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REPORT ON

FEASIBILITY-LEVEL

PIT SLOPE DESIGN CRITERIA

OSISKO CANADIAN MALARTIC PROJECT

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EXECUTIVE SUMMARY

Introduction

This report documents the results of Golder Associates' pit slope design evaluation in support of the Feasibility Study for Osisko Mining Corporation's Canadian Malartic Mine in Quebec. The project site is an area of historic underground mining that has been drilled extensively for the evaluation of underground and selective open pit mining in the past. The deposit is an Archean porphyry gold system hosted in diorite porphyry and metasedimentary rocks.

The scope of work included: assessment of large-scale slope stability to design overall slope angles; assessment of bench-scale and multi-bench stability and inter-ramp slope design; assessment of potential risks to stability and operations associated with underground mine workings; recommended design inter-ramp angles and design bench configurations; recommendations for a geotechnical program during pit development; evaluation of the potential effect of a planned waste rock dump on the stability of the southeast wall of the pit; and recommendations for a monitoring program to support operational safety and security.

Ultimate Pit Plan

The proposed pit has an elliptical shape, with an east-west axis approximately 2,000 m long, and a north-south axis 750 m long (Figure ES-1). Wall heights are a maximum of 380 m. The proposed design provided by Osisko includes inter-ramp slope angles of 53°, and overall slope angles ranging from 53° in the west wall to 47° in the north wall. The pit will be developed with 10 m high production benches based on grade control and operating equipment considerations.

Site Conditions

Geology

The two main rock types in the deposit area are porphyry, and metasedimentary (collectively referred to as "Greywacke," although mudstones and shales also occur) rocks. In addition to the lithologic distinction, geological classification of these units is also based on alteration.

Mafic dikes up to several meters wide that crosscut the area have been regionally metamorphosed to schists. Felsic dikes that may be related to late stages of emplacement of the porphyry are also present.

Overburden consists of glaciolacustrine deposits, till, and local swampy areas containing peat, and is generally believed to be less than 10 m thick in the pit area

Structure

Much of the structure in the deposit area is related to regional deformation that produced schistosity, overturned folds, and conjugate faulting. Modelling of the locations and orientations of the faults and other structural elements is in progress.

Faults are generally characterized by narrow zones of brittle fracturing, usually a few centimeters to less than one meter thick. Clay gouge is present locally, but rarely.

Hydrogeology

A hydrogeological study completed by Golder Associates in support of the Feasibility Study indicates water table elevations in the pit area are near surface, and groundwater flow is generally to the north, except in the vicinity of underground workings, where flow is toward the workings.

Available Data

Data available to support this study include information collected during the geotechnical field investigation; data from development coreholes; geologic maps of the underground workings; historical documents provided by Osisko; and the draft hydrogeological study performed by Golder Associates.

The field investigation began in early November 2007 and continued through January 2008, and included five corehole drilled for geotechnical purposes, and data collection from five development coreholes.

Engineering Characterization

An engineering geologic characterization of the deposit incorporates the following main components:

- Pit geology;
- Geological structure;
- Material properties;
- Rock mass properties; and
- Hydrogeologic conditions.

All rock units at Osisko classify as strong to very strong rock, and generally have low fracture density. All of the units classify as Good Quality rock masses except for SCH, which classifies as a Fair Quality rock mass.

The main structural sets in the Osisko pit area consist of:

- Sladen Fault/greywacke-porphyry contact and parallel structures. Dip ~45°-60°S in the central and east areas of the north wall, subvertical in west area of north wall.
- Steep, northeast-dipping Northwest Conjugate Fault and parallel structures
- Moderately south-east-dipping “Intersection” faults
- Steep, north-dipping bedding
- Northwest-southeast to west-northwest striking, subvertical foliation

Stability Evaluation

Rock mass and kinematic stability analyses indicate that rock quality and structural conditions generally appear favorable for the development of steep inter-ramp slopes in all areas of the pit except the Northeast sector. Specific conclusions supported by the characterization and analyses include:

- Rock mass strength is sufficiently high to preclude rock mass failure of overall slopes.
- Groundwater is not expected to be a control of rock mass stability.
- The waste dump can be located within 50 m of the pit crest with no significant effect on pit slope stability.

- Kinematic stability analyses indicate potential for significant structurally-controlled planar failures along moderately south-dipping structures in the Northeast sector, and bench face angles should be designed at 60° to reduce back-break along these structures.
- Bench and slope stability in the Northeast sector will be sensitive to variations in the dips of in-dipping structures and also to the presence of groundwater pressures.
- In other pit sectors, steep bench face angles should generally be achievable with careful blasting, excavation, and scaling.
- Available data indicates that the schist (metamorphosed mafic dikes) is competent rock.

Slope Design Recommendations

Overburden, All Areas

Overburden is less then 10 m thick and consists of glaciolacustrine deposits, till, and local soft sediment deposits. Overburden should be excavated at 2(H):1(V). The first bench, which will be excavated partially or entirely in overburden, should be a single bench and should incorporate an 8 m catch bench.

Recommended Bedrock Slope Designs

The following design slope configurations are recommended for feasibility-level pit slope designs (Figure ES-2).

RECOMMENDED SLOPE DESIGNS IN BEDROCK					
Wall	**Operating Practices**	**Bench Configuration and Height (m)**	**Catch Bench Width (m)**	**Bench Face Angle (°)**	**Design Inter-Ramp Slope Angle (°)**
All sectors except Northeast sector	Buffer Blasting	Double Bench 2 x 10 m 20 m between catch benches	9	69	50
All sectors except Northeast sector	Pre-Split	Double Bench 2 x 10 m 20 m between catch benches	8.5	75	55
Northeast sector	Controlled Blasting to Break Cleanly Along Structure	Double Bench 2 x 10 m 20 m between catch benches	8	60	46

Incorporate a haul road or wide bench below slopes 200 m to 250 m high to de-couple the slopes.

This is an aggressive slope design that will only be successful where geological conditions are favourable and a high level of skill is applied to perimeter blasting and scaling.

Water Pressures in Slopes

Natural draindown that will develop in the slopes due to the dewatering necessary for operating purposes will likely be adequate for most slopes. However, structurally controlled failures that could develop in the Northeast sector would be sensitive to the presence of groundwater pressures, and these slopes should be effectively de-pressurized in the vicinity of the pit wall to enhance stability.

Risks

Bench scale failures can be expected locally where structural conditions are unfavourable or blasting practices are poor. Additionally, seasonal variations in temperature such as winter freeze and spring thaw may adversely affect bench stability and increase rockfall hazards.

The west half of the Northeast sector should be considered a higher risk area because structural conditions are not well understood and available data are conflicting; careful monitoring and documentation of this area is critical for maximizing stability and optimizing design during slope development.

Abandoned underground stopes and drifts will represent an operating hazard and may require modification to slopes designs.

The risks associated with slope development increase with inter-ramp slope height. It is common practice to break steep inter-ramp slopes with a ramp or stability bench at intervals of 200 m to 250 m to mitigate these risks.

Opportunities

Steeper slope angles may be feasible by optimizing these slope designs based on the documented geological conditions and actual performance achieved in the field. Excellent field performance may warrant increasing the design bench face angle, or slightly reducing the design catch bench width.

There is potential for increasing the bench face angle in the Northeast sector to greater than 60° if the south-southeast-dipping structures are not well developed and benches do not break back along them.

Triple benching may be possible if clean and stable bench faces are developed that would enable safe drilling and excavation of a third bench.

Operational Considerations

Good quality operational practices will be essential for the safe development of stable and steep slopes. The slope design recommendations assume that a work force and supervisors skilled in implementing effective controlled blasting and excavation procedures will be available throughout the mining operations.

Specific recommendations related to operational considerations include:

- Double-benching to maximize slope angles;
- Controlled blasting with pre-split blasting to maximize slope angles
- Thorough bench cleanup and scaling using equipment that can safely reach to the crest of the bench.

Planning Around Underground Workings

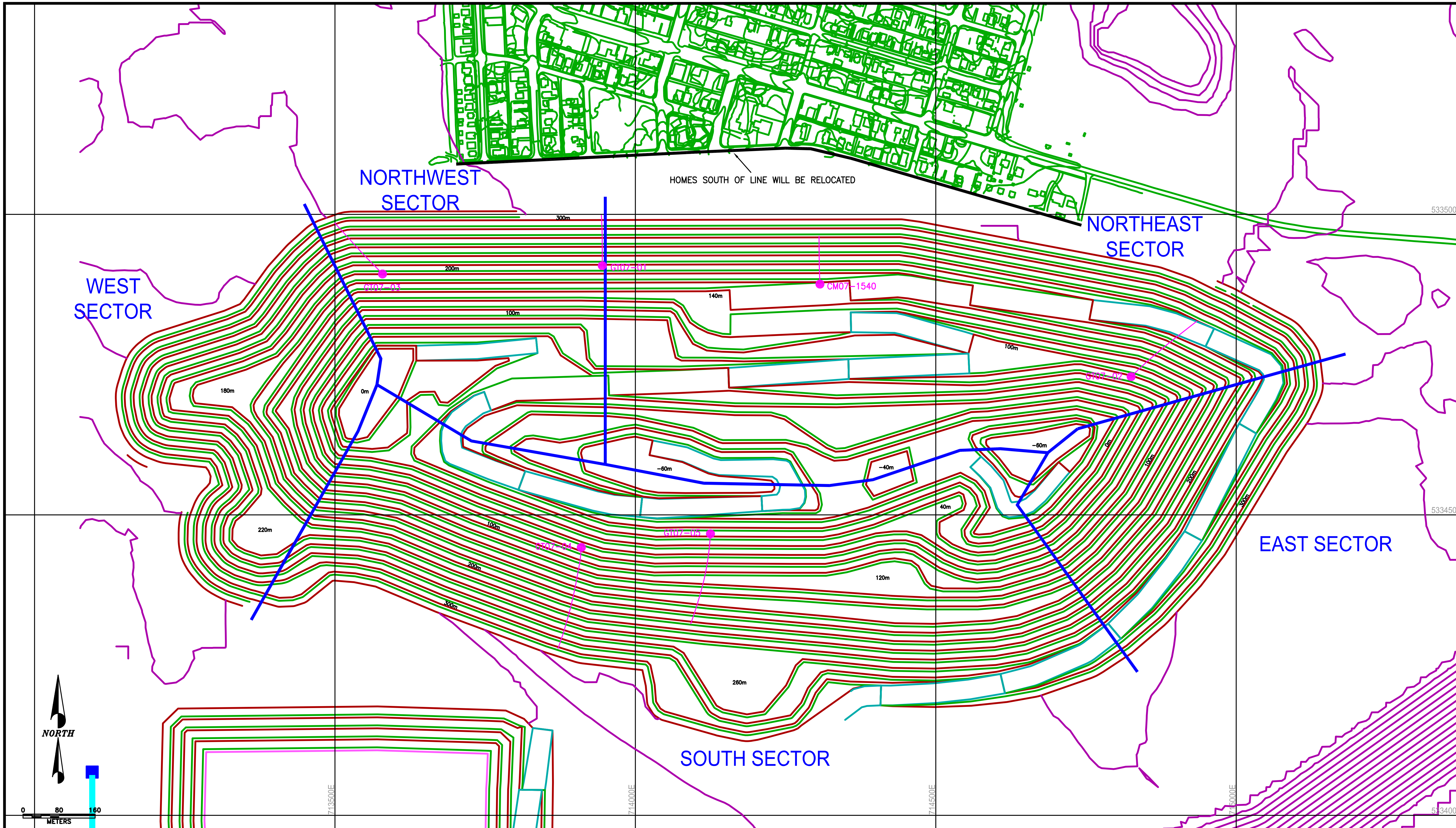
Local modifications to the slope design will be required for safe and stable excavations in areas where stopes intersect the pit wall or floor, or drifts run parallel to the pit wall. Slopes in these areas should be developed with care to ensure the safety of personnel and prevent equipment damage due to collapsing stopes and drifts. Modifications may include excavating the final wall behind stopes and drifts; re-grading overhanging slopes to safe, stable angles; or backfilling the stopes with cemented backfill.

Monitoring and Future Geotechnical Work

There are still significant uncertainties in the extent to which geological structures will be developed, and the manner in which they will affect slope stability, particularly in the Northeast sector of the pit.

