Appendix 5-E

Kemess Underground Project Feasibility Study Surface Infrastructure Design Report

KEMESS UNDERGROUND PROJECT

Application for an Environmental Assessment Certificate

Kemess Underground Project (KUG) Feasibility Study
Surface Infrastructure Design Report
Kemess Mine, British Columbia

Submitted to:
AuRico Metals Inc. – Kemess Mines
Smithers, BC

Submitted by:
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VM00575

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EXECUTIVE SUMMARY

This report presents feasibility level design recommendations for the proposed surface infrastructure for the Kemess Underground Project (KUG) at the Kemess Mine located approximately 250 km north of Smithers and 400 km northwest of Prince George, British Columbia. The mine is owned by AuRico and includes the existing Kemess South Mine, an open-pit gold and copper mine that was operational until 2011.

The KUG project is located approximately 5 km north of the Kemess South Mine, and is designed as an access and utility corridor to serve as the main access route to the decline portals throughout the KUG mining operations. During the development period, the corridor will also serve as the haul road for transportation of waste rock to the open pit, and ore to the mill stockpile. The corridor follows the routing of the overland conveyor system connecting the milling and service infrastructure of the Kemess South Mine with the proposed KUG deposit.

This report provides designs and recommendations with respect to the access corridor, infrastructure, portals and an access tunnel.

The access corridor is discussed in four segments; A through to D, commencing at the Mill site and terminating at the Triple Decline Portal. Design and site preparations of the proposed roads are provided. The proposed Kemess Lake Valley infrastructure such as laydown areas, stockpile and dewatering transfer pond are also discussed.

The proposed conveyor system consists of an underground conveyor suspended from the back of the Access Tunnel, and an overland conveyor transporting the ore to the primary crusher. Design recommendations with respect to the overland conveyor and the Transfer Tower building are discussed in this report, with accompanying site preparation recommendations.

Stability analyses were completed for the rock and soil slopes along the proposed portal cuts. The proposed rock cuts at the portals is 75˚ from the horizontal, with minimum recommended factors of safety, for long term stability under static conditions being achieved under drained conditions.

The proposed soil cuts above the portals will be designed at 60˚ at the North Portal and the Triple Decline Portal. Reinforcement in the form of soil nails and mesh will be required to achieve the minimum recommended factor of safety. The proposed soil nails should be spaced on a 1.5 by 1.5 m grid, and inclined at 15˚. Horizontal drain pipes should be installed to remove water from the soil slope.

The overall terrain at the South Portal is gently sloping, which means the soils slopes above the portal will be developed at slopes of 2H:1V, and should therefore not require reinforcement.

The Access Tunnel has been designed to ensure its stability over a long operating life. Recommendations for support requirements and monitoring are provided.

The geochemistry completed for samples across the proposed development indicate Potentially Acid Generating (PAG) rock within select sections of the Access Tunnel only. The remaining areas were considered to be Non-Acid Generating (NAG).

A preliminary construction schedule and corresponding opinion of probably costs was developed as part of the project. The total cost of the project is estimated to be $36.39M, which includes $6.6M of earthworks contractor indirect costs, $6.23M contingency costs, and $2.13M of QA/QC and detailed engineering costs. The estimate includes 15% contingency, except for costs related to portal cuts, canopy and tunnel construction, where 25% contingency was applied to account for complexities and possible winter conditions.

The initial construction schedule suggests that the project will occur over a period of 2.5 years. The schedule is based around development of the Access Tunnel, which is expected to be constructed between Winter 2016 and Summer 2017. The South Portal must be developed first, and is scheduled for the end of Fall 2016. The North Portal should be completed by Summer 2017 to accommodate tunneling operations. The Triple Decline Portal and conveyor installation is scheduled for Fall 2017 and Summer/ Fall 2018 respectively.

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IMPORTANT NOTICE

1.0 INTRODUCTION

AuRico Metals Inc. ("AuRico") engaged Amec Foster Wheeler Environment & Infrastructure ("Amec Foster Wheeler") to conduct a geotechnical site investigation and provide engineering recommendations for the design of surface infrastructure and access tunnels of the Kemess Underground Project (KUG). The general program scope and objectives of the site investigation and engineering design have been outlined previously in the Amec Foster Wheeler memorandum titled "*Kemess Underground Project 2015 Site Investigation and Feasibility Study Update Confirmation of Amec Foster Wheeler Scope of Work"* submitted on 2 June 2015.

This report, *Kemess Underground Project (KUG) Feasibility Study – Access Tunnel and Surface Infrastructure Design Report*, provides geotechnical design recommendations divided into two main areas of focus: 1) surface infrastructure 2) portals and access tunnel, due to geography and different ground conditions which require individual assessments. The information compiled from the 2015 KUG SI program along with 2015 LiDAR data are applied to the designs of this report. Also included are items pertaining to the collection and conveyance of surface water around the portal areas and general surface water management along the corridor. The design of the overland conveyor system (including elevated sections and transfer towers), main power cable and substations, decline ventilation fans and heaters, underground and pit dewatering pipelines are outside the scope of this report, and are provided by others. In order to simplify the main body of this report, design summaries and recommendations pertaining specifically to each section are addressed. In areas where more extensive and detailed design calculations and recommendations are required, reference to individual appendices have been made.

It must be noted that all the site locations described in this report are based on UTM coordinates rather than the Mine Grid system as per discussion with the client.

2.0 PROJECT BACKGROUND

The Kemess Mine property is located approximately 250 km north of Smithers and 400 km northwest of Prince George, in the north-central region of British Columbia as shown in Drawing VM00575.6.600-001. It is owned by AuRico and includes the existing Kemess South Mine, an open-pit gold and copper mine that was operational from 1998 to 2011, and the proposed Kemess Underground (KUG) project, located approximately 5 km north of the Kemess South mine. Amec Foster Wheeler was retained by AuRico to provide engineering recommendations for the design of an access and utility corridor for the Kemess Underground Project. The corridor will serve as the main access route to the decline portals throughout KUG mining operations. During the development period, the corridor will also function as the haul road for transportation of waste rock (to the open pit) and ore (to the mill stockpile) produced during decline and undercut development. The corridor follows the routing of the overland conveyor system connecting the milling and service infrastructure of the Kemess South Mine with the proposed KUG deposit.

Previously, two preliminary design reports were completed for AuRico; an initial design report in November 2012 titled "*Surface Infrastructure Preliminary Design Report"*, and a second report submitted September 2013 titled *"Surface Infrastructure – Alternative Corridor Preliminary Design Report"*. The November 2012 report was to investigate the proposed access corridor to the KUG deposit aligned along the west side of the Kemess Lake Valley (referred to as "KLV" throughout this report) while the September 2013 report was to investigate an alternative corridor alignment for the KUG conveyor and access road routed along the west side of the mined out Kemess South Open Pit to the ridge above and then down to the head of the KLV using a short tunnel to maintain conveyor design grades. The purpose of the alternative corridor alignment was to avoid the avalanche and geotechnical hazards, and potential environmental impacts associated with traversing the west side of the Kemess Lake valley, as documented in AMEC 2012.

This report is intended to advance the 2013 preliminary design of the alternative corridor to a feasibility design level. As such, a geotechnical site investigation (SI) program was completed in August 2015 by Amec Foster Wheeler to support the design of the corridor based on the recommendations of the alternative corridor alignment design report (AMEC 2013). Refer to Appendix A for the details of the SI program, such as the geotechnical site conditions and laboratory results, included within the factual report titled *"2015 Geotechnical Site Investigation Factual Report – Kemess Underground Project (KUG) Access Tunnel and Surface Infrastructure"*. The engineering design and recommendation of the *Kemess Underground Project (KUG) Feasibility Study – Access Tunnel and Surface Infrastructure Design Report*, will be utilized by AuRico for permitting of the access corridor in early 2016 to support construction in late 2016 to achieve collaring of the Access Tunnel South Portal by November 2016.

3.0 ACCESS CORRIDOR AND SURFACE INFRASTRUCTURE

3.1 General

The KUG deposit is accessed via the Triple Decline tunnels with portals located at the north end of the KLV just south of the watershed divide with the Amazay Lake valley. The existing Kemess South Mine infrastructure will be connected to the portals via an access and utility corridor (herein referred to as the "corridor") along the west side of the open pit to the ridge above and then down to the head of the KLV using a tunnel (herein referred to as the "Access Tunnel") to maintain the design grades required for the overland conveyor. The corridor provides both access to the decline portals as well as routing for the overland conveyor system, 25kV powerline, and underground dewatering pipeline. The general alignment of the corridor is shown in Drawings VM00575.6.600-002.

The corridor will serve as the main access route to the portals throughout KUG mining operations. During the underground mine development period the corridor will serve as a haul road for transportation of waste rock and ore produced during decline and undercut development.

Although primary access to the portals will be provided by the corridor, a secondary (emergency) access route is also available using the existing exploration roads that traverse the hillside directly north of the Kemess South open pit. These roads will be upgraded and used for construction access to initiate clearing, grubbing and portal construction activities, eliminating the need for a pilot road for much of the alignment. The emergency road and Kemess South open pit locations are shown on Drawing VM00575.6.600-002.

The corridor is roughly 5 km in length measured from the existing Kemess South primary crusher chamber at the north end of the mill to the face of the KUG Triple Decline Portals at the north end of the Kemess Lake Valley.

3.2 Access Corridor

3.2.1 General

An overall site plan showing the existing Kemess South camp, open pit, Kemess underground subsidence zone, Kemess underground alignments, and access corridor layout is presented in Drawing VM00575.6.600-002. The access corridor consists of four segments, namely segment A through D described in the following subsections.

The proposed corridor alignment has been divided into four segments based on the varying topography and type of infrastructure. In this report, the segments are presented from segment A at the south to Segment D to the north to follow the direction of the planned construction schedule. However, it should be noted that the direction of the station numbering system of the conveyor starts from north to south, in the opposite direction. The reader is warned that this may cause a confusion, but since other consultants working on the same station numbering system we could not change the stationing to prevent further confusion. The following sections provide brief information for each segment from the Kemess South Waste Dump to the Triple Decline Portals. Plan and profile views of the corridor alignment are shown on Drawings VM00575.6.600-003 to VM00575.6.600-007. Refer to the 2015 KUG SI Report in Appendix A for foundation soil type and conditions.

3.2.2 Segment A: Mill to Kemess South Waste Dump – Sta. 4+928 to 2+875

Segment A consists of a side by side road and conveyor alignment from station 4+865 to the edge of the previously disturbed area of the Kemess South Waste Dumps at station 2+875. In order to reduce the construction requirements of a wider road for haul trucks, the main haul/access road throughout Segment A will be separated from the conveyor alignment and connect to the existing Kemess South Main Access Road as shown on Drawing VM00575.6.600-007. A smaller 5 m wide service road will follow the conveyor alignment from station 4+865 to 3+960 in order to facilitate installation and maintenance of the conveyor system.

Segment A is located through a relatively flat section from the mill and gradually slopes northwards at station 3+400, within the existing disturbed limits of the site as illustrated on Drawing VM00575.6.600-006. Refer to Section 7.2 in this report for the cut and fill volumes of Segment A and working areas within the segment.

3.2.3 Segment B: Waste Dump to South Portal – Sta. 2+875 to 1+525

Segment B consists of a side by side road and conveyor corridor that ascends from the edge of the Waste Dump to the South Portal entrance for the Access Tunnel. The corridor alignment traverses previously undisturbed area. Drawings VM00575.6.600-005 and VM00575.6.600-006 shows the plan and profile of Segment B. Refer to Section 7.2 in this report for the cut and fill volumes of Segment B.

A contractor laydown area (designated South Portal Contractor Laydown Area, 29,000 m^2), is located near the South Portal that will serve as a temporary laydown for development of the Access Tunnel portal and access road, as well as a temporary electrical substation for use in the portal and Access Tunnel development. This area will be used to stockpile processed materials for road surfacing purposes.

Designated stockpiling areas #3 and #4, shown on Drawings VM00575.6.600-005 and VM00575.6.600-006, will be used for storage of organic materials produced from stripping during construction. These stockpiles will be used for reclamation of the road corridor upon closure.

3.2.4 Segment C: Access Tunnel – Sta. 1+525 to 0+655

Segment C consists of an approximately 865 m long Access Tunnel, measuring 5.5 m in width and 5.5 m in height at a 10% grade. The Access Tunnel begins at the South Portal (station 1+525), and ends at the North Portal (station 0+655) as shown on Drawing VM00575.6.600-004. Details of the proposed Access Tunnel cross sections are discussed later in this report. It should be noted that the Access Tunnel dimensions, as provided by AuRico, were determined based on the size of the KUG development equipment, providing adequate room to pass through the Access Tunnel.

While the Access Tunnel primarily provides access for the truck and equipment traffic to the Kemess Lake valley (KLV), the overland conveyor will also be suspended from the back of the tunnel, and gradually transitions through initial elevated sections to an on-grade support system at both ends.

In addition, the tunnel has been aligned in a straight line, in order to facilitate construction and enhance traffic visibility within the tunnel (one way traffic only). There will be at least three 20 m long passing bays equally spaced along the Access Tunnel, to reduce congestion and delays of the haul trucks (radio controlled traffic). Refer to Section 7.2 in this report for the cut and fill volumes of Segment C, and Section 4.4.6 for additional details on the design of the Access Tunnel.

3.2.5 Segment D: North Portal to Triple Decline Portals – Sta. 0+869 to 0+093

Segment D connects the Triple Decline Portals to the North Portal of the Access Tunnel. Supporting infrastructure in Segment D includes the connecting Main Access Road, ore and waste rock stockpiles, office and contractor shop and laydown areas, electrical substation, runoff collection ditches and sedimentation pond, underground dewatering transfer pond, propane tank farm, service roads, on-grade and elevated sections of the overland conveyor (CV-002), and a transfer tower. The transfer tower is required to transition from the underground conveyor (CV-001) to the overland conveyor (CV-002). The proposed location of the transfer tower is shown on Drawings VM00575.6.600-003 and VM00575.6.600-008. The conveyor system and the transfer tower has been designed by Conveyor Dynamic Inc. (CDI). The underground conveyor CV-001 is designed to be suspended from the back of the Triple Decline Tunnel. A section of the CV-001, between the face of the Triple Decline Portal and the transfer tower, has been designed to be elevated (trestle supported). The CV-002 conveyor exits the transfer tower on-grade through a large culvert that allows the Main Access road to cross above the conveyor. From station 0+185 to 0+255 and 0+290 to 0+375 the conveyor will be supported by elevated galleries as designed by CDI. The remaining sections the conveyor will be supported on-grade, using a 9 m wide road to allow access for installation and maintenance. At station 0+540, near the Access Tunnel North Portal, the conveyor becomes elevated once again, such that it can be suspended from the back of the Access Tunnel. The elevated conveyor cross the service road twice stations 0+220 and 0+340, with minimum clearance of 3.5 m at the first crossing, measured from the finished road grade to the underside of the trestle.

The local terrain at KLV is too steep for the Main Access Road to parallel the conveyor alignment. This means that a separate haul road and service road is required, to cross the valley and access the portals and surrounding infrastructure. The Main Access Road alignment begins at the face of the North Portal, and connects to the Triple Decline Portals platform from station 0+869 to 0+000. A loop at the Triple Decline Portal has been provided to allow for return of larger vehicles with trailers. The Main Access Road alignment at KLV is shown on Drawing VM00575.6.600-002 and 008. The access road generally follows the topography along the head of the valley with a maximum grade of 9.3% near the North Portal. The road width along this segment is 12 m, which should provide sufficient width for passing of haul equipment (two way traffic). The road is also routed close to the toe of the ore and waste stockpiles, laydown areas and stripped organics stockpile as shown on Drawing VM00575.6.600-008. This permits roadside access and loading of haul equipment off the end of the stockpiles.

In addition to the Main Access Road, four 5 m wide service roads have been provided, to access various parts of the KLV infrastructure as shown on Drawings VM00575.6.600-008. The first service road is located along the conveyor between stations 0+400 to 0+609. The second service road connects the contractor shop and laydown area to the KUG dewatering transfer pond and pump house, and eventually to the conveyor alignment at approximately station 0+260. The third service road parallels the conveyor from the transfer tower to the sedimentation berm. Portions of this road are parallel to the conveyor, as well as under the conveyor, allowing access to erect the elevated sections of the conveyor. The fourth service road diverts from the Main Access Road at station 0+120 to access the propane tank farm south and below the Triple Decline Portal.

3.3 Road Construction

3.3.1 General

The plan and profile of the Main Access Road is shown in Drawing VM00575.6.600-005, 006, 007 and 009. The cut and fill slopes have typically been designed at 2H:1V, except for the slopes between stations 2+700 to 3+360, which have been designed at 3H:1V. Typical road sections for the KUG surface infrastructure is presented on Drawing VM00575.6.600-010. The designed cut and fill slopes were determined based on information obtained from the previous Duncan Lake terrain hazard assessment by Weiland (2003), and the 2015 KUG SI program. Both assessments indicated that the majority of the road and conveyor cut will be terminated within overburden soils (sand and gravel, and glacial till) or weathered bedrock. The materials between stations 2+700 to 3+600, consist of soft overburden fill from the previously mined open pit, therefore, more shallow slopes are recommended.

The Main Access Road and service road fills should be constructed using well-drained, structural fill materials in accordance with the guidelines outlined in section 3.3.5 below. In general, the existing smaller access roads onsite have been constructed using a side casting methodology, where the cut spoils are simply dumped or pushed to the fill side of the road. This method of construction is not considered to be appropriate for the construction of the KUG access corridor, as it represents a vital component for the proposed underground mine, in addition to also housing the conveyor, power line, and dewatering pipeline.

During construction of the Main Access Road and service roads, underlying subgrade material properties should be examined and confirmed to match the road design. If the soil properties differ substantially from those assumed in the designs, updates to the road designs may be required, to adjust the cut and fill slope angles as well as the material handling. However, specific consideration to the conveyor alignment and grades will be required as these are relatively sensitive to changes in the road alignment.

3.3.2 Existing Exploration Road Improvement

Prior to construction of the main corridor right-of-way, the existing exploration roads could be used for temporary access for construction equipment. However, some of these roads will require upgrading, and widening to permit safe access. It is anticipated that such upgrades are required to gain access to the KLV, for development of the North Portal prior to tunnel breakthrough and Triple Decline Portals. The vent raise access road will also require some improvement of existing exploration roads, to maintain reliable access. Pilot roads within the clearing width should not be required, as the majority of the alignment can be accessed using the existing network of exploration roads, with some improvements and extensions. In isolated areas of less accessible or steeper terrain, it may be necessary to construct a pilot road, which is typically aligned below the flagged centreline on side hills, near the lower clearing width limits. All necessary culverts should be installed coincident with such pilot road construction or initial clearing and grubbing activities.

3.3.3 Right-of-Way Clearing

The corridor right-of-way shall be cleared of all standing timber within the road prism (extents of cut and fill limits), and to a distance of 3 m upslope from the prism, to avoid undercutting roots that may create falling tree hazards, and destabilizing the top of the road cut (MOF 2002).

The trees within the undisturbed areas of the access corridor right-of-way are assumed to be non-merchantable, and less than 12 inches in diameter. It should be noted that all previous site timber cruises have returned non-merchantable timber. Therefore, all trees are assumed to be felled, piled and burned locally along the right-of-way in accordance with BC forest management best practices.

3.3.4 Grubbing and Stripping

After all standing trees within the right-of-way have been felled and burnt/removed, the road prism should be grubbed and stripped of all topsoil and unsuitable mineral soils (assumed to be an average of 0.3 m based on 2015 test pitting). Grubbing includes the removal of stumps, roots, logging slash, and downed or buried logs. Stripping includes the removal of topsoil, or other organic material, and mineral soils unsuitable for use in the road subgrade. Where grubbing operations have removed all organic soil, no stripping is required unless other unsuitable soils are encountered.

For Segment D including the portal laydown, roads and stockpile areas, all grubbing and stripping spoil is to be pushed to an area that can be hauled and dumped in a designated stockpile area shown on Drawing VM00575.6.600-008. The stockpile stripped materials should be stored for potential use in reclamation of the portal area at closure. For Segment B grubbing and stripping spoil along the access road, within the laydown area, and South Portal is to be loaded and hauled to a designated waste stockpile area shown on Drawing VM00575.6.600-005. All grubbing and stripping materials in Segment A are to be hauled to waste stockpile area shown on Drawing VM00575.6.600-006.

3.3.5 Road Fill Construction

All fill zones should be constructed in a bottom-up manner by placing the fill materials in horizontal lifts with a maximum lift thickness of 0.3 m, compacted using loaded haul trucks or roller compactors to 100% Standard Proctor Maximum Dry Density (SPMDD). Locally excavated materials such as weathered bedrock, well drained colluvium, sand, glacial till or borrow rockfill, approved by the engineer could likely be used for the fill zone areas. The fill should be keyed-in or notched into the slope in a stepwise manner as it is constructed up the slope to eliminate a soft toe condition. In some steeper areas, this may require the construction of an approximately 5 m wide step along the toe of the fill.

Properly compacted fills have a higher load-carrying capacity, and tend to shed water rather than absorb it. This results in a more stable, erosion-resistant subgrade, which requires less maintenance while minimizing the potential for adverse environmental impacts (MOF 2002).

In the KLV, it is anticipated that silt fencing or erosion control revegetation mats will be required to control surface erosion, and subsequent sediment transport during active road construction. The contractor should provide an appropriate sediment and erosion control plan prior to the start of construction that is in line with BC forest management best practices. Additional discussion on culverts and runoff collection are provided in Section 3.3.7.

3.3.6 Road Surface Capping Considerations

It is anticipated that the natural overburden materials present along much of the alignment will provide suitable road capping material. However in areas where coarse rockfill or fine grained (silty) materials are exposed at the surface from excavation activities, 0.3 m of road capping material should be placed to provide a smooth running surface. In previous mining operations this was possible by sorting material at the borrow sites, such that fine rockfill was used for the final lift, and graded until smooth with successive passes of the grader. The final road surface should be sloped at 3% towards the hillside to promote drainage of the roadway into the roadside ditches.

3.3.7 Roadside Ditches and Culvert Crossings

Drainage swales and ditches should be constructed on the uphill side of the corridor, to collect and convey hillside drainage along the corridor. The drainage ditches will connect to seasonal drainages where culverts will be installed to convey the runoff across the corridor. Silt fencing and rock check berms should be installed along the ditches, to reduce sediment transport to the culvert crossings and thus the environment.

