APPENDIX A5

PRELIMINARY GEOTECHNICAL ENGINEERING REPORT

FOR THE

State Highway 9 Iron Springs Alignment Environmental Assessment

Prepared for

COLORADO DEPARTMENT OF TRANSPORTATION

FEDERAL HIGHWAY ADMINISTRATION

Report Prepared by

YEH AND ASSOCIATES, INC.

December 2010

APRIL 2014

ADDENDUM/CLARIFICATION TO THE PRELIMINARY GEOTECHNICAL ENGINEERING REPORT STATE HIGHWAY 9 DILLON RESERVOIR ALIGNMENT EVALUATION

The Preliminary Geotechnical Engineering Report was prepared in 2010 before the refinement of the conceptual design for the Proposed Action. As a result, some of the terminology used to describe the project alternative has changed since the preparation of the Preliminary Geotechnical Engineering Report. In addition some details of the conceptual design have changed since the Preliminary Geotechnical Engineering Report was prepared.

The Preliminary Geotechnical Engineering Report provides appropriate background information for this Environmental Assessment (EA) analysis, and thus is included as a supporting technical appendix to the EA. The following changes in terminology and detail since preparation of the report should be noted:

- ***Names of Alternatives:*** The Preliminary Geotechnical Engineering Report provides information on two alignment alternatives. The alternative referred to as the "Lake Dillon Alternative" in Table 1 (page 24) is equivalent to the No Action Alternative (widening on existing alignment) as identified the EA and other supporting technical appendices. The alternative referred to as the "Iron Springs Alignment" in Table 1 is the Proposed Action as identified in the EA and other supporting technical appendices.

- ***Alternative Details:*** Section 10.0 Bridge Foundation Recommendations (page 18) is no longer applicable as the conceptual designs for the Proposed Action and the No Action Alternative (see Appendix A1) do not include any bridges. Table 1 (page 24) indicates "Retaining walls not required" for the Iron Springs Alignment (Proposed Action). This is no longer accurate, as the Proposed Action includes some retaining walls as shown in Appendix A1 and described in the EA and other supporting technical appendices. The retaining walls required in the Proposed Action would be less extensive than those required in the No Action Alternative.

Yeh and Associates, Inc.
Consulting Engineers & Scientists

Preliminary Geotechnical Engineering Report
SH 9 Dillon Reservoir Alignment Evaluation
Summit County, Colorado

December 10, 2010

Project No. 210-147

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1.0 Purpose and Scope of Study

This report presents the results of our preliminary geotechnical study for the evaluation of two alignment alternatives associated with the roadway widening of State Highway 9, west of the Blue River drainage of Dillon Reservoir in Summit County, Colorado with specific emphasis on the eastern side of the alignments that look westward over the Blue River arm of Lake Dillon (Sta. 0+00 through approximately Sta. 35+00 of the first alternative alignment that follows existing State Highway 9) and the fen crossing (natural resources). The study area is located between Swan Mountain Road and Frisco, which corresponds approximately to Mile Markers 93 through 97 (see Appendix A. Project Location Map).

The purpose of our study was to identify and evaluate general geotechnical characteristics of the subsurface soils and bedrock along the highway, to conduct preliminary geotechnical analyses, to provide preliminary geotechnical recommendations and parameters for the proposed cut and fill slopes and structures and provide a general constructability assessment of the two proposed alignment alternatives. The study was conducted in general accordance with our scope of work dated March 17, 2010

A field exploration program consisting of geologic reconnaissance, bedrock structure mapping, geophysical survey and exploratory drilling was conducted to obtain information on subsurface conditions. Between September 28, 2010 and October 6, 2010, 6 geotechnical borings were observed and logged by CDOT representatives and 3 biological borings were observed and logged by a representative with PKM Design Group, Inc. Of these 9 borings, 5 were drilled along SH 9 and 4 were drilled west of SH 9 on the slopes for the alternative new alignment (see Appendix B. Site Investigation Map). Subsurface material samples were obtained during the field exploration and examined by the project personnel. Representative samples were tested in our laboratory to determine the classification and engineering characteristics of the on-site material.

Based on the information obtained, Yeh and Associates has completed an evaluation of subsurface conditions. This report summarizes the data gathered, the

results of our analysis and our recommendations based on the proposed construction, site reconnaissance, geotechnical subsurface investigation, and results of laboratory testing. General cut and fill slopes and structures recommendations and a discussion of the geotechnical engineering considerations are included in this report.

2.0 Proposed Construction

Two alternatives are proposed for the SH 9 alignment through this area. The first alignment will generally follow the existing SH 9 route associated with proposed widening by expanding the current two-lane configuration into a four-lane roadway. The second alignment would follow a more direct route from Farmers corner to Frisco with a four -lane configuration. Both alternatives will require both cut and fill sections on either the existing SH 9 alignment or the open space on the west slope. The construction of a bridge is also proposed over the fen area approximately between Sta. 8+00 and Sta. 9+00.

3.0 Site Conditions

The existing highway slopes from south to north through the study area with an approximate elevation gain of approximately 10 feet between Sta. 0+00 and Sta. 40+00. The existing SH 9 on the northeast side of the project area generally follows the rim of Dillon Reservoir and turns southwest at the north end of the project area. Most of the existing SH 9 in the east side of the project area is cut with cut slopes ranging from 1.3H:1V to 0.4H:1V. The southern section of the existing highway (Sta. 0+00 to 40+00) consists of cut slopes in quartzite of the order of 20 to 30 feet high and as steep as 1.3H:1V in areas. These cuts are generally stable, with some surficial erosion (raveling), except where the middle shale member of the formation is exposed in the slope face (for example approximately Sta. 5+00, first alignment alternative). A slope failure in the cut slope of 1.3H:1V occurs approximately this point. The failed slope comprises sandstone, carbonaceous shale with a little clay. Several minor to severe cracks were observed on the pavement.

The project area consists of undeveloped, public land under the jurisdiction of the National Forest System and Summit County.

The second alignment begins at the Iron Springs Road turnoff from SH 9 and generally follows the existing bike path westward. The existing bike path generally follows the route of SH 9 to Sta. 15+00 of the first alignment alternative and turns west for approximately 2000 feet into the forest and then southwest for approximately another 2000 feet to the limit of the project area. The bike path has changes in elevation over this distance climbing initially before descending to the west.

4.0 Regional Geology

The geologic history of the Frisco quadrangle (US Geological Survey) spans more than 1.7 billion years. The oldest rocks underlie the crest of the Tenmile Range and include biotite-sillimanite schist and gneiss, amphibolite, and quartzite.

The oldest exposed sedimentary rocks in the quadrangle are the brick-red locally conglomeratic sandstones of the Pennsylvanian and Permian Maroon Formation.

The thickest sequence of sedimentary rocks in the Frisco quadrangle is Cretaceous in age and includes almost 3,000 feet of chiefly black to gray-brown shale and brown sandstone of the Upper Cretaceous Pierre Shale. All the Upper Cretaceous formation in the quadrangle (Benton Shale, Niobrara Formation, and Pierre Shale) were deposited in an extensive seaway that covered the entire mid-continent region of North America.

The Laramide Orogeny, between about 70-50 Ma, was a time of major uplift, compressive faulting, and mountain building in the southern Rocky Mountains. Igneous intrusions associated with this mountain building episode crop out in the Frisco quadrangle and a large body of quartz monzonite porphyry, dated at 44 Ma underlies Swan Mountain. Much of the mineralization that is associated with mining in Colorado occurred occur in concert with these intrusive bodies.

Although the Blue River valley in the Frisco quadrangle is not glacially carved (glaciers in the Blue River valley reached the outskirts of Breckenridge, a couple of km north of the quadrangle boundary), large glaciers of at least two major glacial periods (Pinedale, Bull Lake, and possibly pre-Bull Lake) flowed out of valleys in the Gore and Tenmile Ranges to the west.

Colluvium, a mixture of rock fragments and smaller debris from small landslides and soil-creep and sheet-wash deposits, mantles many of the meadows and aspen-covered hillsides in the quadrangle. Alluvium, composed of well-rounded, clast-supported gravels that contain clasts as long as about 6 feet, forms terraces (deposited most prominently during past glacial periods) and the floodplain of the Blue River.

Landslide deposits are extensive in the Frisco quadrangle. These landslides are mostly earth flows and earth slides as well as rock slides and debris slides. Older (pre-Holocene) landslide deposits are extensive while Younger (undifferentiated late Pleistocene and Holocene) deposits commonly form in a number of poorly consolidated units, particularly glacial till of Pinedale age and Bull Lake age (units Qtp and Qtb),

Sandstone slopes are resistant to erosion and form the steep cut slopes visible along the highway. Bedrock encountered during our subsurface investigation was quartzite. Refer to the Geologic Map shown in Figure 1 for the distribution of the various units. More information related to the identification of the various units and structure is provided in Appendix C. Geology Map and Legend.

4.1 Surficial Deposits

The surficial material at the site consists of alluvium and artificial fill. Terrace deposits associated with past fluvial activity were also encountered throughout the project site.

Surficial deposits of varying thickness overlie the bedrock. Generally the thickness of the surficial deposits ranged from 8 to 35 feet, however, in places the thickness in the project is expected to be higher. The various deposits are described in more detail in the following sections.

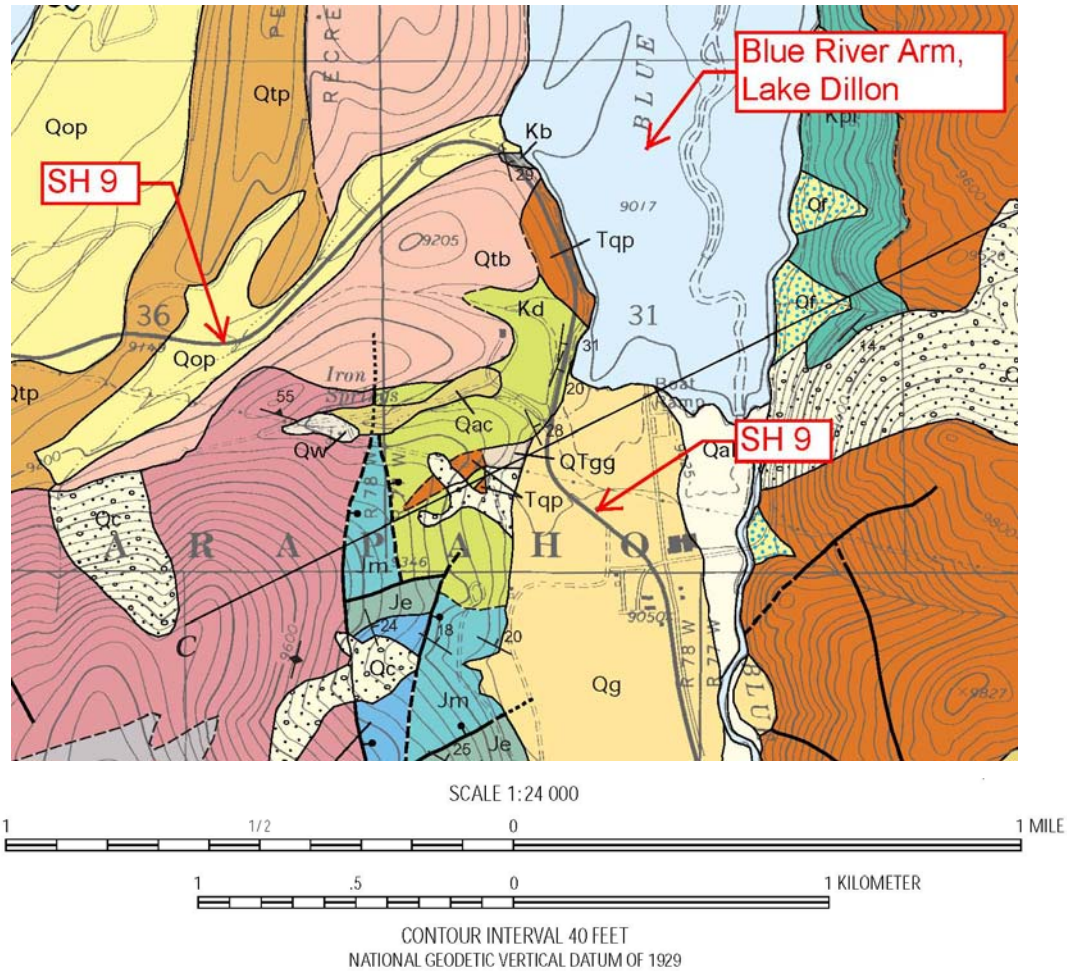


Figure 1-Geology of site and immediate surrounding area

(Kellogg, K.S., Bartos P.J. and Williams, C.L., 2002, Geologic Map of the Frisco Quadrangle, Summit County, Colorado. USGS Miscellaneous Field Studies Map MF-2340.)

Artificial fill

Roadway fill material may be encountered during construction. Natural on-site materials may have been reworked during the original construction of SH 9.

Bouldery gravel of the Gold Run (middle - lower Pleistocene to Pliocene)

The bouldery gravels of the Gold Run (QTgg) are poorly graded, poorly stratified poorly consolidated, subrounded to well-rounded, light tan to grayish-orange bouldery deposits. Deposits are highly weathered and contain clasts mainly of gneiss, Dakota Sandstone and Quartz Monzonite porphyry. Boulders in the deposits are generally less than 7 feet in diameter. These deposits are known for their gold content and were

mined extensively through placer dredging and hydraulic mining. Total thickness locally may more than 150 feet in some instances

Till of Bull Lake glaciation (middle Pleistocene)

Till of Bull Lake glaciations (Qtb) are poorly graded, unstratified, dissected and rounded bouldery till in moraines. Subrounded to subangular clasts are composed of gneiss and granitic rocks. Boulders of gneiss and plutonic in the deposits are slightly to moderately weathered. Bull Lake Till commonly contains 10 percent clasts of Dakota Sandstone. Thickness of the formation may exceed 300 feet in some places.

4.2 Bedrock

Bedrock formations, mostly sedimentary in nature, were encountered during our field investigation. These formations are described in more detail in the following sections.

Dakota Sandstone (Lower Cretaceous)

The Dakota Sandstone (Kd) exists as three layers: an upper quartzite member, a shale member in the middle, and a lower quartzite member. The upper and lower quartzite members generally contain interbedded carbonaceous shale. The upper quartzite member can be 20-65 foot thick and contains light-gray, commonly cross-bedded quartzite in beds 4-12 inch thick, with thin, black, commonly carbonaceous shale interbeds. In places, where it is present, in general Frisco area, the base of the upper member is a massive, 6-30 foot thick, resistant quartzite bed. Joint surfaces contain red, orange, and yellow limonite encrustations

The middle shale member consists of interbedded dark-gray to black, commonly carbonaceous shale and generally thin-to-medium-bedded, medium-grained, equigranular, gray to light-gray quartzite; quartzite beds can be as thick as 6 feet. The thickness of the middle shale member is highly variable but locally at the site where drill core information is available the thickness was about 5 feet.

Quartz Monzonite Porphyry (Eocene)

The quartz monzonite porphyry (Tqp) is a light-gray, conspicuously porphyritic, massive intrusive granite. It weathers to light brown with a flaggy to blocky fracture. The igneous rock is the primary component of Swan Mountain and its finer-grained phase appears in the mapped road cuts on the North-East side of the project near the edge of Lake Dillon (see Figures 3 and 4).

5.0 Seismicity

5.1 Seismic Concern

Large historic earthquakes of magnitude up to 6.5 and some smaller magnitude were recorded in the Front Range region. However, no current active faults are known to exist in the immediate project area. It is believed that the likelihood of a highly damaging earthquake occurring in the near future is low.

5.2 Seismic Design Parameters

Based on the results of the subsurface investigation, the site is classified as Class C. The horizontal peak particle acceleration in the project area was obtained using the Seismic Design Parameters software program, Version 2.10, developed by Leyendecker, Frankel, and Rukstales (2008). This program utilizes the 2002 United States Geological Survey (USGS) National Seismic Hazard Maps (Frankel, et. al., 2002) for horizontal peak ground acceleration (PGA) with a 7 percent probability of exceeding in 75 years (approximate 1034-year return period). For the subject site, the PGA is reported to be 7.5 percent of gravity (0.075g). This PGA value should be multiplied by the appropriate site factors as listed in AASHTO Table 3.10.3.2-1 to arrive at the factored PGA. The seismic design spectra are provided in Appendix D.

6.0 Bedrock Outcrop Mapping

6.1 Cut Slope Bedrock Mapping and Kinematic Analysis

Selected bedrock outcrops of the existing cutslopes within the project limits were mapped and photographed. Rock discontinuities, such as joints and fractures, were

noted and their orientations measured for stereonet kinematic analysis, which is a technique for projecting multiple three-dimensional planes of varying orientations onto a two-dimensional surface, in order to visualize how they interact. In this way, the most likely rock block failure modes can be determined (i.e. planar, wedge, or toppling). Discontinuity orientations were measured using a standard pocket transit (Brunton).

Discontinuity orientations were plotted for short segments (windows) of the roadway using stereonet analysis software (Dips, Version 5.108, developed by Rocscience, Inc.). Kinematic analyses were performed for each segment. Stereonet plots and photographs of each segment can be found in Appendix E. Existing Rockcut Evaluation

Geologic scan line surveys were performed in 4 locations for rock slope mapping between Station 5+00 and Station 25+00 for the first realignment alternative to evaluate the existing cut slopes and determine the overall suitability of rock-excavation into the slopes. See the Site Investigation Map in Appendix B for geologic scan line locations. Note that there are no rock outcrops available for performing such surveys on the second realignment alternative in the area of interest. Various parameters were evaluated to determine an overall suitability for excavating the rock, including existing condition of the cutslopes, predominant failure modes, constructability issues, and maintenance concerns. Dip angles and dip directions of the major discontinuities were recorded, and likely failure modes were determined.

In general, the existing bedrock outcrops range from approximately 1.3H:1V to 0.4H:1V slopes. Rock cuts in the study area have either shown erosion or have had rocks fall out of the rock cuts, onto the bike path, the shoulder and the roadway, over time.

The existing rock outcrops exhibited planar and wedge failure mechanisms that contribute to localized unstable slopes and rockfall. Since stability is directional it should be noted that a cut section that is currently stable may become unstable when new cutslopes with a different orientation are constructed, or vice versa.

The proposed rock cut slopes were divided into 3 sections based on the feature and our field surveys. Rock slope stability and rockfall issues related to each section are summarized.

Proposed Rock Cut Section 1 – Approximately Sta. 1+00 to 10+00

This section is on the west side of SH 9. Geologic scan line 1 (SL 1) was performed in this section approximately at the station 5+00. The existing and proposed rock cuts height in this section is not provided at the time of this report. Based on the field mapping, the existing cutslope (which has failed along a carbonaceous shale - quartzite bedding plane back to a stable configuration) currently has a slope of approximately 37º measuring from horizontal (approximately 1.3H:1V). Overall this section has little vegetated cover on the slope. Several unfavorable planar failures along the weak bedding plane are observed in this section which dips at approximately 22º from the horizontal. This planar failure condition is exacerbated by the existence of a near vertical joint set that intersects this planar set at 90 degrees to the dip direction of the planar set. The stereonet kinematic analyses in this section indicate that the slope is potentially moving downward to the east which corresponds to our field observation. Cuts through this section steeper than about 22 º (~2.5H:1V) will likely be unstable if this weak shale bedding layer daylights directly into the cuts. The analyses also demonstrate that minor toppling or overturning failures may occur in this section although volume is likely to be small because of the closely spaced bedding discontinuities. This section may require additional stabilization measures which will need to be considered during design and evaluated during construction. Figure 2 depicts the unfavorable planar failure. Potential rockfall generated from outside of the ROW is possible in this section; it appears that the velocities and energies would be relatively low, but this has not been fully evaluated at this time.

Figure 2 - Planar Failure at Approximately Sta. 5+00 (existing SH 9 alignment)

Proposed Rock Cut Section 2 – Approximately Sta. 13+50 to 16+00

This section is on the east side of SH 9 adjacent to Dillon Reservoir and is composed of a somewhat finer grained variety of the coarser quartz monzonite porphyry bedrock prevalent at Swan Mountain. Geologic scan line 2 (SL 2) was performed in this section approximately at the station 15+00. In this section existing rock cuts range up to 24 feet in height with existing slope angles of approximately 70º (See Figure 3). Proposed vertical cut heights and slope angle in this section will be approximately 24 feet and 26º respectively. Stereonet kinematic analyses indicate that minor small scale wedge failures may occur in this section but toppling failure is not an issue. The current slope face geometry more or less mimics a potential planar failure discontinuity set and is therefore generally stable. Careful evaluation of this section will be required during design and recommendations re-evaluated during construction.

Figure 3. Proposed Rock Cut Area - Section 2 (approximately Sta. 13+50 to 16+00)

Potential rockfall generated from outside of the ROW is possible in this section; it appears, however, that the velocities and energies would be relatively low, but this has not been fully evaluated at this time.

