Geological Hazard and Geotechnical Investigation
Lot 5R, Powder 11 at Powder Mountain Subdivision
Weber County, Utah

February 10, 2021

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Geological Hazard and Geotechnical Investigation
Proposed Single Family Residence
6615 North Powder Mountain Road
Lot 5R, Powder 11 at Powder Mountain Subdivision
Weber County, Utah
CG Project No.: 153-006

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1.0 INTRODUCTION

1.1 PURPOSE AND SCOPE OF WORK

This report presents the results of a geological hazards evaluation and a geotechnical investigation that were performed for Lot 5R, Powder 11 at Powder Mountain Subdivision, located at 6615 North Powder Mountain Road in the Powder Mountain Resort area of Weber County, Utah. The site is located as shown on Plate1, Vicinity Map. The proposed development parcel consists of an irregularly shaped parcel that includes an approximately 0.31-acre residential property. Plate 2, Site Plan, provides aerial coverage of the site and details of the current (2018) layout of the site vicinity.

In general, the purposes of this investigation were to provide a site-specific geological hazards study and a geotechnical engineering evaluation to support the proposed site development. The geological hazards study was conducted to evaluate the site relative to potential geologic hazards as outlined in the Weber County Code, Section 108-22 Natural Hazard Areas. These hazards include, but are not limited to: Surface-Fault Ruptures, Landslide, Tectonic Subsidence, Rock Fall, Debris Flows, Liquefaction Areas, Flood, or other Hazardous Areas (Weber County Code, 2020). The geotechnical engineering evaluation was conducted to evaluate the subsurface conditions and the nature and engineering properties of the subsurface soils and rock, to evaluate slope stability conditions, and to provide recommendations for general site grading and for the design and construction of floor slabs and foundations. This investigation included subsurface exploration, representative soil sampling, field and laboratory testing, and engineering analysis.

1.2 PROJECT DESCRIPTION

Based on conversations with our client, we understand that the proposed construction on the lot is to consist of a single-family residence. The plans and layout for the proposed residence have not been finalized at this time; however, we expect the structure to be one to two stories in height with a basement level. The footing loads for the proposed structure are anticipated to be on the order of 3 to 4 klf for walls and 150 psf for floors. If the actual structural loads are different from those anticipated, Christensen Geotechnical should be notified in order to reevaluate our recommendations.

1.3 WEBER COUNTY GEOLOGIC HAZARDS REGULATIONS

Due to the steep slopes on the lot and mapped landslides in the vicinity of the site, Weber County requires that a geological site reconnaissance be performed to assess whether all or parts of the site and the proposed improvements are exposed to the hazards that are included in the Weber County Code, Section 108-22 Natural Hazard Areas. These hazards include, but are not limited to: Surface-

Fault Ruptures, Landslide, Tectonic Subsidence, Rock Fall, Debris Flows, Liquefaction Areas, Flood, or other Hazardous Areas.

The purpose of the Geological Site Reconnaissance and Review is to evaluate whether the proposed development is outside or within areas identified as Natural Hazards Area, and, if within a hazard area, to recommend appropriate additional studies that comply with the purpose and intent of the Weber County Natural Hazards Area guidelines and standards in order to be "cleared" for building permit issuance by the county, as outlined by the Weber County Development Process packet provided by the Weber County Building Inspection Department (2020).

1.4 SCOPE OF WORK

The objectives and scope of this study were presented to Mr. Carson Young (**Client**) through verbal communication.

2.0 METHODS OF STUDY

2.1 LITERATURE AND RESOURCE REVIEW

The site conditions and site geology were interpreted through an integrated compilation of data including a review of literature and mapping from previous studies conducted in the area (Bryant, 1988; King and McDonald, 2014; Coogan and King, 2016; and McDonald, 2020); a photogeologic analyses of 2012 and 2018 orthorectified imagery shown on Plate 2 and Plate 5; a review of historical stereoscopic 1:22.000 scale imagery flown in 1952 (frames AAI_3K-132 and AAI_3K-133); a review of Google Earth® imagery sequence of the site between the dates of 1993 and 2020; a GIS analysis of elevation and geoprocessed LiDAR terrain data as shown on Plate 4 LiDAR Analysis; a field reconnaissance of the general site area; and the interpretation of the test pits excavated on the site as part of our field program, located as shown on Plate 5, Site Evaluation. Seismic hazards information was developed from United States Geologic Survey (USGS) databases (Peterson and others, 2014).

2.2 FIELD INVESTIGATION

The site was reconnoitered December 8, 2020 by Senior Geologist Dr. Gregory Schlenker, P.G. and Senior Engineer Mr. Mark Christensen, P.E. During the field reconnaissance, the mapped site geological conditions were confirmed and/or amended as shown on Plate 5.

The subsurface conditions at the site were explored by excavating and logging a walk-in test pit that was excavated to bedrock refusal at a depth of 2 feet below the existing site grade, as well as the inspection of a rock outcrop on the north side of the site. The test pit and outcrop locations are shown on Plate 5. The subsurface conditions as encountered in the test pit were recorded at the time of excavation and are presented on the attached Plate 6, Test Pit Log.

The test pit was excavated using an approximately 18-ton trackhoe excavator. Due to the granular nature of the subgrade soils and rock, only disturbed samples were collected from the test pit sidewall at the time of excavation. The disturbed samples were collected and placed in bags and buckets. The samples were visually classified in the field and portions of each sample were packaged and transported to our laboratory for testing. The classifications for the individual soil units are shown on the attached Plate 6, Test Pit Log.

2.3 LABORATORY TESTING

Of the soils collected during the field investigation, representative samples were selected for testing in the laboratory in order to evaluate the pertinent engineering properties. The laboratory testing included a moisture content determination, Atterberg limits evaluations, and a gradation analysis. A summary of our laboratory testing is presented in the table below:

Table No. 1: Laboratory Test Results

TEST HOLE NO.	DEPTH (ft.)	NATURAL DRY DENSITY (pcf)	NATURAL MOISTURE (%)	ATTERBERG LIMITS		GRAIN SIZE DISTRIBUTION (%)			SOIL TYPE
				LIQUID LIMIT	PLASTICITY INDEX	GRAVEL (+ #4)	SAND	SILT/ CLAY (- #200)	
TP-1	1		7.9	NP	NP	51.7	23.0	25.3	GM

The results of our laboratory tests are also presented on the Test Pit Log, Plate 6, and more detailed laboratory results are presented on the laboratory testing plate, Plate 8.

Samples will be retained in our laboratory for 30 days following the date of this report, at which time they will be disposed of unless a written request for additional holding time is received prior to the disposal date.

2.4 SLOPE ANALYSIS

The elevation data for the site consisted of 2016 geoprocessed 0.5-meter bare earth LiDAR imagery data which was obtained from Utah Automated Geographic Reference Center (AGRC). These data were geoprocessed with the QGIS® GIS platform; and using the r.slope, r.shaded.relief and r.contour.level GRASS® (Geographic Resources Analysis Support System) modules, the slope percentages, surface renderings and elevation contours were calculated for the site area. Additionally, the referenced historical aerial photography was georeferenced to the GIS layering to support our geological interpretation of the site.

Plate 4, LiDAR Analysis, presents the results of the LiDAR slope analysis. Shown on Plate 4 are the slope percentage gradients over a rendered shaded relief surface. The surface of the site is shown to slope moderately to the south. The limiting steep slope gradient for development considerations according to the Weber County Code is 25-percent (Weber County Code, 2020).

3.0 GENERAL SITE CONDITIONS

3.1 SURFACE CONDITIONS

As shown on Plate 2, the site is an approximately 0.31-acre residential property that is currently vacant and undeveloped. Elevations across the site range from a low of 8176 feet on the south side to 8230 feet on the north side of the site. The surface of the site generally consists of moderately steep mountain terrain, with the surface generally sloping downward to the south. The surface vegetation consists of open areas of grasses with weeds and sage brush and with clusters of aspen trees. At the time of our reconnaissance, adjacent properties were observed to be occupied with single-family residences similar to the one proposed.

3.2 SUBSURFACE CONDITIONS

3.2.1 Soil and Rock

Based on the test pit that was completed for this investigation, the subsurface materials at the site consist of approximately 1 foot of topsoil overlying Silty GRAVEL with sand (GM). The Silty GRAVEL with sand (GM) was medium dense and extended to a depth of approximately 2 feet. Below the gravel soils, quartzite bedrock was exposed. The bedrock was slightly weathered, strong, and highly fractured. The trackhoe that was used for the excavation experienced excavation refusal at a depth of 2 feet below existing site grade.

