Geotechnical Investigation
and Geologic Hazards Evaluation
Proposed Single-Family Residence
Lot 28 Pole Patch No. 2 Subdivision
Pleasant View, Utah



April 15, 2022

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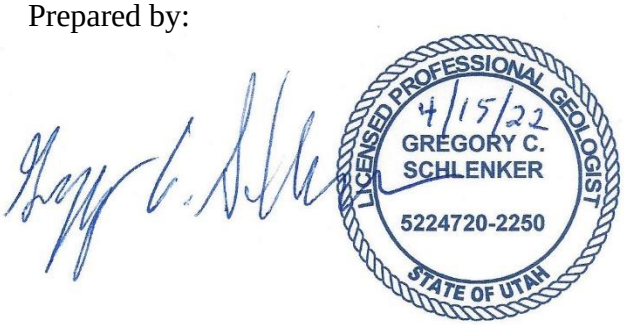
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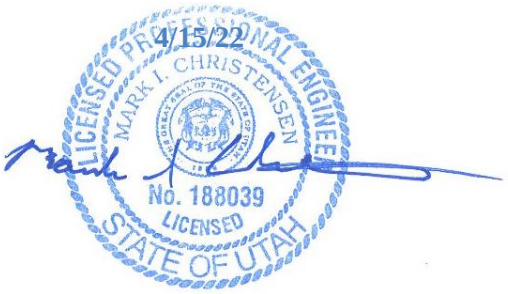
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Pleasant View, Utah

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1.0 INTRODUCTION

1.1 PURPOSE AND SCOPE OF WORK

This report presents the results of a combined geological hazards - surface fault rupture hazard evaluation and geotechnical investigation that were performed for the proposed single-family residence for Lot 28 Pole Patch No. 2 Subdivision development project located at approximately 5090 North Ridge Canyon Road in Pleasant View, Utah. As shown on Plate 1, Vicinity Map, the site for the proposed development parcel consists of an irregularly shaped property occupying parts of the northwestern quarter of Section 7 (T7N, R1W, SLBM). The property includes a 9.06-acre parcel on mountainside terrain where the owner intends to construct a single-family residence. Plate 2, Site Plan, provides aerial coverage of the site and detail of the current (2021) layout of the site vicinity and existing buildings, as well as the Lot 28 parcel boundary and the building area for the proposed single-family residence. The site is located at the transition between the Wasatch Mountains to the north and east of the site, and the Wasatch Front Valleys of the Basin and Range to the south and west of the site (Stokes, 1977). The terrain on and in the vicinity of the site includes steep mountain slopes, moderately sloping range-front foothills and coalesced alluvial fan surfaces which transition southwestward to a nearly level valley floor surface.

The Lot 28 parcel is located within an area of Pleasant View that the City Code defines as a Sensitive Area Overlay Zone (Chapter 18.38). As such, the Sensitive Area Overlay Zone rules require geological and environmental studies for land use permitting. For the geological concerns the Sensitive Area Overlay Zone requires site specific studies to *include, but not be limited to: earthquake faults, potential landslide areas, steep unstable terrain, areas of potential rock fall, subsurface waters and flooding* (Pleasant View City, 2022). For these purposes, this Geotechnical Investigation and Geologic Hazards Evaluation has been prepared to suffice the required geologic studies for the Lot 28 site development.

In general, the purposes of this investigation were to provide both a site-specific geological hazards study including surface fault rupture hazard study and a geotechnical engineering evaluation to support the proposed site development and design. The geological hazards study was conducted to evaluate the site relative to geologic hazards as listed in the Sensitive Area Overlay Zone, and specifically to evaluate the proposed site development to surface fault rupture hazard exposure which specifies, *"Structures shall not be placed on or across a fault nor within fifty feet unless it is determined by professional experts that the distance of fifty feet may be reduced without creating a potential hazard…"* Because the site is located in close proximity to the Wasatch fault zone, and is within the Surface Fault Rupture Hazard Special Studies zone as mapped by McDonald and others (2018) and shown on Plate 3, Fault Hazards, a surface fault rupture hazard evaluation was performed for the proposed building area, to current standard of care as prescribed by Lund and others (2020) of the Utah Geological Survey (UGS).

The geotechnical engineering evaluation was conducted to evaluate the subsurface conditions and the nature and engineering properties of the subsurface soils, 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 development is to consist of a single-family residential homesite. The proposed structure is expected to be one to two stories in height with a basement. The structural loads for the proposed residence are anticipated to be on the order of 3 to 4 klf for walls and 150 psf for floors. The site development will also include driveway grading, installation of utilities, and landscaping. If the actual structural loads are different from those anticipated, Christensen Geotechnical should be notified in order to reevaluate our recommendations.

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 on and in the vicinity of the site by Crittenden and Sorensen (1985), Lowe (1989), Personius (1990), King and McDonald (2014), and Coogan and King (2016) which in part is shown on Plate 4, Geologic Mapping; a photogeologic analyses of 2012 and 2021 orthoimagery as shown on Plate 2, and an analysis of historical stereoscopic 1; 15,840 scale imagery flown in 1963 (frames ELK-1-86 and ELK-1-86) and 1:12,000 scale imagery flown in 1970 (frames WF-N-2-9-185 and WF-N-2-9-186); a GIS analysis of geoprocessed 2014 0.5-meter bare-earth LiDAR terrain data as shown on Plate 5, LiDAR Analysis; a field reconnaissance of the general site area, and the subsurface evaluation of an exploration trench excavated at the site as part of our field program. Seismic hazards information was developed from United States Geologic Survey (USGS) databases by Petersen and others, (2014), and UGS research by DuRoss and others (2012).

2.2 FIELD INVESTIGATION

The site was reconnoitered March 4, 2022 by Dr. Gregory Schlenker, Senior Geologist, P.G. and Principal Engineer Mr. Mark Christensen, P.E. During the field reconnaissance, the mapped site geological conditions were confirmed and/or amended as shown on Plate 4.

The subsurface conditions at the site were explored by excavating a 172-foot-long exploration trench, 5.0- to 7.0-feet in depth, across the proposed building area located on the southwest side

of the property adjacent to Ridge Canyon Road, as shown on Plate 6, Site Evaluation. The subsurface conditions as encountered in the trench were logged for geological interpretation, and soil samples were collected from the trench for geotechnical engineering soils laboratory testing. The graphical log of the trench is presented on the attached Plate 7, Log of Trench.

The trench was excavated using an approximately 20-ton trackhoe excavator. Due to the granular nature of the soils encountered, only disturbed soil samples were collected from the test pit sidewalls at the time of excavation. The locations within the trench where the samples were collected are shown on Figure 7. 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.

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 performed included moisture content determinations, Atterberg limits evaluations, and gradation analyses. A summary of our laboratory testing is presented in the table below:

Table No. 1: Laboratory Test Results

Station	Depth (ft.)	Dry Density (pcf)	Moisture Content (%)	Atterberg Limits		Grain Size Distribution (%)			Soil Type
				LL	PI	Gravel (+#4)	Sand	Silt/Clay (- #200)	
50	5.5		3.8	NP	NP	73.5	24.7	1.8	GP
104	3.5		3.2	NP	NP	73.8	23.6	2.6	GP
150	3.5		2.8	NP	NP	71.0	27.8	1.1	GP

The results of our laboratory tests are also 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 LiDAR ANALYSIS

The elevation data for the site consisted of 2014 geoprocessed 0.5-meter bare earth LiDAR Data obtained from Utah State UGRC GIS agency. 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, slope percentages, surface renderings and elevation contours were calculated for the site area.

