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ROCK SLOPE ENGINEERING DESIGN
Sheep Mountain Rock Cut
Km 1705 Alaska Highway #1
March 2006

CREATING AND DELIVER

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Government of the Yukon

ROCK SLOPE ENGINEERING DESIGN
SHEEP MOUNTAIN ROCK CUT
KM 1705, ALASKA HIGHWAY
YUKON

1200128

March 2006

Transportation Engineering
Geotechnical Services

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1.0 INTRODUCTION

1.1 GENERAL

The Government of Yukon (YTG) is planning to realign a section of the Alaska Highway at Sheep Mountain, located approximately 280 km west of Whitehorse, YT. This will require a 550 m long rock cut between Station 1704+600 and Station 1705+150 that will be more than 50 m high in places. Rockfall from the existing cut slope is presently a concern and it is intended that the new construction will reduce the rockfall hazard. The project location is shown on Figure 1 and the proposed new alignment is presented in Figure 2.

EBA Engineering Consultants Ltd (EBA) were retained by YTG to carry out geotechnical mapping of the existing rock cut and provide design recommendations for the new rock cut.

1.2 SITE CONSTRAINTS

The project site is situated within Kluane National Park and the road follows the shore of Kluane Lake (see Photo 1, Appendix A). It is understood that it is unacceptable to have fly rock or construction debris enter the lake. Explosives must have non-toxic residues as the blast rock may be used as embankment fill in or near Kluane Lake.

The Climate at the site is dry and windy, which results in sparse snow cover during the winter. As a result the area is frequented by Dhal Sheep, who graze on the slopes above the rock cut throughout the winter. Kluane Lake also provides the sheep with a source of water during summer months. The steep gradients of the existing rock cut and slope immediately above provide the sheep with habitat that is safe from predators and close to the lake. Consequently, rockfall mitigation measures such as rockfall nets; slope mesh and guard rails that would disrupt the passage of sheep are not acceptable. Likewise a near vertical and/or high rock cut that might restrict sheep passage should be avoided if possible.

The construction schedule for the project will be influenced by potential work restrictions related to the sheep during the fall rutting season and spring calving season. In addition, heavy tourist traffic during summer months will limit allowable road closures.

1.3 REPORT CONVENTIONS AND QUALIFICATIONS

Preliminary alignment plans prepared by YTG dated November 2004 were used by EBA for reference of chainage locations and design purposes. Selected cross sections from the preliminary design alignment are included in Figures 4 and 5 for reference, although these do not accurately reflect the final cut slope designs. Any subsequent alignment changes may result in subtle shifts in chainages from those used in this report. For future reference in this event, the southern limit of the rock cut is located at station 1704 + 590 and the northern limit of bedrock where it is in contact with overburden materials is located at approximately station 1705+075.

The highway is oriented approximately southwest-northeast through the subject area, however, for purposes of this report "North" is defined as the direction of the road leading to Alaska, and "South" is defined as the direction of the road leading to Haines Junction.

All bedrock structural orientation data in this report is presented using the dip and dip direction convention unless noted otherwise. Further, structural orientation data has been corrected for magnetic declination, which is 32 degrees east from true north at this site.

2.0 FIELD INVESTIGATION

Mr. Jack Dennett, P.Geo. (BC), Senior Geologist with EBA, carried out site investigations on November 30, 2004. Mr. Dennett was accompanied by Mr. Chadwyck Cowan, P.Eng. of EBA and together they completed the geological and structural mapping in the vicinity of the proposed rock cut.

Subsequently, Mr. Scott Sylte, P.Eng. (BC), EBA's senior rockwork engineer, and Mr. Richard Trimble, P.Eng., Manager of EBA's Yukon Operations, met with representatives of the Yukon Transportation Division on December 1, 2004, and conducted a site reconnaissance on December 2, 2004.

During the site investigations, traverses were made along the ditch line of the existing rock cut as well as along the original Alaska Highway trail above the cut. As the traverses were confined to established roads and trails, there was not judged to be a significant hazard of rocks being inadvertently dislodged and posing a hazard to motorists, hence traffic control was limited to un-manned work ahead and speed advisory signs.

Geological mapping of the existing cut slope was completed to delineate rock types, geologic contacts and major structures such as faults, and shears. Geological mapping of the existing highway rock cut is presented on an elevation view of the cut in Figure 3. Structural rock mapping was completed along the existing rock cut to characterize structural features in the rock mass within each geologically distinct area of the cut. Structural mapping data is presented in Appendix B. These measurements provide a small, but representative sampling of the more important rock mass characteristics from a rock slope design perspective.

The original Alaska Highway trail provided access to the area above the crest of the proposed rock cut. The trail is located along the break in slope, with the slope below it (towards the highway) being much steeper than the slope above. The slope between the trail and the existing rock cut ranges from 60% to 70% grade and is mostly covered by unstable, unconsolidated scree (see Photo 2). The thickness of the scree is variable, but is likely less than 3 m on average as bedrock is exposed in some areas.

The existing cut slope was examined closely to identify areas of past instability such as wedge or other failures as these provide a direct indication of how the new cut may perform. Evidence of rockfalls originating from the scree slope above the existing highway was noted, along with localized areas of past or potential instability in the existing rock cut.

EBA's traverse chainage measurements along the existing highway were made with a tape measure working from pre-existing survey stakes located along the existing ditch. Measurements have an accuracy of about +/- 2m.

3.0 GEOTECHNICAL CONDITIONS

3.1 GEOLOGIC SETTING

The proposed rock cut is located above the western shore of Kluane Lake in the Shakwak Valley, a large trench-like valley that flanks the eastern edge of the St. Elias Mountains. The Shakwak Trench is a result of the Denali Fault System that controls the major northeast-southwest physiographic and structural trends in the region.

The study area lies within the southern fringe of the permafrost zone. There is a low probability of permafrost underlying the study area, which consists of southeast-facing, low elevation slopes adjacent to Kluane Lake.

A catastrophic landslide on the northeast flank of Sheep Mountain referred to as the Sheep Mountain Landslide is situated about 1 km north of the rock cut. This slide deposited thick blocky rubble extending into Kluane Lake. Geologic investigations (Clague, 1981) suggest that the slide occurred in at least two events, the first between 1200 and 1950 years B.P. and the second between 500 and 1200 years B.P.

Approximately 500 m south of the rock cut, the highway crosses the fan of debris flow deposits at the south end of Kluane Lake. A debris flow at this location apparently blocked the highway in 1967. A rock glacier is situated about 800 m upslope from the rock cut (Clague, 1981). Talus and fragmental rockfall debris are also present on the slopes above the rock cut but these do not appear to have a distinct source area. Immediately upslope of the rock cut these deposits consist of rubble colluvium.

The project area is located in the Wrangellia terrane of the Insular Tectonic Belt. It is an area of complex structural geology, as it is located near the boundaries of the Alexander terrane (within the Insular belt) to the south west, and the boundary of the Coast Tectonic Belt to the north east (boundary occurs along the Denali Fault). The area is known for its high seismicity.

Bedrock in the study area is composed of faulted volcanics, limestone and other sedimentary units complexed by the Denali fault system as it transects the mountains. These units are intruded by younger granitic rocks north of the Slims River. Lithology includes shale, basaltic and andesitic flows, breccia, thin-bedded bioclastic limestone, conglomerate, phyllite, thin-bedded argillite, siltstone, and/or greywacke.

3.2 ROCK CUT GEOMETRY AND TERRAIN CHARACTERISTICS

The proposed rock cut is located on the west (upslope) side of the existing road alignment and will be substantially higher than the existing cut. South of approximately Sta. 1704+600 the new alignment is coincident with the existing road or will be on a fill embankment (see Figure 2). From approximately 1704+600 to 1704+820 the new alignment parallels the west side of the existing alignment and will require a rock cut up to about 35 m high (see cross sections 1 to 3 on Figure 4). Between 1704+820 and 1705+020, the new alignment cuts through a bedrock ridge that is skirted by the existing highway, and will form a through cut with a maximum cut height of approximately 50 m on the west side (see Figures 4 and 5). From 1705+020 to 1705+150, the new alignment rejoins the existing alignment and the cut height diminishes from about 35 m down to about 10 m or less (see Figure 5).

The existing rock cut is comprised of three distinct lithologies, all of which are intruded by mafic dikes and contain several faulted or sheared zones (see Figure 3). Thinly bedded argillite and phyllite (metasediments) are found from the start of the rock cut at 1704+590 to 1704+720 where they are in contact with a strong quartzite unit. The quartzite unit continues until approximately 1704+820 where a mafic dike occurs along the contact with a volcanic unit. The volcanics are present from 1704+820 to 1705+025 where the rock changes again to the argillite unit. There is a small inclusion of limestone within the mafic volcanics from 1704+950 to 1704+965.

Very little accumulated rockfall debris was present along the original Alaska Highway grade situated about 60 m above the existing highway, suggesting that the gentler slopes above the original grade have not been a significant source of rockfall.

There is a remnant bench or pioneer construction access road along the crest of the existing rock cut between about 1704+500 to 1704+800 (see Photo 3). In places, this has been completely covered with fragmental rockfall debris originating from the steep rubble covered colluvial slopes situated between the existing rock cut and the original Alaska Highway grade. It is possible that little if any of this rockfall material has thus far reached the existing highway, since most appears to have been retained by the bench. However, in areas where the bench is now full, future rockfall could reach the existing highway grade. In view of the large accumulation of rockfall debris on the remnant bench, the steep colluvial slopes must be considered as a significant rockfall source area in the design of the new rock cut.

The ditch along the existing rock cut is typically up to about 2 m wide but narrows to about 1 m through the central part of the cut, which is upwards of 20 m high. As such, the existing ditch effectiveness is judged to be limited and EBA understands that some rockfall from the existing cut does reach the traveled road surface. The existing cut slope angle within the highest central portion of the existing rock cut ranges from 77° to 82°, with one short area near 1704+800 being up to 88°.

Towards the north and south ends of existing the cut in the thinly bedded argillites, individual rockfall particles are likely to be small (typically less than 0.05 m^3). However, in the more massive quartzite and volcanic rocks, there is evidence of past block or planar failures which may have generated rockfall volumes of several cubic metres with individual fragments of 1 m^3 or larger.

Traces of small diameter (~ 25 mm) trim blast holes spaced approximately 0.8 m apart were evident along the brow of the cut at several locations between approximately 1704+780 and 1704+925 (see Photo 4). These were drilled with hand equipment, possibly during the original construction to form a pioneer bench for track-mounted drills, or perhaps at a later date to trim off larger potential rockfall masses.

3.3 DELINEATION OF STRUCTURAL DOMAINS

For rock slope design, the alignment is first subdivided into "structural domains" which are areas that have unique rock mass structural characteristics. Structural domain boundaries often coincide with changes in rock type or regional faults, but may also be indistinct and difficult to identify. Once structural domain boundaries are identified, all mapping data within each structural domain is lumped together to develop a statistical representation of the rock mass characteristics for each structural domain.

The structural rock mapping data was compiled and plotted using DIPS (stereonet software) to determine the prevalent joint set orientations and characteristics. Stereonet plots presenting this data are included in Appendix C. Based on consideration of this data along with the geologic mapping data, the slope was subdivided into three structural domains representing areas with different rock mass characteristics.

Structural domains were identified for this rock cut based primarily on differences in rock type, which appears to be the dominant control on joint orientations, rock strength and weathering. These include the argillite (Structural Domain 1), the quartzite (Structural Domain 2) and the volcanics (Structural Domain 3). Several mafic intrusive dikes are present within all of the structural domains, and a small pod of limestone is present within the volcanics at Sta. 1704 + 950. While these may warrant specific consideration or treatment during construction, they are of limited extent and are not considered separately for design purposes.

The argillite has well defined bedding planes and occurs along the existing rock cut from Sta. 1704 + 590 to Sta. 1704 + 720 and again from Sta. 1705 + 025 to Sta. 1705 + 100. The quartzite has poorly defined foliation and occurs from Sta. 1704 + 720 to Sta. 1704 + 820. The volcanics are a massive unit that occurs from Sta. 1704 + 820 to Sta. 1705 + 025. Structural discontinuities including dominant joint sets and shear zones for each structural domain are described in the following sections.

3.4 TABULATION OF JOINT SET CHARACTERISTICS

The following summarizes the characteristics of each joint set for each structural domain considered during design of the new rock cut. Each joint set is described in terms of the mean orientation and orientation range, as dip and dip direction. Joint orientation, persistence, spacing and other attributes affect the likelihood and potential size of rockfalls, and provide a basis for determining which joint sets are most critical. A summary of all joint sets for all domains is in provided in Table 1.

3.4.1 Structural Domain 1: Argillite

The argillite unit is a highly weathered weak rock that has conspicuous, closely spaced bedding planes dipping moderately to steeply towards the southwest. The argillite contains an abundance of shear zones and mafic dyke intrusions. This is the structural domain of Design Sectors 1 and 4, and contains seven notable joint sets, derived from 50 individual measurements. Photos 5 and 6 show representative exposures of the argillite in Design Sectors 1 and 4 respectively

Joint Set 1 (J1):

Peak at 69/219, range from 58 to 82 (dip) and 194 to 243 (dip direction). Joint Set 1 is parallel to bedding and is the most prominent joint set (32% of measurements in Domain 1). This joint set is present in all Structural Domains and is also denoted as J1 in the quartzite and volcanic units. Joints are persistent with a moderate spacing and have curved, rough surfaces.

Joint Set 2 (J2):

Peak at 28/326 (gentle dip to northwest), range from 25 to 36 (dip) and 298 to 346 (dip direction). This joint set is infrequent (6% of measurements) but is also found in the quartzite unit (denoted as J2).

Joint Set 3 (J3):

Peak at 45/169 (moderate dip to south), range from 34 to 56 (dip) and 149 to 179 (dip direction). Joint Set 3 is relatively frequent (16% of measurements) and is also found in the volcanic unit (denoted as J3) where it appears to have formed several planar failure planes. Joints generally have a medium persistence although occasional very widely spaced joints are very persistent.

Joint Set 4 (J4):

Peak at 83/126 (steep dip to southeast), range from 76 to 90 (dip) and 113 to 137 (dip direction). This joint set is infrequent (10% of measurements) and is also found in the quartzite unit (denoted as J4). Joints are wide to very widely spaced and have medium persistence.

Joint Set 5 (J5):

Peak at 74/344 (steep dip to the north), range from 68 to 82 (dip) and 337 to 347 (dip direction). This joint set is infrequent (6% of measurements).

Joint Set 6 (J6):

Peak at 58/305 (moderate to steep dip to the northwest), range from 48 to 72 (dip) and 297 to 320 (dip direction). This joint set is infrequent (8% of measurements).

Joint Set 7 (J7):

Peak at 71/158 (moderate to steep dip to the south), range from 66 to 78 (dip) and 155 to 163 (dip direction). This joint set is infrequent (6% of measurements). Joints are wide to very widely spaced and have medium persistence.

3.4.2 Structural Domain 2: Quartzite

The quartzite unit is a very strong and slightly weathered rock. Bedding planes are often inconspicuous and have a steep dip towards the southwest. The rock contains an abundance of shear zones and mafic dyke intrusions. This structural domain contains six notable joint sets, derived from 20 individual measurements. Photos 4 and 7 show representative exposures of the quartzite exposed in Design Sector 2.

Joint Set 1 (J1):

Peak at 69/233 (moderate to steep dip to the southwest), range from 62 to 73 (dip) and 217 to 242 (dip direction). Joint Set 1 is parallel to bedding and is the most prominent joint set representing 30% of the measurements in this domain. This joint set is present in all Structural Domains and is also denoted as J1 in the argillite and volcanic units. Joints are widely spaced, highly persistent, and have undulating smooth surfaces.

Joint Set 2 (J2):

Oriented at 20/311 (gentle dip to the northwest). This joint set is infrequent (5% of measurements) and is also found in the argillite unit (denoted as J2). Joints have low persistence and rough, undulating surfaces.

Joint Set 3 (J3):

Peak at 28/119 (shallow dip to the southeast), range from 24 to 34 (dip) and 103 to 141 (dip direction). This joint set is infrequent (10% of measurements).

Joint Set 4 (J4):

Peak at 82/119 (steep to near vertical dip to the southeast and northwest), range from 66 to 82 (dip) and 125 to 293 (dip direction). This joint set is infrequent (10% of measurements) and is also found in the argillite unit (denoted as J4).

Joint Set 5 (J5):

Peak at 88/174 (steep to near vertical dip to the north and south), range from about 70 south to 70 north (dip) and 166 to 183 south and 347 to 003 north (dip direction). This joint set is frequent (25% of measurements). Joints have medium to high persistence, moderate to wide spacing, and smooth, planar surfaces.

Joint Set 6 (J6):

Peak at 60/042 (moderate dip to the northeast), a constant dip of 60 and a range of 041 to 042 (dip direction). This joint set is infrequent (10% of measurements).

3.4.3 Structural Domain 3: Meta-Volcanics

The meta-volcanic unit is a strong, relatively massive, slightly weathered rock that contains an abundance of shear zones and mafic dyke intrusions. This is the structural domain of Design Sectors 3 and 4, and contains six notable joint sets, derived from 32 individual measurements. Photo 8 shows a representative exposure of the meta-volcanics in Design Sector 3.

Joint Set 1 (J1):

Peak at 74/198 (moderate to steep dip to the southwest), range from 68 to 80 (dip) and 179 to 217 (dip direction). This joint set is fairly frequent (13% of measurements) and is present in all Structural Domains (denoted as J1 in both the quartzite and argillite units). Joints generally have medium persistence although occasional very widely spaced joints are very persistent. Joint surfaces are planar but rough.

Joint Set 2 (J2):

Peak at 73/313 (moderate to steep dip to the northwest), range from 64 to 80 (dip) and 297 to 323 (dip direction). This joint set is fairly frequent (16% of measurements). Joints have moderate to wide spacing, medium persistence, and smooth, undulating surfaces.

Joint Set 3 (J3):

Peak at 34/169 (shallow dip to the south), range from 30 to 40 (dip) and 160 to 183 (dip direction). This joint set is infrequent (9% of measurements) but is also found in the argillite unit (denoted as J3). Joints have medium to high persistence but are widely to very widely spaced and generally only present in the upper 8 m of the existing rock cut.

Joint Set 4 (J4):

Peak at 33/342 (shallow dip to the northwest), range from 24 to 44 (dip) and 329 to 007 (dip direction). This joint set is fairly frequent (16% of measurements). Joints have a low to high persistence, wide to very wide spacing, and smooth, undulating surfaces.

Joint Set 5 (J5):

Peak at 80/003 (steep to vertical dip to the north and south), range from 70 to 90 (dip) and 197 and 347 to 013 (dip direction). This joint set is frequent (19% of measurements) but has low persistence.

Joint Set 6 (J6):

Peak at 83/253 (steep to vertical dip to the east and west), range from 70 to 90 (dip) and 067 and 245 to 267 (dip direction). This joint set is infrequent (9% of measurements) has low persistence, and wide to very wide spacing.

3.5 DESIGN SECTORS

The structural domains have been subdivided into five separate "design sectors" that represent areas having similar structural geology, rock cut orientation, height, or other characteristics that may necessitate a unique rock cut design. Design sector locations are shown on Figure 2 and include Design Sectors I, II, IIIa, and IV on the west (left) side of the highway and Design Sector IIIb which forms the right side of the through cut on the east side of the highway.

4.0 ROCK CUT DESIGN BASIS

4.1 GENERAL DESIGN CRITERIA

EBA understand that YTG are designing the proposed highway realignment with a design speed of 90 km/h to 100 km/h and prefer to use a ditch with a 4:1 (H:V) "recoverable" side slope without a guard rail so as not to impede the movement of sheep.

Section 52 of the Yukon Workers' Compensation Board Mine Safety Regulations allows a maximum bench height of 20 m with a bench width sufficient to permit safe access, but in no case less than 8 m. As this would involve an overall 68 degree or flatter cut slope, which is considered unrealistically flat for an engineered highway rock cut, EBA have assumed for design purposes that a variance to these requirements will be obtained.

Although EBA are not aware of established guidelines for sheep traverse of steep slopes, the following are suggested for Sheep Mountain based on experience in other areas:

- Design Sector I (Argillite) – a 76° cut slope (1H:4V) should be marginally passable due to the surface roughness offered by closely spaced bedding planes.
- Design Sector II (Quartzite) – a 76° cut slope (1H:4V) will have structurally controlled ledges and irregularities that may locally be passable, but it is doubtful that a high, uniform cut slope developed at 76° would contain a continuous traverse route for sheep. A flatter 71.6° (1H:3V) slope may offer some passable routes.

- Design Sectors IIIa and IIIb (Volcanics) – similar to Design Sector II, a 71.6° (1H:3V) slope may offer some passable routes, but a steeper 76° slope is likely not passable for higher cuts.
- Design Sector IV (Argillite) – similar to Design Sector I, a 76° cut slope (1H:4V) should be marginally passable due to the surface roughness offered by closely spaced bedding planes.

4.2 KINEMATIC STABILITY ANALYSES OF CUT SLOPES

The objective of kinematic stability analyses is to identify possible modes of failure (wedge, planar, toppling) for various cut slope inclinations to assist in selecting a cut slope angle which avoids most potential instability and provides a balance between excavation costs and the cost of stabilization measures such as rock bolting.

For detailed kinematic design, it is important to recognize that the dip of a particular planar or wedge failure mechanism has a statistical distribution about a mean value. The consequence of this with respect to cut slope design is that a cut which is steeper than the mean dip of the failure mechanism may still be feasible, but will result in more frequent or larger rockfalls, which in turn may require a wider ditch and more frequent ditch maintenance. Hence, optimization of cut slope angle, ditch width, and stabilization measures, requires careful consideration of capital costs versus maintenance costs, cut slope performance, and the acceptable degree of rockfall hazard.

In carrying out kinematic analyses, it is significant to note that most of the joint sets at Sheep Mountain are widely spaced and infrequent. This will tend to result in isolated areas of potential instability associated with a specific joint or discontinuity. These can often be managed most economically by localized stabilization measures such as rock bolting, rather than by flattening the overall cut slope.

EBA carried out kinematic analyses for each design sector based on the joint set characteristics contained in Table 1 and described above, and stereonets provided in Appendix C. Table 2 indicates the kinematically possible failure modes for each design sector and lists the theoretical dip angle of these failure modes with respect to the cut orientation.

Based on the kinematic analyses, the recommended cut slope inclinations for each design sector are discussed below and summarized in Table 3. The cut slope angles noted below are overall cut slope angles. The individual bench face angles will be slightly steeper to accommodate drill offsets between each lift of excavation. The relationship between the overall cut angle, bench face angle, lift height, and drill offset is illustrated in Figure 11.

Design Sector 1:

Design Sector 1 is within the argillite from Sta. 1704+590 to Sta. 1704+720. The average new highway alignment azimuth for this sector is 069° with the new cut dipping southeast towards 159°.

There is the potential for planar failures along J3 and J7, both of which have a strike within 10° of the cut azimuth. J3 and J7 have apparent dips of 45° and 71° respectively. Planar sliding is also possible on J4 but the strike of these joints is almost 35° off the highway azimuth, hence pure planar failures would be localized.

Toppling may occur on Set J5, which has a strike within 5° of the cut azimuth and dips at 74° into the slope, but these joints are too widely spaced to cause pervasive toppling instability.

There is the potential for several wedges to form. Joint Set J1 may form potential wedges with sets J3, J4 and J7. J4 may also form potential flat wedge intersections with set J7.

The flattest potential failure modes dip at 43° to 45° out of the cut and are associated with J3. Such failures would be unlikely or infrequent due to the moderate dip; and this is borne out by the performance of the existing cut slope. At 69° to 73° several variations of planar or wedge failures involving J1, J4 and J7 become kinematically possible. Thus, a cut slope angle of 72° is recommended to minimize ravelling. A slightly steeper slope of say 76° could also be considered provided that the ditch is sized to accommodate a substantially greater volume of material from plane or wedge failures.

Design Sector 2:

Design Sector 2 is within the quartzite from Sta. 1704+720 to Sta. 1704+815. The average new highway alignment azimuth for this sector is 071° with the new cut dipping southeast towards 161°.

There is the potential for planar failures along J5, which is oriented within 13° of the new cut orientation with an apparent dip of 88° and flattest dip of about 76°.

There is also the potential for several wedge failures associated with J4 in combination with either J1 or J5; these have apparent dips of 65° and 85° respectively.

The flattest potential failure mode involves wedges formed by J1 and J4 dipping at 65°. Although J1 is prominent and usually persistent, J4 is infrequent, hence this type of wedge failure will be localized only and not pervasive throughout the rock mass. The next steepest mode of failure is wedge failures dipping at 85° associated with J5. As J5 is quite frequent and can form persistent, smooth planar surfaces, these joints should not be undercut. Therefore, a cut slope of 76°, which is flatter than virtually all of the statistical distribution of mapped J5 joints, is recommended. A steeper cut up to about 82° is also feasible but may require significantly more trimming and/or bolting to address potential planar or wedge failures associated with J5.

Design Sector 3a:

Design Sector 3a is within the volcanics from Sta. 1704+815 to Sta. 1705+020 and forms the left side of a through cut heading north. The new alignment azimuth for this sector ranges from 045° to 058° with the new cut dipping southeast towards 135° to 148°.

There is a potential for wedge failures formed by J1 and J6 with an apparent dip of 76° out of the cut slope. Since J6 has a near vertical dip, low persistence, wide spacing and is oriented almost perpendicular to the cut, stability of these wedges will be governed by sliding on J1, which is oriented oblique to the cut. Therefore, while wedge failures at a cut slope of 76° are feasible, they will be infrequent and localized. Toppling may also occur on Set J2, which is oriented within 14° of the new cut and dips at 72° into the slope.

In consideration of the potential for toppling and the occasional wedge failures, a cut slope inclination of 76° is recommended, however, a steeper cut slope of up to say 82° is also feasible provided the ditch is sized to accommodate a greater volume of failed material and/or potentially unstable material is stabilized by bolting.

Design Sector 3b:

Design Sector 3b is within the volcanics from Sta. 1704+825 to Sta.1705+020 and forms the right side of a through cut heading north. The new alignment azimuth for this sector ranges from 045° to 058° with the new cut dipping northwest towards 315° to 328°.

There is the potential for planar failures to occur on J2, which is oriented within 14° of the new cut orientation and has an average apparent dip of 72° out of the slope. There is also the potential for wedge failures formed by J5 and J6 with an apparent dip of about 74° out of the slope.

As J2 is fairly frequent and has a well clustered dip distribution of about 64° to 80°, it should be undercut only with caution and the understanding that stabilization measures may be necessary. Therefore, we recommend a cut slope inclination no greater than the average apparent dip of 72°, and preferably about 65° to minimise or eliminate potential stabilization requirements.

Design Sector 4:

Design Sector 4 is within the argillite from Sta. 1705+020 to Sta. 1705+100. The new alignment azimuth for this sector is 030° with the new cut dipping southeast towards 120°.

There is the potential for planar failures along J4, which is oriented within 6° of the new cut orientation and has an apparent dip of 82°. Wedge failures may also occur with J1 in combination with J3, but these have a true plunge of 43° and are generally likely to be stable. Toppling may occur on J6, which is oriented within 5° of the new cut orientation and dips at 58° into the slope, but these joints are too widely spaced to cause pervasive toppling instability.