The hillside catchment area for both Segments A and B is roughly 50 ha in size, reporting to the culvert crossings which contribute a maximum peak instantaneous flow of 1.0 m^3/s based on correlation with the BC peak flood maps for the 1:200 year, 24-hour storm event (snowmelt not included) (from British Columbia Streamflow Inventory by C.H. Coulson and Q. Obedkoff, March 1998). In order to pass such a design flow, a minimum culvert diameter of 900 mm is required at a grade of 5.0% (assuming HDPE with manning's n = 0.012). The typical culvert crossing detail is shown on Drawing VM00575.6.600-011. It is assumed that a sufficient quantity of 900 mm (or larger) diameter pipe (HDPE) is available onsite (left over from Kemess South Operations), to be used for culvert crossings. An existing culvert traversing under the Main Access Road approximately at conveyor alignment station 3+500 (Drawing VM00575.6.600.006) should be replaced with an appropriately sized culvert for the highwall diversion ditch (AMEC 2012b). In general, for culverts installed through large road fills, additional riprap armoring around and below the pipe inlet and outlet, or the installation of a flume down the fill slope may be required to prevent erosion of the road fills. Additional riprap materials should be field fitted downstream of the culvert flume, to disperse flow and to reduce local erosion, as required based on the slopes and topographical conditions.

3.4 KLV Infrastructure

3.4.1 Laydown Areas

In order to construct the KLV components; one large laydown (designated Contractor Shop & Laydown area) and a smaller laydown pad (designated Office Laydown) will be developed as illustrated on Drawing VM00575.6.600-009, accessible directly from the Main Access Road. It is anticipated that the Office Laydown will be developed first, since it will be used to facilitate initial mobilization of equipment and materials, to begin construction of the North Portal, and the Main Access Road between the triple decline and the Access Tunnel portals. The Office Laydown area provides an initial 2,400 m^2 of space accessible by an existing exploration road. The excavated material from development of the portal cuts and connecting portal haul road, will be utilized to construct the remaining laydowns and pads. The Contractor Laydown will provide an additional approximately 11,500 m^2 of maintenance and warehousing space, to support construction of the local infrastructure, mine development and eventually operations. The permanent KUG surface electrical substation (by StruthersTech) will also be housed on the edge of the Contractor Laydown as shown on Drawing VM00575.6.600-009.

3.4.2 Decline Portal Area Stockpiles

As shown on Drawing VM00575.6.600-009, the area adjacent to the Triple Decline Portal has been designated for development ore and waste stockpiling, prior to transporting (surface hauled) to the mill and open pit, respectively. The stockpile configuration shown, is based on loose dumping on a flat platform ground (elevation 1384.6 m), stripped of topsoil and excavated with 2H:1V cut slope at the upstream side. The ore and waste rock fill slope angle are 2H:1V and constitutes storage for approximately 24,000 m^3 (46,000 dry metric tonnes) of ore and waste rock (assuming the dry density of the dumped material is 1.9 tonnes/m^3). The stockpile area is anticipated to provide storage space for just over 50% (roughly 1.5 months), of the anticipated quarterly waste production rate during Triple Decline development, which averages about 75,000 tonnes/quarter or roughly two weeks of ore storage during mine development, based on the mine development plan provided by AuRico on October 28, 2015. The total tonnage of ore and waste rock to be stockpiled and hauled along the corridor is estimated at about 1.8 million tonnes between 2017 and Q3 2020.

3.4.3 Underground Dewatering Transfer Pond

The pipeline and associated dewatering pump house were previously designed by Tetratech as part of the 2012 FS, and did not form part of Amec Foster Wheeler scope of work. The underground dewatering system will include two sumps and pumps, which will pump daily water inflows including seepage and processed water directly from the sumps, to a dewatering transfer pond at surface. A suitable area for the KUG dewatering transfer pond and pump house has been identified such that a pond can be constructed with minimal excavation work. The KUG dewatering transfer pond and pump house pond location is shown on Drawing VM00575.6.600-012 and VM00575.6.600-013.

This pond, which will act as collection/sedimentation pond for contact water, will be lined to prevent contamination, and will host two pumps with 1400 gpm (88L/s) capacity. The pumps will be driven by a 725 hp (540 kW) motor against a total dynamic head of 403 m. Some of the water in this pond will be reused for underground process water, while the remaining will be pumped into the open pit, via a 200 mm pipeline that runs through the conveyor structure. It is anticipated that one pump will be in use primarily, with the second pump being a backup, available for maintenance or on standby during the operation.

3.4.4 Vent Raise Access Road Extension

The KUG exhaust ventilation raise is located in a high alpine saddle at about El. 1900 m as shown on Drawing VM00575.6.600-002. The initial mine design by AuRico, required year around access to the vent raises, to service and provide propane to the heating units that will heat the intake air to the underground mine (AMEC 2012a). However, a third decline with heating capabilities was added to the portal area, eliminating the requirement for year round access to the vent raise.

To provide access during construction and ongoing seasonal maintenance, the vent raise area will be accessed via a 1.2 km extension to the existing exploration roads that start on the watershed divide west of the portals, and head north up the mountainside. It is assumed that the new extension road will not require large volumes of cut and fill, as the alignment will traverse relatively gentle alpine topography to the saddle area. The condition of the existing exploration road with grades from 0 to 30 %, is currently unknown. However it is assumed that some upgrading will be required, in order to accommodate construction and maintenance equipment (maximum 3 m equipment width) in accordance to the MOF (2002) guidelines.

The total length of the existing exploration road from the office laydown areas of KLV to the vent raise, including the road extension, is approximately 5,200 m. The vent raise access road, including the extension, emergency road/access roads are shown on Drawing VM00575.6.66-002.

3.4.5 Propane Tank Farm

A secondary platform will be developed below the Triple Decline Portals to host a 30,000 US gallon (340,000 liters) propane tank farm, to feed the heating units that will heat the intake air to the underground mine as designed by Mine Ventilation Services, Inc. (Stinnette, December 2015 E-mail). Each tank is 3 m (10 feet) in diameter and 11 m (37 feet) long. The peak propane consumption rate was estimated at 13,000 l/day, such that the tanks were estimated to provide approximately 3 weeks-worth of propane should the camp site main road be closed during a major winter storm event. The design criteria was based on a 2-week road closure plus a safety factor of 1.5 for a total storage of 3 weeks. The propane tank farm platform is shown in Drawing VM00575.6.600-003.

3.4.6 Concrete Plant

Concrete and shotcrete is required for KUG infrastructure construction. Concrete will be used for the support of the overland and elevated portions of the conveyor, as well as underground road ways and draw points. Shotcrete application will include the access and decline tunnels, portals soil and rock stabilization and various underground applications. Concrete and shotcrete will be delivered from the concrete plant directly to the required locations along the corridor, and underground via transmixers.

In order to produce the required concrete and shotcrete volumes, a concrete batch plant has been designated at the east side of the existing Kemess South mill with an area of approximately 9,100 m^2. The designated area shown on Drawing VM00575.6.600-002 will include aggregate bins, conveyors, control room, two silos for cementitious materials and aggregate stockpiles. The concrete plant will be sized to generate roughly 50 m^3 of concrete and shotcrete per day based on the underground development quantities provided by AuRico.

A proposed concrete plant provided by Multicrete System Inc. has been provided in Appendix B of this report. It should be noted that proposed concrete plant is capable of producing up to 40 m^3 concrete product per hour, given all required aggregates are available on site. Since the volume of the required concrete/shotcrete is considerably lower than the proposed plant production capability, a smaller concrete plant may be considered in the subsequent design phase of the project.

The concrete plant will require the following five operators:

1- Control room operator
2- QC person
3- Two drivers
4- One labourer.

It should also be noted that the proposed concrete plant cost estimate provided in Appendix B is not included in our cost estimate, provided herein, to prevent duplication. We assumed that the cost of the concrete plant will be captured under general cost estimate provided by SRK.

3.4.7 Explosives and Detonator Magazines

All explosive magazines will be delivered to site by an explosives truck from the existing Kemess South bulk explosive plant (BXL) located between pump house #2 and the ball diamond laydown area. There will be no need to construct a magazine facility at KLV, as the existing facility can provide the required services. All magazines and detonators used for the open pit, are still current, and suitable to use for the underground development. These will be transported to the KUG facilities when required.

3.5 KLV Sedimentation Pond

The surface water management of the upper KLV catchment area, consists of culverts and ditches parallel to the Main Access Road that will be collecting the water, and routing it to the natural pond at the bottom of valley previously designated as Condor Creek Pond B (Condor Pond). As part of this design, the Condor Pond will be upgraded to operate as a sedimentation pond (KLV Sedimentation Pond). At this stage the design of the sedimentation pond is at conceptual level. The design of the upgraded sedimentation pond is based on the BC Guidance for Assessing the Design, Size and Operation of Sedimentation Ponds Used in Mining (Guidance Sedimentation Pond Design) (BCMOE, 2015).

The KLV sedimentation pond is located approximately 250 m east of the main haul road alignment as shown in Drawing VM00575.6.600-008. Since the runoff water from the upper KLV catchment area may come in contact with the disturbed area, the current Condor Pond will have to be upgraded to capture sediments.

This section describes the design basis for the sedimentation pond. The secondary road ditches, portal diversion and culverts will be assessed in detail at the detailed design stage, but generally should be designed to pass the 1 in 50 year, 24 hour storm event.

3.5.1 Design Criteria

According to the Guidance for Sedimentation Pond Design (BCMOE, 2015) for sedimentation ponds, the design flow for removal of suspended solids should correspond to the 1 in 10 year, 24-hour flood flow. The sedimentation pond should also have an emergency spillway which is designed to route the 1 in 200-year flood, with a freeboard of 0.5 m.

The KLV sedimentation pond reassessment was based on the following available data and assumptions. Refer to Table 3.1 below for the summary of data used in the design.

Table 3.1 – Summary of Data for Condor Sedimentation Pond Design

Data Type	Reference	Key Values
Topography	1m Topo LIDAR provided by Kemess mine. (2015)	Catchment area: 0.62km^2
Existing Condor Creek Pond B. Bathymetry	(Hatfield Consultants, 2013)	Pond Capacity: 17,000m^3 Surface area at Elev:1314m = 4,700m^2
Design Storms	(Knight Piesold, 1996)	• 1 in 10 peak storm: 48mm • 1 in 200 peak storm: 67mm

The following assumptions were made as part of this design:

- Particle size assumed: 10 micron
- Settling Velocity assumed: 5 x10^{-5} m/s
- Particles are spherical and smooth with Specific Gravity of 2.7
- Surface elevation of the bathymetry data was 1314 m
- Runoff coefficient is assumed to be 0.8 for average flow and 1 in 10 return period storm
- At the start of the 1 in 10 year storm, the pond will be at maximum normal operational level (1311 m)

3.5.2 Methodology

The following values are estimated as part of evaluating the upgrade of the sedimentation pond:

- Maximum monthly inflow rate into the pond;
- Total volume generated by the 1 in 10-year storm; and
- The 1 in 200 year peak flow.

The annual average precipitation was estimated as 715 mm from the program Climate, BC (UBC, 2014). This value was verified with the Annual Precipitation map from the Atmospheric Environment Service (1975).

Maximum monthly precipitation is approximately 93 mm, based on the monthly precipitation of the nearby Wire Station, which is approximately 115 km from the site. The average monthly runoff from the maximum monthly precipitation of 93 mm, is equivalent to 0.02 m^3/s based on the catchment area of 0.62 km^2 and a runoff coefficient of 0.8. Refer to Table 3.2 below for the monthly precipitation of Wire Station.

Table 3.2 – Monthly Precipitation at Kemess Site and Wire Station

Period of Record	Kemess Site Estimated Monthly Precipitation (mm)	Monthly Distribution from Wire Station	Wire Station (1966-1987) (mm)
Jan	70	10%	43.2
Feb	49	7%	30.3
Mar	24	3%	14.9
Apr	17	2%	10.7
May	63	9%	39.1
Jun	70	10%	43.6
Jul	93	13%	57.6
Aug	74	10%	46.2
Sep	64	9%	39.7
Oct	50	7%	30.9
Nov	60	8%	37.3

Period of Record	Kemess Site Estimated Monthly Precipitation (mm)	Monthly Distribution from Wire Station	Wire Station (1966-1987) (mm)
Dec	80	11%	49.6
Total	715		443.0
Max	**93**		57.6

According to Knight Piesold (1996), a 1 in 10 year storm is 48 mm as presented in Figure 3-1. The total volume generated from the 1 in 10 year storm based on a runoff coefficient of 0.8 and catchment area of 0.62 km^2 is approximately 24,000 m^3.

The peak flood resulting from the 1 in 200 year storm based on the BC Peak Flow Maps information (Ministry of Environment, Lands and Parks, 1998) is 1.35 m^3/s, which was used for spillway design.

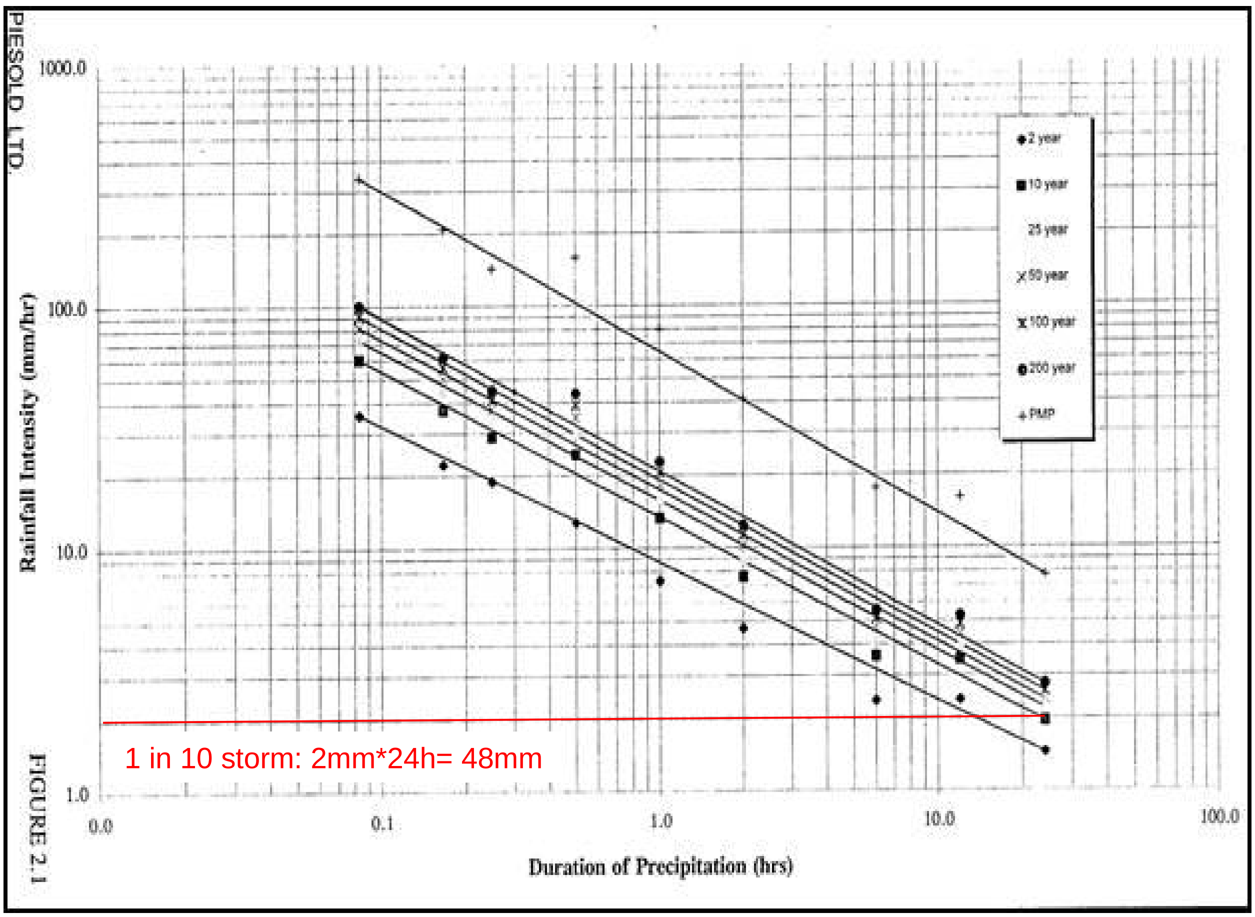

Figure 3-1: Storm Design for 1 in 10 Year Storm (Knight Piesold, 1996)

3.5.3 Sedimentation Pond Design

The assessment was based on Method 3 suggested in the Guidance for Sedimentation Pond Design (BCMOE, 2015). This method is based on the conservative assumption that the particles' size is between 5-10 micron, and that the settling velocity will be in a range of $2x10^{-5}$ m/s to $5x10^{-5}$ m/s. Based on the settling velocity of $5x10^{-5}$ m/s it will take a particle 8.4 hours to settle over the minimum required sedimentation pond depth of 1.5 m.

Based on the 2013 bathymetry for the pond, at the assumed surface elevation of 1,314 m the pond is up to 7 m deep with a with a total volume of 17,000 m^3 and surface area of 4,700 m^2.

Taking advantage of the natural geometry of the existing pond, the design of the sedimentation pond involved the following:

- Increasing the pond storage capacity by constructing a downstream embankment of 2.6 m;
- Install a decant system to regulate and control normal operation discharge and the 1 in 10 year flood from the pond; and
- Install a spillway to route the 1 in 200 year flood.

Refer to Drawing VM00575.6.600-012 for details of the sedimentation pond design.

Decant Structure Design

The decant structure will control the discharge from the sedimentation pond. The decant structure has a lower outlet and upper outlet. During normal operation, the lower decant outlet will discharge up to the maximum monthly average rate (0.02 m^3/s).

During the 1 in 10 year storm event, the upper outlet weir and lower outlet will discharge a maximum of 0.15m^3/s and 0.045m^3/s respectively, for a total of 0.2 m^3/s. Based on these discharge rates the surface loading to meet the Guidance Sedimentation Pond Design (BCMOE, 2015) is approximately 4,000 m^2. This is less than the surface pond area, demonstrating that the pond will meet surface loading requirement.

The decant structure proposed includes a 12" pipe (upper outlet) with two 2" holes (lower outlet) perforated at the bottom. The basis of the decant design is presented in Table 3.3.

Table 3.3 – Decant Structure Design Parameters

Methodology of Design	Orifice	Overtopping Weir
Outlet bottom elevation	1310	1315
Discharge Coefficient (C)	0.6	1.7
Maximum Head (m)	5	0.2
Diameter (inches)	2.75	12
Maximum discharge in a 1 in 10 storm event (m^3/s)	0.045	0.15

Spillway Design

Spillway depth and width were estimated using the general broad crested weir equation. The basis of the spillway structure design is provided in Table 3.4 below.

Table 3.4 – Spillway Structure Design

Parameter	Value
24 hours Design Flow. 1 in 200 year (m^3/s)	1.35
Discharge Coefficient (C)	1.5
Width (m)	2
Water depth (m)	0.6
Freeboard (m)	0.5
Invert Bottom Elevation (m)	1315.5

3.5.4 Design Recommendations

Based on the assessment provided, the current Condor Creek Pond B could be upgraded as a sedimentation pond. Figure 3-2 and Table 3.5 provide summary of the design.

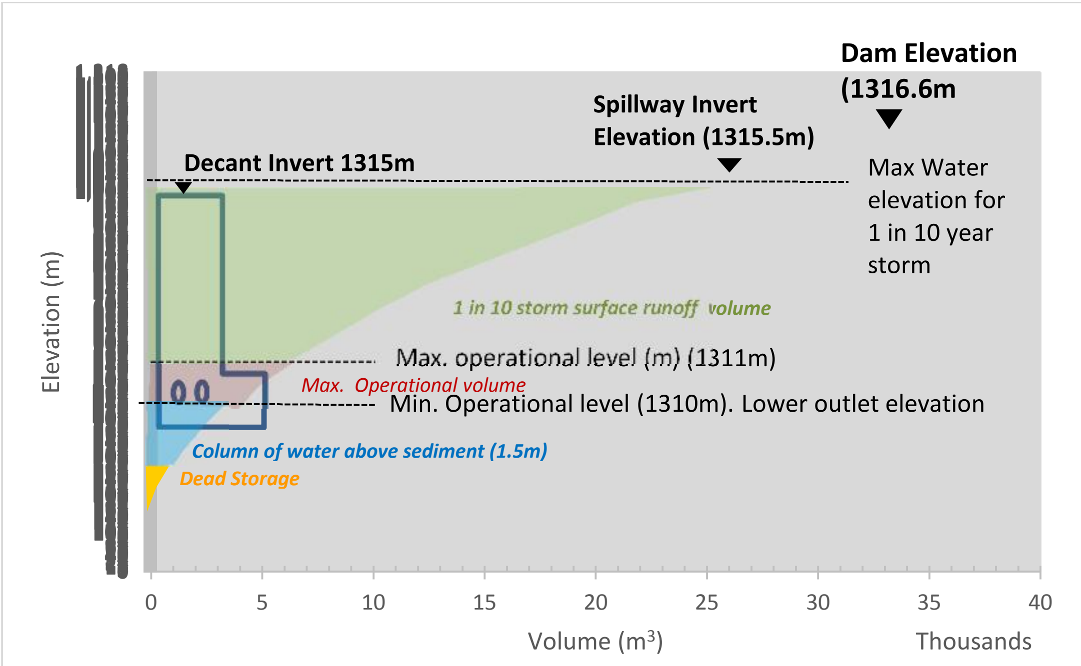

Figure 3-2: Storage Elevation Curve. Combined data sources: (Hatfield Consultants, 2013) and 1 m LIDAR.

Table 3.5 – Key elevations and volumes for design

Parameters	Elevation (m)	Cumulative Volume (m^3)	Lower Outlet Discharge (m^3/s)	Upper Outlet Discharge (m^3/s)	Emergency Spillway Discharge (m^3/s)	Total Discharge (m^3/s)
Pond bottom	1307.5	0	0	0	0	0
Dead Volume	1310	3,972	0	0	0	0
Maximum normal operational level	1311	6,570	0.02	0	0	0.02
Decant Invert	1315	23,613	0.045	0	0	0.05
Maximum water elevation during 1 in 10 year flood	1315.2	25,602	0.046	0.15	0	0.20
Emergency Spillway Invert	1315.5	26,929	0.047	0.58	0	0.62
Maximum water elevation during 1 in 200 year flood	1316.1	31,307	0.050	1.16	1.35	2.56
Dam elevation	1316.6					

The sedimentation pond has been designed to conceptual level as presented in Drawing No. VM00575.6.600-012. As part of the feasibility or detailed design, it is recommended that the following should be evaluated:

- Update of site hydrology (design storms and design flows).