Proposed Rock Cut Section 3 – Approximately Sta. 17+00 to 25+00

This section is on the west side of SH 9. Two scan line locations were identified in this section.

Scan line 3 (SL 3) was performed approximately at station 18+00. The existing cutslope has an angle of approximately 70º measured from the horizontal. This section is partially covered by vegetation. Using an assumed sliding friction angle of 30º several unfavorable planar failures dipping at about 34º are observed in this section.

Figure 4. Proposed Rock Cut Area - Section 3 (approximately Sta. 17+00 to 25+00)

The stereonet kinematic analyses of SL 3 indicate that in addition to planar failures and wedge failures (both assume a sliding friction angle of 30 degrees), potentially significant toppling failures may also occur in this section. Due to the closely jointed nature of the rock mass these failures will likely be of low volume but will require maintenance. A catch ditch will likely be required in this section. Attention to all of these failure modes will be required during design and construction in this vicinity.

Geologic scan line 4 (SL 4) was situated approximately 400 feet southwest of SL 3 at approximately station 22+00. It was obscured by scree and soil, limiting the visibility of the orientations of joints and other discontinuities. However, from overall field observation, SL 4 displays very similar joint characteristics to those measured at SL 3. A catch ditch will likely be required in this section. Attention to all of these failure modes will be required during design and construction in this vicinity.

Potential rockfall generated from outside of the ROW is possible in this section; it appears that the velocities and energies would be relatively low, but this has not been fully evaluated at this time. Proposed rock cut heights in this section were not provided at the time of this report.

7.0 Subsurface Investigation

7.1 Geotechnical Borings

The general subsurface conditions along the proposed roadway were explored between September 28, 2010 and October 6, 2006 utilizing a Colorado Department of Transportation (CDOT) truck mounted CME 75 drill rig and a CDOT rubber tire CME 55 drill rig equipped with HQ size casing and Hollow Stem Auger. See the Site Investigation Map in Appendix B for boring locations. Engineering Geology Sheets and Boring logs can be found in Appendix F.

A total of six borings were advanced vertically using HQ size casing while three borings were drilled vertically in the Fen area using a Hollow Stem Auger. Core samples were collected, logged, and photographed in the core sample boxes (see Appendix E, Photographs of Rock Core). Soils samples were collected typically at five-foot intervals for each boring utilizing a Standard Split Spoon Sampler or a Modified California Sampler driven by a 140-lb safety hammer with a 30-in stroke, using a procedure similar to the Standard Penetration Test (ASTM D1586). The number of blows required driving the Standard Split Spoon Sampler 12 inches after an initial penetration of 6 inches, or driving the modified California Sampler 12 inches without an initial penetration of 6 inches, and constitute the N-Value and blow counts as shown on the Boring Logs. For cohesionless soils, N-Values and blow counts can be correlated to the relative density of the material. N-Values from 0 to 4 or blow counts from 0 to 5 correspond to very loose materials, N-Values from 4 to 10 or blow counts from 5 to 12 correspond to loose materials, N-Values from 10 to 30 or blow counts from 12 to 35 correspond to medium dense materials, N-Values from 30 to 50 or blow counts from 35 to 60 correspond to dense materials, and N-Values greater than 50 and blow counts greater than 50 corresponds to very dense materials.

Borings YA-1, YA-2 and YA-3 were advanced along SH 9 to explore the subsurface conditions for the existing SH 9 widening (alternative alignment 1). In general, the subsurface conditions consisted of 10 to 16 feet of fill described as loose to medium dense gravels with sand, clay and silt in various mixtures and occasional cobbles and boulders. In Boring YA-1, organic materials were encountered at depth of 20 to 24 feet overlying glacial till which extended to the end of exploration of 35.3 feet. In Boring YA-2, bedrock was encountered at depth of 12 feet determined as very hard quartzite. A 5 foot layer of carbonaceous shale was encountered interbedded at 18 feet in the quartzite formation. In Boring YA-3, glacial till underlain by fill materials was encountered at depth of 10 feet and extended to the bottom of boring at 35 feet.

Borings YA-4, YA-5 plus an additional field boring located at the time of investigation were advanced on either side of SH 9 to explore the biological condition of the fen. These three biological borings were logged and observed by Dr. Chuck Schrader of PKM Design Group, Inc.

Borings YA-6, YA-7 and YA-8 were advanced on the west slope to explore the subsurface condition for the second alignment alternative. Boring YA-6 and YA-7 consisted of respectively 15 feet and 8 feet of very dense sand and gravels overlying quartzite bedrock to the depth of 30 feet and 27 feet, respectively. The sand and gravels are described as glacial till in our boring logs and are known as Till of the Bull Lake glaciations based on USGS findings. Boring YA-8 was drilled at the site of an artificial berm. The subsurface condition consists of up to 20 feet of fill described as medium dense clayey sand and gravel overlying glacial till which extended to the end of boring of 34.7 feet. All the borings except for biological borings are advanced to the depth of 27 feet to approximately 35 feet. Borings are backfilled and resurfaced with the drill cuttings.

Groundwater was not encountered in any of these borings at the time of subsurface exploration. Groundwater conditions in the area will likely vary considerably throughout the year. Variations can occur during different seasons, following precipitation events, after construction and site grading, and due to changes in surface and subsurface drainage characteristics of the surrounding area.

7.2 Geophysical Survey

Geophysical surveys were conducted by Yeh and Associates on September 28, 2010 using a seismic tomograph to model the subsurface and bedrock conditions. The seismic survey was performed to evaluate below ground features and to indicate areas for potential subsurface exploration within the proposed project site.

Seismic tomography is a methodology for estimating soil and bedrock properties. Seismic tomography is branch of seismic imaging using shock wave velocities to estimate density of subsurface materials. Seismic tomography operates on the principal that vibrations travel faster through hard materials and slower through soft materials. The process consisted of placing 24 geophones on the ground in a known array and striking a sledgehammer on the metal plate next to the geophones to generate a shockwave. The array consisted of two parallel geophones spaced 5 feet apart, with geophone-to-geophone spacing of 5 feet for a total spread length of 55 feet Typically 5 to 10 shot records were stacked to provide the final waveform data that was used to pick first arrivals. Computer modeling of the data allowed real-time results in the field.

Three geophysical surveys were located in the field approximately along the proposed cut section on the roadway realignments by pacing from existing site features. The geophysical survey 1 and 3 were placed on the existing asphalt-paved bike path and the geophysical survey 2 was placed on the native soils on the west slope in the vicinity of Boring YA-7. Geophysical survey location is presented on the Site Investigation Map in Appendix B and 3 tomographs are presented for each array in Appendix H.

Geophysical survey 1 shows a distinct layer approximately at depth of 9 feet. The P-wave velocities in the upper layer generally range from 1000 to 5000 feet per second with several scattered spot with velocity over 6000 feet per second. The P-wave velocity in the lower layer is generally greater than 5250 feet per second. The upper layer is likely gravelly sand with cobbles and boulders, and the lower layer is likely bedrock. This result is consistent with the nearby Boring YA-2.

The velocities from survey 2 generally range from 750-2250 feet per second. This area has some tree growth but appears that some clearing and possibly soil disturbance may have occurred in the past. No distinct layering appears to exist within the depth of signal penetration of the energy source (sledge hammer) potentially because the surface soil is relatively soft which tends to dissipate energy (survey 2 was placed near Boring YA-7 where the ground surface was covered with vegetation and pine needles). Generally upper layer P wave velocities below 1550 feet per second indicate soft, unconsolidated materials such as organic layers and loose disturbed soils (e.g. some tills display low velocities). The increase in velocity noted with depth is gradual and locally is related to the presence of large cobbles and boulders. The upper layer is likely gravelly sand with cobbles and boulders (fill and natural soils) and the lower layer is likely a similar material with an increase in the number of cobbles and boulders. This material is identified as till in the boring logs. The data suggest that bedrock was not encountered within the depth of resolution achievable by the energy source. The effective depth of the seismic investigation at this location is estimated to be 8 feet.

For the geophysical survey 3, it is likely that a distinct layer (seen as green in the tomograph) exists approximately at a depth of 9 feet. The upper layer is likely colluvium and possibly some till comprising sandy gravel with clusters of cobbles and boulders. The P-wave velocity for the lower layer (approximately 6000 feet per second) is likely bedrock.

8.0 Laboratory Testing

A total of 11 soil samples were tested in Yeh and Associates Inc's laboratory to determine the classification and engineering characteristics of the on-site soil and rock. Laboratory tests performed included sieve analysis, Atterberg limits, natural moisture content, water soluble sulfates, and pH (See Appendix I, Laboratory Test Results).

9.0 Geologic Hazards and Engineering Constraints

Based on our review of the project site it appears that concerns for design and construction of the proposed widening in this area include loose subsurface materials

and potential slope failures in the cut areas and fill areas along the existing SH 9 alignment.

9.1 Loose Subsurface Materials

The existing density of the native sand and gravel material along the existing SH 9 appears to range from very loose to very dense. In the area of Boring YA-1, very loose organic sands were encountered at depth of 20 feet. In the area of Boring YA-3, very loose clayey gravel layer were encountered at depth of 11 feet (right below the fill layer). These low strength materials in the soil profile likely result in distress such as cracks, settlement and slides. Depending on the structure type selected, all foundation alternatives should address bearing capacity and global stability.

9.2 Landslides

Reconnaissance of the project area revealed a significant cut slope failure at approximately Sta. 5+00 due to a weak, friable carbonaceous shale layer which resulted in a combination planar-wedge failure. Further cuts in this area could pose additional slope concerns. Where the middle shaly layer of the Upper quartzite member of the Dakota formation is exposed in a critically oriented cut, for example approximately Sta. 5+00 of the first alternative alignment, stabilization or mitigation approaches need to be instituted as this carbonaceous member is weak and disintegrates when exposed. Care needs to be taken when excavating in this particular area since a cut through this section steeper than about 24 º (~2.5H:1V) will likely be unstable if this weak shale bedding layer daylights directly into the cut. Any cuts in this area may require stabilization, design accommodation or retaining structures to resist potential slope failure. These options will need to be considered during design and evaluated during construction.

Although not currently active, portions of the SH 9 embankment has experienced slope failures along Dillon Reservoir. Failures have occurred near Sta. 7+50 and 20+10 during periods of heavy precipitation and rapid drawdown

of the reservoir. Any proposed retaining walls or expansion of the roadway embankment in these areas should be evaluated for potential slope instability.

9.3 Soil Corrosivity

Soil samples collected during the subsurface investigation were tested to determine the water soluble sulfate concentration and pH. The concentration of water-soluble sulfates in the on-site soils ranges from 0.001 percent to 0.008 percent. The concentration of water-soluble sulfates represents a Class 0 degree of sulfate attack on concrete exposed to these materials. Test results on acidity of the soil samples indicated pH levels in the range of 8.0 to 8.4. The pH of the soils at the site indicates the soils are slightly basic and should represent a negligible degree of pH attack on concrete and metal exposed to these materials. The degree of attack is based on a range of Class 0 (negligible) to Class 3 (very severe) as presented in the Colorado Department of Transportation Standard Specifications for Road and Bridge Construction. Based on the information, we believe the use of special sulfate and acid resistant material will not be required for concrete and metal exposed to the on-site soils.

10.0 Bridge Foundation Recommendations

A new bridge structure is proposed to be constructed to carry the highway over the fen area. A potential wildlife crossing is also being considered in the study area. Since the structure selection and the location have not been provided, no exploration drilling has been conducted to evaluate the subsurface conditions for bridge foundations. The recommendation provided here is based on the preliminary field reconnaissance and observation. In general, bridge structures would likely require deep foundations extended to bedrock due to the loose nature of the subsurface soils. Design parameters and details for foundation options can be further analyzed once a final structure selection is provided.

11.0 Wall Recommendations

Retaining walls may be expected for either cut or fill slopes throughout the project limits. No subsurface investigation has been performed for earth retaining structures at this stage. The recommendations presented below are based on our reconnaissance of the site and our experience in the area. Further investigation and evaluation will be completed during the final geotechnical investigation phase.

In general, retaining walls can be placed in two categories: cut slope walls and fill slope walls. Many of the recommended retaining systems can be applied to either condition and some systems may be combined to stabilize slopes for unique cases. The following sections summarize our preliminary recommendations for each of the proposed wall alternatives.

12.0 Cut Walls

12.1 Ground Nail Walls

This system involves installing reinforcing elements as passive inclusions in an earth mass to provide stabilization. Although identified as “ground” or “soil” nails, the system is utilized to stabilize either rock and soil conditions, or a combination of both.

12.2 Ground Anchor Walls

A ground anchor wall can be used in place of a soil nail wall if excessive groundwater is present (and where, correspondingly, soil nailing is not effective) or if additional support is required for cut slope walls, such as areas with potentially unstable slopes. In the case of ground anchors, it will likely be necessary to establish depth to bedrock in order to bond the anchor to substantially increase the load carrying capacity. Ground anchors are a post-tensioned reinforcement that can be used to minimize slope movement and wall deformation.

13.0 Fill Walls

Fill walls located adjacent to Dillon Reservoir would likely require deep foundations extended to bedrock due to the loose nature of the subsurface soils and the effect of groundwater fluctuation caused by the variation in seasonal precipitation. Changes in the water elevation in Lake Dillon could also create groundwater fluctuations causing a potential of instability for the proposed fill walls and exsitng embankment. The instability can be be created during a rapid drawdown condition as the reservoir water level is dropped during periods of high demand for water downstream and in Denver. Additional fill wall recommendations are as follows:

13.1 Mechanically Stabilized Earth (MSE) Walls

MSE retaining walls may be feasible for the fill slope retaining structures. For stability purposes, all MSE walls should be designed with a minimum embedment of 3 to 5 feet. The reinforcement lengths should be a minimum of 8 feet, 70% of the design height, or the length required to satisfy external stability requirements, whichever is greater. Global stability is usually the controlling factor for the design evaluation.

13.2 Cantilever Walls

Cantilever retaining walls may also be feasible for use as either cut or fill retaining structures. Cantilever walls may be either pre-cast or cast in place (CIP) and may be placed on shallow foundations or deep foundations. The use of cantilever walls may require a deep foundation system to transfer the wall loading either into bedrock or to dense gravels or sands deeper in the soil profile to avoid bearing capacity and settlement issues. Deep foundations could consist of driven piles, drilled shafts, or micropiles.

14.0 Site Grading

All site grading should conform to the Colorado Department of Transportation Standard Specifications for Road and Bridge Construction.

14.1 Cut Slopes

The proposed cut slopes in the project limits will encounter a variety of conditions. All excavations must comply with the applicable local, State, and Federal safety regulations, and particularly with the excavation standards of the Occupational Safety and Health Administration (OSHA). The maximum slope angle for the rock slopes should not exceed 0.5H:1V. Cut slopes in any other materials should not exceed 1.5H:1V. These values are general and approximate and require re-evaluation once a final geotechnical investigation has been performed. Flatter slopes may be required if excavations extend into the groundwater or the slopes will be exposed for an extended period of time.

Surface water runoff should be directed away from the top of the slope and should not be allowed to pond at the top or bottom of the slope. Erosion control may also be an issue in areas with finer grained materials. Steep cut slopes in the locations of existing slope failures such as at Sta. 5+00 should be avoided. Stabilization or retaining structures may be required in areas where potential slope failures may occur due to the presence of water and loose material.

If excessive water seepage is observed during the excavation, Yeh and Associates, Inc. should be notified to verify the validity of these recommendations.

14.2 Fill Slopes

Permanent fill slopes shall be constructed to a maximum slope angle of 2H:1V. Individual areas may be constructed steeper than 2H:1V but must be evaluated independently and constructed with angular rocky material. Natural slopes or where grades are 20 percent (5H:1V) or steeper shall be benched prior to fill placement. The ground surface underlying all fills should be carefully prepared and new fill properly benched in accordance with the Colorado Department of Transportation Standard Specifications for Road and Bridge Construction.

Settlement of embankment fill is anticipated during and after construction of the embankment fill. As the source of the fill material which will make up the majority of the embankment is unknown, a further evaluation for the settlement will be completed when the final design and the fill materials are identified.

It is our understanding that proposed slopes in the project area will have an approximate 2H:1V slope to 3H:1V slope. We believe the proposed slopes should remain stable with proper construction. Surface water runoff should be directed away from the top of the slope and should not be allowed to pond at the top or bottom of the slope.

Changes in the water elevation in Lake Dillon could create groundwater fluctuations with the potential for instability of the fill slopes during a rapid drawdown of a high-water condition of the reservoir if the fill material does not drain freely. Additional evaluation of the embankment stability will be required once the final configuration is determined.

15.0 Surface Drainage

Positive drainage should be provided during construction and maintained throughout the life of the proposed structures. Surface runoff and infiltration of water into subgrade materials must be prevented during construction. Surface features that could retain water in areas adjacent to the paving should be sealed or eliminated. Any permanent drainage swale adjacent to the paving must be properly graded and established for prevention of ponding of water on or immediately adjacent to pavement areas. Concentrated runoff should be avoided in areas susceptible to erosion and slope instability.

16.0 Comparison of Alignment Alternatives

The first alternative follows the current SH 9 alignment along Dillon Reservoir. This alignment would require cut slope development in hard rock which would entail significant blasting (Sta. 5+00 through approximately Sta. 30+00). Fill slopes along the reservoir margin in this same area appear to encroach on the reservoir and require retaining walls in some locations. Changes in the water elevation in Lake Dillon could

create groundwater fluctuations with the potential for instability of the fill slopes during a rapid drawdown condition of the reservoir if the fill does not drain freely. In addition the cut in the vicinity of Sta. 5+00 would require laying back the slope in the immediate area at about 2.5H:1V or alternatively some other form of stabilization.

Fill walls located adjacent to Dillon Reservoir would likely require deep foundations extended to bedrock due to the loose nature of the subsurface soils and the effect of groundwater fluctuation caused by changes in the water elevation. As indicated previously groundwater fluctuation could potentially create instability of the walls during a rapid drawdown condition of the reservoir if the backfill does not drain freely. This can be accommodated by addressing this during design of the walls.

The second alternative (Iron Springs) runs from Farmer's Corner to Frisco and follows the existing bike path for a portion of the alignment. This alignment occurs mostly in glacial till and generally can be excavated using conventional methods, though in places, hard to very hard bedrock may be encountered which would likely require blasting. Based on the preliminary findings, soil cuts in this area can be constructed at a 2H:1V provided that ground water is not encountered and surfical erosion is controlled. This alignment has been potentially identified for a wildlife crossing at approximately Sta. 25+00. It is anticipated that the crossing will be a vegetated concrete structure constructed over the highway. Depth to bedrock is not available at this location since drilling did not intersect bedrock because of thick fill used to construct a disused impoundment in the immediate area. The material encountered in the subsurface investigation generally consists of glacial till overlying bedrock. The estimated depth to bedrock is about 35-40 feet at approximately Sta. 25+00, discounting the thickness of the abandoned impoundment berm. Depth to bedrock should be determined in the final geotechnical investigation.

Based on the information obtained during the preliminary site investigation the Iron Springs alternative alignment should be less difficult to design and construct from a geotechnical perspective than following the current Dillon Reservoir alignment. The following table (Table 1) summarizes the geotechnical conditions encountered on each alignment.

Table 1. Summary of geotechnical issues along the SH 9 alignments.

Geotechnical Condition	Lake Dillon Alignment	Iron Springs Alignment
Cut slope stability	***Issue:*** Existing cut slope at Sta. 5+00 has experienced a previous slope failure ***Mitigation:*** Layback slope and remove slide, buttress and/or ground anchors	***Issue:*** No existing slope failures observed ***Mitigation:*** 2H:1V cut slopes would likely be adequate
Fill Slope Stability	***Issue:*** Existing fill slopes at Sta. 7+50 and 20+10 have experienced previous slope failures ***Mitigation:*** Avoidance, light weight fill of ground improvements	***Issue:*** No existing slope failures observed ***Mitigation:*** 2H:1V to 3H: 1V fill slopes would likely be adequate
Retaining walls	***Issue:*** Existing fill slopes at Sta. 7+50 and 20+10 have experienced previous slope failures ***Mitigation:*** Avoidance (shift alignment to the cut), light weight fill of ground improvements	***Issue:*** Retaining walls not required ***Mitigation:*** N/A
Groundwater	***Issue:*** Rapid down draw in Dillon Reservoir may have a detrimental effect of stability ***Mitigation:*** Avoidance (shift alignment to the cut), light weight fill of ground improvements	***Issue:*** Although not encountered in the site investigation, the presence of Iron Springs could be an indication that shallow ground water, seeps and springs may be present along alignment. ***Mitigation:*** Avoidance, horizontal and cut-off drains.