For this design sector, planar sliding on J4 governs stability and hence a cut slope inclination of 82° is the maximum recommended. A flatter cut inclination of about 76° or less which encompasses virtually all of the statistical distribution of mapped J4 joints may also be considered to minimise ravelling and long term maintenance issues, and better facilitate sheep passage.

4.3 DITCH DESIGN CRITERIA AND METHODOLOGY

EBA have assumed that ditches for the Sheep Mountain rock cut will have a 4:1 (H:V) recoverable side slope without guard rail and with a minimum width and depth of 4.0 m and 1.0 m respectively. Site distance criteria may require an increased width in some areas.

Historically, the most common criteria used for ditch design are those proposed by Ritchie (1963). These criteria acknowledge that for cuts inclined at approximately 76 degrees or steeper, rock particles tend to fall down to the ditch with minimal bounce en-route. At progressively flatter cut angles, rocks tend to roll and/or bounce down the cut slope, which requires a wider and/or deeper catchment ditch. In general, the Ritchie criteria are based on a deeper ditch with steeper side slope than proposed for Sheep Mountain, hence these criteria can not be applied without considerable judgement.

More recently, the Oregon Department of Transportation (ODOT) Rockfall Catchment Area Design Guide (2001), which is based on extensive field testing, has replaced the Ritchie criteria as a design guideline for catchment ditches. Several limitations of the ODOT guide should be noted as follows:

- The ODOT guide is based on field tests of rockfalls on slopes up to 24 m in height. This requires extrapolation of the results to address the higher cut slopes at Sheep Mountain.
- The ODOT field tests were conducted on very uniform slopes that may not reflect "real world" slope irregularities that tend to deflect rockfalls farther than anticipated.
- The ODOT guide does not address rockfalls that originate above the cut and already have some angular and linear momentum as they pass over the crest of the cut. Fortunately, this effect can be largely discounted since the slopes above the cut at Sheep Mountain are gentle enough that the initial momentum at the crest of the cut will be relatively small.

EBA have relied on the ODOT guide for design of catchment ditches for the Sheep Mountain cut. As a minimum, designs provide a sufficient ditch width to ensure that at least 98% of all rockfall impacts land within the ditch, plus at least 1 m to 2 m of additional width depending on the cut height.

After impact, rockfalls tend to bounce once or twice, then roll out of the ditch at a low height which poses a comparatively low hazard to motorists. Consequently, ditches have generally been designed on the basis of 95% to 98% retention of rockfalls for low to moderate height cuts. For higher cuts, designs have been based on a reduced retention percentage of 90% to 95%, but in conjunction with a more conservative cut slope design angle and/or increased reliance on stabilization measures such as bolting for the higher portions of the cut to reduce the rockfall potential.

The most frequent sources of rockfall will be the rubble colluvium on the existing slope above the crest of the new cut, the overburden cut slope along the brow of the rock cut, and weathered, near surface bedrock near the crest of the rock cut. Consequently, the full cut height has been considered for ditch design.

The ditch design recommendations presented herein do not provide absolute security against rockfall but are considered to provide an acceptable balance between cost and risk. The ditch geometry has adequate capacity to substantially contain rockfalls in the order of 5 m^3 total volume, which is as large as should be expected if the cut slope angle respects the kinematic analyses. Any larger potential failures must be identified and stabilized during construction.

5.0 CUT SLOPE DESIGN RECOMMENDATIONS

5.1 DESIGN OPTIONS

In developing a cut slope design, EBA has attempted to balance the conflicting requirements to minimize capital and maintenance costs, provide an acceptable level of rockfall hazard, and accommodate sheep passage through the area. This has led to the development of three design options as described below.

Option 1 – Rock Cut Without Bench:

This option involves a conventional, continuous cut slope without benches. The cut slope would be at a relatively stable angle from a kinematic perspective of 76° (except for 72° in Design Sector 1 and 68° in Design Sector 3a) and range in height from less than 10 m up to approximately 50 m. The ditch width would vary from 4 m up to 13 m according to the cut height to provide adequate retention of rockfalls. As such, the high, central portion of the cut is unlikely to be passable by sheep, either up and down the slope, or traversing north-south across the cut slope. However, sheep could likely climb the lower cut slopes in argillite at both ends of the rock cut in Design Sectors 1 and 4.

The advantages of Option 1 are relatively simple excavation geometry and slightly reduced reliance on rock support through the high, central section due to the very wide ditch. The excavation volume will be similar to, or marginally greater than Option 2.

Option 2 – Rock Cut With Midslope Bench:

Option 2 utilizes a midslope bench through the high, central high portion of the cut to limit the effective slope height below the bench to a maximum of 30 m, which allows the ditch width to be limited to a maximum of 8 m. The cut slope below the bench would be at a less conservative angle from a kinematic perspective of 82° (except for 72° in Design Sector 1 and 68° in Design Sector 3a). This may require additional rock support compared to Option 1, but is considered feasible because the effective slope height below the bench is limited and the rock mass quality is expected to improve with depth as the effects of surface weathering diminish. The cut slope above the bench would be developed at 76° in the near surface weathered rock.

The bench would begin at highway grade at the south end of Design Sector 1, then traverse up the slope across the top of the cut in Design Sectors 1 and 2. The cut slope above the bench in this area would have a maximum height of 10 m and will require a minimum bench width of 8 m for maintenance access and so that rockfalls from the upper slope are retained on the bench, or have their momentum reduced to near zero before rolling off the bench. The bench would climb to a maximum height of 30 m above the ditch at approximately the boundary of Design Sectors 2 and 3a, at which point it would traverse horizontally across Design Sector 3a, and the bench width would increase to a maximum of 11 m to provide adequate rockfall catchment as the cut height above the bench increases.

The primary advantage of Option 2 over Option 1 is the avoidance of a very high, continuous cut slope, which can generate very high energy rockfalls that are difficult to control, even with a wide ditch. Option 2 also provides for better management of rockfall originating above the cut slope, although with increased maintenance cost if the bench must be cleaned. Although movement of sheep up and down the slope will still be blocked as with Option 1, the midslope bench and access ramp will provide a safe north-south traverse route for sheep across the high central portion of the cut slope. Another advantage of Option 2 is that early establishment of a midslope bench will allow simultaneous excavation in several areas of the cut, thus speeding construction and shortening the construction duration.

Option 3 – Rock Cut / Tunnel Combination:

Within the central, through cut portion of the realignment in Design Sector 3a/ 3b, the average excavation volume per meter length of highway will be on the order of 1,000 m^3/m and the high, steep cut slope will block east-west sheep passage from the slopes above the highway, down to the highway and lake shore. Option 3 involves construction of an approximately 130 m long tunnel between approximately 1704+850 and 1704+980. Rock cuts leading up to the tunnel portals would be in accordance with either Option 1 or 2.

Although the overall cost of Option 3 is expected to be highest, the additional cost may not be that great since the tunnel replaces a section of through cut with a ten-fold greater excavation volume. The prime advantages of Option 3 are that it eliminates the high central portion of the cut that poses the greatest rockfall hazard, it provides an uninterrupted corridor approximately 100 m wide for east-west sheep passage over the new highway rather than across it, and it will significantly reduce construction impacts on the existing highway such as closures for blasting. In addition, Option 3 may reduce the duration of excavation (blasting) by several months as compared to the open cut options.

Recognizing that a tunnel was not contemplated at the preliminary design stage, we have only provided a conceptual design for Option 3 at this stage as described below and illustrated in Figure 9. The proposed tunnel would accommodate two 3.65 m wide lanes (one each direction) with a 2.0 m shoulder on each side. Oversize loads as well as pedestrian traffic need not be accommodated in the tunnel as this traffic could bypass the tunnel on the existing highway grade.

For the 130 m +/- length of tunnel anticipated, emergency egress tunnels (typically spaced at 200 m +/-) monitoring systems, fire suppression systems, and ventilation would not be necessary. Lighting would also likely be unnecessary. Consequently, capital and operating costs would be minimized relative to longer highway tunnels.

The tunnel would be excavated by drill and blast, likely with a single (or double) top heading and bench approach. Considering the generally favourable ground conditions, temporary support would be by means of rock bolts and shotcrete alone; ribs or lattice girders are unlikely to be necessary.

Although a permanent lining could be developed by thickening the shotcrete, as is common in Northern European highway or railway practice, the conceptual design assumes a permanent concrete lining typical of North American highway tunnels. For the short length of tunnel, it is unlikely to be economical to fabricate and mobilize a full section reusable form for concrete lining. Therefore, we considered that the concrete lining would likely be placed in two stages. First, the tunnel walls would be poured using a simple reusable steel or wood form anchored to the rock. Then, the tunnel arch would be poured using a large, standard size corrugated steel culvert or liner plate section ("Armco" or similar) as formwork, which would be left in place.

The final lining would be extended 5 m to 10 m beyond each portal to provide rockfall protection.

5.2 DETAILED DESIGN RECOMMENDATIONS

5.2.1 Overburden Cut Slopes

For all three design options, the overburden cut slopes along the crest of rock cuts is a potential source of rockfall that must be addressed during the initial stages of construction to provide safe working conditions. Overburden including talus like material should be removed for a distance of 3 m back from the crest of the rock cut and be sloped back at a stable angle (approximately 1.5H:1V for the rubble colluvium material). Unfortunately, the stable cut angle for the colluvium is very close to the topographic slope, hence the overburden cuts may need to be over-steepened to avoid having them extend far up the slope as a sliver cut. Since over-steepened cuts will have a greater tendency to experience ravelling and regression causing rockfall, stabilization of the overburden cuts will be required in some areas (see Section 6.1).

For the high rock cut in the central section of Option 1, prevention of rockfalls due to ravelling of the overburden cut is essential. For Option 2, which has a midslope bench that will retain most rockfall from the overburden, and that can be maintained, the requirement for stabilization of the overburden cuts is less critical.

5.2.2 Design Sector 1

For both Options 1 and 2, a cut slope inclination of 72° (1H:3V) is recommended (see Table 3). For Option 2 (rock cut with midslope bench) an 8 m wide ramp providing access to the bench would be situated at the crest of the cut as shown on Figure 6. Note that for both options, the ditch width varies as a function of cut height to minimize excavation volumes, while still providing adequate rockfall catchment.

5.2.3 Design Sector 2

The recommended rock cut geometry for design Options 1 and 2 in Design Sector 2 is shown on Figure 6 and summarized on Table 3. For Option 1, a uniform cut slope inclination of 76° (1H:4V) is recommended. For Option 2, which includes an access ramp/ bench across the crest of the cut, a cut slope of 76° (1H:4V) above the bench and 82° (1H:7V) below the bench is recommended. Note that for both options, the ditch width varies as a function of cut height to minimize excavation volumes, while still providing adequate rockfall catchment.

5.2.4 Design Sector 3a

The recommended rock cut geometry for design Options 1 and 2 in Design Sector 3a is shown on Figure 7 and summarized on Table 3. For Option 1, a uniform cut slope inclination of 76° (1H:4V) is recommended. For Option 2, which includes a midslope bench across the high, central portion of the cut, a cut slope of 76° (1H:4V) above the bench and 82° (1H:7V) below the bench is recommended.

Note that for both options, the ditch width varies as a function of cut height to minimize excavation volumes, while still providing adequate rockfall catchment. Similarly, the bench width for Option 2 varies according to the height of the cut above the bench.

5.2.5 Design Sector 3b

The recommended rock cut geometry for design Options 1 and 2 in Design Sector 3b is shown on Figure 8 and summarized on Table 3. For both options, a uniform cut slope inclination of 68° (1H:2.5V) is recommended. For both options, the ditch width varies as a function of cut height to minimize excavation volumes, while still providing adequate rockfall catchment.

The thin razorback ridge formed at the crest of the cut by the intersection of the new and existing cut slopes is likely to contain a significant quantity of blast loosened rock. Any such material must be removed to ensure that the safety of the existing highway is not compromised during the construction period. Note also that the top of this ridge could be easily cut down to reduce the cut height, and thereby permit a narrower ditch.

5.2.6 Design Sector 4

The recommended rock cut geometry for design Options 1 and 2 in Design Sector 4 is shown on Figure 8 and summarized on Table 3. For Option 1, a uniform cut slope inclination of 76° (1H:4V) is recommended. For Option 2, the midslope bench in Design Sector 3a will need to slope down towards the north in Design Sector 4 to follow the crest of the cut and maintain the minimum 8 m bench width. At the north end of Design Sector

4 where the new cut rejoins the existing highway cut, the bench need not be extended all the way down to the highway grade, but to do so may help facilitate sheep passage.

For Option 2, a cut slope of 76° (1H:4V) above the bench and 82° (1H:7V) below the bench is recommended. Note that for both options, the ditch width varies as a function of cut height to minimize excavation volumes, while still providing adequate rockfall catchment.

6.0 CUT SLOPE STABILIZATION RECOMMENDATIONS

Stabilization measures such as concrete guard rail, rockfall fences, or slope mesh, that would impede sheep passage or that may be aesthetically unpleasing, have not been considered.

6.1 OVERBURDEN

Potential instability of overburden cut slopes along the brow of the cut will pose a significant problem in many areas due to the steep natural slopes, loose nature of the coarse colluvial overburden, occasional seepage/runoff flowing across the bedrock surface during wet weather, and dislodgement of material by sheep. In several areas, evidence of previous failures of the overburden down to bedrock were observed. On a smaller scale, it is expected that overburden cut slopes will ravel, and in so doing periodically release large rock blocks as rockfalls. As this can pose a significant hazard both during and after construction, it is essential the overburden cuts be properly stabilized when first exposed. The following provides a brief summary of several methods that may be considered.

Where larger "framework" grains (e.g. boulders and larger) in the overburden are widely spaced and not in contact, and the interstitial soil matrix consists or erodeable material, stabilization measures must target the soil matrix. In this case, hydroseeding, erosion control mats, or even shotcrete shoring might be considered. However, hydroseeding is unlikely to be effective at Sheep Mountain without use of fertilizers.

The most practical method of stabilizing the overburden cuts will be to support the toe of the cut slope where it is in contact with bedrock. The recommended approach is to construct a low retaining wall anchored to the rock with dowels. The face of the wall will consist of hexagonal gabion mesh covered with shotcrete. The recommended installation sequence is as follows:

- Install dowels grouted into rock and protruding to the required wall height along the toe of the overburden cut slope (see table below).
- Install horizontal reinforcing on upslope side of dowels (see table below).
- Install mesh (11 gauge galvanized hexagonal triple twist, Maccafferri or equivalent) against the upslope side of the horizontal reinforcing.
- Install 100 mm dia. perforated PVC drains on rock surface at nominal 2 m spacing.

- Backfill against mesh (upslope side) with loosely placed rock from the colluvial slope above the retaining wall.
- Shotcrete the face of the wall to fully encase dowels, horizontal reinforcing bars, and mesh (see table below for shotcrete thickness).

Layout dimensions for three wall heights are as follows:

OVERBURDEN RETAINING WALL DIMENSIONS						
Wall (Dowel) Height above Bedrock (m)	Dowel Diameter (see note) (mm)	Dowel Embedment in Bedrock (m)	Dowel Spacing (m)	Horizontal Reinforcing	Shotcrete Thickness (mm)	
					Top of Wall	Toe of Wall
0.4	25	0.6	0.8	2 – 16M	100	200
0.6	32	0.7	0.8	3 – 16M	100	250
0.9	44	0.8	0.9	4 – 16M	100	300

Note: Dowels to be Grade 517/690 MPa galvanized Dywidag bar or equal and be drilled and grouted into bedrock.

In some instances it may be prudent to temporarily cover the overburden cut slope with mesh anchored to the bedrock and/or install a temporary chain link fence to minimise the rockfall hazard during construction.

An alternative to a dowel/shotcrete retaining wall as described above would be a conventional gabion wall set on a concrete starter footing that is doweled into the bedrock. In this case the dowels should protrude through the gabion to prevent sliding failure of the gabion baskets.

6.2 BROW OF ROCK CUT

Potentially unstable bedrock situated along the brow of the cut should either be removed (scaling, trim blasting, etc.) or be stabilized by bolting. Removal at the time of construction is generally the cheapest hence preferred approach.

6.3 BOLTING

"Rock Bolts" as used in this report refers to both tensioned "bolts" or untensioned "dowels" of 25 mm diameter. In either case, rock bolts should be fully grouted with either cement or resin grout. The required number, length, location, orientation, and need for tensioning rock bolts must be determined by the Engineer based on inspection of the actual rock structure(s) exposed in the cut slope. Where a need for bolts is identified, they should be installed as soon as possible, and preferably prior to further work in the area.

The following abbreviated specifications provide details of the rock bolt materials and installation procedures envisaged for the Sheep Mountain rock cut.

Materials:

.1 Rock Bolts shall be 25 mm diameter, Grade 517/690 MPa deformed steel bars conforming to CAN/CSA G30.18, such as "Dywidag Threadbar" manufactured by Dywidag Canada Limited, or approved equal. Rock Bolt bars shall be delivered to the site in mill lengths of 12.19 m (40') that can be field cut to the required lengths.

.2 Steel bearing plates shall conform to CAN/CSA-G40.21, Grade 300 W and have minimum dimensions of 10 mm by 150 mm by 150 mm. Plates shall be of "calotte" or similar style to accommodate non-perpendicular alignment of the bolt and plate. Nuts shall be hexagonal head, with hemispherical end matching the bearing plate.

.3 Rock Bolts and all associated hardware shall be hot-dip galvanized to CSA G164.

.4 Resin grout or cementitious grout may be used however, resin grout shall not be used where the rock is excessively fractured or wet. Resin grout shall be the product of an established manufacturer, and shall be supplied with fast and slow gel set times respectively for the anchorage and free-stressing lengths of the bolt. Cement grout shall be a pre-bagged, non-shrink product such as "Microsil Anchor Grout" produced by Basalite Concrete Products.

Installation:

.1 For resin grouted rock bolts, drill holes to a diameter recommended by the resin manufacturer. For cement grouted bolts, drill holes to a minimum diameter of 60 mm, or smaller if required to accommodate an expansion shell anchor.

.2 Installation – Resin Grouted Rock Bolts

Use fast setting resin at the bottom of the hole for anchorage, and slow setting resin for the remainder of the hole. Installation must be in strict accordance with the resin manufacturers recommendations to ensure correct mixing of the resin and complete encapsulation of the bar. After allowing the fast setting resin to set, but prior to the gel time of the slower resin, perform testing and tensioning as required.

.3 Installation – Cement Grouted Rock Bolts

Cement grouted bolts that are to be tensioned shall use either resin cartridges or an expansion shell anchor for the bond length anchorage. Cement grout shall be placed by pumping through a delivery line that extends to the lowest end of the hole. Prior to the grout setting, perform any necessary testing and tensioning.

.4 Testing

Testing equipment shall include a hollow core hydraulic jack with pressure gauge and an independently mounted dial gauge for measuring the strain of the bolt under load. Rock bolts will be selected at random by the Engineer for testing. The first ten (10) of each type shall be tested; thereafter, 20% of the rock bolts shall be tested. Bolts shall be either Proof Tested or Pull Tested as directed by the Engineer.

.5 Tensioning

Tensioning equipment shall consist of the hollow core jack used for testing. A calibrated impact or torque wrench may be used for light tension loads, subject to approval by the Engineer. Bolts shall be tensioned and locked-off at loads ranging from 50 kN to 158 kN as directed by the Engineer. Untensioned bolts shall be nominally tensioned to 25 kN.

The rock cut slope angles have been designed based on kinematic analyses to minimize reliance on rock bolts for support. Nevertheless, occasional joints with random orientation, as well as those joints within the identified joint set populations whose dip is flatter than the cut slope, will give rise to some potentially unstable features that require bolting.

The most critical area where bolting may be required is higher portions of the high rock cuts situated in Design Sectors 2 and 3a, in particular, those portions of the cut that are greater than 20 m above the ditch and/or 15 m above the midslope bench. Assuming an average bolting density of about one bolt per 30 square meters for these areas, it is estimated that up to about 200 bolts could be required. Bolt lengths will typically vary from 3 m to 6 m depending on the feature being addressed.

6.4 SHOTCRETE

Shotcrete will not be necessary as a covering of the rock face. However, as noted previously, shotcrete may be very useful for stabilizing oversteepened overburden cut slopes that would otherwise be prone to ravelling. Owing to the remote location and small application volumes at any given time, it is anticipated that dry mix shotcrete will be used. This should be a standard mix design from a recognized supplier such as Basalite Concrete Products, and should contain silica fume and steel fibres.

7.0 CONSTRUCTION ISSUES

7.1 EXCAVATION SEQUENCE AND PIONEER ACCESS

Pioneer access to the crest of the cut slope in Design Sectors 1 and part of Design Sector 2 can be easily established by excavation to widen the existing remnant access bench (see Photo 3) that begins at about 1704+500. As this is the most logical haul route for muck from the high central portion of the cut in Design Sector 3, the pioneer ramp should be a full 8 m wide to facilitate haul traffic. If this ramp is developed on the line and grade required to access the midslope bench for Option 2, then Option 2 may be the favoured rock cut design. In this case, the pioneer access will need to be extended north of approximately 1704+750 to reach the mid slope bench elevation 30 m above the existing highway grade in Design Sector 3a.

Equipment (drill and excavator) could access the crest of the cut in Design Sector 3a relatively easily from the old pipeline road, which is situated about 10 m to 15 m above the cut at this location. However, this would not provide a favourable route for haul traffic, due to the adverse grade for trucks to climb up to the pipeline road, and then the long haul distance to descend down to the existing highway. Therefore, for the top portion of the cut in Design Sector 3a, the best approach may be to sidecast the excavated rock down the slope, and then load this material into haul trucks on a midslope bench situated about 30 m above the existing highway, as illustrated on Figure 10. For this approach, the midslope bench must be developed first, and must be wide enough to retain all the material that is sidecast down the slope from above.

An important feature of the approach illustrated on Figure 10 is that once access to the middle and top of the slope is developed (Steps 1 and 2 on Figure 10) it will be possible to carry out further excavation from the upper and midslope benches simultaneously (Steps 3 through 6). For the final stages of excavation (Steps 7 through 9) the bench width is adequate for two drills to work side by side. This approach may significantly shorten the construction duration.

Pioneer access to the crest of the new cut slope in Design Sector 4 could be developed by traversing southwards up the slope starting from about 1705+200. However, it may be easier to develop pioneer access across the crest of Design Sector 4 by working north from a midslope stage of excavation in Design Sector 3.

A major consideration in the sequencing of excavation is to minimize highway closures, which will likely be limited to 15 minutes without advertisement, or up to 2 hours with prior advertisement. Although a blast could be shot within a 15 minute closure, it typically requires at least another hour to remove loose material from the blast that may pose a hazard to motorists and to clear the highway. For the highest portion of the cut in Design Sector 3, it would be very difficult and time consuming to make the slope safe unless there is a midslope bench as shown in Figure 10. However, once this bench is established, the bulk of the excavation in the central through cut section could likely be carried out within 15 minute closures, except for benches 3B and 4B as shown on Figure 10.

Much of the excavation for the lower cuts situated in Design Sectors 1, 2, and 4 that are adjacent to the existing highway will require closures longer than 15 minutes to ensure loose material is adequately removed prior to passing traffic. The use of single lane traffic on the northbound lane, with a catchment berm constructed on the southbound lane adjacent to the cut slope will help minimize closure durations.

7.2 CONSTRUCTION SAFETY

As noted previously, ravelling of the overburden cut slope and loose collivial rock debris on the slope poses one of the greatest hazards to construction crews excavating the rock cut. This material must be stabilized, and temporary worker protection such as rock catchment fencing installed if necessary, when the pioneer bench is first advanced across the slope and there is still good access for construction equipment.

Vibrations from blasting and removal of support of the backslope by excavation will result in some areas of marginal stability. Although the Contractor is ultimately responsible for construction safety, it is essential that procedures be in place to identify and rectify areas of potential instability at the earliest opportunity. These measures should include:

- Thorough scaling of freshly exposed cut slopes.
- Inspection by the Contractor and Geotechnical Engineer following every blast and/or during muck removal as necessary to identify areas of potential instability.
- Implementation of remedial measures such as trim blasting or bolting prior to further work below an area of suspect stability.
- Training of equipment operators through a mandatory safety program to identify and immediately report any conditions of concern.

It is essential that areas of potential instability be identified and addressed in a timely fashion when the bench is still at the elevation of concern and equipment can still access the area from the bench or muck pile. If excavation is allowed to progress without first addressing potential problem areas, the remedial work will be more costly to implement and worker safety may be jeopardised in the interim.

Blasting may destabilize areas of the slope below the bench, which could pose a rockfall hazard to motorists subsequent to the blast. Such failures are generally caused by:

- Down slope displacement of some of the muck pile, usually because the blast configuration endeavoured to break more rock than could be retained on the bench, or the delay pattern was not sufficiently biased to break towards the adjacent built road.
- Overbreak of the blast along joints in the bedrock along the outside edge of the bench. A reduced explosives loading should be used where there is a potential for instability.
- Blasting vibrations can cause the marginally stable colluvial rock debris on the slope to become unstable. Appropriate precautions include using a reduced charge weight per delay to limit vibrations, and inspection/scaling of suspect areas after each blast.

In all of the above cases, the likelihood of a down slope failure can be minimised by using carefully controlled and conservative blasting practices

7.3 BLAST DESIGN RECOMMENDATIONS

7.3.1 General

Careful blast design will be required to limit damage to the backslope, to create a useable blast rock gradation for embankment construction, and to prevent flyrock from entering Kluane Lake. As with any blasting project, blasting procedures (hole layout, loading, delay patterns, etc.) will need to be modified on site on a blast by blast basis to obtain the best results. This is especially true for the Sheep Mountain rock cut where the bedrock geology is extremely variable over short distances. In addition, the layout and loading of holes along the outer edge of each bench must be adapted to the local topography.

Controlled Blasting techniques in general as outlined in Clause 22, Section 01010 of the specifications for km 1664.0 to km 1674.2 of the Shakwak Highway Project (or similar specifications) are generally appropriate for this project with a few exceptions or additional requirements as noted in the following sections.

Wright and Hopky (1988) provide guidelines for the use of explosives near fisheries waters that are followed by the Department of Fisheries and Oceans (DFO). These guidelines have two requirements with respect to charge weight per delay. First, the weight of explosives must be limited such that the instantaneous pressure change in fish habitat does not exceed 100 kPa. Secondly, the weight of explosives must be limited such that the peak particle velocity does not exceed 13 mm/s in a spawning bed. At each end of the project where blasting will be closest to Kluane Lake (about 25 m minimum) the charge weight per delay will need to be limited to 25 kg to meet the 100 kPa requirement. However, most areas are at least 30 m from the lake, and in this case up to 35 kg of explosives could be used.

EBA anticipate that the second criteria (maximum 13 mm/s peak particle velocity in a spawning bed) would not be applicable as it is unlikely the steep rocky shoreline would be classified as spawning habitat. However, if this criterion did apply, the charge weight per delay would be limited to as little as 4 kg at a distance of 30 m.