- Collection of soil samples in the catchment area for sieve analysis to estimate the particle size distribution of sediments.
- Evaluation of sediments specific yield for the catchment area.
- Review of site monitoring data for TSS including any data collected downstream of the sedimentation pond. If no data exists for the downstream of the sedimentation pond, TSS should be monitored.
- Demonstrate that the sedimentation pond will meet discharge total suspended solids discharge criteria in accordance with the mine permit.
- Detailed survey of the sedimentation pond site.
- Consider using a 2D finite element or finite difference dynamic model or other approach to estimate the flow pattern in sedimentation pond. This will help strategically place a berm to limit short circuits and force the water and sediment to travel the greatest distance.
- Evaluation of the decant outlet structure. This will include development of more detailed operation rules and consideration for constructability and protection of lower outlet from clogging due to debris. Consider including screens to limit debris entrainment.
- Alignment of outlet channel; evaluate options and design for erosion protection and design of a stilling basin.

3.6 Conveyor Foundation Design

3.6.1 General

The KUG project will utilize the existing processing plant at the Kemess South mine to mill the ore material. This ore material, along with generated waste rock, will be transported by means of a conveying system. Conveyor Dynamics Inc. (CDI) was retained by AuRico to provide the design for the conveyor system. According to the CDI design report, the conveyor system consists of:

- Feeder FE-01: A straight 8m long belt feeder, located in the underground mine, which withdraws material from the surge pocket below the primary crusher.
- Conveyor CV-001: a straight uphill underground conveyor that transports material to surface. It will receive the ore and waste rock from the primary crusher located underground and transport it to the transfer tower, located just outside of the triple-decline tunnel portal. It is approximately 3.21 km long and gains approximately 305 m of elevation.
- Conveyor CV-002: an overland conveyor that transports material from the transfer tower to an existing stockpile conveyor at the Kemess South ore processing plant. It is approximately 4.93 km long and loses approximately 98 m of elevation (Drawing VM00575.6.600-003).

The foundation recommendations provided herein pertain to the overland conveyor only (CV-002). Since the conveyor CV-001 is underground, and suspended from the roof of the tunnel, it will not be discussed any further in this section

3.6.2 Conveyor CV-002

Drawings VM00575.6.600-003 to VM00575.6.600-007 outline the alignment of conveyor CV-002, and include a profile view along the whole alignment. The conveyor initiates from the transfer tower (station 0+000) and extends to the existing primary crusher chamber at the Kemess South ore processing plant (Station 4+900). CV-002 starts from the transfer tower and run through an approximately 70 m long buried culvert underneath the access road (stations 0+005 to 0+075). The conveyor then crosses the KLV (stations 0+075 to 0+637) and through the Access Tunnel (stations 0+637 and 1+523). Once out of the Access Tunnel, it continues along its route to the primary crusher (stations 0+637 and 1+523).

Most of the conveyor is founded on surface and will be serviced by an access road. The exceptions are the elevated sections, located approximately at stations 0+190 to 0+255 and stations 0+285 to 0+380, where access to the conveyor is provided by walkways.

3.6.3 Scope of Work

Based on the results of the 2015 site investigation (details of which are included in Appendix A) the following geotechnical design recommendations for conveyor CV-002 have been completed:

- An overview of terrain and subsurface stratigraphy encountered along the conveyor alignment;
- Recommended foundation sizing based on the loadings provided by CDI for the on-surface and elevated conveyor, consistent with limit states design methodology;
- Foundation recommendation for Transfer Tower building (Station 0+000 in Drawings VM00575.6.600-009 and 011), consistent with limit states design methodology;
- Foundation and design support recommendations, consistent with limit states design methodology, for selection and design of the conveyor culvert crossing (Station 0+005 to Station 0+070, Drawings VM00575.6.600-011); and
- General considerations which include recommendations for selection of cement type, recommendations for buried steel structures and seismic site classification.

3.6.4 Overview of Subsurface Stratigraphy and Soil Properties

A total of 9 boreholes and 35 test pits were completed along and adjacent to conveyor CV-02 alignment. Borehole and test pit logs should be referred to in Appendix A.

Considering terrain and subsurface conditions, the conveyor alignment is divided into the following four sections:

<u>Station 0+000 to Station 0+630</u>

This portion of the overland conveyor crosses the KLV. As can be seen in Drawing VM00575.6.600-003, there is an elevation difference of approximately 50 m along the conveyor alignment, with slope angles of 25-30 degrees along the alignment, and 20-25 degrees in the traverse direction.

According to the terrain assessment carried out as part of the 2015 SI program, the soil deposits encountered on surface at the KLV were identified as glacial deposits comprising outwash, glacio-fluvial and tills. Several test pits and boreholes were drilled along this section of the conveyor, with a maximum drill depth of 27.7 m.

The subsurface stratigraphy generally consisted of coarse-grained native deposits (sandy gravel, gravelly sand and/or sand), with occasional cobbles and varying amounts of fines, ranging between <5% to 30%. The only exception is TP15-09, located near the North Portal at approximately station 0+480, where a 2 m thick silt deposit was encountered at a depth of 0.5 m.

The SPT N-Value results were plotted versus depth for the boreholes completed along the conveyor alignment in the Kemess Lake valley. Refer to Figure H1 in Appendix H.

<u>Station 0+630 to Station 1+536</u>
This section of the conveyor crosses through the Access Tunnel as shown in Drawing VM00575.6.600-004, and it is discussed further in Section 4.4.

<u>Station 1+536 to Station 2+800</u>
As shown in Drawings VM00575.6.600-004 and 005, the topography in this section generally consisted of a gently sloping ground towards the south, with a total elevation loss of approximately 100 m over a distance of approximately 1,264 m.

The soil stratigraphy generally consisted of top soil (up to 0.7 m in thickness with an average of 0.3 m) underlain by sand/gravelly sand deposits of varying thicknesses, over weathered to highly weathered bedrock. Bedrock was encountered anywhere between 0.5 m and 3.0 m below ground surface, with the exception of one test pit, TP15-23, where bedrock was not encountered in the 4.1 m deep excavation.

<u>Station 2+800 to Station 4+900</u>
The topography in this section generally consisted of a gently sloping ground towards the south between station 2+800 and 3+750, and a relatively flat ground surface between stations 3+750 and 4+900. The total elevation loss is approximately 100 m, primarily between stations 2+800 and 3+750. This section is shown in Drawings VM00575.6.600-006 and 007.

The subsurface stratigraphy in this section generally consisted of waste rock and/or fill encountered at ground surface and extending beyond the depth of the test pits (typically at 5 m). The major exceptions are:

- TP15-33 where waste rock was encountered to a depth of 2.5 m and was underlain by 0.2 m of topsoil and bedrock; and

- Test pit TP15-37 where waste rock extended to a depth of 1.2 m and was underlain by native ground consisting organic silt (1.2 – 1.3 m), gravel/sand (1.3-1.7 m) and below 1.7 m.

The SPT N-Value results were plotted versus depth for the boreholes completed along the conveyor alignment south of the Access Tunnel, between stations 1+536 and 4+900. Refer to Figure H2 in Appendix H.

3.6.4.1 Groundwater Conditions

Ground water conditions along the conveyor alignment were evaluated based on seepage observations made during and upon completion of borehole drilling and excavation of test pits.

In general, groundwater was not encountered in the majority of the test pits and boreholes completed along the conveyor alignment. Except at stations 0+470, 1+60 to 1+840 and 3+330, where seepage was observed.

It should be noted that due to seasonal fluctuations, the seepage conditions may vary from those encountered during the field program.

3.6.4.2 Frost Depth and Heave Considerations

A seasonal frost depth of 2 m, assuming snow cover, was adopted for assessment of potential of frost heaving and foundation design. This value is reasonable and close to frost depth values recommended by Amec Foster Wheeler on nearby projects.

The potential of frost heaving was assessed on the basis of an assumed frost depth of 2 m, a freezing time of 2880 hours (i.e. 4 months), groundwater conditions and fines content of the soils in the top 2 to 3 m.

Two subsurface conditions were used to assess frost heaving potential; the first represents the areas where no groundwater was encountered, while the second represents areas where groundwater conditions were observed in the top 2 to 3 m. Refer to Appendix C for summaries of soil index properties in the top 3m and groundwater observations for the two subsurface conditions.

It was determined that there is a low potential for significant heaving in areas where no groundwater was encountered in the top 2-3 m. However, where groundwater seepage was encountered in the top 3 m and based on the soils fines content, differential frost heave could be up to 15 mm.

It should be noted that if the snow cover is reduced in thickness, disturbed, or removed, the seasonal frost penetration depth could be considerably greater.

3.6.5 Foundation Recommendations

The foundation recommendations provided in this section are in general accordance with Limit States Design (LSD) methodology, and include the conveyor, transfer tower building and culvert crossing design.

3.6.5.1 Limit States Foundation Design

Limit states are defined as conditions under which a structure or its component members no longer perform their intended function, and are generally classified into the main groups of ultimate limit state and serviceability limit state. Each of these limit states are discussed in more detail below.

Ultimate Limit States

Ultimate Limit State (ULSs) are primarily concerned with collapse mechanisms for the structure and, hence, safety. Foundation designs using a limit states design approach is described in detail in the Canadian Foundation Engineering Manual (CFEM) 2006, and should satisfy the following design equation:

$$\Phi R_n \geq \Sigma \alpha_i S_{ni}$$

where:

ΦR_n	is the factored geotechnical resistance.
Φ	is the geotechnical resistance factor.
R_n	is the nominal (ultimate) geotechnical resistance determined using unfactored geotechnical parameters.
$\Sigma\alpha_i S_{ni}$	is the summation of the factored overall load effects for a given load combination condition.
α_i	is the load factor corresponding to a particular load, as defined by the National Building Code of Canada (NBCC).
S_{ni}	is a specified load component of the overall load effects; for example, dead load due to: weight of structure or live load due to wind.
i	represents various types of loads such as dead load, live load, wind load, etc.

Geotechnical resistance factors as provided by the National Building Code of Canada (NBCC, 2010) for shallow and deep foundations are shown Table 3.6 below. The critical design events and their corresponding load combination and load factors should be determined by the structural engineer.

Table 3.6 − Geotechnical Resistance Factor of Foundations

Foundation Type	Loading Condition	Geotechnical Resistance Factor
Shallow Foundation	Vertical bearing resistance from semi-empirical analysis	0.5
	Horizontal resistance against sliding (i) based on friction (c=0) (ii) based on cohesion/adhesion (tan Φ = 0)	 0.8 0.6
Deep Foundation	Resistance to axial load: (i) semi-empirical analysis; (ii) analysis using static loading test results; (iii) analysis using dynamic monitoring results; (iv) uplift resistance by semi-empirical analysis; and (v) uplift resistance using load test results.	 0.4 0.6 0.5 0.3 0.4
	Resistance to horizontal load	0.5

Serviceability Limit State – Limit States Design

Serviceability limit states (SLS) are primarily concerned with mechanisms that restrict or constrain the intended use, occupancy, or function of the structure. For foundation design, serviceability limit states are usually associated with:

- Excessive foundation movements; for example, settlement, differential settlement, heave, and so on; and
- Unacceptable foundation vibrations.

In general, the format criteria for serviceability limit states can be expressed as follows:
Serviceability Limit ≥ Effect of Service Loads.

Serviceability limit states are evaluated using unfactored geotechnical settlement properties, such as compressibility, Young's Modulus, and so on, to determine an SLS bearing pressure which, when applied to the foundation soil, will not exceed a specified serviceability criteria.

3.6.5.2 Conveyor Foundation Recommendations

Most of the conveyor is founded on grade, with the exception of the elevated sections located approximately at stations 0+190 to 0+255, stations 0+285 to 0+310 and stations 0+310 to 0+380. Structural loading has been provided by CDI for both the on-grade and elevated section, and are included in Appendix H.

Recommended Foundations for On Grade Conveyor Section

According to CDI design, the foundation supports for the on-grade conveyor sections are spaced 4 m apart, with the structural loads from the conveyor transmitted via two nodes, spaced 1.54 m apart per foundation support.

Based on the structural loading, current ground conditions and recommended fill compaction placement, the use of precast concrete sleepers is deemed acceptable for the on-grade sections of conveyor CV-002. Recommended dimensions and embedment depth for concrete sleepers are summarized in Table 3.7 below and as shown in Drawing VM00575.6.600-013.

Table 3.7 − Summary of Concrete Sleeper Dimensions

I) Concrete Sleeper Dimensions			
Length, L (m) (x-direction)	2.0	Width, B (m) (z-direction)	0.51
Thickness, T (m)	0.3	Embedment Depth, D (m)	0.2
II) Bearing Capacity and Settlement Analysis			
Concrete Sleeper, Nodes N3 & N5 (Refer to Ground Based Stringer Loadings, Appendix D)			
a) Bearing Capacity Check			
Ultimate Unfactored Bearing Pressure (kPa)	150	Geotechnical Resistance Factor	0.5
Factored Bearing Pressure (kPa)	75	Factored Structural Pressure (kPa) (Based on structural loading provided by CDI)	40
b) Settlement Check			
Unfactored Applied Load (kN)	32.0	Unfactored Applied Moment (kN.m)	-1.2
Settlement, w (mm)	1.2	Foundation Tilting, tgϕ (°)	-0.1
Concrete Sleeper, Nodes N4 & N6 (Refer to Ground Based Stringer Loadings, Appendix D)			
a) Bearing Capacity Check			
Ultimate Unfactored Bearing Pressure (kPa)	150	Geotechnical Resistance Factor	0.5
Factored Bearing Pressure (kPa)	75	Factored Structural Pressure (kPa) (Based on structural loading provided by CDI)	41.6
b) Settlement Check			
Unfactored Applied Load (kN) (Based on structural loading provided by CDI)	40.7	Unfactored Applied Moment (kN.m) (Based on structural loading provided by CDI)	-0.8
Settlement, w (mm)	1.6	Foundation Tilting, tgϕ (°)	-0.1

Settlement in the order of 1-2 mm was calculated for the assumed loading and subsurface conditions, with negligible differential settlement in the order of 1 mm. Given that the concrete sleepers are founded on surface and will not be heated during the winter months, frost heave might be the governing factor in terms of SLS conditions. As noted in section 3.6.5.2 above, it was determined that differential frost heave, up to 15 mm, might be expected where groundwater or seepage exists in the top 3 m in the ground. As such, for areas where groundwater is expected to be near surface, the settlement criterion for the concrete sleepers will be governed by the frost heave in the winter months followed by thaw settlement in summer.

In addition, it is important to recognize that differential settlement in the order of 13-15 mm could be expected between sections where frost heave is expected to occur, and those where heaving is expected to be negligible.

It is important to note that the calculated settlement and frost heave values do not account for construction defects or site conditions different from what was adopted for the design. It is prudent for the conveyor designer to assume that the differential settlements/movement could be in the order of 15 to 20 mm along the whole conveyor alignment.

If such settlement is deemed excessive by CDI, mitigation include the use of insulation to limit the frost penetration depth or the implementation of a seepage cut-off system to reduce or eliminate groundwater/seepage. An alternative foundation would be a shallow foundation founded below the frost depth.

Detailed site preparation recommendations are provided in the Foundation Design document of Appendix C.

Recommended Foundations for Elevated Conveyor Section

Shallow foundations, consisting of a combined footing founded below frost level, with structural columns extending to ground surface are considered appropriate for the support of the towers and abutments of the elevated sections.

Table 3.8 below summarizes the recommended dimensions and embedment depths of the shallow foundations for the abutments, the two-legged and the four-legged towers. Refer to Drawings VM00575.6.600-014 to 016 for the structural foundation details.

Bearing capacity was assessed assuming 30 degrees ground inclination, which is comparable to the observed slopes in the Kemess Lake Valley. All shallow foundations are founded below the frost level, defined at a depth 2 m, as a protection from frost action.

Table 3.8 − Summary of Recommended Foundation Dimensions, Expected Settlement, Bearing Capacity and Uplift Resistance for Elevated Conveyor Sections

I) Foundation Dimensions			
	Two-legged Tower	**Four-Legged Tower**	**Abutment**
Length, L (m) (z-direction)	6.1	6.1	3.5
Width, B (m) (x-direction)	2.0	3.1	2.0
Thickness, T (m)	1.0	1.0	0.5
Embedment Depth D (m)	3.0	3.0	3.0
II) Bearing Capacity and Settlement Analysis (Refer to Elevated Gallery Loadings, Appendix D)			
a) Settlement Check			
	Two-legged Tower	**Four-Legged Tower**	**Abutment**
Unfactored Applied Load (kN) (Based on structural loading provided by CDI)	1566.6	1988	740.1
Unfactored Applied Moment (kN.m) (Based on structural loading provided by CDI)	-500.6	-600.2	-244.0
Settlement, w (mm)	8.1	7.9	4.8
Foundation Tilting, tgϕ (°)	-1.1	-0.9	-2.7
b) Bearing Capacity Check			
	Two-legged Tower	**Four-Legged Tower**	**Abutment**
Ultimate Unfactored Bearing Pressure (kPa)	527	638	427
Geotechnical Resistance Factor	0.5		
Factored Bearing Pressure (kPa)	263	319	211
Factored Structural Pressure (kPa) (Based on structural loading provided by CDI)	131.5	121.5	142.3
C) Uplift Resistance			
	Two-legged Tower	**Four-Legged Tower**	**Abutment**
Ultimate Unfactored Uplift Resistance (kN) (Assuming no groundwater)	1093	1536	577
Geotechnical Resistance Factor	0.3		
Factored Uplift Resistance (kN)	328	461	173

Resistance to adfreezing stresses (frost jacking) on the concrete columns will be provided by the combined weight of the foundation, soil cover and by sustained compressive loads. For foundation design purposes, an unfactored adfreezing uplift pressure of 65 kPa applied over a depth of frost penetration of 2.0 m should be used. A load factor (α) of 1.2 should be adopted for frost jacking.

To determine the factored uplift resistance against frost jacking in terms of ULS, a resistance factor, Φ, of 0.8 should be applied to the unfactored shaft resistance values. In case of caissons subjected to live uplift loads as well as to frost jacking forces, the live uplift load need not be additive to the frost jacking forces.

Special care must be taken to protect the foundations against frost action during and after construction, before backfill is completed.

Detailed site preparation recommendations are provided in the Foundation Design document of Appendix C.

3.6.5.3 Foundation Recommendations for Transfer Tower Building

The transfer tower building, located at station 0+000, is the point where the ore and waste rock materials are transferred from conveyor CV-001 to conveyor CV-002 (refer to Drawings VM00575.6.600-003 and 011). The building has a setback distance of roughly 23.5 m from the face of the Triple Decline Portal tunnels. Its footprint is approximately 17 m x 20 m. The foundation slab, founded at approximate elevation of 1,375 m, is tiered with an approximate difference in height of 2.3 m between the two levels. The north wall of the building will be resting against native ground and is roughly 6.5 m high.

The use of a slab on grade as a building foundation for the transfer tower is acceptable. Recommendations will be provided for a flexible and rigid (mat) foundation slab. Recommendations for a flexible foundation slab include a modulus of subgrade reaction, while recommendations for a rigid foundation slab consist of ultimate unfactored bearing pressure and factored bearing pressure for ULS conditions and allowable bearing pressure for SLS conditions.

Modulus of Subgrade Reaction for a Flexible Foundation Slab

For a slab on grade assumed to behave in a non-rigid manner, a modulus of subgrade reaction, K_{s1}, of about 150 MPa/m is recommended for design. K_{s1} represents the modulus value for a 12-inch (0.3 m) diameter plate that must be adjusted for actual size of the foundation. It should be noted that the modulus of sub-grade reaction is not an intrinsic soil property, but is rather dependent upon the size and shape of the foundation, as well as the distribution of load throughout the foundation. Settlements values up to 50 mm can be expected in foundation slabs.

The K_{S1} recommended is a high value and is representative of very dense ground conditions. The recommendation is based on the assumption that bedrock will be encountered at an elevation of 1,375 m, based on nearby borehole BH15-13.

Unfactored and Allowable Bearing Capacities for Rigid Foundation Slab

Assuming that the slab on grade is sufficiently thick and rigid to cause the underlying soil mass to fail in general shear, an ultimate unfactored bearing pressure of 1,500 kPa for ULS conditions and an allowable bearing capacity of 500 kPa for SLS conditions would be applicable. The allowable bearing capacity provided for SLS conditions corresponds to an allowable settlement in the range of 50 to 75 mm.

A geotechnical resistance factor of 0.5 shall be used to determine the factored bearing pressure for ULS conditions.

Discussion

It should be noted that a slab on grade foundation for unheated structures will experience frost heave during winter months followed by thaw settlement in summer. Frost heave of up to 30 mm is estimated. If such movement is deemed unacceptable, insulation could be used to reduce the depth of frost penetration. An insulation layer installed below the foundation will reduce the seasonal frost penetration, resulting in reduction of the frost heave. An alternative foundation option would be a strip or square footing founded below the depth of frost penetration. Further recommendations could be provided in the detailed design phase or as requested.

The design elevation of the foundation slab, approximately 1,375 m, is between 15 to 20 m below the current ground surface. Foundation excavation must conform to all applicable occupational health and safety regulations in the Province of British Columbia and to any site specific regulations.

A discussion on the detailed site preparation recommendations are provided in the Foundation Design document of Appendix C.

Foundation Wall and Lateral Earth Pressure

As mentioned earlier, the north wall of the building will be below the finished ground surface. This wall should be designed to resist horizontal loads from the soils and any surcharge loading. Static “at-rest” triangular lateral earth pressure distribution should be used for structures restrained from lateral or rotational movement. The at-rest lateral soil force, F, on a unit width can be computed using the following equation:

$$F = \frac{1}{2} K_o \gamma H^2$$

Where:

k_o = coefficient of lateral earth pressure “at rest” condition

γ (kN/m^3) = unit weight of the soil (submerged unit weight below the water table)

H (m) = height of the structure below the ground surface (from the ground surface to the foundation base).

The pressure distribution described above does not include any surcharge loads, which should be considered to the lateral earth pressures, if any.

For compacted structural fill, defined as washed or crushed gravel, the at-rest earth pressure coefficients and other soil parameters corresponding to the slope of the backfill are shown in Table 3.9. The use of an alternative fill material is permissible upon approval by an Amec Foster Wheeler geotechnical engineer.

Table 3.9 − Recommended Soil parameters and Lateral Earth Pressure Coefficients

Fill Material Type	Unit Weight, γ (kN/m^3)	Friction Angle, Φ (°)	Coefficient of Lateral Earth Pressure at Rest, k_o	Coefficient of Lateral Active Earth Pressure, k_a	Coefficient of Lateral Passive Earth Pressure, k_p
Structural Fill (washed or crushed gravel)	19.0	35	0.43	0.27	3.70

Note: The above lateral earth pressure coefficients represent a horizontal ground surface at the top of the wall

A discussion on the detailed site preparation recommendations are provided in the Foundation Design document of Appendix C.

3.6.6 Culvert Design

The current layout of the surface infrastructure requires the construction of a culvert, shown in VM00575.6.600-003, over the conveyor to allow the safe crossing of 40 ton articulated trucks over a 12 m wide Main Access Road. The culvert dimensions are as follows:

- Approximately 70 m in length, between stations 0+005 to 0+075 culvert.
- Inside width of 3 m to accommodate the conveyor and access for personnel.
- A height clearance of 2.5 m.
- Up to 15 m of soil cover, measured from the crown of the culvert.