The risks associated with these uncertainties should be managed by a program of ongoing geotechnical documentation and monitoring, including:

- Updating the geotechnical model as additional data becomes available prior to mining
- Pit documentation
- Slope monitoring, including
  - Visual inspection
  - Surface displacement monitoring
  - Subsurface displacement monitoring
  - Water level monitoring and monitoring of piezometric pressures in the Northeast sector
  - Blasting-related monitoring



Explanation

GT07-02 Geotechnical Corehole

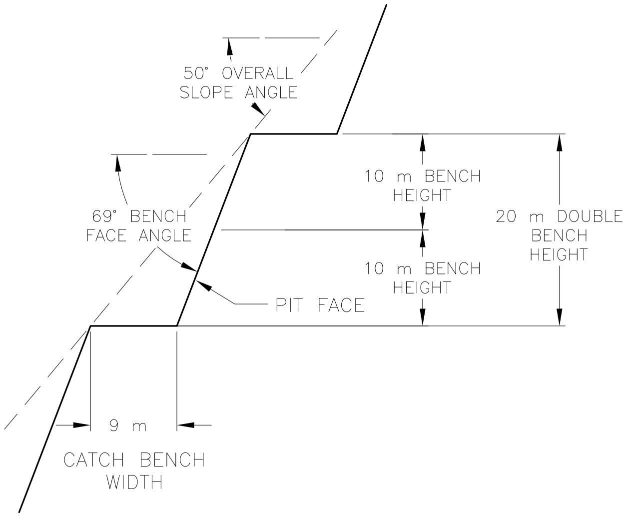
Sector Boundary

Golder Associates
RENO, NEVADA

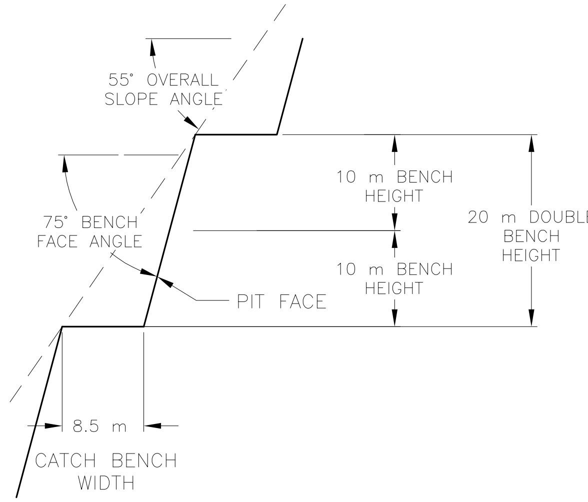
PROJECT No. 07-1221-0028 DATE 06/30/08

ULTIMATE PIT PLAN SHOWING DRILLHOLE AND SECTOR LOCATIONS

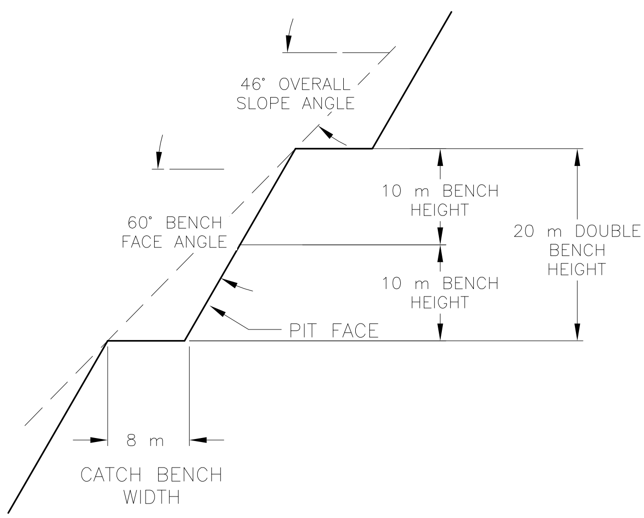
FIGURE ES-1



ALL SECTORS EXCEPT NORTHEAST SECTOR
GOOD BUFFER BLASTING



ALL SECTORS EXCEPT NORTHEAST SECTOR
EFFECTIVE PRE-SPLIT BLASTING



NORTHEAST SECTOR

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TITLE
RECOMMENDED DESIGN SLOPE CONFIGURATIONS

CLIENT/PROJECT
OSISKO
CANADIAN MALARTIC PROJECT

DRAWN RJT	DATE 07/07/08	JOB NO. 07-1221-0028
CHECKED GM	SCALE AS SHOWN	DWG NO./REV. NO. S:\OSISKO\REPORT...\FIGURE ES-2.DWG
REVIEWED	FILE NO.	FIGURE ES-2

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1.0 INTRODUCTION

1.1 General

Osisko Mining Corporation is completing a Feasibility Study to evaluate development of their Canadian Malartic Mine in Quebec by open pit methods (Figure 1). Golder Associates, Inc. (“Golder”) is pleased to present this pit slope design evaluation in support of the Feasibility Study. The project site is an area of historic underground mining that has been drilled extensively for the evaluation of underground and selective open pit mining in the past. The deposit is an Archean porphyry gold system hosted in diorite porphyry and metasedimentary rocks.

1.2 Scope of Work

The scope of work for this study was outlined in our 21 November, 2007 Technical Memorandum to Paul Johnson of Osisko Mining Corporation (“Osisko”) entitled “Osisko Canadian Malartic Project Pit Slope Geotechnical Program,” and included the following:

- Documentation of the geotechnical model on which the slope design recommendations are based;
- Assessment of large-scale slope stability to design overall slope angles and evaluate risks of slope failures that could affect facilities beyond the slope crest and;
- Analysis of the effects of bedding or other structures identified in the unsilicified sedimentary rock unit in the south slope by limit equilibrium or numerical modeling;
- Assessment of bench-scale and multi-bench stability that could affect operations within the pit and inter-ramp slope design;
- Perimeter blast designs to achieve the recommended design bench configuration;
- Assessment of potential risks to stability and operations associated with underground mine workings;
- Recommendations for phase pit slopes based on phase pit designs;
- Recommended design inter-ramp angles and design bench configurations based on geological conditions and the operating bench height developed from the block model;
- Recommendations for a geotechnical program during pit development to mitigate risks and to support optimization of slope designs; and
- Recommendations for a monitoring program to support operational safety and security of facilities in the vicinity of the pit crest.

Additionally, Osisko requested that Golder evaluate the stability of the southeast wall of the pit in the vicinity of a planned waste rock dump, and develop recommendations for setback distance between the waste dump toe and the pit crest.

1.3 Method of Work

Over November 6 and 7, 2007, Graeme Major, Principal Geotechnical Engineer in Golder's Reno office, visited the Canadian Malartic site for an initial review of site conditions, and to provide recommendations for a geotechnical program to support Feasibility-level pit slope designs. The site visit was documented in a Technical Memorandum dated 21 November, 2007, which included preliminary pit slope design criteria and the recommended geotechnical program.

Simon Beaulieu, Project Engineer in Golder's Montreal office, and Rhonda Knupp, project geologist in Golder's Reno, Nevada office, visited the site on 15-16 January, 2008. The purpose of the site visit was to evaluate core drilling, geotechnical core logging, and core orientation procedures; examine representative drill core, review site geology, collect historical reports and data; and review Osisko's three-dimensional ore zones model of the project.

Televiewer surveys were completed in four development coreholes by Bruno Boussicault, Project Geophysicist from Golder's Toronto office between October 30 and November 7, 2007. The images were processed, and structural data was collected from the images. The logging procedures, televiewer images, and structural data were compiled in a report issued from Golder's Toronto office (Golder, 2008).

Geotechnical core drilling commenced on November 10, 2007 and ended on January 25, 2008. Drill core was logged and oriented by Golder personnel. Additionally, point load tests were performed, and representative samples were collected for laboratory strength testing.

The information gathered during the site visits, data from the geotechnical coreholes, televiewer surveys of development coreholes, and the proposed Ultimate pit design provided by Osisko form the basis of the recommendations documented in this report. These data sources are discussed in detail in Section 4.

2.0 SITE CONDITIONS

2.1 Introduction

The Canadian Malartic property is located south of, and adjacent to the town of Malartic, approximately 25 km west of Val d'Or, Quebec. The location of the ultimate pit relative to the town is shown on Figure 2, which also shows the principal named mineralized zones within the pit. The current pit design encroaches on residential areas in the south part of the town, and will require the relocation of some residences and protection of others.

2.2 Climate

Temperatures in the Val d'Or area can reach up to 35°C in summer and -40°C in winter. Average daily temperatures range from highs of 23°C and lows of 11°C in July, to average daily highs of -11°C and lows of -24°C in January. Temperatures can drop below freezing from September through June. Snow generally falls from October through April, with an average annual snowfall of 610 mm, and average snow depths of up to 60 mm in February. Average annual precipitation is 914 mm.

2.3 Topography

Topography across the pit area slopes gently to the east, from approximately 340 m to 320 m. A few small hills reach a maximum elevation of nearly 390 m in the west part of the property.

2.4 Geological Conditions

2.4.1 Regional Geology

The Malartic project is located in the Pontiac Subprovince, immediately south of the boundary with the east part of the Abitibi Subprovince, an Archean greenstone belt in the Canadian Shield. The Pontiac and Abitibi Subprovinces are separated by the east-west oriented Cadillac-Larder Lake Fault. North of the Cadillac-Larder Lake Fault is the Porcupine-Destor Fault, also oriented east-west, which separates older rocks to the north from younger rocks to the south. A large number of mineral deposits are concentrated in the area between the two faults.

Rocks in the region generally consist of alternating volcanic and sedimentary units that are separated by fault zones and have been metamorphosed to greenschist facies or higher. The Canadian Malartic deposit is located on the south limb of the Malartic Syncline (also called the Cadillac Syncline), a regional-scale fold that trends west-northwest and plunges steeply north.

2.4.2 Description of Geologic Units

The two main rock types in the deposit area are porphyry, and metasedimentary (collectively referred to as "Greywacke," although mudstones and shales also occur) rocks. In addition to the lithologic distinction, geological classification of these units is also based on alteration. Increasing quantities of fluids resulted in zonation in both main rock types, although it is better-developed in the Greywacke. These zones are described briefly below, in order of increasing alteration.

- **Biotite Zone (AGR, APO)** – lowest grade of alteration in the greywacke and porphyry, respectively. Widespread in the regionally-metamorphosed Pontiac Group greywacke. These units are considered unaltered with respect to the deposit. AGR commonly contains planar bedding.
- **Carbonate Zone (CGR, CPO)** – recognized by the presence of calcite pervasively in the greywacke or porphyry, respectively, including disseminated calcite as well as stockwork calcite veins.
- **Silica Zone (SGR, AGR)** – greywacke or porphyry is silicified, causing an increase in strength, but calcite alteration is still present.
- **Silica-Replaced Zone (REMGR, REMPO)** – complete replacement of the host rock by silica; no calcite remains.
- **Brecciated Zone (BRGR, BRPO)** – the highest stage of alteration, characterized by brecciation and intense silicification. BRGR contains fragments of greywacke only, and BRPO contains fragments of porphyry only.
- **Megabreccia (MBRH)** – same as BRGR and BRPO, except located along the porphyry contact, so rock fragments consist of both greywacke and porphyry.

Mafic dikes up to several meters wide crosscut the area. These have been regionally metamorphosed to schists (SCH), primarily chlorite and biotite schists, and locally exhibit strong foliation and mineral segregation. Felsic dikes that may be related to late stages of emplacement of the porphyry are also present. The dikes are light-colored, competent, and in sharp contact with the Greywacke; contact metamorphism is generally minimal or absent. The orientations of the dikes are unknown.

Overburden thickness is believed to be less than 10 m in the pit area, and consists of glaciolacustrine deposits, till, and local swampy areas containing peat.

2.4.3 Structure

Much of the structure in the deposit area is related to regional deformation that produced schistosity, overturned folds, and conjugate faulting. As summarized in the following points, our understanding of major structural features in the pit area is taken from Sansfacon and others (1987a and b; Figures 3 and 4):

- **Foliation** formed during regional deformation, and is subvertical and strikes northwest-southeast to west-northwest. Foliation is distinct from bedding. It is generally not as visible as bedding, and is only occasionally recognizable in core;
- **Sladen Fault** strikes east-west across the north part of the pit. It dips sub-vertically in the west half of the deposit, but flattens to the east. In the central and east areas the fault dips 60°S to a depth of approximately 150 m, then flattens to 45° or less. The fault is characterized by 1 to 15 cm of clay gouge, and is situated along the contact between the greywacke and the porphyry, although it does not form the contact.
- **Greywacke – Porphyry contact** strikes east-west across the north part of the pit. It dips subvertically in the west part of the deposit, and flattens to 45°S or less to the east, similar to the orientation of the Sladen Fault. The contact parallels the Sladen Fault, but is distinct from the fault.
- Sets of steeply-dipping **Conjugate Faults** strike northwest and northeast. A fault in the northwest-striking set (strike 320°, dip 70°NE) offsets the Sladen Fault at the west end of the deposit. For subsequent discussion purposes, this particular structure is referred to as the “Northwest Conjugate Fault”.
- Large **drag folds** in the form of an anticline/syncline pair are associated with the Northwest Conjugate Fault. The axes of these folds trend southeast and plunge 65°. The folds are located in the west part of the pit and deform both the Greywacke and the Porphyry. The syncline occurs on the east side of the Northwest Conjugate Fault, and the anticline on the west side.
- The **“Intersection” Fault Zone** occurs along the west side of the deposit, at the intersection of the Sladen Fault and the Northwest Conjugate Fault. Faults within this zone strike 030° to 060° and dip 55°-60° southeast.
- **Bedding** in the Greywacke generally strikes east-west and dips vertical to 60°-70° north. Bedding is evident in the Greywacke only when alteration is absent or low in intensity. In the core inspected from corehole CM06-692, AGR was a competent rock with intermittent fractures along bedding typically spaced at distances of meters rather than centimeters. Lamination visible in core is usually bedding

Although Sansfacon and others (1987a and b) provide a reasonable understanding of structural geological conditions, the exact locations and orientations the faults and the folds are not well-defined at this time. Modeling of the locations and orientations of the faults and other structural elements is in progress.