There are no nearby structures that might be impacted by blasting vibrations, and therefore, the charge weight per delay need not be restricted for this reason. However, we recommend that a maximum charge weight per delay of 35 kg be established for this project to limit potential damage to the rock mass beyond the excavation lines and to minimize instability of the colluvial overburden caused by excessive blast vibrations. A reduced value would be required within 30 m of the lake to meet the DFO 100 kPa requirement. For a powder factor of 0.75 kg/m^3 which may be suitable for most of the rock at site, the 35 kg charge weight would limit the pattern size to 2.1 m x 2.7 m (burden x spacing) for 8.0m deep holes detonated individually. This is also close to the maximum pattern size that can be used efficiently with 8 m deep holes. A tighter pattern (or shear line) with light loading could use two or more holes per delay.

As some of the rock produced by excavation will be used for embankment construction in or near Kluane Lake, the explosives must produce a minimal amount of ammonia or similar chemical residues that may be toxic to fish. Therefore, explosive products such as Ammonium Nitrate and Fuel Oil (ANFO) must not be used for this project. There is a wide range of products from various suppliers that fall into the category of "Dynamites" (nitroglycerin based) or "Emulsions". However, some of these contain a significant amount of ammonium nitrate and thus should be avoided.

7.3.2 Backline Holes

In EBA's experience, preshear blasting often produces mediocre results in the weathered surface rock layer (first lift of excavation) due to overbreak along weathered discontinuities and inadequate confinement. Thus we recommend that cushion blasting be used for the first one or two lifts of excavation until there is at least 8 m horizontally of rock confinement in front of the backline before attempting preshear blasts.

Based on observation of some existing small diameter blast hole traces spaced about 0.8 m apart, EBA believe smooth wall blasting can provide good results. Smooth wall holes should be spaced at not more than 0.75 m and be loaded with a continuous, light explosives load product such as "Primaflex" or similar. The "shear factor" (explosives load per unit area of backwall) should be about 0.6 kg/m^2 as a starting point for preshear holes and 0.75 kg/m^2 for cushion blast holes. In order to limit blast vibrations, the preshear line should be delayed with no more than 8 holes per delay, with a delay of 10 to 20 ms between each group of holes.

For developing a pioneer access bench across the crest of the cut, the backline holes will necessarily be drilled sub-horizontal. Due to limitations with most drilling equipment, there will be a tendency for these holes to fan out, providing uneven spacing between holes. To counter this tendency, the length of pioneer advance rounds should be limited to not more than 5 m. In addition, the backline holes should be spaced not more than 0.6 m apart and be loaded/delayed as a cushion blast.

Aside from the first lift of excavation, backline holes for all subsequent lifts must be collared 0.3 m to 0.5 m away from the excavated face to allow for drill boom clearances. The need to accommodate this "drill step-out" often causes confusion, especially with high cuts, regarding whether or not the survey layout of the crest of the cut should be adjusted to allow for multiple drill step-outs. Figure 11 provides clarification of the positioning requirements for backline holes with respect to the overall design backslope line. A maximum lift height of 8 m is recommended to minimize problems with misalignment of the backline holes.

7.3.3 Production Bench Blasts

As noted above, the maximum charge weight per delay should be no more than 35 kg. In addition, the depth of production holes should be limited to 8 m to coincide with the 8 m limit on backline hole depth.

Production blasts should generally be delayed to break either north or south towards a previously excavated bench to minimize displacement of the muck down the slope. Likewise, the length of a blast along a bench as measured parallel to the highway should be limited to about 15 m to ensure reasonable relief towards the excavated bench rather than towards the highway. Near the top of the cut where the bench width is limited, this will limit the maximum size of a blast to about 1,000 m^3. Lower in the cut where the bench width increases to 15 m or more, a blast volume of up to 2,000 m^3 may be acceptable.

For normal production blasting with 8 m deep holes where good fragmentation is the objective, a 2.1 m x 2.7 m (burden x spacing) pattern is about the largest that can be used for practical reasons, but this would require a hole diameter of at least 90 mm to accommodate the explosives. Therefore, a tighter pattern of 1.8 m x 2.4 m that can be managed with a 75 mm diameter hole size (depending on explosives) is recommended for optimum production and fragmentation.

For maximizing the production of rip rap, a "square" pattern with equal burden and spacing of about 2.1 m x 2.1 m, or even with burden slightly greater than the spacing, combined with light loading, will provide the a coarsest possible gradation.

7.3.4 Estimated Blast Rock Gradation

It is understood that YTG would like to use the blast rock from the rock cut for highway embankment construction in or near Kluane Lake. Further, the predominant gradation requirement will be for Class II size rip-rap material. For reference, the appro9ximate gradations for Class I, II, and III rip-rap are provided on Table 4.

The gradation of the excavated material will be largely dependant on the geologic discontinuities present in the rock, as well as the blast design. Table 4 provides an estimate of the proportion of the three rip-rap classes which could be produced for each of the argillite, quartzite and volcanic rock present at the site.

In general, it is anticipated that the argillite will tend to break along closely spaced bedding planes, with the result that only a limited proportion of Class II or III material can be produced, regardless of the blast pattern. In addition, the argillite may produce a relatively high proportion of platey material which will be less stable (lower friction angle) for embankment construction. In short, the argillite may produce a well graded rock fill but is unlikely to produce durable, large sized particles suitable for rip-rap.

Conversely, it is anticipated that the meta-volcanic rock will produce a significantly greater percentage of Class II material and some Class III material. Bedding planes within the quartzite are sometimes quite conspicuous, and hence it is anticipated that this rock will produce less coarse material than the volcanics, but more than the argillite.

The blasting patterns presented in Table 4 are intended as a starting point only, and will depend on hole depth, the type of explosives used, and other factors. Table 4 also includes an estimate of the bulking factor for rock which is blasted and then recompacted.

8.0 CLOSURE

This report has been prepared for the exclusive use of the Government of the Yukon. Use of this report is subject to the Geotechnical Report General Conditions included in Appendix D.

Recommendations presented in this report are based on engineering judgement and qualitative and quantitative analyses. The recommendations have been prepared in accordance with generally accepted rock slope engineering practices and attempt to provide practical and economic solutions with a reasonable balance between cost, benefit, and risk.

EBA trust that this report satisfies YTG's present requirements. We will be pleased to provide any further assistance that may be needed during detailed design and construction of the proposed remedial works. Please feel free to contact our office should you require additional information or wish to discuss the findings and recommendations presented herein.

Respectfully Submitted,
EBA Engineering Consultants Ltd.

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Reviewed by:

PROFESSIONAL ENGINEER
YUKON TERRITORY
J. RICHARD TRIMBLE
April 10/06

J. Richard Trimble, P.Eng.
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REFERENCES

Clague, John J., January 27, 1981, Landslides at the south end of Kluane Lake, Yukon Territory, Canadian Journal of Earth Sciences, Vol. 18.

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Ritchie, A.M., 1963. The evaluation of rockfall and its control. Highway Record. Vol 17.

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TABLES

Table 1 - Joint Set Characteristics

Structural Domain	Design Sector	Sector ID	Structure Type	Orientation Description	DIP min	DIP max	DIP DIR min	DIP DIR max	Persis-tence	Spacing	% of Observ-ations	Distrib-ution	Comments
1 (Argillite)	1 & 4	J1	Joint	Moderate dip to SW	58	82	194	243	H	M	32	P	
		B	Bedding	Moderate dip to SW	64	82	207	246					Same as J1
		J2	Joint	Shallow dip to NW	25	36	298	346			6	L	
		J3	Joint	Shallow dip to S	34	56	149	179	M	VW	16	C	
		J4	Joint	Steep dip to SE/NW	76	90	113	137	M	W - VW	10	L	
		J5	Joint	Moderate dip to N	68	82	337	347			6	L	
		J6	Joint	Moderate dip to NW	48	72	297	320			8	L	
		J7	Joint	Moderate dip to S	66	78	155	163	M	W - VW	6	L	
2 (Quartzite)	2	J1	Joint	Moderate dip to SW	62	73	217	242	H	W	30	F	
		B	Bedding	Moderate dip to SW	68	70	229	233					Same as J1
		J2	Joint	Shallow dip to NW	20	20	311	311	L		5	L	
		J3	Joint	Shallow dip to SE	24	34	103	141			10	L	
		J4	Joint	Steep dip to SE/NW	66	82	125	293			10	L	Steep joints cutting into the face.
		J5	Joint	Steep dip to N/S	70S	70N	166/347	183/363	M - H	M - W	25	F	Flat Joints form face of cut.
		J6	Joint	Moderate dip to NE	60	60	041	042			10	L	
3 (Volcanics)	3	J1	Joint	Moderate dip to SW	68	80	179	217	M	VW	13	C	
		J2	Joint	Moderate dip to SE/NW	64	80	297	323	M	M - W	16	C	
		J3	Joint	Shallow dip to S	30	40	160	183	M - H	W - VW	9	L	
		J4	Joint	Shallow dip to N	24	44	329	367	L - H	W - VW	16	C	
		J5	Joint	Steep dip to N/S	70	88	197/347	13	L - H	VW	19	C	
		J6	Joint	Steep dip to E/W	70	80	067/245	267	L	W - VW	9	L	

Persistance:

Very Low	< 1 m	VL
Low	1 - 3 m	L
Medium	3 - 10 m	M
High	10 - 20 m	H
Very High	> 20 m	VH

Spacing:

Extremely Close	< 0.02 m	EC
Very Close	0.02 - 0.06 m	VC
Close	0.06 - 0.2 m	C
Moderate	0.2 - 0.6 m	M
Wide	0.6 - 2.0 m	W
Very Wide	2.0 - 6.0 m	VW
Extremely Wide	> 6.0 m	EW

Distribution:

P - Pervasive (> 30%)	Dominant structural feature present throughout rock mass (e.g. foliation).
F - Frequent (21 - 30%)	Frequently occurring structural feature with homogeneous distribution in the rock mass.
C - Common (11 - 20%)	Infrequently occuring features but with overall homogeneous distribution.
L - Localized (6 - 10%)	Random or localized feature(s) with heterogeneous distribution in the rock mass.
R - Rare (0 - 5%)	Isolated structural feature(s); not characteristic of most of the rock mass.

Notes:

1. Orientations listed as dip and dip direction.

Table 2 - Kinematic Analyses

Design Sector	Structural Domain	Proposed Route Azimuth	Dip Direction of Cut	Kinematically Possible Failure Modes		Comments	Dip of Plane Perpendicular to Cut	Apparent Dip of Wedge
1	1 (Argillite)	069°	159°	Wedge:	J1 with J3	Intersection line 43° plunge, 81° from cut orientation		43°
				Wedge:	J1 with J4	Intersection line 68° plunge, 49° from cut orientation		73°
				Planar:	J3	Within 10° of new cut orientation	45°	
				Wedge:	J1 with J7	Intersection line 67° plunge, 54° from cut orientation		72°
				Toppling:	J5	Dips 74° into the cut, 5° from cut orientation		
				Wedge:	J4 with J7	Intersection line 65° plunge, 49° from cut orientation		69°
				Planar:	J7	Within 1° of new cut orientation	71°	
2	2 (Quartzite)	071°	161°	Wedge:	J1 with J4	Intersection line 63° plunge, 59° from cut orientation		65°
				Wedge:	J4 with J5	Intersection line 80° plunge, 24° from cut orientation		85°
				Planar:	J5	Within 13° of new cut orientation	88°	
3a	3 (Volcanics)	045°-058°	135°-148°	Wedge:	J1 with J6	Intersection line 72° plunge, 37°-50° from cut orientation		76°
				Toppling:	J2	Dips 72° into the cut, 2°-14° from cut orientation		
3b	3 (Volcanics)	225°-238°	315°-328°	Planar:	J2	Within 14° from cut orientation	72°	
				Wedge:	J5 with J6	Intersection line 73° plunge, 80° from cut orientation		74°
4	1 (Argillite)	030°	120°	Wedge:	J3 with J1	Intersection line 43° plunge, 60° from cut orientation		45°
				Planar:	J4	Within 6° of new cut orientation	82°	
				Toppling:	J6	Dips 58° into the cut, 5° from cut orientation		

Table 3 - Rock Slope and Ditch Design Summary

DESIGN SECTOR	LOCATION			RECOMMENDED CUT SLOPE GEOMETRY									SUPPORT / COMMENTS
				OPTION 1 (No Midslope Bench)			OPTION 2 (With Midslope Bench)						
							Lower Cut Slope			Upper Cut Slope			
	From[1]	To[1]	Offset	Cut Angle	Cut Height (m)	Ditch Width[2] (m)	Cut Angle	Cut Height (m)	Ditch Width[2] (m)	Cut Angle	Cut Height (m)	Bench Width[3] (m)	
I	1704+590	1704+720	Left	72°	0 - 10	4.0	72°	0 - 10	4.0	72°	0 - 10	8.0	
(Argillite)				(1H:3V)	>10 - 15	5.0	(1H:3V)	>10 - 15	5.0	(1H:3V)			
					>15 - 20	6.0		>15 - 20	6.0				
													Bolting may be required for portions
II	1704+720	1704+815	Left	76°	>20 - 25	7.0	82°	>20 - 25	7.0	76°	0 - 10	8.0	of cut slope that are greater than
(Quartzite)				(1H:4V)	>25 - 30	8.0	(1H:7V)	>25 - 30	8.0	(1H:4V)			20 m above ditch and / or greater
					>30 - 35	9.0		30[4]	8.0[4]				15 m above midslope bench.
III a	1704+815	1705+020	Left	76°	>25 - 30	8.0	82°	>25 - 30	8.0	76°	0-10	8.0	Bolting may be required for portions
(Volcanics)				(1H:4V)	>30 - 35	9.0	(1H:7V)	30[4]	8.0[4]	(1H:4V)	>10-15	9.0	of cut slope that are greater than
					>35 - 40	10.0					>15 - 20	10.0	20 m above ditch and / or greater
					>40 - 45	11.0					>20 - 25	11.0	15 m above midslope bench.
					>45 - 50	12.0					>25 - 30	12.0	
					>50	13.0							
					0 - 10	4.0		0 - 10	4.0	n.a	n.a.		Cut down top of ridge between
III b	1704+825	1705+020	Right	68°	>10 - 15	5.0	68°	>10 - 15	5.0	n.a	n.a.		new & existing cut to decrease
(Volcanics)				(1H:2.5V)	>15 - 20	6.0	(1H:2.5V)	>15 - 20	6.0	n.a	n.a.		cut height and reduce ditch width.
IV	1705+020	1705+100	Left	76°	0 - 10	4.0	76°	0 - 10	4.0	76°	0 - 10	8.0	Bolting may be required for portions
(Argillite)				(1H:4V)	>10 - 15	5.0	(1H:4V)	>10 - 15	5.0	(1H:4V)	>10 - 15	9.0	of cut slope that are greater than
					>15 - 20	6.0		>15 - 20	6.0				20 m above ditch and / or greater
					>20 - 25	7.0		>20 - 25	7.0				15 m above midslope bench.
					>25 - 30	8.0		>25 - 30[4]	8.0[4]				

Notes:

1. Chainage location of start and end of each Design Sector at the new alignment backslope location are estimates only based on projection from existing rock cut.
2. Ditch Width is based on 4:1 (H:V) "recoverable" ditch sideslope without guard rail.
3. Bench Width is based on 5% bench slope towards cut slope above bench.
4. Maximum height to bench for Option 2 is 30 m.

Table 4 - Estimate of Rock Fragmentation and Bulking

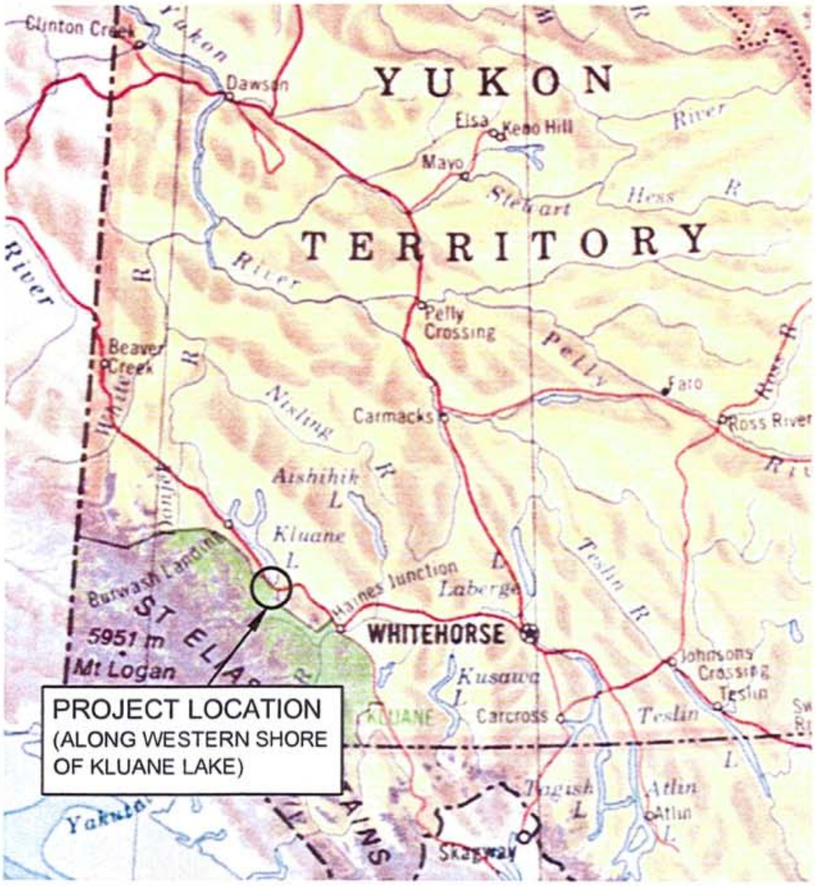
DESIGN SECTOR	LOCATION			TARGET GRADATION	STARTING BLAST PATTERN		ESTIMATED PRODUCTION[2]				COMMENTS
	From[1]	To[1]	Offset		Burden (m)	Spacing (m)	Oversize (%)	In Range (%)	Undersize (%)	BULKING FACTOR[3]	
I & IV	1704+590	1704+720	Left	Class 1 Rip-Rap	2.1	2.1	15	60	25		Platey texture of argillite may exceed
(Argillite)		and		Class 2 Rip-Rap	2.1	1.8	5	35	60	1.4	3:1 dimension ratio.
	1705+020	1705+100	Left	Class 3 Rip-Rap	2.4	1.8	0	15	85		
II	1704+720	1704+815	Left	Class 1 Rip-Rap	1.8	2.1	20	55	25		
(Quartzite)				Class 2 Rip-Rap	2.1	2.1	5	45	50	1.4	
				Class 3 Rip-Rap	2.4	1.8	0	25	75		
III a & III b	1704+815	1705+020	Left	Class 1 Rip-Rap	1.8	2.4	25	55	20		
(Volcanics)		and		Class 2 Rip-Rap	2.1	2.1	10	55	35	1.4	
	1704+825	1705+020	Right	Class 3 Rip-Rap	2.1	1.8	5	30	65		

Notes:

1. Chainage location of start and end of each Design Sector at the new alignment backslope location are estimates only based on projection from the existing rock cut.
2. Product gradation percentages are estimates only and may vary depending on many factors.
3. Estimated Bulking Factor is rock rock which is blasted and then re-compacted. Actual bulking will depend on gradation and compaction effort.

Nominal Particle Size (mm)	Percent Passing		
	Rip-Rap Class I	Rip-Rap Class II	Rip-Rap Class III
1200			100
900			<80
800		100	<50
600		<80	
500		<50	<20
450	100		
350	<80		
300	<50	<20	
200	<20		
100			

FIGURES



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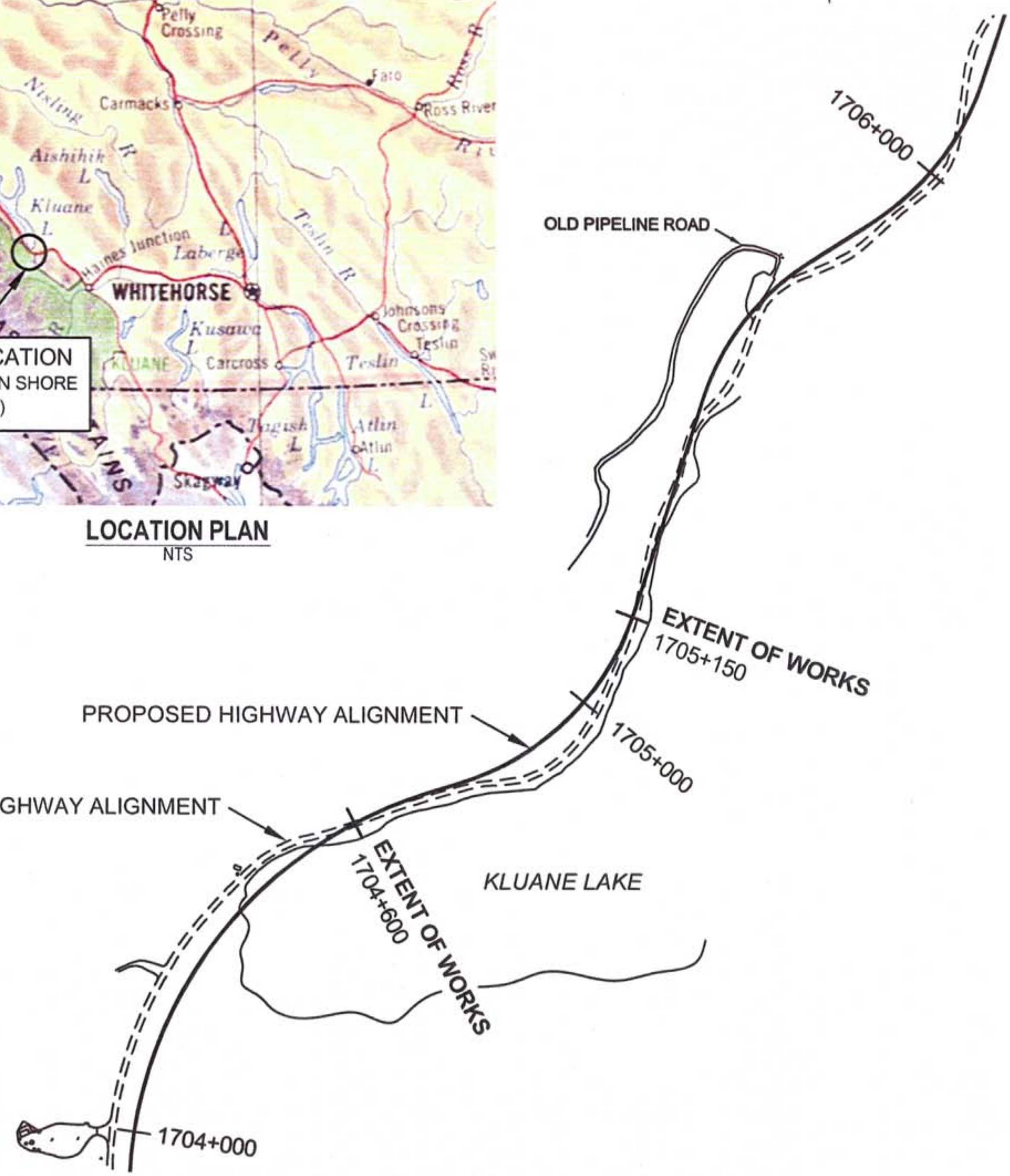

SITE PLAN

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EBA Engineering Consultants Ltd.		PROJECT	km 1705 - ALASKA HIGHWAY SHEEP MOUNTAIN ROCK CUT	
CLIENT: Yukon Highways and Public Works Property Management Branch		TITLE	SITE LOCATION PLAN	
DATE 2006/03/31	DWN. JSB / CHKD. SS	FILE NO.	1200128	FIGURE 1

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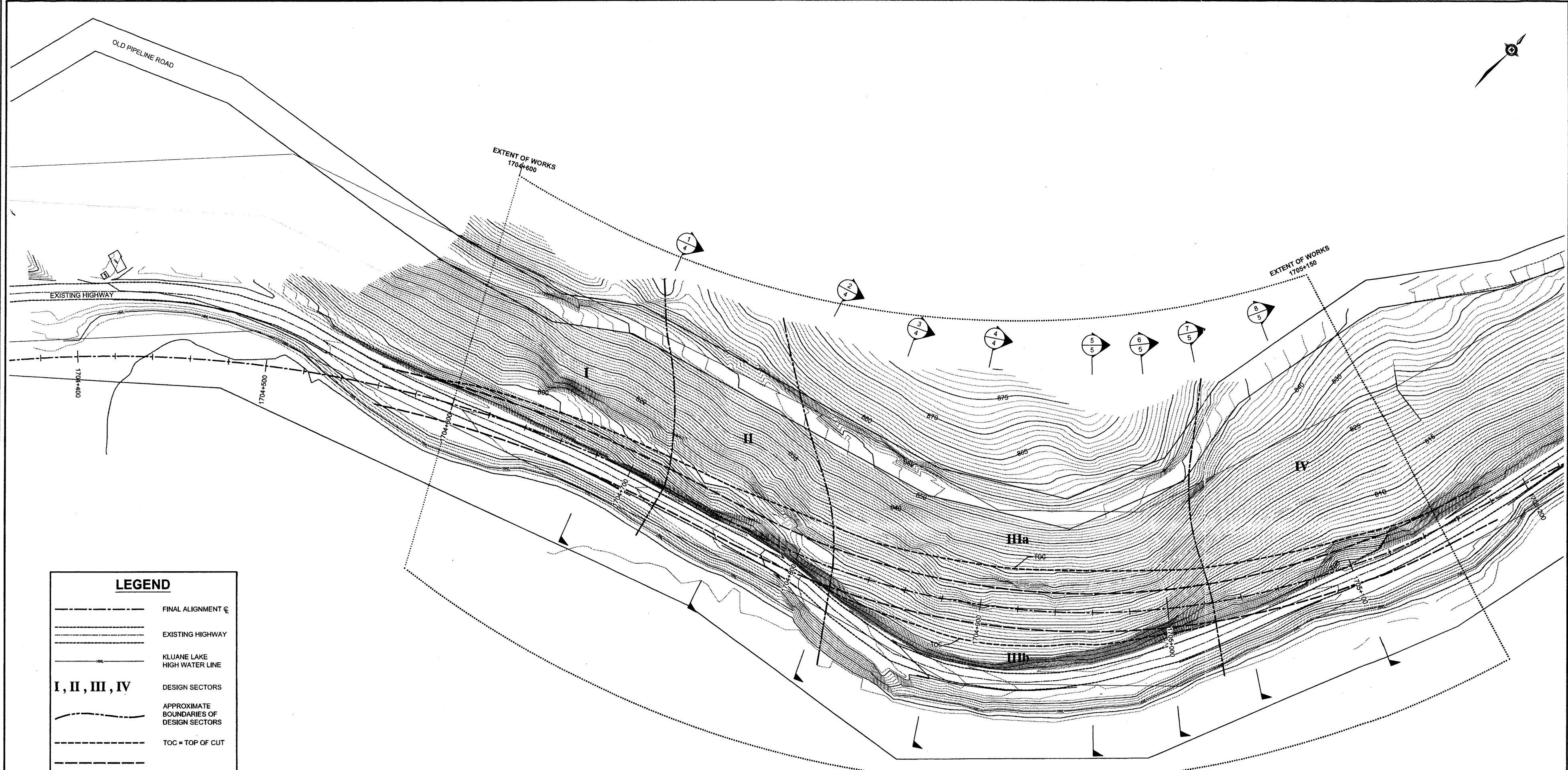

