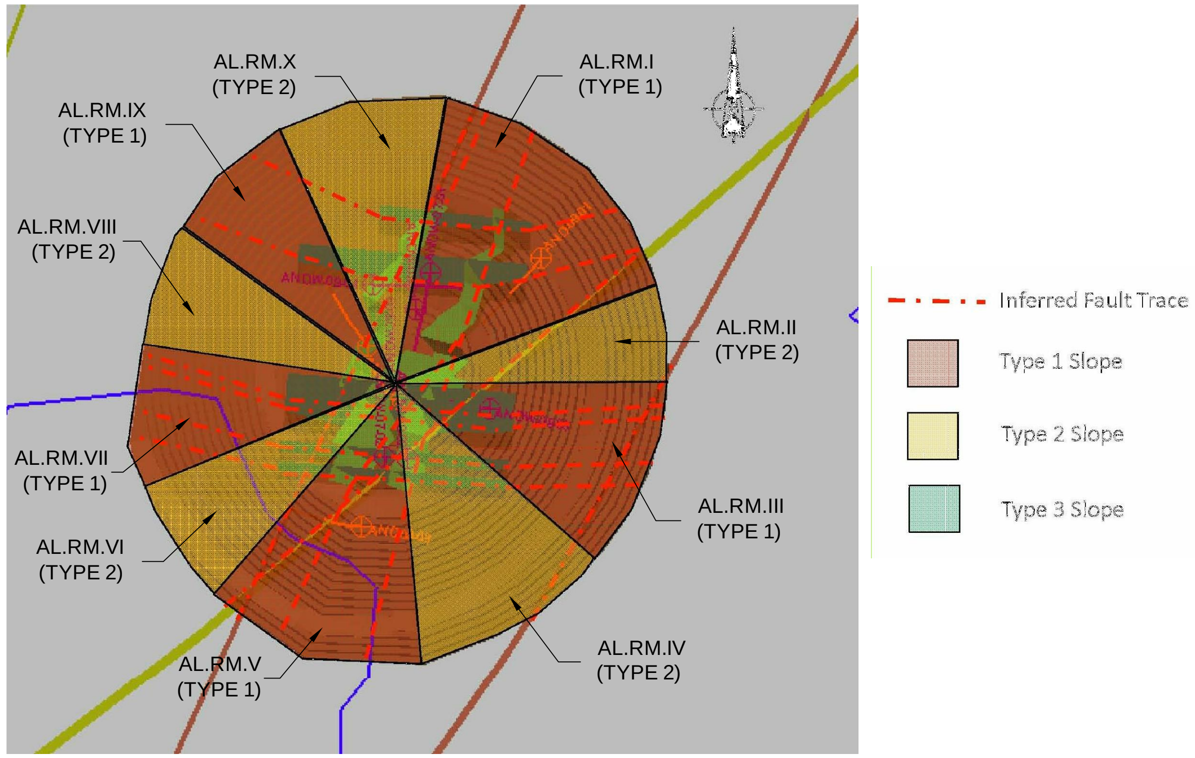
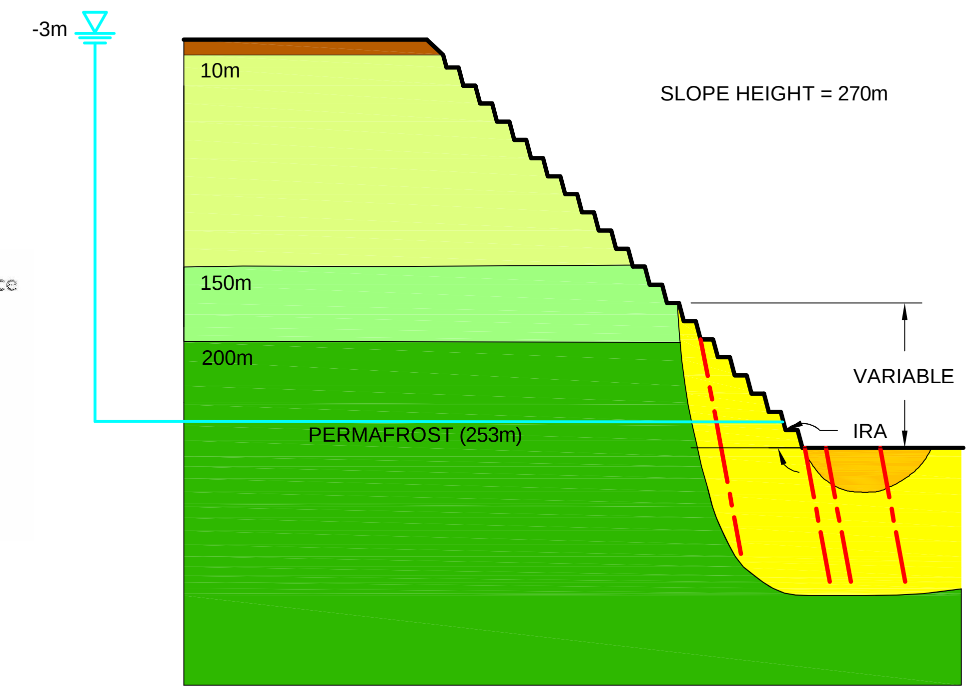
ANDREW LAKE PIT - INFERRED ROCK MASS DESIGN SECTORS



TYPE 2: LOWER SLOPE ALTERATION

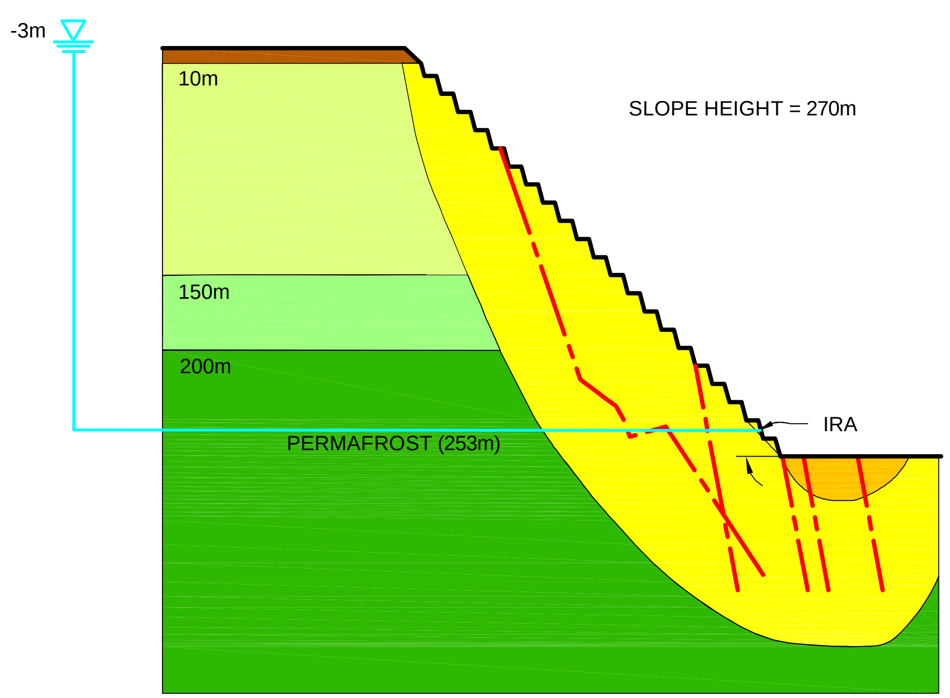


LOWER SLOPE AND FLOOR IS COMPRISED OF FAULTING AND ORE ZONE HALO ALTERATION.

ROCK MASS DOMAINS:

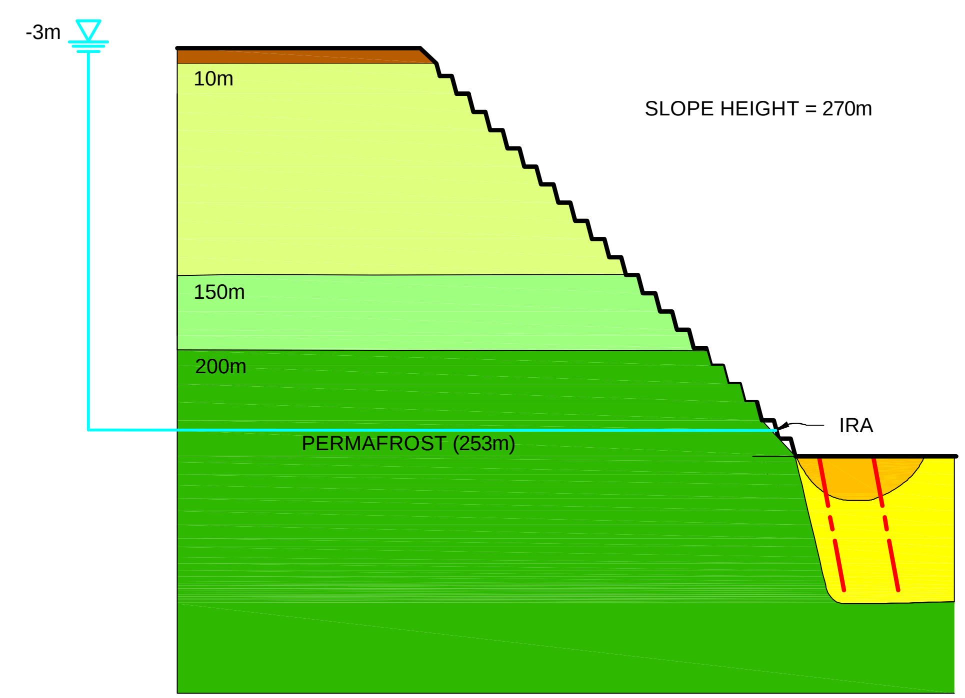
ZONE		RMR (1976)	UCS (MPa)
UPPER METASEDIMENTS		55 (fair)	30 (moderately strong)
GRANITOIDS		55 (fair)	50 (moderately strong to strong)
LOWER SEDIMENTS		65 (good)	65 (strong)
FAULT ZONE ALTERATION		50 (fair)	15 to 25 (weak)
ORE HALO ALTERATION		50 (fair)	10 (very weak to weak)

TYPE 1: FULL SLOPE ALTERATION



SLOPE AND FLOOR IS ENTIRELY COMPRISED OF FAULTING AND ORE ZONE HALO ALTERATION.

TYPE 3: NO SLOPE ALTERATION



SLOPE IS MAINLY UNALTERED. FLOOR IS FAULTED AND ALTERED

REFERENCES:

1. SURPAC GEOLOGY MODEL RECEIVED FROM AREVA (2009).

100 0 100

SCALE 1:5000 METRES

PROJECT

AREVA

AREVA RESOURCES INC.
2009 KIGGAVIK PRELIMINARY
PIT SLOPE DESIGN

TITLE

ANDREW LAKE GENERALIZED PIT SLOPE CONFIGURATIONS FOR STABILITY ANALYSES

Golder Associates
Saskatoon, Saskatchewan, Canada

PROJECT No.		09-1362-0613	FILE No. 0913620613AB0D1.dwg
DESIGN		11/16/2009	SCALE AS SHOWN REV. A
CAD	JS	12/9/2009	FIGURE
CHECK		12/9/2009	
REVIEW		12/9/2009	D-1

PLOT DATE: December 9, 2009
FILENAME: T:\Projects\2009\09-1362-0613 (AREVA, Kiggavik)\-AB-\0913620613AB0D1.dwg

The pit was assumed to be comprised of two types of slopes, with varying degrees of rock mass alteration. The altered rock slopes included zones of the rock mass in close proximity to interpreted faulting or ore zone halo alteration. The less altered rock zones assumed a generalized distribution of lithological domains (i.e., upper metasediments, lower metasediments, and granites) with depth. The slope configuration models, denoted Type 1, Type 2, and Type 3, are described as follows:

- ***Type 1 – Full Slope Alteration:*** Assumes the majority of the slope is altered and weakened due to the presence of faulting or ore zone halo alteration. For the Andrew Lake, Type 1 slopes have been assumed and assigned to pit walls where faulting and associated alteration halos, in particular where more than one fault in close proximity, are interpreted to control the material properties of the open pit wall.
- ***Type 2 – Lower Slope Alteration:*** Assumes the lower slope, being the lower one half to one quarter of the overall slope height (depending on wall inclination), is altered and weakened due to the presence of faulting or mineralization (ore) zone alteration halo. This includes fault zone exposures on the lower portions of pit walls, or interpreted ore zone alteration halos impacting that extend to the lower portions of the walls and to the open pit floors. The upper portions of the slope, away from the faulting or alteration halos are assumed to be relatively unaltered country rock of better strength and quality.
- ***Type 3 – No Slope Alteration:*** Assumes predominantly non-altered rock mass conditions throughout the full slope height. Includes zones of the pit wall away from faulting and mineralization. For the Andrew Lake pit, the pit floor can still be assumed altered and weaker due to faulting or due to alteration. These slopes can include the paleo-weathered metasediments.

The Andrew Lake pit has been divided into eight rock mass sectors as shown on Figure D1. The Andrew Lake pit is interpreted to be predominantly comprised of Type 1 or Type 2 rock mass conditions due to the numerous faults crossing the pit walls and floor, correlated to geotechnical information from 2007 to 2009 boreholes. The main fault trends at Andrew Lake strike north-south to northeast-southwest, or east-west, therefore the wall orientations on-strike with these trends have been interpreted to be to be mainly Type 1 slopes. The borehole AND09-01 is an example of a borehole assumed to entirely penetrate a Type 1, fault and/or alteration influenced slope. Similarly, the majority of the lower slopes at Andrew Lake (lower 100 m) appears to be predominantly within either fault or ore halo influenced alteration, based on cross-sections on which field estimated rock strength are plotted.

2.1.2 Material Parameters

The SLIDE stability model 2D geometries and geotechnical units are outlined on Figure D2 and the material properties used in the slope stability analyses are presented in Table D1. These parameters were developed from the results of the strength testing and rock mass classification work presented in the earlier appendices. Due to a lack of data, some of the parameters are estimates, derived from review of trends on of rock mass quality or strength with depth. Some interpretation was required for assessing the strengths and qualities for the fault altered material at the Andrew Lake pit with depth, as insufficient data was available to accurately characterize variability in strength with alteration through the various rock units.

The rock mass strength was estimated following the generalized Hoek-Brown failure criterion (Hoek et al. 2002). This criterion uses the rock mass Geological Strength Index (GSI) which is related to the Rock Mass Rating (RMR) (1976 version) to estimate the rock mass strength from its intact material strength (UCS). The Hoek-Brown material coefficient, m_i values, were estimated from the approximated ratio of compressive strength to the material's tensile strength, which, in the absence of other data, can be considered as a value slightly greater than the PLT to UCS correlation factor K. These inferred m_i values correspond to recommended values for moderately strong to strong, crystalline metasedimentary rocks (RocLab, Rocscience 2007). Brazilian tensile strength testing, which was not carried out as part of this assessment, would be required to confirm this assumption.

Table D1: Slope Stability Analyses - Rock Mass Parameters.

Rock Mass Units	GSI = RMR	UCS (MPa)	m_i	D	γ (kN/m^3)	c (MPa)	φ (˚)
ANDREW LAKE							
Overburden	-	-	-	-	21	0	35
Upper Metasediments	55	30	10	0.5 or 1	24	-	-
Granites	55	50	12	0.5 or 1	25	-	-
Lower Metasediments	65	65	12	0.5 or 1	25	-	-
Fault Altered – Upper Metasediments	50	15	10	0.5 or 1	23.5	-	-
Fault Altered – Granites	50	20	10	0.5 or 1	24	-	-
Fault Altered – Lower Metasediments	50	25	10	0.5 or 1	24	-	-
Ore Halo Alteration	50	10	10	0.5 or 1	23.5	-	-

*GSI = Geological Strength Index (Hoek-Brown), RMR = Rock Mass Rating (1976), UCS = unconfined compressive strength, m_i = Hoek-Brown material coefficient, γ = unit weight, D = Hoek-Brown disturbance factor, c = cohesion (Mohr-Coulomb), ϕ = angle of friction (Mohr-Coulomb).

In general, the Andrew Lake rock mass is comprised of fair to good quality, moderately strong to strong rock mass units. Type 1 and Type 2 slopes assume considerable zones of fault or mineralization alteration, reducing the rock strengths to very weak or weak.

A disturbance factor (D) is used in the Hoek-Brown criterion to downgrade the rock mass strength for considerations related to rock mass damage and disturbance such as blasting or stress relief due to excavation. This disturbance factor might also be considered for potential deterioration of the rock mass quality due to thawing and unravelling of the exposed frozen ground in warmer months. There is uncertainty estimating a relevant value of D for the rock mass comprising these pit slopes. At Andrew Lake, the already low to moderate rock strengths and fair rock mass qualities would not likely be deteriorated significantly further than those already estimated, therefore a minimal disturbance factor might be considered (D = 0.5). However, in the better quality and stronger rock units, large scale production blasting may act to reduce the rock mass strength significantly, which is modelled by assigning a higher disturbance factor (D = 1). Because of this uncertainty, the models were analysed with both disturbance factors in order to produce a range of results, from conservative to optimistic assumptions. For the Andrew Lake slopes, slope design is very sensitive to the disturbance factor. Slope performance observations and study of the effects of blasting will help reduce blast damage to the walls, with potential for slope steepening if successful. Should the damage factor to slopes be high, slope flattening may be required.

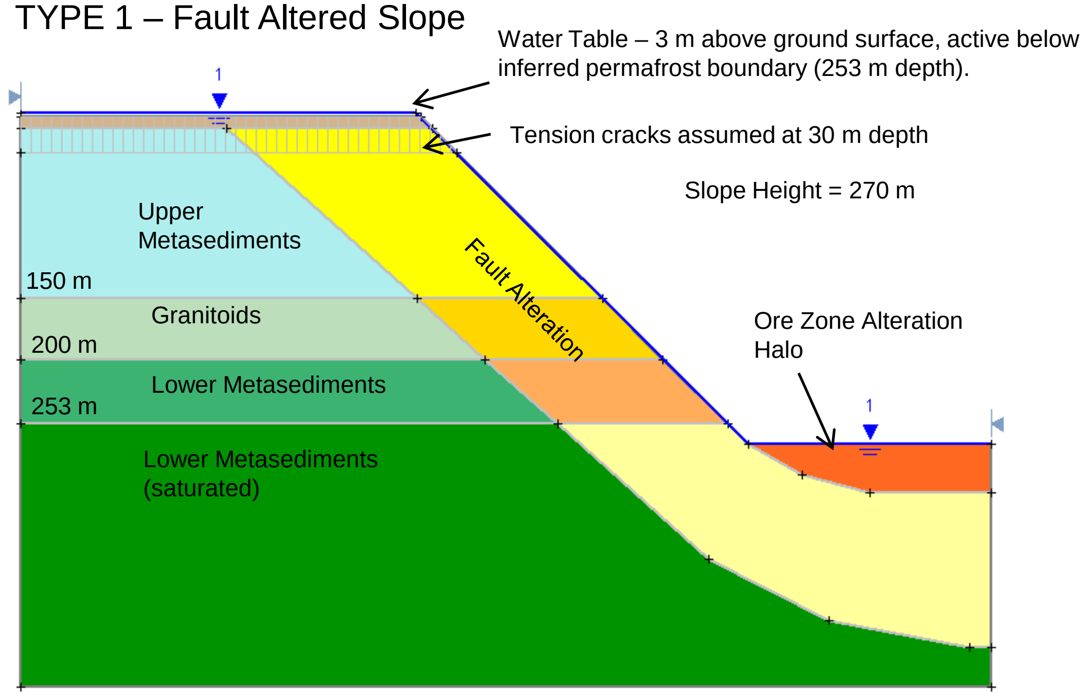
Material unit weights were estimated from the density of the rock samples. Overburden was assumed to be cohesionless with an angle of friction representative of a dense sandy material. However, the overburden units in these models carry no resistance to failure due to assumed tension cracking extending through the upper portions of rock into the overburden. The overburden essentially acts as load imparted to the rock mass.

ANDREW LAKE
SLOPE STABILITY ASSESSMENT
SLIDE MODEL CONFIGURATIONS

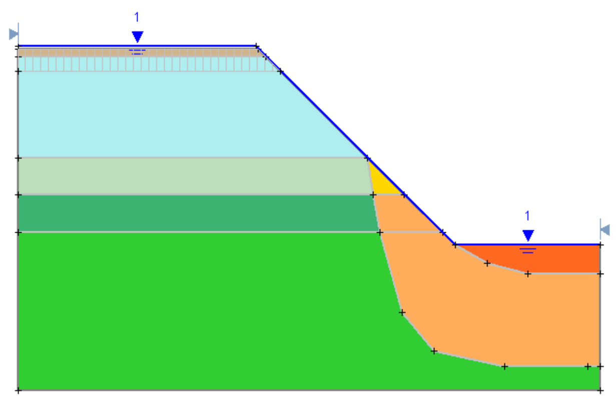
FIGURE D2

ANDREW LAKE

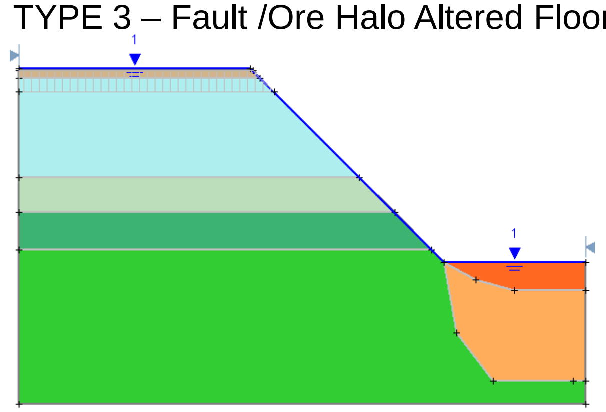
TYPE 1 – Fault Altered Slope



TYPE 2 – Fault Altered Lower Slope



TYPE 3 – Fault /Ore Halo Altered Floor



Notes: See text for material strength parameters information. Material strength parameters considered high blast damage (D=1) or low disturbance conditions (D=0.5). Andrew Lake models were assessed for dry and saturated conditions (below permafrost boundary at Anderw Lake) .

Date: November 2009

Project: 09-1362-0613

Golder Associates

Drawn: BC

Chkd:: MR

2.1.3 Tension Cracking and Water Conditions

Figure D2 shows the various slope model configurations, with tension cracking and piezometric surfaces.

Tension cracking was assumed to develop in the upper slope to depths of between 20 m and 30 m for the varying pit slope heights. The tension cracks carry no shear strength, and act as release planes from which failure surfaces would develop into the rock mass. The tension cracks were assumed to be unfilled, with no water pressure acting within the fractures.

Both wet and dry conditions were analysed for the Andrew Lake slope configurations. The “wet” conditions considered a piezometric surface along the face of the pit slope, and extending to above the ground surface to represent artesian pressure conditions which were measured at depth below the permafrost boundary during the geotechnical investigation. A differential pressure head of approximately 3 m above ground surface (mags) was measured at the Andrew Lake pit (Golder 2009).

In the wet models, the water pressures are assumed to act within the lower portions of the slopes only, either below or slightly above the permafrost boundary. For the Andrew Lake wet models, the zone of saturation was assumed to act below the base of the permafrost, at approximately 253 m depth or 17 m above the slope toe. The use of the water table in the Slide models might be considered conservative, as the full hydrostatic pressures acting from above ground surface were applied to the models. In reality, ground water depressurization through the drilling of drain holes would considerably reduce the water pressures acting in the slope.

2.2 ANDREW LAKE PIT SLOPE STABILITY

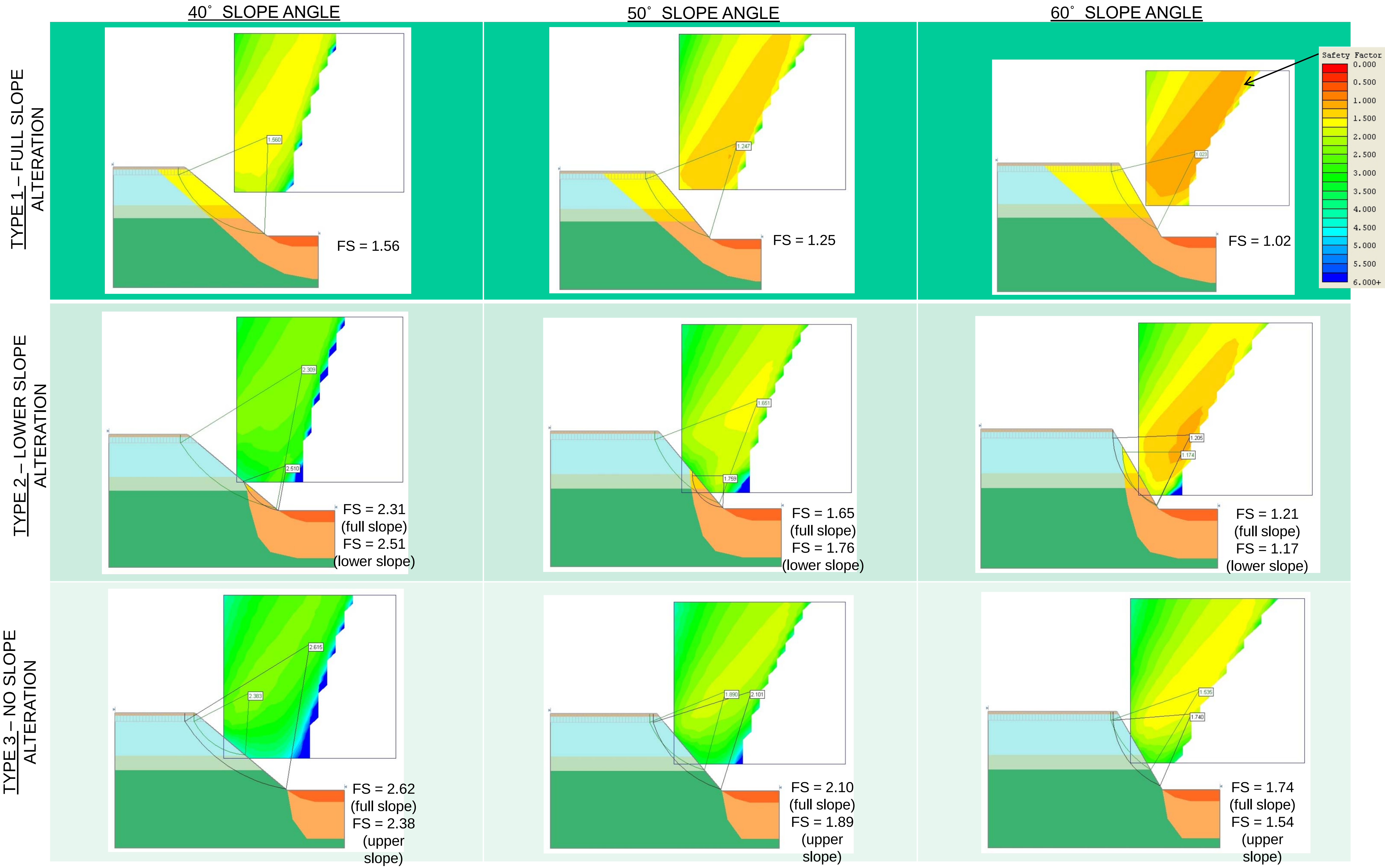
The Andrew Lake Type 1, Type 2, and Type 3 rock mass slope configurations were analysed with varying degrees of slope inclination, ranging from 35˚ to 60˚. Several screen captures from the Slide models are plotted on Figure D3 which illustrate the ranges of factor of safety (FS), and relative decrease in FS with increased inclination.

The FS versus the overall slope angle results for the Andrew Lake Type 1 slope are plotted on Figure D4. There is shown to be a considerable range of factor of safety depending on the assumptions of rock mass disturbance (D). The overall slope stability appears to be most sensitive to the assumptions on rock mass disturbance due to blasting or stress/strain relief. The water pressures acting along the slope toe are also shown to reduce the factor of safety, although not to a considerable extent.

*Slide results shown are dry, with damage factor, D = 0.5.

ANDREW LAKE PIT
SLOPE STABILITY ASSESSMENT
SLIDE ANALYSES RESULTS*

FIGURE D3



Date: November 2009

Project: 09-1362-0613

Golder Associates

Drawn: BC

Chkd:: MR

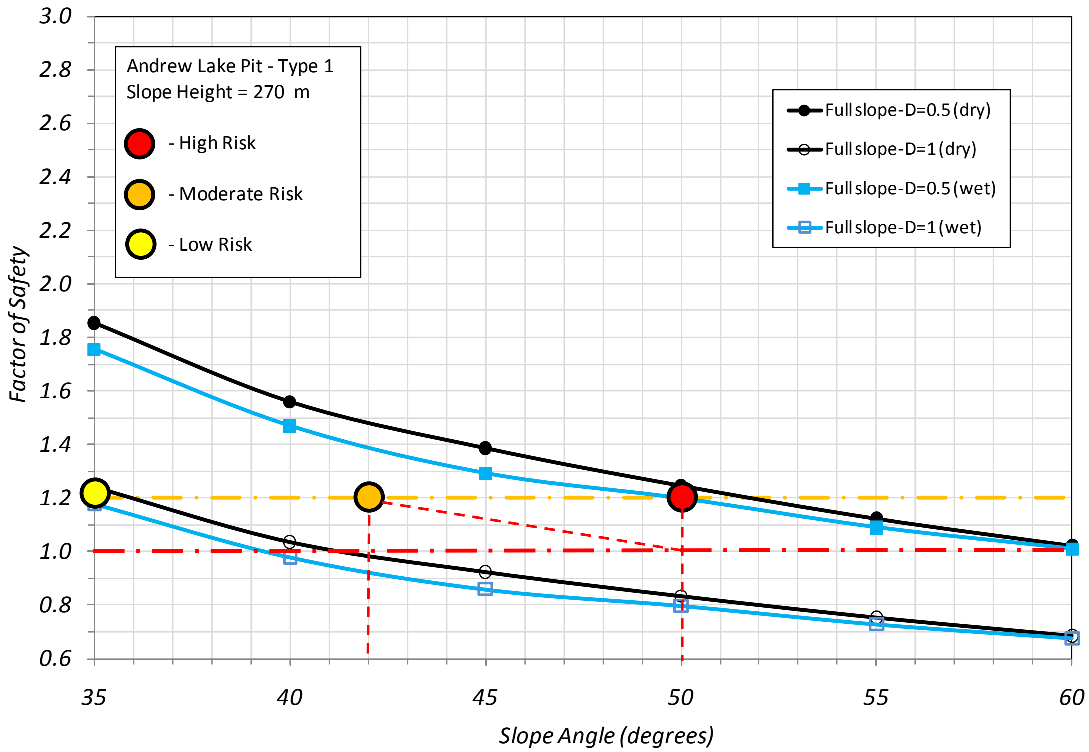


Figure D4: Andrew Lake Type 1 Slope - Slope Angle versus Factor of Safety for the Full Slope Altered Rock Mass Configuration

A 'High Risk', 'Moderate Risk', and 'Low Risk' slope angle have been highlighted on Figure D4 based on the relative FS and the design assumptions considered. Descriptions of these risk ratings are as follows:

- *High Risk* – slope angle corresponding to the minimum disturbance factor (D=0.5) with a safety factor of 1.2. Assumes that blasting and pit excavation will only marginally deteriorate the rock mass quality and strength. The high risk stable slope angle would be considered aggressive.
- *Moderate Risk* – slope angle corresponding to a moderate disturbance factor (D=0.5 to 1) with a safety factor of 1.2. Assumes good blasting practices will be employed to reduce potential damage to the rock mass.
- *Low Risk* – slope angle corresponding to a high disturbance factor (D=1) with a safety factor of 1.2. Assumes large scale blasting and excavation will result in a high degree of disturbance to the rock mass. The low risk stable slope angle would be considered conservative.

As shown in Figure D4, the slope angles range between 35° and 50°. A 35° slope angle is likely too conservative for the open pit, although a 50° slope angle would likely be too aggressive based on the assumed disturbance factor of D = 0.5. An achievable slope angle with the Type 1 rock mass conditions would likely be around 44°, and with the adoption of good blasting practices, a slope angle of closer to 47° might be achievable.

The Andrew Lake Type 2 model results are illustrated on Figure D3, and the results of the assessment for the various slope angles are plotted on Figure D5. The model results show the Type 2 slope configurations have two main tendencies of failure, full slope failure through the majority of the pit wall height, or lower slope failure through the altered rock mass zones in the lower one quarter to one half of the slope. Under dry conditions, the full slope failure mechanism shows the lower factor of safety, but under wet conditions with water pressures acting within the altered rock mass near the base of the slope, the lower slope failure mechanism has a reduced factor of safety. Generally, the stable slope angles show are highly sensitive to the assumed rock mass disturbance conditions, and less sensitive to the presence of water pressures acting near the base of the slope. High, moderate and low risk slope angles have been assessed, ranging between 44° and 59°. This is a considerable increase in angle compared to the Type 1 slope condition. An achievable overall slope angle with good blasting practices and drainage would likely be around 50° for the rock mass conditions considered. Since the lower slope would be at higher risk of failure due to the alteration of the rock as well as the possibility of water pressures, a reduced angle in the lower slope rock mass units should be considered.

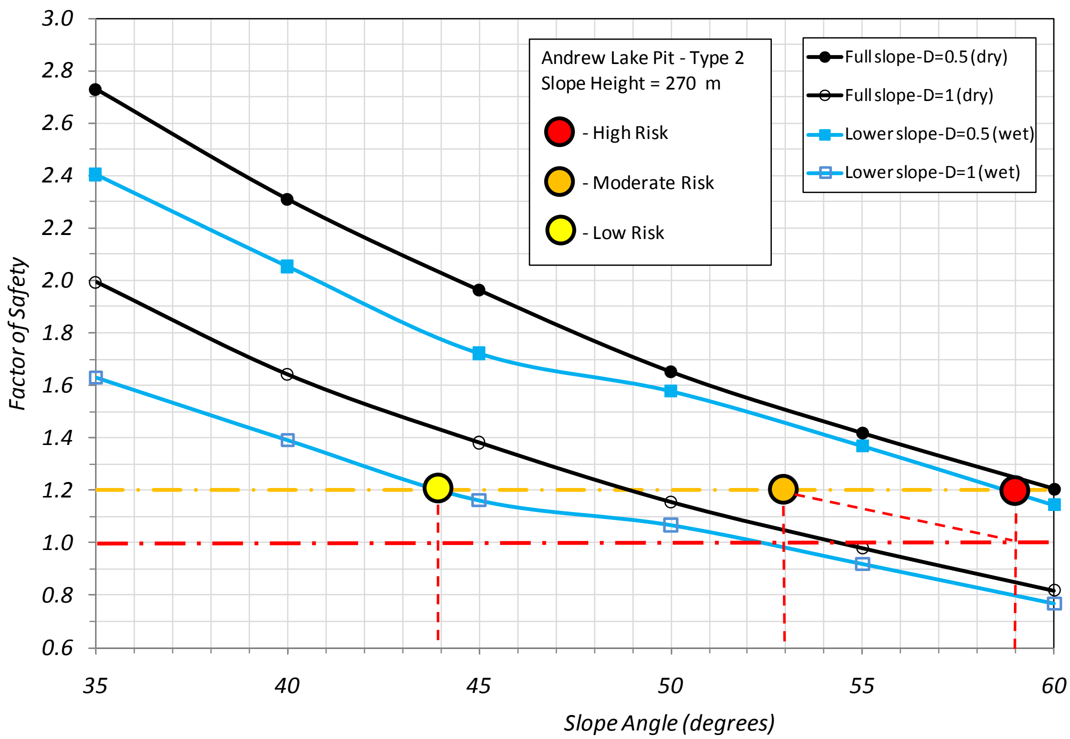


Figure D5: Andrew Lake Type 2 Slope - Slope Angle versus Factor of Safety for the Lower Slope Altered Rock Mass Configuration

The Andrew Lake Type 3 results are plotted on Figure D6. The results show a tendency for rock mass failure both through the full slope height, as well as locally in the upper wall slope, occurring in the relatively weaker upper metasediment rock units. Water pressures are shown to slightly lower the stable slope angle against full slope failure, but do not affect the stability in the upper slope away from the zones of water pressure. The FS results are considerably higher than the Type 1 and Type 2 slope conditions. An achievable slope angle of greater than 55° could be considered against full slope failure in Type 1 conditions; however the slope angle might be limited to 55° in the upper slope due to the relatively weaker rock mass units

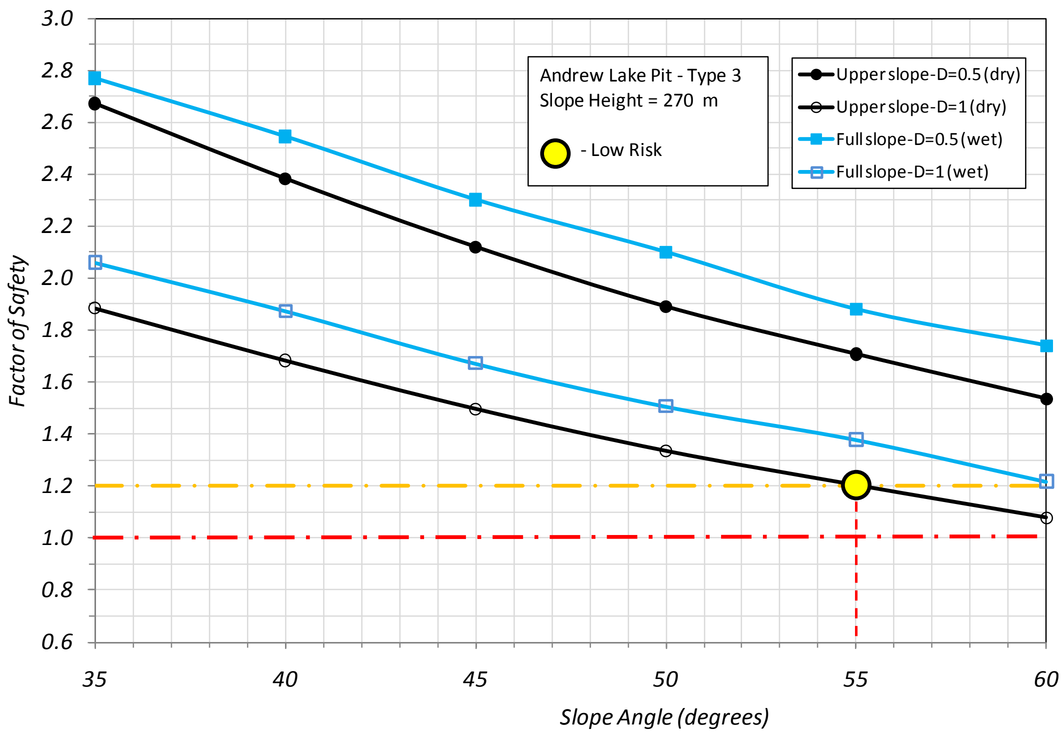


Figure D6: Andrew Lake Type 3 Slope - Slope Angle versus Factor of Safety for the Non-Altered Rock Mass Configuration

Based on the results, recommended slope angles for the Type 1, Type 2 and Type 3 Andrew Lake rock mass slope configurations are tabulated in Table D2. Where variable rock mass conditions are encountered in the pit wall, varying angles for the upper slope (0 m to 150 m vertical depth), lower slope (150 m to 270 m depth), and full slope (270 m depth) are considered. The perceived risk for the recommended slope angles is also given. In general, pit slope walls developed through the very weak to weak rock mass units associated with alteration should limit the slope angle to 44° due to the moderate to high sensitivity related to the inferred rock mass parameters. Pit slopes developed through the unaltered ground could achieve slope angles of up to 55°. Good blasting practices, adequate drainage, and ground support would help reduce the level of risk. Enhanced monitoring would also be required for moderate to high risk slopes.

Table D2: Andrew Lake - Recommended Maximum Slope Angles for Rock Mass Slope Stability, Full 270 m Slope Height.

Rock Mass Slope Configuration		Overall Slope Net of Upper and lower slope (0 m to 270 m)	Upper Slope (0 m to 150 m)	Lower Slope (150 m to 270 m)	Perceived Risk/Sensitivity of the design to variations in strength
Type 1	Full slope alteration	44˚	44˚	44˚	Moderate to high risk
Type 2	Lower slope alteration	50˚	55˚	44˚	Moderate risk
Type 3	No slope alteration	55˚	55˚	55˚	Low risk

3.0 FLOOR HEAVE STABILITY ANALYSES

3.1 METHODOLOGY

A simple analysis was carried out to assess the depth at which floor heave due to artesian water pressures would become an issue at the Andrew Lake site. These analyses follow the assumption that the presence of pressurized water might be encountered below the permafrost boundary. During mining of the open pit, the excavation of the rock mass will reduce the overburden pressure counteracting the water pressure at depth. If a critical depth is reached without the reduction in water pressure, the potential exists for floor heave and water infiltration into the base of the pit. This could present considerable problems for mine operations, as well as potentially reducing the stability of the pit walls. The floor heave analyses follow previous work conducted by Golder (1989), except that both self weight resistance to floor heave, as well as rock mass failure were both considered.