The outcrop location on the north side of the site exposed Mutual Formation (**Zm**) rocks, which are described by Coggan and King (2016) as: Neoproterozoic age, grayish-red to purplish-gray, medium to thick-bedded quartzite with pebble conglomerate lenses; also reddish-gray, pink, tan, and light-gray in color…contains argillite beds...2556 feet thick in James Peak quadrangle (Coogan and King, 2016). The outcrop of rocks on the site were observed to be pale-orange, slightly weathered, very strong, with bedding locally observed to be massive (>1.0 feet) with beds irregularly spaced generally striking northwest southeast and dipping 44° northeastward. Vertical and near-horizontal jointing and fracturing persists, with discontinuities spaced one- to six-inches, and filled with calcite and competent fines.

3.2.2 Groundwater

Groundwater was not encountered within the test pit at the time of excavation. It should be understood that groundwater may fluctuate in response to seasonal changes, precipitation, and irrigation.

4.0 GEOLOGIC CONDITIONS

4.1 GEOLOGIC SETTING

The site is located on the southern end of the Bear River Range of Utah and Idaho, which is a mountain system that is flanked on its eastern side by the Monte Cristo Range, and on the west by Ogden Valley (Avery, 1994). The vicinity of the site is mountainous terrain which was formed by the eastward extension of the Willard Thrust sheet, which is believed to have moved onto the vicinity during the Cretaceous Sevier orogeny, approximately 140 million years ago (ma). The thrust sheet rocks consist of older Paleozoic-aged rocks (500-350 ma) that have experienced significant folding and faulting, and are now covered in parts by more gently folded to horizontally bedded Tertiary-aged (65-35 ma) rocks at the surface (Coogan and King, 2016). Regional uplift during the Laramide orogeny between 70 and 40 ma gave rise to the area, and more recently, movement along high-angle faults along the Wasatch fault during the late Tertiary and Quaternary age (Bryant, 1988). These resultant mountainous terrains have subsequently been modified by Quaternary age glacial erosion and deposition, and fluvial landscape incision and erosion; with localized late-Quaternary stream deposition, residual soil weathering and development, and mass movement processes on the surface (King and McDonald, 2014; Coogan and King, 2016; and McDonald, 2020).

The site is located to the east of Ogden Valley, which is on the east side of the Wasatch Range, the western side of which is the Wasatch Front, marked by the Wasatch fault. The Wasatch fault is approximately 8.6 miles west of the site, and provides the basis of division between the Middle Rocky Mountain Physiographic Province on the east and the Basin and Range Physiographic Province on the west. The Basin and Range Physiographic Province is characterized by approximately north-south trending valleys and mountain ranges that have been formed by extensional tectonics and displacement along normal faults and extends from the Wasatch Range on the east to the Sierra Nevada Range on the west (Hunt, 1967).

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

Topographically, the site is located on gently eastward dipping ridge-crest slopes near the headwaters of Wolf Creek, a tributary of the North Fork of the Ogden River. The topography of the site and vicinity consists of gently and moderately sloping plateau and mountain peak surfaces surrounded by steeply incised drainages. The elevation of the Lot 5R Property is roughly 8200 feet. Slope mapping on Plate 4 illustrates the slope conditions and the topographic relief for the site vicinity, and shows that the property is situated primarily on a moderately steep south-facing slope.

4.2 SURFICIAL GEOLOGY

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

> **Qh -** Human disturbances (Historical) Disturbances that obscure original deposits or rocks by cover or removal...
>
> **Qmc -** Landslide and colluvial deposits, undivided (Holocene and Pleistocene) – Poorly sorted to unsorted clay- to boulder-sized material...(slopewash and soil creep)...These deposits are as unstable as other landslide units...
>
> **Cbm -** Middle limestone member - Bloomington Formation (Middle Cambrian) - Dark to medium-gray, thick- to thin-bedded, argillaceous limestone with tan-, yellow-, and red-weathering, wavy, silty layers and partings; contains subordinate olive-gray and tan-gray, thin-bedded, shale and micaceous argillite...
>
> **Cbh -** Hodges Shale Member - Bloomington Formation (Middle Cambrian) - Brown-weathering, slope-forming, olive-gray to tan-gray, thin-bedded, shale and micaceous argillite, and thin- to thick-bedded, dark- to medium-gray limestone with tan-, yellow-, and red-weathering, wavy, silty layers and partings...
>
> **Cbk -** Blacksmith Formation (Middle Cambrian) – Typically medium-gray, very thick to thick-bedded, dolomite and dolomitic limestone with tan-weathering, irregular silty partings to layers...
>
> **Cu -** Ute Formation (Middle Cambrian) – Interbedded gray thin- to thick-bedded limestone with tan-, yellowish-tan-, and reddish-tan-weathering, wavy, silty layers and partings, and olive-gray to tan-gray, thin-bedded shale and micaceous argillite; and minor, medium-bedded, gray to light-gray dolomite...
>
> **Cl -** Langston Formation (Middle Cambrian) – Upper part is gray, sandy dolomite and limestone that weathers to ledges and cliffs; middle part is yellowish- to reddish-brown to gray weathering, greenish-gray, fossiliferous shale and lesser interbedded gray, laminated to very thin-bedded, silty limestone...

> **Cgc -** Geertsen Canyon Quartzite (Middle and Lower Cambrian and possibly Neoproterozoic) – In the west mostly buff (off-white and tan) quartzite, with pebble conglomerate beds; pebbles are mostly rounded light colored quartzite; contains cross bedding, and pebble layers and lenses; colors vary from tan and light to medium gray, with pinkish, orangish, reddish, and purplish hues...
>
> **Zm -** Mutual Formation (Neoproterozoic) – Grayish-red to purplish-gray, medium to thick-bedded quartzite with pebble conglomerate lenses; also reddish-gray, pink, tan, and light-gray in color…contains argillite beds...

The Lot 5R site location is mapped Neoproterozoic **Zm -** Mutual Formation, a quartzite bedrock unit which dips steeply to the east.

4.3 SEISMIC HAZARDS/CHARACTERIZATION

4.3.1 Strong Ground Motion

Strong ground motion originating from the Wasatch fault or other nearby seismic sources is capable of impacting the site. The Wasatch fault zone is considered active and capable of generating earthquakes as large as magnitude 7.3 (Arabasz and others, 1992). Based on probabilistic estimates (Peterson and others, 2014) queried for the site (41.3791º N -111.7846º E), 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 0.17g, and from an earthquake with a two-percent probability of exceedance in 50 years, as high as 0.37g.

The ten-percent probability of exceedance in 50 years event has a return period of 475 years. The 0.17g acceleration for this event corresponds "strong" perceived shaking with "light" potential damage based on instrument intensity correlations. The two-percent probability of exceedance in 50 years event has a return period of 2475 years, and the 0.37g acceleration for this event corresponds with "severe" perceived shaking with "moderate to heavy" potential damage based on instrument intensity correlations (Wald and others, 1999).

4.3.2 Active Earthquake Faults

Based upon our review of available literature and mapping, no active Holocene-aged faults are known to pass through or are immediately adjacent to the site. The nearest active (Holocene) earthquake fault to the site is the Weber segment of the Wasatch fault zone (UT2351E), which is located 8.6 miles west of the site; thus, active surface fault rupture hazards are not considered present (Black and others, 2004).

4.3.3 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 the liquefaction potential of a soil deposit are: (1) magnitude and duration of seismic ground motions, (2) soil type and consistency, and (3) occurrence and depth to groundwater.

Liquefaction potential hazards have not been studied or mapped for the Ogden Valley area, as has occurred in other parts of northern Utah (Anderson and others, 1994). Liquefaction more commonly occurs in saturated non-cohesive finer-grained soils such as floodplain alluvium and lacustrine sediments (Anderson and others, 1994), which are not mapped for the property; consequently, the conditions susceptible to liquefaction do not appear to be present at the site.

4.3.4 Site Seismic Response

The State of Utah and Utah municipalities have adopted the 2018 International Building Code (IBC) for seismic design. The IBC seismic design is based on seismic hazard maps which depict probabilistic ground motions and spectral response; the maps, ground motions, and spectral response having been developed by the United States Geological Survey (USGS). Seismic design values, including the design spectral response, may be calculated for a specific site using the web-based application by the Applied Technology Council (ATC), the project site’s approximate latitude and longitude, and its Site Class. Based on our field exploration, it is our opinion that this location is best described as a Site Class C, which represents a “very dense soil and soft rock” profile. The spectral acceleration values obtained from the ATC web-based application are shown below.

