ENGINEERING GEOLOGY AND GEOTECHNICAL ENGINEERING REPORT
PROPOSED ALICE CLAIM DEVELOPMENT
PARK CITY, UTAH

Submitted To:

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October 21, 2014

Project No. 6-817-005165

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Attention: Mr. Gregg Brown

**SUBJECT: Engineering Geology and Geotechnical Engineering Report
Alice Claim Development
Park City, Utah
AMEC Project No. 6-817-005165**

1. INTRODUCTION

1.1 Objectives and Scope

This report presents the results of our geotechnical investigation for the proposed Alice Claim residential development to be located in Woodside Gulch south of the intersection of King Road and Ridge Avenue in Park City, Utah. The objective of our study was to evaluate engineering geology and geotechnical engineering characteristics of project area and develop recommendations for design and construction of the project. The studies were conducted in accordance with the scope of work outlined in AMEC proposal No. PL06-074 dated June 8, 2006 and authorized by King Development Group, LLC on June 16, 2006. The scope of work included a site reconnaissance, field explorations, laboratory testing, engineering analyses, and report preparation.

2. PROPOSED CONSTRUCTION

We understand the project will include development of private streets and utility access to nine proposed residential lots that range from about 0.22 acres to 3.0 acres in area. Proposed building and grading plans for the individual lots have not been finalized. The project also includes 3.05 acres of natural open space, 0.37 acres of landscaped open space, and 0.34 acres dedicated to Park City Municipal Corporation.

3. SITE DESCRIPTION

3.1 Site Conditions

The project site is located in an undeveloped area of Woodside Gulch at the south end of old-town Park City. Woodside Gulch is a north-trending drainage with east and west facing side slopes. An abandoned mine dump was located on the east side of the drainage bottom. An abandoned water storage reservoir is located on the southern portion of the property on the ridge top between Woodside Gulch and Daly Canyon. Ground surface vegetation consists primarily of oak brush and scattered deciduous and evergreen trees.

3.2 Topography

Slope angles range from about 10 degrees in the bottom of Woodside Gulch up to about 37 degrees on the side slopes of the drainages, and up to about 60 degrees at localized rock outcrops on the western slope of the drainage. Ground surface elevations range from about 7,490 feet on the western and eastern slopes of Woodside Gulch to about elevation 7,300 feet at King Road.

3.3 Geology

The project site is located in the Middle Rocky Mountain physiographic province. The Middle Rocky Mountain physiographic province is characterized by a complex system of mountain ranges with intermountain basins and plains formed during mountain building episodes, the latest of which, known as the Laramide Orogeny occurred about 70 to 40 million years ago (late Cretaceous and early Tertiary periods).

Seismically, the project site is located within the Intermountain Seismic Belt (ISB), a zone of earthquake activity that runs north-south through the Intermountain West from northwestern Montana in the North, through Wyoming, Idaho, and Utah, and southern Nevada/northern Arizona to the south. Most earthquakes in the ISB are shallow and occur at depths less than 12 miles (20 km). There have been approximately 50 moderate-to-large (magnitude 5.5 to 7.5) earthquakes in this zone since 1900.

The Wasatch fault is located within the ISB and delineates the boundary between the Basin and Range and Middle Rocky Mountain and Colorado Plateau physiographic provinces. The Wasatch fault is considered active and, although has not produced large earthquakes in historic time, is believed capable of producing earthquake magnitudes greater than 7.0 (Richter scale). According to McCalpin and Nishencko (1996), the combined average repeat time for large earthquakes (magnitude greater than 7) on any of the 5 central segments (Brigham City, Weber, Salt Lake City, Provo, and Nephi segments) of the Wasatch fault zone is 350 years. The average return time on any single segment ranges from about 1,200 to 2,600 years. The time since the last earthquakes on the 5 central segments ranges from 620 to 2,120 years.

Based on a review of a geologic map prepared by Bromfield and Crittenden, Jr., (1971) the project site is underlain by the Pennsylvanian-age Weber Quartzite Formation, consisting of pale-gray and tan-weathering quartzite and limy sandstone with some interbedded layers of gray to white limestone and dolomite.

4. FIELD EXPLORATION & LABORATORY TESTING

4.1 Field Exploration

4.1.1 Geologic Reconnaissance

A ground level reconnaissance of the project area was completed on July 12, 2006 by a licensed geologist and geotechnical engineer in the State of Utah. Outcrops of hard, fractured quartzite bedrock were observed in the road cut in the bottom of Woodside Gulch and on the adjacent drainage slopes. The bedding plane orientation of the rock dips steeply in varying directions. Field measurements of bedding plane orientations (strike and dip) ranged from N35E 64NW in the road cut in the bottom of Woodside Gulch to N30W 86NE on the ridge top north of the abandoned reservoir.

Evidence of previous mining activity was apparent in the form of a mine waste dump in the bottom of the drainage. A mine shaft and adit was discovered in one of the test pits made for the field exploration. Mine waste was also observed east of the project area on the west slope of Daly Canyon. No openings were observed, but it appears from aerial photographs (See Figure 2) that there may have been two mine prospects in that area at one time.

Evidence of deep-seated landsliding was not observed on the natural slopes within the project area. Some raveling and shallow sloughing was observed in unvegetated areas on the slope above the mine waste dump on the east slope of Woodside Gulch.

4.1.2 Test Pits

Subsurface materials and conditions at the project site were investigated on June 28, 2006 with 5 test pits designated TP-1 through TP-5. The approximate locations of the test pits are shown on Figure 2, Site Plan. All field operations were observed by a staff engineer provided by our firm, who maintained a detailed log of the materials and conditions encountered in each test pit and directed the sampling operation. A detailed description of the field exploration program is presented in Appendix A.

4.2 Laboratory Testing

Laboratory testing completed for the project included determinations of natural moisture content, grain size analysis, Atterberg limits, pH, resistivity, and soluble sulfate concentration. A description of the test procedures and results is presented in Appendix B, Laboratory Testing.

5. SUBSURFACE CONDITIONS

5.1 Soil and Rock

Logs of Test Pits TP-1 through TP-5 are presented on Figures 3A through 3E. The terms used to describe the soil and rock disclosed by the test pits are defined on Figures 4 and 5, respectively. For the purpose of discussion, the materials disclosed by the explorations have been grouped into 2 major units based on their physical characteristics and engineering properties. The units are:

1. **Clayey Sand and Gravel (Colluvium)**
2. **Quartzite (Weber Quartzite Formation)**

1. Clayey Sand and Gravel (Colluvium). Sand and gravel soils containing variable percentages of silt and clay and angular cobbles were encountered at the ground surface extending to depths ranging from about 1.5 to 3 feet below the ground surface. The sand and gravel soils are typically dark brown and contain roots and organic material. Gravel clasts are comprised of tan to yellowish-brown angular quartzite. The relative density is estimated at loose to medium dense based on excavation effort.