17.0 Limitations

This study has been conducted in accordance with generally accepted geotechnical engineering practices in this area for use by the client for design purposes. The conclusions and recommendations submitted in this report are based upon the data obtained from exploratory borings and field reconnaissance. The nature and extent of subsurface variations across the site may not become evident until excavation is performed. We recommend on-site observation of excavations and foundation bearing strata by a representative of the geotechnical engineer.

Respectfully Submitted,
Yeh and Associates, Inc.

Prepared by:	Reviewed by:
_____	_____
I-Ping Chen	Howard Hume, Ph. D, P.G.
Staff Engineer	Senior Project Manager

Enc: 6 copies

18.0 References

1. Kellogg, Karl S., et al., 2002, Geologic Map of the Frisco Quadrangle, Summit County, Colorado: Scale 1:24,000

APPENDIX A- Project Location Map

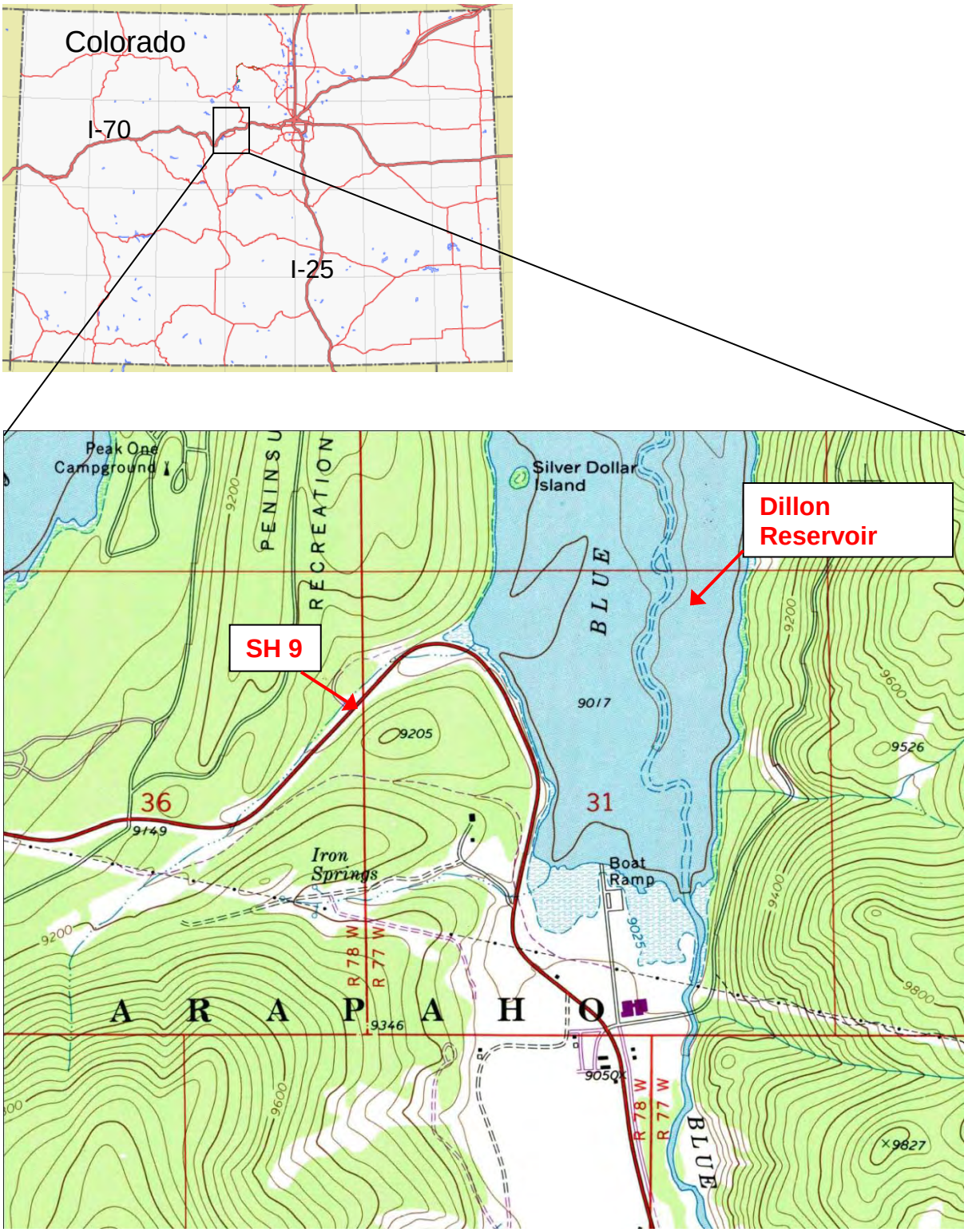
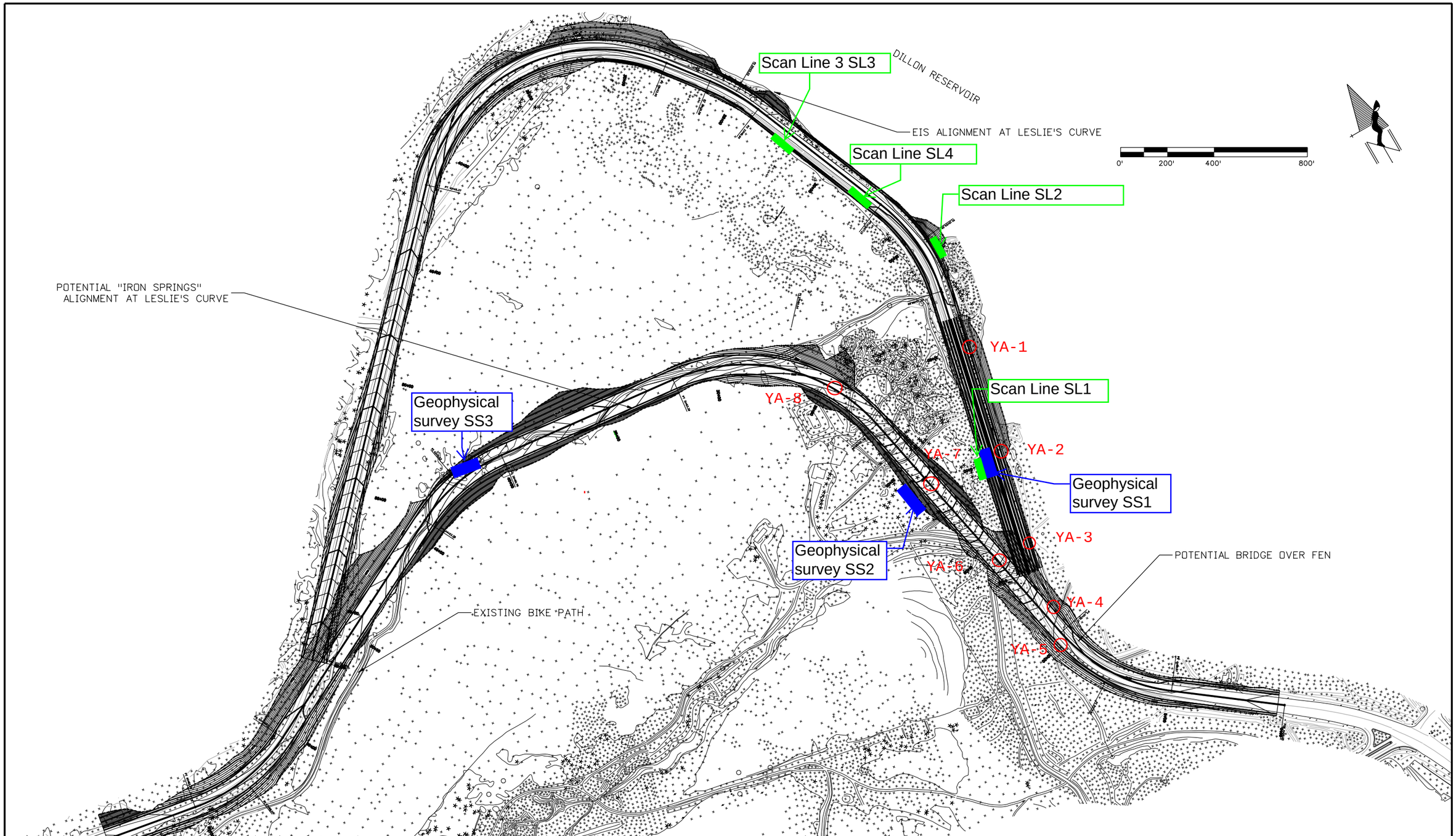


Figure A-1. Project Location Map

APPENDIX B- Site Investigation Map



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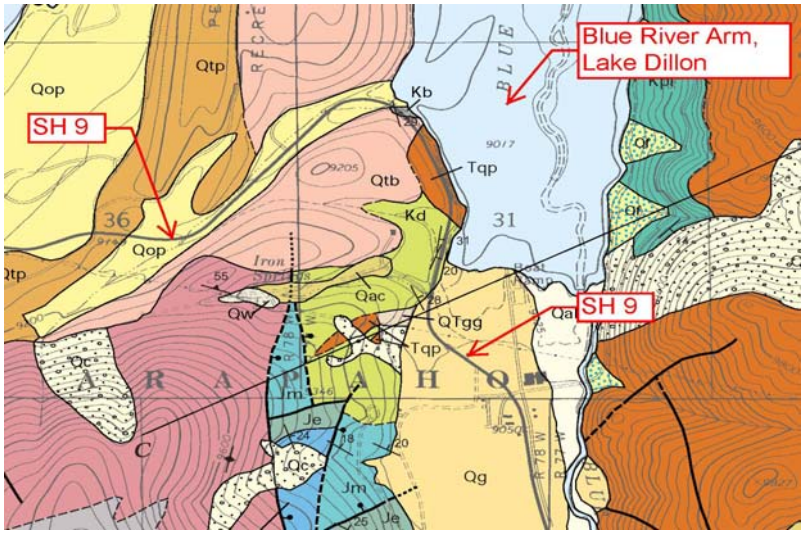
Horiz. Scale: 1:400 Vert. Scale: As Noted					Region 1 WDS
Unit Information Unit Leader Initials					

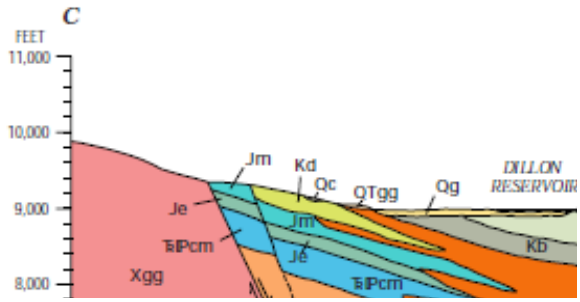
As Constructed	POTENTIAL ALIGNMENT			Project No./Code
No Revisions:				_____
Revised:	Designer: G. Anderson	Structure Numbers	X-XX-XX	_____
	Detailer: G. Anderson		X-XX-XX	
Void:	Sheet Subset: _____	Subset Sheets:	1 of X	Sheet Number XXX

APPENDIX C- Geology Map and Legend

LIST OF MAP UNITS

af	Artificial fill (recent)
dt	Dredge tailings (recent)
Qal	Alluvium (Holocene)
Qw	Wetland deposits (Holocene)
Qav	Avalanche deposits (Holocene)
Qry	Active rock-glacier deposits (Holocene)
Qtr	Travertine (Holocene)
Qr	Inactive rock-glacier deposits (Holocene and upper Pleistocene)
Qf	Fan deposits (Holocene and upper Pleistocene)
Qt	Talus (Holocene and upper Pleistocene)
Qc	Colluvium (Holocene and upper Pleistocene)
Qac	Alluvium and colluvium, undivided (Holocene and upper Pleistocene)
Qls	Younger landslide deposits (Holocene and upper Pleistocene)
Qls (Kb)	Large landslide deposit composed entirely of Benton Shale
Qg	Terrace gravel (Holocene to middle Pleistocene)
Qop	Pinedale outwash deposits (upper Pleistocene)
Qtp	Till of Pinedale glaciation (upper Pleistocene)
Qtb	Till of Bull Lake glaciation (middle Pleistocene)
Qgo	Older outwash gravel (middle or lower Pleistocene)
QTd	Diamicton (middle Pleistocene to Pliocene?)
QTls	Older landslide deposits (middle Pleistocene to Pliocene)
QTgm	Bouldery gravel of Mesa Cortina ("Buffalo placers") (middle Pleistocene to Pliocene?)
QTgg	Bouldery gravel of Cold Run (middle or lower Pleistocene to Pliocene)
Tqp	Quartz monzonite porphyry (Eocene)
Tmp	Hornblende-biotite monzonite porphyry (Eocene)
	Pierre Shale (Upper Cretaceous)
Kpm	Shale and sandstone member
Kps	Kremmling Sandstone Member
Kpl	Lower shale member
Kn	Niobrara Formation (Upper Cretaceous)
Kb	Benton Shale (Upper Cretaceous)
Kd	Dakota Sandstone (Lower Cretaceous)

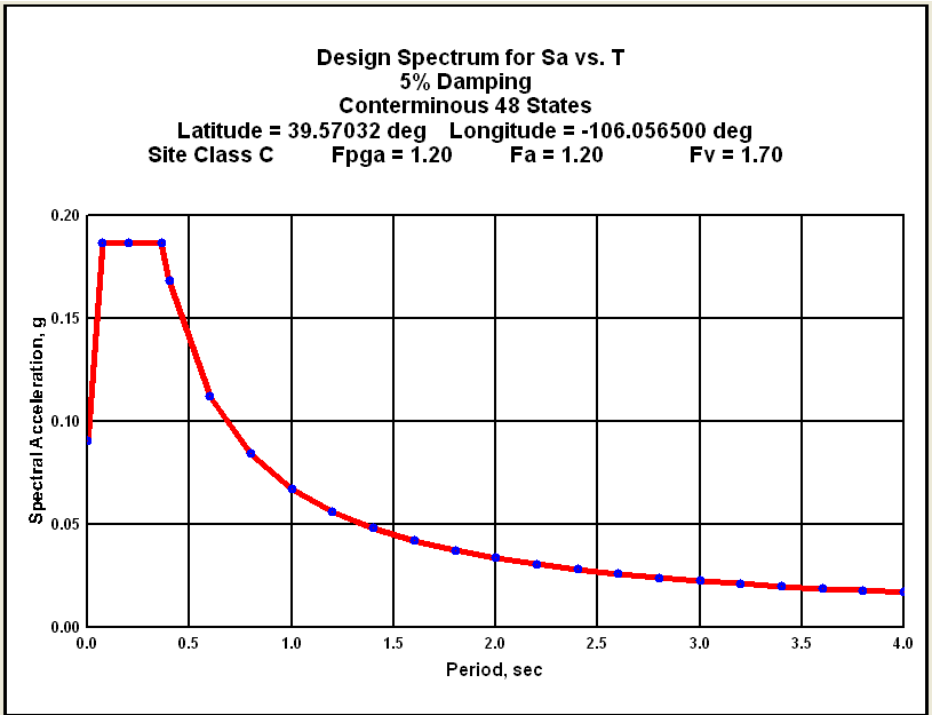




U.S. DEPARTMENT OF THE INTERIOR
U.S. GEOLOGICAL SURVEY

Geology Map and Legend

APPENDIX D- Seismic Design Spectra



Graph Data	
Period, sec	Sa, g
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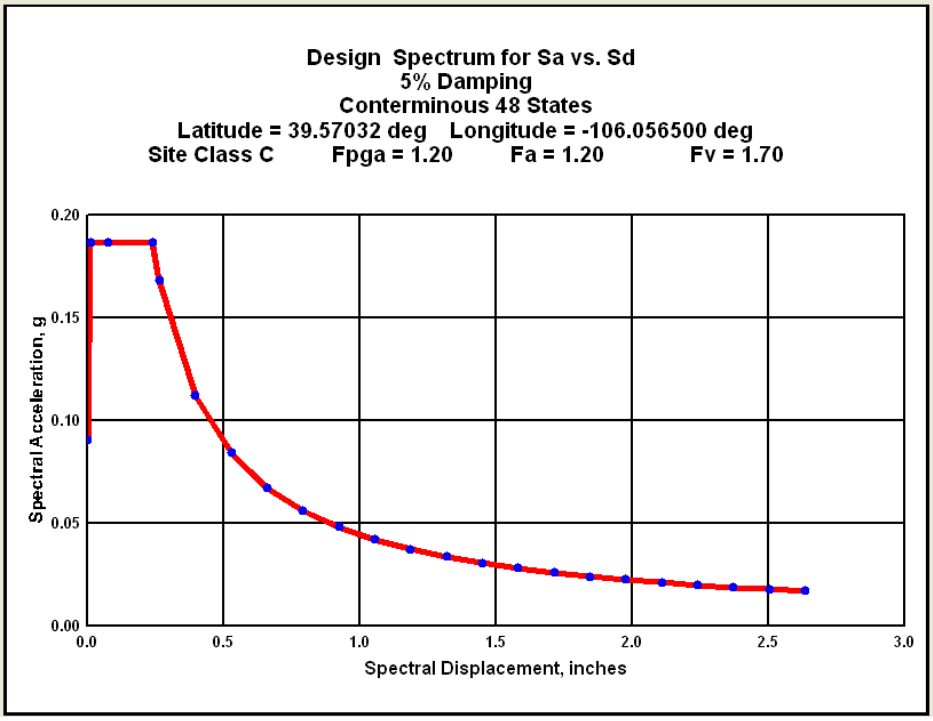
0.07	0.1867
0.20	0.1867
0.36	0.1867
0.40	0.1684
0.60	0.1123
0.80	0.0842
1.00	0.0674
1.20	0.0561
1.40	0.0481
1.60	0.0421
1.80	0.0374
2.00	0.0337
2.20	0.0306
2.40	0.0281
2.60	0.0259
2.80	0.0241
3.00	0.0225
3.20	0.0211
3.40	0.0198
3.60	0.0187
3.80	0.0177
4.00	0.0168



Graph Data	
Sd, inches	Sa, g
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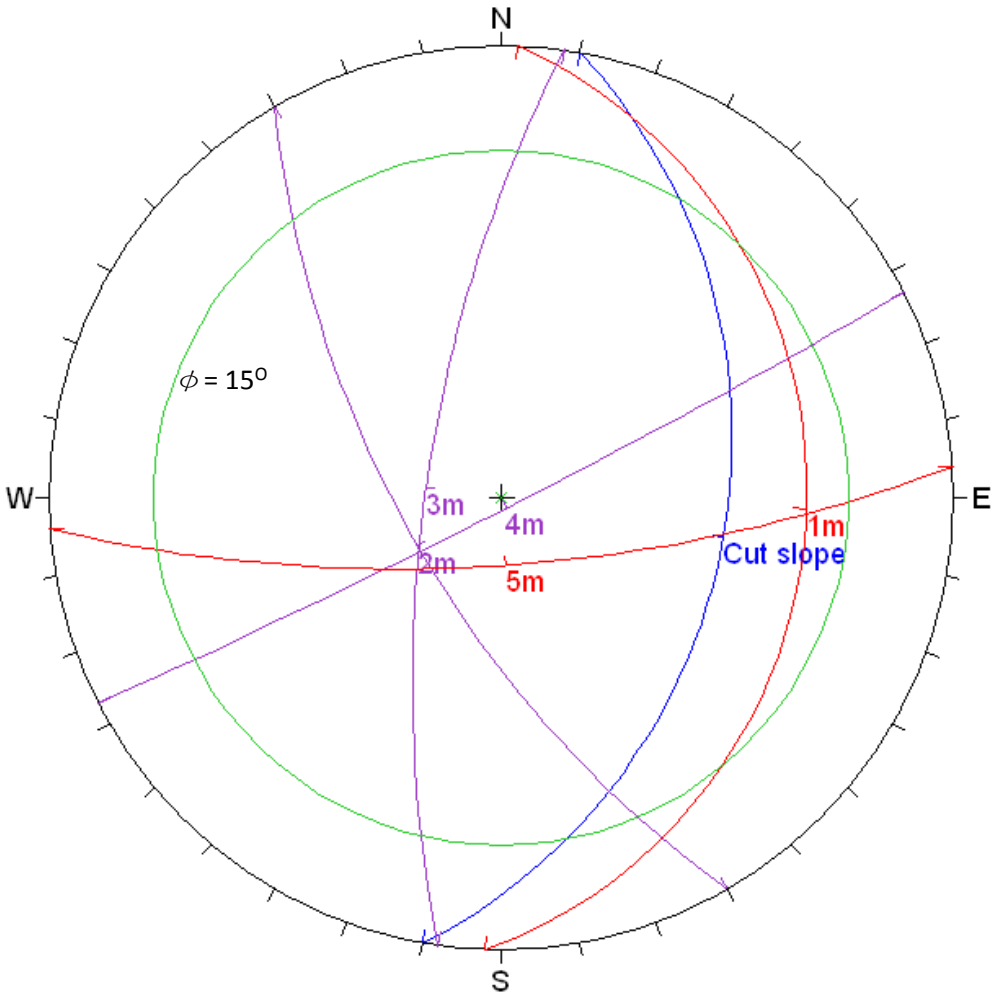
0.07	0.1867
0.24	0.1867
0.26	0.1684
0.39	0.1123
0.53	0.0842
0.66	0.0674
0.79	0.0561
0.92	0.0481
1.05	0.0421
1.18	0.0374
1.32	0.0337
1.45	0.0306
1.58	0.0281
1.71	0.0259
1.84	0.0241
1.97	0.0225
2.11	0.0211
2.24	0.0198
2.37	0.0187
2.50	0.0177
2.63	0.0168