As described above, the dips of the Sladen Fault and the greywacke-porphyry contact are near vertical in the west and flatten to the east, with 60°S dips near surface flattening to 45° or less below about 150 m.

The location and orientation information presented above for the various structures is the best information available at this time, and will be used as a basis for the analyses in this study.

Faults are generally characterized by narrow zones of brittle fracturing, usually a few centimeters to less than one meter thick. A fault inspected in representative core from Corehole CM05-672 occurred close to the hanging wall of the contact breccia (MBRH), over the depth interval of 125 m to 140 m, and was characterized by multiple zones of brittle fracturing, each typically 10 cm to 30 cm wide and separated by 1-3 m of unfractured rock. Approximately 20% of the fault zone was estimated to be rubble. No clay alteration or significant weakening of the intact rock associated with this particular fault zone was observed, although clay gouge occurs in some faults. Approximately one centimeter of clay gouge was observed in a fault in an exploration corehole that was being logged during our site visit. We understand that clay gouge occurrence is unusual; no clay gouge was observed in the geotechnical coreholes.

The distributions, orientations, and character of the schists (metamorphosed mafic dikes) are poorly defined, but they occur more frequently to the south of the mineralization. They may be continuous, and typically appear to have lower rock strength than the surrounding rock. Therefore, they have the potential to form continuous zones of weakness within the rock mass. In contrast, the felsic dikes are competent, and are not indicated to represent significant weakness zones within the rock mass.

2.4.4 Mineralization and Mineralization Controls

Gold occurs in a zone of silica replacement of the metasedimentary rocks of the Pontiac Group. The mineralizing fluids appear to have moved through voids in the sedimentary rocks, and caused brecciation in some of the main fluid conduits. Gold also occurs along the contact between the sedimentary rocks and a diorite porphyry along the north margin of the deposit.

Mineralization associated with the porphyry likely continues at depth in the root of the intrusive body, but for current purposes a depth of approximately 350 m is taken as the base of mineralization extractable by the open pit.

2.5 Hydrogeology

A hydrogeological study was completed in support of the Feasibility Study. Water table elevations in the pit area range from 301.15 m to 334.41 m. Groundwater flow is generally to the north, except in the vicinity of the underground workings, where flow is toward the workings. Osisko operates two pumping wells to control groundwater levels in the underground workings. One pump is in operation from the beginning of April through early December, and the other is in operation from mid-April through the end of May. The groundwater elevation measured in the workings when the pumps were not active and the water level was recovering was 301 m.

3.0 MINING HISTORY AND PLANNED PIT DEVELOPMENT

3.1 Background

The Gouldie Brothers discovered gold in the Malartic area in 1923. In 1927, the Malartic Gold Mines began trenching and limited underground development of a nearby area. This activity ceased in 1929 with the arrival of the Great Depression. The Canadian Malartic Gold Mines reopened the Malartic Mines in 1933, and produced ore from these and other underground mines in the area until 1965.

The nearby Sladen and East Malartic underground mines opened in 1938 and operated until 1970 and 1983, respectively. Since 1983 the properties were owned by several different companies, and while some of the owners continued exploration or commissioned environmental and slope stability studies, no additional ore was produced.

Between November 2004 and March 2006, Osisko acquired the claims that now comprise the Canadian Malartic property. Osisko subsequently completed core drilling programs for resource estimation purposes in compliance with Canadian National Instrument 43-101, with the objective of defining mineralization with drilling on 30 m centers.

3.2 Current Pit Design

The proposed pit has an elliptical shape, with an east-west axis approximately 2,000 m long, and a north-south axis 750 m long (Figure 5). Wall heights are a maximum of 380 m in the east wall, with uninterrupted slope heights of 320 m in the west wall and 300 m in the west ends of the north and south walls. The proposed design provided by Osisko includes inter-ramp slope angles of 53°, and overall slope angles ranging from 53° in the west wall to 47° in the north wall, which contains two switchbacks in the ramp. The pit design includes two small, horseshoe-shaped excavations in the upper benches of the west wall, and another in the upper benches of the south wall.

The pit will be developed with 10 m high production benches based on grade control and operating equipment considerations. The main loading units will be two OK RH340B hydraulic mining excavators which have standard bucket capacities of 34 m^3. These very large excavators have a maximum reach of approximately 15 m so will easily reach to the crest of production benches for scaling. A Cat 994F Loader will be used as a backup unit. The maximum height of the raised bucket in a standard configuration is approximately 10 m, so these units could also reach to the production bench crests for scaling. The initial truck fleet will be twelve Cat 793F, 227 tonne trucks.

3.3 Underground Workings

Extensive underground mining has been conducted in the high grade zones near the porphyry-greywacke contact and in vein systems to the south. Some large stopes that extend close to the ground surface have developed glory holes, while crown pillars remain in place above others. We understand that the underground mining operations were characterized by large and stable stopes, with little structural control on stope dimensions

Osisko provided electronic three-dimensional shapes of the underground workings in the Canadian Malartic and Sladen Mines. A comparison of available hard-copy geological maps of the underground workings with the shapes indicates that not all of the workings were included in the electronic file. Additionally, in the three-dimensional shapes many of the stopes are the same height as the drifts. This may be inaccurate, as stopes usually are developed above or below the drifts, or both. The following discussion is based on the electronic shapes provided, and will need to be updated with the locations of additional workings if they become available, and if the heights of some of the stopes are determined to be in error.

Figure 6 is a plan view of the locations of the Canadian Malartic and Sladen underground workings from the electronic file in relation to the proposed pit. The Canadian Malartic workings mainly targeted mineralization in the Sladen Fault and along the greywacke-porphyry contact, and in the Northwest Conjugate Fault. Most stopes are located inside the pit and will be mined out, and a few stopes in the lower levels are located outside of the pit limits. However, seven stopes will intersect the pit walls, and four drifts run along the pit walls, as summarized below. Note that drifts perpendicular to the pit wall and small stopes are unlikely to have more than a local effect on slope stability, and so are not included in the summary.

In the proposed pit area, the Sladen Mine targeted mineralization in the Sladen Fault. As with the Canadian Malartic workings, most of the larger stopes in this mine are either located inside the pit and will be mined out, or are located well outside the pit walls. One high, narrow stope will intersect the upper benches in the east part of the northeast wall, and one smaller stope will intersect the lower benches in the east part of the north wall.

Intersections of Larger Canadian Malartic Stopes and Drifts with Pit Wall				
Mine	**Type**	**Easting Range**	**Northing Range**	**Elevation Range (m)**
Canadian-Malartic	Drift	713860-713928	5334664	-16 to -19
	Drift	713891-713977	5334674	19-21
	Drift	713963-713999	5334521-5334536	-16 to -19
	Drift	714076-714112	5334863	212-216
	Stope	713847-713871	5334883	171-173
	Stope	713980-714030	5334883	171-173
	Stope	714068-714085	5334883	171-173
	Large Stope	713770-713847	5334883	160-180
	Large Stope	713929-713992	5334875-5334879	138-158
	Large Stope	713936-713959	5334696-5334711	28-40
	Large Stope	714077-714102	5334196-5334233	285-300
Sladen	Stope	714669-714690	5334701-5334707	31-40
	Large Stope	714960-715011	5334740-5334750	168-220

4.0 AVAILABLE DATA

The following sections describe the information that is available to support this study, which includes data collected during the 2007-2008 geotechnical field investigation; structure data collected from televiewer logging of Osisko development coreholes; geologic maps and three-dimensional models of the underground workings; and geotechnical core logging data and historical documents provided by Osisko. No previous pit geotechnical investigations have been completed for the area of the proposed pit.

4.1 Previous Studies

Golder Associates (1981) completed a preliminary investigation of caving stopes in the Sladen Mine along the East Malartic Mine access road for Long Lac Mineral Exploration Ltd. However, the west edge of the study area is approximately 200 m east of the east crest of the proposed pit. Golder concluded that a number of factors contributed to the collapse of the stopes. For stopes that were mined in peridotites that had altered to chlorite schists, contributing factors were incompetent bedrock and percolation of groundwater into the stopes. For stopes mined in other host rocks, contributing factors were cross-faulting at the intersection of the Sladen Fault with the Cadillac Fault, and mining too close to the top of bedrock. The host rocks and local structural environment in the area of the caving stopes are different from those in the Canadian Malartic pit area, so the findings from Golder (1981) are not directly applicable to the current study.

4.2 Field Investigation

A field investigation to support this study was started in early November 2007 and continued through January 2008. This program included five coreholes drilled specifically for geotechnical purposes; and data collection from five additional coreholes drilled for exploration/development purposes. The locations, orientations, and lengths of these coreholes are summarized below, and illustrated in Figure 5.

GEOTECHNICAL AND DEVELOPMENT COREHOLES						
Drillhole ID	**Easting**	**Northing**	**Elevation (m)**	**Azimuth (°)**	**Inclination (°)**	**Length (m)**
GT07-01	713,944.98	5,334,914.68	318.08	0°	-60°	174.00
GT07-02	714,825.16	5,334,730.08	320.64	45°	-60°	282.00
GT07-03	713,579.20	5,334,900.86	323.86	315°	-60°	228.00
GT07-04	713,910.11	5,334,446.49	320.43	180°	-70°	369.00
GT07-05	714,125.05	5,334,469.87	320.96	180°	-70°	381.00
CM07-1377	713,942.68	5,334,923.10	317.90	180°	-45	129.13
CM07-1446	714,156.19	5,334,925.79	319.79	180°	-70	165.00
CM07-1490	713,820.05	5,334,885.39	320.03	330°	-85	321.00
CM07-1500	714,248.08	5,334,898.66	320.86	0°	-70	150.00
CM07-1540	714,307.26	5,334,883.96	320.38	0°	-50°	128.00

Note: Azimuths and inclinations are nominal

The types of data collected from each of these coreholes are summarized below, and discussed in the following sections.

DATA COLLECTED BY COREHOLE			
Drillhole ID	**Geotechnical Logging Data**	**Strength Testing Data**	**Structural Orientation Data**
GT07-01	Detailed	Point Load, Lab Samples	Oriented Core
GT07-02	Detailed	Point Load, Lab Samples	Oriented Core
GT07-03	Detailed	Point Load, Lab Samples	Oriented Core
GT07-04	Detailed	Point Load, Lab Samples	Oriented Core
GT07-05	Detailed	Point Load, Lab Samples	Oriented Core
CM07-1540	Detailed	Point Load, Lab Samples	Oriented Core
CM07-1377	None	None	Televiewer
CM07-1446	Quick	Point Load	Televiewer
CM07-1490	Quick	None	Televiewer
CM07-1500	Quick	None	Televiewer

4.2.1 Geotechnical Core Logging

All geotechnical core logging was completed by Golder personnel to two different levels of detail. A "quick" logging format was applied for core from the development coreholes, which was boxed and transported to the core shed prior to logging. This format included lithology, total core recovery, solid core recovery, Rock Quality Designation (RQD), fracture frequency, ISRM strength index (Table 1); and Joint Condition Rating (JCR, Table 2).

Data collection for the geotechnical coreholes was carried out at the rig, while the core was still in split tubes, according to a "detailed" logging format. This format included the parameters listed above for the "quick" logging, plus detailed descriptions of individual discontinuities according to the following parameters:

- Depth
- Discontinuity type
- Joint Condition Rating
- Roughness
- Shape
- Infilling type
- Infilling thickness.

4.2.2 Core Orientation

Core orientation was carried out in the geotechnical coreholes (GT07-01-05) and in development drillhole CM07-1540 with the Ace Core Orientation Tool, an electronic core-stub orienting device. Discontinuity orientation data was collected during geotechnical logging, and included dip angle with respect to the core axis (α), and the circumferential angle between top-of-core and the apparent "dip direction" of the discontinuity (β, measured clockwise from top-of-core to point on fracture surface furthest downhole).

4.2.3 Televiewer Surveys

Optical and acoustic televiewer surveys were completed by Golder along the entire lengths of drillholes CM07-1377, CM07-1446, CM07-1490, and CM07-1500. The purpose of the surveys was to acquire additional structural orientation data in the north wall. Osisko completed downhole surveys of these coreholes with FLEXIT equipment.

Structures that intersect the borehole walls are clearly identifiable in televiewer images, but televiewer images by themselves often do not allow positive identification of structure type, or whether structures are "open" or "healed". Televiewer images also do not allow evaluation of discontinuity properties (roughness, infilling type, etc.). Positive evaluation of these details requires correlation of the image logs with drill core. This type of correlation was not completed for the above coreholes. However, we assigned a probable structure type to each feature that was identified. In general, veins, dikes, joints, faults, and contacts are recognizable in televiewer images. Foliation and bedding are not as easy to recognize, and differentiating between the two can also be difficult. Additional details of the televiewer surveys are provided in Appendix D.

4.2.4 Laboratory Testing

Representative samples for possible laboratory testing were collected from the geotechnical coreholes and CM07-1540. A total of 95 samples were shipped to the Queen's University rock mechanics laboratory in Kingston, Ontario, for strength testing. Of these, 41 were used for unconfined compressive strength (UCS) and elastic properties testing, 41 for triaxial compressive strength testing, and 10 for direct shear testing along discontinuities.