3.0 GENERAL SITE CONDITIONS

3.1 SURFACE CONDITIONS

As shown on Plate 2, the site is a currently undeveloped 9.06-acre parcel on mountainside terrain, roughly 1250 feet northeast to southwest by 380 feet northwest to southeast in maximum plan dimensions. The proposed building area occupies an approximately 0.32-acre area on the southwest side of the site adjacent to Ridge Canyon Road. The vegetative cover on the site consists of dense clusters of scrub oak and maple trees, with sage brush, grasses and weeds covering the open areas on the site. Elevations across the site range from a low of 5510 feet on the southwest side of the property adjacent to Ridge Canyon Road to 6124 feet on the northeast side of the property. The slopes on the property generally exceed 25-percent, with slopes of a lesser gradient observed on and in the vicinity of the proposed building area adjacent to Ridge Canyon Road.

3.2 SUBSURFACE CONDITIONS

3.2.1 Soils

The soils exposed in the exploration trench during our field investigation were observed to be alluvial fan sediments overlain by surficial topsoil (Soil A Horizon) accumulations. As logged in the 5.0- to 7.0-foot-deep exploration trench, the soils consisted of Sandy Gravel GP, with Cobbles and Boulders; medium to coarse Sand, fine and coarse Gravel; light yellowish brown and brown; dense; slightly moist; with Gravel and Cobble particles horizontally oriented (imbricated). A topsoil A-horizon sequence abundant with herbaceous roots and organic fines was observed on the surface of the trench.

3.2.2 Groundwater

Groundwater was not encountered in the exploration trench, and shallow groundwater indicators were not observed on or in the vicinity of the proposed building area during our field investigation. Static groundwater is believed to be more than 20 feet in depth below the proposed building area location. It should be understood that groundwater and soil water conditions may fluctuate in response to seasonal fluctuations, precipitation and snowmelt events.

4.0 GEOLOGIC CONDITIONS

4.1 GEOLOGIC SETTING

The site is located on the southwestern flank of Ben Lomond Peak, a 9712-foot-high peak, the western flank of which comprises the Wasatch Front. The surficial geology of the site vicinity is the result of the uplift and exposure of older Precambrian rocks which form the crest of the peak northeast of the site. This exposure was the result of movement along high-angle faults during the late Tertiary and Quaternary ages (Bryant, 1988) which lifted the peaks to over 9000 feet in elevation above the Bonneville Basin on the west. Bounding on the south, southwest, and west of the peak is a series of coalesced alluvial fans that form a valley-margin piedmont slope upon which the site is located. Down-gradient from the site, the terrain flattens towards the basin, where late-Pleistocene pluvial lakes (Lake Bonneville) inundated the basin, leaving eroded shorelines, and beach and off-shore sediments (Currey and Oviatt, 1985).

Topographically, the site is located on the base of mountain slopes on the southwest side of Ben Lomond Peak. The topography of the site includes steep bedrock-controlled mountain slopes covered with thin colluvial deposits, with lesser-grade sloping alluvial-fan deposits covering the base of the steeper slopes on and in the vicinity of the proposed building area.

4.2 SURFICIAL GEOLOGY

The surficial geology of the site vicinity is presented on Plate 4 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):

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

Qaf1 - **Qafy** - Younger alluvial-fan deposits (Holocene and uppermost Pleistocene) – Like undivided alluvial fans but height above present drainages is low and is within certain limits...All of these fans should be considered active; Qaf1 fans are active because they impinge on and deflect present-day drainages; Qafy fans are active, impinge on present-day drainages, and may be partly older than Qaf1 fans...

Qaf - Alluvial-fan deposits (Holocene and Pleistocene) – Mostly sand, silt, and gravel that is poorly bedded and poorly sorted...

Qafoe? – Alluvial fan deposits – older (middle and lower Pleistocene) - Mostly poorly bedded and poorly sorted sand, silt, and gravel; fan remnants higher than Lake Bonneville deposits...

Qafpb - Lake Bonneville-age alluvial-fan deposits (upper Pleistocene) — Related to shorelines of Lake Bonneville, mostly sand, silt, and gravel that is poorly bedded and poorly sorted...

Ct - Tintic Quartzite (Middle and Lower Cambrian) – Tan-weathering, cliff-forming, very well-cemented quartzite, with lenses and beds of quartz-pebble conglomerate, and lesser thin argillite layers…

Xfcg - Farmington Canyon Complex (Paleoproterozoic) – Granitic gneiss, quartz-rich gneiss, and biotite-rich schist, with lesser layers to pods of white quartzite, pegmatite, amphibolite, mafic rocks, and meta-ultramafic rocks...

4.3 SURFACE FAULT RUPTURE HAZARDS

Mapped traces of the Brigham City section of the Wasatch fault zone (Black and others, 2004; DuRoss and others, 2012), has been mapped by Personius (1990), Coogan and King (2016), and McDonald and others (2018) in the vicinity of the site. The mapped faulting on Plate 3 shows an active fault trace as trending northwest to southeast across the southwest part of the property, and shown to be downthrown to the southwest in the vicinity of the site. To evaluate the exposure to active fault hazards for the proposed building area, where the proposed single-family residence is to be constructed, our exploration trench was excavated across the proposed building area as shown on Plate 6.

The trench was excavated on a north-to-south axis to intercept evidence of deformation and/or fault movement on the generally northwest-southeast trending faults mapped in the vicinity of the site. The natural deposits observed in the trench and illustrated on Plate 7 consisted of alluvial fan (**Qafy**) sediments that are believed to have been deposited on the site prior to mid-Holocene time approximately 6,000 years ago (King and McDonald, 2014; Coogan and King, 2016) as part of a coalesced alluvial fan sequence that has developed along the base of Ben Lomond Peak.

DuRoss and others (2012), estimate that the Brigham City section of the Wasatch fault zone has experienced as many as six earthquake faulting events in the past 11,000 years, with four faulting events occurring in the past 6,000 years (DuRoss and others, 2012).

4.3.1 Faulting

As shown on Plate7, the fault displacement of alluvial fan (**Qafy**) sediments was observed between STA 8 feet South and STA 11 feet South at the base of our exploration trench. The zone of fault displacement consisted of two closely-spaced faults STA 8 feet South (North fault) and STA 11 feet South (South fault), and is described as follows:

