

REPORT

Mine Operations Plan

Coffee Gold Mine

Submitted By:

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REVISION TRACKING LOG

Revision Tracking Log			
Version	Date	Section Updated	Description of Update
0	November 2023	-	First submission of the Mine Operations Plan
1	October 2024	Various	Updated in response to QML IR1.
2			

TABLE OF CONTENTS

LIST OF ACRONYMS AND ABBREVIATIONS.....	V
LIST OF SYMBOLS AND UNITS OF MEASURE.....	VI
INFORMATION REQUIREMENTS FOR QUARTZ MINE LICENSE AND WATER LICENSE	VII
1.0 INTRODUCTION.....	1
1.1 Project Summary.....	1
1.2 Scope and Objectives	4
1.3 Incorporation of Traditional Knowledge and Consultation Feedback	4
1.4 Synergies with Newmont Standards and Requirements and Other Project Documents	4
1.5 Roles and Responsibilities of Key Personnel	6
2.0 APPLICABLE LEGISLATION FOR MINE OPERATIONS	7
3.0 DESIGN BASIS AND CRITERIA	7
3.1 Geology	7
3.2 Open Pit Optimization	9
3.3 Geotechnical Assessment	11
3.3.1 Wall Design and Overburden Stability	11
3.3.2 Seismic Design Evaluation	15
4.0 OPEN PIT MINING DESIGN AND METHODS	16
4.1 Mine Design	16
4.2 Material Release Schedule	30
4.3 Blasting and Wall Control.....	35
4.4 Pit Dewatering.....	36
4.5 In-pit Haul Road Design.....	37
4.6 Back Haul of Waste Rock	38
5.0 REFERENCES.....	39

LIST OF TABLES (WITHIN TEXT)

Table 1-1	Newmont Global Standards	5
Table 1-2	Key Project Personnel	6
Table 3-1	Open Pit Optimization Input Parameters	9
Table 3-2	Coffee Gold Open Pit Mineralized Material Parameters.....	10
Table 3-3	Coffee Gold Mineralized Material by Oxidation Type Parameters.....	10
Table 3-4	Summary of Rock Mass Characteristics	11

Table 4-1	Open Pit Dimensions and Life of Mine Quantities of Contained Mineralized Material and Waste Rock	16
Table 4-2	Approximate Annual Material Mined from the Open Pits	32
Table 4-3	Annual Heap Leach Feed Source and Run-of-mine Stockpile Balance	34
Table 4-4	Typical Blasthole Loading by Material	35
Table 4-5	In-Pit Haul Road Design Parameters	37

LIST OF FIGURES (WITHIN TEXT)

Figure 1-1	Coffee Project Location and Northern Access Route	2
Figure 1-2	General Arrangement - Mine Site	3
Figure 3-1	Distribution of Mineral Domains	8
Figure 3-2	Pit Slope Design Components	12
Figure 3-3	Pit Slope Design Recommendations	14
Figure 4-1	Double Double Shell	17
Figure 4-2	Double Double Cross Section	18
Figure 4-3	Double Double Pit Detail	19
Figure 4-4	Latte Phase 01 East and West Shells.....	20
Figure 4-5	Latte Phase 01 West Cross Section	21
Figure 4-6	Latte Pit Detail	22
Figure 4-7	Kona Shell	23
Figure 4-8	Kona Cross Section	24
Figure 4-9	Kona Pit Detail	25
Figure 4-10	Supremo Phase 01, 02A, 02B, and 03 Shells	26
Figure 4-11	Supremo Phase 01 Cross Section	26
Figure 4-12	Supremo Phase 02 Cross Section	27
Figure 4-13	Supremo Phase 04A, 04B, and 05 Shells	27
Figure 4-14	Supremo Phase 04A and 05 Cross Section	28
Figure 4-15	Supremo Pit Detail	29
Figure 4-16	Ore and Waste Rock Mining Schedule by Pit Parameters	31
Figure 4-17	Direct Feed and Stockpiled Ore Schedule Parameters	31
Figure 4-18	Typical Bench Loading Configurations	36

APPENDICES

Appendix A	Mine Operations Plan Table of Concordance
Appendix B	Feasibility Pit Slope Stability Update Coffee Gold Project Yukon Territory, Canada
Appendix C	Seismic Evaluation Report

LIST OF ACRONYMS AND ABBREVIATIONS

Acronym / Abbreviation	Definition
AN	ammonium nitrate
ANFO	Ammonium nitrate/fuel oil
H&S	Health & Safety
HLF	heap leach facility
ISA	inter-ramp slope angle
JDS	JDS Energy and Mining
LG	Lerchs-Grossman
LOM	Life of Mine
NAR	Northern Access Route
OSA	overall slope angle
Project	Coffee Gold Mine
QA/QC	quality assurance and quality control
QPs	qualified professionals
RMR	rock mass rating
ROM	run of mine
RQD	rock quality design
SER	Sustainability and External Relations
TDG	Transportation of Dangerous Goods
UCS	unconfined compressive strength
WROMP	Waste Rock and Overburden Management Plan
WRSF	Waste Rock Storage Facility

LIST OF SYMBOLS AND UNITS OF MEASURE

Symbol / Unit of Measure	Definition
%	percent
°	degree (angle)
Au	gold
g	gram
<i>g</i>	gravity acceleration
g/t	grams per tonne
ha	hectares
kg/t	kilogram per tonne
km	kilometre
km ²	square kilometres
koz	thousand ounces
kt	kilotonne
L	litre
m	metre
m ²	square metres
m ³	cubic metre
mm	millimetre
MPa	megapascal
Mt	million tonnes
Mt/a	million tonnes per annum
oz	ounce
t	tonne
t/d	tonnes per day
t/m	tonnes per meter
t:t	ratio of tonnes of waste rock to tonnes of ore

INFORMATION REQUIREMENTS FOR QUARTZ MINE LICENSE AND WATER LICENSE

Information Requirement	Location in this Plan
Table of Concordance	Appendix A: Mine Operations Plan Table of Concordance
Revision Log	Page i
Provide the design criteria that guide the open pit and/or underground design including project constraints, regulatory and guidance-based criteria, and other criteria.	Section 3.0: Design basis and Criteria
Provide an overview of the geology of the deposit and the ore zones. Include ore reserves estimate, the planned annual mining rate, and the mine life based on the reserves and annual mining rate. Provide an outline of ore tonnages and grades, daily mining rate, and planned monthly mining rates with diluted averages.	Section 3.0: Design basis and Criteria; Section 4.2: Material Release Schedule
Provide a geotechnical assessment that describes the competency of the rock and describe the methods that were used to determine the rock mass quality on site rock masses on site, and a summary of the typical rock mass qualities for all ore bodies and zones. Provide the point load strength index for the different rock types. Describe seismic design events, factors of safety, and slope angles.	Section 0: Geotechnical Assessment; General Mine Development Plan; Waste Rock and Overburden Management Plan.
Describe site clearing, stripping, grubbing and foundation preparation. Include areas to be cleared and volumes of material removed. Identify storage locations for waste materials generated from construction activities. For foundation preparation, describe how unsuitable in-situ materials, such as ice-rich permafrost, soft, or weak materials will be identified and addressed.	General Mine Development Plan; Waste and Hazardous Materials Management Plan; Waste Rock and Overburden Management Plan
Describe the construction Quality Assurance/Quality Control (QA/QC) program that will be implemented to ensure that construction activities will achieve expected performance. Also describe how the construction quality assurance/quality control program will ensure appropriate implementation of construction constraints related to climatic conditions like wet, dry and freezing conditions.	General Mine Development Plan
Describe the results of stability and settlement analyses for proposed facilities.	Section 3.3.1: Wall Design and Overburden Stability; General Mine Development Plan;
Describe the construction schedule, addressing proposed construction phases, and also seasonal scheduling. Identify schedule constraints related to climatic conditions.	General Mine Development Plan
Provide a material release schedule for all open pits and underground workings.	Section 4.2: Material Release Schedule
Provide a description of how ore will be handled on site, including location and design of stockpiles, segregation protocols, stockpile inventories, and projected life of stockpiles.	Section 4.1: Mine Design; Section 4.2: Material Release Schedule; Heap Leach and Process Facilities Plan; Waste Rock and Overburden Management Plan.
Provide a description of the crushing process, including how ore is fed into the crusher(s), the capacity of the crusher(s), the components and equipment involved in each crushing circuit, the crush size achieved, and any dust suppression or collection systems that will be employed.	Heap Leach and Process Facilities Plan

Information Requirement	Location in this Plan
Describe the power plant facilities, the generation capacity, the capacity that it will be run at on a daily basis and any backup facilities available. Provide layout drawings, issued for construction drawings, and electrical diagrams.	General Mine Development Plan
Provide a description and diagram showing all infrastructure associated with the mine.	General Mine Development Plan
Include a singleline diagram of the power facilities on site, stamped by a professional engineer.	General Mine Development Plan
Provide a brief description of the communication capabilities on site, and if compressed air will be utilized.	General Mine Development Plan
Provide a brief description of the waste rock and high grade and low-grade ore storage locations and the segregation protocols. Describe any dynamic stockpiles that will be on site, and the timing of these facilities.	Section 4.1: Mine Design; Heap Leach Facility Operations, Maintenance, and Surveillance Manual; Waste Rock and Overburden Management Plan.
Include a description of any additional areas that will serve as staging areas for equipment and any buildings that will be used to house or service these pieces of equipment.	General Mine Development Plan
Provide the numbers of gasoline and diesel fuel tanks required, the capacity of each tank, and the storage location. Include a fuel storage site layout diagram. Include the number of propane tanks required, the capacity of each tank, the storage location(s), the quantities of propane required for major mine components, and how long the quantities will last during winter operations.	General Mine Development Plan
Provide the detailed designs for the open pit; include the final pit walls, the bench spacing, depth of final pit, and access roads. A detailed schematic of the starter pit, pit stages, and final pit designs must be included.	Section 4.0: Open Pit Mining Design and Methods
Describe how the wall design considers overburden stability, include the factor of safety and the probability of failure. Provide the bench sizes that will be utilized to ensure the pit walls will remain stable throughout pit development.	Section 3.3.1: Wall Design and Overburden Stability
Provide a detailed description of the monitoring program that will be used to assess the pit wall stability. Include a list of all instrumentation equipment, where it will be installed, and the frequency of monitoring. Describe both the instrumental monitoring and visual observations that will take place.	EMAMP: Physical Monitoring Plan
For each open pit provide the blasting procedures, the bench height ratio, and the wall stabilization practices that will be used. Include diagrams of the typical bench loading configurations.	Section 4.3 Blasting and Wall Control
Include the design parameters for haul roads within the pit including road width, safety berms, and emergency routes. Describe the procedures for operating vehicles and equipment within the pit walls and outline the vehicle passing procedures and determination of right of way.	Section 4.5 In-pit Haul Road Design

1.0 INTRODUCTION

1.1 Project Summary

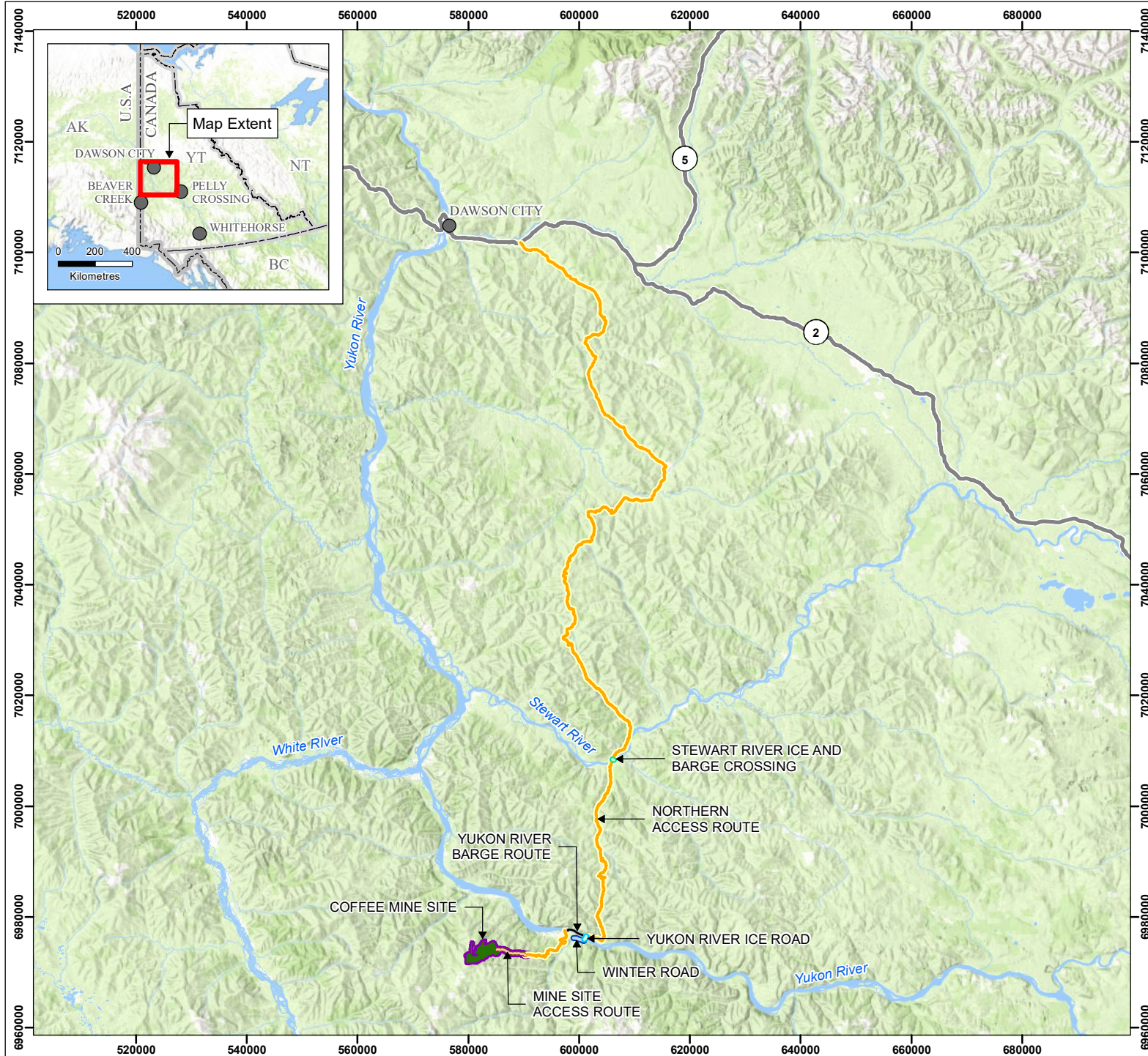
The proposed Coffee Gold Mine (the Project) is an open-pit gold mine owned by Goldcorp Kaminak Ltd., a wholly owned subsidiary of Newmont Corporation (Newmont), located on the south side of the Yukon River in the White Gold District of west-central Yukon. The Project site is approximately 130 km south of the City of Dawson, 140 km west of Pelly Crossing, 95 km north-east of Beaver Creek, and 340 km northwest of Whitehorse. The Project is located wholly within the traditional territory of Tr'ondëk Hwëch'in, partially within the traditional territory of Selkirk First Nation and First Nation of Na-cho Nyäk Dun, and partially within the asserted territory of White River First Nation. The Project contains several gold occurrences within an exploration concession covering an area of more than 600 km². The Mine Site will be accessed by road from Dawson via a 16-km stretch of Klondike Highway and 192-km all-season road, referred to as the Northern Access Route (NAR) (Figure 1-1). The NAR includes seasonal barge crossings on both the Stewart and Yukon rivers, with ice bridges and a seasonal winter road in the winter months.

The Project is comprised of four open pits: Supremo, Latte, Double Double, and Kona. Waste rock is proposed to be permanently stored in the Alpha Waste Rock Storage Facility (WRSF) (Figure 1-2). The ore production rate is proposed to be up to approximately 9.0 million tonnes (Mt) per year, producing an estimated total of 67 Mt of heap leach feed over the 10-year Operation Phase. The conceptual-level estimate for waste material to be moved over the life of mine (LOM) is up to approximately 330 Mt based on an average strip ratio of 5.0:1. The ore will be crushed and transported to the Heap Leach Facility (HLF) via overland conveyor or trucks for nine months of the year. During the three coldest months of winter, run-of-mine (ROM) ore will be stockpiled in the ROM stockpile. Gold will be extracted from gold-bearing leach solution by a six tonnes per day (t/d) adsorption, desorption, recovery carbon plant with mercury retorting to produce a final gold doré product. A total of 2.6 million ounces of gold is planned to be recovered over a 10-year mine life.

The Project phases are defined as follows:

- Construction Phase: Q2 Year –3 to end of Year –1 (30 months)
- Operation Phase: Year –1 to end of Year 9 (10 years)
- Reclamation and Closure Phase: Year 10 to end of Year 21, including a 6-year Post-Mining Closure Stage and a 5-year Active Closure Stage (11 years)
- Post-Closure Phase: Year 21 onwards as determined to be required.

These phases broadly describe the activities occurring within a particular time period; however, some activities will continue from one phase to another as mine site development advances with operational activities (e.g., Open Pits, WRSF). When areas that support mine operations are no longer required, they will be progressively reclaimed. The overall Project schedule is the general expected scenario for mine construction and operation; detailed activities are subject to change depending on detailed mine planning and the timing of receipt of authorizations.



COFFEE GOLD MINE

Coffee Project Location and Northern Access Route

Legend

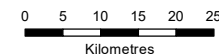
- Stewart River Ice and Barge Crossing
- Yukon River Barge Route
- Yukon River Ice Road
- Winter Road
- Mine Site Access Route
- Northern Access Route
- Project Area
- Project Footprint
- Highway
- Waterbody

Notes

1. This map is not intended to be a "stand-alone" document, but a visual aid of the information contained within the referenced Report. It is intended to be used in conjunction with the scope of services and limitations described therein.
2. Contains information licensed under the Open Government Licence - Yukon Territory
3. Basemap: ESRI World Topographic Map
4. Inset Basemap: ESRI World Topographic Map



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NAD 1983 UTM Zone 7N

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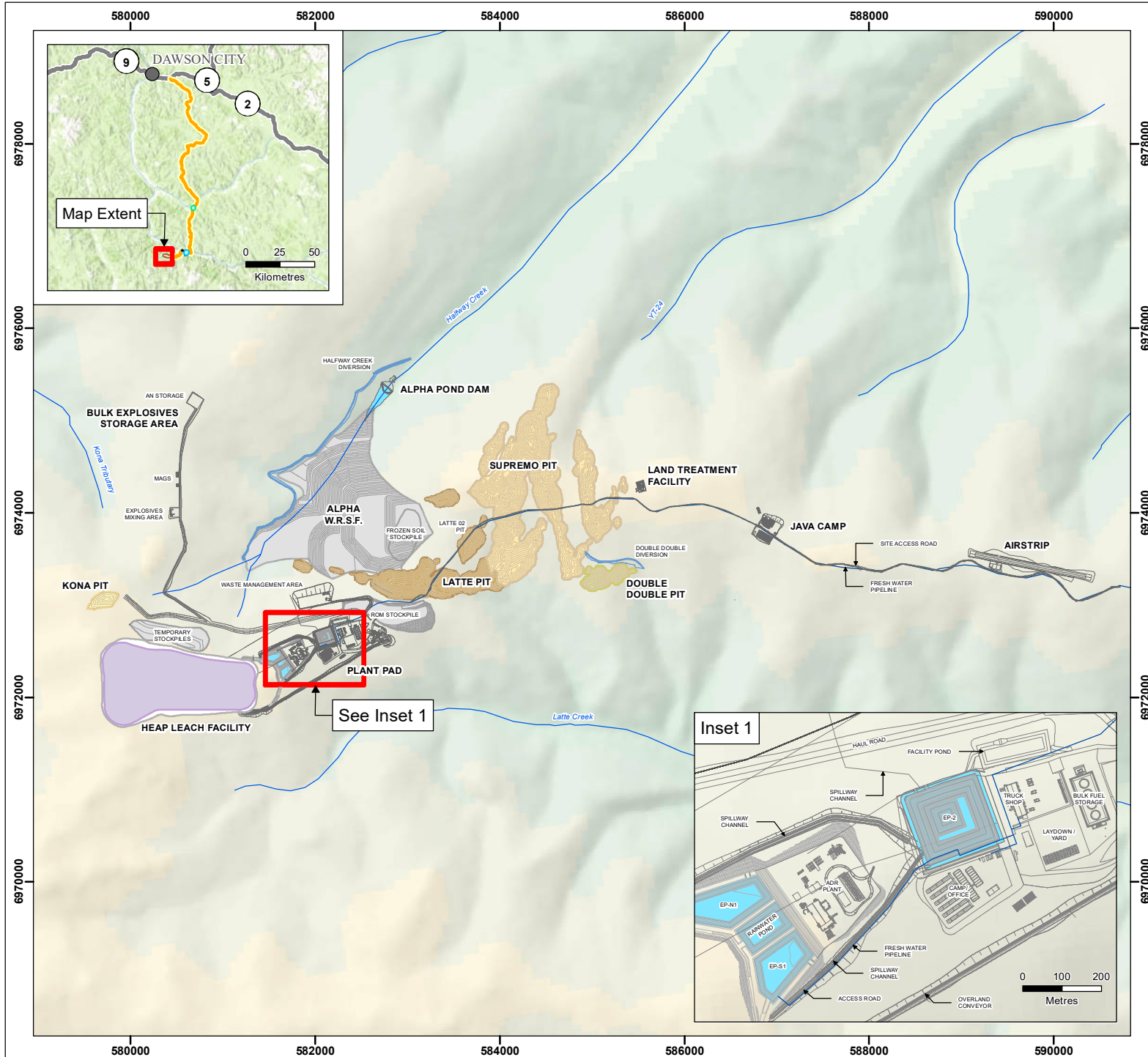
Figure 1-1

Date:
Mar 2, 2023

Drawn by:
AS

Reviewed:
KP

Newmont
COFFEE



COFFEE GOLD MINE

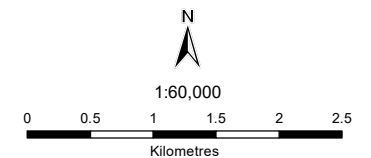
General Arrangement - Mine Site

Legend

- Double Double Pit
- Kona Pit
- Latte Pit
- Supremo Pit

Notes

1. This map is not intended to be a "stand-alone" document, but a visual aid of the information contained within the referenced Report. It is intended to be used in conjunction with the scope of services and limitations described therein.
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3. Inset Basemap: ESRI World Topographic Map



NAD 1983 UTM Zone 7N

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Figure 1-2

Date:
Oct 6, 2023

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AS

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CH

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1.2 Scope and Objectives

This Mine Operations Plan (Plan) describes the design criteria, open pit mine design, material scheduling, and general operations of the four open pits at the Project. This Plan was developed as a living document that may be updated as the Project advances. Construction and site preparation protocols for development of ancillary infrastructure are provided in the General Mine Development Plan.

This Plan is provided in conjunction with the General Mine Development Plan and other management and monitoring plans to meet the requirements for a Mine Development and Operations Plan provided in the Plan Requirement Guidance for Quartz Mining Projects (YWB and EMR 2013). This Plan is also provided in conjunction with other plans to meet the requirements for quartz mine applications for a Quartz Mining License and a Water Use Licence under the *Quartz Mine Act* and the *Waters Act*, respectively. The requirements along with corresponding sections and/or plans where information is provided are outlined in the Information Requirements for Quartz Mine License and Water Use License Table above. Other applicable regulations related to mine development are detailed in Section 2.0.

Commitments that were made during the YESAB screening process related to the Mine Operations Plan are incorporated into the plan where possible and are summarized in the Mine Operations Plan Table of Concordance (Appendix A). The Table of Concordance includes commitments made during the assessment process only, as there were no Decision Document conditions related to mine operations at the site.

1.3 Incorporation of Traditional Knowledge and Consultation Feedback

First Nation engagement and consultation is ongoing, and any feedback that is received or concerns that are heard during consultation regarding general operations of the four open pits will be incorporated into this plan, where applicable.

1.4 Synergies with Newmont Standards and Requirements and Other Project Documents

As an important part of Newmont's internal governance process, Newmont has implemented Global Policies and Standards (Global Standards) that are reviewed and preliminarily approved by a Global Policies & Standards Committee. The Global Standards provides the framework and standards for Newmont sustainability management and ensures a consistency of approach for implementing these global policies across the Company.

The Global Standards apply to all directors, officers and employees of Newmont Corporation, its subsidiaries, and any other entities that it controls. A variance request process for existing or future conditions is in place. The process provides an alternative mechanism for those instances where a Newmont site/operation cannot logistically or feasibly conform to a requirement established in a Standard due to special conditions or unique hardships. The Global Standards are intended to be fully integrated into all core business functions, and they emphasize sustainability, responsibility, and accountability at all organizational levels.

The list of Global Standards is provided in Table 1-1.

Table 1-1 Newmont Global Standards

Global Standards	
Air Emissions Management Standard	Land Acquisition and Involuntary Resettlement Standard
Biodiversity Management Standard	Local Procurement and Employment Standard
Closure and Reclamation Management Standard	People Policy
Code of Conduct	Product Stewardship Standard
Community Investment and Development Standard	Social Baseline and Impact Assessment Standard
Cultural Resource and Management Standard	Stakeholder Relationship Management Standard
Drug and Alcohol Policy (Coffee-Specific)	Tailings Storage Facility and Heap Leach Facility Environmental Standard
Hazardous Materials Management Standard	Tailings Storage Facilities Technical and Operations Standard
Health and Safety Policy	Waste Management Standard
Human Resources Standard	Waste Rock and Ore Stockpile Management Standard
Human Rights Standard	Water Management Standard
Indigenous Peoples Standard	

While this Plan can be reviewed in isolation to inform the specific actions for mine operations related to the open pits, this plan should be viewed in concert with the following additional management plans in order to attain a holistic understanding of the Project:

- General Mine Development Plan
- Explosives Management Plan
- Spill Contingency Plan
- Waste Rock and Overburden Management Plan
- Heap Leach and Process Facilities Plan
- Water Management Plan
- Physical Monitoring Plan.

This plan includes operational information related to open pit development. Information related to mine infrastructure development, site preparation activities, potable and fire water storage, and material availability, is provided in the General Mine Development Plan. Information related to waste rock and overburden management and design criteria is provided in the Waste Rock and Overburden Management Plan. Site water management infrastructure, including the Alpha and Facility sedimentation ponds and conveyance structures, is described in the Water Management Plan. Explosives storage and handling procedures are described in the Explosives Management Plan.

1.5 Roles and Responsibilities of Key Personnel

Newmont has committed to providing the necessary human, material, and financial resources to implement and maintain the Mine Operations Plan. The Mine General Manager will be responsible for overseeing the requirements outlined in these plans, which includes hiring qualified professionals (QPs) to implement the responsibilities outlined in Table 1-2. Key Project personnel responsible for being familiar with the contents of this Plan are shown in Table 1-2.

Table 1-2 Key Project Personnel

Role	Responsibility
Mine General Manager	Overall responsibility for Mine Site management.
Operations Manager	Responsible for mine planning and production, mine technical monitoring, and mine regulatory compliance.
Health and Safety (H&S) Manager	Responsible for conducting regular safety site inspections and implementing the appropriate controls in a timely manner. The H&S Manager shall maintain records of all safety inspections as well as any actions taken because of these inspections throughout the life of the Project. Where safety inspections show the potential for environmental effects, the H&S Manager will work in collaboration with the Environment Department.
Sustainability and External Relations (SER)	Responsible for recording and addressing any complaints received from nearby land users, or other interested parties regarding mine operations-related impacts off-site. The SER department shall maintain records of all complaints received as well as any actions taken because of these complaints.
Environmental Manager	Responsible for permitting, environmental monitoring, and regulatory compliance.
Pit Superintendent	Responsible for all operational aspects of the mine.
Mining Engineer	Mine design and its operation
Explosives Contractor	Responsible for supplying, transporting, and storing blasting agents; manufacturing explosive products on-site; delivering explosives products to blast sites; and inspections of explosives blend trucks and parking garage.
Drill and Blast Supervisor	Responsible for security and safety during a blast; all blasting activities and explosives, including charging the holes; and logging any misfires, cut-offs, and observations.
Blaster	Responsible for maintaining complete control of the blast site; sign in and sign out procedures for blasting accessories from the magazines; accounting for blasting accessories; and acknowledging and recording ANFO or emulsion delivered to the blast site.

2.0 APPLICABLE LEGISLATION FOR MINE OPERATIONS

Development of a mining property to support operational mining is governed by federal and territorial legislation, including but not limited to the following:

Federal Legislation

- *Explosives Act* (RSC 1985 c.E-17)
- *Fisheries Act* (RSC 1985, c. F-14)
- *Transportation of Dangerous Goods Act*, 1992 (SC 1992, c.34).

Yukon Legislation

- *Dangerous Goods Transportation Act* (RSY 2002, c.50)
- *Environment Act* (RSY, c. 76)
- *Quartz Mining Act* (SY 2003, c.14)
- *Waters Act* (SY 2003, c. 19)
- *Territorial Lands (Yukon) Act* (SY 2003, c.17)
- *Workers' Safety and Compensation Act* (SY 2021, c. 11)).

3.0 DESIGN BASIS AND CRITERIA

3.1 Geology

Gold mineralization is located within a series of steeply dipping structures that crosscut all rock units on the property. The structural zones are identified in drill core and from surface mapping and trenching. These structural zones may exhibit faulting, brecciation, silicification, alteration, and local sulphide veining, and can be traced over strike lengths of up to 3.5 km. Individual mineralized structures exhibit localized flexures, anastomosing patterns, and pinch and swell geometries over scales of tens to hundreds of meters.

The mineralized structures have undergone extensive preferential weathering and oxidation of iron-bearing minerals as a result of meteoric fluids percolating from surface downwards through the permeable structural corridors. It is common to find intense oxidation at depths below 200 meters (m) from surface within these structures. Conversely, unfractured and unaltered country rock is typically fresh (unoxidized) at surface. The breakdown of the iron-bearing minerals such as arsenian pyrite to Fe-oxides such as limonite and hematite resulted in the liberation of gold, which had been structurally bound within the arsenian pyrite. As a result of the fine-grained nature of the formally refractory gold, rapid and complete leaching by cyanide is possible.

The mineralized zones have been grouped into several deposits, four of which comprise the Project reserves, namely Supremo, Latte, Double Double, and Kona (Figure 3-1).

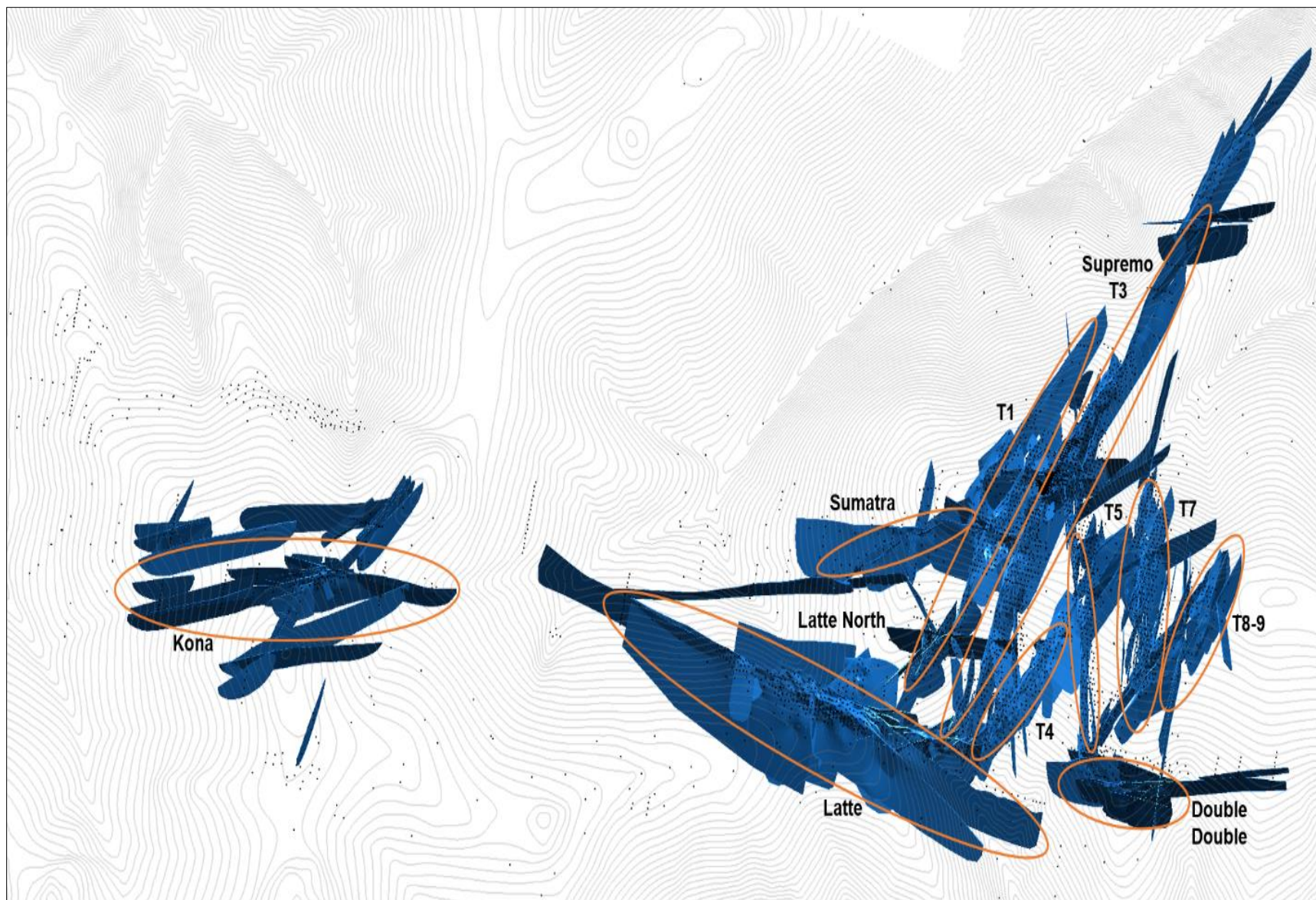


Figure 3-1 Distribution of Mineral Domains

3.2 Open Pit Optimization

The depth and extents of the various open pits were determined by a Lerchs-Grossman (LG) pit optimization of the Project's resource model. Key input parameters for the LG runs are provided in Table 3-1. The optimum pit shell for each deposit was selected as the basis for the pit designs. The details of the final open pit designs are provided in Section 4.1. Mineral resource estimates for each deposit are summarized in Table 3-2 and broken out by oxidation type in Table 3-3.

The open pit designs presented here may be further refined based on the actual geological conditions at the site. As such, additional pit shell optimizations may be considered and may result in modifications to the current pit shell shapes. These optimizations and potential modifications from the current pit design will consider any resulting impacts to long-term flows and water quality in refinement of the final pit design.

Table 3-1 Open Pit Optimization Input Parameters

Parameter	Unit	Value
Recovery and Dilution		
External Mining Dilution – Supremo	%	0
External Mining Dilution – Double Double	%	0
External Mining Dilution – Kona	%	0
External Mining Dilution – Latte	%	0
Mining Recovery – All	%	100
Gold Recovery		
Oxide Leach Gold Recovery – Supremo	%	88
Oxide Leach Gold Recovery – Double Double	%	90
Oxide Leach Gold Recovery – Kona	%	83
Oxide Leach Gold Recovery – Latte	%	88
Upper Transition Leach Gold Recovery – Supremo	%	80
Upper Transition Leach Gold Recovery – Double Double	%	75
Upper Transition Leach Gold Recovery – Kona	%	67
Upper Transition Leach Gold Recovery – Latte	%	77
Middle Transition Leach Gold Recovery – Supremo	%	55
Middle Transition Leach Gold Recovery – Double Double	%	55
Middle Transition Leach Gold Recovery – Kona	%	47
Middle Transition Leach Gold Recovery – Latte	%	58
Lower Transition Leach Gold Recovery – Supremo	%	25
Lower Transition Leach Gold Recovery – Double Double	%	25
Lower Transition Leach Gold Recovery – Kona	%	16
Lower Transition Leach Gold Recovery – Latte	%	25
Cut-off Grade Calculations		
Oxide Gold Cut-off Grade – Supremo	g/t Au	0.27
Oxide Gold Cut-off Grade – Double Double	g/t Au	0.25
Oxide Gold Cut-off Grade – Kona	g/t Au	0.26
Oxide Gold Cut-off Grade – Latte	g/t Au	0.24

Parameter	Unit	Value
Upper Transition Gold Cut-off Grade – Supremo	g/t Au	0.29
Upper Transition Gold Cut-off Grade – Double Double	g/t Au	0.30
Upper Transition Gold Cut-off Grade – Kona	g/t Au	0.32
Upper Transition Gold Cut-off Grade – Latte	g/t Au	0.27
Middle Transition Gold Cut-off Grade – Supremo	g/t Au	0.43
Middle Transition Gold Cut-off Grade – Double Double	g/t Au	0.41
Middle Transition Gold Cut-off Grade – Kona	g/t Au	0.46
Middle Transition Gold Cut-off Grade – Latte	g/t Au	0.36
Lower Transition Gold Cut-off Grade – Supremo	g/t Au	0.94
Lower Transition Gold Cut-off Grade – Double Double	g/t Au	0.91
Lower Transition Gold Cut-off Grade – Kona	g/t Au	1.35
Lower Transition Gold Cut-off Grade – Latte	g/t Au	0.83
Other		
Overall Pit Slope Angles	degrees	65 - 75
Heap Leach Production Rate	Mt/a	6.0

Source: Newmont 2021

Table 3-2 Coffee Gold Open Pit Mineralized Material Parameters

Deposit	Category	Total Ore ('000's t)	Gold Cut-off Grade (Oxide) (g/t)	Average Gold Grade (g/t)	Contained Gold ('000's oz)
Latte	Resource (M,I&I)	20,962	0.24	1.24	835
Double Double	Resource (M,I&I)	1,340	0.25	2.68	115
Kona	Resource (M,I&I)	946	0.26	1.07	32
Supremo	Resource (M,I&I)	36,902	0.27	1.33	1,583
Total Mineral Resource	Resource (M,I&I)	60,150	0.26	1.33	2,566

Source: Newmont 2021

Table 3-3 Coffee Gold Mineralized Material by Oxidation Type Parameters

Deposit	Oxide Ore ('000's t)	Upper Transitional Ore ('000's t)	Middle Transitional Ore ('000's t)	Lower Transitional Ore ('000's t)	Sulfide Ore ('000's t)
Latte	13,491	4,011	2,186	1,270	4
Double Double	605	521	185	30	0
Kona	791	144	10	0	0
Supremo	34,427	1,725	562	188	0
Total Mineral Resource	49,314	6,401	2,943	1,488	4

Source: Newmont 2021

3.3 Geotechnical Assessment

Field data collection consists of geotechnical core logging and discontinuity orientation, point load testing, and laboratory rock strength testing. Results of the geotechnical characterization program suggest that with the exception of the oxide materials, which will be mostly mined and processed, the rock mass is generally of good geotechnical quality. Table 3-4 contains a summary of rock mass characteristic data derived from the 2015 geotechnical drilling program for each of the three primary lithology types (SRK 2016a). Further geotechnical details for the site are provided in the following reports:

- Fall 2017 Testpitting Program Report (Tetra Tech 2017a)
- Summer 2017 Geotechnical Investigation Data Report (Tetra Tech 2017b)
- Fall 2018 Testpitting Program Reports (Tetra Tech 2018).

Table 3-4 Summary of Rock Mass Characteristics

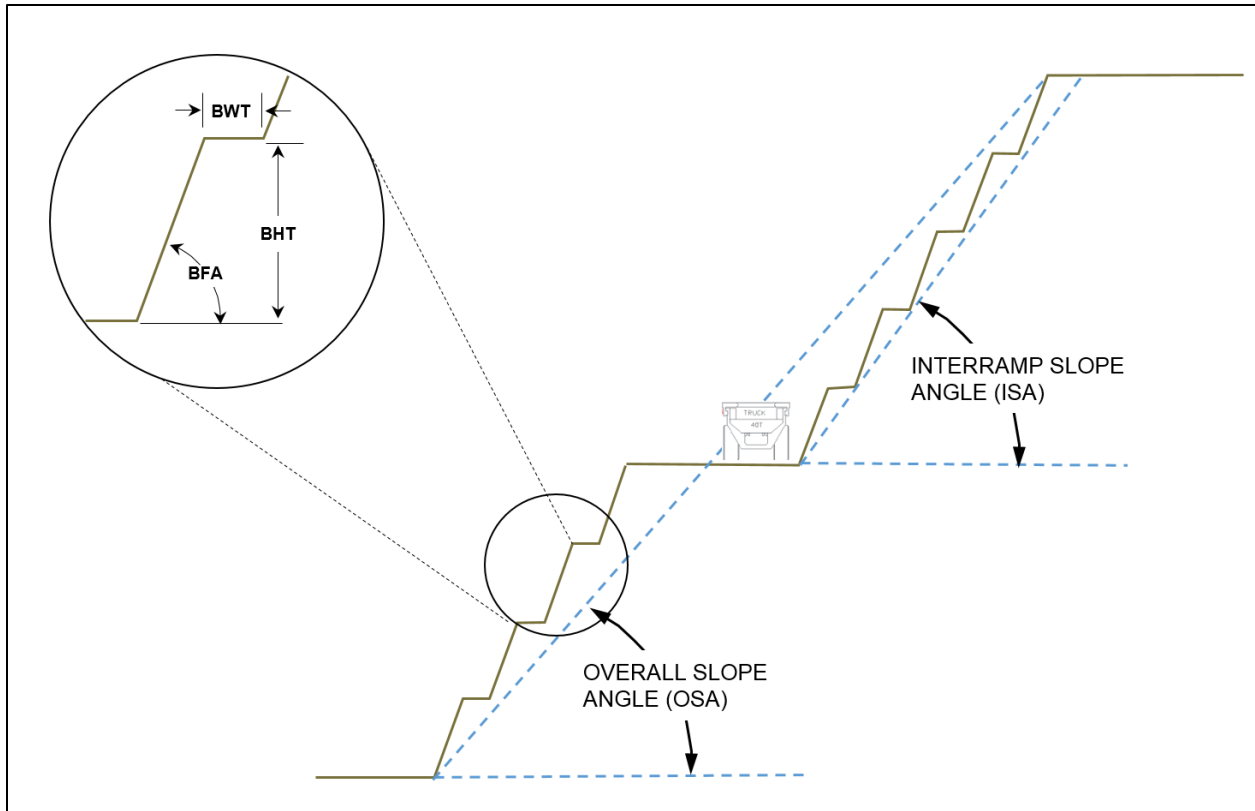
Lithology	Pit	Average UCS (MPa)	No. Valid UCS Tests	Average RMR (1989)	Average RQD (%)	Total Core Length (m)
Gneiss	Supremo and Double Double	90	9	64	81	515
Schist	Latte	94	4	64	87	227
Granite	Kona	130	2	76	95	96

Source: SRK 2016a

UCS: unconfined compressive strength; RMR: rock mass rating; RQD: rock quality design.

3.3.1 Wall Design and Overburden Stability

Slope design involves analysis of the three major components of an open pit slope, i.e., bench configuration, inter-ramp slope angle (ISA) and overall slope angle (OSA) as shown in Figure 3-2. The bench configuration, which is defined by the bench face angle, bench height, and berm width, defines the ISA. The OSA includes inter-ramp slope sections separated by wide step-outs for haul roads, mine infrastructure or geotechnical purposes. The OSA will be approximately equal to the corresponding ISA except in areas where a haul road exists; for more slope stability information, refer to Appendix B - Feasibility Pit Slope Stability Update Coffee Project Yukon Territory, Canada.



Source: SRK 2016a

Figure 3-2 Pit Slope Design Components

Given the overall good rock mass quality anticipated to comprise the pit walls at Coffee Gold, the bench configuration was analyzed first to determine the maximum achievable bench face angle based on geologic structure alone. Bench scale and lower inter-ramp slopes are most realistically assessed using stochastic models that evaluate structurally controlled failure mechanisms. The following was concluded from the analyses:

- Due mostly to the north-south orientation of the pit, benches at Supremo are not anticipated to be substantially affected by structurally controlled instabilities. As such, a maximum achievable bench face angle of 75° was estimated for Supremo based on operational constraints as discussed below.
- The stability of benches on the north Latte and Double Double pit walls will likely be controlled by the dominant southerly (inward) dipping foliation discontinuities. The analyses indicate a maximum achievable bench face angle of approximately 65° for the Latte and Double Double north walls. Stability of the south and end pit wall benches are not anticipated to be governed by geologic structure controls and were therefore estimated to have a maximum achievable bench face of 75° based on operational considerations.
- Benches on the west Kona pit wall have a slightly higher probability of structurally controlled instabilities fostered by a moderately eastward dipping joint set. The analyses indicate a maximum bench face angle of approximately 70° is achievable for this portion of the pit. The remaining pit areas are not anticipated to be substantially affected by geologic structure-controlled instabilities. A maximum achievable bench face of 75° was estimated for the remainder of the pit based on operational constraints.

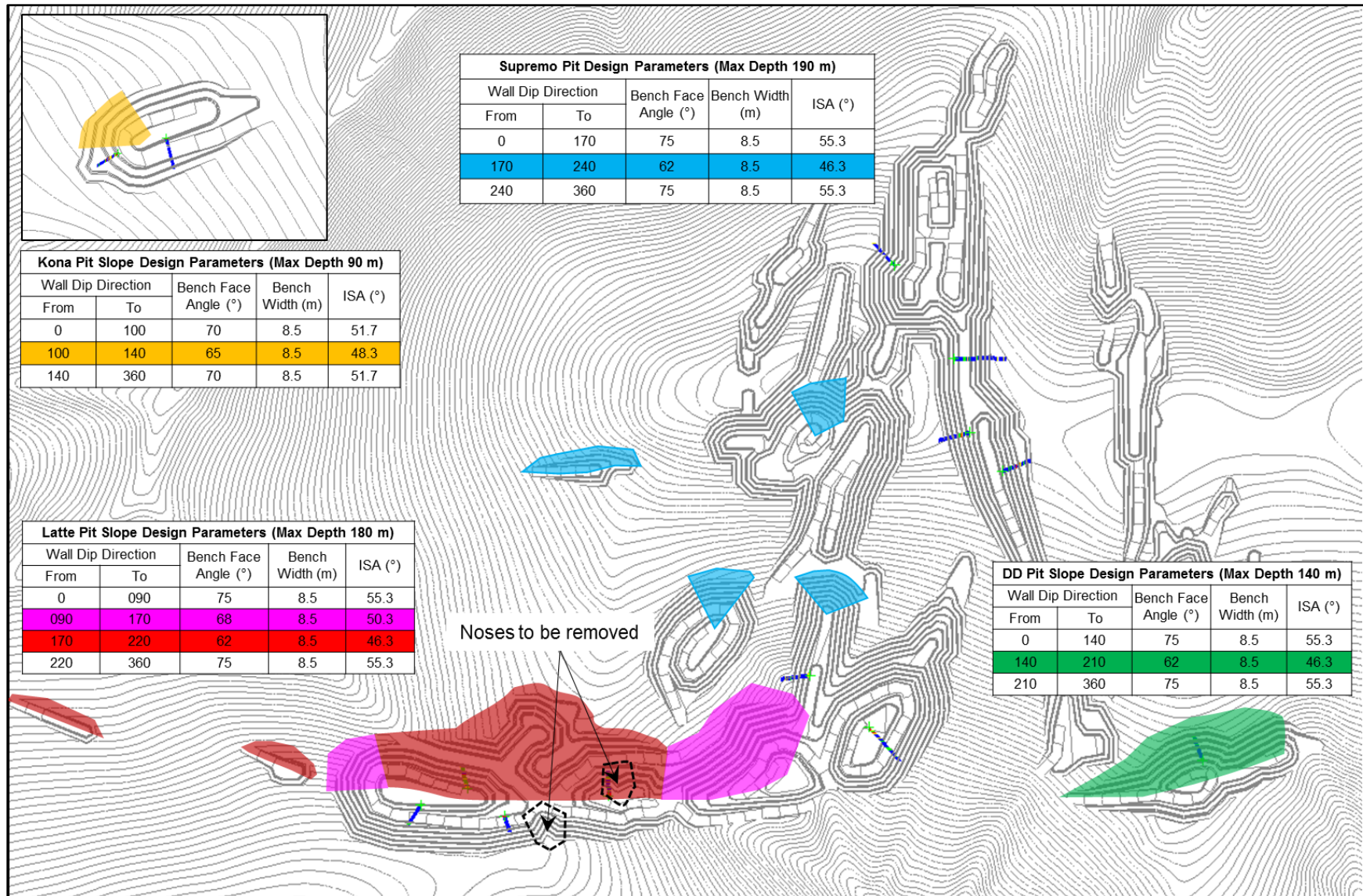
The geotechnical pit slope design is shown in Figure 3-3. These are based on dip direction of the pit wall (e.g., for an east-west trending wall, facing south, the slope dip direction would be 180° azimuth).

Double benching will take place in fresh, competent rock. The double, 20-m-high benching will permit the incorporation of more adequately sized berms for rock-fall control, provided the drilling and, to a greater extent, blasting practices meet best practice standards, thereby reducing the number of crests and toes that are subject to potential damage.

While there are no Yukon regulations regarding pit slope designs, Northwest Territories Mine Health and Safety Regulations (15.95) (SNWT 1995) state that each catchment berm shall be designed so that its final width will not be less than 8 m. The design bench width will be 8.5 m in all areas, exceeding the minimum criteria.

An immaterial amount of overburden is expected within the open pit area. If present, overburden material will be removed 2 m back from the pit crest and sloped to less than the angle of repose as per the Yukon *Workers' Safety and Compensation Act* (15.40) (Yukon Legislative Counsel Office 2017). Stripped topsoil and subsoil will be transported to the soil stockpiles, located immediately north of the heap leach pad and topographically above the Alpha WRSF, for use in reclamation or for permanent disposal. The soil stockpile is shown on the general arrangement map in Figure 1-2.

Frozen soils will be placed in the frozen soil storage area within the Alpha WRSF (Figure 1-2). The frozen soils will be segregated by type, defined primarily by ice content and grain size, in the frozen soil storage area to permit recovery of materials that can be utilized in reclamation. If excess to closure requirements as determined through an updated material balance evaluation, some of the frozen soil may be co-disposed of in thin horizontal lifts within the Alpha WRSF. Materials placed in the Alpha WRSF will be placed with coarse, durable waste rock at a ratio such that the overall strength of the mixture ensures stability. Additional details regarding overburden management in the open pit area are provided in the Waste Rock and Overburden Management Plan (WROMP).



Source: JDS 2019

Figure 3-3 Pit Slope Design Recommendations

Stability of the overall slopes in the detailed design was confirmed using limit equilibrium slope stability modelling software. Rock mass shear strengths were developed for each rock type based on the results of the field and laboratory test work. Several variations of rock mass strengths were evaluated for each cross section to investigate the amount of influence different joint lengths may have on the overall rock mass strength. Results of the overall slope stability analyses demonstrate that the bench configuration-based slope angles either meet or exceed the acceptable safety factor of 1.3. Further details on the assessment of the open pit slope stability can be found in Appendix B - Feasibility Pit Slope Stability Update Coffee Project Yukon Territory, Canada.

A thorough geological and geotechnical bench face mapping program will be undertaken beginning in the early stages of development to verify that the geologic structural conditions encountered are consistent with the assumptions and estimates used in this plan, and to identify local variations in structural conditions that might increase the risk of localized instabilities. Bench design will be refined, and inter-ramp and overall slope angles will be optimized based on the geotechnical database developed during this program. The bench face mapping program will include the following components:

- Recording orientations, lengths, and frequencies of dominant discontinuity sets
- Identification of local variations in structural conditions that might increase the risk of localized instabilities
- Mapping the location, orientation, and conditions of discrete major fault structures
- Identification of potential failure mechanisms such as wedges, planar, toppling and/or pseudo-rotational failures
- Mapping of areas of poor rock quality, tension cracks or signs of potential displacement
- Taking photographs of each bench for later reference, if needed
- Direct comparison of 3D pit wall scans to their respective designs to evaluate wall control blasting performance and verify bench designs are being achieved.

Mapping will take place on newly exposed walls on a weekly to monthly basis depending on mining rates. Slope designs will be reviewed regularly and optimized as needed based on the geotechnical mapping database.

3.3.2 Seismic Design Evaluation

A seismic study was conducted for the Project to identify seismic source zones and to determine the probably and extent of an earthquake event that is likely to occur within the vicinity of the Project. For the open pits, the seismic design criteria are evaluated within the context of slope stability using an analytical hierarchy that considers various elements of slope stability during a seismic event. The analysis determined there to be a low risk of seismically induced instability due to the low probability of a major earthquake event. The study determined however that increased rockfall on mine benches and/or other exposed slopes may result from a major earthquake event (magnitude 5.5 or greater). Further details on the seismic design evaluation for the Project are provided in Appendix C.

4.0 OPEN PIT MINING DESIGN AND METHODS

4.1 Mine Design

The final open pit design is split into several pits and phases, including Latte (two main phases, and three satellite pits), Double-Double (single phase), Supremo (seven phases), and Kona (single phase). The detailed pit designs incorporate all geotechnical pit slope design criteria and road width criteria discussed in Section 3.3.1 and Section 4.5. The approximate dimensions are shown in Table 4-1 with plan views, and cross sections, and final pit detail of each open pit design shown in Figure 4-1 to Figure 4-15. The detailed figures illustrate the current designed configuration of the pits (including benches and haul roads). These details, including exact pit footprints and volumes of extracted material, are subject to change over the LOM based on various factors considered in long and short range mine planning including, but not limited to; commodity pricing, efficiency of equipment, safety procedures etc. Extracted material will not exceed volumes assessed during the YESAA assessment, and material changes to pit design details will be communicated to Energy, Mines and Resources, Minerals through regular reporting or updates to the Mine Operations Plan. Figure 1-2 depicts the general site arrangement including pits, waste rock storage facility, and supporting infrastructure.

Table 4-1 Estimated Open Pit Dimensions and Life of Mine Quantities of Contained Mineralized Material and Waste Rock

Open Pit	Max. Length (m)	Max. Width (m)	Max. Depth (m)	Mineralized Material (kt)	Waste Rock (kt)
Latte	1500	360	140	20962	62505
Double Double	640	270	140	1340	9281
Kona	375	170	90	946	2933
Supremo	2150	570	225	36902	223959
Total				60150	298678

Source: Newmont 2021

Open pit mining operations propose the use of a fleet comprising 15 m³ shovels, 12 m³ front-end loaders, hydraulic excavators, and haul trucks in the range of 144 tonnes, though the exact size of equipment is subject to change based on ongoing and continuous mine planning over the LOM. The primary fleet will be supported by drills, graders, and track and rubber-tire dozers.

Mineralized material hauled from the open pits will either be transported to the crusher system or to the ROM stockpile for temporary storage until it is crushed. Much of the mineralized material will be sent directly to the primary crusher and, after crushing, re-handled from the primary crusher stockpile to the HLF. Mineralized overburden material that is encountered during site development activities may also be processed with the open pit mineralized material. Mineralized material will likely be stockpiled on the ROM stockpile when the crusher is not running and crushed material is not being added to the HLF (i.e., January through March), or during scheduled periods when the mining rate or mineralized material exceeds the throughput rate of the crusher.

Measures to track the PAG ore that will be temporarily stored within the ROM stockpile are described in the WROMP. A drainage ditch will be constructed along the perimeter of the ROM ore stockpile to facilitate drainage of runoff and stockpile seepage towards Facility Pond, from which the water may be used as makeup water for the HLF, used for dust suppression or discharged via pumped pipeline towards Alpha

Pond, dependent on water quality. Details regarding the water management strategy for the ROM stockpile are provided in the Water Management Plan.

Stockpiled material will be re-handled and sent to the primary crusher during the months when stacking is occurring on the HLF, and the ROM stockpile will be operational over the entire mine life, starting during pre-production. Currently there are no plans to create separate high-grade and low-grade material, although this may be investigated further during operational optimization to improve recoveries and control the grade of ore reporting to the HLF. Table 4-3 shows the ROM stockpile balance over the life of the mine. The ROM stockpile is located close to the primary crusher to minimize re-handle. Details regarding the crushing and heap leaching are provided in the Heap Leach and Process Facilities Plan.

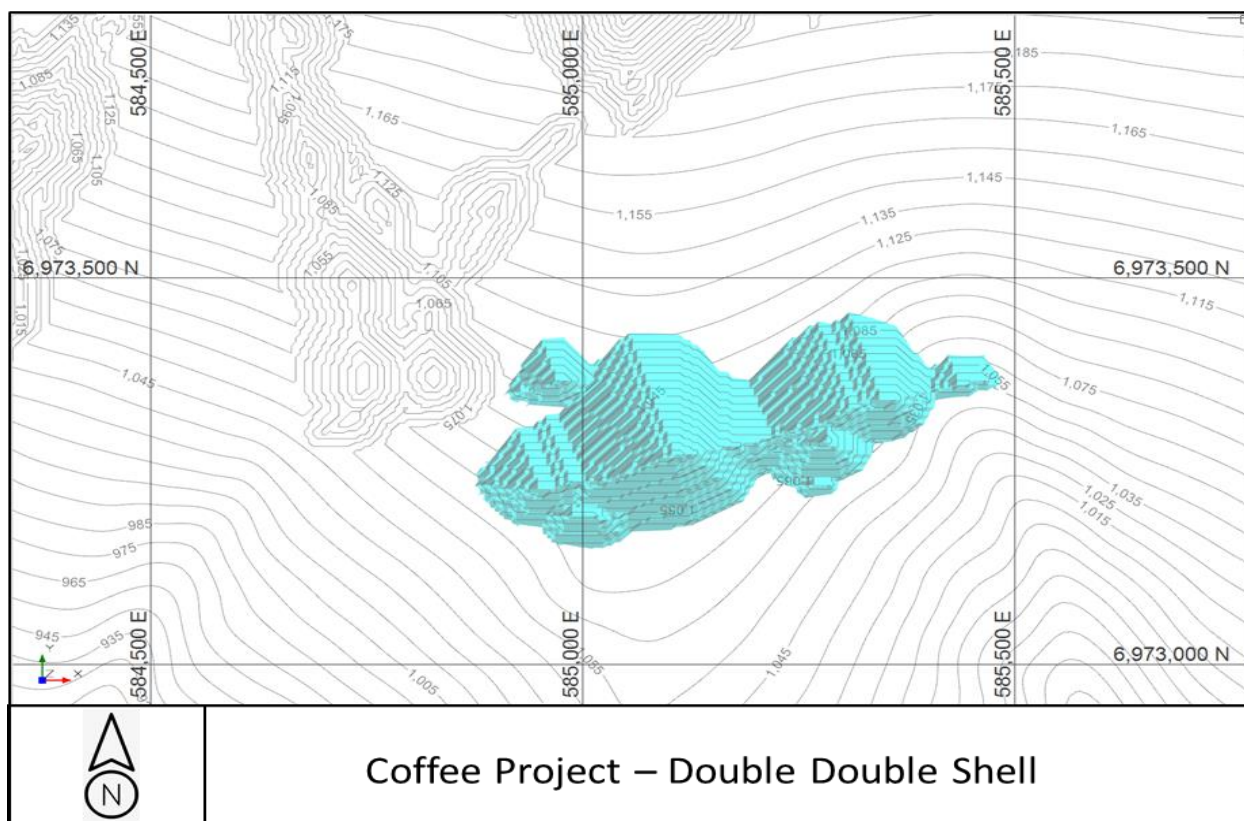


Figure 4-1 Double Double Shell

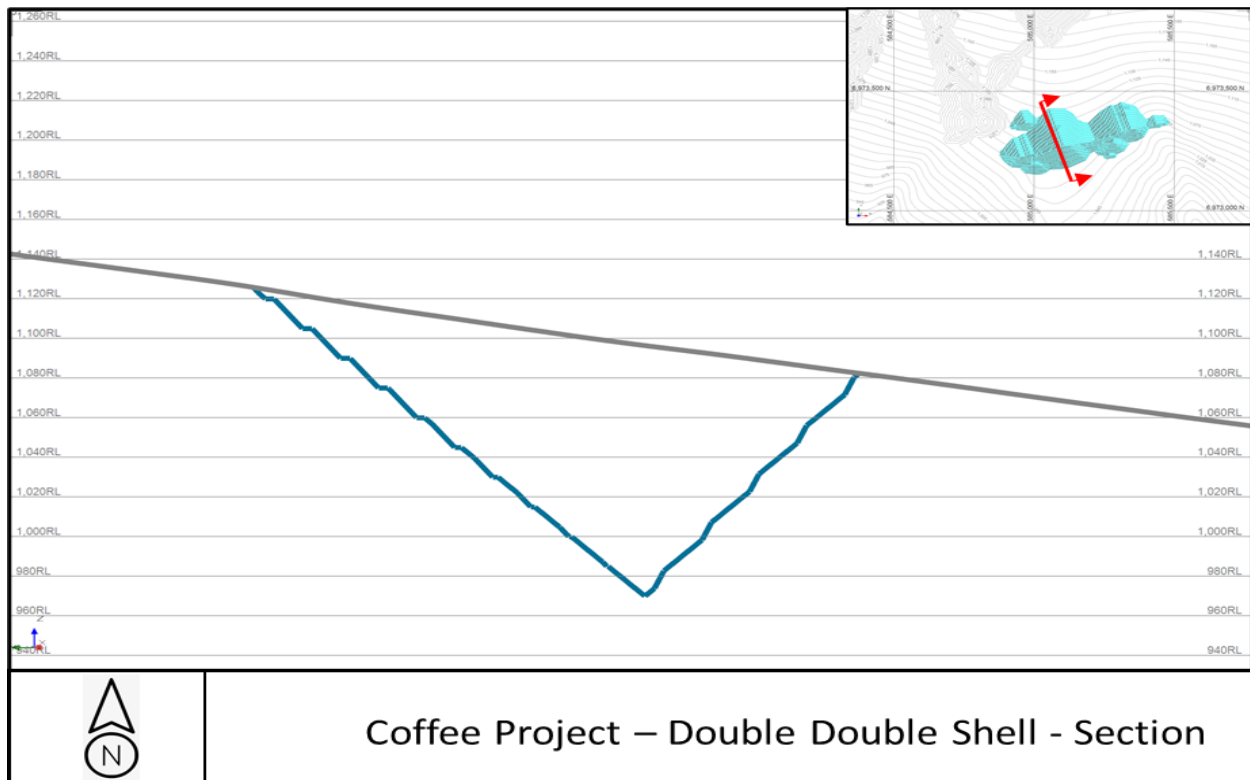


Figure 4-2 **Double Double Cross Section**

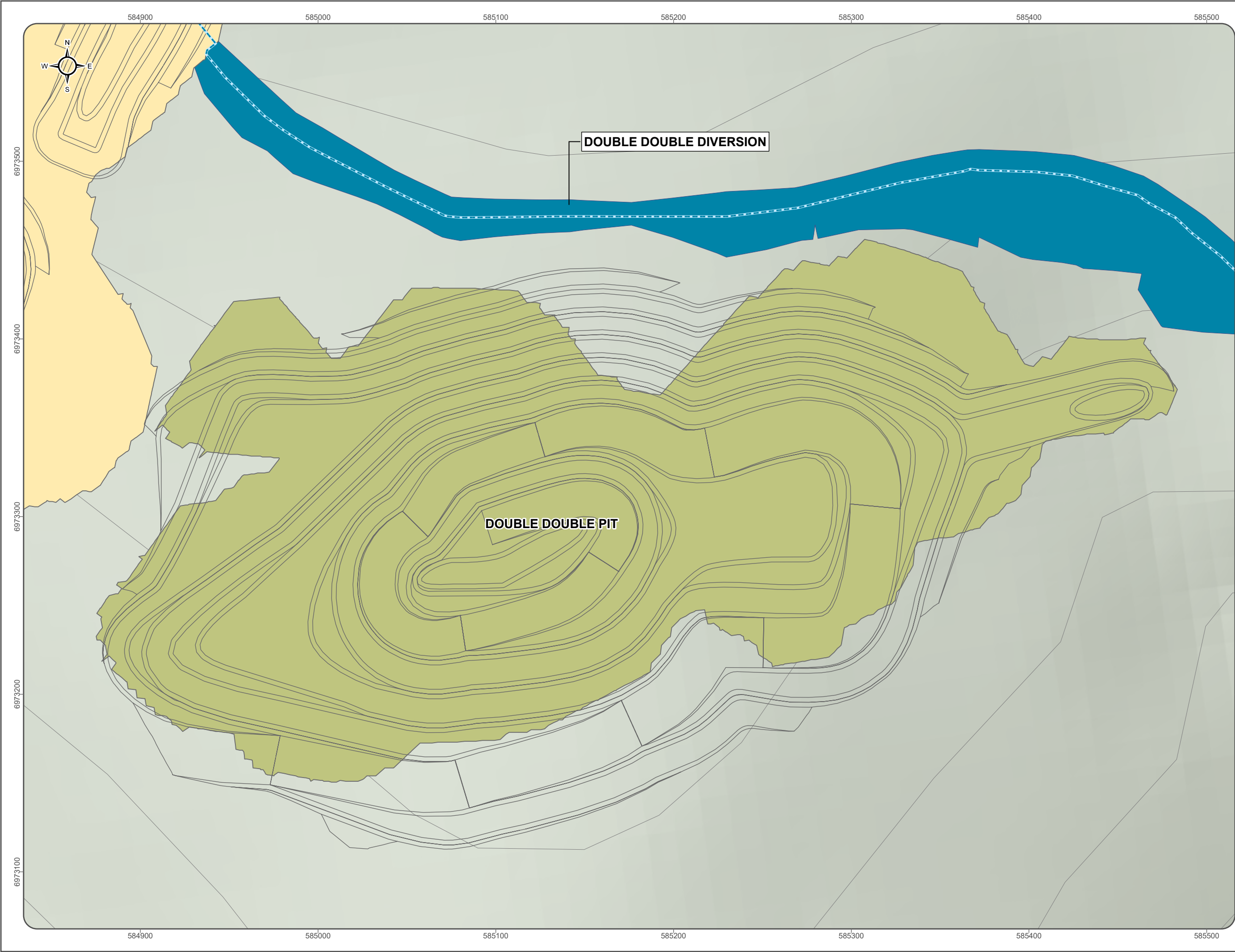


FIGURE 4-3
DOUBLE DOUBLE
PIT DETAILS

SEPTEMBER 2024

- Double Double Pit
- Supremo Pit
- Water Management Area
- Diversion Ditch / Spillway
- Mine Site Road

1:2,006 when printed on 11 x 17 inch paper
0 25 50 75 100
Meters



Imagery: Yukon Government High Resolution Satellite Imagery. URL: <https://mapservices.gov.yk.ca/>
Watercourse and elevation data: CANVEC dataset. Compiled by Natural Resources Canada.

Map Datum: NAD 83; Projection: UTM Zone 7N

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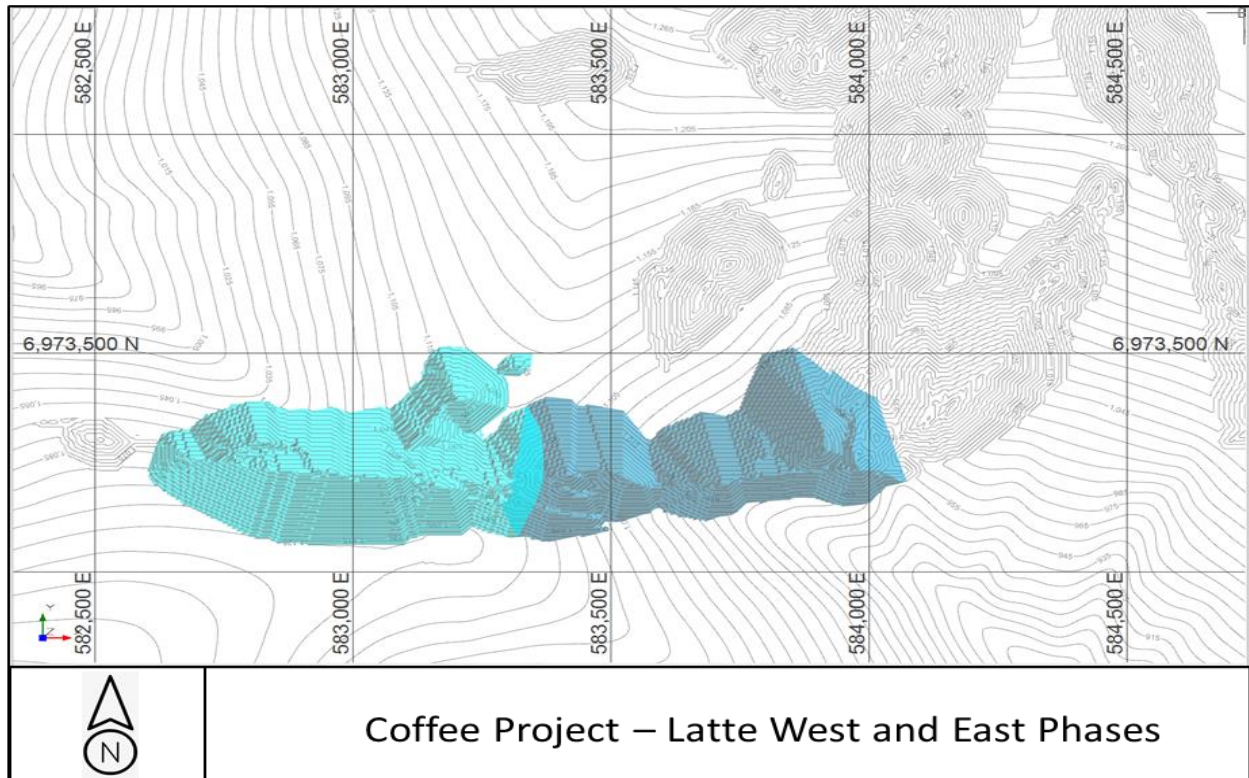


Figure 4-4 Latte Phase 01 East and West Shells

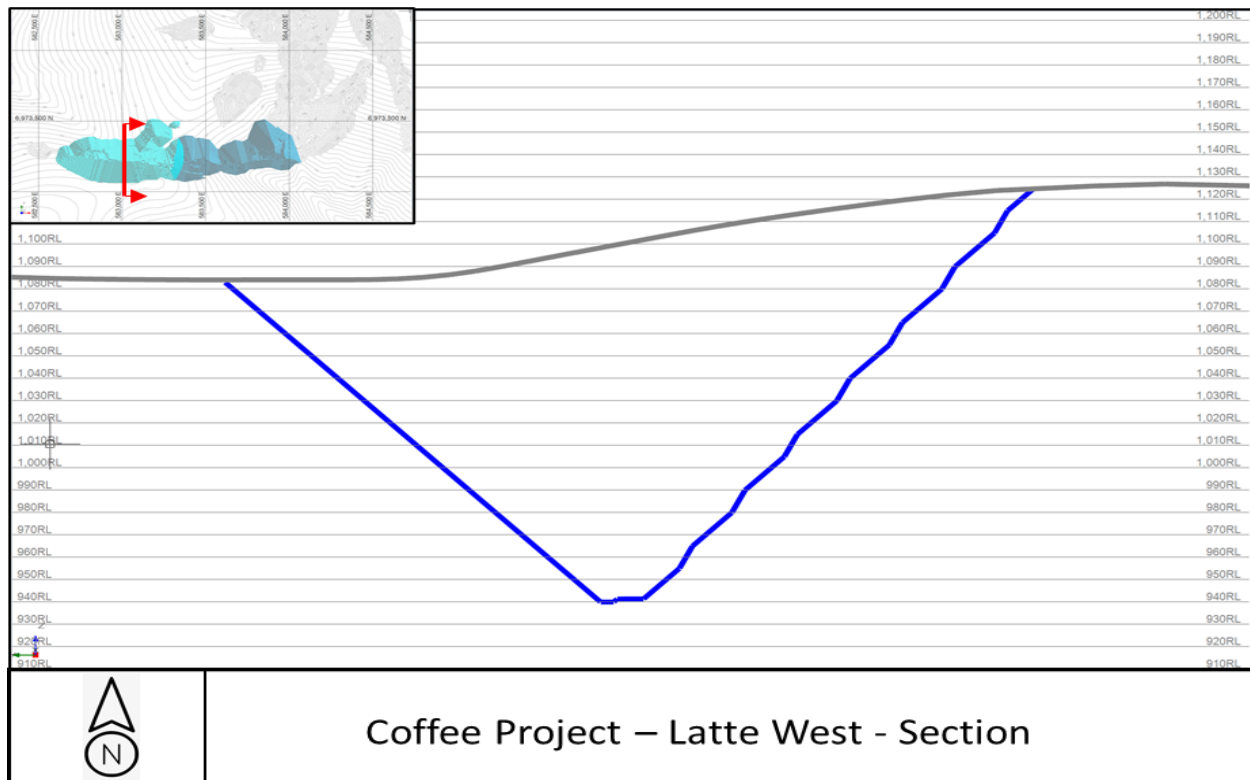


Figure 4-5 Latte Phase 01 West Cross Section

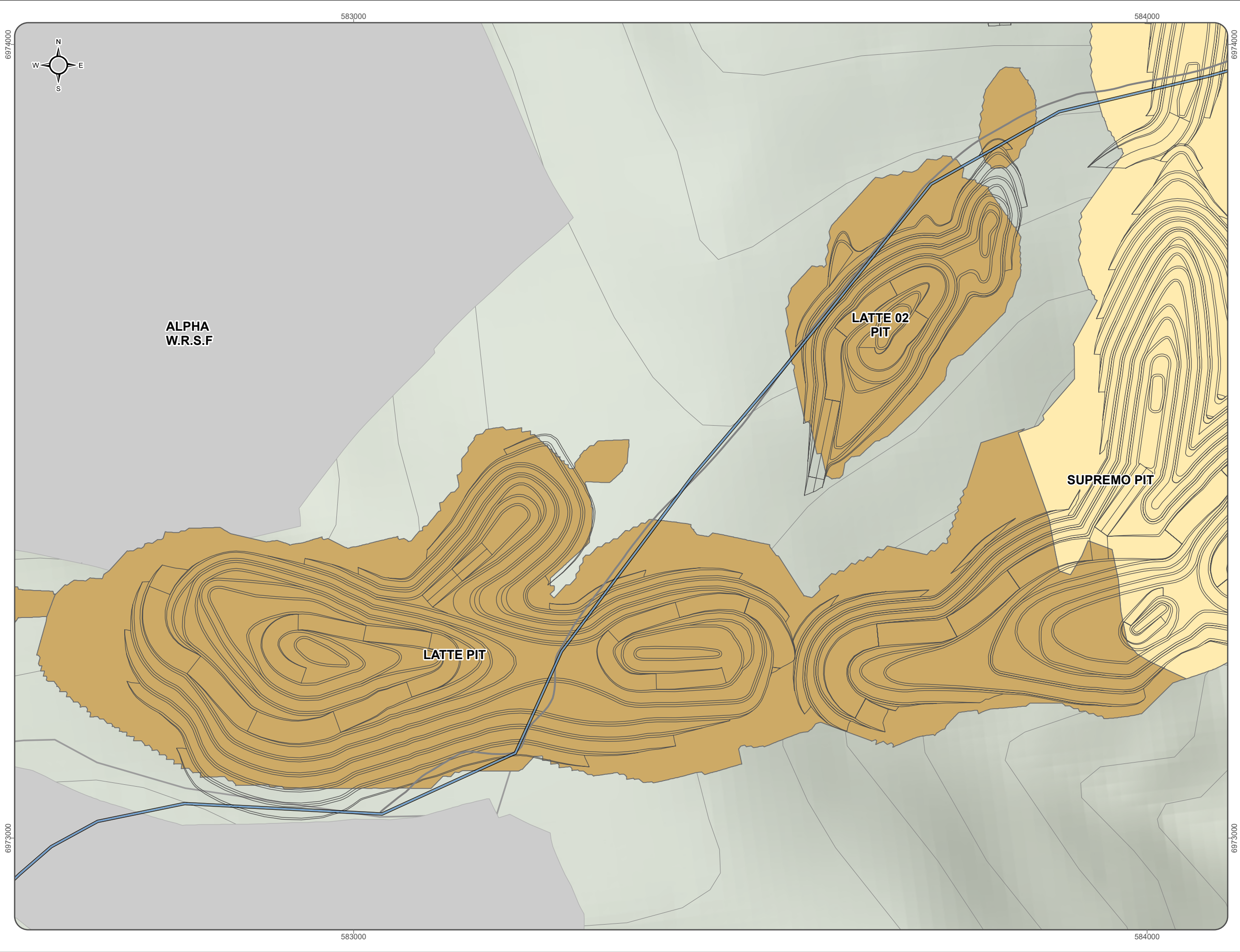


FIGURE 4-6

LATTE PIT DETAILS

SEPTEMBER 2024

- Supremo Pit
- Latte Pit
- Mine Feature Footprint
- Pipeline
- Project Access Road
- Mine Site Road

1:4,500 when printed on 11 x 17 inch paper

0 50 100 150 200

Meters



Imagery: Yukon Government High Resolution Satellite Imagery. URL: <https://mapservices.gov.yk.ca/>
Watercourse and elevation data: CANVEC dataset. Compiled by Natural Resources Canada.

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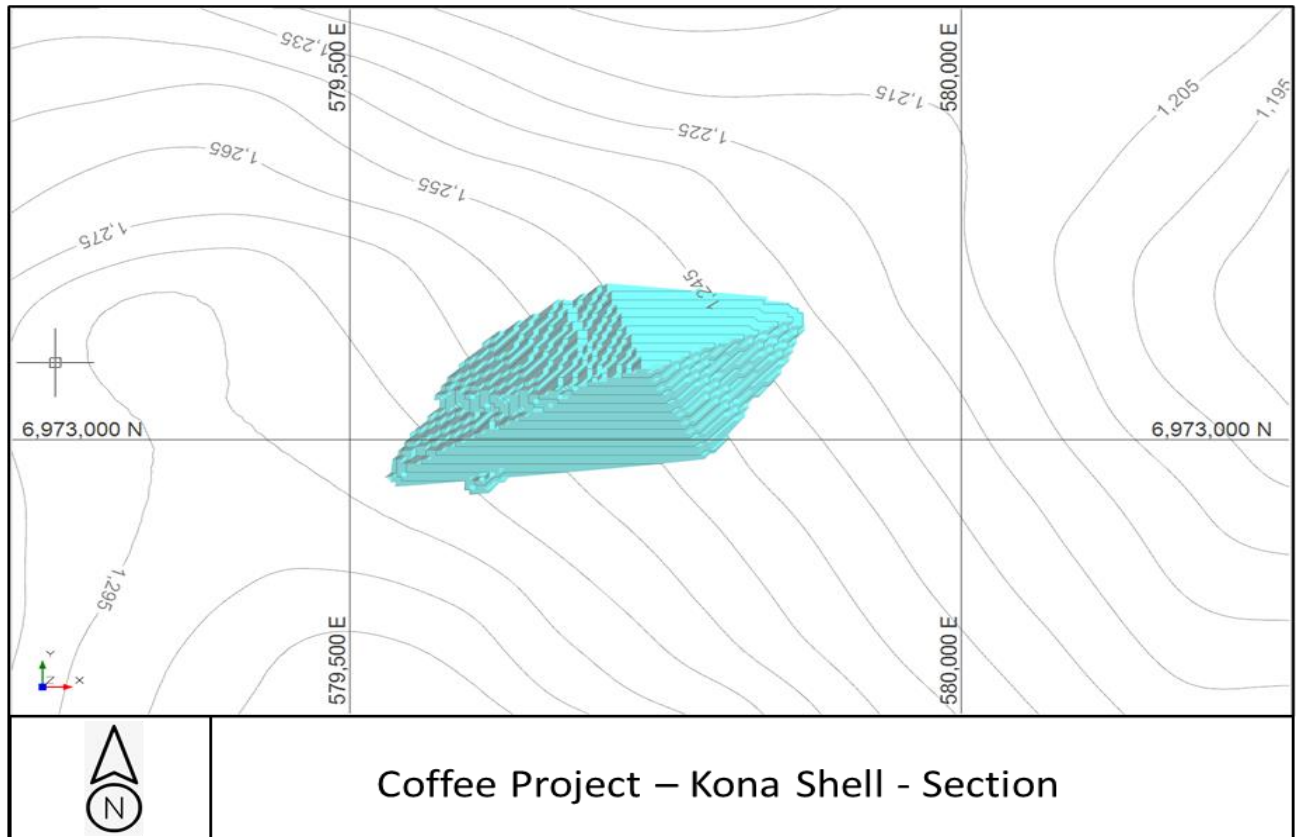


Figure 4-7 Kona Shell

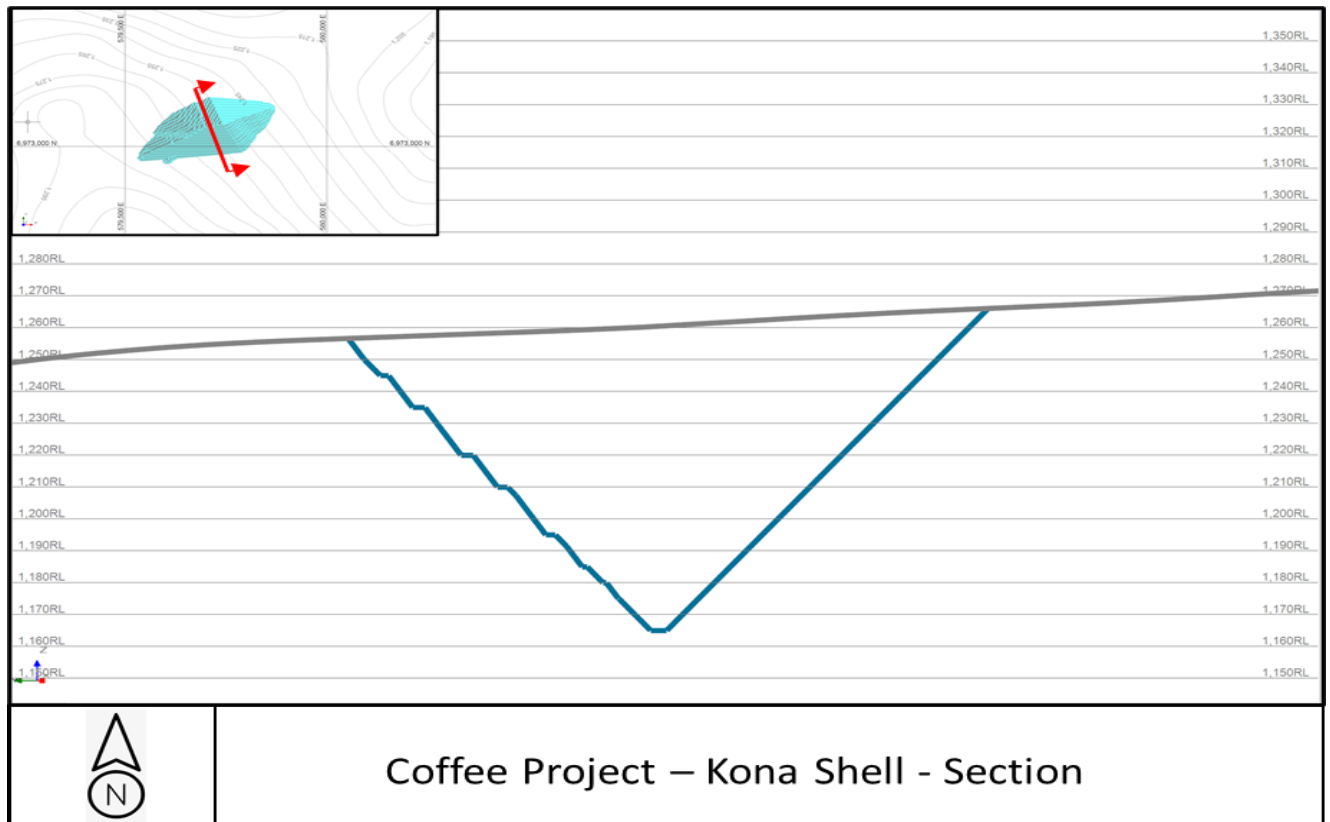
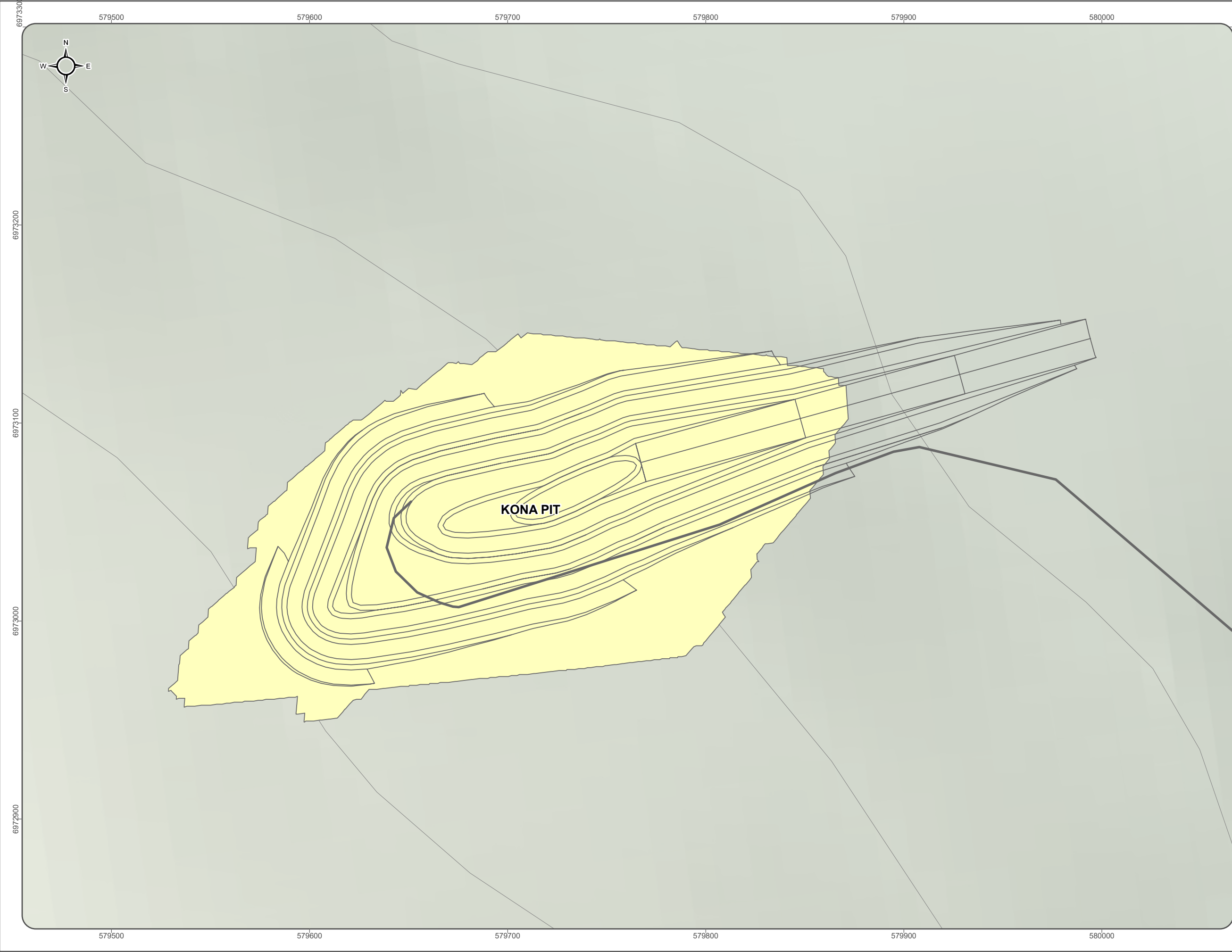


Figure 4-8 Kona Cross Section



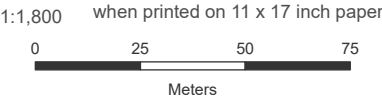
COFFEE EXPLORATION PROJECT

FIGURE 4-9

KONA PIT DETAILS

SEPTEMBER 2024

-  Kona Pit
-  Mine Site Road



Imagery: Yukon Government High Resolution Satellite Imagery. URL: <https://mapservices.gov.yk.ca/>
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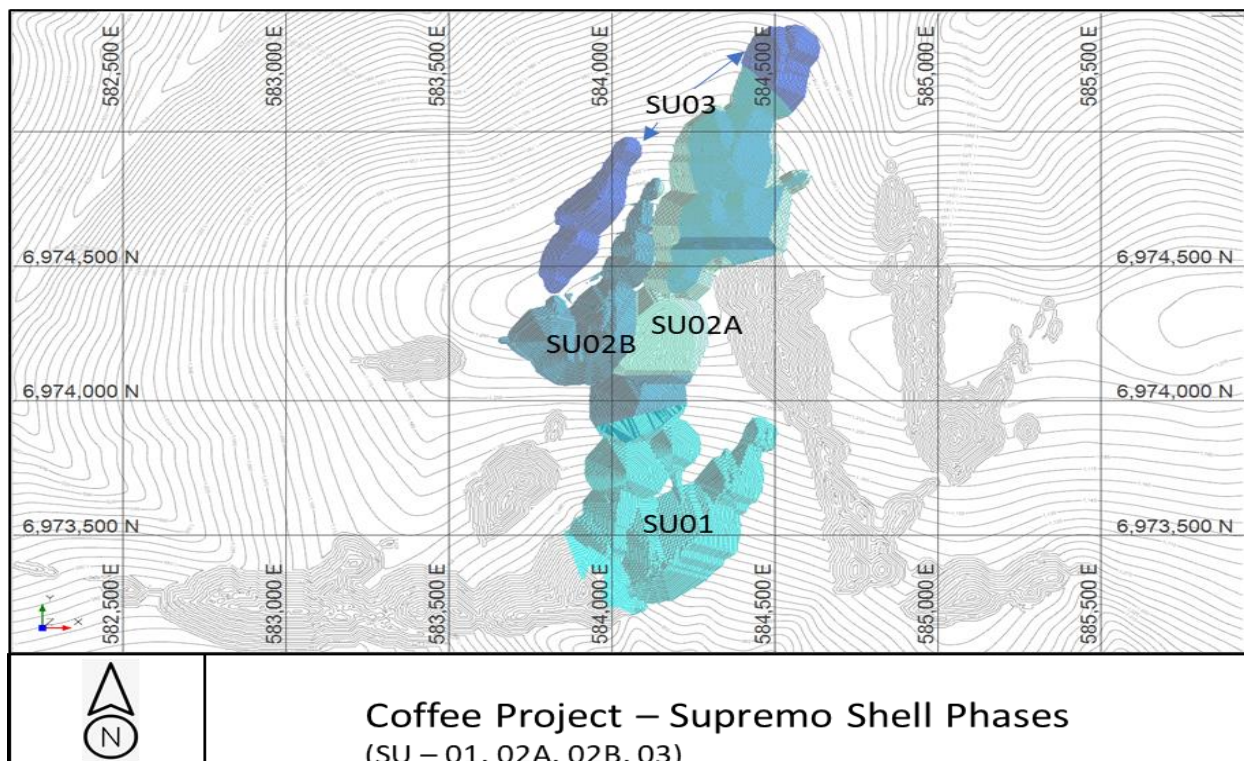