APPENDIX E- Existing Rockcut Evaluation

210-147 SH 9 Frisco to Breckenridge SL 1



Fisher
Concentrations
% of total per 1.0 % area

0.00 ~ 3.50 %
3.50 ~ 7.00 %
7.00 ~ 10.50 %
10.50 ~ 14.00 %
14.00 ~ 17.50 %
17.50 ~ 21.00 %
21.00 ~ 24.50 %

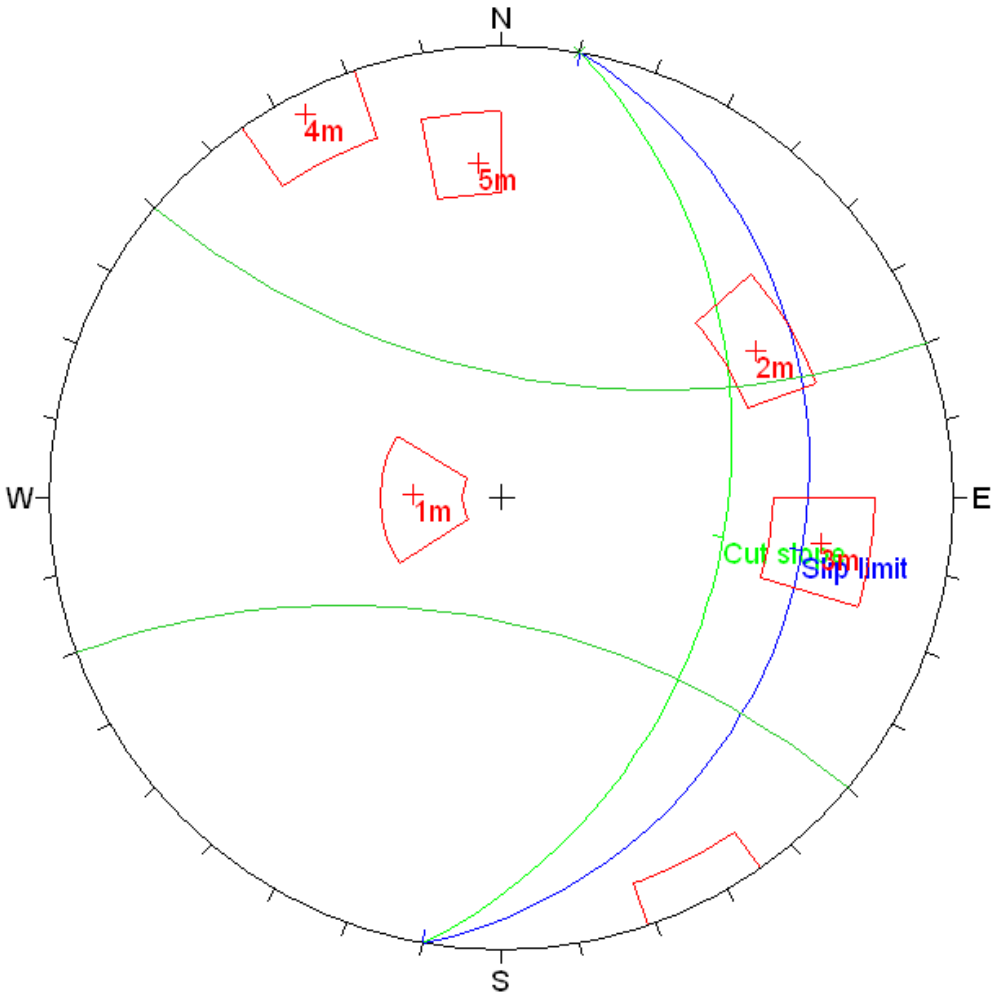
No Bias Correction
Max. Conc. = 21.6962%

Equal Angle
Lower Hemisphere
25 Poles
25 Entries

Set	Dip (degree)	Dip Direction (degree)
1m	22	92
2m	66	240
3m	71	278
4m	87	153
5m	73	176

1m = discontinuity set 1 (unweighted mean plane)

210-147 SH 9 Frisco to Breckenridge SL 1



Fisher
Concentrations
% of total per 1.0 % area

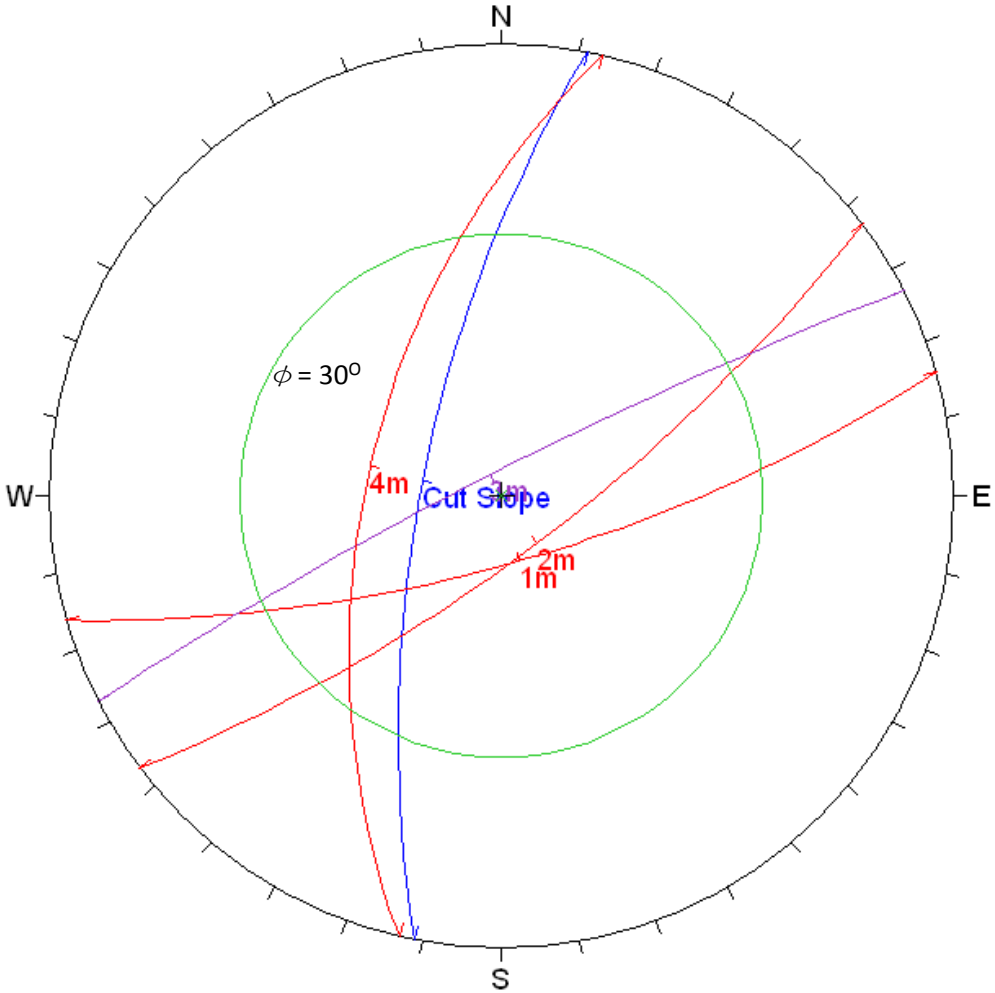
0.00 ~ 3.50 %
3.50 ~ 7.00 %
7.00 ~ 10.50 %
10.50 ~ 14.00 %
14.00 ~ 17.50 %
17.50 ~ 21.00 %
21.00 ~ 24.50 %

No Bias Correction
Max. Conc. = 21.6962%

Equal Angle
Lower Hemisphere
25 Poles
25 Entries

1m = discontinuity set 1 (unweighted mean pole)

210-147 SH 9 Frisco to Breckenridge SL 2



Fisher
Concentrations
% of total per 1.0 % area

0.00 ~ 4.00 %
4.00 ~ 8.00 %
8.00 ~ 12.00 %
12.00 ~ 16.00 %
16.00 ~ 20.00 %
20.00 ~ 24.00 %
24.00 ~ 28.00 %

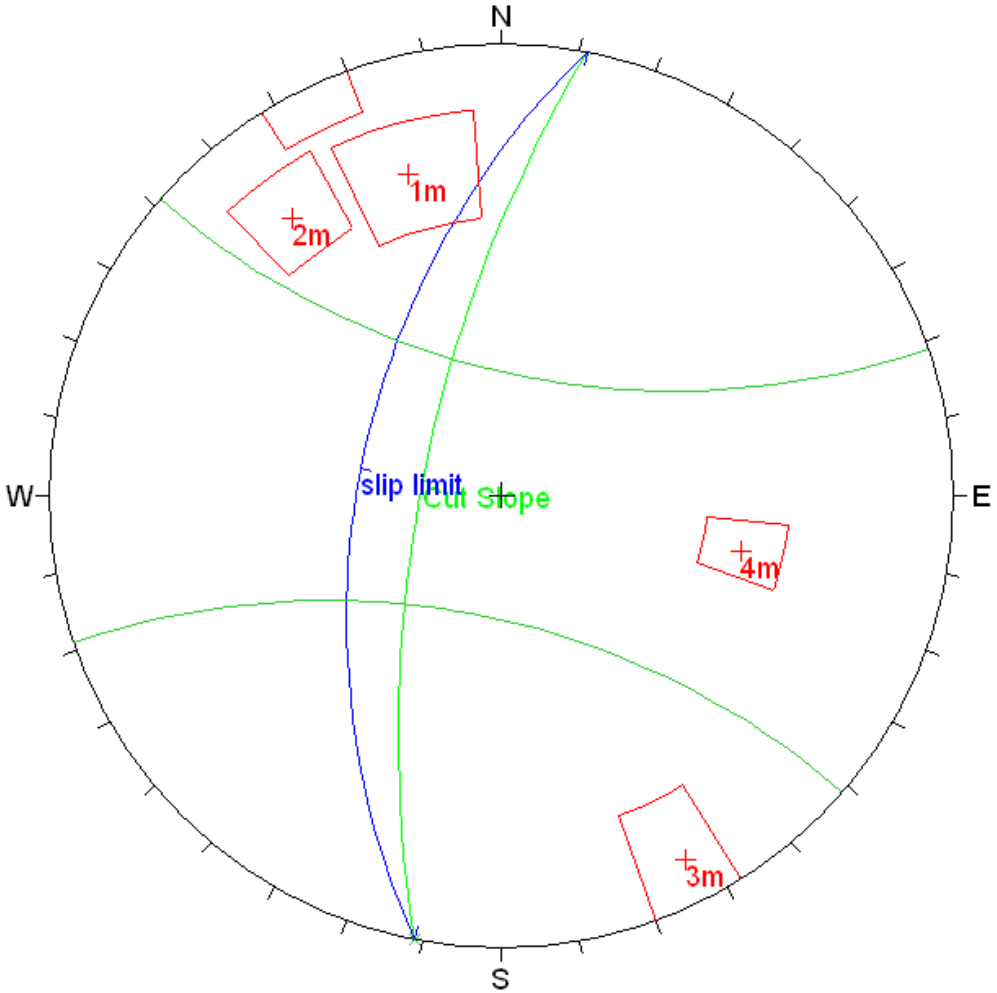
No Bias Correction
Max. Conc. = 26.1173%

Equal Angle
Lower Hemisphere
35 Poles
35 Entries

Set	Dip	Dip Direction
1m	73	164
2m	75	143
3m	84	333
4m	57	283

1m = discontinuity set 1 (unweighted mean plane)

210-147 SH 9 Frisco to Breckenridge SL 2



Fisher
Concentrations
% of total per 1.0 % area

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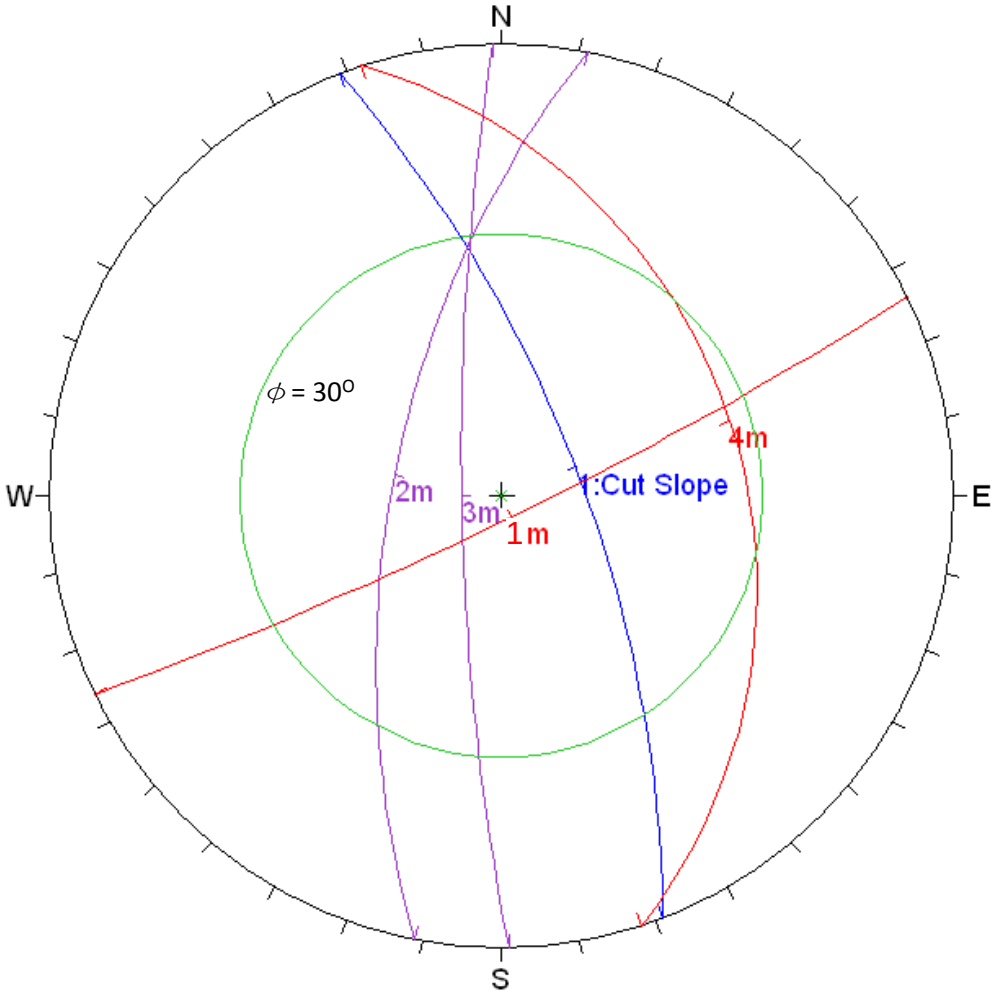
4.00 ~ 8.00 %
8.00 ~ 12.00 %
12.00 ~ 16.00 %
16.00 ~ 20.00 %
20.00 ~ 24.00 %
24.00 ~ 28.00 %

No Bias Correction
Max. Conc. = 26.1173%

Equal Angle
Lower Hemisphere
35 Poles
35 Entries

1m = discontinuity set 1 (unweighted mean pole)

210-147 SH 9 Frisco to Breckenridge SL 3



Fisher
Concentrations
% of total per 1.0 % area

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3.50 ~ 7.00 %
7.00 ~ 10.50 %
10.50 ~ 14.00 %
14.00 ~ 17.50 %
17.50 ~ 21.00 %
21.00 ~ 24.50 %

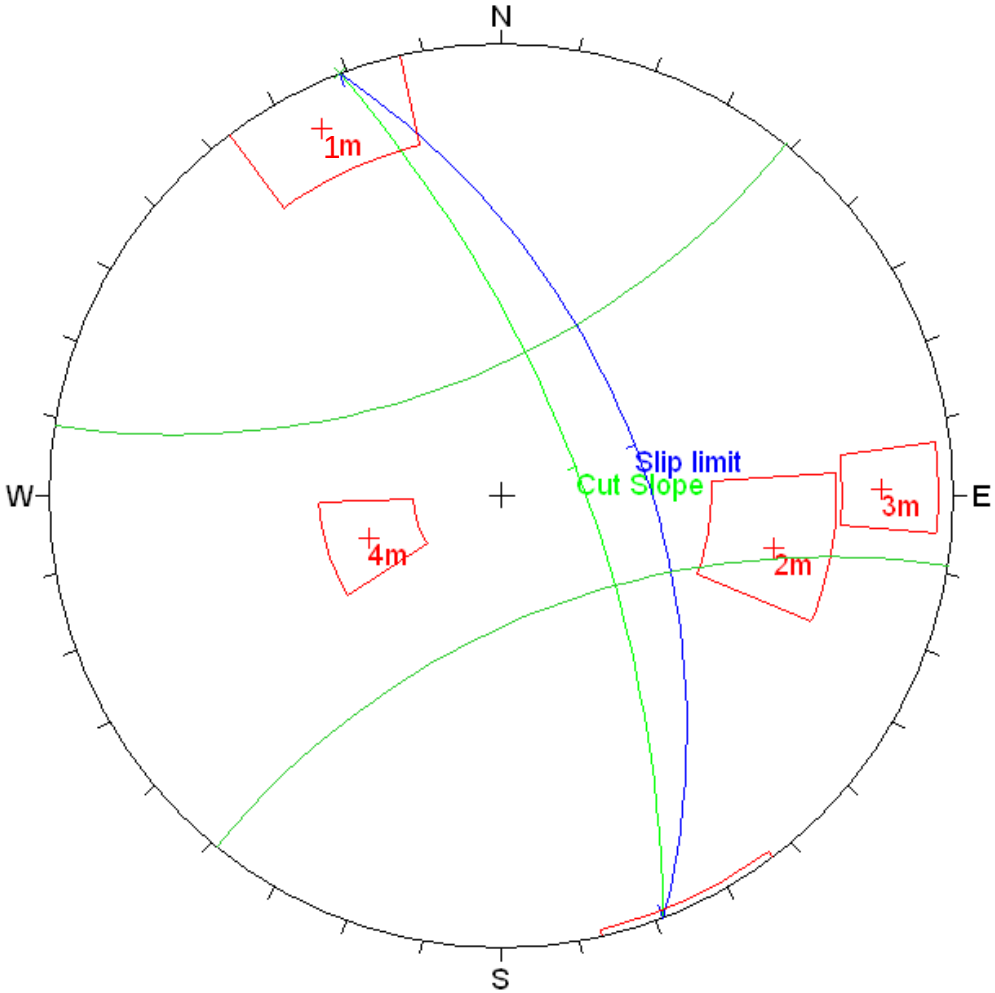
No Bias Correction
Max. Conc. = 23.0640%

Equal Angle
Lower Hemisphere
29 Poles
29 Entries

Set	Dip	Dip Direction
1m	84	154
2m	63	281
3m	80	269
4m	34	72

1m = discontinuity set 1 (unweighted mean plane)

210-147 SH 9 Frisco to Breckenridge SL 3



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Concentrations
% of total per 1.0 % area