LEGEND

—·—·—	FINAL ALIGNMENT ℄
———	EXISTING HIGHWAY
——HWL——	KLUANE LAKE HIGH WATER LINE
I , II , III , IV	DESIGN SECTORS
—·—·—	APPROXIMATE BOUNDARIES OF DESIGN SECTORS
- - - -	TOC = TOP OF CUT
———	NEW HIGHWAY

NOTES:

1. ELEVATIONS & ALL UNITS ARE IN METRES.

1:1000 20 0 20 40m

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SCALE 1:1000 20 0 20

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DRAWN		DATE
CHECKED	SS	DATE 2006/03/31
APPROVED		DATE

PROFESSIONAL SEAL	DWG. No.	Description	Rev	Date	Description	Signature
			Pa	21/01/05	PRELIMINARY LAYOUT	
	REFERENCE DRAWINGS		REVISIONS			

Yukon Highways and Public Works Transportation Engineering Branch

Preliminary Alignment

Alaska Highway
Sheep Mountain
KM 1704+600 to KM 1705+150

SENIOR DESIGN ENGINEER		DATE		
FILE No.	PROJECT No.	REG.	DRAWING No.	
1200128	09901–____	1	FIGURE 2	Pa

CANCEL PRINTS BEARING PREVIOUS LETTER

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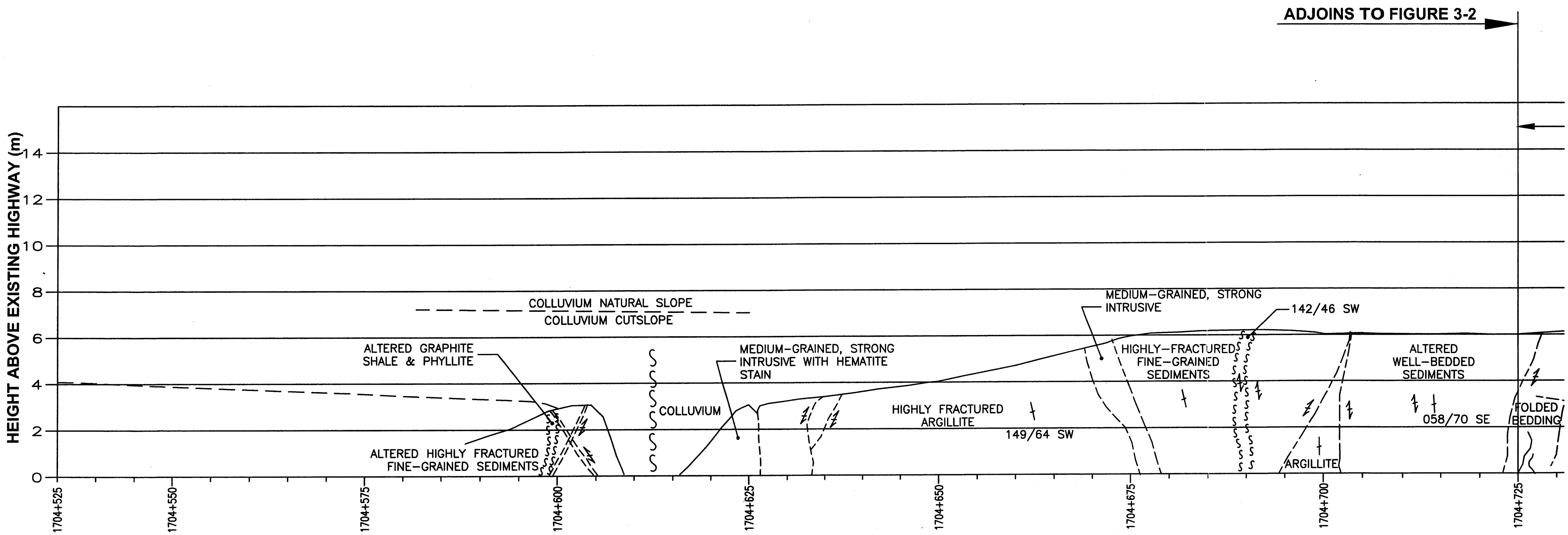

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- SHEAR ZONE/SHEARED JOINTING ORIENTATION
- BEDDING ORIENTATION
- GEOLOGY CONTACT, ASSUMED OR APPROXIMATE
- JOINTING, STRUCTURE CONTACTS
- 142/46 SW STRIKE, DIP, DIP DIRECTION

NOTE:
ALL DIMENSIONS ARE IN METRES.

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Property Management Branch

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DRAWN BY:	JSB/TRH
DATE ISSUED:	MARCH 31, 2006
PROJECT NUMBER:	1200128
FILE NAME:	
REVISION:	2

km 1705 - ALASKA HIGHWAY
SHEEP MOUNTAIN ROCK CUT

GEOLOGIC PROFILE OF
EXISTING ROCK SLOPE

SHEET 1 of 3

FIGURE NO. 3-1

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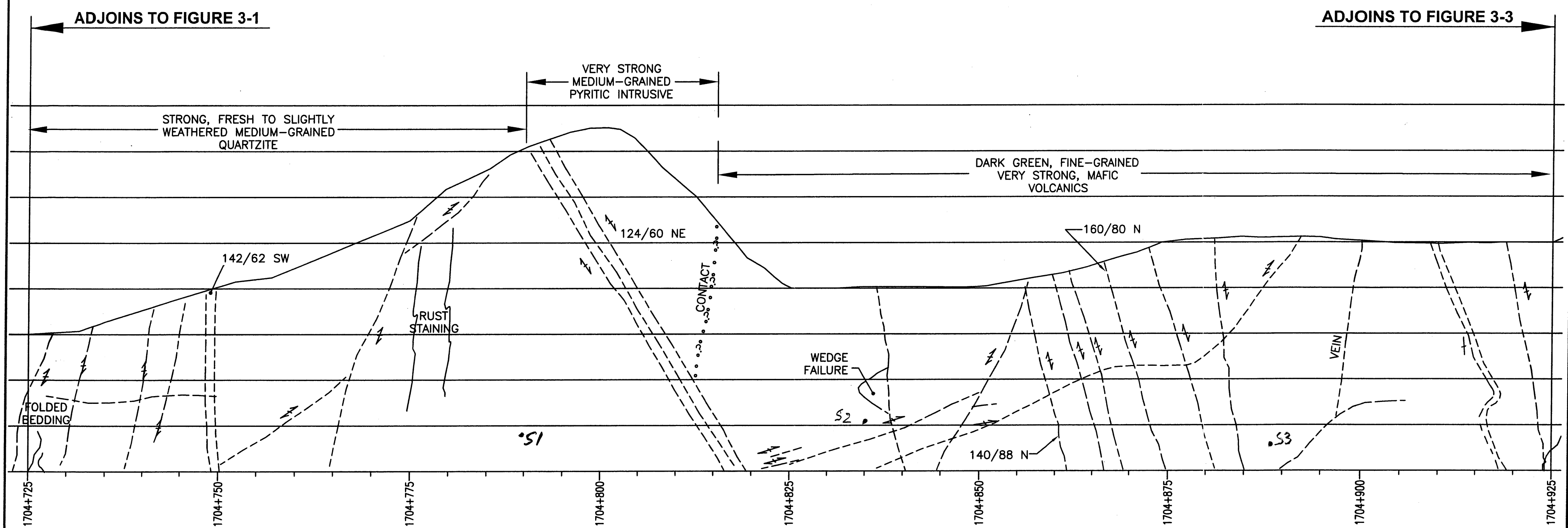

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- BEDDING ORIENTATION
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- JOINTING, STRUCTURE CONTACTS
- 142/46 SW STRIKE, DIP, DIP DIRECTION

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REVISION:	2

km 1705 - ALASKA HIGHWAY
SHEEP MOUNTAIN ROCK CUT

GEOLOGIC PROFILE OF
EXISTING ROCK SLOPE

SHEET 2 of 3

FIGURE NO. 3-2

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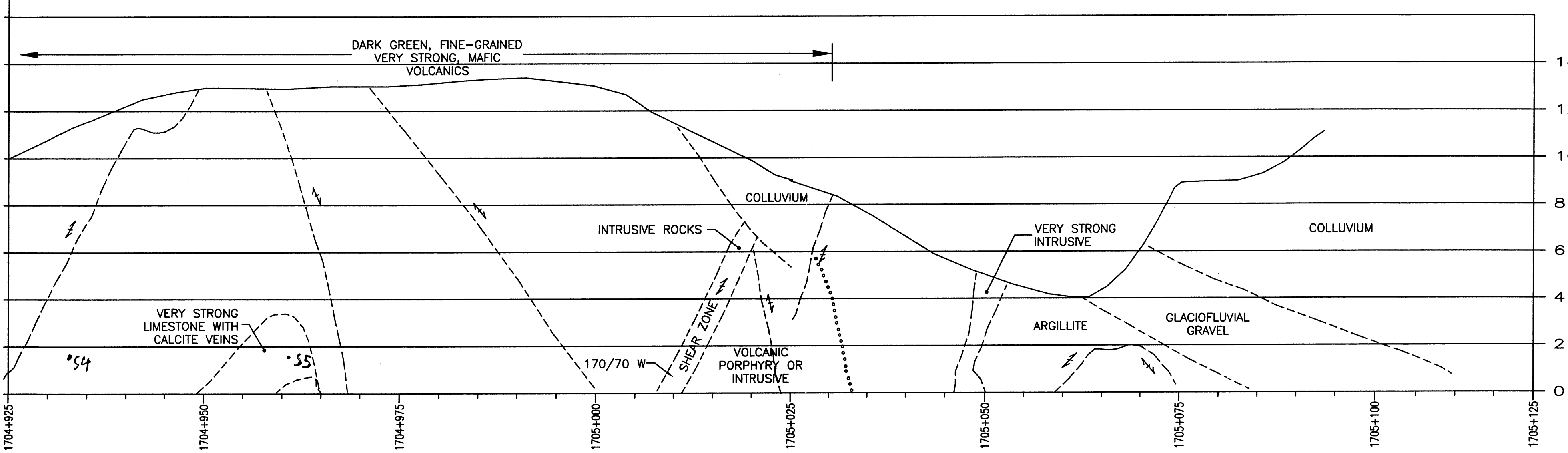
ADJOINS TO FIGURE 3-2



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- BEDDING ORIENTATION
- GEOLOGY CONTACT, ASSUMED OR APPROXIMATE
- JOINTING, STRUCTURE CONTACTS
- 142/46 SW STRIKE, DIP, DIP DIRECTION

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PROJECT NUMBER:	1200128
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REVISION:	2

km 1705 - ALASKA HIGHWAY
SHEEP MOUNTAIN ROCK CUT

GEOLOGIC PROFILE OF
EXISTING ROCK SLOPE

SHEET
3 of 3

FIGURE NO.
3-3

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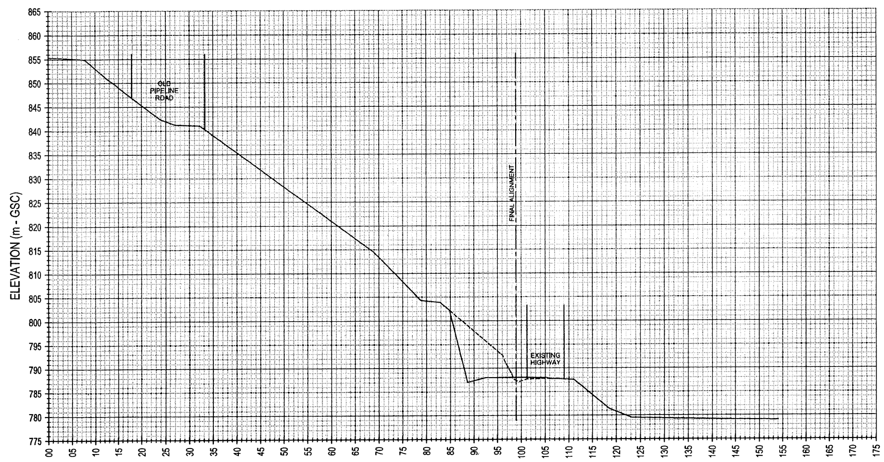

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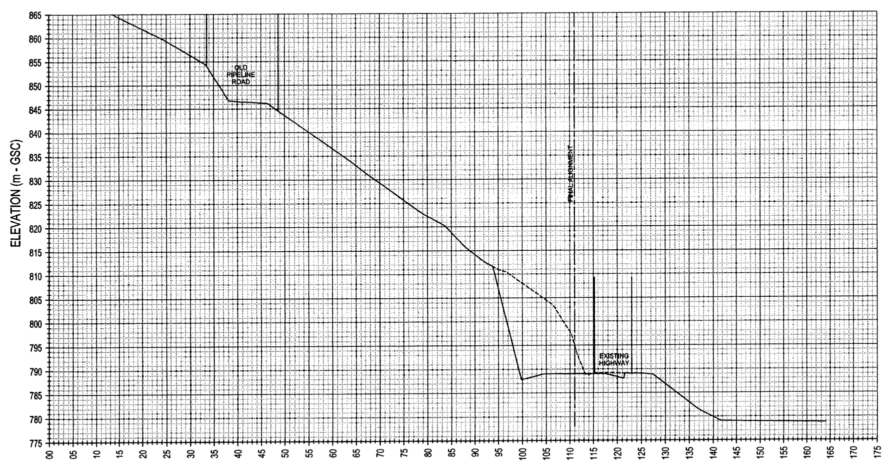

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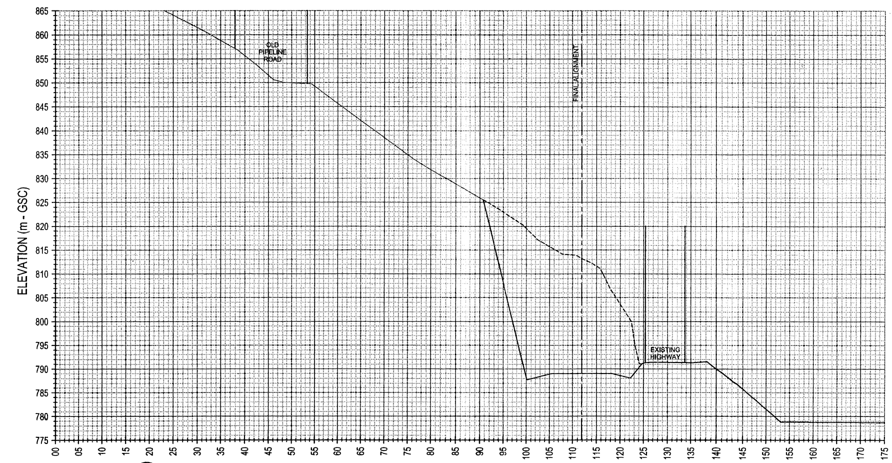

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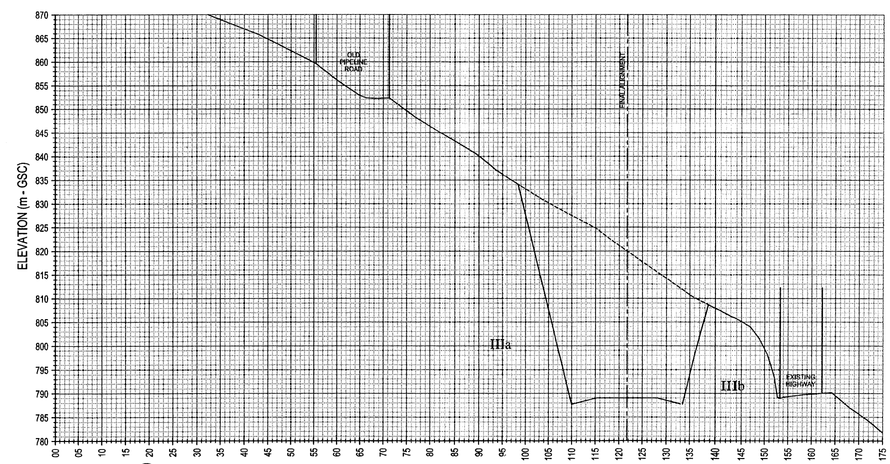

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SCALE

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DRAWN ______ DATE ______
CHECKED SS DATE 2006/03/21
APPROVED ______ DATE ______

PROFESSIONAL SEAL	DWG. No.	Description	Rev	Date	Description	Signature
	REFERENCE DRAWINGS		REVISIONS			

Yukon
Highways and Public Works
Transportation Engineering Branch

Preliminary Alignment
Alaska Highway
Sheep Mountain
KM 1704+600 to KM 1705+150
CROSS SECTIONS 1 – 4

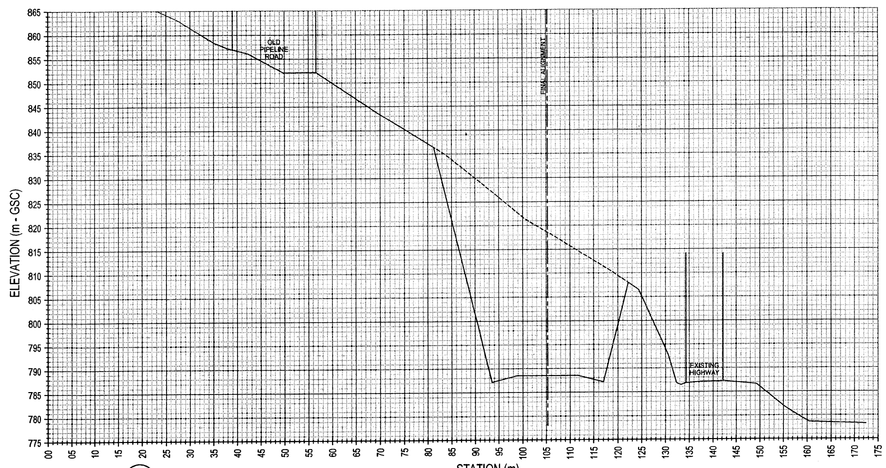
SENIOR DESIGN ENGINEER ______ DATE ______

FILE No.	PROJECT No.	REV.	DRAWING No.
1200128	09901–____	1	FIGURE 4

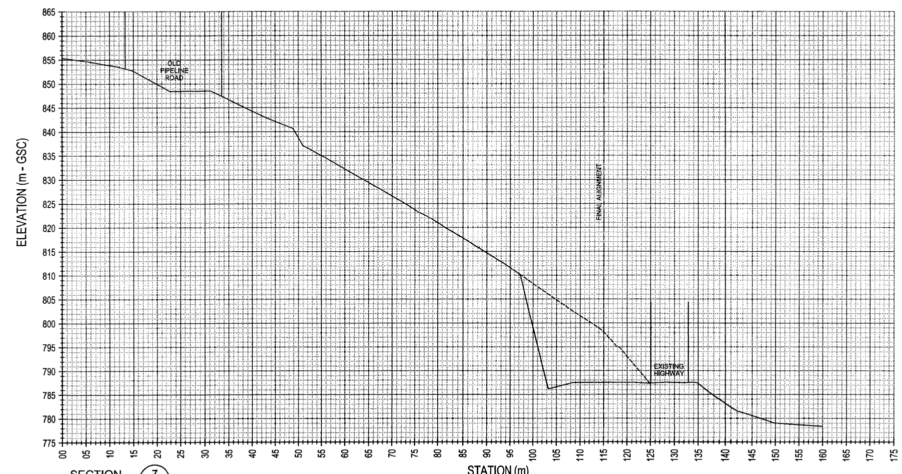
CANCEL PRINTS BEARING PREVIOUS LETTER

DATE: 2006/03/31 – 4:01pm
PATH: K:\0201-WH\1200128\2006-03-31_Final Report\\1200128_FIG 4-5.dwg
LAYOUT: FIG-4

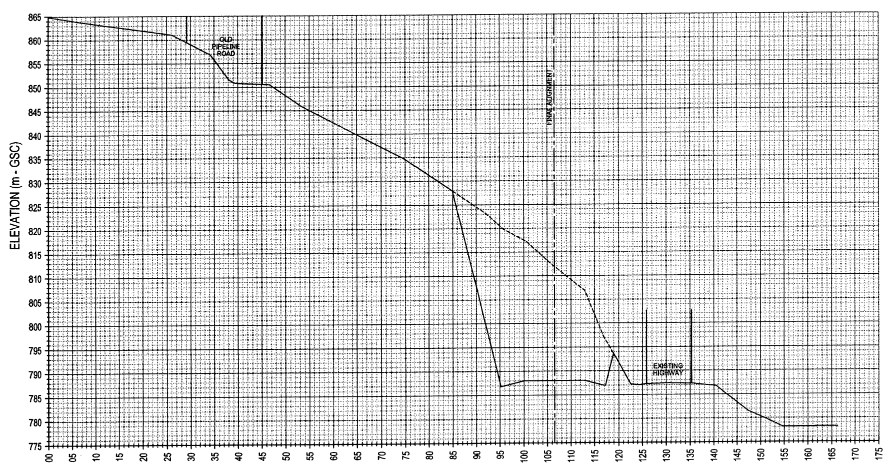
K:\0201-WH\1200128\2006-03-31_Final Report\1200128_FIG 4-5.dwg [FIG-4] March 31, 2006 - 4:01pm



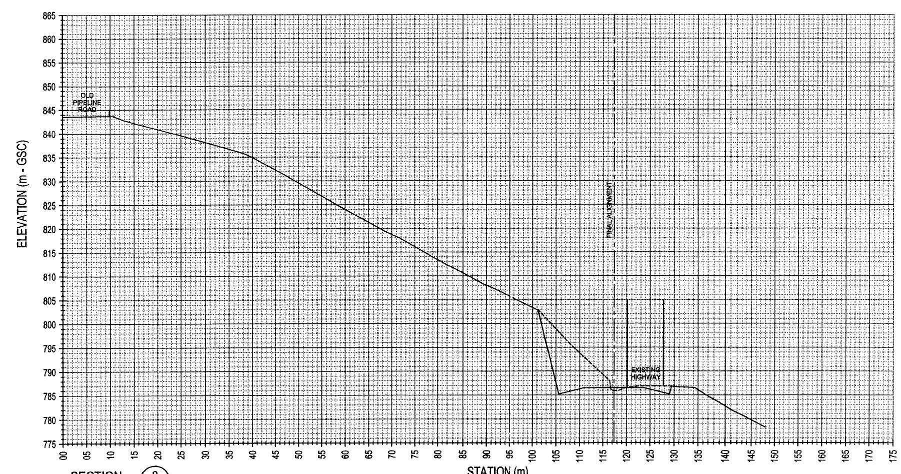
SECTION 5/2 SECTION AT STA. 1704+960
SCALE 1 : 500



SECTION 7/2 SECTION AT STA. 1705+040
SCALE 1 : 500



SECTION 6/2 SECTION AT STA. 1705+000
SCALE 1 : 500



SECTION 8/2 SECTION AT STA. 1705+100
SCALE 1 : 500

EBA Engineering Consultants Ltd.