Figure 4-10 Supremo Phase 01, 02A, 02B, and 03 Shells

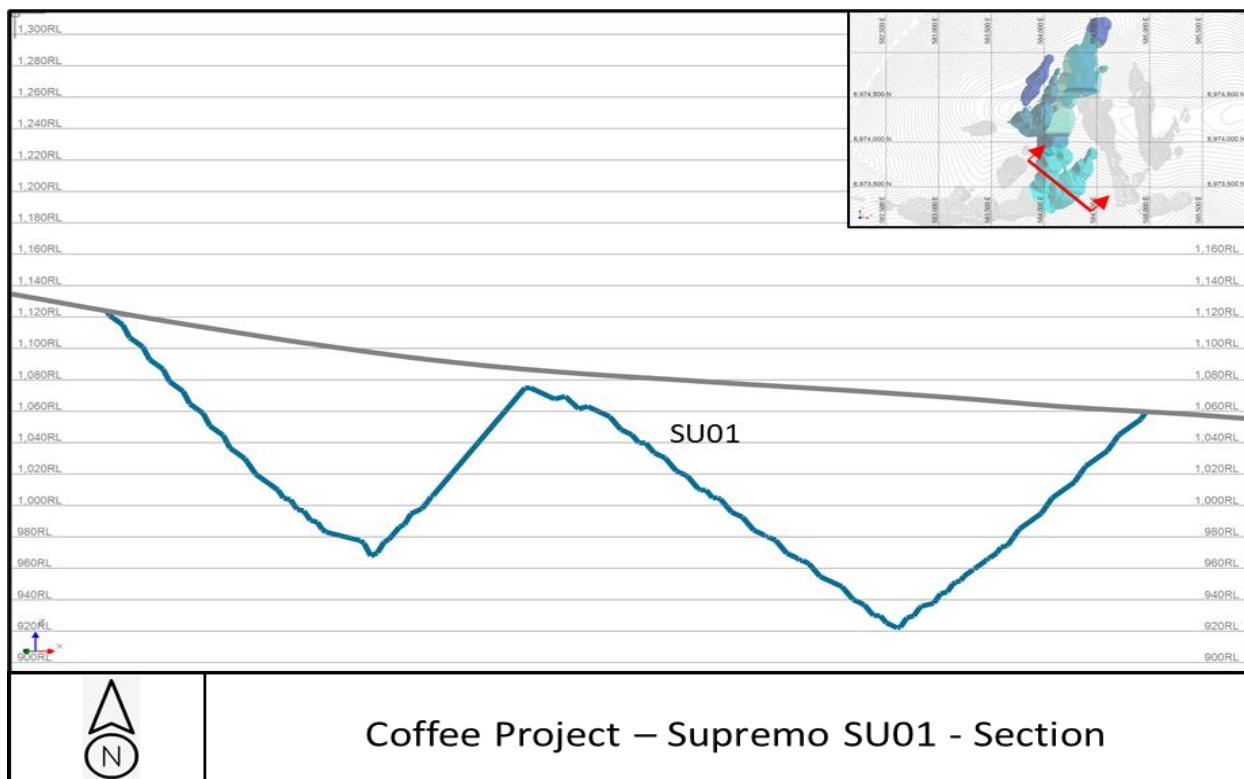


Figure 4-11 Supremo Phase 01 Cross Section

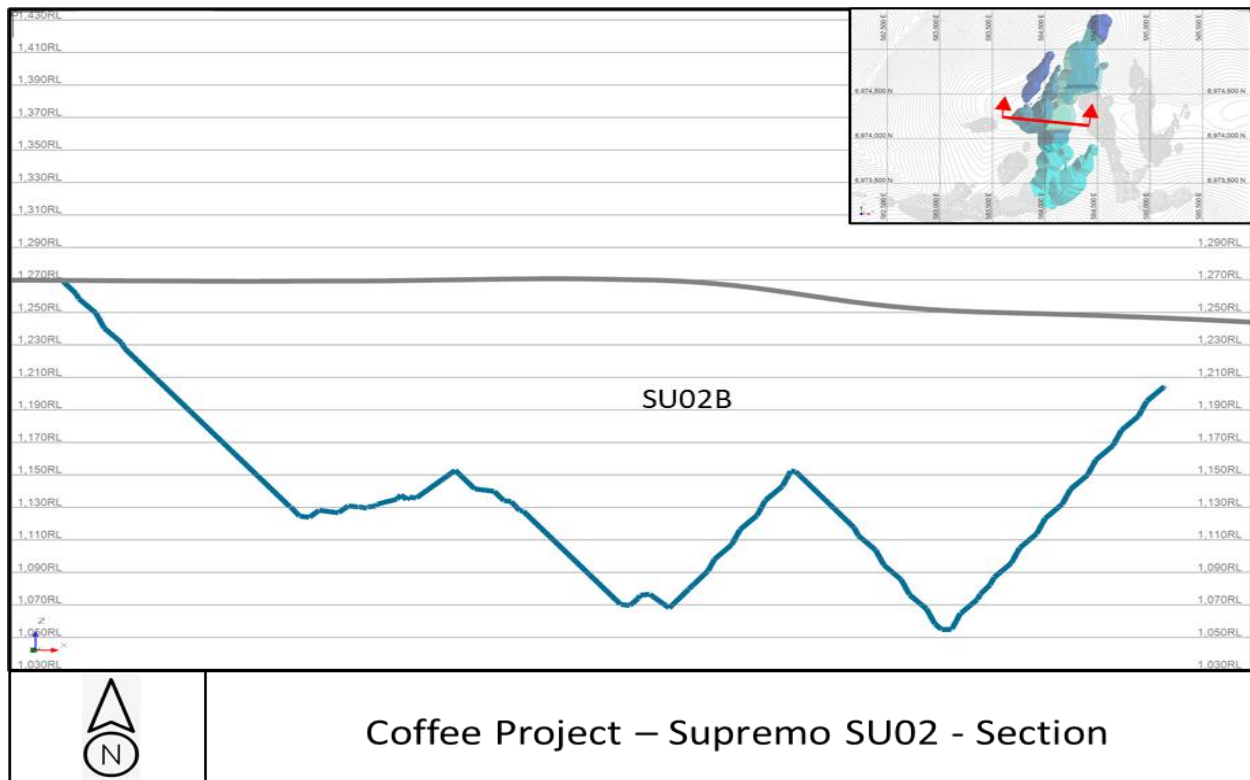


Figure 4-12 Supremo Phase 02 Cross Section

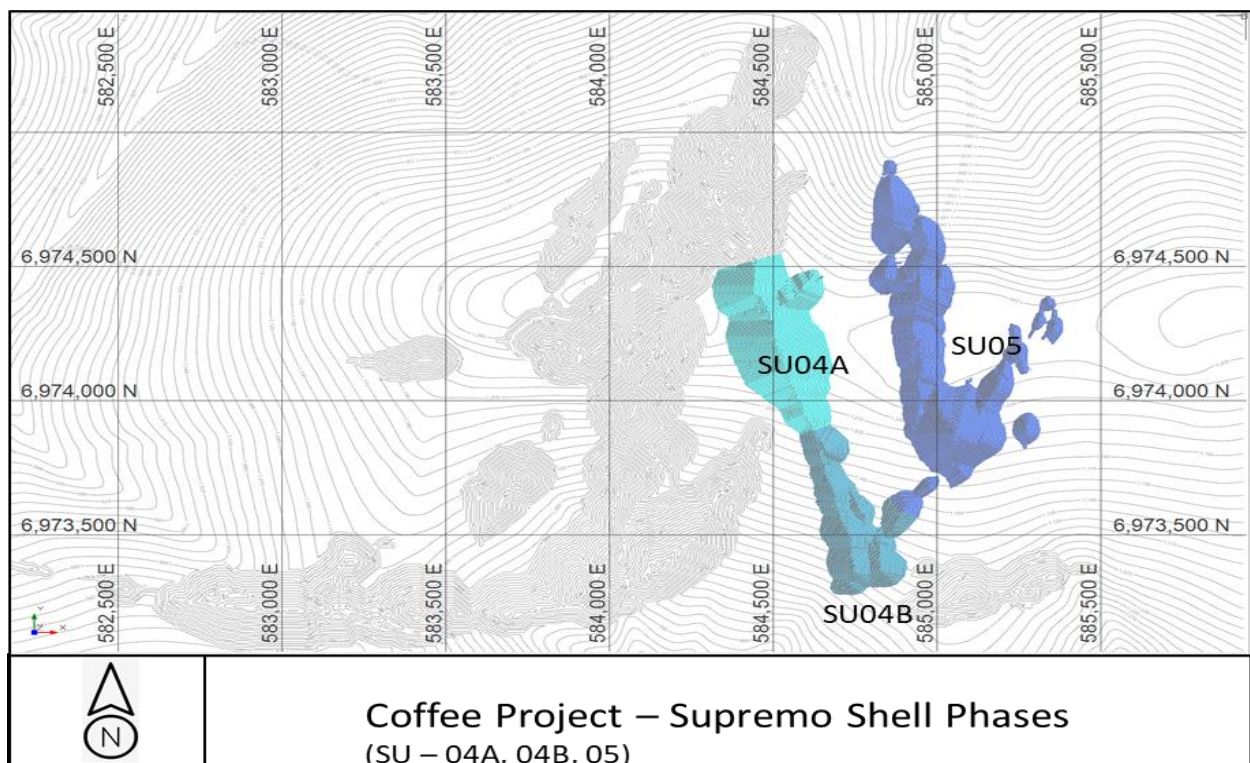


Figure 4-13 Supremo Phase 04A, 04B, and 05 Shells

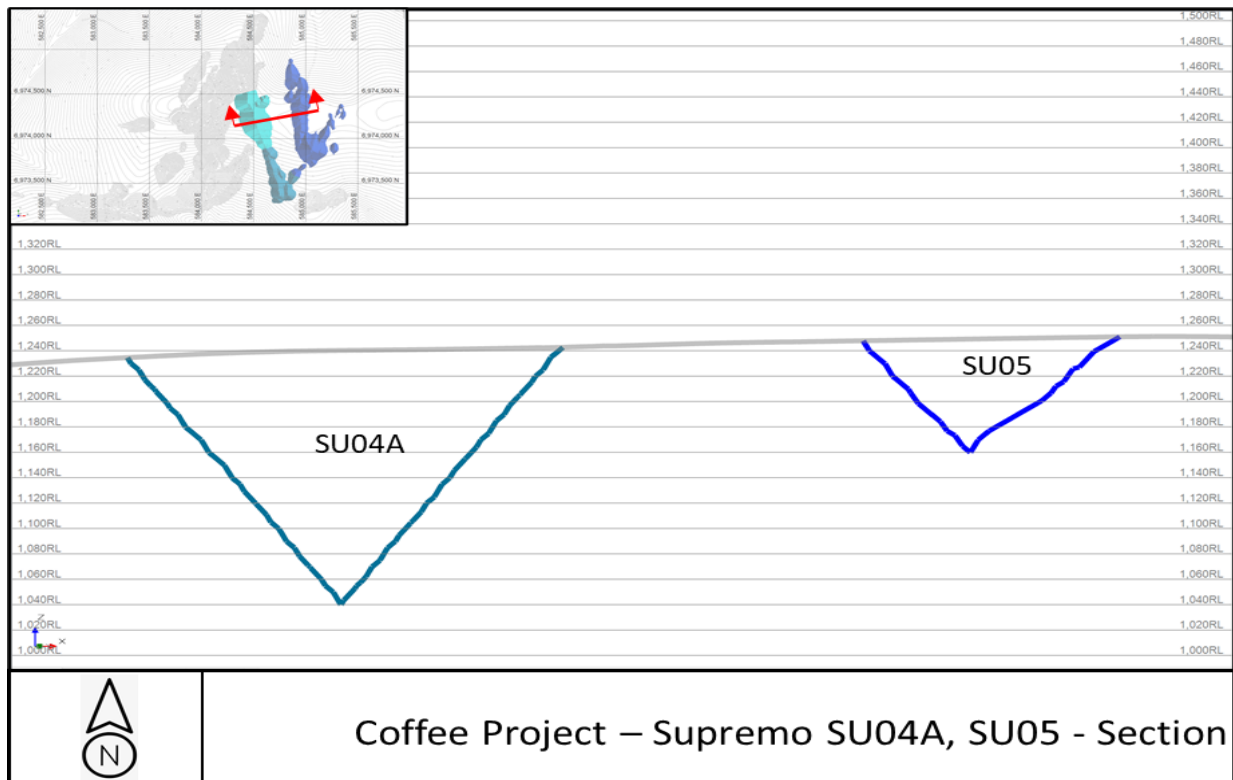


Figure 4-14 Supremo Phase 04A and 05 Cross Section

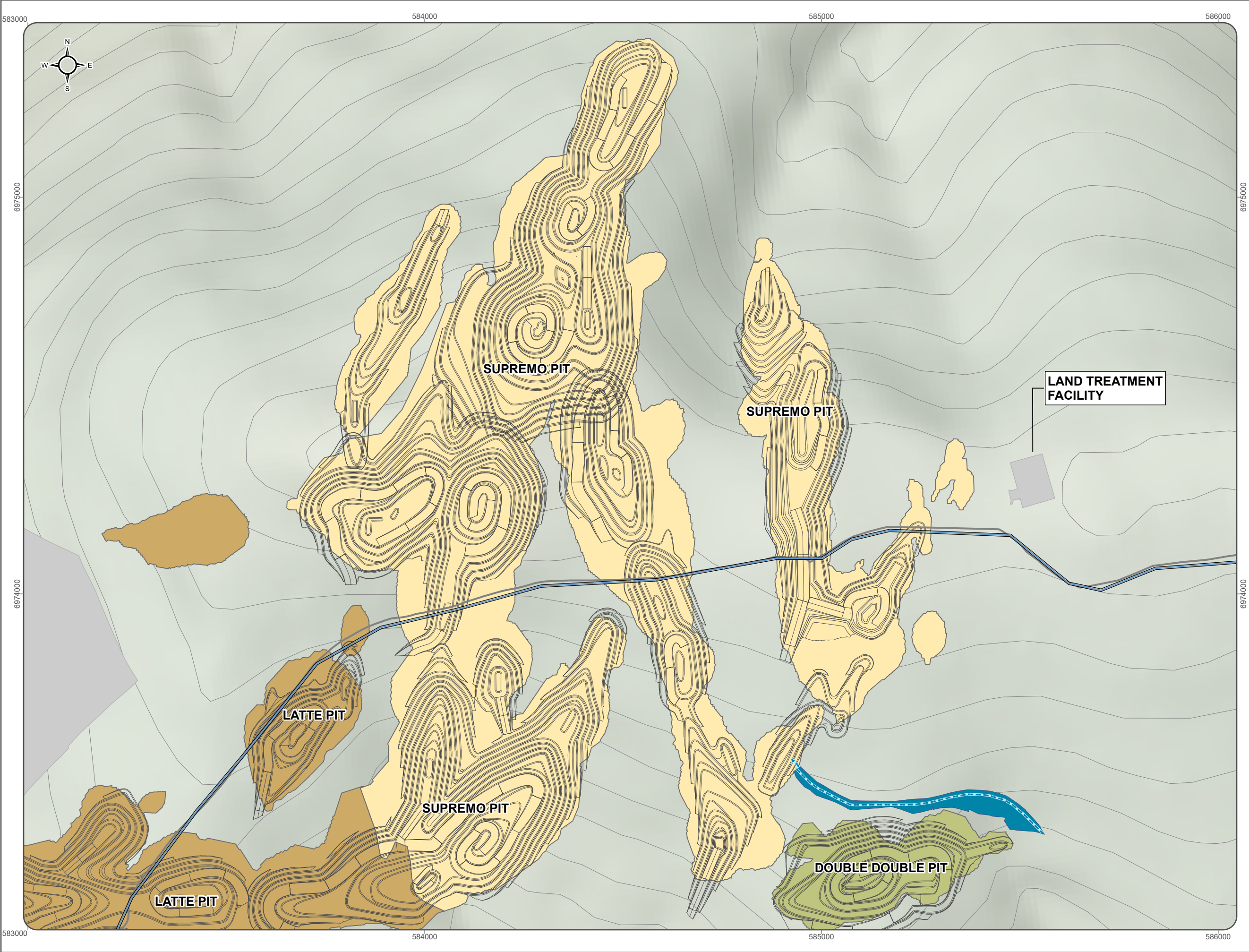


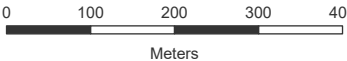
Figure 4-15

SUPREMO PIT DETAILS

SEPTEMBER 2024

- Double Double Pit
- Supremo Pit
- Latte Pit
- Mine Feature Footprint
- Water Management Area
- Diversion Ditch / Spillway
- Pipeline
- Project Access Road
- Pit Contours

1:9,000 when printed on 11 x 17 inch paper



Imagery: Yukon Government High Resolution Satellite Imagery. URL: <https://mapservices.gov.yk.ca/>
Watercourse and elevation data: CANVEC dataset. Compiled by Natural Resources Canada.

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4.2 Material Release Schedule

Based on the above identified resources, the four pits are expected to produce a total of 60 Mt of ore at an average gold head grade of 1.33 g/t. The contained gold is estimated to be 2.57 million ounces. Ore mined during the pre-production period in Year –1 will be placed in the ROM stockpile until the heap leach pad is ready to load ore. The process plant is planned to be operational by early Q2 of Year –1, allowing for commencement of leach solution application at the heap leach pad and production of first gold in Q2 of Year –1. During winter (January to March inclusive), ROM ore from the mine may be placed onto a stockpile for crushing and processing during the remainder of the year. By the end of Year 9, open pit mining will cease, and the ROM stockpile will be depleted (Figure 4-16 and Figure 4-17).

Table 4-2 shows the estimated annual material release schedule from the mine (excluding re-handle). Material extraction is coordinated to maximize heap leach extraction, and to take advantage of empty pits for waste rock. The Latte and Supremo pits are mined first (Year -1), mining of the Double Double pit starts in Year 1 and ends in Year 3, and mining of the Kona pit starts in Year 8 and ends in Year 9 (Figure 4-16). Should suitable waste rock from within the pit footprints be required as construction material earlier in the mining sequence than anticipated, the mining schedule will be adjusted, and this material will be mined for use based on the site development requirements (i.e., Alpha pond dam).

Table 4-3 outlines the anticipated stockpile and Heap Leach feed schedule. The majority of ore is trucked directly to the primary crusher, with some trucked to the stockpile and some removed from the stockpile as required throughout the year. The balance is left on the stockpile for use the following year (Figure 4-17).

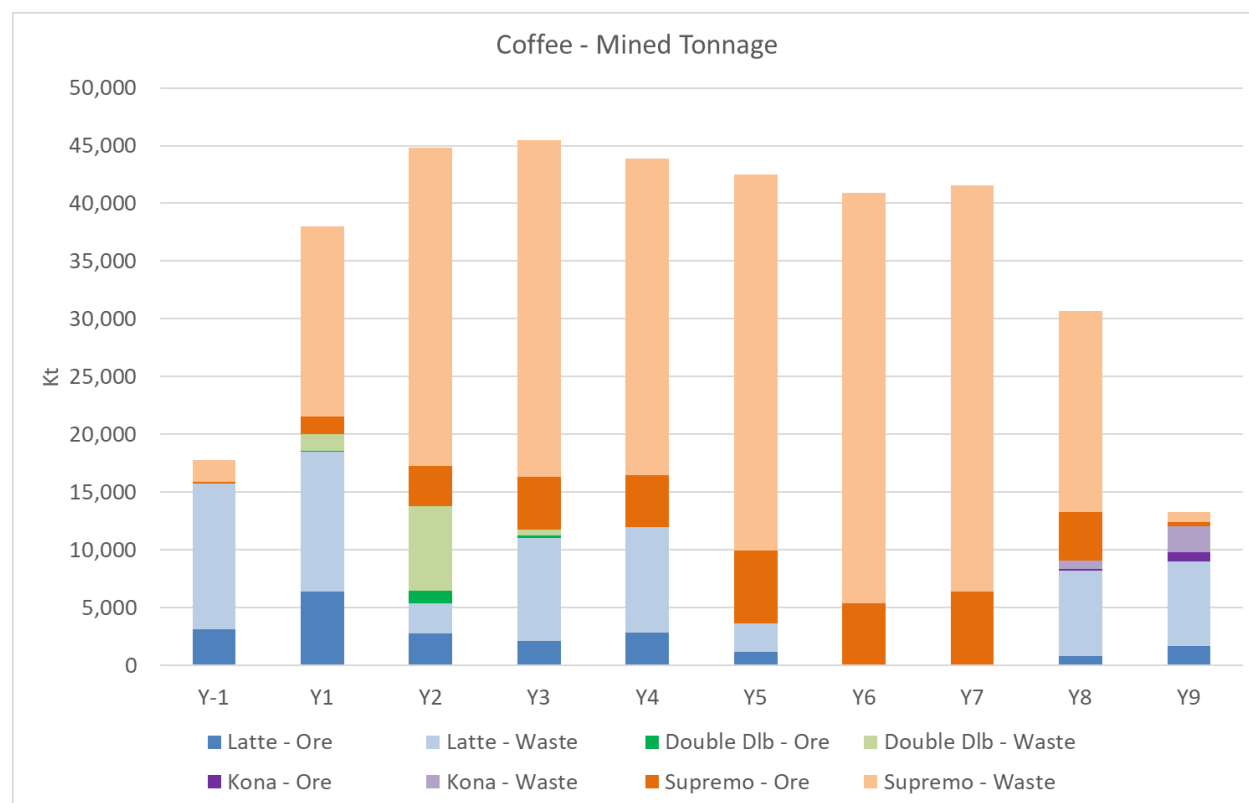


Figure 4-16 Ore and Waste Rock Mining Schedule by Pit Parameters

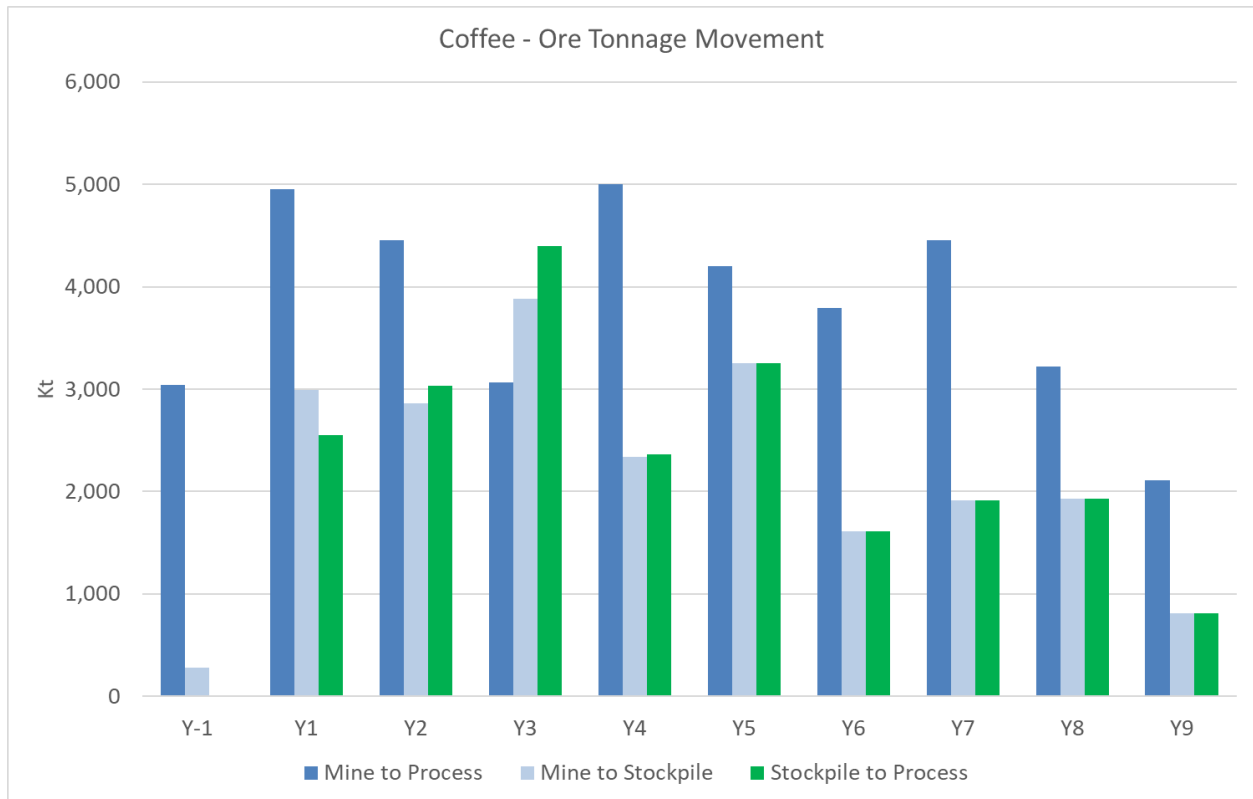


Figure 4-17 Direct Feed and Stockpiled Ore Schedule Parameters

Table 4-2 Approximate Annual Material Mined from the Open Pits

Description	Unit	Total	Year –1	Year 1	Year 2	Year 3	Year 4	Year 5	Year 6	Year 7	Year 8	Year 9
Latte												
Mineralized Material	kt	20,962	3,163	6,365	2,801	2,100	2,856	1,155	0	0	812	1,710
Au grade	g/t	1.24	1.24	1.43	1.38	1.06	0.95	1.03	0.00	0.00	0.91	1.28
Au contained	koz	835	126	293	124	71	88	38	0	0	24	71
Waste	kt	62,505	12,587	12,130	2,608	8,900	9,144	2,468	0	0	7,388	7,279
Strip Ratio	t:t	3.0	4.0	1.9	0.9	4.2	3.2	2.1	0.0	0.0	9.1	4.3
Total Material	kt	83,467	15,750	18,495	5,409	11,000	12,000	3,623	0	0	8,200	8,990
Double Double												
Mineralized Material	kt	1,340	0	43	1,071	227	0	0	0	0	0	0
Au grade	g/t	2.68	0.00	1.98	2.64	2.98	0.00	0.00	0.00	0.00	0.00	0.00
Au contained	koz	115	0	3	91	22	0	0	0	0	0	0
Waste	kt	9,281	0	1,457	7,325	499	0	0	0	0	0	0
Strip Ratio	t:t	6.9	0.0	34.1	6.8	2.2	0.0	0.0	0.0	0.0	0.0	0.0
Total Material	kt	10,622	0	1,500	8,396	726	0	0	0	0	0	0
Kona												
Mineralized Material	kt	946	0	0	0	0	0	0	0	0	145	800
Au grade	g/t	1.07	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.80	1.11
Au contained	koz	32	0	0	0	0	0	0	0	0	4	29
Waste	kt	2,933	0	0	0	0	0	0	0	0	707	2,226
Strip Ratio	t:t	3.1	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	4.9	2.8
Total Material	kt	3,878	0	0	0	0	0	0	0	0	852	3,026
Supremo												
Mineralized Material	kt	36,902	154	1,533	3,445	4,620	4,481	6,296	5,402	6,369	4,192	410
Au grade	g/t	1.33	4.16	2.50	1.39	1.35	1.38	1.17	1.17	1.20	1.39	1.03
Au contained	koz	1,583	21	123	154	200	198	237	202	246	188	14

Description	Unit	Total	Year –1	Year 1	Year 2	Year 3	Year 4	Year 5	Year 6	Year 7	Year 8	Year 9
Waste	kt	223,959	1,842	16,478	27,562	29,130	27,427	32,612	35,494	35,159	17,417	837
Strip Ratio	t:t	6.1	11.9	10.8	8.0	6.3	6.1	5.2	6.6	5.5	4.2	2.0
Total Material	kt	260,862	1,996	18,011	31,007	33,750	31,908	38,908	40,896	41,528	21,609	1,247
Total Open Pits												
Mineralized Material	kt	60,150	3,317	7,940	7,317	6,946	7,337	7,451	5,402	6,369	5,149	2,921
Au grade	g/t	1.33	1.38	1.64	1.57	1.31	1.21	1.15	1.17	1.20	1.30	1.20
Au contained	koz	2,566	147	419	369	293	286	275	202	246	215	113
Waste	kt	298,678	14,429	30,066	37,495	38,529	36,571	35,081	35,494	35,159	25,512	10,343
Strip Ratio	t:t	5.0	4.3	3.8	5.1	5.5	5.0	4.7	6.6	5.5	5.0	3.5
Total Material	kt	358,828	17,746	38,006	44,812	45,476	43,908	42,532	40,896	41,528	30,661	13,263
Average Mined	t/d	99,924	64,766	104,126	122,772	124,591	120,297	116,525	112,044	113,775	84,004	36,338

Table 4-3 Annual Heap Leach Feed Source and Run-of-mine Stockpile Balance

Description	Unit	Total	Y-1	Y1	Y2	Y3	Y4	Y5	Y6	Y7	Y8	Y9
Open Pit												
Mineralized Material Mined	kt	60,150	3,317	7,940	7,317	6,946	7,337	7,451	5,402	6,369	5,149	2,921
Au grade	g/t	1.33	1.38	1.64	1.57	1.31	1.21	1.15	1.17	1.20	1.30	1.20
Au contained	koz	2,566	147	419	369	293	286	275	202	246	215	113
Heap Leach Feed												
<u>Direct Mine to HL</u>	kt	38,289	3,042	4,949	4,455	3,063	4,998	4,201	3,795	4,455	3,219	2,113
Au grade	g/t	1.36	1.41	1.64	1.61	1.29	1.26	1.24	1.24	1.21	1.39	1.23
Au contained	koz	1,677	137	261	230	127	202	168	151	174	144	84
<u>To stockpile</u>	kt	21,861	275	2,991	2,862	3,883	2,340	3,250	1,607	1,914	1,930	808
Au grade	g/t	1.26	1.06	1.64	1.51	1.33	1.12	1.03	1.00	1.17	1.15	1.13
Au contained	koz	889	9	158	139	166	84	108	52	72	71	29
<u>From Stockpile</u>	kt	21,861	0	2,551	3,035	4,400	2,365	3,250	1,607	1,914	1,930	808
Au grade	g/t	1.26	0.00	1.55	1.50	1.40	1.12	1.03	1.00	1.17	1.15	1.13
Au contained	koz	889	0	127	146	198	85	108	52	72	71	29
<u>Total HL feed</u>	kt	60,150	3,042	7,500	7,490	7,463	7,363	7,451	5,402	6,369	5,149	2,921
HL Head grade	g/t	1.33	1.41	1.61	1.56	1.36	1.21	1.15	1.17	1.20	1.30	1.20
Au contained	koz	2,566	137	388	376	325	287	275	202	246	215	113
Stockpile												
Balance	kt	0	275	715	542	25	0	0	0	0	0	0
Au grade	g/t	0.00	1.06	1.76	1.91	1.60	0.00	0.00	0.00	0.00	0.00	0.00
Au contained	koz	0	9	40	33	1	0	0	0	0	0	0

4.3 Blasting and Wall Control

Pit walls will be pre-sheared at a 20-m height with 10-m catchment benches in between. Pre-shear drilling is an effective way to ensure that designed wall angles are achieved. Closely spaced, small-diameter holes, drilled to follow the final contour of the wall, are loaded with light charges, and groups of holes are fired simultaneously, which encourages the formation of a fracture plane between them. Much of the strain energy from production blasts will be reflected back upon meeting this discontinuity, instead of continuing into the wall where it would result in back-break.

For production, 10-m benches will be blasted with an average mixture of 30% ammonium nitrate/fuel oil (ANFO) to 70% emulsion (to account for wet loading conditions), which will be subject to actual operational conditions. ANFO consists of a mixture of ammonium nitrate (AN) prill and fuel oil (94% AN prill and 6% diesel fuel). The decision to adjust the ANFO:emulsion blend is at the engineer's and blaster's discretion during operations. Dry product will be preferred; however, water conditions may require the use of emulsion. The overall powder factor is estimated at 0.26 kg of explosives per tonne of rock. Table 4-4 shows typical blasthole loading by material.

Table 4-4 Typical Blasthole Loading by Material

Heading	Mineralized Material	Waste	Pre-shear
Bench Height (m)	10.0	10.0	20.0
Hole Diameter (mm)	203.2	203.2	76.2
Burden (m)	6.0	6.45	-
Spacing (m)	6.87	7.39	1.5
Sub Drill (m)	1.7	1.7	0.64
Hole Length (m)	11.7	11.7	20.64
Stemming (m)	4.0	4.0	-
Powder Factor (kg/t)	0.27	0.23	0.16

Information regarding type of explosives, safety considerations for transportation and storage of explosives, safe handling procedures, and measures to minimize the release of ammonia and nitrate to the receiving environment are described within the Explosives Management Plan.

Two types of blast hole drilling are proposed:

- Blast pattern drilling to allow for fragmenting of rock for mining; and
- Wall stability control and secondary drilling, as required.

Production drilling is planned to be performed on 10 m benches with drill holes of 203 mm diameter. The typical hole size, burden and depth of explosive plus collar length are shown on Figure 4-18. Hole spacing would be 1.5 m for the pre-strip holes and 7.4 m for all the other hole types. This configuration will be optimized during the mine Construction Phase and/or Operations Phase.

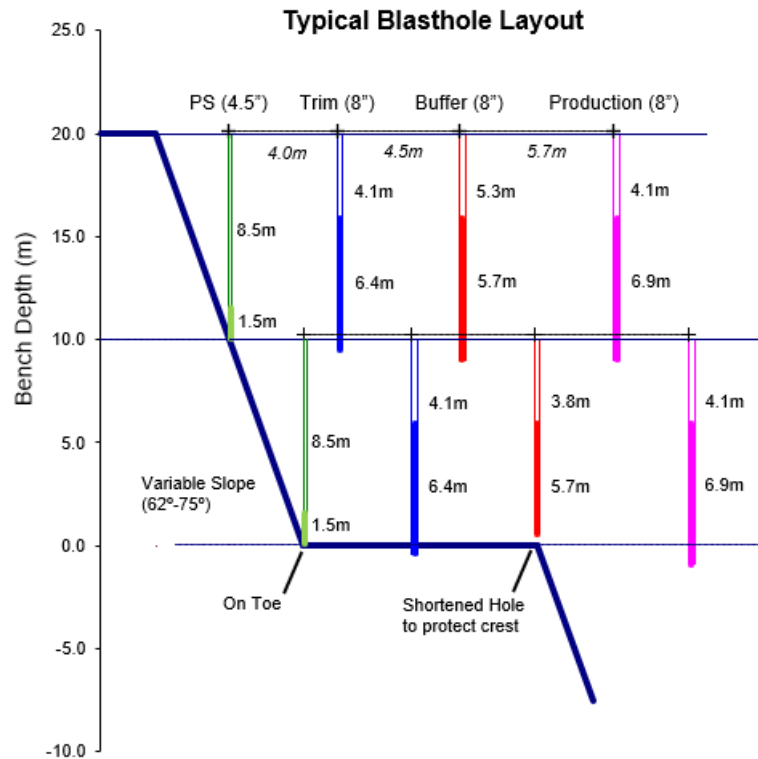


Figure 4-18 Typical Bench Loading Configurations

4.4 Pit Dewatering

Meteoric water collected in active mining areas will be managed using pumping systems or water trucks. The general strategy is to transport water from active mining areas, where in-pit management is not possible, to the closest available storage reservoir for sediment control, if required, until site-specific effluent quality standards are met, and water can be discharged. The available water storage facilities to provide sedimentation control for pit water will vary throughout the life of mine.

During early mine operations, the primary water storage facility on site will be the Alpha Pond and the Facility Pond. During this time, water collected in active pits will be pumped towards Facility Pond, used for dust suppression or makeup water to the HLF, or pumped towards the Alpha Pond. Once active mining concludes in the Double Double Pit and isolated areas of the Latte Pit, water collected from active pit areas may be discharged for storage to any one of the Facility Pond, the Alpha Pond, Double Double Pit, or inactive sections of the Latte Pit.

The specific routing of pit water is mostly dependent on the proximity of the storage reservoir to a given pit area and will be determined on an individual basis based on pump and pipeline availability, water storage capacity in any of the above facilities, and the associated water quality.

Latte Pit and Double Double Pit are both planned to be backfilled later in mine operations. Any water stored temporarily in either pit will be dewatered, to the extent required, to the Facility Pond, which will convey

water towards Alpha Pond, in preparation for the backfilling operation. Double Double Pit will be completely backfilled, while Latte Pit will only be partially backfilled and is expected to be reflooded in support of the development of a pit lake.

Water quality and water levels will be actively monitored in all pit lakes prior to the lakes reaching their final spill point elevations. Corrective actions, such as water treatment, will be pursued in pit lakes that have elevated water quality relative to the effluent discharge limits, in advance of pits discharging to environment. All final discharge locations will be engineered to control erosion in the receiving environment. Additional information regarding pit water management is provided in the Water Management Plan.

4.5 In-pit Haul Road Design

In-pit haul roads are designed based on the largest haulage vehicle being a Komatsu 1500-7 (144 t)-type haul truck. As per Yukon *Occupational Health and Safety Act* (15.43) (YLCO 2017), the running surface width is set to three times the operating width for two-way traffic, or two times the operating width for one-way traffic, and the safety berm height is set to 75% of the diameter of the largest haulage unit tire. Using these inputs, the in-pit haul road width will be, at minimum, 27 m for two-way traffic and 20 m for one-way traffic. Table 4-5 shows the width of each component of the in-pit haul roads. Ramps are designed with a maximum grade of 10% and may be steepened to 12% for final access to deeper portions of the open pits.

Table 4-5 In-Pit Haul Road Design Parameters

Item	Double lane (m)	Single lane (m)
Truck (144 t) operating width	6.9	6.9
Running surface – truck width	20.7	13.8
Berm height (Three-quarters tire height)	2.2	2.2
Berm width at 40° slopes	4.4	4.4
Ditch width	1.9	1.8
Minimum Total Road Allowance	27.0	20.0

Clearly marked emergency runaway lanes or retardation barriers will be provided and maintained at suitable locations per the Yukon *Workers' Safety and Compensation Act* (15.43) (YLCO 2017) and will be capable of safely bringing a runaway vehicle to a stop, where the road grade exceeds 5%.

Mine management will prepare traffic control procedures, showing the maximum allowable speeds for the vehicles in use; rules for passing; stop and yield rules; priority rules for various vehicles; rules for night operation; maximum operating grades; emergency runoff protection; shoulder barriers; and any other information that may be required to ensure the safe operation of all types of vehicles on the Mine Site.

The operator of any unit of mobile equipment shall be directly responsible for its safe operation and for maintaining full control of the unit and complying with all provisions of the *Quartz Mining Act*, the regulations, and the code pertaining to equipment operation. Operators shall wear their seatbelts, drive with their headlights on, and where required, use a flag-equipped whip antenna light or a flashing light at all times.

Traffic rules, signals, and warning signs will be standardized and posted at the Mine Site. Rules governing speed, right-of-way, direction of movement, and the use of headlights to assure appropriate visibility shall be established and followed at the mine.

4.6 Back Haul of Waste Rock

Although the majority of waste rock from the open pits is planned to be deposited in the Alpha WRSF, some suitable waste rock will be used in the construction of Mine Site infrastructure (e.g., site roads, sedimentation embankments, water management structures, laydown areas, heap leach pad, and associated ponds). Additionally, waste rock will be used to selectively backfill mined out pits at Latte, Supremo, Double Double, and Kona to create causeways that shorten the ore haul distance to the crusher system (compared to having to haul material around the pits), and to minimize contact water catchment area. The design criteria for the mine site infrastructure, including the haul roads and waste rock type, are provided in the WROMP.

All equipment and vehicles will be operated by employees who are properly trained and qualified. Communication will be maintained between mining equipment and vehicle operators while working within the open pit area, and all radio communication will be acknowledged before carrying out the task. A minimum of 60 m will be maintained behind heavy mobile equipment for light vehicles, which will increase during low visibility or poor driving conditions. Vehicles will only overtake other vehicles once it has been permitted through radio communication for each vehicle that intends to pass.

Right-of-ways for the haul roads will be indicated by traffic signage; however, the following vehicles will have the right-of-way in the absence of signage (listed in priority):

- Emergency vehicles with lights flashing
- Trucks transporting dangerous goods (Transportation of Dangerous Goods (TDG) signage)
- Haul trucks
- Heavy equipment.

When driving in proximity to equipment, vehicles will be required to approach the equipment from the 10 O'clock position and to maintain a line of sight with the equipment operator.

5.0 REFERENCES

- JDS Energy and Mining (JDS). 2019. Draft Feasibility Pit Slope Stability Update Coffee Gold Project, Yukon Territory, Canada. Report prepared for Newmont Goldcorp Corp. June 07, 2019.
- Northwest Territories Mine Health and Safety Regulations (15.95) (SNWT (1995). *Mine Health and Safety Act*. S.N.W.T. 1994,c.25. In force December 15, 1995; SI-014-95. Accessed from <https://www.canlii.org/en/nt/laws/stat/snwt-1994-c-25/latest/part-1/snwt-1994-c-25-part-1.pdf>
- SRK Consulting (U.S.) Inc. (SRK). 2016a. Feasibility Pit Slope Stability Evaluation, Coffee Gold Project, Yukon Territory, Canada. Report prepared for Kaminak Gold Corporation. March 4, 2016.
- SRK Consulting, Inc. (SRK). 2016. 2015 Geotechnical Field Investigation Report - Coffee Gold Project – Yukon Territory, Canada. January 2016.
- Tetra Tech Canada Inc. (Tetra Tech). 2017a. Fall 2017 Testpitting Program Report (Issued for Use), Coffee Mine Site. December 2017.
- Tetra Tech. 2017b. Summer 2017 Geotechnical Investigation Data Report (Issued for Use), Coffee Mine Site, Coffee Gold Project. November 2017.
- Tetra Tech. 2018. Fall 2018 Testpitting Program Data Report (Issued for Use), Coffee Gold Project. January 2019.
- Yukon Legislative Counsel Office (YLCO). 2017. Transportation of Dangerous Goods (TDG. ST 2017,c.17. Accessed from http://www.gov.yk.ca/legislation/acts/ochesa_c.pdf

APPENDIX A

Mine Operations Plan Table of Concordance

Proponent Commitments Table of Concordance - Mine Operations Plan

Mitigation Name	Reference	Proponent Committed Mitigation	Management Plan	Commitment Incorporated	Section Location	Notes
Backfilling	SIR4-R4-2 (YOR 2017-0211-4259)	Although the majority of waste rock from the open pits is planned to be deposited in the Alpha WRSF, some suitable waste rock will be used in the construction of Mine Site infrastructure (e.g., site roads, sedimentation embankments, water management structures, laydown areas, heap leach pad and associated ponds). As opportunities arise operationally, pits may be selectively backfilled, in addition to the planned backfill locations.	MOP	Yes	MOP Section 4.6: Back Haul of Waste Rock	
Backfilling		Soil and overburden mined from the pit footprints or construction foundations will be stored in the frozen soil storage area (if visible frozen soils present) or one of the soil and overburden stockpiles for use in reclamation or for permanent disposal.	MOP	Yes	MOP Section 3.3.1: Wall Design and Overburden Stability	
Backfilling		Materials placed in the Alpha WRSF will be placed with coarse, durable waste rock at a ratio such that the overall strength of the mixture ensures stability.	MOP	Yes	MOP Section 3.3.1: Wall Design and Overburden Stability	
Backfilling of pits	PP Table 32-1 (YOR 2017-0211-027-1)	Waste rock will be used to selectively backfill mined out pits at Latte, Supremo, and Double Double, to create causeways that shorten the ore haul distance to the crusher system (compared to having to haul material around the pits), and to minimize contact water catchment area.	MOP	Yes	MOP Section 4.6: Back Haul of Waste Rock	
Doré Production and Transport	PP Table 32-1 (YOR 2017-0211-027-1)	On-site production of gold doré bars allowing for frequent and secure transfer via air to offsite refineries for further processing.	HLPF Plan	Yes	HLPF Plan Section 4.3.2.2: Carbon Acid Wash, Elution (Stripping), and Regeneration and Gold Refining	
Geological and Geomechanical Bench Face Mapping	PP Table 32-1 (YOR 2017-0211-027-1)	A thorough geological and geomechanical bench face mapping program will be undertaken beginning in the early stages of development to verify that the geologic structural conditions encountered are consistent with the assumptions and estimates used in the Feasibility Study analyses, and to identify local variations in structural conditions that might increase the risk of localized instabilities. Bench design will be refined, and inter-ramp and overall slope angles optimized based on geotechnical database developed during this program.	MOP	Yes	MOP Section 3.3.1: Wall Design and Overburden Stability	
Geological and Geomechanical Bench Face Mapping	SIR4-R4-2 (YOR 2017-0211-4259)	Geotechnical investigations were undertaken to characterize the subsoil conditions within the Alpha WRSF foundation footprint to inform its designs.	WROMP	Yes	WROMP Section 3.1.1: Foundation Design	
Groundwater Protection and Management	PP Table 32-1 (YOR 2017-0211-027-1)	Rock drains will be installed in the base of Alpha WRSF to allow water flow through the base.	WROMP	Yes	WROMP Section 3.1.3: Rock Drain and Seepage Design	
Groundwater Protection and Management	SIR2-4.1-R2-22 (YOR 2017-0211-411-1)	Additional groundwater modeling runs related to thawing of permafrost, will be completed once the details of the closure phase are optimized and closure design for facilities are better constrained. Results of these groundwater modeling runs can then be used to update the water balance and water quality model.	Water Balance and Water Quality Report	No		The current WBWQM does not currently contain additional runs of the GW model that consider thawing permafrost.
Heat	SIR4-R4-2 (YOR 2017-0211-4259)	As detailed Project engineering related to power generation is advanced, the Proponent will continue to look for opportunities for sustainable power generation including, but not limited to solar, wind, or biomass.	GMDP	Yes	GMDP Section 4.3.5: Power Plant	
In-Pit Sumps	SIR2-3.1-R2-16 (YOR 2017-0211-411-1)	It is expected that any excess water that accumulates in the pits will be pumped to the receiving streams on roughly the same day that it reports to the pit in order to allow mining operations to continue. In-pit sumps will be constructed as required throughout the mine's operations.	MOP	No	MOP Section 4.4: Pit Dewatering	Water will be transported from active mining areas to the closest available storage reservoir for sediment control.
Management of Potential ARD from Kona Pit	SIR1-4.3-R77 (YOR 2017-0211-324-1)	To reduce ARD potential for granite ore to be exposed on Kona pit walls, the pit will be backfilled with non-potentially ARD generating waste rock at closure. Backfilling will occur during winter months to facilitate freezing of the Kona Pit backfill, however, ARD mitigation is not reliant on the re-establishment of permafrost.	WROMP	Yes	WROMP Section 3.2.1: Open Pit Backfill Design	

Proponent Commitments Table of Concordance - Mine Operations Plan

Mitigation Name	Reference	Proponent Committed Mitigation	Management Plan	Commitment Incorporated	Section Location	Notes
Management of Potential ARD from Kona Pit	SIR7-R7-21 (YOR 2017-0211-2174)	All granite waste rock generated from Kona Pit will be sub-aerially disposed of in the Alpha Dump. This will ensure that no granite waste rock is backfilled into mine pits where it may potentially become submerged.	WROMP	Yes	WROMP Section 3.2.1: Open Pit Backfill Design	
Management of Potential ARD in ROM Stockpile	SIR4-R4-2 (YOR 2017-0211-4259)	The waste rock foundation pad will require approximately 3.6Mm3 of material to construct. A 20-m wide safety berm or offset will be left between the crest of the waste rock foundation pad and the toe of the ROM stockpile along the south side where the pad height will reach its maximum fill depth of 50 m.	WROMP	Yes	WROMP Section 3.3.1: ROM Stockpile Design	
Management of Potential ARD in ROM Stockpile	SIR2-3.1-R2-18 (YOR 2017-0211-411-1)	Seepage from the ROM will be monitored as part of the seepage monitoring component of the Surface Water Quality Monitoring Program.	WROMP	Yes	WROMP Section 3.3.1: ROM Stockpile Design	
ML/ARD Seepage Mitigation	SIR1-9.4-R201 (YOR 2017-0211-326-2)	No crushed ore will be left on the stockpile during winter months when ore is not being stacked on the HLF. When active, drainage from the crushed ore stockpile will be collected along with runoff from the ROM stockpile and used as HLF makeup water.	HLPF Plan	Yes	HLPF Plan Section 4.1.2: Crushing Method; Section 4.4: Solution Management	The crushing plant will operate for a minimum of 275 days per year with no heap leach loading during the coldest period of the year; however, this will depend directly on the seasonal conditions each year and the amount of heat available in the HLF for processing. If conditions allow, crushing may be performed year-round.
Road Structures	SIR4-R4-2 (YOR 2017-0211-4259)	The mine site roads, which are designed for light and medium trucks, will be constructed with embankment (rock) fill material with a minimum travelling surface width of 8 m and a maximum grade of 10%. The embankment material will be sourced from infrastructure earthwork activities or from open pit waste material.	GMDP	Yes	GMDP Section 4.8: Mine Site and Haul Roads	
Road Structures	PP Table 32-1 (YOR 2017-0211-027-1)	Existing road (Java or mine access road) connecting the proposed mine site area to the existing exploration camp totaling 25 km which will require upgrades and ongoing maintenance during mine operations.	GMDP	Yes	GMDP Section 4.8: Mine Site and Haul Roads	
Surface Water Protection and Management	PP Table 32-1 (YOR 2017-0211-027-1)	Waste rock benches will be designed to slope inwards from WRSF crest. Runoff will be concentrated along the toe of each bench and prevented from running over the WRSF face by a series of diversion berms.	WROMP	Yes	WROMP Section 3.1.2: Alpha WRSF Design	
Surface Water Protection and Management	SIR1-2.8-R28 (YOR 2017-0211-323-1)	A seepage monitoring program during the Construction Phase is to strategically target potential seepage areas downgradient of the Alpha WRSF as well as within in upper Latte Creek, downgradient of the HLF and the small catchment draining into Latte Creek, south and west of Double Double.	Water Management Plan; EMAMP	Yes	Water Management Plan Section 4.3.8: Alpha Pond EMAMP - Appendix C: Groundwater Monitoring Plan Section 2.3.5: Seepage Monitoring	A seepage monitoring program will be implemented during the Construction Phase to monitor seepage water quality downstream of the Alpha Pond. In addition, the HLF, the downgradient perimeter of the LTF, and the downstream embankments of the landfill will be inspected for evidence of seepage.
Surface Water Protection and Management	SIR4-R4-2 (YOR 2017-0211-4259)	Runoff from Kona Pit will be collected and removed. Therefore, it is not expected that there will be any surface or groundwater interaction between the Alpha WRSF and Kona Pit during operations, until backfill occurs.	Water Management Plan	Yes	WMP Section 3.2.4.3: Kona Pit	Meteoric water collected in the Kona Pit will be pumped towards the Alpha Pond via the Alpha WRSF rock drain. Water may also be pumped towards Event Ponds for water treatment, if it does not meet the Alpha Pond EQS.
Surface Water Protection and Management	SIR1-9.4-R203 (YOR 2017-0211-326-2)	If the evaluation of monitoring and predicted water quality data indicates an increasing trend over time, then mitigation measures can be implemented prior to the pits spilling. Examples of mitigation measures includes in situ treatment of pit lake water, passive biological reactors at the pit spill points, or alteration of pit spill routing to improve water quality within the pit lakes prior to discharge.	Water Management Plan	Yes	WMP Section 4.3.11: Pit Water Management	
Use of Fugitive Waste Heat	PP Table 32-1 (YOR 2017-0211-027-1)	Buildings will be heated primarily by heat recovered from the power plant. Excess heat from the generators, when available, will be used to heat the barren solution.	AQ and GHG Management Plan	Yes	AQ GHG Management Plan Section 7.1.2: Using Fugitive Waste Heat	
WRSF Stability	PP Table 32-1 (YOR 2017-0211-027-1)	The WRSFs will be designed to meet or exceed the acceptable design criteria for short-term conditions during operations (FOS 1.3) and long-term closure criteria (FOS 1.5).	WROMP	Yes	WROMP Section 2.6.2: Stability and Settlement	WRSF will meet acceptable design criteria.

Proponent Commitments Table of Concordance - Mine Operations Plan

Mitigation Name	Reference	Proponent Committed Mitigation	Management Plan	Commitment Incorporated	Section Location	Notes
WRSF Stability	SIR4-R4-1a (YOR 2017-0211-4259)	The crushing plant is planned to operate at an average 28,000 tpd throughput rate, with a maximum of 40,000 tpd. During mine operation, this rate fluctuates as processes and stacking plans are optimized. Current estimates indicate that the crushing circuit will run for approximately 275 days per year; however, this will depend directly on the seasonal conditions each year and the amount of heat available in the HLF for processing. If conditions allow, crushing may be performed year-round.	HLPF Plan	Yes	HLPF Plan Section 2.1: Ore Quantities; Section 4.1.2: Crushing Method	Crushing and stacking will occur at an estimated average rate of 27,000 tpd.
WRSF Stability	SIR1-Appendix 2-A (YOR 2017-0211-327-1)	Slope stability monitoring will be conducted to mitigate risk of slope failure on the Alpha WRSF due to ponding water on upstream of rock drain.	WROMP	Yes	WROMP Section 2.6.2: Stability and Settlement	
WRSF Stability	SIR1-Appendix 2-C (YOR 2017-0211-329-1)	WRSF Performance Monitoring: • Visual Inspections Routine monitoring should be completed on a daily basis by the dump dozer operator to look for and note any signs of cracking at or near the dump crest. In addition, monthly inspections should also be completed by mine staff and including the following: • External slopes for any signs of distress; • Crest of the dump for any signs of cracking; and • Dump and bench toes for any signs of sloughing, deformation, or seepage. Vibrating wire piezometers are recommended to be installed in natural ground immediately within the toes of the dumps to monitor for the development of excess pore water pressures. The final location of the piezometers should be determined upon finalization of the dump designs and be determined by a qualified geotechnical engineer. Instrumentation Monitoring: Piezometer readings should be taken monthly between freshet to the end of the summer season. The threshold warning levels (triggers for action) should be determined following completion of the instrumentation installation and should consider stability analysis results. Surveying Requirements: The crest and toes of the dumps should be surveyed at the completion of each construction lift to compare the as-built geometry to the design surface and to monitor for any deformations within previous lifts. Any significant deviations that could affect dump stability should be brought to the attention of the responsible geotechnical engineer for further evaluation.	EMAMP	Yes	EMAMP - Appendix K: Physical Monitoring Plan Section 3.0: Waste Rock Storage Facility and Stockpiles Methods	

APPENDIX B

Feasibility Pit Slope Stability Update Coffee Gold Project Yukon Territory, Canada

NEWMONT CORPORATION COFFEE GOLD PROJECT



FEASIBILITY PIT SLOPE STABILITY UPDATE

REPORT DATE: AUGUST 9, 2023



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Table of Contents

1	Introduction and Scope of Work	1-1
1.1	Project Background	1-1
1.2	Scope of Work	1-1
2	Geological Setting	2-1
2.1	Property Geology	2-1
2.2	Site Structural Geology	2-3
2.2.1	Metamorphic Foliation	2-4
2.2.2	Jurassic Shearing	2-4
2.2.3	Brittle Fracturing and Faulting	2-4
3	Field Data Collection Program	3-1
3.1	Geotechnical Core Logging	3-2
3.2	Point Load Testing	3-4
3.3	Orientation of Discontinuities	3-5
3.4	Test Pitting Program	3-6
4	Laboratory Testing Program	4-1
4.1	Unconfined Compressive Strength and Elastic Properties	4-1
4.2	Triaxial Compressive Strength	4-2
4.3	Indirect Tensile Strength Testing	4-3
4.4	Direct Shear Strength Testing	4-3
4.5	Unit Weight	4-4
5	Rock Mass Characterization	5-1
5.1	Rock Mass Domains	5-1
5.2	Intact Rock Strength	5-1
5.3	Rock Mass Classification	5-3
5.4	Rock Mass Shear Strength	5-5
5.5	Groundwater	5-7
6	Geologic Discontinuities	6-1
6.1	Structural Domains	6-1
6.2	Kinematic Assessment	6-6
7	Bench Design Considerations	7-1
7.1	Description of Methodology	7-2
7.2	Bench Design Conclusions	7-4

8	Interramp / Overall Slope Stability Analyses	8-1
8.1	Slope Stability Sections	8-2
8.2	Methodology	8-2
8.3	Geomechanical Parameters	8-3
8.4	Results of Interramp/Overall Stability Analysis.....	8-4
9	Pit Slope Design Recommendations.....	9-1
10	Assessment of Future Geomechanical Work.....	10-1
11	Closure	11-1
12	References	12-1

List of Tables

Table 2-1:	Main Rock Units in the Coffee Gold Project Area.....	2-3
Table 2-2:	Tectonic Events at Coffee.....	2-3
Table 3-1:	Summary of Geomechanical Drillholes	3-1
Table 3-2:	Summary of PLT Is(50) Results.....	3-4
Table 4-1:	Average Intact Rock Properties Derived from Laboratory Test Results.....	4-2
Table 4-2:	Direct Shear Test Results.....	4-4
Table 5-1:	Summary of UCS Values Correlated from PLTs	5-3
Table 5-2:	Summary of Rock Mass Classification Data.....	5-4
Table 5-3:	Design Rock Mass Parameters	5-7
Table 6-1:	Supremo East and West Domain Discontinuity Sets.....	6-3
Table 6-2:	Latte and Double Double Domain Discontinuity Sets.....	6-5
Table 6-3:	Kona Domain Discontinuity Sets	6-6
Table 8-1:	Results of Overall / Interramp Slope Stability Modeling	8-4
Table 9-1:	Recommended Pit Slope Design Parameters	9-2
Table 10-1:	Recommended Additional Geotechnical Drillholes.....	10-1

List of Figures

Figure 2-1: Geology in the Supremo, Latte, Double Double, and Kona Areas.....	2-2
Figure 3-1: Location of 2015 and 2018 Geotechnical Drillholes	3-2
Figure 5-1: Correlation of Laboratory UCS and Field Point Load Index Test Results	5-2
Figure 6-1: Supremo Pit Domain and Discontinuity Pole Plots.....	6-2
Figure 6-2: Latte and Double Double Pit Domain and Discontinuity Pole Plots	6-4
Figure 6-3: Kona Pit Domain and Discontinuity Pole Plot.....	6-4
Figure 7-1: Explanation of Bench Design Terminology	7-2
Figure 8-1: Pit Slope Design Components.....	8-1
Figure 8-2: Location of Critical Slope Stability Cross Sections.....	8-2
Figure 9-1: Pit Slope Design Recommendations	9-1

List of Appendices

Appendix A: Geotechnical Core Logs
Appendix B: Laboratory Test Results
Appendix C: Interramp/Overall Stability Analyses

1 INTRODUCTION AND SCOPE OF WORK

Newmont Corporation. (Newmont) requested that JDS Energy & Mining, Inc. (JDS) carry out a supplemental pit slope stability evaluation update for the Coffee Gold Project located in west-central Yukon Territory, Canada. This supplemental pit slope evaluation was carried out to investigate the possibility of increasing pit slope angles in order to reduce waste stripping. The original pit slope evaluation and geotechnical design criteria was carried out by SRK (2016) as part of the project Feasibility Study (JDS 2016).

This report and supplemental pit slope stability evaluation update is primarily based on a Geotechnical Investigations (GI) conducted in 2018 by JDS. A minor supplemental test pitting program was completed across the open pits in 2018 (after the 2018 GI) to provide better estimates of the amount of organics and overburden soils that will be encountered. Relevant details regarding the 2018 test pitting program are summarized in Section 3.4.

Newmont has indicated that, although some updated pit shells have been run more recently, the pit designs assessed in 2018 still remain current.

1.1 Project Background

The Coffee Gold Project is located in the White Gold district, approximately 130 km south of Dawson City and 85 km east of the Alaskan border near the confluence of White and Yukon Rivers. The area consists of low mountainous terrain with steep valleys and rounded ridges, typical of central Yukon. The elevation of the mine site is approximately 1,300 m above sea level.

The Project is currently comprised of four separate open pits (Supremo, Latte, Double Double and Kona), a permanent waste rock storage facility and heap leach pad and supporting infrastructure facilities and roadways. The development is primarily located on a central ridge line between Halfway Creek and Latte Creek, with limited impacts beyond the slope of the ridgeline.

Based on the previous Feasibility Study (JDS, 2016), a nominal ore mining rate of 5 million tonnes per annum would be mined using conventional truck and shovel methods for approximately ten years. A total of 312 Mt of material would be mined to produce 46.4 Mt of ore (5.7:1 strip ratio).

Open pit mining operations will currently use a fleet comprising industry-standard hydraulic shovels, front-end loaders, excavators, and haul trucks. This fleet will be supplemented by appropriately sized drills, graders, and dozers. The fleet selected results in a design ramp width of 27 m.

1.2 Scope of Work

The scope of work for the pit slope stability update included the following:

- Recommendation of the number, location and orientation of core holes believed to be appropriate to update or refine the original SRK (2016) geotechnical design;

- Geotechnical core logging, point load testing and orientation of discontinuities intersecting core recovered from the drillholes in order to characterize pit wall geotechnical conditions;
- Selection of representative drill core samples and management of the laboratory strength testing program;
- Analyses and interpretation of the geotechnical field data and laboratory test results to develop a comprehensive geotechnical model to serve as the basis for slope stability modeling;
- Geotechnical modeling of bench, interramp and overall slopes for the four proposed open pits;
- Assess impact on geotechnical design recommendations of the 2018 test pitting program that was conducted to provide better estimates of the amount of organics and overburden soils; and
- Provide geotechnical design recommendations based on the analyses and compilation of a feasibility-level Pit Slope Stability Evaluation Update report.

2 GEOLOGICAL SETTING

The following summary has been reproduced from Section 7 of the JDS (2016) NI 43-101 Feasibility Study Technical Report for the Coffee Gold Project.

2.1 Property Geology

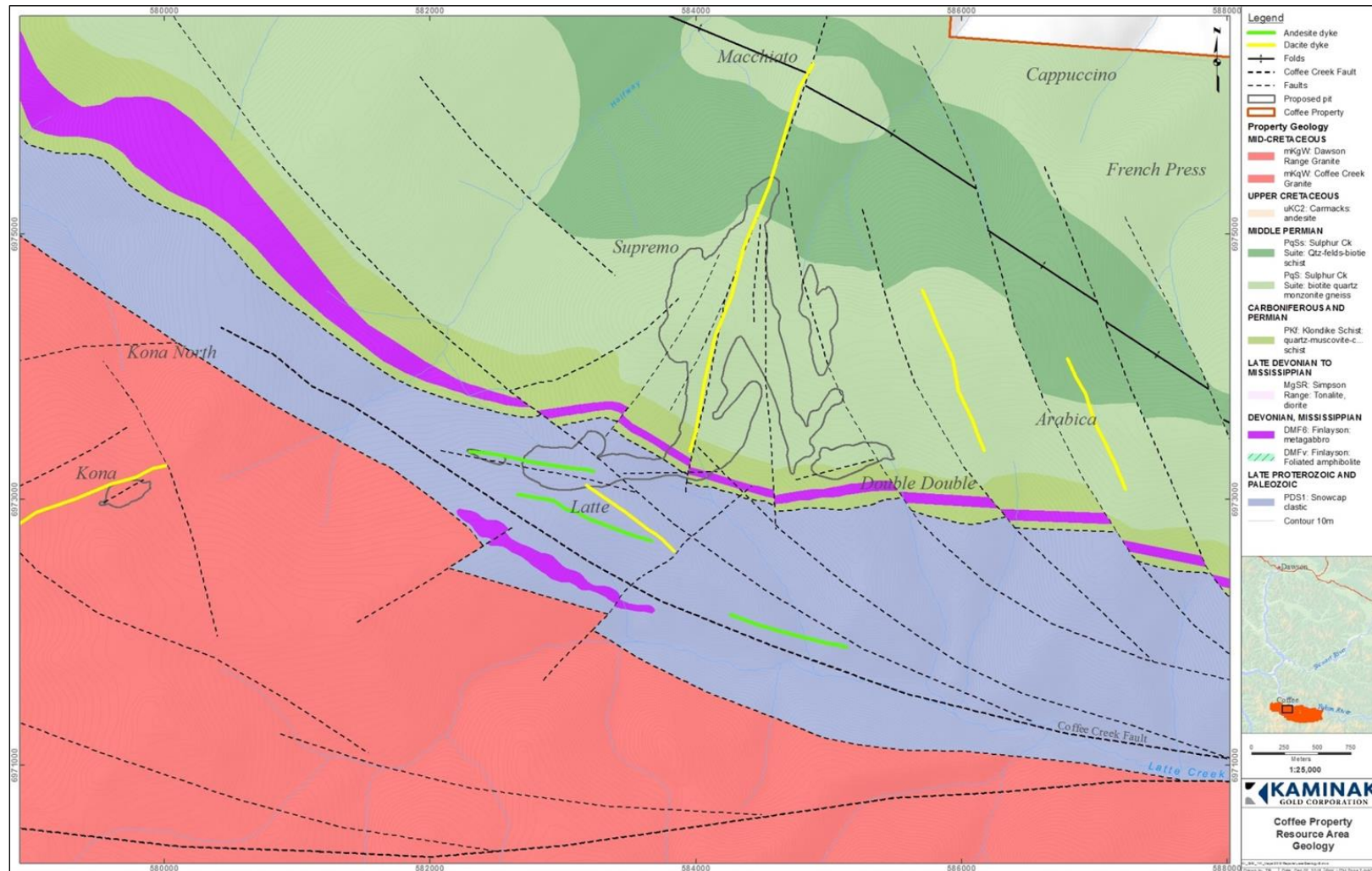
The Coffee Project area is underlain by a package of metamorphosed Paleozoic rocks of the Yukon–Tanana Terrane (YTT) that was intruded by a large granitic body in the Late Cretaceous. The Paleozoic rock package consists of a mafic schistose to gneissic panel which overlies the Sulphur Creek orthogneiss. Both packages form the southwestern limb of a northwest-trending antiformal fold with limbs dipping shallowly to the northeast and southwest.

The schistose and gneissic mafic rock package comprises a thick panel of biotite (+ feldspar + quartz + muscovite ± carbonate) schist with rare lenses of amphibolite which overlies a panel of amphibolite and metagabbro with arc-derived geochemical signatures. Within the schistose panel, slices of 20 m thick serpentinized ultramafic are in tectonic contact with the surrounding rocks. This rock sequence overlies the augen orthogneiss. These rocks are in contact to the southwest with the 98.2 ± 1.3 Ma Coffee Creek granite. Both the Paleozoic metamorphic rocks and Cretaceous granite are cut by intermediate to felsic dykes of andesitic to dacitic composition.

Due to only rare outcrop exposure on the property (<5%), the geological map (Figure 2-1) has been compiled from a combination of geological traverses, bedrock mapping, borehole data, soil geochemistry, and geophysics (magnetic and radiometric).

The magnesium number from soil samples ($Mg\# = Mg/Mg+Fe$) was used to discern mafic from felsic units with the granite being the most felsic, followed by the felsic gneiss. The mafic schist unit was further subdivided into felsic-intermediate schist, biotite schist, amphibolite, and ultramafic rocks (Table 2-1).

Figure 2-1: Geology in the Supremo, Latte, Double Double, and Kona Areas



Source: Grodzicki, K. R., Allan, M. M., Hart, C.J.R., and Smith, T. 2015. Geologic Map of the Coffee Gold deposit area, western Dawson Range, Yukon (MDRU Map M-9)

Table 2-1: Main Rock Units in the Coffee Gold Project Area

Rock Unit	Description
Felsic Gneiss	Variable quartz + feldspar augen + biotite + muscovite. Typical Mg# 2-28. Low in potassium. Host to gold mineralized zones at Supremo.
Biotite Schist	Biotite+/-feldspar+/-quartz+/-muscovite+/- amphibole. Commonly carbonate-rich. High in potassium. Typical Mg# 20 - 40. Locally mylonitic. Host to gold mineralized zones at Latte.
Muscovite Schist	Mainly quartz + muscovite. Typical Mg# 10 - 20. Locally mylonitic.
Biotite Amphibolite	Amphibole + feldspar + biotite. Typical Mg# 20 -40. Biotite and amphibole both Fe-rich. Contains up to 20% biotite.
Amphibolite	Found within the lower mafic footwall. Amphibole + feldspar ± biotite. Typical Mg# 30-50, biotite and amphibole more Mg rich than biotite amphibolite. Contains up to 15% biotite.
Metagabbro/Amphibolite	Interleaved metagabbro with coarse magnesiohornblende + feldspar, and fine-grained, massive amphibolite with >95% magnesiohornblende. Moderate to strong retrogression to actinolite. High Mg content of biotite, amphibole.
Ultramafics	Serpentinite, pyroxenite or listwaenite. Typical Mg# 50 - 73, higher than all amphibolites and metagabbro. Very high in chromium and nickel.
Granite	Coffee Creek granite and Dawson Range batholith. Both are phases of the Whitehorse Plutonic suite and are uranium-rich. Dawson Range batholith higher in Thorium. Both are identifiable using airborne radiometrics.
Dacite Dykes	Quartz + feldspar phenocryst porphyry. Generally, strongly silicified and sericitized. Strong spatial association with mineralized gold zones.
Andesite Dykes	Feldspar phenocrystic. Aphanitic in gold-bearing structures where all original textures are destroyed by intense silicification and sericitization. Strong spatial association with mineralized gold zones.

Source: JDS (2016)

2.2 Site Structural Geology

Rocks at the Coffee Project were deformed by a series of three YTT-wide tectonic events (Table 2-2). Gold mineralization at Coffee occurred during the Cretaceous event.

Table 2-2: Tectonic Events at Coffee

Event	Age	Structures	Mineralization
Extension	Cretaceous	Brittle Fractures	Main Coffee Gold mineralization
		Dextral normal faults	

Event	Age	Structures	Mineralization
YTT-Laurentia Collision	Jurassic	East-west shears and thrust faults	Quartz veining, sericite alteration
		Slices of ultramafic rocks	
Klondike Orogeny	Pre- to late-Permian	Metamorphic gneissosity and schistosity	

Source: JDS (2016)

2.2.1 Metamorphic Foliation

Gneissose and schistose metasedimentary rocks at Coffee contain a shallowly-to-moderately southwest dipping penetrative cleavage (S2 foliation of Berman et al. (2007). The foliation becomes steeper-dipping to the south. Structural data collected from oriented drill core show the following average orientations:

- Supremo: 20° to 40° dip to the south-southwest (190° to 230°);
- Latte: 35° to 55° dip to the south-southwest (180° to 210°); and
- Double Double: 35° to 65° dip to the south-southwest (170° to 200°).

2.2.2 Jurassic Shearing

As the YTT-Laurentia collision continued and the Slide Mountain Ocean was completely closed, the rocks in the Coffee area developed roughly east-west brittle-ductile shears and younger rocks were thrust north over older rocks. This deformation corresponds to the D3 deformation of Berman et al., 2007. This deformation is best seen in the more mafic rocks of the southern schistose panel where intervals of mylonitic rocks are traceable between multiple sections.

2.2.3 Brittle Fracturing and Faulting

Following post-collision uplift and erosion in the YTT, steep-to-vertical brittle fractures and normal faulting affected all lithologies at Coffee. These brittle structures are the hosts to gold mineralization at Coffee. This deformation corresponds to the D5 deformation of Berman et al. (2007). The faults and fractures are splays of the regional Big Creek fault to the southeast of the property. The faults may have locally followed pre-existing Jurassic shear zones. The faults both deflect along the northern edge of the Coffee Creek granite and cut the granite and therefore are syn-to-post granite emplacement (~98 Ma). Younger dacite and andesite dykes intruded into these brittle fractures.

Gold mineralized structures comprise strike-extensive planar zones exhibiting a continuum of deformation intensity from crackle breccia/stockwork fracture systems through to polyphaser

high-energy matrix-supported breccias with intensely altered and reworked clasts. Individual mineralized structures exhibit localized flexures, anastomosing patterns and pinch and swell geometries over scales of tens to hundreds of meters. Overall however, gold mineralization, accompanied by elevated arsenic and antimony, wallrock alteration, deformation intensity, the presence of sub-parallel pre-mineralization dykes, and post-mineral oxidation in the upper 0 to 300 m below surface, display continuity over hundreds of meters in strike and dip, and over 2 km along strike at Supremo T3 and Latte.

Structural measurements of vein orientations and deformation fabrics from oriented drill core provide hard evidence on the structural geometries but are often not available in the mineralized zones due to the disaggregated nature of fractured and often clay-altered core. Where intact core is able to be measured, various structural fabrics from within mineralized zones are used to measure local orientation of mineralization and guide 3D geometric interpretation of mineralization on section and from section to section. Fabrics measured include the dominant fracture orientation, internal fracture or shear fabric, breccia margin, and vein or dyke margin orientation.

The planar gold mineralized zones at Coffee exhibit a number of strike orientations, dominated by east-west, north-south and east-northeast–west-southwest strike directions. Structures typically have sub-vertical dip, with the exception of western Latte which dips 60° to 70° south.

3 FIELD DATA COLLECTION PROGRAM

The 2018 Geotechnical Investigation (GI) was designed with the primary objective of rock mass characterization and discontinuity orientation to augment the previous Geomechanical Model. The 2018 GI was designed very similar to the 2015 GI to allow integration of the two data sets. Both Programs consisted of geotechnical core logging and discontinuity orientation, point load testing and laboratory rock strength testing. The 2018 GI also included downhole surveying of discontinuities with a televiewer. This section focuses on details of the 2018 Geotechnical Investigation. Additional details regarding the 2015 Geotechnical Investigation are contained in SRK, 2016.

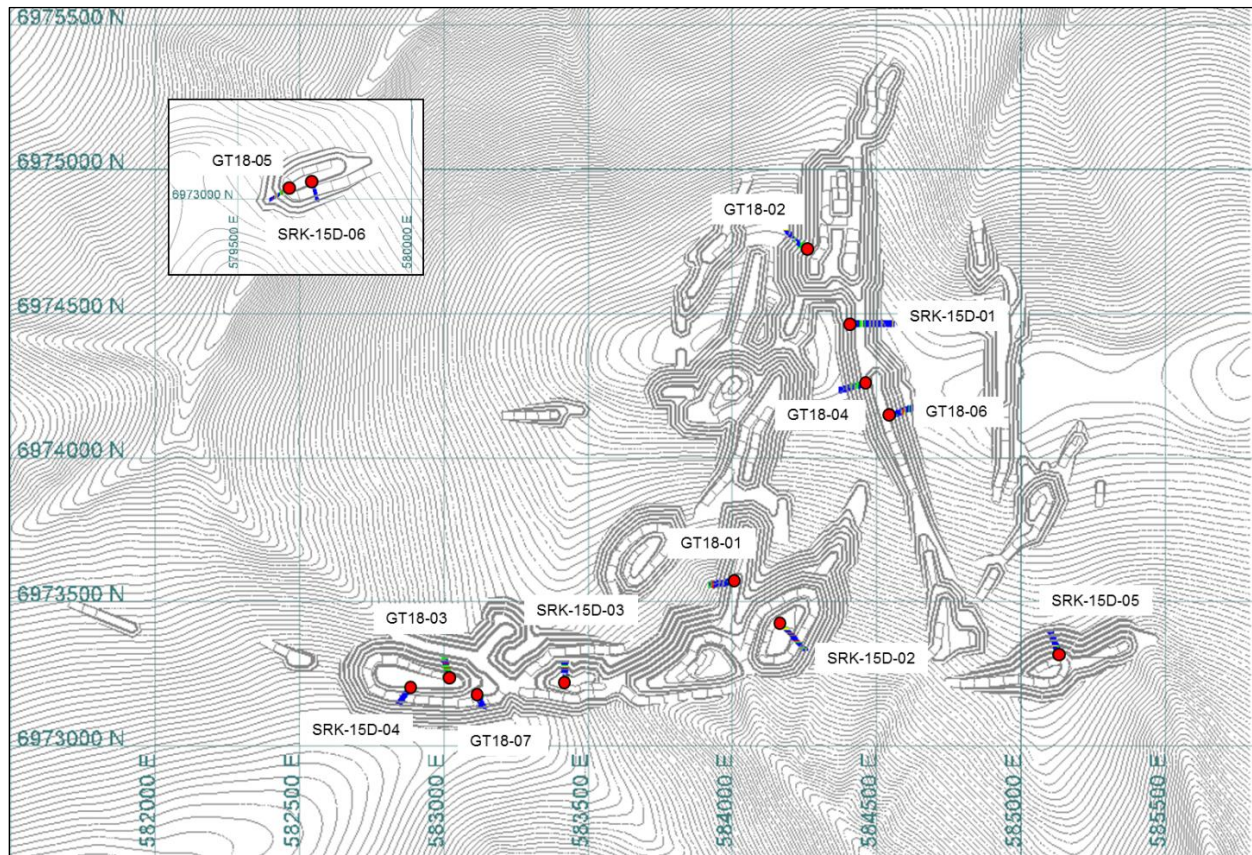
A total of 13 HQ3 diameter core holes were logged and tested between the two programs totaling 2,105 m in length. Drillhole inclinations of approximately 60° below horizontal were targeted since they were judged to be more likely, than would vertical holes, to intersect geologic structures such as joints and faults which would most adversely influence pit slope stability. Collar locations and drillhole orientations of the combined 13 geotechnical specific drillholes from the 2015 and 2018 Programs are summarized in Table 3-1 and shown on Figure 3-1. The geotechnical drillholes targeted the final pit walls. Due to the narrow width of the mineralization and pits, very few, if any, phased pit walls will exist.

Table 3-1: Summary of Geomechanical Drillholes

Program	Hole ID	Deposit	Easting (UTM)	Northing (UTM)	Elevation (masl)	Depth (m)	Azi. (°)	Inc. (°)
2015	SRK-15D-01	Supremo	584,411	6,974,465	1,256	245	090	-51
	SRK-15D-02	Supremo	584,168	6,973,424	1,031	215	137	-52
	SRK-15D-03	Latte	583,421	6,973,227	1,098	110	359	-50
	SRK-15D-04	Latte	582,881	6,973,203	1,099	152	264	-60
	SRK-15D-05	Double Double	585,127	6,973,328	1,106	134	340	-55
	SRK-15D-06	Kona	579,716	6,976,046	1,262	100	340	-55
2018	GT18-01	Supremo	584,000	6,973,570	1,053	175	263	-61
	GT18-02	Supremo	584,243	6,974,730	1,251	192	316	-62
	GT18-03	Latte	583,016	6,973,249	1,094	155	346	-65
	GT18-04	Supremo	584,459	6,974,257	1,252	196	256	-61
	GT18-05	Kona	579,628	6,973,019	1,276	105	239	-65
	GT18-06	Supremo	584,552	6,974,146	1,237	180	069	-61
	GT18-07	Latte	583,122	6,973,169	1,117	145	161	-72

Source: JDS (2019)

Figure 3-1: Location of 2015 and 2018 Geotechnical Drillholes



Source: JDS (2019)

3.1 Geotechnical Core Logging

Geotechnical logging, field point load testing and orientation of discontinuities intersected by core recovered from a total of seven drillholes were conducted to support this update. Drilling was carried out using 1.5 m long, HQ3 diameter triple tube core barrels to preserve the in situ condition of the rock to the largest extent possible. To minimize disturbance, core retrieved from the seven holes was logged on a 24-hour per day basis, at the rig, in the liners prior to boxing and transporting.

The geotechnical core logging program was designed to provide information pertinent to pit slope stability evaluation, such as geologic contacts, profiles of rock strength, and characterization and frequency of discontinuities. Specific parameters that were logged included:

- General lithology and structures;

- Total core recovery;
- Rock Quality Designation (RQD);
- Rock weathering/alteration and intact strength indices;
- Frequency of discontinuities;
- Discontinuity characteristics (type, roughness, infillings and wall condition);
- Micro-defect intensity and strength; and
- Discontinuity orientation (when possible).

Core retrieved from the geotechnical drillholes was logged at the rig by trained geologists or geotechnical engineers before transportation to the core logging facility. Geotechnical markings on the core were made using a standardized system. Care was taken to exclude fractures that were caused during drilling and handling as the inclusion of such would unnecessarily lower rock quality classifications. Geotechnical core logs are presented in Appendix A.

Information regarding major geologic structures, such as faults and shear zones, was also obtained during the logging program. Major structures, as observed in core, often include heavily fractured or broken zones, sheared zones and/or sections of gouge. These large-scale structures are weakening features that are captured separately from the basic logging parameters. They are commonly widely spaced and can serve as a reference to define geotechnical zones or domains. The following information was recorded for each major structure:

- Depth of the feature;
- A quality descriptor of the feature (jointed, broken, sheared or gouged);
- The number of joints or fractures attributed to the feature; and
- The orientation or angle of the structure with respect to the core axis (when possible).

During core logging, representative core samples were selected from each drillhole to provide specimens for laboratory strength testing. Samples were collected at approximately 15 m intervals, or when significant rock type or strength changes were apparent. Each sample was sealed and safely stored at the time of collection. Upon completion of the drilling, test samples were shipped to the Queen's University Rock Mechanics Laboratory in Kingston, Ontario for testing. Details of the laboratory testing program are contained in Section 4.

Following the completion of geomechanical logging and sampling, each of the geotechnical drillholes was logged for lithology and alteration by Goldcorp (previously Kaminak) geologists to assure the best possible correlation of the geotechnical data with the current Goldcorp Geologic Model.

3.2 Point Load Testing

Point load testing provides a quick, inexpensive method of relative strength testing of rock core that can be conducted in the field allowing a large number of tests to be carried out. Although not a precise measurement of rock strength, this method typically provides a more accurate estimation of variability compared to laboratory testing as smaller and weaker samples can be immediately tested at their natural moisture content and without potential disturbance that can occur during shipping.

Point load tests (PLT) were routinely conducted during the 2018 core logging program at a frequency of approximately one test per 1 m using a Matest A125N test machine, thereby providing detailed and nearly continuous profiles of relative rock hardness. In addition to the routine tests, approximately three additional tests were also conducted on core immediately adjacent above and below each uniaxial compressive strength (UCS) sample obtained for laboratory testing. The pairing of PLT results and UCS samples was undertaken to permit estimation of a correlation factor for conversion of the field PLT tests to laboratory UCS values as discussed in Section 5.2. PLTs were conducted according to International Society for Rock Mechanics (ISRM) procedures (ISRM, 1985).

A total of 1,242 point load tests were conducted on core from the seven, 2018 geotechnical drillholes; of those, 853 (69%) met test criteria for valid test results. Point load indices ($Is(50)$) were calculated from the field PLT data using the ISRM (1985) suggested method. The combined 2015 and 2018 PLT results are summarized in Table 3-2 for each primary rock type and oxidation group. Only tests considered valid per ISRM standards, i.e., those tests in which the core did not break along pre-existing weakness, were utilized for the analysis. Invalid or failing tests that broke along a pre-existing defect such as a healed joint or foliation plane were considered separately and used as an indicator of the potential strength anisotropy of the rock types.

Table 3-2: Summary of PLT $Is(50)$ Results

Pit	Lithology	Oxide Zone	No. of Tests	$Is(50)$ (MPa) ¹			
				Average	Std. Dev.	Min.	Max.
Supremo	Gneiss (MXF/MxM/FG)	Oxide	16	3.7	1.9	0.5	7.4
		Upper Trans	12	5.4	2.3	0.6	9.2
		Middle Trans	54	5.5	2.3	0.8	11.9
		Lower Trans	94	6.2	2.3	0.5	10.1
		Fresh	260	6.6	2.2	0.2	12.5
Double Double	Gneiss (MXF/MxM)	Oxide	-	-	-	-	-
		Upper Trans	-	-	-	-	-
		Middle Trans	3	2.2	0.4	1.7	2.5
		Lower Trans	11	2.7	1.2	1.3	4.5
		Fresh	14	4.1	2.4	0.5	7.9
	Schist (BtS)	Fresh	20	4.3	2	1.6	8.1

Pit	Lithology	Oxide Zone	No. of Tests	IS ₍₅₀₎ (MPa) ¹			
				Average	Std. Dev.	Min.	Max.
Latte	Schist (BtS/Bts_carb)	Oxide	1	3.8	-	3.8	3.8
		Upper Trans	10	3.7	1.7	1.1	7.6
		Middle Trans	21	3.4	1.2	1.7	5.9
		Lower Trans	48	3.3	1.8	0.3	7.8
		Fresh	49	2.7	1.7	0.3	9.4
Kona	Granite (GG)	Oxide	-	-	-	-	-
		Upper Trans	3	4	0.5	3.6	4.4
		Middle Trans	5	3.7	1.2	2.3	4.9
		Lower Trans	8	5	0.9	3.9	6.2
		Fresh	26	5.6	1.7	1.9	8.8

Source: JDS (2019)

3.3 Orientation of Discontinuities

An ACTIII Core Orientation System manufactured by Reflex Instruments was used to attempt to orient the discontinuities in each run for both the 2015 and 2018 Programs. The depth, alpha angle, and beta angle were measured for each discontinuity on all core runs that were successfully oriented. The beta angle, i.e., the angle from the lowest part of the ellipse formed by the intersection of each discontinuity with the core, was measured from the bottom of the core in a clockwise direction when looking down hole. The alpha angle was also measured and is defined as the maximum angle made by the discontinuity with respect to the core axis.

It was possible to orient a total of 1,924 discontinuities out of the total 3,479 (55%) discontinuities logged for the seven, 2018 geotechnical holes, excluding “added” joints where a frequency of 4 joints per 10 cm was added to account for intensely fractured or rubbelized zones where counting of discrete joints was not possible. For the 2015 Program it was possible to orient a total of 1,581 discontinuities out of the total 2,158 (73%) discontinuities logged for the six geotechnical drillholes, excluding “added” joints.

Discontinuity orientations and conditions for the 2018 Program were also characterized using downhole optical (OTV) and acoustical (ATV) televiewing. DGI Geoscience Inc., of Toronto, ON completed the surveys which collected the depth, in situ orientation, width (mm) and an inferred type of discontinuity. A total of 5,811 discontinuities were identified by the televiewer between the seven 2018 drillholes (although only 1,190 (20%) had open apertures). Although similar, the ATV data set appears more consistent and reliable than the oriented core data sets and was therefore relied upon mostly for analyses of the 2018 data.

In addition to discontinuity orientations obtained from the 13 geotechnical specific drillholes listed above, a significant percentage of the diamond resource drillholes were also oriented by the previous owner, Kaminak Gold Corporation using the ACT orientation tool. These drillholes were

used to augment the geotechnical specific drillhole data set where possible. Additional details regarding the resource drillhole orientation database are contained in SRK, 2013 SRK, 2016.

3.4 Test Pitting Program

A supplemental test pitting program was completed across the open pits in 2018 (after the 2018 GI) to provide better estimates of the amount of organics and overburden soils that will be encountered. A total of 28 test pits were excavated to bedrock across the Supremo, Latte and Double-Double open pits. Results indicate that the depth of organics typically ranges from 0.2 to 0.7 m with up to approximately 2.5 m of overburden soils beneath. The materials were mostly unfrozen or ice-poor but materials with visible ice were encountered locally. Additional details regarding the test pitting program are summarized in the Tetra Tech (2019) report.

JDS has reviewed the results of this 2018 test pitting program, and given the limited organic and overburden depths encountered, that the results and recommendations of the 2018 analyses are not impacted.

4 LABORATORY TESTING PROGRAM

A total of 73 laboratory tests were performed on core samples selected to represent the range of the rock conditions observed in core from the 2018 Geotechnical Investigation. The overall laboratory testing consisted of uniaxial compressive, triaxial compressive, indirect tensile (Brazilian) and natural joint direct shear strength tests, as well as measurements of unit weight and elastic properties. The 2018 laboratory testing was carried out by the Queen's University Rock Mechanics Laboratory located in Kingston, Ontario. Raw laboratory test data are included in Appendix B. An additional 59 tests were carried out by Agapito Associates Inc. located in Grand Junction, Colorado and the University of Arizona Rock Mechanics Laboratory in Tucson, Arizona as part of the 2013 and 2015 test programs.

After completion of the laboratory testing program, before and after photographs of the test samples were forensically reviewed to confirm validity of test results. The type and quality of break were assessed through the sample photographs. Test samples that broke through a pre-existing structure or fabric and not through intact rock were discounted from the analyses, where possible. Due to the relatively high number of schist samples that broke along the schistose fabric or foliation, most of the strength values summarized below are influenced to some extent by the fabric/foliation.

4.1 Unconfined Compressive Strength and Elastic Properties

The uniaxial compressive strength (UCS) test is the most commonly used measure of intact rock strength and is one of the primary inputs to basic rock mechanics classifications and strength criteria such as rock mass rating systems and the Hoek-Brown (Hoek, et al., 2002) shear strength criteria, both of which are discussed in detail in Section 5.

UCS testing involves the application of a steadily increasing axial load upon a core sample with a length-to-diameter (L/D) ratio of, ideally, between 2.0 and 2.5. The uniaxial compressive strength of the sample is the applied load that produces failure divided by the cross-sectional area of the core.

For selected UCS tests, strain gauges were applied to the samples to monitor longitudinal and lateral strains produced by the axial loading. The elastic properties are derived from the strain gauge output; specifically, Young's Modulus (E) is the ratio of the vertical stress to the longitudinal strain, while Poisson's Ratio (ν) describes the relationship between the lateral strain and the longitudinal strain.

UCS testing was conducted on a total of 65 samples between the 2013, 2015 and 2018 testing programs according to ASTM Method D7012. Elastic properties (Young's Modulus and Poisson's Ratio) were measured for 20 of the 65 UCS samples. Young's moduli were also calculated for an additional 15 triaxial compressive strength tests as discussed below. The average laboratory UCS results for the combined 2015 and 2018 programs are summarized in Table 4-1 by primary rock type and oxidation zone, along with other laboratory test results.

Table 4-1: Average Intact Rock Properties Derived from Laboratory Test Results

Lithology	Oxide	γ (kN/m ³) ¹	BTS ¹ (MPa)	UCS ^{1,2} (MPa)	E _i (GPa) ¹	ν ¹	m _i	σ_{ci} ² (MPa)
Gneiss	All	25.4 (56)	11.9 (19)	81 (31)	26 (22)	0.25 (11)	18	89
	Oxide	24.7 (2)	-	67 (2)	-	-		
	Transition	25.4 (19)	8.4 (5)	77 (12)	30 (7)	0.26 (5)		
	Fresh	25.5 (35)	13.2 (14)	82 (18)	24 (15)	0.25 (6)		
Schist	All	25.4 (31)	7.7 (8)	69 (13)	16 (10)	0.24 (6)	19	54
	Oxide	26.1 (3)	-	58 (2)	12 (1)	0.11 (1)		
	Transition	24.6 (15)	4.6 (5)	66 (6)	16 (7)	0.26 (5)		
	Fresh (valid tests)	26.3 (31)	12.7 (3)	76 (5)	19 (2)	-		
	Fresh (all tests)	26.3 (31)	12.7 (3)	51 (9) ³	19 (2)	-		
Granite	All	24.6 (8)	7.1 (3)	70 (6)	29 (3)	0.19 (3)	-	-
	Oxide	-	-	-	-	-		
	Transition	23.9 (4)	2.6 (1)	41 (4)	15 (2)	0.14 (2)		
	Fresh	25.2 (4)	9.4 (2)	130 (2)	55 (1)	0.30 (1)		

Source: JDS (2019)

Notes:

¹ Numbers in parentheses indicate the number of test results available to calculate the average intact rock parameters.

² 'UCS' indicates lab UCS test results while ' σ_{ci} ' represents the intercept of the triaxial strength envelope with the σ_3 axis.

³ Excludes breaks where the sample broke along a pre-existing weakness such as healed joints or foliation.

4.2 Triaxial Compressive Strength

Triaxial compressive strength (TCS) tests were conducted to calculate the Hoek-Brown (Hoek, et al, 2002) material coefficient m_i as discussed in Section 5.4. The TCS test involves encasing a core sample in an impervious membrane and subjecting the sample to a selected confining pressure (σ_3) while the sample is loaded axially (σ_1) until failure occurs. The applied load that results in failure divided by the cross-sectional area of the core is the triaxial compressive strength, referenced to the appropriate confining pressure.

Multi-stage TCS tests were conducted on 15 samples (46 individual test points) from the 2018 GI using ASTM Method D7012. Young's moduli were also calculated from each TCS. The samples were tested at confining pressures ranging between 0.5 and 20 MPa. The maximum confining pressure was selected to be approximately of one-half of the UCS values, as recommended by Hoek and Brown (1997). The raw TCS testing data are contained in Appendix B.

Regression analyses were conducted on the triaxial test data set for each rock type using RocData software (Rocscience 2017) to estimate m_i values. Values of 18 and 19 were estimated for the Gneiss and Schist rock types respectively. Insufficient data was available to develop a

triaxial curve for the Kona Granite due to the shallow pit depth and limited number of samples obtained from the geotechnical drillholes.

4.3 Indirect Tensile Strength Testing

The indirect (Brazilian) tensile strength (BTS) test measures the tensile strength of the intact rock by applying pressure to opposite sides of a relatively thin disc sample, similar to a diametral field point load test, but in a controlled laboratory environment. The BTS test results provide some level of verification of field point load testing values and, in conjunction with UCS data, the determination of the Hoek-Brown (Hoek, et al., 2002) m_i value as discussed in Sections 5.4.

A total of 37 BTS tests were conducted according to ASTM Method D3967 between the 2013, 2015 and 2018 Geotechnical Investigations. The average laboratory BTS test value are shown by rock type and oxidation group in Table 4-1 along with the other laboratory test results.

4.4 Direct Shear Strength Testing

Direct shear testing is commonly used for estimating the expected shear strength along natural rock discontinuities such as joints, bedding or foliation planes and faults. Since the stress levels developed within open pits are usually much lower than the rock substance or intact strength, displacement frequently occurs along pre-existing geologic discontinuities, making the determination of discontinuity shear strength a necessity.

The direct shear test involves the application of a load perpendicular (normal) to a discontinuity separating two intact blocks of rock, with the simultaneous application of a shear load parallel to the discontinuity of sufficient magnitude to induce displacement of the blocks relative to each other. To define the overall shear strength envelope, tests are conducted at three or more normal stresses, with continuous displacement/shear stress data being obtained at each of the normal loads. For each normal load, the peak (maximum) and residual (steady state relative to displacement) shear stresses are recorded, thereby defining the peak and residual shear strengths given a normal stress. The relationship between an applied normal stress and the resulting shear strength defines a point on the shear strength envelope. Peak and residual shear strength envelopes can then be determined from the shear strength/normal stress points using statistical regression methods.

A total of 20 core samples were selected between the 2013, 2015 and 2018 drillholes for four-point, small scale direct shear (SSDS) tests in accordance with ASTM Method D5607. Natural fractures preserved in the field were used for all of the direct shear tests. The range of normal stresses applied during testing, approximately 170 to 4,880 kPa, was selected to approximate the in situ stresses that are expected to develop within the slopes.

In order to fit a shear strength envelope to the laboratory data points, a linear or curvilinear regression analysis is typically conducted. For a linear fit, the envelope is presented according to the Mohr-Coulomb, or linear, criterion, i.e., in the form of an angle of internal friction (Φ), which corresponds to the inverse tangent of the slope of the least-squares regression line, and an apparent cohesion (c), which corresponds to the shear strength intercept at zero normal stress. Discontinuity shear strengths derived from the direct shear tests are summarized by rock type

and fracture type in Table 4-2 for the combined 2015 and 2018 testing programs. Raw test data is contained in Appendix B. Cohesion values listed in Table 4-2 were obtained directly from linear regression analysis of the raw laboratory test data and, in some cases, are considered unrealistically high. Joint and foliation strength cohesions were typically reduced to 10 kPa for the subsequent stability analyses as discussed in Section 5.4.

Table 4-2: Direct Shear Test Results

Fracture Type	No.	ϕ (°)	Cohesion (kPa)
Schist Foliation	7	29	28
Gneiss Foliation	4	31	47
Gneiss Joint	6	29	40
Schist Joint	1	29	53
Granite Joint	1	38	53

Source: JDS (2019)

4.5 Unit Weight

The unit weight or density of each material type is required for slope stability modeling as both the resisting and driving forces of potential instabilities are functions of weight (or stress). Prior to actual testing of rock core samples, sample dimensions and weights were measured and used to calculate total unit weights for each sample. The combined data set includes 96 measurements between the 2015 and 2018 programs and ranges from 22.6 to 29.5 kN/m³ with a mean value of 25.9 kN/m³, for all samples. The average unit weights for each rock type and oxidation zone are summarized in Table 4-1 along with the previously discussed intact rock properties.