Zone of Fault Displacement

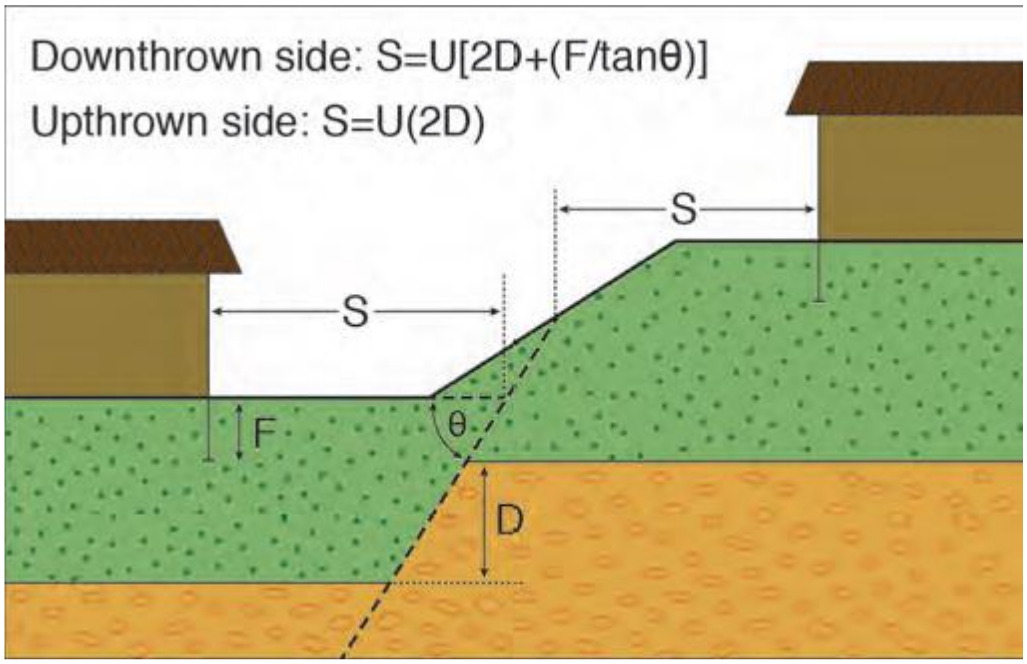
North Fault: Dip - 68° S (displacement down to south)

South Fault: Dip - 60° S (displacement down to south)
Total observed vertical displacement across zone: ~4.0 feet, down to South
Fault zone strikes 342° W through trench

The faulting observed in the trench displaced sediments that were interpreted to be upper Pleistocene to Holocene in age and was therefore considered active. According to the Western States Seismic Policy Council (2011) and adopted by the UGS (Lund and others, 2020), active (Holocene age) earthquake faults are generally considered to be faults which have disrupted the ground surface within the past 11,700 years before present. Implied with this definition is that such faults are likely to disrupt the ground surface in the relatively near future.

4.3.2. UGS Fault Setback Guidelines

To establish reasonable setbacks from the faulting for the proposed structures, we used setback calculation criteria provided by Utah Geological Survey guidelines (Lund and others, 2020), which are based on the schematic diagram shown below:



(Diagram from Lund and others, 2020)

The fault setback distances are calculated as follows:

Downthrown Side:

$$S = U\,[2D + (F/\tan \varnothing)]$$

and Upthrown Side:

S= U (2D)

where:

S = Setback within which buildings are not permitted.
U = Risk Category, based on the IBC building occupancy class (Table 13, Lund and others 2020).
D = Expected fault displacement per event…
F = Maximum depth of footing or subgrade portion of the building.
Ø= Dip of the fault (degrees)…

4.3.3 Site Specific Fault Setback Calculations

The setbacks for the Lot 28 site are calculated using the following parameters:

S = Setback within which buildings are not permitted.
U = II(a)— Single family dwellings. (U = 1.5, or minimum setback of 15 feet)
D = 4.0 feet (greatest expected displacement)
F = 10.0 feet (assumed basement grade maximum footing depth for proposed residential structure).
Ø = 60º lowest dip angle observed on the fault

Downthrown Side Setback:

$$\mathbf{S = 20.7}' = 1.5\ [8.0' + (\tfrac{10}{1.73})]$$
$$5.8$$

and Upthrown Side Setback:

S = 12.0' = 1.5 (8.0)

On the basis of these calculations, we compute setbacks of 20.7 feet for the downthrown side and 15.0 feet (minimum setback) for the upthrown side from the observed faulting. To avoid exposure to surface fault rupture hazards, we recommend that the proposed structures be placed outside of the Fault Hazard Setback zone on the site and also within the proposed building area as shown on Plate 6. From a life safety standpoint, we believe that the specified setbacks will adequately reduce exposure to surface fault rupture hazards in the event of displacement during a future earthquake event on the Brigham City section of the Wasatch fault zone during the design life of the proposed structure. However, damage to property from ground shaking may occur during a future earthquake event.

4.4 GEOSEISMIC HAZARDS

4.4.1 Strong Ground Motion

Strong ground motion originating from the Wasatch fault or other nearby seismic sources is capable of impacting the site as well as the surrounding region. 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 (Petersen and others, 2014) queried for the site (41.3478º N -111.9795º 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 0.23*g*, and for a two percent probability of exceedance in 50 years is as high as 0.69*g*.

The ten-percent probability of exceedance in 50 years event has a return period of 475 years, and the 0.23*g* acceleration for this event corresponds with "very strong" perceived shaking with "moderate" 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 0.69*g* acceleration for this event corresponds with "severe " perceived shaking with "heavy" potential damage based on instrument intensity correlations (Wald and others, 1999).

4.4.2 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 hazard mapping for Northern Utah by Anderson and others (1994) classifies the site location as within a “Very Low” Liquefaction Potential area. A site-specific liquefaction assessment was outside the scope of our services for this project. If a liquefaction assessment for this development is desired, Christensen Geotechnical should be contacted to discuss the additional work required.

4.4.3 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) and the project site's approximate latitude and longitude and Site Class. Based on our field exploration, it is our opinion that this location is best described as a Site Class D, which represents a "stiff soil" profile. The spectral acceleration values obtained from the ATC web-based application are shown below.

Table 2: IBC Seismic Response Spectrum Values

Site Location: 41.347786º N -111.979625º W	
Name	**Response Spectral Value**
S_S	**1.515**
S_1	**0.5062**
S_{MS}	**1.515**
S_{M1}	**See ASCE Section 11.4.8**
S_{DS}	**1.01**
S_{D1}	**See ASCE Section 11.4.8**
PGA	**0.693**
PGA_M	**0.762**

4.5 ENGINEERING GEOLOGY

The engineering geology findings presented in this section pertain to the natural and geological hazards included in the Pleasant View City Subdivision Sensitive Area Overlay Zone Chapter 18.38 which requires site specific studies *to include, but not be limited to: earthquake faults, potential landslide areas, steep unstable terrain, areas of potential rock fall, subsurface waters and flooding* (Pleasant View City, 2022). The exposure to earthquake faulting and related seismicity has previously been discussed in Sections 4.3 and 4.4 of this report.

4.5.1 Landsliding - Slope Stability

The nearest potentially active landslide units are mapped as **Qms** deposits by Coogan and King (2016), and are located approximately 880 feet to the southeast of the site as shown on Plate 4, and as located, do not appear to be a risk to the site.

4.5.2 Sloping Surfaces

The site vicinity slopes developed from our LiDAR analysis are shown to range from level to well over 50 percent as shown on Plate 5, which illustrates slope percentage gradients over a rendered shaded relief surface. Much of the 9.06-acre property slopes steeply to the southwest, with moderate slopes occurring on and in the vicinity of the proposed building area. The average slope gradient for the 9.06-acre property area is calculated to be 61.0-percent, and 19.7-percent for the proposed building area. The Sensitive Area Overlay Zone Section 18.38.070 Site Plan Requirements and Considerations specifies that *no building will be permitted in a sensitive area zone on land where the natural terrain has a slope greater than twenty-five percent* (Pleasant View City, 2022).