Yukon Highways and Public Works Transportation Engineering Branch

SCALE

DESIGNED ____ DATE ____
DRAWN ____ DATE ____
CHECKED SS DATE 2006/03/31
APPROVED ____ DATE ____

Preliminary Alignment

Alaska Highway
Sheep Mountain
KM 1704+600 to KM 1705+150
CROSS SECTIONS 5 – 8

PROFESSIONAL SEAL	DWG. No.	Description	Rev	Date	Description	Signature
	REFERENCE DRAWINGS		REVISIONS			

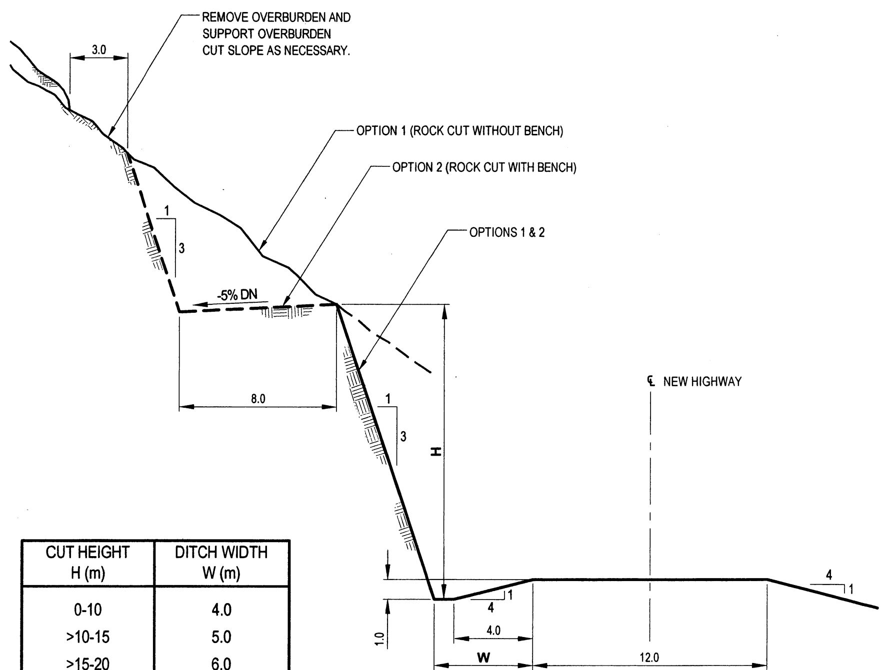
SENIOR DESIGN ENGINEER ____ DATE ____

FILE No.	PROJECT No.	REV.	DRAWING No.
1200128	09901–____	1	FIGURE 5

Pa

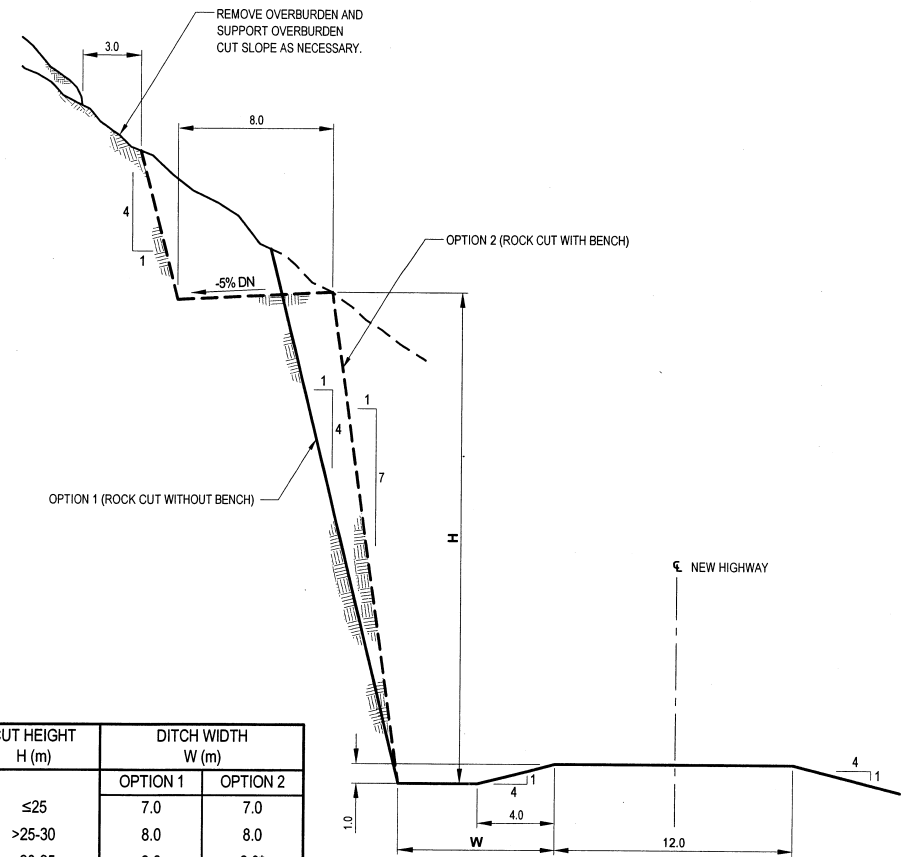
CANCEL PRINTS BEARING PREVIOUS LETTER

DATE: 2006/03/31 – 4:03pm
PATH: K:\0201-WH\1200128\2006-03-31_Final Report\1200128_FIG 4-5.dwg
LAYOUT: FIG-5
K:\0201-WH\1200128\2006-03-31_Final Report\1200128_FIG 4-5.dwg [FIG-5] March 31, 2006 – 4:03pm thennecker



CUT HEIGHT H (m)	DITCH WIDTH W (m)
0-10	4.0
>10-15	5.0
>15-20	6.0

DESIGN SECTOR I (STA. 1704+590 TO 1704+720)



CUT HEIGHT H (m)	DITCH WIDTH W (m)	
	OPTION 1	OPTION 2
≤25	7.0	7.0
>25-30	8.0	8.0
>30-35	9.0	8.0*

* BENCH AT MAXIMUM 30m HEIGHT

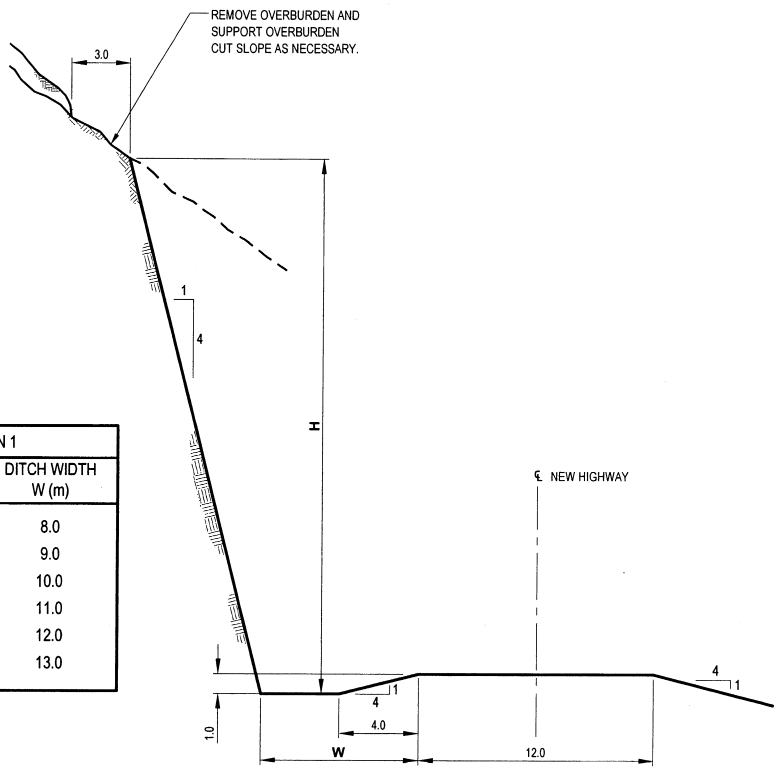
DESIGN SECTOR II (STA. 1704+720 TO 1704+815)

NOTE:
ALL DIMENSION ARE IN METRES,
UNLESS NOTED OTHERWISE.

SCALE 1:250

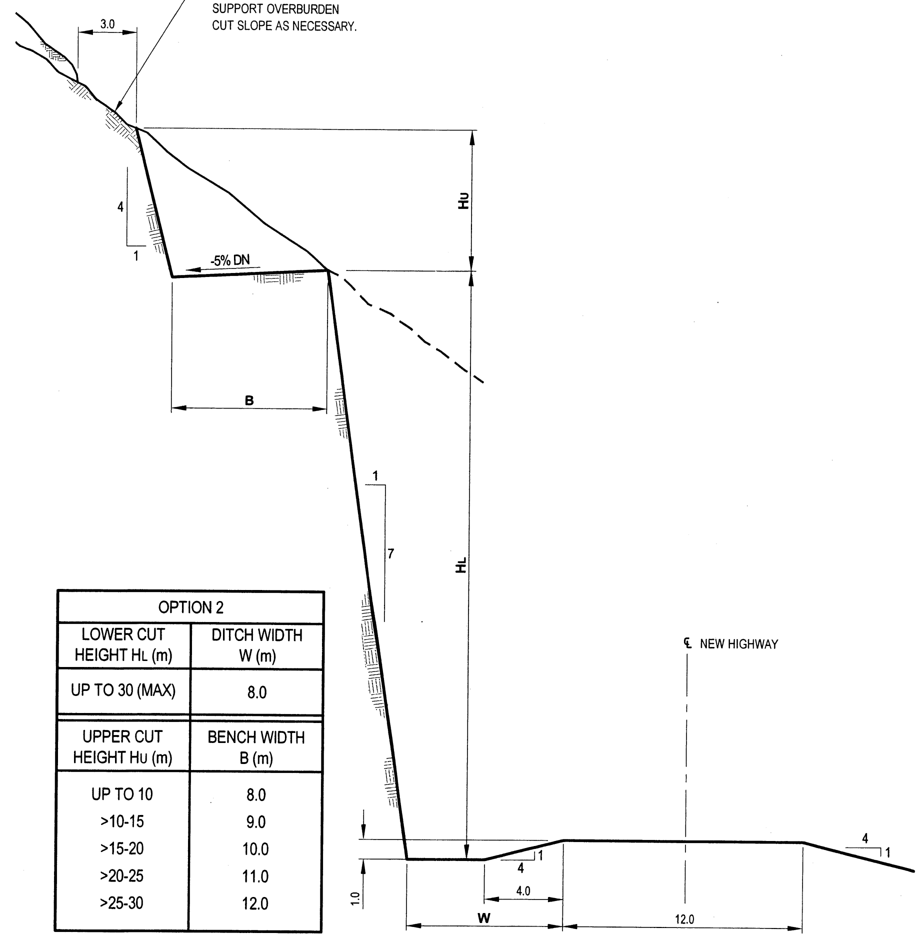
EBA Engineering Consultants Ltd.		PROJECT	km 1705 - ALASKA HIGHWAY SHEEP MOUNTAIN ROCK CUT
CLIENT	Yukon Highways and Public Works Property Management Branch	TITLE	ROCK CUT DESIGN DESIGN SECTORS I & II
DATE 2006/03/31	DWN. TRH / CHKD. SS	FILE NO. 1200128	FIGURE 6

K:\0201-WHI\1200128\2006-03-31_Final Report\1200128_FIG 1,6-11.dwg [FIG 6] March 31, 2006 - 4:10pm tbennecker



OPTION 1	
CUT HEIGHT H (m)	DITCH WIDTH W (m)
>25-30	8.0
>30-35	9.0
>35-40	10.0
>40-45	11.0
>45-50	12.0
>50	13.0

DESIGN SECTOR IIIa (STA. 1704+815 TO 1705+020)
OPTION 1 (ROCK CUT WITHOUT BENCH)



OPTION 2	
LOWER CUT HEIGHT HL (m)	DITCH WIDTH W (m)
UP TO 30 (MAX)	8.0
UPPER CUT HEIGHT Hu (m)	BENCH WIDTH B (m)
UP TO 10	8.0
>10-15	9.0
>15-20	10.0
>20-25	11.0
>25-30	12.0

DESIGN SECTOR IIIa (STA. 1704+815 TO 1705+020)
OPTION 2 (ROCK CUT WITH BENCH)

NOTE:
ALL DIMENSION ARE IN METRES,
UNLESS NOTED OTHERWISE.

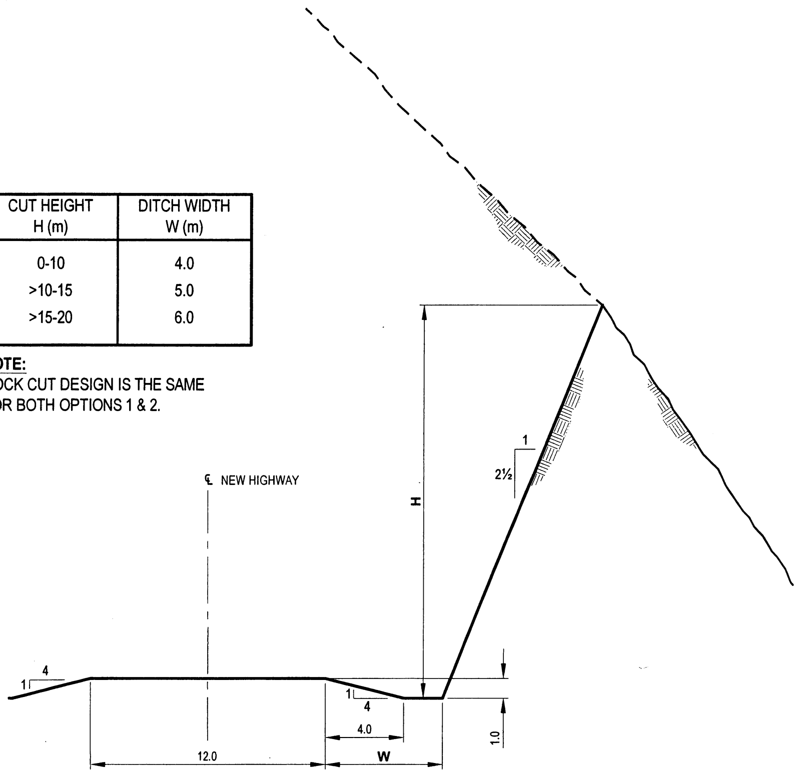
SCALE 1:250
2 0 2 6 10m

EBA Engineering Consultants Ltd.				PROJECT	km 1705 - ALASKA HIGHWAY SHEEP MOUNTAIN ROCK CUT
CLIENT: Yukon Highways and Public Works Property Management Branch				TITLE	ROCK CUT DESIGN DESIGN SECTOR IIIa (LEFT) (OPTIONS 1 & 2)
DATE 2006/03/31	DWN. TRH	CHKD. SS		FILE NO. 1200128	FIGURE 7

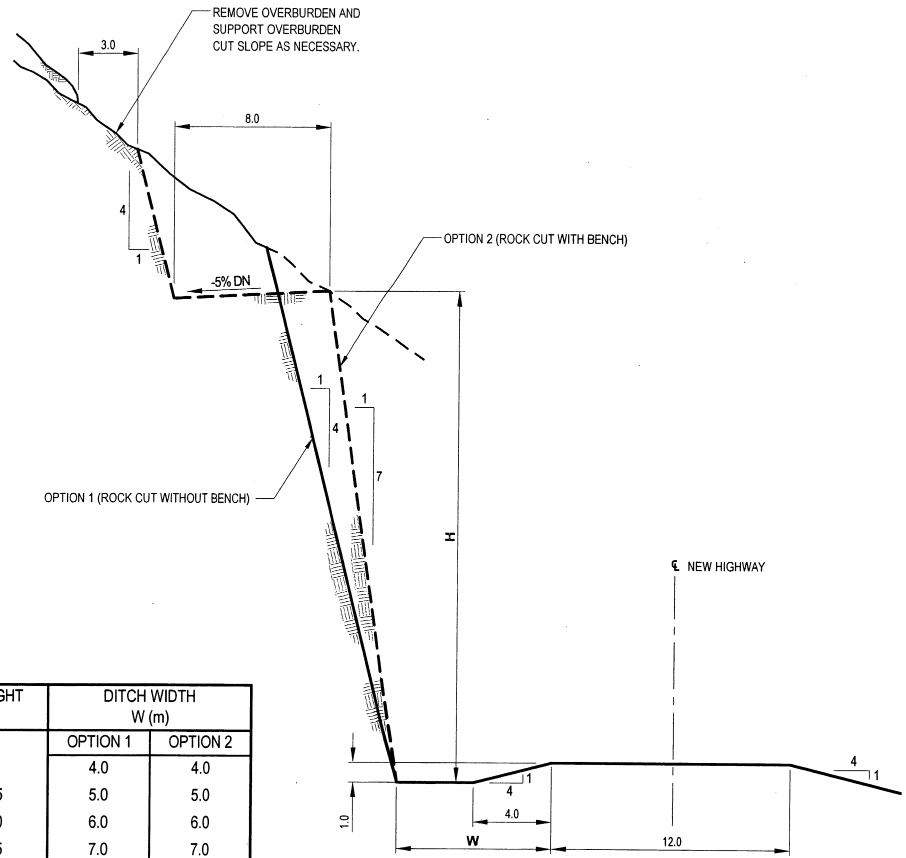
K:\0201-WHI\1200128\2006-03-31_Final Report\1200128_FIG 1,6-11.dwg [FIG 7] March 31, 2006 - 4:10pm thennecker

CUT HEIGHT H (m)	DITCH WIDTH W (m)
0-10	4.0
>10-15	5.0
>15-20	6.0

NOTE:
ROCK CUT DESIGN IS THE SAME FOR BOTH OPTIONS 1 & 2.



DESIGN SECTOR IIIb (STA. 1704+825 TO 1705+020)



CUT HEIGHT H (m)	DITCH WIDTH W (m)	
	OPTION 1	OPTION 2
0-10	4.0	4.0
>10-15	5.0	5.0
>15-20	6.0	6.0
>20-25	7.0	7.0
>25-30	8.0	8.0*

* BENCH AT MAXIMUM 30m HEIGHT

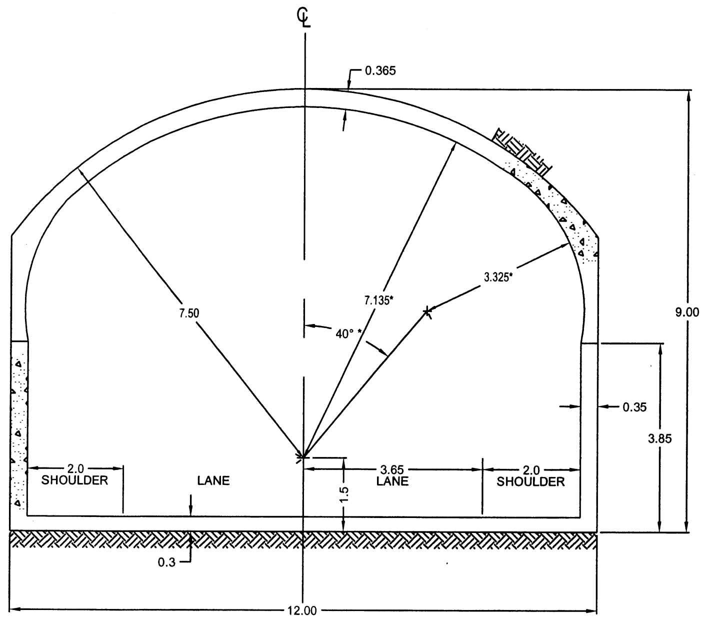
DESIGN SECTOR IV (STA. 1705+020 TO 1705+100)

NOTE:
ALL DIMENSION ARE IN METRES, UNLESS NOTED OTHERWISE.

SCALE 1:250
2 0 2 6 10m

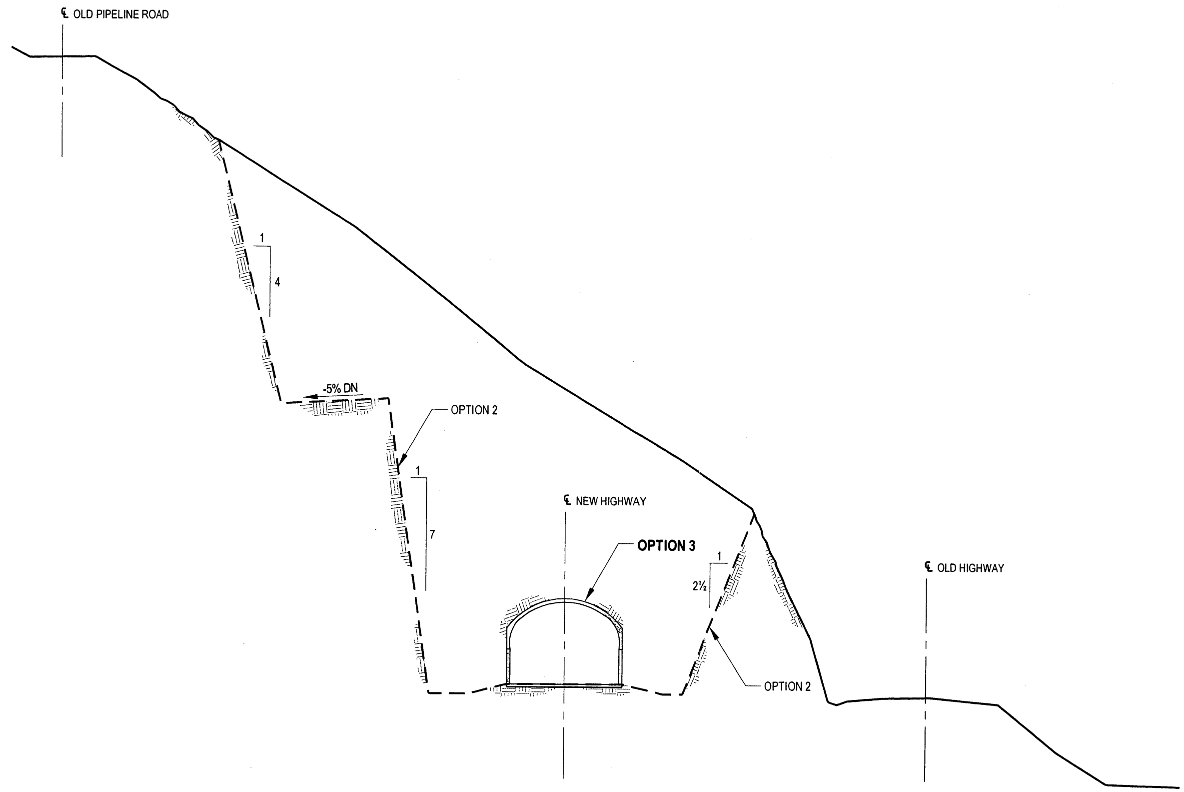
EBA Engineering Consultants Ltd.				PROJECT	km 1705 - ALASKA HIGHWAY SHEEP MOUNTAIN ROCK CUT	
CLIENT	Yukon Highways and Public Works Property Management Branch			TITLE	ROCK CUT DESIGN DESIGN SECTORS IIIb (RIGHT) & IV (OPTIONS 1 & 2)	
DATE 2006/03/31	DWN. TRH	CHKD. SS		FILE NO.	1200128	FIGURE 8

K:\0201-WHI\1200128\2006-03-31_Final Report\1200128_FIG 1,6-11.dwg [FIG 8] March 31, 2006 - 4:10pm tnennecker



CONCEPTUAL TUNNEL CROSS-SECTION

SCALE 1:100



COMPARISON BETWEEN CUT AND TUNNEL OPTIONS CROSS-SECTION (STA. 1704+960)

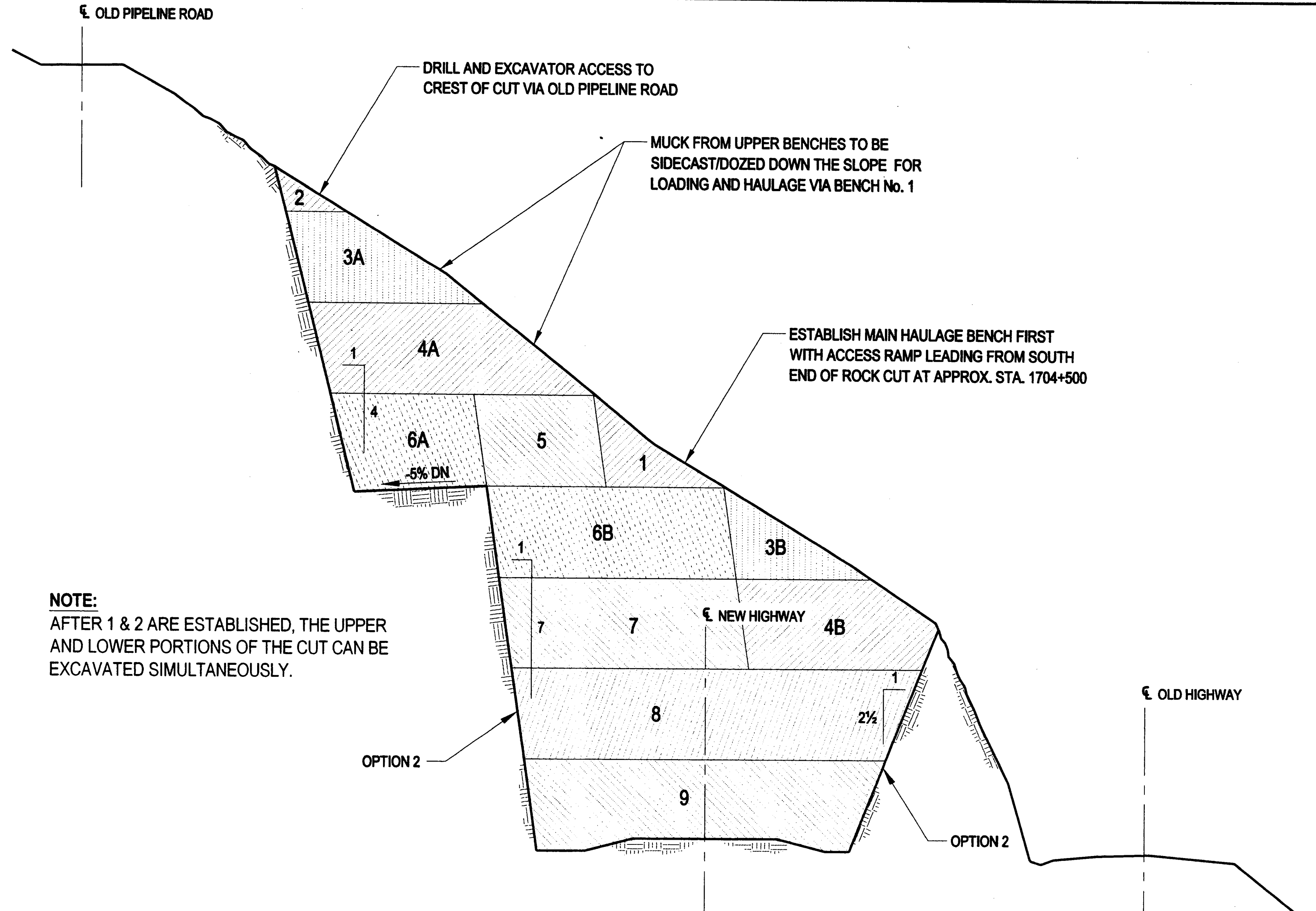
SCALE 1:500

NOTES:

1. ALL DIMENSION ARE IN METRES, UNLESS NOTED OTHERWISE.
2. ARCH DIMENSIONS BASED ON ARMTEC CONSTRUCTION PRODUCTS LOW PROFILE ARCH NO. 123A42.

EBA Engineering Consultants Ltd.		PROJECT	km 1705 - ALASKA HIGHWAY SHEEP MOUNTAIN ROCK CUT
CLIENT	Yukon Highways and Public Works Property Management Branch	TITLE	CONCEPTUAL TUNNEL DESIGN DESIGN SECTOR III - OPTION 3
DATE 2006/03/31	DWN. TRH/JSB	CHKD. SS	FILE NO. 1200128 — FIGURE 9

K:\0201-WHI\1200128\2006-03-31_Final Report\1200128_FIG 1,6-11.dwg [FIG 10] March 31, 2006 - 4:11pm thennecker



NOTE:
AFTER 1 & 2 ARE ESTABLISHED, THE UPPER AND LOWER PORTIONS OF THE CUT CAN BE EXCAVATED SIMULTANEOUSLY.