Table 2: IBC Seismic Response Spectrum Values

Site Location: 41.38003^{0} N -111.78402^{0} W	
Name	**Response Spectral Value**
S_S	**0.855**
S_1	**0.298**
S_{MS}	**1.026**
S_{M1}	**0.447**
S_{DS}	**0.684**
S_{D1}	**0.298**
PGA	**0.374**
PGA_M	**0.449**

4.4 ENGINEERING GEOLOGY

The engineering geology findings presented in this section pertain to the natural and geological hazards included in the potential geologic hazards as outlined in the Weber County Code, Section 108-22 Natural Hazard Areas. These hazards include, but are not limited to: Surface-Fault Ruptures, Landslide, Tectonic Subsidence, Rock Fall, Debris Flows, Liquefaction Areas, Flood, or other Hazardous Areas (Weber County Code, 2020).

4.4.1 Landsliding - Slope Stability

The nearest potentially active Holocene landslide units are mapped as **Qms** deposits (Landslide deposits - Holocene and upper and middle? Pleistocene) by Coogan and King, (2016), and are located approximately 4200 feet to the northeast of the site (not shown on Plate 3). The **Qms** deposits are distant and should not potentially impact the site or the proposed improvements.

4.4.2 Sloping Surfaces

The site vicinity slopes developed from our LiDAR analysis range from level to well over 50 percent as shown on Plate 4. The calculated average slope gradient for the 0.31-acre property is 29.4 percent. The threshold gradient for slope development considerations and hillside review according to the Weber County Section 108-14-3 includes slopes greater than 25 percent (Weber County Code, 2020).

4.4.3 Alluvial Fan - Debris Flow Processes

Alluvial fan/debris flow processes include flash flooding and debris flow hazards. The active potential debris flow process deposits nearest to the site are mapped as **Qmdf** deposits (debris- and mud-flow deposits - Holocene and upper and middle? Pleistocene) by Coogan and King, (2016) and are located 4250 feet west of the site along Wolf Creek (not shown on Plate 3), which is located down-gradient of the site, and do not appear to be a potential impact to the site.

4.4.4 Surface Fault Rupture Hazards and Liquefaction

These hazards were discussed previously in Section 4.3 of this report.

4.4.5 Flooding Hazards

No significant waterways pass in the vicinity of the site, and flood insurance rate mapping by Federal Emergency Management Agency for Morgan County classifies the site location as within "Zone X - Area of Minimal Flood Hazard" (FEMA, 2005).

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

4.4.6 Rockfall and Avalanche Hazards

The site is not located directly below rock cliff faces where rockfall hazards may originate, and no indices or set-up conditions for snow avalanche development (Perla and Martinelli, 1976) were observed for the site vicinity during our analysis or reconnaissance of the site.

5.0 ENGINEERING ANALYSIS AND RECOMMENDATIONS

5.1 GENERAL CONLUSIONS

Based on the results of our field and laboratory investigations, it is our opinion that the subject site is suitable for the proposed construction provided that the recommendations contained in this report are incorporated into the design and construction of the project.

5.2 EARTHWORK

5.2.1 General Site Preparation and Grading

Prior to site grading operations, all vegetation, topsoil, undocumented fill soils, and loose or disturbed soils should be stripped (removed) from the building pad and flatwork concrete areas. Following the stripping operations, the exposed soils should be proof rolled to a firm, unyielding condition. Site grading may then be conducted to bring the site to design grade. Where over-excavation is required, the excavation should extend at least 1 foot laterally for every foot of over-excavation. A Christensen Geotechnical representative should observe the site grading operations.

5.2.2 Soft Soil Stabilization

Once exposed through excavation, all subgrade soils should be proof rolled with a relatively large-wheeled vehicle to a firm, unyielding condition. Where localized soft areas are encountered, they should be removed and replaced with granular structural fill. If soft areas extend more than 18 inches deep, or where large areas are encountered, stabilization may be considered. The use of stabilization should be approved by the geotechnical engineer, but would likely consist of over-excavating the area by at least 18 inches and then placing a geofabric (such as Mirafi RS280i) at the bottom of the excavation. Over this, a stabilizing fill, consisting of angular coarse gravel with cobbles, would be placed to the design subgrade.

5.2.3 Temporary Construction Excavations

Based on OSHA requirements and the soil conditions encountered during our field investigation, we anticipate that temporary construction excavations at the site that have vertical walls that extend to depths of up to 5 feet may be occupied without shoring; however, where groundwater or fill soils are encountered, flatter slopes may be required. Excavations that extend to more than 5 feet in depth should be sloped or shored in accordance with OSHA regulations for a type C soil. The stability of construction excavations is the contractor’s responsibility. If the stability of an excavation becomes questionable, the excavation should be evaluated immediately by qualified personnel.

5.2.4 Structural Fill and Compaction

All fill placed for the support of structures and concrete flatwork should consist of structural fill. Structural fill may consist of the native gravel soil with particles larger than 4 inches in diameter removed. Imported structural fill, if required, should consist of a relatively well-graded granular soil with a maximum particle size of 4 inches, with a maximum of 50 percent passing the No. 4 sieve and a maximum of 30 percent passing the No. 200 sieve. The liquid limit of the fines (material passing the No. 200 sieve) should not exceed 35 and the plasticity index should be less than 15. Additionally, all structural fill, whether native soils or imported material, should be free of topsoil, vegetation, frozen material, particles larger than 4 inches in diameter, and any other deleterious materials. Any imported materials should be approved by the geotechnical engineer prior to importing.

The structural fill should be placed in loose lifts that are a maximum of 8 inches thick. The moisture content should be within 3 percent of optimum and the fill should be compacted to at least 95 percent of the maximum density as determined by ASTM D 1557. Where fill heights exceed 5 feet, the level of compaction should be increased to 98 percent.

5.2.5 Excavatability

As indicated in Section 3.2.1, strong bedrock was encountered 0 to 2 feet below present site grades within our test pits. The trackhoe experienced practical equipment refusal at a depth of 2 feet below grade. We anticipate that the minimum equipment required for footing excavations within the bedrock would be the use of a heavy excavator with a ripper tooth or the use of a hoe-ram. Prior to bidding, the contractor should be provided this report in order to be made aware of the subsurface conditions so that they can assess the type of equipment that will be best suited for these conditions.

5.2.6 Permanent Cut and Fill Slopes

Existing slopes should not be over steepened by cutting or filling. We recommend that all non-retained cut and fill slopes be graded no steeper than a 2 to 1 (horizontal to vertical) grade. If steeper grades are required, then retaining structures should be used. We are able to provide retaining walls recommendations if they are desired.

5.3 FOUNDATIONS

The foundations for the planned structure may consist of conventional continuous and/or spread footings established entirely on undisturbed native soil or entirely on bedrock. If the footing excavations expose both soil and bedrock, the bedrock should be over-excavated to allow the placement of at least 12 inches of structural fill. The footings for the proposed structure should be a minimum of 20 inches and 30 inches wide for continuous and spot footings, respectively. Exterior footings should be established at a minimum of 30 inches below the lowest adjacent grade to provide frost protection and confinement. Interior footings not subject to frost should be embedded a minimum of 18 inches for confinement.

Continuous and spread footings that are established on undisturbed native soil, bedrock, or structural fill may be proportioned for a maximum net allowable bearing capacity of 3,500 psf. A one-third increase may be used for transient wind or seismic loads. All footing excavations should be observed by the geotechnical engineer prior to the construction of footings.

5.4 ESTIMATED SETTLEMENT

If the foundations are designed and constructed in accordance with the recommendations presented in this report, there is a low risk that total settlement will exceed 1 inch and a low risk that differential settlement will exceed ½ inch for a 30-foot span.

5.5 LATERAL EARTH PRESSURES

Buried structures, such as basement walls, should be designed to resist the lateral loads imposed by the soils retained. The lateral earth pressures on the below-grade walls and the distribution of those pressures will depend upon the type of structure, hydrostatic pressures, in-situ soils, backfill, and tolerable movements. Basement and retaining walls are usually designed with triangular stress distributions, which are based on an equivalent fluid pressure and calculated from lateral earth pressure coefficients. If soils similar to the native soils are used to backfill basement walls, then the walls may be designed using the following ultimate values:

Table No. 3: Lateral Earth Pressures

Condition	Lateral Pressure Coefficient	Equivalent Fluid Density (pcf)
Active Static	0.27	33
Active Seismic	0.12	14
At-Rest	0.43	51
Passive Static	3.69	443
Passive Seismic	-0.32	-38

We recommend that walls which are allowed little or no wall movement be designed using “at rest” conditions. Walls that are allowed to rotate at least 0.4 percent of the wall height may be designed with “active” pressures. The coefficients and densities presented above assume level backfill with no buildup of hydrostatic pressures. If anticipated, hydrostatic pressures and any surcharge loads should be added to the presented values. If sloping backfill is present, we recommend that the geotechnical engineer be consulted to provide more appropriate lateral pressure parameters once the design geometry is established.