2. Quartzite. Beneath the colluvial soils, the test pits encountered hard quartzite of the Weber Quartzite Formation. The quartzite is tan to yellowish brown in color, hard (RH-4), moderately weathered and has close to very closely spaced joints. Practical excavation refusal was encountered in TP-3 and TP-4 on hard rock at depths of 5 and 12 feet, respectively.

5.2 Groundwater

Groundwater was not encountered in the test pit excavations at the time the field explorations were performed. Fluctuations in groundwater levels can occur due to variations in precipitation, runoff, water levels in nearby ditches, drainages and other factors. Longer-term groundwater fluctuations should be anticipated with the highest seasonal levels generally occurring during the late spring and early summer.

Perched groundwater conditions, seeps and springs should be anticipated on hillsides and near the bottoms of local drainages during and following periods of prolonged precipitation and snow melt. The potential for perched groundwater, seeps and springs is enhanced by the presence of shallow bedrock and topographic relief across the site.

6. CONCLUSIONS AND RECOMMENDATIONS

6.1 General

The project can be developed with careful planning and engineering. The most significant engineering geology and geotechnical aspects which could affect design and construction at the site are:

1. **Previous Mining Activities**
2. **Strong Ground Motion**
3. **Slope Stability**
4. **Debris Flow and Avalanche**
5. **Shallow Bedrock**
6. **Perched Groundwater (Seeps And Springs)**

More detailed discussions pertaining to the engineering geology and preliminary geotechnical engineering recommendations are presented in the following sections.

6.2 Engineering Geology

6.2.1 Hazards

The term geologic hazard refers to a geologic condition, either natural or man-made, that poses a potential danger to life and property. Common examples include earthquakes, landslides, flooding, volcanoes, and tsunamis. Specific geologic hazards vary with location. In Utah, potential geologic hazards include seismically-induced ground motion, surface fault rupture, liquefaction, landslides, debris flow, avalanche and rockfall. Another potential hazard related to geology is mining. The following sections briefly describe these potential hazards and present information pertinent to the project site.

6.2.1.1 Previous Mining Activities

A mine shaft and associated adit was encountered in test pit TP-1 located about 10 feet south of the center of Lot 4 (see Figure 2). Measurements indicate that the shaft has a diameter of 6 to 8 feet and a depth of over 230 feet. The adit (horizontal opening) is located just below the ground surface and extends from the shaft into the hillside for an unknown distance. An approximate bearing of N20W was estimated for the trend of the adit at the shaft opening.

The shaft and adit represent a public safety hazard and a potential for property damage resulting form ground subsidence. In our opinion, the openings should be closed to prevent accidental entry and potential subsidence. Typically mine openings are closed by backfilling and capping with concrete. Closure should be performed in accordance with Utah Division of Oil & Gas and Mining Abandoned Mine Reclamation Program Guidelines. Structures should not be located over the closed shaft and adit.

The existing mine dump materials are unsuitable for support of roadways, utilities, or other structures.

6.2.1.2 Seismic Ground Motion

The International Building Code (IBC) 2012 determines the seismic hazard for a site based upon regional acceleration mapping prepared by the United States Geologic Survey (USGS) and the soil site class. The structures should be designed in accordance with the procedures presented in Chapter 16 of the IBC 2012 edition.

Design spectral acceleration values are based on information obtained from the USGS 2008 Hazard Data for the maximum considered earthquake (MCE). For the Wasatch fault zone, the MCE ground acceleration is associated with approximately a 2 percent probability of being exceeded in 50 years or a 2,475-yr return period. Design spectral acceleration values are calculated as 2/3 of the maximum values.

The results of the investigation indicate that Site Class B (Rock) as described in Section 1613.3.2 of the 2012 edition of the International Building Code (IBC) best characterizes the site class definition for the project area. Using 40.6371 degrees north latitude and 111.4972 degrees west longitude as the project coordinates; seismic design criteria based on the maximum considered earthquake are summarized below.

TABLE 1. Seismic Design Criteria

Latitude/Longitude	40.6371° North, 111.4972° West		
Design Level	MCE (2,475-yr Return Period)		
Site Class	B		
	Period, T		
Parameter	T = 0 Sec	T = 0.2 Sec	T = 1.0 Sec
Spectral Acceleration for Site Class B (Rock)	PGA = 0.253 g	S_S = 0.641 g	S_1 = 0.214 g
Site Coefficient	F_{pga} = 1.0	F_a = 1.0	F_v = 1.0
Maximum Spectral Acceleration	PGA_M = 0.253 g	S_{MS} = 0.641 g	S_{M1} = 0.214 g
Design Spectral Acceleration	PGA_D = 0.253 g	S_{DS} = 0.427 g	S_{D1} = 0.143 g

6.2.1.3 Slope Stability

Active landslides were not identified in the office studies or during the field reconnaissance completed for the project. Although the steep site topography appears to be an expression of relatively strong rock materials and stable slopes, the risk of slope instability generally increases with increasing slope inclination. Site specific grading and development plans for individual lots should be reviewed by a geotechnical engineer.

6.2.1.4 Surface Fault Rupture

Large earthquakes can produce offset at the ground surface. Surface fault rupture represents a severe hazard to structures and the most common mitigation method is establishing a minimum setback distance to avoid the hazard. Active faults are not mapped in the project area; therefore, the risk of surface fault rupture affecting the project site is very low.

6.2.1.5 Liquefaction

Liquefaction is the condition where sandy soils that are submerged below groundwater loose shear strength because of increased pore water pressure induced by earthquake ground shaking. When soil liquefies, it loses strength and behaves as a viscous liquid. Structures supported on liquefiable soils can experience large settlements and buried tanks can rise to the ground surface. Loss of shear strength induced by liquefaction can also result in slope failures and lateral spreading and flow-related ground failures. In general, soils most susceptible to liquefaction are located along rivers, streams, and lake shorelines. The gravelly soils and quartzite bedrock underlying the project site are not susceptible to liquefaction.

6.2.1.6 Debris Flow, Avalanche and Rockfall

Civil design should consider hydrological aspects of the local drainages. Removal of surface vegetation resulting from grading will increase the potential for debris flows during peak storm events.

A review of the topography indicates that slopes in excess of 30 degrees are common in the project area on varying aspects, primarily east and west facing slopes. An avalanche expert should be consulted to evaluate avalanche potential and develop appropriate design impact pressures for structures.