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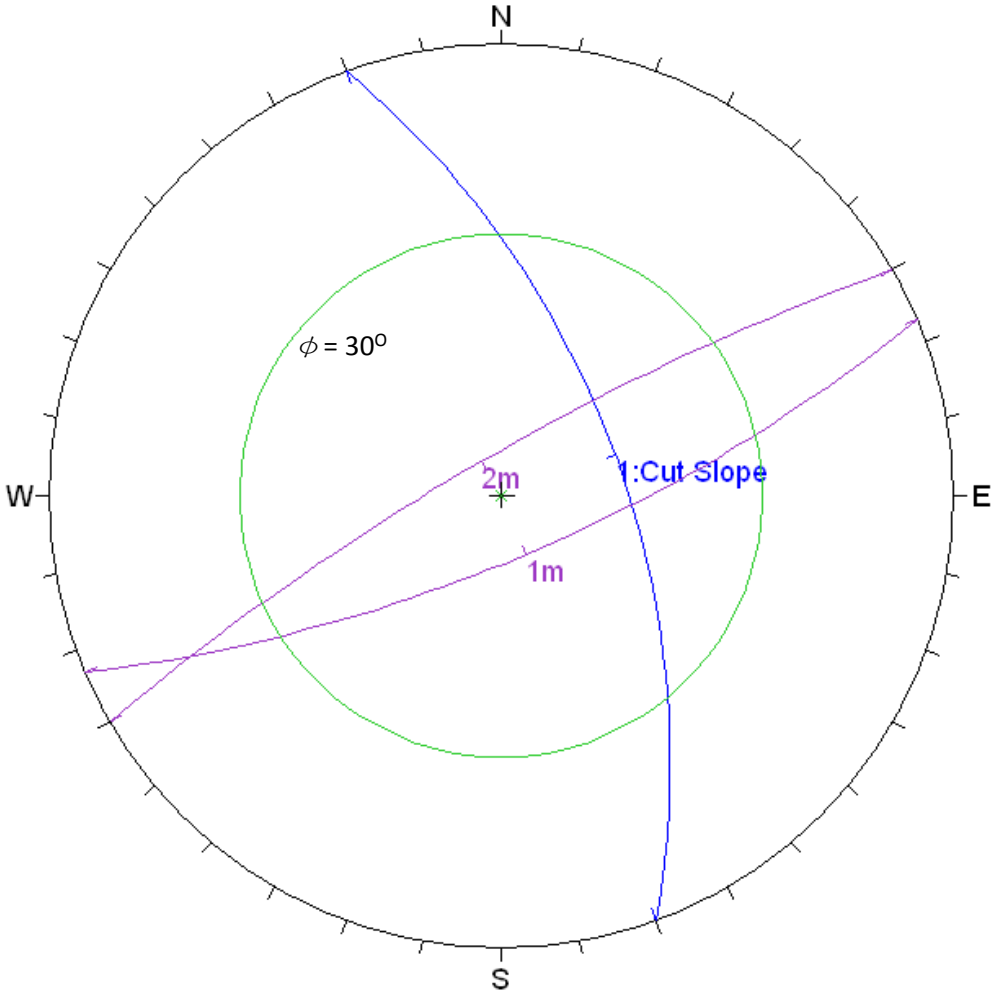
7.00 ~ 10.50 %
10.50 ~ 14.00 %
14.00 ~ 17.50 %
17.50 ~ 21.00 %
21.00 ~ 24.50 %

No Bias Correction
Max. Conc. = 23.0640%

Equal Angle
Lower Hemisphere
29 Poles
29 Entries

1m = discontinuity set 1 (unweighted mean pole)

210-147 SH 9 Frisco to Breckenridge SL 4



Fisher
Concentrations
% of total per 1.0 % area

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11.00 ~ 16.50 %
16.50 ~ 22.00 %
22.00 ~ 27.50 %
27.50 ~ 33.00 %
33.00 ~ 38.50 %

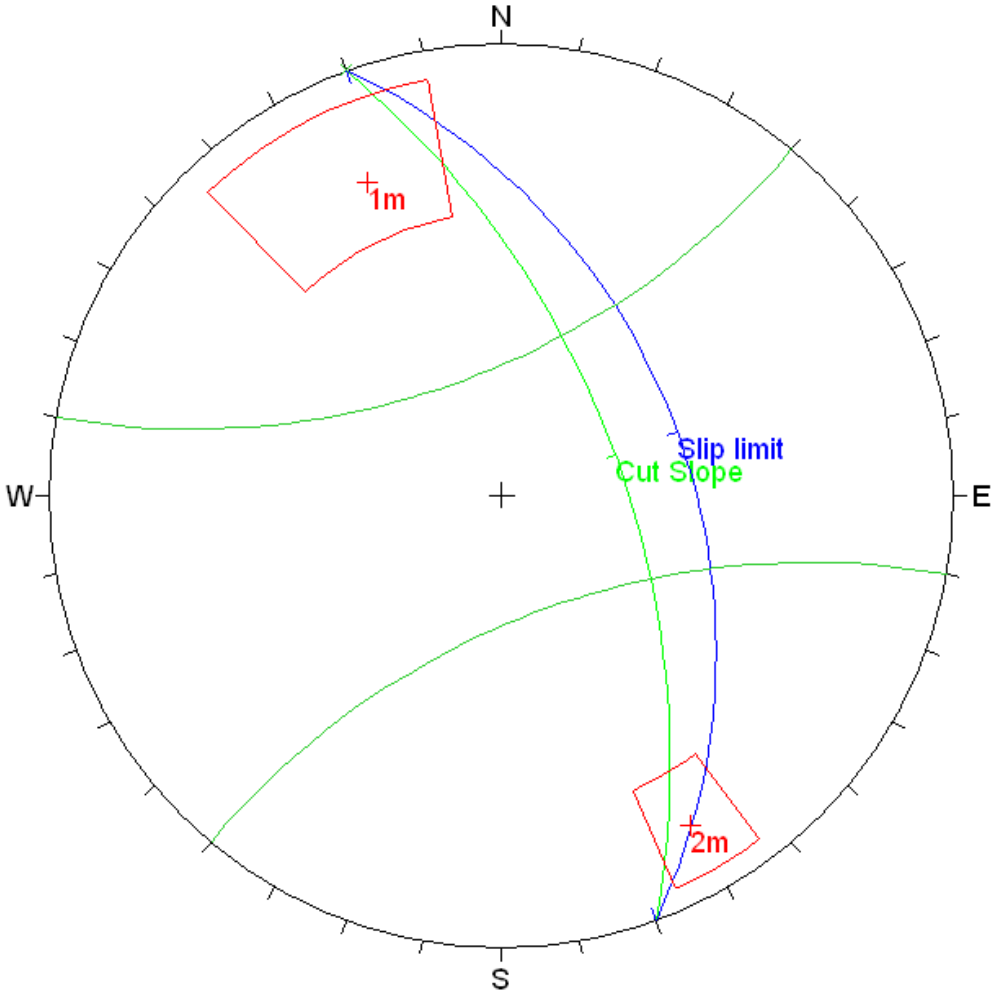
No Bias Correction
Max. Conc. = 37.2754%

Equal Angle
Lower Hemisphere
20 Poles
20 Entries

Set	Dip	Dip Direction
1m	74	157
2m	80	330

1m = discontinuity set 1 (unweighted mean plane)

210-147 SH 9 Frisco to Breckenridge SL 4



Fisher
Concentrations
% of total per 1.0 % area

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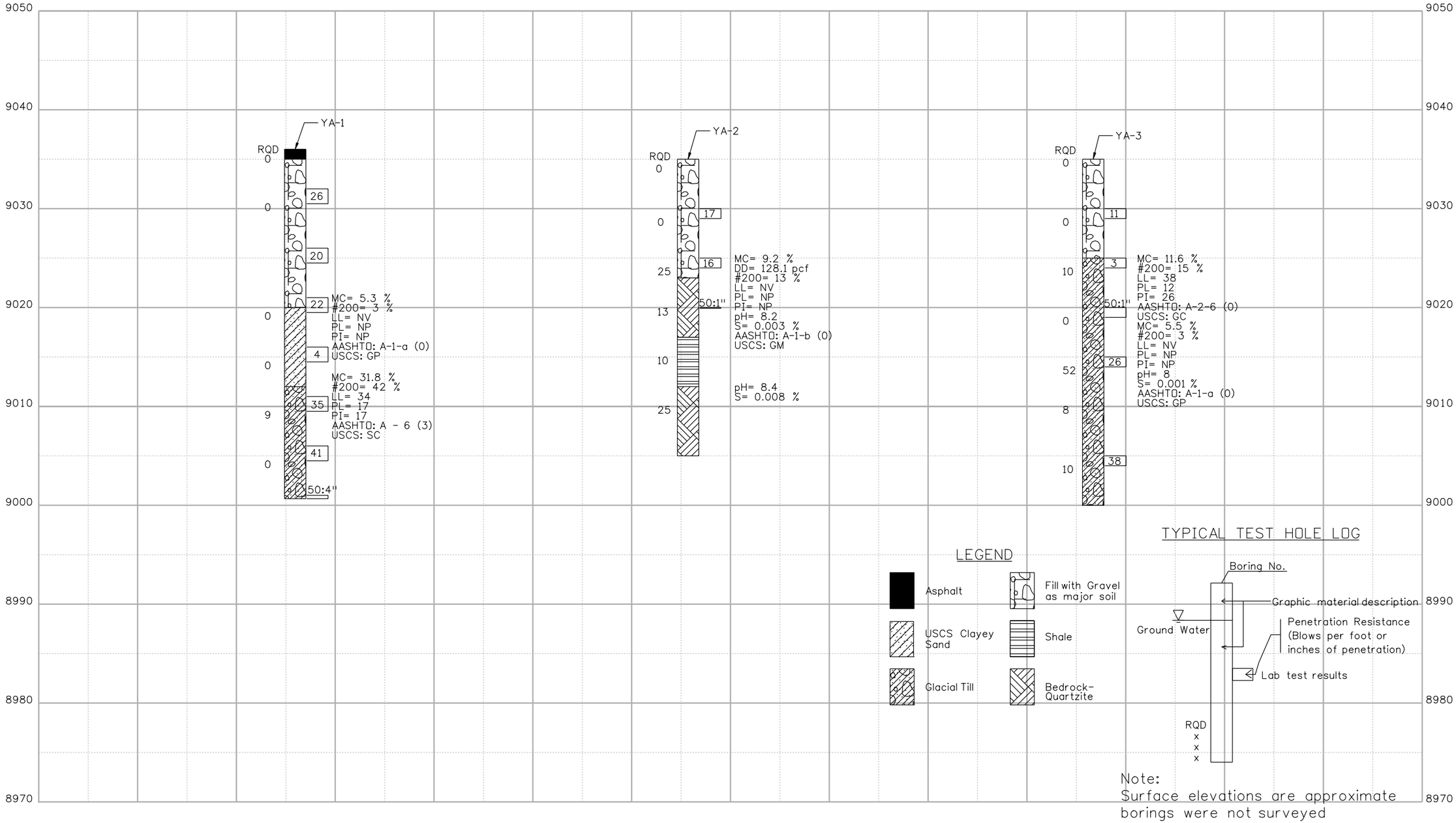
5.50 ~ 11.00 %
11.00 ~ 16.50 %
16.50 ~ 22.00 %
22.00 ~ 27.50 %
27.50 ~ 33.00 %
33.00 ~ 38.50 %

No Bias Correction
Max. Conc. = 37.2754%

Equal Angle
Lower Hemisphere
20 Poles
20 Entries

1m = discontinuity set 1 (unweighted mean pole)

APPENDIX F- Engineering Geology Sheets and Boring Logs



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Horiz. Scale: 1:10 Vert. Scale: As Noted				Region 1 WDS	Revised:
Unit Information Unit Leader Initials					Void:
Yeh and Associates, Inc. Consulting Engineers & Scientists					

ENGINEERING GEOLOGY				Project No./Code
Designer: H. HUME	Structure Numbers			Project Number
Detailer: M. WALZ				Code
Sheet Subset:	Subset Sheets:			Sheet Number

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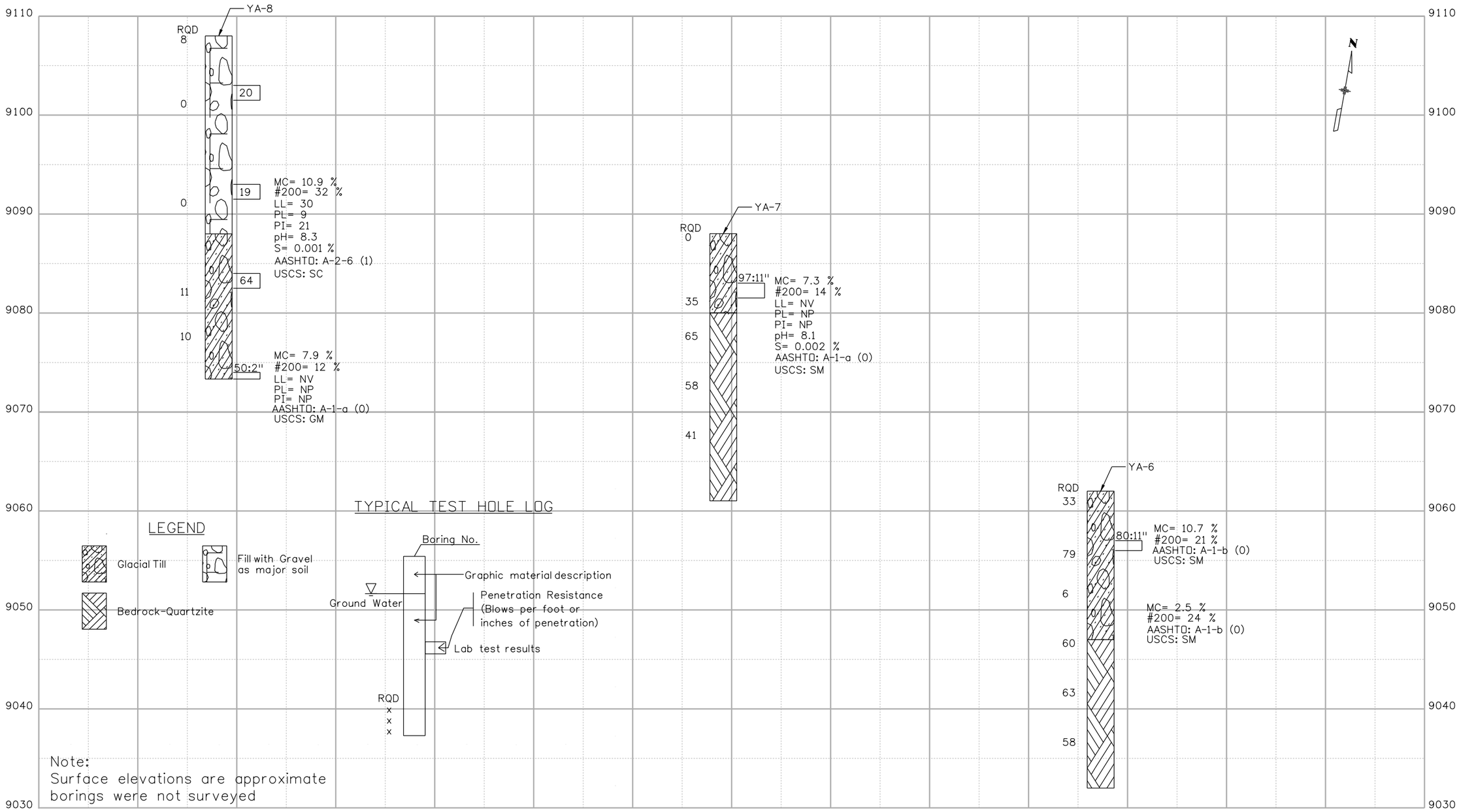

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Horiz. Scale: 1:10	Vert. Scale: As Noted	R-X		
Unit Information	Unit Leader Initials			

Yeh and Associates, Inc.
Consulting Engineers & Scientists

Colorado Department of Transportation
DOT
DEPARTMENT OF TRANSPORTATION
Moutain Residency P.O. Box 399
Dumont, CO 80436
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Region 1 WDS

As Constructed
No Revisions:
Revised:
Void:

ENGINEERING GEOLOGY

Designer:	H. HUME	Structure Numbers	
Detailer:	M. WALZ		
Sheet Subset:		Subset Sheets:	

Project No./Code
Project Number
Code
Sheet Number

Mike Walz 1:26:57 PM W:\2010 Projects\210-147 SH 9 Frisco to Breckenridge\Dwg\210-147 Engineering Geology02.dgn

YEH AND ASSOCIATES, INC.
GEOTECHNICAL ENGINEERING CONSULTANTS

Project: SH 9 Frisco to Breckenridge
Project Number: 210-147 Date:

Boring: **YA-1**
Sheet 1 of 1

Boring Began: 10/4/2010
Drilling Method: Coring
Drill: CME 75
Driller: CDOT - Dave
Logged By: I. Ksouri
Final By: M. Norris
Inclination: Vertical

Completed: 10/4/2010
Drill Bit:
Casing:
Weather:

Total Depth: 35.3 ft
Ground Elevation:
Location:
Coordinates: N: E:

Ground Water Notes: Not encountered during drilling

Depth	-	-	-	-
Date	-	-	-	-
Time	-	-	-	-

Elevation (feet)	Depth (feet)	Run / Sample Type	Recovery (%)	Rock RQD	Soil Samples Blows per 6 in	Soil Samples N	Lithology	Material Description	Field Notes and Lab Tests
								0.0 - 1.0 ft. ASPHALT, no.	
			8	0				**1.0 - 16.0 ft. FILL GRAVEL** with sand, cobbles and boulders, tan to gray - brown, medium dense, thin layers of clay.	
					12/12/14	26			
			22	0					
	10				8/9/11	20			
					9/13/9	22			MC= 5.3 % #200= 3 % LL= NV PL= NP PI= NP AASHTO: A-1-a (0) USCS: GP
			0	0				**16.0 - 24.0 ft. clayey SAND**, medium plasticity, very loose.	
	20				2/2/2	4		At 20-24 ft. with organic black material.	
			14	0					MC= 31.8 % #200= 42 % LL= 34 PL= 17 PI= 17 AASHTO: A - 6 (3) USCS: SC
					3/11/24	35		**24.0 - 35.3 ft. GRAVEL** with sand, boulders and cobbles, tan to gray - brown, dense to very dense, Glacial Till.	
			36	9					
	30				25/20/21	41			
			17	0					
					50:4"	50:4"		Bottom of Hole at 35.3 ft.	

YEH AND ASSOCIATES, INC.
GEOTECHNICAL ENGINEERING CONSULTANTS

Project: SH 9 Frisco to Breckenridge
Project Number: 210-147 Date:

Boring: **YA-2**
Sheet 1 of 1

Boring Began: 9/29/2010
Drilling Method: Coring
Drill: CME 55
Driller: CDOT - Dave
Logged By: I. Ksouri
Final By: M. Norris
Inclination: Vertical

Completed: 9/29/2010
Drill Bit:
Casing:
Weather:

Total Depth: 30.0 ft
Ground Elevation:
Location:
Coordinates: N: E:

Ground Water Notes: Not encountered during drilling

Depth	-	-	-	-
Date	-	-	-	-
Time	-	-	-	-

Elevation (feet)	Depth (feet)	Run / Sample Type	Recovery (%)	Rock RQD	Soil Samples Blows per 6 in	Soil Samples N	Lithology	Material Description	Field Notes and Lab Tests
			28	0				**0.0 - 12.0 ft. FILL GRAVEL** with silt, sand, cobbles and boulders, tan to gray - brown, no plasticity, medium dense, thin layers of clay.	
					8/9	17			
			25	0					
	10				5/11	16			MC= 9.2 %
			81	25				**12.0 - 18.0 ft. QUARTZITE**, light brown to light gray, very hard, highly fractured in all directions, iron stains at joints.	DD= 128.1 pcf #200= 13 % LL= NV PL= NP PI= NP pH= 8.2 S= 0.003 % AASHTO: A-1-b (0) USCS: GM
					50:1"	50:1"			
			98	14				**18.0 - 23.0 ft. CARBONACEOUS SHALE**, black.	
	20		98	10				**23.0 - 30.0 ft. QUARTZITE**, light brown to light gray, very hard.	pH= 8.4 S= 0.008 %
			50	25					
	30							Bottom of Hole at 30.0 ft.	

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YEH AND ASSOCIATES, INC.
GEOTECHNICAL ENGINEERING CONSULTANTS

Project: SH 9 Frisco to Breckenridge
Project Number: 210-147 Date:

Boring: **YA-3**
Sheet 1 of 1

Boring Began: 9/30/2010
Drilling Method: Coring
Drill: CME 75
Driller: CDOT - Dave
Logged By: I. Ksouri
Final By: M. Norris
Inclination: Vertical

Completed: 9/30/2010
Drill Bit:
Casing:
Weather:

Total Depth: 35.0 ft
Ground Elevation:
Location:
Coordinates: N: E:

Ground Water Notes: Not encountered during drilling

Depth	-	-	-	-
Date	-	-	-	-
Time	-	-	-	-

Elevation (feet)	Depth (feet)	Run / Sample Type	Recovery (%)	Rock RQD	Soil Samples Blows per 6 in	Soil Samples N	Lithology	Material Description	Field Notes and Lab Tests
			17	0				**0.0 - 10.0 ft. FILL GRAVEL** with sand, cobbles and boulders, tan to gray - brown, medium plasticity, loose, thin layers of clay.	
					3/8	11			
			15	0					
	10				1/2	3		**10.0 - 35.0 ft. clayey GRAVEL** poorly graded, with cobbles and boulders, tan to gray - brown, no plasticity, very loose to very dense, some clay, Glacial Till.	MC= 11.6 % #200= 15 % LL= 38 PL= 12 PI= 26 AASHTO: A-2-6 (0) USCS: GC
			50	11					
					50:1"	50:1"			MC= 5.5 % #200= 3 % LL= NV PL= NP PI= NP pH= 8 S= 0.001 % AASHTO: A-1-a (0) USCS: GP
			90	0					
	20				4/22	26			
			100	52					
			80	8					
	30				2/36	38			
			63	11					
								Bottom of Hole at 35.0 ft.	

