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GEOTECHNICAL & GEOLOGIC HAZARD INVESTIGATION (Rev. 1)
Copper Crest - East
Powder Mountain Resort
Weber County, Utah

IGES Project No. 01628-010

July 15, 2016

Prepared for:

Summit, LLC

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IGES Project No. 01628-010

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July 15, 2016

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1.0 INTRODUCTION

1.1 PURPOSE AND SCOPE OF WORK

This report presents the results of a geotechnical and geologic hazard investigation conducted for the *Copper Crest – East t*ownhome development, a part of the currently on-going expansion at the Powder Mountain Ski Resort in Weber County. The purposes of our investigation was to assess the nature and engineering properties of the subsurface soils at the proposed townhome site and to provide recommendations for the design and construction of foundations, grading, and drainage. The scope of work completed for this study included subsurface exploration, literature review, engineering analyses, and preparation of this report. This report has been revised from the original report dated November 14, 2014; the revised report remains largely unchanged from the original report, with the exception that the revised report has been reviewed by signed by a licensed geologist (Section 3.1 has been updated accordingly).

Our services were performed in accordance with our proposal to Summit, LLC (Client), dated October 6, 2014. The recommendations presented in this report are subject to the limitations presented in the "Limitations" section of this report (Section 6.1).

1.2 PROJECT DESCRIPTION

Our understanding of the project is based primarily on our previous involvement with the Powder Mountain resort project, which included two geotechnical investigations for the greater 200-acre Powder Mountain Resort expansion project (IGES, 2012a and 2012b) and subsequent geotechnical consulting for several other aspects of the project.

The Powder Mountain Resort expansion project is located southeast of SR-158 (Powder Mountain Road), south of previously developed portions of Powder Mountain Resort, in unincorporated Weber County, Utah. The project is accessed by Powder Ridge Road. The *Copper Crest - East* townhomes will be located within the sub-development *The Village* (see *Site Vicinity Map*, Figure A-1 in Appendix A). The approximately ½-acre *Copper Crest - East* project will consist of nine residential units, presumably intended to be vacation homes. The entire townhome structure is expected to have a structural footprint on the order of 15,000 square feet. The units will have three levels – the south-end of the townhomes will have, in effect, a walk-out basement (the portion of the building adjacent to the street will be subterranean). Individual units will have a single-car garage, with a possible storage space below the garage floor.

2.0 METHOD OF STUDY

2.1 LITERATURE REVIEW

The earliest geotechnical report for the area is by AMEC (2001), which was a reconnaissance-level geotechnical and geologic hazard study. IGES later completed a geotechnical investigation for the Powder Mountain Resort expansion in 2012 (2012a, 2012b). Our previous work included twenty-two test pits and one soil boring excavated at various locations across the 200-acre development; as a part of this current study, the logs from relevant nearby test pits and other data from our reports were reviewed. In addition, Western Geologic (2012) completed a geologic hazard study for the greater 200-acre Powder Mountain expansion project – this report was reviewed to assess the potential impact of geologic hazards on the *Copper Crest – East* townhomes.

2.2 FIELD INVESTIGATION

Subsurface soils were investigated by excavating two test pits at representative locations. The approximate location of the test pits are illustrated on the *Geotechnical Map* (Figure A-2 in Appendix A). The soil types were visually logged at the time of our field work in general accordance with the *Unified Soil Classification System* (USCS). Soil classifications and descriptions are included on the test pit logs, Figures A-3 and A-4 in Appendix A. A key to USCS symbols and terminology is included as Figure A-5.

2.3 LABORATORY TESTING

Samples retrieved during the subsurface investigation were transported to the laboratory for evaluation of engineering properties. Specific laboratory tests include:

- Atterberg Limits
- Grain-Size Distribution
- Insitu Moisture Content
- Soluble Sulfate, Soluble Chloride, pH and Resistivity

Results of the laboratory testing are discussed in this report and presented in Appendix B. Some test results, including moisture content and Atterberg Limits, have been incorporated into the test pit logs (Figures A-3 and A-4).

3.0 GEOLOGIC CONDITIONS

3.1 GEOLOGY AND GEOLOGIC HAZARDS

Geology and geologic hazards have been previously addressed by Western Geologic in a separate submittal (Western Geologic, 2012). This work has also been referenced in our previous geotechnical report for the project (IGES, 2012b). The report by Western Geologic indicates that the townhome site is located outside of known geologically unstable areas. The Western Geologic report also includes a large-scale geologic map that shows the development is in an area mapped as "Wasatch Formation". The Wasatch Formation is a bedrock unit that typically consist of reddish-brown conglomerate with less common sandstone, siltstone, and mudstone. Earth materials observed during our subsurface investigation consisted of colluvium (clayey gravel); however, it is conceivable that the earth materials observed consisted of highly weathered/decomposed Wasatch Formation, which can be indistinguishable from soil.

During our 2014 subsurface investigation, potentially adverse geologic structures (e.g., evidence of faulting or landslides) were not evident in the test pits. Also, geomorphic expressions of shallow, surficial landslides were not observed within the site. Subsequent geologic mapping for the Village Lifts area performed in June of 2016 by IGES, which includes the Copper Crest East property, found the subject property to be entirely underlain by colluvium derived from the Wasatch Formation (conglomerate bedrock). No geologic hazards were observed on or adjacent to the property during the recent field mapping exercise. These recent findings are consistent with what was determined at the time of the geotechnical subsurface exploration in 2014.

Based on currently available data and our observations, the potential for geologic hazards such as landslides, liquefaction, or surface fault rupture impacting the site is considered low.

3.2 SEISMICITY

Following the criteria outlined in the 2012 International Building Code (IBC, 2012), spectral response at the site was evaluated for the *Maximum Considered Earthquake* (MCE) which equates to a probabilistic seismic event having a two percent probability of exceedance in 50 years (2PE50). Spectral accelerations were determined based on the location of the site using the *U.S. Seismic "DesignMaps" Web Application* (USGS, 2012); this software incorporates seismic hazard maps depicting probabilistic ground motions and spectral response data developed for the United States by the U. S. Geological Survey as part of NEHRP/NSHMP (Frankel et al., 1996). These maps have been incorporated into both *NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures* (FEMA, 1997) and the *International Building Code* (IBC) (International Code Council, 2012).

To account for site effects, site coefficients that vary with the magnitude of spectral acceleration and *Site Class* are used. Site Class is a parameter that accounts for site amplification effects of soft soils and is based on the average shear wave velocity of the upper 100 feet; based on our field exploration and our understanding of the geology in this area, the subject site is appropriately classified as Site Class D (*stiff soil*). Based on IBC criteria, the short-period (F_a) coefficient is 1.176 and long-period (F_v) site coefficient is 1.863. Based on the design spectral response accelerations for a *Building Risk Category* of I, II or III, the site's *Seismic Design Category* is D. The short- and long-period *Design Spectral Response Accelerations* are presented in Table 3.2; a summary of the *Design Maps* analysis is presented in Appendix B. The *peak ground acceleration* (PGA) may be taken as $0.4*S_{MS}$.

Table 3.2
Short- and Long-Period Spectral Accelerations for MCE

Parameter	**Short Period (0.2 sec)**	**Long Period (1.0 sec)**
MCE Spectral Response Acceleration (g)	$S_S = 0.810$	$S_1 = 0.269$
MCE Spectral Response Acceleration Site Class B (g)	$S_{MS} = S_sF_a = 0.953$	$S_{M1} = S_1F_v = 0.500$
Design Spectral Response Acceleration (g)	$S_{DS} = S_{MS}*{}^2/_3 = 0.635$	$S_{D1} = S_{M1}*{}^2/_3 = 0.334$

4.0 GENERALIZED SITE CONDITIONS

4.1 SURFACE CONDITIONS

At the time of our field work the site was in a relatively natural state and was covered with a variety of vegetation including several mature aspens, native grasses and shrubs. Signs indicating lot number, silt fencing, and a damaged observation deck were the only man-made improvements evident.

4.2 SUBSURFACE CONDITIONS

The subsurface soil conditions were explored at the subject property by excavating two test pits at the north and south ends of the proposed townhome building. Subsurface soil conditions were logged during our field investigation and are included in the exploration logs in Appendix A at the end of this report (Figures A-3 and A-4). The soil and moisture conditions encountered during our investigation are discussed in the following paragraphs.

4.2.1 Earth Materials

Topsoil: Topsoil was encountered in both test pits – where observed, topsoil thickness generally ranged from 2 to 2½ feet, and was observed to be as thick as 3 feet locally. When queried, construction personnel from Geneva indicated that the topsoil conditions observed were 'common' in the immediate vicinity. The topsoil generally consisted of a well-developed "A Horizon" and was generally well-rooted (including tree roots) and had a *loamy* appearance.