The depth of the permafrost boundary, and assumed water pressure conditions used in the analyses are summarised in Table D3. This data was cited from the geotechnical investigation factual report (Golder 2009). An assumed value was used in the analyses where a range of values was recorded.

Table D3: Permafrost depths and hydraulic heads used in the floor heave analyses.

Site	Depth of Permafrost – Measured	Hydraulic Head above ground surface
Andrew Lake	253 m	3.5 m

Rock mass strength parameters and density were required to assess the rock mass pressure and strength for resistance to floor heave. The rock mass in the floor of the planned open pits is inferred to be predominantly altered and faulted, therefore the weakest material parameters were assumed in the analyses (Table D4). A disturbance factor (D) of 0.5 and 1 was also considered due to potential reduction in rock mass strength related to blasting and stress relief. This is likely a conservative estimation, particularly when considering the rock mass at considerable depth away from the pit floor.

Table D4: Floor Heave Analyses - Rock mass parameters.

Rock Type	GSI = RMR	UCS (MPa)	m_i	D	ρ (kg/m^3)
Ore Halo Alteration	50	10	10	0.5 or 1	2,450

*GSI = Geological Strength Index (Hoek-Brown), RMR = Rock Mass Rating (1976), UCS = unconfined compressive strength, m_i = Hoek-Brown material coefficient, ρ = bulk density, D = Hoek-Brown disturbance factor

The general layout of the pit and hydraulic/permafrost considerations is shown on Figure D7. The water pressure (Pw) acting at depth below the permafrost boundary is calculated from the depth of the permafrost (Dp) and hydraulic head above ground surface (Hw). The rock pressure (Pr) resisting the potential floor heave and water infiltration are assessed from the depth of the rock from the floor to the permafrost boundary (Dr). The rock pressure considered both the weight of the rock mass only, as well as the weight and strength components of the rock mass. The strength component was calculated from the rock mass shear strength (τ_{rm}), derived from the Hoek-Brown criterion. A confining pressure of two-thirds of the depth of the rock (2/3Dr) was used in calculating the rock mass shear strength. A pressure differential (dP=Pr-Pw), being rock pressure minus the water pressure is calculated for varying pit slope depths (Ho). As the depth of the pit increases, the pressure differential decreases. When the water pressure exceeds the rock pressure (negative differential pressure), there exists the risk for floor heave and water infiltration.

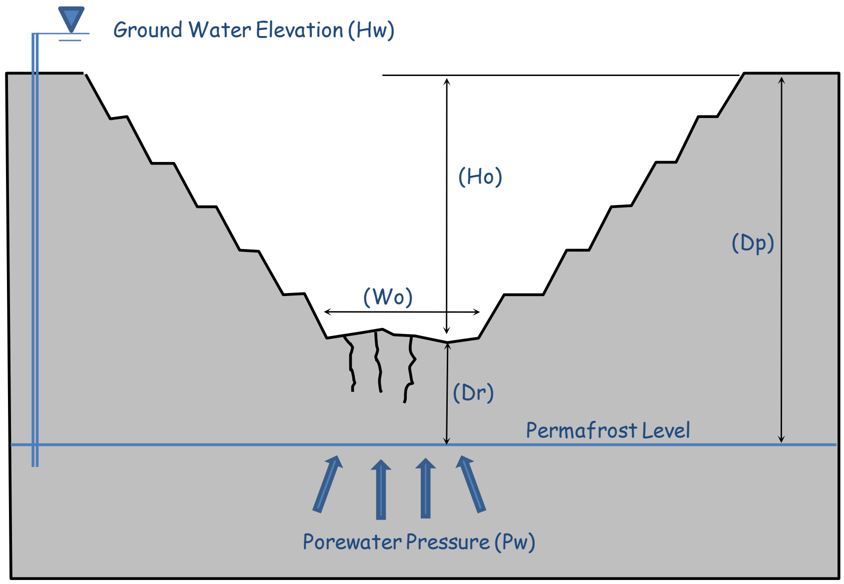


Figure D7: General Representation of the Components Used in the Floor Heave Analysis

3.2 FLOOR HEAVE ANALYSIS RESULTS

The floor heave analysis results for Andrew Lake, plotting the differential pressure versus pit depth are shown on Figure D8. The results compare dP calculated from the mass of the rock only, as well as dP considering a rock mass shear strength component for a disturbance factor D = 0.5 and 1. Negative differential pressures, related to unstable conditions, are shown to occur at between 145 m and 185 m. A pit depth of 145 m is recommended for consideration of alleviation of floor water pressures, which corresponds to approximately one half of the planned pit depth.

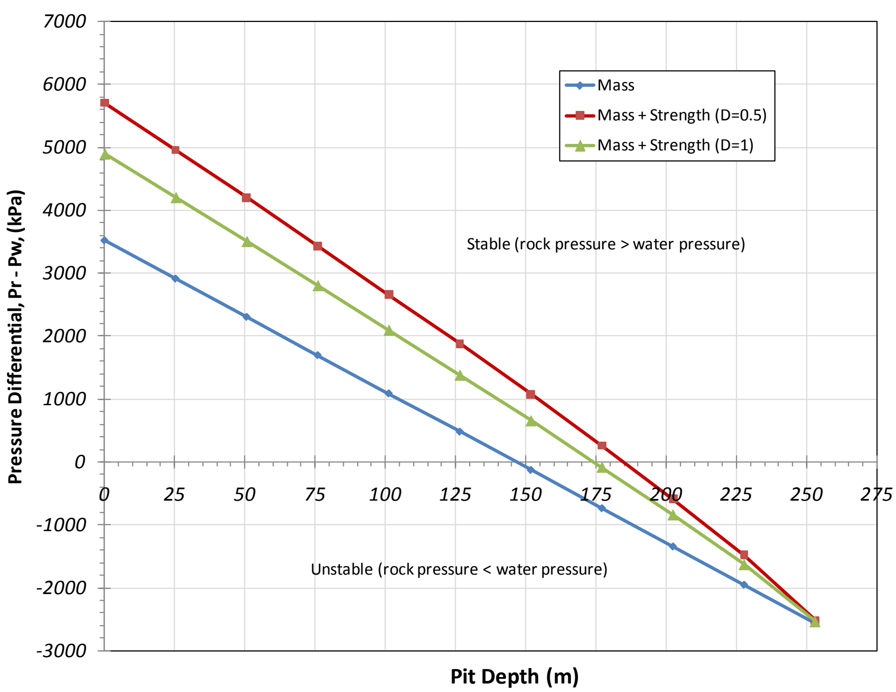


Figure D8: Andrew Lake - Floor Heave Analysis Results Showing Differential Pressure versus Pit Depth

The recommended critical depths of mining for floor heave drainage considerations are given on Table D5. These depths represent the rock mass components of resisting pressure only. The strength components of the rock mass are assumed to be an additional factor of safety. If floor drainage systems are proven to be effective at reducing the water pressures below the permafrost boundary, full depth of mining would be achievable without risk of floor heave.

Table D5: Recommended critical depths at which remedial measures such as vertical pressure relief drains may be required to prevent floor heave – self weight and rock mass failure.

Site	Planned Depth of Pit (m)	Expected Depth to Permafrost (m)	Critical Depth[(a)] (m) – Self Weight	Critical Depth[1] (m) – Rock Mass Failure
Andrew Lake	270 m	250	145 m	175 m

a = Critical depth of the pit floor at which floor pore water pressure reductions should start assuming self weight resistance only. Consideration should be given to establishing a geotechnical bench at this depth for depressurization.

4.0 REFERENCES

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Hoek, E., Carranza-Torres, C., Corkum, B. 2002. Hoek-Brown Failure Criterion – 2002 Edition. www.rocscience.com

RocLab v.1031 (www.rocscience.com)

n:\active\2009\1362\09-1362-0613 areva kiggavik geotechnical baker lake nunavut\pit slope report\final reports - february 2011\andrew lake\app d\app 11 feb 18 andrew lake appendix d - rock mass stability final_eam.docx

APPENDIX E

Kinematic Analysis

Table of Contents

TABLES

FIGURES

1.0 INTRODUCTION

The slopes in the Andrew Lake pit will be susceptible to kinematic structurally controlled failure. The kinematics were analyzed using the structural data, outlined in Appendix C. The methodology used to analyze the potential kinematic controls was deterministic in nature, and results are presented below.

2.0 STRUCTURALLY CONTROLLED FAILURE MECHANISMS

Structurally controlled failure in rock occurs as the result of movement along pre-existing geological discontinuities. The three basic mechanisms of structurally controlled failure in rock slopes are planar failures, wedge failures, and toppling failures, as described below.

A **planar failure** may occur when a geological discontinuity dips out of a rock slope at an angle that is shallower than the inclination of the slope and steeper than the effective angle of friction on the discontinuity. Planar failures will generally only develop to a significant extent if the strike of the geologic discontinuity is within $\pm 20°$ of the strike of the rock slope. In some cases, this range was expanded in the analyses to $\pm 30°$ to account for variability in slope dip direction for each design sector.

Wedge failure may occur when two or more geological discontinuities intersect to form an unstable wedge. In order for a wedge to fail, the line of intersection of the wedge must dip out of the slope at an inclination that is shallower than the inclination of the slope face, but steeper than the effective angle of friction along the discontinuities. Wedge failures will only develop to a significant extent if the azimuth of the line of intersection is within $\pm 45°$ of the dip direction of the slope face.

Toppling failure may develop when a rock mass contains multiple, parallel, steeply dipping continuous geologic structures, that strike nearly parallel to the strike of the face of the rock slope. Toppling failure will generally only develop when the strike of the structures is within $\pm 10°$ of the azimuth of the slope face. Kinematically, the potential for toppling failure is determined by the spacing (separation), inclination and continuity of the toppling blocks and the slope angle. Wide spacing and/or discontinuous structures will mitigate the potential for toppling. At a bench scale, this failure mechanism is controlled by berm width and/or the inclusion of mid-slope catch berms, both which improve stability by reducing the effective length of the toppling blocks.

All structurally controlled failure modes are aggravated by water pressures within the slope, particularly toppling failures. Water pressure was not included in the kinematic analysis, as dry conditions were assumed in all pit slopes. It will be important to monitor the groundwater elevations at each deposit and, where required, install long horizontal drains.

The magnitude and frequency of structurally controlled failures are directly related to the continuity and spacing of the structures along which sliding can occur. Rock mass structures that exhibit limited continuity, such as joints, may result in small bench scale failures that are rarely of consequence to overall slope stability but may adversely affect access ramps or equipment installations. Conversely, larger scale failures can occur along continuous, through-going structures, such as faults. It is, therefore, these more continuous structures that are of primary concern.

2.1 Design Sectors

Figure E2 present the kinematic design sectors for Andrew Lake. At Andrew Lake, kinematic design sectors were based on the proposed pit geometry. Because the pit geometry is essentially circular, the Andrew Lake pit walls were divided into eight different kinematic design sectors, with division of the sectors every 45°.

2.2 Slope Design Definitions

A pit slope has three major components: bench configuration, inter-ramp slope and overall slope, as illustrated on Figure E1. The bench configuration is defined by vertical bench separation (or bench height), catch berm width (or berm width) and bench face angle (or batter). The inter-ramp slope is formed by a series of uninterrupted benches and the overall slope is formed by a series of inter-ramp slopes separated by haul roads.

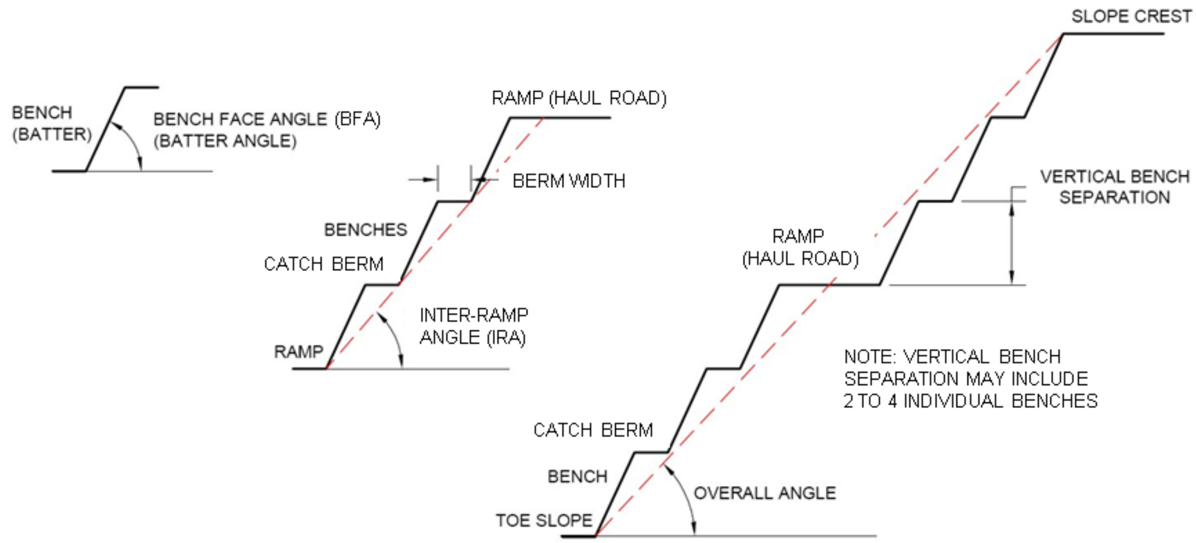
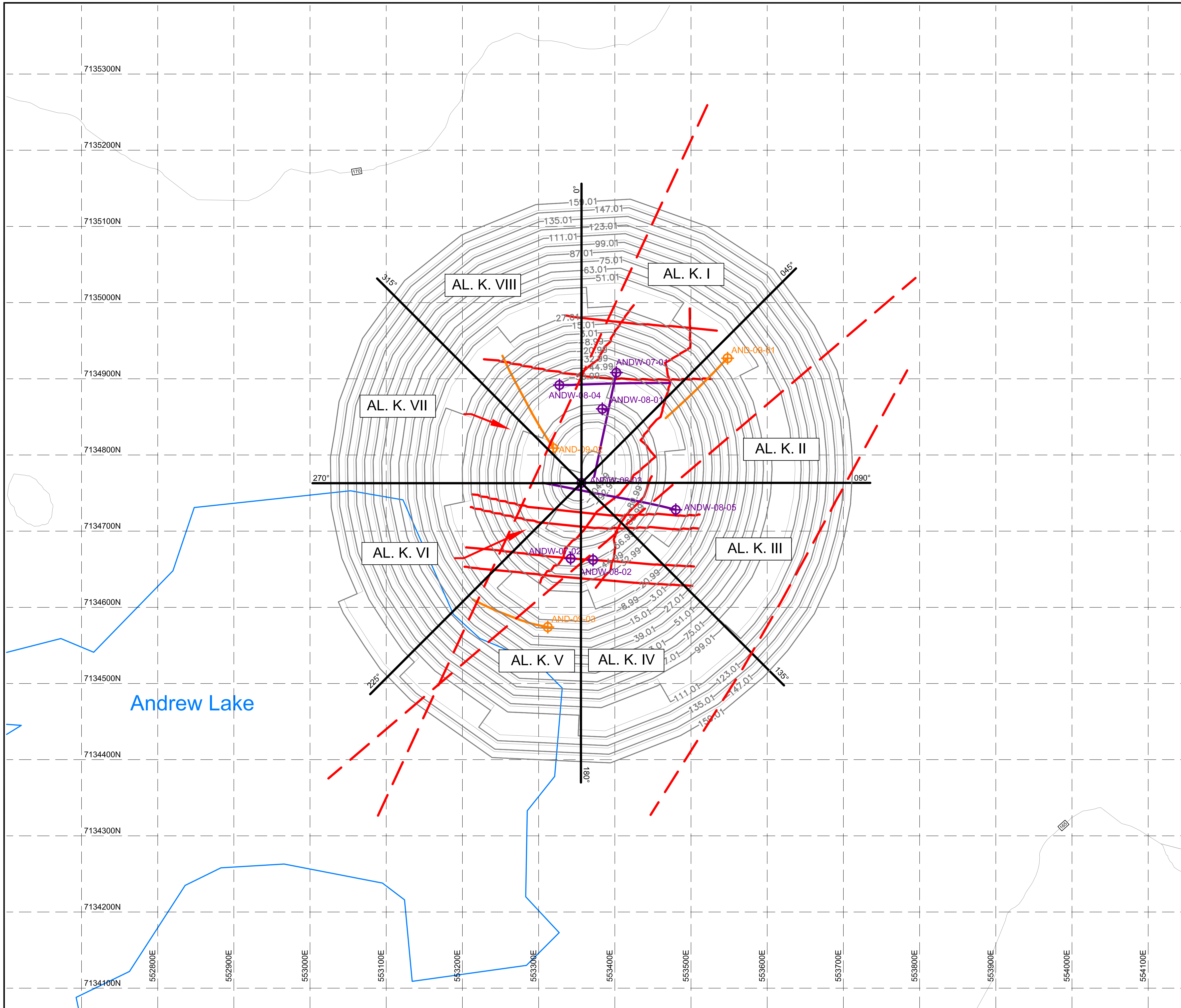


Figure E1: Schematic Representation of Bench Face Angle (BFA) and Inter-ramp Angle (IRA)



LEGEND:

ANDW-08-01 DRILLHOLES FROM 2007 & 2008 INVESTIGATIONS (SRK)

AND-09-01 DRILLHOLES FROM 2009 INVESTIGATIONS (GOLDER)

REGIONAL FAULT TREND (REF. 1)

INFERRED FAULT INTERSECTION ON PIT WALL (REF. 2)

REFERENCES:

1. BASED ON REGIONAL GEOLOGY DRAWINGS (MAIN TEXT FIGURES 4 & 5).
2. AREVA GEOLOGICAL MODEL (2009).

100 0 100
SCALE 1:5000 METRES

REV	DATE	DES	REVISION DESCRIPTION	CAD	CHK	RVW

PROJECT

AREVA

AREVA RESOURCES CANADA INC.
KIGGAVIK PROJECT

TITLE

KINEMATIC DESIGN SECTORS
ANDREW LAKE

Golder Associates
Saskatoon, Saskatchewan, Canada

PROJECT No.	09-1362-0613	FILE No. 0913620613AB0E2.dwg	
DESIGN		12/8/2009	SCALE AS SHOWN REV. A
CAD	JS	12/22/2009	FIGURE
CHECK	EAM	12/22/2009	E2
REVIEW	MR	12/22/2009	

PLOT DATE: February 25, 2011
FILENAME: G:\2009\1362\09-1362-0613 Areva Kiggavik Golder Internal Field Manual\CAD\from Mississauga\-AB-\0913620613AB0E2.dwg

2.3 Kinematic Assessment

2.3.1 Rock Mass Fabrics

The structural rock mass fabric for Andrew Lake is shown on Figure E3. This figure presents the discontinuity sets that will be used in the kinematic analysis. The selection of discontinuity sets for the kinematic analyses are discussed in Appendix C.

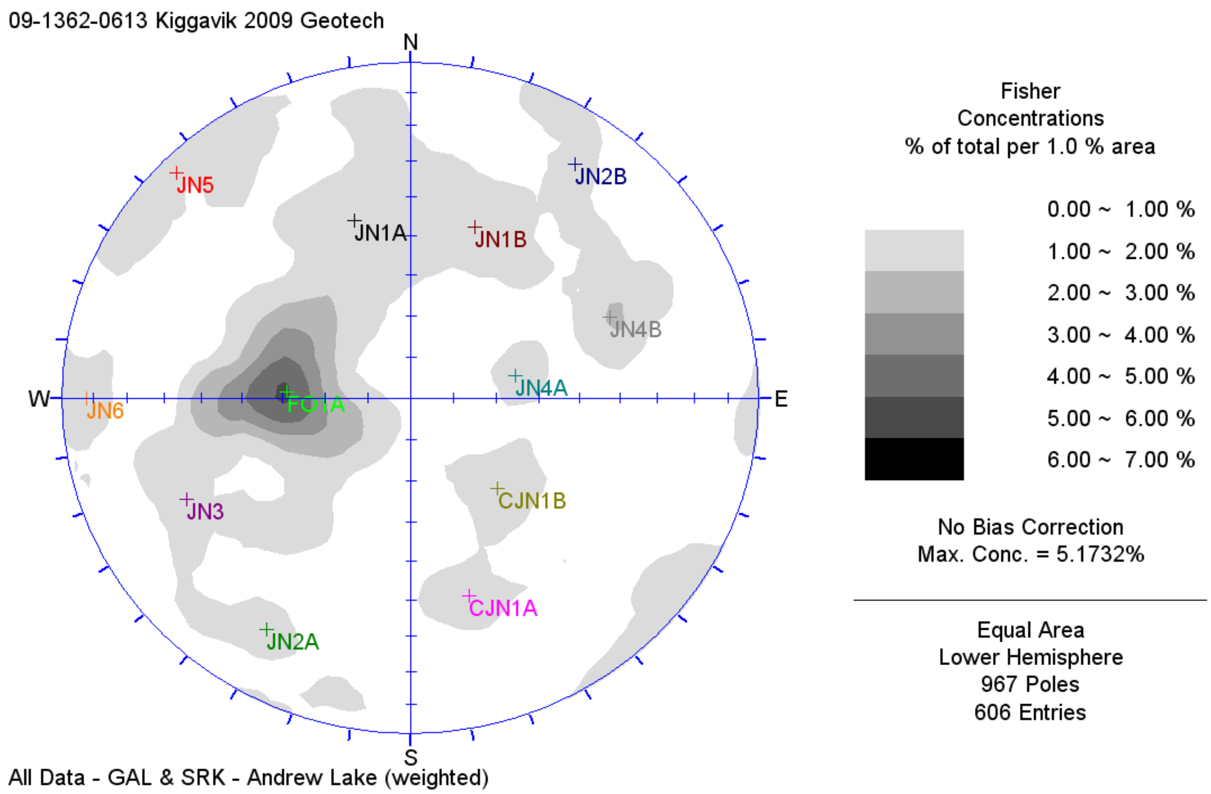


Figure E3: Andrew Lake - All Validated Oriented Core Data (contoured) with Selected Discontinuity Sets

It is recognized that one of the limitations of core orientation is the interpretation of continuity and frequency for a given discontinuity set. Large scale characteristics can only reasonably be assessed from actual mining exposures. For example, when a discontinuity set is very continuous but widely spaced, it would show a low concentration of poles and be classified as minor set in the stereonet, whereas a closely spaced, but discontinuous set would show a high concentration of poles and be classified as a major set in the stereographic projections, based on the frequency of data measurements. In addition, for identically spaced discontinuities, the smaller the angle at which the structures intersect the borehole axis, the fewer the number of structures that will be encountered. An effect of this condition is that boreholes have a “blind zone”, such that structures that are approximately parallel to the borehole are rarely encountered. Engineering judgement is therefore required for selecting the sets that would be considered continuous with the potential to affect multiple benches (i.e., be more than 20 m to 30 m long), for the application of the kinematic analysis. For this judgment, the following information is used:

- regional and local geology;
- interpretation of orientation of the mineralized zones;
- occurrence and the orientation of the major geological structures, such as fault, shear zones and dykes;
- outcrop exposures;
- pole concentrations in the stereographic projections; and
- type of discontinuity (e.g., vein, foliation, bedding, etc.).

At Andrew Lake, all identified sets were considered to be major due to the fact that the data set was relatively small. Therefore, all the sets at Andrew Lake were considered to be continuous across the Andrew Lake pit footprint for the kinematic analysis.

Similar to the discussion of structural domains in Appendix C, the determination of the continuity of the sets identified at Andrew Lake was based on a relatively small data set. This should be reassessed as more orientation data for each deposit becomes available.

2.3.2 Shear Strength

Direct shear testing was not conducted for Andrew Lake; therefore, discontinuity shear strength properties were estimated based on core log data. Friction angles at Andrew Lake were estimated based on the logged Joint Roughness Number (Jr) and Joint Alteration Number (Ja) parameters for each discontinuity. Jr and Ja are numerical values used in Barton's Q rock mass classification system, and they attempt to describe the friction and strength conditions along the discontinuity face. Jr, and Ja are described in detail in the geotechnical data report prepared by Golder entitled "2009 Kiggavik Geotechnical and Hydrogeological Investigation Data Report".

Two methods were used to estimate the friction angle for each discontinuity set. The ratio of Jr/Ja developed by Barton et al. (1974) for the Q system attempts to quantify the strength of the discontinuity surface. A rough estimate of the peak discontinuity friction angle can be made using the $\tan^{-1}$ (Jr/Ja). The second method utilized correlations developed between the Ja parameter and the residual friction angle, as presented by Barton and Grimstad (1994). Generally, the two methods were comparable to each other, however, estimating the friction angle based on $\tan^{-1}$ (Jr/Ja) tended to overestimate the friction angle when the discontinuity surface was clean or rough. If there was a discrepancy between the two methods used to estimate the friction angle, the residual friction angle estimated based on Barton and Grimstad (1994) was used, as it was the more conservative value.

For the current kinematic analyses under dry slope conditions, a shear strength along the discontinuity sets was assumed to be represented by zero cohesion and friction angles varying from ϕ = 30° to 35°, as summarized on Table E1. Given that the cohesion component can be reduced or eliminated by freeze-thaw, dilation related to blasting or stress relief, or other factors, the assumption of zero cohesion along discontinuities is reasonable. The friction angle can be considered as a conservative value for some of the discontinuities with clean and slightly altered surfaces, however, it is considered representative of the relatively high frequency of coated joints (predominantly calcite and quartz, with some chlorite and clay coatings).

It must be recognized that for faults and shear zones, the friction angle is likely to be lower than 35° due to soft material infilling on these features. It is possible in these zones that the shear strength would typically vary from 20° to 25°. Therefore, it is important to properly locate the known faults and shear zones on the geological model when designing the pit shell, particularly for the location of the access ramps.

Table E1: Andrew Lake Kinematic Sets Showing Friction Angle and Spacing

Set #	Dip	Dip Dir	Φ(˚)	Spacing (m)
FO1A	29	93	35	0.77
JN1A	46	163	35	0.71
JN1B	45	200	35	
JN2A	69	31	30	0.62
JN2B	73	214	35	
JN3	60	65	35	0.85
JN4A	25	257	30	0.89
JN4B	52	247	35	0.54
JN5	84	135	35	0.5
CJN1A	51	344	30	0.67
CJN1B	30	317	35	0.43
JN6	82	90	35	0.66

Dip Dir = Dip Direction; Φ = friction angle. Spacing taken as average for discontinuity sets indicated. Actual spacing may vary significantly.

2.3.3 Set Spacing

As shown in Table E1, the discontinuity spacing for each set was estimated from the oriented core data. This was done by identifying features that were part of the same discontinuity set, based on the stereonet analysis from each borehole. These sets were then grouped, and the average spacing between them was calculated. It should be noted that the actual spacing between discontinuities may vary significantly from the average spacing presented, as there are a number of limitations and biases associated with this approach. This was especially true for Andrew Lake where a significant proportion of this data was discarded due to quality control.

2.3.4 Kinematic Design Criteria

The following criteria have been used to define the potential level for planar, wedge and toppling modes of failure, based on the major and minor sets identified at Andrew Lake.

2.3.4.1 Planar Failure

Major Potential – When involving a major set, as it is assumed to be continuous and could impact both the bench face (BFA) and inter-ramp (IRA) angles, with potential to affect multiple benches.

Minor Potential – When involving a minor set, as it is considered to be discontinuous compared to the bench height.

A planar failure is considered if the following conditions are met:

- the plane on which sliding occurs must strike within approximately ±30˚ to the slope face;
- the dip of the sliding plane is less than the dip of the slope face (daylights on the slope face); and
- the dip of the sliding plane is greater than the angle of friction of this plane.

2.3.4.2 Wedge Failure

Factors of safety (FS) were calculated for the potential wedges that could form by the intersection of any two discontinuity sets. A wedge would be considered stable for a FS greater than 1.2. A wedge with a FS less than 1.2 would require additional consideration (flatten the bench face angle or widening the benches, depending upon whether the wedge is created by major or minor fabric as discussed below). The FS for wedges was calculated using a deterministic limiting equilibrium approach detailed in Appendix 3 of Rock Slope Engineering 4th Edition (Wyllie 2007). In the deterministic approach the major and minor peak set orientations established through the structural fabric analysis represented the orientation of the failure surfaces. The deterministic approach considered:

- shear strength represented by cohesion and angle of friction (ϕ), as presented in Table E1, was used for the analyses and served as a screening process to identify the critical wedges (i.e., those that showed FS $\leq$ 1.2). This screening was important to define the potential level for planar, wedge or toppling failures, as described below, and
- dry conditions without excess pore water pressure acting within the discontinuity surface.

The following are the potential levels used in the kinematic analysis of wedges:

Major Potential – When the wedge has a FS less than 1.2 and is formed by the intersection of two major sets (i.e., formed by major rock mass fabric), and at least one of them is considered to be very persistent (e.g., foliation or parallel to fault sets). It is considered that it can impact the stability of both the BFA and IRA. In general, consideration is given to flatten the BFA and IRA.

Moderate Potential – When the wedge is formed by the intersection of a major and a minor set, it is considered that it would locally impact the BFA stability and could also impact the IRA. If the wedge is sliding along the plane of the major set, then the IRA should, as much as practically possible, be within the plunge of this wedge. Subsequent structural mapping of the exposed bench faces during the initial production stage should confirm if the minor set is discontinuous as assumed in the analysis.

Minor Potential – When the wedge is formed by the intersection of two minor sets, which are considered to be discontinuous, then it may only locally affect the bench face (or batter) stability and it is considered that the wedge will be retained by the catch-berm. Consideration is given to widening the benches, particularly, if a steeper BFA is used.

2.3.4.3 Toppling

The following was considered in assessing toppling potential for the kinematic design sectors:

Major Potential – When it involves a major steeply dipping set and a shallow dipping set, which would facilitate toppling. In addition, the major discontinuity set also shows small spacing between the planes, and have a preferential orientation to the slope face for toppling.

Moderate Potential – When it involves a major set, but there is no shallow dipping set, or the orientation of the major set compared to the slope face was not preferential to toppling. Development of toppling becomes more difficult in this case.

Minor Potential, Minor Likelihood or Not Likely – When involving a major or minor set that has wide spacing or a minor set, which is assumed discontinuous.

The potential for toppling was assessed based on the shape of the blocks (block shape test), the relationship between the dip of the planes forming the slabs and the face angle (inter-layer slip test) and the orientation of the toppling set with respect to the wall (block alignment test). The block shape test states that if the ratio of the width of the block to the height of the block is less than the tangent of the base plane angle, then the centre of gravity of the block lies outside the base of the block, and the block is susceptible to toppling. The condition for inter-layer slip states that for toppling to occur, the dip of the wall must be greater than or equal to ninety degrees minus the dip of the toppling set plus the friction angle of the toppling set. The block alignment kinematic condition for toppling is that the steep inwardly dipping toppling set should strike approximately parallel to the wall face. The dip direction of the toppling set should be within about ± 10° of the wall dip direction (Wyllie 2007).

2.4 Results of Kinematic Assessment

Figures E4 to E11 illustrate the planar, wedge, and toppling potentials in terms of the major and minor set arrangements at Andrew Lake. Table E2 presents the results of the kinematic analyses for the Andrew Lake deposit location.

General observations from the kinematic analyses from the Andrew Lake data include:

- a number of inferred major discontinuity sets (JN1A/JN1B, FO1A, CJN1A and JN4B) show potential for low angle planar failures or wedge formations of less than 50° plunge. These inclined sets will impart IRA controls, and,
- a number of high angle discontinuity sets (JN2A, JN2B, JN3, JN5 and JN6) show potential for high angle planar failures on bench faces as well as for toppling or high angle wedge formations along kinematically favourable wall orientations. These high angle failure surfaces generally range between 60° and 82°, setting controls on BFA.

Table E2: Andrew Lake - Kinematic Design Sectors and Slope Stability Controls

Andrew Lake Walls		Kinematic Controls							
Kinematic Sectors		Toppling		Planar		Wedge (F.S. ≤ 1.2)			Comments
		Set	Dip (°)	Set	Dip (°)	Combination		Dip (°)	
AL.K.I	Dip Dir: 202˚ (180˚ to 225˚)	**JN2A**	69	**JN1B**	45	**JN1A**	**JN4B**	40	Major planar set (JN1B) and major toppling set identified are likely to limit IRA. Nine major wedge combinations dipping between 40 to 73° may limit BFA and IRA.
				JN2B	73	**JN1B**	**JN6**	42	
						JN1B	**JN5**	43	
						JN1B	**JN1A**	44	
						JN1B	**JN4B**	45	
						JN1A	**JN6**	46	
						JN4B	**JN5**	48	
						JN2B	**JN6**	64	
						JN2B	**JN5**	73	
AL.K.II	Dip Dir:247 ˚ (225˚ to 270˚)	**JN3**	60	JN4A	25	**JN1B**	**JN5**	43	Major planar set (JN4B) likely to limit IRA. Major toppling set may limit BFA. Four major wedge combinations dipping between 43 to 73° may limit BFA and IRA.
				JN4B	52	**JN1B**	**JN4B**	45	
						JN4B	**JN5**	48	
						JN2B	**JN5**	73	
AL.K.III	Dip Dir: 292˚ (270˚ to 315˚)	**JN5**	84	CJN1B	30	**CJN1A**	**JN4B**	40	Strong control through CJN1A. Strong sets combine to form wedges that are interpreted to be the major control on bench configuration. Toppling considered only a local concern, given the steepness of the sets.
		JN6	82			**CJN1A**	**JN2A**	50	
AL.K.IV	Dip Dir: 337˚ (315˚ to 360˚)	**JN1A**	46	**CJN1A**	51	**CJN1A**	**JN3**	47	Major planar set (CJN1A) likely to limit IRA. Major toppling set JN1A may limit BFA, while toppling set JN5 may limit IRA if the slope orientation is favourable. Three major wedge combinations dipping between 47 to 69° may limit BFA and IRA.
		JN5	84	CJN1B	30	**CJN1A**	**JN6**	48	
						JN2A	**JN6**	69	
AL.K.V	Dip Dir: 022˚ (000˚ to 045˚)	**JN1B**	45	**JN2A**	69	**CJN1A**	**JN3**	47	Major planar set (JN2A) likely to limit BFA. Toppling sets have potential to limit BFA and IRA, if slope orientation is favourable. Five major wedge combinations that dip between 47-69° may limit BFA and IRA.
		JN2B	84			**CJN1A**	**JN6**	48	
						JN3	**JN5**	60	
						JN2A	**JN5**	67	
						JN2A	**JN6**	69	

Table E2: Andrew Lake - Kinematic Design Sectors and Slope Stability Controls (continued)

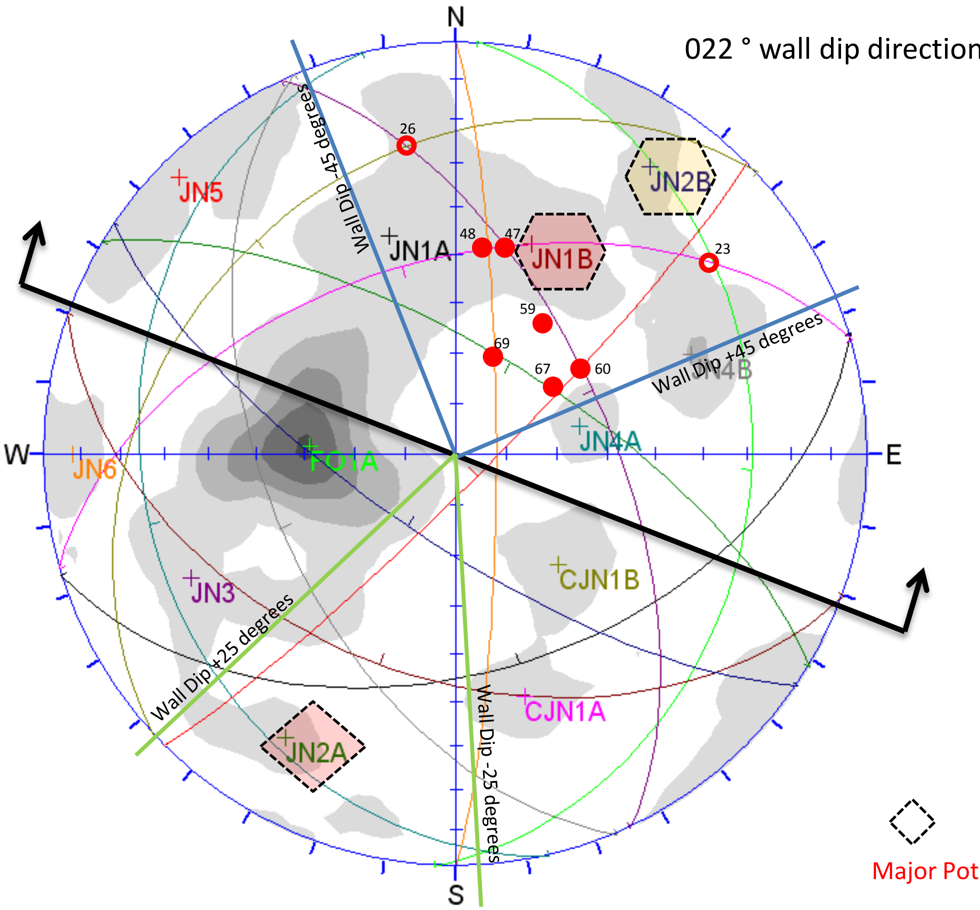
Andrew Lake Walls		Kinematic Controls							
Kinematic Sectors		Toppling		Planar		Wedge (F.S. ≤ 1.2)			Comments
		Set	Dip (°)	Set	Dip (°)	Combination		Dip (°)	
AL.K.VI	Dip Dir: 067˚ (045˚ to 090˚)	**JN4B**	**52**	**JN3**	**63**	**JN3**	**JN2A**	59	Strong planar and wedge controls through JN3. The controls on this orientation will control the bench face angle. Potential scatter of FO1A (up to 45 deg) would likely further limit IRA.
		JN2B	73	**FO1A**	**29**	**JN3**	**JN5**	60	
				JN6	82	**JN2A**	**JN5**	67	
						JN6	**JN5**	82	
AL.K.VII	Dip Dir: 112˚ (090˚ to 135˚)	N/A	N/A	**FO1A**	**29**	**JN1A**	**JN3**	40	Strong wedge controls through JN1A. Major planar set (FO1A) identified is likely dipping shallow enough to be less than the angle of friction, except possibly in fault altered ground. Potential scatter of FO1A up to 45 deg. Four major wedge combinations dipping between 40 to 82° may impact BFA and IRA. No toppling sets identified.
				JN5	84	**JN1A**	**JN2B**	44	
				JN6	82	**JN3**	**JN2A**	59	
						JN6	**JN5**	82	
K.VIII	Dip Dir: 157˚ (135˚ to 180˚)	**CJN1A**	51	**JN1A**	46	**JN1A**	**JN3**	40	Major planar set (JN1A) identified is likely to limit IRA. Major toppling set may limit BFA. Seven major wedge combinations dipping between 40 to 64° may limit BFA and IRA.
						JN1A	**JN4B**	40	
						JN1B	**JN6**	42	
						JN1A	**JN2B**	44	
						JN1B	**JN1A**	44	
						JN1A	**JN6**	46	
						JN2B	**JN6**	64	

Design Sectors:
Dip Direction: 000-045

- Design window for wedges = +/- 45 °
- Design window for planar failure = +/- 25 °

Kiggavik – Andrew Lake
Kinematic Assessment
Wall Dip Direction 022 Degrees

Figure E4



Set #	Dip	Dip Dir	ϕ	Spacing (m)*
FO1A	29	93	35	0.77
JN1A	46	163	35	0.71
JN1B	45	200	35	
JN2A	69	31	30	0.62
JN2B	73	214	35	
JN3	60	65	35	0.85
JN4A	25	257	30	0.89
JN4B	52	247	35	0.54
JN5	84	135	35	0.5
CJN1A	51	344	30	0.67
CJN1B	30	317	35	0.43
JN6	82	90	35	0.66

Wedge Analysis

Set 1	Set 2	Plunge	Trend	FS
FO1A	CJN1A	23	54	1.77
CJN1B	JN3	26	351	1.71
CJN1A	JN3	47	13	0.67
CJN1A	JN6	48	9	0.72
JN3	JN5	60	55	0.49
JN2A	JN5	67	59	0.38
JN2A	JN6	69	21	0.30

Planar Failures

Major Potential – set JN2A (69 °)

Toppling Failures

Moderate Potential – set JN2B (84 °)
Major Potential – set JN1B (45 °)

Potential Wedge Failures FS < 1.2 (Major-Major)

Potential Wedge Failures 1.2 < FS < 2 (Major-Major)

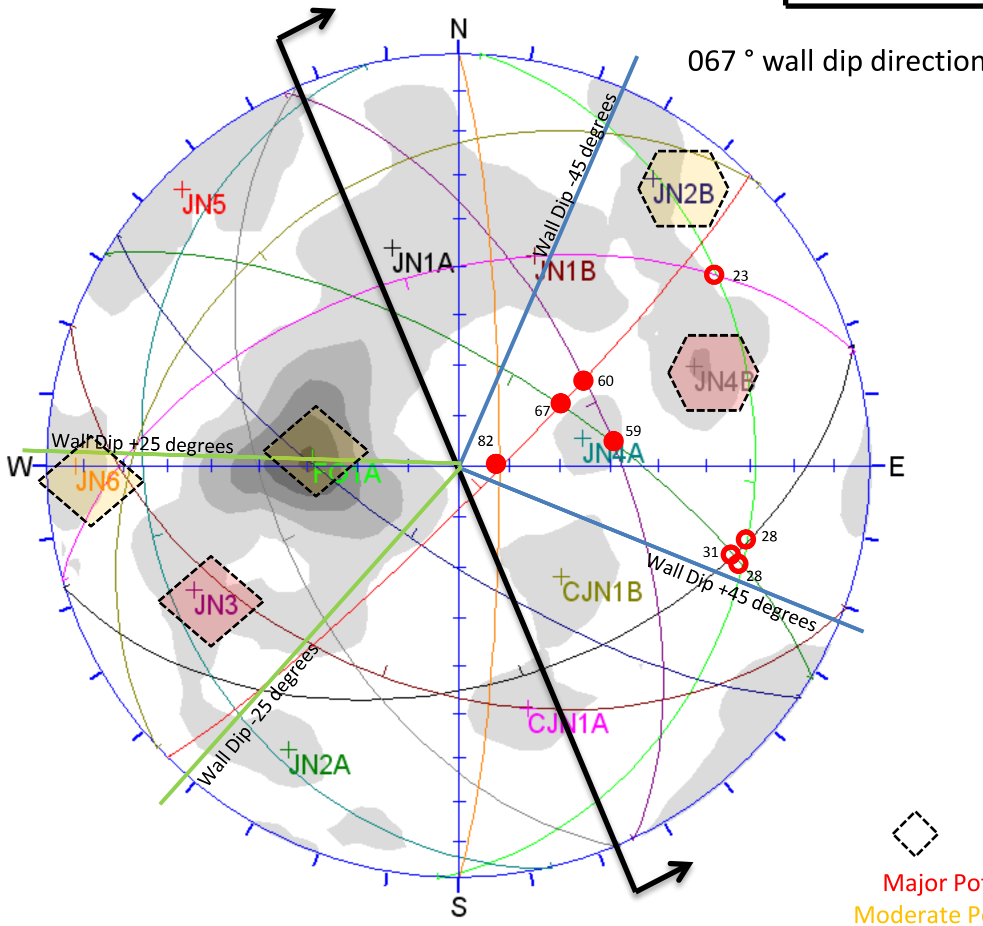
•Spacings taken as average for discontinuity sets indicated. Actual spacing may vary significantly.