Localized areas may be subject to rockfall hazard. Typically, these areas are associated with rock outcrops and steep terrain. Development in these areas should be evaluated by a qualified engineering geologist or geotechnical engineer.

6.3 Geotechnical Recommendations

6.3.1 Earthwork

Site civil design was in progress at the time this report was prepared and plans showing locations of roadways, proposed grading and specific structures was not available. We anticipate that some earthwork will be required to construct roadways to provide access to the lots. Because of shallow rock conditions, we recommend that civil design consider minimizing cut and fill heights to reduce rock excavation costs. The following earthwork sections provide preliminary recommendations pertaining to earthwork.

6.3.1.1 Site Preparation

The ground surface should be stripped of all vegetation, organic material, unsuitable fill, or any other deleterious material within the building and pavement areas or areas to receive structural fill. The spoil materials should be removed from the site or stockpiled on-site for use as fill in landscaped areas. Upon completion of the site stripping, the exposed subgrade should be observed by a qualified soils engineer or engineering geologist. Proof rolling with rubber-tire construction equipment may be part of this evaluation. Any soft areas during the subgrade evaluation should be over-excavated to firm undisturbed soil and backfilled with structural fill.

6.3.1.2 Excavations

We anticipate that excavations up to about 10 to 12 feet in depth will be required for roadway, and utility construction. Excavation refusal was encountered at depth ranging from about 5 to 12 feet below the ground surface in the test pits excavated for this investigation. It should be anticipated that large hydraulic excavators equipped with rock teeth, rock splitting tools, and possibly drilling and blasting techniques will be required to excavate the rock.

Temporary construction excavations in soils/bedrock not exceeding 4 feet in depth may be constructed with near-vertical side slopes. Temporary excavations slopes up to 12 feet in height may be constructed no steeper than one-half horizontal to one vertical (½H:1V). Excavation slopes greater than 12 feet and up to 20 feet should be constructed no steeper than ¾H:1V. Excavations up to 12 feet in stable bedrock may be constructed no steeper than ¼H:1V. Loose rock on the sides of the excavation should be scaled or covered with a wire mesh or some other covering to prevent rock fall. The inclination of permanent cut slopes will depend on the type of material. For planning purposes, it should be anticipated that cut steeper than 2H:1V will require retaining walls.

The contractor is solely responsible for designing and constructing stable, temporary evaluations and should shore, slope, or bench the sides of the excavations as required to maintain stability of both the excavation sides and bottom. The contractor's responsible person, as defined in 29 CFR Part 1926, should evaluate the soil exposed in the excavations as part of the contractor's safety procedures. In no case should scope height, slope inclination, or excavation depth, including utility trench evacuation depth, exceed those specified in local, State, and Federal safety regulations.

6.3.1.3 Structural Fill

On-site or imported, organic-free, fine-grained soils approved by the geotechnical engineer may be used to construct structural fills. However, fine-grained soils are sensitive to moisture content and should be placed only during the dry summer months. During the wet winter and spring months, fills should be constructed using imported, relatively clean, granular materials. All structural fills should extend to a minimum horizontal distance of 10-feet beyond the limits of buildings.

Approved, organic-free, fine-grained soils used to construct structural fills should be placed in 9-inch-thick lifts (loose) and compacted using pneumatic or segmented pad rollers to a density not less than 95 percent of the maximum dry density as determined by ASTM D 1557. Fill placed in landscaped areas should be compacted to a minimum of 90 percent ASTM D 1557. In our opinion, the moisture content of fine-grained soils at the time of compaction should be controlled to within 3 percent of optimum. Some aeration and drying of the on-site fine-grained soils may be required to meet the above recommendations for compaction.

All backfill placed in utility trench excavations within the limits of the buildings and paved areas should consist of sand, sand and gravel, or crushed rock with a maximum size of up to 1½-inch, and with not more than 5 percent passing the No. 200 sieve (washed analysis). In our opinion, the granular backfill should be placed in 9-inch-thick lifts (loose) and compacted using vibratory plate compactors or tamping units to at least 95 percent of the maximum dry density as determined by ASTM D 1557. Flooding or jetting the backfilled trenches with water to achieve the recommended compaction should not be permitted.

Fill slopes should be constructed no steeper than 2H:1V. Fills constructed on natural slopes steeper than 5H:1V should be keyed in at the toe a minimum of 2-feet below the stripped ground surface and benched into the existing hillside as the fill is constructed. The benches should be at least 8-feet wide and should be cut into the slope every 4-feet of vertical rise. The naturally occurring existing soils should be prepared and fill placed in accordance with the previously described structural fill guidelines. A representative of the geotechnical engineer should monitor the benching and fill placement operations.

6.3.1.4 Subdrainage

It should be anticipated that subdrains will be required to control groundwater flow in certain areas of mass grading, such as at the base of fills in the natural drainages. The proposed grading plans should be reviewed by the geotechnical engineer to determine possible locations for subdrains. The actual locations of the subdrains should be determined by a representative of the geotechnical engineer during construction.

Structures with embedded walls and floors should be provided with adequate drainage to reduce the potential for buildup of hydrostatic pressures behind walls and reduce the potential for water entering the building space.

6.3.2 Foundations

We anticipate that most building structures can likely be supported on conventional spread footing foundations established on suitable on-site soils, on structural fill, or on bedrock. Allowable bearing pressures will depend on the specific structure and the soil and rock conditions at the specific locations. For residential foundations, a maximum allowable bearing pressure of 2,000 psf is recommended. This allowable bearing pressure may be increased by 50 percent for wind and seismic loads.

Foundations should be established to a minimum of 42-inches below the ground surface for frost protection. Continuous and isolated column footings should have minimum dimensions of

18-inches and 24-inches, respectively. A summary of foundation design recommendations are presented below.

TABLE 2. Spread Footing Design Parameters

Minimum Embedment Depth for Frost Protection	42 in.
Minimum Width for Continuous Wall Footings	18 in.
Minimum Width for Footings Isolated Column Footings	24 in.
Net Allowable Bearing Pressure for Real Load Conditions	2,000 psf
Bearing Pressure Increase for Seismic Loading	50 percent

It should be anticipated that some overexcavation and replacement will be required to remove unsuitable soils, such as hydro-collapsible or expansive soils beneath foundations during construction.

Footings for buildings should bear on similar materials. We recommend that footing excavations that encounter relatively hard rock are overexcavated and backfilled with granular material to a depth of approximately 2-feet. The footings will then bear on more similar materials to reduce the magnitude of the potential differential settlement.