Colluvium: Underlying the topsoil, the soils consisted of coarse colluvium, likely derived from the Wasatch Formation (conglomerate). The colluvium generally consisted of loose to medium dense clayey gravel with cobbles to 6 inches. A few boulder-size constituents to 3 feet in diameter were observed. The colluvium did not appear to be particularly difficult to excavate with the equipment used (CAT 320C tracked excavator).

Detailed descriptions of earth materials encountered are presented on the test pit logs, Figures A-3 and A-4, in Appendix A. Due to the nature and depositional characteristics of the native earth materials, care should be taken in interpolating subsurface conditions between and beyond the exploration locations.

4.2.2 Groundwater

Groundwater was not encountered in the test pit excavations. Based on our observations, groundwater is not anticipated to adversely impact the proposed development. However, groundwater levels could rise at any time based on several factors including recent precipitation, on- or off-site runoff, irrigation, time of year (e.g., spring run-off), or modifications to existing natural grade. Should the groundwater become a concern during the proposed construction, IGES should be contacted so that dewatering recommendations may be provided.

5.0 CONCLUSIONS AND RECOMMENDATIONS

5.1 GENERAL CONCLUSIONS

Based on the results of the field observations, literature review, and previously completed geotechnical investigation (IGES, 2012a), the subsurface conditions are considered suitable for the proposed development provided that the recommendations presented in this report are incorporated into the design and construction of the project.

Supporting data upon which the following recommendations are based have been presented in the previous sections of this report. The recommendations presented herein are governed by the physical properties of the earth materials encountered in the subsurface explorations. If subsurface conditions other than those described herein are encountered in conjunction with construction, and/or if design and layout changes are initiated, IGES must be informed so that our recommendations can be reviewed and revised as deemed necessary.

5.2 EARTHWORK

5.2.1 General Site Preparation and Grading

Below proposed structures, fills, and man-made improvements, all vegetation, topsoil, debris and undocumented fill (if any) should be removed. Any existing utilities should be re-routed or protected in place. The exposed native soils should then be proof-rolled with heavy rubber-tired equipment such as a scraper or loader*. Any soft/loose areas identified during proof-rolling should be removed and replaced with structural fill. All excavation bottoms should be observed by an IGES representative during proof rolling or otherwise prior to placement of engineered fill to evaluate whether soft, loose, or otherwise deleterious earth materials have been removed and to assess compliance with the recommendations presented in this report.

*not required where bedrock is exposed in the foundation subgrade

5.2.2 Excavations

Soft, loose, or otherwise unsuitable soils beneath structural elements, hardscape or pavements may need to be over-excavated and replaced with structural fill. If over-excavation is required, the excavations should extend one foot laterally for every foot of depth of over-excavation. Excavations should extend laterally at least two feet beyond flatwork, pavements, and slabs-on-grade. Structural fill should consist of granular materials and should be placed and compacted in accordance with the recommendations presented in this report.

Prior to placing engineered fill, all excavation bottoms should be scarified to at least 6 inches, moisture conditioned as necessary at or slightly above optimum moisture content (OMC), and compacted to at least 90 percent of the maximum dry density (MDD) as determined by ASTM D-1557 (Modified Proctor). Scarification is not required where bedrock is exposed.

5.2.3 Excavation Stability

The contractor is responsible for site safety, including all temporary trenches excavated at the site and the design of any required temporary shoring. The contractor is responsible for providing the "competent person" required by Occupational Safety and Health (OSHA) standards to evaluate soil conditions. For planning purposes, Soil Type C is expected to predominate at the site (sands and gravels). Close coordination between the competent person and IGES should be maintained to facilitate construction while providing safe excavations.

Based on OSHA guidelines for excavation safety, trenches with vertical walls up to 5 feet in depth may be occupied. Where very moist soil conditions or groundwater is encountered, or when the trench is deeper than 5 feet, we recommend a trench-shield or shoring be used as a protective system to workers in the trench. As an alternative to shoring or shielding, trench walls may be laid back at one and one half horizontal to one vertical (1½H:1V) (34 degrees) in accordance with OSHA Type C soils. Trench walls may need to be laid back at a steeper grade pending evaluation of soil conditions by the geotechnical engineer. Soil conditions should be evaluated in the field on a case-by-case basis. Large rocks exposed on excavation walls should be removed (scaled) to minimize rock fall hazards.

5.2.4 Structural Fill and Compaction

All fill placed for the support of structures, flatwork or pavements should consist of structural fill. Structural fill should consist of granular native soils, which may be defined as soils with less than 25% fines, 10-60% sand, and contain no rock larger than 4 inches in nominal size (6 inches in greatest dimension). Structural fill should also be free of vegetation and debris. All structural fill should be 1 inch minus material when within 1 foot of any base coarse material. Soils not meeting these criteria may be suitable for use as structural fill; however, such soils should be evaluated on a case by case basis and should be approved by IGES prior to use.

All structural fill should be placed in maximum 4-inch loose lifts if compacted by small hand-operated compaction equipment, maximum 6-inch loose lifts if compacted by light-duty rollers, and maximum 8-inch loose lifts if compacted by heavy duty compaction equipment that is capable of efficiently compacting the entire thickness of the lift. Additional lift thickness may be allowed by IGES provided the Contractor can demonstrate sufficient compaction can be achieved with a given lift thickness with the equipment in use. We recommend that all structural fill be compacted on a horizontal plane, unless otherwise approved by IGES. Structural fill underlying all shallow footings and pavements should be compacted to at least 95 percent of the MDD as determined by ASTM D-1557. **The moisture content should be at, or slightly above, the OMC for all structural fill**. Any imported fill materials should be approved prior to importing. Also, prior to placing any fill, the excavations should be observed by IGES to confirm that unsuitable materials have been removed. In addition, proper grading should precede placement of fill, as described in the General Site Preparation and Grading subsection of this report.

Specifications from governing authorities such as Weber County and/or special service districts having their own precedence for backfill and compaction should be followed where more stringent.

5.2.5 Oversize Material

Based on our observations there is a significant potential for the presence of oversize materials (larger than 6 inches in greatest dimension). Large rocks, particularly boulders (>12 inches), may require special handling, such as segregation from structural fill, and disposal. Particularly large boulders may require special equipment for removal during excavation of western half of the townhomes.

5.2.6 Utility Trench Backfill

Utility trenches should be backfilled with structural fill in accordance with Section 6.2.4 of this report. Utility trenches can be backfilled with the onsite soils free of debris, organic and oversized material. Prior to backfilling the trench, pipes should be bedded in and shaded with a uniform granular material that has a Sand Equivalent (SE) of 30 or greater. Pipe bedding may be water-densified in-place (jetting). Alternatively, pipe bedding and shading may consist of clean ¾-inch gravel, which generally does not require densification. Native earth materials can be used as backfill over the pipe bedding zone. All utility trenches backfilled below pavement sections, curb and gutter, and hardscape, should be backfilled with structural fill compacted to at least 95 percent of the MDD as determined by ASTM D-1557. All other trenches should be backfilled and compacted to approximately 90 percent of the MDD (ASTM D-1557). However, in all cases the pipe bedding and shading should meet the design criteria of the pipe manufacturer. Specifications from governing authorities having their own precedence for backfill and compaction should be followed where they are more stringent.

5.3 FOUNDATION RECOMMENDATIONS

Based on our field observations and considering the presence of relatively competent native earth materials, we recommend that the footings for proposed townhome structure be founded either *entirely* on competent native soils <u>or</u> *entirely* on structural fill. Native/fill transition zones are not allowed. If soft, loose, or otherwise deleterious earth materials are exposed in the footing excavations, then all footings must be deepened such that all footings bear on relatively uniform, competent native earth materials. Alternatively, the foundation excavation may be over-excavated a minimum of 2 feet below the bottom of proposed footings and replaced with structural fill, such that the footings bear entirely on a uniform fill blanket. We recommend that IGES assess the bottom of the foundation excavation prior to the placement of steel or concrete to identify the competent native earth materials as well as any unsuitable soils or transition zones. Additional over-excavation may be required based on the actual subsurface conditions observed.

Shallow spread or continuous wall footings constructed entirely on competent, uniform native earth materials or on a minimum of 2 feet of *structural fill* may be proportioned utilizing a maximum net allowable bearing pressure of **2,400 pounds per square foot (psf)** for dead load plus live load conditions. The net allowable bearing value presented above is for dead load plus live load conditions. The minimum recommended footing width is 20 inches for continuous wall footings and 30 inches for isolated spread footings.