Kiggavik – Andrew Lake
Kinematic Assessment
Wall Dip Direction 067 Degrees

Figure E5

Design Sectors:
Dip Direction: 045-090

- Design window for wedges = +/- 45 °
- Design window for planar failure = +/- 25 °



Set #	Dip	Dip Dir	ϕ	Spacing (m)*
FO1A	29	93	35	0.77
JN1A	46	163	35	0.71
JN1B	45	200	35	
JN2A	69	31	30	0.62
JN2B	73	214	35	
JN3	60	65	35	0.85
JN4A	25	257	30	0.89
JN4B	52	247	35	0.54
JN5	84	135	35	0.5
CJN1A	51	344	30	0.67
CJN1B	30	317	35	0.43
JN6	82	90	35	0.66

Wedge Analysis

Set 1	Set 2	Plunge	Trend	FS
FO1A	CJN1A	23	54	1.77
FO1A	JN2A	28	109	1.59
FO1A	JN1A	28	105	1.33
JN1A	JN2A	31	108	1.69
JN3	JN2A	59	81	0.73
JN3	JN5	60	55	0.49
JN2A	JN5	67	59	0.38
JN6	JN5	82	93	0.10

Planar Failures

Major Potential – set JN3 (63 °)
Moderate Potential – Set FO1A (29°)
Moderate Potential – Set JN6 (82°)

Toppling Failures

Major Potential – set JN4B (52°)
Moderate Potential – set JN2B (73°)

Potential Wedge Failures FS < 1.2 (Major-Major)

Potential Wedge Failures 1.2 < FS < 2 (Major-Major)

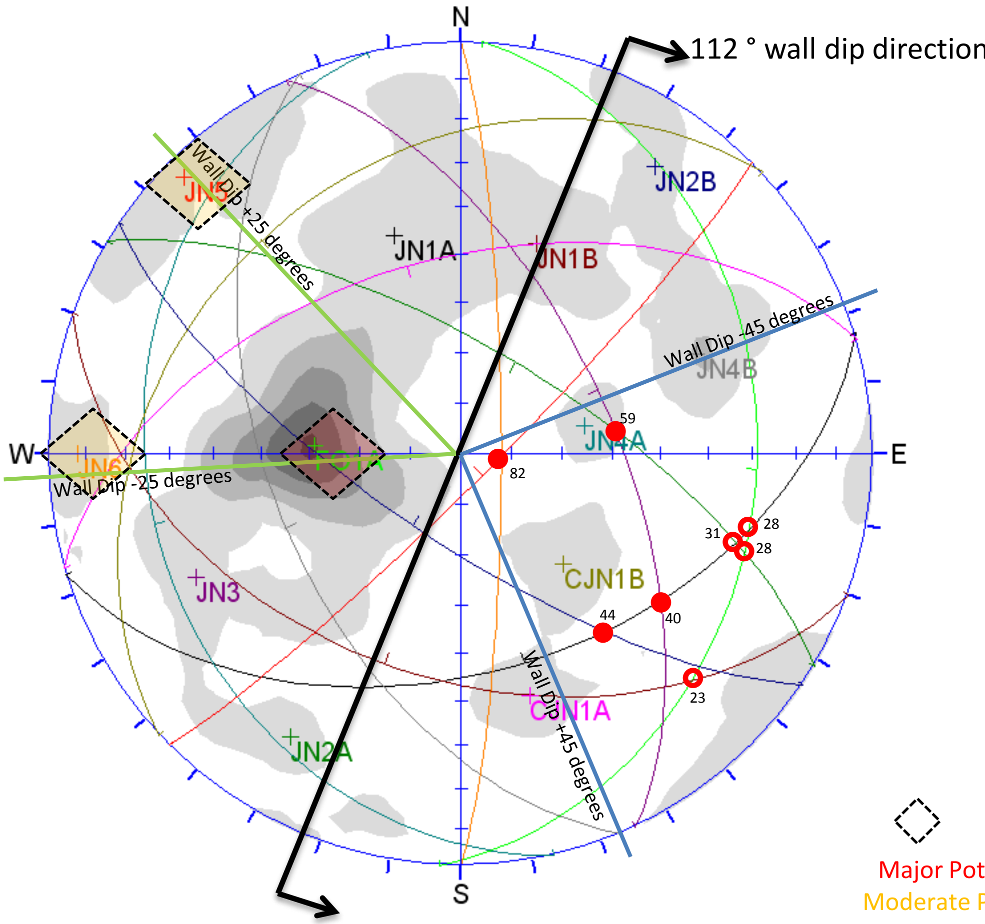
•Spacings taken as average for discontinuity sets indicated. Actual spacing may vary significantly.

Design Sectors:
Dip Direction: 090-135

- Design window for wedges = +/- 45 °
- Design window for planar failure = +/- 25 °

Kiggavik – Andrew Lake
Kinematic Assessment
Wall Dip Direction 112 Degrees

Figure E6



Set #	Dip	Dip Dir	ϕ	Spacing (m)*
FO1A	29	93	35	0.77
JN1A	46	163	35	0.71
JN1B	45	200	35	
JN2A	69	31	30	0.62
JN2B	73	214	35	
JN3	60	65	35	0.85
JN4A	25	257	30	0.89
JN4B	52	247	35	0.54
JN5	84	135	35	0.5
CJN1A	51	344	30	0.67
CJN1B	30	317	35	0.43
JN6	82	90	35	0.66

Wedge Analysis

Set 1	Set 2	Plunge	Trend	FS
FO1A	JN1B	23	135	1.90
FO1A	JN2A	28	109	1.59
FO1A	JN1A	28	105	1.33
JN1A	JN2A	31	108	1.69
JN1A	JN3	40	126	1.04
JN1A	JN2B	44	141	1.12
JN3	JN2A	59	81	0.73
JN6	JN5	82	93	0.10

Planar Failures

Major Potential – set FO1A (29°)
Moderate Potential – set JN5 (84 °)
Moderate Potential – set JN6 (82 °)

Toppling Failures

None Identified

Potential Wedge Failures FS < 1.2 (Major-Major)

Potential Wedge Failures 1.2 < FS < 2 (Major-Major)

- Spacings taken as average for discontinuity sets indicated. Actual spacing may vary significantly.

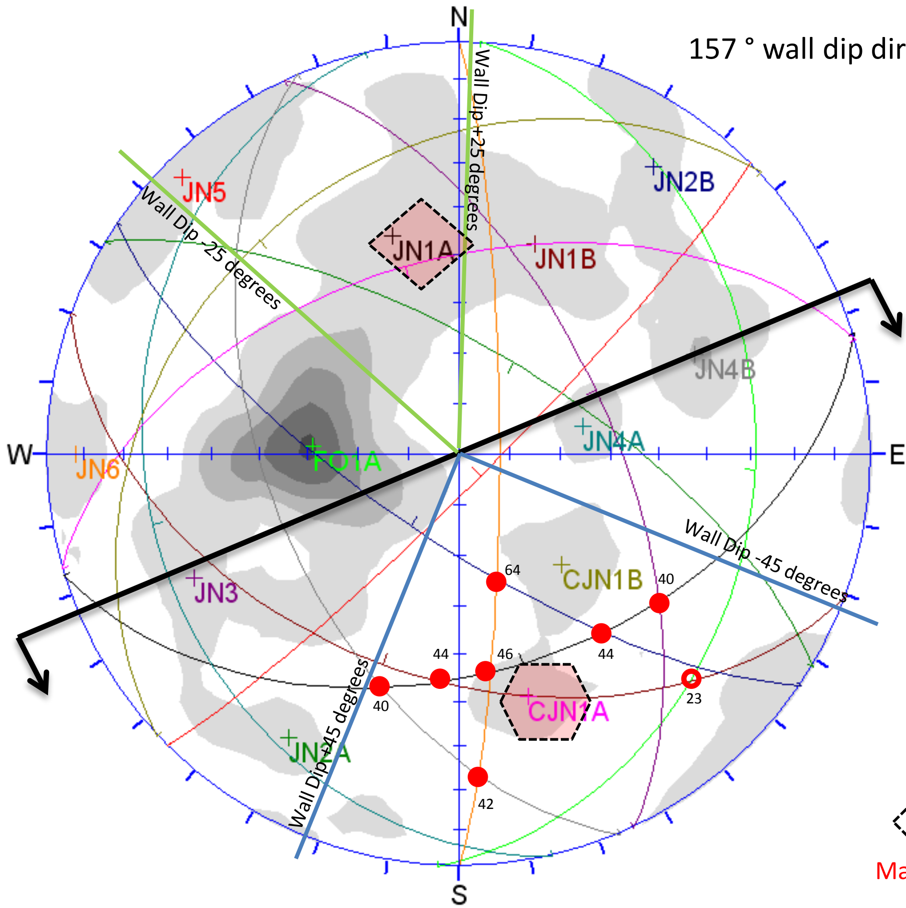
Project: 09-1362-0613 Drawn: EAM Reviewed: Rev.: 24 Nov 09

Design Sectors:
Dip Direction: 135-180

•Design window for wedges = +/- 45 °
•Design window for planar failure = +/- 25 °

Kiggavik – Andrew Lake
Kinematic Assessment
Wall Dip Direction 157 Degrees

Figure E7



Set #	Dip	Dip Dir	ϕ	Spacing (m)*
FO1A	29	93	35	0.77
JN1A	46	163	35	0.71
JN1B	45	200	35	
JN2A	69	31	30	0.62
JN2B	73	214	35	
JN3	60	65	35	0.85
JN4A	25	257	30	0.89
JN4B	52	247	35	0.54
JN5	84	135	35	0.5
CJN1A	51	344	30	0.67
CJN1B	30	317	35	0.43
JN6	82	90	35	0.66

Wedge Analysis

Set 1	Set 2	Plunge	Trend	FS
FO1A	JN1B	23	135	1.90
JN1A	JN3	40	126	1.04
JN1A	JN4B	40	198	0.96
JN1B	JN6	42	173	1.04
JN1A	JN2B	44	141	1.12
JN1B	JN1A	44	184	0.75
JN1A	JN6	46	172	0.78
JN2B	JN6	64	163	0.66

Planar Failures
Major Potential – set JN1A (46 °)

Toppling Failures
Major Potential – set CJN1A (51°)

Potential Wedge Failures FS < 1.2 (Major-Major)

Potential Wedge Failures 1.2 < FS < 2 (Major-Major)

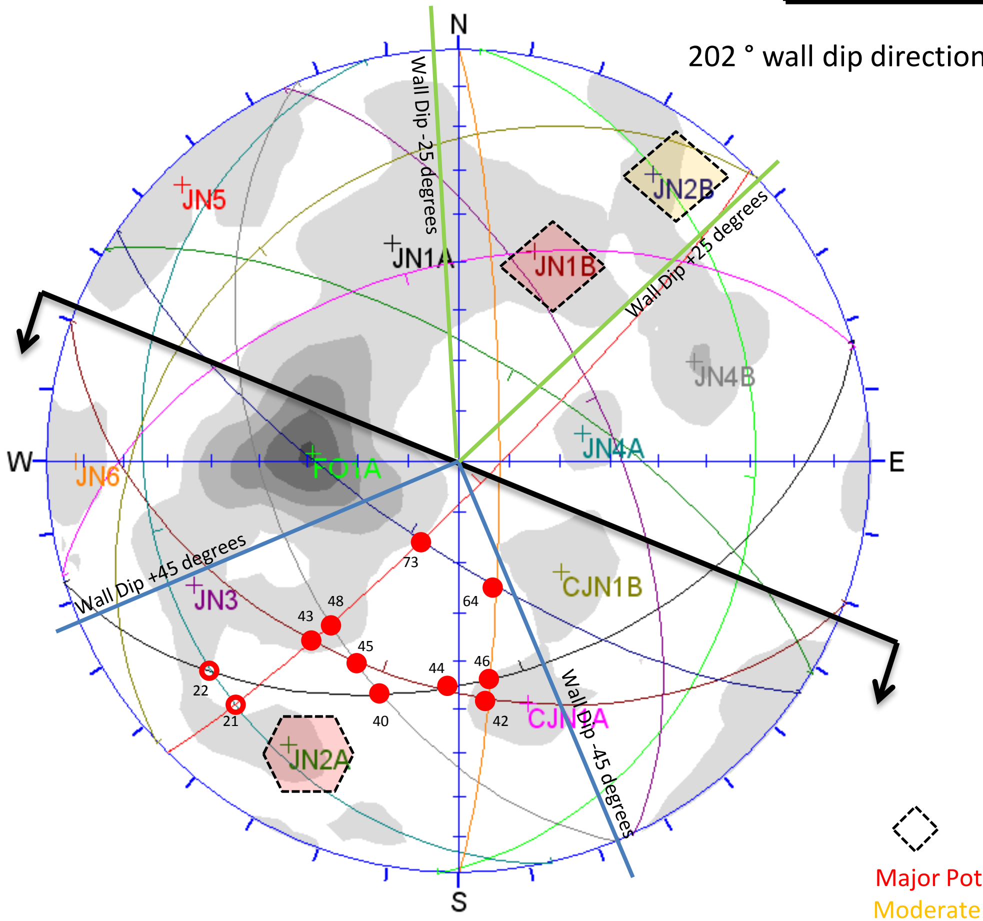
•Spacings taken as average for discontinuity sets indicated. Actual spacing may vary significantly.

Design Sectors:
Dip DIrection: 180-225

•Design window for wedges = +/- 45 °
•Design window for planar failure = +/- 25 °

Kiggavik – Andrew Lake
Kinematic Assessment
Wall Dip Direction 202 Degrees

Figure E8



Set #	Dip	Dip Dir	ϕ	Spacing (m)*
FO1A	29	93	35	0.77
JN1A	46	163	35	0.71
JN1B	45	200	35	
JN2A	69	31	30	0.62
JN2B	73	214	35	
JN3	60	65	35	0.85
JN4A	25	257	30	0.89
JN4B	52	247	35	0.54
JN5	84	135	35	0.5
CJN1A	51	344	30	0.67
CJN1B	30	317	35	0.43
JN6	82	90	35	0.66

Wedge Analysis

Set 1	Set 2	Plunge	Trend	FS
JN4A	JN5	21	223	1.94
JN4A	JN1A	22	229	1.58
JN1A	JN4B	40	198	0.96
JN1B	JN6	42	173	1.04
JN1B	JN5	43	219	0.98
JN1B	JN1A	44	184	0.75
JN1B	JN4B	45	208	0.72
JN1A	JN6	46	172	0.78
JN4B	JN5	48	218	0.88
JN2B	JN6	64	163	0.66
JN2B	JN5	73	205	0.24

Planar Failures
Major Potential – set JN1B (45 °)
Moderate Potential – JN2B (73 °)

Toppling Failures
Major Potential – set JN2A (69 °)

Potential Wedge Failures FS < 1.2 (Major-Major)

Potential Wedge Failures 1.2 < FS < 2 (Major-Major)

•Spacings taken as average for discontinuity sets indicated. Actual spacing may vary significantly.

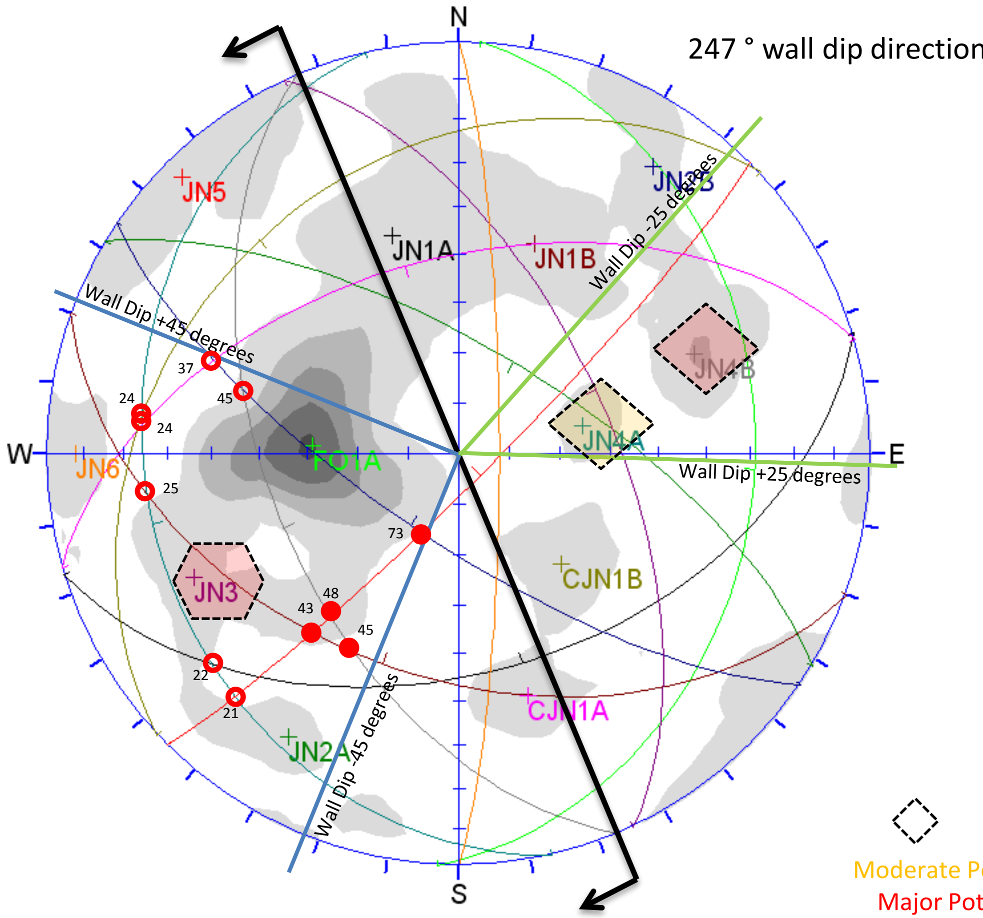
Project: 09-1362-0613 Drawn: EAM Reviewed: Rev.: 24 Nov 09

Design Sectors:
Dip Direction: 225-270

- Design window for wedges = +/- 45 °
- Design window for planar failure = +/- 25 °

Kiggavik – Andrew Lake
Kinematic Assessment
Wall Dip Direction 247 Degrees

Figure E9



Set #	Dip	Dip Dir	ϕ	Spacing (m)*
FO1A	29	93	35	0.77
JN1A	46	163	35	0.71
JN1B	45	200	35	
JN2A	69	31	30	0.62
JN2B	73	214	35	
JN3	60	65	35	0.85
JN4A	25	257	30	0.89
JN4B	52	247	35	0.54
JN5	84	135	35	0.5
CJN1A	51	344	30	0.67
CJN1B	30	317	35	0.43
JN6	82	90	35	0.66

Wedge Analysis

Set 1	Set 2	Plunge	Trend	FS
JN4A	JN5	21	223	1.94
JN4A	JN1A	22	229	1.58
JN4A	CJN1B	24	277	1.43
JN4A	CJN1A	24	275	1.38
JN4A	JN1B	25	262	1.41
CJN1A	JN2B	37	291	1.37
JN1B	JN5	43	219	0.98
JN4B	JN2B	45	286	1.71
JN1B	JN4B	45	208	0.72
JN4B	JN5	48	218	0.88
JN2B	JN5	73	205	0.24

Planar Failures

Moderate Potential – set JN4A (25 °)

Major Potential – Set JN4B (52°)

Toppling Failures

Major Potential – set JN3 (60°)

Potential Wedge Failures FS < 1.2 (Major-Major)

Potential Wedge Failures 1.2 < FS < 2 (Major-Major)

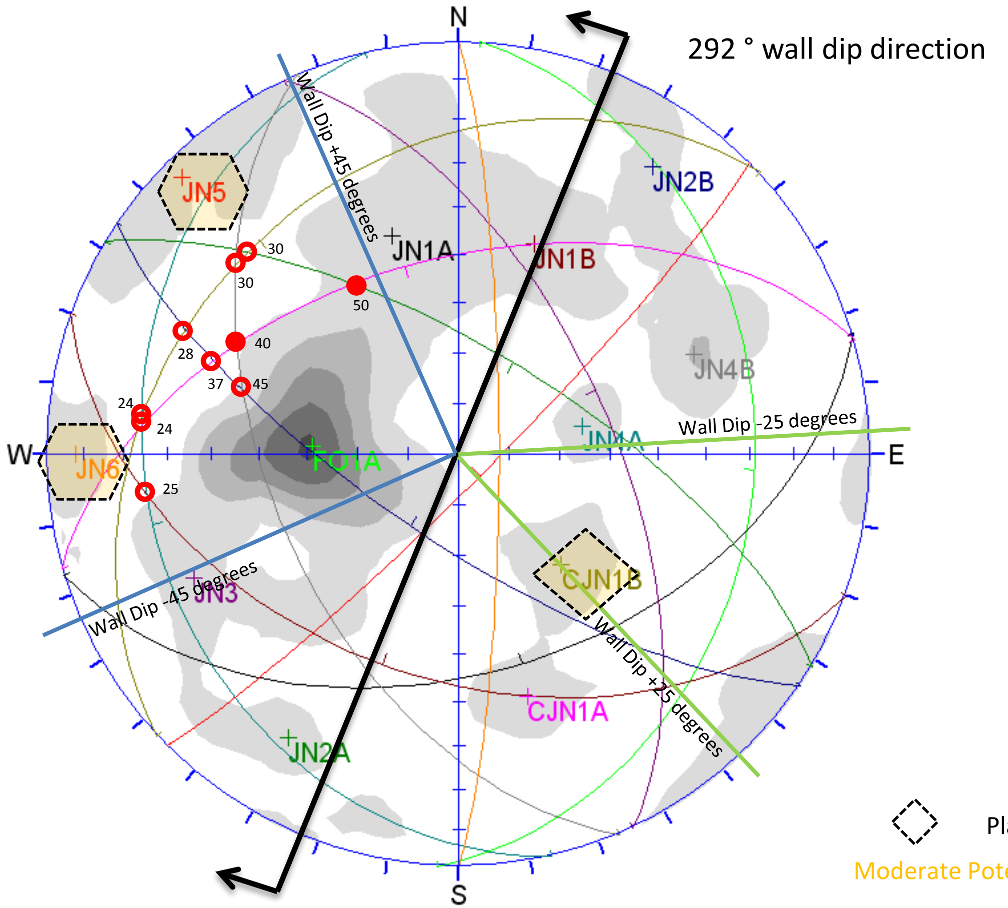
- Spacings taken as average for discontinuity sets indicated. Actual spacing may vary significantly.

Design Sectors:
Dip Direction: 270-315

•Design window for wedges = +/- 45 °
•Design window for planar failure = +/- 25 °

Kiggavik – Andrew Lake
Kinematic Assessment
Wall Dip Direction 292 Degrees

Figure E10



Set #	Dip	Dip Dir	ϕ	Spacing (m)*
FO1A	29	93	35	0.77
JN1A	46	163	35	0.71
JN1B	45	200	35	
JN2A	69	31	30	0.62
JN2B	73	214	35	
JN3	60	65	35	0.85
JN4A	25	257	30	0.89
JN4B	52	247	35	0.54
JN5	84	135	35	0.5
CJN1A	51	344	30	0.67
CJN1B	30	317	35	0.43
JN6	82	90	35	0.66

Wedge Analysis

Set 1	Set 2	Plunge	Trend	FS
JN4A	CJN1B	24	277	1.43
JN4A	CJN1A	24	275	1.38
JN4A	JN1B	25	262	1.41
CJN1B	JN2B	28	295	1.50
CJN1B	JN4B	30	310	1.25
CJN1B	JN2A	30	314	1.26
CJN1A	JN2B	37	291	1.37
CJN1A	JN4B	40	296	0.94
JN4B	JN2B	45	286	1.71
CJN1A	JN2A	50	328	0.73

Planar Failures
Moderate Potential – set CJN1B (30 °)

Toppling Failures
Moderate Potential – set JN5 (84 °)
Moderate Potential – set JN6 (82 °)

Potential Wedge Failures FS < 1.2 (Major-Major)

Potential Wedge Failures 1.2 < FS < 2 (Major-Major)

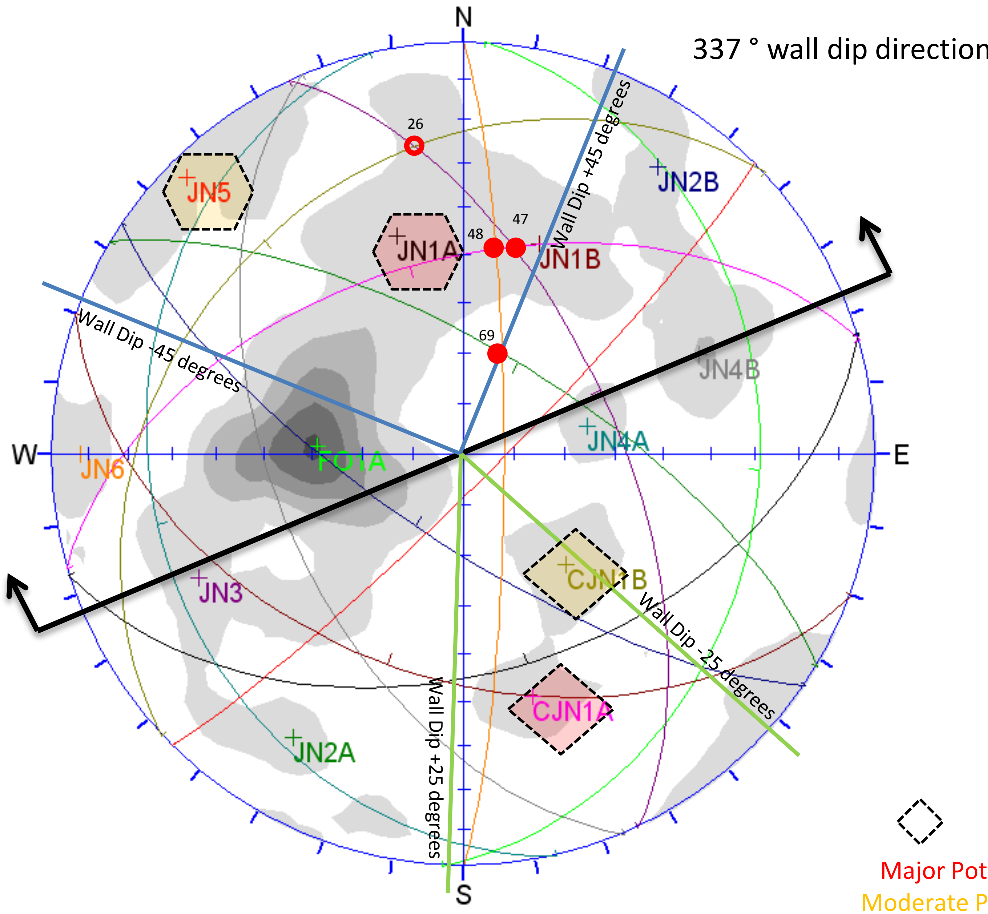
•Spacings taken as average for discontinuity sets indicated. Actual spacing may vary significantly.

Design Sectors:
Dip Direction: 315-360

- Design window for wedges = +/- 45 °
- Design window for planar failure = +/- 25 °

Kiggavik – Andrew Lake
Kinematic Assessment
Wall Dip Direction 337 Degrees

Figure E11



Set #	Dip	Dip Dir	ϕ	Spacing (m)*
FO1A	29	93	35	0.77
JN1A	46	163	35	0.71
JN1B	45	200	35	
JN2A	69	31	30	0.62
JN2B	73	214	35	
JN3	60	65	35	0.85
JN4A	25	257	30	0.89
JN4B	52	247	35	0.54
JN5	84	135	35	0.5
CJN1A	51	344	30	0.67
CJN1B	30	317	35	0.43
JN6	82	90	35	0.66

Wedge Analysis

Set 1	Set 2	Plunge	Trend	FS
CJN1B	JN3	26	351	1.71
CJN1A	JN3	47	13	0.67
CJN1A	JN6	48	9	0.72
JN2A	JN6	69	21	0.30

Planar Failures
Major Potential – set CJN1A (51 °)
Moderate Potential – Set CJN1B (30°)

Toppling Failures
Major Potential – set JN1A (46 °)
Moderate Potential – set JN5 (84 °)

Potential Wedge Failures FS < 1.2 (Major-Major)

Potential Wedge Failures 1.2 < FS < 2 (Major-Major)

•Spacings taken as average for discontinuity sets indicated. Actual spacing may vary significantly.

3.0 REFERENCES

Barton, N.R., Lien, R. and Lunde, J. 1974. Engineering classification of rock masses for the design of tunnel support. *Rock Mech.* **6**(4), 189-239.

Barton, N. and Grimstad, E. 1994. The Q-system following twenty years of application in NMT support selection. 43rd Geomechanic Colloquay, Salzburg. Felsbau, 6/94. 428-436.

Wyllie, D.C. and Mah, C.W., 2007. Rock Slope Engineering Civil and Mining, 4th Edition. Spon Press, New York.

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Africa	+ 27 11 254 4800
Asia	+ 852 2562 3658
Australasia	+ 61 3 8862 3500
Europe	+ 356 21 42 30 20
North America	+ 1 800 275 3281
South America	+ 55 21 3095 9500

solutions@golder.com
www.golder.com

Golder Associates Ltd.
1721 8th Street East
Saskatoon, Saskatchewan, Canada S7H 0T4
Canada
T: +1 (306) 665 7989

Attachment B Kiggavik Open Pit Design Report

August 27, 2014

REPORT

FINAL REPORT

Geotechnical Recommendations for the Proposed Kiggavik Main and Kiggavik Centre Open Pits - Support Document for Permit Application

Submitted to:
AREVA Resources Canada Inc.
817 - 45th Street West
P.O. Box 9204
Saskatoon, Saskatchewan
S7K 3X5

Attention: Mr. Bradley Schmid

Reference Number: 1213620158-005-Rev0-3000

Distribution:Distribution:Distribution:

1 Electronic Copy (Pdf) - AREVA Resources Canada Inc.
1 Electronic Copy (Word) - AREVA Resources Canada Inc.
2 Hard Copies - Golder Associates Ltd.

Executive Summary

GENERAL

Golder Associates Ltd. was retained by AREVA Resources Canada Inc. to carry out geotechnical studies on the Kiggavik Project located near Baker Lake, Nunavut. This report presents the results of the investigations undertaken in 2009 to develop geotechnical criteria for the purpose of slope design and has been prepared as a supporting document to the EIS. However the recommendations presented in this report are intended to be dealt with during the detailed design stage for licencing, and are not a requirement to increase the confidence or robustness of the EIS.

The Kiggavik project is located approximately 80 km west of Baker Lake, and 400 km from the west coast of Hudson Bay. The project area consists of flat lying or gently sloping terrain. A series of low-lying scarps oriented west-southwest to east-northeast separate local drainage patterns. The vegetation is typical of the Tundra Barren Lands.

Kiggavik is located in the zone of continuous permafrost. The active layer is considered to vary between 1.3 m and 1.5 m depending upon the nature of the underlying material. Evidence to date suggests that the permafrost thickness is between 210 m to 250 m based on drilling experience and the limited thermistor data available.

The uranium ore bodies of Kiggavik occur within an early Aphebian sequence of meta-arkoses and metapelites which are overlain by orthoquartzites. Hudsonian granite has intruded in the vicinity of one of the ore zones. The alteration and associated mineralization occurs in three principal zones (the Main Zone and Centre Zone at Kiggavik and the Andrew Lake Zone at Sissons) for which open pit mining has been proposed. This report focuses on the Main Zone and Centre Zone deposits.

INTERPRETATION OF EXISTING INFORMATION

The existing data provided to Golder by AREVA in support of the study of the Kiggavik Main Zone and Centre Zone sites included:

- SRK core orientation data and geotechnical logs. Peak orientations of structure data captured by the SRK oriented boreholes at Kiggavik were included in the assessment of the rock mass fabric (Appendix C). This was done to maximize the available data for interpretation, and to help eliminate bias by providing data from a complimentary borehole orientation. Rock mass and strength estimation data by SRK were incorporated onto the geotechnical cross-sections in Appendix B, to help categorize the rock masses of the pit floors and wall rock as altered and weak, or not. The key deficiency of the SRK data was that the borehole locations do not provide information on the pit wall rock.
- Interpreted regional fault traces based on geophysics (Beaudemont) as provided by AREVA.

- Shape files of the top of granitic basement and other rock types. Note that no shape file was provided for the north-south dyke mentioned in the Golder 1989 report, which is interpreted to sit above the local rise in the granitic basement that helps to cause the "Figure 8" character and central pillar of the Kiggavik Main Zone open pit. Golder understands from discussion with AREVA that this feature should be assumed to exist and be exposed on the pit walls. Because of the absence of a shape file for this feature, it is not projected or otherwise shown on figures and drawings in this report. The rock within the lower central pillar was assumed to be altered related to faulting and mineralization, and this alteration was inferred to reduce with distance away from mineralization.
- Shape files of the local fault interpretations. It is important to note that the fault extents were very limited, and were not shown to extend to the pit walls in most cases. These faults were assumed to be associated with an alteration halo, most intense near mineralization, resulting in a weaker rock mass. Faults were assumed to have greater continuity than provided by AREVA, and their projections onto the pit walls and stability implications are addressed in this report.

KIGGAVIK ROCK SLOPE DESIGNS

The wall rocks for the Kiggavik Main Zone and Kiggavik Centre Zone pits are generally competent to very competent rocks which exhibit brittle rock characteristics. The orthoquartzites and the granitic intrusive in particular are very strong rocks. The metasediments are not of the same strength, but are still very competent with respect to providing a stable wall for an open pit. The only low strength materials are the altered rocks associated with the mineralization and thus will generally be removed as ore. The stability of slopes excavated under these conditions will therefore be primarily controlled by the orientations, spacing, persistence and strength of the discontinuities that exist within the rock mass.

Interpretations of outcrop mapping and oriented core data suggest that the major controlling structural systems in the vicinity of the open pits are very consistent. A stereographic study of the kinematics of failure was conducted for a range of proposed slope orientations to identify discontinuities oriented such that they could give rise to slope failure.

The kinematics of failure study showed that the most likely potential modes of failure are planes and wedges. A previous study by Golder in 1989 suggested that toppling would be the main control. New information on horizontal and sub-vertical joint spacing suggests that block shapes with potential toppling are not likely. Rather, blocks will occur, and ravel rather than topple. This type of failure is likely to be confined to near surface, bench scale instability in the south and east walls of both pits. The potential for small scale wedge and planar type failures has also been identified but is again likely to be confined to bench scale. The conclusion of the kinematic study is that slopes of 50º to 55º could be mined however the requirement for safety berms to contain bench scale failure will determine the inter-ramp slope geometry.

BENCH CONFIGURATIONS

Catch bench widths, bench face angles and bench heights, which define the inter-ramp slope geometry, were determined on the basis of:

- The results of the kinematic stability analysis.
- The nominal 9 m bench height, with alternate configurations for 12 m bench heights presented for comparison, at AREVA's request.
- The likelihood of catch bench debris accumulation and the requirement for catch bench cleanup.
- A minimum bench width of 8 m, to conform to Nunavut mining regulations.
- Bench configurations were also adjusted based on the results of rock mass failure analyses for overall slopes, to reduce the inter-ramp and overall slope angles to conform to the stability analysis results.
- A maximum bench face angle of 75º.
- Single benching (9 m or 12 m vertical) maximum in poorer rock masses.

The resulting slope designs are based upon a base-case configuration with 8 m catch benches, bench slope angles of 75° and single bench heights (in waste material) of either 9 m or 12 m.

Analyses were originally carried out using 12m bench heights. Following issue of the initial draft of the Main Zone and Centre Zone pit slope recommendations in early December 2009, AREVA asked for slope design recommendations assuming a vertical bench height of 9 m at both Main and Centre Zones, in addition to the height of 12 m that was assumed for the initial analysis. Given that the Kiggavik slopes are considered to be controlled by kinematic failure mechanisms, there will be no change in the recommended inter-ramp angles for this decrease in bench height to 9 m, because for the double bench configuration considered, the recommended inter-ramp angles can be achieved while maintaining a minimum 8 m catch berm. The difference to the operation will be total number of benches per slope, which will increase with decreased operating bench height.

GEOTECHNICAL BERMS

It is becoming common practice to periodically include extra wide catch benches at regular intervals in open pits, with a purpose to provide additional safety to operators and equipment, particularly on portions of the open pit walls not crossed by a ramp. The extra wide benches provide catchment to rock fall and debris, and can provide some flexibility to the mine plan. The inclusion of geotechnical berms into the pit slope designs is appropriate at Main Zone, where significant slope heights are developed. These benches are recommended to be 15 m to 20 m wide. For planning purposes it is recommended that the pit have a geotechnical berm at approximately 60m below ground surface, and a minimum of one other geotechnical bench. This second geotechnical bench should be established at an elevation to optimize the safe drilling and installation of the initial pit slope and pit floor depressurization vertical drains, with a view that this bench requires permanent access for the remainder of mine life.

OVERBURDEN SLOPES

Overburden material will be stripped back from the pit crests and trimmed to a slope angle of nominally 35°. The slope will be protected from thaw and sloughing by placing a protective layer of good quality waste rock against the slope. Perimeter diversion ditches are recommended at the overburden crest, and at the base of the overburden. A catch bench of minimum 12.5 m width is recommended at the base of the overburden slope, to allow room for slope maintenance and drainage control over the life of mine. Geotechnical investigations will be required to verify the suitability of this nominal design.

OPERATIONAL CONSIDERATIONS

Operational considerations for the Kiggavik pit designs include the following:

- Controlled blasting will be required to ensure minimum overbreak in final wall slopes in order to reduce bench break-back.
- Artificial slope support in the form of rock anchors may be a viable option for ground control in areas such as bench slopes adjacent to haul ramps.
- The shape of the pit should be designed to avoid convex slopes or "noses" which are invariably more unstable than concave slopes.
- Pit floor depressurization may be required late in the life of the Main Zone pit in order to relieve ground water pressures below the permafrost layer.
- Surface water protection will be required around the open pits for the brief period of heavy runoff experienced in the spring.
- Additional consideration and testing will need to be given to the blast plan to overcome challenges associated with permafrost conditions.

These design criteria provide the basis for discussions with mine planning personnel with a view to finalizing pit designs.

Study Limitations

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TABLES

FIGURES

APPENDICES

1.0 INTRODUCTION

Golder Associates Ltd. (Golder) was retained by AREVA Resources Canada Inc. (AREVA) to carry out geotechnical and hydrogeological studies on the Kiggavik Project located near Baker Lake, Nunavut. This report presents the results of the investigations undertaken in 2009 to develop geotechnical criteria for the purpose of slope design and has been prepared as a supporting document to the EIS. However the recommendations presented in this report are intended to be dealt with during the detailed design stage for licencing, and are not a requirement to increase the confidence or robustness of the EIS.

