Kiggavik Project

Final Environmental Impact Statement

Tier 3 Volume 2V: Mine Geotechnical Reports

September 2014

History of Revisions

Revision Number	Date	Details of Revisions
01	December 2011	First Issue with Draft Environmental Impact Statement
02	September 2014	Issued for Final Environmental Impact Statement

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Attachments

1 Preface

This volume consists of reports that provide detailed analyses, statistical and interpretive, of the engineering geology and the controls on rock slope design for the Andrew Lake open pit and the Kiggavik open pits in parts A and B of this document. Discussions during the technical comment period of the Environmental Assessment process produced a summary of the design criteria from the reports contained in this Volume. This summary is provided below to assist the reader. The complete details of the slope designs are contained in Parts A and B of this appendix.

The design of open pit mines may be considered to follow a slope design approaches sometimes termed "traditional" or "observational". Table 1-1 below summarizes the selection of geotechnical parameters and factor of safety criteria for these approaches.

Table 1-1 Traditional and Obersvational Slope Design Approach Criteria

Slope Design Approach	Selection of Geotechnical Parameters	Factor of Safety Acceptance Criteria (FoS)
"Traditional"	"Reasonably Conservative" "Reasonably Worst Case" *Select Lower Bound Parameters for stability analyses.*	Minimum Factor of Safety (FoS) FoS =1.2 "Short Term" FoS =1.5 "Long Term"
"Observational" For the observational approach, monitoring of the performance of the structure, during and after construction, is carried out and the design is modified accordingly. *(As described in the preamble to Information Request AANDC 026)*	"Base Case" *Select Average Parameters for stability analyses.* *The design is modified during operations.*	Minimum FoS FoS = 1.3

While monitoring and re-assessment indicated in Table 1-1 are principle components of the observational approach, these activities are not unique to the observational approach. Slope performance assessments, instrumentation, monitoring, routine mapping, and periodic review of the continued validity of the slope design assumptions must be part of the best practices of any open pit operation.

The design of the open pits for the proposed Kigavik Project was based on the following:

- “Observational” and deterministic in terms of rock mass failure assessment – average parameters were selected with a Minimum FoS of 1.3 for “Reasonably Worst Case”.
- “Observational” and deterministic in terms of kinematic assessments slope designs at the bench scale. The minimum FoS at the bench scale was FoS = 1.1 (Read, Stacey, 2009, “Guidelines of Open Pit Slope Design, Table 1.1, page 9.)
- No distinction was made between assessments of short and long term conditions with respect to rock mass failure.
- The benefits of frozen ground for both rock mass stability and kinematic stability have been ignored.
- In terms of the engineering geology model of the open pit walls, the projections were based on a “Traditional” approach in that “Reasonably Worst Case” conditions, namely that the worst case rock mass conditions (altered Type 1 Rock Mass), were assigned to over 60% of the pit slopes.

The outcome of these assessments was the selection of reasonably conservative or assumed-worst case rock mass parameters and/or structural controls assigned to geotechnical domains based on conservative interpretations of the engineering geology model. Slope designs are mainly controlled by rock mass quality, with secondary controls being failures along discontinuities on bench faces.

The typical industry standard acceptance criteria for open pit designs are presented in Table 1-2.

Table 1-2 Typical Factor of Safety (FoS) and Probability of Failure (FoS) acceptance criteria (Read, Stacey, 2009)

Slope Scale	Consequence of failure	Acceptance Criteria[a]		
		FoS (min) (static)	**FoS (min) (dynamic)**	**PoF (max) P(FoS <= 1)**
Bench	Low-high[b]	1.1	NA	25–50%
Inter-ramp	Low	1.15-1.2	1.0	25%
	Moderate	1.2	1.0	20%
	High	1.2 – 1.3	1.1	10%
Overall	Low	1.2 – 1.3	1.0	15-20%
	Moderate	1.3	1.05	10%
	High	1.3 – 1.5	1.1	5%
a: Needs to meet all acceptance criteria				
b: Semi-quantitatively evaluated				

The 8m minimum catch-berm width (horizontal portion of a bench) that was incorporated into the designs was selected to conform to regulations in British Columbia.

Attachment A Andrew Lake Open Pit Design Report

August 28, 2014

FINAL REPORT

Geotechnical Recommendations for the Proposed Andrew Lake Open Pit - Support Document For Permit Application

Submitted to:
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PO Box 9204
Saskatoon, SK
S7K 3X5

Attention: Mr. Bradley Schmid

REPORT

Reference Number: 1213620158-006-R-Rev0-3000

Distribution:

1 Electronic Copy (pdf) - AREVA Resources Canada Inc.

1 Electronic Copy (Word) - AREVA Resources Canada Inc.

2 Hard Copies - Golder Associates Ltd.

Golder Associates

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 and 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 Andrew Lake deposit.

INTERPRETATION OF EXISTING INFORMATION

The existing data provided to Golder Associates Ltd. by AREVA Resources Canada Inc. in support of this study included:

- Golder 1989 interpretations and reporting.
- AREVA Resources Canada Inc. alteration logs for the 2009 geotechnical holes. This data were used to attempt to correlate strength and rock mass quality to intensity of alteration, as outlined in Appendices A and B.
- SRK core orientation data and geotechnical logs. Structure data captured by the one SRK oriented core hole at Andrew Lake were included in the kinematic assessment. This was done to maximize the available data for interpretation, and to help eliminate bias by providing data from a complimentary drillhole 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 floor and wall rock as altered and weak, or not. The key deficiency of the SRK data was that the hole locations do not provide information on the pit wall rock.

- AREVA Resources Canada Inc.'s interpreted regional fault traces based on geophysics anomalies (Beaudemont). These were conservatively assumed to occur within the pit footprint, because their interpreted trace locations are shown to do so.

- Shape files of the local fault interpretations, which trend east-west and northeast-southwest. It is important to note that the fault extents provided as shapes were not shown to extend to the pit walls in most cases. The conservative assumption used in this assessment was that the weaker rock mass extends horizontally 100 m from the base of the ultimate pit, resulting in weaker lower slopes all around the pit. This poorer rock quality due to alteration is interpreted to be most intense where the two fault trends intersect and/or within or in close proximity to mineralization zones. The weaker altered ground was also assumed to extend to the top of bedrock for pit walls along the regional northeast-southwest trend, as shown on Figure 9, based on 2009 drillhole data. Wall rocks away from the main northeast-southwest mineralization trend and away from cross-faulting and mineralization were assumed to be less altered, as shown on Figure 6, based on the results at depth at AND09-02.

ANDREW LAKE ROCK SLOPE DESIGNS

Very little factual information is known about the wall rocks at Andrew Lake, other than that which has been determined from the geotechnical holes drilled in 2009 as part of this study, and other information available from exploration holes that incidentally intersect wall rock on the margins of mineralization near where the toes of pit wall slopes will occur. Conservative interpretation of available information suggests that some of the pit slopes may be entirely comprised of weak, deformable rock masses due to clay and chlorite alteration associated with mineralization and faulting. The lower portions of all rock wall slopes and the Andrew Lake pit floor are interpreted to intersect a similarly weak, deformable rock masses. Away from the faulting and alteration halos, more competent rocks are interpreted to occur.

Stability analyses indicate that in the weaker rocks, the potential for rock mass failure will control the slope design, exacerbated by kinematic controls. Risk of instability in the weaker, altered rocks increases with the presence of groundwater pressure, as might occur on the lower slopes, at or near the base of permafrost, should slope depressurization methods not be sufficient. These concerns are based on the conservative assumption that the pit floor and lower pit walls will at some depth begin to be adversely affected by uplift (artesian) groundwater pressures while still within the permafrost horizon, because the pit floor will be very near the base of permafrost.

On portions of the rock slopes away from faulting and alteration, slope designs will be controlled by potential for plane and wedge type failures. Toppling failure may also occur, however, the 2009 information on joint spacing suggests that blocks with the tall column shape needed for toppling are unlikely, given the prevalence of sub-horizontal discontinuities.

The overall pit slope recommendations for Andrew Lake are considered to be conservative, based on assumptions made to infill data gaps within the analysis. There may be opportunity to optimize the pit slope angles once further investigation and additional analysis is complete.

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 12 m bench height.
- 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 75°.
- Single benching (12 m vertical) maximum in poorer rock masses.

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

Following issue of the initial draft of Andrew Lake pit slope recommendations in early December 2009, AREVA Resources Canada Inc. asked for slope design recommendations assuming a vertical bench height of 9 m at Andrew Lake, in addition to the height of 12 m that was assumed for the initial analysis. Golder Associates Ltd. has not carried out any detailed review with purpose to optimize this different bench configuration; however inter-ramp slope angles have been presented based on the same bench face angles proposed for a 12 m high bench height by design sector, single benching operations, and a minimum 8 m wide berm.

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 Andrew Lake, 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 60 m 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 overburden, to allow room for slope maintenance and drainage control over the life of mine.

OPERATIONAL CONSIDERATIONS

Operational considerations for the Andrew Lake pit design 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 Andrew Lake pit in order to relieve ground water pressures below the permafrost layer.
- Surface water protection will be required around the open pit 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

Golder Associates Ltd. (Golder) has prepared this document in a manner consistent with that level of care and skill ordinarily exercised by members of the engineering and science professions currently practising under similar conditions in the jurisdiction in which the services are provided, subject to the time limits and physical constraints applicable to this document. No warranty, express or implied, is made.

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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, hydrogeological testing, groundwater sampling, instrumentation, 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 mine at Andrew Lake. Although geotechnical core logging was conducted at the End Grid deposit and at Kiggavik Main Zone and Centre Zone, geotechnical aspects of mine development for these deposits will not be discussed in this report. The data findings for End Grid and Kiggavik Main Zone and Centre Zone from the 2009 field investigation are presented in a separate report.

Legend

Community

Proposed Kiggavik - Sissons Location

Reference
ESRI Canada Data
Atlas of Canada Data

PROJECT

AREVA

AREVA RESOURCES CANADA INC.
KIGGAVIK PROJECT

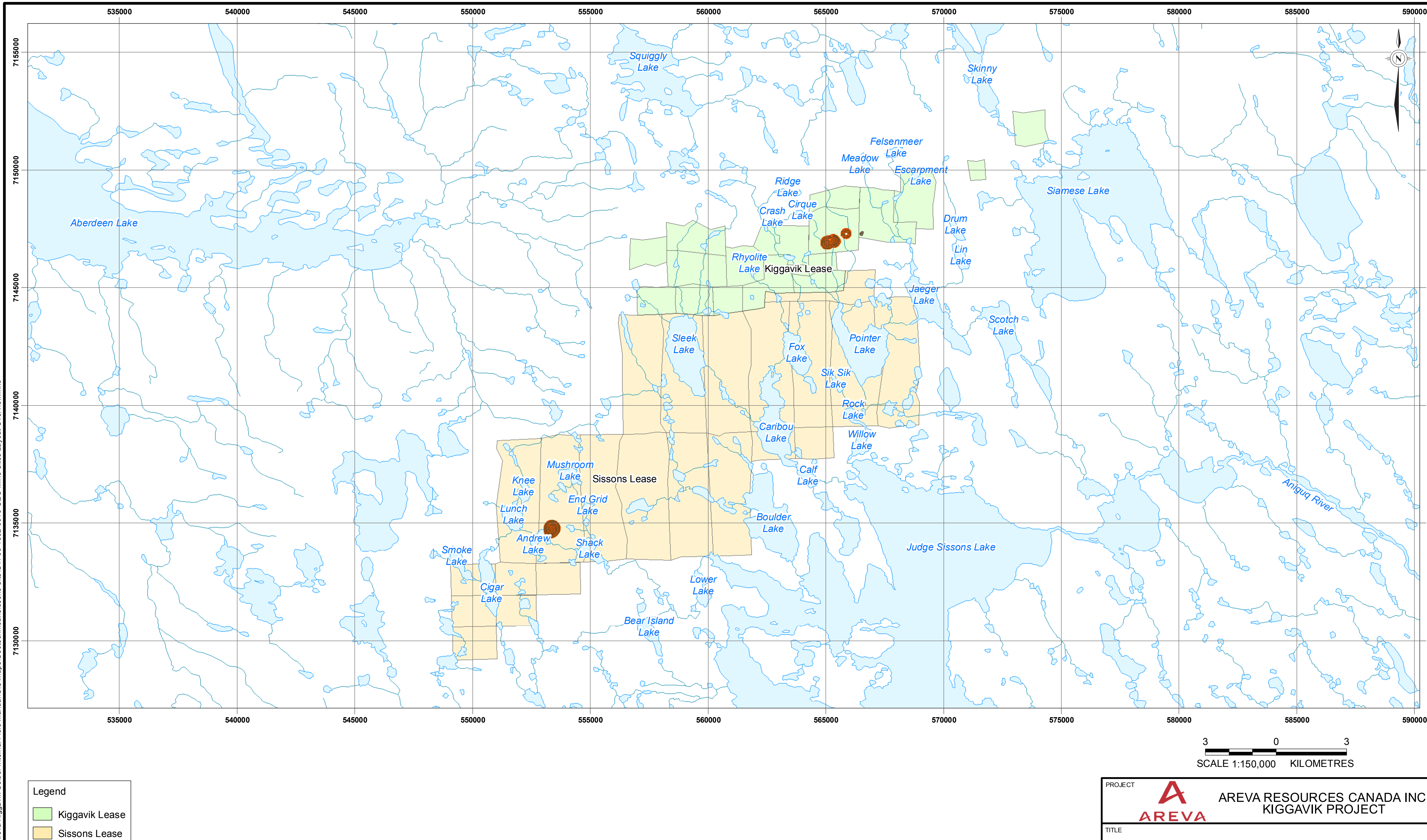
TITLE

GENERAL SITE LOCATION

Golder Associates
Saskatoon, Saskatchewan

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		

G:\2009\1362\09-1362-0613 Areva Kiggavik Golder Internal Field Manual\GIS\Maps\Geotechnical\Sissons 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\Sissons 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 this study included:

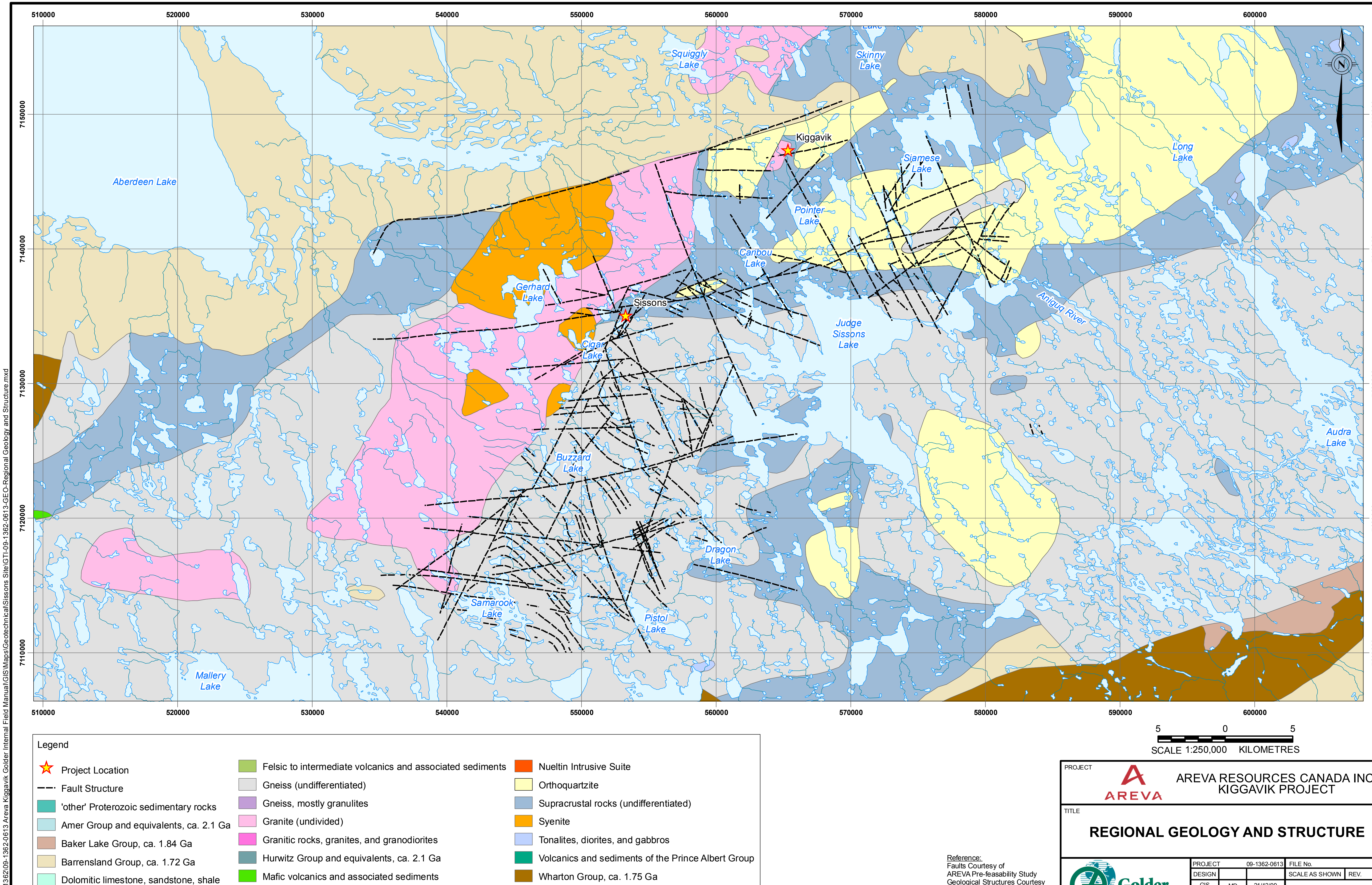
- AREVA alteration logs for the 2009 geotechnical holes. These data were used to attempt to correlate strength and rock mass quality to intensity of alteration, as outlined in Appendices A and B.
- SRK core orientation data and geotechnical logs. Structure data captured by the one SRK oriented borehole at Andrew Lake were included in the kinematic assessment. 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 floor 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.
- AREVA's interpreted regional fault traces based on geophysics anomalies (Beaudemont). These were conservatively assumed to occur within the pit footprint when shown to do so.
- Shape files of the local fault interpretations, which trend east-west and northeast-southwest. 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. The conservative assumption used was that the weaker rock mass extends horizontally 100 m from the base of the ultimate pit, resulting in weaker lower slopes all around the pit. This poorer rock quality due to alteration is interpreted to be most intense where the two fault trends intersect and/or in within or in close proximity to mineralization zones. The weaker altered ground was also assumed to extend to the top of bedrock for pit walls along the regional northeast-southwest trend, as shown on Figure 9, based on 2009 borehole data. Wall rocks away from the main northeast-southwest mineralization trend and away from cross-faulting and mineralization were assumed to be less altered, as shown on Figure 6, based on the results at depth at AND09-02.

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 64º26' 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 northeastern 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.



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.	
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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\Sissons Site\GTI-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 Sissons Area Deposits

The Sissons deposits are located in pelite and arenitic metasediments overlying granitic gneisses and granodiorites (AREVA 2007). These formations have been strongly metamorphosed and altered, tectonised, and intruded by lamprophyres, syenites and granites. The rocks have gently dipping foliation, small scale recumbent folding and low angle thrusting. The area also consists of steeply dipping brittle deformation zones that trend east-northeast to north-northeast as a conjugate set. Three main faults (Sissons North, Buzzard Lake and Andrew Lake) intersect the area.

The Andrew Lake deposit is located on a major east-northeast structure (AREVA 2007) (Figure 3). This region has seen several episodes of hydraulic brecciation mainly within the granite and syenite rocks, and to a lesser extent in the metasediment units. The subvertical faulting associated with the Andrew Lake deposit governs the extension of the mineralization.

Three main mineralized lenses have been identified at Andrew Lake (AREVA 2007). These are associated with strongly altered metasediments (metagreywackes and metapelites), altered paragneiss, and less altered metasediments. The lenses overlie each other, and are separated by a quartz breccia and paragneiss. Mineralization within the Andrew Lake area occurs between 70 mbgs and 270 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, three boreholes were drilled at Andrew Lake during the 2009 season. All three holes were geotechnically core logged with core orientation, two holes were tested for hydraulic conductivity, and one instrument was installed. Details of the drill program conducted at Andrew Lake 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" (Golder 2009c).

In addition to the information collected at Andrew Lake in 2009, historical information was also considered when conducting the pit slope analysis. 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.

Table 1: Summary of Oriented Boreholes from 2007, 2008, and 2009 from Andrew Lake

Borehole #	Year Drilled	Northing	Easting	Collar Elevation (masl)*	Azimuth	Dip	Drill Depth (mAH)	Vertical Depth (mbgs)	Bit Size
AND09-01	2009 (G)	7134927	553548	167.94	228	-70	342	321	NQ3
AND09-02	2009 (G)	7134809	553319	167.07	330	-65	330	299	NQ3
AND09-03	2009 (G)	7134574	553312	166.21	284	-70	327	307	NQ3
ANDW-08-04	2008 (SRK)	7134893	553326	168	092	-60	300	260	HQ3

G = Golder (2009)

SRK (2007/2008)

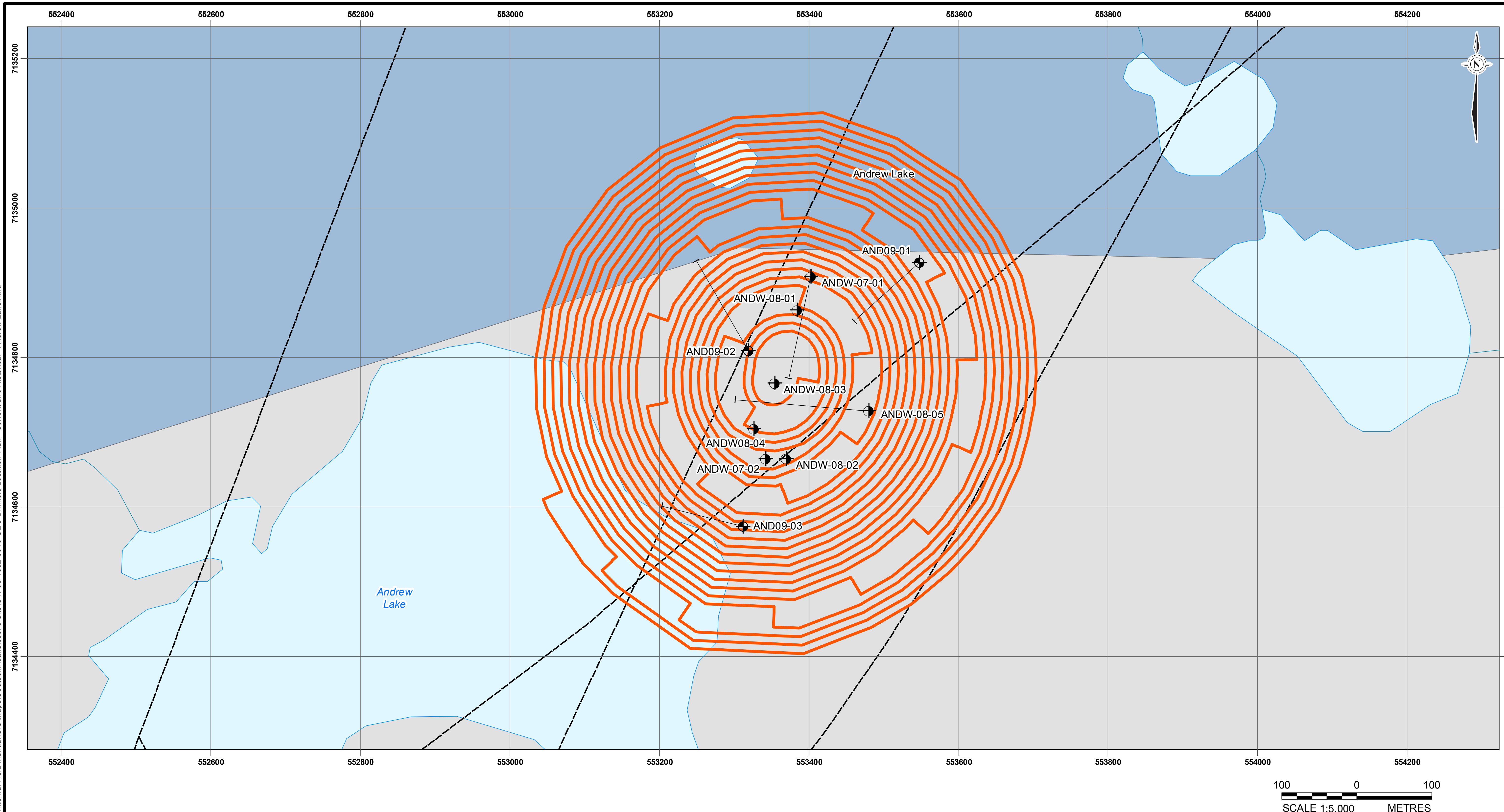
Coordinates in UTM NAD 83 Zone 14

* 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



Legend

- Borehole
- Historical Borehole
- Hole Trace
- Andrew Lake Pit
- Fault
- Gneiss (undifferentiated)
- 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
ANDREW LAKE

Golder Associates
Saskatoon, Saskatchewan

PROJECT		09-1362-0613	FILE No.	
DESIGN			SCALE AS SHOWN	REV. 0
GIS	MD	21/12/09	**FIGURE: 4**	
CHECK	EM	21/12/09		
REVIEW	LG	22/12/09		

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

2.0 ENGINEERING GEOLOGY

The Andrew Lake host rock consists of moderately strong hematite paleo weathered metasediment and granitoid rocks, with a generally east dipping trend. In areas where alteration does not mask rock structure, a network of sub vertical to sub horizontal microfractures and veining are visible. The Andrew Lake area has varying degrees of leaching and alteration, including hematite, chlorite, sericite and clay alteration which is usually structurally related. More intense alteration is generally associated with zones of faulting and brecciation, and with the ore zones, which decreases the quality of the rock mass. The ore bodies at Andrew Lake are generally flat lying, are separated by zones of brecciation and paragneiss, and generally appear to be fault controlled. The wall rocks of the proposed Andrew Lake pit will be constructed in metasediments, brecciated zones, and paragneiss, all with varying degrees of alteration.

The derivation of a stable economic pit design for this deposit 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 best-guess interpretations of the wall rock character.

2.1 Material Properties

Representative samples for the Andrew Lake pit were sampled 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 19 Unconfined Compressive Strength (UCS) tests were carried out. 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 199 valid PLT tests were carried out on the Andrew Lake rock types respectively. Detailed results of the material strength testing program are presented in Appendix A.

An assessment of mineral alteration (argilization and chloritization) and its effect on the intact strength and density of the rock were carried out for the Andrew Lake rock samples. Alteration indices were logged by AREVA for the 2009 Andrew Lake geotechnical boreholes, and an index value was assigned to reflect the relative degree of alteration (0 = non-altered and 4 = intensely altered). The strength testing results show a good correlation between the degree of alteration, related decrease in material density, and decrease in strength.

Based on the results of the assessment, alteration is assumed to be the main control on rock strength at the Andrew Lake site, where faulting and ore zone halo alteration are more pronounced. An example figure illustrating the measured material density with UCS strength is shown on Figure 5. The density determinations presented in Figure 5 show that there is a noticeable decrease in density with increasing alteration.

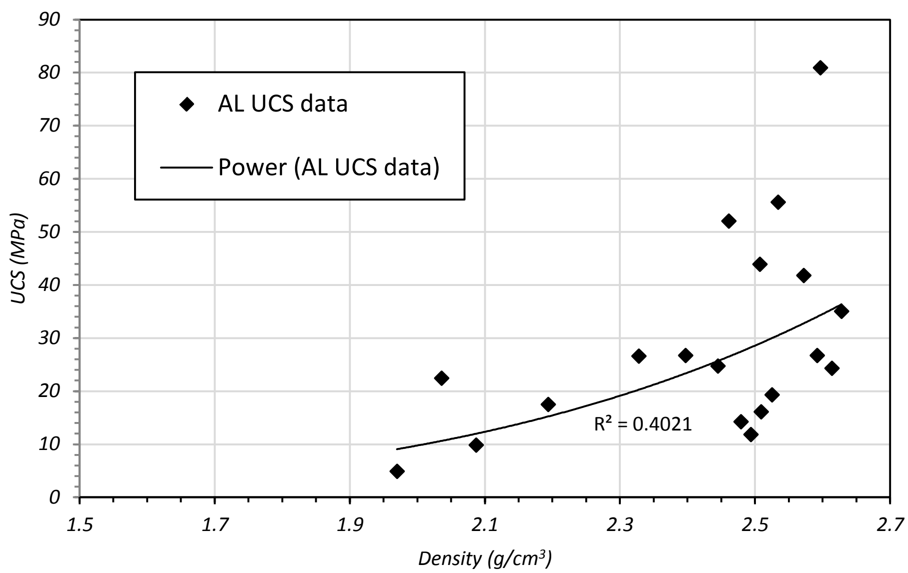


Figure 5: UCS versus Density for the 2009 Andrew Lake Testing (see Appendix A)

Very little factual information is known about the wall rocks at Andrew Lake, other than that which has been determined from the geotechnical holes drilled in 2009 as part of this study, and other information available from exploration holes that incidentally intersect wall rock slope toes. Conservative interpretation of available information suggests that some of the pit slopes may be entirely comprised of weak, deformable rock masses due to a combination of regional faulting and clay and chlorite alteration. Similarly, the lower portions of all rock slopes are interpreted to also intersect weak, deformable rock masses. Away from the faulting and alteration halos, competent rocks are interpreted to occur. The granites and metasediments at depth (gneisses) away from zones of alteration are shown to be moderately strong to strong. Results from the laboratory strength testing program are summarised in Table 2.

Table 2: Andrew Lake - Summary of UCS Testing by Rock Type and Alteration (see Appendix A).

Rock Type	Alteration	# Tests	UCS (MPa)	Youngs Modulus (GPa)	Density (g/cm^3)	Poissons Ratio
Metasediments < 200 m depth	Low to Mod. Alt (Af = 1 to 2)	8	29.9 +/- 13.2	11.3 +/- 6.5	2.46 +/- 0.17	0.08 +/- 0.04
	Mod. to High Alt (Af = 3 to 6)	3	15.9 +/- 1.7	2.8 +/- 1.5	2.39 +/- 0.17	0.09 +/- 0.05
Granite	Low to Mod. Alt (Af = 1 to 2)	2	66.5 +/- 20.4	22.4 +/- 2.5	2.53 +/- 0.10	0.15 +/- 0.02
	Mod. to High Alt (Af = 3)	1	24.7	6.1	2.45	0.24
Inferred Fault	Low to High Alt. (Af = 1 to 6)	2	7.4 +/- 3.5	1.4 +/- 1.4	2.03 +/- 0.08	0.05 +/- 0.01
Quartz Breccia	Low Alt (Af = 1)	1	35.0	19.9	2.63	0.16

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

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 Andrew Lake site, particularly for rock units where alteration influenced weakening of the rock mass appears to be prevalent. 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 Andrew Lake Rock Mass Quality

A significant amount of interpretation and assumptions were required for developing a generalized model of the rock mass conditions at Andrew Lake. Based on the geology models (AREVA 2009) and regional geology (Figure 3), the Andrew Lake pit is inferred to be cross-cut by numerous fault structures, trending N-S to NE-SW, or orthogonally trending E-W. There is a general lack of understanding regarding the locations and conditions of these fault structures. It was assumed that these faults would be persistent to a substantial depth into the proposed pit shells. Borehole AND09-01 was drilled sub-parallel to some inferred fault structures, and ground conditions were shown to be poor to fair throughout the borehole. In both AND09-02 and AND09-03, the rock mass conditions at depth, away from the inferred zones of mineralization show improvement in both strength and quality. The extents of alteration or weakening of ground on either side of these faults is uncertain, however, if multiple faults were shown to intersect the pit shell in close proximity to one another, it was assumed that the majority of ground would show some deleterious conditions.