5 ROCK MASS CHARACTERIZATION

Rock mass models are developed to provide a framework for interramp / overall slope stability modeling by mathematically simulating site geotechnical conditions. The term “rock mass” refers to the entire body of rock, including discontinuities. In contrast, “intact rock” or “substance strength” refers to the rock between discontinuities in a rock mass. A typical rock mass model is heavily influenced by site geologic information such as lithologic, alteration and/or structural models.

5.1 Rock Mass Domains

Geotechnical models consist of individual domains, or volumes of rock, each of which comprises materials exhibiting similar geotechnical properties, such as rock mass strength, porewater pressures and orientation of geologic structures. Pertinent geotechnical parameters are assigned to each domain, based on engineering properties that are determined during field and laboratory data collection programs.

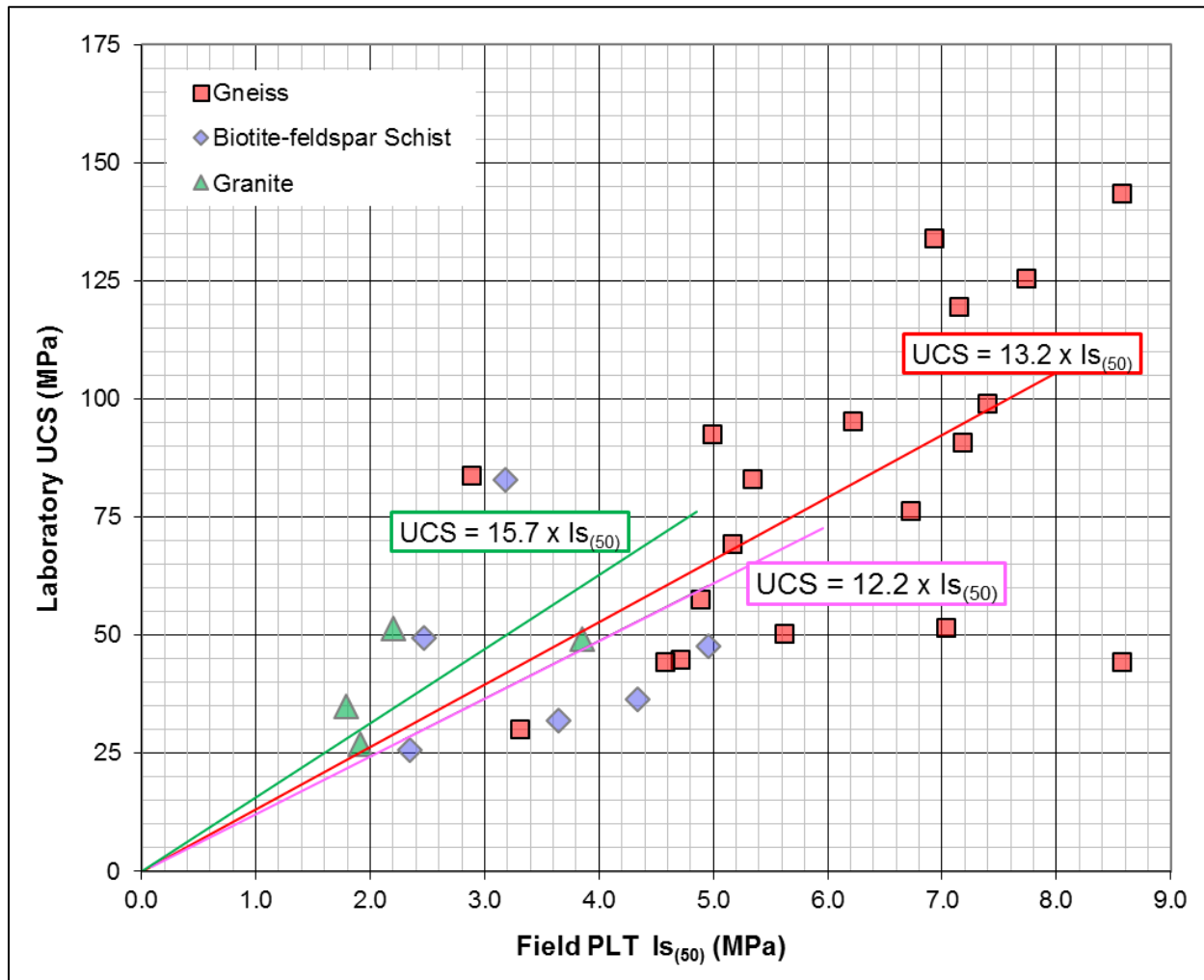
Following the completion of the 2018 GI the data was compiled with the previous 2015 GI data to form a complete database for analysis. The geotechnical properties were first analyzed by primary rock type and then oxidation group for each drillhole and then geotechnically similar rock types were combined to form geotechnical or rock mass domains. It was concluded from the analyses that the, while the three primary lithologies (i.e. schist, gneiss or granite) were generally similar in their characteristics, rock mass quality may be best delineated by the various oxide zones within each lithology. Additional information regarding geologic structure and structural domains is contained in Section 6.

5.2 Intact Rock Strength

The term “intact rock” or “substance strength” refers to the rock between discontinuities in a rock mass; in contrast, “rock mass” refers to the entire body of rock, including discontinuities. Intact rock strengths were assessed in the field by logging the ISRM (1978) qualitative ‘hardness’ index for each core run and by conducting field PLTs according to the ISRM (1985) recommended procedure, as discussed in Section 3.2. Samples of rock core were also selected for subsequent laboratory UCS, TCS and BTS tests described in Section 4.

Each laboratory UCS test sample was paired with adjacent PLT $I_s(50)$ values in order to estimate a correlation coefficient enabling approximations UCS from the field PLT results. Figure 5-1 contains a plot of the 2018 UCS vs. PLT $I_s(50)$ correlation for each of the three primary rock types. The approximation of UCS values for each of the field $I_s(50)$ tests permitted the development of essentially continuous profiles of intact rock strength down each drillhole.

Figure 5-1: Correlation of Laboratory UCS and Field Point Load Index Test Results



Source: JDS (2019)

The strength profile allowed weaker zones to be identified that are not commonly sampled for laboratory testing due to either difficulties in obtaining solid intact samples and/or the susceptibility of the samples to damage during shipping. The PLT's also provide a check against the IRS hardness values recorded by the loggers. Table 5-1 contains a summary of UCS values correlated from PLTs by rock type and oxide zone for the combined 2015 and 2018 geotechnical investigations.

Laboratory UCS and field PLT $I_{s(50)}$ results are plotted with the field estimates of intact rock strength or hardness on the geotechnical core logs presented in Appendix A. As illustrated on the logs presented in Appendix A, the PLT derived UCS strengths are generally consistent with the values and variation of the laboratory UCS and the intact rock strength estimates recorded

by the loggers. In some cases the field logging estimates were higher than the laboratory and PLT derived UCS values.

Table 5-1: Summary of UCS Values Correlated from PLTs

Pit	Rock Types	Oxide Zone	No. of Tests	UCS Converted from PLT (MPa) ¹			
				Average	Std. Dev.	Min.	Max.
Supremo	Gneiss (MXF/MxM/FG)	Oxide	16	49	25.1	7	97
		Upper Trans	12	71	29.9	7	120
		Middle Trans	54	73	30.8	11	156
		Lower Trans	94	81	30.5	6	133
		Fresh	260	87	28.6	3	164
Double Double	Gneiss (MxF/MxM)	Oxide	-	-	-	-	-
		Upper Trans	-	-	-	-	-
		Middle Trans	3	29	5.6	22	33
		Lower Trans	11	35	16.2	17	59
		Fresh	14	54	31.5	7	103
	Schist (BtS)	Fresh	20	92	43.6	34	176
Latte	Schist (BtS/Bts_carb)	Oxide	1	46	-	46	46
		Upper Trans	10	58	28.8	23	108
		Middle Trans	21	42	14.6	20	72
		Lower Trans	48	53	27.7	7	124
		Fresh	49	45	27.6	7	127
Kona	Granite (GG)	Oxide	-	-	-	-	-
		Upper Trans	3	62	7.2	56	70
		Middle Trans	5	60	20.1	36	85
		Lower Trans	8	87	15.9	67	107
		Fresh	26	97	30.1	33	151

Source: JDS (2019)

Notes:

¹ Excludes breaks where the sample broke along a pre-existing weakness such as healed joints or foliation.

5.3 Rock Mass Classification

Core logging and laboratory test data obtained from the 2015 geomechanical drillholes was used for empirical rock mass characterization according to the Bieniawski (1989) Rock Mass Rating

(RMR) system. The RMR system consists of five primary parameters: unconfined compressive strength (UCS), rock quality designation (RQD), discontinuity spacing, joint conditions (Jc) and groundwater. Dry conditions were assumed for the RMR calculations as groundwater pressures are commonly accounted for during slope stability analysis using effective stress type analyses. Similarly, the RMR values were not discounted for adverse joint set orientations joint orientations. The RMR values, with the maximum possible (or most competent rock mass) being 100 for each run, are displayed on the geomechanical core logs presented in Appendix A.

Overall, RMR characterization provides a useful starting point for determining trends in overall rock mass quality and delineating geomechanical domain. However, it is important to note that when combining or averaging the individual RMR parameters over a distance such as a core run interval, important small-scale details such as weak features in the core can become obscured or hidden by the overall RMR value calculated for the interval. For this reason it is important that major structures or weak features observed in the core are logged and analyzed independent of the combined RMR data as was done for the geomechanical drillholes.

Results of the rock mass characterization suggest that, with the exception of the oxide materials which will be mostly mined and processed, the rock mass at Coffee is generally of 'Fair' to 'Good' geomechanical quality according to the Bieniawski (1989) criteria. Table 5-2 contains a summary of rock mass characteristic data derived from the combined 2015 and 2018 GI for each of the three primary rock types and oxide zones.

Table 5-2: Summary of Rock Mass Classification Data

Pit	Lithology	Oxide Zone	No. of Tests	RMR (Bieniawski, 1989)			
				Average	Std. Dev.	Min.	Max.
Supremo	Gneiss (MXF/MxM/FG)	Oxide	27	59	9.9	39	76
		Upper Trans	32	61	11.1	35	77
		Middle Trans	87	61	9.1	34	78
		Lower Trans	180	66	11.2	34	91
		Fresh	391	69	10.8	31	91
Double Double	Gneiss (MxF/MxM)	Oxide	-	-	-	-	-
		Upper Trans	-	-	-	-	-
		Middle Trans	5	58	6.3	50	68
		Lower Trans	13	66	11.5	42	82
		Fresh	20	62	8.7	48	80
	Schist (BtS)	Fresh	21	64	9.7	42	93
Latte	Schist (BtS/Bts_carb)	Oxide	28	54	8.2	32	73
		Upper Trans	16	56	13.0	36	78
		Middle Trans	34	69	9.5	37	83
		Lower Trans	100	67	10.5	33	91
		Fresh	90	61	10.2	28	82

Pit	Lithology	Oxide Zone	No. of Tests	RMR (Bieniawski, 1989)			
				Average	Std. Dev.	Min.	Max.
Kona	Granite (GG)	Oxide	-	-	-	-	-
		Upper Trans	10	58	16.0	30	77
		Middle Trans	27	69	8.1	38	77
		Lower Trans	6	77	9.2	61	85
		Fresh	34	77	8.1	81	93

Source: JDS (2019)

The values in Table 5-2 were calculated excluding the upper, surface weathered materials. Soil coverage is very minimal in the pit areas. The depth of soils and heavily weathered bedrock is typically between 3 and 8 m below ground surface with local areas reaching up to 15 m, based on the 13 geotechnical drillholes.

5.4 Rock Mass Shear Strength

The shear strength/normal stress relationship describes the ultimate shear strength available at a given point within a slope as a function of the effective normal stress acting on that point. Based on review of the lithology, RQD, rock mass rating and intact rock strength, three geomechanical units were identified; the Gneiss, Schist and Granite. Rock mass shear strength/normal stress relationships were developed for the using the Generalized Hoek-Brown criterion (Hoek, et al., 2002).

The Generalized Hoek-Brown criterion defines curvilinear shear strength envelopes that are considered effective representations of intact rock and heavily jointed rock mass behavior. As discussed in Hoek, et al (2002), the following equation defines the rock mass shear strength envelope according to the Generalized Hoek-Brown criterion:

$$\sigma'_1 = \sigma'_3 + \sigma_{ci} \left(m_b \frac{\sigma'_3}{\sigma_{ci}} + s \right)^a \quad (1)$$

$$m_b = m_i e^{\left(\frac{GSI-100}{28-14D} \right)} \quad (2)$$

$$s = e^{\left(\frac{GSI-100}{9-3D} \right)} \quad (3)$$

$$a = \frac{1}{2} + \frac{1}{6} \left(e^{-GSI/15} - e^{-20/3} \right) \quad (4)$$

Where σ_1' and σ_3' are the effective major and minor principal stresses and σ_{ci} is the uniaxial compressive strength of the rock.

The Geologic Strength Index (GSI) was developed as a qualitative assessment of the lithology, structure/blockiness and condition of discontinuity surfaces in a rock mass by Hoek as input into the Generalized Hoek-Brown shear strength criterion in lieu of the RMR due to increasing realization that a system based more heavily on fundamental, largely subjective geological observations than on potentially less reflective, wholly objective, “numbers” was needed (Hoek, 1994). In practice, the GSI ranges from a value of zero for very weak, intensely sheared and fractured rock masses up to a maximum value of 100 for intact or very massive rock with very few, widely spaced discontinuities. GSI was estimated using the relationship proposed by Hoek et al. (1997): $GSI \sim RMR89 - 5$.

The disturbance factor, D, depends on the degree of disturbance that the rock mass is expected to be subject to as a result of blast damage and stress relaxation caused by excavation of the pit. The disturbance factor ranges from zero, for wholly undisturbed, confined rock masses, to 1 for very disturbed rock masses. For surface mining operations, Hoek et al., 2002 recommends a D of 0.7 for carefully controlled blasting and a D of 1 for heavy large-scale production blasting. The rock mass disturbance zone is typically on the order of 10 m to 50 m and does not apply to the entire rock mass behind the slope.

As described by Hoek (1983), the Hoek-Brown intact material constant m_i is very approximately analogous to the angle of friction of the conventional Mohr-Coulomb failure criterion, controlling the curvature of the intact failure envelope and thus, the rate at which the shear strength increases with confinement. Higher m_i values (in the order of 15 to 32), give steeply inclined strength envelopes and high instantaneous friction angles at low normal stress levels. These high m_i values tend to be associated with stronger brittle igneous and metamorphic rocks such as andesites, gneisses and granites. Lower m_i values (in the order of 3 to 7), give lower instantaneous friction angles and tend to be associated with more ductile carbonate rocks such as limestone or dolomite. Different samples of the same rock type can also have varying m_i values depending on the granularity and interlocking of the crystal structure (Marinos and Hoek, 2000).

As discussed in Section 4.2, m_i values of 18 and 19 were estimated for the Gneiss and Schist rock types respectively. Insufficient data was available to develop a triaxial curve for the Kona Granite and as such, a conservative estimate of 23 was made based on SRK (2016) and typical values published by Marinos & Hoek (2000). Rock mass strength parameters derived from the 2015 and 2018 geotechnical logging and laboratory test data are summarized in Table 5-3.

Table 5-3: Design Rock Mass Parameters

Pit	Rock Type	Oxidation Group	Intact Rock		Rock Mass		
			m _i	UCS (MPa)	GSI	D	Density (kN/m ³)
Supremo	Gneiss	Oxide	18	49	54	0.7	26.0
		Upper & Middle Trans	18	70	56		
		Lower Trans & Fresh	18	87	63		
Latte	Schist	Oxide	19	46	49	0.7	26.0
		Upper Trans	19	58	51		
		Middle & Lower Trans	19	52	62		
		Fresh	19	45	56		
Double Double	Gneiss & Schist	Middle & Lower Trans	18	34	62	0.7	26.0
		Fresh	18	54	58		
Kona	Granite	Middle Trans	23 ¹	60	64	0.7	25.1
		Fresh	23 ¹	97	72		

Source: JDS (2019)

Notes:

¹ Estimated based on published values due to insufficient number of laboratory test results.

5.5 Groundwater

Groundwater (pore water) pressure is an important component of slope stability. Pore water pressures act in direct opposition (as buoyant forces) to stabilizing forces, and as such, must be considered for the results of stability modeling to be realistic. A relatively free-draining slope will typically allow drawdown of the groundwater surface sufficiently deep within the slope so that pore water pressures are of minimal impact to slope stability. Since the rock mass comprising open pit benches has usually been at least moderately disrupted by production blasting, it is usually free-draining and, therefore, pore water pressures are seldom considered in bench scale stability analyses.

Groundwater levels for the stability analyses were estimated based on field hydrogeological investigations conducted by SRK (2015) and Lorax (2016) and subsequent three-dimensional groundwater modeling performed by Lorax (2017). The end-of-mining groundwater surface predicted by the Lorax (2017) groundwater model was used as the basis for the estimations. The 2017 numerical groundwater model was based on a March 2017 pit design that is slightly deeper in some areas. However, the surface is still considered a reasonable starting point for depressurization in lieu of any more specific information.

6 GEOLOGIC DISCONTINUITIES

The term discontinuity refers to any significant mechanical break or fracture in rock, across which there is negligible tensile strength. Discontinuities are formed by a wide range of geological processes, collectively including joints, faults, fissures, fractures, veins, bedding planes, foliation surfaces, shear zones, dikes and contacts. The most common discontinuities at Coffee are joints and foliation. Faults and shear zones were also occasionally encountered in the geotechnical drill core; however, due to the relatively poor rock quality commonly associated with larger scale structures such as faults and/or shear zones, relatively few were able to be confidently oriented.

Minor discontinuities, such as joints, fractures, foliation and bedding planes, represent an infinite population for practical purposes and, due to sampling limitations, are best modeled with stochastic (probabilistic) techniques. Major geologic structures are those features, such as faults, dikes, shear zones, and contacts that have dimensions on the same order of magnitude as the area being characterized. These structures are treated as individual elements for design purposes, as opposed to joints, which are handled statistically.

6.1 Structural Domains

Structural domains were developed based primarily on dominant discontinuity orientations obtained from the combined 13 geotechnical drillholes. These domains define reasonably large volumes where discontinuity patterns are similar enough on pit wall scale that they can be grouped together for analysis.

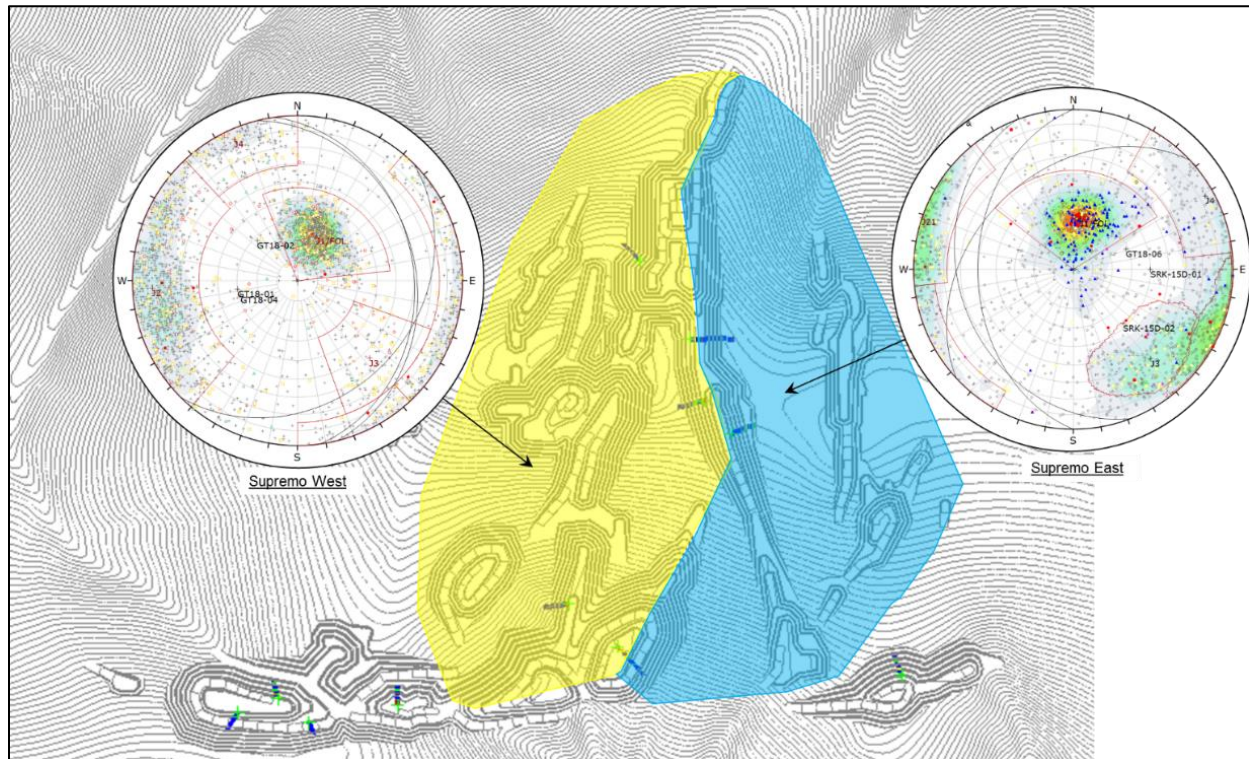
The pit areas were initially broken down into volumes or rock bound by known major faults and/or lithological contacts. Discontinuity orientation data from within each of these volumes was contoured on lower hemisphere, equal area percentage plots to identify dominant discontinuity set orientations using Dips software (Rocscience, 2019). A discontinuity 'set' is generally defined as a group of discontinuities that are anticipated to exhibit similar geotechnical conditions and are within a similar range of orientations, i.e., dip and dip direction. The discontinuity orientation data for each geotechnical drillhole and primary lithology were initially analyzed independently of each other to identify significant variations in the presence and the orientation of the various sets spatially across the pits.

Where similar, the initial areas were combined with neighboring areas forming larger volumes or structural domains. Directional biases were considered for each drillhole. Drillholes are directionally biased against encountering structures with dips and dip directions sub-parallel to the drillhole's plunge and bearing.

A total of seven separate structural domains were developed for the pit slope stability analyses using this methodology. The Supremo, Latte and Double Double pits were divided into two separate domains each while Kona was considered as a single domain due to its small size. The individual domains delineated and their respective discontinuity orientation plots or stereonets are shown on Figure 6-1, Figure 6-2 and Figure 6-3 for the Supremo, Latte - Double Double and Kona pits, respectively. The discontinuity sets identified in the Supremo, Latte-Double Double and Kona pit domains are summarized in Table 6-1, Table 6-2 and Table 6-3, respectively. The

numbers in parentheses in Table 6-1 through Table 6-3 indicate the percentage of the total discontinuities identified in each domain that are contained within the respective set.

Figure 6-1: Supremo Pit Domain and Discontinuity Pole Plots



Source: JDS (2019)

Table 6-1: Supremo East and West Domain Discontinuity Sets

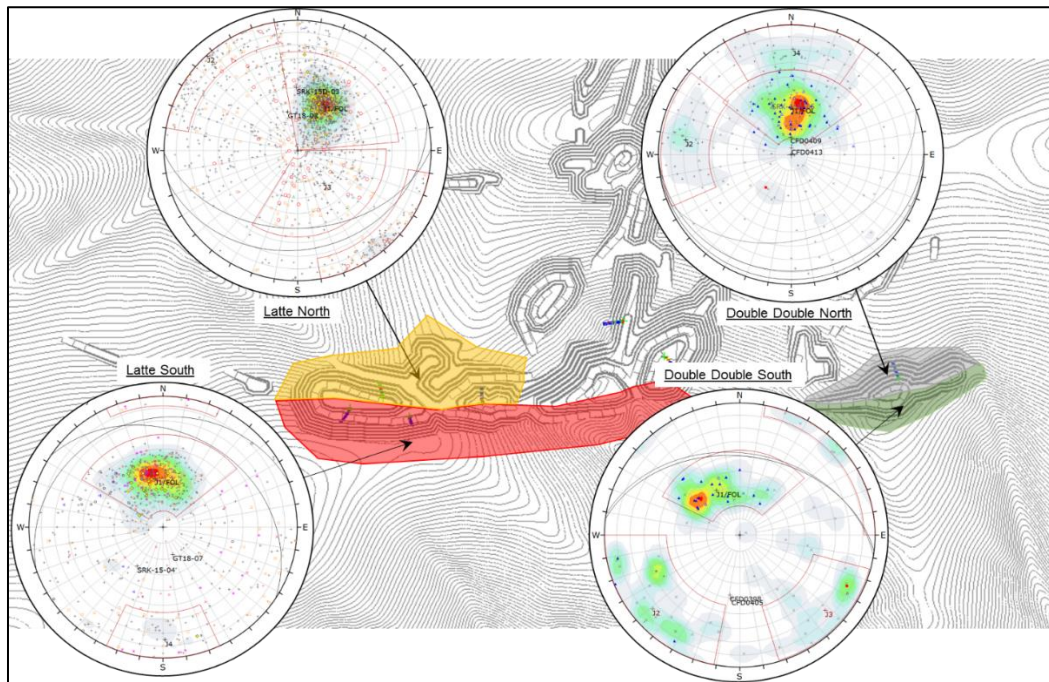
Pit Sector	Set ID	No. ¹	Mean Orientation		Potential Impact on Design
			Dip	Dip Dir.	
Supremo East	J1/FOL	451 (28%)	26	188	Very minimal impacts on stability due to dip direction perpendicular to pit walls except for a few very local areas.
	J2	363 (22%)	90	109	Minimal impacts on bench stability given near vertical dip. May cause localized sloughing where oblique to bench face.
	J3	275 (17%)	63	319	Minimal impacts on interramp/overall slope stability. Potential for planar instability along NW dipping, T4 pit wall benches, if present.
	J4	184 (11%)	84	241	Minor joint set. Minimal impacts on bench stability given near vertical dip.
Supremo West	J1/FOL	949 (33%)	24	203	Very minimal impacts on stability due to dip direction being perpendicular to pit walls except for a few very local areas.
	J2	1,263 (44%)	77	087	Minimal impacts on stability given sub-vertical dip. May be advantageous for N-NW striking walls, providing steep, stable surfaces to break to.
	J3	117 (4%)	54	317	Minor joint set. Potential to cause planar bench instability for NW dipping pit walls if becomes more prominent. Confirm during mining.
	J4	261 (9%)	83	156	Minor joint set, minimal impacts on bench stability given near vertical dip.

Source: JDS (2019)

Notes:

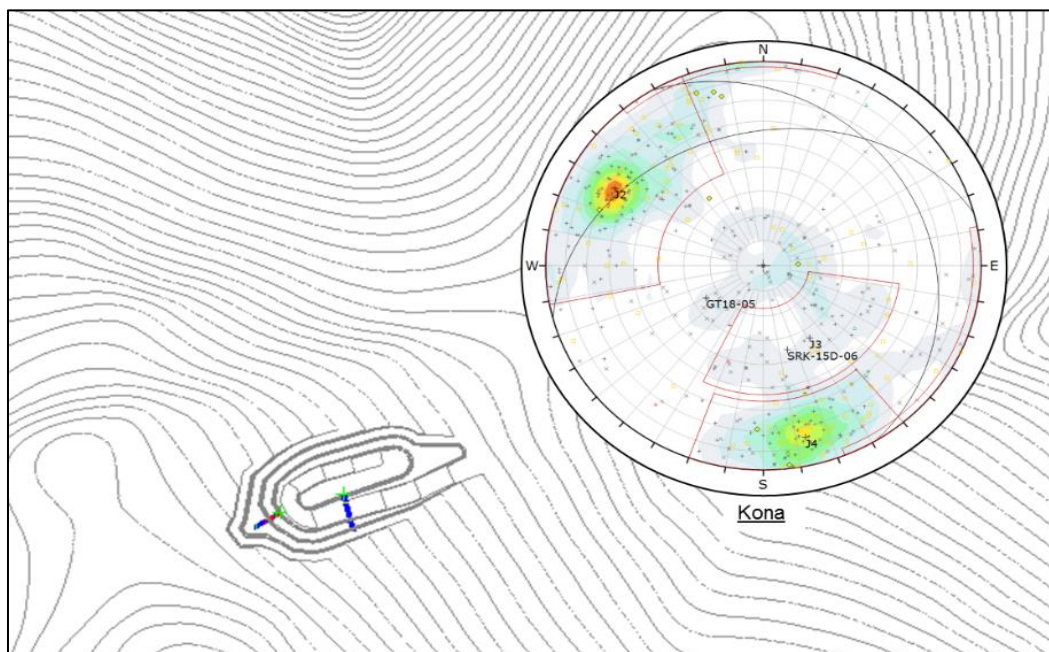
¹ Total number of discontinuities oriented per set. Number in parentheses is the percentage of the total number of discontinuities oriented.

Figure 6-2: Latte and Double Double Pit Domain and Discontinuity Pole Plots



Source: JDS (2019)

Figure 6-3: Kona Pit Domain and Discontinuity Pole Plot



Source: JDS (2019)

Table 6-2: Latte and Double Double Domain Discontinuity Sets

Pit Sector	Set ID	No. ¹	Mean Orientation		Potential Impact on Design
			Dip	Dip Dir.	
Latte North	J1/FOL	954 (58%)	29	57	Significant potential to impact stability of bench and interramp slope stability for north wall due to dip direction being parallel to wall dip direction.
	J2	174 (11%)	89	137	Minor joint set. Minimal impacts on bench stability given near vertical dip.
	J3	222 (14%)	26	321	Low confidence set. Potential to cause planar bench instability for NW dipping pit walls but generally has shallow dip. Needs confirmed during mining.
Latte South	J1/FOL	378 (57%)	31	171	Low risk of instability due to shallow dip into wall. Potential for step-path instability in south wall bench faces if combined with J4.
	J4	61 (9%)	75	359	Minor/low confidence set. If present, has potential to cause planar bench instability or, if combined with J1, step-path and/or toppling instability in south wall benches. Needs confirmed during mining.
DD North	J1/FOL	111 (45%)	30	180	Potential to impact stability of bench and interramp slope stability for north wall due to dip direction being parallel to wall dip direction.
	J2	32 (13%)	69	098	Minor joint set. Potential wedge formed with J4 but with moderate to steep LOI (63/137). Minimal potential for plane shear due to strike perpendicular to wall.
	J4	37 (15%)	70	181	Minor joint set. Potential wedge formed with J2 but with moderate to steep LOI (63/137). Minor potential for planar failures due to steep dip. May act as release for J1 planar instabilities. Needs confirmed during mining.
DD South	J1/FOL	25 (32%)	31	153	Minimal impact to south wall stability due to dip into pit wall.
	J2	27 (35%)	74	049	Minor joint set. Potential wedge formed with J3 but steep LOI (66/000). Minor potential for planar failures due to orientation relative to pit walls.
	J3	13 (17%)	74	312	Appears as minor joint set. Potential wedge formed with J2 but steep LOI (66/000). Low potential for planar failures for south wall due to steep dip.

Source: JDS (2019)

Notes:

¹ Total number of discontinuities oriented per set. Number in parentheses is the percentage of the total number of discontinuities oriented.

Table 6-3: Kona Domain Discontinuity Sets

Pit Sector	Set ID	No. ¹	Mean Orientation		Potential Impact on Design
			Dip	Dip Dir.	
Kona	J2	139 (33%)	67	119	Low probability of planar instabilities for S-SE dipping walls due to steep dip and curved NW pit walls. Potential wedge formed with J4 (49/058).
	J3	48 (12%)	34	329	Low confidence joint set. Minor potential for planar instabilities along SE wall if persistent but has shallow dip. Confirm during mining.
	J4	102 (24%)	75	347	Low probability of creating planar instabilities for N-NW dipping walls due to steep dip and curved nature of pit northerly dipping pit walls. Potential wedge formed with J2 (49/058).

Source: JDS (2019)

Notes:

¹ Total number of discontinuities oriented per set. Number in parentheses is the percentage of the total number of discontinuities oriented.

6.2 Kinematic Assessment

To assess which of the various discontinuity sets identified in each domain have potential to adversely impact bench stability for the various pit slope orientations, an initial kinematic assessment was completed for each pit using stereonet created with Dips Software (Rocscience, 2019). The orientation and variability of each set was graphically compared to the various pit slope dip directions to assess which sets, or combination of sets, have the potential to form 3D (planar or wedge) blocks that could displace under gravitational forces. Kinematic analyses can be considered conservative in that they do not consider the length or spacing of discontinuities which are more realistically assessed with more complex probabilistic type models.

Where relatively high likelihood of instabilities are suggested by the initial kinematic analyses, more detailed quantitative and/or probabilistic analyses should be carried out. Based on the structural domaining and kinematic assessment, the following conclusions can be made:

- The most dominant structural trend at Coffee is the south-southwest shallow to moderate dipping foliation and a vertical to steep east to east-southeast dipping joint set (J2) present at Supremo, Latte and Double Double;
- Foliation has a high potential to cause planar type instabilities in the north Latte pit wall due to the intense foliation. The foliation has similar potential in the Double Double north wall but is only considered moderate due to foliation being less intense in the gneiss. The J1/Foliation sets are likely to impact bench and interramp stability and should be accounted for in the north wall slope design parameters. Pit wall dip directions between 150° and 240° azimuth

will be susceptible to planar type displacements if bench face and interramp slope angles are over steepened;

- The vertical to steep east to east-southeast dipping joint set (J2) at Supremo, Latte and Double Double is not expected to have a significant impact on bench or interramp slope stability. It may cause local sloughing where persistent discontinuities strike slightly oblique to bench face. The set may even be advantageous for Supremo west walls providing a clean surface for the bench faces to break back to;
- The steeply south dipping set (J4) identified in Latte and Double Double could be considered as questionable due its being poor representation in the data set. If J4 does exist, it may combine with J1/foliation forming step-path, bench scale instabilities. Mapping should be carried out during initial pit excavation to confirm the presence of J4 and its potential implications;
- Double Double sets J2 and J4 have potential to combine forming wedge type bench instabilities if present. However given the 63° plunge of their line-of-intersection (LOI) and the southeast direction of its dip, such wedges are expected have minimal impact on wall stability;
- Double Double sets J2 and J3 also have potential to combine forming wedge type bench instabilities with their LOI plunging approximately 66° towards north. While the northerly plunge direction makes such a wedge kinematically viable, the 66° LOI dip suggests that the impact would be minor;
- Steep, north-northwest (J4) and southeast (J2) dipping joint sets are the most dominant trends at Kona. The sets have potential to for planar instabilities if undercut; however, given the J4 dip is near design bench face angle and that the northwest pit wall has such a tight radius of curvature (J2), impacts on pit wall stability are anticipated to be minimal;
- The combination of Kona sets J4 and J2 potentially form a kinematically viable wedge with a LOI plunge of 49° towards 058° which could limit achievable bench face angles for southwest corner of the pit. However impacts are expected to be minor since the portion of the pit wall susceptible to such instabilities is small section due to the tight radius of curvature in the southwest corner of the pit; and
- Based on the current understanding of major geologic structures and the current three-dimensional fault model (SRK, 2017), most major fault structures are either near vertical or are mined out with the ore and therefore are not expected to significantly impact final pit wall stability.

As is the case at all mine sites, deviation from the discontinuity sets identified in Table 6-1, Table 6-2 and Table 6-3 and/or additional sets not encountered by the oriented core or televiwer should be anticipated during mining of the pits. It is anticipated that the major, pit wall controlling structural trends have been identified during the current program and that additional sets or variations encountered during mining should have relatively minor, bench scale impacts. A proper geotechnical mapping program during the early stages of pit development will be necessary to verify the current assumptions and allow for slope design modifications, if appropriate.

7 BENCH DESIGN CONSIDERATIONS

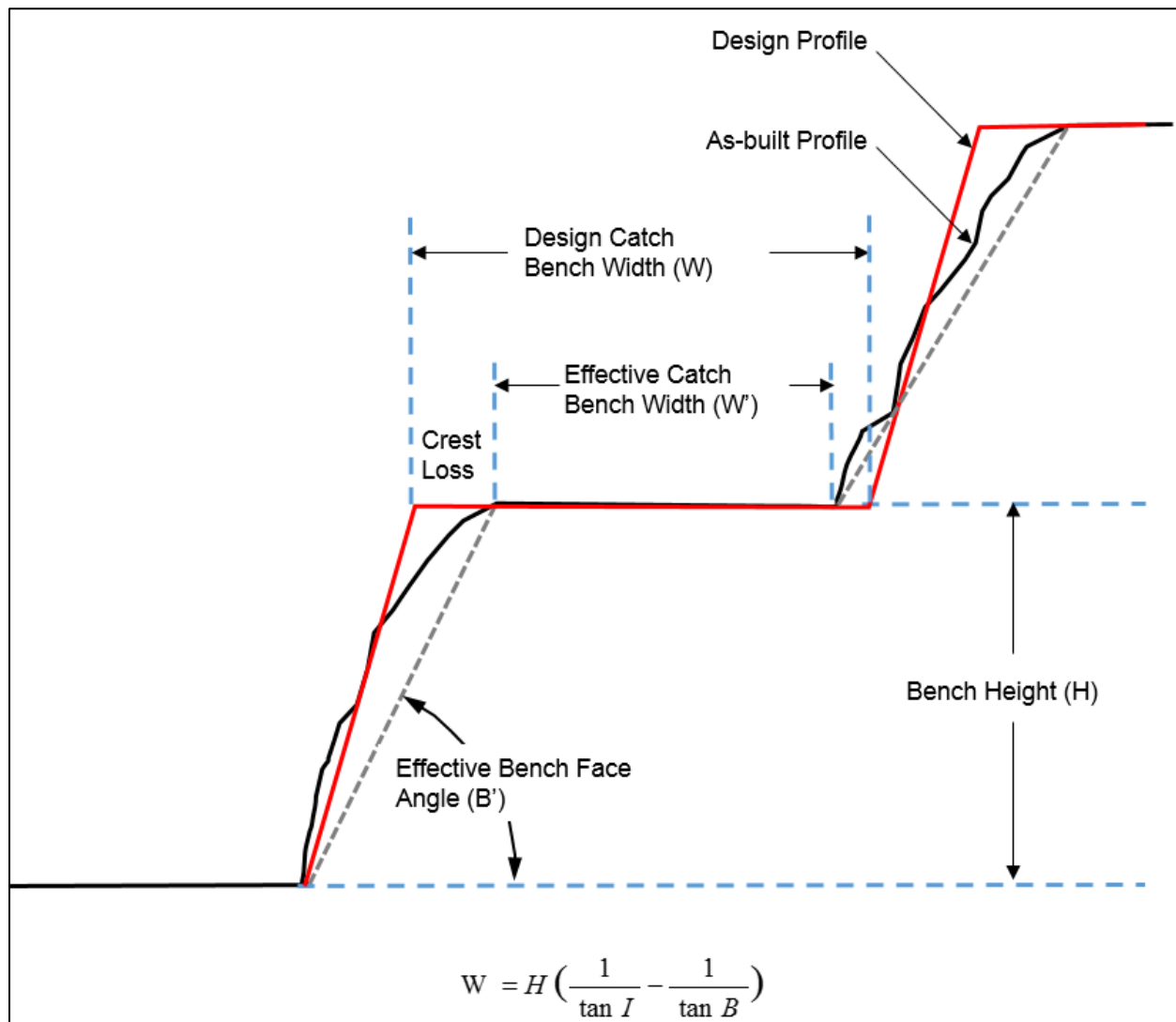
The consequences of an overall or high interramp slope failure on a final pushback commonly produce significant impact on mine economics. This is due to the fact that a substantial quantity of ore is frequently rendered uneconomic by the additional, unanticipated cost of removing the resulting failed material. The evaluation of the anticipated stability of final design slopes is therefore necessary and must be incorporated into final design recommendations.

Of similar importance and impact on the project economics, though not nearly as dramatic as large-scale slope failures, are the design and excavation of the benches and lower height interramp slopes, i.e., those slopes comprised of two to three benches. The overall slope designs cannot be achieved if benches cannot be safely and effectively established.

Although the expected performance of the overall and higher interramp slopes comprising the open pit can best be predicted and subsequently examined using rock mass failure models, the anticipated behavior of the bench and lower interramp slopes is most realistically assessed using analytical models that evaluate structurally controlled failure mechanisms. This is because rock structure, i.e., joints and other non-fault discontinuities, will most likely facilitate structurally controlled failures, whenever the site materials have relatively high rock mass strengths.

In strong, competent materials such as at Coffee, the development of rock mass failure in benches and in lower height interramp slopes is essentially precluded. Consequently, the evaluation of structurally controlled, or kinematic failure potential of benches and lower height interramp scale slopes, is the dominant consideration in the formulation of bench design recommendations. Common bench design terminology is explained in Figure 7-1.

Figure 7-1: Explanation of Bench Design Terminology



Source: JDS (2019)

7.1 Description of Methodology

The rock structure controlled primary failure mechanisms considered were simplified into one of three categories, i.e.:

- Plane shear failure, defined as translation (sliding) of a failure mass on a single geologic structure oriented essentially parallel with the slope being modeled;

- Simple wedge failure, defined as translation of a tetrahedral-shaped failure mass (bounded by the slope face, the essentially flat upper surface and two geologic structures, each oriented obliquely to the slope face) on either of the two lower bounding geologic structures or on both, i.e., down the line of their intersection; or
- Step path failure, defined as a translation similar to that of a plane shear mode but due to insufficient length of the plane shear set to foster a through-going failure surface, the failure path must step up to another discontinuity of the same set via a sub-perpendicular discontinuity joint set failing in tension, and with this alternation of sliding and stepping occurring until a path daylight at the top of the bench.

In all instances, the failure mechanism is only viable if:

- A potential failure mass exists, that is, if a geologic structure or structure set is present in a plane shear orientation relative to the slope being evaluated, or if two geologic structures or structure sets are present and oriented relative to each other and to the slope being studied in such a manner that a wedge tetrahedron; and
- Either the plane shear or the wedge line of intersection (between the two geologic structures) is “daylighted”, i.e., the potential plane(s) intersects the bench level above and the bench face and is flatter than the slope angle.

Geologic structure-controlled failure is only possible if discontinuities occur spatially so that a potential failure mass exists, and that mass is unconstrained at the slope face. Once it is determined that a viable potential failure mass does exist, the likelihood of geologic structure-controlled failure can be assessed by determining if the maximum shear resistance, which can be developed along the potential failure surface or surfaces, is greater than the driving forces acting to destabilize the mass.

The maximum available shear resistance is a function of both the structure continuity and its shear strength. For failure to occur along a discontinuous geologic structure, the non-fractured rock forming the intact portion(s) of the potential failure surface must fail. Intact rock strength for all but the weakest of rock tends to be higher than the stresses developed in bench and lower height interramp slopes. Consequently, the strength of intact rock is rarely exceeded. Seldom do failures of such scale occur along surfaces that are not of through-going, continuous geologic structures. The models considered for bench and lower height interramp slopes within the proposed open pit assume that failures could only develop along geologic structures which were continuous through the slopes. The available shear resistance then becomes a function solely of the discontinuity shear strength along the surface, given that a continuous structure(s) is present.

With the assumption of the necessity of continuous geologic structure(s) relative to the slope under consideration, the likelihood of the failure of bench or lower height interramp slope simplifies, becoming a function of:

- The likelihood that potential failure surface forming geologic structure(s) occur in the required orientation relative to the slope and, in the case of wedge failure, to each other;
- The likelihood that the shear stresses along the potential failure surface(s) exceed the maximum available shear resistance;

- The likelihood of geologic structure(s) continuity or, in the case of step path failure, path continuity on the appropriate scale;
- The likelihood of the potential failure surface(s) being daylighted; and
- The number of such structure(s).

7.2 Bench Design Conclusions

A comprehensive Probabilistic Bench Design Analysis was carried out by SRK (2016) using distributions of joint set properties (orientation, spacing and length) to evaluate structurally controlled failure mechanisms. Review of the fracture orientation data from the 2018 GI indicates refined but very similar dominant structural trends compared to 2015 investigation which was the basis of the SRK (2016) detailed Probabilistic Bench Design Analyses. As such, detailed bench design analyses were not carried out as part of the current update. The impact of discontinuities on stability was assessed with a more qualitative, kinematic assessment as discussed in Section 6.2.

As discussed in Section 6.2, the highest likelihood of bench instability is from planar failures fostered by foliation discontinuities in the Latte and Double Double north walls. The SRK (2016) detailed Probabilistic Bench Design Analyses were reviewed relative to the updated information and used to develop appropriate bench design recommendations for the Latte and Double Double north walls.

The following was concluded from the kinematic assessment (Section 6.2) regarding bench stability:

- Due mostly to the north-south orientation of the pit, benches at Supremo are not anticipated to be significantly impacted by structurally controlled instabilities. As such, a maximum achievable bench face of 75° was estimated for Supremo based on operational constraints as discussed below;
- The stability of benches on the north Latte and Double Double pit walls will likely be controlled by the dominant southerly (inward) dipping foliation discontinuities. The analyses indicate a maximum achievable bench face angle of approximately 65° for their north walls. Stability of the south and end pit wall benches are not anticipated to be governed by geologic structure controls and were, therefore, estimated to have a maximum achievable bench face of 75° based on operational considerations;
- Benches on the northwest Kona pit have a slightly higher probability of structurally controlled instabilities as a result of a moderately southeast dipping joint set (J2). The analyses indicate a design bench face angle of approximately 65° is appropriate for this sector of the pit. The remaining pit areas are not anticipated to be significantly impacted by geologic structure-controlled instabilities. A maximum achievable bench face of 75° was estimated for the remainder of the pit based on operational constraints; and
- Based on the Modified Ritchie Criteria as described by Call (1992), a minimum catch bench width of 8.5 m is recommended.

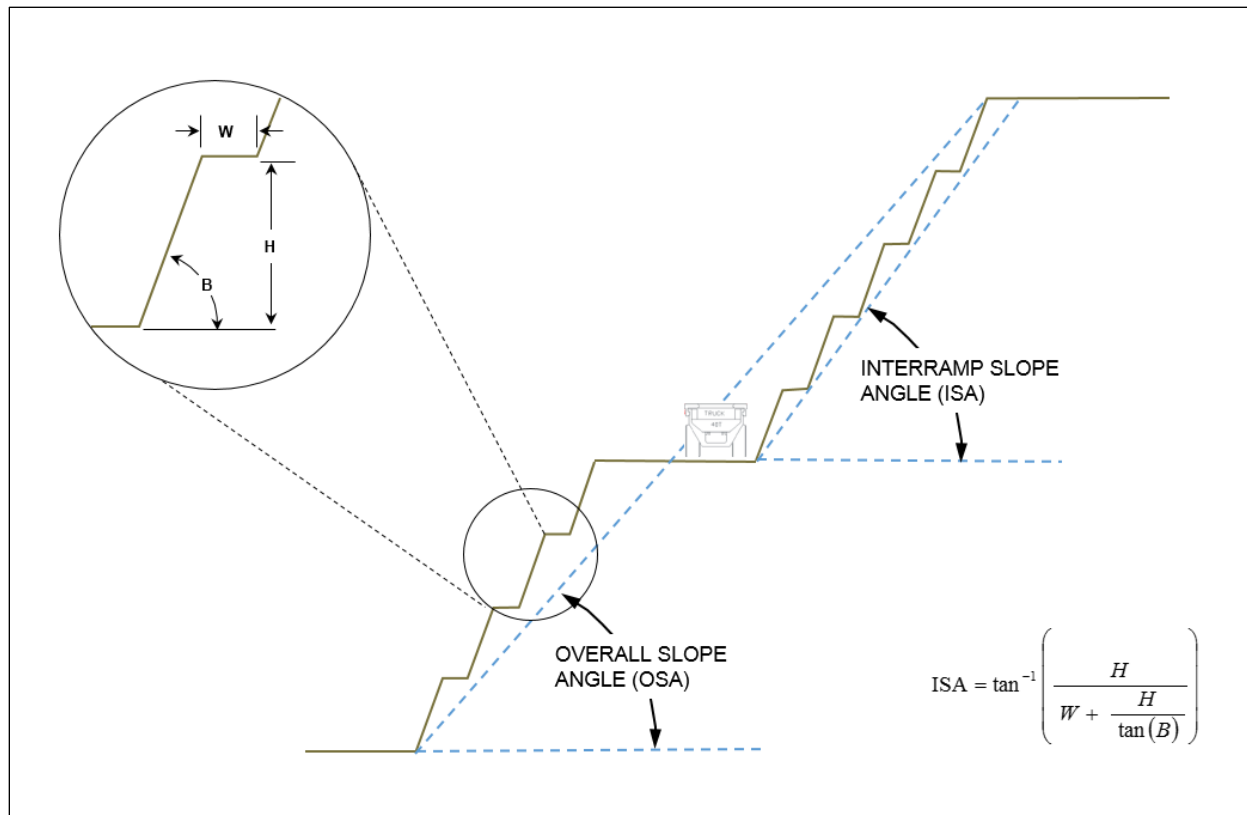
It should be noted that bench design and kinematic analyses are based solely on orientations of geologic structure and do not directly consider effects of weathering, alteration, blasting or excavation techniques. Depending on the quality of blasting and excavation practices, achievable bench face angles may be significantly reduced from the theoretical angles determined by these analyses. When taking these operational effects into consideration, it is rare to achieve bench face angles greater than about 70 to 75°, unless there is a steeper structure controlling the bench geometry. Increasing bench face angles to greater than about 75° may be achievable, but usually require more rigorous drilling and specialized controlled blasting techniques than are commonly practiced.

8 INTERRAMP / OVERALL SLOPE STABILITY ANALYSES

Based on the results of the Bench Design Analyses, recommendations for bench configurations and the resulting maximum interramp slope design were provided to Goldcorp for detailed pit designs incorporating necessary pit access ramps and infrastructure. Stability of the high interramp and overall slopes of the initial detailed pit designs were then evaluated.

Slope design involves analysis of the three major components of a pit slope: bench configuration, interramp angle, and overall slope angle, all as defined on Figure 8-1. The bench configuration, which is controlled by the bench face angle (BFA), bench height (BHT), and berm width (BWT), defines the interramp angle (IRA). The overall slope angle (OSA) consists of interramp sections separated by wide step-outs for haulage roads or mine infrastructure.

Figure 8-1: Pit Slope Design Components

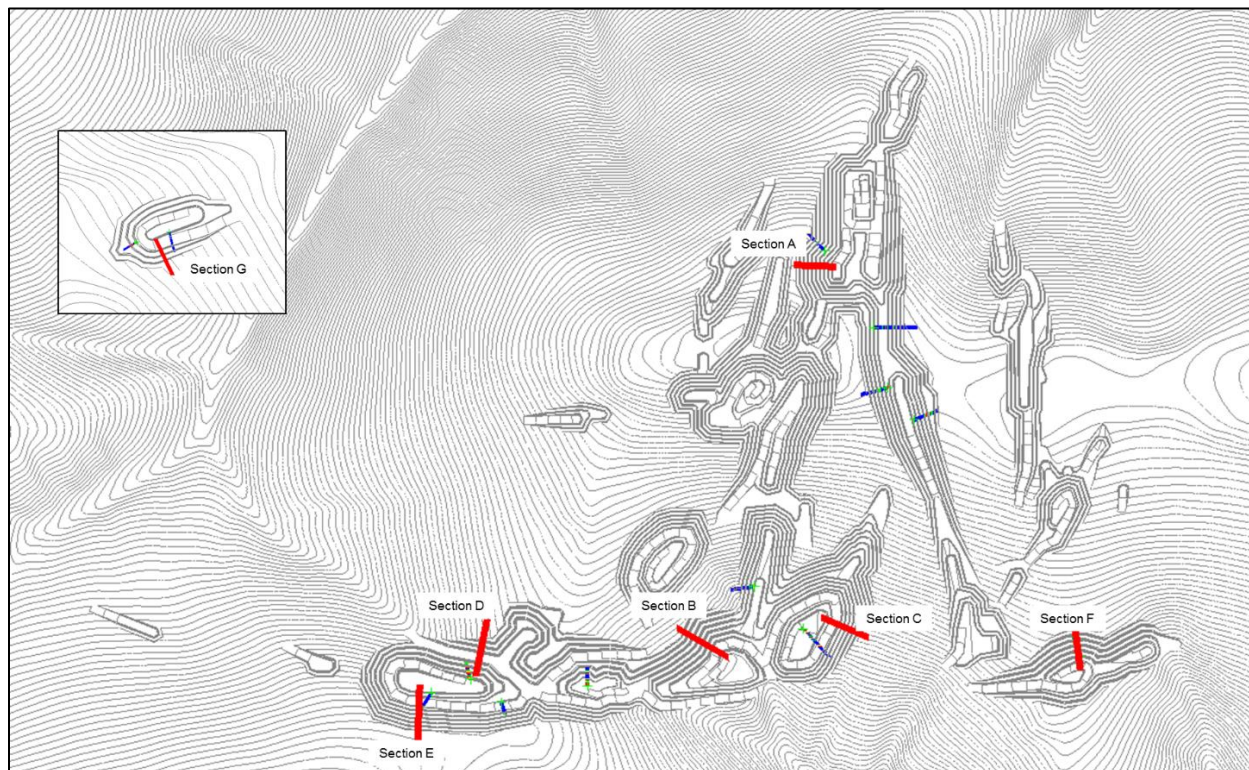


Source: JDS (2019)

8.1 Slope Stability Sections

Based on the results of the geomechanical characterization testing and initial detailed pit design geometries, critical slope stability cross-sections were selected for analysis. A total of seven critical sections were selected to verify stability of the ultimate pit designs. Critical sections are selected at locations where slope stability conditions are anticipated to be the most adverse such as where the slope height is at its maximum, pit wall materials are low strength and/or pore water pressures may be the highest. The traces of the critical sections are shown on Figure 8-2 with the geometry of each individual section shown in Appendix C.

Figure 8-2: Location of Critical Slope Stability Cross Sections



Source: JDS (2019)

8.2 Methodology

The seven critical cross-sections were analyzed using the slope stability modeling software, Slide (Rocscience, 2018), which uses two-dimensional, limited equilibrium methods with output in

terms of safety factors. Spencer's method of slices was used for the analyses due its consideration of both force and moment equilibrium and is typically less conservative compared to methods that consider only force equilibrium.

To construct the model geometries, vertical cross sections were cut through the respective final pit designs and the primary structures using Vulcan Mine Planning Software (Maptek, 2018). Multiple failure modes were analyzed including overall and interramp slopes and localized failures associated with geological contacts and/or faults. The non-circular, "path" search method was used for the final analyses in all cases.

8.3 Geomechanical Parameters

As discussed in Section 5.5, the Hoek-Brown (Hoek, et al., 2002) criteria was used to represent the shear strength–normal stress relationship for the rock masses for slope stability modeling. The respective average properties were used to represent the rock mass and joint strengths for each of the primary rock types. The rock mass and joint strength parameters are summarized in Table 4-2 and Table 5-3, respectively.

Slide's Generalized Anisotropic shear strength model was used to simulate the weaker strength of the rock parallel to foliation and, to a lesser extent, the dominant joint sets. The Generalized Anisotropic Strength model assigns a weaker strength to any portion of a potential failure surface that is parallel to the joint or foliation planes. The foliation and joint parallel shear strengths were developed using an equivalent strength material approach based on Jennings' (1972) equations:

$$c_{eq} = (1 - k)c + kc_j \quad (5)$$

$$\tan(\phi_{eq}) = (1 - k)\tan(\phi) + k \tan(\phi_j) \quad (6)$$

Where c and ϕ are the cohesion and friction angle, respectively, of the intact rock bridges, c_j and ϕ_j are the cohesion and friction angle of the discontinuities and k is the coefficient of continuity along the failure plane. Given the lack of discontinuity length information at this stage of the project, the analyses were conducted using conservative continuity coefficients of 0.7 for joints, 0.80 for the intense schist foliation and 0.75 for the gneiss foliation. The isotropic, Hoek-Brown rock mass shear strength was used to represent the intact rock bridges.

Based on anecdotal evidence that earthquake ground accelerations are not known to have been the cause of any rock slope failures in mining and that there is little or no experience to suggest that rock slope stability is susceptible to seismic loading, a seismic (or pseudostatic) stability analysis was not performed. This is a common industry assumption for mining rock slope stability.

8.4 Results of Interramp/Overall Stability Analysis

Based on accepted engineering experience, interramp/overall slope designs that yield factors of safety (FOS) of 1.3 for slopes with high failure consequences and 1.2 for low failure consequences are appropriate for most open pit mines. Slopes of high failure consequence are generally those slopes that are critical to mine operations, such as those on which major haul roads are established, those providing ingress or egress points to the pit, or those underlying infrastructure such as processing facilities or structures.

The results of the overall slope angle (OSA) and interramp slope angle (ISA) stability analyses are summarized in Table 8-1 for each of the seven critical sections analyzed. Graphical output files showing the critical failure and FOS calculated by Slide for each individual analysis are presented in Appendix C.

Table 8-1: Results of Overall / Interramp Slope Stability Modeling

Pit	Section	Slope Design			Safety Factor	
		Max. Height (m)	OSA (°)	ISA (°)	ISA	OSA
Supremo	A	190	56	55	3.8	2.8
	B	170	49	51	3.3	2.9
	C	180	53	55	2.1	2.3
Latte	D	150	44	47	1.3	1.3
	E	180	53	55	2.1	2.0
Double Double	F	140	44	46	1.7	1.9
Kona	G	90	47	52	3.4	3.9

Source: JDS (2019)

Results of the overall slope stability analyses demonstrate that the bench configuration-based slope angles either meet or exceed the minimum acceptable safety factor of 1.3 for slopes with high failure consequences. Critical failure surfaces were typically non-circular, partially through the intact rock mass and partially along the discontinuities as defined in Section 8.3.

The results indicate that the stability of the Coffee open pit slopes is anticipated to be controlled by achievable bench face angles and not the stability of interramp or overall slope angles. Calculated safety factors for some sections are considered relatively high for typical open pit slope designs; however, steepening of the interramp slope angles would require either steeper bench face angles or reducing the design catch bench width which is not recommended due to the lack of outcrop to record actual structural information. With detailed geomechanical/geological bench mapping and good quality wall control blasting practices during operation, opportunity may exist to steepen the interramp angles based on the newly acquired and more accurate information.

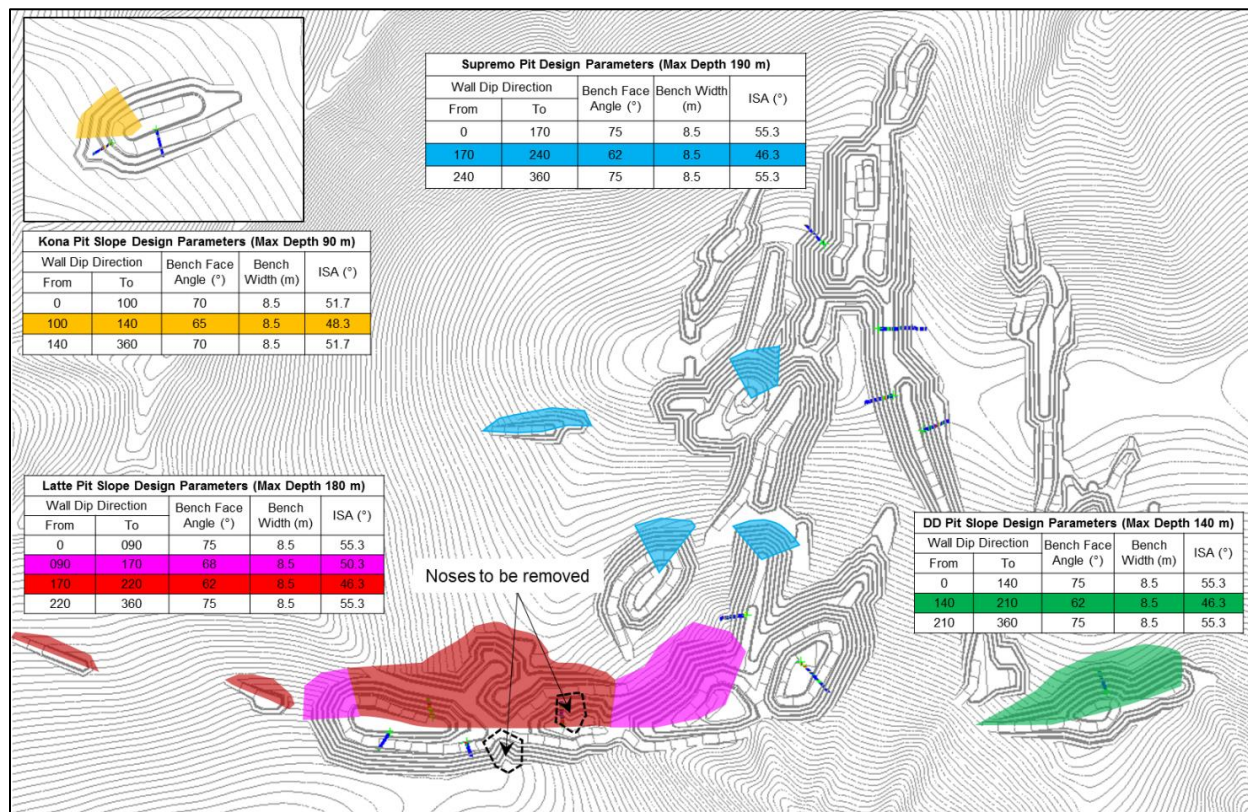
The use of a disturbance factor “D” equal to 0.7 for the entire rock mass is considered conservative given that blast damage and relaxation for relatively small open pits such as the Coffee pits would typically only extend a maximum distance of up to approximately 50 m. Beyond this disturbed zone, a D factor of zero should typically be used which would increase the rock mass strength and safety factors. However, given that the critical surfaces produced by the models are typically near 50 m from the face (approximate limits of the disturbed zone), more detailed modeling or zoning of the D factor would not have a significant impact on the results.

9 PIT SLOPE DESIGN RECOMMENDATIONS

Recommendations for Geotechnical Pit Slope design Parameters are summarized in Table 9-1 and shown graphically on Figure 9-1. In addition to the recommendations in Table 9-1 and Figure 9-1, continuous bench stacks or interramp slope heights should not exceed a height of approximately 100 m (vertically) without a minimum 15 m wide geotechnical berm, haul road or other wide step-out for infrastructure.

The current pit design incorporates multiple ‘noses’ which typically perform poorly in pit walls due to the lack of confinement on their sides and the increased probability of a geologic structure(s) penetrating from one side to the other, resulting instability. Given the dominant east-west structure at Latte and Double Double, the likelihood of a structure penetrating fully through the nose and causing a bench or multi bench failure is high. Interramp slopes within the noses should be reduced or the noses should be removed completely.

Figure 9-1: Pit Slope Design Recommendations



Source: JDS (2019)

Table 9-1: Recommended Pit Slope Design Parameters

Pit	Max. Slope Height (m)	Wall Dip Direction		Bench Face Angle (°)	Bench Height (m)	Min. Bench Width (m)	Max. ISA (°)
		From (°)	To (°)				
Latte	170	0	90	75	20	8.5	55.3
		90	170	68	20	8.5	50.3
		170	220	62	20	8.5	46.3
		220	360	75	20	8.5	55.3
Supremo	180	0	170	75	20	8.5	55.3
		170	240	62	20	8.5	46.3
		240	360	75	20	8.5	55.3
Double Double	135	0	140	75	20	8.5	55.3
		140	210	62	20	8.5	46.3
		210	360	75	20	8.5	55.3
Kona	100	0	100	70	20	8.5	51.7
		100	140	65	20	8.5	48.3
		140	360	70	20	8.5	51.7

Source: JDS (2019)

Double (20 m high) benches are recommended instead of single (10 m high) benches because properly executed double benches typically result in steeper effective bench face angles due to the confinement on the lower lift(s). For example, the upper bench lift, adjacent to a planned catch bench, is unconfined or free-faced in the upward direction and, consequently, some level of crest loss is unavoidable; however, for the lower lift(s), where there will not be a catch bench left adjacent to the blast, the confinement from the rock above effectively eliminates the potential for crest loss, assuming proper controlled blasting procedures are followed.

In addition, the probability of a discontinuity penetrating through a double bench is lower than that of a single bench, given the increase in bench height and corresponding increase in discontinuity length required to penetrate the bench. The use of double benches also results in wider catchments which are more likely to retain rock fall and provide access to benches for cleaning. Careful cleaning and scaling is required for double benches which, considering the bench height relative to scaling equipment reaching capabilities, must be done after each lift.

The recommended bench design assumes high quality wall control blasting including properly designed pre-split and/or trim blasting adjacent to final walls. Depending on the quality of blasting and excavation techniques, achievable bench face angles might be greatly reduced from the theoretical angles determined by these analyses. It is recommended that field trials be performed of various controlled blasting techniques, carefully documenting the results to confirm that the actual slope designs are being achieved or, if necessary, to serve as the basis of slope angle refinements.

10 ASSESSMENT OF FUTURE GEOMECHANICAL WORK

Two additional diamond geotechnical drillholes are recommended to be completed with future resource drilling. The collar coordinates and drillhole orientations are summarized in Table 10-1. Both holes should be surveyed with televiewer equipment similar to what was completed for the 2018 GI.

Table 10-1: Recommended Additional Geotechnical Drillholes

Hole ID	Easting (UTM)	Northing (UTM)	Elevation (masl)	Depth (m)	Azi. (°)	Inc. (°)
GT19-08	583,845	6,973,327	1,030	160	315	-60
GT19-09	582,770	6,973,200	1,100	190	200	-60

Proposed drillhole GT19-08 is designed to characterize the northeast-southwest oriented wall between the southern Supremo T3 and Latte structures that did not exist in previous pit designs. The purpose of drillhole GT19-09 is to better characterize geologic structure in Latte south wall. It is currently unclear whether an east-west striking, sub-vertical (orthogonal to foliation) joint set exists. If present, this set could lead to step path or toppling type instability in the south wall.

A thorough Geological and Geomechanical Bench Face Mapping Program should be undertaken, on a continuing basis, beginning in the early stages of development to verify structural conditions are consistent with assumptions presented herein and to identify local variations in structural conditions that might increase the risk of localized instabilities. The geomechanical data collection should concentrate on providing important data such as discontinuity persistence, spacing and variations in orientation that will allow further refinement of the bench design. The data collected should be used to confirm parameters used in the Geotechnical Models contained herein and, if determined to be other than assumed in this study, to further refine the analyses providing more accurate estimates of anticipated slope behavior.

As part of the Geologic Mapping Program, any significant structures or fault zones encountered should be mapped and digitized electronically and incorporated into the 3D Fault Model. This will allow projection of such structures to future pit slopes, highlighting areas of potentially instability and allow refinements to the slope design, if necessary. The accurate orientation and projection of fault structures is difficult based strictly on core drilling unless the structures cause a significant offset in mineralization, or a marker horizon is present. As such, the identification, mapping and analysis of fault structures of identified in pit walls will be a necessary and ongoing process during pit development.

A Slope Monitoring Program should be designed to ensure that the slopes are behaving as anticipated and warn if significant movements occur. The monitoring program should include a network of survey prisms monitored and analyzed regularly. If significant movements are noted, additional prisms should be installed along with extensometers to monitor any tension cracking.

11 CLOSURE

Analyses and recommendations presented herein are based on ultimate pit designs and resource as described in this report, and, as such, any significant changes to mine plans or pit configuration should be reviewed by JDS to verify that recommendations will remain valid for the new plans.

JDS is pleased to have the opportunity to be of service to Newmont and trusts that we have addressed the pertinent issues related to the Coffee feasibility pit slope update at this time. Should you, however, have any queries or comments on our visit or on the contents of this report, please do not hesitate to contact us.

Signed on this 9th day of August 2023.


Michael Levy, P.Eng.
Geotechnical Manager



12 REFERENCES

- Bieniawski Z.T., Engineering Rock Mass Classifications, John Wiley & Sons, New York, 1989.
- Call, R.D., Slope Stability, SME Mining Engineering Handbook, Ch.10.4, 1992.
- Hoek E., Strength of jointed rock masses. Twenty-third Rankine Lecture, Geotechnique, 1983:23 (3):187-223.
- Hoek E. and Brown E.T., Practical Estimates of Rock Mass Strength. International Journal of Rock Mechanics and Mining Sciences & Geomechanics Abstracts, 1997:34 (8):1165-1186.
- Hoek, E., Carranza-Torres CT, Corkum B., Hoek-Brown Failure Criterion – 2002 Edition. In: Proceedings of the Fifth International North American Rock Mechanics Symposium, Toronto, Canada, Vol. 1, 2002. p. 267-273.
- JDS Energy & Mining, Inc. (2016) NI 43-101 Feasibility Study Technical Report for the Coffee Gold Project, Report Date: February 18, 2016.
- Lorax Environmental Ltd., 2016, Coffee Gold Feasibility Study: Appendix J2: Hydrogeology, report prepared for Kaminak Gold Corporation dated February 1, 2016.
- Maptek Pty. Ltd., Lakewood, Colorado, 2015 Vulcan 9.1.
- Marinos, P. Hoek, E., GSI: Geologically Friendly Tool for Rock Mass Strength Estimation, Proc. GeoEng2000 Conference, Melbourne. 1422-1442.
- SRK Consulting (U.S.), Inc., Updated Preliminary Geotechnical Assessment for the Coffee Gold Project, draft report dated November 11, 2013.
- SRK Consulting (U.S.), Inc., 2015. Hydrogeologic Investigations Report, prepared for Kaminak Gold Corporation dated December 18, 2015.
- SRK Consulting (U.S.), Inc., 2016. Feasibility Pit Slope Stability Evaluation, Coffee Gold Project, prepared for Kaminak Gold Corporation dated March 11, 2016.
- Tetra Tech Canada Inc. (2019) Fall 2018 Testpitting Program Data Report, Coffee Gold Project. Report prepared for JDS Energy & Mining Inc. dated January 9, 2019.
- Rocscience, Inc., Toronto, Ontario, 2017. RocData, 5.009, Rock, soil and discontinuity strength analysis software.
- Rocscience, Inc., Toronto, Ontario, 2018. Slide 8.023, 2-dimensional limit equilibrium slope stability analysis software.
- Rocscience, Inc., Toronto, Ontario, 2019. Dips 7.015, Visualization and analysis software for orientation based geologic data.



APPENDIX A

GEOTECHNICAL CORE LOGS



DRILLHOLE LOG: GT18-01

AREA:
Supremo

PROJECT: Coffee Slope Stability Update

LOCATION: Yukon, Canada

SITE & PROJECT No: GOLCOF02E

BORING DATE: 2018-10-15 TO: 2018-10-18

DIP: -60

AZIMUTH: 255

COORDINATES: E: 584000.058 N: 6973569.835 Elev.: 1053.376

TOTAL DEPTH: 175

PAGE: 1 OF 4

DRILL TYPE: Diamond

CORE DIA.: HQ3

CASING DEPTH: 9.05

PLANNED HOLE ID: GT18-01

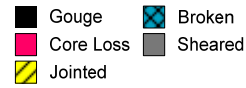
DEFINITIONS

IRS: Intact Rock Strength (field est.) UCS: Uniaxial Compressive Strength (MPa)
Pt Load: Point Load Test (MPa) FF/m: Fracture Frequency per metre
TCR: Total Core Recovery RQD: Rock Quality Designation
RMR: Rock Mass Rating

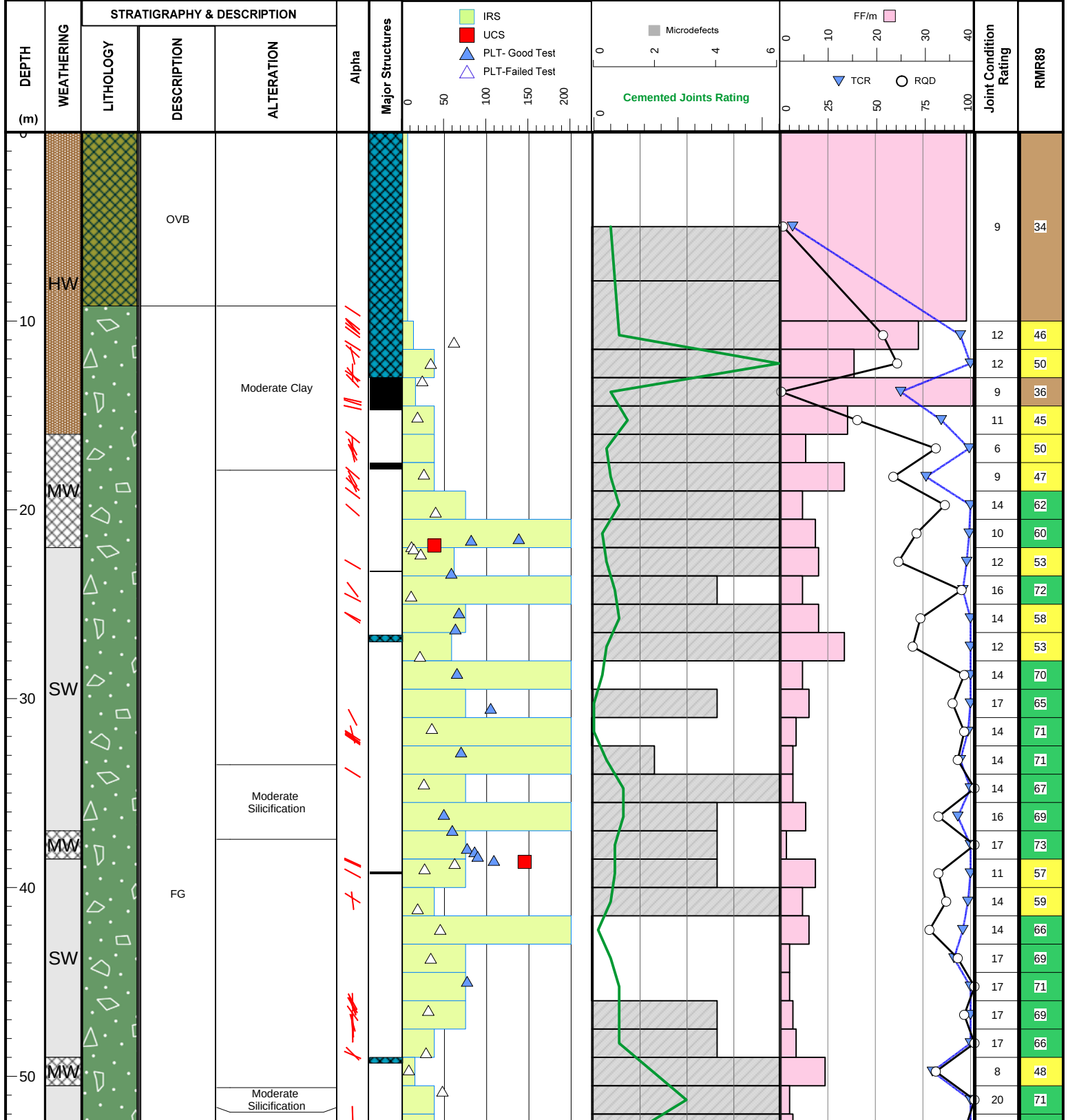
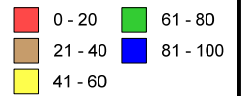
MAJOR LITHOLOGY



MAJOR STRUCTURES



LEGEND OF RMR





PLANNED HOLE ID: GT18-01

	0 - 20		61 - 80
	21 - 40		81 - 100
	41 - 60		

DEPTH (m)	WEATHERING	STRATIGRAPHY & DESCRIPTION			Alpha	Major Structures	TEST DATA				Cemented Joints Rating	FF/m	TCR	RQD	Joint Condition Rating	RMR89
		LITHOLOGY	DESCRIPTION	ALTERATION			IRS	UCS	PLT- Good Test	PLT- Failed Test						
58	SW	MxM													21	70
60	SW			Strong Silicification											21	68
62	SW														21	54
64	SW														21	53
66	SW														21	73
68	SW														21	73
70	SW														21	57
72	SW														21	61
74	SW														21	69
76	SW														21	68
78	SW														21	69
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152	SW														21	72
154	SW														21	67
156	SW														21	73
158	SW														21	72
160	SW														21	67
162	SW															

[illegible]



DRILLHOLE LOG: GT18-01

AREA:
Supremo

PROJECT: Coffee Slope Stability Update

LOCATION: Yukon, Canada

SITE & PROJECT No: GOLCOF02E

BORING DATE: 2018-10-15 TO: 2018-10-18

DIP: -60

AZIMUTH: 255

COORDINATES: E: 584000.058 N: 6973569.835 Elev.: 1053.376

TOTAL DEPTH: 175

PAGE: 4 OF 4

DRILL TYPE: Diamond

CORE DIA.: HQ3

CASING DEPTH: 9.05

PLANNED HOLE ID: GT18-01

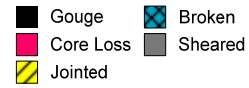
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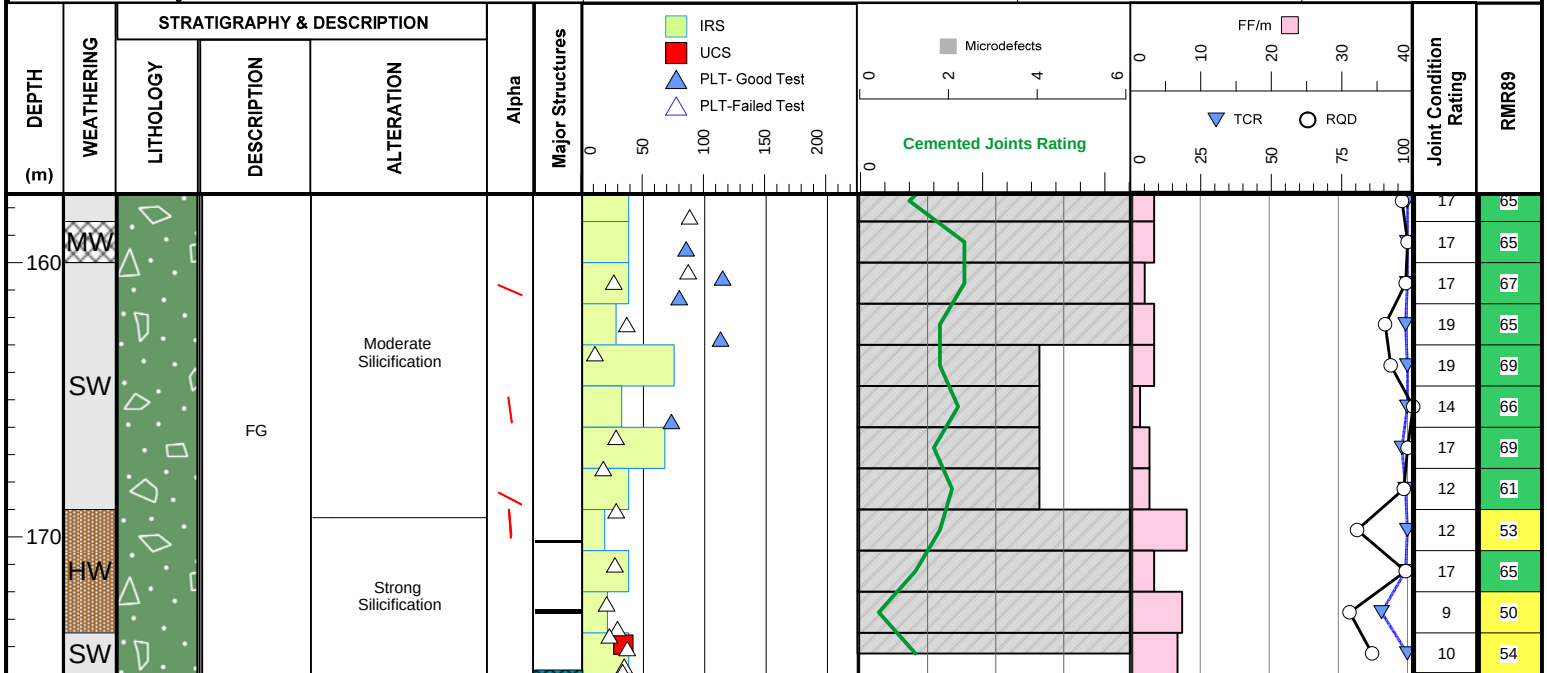
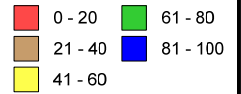
MAJOR LITHOLOGY



MAJOR STRUCTURES



LEGEND OF RMR





PLANNED HOLE ID: GT18-02

IRS: Intact Rock Strength (field est.)	UCS: Uniaxial Compressive Strength (MPa)
Pt Load: Point Load Test (MPa)	FF/m: Fracture Frequency per metre
TCR: Total Core Recovery	RQD: Rock Quality Designation
RMR: Rock Mass Rating	

	OVB		GG		Ylim
	BtS		HU		YC
	BtS_carb		IV		PB
	FC		MxM		YO
	FG		Yx		UM
	FLT		MxF		OG

 Gouge Broken
 Core Loss Sheared
 Jointed

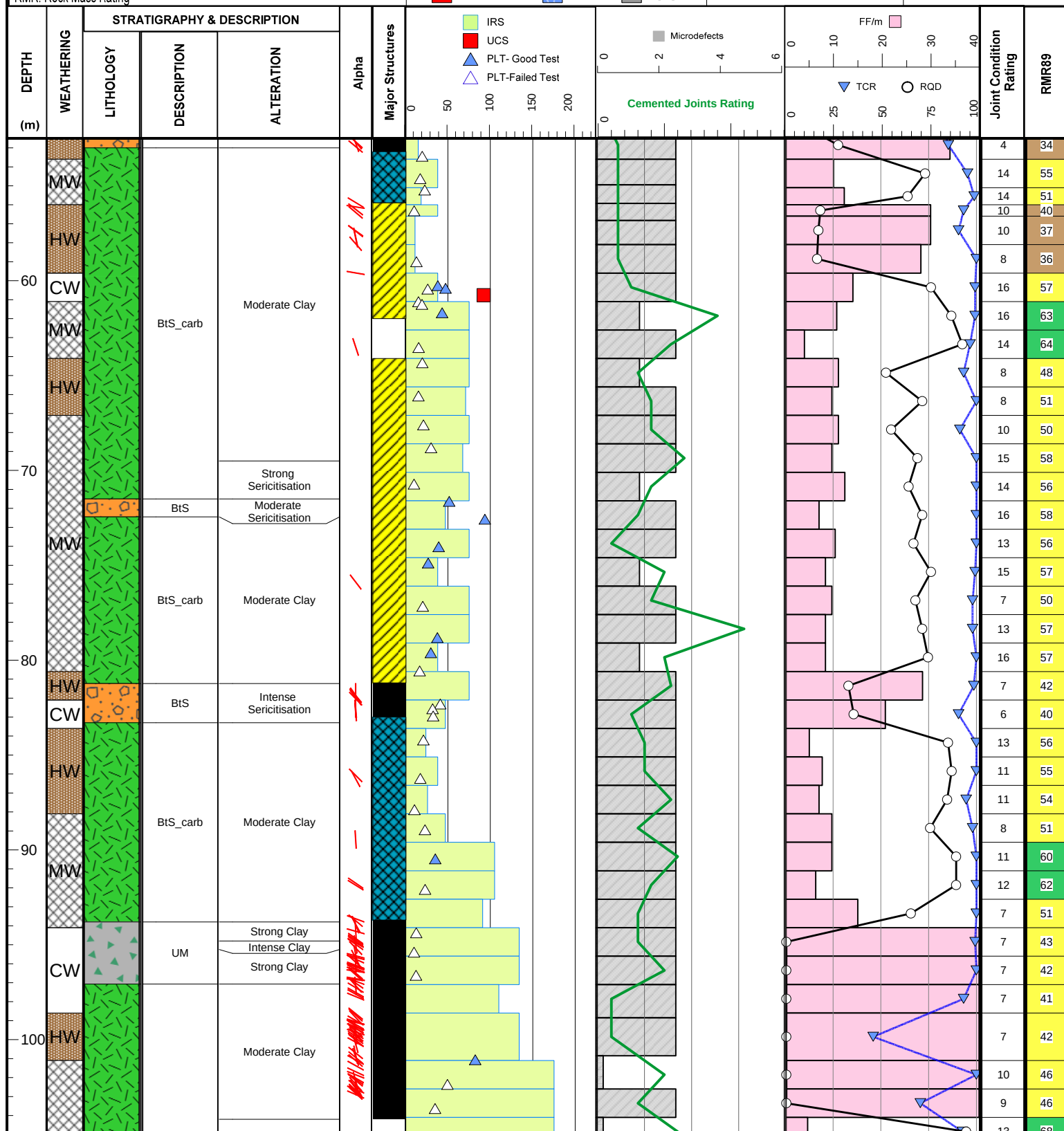
0 - 20	61 - 80
21 - 40	81 - 100
41 - 60	

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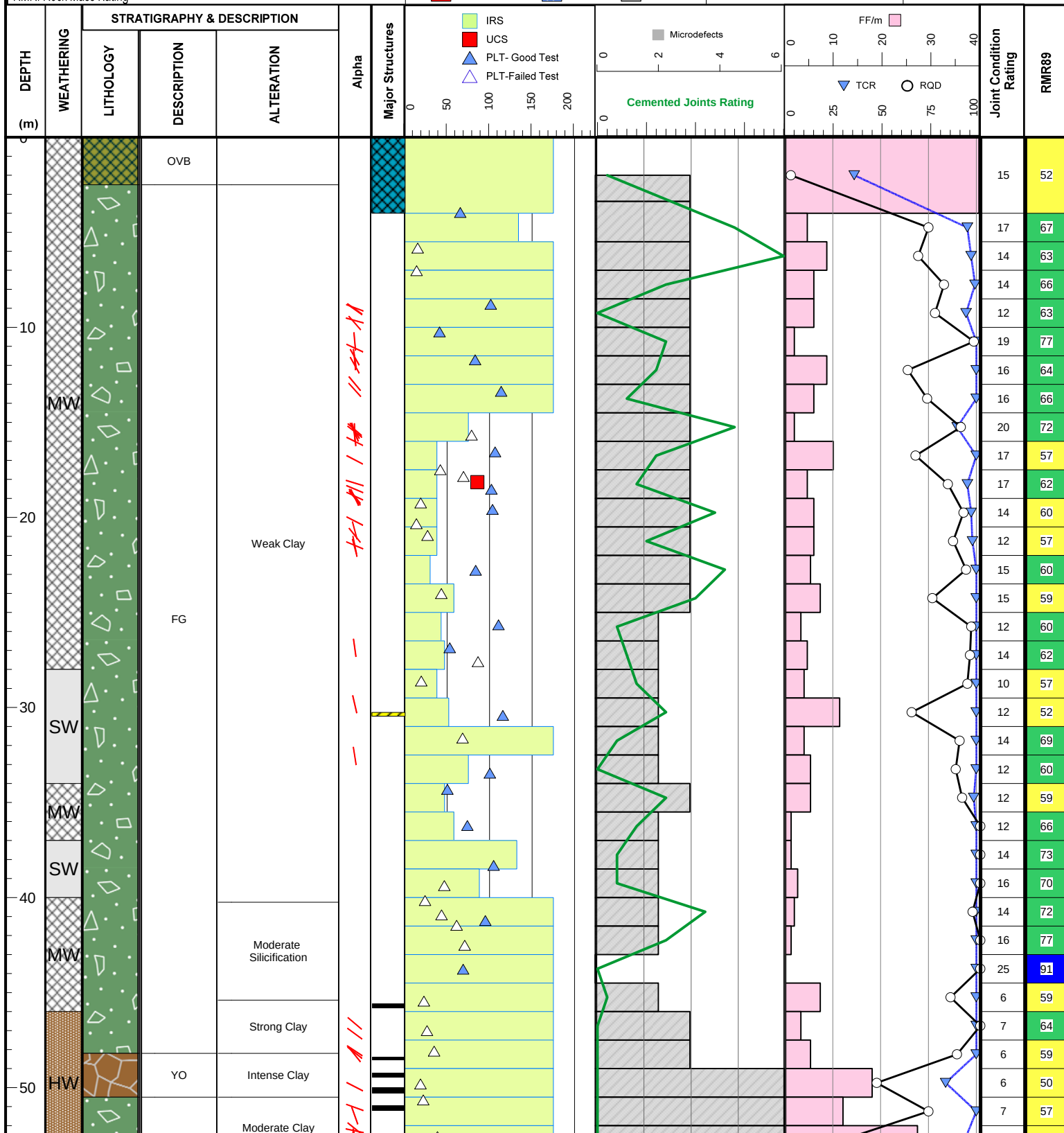
DEPTH (m)	WEATHERING	STRATIGRAPHY & DESCRIPTION			Alpha	Major Structures	Legend		Cemented Joints Rating		FF/m		Joint Condition Rating	RMR89
		LITHOLOGY	DESCRIPTION	ALTERATION			IRS UCS PLT- Good Test PLT-Failed Test	Microdefects	TCR RQD	TCR RQD				
110	SW	FG	Moderate Silicification	-	-	-	-	-	-	-	-	-	16	71
													12	72
													25	85
													19	75
													17	77
													12	72
													25	82
													14	71
													12	69
													17	69
													17	69
													12	62
													12	69
													25	82
													12	64
													12	64
													17	73
													17	67
													14	71
													12	62
													12	69
													19	76
													19	76
													21	78
													25	85
													25	85
													14	74
	MW												19	73
	SW												25	85
													17	71
													13	67
													17	77
													25	85
													17	68

[illegible]

[illegible]



DEPTH (m)	WEATHERING	STRATIGRAPHY & DESCRIPTION			Alpha	Major Structures	Geotechnical Data		Cemented Joints Rating		FF/m		Joint Condition Rating	RMR89		
		LITHOLOGY	DESCRIPTION	ALTERATION			IRS	UCS	PLT- Good Test	PLT-Failed Test	Microdefects				TCR	RQD
110	MW	BtS_carb	Weak Clay												14	69
														14	71	
														16	69	
														14	66	
														10	60	
														12	61	
														14	66	
														7	59	
120	MW		Strong Chlorite												15	67
														12	63	
														7	57	
														13	66	
130			Weak Clay												8	57
														14	65	
														14	65	
														9	61	
													10	58		
													16	64		
													9	58		
													15	62		
140	SW													9	56	
													13	59		
													17	68		
													13	62		
													15	65		
													5	51		
													13	64		
													16	69		
150	MW													10	55	
													17	69		
													16	70		
													16	67		
	SW														17	70





COORDINATES: E: 584458.692 N: 6974257.559 Elev.: 1251.525

PLANNED HOLE ID: GT18-04

IRS: Intact Rock Strength (field est.) UCS: Uniaxial Compressive Strength (MPa)
Pt Load: Point Load Test (MPa) FF/m: Fracture Frequency per metre
TCR: Total Core Recovery RQD: Rock Quality Designation
RMR: Rock Mass Rating

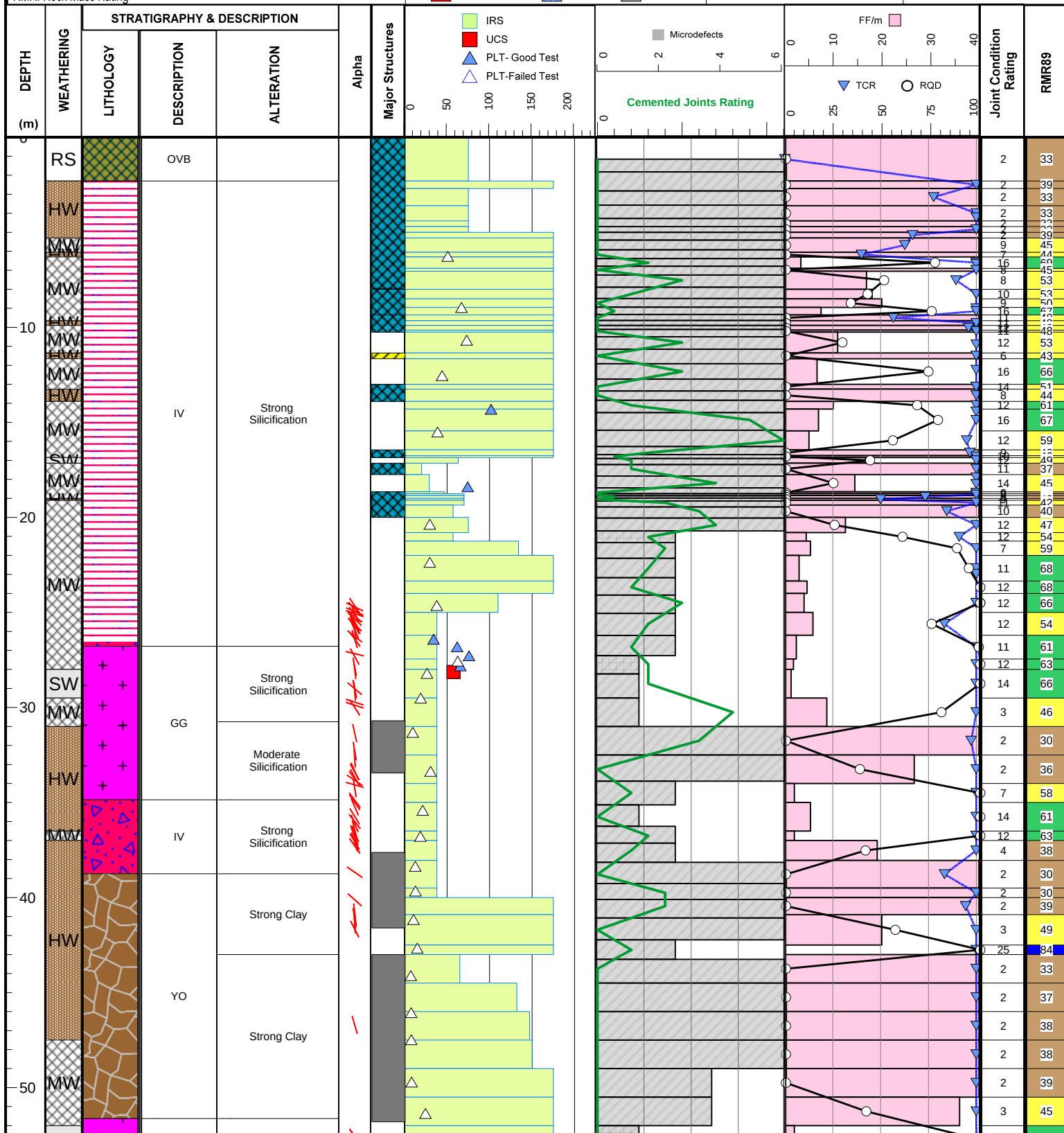
 Gouge Broken
 Core Loss Sheared
 Jointed