The Kiggavik property is situated approximately 80 km west of Baker Lake, Nunavut (Figure 1). The Kiggavik project can be divided into two main deposit areas: the Kiggavik area and the Sissons area. Main, Centre and East Zone deposits are located within the Kiggavik area, while the Andrew Lake and End Grid deposits are located within the Sissons area, approximately 15 km to 17 km south of the Kiggavik area, as shown on Figure 2.

The terms of reference for the project were based on the request for proposal (RFP) issued by AREVA on February 19, 2009, in conversations with and clarifications provided by AREVA personnel and on Golder's knowledge of the Kiggavik site. These included:

- collect geotechnical and hydrogeological data for the Kiggavik Main Zone, Kiggavik Centre Zone and Andrew Lake sites in order to confirm the existing open pit design assumptions and to provide recommendations regarding pit slope design criteria; and
- collect representative geotechnical and hydrogeological data to assist in the design of End Grid underground development.

The work program to achieve these objectives included review of the available data relevant to the project, geotechnical logging, hydrogeologic testing, groundwater sampling, instrumentation, data analysis and report preparation.

Deliverables for the project included:

- daily progress reports during field investigations;
- a factual site investigation report, summarizing the geotechnical/hydrogeological data collected; and
- a pit slope stability analysis report for each of the open pit sites (Kiggavik Main Zone, Kiggavik Centre Zone and Andrew Lake).

This report presents the results of the recommended geotechnical criteria for the proposed open pit mines at Kiggavik Main Zone and Centre Zone. Although geotechnical core logging was conducted at the proposed Andrew Lake pit location, and at the End Grid deposit, geotechnical aspects of mine development for these deposits will not be discussed in this report. The data findings for End Grid and Andrew Lake from the 2009 field investigation are presented in a separate report.

150 0 150

SCALE 1:7,500,000 KILOMETRES

Legend

- Community
- Proposed Kiggavik - Sissons Location

PROJECT

AREVA

AREVA RESOURCES CANADA INC.
KIGGAVIK PROJECT

TITLE

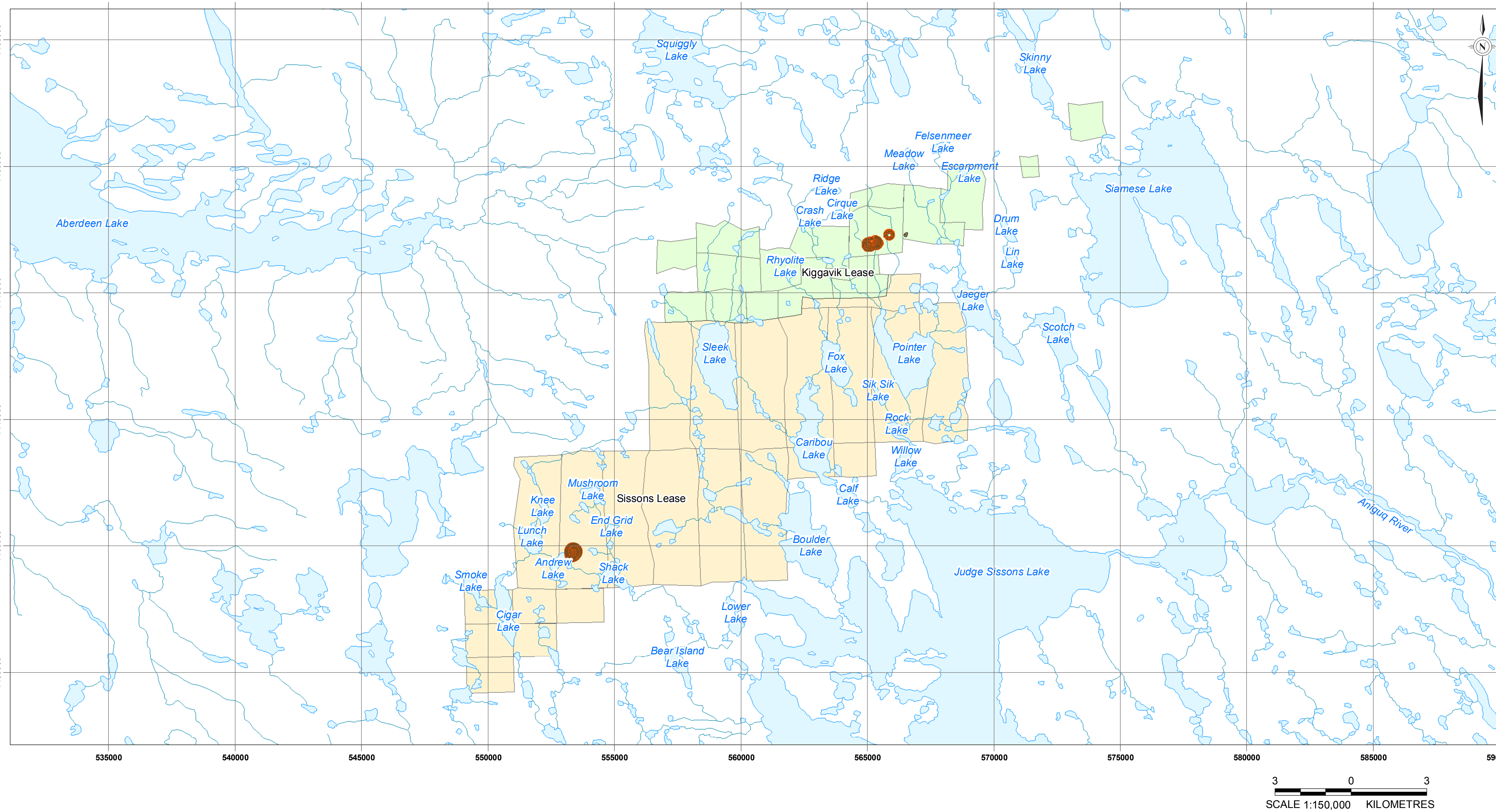
GENERAL SITE LOCATION

PROJECT		09-1362-0613	FILE No.	
DESIGN			SCALE AS SHOWN	REV. 0
GIS	MD	22/12/09	**FIGURE: 1**	
CHECK	EM	22/12/09		
REVIEW	LG	22/12/09		

Golder Associates
Saskatoon, Saskatchewan

Reference
ESRI Canada Data
Atlas of Canada Data

G:\2009\1362\09-1362-0613 Areva Kiggavik Golder Internal Field Manual\GIS\Maps\Geotechnical\Kiggavik Site\GTI-09-1362-0613-3100-GEO-General Location.mxd



PROJECT
AREVA
AREVA RESOURCES CANADA INC.
KIGGAVIK PROJECT

TITLE
SITE MAP

Reference:
AREVA Resources Canada Inc.
NTS Mapsheets 56D, 66A, 66B
NAD 83 UTM Zone 14

Golder Associates
Saskatoon, Saskatchewan

PROJECT		09-1362-0613	FILE No.	
DESIGN			SCALE AS SHOWN	REV. 0
GIS	MD	22/12/09		
CHECK	EM	22/12/09	FIGURE: 2	
REVIEW	LG	22/12/09		

G:\2009\1362\09-1362-0613 Areva Kiggavik Golder Internal Field Manual\GIS\Maps\Geotechnical\Kiggavik Site\GTI-09-1362-0613-GEO-Mine Sites Layout Overview.mxd

1.1 Interpretation of Existing Information

The existing data provided to Golder by AREVA in support of the study of the Kiggavik Main Zone and Centre Zone sites included:

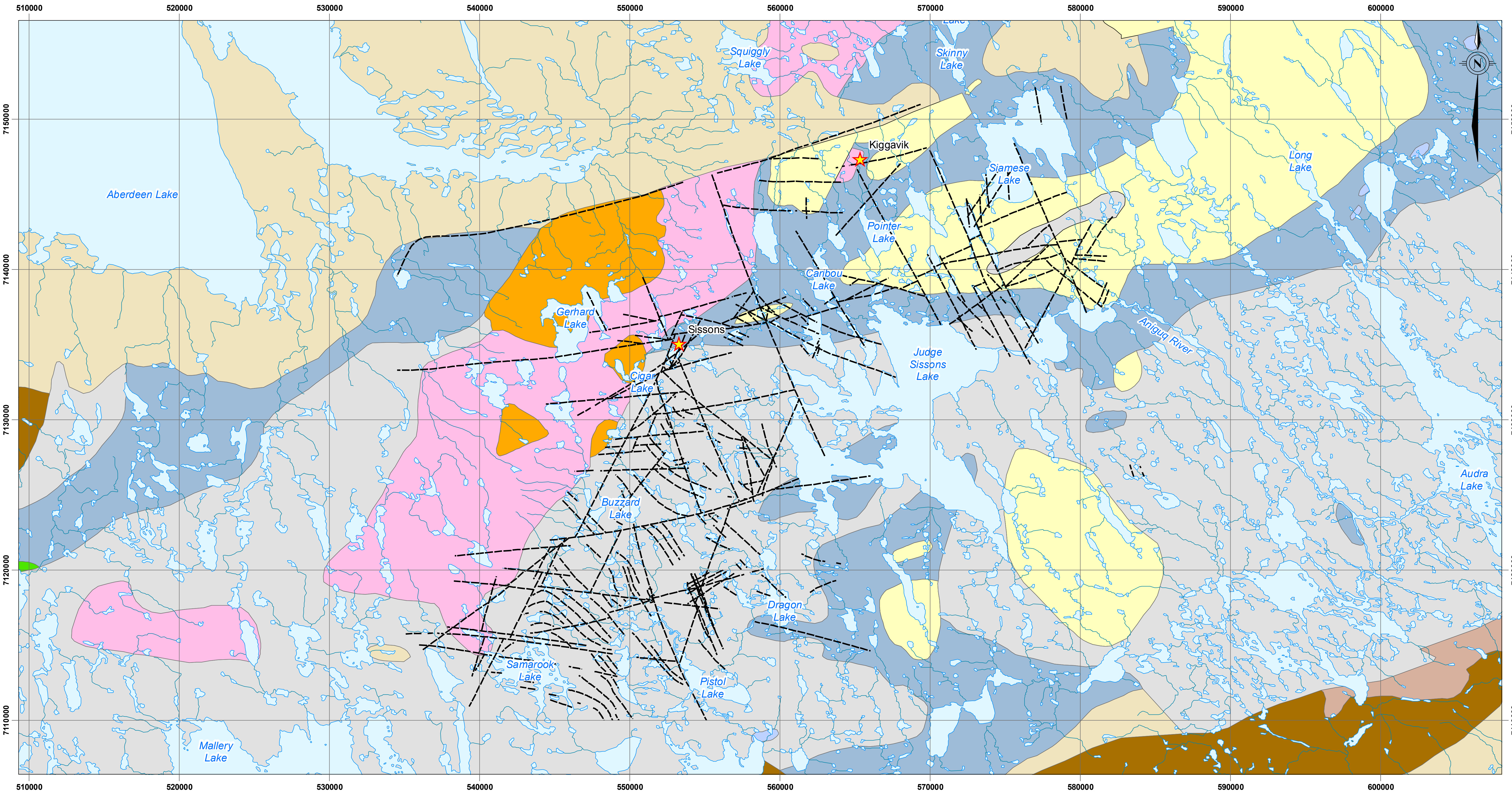
- SRK core orientation data and geotechnical logs. Peak orientations of structure data captured by the SRK oriented boreholes at Kiggavik were included in the assessment of the rock mass fabric (Appendix C). This was done to maximize the available data for interpretation, and to help eliminate bias by providing data from a complimentary borehole orientation. Rock mass and strength estimation data by SRK were incorporated onto the geotechnical cross-sections in Appendix B, to help categorize the rock masses of the pit floors and wall rock as altered and weak, or not. The key deficiency of the SRK data was that the borehole locations do not provide information on the pit wall rock.
- Interpreted regional fault traces based on geophysics (Beaudemont) as provided by AREVA.
- Shape files of the top of granitic basement and other rock types. Note that no shape file was provided for the north-south dyke mentioned in the Golder 1989 report, which is interpreted to sit above the local rise in the granitic basement that helps to cause the “Figure 8” character and central pillar of the Kiggavik Main Zone open pit. Golder understands from discussion with AREVA that this feature should be assumed to exist and be exposed on the pit walls. Because of the absence of a shape file for this feature, it is not projected or otherwise shown on figures and drawings in this report. The rock within the lower central pillar was assumed to be altered related to faulting and mineralization, and this alteration was inferred to reduce with distance away from mineralization.
- Shape files of the local fault interpretations. It is important to note that the fault extents were very limited, and were not shown to extend to the pit walls in most cases. These faults were assumed to be associated with an alteration halo, most intense near mineralization, resulting in a weaker rock mass. Faults were assumed to have greater continuity than provided by AREVA, and their projections onto the pit walls and stability implications are addressed in this report.

1.2 Location

The Kiggavik project is located in the Keewatin District of Nunavut approximately 80 km west of Baker Lake, and 400 km from the west coast of Hudson Bay. Latitude is approximately 64°26’ North and longitude 97º39’ West. Figure 1 shows the general project location.

1.3 General Geological Setting

The Kiggavik project is located within the Rae Province (AREVA 2007), at the southwest end of the Archean Woodburn Group, and at the northeasten end of the Thelon Basin which formed after the Hudsonian Orogeny (Golder 1989). The Woodburn group consists of metavolcanic and metasedimentary assemblages, which are in tectonic contact and structurally overlie Archean basement granitic to minor amphibolitic gneisses (AREVA 2007), while the Thelon Sandstone unconformably overlies a series of geological units ranging from Archean Basement to Aphebian rocks of various metamorphic grades (Golder 1989). A regional geological map is presented in Figure 3.



5 0 5

SCALE 1:250,000 KILOMETRES

Legend

Project Location	Felsic to intermediate volcanics and associated sediments	Nueltin Intrusive Suite
Fault Structure	Gneiss (undifferentiated)	Orthoquartzite
'other' Proterozoic sedimentary rocks	Gneiss, mostly granulites	Supracrustal rocks (undifferentiated)
Amer Group and equivalents, ca. 2.1 Ga	Granite (undivided)	Syenite
Baker Lake Group, ca. 1.84 Ga	Granitic rocks, granites, and granodiorites	Tonalites, diorites, and gabbros
Barrensland Group, ca. 1.72 Ga	Hurwitz Group and equivalents, ca. 2.1 Ga	Volcanics and sediments of the Prince Albert Group
Dolomitic limestone, sandstone, shale	Mafic volcanics and associated sediments	Wharton Group, ca. 1.75 Ga

Reference:
Faults Courtesy of
AREVA Pre-feasability Study
Geological Structures Courtesy
of Natural Resources Canada 2002
NTS Mapsheets 56D, 66A, 66B
NAD83 UTM Zone 14

PROJECT

AREVA

AREVA RESOURCES CANADA INC.
KIGGAVIK PROJECT

TITLE

REGIONAL GEOLOGY AND STRUCTURE

Golder Associates
Saskatoon, Saskatchewan

PROJECT	09-1362-0613	FILE No.	
DESIGN		SCALE AS SHOWN	REV. 0
GIS	MD	21/12/09	
CHECK	EM	21/12/09	
REVIEW	LG	21/12/09	

FIGURE: 3

G:\2009\1362\09-1362-0613 Areva Kiggavik Golder Internal Field Manual\GIS\Maps\Geotechnical\Kiggavik Site\GT\09-1362-0613-GEO-Regional Geology and Structure.mxd

Overlying the Woodburn Group is the Meso-Proterozoic Dubwant Supergroup (AREVA 2007). The Dubwant Supergroup can be subdivided into the Baker Lake, Wharton and Barrensland Groups, in ascending order. The Baker Lake Group consists of several sedimentary redbeds which are not exposed near the Kiggavik area, as well as the Christopher Island Formation, which may have intrusive equivalents present as syentic dykes within the Sissons area. The Wharton Group unconformably overlies the Baker Lake Group, and consists of the felsic volcanic Pitz Formation and a fluorite-bearing granite (AREVA 2007). The Pitz Formation is not present in the Sissons area, and the fluorite-bearing granite has been locally named the Lone Gull Granite. The Lone Gull Granite has been interpreted to be older than the Wharton Group. The Barrensland Group unconformably overlies the Wharton Group (AREVA 2007). It mainly consists of the Thelon Formation, which is exposed to the north of the Sissons area, and the MacKenzie diabase dykes, which are the youngest rocks within the project area.

1.3.1 Kiggavik Area Deposits

The Kiggavik area is located between two regional fault zones (AREVA 2007). The Thelon fault is located to the north, while the Sissons fault is located to the south. The Kiggavik deposits (Main, Centre, and East Zones) follow a local 65° east-northeast trending shear zone (Figure 3). Basement host rocks are composed of metasediments (mainly metaarkoses and metapelites overlain by orthoquartzites), and to a lesser extent altered granite and intrusives. Despite their considerable metamorphic overprint these rocks appear to be essentially flat lying with the foliation/bedding dipping north at 10º to 20º (Golder 1989).

The lithology in the Main Zone deposit consists of granite and metasediments, with the near vertical fault serving as the contact between the two units. At Main Zone, a northwest-southeast trending dyke of MacKenzie diabase cuts through the middle of the deposit, and is unmineralized (AREVA 2007).

Main Zone ore body consists of two parallel running major lenses, which are elongated along strike (AREVA 2007) and are approximately 20 m to 30 m thick, as well as two minor lenses. The major lenses trend east-northeast with a plunge of approximately 25°, and are controlled by the intersection of the shear zone with a near vertical northeast trending fault. Generally mineralization at Main Zone occurs to a depth of approximately 150 to 190 m below ground surface (mbgs). Mineralized zones are associated with an intensive alteration halo, characterized by desilicification and argillization with mainly illite and sericite.

Centre Zone is located approximately 600 m to the east of Main Zone, along strike of the shear zone (AREVA 2007). Mineralization occurs in two lenses, extending to a depth of approximately 100 mbgs. The footwall rock consists of an orthoquartzite horizon, which controls the dip of the mineralized lenses.

East Zone is located approximately 500 m to the east of Centre Zone, along strike of the shear zone (AREVA 2007). Mineralization within the East Zone is similar to that of Centre Zone, and occurs up to 60 mbgs.

1.4 Data Collection Program

Between May 21, 2009 and August 25, 2009, Golder was involved in a drilling program conducted at the Kiggavik property. In general, Golder's work relating to the open pit deposits included:

- selection, geotechnical core logging and orientation of pit wall investigation holes at Andrew Lake, Main Zone and Centre Zone;
- laboratory strength testing of selected rock core samples;
- in-situ hydrogeological testing to obtain information to assess the groundwater flow regime at each deposit;
- groundwater sample collection from the sub-permafrost bedrock to assess water quality; and
- instrument installation to measure the depth of permafrost at each deposit, as well as the groundwater pressure at each instrument location.

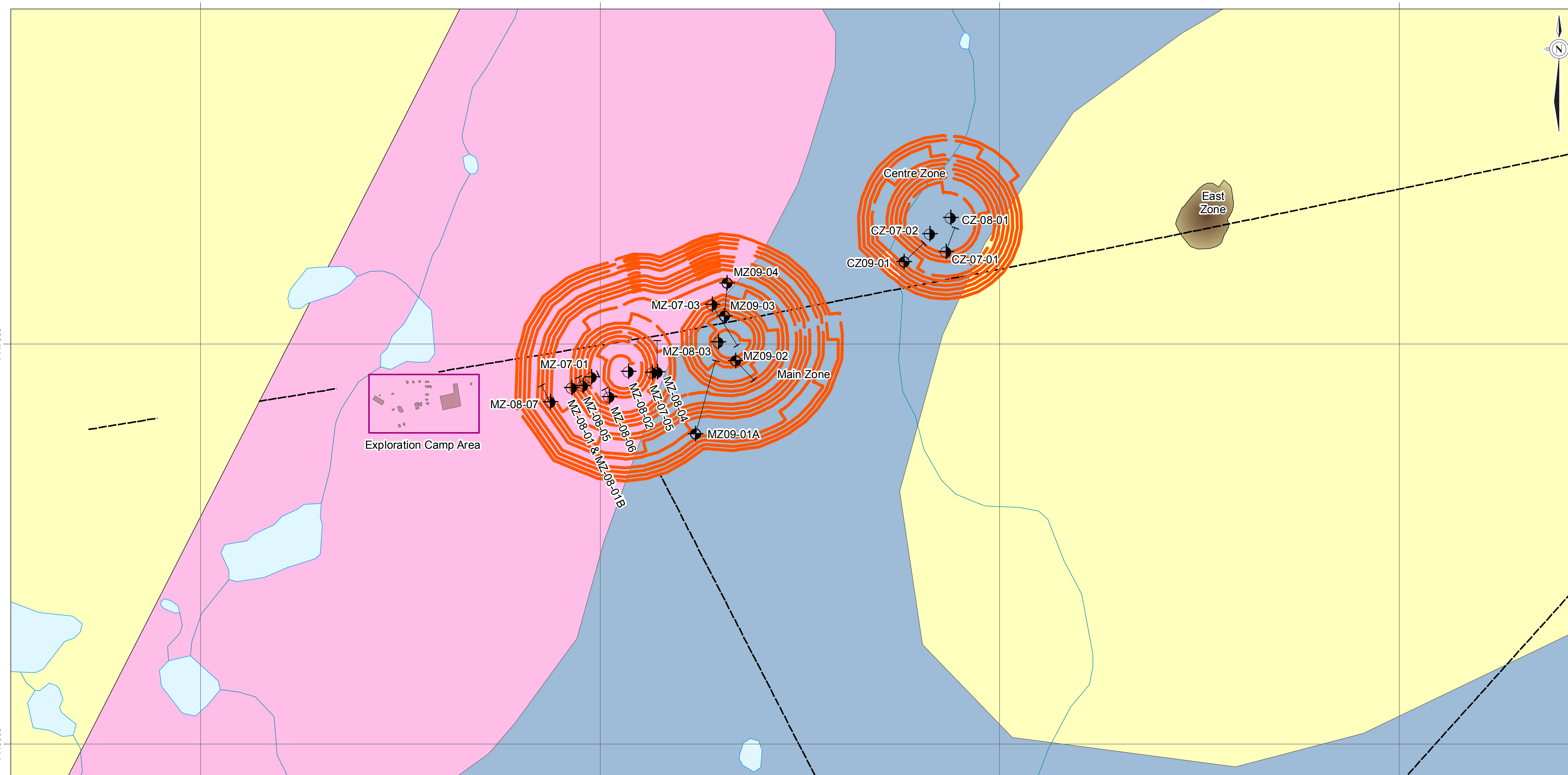
In total, 4 boreholes were drilled at Main Zone and 1 borehole was drilled at Centre Zone during the 2009 season. All 5 holes were geotechnically core logged, 4 of which had core orientation, 4 holes were tested for hydraulic conductivity, 2 instruments were installed, and 1 groundwater sample from bedrock was collected. Details of the drill program conducted at Main Zone and Centre Zone during the 2009 season are shown on Table 1. The locations of the boreholes are shown on Figure 4. A summary of the data collected during this field program can be found in Golder's report "2009 Kiggavik Geotechnical and Hydrogeological Investigation Data Report", submitted in draft to AREVA in November 2009 (Golder 2009c).

In addition to the information collected at Main Zone and Centre Zone in 2009, historical information was also considered when conducting the pit slope analysis at each location. SRK geotechnically logged the boreholes completed in 2007 and 2008. Details of these holes are also included in Table 1 for completeness, and are shown on Figure 4 as well. Geotechnical information collected by Golder in 1989 was considered in the analysis also, but is discussed in more detail within the appendices.

Table 1: Summary of Oriented Boreholes from 2007, 2008, and 2009 from Main Zone and Centre Zone

Borehole #	Year Drilled	Northing	Easting	Collar Elevation (masl)*	Azimuth	Dip	Drill Depth (mAH)	Vertical Depth (mbgs)	Bit Size
Main Zone									
MZ09-01A	2009 (G)	7146776	565239	173.73	015	-52	309	243	NQ3
MZ09-02	2009 (G)	7146959	565340	176.03	137	-75	260	251	NQ3
MZ09-03	2009 (G)	7147070	565312	184.48	004	-70	248	233	NQ3
MZ-08-04	2008 (SRK)	7146929	565143	179	244	-76	341	330	HQ/HQ3
MZ-08-05	2008 (SRK)	7146895	564956	187	333	-80	150	148	HQ
MZ-08-06	2008 (SRK)	7146867	565022	188	333	-75	96	93	HQ
MZ08-07	2008 (SRK)	7146854	564875	187.5	333	-67	126	116	HQ/NQ
MZ-07-01	2008 (SRK)	7146907	564969	185.93	065	-85	238	237	NQ3
MZ-07-03	2008 (SRK)	7147097	565282	187.85	148	-60	243	210	NQ3
Centre Zone									
CZ09-01	2009 (G)	7147207	565761	179.33	047	-75	270	261	NQ3
CZ-07-01	2007 (SRK)	7147336	565808	180.36	111	-60	135	117	NQ3

G = Golder (2009), SRK (2007/2008); Coordinates in UTM NAD 83 Zone 14, * Collar elevation for 2009 boreholes are an estimate based on point data collected from a LiDAR survey, masl = metres above sea level; mAH = metres along hole; mbgs = metres below ground surface



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Legend

- Borehole
- Historical Borehole
- Hole Trace
- Main and Centre Pit
- Camp Boundary
- Camp Infrastructure
- East Zone Pit
- Fault
- Granite (undivided)
- Orthoquartzite
- Supracrustal rocks (undifferentiated)

Reference:
Faults Courtesy of
AREVA Pre-feasability Study
Geological Structures Courtesy
of Natural Resources Canada 2002
NTS Mapsheets 56D, 66A, 66B
NAD83 UTM Zone 14

PROJECT
AREVA
AREVA RESOURCES CANADA INC.
KIGGAVIK PROJECT

TITLE
DRILLHOLE LOCATION PLAN
MAIN AND CENTRE ZONES

Golder Associates
Saskatoon, Saskatchewan

PROJECT		09-1362-0613
DESIGN		
GIS	MD	21/12/09
CHECK	EM	21/12/09
REVIEW	LG	22/12/09

FILE No.
SCALE AS SHOWN REV. 0

FIGURE: 4

G:\2009\1362\09-1362-0613 Areva Kiggavik Golder Internal Field Manual\GIS\Maps\Geotechnical\Kiggavik Site\GTI-09-1362-0613-3100-GEO-Drillhole Location Plan - Current and Historical - Main and Centre Zones.mxd

2.0 ENGINEERING GEOLOGY

The Kiggavik uranium deposits discovered to date include two major zones of interest, Main Zone and Centre Zone. The Main Zone mineralization generally occurs in two cigar-shaped zones which strike east-northeast and plunge at 25° in the same direction. The host rocks are predominantly north-northwesterly dipping quartz-feldspathic metasediments with intrusions of granite and minor feldspar porphyries in some places. A near vertical diabase dyke has intruded the centre of the Main Zone orebodies and strikes approximately south-southeast. Minor feldspar and quartz-feldspar porphyries are found interspersed within the Main Zone sequence. Pit slopes for the Main Zone will be predominantly in meta-arkose sediments with the exception of the southwest quadrant where the lower portion of the wall will be in granite.

For the Centre Zone, the relatively flat lying ore zone occurs in meta-arkoses and schists. A barren orthoquartzite horizon controls the form and separates the orebody into two lenses. The host rocks of the Centre Zone dip to the north-northwest and generally comprise meta-arkoses overlain by orthoquartzites which are interfingered with sericite schists. Quartz-feldspar and feldspar porphyries intrude the sequence along with minor lampropyres. The wall rocks of the proposed Centre Zone pit will predominantly be in meta-arkose, and quartzite with some schists which are generally confined to the upper portion of the north wall.

The derivation of stable economic pit designs for these deposits requires a detailed understanding of the major factors controlling slope stability. The major engineering geological factors influencing the stability of a slope can be listed as:

- The strength properties of the rock/soil materials.
- The properties of the fracture surfaces existing within the rock mass (i.e., orientation in relation to the dip direction of the slope, continuity, frequency, strength, spacing, etc.).
- The condition, quality and strength of the rock mass assemblages.
- Water pressures and/or permafrost conditions within the slopes and pit floor.

Based on the assumptions indicated earlier in the report, this study provides interpretations of the wall rock character.

2.1 Material Properties

Representative samples for the Kiggavik Main and Centre Zone pits were collected as part of the 2009 geotechnical investigation, and strength tested in the laboratory. The testing was conducted at the University of Saskatchewan's Rock Mechanics Laboratory. A total of 16 Unconfined Compressive Strength (UCS) tests were carried out for the Main Zone and Centre Zone samples. The measured material parameters included:

- UCS;
- Young's Modulus;
- Poisson's Ratio; and
- Bulk Density (based on physical measurements of the sample dimensions and weight prior to testing).

To complement the laboratory testing program a field estimate for rock strength according to the International Society for Rock Mechanics (ISRM) standards for assessing rock hardness (ISRM 1981) was undertaken during geotechnical core logging. Also, point load tests (PLT) for the estimation of the PLT index strength ($I_{s(50)}$) were carried out at the site on selected rock intervals. A total of 244 valid PLT tests were carried out on the Main Zone and Centre Zone rock types. Detailed results of the material strength testing program are presented in Appendix A.

Based on the results of the assessment, alteration is assumed to be the main control on rock strength, particularly in zones where faulting and ore zone halo alteration are more pronounced. The Main and Centre Zones show substantially less alteration, resulting in considerably stronger rock conditions. An example figure illustrating the measured material density with UCS strength is shown on Figure 5. The density determinations shown in Figure 5 show that there is a noticeable decrease in strength with decreasing density.

The wall rocks for the Kiggavik Main and Centre pits are generally competent to very competent rocks and exhibit brittle rock characteristics. The unaltered granites and metasediment rock units showed similar strengths, ranging from strong to very strong. The metasediments were shown to be slightly weaker than the granites, but are still very competent with respect to providing a stable wall for an open pit. Altered zones related to faulting or ore zone halo alteration are expected to reduce the rock strengths considerably. The ISRM weathering index (ISRM 1981) as assessed during the borehole investigation was used for assessment of weathering/alteration at the MZ/CZ. The potentially altered zones on the final pit walls were assessed for rock mass stability considerations. Results from the laboratory strength testing program are summarised in Table 2 below.

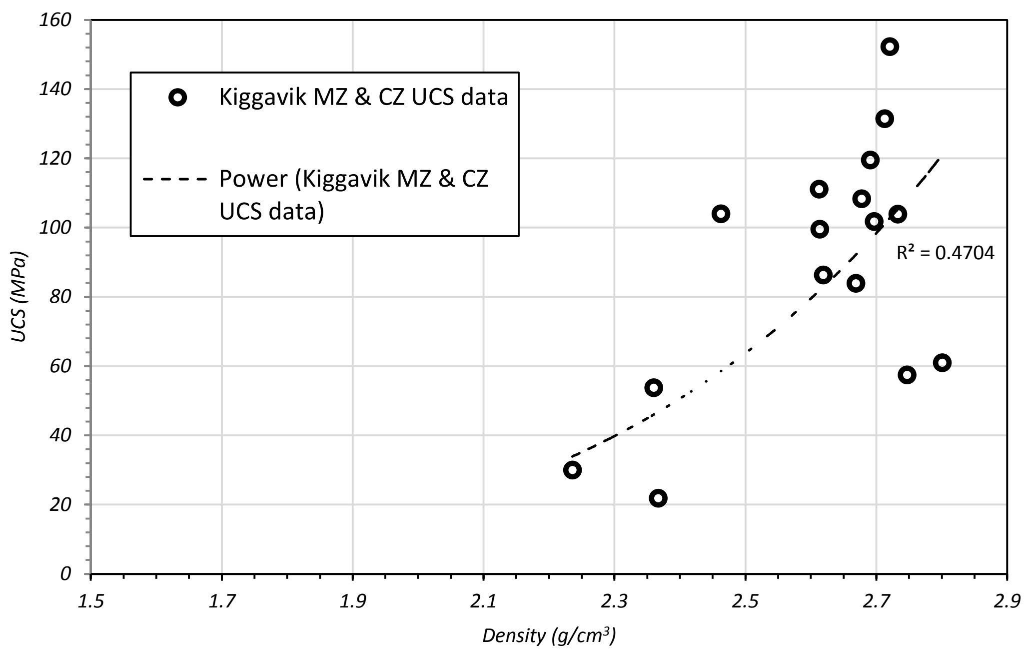


Figure 5: UCS versus Density for the 2009 Main Zone and Centre Zone Testing (see Appendix A)

Table 2: Kiggavik Main & Centre Zones - Summary of UCS Tests by Rock Type and Alteration

Rock Type	Alteration (ISRM)	# Tests	UCS (MPa)	Youngs Modulus (GPa)	Density (g/cm^3)	Poissons Ratio
Metasediments	Highly altered (W5)	1	21.8	6.6	2.37	0.01
	Fresh (W1)	8	98.4 +/- 26.1	44.6 +/- 6.3	2.69 +/- 0.10	0.17 +/- 0.02
Granite	Slightly to Moderately altered (W2 to W3)	3	55.9 +/- 27.0	23.9 +/- 18.2	2.42 +/- 0.22	0.10 +/- 0.03
	Fresh (W1)	4	112.3 +/- 28.5	45.1 +/- 3.4	2.64 +/- 0.05	0.15 +/- 0.01

MPa = mega pascal, GPa = giga pascal, g/cm^3 = grams per cubic centimetre

All rock strength testing was conducted on thawed core. The strength characteristics of rock types tested are unlikely to differ significantly in a frozen state due to very low moisture content/void ratios. The highly altered rocks such as the ore zone materials have a relatively high void ratio which implies that they are likely to be significantly stronger in a frozen state than in an unfrozen condition. The potential reduction in strength with thawing may contribute to unravelling of the rock on the slope face, which would be exacerbated in the blocky rock conditions associated with high alteration.

2.2 Rock Mass Quality

An assessment of rock mass quality was carried out for the geotechnical data obtained from the 2009 borehole program. Little historic information was available with regards to the engineering geology of the various rock units at the two sites. The rock mass quality assessment was carried out by calculating a Rock Mass Rating (RMR) index value (0 to 100) on a per run basis (typically 3 m in length). The methodology and discussion of the rock mass classification results are presented in detail in Appendix B.

2.2.1 Main and Centre Zone Rock Mass Quality

The Kiggavik Main Zone and Centre Zone rock masses have been classified together for lack of any additional information.

Geotechnical sections were interpreted based on the distributions of rock mass qualities in the 2008 and 2009 geotechnical boreholes. An example section is plotted on Figure 6. The lower quality and strength domains appear to be associated with faulting and mineralization horizons. At Kiggavik Main Zone and Centre Zone the geology and alteration profiles appear to be less complicated, with less mineralization and faulting transecting the pit walls and floor, than other deposit locations within the Kiggavik area. Faulting has been inferred to alter the rock strengths in the Main Zone pit walls following the east-northeast to east-southeast trend of the faults.

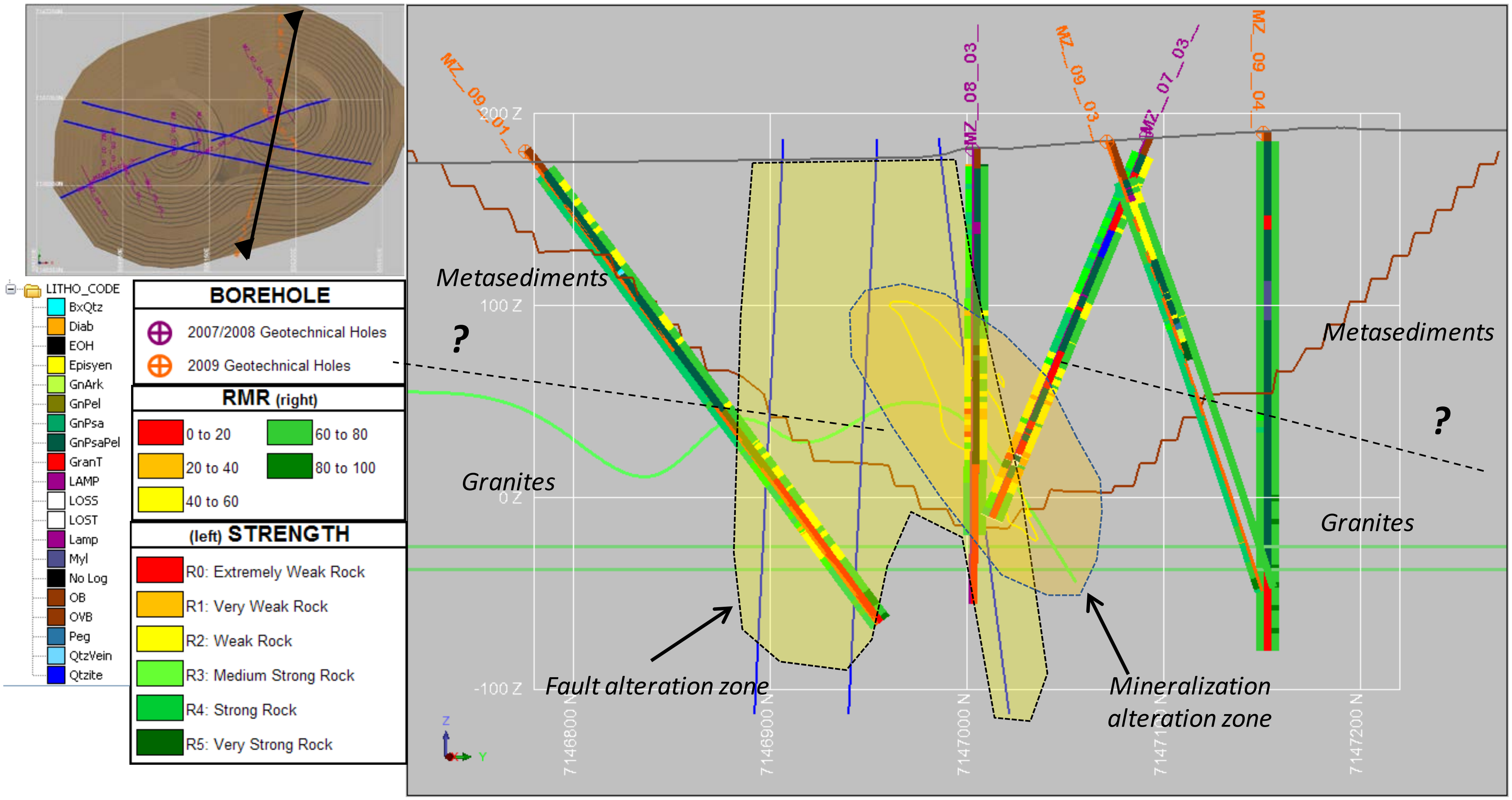


Figure 6: Generalized Geotechnical Section Illustrating the Inferred Fault and Alteration Zones Based on Main Zone, with Borehole Traces showing Rock Strength, Lithology and RMR

AREVA's geological model indicates that the Main Zone pit walls will be comprised of mainly strong, and fair to good quality granites in the western pit walls, and predominantly metasediments in the planned pit hanging wall and eastern walls. The distribution of rock types noticed in the geotechnical boreholes do however suggest some granites could transect the lower hanging wall and eastern wall in the pit. An upper metasediment and lower metasediment rock mass domain was assessed based on the relative increase in rock mass quality with depth. The granites generally showed a similar quality throughout away from the fault or mineralization zones.

The geological model for the Centre zone does not show any major faulting transecting the pit boundary; however the regional fault trace has been interpreted to intersect the upper footwall of the Centre Zone pit. A general alteration profile, inferred to be related to the mineralization alteration, was interpreted in the Centre Zone pit, possibly transecting the majority of the pit end walls, and the toe of the hang wall and footwall. This is a fairly conservative interpretation based on limited information.

Recommendations for RMR and strengths for the rock units at Kiggavik Main and Centre are given in Table 3. Rock mass qualities for the majority of the pit walls are generally fair to good, with strong to very strong intact rock strength. Some rock mass stability concerns might however be considered where faulting or mineralization alteration transects the pit boundaries, as the presence of moderately to highly altered rock reduces the strengths considerably.