6.3.2.1 Lateral Resistance

Horizontal shear forces can be resisted partially or completely by frictional forces developed between the base of spread footings and the underlying soil and by soil passive resistance. The total frictional resistance between the footing and soil is the normal force times the coefficient of friction between the soil and the base of the footing. The normal force is the sum of the vertical forces (dead load plus real live load). We recommend ultimate values of 0.30 and 0.40 for the coefficient of friction for footings established and clay and gravel, respectively. If additional lateral resistance is required, passive earth pressures against embedded footings can be computed on the basis of an equivalent fluid having a unit weight of 300 pcf. This design passive earth pressure would be applicable only if the footing is cast neat against undisturbed soil, or if backfill for the footings is placed as granular structural fill. A combination of passive earth resistance and friction may be utilized provided that the friction component of the total is divided by 1.5.

6.3.2.2 Lateral Earth Pressures

Design lateral earth pressures for embedded walls depend on the type of construction, i.e., the ability of the wall to yield. The two possible conditions regarding the ability of the wall to yield include the at-rest and the active earth pressure cases. The at-rest earth pressure case applies to walls that are relatively rigid and laterally supported at top and bottom and therefore unable to yield. The active earth pressure case applies to walls that are capable of yielding slightly away from the backfill by either sliding or rotating about the base. A conventional cantilevered retaining wall is an example of a wall that develops the active earth pressure case by yielding.

Yielding and non-yielding walls can be designed using a lateral earth pressure based on an equivalent fluid having a unit weight of 35 and 55 pcf, respectively. The recommended lateral earth pressures are for level backfill and free-draining backfill conditions. Lateral earth pressures from seismic forces can be computed based on an equivalent fluid having a unit weight of 15 pcf and 45 pcf for the active and at-rest cases, respectively.

The total seismic lateral earth pressure is the sum of the static and seismic pressures. In contrast to the static pressure, which is represented by a triangular pressure distribution that increases in the downward direction and the resultant force is applied at 1/3H, where H is the embedded height of the wall, the seismic pressure is applied as an inverted triangular pressure distribution with the maximum at the top of the backfill and the resultant force is applied at a distance of 0.6H up from the base of the backfilled wall.

Surcharge-induced lateral loads such as wheel loads associated with traffic on the backfill behind the walls are not included. In this regard, heavy compactors and large pieces of construction equipment should not operate within a horizontal distance equal to the height of the embedded wall. Compaction close to the walls should be accomplished with hand-operated compactors.

The backfill behind embedded walls must be fully drained. The drainage system should consist of a minimum 2-foot-wide zone of free-draining granular fill adjacent to the embedded walls. The drainage layer should consist of ¾- to ¼-inch crushed rock, or similar gap graded drain rock, containing less than 2 percent passing the No. 200 sieve. A 4-inch-diameter, rigid, perforated drain pipe should be provided near the bottom of the embedded wall. A nonwoven geotextile filter fabric, such as AMOCO 4545, is recommended between the free-draining backfill and the general wall backfill to prevent contamination of the wall drain system.

6.3.3 Floor Support

To provide uniform support for the floor slab and a capillary break, we recommend the floor slab be underlain by a minimum 4-inch-thick layer of granular base course. The base course material should consist of crushed rock of up to 1-inch maximum size, with less than about 5 percent passing the No. 200 sieve (washed analysis). This material should be placed in a single lift and compacted until well keyed using a minimum of four passes with a medium- to heavy-weight vibratory roller.

Floor slab subgrade preparation should be conducted in accordance with recommendations in Section 6.3.1.1, Site Preparation prior to placement of the granular base course.

If moisture-sensitive flooring will be placed on the slab, it may be appropriate to install a suitable vapor-retarding membrane, such as MoistStop beneath slab-on-grade floors. Membranes should be installed in accordance with manufacturer's recommendations.

6.3.4 Pavement

The fine-grained soils that mantle the site will provide fair pavement support properties. For design purposes, we have assumed a CBR value of 5 for the subgrade soils. A suitable pavement section resulting in adequate pavement performance is highly dependent on actual traffic loading, typically expressed as 18-kip Equivalent Single Axle Loads ESALs. Typical Light Trucks impart 0.25 to 0.50 ESAL's per truck; medium sized trucks and school buses impart 1.0 to 1.5 ESAL's per truck; heavy trucks impart 2.0 to 2.5 ESAL's per truck. It takes approximately 1,200 passenger cars to impart 1 ESAL.

Design traffic information has been estimated based on the anticipated usage for similar projects. Based on our understanding of the proposed traffic and the anticipated subgrade soil types and conditions, the pavement sections presented on the following table are recommended. Pavement subgrade should be prepared and proof rolled prior placement of base course and pavement as described in Section 6.3.1.1 Site Preparation. The following parameters were used in the pavement design:

Pavement Design Parameters

Design Life	20 years
Initial Serviceability	4.5
Terminal Serviceability	2.5
Reliability	95%
Std Deviation - Flexible	0.4
Std Deviation - Rigid	0.35
AC Structural Coefficient	0.4
Untreated Road Base	0.10
Granular Subbase	0.08
Design CBR	5

Flexible Pavement

Pavement Use	Design 18-kip ESALs	Layer Thickness (inches)	
		AC	Base Course
Auto and Light Truck Traffic	30,000	3	8

If the design team considers that the assumptions presented above are not accurate, AMEC should be informed so that we can review the pavement designs as necessary. Similarly, AMEC should be contacted if alternate designs are needed. The pavement materials and placement should be in accordance with the Utah Department of Transportation (UDOT) or American Public Works Association (APWA) specifications.

6.3.5 Final Grading

Final grading should be constructed and maintained to convey water away from foundation walls and backfill. Down spouts should discharge outside of the foundation backfill at least 10 feet away from the building. Irrigation above or near wall backfill should be minimized. We recommend that landscaped surfaces adjacent to buildings be sloped down away from the buildings at a minimum slope of 5 percent. Concrete flatwork or pavement adjacent to buildings should slope down away from the buildings at a slope of 2 percent or more.

6.4 Soil Corrosivity

A soil sample collected from the site was tested to determine pH and resistivity values. The measured pH value was 6.2 and the measured resistivity was 18,607 ohm-cm. The results are included in Appendix B. These values are indicative of a mildly corrosive environment.

6.5 Cement Types

A soluble sulfate concentration of 175 parts per million (ppm) was measured from a representative sample of on-site soil collected from the site. This result indicates that the site soils contain negligible amounts of water soluble sulfates and standard Type I-II cement may be used for concrete in contact with the on-site soils.