The seismic active and passive earth pressure coefficients provided in the table above are based on the Mononobe-Okabe method and only account for the dynamic horizontal force produced by a seismic event. The resulting dynamic pressure should therefore be added to the static pressure to determine the total pressure on the wall. The dynamic pressure distribution may be approximated as an inverted triangle, with stress decreasing with depth and the resultant force acting approximately 0.6 times the height of the retaining wall, measured upward from the bottom of the wall.

Lateral building loads will be resisted by frictional resistance between the footings and the foundation soils and by passive pressure developed by backfill against the wall. For footings on native soils, we recommend that an ultimate coefficient of friction of 0.45 be used. If passive resistance is used in conjunction with frictional resistance, the passive resistance should be reduced by ½. Passive earth pressure from soils subject to frost or heave should usually be neglected in design.

The coefficients and equivalent fluid densities presented above are ultimate values and should be used with an appropriate factor of safety against overturning and sliding. A value of 1.5 is typically used.

5.6 CONCRETE SLAB-ON-GRADE CONSTRUCTION

Concrete slabs-on-grade should be constructed over at least 4 inches of compacted gravel to help distribute floor loads, break the rise of capillary water, and to aid in the curing process. The gravel should consist of free-draining gravel compacted to a firm, unyielding condition. To help control normal shrinkage and stress cracking, the floor slab should have adequate reinforcement for the anticipated floor loads, with the reinforcement continuous through the interior joints. In addition, we recommend adequate crack control joints to control crack propagation.

5.7 MOISTURE PROTECTION AND SURFACE DRAINAGE

Any wetting of the foundation soils will likely cause some degree of volume change within the soil and should be prevented both during and after construction. We recommend that the following precautions be taken at this site:

1. The ground surface should be graded to drain away from the structures in all directions, with a minimum fall of 8 inches in the first 10 feet.
2. Roof runoff should be collected in rain gutters with downspouts that are designed to discharge well outside of the backfill limits.
3. Sprinkler heads should be aimed away from and placed at least 12 inches from foundation walls.
4. There should be adequate compaction of backfill around foundation walls, to a minimum of 90% density (ASTM D 1557). Water consolidation methods should not be used.

5.8 SUBSURFACE DRAINAGE

Due to the shallow bedrock and the elevation of the subject site, we recommend that all basement walls incorporate a foundation drain. The foundation drain should consist of a 4-inch-diameter slotted pipe placed at or below the bottom of footings and encased in at least 12 inches of free-draining gravel. The gravel should extend up the foundation wall to within 2 feet of the final ground surface, and a filter fabric, such as Mirafi 140N, should separate the gravel from the native soils. The pipe should be graded to drain to the land drains, a storm drain or other free-gravity outfall unless provisions for pumped sumps are made. The gravel which extends up the wall may be replaced by a fabricated drain panel such as Mirafi G200N or equivalent.

5.9 SLOPE STABILITY

Due to the steep slope on the lot, a slope stability analysis was performed. The slope stability assessment was performed using the Slide computer program and the modified Bishop's method of slices. The profile assessed is labeled line *A-A'*, as shown on Plate 5. This profile was based on the subsurface conditions which were encountered within our test pit at the site. The Silty GRAVEL with sand (GM) soils were assumed to have a soil strength consisting of an angle of internal friction of 36 degrees and a cohesion of 50 psf. The bedrock was assumed to have a strength consist of a cohesion of 10,000 psf.

The profile was assessed under static and pseudo static conditions. The pseudo static condition is used to assess the slope during a seismic event. As indicated in Section 4.3.4, the peak ground acceleration at this site is estimated to be 0.449g. As is common practice, half of this value was used in our pseudo static assessments. Minimum factors of safety of 1.5 and 1.1 for static and seismic conditions, respectively, were considered acceptable. Our analyses indicate that the site has safety factors greater than 1.5 and 1.1 for the static and pseudo static conditions and is therefore considered adequate for residential development. The results of our slope stability assessment are shown on Plates 9 and 10.

The slope stability analyses presented above are based on the assumption that no significant cuts or fills will occur during the development of the site. Significant changes to the site grade, such as the steepening of slopes with cuts or fills, may adversely affect the stability of the slopes at the site and may increase the risk of slope failures. If cuts or fills over 10 feet are planned at the site, additional slope stability assessments may be necessary and Christensen Geotechnical should be contacted to provide the additional assessments.

6.0 CONCLUSIONS

Based upon the findings of this study, we believe that the proposed homesite development will not be adversely exposed to the geological hazards addressed in this report. It is our opinion that the buildable area of the site as shown on Plate 5 is suitable for the proposed development from both a geological hazard and a geotechnical engineering perspective. Our conclusion assumes that the proposed construction is to occur as shown on Plate 5, that the geotechnical engineering recommendations provided herein are followed, and that the final site development and grading does not adversely affect the site's slope stability in its present condition.

7.0 LIMITATIONS

The recommendations contained in this report are based on limited field exploration, laboratory testing, and our understanding of the proposed construction. The subsurface data used in this report was obtained from the explorations that were made specifically for this investigation. It is possible that variations in the soil and groundwater conditions could exist between and beyond the points explored. The nature and extent of variations may not be evident until construction occurs. If any conditions are encountered at this site that are different from those described in this report, Christensen Geotechnical should be immediately notified so that we may make any necessary revisions to the recommendations contained in this report. In addition, if the scope of the proposed construction changes from that described in this report, Christensen Geotechnical should be notified.

This report was prepared in accordance with the generally accepted standard of practice at the time the report was written. No other warranty, expressed or implied, is made.

It is the client's responsibility to see that all parties to the project, including the designer, contractor, subcontractors, etc., are made aware of this report in its entirety. The use of information contained in this report for bidding purposes should be done at the contractor's option and risk.

8.0 REFERENCES

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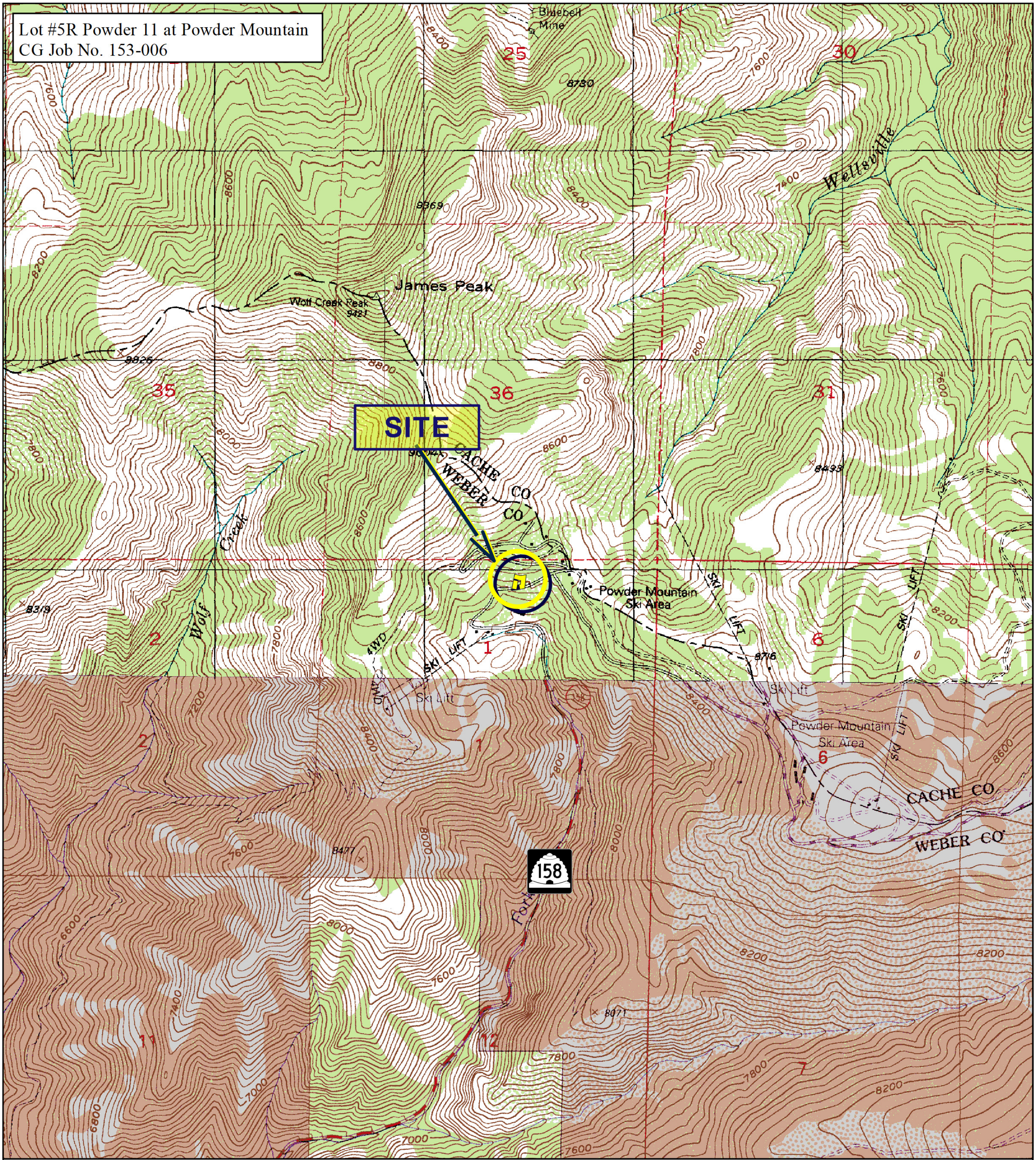
Lot #5R Powder 11 at Powder Mountain
CG Job No. 153-006