All conventional foundations exposed to the full effects of frost should be established at a minimum depth of 42 inches below the lowest adjacent final grade. Interior footings, not subjected to the full effects of frost (i.e., *a continuously heated structure*), may be established at higher elevations, however, a minimum depth of embedment of 12 inches is recommended for confinement purposes.

Foundation drains should be installed around below-ground foundations (e.g., basement walls) to minimize the potential for flooding from shallow groundwater, which may be present at various times during the year, particularly spring run-off.

5.4 SETTLEMENT

5.4.1 Static Settlement

Static settlements of properly designed and constructed conventional foundations, founded as described in Section 5.3, are anticipated to be on the order of 1 inch or less. Differential settlement is expected to be half of total settlement over a distance of 30 feet.

5.4.2 Dynamic Settlement

Dynamic settlement (or seismically-induced settlement) consists of dry dynamic settlement of unsaturated soils (above groundwater) and liquefaction-induced settlement (below groundwater). During a strong seismic event, seismically-induced settlement can occur within loose to moderately dense sandy soil due to reduction in volume during, and shortly after, an earthquake event. Settlement caused by ground shaking is often non-uniformly distributed, which can result in differential settlement.

Based on the subsurface conditions encountered, dynamic settlement arising from a MCE seismic event is expected to be on the low; for design purposes, settlement on the order of ½ inch over 40 feet may be assumed.

5.5 EARTH PRESSURES AND LATERAL RESISTANCE

Lateral forces imposed upon conventional foundations due to wind or seismic forces may be resisted by the development of passive earth pressures and friction between the base of the footing and the supporting soils. In determining the frictional resistance against concrete, a coefficient of friction of 0.45 for sandy native soils or structural fill should be used.

Table 5.5
Lateral Earth Pressure Coefficients

Condition	Level Backfill		2H:1V Backfill	
	Lateral Pressure Coefficient	Equivalent Fluid Density (pcf)	Lateral Pressure Coefficient	Equivalent Fluid Density (pcf)
Active (Ka)	0.33	35	0.53	56
At-rest (Ko)	0.50	55	0.80	85
Passive (Kp)	3.0	320	—	—

Ultimate lateral earth pressures from *granular* backfill acting against retaining walls, temporary shoring, or buried structures may be computed from the lateral pressure coefficients or equivalent fluid densities presented in Table 5.5. These lateral pressures should be assumed even if the backfill is placed in a relatively narrow gap between a vertical bedrock cut and the foundation wall. These coefficients and densities assume no buildup of hydrostatic pressures. The force of water should be added to the presented values if hydrostatic pressures are anticipated.

Clayey soils drain poorly and may swell upon wetting, thereby greatly increasing lateral pressures acting on earth retaining structures; therefore, clayey soils should not be used as retaining wall backfill. Backfill should consist of native granular soil with an Expansion Index (EI) less than 20.

Walls and structures allowed to rotate slightly should use the active condition. If the element is to be constrained against rotation (i.e., a basement wall), the at-rest condition should be used. These values should be used with an appropriate factor of safety against overturning and sliding. A value of 1.5 is typically used. Additionally, if passive resistance is calculated in conjunction with frictional resistance, the passive resistance should be reduced by ½.

5.6 CONCRETE SLAB-ON-GRADE CONSTRUCTION

To minimize settlement and cracking of slabs, and to aid in drainage beneath the concrete floor slabs, all concrete slabs should be founded on a minimum 4-inch layer of compacted gravel overlying properly prepared subgrade. The gravel should consist of free-draining gravel or road base with a 3/4-inch maximum particle size and no more than 5 percent passing the No. 200 mesh sieve. The layer should be compacted to at least 95 percent of the MDD as determined by ASTM D-1557.

All concrete slabs should be designed to minimize cracking as a result of shrinkage. Consideration should be given to reinforcing the slab with a welded wire fabric, re-bar, or fibermesh. Slab reinforcement should be designed by the structural engineer; however, as a minimum, slab reinforcement should consist of 4’’×4’’ W4.0×W4.0 welded wire mesh within the middle third of the slab. We recommend that concrete be tested to assess that the slump and/or air content is in compliance with the plans and specifications. We recommend that concrete be placed in general accordance with the requirements of the American Concrete Institute (ACI). A Modulus of Subgrade Reaction of **250 psi/inch** may be used for design.

A moisture barrier (vapor retarder) consisting of 10-mil thick Visqueen (or equivalent) plastic sheeting should be placed below slabs-on-grade where moisture-sensitive floor coverings or equipment is planned. Prior to placing this moisture barrier, any objects that could puncture it, such as protruding gravel or rocks, should be removed from the building pad. Alternatively, the subgrade may be covered with 2 inches of clean sand**.**

5.7 MOISTURE PROTECTION AND SURFACE DRAINAGE

Moisture should not be allowed to infiltrate into the soils in the vicinity of the foundations. As such, design strategies to minimize ponding and infiltration near the townhome structure should be implemented.

We recommend roof runoff devices be installed to direct all runoff a minimum of 10 feet away from the townhome foundations. The builder should be responsible for compacting the exterior backfill soils around the foundation, particularly around basement walls. Additionally, the ground surface within 10 feet of the structure should be constructed so as to slope a minimum of **five** percent away. Pavement sections should be constructed to divert surface water off the pavement into storm drains, curb/gutter, or another suitable location.

For the subterranean portion of the townhome, IGES recommends a perimeter foundation drain be constructed in accordance with the International Residential Code (IRC).

5.8 SOIL CORROSION POTENTIAL

Laboratory testing of a representative soil sample obtained from the Test Pit 1 indicated that the soil sample tested had a sulfate content of 18 ppm. Accordingly, the soils are classified as having a ‘low’ potential for deterioration of concrete due to the presence of soluble sulfate. As such, conventional Type I/II Portland cement may be used for all concrete in contact with site soils.

To evaluate the corrosion potential of ferrous metal in contact with onsite native soil a sample was tested for soil resistivity, soluble chloride and pH. The test indicated that the onsite soil tested has a minimum soil resistivity of 7,330 OHM-cm, soluble chloride content of 5.4 ppm and a pH of 5.4. Based on this result, the onsite native soil is considered to be *moderately corrosive* to ferrous metal. Consideration should be given to retaining the services of a qualified corrosion engineer to provide an assessment of any metal that may be associated with construction of ancillary water lines and reinforcing steel, valves etc.

5.9 CONSTRUCTION CONSIDERATIONS

5.9.1 Temporary Shoring

Temporary shoring may be required during excavation of the lower floors, particularly below the planned garage level, if the earth material below the garage will be left in-place. If a temporary storage area is constructed below the garages, temporary shoring may also be required to protect the street, particularly if utilities have been installed that preclude the possibility of laying-back the slope.

If the area below the garage is laid-back during construction of the foundation wall, the entire garage slab should be underlain by a minimum of 3 feet of structural fill (to minimize excessive differential fill thicknesses below the structure).

5.9.2 Over-Size Material

Several large boulders (up to 36 inches) were observed within the test pits; as such, excavation of the basement may generate an abundance of over-size material that may require special handling, processing, or disposal.

6.0 CLOSURE

6.1 LIMITATIONS

The recommendations presented in this report are based on limited field exploration, review of existing hazard studies and other geotechnical data, and our understanding of the proposed construction. The subsurface data used in the preparation of this report were obtained from the explorations made 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, we should be immediately notified so that we may make any necessary revisions to recommendations contained in this report. In addition, if the scope of the proposed construction changes from that described in this report, IGES should also be notified.

This report was prepared in accordance with the generally accepted standard of practice at the time the report was written. No 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.

6.2 ADDITIONAL SERVICES

The recommendations made in this report are based on the assumption that an adequate program of tests and observations will be made during the construction. IGES staff or other qualified personnel should be on site to verify compliance with these recommendations. These tests and observations should include at a minimum the following:

- Observations and testing during site preparation, earthwork and structural fill placement.
- Consultation as may be required during construction.
- Quality control on concrete placement to verify slump, air content, and strength.
- Quality control and testing during placement and compaction of asphalt.

We also recommend that project plans and specifications be reviewed by us to verify compatibility with our conclusions and recommendations. Additional information concerning the scope and cost of these services can be obtained from our office.

We appreciate the opportunity to be of service on this project. Should you have any questions regarding the report or wish to discuss additional services, please do not hesitate to contact us at your convenience (801) 748-4044.