The locations of mineralization horizons at Andrew Lake are better understood, and have been defined in AREVA's geology models. The 2009 boreholes however did not target these zones of mineralization, therefore no direct assessment of the ore alteration 'halo' was carried out to any detail. It does appear that alteration related to mineralization would result in deleterious ground conditions, and in particular, reduced material strengths.

Geotechnical sections were developed based on the available 2009 data as well as 2008 holes carried out by others (SRK 2009). These sections are quite generalized due to the limited information available, particularly in the pit walls. An example section is plotted on Figure 6. A general lithological arrangement of upper metasediments, granitics, and lower metasediments (gneisses) was assumed throughout. These arrangements in rock types were not always observed in all of the 2008/2009 geotechnical boreholes, but these assumptions agree with the updated geological model (SRK 2008c) also shown on Figure 6.

At Andrew Lake, considerable variability in RMR was observed in the boreholes due to improved ground conditions with depth, as well as due to the inferred proximity to faulting and mineralization zones. The data was grouped by geology and alteration in order to make a better assessment of the varying geotechnical qualities for the various rock mass domains. The recommended RMR and strengths based on the statistical significant intervals of quality and strength for the upper metasediments, granitics, lower metasediments, and alteration zones and summarised in Table 3. The ranges in RMR reflect the variability in values for the upper bound, lower bound, and average cumulative distributions of the data. A number of comments are included on this table to substantiate the recommended values.

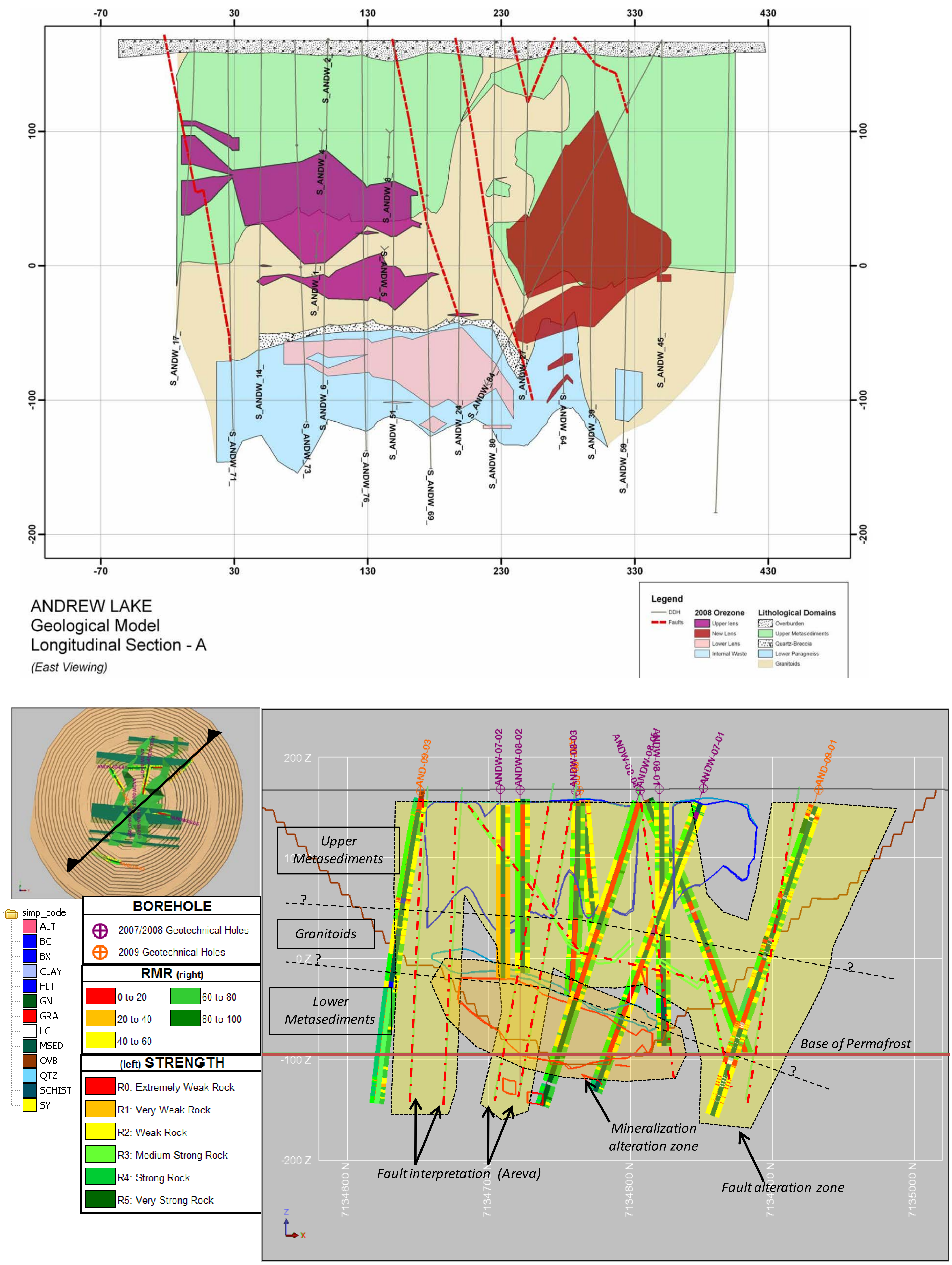


Figure 6: Generalized Geotechnical Section with Inferred Fault and Mineralization Altered Zone with Rock Strength, Lithology and RMR Plotted Along the Borehole Traces; Geological Model (SRK 2008c)

Table 3: Andrew Lake - Recommended RMR and Strength Parameters

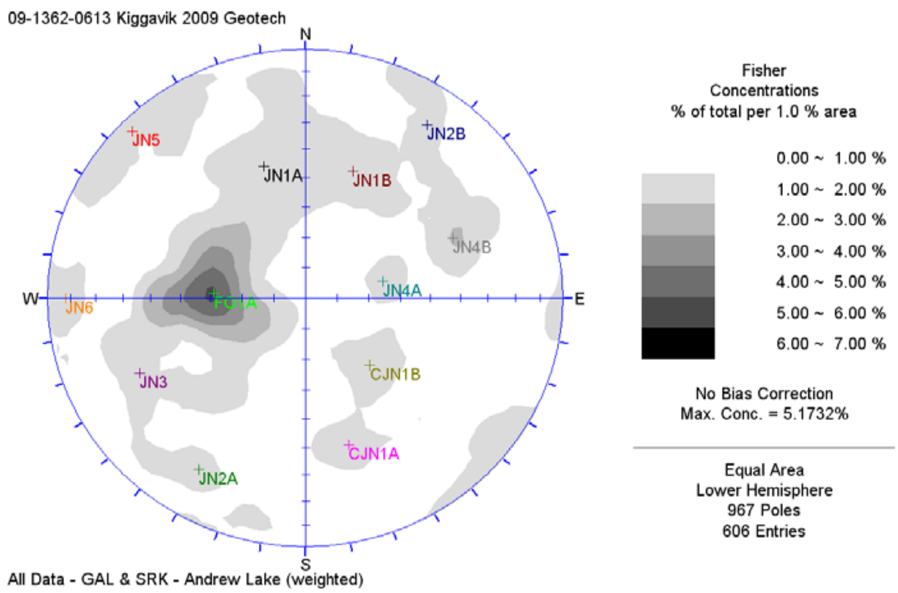
Rock Unit	RMR (1976)	Comment	Strength	Comment
Upper Metasediments (approximately less than 200 m depth)	**42 to 62** (fair)	Range of RMR for lower to upper bound limits for the 2009 Upper Metasediment data. Average RMR of nil to moderately altered ground. (Appendix B)	**R2/R3** (weak to moderately strong)	Average UCS = 29.9 MPa and average $Is_{(50)}$ = 2.2 MPa for upper metasediments (Appendix A)
Granites (all)	**52 to 65** (fair to good)	Range of RMR for lower to upper bound limits for the 2009 Granite data. Average RMR of nil to moderately altered ground. (Appendix B)	**R3/R4** (moderately strong to strong)	Average UCS = 66.5 MPa and $Is_{(50)}$ = 5.5 MPa (Appendix A)
Lower Metasediments/ Paragneiss (approximately greater than 200 m depth)	**52 to 74** (fair to good)	Range of RMR for lower to upper bound limits for the 2009 Lower Metasediments data. Average RMR of nil to slightly altered ground. (Appendix B)	**R4** (strong)	Average $Is_{(50)}$ = 7.3 MPa (Appendix A)
Fault or Mineralization Altered Zones	**42 to 52** (poor to fair)	Lower Bound RMR of all 2009 lithology data. Average RMR of 2009 highly altered data.(Appendix B)	**R2** (weak)	Average UCS = 7.4 to 24.7 MPa for moderate to highly altered rock or fault related zones (Appendix A)

2.3 Rock Mass Fabric

Structural data was collected through oriented core drilling during the 2009 geotechnical investigation. During the core logging process, Golder personnel used the driller's reference mark to scribe an orientation line on the low side of the drill core along the length of the drill run. A good match of reference lines between consecutive drill core runs was considered valid with 'high confidence'. When the orientation lines varied at approximately 40° to 60° from one another, this data was considered valid with 'moderate confidence'. In some cases, poor core conditions in zones of highly fractured or altered core prevented the line from being extended the length of the drill run and the core orientation line was lost. This frequently occurred for the Andrew Lake holes as the drill core was often broken and damaged in the weaker ground. When there was no match between orientation lines, or no continuity between oriented zones, the data was considered invalid or 'low' confidence.

A total of 1,243 oriented features were logged in 2009 at Andrew Lake. Quality control of the data as discussed above reduced the number of high to moderate confidence oriented features to 487 for the analysis as there was considerable difficulty obtaining validated oriented core data in the weak and altered rock units (refer to Figure C4 in Appendix C). The available structural data from the 2008 (hole ANDW-08-04) and 2009 (AND09-01 to -03) geotechnical programs was combined (606 features), and an assessment of discontinuity sets by borehole and rock type was carried out. The distribution of the oriented core data was fairly scattered, and due to the relatively low quantity of data, it was difficult to make an assessment on the relative persistence of any given set. Overall it was decided to define the Andrew Lake rock mass with a single structural domain based on the culmination of all data. The procedure and discussion related to the assessment of the rock mass structure is given in Appendix C.

The selected discontinuity sets are plotted with all contoured data (2008 and 2009) in the stereonet on Figure 7. A conventional naming system was used for the various sets (i.e., FO = foliation, JN = joint, CJN = conjugate joint). 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’. For the kinematic analyses, all discontinuity sets were conservatively assumed to be persistent or major, given the small data set of moderate to high confidence core orientation data for discontinuities relative to the size and extent of the pit walls. Future investigations, including detailed mapping on exposed benches should focus on better defining the rock mass structure, with possible delineation of structural domains and major/minor structures.



Set	Dip	Dip Dir
FO1A	29	93
JN1A	46	163
JN1B	45	200
JN2A	69	31
JN2B	73	214
JN3	60	65
JN4A	25	257
JN4B	52	247
JN5	84	135
CJN1A	51	344
CJN1B	30	317
JN6	82	90

Figure 7: Andrew Lake Oriented Core Data from 2008 and 2009 (contoured) with Selected Discontinuity Sets

Due to the general lack of oriented core data at Andrew Lake, it was difficult to make an accurate interpretation as to the deviation of structure within the various rock units. Therefore, as a conservative assumption, a large number of sets 8 main sets (12 sub-sets) were identified. The scatter in the data is likely associated with the data collection due to difficult drilling conditions, but also possibly associated with localized folding and faulting in the rock mass. The most dominant set is associated with the bedding/foliation (FO1A), and the other prominent set could be related to the east-west trend fault structures cross-cutting the pit (JN1). Sets JN2, JN3, JN5, and JN6 are predominantly high angle joint sets. Sets CJN1 and JN4 are shallow to inclined and considered to be sets possibly conjugate to the main foliation/ bedding orientation.

2.3.1 Rock Discontinuity Properties

At the Andrew Lake site, the rock conditions are weaker and more altered. Typical shear strength values back analyzed from the logged discontinuity roughness and alteration data are in the order of 30° to 35° depending on the relative alteration of the given discontinuity set.

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. These features sub-parallel the regional trend on which the End Grid and Andrew Lake deposits occur. While it is recognized that these features, interpreted by Beaudemont at a more regional scale, may not actually be accurate located at the pit scale, there did not appear to be better regional trend interpretations provided by previous work to project onto or along the pit shell. Consequently these regional fault interpretations were considered possible local controls on where weakest rocks may occur on the pit walls.

At the Andrew Lake deposit, the regional Andrew Lake Fault, which is steeply dipping and trends northeast at 030° (AREVA 2007), is interpreted to lie within the footprint of the proposed pit. It is possible that four dominant north-northeast trending faults mapped from historical core data are related to the Andrew Lake fault. The four mapped fault features appear to control the lateral extents of the uranium mineralization at Andrew Lake, and strike in the same orientation as the mineralization (SRK 2008c). At Andrew Lake, there is also evidence of structural features which cross-cut these faults, although there is limited drill core data to support this (SRK 2008). It is estimated that these cross-cutting features run sub-parallel to the north-northeast trending faults, and may have an influence on the mineralization at Andrew Lake (SRK 2008c). It has been estimated these cross cutting features dip steeply to the southwest (SRK 2008c).

The inferred faults provided in the geological model supplied by AREVA, as well as the regional fault trends for Andrew Lake are plotted on Figure 8. A number of fault structures are inferred to intercept the pit walls and floor at Andrew Lake, mainly trending N-S to NE-SW, or orthogonal to this trend striking E-W. Again there is not always agreement between the fault interpretations as given in the geological model, and the regional fault trends due to the method in which the regional fault trends were identified. Future work may be considered to clarify the locations and orientations of the major fault structures.

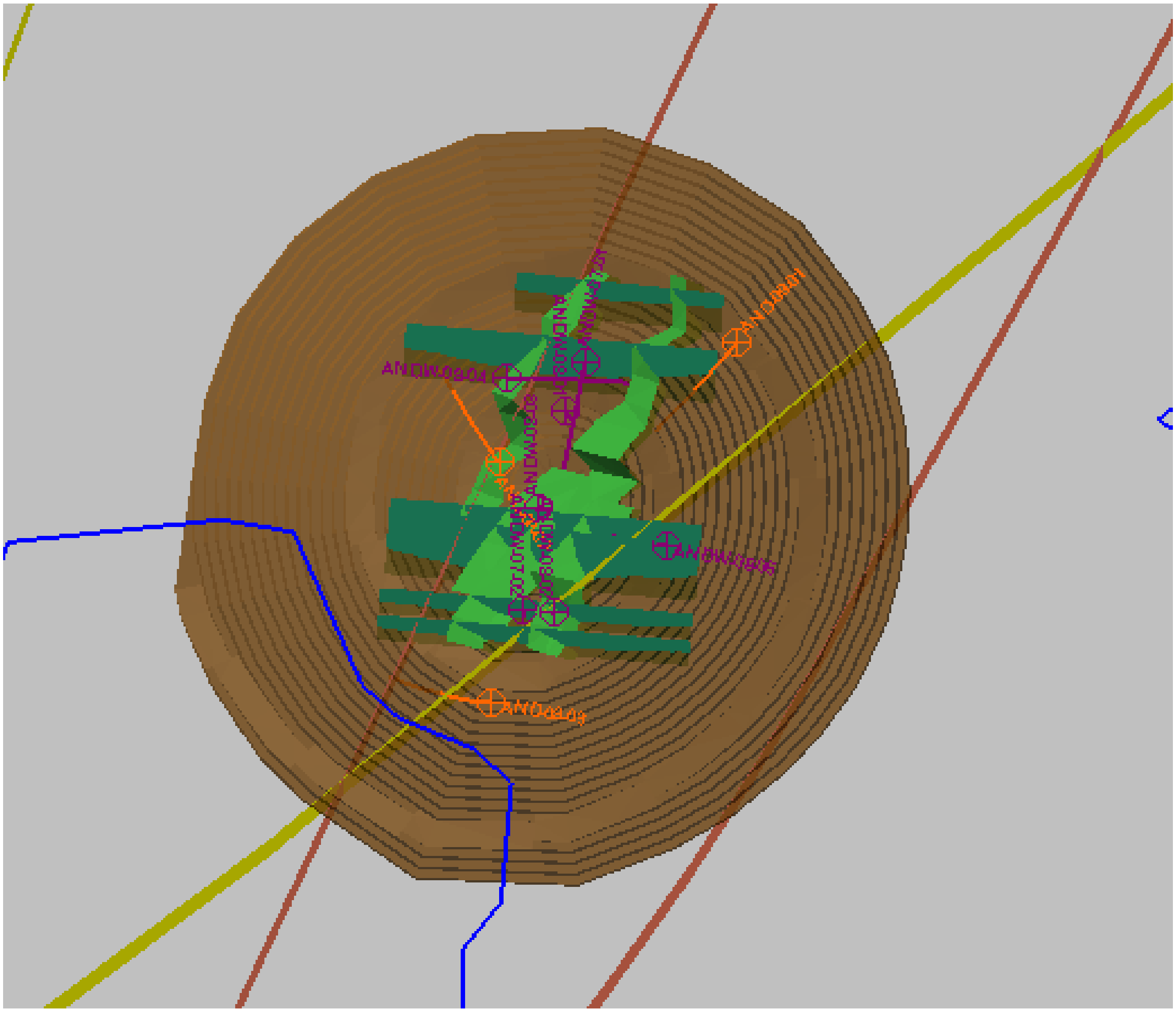


Figure 8: Andrew Lake Fault Traces from the Geological Model (green) with the Approximate Regional Geology Fault Traces (brown and yellow)

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 using 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. Based on these estimations the average joint spacing assumed for Andrew Lake ranged between 0.5 m to 0.9 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, one instrument equipped with both a thermistor string and a vibrating wire piezometer was installed at Andrew Lake. Data collected from this instrument are 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 at Andrew Lake is approximately 250 m below ground surface (mbgs) (-87 m above sea level [masl]).

Piezometer pressure readings documented and presented in Golder 2009(c) indicate that artesian pressure conditions exist in the groundwater below the Andrew Lake pit 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 in the area with sufficient depth that a talik has formed beneath it.

The Andrew Lake pit will be excavated to an ultimate depth at 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. This assumes that flow or pressure from below the permafrost layer that can contribute to instability will begin to influence the floor and walls before the pit floor actually breaches the base of permafrost horizon due to changes in ground temperature and rock mass conditions in response to mining, as the pit deepens.

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%. These water contents are all higher than the estimated 7% 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 occurs 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. Given the potential for poor rock mass conditions based on the very limited wall rock borehole information and the absence of alteration or structural interpretation projections for the pit walls, ravelling is conservatively considered a significant potential for the Andrew Lake pit benches. Consequently, double or triple benches are not recommended.

The angle 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 access ramp 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 three 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 end loader or hydraulic excavator, and trucks will be used for hauling;
- the practice of multi-benching will not 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 at Andrew Lake will be approximately 270 m.

AREVA has indicated to Golder that 12 m bench heights are being considered for Andrew Lake, while 9 m bench heights are being considered for the nearby Kiggavik sites. AREVA requested comment on how 9 m bench heights would change bench designs at Andrew Lake. Therefore, slope configurations have been formulated for both scenarios, see Section 5.

3.4 Geotechnical Benches

It is becoming common practice to periodically include extra wide catch benches at regular intervals in open pits, with 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 design is appropriate at Andrew Lake, 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 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 failure 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 failure. 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 Andrew Lake pit.

- **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 failure. 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 Andrew Lake pit.

- **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 the Andrew Lake pit.
- **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 2009 work estimated joint spacing on all sets, and 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 not 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. Ravelling failure might be considered somewhat problematic for the weaker and altered rock units at Andrew Lake.

3.6 Rock Mass Failure

A series of rock slope stability analyses were carried out for various rock mass configurations for the Andrew Lake pit. 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 and outlined in Appendices A and B. Appendix D discusses the methodology and results of the stability analyses in detail.

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 non-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 Andrew Lake, Type 1 slopes have been assumed and assigned to pit walls where faulting and associated alteration halos, in particular where there is 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 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 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, based on correlated geotechnical information from 2007 to 2009 boreholes. The main fault trends at Andrew Lake strike N-S to NE-SW, or E-W, therefore the wall orientations on-strike with these trends have been interpreted to be to be mainly Type 1 slopes. 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 80 m to 120 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. Figure D1 (Appendix D) illustrates the inferred rock mass domains considered at the Andrew Lake pit.

The results of the rock mass stability analyses for Andrew Lake are presented in Table 4 below. Due to the potential variability in alteration for the Type 2 slope, the overall slope angle recommendations have been divided into three domains; overall slope, upper slope (0 m to 150 m depth), and lower slope (150 m to 270 m depth). Rock mass failure appears to be a risk for the Andrew Lake rock mass under weakened and altered ground conditions in the Type 1 and Type 2 slope configurations. A perceived risk/sensitivity is included for each slope configuration based on the potential risks associated with varying ground conditions. In Type 1 ground conditions, the overall slope angle will likely have to be limited to 44° to maintain a reasonable factor of safety against failure. In Type 2 ground conditions, the overall slope angle can increase to 50˚, however, the lower slopes within the altered ground conditions should be limited to 44˚. If it can be proven that ground conditions exist within the pit walls for Andrew Lake, a significant increase in slope angle could be considered.

Table 4: 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, all parts of wall have same bench configuration	44˚	44˚	44˚	Moderate to high risk.
Type 2	Lower slope alteration, upper and lower slopes can have different bench configurations	50˚	55˚	44˚	Moderate risk.
Type 3	No slope alteration	55˚	55˚	55˚	Low risk.

3.7 Kinematics of Slope Failures

Kinematic analyses were carried out on the various slope kinematic design sectors for the Andrew Lake pit using the rock mass structural data derived from the 2009 program.

A stereographic projection (stereonet) was 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 were 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 Andrew Lake. 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 described in 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.

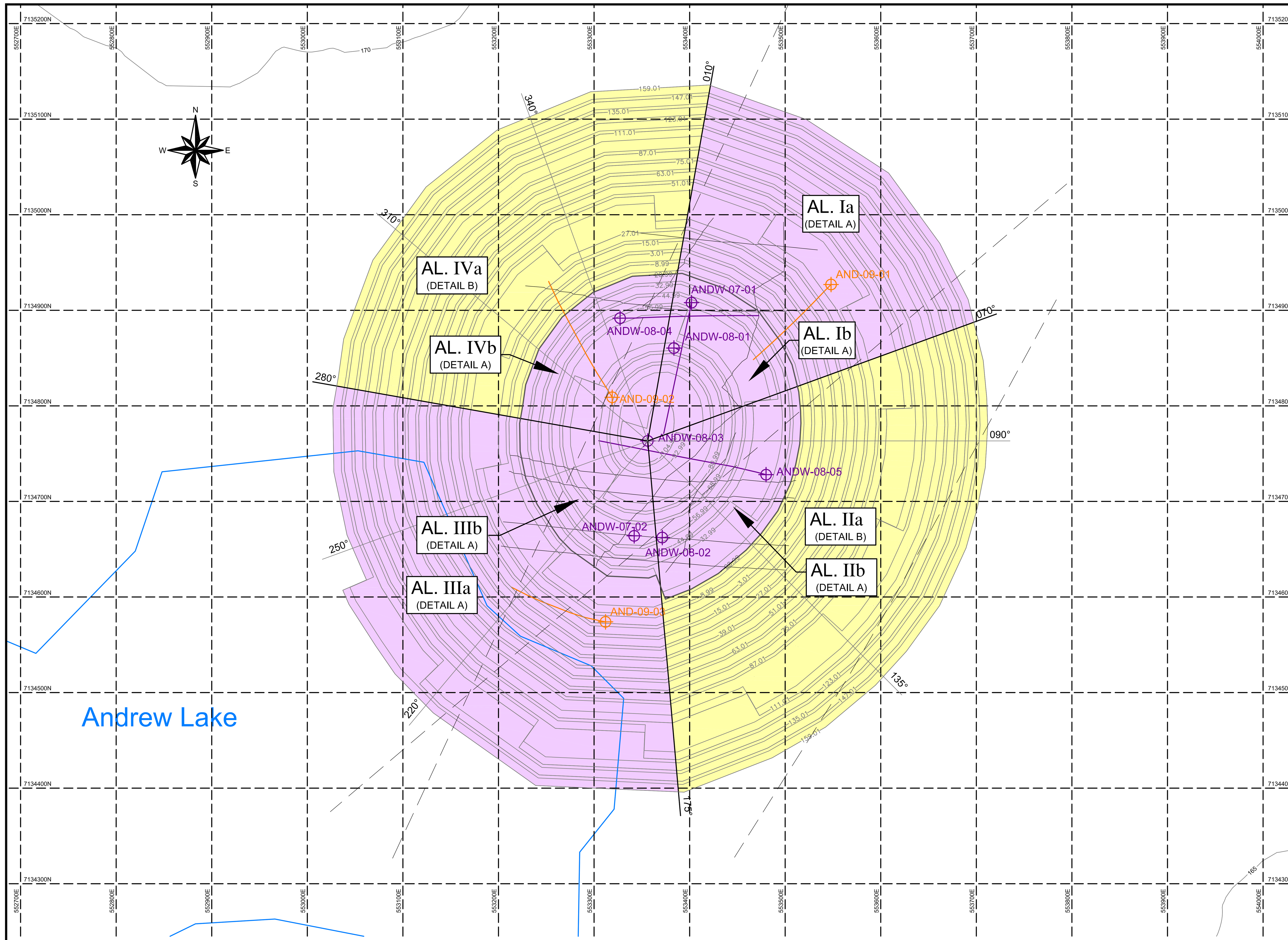
To assist stability analysis the Andrew Lake pit has been divided into a series of tentative design sectors. These sectors are very general and have been selected primarily on the basis of pit geometry since rock mass conditions are perceived to be main limiting factor on slope design. A total of 8 design sectors were analysed for the Andrew Lake pit geometry.

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

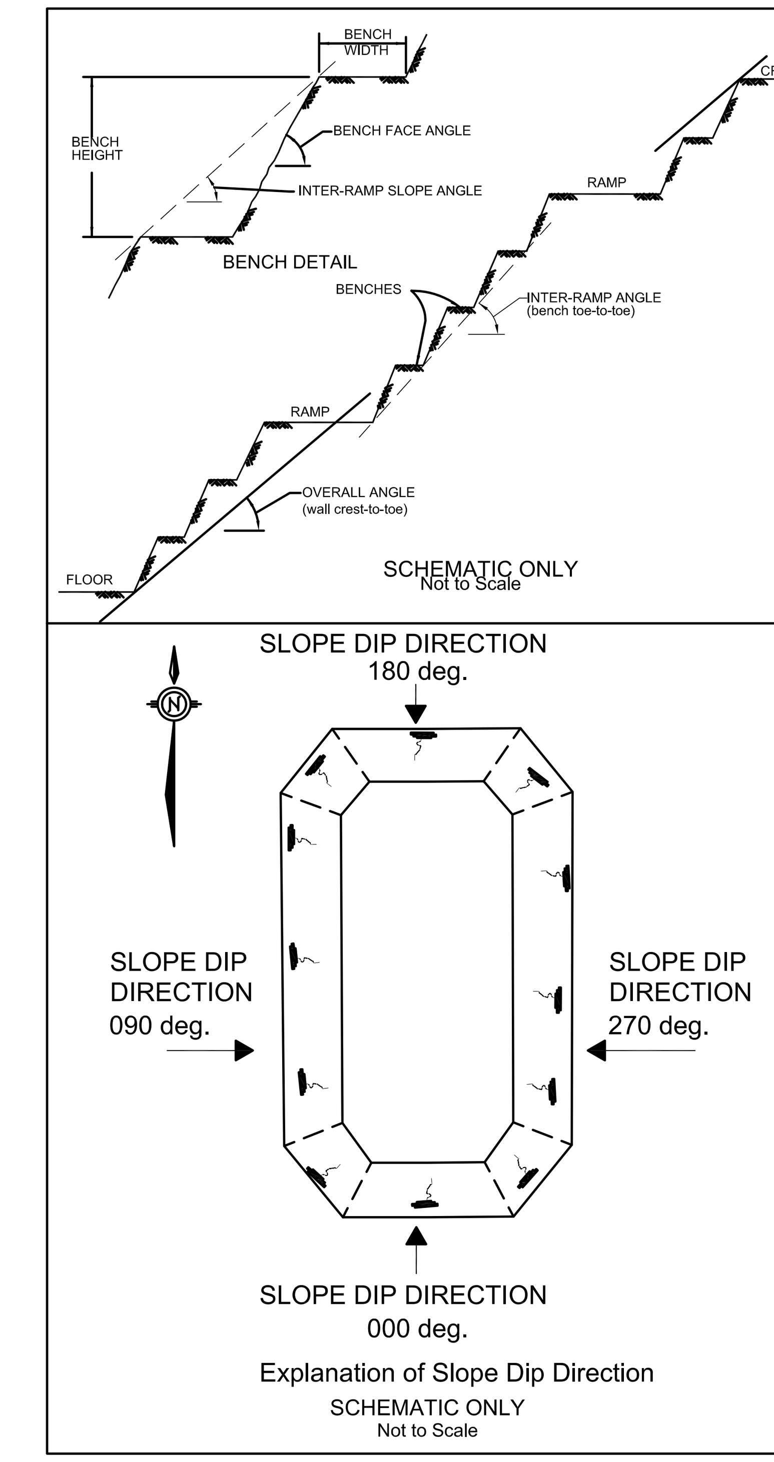
A series of stereo diagrams have been constructed showing the discontinuity sets which have been assumed for the structure associated with the Andrew Lake pit walls. Potential wedges, planar and toppling failure sets have been identified on Figures E1.1 through E1.8 for Andrew Lake (Appendix E).