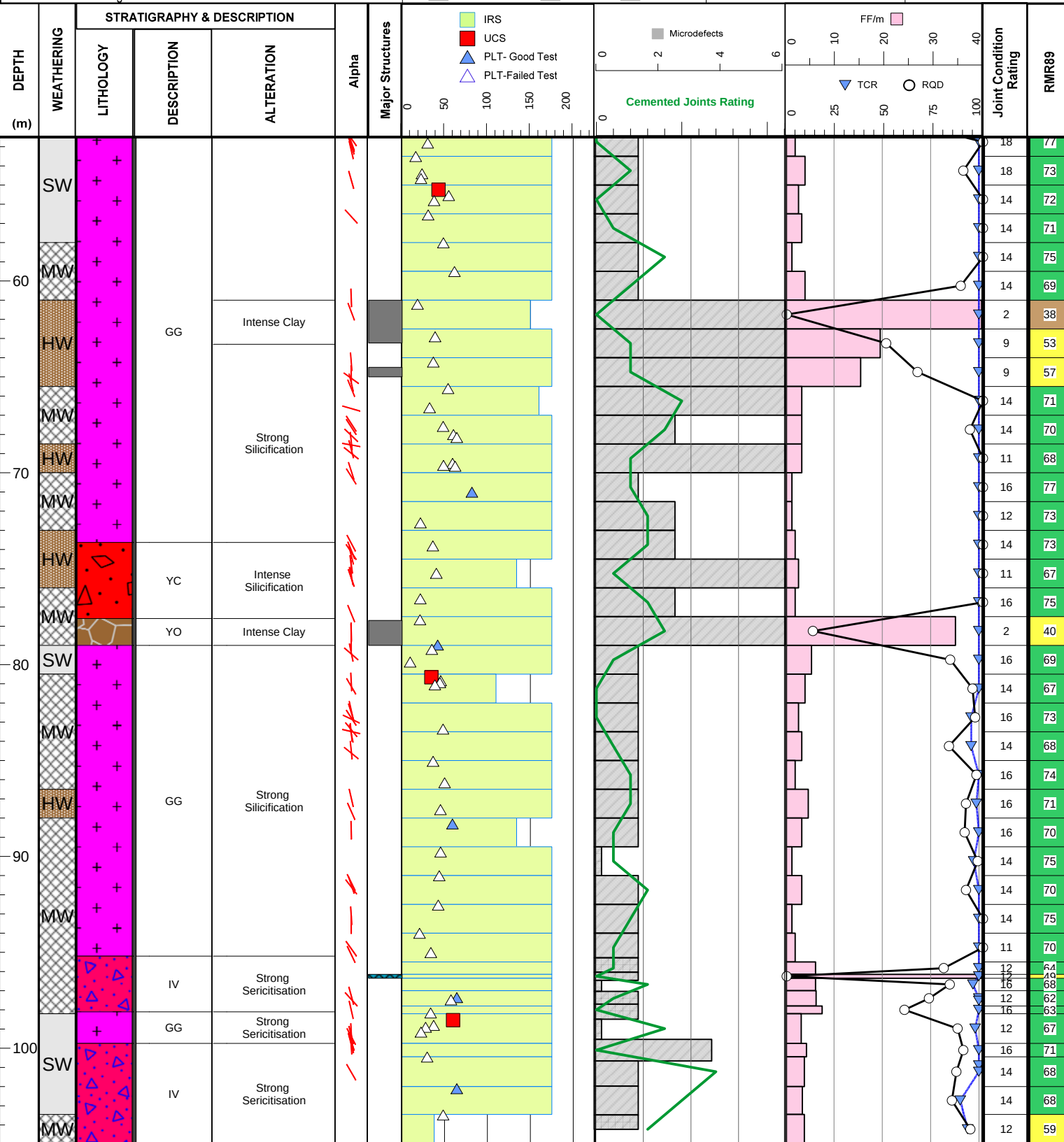
0 - 20	61 - 80
21 - 40	81 - 100
41 - 60	

DEPTH (m)	WEATHERING	STRATIGRAPHY & DESCRIPTION			Alpha	Major Structures	Cemented Joints Rating		FF/m		Joint Condition Rating	RMR89		
		LITHOLOGY	DESCRIPTION	ALTERATION			IRS	UCS	PLT- Good Test	PLT- Failed Test			Microdefects	TCR
58	MW	FG	Moderate Clay	-	-	IRS	UCS	PLT- Good Test	PLT- Failed Test	Microdefects	TCR	RQD	4	45
60	HW					IRS	UCS	PLT- Good Test	PLT- Failed Test	Microdefects	TCR	RQD	8	59
62	MW					IRS	UCS	PLT- Good Test	PLT- Failed Test	Microdefects	TCR	RQD	7	44
64	SW					IRS	UCS	PLT- Good Test	PLT- Failed Test	Microdefects	TCR	RQD	9	47
66	SW					IRS	UCS	PLT- Good Test	PLT- Failed Test	Microdefects	TCR	RQD	9	54
68	MW					IRS	UCS	PLT- Good Test	PLT- Failed Test	Microdefects	TCR	RQD	12	65
70	SW					IRS	UCS	PLT- Good Test	PLT- Failed Test	Microdefects	TCR	RQD	17	68
72	SW					IRS	UCS	PLT- Good Test	PLT- Failed Test	Microdefects	TCR	RQD	17	72
74	SW					IRS	UCS	PLT- Good Test	PLT- Failed Test	Microdefects	TCR	RQD	16	70
76	SW					IRS	UCS	PLT- Good Test	PLT- Failed Test	Microdefects	TCR	RQD	17	64
78	SW	IV	Moderate Silicification	-	-	IRS	UCS	PLT- Good Test	PLT- Failed Test	Microdefects	TCR	RQD	17	67
80	SW					IRS	UCS	PLT- Good Test	PLT- Failed Test	Microdefects	TCR	RQD	15	69
82	SW					IRS	UCS	PLT- Good Test	PLT- Failed Test	Microdefects	TCR	RQD	15	69
84	SW					IRS	UCS	PLT- Good Test	PLT- Failed Test	Microdefects	TCR	RQD	14	68
86	SW					IRS	UCS	PLT- Good Test	PLT- Failed Test	Microdefects	TCR	RQD	15	70
88	SW					IRS	UCS	PLT- Good Test	PLT- Failed Test	Microdefects	TCR	RQD	15	65
90	SW					IRS	UCS	PLT- Good Test	PLT- Failed Test	Microdefects	TCR	RQD	17	74
92	SW					IRS	UCS	PLT- Good Test	PLT- Failed Test	Microdefects	TCR	RQD	17	74
94	SW					IRS	UCS	PLT- Good Test	PLT- Failed Test	Microdefects	TCR	RQD	15	70
96	SW					IRS	UCS	PLT- Good Test	PLT- Failed Test	Microdefects	TCR	RQD	15	72
98	SW	V	Moderate Silicification	-	-	IRS	UCS	PLT- Good Test	PLT- Failed Test	Microdefects	TCR	RQD	15	68
100	SW					IRS	UCS	PLT- Good Test	PLT- Failed Test	Microdefects	TCR	RQD	15	59
102	SW					IRS	UCS	PLT- Good Test	PLT- Failed Test	Microdefects	TCR	RQD	14	63
104	SW					IRS	UCS	PLT- Good Test	PLT- Failed Test	Microdefects	TCR	RQD	15	64
106	SW					IRS	UCS	PLT- Good Test	PLT- Failed Test	Microdefects	TCR	RQD	12	61
108	SW					IRS	UCS	PLT- Good Test	PLT- Failed Test	Microdefects	TCR	RQD	16	62
110	SW					IRS	UCS	PLT- Good Test	PLT- Failed Test	Microdefects	TCR	RQD	18	69
112	SW					IRS	UCS	PLT- Good Test	PLT- Failed Test	Microdefects	TCR	RQD	14	62
114	SW	VI	Moderate Silicification	-	-	IRS	UCS	PLT- Good Test	PLT- Failed Test	Microdefects	TCR	RQD	16	57
116	SW					IRS	UCS	PLT- Good Test	PLT- Failed Test	Microdefects	TCR	RQD	16	62
118	SW					IRS	UCS	PLT- Good Test	PLT- Failed Test	Microdefects	TCR	RQD	12	54
120	SW					IRS	UCS	PLT- Good Test	PLT- Failed Test	Microdefects	TCR	RQD	14	61
122	SW	VII	Moderate Silicification	-	-	IRS	UCS	PLT- Good Test	PLT- Failed Test	Microdefects	TCR	RQD	14	60
124	SW					IRS	UCS	PLT- Good Test	PLT- Failed Test	Microdefects	TCR	RQD	18	68
126	SW	VIII	Moderate Silicification	-	-	IRS	UCS	PLT- Good Test	PLT- Failed Test	Microdefects	TCR	RQD	18	68
128	SW					IRS	UCS	PLT- Good Test	PLT- Failed Test	Microdefects	TCR	RQD	18	68

DEPTH (m)	WEATHERING	STRATIGRAPHY & DESCRIPTION			Alpha	Major Structures	Geotechnical Data		Cemented Joints Rating	FF/m	TCR	RQD	Joint Condition Rating	RMR89		
		LITHOLOGY	DESCRIPTION	ALTERATION			IRS	UCS							PLT- Good Test	PLT- Failed Test
110	FR	FR	FG	Moderate Silicification		IRS	UCS	PLT- Good Test	PLT- Failed Test	Microdefects	TCR	RQD	9	65		
															16	62
															12	61
															9	63
															16	63
															19	68
															19	77
															25	34
															17	65
															17	77
120		SW	FG	Moderate Silicification									14	67		
															19	35
															25	91
															12	75
															19	35
															16	32
															14	71
															14	72
															25	91
															17	74
130	FR	FR	FG										19	75		
															25	91
															16	77
															14	80
															14	72
															14	73
															14	72
															14	80
															12	73
															25	91
140	MW	SW	FG										25	34		
															25	91
															25	91
															25	91
															25	91
															16	32
															12	78

DEPTH (m)	WEATHERING	STRATIGRAPHY & DESCRIPTION			Alpha	Major Structures	Cemented Joints Rating		FF/m		Joint Condition Rating	RMR89
		LITHOLOGY	DESCRIPTION	ALTERATION			IRS	UCS	PLT- Good Test	PLT-Failed Test		
160	FR			Moderate Silicification						25	86	
										12	73	
										16	82	
										19	80	
										12	69	
										12	70	
										25	91	
										15	81	
										17	76	
										12	70	
	SW			Moderate Silicification						19	77	
										14	73	
										12	78	
										14	75	
										25	91	
										14	75	
										14	73	
										18	84	
										14	72	
										25	84	
	FR			Moderate Silicification						16	76	
										25	85	
										16	76	
										18	84	
										12	71	
										25	85	
										16	76	
										18	84	
										12	71	
										25	85	







PLANNED HOLE ID: GT18-06

IRS: Intact Rock Strength (field est.) UCS: Uniaxial Compressive Strength (MPa)
Pt Load: Point Load Test (MPa) FF/m: Fracture Frequency per metre
TCR: Total Core Recovery RQD: Rock Quality Designation
RMR: Rock Mass Rating

Figure 1 displays 15 color-coded icons representing different land use/cover types, arranged in a 3x5 grid. The icons are as follows:

- Row 1: OVB (orange), GG (green), YLim (yellow)
- Row 2: BtS (blue), HU (brown), YC (red)
- Row 3: BtS_carb (blue), IV (purple), PB (purple), FC (green), MxM (green), YO (green), FG (green), Yx (green), UM (green), FLT (red), MxF (blue), and OG (grey)

 Gouge Broken
 Core Loss Sheared
 Jointed

0 - 20
 61 - 80

21 - 40
 81 - 100

41 - 60

[illegible]



DRILLHOLE LOG: GT18-06

AREA:
Supremo

PROJECT: Coffee Slope Stability Update

LOCATION: Yukon, Canada

SITE & PROJECT No: GOLCOF02E

BORING DATE: 2018-10-24 TO: 2018-10-27

DIP: -60

AZIMUTH: 70

COORDINATES: E: 584552.186 N: 6974147.563 Elev.: 1236.928

TOTAL DEPTH: 179.5

PAGE: 2 OF 4

DRILL TYPE: Diamond

CORE DIA.: HQ3

CASING DEPTH:

PLANNED HOLE ID: GT18-06

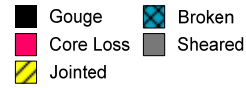
DEFINITIONS

IRS: Intact Rock Strength (field est.) UCS: Uniaxial Compressive Strength (MPa)
Pt Load: Point Load Test (MPa) FF/m: Fracture Frequency per metre
TCR: Total Core Recovery RQD: Rock Quality Designation
RMR: Rock Mass Rating

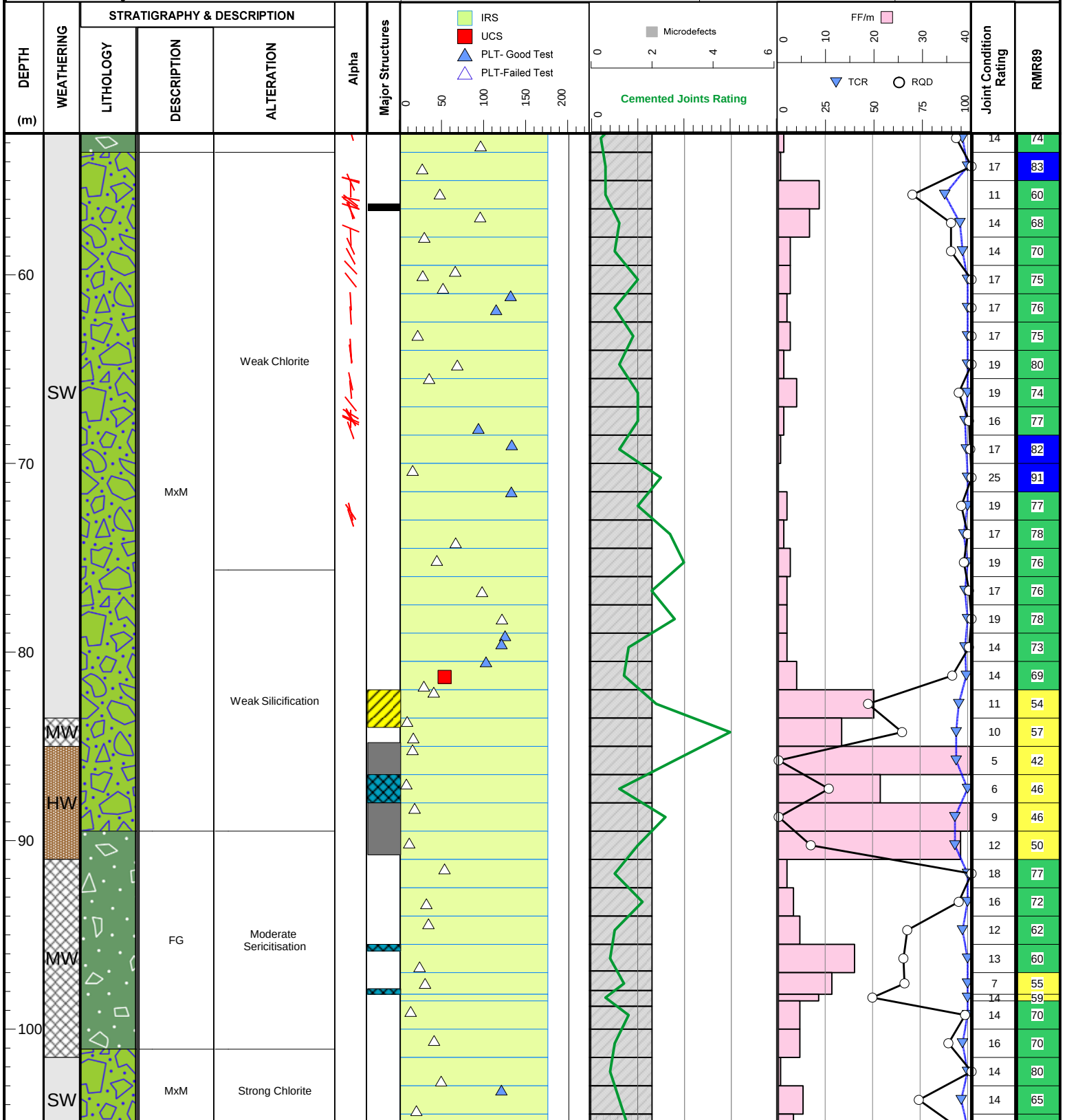
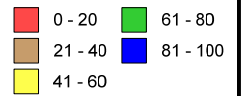
MAJOR LITHOLOGY



MAJOR STRUCTURES



LEGEND OF RMR



DEPTH (m)	WEATHERING	STRATIGRAPHY & DESCRIPTION			Alpha	Major Structures	Geotechnical Data		Cemented Joints Rating	Rock Quality Indicators		Joint Condition Rating	RMR89			
		LITHOLOGY	DESCRIPTION	ALTERATION			IRS	UCS		PLT- Good Test	PLT-Failed Test			FF/m	TCR	RQD
110	MW	FG		Moderate Sericitisation								12	68			
														12	60	
														8	51	
														5	47	
														5	53	
														13	57	
														14	66	
														17	74	
														17	76	
														17	75	
120	SW		Weak Silicification									15	71			
												12	77			
												12	71			
												17	83			
												12	61			
												12	68			
												16	66			
												14	66			
												16	72			
												12	71			
												12	66			
												14	72			
140	SW		FG	Weak Clay									17	78		
															25	91
															14	73
															16	77
															16	75
															14	80
															14	68
															16	72
															14	79
															14	74
												14	72			
												14	75			
												25	91			

[illegible]

[illegible]

DRILLHOLE LOG:
GT18-07

AREA:
Latte

PROJECT: Coffee Slope Stability Update

LOCATION: Yukon, Canada

SITE & PROJECT No: GOLCOF02E

BORING DATE: 2018-10-01 **TO:** 2018-10-03

DIP: -70

AZIMUTH: 160

COORDINATES: E: 583126.872 N: 6973167.4

Elev.: 1115.343

TOTAL DEPTH: 145

PAGE: 2 OF 3

DRILL TYPE: Diamond

CORE DIA.: HQ3

CASING DEPTH: 3.91

PLANNED HOLE ID: GT18-07

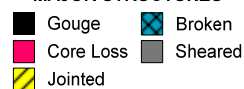
DEFINITIONS

IRS: Intact Rock Strength (field est.) UCS: Uniaxial Compressive Strength (MPa)
Pt Load: Point Load Test (MPa) FF/m: Fracture Frequency per metre
TCR: Total Core Recovery RQD: Rock Quality Designation
RMR: Rock Mass Rating

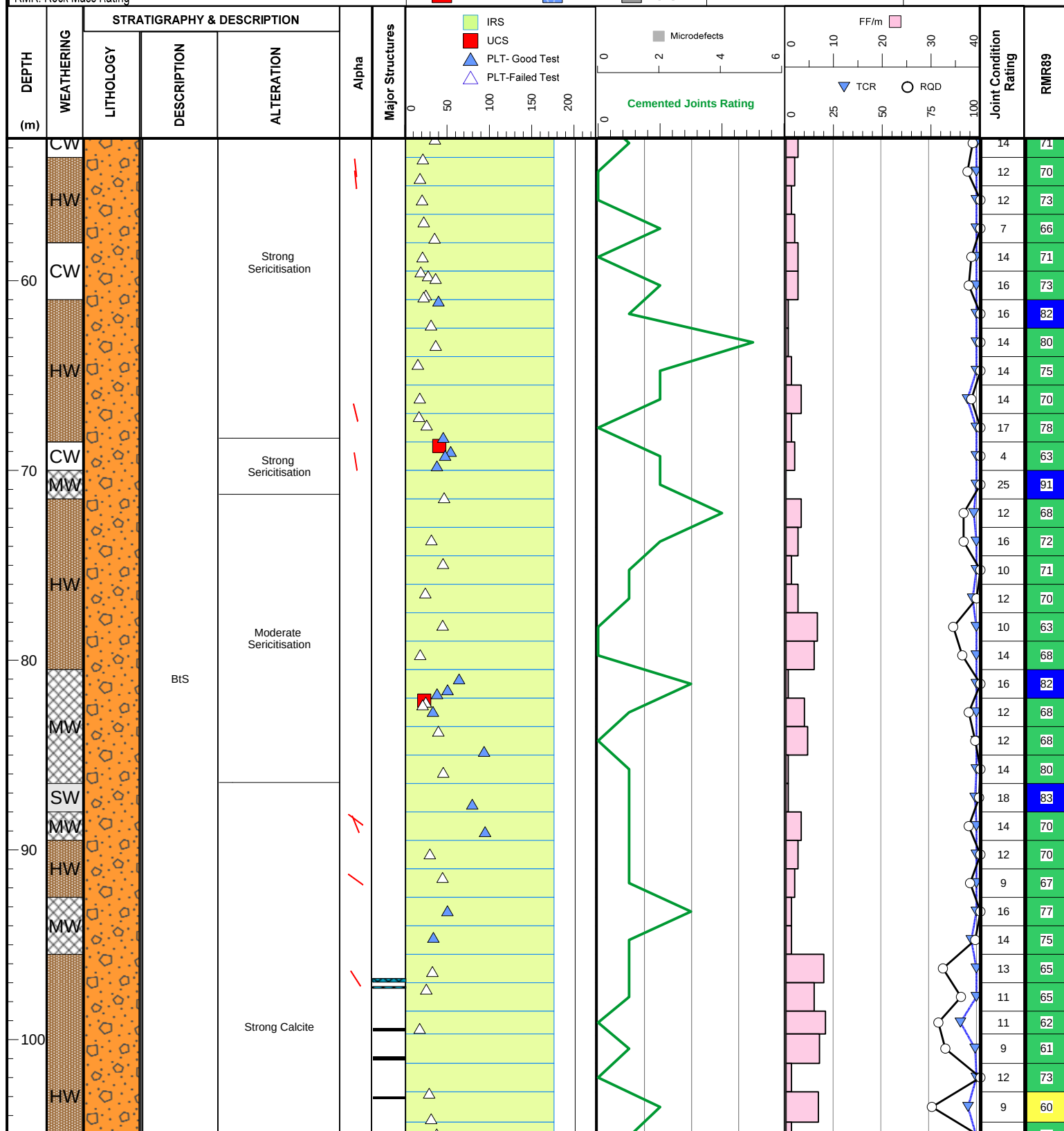
MAJOR LITHOLOGY



MAJOR STRUCTURES



LEGEND OF RMR





DRILLHOLE LOG: GT18-07

AREA:
Latte

PROJECT: Coffee Slope Stability Update

LOCATION: Yukon, Canada

SITE & PROJECT No: GOLCOF02E

BORING DATE: 2018-10-01 TO: 2018-10-03

DIP: -70

AZIMUTH: 160

COORDINATES: E: 583126.872 N: 6973167.4

Elev.: 1115.343

TOTAL DEPTH: 145

PAGE: 3 OF 3

DRILL TYPE: Diamond

CORE DIA.: HQ3

CASING DEPTH: 3.91

PLANNED HOLE ID: GT18-07

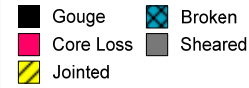
DEFINITIONS

IRS: Intact Rock Strength (field est.) UCS: Uniaxial Compressive Strength (MPa)
Pt Load: Point Load Test (MPa) FF/m: Fracture Frequency per metre
TCR: Total Core Recovery RQD: Rock Quality Designation
RMR: Rock Mass Rating

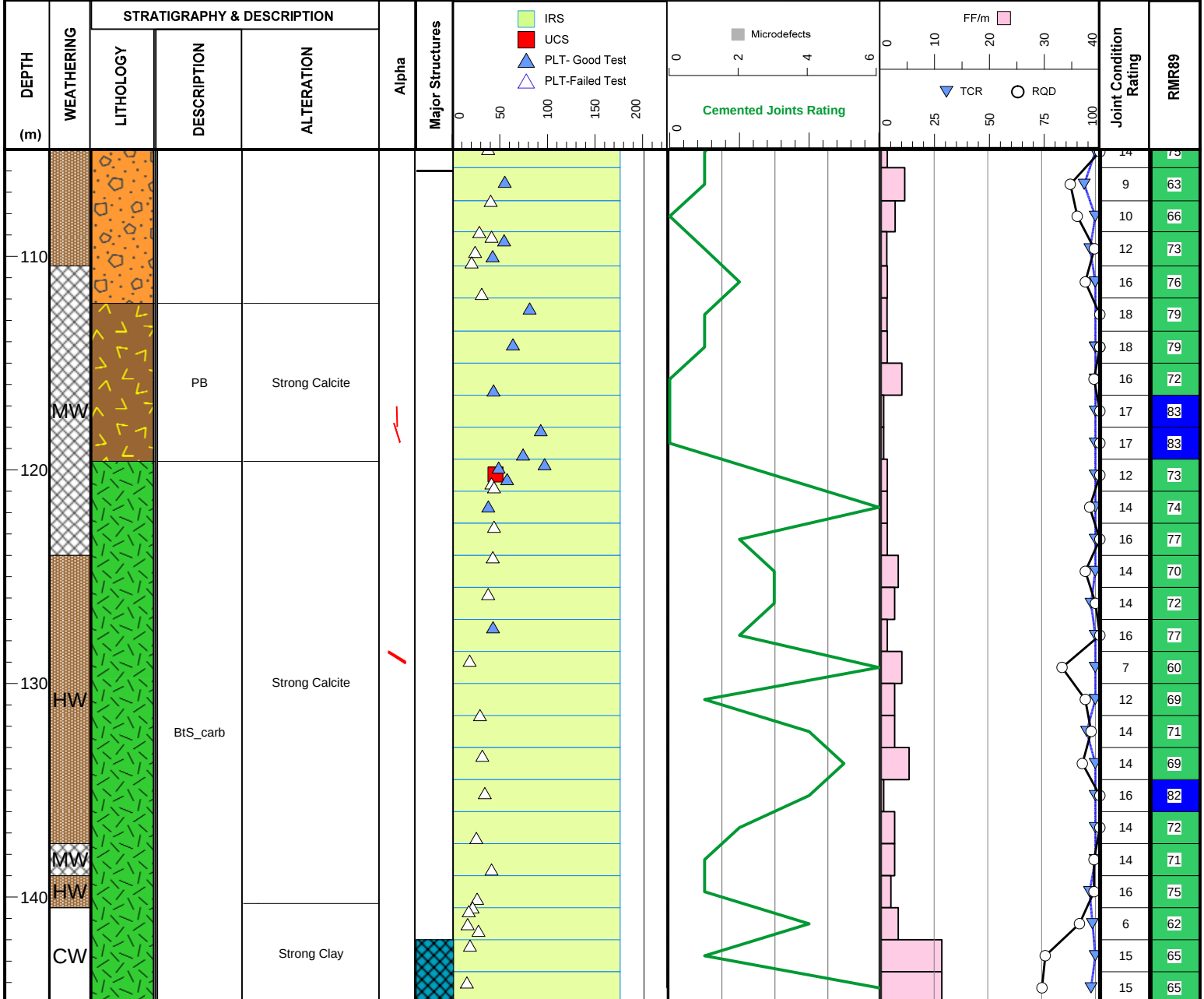
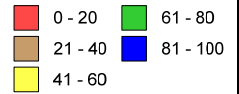
MAJOR LITHOLOGY



MAJOR STRUCTURES



LEGEND OF RMR



APPENDIX B

LABORATORY TEST RESULTS



DEPARTMENT OF
MINING ENGINEERING

Goodwin Hall
Queen's University
Kingston, Ontario, Canada K7L 3N6
Tel 613 533-2230
Fax 613 533-6597

March 11, 2019

Mr. Michael Levy
JDS Mining & Energy Inc.
14143 Denver West Parkway
Suite 100
Golden, CO 80401

Re: Core Testing – JDS Energy Corp. Coffee Project (P.O. #6448)

Mr. Masson:

One shipment of containers containing a total of sixty-nine (69) rock core specimens, including eleven spare direct shear and eight broken or limited length (too short to test) specimens was delivered to Queen's University for the purpose of conducting various strength tests (including UCS, triaxial confined compression, joint direct shear and Brazilian indirect tensile strength) on these specimens.

Tests were conducted according to the standards of the following organizations:

- The Complete ISRM (International Society of Rock Mechanics) Suggested Methods for Rock Characterization, Testing and Monitoring: 1974-2006. April, 2007 Edition Prepared by the Commission on Testing Methods of the International Society for Rock Mechanics. Edited by R. Ulusay and J. A. Hudson. ISBN 978-975-93675-4-1.
- ASTM International Standards

Specific test protocols of each organization that were adhered to in conducting determinations that were undertaken for this report included:

-ISRM Suggested Methods for Determining Water Content, Porosity, Density, Absorption and Related Properties, pp. 85-98.

-ISRM Suggested Methods for Determining the Uniaxial Compressive Strength and Deformability of Rock Materials, pp. 151-156.

(also in accordance with Standard Test Methods for Compressive Strength and Elastic Moduli of Intact Rock Core Specimens Under Varying States of Stress and Temperatures – ASTM D 7012-14)

-ISRM Suggested Methods for Determining the Strength of Rock Materials in Triaxial Compression, pp. 157-164.

(Standard Test Methods for Compressive Strength and Elastic Moduli of Intact Rock Core Specimens Under Varying States of Stress and Temperatures – ASTM D 7012-14)

-ISRM Suggested Methods for Determining Tensile Strength of Rock Materials, (a) (Direct Tensile Strength); (b) (Brazilian Indirect Tensile Strength), pp. 182-183.

(also in accordance with (a) (Standard Test method for Direct Tensile Strength of Intact Rock Core Specimens – ASTM D 2936 – 08) and

(b) (Standard Test Method for Splitting Tensile Strength of Intact Rock Core Specimens – ASTM D 3967 – 08)), and

-ISRM Suggested Methods for Determining Shear Strength, pp. 165-176.

(Standard Test Method for Performing Laboratory Direct Shear Strength Tests on Rock Specimens Under Constant Normal Force – ASTM D 5607-08)

The samples that were subjected to unconfined and confined compression failure testing were subjected to a process of preparation that included:

- diamond sawing to prepare cylindrical samples having nearly parallel end faces at length-to-diameter aspect ratios no greater than 2.5-to-1 and no less than 2.0-to-1, where possible and depending on core lengths delivered

- diamond lathing, to prepare sample faces parallel to within ± 0.025 mm

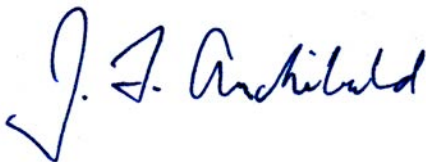
- testing to failure within a Materials Testing System (MTS) Model 815 servo-controlled, electrohydraulic compression testing frame

These compression tests were performed under axial strain control at rates approximating 10^{-5} s⁻¹. For each unconfined compression strength test, simultaneous recordings of axial force, axial deformation (using strain gauges) and circumferential deformation (using strain gauges) were performed from which determination of standard failure parameters (UCS, Young's Modulus and Poisson's ratio) were made. For triaxial confined compression tests, no circumferential (ie.- lateral) deformation/strain was measured, thus no Poisson's ratio determinations were made. A summary of test data achieved for all successful tests, including photographs of pre- and post-test Brazilian tensile strength test specimens, is included in this portion of the submitted report (PART A).

Separate electronic file summaries, according to ASTM requirements that illustrate the pre- and post-test images and stress/strain responses of unconfined, triaxial confined and direct shear specimens, are also attached in a separate submission as PART B of this report.

Should you also require any additional information concerning work that has been performed, please do not hesitate to contact me by telephone at (613)-533-2198 or by FAX at (613)-533-6597.

Yours sincerely,

A handwritten signature in blue ink, reading "J. F. Archibald". The signature is fluid and cursive, with the first letters of each name being capitalized and prominent.

J. F. Archibald, Ph.D., P. Eng., FCIM

Summary of Strength Test Results
JDS Energy & Mining Inc. Coffee Gold Project (March, 2019)

Borehole (Depth, m)	Density (g/cm ³)	Young's Modulus (GPa)	Poisson's ratio	UCS (MPa)	Sigma 1 (MPa)	Sigma 3 (MPa)	Brazilian Indirect Tensile Strength (Values)/Average (MPa)
GT18-01 (21.79-22.00)	2.63	---	---	30.2 pf	---	---	---
GT18-01 (38.50-38.79)	2.65	23.892	0.16	134.2 pf	---	---	(3.4, 9.8, 10.9, 9.5)/ 8.4
GT18-01 (59.50-59.74)	2.70	---	---	57.6 pf	---	---	---
GT18-01 (73.00-73.45)	2.64	18.636	---	---	36.1 (f)	0.5	---
					---	2.0	
					---	5.0	
					---	10.0	
GT18-01 (99.42-99.70)	2.67	18.835	---	49.4 f	---	---	(16.4, 11.9, 12.4)/
GT18-01 (119.50-119.72)	2.62	Broken	Broken	Broken		1.0	---
						4.0	---
						10.0	---
						20.0	--
GT18-01 (139.68-139.92)	2.69	---	---	48.0 pf	---	---	---
GT18-01 (78.27-78.77)	2.67	18.784	---	---	46.5	1.0	---
					59.7	3.0	
					70.0	6.0	
					88.5 (pf)	12.0	
GT18-01 (173.80-174.05)	2.55	---	---	26.0 f	---	---	---
GT18-02 (19.98-20.23)	2.61	---	---	69.4 pf	---	---	(10.1, 10.2, 10.9)/ 10.4
GT18-02 (40.05-40.34)	2.65	26.802	---	---	128.4	1.0	---
					179.1	3.0	
					194.2	6.0	
					234.7 (pf)	12.0	
GT18-02 (61.23-61.50)	Broken	Broken	Broken	Broken	---	---	---
GT18-02 (82.06-82.28)	2.60	24.734	---	---	124.1	1.5	---
					155.6	4.0	
					196.9	9.0	
					227.9	15.0	
GT18-02 (100.21-100.50)	Broken	Broken	Broken	Broken	---	---	(4.3, 2.63, 16.6, 15.0)/ 9.6
GT18-02 (119.45-119.71)	2.92	---	---	47.2 f	---	---	(13.2, 10.7, 13.7)/ 12.5
GT18-02 (140.03-140.27)	2.64	27.806	---	---	239.2	2.0	---
					241.5	5.0	
					---	11.0	
					---	25.0	
GT18-02 (143.52-144.00)	2.61	---	---	83.2 (pf)	---	---	---
GT18-02 (159.69-159.93)	2.63	---	---	95.4 pf	---	---	(17.3, 22.1, 16.8)/ 18.7
GT18-02 (161.51-161.75)	256	19.111	0.11	45.0 pf	---	---	---
GT18-02 (181.16-181.40)	Broken	Broken	Broken	Broken	---	---	---
GT18-03 (13.00-13.26)	2.70	12.342	0.11	25.6 (pf)	---	---	---
GT18-03 (40.69-40.93)	Broken	Broken	Broken	Broken	---	---	(5.2, 3.9, 4.0, 4.4, 5.4, 2.6)/ 4.3

- (pf) - indicates that failure occurred partially along pre-existing foliation surface
(f) - indicates that failure occurred fully along pre-existing foliation surface
(*) - direct shear sample failure data listed in following table

Borehole (Depth, m)	Density (g/cm ³)	Young's Modulus (GPa)	Poisson's ratio	UCS (MPa)	Sigma 1 (MPa)	Sigma 3 (MPa)	Brazilian Indirect Tensile Strength (Values)/Average (MPa)
GT18-03 (60.63-60.90)	2.70	17.216	0.09	82.9 (pf)	---	---	(6.4, 7.2, 14.4)/ 9.3
GT18-03 (118.82-119.03)	2.82	22.529	---	---	57.6	1.0	---
					82.0	4.0	
					120.9	10.0	
					180.6 (pf)	20.0	
GT18-03 (139.55-139.78)	2.70	---	---	49.4 pf	---	---	---
GT18-03 (154.02-154.26)	2.77	14.911	---	---	102.5	1.5	---
					115.1	4.0	
					131.3	9.0	
					150.9 (pf)	15.0	
GT18-04 (18.01-18.29)	2.62	---	---	76.5	---	---	(12.8, 9.6, 14.1, 10.5)/ 11.8
GT18-04 (41.50-41.70)	2.58	15.318	---	---	42.4	0.5	---
					92.1	3.0	
					137.4	6.0	
					147.5 (f)	12.0	
GT18-04 (60.46-60.78)	2.62	28.037	0.18	119.7	---	---	---
GT18-04 (80.50-80.85)	2.88	---	---	51.7 (pf)	---	---	---
GT18-04 (100.10-100.30)	2.63	27.447	---	---	22.1	1.0	---
					40.5	4.0	
					68.5	10.0	
					160.8 (pf)	20.0	
GT18-04 (122.10-122.36)	2.61	19.578	0.28	99.0	---	---	(18.0, 12.9, 17.4, 19.6)/ 17.0
GT18-04 (140.91-141.24)	2.60	---	---	50.3 (pf)	---	---	---
GT18-04 (143.86-144.34)	2.62	---	---	47.7 (pf)	---	---	---
GT18-04 (158.50-158.75)	2.66	---	---	125.5 (pf)	---	---	(13.7, 16.8, 17.6)/ 16.0
GT18-04 (180.31-180.57)	2.65	26.523	---	---	150.4 (pf)	2.0	---
					---	5.0	
					---	11.0	
					---	25.0	
GT18-04 (194.72-194.99)	2.61	---	---	92.6 (pf)	---	---	(16.1, 9.0, 13.1)/ 12.7
GT18-05 (28.00-28.27)	2.57	17.393	0.20	49.1 (pf)	---	---	---
GT18-05 (55.10-55.39)	2.43	---	---	35.0	---	---	(2.4, 2.2, 3.2)/ 2.6
GT18-05 (68.65-68.89)	Broken	Broken	Broken	Broken	---	---	---
GT18-05 (80.55-80.78)	2.37	---	---	26.9	---	---	---
GT18-05 (98.42-98.65)	2.60	13.568	0.07	51.5	---	---	---
GT18-06 (19.92-20.27)	2.63	---	---	83.9 (pf)	---	---	---
GT18-06 (40.00-40.27)	2.65	30.683	0.11	143.5 (pf)	---	---	---
GT18-06 (60.45-60.66)	2.85	28.359	---	---	29.9	1.0	---
					32.2	3.0	
					40.2	6.0	
					136.2	12.0	

(pf) - indicates that failure occurred partially along pre-existing foliation surface

(f) - indicates that failure occurred fully along pre-existing foliation surface

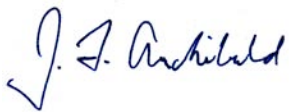
(*) - direct shear sample failure data listed in following table

Borehole (Depth, m)	Density (g/cm ³)	Young's Modulus (GPa)	Poisson's ratio	UCS (MPa)	Sigma 1 (MPa)	Sigma 3 (MPa)	Brazilian Indirect Tensile Strength (Values)/Average (MPa)
GT18-06 (81.20-81.44)	3.01	---	---	44.4 (pf)	---	---	---
GT18-06 (120.43-120.70)	Broken	Broken	Broken	Broken	---	---	(5.7, 4.7, 8.7)/ 6.4
GT18-06 (139.00-139.30)	2.66	25.600	---	---	90.5	1.0	---
					117.4	4.0	
					146.2	10.0	
					166.7	20.0	
GT18-06 (160.47-160.78)	2.43	---	---	44.5	---	---	(4.8, 4.7, 4.9)/ 4.8
GT18-06 (178.37-178.63)	2.65	---	---	90.9 (pf)	---	---	---
GT18-07 (15.25-15.51)	2.30	---	---	19.4 (pf)	---	---	(3.6, 2.6, 1.5)/ 2.6
GT18-07 (40.00-40.24)	2.64	23.258	---	---	53.1	0.5	---
					61. (pf)5	2.0	
					---	5.0	
					----	10.0	
GT18-07 (68.58-68.84)	2.54	4.984	0.09	32.0 (pf)	---	---	---
GT18-07 (82.00-82.27)	2.45	---	---	14.8 (pf)	---	---	(3.4, 3.9, 4.7)/ 4.0
GT18-07 (109.39-109.63)	2.36	17.346	---	---	35.6	0.5	---
					48.7	3.0	
					71.6	6.0	
					107.1 (pf)	12.0	
GT18-07 (120.06-120.38)	2.52	13.617	0.29	36.4 (pf)	---	---	(2.9, 1.8, 4.2)/ 3.0
GT18-03 (62.88-63.18)*	---	---	---	---	---	---	---
GT18-07 (56.50-56.80)*	---	---	---	---	---	---	---
GT18-01 (148.33-148.68)*	---	---	---	---	---	---	---
GT18-04 (164.50-165.14)*	---	---	---	---	---	---	---
GT18-04 (176.14-176.59)*	---	---	---	---	---	---	---
SPARE Direct Shear	Not	Tested					
DS-01 (9.70-9.98)*	---	---	---	---	---	---	---
DS-01 (13.00-13.66)*	---	---	---	---	---	---	---
DS-01 (62.60-62.88)*	---	---	---	---	---	---	---
DS-01 (73.00-73.45)*	---	---	---	---	---	---	---
DS-01 (78.27-78.77)*	---	---	---	---	---	---	---
DS-01 (81.81-82.24)*	---	---	---	---	---	---	---
DS-01 (143.52-144.00)*	---	---	---	---	---	---	---
DS-01 (143.86-144.34)*	---	---	---	---	---	---	---
DS-01 (148.33-148.68)*	---	---	---	---	---	---	---
DS-01 (150.23-150.62)*	---	---	---	---	---	---	---
DS-01 (193.20-193.68)*	---	---	---	---	---	---	---

- (pf) - indicates that failure occurred partially along pre-existing foliation surface
(f) - indicates that failure occurred fully along pre-existing foliation surface
(*) - direct shear sample failure data listed in following table

Summary of Residual Direct Shear Strength Test Results

Borehole-Sample (depth)	Apparent Cohesion (kPa)	Internal Friction Angle (°)
GT18-03 (62.88-63.18)	20	21.3
GT18-07 (56.50-56.80)	0	20.6
GT18-01 (148.33-148.68)	69	16.1
GT18-04 (164.50-165.14)	0	24.6
GT18-04 (176.14-176.59)	0	22.0
SPARE Samples (Holdover)		
<i>DS-01 (9.70-9.98)</i>	---	---
<i>DS-01 (13.00-13.66)</i>	---	---
<i>DS-01 (62.60-62.88)</i>	---	---
<i>DS-01 (73.00-73.45)</i>	---	---
<i>DS-01 (78.27-78.77)</i>	---	---
<i>DS-01 (81.81-82.24)</i>	---	---
<i>DS-01 (143.52-144.00)</i>	---	---
<i>DS-01 (143.86-144.34)</i>	---	---
<i>DS-01 (148.33-148.68)</i>	---	---
<i>DS-01 (150.23-150.62)</i>	---	---
<i>DS-01 (193.20-193.68)</i>	---	---



J. F. Archibald, Ph.D., P. Eng., FCIM

Pre-and Post-Test Photos of Brazilian Tensile Strength Samples



18-01 (38.50-38.79))



18-01 (99.42-99.70)



18-02 (19.98-20.23)



Post-test photo image missing

18-02 (100.21-100.50)



18-02 (119.45-119.71)



18-02 (159.69-159.93)



18-03 (40.69-40.93)



18-03 (60.63-60.90)



18-04 (18.01-18.29)





18-04 (122.10-122.36)

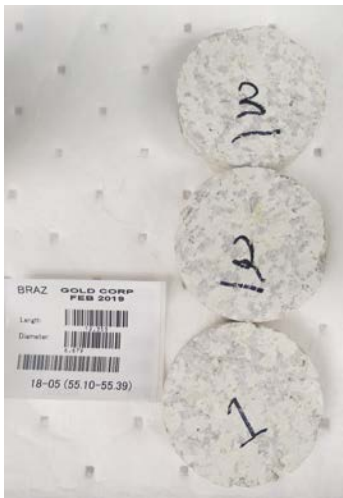


18-04 (158.50-158.75)

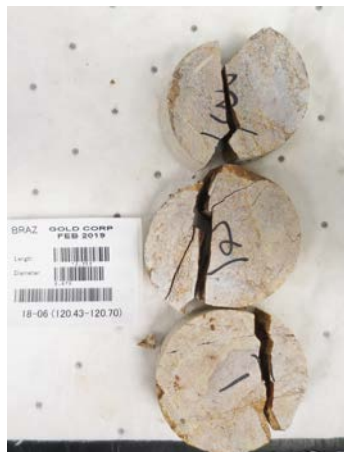
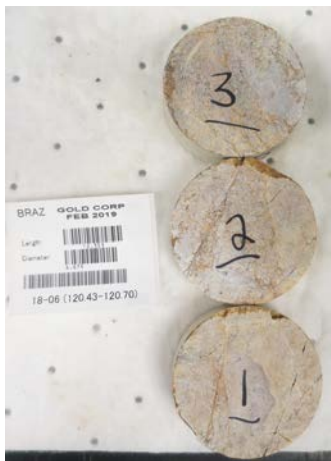


18-04 (194.72-194.99)





18-05 (55.10-55.39)



18-06 (120.43-120.70)



18-06 (160.47-160.78)



18-07 (15.25-15.51)



18-07 (82.00-82.27)



18-07 (120.06-120.38)



JDS Energy & Mining Inc.

Coffee Project

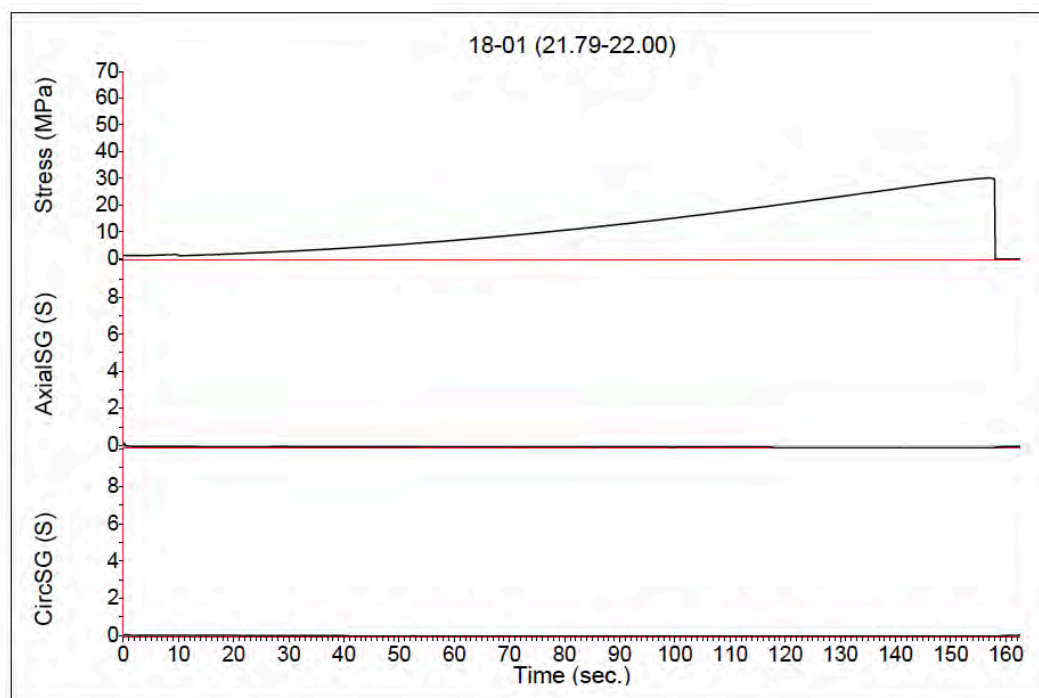
Core Sample Strength Tests

ASTM Data Sheets

Client: JDS Mining & Energy Inc.	Lab: Queen's University
Project/Site: Coffee Gold Mine	Project No.: Unknown
Borehole No.: GT18-01	Sample ID: UCS (no moduli)
Depth: (21.79-22.00 m)	Date Sampled: NA
Lithological Description: Felsic Gneiss	

Specimen Parameters	
Diameter, cm	6.07
Length, cm	14.64
Height-to-Diameter Ratio	2.41/1
Bulk Density, kg/m ³	2633
Moisture Condition	NA

Unconfined Compressive Strength (MPa)	30.2
Young's Modulus, E (GPa)	---
Poisson's ratio	---



Description: 18-01 (21.79-22.00)

Remarks: Failed partially along pre-existing foliation at 30° to core axis (50% failure on foliation surface)

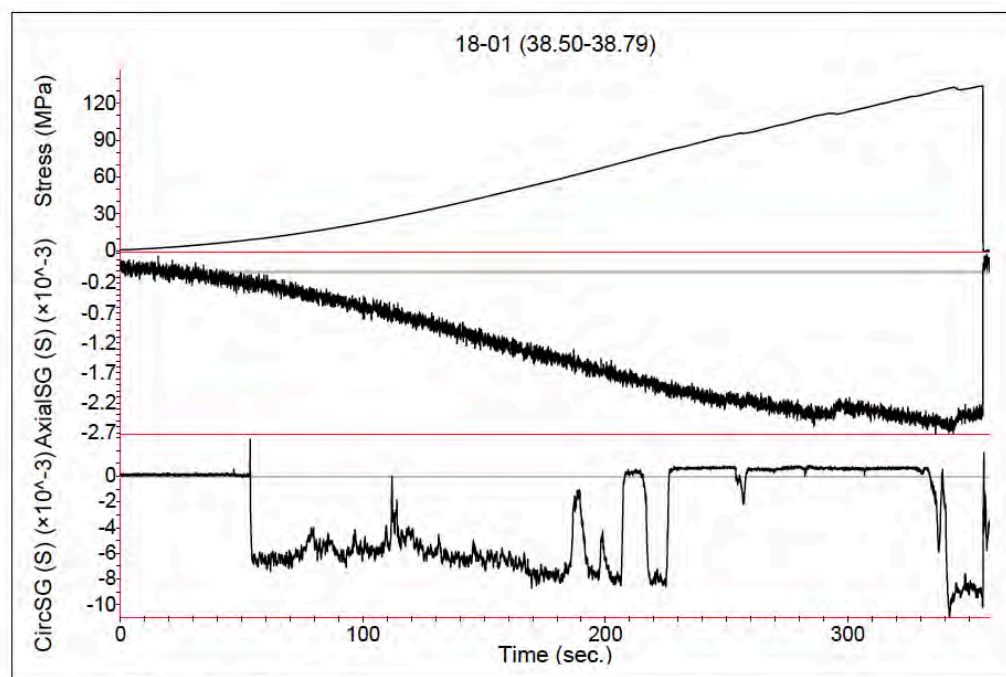
Performed By: P. Ross	Date: February 25, 2019
Verified By: J. F. Archibald	Date: February 25, 2019

Elastic Moduli of Intact Rock Core Specimens in Uniaxial Compression (ASTM D7012 D)

Client: JDS Mining & Energy Inc.	Lab: Queen's University
Project/Site: Coffee Gold Mine	Project No.: Unknown
Borehole No.: GT18-01	Sample ID: UCS
Depth: (38.50-38.79 m)	Date Sampled: NA
Lithological Description: Felsic Gneiss	

Specimen Parameters	
Diameter, cm	6.05
Length, cm	14.96
Height-to-Diameter Ratio	2.47/1
Bulk Density, kg/m ³	2650
Moisture Condition	NA

Unconfined Compressive Strength (MPa)	134.2
Young's Modulus, E (GPa)	23.892
Poisson's ratio	0.16



Description: 18-01 (38.50-38.79)

Remarks: Failed partially along pre-existing foliation at 20" to core axis (50% failure on foliation surface)

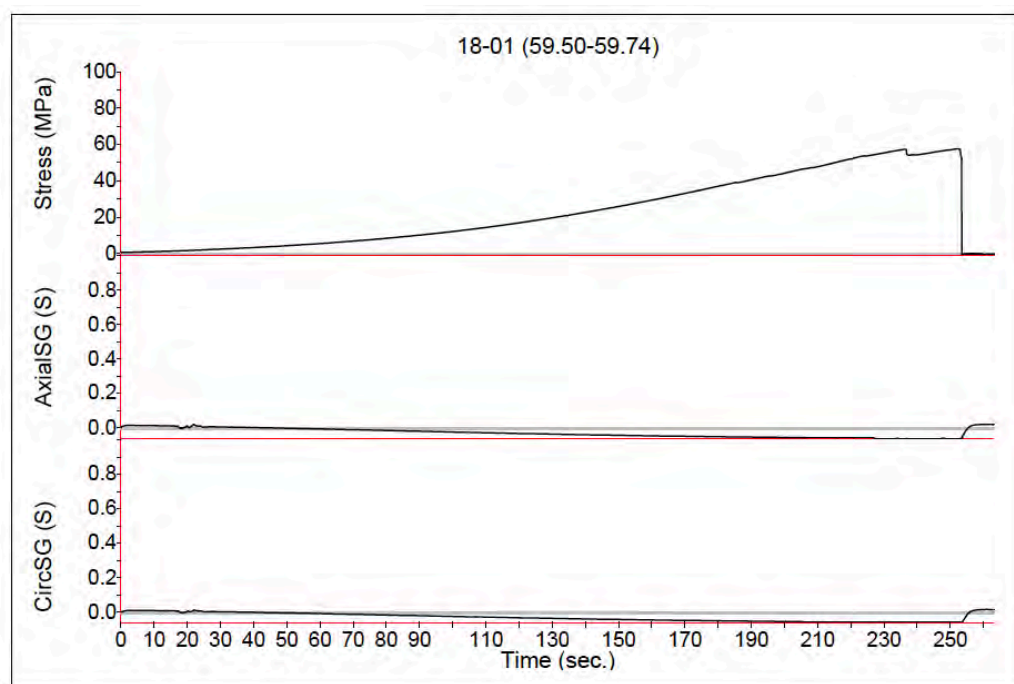
Performed By: P. Ross	Date: February 26, 2019
Verified By: J. F. Archibald	Date: February 26, 2019

Elastic Moduli of Intact Rock Core Specimens in Uniaxial Compression (ASTM D7012 D)

Client: JDS Mining & Energy Inc.	Lab: Queen's University
Project/Site: Coffee Gold Mine	Project No.: Unknown
Borehole No.: GT18-01	Sample ID: UCS (no moduli)
Depth: (59.50-59.74 m)	Date Sampled: NA
Lithological Description: Felsic Gneiss	

Specimen Parameters	
Diameter, cm	6.06
Length, cm	14.71
Height-to-Diameter Ratio	2.43/1
Bulk Density, kg/m ³	2696
Moisture Condition	NA

Unconfined Compressive Strength (MPa)	57.6
Young's Modulus, E (GPa)	---
Poisson's ratio	---



Description: 18-01 (59.50-59.74)

Remarks: Failed partially along pre-existing foliation at 30" to core axis (90% failure on foliation surface)

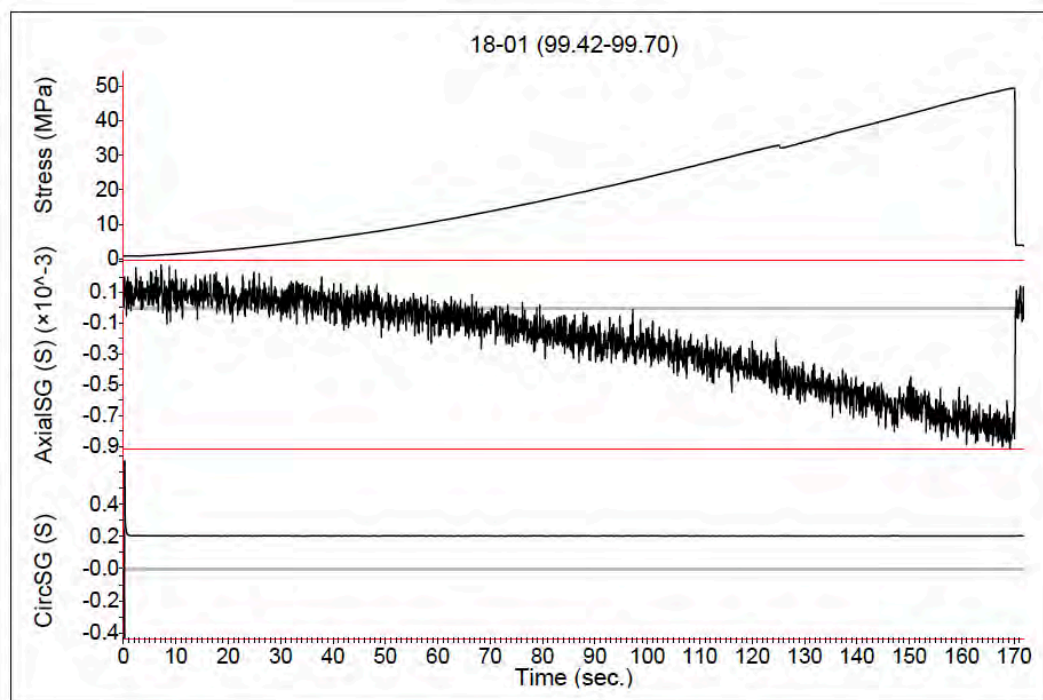
Performed By: P. Ross	Date: February 25, 2019
Verified By: J. F. Archibald	Date: February 25, 2019

Elastic Moduli of Intact Rock Core Specimens in Uniaxial Compression (ASTM D7012 D)

Client: JDS Mining & Energy Inc.	Lab: Queen's University
Project/Site: Coffee Gold Mine	Project No.: Unknown
Borehole No.: GT18-01	Sample ID: UCS
Depth: (99.42-99.70 m)	Date Sampled: NA
Lithological Description: Gneiss	

Specimen Parameters	
Diameter, cm	6.07
Length, cm	14.93
Height-to-Diameter Ratio	2.46/1
Bulk Density, kg/m ³	2670
Moisture Condition	NA

Unconfined Compressive Strength (MPa)	49.4
Young's Modulus, E (GPa)	18.835
Poisson's ratio	---



Description: 18-01 (99.42-99.70)

Remarks: Failed fully along pre-existing foliation at 30" to core axis (100% failure on foliation surface)

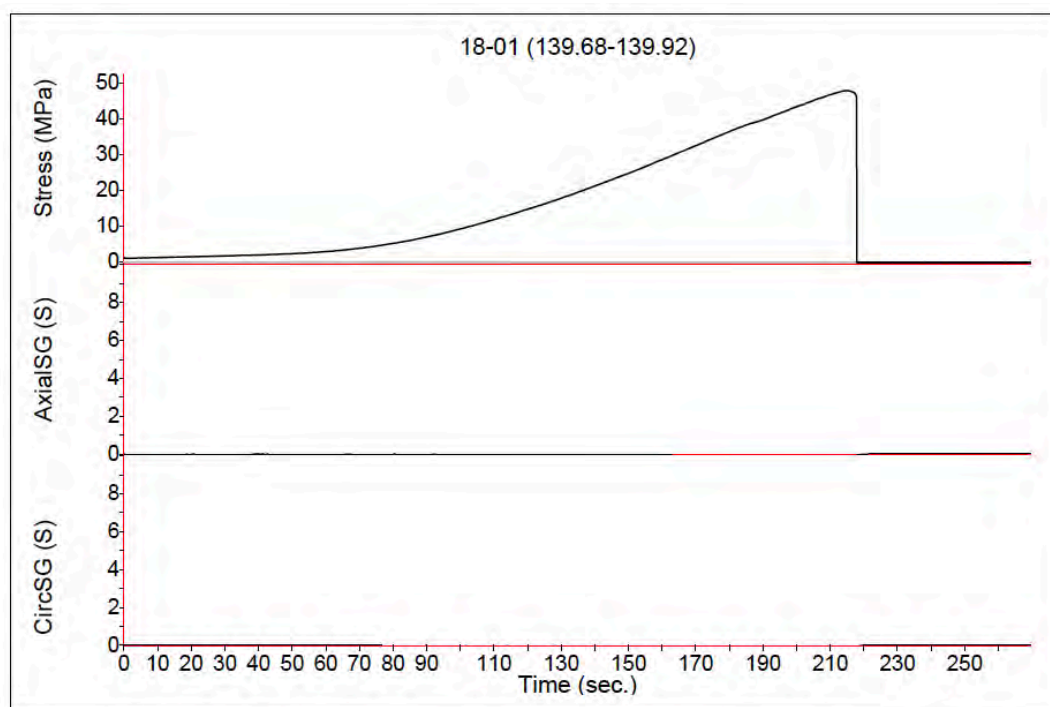
Performed By: P. Ross	Date: February 26, 2019
Verified By: J. F. Archibald	Date: February 26, 2019

Elastic Moduli of Intact Rock Core Specimens in Uniaxial Compression (ASTM D7012 D)

Client: JDS Mining & Energy Inc.	Lab: Queen's University
Project/Site: Coffee Gold Mine	Project No.: Unknown
Borehole No.: GT18-01	Sample ID: UCS (no moduli)
Depth: (139.68-139.92 m)	Date Sampled: NA
Lithological Description: Felsic Gneiss	

Specimen Parameters	
Diameter, cm	6.06
Length, cm	13.61
Height-to-Diameter Ratio	2.25/1
Bulk Density, kg/m ³	2695
Moisture Condition	NA

Unconfined Compressive Strength (MPa)	48.0
Young's Modulus, E (GPa)	---
Poisson's ratio	---



Description: 18-01 (139.68-139.92)

Remarks: Failed partially along pre-existing foliation at 30" to core axis (90% failure on foliation surface)

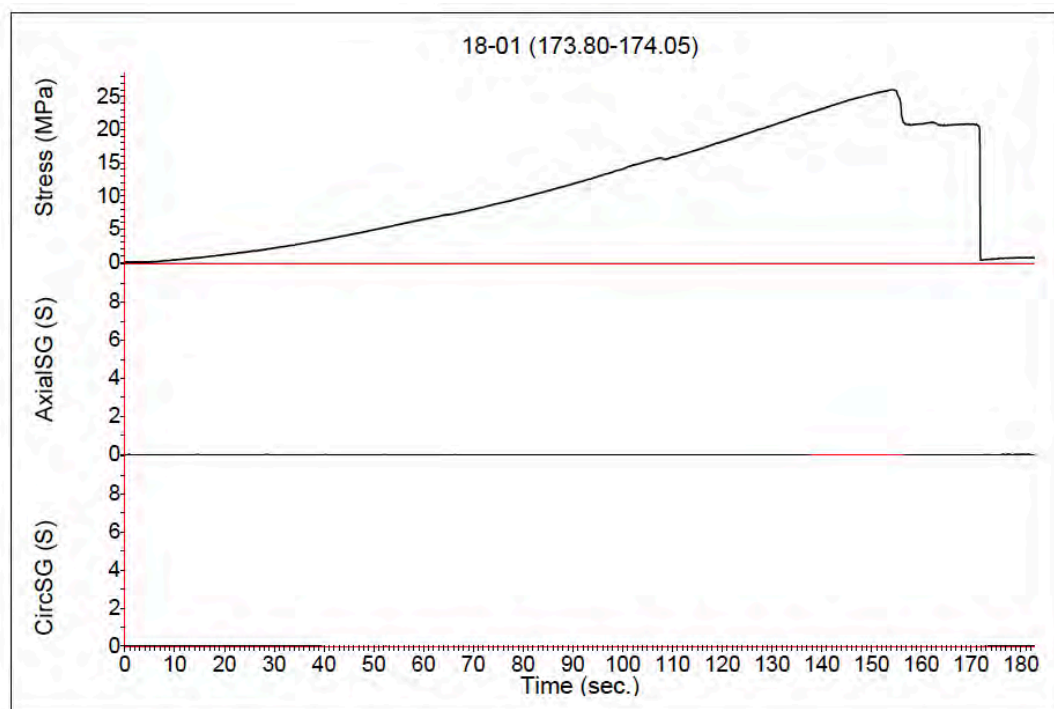
Performed By: P. Ross	Date: February 25, 2019
Verified By: J. F. Archibald	Date: February 25, 2019

Elastic Moduli of Intact Rock Core Specimens in Uniaxial Compression (ASTM D7012 D)

Client: JDS Mining & Energy Inc.	Lab: Queen's University
Project/Site: Coffee Gold Mine	Project No.: Unknown
Borehole No.: GT18-01	Sample ID: UCS (no moduli)
Depth: (173.80-174.05 m)	Date Sampled: NA
Lithological Description: Felsic Gneiss	

Specimen Parameters	
Diameter, cm	6.06
Length, cm	14.89
Height-to-Diameter Ratio	2.46/1
Bulk Density, kg/m ³	2546
Moisture Condition	NA

Unconfined Compressive Strength (MPa)	26.0
Young's Modulus, E (GPa)	---
Poisson's ratio	---



Description: 18-01 (173.80-174.05)

Remarks: Failed fully along pre-existing foliation at 10" to core axis (100% failure on foliation surface)

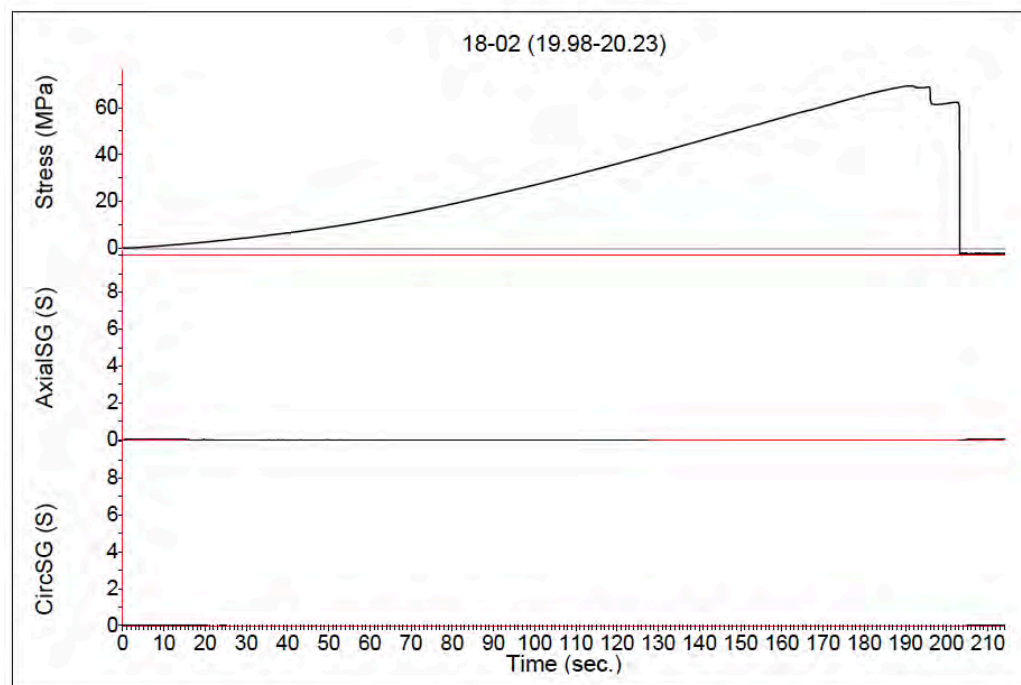
Performed By: P. Ross	Date: February 25, 2019
Verified By: J. F. Archibald	Date: February 25, 2019

Elastic Moduli of Intact Rock Core Specimens in Uniaxial Compression (ASTM D7012 D)

Client: JDS Mining & Energy Inc.	Lab: Queen's University
Project/Site: Coffee Gold Mine	Project No.: Unknown
Borehole No.: GT18-02	Sample ID: UCS (no moduli)
Depth: (19.98-20.23 m)	Date Sampled: NA
Lithological Description: Felsic Gneiss	

Specimen Parameters	
Diameter, cm	6.09
Length, cm	13.41
Height-to-Diameter Ratio	2.20/1
Bulk Density, kg/m ³	2608
Moisture Condition	NA

Unconfined Compressive Strength (MPa)	69.4
Young's Modulus, E (GPa)	---
Poisson's ratio	---



Description: 18-02 (19.98-20.23)

Remarks: Failed partially along pre-existing foliation at 30" to core axis (90% failure on foliation surface)

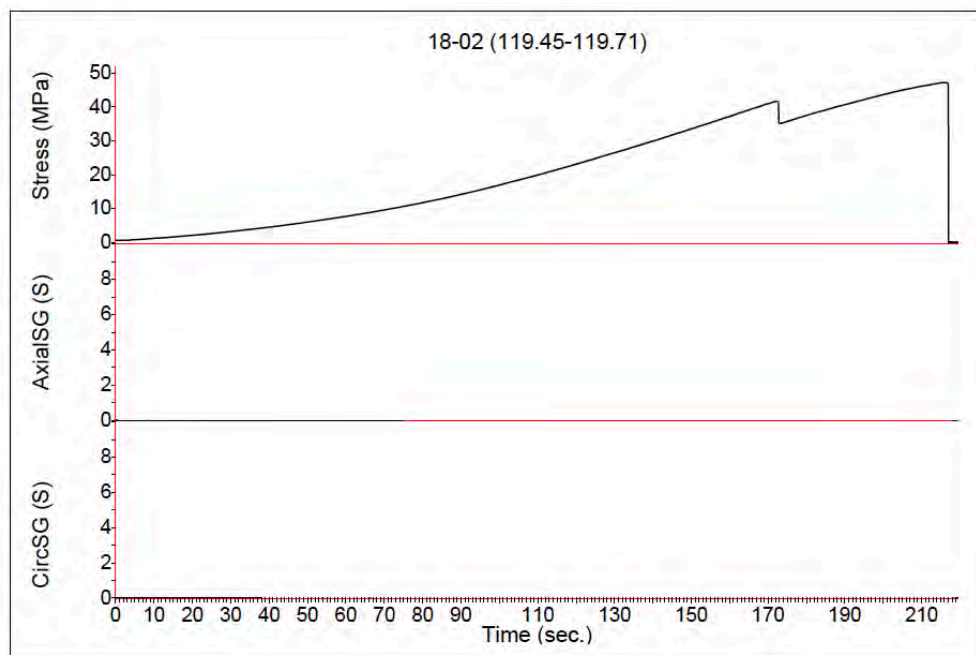
Performed By: P. Ross	Date: February 25, 2019
Verified By: J. F. Archibald	Date: February 25, 2019

Elastic Moduli of Intact Rock Core Specimens in Uniaxial Compression (ASTM D7012 D)

Client: JDS Mining & Energy Inc.	Lab: Queen's University
Project/Site: Coffee Gold Mine	Project No.: Unknown
Borehole No.: GT18-02	Sample ID: UCS (no moduli)
Depth: (119.45-119.71 m)	Date Sampled: NA
Lithological Description: Felsic Gneiss	

Specimen Parameters	
Diameter, cm	6.10
Length, cm	14.99
Height-to-Diameter Ratio	2.46/1
Bulk Density, kg/m ³	2922
Moisture Condition	NA

Unconfined Compressive Strength (MPa)	47.2
Young's Modulus, E (GPa)	---
Poisson's ratio	---



Description: 18-02 (119.45-119.71)

Remarks: Failed fully along pre-existing foliation at 30" to core axis (100% failure on foliation surface)

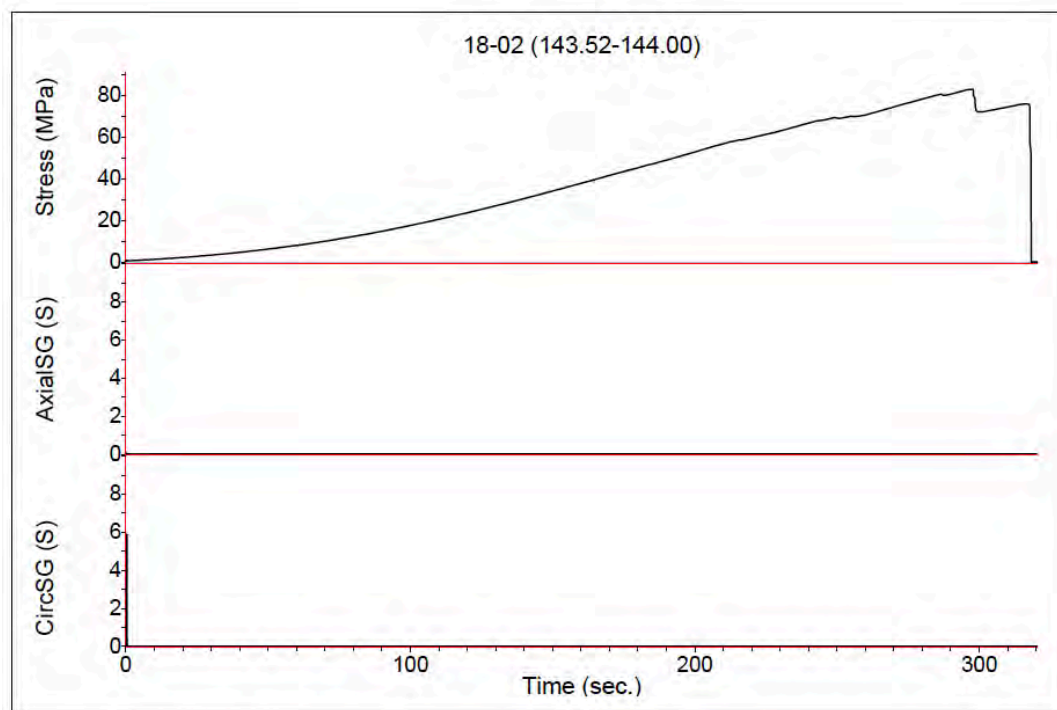
Performed By: P. Ross	Date: February 25, 2019
Verified By: J. F. Archibald	Date: February 25, 2019

Elastic Moduli of Intact Rock Core Specimens in Uniaxial Compression (ASTM D7012 D)

Client: JDS Mining & Energy Inc.	Lab: Queen's University
Project/Site: Coffee Gold Mine	Project No.: Unknown
Borehole No.: GT18-02	Sample ID: UCS (no moduli)
Depth: (143.52-144.00 m)	Date Sampled: NA
Lithological Description: Biotite-feldspar schist with carbonate	

Specimen Parameters	
Diameter, cm	6.10
Length, cm	14.93
Height-to-Diameter Ratio	2.45/1
Bulk Density, kg/m ³	2610
Moisture Condition	NA

Unconfined Compressive Strength (MPa)	83.2
Young's Modulus, E (GPa)	---
Poisson's ratio	---



Description: 18-02 (143.52-144.00)

Remarks: Failed partially along pre-existing foliation at 20" to core axis (10% failure on foliation surface)

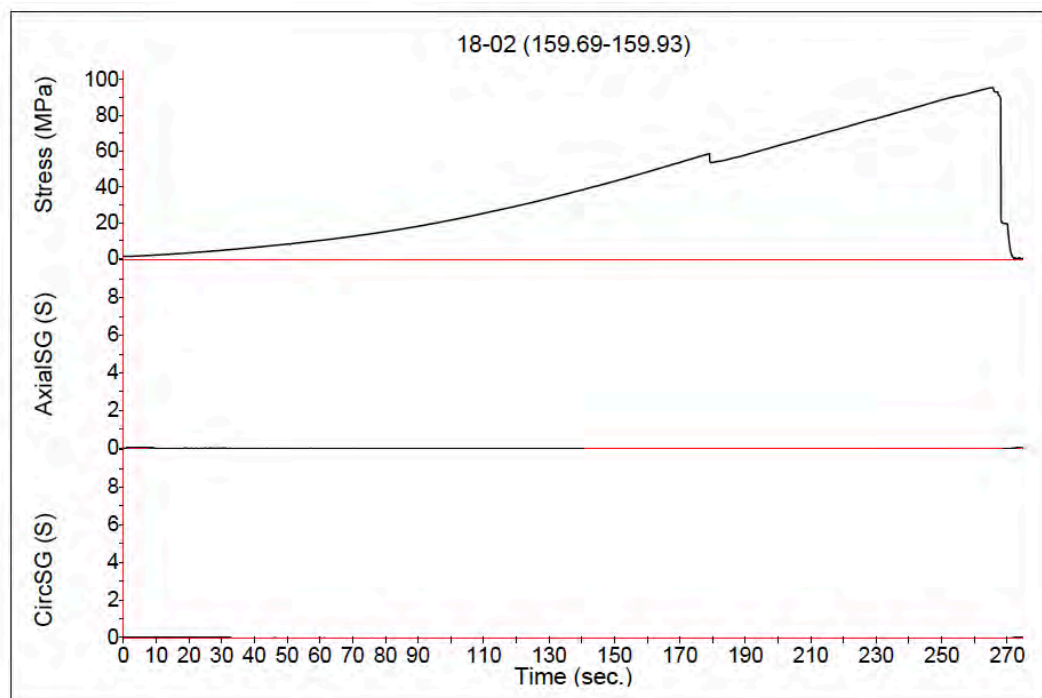
Performed By: P. Ross	Date: February 25, 2019
Verified By: J. F. Archibald	Date: February 25, 2019

Elastic Moduli of Intact Rock Core Specimens in Uniaxial Compression (ASTM D7012 D)

Client: JDS Mining & Energy Inc.	Lab: Queen's University
Project/Site: Coffee Gold Mine	Project No.: Unknown
Borehole No.: GT18-02	Sample ID: UCS (no moduli)
Depth: (159.69-159.93 m)	Date Sampled: NA
Lithological Description: Felsic Gneiss	

Specimen Parameters	
Diameter, cm	6.08
Length, cm	13.58
Height-to-Diameter Ratio	2.23/1
Bulk Density, kg/m ³	2633
Moisture Condition	NA

Unconfined Compressive Strength (MPa)	95.4
Young's Modulus, E (GPa)	---
Poisson's ratio	---



Description: 18-02 (159.69-159.93)

Remarks: Failed partially along pre-existing foliation at 15" to core axis (90% failure on foliation surface)

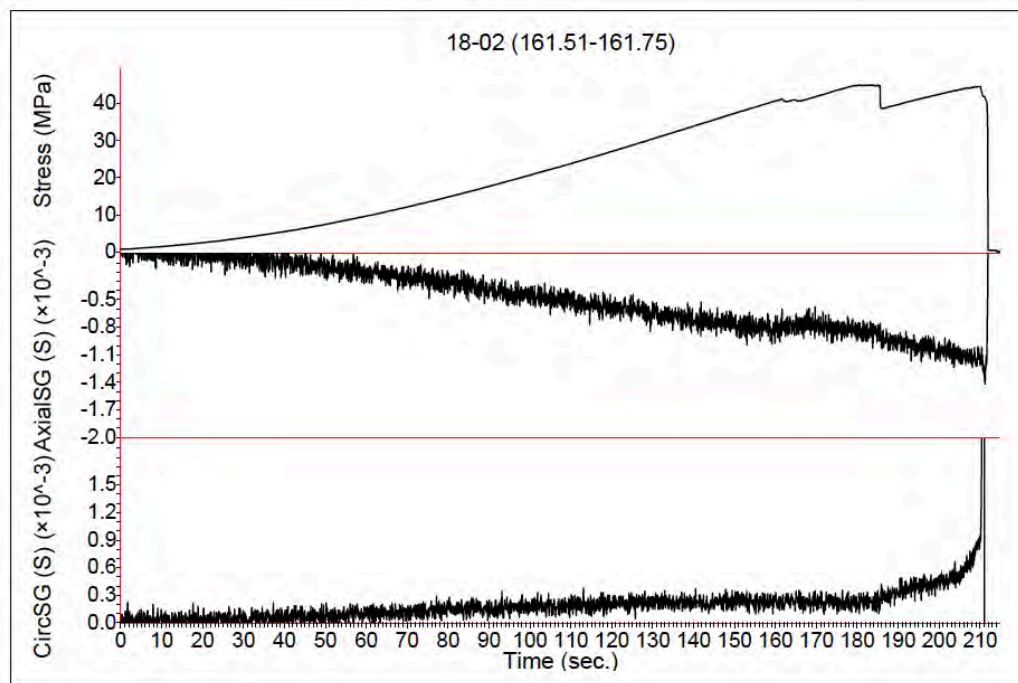
Performed By: P. Ross	Date: February 25, 2019
Verified By: J. F. Archibald	Date: February 25, 2019

Elastic Moduli of Intact Rock Core Specimens in Uniaxial Compression (ASTM D7012 D)

Client: JDS Mining & Energy Inc.	Lab: Queen's University
Project/Site: Coffee Gold Mine	Project No.: Unknown
Borehole No.: GT18-02	Sample ID: UCS
Depth: (161.51-161.75m)	Date Sampled: NA
Lithological Description: Felsic Gneiss	

Specimen Parameters	
Diameter, cm	6.08
Length, cm	14.69
Height-to-Diameter Ratio	2.42/1
Bulk Density, kg/m ³	2560
Moisture Condition	NA

Unconfined Compressive Strength (MPa)	45.0
Young's Modulus, E (GPa)	19.111
Poisson's ratio	0.11



Description: 18-02 (161.51-161.75)

Remarks: Failed partially along pre-existing foliation at 30" to core axis (75% failure on foliation surface)

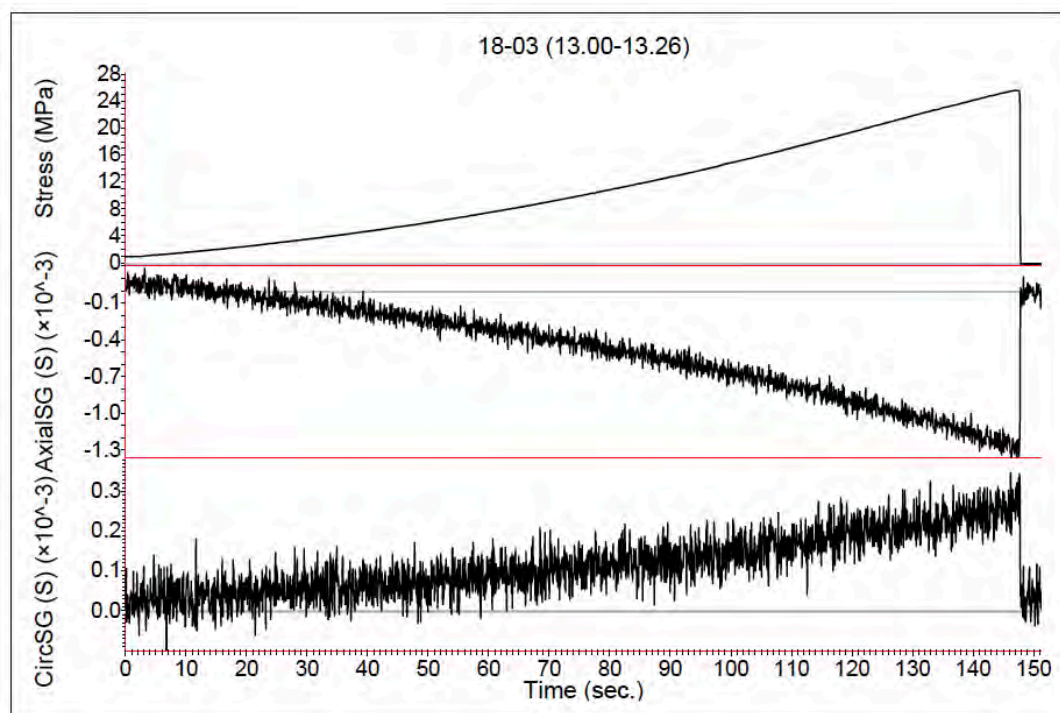
Performed By: P. Ross	Date: February 26, 2019
Verified By: J. F. Archibald	Date: February 26, 2019

Elastic Moduli of Intact Rock Core Specimens in Uniaxial Compression (ASTM D7012 D)

Client: JDS Mining & Energy Inc.	Lab: Queen's University
Project/Site: Coffee Gold Mine	Project No.: Unknown
Borehole No.: GT18-03	Sample ID: UCS
Depth: (13.00-13.26 m)	Date Sampled: NA
Lithological Description: Biotite-feldspar schist with carbonate	

Specimen Parameters	
Diameter, cm	6.06
Length, cm	14.94
Height-to-Diameter Ratio	2.47/1
Bulk Density, kg/m ³	2700
Moisture Condition	NA

Unconfined Compressive Strength (MPa)	25.6
Young's Modulus, E (GPa)	12.342
Poisson's ratio	0.11



Description: 18-03 (13.00-13.26)

Remarks: Failed partially along pre-existing foliation at 30" to core axis (30% failure on foliation surface)

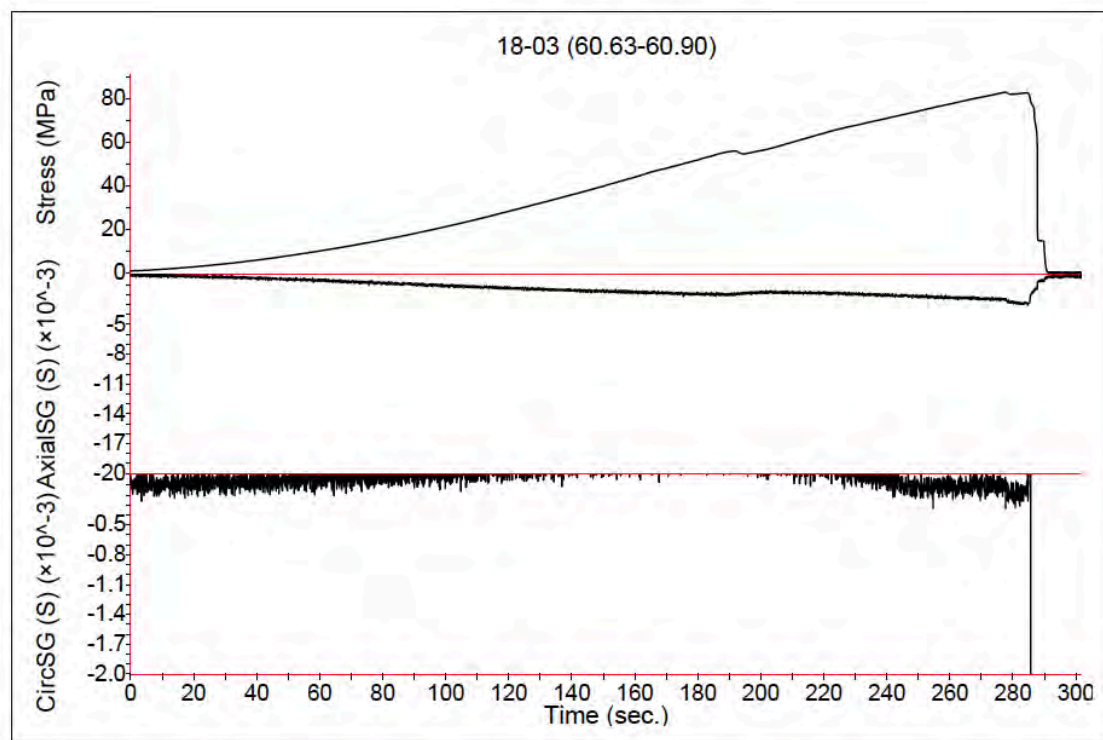
Performed By: P. Ross	Date: February 26, 2019
Verified By: J. F. Archibald	Date: February 26, 2019

Elastic Moduli of Intact Rock Core Specimens in Uniaxial Compression (ASTM D7012 D)

Client: JDS Mining & Energy Inc.	Lab: Queen's University
Project/Site: Coffee Gold Mine	Project No.: Unknown
Borehole No.: GT18-03	Sample ID: UCS
Depth: (60.63-60.90 m)	Date Sampled: NA
Lithological Description: Biotite-feldspar schist with carbonate	

Specimen Parameters	
Diameter, cm	6.06
Length, cm	13.08
Height-to-Diameter Ratio	2.16/1
Bulk Density, kg/m ³	2700
Moisture Condition	NA

Unconfined Compressive Strength (MPa)	82.9
Young's Modulus, E (GPa)	17.216
Poisson's ratio	0.09



Description: 18-03 (60.63-60.90)

Remarks: Failed partially along pre-existing foliation at 50" to core axis (75% failure on foliation surface)

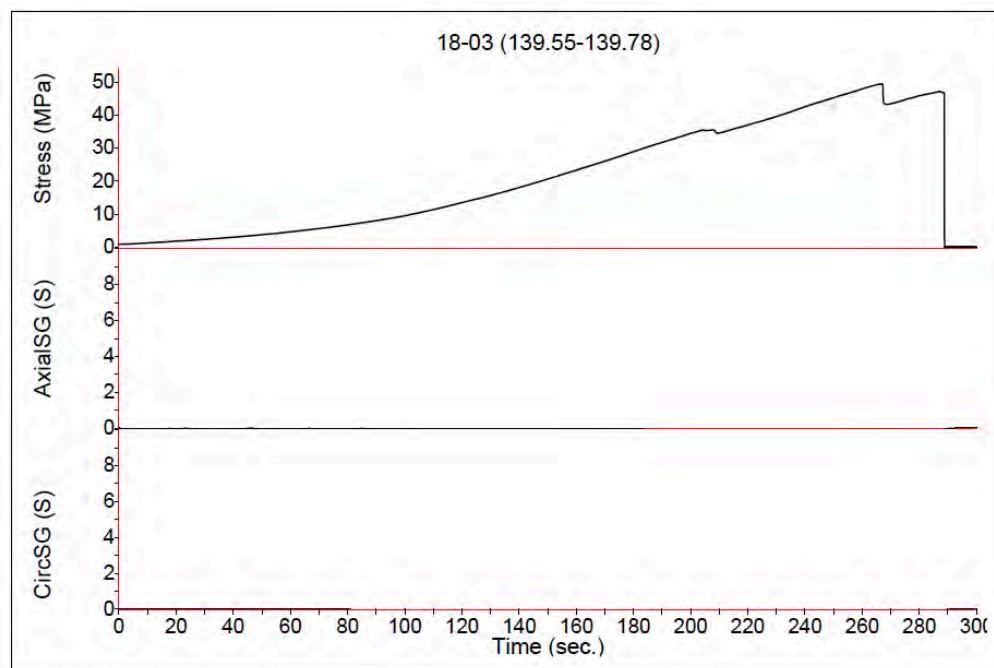
Performed By: P. Ross	Date: February 26, 2019
Verified By: J. F. Archibald	Date: February 26, 2019

Elastic Moduli of Intact Rock Core Specimens in Uniaxial Compression (ASTM D7012 D)

Client: JDS Mining & Energy Inc.	Lab: Queen's University
Project/Site: Coffee Gold Mine	Project No.: Unknown
Borehole No.: GT18-03	Sample ID: UCS (no moduli)
Depth: (139.55-139.78 m)	Date Sampled: NA
Lithological Description: Biotite-feldspar schist with carbonate	

Specimen Parameters	
Diameter, cm	6.07
Length, cm	14.84
Height-to-Diameter Ratio	2.44/1
Bulk Density, kg/m ³	2700
Moisture Condition	NA

Unconfined Compressive Strength (MPa)	49.4
Young's Modulus, E (GPa)	---
Poisson's ratio	---



Description: 18-03 (139.55-139.78)

Remarks: Failed partially along pre-existing foliation at 20° to core axis (75% failure on foliation surface)

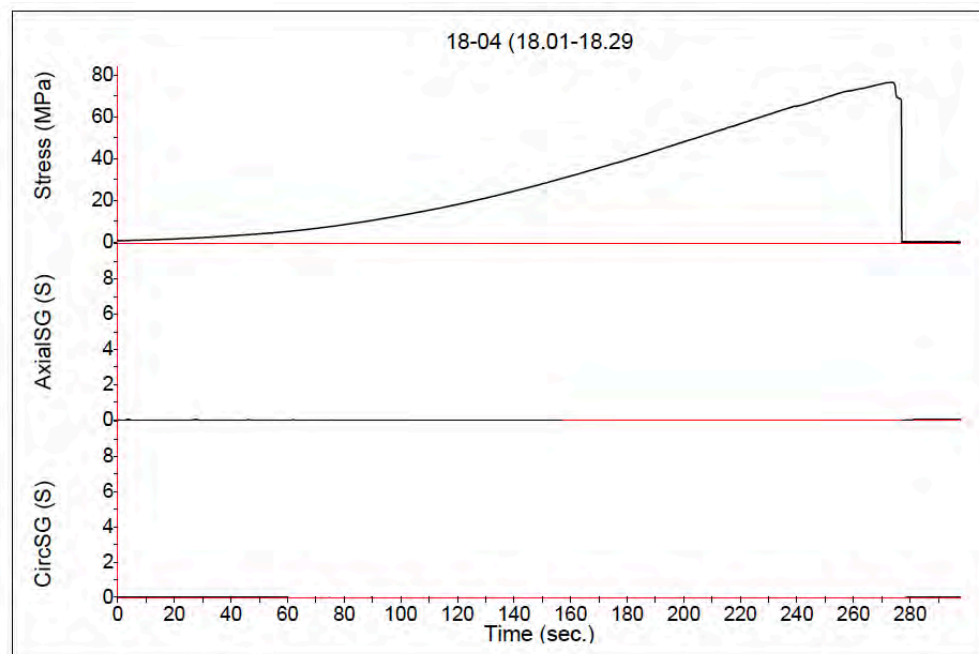
Performed By: P. Ross	Date: February 25, 2019
Verified By: J. F. Archibald	Date: February 25, 2019

Elastic Moduli of Intact Rock Core Specimens in Uniaxial Compression (ASTM D7012 D)