7.0 REFERENCES

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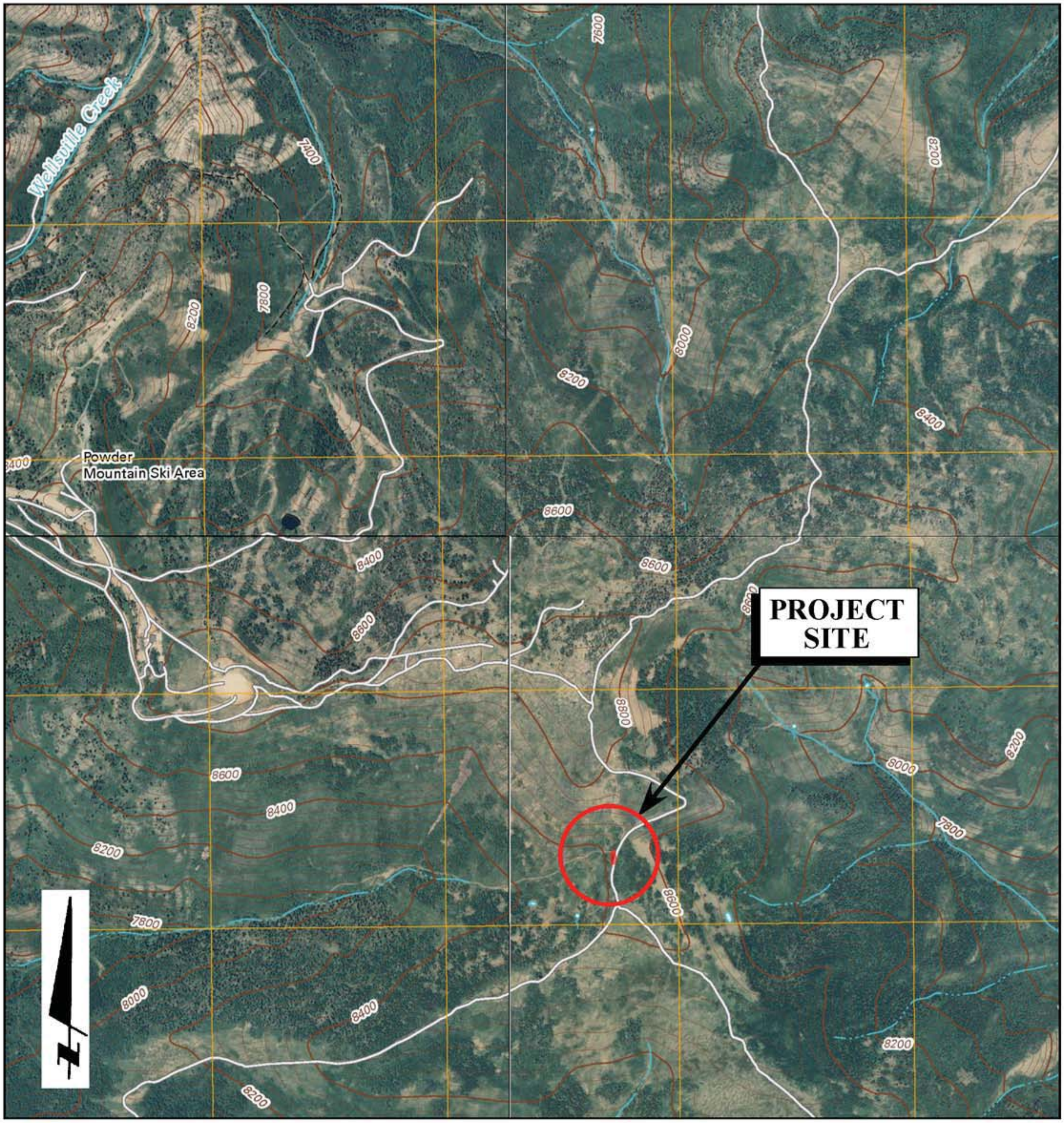
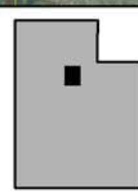
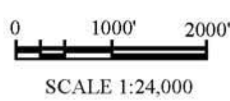
APPENDIX A



BASE MAP:
USGS Huntsville, Browns Hole, James Peak and Sharp Mountain
7.5-Minute Quadrangle Topographic Maps (2011)



MAP LOCATION



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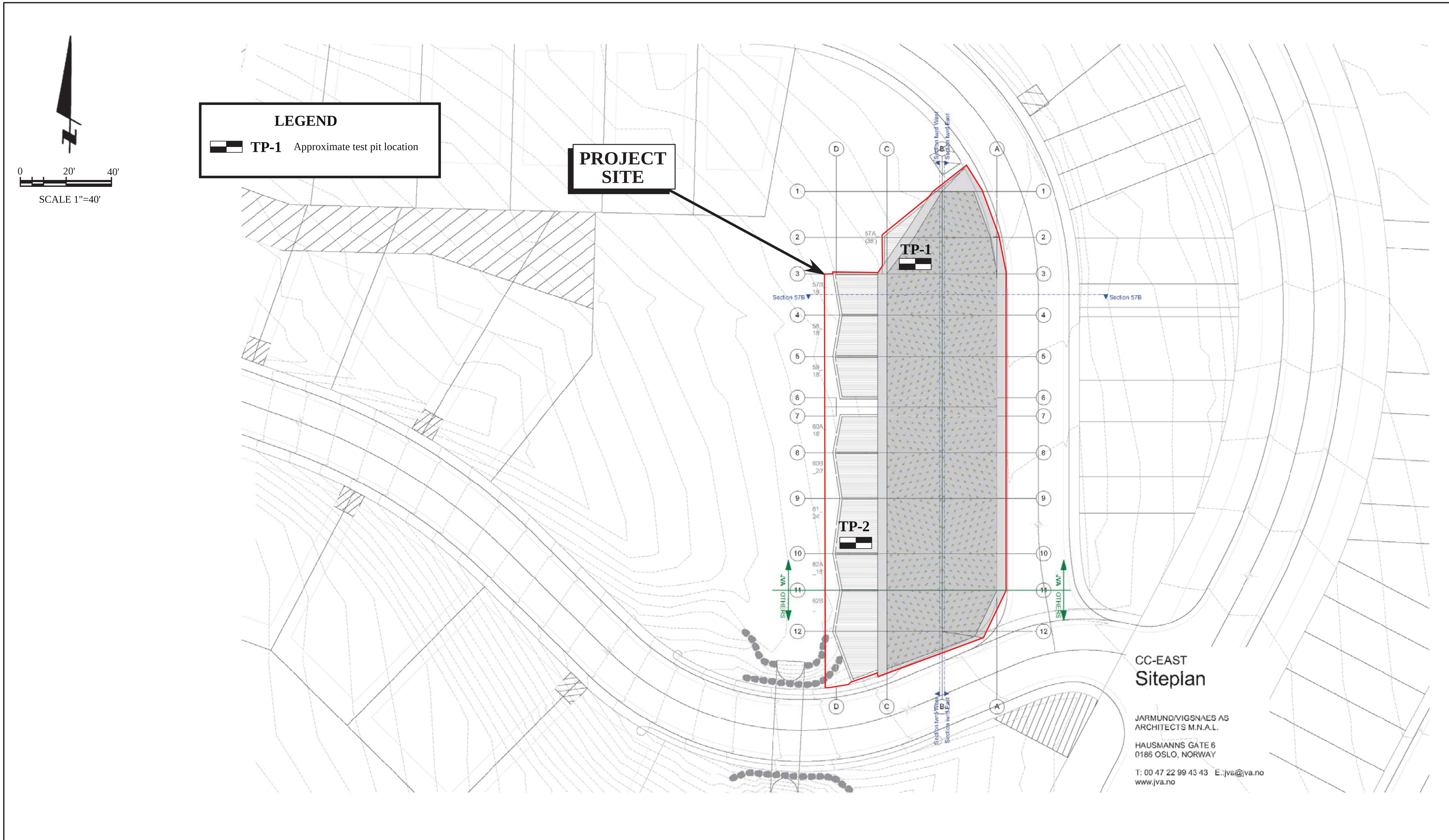
Geotechnical Investigation
Copper Crest East
Powder Mountain Resort
Weber County, Utah

SITE VICINITY MAP

Figure

A-1



Basemap: Undated 32-scale site plan prepared by Jarmund/Vigsnaes AS Architects M.N.A.L. presented in the plan set titled "Copper Crest - East, Preliminary Concept Design, Second Phase", dated 09-29-14

IGES®

Project No. 01628-010

Geotechnical Investigation
Copper Crest East
Powder Mountain Resort
Weber County, Utah