CROSS-SECTION (STA. 1704+960)

SCALE 1:500
5 0 5 10 20m

EBA Engineering Consultants Ltd.				PROJECT km 1705 - ALASKA HIGHWAY SHEEP MOUNTAIN ROCK CUT	
CLIENT Yukon Highways and Public Works Property Management Branch				TITLE CONCEPTUAL EXCAVATION SEQUENCE DESIGN SECTOR III	
DATE 2006/03/31	DWN. TRH	CHKD. SS		FILE NO. 1200128	FIGURE 10

$$\beta = \tan^{-1}\left[\frac{h * \tan\gamma}{(h - d * \tan\gamma)}\right]$$

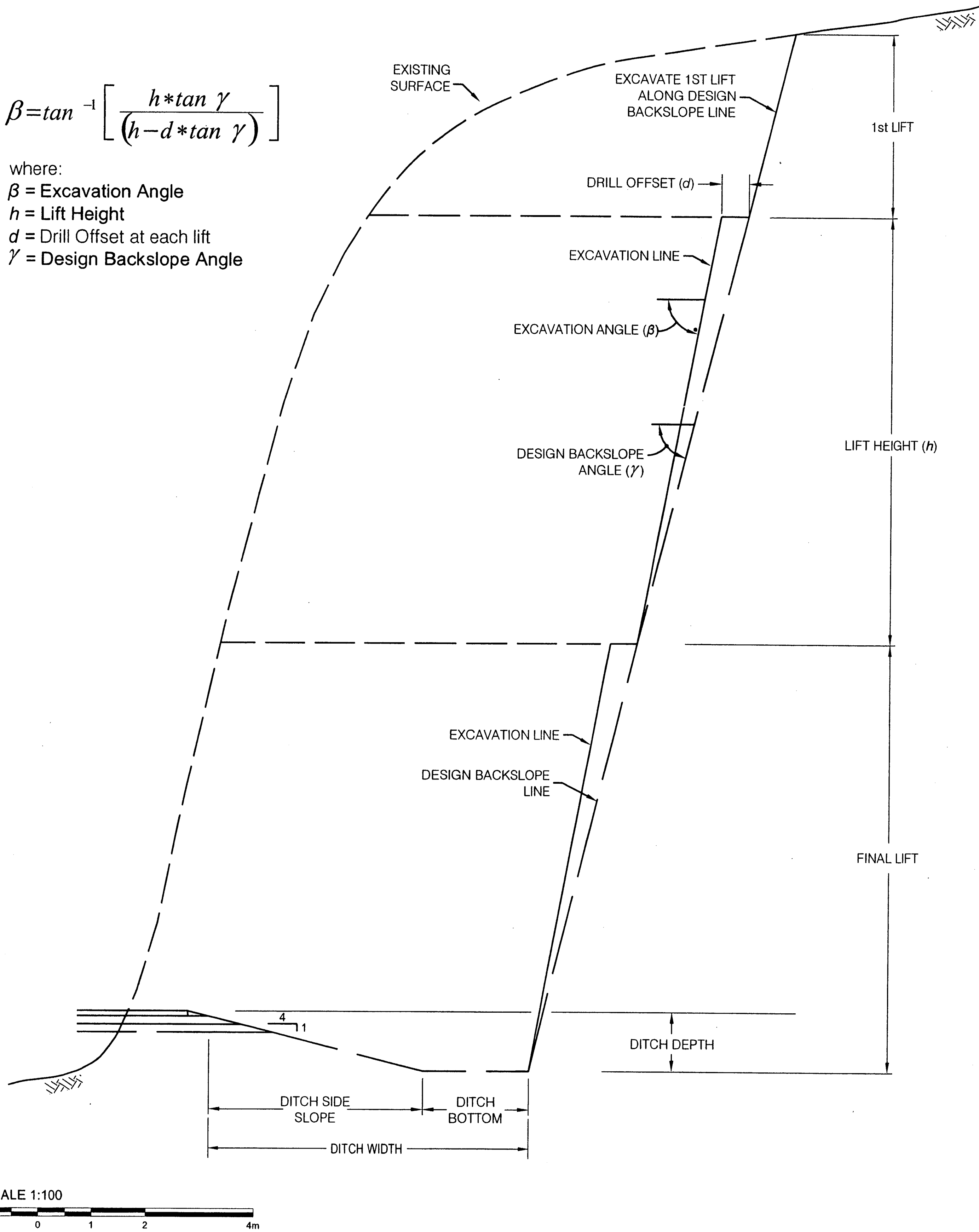
where:

β = Excavation Angle

h = Lift Height

d = Drill Offset at each lift

γ = Design Backslope Angle



EBA Engineering Consultants Ltd.		PROJECT	km 1705 - ALASKA HIGHWAY SHEEP MOUNTAIN ROCK CUT
CLIENT Yukon Highways and Public Works Property Management Branch		TITLE	CALCULATION OF BACKSLOPE EXCAVATION ANGLE
DATE 2006/03/31	DWN. JSB	CHKD. SS	FILE NO. 1200128 — FIGURE 11

APPENDIX

APPENDIX A PHOTOGRAPHS

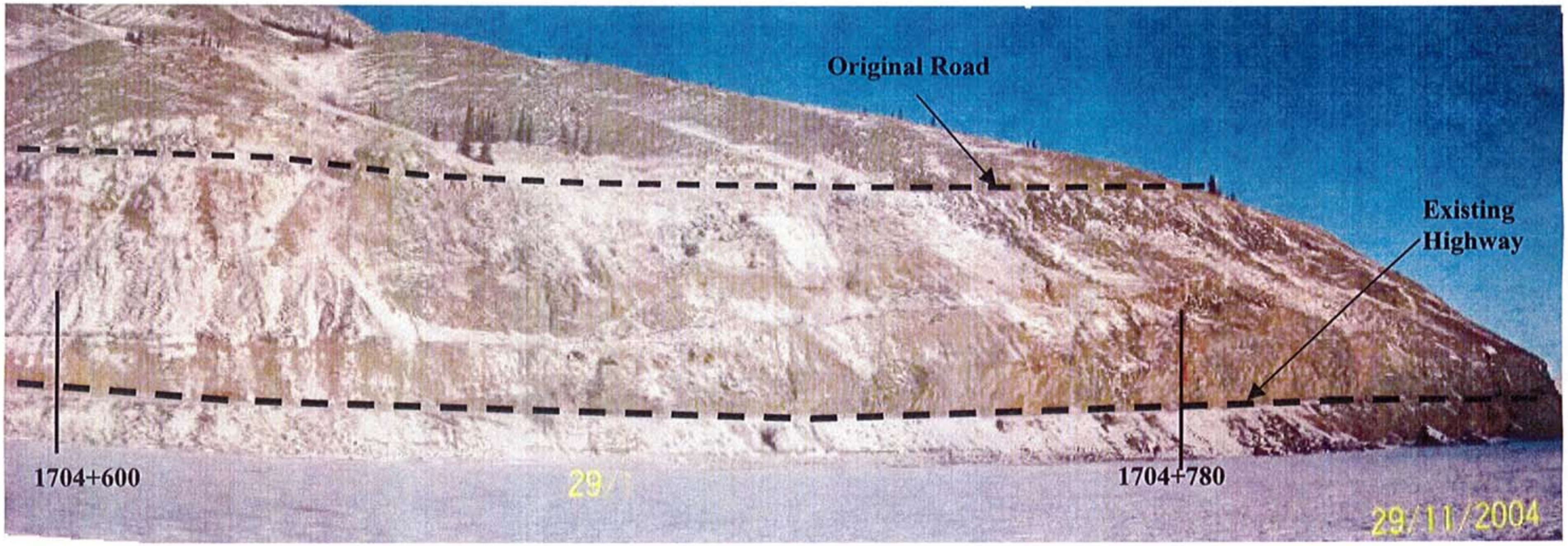


Photo 1: Sheep Mountain Rock Cut – Overview

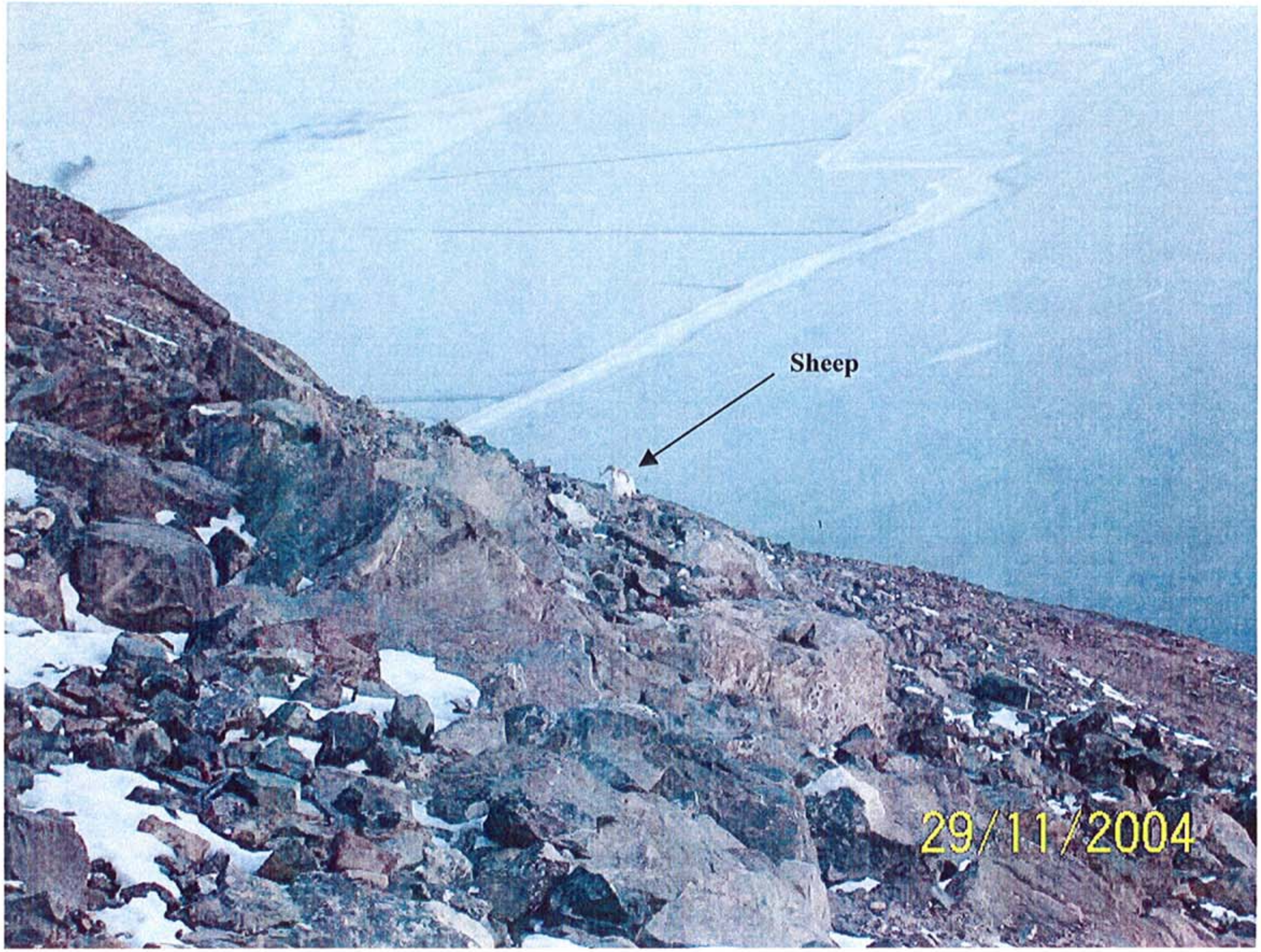


Photo 2: Moderately steep gradient slopes of colluvium and bedrock upslope of existing highway rock cut.

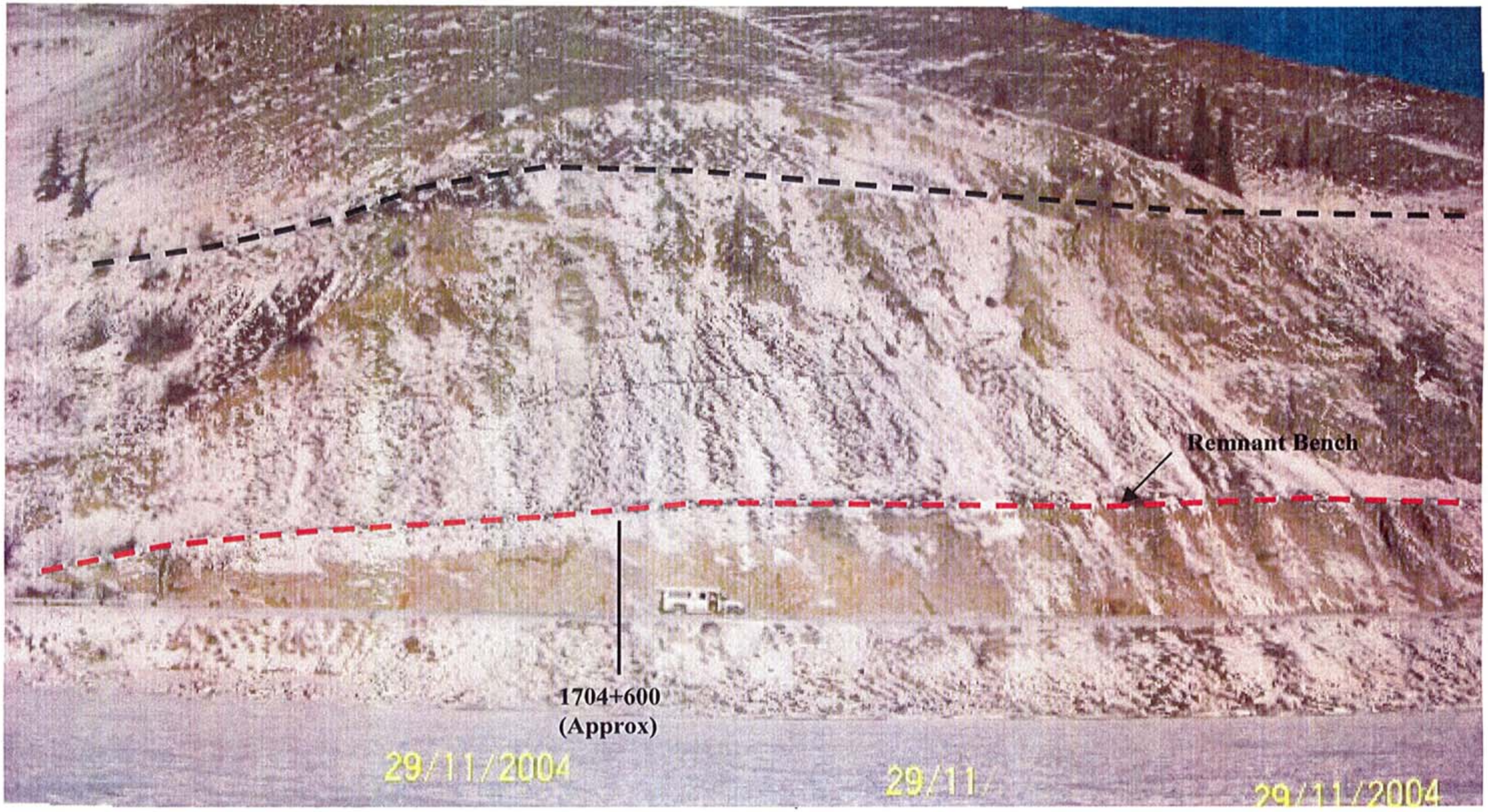


Photo 3 – Note Remnant bench or construction access trail from construction of existing rock cut.

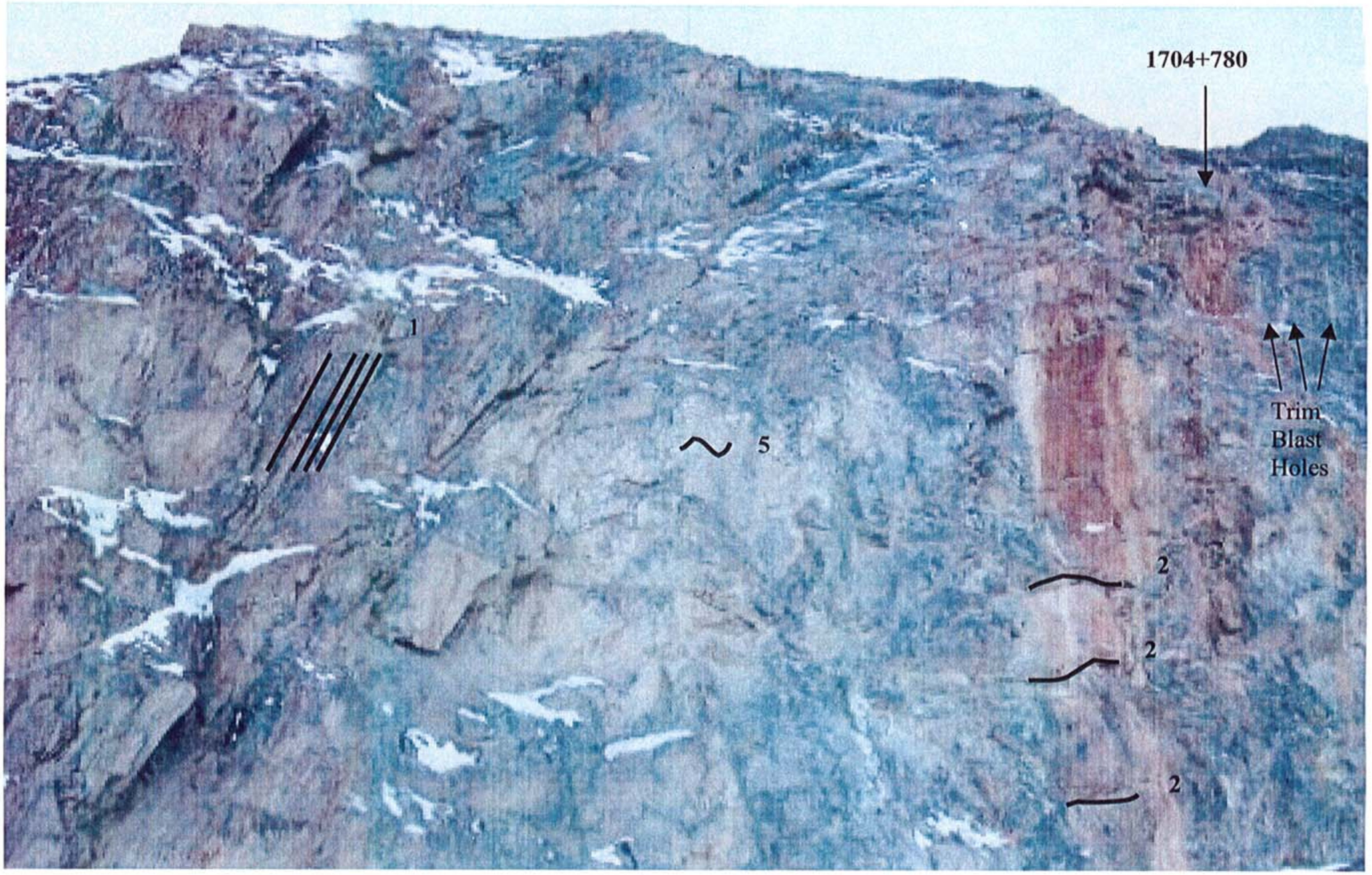


Photo 4: Structural Domain 2 (Quartzite) with selected joint sets highlighted. Note traces of small diameter trim blast holes spaced at approximately 0.8 m apart.

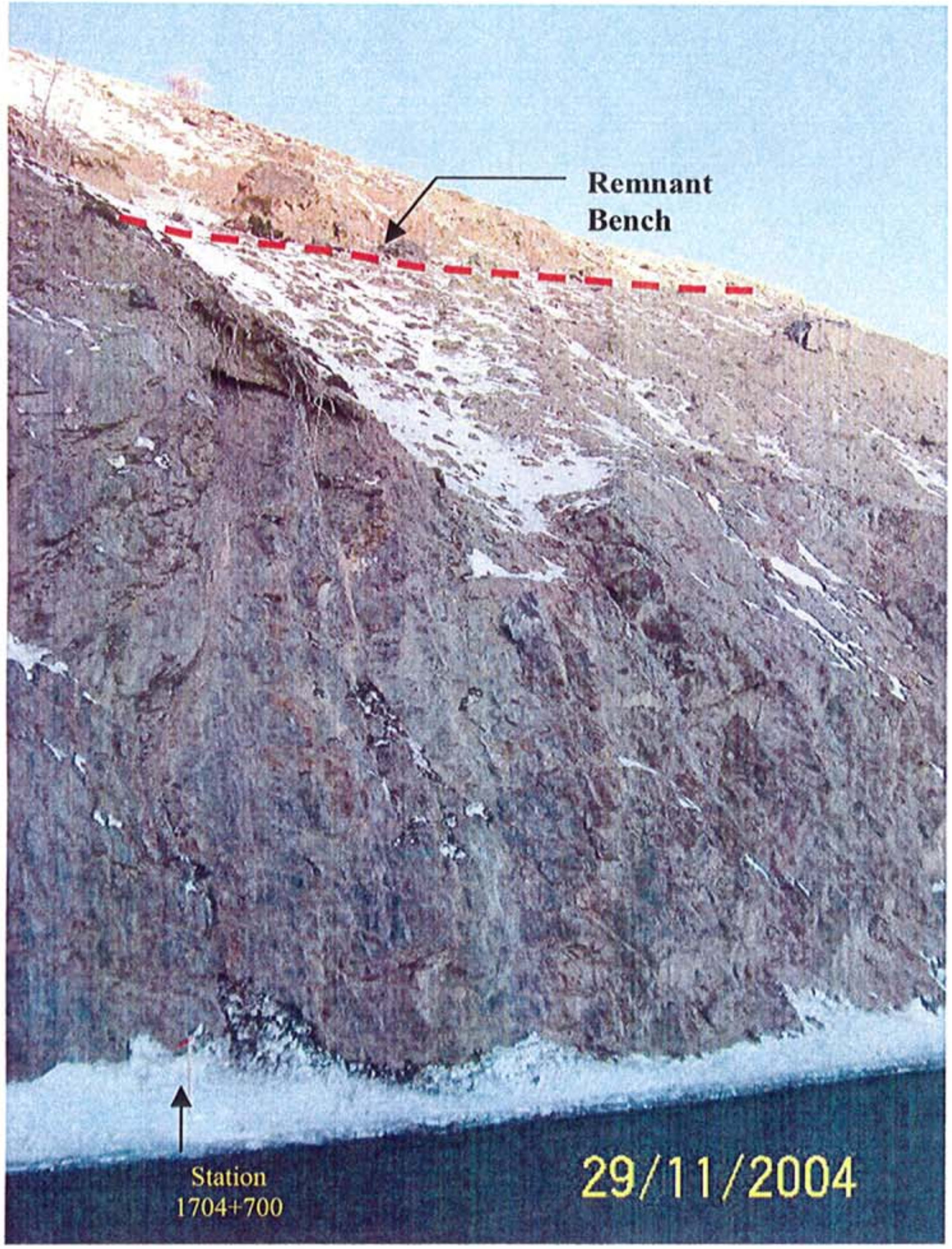


Photo 5: Northern end of the Argillite in Design Sector 1.

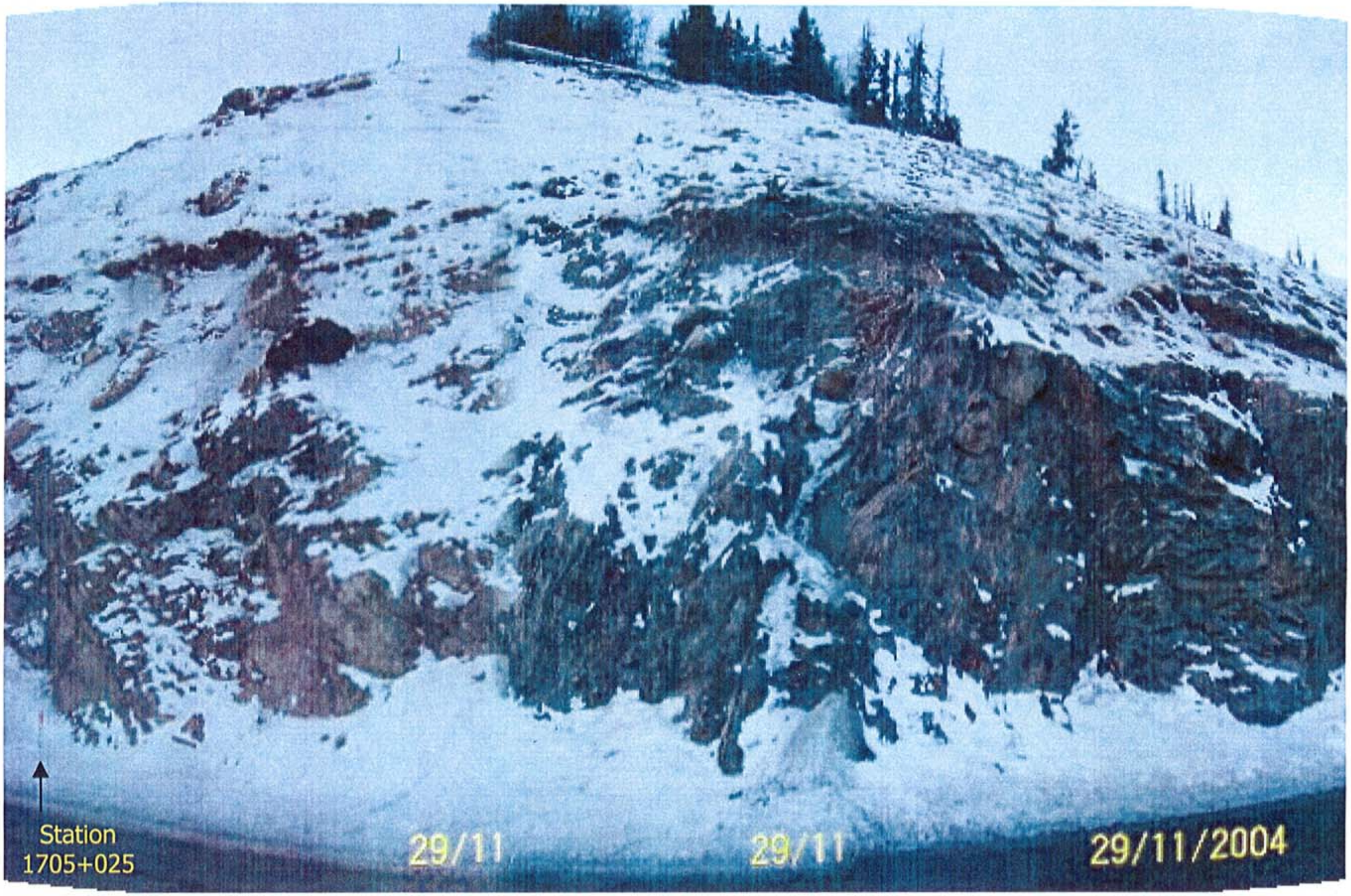


Photo 6: Southern end of the argillite in Design Sector 4.

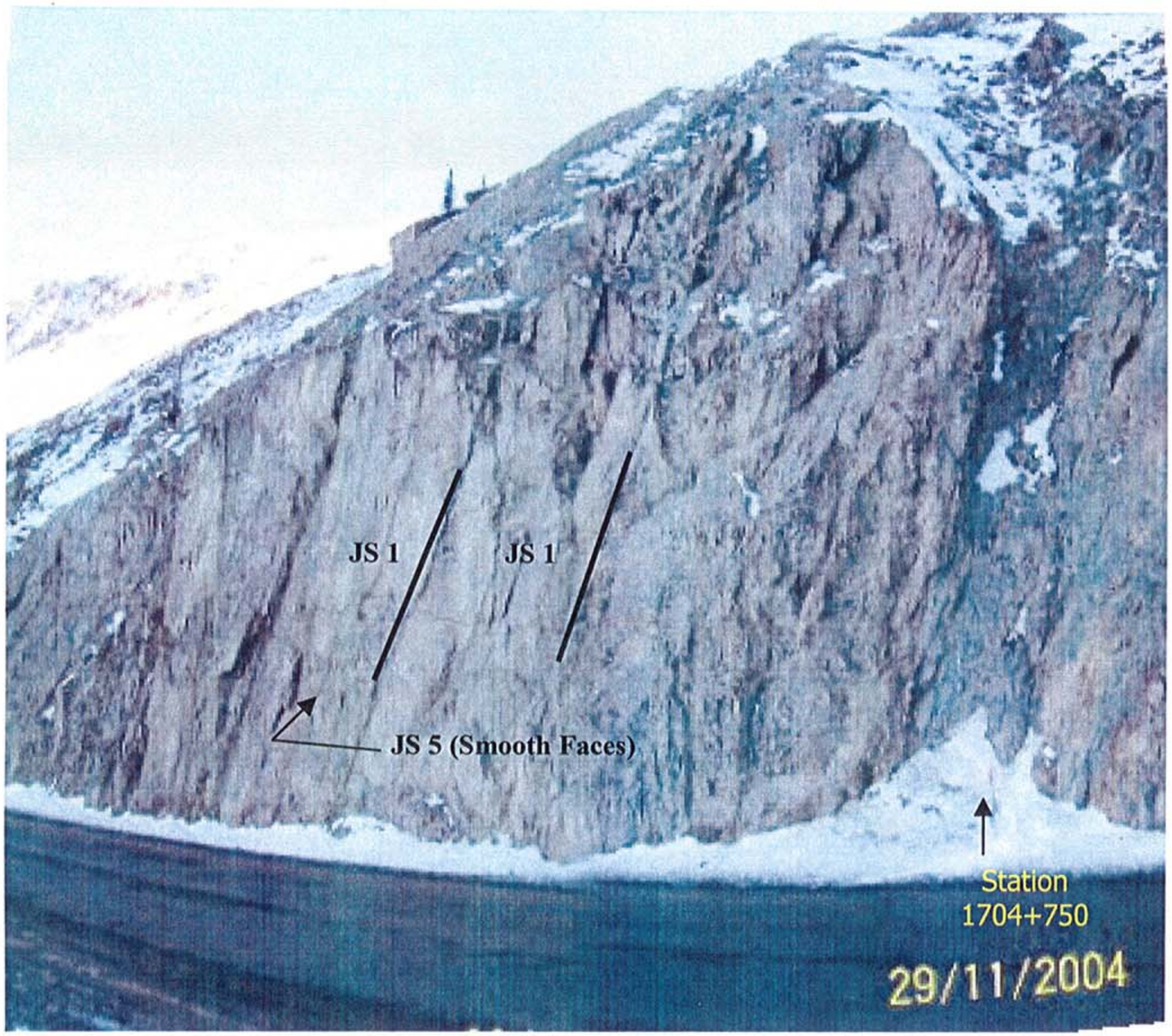


Photo 7: Locally pronounced joints of Joint Sets 1 and 5 in the quartzite in Design Sector 2.