4.5.3 Alluvial Fan - Debris Flow Processes

The geologic mapping on Plate 4 indicates the proposed building area is located in part on the margin of Holocene and uppermost Pleistocene age alluvial fan surface (**Qafy**), with more active (**Qaf1**), younger alluvial-fan deposits, Holocene and uppermost Pleistocene, emanating from a small, unnamed canyon approximately 100 feet southeast of the site, and shown on Plate 6 as potential debris flow deposits. The **Qaf1** deposits as shown on Plate 3, and Plate 6, were mapped on the basis as part of our site-specific aerial photography analysis and our field reconnaissance of the site vicinity, and the review of the Delta Geotechnical Consultants, Inc. (1993) study for the property on the east side of Lot 28. Because the source and the deposition of the **Qaf1** deposits are down-slope and off-site, active debris flow hazards are not believed to be present at the proposed building area on the site. Although deposits comprised of coarse alluvium, indicative of debris flow processes were observed in our exploration trench, the observed fault displacement of these deposits indicate that active deposition on the site precludes at least the most recent faulting event on the event on the Brigham City section of the Wasatch fault zone, which is estimated to have occurred approximately 2100 years ago according to DuRoss and others (2012).

4.5.4 Flooding Hazards

No significant waterways pass in the vicinity of the site, and flood insurance rate mapping by Federal Emergency Management Agency for Pleasant View City classifies the site vicinity as "Zone X - Area of Minimal Flood Hazard" (FEMA, 2015). Local sheet flow, slope wash, and seasonally perched soil water, typical for sloping sites, should be anticipated for the site and site improvements.

4.5.5 Rockfall and Avalanche Hazards

The site is within a broad area along the mountain front classified as *rock fall hazard special studies zone* as mapped Lowe (1989). No evidence of fallen rock clasts (including large surface boulders) were observed on the site, or within the site vicinity during our site reconnaissance. No

rockfall source outcrops were observed upslope of the site, and the upslope areas were observed to be densely covered with woody vegetation. Additionally, no indices or set-up conditions for snow avalanche development (Perla and Martinelli, 1976) were observed on or upslope from the site during our analysis or reconnaissance of the site location.

5.0 ENGINEERING ANALYSIS AND RECOMMENDATIONS

5.1 GENERAL CONCLUSIONS

Based upon the findings of this study, it is our opinion that the proposed subdivision will not be adversely exposed to the geological hazards addressed in this report and that the property is suitable for residential development from both a geological hazard and a geotechnical engineering perspective. It is also 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. The structural fill may consist of the native gravel soils or an imported material. 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.

5.3 FOUNDATIONS

The foundations for the planned structure may consist of conventional continuous and/or spread footings established either on undisturbed native soil or on properly placed and compacted structural fill which extends down to undisturbed native soils. 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 soils or structural fill may be proportioned for a maximum net allowable bearing capacity of 2,500 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	0.24	29
Active Seismic	0.30	36
At-Rest	0.43	51
Passive Static	9.96	1195
Passive Seismic	-2.63	-316

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 0.40 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 location of the subject lot in the mountain foothills, 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 to another 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.

6.0 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.

The recommendations presented within this report are based on the assumption that an adequate program of tests and observations will be followed during construction to verify compliance with our recommendations. We also assume that we will review the project plans and specifications to verify that our conclusions and recommendations are incorporated and remain appropriate (based on the actual design).

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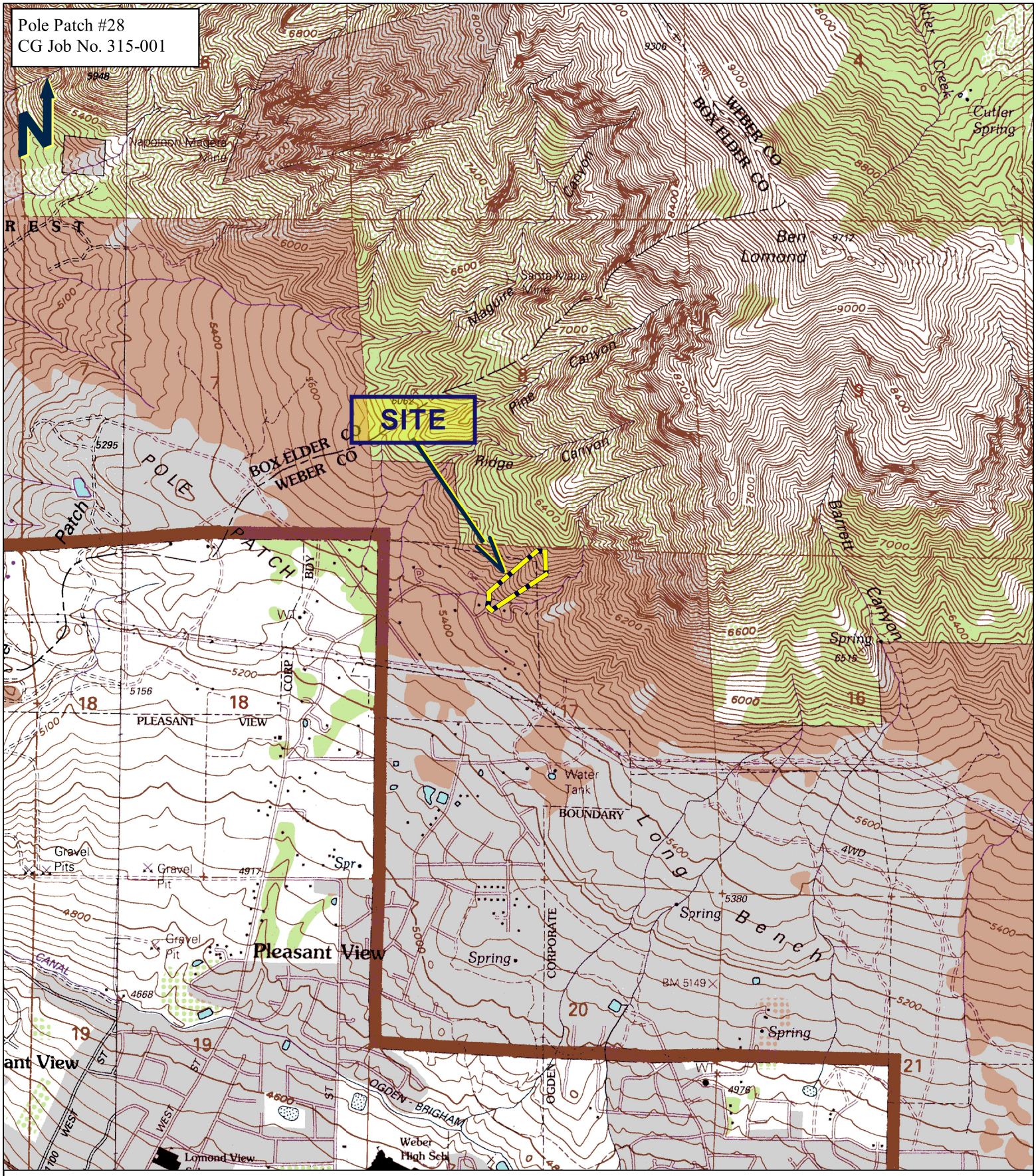
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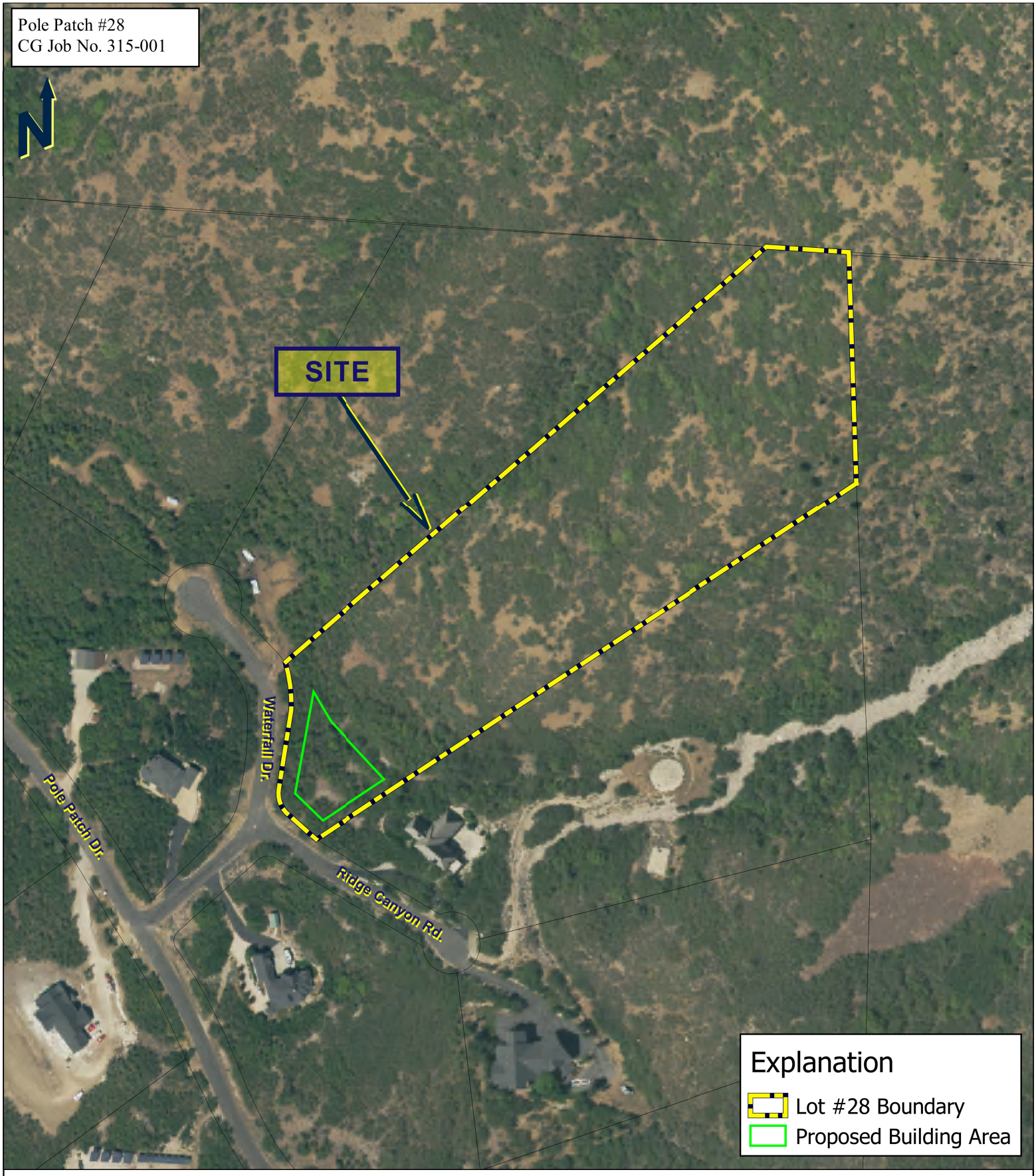
Base:
1998 USGS 7.5 Minute topographic maps titled "North Ogden, Utah" and "Plain City, Utah" from Utah UGRC; http://gis.utah.gov/