GEOTECHNICAL MAP

Figure

A-2

DATE	
STARTED:	11/6/14
COMPLETED:	11/6/14
BACKFILLED:	11/6/14

Geotechnical Investigation
Copper Crest - East
Powder Mountain Resort
Weber County, Utah

Project Number 01628-010

IGES Rep: DAG

Rig Type: 320C

TEST PIT NO: **TP-1**

Sheet 1 of 1

LOCATION

LATITUDE LONGITUDE ELEVATION 8,612

Approx. 57A

ELEVATION	DEPTH (FEET)	SAMPLES	WATER LEVEL	GRAPHICAL LOG	UNIFIED SOIL CLASSIFICATION	MATERIAL DESCRIPTION	Dry Density(pcf)	Moisture Content %	Percent minus 200	Liquid Limit	Plasticity Index	Moisture Content and Atterberg Limits (Plastic Limit, Moisture Content, Liquid Limit; 10 20 30 40 50 60 70 80 90)
	0					Topsoil - Silty CLAY, moist, dark brown, well-rooted with several tree roots, loamy appearance, soft, low plasticity, well-developed topsoil						
8610					GC	@ 2½' **Colluvium (Qc)** Clayey GRAVEL, loose to medium dense, coarse, subrounded gravel and cobble to 4 inches, some rounded boulders to 3 feet, light brown, slightly moist, trace roots, low plasticity (clayey matrix is below plastic limit), clast-supported						
		Grab sample						3.4	22.1	37	14	
	5											
8605												
	10					Total depth 9½ feet No groundwater No caving Bottom of Test Pit @ 9.5 Feet						
8600												

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SAMPLE TYPE
- GRAB SAMPLE
- 3" O.D. THIN-WALLED HAND SAMPLER

WATER LEVEL
- MEASURED
- ESTIMATED

NOTES:

FIGURE

A - 3

DATE		
STARTED:	11/6/14	
COMPLETED:	11/6/14	
BACKFILLED:	11/6/14	

Geotechnical Investigation
Copper Crest - East
Powder Mountain Resort
Weber County, Utah

Project Number 01628-010

IGES Rep: DAG
Rig Type: 320C

TEST PIT NO: **TP-2**

Sheet 1 of 1

LOCATION

LATITUDE | LONGITUDE | ELEVATION 8,605

Approx. 61

ELEVATION	DEPTH (FEET)	SAMPLES	WATER LEVEL	GRAPHICAL LOG	UNIFIED SOIL CLASSIFICATION	MATERIAL DESCRIPTION	Dry Density(pcf)	Moisture Content %	Percent minus 200	Liquid Limit	Plasticity Index	Moisture Content and Atterberg Limits (Plastic Limit, Moisture Content, Liquid Limit; 10 20 30 40 50 60 70 80 90)
	0					@ 0' Topsoil - Silty CLAY, low plasticity, moist, dark brown, loamy appearance, well-rooted, soft, up to 3 feet thick locally						
					GC	@ 2½' **Colluvium (Qc)** Clayey GRAVEL, medium dense, coarse, light brown, moist, subrounded gravel and cobble within a clayey matrix, clast-supported, occasional boulders to 3 feet, most cobble-size constituents are <6 inches, trace roots, well-cemented			19.6			
8600	5											
8595	10					Total depth 9 feet No groundwater No caving Bottom of Test Pit @ 9 Feet						

SAMPLE TYPE
- GRAB SAMPLE
- 3" O.D. THIN-WALLED HAND SAMPLER

WATER LEVEL
- MEASURED
- ESTIMATED

NOTES:

FIGURE A - 4

UNIFIED SOIL CLASSIFICATION SYSTEM

MAJOR DIVISIONS			USCS SYMBOL	TYPICAL DESCRIPTIONS
COARSE GRAINED SOILS (More than half of material is larger than the #200 sieve)	GRAVELS (More than half of coarse fraction is larger than the #4 sieve)	CLEAN GRAVELS WITH LITTLE OR NO FINES	GW	WELL-GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE OR NO FINES
			GP	POORLY-GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE OR NO FINES
		GRAVELS WITH OVER 12% FINES	GM	SILTY GRAVELS, GRAVEL-SILT-SAND MIXTURES
			GC	CLAYEY GRAVELS, GRAVEL-SAND-CLAY MIXTURES
	SANDS (More than half of coarse fraction is smaller than the #4 sieve)	CLEAN SANDS WITH LITTLE OR NO FINES	SW	WELL-GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE OR NO FINES
			SP	POORLY-GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE OR NO FINES
		SANDS WITH OVER 12% FINES	SM	SILTY SANDS, SAND-GRAVEL-SILT MIXTURES
			SC	CLAYEY SANDS SAND-GRAVEL-CLAY MIXTURES
FINE GRAINED SOILS (More than half of material is smaller than the #200 sieve)	SILTS AND CLAYS (Liquid limit less than 50)		ML	INORGANIC SILTS & VERY FINE SANDS, SILTY OR CLAYEY FINE SANDS, CLAYEY SILTS WITH SLIGHT PLASTICITY
			CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
			OL	ORGANIC SILTS & ORGANIC SILTY CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS (Liquid limit greater than 50)		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SAND OR SILT
			CH	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS
			OH	ORGANIC CLAYS & ORGANIC SILTS OF MEDIUM-TO-HIGH PLASTICITY
HIGHLY ORGANIC SOILS			PT	PEAT, HUMUS, SWAMP SOILS WITH HIGH ORGANIC CONTENTS

LOG KEY SYMBOLS

BORING SAMPLE LOCATION

TEST-PIT SAMPLE LOCATION

WATER LEVEL (level after completion)

WATER LEVEL (level where first encountered)

CEMENTATION

DESCRIPTION	DESCRIPTION
WEAKELY	CRUMBLES OR BREAKS WITH HANDLING OR SLIGHT FINGER PRESSURE
MODERATELY	CRUMBLES OR BREAKS WITH CONSIDERABLE FINGER PRESSURE
STRONGLY	WILL NOT CRUMBLE OR BREAK WITH FINGER PRESSURE

OTHER TESTS KEY

C	CONSOLIDATION	SA	SIEVE ANALYSIS
AL	ATTERBERG LIMITS	DS	DIRECT SHEAR
UC	UNCONFINED COMPRESSION	T	TRIAXIAL
S	SOLUBILITY	R	RESISTIVITY
O	ORGANIC CONTENT	RV	R-VALUE
CBR	CALIFORNIA BEARING RATIO	SU	SOLUBLE SULFATES
COMP	MOISTURE/DENSITY RELATIONSHIP	PM	PERMEABILITY
CI	CALIFORNIA IMPACT	-200	% FINER THAN #200
COL	COLLAPSE POTENTIAL	Gs	SPECIFIC GRAVITY
SS	SHRINK SWELL	SL	SWELL LOAD

MODIFIERS

DESCRIPTION	%
TRACE	<5
SOME	5 - 12
WITH	>12

GENERAL NOTES

1. Lines separating strata on the logs represent approximate boundaries only. Actual transitions may be gradual.
2. No warranty is provided as to the continuity of soil conditions between individual sample locations.
3. Logs represent general soil conditions observed at the point of exploration on the date indicated.
4. In general, Unified Soil Classification designations presented on the logs were evaluated by visual methods only. Therefore, actual designations (based on laboratory tests) may vary.