The design of the culvert crossing will be performed in general accordance with limit states design methodology; see Section 3.6.7 for the applicable geotechnical resistance factors based on NBCC (2010).

3.6.6.1 Foundation Recommendations

The use of a strip footing is deemed acceptable to provide a stable foundation against vertical and horizontal loading, with dimensions and embedment depth summarized in Table 3.10 below. The allowable bearing capacity for SLS design conditions are consistent with total and differential settlements of less than about 25 mm and 15 mm, respectively. It should be noted that the settlement calculations were evaluated assuming static loads only.

The soil bearing capacity was estimated using the methodology described in Section 10.2 of the 4th Edition of the CFEM and taking into consideration the inferred soil profile and groundwater conditions, and assuming that loads are vertical and applied concentrically with the foundations.

Table 3.10 − Summary of Concrete Strip Footing Dimensions, Expected Settlement and Bearing Capacity

I) Strip footing Dimensions			
Length, L (m)	70	Width, B (m)	1.5
Thickness, T (m)	0.5	Embedment Depth, D (m)	1.0
II) Bearing Capacity Check (ULS & SLS conditions)			
Ultimate Unfactored Bearing Pressure (kPa)	550	Geotechnical Resistance Factor	0.5
Factored Bearing Pressure (kPa)	275	Allowable bearing Pressure (SLS condition, expected settlement limited to 25 mm)	150
Note: Ultimate bearing capacity assumes a failure surface propagates into the inside of the culvert arch and does not account for support provided by the arching effect of the culvert, which tends to produce a conservative result.			

Lateral Earth Pressure

Lateral loads acting on the culvert, and ultimately the foundation, will be resisted by the sliding resistance between the foundation and subgrade, and the lateral earth pressure acting on the sides of the RC sleeves.

The sliding resistance between the foundation and the subgrade may be determined by multiplying the average pressure acting on the foundation base by the coefficient of friction between the concrete and the underlying soils consisting of 25 mm minus crushed gravel. A coefficient of friction of 0.55 between mass concrete and pit run gravel is recommended.

The lateral earth pressure, acting on the sides of the culvert by the fill material, may be assumed to be trapezoidal in shape and increase linearly with depth according to:

$p = K\sigma'$

Where:

p (kPa) = lateral earth pressure at depth z

K = lateral earth pressure coefficient

σ' (kPa) = effective stress at depth z, i.e. γ' times z

γ' (kN/m^3) = unit effective weight of the soil

z (m) = depth below the existing tailings road

The magnitude of lateral earth pressure mobilized is a function of the deformation that the soil experiences. If the soil undergoes negligible movement, the lateral earth pressure it generates will correspond to "at-rest" conditions with $k = k_o$. If the soil mass experiences sufficient deformation, the lateral earth pressure mobilized will correspond to either an active or passive state depending on the relative direction of movement, with $k=k_a$ or $k=k_p$, respectively. The adoption of either lateral earth pressure coefficient depends on the relative stiffness of the culvert material and on the acceptable allowable movements.

For compacted structural fill or waste rock, the at-rest earth pressure coefficients and other soil parameters corresponding to the slope of the backfill are shown in Table 3.11. The use of an alternative fill material is permissible upon approval by an Amec Foster Wheeler geotechnical engineer.

Table 3.11 − Recommended Soil parameters and Lateral Earth Pressure Coefficients

Fill Material Type	Total Unit Weight (kN/m^3)	Friction Angle (°)	Lateral Earth Pressure Coefficients		
			At Rest, K_o	Active, K_a	Passive, K_p
Structural Fill (Washed or Crushed Gravel)	20	35	0.43	0.27	3.7
Common Fill (Waste Rock)	19-20	33-37	0.45-0.4	0.29-0.25	3.4-4.0

Note: The above lateral earth pressure coefficients represent a horizontal ground surface at the top of the wall.

A discussion on the detailed site preparation recommendations is provided in the Foundation Design document of Appendix C.

3.6.7 Seismic Site Classification

The seismic response of the site was classified according to the National Building Code of Canada 2010 (NBCC), which categorizes the soil conditions into 6 types - Class ‘A’ to ‘F’. This classification is based on the average shear wave velocity, SPT ‘N’ values, or undrained shear strength over the top 30 m of the soil profile. Based on average standard penetration N values in the upper 30 m of the site obtained from boreholes along the conveyor alignment, the site is classified as class D.

The 5% damped spectral response acceleration values for the firm ground (Class C sites) are as follows: Sa(0.2)=0.095, Sa(0.5)=0.058, Sa(1.0)=0.035 and Sa(2.0)=0.023. These values must be multiplied by the factors in tables 4.1.8.4.B and 4.1.8.4.C to obtain values appropriate for a Class D site. The horizontal peak firm ground acceleration for Kemess mine is 0.052 g.

3.6.8 Recommendations for Cement Type and Buried Steel Structures

The sulphate concentrations from six samples tested were less than 0.02 percent which is considered to be “negligible” potential for sulphate attack on buried concrete. All concrete design and construction should be carried out in general accordance with the current CAN/CSA-A23.1 specifications. Air entrainment is recommended for all concrete exposed to freeze-thaw cycles to enhance durability.

For the chloride testing, the concentrations were less than 0.001 percent which is considered negligible. The pH of the soil tested ranged from 5.35 to 6.73 with an average of 6, indicating low corrosion potential. Based on the combined results of pH, sulphate and chloride, corrosion of the risk of corrosion to buried steel is low.

Refer to Appendix C for further information regarding the chemical testing program completed on selected grab sample.

4.0 PORTALS AND TUNNEL

4.1 General

The following sections describe the design and design recommendations for development of the South Portal, North Portal, Access Tunnel and the Triple Decline Portal. Each of these structures are shown on Drawing VM00575.6.600-002. Within each section, the analyses completed and the respective designs have been described. Given the similarity in the analyses completed for each of the portals, attempts have been made to reduce repetitive descriptions by making reference to the section where it was initially described in detail, rather than repeating the procedures.

4.2 South Portal

4.2.1 General

The South Portal forms the southern entrance to the Access Tunnel for the KUG deposit, with the centerline located at N 6321727 m E 635177 m. The portal dimension is 5.5 m high by 5.5 m wide, located at El. 1490 m (base elevation). Refer to Drawing VM00575.6.600-017 for the plan and profile of the portal footprint.

4.2.2 Geotechnical Site Conditions and Terrain Stability

The rock and soil characteristics at the South Portal are based mainly on data from boreholes BH15-07 (KH15-15) and BH15-07A. The soil and bedrock contact was interpreted from BH15-07 and BH15-07A, as well as from the depth of refusal recorded from the overburden auger drilling (OB15-XXs) completed within the portal footprint. Bedrock within the South Portal area consists of the Black Lake Formation, consisting primarily of granodiorite. The recorded rock quality designation (RQD) values within the granodiorite, range from very poor to good. However, field strength estimates of intact rock ranged between strong to very strong. The overburden consisted of compact silty sand to a shallow depth overlying weathered bedrock. Detailed borehole logs are included in the 2015 KUG SI report of Appendix A.

Based on review of LiDAR data and air photos, the terrain surrounding the South Portal is considered benign with respect to geohazards. Due to time restraints with respect to the terrain hazard portion of the field work, Amec Foster Wheeler considered it more important to focus on areas known to have potential geohazards issues; namely the North Portal and the Triple Decline Portal. These are discussed in subsequent sections of the report.

4.2.3 Kinematic Analysis

A kinematic analysis was completed for the South Portal rock cut slope and the approach slopes, to determine the potential failure modes present within the rock mass. Initially the analysis for the portal cut slope was completed using the original design configuration (default) with respect to face dip and dip direction (see Table 4.1 below). For completion, a sensitivity analysis was carried out to assess if there was a marked reduction in the probability of failure, by rotating the cut slope face ±10° in either direction from the original orientation. Table 4.1 below summarizes the face orientations analysed.

Table 4.1 − Summary of Kinematic Analysis Parameters for Portal Orientations

Area	Dip (°)	Dip Direction (°)	Friction Angle (°)	Formations Analyzed
South Portal	75	204 (Default)	30	Black Lake
South Portal	75	194	30	Black Lake
South Portal	75	214	30	Black Lake

The approach slopes (East and West approach slopes) were designed approximately perpendicular to the South Portal slope, and consist of a series of 5 m high benches. The overall slope angle ranges between 41 and 43˚, with much steeper individual bench cuts of 75˚. Kinematic analyses were completed for the approach slopes with respect to the overall slope angle, as well as for the individual steeper bench cuts. Assumptions were made for the friction angle of the Black Lake Formation based on published literature and experience of the rock types in the area. Table 4.2 below summarizes the approach slope orientations analysed.

Table 4.2 − Summary of Kinematic Analysis Parameters for Portal Approach Slopes

Area	Approach Slope	Dip (°)	Dip Direction (°)	Friction Angle (°)	Formations Analyzed
South Portal	West (overall)	41	115	30	Black Lake
South Portal	East (overall)	43	295	30	Black Lake
South Portal	West (individual)	75	115	30	Black Lake
South Portal	East (individual)	75	295	30	Black Lake

The computer software DIPS 6.0 by Rocscience Inc. was utilized to complete the kinematic analysis. The structural data obtained from the oriented core drilling was plotted onto stereonets, in order to visually assess the joint sets present within the rock mass. Where clusters of pole vectors form on the stereonet, these represent the most common joint sets present. As expected, randomly distributed joints also occur, although these were typically not considered in the analysis. Stereoplots representing the joint set distribution within the rock mass at each of the portals are shown on Drawing VM00575.6.600-018. Refer to Appendix D for the stereonet plots of the South Portal.

The kinematic analysis considers the following failure modes; planar sliding, wedge sliding, flexural toppling and direct toppling. Resulting number of poles intersecting planes are tabulated and the probability of each failure mode is determined. The results of the kinematic analysis are summarized in Table D1 – D25 in Appendix D.

The kinematic analysis completed for the three face orientations (default and ±10° in either direction) resulted in the following;

- **Planar Sliding**: the highest potentials of failure at the South Portal are for face orientations 75/204° and 75/194°.
- **Wedge Sliding**: the highest potential of wedge sliding at the South Portal is 75/214°.
- **Flexural Toppling**: all orientations at the South Portal have equal potential for flexural toppling.
- **Direct Toppling**: the highest potential of direct toppling at the South Portal are with orientations 75/204° and 75/214°.

The results of the kinematic analysis indicated that the face orientation towards 194˚ is preferred, as the probability of any of the failure modes is lowest for this orientation. However, due to logistical complications and construction issues, the preferred face orientation is not considered practical. Therefore the face orientation towards 204˚ was selected, with the understanding that reinforcement of the cut slopes may be required with respect to small-scale, localized failures and ravelling.

The results of the kinematic analysis for a face orientation of 75˚ towards 204˚ are summarized in Table 4.3 below.

Table 4.3 – Summary of Failure Modes Probabilities for South Portal Orientation 75/204

Failure Mode	Type	Critical	Total	Probability of Failure (%)
Planar Sliding	All	2	13	15.4
Wedge Sliding	Both Planes	13	77	16.9
Wedge Sliding	One Plane	15	77	19.5
Flexural Toppling	All	0	13	0
Direct Toppling	Direction Toppling (Intersection)	6	77	7.8

Failure Mode	Type	Critical	Total	Probability of Failure (%)
Direct Toppling	Oblique Toppling (Intersection)	1	77	1.3
Direct Toppling	Base Plane (All)	2	13	15.4

For the approach slopes, the potential for planar and wedge sliding and flexural toppling is higher along the east slopes than the west slopes. Whereas the potential for direct toppling failure is higher along the west slopes than the east slopes.

4.2.4 Soil Stability Analysis

4.2.4.1 General

A slope stability analysis was completed for the soil cut slopes above the South Portal and approach slopes based on cross section profiles intersecting the portal face orientations. The analysis was completed to determine the potential risks of slope failures at the portal. The slope cuts above the South Portal entrance and the approach slopes were modelled to determine their respective factors of safety (FOS). The stability analysis was completed based on the proposed design profiles, using soil strength parameters and water table data obtained from the field assessment. Additional detail of the analyses completed and sections should be referred to in Appendix E.

4.2.4.2 Methodology

The design parameters were selected from data obtained from the 2015 site investigation program (refer to the 2015 KUG SI Report in Appendix A). The auger borehole and test pit logs from the South Portal were reviewed to determine the properties of the soil material above the bedrock at this location. The design parameters are outlined in Table 4.4 below.

Table 4.4 – Soil Parameters for Slope/W Analysis

Material Type	Unit Weight (kN/m^3)	Cohesion (kPa)	Phi (°)*
Sand and Gravel	20	0	35
Bedrock (Assumed Impenetrable)	-	-	-

Note: Soil parameters based on interpretation of Table 4-13 and Figure 4-12 of EL-6800 Report.

Two-dimensional limit equilibrium analyses were carried out to evaluate the slope stability of the portals using the computer program SLOPE/W (2012), the Morgenstern-Price (1965) method of solution was applied. The entry and exit method was used to determine the most critical failure plane yielding the lowest FOS with all other conditions being equal. The analysis was completed for a section drawn perpendicular and through the centre line of the Access Tunnel. Also a section was cut for each of the approach slopes. Refer to Drawing VM00575.6.600-017 for locations of the cross sections.

Due to the overall topography at this location, the proposed soil cut slopes will be developed at 2H:1V (27˚). The underlying rock cut slopes will be steeper, and are discussed further in subsequent sections. The soil cut slopes along the approach slopes are designed to approximately 60˚.

4.2.4.3 Analysis

The stability analysis was conducted to determine the lowest factory of safety for shallow and deep-seated instability. Since the proposed cut slope was designed to a shallow angle (27˚), the lowest global factor of safety for shallow instability was typically between 1.4 and 1.5 for the South Portal and the approach slopes. The factor of safety for deep-seated instability was greater than 1.5. Refer to Appendix E for the figures of the Slope/W modelling results and Table 4.5 below for the results of the Slope/W analysis of the South Portal.

Table 4.5 – Slope/W Analysis Results for South Portal Factor of Safety

Area	Shallow Failure (Lowest)	Deep Seated
Portal Slope Face	1.49	>1.5
East Approach Slope	1.40	-
West Approach Slope	1.41	-

Based on the factors of safety achieved for the South Portal and the approach slopes, it is unlikely that reinforcement of the slopes will be required. However, it is recommended that the slopes are monitored during construction, to ensure conditions other than those anticipated based on the site investigation program, are not encountered.

4.2.5 Rock Stability Analysis

4.2.5.1 General

In order to determine if the proposed slope profile meets the required minimum factor of safety for long term stability under static conditions (FoS of 1.5), a two-dimensional limit equilibrium stability analysis was completed for the proposed rock cut slope at the South Portal.

The proposed cut slope is approximately 17 m in height, and sloped at 75˚ from the horizontal. A 10 m wide bench is proposed above the portal rock cut, for construction of a catchment and water runoff ditch, as well as to allow access for maintenance equipment. The proposed slope above the bench consists of approximately 7 m of rock overlain by 7 m of soil. The slopes above the bench were not analyzed with respect to rock slope stability, as these have been designed at a much shallower angle, and are not considered to be a cause for concern. Refer to Drawing VM00575.6.600-017 for a profile of the proposed slope geometry at the South Portal.

4.2.5.2 Methodology

Two-dimensional limit equilibrium analyses were completed using the Rocscience SLIDE program (Version 6.0). A section was created along the proposed Access Tunnel, and the factor of safety was calculated using the GLE/Morgenstern-Price slice method. The strength type was assumed to be the Generalized Hoek-Brown, as the overall rock mass is considered to be of moderate strength based on the drilling data. The failure surface within the rock mass was assumed to be non-circular in nature.

The strength parameters used in the analyses were developed based on data obtained from borehole BH15-07 (refer to the 2015 KUG SI report in Appendix A). Data pertaining specifically to the depth at which the portal will be developed, was used to assign the following strength parameters (from the Rocscience RocLab and SLIDE programs):

- Geological Strength Index (GSI) = 35
- Material Constant (m_i) = 29 for granodiorite
- Disturbance Factor (D) = 0.8 based on assumption of moderately controlled blasting will be conducted.

Table 4.6 − Design parameters used to conduct the stability analysis from RocLab

Portal	Average Unit Weight (kN/m3)	Average UCS (MPa)	m_b*	s*	a*
South	27.1	82.6	0.61	5.28	0.52

*Where m_b is a reduced value of mi; "s" is given by s = exp ((GSI-100)/9-3D); "a" is given by
$a = 0.5 + 1/6\ (e^{-GSI/15} - e^{-20/3})$.

4.2.5.3 Analysis

Table 4.7 below summarizes the calculated global minimum and deep-seated factors of safety, under static, dry and partially saturated conditions, as well as for pseudo-static, partially saturated conditions. Reference should be made to Appendix F which further discusses some of the design parameters used in the analyses.

Table 4.7 − Calculated factors of safety under static and pseudo-static conditions

Portal	FoS Static Dry		FoS Static Part. Saturated		FoS Pseudo-Static Part. Saturated	
	Shallow	Deep Seated	Shallow	Deep Seated	Shallow	Deep Seated
South Portal	2.2	N/A	1.4	2.0	1.3	1.4

*Note that for some of the analyses conducted, the potential for deep seated sliding is considered negligible, resulting in no FoS value being calculated (assigned N/A).

For dry, static conditions, the recommended minimum factor of safety of 1.5 was achieved. For shallow instability, under partially saturated, static conditions, the calculated factor of safety is 1.4, and 2.0 for deep-seated instability. This suggests that drained conditions are required to meet the minimum required factor of safety for long-term stability. The minimum required factor of safety (1.1) under pseudo-static conditions was achieved under both dry and partially saturated conditions.

In general the recommended slope designs at the South Portal appear to meet the required factor of safety under dry conditions. However, it must be noted that based on the kinematic analysis completed (see Section 4.2.3 above and Appendix D), there may be a potential for localized toppling failure as a result of the discontinuity distribution. This type of failure can typically be managed through bolting and mesh. This will need to be assessed by an experienced engineer during construction.

As well as review of the requirement for reinforcement during construction of the South Portal, it is recommended that scaling of loose blocks is completed for improved safety under rock slopes. Additionally, it is recommended that a catchment ditch is constructed along the toe of the cut slope, to divert water away from the toe, and to catch any material ravelling from the cut slope. Details of the recommended ditch dimensions are included in Appendix F. To reduce ravelling of small rock fragments, the cut surface directly above the tunnel entrance could be shotcreted.

4.2.6 Support Assessment

4.2.6.1 Soil Nails

Due to the relatively gently sloping terrain at the South Portal, the soil cut slopes above the portal will likely not require reinforcement. The slopes are recommended to be developed at approximately 2H to 1V (27˚). To account for any surface water runoff, and to prevent development of pore pressure behind the underlying rock cut slopes, a water runoff ditch is recommended along the toe of the soil cut slopes. This ditch should be gently inclined (about 2˚) to divert water away from the face.

4.2.6.2 Crown Pillar

In order to determine if the rock mass will require ground support above the tunnel opening, calculations with respect to the geometry of the opening, as well as the rock mass parameters was completed for the South Portal.

In accordance with Carter and Miller (1995), the acceptable risk exposure guidelines developed, indicate that for long term (50 to 100 years), permanent structures such as the portal crown over the tunnel, require a minimum factor of safety of 2.0. Accordingly, analysis for the anticipated factor of safety was completed for the South Portal. Based on drilling data from BH15-07, the following parameters and assumptions were made to calculate the factors of safety;

Table 4.8 − Factor of safety calculation parameters

Portal	Average RQD (%)	*Average J_r	*Average J_a	*Average J_n
South	25	2.0	3.0	12

*The values are obtained from data within the 2015 KUG SI Report (Appendix A)

- Based on the anticipated hydrogeological conditions on site, the calculations have assumed dry conditions i.e. the rock mass is fully drained, and therefore no flowing water (J_w = 1.0), and partially dewatered conditions indicating a moderate flow of water through the rock mass (J_w = 0.66);
- The average, minimum and maximum Q' values obtained from the results of the geotechnical drilling program were used to derive the factor of safety in order to identify the "worst case scenario";

Table 4.9 below summarizes the compressive strength, the crown pillar span and the crown pillar thickness of the tunnel entrance. Refer to Appendix A for UCS and Q' values, and Appendix G for details of individual crown-pillar calculations. The average Q' value was obtained from Q' values obtained within the span of the crown-pillar.

Table 4.9 − A summary of the parameters used to calculate the factors of safety

Portal	Section	UCS Range (MPa)	Crown Pillar Span (m)	Average Q' Portals	Crown Pillar Thickness (m)*
South		54.7 - 97.2	5.5	0.9	10.1

*The crown pillar thickness is taken at the tunnel entrance rather than an average thickness over a 10 m span (the width of the bench above the cut slope). As such, this value is more conservative than what may actually be present based on the declining nature of the tunnel.

Table 4.10 and Table 4.11 below summarize the calculated factors of safety under dry and partially saturated conditions. Table 4.10 presents the results without support, Table 4.11 presents results with support. Refer to Appendix G for FoS calculation worksheets.

Table 4.10 − Summary of the calculated factors of safety under unsupported conditions

Portal	Joint Water (J_w) Conditions	Average FoS	Max. FoS	Min. FoS
South	0.66	1.1	1.4	0.7
	1.0	1.2	1.5	0.8

To increase the calculated factors of safety, the application of reinforcement such as steel sets and shotcrete was considered. To gauge the possible variations in the safety factor, consideration was given to several thicknesses of shotcrete. The results are presented in Table 4.11 below.

Table 4.11 − Summary of the calculated factors of safety (FoS) under reinforced conditions

Portal	Average FoS using 100 mm shotcrete	Average FoS using 150 mm shotcrete	Average FoS using 200 mm shotcrete
South	2.4	2.4	2.5

Discussion of Results

When unsupported, none of the calculated factors of safety for dry or moderate inflow conditions meet the required factor of safety of 2.0, for long-term, permanent stability based on the Acceptable Risk Exposure Guidelines – Comparative Significance of Crown Pillar Failure (modified from Carter & Miller, 1995). As a result, addition of ground support such as steel sets and shotcrete is required to achieve the minimum required factor of safety.

Based on the calculations completed, at least 100 mm of shotcrete together with steel sets will be required to achieve the minimum factor of safety of 2.0. The calculated factors of safety are based on the use of 8 inch I-beam steel sets (W8 x 31) in accordance with ASTM A6.