YEH AND ASSOCIATES, INC.
GEOTECHNICAL ENGINEERING CONSULTANTS

Project: SH 9 Frisco to Breckenridge
Project Number: 210-147 Date:

Boring: **YA-6**
Sheet 1 of 1

Boring Began: 10/6/2010
Drilling Method: Coring
Drill: CME 55
Driller: CDOT - Dave
Logged By: I. Ksouri
Final By: M. Norris
Inclination: Vertical

Completed: 10/6/2010
Drill Bit:
Casing:
Weather:

Total Depth: 30.0 ft
Ground Elevation:
Location:
Coordinates: N: E:

Ground Water Notes: Not encountered during drilling

Depth	-	-	-	-
Date	-	-	-	-
Time	-	-	-	-

Elevation (feet)	Depth (feet)	Run / Sample Type	Recovery (%)	Rock RQD	Soil Samples Blows per 6 in	Soil Samples N	Lithology	Material Description	Field Notes and Lab Tests
			72	33				**0.0 - 15.0 ft. silty SAND** with gravels, cobbles and boulders, tan to gray, no plasticity, very dense, thin layers of clay, Glacial Till.	MC= 10.7 % #200= 21 % AASHTO: A-1-b (0) USCS: SM
					80:11"	80:11"			
			100	79					
	10		32	7					MC= 2.5 % #200= 24 % AASHTO: A-1-b (0) USCS: SM
			83	60				**15.0 - 30.0 ft. QUARTZITE**, light brown to light gray, very hard, iron stains at the joints.	
	20		83	63					
			92	58					
	30							Bottom of Hole at 30.0 ft.	

YEH AND ASSOCIATES, INC.
GEOTECHNICAL ENGINEERING CONSULTANTS

Project: SH 9 Frisco to Breckenridge
Project Number: 210-147 Date:

Boring: **YA-7**
Sheet 1 of 1

Boring Began: 10/5/2010
Drilling Method: Coring
Drill: CME 55
Driller: CDOT - Dave
Logged By: I. Ksouri
Final By: M. Norris
Inclination: Vertical

Completed: 10/5/2010
Drill Bit:
Casing:
Weather:

Total Depth: 27.0 ft
Ground Elevation:
Location:
Coordinates: N: E:

Ground Water Notes: Not encountered during drilling

Depth	-	-	-	-
Date	-	-	-	-
Time	-	-	-	-

Elevation (feet)	Depth (feet)	Run / Sample Type	Recovery (%)	Rock RQD	Soil Samples Blows per 6 in	Soil Samples N	Lithology	Material Description	Field Notes and Lab Tests
			27	0				**0.0 - 8.0 ft. silty SAND** with gravels, cobbles and boulders, tan to gray - brown, no plasticity, very dense, layers of clay, Glacial Till.	
					32/97:11"	97:11"			MC= 7.3 % #200= 14 % LL= NV PL= NP PI= NP pH= 8.1 S= 0.002 % AASHTO: A-1-a (0) USCS: SM
			93	36				**8.0 - 27.0 ft. QUARTZITE**, light brown to light gray, iron stains at the joints, highly fractured in all directions.	
	10		100	65					
			100	58					
	20		100	42					
								Bottom of Hole at 27.0 ft.	
	30								

YEH AND ASSOCIATES, INC.
GEOTECHNICAL ENGINEERING CONSULTANTS

Project: SH 9 Frisco to Breckenridge
Project Number: 210-147 Date:

Boring: **YA-8**
Sheet 1 of 1

Boring Began: 10/5/2010
Drilling Method: Coring
Drill: CME 55
Driller: CDOT - Dave
Logged By: I. Ksouri
Final By: M. Norris
Inclination: Vertical

Completed: 10/5/2010
Drill Bit:
Casing:
Weather:

Total Depth: 34.7 ft
Ground Elevation:
Location:
Coordinates: N: E:

Ground Water Notes: Not encountered during drilling

Depth	-	-	-	-
Date	-	-	-	-
Time	-	-	-	-

Elevation (feet)	Depth (feet)	Run / Sample Type	Recovery (%)	Rock RQD	Soil Samples Blows per 6 in	Soil Samples N	Lithology	Material Description	Field Notes and Lab Tests
			13	8				**0.0 - 20.0 ft. clayey SAND** with gravels, cobbles and boulders, tan to brown-gray, no plasticity, medium dense.	
					6/6/14	20			
	10		49	0					
					6/8/11	19			MC= 10.9 % #200= 32 % LL= 30 PL= 9 PI= 21 pH= 8.3 S= 0.001 % AASHTO: A-2-6 (1) USCS: SC
	20		32	0				**20.0 - 34.7 ft. silty GRAVEL** with sand, cobbles and boulders, tan to brown-gray, no plasticity, very dense, Glacial Till.	
					30/32/32	64			MC= 7.9 % #200= 12 % LL= NV PL= NP PI= NP AASHTO: A-1-a (0) USCS: GM
			56	11					
	30		52	11					
					32/50:2"	50:2"			
								Bottom of Hole at 34.7 ft.	

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APPENDIX G- Photographs of Rock Core

Boring YA-2
Depth 0-20 ft

Boring YA-2
Depth 20-30 ft

Boring YA-3
Depth 0-25 ft

Boring YA-3
Depth 25-35 ft

Boring YA-6
Depth 0-15 ft

Boring YA-6
Depth 15-25 ft

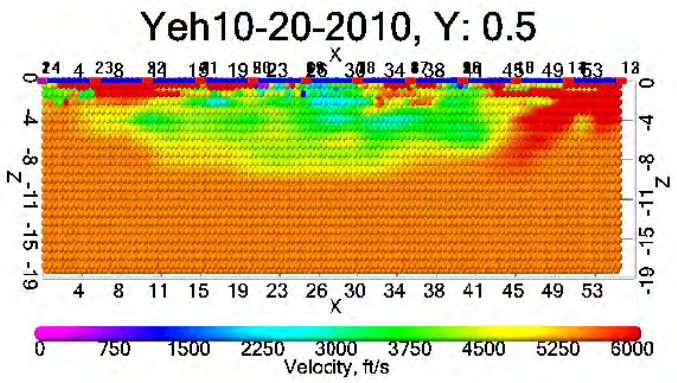
Boring YA-6
Depth 25-30 ft

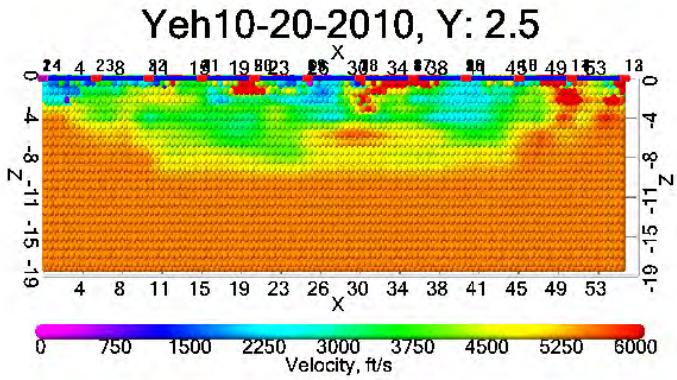
Boring YA-7
Depth 10-20 ft

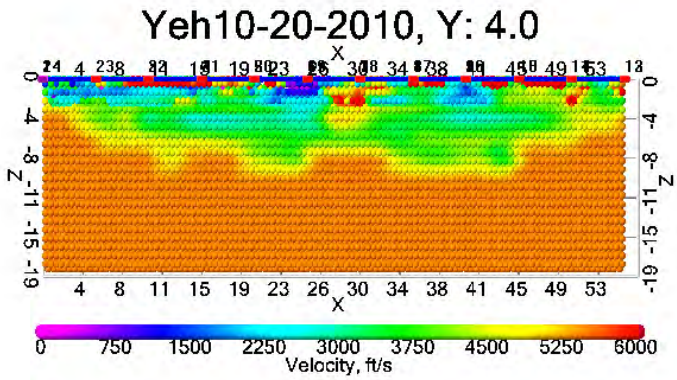
Boring YA-7
Depth 20-27 ft

APPENDIX H- Tomographs

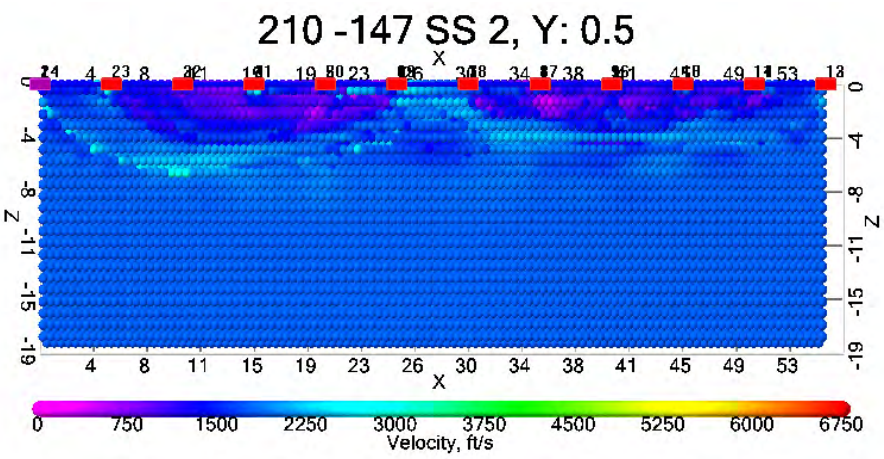
Geophysical Survey 1

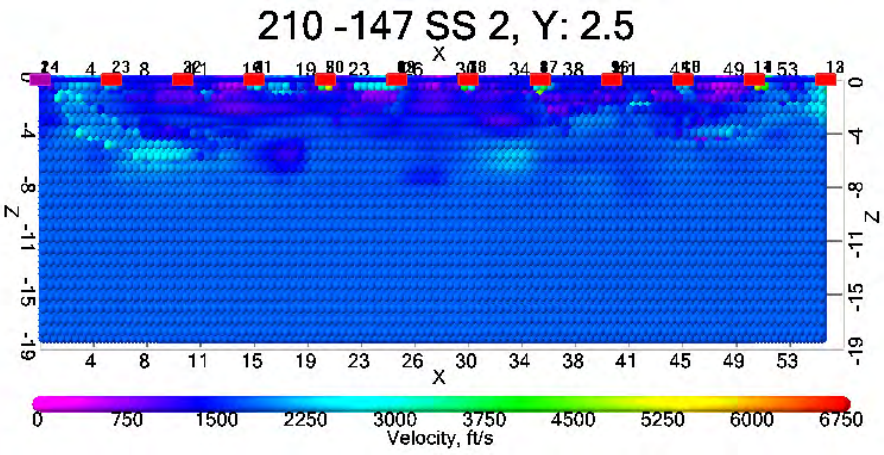




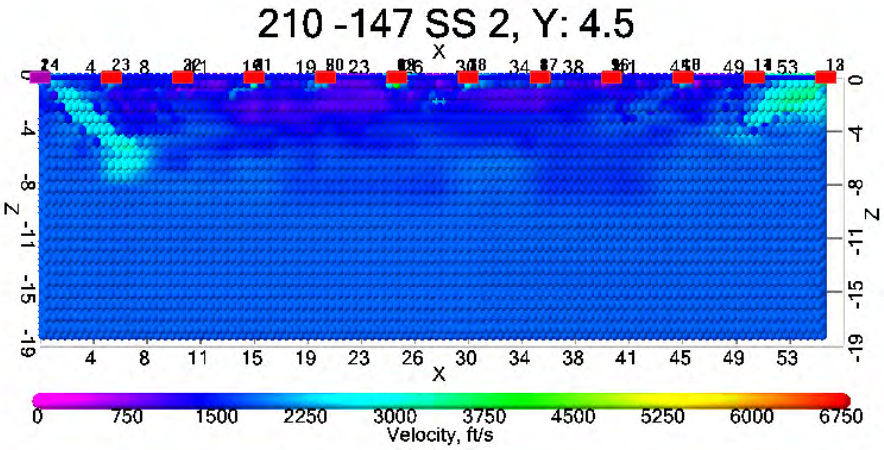


Geophysical Survey 2





Geophysical Survey 2



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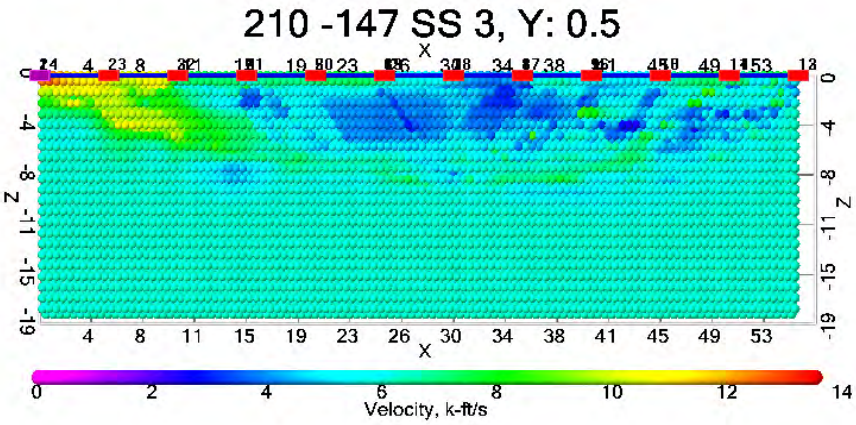
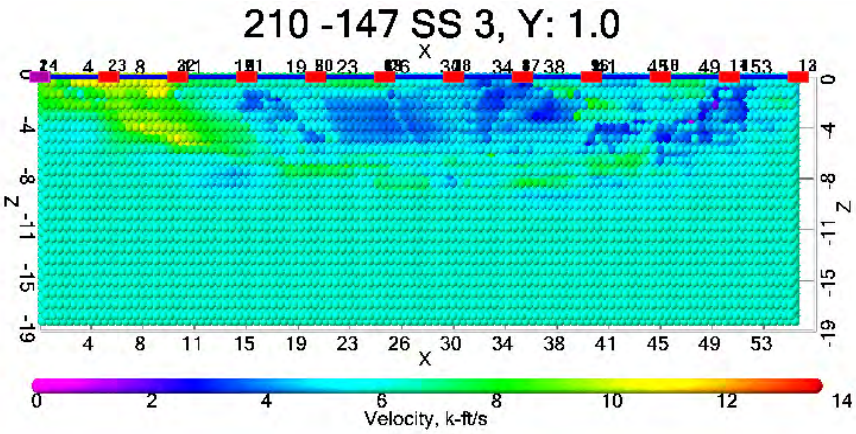
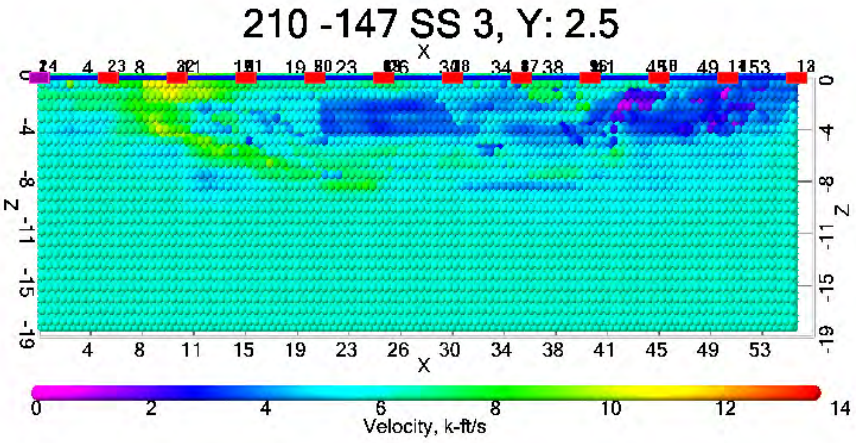






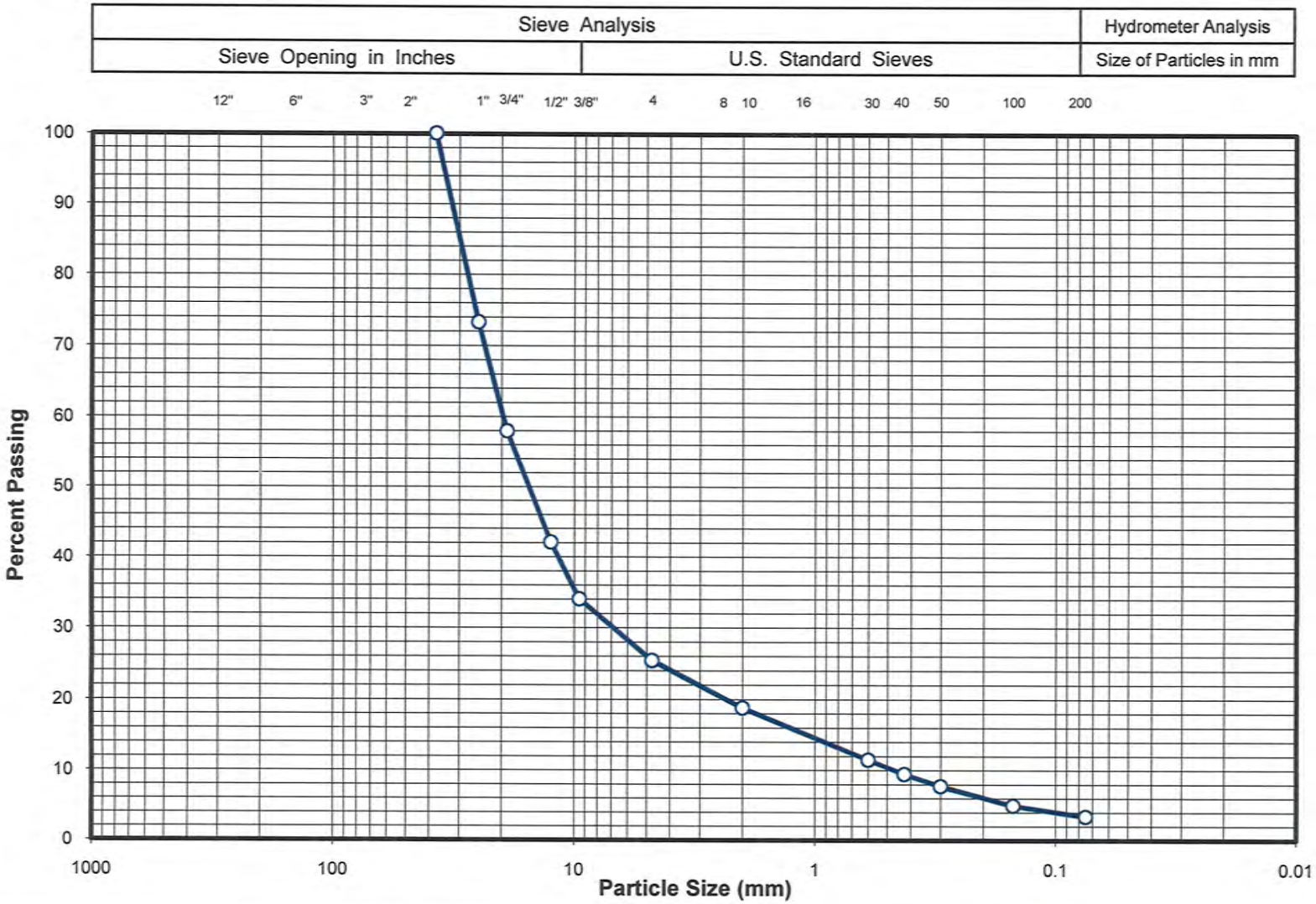
APPENDIX I- Laboratory Test Results

YEH & ASSOCIATES, INC

Summary of Laboratory Test Results

Project No: 210 - 147 Project Name: SH 9 Frisco to Breckenridge Date: 11/10/2010

Sample Location			Natural Moisture Content (%)	Natural Dry Density (pcf)	Gradation			Atterberg			pH	Water Soluble Sulfate %	% Swell (+) / Consoli-dation (-)	Unconf. Comp. Strength (psf)	R-VALUE	CLASSIFICATION	
Boring NO.	Depth (ft)	Sample Type			Gravel > #4 (%)	Sand (%)	Fines < #200 (%)	LL	PL	PI						AASHTO	USCS
YA - 1	15-16.5	SS	5.3	–	75	22	3	NV	NP	NP	–	–	–	–	–	A-1-a (0)	GP
YA - 1	23-25	Grab	31.8	–	17	41	42	34	17	17	–	–	–	–	–	A - 6 (3)	SC
YA - 2	10-11	CA	9.2	128.1	47	40	13	NV	NP	NP	8.2	0.003	–	–	–	A-1-b (0)	GM
YA - 2	23-24	Core	–	–	–	–	–	–	–	–	8.4	0.008	–	–	–	–	–
YA - 3	10-11	Core	11.6	–	61	24	15	38	12	26	–	–	–	–	–	A-2-6 (0)	GC
YA - 3	15-17	Core	5.5	–	69	28	3	NV	NP	NP	8.0	0.001	–	–	–	A-1-a (0)	GP
YA - 6	4	Core	10.7	–	36	43	21	–	–	–	–	–	–	–	–	A-1-b (0)	SM
YA - 6	12	Core	2.5	–	30	46	24	–	–	–	–	–	–	–	–	A-1-b (0)	SM
YA - 7	5-6.5	SS	7.3	–	39	47	14	NV	NP	NP	8.1	0.002	–	–	–	A-1-a (0)	SM
YA - 8	15-16.5	SS	10.9	–	7	61	32	30	9	21	8.3	<0.001	–	–	–	A-2-6 (1)	SC
YA - 8	25-26.5	SS	7.9	–	49	39	12	NV	NP	NP	–	–	–	–	–	A-1-a (0)	GM