DESIGN SECTORS FOR ANDREW LAKE PIT

Design Sector	Azimuth (°)	Dip Direction (°)	Kinematic Sectors	Rock Mass Sectors	Design Detail
AL.I	010 to 070	190 to 250	K.I K.II	RM. I (Type 1)	AL.Ia (Detail A) AL.Ib (Detail A)
AL.II	070 to 175	250 to 355	K.II K.III K.IV	RM.II (Type 2) RM.III (Type 1) RM.IV (Type 2)	AL.IIa (Detail B) AL.IIb (Detail A)
AL.III	175 to 280	355 to 100	K.V K.VI K.VII	RM.V (Type 1) RM.VI (Type 2) RM.VII (Type 1)	AL.IIIa (Detail A) AL.IIIb (Detail A)
AL.IV	280 to 010	100 to 190	K.VII K.VIII	RM.VIII (Type 1) RM.IX (Type 2) RM.X (Type 1)	AL.IVa (Detail B) AL.Ivb (Detail A)

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



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

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:

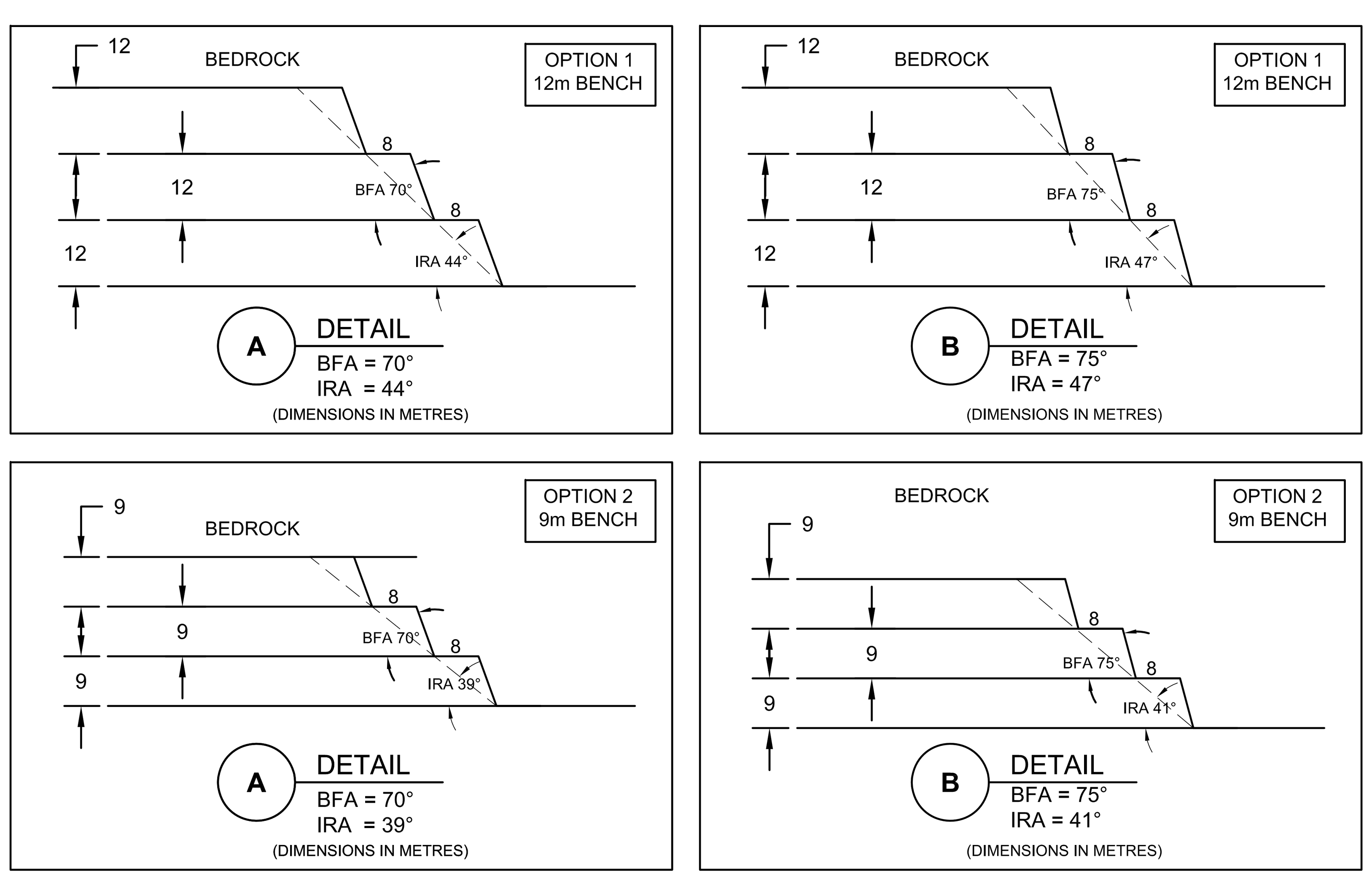
1. BASED ON REGIONAL GEOLOGY DRAWINGS (MAIN TEXT FIGURES 4 & 5)
2. AREVA GEOLOGICAL MODEL (2009).

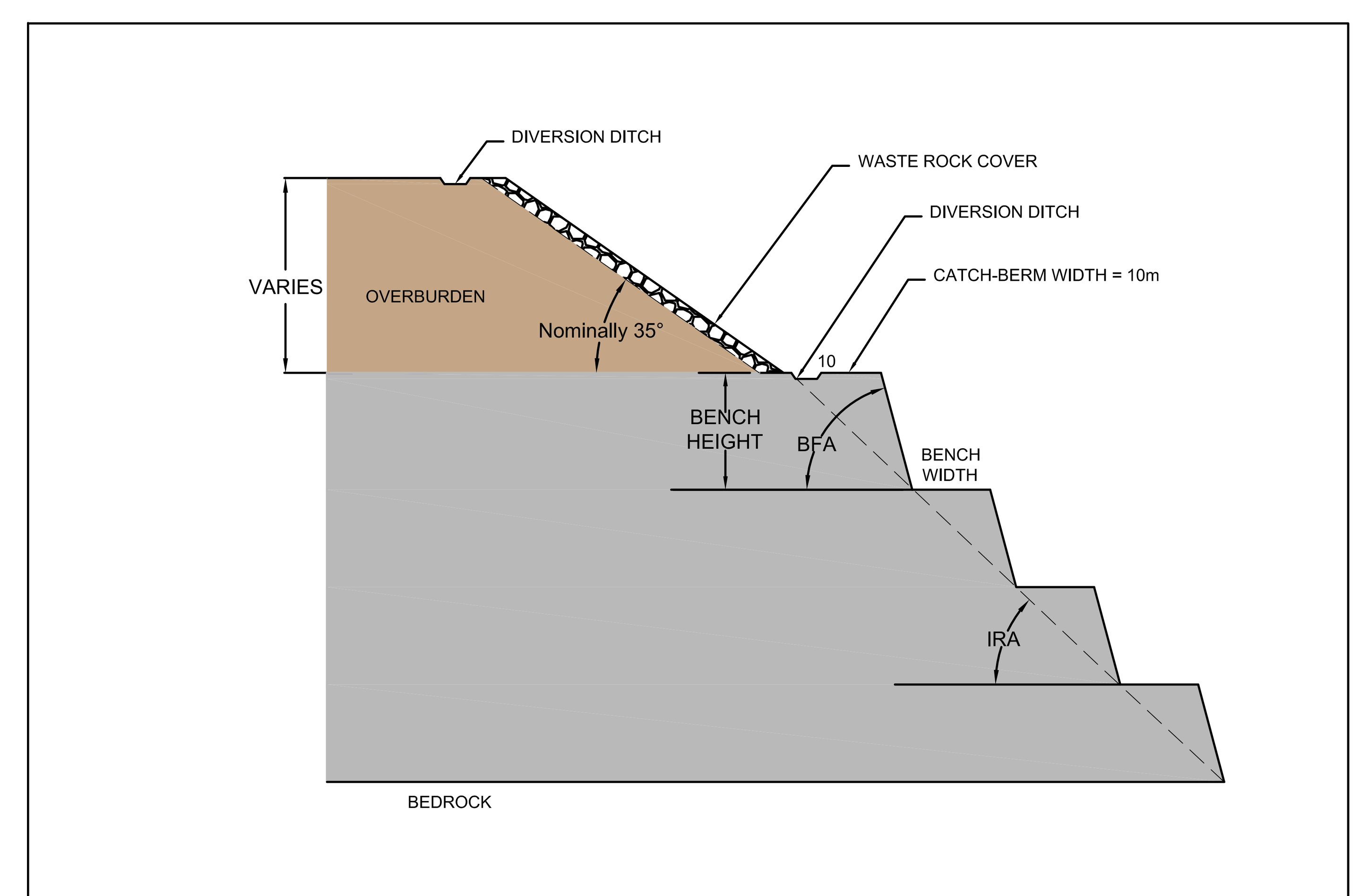
NOTES:

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.

SCALE 1:3000 — METRES (60 0 60 120)

DESIGN SECTOR DETAILS





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

PROJECT: AREVA — AREVA RESOURCES CANADA INC. KIGGAVIK PROJECT

TITLE: **ANDREW LAKE PROPOSED PIT SLOPE DESIGN PARAMETERS**

PROJECT No.		09-1362-0613	FILE No.	0913620613AB0E6.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		

Golder Associates
Saskatoon, Saskatchewan, Canada

PLOT DATE: February 23, 2011
FILENAME: G:\2009\1362\09-1362-0613 Areva Kiggavik Golder Internal Field Manual\CAD\Scissons Site\0913620613AB0E6.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 Andrew Lake site. These analyses assumed that 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. 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 implementation of 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
Andrew Lake	270	250	145	175

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 for depressurization at this depth below ground surface.

5.0 ROCK SLOPE DESIGN RECOMMENDATION

The rock slope design configurations for the Andrew Lake pit are plotted on Figure 9. The pit slope design recommendations are summarised on Table 6. 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. At Andrew Lake, the pit walls have been divided into sub-sectors representing the upper and lower walls. Discussions pertaining to the main design considerations for each sector are also given.

Following issue of the initial draft of Andrew Lake pit slope recommendations in early December 2009, AREVA asked for slope design recommendations assuming a vertical bench height of 9 m at Andrew Lake, in addition to the height of 12 m that was assumed for the initial analysis. Golder has not carried out any detailed review with purpose to optimize this different bench configuration. Table 6 below presents 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 is not considered, as a conservative measure, given the interpreted poorer rock mass quality at Andrew Lake.

Note that bench face angles assume dry slope conditions. Stability analyses indicate that bench faces at 70° at Andrew Lake in the altered (weak rock mass) may be unstable at bench heights of 9 m or 12 m, if the slopes are saturated. Should instability occur, it is interpreted that it will result in sloughing of the faces. This condition may occur on upper slopes near the active zone, or potentially more critically, near the base of the Andrew Lake pit walls, if slope and pit floor depressurization measures are not achieved and the lower slopes can be saturated due to artesian pressures below the pit floor permafrost aquitard.

During operations, if this condition is anticipated to occur based on monitoring, a modified bench configuration should be considered.

Table 6: Andrew Lake – Pit Slope Design Recommendations

Design Sector	Azimuth (°)	Dip Direction (°)	Kinematic Sectors	Rock Mass Sectors	Design Considerations	Sub-sector	BFA (°)	IRA (°)	Bench Height (m)	Berm Width (m)
AL.I	010 to 070	190 to 250	K.I	RM. I (Type 1)	Inferred mainly fully altered slope (Type 1). Transected by NE striking Beaudomont faults and E-W striking faults. Also shows strong kinematic controls on set JN1 (45 deg).	AL.Ia (upper wall)	70	44	12	8
			K.II				70	39	9	8
						AL.Ib (lower wall)	70	44	12	8
							70	39	9	8
AL.II	070 to 175	250 to 355	K.II	RM.II (Type 2)	Inferred mainly lower slope alteration at closer proximity to faulting and mineralization (Type 2). Two possible E-W striking faults may intersect upper wall in some locations. Also shows kinematic controls on set CJN1A (51 deg).	AL.IIa (upper wall)	75	47	12	8
			K.III	RM.III (Type 1)			75	41	9	8
			K.IV	RM.IV (Type 2)		AL.IIb (lower wall)	70	44	12	8
							70	39	9	8
AL.III	175 to 280	355 to 100	K.V	RM.V (Type 1)	Inferred mainly fully altered slope (Type 1). Transected by NE striking Beaudomont faults and E-W striking faults. Potential kinematic controls on BFA by sets JN2A (69 deg.) and JN3 (63 deg). Possible scatter on FO1A (up to 45 deg.) for limitation on IRA.	AL.IIIa (upper wall)	70	44	12	8
			K.VI	RM.VI (Type 2)			70	39	9	8
			K.VII	RM.VII (Type 1)		AL.IIIb (lower wall)	70	44	12	8
							70	39	9	8
AL.IV	280 to 010	100 to 190	K.VII	RM.VIII (Type 1)	Inferred mainly lower slope alteration at closer proximity to faulting and mineralization (Type 2). Two possible E-W striking faults may intersect upper wall in some locations. Possible kinematic controls on sets FO1A dipping east (up to 45 deg.) and set JN1A dipping south (46 deg.)	AL.IIIa (upper wall)	75	47	12	8
			K.VIII	RM.IX (Type 2)			75	41	9	8
				RM.X (Type 1)		AL.IIIb (lower wall)	70	44	12	8
							70	39	9	8

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 the Andrew Lake pit location 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 actual conditions may vary across the site from those indicated in this report. 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 re-vegetation 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 and drainage 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. Preventing run-off onto the weak materials that comprise the Andrew Lake pit slopes in rock, which are considered prone to erosion is considered an important aspect of slope design. 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.

7.0 OPERATIONAL CONSIDERATIONS

This section addresses some of the operational considerations during mining of the Andrew Lake open pit.

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 benches are assumed to remain dry. Therefore, the importance of achieving depressurised conditions cannot be over-emphasized.

For the slopes below the permafrost level at Andrew Lake, 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 pit is 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 pit is 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 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 re-drilling 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 solution for depressurization 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 pit. 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 Andrew Lake 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 Andrew Lake 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 excavation is 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 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 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

When planning future field investigations for the open pit deposit at Andrew Lake, it is recommended that the boreholes proposed by SRK in 2008 and by Golder in 2009 be completed to fulfill the data requirement for feasibility level pit designs. As additional data becomes available, a data gap analysis can be conducted, and the need for further boreholes will be identified.

Additional work should also include the development of a detailed geological, structural and alteration model for the open pit deposit. This is especially important at Andrew Lake, where highly altered rock conditions may impact the overall pit slope stability. Further understanding of the alteration halo at Andrew Lake may allow for the pit slope design to be better optimized.

The drilling and interpretive work on wall alteration should be done in an integrated manner. The upside potential of this work is that more of the Andrew Lake wall rock may be relatively unaltered, where steeper slopes on the order of 47 degrees could be achieved. Consequently, a greater percentage of the pit walls could have the 47 degree inter-ramp bench geometry.

9.0 CLOSURE

This report presents the rationales and slope design recommendations for Andrew Lake open pit. 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.

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REFERENCES

AREVA Resources Canada Inc. (AREVA). 2007. Kiggavik Project Prefeasibility Report – Section 5 Geology and Resources. Revision 1. Pages 5-1 to 5-4. November 2007.

Golder Associates Ltd. (Golder). 1989. Report to Urangesellschaft Canada Limited on the Mining Geotechnical Aspects of the Proposed Kiggavik Uranium Operations. Volume I of II and Volume II of II. Project #882-1421/881-1814G. August 1989.

Golder Associates Ltd. (Golder). 2007a. Report on Preliminary Assessment of Expected Groundwater Conditions Kiggavik-Sissons Project, Nunavut. Project #06-1321-065/5000. March 2, 2007.

Golder Associates Ltd. (Golder). 2007b. Draft Report on Complementary 2006-2007 Geotechnical Work – Kiggavik-Sissons Project, Nunavut. Project #07-1321-065. June 2007.

Golder Associates Ltd. (Golder). 2009a. Draft Technical Memorandum to Frederic Guerin. Borehole Plan for 2009 Kiggavik Geotechnical and Hydrogeological Investigations for Andrew Lake and Kiggavik Pits. Project #09-1362-0613. May 8, 2009.

Golder Associates Ltd. (Golder). 2009b. “Table 0 - Open Pit Summary Program, June 12th, 2009 Revisions (Deferred holes not shown)” Excel Spreadsheet for AREVA. Project #09-1362-0613. June 12, 2009.

International Society for Rock Mechanics (ISRM). 1981. Suggested Methods for Rock Characterization Testing and Monitoring, (ed. E.T. Brown). Commission on Testing Methods, ISRM. Pergamon Press.

Nunavut Worker’s Compensation Board. 2009. Consolidation of Mine Health and Safety Regulations. R-125-95. URL: http://www.wcb.nt.ca/your_wcb/pdf/legislation/NUMineRegulations.pdf.

SRK Consulting (Canada) Inc. 2008a. Technical Memorandum to Frederic Guerin and Cliff Revering of AREVA Resources Canada Inc. 2008 Geotechnical/ Hydrogeological Borehole Planning and Preliminary Pit Design Review. Project #1CA015.003. July 2, 2008.

SRK Consulting (Canada) Inc. 2008b. Draft Memorandum to Frederic Guerin of AREVA Resources Canada Inc. 2008 Kiggavik-Sissons Geotechnical Data Report (Draft). Project #1CA015.003. November 23, 2008.

SRK Consulting (Canada) Inc. 2008c. Technical Memorandum to Frederic Guerin of AREVA Resources Canada Inc. Andrew Lake Geological Model Update. Project #4CA005.001. November 27, 2008.

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

APPENDIX A

Rock Strength

Table of Contents

1.0 INTRODUCTION

This appendix presents the results of the materials testing carried out for the sampled rock material specimens obtained during the 2009 geotechnical drilling program for the Andrew Lake (AL) geotechnical boreholes. Strength testing included Point Load Testing (PLT) and Unconfined/Uniaxial Compressive Strength (UCS) testing. A total of 199 valid PLT and 19 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 Andrew Lake site. Unless otherwise noted, rock type refers to information provided by AREVA Resources Canada Inc. (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 19 tests were carried out on the AL borehole samples. A laboratory testing report by 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 AL data shows 7 preferential failures.

Further discussion and interpretation of these results are provided in the following sections.

Table A1: Andrew Lake – Summary of laboratory UCS test results

Hole	Depth (m)	Lithology	UCS (MPa)	Youngs Modulus (GPa)	Density (g/cm^3)	Poissons Ratio	Type of Failure
AND09-01	78	GnPsaPel	43.9	15.2	2.51	0.03	Intact rock
	84	GnPsaPel	41.8	23.1	2.57	0.08	Intact rock
	120	GranT	24.7	6.1	2.45	0.24	Intact rock
	219	GranT	80.9	24.1	2.60	0.13	Intact rock
	231	GranT	52.0	20.6	2.46	0.16	Intact rock
	255	GnPsaPel/Flt*	4.9	0.4	1.97	0.06	Intact rock
	336	GnPsaPel/Flt*	9.8	2.4	2.09	0.04	Weakness Plane
AND09-02	60	GnPsaPel	17.5	2.2	2.19	0.1	Intact rock
	99	GnPsaPel	14.2	1.8	2.48	0.13	Intact rock
	105	GnPsaPel	16.1	4.5	2.51	0.03	Foliation
	111	GnPsaPel	22.4	5.7	2.04	0.16	Weakness Plane
	129	GnPsaPel	26.7	8.6	2.40	0.49	Intact rock
	135.37	GnPsaPel	55.6	21	2.53	0.07	Intact rock
	150	GnGran	26.6	7.3	2.33	0.08	Intact rock
	168	GnPsa	11.8	5.1	2.49	0.04	Microfracture(s)
	174.64	GnPsa	19.3	6.4	2.53	0.75	Microfracture(s)
AND09-03	45	GnPsaPel	24.3	12.9	2.61	0.12	Weakness Plane
	108	GnPsaPel	26.7	7.3	2.59	0.04	Microfracture(s)
	201	BxQtz	35.0	19.9	2.63	0.16	Intact rock

*Lithology inferred from Golder's geotechnical logs.

2.2 Field PLT and Rock Strength Index Testing

During the 2009 geotechnical drilling program, point load testing was performed in the field at selected depth intervals. There were 199 PLT tests carried out for the AL geotechnical boreholes.

The PLT procedures are discussed in Golder's data report from the 2009 field season (Golder 2009). This 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 logging interval. 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 AL 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: Andrew Lake – summary of average $I_{s(50)}$ (MPa) and R (index) values by borehole

AND09-01				AND09-02				AND09-03			
Depth	Lithology	$I_{s(50)}$ (MPa)	R	Depth	Lithology	$I_{s(50)}$ (MPa)	R	Depth	Lithology	$I_{s(50)}$ (MPa)	R
20	GnPsaPel	0.2	1	39	GnPsaPel	0.9	2	18	Episyen	3.9	4
21.25	GnPsaPel	1.0	1	42	GnPsaPel	0.9	2	36	GnPsaPel	3.7	3
63	GnPsaPel	2.5	2.5	51	GnPsaPel	1.8	2	57	GnPsaPel	2.8	3
66	GnPsaPel	0.4	2.5	57.15	GnPsaPel	1.3	3	72	Episyen	1.9	3
93	GnPsaPel	0.6	2	69	GnPsaPel	1.8	3	108	GnPsaPel	0.8	3
99	GnPsaPel	1.5	2.5	87	GnPsaPel	1.0	3	132	GnPsaPel	4.7	3
102	GnPsaPel	2.9	2.5	97.1	GnPsaPel	0.2	2	135	GnPsaPel	0.5	2
126	GranT	3.5	3	102	GnPsaPel	0.3	2	138	GnPsaPel	0.7	2
126.9	GranT	2.7	3	108	GnPsaPel	0.5	2	141.86	Episyen	4.0	3.5
127.65	GranT	1.8	3	111	GnPsaPel	1.3	2	144	GnPsaPel	1.8	2
132	GranT	0.2	1	120	GnPsaPel	0.7	3	148.36	GnPsaPel	0.2	2
180	GranT	5.9	2.5	126	GnPsaPel	1.1	2	151.06	GnPsaPel	3.3	2
183	GranT	5.7	2.5	132	GnPsaPel	1.4	2	153	GnPsaPel	1.0	2
222	GranT	1.6	3.5	144	GnPsaPel	2.8	3	156	GnAlt	2.9	2
225	GranT	4.1	3.5	148.55	GnPsaPel	1.2	2.5	159	GnAlt	6.7	2
266.22	GnPsaPel/Flt*	1.0	3	150	GnGran	25.5	2.5	162	GnAlt	5.3	2
268.29	GnPsaPel/Flt*	1.2	3	153	GnGran	0.4	2.5	165	GnPsaPel	0.7	2
306	GnPsaPel/Flt*	0.2	2	156	GnGran	1.1	3	168	GnPsaPel	2.4	2
309	GnPsaPel/Flt*	0.2	2	159	GnGran	7.4	3	171	GnPsaPel	2.9	3
				162	GnGran	3.9	3	177	GnPsaPel	0.4	3
				165	GnGran	3.5	3	180	GnPsaPel	2.1	3
				171	GnPsa	1.4	2.5	189	GnPsaPel	2.4	3
				174.64	GnPsa	1.0	2.5	192	GnPsaPel	2.6	3
				180	GnPsa	2.0	3	207	GnPsaPel	0.2	4
				183	GnPsa	8.0	3	210	GnPsaPel	8.3	4
				186	GnPsa	2.0	3	213	GnPsaPel	4.8	4
				189	GnPsa	1.7	3	216	GnQtzFd	6.8	4
				192	GnPsa	2.0	3	219	GnPsaPel	0.5	4
				198	GnPsa	1.6	3	222	GnPsaPel	5.5	4
				201	GnPsa	4.2	3	225	GnPsaPel	6.9	4
				204	GnPsa	0.7	3	228	GnPsaPel	8.3	4

Table A3: Andrew Lake – summary of average $I_{s(50)}$ (MPa) and R (index) values by borehole (continued)

AND09-01				AND09-02				AND09-03			
Depth	Lithology	$I_{s(50)}$ (MPa)	R	Depth	Lithology	$I_{s(50)}$ (MPa)	R	Depth	Lithology	$I_{s(50)}$ (MPa)	R
				207	GnPsa	0.5	3	231	GnPsaPel	8.1	4
				210	GnPsa	1.5	3	234	GnPsaPel	2.2	4
				213	GnPsa	1.7	3	237	GnPsaPel	7.0	4
				237	GnPsa	9.0	3	240	GnPsaPel	4.8	4
				264	GnPsaPel	12.1	5	243	GnPsaPel	8.3	4
				297	GnGran	9.9	5	246	GnPsaPel	0.9	4
				318	GnGran	11.1	5	252	Flt	7.0	4
								255	GranT	9.8	4

*Lithology assessed from Golder's field logs, otherwise rock type based on information provided by AREVA; m = metre, MPa = mega pascal

3.0 STRENGTH TESTING RESULTS

3.1 Andrew Lake

3.1.1 Alteration Factor

From the assessment of the AL strength testing data, it was apparent that the intensity of mineral alteration was one of the main qualitative controls on the intact strength of the metasedimentary and granitic rock masses. The argillization and chloritization indices (denoted A_{arg} and A_{chl} respectively) for the 2009 AL geotechnical holes were obtained from AREVA. The alteration indices used by AREVA are on a scale of 0 to 4, where 0 represents nil alteration, and 4 represents intensely altered rock (greater than 80% alteration). Since it was recognized that both argillization and chloritization were related to a potential degradation in rock strength, an alteration factor (A_f) was assessed for the 2009 geotechnical holes as follows:

If A_{arg} or A_{chl} = 0, then $A_f = A_{arg} + A_{chl}$

If A_{arg} and $A_{chl} \neq 0$, then $A_f = A_{arg} \times A_{chl}$

The A_f value could range between 0 and 16, where 0 represents nil alteration, and 16 represents intensely altered rock both through chloritization and argillization. It is noted that $A_f = 9$ in borehole AND-09-01 was the maximum alteration factor assessed in the 2009 geotechnical holes, and the least overall alteration apparently occurs in borehole AND-09-03.

A comparison of A_f versus downhole depth for the metasedimentary and granitic rock units in the 2009 AL boreholes are shown on Figure A1. In general, the maximum alteration apparently occurs between 100 m and 200 m depth. The AL geotechnical boreholes were drilled at an inclination of between 65˚ and 70˚; therefore a correction would be required to relate the downhole depths to vertical depth below ground surface.

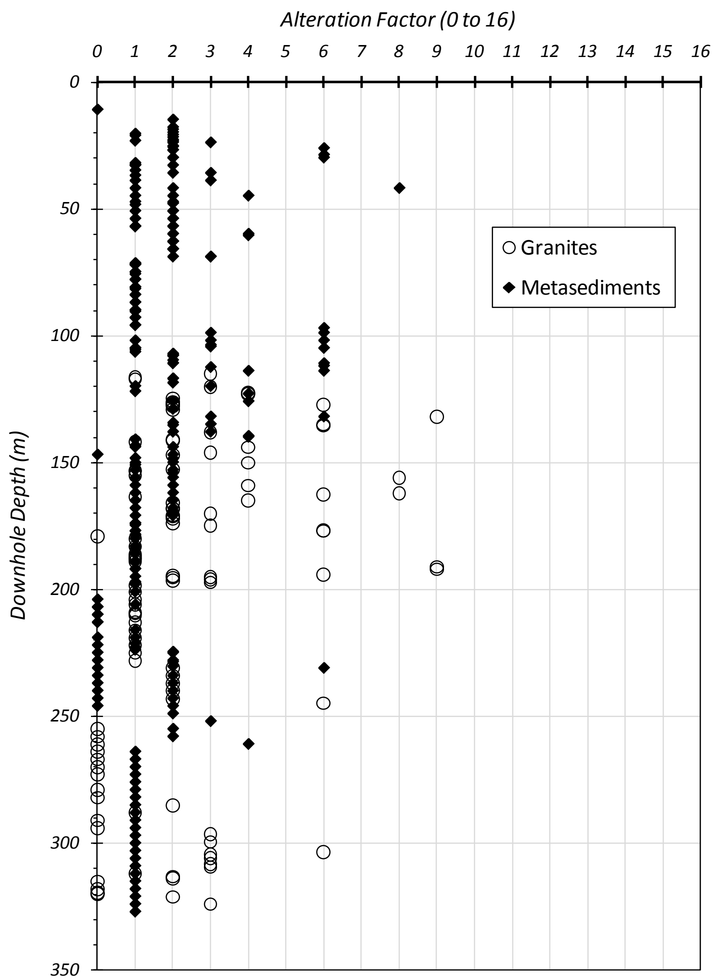

Figure A1: Alteration factor for granites and metasediments for all 2009 Andrew Lake boreholes.

3.1.2 Effect of Alteration on Rock Strength

Reviewing the Andrew Lake UCS and PLT, there is shown to be considerable variability in strength for each rock type. Applying the alteration factor (Af) to the strength data shows more consistent trends relating increased rock alteration with decreased rock strength.

The UCS versus alteration factors are plotted in Figure A2 and show a trend of decreasing strength with increasing alteration, albeit there is still considerable variability in strength data. Based on the general trend, it can inferred that moderate strength (greater than 25 MPa) only occurs in rocks that are not intensely altered.

It is appreciated that mineral alteration, being the replacement of hard and dense minerals, with soft and low density minerals, results in a decrease in rock material density. The trend of increasing UCS with increasing rock density is plotted on Figure A3. These results indicate that mineral alteration and density provide useful correlations to rock strength. Generally, rocks with a density of less than approximately 2.4 g/cm^3 are very weak to weak rock (less than 25 MPa).

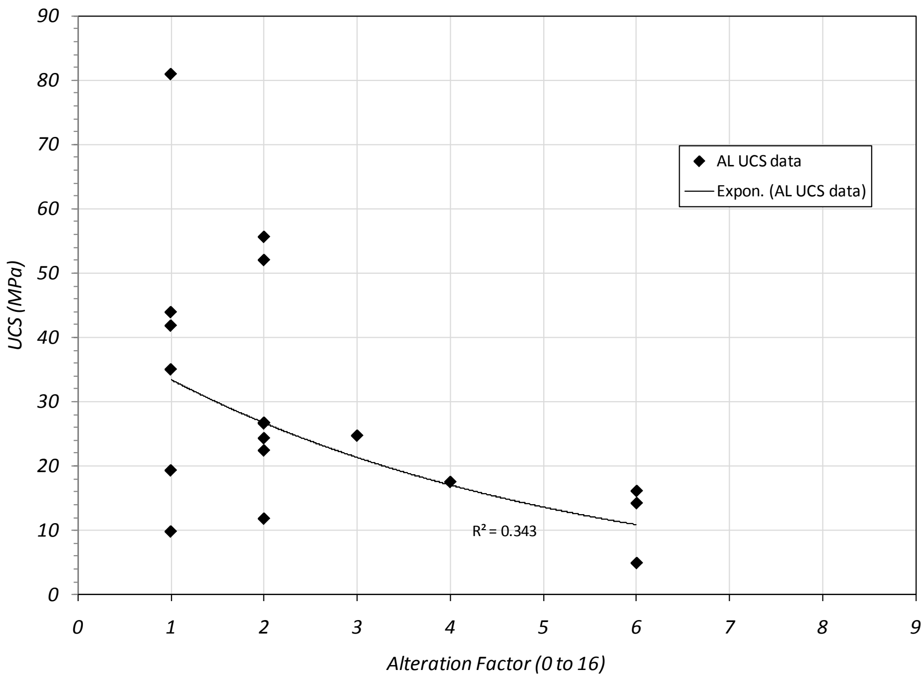

Figure A2: UCS Versus alteration for the 2009 Andrew Lake borehole data.