0 2000 4000 ft

1:24,000

PLATE 1
VICINITY MAP

Christensen Geotechnical



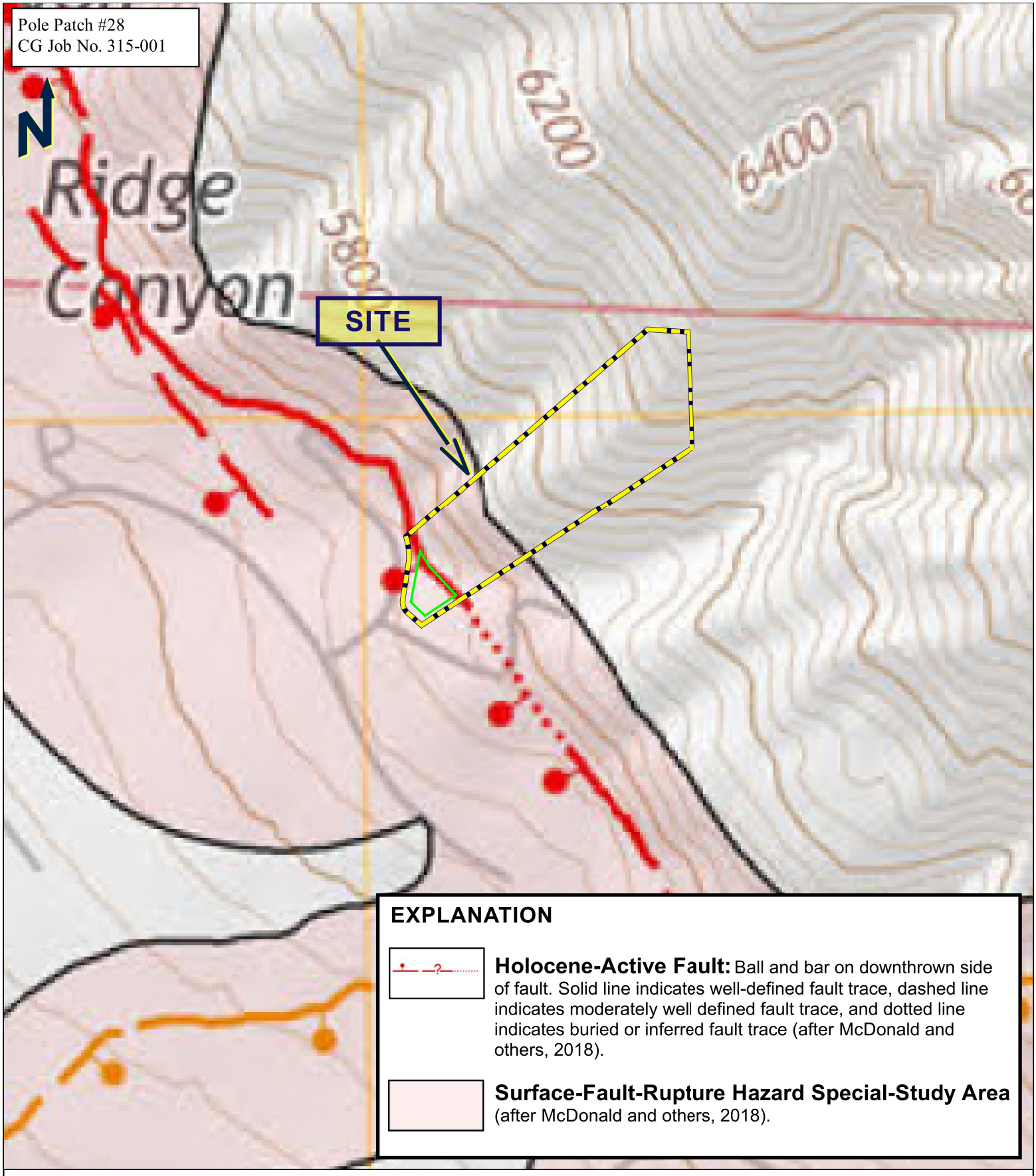
Base:
2021 0.6m NAIP Color Orthoimagery,
from Utah UGRC; http://gis.utah.gov/