4.2.4.1 UCS Test Results

The UCS test results indicate that all the rock types tested except for CGR and BRGR classify as Very Strong Rock (Table 1). CGR and BRGR classify at the upper end of Strong Rock. Summary results for this testing are listed below, and more detailed results are included in Appendix C.

SUMMARY OF UCS TEST RESULTS						
Rock Type	**# of Samples**	**UCS (MPa)**				
		Min	**Max**	**Mean**	**Median**	**Standard Deviation**
AGR	10	78.30	211.40	145.58	153.90	47.77
CGR	3[1]	81.30	125.00	98.07	87.90	23.56
SGR	6	87.80	219.90	162.88	176.95	50.28
REMGR	2[1]	118.90	138.20	128.55	128.55	13.65
BRGR	2	70.60	129.20	99.90	99.90	41.44
APO	3	150.50	208.10	188.63	207.30	33.03
CPO	8[1]	90.00	236.00	162.10	162.00	49.81
SPO	2[1]	179.40	187.10	183.25	183.25	5.44
SCH	1	--	--	*65.00*[2]		--

[1]Number of samples excludes one sample not included in mean UCS calculation because it failed along a pre-existing fracture surface.

[2]Sample failed along a pre-existing fracture surface and UCS strength likely underestimates true rock strength.

4.2.4.2 Triaxial Compressive Strength Testing

Triaxial compressive strength tests were performed to provide data for calculating rock mass shear strength as discussed in Section 5. Test results are summarized in Table 3, and further details are provided in Appendix C.

4.2.4.3 Elastic Properties and Density

Poisson's ratio was determined for UCS test samples, and density and Young's Modulus were measured for all UCS and triaxial compressive strength samples. Summary test results are tabulated below, and detailed results are included in Appendix C.

SUMMARY OF POISSON'S RATIO			
Rock Type	**Number of Samples**	**Poisson's Ratio**	
		Mean	**Standard Deviation**
AGR	10	0.19	0.03
CGR	4	0.18	0.04
SGR	6	0.17	0.02
REMGR	3	0.13	0.05
BRGR	2	0.18	0.01
APO	3	0.17	0.03
CPO	9	0.17	0.03
SPO	2[1]	0.15	0.04
SCH	1	0.15	--

[1]Excludes one sample not included in calculations because results were significantly different from other samples.

SUMMARY OF YOUNG'S MODULUS AND DENSITY DATA					
Rock Type	**Number of Samples**	**Young's Modulus (GPa)**		**Density (g/cm^3)**	
		Mean	**Std. Dev.**	**Mean**	**Std. Dev.**
AGR	19	46.830	10.992	2.76	0.03
CGR	10	46.322	9.505	2.74	0.05
SGR	10	54.107	7.113	2.70	0.03
REMGR	8	55.053	6.071	2.67	0.04
BRGR	2	37.561	3.414	2.58	0.04
APO	8	53.320	3.973	2.68	0.01
CPO	15	45.876	11.679	2.67	0.03
SPO	8[1]	56.436	3.836	2.67	0.02
SCH	4	38.182	7.942	2.80	0.08

[1]Number of samples excludes one sample not included in calculations because test results were significantly different from other samples of same rock type.

4.2.4.4 Direct Shear Testing

Direct shear testing was conducted on samples of natural bedding, chlorite-coated joints, and foliation surfaces in AGR and CGR. The results of this testing are included in Appendix C and summarized below.

SUMMARY OF DIRECT SHEAR TESTING ALONG DISCONTINUITIES					
Discontinuity Type	Drillhole	Depth (m)	Rock Type	Residual Strength Data	
				Friction Angle (°)	Cohesion (kPa)
Bedding	GT07-04	189.09-189.66	AGR	26	550
Chlorite-coated joints	GT07-02	159.05-159.35	AGR	19	850
	GT07-02	165.00-165.33	AGR	20	900
	GT07-04	141.21-141.50	AGR	31	0
Foliation	GT07-02	89.00-89.25	CPO	30	1290
	GT07-04	189.09-189.66	AGR	51	790
		205.09-205.33	AGR	28	185
		227.55-227.99	AGR	34	455
		230.00-232.33	AGR	31	0
	GT07-05	213.00-213.50	AGR	27	16

4.2.5 <u>Point Load Testing</u>

Point load testing provides quantitative estimates of rock compressive strength through a simple field test. These tests are typically completed with a higher frequency than laboratory UCS tests, and so allow closer tracking of compressive strength variations. Axial and diametral point load tests were performed on core from the geotechnical drillholes, and coreholes CM07-1540 and CM07-1446 approximately every 3 m (Appendix B). Data collected for each test included depth, lithology, test orientation, dimensions, failure load, and comments regarding induced failure surfaces. Tests are generally considered valid when failure surfaces were generated between the loading platens; tests in which failure surfaces involve pre-existing, healed structures were considered either “questionable” or perhaps invalid.

Sample dimensions and failure load were used to calculate point load index according to the following equation:

$Is_{50} = P/D_e^2 * (D_e/50)^{0.45}$

where:

Is_{50}= point load index

P = gauge load at failure

De = equivalent diameter

Test results was sorted by test type and by rock type, and the upper and lower 10% of the $I_{s(50)}$ values were deleted unless there were fewer than 10 tests for a particular rock type, in which case only the uppermost and lowermost $I_{s(50)}$ values were deleted. Only three tests were available for REMGR and SCH; no test results were deleted for these units. Basic statistics for $I_{s(50)}$ are summarized in the following tables.

SUMMARY OF DIAMETRAL POINT LOAD TEST RESULTS

Rock Type	Number of Tests	$I_{s(50)}$ (MPa)				
		Min	Max	Mean	Median	Standard Deviation
AGR	395	5.19	12.18	8.17	7.93	1.76
CGR	31	5.81	10.54	7.73	7.24	1.34
SGR	4	4.98	6.90	6.27	6.59	0.90
REMGR	3[1]	2.40	9.20	4.82	2.85	3.80
APO	19	8.64	11.99	10.32	10.51	0.95
CPO	114	3.64	12.10	8.26	8.98	2.45
SPO	22	3.87	11.73	7.67	7.44	1.88
SCH	3[1]	6.54	8.850	7.35	6.65	1.30

[1]Due to the small number of tests, the upper and lower 10% of the values were not deleted from this sample set.

SUMMARY OF AXIAL POINT LOAD TEST RESULTS						
Rock Type	**Number of Tests**	**$I_{s(50)}$ (MPa)**				
		Min	**Max**	**Mean**	**Median**	**Standard Deviation**
AGR	323	3.35	9.56	6.27	6.31	1.50
CGR	28	5.55	10.12	7.45	7.48	1.25
SGR	4	4.38	5.12	4.77	4.79	0.41
REMGR	3[1]	1.98	4.53	2.89	2.17	1.42
APO	16	7.74	9.80	8.47	8.34	0.63
CPO	90	4.20	11.40	8.08	8.76	1.73
SPO	17	6.18	9.54	7.66	7.49	1.09
SCH	3[1]	3.28	9.96	7.53	9.34	3.69

[1]Due to the small number of tests, the upper and lower 10% of the values were not deleted from this sample set.

Use of point load index values to estimate UCS requires the following correlation:

$$UCS = N * I_{s(50)}$$

The conversion factor can be affected by characteristics such as lithologic variations, alteration, and geologic structure. Wyllie (1992) reports that N can vary between sites and rock types within a broad range from 15 to 50, and Bieniawski (1974) suggests that a conversion factor of 24 provides a reasonable estimate of UCS for many rock types.

4.3 Other Data

4.3.1 Optical Televiewer Surveys

Optical televiewer surveys completed by a Terratec Geophysical Services in 2006-2007 were available for a number of drillholes in the deposit. Osisko provided optical images for eight drillholes located in the proposed pit walls as summarized below, and shown in Figure 5.

TELEVIEWER DRILLHOLES PROVIDED BY OSISKO						
Drillhole	**Easting**	**Northing**	**Elevation (m)**	**Azimuth (°)**	**Inclination (°)**	**Length (m)**
CM06-693	713,444.10	5,334,849.98	329.68	--	-90	325.60
CM07-984	714,760.29	5,334,518.79	333.85	185	-47	415.50
CM06-832	714,700.39	5,334,806.50	320.72	--	-90	181.70
CM07-1029	713,349.84	5,334,619.89	335.27	--	-90	353.40
CM07-1100	714,276.74	5,334,243.68	325.38	--	-90	151.60
CM07-1108	713,580.12	5,334,450.65	339.84	--	-90	481.15
CM07-1421	713,751.24	5,334,953.61	321.81	140	-70	250.78

Note: Azimuths and inclinations are nominal

Structure data was not collected from these images beforehand, so Golder compiled structure logs of the images as part of this study. Structure types were assigned to each feature picked in the televiewer data, but as was the case with the televiewer data collection described previously, the images were not correlated with the core. FLEXIT downhole surveys for these coreholes were provided by Osisko.

4.3.2 Underground Mapping Data

Hard copies of geologic maps were available for many of the levels in the Canada Malartic and Sladen underground mines. Structure orientations, types, and locations within approximately 30 m of the pit walls were measured from these maps and compiled in a database.

4.3.3 RQD Database

Osisko personnel record RQD during geological logging of development core, and Osisko provided a database of RQD values for 49 development coreholes numbered between CM06-807 and CM07-1401. This database was used to model rock quality in the ultimate pit walls.

4.3.4 Surpac Database

Osisko provided a Surpac-compatible database that included the pit shape; and locations, downhole survey data, and rock type codes for the exploration drillholes.

5.0 ENGINEERING CHARACTERIZATION

The data sources described in Section 4 were used to develop an engineering geologic characterization of the deposit that consists of the following main components:

- Pit geology;
- Geological structure;
- Material properties;
- Rock mass properties; and
- Hydrogeologic conditions.

The purpose of this characterization is to define the information required to support the stability analyses discussed in Section 6. For material and rock mass properties, the characterization is developed in terms of discrete “design” values for these properties. For geological structure, the characterization includes the orientations of major structures; and “design” orientations for discontinuity sets that are considered to occur systematically in a rock mass.

5.1 Pit Geology

5.2 Geology and Structure in the Ultimate Pit

Three-dimensional solids of the rock/alteration units and the major structures have not yet been developed, as geological work to date has focused on delineating the ore zones. Generally, the central and east parts of the north wall will be developed in porphyry, and the rest of the slopes will expose the various altered greywacke units. The south wall and the west part of the north wall are expected to be developed in relatively unaltered greywacke (AGR).

As discussed previously, the Sladen Fault and the contact are believed to strike east-west, with sub-vertical dips west of about 713950E. These structures will likely be exposed below the mid-height of the north wall in this area. East of about 713950 E, the strike of the Sladen Fault and the contact remains unchanged, but dips flatten to about 60°S from surface to approximately 150 m depth, and to 45° or less below ~150 m. Between 713950E and 714450E, they appear to strike subparallel to the central part of the north wall, and in this area due to their flatter dips they will likely be exposed in the pit floor, rather than in the north wall. East of about 714450E, the north wall changes orientation slightly, and converges with the fault and contact, such that they will likely be exposed along the full height of the wall in the area of the abrupt change in wall orientation in the east part of the pit.

The exact location of the Northwest Conjugate Fault is not well-defined, although it occurs in the west part of the deposit, and likely strikes obliquely into the northwest and south walls. Between these exposures, the fault will be mined out, as will the Intersection Fault Zone. The anticline and syncline pair may be partially or entirely mined out, depending on the extent of the folding.

5.2.1 Geotechnical Units

Geotechnical units are composed of individual geologic units, or combinations of geologic units that can be grouped together for engineering purposes based on similar rock mass and rock structure conditions. For this study, the geotechnical units are consistent with the geologic units that have been classified on the basis of lithology and alteration by Osisko. Subsequent discussions related to rock structure, material properties, and rock mass properties are presented in the context of these units.

5.3 Material Properties

5.3.1 Unconfined Compressive Strength

Laboratory UCS testing results and point load testing data were compared to provide a conversion factor, N, to correlate between rock strength data sources; and to determine design compressive strengths by geotechnical unit.

5.3.1.1 Correlation of UCS and Point Load Testing Data

Laboratory UCS testing data was compared with the point load testing data using two different methods to define N values that should be applied to the point load testing data. The first method involves determination of N values based on average UCS and $I_{s(50)}$ for each geotechnical unit:

COMPARISON OF AVERAGE UCS AND POINT LOAD TEST RESULTS						
Unit	Average UCS (MPa)	Average $I_{s(50)}$		N ($UCS/I_{s(50)}$)		Comments
		Diametral	Axial	Diametral	Axial	
AGR	145.58	8.17	6.27	17.82	23.22	
CGR	98.07	7.73	7.45	12.69	13.16	3 UCS tests; N too low
SGR	162.88	6.27	4.77	25.98	34.15	
REMGR	128.55	4.82	2.89	26.67	44.48	2 UCS tests; axial N too high
APO	188.63	10.32	8.47	18.28	22.27	3 UCS tests
CPO	162.10	8.26	8.08	19.62	20.06	
SPO	183.25	7.67	7.66	23.89	23.92	2 UCS tests
SCH	*65.00**	7.35	7.53	8.84	8.63	Only 1 UCS test; too low

*Sample failed along a pre-existing fracture surface and UCS strength likely underestimates true rock strength.