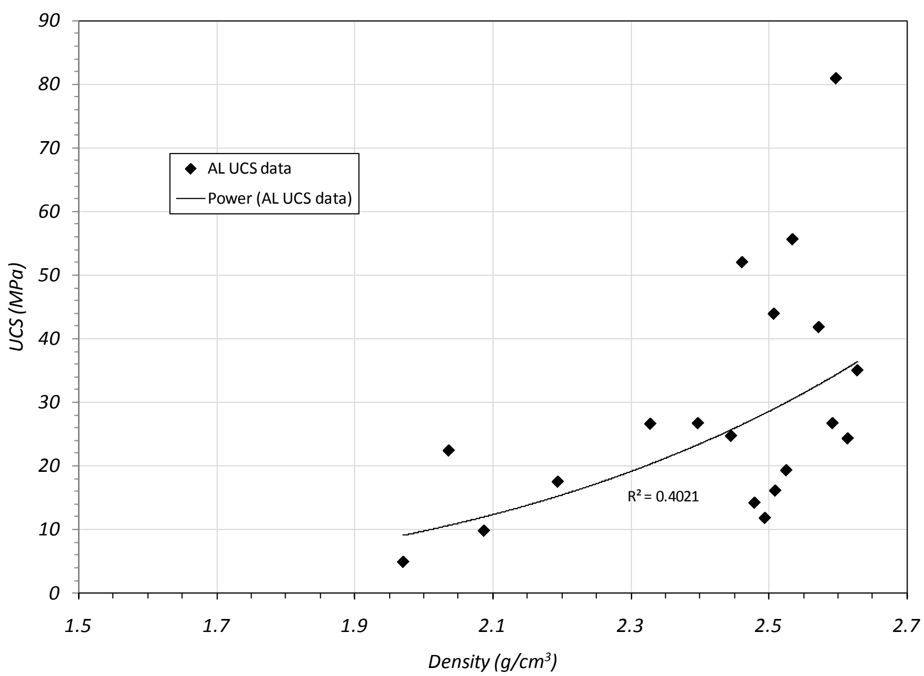

Figure A3: UCS versus density for the 2009 Andrew Lake borehole data.

Previous work by SRK (SRK, 2009) conducted 16 UCS tests within the Andrew Lake rock units. Furthermore, as part of routine core logging, SRK identified the rock units by an alteration/microfracture index on a scale of 0 to 3, where 0 = none and 3 = heavy. Although it is difficult to correlate SRK's alteration index to the alteration factor (A_f), plotting this data qualitatively as shown in Figure A4 does show the effect of alteration on rock strength as we see similar trends of decreasing strength with increased alteration.

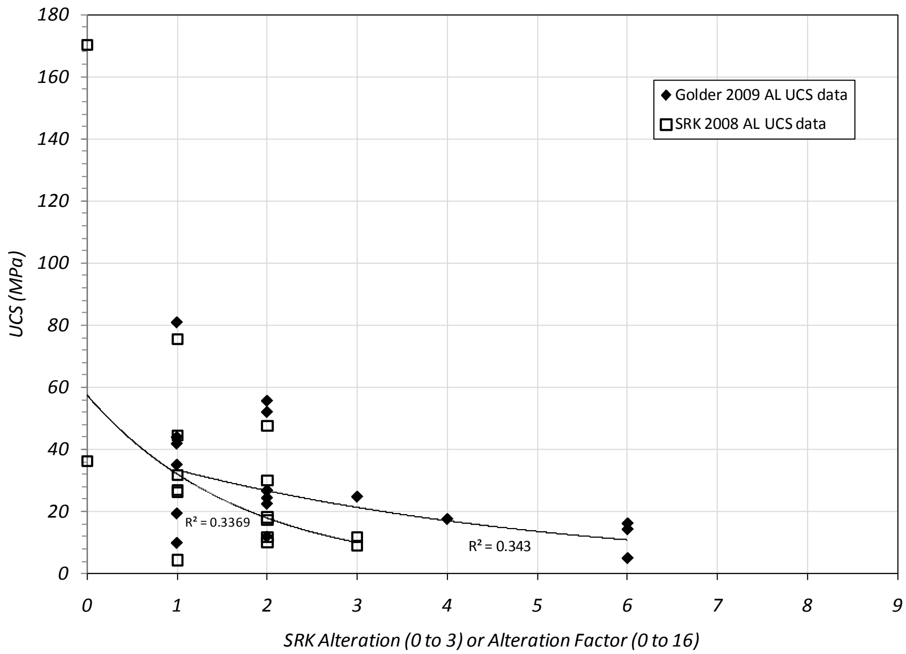

Figure A4: UCS versus alteration for the Golder (2009) and SRK (2008) Andrew Lake borehole data.

The PLT data shows considerable variability in strength results which can be related to both the variability of intact rock strength, as well as testing procedures. Similar to the UCS data, the PLT $I_{s(50)}$ values have been correlated to the alteration factor, and this correlation is plotted in Figure A5. The trend of decreasing strength with increased alteration is less apparent than the UCS data with considerably more variability in values; however the trends are still apparent. The $I_{s(50)}$ data is also plotted against depth in Figure A6. There is shown to be a general increase in strength with increasing depth, particularly below 200 m.

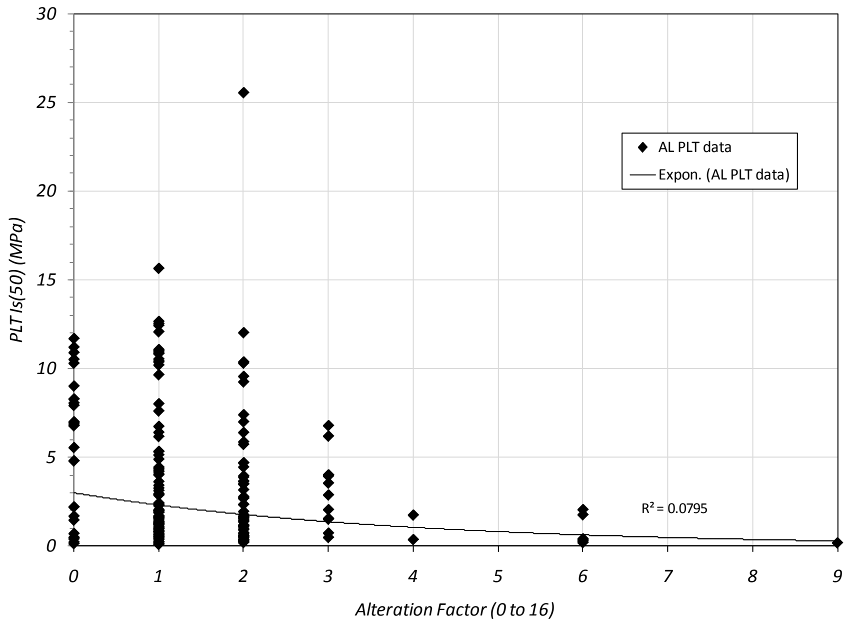

Figure A5: PLT $I_{s(50)}$ versus alteration factor for the 2009 Andrew Lake borehole data.

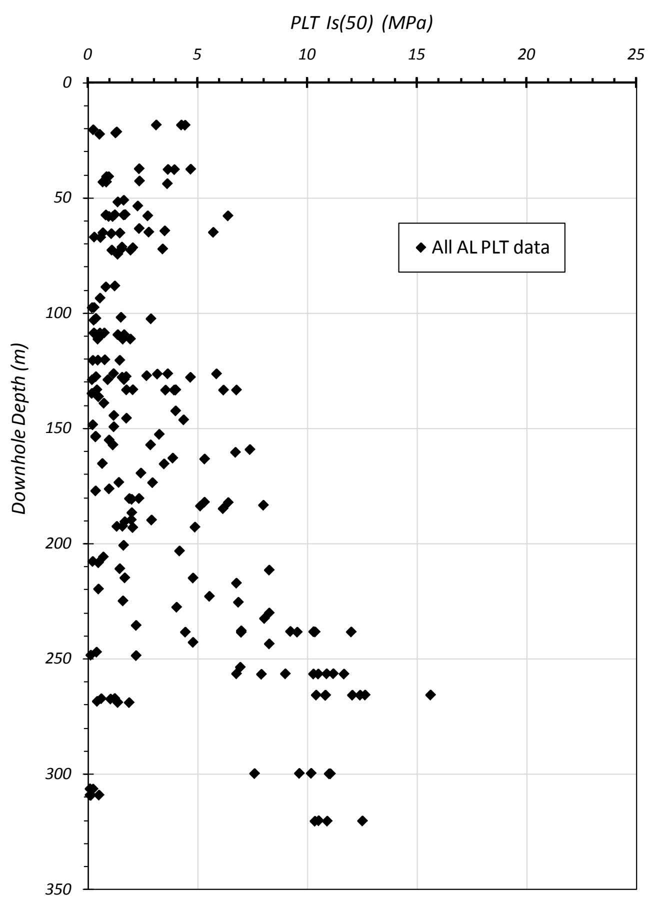

Figure A6: PLT $I_{s(50)}$ versus depth for the 2009 Andrew Lake borehole data.

3.1.3 Summary of Andrew Lake Strength Data

The AL UCS and elastic parameter data has been summarised by rock type and further sub-divided by alteration where sufficient data was available. Table A3 presents the calculated average and standard deviation of this data. Overall, the statistical distribution of this data appears reasonable.

The metasediment rock types tested are shown to be weak to moderately strong, with an average UCS varying between 15.9 and 29.9 MPa for moderate to high, and moderate to low intensities of alteration, respectively. The increased alteration also corresponds with decreased material density. It is noted that the tested metasediment data was taken above 200 m depth, and based on the PLT data, there is likely a corresponding increase of strength with depth below 200 m.

The low alteration granites are shown to be strong, with an average UCS of 66.5 MPa. Only one altered granite sample was available for testing (UCS = 24.7 MPa), but it is likely that further samples of altered granite would yield within the weak to moderately strong range.

Two samples taken from within the inferred fault zone in borehole AND09-01 at considerable depths (255 m and 336 m), show very weak material with very low material densities, with an average UCS of 7.4 MPa and density of only 2.03 g/cm^3. Furthermore, argillitic and chloritic alteration of the inferred fault rock is not appreciably higher than the other rock units. These test results illustrate the contrast of strengths between a potential fault influenced zone and as compared to the non-fault influenced rock units.

Table A4: Andrew Lake - summary of UCS and elastic parameters by rock type and alteration.

Rock Type	Alteration	# Tests	UCS (MPa)	Youngs Modulus (GPa)	Density (g/cm^3)	Poissons Ratio
Metasediments <200 m (GnGran,GnPsa,GnPsaPel)	Low to Mod (Af = 1 to 2)	8	29.9 +/- 13.2	11.3 +/- 6.5	2.46 +/- 0.17	0.08 +/- 0.04
	Mod to High (Af = 3 to 6)	3	15.9 +/- 1.7	2.8 +/- 1.5	2.39 +/- 0.17	0.09 +/- 0.05
Granite (GranT)	Low to Mod (Af = 1 to 2)	2	66.5 +/- 20.4	22.4 +/- 2.5	2.53 +/- 0.10	0.15 +/- 0.02
	Mod to High (Af = 3)	1	24.7	6.1	2.45	0.24
Fault (Flt)	Low to High (Af = 1 to 6)	2	7.4 +/- 3.5	1.4 +/- 1.4	2.03 +/- 0.08	0.05 +/- 0.01
Quartz Breccia (BxQtz)	Low (Af = 1)	1	35.0	19.9	2.63	0.16

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

The AL PLT – $I_{s(50)}$ data is summarised in Table A4. The rock types have again been subdivided by the intensity of alteration. There is shown to be significant statistical variance in the PLT data compared to the UCS data.

Sufficient data was available to subdivide the metasediments with depth, greater or less than 200 m (it is noted that this is the downhole depth). At depth less than 200 m, the high and slightly altered materials showed similar results of weak to moderate strength. At depth greater than 200 m, the slightly altered metasediments showed markedly higher results suggesting strong rock.

The non to slightly altered granitic rock types were shown to be moderately strong to strong. Insufficient data was available to make any determinations for moderate to highly altered granites.

Several tests were available in the inferred fault zone in AND-09-01. In general this rock was shown to be weak, particularly when compared to the non-altered metasediments and granites of similar depth (>250 m).

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. Based on the comparisons of the average UCS and $I_{s(50)}$ values, a K of 8 to 14 appears to be appropriate for the range of materials analysed. An average K of 12 would likely be representative for general estimates of UCS from $I_{s(50)}$. Other work by SRK (SRK, 2009) found PLT-UCS correlation factors of between 7 and 10.2 for the Andrew Lake rock units.

Table A5: Andrew Lake - summary of PLT $I_{s(50)}$ values by rock type and alteration.

Rock Type	Alteration	# Tests	$I_{s(50)}$ (MPa)
Metasediment < 200 m (GnGran,GnPsa,GnPsaPel)	Low to Mod (Af = 0 to 2)	87	2.2 +/- 3.0
	Mod to High (Af = 3 to 6)	21	2.1 +/- 2.0
Metasediment > 200 m (GnGran,GnPsa,GnPsaPel)	Low to Mod (Af = 0 to 2)	44	7.3 +/- 4.3
Granite (GranT)	Low to Mod (Af = 0 to 2)	24	5.5 +/- 3.6
	High (Af = 9)	1	0.2
Fault (Flt)	Low to Mod (Af = 0 to 2)	12	1.2 +/- 1.9
Episyenite (Episyen)	Low (Af = 1)	9	2.8 +/- 1.3

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

4.0 CONCLUSIONS AND RECOMMENDATIONS

Based on the assessment of the UCS and PLT data for the Andrew Lake site, rock strength appears to be influenced by the intensity of mineral alteration (argillization, chloritization). It is hypothesized that within close proximity to the ore zone horizons and associated faults, rock strengths would range from very weak (<10 MPa) to weak (10 MPa to 25 MPa). Within the planned pit walls, it is probable that alteration is less intense or absent, and this reduction in alteration would likely be associated with increased rock strengths of moderately strong (25 MPa to 50 MPa) to strong (50 MPa to 100 MPa). The presence of faults transecting the pit walls, particularly where more than one fault exists in close proximity to one another, may be accompanied with very weak to weak rock conditions. Furthermore, the granitic and lower metasediment (paragneiss) rock units show higher strengths compared to the upper metasediments, which is also possibly associated with proximity to the alteration zones.

The proceeding analyses are based on limited laboratory testing. Additional laboratory strength 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

Golder Associates Ltd., 2009. Draft 2009 Kiggavik Geotechnical and Hydrogeological Investigation Data Report. November, 2009. Project #09-1362-0613. November 2009.

International Society for Rock Mechanics (ISRM),1981. Suggested Methods for Rock Characterization Testing and Monitoring, (ed. E.T. Brown). Commission on Testing Methods, ISRM. Pergamon Press.

International Society for Rock Mechanics (ISRM), 1985. Suggested Method for Determining Point Load Strength Index. Int.J.Rock Mech.Min. Sci. & Geomech. Abstr.Vol.22 No.2 pp. 51-60.

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

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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

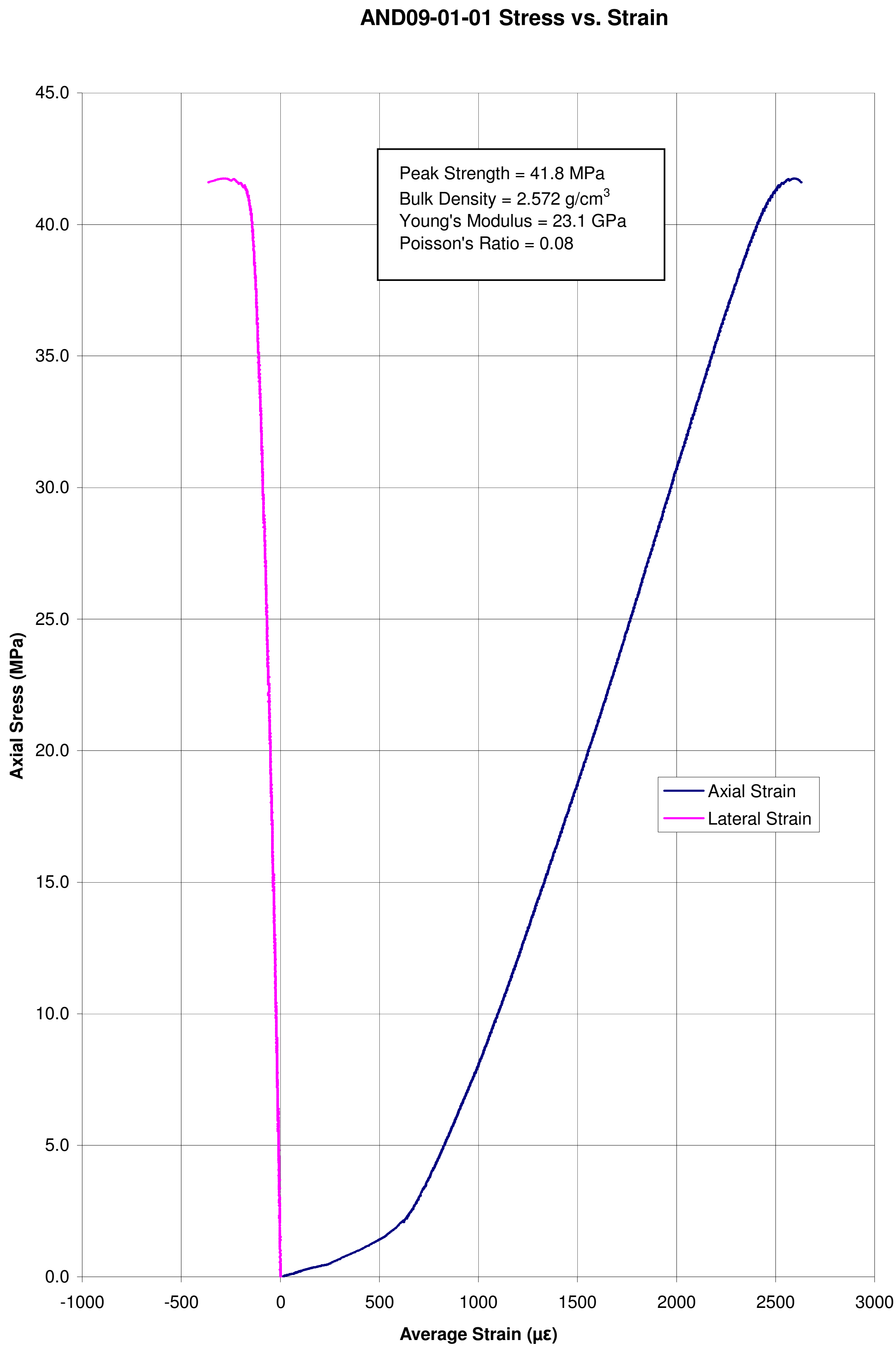
Density: 2.572 g/cm^3

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/cm^3
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

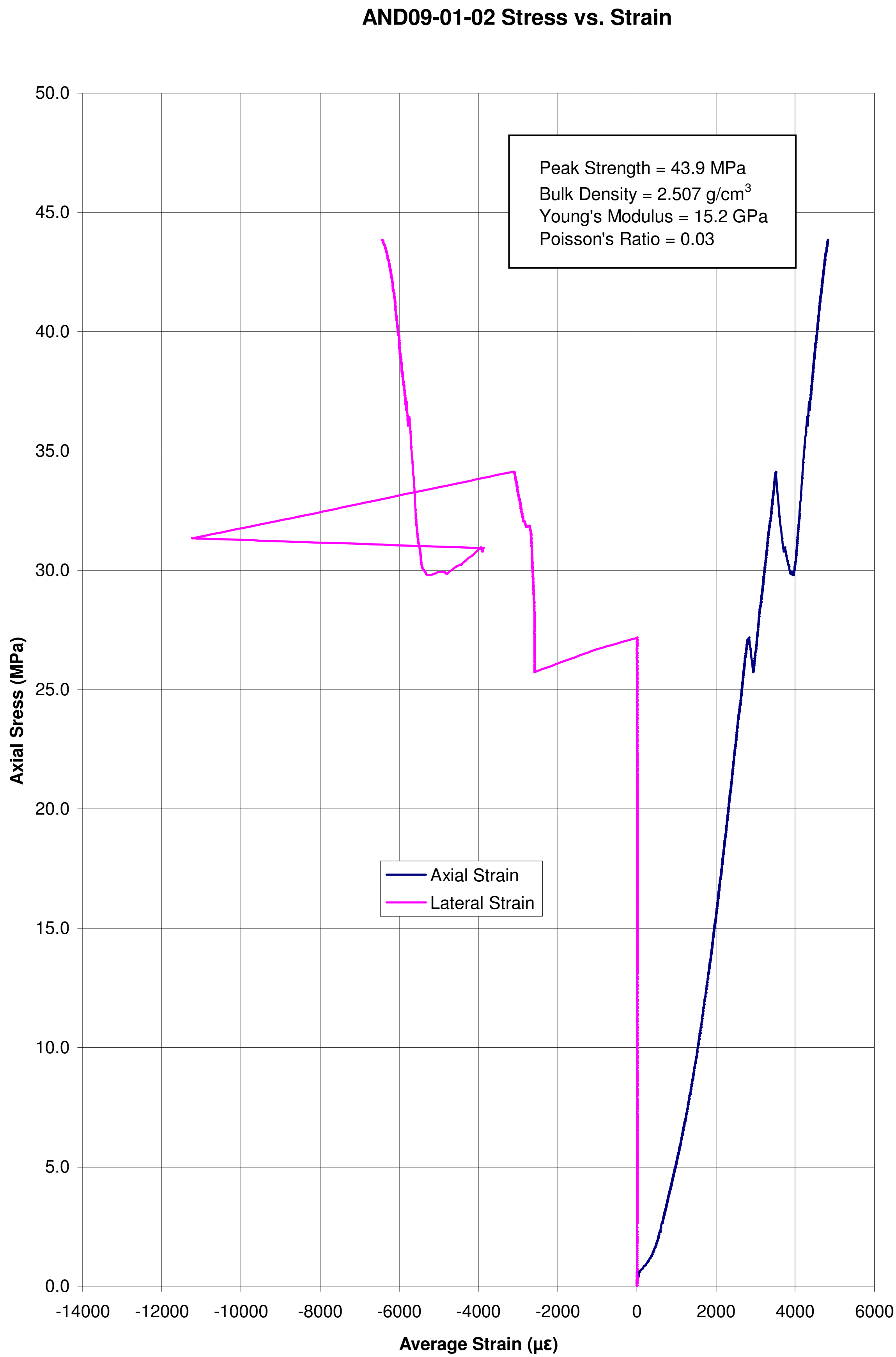
Density: 2.507 g/cm^3

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

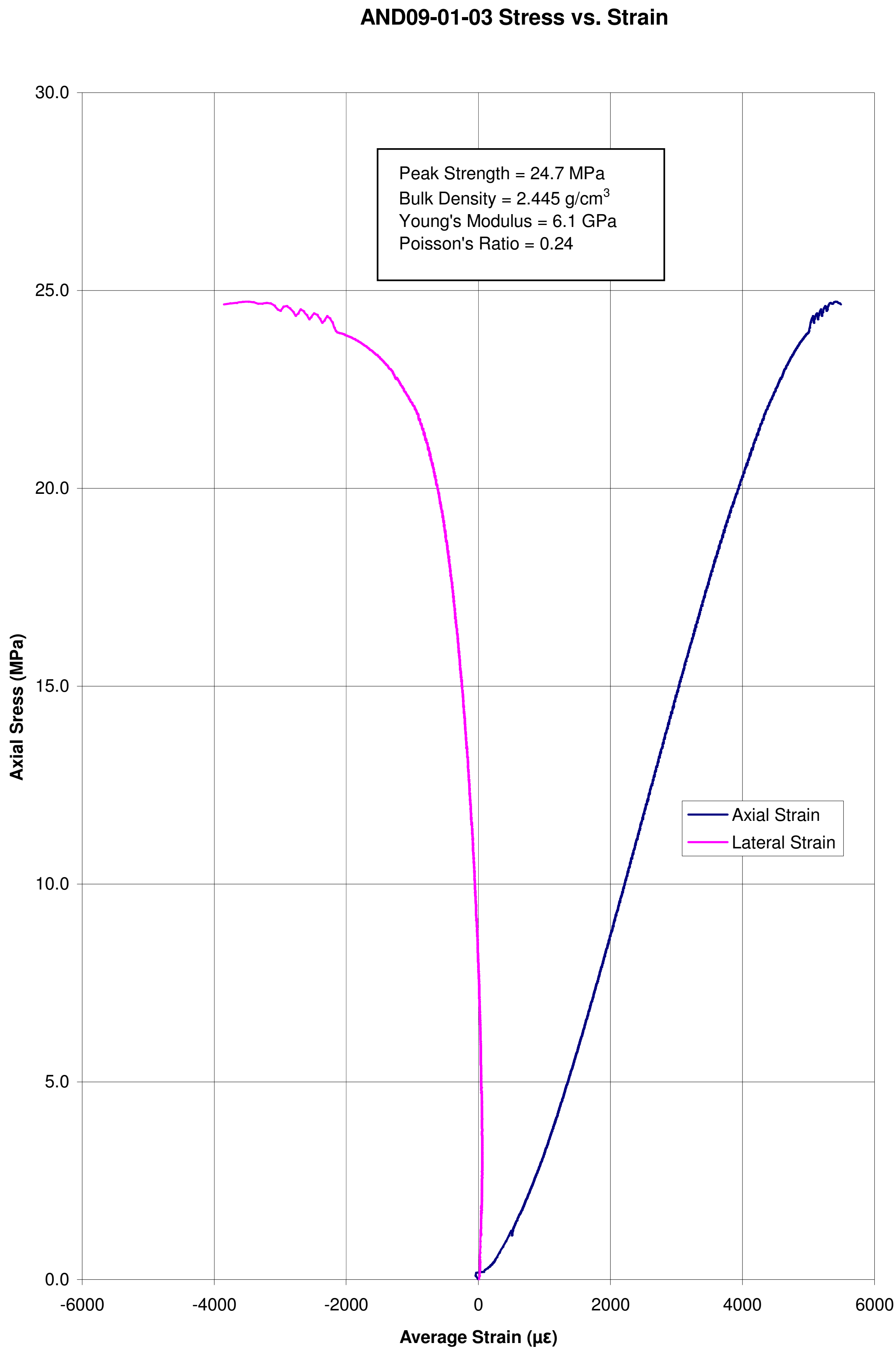
Density: 2.445 g/cm^3

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/cm^3
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