Base:
USGS 7.5 Minute topographic maps titled "Huntsville, Utah 1998; and James Peak, Utah 1991" from Utah AGRC;
http://gis.utah.gov/

0 2000 4000 ft

1:24,000

PLATE 1
VICINITY MAP

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Lot #5R Powder 11 at Powder Mountain
CG Job No. 153-006

Base:
2018 0.6m NAIP Orthoimagery
from Utah AGRC; http://gis.utah.gov/

PLATE 2
SITE PLAN

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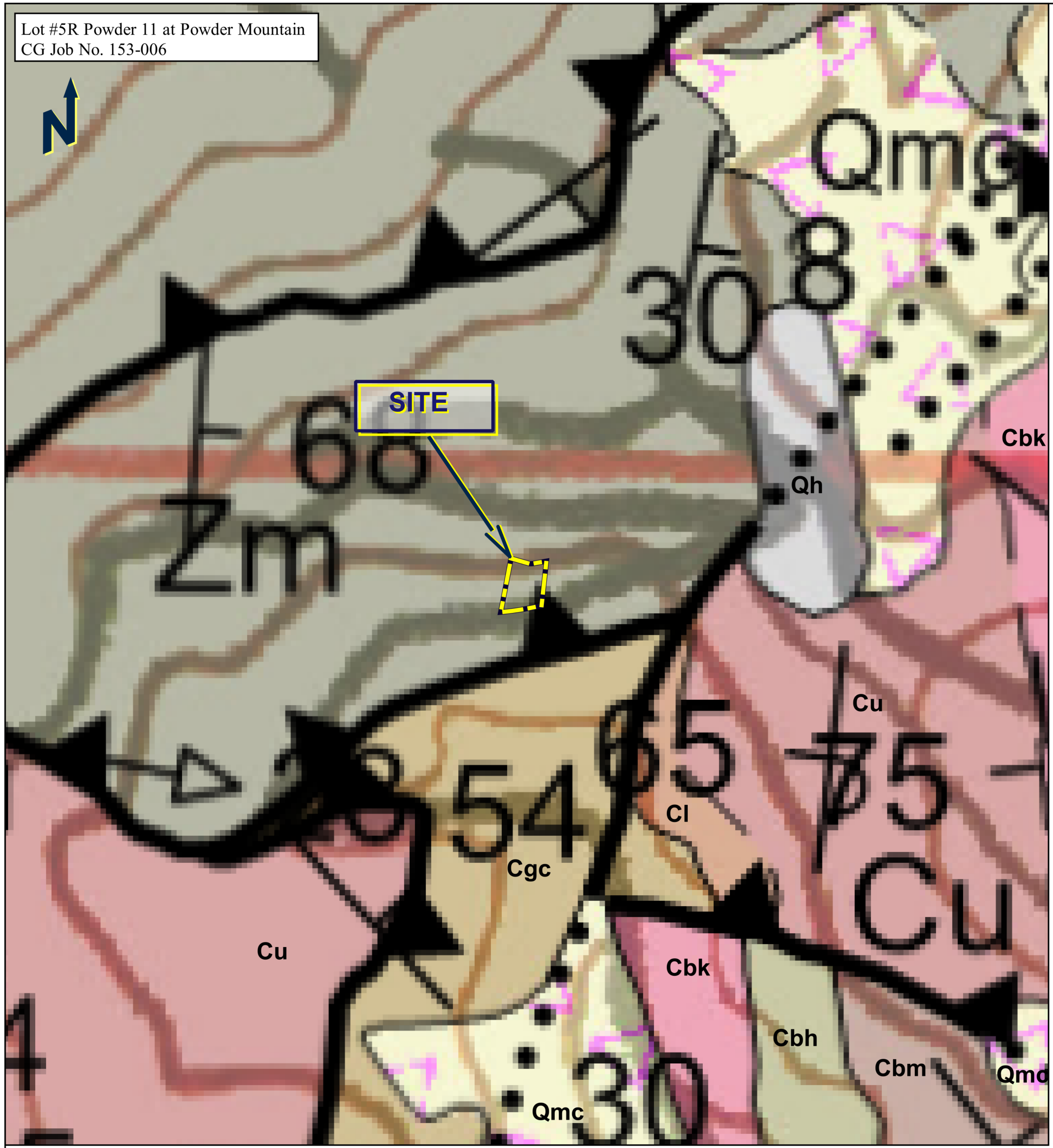
Lot #5R Powder 11 at Powder Mountain
CG Job No. 153-006



Base:
Coogan and King, 2016

GEOLOGCIAL CLASSIFICATION
(after Coogan and King, 2016)

▨ **Qh -** Human disturbances (Historical) Disturbances that obscure original deposits or rocks by cover or removal...

▨ **Qmc -** Landslide and colluvial deposits, undivided (Holocene and Pleistocene) – Poorly sorted to unsorted clay- to boulder-sized material...(slopewash and soil creep)...These deposits are as unstable as other landslide units...

▨ **Cbm -** Middle limestone member - Bloomington Formation (Middle Cambrian) - Dark to medium-gray, thick- to thin-bedded, argillaceous limestone with tan-, yellow-, and red-weathering, wavy, silty layers and partings; contains subordinate olive-gray and tan-gray, thin-bedded, shale and micaceous argillite...

▨ **Cbh -** Hodges Shale Member - Bloomington Formation (Middle Cambrian) - Brown-weathering, slope-forming, olive-gray to tan-gray, thin-bedded, shale and micaceous argillite, and thin- to thick-bedded, dark- to medium-gray limestone with tan-, yellow-, and red-weathering, wavy, silty layers and partings...

▨ **Cbk -** Blacksmith Formation (Middle Cambrian) – Typically medium-gray, very thick to thick-bedded, dolomite and dolomitic limestone with tan-weathering, irregular silty partings to layers...

▨ **Cu -** Ute Formation (Middle Cambrian) – Interbedded gray thin- to thick-bedded limestone with tan-, yellowish-tan-, and reddish-tan-weathering, wavy, silty layers and partings, and olive-gray to tan-gray, thin-bedded shale and micaceous argillite; and minor, medium-bedded, gray to light-gray dolomite...

▨ **Cl -** Langston Formation (Middle Cambrian) – Upper part is gray, sandy dolomite and limestone that weathers to ledges and cliffs; middle part is yellowish- to reddish-brown to gray weathering, greenish-gray, fossiliferous shale and lesser interbedded gray, laminated to very thin-bedded, silty limestone...

▨ **Cgc -** Geertsen Canyon Quartzite (Middle and Lower Cambrian and possibly Neoproterozoic) – In the west mostly buff (off-white and tan) quartzite, with pebble conglomerate beds; pebbles are mostly rounded light colored quartzite; contains cross bedding, and pebble layers and lenses; colors vary from tan and light to medium gray, with pinkish, orangish, reddish, and purplish hues...