MOISTURE CONTENT

DESCRIPTION	FIELD TEST
DRY	ABSENCE OF MOISTURE, DUSTY, DRY TO THE TOUCH
MOIST	DAMP BUT NO VISIBLE WATER
WET	VISIBLE FREE WATER, USUALLY SOIL BELOW WATER TABLE

STRATIFICATION

DESCRIPTION	THICKNESS
SEAM	1/16 - 1/2"
LAYER	1/2 - 12"

DESCRIPTION	THICKNESS
OCCASIONAL	ONE OR LESS PER FOOT OF THICKNESS
FREQUENT	MORE THAN ONE PER FOOT OF THICKNESS

APPARENT / RELATIVE DENSITY - COARSE-GRAINED SOIL

APPARENT DENSITY	SPT (blows/ft)	MODIFIED CA. SAMPLER (blows/ft)	CALIFORNIA SAMPLER (blows/ft)	RELATIVE DENSITY (%)	FIELD TEST
VERY LOOSE	<4	<4	<5	0 - 15	EASILY PENETRATED WITH 1/2-INCH REINFORCING ROD PUSHED BY HAND
LOOSE	4 - 10	5 - 12	5 - 15	15 - 35	DIFFICULT TO PENETRATE WITH 1/2-INCH REINFORCING ROD PUSHED BY HAND
MEDIUM DENSE	10 - 30	12 - 35	15 - 40	35 - 65	EASILY PENETRATED A FOOT WITH 1/2-INCH REINFORCING ROD DRIVEN WITH 5-LB HAMMER
DENSE	30 - 50	35 - 60	40 - 70	65 - 85	DIFFICULT TO PENETRATED A FOOT WITH 1/2-INCH REINFORCING ROD DRIVEN WITH 5-LB HAMMER
VERY DENSE	>50	>60	>70	85 - 100	PENETRATED ONLY A FEW INCHES WITH 1/2-INCH REINFORCING ROD DRIVEN WITH 5-LB HAMMER

CONSISTENCY - FINE-GRAINED SOIL

CONSISTENCY	SPT (blows/ft)	TORVANE UNTRAINED SHEAR STRENGTH (tsf)	POCKET PENETROMETER UNCONFINED COMPRESSIVE STRENGTH (tsf)	FIELD TEST
VERY SOFT	<2	<0.125	<0.25	EASILY PENETRATED SEVERAL INCHES BY THUMB. EXUDES BETWEEN THUMB AND FINGERS WHEN SQUEEZED BY HAND.
SOFT	2 - 4	0.125 - 0.25	0.25 - 0.5	EASILY PENETRATED ONE INCH BY THUMB. MOLDED BY LIGHT FINGER PRESSURE.
MEDIUM STIFF	4 - 8	0.25 - 0.5	0.5 - 1.0	PENETRATED OVER 1/2 INCH BY THUMB WITH MODERATE EFFORT. MOLDED BY STRONG FINGER PRESSURE.
STIFF	8 - 15	0.5 - 1.0	1.0 - 2.0	INDENTED ABOUT 1/2 INCH BY THUMB BUT PENETRATED ONLY WITH GREAT EFFORT.
VERY STIFF	15 - 30	1.0 - 2.0	2.0 - 4.0	READILY INDENTED BY THUMBNAIL.
HARD	>30	>2.0	>4.0	INDENTED WITH DIFFICULTY BY THUMBNAIL.

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Key to Soil Symbols and Terminology

Figure A-5

IGES, Inc. Project No.:01628-010

APPENDIX B

Water Content and Unit Weight of Soil

(In General Accordance with ASTM D7263 Method B and D2216)

IGES

Project: Summit/Copper
No: 01628-010
Location: Weber County, Utah
Date: 11/10/2014
By: NB

Sample Info.	Boring No.	TP-1							
	Sample								
	Depth	4 to 5'							
	Split	Yes							
	Split sieve	3/4"							
	Total sample (g)	23072.40							
	Moist coarse fraction (g)	10412.70							
	Moist split fraction (g)	12659.70							
	Sample height, H (in)								
	Sample diameter, D (in)								
	Mass rings + wet soil (g)								
	Mass rings/tare (g)								
	Moist unit wt., γ_m (pcf)								
Coarse Fraction	Wet soil + tare (g)	10840.40							
	Dry soil + tare (g)	10768.60							
	Tare (g)	698.38							
	Water content (%)	0.7							
Split Fraction	Wet soil + tare (g)	1629.25							
	Dry soil + tare (g)	1563.24							
	Tare (g)	408.83							
	Water content (%)	5.7							
	Water Content, w (%)	**3.4**							
	Dry Unit Wt., γ_d (pcf)								

Entered by:___________

Reviewed:___________

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Liquid Limit, Plastic Limit, and Plasticity Index of Soils

(ASTM D4318)

Project: Summit/Copper
No: 01628-010
Location: Weber County, Utah
Date: 11/10/2014
By: BRR

Boring No.: TP-1
Sample:
Depth: 4 to 5'
Description: Brown lean clay

Preparation method: Wet
Liquid limit test method: Multipoint

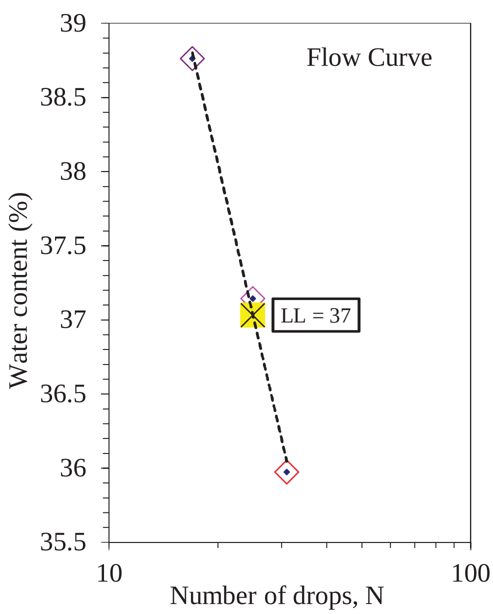
Plastic Limit

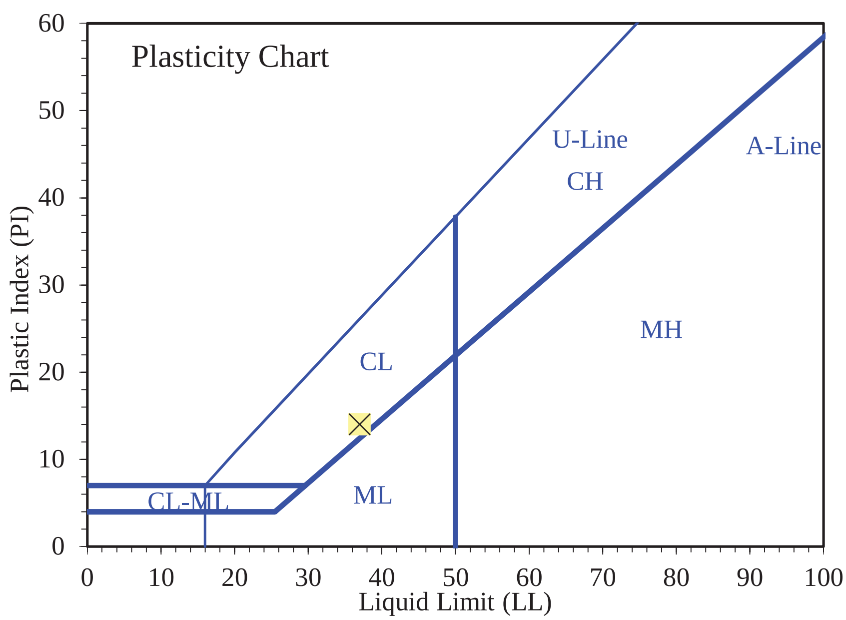
Determination No	1	2				
Wet Soil + Tare (g)	28.04	28.30				
Dry Soil + Tare (g)	26.89	27.08				
Water Loss (g)	1.15	1.22				
Tare (g)	21.97	21.77				
Dry Soil (g)	4.92	5.31				
Water Content, w (%)	23.37	22.98				

Liquid Limit

Determination No	1	2	3			
Number of Drops, N	31	25	17			
Wet Soil + Tare (g)	30.59	30.75	30.70			
Dry Soil + Tare (g)	28.32	28.41	28.32			
Water Loss (g)	2.27	2.34	2.38			
Tare (g)	22.01	22.11	22.18			
Dry Soil (g)	6.31	6.30	6.14			
Water Content, w (%)	35.97	37.14	38.76			
One-Point LL (%)		37				

Liquid Limit, LL (%)	**37**
Plastic Limit, PL (%)	**23**
Plasticity Index, PI (%)	**14**





Entered by:____________
Reviewed:____________

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Particle-Size Distribution (Gradation) of Soils Using Sieve Analysis

(ASTM D6913)