Client: JDS Mining & Energy Inc.	Lab: Queen's University
Project/Site: Coffee Gold Mine	Project No.: Unknown
Borehole No.: GT18-04	Sample ID: UCS (no moduli)
Depth: (18.01-18.29 m)	Date Sampled: NA
Lithological Description: Felsic Gneiss	

Specimen Parameters	
Diameter, cm	6.08
Length, cm	14.85
Height-to-Diameter Ratio	2.44/1
Bulk Density, kg/m ³	2620
Moisture Condition	NA

Unconfined Compressive Strength (MPa)	76.5
Young's Modulus, E (GPa)	---
Poisson's ratio	---



Description: 18-04 (18.01-18.29)

Remarks: Failure completely through intact rock at 20" to core axis

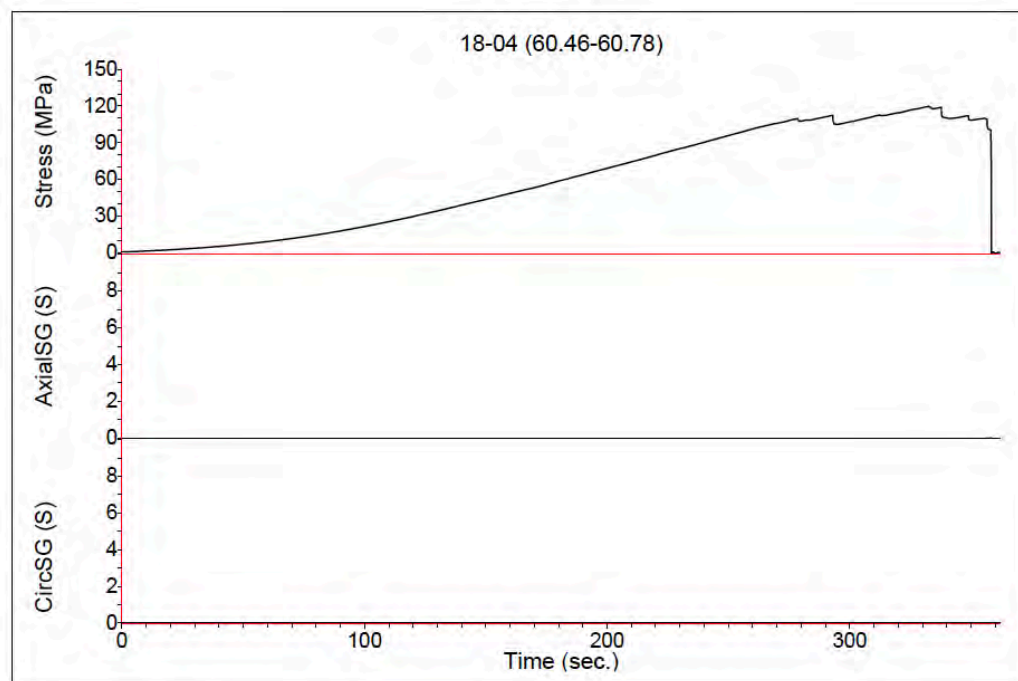
Performed By: P. Ross	Date: February 25, 2019
Verified By: J. F. Archibald	Date: February 25, 2019

Elastic Moduli of Intact Rock Core Specimens in Uniaxial Compression (ASTM D7012 D)

Client: JDS Mining & Energy Inc.	Lab: Queen's University
Project/Site: Coffee Gold Mine	Project No.: Unknown
Borehole No.: GT18-04	Sample ID: UCS
Depth: (60.46-60.78 m)	Date Sampled: NA
Lithological Description: Felsic Gneiss	

Specimen Parameters	
Diameter, cm	6.09
Length, cm	14.87
Height-to-Diameter Ratio	2.44/1
Bulk Density, kg/m ³	2620
Moisture Condition	NA

Unconfined Compressive Strength (MPa)	119.7
Young's Modulus, E (GPa)	28.037
Poisson's ratio	0.18



Description: 18-04 (60.46-60.78)

Remarks: Failure completely through intact rock parallel to core axis

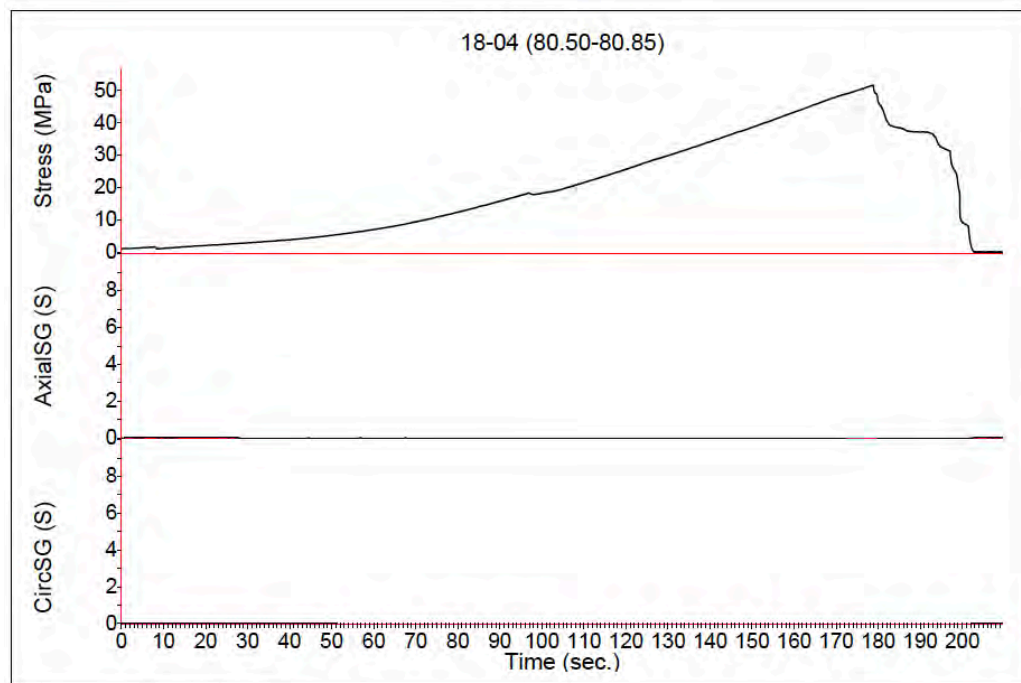
Performed By: P. Ross	Date: February 26, 2019
Verified By: J. F. Archibald	Date: February 26, 2019

Elastic Moduli of Intact Rock Core Specimens in Uniaxial Compression (ASTM D7012 D)

Client: JDS Mining & Energy Inc.	Lab: Queen's University
Project/Site: Coffee Gold Mine	Project No.: Unknown
Borehole No.: GT18-04	Sample ID: UCS (no moduli)
Depth: (80.50-80.85 m)	Date Sampled: NA
Lithological Description: Felsic Gneiss	

Specimen Parameters	
Diameter, cm	6.03
Length, cm	12.56
Height-to-Diameter Ratio	2.08/1
Bulk Density, kg/m ³	2880
Moisture Condition	NA

Unconfined Compressive Strength (MPa)	51.7
Young's Modulus, E (GPa)	---
Poisson's ratio	---



Description: 18-04 (80.50-80.85)

Remarks: Failed partially along pre-existing foliation at 50° to core axis (50% failure on foliation surface)

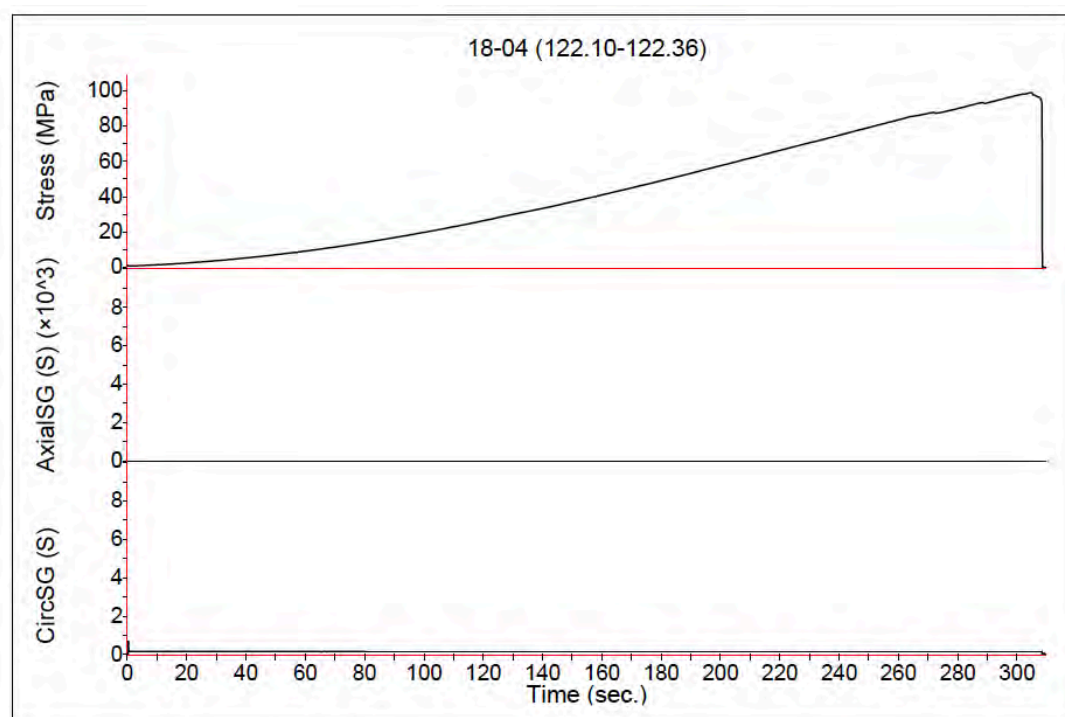
Performed By: P. Ross	Date: February 25, 2019
Verified By: J. F. Archibald	Date: February 25, 2019

Elastic Moduli of Intact Rock Core Specimens in Uniaxial Compression (ASTM D7012 D)

Client: JDS Mining & Energy Inc.	Lab: Queen's University
Project/Site: Coffee Gold Mine	Project No.: Unknown
Borehole No.: GT18-04	Sample ID: UCS
Depth: (122.10-122.36 m)	Date Sampled: NA
Lithological Description: Felsic Gneiss	

Specimen Parameters	
Diameter, cm	6.06
Length, cm	14.85
Height-to-Diameter Ratio	2.45/1
Bulk Density, kg/m ³	2610
Moisture Condition	NA

Unconfined Compressive Strength (MPa)	99.0
Young's Modulus, E (GPa)	19.578
Poisson's ratio	0.28



Description: 18-04 (122.10-122.36)

Remarks: Failure completely through intact rock parallel to core axis

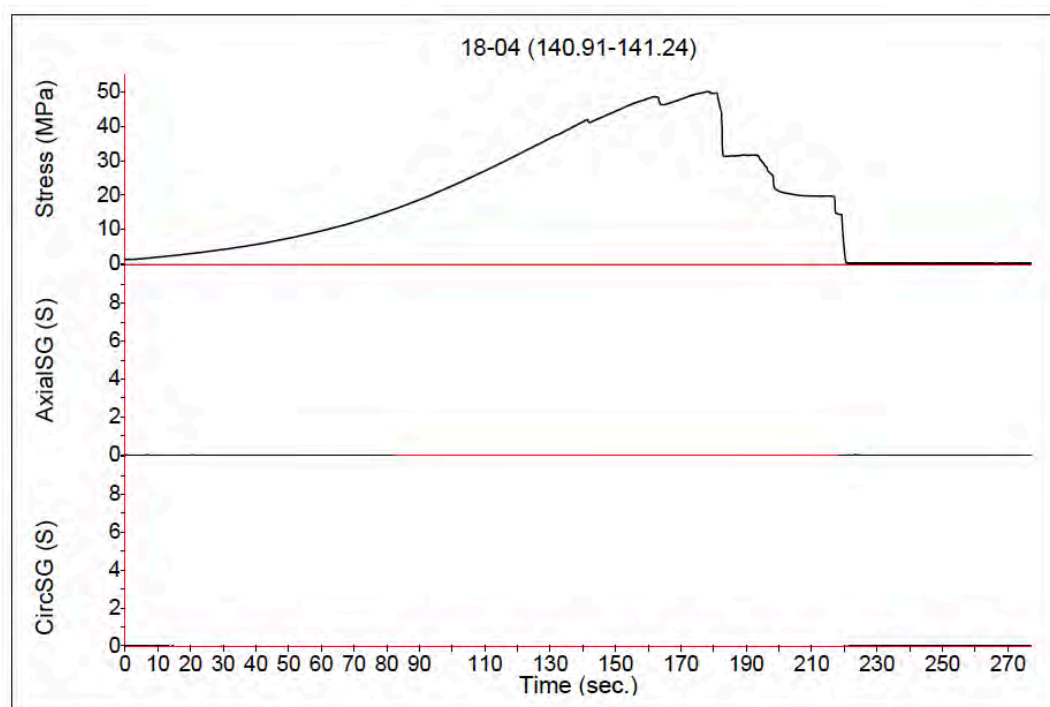
Performed By: P. Ross	Date: February 26, 2019
Verified By: J. F. Archibald	Date: February 26, 2019

Elastic Moduli of Intact Rock Core Specimens in Uniaxial Compression (ASTM D7012 D)

Client: JDS Mining & Energy Inc.	Lab: Queen's University
Project/Site: Coffee Gold Mine	Project No.: Unknown
Borehole No.: GT18-04	Sample ID: UCS (no moduli)
Depth: (140.91-141.24 m)	Date Sampled: NA
Lithological Description: Felsic Gneiss	

Specimen Parameters	
Diameter, cm	6.06
Length, cm	11.88
Height-to-Diameter Ratio	1.96/1
Bulk Density, kg/m ³	2600
Moisture Condition	NA

Unconfined Compressive Strength (MPa)	50.3
Young's Modulus, E (GPa)	---
Poisson's ratio	---



Description: 18-04 (140.91-141.24)

Remarks: Failed partially along pre-existing foliation at 35" to core axis (85% failure on foliation surface)

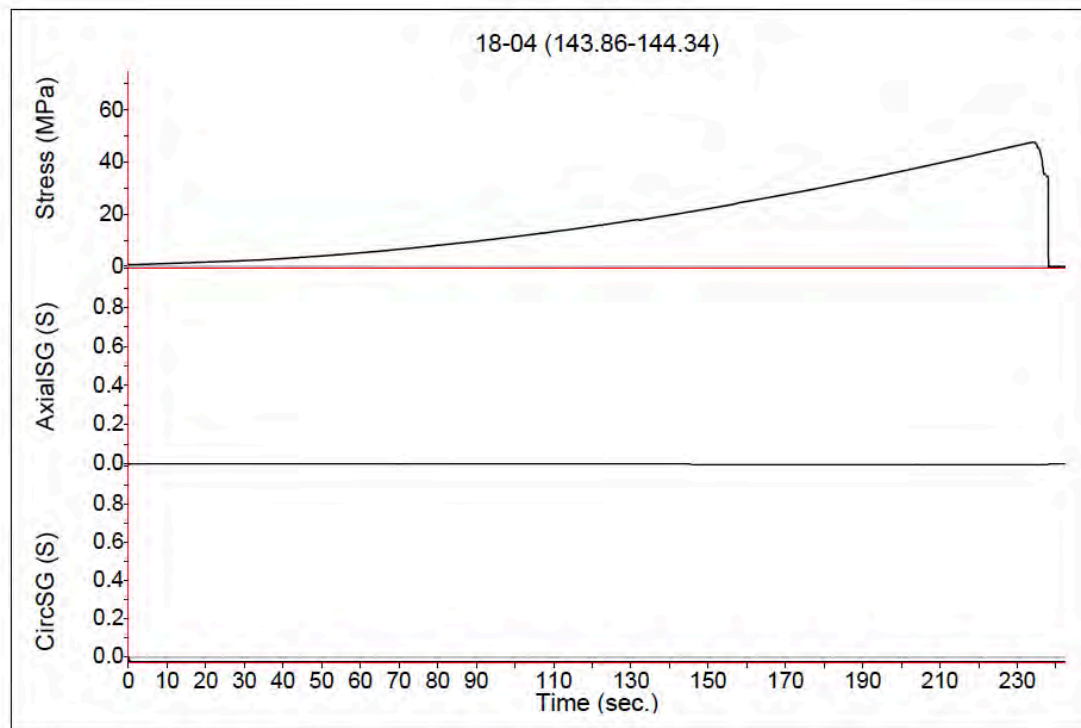
Performed By: P. Ross	Date: February 25, 2019
Verified By: J. F. Archibald	Date: February 25, 2019

Elastic Moduli of Intact Rock Core Specimens in Uniaxial Compression (ASTM D7012 D)

Client: JDS Mining & Energy Inc.	Lab: Queen's University
Project/Site: Coffee Gold Mine	Project No.: Unknown
Borehole No.: GT18-04	Sample ID: UCS (no moduli)
Depth: (143.86-144.34 m)	Date Sampled: NA
Lithological Description: Biotite-feldspar schist with carbonate	

Specimen Parameters	
Diameter, cm	6.08
Length, cm	15.00
Height-to-Diameter Ratio	2.47/1
Bulk Density, kg/m ³	2620
Moisture Condition	NA

Unconfined Compressive Strength (MPa)	47.7
Young's Modulus, E (GPa)	---
Poisson's ratio	---



Description: 18-04 (143.86-144.34)

Remarks: Failed partially along pre-existing foliation at 40° to core axis (60% failure on foliation surface)

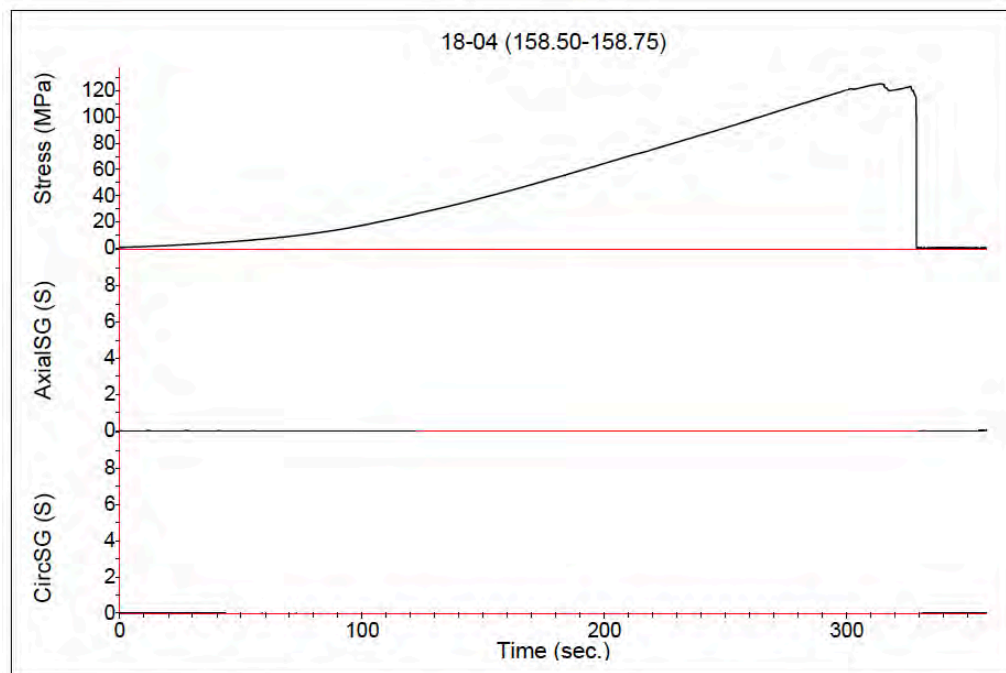
Performed By: P. Ross	Date: February 25, 2019
Verified By: J. F. Archibald	Date: February 25, 2019

Elastic Moduli of Intact Rock Core Specimens in Uniaxial Compression (ASTM D7012 D)

Client: JDS Mining & Energy Inc.	Lab: Queen's University
Project/Site: Coffee Gold Mine	Project No.: Unknown
Borehole No.: GT18-04	Sample ID: UCS (no moduli)
Depth: (158.50-158.75 m)	Date Sampled: NA
Lithological Description: Felsic Gneiss	

Specimen Parameters	
Diameter, cm	6.06
Length, cm	14.98
Height-to-Diameter Ratio	2.47/1
Bulk Density, kg/m ³	2660
Moisture Condition	NA

Unconfined Compressive Strength (MPa)	125.5
Young's Modulus, E (GPa)	---
Poisson's ratio	---



Description: 18-04 (158.50-158.75)

Remarks: Failed partially along pre-existing foliation at 40° to core axis (25% failure on foliation surface)

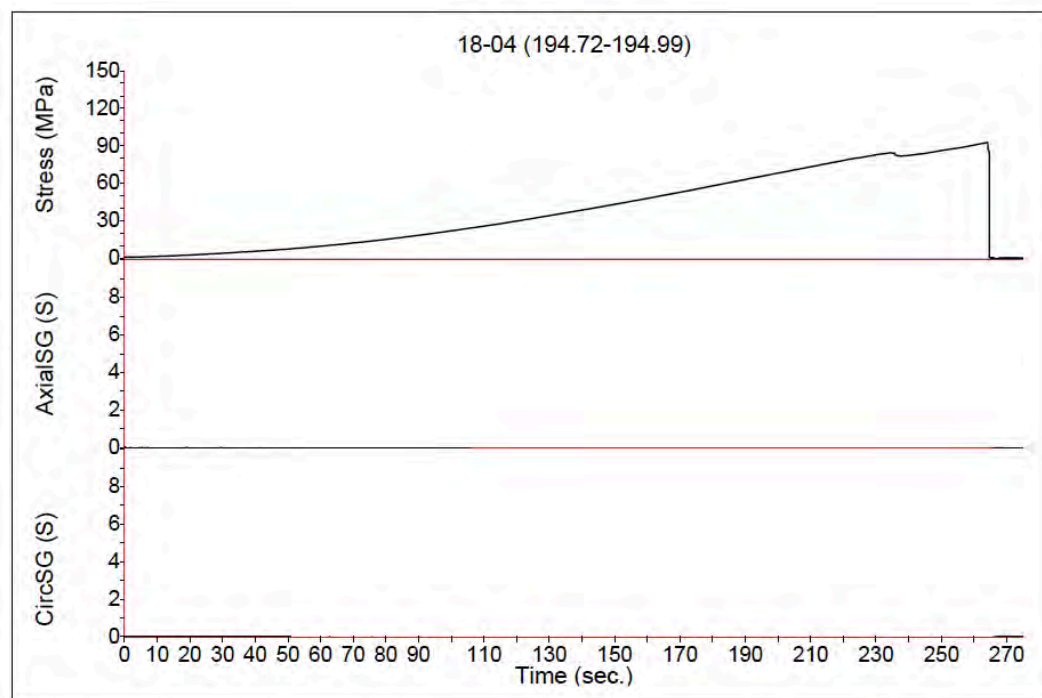
Performed By: P. Ross	Date: February 25, 2019
Verified By: J. F. Archibald	Date: February 25, 2019

Elastic Moduli of Intact Rock Core Specimens in Uniaxial Compression (ASTM D7012 D)

Client: JDS Mining & Energy Inc.	Lab: Queen's University
Project/Site: Coffee Gold Mine	Project No.: Unknown
Borehole No.: GT18-04	Sample ID: UCS (no moduli)
Depth: (194.72-194.99 m)	Date Sampled: NA
Lithological Description: Felsic Gneiss	

Specimen Parameters	
Diameter, cm	6.06
Length, cm	15.14
Height-to-Diameter Ratio	2.49/1
Bulk Density, kg/m ³	2610
Moisture Condition	NA

Unconfined Compressive Strength (MPa)	92.6
Young's Modulus, E (GPa)	---
Poisson's ratio	---



Description: 18-04 (194.72-194.99)

Remarks: Failed partially along pre-existing foliation at 30° to core axis (20% failure on foliation surface)

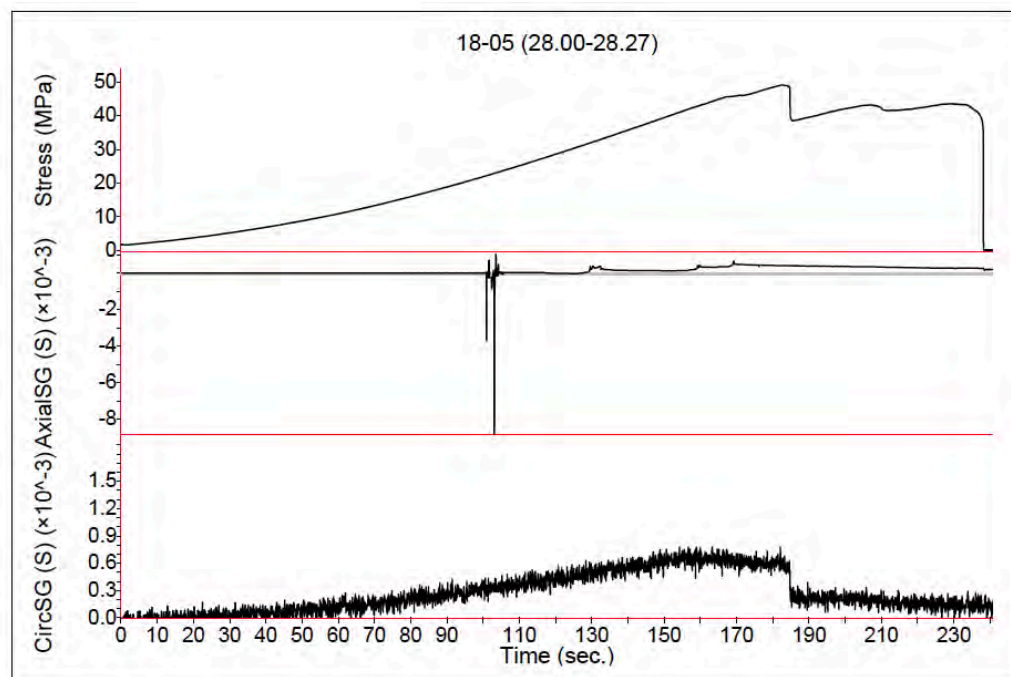
Performed By: P. Ross	Date: February 25, 2019
Verified By: J. F. Archibald	Date: February 25, 2019

Elastic Moduli of Intact Rock Core Specimens in Uniaxial Compression (ASTM D7012 D)

Client: JDS Mining & Energy Inc.	Lab: Queen's University
Project/Site: Coffee Gold Mine	Project No.: Unknown
Borehole No.: GT18-05	Sample ID: UCS
Depth: (28.00-28.27 m)	Date Sampled: NA
Lithological Description: Granite	

Specimen Parameters	
Diameter, cm	6.08
Length, cm	13.46
Height-to-Diameter Ratio	2.21/1
Bulk Density, kg/m ³	2570
Moisture Condition	NA

Unconfined Compressive Strength (MPa)	49.1
Young's Modulus, E (GPa)	17.394
Poisson's ratio	0.20



Description: 18-05 (28.00-28.27)

Remarks: Failed partially along pre-existing foliation at 20" to core axis (60% failure on foliation surface)

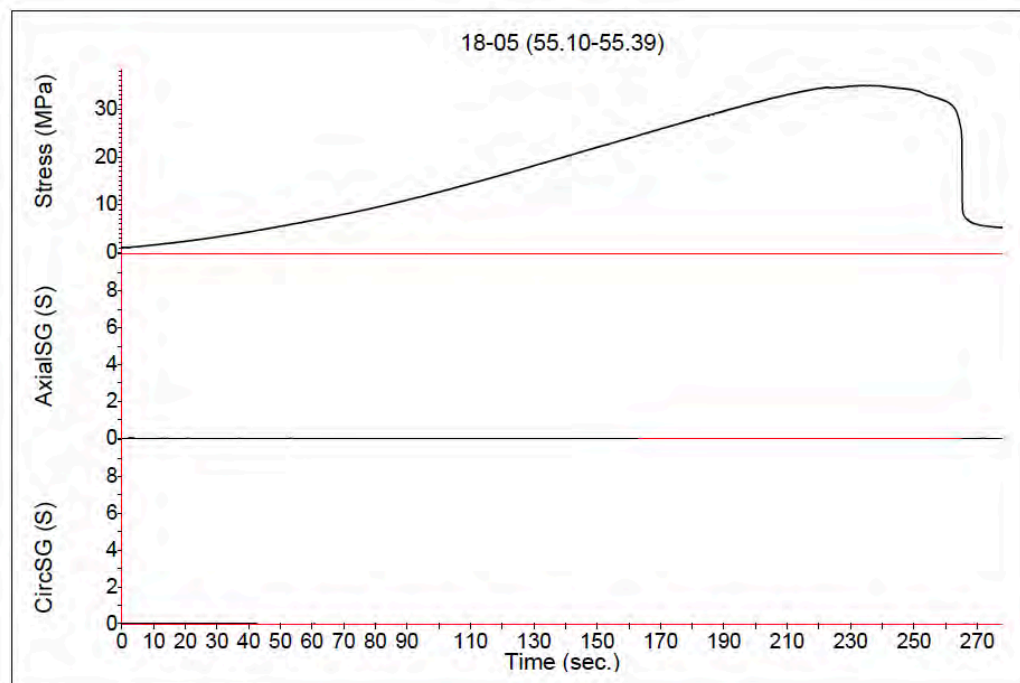
Performed By: P. Ross	Date: February 26, 2019
Verified By: J. F. Archibald	Date: February 26, 2019

Elastic Moduli of Intact Rock Core Specimens in Uniaxial Compression (ASTM D7012 D)

Client: JDS Mining & Energy Inc.	Lab: Queen's University
Project/Site: Coffee Gold Mine	Project No.: Unknown
Borehole No.: GT18-05	Sample ID: UCS (no moduli)
Depth: (55.10-55.39 m)	Date Sampled: NA
Lithological Description: Granite	

Specimen Parameters	
Diameter, cm	6.09
Length, cm	15.00
Height-to-Diameter Ratio	2.46/1
Bulk Density, kg/m ³	2430
Moisture Condition	NA

Unconfined Compressive Strength (MPa)	35.0
Young's Modulus, E (GPa)	---
Poisson's ratio	---



Description: 18-05 (55.10-55.39)

Remarks: Failure completely through intact rock at 10" to core axis

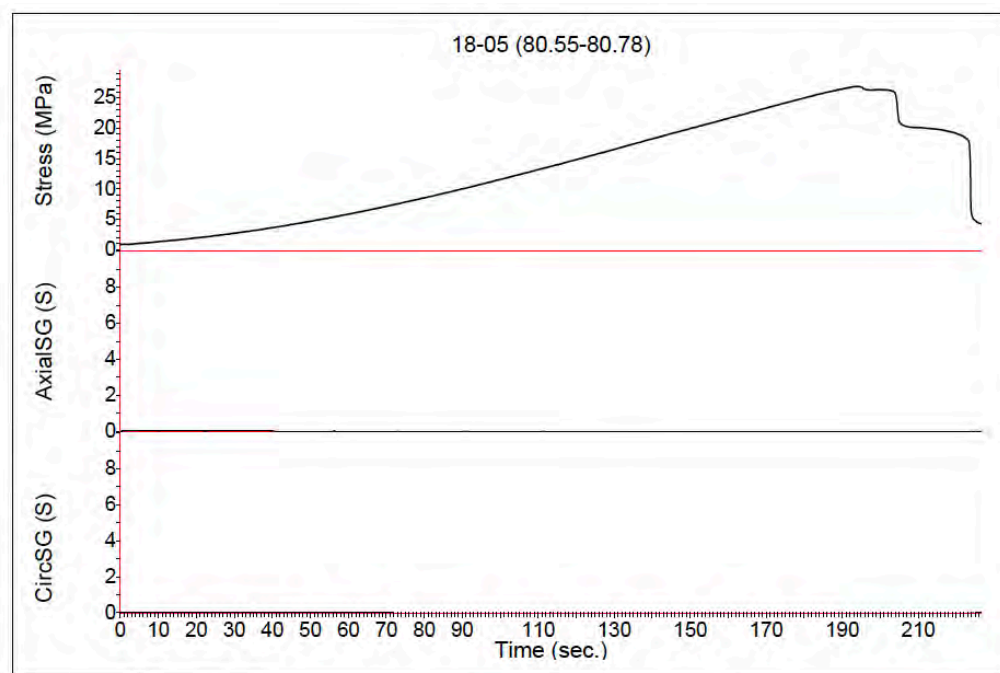
Performed By: P. Ross	Date: February 25, 2019
Verified By: J. F. Archibald	Date: February 25, 2019

Elastic Moduli of Intact Rock Core Specimens in Uniaxial Compression (ASTM D7012 D)

Client: JDS Mining & Energy Inc.	Lab: Queen's University
Project/Site: Coffee Gold Mine	Project No.: Unknown
Borehole No.: GT18-05	Sample ID: UCS (no moduli)
Depth: (80.55-80.78 m)	Date Sampled: NA
Lithological Description: Granite	

Specimen Parameters	
Diameter, cm	6.09
Length, cm	14.95
Height-to-Diameter Ratio	2.45/1
Bulk Density, kg/m ³	2370
Moisture Condition	NA

Unconfined Compressive Strength (MPa)	26.9
Young's Modulus, E (GPa)	---
Poisson's ratio	---



Description: 18-05 (80.55-80.78)

Remarks: Failure completely through intact rock at 20" to core axis

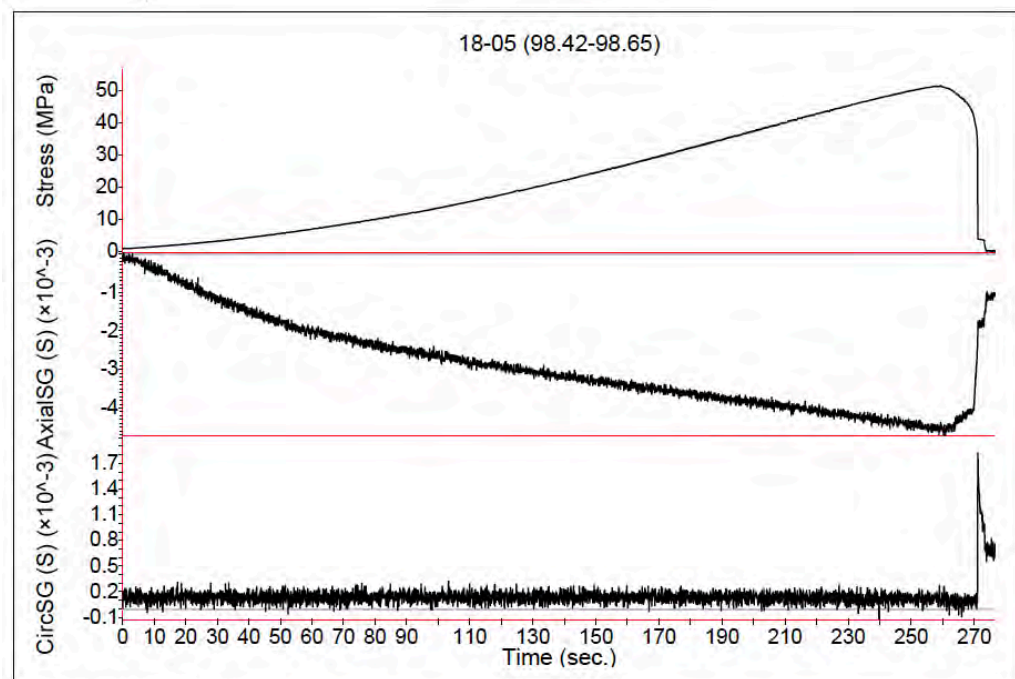
Performed By: P. Ross	Date: February 25, 2019
Verified By: J. F. Archibald	Date: February 25, 2019

Elastic Moduli of Intact Rock Core Specimens in Uniaxial Compression (ASTM D7012 D)

Client: JDS Mining & Energy Inc.	Lab: Queen's University
Project/Site: Coffee Gold Mine	Project No.: Unknown
Borehole No.: GT18-05	Sample ID: UCS
Depth: (98.42-98.65 m)	Date Sampled: NA
Lithological Description: Granite	

Specimen Parameters	
Diameter, cm	6.09
Length, cm	13.74
Height-to-Diameter Ratio	2.26/1
Bulk Density, kg/m ³	2600
Moisture Condition	NA

Unconfined Compressive Strength (MPa)	51.5
Young's Modulus, E (GPa)	13.568
Poisson's ratio	0.07



Description: 18-05 (98.42-98.65)

Remarks: Failure completely through intact rock at 10" to core axis

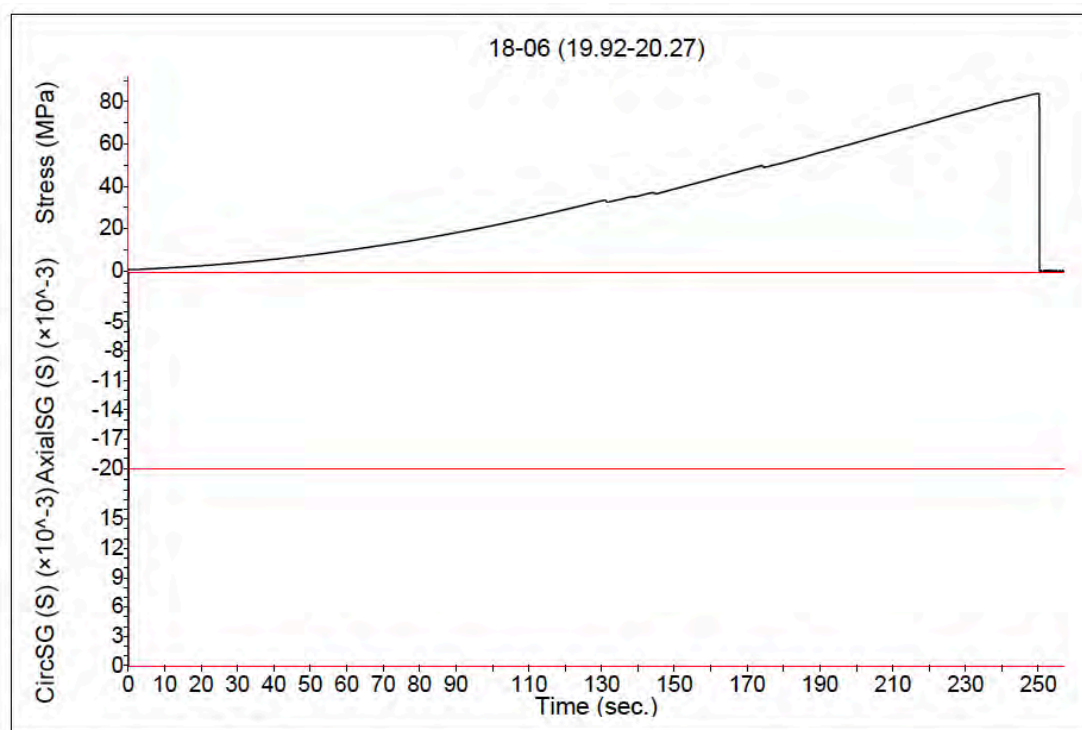
Performed By: P. Ross	Date: February 26, 2019
Verified By: J. F. Archibald	Date: February 26, 2019

Elastic Moduli of Intact Rock Core Specimens in Uniaxial Compression (ASTM D7012 D)

Client: JDS Mining & Energy Inc.	Lab: Queen's University
Project/Site: Coffee Gold Mine	Project No.: Unknown
Borehole No.: GT18-06	Sample ID: UCS (no moduli)
Depth: (19.92-20.27 m)	Date Sampled: NA
Lithological Description: Felsic Gneiss	

Specimen Parameters	
Diameter, cm	6.07
Length, cm	13.52
Height-to-Diameter Ratio	2.23/1
Bulk Density, kg/m ³	2630
Moisture Condition	NA

Unconfined Compressive Strength (MPa)	83.9
Young's Modulus, E (GPa)	---
Poisson's ratio	---



Description: 18-06 (19.92-20.27)

Remarks: Failed partially along pre-existing foliation at 30" to core axis (40% failure on foliation surface)

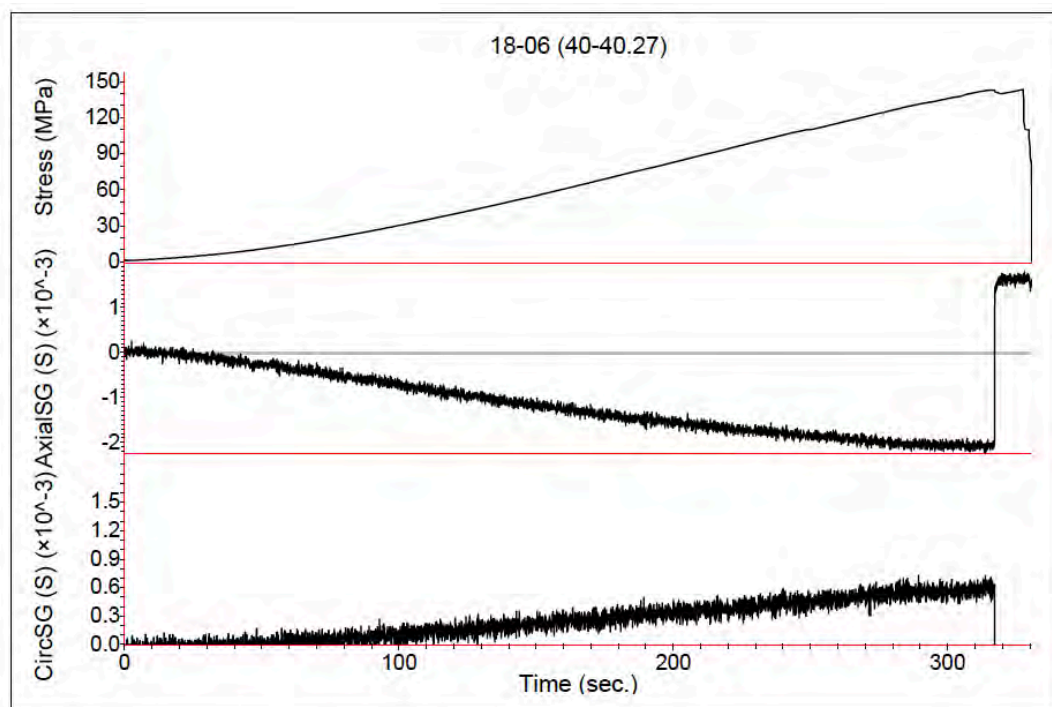
Performed By: P. Ross	Date: February 25, 2019
Verified By: J. F. Archibald	Date: February 25, 2019

Elastic Moduli of Intact Rock Core Specimens in Uniaxial Compression (ASTM D7012 D)

Client: JDS Mining & Energy Inc.	Lab: Queen's University
Project/Site: Coffee Gold Mine	Project No.: Unknown
Borehole No.: GT18-06	Sample ID: UCS
Depth: (40.00-40.27 m)	Date Sampled: NA
Lithological Description: Felsic Gneiss	

Specimen Parameters	
Diameter, cm	6.08
Length, cm	13.52
Height-to-Diameter Ratio	2.22/1
Bulk Density, kg/m ³	2650
Moisture Condition	NA

Unconfined Compressive Strength (MPa)	143.5
Young's Modulus, E (GPa)	30.683
Poisson's ratio	0.11



Description: 18-06 (40-40.27)

Remarks: Failed partially along pre-existing foliation at 15" to core axis (50% failure on foliation surface)

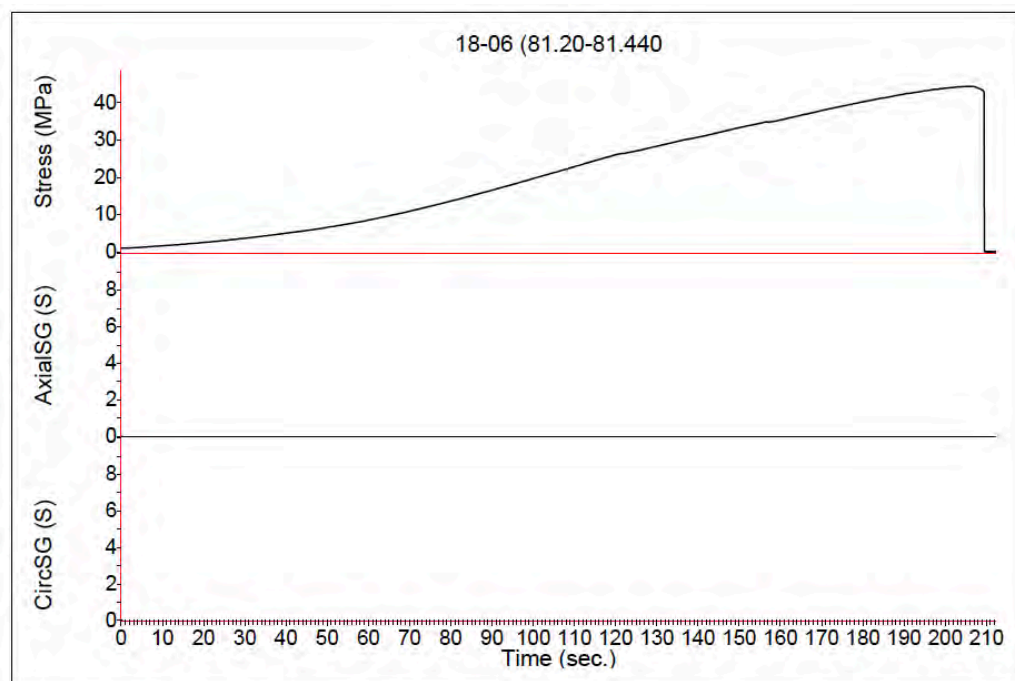
Performed By: P. Ross	Date: February 26, 2019
Verified By: J. F. Archibald	Date: February 26, 2019

Elastic Moduli of Intact Rock Core Specimens in Uniaxial Compression (ASTM D7012 D)

Client: JDS Mining & Energy Inc.	Lab: Queen's University
Project/Site: Coffee Gold Mine	Project No.: Unknown
Borehole No.: GT18-06	Sample ID: UCS (no moduli)
Depth: (81.20-81.44 m)	Date Sampled: NA
Lithological Description: Gneiss	

Specimen Parameters	
Diameter, cm	6.10
Length, cm	14.67
Height-to-Diameter Ratio	2.40/1
Bulk Density, kg/m ³	3010
Moisture Condition	NA

Unconfined Compressive Strength (MPa)	44.4
Young's Modulus, E (GPa)	---
Poisson's ratio	---



Description: 18-06 (81.20-81.440)

Remarks: Failed partially along pre-existing foliation at 10" to core axis (75% failure on foliation surface)

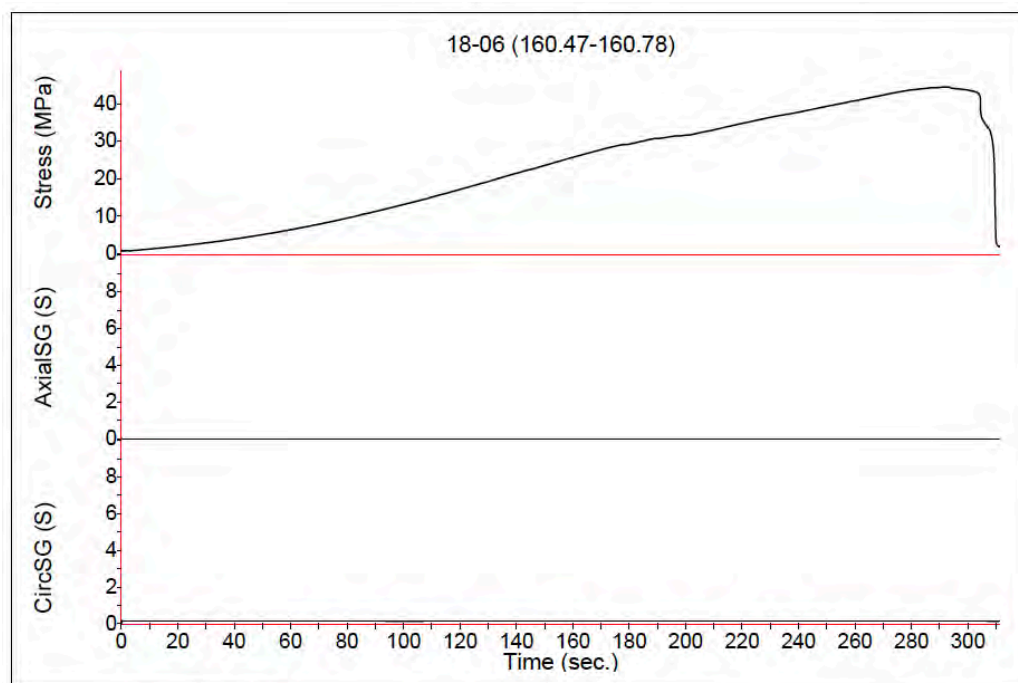
Performed By: P. Ross	Date: February 25, 2019
Verified By: J. F. Archibald	Date: February 25, 2019

Elastic Moduli of Intact Rock Core Specimens in Uniaxial Compression (ASTM D7012 D)

Client: JDS Mining & Energy Inc.	Lab: Queen's University
Project/Site: Coffee Gold Mine	Project No.: Unknown
Borehole No.: GT18-06	Sample ID: UCS (no moduli)
Depth: (160.47-160.78 m)	Date Sampled: NA
Lithological Description: Felsic Gneiss	

Specimen Parameters	
Diameter, cm	6.08
Length, cm	14.93
Height-to-Diameter Ratio	2.46/1
Bulk Density, kg/m ³	2430
Moisture Condition	NA

Unconfined Compressive Strength (MPa)	44.5
Young's Modulus, E (GPa)	---
Poisson's ratio	---



Description: 18-06 (160.47-160.78)

Remarks: Failure completely through intact rock parallel to core axis

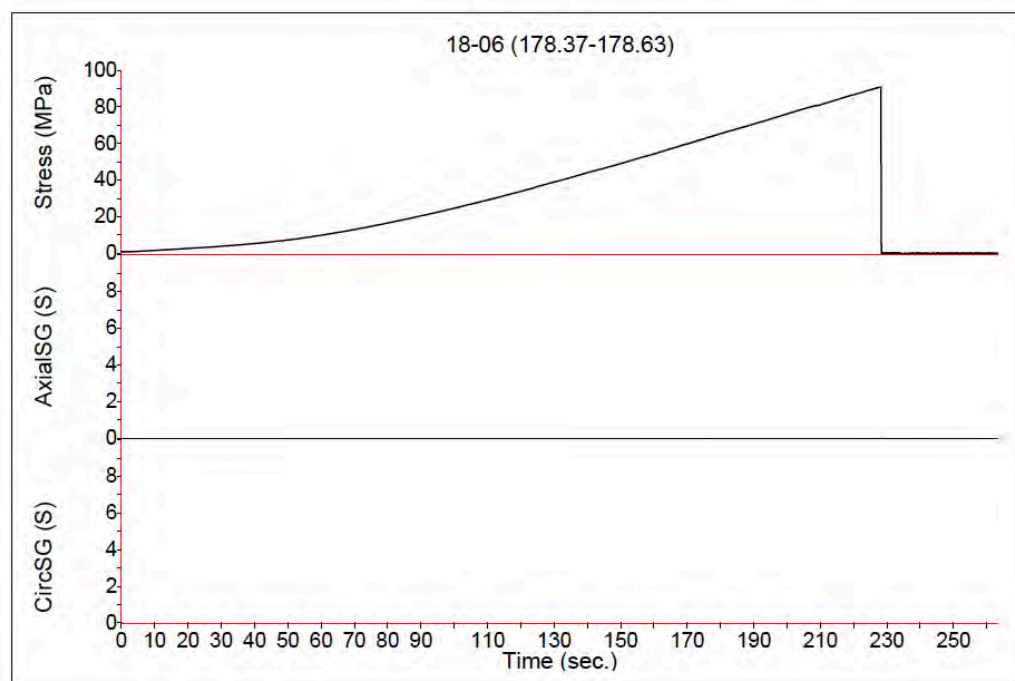
Performed By: P. Ross	Date: February 25, 2019
Verified By: J. F. Archibald	Date: February 25, 2019

Elastic Moduli of Intact Rock Core Specimens in Uniaxial Compression (ASTM D7012 D)

Client: JDS Mining & Energy Inc.	Lab: Queen's University
Project/Site: Coffee Gold Mine	Project No.: Unknown
Borehole No.: GT18-06	Sample ID: UCS (no moduli)
Depth: (178.37-178.63 m)	Date Sampled: NA
Lithological Description: Felsic Gneiss	

Specimen Parameters	
Diameter, cm	6.08
Length, cm	14.66
Height-to-Diameter Ratio	2.41/1
Bulk Density, kg/m ³	2650
Moisture Condition	NA

Unconfined Compressive Strength (MPa)	90.9
Young's Modulus, E (GPa)	---
Poisson's ratio	---



Description: 18-06 (178.37-178.63)

Remarks: Failed partially along pre-existing foliation at 20" to core axis (30% failure on foliation surface)

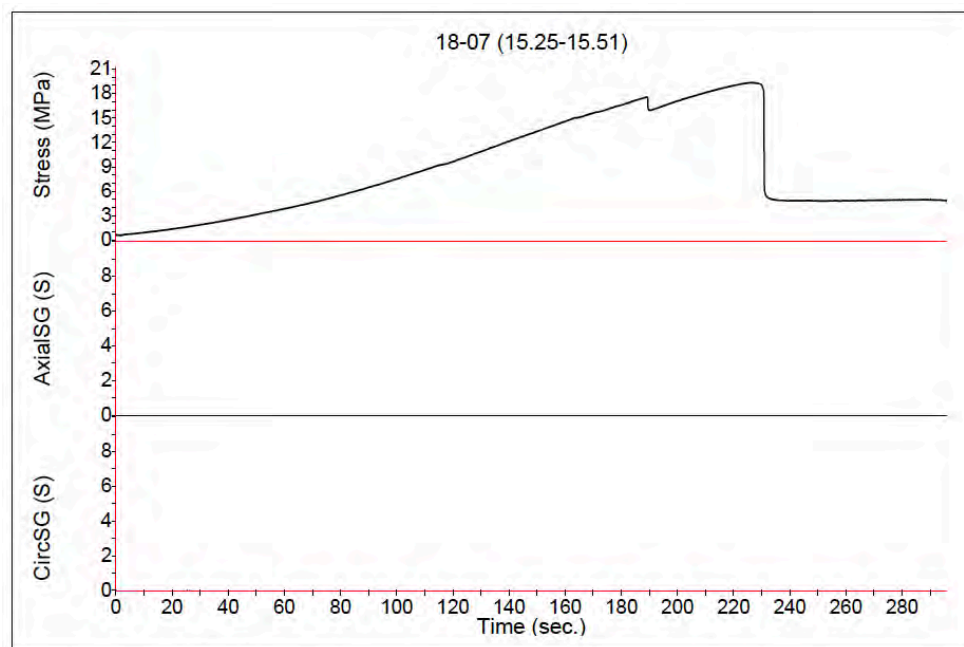
Performed By: P. Ross	Date: February 25, 2019
Verified By: J. F. Archibald	Date: February 25, 2019

Elastic Moduli of Intact Rock Core Specimens in Uniaxial Compression (ASTM D7012 D)

Client: JDS Mining & Energy Inc.	Lab: Queen's University
Project/Site: Coffee Gold Mine	Project No.: Unknown
Borehole No.: GT18-07	Sample ID: UCS (no moduli)
Depth: (15.25-15.51 m)	Date Sampled: NA
Lithological Description: Biotite-feldspar schist	

Specimen Parameters	
Diameter, cm	6.08
Length, cm	14.98
Height-to-Diameter Ratio	2.46/1
Bulk Density, kg/m ³	2300
Moisture Condition	NA

Unconfined Compressive Strength (MPa)	19.4
Young's Modulus, E (GPa)	---
Poisson's ratio	---



Description: 18-07 (15.25-15.51)

Remarks: Failed partially along pre-existing foliation at 40" to core axis (80% failure on foliation surface)

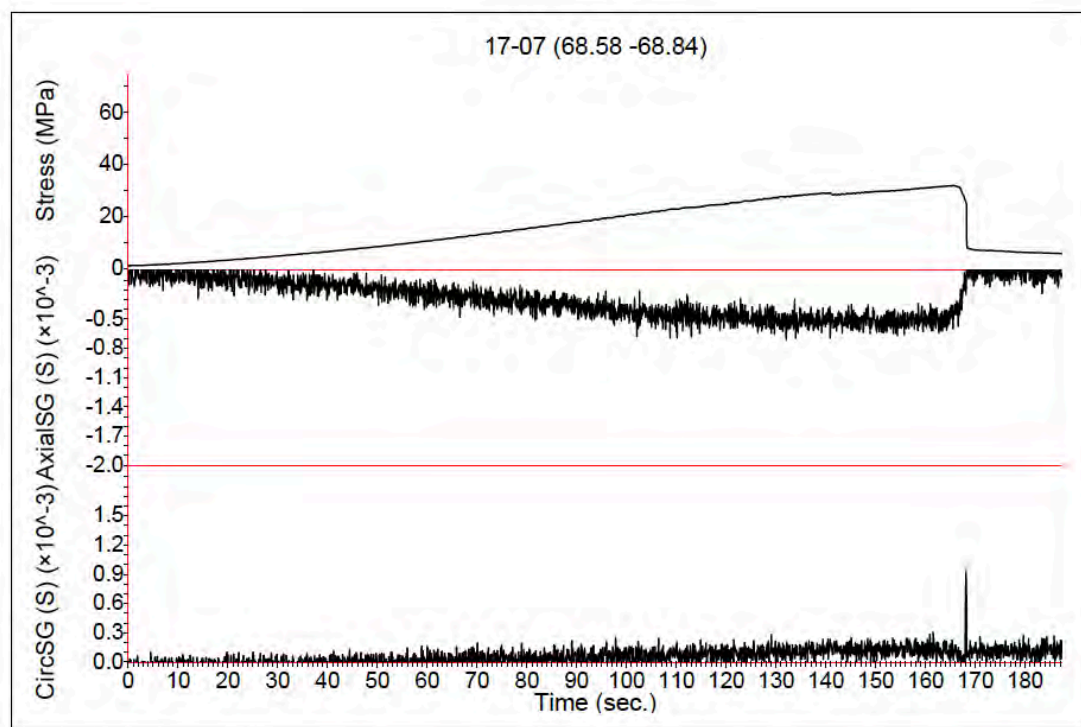
Performed By: P. Ross	Date: February 25, 2019
Verified By: J. F. Archibald	Date: February 25, 2019

Elastic Moduli of Intact Rock Core Specimens in Uniaxial Compression (ASTM D7012 D)

Client: JDS Mining & Energy Inc.	Lab: Queen's University
Project/Site: Coffee Gold Mine	Project No.: Unknown
Borehole No.: GT18-07	Sample ID: UCS
Depth: (68.58-68.84 m)	Date Sampled: NA
Lithological Description: Biotite-feldspar schist	

Specimen Parameters	
Diameter, cm	6.09
Length, cm	14.20
Height-to-Diameter Ratio	2.33/1
Bulk Density, kg/m ³	2540
Moisture Condition	NA

Unconfined Compressive Strength (MPa)	32.0
Young's Modulus, E (GPa)	4.984
Poisson's ratio	0.09



Description: 17-07 (68.58 -68.84)

Remarks: Failed partially along pre-existing foliation at 5" to core axis (50% failure on foliation surface)

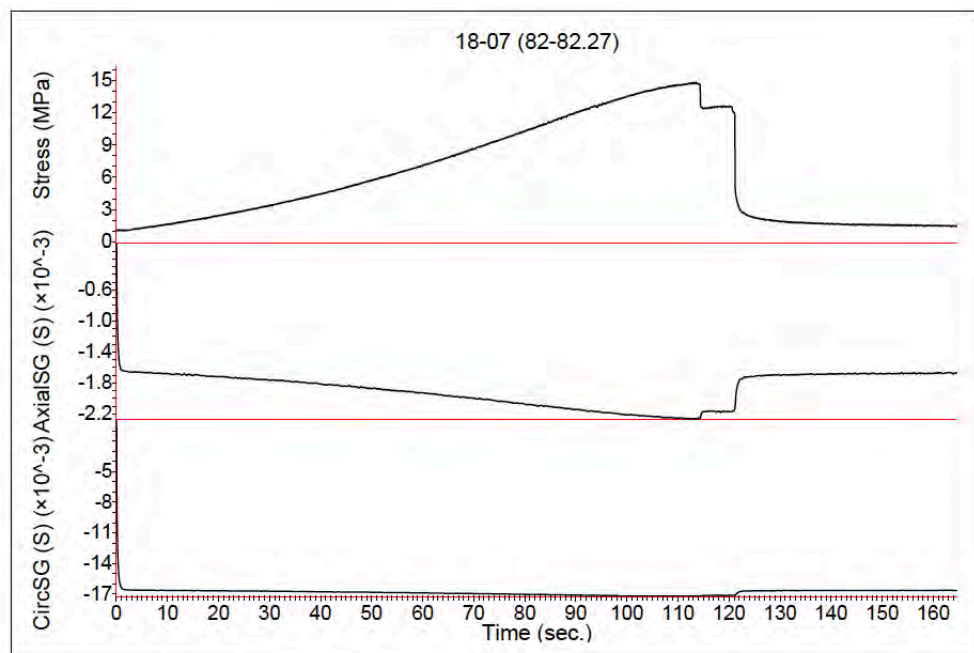
Performed By: P. Ross	Date: February 26, 2019
Verified By: J. F. Archibald	Date: February 26, 2019

Elastic Moduli of Intact Rock Core Specimens in Uniaxial Compression (ASTM D7012 D)

Client: JDS Mining & Energy Inc.	Lab: Queen's University
Project/Site: Coffee Gold Mine	Project No.: Unknown
Borehole No.: GT18-07	Sample ID: UCS (no moduli)
Depth: (82.00-82.27 m)	Date Sampled: NA
Lithological Description: Biotite-feldspar schist	

Specimen Parameters	
Diameter, cm	6.10
Length, cm	14.35
Height-to-Diameter Ratio	2.35/1
Bulk Density, kg/m ³	2450
Moisture Condition	NA

Unconfined Compressive Strength (MPa)	14.8
Young's Modulus, E (GPa)	---
Poisson's ratio	---



Description: 18-07 (82-82.27)

Remarks: Failed partially along pre-existing foliation at 30" to core axis (95% failure on foliation surface)

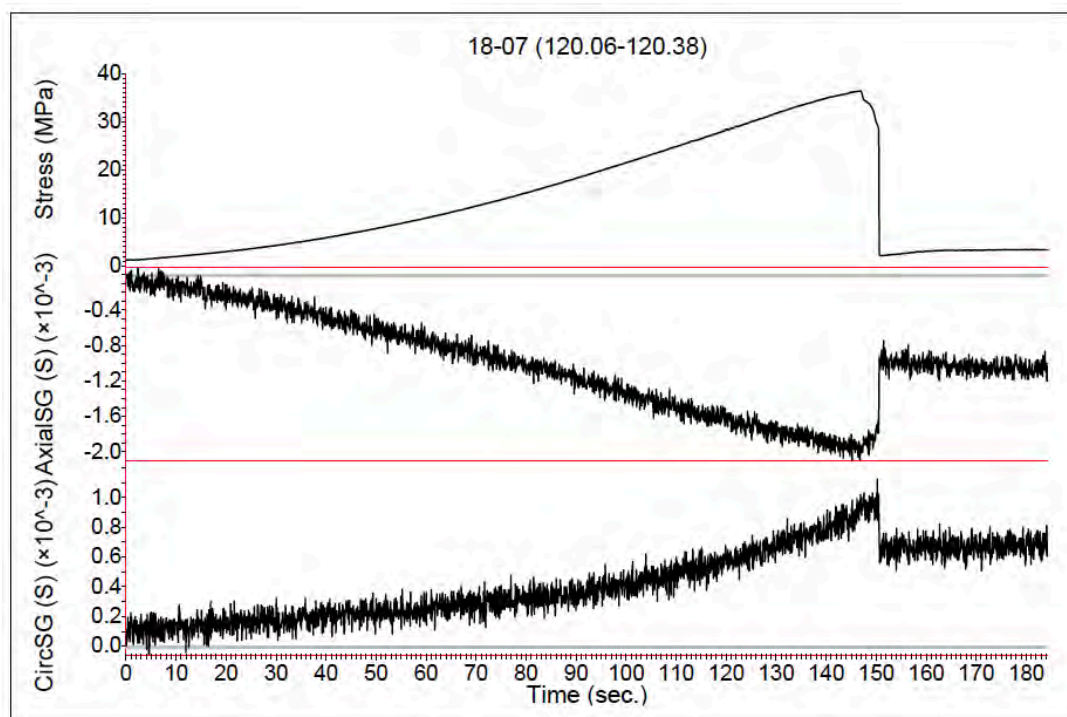
Performed By: P. Ross	Date: February 25, 2019
Verified By: J. F. Archibald	Date: February 25, 2019

Elastic Moduli of Intact Rock Core Specimens in Uniaxial Compression (ASTM D7012 D)

Client: JDS Mining & Energy Inc.	Lab: Queen's University
Project/Site: Coffee Gold Mine	Project No.: Unknown
Borehole No.: GT18-07	Sample ID: UCS
Depth: (120.06-120.38 m)	Date Sampled: NA
Lithological Description: Biotite-feldspar schist with carbonate	

Specimen Parameters	
Diameter, cm	6.10
Length, cm	11.43
Height-to-Diameter Ratio	1.87/1
Bulk Density, kg/m ³	2520
Moisture Condition	NA

Unconfined Compressive Strength (MPa)	36.4
Young's Modulus, E (GPa)	13.617
Poisson's ratio	0.29



Description: 18-07 (120.06-120.38)

Remarks: Failed partially along pre-existing foliation at 35" to core axis (40% failure on foliation surface)

Performed By: P. Ross	Date: February 26, 2019
Verified By: J. F. Archibald	Date: February 26, 2019

Elastic Moduli of Intact Rock Core Specimens in Uniaxial Compression (ASTM D7012 D)

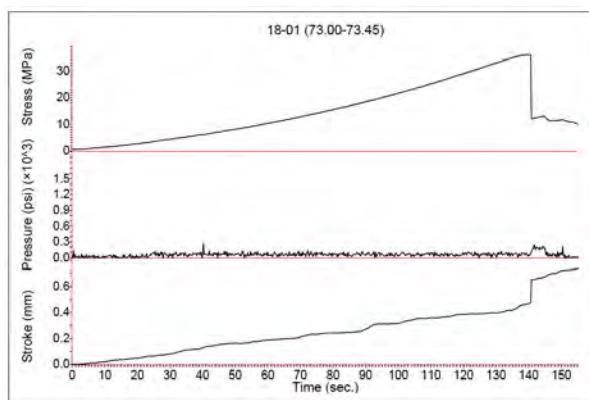
Client: JDS Mining & Energy Inc.	Lab: Queen's University
Project/Site: Coffee Gold Mine	Project No.: Unknown
Borehole: GT18-01	Sample: Triaxial Confined
Depth: (73.00-73.45 m)	Date Sampled: NA
Lithological Description: Gneiss	

Initial Sample Parameters

Specimen	GT18-01
Depth	(73.00-73.45 m)
Diameter, cm	6.07
Height, cm	13.70
Height-to-Diameter Ratio	2.26/1
Volume, cm ³	396.2
Mass, g	1047.3
Bulk Density, kg/m ³	2640
Moisture Condition	As-received

Failed Sample Parameters

Specimen	Sigma 3	Sigma 1
Confining Stress (a) and Fail Stress, MPa	0.5	36.1
Confining Stress (b) and Fail Stress, MPa	2	---
Confining Stress (c) and Fail Stress, MPa	5	---
Confining Stress (d) and Fail Stress, MPa	10	---
Confining Stress (e) and Fail Stress, MPa	---	---
Total Time to Failure, sec	---	---
Strain Rate, microstrain/s	---	34.7



Description: 18-01 (73.00-73.45)

Remarks: Failed fully along pre-existing foliation at 25" to core axis (100% failure on foliation surface); failure occurred during first confinement test thus single stage triaxial test only	
Performed by: Perry Ross	Date: March 8, 2019
Verified by: J. F. Archibald	Date: March 8, 2019

Triaxial Compressive Strength of Undrained Rock Core Specimen Without Pore Pressure Measurements (ASTM D7012 – Method A)

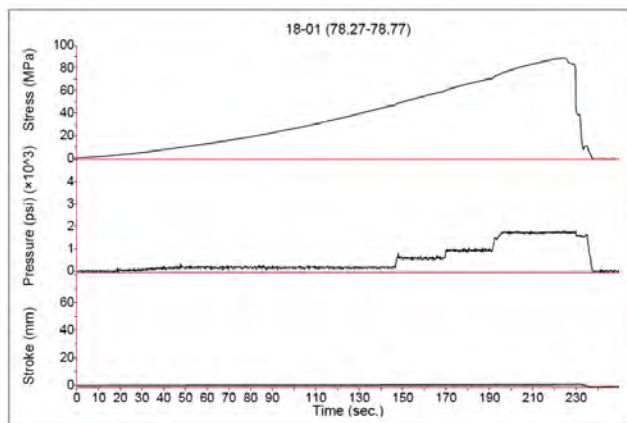
Client: JDS Mining & Energy Inc.	Lab: Queen's University
Project/Site: Coffee Gold Mine	Project No.: Unknown
Borehole: GT18-01	Sample: Triaxial Confined
Depth: (787.27-78.77 m)	Date Sampled: NA
Lithological Description: Gneiss	

Initial Sample Parameters

Specimen	GT18-01
Depth	(78.27-78.77 m)
Diameter, cm	6.08
Height, cm	13.63
Height-to-Diameter Ratio	2.24/1
Volume, cm ³	395.5
Mass, g	1057.7
Bulk Density, kg/m ³	2674
Moisture Condition	As-received

Failed Sample Parameters

Specimen	Sigma 3	Sigma 1
Confining Stress (a) and Fail Stress, MPa	1	46.5
Confining Stress (b) and Fail Stress, MPa	3	59.7
Confining Stress (c) and Fail Stress, MPa	6	70.0
Confining Stress (d) and Fail Stress, MPa	12	88.5
Confining Stress (e) and Fail Stress, MPa		
Total Time to Failure, sec	---	
Strain Rate, microstrain/s	---	34.8



Description: 18-01 (78.27-78.77)

Remarks: Failed partially along pre-existing foliation at 20" to core axis (85% failure on foliation surface)	
Performed by: Perry Ross	Date: March 8, 2019
Verified by: J. F. Archibald	Date: March 8, 2019

Triaxial Compressive Strength of Undrained Rock Core Specimen Without Pore Pressure Measurements (ASTM D7012 – Method A)

Client: JDS Mining & Energy Inc.	Lab: Queen's University
Project/Site: Coffee Gold Mine	Project No.: Unknown
Borehole: GT18-01	Sample: Triaxial Confined
Depth: (119.50-119.72 m)	Date Sampled: NA
Lithological Description: Felsic gneiss	

Initial Sample Parameters

Specimen	GT18-01
Depth	(119.50-119.72 m)
Diameter, cm	6.06
Height, cm	13.70
Height-to-Diameter Ratio	2.26/1
Volume, cm ³	394.9
Mass, g	1036.1
Bulk Density, kg/m ³	2620
Moisture Condition	As-received

Failed Sample Parameters

Specimen	Sigma 3	Sigma 1
Confining Stress (a) and Fail Stress, MPa	1	---
Confining Stress (b) and Fail Stress, MPa	4	---
Confining Stress (c) and Fail Stress, MPa	10	---
Confining Stress (d) and Fail Stress, MPa	20	---
Confining Stress (e) and Fail Stress, MPa		
Total Time to Failure, sec	---	
Strain Rate, microstrain/s	---	34.6



Broken during initial testing – no results

Remarks: Sample failed during test insertion and prior to confinement – no data acquisition	
Performed by: Perry Ross	Date: March 8, 2019
Verified by: J. F. Archibald	Date: March 8, 2019

Triaxial Compressive Strength of Undrained Rock Core Specimen Without Pore Pressure Measurements (ASTM D7012 – Method A)

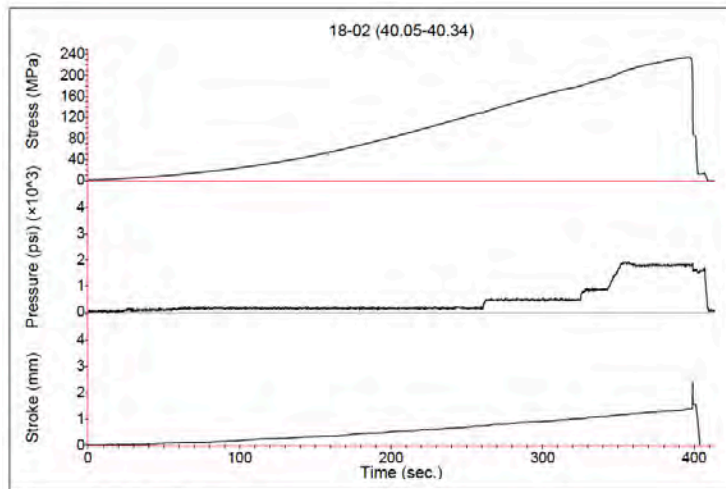
Client: JDS Mining & Energy Inc.	Lab: Queen's University
Project/Site: Coffee Gold Mine	Project No.: Unknown
Borehole: GT18-02	Sample: Triaxial Confined
Depth: (40.05-40.34 m)	Date Sampled: NA
Lithological Description: Felsic gneiss	

Initial Sample Parameters

Specimen	GT18-02
Depth	(40.05-40.34 m)
Diameter, cm	6.10
Height, cm	13.55
Height-to-Diameter Ratio	2.22/1
Volume, cm ³	395.8
Mass, g	1047.6
Bulk Density, kg/m ³	2650
Moisture Condition	As-received

Failed Sample Parameters

Specimen	Sigma 3	Sigma 1
Confining Stress (a) and Fail Stress, MPa	1	128.4
Confining Stress (b) and Fail Stress, MPa	3	179.1
Confining Stress (c) and Fail Stress, MPa	6	194.2
Confining Stress (d) and Fail Stress, MPa	12	234.7
Confining Stress (e) and Fail Stress, MPa		
Total Time to Failure, sec	---	
Strain Rate, microstrain/s	---	35.0



Description: 18-02 (40.05-40.34)

Remarks: Failed partially along pre-existing foliation at 25" to core axis (10% failure on foliation surface)	
Performed by: Perry Ross	Date: March 8, 2019
Verified by: J. F. Archibald	Date: March 8, 2019

Triaxial Compressive Strength of Undrained Rock Core Specimen Without Pore Pressure Measurements (ASTM D7012 – Method A)

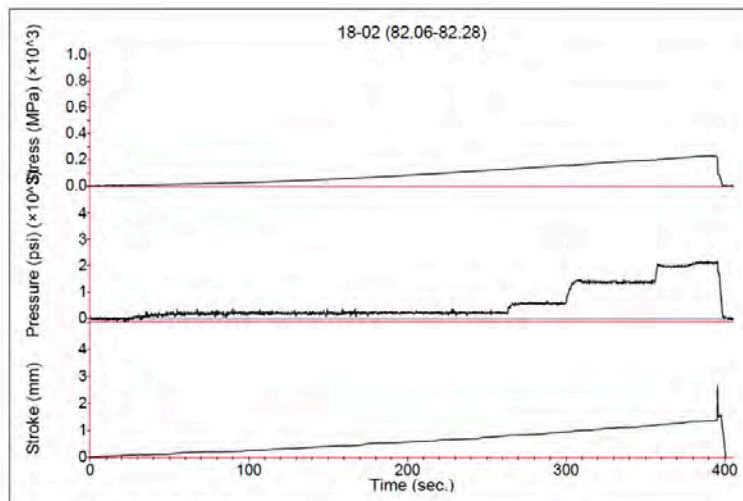
Client: JDS Mining & Energy Inc.	Lab: Queen's University
Project/Site: Coffee Gold Mine	Project No.: Unknown
Borehole: GT18-02	Sample: Triaxial Confined
Depth: (82.06-82.28 m)	Date Sampled: NA
Lithological Description: Felsic gneiss	

Initial Sample Parameters

Specimen	GT18-02
Depth	(82.06-82.28 m)
Diameter, cm	6.10
Height, cm	11.08
Height-to-Diameter Ratio	1.82/1
Volume, cm ³	323.6
Mass, g	840.7
Bulk Density, kg/m ³	2600
Moisture Condition	As-received

Failed Sample Parameters

Specimen	Sigma 3	Sigma 1
Confining Stress (a) and Fail Stress, MPa	1.5	124.1
Confining Stress (b) and Fail Stress, MPa	4	155.6
Confining Stress (c) and Fail Stress, MPa	9	196.9
Confining Stress (d) and Fail Stress, MPa	15	227.9
Confining Stress (e) and Fail Stress, MPa		
Total Time to Failure, sec	---	
Strain Rate, microstrain/s	---	42.8



Description: 18-02 (82.06-82.28)

Remarks: Failed fully along pre-existing foliation at 25" to core axis (100% failure on foliation surface)	
Performed by: Perry Ross	Date: March 8, 2019
Verified by: J. F. Archibald	Date: March 8, 2019

Triaxial Compressive Strength of Undrained Rock Core Specimen Without Pore Pressure Measurements (ASTM D7012 – Method A)

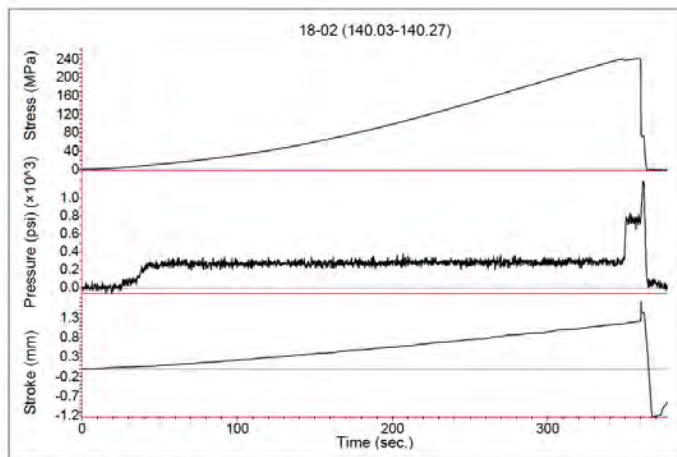
Client: JDS Mining & Energy Inc.	Lab: Queen's University
Project/Site: Coffee Gold Mine	Project No.: Unknown
Borehole: GT18-02	Sample: Triaxial Confined
Depth: (140.03-140.27 m)	Date Sampled: NA
Lithological Description: Felsic gneiss	

Initial Sample Parameters

Specimen	GT18-02
Depth	(140.03-140.27 m)
Diameter, cm	6.10
Height, cm	13.73
Height-to-Diameter Ratio	2.25/1
Volume, cm ³	401.1
Mass, g	1057.4
Bulk Density, kg/m ³	2640
Moisture Condition	As-received

Failed Sample Parameters

Specimen	Sigma 3	Sigma 1
Confining Stress (a) and Fail Stress, MPa	2	239.2
Confining Stress (b) and Fail Stress, MPa	5	241.5
Confining Stress (c) and Fail Stress, MPa	11	--
Confining Stress (d) and Fail Stress, MPa	25	--
Confining Stress (e) and Fail Stress, MPa		
Total Time to Failure, sec	---	
Strain Rate, microstrain/s	---	34.5



Description: 18-02 (140.03-140.27)

Remarks: Sample failed along foliation at 20 ° to core axis after second confinement process step	
Performed by: Perry Ross	Date: March 8, 2019
Verified by: J. F. Archibald	Date: March 8, 2019

Triaxial Compressive Strength of Undrained Rock Core Specimen Without Pore Pressure Measurements (ASTM D7012 – Method A)

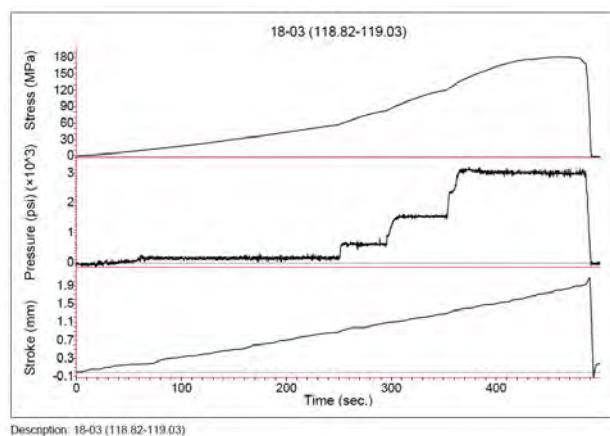
Client: JDS Mining & Energy Inc.	Lab: Queen's University
Project/Site: Coffee Gold Mine	Project No.: Unknown
Borehole: GT18-03	Sample: Triaxial Confined
Depth: (118.82-119.03 m)	Date Sampled: NA
Lithological Description: Biotite-feldspar schist with carbonate	

Initial Sample Parameters

Specimen	GT18-03
Depth	(118.82-119.03 m)
Diameter, cm	6.07
Height, cm	13.62
Height-to-Diameter Ratio	2.24/1
Volume, cm ³	393.3
Mass, g	1112.2
Bulk Density, kg/m ³	2820
Moisture Condition	As-received

Failed Sample Parameters

Specimen	Sigma 3	Sigma 1
Confining Stress (a) and Fail Stress, MPa	1	57.6
Confining Stress (b) and Fail Stress, MPa	4	82.0
Confining Stress (c) and Fail Stress, MPa	10	120.9
Confining Stress (d) and Fail Stress, MPa	20	180.6
Confining Stress (e) and Fail Stress, MPa		
Total Time to Failure, sec	---	
Strain Rate, microstrain/s	---	34.8



Remarks: Failed partially along pre-existing foliation at 30" to core axis (35% failure on foliation surface)	
Performed by: Perry Ross	Date: March 8, 2019
Verified by: J. F. Archibald	Date: March 8, 2019

Triaxial Compressive Strength of Undrained Rock Core Specimen Without Pore Pressure Measurements (ASTM D7012 – Method A)

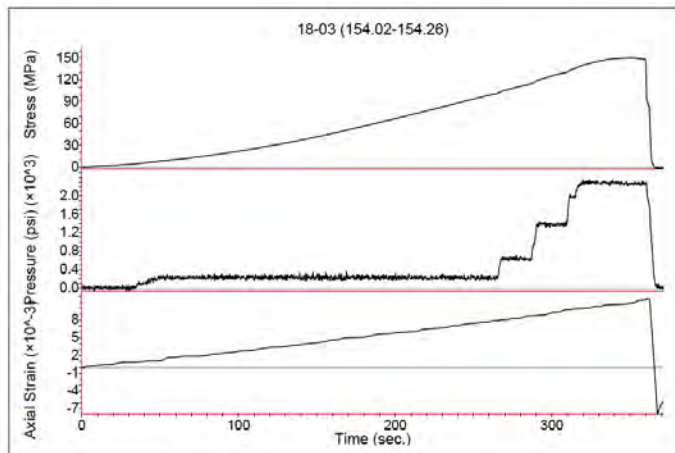
Client: JDS Mining & Energy Inc.	Lab: Queen's University
Project/Site: Coffee Gold Mine	Project No.: Unknown
Borehole: GT18-03	Sample: Triaxial Confined
Depth: (154.02-154.26 m)	Date Sampled: NA
Lithological Description: Biotite-feldspar schist with carbonate	

Initial Sample Parameters

Specimen	GT18-03
Depth	(154.02-154.26 m)
Diameter, cm	6.08
Height, cm	13.50
Height-to-Diameter Ratio	2.22/1
Volume, cm ³	391.8
Mass, g	1087.3
Bulk Density, kg/m ³	2770
Moisture Condition	As-received

Failed Sample Parameters

Specimen	Sigma 3	Sigma 1
Confining Stress (a) and Fail Stress, MPa	1.5	102.5
Confining Stress (b) and Fail Stress, MPa	4	115.1
Confining Stress (c) and Fail Stress, MPa	9	131.3
Confining Stress (d) and Fail Stress, MPa	15	150.9
Confining Stress (e) and Fail Stress, MPa		
Total Time to Failure, sec	---	
Strain Rate, microstrain/s	---	35.1



Description: 18-03 (154.02-154.26)

Remarks: Failed partially along pre-existing foliation at 45" to core axis (10% failure on foliation surface)	
Performed by: Perry Ross	Date: March 8, 2019
Verified by: J. F. Archibald	Date: March 8, 2019

Triaxial Compressive Strength of Undrained Rock Core Specimen Without Pore Pressure Measurements (ASTM D7012 – Method A)

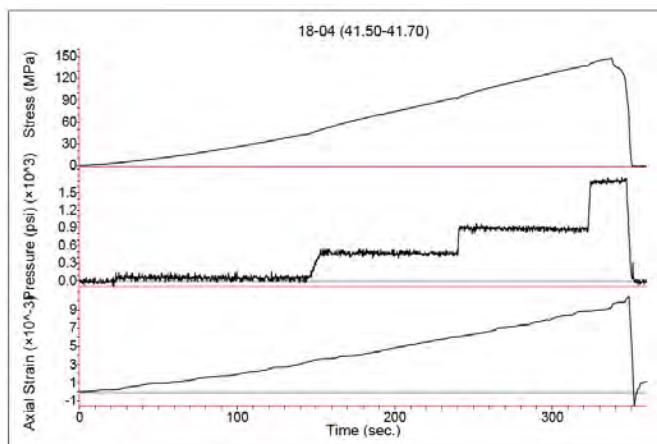
Client: JDS Mining & Energy Inc.	Lab: Queen's University
Project/Site: Coffee Gold Mine	Project No.: Unknown
Borehole: GT18-04	Sample: Triaxial Confined
Depth: (41.50-41.70 m)	Date Sampled: NA
Lithological Description: Felsic gneiss	

Initial Sample Parameters

Specimen	GT18-04
Depth	(41.50-41.70 m)
Diameter, cm	6.07
Height, cm	13.64
Height-to-Diameter Ratio	2.25/1
Volume, cm ³	394.5
Mass, g	1015.9
Bulk Density, kg/m ³	2580
Moisture Condition	As-received

Failed Sample Parameters

Specimen	Sigma 3	Sigma 1
Confining Stress (a) and Fail Stress, MPa	0.5	42.4
Confining Stress (b) and Fail Stress, MPa	3	92.1
Confining Stress (c) and Fail Stress, MPa	6	137.4
Confining Stress (d) and Fail Stress, MPa	12	147.5
Confining Stress (e) and Fail Stress, MPa		
Total Time to Failure, sec	---	
Strain Rate, microstrain/s	---	34.8



Description: 18-04 (41.50-41.70)

Remarks: Failed fully along pre-existing foliation at 15" to core axis (100% failure on foliation surface)	
Performed by: Perry Ross	Date: March 8, 2019
Verified by: J. F. Archibald	Date: March 8, 2019

Triaxial Compressive Strength of Undrained Rock Core Specimen Without Pore Pressure Measurements (ASTM D7012 – Method A)

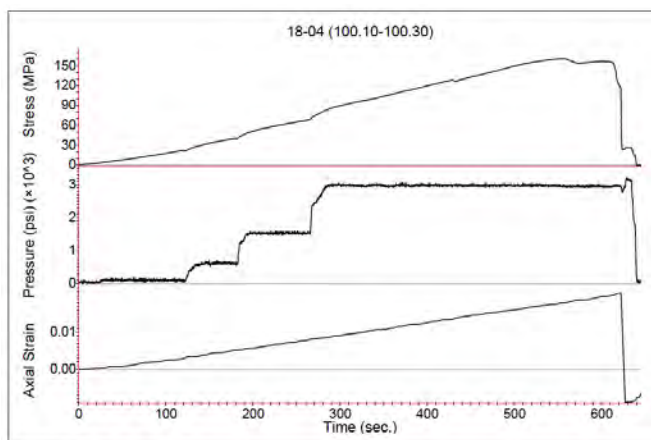
Client: JDS Mining & Energy Inc.	Lab: Queen's University
Project/Site: Coffee Gold Mine	Project No.: Unknown
Borehole: GT18-04	Sample: Triaxial Confined
Depth: (100.10-100.30 m)	Date Sampled: NA
Lithological Description: Felsic gneiss	

Initial Sample Parameters

Specimen	GT18-04
Depth	(100.10-100.30 m)
Diameter, cm	6.06
Height, cm	13.63
Height-to-Diameter Ratio	2.25/1
Volume, cm ³	392.9
Mass, g	1033.9
Bulk Density, kg/m ³	2630
Moisture Condition	As-received

Failed Sample Parameters

Specimen	Sigma 3	Sigma 1
Confining Stress (a) and Fail Stress, MPa	1	22.1
Confining Stress (b) and Fail Stress, MPa	4	40.5
Confining Stress (c) and Fail Stress, MPa	10	68.5
Confining Stress (d) and Fail Stress, MPa	20	160.8
Confining Stress (e) and Fail Stress, MPa		
Total Time to Failure, sec	---	
Strain Rate, microstrain/s	---	34.8



Description: 18-04 (100.10-100.30)

Remarks: Failed partially along pre-existing foliation at 30" to core axis (45% failure on foliation surface)	
Performed by: Perry Ross	Date: March 8, 2019
Verified by: J. F. Archibald	Date: March 8, 2019

Triaxial Compressive Strength of Undrained Rock Core Specimen Without Pore Pressure Measurements (ASTM D7012 – Method A)

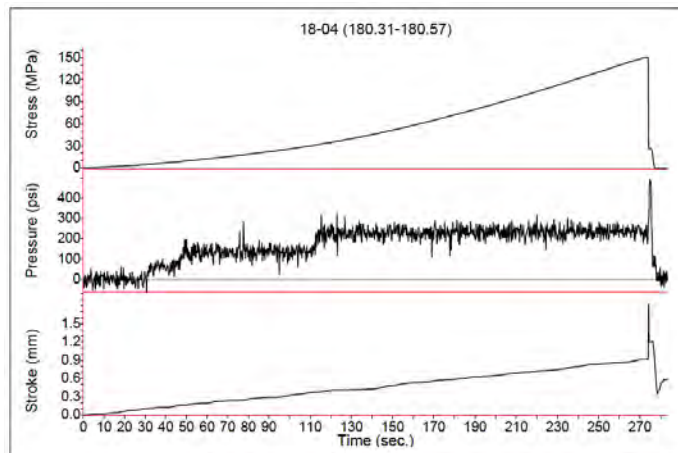
Client: JDS Mining & Energy Inc.	Lab: Queen's University
Project/Site: Coffee Gold Mine	Project No.: Unknown
Borehole: GT18-04	Sample: Triaxial Confined
Depth: (180.31-180.57 m)	Date Sampled: NA
Lithological Description: Felsic gneiss	

Initial Sample Parameters

Specimen	GT18-04
Depth	(180.31-180.57 m)
Diameter, cm	6.05
Height, cm	13.61
Height-to-Diameter Ratio	2.25/1
Volume, cm ³	391.1
Mass, g	1034.8
Bulk Density, kg/m ³	2650
Moisture Condition	As-received

Failed Sample Parameters

Specimen	Sigma 3	Sigma 1
Confining Stress (a) and Fail Stress, MPa	2	150.4
Confining Stress (b) and Fail Stress, MPa	5	---
Confining Stress (c) and Fail Stress, MPa	11	---
Confining Stress (d) and Fail Stress, MPa	25	---
Confining Stress (e) and Fail Stress, MPa		
Total Time to Failure, sec	---	
Strain Rate, microstrain/s	---	34.9



Description: 18-04 (180.31-180.57)

Remarks: Sample failed partially along foliation at 30° to core axis (20% foliation) after first confinement process step – single stage test only	
Performed by: Perry Ross	Date: March 8, 2019
Verified by: J. F. Archibald	Date: March 8, 2019

Triaxial Compressive Strength of Undrained Rock Core Specimen Without Pore Pressure Measurements (ASTM D7012 – Method A)

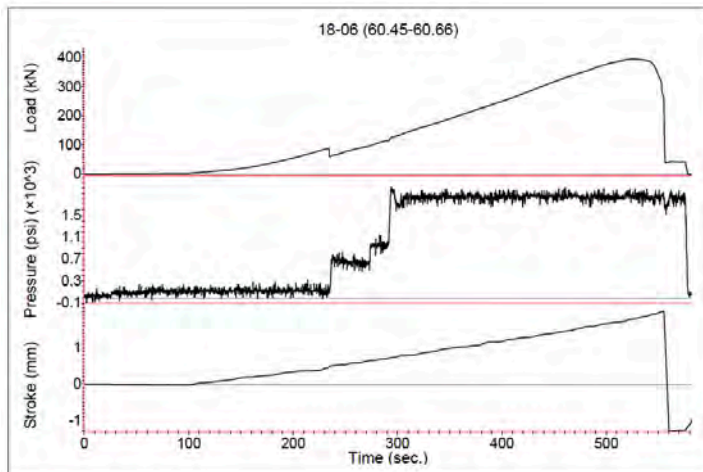
Client: JDS Mining & Energy Inc.	Lab: Queen's University
Project/Site: Coffee Gold Mine	Project No.: Unknown
Borehole: GT18-06	Sample: Triaxial Confined
Depth: (60.45-60.66 m)	Date Sampled: NA
Lithological Description: Gneiss	

Initial Sample Parameters

Specimen	GT18-06
Depth	(60.45-60.66 m)
Diameter, cm	6.08
Height, cm	13.56
Height-to-Diameter Ratio	2.23/1
Volume, cm ³	393.5
Mass, g	1120.9
Bulk Density, kg/m ³	2850
Moisture Condition	As-received

Failed Sample Parameters

Specimen	Sigma 3	Sigma 1
Confining Stress (a) and Fail Stress, MPa	1	29.9
Confining Stress (b) and Fail Stress, MPa	3	32.2
Confining Stress (c) and Fail Stress, MPa	6	40.2
Confining Stress (d) and Fail Stress, MPa	12	136.2
Confining Stress (e) and Fail Stress, MPa		
Total Time to Failure, sec	---	
Strain Rate, microstrain/s	---	35.0