Recommendations

Based on the results of the calculated factors of safety discussed in the sections above, the following reinforcement support will be required to achieve the minimum required factors of safety:

- Steel sets of the W8 x 31 type should be placed every two metres for the first 10 - 12 m from the portal entrance (a length approximately equal to twice the tunnel diameter);
- On the face and 20 m into the portal, 3 m rebar on a 1.2 m x 1.2 m spacing;
- #4 Galvanized Screen;
- ¼ inch Galvanized Steel Strapping placed around the brow;
- 4 to 6 inch (100 – 150 mm) shotcrete on first 20 m of the drive; and
- 5 – 6 m single Garford cable bolts on a 2 m by 2 m spacing in the back for the first 10 m (a length approximately equal to the tunnel diameter).

4.2.7 Inspection and Monitoring

It is recommended that an experienced engineer is present during construction of the South Portal and the approach slopes, in order to monitor any changes in site conditions that may affect the designs completed for this report. In the event significant changes are encountered, updates to the designs may be required.

The bedrock/soil interface should be verified during excavation, to confirm the interpreted surface from the 2015 KUG SI program. In addition, any deviations in rock and soil conditions during excavation in comparison to the drilling program. Also, rock and soil surfaces should be inspected during construction for stability or changing water conditions, which may arise after the slopes are exposed.

Monitoring during mining operations is recommended for slopes without support, to identify hazard of ravelling or other potential instabilities on the slopes. If support is deemed necessary, the frequency of monitoring could be reduced by adding support to the portal structures.

4.3 North Portal

4.3.1 General

The North Portal forms the northern entrance to the Access Tunnel for the KUG deposit, with the centerline located at N 6322516 m E 635532 m. The portal dimension is 5.5 m high by 5.5 m wide, located at El. 1409 m (base elevation). Refer to Drawing VM00575.6.600-019 for the plan and profile of the portal footprint.

4.3.2 Geotechnical Site Conditions and Terrain Stability

The rock and soil characteristics of the North Portal are based mainly on data from borehole BH15-06 (KH15-20). The soil and bedrock interface was interpreted from BH15-06 and depth of refusal from the overburden auger drilling (OB15-XXs) completed within the portal footprint. Bedrock in the North Portal area consist of the Astika Formation with mainly basalt overlying layers of quartz carbonate and greywacke. The recorded RQD values within the basalt range from very poor to excellent. The overburden consisted of sand and gravel fill to shallow depth overlying weathered bedrock. Detailed borehole logs are included in the 2015 KUG SI report of Appendix A.

The terrain hazard assessment conducted at the North Portal, identified several outcrops near the portal footprint, consisting of mainly basalts with iron oxide staining. Minor surface soil creep was noted along the slope face near the portal, but no major instability was identified during the assessment.

4.3.3 Kinematic Analysis

Similarly to the South Portal, a kinematic analysis was conducted at the North Portal and the approach slopes, to identify the potential failure modes present along the exposed rock cuts. Initially the analysis for the portal cut slope was completed using the original design configuration (default) with respect to face dip and dip direction. For completion, a sensitivity analysis was carried out to assess if there was a marked reduction in the probability of failure, by rotating the cut slope face ±10° in either direction from the original orientation. Table 4.12 below summarizes the design parameters used to complete the analysis for the North Portal face orientations.

Table 4.12 − Summary of Kinematic Analysis Parameters for Portal Orientations

Area	Dip (°)	Dip Direction (°)	Friction Angle (°)	Formations Analyzed
North Portal	75	26 (Default)	30	Asitka
North Portal	75	16	30	Asitka
North Portal	75	36	30	Asitka

The approach slopes (East and West approach slopes) were designed approximately perpendicular to the North Portal slope, and consist of a series of 5 m high benches. The overall slope angle ranges between 41 and 50˚, with much steeper individual bench cuts of 75˚. Kinematic analyses were completed for the approach slopes with respect to the overall slope angle, as well as for the individual steeper bench cuts. Assumptions were made for the friction angle of the Astika Formation based on published literature and experience of the rock types in the area. Table 4.13 below summarizes the design parameters used to complete the analysis for the North Portal approach slopes.

Table 4.13 − Summary of Kinematic Analysis Parameters for Portal Approach Slopes

Area	Approach Slope	Dip (°)	Dip Direction (°)	Friction Angle (°)	Formations Analyzed
North Portal	West (overall)	50	113	30	Asitka
North Portal	East (overall)	41	294	30	Asitka
North Portal	West (individual)	75	113	30	Asitka
North Portal	East (individual)	75	294	30	Asitka

Based on the structural data from borehole BH15-06, the DIPS program was used to analyze the potential failure modes for each face orientation. As expected randomly distributed joints also occur, although these were typically not considered in the analysis. Stereoplots representing the joint set distribution within the rock mass at each of the portals are shown on Drawing VM00575.6.600-018. Refer to Appendix D to view the individual stereonet plots.

The kinematic analysis considers the following failure modes; planar sliding, wedge sliding, flexural toppling and direct toppling. Resulting number of poles intersecting planes are tabulated and the probability of each failure mode is determined. Refer to Tables D1 to D25 in Appendix D.

The kinematic analyses completed for the three face orientations (default and 10˚ rotated slopes) resulted in the following conclusions:

- **Planar Sliding**: all three orientations for the North Portal had equal potential of failure.
- **Wedge Sliding**: all three orientations for the North Portal had equal potential of failure.
- **Flexural Toppling**: all three orientations for the North Portal had equal potential of failure.
- **Direct Toppling**: all three orientations for the North Portal had equal potential of failure.

For the portal face, the probability of failure was similar for all three face orientations. Therefore, the default orientation (75/026˚) was selected for the North Portal design. Table 4.14 below summarizes the potential failure probabilities for the selected design face orientation.

Table 4.14 – Summary of Failure Modes Probabilities for North Portal Orientation 75/26°

Failure Mode	Type	Critical	Total	Probability of Failure (%)
Planar Sliding	All	0	4	0
Wedge Sliding	Both Planes	0	6	0
Wedge Sliding	One Plane	0	6	0
Flexural Toppling	All	0	4	0
Direct Toppling	Direction Toppling (Intersection)	0	6	0
Direct Toppling	Oblique Toppling (Intersection)	0	6	0
Direct Toppling	Base Plane (All)	2	4	50

Along the approach slopes, the potential for planar and wedge sliding and flexural toppling is similar for both the west and east slopes. The potential for direct toppling is highest for the west slopes.

4.3.4 Soil Stability Analysis

4.3.4.1 General

Similarly to the South Portal, slope stability analyses were completed for the soil slopes above the North Portal, based on cross section profiles intersecting the portal face orientations (refer to Drawing VM00575.6.600-019). For further detail on the soil stability analysis, reference should be made to Section 4.2.4 of this report. Additional detail of the analyses completed and sections should be referred to in Appendix E.

4.3.4.2 Methodology

The parameters applied to the analysis were based on data from borehole BH15-06 and from the auger borehole and test pit logs from the 2015 site investigation program (refer to the 2015 KUG SI Report in Appendix A).

The North Portal had one section cut at the centerline of the portal face (parallel to the Access Tunnel), as well as a section for each of the west slope and east slope of the approach slopes. Refer to Drawing VM00575.6.600-019 for locations of the cross sections and profiles of the proposed cuts.

Due to the steeply sloping terrain at the North Portal, the proposed soil cut slopes have been designed at 60˚ from the horizontal, in order to keep the amount of material having to be excavated to a minimum. The rock slopes underlying the soils will be cut at a steeper angle, and are discussed in subsequent sections.

Given the steep nature of the proposed soil cuts, reinforcement is required to achieve the minimum recommended factor of safety. Soil slope reinforcement designs are discussed in Section 4.3.6 below.

The modelling set up for the North Portal approach slopes is similar to those of the South Portal, with a series of 5 m high benches cut to 60° from the horizontal within the soil, and 75˚ from the horizontal within the rock cuts (refer to Drawing VM00575.6.600-019).

4.3.4.3 Analysis

The stability analysis was conducted to determine the lowest factory of safety for deep-seated failures at the North Portal faces, by locating the deepest failure surface at FOS = 1.00. Lowest FOS overall was also reviewed in the Slope/W analysis and it was mainly shallow localized failures near the soil slope surface. By designing for deep-seated failure with FOS = 1.00, the design would factor for shallow surface failures on the soil slope. For the approach slopes, only the lowest FOS was considered as the soil slope heights were smaller and unlikely to experience deep seated failures. Refer to Appendix E for the figures of the Slope/W modelling results and Table 4.15 for the results of the Slope/W analysis of the North Portal.

Table 4.15 – Slope/W Analysis Results for North Portal Factor of Safety

Area	Shallow Failure (Lowest)	Deep Seated
Portal Slope Face	0.57	0.97
East Approach Slope	0.56	-
West Approach Slope	1.44	-

The North Portal face and east approach slopes will require support and reinforcements to improve the FOS and minimize the likelihood of rockfalls or sliding failures at the location. Refer to Section 4.3.6 for discussion on the slope reinforcements for the North Portal.

4.3.5 Rock Stability Analysis

4.3.5.1 General

Similarly to the South Portal, a two-dimensional limit equilibrium stability analysis was completed for the proposed rock cut slope at the North Portal.

The proposed cut slope is approximately 17 m in height, and sloped at 75˚ from the horizontal. A 10 m wide bench is proposed above the rock cut. The proposed slope above the bench consists of approximately 7 m of rock overlain by 10 m of soil. The slopes above the bench were not analyzed with respect to rock slope stability, as these have been designed at a much shallower angle. A separate analysis has been completed for the soil slope with respect to reinforcement requirements (Section 4.3.6.1 below). Refer to Drawing VM00575.6.600-019 for a profile of the proposed slope geometry at the North Portal.

4.3.5.2 Methodology

The two-dimensional limit equilibrium analyses completed for the North Portal used similar parameters to the South Portal; the GLE/Morgenstern-Price slice method, and the Generalized Hoek-Brown strength type. The strength parameters were developed based on data obtained from borehole BH15-06 (refer to the 2015 KUG SI report in Appendix A). Since the North Portal is to be developed in basalt, the following parameters were derived using the Rocscience RocLab and SLIDE programs:

- Geological Strength Index (GSI) = 35
- Material Constant (m_i) = 25 for basalt
- Disturbance Factor (D) = 0.8 based on assumption of moderately controlled blasting will be conducted.

Table 4.16 − Design parameters used to conduct the stability analysis from RocLab

Portal	Average Unit Weight (kN/m3)	Average UCS (MPa)	m_b*	s*	a*
North	28.4	56.6	0.52	5.28	0.52

*Where m_b is a reduced value of mi; “s” is given by s = exp ((GSI-100)/9-3D); “a” is given by a = 0.5 + 1/6 ($e^{-GSI/15}$ - $e^{-20/3}$).

4.3.5.3 Analysis

Table 4.17 below summarizes the calculated global minimum and deep-seated factors of safety, under static, dry and partially saturated conditions, as well as for pseudo-static, partially saturated conditions.

Table 4.17 − Calculated factors of safety under static and pseudo-static conditions

Portal	Section ID	FoS Static Dry		FoS Static Part. Saturated		FoS Pseudo-Static Part. Saturated	
		Shallow	Deep Seated	Shallow	Deep Seated	Shallow	Deep Seated
North Portal		1.8	N/A	1.2	1.5	1.1	1.4

*Note that for some of the analyses conducted, the potential for deep seated sliding is considered negligible, resulting in no FoS value being calculated.

For dry, static conditions, the recommended minimum factor of safety of 1.5 was achieved with respect to shallow and deep-seated instability. Under partially saturated, static conditions, the calculated factor of safety is 1.2, and 1.5 for shallow and deep-seated instability respectively. This suggests that drained conditions are required to meet the minimum required factor of safety for long-term stability. The minimum required factor of safety (1.1) under pseudo-static conditions was achieved under both dry and partially saturated conditions. In general the recommended slope designs at the North Portal appear to meet the required factor of safety under dry conditions. However, it must be noted that based on the kinematic analysis completed (see Section 4.3.3 above and Appendix D), there may be a potential for localized failure as a result of the discontinuity distribution. These smaller-scale failures can typically be managed through bolting and mesh. This will need to be assessed by an experienced engineer during construction.

As well as review of the requirement for reinforcement during construction of the South Portal, it is recommended that scaling of loose blocks is completed for improved safety under rock slopes. Additionally, it is recommended that a catchment ditch is constructed along the toe of the cut slope, to divert water away from the toe, and to catch any material revelling from the cut slope. Details of the recommended ditch dimensions are included in Appendix F.

4.3.6 Support Assessment

4.3.6.1 Soil Nails

To reduce the amount of material having to be excavated in preparation for the portal development, the slopes above the North Portal entrance will be developed at fairly steep cut slopes of approximately 60˚ from horizontal. Based on the two-dimensional limit equilibrium stability analysis discussed in the section above, the minimum factor of safety will not be achieved if the slopes remain unsupported. Soil nails and shotcrete surfacing is recommend to support the soil cut slopes.

In order to determine the size and installation requirements of the proposed soil nails, the U.S. Department of Transportation Federal Highway Administration (FHWA) Geotechnical Engineering

Circular No. 7. Soil Nail Walls document was consulted for guidance. This document is commonly used as a standard practice for soil nail designs.

The proposed soil cut slope at the North Portal is about 10 m vertically. The natural slope above the cut is moderately steep at approximately 30˚;

Soil Properties

The soils across the portal typically comprise of granular, cohesionless sands and gravels with varying degree of fines. Coarse gravel and cobbles were also observed in the boreholes and test pits completed across. Based on the SPT's collected from the drilling program, the angle of friction of these soils is estimated at about 35˚. The soil unit weight (ɣ) is estimated at 20 kN/m^{3}. The 2015 KUG SI Report in Appendix A should be referred to for additional detail on the soil properties. There may be unforeseen variations within the overburden that will need to be considered during the nail installation stage.

Soil Nail Parameters

Based on the calculation checks completed for a minimum factor of safety of 1.5 under static conditions (refer to Appendix G), the following soil nail lengths are recommended to support the soil mass above the potential deep seated failure surface (refer to Drawing VM00575.6.600-020):

Table 4.18 − Summary of proposed nail length at the North Portal

	Installation: Soil nails inclined at 15˚ from horizontal, spacing at 1.5 x 1.5 m. Horizontal drains installed at similar spacing in centre of nail pattern.
	Soil Nail Length (m)
Row No.	**North Portal**
1	10-15
2	10-15
3	10-15
4	10
5	10
6	<10
7	<10
8	<10

These soil nails lengths are based on the current overburden/ bedrock contact model. In the event the depth of this contact is different from the existing model, the soil nail lengths may need to be adjusted.

Discussion of Results

The recommended global factor of safety of 1.5 for long term stability under static conditions, is achieved based on the verification calculations discussed in Appendix G. However, it must be noted that variations within the ground conditions may warrant further analysis or adjustment to the design recommendations during the construction stage.

When considering the strength of the soil nails only, the capacity of the #14 Grade 75 soil nails at 60% yield load, is adequate to support the anticipated mass above the a potential deep-seated failure surface. However, should significant variations in overburden thickness be observed during construction, it is recommended the existing geological model be updated accordingly, with subsequent updates to determine the required nail capacity requirements.

The value of the ultimate bond strength appears to vary depending on the type of material as well as on the construction/ installation method. In this case, cohesionless sands and gravels are assumed to make up most of the soils into which soil nails will be installed. Additionally, given the potential presence of coarse gravel and cobbles, the nails will likely be installed through driven casing. The calculated nail lengths beyond the proposed failure surface, depend on the ultimate bond strength value assigned to the soil. If during construction this value is considered to be different from the value assigned 190 kPa (from Table 3.1 of the FWHA manual), these lengths will need to be recalculated. The capacity can also be verified through pull tests.

In areas where the overburden and rock contact is anticipated to be encountered along a proposed nail length, it is recommended that the quality of the rock is assessed in order to determine the required nail installation depth. Where this contact is encountered, it is recommended that the nail is installed past the area of poor quality rock. This will need to be assessed during the installation process. It is not recommended that the nail is installed well into strong competent bedrock.

Additional to updates to design parameters during the installation process, random pull tests of the soil nails should be completed, to ensure ongoing satisfactory compliance in accordance with design specifications.

Other Design Considerations

Water Management

Although the test pits conducted were primarily dry at the time of the investigation, designs should consider diversion of water from the slopes and the soil. Slotted, horizontal PVC drains are recommended within the gaps of the soil nails in order to divert water away from the slope and prevent build-up of pore pressure (see Drawing VM00575.6.600-020). To prevent fines blocking the drain pipe, a geosoc should be used. Additionally, end caps should be used to prevent blockage from the exposed end of the pipe.

It is recommended that a ditch to divert the water away from the rock face below is constructed along the toe of the soil slope. The ditch should be angled gently (say 2˚) to divert water in either direction, away from the cut slope face. Additionally, to manage surface water runoff, a ditch could be constructed along the crest of the soil cut slope. The ditch should be concrete lined to prevent infiltration of water into the soil slope. This ditch could be constructed at the first excavation lift.

Corrosion Protection

Double corrosion protection (DCP) may be used to provide additional protection to the nails. DCP consists of a protective sheathing made from corrugated synthetic material such as High Density Polyethylene (HDPE) or PVC tube surrounding the nail bar. The internal annulus between the protective sheathing and the nail bar is prefilled with grout. Further information on the details of the DCP nails can be obtained from the Dywidag Soil Nail Manual.

Shotcrete

Temporary shotcrete facing will be required during construction of the soil nail wall. The initial facing is typically applied with the purpose of supporting the exposed soil slope during the nail installation and to protect against erosion and sloughing. Upon completion of the nail installation, a permanent facing is applied to provide connection among the soil nails, and to provide a more resistant erosion protection. The total shotcrete thickness should be in the range of 150 to 300 mm, with welded wire mesh reinforcement. The nail heads which are typically welded to a bearing pate should be fully encapsulated in the permanent shotcrete facing.

Soil Nail Installation Recommendations

Analyses and review of published standard practice manuals, have been completed to provide recommendations with respect to reinforcement requirements for the soil cut slopes at the North Portal. The following design criterion are recommended, and are outlined on Drawing VM00575.6.600-020:

- Install Grade 75 (#14) threaded soil nails and hardware (Dywidag Threadbar – ASTM A615);
- Soil nails should be installed in a square pattern, with a regular spacing of 1.5 m;
- The nails should be installed at an angle 15˚ below horizontal;
- The nail lengths should be in accordance with Table 4.18, ensuring embedment at least 4 m beyond the proposed failure surface, unless competent bedrock is encountered;
- Slotted horizontal drain pipes should be installed at an angle between 5 - 10˚ above horizontal to permit drainage of water. The pipes should include an end cap and a geosoc to prevent blocking by fine grained soils;
- Complete pull testing on select nails;
- For protection, the surface should be coated in 150 to 300 mm of shotcrete with wire mesh reinforcement;
- A catchment ditch should be constructed along the toe of the soil slopes to divert water from the horizontal drains away from the crest of the slope to avoid erosion from water as well as pore pressure building up behind the underlying rock cut slope; and

- A surface water run-off ditch should be constructed along the crest of each slope, to divert water away from the surface.

It must be noted that these guidelines and recommendations are based on our current understanding of the ground conditions. In the event variations in ground conditions are observed, the designs may need to be altered accordingly. Therefore it is recommended that an experienced engineer is present during the installation of the soil nails, in order to respond to any immediate alteration requirements.

4.3.6.2 Crown Pillar

Similarly to the South Portal, the required ground support was calculated for the North Portal. The analysis for the anticipated factor of safety was completed based on the following parameters and assumptions (from BH15-06);

Table 4.19 − Factor of safety calculation parameters

Portal	Average RQD (%)	*Average J_r	*Average J_a	*Average J_n
North	42	1.5	3.0	12

*The values are obtained from data within the 2015 KUG SI Report (Appendix A)

- Based on the anticipated hydrogeological conditions on site, the calculations have assumed dry conditions i.e. the rock mass is fully drained, and therefore no flowing water J_w = 1.0), and partially dewatered conditions indicating a moderate flow of water through the rock mass (J_w = 0.66);

- The average, minimum and maximum Q' values obtained from the results of the geotechnical drilling program were used to derive the factor of safety in order to identify the "worst case scenario";

Table 4.20 below summarizes the compressive strength, the crown pillar span and the crown pillar thickness of the tunnel entrance. Refer to Appendix A for UCS and Q' values, and Appendix G for details of individual crown-pillar calculation worksheets.

Table 4.20 − A summary of the parameters used to calculate the factors of safety

Portal	UCS Range (MPa)	Crown Pillar Span (m)	Average Q' Portals	Crown Pillar Thickness (m)*
North	56.0	5.5	1.0	9.6

*The crown pillar thickness is taken at the tunnel entrance rather than an average thickness over a 10 m span (the width of the bench above the cut slope). As such, this value is more conservative than what may actually be present based on the declining nature of the tunnel.

Table 4.21 and Table 4.22 below summarize the calculated factors of safety under dry and partially saturated conditions. Table 4.21 presents the results without support, Table 4.22 presents results with support.

Table 4.21 − Summary of the calculated factors of safety under unsupported conditions

Portal	Joint Water (J_w) Conditions	Average FoS	Max. FoS	Min. FoS
North	0.66	1.1	1.4	0.7
	1.0	1.2	1.5	0.8

To increase the calculated factors of safety, the application of reinforcement such as steel sets and shotcrete was considered. To gauge the possible variations in the safety factor, consideration was given to several thicknesses of shotcrete. The results are presented in Table 4.22 below.

Table 4.22 − Summary of the calculated factors of safety (FoS) under reinforced conditions

Portal	Average FoS using 100 mm shotcrete	Average FoS using 150 mm shotcrete	Average FoS using 200 mm shotcrete
North	2.4	2.5	2.6

Discussion of Results

When unsupported, none of the calculated factors of safety for dry or moderate inflow conditions meet the required factor of safety of 2.0 for long-term, permanent stability based on the Acceptable Risk Exposure Guidelines – Comparative Significance of Crown Pillar Failure (modified from Carter & Miller, 1995). As a result, addition of ground support such as steel sets and shotcrete is required to achieve the minimum required factor of safety.

Based on the calculations completed, at least 100 mm of shotcrete together with steel sets will be required to achieve the minimum factor of safety of 2.0. The calculated factors of safety are based on the use of 8 inch I-beam steel sets (W8 x 31) in accordance with ASTM A6.

Recommendations

Based on the results of the calculated factors of safety discussed in the sections above, the following reinforcement support will be required to achieve the required factors of safety:

- Steel sets of the W8 x 31 type should be placed every two metres for the first 10 - 12 m from the portal entrance (a length approximately equal to the tunnel diameter);
- On the face and 20 m into the portal, 3 m rebar on a 1.2 m x 1.2 m spacing;
- #4 Galvanized Screen;
- ¼ inch Galvanized Steel Strapping placed around the brow;
- 4 to 6 inch (100 – 150 mm) shotcrete on first 20 m of the drive; and
- 5 – 6 m single Garford cable bolts on a 2 m by 2 m spacing in the back for the first 10 m (a length approximately equal to the tunnel diameter).