0 200 400 ft

1:2,400

PLATE 2
SITE PLAN

Christensen Geotechnical



Base:
McDonald and others, 2018

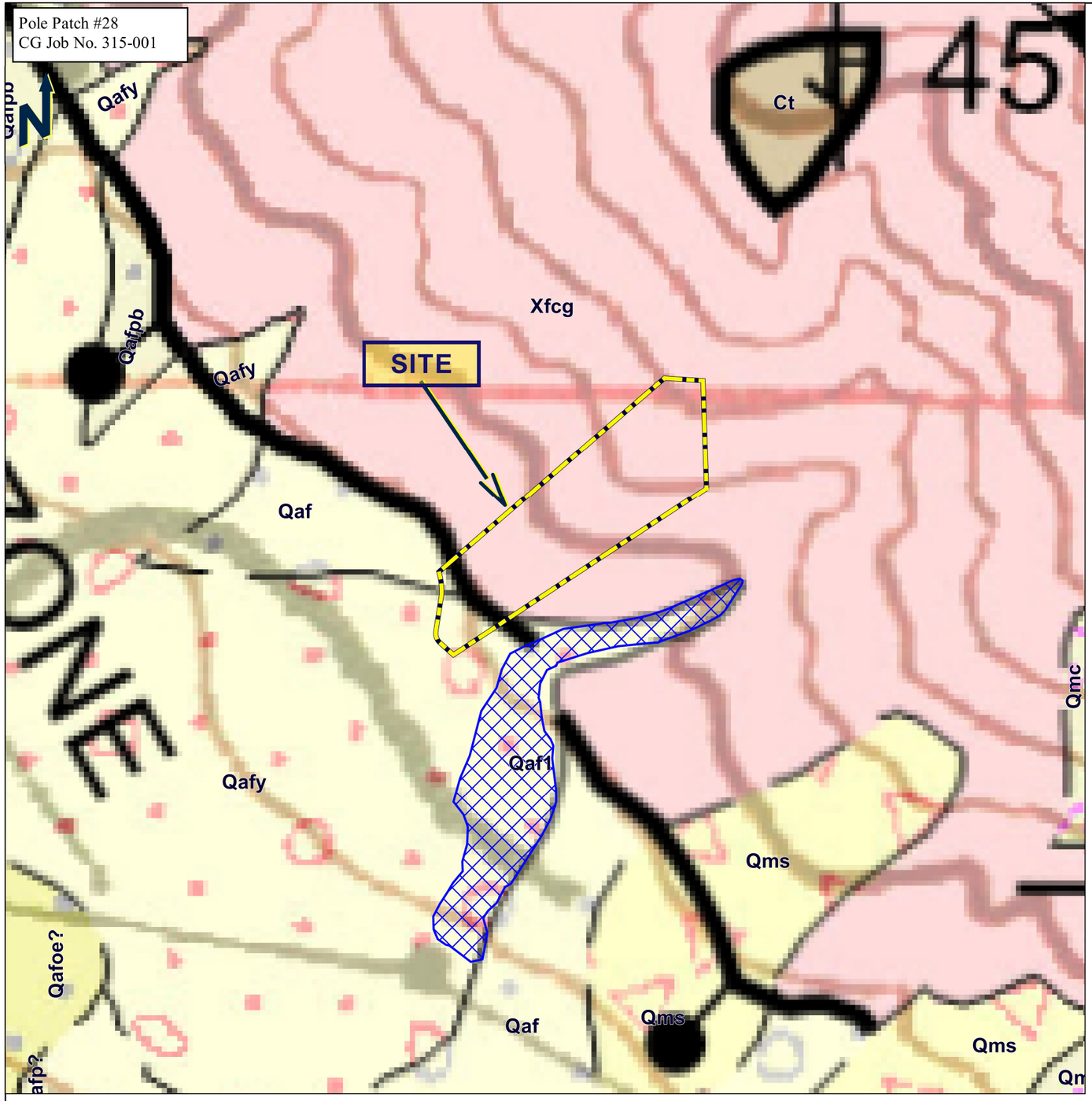
0 400 800 ft

1:4,800

PLATE 3
FAULT HAZARDS

Christensen Geotechnical

Pole Patch #28
CG Job No. 315-001



Base:
Coogan and King, 2016

0 400 800 ft

1:4,800

Geology (after Coogan and King, 2016)

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

Qaf1 - Qafy - Younger alluvial-fan deposits (Holocene and uppermost Pleistocene) – Like undivided alluvial fans but height above present drainages is low and is within certain limits...All of these fans should be considered active; Qaf1 fans are active because they impinge on and deflect present-day drainages; Qafy fans are active, impinge on present-day drainages, and may be partly older than Qaf1 fans...

Qaf - Alluvial-fan deposits (Holocene and Pleistocene) – Mostly sand, silt, and gravel that is poorly bedded and poorly sorted...

Qafoe? – Alluvial fan deposits - older (middle and lower Pleistocene) - Mostly poorly bedded and poorly sorted sand, silt, and gravel; fan remnants higher than Lake Bonneville deposits...

Qafpb - Lake Bonneville-age alluvial-fan deposits (upper Pleistocene) — Related to shorelines of Lake Bonneville, Mostly sand, silt, and gravel that is poorly bedded and poorly sorted...

Ct - Tintic Quartzite (Middle and Lower Cambrian) – Tan-weathering, cliff-forming, very well-cemented quartzite, with lenses and beds of quartz-pebble conglomerate, and lesser thin argillite layers…