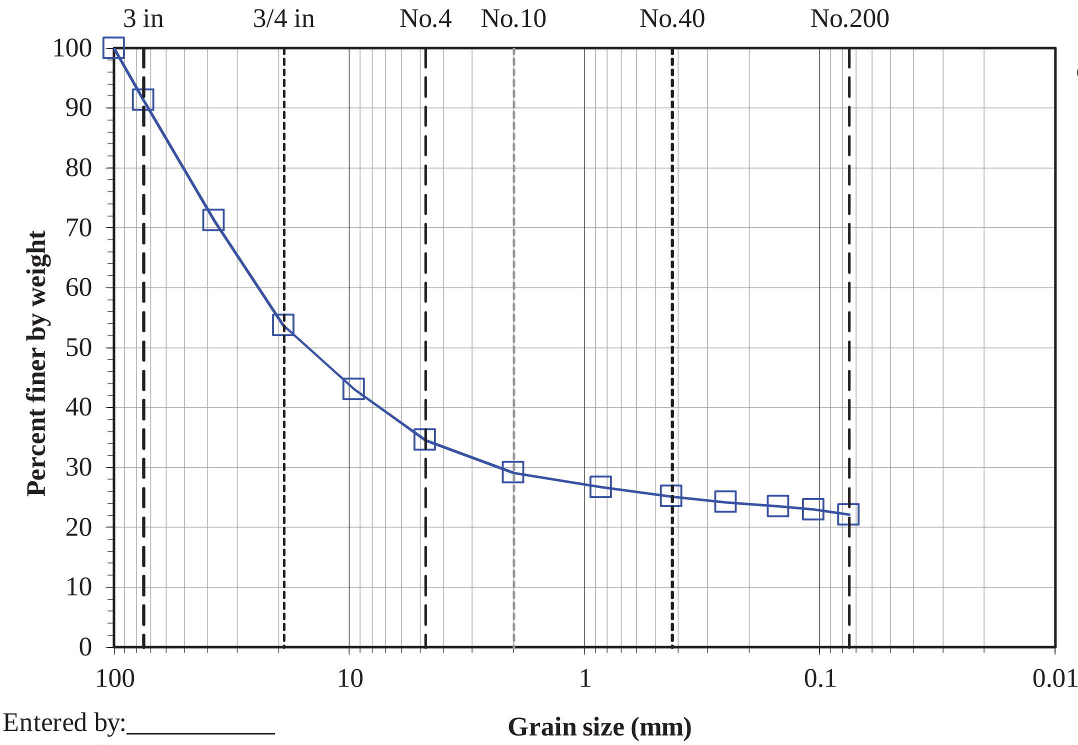
Project: Summit/Copper
No: 01628-010
Location: Weber County, Utah
Date: 11/10/2014
By: NB

Boring No.: TP-1
Sample:
Depth: 4 to 5'
Description: Brown clayey gravel

Split:	Yes	
Split sieve:	3/4"	
	Moist	Dry
Total sample wt. (g):	23072.40	22301.25
+3/4" Coarse fraction (g):	10142.70	10070.90
-3/4" Split fraction (g):	1242.38	1175.18
Split fraction:	0.548	

Water content data	C.F.(+3/4")	S.F.(-3/4")
Moist soil + tare (g):	10840.40	1629.25
Dry soil + tare (g):	10768.60	1563.24
Tare (g):	698.38	408.83
Water content (%):	0.7	5.7

Sieve	Accum. Wt. Ret. (g)	Grain Size (mm)	Percent Finer	
8"	-	200	-	
6"	-	150	-	
4"	-	100	100.0	
3"	1936.59	75	91.3	
1.5"	6427.37	37.5	71.2	
3/4"	10338.98	19	53.6	←Split
3/8"	253.99	9.5	43.0	
No.4	435.05	4.75	34.5	
No.10	551.86	2	29.1	
No.20	603.25	0.85	26.7	
No.40	636.27	0.425	25.1	
No.60	657.14	0.25	24.2	
No.100	672.66	0.15	23.5	
No.140	683.25	0.106	23.0	
No.200	701.42	0.075	22.1	



Gravel (%): 65.5
Sand (%): 12.4
Fines (%): 22.1

Entered by:___________
Reviewed:___________

Particle-Size Distribution (Gradation) of Soils Using Sieve Analysis

(ASTM D6913)

IGES

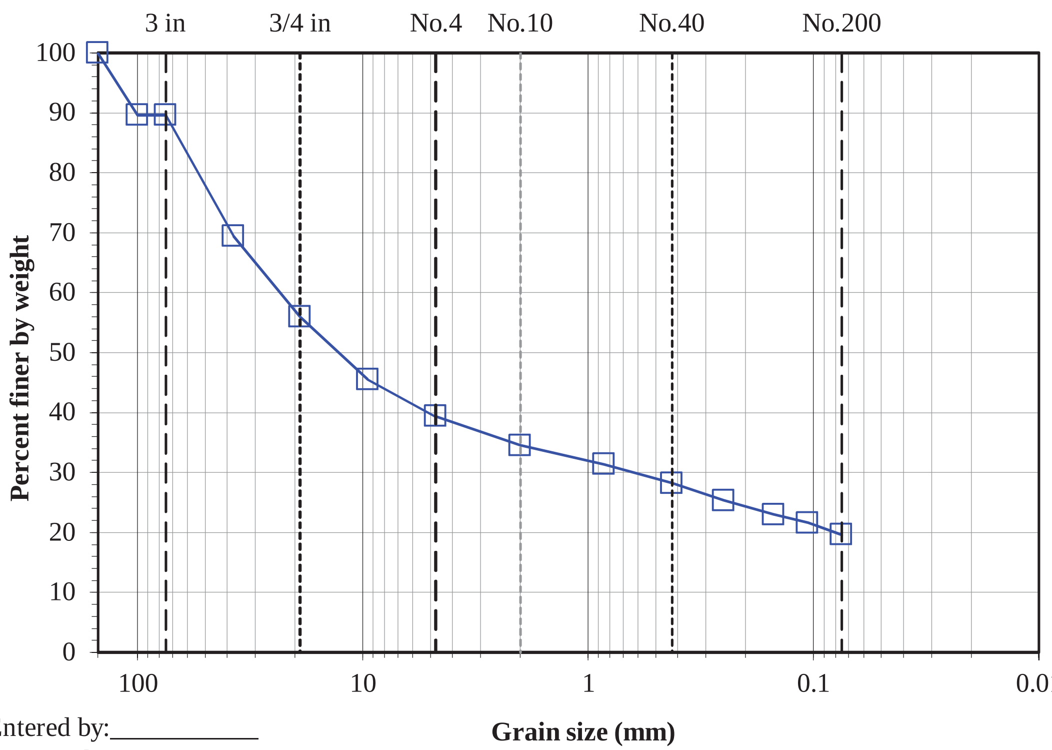
Project: Summit/Copper
No: 01628-010
Location: Weber County, Utah
Date: 11/10/2014
By: NB

Boring No.: TP-2
Sample:
Depth: 3 to 4'
Description: Light brown clayey gravel with sand

Split:	Yes	
Split sieve:	3/4"	
	Moist	Dry
Total sample wt. (g):	19391.80	18742.90
+3/4" Coarse fraction (g):	8380.30	8251.78
-3/4" Split fraction (g):	2262.44	2155.52
Split fraction:	0.560	

Water content data	C.F.(+3/4")	S.F.(-3/4")
Moist soil + tare (g):	9062.80	3312.86
Dry soil + tare (g):	8934.60	3171.74
Tare (g):	703.20	326.70
Water content (%):	1.6	5.0

Sieve	Accum. Wt. Ret. (g)	Grain Size (mm)	Percent Finer
8"	-	200	-
6"	-	150	100.0
4"	1945.20	100	89.6
3"	1945.20	75	89.6
1.5"	5736.75	37.5	69.4
3/4"	8251.78	19	56.0 ←Split
3/8"	404.25	9.5	45.5
No.4	641.46	4.75	39.3
No.10	828.22	2	34.5
No.20	949.48	0.85	31.3
No.40	1071.24	0.425	28.2
No.60	1182.14	0.25	25.3
No.100	1271.81	0.15	22.9
No.140	1325.93	0.106	21.5
No.200	1401.83	0.075	19.6



Gravel (%): 60.7
Sand (%): 19.7
Fines (%): 19.6

Entered by:___________
Reviewed:___________

Minimum Laboratory Soil Resistivity, pH of Soil for Use in Corrosion Testing, and Ions in Water by Chemically Suppressed Ion Chromatography *(AASHTO T 288, T 289, ASTM D4327, and C1580)*

IGES

Project: **Summit/Copper**
No: **01628-010**
Location: Weber County, Utah
Date: 11/10/2014
By: JDF

Section	Field	Approximate Soil condition (%)	Resistance Reading (Ω)	Soil Box Multiplier (cm)	Resistivity (Ω-cm)	Approximate Soil condition (%)	Resistance Reading (Ω)	Soil Box Multiplier (cm)	Resistivity (Ω-cm)
Sample info.	Boring No.	TP-1							
	Sample								
	Depth	4 to 5'							
Water content data	Wet soil + tare (g)	171.02							
	Dry soil + tare (g)	165.74							
	Tare (g)	123.57							
	Water content (%)	12.5							
Chem. data	pH	5.63							
	Soluble chloride* (ppm)	< 5.38							
	Soluble sulfate** (ppm)	18.1							
Resistivity data	Pin method	2							
	Soil box	Miller Small							
		As is	33600	0.67	22512				
		+3	19740	0.67	13226				
		+6	14000	0.67	9380				
		+9	11840	0.67	7933				
		+12	10940	0.67	7330				
		+15	10970	0.67	7350				
	Minimum resistivity (Ω-cm)	**7330**							

* Performed by AWAL using EPA 300.0

** Performed by AWAL using ASTM C1580

Entered by:___________

Reviewed:___________

APPENDIX C

USGS Design Maps Detailed Report

2012 International Building Code (41.3627°N, 111.7445°W)

Site Class D – "Stiff Soil", Risk Category I/II/III

Section 1613.3.1 — Mapped acceleration parameters

Note: Ground motion values provided below are for the direction of maximum horizontal spectral response acceleration. They have been converted from corresponding geometric mean ground motions computed by the USGS by applying factors of 1.1 (to obtain S_S) and 1.3 (to obtain S_1). Maps in the 2012 International Building Code are provided for Site Class B. Adjustments for other Site Classes are made, as needed, in Section 1613.3.3.