7. DESIGN REVIEW AND CONSTRUCTION SERVICES

We welcome the opportunity to review and discuss construction plans and specifications for this project as they are being developed. In addition, AMEC should be retained to review all geotechnical-related portions of the plans and specifications to evaluate whether they are in conformance with the recommendations provided in our report. Additionally, to observe compliance with the intent of our recommendations, design concepts, and the plans and specifications, we are of the opinion that all construction operations dealing with earthwork and foundations should be observed by a representative of AMEC. Our construction-phase services will allow for timely design changes if site conditions are encountered that are different from those described in this report. If we do not have the opportunity to confirm our interpretations, assumptions, and analyses during construction, we cannot be responsible for the application of our recommendations to subsurface conditions that are different from those described in this report.

8. LIMITATIONS

This report has been prepared to aid the architect and engineer in the design of this project. The scope is limited to the specific project and location described herein, and our description of the project represents our understanding of the significant aspects of the project relevant to the design and construction of the earthwork, foundations, and floor slabs. In the event that any changes in the design and location of the building as outlined in this report are planned, we should be given the opportunity to review the changes and to modify or reaffirm the conclusions and recommendations of this report in writing.

The conclusions and recommendations submitted in this report are based on the data obtained from the test pits made at the locations indicated on the Site Plan, Figure 2 and from other sources of information discussed in this report. In the performance of subsurface investigations, specific information is obtained at specific locations at specific times. However, it is acknowledged that variations in soil conditions may exist between explorations. This report does not reflect any variations that may occur between these explorations. The nature and extent of variation may not become evident until construction. If, during construction, subsurface conditions are different from those encountered in the explorations, we should be advised at once so that we can observe and review these conditions and reconsider our recommendations where necessary.

Our professional services have been performed, our findings obtained, and our recommendations prepared in accordance with generally accepted engineering principles and practices at this time along the Wasatch Front.

9. CLOSURE

We appreciate the opportunity to provide this service for you. If you have any questions or require additional information, please do not hesitate to contact us.

Respectfully submitted,
AMEC Environment & Infrastructure, Inc.

REGISTERED PROFESSIONAL ENGINEER
#349987-2202
DAVID K.
FADLING
10/21/14
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Senior Geotechnical Engineer/Geologist

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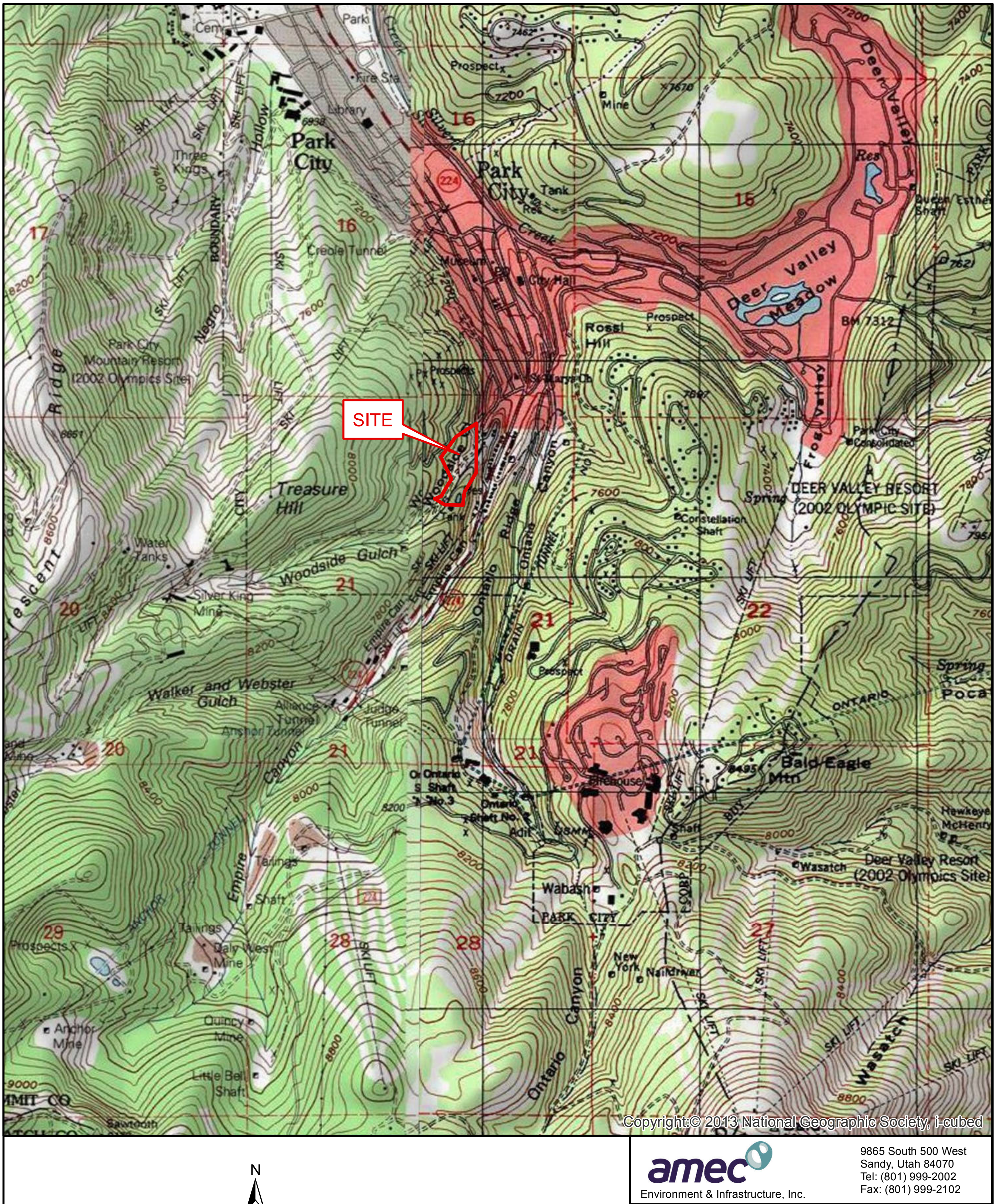

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N

0 0.25 0.5 1 Miles

Heber City, Brighton, Park City West, and Park City East Quadrangles
USGS 7.5 Minute Series (Topographic)

amec
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Figure 1
Vicinity Map

Alice Lode Site
King Development Group, LLC
Park City, Utah

Alice Lode\Drawings\
GIS\2014_Oct\Figure1.mxd
Job No. 5-814-000223

Legend

- Index Contour Interval of 10 feet
- New Ephemeral Stream Alignment
- Waterline
- ▭ Site Boundary
- ○ Tree
- Mineshaft

Property boundary and contour lines provided by DHM Design and Stantec, Inc. October 2014.
The map shown here has been created with all due and reasonable care and is strictly for use with AMEC Project Number: 5-814-000223. This map has not been certified by a licensed land surveyor, and any third party use of this map comes without warranties of any kind as AMEC assumes no liability, direct or indirect, whatsoever for any such third party or unintended use.