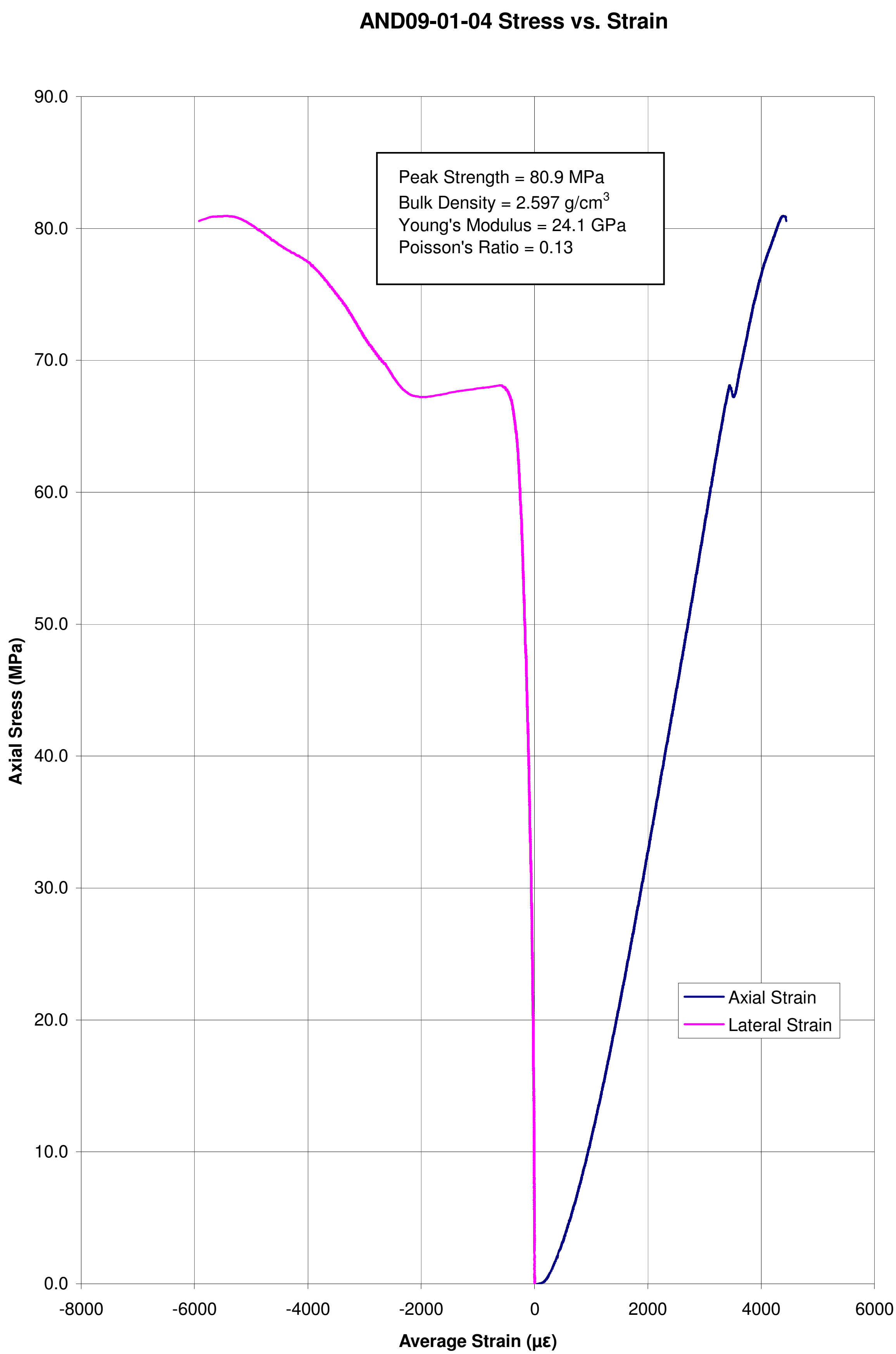
Density: 2.597 g/cm^3

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

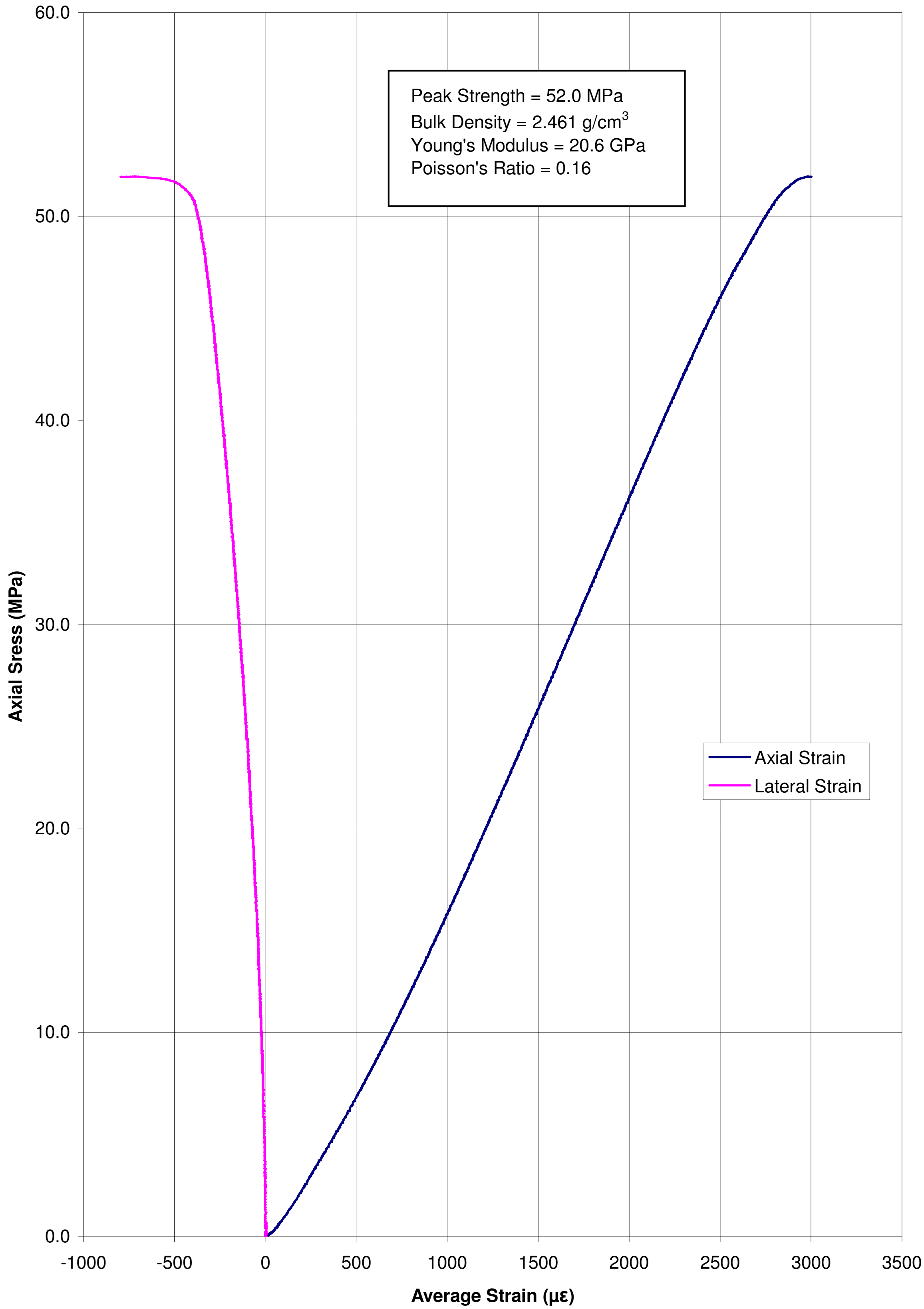
Peak Strength: 52.0 MPa

Young's Modulus: 20.6 GPa

Poisson's Ratio: 0.16

Failure Cause: Intact Rock

AND09-01-05 Stress vs. Strain



SAMPLE: AND09-01-06

Length: 112.65 mm

Diameter: 45.26 mm

Density: 1.970 g/cm^3

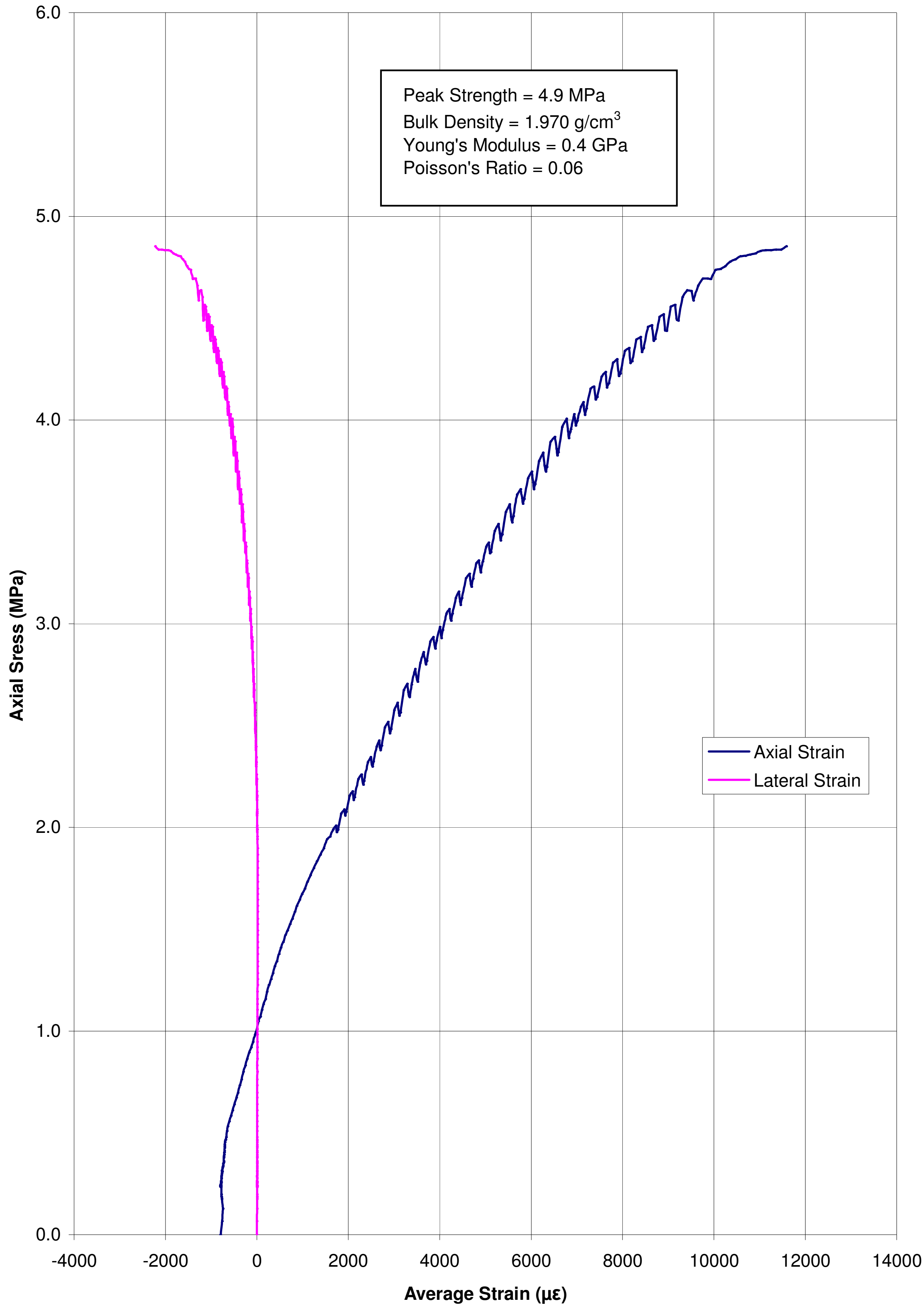
Peak Strength: 4.9 MPa

Young's Modulus: 0.4 GPa

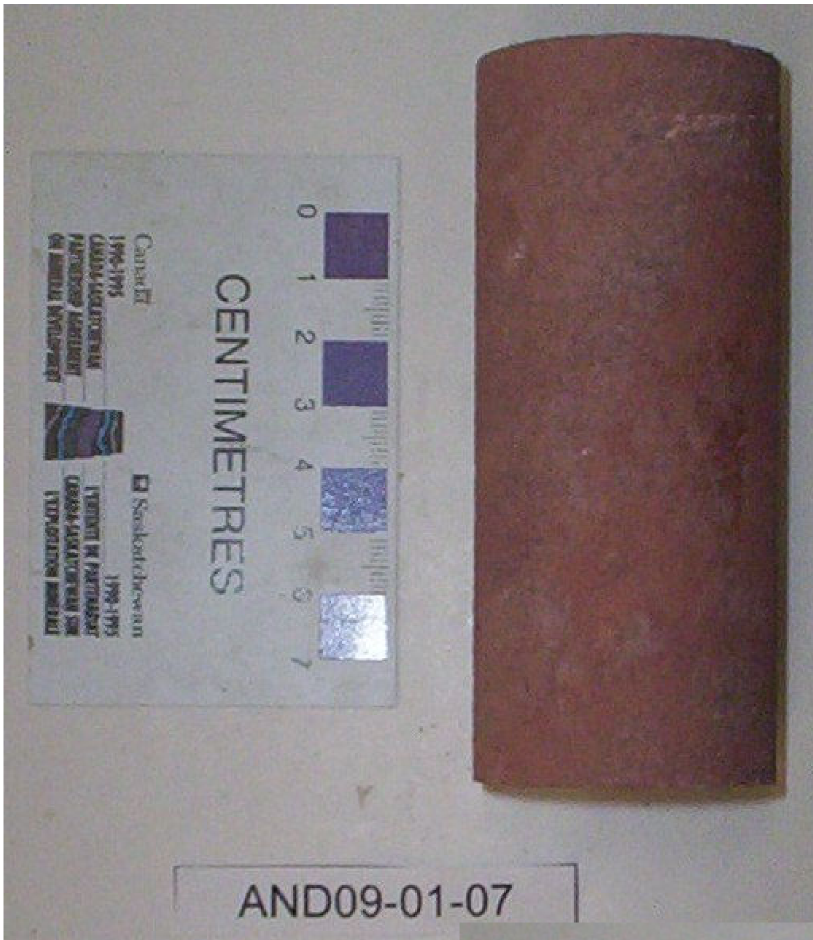
Poisson's Ratio: 0.06

Failure Cause: Intact Rock

Stress vs. Strain



SAMPLE: AND09-01-07



Length: 106.72 mm

Diameter: 45.14 mm

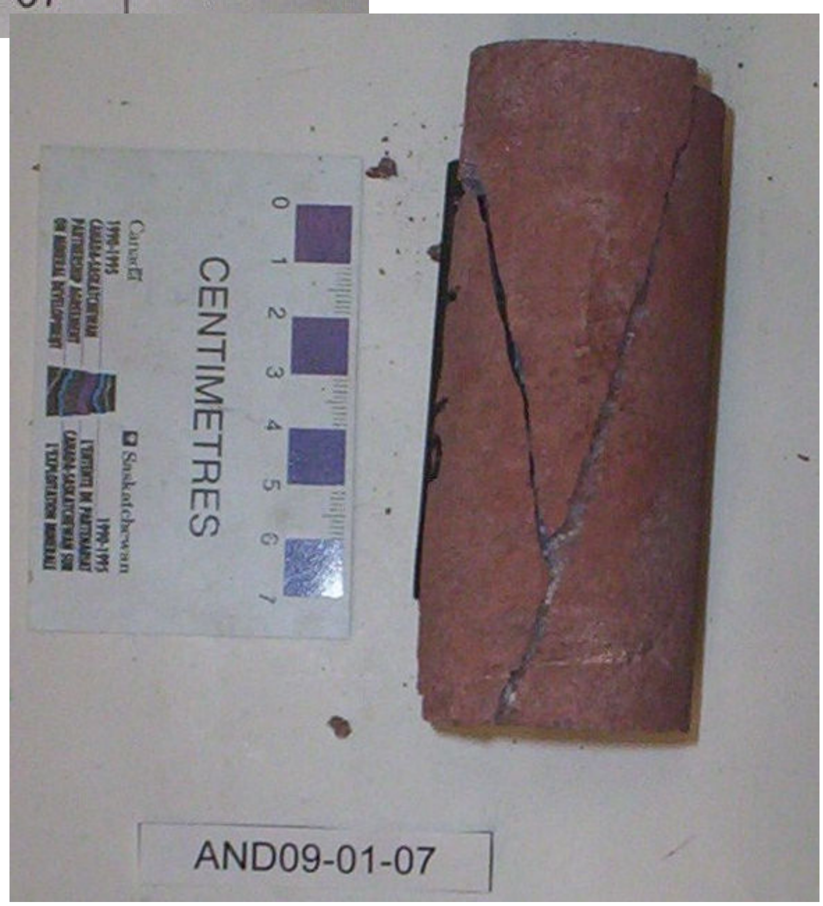
Density: 2.087 g/cm^3

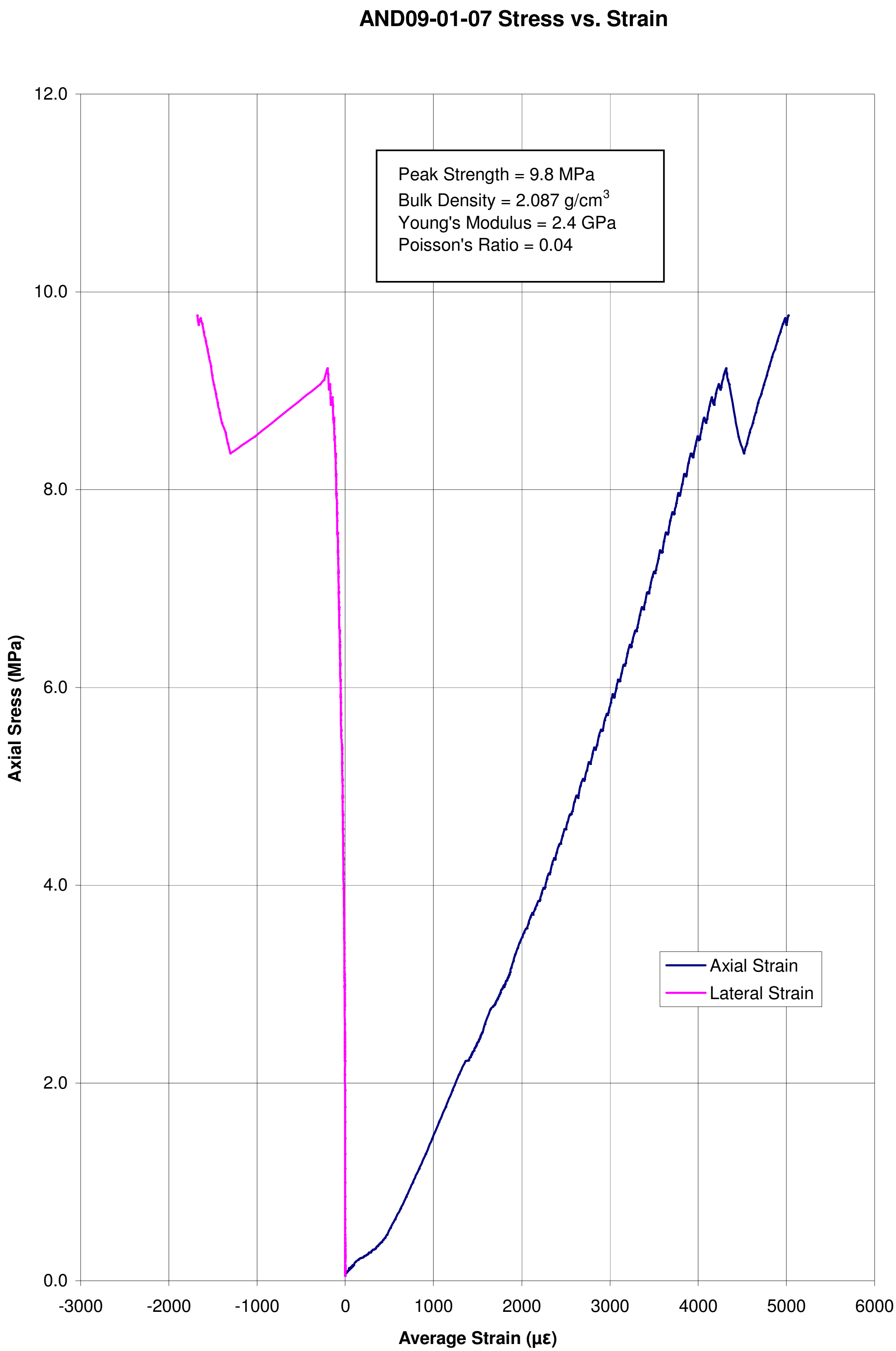
Peak Strength: 9.8 MPa

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/cm^3
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

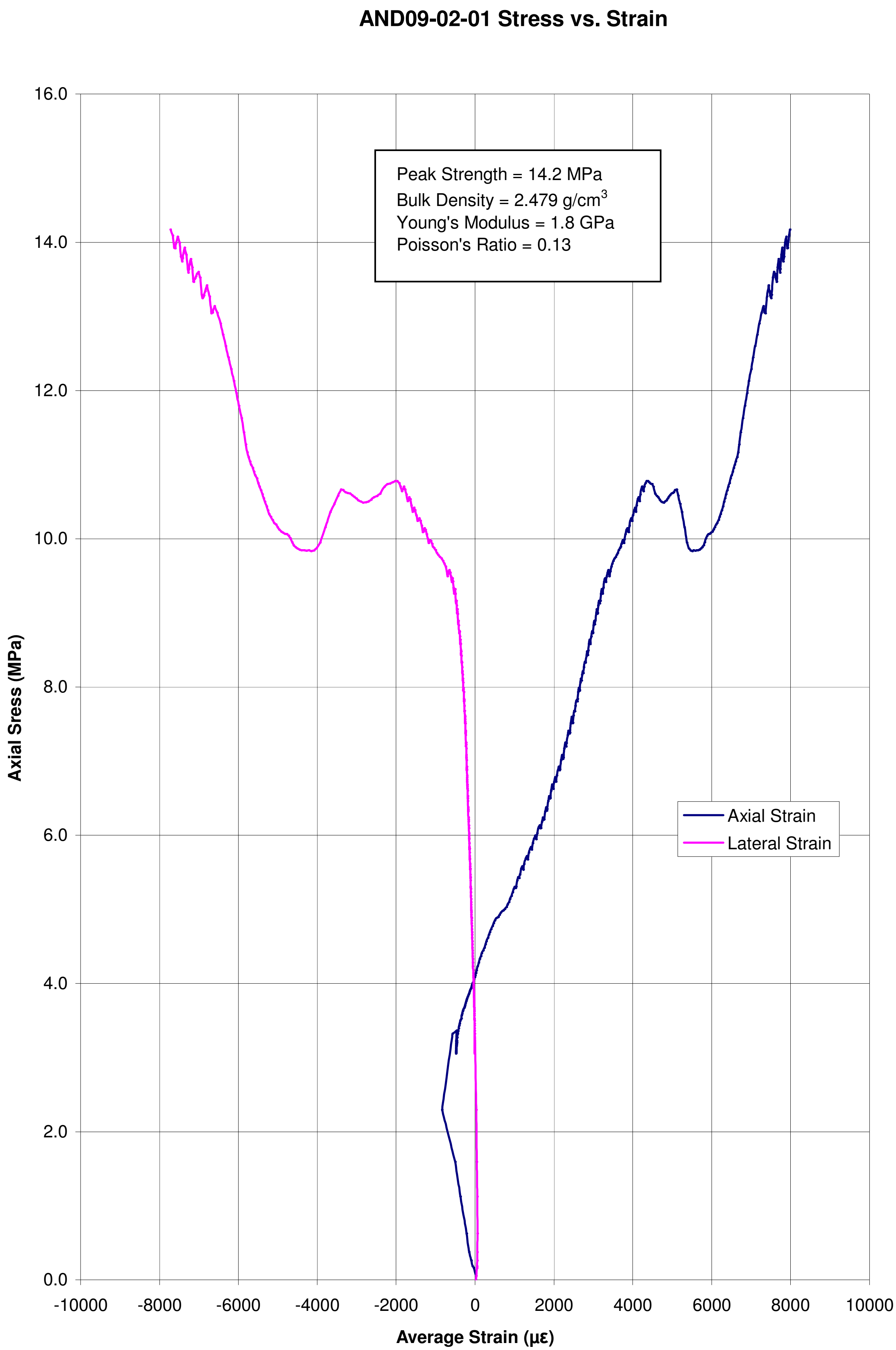
Density: 2.479 g/cm^3

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 Stress (MPa)
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
Average Strain (με)
Axial Strain
Lateral Strain

