the accuracy of information supplied by others, or the use of segregated portions of this report. This report is solely for the use of our client unless noted otherwise. Any reliance on this report by a third party is at the party's sole risk.
3. If changes are made to the nature or design of the project as described in this report, then the conclusions and recommendations presented in this report should be considered invalid. Only our firm can determine the validity of the conclusions and recommendations presented in this report. Therefore, we should be retained to review all project changes and prepare written responses with regards to their impacts on our conclusions and recommendations. However, we may require additional fieldwork and laboratory testing to develop any modifications to our recommendations. Costs to review project changes and perform additional fieldwork and laboratory testing necessary to modify our recommendations are beyond the scope of services presented in this report. Any additional work will be performed only after receipt of an approved scope of services, budget, and written authorization to proceed.
4. The analyses, conclusions and recommendations presented in this report are based on site conditions as they existed at the time we performed our surface and subsurface field investigations. We have assumed that the subsurface soil and groundwater conditions encountered at the location of our exploratory trenches are generally representative of the subsurface conditions throughout the entire project site. However, the actual subsurface conditions at locations between and beyond our exploratory trenches may differ. Therefore, if the subsurface conditions encountered during construction are different than those described in this report, then we should be notified immediately so that we can review these differences and, if necessary, modify our recommendations.
5. The elevation or depth to groundwater underlying the project site may differ with time and location.
6. The project site map shows approximate exploratory trench locations as determined by pacing distances from identifiable site features. Therefore, the trench locations should not be relied upon as being exact nor located with surveying methods.
7. Our geotechnical investigation scope of services did not include evaluating the project site for the presence of hazardous materials or mining features. Although we did not observe evidence of hazardous materials or mining features within the proposed building

area at the time of our field investigation, all project personnel should be careful and take the necessary precautions should hazardous materials and/or mining features be encountered during construction.

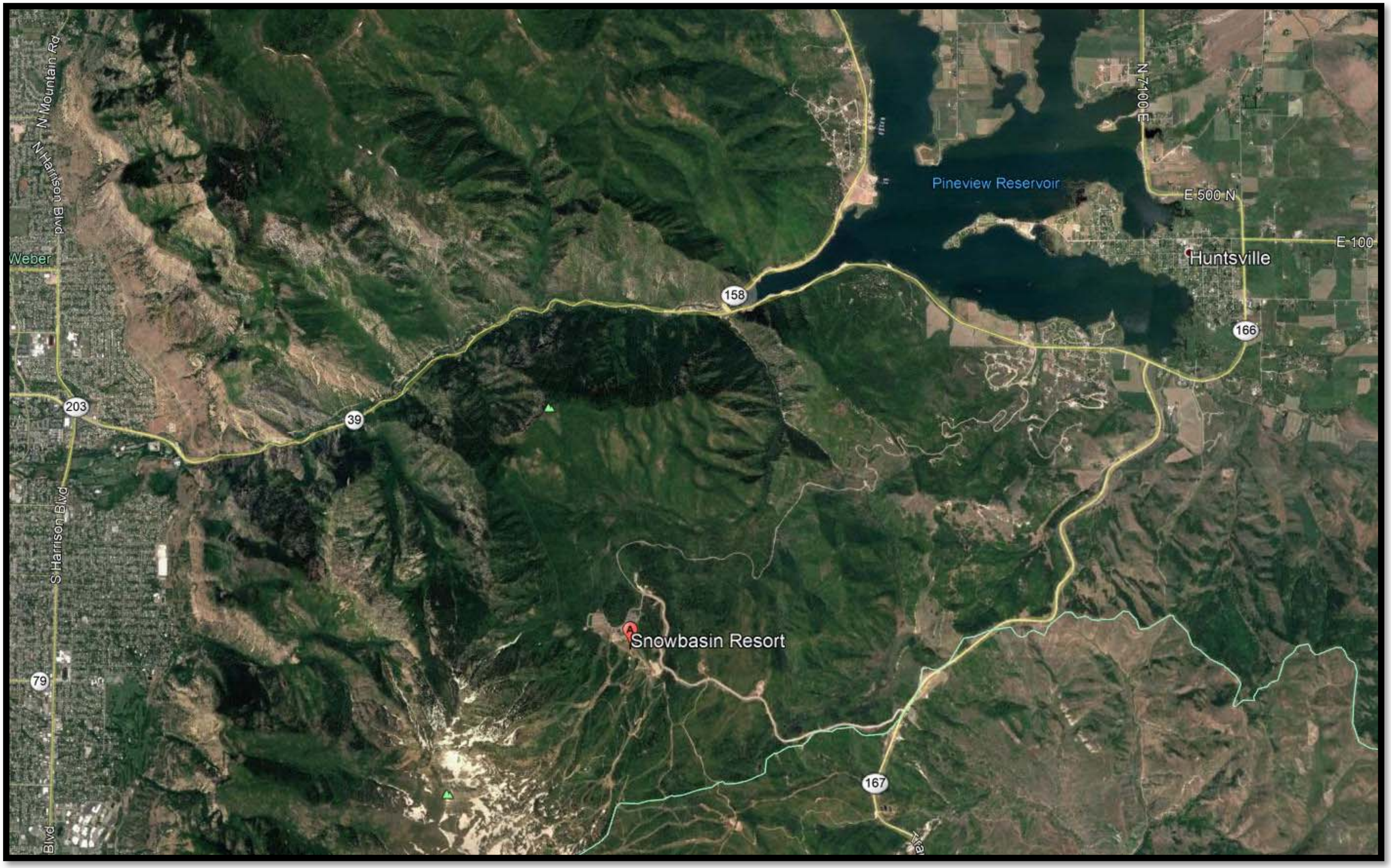
8. The findings of this report are valid as of the present date. However, changes in the conditions of the property can occur with the passage of time. The changes may be due to natural processes or to the works of man, on the project site or adjacent properties. In addition, changes in applicable or appropriate standards can occur, whether they result from legislation or the broadening of knowledge. Therefore, the recommendations presented in this report should not be relied upon after a period of two years from the issue date without our review.