Table 3: Kiggavik Main and Centre - Recommended RMR and Strengths Parameters

Rock Unit	RMR (1976)	Comment	Strength	Comment
Upper Metasediments (approximately less than 75 m depth)	**46 to 66** (fair to good)	Range of RMR from lower to upper bound limits for the 2009 Upper Metasediment data. (Appendix B)	**R4/R5** (strong to very strong)	Average UCS = 93.4 MPa and average $Is_{(50)}$ = 7.9 MPa for slightly altered (Appendix A)
Lower Metasediments (approximately greater than75 m depth)	**62 to 71** (good)	Range of RMR from lower to upper bound limits for the 2009 Lower Metasediment data. (Appendix B)	**R4/R5** (strong to very strong)	Average UCS = 93.4 MPa and average $Is_{(50)}$ = 8.4 MPa for fresh rock (Appendix A)
Granites (all)	**62 to 76** (good)	Range of RMR from lower to upper bound limits for the 2009 Granite data. (Appendix B)	**R4/R5** (strong to very strong)	Average UCS = 112.3 MPa and average $Is_{(50)}$ = 10.1 MPa for fresh rock (Appendix A)
Fault or Mineralization Altered Zones	**46 to 62** (fair to good)	Lower Bound RMR for all 2009 lithology data. (Appendix B)	**R3** (moderately strong)	Average UCS = 21.8 MPa for highly altered metasediments, and average UCS = 55.9 for slightly to moderately altered granites (Appendix A)

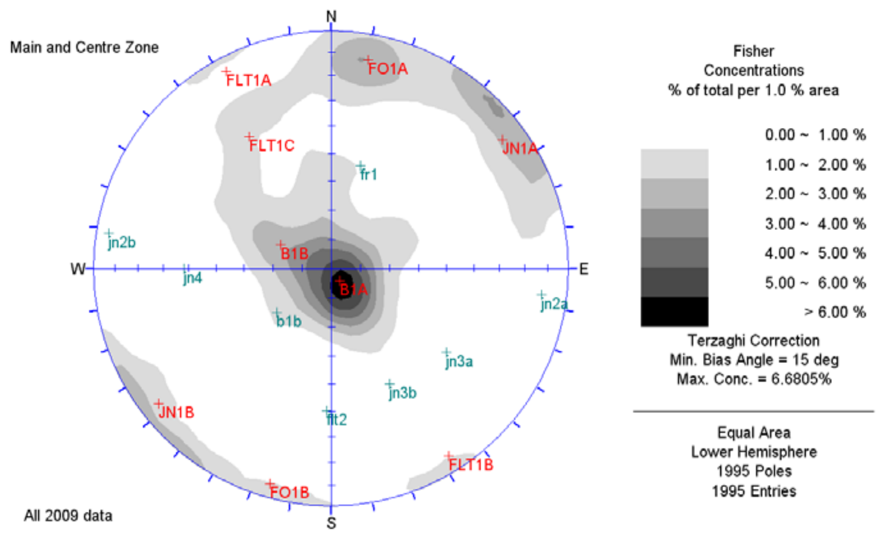
2.3 Rock Mass Fabric

Structural data was collected through oriented core drilling during the 2009 geotechnical investigation. A total of 1,995 oriented features were logged at Main Zone and Centre Zone. Historical data was utilized (Golder 1989 and SRK 2009) for assessment of major and minor discontinuity sets. The procedure and discussion related to the assessment of the rock mass structure is given in Appendix C.

The Kiggavik Main and Centre Zone rock masses were analysed together due to the lack of data in the Centre Zone pit. The quality of data was generally good at the Main Zone and Centre Zone sites, predominantly due to drilling conditions, as well as likely being associated with a more persistent and continuous rock mass fabric.

The 2009 boreholes were analysed by borehole, and selected major and minor discontinuity sets were assessed. These selected sets were also compared to sets identified by Golder (1989) and SRK (2009). The rock mass structure for the metasediments and granites showed quite a similar distribution of sets, therefore a single structural domain was considered to represent the entire rock mass at Main Zone and Centre Zone. Additional data collection at the Centre Zone pit, and western zone of the Main Zone pit would be required for delineation of structural domains and identification of major sets.

The selected major and minor sets for the Main and Centre Zones are plotted in Figure 7. A conventional naming system was used for the various sets (i.e., FO = foliation, JN = joint, FLT = fault, B = bedding). The numbers related to either joint set indicate the relative degree of intensity (1 = most dominant). Sub-sets, or two sets with an apparent relation were identified by the designation ‘A’ and ‘B’. An upper case set name indicates a major ser designation, while a lower case set name indicates a minor set designation. Major sets have been identified which follow the main trends in the data. Overall these major sets agree reasonably well to the Golder 1989 mapping data. A number of minor sets have also been identified, either using the 2009 data or based on the previous work by SRK (SRK 2009). In some instances, SRK have inferred a minor set as a major set based on their data distribution. Some discussion is given in Appendix C with regards to the bias due to drill orientation from both the 2008 and 2009 programs. This should be considered in future investigations.



Set	Dip	Dip Dir
JN1A	79	233
JN1B	81	52
FO1A	78	190
FO1B	83	16
FLT1A	83	152
FLT1B	82	328
FLT1C	55	148
B1A	5	326
B1B	19	115
b1c	24	51
flt2	50	2
jn2a	78	277
jn2b	84	99
jn3a	50	306
jn3b	45	333
jn4	52	90
fr1	37	196

Figure 7: 2009 Oriented Core Data (contoured) from Main Zone and Centre Zone with Selected MAJOR and minor Sets

The rock mass structure for the Main and Centre Zones appear to be comprised of 4 major sets denoted B1, FO1, JN1, and FLT1. Each of these sets was inferred to have two or more subsets (i.e., FO1A, FO1B). Set B1 is associated with bedding and foliation in the metasediments, but is also seen as a prominent joint set in the granitic rock units. Set FO1 is also associated with foliation or veining both within the metasediments and granites, possibly related to a deformational event. Set FLT1 is inclined to steeply dipping, trending near parallel to the main east-northeast trending shear zone associated with mineralization. Set JN1 is an inferred persistent joint set, trending nearly orthogonal to the main shear zone.

2.3.1 Rock Discontinuity Properties

At the Kiggavik Main and Centre Zones, with the exception of the highly altered rocks from faulting or the ore zone, all rock types exhibit strong rock characteristics. The stability of slopes excavated under these conditions will therefore be primarily controlled by the orientation, spacing, persistence and strength of the discontinuities that exist within the rock. The shear strength along discontinuities within the rock mass is an important characteristic with respect to slope stability. Typical shear strength values back analysed from the joint roughness and alteration conditions assessed for the rock units at the Kiggavik site are conservatively in the order of 35° (apparent friction), related to a planar and rough, clean to slightly altered discontinuity. Laboratory shear strength testing would be required to confirm these estimations.

Field investigations suggest that discontinuities at the site will be ice filled which will significantly increase the shear strength of these surfaces. Permafrost is therefore a favourable characteristic with respect to stability as shear strength values are likely to be increased to 40° or more. Even relatively low shear strength discontinuities, such as those altered by clays, are likely to be strengthened considerably by ice. Thawing of the exposed rock faces in the altered rock units could reduce the shear strength of the discontinuities, leading to unravelling of the blocky material.

2.3.2 Faults and Shear Zones

Regional fault traces as shown on Figure 3 are taken from information provided by AREVA (AREVA 2007), and are based on geophysical anomalies that were interpreted to be potential fault traces. The location of these features is approximate, but for the purpose of the pit slope analysis, they were conservatively assumed to have the potential to occur within the pit footprint where shown to do so.

The faults and major shear zones identified to date at Main and Centre Zones are continuous enough to impact overall slope designs, and are shown on Figure 8. Although data is limited, the dominant fault systems are interpreted to strike to 065° and 095° respectively. The 065° faults generally appear to dip steeply to the south, while the 095° system is inferred to dip at approximately 60° to the north. The intersection of these fault zones apparently controls the mineralization and linear shape of the orebodies. There is an apparent weakening and alteration of the rock mass up to several metres on either side of the inferred fault trace.

In the Centre Zone, 065° faulting is combined with postulated low angle shearing parallel to the bedding/foliation of the orthoquartzite meta-arkose contact.

Cross cutting faults which dip steeply and trend north-northwest or north-northeast have also been observed. These faults are considered to be relatively minor as they apparently do not affect the ore bodies except for possible minor variations in plunge. The north-northwest trending fault system has a similar trend to the diabase dykes.

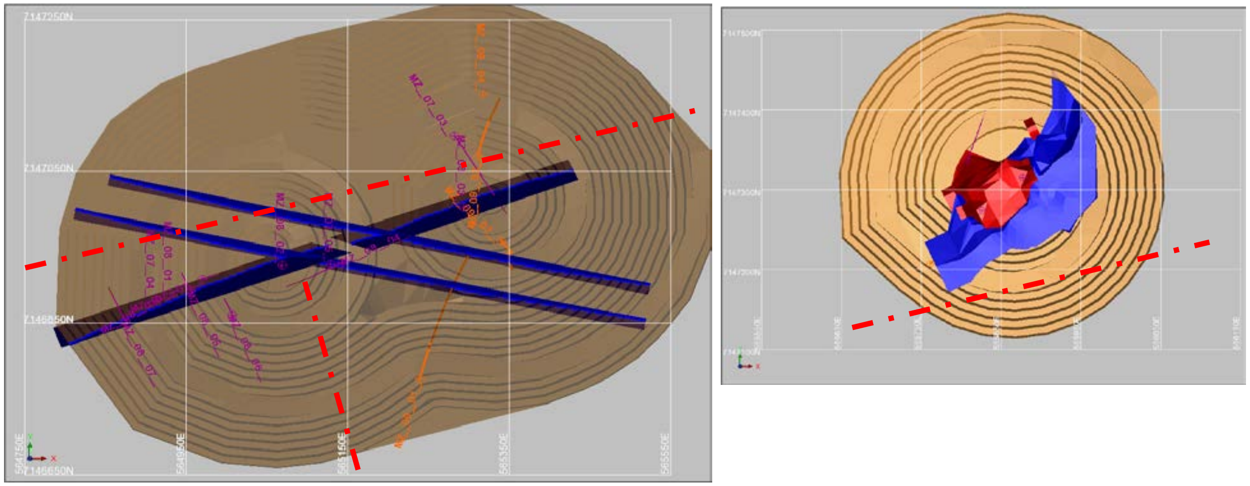


Figure 8: Main Zone and Centre Zone Fault Traces on the Pit Walls (blue) and the Inferred Regional Fault Trends (red dashed)

2.3.3 Joint Spacing

It is very difficult to estimate discontinuity set spacing from core log data. As shown in Appendix E, the discontinuity spacing for each set was estimated from the core logging data. This was done by identifying features that were part of the same discontinuity set, based on the stereonet analysis from each borehole. These sets were then grouped, and the average spacing between them was calculated. The average joint spacings estimated for Main Zone and Centre Zone ranged from 0.7 m to 4.3 m. It should be noted that the actual spacing between discontinuities may vary significantly from the average spacing presented.

2.4 Ground Water and Permafrost

During the 2009 field investigation, two instruments equipped with both a thermistor string and a vibrating wire piezometer were installed at Main Zone. Data collected from these instruments is discussed in detail in Golder's data report from the 2009 field investigations (Golder 2009c). General conclusions presented in this report indicated that the depth to permafrost is at approximately 210 metres below ground surface (mbgs) (-30 metres above sea level [masl]) at Main Zone.

Piezometer pressure readings documented and presented in Golder 2009(c) indicate that artesian pressure conditions exist in the groundwater below the Main Zone location. This corresponds to the artesian conditions recorded in both 1989 by Golder and 2007/2008 by SRK.

The significant conclusion is that the thick permafrost horizon acts as a confined aquifer. Water pressures, based on the installed vibrating wire piezometers, indicate that the piezometric head below the permafrost is at or above ground surface.

The source of this head is not known, but likely to be a large lake or lakes, with sufficient depth that a talik has formed beneath.

The Centre Pit is not expected to mine through the permafrost to expose this confined aquifer. The Main Zone pit will be excavated to ultimate depths at or slightly above the current base of permafrost. As a conservative measure, these basal pore pressures must be considered and addressed with respect to pit floor stability and lower pit wall stability.

2.5 Overburden Material Properties

The following information is cited from Golder 1989, as no overburden investigation has been conducted as part of the 2009 investigation. The investigation in 1989 centered on the Main Zone deposit and proposed pit outline.

> Overburden investigations were conducted for the purpose of evaluating waste dump foundations, overburden stripping in the vicinity of the open pits and haul road design. A drilling program was undertaken using the available flight auger coring rigs. A total of six boreholes were drilled in the vicinity of the mine area, all of which were terminated due to refusal at shallow depths (1.20 m to 2.30 m). It is assumed that refusal occurred when boulders within the glacial till were encountered. Samples from flight auger cuttings and frozen samples obtained with a CREEL core barrel were shipped to Golder Associates' Calgary laboratory. Selected samples were tested for grain size distribution, plasticity and water content. The results of these tests and associated borehole logs are summarized in Table 6 and Appendix C of the Golder 1989 report.
>
> Generally the overburden materials consisted of organic topsoil 0.10 m in thickness, developed over a glacial till. An exception was BH88-IMHR2 where, silty fine sand, 0.45 in thickness, overlies the glacial till. The glacial till varies in texture and composition from well-graded silty sand with some gravel and a trace of clay to well-graded gravelly sand with some silt and a trace of clay. The glacial till samples exhibited little to no plasticity. Oversize material in the till consists of boulders and cobbles which tend to be larger and more frequent with proximity to bedrock. The water content of the glacial till ranged from 8.4 to 13.1 percent. These water contents are all higher than the estimated 7 percent optimum water content for compaction.

3.0 PIT SLOPE STABILITY

3.1 Pit Slope Design Philosophy

The stability of cut slopes in an open pit is basic to the economics of the mining operation. If slopes are designed with very low probabilities of failure and are cut too flat, mining costs rise due to increased stripping ratios. On the other hand, if a slope has a high risk of failure, mining costs will also increase due to loss of production arising from failure. This loss of production can be the result of failure covering up ore reserves or failure involving access haul roads. Therefore, it is the function of a slope stability study to interact with mine planning and balance the above extremes, thereby deriving the most economical slope angle.

The stability of a rock slope is governed by the response of the geological environment and may be affected by environmental changes such as thawing or weathering that occur during the life of mine. The slope design assumptions are then adjusted as more information becomes available.

3.2 Slope Design Procedures

The basic components of a pit slope are the operating **bench height** and the **bench face angle** that can be achieved in the excavation of the lift. The lift height is a function of the type of excavation equipment that is being used. The bench face angle is normally a function of geotechnical factors such as the material strength or the structural discontinuities in the rocks, although, where no such controls are operative, it may be a function of the type of excavation equipment.

It is normal practice to establish **catch benches** on the slope to retain any loose material that may fall from either the immediate bench face, or from the upper part of the wall. This material could otherwise represent a hazard to any crew and equipment working at the toe of the slope. The width of the benches can depend upon a number of factors, including:

- required retention capacity;
- type of equipment available for cleaning the benches; and
- local mining regulations.

A typical minimum bench width is in the order of 8 m.

Where conditions are suitable it is common practice to place catch benches at vertical intervals of two or more operating lift heights, thereby creating what is termed a **"multi-bench slope configuration".** Typically, the vertical separation between catch benches on a multi-benched slope would be two operating lift heights (double benching) with triple benching used under special circumstances since the potential for ravelling material endangering personnel and equipment increases with the face height.

The angel between the toes of the benches on the wall is a basic element of slope design, and is termed the **"inter-ramp slope angle"**. The incorporation of haul ramps into a wall will result in a slope that has a shallower **overall wall angle** than the inter-ramp angle.

The actual slope design involves an iterative procedure involving the derivation of the design criteria as described above. These design criteria together with the access ramps requirements are incorporated into the mine design by the mine planners. The resulting pit design must then be reviewed by the slope designer in terms of overall stability. Operating criteria such as depressurization requirements, artificial slope support and excavation procedures required to attain the design slope angle must also be evaluated and incorporated into the design.

3.3 Basic Assumptions

In the formulation of the slope configurations a series of mine design assumptions were made on the basis of standard practices and the Nunavut Mine Health and Safety Regulations (Nunavut 2009), which require:

- a minimum bench width of 8 m;
- that the height of the working face be no greater than 2 m higher than the reach of the loading equipment, except when double benching is being performed; and
- where dual lane traffic exists, the haul road minimum width will be 3 times the width of the widest vehicle travelling it, plus a safety berm which will be the a minimum of ¾ of the largest tire height.

The mine design assumptions used include:

- mining will be by conventional open pit techniques;
- most of the materials in the pit will be fragmented by blasting prior to excavation;
- excavation will be by either from end loader or hydraulic excavator, and trucks will be used for hauling;
- the practice of multi-benching will be employed;
- the minimum catch bench width will be 8 m for single bench configuration and 12 m for double bench configuration;
- the minimum design haul road allocation will correspond to Nunavut regulations, and be dependent upon the largest machinery planned for travel on the haul road; and
- the final pit depth will be approximately 200 m at Main Zone, and 100 m at Centre Zone.

AREVA has indicated to Golder that 9 m mining bench heights are considered for the Kiggavik Main and Centre pits (9 m benches in waste rock and 3 m benches in ore). AREVA has indicated that Golder should also develop design recommendations for 12 m mining bench heights. Therefore, slope configurations have been formulated for both scenarios.

3.4 Geotechnical Benches

It is becoming common practice to periodically include extra wide catch benches at regular intervals in open pits, with the purpose to provide additional safety to operators and equipment, particularly on portions of the open pit walls not crossed by a ramp. The extra wide benches provide catchment to rock fall and debris, and can provide some flexibility to the mine plan.

The inclusion of geotechnical berms into the pit slope designs is appropriate at Kiggavik Main, where significant slope heights are developed. These benches are recommended to be 15 m to 20 m wide.

For planning purposes it is recommended that the Main Zone pit have a geotechnical berm at approximately 60 m below ground surface, to help collect ravel due to freeze thaw on upper slopes, and a minimum of one other geotechnical bench. This second geotechnical bench should be established at an elevation to optimize the safe drilling and installation of the initial pit slope and pit floor depressurization vertical drains, with a view that this bench requires permanent access for the remainder of mine life.

A third geotechnical bench in between these two can also be considered. However, the ramp may act as the geotechnical berm at some elevations. Inclusion of a third geotechnical bench mid-slope should be part of interactive discussions between the geotechnical engineers and mine planning team.

3.5 Failure Modes

Typical failure modes encountered in rock slopes are discussed below:

- **Planar Failure** can occur when a geological structure has a strike parallel or nearly parallel to the slope face and a dip shallower than the slope angle. The controlling factors in the plane shear analysis are the properties of the discontinuities, which express the likelihood of a fracture being long enough to reach full slope height, and the probability that the fracture is daylighted. To be considered kinematically viable, the fracture set must daylight, and its mean dip direction should be within 20° of the average dip direction of the pit wall in the pertinent design sector. **Step Path Planar Failure** is very similar to that of the plane shear. Sliding is assumed to occur along structures parallel to the slope, however, the failure surface is stepped. The step path model is based on the assumption that failure is due to the combined mechanism of sliding along surfaces dipping out of the slope and either separation along geological structures nearly perpendicular to the sliding surfaces or tensile failure of intact rock connecting the sliding surface. Because the step path failure model does not depend on the presence of a long, continuous geologic structure, it often has a wider applicability than does the plane shear model. Planar type failure modes have the potential to impact bench and overall slopes of the Main Zone and Centre Zone pits.

- **Simple Wedge** geometry is the result of two planar or nearly planar discontinuities intersecting within the slope to form a completely detached prism of material. A major wedge is defined by major structures of nearly continuous lengths. The weight of the material, coupled with hydrostatic forces, tends to drive the prism down the line of intersection for the two planes. Consequently, in order to be kinematically viable, the line of intersection of the two geologic structures must daylight. This implies that the plunge of the line of intersection must not only be shallower than the dip of the slope, but it must daylight within the slope face. **Step Wedge Failure** is very similar to the model for the simple wedge. However, structures that intersect to form the wedge are not necessarily single planar structures. Rather, it is assumed that combinations of different structural sets form at highly irregular intersections. Analysis for the mode of failure is combined with the simple step paths analysis on the assumption that any step wedge would be a combination of previously analysed step paths. Wedge type failure modes have the potential to impact benches and overall slopes at the Main Zone and Centre Zone sites.

- **Material Failure** or rotational shear failure through the rock mass is a model used to assess stability to slopes which are composed of material with low intact strength and sparse or non-existent geologic structure. Development of potential failure paths along discontinuities is precluded by failure through intact material. The low intact strength of the material increases the probability that shear stresses developed in even moderately high slopes will exceed that available shear strength. This model is sometimes applicable to intensely fractured rock that has enough different fracture orientations to make the failure path approximately circular. Material failure has been analysed for Main and Centre Zones.
- **Toppling Failure** involves rotation of columns or blocks of rock about some fixed base. This mode of failure can occur where a steeply dipping discontinuity forms a series of blocks or columns that trend near parallel to the strike of the slope. The continuity of joint sets has an important influence on toppling failure potential. Discontinuous fracture sets are unlikely to form columns, therefore reducing the potential for toppling. The Golder 1989 work suggested that the toppling joint sets were relatively continuous. However, the 2009 work with oriented core used to establish joint spacing on all sets suggests that sub-horizontal discontinuities are more prevalent than toppling set discontinuities. This suggests that while toppling is expected to occur locally, it does not appear likely that blocks with the right tall-narrow aspect ratio required for toppling are prevalent. Consequently, toppling is no longer considered the major kinematic control on slope design.
- **Ravelling Failure** or rock fall usually occurs in slopes where the geological structure is closely spaced. Structures of this type produce a rock mass characterized by multi-sized blocks that are easily detached at any free surface. The detached blocks accumulate as debris on benches and, if not removed they can form talus piles that overflow the bench. The practical approach to the ravelling problem is to minimize the amount of disturbed material by controlled blasting and excavation procedures. High broken ground in a permafrost environment is particularly susceptible to ravelling type failure during the period of thaw. The blocky character of the rock mass at the Kiggavik site is unlikely to be affected by ravelling failure with the localized exception of possibly the broken ground within faults/shear zones and in altered ore zone material.

3.6 Rock Mass Failure

A series of rock slope stability analyses were carried out for various rock mass configurations for the Main and Centre Zone pits. A limit equilibrium approach was followed using the software Slide (Rocscience Inc.). Rock mass slope design sectors were developed from the results of the rock mass classification and material strength assessments as discussed previously. Appendix D discusses the methodology and results of the stability analyses in detail.

Each pit was assumed to be comprised of two or three types of slopes, with varying degrees of rock mass alteration. The altered rock slopes included zones of the rock mass in close proximity to interpreted faulting or ore zone halo alteration. The non-altered rock zones assumed a generalized distribution of lithological domains (i.e., upper metasediments, lower metasediments, granites) with depth. The slope configuration models, denoted Type 1, Type 2, and Type 3, are described as follows:

- ***Type 1 – Full Slope Alteration:*** Assumes the majority of the slope is altered and weakened due to the presence of faulting or ore zone halo alteration. For the Kiggavik Main Zone slopes, Type 1 slopes have been assumed and assigned to pit walls where faulting and associated alteration halos, in particular where more than one fault in close proximity, are interpreted to control the material properties of the open pit wall. At Kiggavik Centre, no major fault zones have been identified, however the ore zone halo alteration is interpreted to potentially affect the majority of the slope height in the end walls.

- ***Type 2 – Lower Slope Alteration:*** Assumes the lower slope, being the lower one half to one quarter of the overall slope height (depending on wall inclination), is altered and weakened due to the presence of faulting or ore zone alteration halo. This includes fault zone exposures on the lower portions of the pit walls (Main Zone), or interpreted ore zone alteration halos that extend to the lower portions of the walls and to the open pit floors (all pits). The upper portions of the slope, away from the faulting or alteration halos, are assumed to be relatively unaltered country rock of better strength and quality.
- ***Type 3 – No Slope Alteration:*** Assumes predominantly non-altered rock mass conditions throughout the full slope height. Included are zones of the pit wall away from faulting and mineralization. For the Main Zone pit, the pit floor can still be assumed altered and weaker due to faulting or due to alteration. These slopes can include the paleo-weathered metasediments.

The Kiggavik Main Zone pit is interpreted to be predominantly Type 2 or Type 3 rock mass slopes. The three identified major fault trends strike east-northeast-west-southwest to east-southeast-west-northwest. In the pit walls on strike with these trends, the rock mass is inferred to be Type 1. The Kiggavik Centre Zone pit is interpreted to be predominantly Type 1 or Type 2 slopes due to the relatively shallow depth of excavation and the inferences made on the extents of the ore halo alteration. This is a relatively conservative estimation of the pit wall quality; however the shallow depth of excavation significantly reduces the likelihood of rock mass instability. Figure D1 (Appendix D) illustrates the inferred rock mass domains at the Main and Centre pits.

The results of the stability analysis for the Main Zone pit are presented on Table 4. A maximum slope angle of 53° is recommended in the Type 1 altered slopes, and with decreased alteration, the allowable slope angles increase considerably. Significant material failures are not expected in the good quality, strong rock units which are inferred to comprise the majority of the Main Zone pit. The relatively shallow depth of the Centre Zone pit, and reasonable rock mass qualities also suggest that deep seated rock mass failure would not be considered a risk at the Centre Zone pit for overall slope angles of less than 55° in altered ground and 60° in predominantly unaltered ground.

Table 4: Kiggavik Main Pit - Recommended Maximum Slope Angles for Rock Mass Slope Stability, Full 200 m Slope Height

Rock Mass Slope Configuration		Overall Slope Angle (°) Net of Upper and Lower Slope (0 m to 200 m)	Upper Slope Angle (°) (0 m to 125 m)	Lower Slope Angle (°) (125 m to 200 m)	Perceived Risk/Sensitivity of the Design to Variations in Strength
Type 1	Full slope alteration (toe to crest)	50	50	50	Moderate risk
Type 2	Lower slope alteration	58	60	55	Low risk
Type 3	No slope alteration (toe to crest)	60	>60	>60	Low risk

3.7 Kinematics of Slope Failures

Kinematic analyses were carried out on the various slope kinematic design sectors for the Main Zone and Centre Zone pits using the rock mass structural data derived from the 2009 program.

A stereographic projection (stereonet) is used to graphically represent the three-dimensional relationship of the bedding surfaces, faults, joint sets or combination of joint sets and to evaluate their relationships to proposed slope orientations. Discontinuities that are oriented such that they could give rise to slope failure are also identified.

3.7.1 Kinematic Design Sectors

A stereographic study of the kinematics of failure has been used to define those fractures or discontinuities which are unfavourably oriented with respect to the slope design sections at the Kiggavik project. This study was undertaken with the aid of stereonets and AREVA's 3-D fault traces and structural interpretation of the orebody.

The mean pole plots of discontinuities along with possible slope geometries have been plotted on a stereonet. This was done for a range of slope orientations and slope angles, coincident with the design sectors shown on Figure 9. A number of zones are described on the stereonet defining discontinuity orientations which are most likely to give rise to failure. If these zones are coincident with the pole points or wedge intersection lines of discontinuity sets for a given slope geometry, failure is kinematically possible.

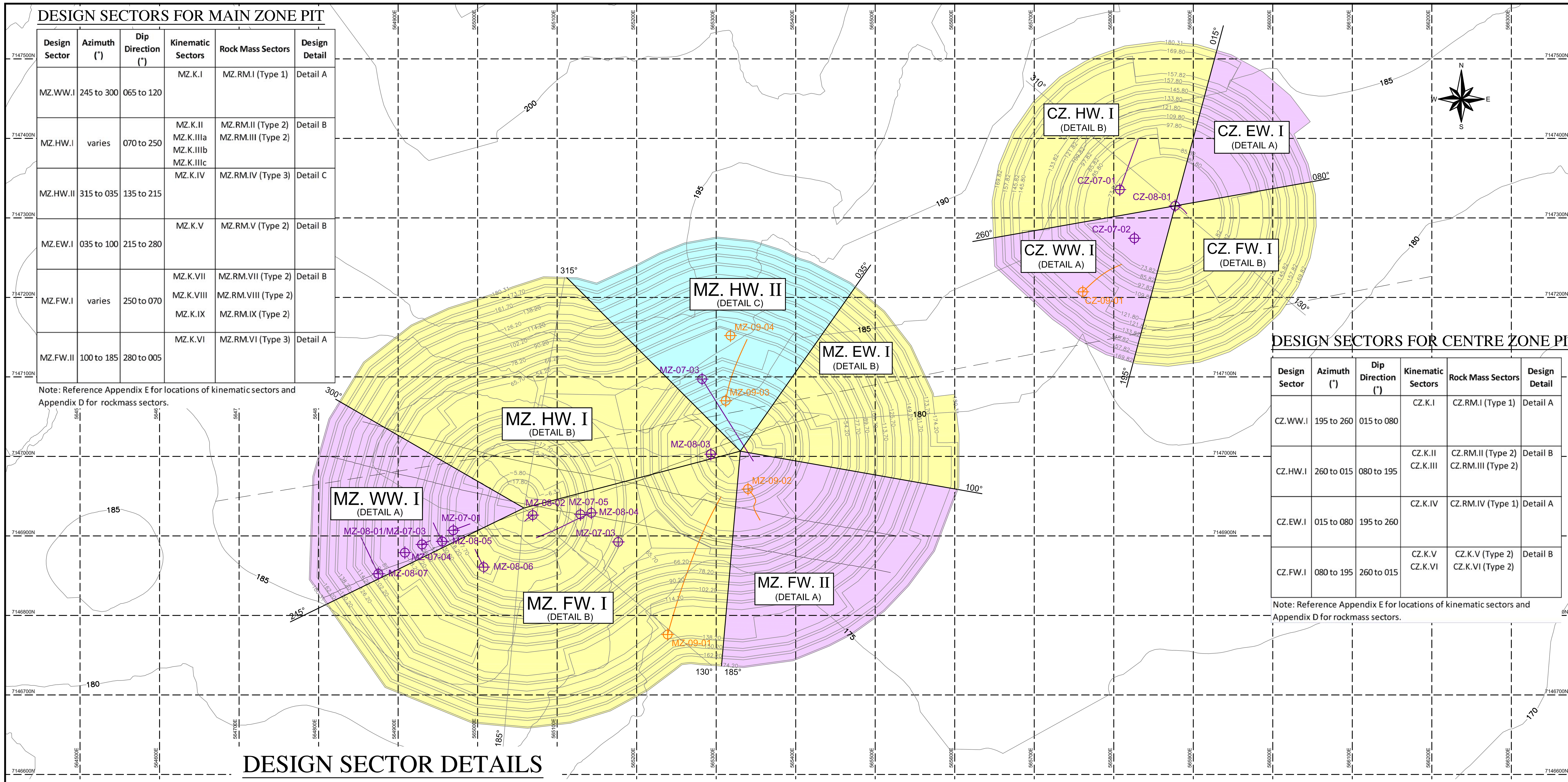
To assist stability analysis the Kiggavik pits have been divided into a series of tentative design sectors. For the Kiggavik Main and Centre pits, the design sectors were selected based on pit geometry as well as the inferred rock mass conditions comprising the pit wall. Because of the central pillar at the Main Zone pit complicating the pit geometry, a total of 13 design sectors were analysed, although a number of these sectors shared the same slope orientation. A total of 7 kinematic design sectors were analysed for the circular Centre Zone pit.

Variations in structural trends and lithologies are represented within each design sector. However the major controlling structural systems in the vicinity of the pits are considered very consistent. For this reason the most important factor in selecting design sectors is the orientation of a proposed pit wall with respect to these major structural systems. It is expected that these sectors may be subdivided on the basis of structural regimes as a more detailed understanding of the discontinuity systems evolves.

The distribution of the tentative design sectors are summarised in Figure 9. These sectors may be re-evaluated due to mine planning considerations, and in particular pit shape.

3.7.2 Kinematic Analyses

A series of stereo diagrams have been constructed showing the major and minor discontinuity sets which have been assumed for the structure associated with the Main Zone and Centre Zone pit walls. Potential wedges, planar and toppling failure sets have been identified on Figures E1.1 through E1.9 (Appendix E).



DESIGN SECTORS FOR MAIN ZONE PIT

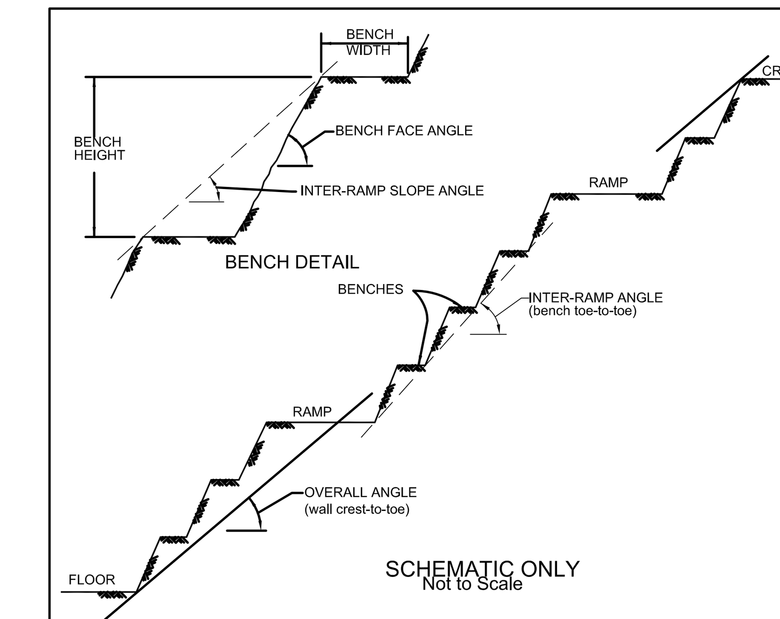
Design Sector	Azimuth (°)	Dip Direction (°)	Kinematic Sectors	Rock Mass Sectors	Design Detail
MZ.WW.I	245 to 300	065 to 120	MZ.K.I	MZ.RM.I (Type 1)	Detail A
MZ.HW.I	varies	070 to 250	MZ.K.II MZ.K.IIIa MZ.K.IIIb MZ.K.IIIc	MZ.RM.II (Type 2) MZ.RM.III (Type 2)	Detail B
MZ.HW.II	315 to 035	135 to 215	MZ.K.IV	MZ.RM.IV (Type 3)	Detail C
MZ.EW.I	035 to 100	215 to 280	MZ.K.V	MZ.RM.V (Type 2)	Detail B
MZ.FW.I	varies	250 to 070	MZ.K.VII MZ.K.VIII MZ.K.IX	MZ.RM.VII (Type 2) MZ.RM.VIII (Type 2) MZ.RM.IX (Type 2)	Detail B
MZ.FW.II	100 to 185	280 to 005	MZ.K.VI	MZ.RM.VI (Type 3)	Detail A

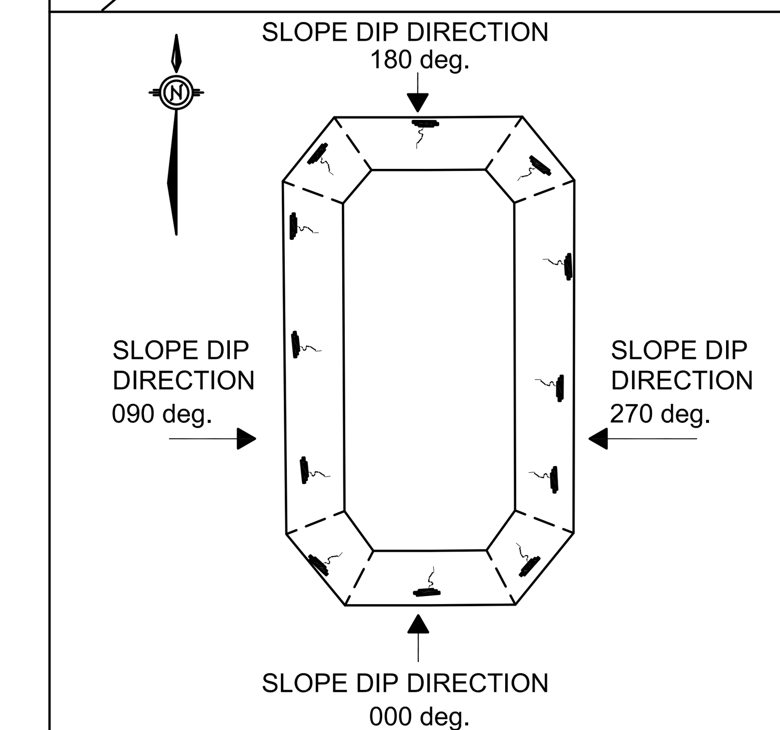
Note: Reference Appendix E for locations of kinematic sectors and Appendix D for rockmass sectors.

DESIGN SECTORS FOR CENTRE ZONE PIT

Design Sector	Azimuth (°)	Dip Direction (°)	Kinematic Sectors	Rock Mass Sectors	Design Detail
CZ.WW.I	195 to 260	015 to 080	CZ.K.I	CZ.RM.I (Type 1)	Detail A
CZ.HW.I	260 to 015	080 to 195	CZ.K.II CZ.K.III	CZ.RM.II (Type 2) CZ.RM.III (Type 2)	Detail B
CZ.EW.I	015 to 080	195 to 260	CZ.K.IV	CZ.RM.IV (Type 1)	Detail A
CZ.FW.I	080 to 195	260 to 015	CZ.K.V CZ.K.VI	CZ.K.V (Type 2) CZ.K.VI (Type 2)	Detail B

Note: Reference Appendix E for locations of kinematic sectors and Appendix D for rockmass sectors.





Explanation of Slope Dip Direction
SCHEMATIC ONLY
Not to Scale

LEGEND:

- ANDW-08-01 — DRILLHOLES FROM 2007 & 2008 INVESTIGATIONS (SRK)
- AND-09-01 — DRILLHOLES FROM 2009 INVESTIGATIONS (GOLDER)
- REGIONAL FAULT TREND (REF. 1)
- INFERRED FAULT INTERSECTION ON PIT WALL (REF. 2)

REFERENCES:

1. BASED ON REGIONAL GEOLOGY DRAWINGS (MAIN TEXT FIGURES 4 & 5)
2. AREVA GEOLOGICAL MODEL (2009).

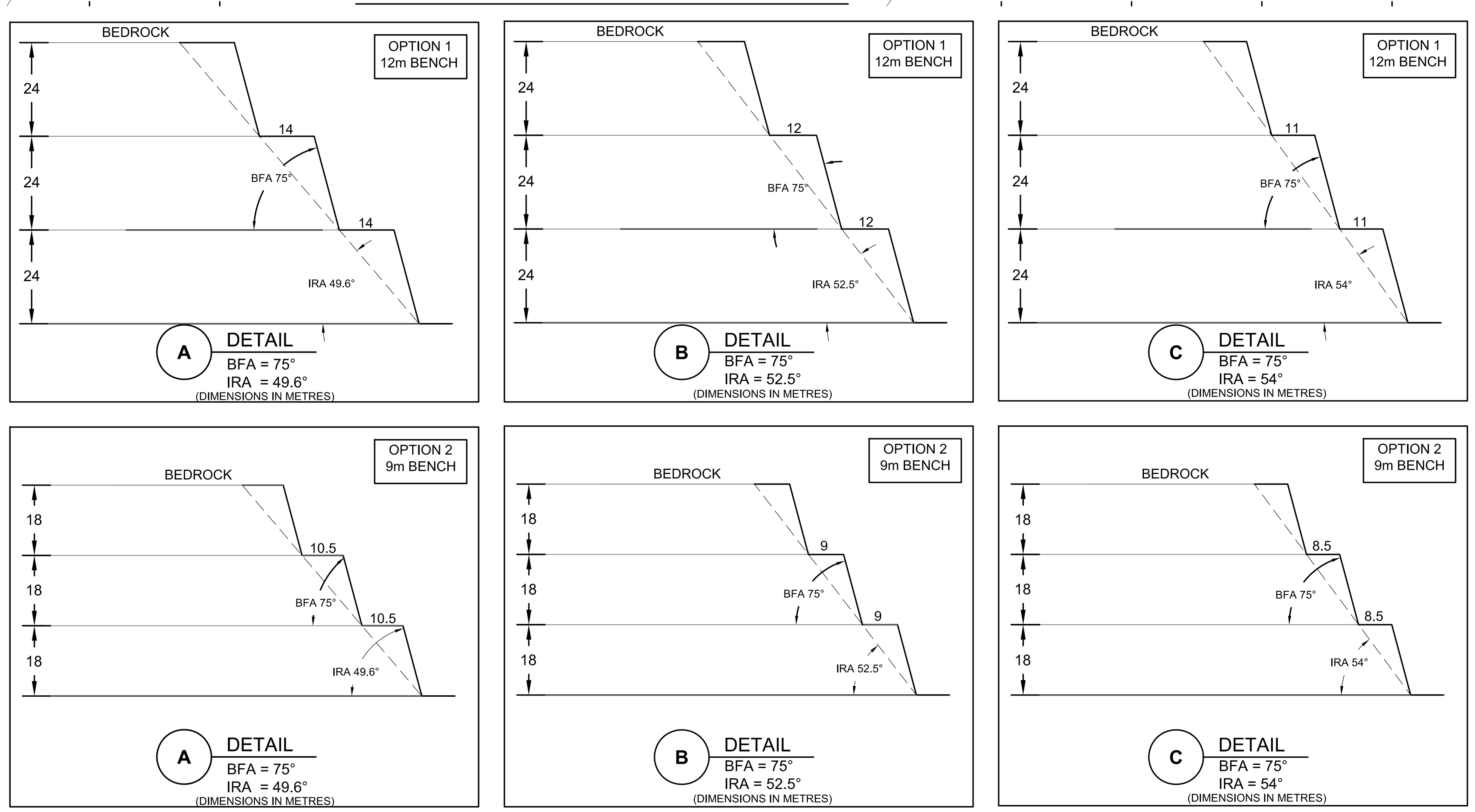
NOTES:

1. SLOPE DESIGN CONSIDERATIONS ARE GIVEN IN THE MAIN TEXT.
2. SLOPE DESIGN DETAILS ARE HIGHLY INFLUENCED BY WEAKER GROUND CONDITIONS ASSOCIATED WITH ZONES OF ALTERATION. THE LOCATION OF FAULTS AND ORE HALO ALTERATION ZONES ON PIT WALLS SHOULD BE CONFIRMED.
3. CONSIDERATION SHALL BE MADE FOR GEOTECHNICAL CATCH BERMS FOR PROTECTION FROM ROCKFALL HAZARDS. SEE RECOMMENDATIONS IN THE MAIN TEXT.
4. OVERBURDEN TOE BERM SHOULD BE SETBACK A MINIMUM OF 10m FROM THE BEDROCK PIT CREST. THE OVERBURDEN SHOULD BE PROTECTED FROM EROSION WITH PLACEMENT OF WASTE ROCK COVER OF MINIMUM 2.5m THICKNESS. A WATER DIVERSION DITCH SHOULD BE INCLUDED ON THE OVERBURDEN SLOPE CREST AND TOE.