Description: 18-06 (60.45-60.66)

Remarks: Failed partially along pre-existing foliation at 30" to core axis (50% failure on foliation surface)	
Performed by: Perry Ross	Date: March 8, 2019
Verified by: J. F. Archibald	Date: March 8, 2019

Triaxial Compressive Strength of Undrained Rock Core Specimen Without Pore Pressure Measurements (ASTM D7012 – Method A)

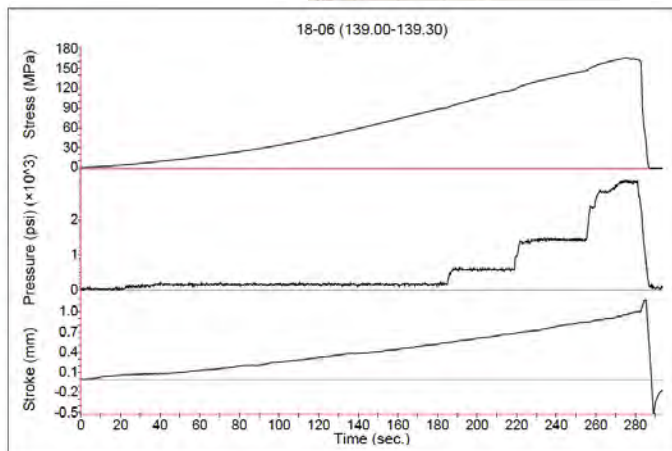
Client: JDS Mining & Energy Inc.	Lab: Queen's University
Project/Site: Coffee Gold Mine	Project No.: Unknown
Borehole: GT18-06	Sample: Triaxial Confined
Depth: (139.00-139.30 m)	Date Sampled: NA
Lithological Description: Felsic gneiss	

Initial Sample Parameters

Specimen	GT18-06
Depth	(139.00-139.30 m)
Diameter, cm	6.06
Height, cm	13.61
Height-to-Diameter Ratio	2.25/1
Volume, cm ³	392.3
Mass, g	1043.1
Bulk Density, kg/m ³	2660
Moisture Condition	As-received

Failed Sample Parameters

Specimen	Sigma 3	Sigma 1
Confining Stress (a) and Fail Stress, MPa	1	90.5
Confining Stress (b) and Fail Stress, MPa	4	117.4
Confining Stress (c) and Fail Stress, MPa	10	146.2
Confining Stress (d) and Fail Stress, MPa	20	166.7
Confining Stress (e) and Fail Stress, MPa		
Total Time to Failure, sec	---	
Strain Rate, microstrain/s	---	34.9



Description: 18-06 (139.00-139.30)

Remarks: Failed partially along pre-existing foliation at 30" to core axis (45% failure on foliation surface)	
Performed by: Perry Ross	Date: March 8, 2019
Verified by: J. F. Archibald	Date: March 8, 2019

Triaxial Compressive Strength of Undrained Rock Core Specimen Without Pore Pressure Measurements (ASTM D7012 – Method A)

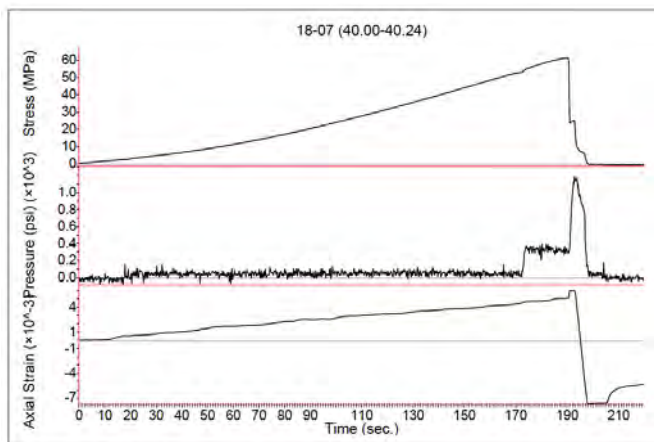
Client: JDS Mining & Energy Inc.	Lab: Queen's University
Project/Site: Coffee Gold Mine	Project No.: Unknown
Borehole: GT18-07	Sample: Triaxial Confined
Depth: (40.00-40.24 m)	Date Sampled: NA
Lithological Description: Biotite-feldspar schist with carbonate	

Initial Sample Parameters

Specimen	GT18-07
Depth	(40.00-40.24 m)
Diameter, cm	6.08
Height, cm	13.74
Height-to-Diameter Ratio	2.26/1
Volume, cm ³	398.7
Mass, g	1051.5
Bulk Density, kg/m ³	2640
Moisture Condition	As-received

Failed Sample Parameters

Specimen	Sigma 3	Sigma 1
Confining Stress (a) and Fail Stress, MPa	0.5	53.1
Confining Stress (b) and Fail Stress, MPa	2	61.5
Confining Stress (c) and Fail Stress, MPa	5	---
Confining Stress (d) and Fail Stress, MPa	10	---
Confining Stress (e) and Fail Stress, MPa		
Total Time to Failure, sec	---	
Strain Rate, microstrain/s	---	34.5



Description: 18-07 (40.00-40.24)

Remarks: Sample failed along partially foliation at 40° to core axis (90% on foliation surface) after second confinement process step	
Performed by: Perry Ross	Date: March 8, 2019
Verified by: J. F. Archibald	Date: March 8, 2019

Triaxial Compressive Strength of Undrained Rock Core Specimen Without Pore Pressure Measurements (ASTM D7012 – Method A)

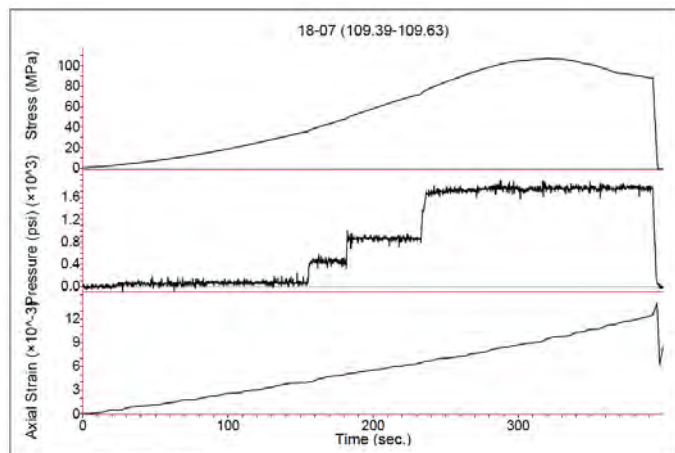
Client: JDS Mining & Energy Inc.	Lab: Queen's University
Project/Site: Coffee Gold Mine	Project No.: Unknown
Borehole: GT18-07	Sample: Triaxial Confined
Depth: (109.39-109.63 m)	Date Sampled: NA
Lithological Description: Biotite-feldspar schist	

Initial Sample Parameters

Specimen	GT18-07
Depth	(109.39-109.63 m)
Diameter, cm	6.11
Height, cm	13.68
Height-to-Diameter Ratio	2.24/1
Volume, cm ³	400.9
Mass, g	944.8
Bulk Density, kg/m ³	2360
Moisture Condition	As-received

Failed Sample Parameters

Specimen	Sigma 3	Sigma 1
Confining Stress (a) and Fail Stress, MPa	0.5	35.6
Confining Stress (b) and Fail Stress, MPa	3	48.7
Confining Stress (c) and Fail Stress, MPa	6	71.6
Confining Stress (d) and Fail Stress, MPa	12	107.1
Confining Stress (e) and Fail Stress, MPa		
Total Time to Failure, sec	---	
Strain Rate, microstrain/s	---	34.7



Description: 18-07 (109.39-109.63)

Remarks: Failed partially along pre-existing foliation at 30" to core axis (70% failure on foliation surface)	
Performed by: Perry Ross	Date: March 8, 2019
Verified by: J. F. Archibald	Date: March 8, 2019

Triaxial Compressive Strength of Undrained Rock Core Specimen Without Pore Pressure Measurements (ASTM D7012 – Method A)

Client: JDS Mining & Energy Inc.	Lab: Queen's University
Project/Site: Coffee Gold Project	Project No.: PO 6448
Borehole/Run: GT 18-04	Sample ID: GT 18-04 (164.50-165.14)
Depth: 164.50-165.14 m	Date Sampled: NA
Lithological Description: Felsic gneiss	

Sample Parameters

Specimen	GT 18-04 (164.50-165.14)
Nominal Cross-Sectional Area, cm ²	36.7
Moisture Condition	As-received

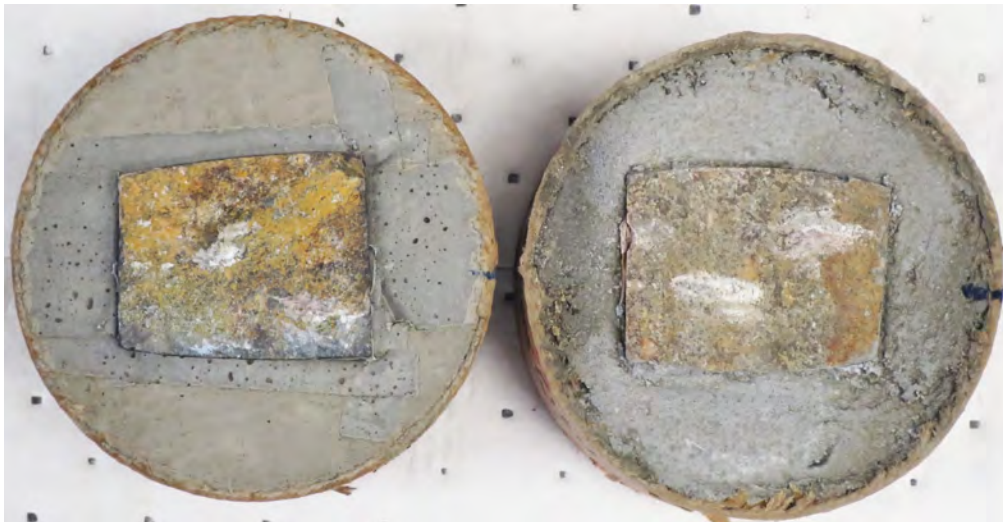
Residual Shear and normal stress data:

Normal Stress, MPa	0.43	0.93	1.16	1.47	1.87	2.20	2.46	2.82	3.18	3.46
Residual Shear Stress, MPa	0.20	0.43	0.52	0.67	0.84	1.00	1.09	1.19	1.42	1.68



Cohesion = 0 kPa

Internal Friction
Angle = 24.6°



Remarks: Undulating, regular shear sliding surface evident with smooth contact layer present across entire sliding plane surface	
Performed by: Perry Ross	Date: March 4, 2019
Verified by: J. F. Archibald	Date: March 4, 2019

Direct Shear Strength Tests of Rock Specimens (ASTM D5607-08)

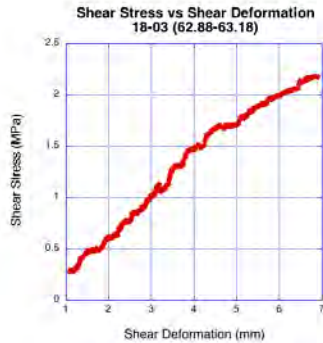
Client: JDS Mining & Energy Inc.	Lab: Queen's University
Project/Site: Coffee Gold Project	Project No.: PO 6448
Borehole/Run: GT 18-03	Sample ID: GT 18-03 (62.88-63.18)
Depth: 62.88-63.18 m	Date Sampled: NA
Lithological Description: Biotite-feldspar schist with carbonate	

Sample Parameters

Specimen	GT 18-03 (62.88-63.18)
Nominal Cross-Sectional Area, cm ²	31.1
Moisture Condition	As-received

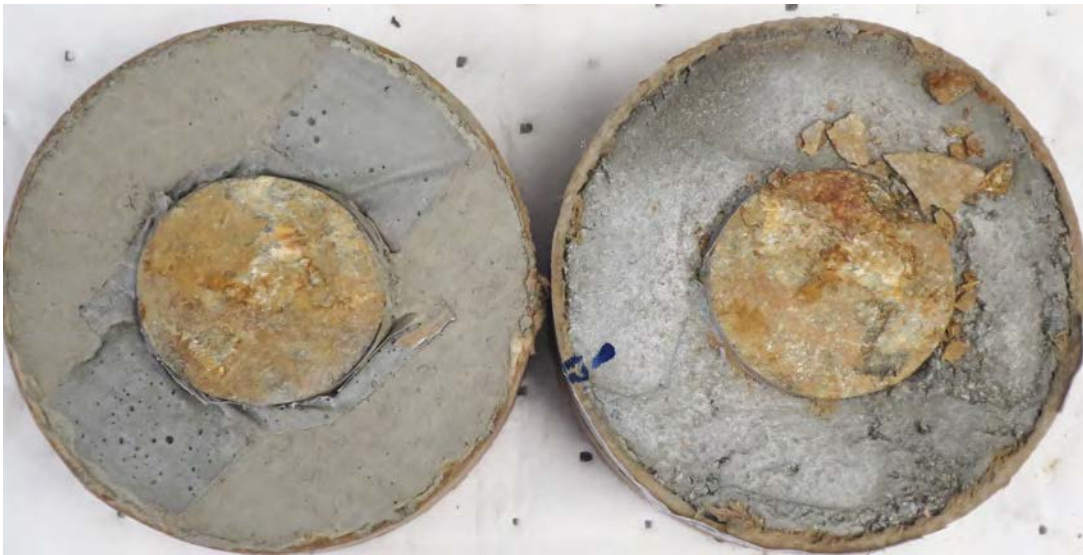
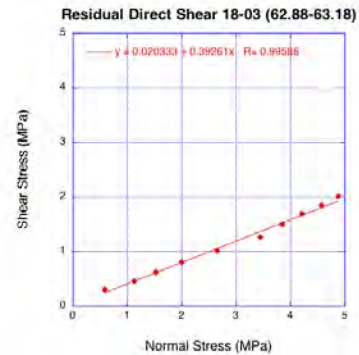
Residual Shear and normal stress data:

Normal Stress, MPa	0.58	1.13	1.53	2.00	2.65	3.45	3.85	4.21	4.57	4.88
Residual Shear Stress, MPa	0.31	0.46	0.63	0.80	1.02	1.26	1.50	1.69	1.85	2.01



Cohesion = 20 kPa

Internal Friction
Angle = 21.3°



Remarks: Regular shear sliding surface evident with smooth, deformable contact layer occurring across sliding plane surface	
Performed by: Perry Ross	Date: March 4, 2019
Verified by: J. F. Archibald	Date: March 4, 2019

Direct Shear Strength Tests of Rock Specimens (ASTM D5607-08)

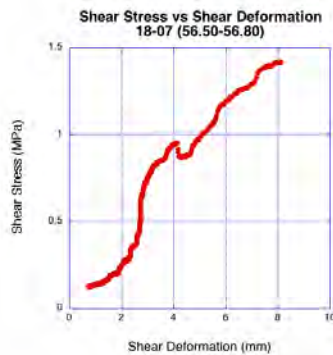
Client: JDS Mining & Energy Inc.	Lab: Queen's University
Project/Site: Coffee Gold Project	Project No.: PO 6448
Borehole/Run: GT 18-07	Sample ID: GT 18-07 (56.50-56.80)
Depth: 56.50-56.80 m	Date Sampled: NA
Lithological Description: Biotite-feldspar schist	

Sample Parameters

Specimen	GT 18-07 (56.50-56.80)
Nominal Cross-Sectional Area, cm ²	48.0
Moisture Condition	As-received

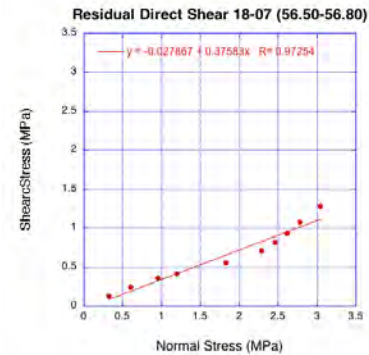
Residual Shear and normal stress data:

Normal Stress, MPa	0.32	0.60	0.95	1.20	1.83	2.29	2.46	2.62	2.78	3.04
Residual Shear Stress, MPa	0.13	0.24	0.36	0.42	0.55	0.71	0.82	0.93	1.08	1.28



Cohesion = 0 kPa

Internal Friction Angle = 20.6°



Remarks: Regular shear sliding surface evident with smooth, deformable contact layer occurring across entire sliding plane surface	
Performed by: Perry Ross	Date: March 4, 2019
Verified by: J. F. Archibald	Date: March 4, 2019

Direct Shear Strength Tests of Rock Specimens (ASTM D5607-08)

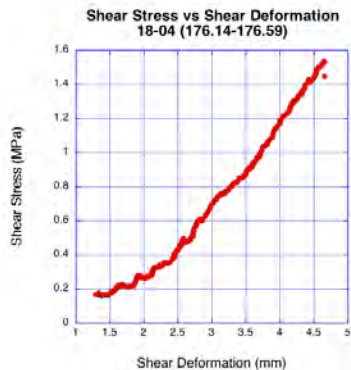
Client: JDS Mining & Energy Inc.	Lab: Queen's University
Project/Site: Coffee Gold Project	Project No.: PO 6448
Borehole/Run: GT 18-04	Sample ID: GT 18-04 (176.14-176.59)
Depth: 176.14-176.59 m	Date Sampled: NA
Lithological Description: Felsic gneiss	

Sample Parameters

Specimen	GT 18-04 (176.14-176.59)
Nominal Cross-Sectional Area, cm ²	47.1
Moisture Condition	As-received

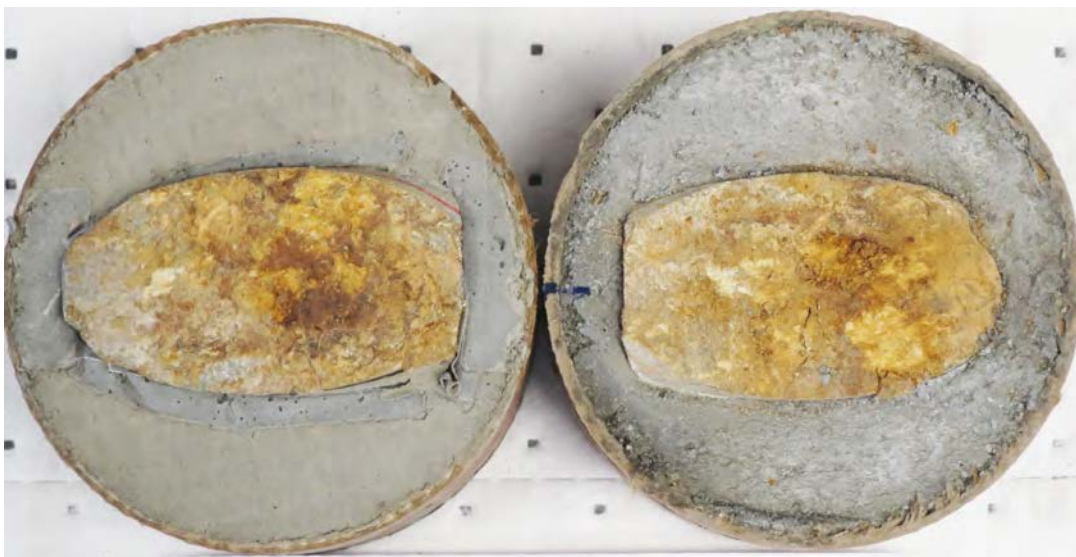
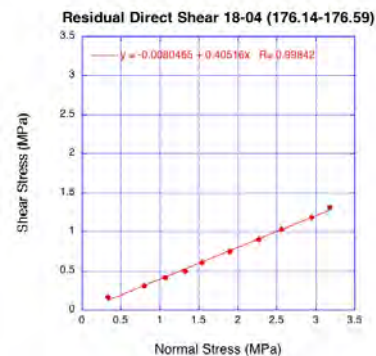
Residual Shear and normal stress data:

Normal Stress, MPa	0.33	0.80	1.07	1.32	1.54	1.90	2.27	2.56	2.95	3.18
Shear Stress, MPa	0.17	0.31	0.42	0.50	0.60	0.75	0.90	1.03	1.19	1.31



Cohesion = 0 kPa

Internal Friction
Angle = 22.0°



Remarks: Regular shear sliding surface evident with smooth, deformable contact layer occurring across entire sliding plane surface	
Performed by: Perry Ross	Date: March 4, 2019
Verified by: J. F. Archibald	Date: March 4, 2019

Direct Shear Strength Tests of Rock Specimens (ASTM D5607-08)

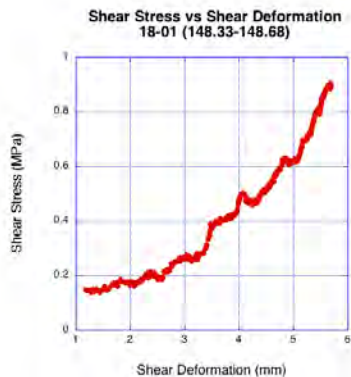
Client: JDS Mining & Energy Inc.	Lab: Queen's University
Project/Site: Coffee Gold Project	Project No.: PO 6448
Borehole/Run: GT 18-01	Sample ID: GT 18-01 (148.33-148.68)
Depth: 148.33-148.68 m	Date Sampled: NA
Lithological Description: Dacite	

Sample Parameters

Specimen	GT 18-01 (148.33-148.68)
Nominal Cross-Sectional Area, cm ²	35.4
Moisture Condition	As-received

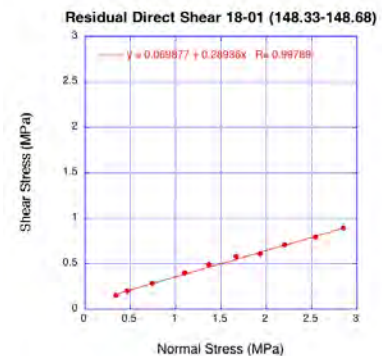
Residual Shear and normal stress data:

Normal Stress, MPa	0.34	0.47	0.74	1.10	1.37	1.67	1.93	2.20	2.54	2.85
Shear Stress, MPa	0.15	0.20	0.28	0.40	0.49	0.58	0.61	0.71	0.79	0.89



Cohesion = 69 kPa

Internal Friction
Angle = 16.1°



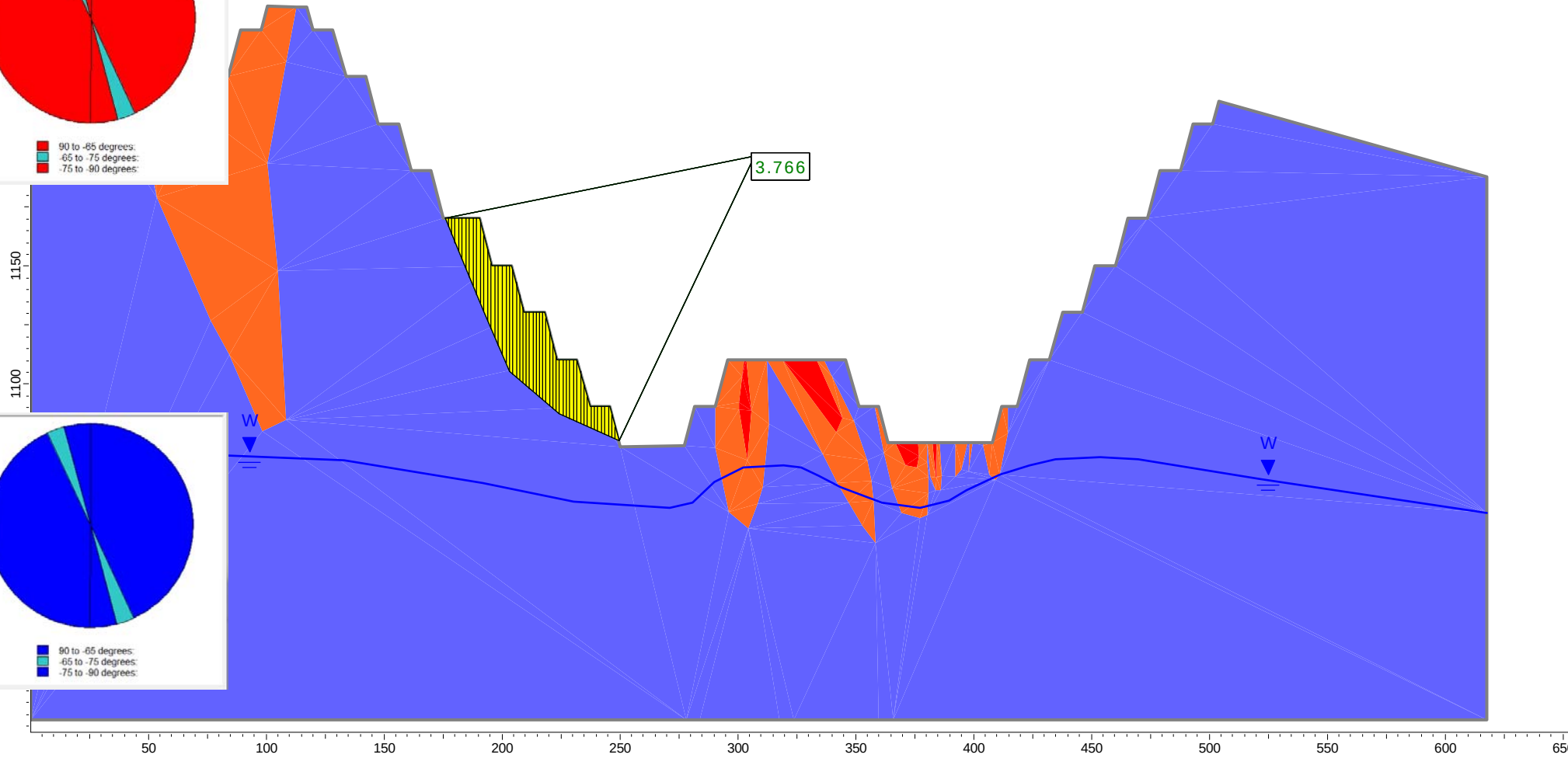
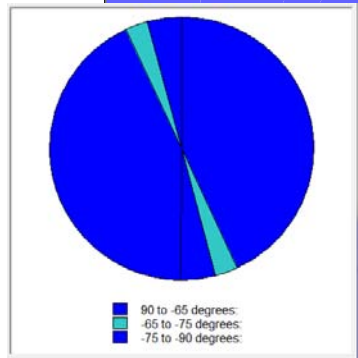
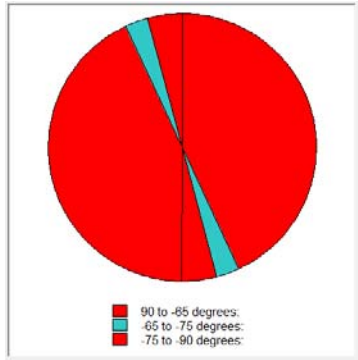
Remarks: Regular shear sliding surface evident with smooth contact layer occurring across sliding plane surface	
Performed by: Perry Ross	Date: March 4, 2019
Verified by: J. F. Archibald	Date: March 4, 2019

Direct Shear Strength Tests of Rock Specimens (ASTM D5607-08)

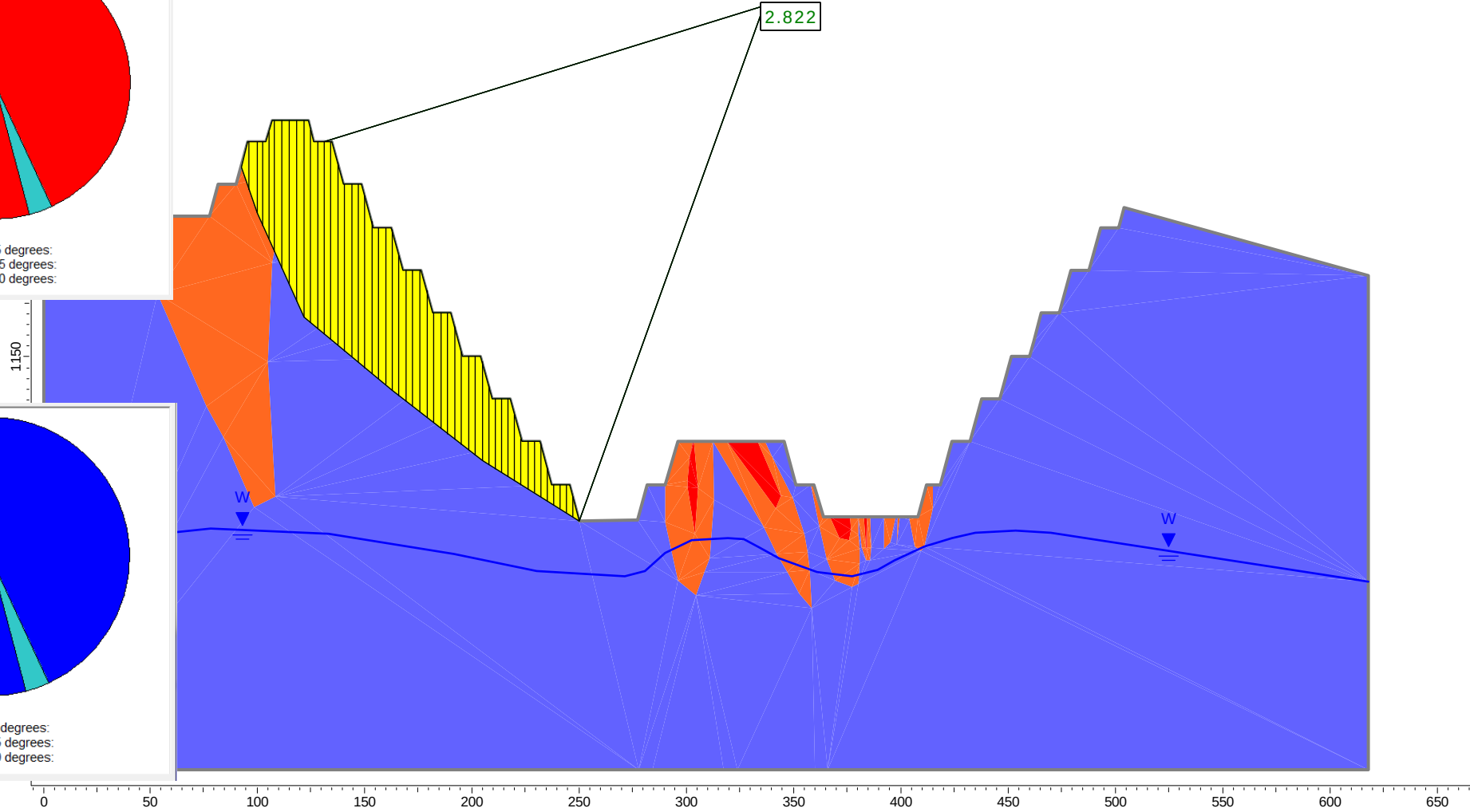
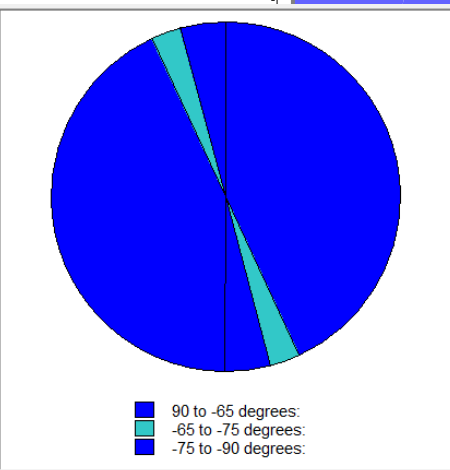
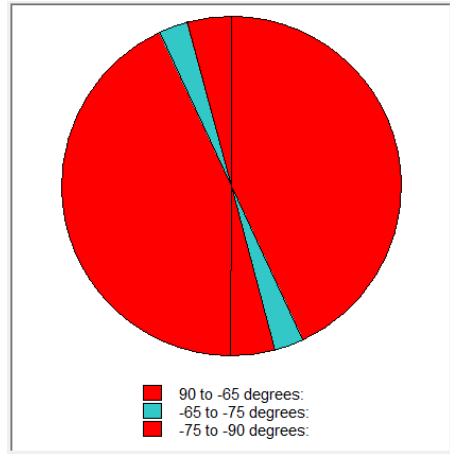
APPENDIX C

INTERRAMP/OVERALL STABILITY ANALYSES

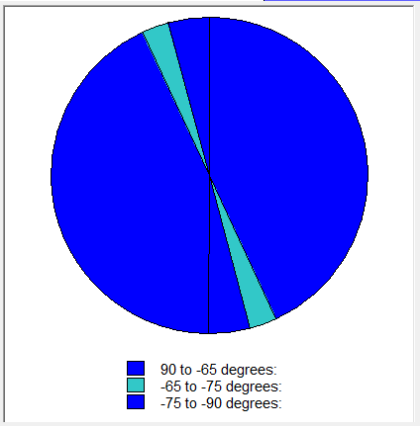
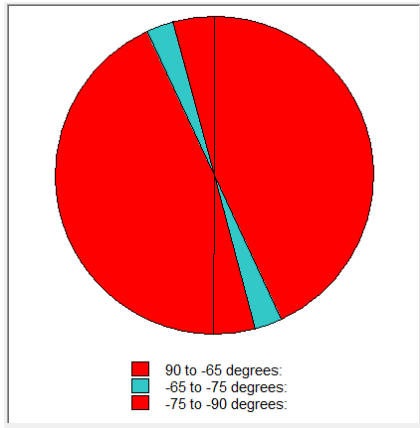
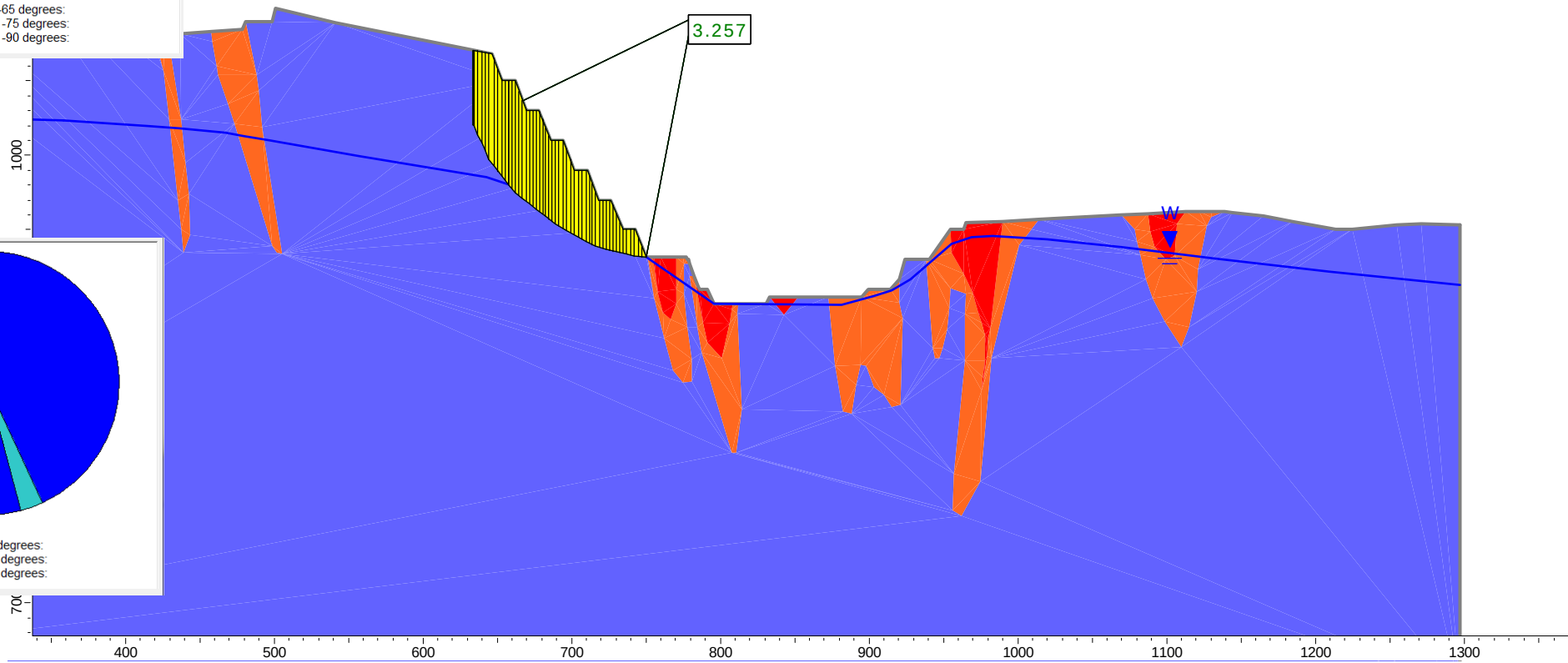
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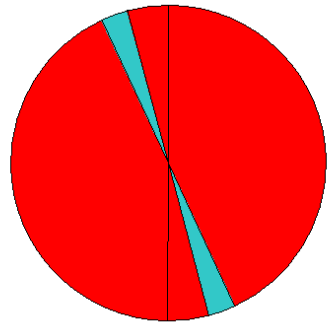
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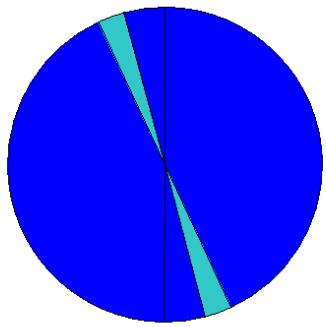
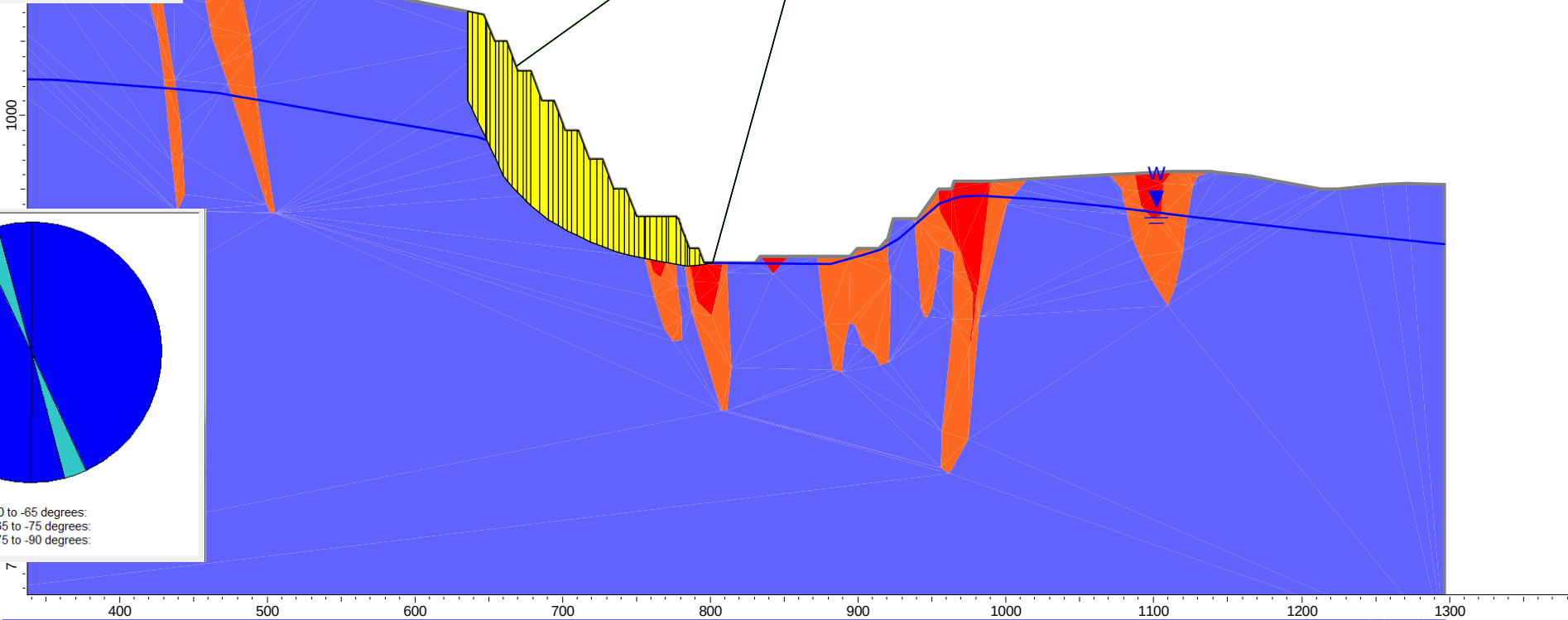
Section B - Interramp Slope



Section B - Overall Slope

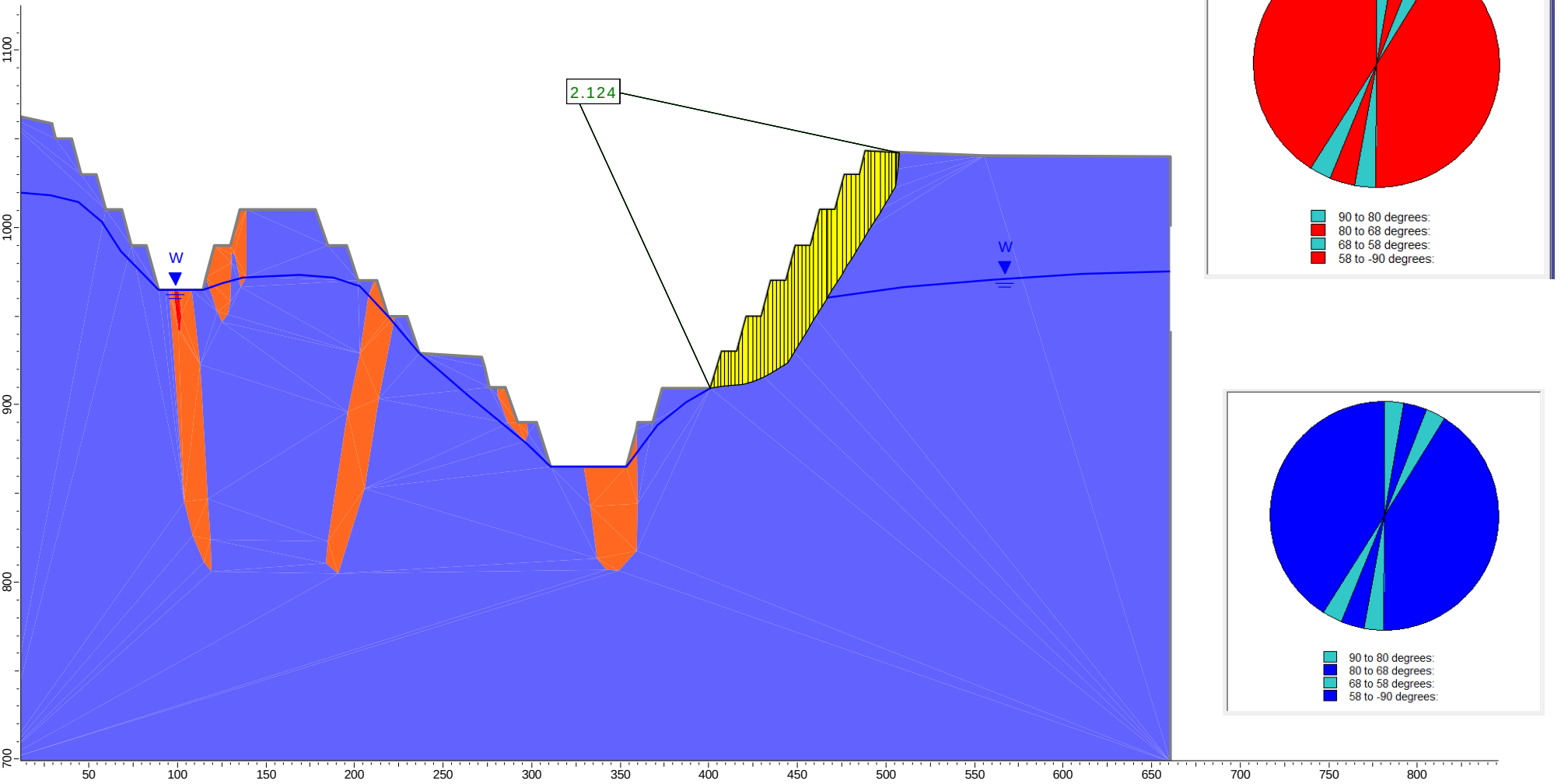


90 to -65 degrees:
-65 to -75 degrees:
-75 to -90 degrees:

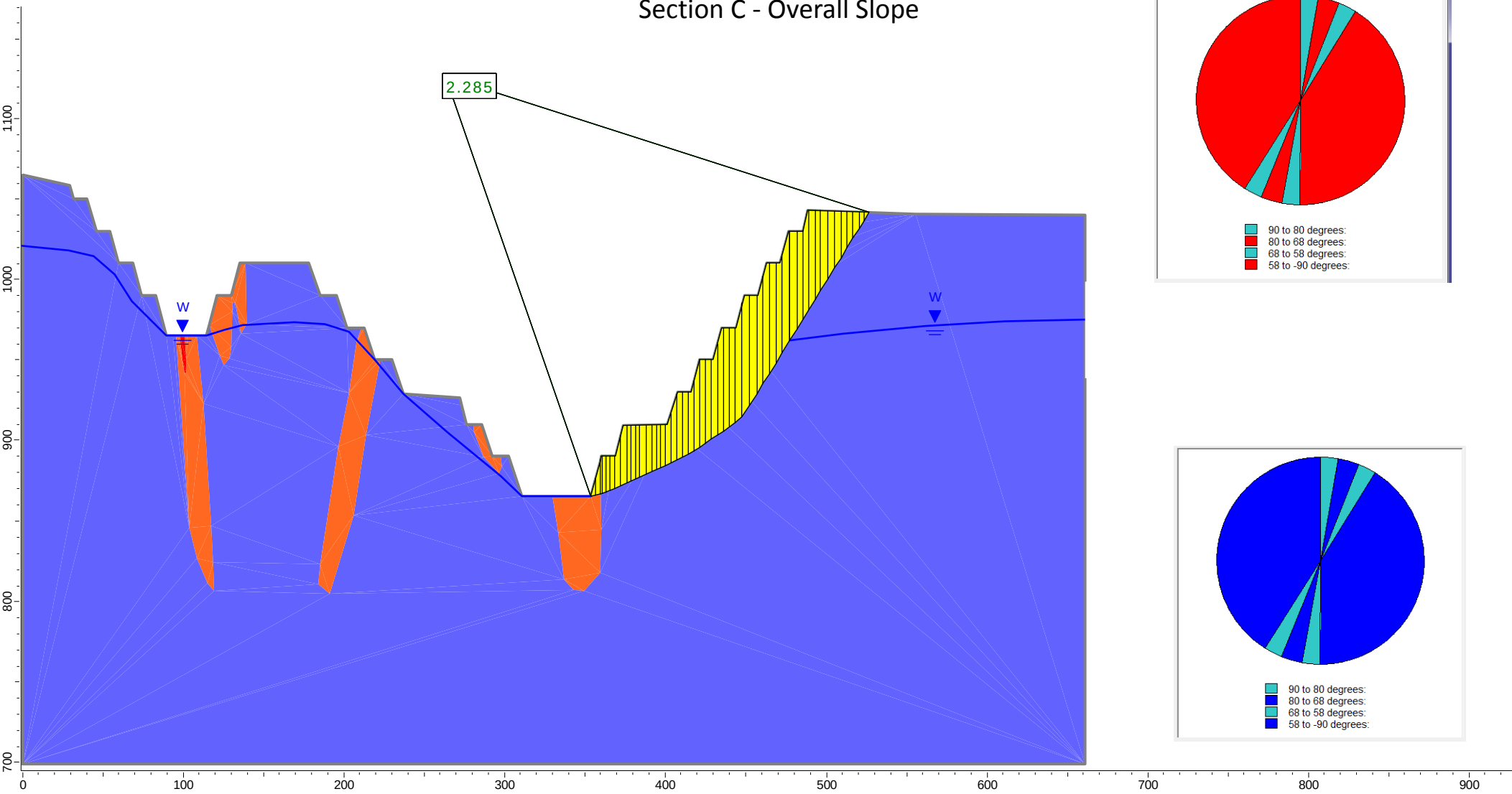


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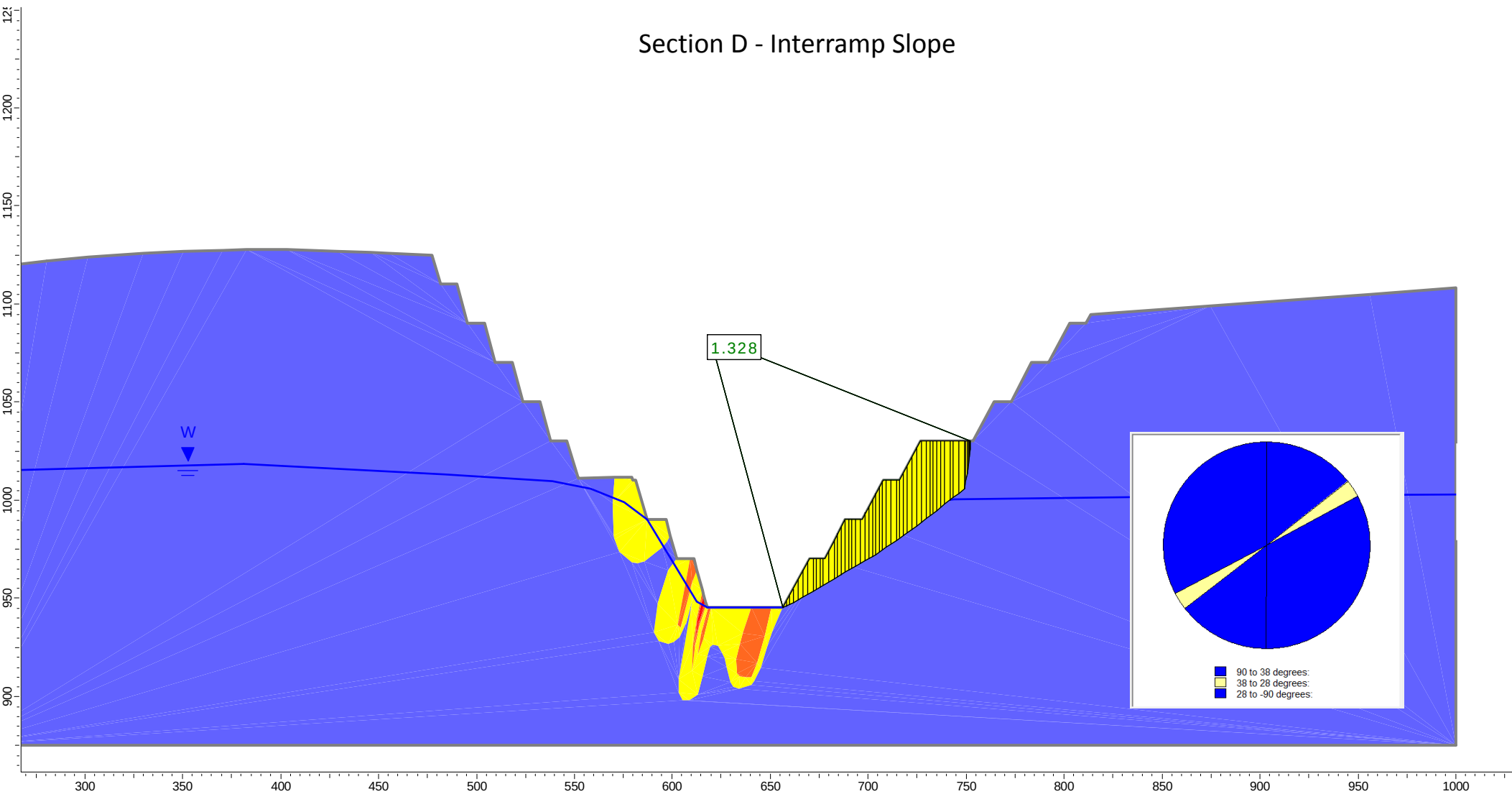
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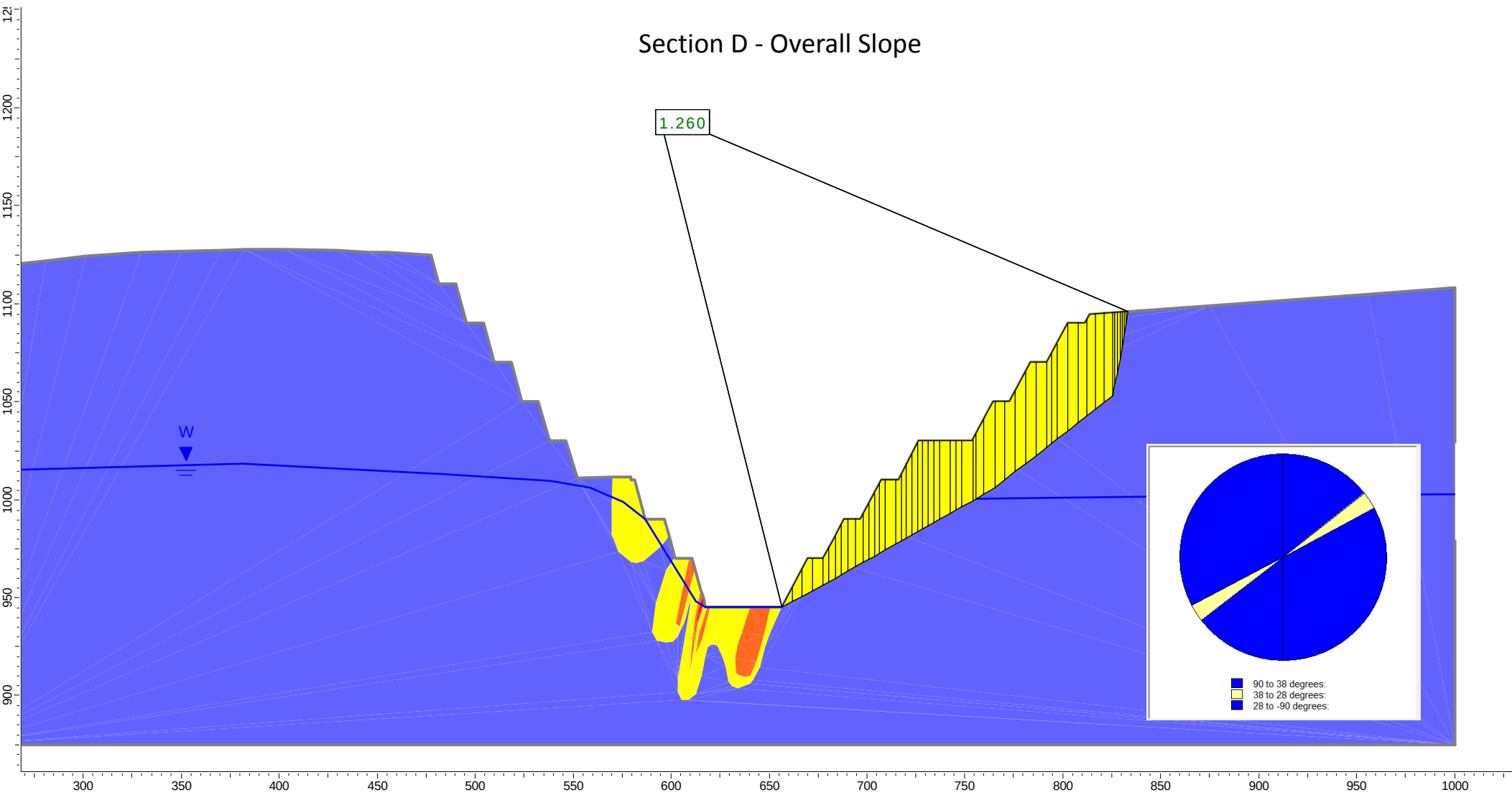
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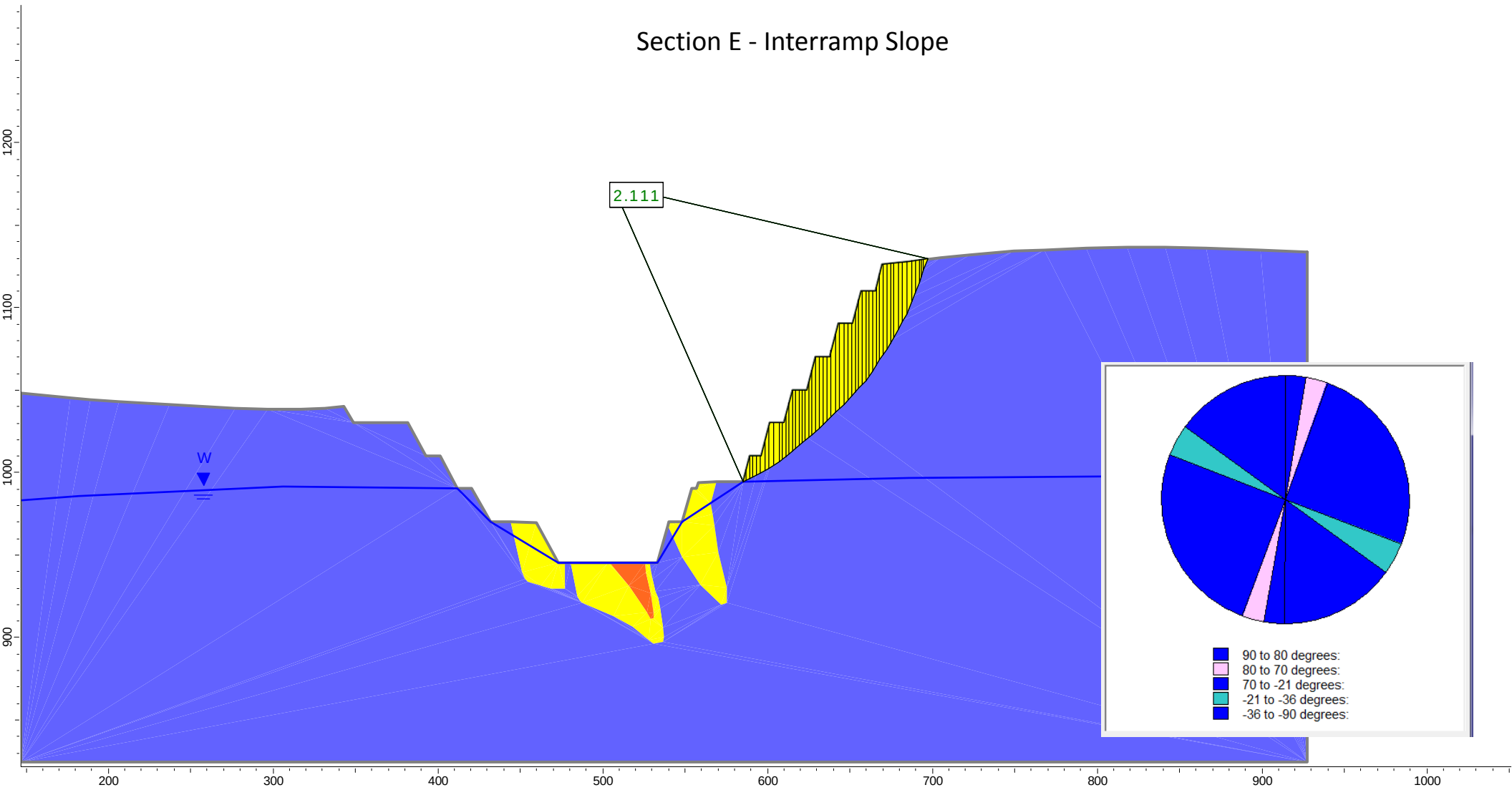
Section D - Interramp Slope



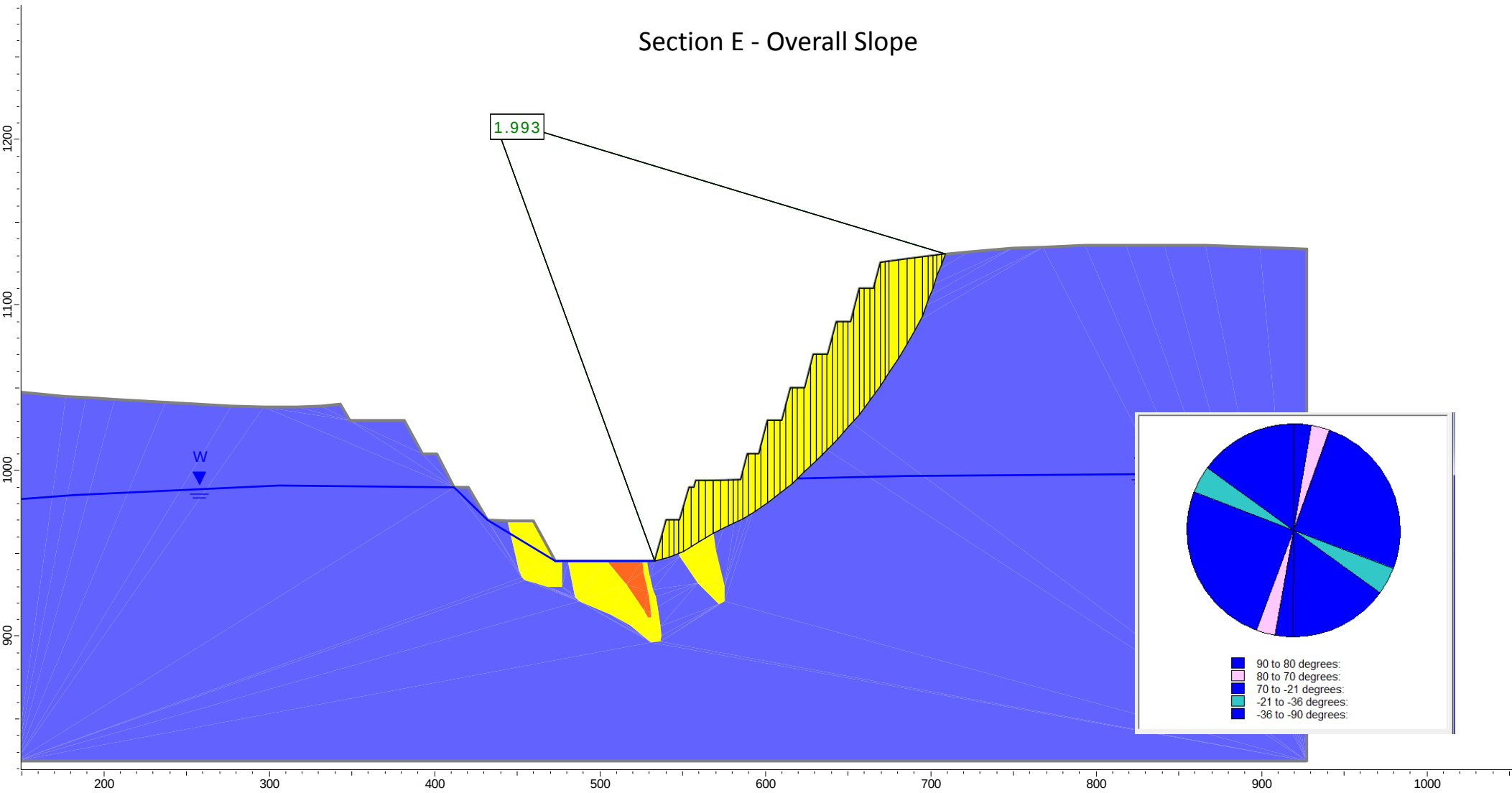
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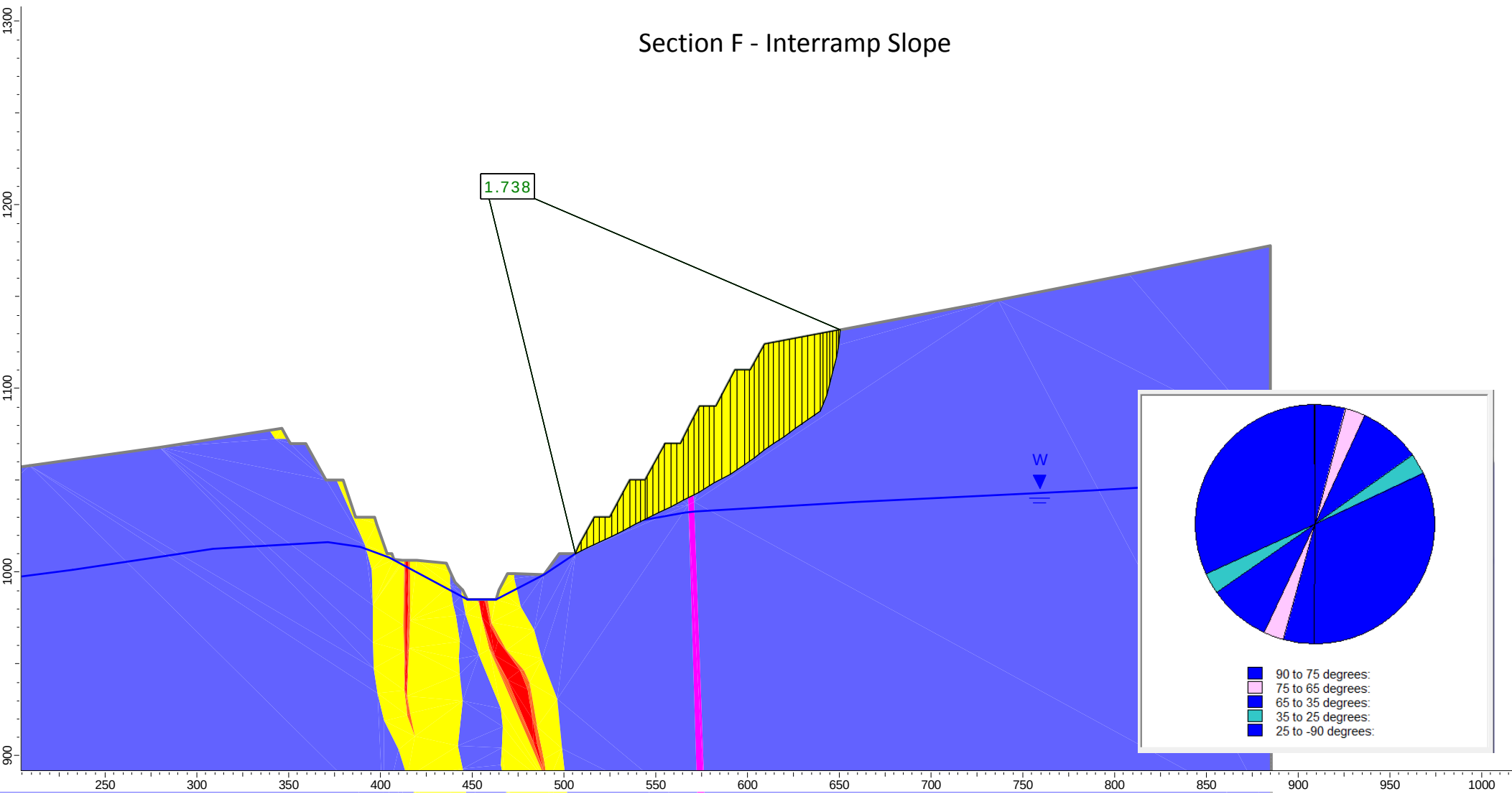
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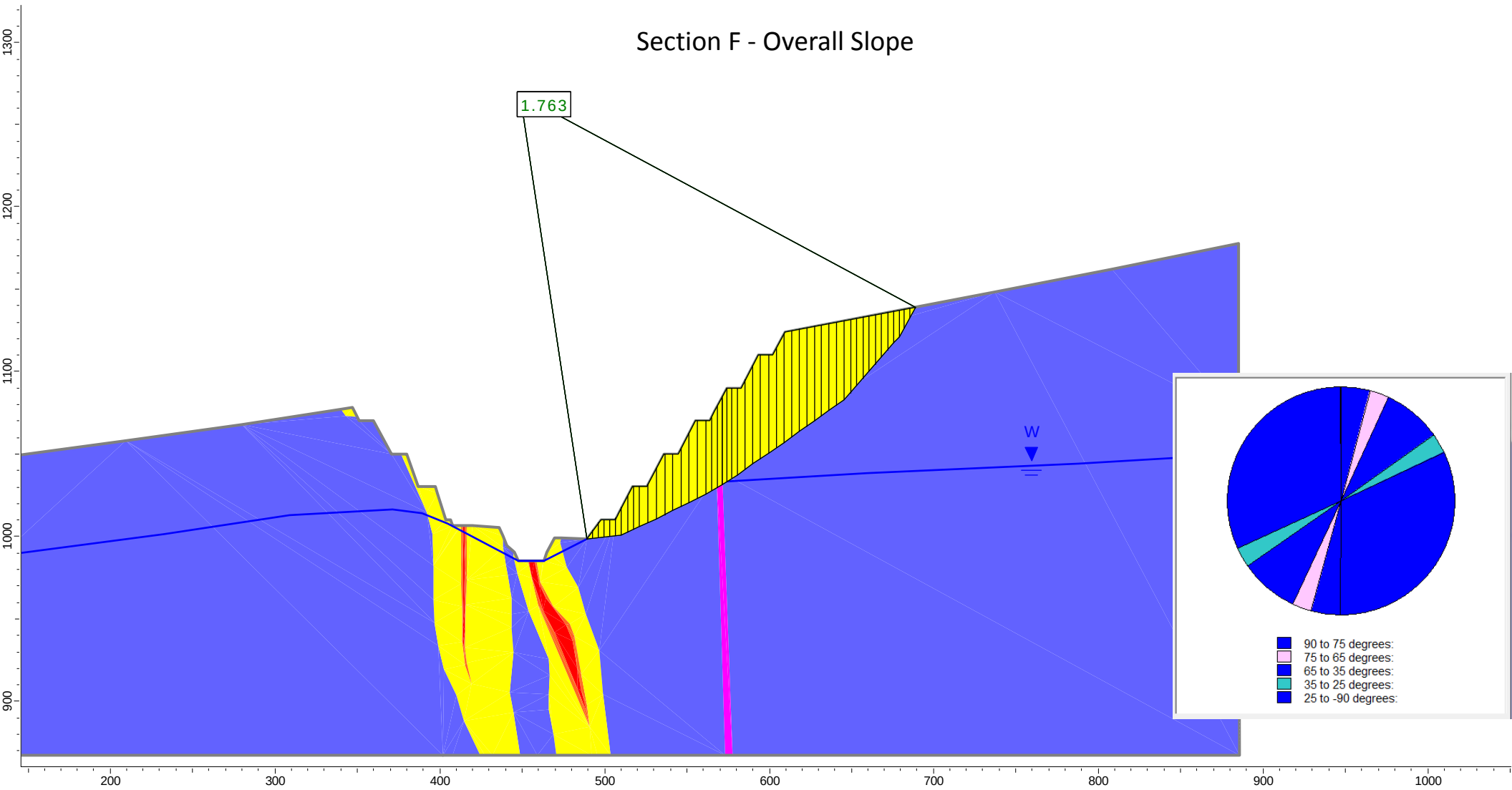
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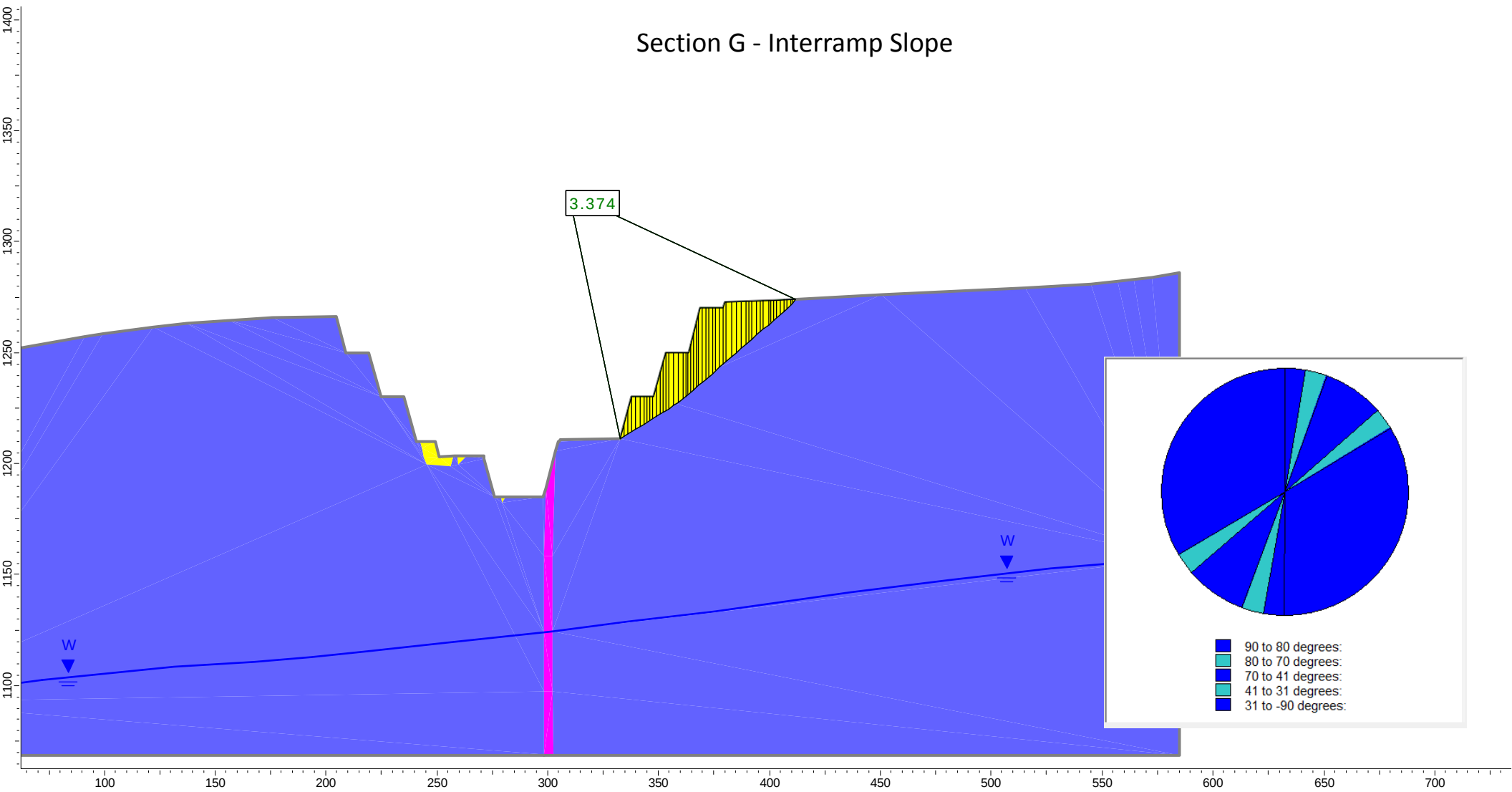
Section F - Interramp Slope



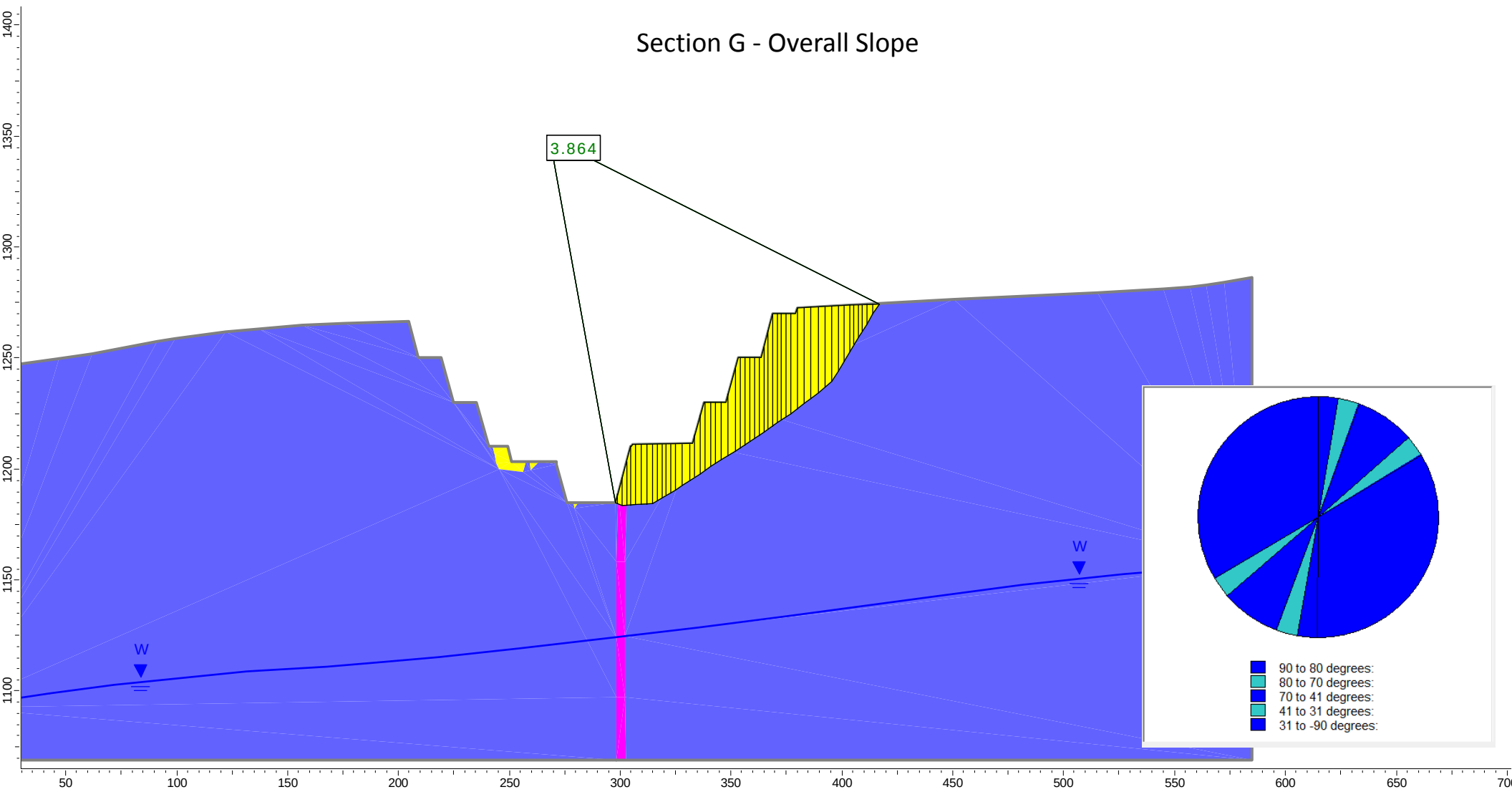
Section F - Overall Slope



Section G - Interramp Slope



Section G - Overall Slope



APPENDIX C

Seismic Evaluation Report

APPENDIX C

..... **SEISMIC EVALUATION REPORT**

Coffee Gold Project Seismic Design Report

Prepared for:

**Kaminak Gold
Vancouver, BC Canada**

Prepared by:



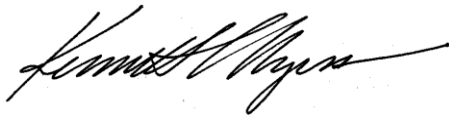
**1325 Airmotive Way, Suite 175u
Reno, Nevada 89502
775-322-7622**

Project No.157801
August 10, 2015

Coffee Gold Project Seismic Design Report

The following documents have been prepared by The MINES Group, Inc. under the professional supervision of the engineer whose seal and signature appear hereon.

The findings are presented within the limits described by the client, and prepared in accordance with generally accepted professional engineering principles and practices. No other warranties either express or implied are made.

A handwritten signature in black ink, appearing to read "Kenneth L. Myers", with a long horizontal flourish extending to the right.

Kenneth L. Myers, P.E.
Principal

Table of Contents

1	EXECUTIVE SUMMARY	1
2	Introduction and Project Description.....	7
3	The Seismic Environment and Historic Data	7
4	Seismic Source Zones.....	15
5	Deterministic Seismic Hazard Analysis (DSHA).....	18
5.1	Tintina North	19
5.2	Inland Alaska.....	23
5.3	Tintina Trench	26
5.4	Denali Fault	30
5.5	Aleutian Megathrust	34
5.6	Fairweather-Queen Charlotte and Transition Fault	40
5.7	DSHA Results Summary	44
6	Probabilistic Seismic Hazard Analysis (PSHA).....	48
7	Seismic Design Criteria.....	51
8	Conclusions and Recommendations	56
9	References	58

List of Tables

Table 1 – Summary of DSHA Results	4
Table 2 – Summary of PSHA Results	6
Table 3 – Summary of Significant Historic Earthquake Events.....	10
Table 4 – Attenuated Sa vs. Natural Period for the Tintina North Seismic Source Zone	21
Table 5 - Attenuated Sa vs. Natural Period for the Tintina Trench Seismic Source Zone	28
Table 6 - Attenuated Sa vs. Natural Period for the Denali Fault Seismic Source Zone	32
Table 7 - Sa vs. Natural Period Extrapolated for the Aleutian Megathrust Seismic Source Zone	39
Table 8 - Attenuated Sa vs. Natural Period for the Fairweather-Queen Charlotte and Transition Fault Seismic Source Zone.....	42
Table 9 – Summary of DSHA Results	45
Table 10 – Summary of PSHA Results	49
Table 11 – Summary of Earthquake Probability Level vs. Radial Site Distance from USGS Model	51
Table 12 – Summary of PSHA Results	57

List of Figures

Figure 1 – Plot of DSHA Results	5
Figure 2 – Plot of PSHA Results	6
Figure 3 – Plot of Significant Earthquake Events in Canada	8
Figure 4 – Relative Seismic Hazard Map of Canada.....	8
Figure 5 – Map of Peak Ground Acceleration (PGA) for Western North America	9
Figure 6 – Plot of Historic Earthquake Events within a 1000 km Radius.....	11
Figure 7 – Historic Earthquake Events Near the Project Site.....	12
Figure 8 – Frequency Distribution for Historic Earthquakes within a 1000 km Radius	13
Figure 9 – Historic Earthquake Magnitude vs. Distance.....	14
Figure 10 – Gutenberg-Richter Relationship for Historic Earthquakes within a 1000 km Radius	14

Figure 11 – Major Faults along the West Coast of North America.....	15
Figure 12 – Major Faults in Southwest Yukon and Southeast Alaska	16
Figure 13 – Significant Seismic Features in Southwest Yukon and Southeast Alaska	16
Figure 14 – Boundaries of Selected Seismic Source Zones	17
Figure 15 – Historic Earthquake Locations in the Tintina North Seismic Source Zone	19
Figure 16 – Frequency Distribution of Magnitude for the Tintina North Seismic Source Zone...	20
Figure 17 - Gutenberg-Richter Relationship for Tintina North Seismic Source Zone	20
Figure 18 - Frequency Distribution of Distance for the Tintina North Seismic Source Zone.....	21
Figure 19 – Attenuated Sa Response Spectra for Tintina North Seismic Source Zone.....	22
Figure 20 - Historic Earthquake Locations in the Inland Alaska Seismic Source Zone	23
Figure 21 - Frequency Distribution of Magnitude for the Inland Alaska Seismic Source Zone ...	24
Figure 22 - Gutenberg-Richter Relationship for Inland Alaska Seismic Source Zone	24
Figure 23 - Frequency Distribution of Distance for the Inland Alaska Seismic Source Zone	25
Figure 24 - Historic Earthquake Locations in the Tintina Trench Seismic Source Zone	26
Figure 25 - Frequency Distribution of Magnitude for the Tintina Trench Seismic Source Zone .	27
Figure 26 - Gutenberg-Richter Relationship for Tintina Trench Seismic Source Zone	27
Figure 27 - Frequency Distribution of Distance for the Tintina Trench Seismic Source Zone.....	28
Figure 28 - Attenuated Sa Response Spectra for Tintina Trench 55 km Seismic Source Zone	29
Figure 29 - Attenuated Sa Response Spectra for Tintina Trench 1 km Seismic Source Zone	29
Figure 30 - Historic Earthquake Locations in the Denali Fault Seismic Source Zone	30
Figure 31 - Frequency Distribution of Magnitude for the Denali Fault Seismic Source Zone	31
Figure 32 - Gutenberg-Richter Relationship for Denali Fault Seismic Source Zone	31
Figure 33 - Frequency Distribution of Distance for the Denali Fault Seismic Source Zone	32
Figure 34 - Attenuated Sa Response Spectra for Denali Fault Seismic Source Zone	33
Figure 35 - Historic Earthquake Locations in the Aleutian Megathrust Seismic Source Zone.....	34
Figure 36 - Frequency Distribution of Magnitude for the Aleutian Megathrust Seismic Source Zone.....	35
Figure 37 - Gutenberg-Richter Relationship for Aleutian Megathrust Seismic Source Zone.....	35
Figure 38 - Frequency Distribution of Distance for the Aleutian Megathrust Seismic Source Zone	36
Figure 39 – PGA vs. Distance for Interface Megathrust Earthquake Events	37
Figure 40 - PGA vs. Distance for Interface Megathrust Earthquake Events with Graphical Extrapolation	37
Figure 41 – Sa 0.2s vs. Distance for Interface Megathrust Earthquake Events with Graphical Extrapolation	38
Figure 42 - Sa 2s vs. Distance for Interface Megathrust Earthquake Events with Graphical Extrapolation	38
Figure 43 - Historic Earthquake Locations in the Fairweather-Queen Charlotte and Transition Fault Seismic Source Zone.....	40
Figure 44 - Frequency Distribution of Magnitude for the Fairweather-Queen Charlotte and Transition Fault Seismic Source Zone	41
Figure 45 - Gutenberg-Richter Relationship for Fairweather-Queen Charlotte and Transition Fault Seismic Source Zone	41
Figure 46 - Frequency Distribution of Distance for the Fairweather-Queen Charlotte and Transition Fault Seismic Source Zone	42
Figure 47 - Attenuated Sa Response Spectra for Fairweather-Queen Charlotte and Transition Fault Seismic Source Zone.....	43
Figure 48 – DSHA Site Specific Sa Response Spectra by Seismic Source Zone	46
Figure 49 - DSHA IBC General Procedure Sa Response Spectra by Seismic Source Zone	46
Figure 50 - DSHA Site Specific Sa Design Response Spectra.....	47
Figure 51 - DSHA IBC General Procedure Sa Design Response Spectra	47

Figure 52 – Plot of PSHA Results and Comparison with DSHA Results	50
Figure 53 – Plot of PSHA Results from EZ-FRISK Model	50
Figure 54 - Plot of Earthquake Probability Level vs. Radial Site Distance from USGS Model ...	51
Figure 55 - Displacement Analysis Results from Seed, 1979 after Pyke, 1999	53
Figure 56 - Pyke’s Recommended Reduction of the Peak Ground Acceleration for Selection of a Seismic Acceleration Coefficient as a Function of Earthquake Magnitude	53
Figure 57 – Plot of PSHA Results	57

1 EXECUTIVE SUMMARY

The Coffee Gold Project is located in west central Yukon Territory, Canada approximately 330 kilometers (km) northwest of Whitehorse. The project will involve the design and construction of a number of facilities associated with the proposed gold mining operation at the site. Facilities may include an open pit, a heap leach pad, various ponds and water storage reservoirs, crushers, pipelines, roads, and various mine and administrative buildings. All planned facilities will need to be designed to resist expected seismic (earthquake) loading.

Western Canada, southeast Alaska, and the Aleutian volcanic arc are among the more seismically active areas in North America. Southwestern Yukon Territory is directly affected by this area of elevated seismicity. The Coffee Gold Project site is located in an area of moderate seismicity known as the Tintina Trench that also holds some of the region's major river systems including the Yukon, Pelly, and Stewart rivers. The northern boundary of the trench is the Tintina fault, a long right lateral strike slip fault responsible both for the formation of the trench and its name. The area to the north of the Tintina fault would seem to be associated with a higher level of historic seismicity than the area within the trench itself. The southern boundary of the trench is the highly active Denali fault, a long right lateral strike slip fault responsible for generating the largest of the earthquakes in close proximity to the site. The area to the southwest of the Denali fault holds some of the most seismically active features in North America, or for that matter, in the world. They include the Fairweather-Queen Charlotte fault and the Aleutian Megathrust zone. The Coffee Gold Project lies very near the geographic center of the trench being approximately 125 km from both the Tintina fault to the northeast and the Denali fault to the southwest.

Most practitioners would suggest that investigations extend for a minimum radius of 150 km from the site. For important projects, a radius of 150 km to 300 km might be more appropriate (the Nuclear Regulatory Commission or NRC requires 200 km). For this project the historic data set was collected for an area within a 1000 km radius. This was done not because damaging impacts are expected from earthquakes originating at a distance of 1000 km, but rather in order to understand more completely the nature of the very complex and highly active sources of seismic energy that begin a mere 125 km southwest of the Coffee Gold Project.

Regional seismic source zones were assigned to a map of historic earthquake events for the specific purpose of evaluating potential seismic loads for the Coffee Gold Project site. We have identified six (6) seismic source zones which (proceeding from northeast to southwest) are as follows:

1. Tintina North Zone
2. Inland Alaska Zone
3. Tintina Trench Zone
4. Denali Fault Zone
5. Aleutian Megathrust Zone
6. Fairweather-Queen Charlotte, Transition Fault Zone

The purpose of identifying discrete seismic source zones is to characterize and quantify the nature of the largest earthquake that is likely to occur within the zone. This information can be utilized in either a deterministic seismic hazard analysis (DSHA) or a probabilistic seismic hazard analysis (PSHA). Although different in approach, they probably have more in common than they have differences. Of interest is the largest earthquake that could reasonably be expected to occur

within the zone, the mean rate of occurrence or recurrence interval, and the location of the earthquake. Seismic source zones come in two (2) varieties:

1. An Aerial Seismic Source where earthquakes are uniformly distributed throughout the area and assumed to be equally likely to occur anywhere within the area.
2. A Linear Seismic Source where earthquakes occur along a narrow linear band (fault), but are again assumed to be equally likely to occur anywhere along the fault line.

For the DSHA approach, the largest earthquake event that could occur within the zone is estimated and then relocated to the position that most closely approaches the site (either along the fault in a narrow linear source zone or within the area boundaries in an aerial source zone). Site specific ground motion parameters are then estimated by attenuating ground motion between the earthquake location and the site. This seems like a very logical “worst case” type of approach, but can sometimes lead to unrealistically severe design criteria. To address this concern, in the PSHA approach the single input parameter of earthquake magnitude is replaced with a cumulative probability distribution relating earthquake magnitude to recurrence rate and an associated exceedance probability. The simple location parameter or distance to the site is replaced by another cumulative probability distribution that addresses the uncertainty associated with where the earthquake event might occur within the source zone (i.e., the variability in the distance to the site). Finally, there is also uncertainty associated with ground motion attenuation and a random component is applied to the ground motion.

The steps involved in a DSHA analysis are as follows:

1. Using information derived from geologic maps, fault maps, and plots of historic earthquake events, identify discrete seismic source zone polygons.
2. Extract “clipped” data sets lying within each seismic source zone that represent the nature of the seismicity within the zone.
3. Estimate the Maximum Credible Earthquake (MCE) associated with each seismic source.
4. Estimate the closest point of approach to the site of interest for each seismic source zone.
5. Estimate site specific ground motions by attenuating motions over the distance between the earthquake epicenter and the site.

Results of analyses for all seismic source zones are summarized in Table 1 and plotted in Figure 1. Most building codes (including the 2010 National Building Code of Canada (NBCC)) allow for either a site specific deterministic design approach or a probabilistic approach. For the site specific DSHA procedure the estimated spectral acceleration values at the various natural periods as shown in Table are used to develop a design spectral acceleration response spectrum. These design accelerations are then used to determine seismic loads for structural design, which will also vary as a function of occupancy and use.

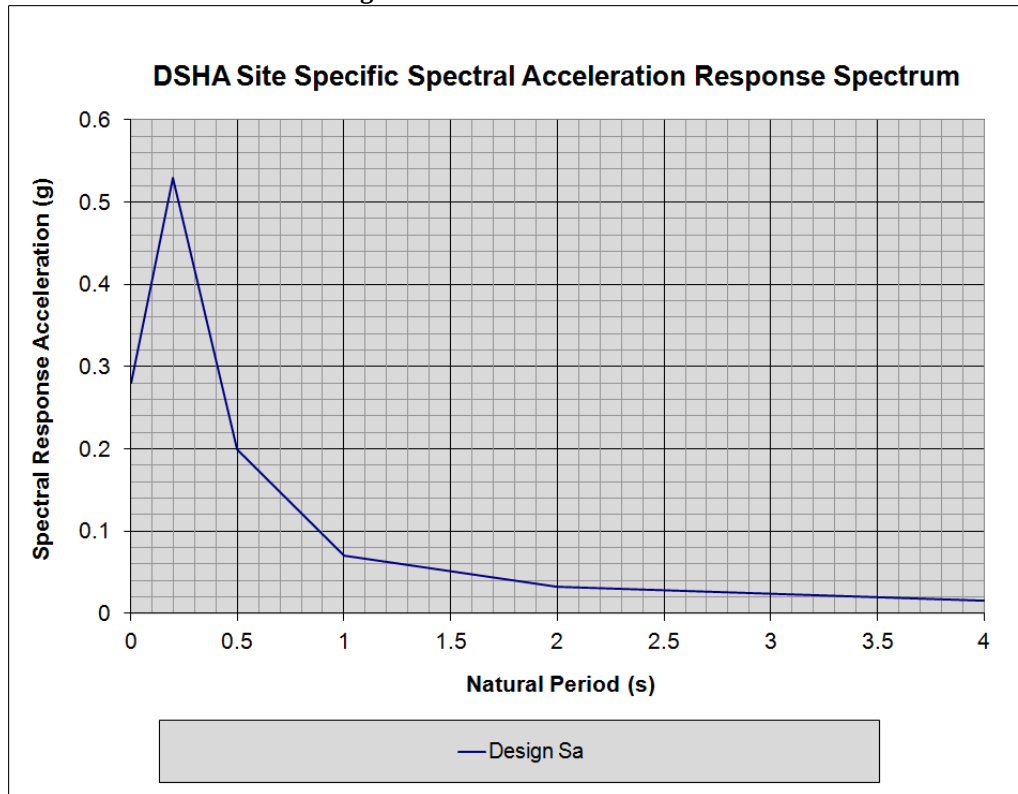
Table 1 – Summary of DSHA Results

Seismic Source Zone	Magnitude				Distance (km)			Pseudo-Spectral Acceleration (g)				
	Max Historic	Gutenberg-Richter	Surface Fault Rupture	MCE	Mean	Median	Closest	PGA	0.2 s	0.5 s	1.0 s	2.0 s
Tintina North	6.9	7.2	---	7.2	547	496	118	.026	0.047	0.038	0.022	0.011
Inland Alaska	7.1	7.1	---	7.1	656	633	420	---	---	---	---	---
Tintina Trench at 55 km	4.6	5.0	---	5.0	550	591	55	0.0085	0.0155	0.0065	0.0002	0.0001
Tintina Trench at 1 km	4.6	5.0	---	5.0	---	---	1	0.28	0.53	0.20	0.070	0.020
Denali Fault	7.9	7.3	8.0	8.0	360	350	105	0.06	0.11	0.085	0.050	0.0025
Aleutian Megathrust	9.2	8.3	---	9.2	662	623	364	0.07*	0.153*	---	---	0.092*
Fairweather-Queen Charlotte and Transition Fault	7.9	8.7	8.0	8.7	509	517	267	0.022**	0.036**	0.084**	0.056**	0.032**

Notes: * - Estimated from 2003 Atkinson and Boore equations for ground-motions in megathrust subduction zones

** - Estimated using M8.5 event at 267 km and the Abrahamson and Silva 2013 NGA West 2 attenuation relationship

Figure 1 – Plot of DSHA Results



The PSHA analysis can be performed using the same seismic source zones and by replacing the maximum credible earthquake with the probability distribution of earthquake events, the site distance with the probability distribution of site distances, and adding a random component to the attenuated spectral acceleration values, then using a Monte Carlo type sampling model to compile a new distribution of spectral accelerations associated with an exceedance probability. However, some developed countries, including the U.S. and Canada, have their own web-based PSHA programs that use regionally based maps of seismic source zones and compute site distances by asking you to enter a specific geographic site location using latitude and longitude.