FIGURES

APPENDIX A PROPOSAL

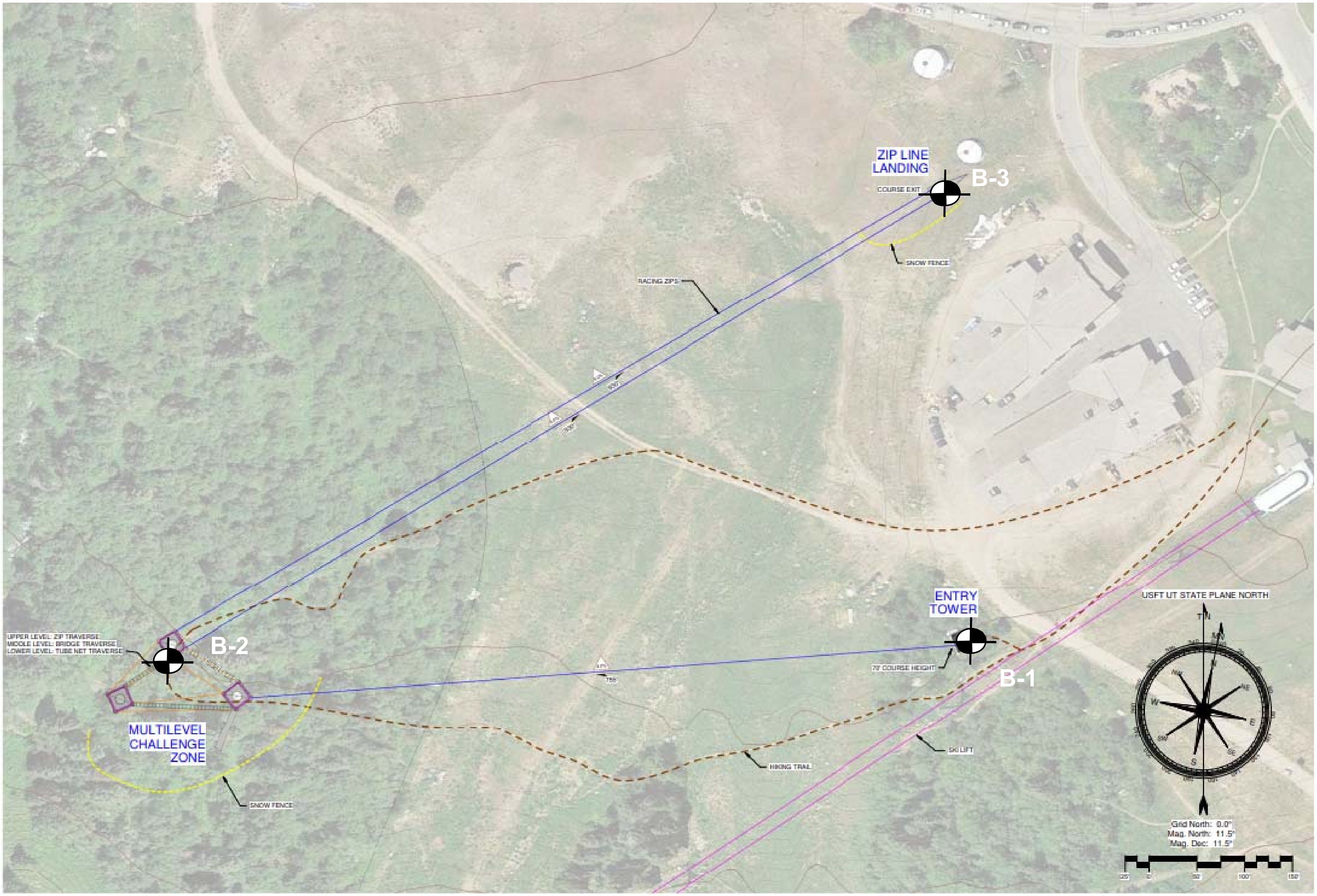
APPENDIX B IMPORTANT INFORMATION ABOUT YOUR GEOTECHNICAL ENGINEERING REPORT (Included with permission of ASFE, Copyright 2004)

APPENDIX C EXPLORATORY TRENCH LOGS



Google Earth ©2018

ANTARES, LLC	**Vicinity Map** Snowbasin Huntsville, Utah	PROJECT NO. 18109-01 August 16, 2018 FIGURE 1



ANTARES, LLC

Exploratory Trench Location Map
Snowbasin
Huntsville, Utah

PROJECT NO. 18109-01

August 16, 2018

FIGURE 2