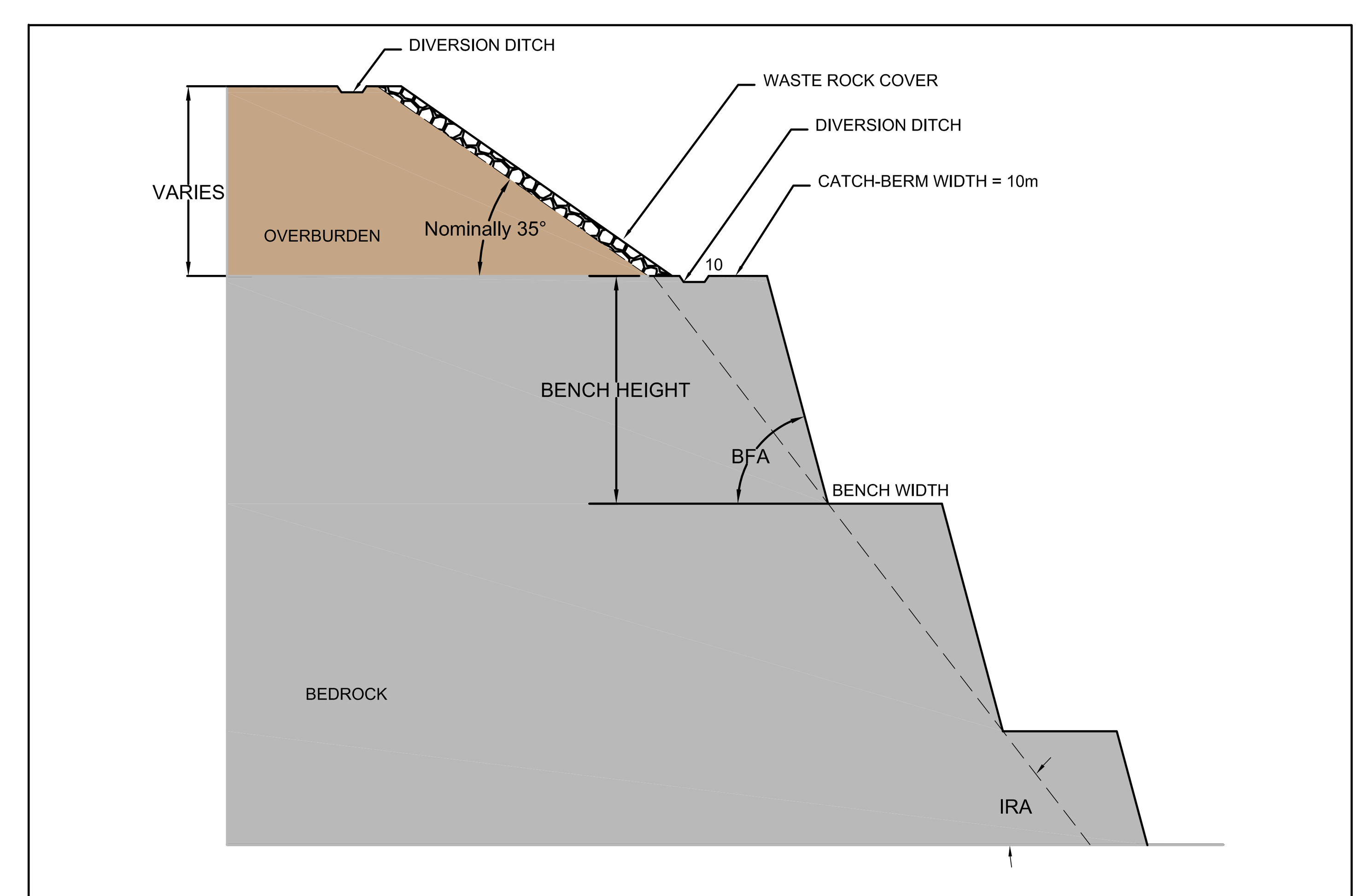
THIS DRAWING MAY HAVE BEEN REDUCED. ALL SCALE NOTATIONS INDICATED (i.e. 1:1000 etc) ARE BASED ON A1 FORMAT DRAWINGS

SCALE 1:3000 — METRES

DESIGN SECTOR DETAILS



DETAIL A: BFA = 75°, IRA = 49.6° (DIMENSIONS IN METRES)
DETAIL B: BFA = 75°, IRA = 52.5° (DIMENSIONS IN METRES)
DETAIL C: BFA = 75°, IRA = 54° (DIMENSIONS IN METRES)



DETAIL D - BEDROCK / OVERBURDEN INTERFACE
(DIMENSIONS IN METRES)

PROJECT	
AREVA	AREVA RESOURCES CANADA INC. KIGGAVIK PROJECT
TITLE	MAIN ZONE AND CENTRE ZONE PROPOSED PIT SLOPE DESIGN PARAMETERS

Golder Associates — Saskatoon, Saskatchewan, Canada

PROJECT No.	09-1362-0613		FILE No.	0913620613AB0E7-JS.dwg
DESIGN	BEC	12/03/2009	SCALE AS SHOWN	REV. A
CAD	JS/SIB	12/22/2009	DRAWING No.	
CHECK	EAM	12/22/2009		9
REVIEW	MR	12/22/2009		

PLOT DATE: February 23, 2011
FILENAME: G:\2009\1362\09-1362-0613 Areva Kiggavik Golder Internal Field Manual\CAD\Kiggavik Site\0913620613AB0E7-JS.dwg

4.0 PIT FLOOR HEAVE MITIGATION

A simple analysis was carried out to assess the depth at which floor heave due to artesian water pressures would become an issue at the Main Zone and Centre Zone sites. These analyses follow the assumption that the presence of pressurized water might be encountered below the permafrost boundary. At either site, during mining of the open pit, the excavation of the rock mass will reduce the overburden pressure counteracting the water pressure at depth. If a critical depth is reached without the reduction in water pressure, the potential exists for floor heave and water infiltration into the base of the pit. This could present considerable problems for mine operations, as well as potentially reducing the stability of the pit walls. Floor heave analysis details and results are given in Appendix D.

The recommended critical depths of mining for floor heave drainage considerations are given on Table 5. Both the critical depths assuming the self-weight resistance to floor heave, as well as critical depths assuming failure through the rock mass (including a rock mass strength component) are given. The strength components of the rock mass are assumed to be an additional factor of safety, and floor heave mitigation measures should be considered at the depth of mining related to the self-weight of the rock. Measures could include establishing a geotechnical bench at this depth for depressurization. If floor drainage systems are proven to be effective at reducing the water pressures below the permafrost boundary, full depth of mining would be achievable without risk of floor heave.

Table 5: Recommended Critical Depths at which Remedial Measures Such as Vertical Pressure Relief Drains May be Required to Prevent Floor Heave – Self Weight and Rock Mass Failure

Site	Planned Depth of Pit (m)	Expected Depth to Permafrost (m)	Critical Depth[(a)] (m) – Self Weight	Critical Depth[1] (m) – Rock Mass Failure
Kiggavik Main	200	210	110	150
Kiggavik Centre	105	210	-	-

a = Critical depth of the pit floor at which floor pore water pressure reductions should start assuming self weight resistance only. Consideration should be given to establishing a geotechnical bench at this depth for depressurization.

5.0 ROCK SLOPE DESIGN RECOMMENDATIONS

The rock slope design configurations for the Main Zone and Centre Zone pits are plotted on Figure 9. The pit slope design recommendations for the Kiggavik pits are summarised on Tables 6 and 7. Included on this table are the azimuth and dip direction ranges for each design sector, as well as the kinematic and rock mass design sectors, as discussed in Appendix E and Appendix D respectively. The recommended bench face angles (BFA), inter-ramp angles (IRA), bench heights and berm widths are also presented along with the design considerations for each wall. Discussions pertaining to the main design considerations for each sector are also given.

Following issue of the first draft of the Main Zone and Centre Zone pit slope recommendations in early December 2009, AREVA asked for slope design recommendations assuming a vertical bench height of 9 m at the Kiggavik pits, rather than the 12 m separation assumed in the initial analysis. Golder has not carried out any detailed review with purpose to optimize this different bench configuration. Tables 6 and 7 below present the inter-ramp angles for 9 m high benches under the following additional assumptions:

- The recommended bench face angles for the 9 m and 12 m benches are the same.
- The minimum 8 m catch bench width must still be respected.
- Double benching can be achieved given the interpreted better rock mass quality. Given that the bench geometries at the Kiggavik pits are considered to be mainly controlled by kinematics, catch bench widths are adjusted to achieve the same inter-ramp angles as recommended for the 12m bench height case.

An initial slope design for the Main Zone pit was conducted by Golder in 1989 (Golder 1989). The main failure mode within this initial assessment was thought to be controlled by toppling. After analysis of the 2009 data, with the historical data from 1989 and 2007/2008, toppling is no longer considered to be a major failure mechanism, due to joint spacing. Mechanisms thought to be more prevalent now are planar and wedge failure modes. However, in 1989, inter-ramp slope angles of 50° to 55° were suggested, which is similar to those suggested in this report. Actual 1989 slope recommendations were 47° to 51°, for 18 m high (double benched) slopes. The 2009 inter-ramp angles are slightly steeper, based on the new information.

6.0 OVERBURDEN SLOPE DESIGN

There have been no geotechnical investigations for the open pit overburden slopes. The overburden slope design presented in Golder 1989, can be considered applicable for both proposed pit locations until investigations can be carried out to confirm conditions. Generally, the overburden material will be stripped back from the pit crest and trimmed to a smooth slope. For slopes constructed primarily in till, a nominal angle of 35°, with an appropriate catch bench, is recommended. Material properties of the overburden have been estimated from those presented in the Golder 1989 report, and may vary between sites. Flatter slope angles may be required if significant thicknesses of loose alluvial materials are encountered. The overburden slope should be unbenched and smooth with a layer of good quality waste rock a minimum of 2.5 m thick placed against it. The waste rock placement may be subject to constructability controls. This configuration will help control erosion and drainage, and facilitate reclamation and revegetation of the slope at the end of mine life. A catch bench, a minimum 12.5 m in width should be incorporated at the toe of the overburden slope. This bench will allow room for slope maintenance, drainage and control over the life of the pit. On higher overburden slopes, the protective waste rock cover may need to be wider at the base, for reasons of stability and constructability.

Perimeter diversion ditches are recommended at the crest and the toe of the overburden slope. The diversion ditch at the crest of the slope will intercept surface drainage, snow melt, or precipitation runoff before it enters the pit. The ditch should be set back an appropriate distance from the crest to mitigate the potential for piping failure between the ditch and the overburden slope. This will help with erosion control of the overburden slopes, and overall water management within the pit. The diversion ditch at the toe of the overburden slope will intercept any drainage from the overburden slope or precipitation runoff, and will help control overall water management within the pit. These controls will be of greatest use during the freshet period.

The general proposed configuration of the overburden slopes are shown as part of the design figure, presented in Figure 9.

Table 6: Main Zone - Pit Slope Design Recommendations

Design Sector	Azimuth (˚)	Dip Direction (˚)	Kinematic Sectors	Rock Mass Sectors	Wall	BFA (˚)	IRA (˚)	Bench Height (m)	Berm Width (m)
MZ.WW.I	245 to 300	065 to 120	MZ.K.I	MZ.RM.I (Type 1)	Inferred full slope alteration. Transected by faults striking ENE to ESE. Altered rockmass conditions control the overall design. Steep BFA and wider berm selected to contain ravelling.	75	50	24	14
						75	50	18	10.5
MZ.HW.I	varies	070 to 250	MZ.K.II	MZ.RM.II (Type 2)	Main kinematic controls include set FLT1C (55 deg) on the upper walls, and sets FO1A (77 deg) and JN1A (77 deg) controlling BFA on lower walls. Inferred lower slope alteration due to faults transecting near lower slope and central pit pillar required to control potential ravelling.	75	53	24	12
			MZ.K.IIIa	MZ.RM.III (Type 2)					
			MZ.K.IIIb			75	53	18	9
			MZ.K.IIIc						
MZ.HW.II	315 to 035	135 to 215	MZ.K.IV	MZ.RM.IV (Type 3)	Kinematically favourable wall. Main kinematic controls include FO1A (77 deg) controlling BFA. Single major wedge at 55 deg. Little inferred alteration due to faulting on lower slope.	75	54	24	11
						75	54	18	8.5
MZ.EW.I	035 to 100	215 to 280	MZ.K.V	MZ.RM.V (Type 2)	Main kinematic control at set JN1A (77 deg) controlling BFA. Major wedges at 64 deg to 78 deg. Single ENE trending fault possibly cross-cutting wall, inferred alteration limited to lower slope in proximity to mineralization.	75	53	24	12
						75	53	18	9
MZ.FW.I	varies	250 to 070	MZ.K.VII	MZ.RM.VII (Type 2)	Main kinematic controls on BFA include sets JN1B (81 deg), FO1B (83 deg), and FLT1B (82 deg). Major wedges dipping at 63 deg to 82 deg. Inferred lower slope alteration due to ENE to ESE striking faults requiring limitations on berm width in lower slope.	75	53	24	9
			MZ.K.VIII	MZ.RM.VIII (Type 2)		75	53	18	9
			MZ.K.IX	MZ.RM.IX (Type 2)					
MZ.FW.II	100 to 180	285 to 005	MZ.K.VI	MZ.RM.VI (Type 3)	Inferred full slope alteration. Transected by faults striking ENE to ESE. Altered rockmass conditions control the overall design. Steep BFA and wider berm selected to contain ravelling.	75	50	24	14
						75	50	18	10.5

Table 7: Centre Zone - Pit Slope Design Recommendations

Design Sector	Azimuth (˚)	Dip Direction (˚)	Kinematic Sectors	Rock Mass Sectors	Design Considerations	BFA (˚)	IRA (˚)	Bench Height (m)	Berm Width (m)
CZ.WW.I	195 to 260	015 to 080	CZ.K.I	CZ.RM.I (Type 1)	Inferred full slope alteration with ore halo alteration following main shear zone trend (striking ENE). Steep BFA and wider berm selected to contain ravelling.	75	53	24	14
						75	53	18	9
CZ.HW.I	260 to 015	080 to 195	CZ.K.II	CZ.RM.II (Type 2)	Main kinematic control for set FLT1A (83 deg), and inclined set FLT1C (55 deg) for IRA. Inferred lower slope alteration limits berm width for unravelling considerations.	75	53	24	12
			CZ.K.III	CZ.RM.III (Type 2)		75	53	18	9
CZ.EW.I	015 to 080	195 to 260	CZ.K.IV	CZ.RM.IV (Type 1)	Inferred full slope alteration with ore halo alteration following main shear zone trend (striking ENE). Steep BFA and wider berm selected to contain ravelling.	75	50	24	14
						75	50	18	10.5
CZ.FW.I	080 to 195	260 to 015	CZ.K.V	CZ.K.V (Type 2)	Main kinematic control for set FLT1B (82 deg). Major wedges plunging at 63 deg to 73 deg. Inferred lower slope alteration limits berm width for unravelling considerations.	75	53	24	12
			CZ.K.VI	CZ.K.VI (Type 2)		75	53	18	9

7.0 OPERATIONAL CONSIDERATIONS

This section addresses some of the operational considerations during mining of the Main Zone and Centre Zone open pits.

7.1 Groundwater Considerations

As explained in Appendix E, all structurally controlled failure modes (i.e., planar, wedge and toppling) and floor heave are aggravated by water pressures within the slope or floor. Water pressure was not included in the kinematic analysis as the majority of the benches are assumed to remain dry. Therefore, the importance of achieving depressurised conditions cannot be over-emphasized.

For the slopes that may possibly extend below the permafrost level at Main Zone, most of the bedrock seepage is anticipated to occur along fracture zones encountered at depth. It is assumed that rock groundwater inflows and direct precipitation in the pit can be collected and pumped out from pit floor sumps. Horizontal drains may be required in localized areas if persistent seepage is noted during pit operations or if adequate pore water depressurization is not achieved. Water flowing from these drainholes should be manifolded into pipes to the sumps, in order to prevent face freeze-up in the winter months. The drainholes will also help reduce icing up of the slope face in the winter, which could lead to ice-falls.

As the pits are excavated, local thawing and re-freezing of the permafrost around the pit walls may occur. Instrumentation should be installed to monitor ground temperature as the pits are developed, and reassessment of the permafrost and water conditions within the pit walls should be conducted regularly during development. This freeze/thaw effect may also cause degradation of the rock mass, so signs of instability or local ravelling should also be monitored.

Pit floor heave due to water pressure must also be addressed. To avoid loss of pit floor monitoring instrumentation and depressurization drains (vertical), it is recommended that a geotechnical bench be established above the base of permafrost, nominally 20 m wide. This bench will act as additional protection against rockfall and debris, and act as a platform from which depressurization and water pressure monitoring can occur. Protective structures may be required to shelter instrumentation from loose rock or debris falls. Similar to the above, instrumentation should be installed to monitor ground temperature, to watch for changes in the depth of the permafrost horizon in response to mining and depressurization. The elevation of this geotechnical/depressurization bench should be based on the estimated minimum allowable thickness of pit floor above the artesian zone to resist heave due either self weight (conservative) or due to rock mass failure (optimistic).

Some of the vertical pressure relief drains may have flowing water, which will require diversion to sumps. Others, located in pressurized but low permeability rock, are more likely to freeze. Consideration should be given to this fact, which could be addressed either by regularly redrilling new pressure relief vertical drains, or by including heat trace lines on the vertical drains during installation, so they can continue to function. However, Golder has some concerns about the long term functionality of heat trace lines, based on experience at other sites, and suggests that the depressurization solution will require a combination of new drains and heated drains. Additional vertical pressure relief drains may be required along the ramp, as the pit floor deepens.

An alternative approach to consider would be to continually drill new vertical drains on the operating pit floor, instead of, or in addition to the perimeter drains on the geotechnical bench. The cost-benefit of this approach, versus the risks should the pit floor become inaccessible for a significant period of time, needs to be reviewed.

7.2 Overburden Slopes

Maintaining the stability of the overburden slopes during and after excavation will require:

- maintaining erosion protection along the overburden slope face through a good quality waste rock cover;
- regular monitoring for tension cracking or other signs of slope instability;
- water management around the crest and toe of the overburden slopes to control seasonal runoff and precipitation, and to help prevent slope erosion; and
- monitoring of the overburden slopes for degradation over time due to freeze/thaw or temperature effects.

7.3 Blasting and Excavation

Some form of controlled blasting and excavation control will be necessary during the drilling, blasting and the excavation of the open pits. This will be especially important in areas of highly altered or poor rock quality, as these zones are more likely to become instable with uncontrolled excavation methods. It is recommended that pit specific, optimized controlled blasting designs be developed early in the mine life for use on long-term and final slopes to improve surficial stability of the bench faces. Blasting experience and trials should be developed and optimized in the interior of the open pit prior to applying it to the final slopes.

In addition to blasting control, other important excavation control issues include:

- maintaining full design catch bench widths by cleaning the toe of the bench slopes fully;
- avoiding undercutting at the toe of the bench slopes, which may increase toppling or other modes of failure;
- minimizing the practice of working at the base of major slopes during the active runoff and snowmelt period;
- the shape of the pit should be designed to avoid convex slopes or “noses” which are invariably more unstable than concave slopes;
- maintaining strict grade control and altering excavation practices near the highly altered ore zones; and
- conducting periodic scaling of the bench crests to clean off loose rock fragments.

Blasting in permafrost will have challenges similar to those faced by open pit mining operations during cold winter conditions. These challenges revolve around delays between drilling, loading and blasting. The longer blasts remain loaded but not detonated (“sleep time”) the more likely the potential for reduced explosive performance.

As is the case at all operations, test blasting will be required to optimize loads, burdens, powder factor and sleep time of the hole.

Considerations associated with drilling blast holes in frozen ground include:

- The drillholes may fill with water and freeze over, requiring re-drilling if not loaded promptly.
- For the Main Zone site, this is most likely to occur on the upper benches, which may fill due to surface runoff and melt, and for the lower benches, due to upwelling of confined aquifer water that will occur proximal to the base of each of these pits. For the Centre Zone site, this is most likely to occur on the upper benches only, as the projected base of the pit is well above the base of permafrost.
- For the Main Zone site, the recommended pit floor advanced depressurization measures to control floor heave will also help mitigate hole-filling.

Considerations associated with loading explosives into accessible blastholes in frozen ground include:

- Can the explosives tolerate extended periods at very low temperatures?
- Do the explosives (such as emulsions) become desensitized with exposure to severe cold?
- How does the length of time that the explosives "sleep" at very low temperatures exacerbate the above?
- Will increased powder factors be required to overcome the above?
- Can the "sleep time" be limited if the explosive performance is likely to be affected?
- Increased powder factors may also be required to overcome the additional strength that being frozen provides the wall rock.

The explosives manufacturer/distributer should be consulted. They will have information regarding the most appropriate product for the intended operation and recommended procedures to obtain the best blast result for the anticipated conditions and mine requirements.

With respect to operator safety, upper benches in near-surface bedrock may be more intensely fractured than rock at depth, due to recent or historical freeze-thaw and weathering. At some frozen-ground sites, such as the small open pits at an operating mine in the Ungava Peninsula region of northern Quebec, there can be greatly increased fly rock generated from the upper bench blasts, because of the greater intensity of fracturing. In winter months, higher powder factors were required. However, in summer months, when and where similar upper bench wall rock was locally unfrozen, the same powder factors resulted in greater than expected fragmentation and flyrock, sending debris over the tundra around the pit perimeter. Consequently, blast performance evaluations should consider season changes in ground temperatures, when determining how to modify blasting practices for both efficiency and safety reasons.

7.4 Ground Support

A budget provision should be included for the eventual requirement of localized reinforcement of rock slopes if the need should arise, especially around the highly altered zone. This reinforcement, if necessary, could include the use of cable-bolts, mesh and shotcrete.

7.5 Monitoring Program

The ongoing development of the pit will require an observational approach. With this method, which is common practice in the mining industry, the initial pit excavations are monitored and the pit slope designs are modified on an ongoing basis throughout the life of the pit. It is expected that revisions will be made based on further review and mapping and stability performance monitoring, as mining exposes subsurface geology in the proposed pit.

A pit slope monitoring program should be established early in the life the pits. The monitoring program is intended to both confirm the assumptions made regarding the structural and geologic models and to detect unexpected conditions in sufficient time that remedial measures can be adopted. This program should include both pit mapping to confirm the engineering geology model upon which the designs are based, as well as monitoring to detect any movement in the slopes.

The program should be intended to be conducted largely by the mine geotechnical staff; although periodic reviews by an experienced rock slope design engineer is recommended. It is recommended that the monitoring program include aspects relating to the following:

- Geologic Mapping - in order to confirm the geological model on which the current slope designs are based and assess the potential for slope steepening, routine geologic mapping should be carried out as the slopes are excavated.
- Slope Monitoring - regular visual inspections of the bench faces and the crest areas should be conducted for early evidence of slope instability. Occurrences of tension cracks behind the slope crest are indicators of movements and the beginning of instability in the slopes. Instrumentation should also be installed around the perimeter of each pit to monitor slope movement as the excavation progresses.

7.6 Pit Floor Heave

Pressure readings taken from vibrating wire piezometers installed in the proposed Main Zone pit location in 2009 indicate that groundwater is present at a significant pressure under the zone of permafrost. As the pits are excavated, this groundwater pressure should remain relatively unchanged, while the rock above the permafrost interface will be reduced. This may lead to pit floor heave. It is recommended that vertical pressure relief drains be installed in the pit floor as the pit excavation progresses, with preference for a ring of drains on a geotechnical bench at a suitable elevation above the base of permafrost. These will depressurize the pit floor, and reduce the potential for heave. Water from the drain holes will be collected within the pit sumps and removed from the pit as part of the water management program.

8.0 RECOMMENDATIONS FOR FUTURE WORK

Future drilling investigation work at Kiggavik Main Zone for the purpose of slope design optimization is not considered a priority because of the interpreted favourable rock mass conditions. More will be learned by careful documentation of the slope stability performance of initial benches, combined with structural mapping. Where future drilling investigations could be helpful is to further develop a 3-D and geotechnical understanding of the dyke that is interpreted to overlie the mid-pit pillar of barren rock that causes the figure-eight shape of the pit shell. This dyke rock may be well-suited for use as backfill at End Grid. To optimize the End Grid mine plan, it may help to have the best possible volumetric and quality information on this dyke. However, before drilling, a detailed 3-D interpretation of existing geological and geotechnical data on this feature, particularly strength, could be carried out, as that may suffice for this level of economic investigation.

9.0 CLOSURE

This report presents the rationales and slope design recommendations for the Kiggavik Main Zone, and Kiggavik Centre Zone open pits. This document has incorporated initial comments received from AREVA. We trust this report meets your requirements at this time. Should any questions arise from this report, please feel free to contact the undersigned at our office.

GOLDER ASSOCIATES LTD.

REGISTERED PROFESSIONAL ENGINEER
ROSS D. HAMMETT
LICENSEE
N.W.T.

Ross Hammett, B.Eng.
Principal, Senior Civil/Mining Engineer

Marc Rougier

Marc Rougier, B.Sc.
Principal, Senior Geological Engineer

AH/EAM/BC/LFG/MR/TGC/pls/rs/ls

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PROFESSIONAL ENGINEERS,
GEOLOGISTS and GEOPHYSICISTS
OF THE NORTHWEST TERRITORIES
PERMIT NUMBER
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ASSOCIATES LTD.

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APPENDIX A

Rock Strength

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TABLES

APPENDICES

1.0 INTRODUCTION

This appendix presents the results of the materials testing conducted on the sampled rock material specimens obtained during the 2009 geotechnical drilling program for Kiggavik Main Zone (MZ) and Centre Zone (CZ) geotechnical boreholes. Strength testing included Point Load Testing (PLT) and Unconfined/Uniaxial Compressive Strength (UCS) testing. A total of 244 valid PLT and 16 UCS tests were carried out as part of the strength testing program. Rock hardness assessment according to the International Society of Rock Mechanics (ISRM, 1981) procedures was also carried out as part of the geotechnical core logging. The rock hardness data is presented in detail in the 2009 data report (Golder 2009).

The following appendix discusses the rock materials testing program and presents a summary of strength test results for the Kiggavik sites. Unless otherwise noted, rock type refers to information provided by AREVA, based on the following:

- GnPsaPel – Psammo-pelitic Gneiss;
- Flt – Fault;
- GranT – Granite;
- BxQtz – Quartz Breccia;
- Lamp – Lamprophyre;
- GnGran – Granitic Gneiss;
- Episyen – Episyenite;
- GnAlt – Altered Gneiss;
- GnQtzFd – Quartzo-feldspathic Gneiss;
- GnPel – Pelitic Gneiss;
- GnPsa - Psammitic Gneiss;
- GnArk – Arkosic Gneiss; and
- Myl – Mylonite.

2.0 ROCK MATERIALS TESTING

2.1 Laboratory UCS Testing

A number of representative samples were collected from the 2009 geotechnical boreholes and sent to the University of Saskatchewan's Rock Mechanics Laboratory (U of S) for UCS testing and elastic properties determination. A total of 16 tests were carried out on the MZ/CZ borehole samples. A laboratory testing report by the U of S, including photos of the test specimens before and after breaking, is presented in sub-Appendix A1.

The UCS, elastic properties (Young's Modulus and Poisson's ratio), and material bulk density (based on physical measurements of the sample dimensions and mass) results are presented in Table A1. The representative lithology provided by AREVA is also included. The depth increment indicated in these tables is the nearest run interval (true depths given in Appendix A1). The type of sample breakage has been interpreted by the U of S technicians which indicate whether failure occurred through the intact rock material, or preferentially along a weakness or foliation plane. The MZ/CZ data shows one preferential failure.

Table A1: Kiggavik Main and Centre Zones – Summary of laboratory UCS test results

Hole	Depth (m)	Lithology	UCS (MPa)	Youngs Modulus (GPa)	Density (g/cm^3)	Poissons Ratio	Type of Failure
CZ09-01	96	GnPsaPel	61.0	39.8	2.80	0.16	Intact rock
	123	GnPsaPel	57.4	38.2	2.75	0.14	Intact rock
MZ09-01	54.00	GnPsaPel	108.3	49.9	2.68	0.17	Intact rock
	141.00	GnPsaPel	131.4	54.3	2.71	0.19	Intact rock
	222.55	GranT	86.3	43.4	2.62	0.15	Intact rock
	244.88	GranT	30.0	8.8	2.24	0.07	Intact rock
	255.00	GranT	53.7	18.7	2.36	0.12	Intact rock
MZ09-02	26	GnPsaPel	21.8	6.6	2.37	0.01	Intact rock
	59	GnPsaPel	104.0	48.3	2.46	0.15	Intact rock
	119	GnPsaPel	103.9	42	2.73	0.21	Intact rock
	184.2	GranT	83.9	44.1	2.67	0.12	Weakness Plane
	230	GranT	152.2	50	2.72	0.16	Intact rock
	242	GranT	111.1	42.4	2.61	0.13	Intact rock
MZ09-03	114	GnPsaPel	101.7	36.8	2.70	0.17	Intact rock
	147	GranT	99.5	44.6	2.61	0.15	Intact rock
	189	GnPsaPel	119.5	47.8	2.69	0.17	Intact rock

m = metre, MPa = mega pascal, GPa = giga pascal, g/cm^3 = grams per cubic centimetre; rock type based on information provided by AREVA

2.2 Field PLT and Rock Strength Index Testing

During the 2009 geotechnical drilling program, point load testing (PLT) was performed in the field at selected depth intervals. There were 244 valid PLT tests carried out for the MZ/CZ geotechnical boreholes.

The PLT procedures are discussed in Golder's data report from the 2009 field season (Golder 2009). The PLT index value $I_{s(50)}$ is calculated from the load required to break the core specimen using the PLT apparatus. The $I_{s(50)}$ value is an index value (MPa) that can be correlated to the expected UCS of the intact rock, but is also useful for illustrating the strength variability throughout the various rock units and geotechnical domains. The tests were performed diametrally, or orthogonal to the core axis. Multiple tests were performed at the selected depth to reduce statistical bias often resulting from breakage of the rock along pre-existing weakness planes. Valid tests were identified according to ISRM procedures (ISRM 1985).

Rock strength index (R) values were assigned according to ISRM procedures as discussed in the data report (Golder 2009). A minimum of one index value was assessed for every drilling interval (drill run). This index is a simple estimate of rock hardness related to rock strength based on the rock materials hardness and resistance to fracturing.

The calculated $I_{s(50)}$ strengths, taken as the average of valid tests at the selected depth, are presented in Table A2 for the MZ/CZ geotechnical holes. The 'R' value as assessed in the field is also presented for comparison. The indicated depths represent the nearest drill run depth interval. The lithologies were provided by AREVA's geologists unless otherwise noted.

Table A2: Kiggavik Main and Centre Zones - summary of average $I_{s(50)}$ (MPa) and R (ISRM index) values by borehole

CZ-09-01				MZ-09-01				MZ-09-02			
Depth (m)	Lithology	$I_{s(50)}$ (MPa)	R	Depth (m)	Lithology	$I_{s(50)}$ (MPa)	R	Depth (m)	Lithology	$I_{s(50)}$ (MPa)	R
42	GnPsaPel	1.5	2.5	15.00	GnPsaPel	8.2	4	56	GnPsaPel	10.7	4
45	GnPsaPel	1.7	2.5	39.00	GnPsaPel	11.0	4	92	GnArk	10.8	5
48	GnPsaPel	2.8	2.5	54.00	GnPsaPel	9.8	4	93.25	GnArk	12.7	5
90	GnPsaPel	5.1	3.5	144.00	GnPsaPel	9.1	4	107	GnPsaPel	10.8	3
93	GnPsaPel	6.4	3.5	153.00	GnPsaPel	7.3	4	110	GnPsaPel	5.8	3
96	GnPsaPel	2.7	3.5	162.00	GnPsaPel	9.4	4.5	122	GnPsaPel	10.2	3
99	GnPsaPel	2.2	3	165.00	GnPsaPel	6.1	4	131	GnPsaPel	5.4	3
102	GnPsaPel	5.2	3	203.85	GnPel	6.6	3	143	GnPsaPel	5.0	3
105	GnPsaPel	8.9	3	219.00	GranT	7.6	4	149	GnPsaPel	13.0	3.5
111	GnPsaPel	3.5	3	249.00	GranT	3.7	2	176	GnPsaPel	15.1	4
114	GnPsaPel	6.1	3	273.00	GranT	8.5	4	179	GnPsaPel	10.0	4
117	GnPsaPel	4.9	3	297.00	GranT	9.9	4.5	184.2	GranT	10.5	3
120	GnPsaPel	4.4	3					191	GranT	7.8	3
168.48	GnPsaPel	5.5	4					200	GranT	10.2	4
170.25	GnPsaPel	6.5	4					227	GranT	10.3	4
186	GranT	9.4	4					233	GranT	8.9	4
192	GnPsaPel	11.7	3.5					238.08	GranT	10.4	5
195	GnPsaPel	2.3	3.5					239	GranT	10.2	5
204	GnPsaPel	10.8	4					242	GranT	13.6	5
252	GranT	11.3	5					251	GranT	12.5	5
255	GranT	12.3	5					257	GranT	11.9	5
261	GnPsaPel	4.2	5								
MZ-09-03				**MZ-09-04**							
Depth (m)	**Lithology**	**$I_{s(50)}$ (MPa)**	**R**	**Depth (m)**	**Lithology**	**$I_{s(50)}$ (MPa)**	**R**				
27.14	GnPsaPel	11.0	4	6	GnPsaPel	4.4	5				
30	Lamp	5.6	5	9	GnPsaPel	7.3	5				
33	Episyen	14.5	5	12	GnPsaPel	7.9	5				
36	Episyen	11.2	5	27	GnPsaPel	12.0	5				
45	Episyen	10.5	5	51	GnPsaPel	6.9	5				
48	Episyen	15.6	5	87	Myl	7.4	5				
51	Episyen	12.3	5	117	GnPsaPel	7.3	5				
66	GnPsaPel	13.6	5	147	GnPsaPel	4.6	3				
78	GnPsaPel	11.0	4	150	GnPsaPel	10.8	5				
81	GnPsaPel	9.1	4	177	GnPsaPel	12.7	5				
99	GnPsaPel	7.5	4	189	GnPsaPel	10.0	5				
120	GnPsaPel	5.5	4	219	GnPsaPel	9.1	5				
135	GnPsaPel	13.5	4	228	GranT	10.0	5				
138.2	GranT	9.0	4								
141	GranT	8.7	4								
150	GranT	13.1	4								
159	GranT	8.6	4								
162	GranT	6.1	4								
177	GnPsaPel	12.9	4								
189	GnPsaPel	9.4	4								
210	GnPsaPel	11.6	4								
219	GranT	9.6	2.5								

m = metre, MPa = mega pascal; rock type based on information provided by AREVA

3.0 STRENGTH TESTING RESULTS

The Kiggavik UCS and elastic parameters results are summarised in Table A3 by rock type and alteration. Where sufficient data was available, the rock types have been subdivided into their respective degrees of weathering/alteration. This alteration designation was based on the ISRM weathering indices assessed by Golder's field staff, which is described in the 2009 data report (Golder 2009).

Based on the available data, where alteration is present, strength (and density) of the intact rock are noticeably lower. These alteration horizons are likely associated with the halos of mineralization and/or associated faulting. In general, the non-altered metasediments and granites are shown to be strong (>75 MPa) to very strong (100 MPa to 150 MPa).

Table A3: Kiggavik Main and Centre Zone - summary of UCS and elastic parameters by rock type and alteration

Rock Type	Alteration (ISRM)	# Tests	UCS (MPa)	Youngs Modulus (GPa)	Density (g/cm^3)	Poissons Ratio
Metasediment (GnPsaPel)	Highly altered (W5)	1	21.8	6.6	2.37	0.01
	Fresh (W1)	8	98.4 +/- 26.1	44.6 +/- 6.3	2.69 +/- 0.10	0.17 +/- 0.02
Granite (GranT)	Slightly to Moderately altered (W2 to W3)	3	55.9 +/- 27.0	23.9 +/- 18.2	2.42 +/- 0.22	0.10 +/- 0.03
	Fresh (W1)	4	112.3 +/- 28.5	45.1 +/- 3.4	2.64 +/- 0.05	0.15 +/- 0.01

*includes average and standard deviation (+/-); rock type based on information provided by AREVA, MPa = mega pascal; GPa = giga pascal, g/cm^3 = grams per cubic centimetre

The Kiggavik PLT $I_{s(50)}$ results are summarised in Table A4 by rock type and alteration. The rock units were subdivided by alteration according to the ISRM designations. It was noted that insufficient PLT tests were carried out on the moderately to highly altered rock units.

The general trends of the PLT data agree reasonably well to the UCS results. The non-altered granites show slightly higher average strength compared to the metasediments. In general, the fresh to slightly altered rock units range in strength from strong (>75 MPa) to very strong (100 MPa to 150 MPa).

For comparing the PLT – $I_{s(50)}$ results to the material UCS, a correlation factor (K) can be used as follows:

$$UCS_{plt} = I_{s(50)} \times K$$

The K factor can often vary significantly depending on the rock material and microstructure. For comparing the PLT $I_{s(50)}$ data to an equivalent UCS, based on the averaged data, a correlation factor (K) of 10 to 12 is estimated. An average K of 12 should produce reasonable correlations. Other work by SRK (SRK, 2009) found correlation factors of between 11.2 and 14.5 for the Kiggavik rock types.

Table A4: Kiggavik Main and Centre Zone - summary of PLT $I_{s(50)}$ values by rock type and alteration.

Rock Type	Alteration (ISRM)	# Tests	$I_{s(50)}$ (MPa)
Metasediments (GnArk,GnPel,GnPsaPel)	Slightly altered (W2)	30	7.9 +/- 3.6
	Fresh (W1)	147	8.4 +/- 3.7
Granite >140 m (GranT)	Slightly altered (W2)	9	5.7 +/- 2.8
	Fresh (W1)	45	10.1 +/- 2.3
Episyenite (Episyen)	Fresh (W1)	7	12.3 +/- 2.4

*includes average and standard deviation (+/-); rock type based on information provided by AREVA, MPa = mega pascal

4.0 CONCLUSIONS AND RECOMMENDATIONS

The Kiggavik Main and Centre Zones generally exhibit strong (>75 MPa) to very strong (100 MPa to 150 MPa) rock conditions. Some alteration is present in the 2009 boreholes, likely associated with faulting and mineral alteration. These altered zones are shown to be weak (<25 MPa) to moderately strong (<50 MPa) depending on the intensity of alteration. Where faulting or ore zone alteration 'halos' intersect the planned pit walls and floor, weak to moderately strong rock conditions might be encountered.

The proceeding analyses are based on limited laboratory testing. Additional laboratory testing in the form of triaxial and tensile strength testing would be required to develop the rock failure envelopes at various degrees of confinement. Moreover, additional work should be undertaken to better understand the alteration profiles into the planned pit walls and floor, as alteration is shown to relate closely with rock strength.

5.0 REFERENCES

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SRK Consulting (Canada) Inc., 2009. Kiggavik-Sissons Updated Geotechnical Data Report 2007/2008. Project #1CA015.003. March 2009.

\\sas1-s-filesrv1\data\active\2009\1362\09-1362-0613 areva kiggavik geotechnical baker lake nunavut\pit slope report\final reports - february 2011\main and centre\app a\app 11 feb 17 main and centre appendix a - rock strength final_eam.docx

APPENDIX A1

University of Saskatchewan Laboratory Test Results

UNIVERSITY OF SASKATCHEWAN

Rock Mechanics Laboratory
Geological Engineering Program
University of Saskatchewan

Unconfined Compressive Strength and Elastic Properties Determinations

Report Submitted
to
Golder Associates

D. Milne and F. Monsman
October 2009

1. Introduction

This report contains the results of strength tests performed on rock samples provided by Golder Associates. Testing was done at the Rock Mechanics Laboratory in the Department of Geological Sciences at the University of Saskatchewan.