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Figure 2
Site Map - Test Pit Locations

Alice Lode Site
King Development Group, LLC
Park City, Utah

Alice Lode\Drawings\GIS\Request121508\Figure_2_Site_Map.mxd
Job No. 5814000223

Document Path: P:\Environmental\Projects\2005 Projects (0209-0248)\54-0223 Alice Lode\Drawings\GIS\2014_Oct\Figure2_site_map_10-2014.mxd

LOG OF TEST PIT NO. TP-1

Project Name: **Alice Claim Development**
Location: **Woodside Gulch**
Park City, Utah
Project No: **6-817-005165** Sheet 1 of 1

Date Excavated: **6/28/06**
Backhoe Type: **JCB 214S**
Excavated By: **Skyline**
Logged By: **R. Buxton**

Surface El.:

Elevation, feet	Depth, feet	Graphic Log	MATERIAL DESCRIPTION	Samples	Unit Dry Weight, pcf	Water Content, %	% Passing No. 200 Sieve	Liquid Limit	Plasticity Index	REMARKS
	0 – 2.0	▨	Clayey SAND with some gravel (SC); dark brown; dry; loose to medium dense; major roots; (topsoil) to 2"							
	2.0 – 6.0	⁘	QUARTZITE; yellowish-brown; moderately weathered, hydrothermally altered; hard (RH-4); close to very close joint spacing (WEBER QUARTZITE) Mine shaft and adit encountered							
	6.0		Bottom of Excavation @ 6.0' Groundwater Not Encountered							

Depth scale: 5, 10, 15, 20, 25

Remarks:

Water Level Observations		
∇		
▼		

The discussion in the report is necessary for a proper understanding of the nature of subsurface materials.

Figure 3A

LOG OF TEST PIT NO. TP-2

Project Name: **Alice Claim Development**
Location: **Woodside Gulch**
Park City, Utah
Project No: **6-817-005165** Sheet 1 of 1

Date Excavated: **6/28/06**
Backhoe Type: **JCB 214S**
Excavated By: **Skyline**
Logged By: **R. Buxton**

Elevation, feet	Depth, feet	Graphic Log	Surface El.: MATERIAL DESCRIPTION	Samples	Unit Dry Weight, pcf	Water Content, %	% Passing No. 200 Sieve	Liquid Limit	Plasticity Index	REMARKS
		▨	Clayey SAND with some gravel (SC); some roots; (topsoil)							
	1.5									
		○	QUARTZITE; brown; slightly to moderately weathered; hard (RH-4); close to very close joint spacing; some clay infilling (WEBER QUARTZITE)	SB-1		17				
	5									
				SB-2		12	6	NP	NP	
	10									
	12.0		Bottom of Excavation @ 12.0' Groundwater Not Encountered							
	15									
	20									
	25									

Remarks:

Water Level Observations		
▽		
▼		

The discussion in the report is necessary for a proper understanding of the nature of subsurface materials.

Figure 3B

LOG OF TEST PIT NO. TP-3

Project Name: **Alice Claim Development**
Location: **Woodside Gulch**
Park City, Utah
Project No: **6-817-005165** Sheet 1 of 1

Date Excavated: **6/28/06**
Backhoe Type: **JCB 214S**
Excavated By: **Skyline**
Logged By: **R. Buxton**

Surface El.:

Elevation, feet	Depth, feet	Graphic Log	MATERIAL DESCRIPTION	Samples	Unit Dry Weight, pcf	Water Content, %	% Passing No. 200 Sieve	Liquid Limit	Plasticity Index	REMARKS
			Clayey SAND with some gravel (SC); brown; loose to medium dense; dry; major roots from topsoil to 1"							
			2.0							
			Clayey GRAVEL with sand and cobbles (GC); brown; dry; very dense; (colluvium)	SB-1		4				PH=6.2 RES=18,607 ohm-cm
	5		5.0							
			Backhoe Refusal on Hard (RH-4) Quartzite at 5' Groundwater Not Encountered							
	10									
	15									
	20									
	25									

Remarks:

Water Level Observations		
∇		
▼		

The discussion in the report is necessary for a proper understanding of the nature of subsurface materials.

Figure 3C

LOG OF TEST PIT NO. TP-4

Project Name: **Alice Claim Development**
Location: **Woodside Gulch**
Park City, Utah
Project No: **6-817-005165** Sheet 1 of 1

Date Excavated: **6/28/06**
Backhoe Type: **JCB 214S**
Excavated By: **Skyline**
Logged By: **R. Buxton**

Surface El.:

Elevation, feet	Depth, feet	Graphic Log	MATERIAL DESCRIPTION	Samples	Unit Dry Weight, pcf	Water Content, %	% Passing No. 200 Sieve	Liquid Limit	Plasticity Index	REMARKS
		▨	Clayey SAND with some gravel (SC); dark brown; dry; medium dense; fine to coarse, subangular to subrounded gravel	SB-1		9				
			3.0							
		●	Sandy clayey GRAVEL (GC) with occassional cobbles; brown; angular to subangular, fine to coarse gravel; fine sand; damp; dense; (colluvium)	SB-2		8				
	5									
	10			SB-3		11				
			12.0							
			Backhoe Refusal on Hard (RH-4) Quartzite @ 12' Groundwater Not Encountered							
	15									
	20									
	25									

Remarks:

Water Level Observations		
∇		
▼		

The discussion in the report is necessary for a proper understanding of the nature of subsurface materials.

Figure 3D

LOG OF TEST PIT NO. TP-5

Project Name: **Alice Claim Development**
Location: **Woodside Gulch**
Park City, Utah
Project No: **6-817-005165** Sheet 1 of 1

Date Excavated: **6/28/06**
Backhoe Type: **JCB 214S**
Excavated By: **Skyline**
Logged By: **R. Buxton**

amec

Surface El.:

Elevation, feet	Depth, feet	Graphic Log	MATERIAL DESCRIPTION	Samples	Unit Dry Weight, pcf	Water Content, %	% Passing No. 200 Sieve	Liquid Limit	Plasticity Index	REMARKS
	0–2.0	▨	Clayey SAND with some gravel (SC); dark brown; fine to coarse, angular to subangular gravel; fine to coarse sand; major roots (topsoil) to 2"; dry; medium dense	SB-1		5	17			
	2.0–7.5	○		SB-2		3				
	7.5		Bottom of Excavation @ 7.5' Groundwater Not Encountered							

Depth scale: 5, 10, 15, 20, 25

Remarks:

Water Level Observations		
∇		
▼		

The discussion in the report is necessary for a proper understanding of the nature of subsurface materials.