4.3.7 Inspections and Monitoring

Similarly to the South Portal, it is recommended that an experienced engineer is present during construction of the North Portal. The main purpose of this will be to confirm that the ground conditions are similar to those used for the designs, based on the data obtained from the 2015 SI program. Any significant variations in the ground conditions may result in changes to the designs.

4.4 Access Tunnel

4.4.1 General

The Triple Decline tunnel forms a part of the segment 1 of the access corridor as shown on Drawing VM00575.6.600-003. However, the Triple Decline Portal and the first 10 m of the Triple Decline tunnel from the portal is included in Amec Foster Wheeler's scope of work. Segment C of the access corridor segment consists of an approximately 889 m long tunnel (herein referred to as the "Access Tunnel") measuring 5.5 mW x 5.5 mH x 889 mL at a –5° decline and azimuth of 204° (measured clockwise from true north as defined by construction design. The Access Tunnel begins at approximately station 0+660 and ends at station 1+520, as shown on Drawing VM00575.6.600-002.

The short tunnel is proposed to be driven through a ridge as shown in VM00575.6.600-004 based on the dimensions provided by AuRico. The following sections provide a brief summary of the design basis and criteria of the Access Tunnel, along with the design recommendation for the tunnel. Refer to Appendix H for the details of the calculations and details such as summary tables and support design packages.

4.4.2 Geotechnical Site Conditions

Rock mass characterization for the short tunnel has been performed based on the geotechnical data collected from oriented core (2015 KUG SI report).

The main rock type encountered for the short Access Tunnel was basalt, belonging to the Asitka Volcanic group. Granodiorite, belonging to the Black Lake Intrusive group, was encountered mainly in the South end of the tunnel. Overburden along the tunnel alignment was generally not recovered as the drill holes were cased from surface to bedrock. Soil observed from road cuts and drill pads comprised primarily of silty sand and gravel. Approximate overburden thicknesses were found to be 9.25 m above the ridge, 3.5 m at the South Portal and 1.5 m at the North Portal.

Core recovery within the basalt was high with most of the cores exhibiting recovery above 90%. While RQD ranged from very poor to excellent (18.7% to 100%), with 50 % of the runs having a good RQD (> 75%). The granodiorite also exhibited recovery above 90% and RQD ranged from very poor to excellent with an average RQD of 68%. Both rock types exhibited high intact strength, generally assessed in the field as medium strong to very strong with average UCS in the range of 154 to 168 MPa. For further details refer to the 2015 KUG SI Report in Appendix A.

For the Access Tunnel, the design rock mass domain is considered equivalent to 2 x the tunnel diameters at the widest part of the tunnel. Since the Access Tunnel also encompasses two bays each 9 mW x 20 mL, the design domain is 20 m surrounding the tunnel. The borehole intersections (KH15-3, KH15-4, KH15-10, KH15-11, KH15-13, KH15-14) in the Access Tunnel area, indicate that the approximately 680 m of the tunnel on the North Portal side is mainly driven through domain A (blue - mainly basalt, with thin layers of feldspar porphyry, andesite, chert, quartz carbonate and greywacke of the Asitka formation) while the rest of the tunnel (209 m) through domain B (pink - mainly granodiorite with thin layers of feldspar porphyry of the Black Lake formation).

4.4.3 Rock Mass Rating

Oriented core logging data from the drill holes in the Access Tunnel footprint were processed to obtain parameters and the ratings for the joint roughness (Jr), joint alteration (Ja) and the number of joint sets (Jn) used in calculation of the Norwegian Geotechnical Institute (NGI) Q-system and modified Q' (Barton, 1974, 1980). Parameters for determination of RMR'76 (Bieniawski, 1976) and RMR'89 (Bieniawski, 1989) were also obtained. Refer to Appendix H for details of the calculation for the rock mass rating for the Access Tunnel.

4.4.4 Fault Structure

As it can be seen in Figure H3 (Appendix H) that although the area is characterized by some of the major fault systems, the Access Tunnel does not appear to be intercepted by any them. However, some minor fault zones have been intercepted in the geotechnical drill holes. The rock mass quality around those zones may be expected to be inferior relative to elsewhere in the tunnel, and can be dealt with of by installing appropriate ground support which will be described in the ground support section.

4.4.5 In-Situ Regime

In absence of any recent *in situ* stress test data on the site, the in-situ stress regime is adopted from the SRK investigation program in completed in 2013 using over coring technique (Deep Door Stopper Gauge System) in four boreholes to complete nine tests. Refer to Appendix H for details of the interpretation of the SRK data.

4.4.6 Tunnel Design

The Access Tunnel is a life of mine service excavation. It is therefore necessary to design the tunnel to ensure its stability over a longer operational life.

It is required to assess the stability of the tunnel along the entire length, the stability of an excavation is a function of mainly the rock mass quality, rock strength, existence of any major geological structure such as faults, dykes, shear zones, etc and rock joint fabric. Additionally, since the Access Tunnel is located in the Kemess North Area (latitude 57°11' and longitude 126°50') within the Northern British Columbia (NBC) seismic source zone, seismic loading conditions is also taken into consideration for the tunnel design. As noted above the tunnel will be 5.5 mW x 5.5 mH x 889 mL at a –5° decline and azimuth of 204°. Refer to Appendix H for detailed design analysis of the expected failure modes, back profile and tunnel size assessments, seismic consideration and ground support for the Access Tunnel.

4.4.7 Recommendations

Based on the review of the available geotechnical data, report and drawings and findings of the current study, the following conclusions and recommendations are made:

- Due to access restriction along the northern end of the tunnel, it was not possible to complete additional geotechnical drilling. Therefore, designs along this part of the tunnel are based on one borehole only. In order to ensure the ground conditions along this section of the tunnel are in accordance with the design parameters selected, it is recommended that regular monitoring by an experienced engineer is completed during the tunnel development. If the ground conditions prove to be significantly different from those anticipated, it is recommended that sub-horizontal drilling is completed within the tunnel, in order to obtain data for both RMR'89 and Q rock mass ratings;
- If additional sub-horizontal drilling is completed during the tunnel excavation, it is recommended that an acoustic televiewer survey is completed to capture structural data;
- It is recommended that geotechnical mapping of the tunnel is performed by an experienced engineer during construction of the tunnel;

- It has been identified that currently only a limited number of laboratory strength tests have been completed. It is recommended that laboratory testing for intact rock strength be performed if additional drilling is carried out during the tunnel excavation.
- Upon completion of the tunnel, routine physical inspections by an experienced engineer will be required to assess any visual signs of rock mass movement, damage of ground support elements, bagging inside the screens, cracking of shotcrete or actual fall of ground;
- Installation of borehole extensometers in the back, along the length of the tunnel at regular intervals (maximum three including at least one in the passing bay) to measure displacement within the rockmass;

4.5 Triple Decline Portal

4.5.1 General

The Triple Decline Portal is located at the north end of the Kemess Lake Valley, connecting the conveyor system to the Access Tunnel for the KUG deposit. The portal footprint is split into three separate portals: conveyor decline, Access Tunnel decline and the intake air decline, with the Access Tunnel decline intersecting the tunnel centerline located at N 6323124 m E 635873 m and approximately El. 1490 m. The total width of the Triple Decline Portal floor is 60 m wide. Refer to Table 4.23 below for the individual portal dimensions.

Table 4.23 – KUG Decline Portal Dimensions

Triple Decline Portal	Height (m)	Width (m)
Conveyor Decline	5.25	4.5
Access Tunnel Decline	5.75	5.5
Intake Air Decline	6.0	6.0

Refer to Drawings VM00575.6.600-022 to 024 for the plan and profile of the portal footprint. The sections below discuss the subsurface conditions and design of the Kemess Triple Decline Portal.

4.5.2 Geotechnical Site Conditions and Terrain Stability

The rock and soil characteristics of the Triple Decline Portal are based on the field logging data of BH15-01 (KH15-27) and BH15-12 (KH15-19). Soil and bedrock interface was interpreted from BH15-01, BH15-12 and depth of refusal from the overburden auger drilling (OB15-XXs) in the portal footprint. Bedrock in the Triple Decline Portal area consist of mainly basalt down to El. 1248 m, overlying granodiorite to termination depth.

The upper 20 m of basalt, which is expected to be within the depth range which will comprise of the portal and portal crown pillar, had an average RQD of approximately 70%. Soil encountered was mainly a trace to silty, sand and gravel mixture, with SPT values ranging from compact to very dense.

The terrain assessment for the Triple Decline Portal identified highly fractured and weathered basalt outcrops at the shallow road cuts near the portal footprint. Further aerial and ground reconnaissance did not identify major terrain geohazards but steep slopes standing at or near angle of repose were observed. Slope cuts in the footprint may cause raveling and instability unless properly designed and supported. Refer to Appendix A for the 2015 KUG SI report for further details on the boreholes and terrain assessment in the Triple Decline Portal footprint.

4.5.3 Kinematic Analysis

The orientations of the Triple Decline Portal and approach slopes for the tunnel were analyzed to determine the kinematic risk of failures for construction. For the portal orientations, the dip/dip direction orientation in the original design was reviewed and a sensitivity analysis was completed by rotating the dip direction ±10° for each portal. For the portal approach slopes, the overall dip angles of each approach slope was reviewed to determine the stability of the cut into the tunnel. In addition, the individual bench cut angle at 75° was also examined for localized failures. Assumptions were made for the friction angle of the Asitka and Black Lake Formations based on literature and experience of the rock types in the area. Refer to Table 4.24 for summary of the parameters and assumptions for the portal face analysis.

Table 4.24 − Summary of Kinematic Analysis Parameters for Portal Orientations

Area	Dip (°)	Dip Direction (°)	Friction Angle (°)	Formations Analyzed
Triple Decline Portal	75	194 (Default)	30	Asitka/Black Lake
Triple Decline Portal	75	184	30	Asitka/Black Lake
Triple Decline Portal	75	204	30	Asitka/Black Lake

The approach slopes (East and West approach slopes) were designed approximately perpendicular to the Triple Decline slope, and consist of a series of 5 m high benches. The overall slope angle ranges between 41 and 44˚, with much steeper individual bench cuts of 75˚. Kinematic analyses were completed for the approach slopes with respect to the overall slope angle, as well as for the individual steeper bench cuts. Assumptions were made for the friction angle of the Asitka and Black Lake Formations based on published literature and experience of the rock types in the area. Refer to Table 4.25 for summary of the parameters and assumptions for the approach slopes.

Table 4.25 − Summary of Kinematic Analysis Parameters for Portal Approach Slopes

Area	Approach Slope	Dip (°)	Dip Direction (°)	Friction Angle (°)	Formations Analyzed
Triple Decline Portal	West	41	130	30	Asitka/Black Lake
Triple Decline Portal	East	44	261	30	Asitka/Black Lake
Triple Decline Portal	West	75	130	30	Asitka/Black Lake
Triple Decline Portal	East	75	261	30	Asitka/Black Lake

The computer software DIPS 6.0 by Rocscience Inc. was utilized to complete the kinematic analysis. The analysis was based on the fracture data logged from diamond drill hole core samples taken by Amec Foster Wheeler in the 2015 KUG site investigation program. The structural data obtained from the oriented core drilling was plotted onto stereonets, in order to visually assess the joint sets present within the rock mass. Where clusters of pole vectors form on the stereonet, these represent the most common joint sets present. As expected randomly distributed joints also occur, although these were typically not considered in the analysis. Stereoplots representing the joint set distribution within the rock mass at each of the portals are shown on Drawing VM00575.6.600-018. Refer to Appendix D for the stereonet plots of the Triple Decline Portal.

The kinematic analysis was conducted to determine the potential risks for the following failure modes: planar sliding, wedge sliding, flexural toppling and direct toppling. Resulting number of poles intersecting planes are tabulated and determines the probability of an instability failure. The results for the Triple Decline Portal face and approach slopes are summarized in Tables D1 – D25 separated by hazard types in Appendix D.

In summary, the highest potential risk of failure for each type of failure mechanism at the Triple Decline Portal face are the following:

- **Planar Sliding**: the highest potentials of failure at the Triple Decline Portal is orientation 75/204;
- **Wedge Sliding**: the highest potentials of failure at the Triple Decline Portal is orientation 75/204;
- **Flexural Toppling**: the highest potential of flexural toppling are 75/194° and 75/184; and
- **Direct Toppling**: the highest potentials of direct toppling is orientation 75/204.

Based on the results of the kinematic analysis, the optimal portal dip/direction orientation for the Triple Decline Portal is 75/194°, as it has the lowest potential risk of failure. Table 4.26 below summarizes the probability of each failure mode at orientation 75/194°.

Table 4.26 – Summary of Failure Modes Probabilities for Triple Decline Portal Orientation 75/194°

Failure Mode	Type	Critical	Total	Probability of Failure (%)
Planar Sliding	All	8	58	13.8
Wedge Sliding	Both Planes	215	1648	13.1
Wedge Sliding	One Plane	293	1648	17.8
Flexural Toppling	All	6	58	10.3
Direct Toppling	Direction Toppling (Intersection)	144	1648	8.7
Direct Toppling	Oblique Toppling (Intersection)	34	1648	2.1
Direct Toppling	Base Plane (All)	13	58	22.4

For the Triple Decline approach slopes, the potential for planar and wedge sliding and flexural toppling is higher along the east slopes than the west slopes. Whereas the potential for direct toppling failure is higher along the west slopes than the east slopes.

Refer to Section 4.5.6 for the discussion of stability support for the Triple Decline Portal face and approach slopes.

4.5.4 Soil Stability Analysis

4.5.4.1 General

A slope stability analysis was completed for the soil slopes above the Triple Decline Portal of the Kemess Underground tunnels based on cross section profiles intersecting the portal face orientations. The analysis was required to determine the potential risks of slope stability failures at the portals. The slope cuts above the Triple Decline Portal entrance and the side approach slopes were modelled to determine their respective factors of safety (FOS). The stability analysis was carried out based on the design cuts with assumed strength and pore pressure conditions. Additional detail of the analyses completed and sections should be referred to in Appendix E.

4.5.4.2 Methodology

The parameters applied to the analysis were established based on data from boreholes BH15-01 and BH15-12 from the 2015 KUG SI program. Refer to Section 4.2.4.2 for further details on the parameters and methodology applied in the slope stability analysis.

The Triple Decline Portal had three sections cut for the portal face; one parallel to each of the tunnels.

Due to the steeply sloping terrain at the Triple Decline Portal, the proposed soil cut slopes have been designed at 60˚ from the horizontal, in order to keep the amount of material having to be excavated to a minimum. The rock slopes underlying the soils will be cut at a steeper angle, and are discussed in subsequent sections.

Given the steep nature of the proposed soil cuts, reinforcement is required to achieve the minimum recommended factor of safety. Soil slope reinforcement designs are discussed in Section 4.5.6 below.

The modelling set up for the Triple Decline Portal approach slopes is similar to those of the South Portal, with a series of 5 m high benches cut to 60° from the horizontal within the soil, and 75˚ from the horizontal within the rock cuts (refer to Drawing VM00575.6.600-014 to 024).

4.5.4.3 Analysis

The stability analysis was conducted to determine the lowest global factory of safety, as well as identify the potential failure surface for deep-seated instability with a FOS of 1. The deep-seated failure surface was required to determine the reinforcement requirements (discussed in Section 4.5.6). For the approach slopes, only the lowest global FOS was considered. Refer to Appendix E for the figures of the Slope/W modelling results and Table 4.27 for the results of the Slope/W analysis of the Triple Decline Portal.

Table 4.27 – Slope/W Analysis Results for North Portal Factor of Safety

Area	Shallow Failure (Lowest)	Deep Seated
Portal Slope Face	0.59	1.04
East Approach Slope	0.59	-
West Approach Slope	1.31	-

The Triple Decline Portal face and east approach slopes will require support and reinforcements to improve the FOS and minimize the likelihood of rockfall or slope instability at the portal location. Refer to Section 4.5.6 for discussion of slope stability support below.

4.5.5 Rock Stability Analysis

4.5.5.1 General

Similarly to the South and North Portals, two-dimensional limit equilibrium stability analyses was completed for the proposed rock cut slope at the Triple Decline Portal. Since there are three separate tunnel at the Triple Decline Portal, analyses were completed along sections drawn parallel to each of the tunnels.

The average cut slope height along Sections A-A’, B-B’ and C-C’ is 18 m (ranging from 17 m to 19 m), and sloped at 75˚ from the horizontal. As is the case for the South and North Portals, a 10 m wide bench is proposed above the rock cut. The proposed slope above the bench consists of approximately 7 m of rock overlain by 5 to 10 m of soil. The slopes above the bench were not analyzed with respect to slope stability, as these have been designed at a much shallower angle. A separate analysis has been completed for the soil slope with respect to reinforcement requirements (Section 4.5.6 below). Refer to Drawings VM00575.6.600-022 to 024 for profiles of the proposed slope geometry at the Triple Decline Portal.

4.5.5.2 Methodology

The two-dimensional limit equilibrium analyses completed for the Triple Decline Portal used similar parameters to the South and North Portals; the GLE/Morgenstern-Price slice method, and the Generalized Hoek-Brown strength type. The strength parameters were developed based on data obtained from borehole BH15-01 and BH15-12 (refer to the 2015 KUG SI report in Appendix A). Since the Triple Decline Portal is to be developed in granodiorite as well as basalt, the following parameters were derived using the Rocscience RocLab and SLIDE programs:

- Geological Strength Index (GSI) = 35
- Material Constant (m_i) = 25 for basalt;
- Material Constant (m_i) = 29 for granodiorite; and
- Disturbance Factor (D) = 0.8 based on assumption of moderately controlled blasting will be conducted.

Table 4.28 − Design parameters used to conduct the stability analysis from RocLab

Portal		Average Unit Weight (kN/m3)	Average UCS (MPa)	m_b*	s*	a*
Triple Decline	A-A'	26.7	118.1	0.61	5.28	0.52
	B-B'	26.7	118.1	0.61	5.28	0.52
	C-C'	29.5	69.6	0.52	5.28	0.52

*Where m_b is a reduced value of mi; "s" is given by s = exp ((GSI-100)/9-3D); "a" is given by a = 0.5 + 1/6 ($e^{-GSI/15}$ - $e^{-20/3}$).

4.5.5.3 Analysis

Table 4.29 below summarizes the calculated global minimum and deep-seated factors of safety, under static, dry and partially saturated conditions, as well as for pseudo-static, partially saturated conditions.

Table 4.29 − Calculated factors of safety under static and pseudo-static conditions

Portal	Section ID	FoS Static Dry		FoS Static Part. Saturated		FoS Pseudo-Static Part. Saturated	
		Shallow	Deep Seated	Shallow	Deep Seated	Shallow	Deep Seated
Triple Decline	A-A’	2.4	N/A	1.8	2.0	1.5	N/A
	B-B’	2.5	N/A	1.8	2.0	1.6	1.4
	C-C’	1.9	N/A	1.2	1.9	0.9	1.4

*Note that for some of the analyses conducted, the potential for deep seated sliding is considered negligible, resulting in no FoS value being calculated.

For dry, static conditions, the recommended minimum factor of safety of 1.5 was achieved with respect to shallow and deep-seated instability. Under partially saturated, static conditions, the recommended minimum factor of safety was not achieved along Section C-C’ for shallow instability. This suggests that drained conditions are required to meet the minimum required factor of safety for long-term stability. The minimum required factor of safety (1.1) under pseudo-static conditions was achieved under dry conditions, but not for partially saturated conditions along section C-C’ for shallow instability.

In general the recommended slope designs at the Triple Decline Portal appear to meet the required factor of safety under dry conditions. However, it must be noted that based on the kinematic analysis completed (see Section 4.5.3 above and Appendix D), there may be a potential for localized wedge and toppling failure as a result of the discontinuity distribution. These smaller-scale failures can typically be managed through bolting and mesh. This will need to be assessed by an experienced engineer during construction.

As well as review of the requirement for reinforcement during construction of the Triple Decline Portal, it is recommended that scaling of loose blocks is completed for improved safety under rock slopes. Additionally, it is recommended that a catchment ditch is constructed along the toe of the cut slope, to divert water away from the toe, and to catch any material ravelling from the cut slope. Details of the recommended ditch dimensions are included in Appendix F.

4.5.6 Support Assessment

4.5.6.1 Soil Nails

The slopes above the Triple Decline Portal entrance will be developed at fairly steep cut slopes of approximately 60˚ from horizontal. Based on the two-dimensional limit equilibrium stability analysis discussed in the section above, the minimum factor of safety will not be achieved if the slopes remain unsupported. Soil nails and shotcrete surfacing is recommend to support the soil cut slopes.

Similarly to the North Portal, the U.S. Department of Transportation Federal Highway Administration (FHWA) Geotechnical Engineering Circular No. 7. Soil Nail Walls document was consulted for guidance.

The proposed soil cut slopes at the Triple Decline Portal range from about 6.5 to 11 m vertically. The natural slope above the cut is moderately steep at approximately 30 to 34˚;

Soil Properties

The soil properties across the Triple Decline Portal is similar to what was observed along the South and North Portals; granular, cohesionless sands and gravels with varying degree of fines. The angle of friction of these soils is estimated at about 35˚, and the soil unit weight (ɣ) is estimated at 20 kN/m$^{3.}$ The 2015 KUG SI Report in Appendix A should be referred to for additional detail on the soil properties. There may be unforeseen variations within the overburden that will need to be considered during the nail installation stage.

Soil Nail Parameters

Based on the calculation checks completed for a minimum factor of safety of 1.5 under static conditions (refer to Appendix G), the following soil nail lengths are recommended to support the soil mass above a potential deep seated failure surface (refer to Drawing VM00575.6.600-25).

Table 4.30 − Summary of proposed nail length at each portal

	Installation: Soil nails inclined at 15˚ from horizontal, spacing at 1.5 x 1.5 m. Horizontal drains installed at similar spacing in centre of nail pattern.		
	Soil Nail Length (m)		
	Triple Decline		
Row No.	**Section A**	**Section B**	**Section C**
1	15	15	15
2	15	15	10-15
3	15	10-15	10-15
4	15	10	10-15
5	10-15	<10	<10
6	10	<10	<10
7	<10	<10	<10
8	<10	-	-

These soil nails lengths are based on the current overburden/ bedrock contact model. In the event the depth of this contact is different from the existing model, the soil nail lengths may need to be adjusted.

Discussion of Results

The recommended global factor of safety of 1.5 for long term stability under static conditions, is achieved based on the verification calculations discussed in Appendix G. However, it must be noted that variations within the ground conditions may warrant further analysis or adjustment to the design recommendations during the construction stage.