The second method consists of comparison of specific point load and UCS tests performed at approximately the same depth intervals:

COMPARISON OF UCS AND POINT LOAD TEST RESULTS BY DEPTH								
DHID	Unit	UCS Depth (m)	UCS (MPa)	$I_{s(50)}$ Depth (m)	$I_{s(50)}$ (MPa)		N (UCS/ $I_{s(50)}$)	
					Diam	Ax	Diam	Ax
GT07-03	AGR	49.95-50.5	179.00[2]	48	10.48[5]	4.64[3]	17.08	38.57
GT07-03	AGR	142.9-144.05	137.48[4]	144	6.49[5]	3.12[3]	21.17	44.02
GT07-03	AGR	179.15-179.59	105.75[2]	180	9.21[4]	6.40[3]	11.48	16.53
GT07-01	AGR	152.3-152.7	168.20[2]	150	6.75[3]	5.07[3]	24.92	33.19
GT07-04	SGR	17.6-17.9	191.75[2]	12	8.67[3]	4.31[3]	22.11	44.48
GT07-05	REMGR	38.82-39.0	118.90[1]	36	4.82[3]	2.89[3]	24.69	41.09
GT07-02	APO	114.27-114.52	150.50[1]	114		9.36[2]		16.09
GT07-02	CPO	121.6-121.82	209.00[1]	120	11.43[2]	6.14[2]	18.29	34.04
GT07-02	CPO	194.22-194.52	191.60[2]	192	9.70[2]	4.97[2]	19.74	38.57
CM07-1540	CPO	70.24-70.49	113.00[1]	72	10.15[4]	9.33[3]	11.13	12.12
CM07-1540	CPO	110.04-110.52	133.40[2]	111	8.97[4]	8.19[3]	14.88	16.29
CM07-1540	CPO	123.6-124.07	162.40[2]	123	4.41[4]	8.65[3]	36.86	18.78

[1]One test; [2]Average of two tests; [3]Average of three tests; [4]Average of four tests; [5]Average of five tests

N values based on the two methods are summarized in the following table:

SUMMARY OF CALCULATED CONVERSION FACTORS					
Unit	N Calculated from Average Test Data		N Calculated from Tests at Similar Depths		Comments
	Diametral	Axial	Diametral	Axial	
AGR	18.66[1]	33.08[1]	17.82	23.22	
CGR			12.69	13.16	Too low (there was no comparison for CGR based on sampling at similar depths in the table above)
SGR	22.11	44.48	25.98	34.15	
REMGR	24.69	41.09	26.67	44.48	
APO		16.09	18.28	22.27	
CPO	20.18[2]	23.96[2]	19.62	20.06	
SPO			23.89	23.92	There was no comparison for SPO based on sampling from similar depths
SCH			8.84[3]	8.63[3]	Too low (there was no comparison for SCH based on sampling from similar depths in the table above)

[1]Average of four tests
[2]Average of five tests
[3]Failed along a pre-existing fracture surface during UCS testing

The N values calculated for AGR, APO, CPO, REMGR, and SGR using the two different methods agree reasonably well, but values for CGR and SCH fall below the normal range. For CGR, three UCS tests may not be a sufficient number of tests to calculate a realistic average. The low N calculated for SCH is likely due to an underestimation of the actual rock strength by the UCS test, as the sample failed along a pre-existing, healed structure during testing.

The conversion factors for the metasedimentary units from axial point load testing data are generally higher than those from diametral point load testing data, suggesting that compressive strength in these units may be anisotropic. For current characterization purposes, design UCS values are based on the lower-strength direction, i.e., the diametral testing data, which should provide conservative estimates of UCS. Accordingly, N values of 18 and 24 were selected for unsilicified sedimentary rocks and silicified sedimentary rocks, respectively.

Point load testing data do not indicate anisotropy for the porphyry, and an N value of 20 was selected for this unit. Given the poor correlation between UCS and $I_{s(50)}$ for SCH, a conservative N value of 15 was used to estimate UCS from point load testing data for this unit.

5.3.1.2 Design UCS Values

Design UCS values are summarized below by geotechnical unit:

DESIGN UCS VALUES BY GEOLOGIC UNIT	
Unit	**Design UCS (MPa)**
AGR	145
CGR	140
SGR	150
REMGR	115
BRGR	100
APO	205
CPO	165
SPO	150
SCH	110

5.3.2 Design Elastic Constants and Unit Weight

Design Young's Modulus, Poisson's Ratio, and unit weights were determined by laboratory testing:

DESIGN UNIT WEIGHT AND ELASTIC CONSTANTS			
Geotechnical Unit	**Young's Modulus (GPa)**	**Poisson's Ratio**	**Unit Weight (g/cm^3)**
AGR	46.830	0.19	2.76
CGR	46.322	0.18	2.74
SGR	54.107	0.17	2.70
REMGR	55.053	0.13	2.67
BRGR	37.561	0.18	2.58
APO	53.320	0.17	2.68
CPO	45.876	0.17	2.67
SPO	56.436	0.15	2.67
SCH	38.182	0.15	2.80

5.3.3 Design Discontinuity Shear Strength

Estimates of shear strength along discontinuities are required in order to analyze potential kinematic failure modes. The following equation was used for this purpose (Patton, 1966):

$$\tau = c + \sigma' \tan(\phi_b + i)$$

where: τ = shear strength
c = cohesion
σ' = effective normal stress
ϕ_b = base friction angle (i.e., ϕ for a smooth surface)
i = discontinuity roughness angle

Laboratory direct shear testing of natural bedding, foliation, and chlorite-coated fractures surfaces completed for this study provide valid measurements of ϕ_b+i:

DESIGN DISCONTINUITY SHEAR STRENGTH		
Discontinuity Type	**ϕ_b+i (°)**	**Cohesion (MPa)**
Bedding	25	0
Foliation	30	0
Chlorite-coated joint	19	0

Values of ϕ_b+i for "clean" joints is likely in the range of 30˚ or higher based on the characterization of these surfaces from detailed discontinuity logging as being somewhat more rough than bedding. However, the distribution of chlorite-coated joints compared to the distribution of clean joints across the deposit can not be established with available data. Therefore, the design discontinuity shear strength of chlorite-coated joints is an appropriate representation of discontinuity shear strength for current purposes.

Laboratory testing indicated cohesion along discontinuity surfaces. Given that cohesion can be reduced or eliminated by freeze-thaw, dilation related to blasting or stress relief, or other factors, we have assumed zero cohesion along discontinuities.

5.4 Rock Mass Properties

5.4.1 <u>Rock Quality Designation (RQD)</u>

RQD values rate rock quality on a scale of 0 to 100, with 90-100 being excellent quality rock (Table 4). Based on our observations of the core and the core logging data, rock quality is Good to Excellent in most areas of the proposed pit. The following table shows weighted average RQD values for each rock type from the geotechnical and exploration core logged for this study.

WEIGHTED AVERAGE RQD BY ROCK TYPE				
Unit	Geotechnical Drillholes		Exploration Drillholes	
	RQD (%)	Meters logged	RQD (%)	Meters logged
AGR	90	1168.50	90	243.00
APO	89	9.00	91	30.00
BRGR	94	3.00	74	3.00
CGR	80	93.00	86	21.00
CPO	82	60.00	84	373.50
REMGR	77	9.00	--	--
SCH	77	12.00	93	6.00
SGR	71	30.00	--	--
SPO	91	6.00	88	39.00

All of the geotechnical units at Canada Malartic except SGR classify as Good quality rock or better. Although relatively few meters of core were logged in the APO, BRGR, REMGR, SCH, and SPO units, based on our observations of the core and on the UCS and point load test results, a classification of Good quality rock is reasonable for these units. Only 30m of SGR was logged, but this unit was encountered in four different geotechnical drillholes, and a classification of Fair quality rock is considered representative based on these intersections.

Osisko's database of RQD logging from 49 exploration coreholes is another source of RQD logging data. An RQD block model was developed from this database in Surpac, and the block model was intersected with the ultimate pit shell to develop an understanding of RQD distribution in the pit walls (Figure 7). This figure shows that where RQD data is available, the majority of the pit walls will be developed in Good to Excellent quality rock, with only isolated areas of RQD values below 50%.

5.4.2 Rock Mass Rating

The Rock Mass Rating (RMR) classification developed by Bieniawski (1976) incorporates UCS, RQD, joint spacing, joint condition, and groundwater condition. Each of these parameters is assigned a rating value, and the sum of these yields the RMR, which can vary between 0 and 100 (Table 2; note that the RMR system has been revised several times, but the 1976 version was used in development of the rock mass shear strength criterion that is applied in this study).

Weighted average RMR values calculated for each unit from the geotechnical logging data and the design compressive strength values are summarized below. Because the geotechnical coreholes were drilled using triple tube equipment and the core from these holes was logged before being boxed and transported, the RMR ratings for core from the geotechnical drillholes are considered more representative of the rock masses than those from development core. Therefore, data from the geotechnical coreholes was used for the RMR calculations for all of the units except for APO and SPO, which were not encountered in the geotechnical drillholes.

RMR_{1976} RATINGS

Unit	Strength Rating[1]	RQD Rating	Fracture Frequency Rating	Joint Condition Rating	Groundwater Rating	Total RMR	Meters of Core
AGR	12	18	21	14	10	75[2]	1168.5
CGR	12	16	16	14	10	68[2]	93.0
SGR	12	14	17	14	10	67[2]	30.0
APO	14	18	16	10	10	68[3]	30.0
CPO	13	16	14	15	10	68[2]	60.0
SPO	12	17	14	11	10	64[3]	39.0
SCH	10	15	11	12	10	58[2]	12.0

[1]Based on design UCS values
[2]Calculated using logging data from geotechnical core except for strength rating
[3]Calculated using logging data from exploration core except for strength rating

All of the geotechnical units classify as Good Quality rock masses except for SCH, which classifies as a Fair Quality rock mass.

5.4.3 Rock Mass Strength

5.4.3.1 Hoek-Brown Failure Criterion

Methods for determining unconfined and triaxial compressive strengths of intact samples, and shear strength of distinct discontinuities are well-established. However, estimation of the shear strength of *rock masses*, which reflect the combined influences of intact rock, discontinuity shear strength, the frequency and continuity of fractures, etc., is more difficult. The Hoek-Brown failure criterion (Hoek and Brown, 1988) provides an empirical approach to this issue, with a criterion expressed in the following equation:

$$\sigma_1 = \sigma_3 + \sqrt{m \cdot \sigma_{ci} \cdot \sigma_3 + s\sigma_{ci}^{\ 2}}$$

where:

σ_1 = the major principal stress at failure
σ_3 = the minor principal stress (confining stress)
m, *s* = Hoek-Brown material constants for jointed rock mass
σ_{ci} = the uniaxial compressive strength of intact rock

Values for *m* and *s* vary with rock mass quality and lithology, with the upper limits of the value ranges defined by m and s for intact rock (i.e., intact core samples). Triaxial compressive strength testing data can be used to determine m for intact rock (termed m_i); alternatively, published data are available for this purpose. The value of s for intact rock is 1.0.

Given the values of m_i and s for intact rock, the parameters *m* and *s* for the broken rock mass are calculated based on the following equations (Hoek and others, 2002); in effect, these equations reduce m and s from the values for intact rock on the basis of a measure of rock mass quality, RMR. A "disturbance" criteria, which relates to the effects of blasting/excavation and stress relief on rock mass integrity, is also applied.

$$m / m_i = \exp [(RMR - 100) / (28-14D)]$$

$$s = \exp [(RMR - 100) / (9-3D)]$$

where D is a factor that depends on the degree of disturbance that the rock mass has been subjected to by blast damage and stress relaxation. It varies from 0 for undisturbed rock to 1.0 for very disturbed rock.

5.4.3.2 Design m_i Values

The laboratory testing data developed for this study were processed using the computer program Rocdata (Rocscience, 2007a) to compute best fit m_i parameters for the geotechnical units. This process involved solving the above equation relating σ_1 and σ_3 for m_i, assuming that s=1.

The m_i constants calculated from the triaxial compressive strength testing are:

CALCULATED m_i VALUES FROM TRIAXIAL TESTING DATA				
Unit	**UCS (MPa)**	**RMR**	m_i	**# of Samples**
AGR	145	75	23	19
CGR	140	68	13	9
SGR	150	67	34	10
APO	205	68	35	8
CPO	165	68	27	14
SPO	150	64	24	8
SCH	110	58	7	4

Published m_i values are shown in Table 5. The m_i values determined by laboratory testing for AGR, CGR, SPO, and SCH are consistent with the published values for similar rock types. However, m_i values for APO, CPO, and SGR based on laboratory testing are higher than published values by as much as 13. Because m_i values calculated from site-specific data are preferred over published values, laboratory-based values were used to calculate rock mass shear strength for all geotechnical units.