From Figure 1613.3.1(1) [1] $S_S = 0.810$ g

From Figure 1613.3.1(2) [2] $S_1 = 0.269$ g

Section 1613.3.2 — Site class definitions

The authority having jurisdiction (not the USGS), site-specific geotechnical data, and/or the default has classified the site as Site Class D, based on the site soil properties in accordance with Section 1613.

2010 ASCE-7 Standard – Table 20.3-1
SITE CLASS DEFINITIONS

Site Class	$\bar{v}_s$	$\bar{N}$ or $\bar{N}_{ch}$	$\bar{s}_u$
A. Hard Rock	>5,000 ft/s	N/A	N/A
B. Rock	2,500 to 5,000 ft/s	N/A	N/A
C. Very dense soil and soft rock	1,200 to 2,500 ft/s	>50	>2,000 psf
D. Stiff Soil	600 to 1,200 ft/s	15 to 50	1,000 to 2,000 psf
E. Soft clay soil	<600 ft/s	<15	<1,000 psf
	Any profile with more than 10 ft of soil having the characteristics: • Plasticity index *PI* > 20, • Moisture content $w \geq 40\%$, and • Undrained shear strength $\bar{s}_u$ < 500 psf		
F. Soils requiring site response analysis in accordance with Section 21.1	See Section 20.3.1		

For SI: 1ft/s = 0.3048 m/s 1lb/ft² = 0.0479 kN/m²

Section 1613.3.3 — Site coefficients and adjusted maximum considered earthquake spectral response acceleration parameters

TABLE 1613.3.3(1)
VALUES OF SITE COEFFICIENT F_a

Site Class	Mapped Spectral Response Acceleration at Short Period				
	$S_S \le 0.25$	$S_S = 0.50$	$S_S = 0.75$	$S_S = 1.00$	$S_S \ge 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	See Section 11.4.7 of ASCE 7				

Note: Use straight–line interpolation for intermediate values of S_S

For Site Class = D and S_S = 0.810 g, F_a = 1.176

TABLE 1613.3.3(2)
VALUES OF SITE COEFFICIENT F_v

Site Class	Mapped Spectral Response Acceleration at 1–s Period				
	$S_1 \le 0.10$	$S_1 = 0.20$	$S_1 = 0.30$	$S_1 = 0.40$	$S_1 \ge 0.50$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	2.4
F	See Section 11.4.7 of ASCE 7				

Note: Use straight–line interpolation for intermediate values of S_1

For Site Class = D and S_1 = 0.269 g, F_v = 1.863

Equation (16-37): $S_{MS} = F_aS_S = 1.176 \times 0.810 = 0.953$ g

Equation (16-38): $S_{M1} = F_vS_1 = 1.863 \times 0.269 = 0.500$ g

Section 1613.3.4 — Design spectral response acceleration parameters

Equation (16-39): $S_{DS} = ⅔\, S_{MS} = ⅔ \times 0.953 = 0.635$ g

Equation (16-40): $S_{D1} = ⅔\, S_{M1} = ⅔ \times 0.500 = 0.334$ g

Section 1613.3.5 — Determination of seismic design category

TABLE 1613.3.5(1)
SEISMIC DESIGN CATEGORY BASED ON SHORT-PERIOD (0.2 second) RESPONSE ACCELERATION

VALUE OF S_{DS}	RISK CATEGORY		
	I or II	III	IV
$S_{DS} < 0.167g$	A	A	A
$0.167g \le S_{DS} < 0.33g$	B	B	C
$0.33g \le S_{DS} < 0.50g$	C	C	D
$0.50g \le S_{DS}$	D	D	D

For Risk Category = I and S_{DS} = 0.635 g, Seismic Design Category = D

TABLE 1613.3.5(2)
SEISMIC DESIGN CATEGORY BASED ON 1-SECOND PERIOD RESPONSE ACCELERATION

VALUE OF S_{D1}	RISK CATEGORY		
	I or II	III	IV
$S_{D1} < 0.067g$	A	A	A
$0.067g \le S_{D1} < 0.133g$	B	B	C
$0.133g \le S_{D1} < 0.20g$	C	C	D
$0.20g \le S_{D1}$	D	D	D

For Risk Category = I and S_{D1} = 0.334 g, Seismic Design Category = D

Note: When S_1 is greater than or equal to 0.75g, the Seismic Design Category is **E** for buildings in Risk Categories I, II, and III, and **F** for those in Risk Category IV, irrespective of the above.

Seismic Design Category ≡ “the more severe design category in accordance with Table 1613.3.5(1) or 1613.3.5(2)” = D

Note: See Section 1613.3.5.1 for alternative approaches to calculating Seismic Design Category.

References

1. *Figure 1613.3.1(1)*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/IBC-2012-Fig1613p3p1(1).pdf
2. *Figure 1613.3.1(2)*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/IBC-2012-Fig1613p3p1(2).pdf

Design Maps Summary Report

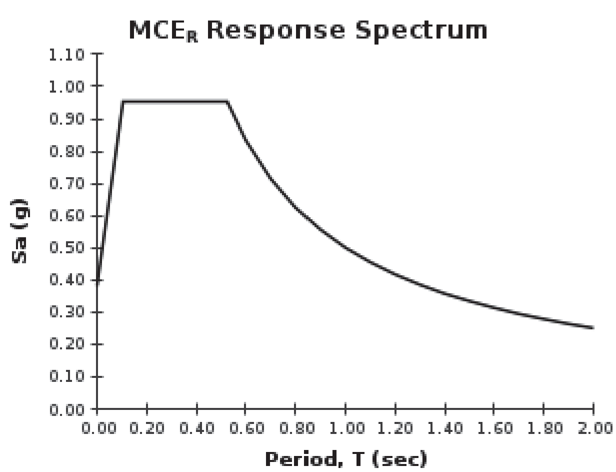
User–Specified Input

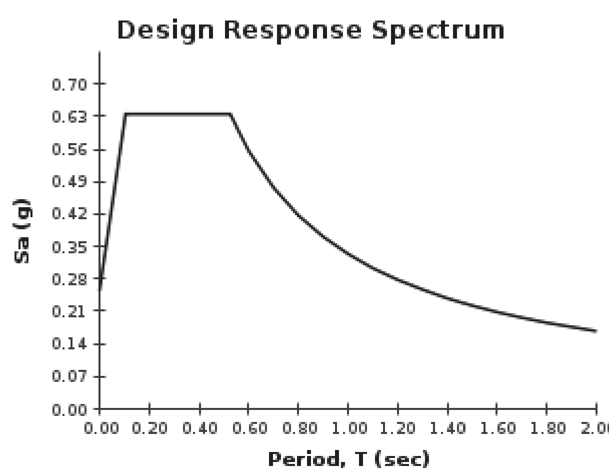
Report Title	Copper Crest - East Wed November 12, 2014 01:02:55 UTC
Building Code Reference Document	2012 International Building Code (which utilizes USGS hazard data available in 2008)
Site Coordinates	41.3627°N, 111.7445°W
Site Soil Classification	Site Class D – "Stiff Soil"
Risk Category	I/II/III

USGS–Provided Output

S_S =	0.810 g	S_{MS} =	0.953 g	S_{DS} =	0.635 g
S_1 =	0.269 g	S_{M1} =	0.500 g	S_{D1} =	0.334 g

For information on how the SS and S1 values above have been calculated from probabilistic (risk-targeted) and deterministic ground motions in the direction of maximum horizontal response, please return to the application and select the "2009 NEHRP" building code reference document.





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