When considering the strength of the soil nails only, the capacity of the #14 Grade 75 soil nails at 60% yield load, is adequate to support the anticipated mass above a potential deep seated failure surface. However, should significant variations in overburden thickness be observed during construction, it is recommended the existing geological model be updated accordingly, with subsequent updates to determine the required nail capacity requirements.

The value of the ultimate bond strength appears to vary depending on the type of material as well as on the construction / installation method. In this case, cohesionless sands and gravels are assumed to make up most of the soils into which soil nails will be installed. Additionally, given the potential presence of coarse gravel and cobbles, the nails will likely be installed through driven casing. The calculated nail lengths beyond the proposed failure surface, depend on the ultimate bond strength value assigned to the soil. If during construction this value is considered to be different from the value assigned 190 kPa (from Table 3.1 of the FWHA manual), these lengths will need to be recalculated. The capacity can also be verified through pull tests.

In areas where the overburden and rock contact is anticipated to be encountered along a proposed nail length, it is recommended that the quality of the rock is assessed in order to determine the required nail installation depth. Where this contact is encountered, it is recommended that the nail is installed past the area of poor quality rock. This will need to be assessed during the installation process. It is not recommended that the nail is installed well into strong competent bedrock.

Additional to updates to design parameters during the installation process, random pull tests of the soil nails should be completed, to ensure ongoing satisfactory compliance in accordance with design specifications.

Other Design Considerations

As discussed for the North Portal, horizontal drain pipes should be used to divert water away from the slope. Additionally, a runoff diversion ditch should be constructed to remove water from the toe and crest of the soil slope. Refer to Drawing VM00575.6.600-025. The proposed installation details are discussed in Appendix G as well as in the section for the North Portal.

As previously discussed, Double corrosion protection (DCP) may be used to provide additional protection to the nails.

Temporary shotcrete facing will be required during construction of the soil nail wall. The total shotcrete thickness should be in the range of 150 to 300 mm, with welded wire mesh reinforcement. The nail heads which are typically welded to a bearing pate should be fully encapsulated in the permanent shotcrete facing.

Soil Nail Installation Recommendations

Similarly to the North Portal, the following design criterion are recommended, and are outlined on Drawing VM00575.6.600-025:

- Install Grade 75 (#14) threaded soil nails and hardware (Dywidag Threadbar – ASTM A615);
- Soil nails should be installed in a square pattern, with a regular spacing of 1.5 m;
- The nails should be installed at an angle 15˚ below horizontal;
- The nail lengths should be in accordance with Table 1, ensuring embedment at least 4 m beyond the proposed failure surface, unless competent bedrock is encountered;
- Slotted horizontal drain pipes should be installed at an angle between 5 - 10˚ above horizontal to permit drainage of water. The pipes should include an end cap and a geosoc to prevent blocking by fine grained soils;
- Complete pull testing on select nails;
- For protection, the surface should be coated in 150 to 300 mm of shotcrete with wire mesh reinforcement;
- A catchment ditch should be constructed along the toe of the soil slopes to divert water from the horizontal drains away from the crest of the slope to avoid erosion from water as well as pore pressure building up behind the underlying rock cut slope; and
- A surface water run-off ditch should be constructed along the crest of each slope, to divert water away from the surface.

It must be noted that these guidelines and recommendations are based on our current understanding of the ground conditions. In the event variations in ground conditions are observed, the designs may need to be altered accordingly. Therefore it is recommended that an experienced Geotechnical Engineer is present during the installation of the soil nails, in order to respond to any immediate alteration requirements.

4.5.6.2 Crown Pillar

Similarly to the South and North Portals, the required ground support was calculated for the Triple Decline Portal. The analysis for the anticipated factor of safety was completed based on the following parameters and assumptions (from BH15-01 and BH15-12);

Table 4.31 − Factor of safety calculation parameters

Portal	Average RQD (%)	*Average J_r	*Average J_a	*Average J_n
Triple Decline	20	1.5	2.0	12

*The values are obtained from data within the 2015 KUG SI Report (Appendix A)

- Based on the anticipated hydrogeological conditions on site, the calculations have assumed dry conditions i.e. the rock mass is fully drained, and therefore no flowing water J_w = 1.0), and partially dewatered conditions indicating a moderate flow of water through the rock mass (J_w = 0.66);
- The average, minimum and maximum Q' values obtained from the results of the geotechnical drilling program were used to derive the factor of safety in order to identify the "worst case scenario"; and

Table 4.32 below summarizes the compressive strength, the crown pillar span and the crown pillar thickness of the tunnel entrance. Refer to Appendix A for UCS and Q' values, and Appendix G for details of individual crown-pillar calculations. Since the Triple Decline consist of three separate tunnels, it was considered prudent to evaluate the factor of safety along each tunnel.

Table 4.32 − A summary of the parameters used to calculate the factors of safety

Portal	Section	UCS Range (MPa)	Crown Pillar Span (m)	Average Q' Portals	Crown Pillar Thickness (m)*
Triple Decline	A	19.2 - 121.2	5.5	0.6	10.7
	B	19.2 - 121.2	5.5	0.6	10.3
	C	19.2 - 121.2	5.5	0.6	9.3

*The crown pillar thickness is taken at the tunnel entrance rather than an average thickness over a 10 m span (the width of the bench above the cut slope). As such, this value is more conservative than what may actually be present based on the declining nature of the tunnel.

Table 4.33 and Table 4.34 below summarize the calculated factors of safety under dry and partially saturated conditions at each of the sections. Table 4.33 presents the results without support, Table 4.34 presents results with support.

Table 4.33 − Summary of the calculated factors of safety under unsupported conditions

Portal	Section	Joint Water (J_w) Conditions	Average FoS	Max. FoS	Min. FoS
Triple Decline	A	0.66	0.9	1.1	0.8
		1.0	1.0	1.2	0.9
	B	0.66	1.0	1.3	0.8
		1.0	1.1	1.4	0.9
	C	0.66	0.9	1.0	0.8
		1.0	1.0	1.1	0.9

To increase the calculated factors of safety, the application of reinforcement such as steel sets and shotcrete was considered. To gauge the possible variations in the safety factor, consideration was given to several thicknesses of shotcrete. The results are presented in Table 4.34 below:

Table 4.34 − Summary of the calculated factors of safety (FoS) under supported conditions

Portal	Section	Average FoS using 100 mm shotcrete	Average FoS using 150 mm shotcrete	Average FoS using 200 mm shotcrete
Triple Decline	A	2.1	2.1	2.2
	B	2.1	2.2	2.3
	C	2.4	2.5	2.5

Discussion of Results

Similarly to the South and North Portals, when unsupported, none of the calculated factors of safety for dry or moderate inflow conditions meet the required factor of safety of 2.0 for long-term, permanent stability based on the Acceptable Risk Exposure Guidelines – Comparative Significance of Crown Pillar Failure (modified from Carter & Miller, 1995). As a result, addition of ground support such as steel sets and shotcrete is required to achieve the minimum required factor of safety.

Based on the calculations completed, at least 100 mm of shotcrete together with steel sets will be required to achieve the minimum factor of safety of 2.0. The calculated factors of safety are based on the use of 8 inch I-beam steel sets (W8 x 31) in accordance with ASTM A6.

Recommendations

Based on the results of the calculated factors of safety discussed in the sections above, the following reinforcement support will be required to achieve the required factors of safety:

- Steel sets of the W8 x 31 type should be placed every two metres for the first 10 - 12 m from the portal entrance (a length approximately equal to the tunnel diameter);
- On the face and 50 m into the portal, 3 m rebar on a 1.2 m x 1.2 m spacing;
- #4 Galvanized Screen;
- ¼ inch Galvanized Steel Strapping placed around the brow;
- 4 to 6 inch (100 – 150 mm) shotcrete on first 20 m of the drive; and
- 5 – 6 m single Garford cable bolts on a 2 m by 2 m spacing in the back for the first 10 m (a length approximately equal to the tunnel diameter).

4.5.7 Inspections and Monitoring

Similarly to the South Portal, it is recommended that an experienced engineer is present during construction of the North Portal. The main purpose of this will be to confirm that the ground conditions are similar to those used for the designs, based on the data obtained from the 2015 SI program. Any significant variations in the ground conditions may result in changes to the designs

5.0 GEOCHEMISTRY

5.1.1 Corridor Geochemistry

The geochemical investigation was conducted at the proposed site for the Access Tunnel and surface infrastructure of the Kemess Underground Project. The purpose of this geochemical study is to understand the potential for metal leaching and acid rock drainage (ML/ARD) from materials that will be removed from the Access Tunnel and surface infrastructure area as well as from the rock surfaces that will be exposed after the completion of the tunnel and portals.

During the 2015 SI program, core samples were collected from core holes located at the Access Tunnel footprint and in the vicinity of the proposed South, North and Triple Decline Portals. In addition, soil samples were collected from the test pits located at the proposed Infrastructure facility.

In total, 793 core samples collected from those boreholes and 4 soil samples collected from the test pits underwent geochemical testing. All samples underwent elemental analysis mainly by four acid digestion with ICP-MS finish. Acid base accounting (ABA) was conducted on 217 samples, while the shake flask extraction (SFE) test to assess the content of readily leachable metals was performed on 210 samples. Four soil samples were analysed for metals, total inorganic/organic carbon and cation exchange capacity.

The summary of geochemical investigation results are presented below; details of the geochemical testing are reported in the 2015 KUG SI Report (Appendix A).

5.1.2 Geochemical Characterization

From a regulatory standpoint, assessment and management of ML/ARD is conducted on a site-specific basis. However, generic classification of rock as either potentially acid generating (PAG) or non-potentially acid generating (NAG) is done using the measured neutralization potential ratio (NPR) of a sample or rock unit. For this current feasibility study the NPR of 2 is proposed as the threshold value to separate PAG rock and NAG rock at the Access Tunnel, portals and infrastructure corridors.

5.1.2.1 South Portal

In the borehole located near the proposed South Portal (Drawing VM00564.6.600-028), Black Lake granodiorite was encountered from approximately 7.5 m below surface to borehole termination depth of 45.0 m. Hematite was identified as the main alteration faces in this borehole.

ABA results for one sample collected around 8.0 m depth and another other sample from around 30 m depth indicated both samples were NAG with high NPR values. The low sulphur ICP contents (equal or less than 0.02 %) were identified for 26 samples collected from 6 m to 45 m depth.

The majority of samples were enriched in bismuth and antimony. The elevated concentrations of bismuth and antimony in some samples are likely due to the high detection limits associated with these samples. The short term leaching test results for two samples suggest there was no evidence for soluble metal phases that would be leached under neutral drainage conditions.

5.1.2.2 North Portal

Slightly weathered Asitka Group basalt was encountered at 10.5 m below the overburden surface in the borehole located in the vicinity of the North Portal (Drawing VM00575.6.600-026 and 027). A series of sedimentary rocks of the Asitka Group (quartz carbonate and greywacke) were identified, from around 31.5 m to termination depth of 65 m. Pyrite was observed on core samples collected from 25.5 m to 61.5 m depth and occurred as finely disseminated and veinlets pyrite.

ABA tests conducted on two samples collected from 30.2 m and 34.4 m depth, indicated both samples were PAG. Sulphur speciation results on two samples indicate sulphide was the primary form of sulphur. The median sulphur ICP results of 36 samples was low (0.16%). The low sulphur contents (less than 0.01%) were measured in the samples collected up to 21 m depth while the relatively high sulphur contents (around 1% or greater) were measured on samples collected from 28.8 m to 31.1 m depth.

The majority of samples from the North Portal borehole were enriched in bismuth and antimony and less frequently for arsenic. The elevated concentrations of bismuth and antimony in some samples are likely due to the high detection limits associated with these samples. The short term leaching test results suggest there was little evidence of concern for soluble metal phases to be released under neutral drainage conditions.

5.1.2.3 Access Tunnel

Six HQ drill holes located along the alignment of the proposed Access Tunnel were completed during the 2015 Site Investigation program. In total, 668 core samples were collected for the geochemistry testing with 40 samples estimated to be within the Access Tunnel volume.

Two rock domains, Asitka Group and Blake Lake Intrusives were encountered in the boreholes located at the Access Tunnel footprint. The Blake Lake Intrusives were identified at the South end of the tunnel. The majority of samples collected from Asitka Group rocks were basalts; volcanic breccia was encountered at one borehole. Granodiorite was the rock type encountered within the Blake Lake Intrusives.

Pyrite was observed visually on most of the core samples with the exception of granodiorite core samples from the borehole located at south end of tunnel. Pyrite concentrations ranged from trace to 8%. Chalcopyrite was also observed in core samples with content less abundant compare to pyrite. Molybdenite was also observed visually in the core samples of certain boreholes.

ABA results indicate that PAG rock (NPR<2) represents the Access Tunnel volume with the exception of South end of the tunnel. Granodiorite rock that was encountered at the South end of tunnel are NAG rock. Drawing VM00575.6.600-027 shows the distribution PAG and NAG rocks in the boreholes located at the Access Tunnel.

The majority of samples from the Access Tunnel volume were enriched in bismuth and antimony and less frequently for arsenic and molybdenum. The elevated concentrations of bismuth and antimony in some samples are related to the high detection limits used for these samples. The short term leaching test results suggest that arsenic and molybdenum could require further consideration.

5.1.2.4 Triple Decline Portal

Basalt of the Asitka Group was encountered at two boreholes located north and north-east of Triple Decline Portal (Drawing VM00565.6.600-026). While granodiorite of the Blake Lake group with rock type was identified at one borehole located at the north-west of the proposed Triple Decline Portal. Minor pyrite was observed on core samples (0.1% or less). Sparse molybdenite was also observed in some core samples.

The ABA test conducted only on one sample collected 22.5 m below the surface from the borehole located to the north-east of the Triple Decline Portal. The rock sample is PAG. The sulphur ICP contents of samples collected from all three boreholes were quite variable.

Most samples collected from all three boreholes had bismuth and antimony enrichment. The elevated concentrations of bismuth and antimony in some samples are related to the high detection limits used for these samples. Molybdenum enrichment measured at various depth of the core samples as molybdenite was also observed visually in these samples. The short term leaching test results conducting on one sample suggest that molybdenum could require further consideration.

5.1.3 Waste Rock Management

The recent ARD/ML characterization study indicates that both PAG and Non-PAG rock will be encountered within the Access Tunnel volume. Rock segregation based on the acid generation potential is recommended during the excavation, therefore the Non PAG rock can be utilized for fill materials or the other purposes.

The geochemical investigation results on the Access Tunnel suggest that the rock belonging to the Asitka Group are PAG and the Blake Lake Intrusives are NAG. Asitka Group rock was encountered within the Access Tunnel from the north end of the tunnel to approximately 0.8 km from the north end. The remainder of the Access Tunnel including the South Portal is considered to be NAG rock. PAG rock accounts for approximately 80% of the Access Tunnel volume. The North Portal and Triple Decline Portal volumes appear to be composed of PAG rock.

Rock that will be removed from the Access Tunnel and portal areas will follow the management practice for waste rock from the proposed underground mine. All PAG rock from the Access Tunnel and portals which is not suitable for construction materials will be placed in the Kemess South open pit while the NAG rock is recommended to be reused as fill material.

5.1.4 Seepage Quality

During the Access Tunnel construction, the bedrock will be excavated and the PAG and NAG rock comprising the tunnel surfaces will be exposed during the mine life. It is estimated that the steady state seepage rate around 2 L/s will be generated from the Access Tunnel and will contact PAG and NAG rock walls of the tunnel.

Seepage contacted from or in contact with the NAG rock is not expected to be problematic based on the short-term leaching testing results. Also numerous access and haul roads around the Kemess South site have been constructed using similar NAG waste rock produced from the open pit; it is Amec Foster Wheeler's understanding that water quality associated with runoff from these materials have been non-problematic.

However the exposed PAG rock surfaces can potentially generate acidic conditions during the mine life and may affect the quality of seepage draining from the Access Tunnel. Management of the tunnel drainage may be required under these conditions.

The prediction of seepage quality from the Access Tunnel has not been conducted in this feasibility study due to the lack of data from the project area. However, data is available from related mine rock geochemistry studies at Kemess Mine. It is therefore recommended that a seepage quality estimation is performed in the next phase of design using the information from previous mine site geochemical studies.

6.0 RECLAMATION AND CLOSURE ASPECTS

6.1.1 General

Following the cessation of active mining activities for KUG the site will be reclaimed to return the site to as natural a state as is practical. The primary objective for reclamation will be to achieve self-sustaining landforms that enhance the local ground stability and reduce erosion potential on and adjacent to the remaining access road components. The following objectives are implicit in achieving this goal:

- Regrading and reclamation of all access roads, ponds, ditches and borrow areas that are not required beyond mine closure, and maintenance of those roads that will be required for long term monitoring and maintenance of the portal area.
- Long-term stabilization of all exposed erodible materials.
- Natural integration of disturbed lands into the surrounding landscape after mining ceases and, to the greatest extent practicable, restoration of the area’s natural appearance.
- Establishment of self-sustaining vegetation, where appropriate, consistent with existing forestry and wildlife needs.

The Kemess South Mine Reclamation and Closure Plan (KS RCP) was updated in 2010 and provides detailed reclamation strategies and prescriptions for the general Kemess Mine Site (Northgate 2010b). Additional details pertaining to the proposed KUG access corridor and portal area are discussed in subsequent sections.

6.1.2 Portal Laydown Areas

Following closure and decommissioning of the underground mine the portal laydown areas and any other disturbed areas in the vicinity of the portals shall be reclaimed. The areas will be covered with locally available overburden stockpiled during development. Sloping surfaces should then be roughened using an excavator and/or ripped using a dozer as appropriate to obtain a “rough and loose” final surface designed to reduce soil erosion. Final reclamation should include placement of coarse woody debris spread over the slopes to provide wildlife habitat and to further encourage establishment of micro-sites for ingress of native vegetation followed by fertilization and planting with plugs of willow, lupine, and native grasses as specified in Northgate (2010b).

6.1.3 Ventilation Raise Extension Access Road

The ventilation raise access road shall be deactivated in accordance with forest best management practices (MOF 2002). The road fill will be pulled back and regraded to place the road in a self-sustaining state. The deactivated road should be planted with plugs of willow, lupine, and native grasses as specified in Northgate (2010b). It should be noted that the cost of reclamation work for the existing exploration road is not part of the cost estimate presented in this report as it is assumed to be covered under the existing Kemess South reclamation budget.

6.1.4 Conveyor Right-of-Way

Following removal of the overland conveyor (including the hung portion within the Access Tunnel and elevated sections) the conveyor right-of-way shall be deactivated in accordance with forest best management practices (MOF 2002). The road fill will be pulled back and regraded to place the road in a self-sustaining state. The deactivated road should be planted with plugs of willow, lupine, and native grasses as specified in Northgate (2010b).

6.1.5 Long-term Portal Access

It is understood that long-term access to the decline portals will be required following closure of the Kemess Mine in order to convey seepage from the underground workings to a treatment plant located near the existing mill complex using the dewatering pipeline installed during operations along the corridor. In order to provide such long-term access, Segments 1, 2, and 3 will be partially decommissioned and reclaimed to provide single lane access to the portals. The reduced access corridor would remain until such time as collection and treatment of underground seepage is no longer required based on site water quality objectives at which point full road deactivation activities could be undertaken to permanently deactivate and reclaim the corridor.

6.1.6 Decline and Access Tunnel Portals

It is understood that the decline portals will be plugged at the end of mining with installation of piping and pumps to transfer groundwater/seepage collected within the decline tunnels to the treatment facility near the existing mill complex. It is assumed that the dewatering system (pump house and pipeline) installed for KUG operations can continue to be utilized for this purpose throughout closure with minor modification and that treatment will continue until the water quality meets the requirements so it can be released in to the environment. It is assumed that the rock within the short tunnel is NAG, therefore, no additional treatment or plugging is required based on the 2015 KUG geochemistry program.

7.0 QUANTITIES, SCHEDULE AND COST ESTIMATE

7.1 Infrastructure Design Assumptions

Several main criteria and assumptions were established in order to design the various surface infrastructure. Refer to Section 3.2 for the design criteria. All notes, criteria, and assumptions related to evaluation of quantities for cost estimation purposes are presented in Table I-1 in Appendix I. Sample calculations used to define unit costs for the construction operations are also presented in Appendix I.

For all earthworks designed, a stripping thickness of 0.3 m was assumed based on general observations of nearby exploration roads. Service Road profiles were designed such that grades are below 20%. Side slopes of 2H:1V were used for all daylighting cuts and fills.

7.2 Construction Quantities

A summary of the neat-line cut and fill volumes required for the construction of the access corridor, laydown areas, service roads and vent raise access road (as illustrated on Drawings VM00575.6.600-005 to 009) is provided in Table 7.1 below. The quantities listed below were estimated using the Civil3D package for AutoCAD and are based on area averaging of the cut and fill sections on a 25 m stationing interval. The table also provides comments on the use of surplus excavation materials in fill deficit areas.