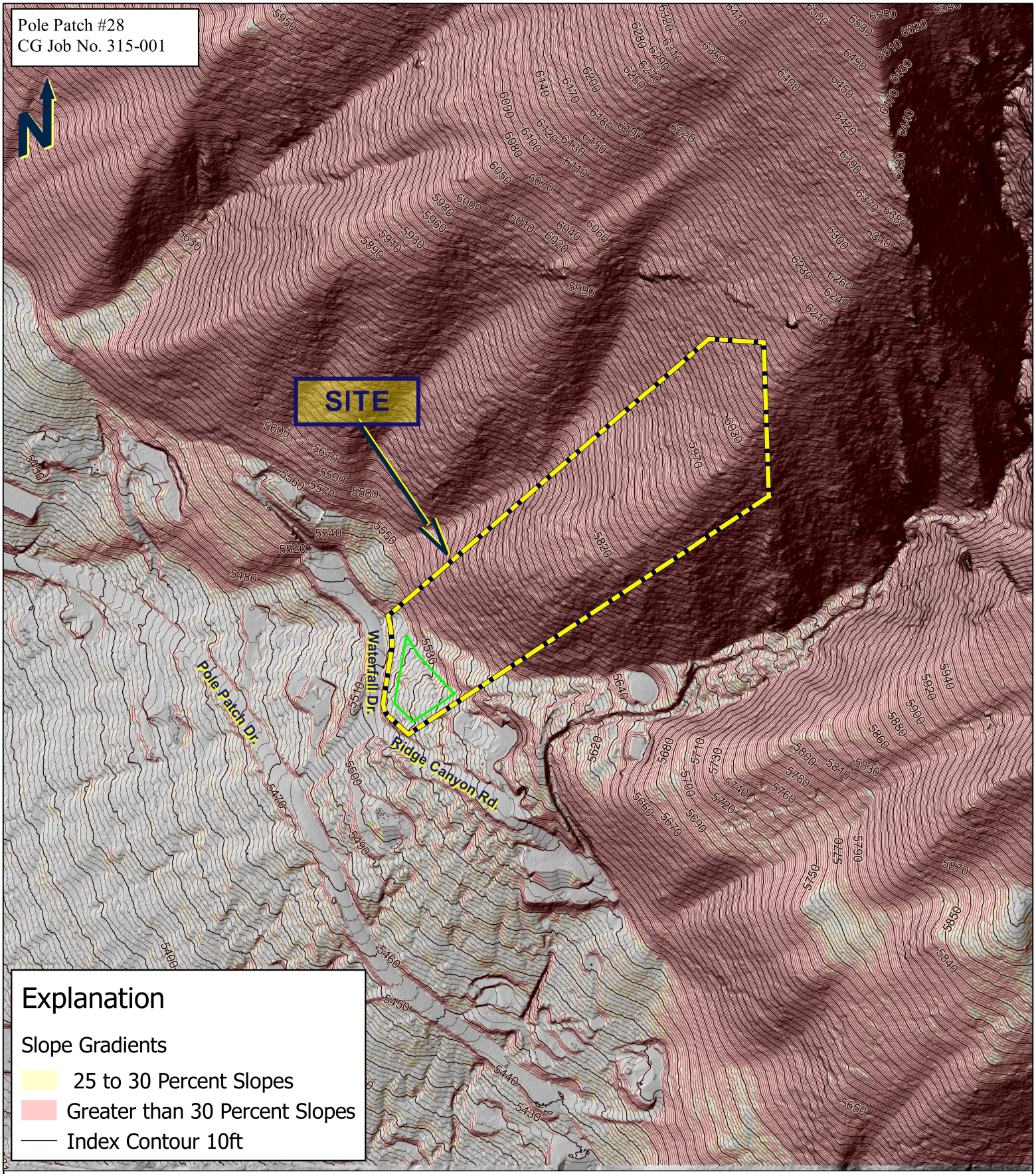
Xfcg - Farmington Canyon Complex (Paleoproterozoic) – Granitic gneiss, quartz-rich gneiss, and biotite-rich schist, with lesser layers to pods of white quartzite, pegmatite, amphibolite, mafic rocks, and meta-ultramafic rocks...

Normal Fault Bar on downthrown dashed where concealed

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

PLATE 4
GEOLOGIC MAPPING

Christensen Geotechnical



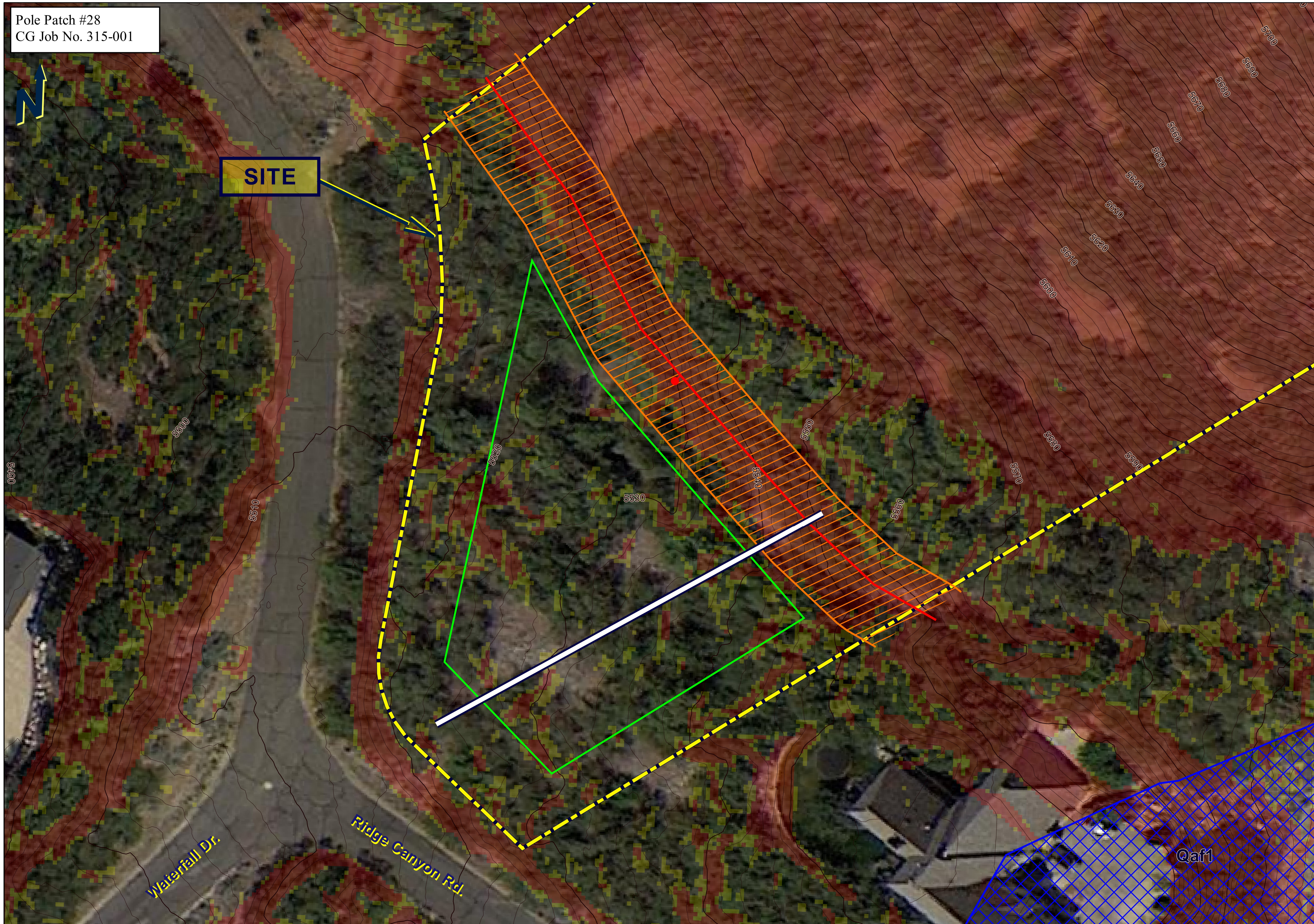
Base:
2014 0.5 Meter Bare Earth LiDAR DEM,
from Utah UGRC; http://gis.utah.gov/

0 300 600 ft

1:3,600

PLATE 5
LiDAR ANALYSIS

Christensen Geotechnical



Explanation

- Lot #28 Boundary
- Proposed Building Area
- Trench Location
- Site Specific Faulting
- Site Specific Fault Hazard Setback
- Qaf1 Potential Debris Flow Deposits