SAMPLE: AND09-02-02

Length: 111.62 mm

Diameter: 45.34 mm

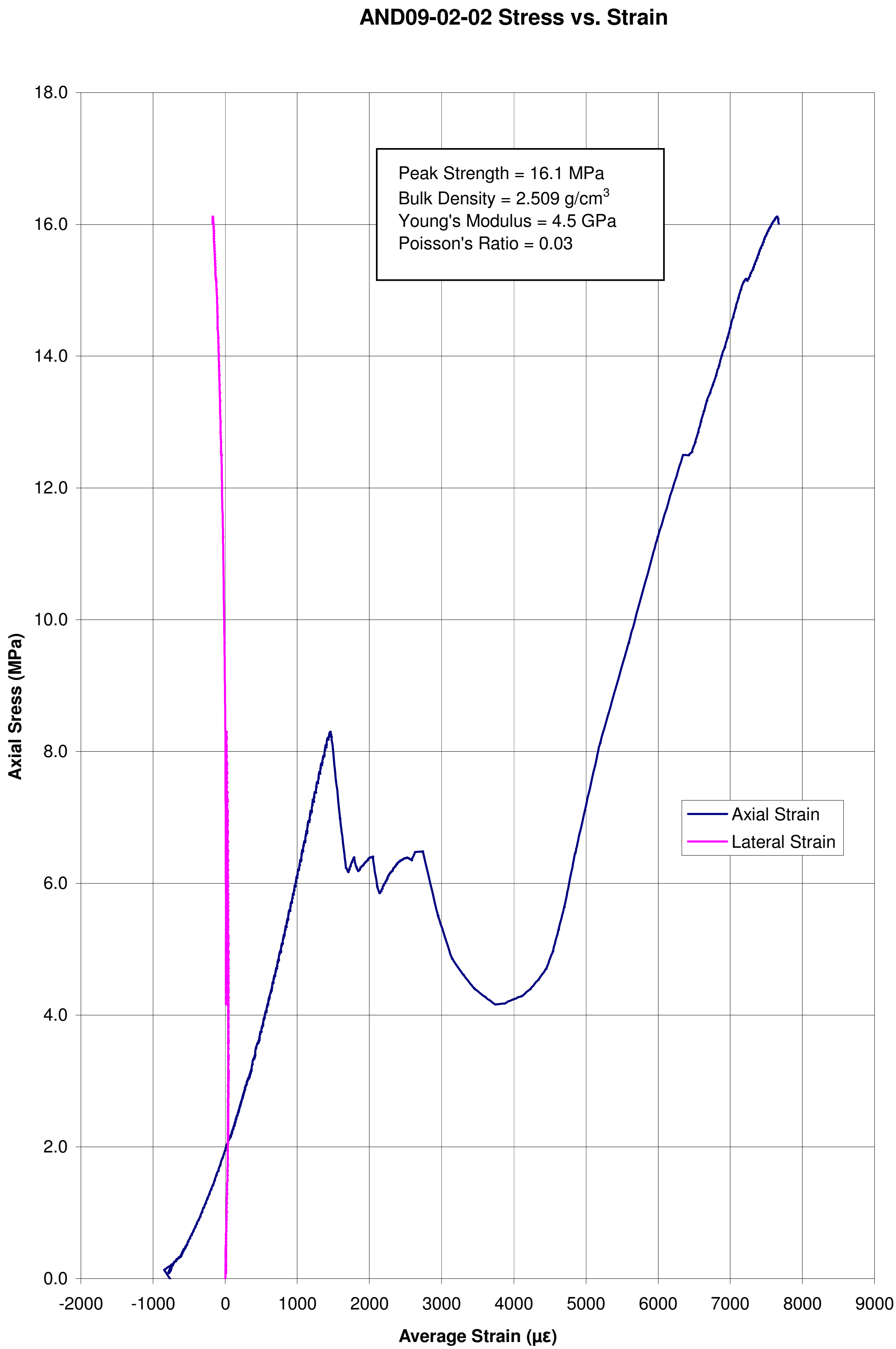
Density: 2.509 g/cm^3

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

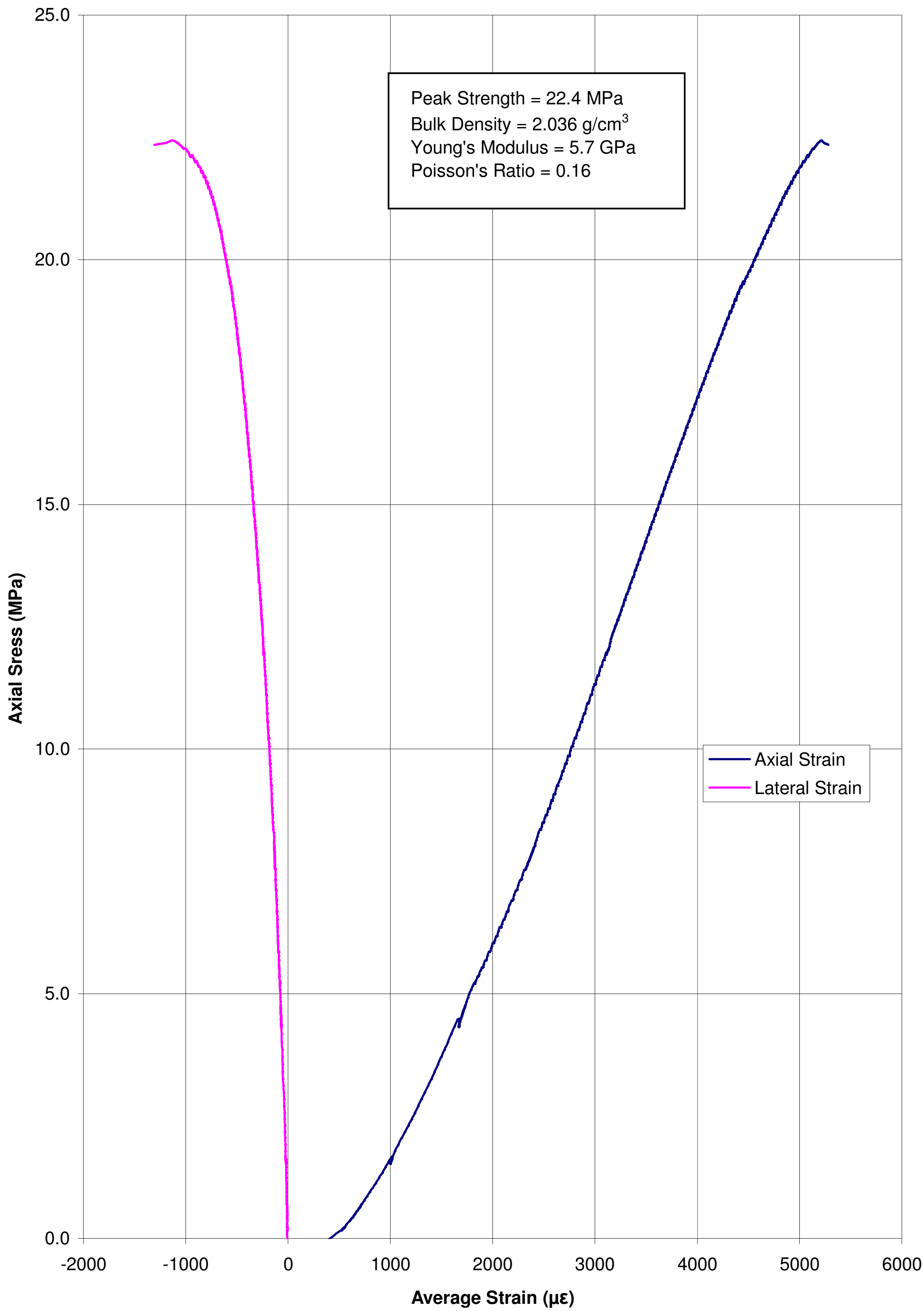
Peak Strength: 22.4 MPa

Young's Modulus: 5.7 GPa

Poisson's Ratio: 0.16

Failure Cause: Weakness Plane

AND09-02-03 Stress vs. Strain



SAMPLE: AND09-02-04

Length: 107.82 mm

Diameter: 45.25 mm

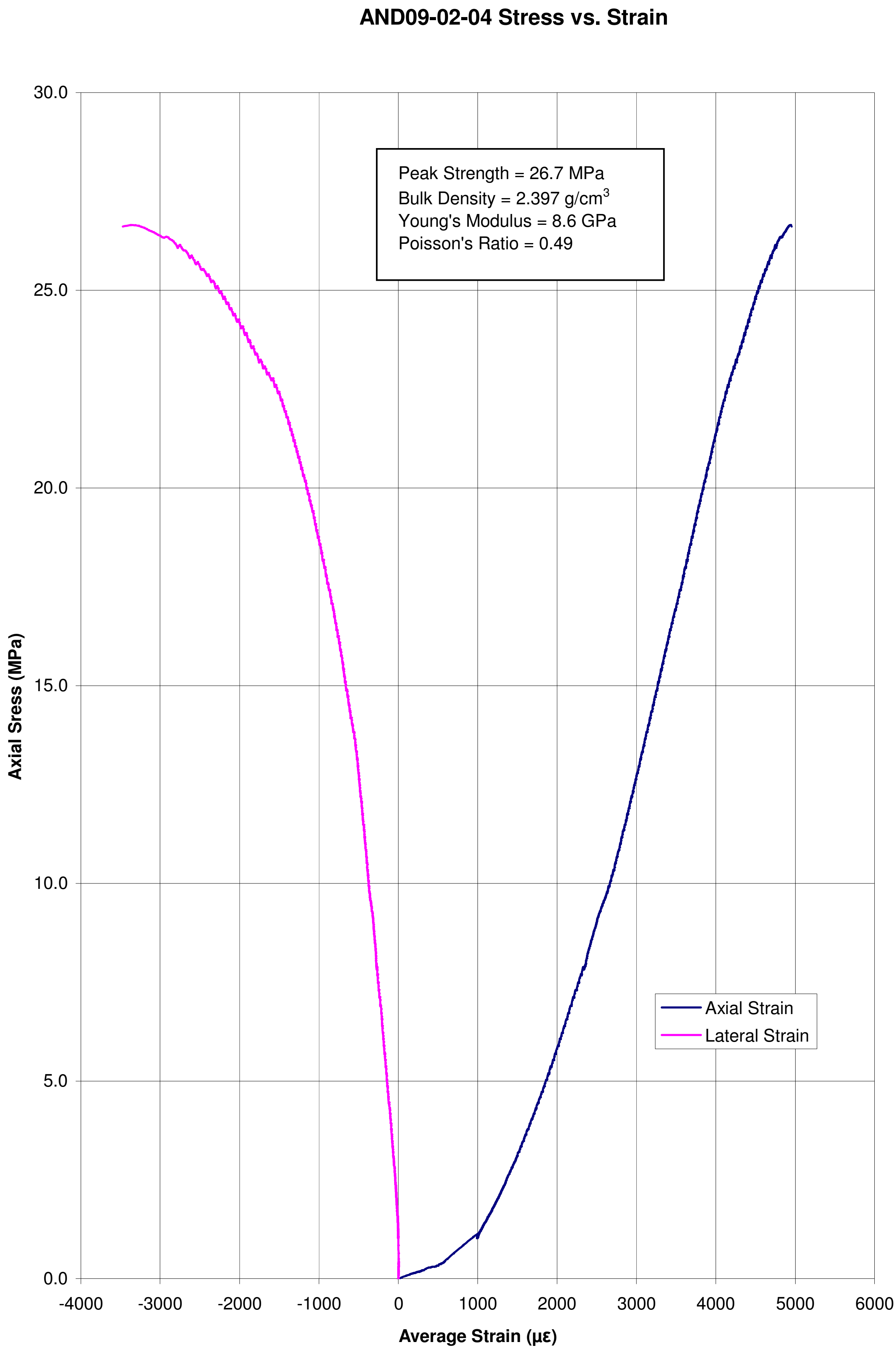
Density: 2.397 g/cm^3

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

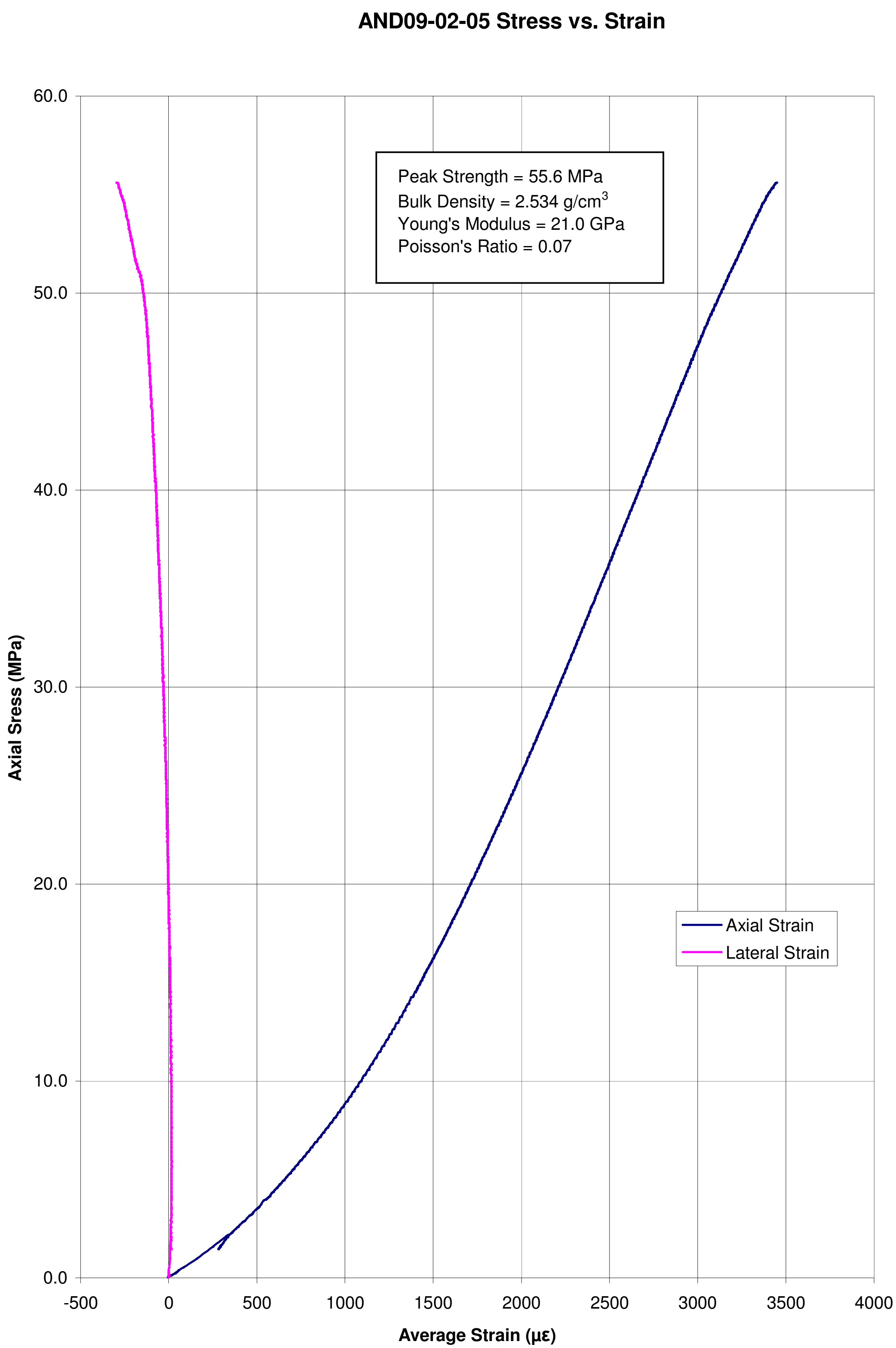
Density: 2.534 g/cm^3

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

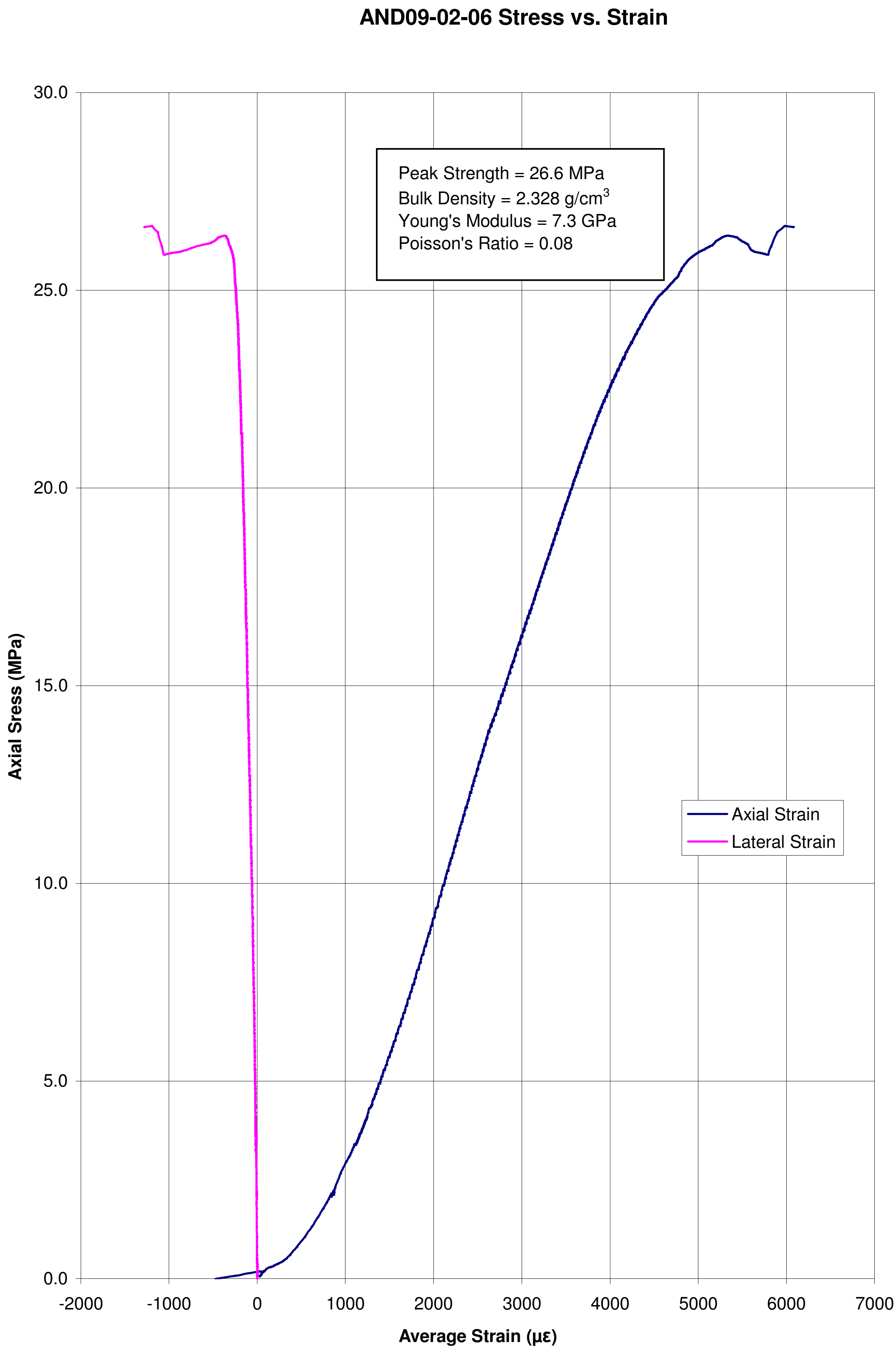
Density: 2.328 g/cm^3

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

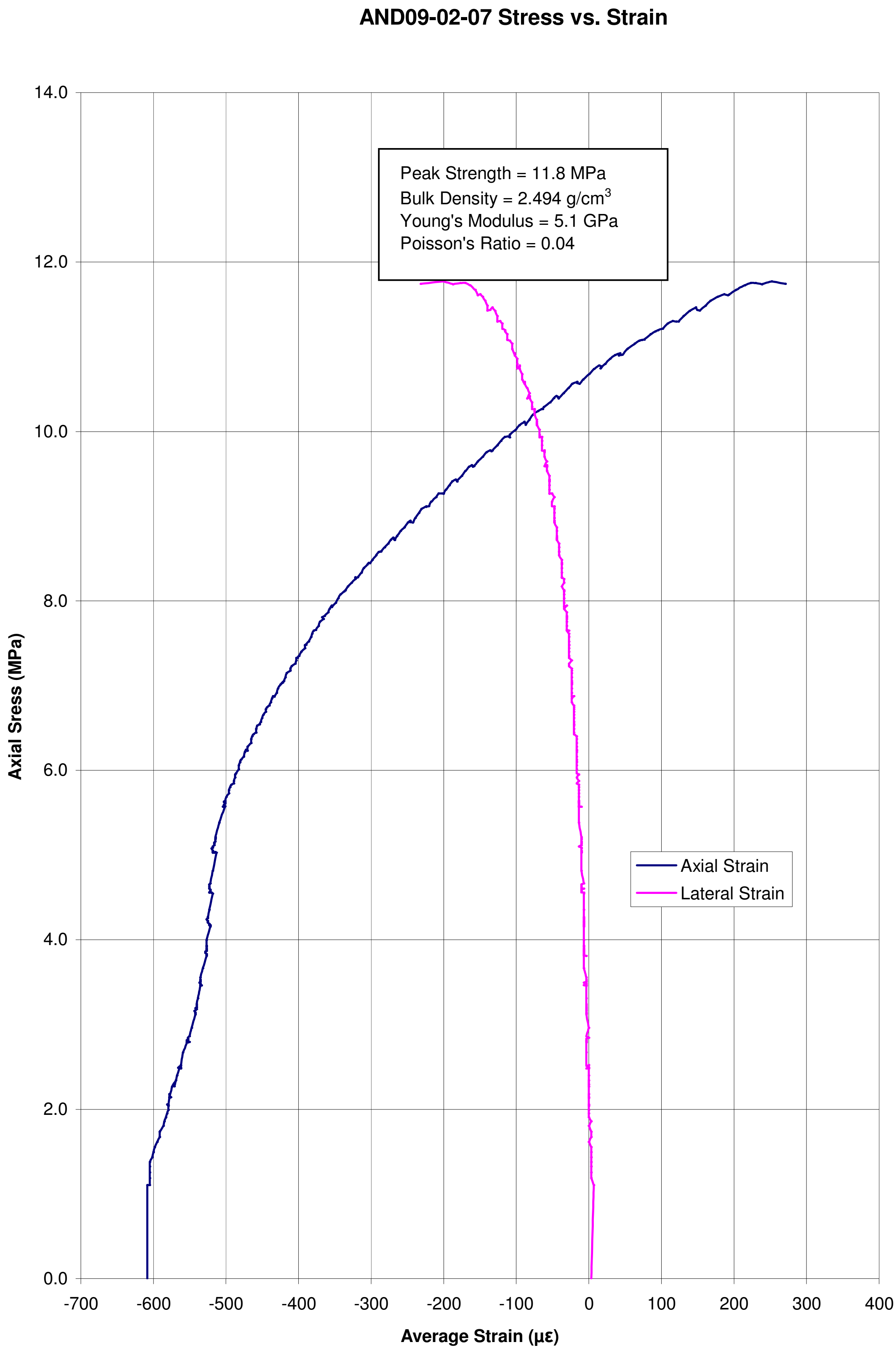
Density: 2.494 g/cm^3

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
Peak Strength = 11.8 MPa
Bulk Density = 2.494 g/cm3
Young's Modulus = 5.1 GPa
Poisson's Ratio = 0.04
Axial Strain
Lateral Strain
Axial Sress (MPa)
Average Strain (με)
14.0
12.0
10.0
8.0
6.0
4.0
2.0
0.0
-700
-600
-500
-400
-300
-200
-100
0
100
200
300
400

SAMPLE: AND09-02-08

Length: 94.50 mm

Diameter: 44.07 mm

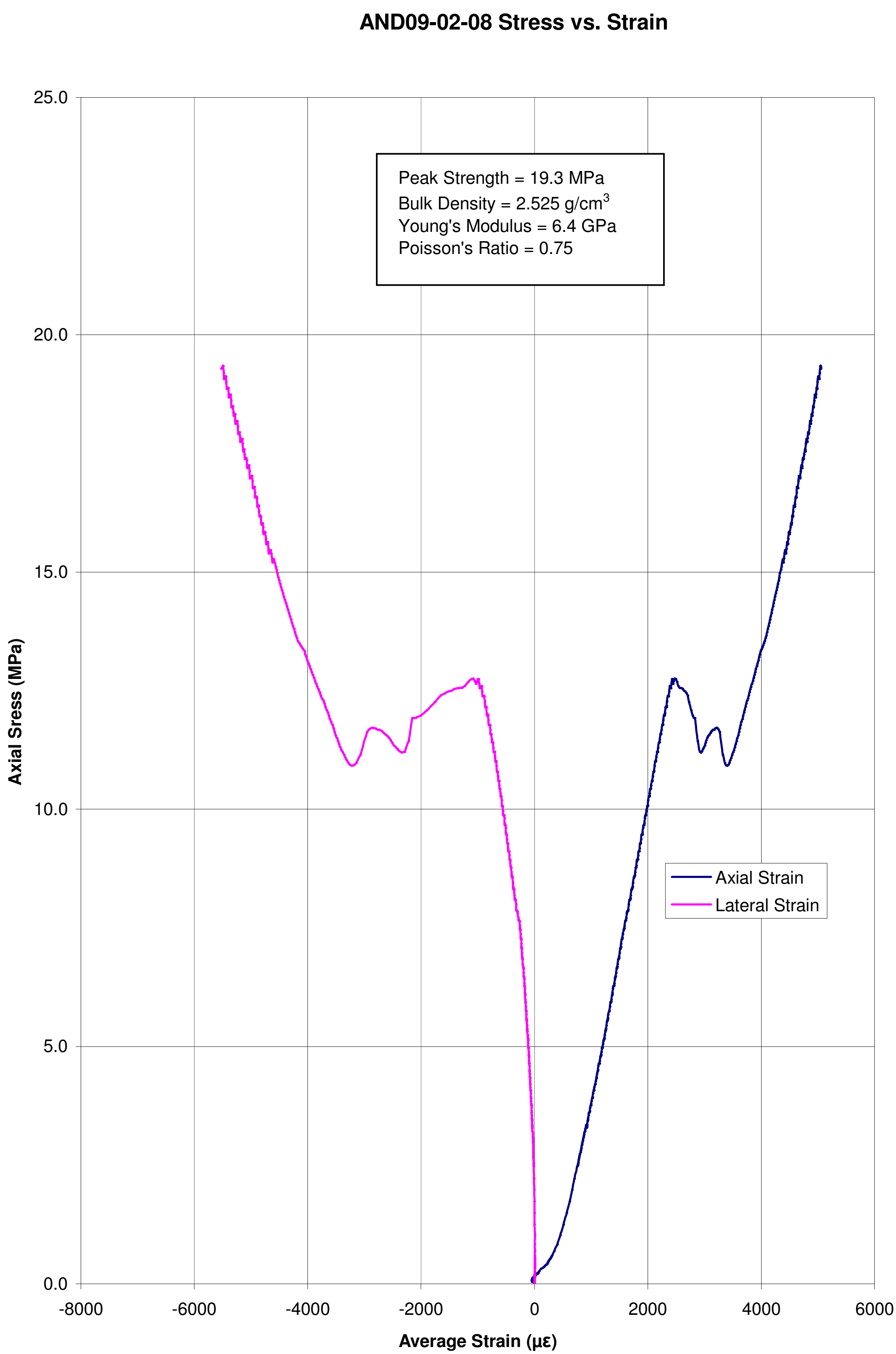
Density: 2.525 g/cm^3

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 Strain
Lateral Strain
Axial Sress (MPa)
Average Strain (με)
25.0
20.0
15.0
10.0
5.0
0.0
-8000
-6000
-4000
-2000
0
2000
4000
6000

SAMPLE: AND09-02-1A

Length: 72.18 mm

Diameter: 45.14 mm

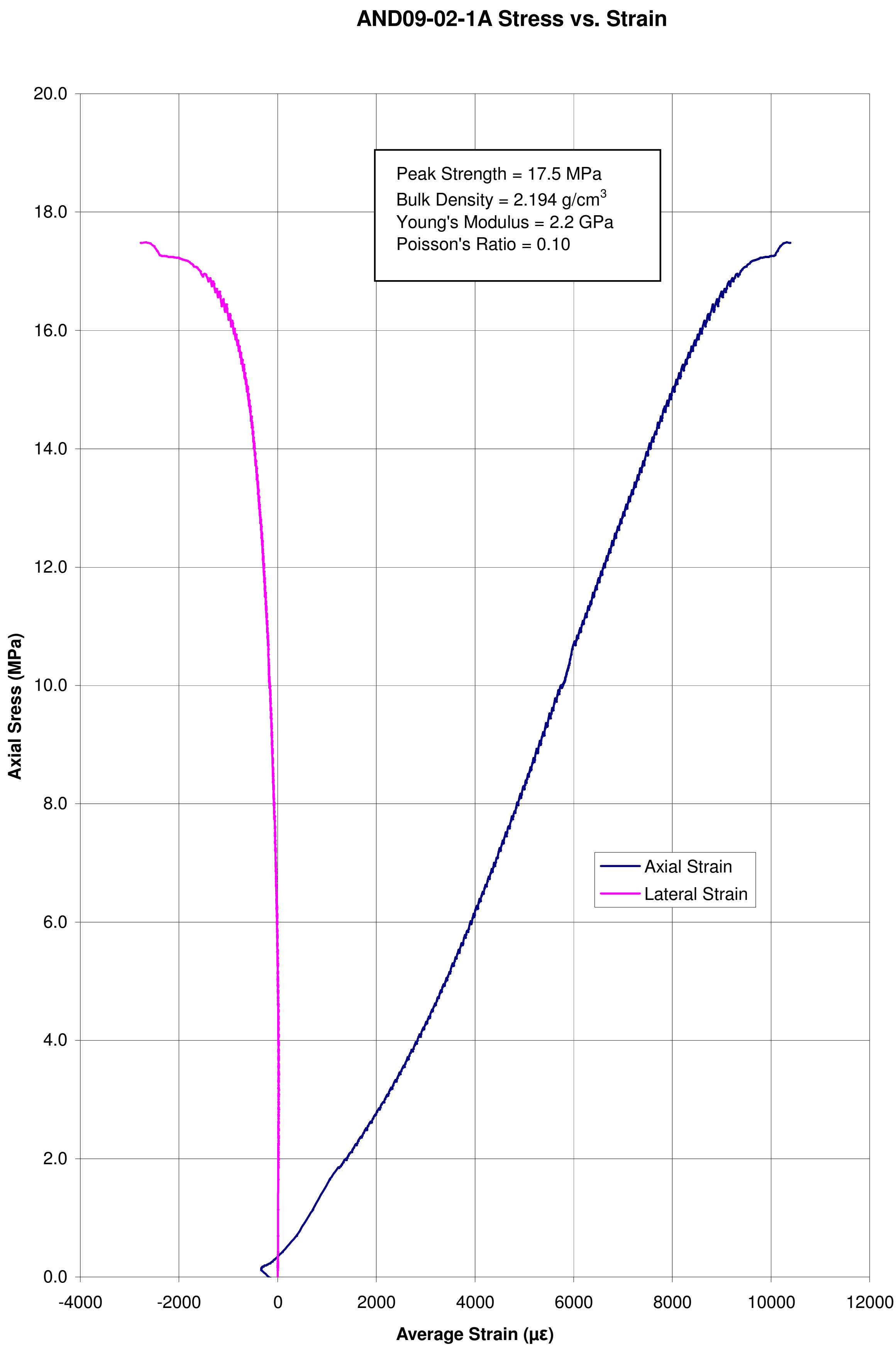
Density: 2.194 g/cm^3

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
Peak Strength = 17.5 MPa
Bulk Density = 2.194 g/cm3
Young's Modulus = 2.2 GPa
Poisson's Ratio = 0.10
Axial Strain
Lateral Strain
Axial Sress (MPa)
Average Strain (με)
20.0
18.0
16.0
14.0
12.0
10.0
8.0
6.0
4.0
2.0
0.0
-4000
-2000
0
2000
4000
6000
8000
10000
12000

SAMPLE: AND09-03-02

Length: 112.13 mm

Diameter: 45.16 mm

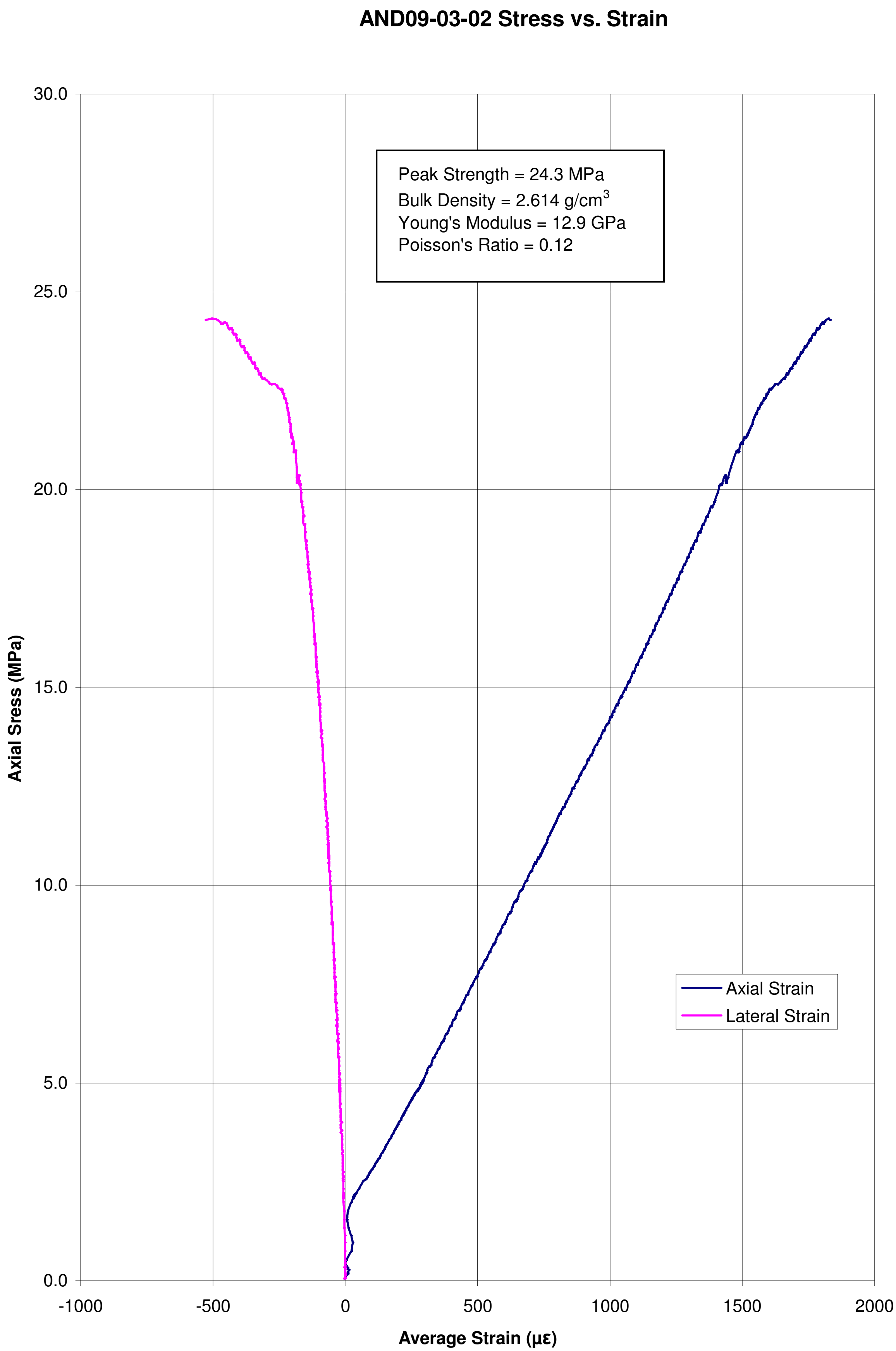
Density: 2.614 g/cm^3

Peak Strength: 24.3 MPa

Young’s Modulus: 12.9 GPa

Poisson’s Ratio: 0.12

Failure Cause: Weakness Plane?



AND09-03-02 Stress vs. Strain
Peak Strength = 24.3 MPa
Bulk Density = 2.614 g/cm3
Young's Modulus = 12.9 GPa
Poisson's Ratio = 0.12
Axial Strain
Lateral Strain
Axial Sress (MPa)
Average Strain (με)
30.0
25.0
20.0
15.0
10.0
5.0
0.0
-1000
-500
0
500
1000
1500
2000

SAMPLE: AND09-03-03

Length: 96.76 mm

Diameter: 45.06 mm

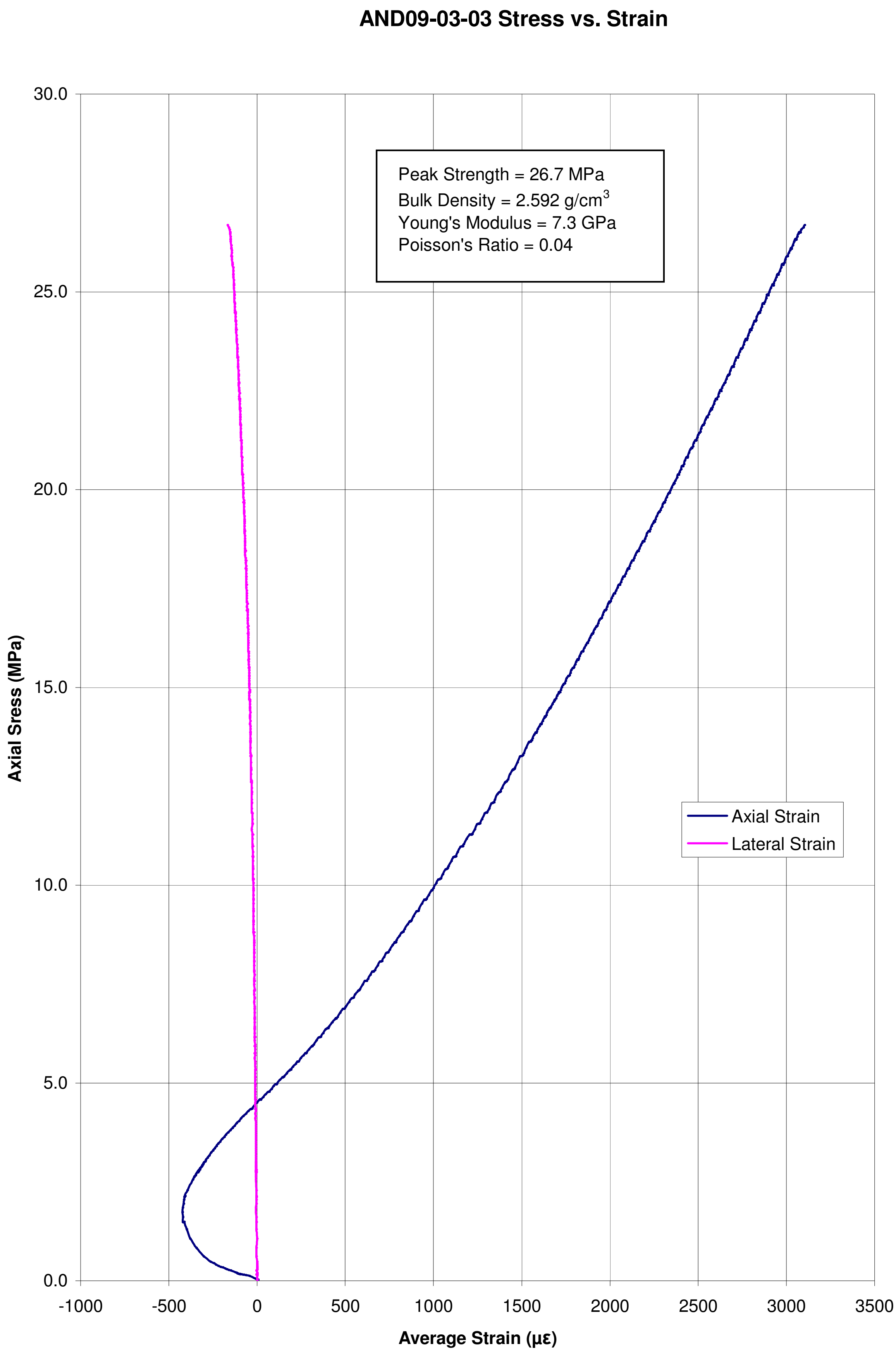
Density: 2.592 g/cm^3

Peak Strength: 26.7 MPa

Young's Modulus: 7.3 GPa

Poisson's Ratio: 0.04

Failure Cause: Microfracture



AND09-03-03 Stress vs. Strain
Peak Strength = 26.7 MPa
Bulk Density = 2.592 g/cm3
Young's Modulus = 7.3 GPa
Poisson's Ratio = 0.04
Axial Strain
Lateral Strain
Axial Sress (MPa)
Average Strain (με)
30.0
25.0
20.0
15.0
10.0
5.0
0.0
-1000
-500
0
500
1000
1500
2000
2500
3000
3500