Figure 3E

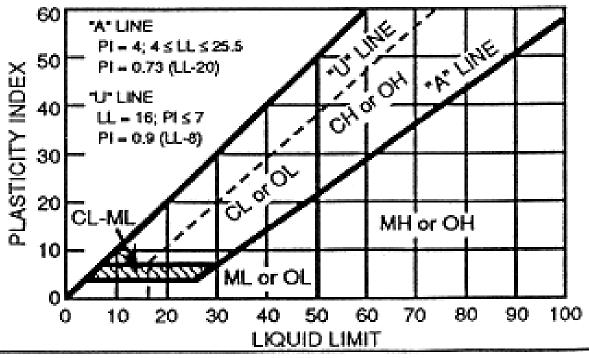
UNIFIED SOIL CLASSIFICATION SYSTEM

Soils are visually classified for engineering purposes by the Unified Soil Classification System. Grain-size analyses and Atterberg Limits tests often are performed on selected samples to aid in classification. The classification system is briefly outlined on this chart. Graphic symbols are used on boring logs presented in this report. For a more detailed description of the system, see "Standard Practice for Description and Identification of Soils (Visual-Manual Procedure)" ASTM Designation: 2488-84 and "Standard Test Method for Classification of Soils for Engineering Purposes" ASTM Designation: 2487-85.

MAJOR DIVISIONS				GRAPHIC SYMBOL	GROUP SYMBOL	TYPICAL NAMES
COARSE-GRAINED SOILS Less than 50% passes No. 200 sieve)	GRAVELS (50% or less of coarse fraction passes No. 4 sieve)	CLEAN GRAVELS (Less than 5% passes No. 200 sieve)			GW	Well graded gravels, gravel-sand mixtures, or sand-gravel-cobble mixtures
					GP	Poorly graded gravels, gravel-sand mixtures, or sand-gravel-cobble mixtures
		GRAVELS WITH FINES (More than 12% passes No. 200 sieve)	Limits plot below "A" line & hatched zone on plasticity chart		GM	Silty gravels, gravel-sand-silt mixtures
			Limits plot above "A" line & hatched zone on plasticity chart		GC	Clayey gravels, gravel-sand-clay mixtures
	SANDS (50% or more of coarse fraction passes No. 4 sieve)	CLEAN SANDS (Less than 5% passes No. 200 sieve)			SW	Well graded sands, gravelly sands
					SP	Poorly graded sands, gravelly sands
		SANDS WITH FINES (More than 12% passes No. 200 sieve)	Limits plot below "A" line & hatched zone on plasticity chart		SM	Silty sands, sand-silt mixtures
			Limits plot above "A" line & hatched zone on plasticity chart		SC	Clayey sands, sand-clay mixtures
FINE-GRAINED SOILS (50% or more passes No. 200 sieve)	SILTS Limits plot below "A" line & hatched zone on plasticity chart	SILTS OF LOW PLASTICITY (Liquid Limit less than 50)			ML	Inorganic silts, clayey silts of low to medium plasticity
		SILTS OF HIGH PLASTICITY (Liquid Limit 50 or more)			MH	Inorganic silts, micaceous or diatomaceous silty soils, elastic silts
	CLAYS Limits plot above "A" line & hatched zone on plasticity chart	CLAYS OF LOW PLASTICITY (Liquid Limit less than 50)			CL	Inorganic clays of low to medium plasticity, gravelly, sandy, and silty clays
		CLAYS OF HIGH PLASTICITY (Liquid Limit 50 or more)			CH	Inorganic clays of high plasticity, fat clays, sandy clays of high plasticity
	ORGANIC SILTS AND CLAYS	ORGANIC SILTS AND CLAYS OF LOW PLASTICITY (Liquid Limit less than 50)			OL	Organic silts and clays of low to medium plasticity, sandy organic silts and clays
		ORGANIC SILTS AND CLAYS OF HIGH PLASTICITY (Liquid Limit 50 or more)			OH	Organic silts and clays of high plasticity, sandy organic silts and clays
ORGANIC SOILS		PRIMARILY ORGANIC MATTER (dark in color and organic odor)			PT	Peat

NOTE: Coarse-grained soils with between 5% and 12% passing the No. 200 sieve and fine-grained soils with limits plotting in the hatched zone on the plasticity chart have dual classifications.

PLASTICITY CHART

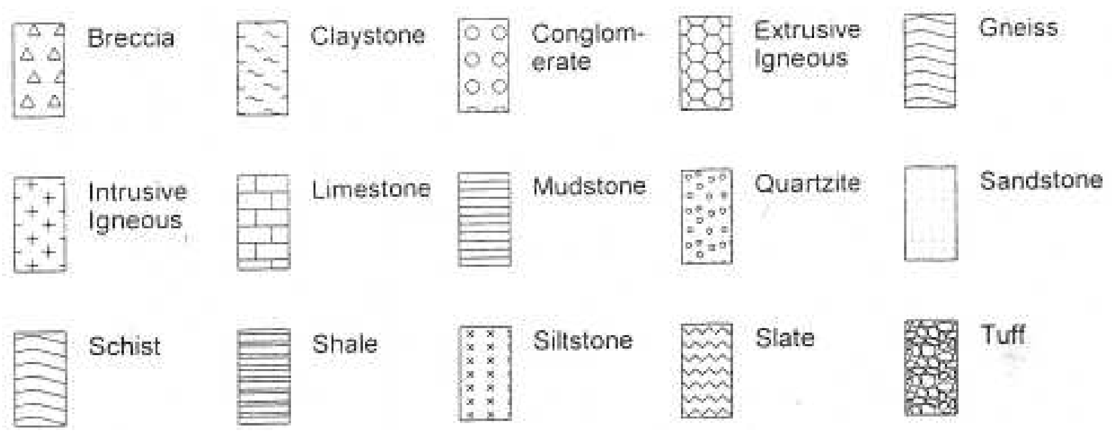


DEFINITION OF SOIL FRACTIONS

SOIL COMPONENT	PARTICLE SIZE RANGE
Boulders	Above 12 in.
Cobbles	12 in. to 3 in.
Gravel	3 in. to No. 4 sieve
Coarse gravel	3 in. to 3/4 in.
Fine gravel	3/4 in. to No. 4 sieve
Sand	No. 4 to No. 200 sieve
Coarse sand	No. 4 to No. 10 sieve
Medium sand	No. 10 to No. 40 sieve
Fine sand	No. 40 to No. 200 sieve
Fines (silt and clay)	Less than No. 200 sieve