Sieve Size	% Passing
3"	-
2 ½"	-
2"	-
1 ½"	100
1"	73
¾ "	58
½"	42
⅜"	34
#4	25
#10	19
#40	9
#200	3

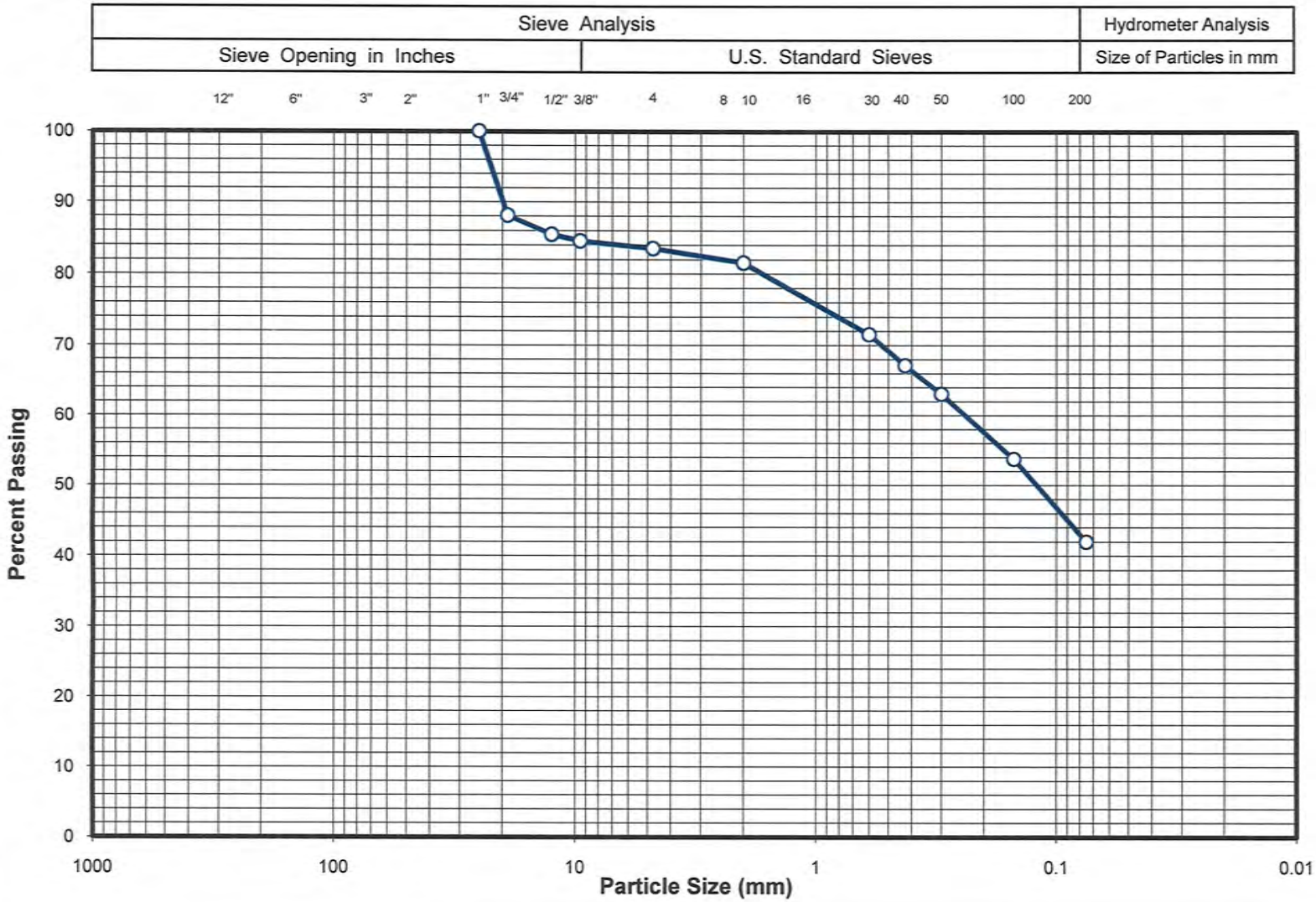
Gravel (%)	75	LL	NV	Project Name:	SH 9 Frisco to Breckenridge
Sand (%)	22	PL	NP	Sample ID:	YA - 1
Fines (%)	3	PI	NP	Sample Depth (ft.):	15 - 16.5
Sample Description:	GP / A - 1 -a (0)				

Yeh & Associates, Inc.
Geotechnical Engineering Consultants

SIEVE ANALYSIS

Drawn By:	MA	Project No.:	210 - 147
Checked By:	Howard		
Date:	10/25/10	Figure No.:	-

Revised 04/27/2004



Sieve Size	% Passing
3"	-
2 ½"	-
2"	-
1 ½"	-
1"	100
¾ "	88
½"	85
⅜"	84
#4	83
#10	81
#40	67
#200	42

Gravel (%)	17	LL	34	Project Name:	SH 9 Frisco to Breckenridge
Sand (%)	41	PL	17	Sample ID:	YA - 1
Fines (%)	42	PI	17	Sample Depth (ft.):	23 - 25
Sample Description:	SC / A - 6 (3)				

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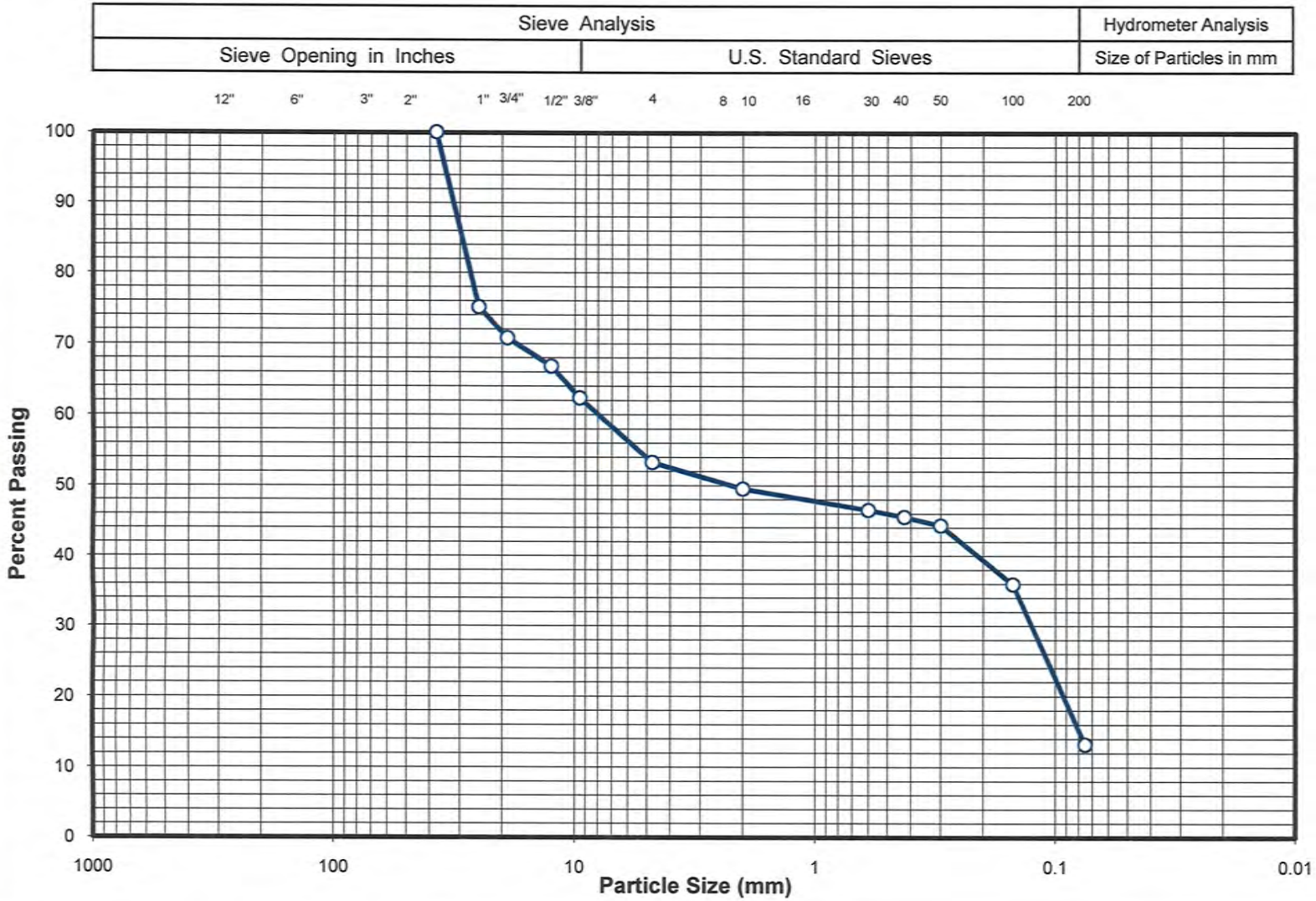
Geotechnical Engineering Consultants

SIEVE ANALYSIS

Drawn By:	MA	Project No.:	210 - 147
Checked By:	Howard		
Date:	10/25/10	Figure No.:	-

Revised 04/27/2004



Sieve Size	% Passing
3"	-
2 ½"	-
2"	-
1 ½"	100
1"	75
¾ "	71
½"	67
⅜"	62
#4	53
#10	49
#40	46
#200	13

Gravel (%)	47	LL	NV	Project Name:	SH 9 Frisco to Breckenridge
Sand (%)	40	PL	NP	Sample ID:	YA - 2
Fines (%)	13	PI	NP	Sample Depth (ft.):	10-11
Sample Description:	GM / A - 1 - b (0)				

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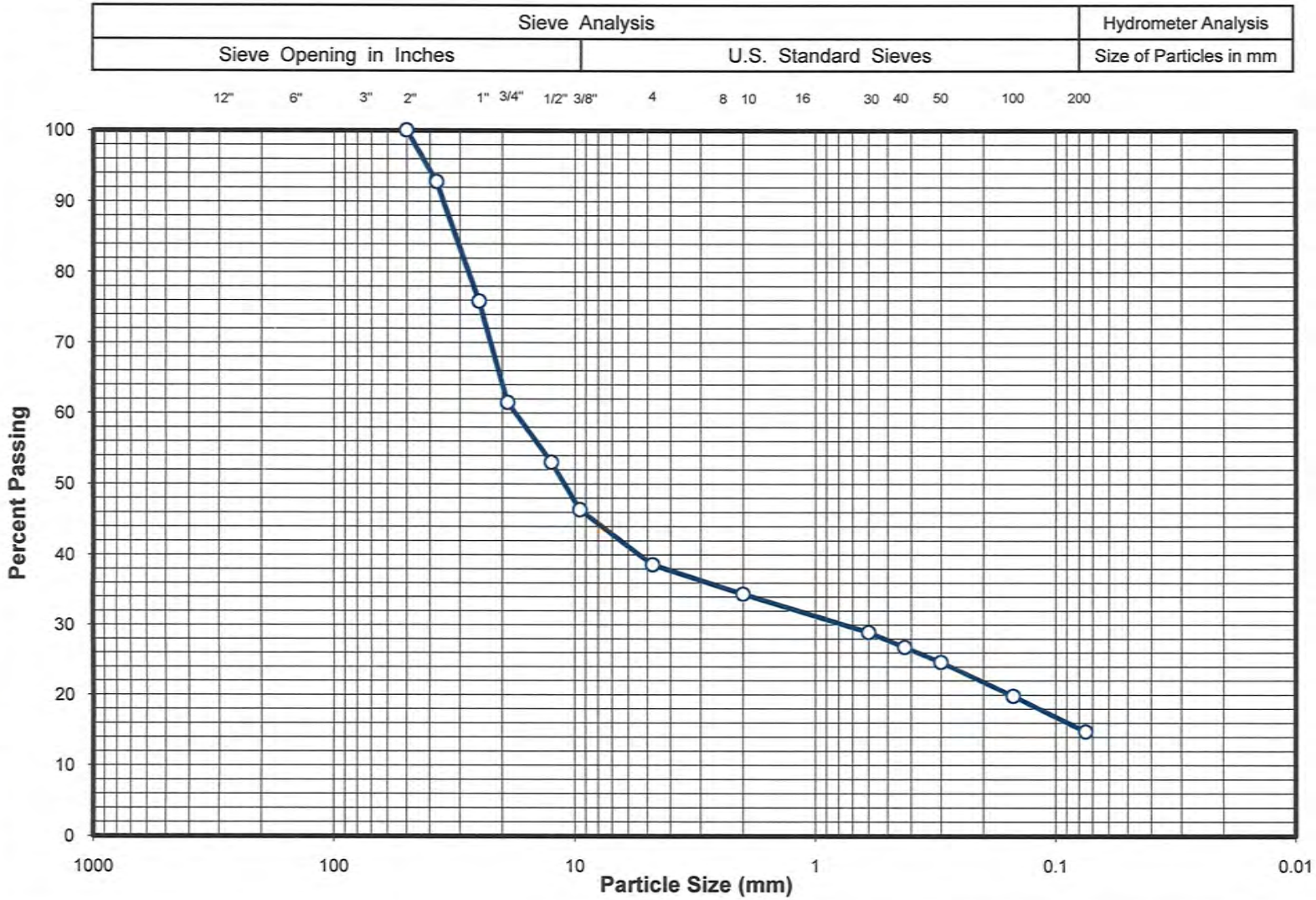
SIEVE ANALYSIS

Drawn By:	MA	Project No.:	210 - 147
Checked By:	Howard		
Date:	10/25/10	Figure No.:	-

Revised 04/27/2004



Sieve Size	% Passing
3"	-
2 ½"	-
2"	100
1 ½"	93
1"	76
¾ "	61
½"	53
⅜"	46
#4	39
#10	34
#40	27
#200	15

Gravel (%)	61	LL	38	Project Name:	SH 9 Frisco to Breckenridge
Sand (%)	24	PL	12	Sample ID:	YA - 3
Fines (%)	15	PI	26	Sample Depth (ft.):	10 - 11
Sample Description:	GC / A - 2 - 6 (0)				

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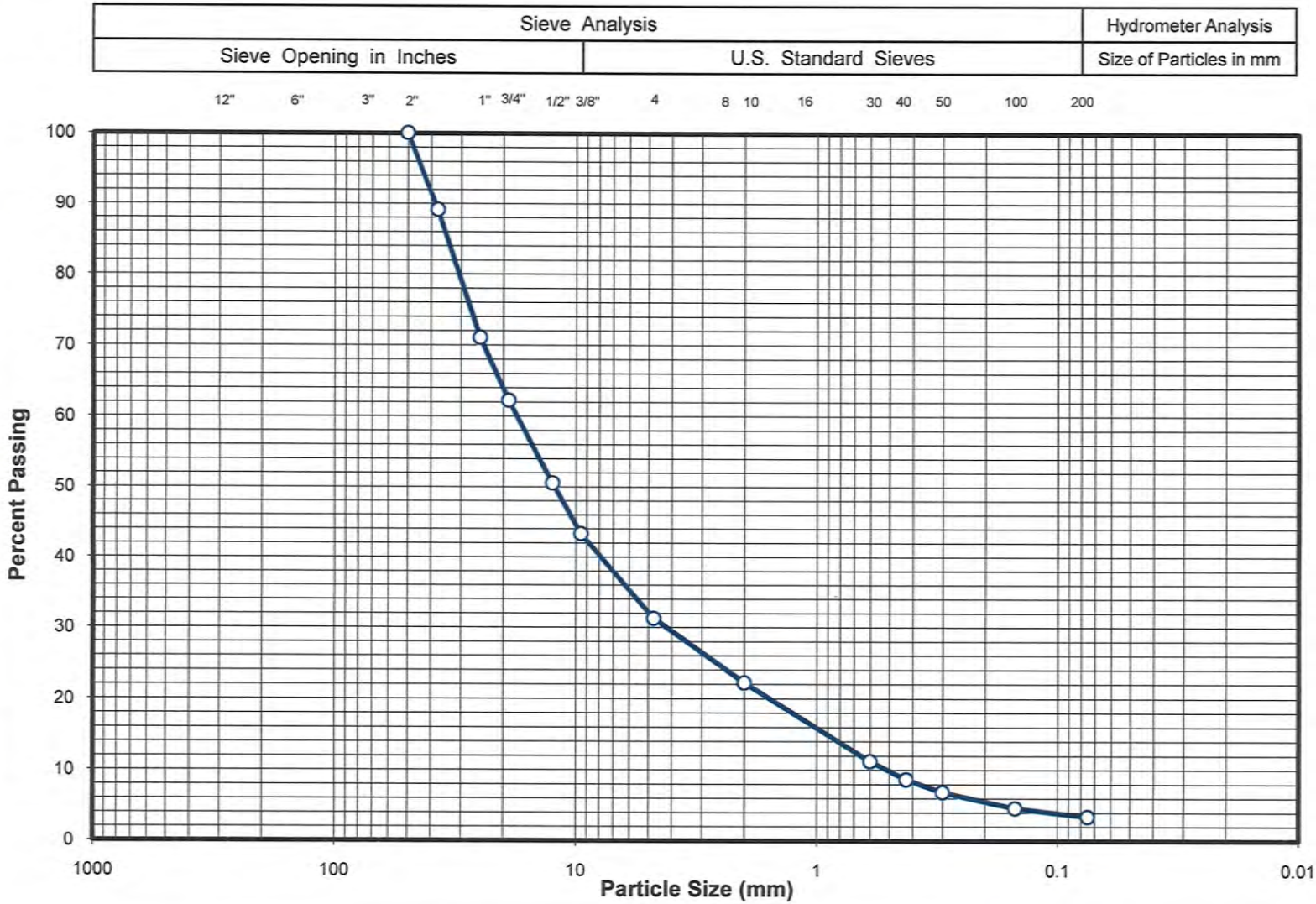
Geotechnical Engineering Consultants

SIEVE ANALYSIS

Drawn By:	MA	Project No.:	210 - 147
Checked By:	Howard		
Date:	10/25/10	Figure No.:	-

Revised 04/27/2004



Sieve Size	% Passing
3"	-
2 ½"	-
2"	100
1 ½"	89
1"	71
¾ "	62
½"	50
⅜"	43
#4	31
#10	22
#40	9
#200	3