▨ **Zm –** Mutual Formation (Neoproterozoic) – Grayish-red to purplish-gray, medium to thick-bedded quartzite with pebble conglomerate lenses; also reddish-gray, pink, tan, and light-gray in color…contains argillite beds...

Normal Fault Bar on downthrown dashed where concealed

Thrust Fault Dashed where covered

Strike and Dip Long bar represents strike axis, numerical value equals dip angle

PLATE 3
GEOLOGIC MAPPING

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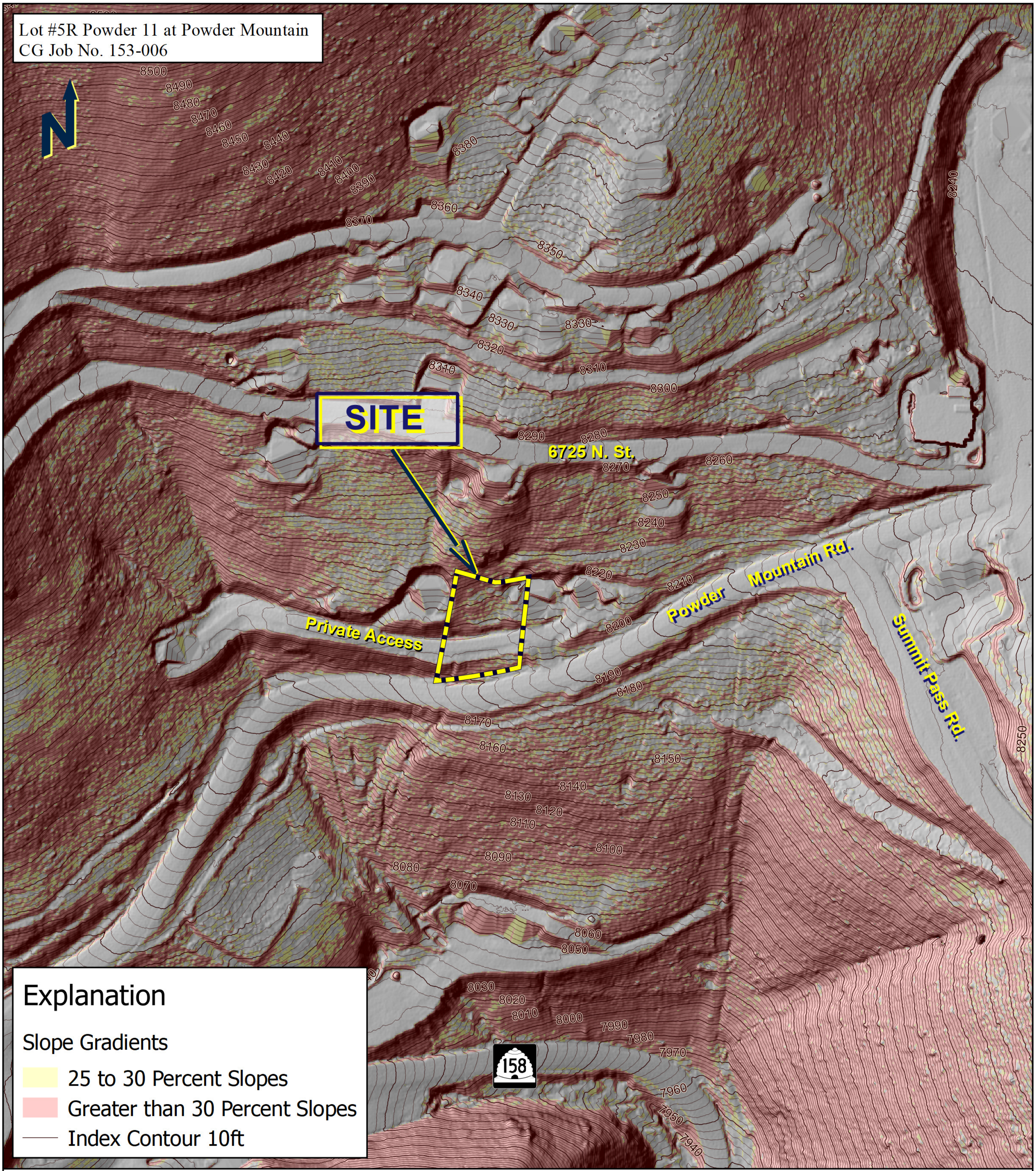

Base:
2016 0.5m LiDAR Imagery,
from Utah AGRC; http://gis.utah.gov/

0 200 400 ft

1:2,400

PLATE 4
LiDAR ANALYSIS

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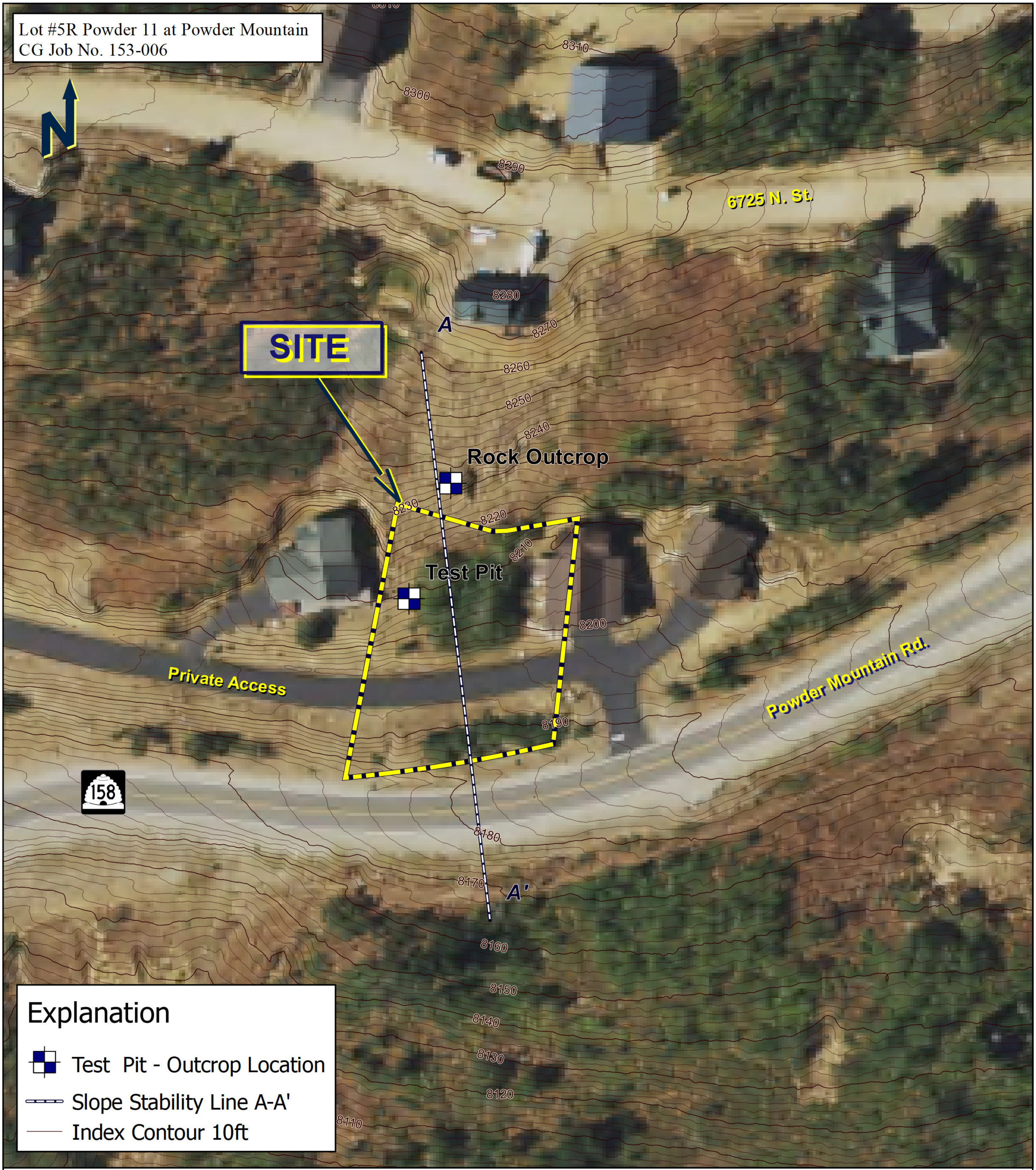
CG Job No. 153-006



Base:
2006 1.0ft HRO Orthoimagery
from Utah AGRC; http://gis.utah.gov/

0 80 160 ft

1:960

PLATE 5
SITE EVALUATION

Christensen
Geotechnical

Date		
Started:	12/10/2021	
Completed:	12/10/2021	
Backfilled:	12/10/2021	

TEST PIT LOG

Logged By: M Christensen
Equipment: Trackhoe
Location: See Plate 2

Test Pit No.