Figure 4

ROCK TYPE AND DESCRIPTION KEY



RELATION OF RQD & ROCK QUALITY (DEERE 1968)

RQD, Rock Quality Designation %	Description of Rock Quality
0-25	Very Poor
25-50	Poor
50-75	Fair
75-90	Good
90-100	Excellent

DESCRIPTIVE TERMINOLOGY FOR JOINT SPACING

Spacing of Joints	Descriptive Terms
< 2 in	Very Close
2 in – 1 ft	Close
1 ft – 3 ft	Moderately Close
3 ft – 10 ft	Wide
> 10 ft	Very Wide

Description of Relative Hardness / Strength

RH 0	Extremely Soft	Can be indented with difficulty by thumbnail. May be moldable or friable with finger pressure.
RH 1	Very Soft	Crumbles under firm blows with point of a geology pick. Can be peeled by a pocket knife. Scratched with finger nail.
RH 2	Soft	Can be peeled by a pocket knife with difficulty. Cannot be scratched with fingernail. Shallow indentation made by firm blow of geology pick.
RH 3	Medium Hard	Can be scratched by knife or pick. Specimen can be fractured with a single firm blow of hammer/geology pick.
RH 4	Hard	Can be scratched with knife or pick only with difficulty. Several hard hammer blows required to fracture specimen.
RH 5	Very Hard	Cannot be scratched by knife or sharp pick. Specimen requires many blows of hammer to fracture or chip. Hammer rebounds after impact.

Term Used to Describe the Degree of Weathering

Fresh	Crystals are bright. Discontinuities may show some minor surface staining. No discoloration in rock fabric.
Slightly	Rock mass is generally fresh. Discontinuities are stained and may contain clay. Some discoloration in rock fabric. Decomposition extends up to 1 inch into rock.
Moderately	Rock mass is decomposed 50% or less. Significant portions of rock show discoloration and weathering effects. Crystals are dull and show visible chemical alteration. Discontinuities are stained and may contain secondary mineral deposits.
Predominately	Rock mass is more than 50% decomposed. Rock can be excavated with geologists' pick. All discontinuities exhibit secondary mineralization. Complete discoloration of rock fabric. Surface of core is friable and usually pitted due to washing out of highly altered minerals by drilling water.
Decomposed	Rock mass is completely decomposed. Original rock "fabric" may be evident. May be reduced to soil with hand pressure.

Figure 5

APPENDIX A

FIELD EXPLORATIONS

APPENDIX A

FIELD EXPLORATION

FIELD EXPLORATION

General

Subsurface materials and conditions at the project site were investigated on June 28, 2006 with 5 test pits designated TP-1 through TP-5. The approximate locations of the test pits are shown on Figure 2, Site Plan. All field operations were observed by a staff engineer provided by our firm, who maintained a detailed log of the materials and conditions encountered in each boring and directed the sampling operations.

Test Pits

The test pits were excavated with a Volvo JCB 214S excavator provided and operated by Skyline of Salt Lake City, Utah. The test pits were excavated to depths of 5 to 12 feet below the ground surface. Disturbed samples were obtained from the test pits at appropriate intervals. The soil samples obtained were carefully examined in the field, and representative portions were saved in plastic bags and transported to our laboratory for further examination and physical testing.

The field program was supervised by a member of our geotechnical staff who maintained a continuous log of the subsurface conditions encountered. The soils were classified by visual and textural examination in the field. These classifications were later reviewed by subsequent re-examination of the soil samples in our laboratory. Graphical representations of the subsurface conditions encountered are presented on Figures 3A through 3E, Log of Test Pits. Terms used to describe the soil and rock are presented on Figure 4, Unified Soil Classification System and Figure 5, Rock Type and Description Key. The stratification boundaries indicated on the logs are approximate. Actual transitions between differing materials may be gradual.

APPENDIX B

LABORATORY TESTING

APPENDIX B

LABORATORY TESTING

LABORATORY TESTING

General

All samples obtained from the field were transported to our laboratory for examination and testing. The physical characteristics were noted, and the field classifications were modified where necessary. The laboratory testing program was conducted to provide data for our engineering analyses. The laboratory program included determinations of natural moisture content, grain size distribution, partial sieve analysis, Atterberg limits tests and corrosion tests. The following sections describe the testing program in more detail.

Natural Moisture Content

Natural moisture content determinations were made in conformance with ASTM D 2216. The results are presented on Figures 3A through 3E, Log of Test Pits.

Grain Size Distribution

A determination of grain size distribution was conducted on a selected sample of the on-site soil in general conformance with ASTM 422. The result of the test is summarized in the following table.

SUMMARY OF
GRAIN SIZE ANALYSIS DETERMINATIONS

	Percent Passing By Dry Weight													
Test Pit	**Depth (feet)**	**3”**	**2”**	**1-1/2”**	**1"**	**3/4"**	**1/2"**	**3/8"**	**No. 4**	**No. 10**	**No. 20**	**No. 40**	**No. 100**	**No. 200**	**Unified Soil Classification**
TP-2	6.0	73	67	58	47	41	35	32	22	17	11	9	7	6	GP-GM

Percent Passing the No. 200 Sieve (Washed Sieve Analysis)

The silt and clay content (percent passing the No. 200 sieve) was evaluated for selected soil samples in general conformance with ASTM D 1140. Oven-dried samples were weighed and placed on the No. 200 sieve. The silt and clay were washed through the sieve, and the sample remaining on the sieve was oven-dried and weighed. The change in sample weight is used to calculate the percent of material passing than the No. 200 sieve. The test results are summarized below.

SUMMARY OF
GRAIN SIZE ANALYSIS DETERMINATIONS

TP	Depth, ft	Percent Passing No. 200 Sieve	Classification
TP-5	0.0	17	Clayey Sand (SC)

Atterberg Limits

Atterberg Limit tests were performed in accordance with ASTM D 4318 on a representative sample of the native soil encountered at the site to verify field classifications. The test results are tabulated below:

Test Pit No.	Sample Depth (ft)	Unifed Soil Classification System Group Symbol	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)
TP-2	6.0	GP-GM	NP	NP	NP

Analytical Tests

Analytical tests were conducted on a representative sample collected from the site. The pH test was conducted by AMEC in our laboratory. The water soluble sulfate test was performed by TEI Testing Services, Inc. of Salt Lake City, Utah. The results are summarized in the following table.

Test Pit No.	Sample Depth (ft)	Unifed Soil Classification System Group Symbol	pH	Resistivity (ohm-cm)	Water Soluble Sulfate Concentration (ppm)
TP-3	2.0-4.5	GC	6.2	18,607	175