5.4.3.3 Design Rock Mass Strength Parameters

Hoek and Karzulovic (2001) recommend using “undisturbed” rock mass parameters, and accounting for disturbance due to stress relaxation or blast damage based on slope deformations and blasting performance. They suggest that the zone of blast damage resulting from open-pit mine production blasting typically extends from 0.3 to 0.5 times the production bench height for careful controlled blasting. For production blasting with some control (e.g., one or more buffer rows and blasting to a free face), they suggest that the zone of blast damage extends to between 0.5 and one times the production bench height. These ranges corresponds to a depth of blast damage of approximately 5 to 15 m, which can be considered bench-scale damage that would not significantly affect rock mass strength for a large-scale, overall slope failure. They also indicated that stress relaxation will result in rock mass disturbance and reduced shear strength, but provide no guidance in accounting for stress relaxation in pit slope design.

Although Hoek and Karzulovic’s recommendations imply that the use of undisturbed rock mass parameters may be appropriate for this case, we do not recommend using undisturbed parameters because there is limited experience with application of these parameters to rock slope design. It is therefore not well established whether these parameters are conservative, accurate, or unconservative. For this reason, we prefer to consider “partially disturbed” strength parameters between Hoek and

Brown's disturbed and undisturbed estimates for the material constants. With limits defined by 0% disturbance (undisturbed) and 100% disturbance (completely disturbed), Hoek and others (2002) recommend values of 70% to 100% disturbed for typical surface mining applications, with 100% disturbance associated with heavy production blasting, and 70% disturbance with careful controlled blasting.

Design rock mass shear strength parameters for partially (70%) disturbed rocks at Osisko are:

PARAMETERS FOR HOEK-BROWN CRITERION			
Unit	m_i	m_b	s
AGR	23	5.7820	0.0267
CGR	13	2.2028	0.0097
SGR	34	5.5917	0.0084
APO	35	6.1047	0.0097
CPO	27	4.6708	0.0097
SPO	24	3.3025	0.0054
SCH	7	0.7249	0.0023

5.5 Hydrogeology

Groundwater levels during pit development and the degree of depressurization in the pit walls are important for direct input to stability analyses. In a qualitative sense, groundwater conditions are also important in developing conclusions about other aspects of slope designs, including the tendency for bench faces to generate rock fall; the effect that adverse structure may have on bench-scale slope performance; and the transmission of blast-related energy into the pit walls.

Osisko is evaluating options for a dewatering program. The hydrogeological study in support of the Feasibility Study modelled cones of depression after 10 and 15 years of pumping according to two scenarios: (A) water levels at least 50 m below the pit bottom, and (B) complete dewatering of the underground workings.

5.6 Rock Structure Characterization

5.6.1 General

Rock structure characterizations typically include large-scale structures defined in the geologic model; systematic discontinuity sets within each geotechnical unit; and other unique structures that could impact stability but do not necessarily conform to systematic structure or other large-scale faults. For the purposes of this study, the rock structure characterization was used for kinematic stability analyses, which focus on potential failure modes or slope angle constraints related to geologic structure.

Use of the structural data in developing rock structure characterizations involved plotting stereonets of the data from individual data sources, sometimes subdivided by location; identifying prominent pole concentrations on these stereonets; and determining average orientations for these concentrations. This last step in the process effectively reduces concentrations of individual structures to a single average dip and dip direction, or “design” orientation. Thus, identification of design structure orientations involved a progression of pole plots, contour plots, and plane plots of design orientations, all of which were generated with the computer stereonet program DIPS (Rocscience, 2008a).

Structural data is available from underground mapping data, and the drillhole-based sources described in Section 3. The number of poles available for each data source is summarized in the table below.

SUMMARY OF AVAILABLE STRUCTURAL DATA								
Data Source	Number of Poles by Structure Type							
	Faults and Shears	Foliation and Bedding	Joints	Veins	Dikes	Contacts	Unknown	Total
Underground Mapping	75	0	609	230	0	14	139	1,067
Oriented Core	7	116	2,793	144	4	5	1	3,070
Televiewer Surveys	23	1,518	1,366	5,633	7	80	75	8,702

As described previously, televiewer images were not correlated with the core, so structure types assigned are unconfirmed, and structures picked may not represent “open” discontinuities in the core.

Still, the major structural trends that occur in the televiewer data are generally consistent with those in the underground mapping and oriented core data, so the televiewer data were considered in the kinematic analyses.

5.6.2 Correction for Directional Bias

Structural data collected from drillholes is subject to directional bias, because the number of joints intersected by the line sample is a function of the discontinuity spacing and the orientation of the structures relative to the drillhole. For identically-spaced discontinuities, the smaller the angle at which the structures intersect the sample line, the fewer the number of structures that will be encountered. An effect of this condition is that drillholes have a “blind zone,” such that structures that are approximately parallel to the drillhole are rarely encountered. For practical purposes, the blind zone is considered to apply to structures oriented within about 20° of the drillhole axis. On a stereonet, the blind zone generally appears as a region within which little or no data appears for a given drillhole orientation.

To account for directional bias, the “Terzaghi correction” was applied in analyzing the oriented core and televiewer data (Terzaghi, 1965). This correction is applied by DIPS on contoured stereoplots (Appendix G), by weighting each pole by a factor (W) prior to contouring:

$W = \text{cosecant}(\alpha)$, where:

α = discontinuity dip angle with respect to core axis

This correction effectively increases the “quantity” of a single data point from 1 to a value greater than 1, which in turn affects contouring of the data.

5.6.3 Major Structures

The greywacke-porphyry contact and the Sladen Fault, which is situated along the contact, strike east-west in the north part of the deposit. These structures are sub-vertical west of about 713950 E; to the east, the dips flatten to about 60°S from surface to approximately 150 m depth, and to 45° or less below ~150 m. The Northwest Conjugate Fault strikes northwest and dips 70° NE in the west part of the deposit. The Sladen Fault is offset in the west by the Northwest Conjugate Fault. The orientations of the major structures are shown on the stereonet in Figure 8.

5.6.4 Pit Sectors

The proposed pit was divided into five design sectors for stability evaluations. Design sectors can be based on geotechnical conditions, pit geometry, or both. In the Osisko ultimate pit, sectors are based mainly on pit geometry, but north wall sectors are also defined by differences in structure. Details of the sectors are summarized below; sector locations are shown in Figure 5.

CANADIAN MALARTIC PIT SECTORS				
Sector	**Wall Height (m)**	**Wall Orientation (Dip Direction, °)**	**Main Lithology**	**Large-Scale Structure**
Northwest	260-320	180	AGR	Sladen Fault, greywacke-porphyry contact, Northwest Conjugate Fault, syncline
Northeast	320-360	180-210	Porphyry, various alteration types	Sladen Fault, greywacke-porphyry contact
East	380	300-310	Greywacke, various alteration types	
South	340-380	330-025	AGR	Northwest Conjugate Fault
West	320	325-160	Greywacke, various alteration types	Northwest Conjugate Fault

The structural data from sources within each sector was plotted on stereonets using an equal-area, lower hemisphere projection, and contoured using the Schmidt method. To avoid biasing the data sets for each sector in favor of individual data sources with a large number of points (particularly televiewer drillholes), data from each source was plotted individually. The structural trends identified from the concentrations of poles in the stereonets are discussed below. The stereonets are included as Appendix G.

5.6.5 Northwest Sector

Prominent structure concentrations indicated in structure data sources for the Northwest Sector are (Appendix G.1):

NORTHWEST SECTOR STRUCTURE CONCENTRATIONS				
Data Source	**Set ID**	**Dip°/ Dip Direction°**	**Concentration (%)**	**Comments**
UG Mapping Data	2	42/135	6.0	Faults, joints, and veins
	4	41/176	7.2	Veins, faults, and joints
GT07-01	1a	69/174	5.8	Joints, veins, and foliation
	1b	64/358	4.0	Foliation, joints, and veins; same as 1a
GT07-03	1a	89/179	3.0	Foliation and joints
	1b	79/346	10.6	Foliation and joints; same as 1a
	2	59/127	1.5	Mostly joints
	5	24/320	3.0	Joints and veins
CM07-1377	1a	78/164	5.0	Foliation/bedding, veins, and joints
	1b	81/355	5.2	Foliation/bedding, veins, and joints; same as 1a
	3	51/062	3.0	Mostly veins
CM07-1490	1b	59/022	12.0	Foliation/bedding, veins, and joints
	2	60/109	1.5	Mostly veins
CM07-1421	1b	52/352	13.5	Foliation/bedding, joints, and veins
	2	31/132	6.0	Joints and veins

Set 1 and Set 2 are the most prominent concentrations of poles in the Northwest sector. Set 1 consists of steep, east-west-striking foliation/bedding, joints, and veins that are parallel to the Sladen Fault (note that “foliation” logged in the geotechnical and televiewer coreholes is probably bedding). Although prominent in all of the drillholes, this set is not strongly developed in the underground mapping data, possibly because it strikes parallel to the drifts.

Set 2 is comprised of faults, joints, and veins that occur in all data sets. Structures in this set parallel the “intersection” fault system between the Sladen Fault and the Northwest Conjugate Fault.

Set 3 in corehole CM07-1377 consists mostly of veins that parallel to the Northwest Conjugate Fault. These structures are also present in CM07-1421 and CM07-1490, although not in strongly-developed concentrations.

Set 4 consists of veins, faults, and joints that are well-represented in the underground mapping data, but weakly developed in drillholes GT07-03, CM07-1490, and GT07-01. Drillhole CM07-1377 is oriented such that it would not pick up this set, and only a few poles at this orientation are present in CM07-1421.

Joints and veins in Set 5 occur in coreholes GT07-03 and GT07-01, but not in other data sources.

5.6.6 Northeast Sector

Prominent structure concentrations indicated in structure data sources for the Northeast Sector are (Appendix G.2):

NORTHEAST SECTOR STRUCTURE CONCENTRATIONS				
Data Source	**Set ID**	**Dip°/ Dip Direction°**	**Concentration (%)**	**Comments**
UG Mapping Data	1	34/141	6.0	Joints, veins, and faults
	2a	62/175	6.0	Joints, veins, faults, and contacts
	2b	73/011	2.0	Joints and faults, same as 2a
	4	48/087	3.0	Joints and veins
	5	59/264	2.0	Veins and joints
GT07-02	1	46/154	3.5	Mostly joints
	2a	73/185	4.7	Mostly joints
	2b	86/005	4.0	Joints; same as 2a
	3	08/325	4.0	Joints
CM06-832	1	31/148	12.5	Joints, bedding/foliation, veins, and dikes
CM07-1446	6	51/161	9.6	Bedding/foliation, veins, and joints; possibly biased flatter and same as 2a
	2a	88/189	3.0	Veins and joints
	2b	82/001	4.0	Bedding/foliation, veins, joints, and contacts; same as 2a
CM07-1500	1	36/172	15.6	Veins, bedding/foliation, and joints
CM07-1540	1	42/173	5.5	Joints, veins, and foliation
	2a	84/201	2.0	Joints and veins
	2b	84/019	3.0	Mostly joints; same as 2a

The most prominent structures in this sector are represented by Set 1, which occurs in the underground mapping data and in drillholes CM06-1500, CM07-1540, CM06-832, and GT07-02. This set does not appear to be present in CM07-1446, but it is strongly developed in all of the other data sources. Southeast-dipping structures in this set are parallel to the “intersection” fault zone. The more southerly-dipping structures in this set may represent the Sladen Fault/greywacke-porphyry contact, which as discussed previously dips flatter in the east part of the pit than in the west. Set 2a in the underground mapping data dips 62°S, parallel to the fault/contact above ~150 m depth. Strong concentrations of poles in Set 1 dip 31°-46° south, suggesting that the dip of the fault/contact may be as flat as ~30°.

The dip directions for Set 1 vary by the data source. The stereoplots for televiewer drillholes CM06-832 and CM07-1500 contain concentrations of poles with dip directions to the south, while the oriented coreholes and underground mapping data show concentrations with dip directions more to the southeast. (Although the average dip direction of Set 1 in CM07-1540 is 173°, the main concentrations of poles dip 25° east and 20° west of south.). Because the televiewer images were not compared with the core, it is unclear whether the south-dipping structures in CM06-832 and CM07-1500 represent open discontinuities such as joints, or intact features such as veins.

Set 2 is strongly-developed in the underground mapping data, CM07-1446, CM07-1540, and GT07-02. This set is also present but weakly developed in CM07-1500. Set 2 is not present in CM08-832, but this drillhole is oriented vertically and so would tend not to intersect steep structures. In the stereoplot for CM07-1446, the strong concentration of poles designated Set 6 lies adjacent to the blind zone. This suggests the possibility that the concentration is "truncated" by the blind zone, in which case the average orientation indicated by the data from CM07-1446 may be biased flatter, and Set 6 may actually be part of Set 2. The steeper structures in Set 2 are parallel to bedding, while the flatter structures in Set 2a may represent structures associated with the Sladen Fault.

Shallow, north-dipping joints (Set 3) are well-developed in GT07-02, and are present but poorly developed in CM07-1446 and CM06-832. Joints in Set 4 are present in the underground mapping data and several of the drillholes, but not in strong concentrations.