SAMPLE: AND09-03-04

Length: 90.03 mm

Diameter: 45.16 mm

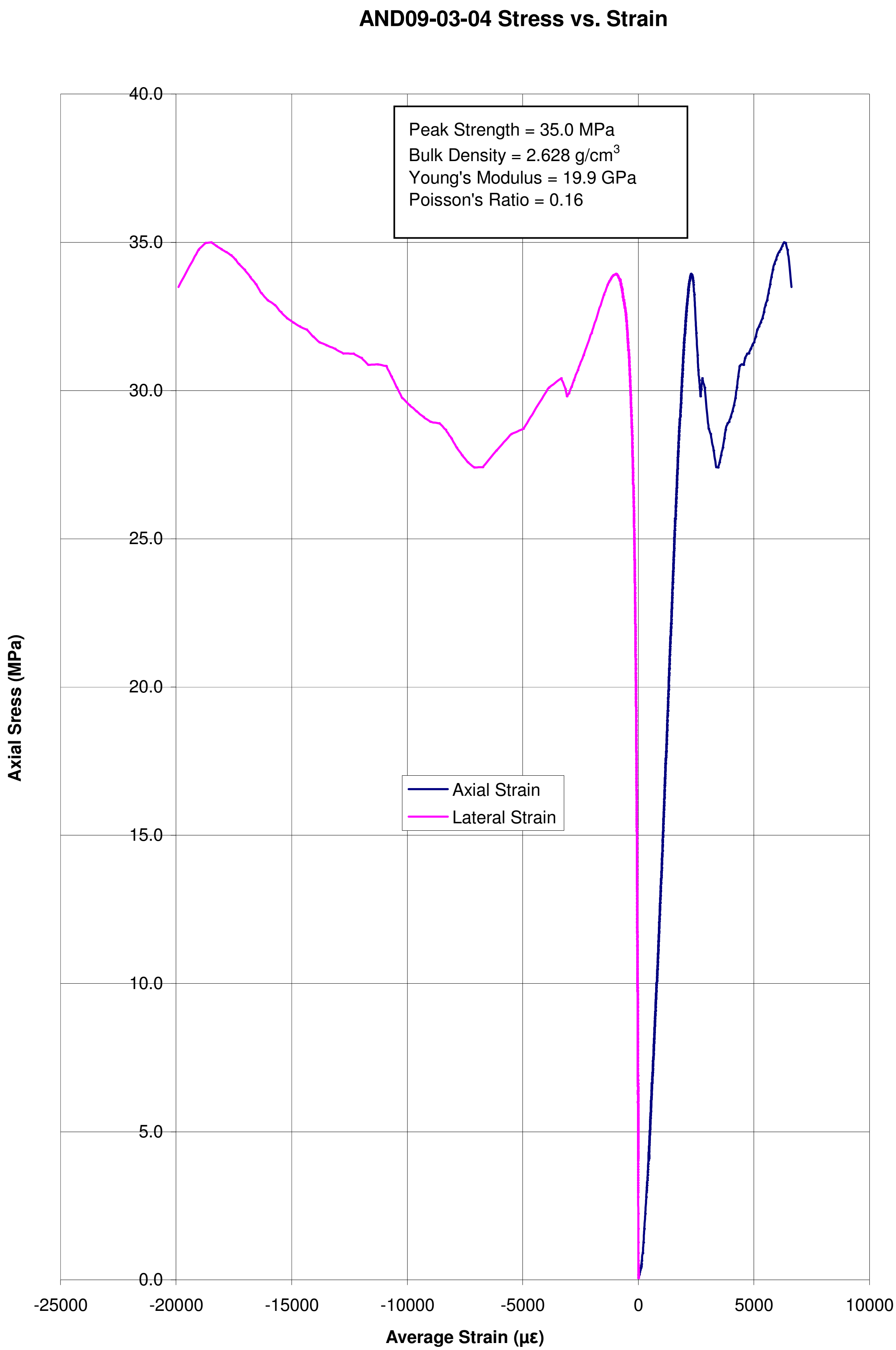
Density: 2.628 g/cm^3

Peak Strength: 35.0 MPa

Young’s Modulus: 19.9 GPa

Poisson’s Ratio: 0.16

Failure Cause: Intact Rock



AND09-03-04 Stress vs. Strain
Peak Strength = 35.0 MPa
Bulk Density = 2.628 g/cm3
Young's Modulus = 19.9 GPa
Poisson's Ratio = 0.16
Axial Strain
Lateral Strain
Axial Sress (MPa)
Average Strain (με)
40.0
35.0
30.0
25.0
20.0
15.0
10.0
5.0
0.0
-25000
-20000
-15000
-10000
-5000
0
5000
10000

SAMPLE: CZ09-01-04

Length: 108.77 mm

Diameter: 45.43 mm

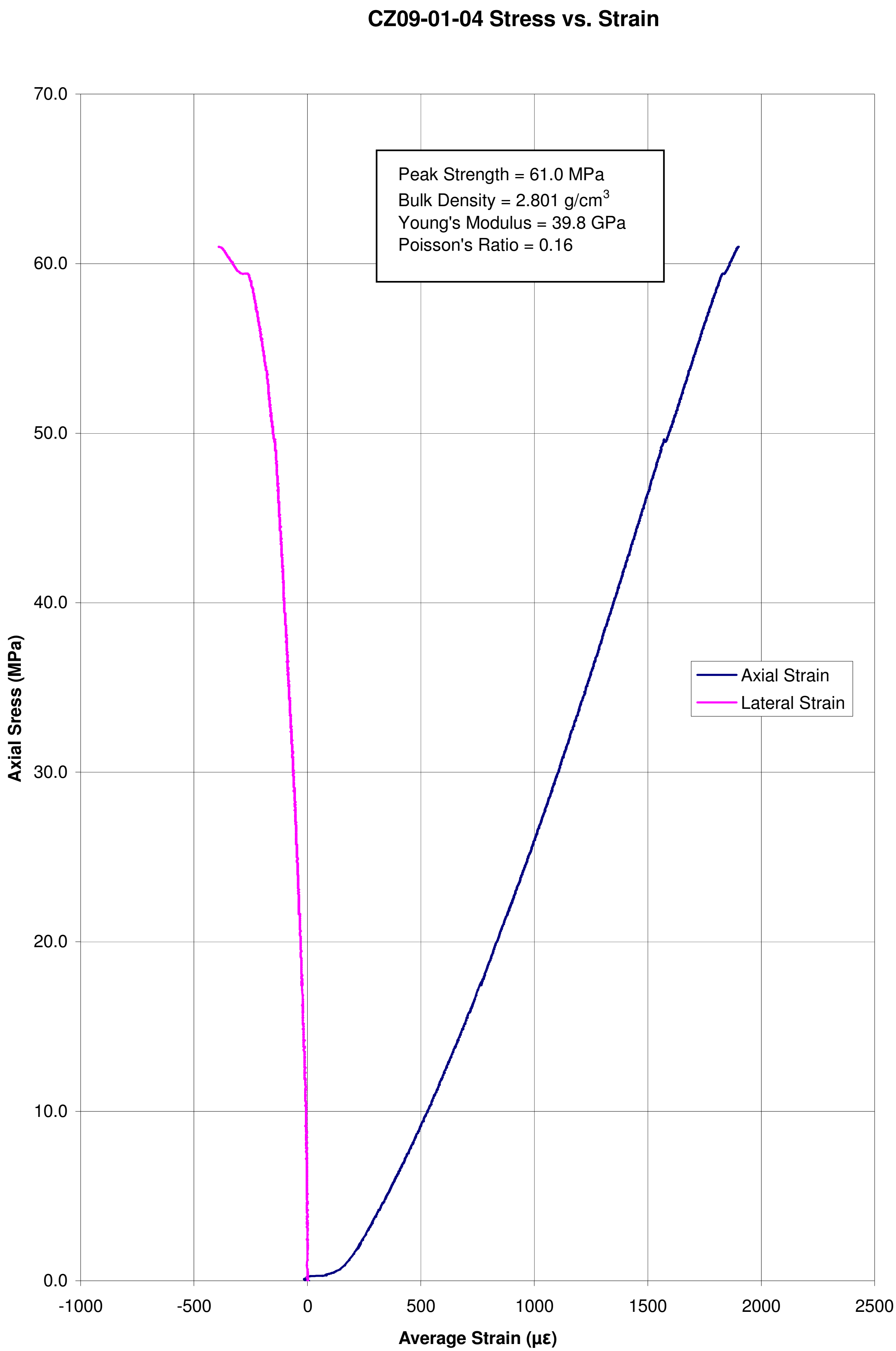
Density: 2.801 g/cm^3

Peak Strength: 61.0 MPa

Young's Modulus: 39.8 GPa

Poisson's Ratio: 0.16

Failure Cause: Intact Rock



CZ09-01-04 Stress vs. Strain
Peak Strength = 61.0 MPa
Bulk Density = 2.801 g/cm3
Young's Modulus = 39.8 GPa
Poisson's Ratio = 0.16
Axial Strain
Lateral Strain
Axial Sress (MPa)
Average Strain (με)
70.0
60.0
50.0
40.0
30.0
20.0
10.0
0.0
-1000
-500
0
500
1000
1500
2000
2500

SAMPLE: CZ09-01-05

Length: 108.58 mm

Diameter: 45.4 mm

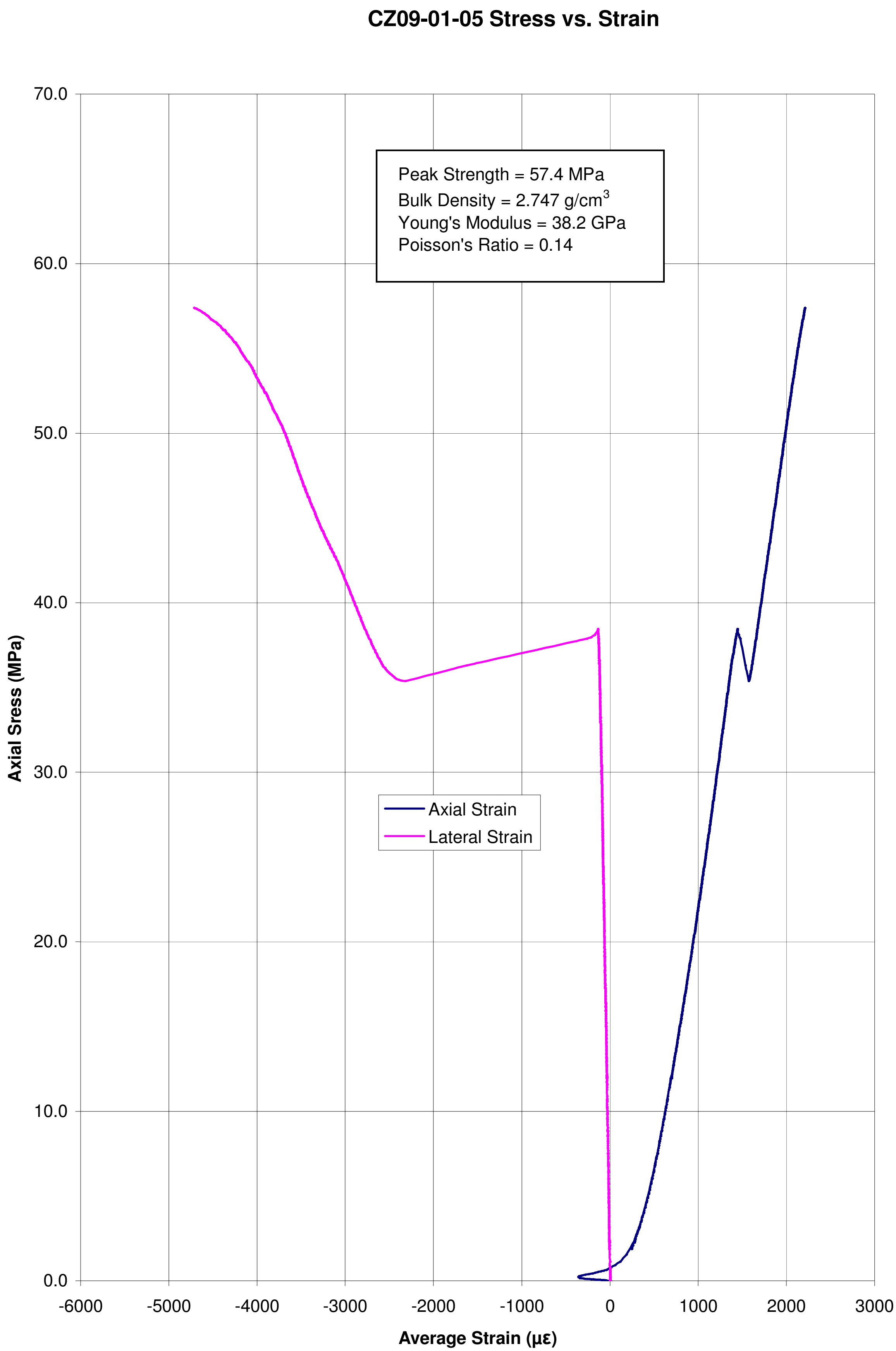
Density: 2.747 g/cm^3

Peak Strength: 57.4 MPa

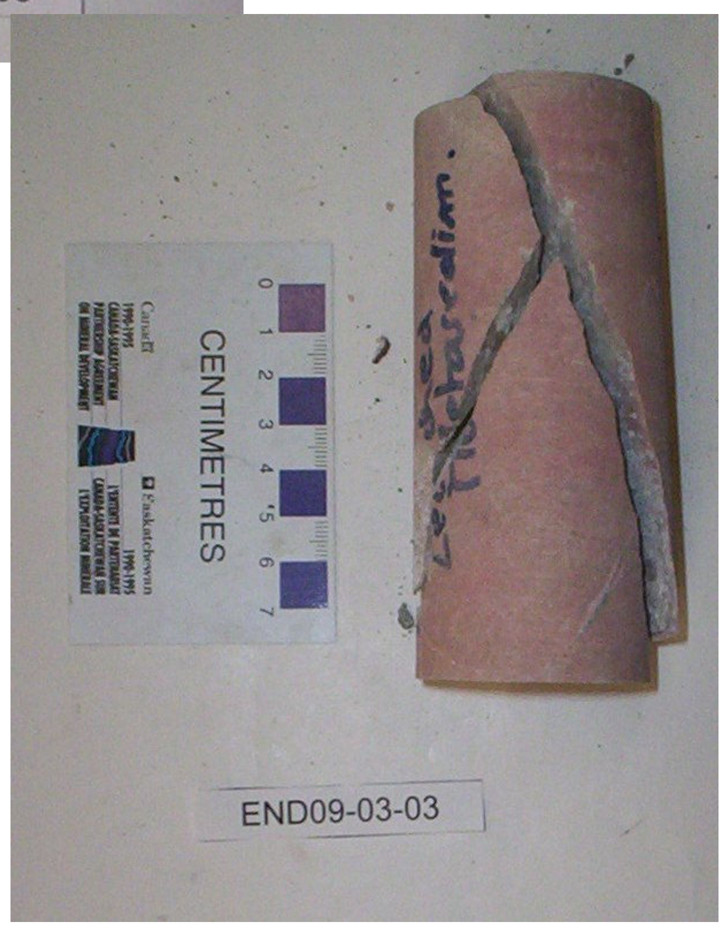
Young’s Modulus: 38.2 GPa

Poisson’s Ratio: 0.14

Failure Cause: Intact Rock


CZ09-01-05 Stress vs. Strain
Peak Strength = 57.4 MPa
Bulk Density = 2.747 g/cm3
Young's Modulus = 38.2 GPa
Poisson's Ratio = 0.14
Axial Strain
Lateral Strain
Axial Sress (MPa)
Average Strain (με)
70.0
60.0
50.0
40.0
30.0
20.0
10.0
0.0
-6000
-5000
-4000
-3000
-2000
-1000
0
1000
2000
3000

SAMPLE: END09-03-03



Length: 108.99 mm

Diameter: 47.78 mm

Density: 2.373 g/cm^3

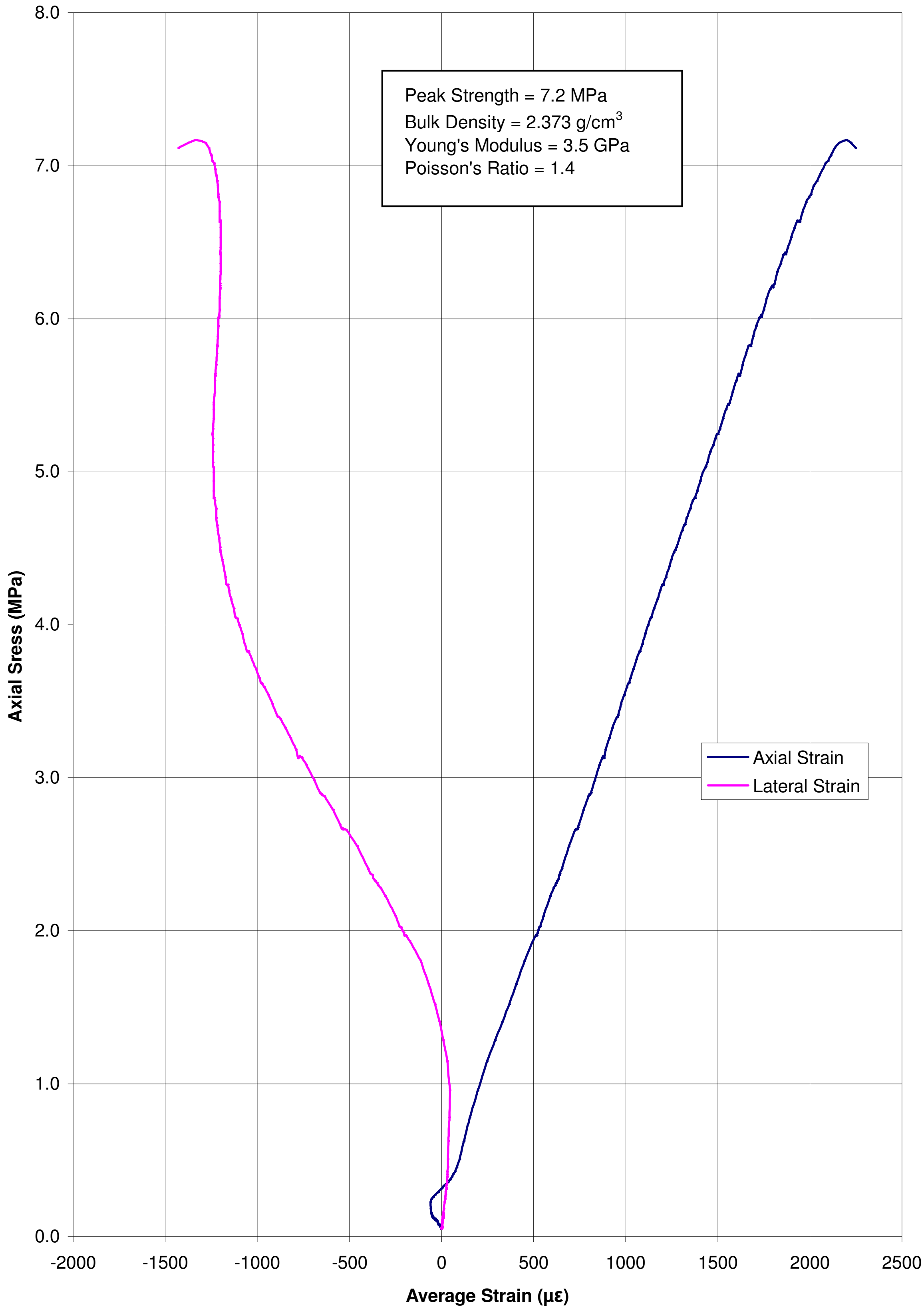
Peak Strength: 7.2 MPa

Young's Modulus: 3.5 GPa

Poisson's Ratio: 1.4 ?

Failure Cause: Weakness Plane

END09-03-03 Stress vs. Strain



SAMPLE: END09-03-04

Length: 114.07 mm

Diameter: 46.83 mm

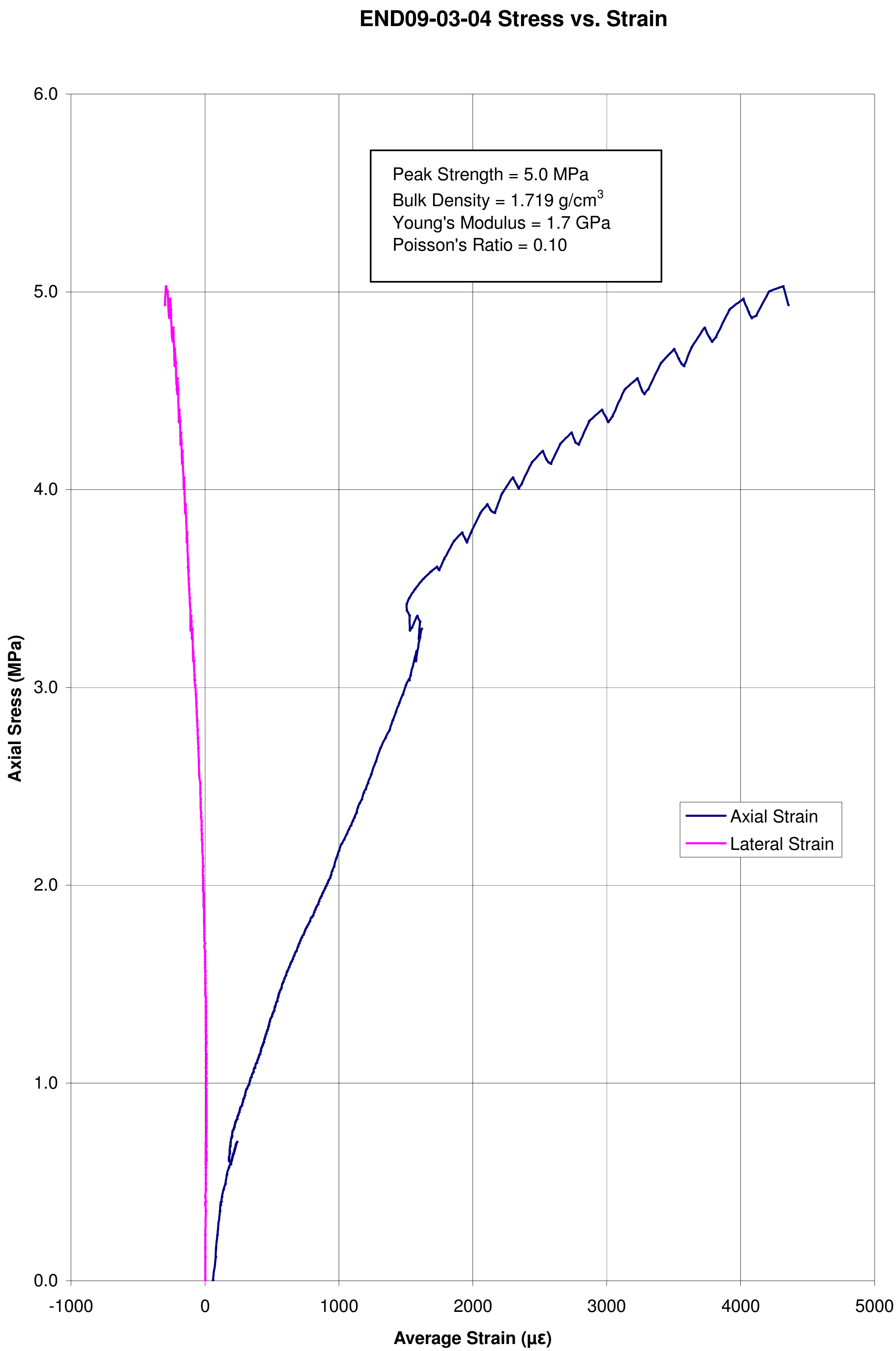
Density: 1.719 g/cm^3

Peak Strength: 5.0 MPa

Young's Modulus: 1.7 GPa

Poisson's Ratio: 0.10

Failure Cause: Intact Rock



END09-03-04 Stress vs. Strain
Peak Strength = 5.0 MPa
Bulk Density = 1.719 g/cm3
Young's Modulus = 1.7 GPa
Poisson's Ratio = 0.10
Axial Sress (MPa)
6.0
5.0
4.0
3.0
2.0
1.0
0.0
-1000
0
1000
2000
3000
4000
5000
Average Strain (με)
Axial Strain
Lateral Strain

SAMPLE: END09-03-06

Length: 111.12 mm

Diameter: 47.77mm

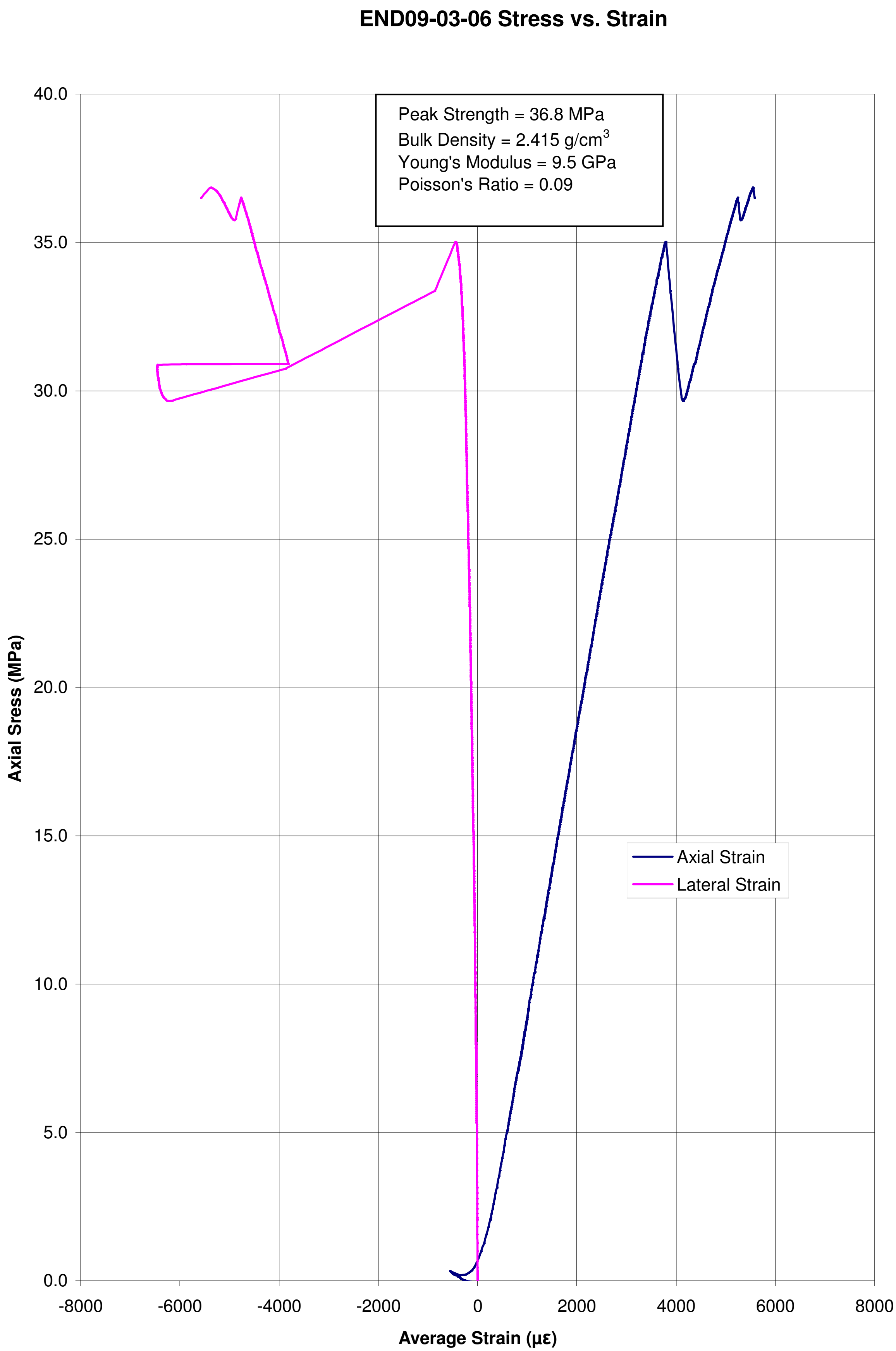
Density: 2.415 g/cm^3

Peak Strength: 36.8 MPa

Young's Modulus: 9.5 GPa

Poisson's Ratio: 0.09

Failure Cause: Weakness Plane



END09-03-06 Stress vs. Strain
Peak Strength = 36.8 MPa
Bulk Density = 2.415 g/cm3
Young's Modulus = 9.5 GPa
Poisson's Ratio = 0.09
Axial Sress (MPa)
Average Strain (µε)
Axial Strain
Lateral Strain
40.0
35.0
30.0
25.0
20.0
15.0
10.0
5.0
0.0
-8000
-6000
-4000
-2000
0
2000
4000
6000
8000