2. Sample Preparation

Fifty-six NQ core waste rock samples and 14 NQ core radioactive samples were received at the U of S Rock Mechanics Lab. The samples were wrapped in plastic bags and duct tape.

Of the 56 waste rock samples received, 54 were long enough to be cut to a length to diameter ratio of 2:1, as required for unconfined compressive strength testing (UCS). For a few of these samples this ratio is slightly lower due to fractures in the sample that required the sample to be cut shorter, but testing was still done. Of the 14 radioactive samples, 12 were of adequate length. These samples were subjected to UCS tests, in which the load was applied perpendicular to the cut ends.

3. Testing Procedure

After being cut and ground flat on each end, the samples were placed into a UCS testing machine. A constant rate of load was applied until the sample failed. The peak load, F, acquired by the data logger was used to calculate UCS by dividing by the surface area of the end of the sample in the following equation:

$$UCS = \frac{F}{\pi r^2}$$

Data acquired from two linear displacement transducers was averaged to determine axial displacement. The axial strain was calculated by dividing the change in length by the initial length of the sample using the formula: $\varepsilon_A = \frac{\Delta L}{L_i}$

A circumferential extensometer was used to measure radial displacement and calculate radial strain by dividing the change in circumference by initial sample circumference using the formula: $\varepsilon_R = \frac{\Delta C}{\pi D_i}$

Young's Modulus was determined by plotting stress vs. strain and finding the slope of the stress-axial strain curve at 50% of peak strength.

Poisson's ratio was determined by dividing radial strain by axial strain at 50% of peak strength: $\nu = \frac{\varepsilon_R}{\varepsilon_A}$

4. Test Results

Results of the tests for the waste rock are tabulated in Table 1, while the results for the radioactive samples are shown in Table 2. Appendix A and Appendix B contain results from each test and photographs of samples before and after testing. Test results show a slight increase in rock UCS with increasing density. Sample inspection indicated that the samples that could not be tested appeared to have significantly lower strength.

Table 1: Strength Test Results-Waste Rock Samples

Sample No.	Depth (m)	Length (mm)	Diameter (mm)	Density (g/cc)	Peak Strength (MPa)	Young's Modulus (GPa)	Poisson's Ratio	Cause of Failure
AND09-01-01	86.40-86.65	108.03	45.11	2.572	41.8	23.1	0.08	Intact
AND09-01-02	80.02-80.27	108.03	45.11	2.507	43.9	15.2	0.03	Intact
AND09-01-03	120.95-121.23	110.13	45.16	2.445	24.7	6.1	0.24	Intact
AND09-01-04	221.24-221.44	107.81	45.1	2.597	80.9	24.1	0.13	Intact
AND09-01-05	232.64-232.89	108.42	45.41	2.461	52	20.6	0.16	Intact
AND09-01-06	257.26-257.57	112.65	45.26	1.970	4.9	0.4	0.06	Intact
AND09-01-07	339.84-340.04	106.72	45.14	2.087	9.8	2.4	0.04	Weakness plane
AND09-02-01	102.09-102.20	86.53	45.34	2.479	14.2	1.8	0.13	Intact
AND09-02-02	105.64-105.87	111.62	45.34	2.509	16.1	4.5	0.03	Foliation
AND09-02-03	111.73-111.93	109.02	45.2	2.036	22.4	5.7	0.16	Weakness plane
AND09-02-04	129.18-129.32	107.82	45.25	2.397	26.7	8.6	0.49	Intact
AND09-02-05	137.20-137.48	108.76	45.67	2.534	55.6	21.0	0.07	Intact
AND09-02-06	151.81-151.96	111.83	45.39	2.328	26.6	7.3	0.08	Intact
AND09-02-07	168.68-168.90	103.18	44.32	2.494	11.8	5.1	0.04	Microfracture
AND09-02-08	175.18-175.27	94.5	44.07	2.525	19.3	6.4	0.75 ?	Microfracture
AND09-02-1A	60.12-60.22	72.81	45.14	2.194	17.5	2.2	0.10	Intact
AND09-03-02	46.74-46.90	112.13	45.16	2.614	24.3	12.9	0.12	Weakness plane
AND09-03-03	110.33-110.43	96.76	45.06	2.592	26.7	7.3	0.04	Microfracture
AND09-03-04	201.29-201.49	90.03	45.16	2.628	35	19.9	0.16	Intact
CZ09-01-02	39.20-39.32	Broke						
CZ09-01-04	97.85-98.00	108.77	45.43	2.801	61	39.8	0.16	Intact
CZ09-01-05	123.27-123.47	108.58	45.4	2.747	57.4	38.2	0.14	Intact
END09-03-03	165.50-165.63	108.99	47.78	2.373	7.2	3.5	1.4 ?	Weakness plane
END09-03-04	129.50-129.76	114.07	46.83	1.719	5	1.7	0.10	Intact
END09-03-06	237.76-237.91	111.12	47.77	2.415	36.8	9.5	0.09	Weakness plane
END09-03-07	470.32-470.53	112.13	47.59	2.441	27.5	11.5	0.05	Weakness plane
END09-04-01	213.80-213.97	111.75	45.25	2.112	32.1	3.3	0.09	Intact
END09-04-03	225.22-225.38	91.27	44.83	2.021	18.9	2.8	0.06	Intact
END09-05-02	208.40-208.54	Broke						
END09-06-02	198.57-198.75	106.33	45.13	1.987	11.3	0.7	0.07	Intact
END09-06-03	229.55-229.72	109.25	45.12	2.444	16.2	9.1	0.06	Weakness Plane
END09-06-04	239.76-239.90	93.83	45.11	2.326	40.1	12.7	0.08	Foliation
END09-07-01	178.87-178.98	91.01	47.6	2.168	26.7	6.6	0.30	Intact
END09-07-02	206.14-206.26	78.89	45.31	2.457	48.5	18.6	0.13	Microfracture
END09-07-03	210.87-211.00	99.18	45.08	2.662	64.2	31.9	0.11	Intact
END09-11-02	61.05-61.32	100.51	47.25	2.029	10.1	10.0	0.14	Intact
END09-11-03	204.59-204.75	112.01	47.7	2.463	30.7	15.2	0.08	Intact
END09-11-04	308.47-308.70	113.6	47.47	2.521	26	15.6	0.44	Intact
END09-11-06	289.38-289.55	112.26	47.51	2.451	42.4	17.5	0.06	Microfracture
END09-11-09	315.24-315.57	103.02	47.06	2.202	28.7	3.8	0.12	Intact
END09-12-04	205.94-206.06	94.16	47.78	2.491	38.5	18.8	0.14	Intact
END09-12-06	435.23-435.35	97.82	47.75	2.601	119.6	39.2	0.18	Intact
MZ09-01A-02	55.47-55.67	107.28	44.86	2.678	108.3	49.9	0.17	Intact
MZ09-01A-03	142.83-143.07	108.29	45.01	2.713	131.4	54.3	0.19	Intact
MZ09-01A-05	223.09-223.29	106.62	44.61	2.619	86.3	43.4	0.15	Intact
MZ09-01A-06	245.18-245.34	109.53	44.69	2.236	30	8.8	0.07	Intact
MZ09-01A-07	257.04-257.19	108.02	44.48	2.360	53.7	18.7	0.12	Intact
MZ09-02-01	28.84-29.00	89.15	44.95	2.367	21.8	6.6	0.01	Intact
MZ09-02-02	59.19-59.44	108.45	44.78	2.463	104	48.3	0.15	Intact
MZ09-02-03	120.77-120.92	109.64	44.85	2.733	103.9	42.0	0.21	Intact
MZ09-02-04	182.61-182.78	106.41	44.98	2.669	83.9	44.1	0.12	Weakness plane
MZ09-02-05	230.13-230.26	106.91	44.91	2.721	152.2	50.0	0.16	Intact
MZ09-02-06	243.28-243.44	109.13	45.14	2.613	111.1	42.4	0.13	Intact
MZ09-03-03	115.46-115.66	108.46	45.09	2.697	101.7	36.8	0.17	Intact
MZ09-03-04	145.80-145.93	109.51	45.03	2.614	99.5	44.6	0.15	Intact
MZ09-03-05	190.11-190.24	108.63	45.03	2.691	119.5	47.8	0.17	Intact

Table 2: Strength Test Results-Radioactive Rock Samples

Hole No.	Depth (m)	Length (mm)	Diameter (mm)	Density (g/cc)	Peak Strength (MPa)	Young's Modulus (GPa)	Poisson's Ratio
END 09-02	210.24-210.42	94.45	45.02	2.399	4.3	2.8	0.49
END 09-02	215.48-215.67	99.48	45.05	2.352	8.4	6.7	0.02
END 09-02	234.67-234.89	98.07	45.12	2.440	10.3	5.0	0.13
END 09-02	246.92-247.04	102.86	45.05	2.447	15.2	7.5	0.56
END 09-02	272.81-272.95	100.68	45.03	2.473	24.0	14.0	0.09
END 09-02	284.00-284.30	98.16	44.96	2.460	13.4	13.1	0.11
END 09-02	291.40-291.50	short					
END 09-02	302.10-302.43	101.08	45.08	2.604	43.0	11.8	0.75
END 09-02	312.35-312.55	102.08	44.95	2.415	5.6	2.0	0.93
END 09-02	313.95-314.12	101.57	44.97	2.476	12.9	7.4	0.51
END 09-02	319.50-319.60	fractured					
END 09-02	329.30.329.50	103.53	44.97	2.377	23.9	9.5	0.32
END 09-02	333.25-333.42	101.03	45.13	2.524	7.4	6.9	0.31
END 09-02	341.20-341.40	100.13	45.05	2.440	9.4	5.1	0.04

APPENDIX A
WASTE ROCK SAMPLES

SAMPLE: AND09-01-01

Length: 108.03 mm

Diameter: 45.11 mm

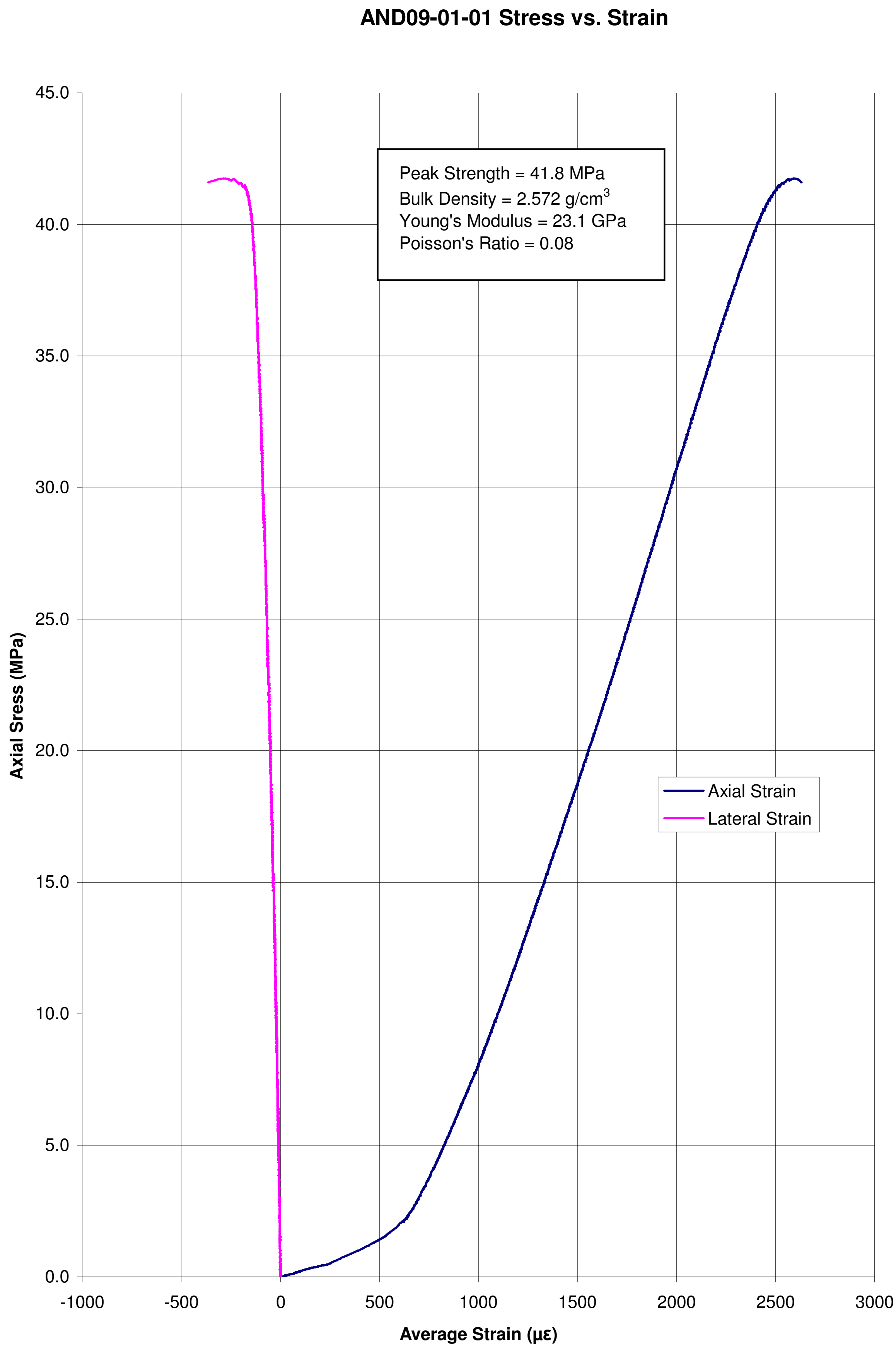
Density: 2.572 g/cm^3

Peak Strength: 41.8 MPa

Young’s Modulus: 23.1 GPa

Poisson’s Ratio: 0.08

Failure Cause: Intact Rock


AND09-01-01 Stress vs. Strain
Peak Strength = 41.8 MPa
Bulk Density = 2.572 g/cm3
Young's Modulus = 23.1 GPa
Poisson's Ratio = 0.08
Axial Strain
Lateral Strain
Axial Sress (MPa)
Average Strain (με)
45.0
40.0
35.0
30.0
25.0
20.0
15.0
10.0
5.0
0.0
-1000
-500
0
500
1000
1500
2000
2500
3000

SAMPLE: AND09-01-02

Length: 108.03 mm

Diameter: 45.11 mm

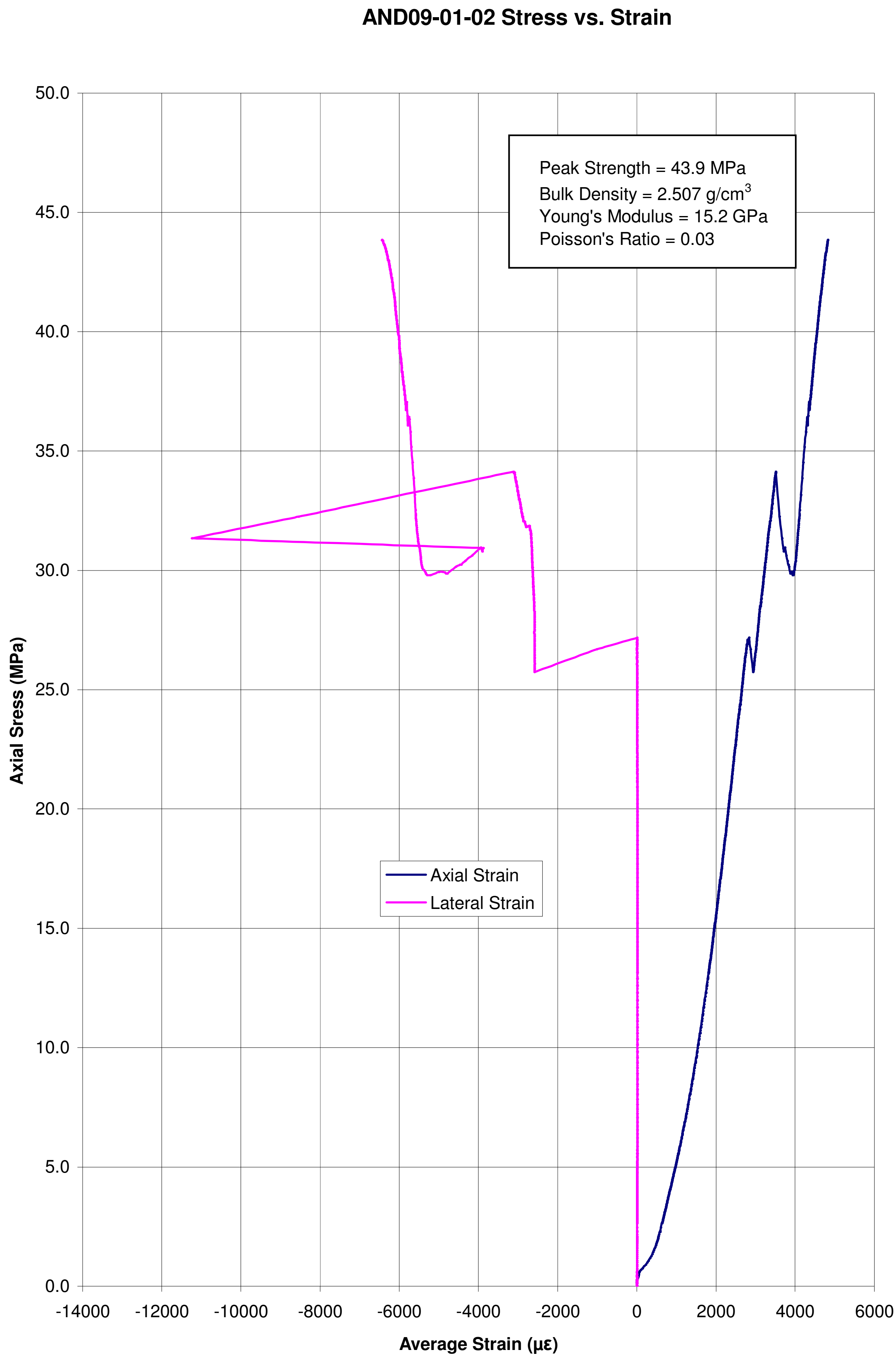
Density: 2.507 g/cm^3

Peak Strength: 43.9 MPa

Young's Modulus: 15.2 GPa

Poisson's Ratio: 0.03

Failure Cause: Intact Rock


AND09-01-02 Stress vs. Strain
Peak Strength = 43.9 MPa
Bulk Density = 2.507 g/cm3
Young's Modulus = 15.2 GPa
Poisson's Ratio = 0.03
Axial Strain
Lateral Strain
Axial Sress (MPa)
Average Strain (με)
50.0
45.0
40.0
35.0
30.0
25.0
20.0
15.0
10.0
5.0
0.0
-14000
-12000
-10000
-8000
-6000
-4000
-2000
0
2000
4000
6000

SAMPLE: AND09-01-03

Length: 110.13 mm

Diameter: 45.16 mm

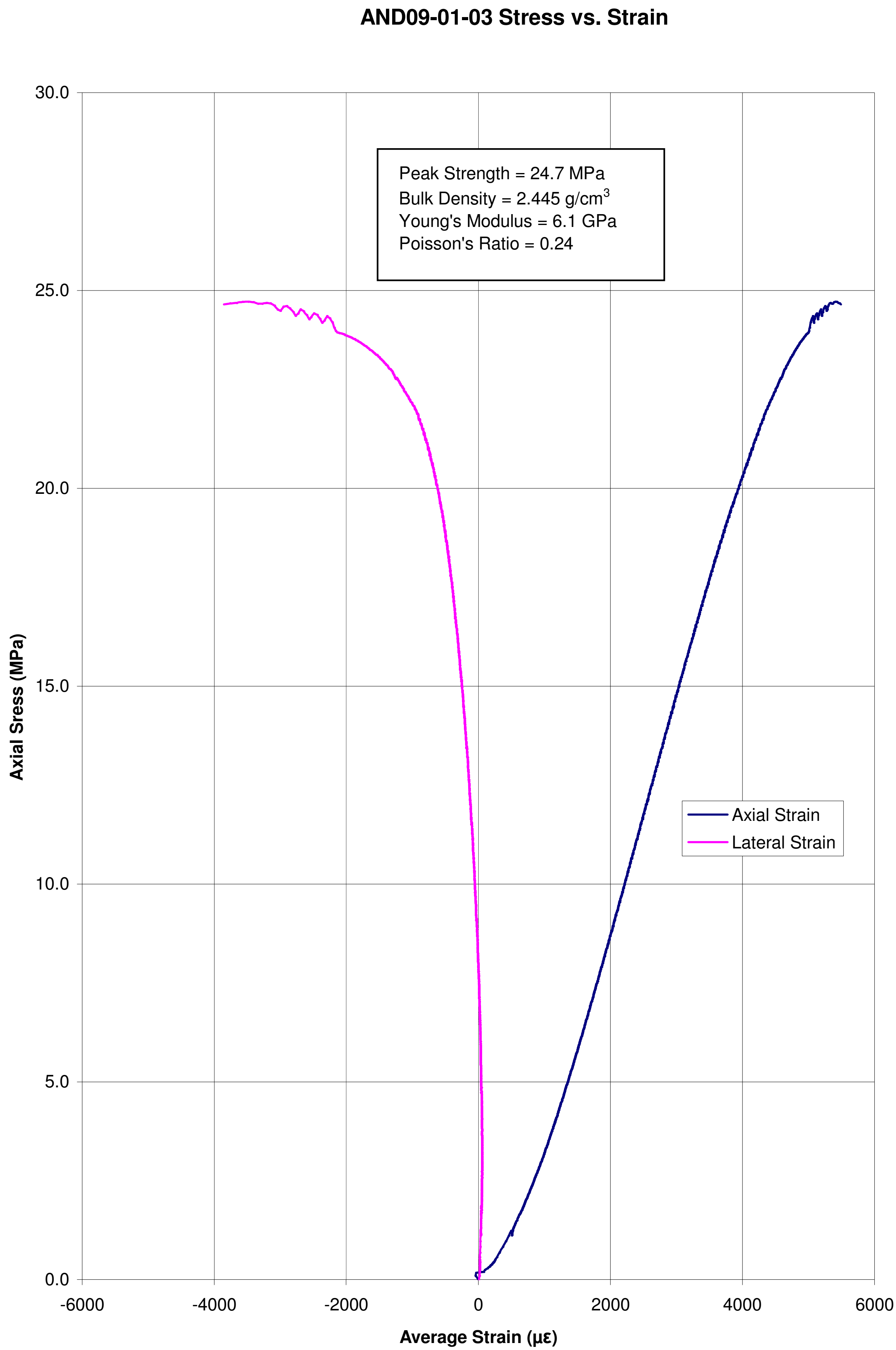
Density: 2.445 g/cm^3

Peak Strength: 24.7 MPa

Young's Modulus: 6.1 GPa

Poisson's Ratio: 0.24

Failure Cause: Intact Rock


AND09-01-03 Stress vs. Strain
Peak Strength = 24.7 MPa
Bulk Density = 2.445 g/cm3
Young's Modulus = 6.1 GPa
Poisson's Ratio = 0.24
Axial Strain
Lateral Strain
Axial Sress (MPa)
Average Strain (με)
30.0
25.0
20.0
15.0
10.0
5.0
0.0
-6000
-4000
-2000
0
2000
4000
6000

SAMPLE: AND09-01-04

Length: 107.81 mm

Diameter: 45.1 mm

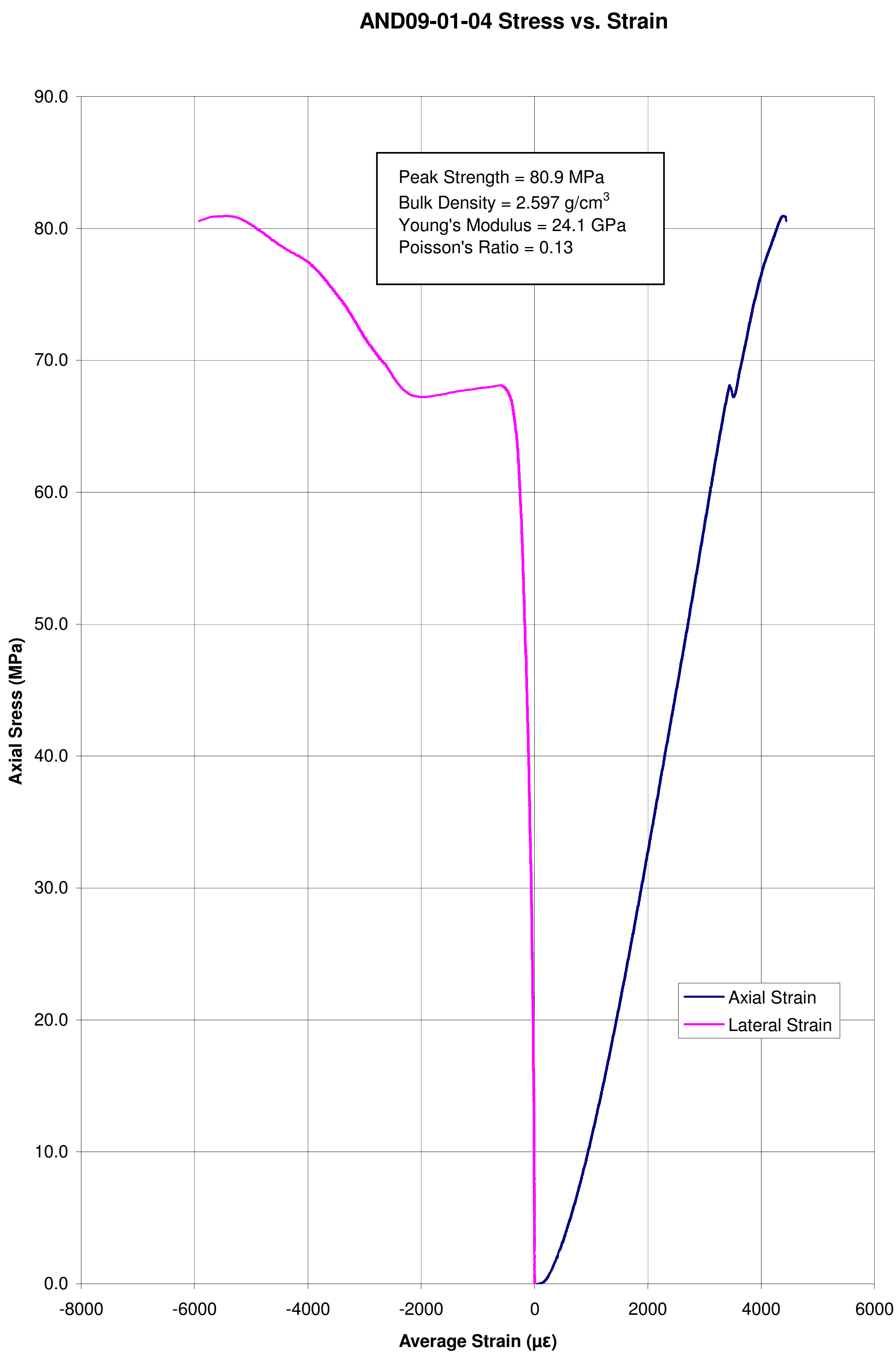
Density: 2.597 g/cm^3

Peak Strength: 80.9 MPa

Young's Modulus: 24.1 GPa

Poisson's Ratio: 0.13

Failure Cause: Intact Rock



AND09-01-04 Stress vs. Strain
Peak Strength = 80.9 MPa
Bulk Density = 2.597 g/cm3
Young's Modulus = 24.1 GPa
Poisson's Ratio = 0.13
Axial Strain
Lateral Strain
Axial Sress (MPa)
Average Strain (με)
90.0
80.0
70.0
60.0
50.0
40.0
30.0
20.0
10.0
0.0
-8000
-6000
-4000
-2000
0
2000
4000
6000

SAMPLE: AND09-01-05

Length: 108.42 mm

Diameter: 45.41 mm

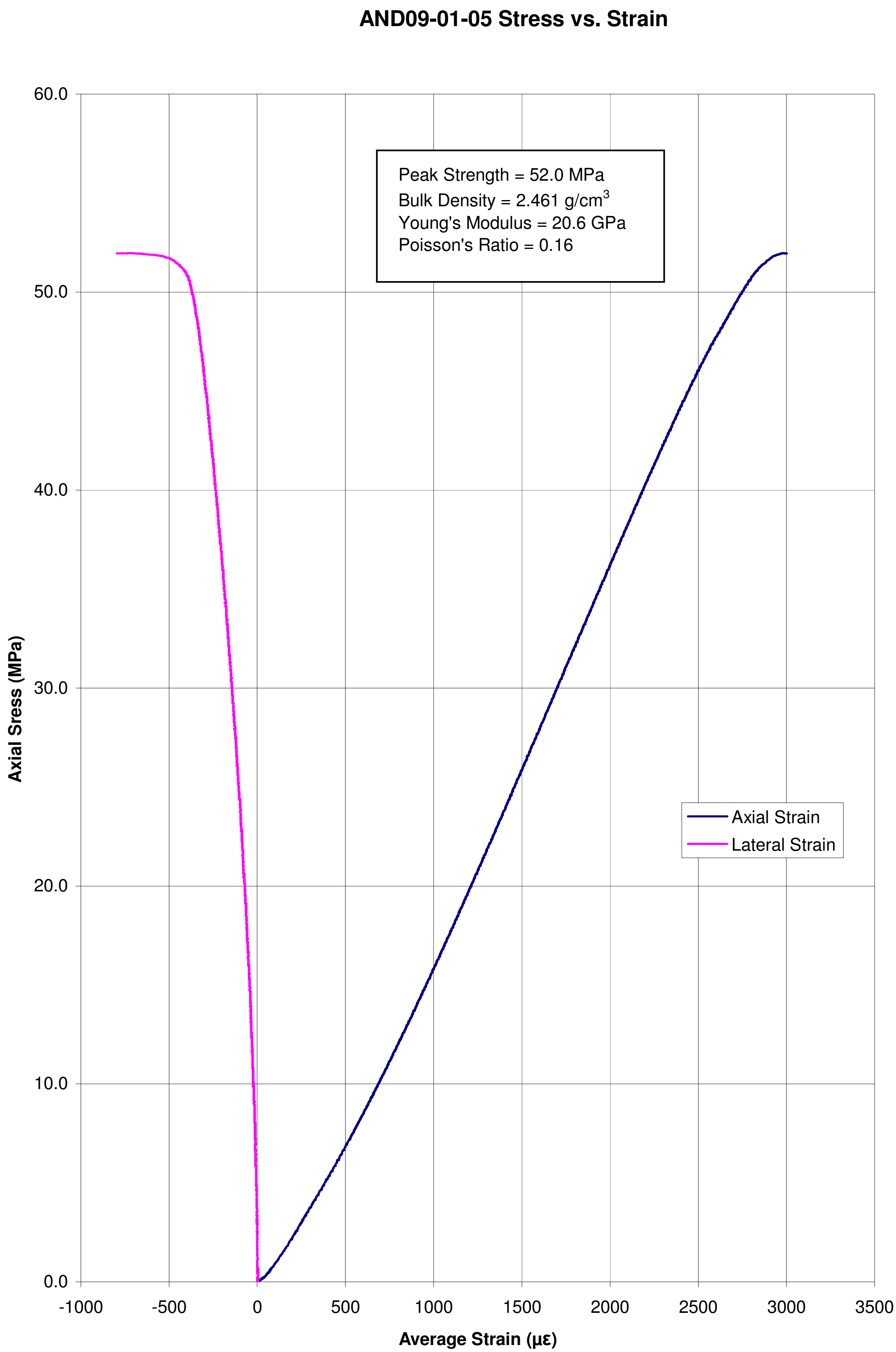
Density: 2.461 g/cm^3

Peak Strength: 52.0 MPa

Young's Modulus: 20.6 GPa

Poisson's Ratio: 0.16

Failure Cause: Intact Rock


AND09-01-05 Stress vs. Strain
Peak Strength = 52.0 MPa
Bulk Density = 2.461 g/cm3
Young's Modulus = 20.6 GPa
Poisson's Ratio = 0.16
Axial Strain
Lateral Strain
Axial Sress (MPa)
Average Strain (με)
60.0
50.0
40.0
30.0
20.0
10.0
0.0
-1000
-500
0
500
1000
1500
2000
2500
3000
3500

SAMPLE: AND09-01-06

Length: 112.65 mm

Diameter: 45.26 mm

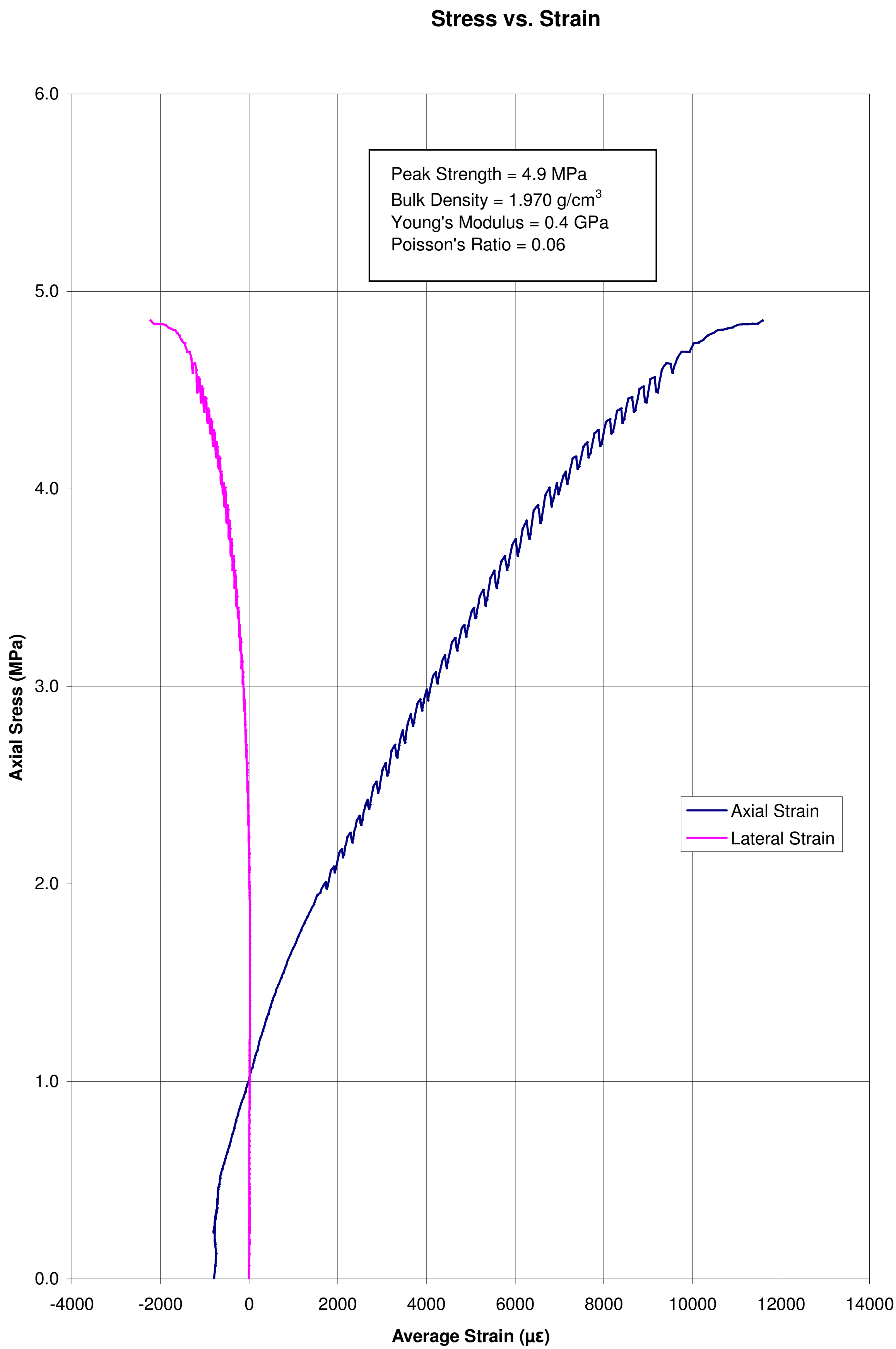
Density: 1.970 g/cm^3

Peak Strength: 4.9 MPa

Young's Modulus: 0.4 GPa

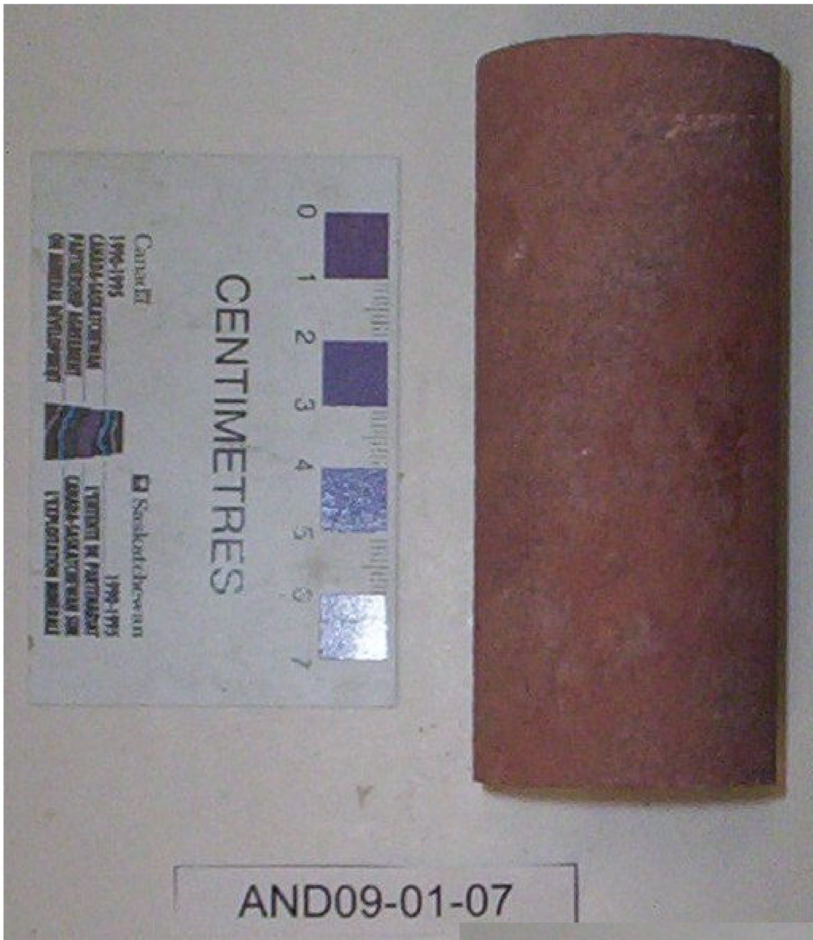
Poisson's Ratio: 0.06

Failure Cause: Intact Rock



Stress vs. Strain
Peak Strength = 4.9 MPa
Bulk Density = 1.970 g/cm^3
Young's Modulus = 0.4 GPa
Poisson's Ratio = 0.06
Axial Strain
Lateral Strain
Axial Sress (MPa)
Average Strain (με)
6.0
5.0
4.0
3.0
2.0
1.0
0.0
-4000
-2000
0
2000
4000
6000
8000
10000
12000
14000