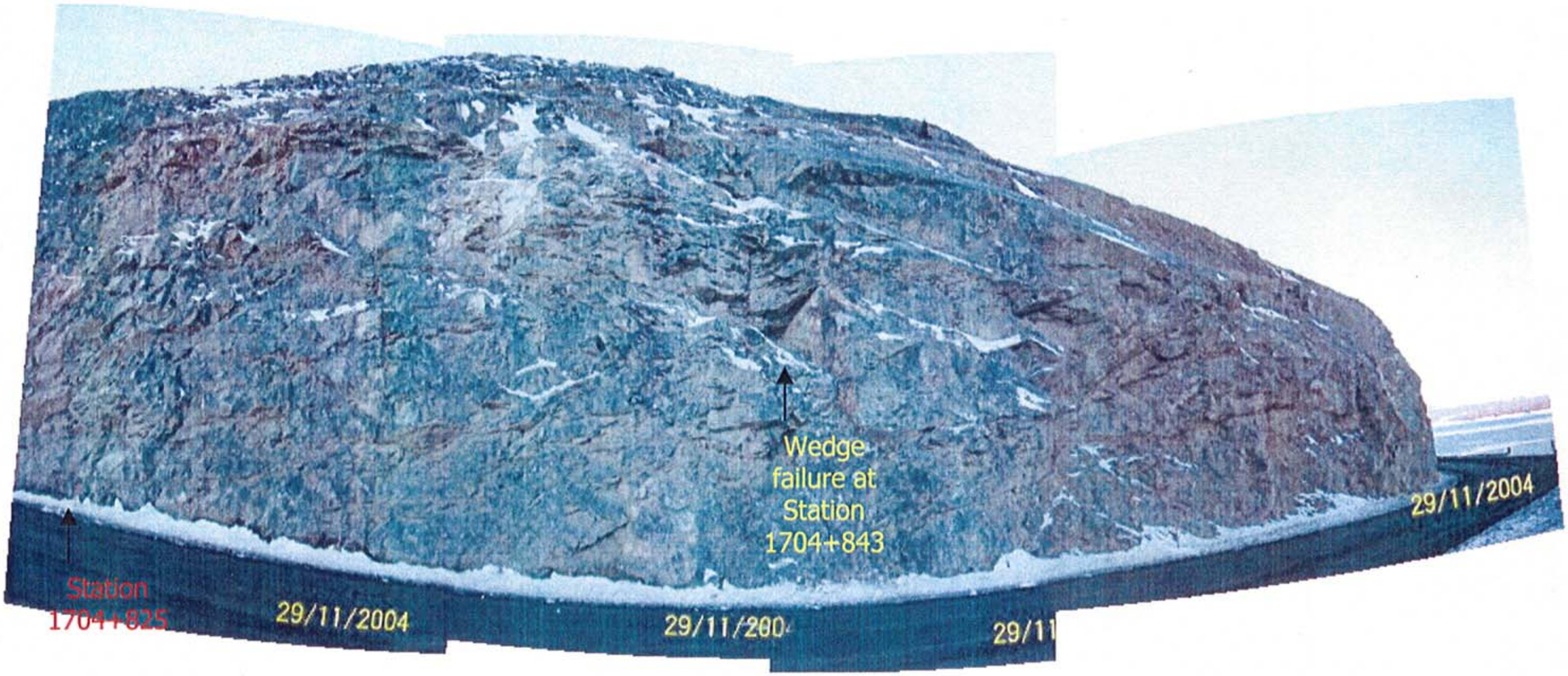


Photo 8: Typical exposure of the meta-volcanics in Design Sector 3.

APPENDIX

APPENDIX B BEDROCK MAPPING DATA

APPENDIX B - BEDROCK MAPPING DATA

LOCATION		BEDROCK			DISCONTINUITY										
													Aperature		
Station (m)	Offset (m)	Type	Strength	Weathering	Type	Dip	Dip Direction - Revised for Correct Declination	Persistence (m)	Spacing (m)	No. of Features	Shape	Roughness (JRC)	Open /Infill /tight	Width	Infill Type
1704 + 600	6	Phylite	Very Weak	Highly Weathered	Bedding	76	217	3.0	0.002	Extreme	planar	11	Open	0.002	Clay
1704 + 600	6	Phylite	Very Weak	Highly Weathered	Bedding	64	207	3.0	0.002	Extreme	planar	7	Open	0.002	Clay
1704 + 600	6	Phylite	Very Weak	Highly Weathered	Bedding	74	227	3.0	0.002	Extreme	planar	7	Open	0.002	Clay
1704 + 600	6	Phylite	Very Weak	Highly Weathered	Bedding	82	225	3.0	0.002	Extreme	planar	5	Open	0.002	Clay
1704 + 600	6	Phylite	Very Weak	Highly Weathered	Fragmentation Zones	90	137	3.0	?	Extreme		9	Open	0.002	Clay
1704 + 600	6	Phylite	Very Weak	Highly Weathered	Fragmentation Zones	82	347	3.0	0.002	Extreme		9	Open	0.002	Clay
1704 + 600	6	Phylite	Very Weak	Highly Weathered	Fragmentation Zones	78	137	3.0	0.002	Extreme		3	Open	0.002	Clay
1704 + 600	6	Phylite	Very Weak	Highly Weathered	Fragmentation Zones	90	119	3.0	0.002	Extreme		5	Open	0.002	Clay
1704 + 600	6	Phylite	Very Weak	Highly Weathered	Fragmentation Zones	78	163	3.0	0.002	Extreme		13	Open	0.002	Clay
1704 + 600	6	Phylite	Very Weak	Highly Weathered	Fragmentation Zones	81	123	3.0	0.002	Extreme		5	Open	0.002	Clay
1704 + 600	6	Phylite	Very Weak	Highly Weathered	Fragmentation Zones	72	299	3.0	0.002	Extreme		7	Open	0.002	Clay
1704 + 600	6	Phylite	Very Weak	Highly Weathered	Shear zones	58	229	3.0	0.002	Extreme		7	Open	0.002	Clay
1704 + 600	6	Phylite	Very Weak	Highly Weathered	Shear zones	30	95	3.0	0.002	Extreme		13	Open	0.002	Clay
1704 + 600	6	Phylite	Very Weak	Highly Weathered	Fragmentation Zones	72	347	3.0	0.002	Extreme		17	Open	0.002	Clay
1704 + 600	6	Phylite	Very Weak	Highly Weathered	Fragmentation Zones	68	337	3.0	0.002	Extreme		11	Open	0.002	Clay
1704 + 617	7	Mafic Intrusive	Strong	Slightly Weathered	Jointing	90	119	2.0	0.01	7	planar	5	Open	0.001	Clay
1704 + 617	7	Mafic Intrusive	Strong	Slightly Weathered	Fragmentation Zones	55	104	2.0	0.01	7		5	Open	0.001	Clay
1704 + 617	7	Mafic Intrusive	Strong	Slightly Weathered	Fragmentation Zones	78	44	2.0	0.01	7		5	Open	0.001	Clay
1704 + 617	7	Mafic Intrusive	Strong	Slightly Weathered	Fragmentation Zones	40	210	2.0	0.01	7		5	Open	0.001	Clay
1704 + 619	7	Mafic Intrusive	Very Weak	Extreme	Shear Zones	40	235	2.0	0.01	1			Open	0.02	
1704 + 619	7	Mafic Intrusive	Strong	Slight Weathering	Jointing	66	235	2.0	0.001 - 0.01	Multiple	planar	3	Open	0.001	Clay
1704 + 619	7	Mafic Intrusive	Strong	Slight Weathering	Jointing	36	117	2.0	0.001 - 0.01	Multiple	planar	9	Tight	-	-
1704 + 619	7	Mafic Intrusive	Strong	Slight Weathering	Jointing	56	233	2.0	0.001 - 0.01	Multiple	planar	15	Open	0.001	Rust
1704 + 619	7	Mafic Intrusive	Strong	Slight Weathering	Jointing	90	157	2.0	-	2	planar	13	Open	0.001	Calcite

Mapped By: CPC JTD

APPENDIX B - BEDROCK MAPPING DATA

LOCATION		BEDROCK			DISCONTINUITY										
													Aperature		
Station (m)	Offset (m)	Type	Strength	Weathering	Type	Dip	Dip Direction - Revised for Correct Declination	Persistence (m)	Spacing (m)	No. of Features	Shape	Roughness (JRC)	Open /Infill /tight	Width	Infill Type
1704 + 619	7	Mafic Intrusive	Strong	Slight Weathering	Jointing	42	169	2.0	-	2	planar	7	Open	0.001	Calcite
1704 + 619	7	Mafic Intrusive	Strong	Slight Weathering	Jointing	80	111	2.0	-	5	planar	7	Open	0.001	Clay
1704 + 619	7	Mafic Intrusive	Strong	Slight Weathering	Jointing	78	87	2.0	0.02 - 0.01	7	planar	7	Open	0.001	Clay
1704 + 619	7	Mafic Intrusive	Strong	Slight Weathering	Jointing	67	17	2.0	0.02 - 0.01	7	planar	7	Open	0.001	Clay
1704 + 623	7	Mafic Intrusive	Very Weak	Highly Weathered	Shear Zones	80	31	3.0	0.02 - 0.01	7		7	Open	0.001	Clay
1704 + 623	7	Mafic Intrusive	Strong	Highly Weathered	Shear Zones	70	219	3.0	0.02 - 0.01	7		7	Open	0.001	Clay
1704 + 623	7	Mafic Intrusive	Strong	Slightly Weathered	Jointing	24	167	3.0	0.1	3	planar	13	Open	0.001	Clay
1704 + 623	7	Mafic Intrusive	Strong	Slightly Weathered	Jointing	26	151	3.0	0.1	3	planar	13	Open	0.001	Clay
1704 + 623	7	Mafic Intrusive	Strong	Slightly Weathered	Jointing	76	65	3.0	0.1	3	planar	13	Open	0.001	Clay
1704 + 623	7	Mafic Intrusive	Strong	Slightly Weathered	Jointing	84	179	3.0	0.1	3	planar	11	Open	0.001	Clay
1704 + 623	7	Mafic Intrusive	Strong	Slightly Weathered	Jointing	34	201	3.0	0.1	3	planar	9	Open	0.001	Clay
1704 + 623	7	Mafic Intrusive	Strong	Slightly Weathered	Jointing	74	101	3.0	0.1	3	planar	13	Open	0.001	Clay
1704 + 623	7	Mafic Intrusive	Strong	Slightly Weathered	Jointing	83	21	3.0	0.1	3	planar	15	Open	0.001	Clay
1704 + 623	7	Mafic Intrusive	Strong	Slightly Weathered	Jointing	36	323	3.0	0.1	3	planar	11	Open	0.001	Clay
1704 + 623	7	Mafic Intrusive	Strong	Slightly Weathered	Jointing	58	161	3.0	0.1	3	planar	13	Open	0.001	Clay
1704 + 623	7	Mafic Intrusive	Strong	Slightly Weathered	Jointing	70	359	3.0	0.1	3	planar	13	Open	0.001	Clay
1704 + 623	7	Mafic Intrusive	Strong	Slightly Weathered	Jointing	72	33	3.0	0.1	3	planar	11	Open	0.001	Clay
1704 + 623	7	Mafic Intrusive	Strong	Slightly Weathered	Jointing	54	77	3.0	0.1	3	planar	9	Open	0.001	Clay
1704 + 632	7	Argillite	Medium	Slightly Weathered	Jointing	64	194	4.0	0.03 - 0.15	3	planar	9	Open	0.001	Clay
1704 + 632	7	Argillite	Strong	Slightly Weathered	Jointing	48	172	4.0	0.03 - 0.15	3	planar	5	Open	0.001	Clay
1704 + 632	7	Argillite	Strong	Slightly Weathered	Jointing	58	203	4.0	0.03 - 0.15	3	planar	9	Open	0.001	Clay
1704 + 632	7	Argillite	Strong	Slightly Weathered	Jointing	25	327	4.0	0.03 - 0.15	3	planar	9	Open	0.001	Clay
1704 + 632	7	Argillite	Strong	Slightly Weathered	Bedding	64	246	4.0	0.001 - 0.03	3	planar	11	Open	0.001	Clay
1704 + 632	7	Argillite	Strong	Slightly Weathered	Shear Zones	35	229	4.0	0.001 - 0.03	3		9	Open	0.001	Clay

Mapped By: CPC JTD

APPENDIX B - BEDROCK MAPPING DATA

LOCATION		BEDROCK			DISCONTINUITY										
													Aperature		
Station (m)	Offset (m)	Type	Strength	Weathering	Type	Dip	Dip Direction - Revised for Correct Declination	Persistence (m)	Spacing (m)	No. of Features	Shape	Roughness (JRC)	Open /Infill /tight	Width	Infill Type
1704 + 634	7	Argillite	Strong	Slightly Weathered	Jointing	28	298	4.0	0.005 - 0.1	3	planar	11	Open	0.001	Clay
1704 + 634	7	Argillite	Strong	Slightly Weathered	Jointing	56	179	4.0	0.005 - 0.1	3	planar	13	Open	0.001	Clay
1704 + 637	7	Argillite	Strong	Slightly Weathered	Jointing	54	320	4.0	0.005 - 0.1	3	planar	9	Open	0.001	Clay
1704 + 637	7	Argillite	Strong	Slightly Weathered	Jointing	55	177	4.0	0.005 - 0.1	3	planar	11	Open	0.001	Clay
1704 + 637	7	Argillite	Strong	Slightly Weathered	Bedding	68	237	4.0	0.005 - 0.1	3	planar	9	Open	0.001	Clay
1704 + 637	7	Argillite	Strong	Slightly Weathered	Jointing	80	279	4.0	0.005 - 0.1	3	planar	15	Open	0.001	Clay
1704 + 675	7	Argillite	Strong	Slightly Weathered	Jointing	48	308	4.0	0.005 - 0.1	3	planar	5	Open	0.001	Clay
1704 + 676	7	Dyke	Strong	Slightly Weathered	Jointing	28	29	4.0	0.2 - 0.4	3	planar	3	Open	0.001	Clay
1704 + 676	7	Dyke	Strong	Slightly Weathered	Jointing	40	201	4.0	0.05 - 0.2	3	planar	9	Open	0.001	Clay
1704 + 676	7	Dyke	Strong	Slightly Weathered	Jointing	60	301	4.0	0.05 - 0.2	3	planar	5	Open	0.001	Clay
1704 + 676	7	Dyke	Strong	Slightly Weathered	Jointing	60	115	4.0	0.05 - 0.2	3	planar	11	Open	0.001	Clay
1704 + 676	7	Dyke	Strong	Slightly Weathered	Jointing	54	167	4.0	0.05 - 0.2	3	planar	11	Open	0.001	Clay
1704 + 676	7	Dyke	Strong	Slightly Weathered	Jointing	32	81	4.0	0.05 - 0.2	3	planar	3	Open	0.001	Clay
1704 + 681	7	Argillite	Medium Strong	Slightly Weathered	Jointing	60	297	4.0	0.005 - 0.05	3	planar	7	Open	0.001	Clay
1704 + 681	7	Argillite	Medium Strong	Slightly Weathered	Jointing	80	201	4.0	0.005 - 0.05	3	planar	7	Open	0.001	Clay
1704 + 681	7	Argillite	Medium Strong	Slightly Weathered	Jointing	76	187	4.0	0.005 - 0.05	3	planar	5	Open	0.001	Clay
1704 + 681	7	Argillite	Medium Strong	Slightly Weathered	Bedding	70	233	4.0	0.005 - 0.05	3	planar	9	Open	0.001	Clay
1704 + 681	7	Argillite	Medium Strong	Slightly Weathered	Bedding	72	230	4.0	0.005 - 0.05	3	planar	7	Open	0.001	Clay
1704 + 681	7	Argillite	Medium Strong	Slightly Weathered	Bedding	74	238	4.0	0.005 - 0.05	3	planar	9	Open	0.001	Clay
1704 + 681	7	Argillite	Medium Strong	Slightly Weathered	Jointing	40	171	4.0	0.03 - 0.2	3	planar	3	Open	0.001	Clay
1704 + 681	7	Argillite	Medium Strong	Slightly Weathered	Jointing	34	173	4.0	0.03 - 0.2	3	planar	7	Open	0.001	Clay
1704 + 681	7	Argillite	Medium Strong	Slightly Weathered	Jointing	42	163	4.0	0.03 - 0.2	3	planar	3	Open	0.001	Clay
1704 + 681	7	Argillite	Medium Strong	Slightly Weathered	Jointing	78	210	4.0	0.03 - 0.2	3	planar	9	Open	0.001	Clay
1704 + 695	7	Argillite	Medium Strong	Slightly Weathered	Shear Zones	46	239	8.0	0.03 - 0.2	3		11	Open	0.001	Clay

Mapped By: CPC JTD

APPENDIX B - BEDROCK MAPPING DATA

LOCATION		BEDROCK			DISCONTINUITY										
													Aperature		
Station (m)	Offset (m)	Type	Strength	Weathering	Type	Dip	Dip Direction - Revised for Correct Declination	Persistence (m)	Spacing (m)	No. of Features	Shape	Roughness (JRC)	Open /Infill /tight	Width	Infill Type
1704 + 695	7	Argillite	Medium Strong	Slightly Weathered	Jointing	66	157	8.0	0.03 - 0.2	3	planar	11	Open	0.001	Clay
1704 + 700	7	Argillite	Medium Strong	Slightly Weathered	Jointing	36	346	8.0	0.03 - 0.2	3	planar	13	Open	0.001	Clay
1704 + 700	7	Argillite	Medium Strong	Slightly Weathered	Jointing	86	61	8.0	0.03 - 0.2	3	planar	11	Open	0.001	Clay
1704 + 701	7	Argillite	Medium Strong	Slightly Weathered	Shear Zones	66	217	8.0	0.03 - 0.2	3		11	Open	0.001	Clay
1704 + 701	7	Argillite	Medium Strong	Slightly Weathered	Bedding	70	243	8.0	0.03 - 0.2	3	planar	9	Open	0.001	Clay
1704 + 706	7	Argillite	Medium Strong	Slightly Weathered	Shear Zones	36	149	8.0	0.03 - 0.2	3		9	Open	0.001	Clay
1704 + 714	7	Argillite	Medium Strong	Slightly Weathered	Bedding	70	155	8.0	0.03 - 0.2	3	planar	7	Open	0.001	Clay
1704 + 714	7	Argillite	Medium Strong	Slightly Weathered	Jointing	76	113	8.0	0.03 - 0.2	3	planar	5	Open	0.001	Clay
1704 + 714	7	Argillite	Medium Strong	Slightly Weathered	Jointing	66	125	8.0	0.03 - 0.2	3	planar	13	Open	0.001	Clay
1704 + 714	7	Argillite	Medium Strong	Slightly Weathered	Jointing	50	161	8.0	0.03 - 0.2	3	planar	13	Open	0.001	Clay
1704 + 722	7	Quartzite	Very Strong	Slightly Weathered	Jointing	80	171	8.0	0.03 - 0.2	3	planar	7	Open	0.001	Clay
1704 + 722	7	Quartzite	Very Strong	Slightly Weathered	Bedding	68	229	8.0	0.03 - 0.2	3		7	Open	0.001	Clay
1704 + 722	7	Quartzite	Very Strong	Slightly Weathered	Jointing	82	347	8.0	0.03 - 0.2	3	planar	7	Open	0.001	Clay
1704 + 732	7	Quartzite	Very Strong	Slightly Weathered	Shear Zones	70	237	10.0	0.03 - 0.2	3		7	Open	0.001	Clay Gouge
1704 + 732	7	Quartzite	Very Strong	Slightly Weathered	Jointing	76	183	10.0	0.03 - 0.2	3	planar	7	Open	0.001	Clay Gouge
1704 + 732	7	Quartzite	Very Strong	Slightly Weathered	Jointing	34	103	10.0	0.03 - 0.2	3	planar	5	Open	0.001	Clay Gouge
1704 + 732	7	Quartzite	Very Strong	Slightly Weathered	Shear Zones	66	125	10.0	0.03 - 0.2	3		13	Open	0.001	Clay Gouge
1704 + 746	7	Quartzite	Very Strong	Slightly Weathered	Jointing	58	327	10.0	0.03 - 0.2	3	planar	9	Open	0.001	Clay
1704 + 750	7	Quartzite	Very Strong	Slightly Weathered	Shear Zones	62	239	10.0	0.03 - 0.2	3		9	Open	0.001	Clay
1704 + 760	7	Quartzite	Very Strong	Slightly Weathered	Bedding	70	233	10.0	0.03 - 0.2	3	planar	5	Open	0.001	Clay
1704 + 760	7	Quartzite	Very Strong	Slightly Weathered	Jointing	82	293	10.0	0.03 - 0.2	3	planar	11	Open	0.001	Clay
1704 + 760	7	Quartzite	Very Strong	Slightly Weathered	Jointing	72	363	10.0	0.03 - 0.2	3	planar	7	Open	0.001	Clay
1704 + 760	7	Quartzite	Very Strong	Slightly Weathered	Jointing	20	311	10.0	0.03 - 0.2	3	planar	13	Open	0.001	Clay
1704 + 764	7	Intrusive	Very Strong	Slightly Weathered	Contact	62	263	18.0	0.05 - 0.7	3	undulated	9	Open	0.001	Clay

Mapped By: CPC JTD

APPENDIX B - BEDROCK MAPPING DATA

LOCATION		BEDROCK			DISCONTINUITY										
													Aperature		
Station (m)	Offset (m)	Type	Strength	Weathering	Type	Dip	Dip Direction - Revised for Correct Declination	Persistence (m)	Spacing (m)	No. of Features	Shape	Roughness (JRC)	Open /Infill /tight	Width	Infill Type
1704 + 764	7	Intrusive	Very Strong	Slightly Weathered	Jointing	76	21	18.0	0.05 - 0.7	3	planar	5	Infill	0.002	Clay
1704 + 764	7	Intrusive	Very Strong	Slightly Weathered	Jointing	90	179	18.0	0.05 - 0.7	3	planar	5	Open	0.001	Clay
1704 + 764	7	Intrusive	Very Strong	Slightly Weathered	Jointing	28	363	18.0	0.05 - 0.7	3	planar	7	Open	0.001	Clay
1704 + 764	7	Intrusive	Very Strong	Slightly Weathered	Jointing	64	101	18.0	0.05 - 0.7	3	planar	11	Infill	0.002	Clay
1704 + 764	7	Intrusive	Very Strong	Slightly Weathered	Jointing	30	33	18.0	0.05 - 0.7	3	planar	9	Open	0.001	Clay
1704 + 764	7	Intrusive	Very Strong	Slightly Weathered	Jointing	68	65	18.0	0.05 - 0.7	3	planar	15	Open	0.001	Clay
1704 + 764	7	Intrusive	Very Weak	Slightly Weathered	Shear Zones	80	73	18.0	0.05 - 0.7	3	planar	11	Infill	0.001	Clay
1704 + 764	7	Intrusive	Very Strong	Slightly Weathered	Jointing	54	199	18.0	0.3	3	planar	13	Infill	0.001	Calcite
1704 + 764	7	Intrusive	Very Strong	Slightly Weathered	Jointing	64	196	18.0	-	3	planar	15	Tight	0.001	Clay
1704 + 805	7	Intrusive	Very Strong	Slightly Weathered	Jointing	70	217	18.0	0.3 - 0.7	3	planar	13	Tight	0.001	Clay
1704 + 805	7	Intrusive	Very Strong	Slightly Weathered	Jointing	73	242	18.0	0.3 - 0.7	3	planar	11	Tight	0.001	Clay
1704 + 805	7	Intrusive	Very Strong	Slightly Weathered	Jointing	80	64	18.0	0.3 - 0.7	3	planar	11	Tight	0.001	Clay
1704 + 814	7	Intrusive	Very Strong	Slightly Weathered	Shear Zones	60	42	12.0	0.05 - 0.9	3	planar	5	Tight	0.001	Clay Gouge
1704 + 814	7	Intrusive	Very Strong	Slightly Weathered	Jointing	80	166	12.0	0.05 - 0.9	3	planar	13	Infill	0.001	Calcite
1704 + 815	7	Intrusive	Very Strong	Slightly Weathered	Shear Zones	60	41	16.0	0.5	3	planar	7	Infill	0.001	Clay Gouge
1704 + 815	7	Intrusive	Very Strong	Slightly Weathered	Jointing	24	141	16.0	0.5	3	planar	13	Infill	0.001	Clay
1704 + 830	7	Meta Vol - Extrusive	Very Strong	Slightly Weathered	Shear Zones	32	336	16.0	0.3 - 0.6	3	planar	9	Infill	0.001	Clay
1704 + 843	7	Meta Vol - Extrusive	Very Strong	Slightly Weathered	Jointing	80	245	16.0	0.3 - 0.6	3	planar	13	Infill	0.001	Quartz Gouge
1704 + 843	7	Meta Vol - Extrusive	Very Strong	Slightly Weathered	Jointing	64	315	16.0	0.3 - 0.6	3	planar	15	Infill	0.001	Quartz Gouge
1704 + 843	7	Meta Vol - Extrusive	Very Strong	Slightly Weathered	Jointing	32	167	16.0	0.3 - 0.6	3	planar	15	Infill	0.001	Quartz Gouge
1704 + 848	7	Meta Vol - Extrusive	Very Strong	Slightly Weathered	Shear Zones	30	239	13.0	0.3 - 0.6	3	planar		Infill	0.001	Clay
1704 + 853	3	Meta Vol - Extrusive	Very Strong	Fresh to Slightly Weathered	Shear Zones	24	353	13.0	0.3 - 1.5	4	slightly undulated	15	Infill	0.01	Clay
1704 + 880	4.5	Meta Vol - Extrusive	Strong	Fresh to Slightly Weathered	Shear Zones	88	197	13.0	0.05 - 0.3	3	slightly undulated	13	Tight	0.01	Clay
1704 + 880	4.5	Meta Vol - Extrusive	Very Strong	Fresh to Slightly Weathered	Shear Zones	20	282	10.0	0.3 - 0.9	3	planar	9	Tight	0.01	Clay