The Canadian PSHA model is located on the Natural Resources of Canada website and was utilized to generate the PSHA results summarized in Table and plotted in Figure 2. The results assume a Site Class C (stiff soil and/or soft rock in the upper 30 m of the subsurface profile) and are consistent with the observed site conditions at the Coffee Gold Project site.

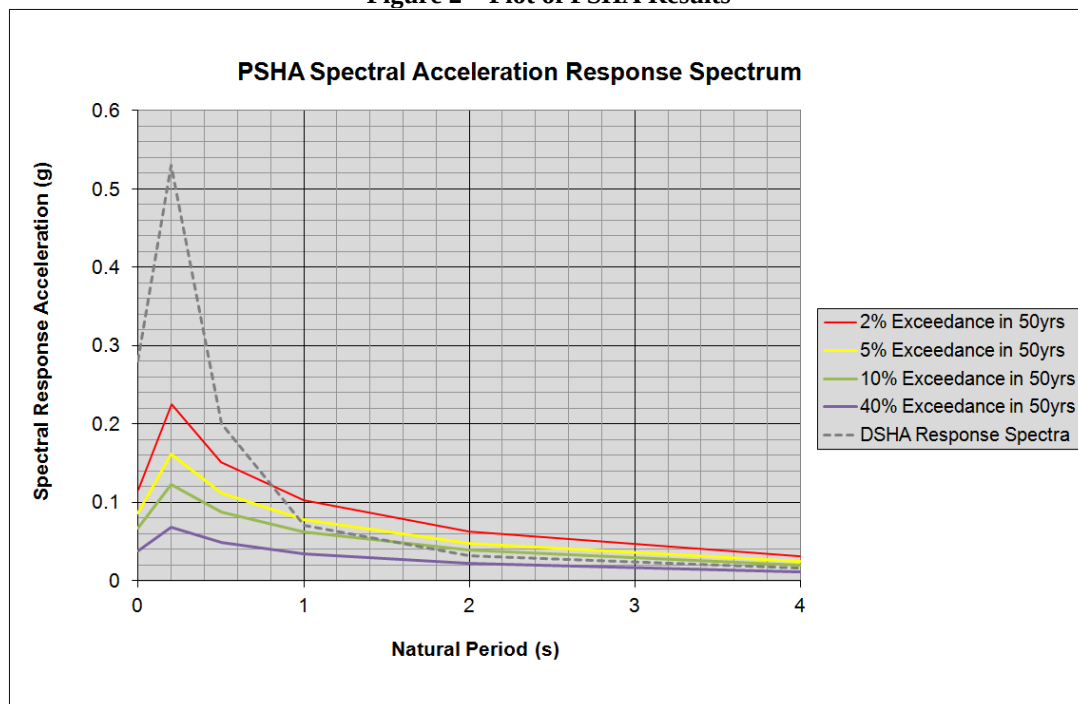
Figure 2 plots the DSHA spectral response spectra (shown as a dashed gray line) on the same plot as the PSHA spectral response spectra in order to permit a comparison of the two methods. As expected the design ground motions from the DSHA analysis are similar but more conservative, at least for natural periods less than 0.8 s. The reduced probability of larger earthquakes occurring in very close proximity to the site moderates the distribution of spectral accelerations resulting in reduced short period Sa values from the PSHA. Beyond the natural period of 0.8 s the design Sa is slightly less than both the 2% and 5% exceedance earthquakes (return periods of 2475 yrs and 1000 yrs respectively), implying that these rare, large earthquakes occurring at larger radial distances have a magnitude higher than the MCEs selected for the DSHA analysis.

Table 2 – Summary of PSHA Results

Annual Exceedance Probability	0.01	0.0021	0.001	0.000404
Recurrence Interval (yrs)	100	475	1000	2475
Probability of Exceedance in 50yrs	40%	10%	5%	2%
Period (s)	Sa (g)	Sa (g)	Sa (g)	Sa (g)
PGA	0.038	0.067	0.086	0.116
0.2	0.068	0.123	0.161	0.225
0.5	0.049	0.087	0.112	0.151
1	0.034	0.062	0.078	0.103
2	0.022	0.039	0.048	0.063
4*	0.011	0.0195	0.024	0.0315

Note: * - Per the 2010 NBCC the 4s Sa = (2s Sa)/2

Figure 2 – Plot of PSHA Results



For the design of buildings and other structures at the Coffee Gold Project site the PSHA results shown in Table and Figure 2 for the earthquake event having a 2% probability of exceedance in 50 yrs coupled with the criteria and design procedures for estimating seismic loads found in the 2010 NBCC should be applied for structural design.

Seismic design in connection with earthen geotechnical structures (earth dams, embankments, excavations, etc.) is usually associated with slope stability analysis. Current practice in slope design incorporates a hierarchy of slope analysis procedures that progress from simple and conservative, to more rigorous but less conservative. The first level in the hierarchy is the pseudostatic analysis. For the Coffee Gold Project site, the recommended pseudostatic acceleration coefficient for use in pseudostatic stability analysis is 0.06g. The second level in the hierarchy of seismic stability analysis is displacement analysis indicating acceptably small displacements. For critical structures,

the last level in the hierarchy of seismic stability analysis tools is a full dynamic analysis, again indicating acceptably small displacements.

2 Introduction and Project Description

The Coffee Gold Project is located in west central Yukon Territory, Canada approximately 330 kilometers (km) northwest of Whitehorse. The project will involve the design and construction of a number of facilities associated with the proposed gold mining operation at the site. Facilities may include an open pit, a heap leach pad, various ponds and water storage reservoirs, crushers, pipelines, roads, and various mine and administrative buildings. All planned facilities will need to be designed to resist expected seismic (earthquake) loading.

Seismic events or earthquakes have long been known to be associated strongly with tectonic movements of large masses of the earth's crust or *plates*. Earthquakes can and do occur virtually anywhere in the earth's crust, but tend to concentrate, both in space and time, near the boundaries of the major plates. They can be associated with magmatic movement and volcanism, with areas of crustal thinning or spreading centers (rift zones), or even from elastic crustal rebound and isostatic uplift following the removal of a great thickness of glacial ice. However, the largest earthquakes and the greatest frequency of earthquakes seem to be associated with movement along plate boundaries and *subduction* zones where edge of one plate is sinking beneath the edge of an adjacent plate that is overriding it. Most earthquakes are directly associated with the breaking or rupture of the crust along a fault line. The size or *magnitude* of the earthquake is a function of a number of factors including the type of rock present, the dip angle of the subduction zone, the depth of the rupture, the creep rate and magnitude of the stress drop during a rupture, the areal extent of the rupture, and so on. This report will describe the seismic environment surrounding the site, identify seismic source zones (the origins of earthquakes), characterize the nature of the seismic hazard, and propose appropriate seismic design criteria.

3 The Seismic Environment and Historic Data

Western Canada, southeast Alaska, and the Aleutian volcanic arc are among the more seismically active areas in North America as seen on Figure 3 through Figure 5. Southwestern Yukon Territory is directly affected by this area of elevated seismicity.

Figure 3 – Plot of Significant Earthquake Events in Canada

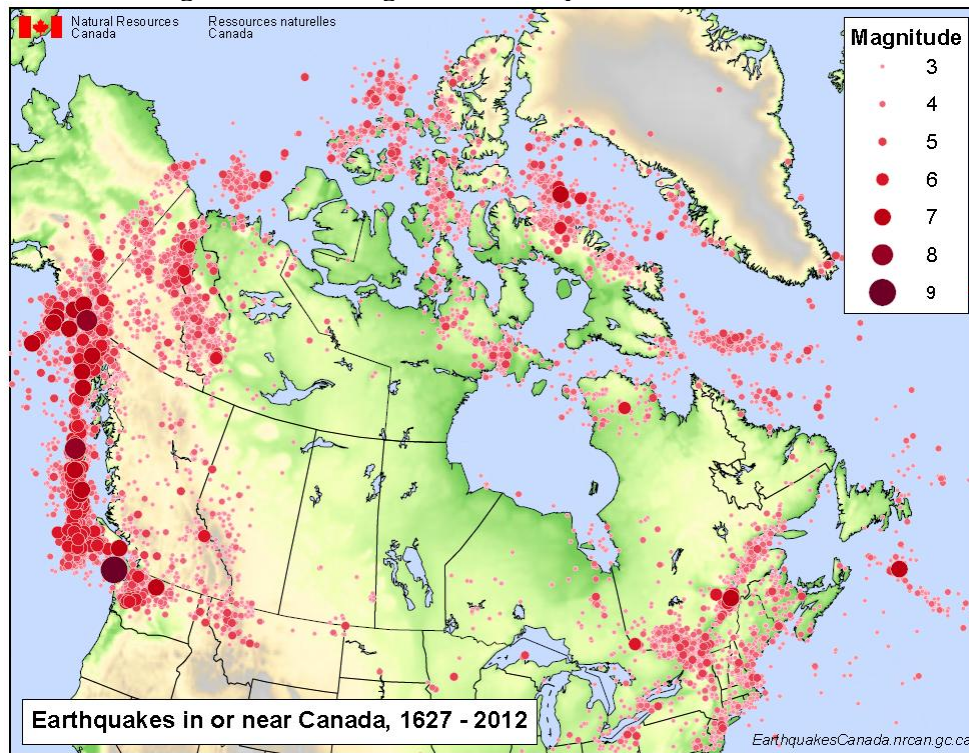


Figure 4 – Relative Seismic Hazard Map of Canada

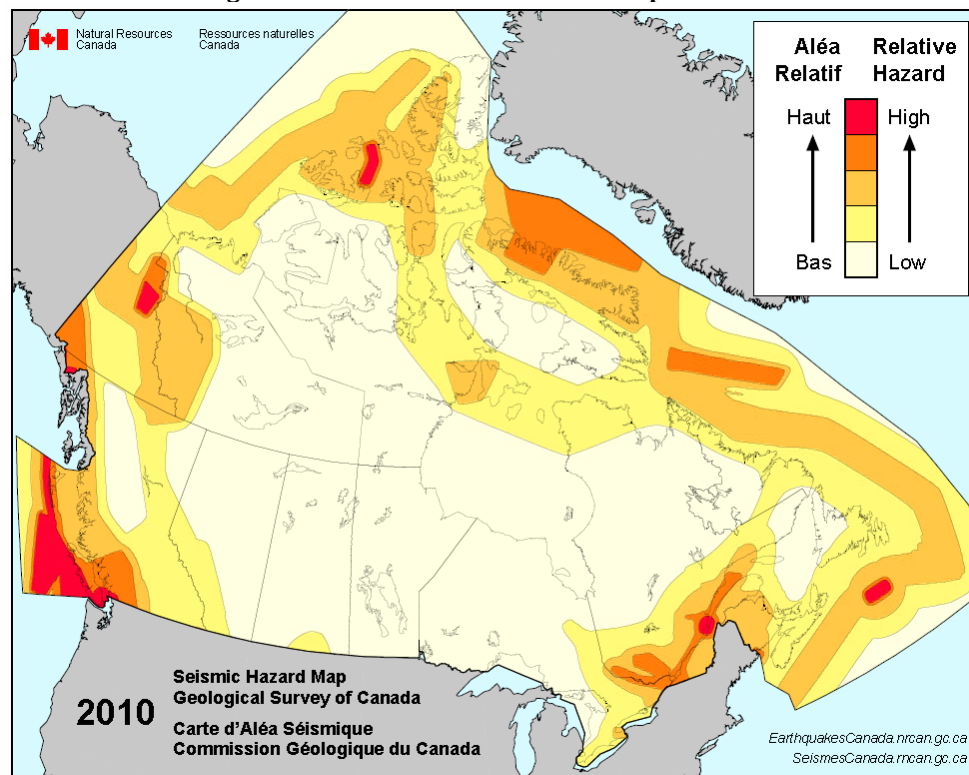
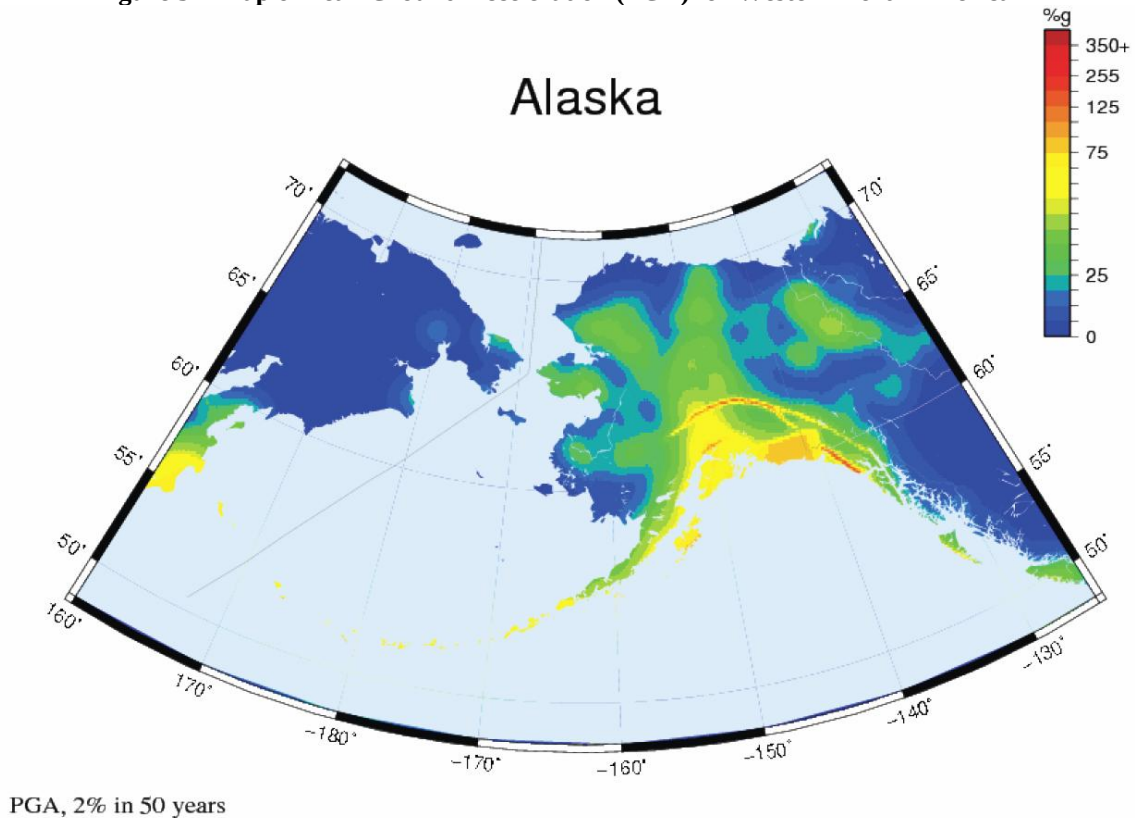


Figure 5 – Map of Peak Ground Acceleration (PGA) for Western North America



The Coffee Gold Project site is located in an area of moderate seismicity known as the Tintina Trench that also holds some of the region's major river systems including the Yukon, Pelly, and Stewart rivers. The northern boundary of the trench is the Tintina fault, a long right lateral strike slip fault responsible both for the formation of the trench and its name. The area to the north of the Tintina fault would seem to be associated with a higher level of historic seismicity than the area within the trench itself. The southern boundary of the trench is the highly active Denali fault, a long right lateral strike slip fault responsible for generating the largest of the earthquakes in close proximity to the site. The area to the southwest of the Denali fault holds some of the most seismically active features in North America, or for that matter, in the world. They include the Fairweather-Queen Charlotte fault and the Aleutian Megathrust zone. The Coffee Gold Project lies very near the geographic center of the trench being approximately 125 km from both the Tintina fault to the northeast and the Denali fault to the southwest (see Figure 7).

When evaluating the potential impact of seismic activity at a specific site it is not necessary to investigate events at great distances from the site of interest, as the damaging effects of earthquake ground motion dissipate rapidly with distance. The response of structures to vibratory loading is a function of harmonics and the *natural period* of the structure. Short, stiff structures tend to respond more to high frequency vibrations while taller more flexible structures tend to respond more to low frequency vibrations. Earthquake events produce a mixture of both high and low frequency vibration. However, the high frequency end of the spectrum tends to attenuate rather quickly over short distances while the low frequencies persist longer over much larger distances. This means that a structure with a low frequency natural period may still experience damaging motion at a distance of 100 km from the earthquake while structures with a high natural period may exhibit little or no response.

Most practitioners would suggest that investigations extend for a minimum radius of 150 km from the site. For important projects, a radius of 150 km to 300 km might be more appropriate (the Nuclear Regulatory Commission or NRC requires 200 km). For this project the historic data set was collected for an area within a 1000 km radius. This was done not because damaging impacts are expected from earthquakes originating at a distance of 1000 km, but rather in order to understand more completely the nature of the very complex and highly active sources of seismic energy that begin a mere 125 km southwest of the Coffee Gold Project.

Figure 6 shows a plot of the historic recorded earthquake events within a 1000 km radius of the Coffee Gold Project site. This data set was created by downloading the events from the Natural Resources Canada (NRC) database and combining those events with another set of events from the United States Geological Survey (USGS) database. The combined dataset was then screened for duplicates and the resulting filtered dataset is the final dataset displayed in Figure 6. For purposes of this report, the Coffee Gold Project site is assumed to be located as follows:

Lat-Long:

62.8745° Latitude, -139.1819° Longitude

Universal Transverse Mercator (UTM) Projection, Zone 7N:

UTM Easting 592,477 m, UTM Northing 6,972,913 m

A more detailed plot of the location of historic earthquake events overlaid on a topographic map of the region surrounding the site is shown on Figure 7. The increase in both magnitude and frequency of earthquake events in the area to the southwest of the project is evident in this figure.

Using the full 1000 km radius dataset, the event specific information in Table 3 was extracted. Figure 8 shows the frequency distribution of earthquakes for the full data set.

Table 3 – Summary of Significant Historic Earthquake Events

	Magnitude	Epicenter Distance (km)	Year	Month	Day
Largest Earthquake:	9.3	481	1964	3	28
Closest Earthquake:	4.5	55	1993	12	7
Closest Earthquake with Magn >= 5:	5	154	2002	11	8
Closest Earthquake with Magn >= 6:	7.1	279	1979	2	28
Closest Earthquake with Magn >= 7:	7.1	279	1979	2	28
Closest Earthquake with Magn >= 8:	9.3	481	1964	3	28
Closest Earthquake with Magn >= 9:	9.3	481	1964	3	28

It should be noted that the combined NRC/USGS dataset contains a variety of different methods for the measurement of magnitude including surface wave, body wave, the so called Richter magnitude, and moment magnitude. The current standard is moment magnitude which requires a map of hypocenters of rupture locations (defining rupture length and width) along with an estimate of the average slip. This kind of information was simply not available in the early days of data collection on earthquakes. All of the early methods of magnitude measurement have limitations, i.e. they “saturate” at magnitudes somewhere around 8.0 to 8.5 (meaning they would record events larger than magnitude 8.5 as a magnitude 8.0 to 8.5 event). Most of the more recent events in the dataset would report a moment magnitude (Mo).

Figure 9 shows a plot of earthquake magnitude vs. distance for the full dataset and the number of events within each magnitude range. The envelope line (the yellow line at the left of the plot) shows a very rough estimate of the earthquake magnitude that could be expected to occur at the

location of the site (approximately magnitude 4). Figure 10 is a plot of the Gutenberg-Richter relationship for the full 1000 km radius dataset. The relationship predicts the maximum magnitude earthquake in the region to be approximately M8.8. The plot also reveals some of the problems and limitations of the Gutenberg-Richter relationship. The curve “tails off” at both the top or high magnitude end of the plot (the result of a limited record length that understates the number of large, infrequent events) and at the bottom (the result of widely spaced instrumentation, especially in remote regions, that cannot record the smaller magnitude events).

Ground motion response to earthquakes depends not just on the character of the earthquake, but also the character of the subsurface conditions at the site. Of concern is the nature of the soil/rock in the upper 30 m of soil/rock profile. Test pits and drilling at the site indicate that the soil cover is relatively thin and the underlying rock moderately to highly weathered. Therefore, for the purpose of this investigation, the site was assigned to Site Class C (stiff soil or soft rock) with an assumed representative shear velocity (V_{s30}) of 360 m/s to 760 m/s consistent with the recommendations in the 2010 National Building Code of Canada.

Although interesting and informative, evaluations that treat the entire regional dataset as a uniformly distributed source of earthquakes and seismic energy are inappropriate for the assignment of seismic design criteria to a specific site. In the next section we will examine the 1000 km radius dataset in more detail and attempt to identify specific, individual seismic sources.

Figure 6 – Plot of Historic Earthquake Events within a 1000 km Radius

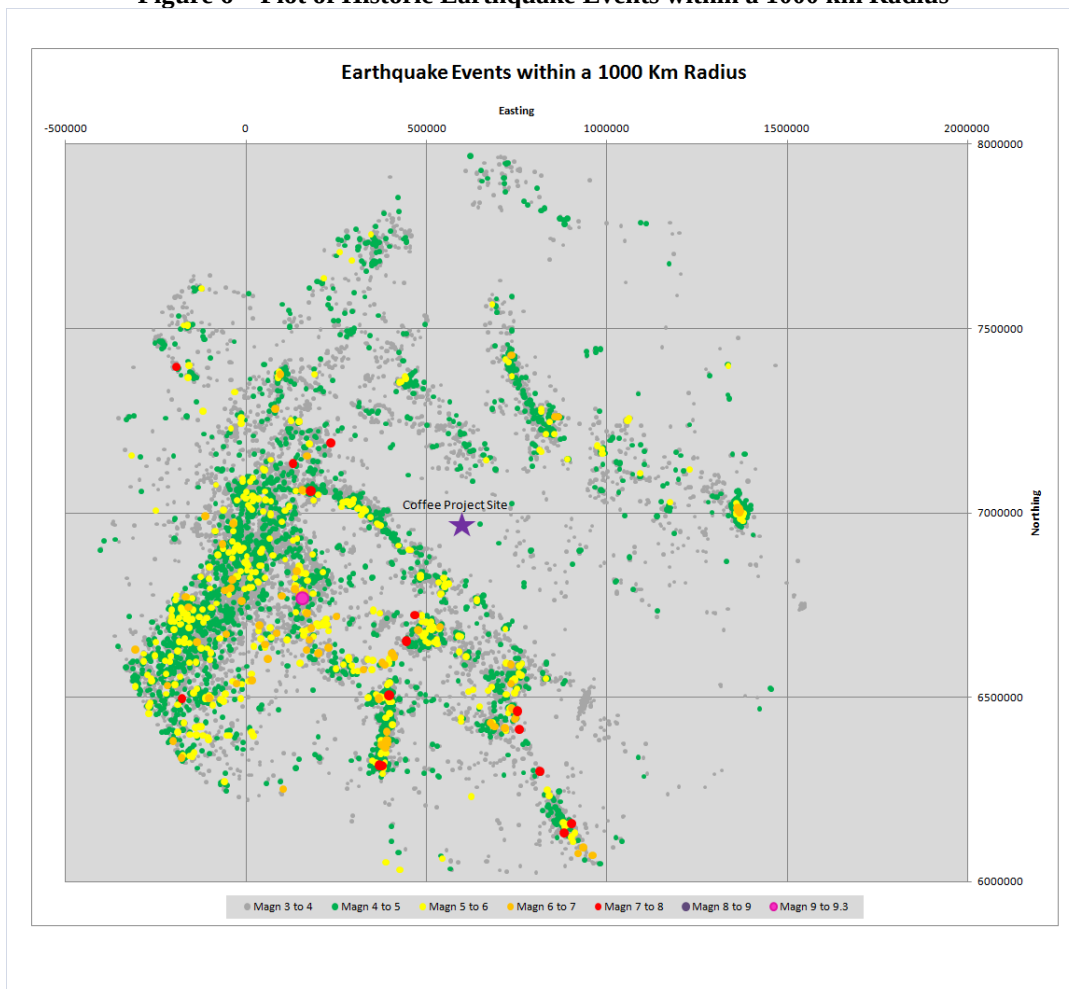


Figure 7 – Historic Earthquake Events Near the Project Site
Regional Site Seismicity Map

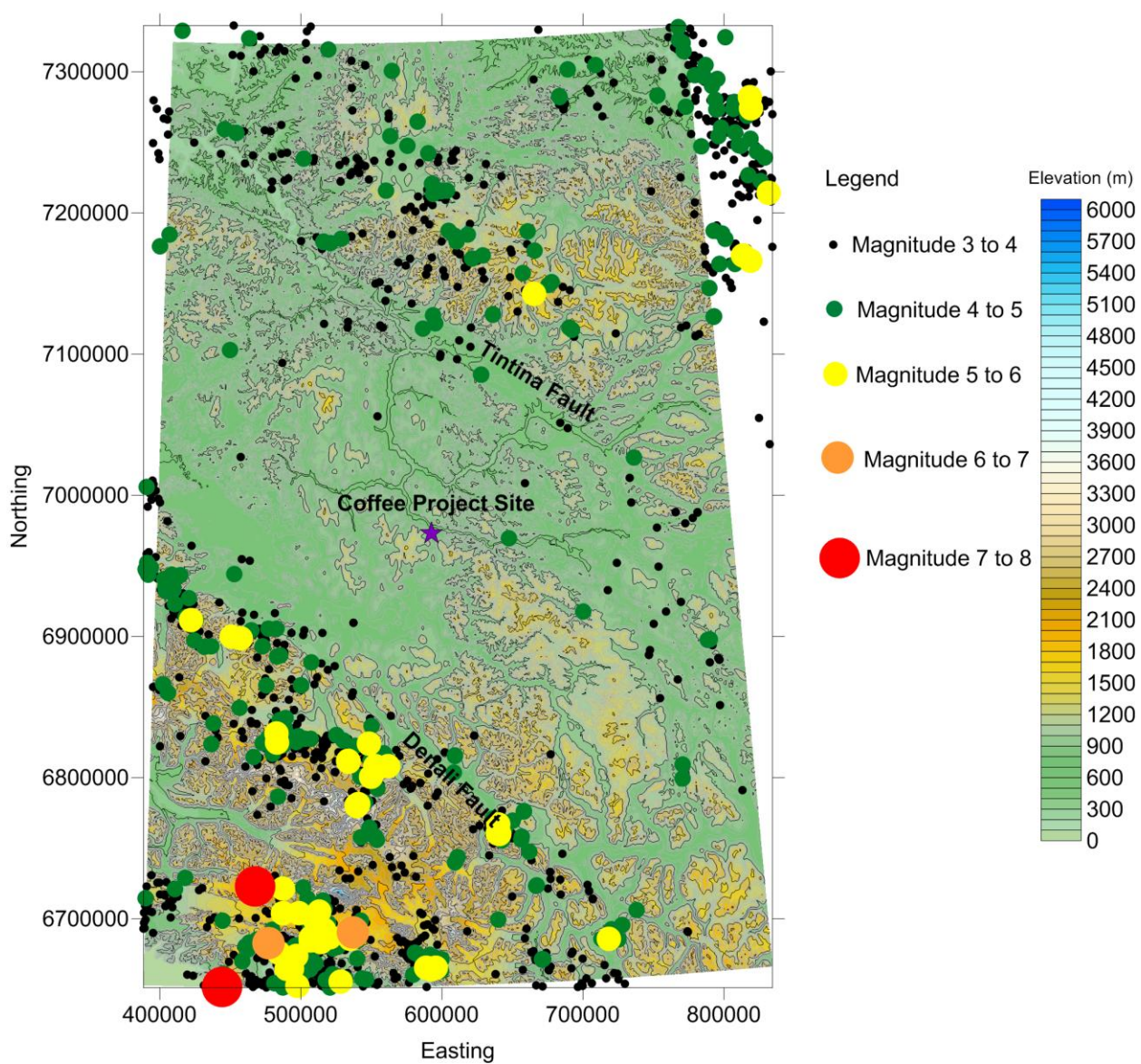


Figure 8 – Frequency Distribution for Historic Earthquakes within a 1000 km Radius

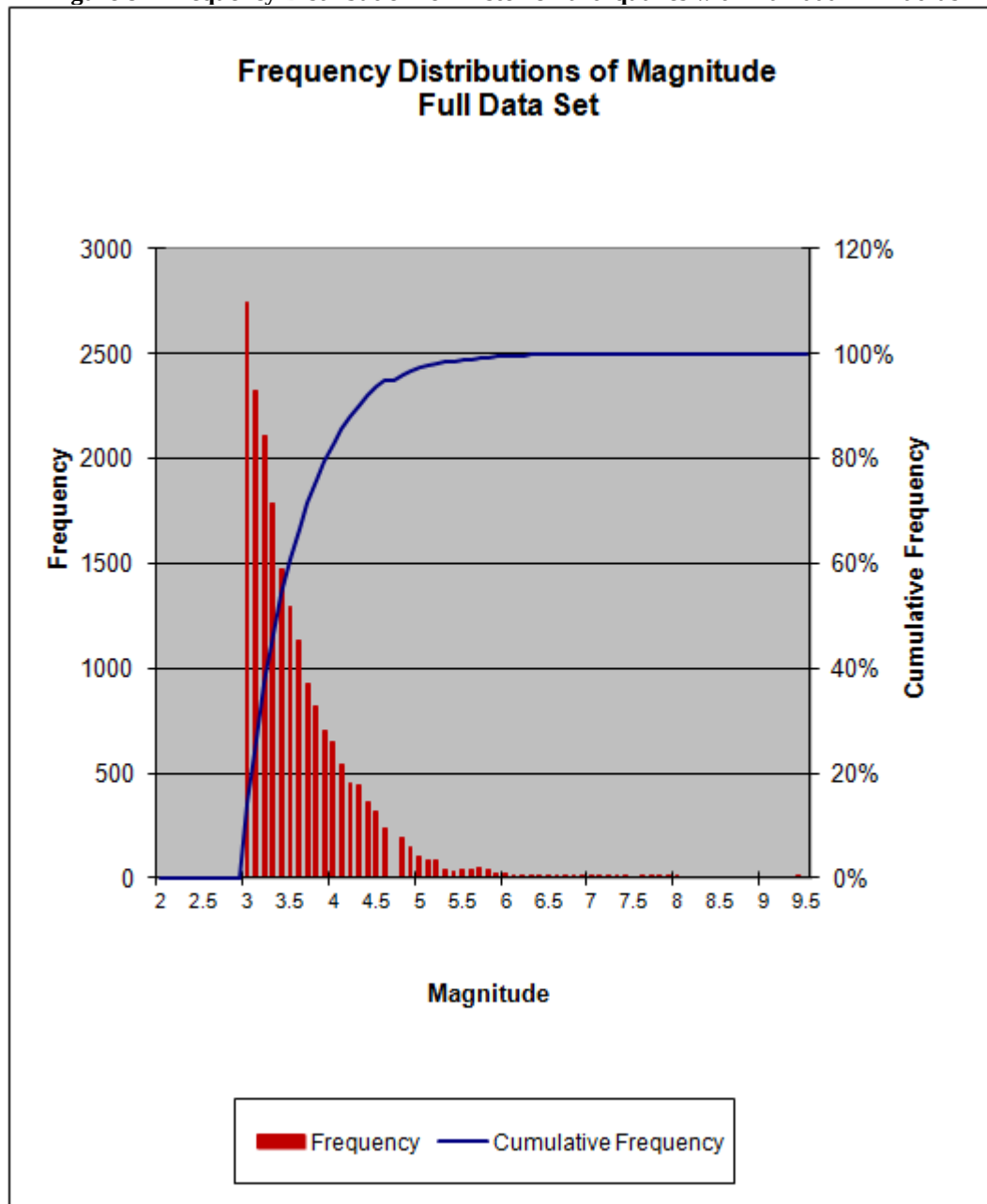


Figure 9 – Historic Earthquake Magnitude vs. Distance

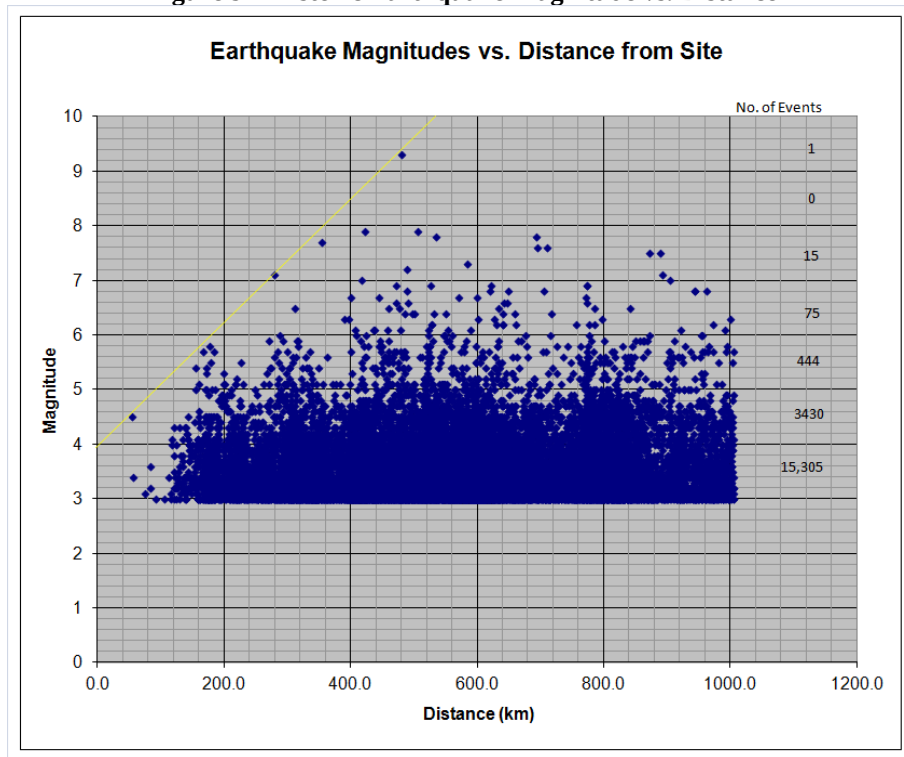
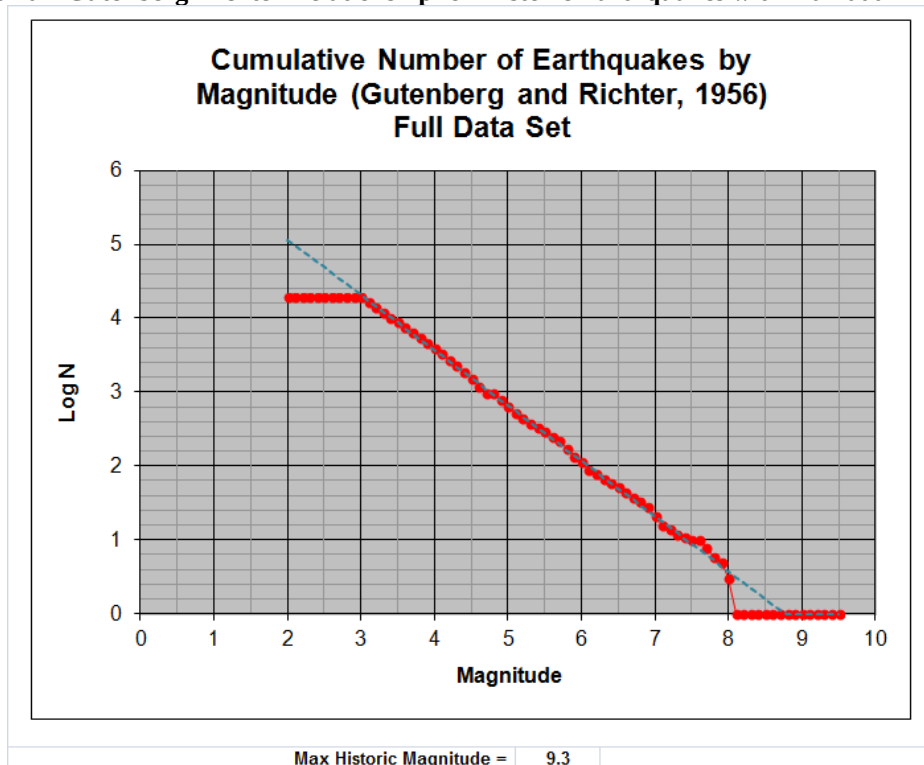


Figure 10 – Gutenberg-Richter Relationship for Historic Earthquakes within a 1000 km Radius



4 Seismic Source Zones

The maps that follow show, at progressively larger scale, the location of the major seismogenic fault systems in southwest Yukon Territory and southeast Alaska (see Figure 11 through Figure 13). Figure 14 shows the regional seismic source zones assigned to the map of historic earthquake events for the specific purpose of evaluating potential seismic loads for the Coffee Gold Project site. We have identified six (6) seismic source zones which (proceeding from northeast to southwest) are as follows:

1. Tintina North Zone
2. Inland Alaska Zone
3. Tintina Trench Zone
4. Denali Fault Zone
5. Aleutian Megathrust Zone
6. Fairweather-Queen Charlotte, Transition Fault Zone

The purpose of identifying discrete seismic source zones is to characterize and quantify the nature of the largest earthquake that is likely to occur within the zone. This information can be utilized in either a deterministic seismic hazard analysis (DSHA) or a probabilistic seismic hazard analysis (PSHA). Although different in approach, they probably have more in common than they have differences. Of interest is the largest earthquake that could reasonably be expected to occur within the zone, the mean rate of occurrence or recurrence interval, and the location of the earthquake. Seismic source zones come in two (2) varieties:

1. An Aerial Seismic Source where earthquakes are uniformly distributed throughout the area and assumed to be equally likely to occur anywhere within the area.
2. A Linear Seismic Source where earthquakes occur along a narrow linear band (fault), but are again assumed to be equally likely to occur anywhere along the fault line.

Figure 11 – Major Faults along the West Coast of North America

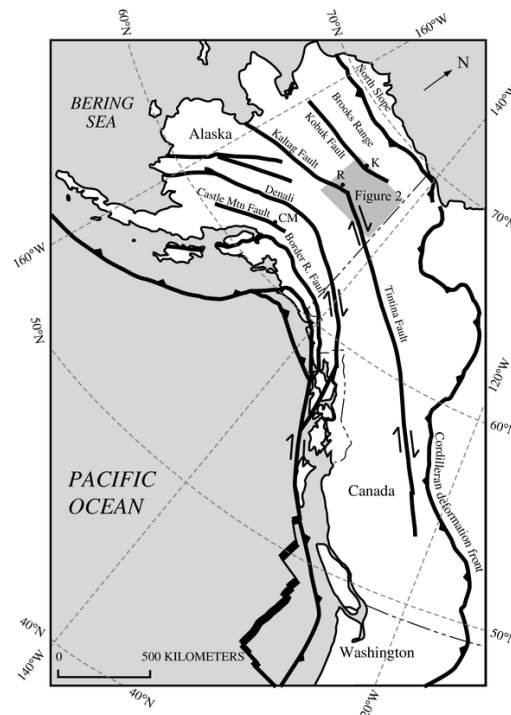


Figure 12 – Major Faults in Southwest Yukon and Southeast Alaska

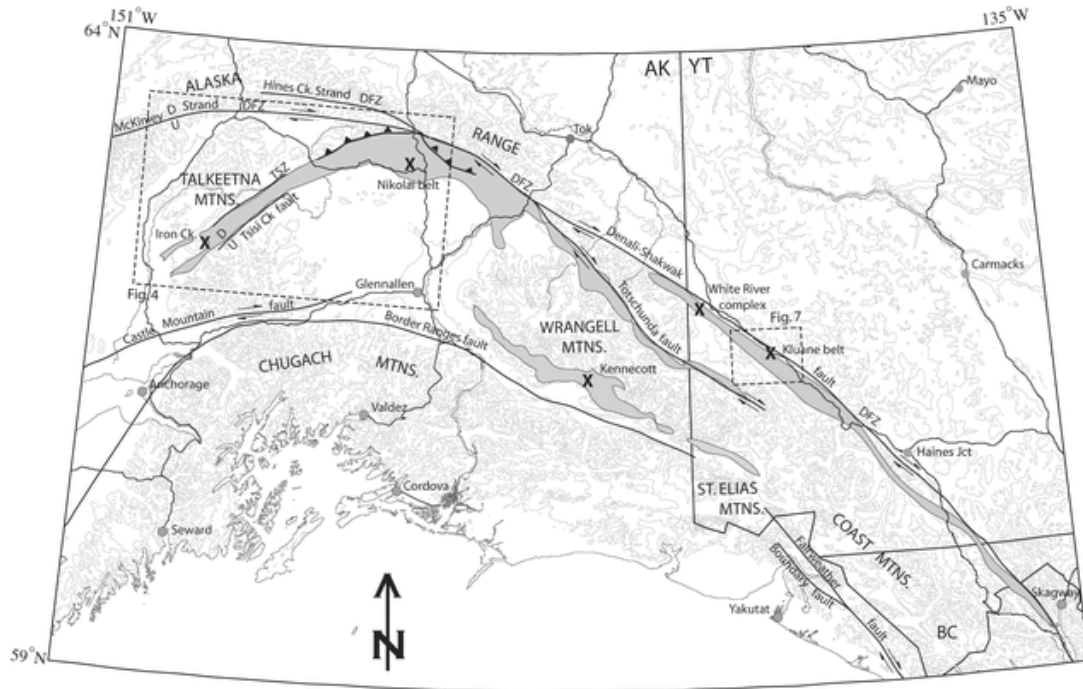


Figure 13 – Significant Seismic Features in Southwest Yukon and Southeast Alaska

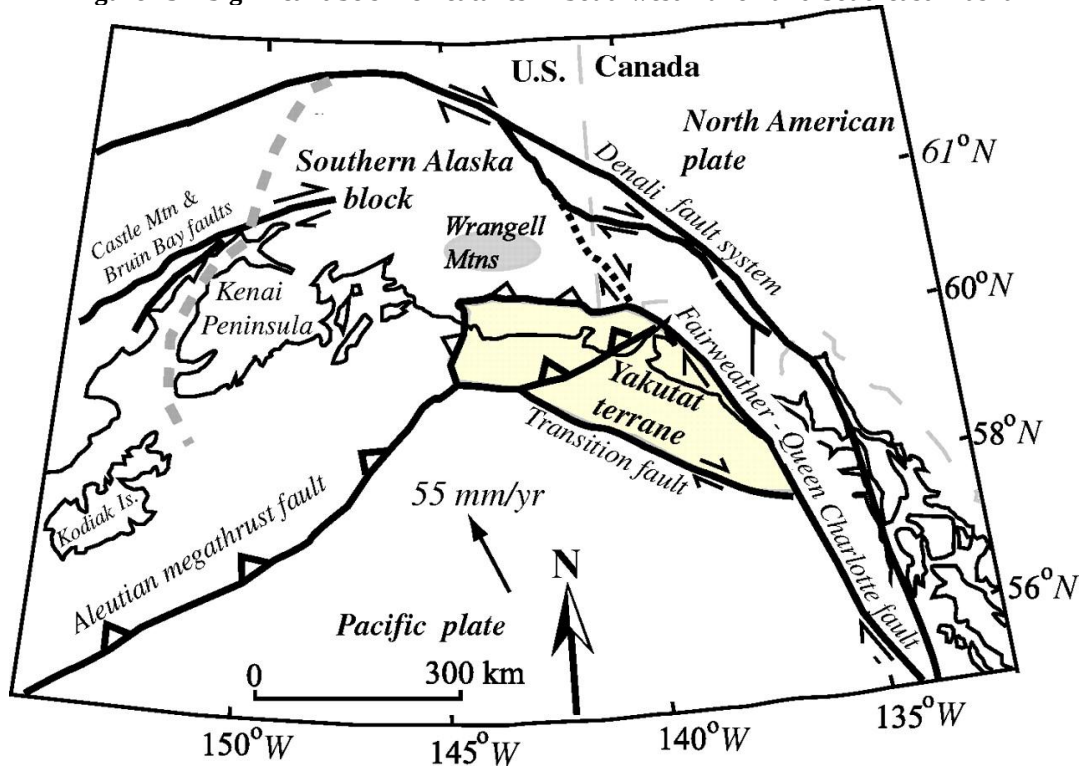
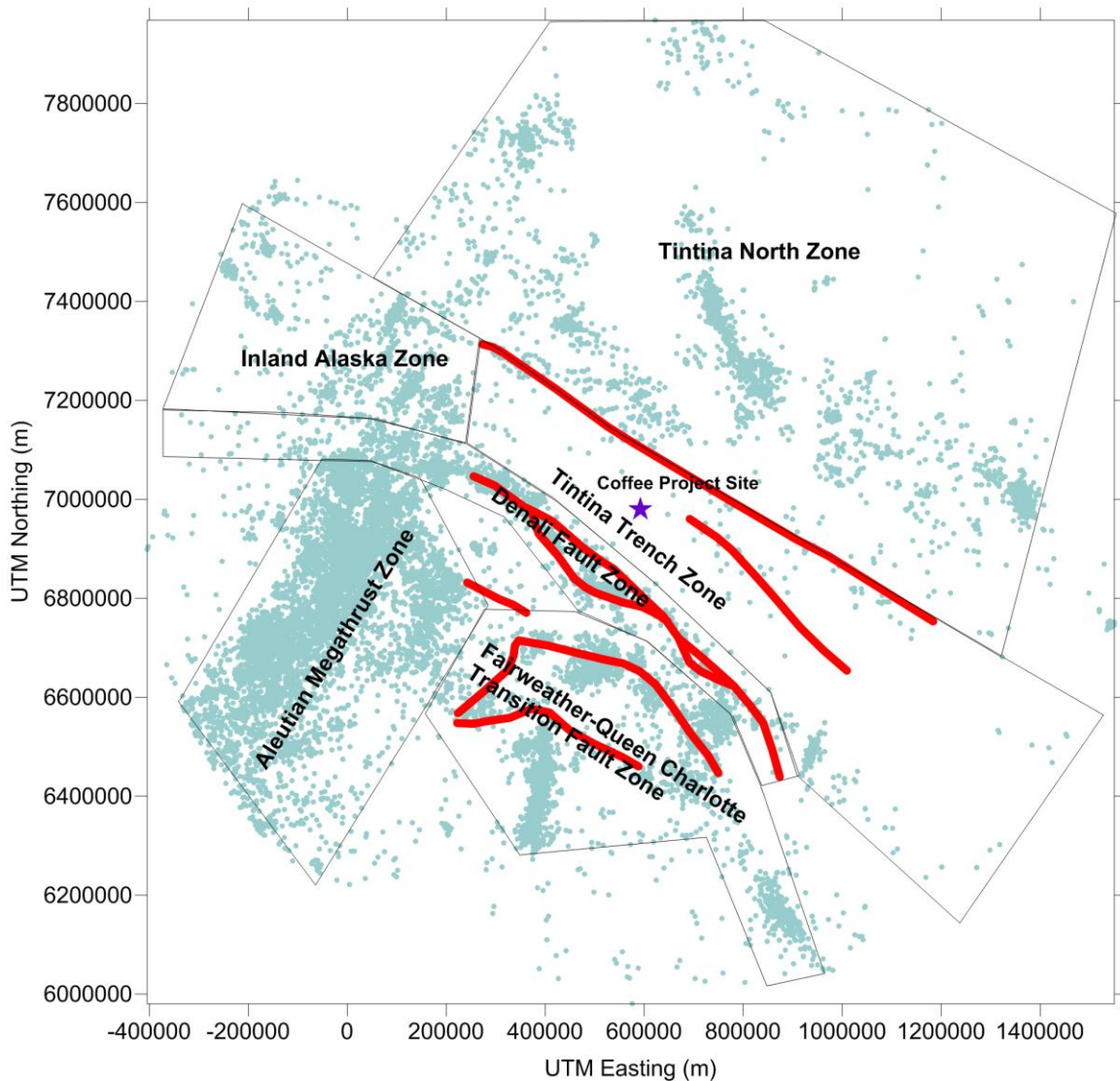


Figure 14 – Boundaries of Selected Seismic Source Zones
Coffee Project Seismic Source Zones



Note: The mapped fault locations (shown in red above) are based on the Regional Tectonics and Historical Seismicity map (Figure 1) in the Knight Piesold Consulting, Preliminary Seismic Hazard Assessment, dated December 9, 2014.

For the DSHA approach, the largest earthquake event that could occur within the zone is estimated and then relocated to the position that most closely approaches the site (either along the fault in a narrow linear source zone or within the area boundaries in an aerial source zone). Site specific ground motion parameters are then estimated by attenuating ground motion between the earthquake location and the site. This seems like a very logical “worst case” type of approach, but can sometimes lead to unrealistically severe design criteria. To address this concern, in the PSHA approach the single input parameter of earthquake magnitude is replaced with a cumulative probability distribution relating earthquake magnitude to recurrence rate and an associated exceedance probability. The simple location parameter or distance to the site is replaced by another cumulative probability distribution that addresses the uncertainty associated with where the earthquake event might occur within the source zone (i.e., the variability in the distance to the

site). Finally, there is also uncertainty associated with ground motion attenuation and a random component is applied to the ground motion.

5 Deterministic Seismic Hazard Analysis (DSHA)

The steps involved in a DSHA analysis are as follows:

1. Using information derived from geologic maps, fault maps, and plots of historic earthquake events, identify discrete seismic source zone polygons.
2. Extract “clipped” data sets lying within each seismic source zone that represent the nature of the seismicity within the zone.
3. Estimate the Maximum Credible Earthquake (MCE) associated with each seismic source.
4. Estimate the closest point of approach to the site of interest for each seismic source zone.
5. Estimate site specific ground motions by attenuating motions over the distance between the earthquake epicenter and the site.

The MCE is the larger of the magnitude predicted by the Gutenberg-Richter relationship for the clipped data set, the mapped fault characteristics (typically surface rupture length, subsurface rupture length, rupture area, or maximum displacement), or the maximum recorded historic earthquake within the seismic source zone. Sometimes in areas of low seismicity, from 0.5 to 1.0 units of magnitude will be added to the maximum recorded historic earthquake. That will not apply in this highly active seismic area. The most commonly used relationships between earthquake magnitude and fault geometry are those of Wells and Coppersmith, 1994 which were used in the completion of this evaluation.

The attenuation relationships used in this evaluation are the most current Next Generation Attenuation equations (NGA) from 2013 assuming 5% damping. Three (3) relationships were selected representing western North America and coastal plate boundary environments. They include:

- Abrahamson, Silva, and Kamai (2013) – NGA West 2
- Campbell and Bozorgnia (2013) – NGA West 2
- Chiou and Youngs (2013) – NGA West 2

A brief description of each seismic source zone will follow along with a summary of any pertinent assumptions made with respect to the zone.

5.1 Tintina North

The Tintina North Zone is treated as aerial seismic source zone despite the presence of the long right lateral strike slip Tintina fault along the full length of its southwestern border. Examination of the pattern of earthquake events in the historic record does not disclose a concentration of events along the alignment of the fault. A plot of the pattern of historic events is shown in Figure 15. Figure 16 shows the frequency distribution of earthquake magnitudes within the source zone which is then plotted in the Gutenberg-Richter relationship as shown in Figure 17 and shows an estimated maximum event magnitude of 7.2. This exceeds the maximum recorded historic event of magnitude 6.9. Figure 18 shows the distribution of the distances between event epicenters and the site. The statistics of this distribution are as follows:

- Mean = 547 km
- Median = 496 km
- Minimum (closest) = 118 km

Figure 15 – Historic Earthquake Locations in the Tintina North Seismic Source Zone

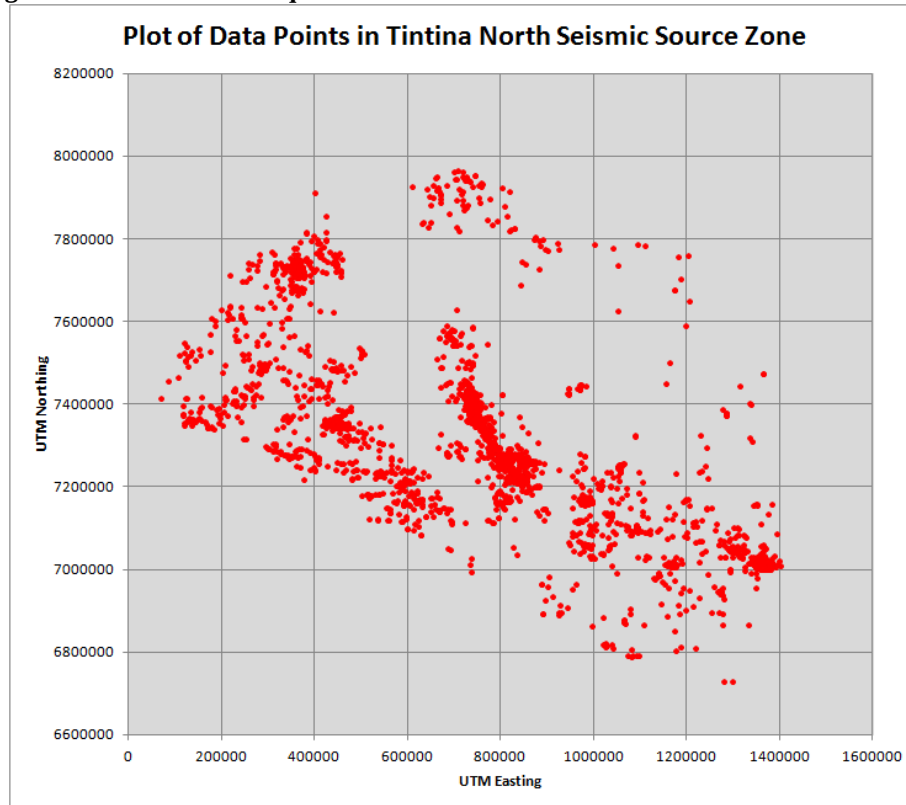


Figure 16 – Frequency Distribution of Magnitude for the Tintina North Seismic Source Zone

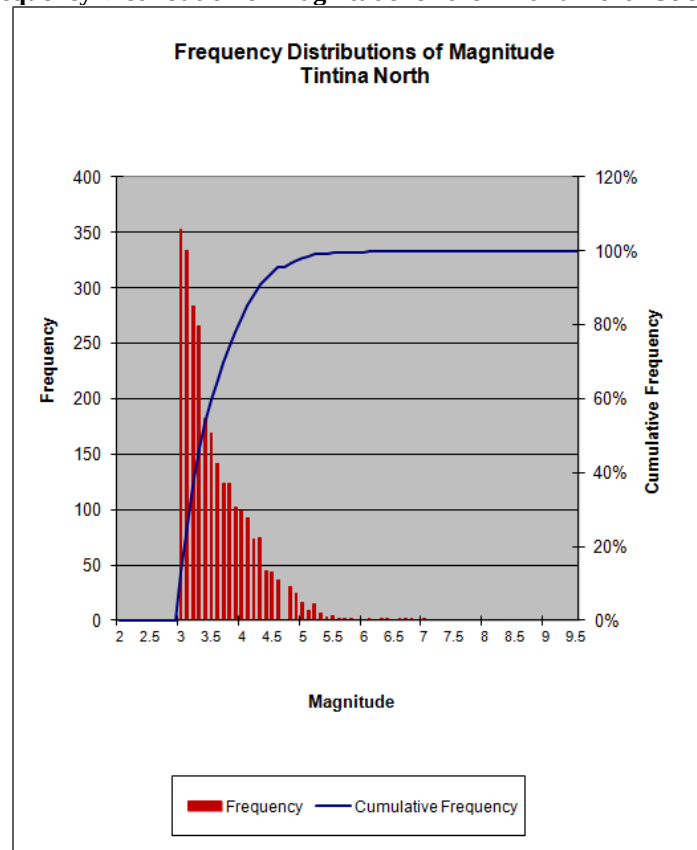


Figure 17 - Gutenberg-Richter Relationship for Tintina North Seismic Source Zone

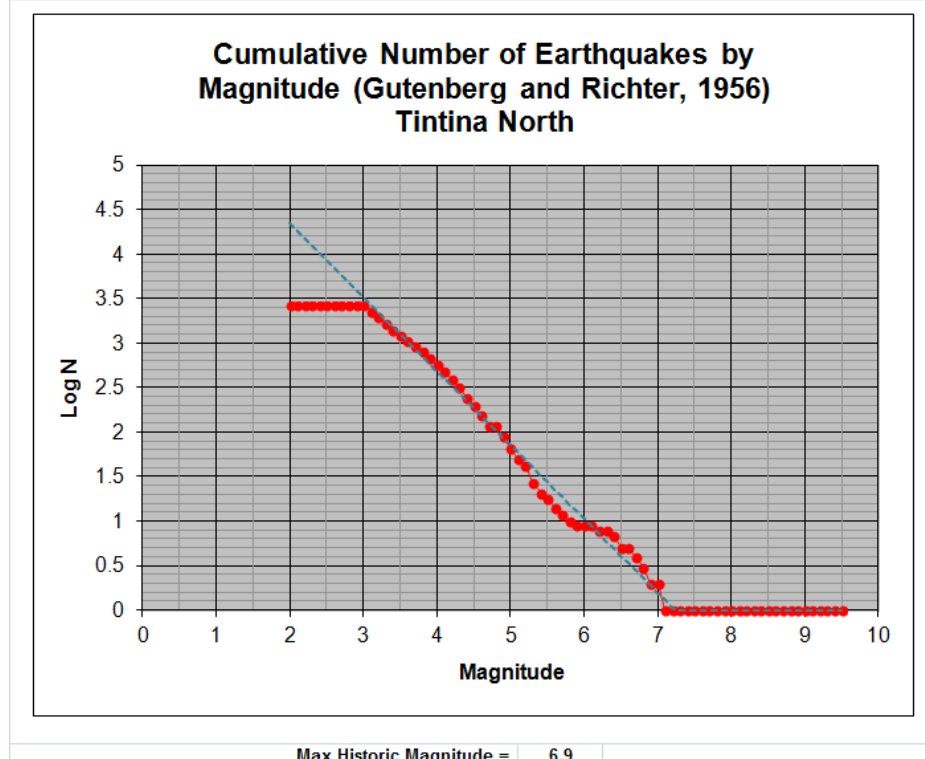
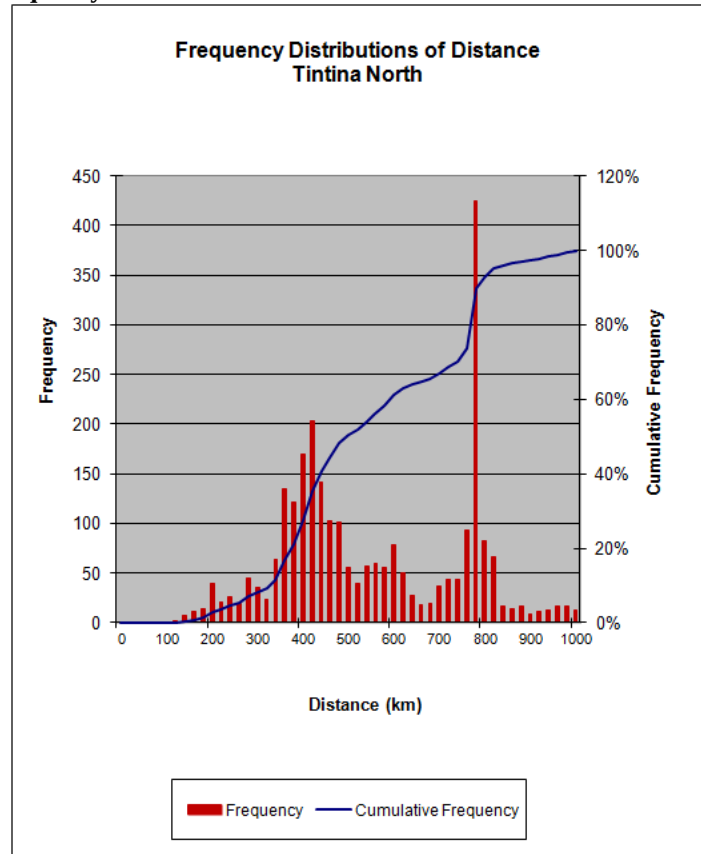


Figure 18 - Frequency Distribution of Distance for the Tintina North Seismic Source Zone

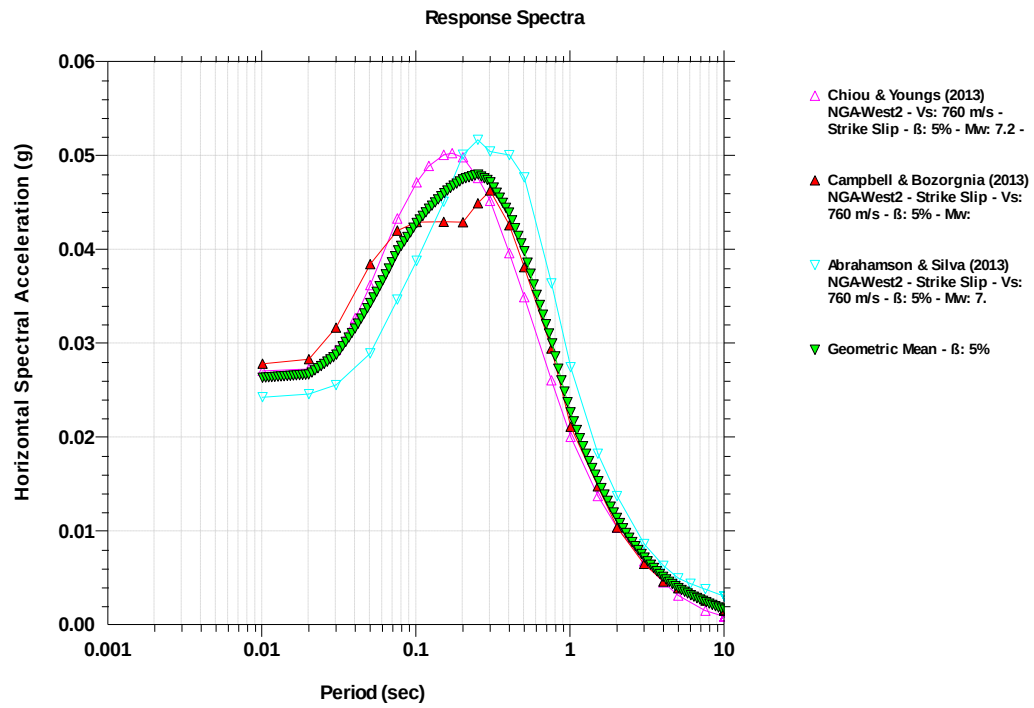


Using the maximum magnitude of 7.2 at the minimum distance of 118 km the attenuated ground motion response spectra shown in Figure 19 was generated. Mean values of pseudo-spectral acceleration were then extracted from this figure and are shown in Table 4.

Table 4 – Attenuated S_a vs. Natural Period for the Tintina North Seismic Source Zone

Natural Period (s)	Spectral Acceleration (g)
PGA (0 s)	.026
0.2	.047
0.5	.038
1.0	.022
2.0	.011

Figure 19 – Attenuated Sa Response Spectra for Tintina North Seismic Source Zone



5.2 Inland Alaska

The Inland Alaska seismic source zone is a westward extension of the Tintina Trench across the U.S.-Canadian border and into Alaska. It is modeled as an aerial seismic source zone (see Figure 20). Figure 21 shows the frequency distribution of earthquake magnitudes within the source zone which is then plotted in the Gutenberg-Richter relationship as shown in Figure 22 and shows an estimated maximum event magnitude of 7.1. This estimate is identical to the maximum recorded historic event of magnitude 7.1. Figure 23 shows the distribution of the distances between event epicenters and the site. The statistics of this distribution are as follows:

- Mean = 656 km
- Median = 633 km
- Minimum (closest) = 420 km

Figure 20 - Historic Earthquake Locations in the Inland Alaska Seismic Source Zone

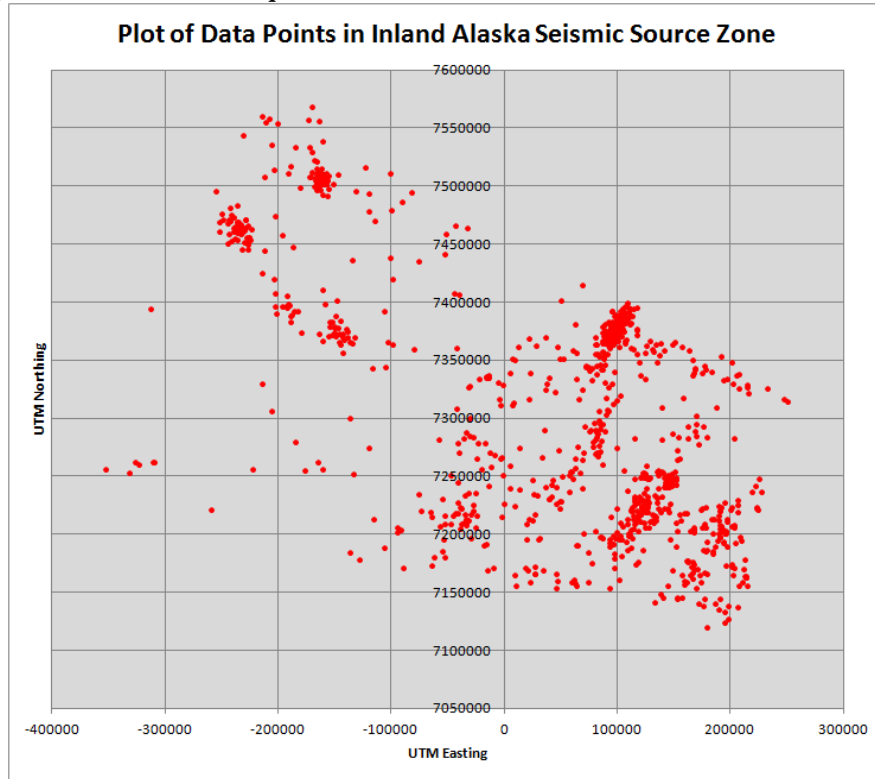


Figure 21 - Frequency Distribution of Magnitude for the Inland Alaska Seismic Source Zone

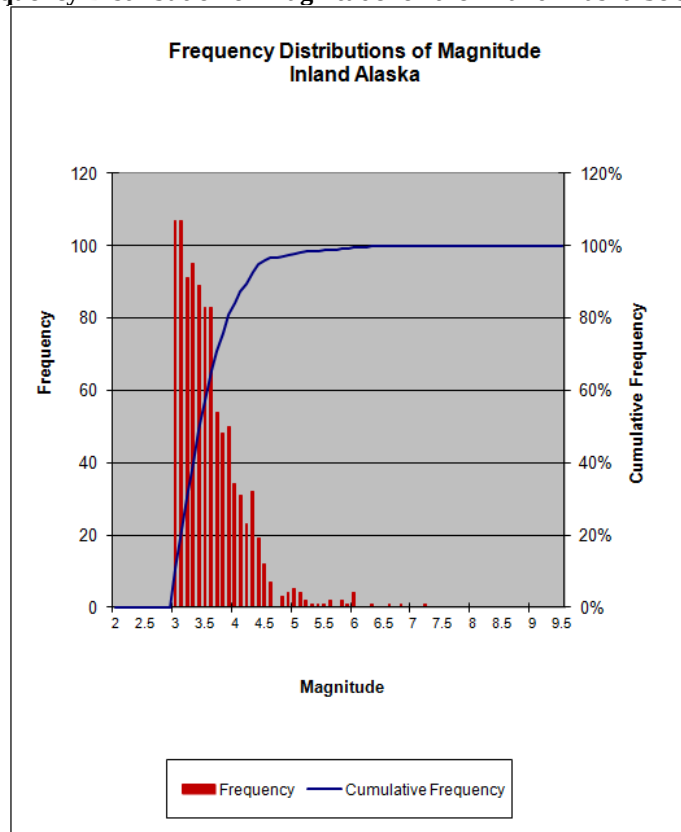


Figure 22 - Gutenberg-Richter Relationship for Inland Alaska Seismic Source Zone

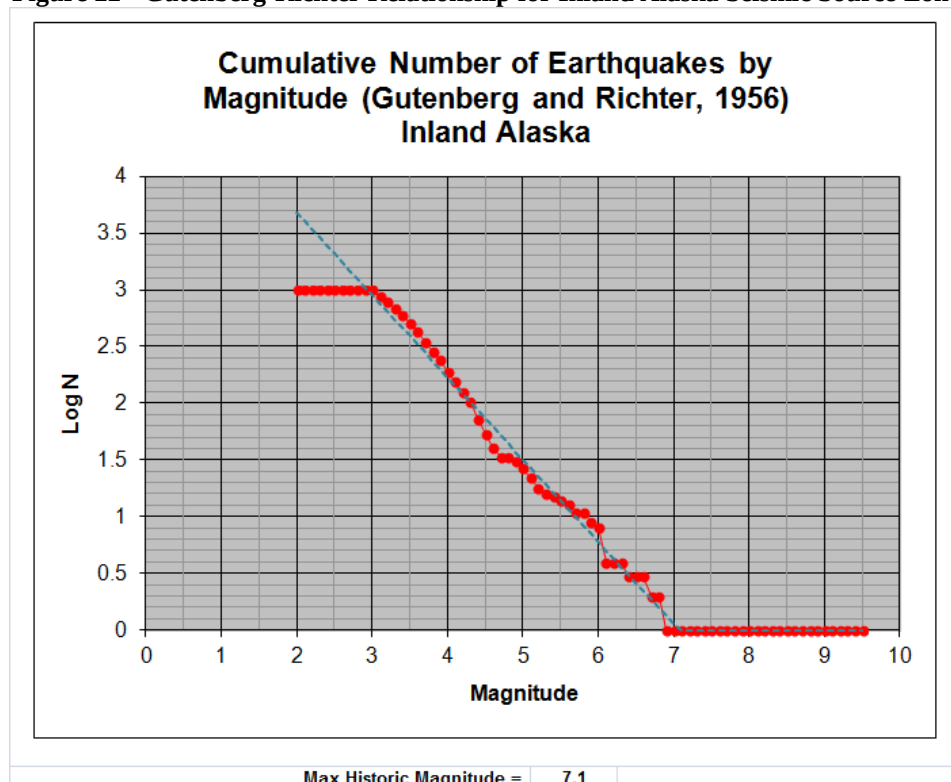
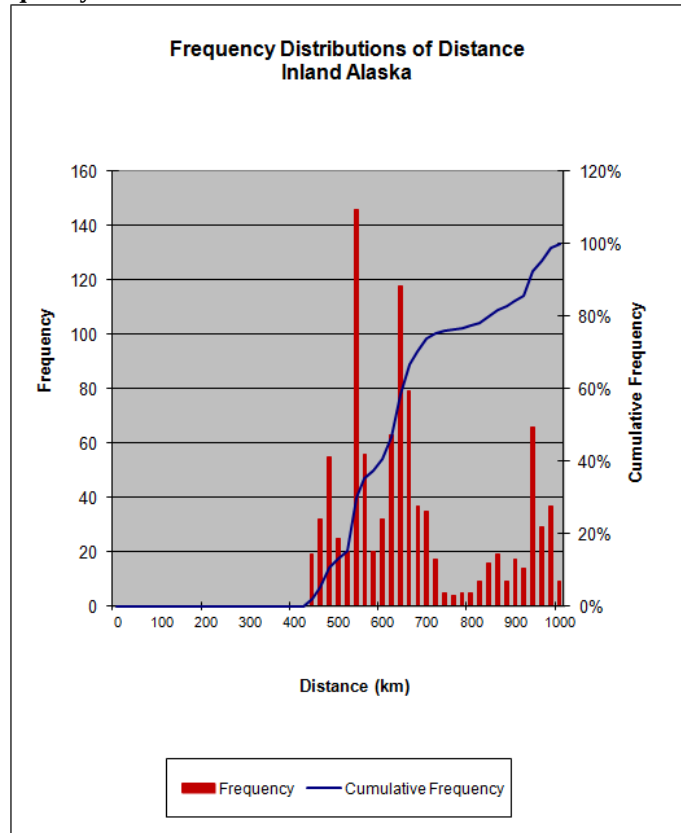


Figure 23 - Frequency Distribution of Distance for the Inland Alaska Seismic Source Zone



Application of the 2013 version of the NGA relationships are restricted to earthquake magnitudes of 5.0 to 8.5 and for distances not exceeding 300 km. Given the closest approach at 420 km, no attenuated ground motions can be estimated for this source zone using the NGA West 2 relationships. However, given the 7.1 magnitude and 420 km distance compared to the 7.2 magnitude and 118 km distance obtained for Tintina North, the expected ground motion would be less than that calculated for Tintina North and would not control design parameters.

5.3 Tintina Trench

The Tintina Trench seismic source zone is a broad valley lying between the Tintina Fault on its northeast boundary and the Denali Fault on its southwest boundary. It is modeled as an aerial seismic source zone (see Figure 24). Figure 25 shows the frequency distribution of earthquake magnitudes within the source zone which is then plotted in the Gutenberg-Richter relationship as shown in Figure 26 and shows an estimated maximum event magnitude of 5.0. This estimate exceeds the maximum recorded historic event of magnitude 4.6. Figure 27 shows the distribution of the distances between event epicenters and the site. The statistics of this distribution are as follows:

- Mean = 550 km
- Median = 591 km
- Minimum (closest) = 55 km

However, the project site itself lies roughly near the geographic center of this source zone. Therefore in theory, given that earthquakes are equally likely to occur anywhere within this seismic source zone, the maximum credible earthquake could occur directly below the project site (distance $D = 0$ km). Therefore, for this zone two (2) ground motion attenuation profiles were developed, one at the 55 km historic minimum and one for a distance of 1 km (1 instead of 0 to preclude any potential divide by zero issues).

Figure 24 - Historic Earthquake Locations in the Tintina Trench Seismic Source Zone

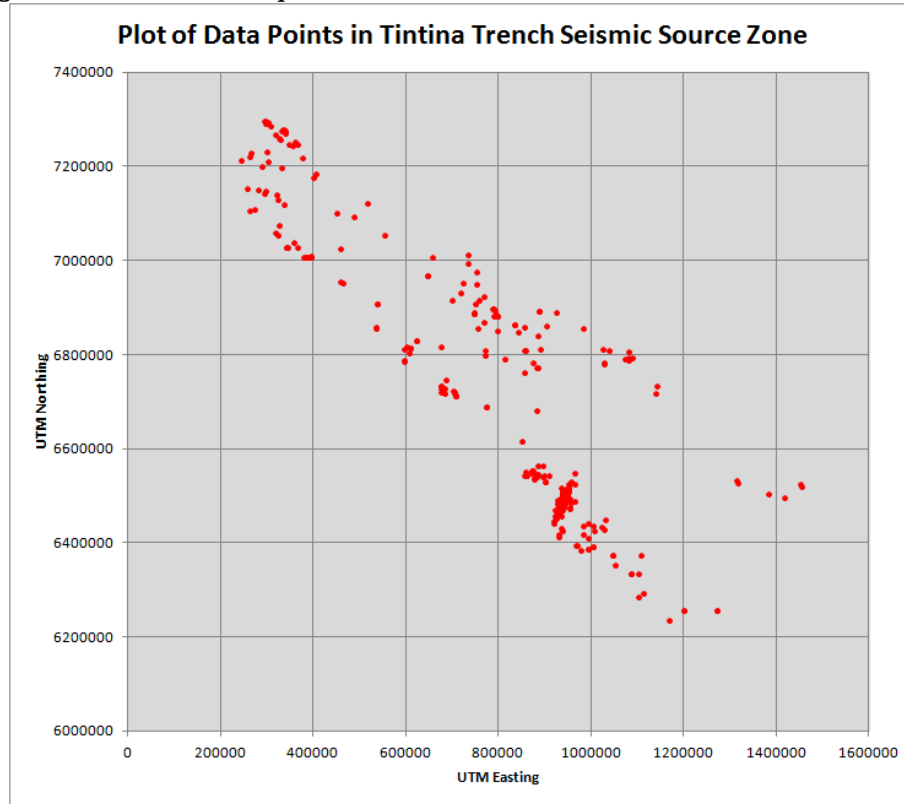


Figure 25 - Frequency Distribution of Magnitude for the Tintina Trench Seismic Source Zone

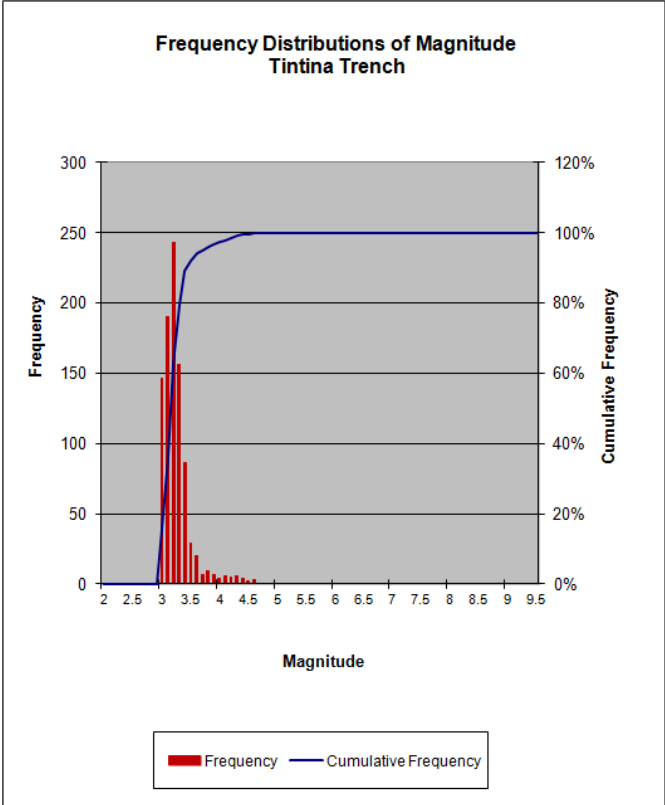


Figure 26 - Gutenberg-Richter Relationship for Tintina Trench Seismic Source Zone

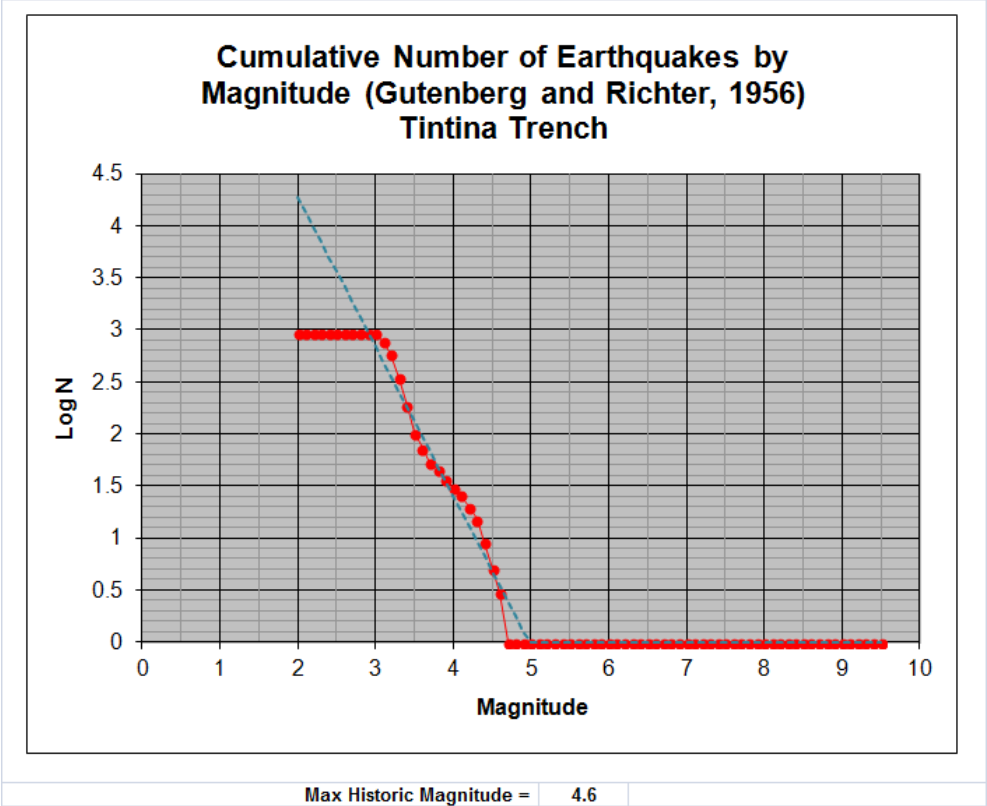
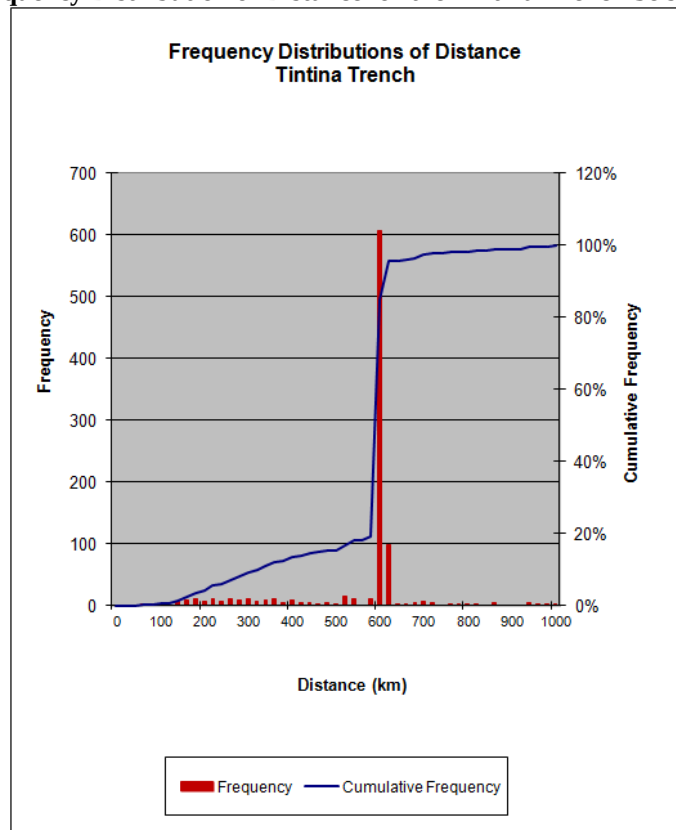


Figure 27 - Frequency Distribution of Distance for the Tintina Trench Seismic Source Zone



Using the maximum magnitude of 5.0 at the minimum distances of 55 km and 1 km the attenuated ground motion response spectra shown in Figure 28 and Figure 29 were generated. Mean values of pseudo-spectral acceleration were then extracted from these figures and are shown in Table 5.

Table 5 - Attenuated Sa vs. Natural Period for the Tintina Trench Seismic Source Zone

Natural Period (s)	Spectral Acceleration (g)	
	D = 55 km	D = 1 km
PGA (0 s)	.0085	.28
0.2	.0155	.53
0.5	.0065	.20
1.0	.0002	.07
2.0	.0001	.02

Figure 28 - Attenuated Sa Response Spectra for Tintina Trench 55 km Seismic Source Zone

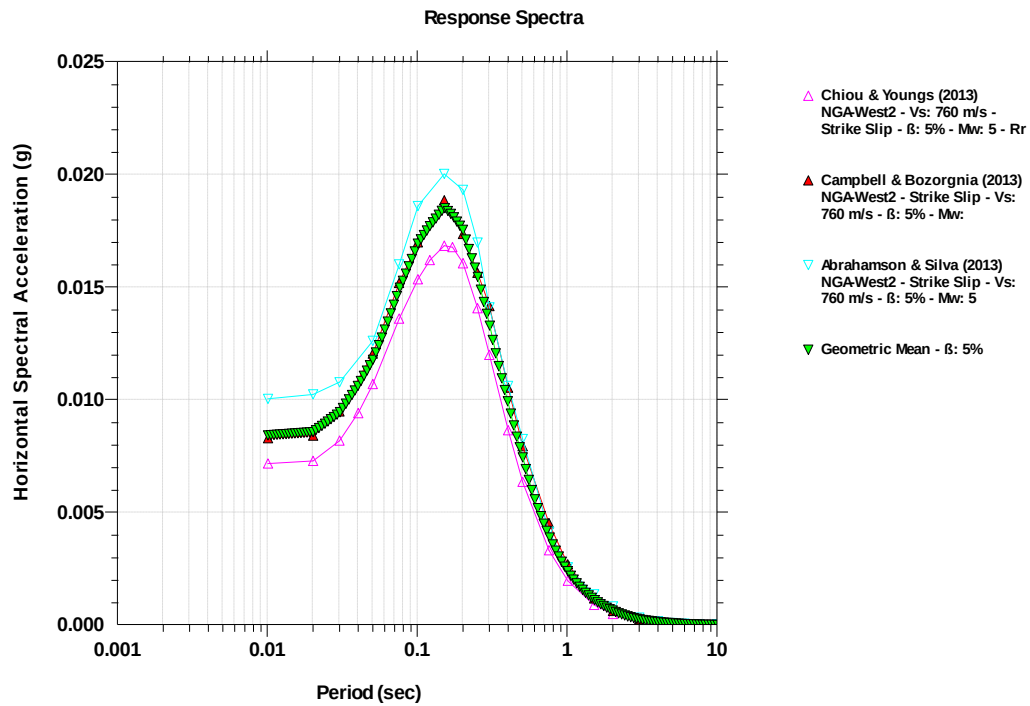
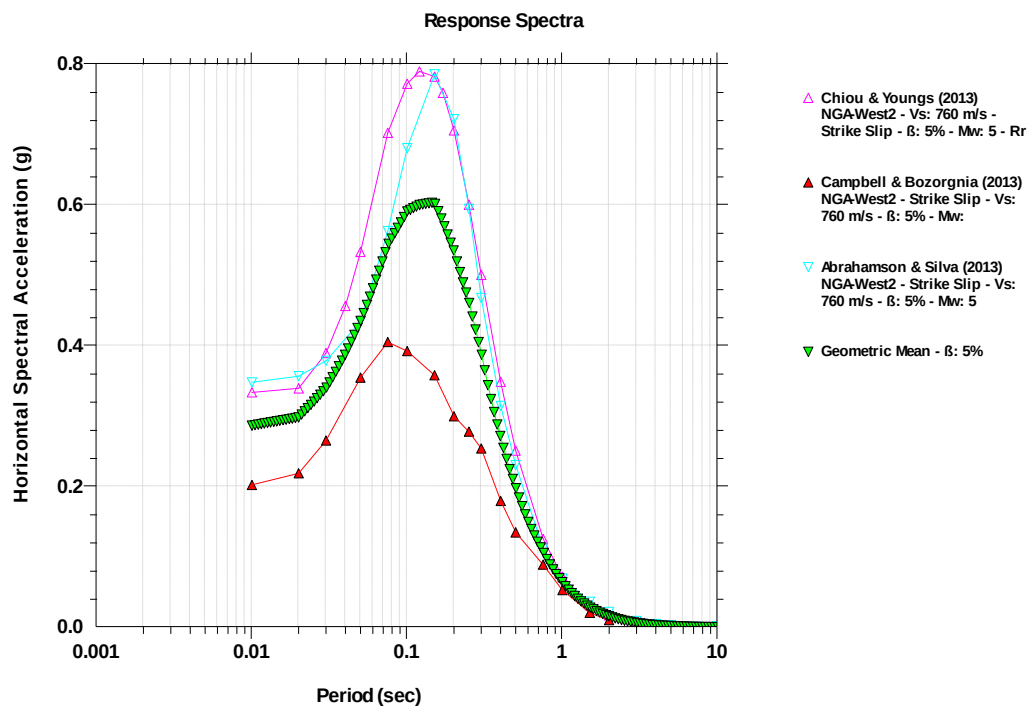


Figure 29 - Attenuated Sa Response Spectra for Tintina Trench 1 km Seismic Source Zone



5.4 Denali Fault

The Denali Fault seismic source zone is a long right lateral strike slip fault that extends from the southeast of the site to the northwest, curving westward as it moves across the U.S.-Canadian border and into Alaska. It is modeled as a linear seismic source zone (see Figure 30). Figure 31 shows the frequency distribution of earthquake magnitudes within the source zone which is then plotted in the Gutenberg-Richter relationship as shown in Figure 32 and shows an estimated maximum event magnitude of 7.3. This estimate is less than the maximum recorded historic event of magnitude 7.9. Figure 33 shows the distribution of the distances between event epicenters and the site. The statistics of this distribution are as follows:

- Mean = 360 km
- Median = 350 km
- Minimum (closest) = 105 km

The Denali fault is nearly 2000 km long extending from the east end of our seismic source zone to the southeast across all of Yukon Territory and into British Columbia, and from the west end of our seismic source zone southwest across all of southern Alaska. Estimation of the MCE as a function of surface fault rupture requires an estimate of the maximum expected rupture length for a single earthquake event. The long strike slip faults never rupture along their full length in a single event. Instead they are segmented. Segment lengths can often be observed from mapped fault traces as structural discontinuities (offsets normal to the fault trace or sudden “kinks” or changes in direction of the trace – Slemmons, 1977). However, major strike slip faults rarely exhibit any significant structural discontinuities. Various rules of thumb for maximum surface rupture length have been proposed in the literature over the years including one-half of the mapped total length (Wentworth et. al., 1969) or 17% to 38% of the mapped total length (Slemmons, 1977). A research study on the M7.9 November 3, 2002 event on the Denali fault documented a surface rupture length of 340 km (Eberhart-Phillips and Haeussler, 2003). Using a surface rupture length (SRL) of 340 km the Wells and Coppersmith equation generates a magnitude of 8.0.