SAMPLE: AND09-01-07



Length: 106.72 mm

Diameter: 45.14 mm

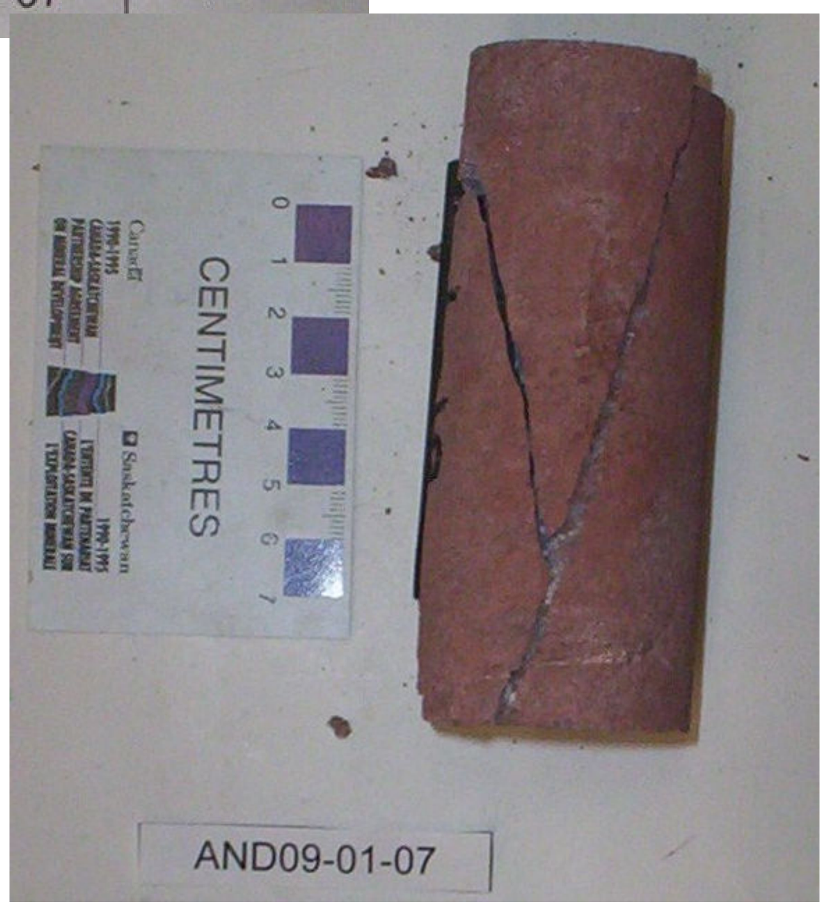
Density: 2.087 g/cm^3

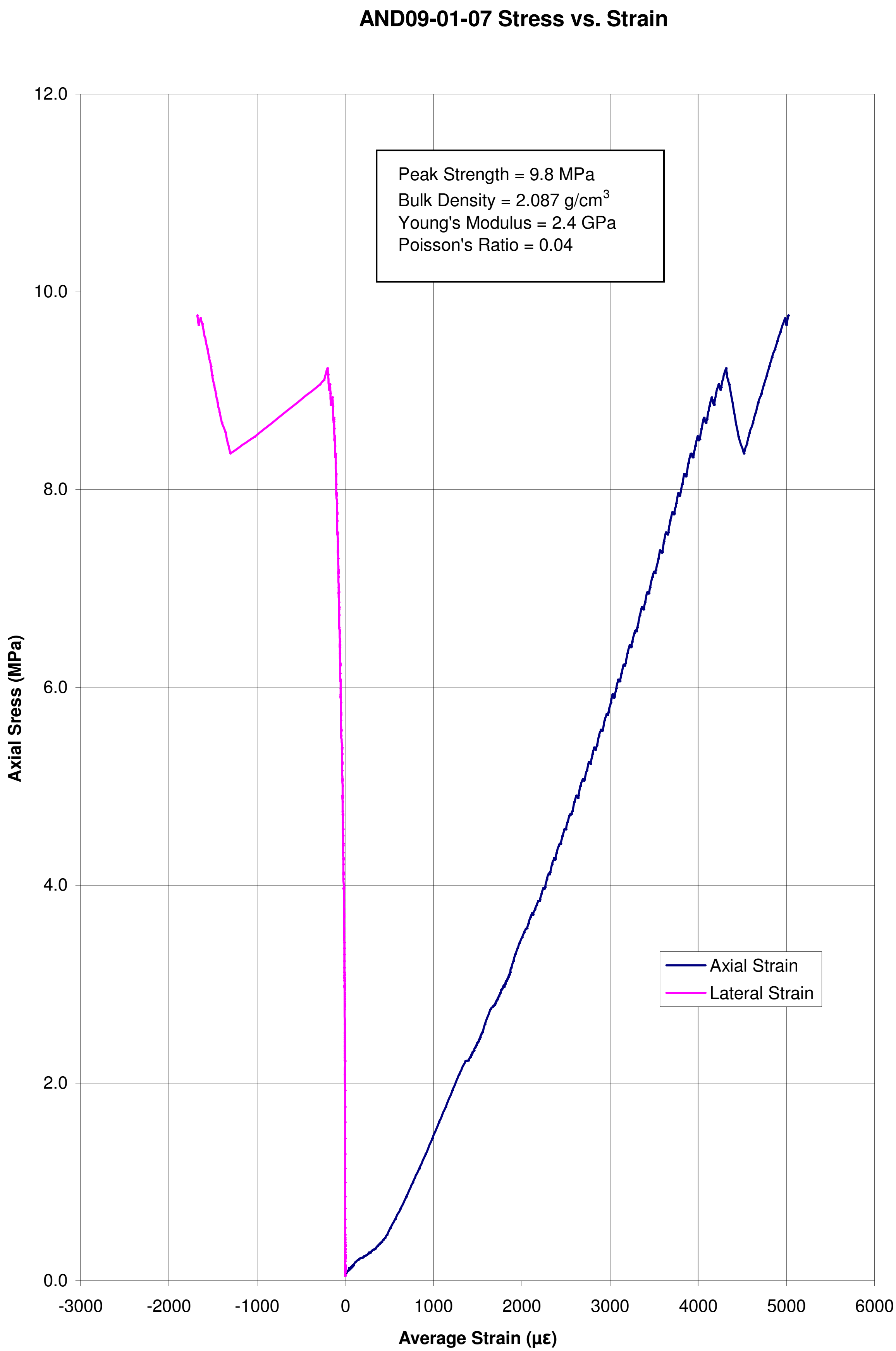
Peak Strength: 9.8 MPa

Young’s Modulus: 2.4 GPa

Poisson’s Ratio: 0.04

Failure Cause: Weakness Plane




AND09-01-07 Stress vs. Strain
Peak Strength = 9.8 MPa
Bulk Density = 2.087 g/cm3
Young's Modulus = 2.4 GPa
Poisson's Ratio = 0.04
Axial Strain
Lateral Strain
Axial Sress (MPa)
Average Strain (με)
12.0
10.0
8.0
6.0
4.0
2.0
0.0
-3000
-2000
-1000
0
1000
2000
3000
4000
5000
6000

SAMPLE: AND09-02-01

Length: 86.53 mm

Diameter: 45.34 mm

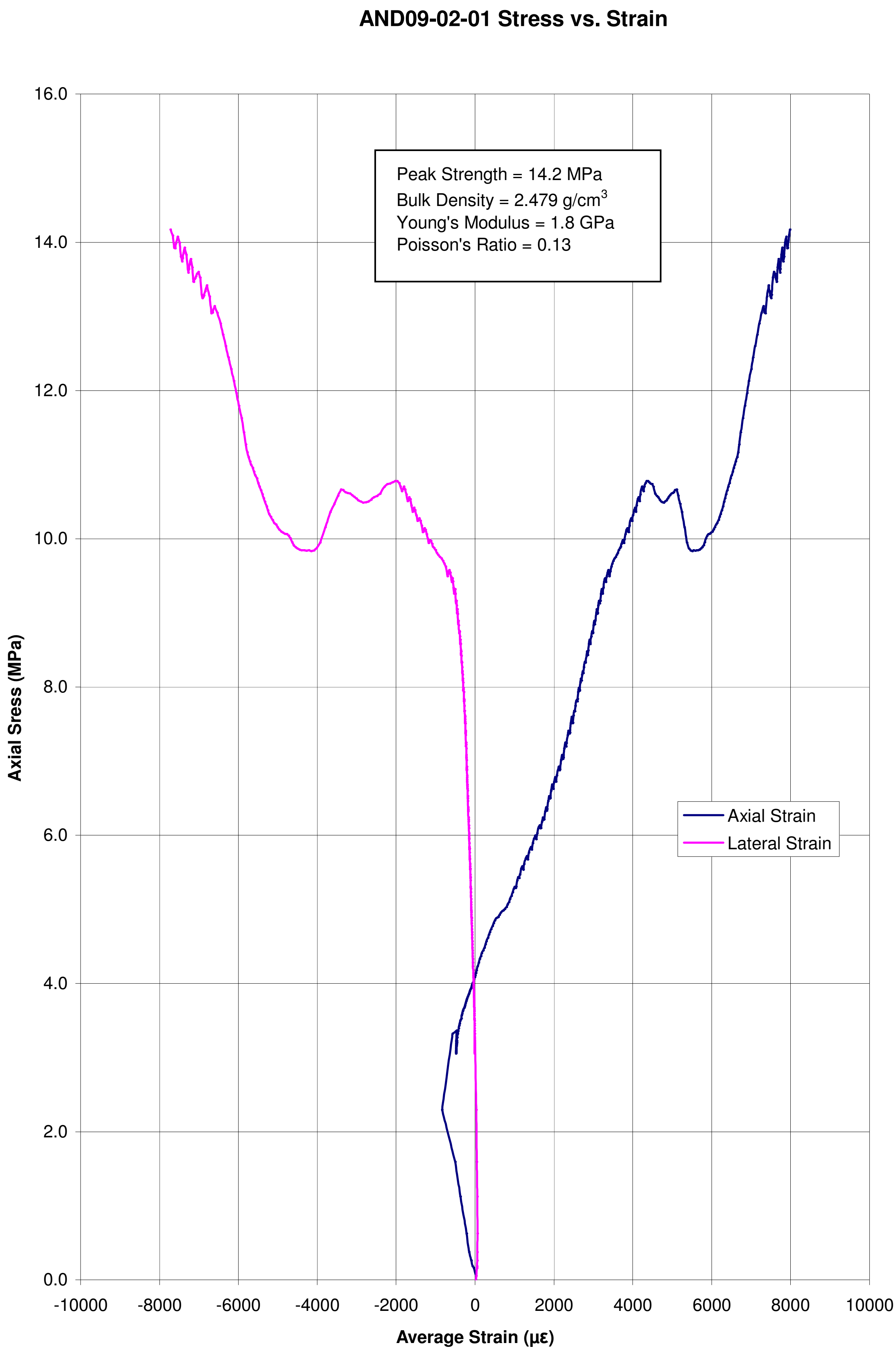
Density: 2.479 g/cm^3

Peak Strength: 14.2 MPa

Young's Modulus: 1.8 GPa

Poisson's Ratio: 0.13

Failure Cause: Intact Rock


AND09-02-01 Stress vs. Strain
Peak Strength = 14.2 MPa
Bulk Density = 2.479 g/cm3
Young's Modulus = 1.8 GPa
Poisson's Ratio = 0.13
Axial Strain
Lateral Strain
Axial Sress (MPa)
Average Strain (µε)
16.0
14.0
12.0
10.0
8.0
6.0
4.0
2.0
0.0
-10000
-8000
-6000
-4000
-2000
0
2000
4000
6000
8000
10000

SAMPLE: AND09-02-02

Length: 111.62 mm

Diameter: 45.34 mm

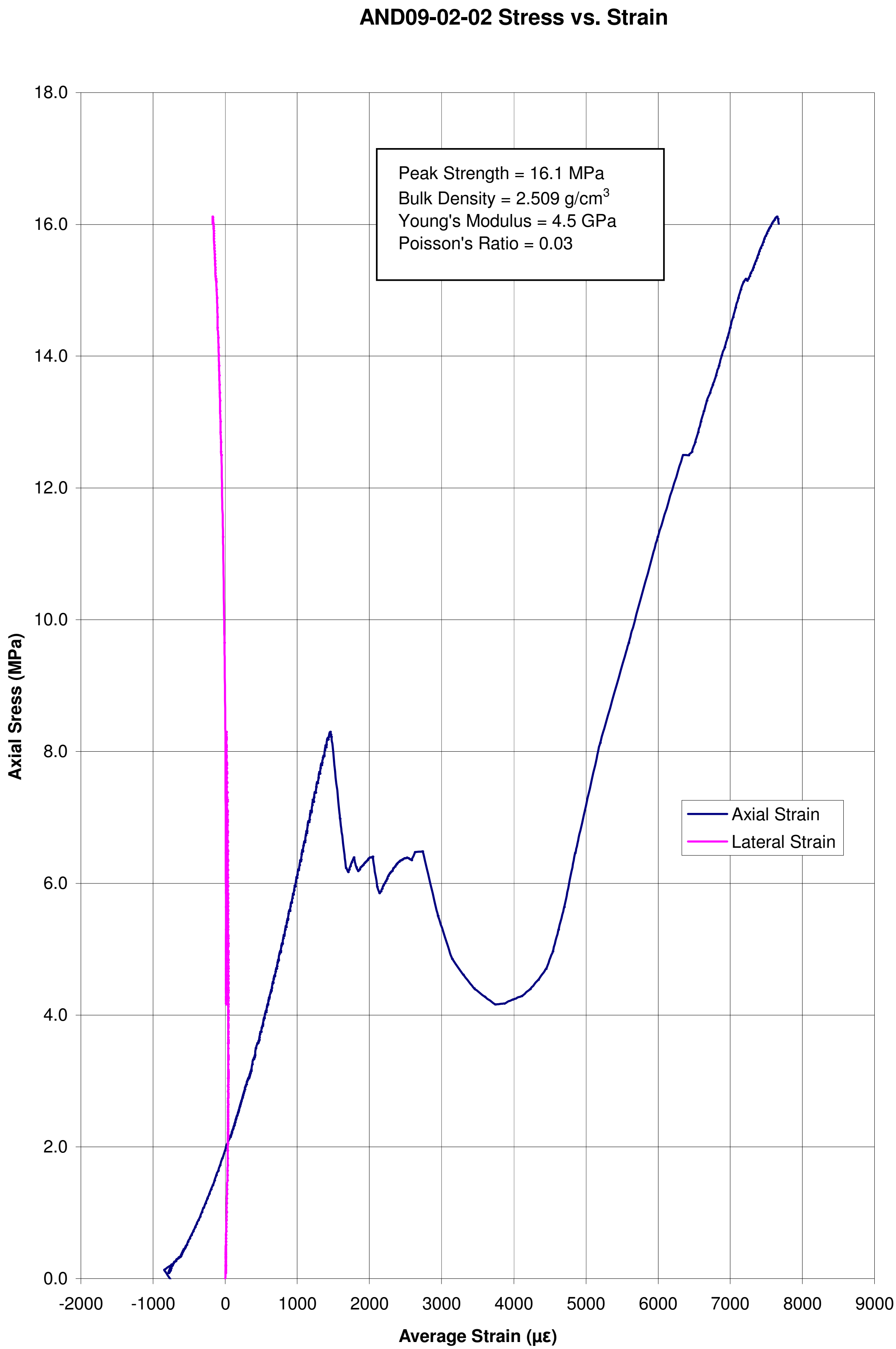
Density: 2.509 g/cm^3

Peak Strength: 16.1 MPa

Young’s Modulus: 4.5 GPa

Poisson’s Ratio: 0.03

Failure Cause: Foliation


AND09-02-02 Stress vs. Strain
Peak Strength = 16.1 MPa
Bulk Density = 2.509 g/cm3
Young's Modulus = 4.5 GPa
Poisson's Ratio = 0.03
Axial Strain
Lateral Strain
Axial Sress (MPa)
Average Strain (με)
18.0
16.0
14.0
12.0
10.0
8.0
6.0
4.0
2.0
0.0
-2000
-1000
0
1000
2000
3000
4000
5000
6000
7000
8000
9000

SAMPLE: AND09-02-03

Length: 109.02 mm

Diameter: 45.2 mm

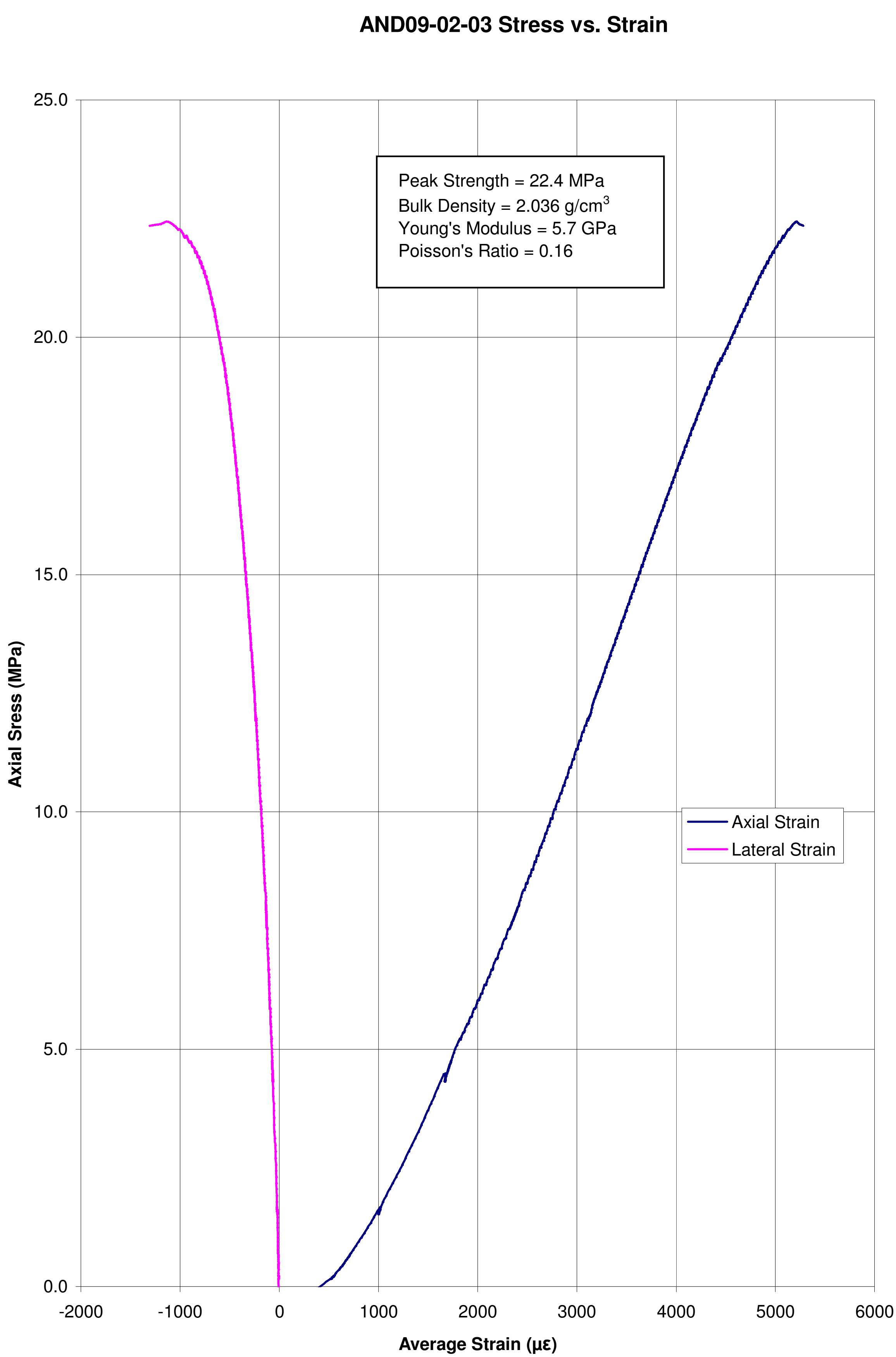
Density: 2.036 g/cm^3

Peak Strength: 22.4 MPa

Young's Modulus: 5.7 GPa

Poisson's Ratio: 0.16

Failure Cause: Weakness Plane


AND09-02-03 Stress vs. Strain
Peak Strength = 22.4 MPa
Bulk Density = 2.036 g/cm3
Young's Modulus = 5.7 GPa
Poisson's Ratio = 0.16
Axial Strain
Lateral Strain
Axial Sress (MPa)
Average Strain (με)
25.0
20.0
15.0
10.0
5.0
0.0
-2000
-1000
0
1000
2000
3000
4000
5000
6000

SAMPLE: AND09-02-04

Length: 107.82 mm

Diameter: 45.25 mm

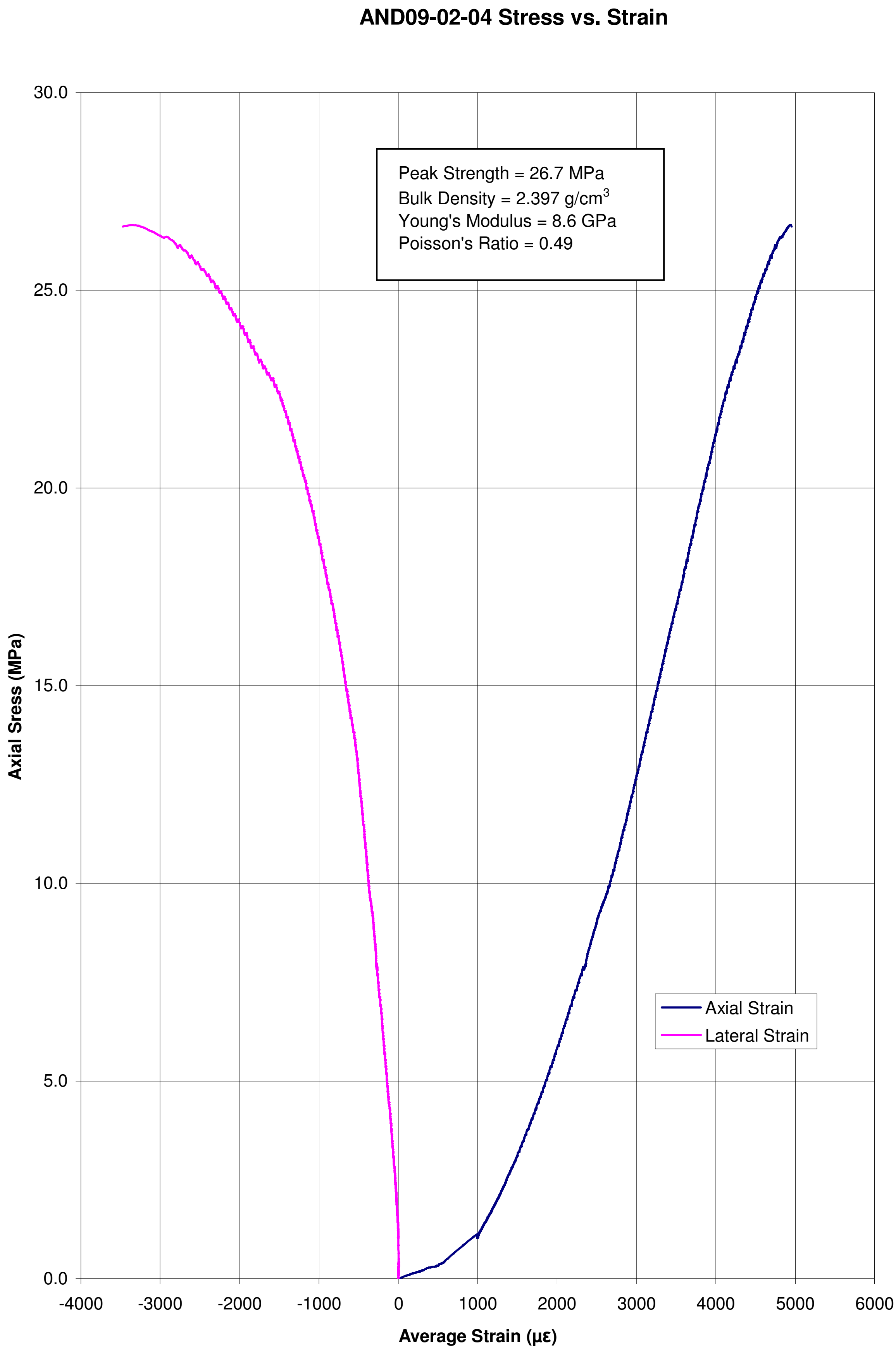
Density: 2.397 g/cm^3

Peak Strength: 26.7 MPa

Young’s Modulus: 8.6 GPa

Poisson’s Ratio: 0.49

Failure Cause: Intact Rock


AND09-02-04 Stress vs. Strain
Peak Strength = 26.7 MPa
Bulk Density = 2.397 g/cm3
Young's Modulus = 8.6 GPa
Poisson's Ratio = 0.49
Axial Strain
Lateral Strain
Axial Sress (MPa)
Average Strain (με)
30.0
25.0
20.0
15.0
10.0
5.0
0.0
-4000
-3000
-2000
-1000
0
1000
2000
3000
4000
5000
6000

SAMPLE: AND09-02-05

Length: 108.76 mm

Diameter: 45.67 mm

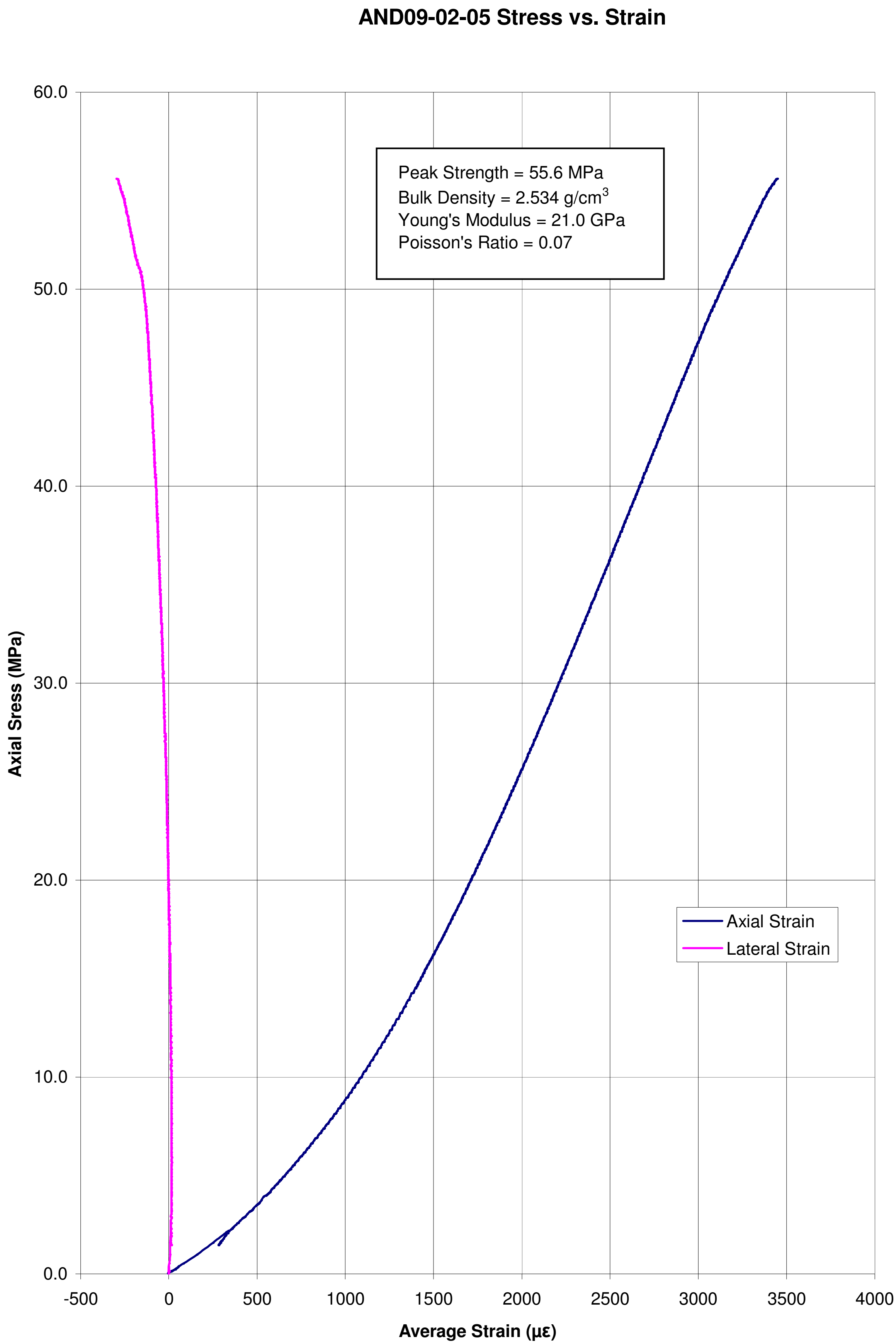
Density: 2.534 g/cm^3

Peak Strength: 55.6 MPa

Young's Modulus: 21.0 GPa

Poisson's Ratio: 0.07

Failure Cause: Intact Rock


AND09-02-05 Stress vs. Strain
Peak Strength = 55.6 MPa
Bulk Density = 2.534 g/cm3
Young's Modulus = 21.0 GPa
Poisson's Ratio = 0.07
Axial Strain
Lateral Strain
Axial Sress (MPa)
Average Strain (µε)
60.0
50.0
40.0
30.0
20.0
10.0
0.0
-500
0
500
1000
1500
2000
2500
3000
3500
4000

SAMPLE: AND09-02-06

Length: 111.83 mm

Diameter: 45.39 mm

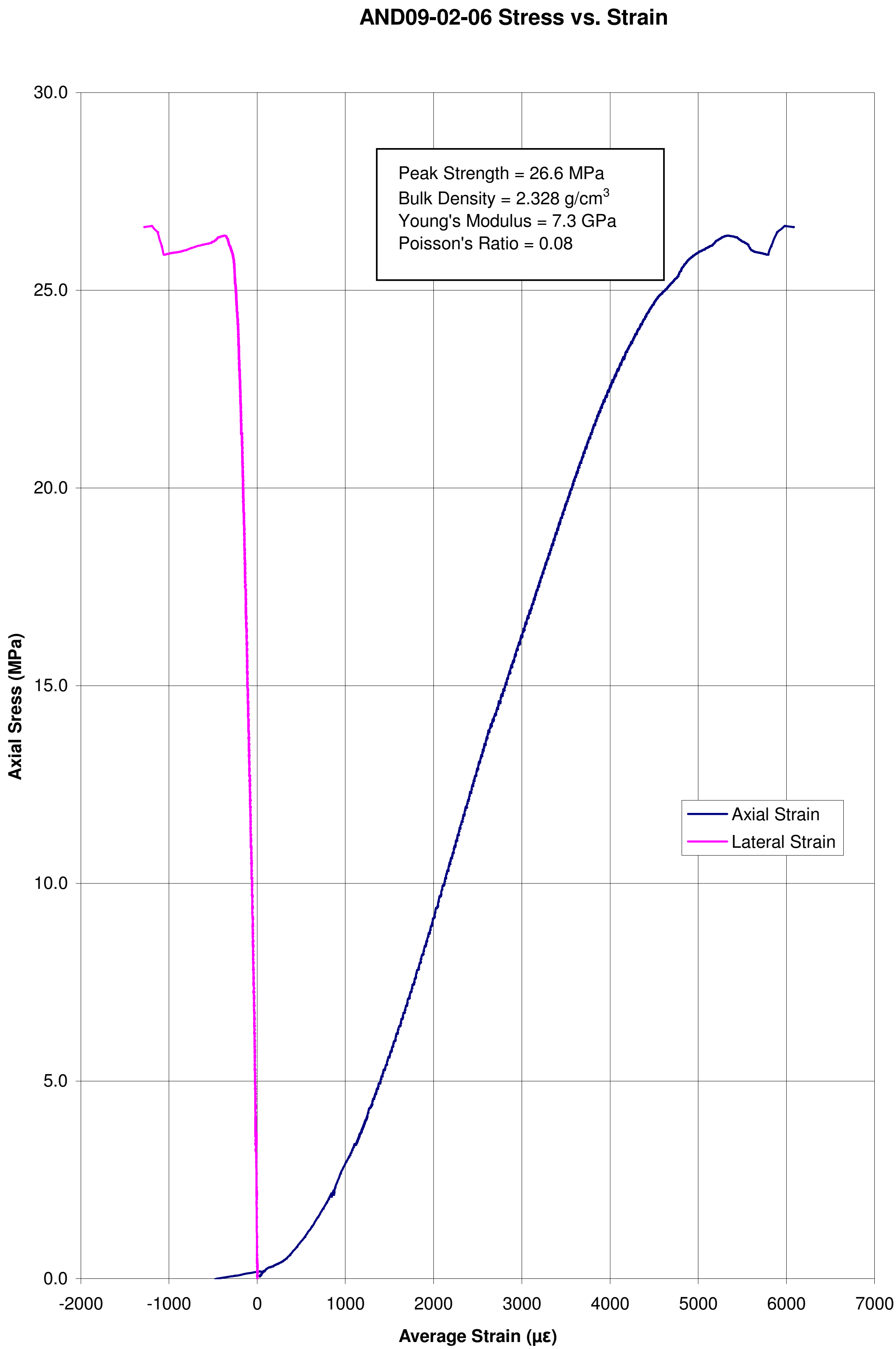
Density: 2.328 g/cm^3

Peak Strength: 26.6 MPa

Young’s Modulus: 7.3 GPa

Poisson’s Ratio: 0.08

Failure Cause: Intact Rock



AND09-02-06 Stress vs. Strain
Peak Strength = 26.6 MPa
Bulk Density = 2.328 g/cm3
Young's Modulus = 7.3 GPa
Poisson's Ratio = 0.08
Axial Strain
Lateral Strain
Axial Sress (MPa)
Average Strain (με)
30.0
25.0
20.0
15.0
10.0
5.0
0.0
-2000
-1000
0
1000
2000
3000
4000
5000
6000
7000

SAMPLE: AND09-02-07

Length: 103.18 mm

Diameter: 44.32 mm

Density: 2.494 g/cm^3

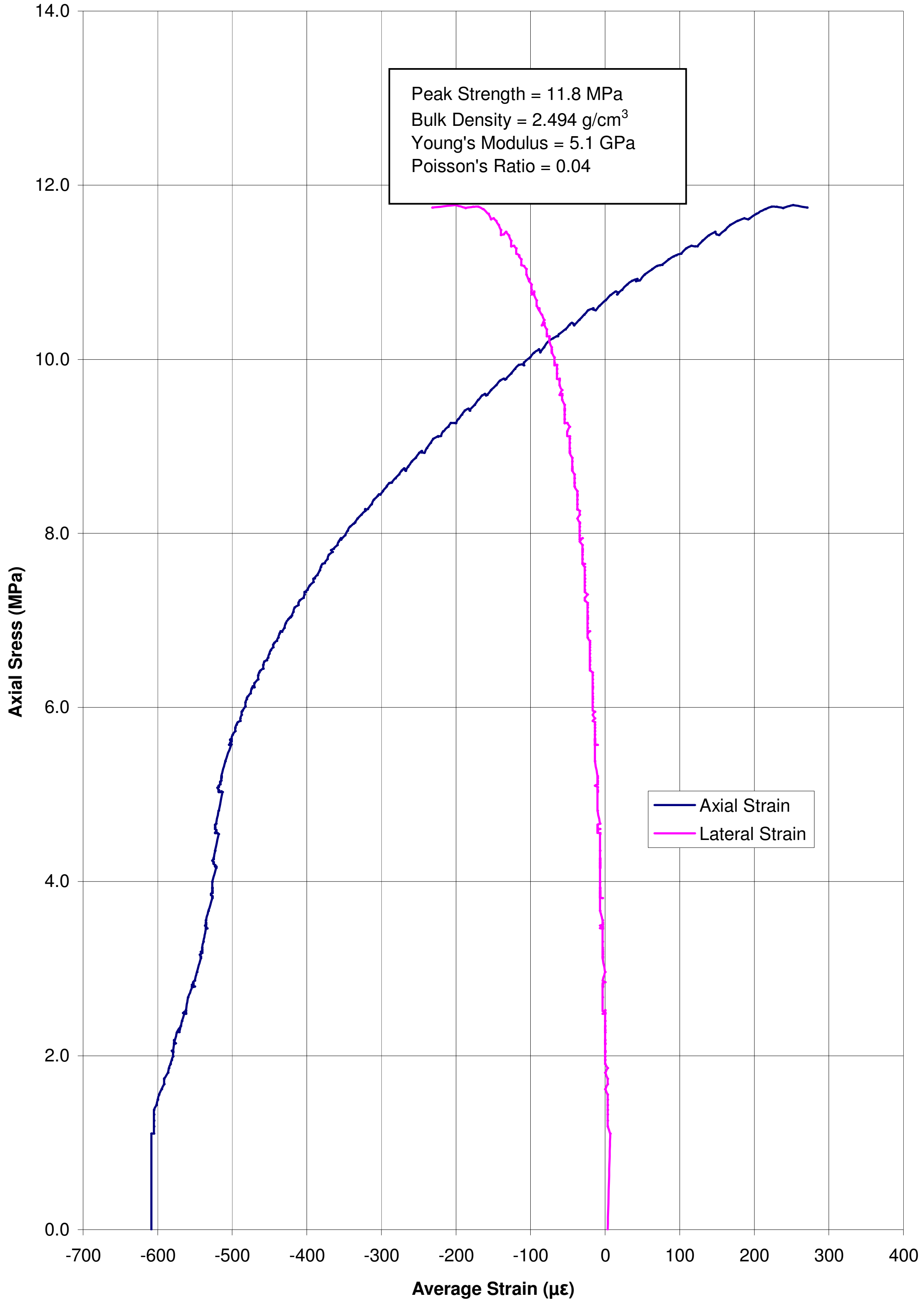
Peak Strength: 11.8 MPa

Young's Modulus: 5.1 GPa

Poisson's Ratio: 0.04

Failure Cause: Microfracture?

AND09-02-07 Stress vs. Strain



SAMPLE: AND09-02-08

Length: 94.50 mm

Diameter: 44.07 mm

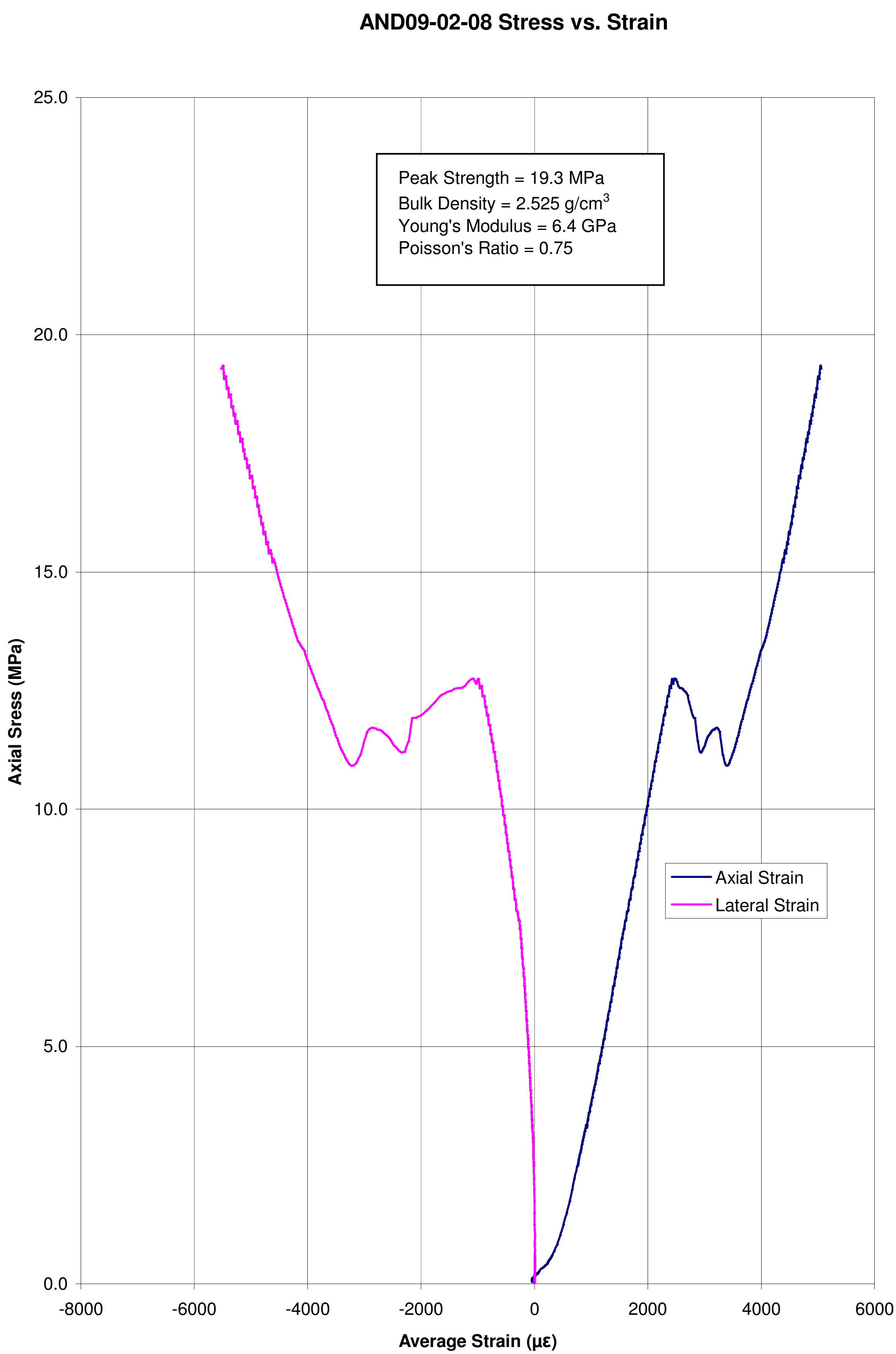
Density: 2.525 g/cm^3

Peak Strength: 19.3 MPa

Young’s Modulus: 6.4 GPa

Poisson’s Ratio: 0.75 ?

Failure Cause: Microfracture


AND09-02-08 Stress vs. Strain
Peak Strength = 19.3 MPa
Bulk Density = 2.525 g/cm3
Young's Modulus = 6.4 GPa
Poisson's Ratio = 0.75
Axial Sress (MPa)
25.0
20.0
15.0
10.0
5.0
0.0
-8000
-6000
-4000
-2000
0
2000
4000
6000
Average Strain (με)
Axial Strain
Lateral Strain

SAMPLE: AND09-02-1A

Length: 72.18 mm

Diameter: 45.14 mm

Density: 2.194 g/cm^3

Peak Strength: 17.5 MPa

Young's Modulus: 2.2 GPa

Poisson's Ratio: 0.10

Failure Cause: Intact

AND09-02-1A Stress vs. Strain