5.6.7 East Sector

Prominent structure concentrations indicated in structure data sources for the East Sector are (Appendix G.3):

EAST SECTOR STRUCTURE CONCENTRATIONS				
Data Source	**Set ID**	**Dip°/ Dip Direction°**	**Concentration (%)**	**Comments**
UG Mapping Data	1a	55/167	20.6	Joints and contacts
	1b	77/349	5.0	Joints; same as 1a
CM06-984	2a	67/015	16.6	Foliation and veins
	2b	89/173	6.0	Foliation and veins, same as 2a
	3	17/320	4.0	Joints and veins

Set 1a in the underground mapping data dips 55° south, parallel to the greywacke-porphyry contact and the Sladen Fault in this area. Set 1b is parallel to, and probably represents bedding. Set 2a in CM06-984 dips steeply north to moderately northeast, and is probably a combination of north-dipping bedding and northeast-dipping foliation (and foliation-parallel veins). Shallow, northwest-dipping joints and veins in Set 3 form a smaller concentration of poles in CM06-984, but do not occur in the underground mapping data.

5.6.8 South Sector

Prominent structure concentrations indicated in structure data sources for the South Sector are (Appendix G.4):

SOUTH SECTOR STRUCTURE CONCENTRATIONS				
Data Source	**Set ID**	**Dip°/ Dip Direction°**	**Concentration (%)**	**Comments**
UG Mapping Data	2a	87/176	19.4	Joints, faults, and veins
	2b	87/356	16.0	Joints and faults; same as 1a
GT07-04	2a	89/187	7.0	Joints; same as 1a
	2b	82/007	8.0	Joints, veins, and foliation
	3	07/198	3.0	Joints
	5	71/069	2.0	Joints
GT07-05	1	68/037	6.0	Joints, foliation, and veins
	2b	75/014	7.2	Joints, foliation, dikes, and veins
	3	06/218	3.0	Joints
CM07-1100	1	60/038	10.0	Bedding/foliation, veins, and joints
	4	12/350	15.1	Joints and veins
CM07-1108	1	65/039	7.0	Veins, bedding/foliation, and joints
	4	22/359	5.0	Veins and joints

Sets 1 and 2 represent distinct concentrations within a broader, northwest-striking structural trend. Moderate to steep, northeast-dipping structures comprising Set 1 occur in all of the data sources, although they are not strongly-developed in the underground mapping data. The average dip of Set 1 in drillholes CM07-1100 and CM07-1108 may be biased flatter, as these were drilled vertical, and would tend not to encounter steeper structures. Set 1 is parallel to the Northwest Conjugate Fault, and also to foliation.

Set 2 is present in the underground mapping data and the oriented coreholes GT07-04 and GT07-05. These structures probably represent bedding. They do not occur in data from drillholes CM07-1100 and CM07-1108, but these are vertical drillholes, and would therefore tend not to intersect Set 2 structures.

Sets 3 and 4 are comprised of shallow, south- or north-dipping joints and veins that are present in the drillhole-based data sources, but not in the underground mapping data.

5.6.9 West Sector

Prominent structure concentrations indicated in structure data sources for the West Sector are (Appendix G.5):

WEST SECTOR STRUCTURE CONCENTRATIONS				
Data Source	**Set ID**	**Dip°/ Dip Direction°**	**Concentration (%)**	**Comments**
UG Mapping Data	1b	86/225	10.5	Joints
	1a	90/060	12.0	Joints
	4	42/090	12.0	Joints
	2b	88/190	6.0	Joints
	5	71/088	6.0	Joints
CM05-693	1a	67/060	8.8	Veins and joints
	2a	65/007	3.0	Veins and joints
	2b	77/176	3.0	Veins and joints
	3	20/011	3.0	Veins and joints
CM07-1029	1a	57/042	5.0	Bedding, veins, and joints
	3	20/035	6.9	Veins and joints

Set 1 occurs in all data sources, and represents steep, northwest-striking structures parallel to foliation and the Northwest Conjugate Fault. Set 2, present in the underground mapping data and in drillhole CM05-693, is comprised of steep, east-west-striking joints and veins that are parallel to bedding.

East-dipping joints in Sets 4 and 5 form concentrations of poles in the underground mapping data but are relatively sparse in the televiewer drillholes. Shallow, north to northeast-dipping joints comprising Set 3 are present in the televiewer drillholes but not in the underground mapping data.

5.6.10 Design Rock Structure Characterizations

For stability analysis purposes, structural sets that are considered to be systematic in a design sector are designated as “Design Sets.” These correspond with rock structure that occurs in multiple data sources as described in the previous sections; and with the orientations of major faults.

Concentrations of structures that are not strongly developed in the rock structure data are considered to be of secondary importance for pit slope designs. These structures could affect slope performance locally, but are not regarded as potential controls of slope angles.

Design discontinuity sets by design sector are (Figures 9-11):

DESIGN DISCONTINUITY SETS			
Sector	**Structure ID**	**Dip°/ DDR°**	**Comments**
Northwest	1a	79/172	Sladen Fault/greywacke-porphyry contact
	1b	67/359	Bedding Set
	2	48/126	"Intersection" fault zone
	3	70/050	Northwest Conjugate Fault
Northeast	1	38/158	Sladen Fault/greywacke-porphyry contact?
	2a	77/188	Bedding/Sladen Fault/greywacke-porphyry contact?
	2b	81/009	Bedding
	6	51/161	Sladen Fault/greywacke-porphyry contact?
	7	60/180	Nominal orientation of Sladen Fault/greywacke-porphyry contact
East	1a	55/167	Sladen Fault/greywacke-porphyry contact
	1b	77/349	Bedding-parallel joints
	2a	67/015	Foliation/bedding
	2b	89/173	Foliation/bedding
South	1	64/038	Northwest Conjugate Fault, foliation
	2a	88/182	Bedding
	2b	81/006	Bedding
West	1a	71/054	Northwest Conjugate Fault, foliation
	1b	86/225	Foliation
	2a	65/007	Bedding
	2b	83/183	Bedding

6.0 STABILITY EVALUATION

6.1 Kinematic Analyses

The stability of slopes in competent rock can be controlled by structures, or combinations of structures that define kinematic failure modes. These involve movement of intact blocks along one or more discontinuities, typically in planar, wedge, toppling, or combination modes. Stability with respect to these failure modes is a function of slope orientation, discontinuity orientation, discontinuity shear strength, groundwater conditions, and external forces. The rock structure and discontinuity shear strength characterizations detailed in the previous section comprise the engineering geological input to kinematic analyses; slope geometry information was taken from the ultimate pit plan in Figure 5.

Kinematic failure modes can affect bench-scale, inter-ramp, or overall slopes, depending on the continuity of the structures involved. As such, persistence is an important consideration in evaluating the significance of any potential failure modes that are indicated by stereonet-based analyses. Until exposure during mining allows persistence to be evaluated directly, structures represented by the Design Sets (Figures 9 to 11) must all be considered continuous on an inter-ramp scale.

As a first approximation, the potential for kinematic failure modes can be determined stereographically, and the main elements of this type of analysis are the structure data, slope orientation, and discontinuity friction angle. These are shown on the major planes plots in Figures 9-11. If stereonet-based analyses indicates the potential for planar or wedge failure modes, the Factors of Safety (FOS) with respect to these modes can be determined with limit equilibrium stability analyses. Limit equilibrium solutions are also available for analysis of toppling failures, but do not result in explicit FOS.

6.1.1 Northwest Sector

The Northwest Sector is the north wall, west of about 712950E, where the wall dip direction is essentially constant at 180˚. The Sladen Fault is expected to intersect the wall in this sector in the middle benches or below. The Northwest Conjugate Fault is expected to intersect the wall immediately east of the boundary between this sector and the West sector.

Potential kinematic controls of stability in this sector (Figure 9) are toppling associated with bedding (Set 1b, 67°/359°); wedge intersections involving the “intersection” fault zone (Set 2; 48°/126°) and structures parallel to the Northwest Conjugate Fault (Set 3; 70°/050°), with trend/plunge of 117°/48° and FOS<1; and steep planar control of bench face angles along the Sladen Fault and parallel structures (Set 1a, 79°/172°).

The analysis indicates there is potential for toppling along well-developed, east-west-striking structures that dip steeply into the walls. Toppling may be further facilitated by shallow joints that occur throughout the pit, which would act as base planes for toppling blocks. Although toppling may affect bench stability locally in this sector, we do not consider this failure mode to be a limitation on slope designs due to the high intact and rock mass strengths, and the lack of extensive clay development along structures.

Wedges form only for slope angles greater than about 65°, so instability should be limited to back-break of bench faces and local reduction of catch bench width, and they are not considered a constraint on pit slope designs.

Provided rock quality is not significantly reduced in the vicinity of the Sladen Fault, this fault and structures parallel to it may enhance stability by facilitating the development of steep bench face angles.

6.1.2 Northeast Sector

The Northeast Sector is the north wall, east of about 712950E. Wall dip direction in this sector is 180˚ between 712950E and about 714450E, and changes to 210˚ in the east. Potential structural controls of stability in this sector (Figure 9) include toppling associated with Set 2b (81°/009°); and planar control along structures in Set 1 (38°/158°), Set 6 (51˚/161˚), and Set 7 (60°/180°).

In the Northeast sector, statistical average dip directions for Set 1 vary in the individual data sources. In most data sources, the average dip directions and dips indicate that Set 1 will not be problematic with respect to slope stability. However, the average dip directions of Set 1 in CM07-1500 and CM07-1540 are within 10° of the dip direction of the wall in the west half of the sector. Additionally, the dip direction of Set 6 in CM07-1446 is within 20° of the wall dip direction in the west half of the sector. Because these structures are indicated in televiewer data, whether they represent open discontinuities or intact features is unclear. In the west half of the sector, where these drillholes are located, open discontinuities in Set 1 and Set 6 could control bench stability where they dip steeper than the discontinuity shear strength.

Set 7 is the nominal orientation of the Sladen Fault/greywacke-porphyry contact above approximately 150 m depth. Structures with this orientation are present in the pit wall, as indicated by Set 2a in the underground mapping data. Bench faces may break back along these structures where they are well-developed in the pit wall.

6.1.3 East Sector

The East Sector comprises the east wall of the pit, with wall dip directions of 300˚ to 310˚. No strong indications of structural control of overall slope angles or bench configurations are indicated for this sector (Figure 10).

The Sladen Fault and the porphyry-greywacke contact will intersect the wall in the area of the sharp transition in wall orientation between the Northeast and East Sectors. These structures will strike roughly perpendicularly to the wall in this area. Depending on rock quality, bench-scale instability and rock fall could be a concern between these structures, but we do not consider them to be a constraint on pit slope designs.

6.1.4 South Sector

The South Sector comprises the entire south wall, which has a dip direction range of 330˚ to 025˚. Potential structural controls in this sector (Figure 10) include steep planar control of bench face angles along Set 2b (81°/006°) in the east half of the sector; and toppling associated with Set 2a (88°/182°). The Northwest Conjugate Fault is expected to intersect the south wall, possibly in the central area, but is not unfavorably oriented with respect to stability. As discussed above, toppling may locally affect bench stability, but is not considered a limit on slope design. Steeply-dipping set 2a may enhance stability by facilitating the development of steep bench face angles.

6.1.5 West Sector

The West Sector is the west wall of the pit, where the dominant wall dip directions are 125˚ and 70˚, although the full range of wall orientations is much wider (325°-000°-160°) because of the horseshoe-shaped excavations in the upper benches. As shown in Figure 11, potential structural controls of slope angles indicated by the rock structure characterization for this sector are limited to wedges formed by the intersection of bedding (Set 2b, 83°/183°) with structures parallel to the Northwest Conjugate Fault (Set 1a, 71°/054°). The nominal trend/plunge of the wedges is 106°/61°, with FOS<1. These wedges are adversely-oriented for the north end of this sector (wall dip direction 125˚), but given that the plunge of these wedges is steeper than the inter-ramp angles, related instability should be limited to back-break of bench faces, and local reduction of catch bench width.

6.2 Rock Mass Stability Analyses

Where there is no simple structural control of stability, estimates of rock mass strength are used to evaluate the stability of slopes in rock masses. Rock strengths and RQD are high at the Canadian Malartic project, and rock mass ratings and Hoek-Brown strength parameters indicate good quality rock masses (except for SCH, for which little data is available). This indicates a low probability for rock mass failure for the moderate slope heights in the Canadian Malartic pit. However, analyses completed to support an evaluation of the setback distance for the waste dump that will be constructed south of the pit provide a measure of rock mass stability that is representative of other slopes developed in similar competent rock.

6.3 Placement of Waste Dump

The ultimate pit plan shows a waste dump approximately 280 m south of the ultimate pit crest (Figure 12). The waste dump is 85 m high and approximately 2300 m long, with outslopes at 2(H):1(V), or 27°. Osisko is evaluating the viability of moving the waste dump closer to the pit crest, provided that stability analyses demonstrate that the waste dump would not adversely affect the stability of the south and southeast walls of the pit.