SAMPLE: END09-03-07

Length: 112.13 mm

Diameter: 47.59mm

Density: 2.441g/cm^3

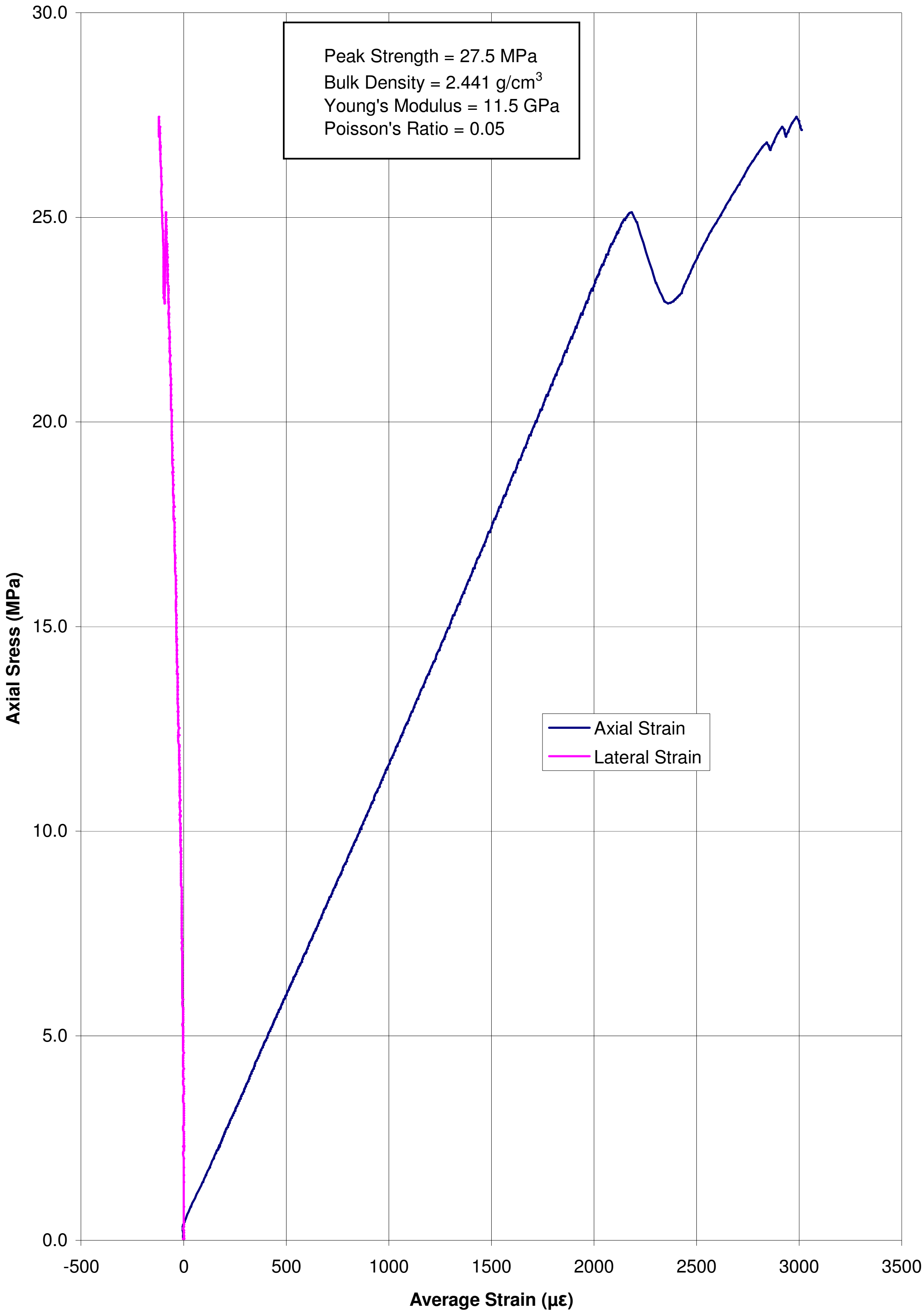
Peak Strength: 27.5 MPa

Young's Modulus: 11.5 GPa

Poisson's Ratio: 0.05

Failure Cause: Weakness Plane

END09-03-07 Stress vs. Strain



SAMPLE: END09-04-01

Length: 111.75mm

Diameter: 45.25mm

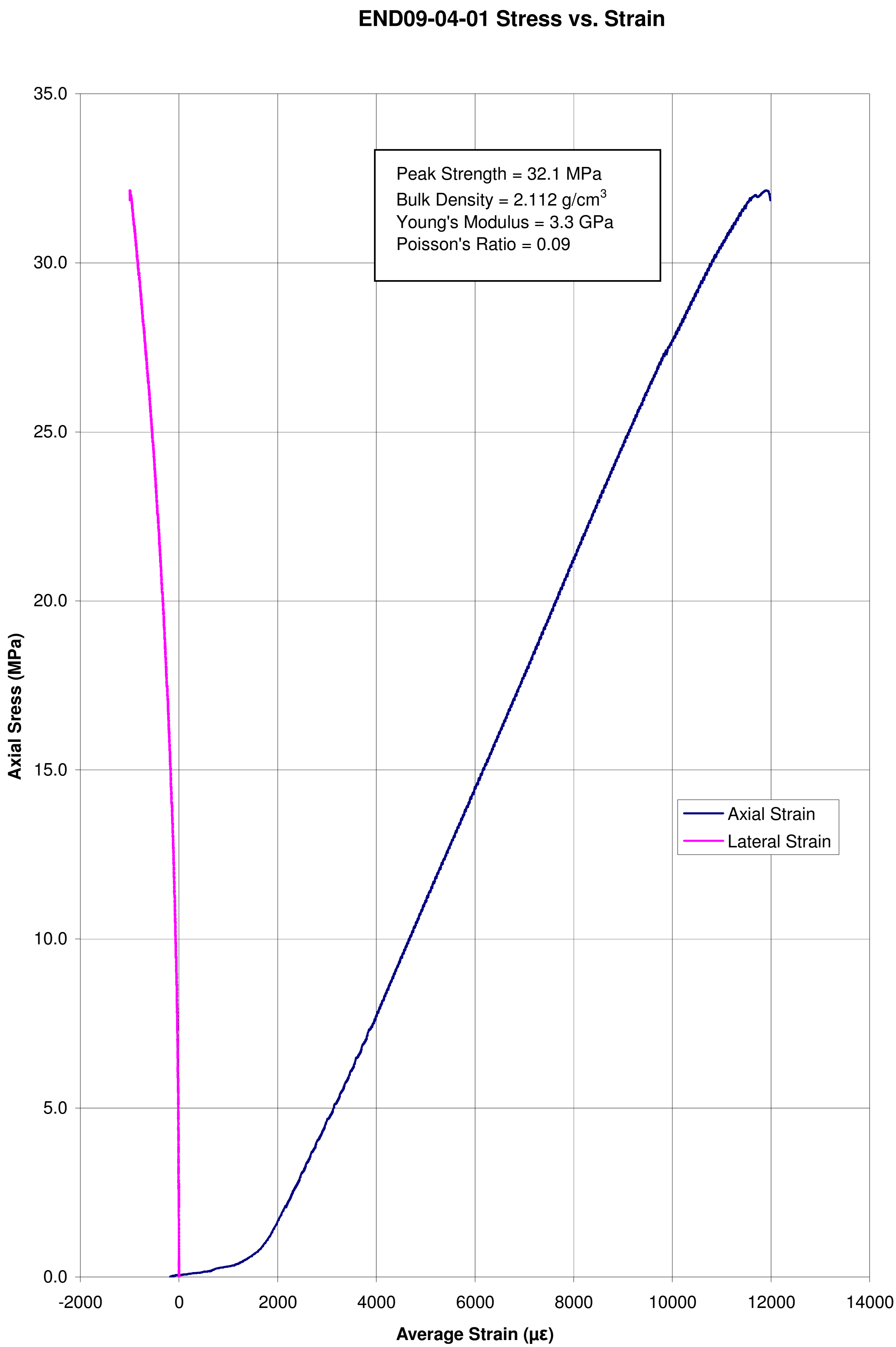
Density: 2.112g/cm^3

Peak Strength: 32.1 MPa

Young's Modulus: 3.3 GPa

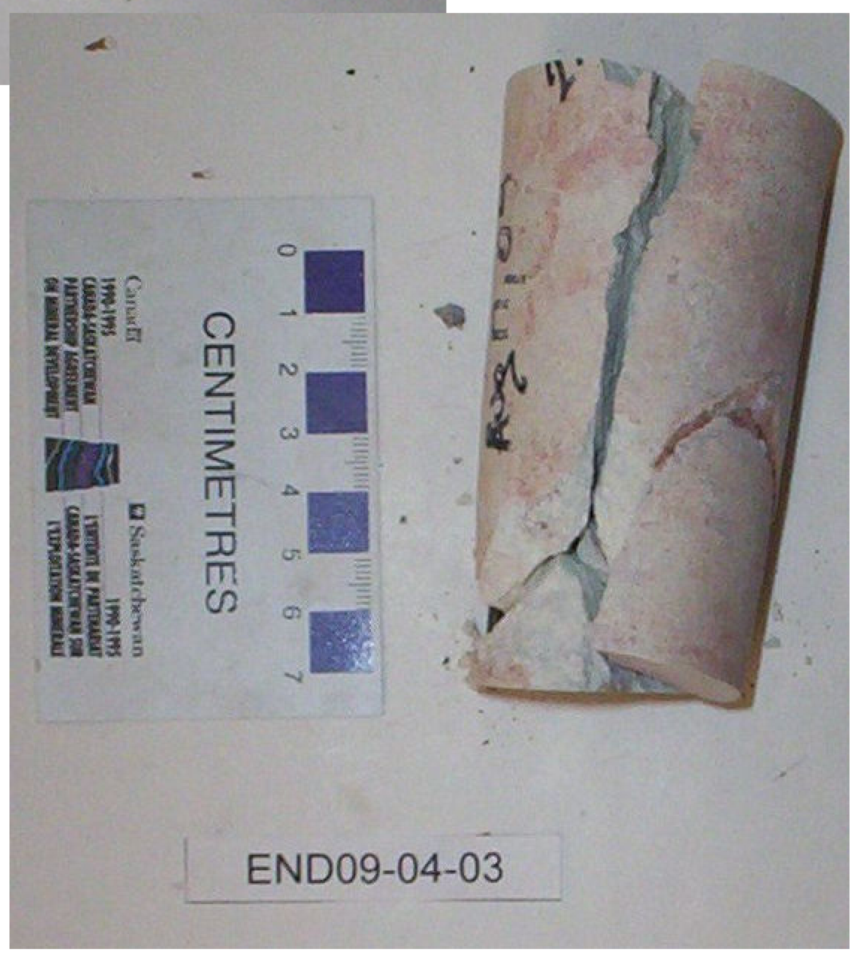
Poisson's Ratio: 0.09

Failure Cause: Intact Rock



END09-04-01 Stress vs. Strain
Peak Strength = 32.1 MPa
Bulk Density = 2.112 g/cm3
Young's Modulus = 3.3 GPa
Poisson's Ratio = 0.09
Axial Strain
Lateral Strain
Axial Sress (MPa)
Average Strain (με)
35.0
30.0
25.0
20.0
15.0
10.0
5.0
0.0
-2000
0
2000
4000
6000
8000
10000
12000
14000

SAMPLE: END09-04-03



Length: 91.27mm

Diameter: 44.83mm

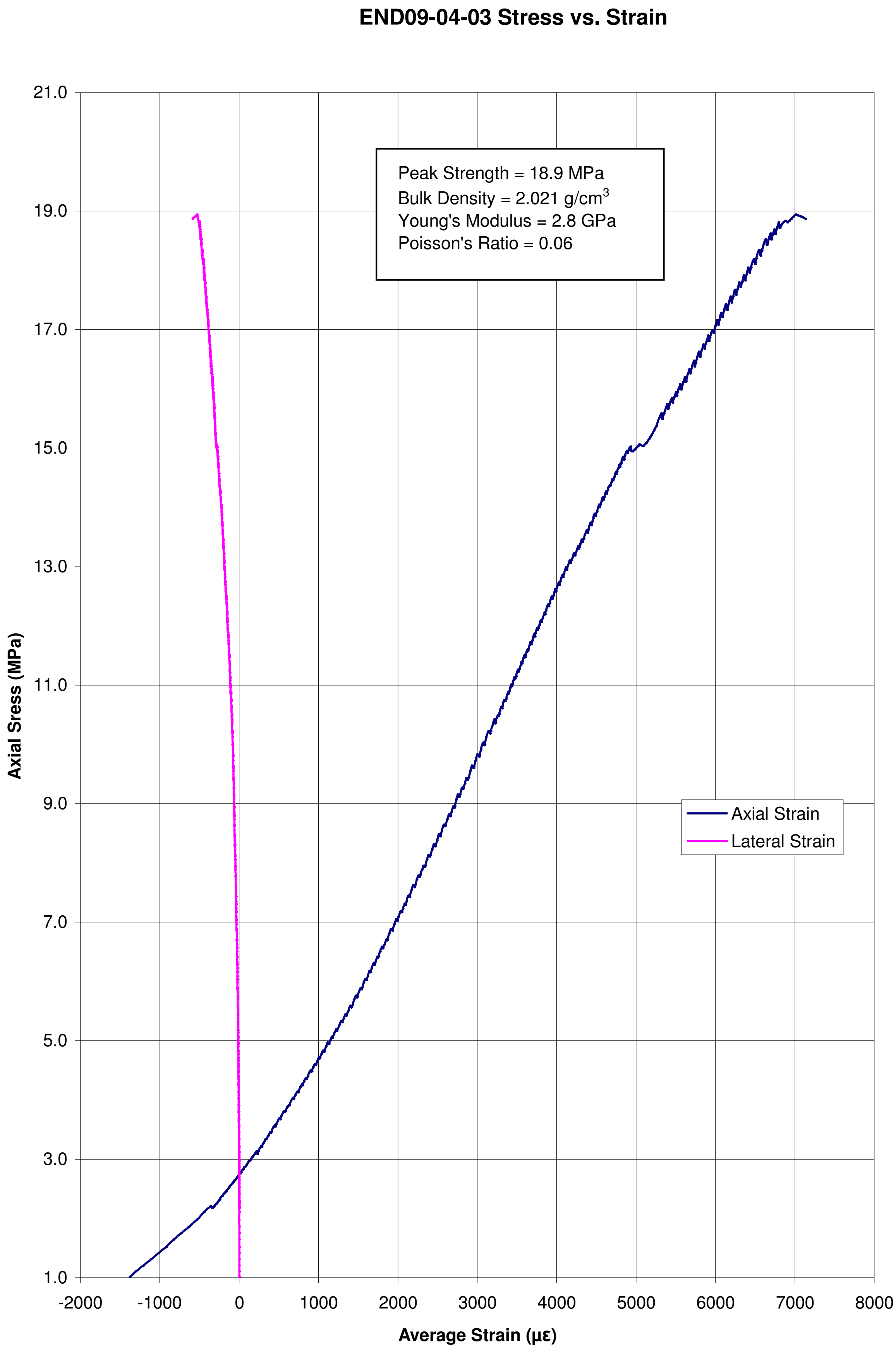
Density: 2.021g/cm^3

Peak Strength: 18.9 MPa

Young's Modulus: 2.8 GPa

Poisson's Ratio: 0.06

Failure Cause: Intact Rock


END09-04-03 Stress vs. Strain
Peak Strength = 18.9 MPa
Bulk Density = 2.021 g/cm3
Young's Modulus = 2.8 GPa
Poisson's Ratio = 0.06
Axial Sress (MPa)
21.0
19.0
17.0
15.0
13.0
11.0
9.0
7.0
5.0
3.0
1.0
-2000
-1000
0
1000
2000
3000
4000
5000
6000
7000
8000
Average Strain (με)
Axial Strain
Lateral Strain

SAMPLE: END09-06-02

Length: 106.33mm

Diameter: 45.13 mm

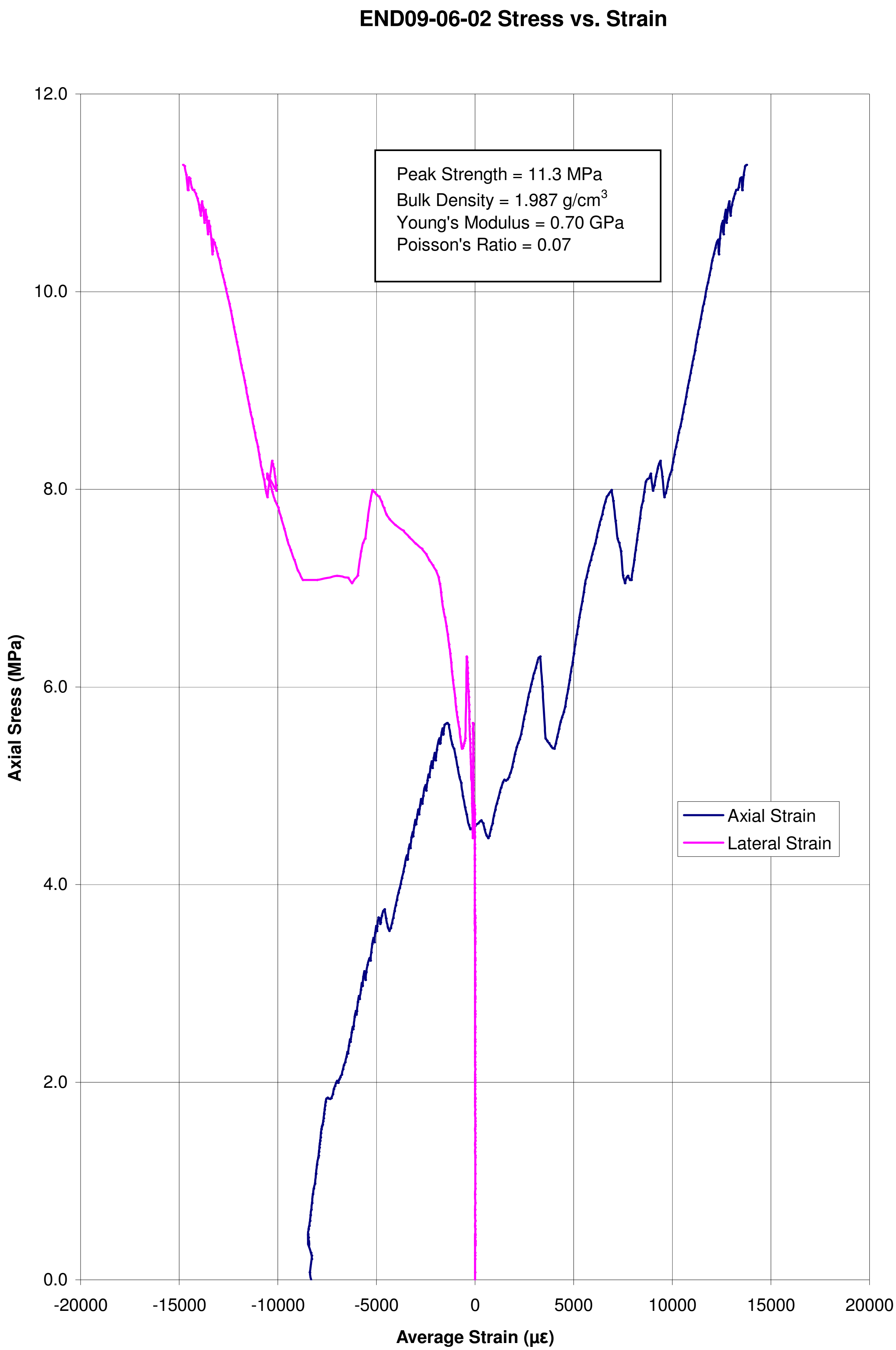
Density: 1.987 g/cm^3

Peak Strength: 11.3 MPa

Young’s Modulus: 0.7 GPa

Poisson’s Ratio: 0.07

Failure Cause: Intact Rock


END09-06-02 Stress vs. Strain
Peak Strength = 11.3 MPa
Bulk Density = 1.987 g/cm3
Young's Modulus = 0.70 GPa
Poisson's Ratio = 0.07
Axial Strain
Lateral Strain
Axial Sress (MPa)
Average Strain (με)
12.0
10.0
8.0
6.0
4.0
2.0
0.0
-20000
-15000
-10000
-5000
0
5000
10000
15000
20000

SAMPLE: END09-06-03

Length: 109.25mm

Diameter: 45.12 mm

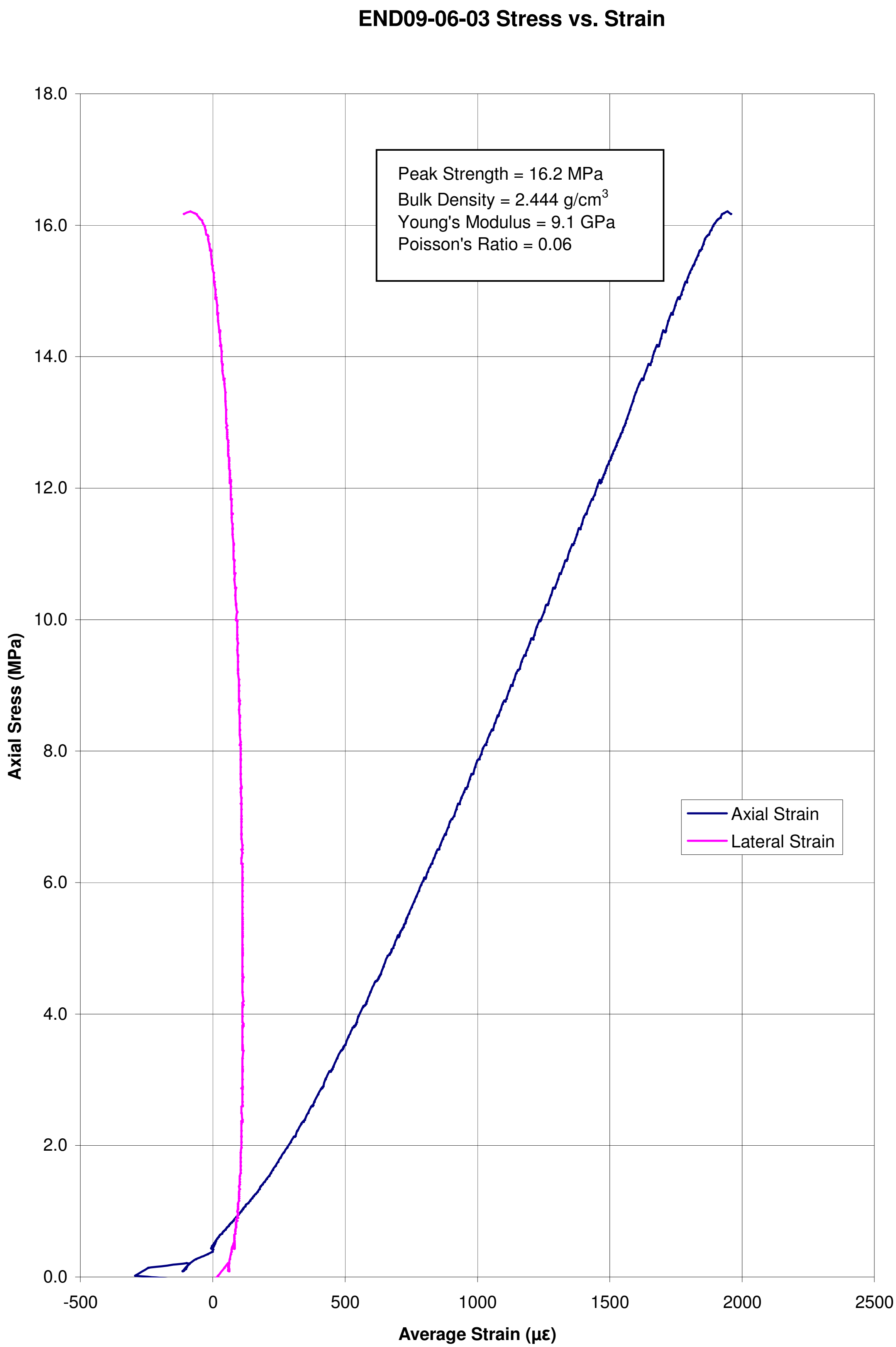
Density: 2.444 g/cm^3

Peak Strength: 16.2 MPa

Young's Modulus: 9.1 GPa

Poisson's Ratio: 0.06

Failure Cause: Weakness Plane



END09-06-03 Stress vs. Strain
Peak Strength = 16.2 MPa
Bulk Density = 2.444 g/cm3
Young's Modulus = 9.1 GPa
Poisson's Ratio = 0.06
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
-500
0
500
1000
1500
2000
2500

SAMPLE: END09-06-04

Length: 93.83 mm

Diameter: 45.11 mm

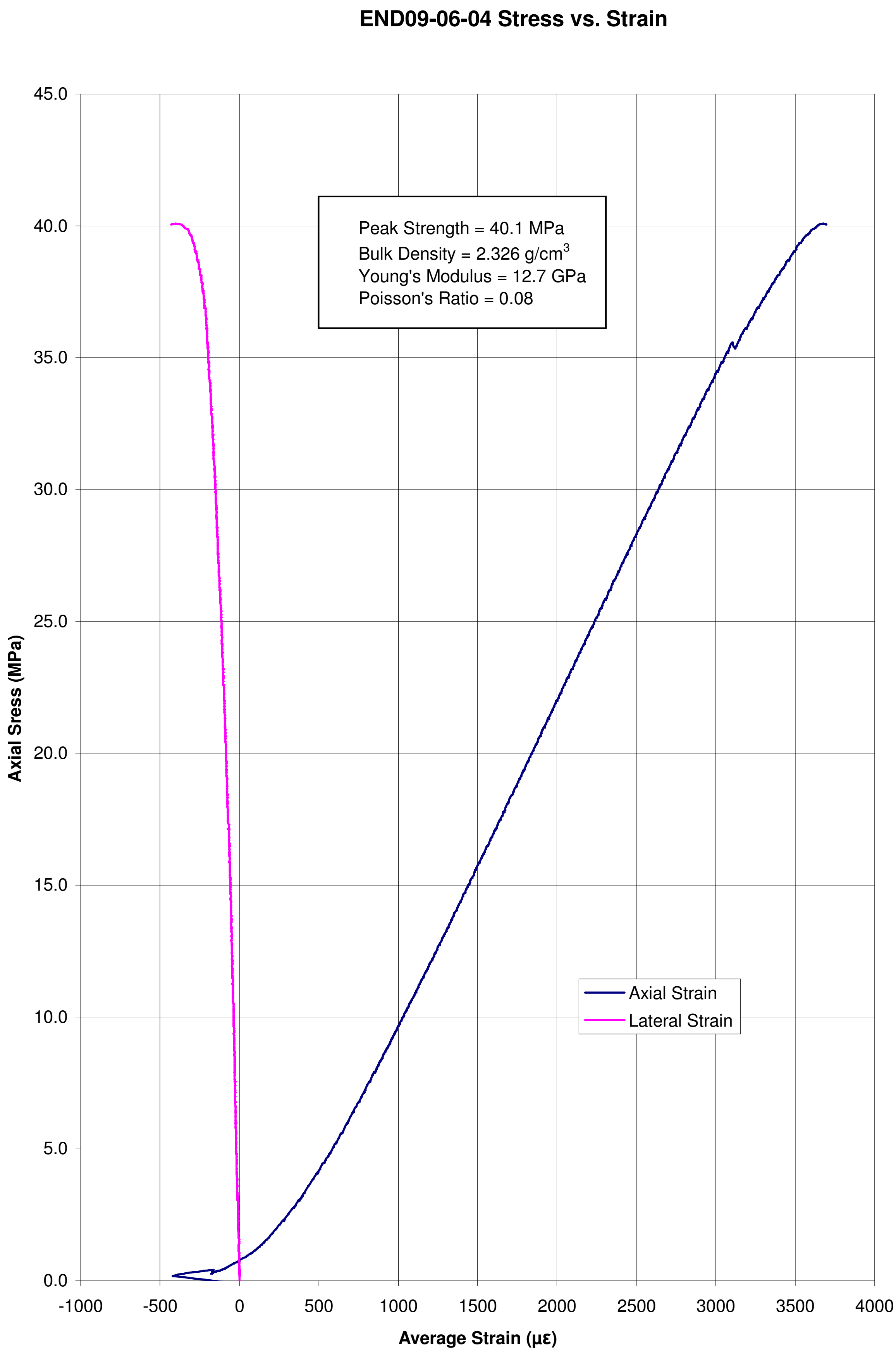
Density: 2.326 g/cm^3

Peak Strength: 40.1 MPa

Young's Modulus: 12.7 GPa

Poisson's Ratio: 0.08

Failure Cause: Foliation


END09-06-04 Stress vs. Strain
Peak Strength = 40.1 MPa
Bulk Density = 2.326 g/cm3
Young's Modulus = 12.7 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
3500
4000

SAMPLE: END09-07-01

Length: 91.01 mm

Diameter: 47.60 mm

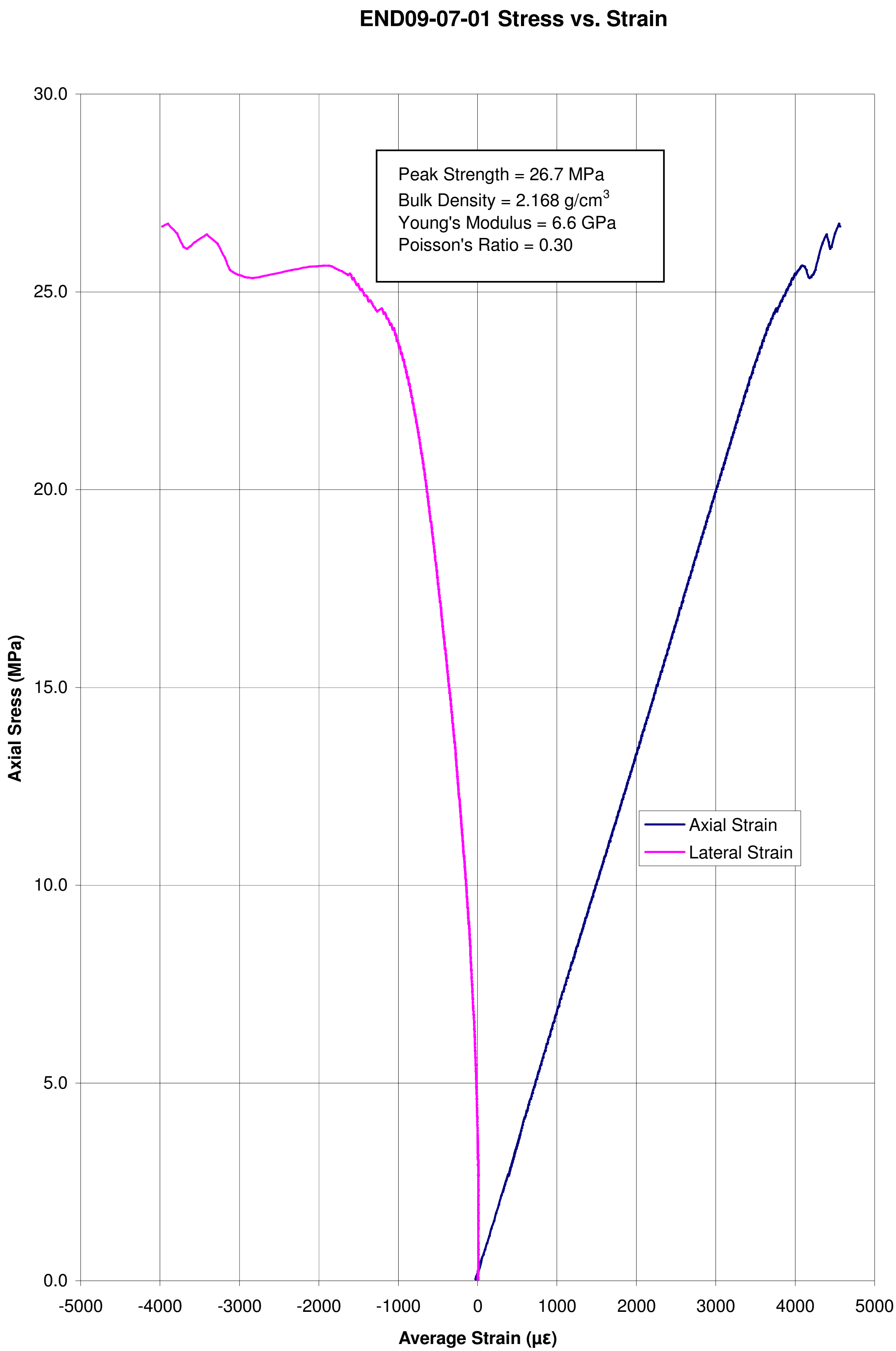
Density: 2.168 g/cm^3

Peak Strength: 26.7 MPa

Young's Modulus: 6.6 GPa

Poisson's Ratio: 0.30

Failure Cause: Intact Rock



END09-07-01 Stress vs. Strain
Peak Strength = 26.7 MPa
Bulk Density = 2.168 g/cm^3
Young's Modulus = 6.6 GPa
Poisson's Ratio = 0.30
Axial Strain
Lateral Strain
Axial Sress (MPa)
Average Strain (με)
30.0
25.0
20.0
15.0
10.0
5.0
0.0
-5000
-4000
-3000
-2000
-1000
0
1000
2000
3000
4000
5000