Slope Gradients

- 25 to 30 Percent Slopes
- Greater than 30 Percent Slopes
- Index Contour 10ft

Base:
2021 ~0.2m Color Orthoimagery,
from Google Earth®

0 40 80 ft

1:480

PLATE 6
SITE EVALUATION

Christensen Geotechnical

Pole Patch #28
CG Job No. 315-001

East Wall of Trench

STA South 00 + | 10 + | 20 + | 30 + | 40 + | 50 + | 60 + | STA South 70 +

Alluvium – Sandy Gravel GP, with few Cobbles; medium to coarse Sand, fine and coarse Gravel; light yellowish brown; dense; dry; with sub-rounded Gravel particles weakly bedded imbricated (**Qafy** deposits after Coogan and King, 2016)

Soil A Horizon

Alluvium – Sandy Gravel GP, with few Cobbles and Boulders; medium to coarse Sand, fine and coarse Gravel; light brown; dense; slightly moist; with sub-angular Gravel and Cobble particles imbricated (**Qafy** deposits after Coogan and King, 2016)

Alluvium – Sandy Gravel GP, with few Cobbles; medium to coarse Sand, fine and coarse Gravel; light yellowish brown; dense; dry; with sub-rounded Gravel and Cobble particles weakly bedded imbricated (**Qafy** deposits after Coogan and King, 2016)

— Line >

Zone of Fault Displacement
North Fault: Dip - 68° S (displacement down to south)
South Fault: Dip - 60° S (displacement down to south)
Total observed vertical displacement across zone: ~4.0 feet, down to South
Fault zone strikes 342° W through trench

B

Geotechnical Sample #1

STA South 70 + | 80 + | 90 + | 100 + | 110 + | 120 + | 130 + | STA South 140 +

Soil A Horizon

Alluvium – Sandy Gravel GP, with few Cobbles; medium to coarse Sand, fine and coarse Gravel; light yellowish brown; dense; dry; with sub-rounded Gravel and Cobble particles weakly bedded imbricated (**Qafy** deposits after Coogan and King, 2016)

— Line >

B

Geotechnical Sample #2

STA South 140 + | 150 + | 160 + | 170 + | STA South 180 +

Soil A Horizon

— Line >

B

Geotechnical Sample #3

Alluvium – Sandy Gravel GP, with few Cobbles; medium to coarse Sand, fine and coarse Gravel; light yellowish brown; dense; dry; with sub-rounded Gravel and Cobble particles weakly bedded imbricated (**Qafy** deposits after Coogan and King, 2016)

5
0 5
Scale In Feet

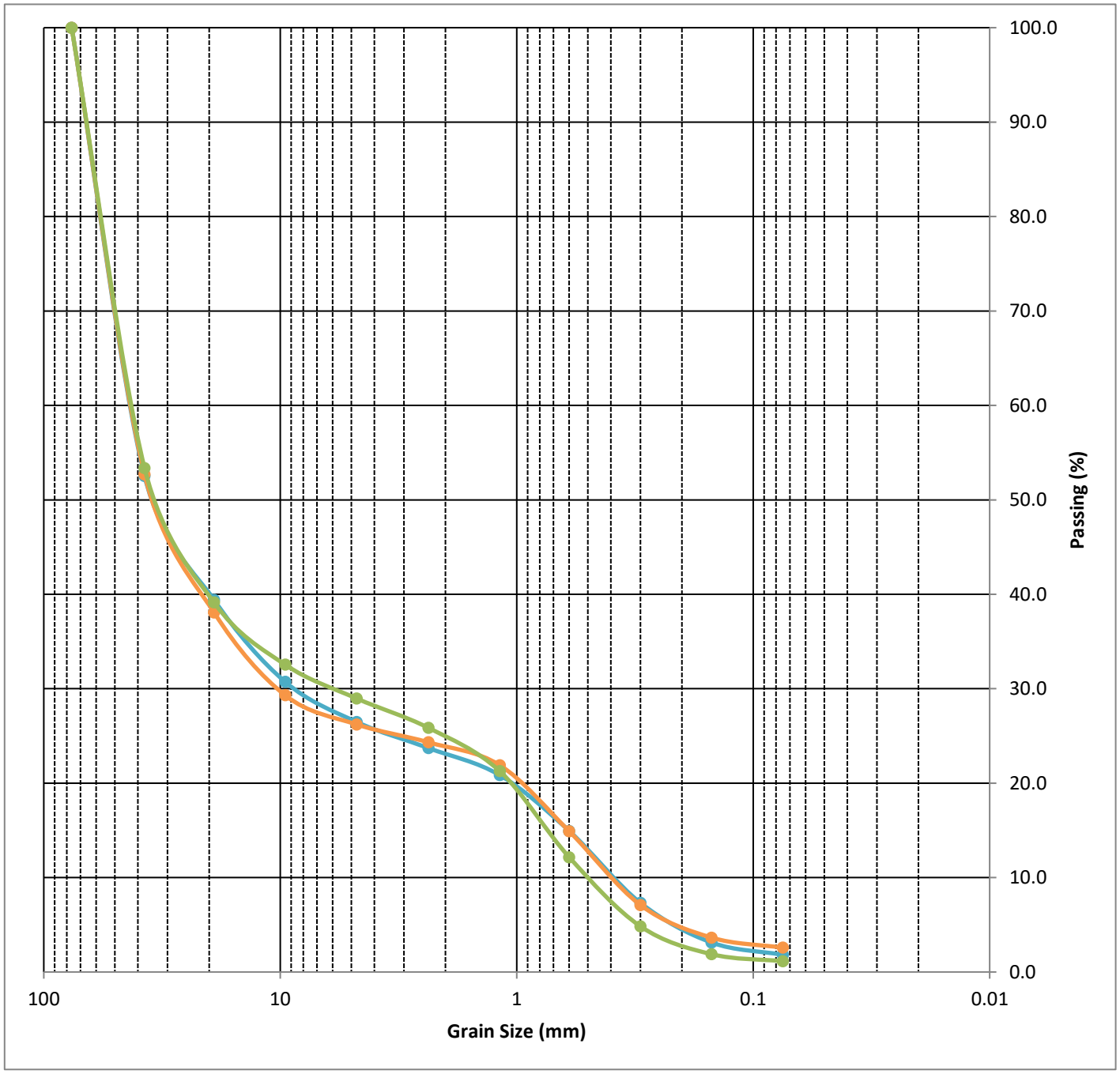
Geotechnical Sample Location
B Bulk Sample

LOGGED BY: GCS 3/4/22 | CHECKED BY MC 4/7/22

FIGURE 7
LOG OF TRENCH
STA 00 to 172 South

Christensen Geotechnical

Grain Size Distribution



Location	Depth		Classification	% Gravel	% Sand	% Silt and Clay
Station 50	5.5	●	Poorly Graded GRAVEL with sand	73.5	24.7	1.8
Station 104	3.5	●	Poorly Graded GRAVEL with sand	73.8	23.6	2.6
Station 150	3.5	●	Poorly Graded GRAVEL with sand	71.0	27.8	1.1

Christensen Geotechnical	Pierce Design Build Lot 28 Ploe Patch No. 2 Subdivision Pleasant View, Utah Project No.: 315-001	Plate 8