Mapped By: CPC JTD

APPENDIX B - BEDROCK MAPPING DATA

LOCATION		BEDROCK			DISCONTINUITY								Aperature		
Station (m)	Offset (m)	Type	Strength	Weathering	Type	Dip	Dip Direction - Revised for Correct Declination	Persistence (m)	Spacing (m)	No. of Features	Shape	Roughness (JRC)	Open /Infill /tight	Width	Infill Type
1704 + 900	4.0	Meta Vol - Extrusive	Very Strong	Fresh to Slightly Weathered	Jointing	30	329	10.0	0.04 - 0.1	1	planar	11	Tight	0.01	Clay
1704 + 900	4.0	Meta Vol - Extrusive	Very Strong	Fresh to Slightly Weathered	Jointing	80	13	10.0	0.5	1	planar	14	Tight	0.01	Clay
1704 + 900	4.5	Meta Vol - Extrusive	Very Strong	Fresh to Slightly Weathered	Jointing	40	160	-	0.05 - 0.2	1	planar	7	Tight	0.01	Clay
1704 + 912	4.5	Meta Vol - Extrusive	Very Strong	Fresh to Slightly Weathered	Jointing	66	29	15.0	0.05 - 0.2	1	planar	7	Infill	0.005	Clay
1704 + 913	5.0	Meta Vol - Extrusive	Very Strong	Fresh to Slightly Weathered	Jointing	44	329	5.0	0.2	1	slightly undulated	11	Tight	-	-
1704 + 913	5.0	Meta Vol - Extrusive	Very Strong	Fresh to Slightly Weathered	Jointing	80	217	13.0	0.2 - 0.4	1	planar	13	Tight		Clay
1704 + 925	5.0	Meta Vol - Extrusive	Very Strong	Fresh to Slightly Weathered	Jointing	72	179	-	-	3	planar	11	Infill	0.005	Quartz
1704 + 925	5.0	Meta Vol - Extrusive	Very Strong	Fresh to Slightly Weathered	Shear Zones	38	367	25.0	-	3	slightly undulated	11	Infill	0.01	Quartz
1704 + 950	5.0	Meta Vol - Extrusive	Strong	Fresh to Slightly Weathered	Contact	74	311	25.0	0.5 - 0.9	3	planar	13	Infill	0.002	Calcite
1704 + 950	5.0	Meta Vol - Extrusive	Very Strong	Fresh to Slightly Weathered	Jointing	84	366	15.0	0.5 - 0.9	3	planar	13	Infill	0.002	Clay
1704 + 957	5.0	Lime Stone	Very Strong	Fresh to Slightly Weathered	Jointing	50	33	4.0	0.05 - 0.4	1	planar	9	Tight		Clay
1704 + 957	5.0	Lime Stone	Very Strong	Fresh to Slightly Weathered	Jointing	88	93	4.0	0.05 - 0.8	1	planar	11	Tight		Clay
1704 + 970	5.0	Meta Vol - Extrusive	Very Strong	Fresh to Slightly Weathered	Jointing	80	317	20.0	0.05 - 0.8	2	planar	5	Tight		Clay
1704 + 970	5.0	Meta Vol - Extrusive	Very Strong	Fresh to Slightly Weathered	Jointing	70	347	10.0	0.05 - 0.8	2	planar	7	Infill	0.002	Calcite
1704 + 988	6.0	Meta Vol - Extrusive	Very Strong	Fresh to Slightly Weathered	Jointing	80	127	20.0	0.8	2	planar	7	Infill	0.002	Clay
1704 + 988	6.0	Meta Vol - Extrusive	Very Strong	Fresh to Slightly Weathered	Jointing	70	323	10.0	0.6	2	planar	7	Infill	0.002	Clay
1705 + 009	10.0	Meta Vol - Extrusive	Very Strong	Fresh to Slightly Weathered	Jointing	78	297	10.0	0.1 - 0.9	1	slightly undulated	9	Infill	0.002	Calcite
1705 + 009	10.0	Meta Vol - Extrusive	Very Strong	Fresh to Slightly Weathered	Jointing	40	97	3.0	1.5	1	slightly undulated	9	Infill	0.002	Calcite
1705 + 009	10.0	Meta Vol - Extrusive	Very Strong	Fresh to Slightly Weathered	Jointing	80	67	2.0	2	2	planar	11	Tight	0.002	Clay
1705 + 009	10.0	Meta Vol - Extrusive	Very Strong	Fresh to Slightly Weathered	Jointing	80	367	20.0	2	2	planar	9	Tight	0.002	Clay
1705 + 015	10.0	Meta Vol - Extrusive	Very Strong	Fresh to Slightly Weathered	Shear Zones	70	267	20.0	-	1	slightly undulated	9	Tight	0.002	Clay
1705 + 025	10.0	Intrusive	Very Strong	Fresh to Slightly Weathered	Shear Zones	58	217	10.0	-	1	slightly undulated	11	Infill	0.002	Quartz
1705 + 025	10.0	Intrusive	Very Strong	Fresh to Slightly Weathered	Jointing	74	347	4.0	0.08 - 0.3	1	planar	13	Tight	0.002	Clay
1705 + 025	10.0	Intrusive	Very Strong	Fresh to Slightly Weathered	Jointing	30	183	10.0	0.08 - 0.3	1	planar	7	Tight	0.002	Clay

Mapped By: CPC JTD

APPENDIX B - BEDROCK MAPPING DATA

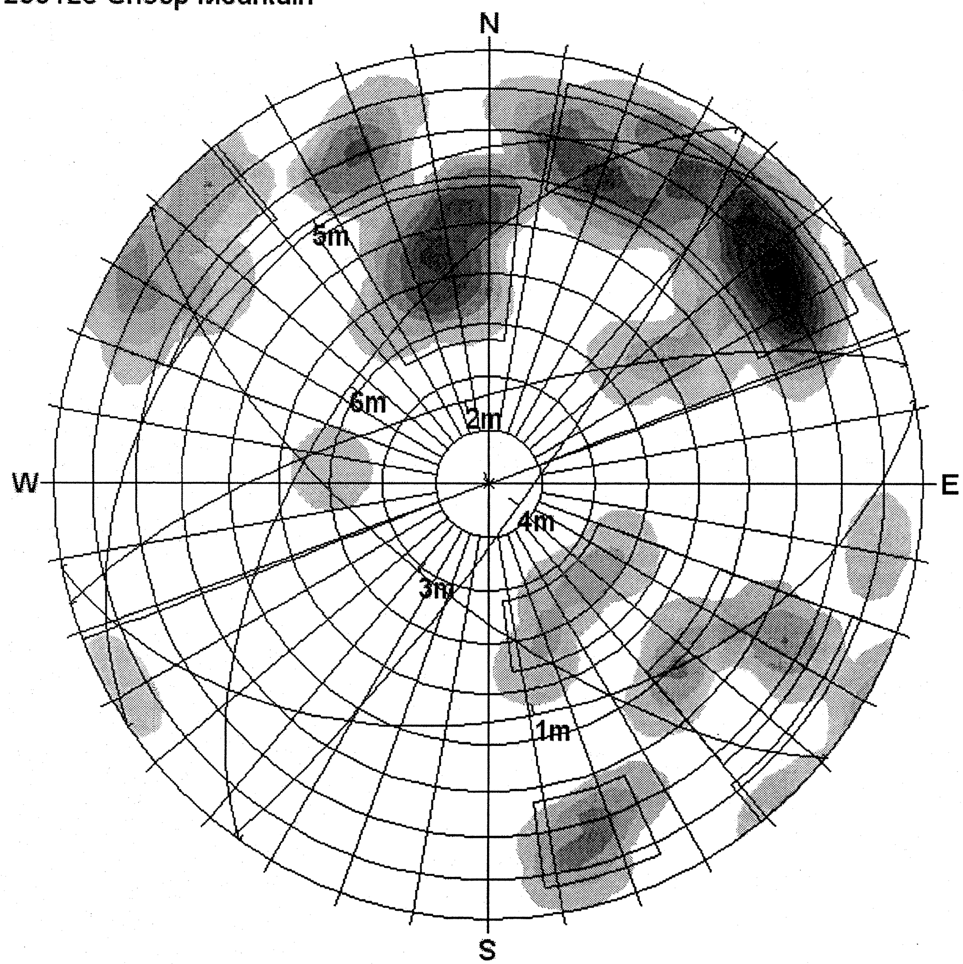
LOCATION		BEDROCK			DISCONTINUITY										
													Aperature		
Station (m)	Offset (m)	Type	Strength	Weathering	Type	Dip	Dip Direction - Revised for Correct Declination	Persistence (m)	Spacing (m)	No. of Features	Shape	Roughness (JRC)	Open /Infill /tight	Width	Infill Type
1705 + 040	5.0	Intrusive	Very Strong	Fresh to Slightly Weathered	Shear Zones	68	199	8.0	0.08 - 0.3	1	planar	9	Tight	0.002	Clay
1705 + 047	5.0	Intrusive	Very Strong	Fresh to Slightly Weathered	Shear Zones	78	197	7.0	0.08 - 0.3	1	planar	15	Infill	0.002	Clay
1705 + 048	5.0	Intrusive Dyke	Very Strong	Fresh to Slightly Weathered	Contact	82	197	7.0	0.08 - 0.3	1	planar	15	Tight	0.002	Clay
1705 + 050	5.0	Intrusive Dyke	Very Strong	Fresh to Slightly Weathered	Contact	88	39	7.0	0.08 - 0.3	1	planar	9	Tight	0.002	Clay
1705 + 059	5.0	Argillite	Very Strong	Fresh to Slightly Weathered	Shear Zones	68	194	6.0	0.08 - 0.3	1	planar	9	Tight	0.002	Clay

Mapped By: CPC JTD

APPENDIX

APPENDIX C STEREONET PLOTS OF MAPPING DATA

1200128-Sheep Mountain



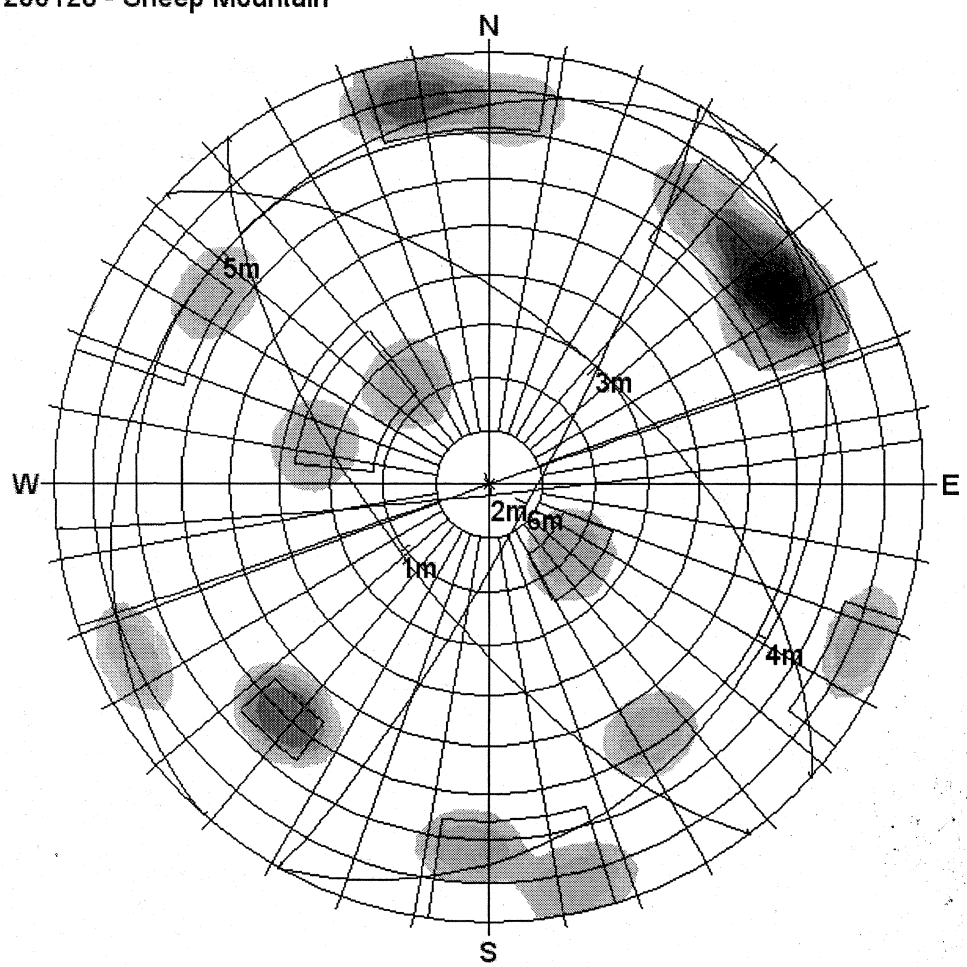
Structural Domain: Argillite - Design Sector 1

Orientations

ID		Dip / Direction	
1		90 / 159	Highway Cut Orientation
1	m	45 / 169	J3
2	m	74 / 344	J5
3	m	69 / 219	J1
4	m	83 / 126	J4
5	m	28 / 326	J2
6	m	58 / 305	J6

Equal Area
Lower Hemisphere
50 Poles
50 Entries

1200128 - Sheep Mountain



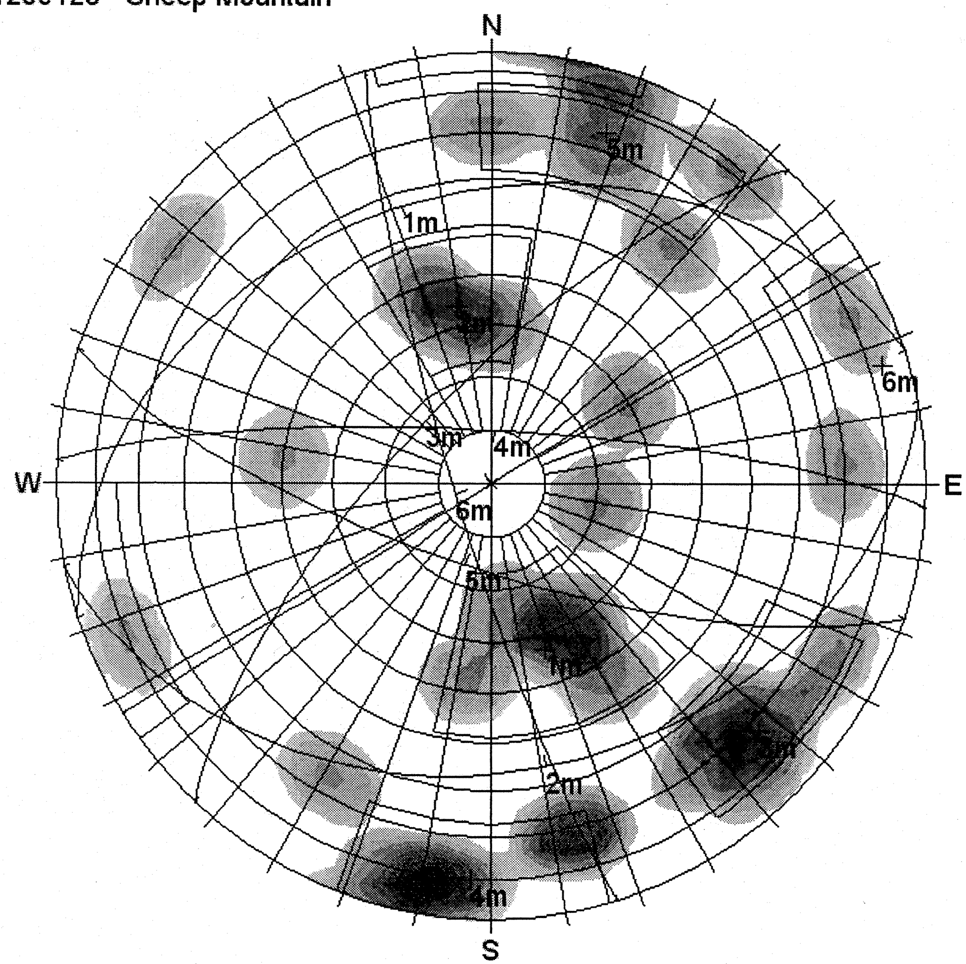
Structural Domain: Quartzite - Design Sector 2

Orientations

ID		Dip / Direction	
1		90 / 161	Highway Cut Orientation
1	m	69 / 233	J1
2	m	88 / 174	J5
3	m	60 / 042	J6
4	m	28 / 119	J3
5	m	20 / 311	J2
6	m	82 / 119	J4

Equal Area
Lower Hemisphere
20 Poles
20 Entries

1200128 - Sheep Mountain



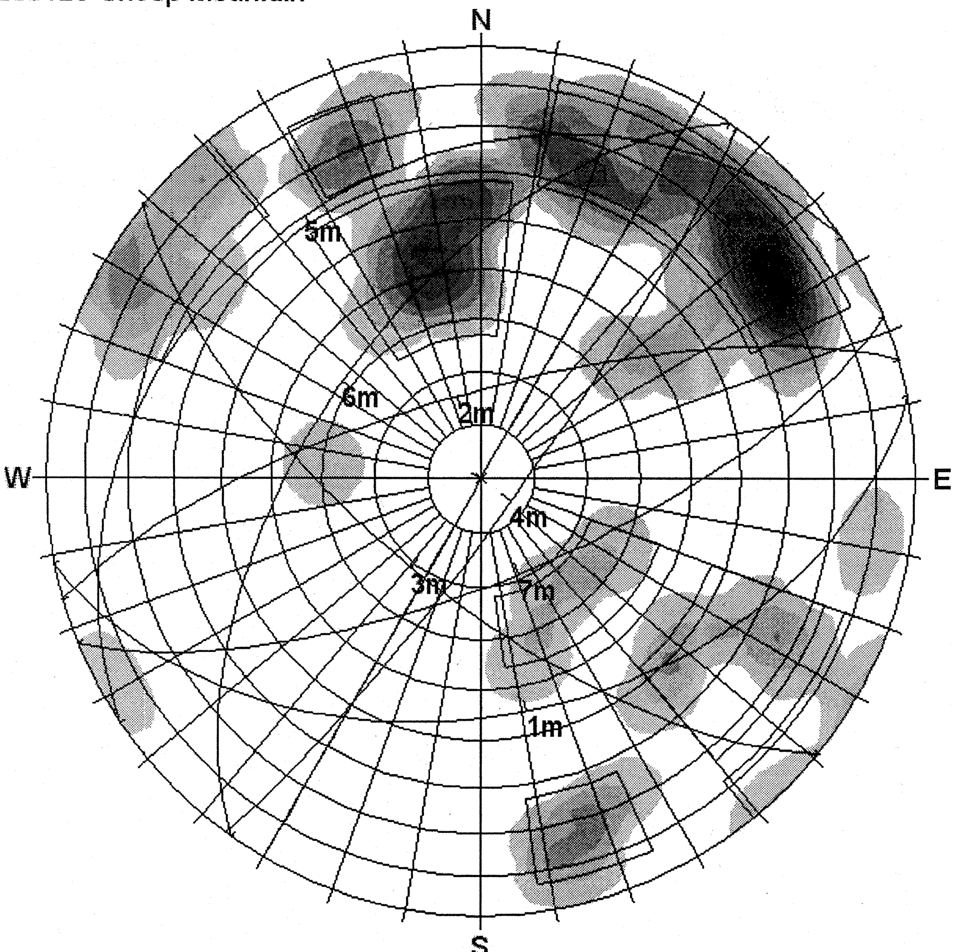
Structural Domain: Volcanics - Design Sector 3

Orientations

ID		Dip / Direction	
1		90 / 148	**Highway Cut Orientation**
1	m	33 / 342	J4
2	m	34 / 169	J3
3	m	73 / 313	J2
4	m	80 / 003	J5
5	m	74 / 198	J1
6	m	83 / 253	J6

Equal Area
Lower Hemisphere
32 Poles
32 Entries

1200128-Sheep Mountain



Structural Domain: Argillite - Design Sector 4

Orientations

ID		Dip / Direction	
1		90 / 120	Highway Cut Orientation
1	m	45 / 169	J3
2	m	74 / 344	J5
3	m	69 / 219	J1
4	m	83 / 126	J4
5	m	28 / 326	J2
6	m	58 / 305	J6
7	m	71 / 158	J7

Equal Area
Lower Hemisphere
50 Poles
50 Entries

APPENDIX

APPENDIX D GEOTECHNICAL REPORT – GENERAL CONDITIONS

GEOTECHNICAL REPORT – GENERAL CONDITIONS

This report incorporates and is subject to these "General Conditions".

1.0 USE OF REPORT AND OWNERSHIP

This geotechnical report pertains to a specific site, a specific development and a specific scope of work. It is not applicable to any other sites nor should it be relied upon for types of development other than that to which it refers. Any variation from the site or development would necessitate a supplementary geotechnical assessment.

This report and the recommendations contained in it are intended for the sole use of EBA's client. EBA does not accept any responsibility for the accuracy of any of the data, the analyses or the recommendations contained or referenced in the report when the report is used or relied upon by any party other than EBA's client unless otherwise authorized in writing by EBA. Any unauthorized use of the report is at the sole risk of the user.

2.0 NATURE AND EXACTNESS OF SOIL AND ROCK DESCRIPTIONS

Classification and identification of soils and rocks are based upon commonly accepted systems and methods employed in professional geotechnical practice. This report contains descriptions of the systems and methods used. Where deviations from the system or method prevail, they are specifically mentioned.

Classification and identification of geological units are judgmental in nature as to both type and condition. EBA does not warrant conditions represented herein as exact, but infers accuracy only to the extent that is common in practice.

Where subsurface conditions encountered during development are different from those described in this report, qualified geotechnical personnel should revisit the site and review recommendations in light of the actual conditions encountered.

3.0 LOGS OF TESTHOLES

The testhole logs are a compilation of conditions and classification of soils and rocks as obtained from field observations and laboratory testing of selected samples. Soil and rock zones have been interpreted. Change from one geological zone to the other, indicated on the logs as a distinct line, can be, in fact, transitional. The extent of transition is interpretive. Any circumstance which requires precise definition of soil or rock zone transition elevations may require further investigation and review.

4.0 STRATIGRAPHIC AND GEOLOGICAL INFORMATION

The stratigraphic and geological information indicated on drawings contained in this report are inferred from logs of test holes and/or soil/rock exposures. Stratigraphy is known only at the locations of the test hole or exposure. Actual geology and stratigraphy between test holes and/or exposures may vary from that shown on these drawings. Natural variations in geological conditions are inherent and are a function of the historic environment. EBA does not represent the conditions illustrated as exact but recognizes that variations will exist. Where knowledge of more precise locations of geological units is necessary, additional investigation and review may be necessary.

5.0 SURFACE WATER AND GROUNDWATER CONDITIONS

Surface and groundwater conditions mentioned in this report are those observed at the times recorded in the report. These conditions vary with geological detail between observation sites; annual, seasonal and special meteorologic conditions; and with development activity. Interpretation of water conditions from observations and records is judgmental and constitutes an evaluation of circumstances as influenced by geology, meteorology and development activity. Deviations from these observations may occur during the course of development activities.

6.0 PROTECTION OF EXPOSED GROUND

Excavation and construction operations expose geological materials to climatic elements (freeze/thaw, wet/dry) and/or mechanical disturbance which can cause severe deterioration. Unless otherwise specifically indicated in this report, the walls and floors of excavations must be protected from the elements, particularly moisture, desiccation, frost action and construction traffic.

7.0 SUPPORT OF ADJACENT GROUND AND STRUCTURES

Unless otherwise specifically advised, support of ground and structures adjacent to the anticipated construction and preservation of adjacent ground and structures from the adverse impact of construction activity is required.

8.0 INFLUENCE OF CONSTRUCTION ACTIVITY

There is a direct correlation between construction activity and structural performance of adjacent buildings and other installations. The influence of all anticipated construction activities should be considered by the contractor, owner, architect and prime engineer in consultation with a geotechnical engineer when the final design and construction techniques are known.

9.0 OBSERVATIONS DURING CONSTRUCTION

Because of the nature of geological deposits, the judgmental nature of geotechnical engineering, as well as the potential of adverse circumstances arising from construction activity, observations during site preparation, excavation and construction should be carried out by a geotechnical engineer. These observations may then serve as the basis for confirmation and/or alteration of geotechnical recommendations or design guidelines presented herein.

10.0 DRAINAGE SYSTEMS

Where temporary or permanent drainage systems are installed within or around a structure, the systems which will be installed must protect the structure from loss of ground due to internal erosion and must be designed so as to assure continued performance of the drains. Specific design detail of such systems should be developed or reviewed by the geotechnical engineer. Unless otherwise specified, it is a condition of this report that effective temporary and permanent drainage systems are required and that they must be considered in relation to project purpose and function.

11.0 BEARING CAPACITY

Design bearing capacities, loads and allowable stresses quoted in this report relate to a specific soil or rock type and condition. Construction activity and environmental circumstances can materially change the condition of soil or rock. The elevation at which a soil or rock type occurs is variable. It is a requirement of this report that structural elements be founded in and/or upon geological materials of the type and in the condition assumed. Sufficient observations should be made by qualified geotechnical personnel during construction to assure that the soil and/or rock conditions assumed in this report in fact exist at the site.

12.0 SAMPLES

EBA will retain all soil and rock samples for 30 days after this report is issued. Further storage or transfer of samples can be made at the client's expense upon written request, otherwise samples will be discarded.

13.0 STANDARD OF CARE

Services performed by EBA for this report have been conducted in a manner consistent with the level of skill ordinarily exercised by members of the profession currently practising under similar conditions in the jurisdiction in which the services are provided. Engineering judgement has been applied in developing the conclusions and/or recommendations provided in this report. No warranty or guarantee, express or implied, is made concerning the test results, comments, recommendations, or any other portion of this report.

14.0 ENVIRONMENTAL AND REGULATORY ISSUES

Unless stipulated in the report, EBA has not been retained to investigate, address or consider and has not investigated, addressed or considered any environmental or regulatory issues associated with development on the subject site.

15.0 ALTERNATE REPORT FORMAT

Where EBA submits both electronic file and hard copy versions of reports, drawings and other project-related documents and deliverables (collectively termed EBA's instruments of professional service), the Client agrees that only the signed and sealed hard copy versions shall be considered final and legally binding. The hard copy versions submitted by EBA shall be the original documents for record and working purposes, and, in the event of a dispute or discrepancies, the hard copy versions shall govern over the electronic versions. Furthermore, the Client agrees and waives all future right of dispute that the original hard copy signed version archived by EBA shall be deemed to be the overall original for the Project.

The Client agrees that both electronic file and hard copy versions of EBA's instruments of professional service shall not, under any circumstances, no matter who owns or uses them, be altered by any party except EBA. The Client warrants that EBA's instruments of professional service will be used only and exactly as submitted by EBA.

The Client recognizes and agrees that electronic files submitted by EBA have been prepared and submitted using specific software and hardware systems. EBA makes no representation about the compatibility of these files with the Client's current or future software and hardware systems.