TP-1

Sheet 1 of 1

Depth (feet)	Sample Type	Groundwater	Graphic Log	Group Symbol	Material Description	Dry Density (pcf)	Moisture Content (%)	Minus #200 (%)	Liquid Limit	Plasticity Index
			▓		Topsoil; Silty GRAVEL with sand - moist, dark brown					
		☒	░	GM	Silty GRAVEL with sand - medium dense, moist, brown		7.9	25.3	NP	NP
			▤		Quartzite Bedrock - slightly weathered, strong, highly fractured, brown					
					Refusal on bedrock at 2 feet					
5										
10										
15										

☒ Bulk/Bag Sample
▥ Undisturbed Sample

▼ StabIlized Groundwater
▽ Groundwater At Time of Excavation

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Solitude Builders
Lot 5R Powder 11 at Powder Mountain
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Plate

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RELATIVE DENSITY – COURSE GRAINED SOILS

Relative Density	SPT (blows/ft.)	3 In OD California Sampler (blows/ft.)	Relative Density (%)	Field Test
Very Loose	<4	<5	0 – 15	Easily penetrated with a ½ inch steel rod pushed by hand
Loose	4 – 10	5 – 15	15 – 35	Difficult to penetrate with a ½ inch steel rod pushed by hand
Medium Dense	10 – 30	15 – 40	35 – 65	Easily penetrated 1-foot with a steel rod driven by a 5 pound hammer
Dense	30 – 50	40 – 70	65 – 85	Difficult to penetrate 1-foot with a steel rod driven by a 5 pound hammer
Very Dese	>50	>70	85 - 100	Penetrate only a few inches with a steel rod driven by a 5 pound hammer

CONSISTENCY – FINE GRAINED SOILS

Consistency	SPT (blows/ft)	Torvane Undrained Shear Strength (tsf)	Pocket Penetrometer Undrained Shear Strength (tsf)	Field Test
Very Soft	<2	<0.125	<0.25	Easily penetrated several inches with thumb
Soft	2 – 14	0.125 – 0.25	0.25 – 0.5	Easily penetrated one inch with thumb
Medium Stiff	4 – 8	0.25 – 0.5	0.5 – 1.0	Penetrated over ½ inch by thumb with moderate effort. Molded by strong finger pressure
Stiff	8 – 15	0.5 – 1.0	1.0 – 2.0	Indented ½ inch by thumb with great effort
Very Stiff	15 – 30	1.0 – 2.0	2.0 – 4.0	Readily indented with thumbnail
Hard	>30	>2.0	>4.0	Indented with difficulty with thumbnail

CEMENTATION

Weakly	Crumbles or breaks with handling or little finger pressure
Moderately	Crumbles or breaks with considerable finger pressure
Strongly	Will not crumble or break with finger pressure

MOISTURE

Dry	Absence of moisture, dusty, dry to the touch
Moist	Damp but no visible water
Wet	Visible water, usually below water table

GRAIN SIZE

Description		Sieve Size	Grain Size (in)	Approximate Size
Boulders		>12”	>12”	Larger than basketball
Cobbles		3” – 12”	3” – 12”	Fist to basketball
Gravel	Coarse	3/4” - 3”	3/4” - 3”	Thumb to fist
	Fine	#4 – 3”	0.19 – 0.75	Pea to thumb
Sand	Coarse	#10 - #4	0.079 – 0.19	Rock salt to pea
	Medium	#40 - #10	0.017 – 0.079	Sugar to rock salt
	Fine	#200 - #40	0.0029 – 0.017	Flour to sugar
Silt/Clay		<#200	<0.0029	Flour sized or smaller

STRATAFICATION

Occasional	One or less per foot of thickness
Frequent	More than one per foot of thickness

MODIFIERS

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

STRATIFICATION

Seam	1/16 to 1/2 inch
Layer	1/2 to 12 inch

NOTES

1. The logs are subject to the limitations and conclusions presented in the report.
2. Lines separating strata represent approximate boundaries only. Actual transitions may be gradual.
3. Logs represent the soil conditions at the points explored at the time of our investigation.
4. Soils classifications shown on logs are based on visual methods . Actual designations (based on laboratory testing)may vary.

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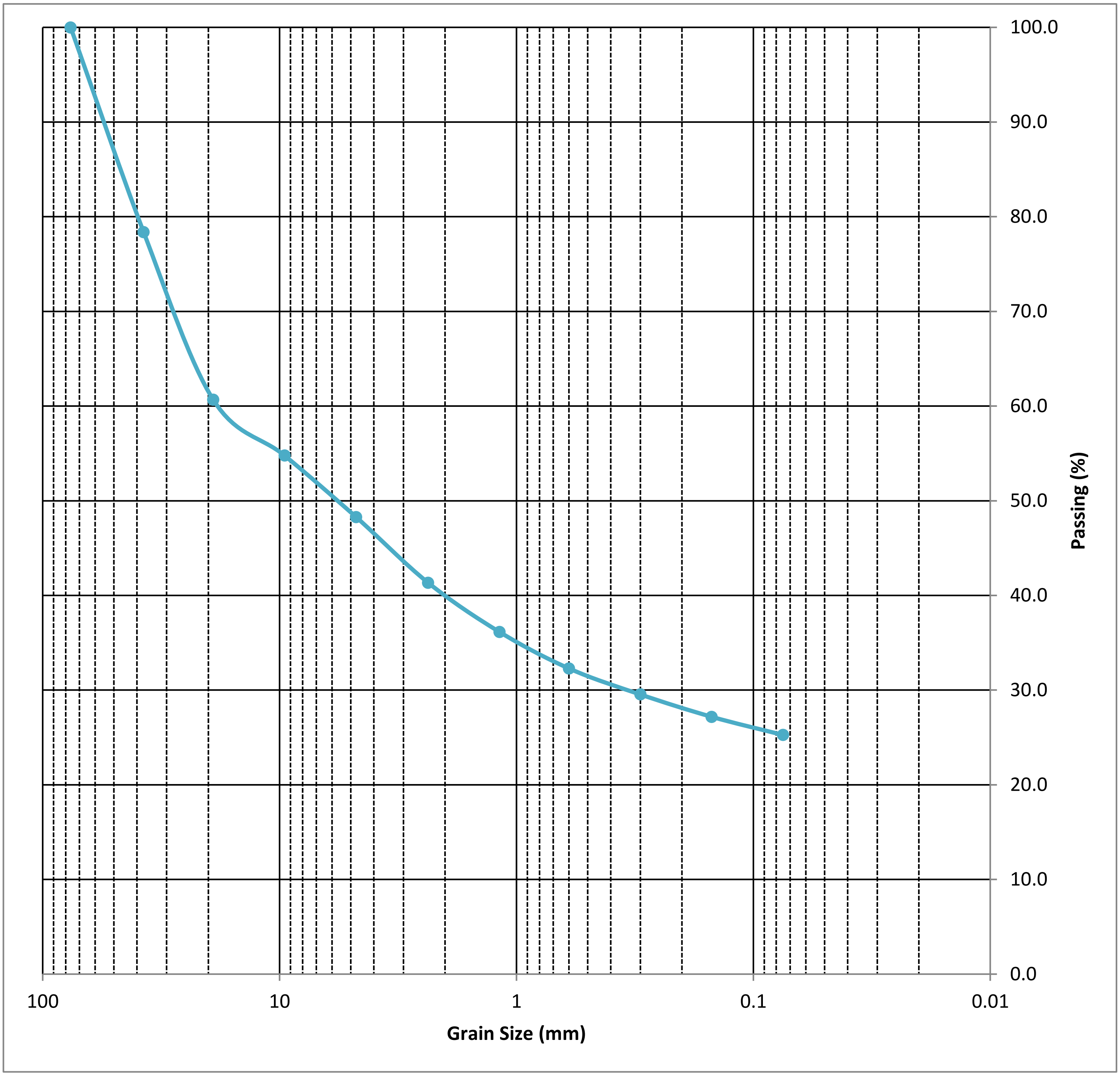
Soil Terms Key