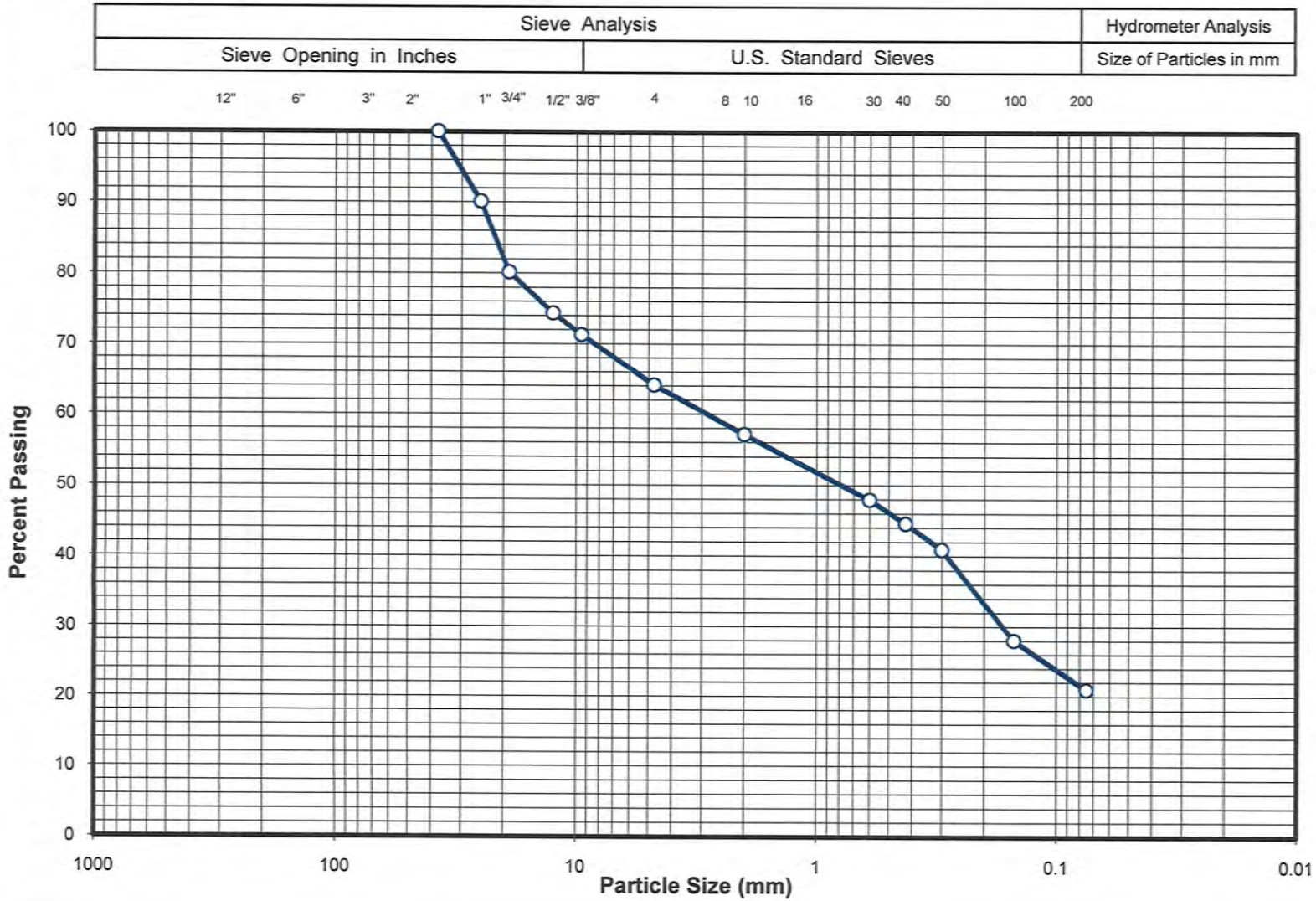
Gravel (%)	69	LL	NV	Project Name:	SH 9 Frisco to Breckenridge
Sand (%)	28	PL	NP	Sample ID:	YA - 3
Fines (%)	3	PI	NP	Sample Depth (ft.):	15 - 17
Sample Description:	GP / A - 1 - a (0)				

Yeh & Associates, Inc.
Geotechnical Engineering Consultants

SIEVE ANALYSIS

Drawn By:	MA	Project No.:	210 - 147
Checked By:	Howard		
Date:	10/25/10	Figure No.:	-

Revised 04/27/2004



Sieve Size	% Passing
3"	-
2 ½"	-
2"	-
1 ½"	100
1"	90
¾ "	80
½"	74
⅜"	71
#4	64
#10	57
#40	45
#200	21

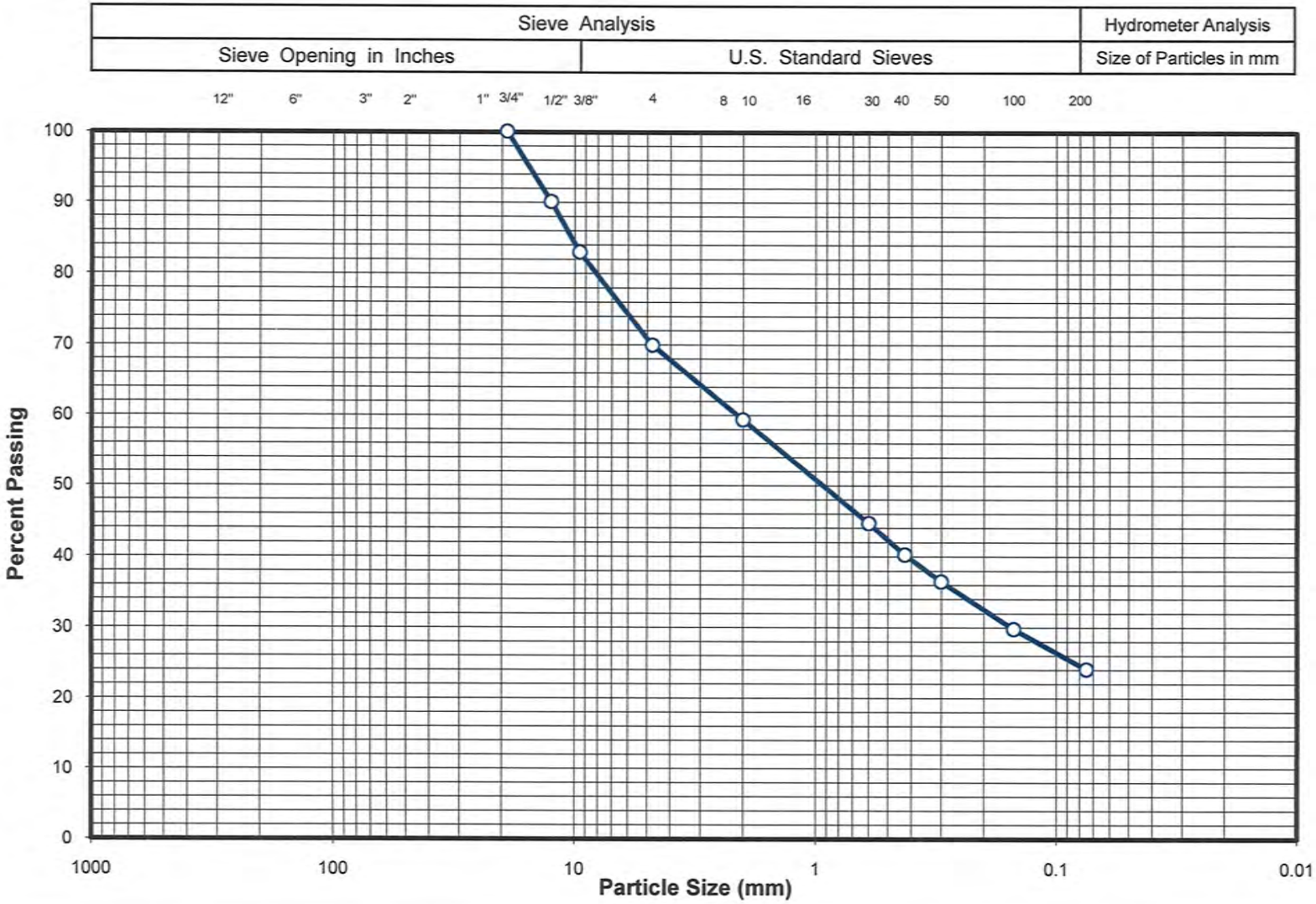
Gravel (%)	36	LL	NV	Project Name:	SH 9 Frisco to Breckenridge
Sand (%)	43	PL	NP	Sample ID:	YA - 6
Fines (%)	21	PI	NP	Sample Depth (ft.):	4
Sample Description:	SM / A-1-b (0)				

Yeh & Associates, Inc.
Geotechnical Engineering Consultants

SIEVE ANALYSIS

Drawn By:	MA	Project No.:	210 - 147
Checked By:	Howard		
Date:	11/08/10	Figure No.:	-

Revised 04/27/2004



Sieve Size	% Passing
3"	-
2 ½"	-
2"	-
1 ½"	-
1"	-
¾ "	100
½"	90
⅜"	83
#4	70
#10	59
#40	40
#200	24

Gravel (%)	30	LL	NV	Project Name:	SH 9 Frisco to Breckenridge
Sand (%)	46	PL	NP	Sample ID:	YA - 6
Fines (%)	24	PI	NP	Sample Depth (ft.):	12
Sample Description:	SM / A-1-b (0)				

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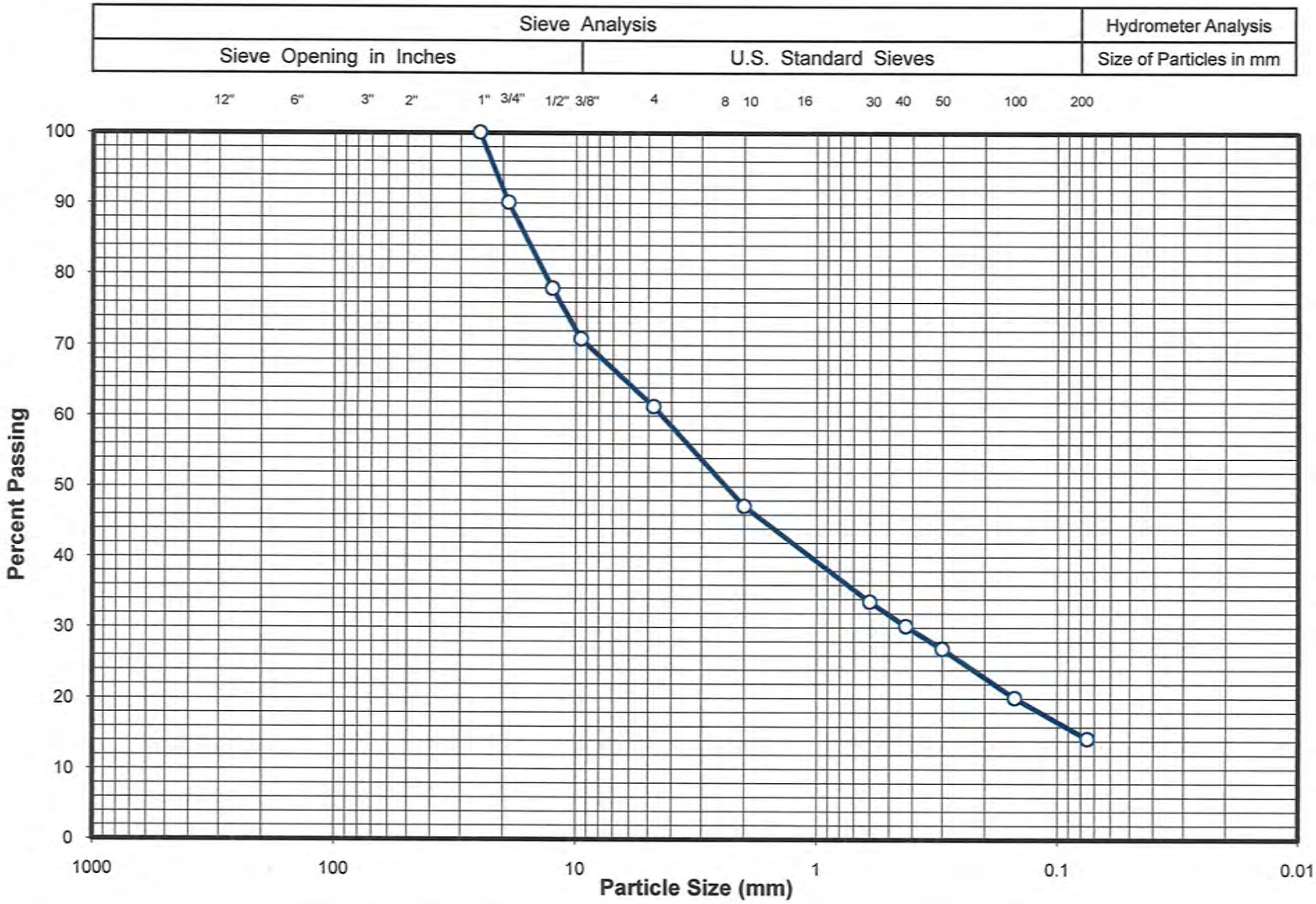
SIEVE ANALYSIS

Drawn By:	MA	Project No.:	210 - 147
Checked By:	Howard		
Date:	11/08/10	Figure No.:	-

Revised 04/27/2004



Sieve Size	% Passing
3"	-
2 ½"	-
2"	-
1 ½"	-
1"	100
¾ "	90
½"	78
⅜"	71
#4	61
#10	47
#40	30
#200	14

Gravel (%)	39	LL	NV	Project Name:	SH 9 Frisco to Breckenridge
Sand (%)	47	PL	NP	Sample ID:	YA - 7
Fines (%)	14	PI	NP	Sample Depth (ft.):	5 - 6.5
Sample Description:	SM / A - 1 - a (0)				

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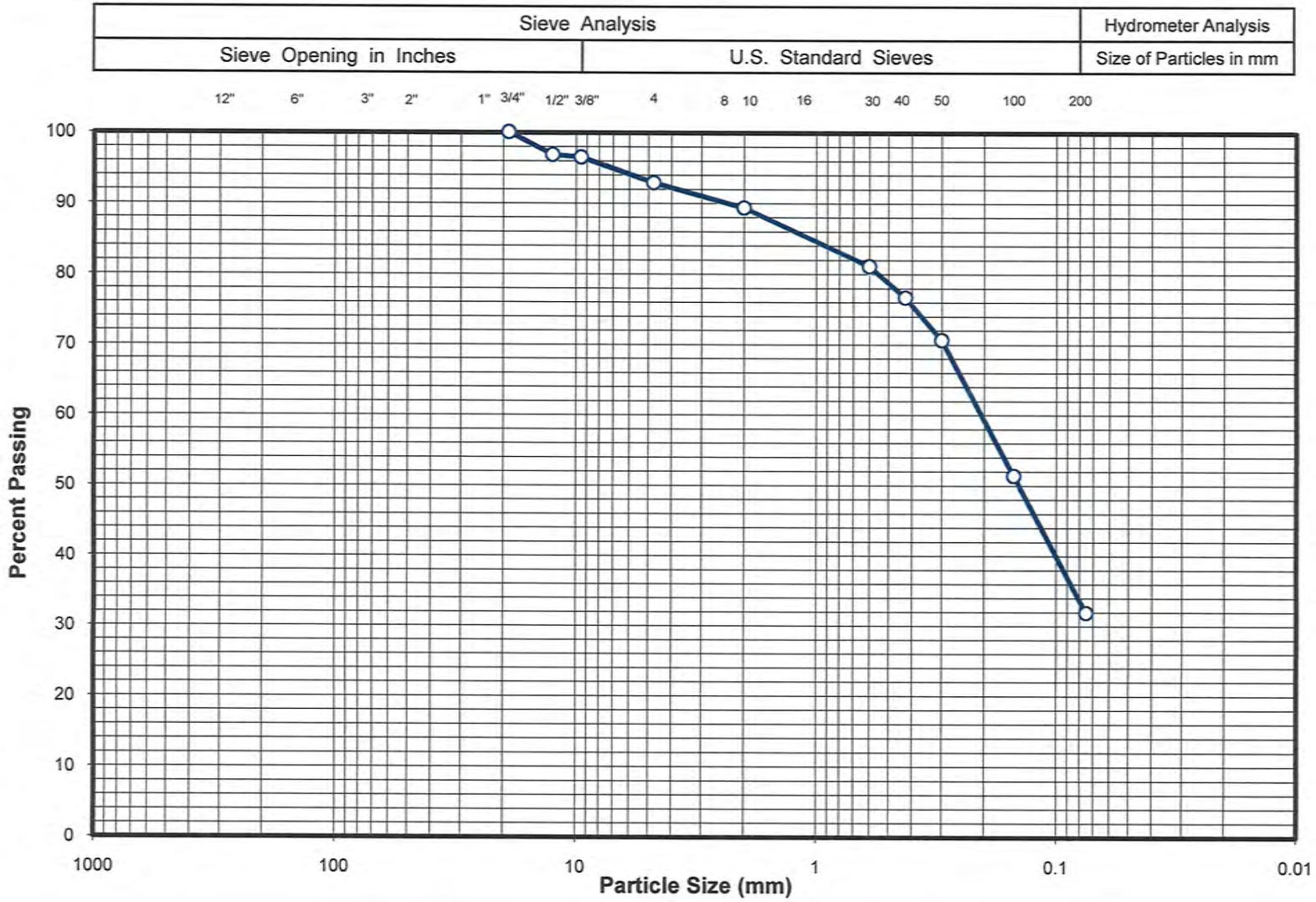
SIEVE ANALYSIS

Drawn By:	MA	Project No.:	210 - 147
Checked By:	Howard		
Date:	10/25/10	Figure No.:	-

Revised 04/27/2004



Sieve Size	% Passing
3"	-
2 ½"	-
2"	-
1 ½"	-
1"	-
¾ "	100
½"	97
⅜"	96
#4	93
#10	89
#40	77
#200	32

Gravel (%)	7	LL	30	Project Name:	SH 9 Frisco to Breckenridge
Sand (%)	61	PL	9	Sample ID:	YA - 8
Fines (%)	32	PI	21	Sample Depth (ft.):	15 - 16.5
Sample Description:	SC / A - 2 - 6 (1)				

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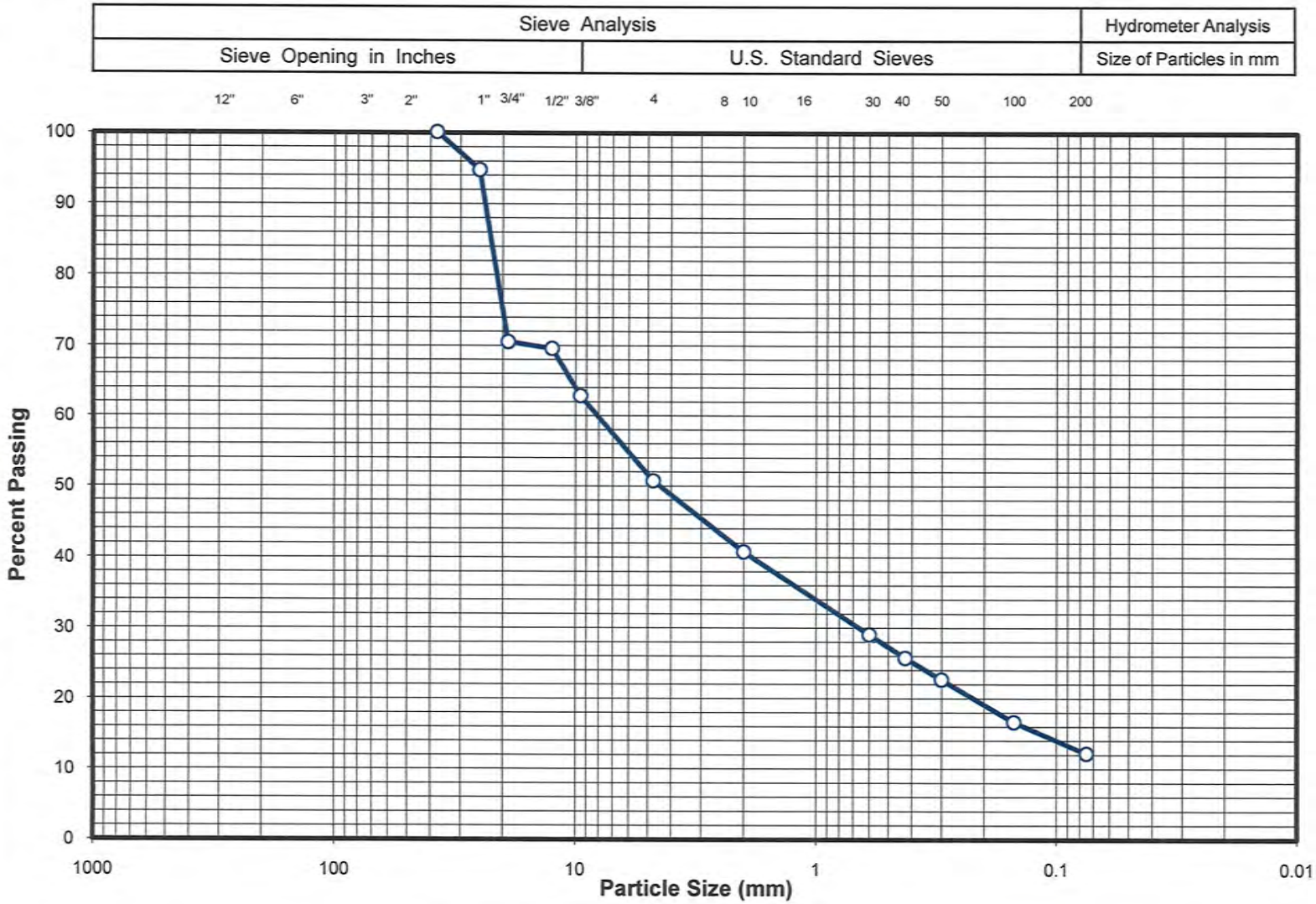
SIEVE ANALYSIS

Drawn By:	MA	Project No.:	210 - 147
Checked By:	Howard		
Date:	10/25/10	Figure No.:	-

Revised 04/27/2004



Sieve Size	% Passing
3"	-
2 ½"	-
2"	-
1 ½"	100
1"	95
¾ "	70
½"	69
⅜"	63
#4	51
#10	41
#40	26
#200	12

Gravel (%)	49	LL	NV	Project Name:	SH 9 Frisco to Breckenridge
Sand (%)	39	PL	NP	Sample ID:	YA - 8
Fines (%)	12	PI	NP	Sample Depth (ft.):	25 - 26.5
Sample Description:	GM / A - 1 -a (0)				

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SIEVE ANALYSIS

Drawn By:	MA	Project No.:	210 - 147
Checked By:	Howard		
Date:	10/25/10	Figure No.:	-

Revised 04/27/2004