Figure 30 - Historic Earthquake Locations in the Denali Fault Seismic Source Zone

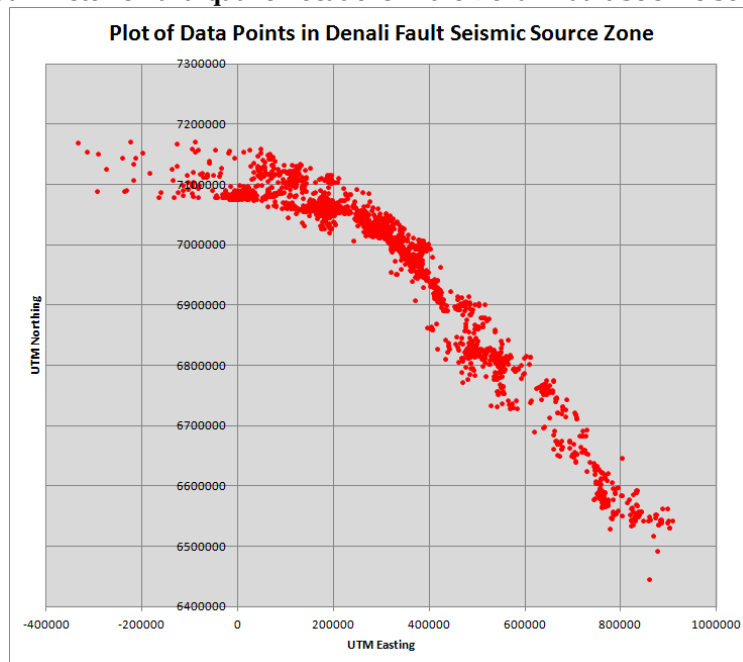


Figure 31 - Frequency Distribution of Magnitude for the Denali Fault Seismic Source Zone

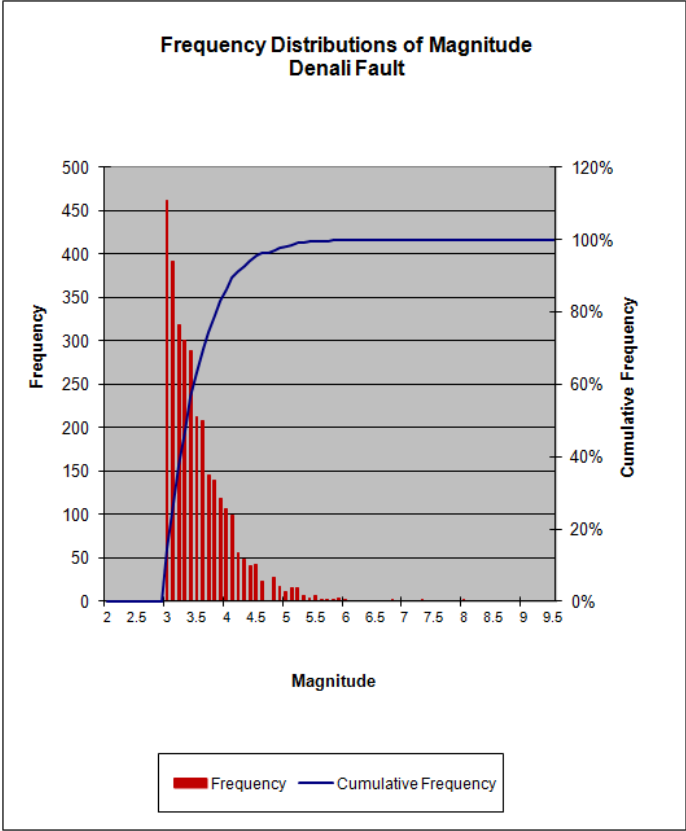


Figure 32 - Gutenberg-Richter Relationship for Denali Fault Seismic Source Zone

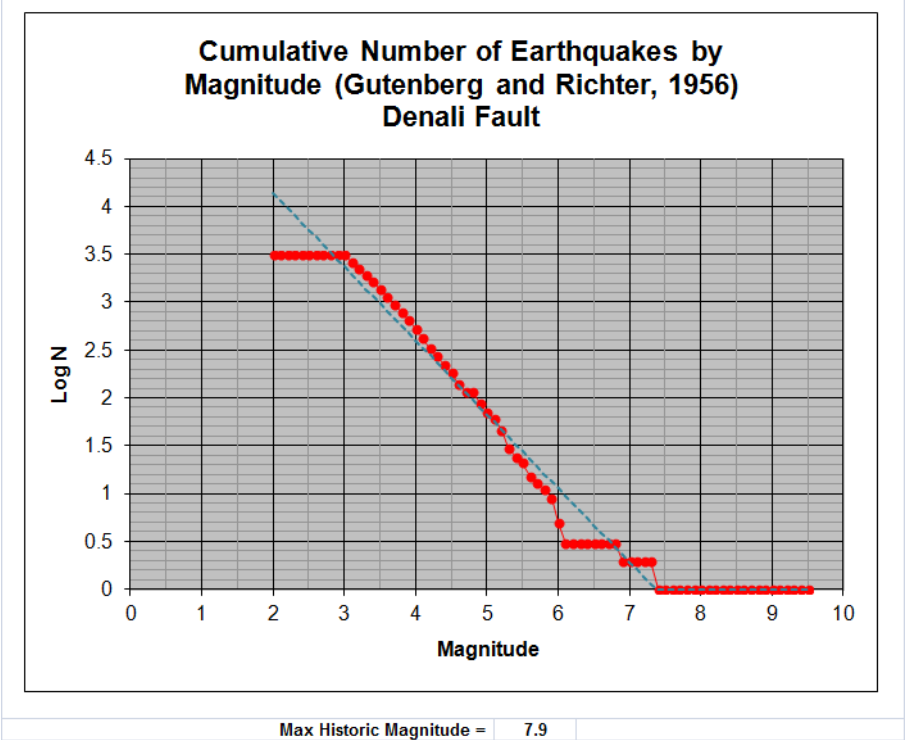
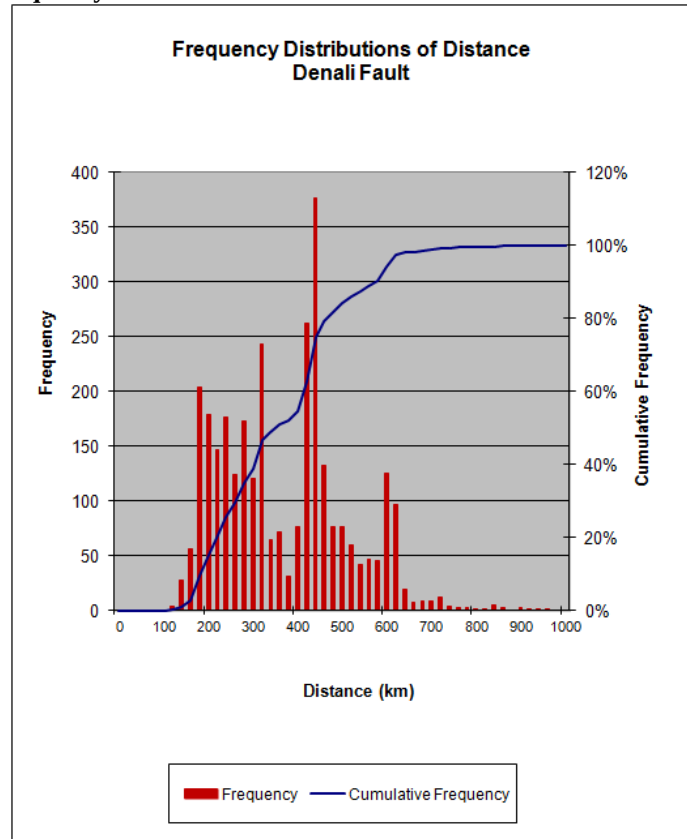


Figure 33 - Frequency Distribution of Distance for the Denali Fault Seismic Source Zone

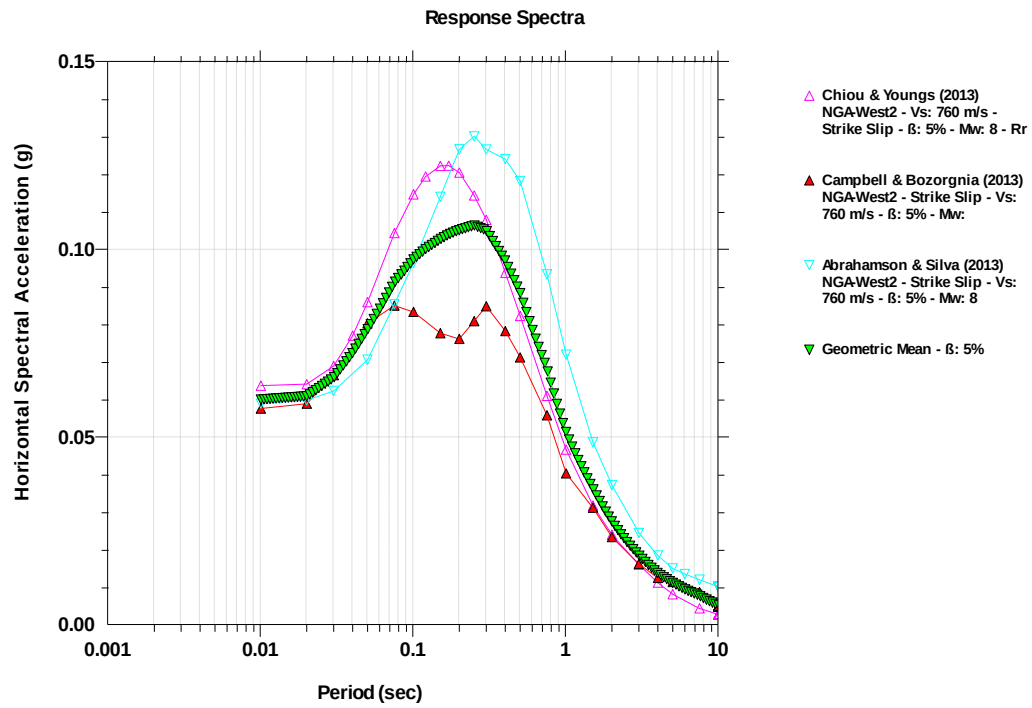


Using the maximum magnitude of 8.0 at the minimum distance of 105 km the attenuated ground motion response spectra shown in Figure 34 was generated. Mean values of pseudo-spectral acceleration were then extracted from this figure and are shown in Table 6.

Table 6 - Attenuated S_a vs. Natural Period for the Denali Fault Seismic Source Zone

Natural Period (s)	Spectral Acceleration (g)
PGA (0 s)	.06
0.2	.11
0.5	.085
1.0	.050
2.0	.0025

Figure 34 - Attenuated Sa Response Spectra for Denali Fault Seismic Source Zone



5.5 Aleutian Megathrust

The Aleutian Megathrust seismic source zone is a wide, rectangular shaped band oriented northwest to southeast and lying just south of the Denali fault. It is modeled as an aerial seismic source zone (see Figure 35). The earthquake pattern reflects a northwest dipping subduction zone/thrust fault where the Pacific Plate is diving below the North American Plate. Figure 36 shows the frequency distribution of earthquake magnitudes within the source zone which is then plotted in the Gutenberg-Richter relationship as shown in Figure 37 and shows an estimated maximum event magnitude of 8.3. This estimate is less than the maximum recorded historic event of magnitude 9.3. Figure 38 shows the distribution of the distances between event epicenters and the site. The statistics of this distribution are as follows:

- Mean = 662 km
- Median = 623 km
- Minimum (closest) = 364 km

Although this seismic source zone is technically a thrust fault, the low angle of the thrust causes the events to be spread out into very broad band that is better viewed as an aerial seismic source zone. The historic events in this zone capture the March 28, 1964 Prince William Sound earthquake which is described in most of the literature as a M9.2 event (although the USGS in its database assigns it M9.3). This event is the second largest earthquake event ever recorded in the world in the history of seismic data collection (second only to the May 22, 1960 M9.5 event in the coastal subduction zone of Chile).

Figure 35 - Historic Earthquake Locations in the Aleutian Megathrust Seismic Source Zone

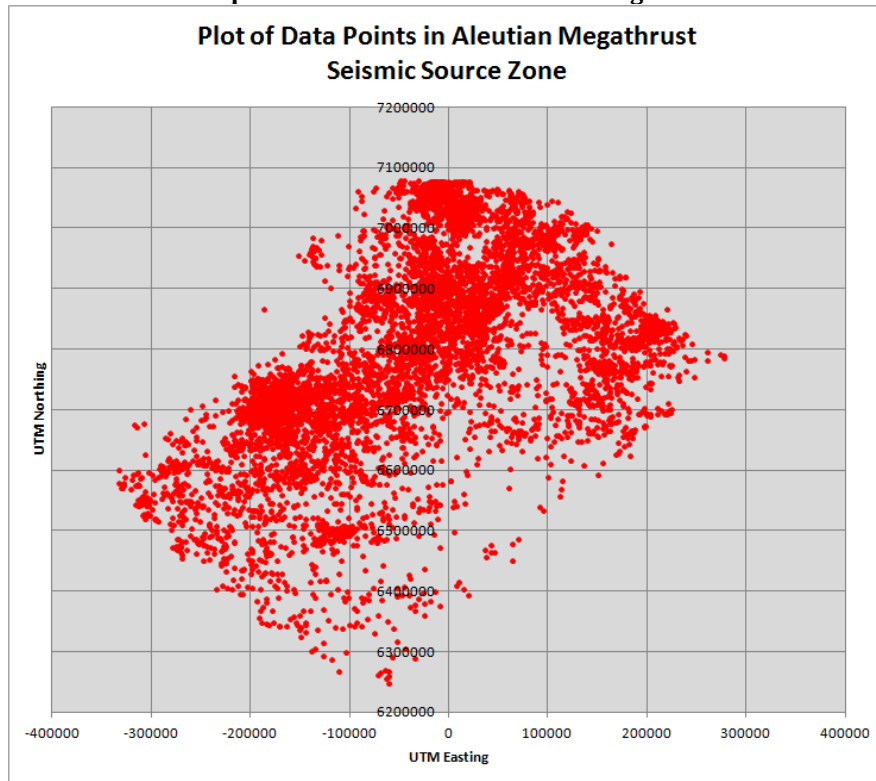


Figure 36 - Frequency Distribution of Magnitude for the Aleutian Megathrust Seismic Source Zone

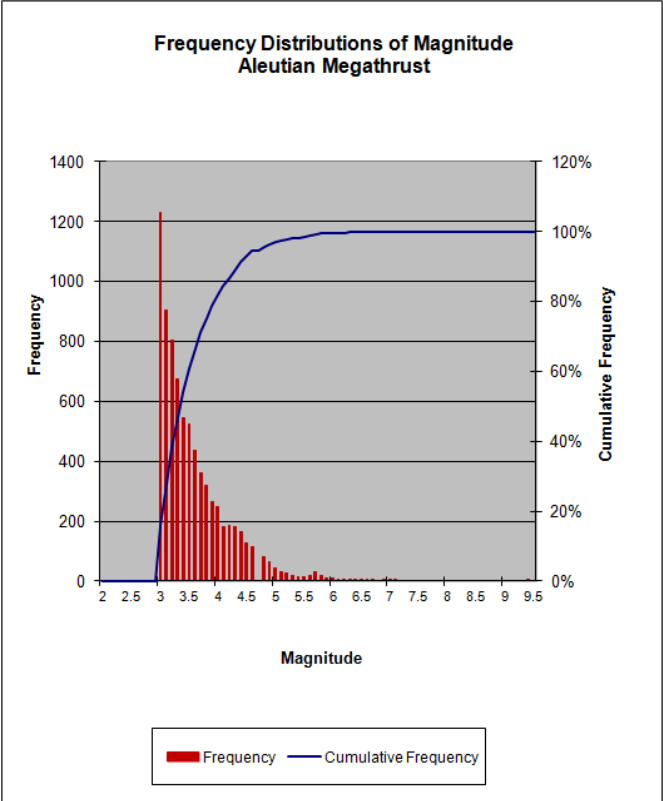


Figure 37 - Gutenberg-Richter Relationship for Aleutian Megathrust Seismic Source Zone

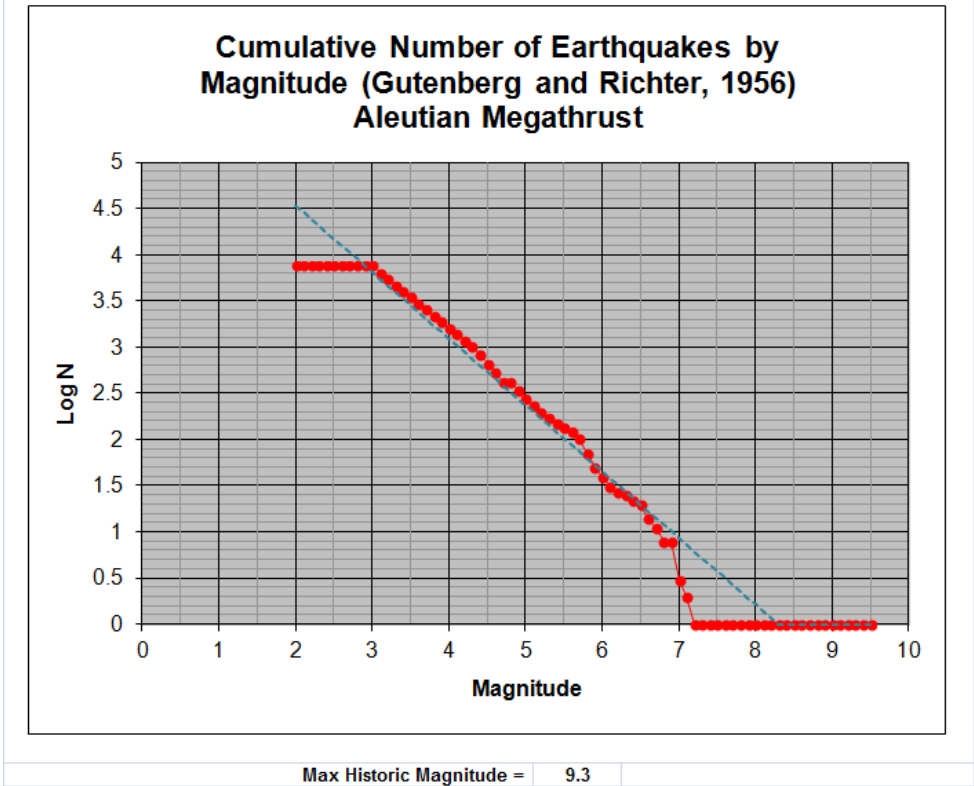
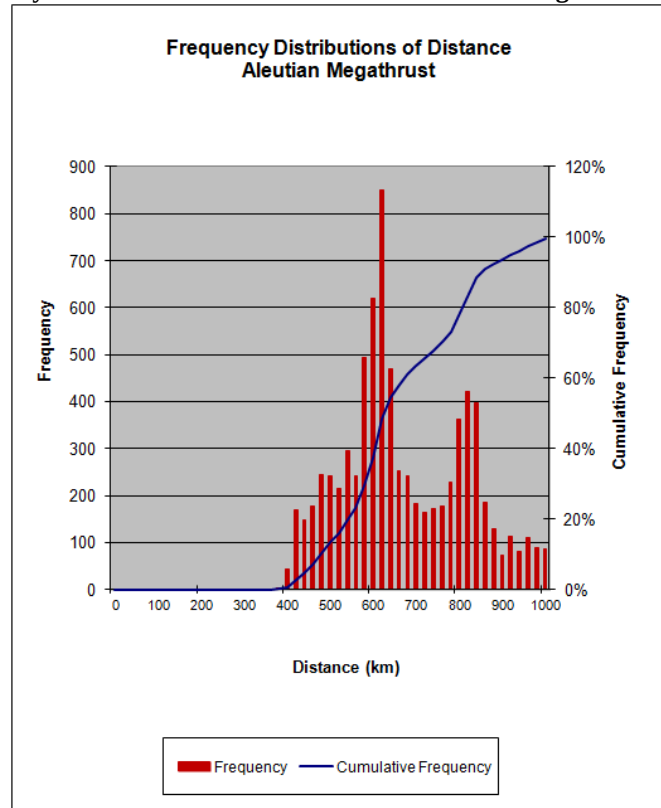


Figure 38 - Frequency Distribution of Distance for the Aleutian Megathrust Seismic Source Zone



The MCE maximum magnitude of 9.3 at the minimum distance of 364 km exceeds the NGA 2013 attenuated ground motion response spectra relationship limitations for both magnitude and distance. Therefore, it is not possible to use these relationships to estimate ground motion parameters at the site.

However, given the magnitude of the actual recorded event and the relatively close approach to the site for such a large event (well under 400 km), it is considered appropriate to explore potential impacts outside of the accepted 2013 NGA attenuation relationships. One of the most pertinent papers addressing ground motions for subduction zone earthquakes is a 2003 paper by Atkinson and Boore entitled “Empirical Ground-Motion Relations for Subduction-Zone Earthquakes and their Application to Cascadia and other Regions”. Some of the principal observations and conclusions from that paper can be summarized as follows. For a megathrust interface earthquake of M8+ at a distance of 100 km from the causative fault the following ground motions could be expected:

- 110 cm/s² at 0.5 Hertz (Hz) (0.112g at a period of 2 s)
- 660 cm/s² at 2.5 Hz (0.673g at 0.4 s)
- 410 cm/s² at 5 Hz (0.418g at 0.2 s)
- PGA of 180 cm/s²@ (PGA of 0.183g)

These damaging ground motions would be experienced over a very large area approximately 300 km wide (presumed to be normal to the fault trace) by 500 km long (presumed to be along the fault trace). Near field ground motions appear to saturate above about magnitude 8 (i.e. ground motions above M8 are not significantly different from those at M8). Figure 39 is a figure from the paper that shows a plot of the predicted PGA vs. magnitude and distance for both rock sites (dashed line) and soil sites (solid line). Examination of this plot shows the saturation effect as the

curves for the M7.5 event and the M8.5 event converge on the left edge of the plot at a fault distance of about 20 km. It also shows the very flat nature of the M8.5 curve up to about 100 km distance from the fault (i.e. virtually no attenuation for the first 100 km). The magnitude range of the dataset used to generate these curves was M6.0 to M8.3. Therefore they still do not apply directly to a M9.2+ event. In Figure 40 a graphical extrapolation of the curves was subjectively applied to represent a potential M9+ event that preserves the saturation over the first 100 km of distance but allows for separation of the curves over larger distances (the red line added to the figure). This was done to provide a rough “ballpark” estimate of the kind of ground motion that might be experienced at the project site in response to another great megathrust earthquake event along the eastern edge of the Aleutian Megathrust seismic source zone. Similarly a graphical extrapolation of the ground motion curves for 5 Hz (0.2 s period) and 0.5 Hz (2 s period) is shown in Figure 41 and Figure 42 respectively.

Figure 39 – PGA vs. Distance for Interface Megathrust Earthquake Events

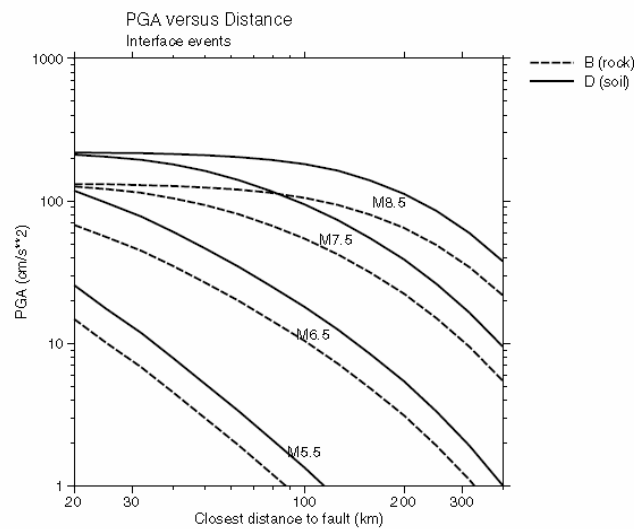


Figure 40 - PGA vs. Distance for Interface Megathrust Earthquake Events with Graphical Extrapolation

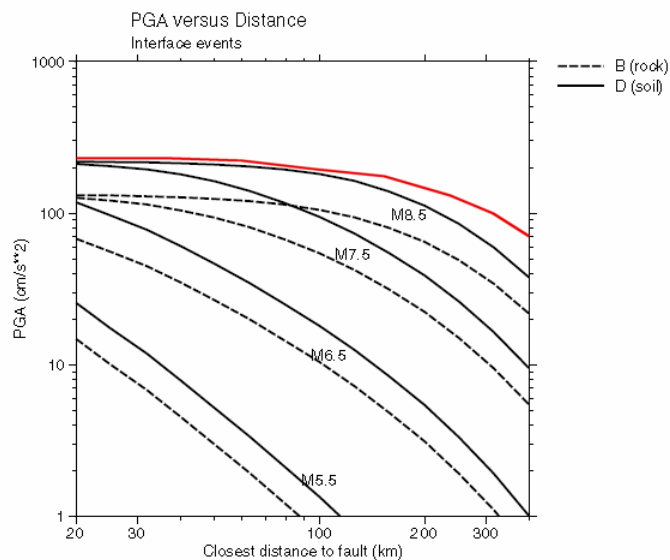


Figure 41 – Sa 0.2s vs. Distance for Interface Megathrust Earthquake Events with Graphical Extrapolation

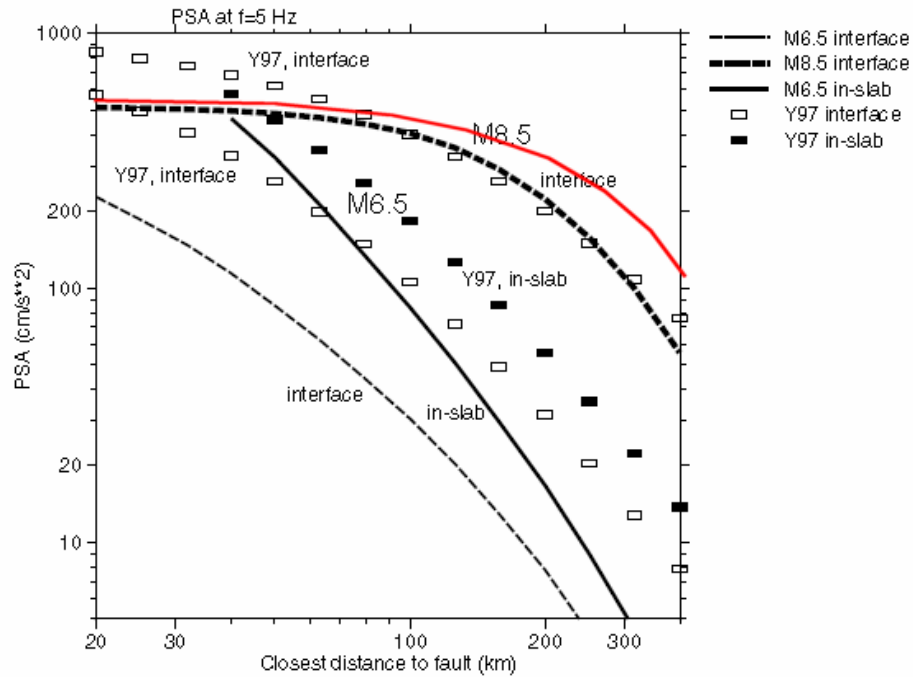
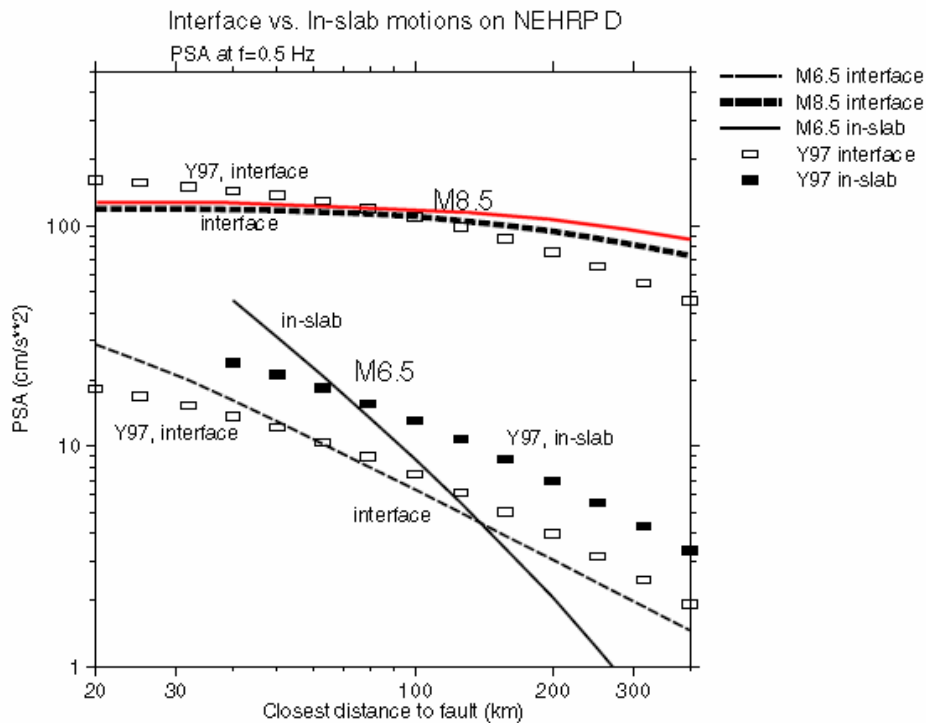


Figure 42 - Sa 2s vs. Distance for Interface Megathrust Earthquake Events with Graphical Extrapolation



Extrapolated values of pseudo-spectral acceleration are shown in Table 7.

Table 7 - Sa vs. Natural Period Extrapolated for the Aleutian Megathrust Seismic Source Zone

Natural Period (s)	Spectral Acceleration (g)
PGA (0 s)	0.07
0.2	0.153
2.0	0.092

5.6 Fairweather-Queen Charlotte and Transition Fault

The Fairweather-Queen Charlotte and Transition Fault seismic source zone contains multiple linear seismic sources including the long right lateral strike slip Fairweather-Queen Charlotte fault that extends from the coastal area immediately south of the site off to the southeast into British Columbia and southeast Alaska and the relatively short Transition fault that borders the Yakutat accretion terrane southeast of the St. Elias Mountains. The distribution of historic earthquakes is shown in Figure 43 Figure 44 shows the frequency distribution of earthquake magnitudes within the source zone which is then plotted in the Gutenberg-Richter relationship as shown in Figure 45 and shows an estimated maximum event magnitude of 8.7. This exceeds the maximum recorded historic event of magnitude 7.9. Figure 46 shows the distribution of the distances between event epicenters and the site. The statistics of this distribution are as follows:

- Mean = 509 km
- Median = 517 km
- Minimum (closest) = 267 km

There is relatively little evidence upon which to base an estimate of maximum SRL. However, the fault appears to have a total length of at least 800 km and using the 38% rule of thumb of Slemmons (1977), a SRL of 300 km would seem reasonable. This SRL results in an estimate of magnitude 8.0.

Figure 43 - Historic Earthquake Locations in the Fairweather-Queen Charlotte and Transition Fault Seismic Source Zone

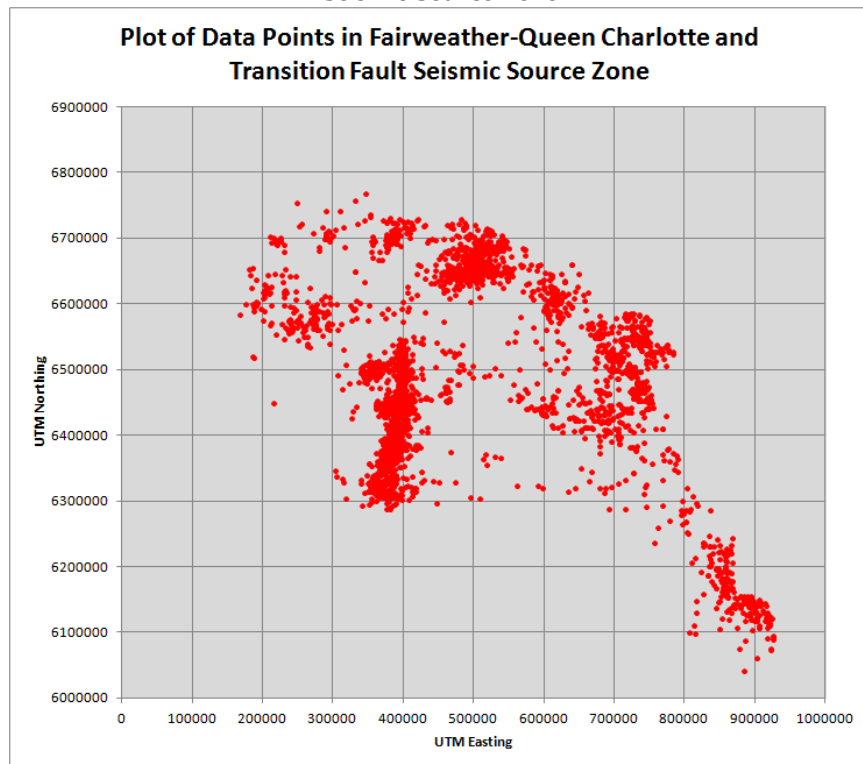


Figure 44 - Frequency Distribution of Magnitude for the Fairweather-Queen Charlotte and Transition Fault Seismic Source Zone

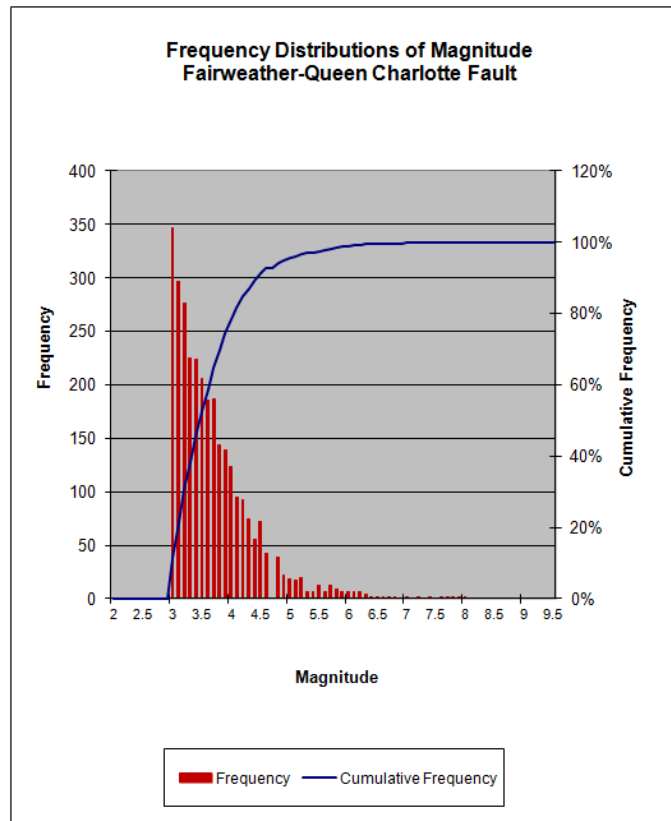


Figure 45 - Gutenberg-Richter Relationship for Fairweather-Queen Charlotte and Transition Fault Seismic Source Zone

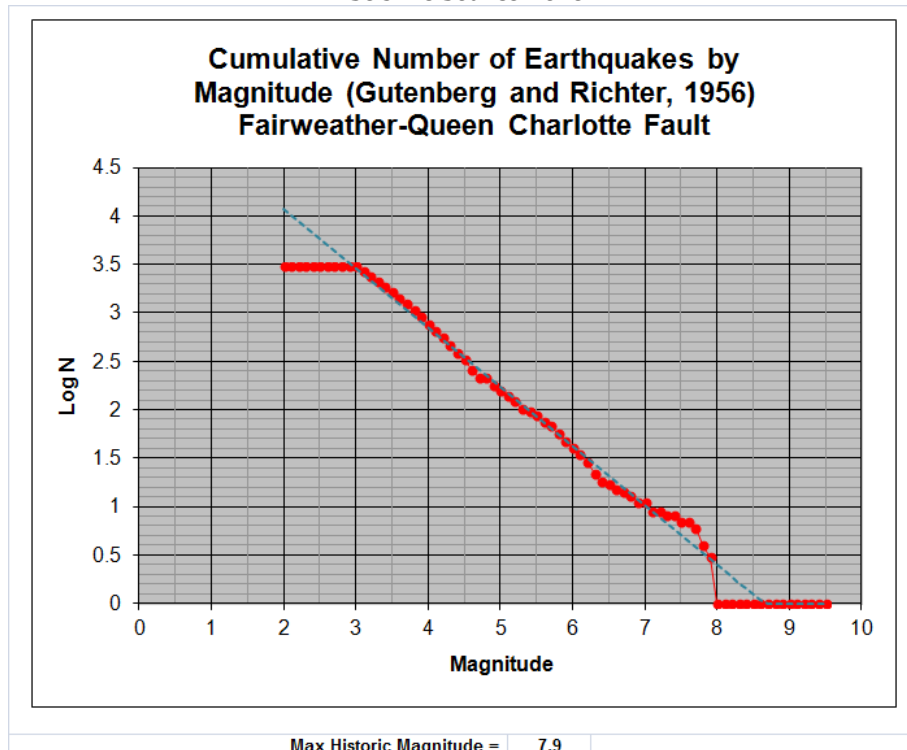
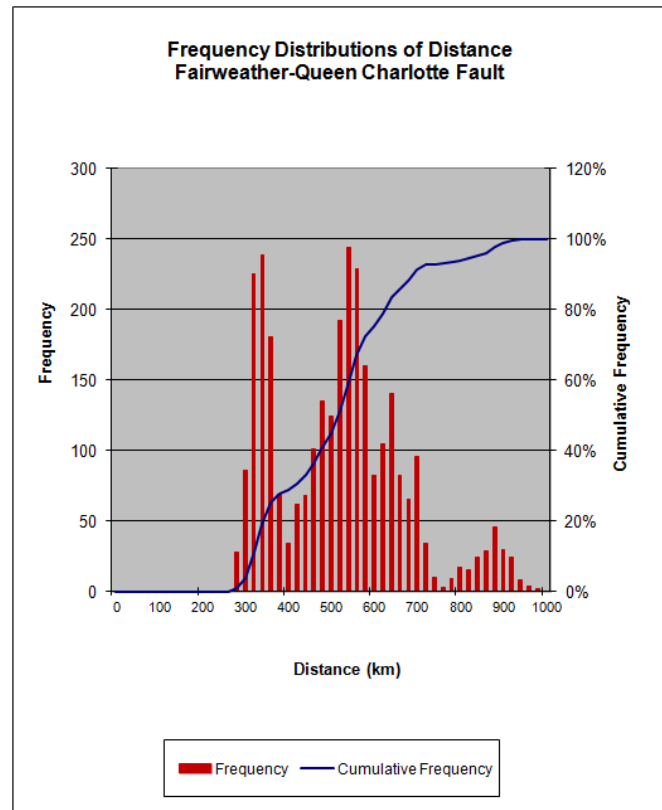


Figure 46 - Frequency Distribution of Distance for the Fairweather-Queen Charlotte and Transition Fault Seismic Source Zone



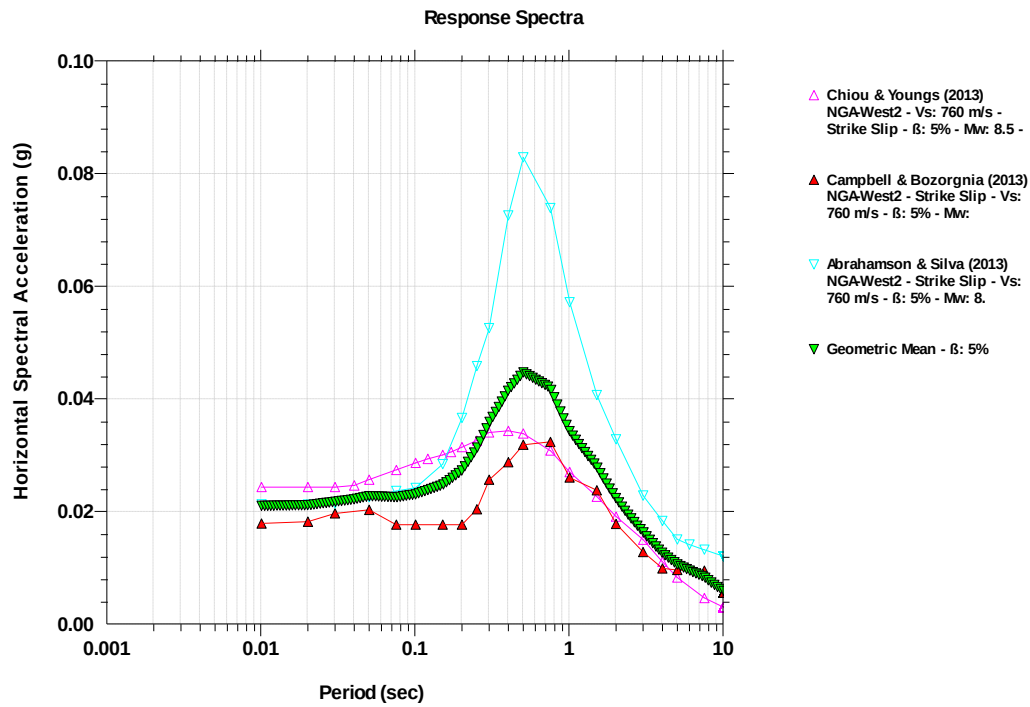
Although the distance falls within the valid range, the magnitude of 8.7 slightly exceeds the magnitude limitation of M8.5. In order to obtain an approximation of expected ground motions, a maximum magnitude of 8.5 was used in the evaluation but rather than the mean parameters, the more conservative estimates of the Abrahamson and Silva NGA West 2 relationship was used to account for the slightly larger MCE of M8.7. Using the maximum magnitude of 8.5 at the minimum distance of 267 km the attenuated ground motion response spectra shown in Figure 47 was generated. Values of pseudo-spectral acceleration were then extracted from this figure (using the Abrahamson and Silva relationship) and are shown in Table 8.

Table 8 - Attenuated Sa vs. Natural Period for the Fairweather-Queen Charlotte and Transition Fault Seismic Source Zone

Natural Period (s)	Spectral Acceleration (g)
PGA (0 s)	.022
0.2	.036
0.5	.084
1.0	.056
2.0	.032

The plot shown in Figure 47 clearly shows the effect of less attenuation of longer period (lower frequency) vibrations at longer site distances.

Figure 47 - Attenuated Sa Response Spectra for Fairweather-Queen Charlotte and Transition Fault Seismic Source Zone



5.7 DSHA Results Summary

Results of analyses for all seismic source zones are summarized in Table 9. Most building codes (including the 2010 National Building Code of Canada (NBCC)) allow for either a site specific deterministic design approach or a probabilistic approach. For the site specific DSHA procedure the estimated spectral acceleration values at the various natural periods as shown in Table 9 are used to develop a design spectral acceleration response spectrum. These design accelerations are then used to determine seismic loads for structural design which will also vary as a function of occupancy and use.

Figure 48 shows a plot of the grouping of spectral acceleration (S_a) vs. natural period (T) for all of the seismic source zones considered including the extrapolated estimates for the Aleutian Megathrust seismic source zone. A final design spectrum is obtained by taking the maximum spectral response acceleration at each natural period attributed to any one of the seismic source zones. However, some special circumstances warrant discussion in the development of the final design spectrum for the Coffee Gold Project site. The first is the Tintina Trench seismic source zone where the Coffee Gold Project site itself is located. The spectral acceleration curve for the minimum recorded site distance of 55 km is shown as the magenta curve on Figure 48. The spectral acceleration curve for an earthquake epicenter at the project site (distance of 1 km) is shown as the red curve on Figure 48. In the absence of any geologic constraints that could preclude earthquake events within the 55 km radius and the assumption that earthquakes are equally likely to occur at any location within seismic source zone it would seem reasonable that the closer (1 km) earthquake control the short period design accelerations. The second issue involves the extrapolated S_a values for the Aleutian Megathrust seismic source zone. As can be seen on Figure 48 the design S_a values are dominated by the Tintina Trench 1 km values until a natural period of approximately 0.75 s when the values for the Aleutian Megathrust zone (shown as a dashed purple line in Figure 48) begin to exceed those from any other seismic source zone. Although the probabilistic aspects of the seismic hazard will be addressed in a following section, it warrants mentioning that the great megathrust interface earthquake event is extremely rare having a high recurrence interval. It would be appropriate to consider this event for important permanent structures with a long expected design life (often 100 yrs +) such as a major dam, a major bridge, or a high rise structure. But the structures anticipated at the Coffee Gold Project are expected to be temporary (an expected mine life less than 20 yrs) and the only permanent structures are likely to be reclaimed earthworks with natural periods likely in the range of 0.1 s to 0.8 s. So the use of the Aleutian Megathrust S_a values would seem to be excessively conservative. The Fairweather-Queen Charlotte and Transition Fault zone begins to dominate spectral acceleration beyond a natural period of 1.6 s.

Seismic design procedures often differ from one code to the next and the NBCC does differ from the procedures of the International Building Code (IBC) commonly used at mining sites around the world. One significant difference is in the procedure for development of the design pseudo-acceleration spectrum. The NBCC for Site Class C assigns values of 1.0 to both the acceleration based site coefficient (F_a) and the velocity based site coefficient (F_v) then applies those coefficients directly to the short and long period values of S_a . The IBC for Site Class C assigns value of 1.2 to F_a and 1.7 to F_v , then applies those coefficients to the short and long period values of S_a and reduces them by 2/3. The different procedures result in significantly different design response spectra as shown in Figure 48 vs. Figure 49 and for the final design spectrum Figure 50 vs. Figure 51. Both spectra have been provided here so that vendors with “standard” facility designs used at other sites can directly compare the differences in design criteria.

Table 9 – Summary of DSHA Results

Seismic Source Zone	Magnitude				Distance (km)			Pseudo-Spectral Acceleration (g)				
	Max Historic	Gutenberg-Richter	Surface Fault Rupture	MCE	Mean	Median	Closest	PGA	0.2 s	0.5 s	1.0 s	2.0 s
Tintina North	6.9	7.2	---	7.2	547	496	118	.026	0.047	0.038	0.022	0.011
Inland Alaska	7.1	7.1	---	7.1	656	633	420	---	---	---	---	---
Tintina Trench at 55 km	4.6	5.0	---	5.0	550	591	55	0.0085	0.0155	0.0065	0.0002	0.0001
Tintina Trench at 1 km	4.6	5.0	---	5.0	---	---	1	0.28	0.53	0.20	0.070	0.020
Denali Fault	7.9	7.3	8.0	8.0	360	350	105	0.06	0.11	0.085	0.050	0.0025
Aleutian Megathrust	9.2	8.3	---	9.2	662	623	364	0.07*	0.153*	---	---	0.092*
Fairweather-Queen Charlotte and Transition Fault	7.9	8.7	8.0	8.7	509	517	267	0.022**	0.036**	0.084**	0.056**	0.032**

Notes: * - Estimated from 2003 Atkinson and Boore equations for ground-motions in megathrust subduction zones

** - Estimated using M8.5 event at 267 km and the Abrahamson and Silva 2013 NGA West 2 attenuation relationship

Figure 48 – DSHA Site Specific Sa Response Spectra by Seismic Source Zone

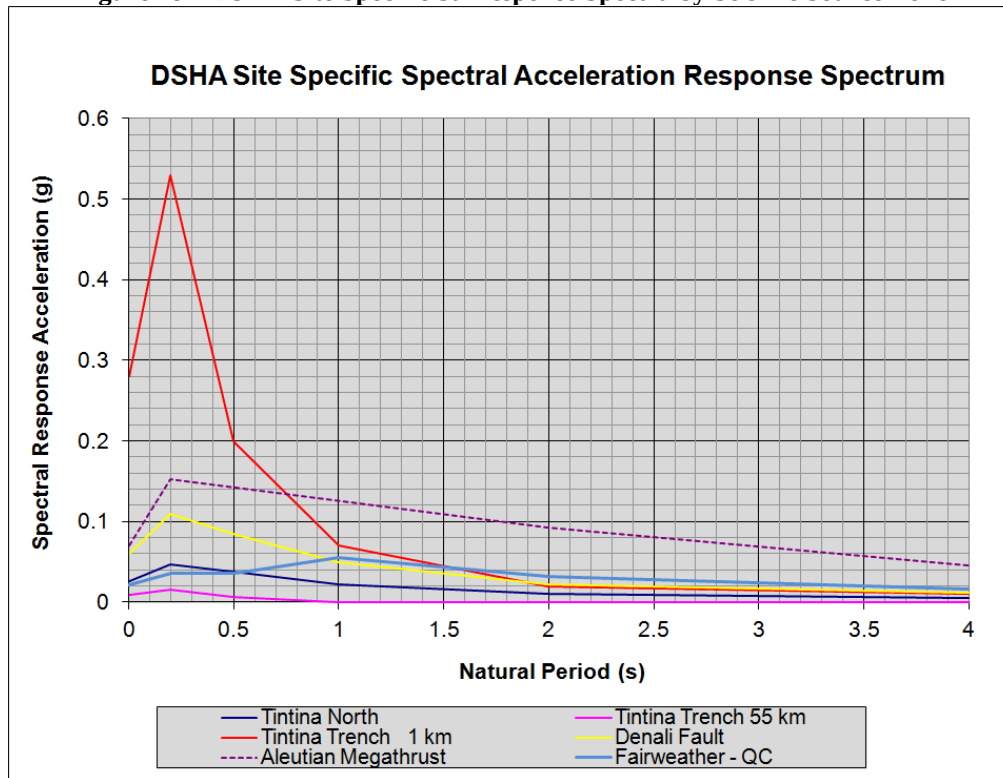


Figure 49 - DSHA IBC General Procedure Sa Response Spectra by Seismic Source Zone

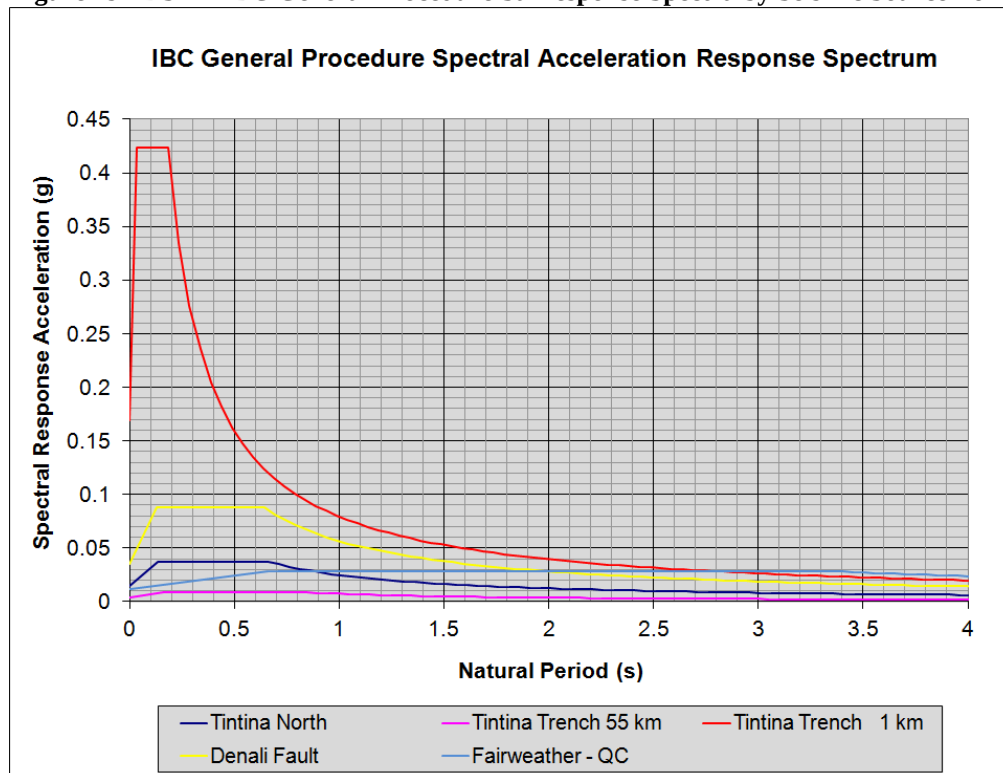


Figure 50 - DSHA Site Specific Sa Design Response Spectra

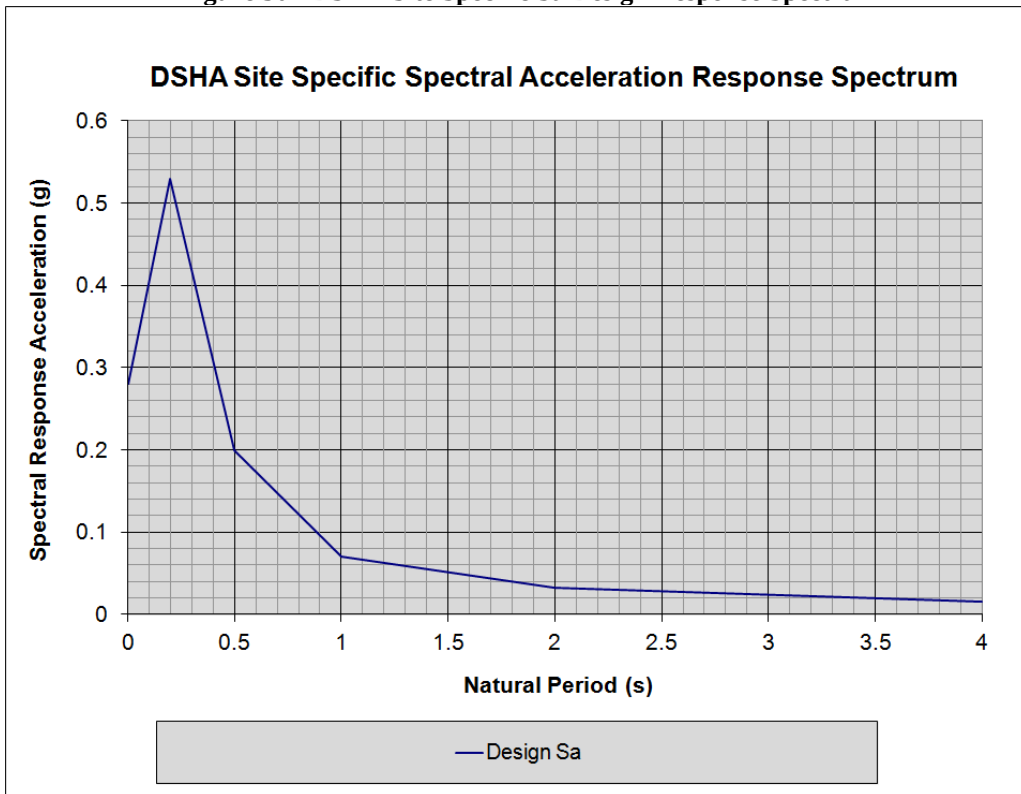
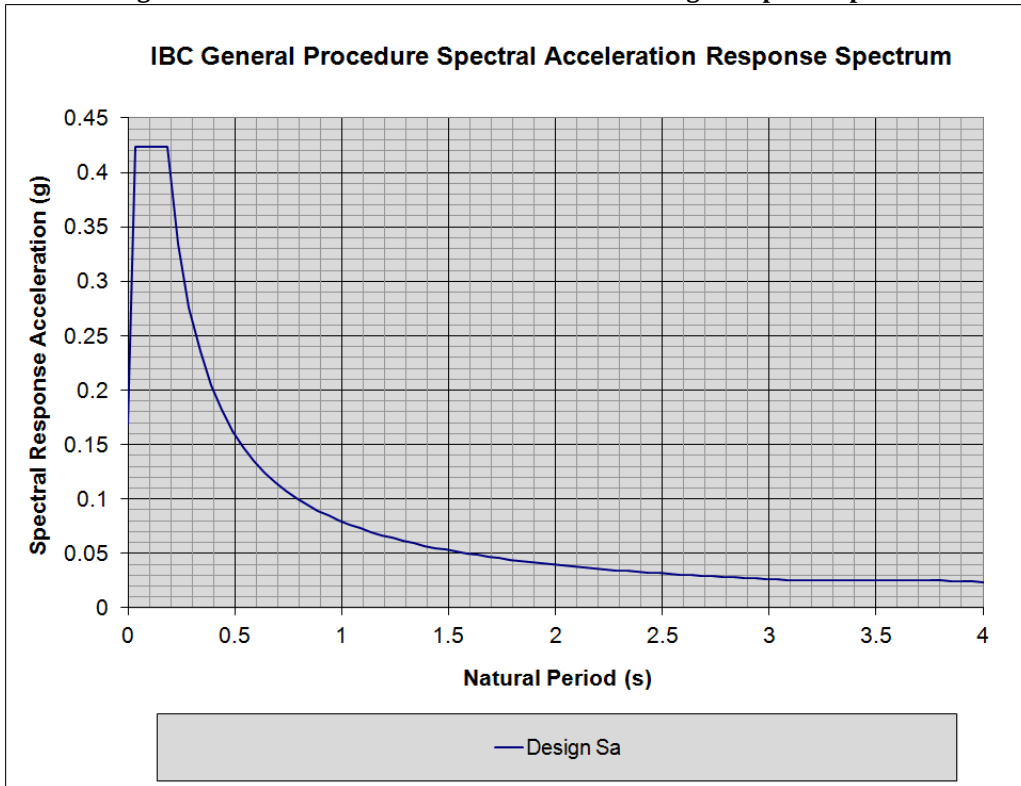


Figure 51 - DSHA IBC General Procedure Sa Design Response Spectra



6 Probabilistic Seismic Hazard Analysis (PSHA)

The PSHA analysis can be performed using the same seismic source zones and by replacing the maximum credible earthquake with the probability distribution of earthquake events, the site distance with the probability distribution of site distances, and adding a random component to the attenuated spectral acceleration values, then using a Monte Carlo type sampling model to compile a new distribution of spectral accelerations associated with an exceedance probability. However, some developed countries, including the U.S. and Canada, have their own web-based PSHA programs that use regionally based maps of seismic source zones and compute site distances by asking you to enter a specific geographic site location using latitude and longitude.

The Canadian PSHA model is located on the Natural Resources of Canada website and was utilized to generate the information found in Appendix A and summarized in Table 10. The results assume a Site Class C (stiff soil and/or soft rock in the upper 30 m of the subsurface profile) and are consistent with the observed site conditions at the Coffee Gold Project site.

In applying the PSHA results to design, there is no longer a single design earthquake event (in the case of DSHA the maximum credible earthquake or MCE). Now it is necessary to choose an appropriate design earthquake that has an associated annual exceedance probability, or its inverse, the recurrence interval. Selection of the appropriate design earthquake is commonly a function of the facilities purpose or function, its importance, and the consequences of failure. In that context, a description of some commonly used definitions would be appropriate.

Earthquake design levels and their definitions can vary with the nature of the facility and the regulatory agency in charge of overseeing their design and operation. Some common definitions will be described here as follows:

- Operating Basis Earthquake (OBE) – Sometimes also referred to as the operating level earthquake (OLE) it is the earthquake for which a structure is designed to remain operational with damage considered to be readily repairable following the event. In terms of probability, it is usually taken to be the earthquake having a 50% probability of exceedance in 50 yrs.
- Design Basis Earthquake (DBE) – Sometimes also referred to as the contingency level earthquake (CLE) it is the earthquake expected to produce significant damage (with the facility not necessarily remaining operational), but with damage that is ultimately repairable. In terms of probability, it is usually taken to be the earthquake having a 10% probability of exceedance in 50 yrs.
- Maximum Credible Earthquake (MCE) – This has already been described under DSHA procedures as the largest earthquake capable of occurring under the known tectonic framework for a specific seismic source. It can sometimes also be referred to as the controlling earthquake.
- Maximum Design Earthquake (MDE) – This is the earthquake that is expected to produce the strongest level of shaking and is often used interchangeably with the MCE event described above, however it is based on *ground motions* **not** earthquake magnitude. It can be determined from deterministic or probabilistic methods.

- **Maximum Considered Earthquake (MCE)** – Often confused with the other MCE (maximum credible earthquake) it is usually intended to be the probabilistic equivalent of the maximum credible earthquake and has meaning only in the context of a PSHA analysis so it is important to make it clear up front, which MCE is being used in any discussion. It is commonly used as the required design criteria in building codes. In terms of probability, it is usually taken to be the earthquake having a 2% probability of exceedance in 50 yrs.

Note: The definition descriptions above have numerous sources, but much of the language was derived from a 2011 presentation by Oregon State University.

Figure 52 plots the DSHA spectral response spectra (shown as a dashed gray line) on the same plot as the PSHA spectral response spectra in order to permit a comparison of the two methods. As expected the design ground motions from the DSHA analysis are similar but more conservative, at least for natural periods less than 0.8 s. The reduced probability of larger earthquakes occurring in very close proximity to the site moderates the distribution of spectral accelerations resulting in reduced short period S_a values from the PSHA. Beyond the natural period of 0.8 s the design S_a is slightly less than both the 2% and 5% exceedance earthquakes (return periods of 2475 yrs and 1000 yrs respectively), implying that these rare, large earthquakes occurring at larger radial distances have a magnitude higher than the MCEs selected for the DSHA analysis.

Table 10 – Summary of PSHA Results

Annual Exceedance Probability	0.01	0.0021	0.001	0.000404
Recurrence Interval (yrs)	100	475	1000	2475
Probability of Exceedance in 50yrs	40%	10%	5%	2%
Period (s)	S_a (g)	S_a (g)	S_a (g)	S_a (g)
PGA	0.038	0.067	0.086	0.116
0.2	0.068	0.123	0.161	0.225
0.5	0.049	0.087	0.112	0.151
1	0.034	0.062	0.078	0.103
2	0.022	0.039	0.048	0.063
4*	0.011	0.0195	0.024	0.0315

Note: * - Per the 2010 NBCC the 4s $S_a = (2s S_a)/2$

Figure 52 – Plot of PSHA Results and Comparison with DSHA Results

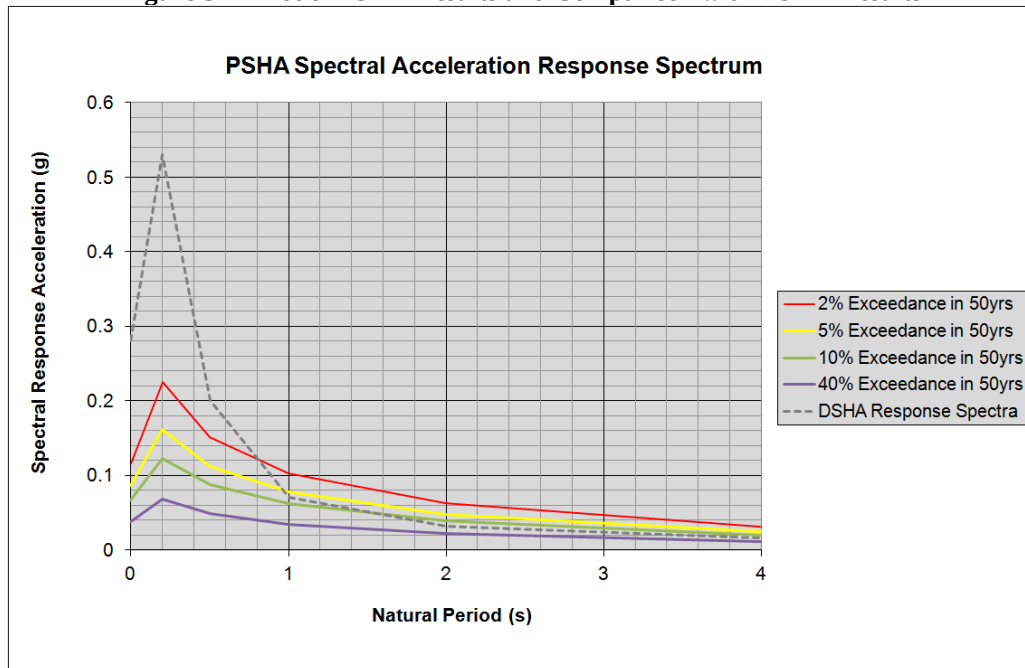
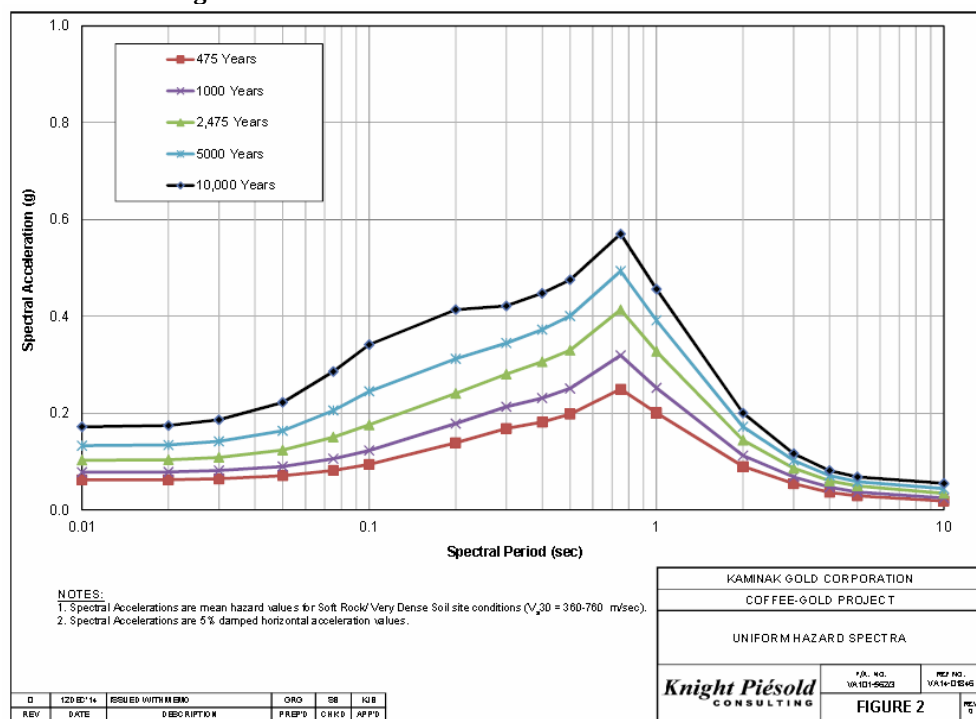


Figure 53 shows the PSHA results from the proprietary EZ-FRISK program taken from the Knight Piesold preliminary seismic study. The S_a values for the 2% exceedance in 50 yrs event are similar up to the 0.2 s natural period, but then go on to peak consistently at a natural period of 0.75 s at a S_a nearly double the 0.2s S_a for all earthquake levels. The reason for this behavior is unclear.

Figure 53 – Plot of PSHA Results from EZ-FRISK Model

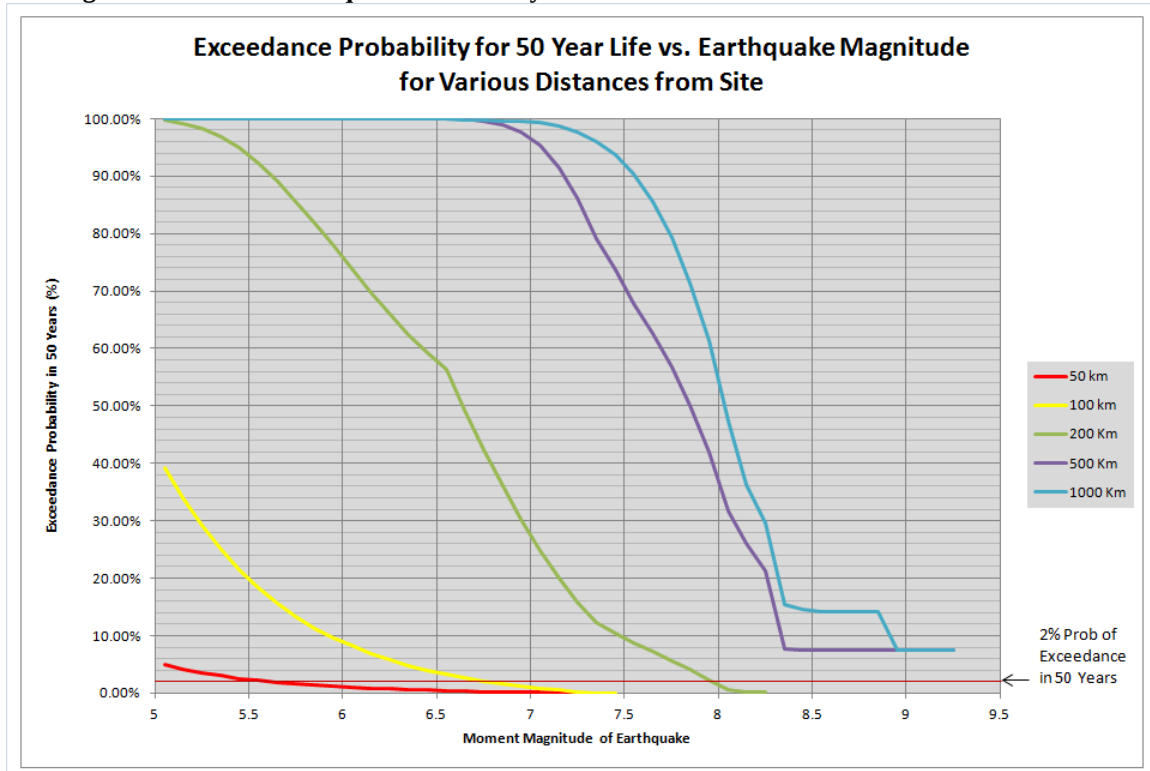


The USGS PSHA web based model functions only within the limits of the United States. However, they do provide custom hazard maps and estimates of earthquake magnitude vs. exceedance probability over a 50 year interval at most world locations based on latitude and longitude coordinates. Figure 54 is a plot of earthquake magnitude vs. Exceedance Probability in 50 years and Table 11 shows the estimated earthquake magnitude as a function of exceedance probability and radial distance from the Coffee Gold Project site.

Table 11 – Summary of Earthquake Probability Level vs. Radial Site Distance from USGS Model

RI (yrs)	Ex. Prob. in 50 yrs	Radial Distance from Site				
		50	100	200	500	1000
100	40%	< 5.0	< 5.0	6.8	7.95	8.1
475	10%	< 5.0	5.9	7.5	8.3	8.9
1000	5%	5.05	6.3	7.8	9.3+	9.3+
2475	2%	5.6	6.8	8.0	9.3+	9.3+
5000	1%	6.05	7.0	8.0	9.3+	9.3+
10,000	0.5%	6.45	7.25	8.1	9.3+	9.3+

Figure 54 - Plot of Earthquake Probability Level vs. Radial Site Distance from USGS Model



7 Seismic Design Criteria

For the design of buildings and other structures at the Coffee Gold Project site the PSHA results shown in Table 10 and Figure 52 for the earthquake event having a 2% probability of exceedance in 50 yrs coupled with the criteria and design procedures for estimating seismic loads found in the 2010 NBCC should be applied for structural design.

The kinds of structures anticipated at the Coffee Gold Project site should generally be associated with natural periods less than 0.8 s. Typical structures could include 1 to 5 story buildings ($T = 0.1$ s to 0.5 s), small concrete gravity structures ($T = 0.3$ s), or earthen structures ($T = 0.1$ s to 0.8 s). Most long natural periods are associated long bridges or tall high rise structures (the Golden Gate bridge with $T = 18$ s or a 70 story skyscraper with $T = 8$ s). A common rule of thumb for moment frame buildings is $T = 0.1 \times (\text{No. of Stories})$. A common rule of thumb for earthen structures is $T = 4H/V_s$, where H is the average height of the potential sliding mass and V_s is the average shear velocity of the sliding mass. As long as the natural period of a structure is less than or equal to 0.8 s the designer can elect to use a site specific deterministic design procedure and remain conservative.

Seismic design in connection with earthen geotechnical structures (earth dams, embankments, excavations, etc.) is usually associated with slope stability analysis. Current practice in slope design incorporates a hierarchy of slope analysis procedures that progress from simple and conservative, to more rigorous but less conservative. The first level in the hierarchy is the pseudostatic analysis. This procedure has a long history apparently dating back to around 1920. It assumes that the impact of earthquake loading on slope stability can be modeled using a conventional limit equilibrium model with a static horizontal force F (equal to mass times acceleration acting **out** of the slope) applied to the centroid of the mass (or in the case of modern method of slices limit equilibrium models, to the centroid of each slice). Vertical acceleration is not considered in the typical pseudostatic analysis because it tends to be smaller than the horizontal component, can act in either an upward or downward direction, and seems to have a very minor impact on the results. Once a seismic acceleration coefficient is selected, a new FOS is computed and, if found to be greater than 1.0, the slope would be considered stable. Although believed to be very conservative, the degree of conservatism was unknown until researchers began investigating the method further in the 1960's and 1970's. The best known of these studies was published in 1979 by Dr. H. Bolton Seed (*Considerations in the Earthquake Resistant Design of Earth and Rockfill Dams*, Geotechnique, Vol. 23, No. 3, 1979). Dr. Seed's conclusions in that paper were that the pseudostatic analysis procedure was in fact very conservative and recommended that, for slopes composed of materials that showed no significant loss of strength as a result of cyclic loading (i.e., no liquefaction), use of the pseudostatic analysis procedure with an appropriate seismic acceleration coefficient and target FOS would assure acceptably small displacements. The seismic acceleration coefficient is not the same as the peak ground acceleration, but is a ratio of some selected design acceleration to the acceleration of gravity (i.e., it is expressed as a percentage of "g"). Dr. Seed tied his recommendations for the design seismic acceleration coefficient to earthquake magnitude recommending 0.05 g for earthquakes with a magnitude of less than 6.5, 0.10 g for earthquakes with magnitude between 6.5 and 7.5, and 0.15 g for earthquakes with magnitude greater than 7.5 and up to 8.25. His study focused on the design of earth and rockfill dams and he recommended obtaining a target factor of safety of 1.15 (allowing for a 15% reduction in strength as a result of shaking).

Beginning in the 1980's it became commonplace for many practitioners seeking a more technically based method for the selection of the seismic acceleration coefficient to select the value as a percentage of the estimated peak ground acceleration (PGA) at the site (Hynes-Griffin and Franklin, 1984), typically using 50% to 60% of the PGA. However, the basis for the 50% or 60% range was pretty arbitrary. One of the more interesting studies pertaining to selection of the seismic acceleration coefficient was done by California consultant Robert Pyke in 1999. Pyke revisited the original work of Seed whose recommendations were actually a simplified interpretation of the displacement analysis shown in Figure 55. Using Seed's threshold of acceptably small displacements, Pyke produced the curve shown in Figure 56 showing the reduction of the estimated PGA (for the purpose of obtaining a seismic acceleration coefficient

for pseudostatic analysis) to range from 20% (0.2) for a magnitude 6.5 earthquake to 50% (0.5) for a magnitude 8.25 earthquake, using intermediate values on the continuous curve for earthquake magnitudes lying between 6.5 and 8.25. Using the Tintina Trench MCE of 5.0 and the associated PGA of 0.28g from the DSHA, application of Pyke's criteria yields a pseudostatic acceleration coefficient of $0.2 \times 0.28 = 0.06g$. Similarly, using the expected PGA of 0.116g associated with the maximum considered earthquake having a 2% probability of exceedance in 50 yrs (expected to be an earthquake with magnitude greater than 8.25) Pyke's criteria yields a pseudostatic acceleration coefficient of $0.5 \times 0.116 = 0.06g$. Therefore, the recommended pseudostatic acceleration coefficient is 0.06g regardless of whether a deterministic or probabilistic approach is used.

Figure 55 - Displacement Analysis Results from Seed, 1979 after Pyke, 1999

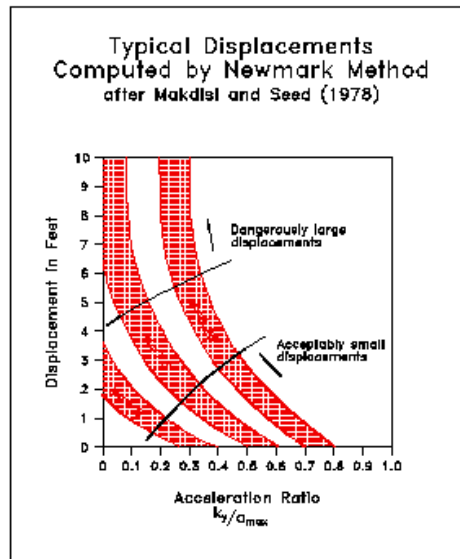
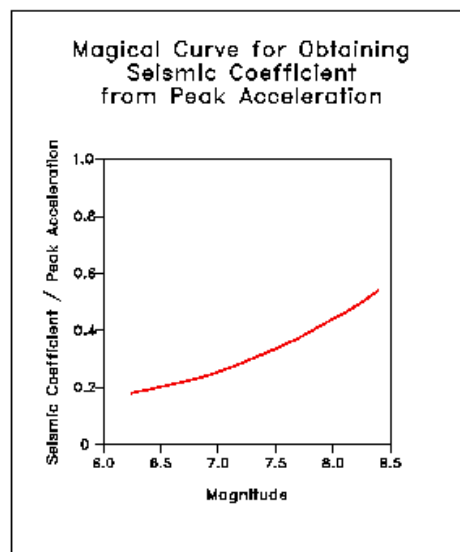


Figure 56 - Pyke's Recommended Reduction of the Peak Ground Acceleration for Selection of a Seismic Acceleration Coefficient as a Function of Earthquake Magnitude



In most cases, if a sufficiently high FOS is computed using the pseudostatic analysis (1.00 for the 1999 Pyke criteria or 1.15 for the 1979 Seed criteria), then it is presumed that displacements during the design earthquake will be acceptably small and no further evaluation of seismic stability impact is necessary. If the FOS is not found to meet the design criteria, then the stability under earthquake loading is considered to be in question, and the designer should proceed to the next level of analysis in the hierarchy to further evaluate seismic stability using a more rigorous analysis procedure.

The second level in the hierarchy of seismic stability analysis is displacement analysis. There are three (3) commonly used procedures for displacement analysis. The first was developed by Newmark and published in 1965 (*Effects of Earthquakes on Dams and Embankments*, Geotechnique, Vol. 15, No. 2, 1965). Although considered a more rigorous analysis method, the Newmark method is also based on a pseudostatic analysis procedure. A simple pseudostatic stability analysis is used to find the critical seismic acceleration coefficient K_c at which the FOS is exactly 1.00. The critical seismic acceleration coefficient is then converted into the yield acceleration a_y which is assumed to be the horizontal acceleration above which the mass will begin to move. An earthquake time history is then selected that is modified and scaled as necessary to represent the expected design earthquake event. An estimate of the cumulative displacement of the potential failure mass is then obtained by mapping the frequency and duration of the acceleration pulses that exceed the yield acceleration and double integrating that portion of the acceleration time history curve to obtain the displacement (i.e., the integral of the acceleration curve yields the velocity, and the integration of the velocity yields the displacement). The cumulative sum of all the increments of displacement represents an estimate of the total permanent displacement.

The second method is a simplified version of the Newmark procedure published by Makdisi and Seed in 1978 (*Simplified Procedure for Estimating Dam and Embankment Earthquake-Induced Deformations*, Journal of Geotechnical Engineering Division, ASCE Vol. 104, No GT7, 1978). It uses the same sliding block type model of Newmark's analysis but utilizes the average (rather than peak) acceleration computed with the procedure of Chopra and the shear beam method. It is very graphically oriented using a plot that relates the average maximum acceleration with the depth of the potential failure surface and another plot of normalized permanent displacement with yield acceleration for earthquakes of various magnitudes.

The third method is a relatively current updated version of a Newmark type of analysis (Bray and Travararou, 2007) that makes use of the more comprehensive database of recorded ground motions since 1965. It continues to make use of the concept of a system's yield coefficient, but accounts for the characteristics of the earthquake event using spectral acceleration (S_a), fundamental (natural) period T_s , and earthquake magnitude M . The procedure can be used stochastically, quantitatively addressing uncertainty in the controlling parameters, or deterministically to evaluate seismic displacement potential for a discrete event.

The displacement methods have no simple pass/fail criteria, nor any stable/unstable threshold. They simply produce an estimate of the maximum permanent displacement and it is up to the designer, owner, and potentially a knowledgeable representative of the regulating agency to decide if that amount of displacement is acceptable or unacceptable. In the case of embankment dams there are often physical consequences to displacement (loss of freeboard, reduction of crest width, embankment cracking that could carry seepage and create a piping risk, etc.). With other non-impounding slope structures detrimental impacts from displacement are less clear. Could the displacement undermine a structure at the top of a slope, or impact a structure at the toe, or cross a critical property boundary? Will pipelines, power lines, or drainage facilities be disrupted?

Mine pit slopes and other non-impounding embankments and slopes often have few direct consequences from displacement. So what represents unacceptable displacement? One way to look at it in slopes of rock or stiff soil would be to limit displacements to something that would not exceed the yield strain (the strain at which peak stress is attained and beyond which the available stress drops quickly to lower residual levels leading to a sudden “brittle” failure and large unpredictable plastic strains). For many intact rock and stiff soil materials the yield strain might be in the range of 0.5% to 1.0%. Therefore, for a small failure plane of 30 m in length, the yield point could potentially be reached with a cumulative displacement of only 15 to 30 cm, or for a larger failure plane of 300 m in length, with a cumulative displacement of 1.5 to 3 m. This sort of approach is similar to the criteria used by Seed as shown in Pyke’s graph (Figure 55) that shows dangerously large displacements beginning in a range of 4 to 6 ft (1.2 to 1.8 m) and becoming clearly unpredictable in the range of 8 to 10 ft (2.4 to 3 m) as displacements are increasing rapidly with little or no increase in acceleration ratio. Interpretations of what represents an acceptably small displacement threshold vary widely in the literature typically ranging from about 5 cm to about 100 cm (1 m). However for large pit slopes or high embankments where failure plane lengths can be as large as 500 to 1000 m long, displacements on the order of 5 m to 10 m can be acceptable. In weaker more ductile materials like soil and soft rock, there may not be any peak shear strain behavior, and therefore no specific limiting displacement. At the very least, the comparison of an estimated cumulative axial strain (using the calculated Newmark displacement divided by the failure plane length) to a threshold axial failure strain in intact rock should be conservative. A fractured rock mass is likely to have a much larger peak failure strain (commonly between 1% and 5%).

So what if a displacement analysis shows the expected displacements to be unacceptable? Must the slope then be considered seismically unstable? Not necessarily. The displacement analysis procedures still remain a pseudostatic type of analysis that estimate displacements using models based on the application of unidirectional static loads on rigid bodies and are an approximation of the expected behavior during earthquakes. For critical structures, the last level in the hierarchy of seismic stability analysis tools is a full dynamic analysis. This approach attempts to quantify the true dynamic shear strength and pore pressure response to earthquake shaking using a program such as the SHAKE program (1-D) at one extreme or something like the PEER OpenSees program (3-D) at the other extreme and an appropriately selected and scaled acceleration time history representing the design earthquake. Permanent deformations are estimated using some form of numerical continuum model (i.e., a finite element, finite difference, or discrete element model of the structure/slope). Dynamic modeling tends to be very slow and expensive due to the more extensive data requirements and the complexity of setting up the models. Dynamic modeling is rarely used in mine pit design or non-impounding embankment design. Although more precise in the manner in which earthquake motions and material shear strength behavior are treated, the final result of the modeling is still a displacement time history and has all the same limitations of subjective interpretation associated with the other displacement methods (i.e., what is an acceptable amount of displacement?).

Seismic risk must necessarily consider the potential consequences of the seismic hazard. The nature of the potential impacts vary depending on whether facilities are considered temporary, have a fixed life, or are considered permanent. Facilities can be impacted in a number of ways including surface ground rupture, strong shaking, liquefaction, and seismically induced instability. Other potential impacts not considered pertinent to this site, and therefore not considered, include seiches and tsunamis.

All fixed structures at the site will be impacted by strong ground motion independent of their position at the site. For most facilities, the magnitude threshold at which some level of damage

might be expected is about 5.0 with the severity of damage increasing with increasing magnitude. Probability of impact is simply the probability of the occurrence of the event.

Liquefaction, or the sudden loss of strength in the foundation soils supporting structures, typically begins at a magnitude threshold on the order of 6.0. However, the phenomenon requires not only strong ground motion, but also the presence of loose, saturated, non-cohesive soil. Although these conditions can exist locally where deeply weathered pockets in the bedrock have been filled with loose granular soil, the conditions would be rare and isolated. A detailed evaluation of site specific liquefaction risk is beyond the scope of this study.

Potential impacts from ground rupture during seismic events affect primarily permanent, fixed linear facilities such as roads, channels, pipelines, and so on. Surface ground rupture has been documented during events as small as magnitude 3.6 (Imperial, California, 1966), however, as a general rule, surface fault rupture requires events larger than magnitude 5.5. No known active faults exist on the site, and therefore the risk of impact from surface fault displacement is considered low.

Another potential consequence of a major seismic event is the occurrence of seismically induced instability. David Keefer of the USGS in Menlo Park, California has prepared most of the work on seismically induced slides. His initial study involved a review of 15 historic earthquakes (Keefer, Wiczorek, Harp, and Tuel, 1979) but was subsequently updated and expanded to include 40 historical earthquakes distributed worldwide (Keefer, 1984). His findings showed that the most common seismically induced events included rockfalls, disrupted soil slides, and rockslides. The minimum magnitude threshold required to induce events of this type was 4.0, however, the frequency of slides was not significant until magnitudes on the order of 5.5 were experienced. Magnitude 5.5 earthquakes caused a few tens of slides at most, while earthquakes with magnitudes exceeding 8 caused several thousands of slides at the very least. The area experiencing seismically induced slides varied from 200 km² at magnitude 5.5 to more than 100,000 km² at magnitude 8.0. The risk of seismically induced instability at the site is low due to the expected low probability/high recurrence interval of a magnitude 5.5+ earthquake. However, a magnitude 5.5+ earthquake near the site would likely cause increased rockfall and raveling on existing mine benches and other slopes of exposed rock.

8 Conclusions and Recommendations

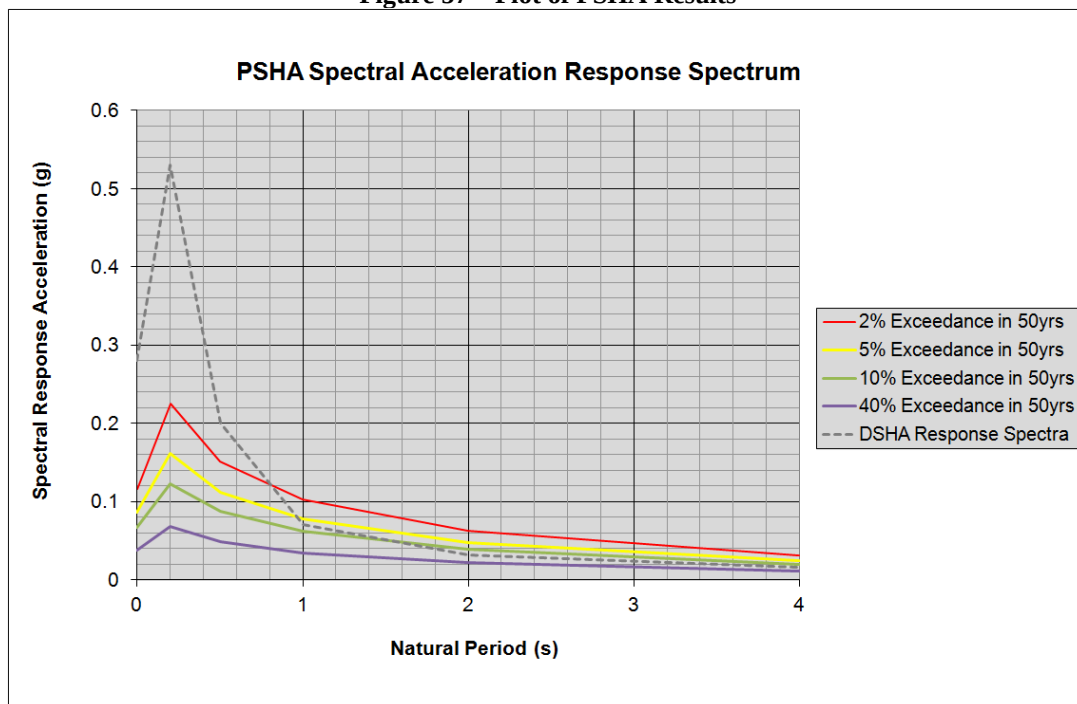
A deterministic approach to seismic hazard analysis or DSHA and a probabilistic approach or PSHA have produced similar design pseudo-acceleration response spectra. For short natural periods between 0.0 s and 0.8 s the deterministic curve is the more conservative (see Figure 57 and Table 12). Beyond a natural period of 0.8 s the PSHA results for a maximum considered earthquake having a 2% probability of exceedance in 50 yrs exceed the DSHA results. This is believed to be the result of changes in the 2010 edition of the NBCC code for PSHA resulting in a reduction in the PGA and short-period spectral values and an increase in the long-period values as well as the influence of larger high recurrence interval earthquakes at larger distances from the site. For most of the structures anticipated to be constructed at the site, use of the PSHA results for the maximum considered earthquake having a 2% probability of exceedance in 50 yrs is recommended.

Table 12 – Summary of PSHA Results

Annual Exceedance Probability	0.01	0.0021	0.001	0.000404
Recurrence Interval (yrs)	100	475	1000	2475
Probability of Exceedance in 50yrs	40%	10%	5%	2%
Period (s)	Sa (g)	Sa (g)	Sa (g)	Sa (g)
PGA	0.038	0.067	0.086	0.116
0.2	0.068	0.123	0.161	0.225
0.5	0.049	0.087	0.112	0.151
1	0.034	0.062	0.078	0.103
2	0.022	0.039	0.048	0.063
4*	0.011	0.0195	0.024	0.0315

Note: * - Per the 2010 NBCC the 4s Sa = (2s Sa)/2

Figure 57 – Plot of PSHA Results



For seismic slope stability analyses in soil and rock, a hierarchy of analysis methods should be implemented with progression to the next method in the hierarchy required only in the event of failure to satisfy the requirements of the previous method. Recommended methods in the order of their application are as follows:

- Pseudostatic stability analysis using a pseudostatic acceleration coefficient of 0.06 g (to be used only at sites with no liquefaction potential).
- Seismic displacement analysis using the procedures of Newmark, 1965, Makdisi and Seed, 1978, or Bray and Travarasrou, 2007 showing acceptably small displacements.
- Full dynamic analysis of soil-structure interaction coupled with continuum modeling showing acceptably small displacements.

9 References

Atkinson, G. M. and Boore, D. M., "Empirical Ground-Motion Relations for Subduction Zone Earthquakes and Their Application to Cascadia and Other Regions", *Bulletin of the Seismological Society of America*, Vol. 93, No. 4, pp. 1703-1729, August 2003.

Baker, J. W., "An Introduction to Probabilistic Seismic Hazard Analysis (PSHA)", a report prepared for the Nuclear regulatory commission, Version 1.3, October 2008.

Bray, J. D. and Travasarou, T., "Simplified Procedure for Estimating Earthquake-Induced Deviatoric Slope Displacements", *Journal of Geotechnical and Geoenvironmental*, Vol. 133, No. 4, April 1, 2007.

Eberhart-Phillips, Donna, et al. "The 2002 Denali fault earthquake, Alaska: a large magnitude, slip-partitioned event." *Science* 300.5622 (2003): 1113-1118.

Hanks, T. C. and Cornell, C. A., "Probabilistic Seismic Hazard Analysis: A Beginner's Guide", *Proceedings of the Fifth Symposium on Current Issues...*, 1994.

Hynes-Griffin, M. E. and Franklin, A. G., "Rationalizing the Seismic Coefficient Method", No. WES/MP/GL-84-13. Army Engineer Waterways Experiment Station Vicksburg MS Geotechnical Lab, 1984.

Keefer, D. K., et al. "Preliminary assessment of seismically induced landslide susceptibility" Progress on seismic zonation in the San Francisco Bay region: *US Geological Survey Circular* 807, pp. 49-60, 1979.

Keefer, D. K. "Landslides caused by earthquakes." *Geological Society of America Bulletin* 95.4, pp. 406-421, 1984

Kyrou, K. "Seismic Slope Stability Analysis.", *Proceedings of the International Conference on Earthquake Risk Minimization*, edited by: G. Petrides, Chrysostomou, K. Kyrou, Hadjigeorgiou, Geological Survey Department, Nicosia. Eds. G. Petrides, and K. Kyrou, 2002.

Makdisi, F. I. and Seed, H. B., "Simplified Procedure for Estimating Dam and Embankment Earthquake-Induced Deformations", *Journal of Geotechnical Engineering Division*, ASCE Vol. 104, No GT7, 1978

Newmark, N. M., "Effects of Earthquakes on Dams and Embankments", *Geotechnique*, Vol. 15, No. 2, 1965

Pyke, Robert, "Selection of Seismic Coefficients for Use in Pseudo-Static Slope Stability Analysis" web based article, bobpyke@tagasoft.com, 1999.

Seed, H. B., "Considerations in the Earthquake Resistant Design of Earth and Rockfill Dams", *Geotechnique*, Vol. 23, No. 3, 1979.

Slemmons, D. B., "State-of-the-Art for Assessing Earthquake Hazards in the United States. Report 6. Faults and Earthquake Magnitude", Report prepared for the U. S. Army Corps of Engineers, May 1977.

Wells, D. L. and Coppersmith, K. J., “New Empirical Relationships among Magnitude, Rupture Length, Rupture Width, Rupture Area, and Surface Displacement”, *Bulletin of the Seismological Society of America*, Vol. 84, No. 4, pp. 974 – 1002, August 1994.

Wentworth, et. al., “Seismic Environment of the Sodium Pump Test Facility at Burro Flats, Ventura County, California”, *Open-File Report, U. S. Geological Survey*, 1969.

Zhang, P., Mao, F., and Slemmons, D. B., “Rupture Terminations and Size of Segment Boundaries from Historical Earthquake Ruptures in the Basin and Range Province”, *Tectonophysics*, Elsevier Publishing, 1999.

Appendix A
Results of PSHA from National Resource of Canada Web
Based Modeling Procedure

2010 National Building Code Seismic Hazard Calculation

INFORMATION: Eastern Canada English (613) 995-5548 français (613) 995-0600 Facsimile (613) 992-8836
Western Canada English (250) 363-6500 Facsimile (250) 363-6565

Requested by: ,

August 06, 2015

Site Coordinates: 62.8745 North 139.1819 West

User File Reference: Coffee Project Site

National Building Code ground motions:

2% probability of exceedance in 50 years (0.000404 per annum)

Sa(0.2)	Sa(0.5)	Sa(1.0)	Sa(2.0)	PGA (g)
0.225	0.151	0.103	0.063	0.116

Notes. Spectral and peak hazard values are determined for firm ground (NBCC 2010 soil class C - average shear wave velocity 360-750 m/s). Median (50th percentile) values are given in units of g. 5% damped spectral acceleration (Sa(T), where T is the period in seconds) and peak ground acceleration (PGA) values are tabulated. Only 2 significant figures are to be used. *These values have been interpolated from a 10 km spaced grid of points. Depending on the gradient of the nearby points, values at this location calculated directly from the hazard program may vary. More than 95 percent of interpolated values are within 2 percent of the calculated values.*

Ground motions for other probabilities:

Probability of exceedance per annum	0.010	0.0021	0.001
Probability of exceedance in 50 years	40%	10%	5%
Sa(0.2)	0.068	0.123	0.161
Sa(0.5)	0.049	0.087	0.112
Sa(1.0)	0.034	0.062	0.078
Sa(2.0)	0.022	0.039	0.048
PGA	0.038	0.067	0.086

References

National Building Code of Canada 2010 NRCC no. 53301; sections 4.1.8, 9.20.1.2, 9.23.10.2, 9.31.6.2, and 6.2.1.3

Appendix C: Climatic Information for Building Design in Canada - table in Appendix C starting on page C-11 of Division B, volume 2

User's Guide - NBC 2010, Structural Commentaries NRCC no. 53543 (in preparation)
Commentary J: Design for Seismic Effects

Geological Survey of Canada Open File xxxx
Fourth generation seismic hazard maps of Canada: Maps and grid values to be used with the 2010 National Building Code of Canada (in preparation)

See the websites www.EarthquakesCanada.ca and www.nationalcodes.ca for more information

Aussi disponible en français