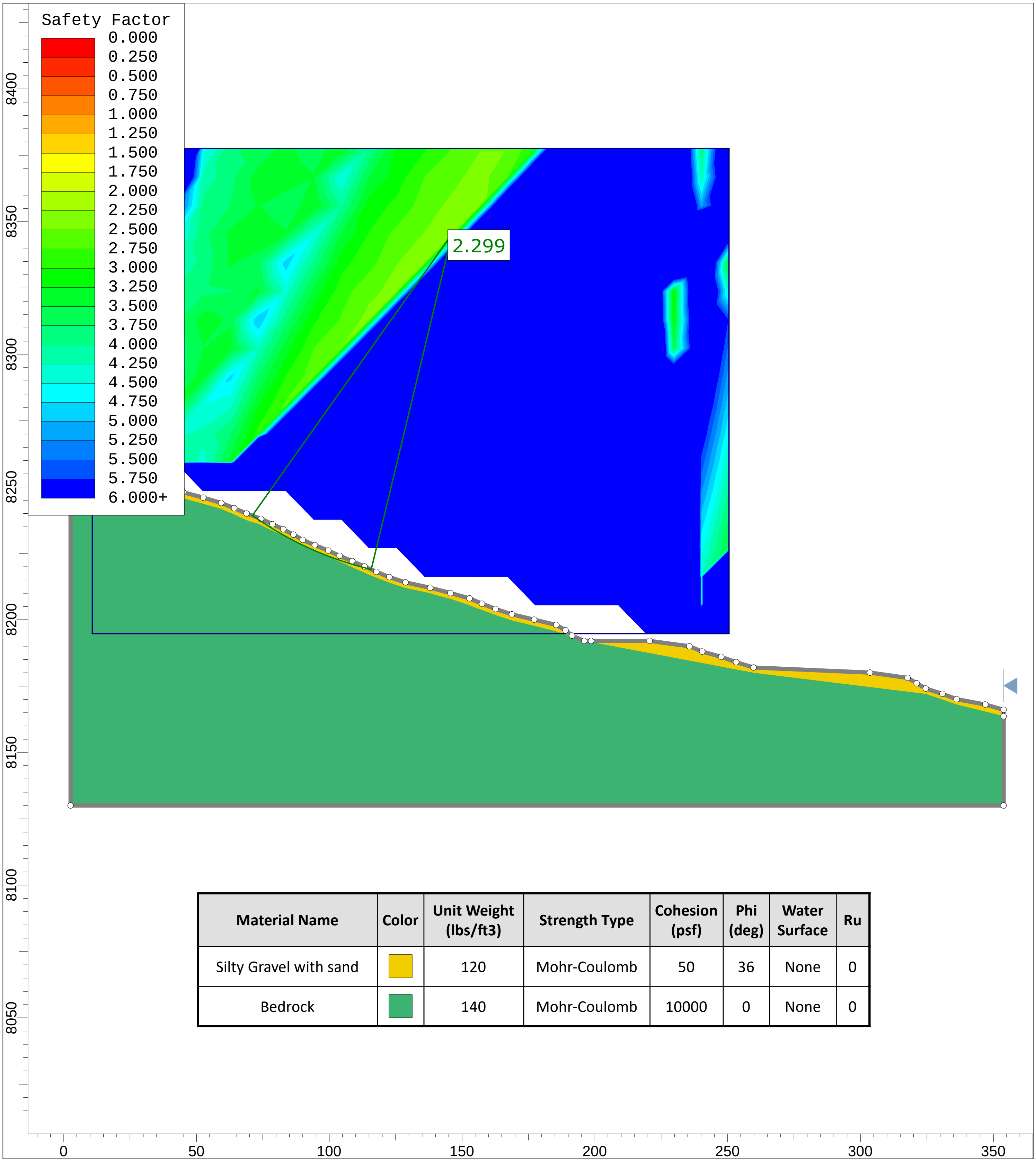
Plate 7

Grain Size Distribution



Location	Depth		Classification	% Gravel	% Sand	% Silt and Clay
TP-1	1	●	Silty GRAVEL with sand	51.7	23.0	25.3

Christensen Geotechnical	**Solitude Builders** **Lot 5R Powder 11 at Powder Mountain** **Weber County, Utah** **Project No.: 153-006**	**Plate** **8**



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Ru
Silty Gravel with sand	🟨	120	Mohr-Coulomb	50	36	None	0
Bedrock	🟩	140	Mohr-Coulomb	10000	0	None	0

Global Stability - Static

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Solitude Builders
Powder 11 Lot 5
Weber County, Utah
153-006

Plate

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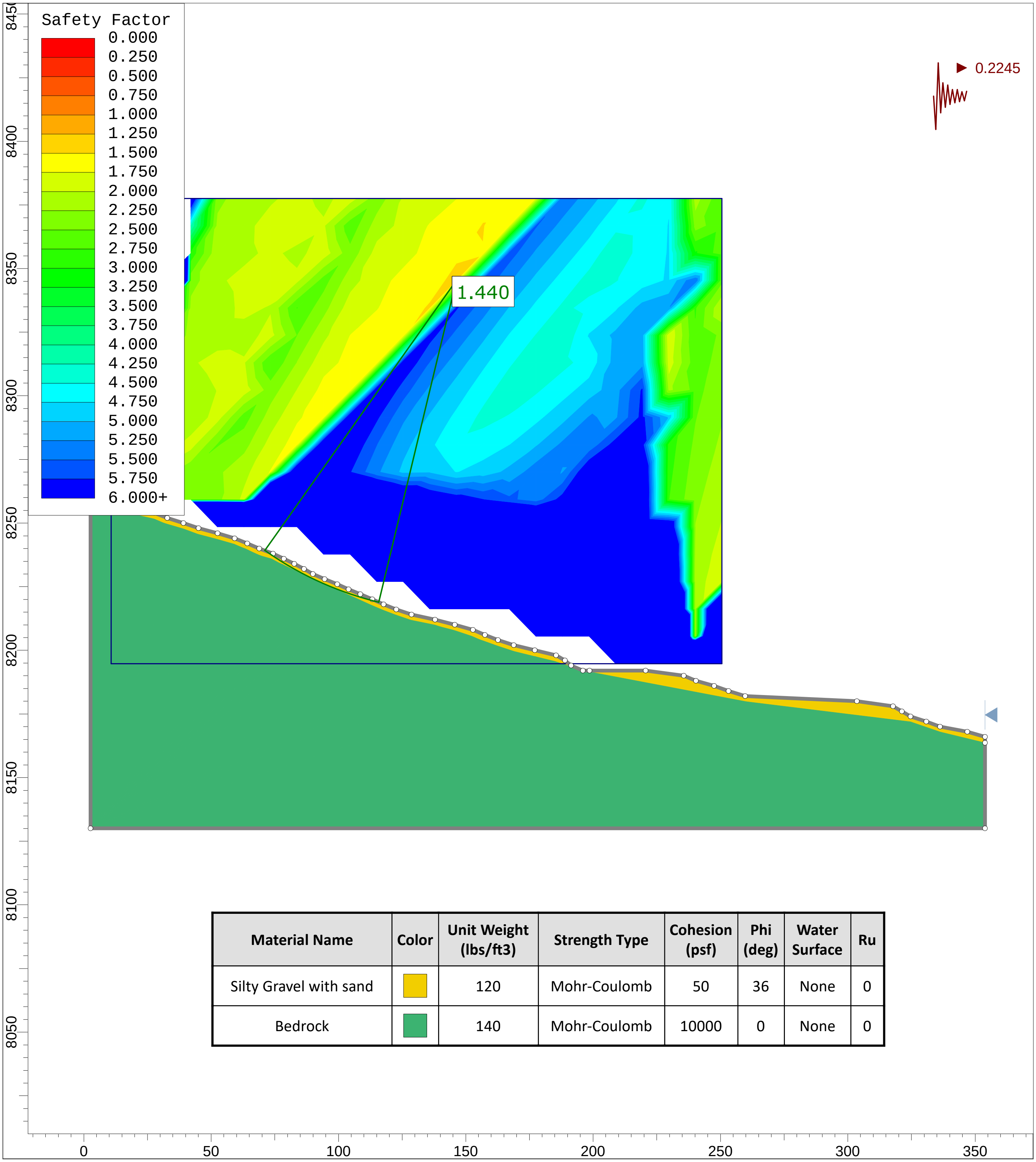

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Ru
Silty Gravel with sand	🟨	120	Mohr-Coulomb	50	36	None	0
Bedrock	🟩	140	Mohr-Coulomb	10000	0	None	0

Global Stability - Pseudo Static

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Powder 11 Lot 5
Weber County, Utah
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Plate
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