Table 7.1 − Surface Infrastructure Cut/Fill Balance Summary Sheet

	Segment	**Reference Alignment**	**Station, From - To**	**Item # in table ******	**Cut Vol.** bcm	**Fill Vol.** ccm	**Net Vol.** ccm	**Comments**
Access Road	A - Mill to Waste Dump	Conveyor	4+865 to 2+875	1.2	131,965	48,842	(93,680)	Surplus material from the excavation to be discarded in designated waste dump
	B - Waste Dump to South Portal	Conveyor	2+875 to 1+525	1.1	33,375	52,483	16,438	Surplus material from the excavation of the access road to be used as fill for South Portal Laydown
	C - South Access Portal to North Access Portal (Access Tunnel)	Conveyor	1+525 to 0+655	5.1	N/A[1]	0	N/A[1]	This segment is the tunnel portion of the access road. The PAG material excavated will be transported to the TSF
	D - North Access Portal to Declines Portals	KLV Access Rd	0+869 to -0+093	3.0	31,859	24,148	(10,260)	Surplus material from the excavation of the access road to be used as fill for service road to propane platform and propane platform
				Sub-Total	**197,199**	**125,473**	**(87,502)**	
Service Roads	Declines Portals to Elevated Conveyor Section #2	KLV Conveyor Service Rd	0+060 to 0+400	3.2	7,842	3,874	(4,596)	Surplus material from the excavation of the service road to be reused in subsequent segment of service road
	Elevated Conveyor Section #2 to KLV Sedimentation Pond	KLV Conveyor Service Rd	0+400 to 0+610	3.2	1,622	3,467	1,716	
	Elevated Conveyor Section #2 to North Portal	Conveyor	0+375 to 0+655	3.2	16,995	761	(17,593)	Surplus material from the excavation to be discarded in designated waste dump

	Segment	Reference Alignment	Station, From - To	Item # in table ****	Cut Vol. bcm	Fill Vol. ccm	Net Vol. ccm	Comments
	Underground dewatering pond	Dewatering pond service RD	0+250 to 0+680	3.2	7,800	2,583	(5,841)	
	Propane Laydown Service Road	KLV Propane Service Rd	0+000 to 0+120	3.2	0	8,640	8,640	Fill to be supplied from excavation of various infrastructure in surrounding areas
				Sub-Total	**34,259**	**19,325**	**(17,675)**	
Portals and Laydowns	Declines Portals[1]	NA	NA	6.1	19,544	0	(21,108)	Surplus material from the excavation to be used as fill for the propane platform
	Ore Stockpile Laydown	NA	NA	6.2	6,792	0	(7,335)	Surplus material from the excavation to be used as fill for the propane platform
	Propane Laydown	NA	NA	6.2	0	47,580	47,580	Fill to be supplied from excavation of various infrastructure in surrounding areas
	North Portal[1]	NA	NA	4.1	16,477	0	(17,795)	Surplus material from the excavation to be discarded in designated waste dump
	North Portal, Office Laydown (North-West)	NA	NA	4.2	1,983	1,025	(1,117)	Surplus material from the excavation to be discarded in designated waste dump
	North Portal, Contractor Laydown, (North-East)	NA	NA	4.2	2,139	14,906	12,596	Surplus material from the excavation to be discarded in designated waste dump
	South Portal[1]	NA	NA	2.1	31,608	0	(34,137)	Fill to be supplied from excavation of various infrastructure in surrounding areas
	South Portal Laydown	NA	NA	2.2	26,601	53,310	24,580	Fill to be supplied from excavation of various infrastructure in surrounding areas

	Segment	**Reference Alignment**	**Station, From - To**	**Item # in table ******	**Cut Vol.** bcm	**Fill Vol.** ccm	**Net Vol.** ccm	**Comments**
				Sub-Total	**105,144**	**116,821**	**3,265**	
Water Mgmt	KLV Sedimentation pond dam	KLV Conveyor Service Rd	0+610 to 0+647	8.1	1051	0	(1,135)	
				Sub-Total	**1,051**	**0**	**(1,135)**	
				Corridor Total	**337,653**	**261,619**	**(103,047)**	
Vent Raise Access Road[2]	Hillside Section	Vent Raise Access Rd	0+000 - 3+200	9.1	107,200	107,200	-	
	Ridgeline Section	Vent Raise Access Rd	3+200 - 5+000	9.1	28,000	28,000	-	
	Saddle Access Section	Vent Raise Access Rd	5+000 - 6+150	9.1	38,525	38,525	-	
				Sub-Total	**173,725**	**173,725**	**-**	
				Grand Total	**511,378**	**435,344**	**(103,047)**	

1- Cut volumes related to the construction of the Access Tunnel and portal bedrock excavation are not considered in the Cut/Fill balance since the excavated material is expected to be PAG waste rock which cannot be reused as fill.

2- Cut/Fill volumes related to the construction of the Vent Raise Access Road taken from AMEC (2012)

Table 7.1 indicates that a surplus of 103,047 ccm will remain from the construction of infrastructure in the KLV and between the mill and South Portal of the Access Tunnel. Thus, borrow pits may not be required to produce general fill for laydowns and road sub-base. However, depending on the quality of the material excavated, a borrow pit may be required to produce suitable material for road topping. For the purpose of this study, the material excavated during infrastructure construction was assumed to meet requirements of road topping material based on the site investigations performed to date.

Table 7.2 below presents the cut/fill balance south of the Access Tunnel and in the KLV. Surplus fill could be used to widen laydown areas and service roads as needed to better suit vehicle mobility.

Table 7.2 – Cut/Fill Balance South of the Access Tunnel and KLV area

	Cut Vol.	**Fill Vol.**	**Net Vol.**
	ccm	ccm	ccm
Mill to South Portal	241,433	154,635	(86,799)
KLV area	123,232	106,984	(16,248)

It must be noted that the haul road from the North to Triple Decline Portals (Segment D), as well as the service roads in the KLV, are already partially constructed. This was not considered in quantity estimations or in scheduling, since these roads were not present at the time of the most recent LiDAR survey.

7.3 Preliminary Cost Estimate

An estimate of the capital costs anticipated to construct the earthworks discussed herein (along with associated assumptions) is provided in Table 7.3 below. The estimate includes additional discussion and calculations on the costs associated with clearing and grubbing, overburden stripping and the use of surplus materials as fill in deficit areas. Table 7.3 provides a summary of the estimated costs by segment and area, however the reader is referred to Appendix I in order to obtain a complete understanding of the assumptions and context of the estimate. The operating costs associated with the installation and maintenance of the conveyor system, pipelines and power cables are not included in this estimate.

Table 7.3 − KUG Surface Infrastructure Cost Estimate

Item No.	Description of work	Direct Cost (2015 M$CDN)	Other AuRico Cost (2015 M$CDN)	Indirect Cost Allocation (2015 M$CDN)	Contingency (2015 M$CDN)[1]	Total Cost (2015 M$CDN)
1.0	Corridor - Mill to Access Tunnel South Portal	2.14	0.00	1.24	0.51	3.88
2.0	Access Tunnel - South Portal	2.14	0.00	1.24	0.76	4.14
3.0	Corridor - Access Tunnel North Portal to Decline Portal	1.70	0.00	0.99	0.40	3.09
4.0	Access Tunnel - North Portal	1.82	0.00	1.05	0.69	3.56
5.0	Access Tunnel	8.59	0.00	0.13	2.15	10.87
6.0	Triple Declines - Portal	2.75	0.00	1.60	1.04	5.39
7.0	Concrete - Conveyor Foundations	1.47	0.00	0.85	0.35	2.67
8.0	Water Management	0.20	0.00	0.12	0.05	0.37
9.0	Vent Raise Access Road Extension	0.12	0.00	0.07	0.03	0.22
10.0	QA/QC and Detailed Engineering	0.00	2.01	0.00	0.30	2.31
11.0	Reclamation	0.00	0.61	0.00	0.09	0.70
	Grand Total (2015 $CDN)	**20.94**	**2.61**	**7.29**	**6.36**	**37.21**

1 - Contingency applied to sum of direct and indirect costs. 15% applied to all items, except 4.0, where 25% was applied due to the nature of construction works and possible winter conditions

7.4 Construction Schedule

A preliminary schedule for construction of the KUG access corridor and associated earthworks was developed based on cut/fill quantities and Amec Foster Wheeler's experience. A high level Gantt chart for corridor construction is provided with the cost estimate in Appendix I.

The schedule is based on single crew per task, assuming 7 day weeks and 12 hour shifts. With the exception of the Access Tunnel excavation, all construction activity durations are based on single shift days. For planning purposes, Amec Foster Wheeler avoided the scheduling of construction activities during the winter as much as possible. Winter was assumed to span the months of November through May.

Tentative resources were allocated to each task to estimate the workforce size and equipment required for the project. Tasks were logically sequenced to best utilize common equipment for similar tasks.

The preliminary schedule presented herein indicates that the project will occur over a period of two years. The schedule mainly revolves around the construction of the Access Tunnel (Segment C), which is expected to occur from winter 2016 to summer 2017. To allow this, the South Portal must be constructed before the tunnelling activities begin, while the North Portal must be ready before the end of the tunnelling activities, expected in summer 2017.

It follows that the South Portal must be accessible by the end of fall 2016, via an access road from the mill to the portal (Segments A and B combined). It must be noted that, as of 2015, the access road from the mill to the portal was partially constructed. For this reason, two segments were defined:

- Segment A, which spans from the mill to the waste dump. AFW considers that this segment is adequate to be used as a pioneer road. However, it will require further construction at later stages of the project.
- Segment B which spans from the waste dump to the South Portal, and will need to be sufficiently constructed to access the South Portal by the end of fall 2016.

Therefore, the construction efforts are first concentrated on Segment B, between the South portal and the waste dump, while Segment A, from the waste dump to the mill, is planned later in summer/fall 2017 since its construction is less critical to the timely delivery of the project.

Due to time constraints and assumed crew size, only a partial buildout of Segment B will be completed in 2016. At this time, the corridor will be sufficiently complete to allow access to the South Portal. The remainder of this road will be completed in summer/fall 2017.

The construction of the South Portal follows during November and December 2016. Simultaneously to this, the North Portal pioneer road and preliminary sediment control structures will be implemented in the KLV. These will be required to initiate North Portal construction in spring 2017.

The tunnelling operations will be carried out throughout the winter and summer 2017, connecting the South and North Portals (Segment C) in August 2017. In order to achieve this, the North Portal will be completed by the same date, and is thus planned for completion in June and July 2017. The haul road from the North to Triple Declines Portal (Segment D), will be constructed during this same period.

The Triple Decline Portal will follow, and is expected to be completed during Fall 2017. Other earthwork structures, such as the laydowns and service roads are also scheduled during the excavation of the Triple Decline Portals

Finally, the construction of the concrete foundations of the conveyor is planned for 2018, where the foundations for the elevated sections of the conveyor will be poured and the pre-cast concrete sleepers will be installed. The duration of this operation was based on the duration of stringer installation, as evaluated by CDI, since this task's progress rate limits the rate at which sleepers can be installed.

The vent raise access road is excluded from the schedule since it will be constructed once mining operations have begun.

8.0 RISK AND CONTINGENCY

The design of the KUG Access Tunnel and surface infrastructure presented herein is based on several key assumptions regarding the topographical, geotechnical and geochemical nature of the alignment that introduce uncertainty and risk in the feasibility of the design. Some of the risks have been eliminated during the 2015 SI program, while other uncertainties will occur during construction of the project. The eliminated and existing risks are discussed and divided in the following sections: *Survey/Topography*, *Subsurface Conditions*, *Schedule*, and *Other* and are expected to be covered by the contingency amount shown in Table I-1 in Appendix I.

8.1 Survey and Topography

- The 2013 *Surface Infrastructure – Alternative Corridor Preliminary Design Report* had highlighted an uncertainty in the topography of the site due to lack of data. This introduced a possible cost increase to the project in the event of any discrepancies between the true existing ground elevation and the topographic survey. In the present study, this risk was reduced by performing a LiDAR survey of the Kemess site in June 2015. This survey improved the precision of the available topographic data, thereby improving the accuracy of the design modelling and cost estimates related to quantity takeoff for surface infrastructure. Although the risk related to survey and topography is greatly reduced by the increased precision of the survey data, there remains some uncertainty, estimated at +/- 5% of the total cut/fill volumes.

8.2 Subsurface Conditions

- The 2015 site investigation program reduced the subsurface conditions risk significantly with borehole and test pit data to provide details of the rock and soil types in the subject area, and material properties through subsequent laboratory testing. While the diamond drill coring identified the bedrock/soil interface at the coring locations, there is still an uncertainty of the interface at some areas where only blind-auger drilling to refusal was performed. There is uncertainty whether the auger rig reached refusal in bedrock or encountered boulders, and supervision during excavation cuts of these areas will be necessary to verify the bedrock/soil interface.

- The uncertainty of the bedrock geochemical properties was reduced by conducting laboratory testing on rock samples recovered during the diamond drilling process. The areas within the Access Tunnel and Portal footprints with NAG and PAG rock were identified, which would improve assumptions previously made in regards to geochemistry of the rock.

Despite the additional site investigation performed, latent undesirable geotechnical conditions still introduce a risk component to the project;

- The topsoil thickness, estimated at a uniform thickness of 0.3m for quantity evaluation purposes, could vary within the footprint of the various surface infrastructure, depending on local vegetation and topography. Because of this, overburden placement and excavation quantities could vary by +/- 5%.
- The Access Tunnel costs are also impacted by latent undesirable geotechnical conditions. For this reason, 25% contingency was applied to tunneling costs, rather than 15%, which was applied to other infrastructure, with the exception of the tunnel portals.

8.3 Schedule

The construction scheduling, presented in figure I-1 of Appendix I and described in section 0, introduces the largest component of risk related to project costing. As previously stated, the scheduling revolves around the construction of the Access Tunnel from January 2017 to August 2017. To allow construction of the Access Tunnel within this timeframe, the portals on either ends of the tunnel must be delivered in coordination with the start and finish of the tunneling operation.

Because of permitting and financing constraints proposed by AuRico, the construction works are scheduled to begin only in September 2016. The construction of the South Portal, required to begin the tunneling, is thus expected to end in December 2016, at a time when winter conditions will be experienced. Depending on the extent of winter climatic events, the South Portal construction cost and duration could increase significantly. Similarly, the North Portal, scheduled to be constructed from June 2017 to July 2017, could increase in cost if winter conditions persist longer than expected. For this reason, a contingency of 25% was applied to costs related to portal construction.

8.4 Other

- A terrain assessment was completed during the 2015 SI program in response to concerns of potential rockfall or avalanche hazards mentioned in the September 2013 design report. The assessment was conducted for the Access Tunnel, North and Triple Decline Portals footprints, which reduced the uncertainty of the terrain for the site area.

9.0 SUMMARY AND PATH FORWARD

The KUG Geotechnical Site Investigation program was conducted between June 29 to August 8, 2015 for the design of surface infrastructure and Access Tunnels. A total of 15 bedrock holes, 75 overburden holes and 49 test pits were completed within the site footprint. In general, the bedrock was encountered at shallower depths at the North and South Portals. Two main domain types were identified for the bedrock on the site; Astika Formation (consisting of mainly basalt) overlying Black Lake formation (consisting of mainly granodiorite).

Geochemistry testing was conducted on recovered bedrock samples, and Potentially Acid Generating (PAG) rock with NPR<2 were identified only within the Access Tunnel area in select boreholes. All other rock samples from the other areas of the KUG footprint were Non-PAG rock.

Hydraulic conductivity ranged from negligible within the bedrock along the Access Tunnel area to 6.6E-7 m/s at the Triple Decline Portal. The boreholes were dry upon completion. In general, the rock quality of the Asitka Formation ranged from very poor to excellent, with the majority of the runs having good RQD values. For the Black Lake Formation, the rock quality ranged from very poor to excellent, and generally had poor to good RQD values.

Analyses were completed for the rock and soil slopes based on data obtained from the 2015 KUG SI program. The portal slopes within the bedrock will be constructed to 75˚ from the horizontal. Soil slopes at the North and Triple Decline Portals will be developed at 60˚, with addition of reinforcement. The slopes at the South Portal will not require reinforcement, as these are designed to a much shallower angle.

The Access Tunnel and crown-pillar at the entrance at tunnel have been designed with recommended for support in order to achieve the minimum required factor of safety for long term stability.

The soil stratigraphy identified from the overburden holes and test pits were mainly coarse-grained native deposits (sandy gravel, gravelly sand and/or sand), with occasional cobbles and varying amounts of fines, or fill deposits of clay, sand and gravel mixture. Shallow foundations were recommended for the surface infrastructure constructed on soil foundation within the Kemess Lake Valley and along the Conveyor Corridor. Concrete sleepers were considered suitable for the on-grade conveyor section while slab on-grade foundation would be used for the transfer tower structure.

The total cost of the project is estimated to be $ 36.39M, which includes $ 6.6M of earthworks contractor indirect costs, $ 6.23M of contingency costs and $ 2.13M of QA/QC and detailed engineering costs. The contingency applied was typically 15%, with the exception of costs related to portal cuts, canopy and tunnel construction, where a 25% contingency was applied to account for complexity of work and possible winter conditions.

Before construction of the KUG Access Tunnel and surface infrastructure can commence, the feasibility report will be submitted to the B.C. Ministry of Mines for permitting and approval process, scheduled to be in Spring 2016. The preliminary schedule indicates that the project will occur over a period of 2.5 years. The schedule mainly revolves around the construction of the Access Tunnel, which is expected to occur from Winter 2016 to Summer 2017. It follows that the South Portal must be accessible for construction by the end of fall 2016, and that the North portal should be completed in summer 2017 to accommodate tunnelling operations. Construction of the Triple Decline Portal and installation of the conveyor will follow in Fall 2017 and Summer/Fall 2018, respectively. It must be noted that the proposed schedule is dependent upon submission and approval of the Environmental Assessment, as well as financing. If there are any significant delays with respect to either of these aspects, the proposed schedule will change as a consequence. In preparation for construction, the next recommended part of the program will be development of construction drawings and specifications.

10.0 LIMITATIONS & CLOSURE

Recommendations presented herein are based on a geotechnical evaluation of the findings of the site investigation noted. If conditions other than those reported are noted during subsequent phases of the project, Amec Foster Wheeler should be notified and be given the opportunity to review and revise the current recommendations, if necessary. Recommendations presented herein may not be valid if an adequate level of review or inspection is not provided during construction.

This report has been prepared for the exclusive use of AuRico Metals Inc. for specific application to the area within this report. Any use which a third party makes of this report, or any reliance on or decisions made based on it, are the responsibility of such third parties. Amec Foster Wheeler accepts no responsibility for damages, if any, suffered by any third party as a result of decisions made or actions based on this report. It has been prepared in accordance with generally accepted soil and foundation engineering practices. No other warranty, expressed or implied, is made.

Respectfully submitted,

Amec Foster Wheeler Environment & Infrastructure,
a Division of Amec Foster Wheeler Americas Limited

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REFERENCES

AMEC Earth & Environmental Limited (2011a). “Kemess Underground Project - East Dam Pre-feasibility Study Report”, May 24.

AMEC Earth & Environmental Limited (2011b). “Kemess Underground Project Feasibility Study - Geotechnical Investigations for Open Pit Tailings Storage, AMEC Scope and Budget Estimate”, August 30.

AMEC Environment & Infrastructure (2012). “Kemess Underground Project (KUG) Feasibility Study – Surface Infrastructure, Preliminary Design Report”, November 09.

AMEC Environment & Infrastructure (2013). “Kemess Underground Project (KUG) Feasibility Study – Surface Infrastructure – Alternative Corridor, Preliminary Design Report”, September 13.

Amec Foster Wheeler (2015). “Kemess Underground Project – 2015 Site Investigation and Feasibility Study Update, Confirmation of Amec Foster Wheeler Scope of Work”, June 02.

Amec Foster Wheeler (2016). “2015 Geotechnical Site Investigation Factual Report – Kemess Underground Project (KUG) Access Tunnel and Surface Infrastructure”, February.

ASTM A615 Standard Specification for Deformed and Plain Carbon-Steel Bars for Concrete Reinforcement.

Atmostpheric Environment Service (1975). “Annual Precipitation Canada”.

Barton, N.R., Lien, R. and Lunde. J, (1974). “Engineering Classification of Rock Masses for the Design of Tunnel Support, Rock Mech 6(4).” Pg. 189-239.

Barton, N., Løset, F., Lien, R. and Lunde, J, (1980). “Application of the Q-System in Design Decisions, In Subsurface Space (ed. M. Bergman).” Pg. 553-561.

Bieniawski, Z. T. (1976). “Rock Mass Classification in Rock Engineering. *In Exploration for rock engineering, proc. of the symp.* (ed. Z. T. Bieniawski) 1, 97-106”.

Bieniawski, Z. T. (1989). “Engineering rock mass classifications”.

Brady, T., Martin, L. and Pakalnis, R. (2009). “Empirical Approaches for Opening Design in Weak Rock Masses. *Transactions of the Institution of Mining and Metallurgy*”*,* Pg. 114: A13-A20.

British Columbia Ministry of Environment (BCMOE) (2015). “Assessing the Design, Size and Operation of Sediment Ponds Used in Mining, Version 1.0”, December.

British Columbia Ministry of Forests (BCMOF) (2002). “Forest Road Engineering Guidebook, 2nd Edition”, June.

British Columbia Ministry of Environment, Lands and Parks (1998). “British Columbia Streamflow Inventory – Instantaneous Peak Flow Isoline.”

Canadian Geotechnical Society (CGS) (2006). “Canadian Foundation Engineering Manual, 4th Edition”.

Church, M. (2014). “The Regulation of Peace River: A Case Study for River Management.”

Coulson, C.H. and Obedkoff, Q. (1998). “British Columbia Streamflow Inventory”, March.

Conveyor Dynamics, Inc. (CDI) (2015). “Design Report and +/- 10% Cost Estimate for the Overland Conveyor System in the Kemess Underground Project”, November 18.

Dywidag-Systems International (DSI). Dywidag Soil Nails Manual

FHWA. (2003). Geotechnical Engineering Circular No. 7. Soil Nail Walls. Report FHWAO-IF-03-017, U.S. Department of Transportation, Federal Highway Administration, Washington D.C.

Grimstad, E. and Barton, N. (1993). “Updating the Q-System for NMT. *Proc. int. symp. on sprayed concrete - modern use of wet mix sprayed concrete for underground support, Fagernes*, (eds Kompen, Opsahl and Berg)”.

Hatfield Consultants (2013). “Bathymetry Assessment of El Condor Creek Pond B.”

Hoek, E. and Brown, E.T. (1980). “Underground Excavations In Rock. The Institution of Mining and Metallurgy, London, England”, Pg. 358.

Hoek, E., Kaiser, P. K. and Bawden W. F. (1993). “Support of Underground excavation in Hard Rock. *3rd printing. Published: Rotterdam, Netherlands; Brookfield, VT, USA: A. A. Balkema*”.

Kaiser, P. K., McCreath, D. R. and Tannant, D. D. (1996). “Canadian Rockburst Support Handbook. Geomechanics Research Centre”.

Klohn Crippen. (2005). “Kemess Mine Environmental Impact Assessment Appendix 6 – Physical Environment Baseline Conditions”

Knight Piesold (1996). “Tailings Storage Facility Design Report, Ref No. 8162/3.”

Martin, C.D., Kaiser, P. K., and McCreath, D. R. (1999). "Hoek-Brown Parameters for Predicting Depth of Brittle Failure around Tunnels."

National Building Council of Canada (2010). "National Building Code of Canada."

Northgate Minerals – Kemess Mine (2010). "Kemess South Mine Reclaimation and Closure"

Oregon Department of Transportation. 2001. Rockfall Catchment Area Design Guide. FHWA-OR-RD-02-04m. 2001.

Ouchi, A., Pakalnis, R. and Brady, T. (2009). "Weak Rock Mass Span Design – Best Practices. ROCKENG09: In Proceedings of the 3rd CANUS Rock Mechanics Symposium", May.

Post-Tensioning Institute. (2001). Recommendations for Pre-stressed Rock and Soil Anchors.

Read, J., & Stacey, P. 2010. Guidelines for Open Pit Design. CRC Press. 2010.

SRK Consulting (Canada) Inc. (2013). "Feasibility Study for the Kemess Underground project, British Columbia, Canada. 2CNO25.001"

Stinette, Dan (2015). "RE: KUG Surface Facilities, Propane Tank Farm Design" E-mail, December 11

University of British Columbia (2014). "ClimateBC".

Weiland Terrain Sciences (2003). "Terrain Stability and Soil Erosion Potential in the Duncan Lake Corridor between Kemess Lake and Attycelley Creek", December.