A stability section was developed for the southeast wall at the location shown in Figure 12. Although a geological model for this area has yet to be developed, geologic logging is available for corehole CM07-984, which was drilled across the section at a 40° angle. The drillhole encountered dominantly

AGR (307 m total), with local narrow intervals of CGR (42 m total), SGR (14 m total), CPO (16 m total), SPO (13 m total), and one interval of SCH (2 m) interspersed throughout the drillhole. While AGR comprises approximately 75% of the rock encountered in the drillhole, the strength of this unit is higher than the others, and using a rock mass strength based solely on AGR could tend to overestimate the actual composite rock mass strength. Instead, the rock strength in the section was modelled using rock mass strength parameters for CGR, which are lower than all other units encountered in corehole CM07-984, except for SCH. Given that SCH occurs in relatively narrow dikes, characterization of the rock mass strength along this section with the parameters for CGR was considered to be a reasonably conservative approach.

Two separate cases were analyzed, one with the waste dump located 100 m from the pit crest, and the second with the waste dump located 50 m from the crest. The slope was conservatively assumed to be fully saturated.

Analyses were run using the Rocscience limit equilibrium analysis program Slide 5.0 (Rocscience, 2007b). This software was used to generate Factors of Safety according to a Spencer solution for circular slip surfaces. Minimum calculated Factors of Safety were 2.1 for the waste dump located 100 m from the pit crest and for the waste dump located 50 m from the pit crest (Figure 14), indicating a low risk of rock mass failure for both cases. The current plan is to construct the waste dump with outslopes at 3(H):1(V), or 18°, rather than the 2(H):1(V), or 27° slopes that were used for the analyses. This will move the crest of the waste dump approximately 80 m farther back from the crest of the pit.

As discussed above, rock mass strength in this section was modelled conservatively using strength parameters for CGR. Also, slope heights and inter-ramp and overall slope angles are the same or lower throughout the pit except for the west end of the South sector, which is 20 m higher. Because the material properties and slope configuration in this section are equivalent to those in the other areas of the pit, this analysis demonstrates that the Factor of Safety against rock mass failure is high throughout the pit.

7.0 DESIGN APPROACH

7.1 Standard of Care in Pit Slope Development

Rock slope design for open pit mines and quarries includes consideration of both mining economics (the steepness and overall stability of the slopes) and operating safety (particularly mitigation of rockfall hazards). Design factors affecting pit economics can be modified to optimize financial returns. Design factors related to safety can not be compromised, whether for permanent or temporary slopes, and slope designs must be implemented to meet the current standard of care in the mining industry for operating safely below rock slopes. This standard includes incorporating effective catch benches into pit slopes.

The minimum standard of care for safety in development of mine slopes in the United States is defined by Federal regulations that are enforced by Mine Safety and Health Administration (MSHA), or by equivalent State agencies using State regulations that can be no less stringent than Federal regulations. In Canada, Provincial mining codes regulate mine safety and slope design, and the Canadian Malartic mine will be regulated under Quebec codes. In addition, operating practices and slope designs to enhance operator safety are often developed at the Corporate level, and these may be supplemented at the Operating level based on site conditions at individual pits.

While the Canadian Malartic mine will be regulated under Quebec codes, North American codes in general aim at achieving safe slope through similar mechanisms of requiring effective catch benches for protection against rockfall, and an understanding of different codes can help in assessing alternative slope design options. Mine slope stability requirements in the United States are regulated by Title 30 of the Code of Federal Regulations, Section 56.3130. This Section requires that mining methods shall maintain slope stability in places where persons work or travel in performing their assigned tasks, and that bench configurations be based on the type of equipment used for scaling.

MSHA provides interpretation guidelines for ground control. These indicate that MSHA requires that a bench adequate to retain rockfall must be maintained above work or travel areas. Where there is not an effective catch bench above a work or travel area, other measures must be taken to protect the miners, such as berming off or ceasing mining in the affected area.

Quebec regulations respecting occupational health and safety in mines are covered by R.S.Q.c.S-2.1, r.19.1 (refer to http://www.canlii.org/qc/laws/regu/s-2.1r.19.1/20080515/; see Appendix H). Division III of this regulation concerns Work Environment, and Sections 40 and Section 41 present relevant requirements, with those in Section 41 specific to open pits developed in bedrock. These regulations indicate that catch benches are required, and that bench faces must be scaled prior to working below. In addition, for pits over 25 m deep,

> "the contours of the profile of the working faces and walls shall be determined so as to ensure their stability and drilling and blasting must be controlled so that the intended profile of the faces and walls may be maintained"

Mine slopes in rock are generally designed with catch benches for the sole purpose of providing safe working conditions; catch benches do not generally enhance the overall stability of the slope. A slope where effective catch benches are not developed in accordance with the design is not in compliance with the important safety considerations of the slope design, and is explicitly not in compliance with Quebec mining regulatory standards. This may arise due to geotechnical conditions that are different from those assumed for the design, or due to operating practices that do not achieve the design bench configurations. In either case, it is important to recognize the failure to meet design criteria early so that suitable modifications to the slope design and/or to the operating procedures can be implemented to maintain an adequate level of safety and protection against rockfall during continued development of the slope.

When changes in operating procedures can not produce adequate catch benches, then design modifications to the slope are required. Most commonly, this will involve incorporating a step-out to provide suitable catchment for any rockfall hazard above that level, and a flatter slope design with wider catch benches below the level at which the design changes are implemented. This will commonly result in reduced recovery of resource. An alternative where there are no property or geological constraints at the slope crest is to lay back the slope to a flatter angle with wider catch benches from the top of the slope, which may result in increased stripping. Mechanical stabilization of benches or restraint of rockfall by fences or berms can be warranted locally in some circumstances, but are rarely feasible alternatives on a large scale.

Because catch benches are designed to provide safe working conditions below the slope, it is equally important to develop effective catch benches for both Ultimate and interim or Phase slopes, since the safety risks are identical for both cases. Where the Ultimate slope is designed assuming expensive perimeter blasting techniques, it may be warranted to avoid perimeter blasting costs for Phase slopes by accepting flatter interim slopes with alternative design bench configurations.

7.2 Controls and Limitations for Slope Design and Development

Rock quality and structural conditions generally appear favorable for the development of steep inter-ramp slopes in all areas of the pit except the Northeast sector. Specific conclusions supported by the characterization and analyses described in the previous sections include:

- Rock mass strength is sufficiently high to preclude rock mass failure of overall slopes. The slope angles that can be achieved will depend on the stable bench configurations that can be developed safely; this will be a function of both structural conditions and operating practices.
- Groundwater is not expected to be a control of rock mass stability, and slope dewatering required for pit operations is likely to be sufficient for maintaining rock mass stability.
- There is little clay alteration evident, and low shear strengths associated with clay alteration are not anticipated to be a control on large-scale or bench stability.
- Kinematic stability analyses indicate potential for significant structurally-controlled planar failures along moderately south-dipping structures in the Northeast sector, and bench face angles should be designed at 60° to reduce back-break along these structures.
- Bench and slope stability in the Northeast sector will be sensitive to variations in the dips of in-dipping structures and also to the presence of groundwater pressures along structures where slope stability is marginal. While pit dewatering is indicated to result in generally depressurized slopes, this should be verified by monitoring, particularly for the footwall slope where stability may be sensitive to groundwater pressures and anisotropy may affect drainage characteristics.
- In other pit sectors, where high risks of planar failures have not been identified, steep bench face angles should generally be achievable with careful blasting, excavation, and scaling. While wedge or toppling failures may occur locally, available structural data does not indicate these failure mechanisms to be a widespread control on bench design.
- Zones of intense fracturing that may be encountered in association with faulting or elsewhere and that could result in a decrease in stable bench face angles are currently understood to be relatively narrow, and will affect bench stability over only limited lengths that should not require major slope re-design.
- Available data indicates that the schist (metamorphosed mafic dikes) is competent rock. The characteristics of the schist should be continually evaluated as additional data becomes available, and as the unit is exposed during mining.

7.3 General Risks Associated With Steep Slope Designs

There appears to be little risk of large-scale rock mass or structurally controlled slope failures developing based on our current understanding of engineering geologic conditions. Risks of failure to achieve design slope angles appear to be predominantly related to the risk of encountering locally unfavorable structural conditions, particularly in the Northeast sector of the pit, else damage to the benches due to poor operating practices. In either case, the result is likely to be a safety hazard resulting from the increased risk of rockfall to benches below, particularly during or immediately following precipitation events or earthquakes. With conservative slope designs, the risk of encountering conditions that will require a modification to the slope design is always less than with more aggressive slope designs. If mining has proceeded below a hazardous area, there is often little alternative for re-establishing safe working conditions other than incorporating a step-out into the slope design to provide additional catchment for rockfall. With an aggressive slope design, the effects of a step-out on inter-ramp slope angles is greater, and the potential for recovering to the original slope design by locally over-steepening the slope is less. This is particularly the case for high slopes that do not incorporate haul ramps to flatten the overall slope angle to less than the inter-ramp slope angle.

8.0 SLOPE DESIGN RECOMMENDATIONS

8.1 General

Since there appears to be little potential for rock mass or structural control of overall or inter-ramp slope angles, achievable pit slope angles will be determined by the bench configurations that can be safely developed and maintained. Bench configurations are defined by production bench height, achievable bench face angle (BFA), and catch bench width, all of which combine to define the inter-ramp angle (IRA) as shown in Figure 15. Our recommended bench configurations are given in terms of these parameters, but include the following assumptions:

- Production bench height is 10 m.
- Maximizing slope angles will require that a multiple bench configuration be mined. A double bench configuration in fresh rock is a reasonable assumption at the current stage of pit planning.
- Catch bench widths should be sufficient to provide effective protection against rockfall. The following modified Ritchie criteria (Ryan and Pryor, 2000) is commonly used for initial estimates of design catch bench width:

  Catch Bench Width (m) = (0.2 x Bench Height) + 4.5 m.
- For 20 m height between catch benches, this results in a design catch bench width of 8.5 m.
- Minimum design catch bench widths in the mining industry are commonly taken as 6 m to allow for back-break and hard toes due to imperfect blasting, and local bench crest failures due to structural conditions; narrower design catch benches are not generally assumed unless operating experience can demonstrate that effective catch benches can be constructed at narrower designs
- BFA and IRA are limited by structural control for slopes oriented within +/-20° of the dip direction of structures involved in planar failure modes, and +/- 45° of the trend of potential wedge failure modes.
- If strong structural control at flatter angles is lacking, the following BFA ranges are typically achievable, depending on rock quality and blasting methods:
  - Standard production blasting - 55°-65°
  - Effective controlled blasting - 65°-70°
  - Best-case controlled blasting - 70°-75°
- Since catch bench width for a given bench height is constant according to the modified Ritchie criteria, maximizing the IRA will be contingent on excavating the BFA as steep as possible. We have assumed a BFA consistent with the implementation of effective controlled blasting at the pit limit.

8.2 Slope Design Rationale

Using the average orientations of systematic discontinuity sets as a basis for evaluating structural control of bench stability is a reasonable approach given the currently available structure data. Kinematic analyses based on these average orientations indicated the potential for structural control of bench face angles along moderately south-dipping structures in the Northeast sector. Bench face angles should be designed at 60° to reduce bench-scale planar failures along these structures.

Slopes could be designed to avoid all possible failures, but this would result in flat slope angles and impacts on economics that may be unnecessary. Rather than designing to avoid all failures, it is generally preferable from a planning perspective to assume the risk that some intermittent failures will occur, provided that failures are not pervasive and can be contained by effective catch benches, and there is a low risk of failures that would jeopardize overall slope stability.

8.3 Recommended Slope Designs and Bench Configurations

8.3.1 All Sectors Except Northeast Sector

Available structural data indicate favorable structural conditions for moderately steep to steep slopes in these areas. Bedrock should be double-benched, with design 9-m catch benches at vertical intervals of 20 m. While it may be possible to triple bench slopes in this excellent rock where structural conditions are favourable and operating practices are exceptional, 30 m between catch benches is very high, and we do not recommend assuming that triple benching will be safely achievable for these relatively high production benches for feasibility-level designs. Triple benching should only be considered as part of the slope optimization process as pit development enables detailed documentation of geological conditions and evaluation of the effect of operating practices on slope performance.

Slopes designed with 9 m catch benches at 20 m intervals and design bench face angles of 69° will result in an inter-ramp slope angle of 50°. This design should be readily achievable with good buffer blasting, and should be used for slopes where an effective pre-split is not implemented due to cost or lack of equipment or a skilled labor force. This design may be appropriate for phase walls if adequate ore production rates can be maintained without maximizing angles for interim slopes.

Steeper slopes will be possible for slopes that warrant the care and cost of an effective pre-split, such as final pit slopes. A pre-split should enable development of steeper, stable bench faces, with design bench face angles of 75°. Slightly narrower catch benches should be acceptable for a slope that is effectively pre-split; slopes designed with 8.5 m catch benches at 20 m intervals and 75° bench face angles will result in an inter-ramp slope angle of 55°. This is an aggressive slope design that will only be successful where geological conditions are favourable and a high level of skill is applied to perimeter blasting and scaling. The cost and level of care in drilling, blasting, and scaling will be similar to that implemented to achieve the steep pit slopes in the Fennell pit at Omai. Drilling a double height (20 m) inclined pre-split angled at 15° from vertical would be the preferred method of developing the bench faces if hole alignment can be maintained over 20 m, and there is not strong structural control of the bench face at a different inclination. Double height pre-splits eliminate the need for two setups, and eliminate the need for a stepout between pre-split rows that often results in a ledge that can launch rockfall and increase its horizontal trajectory. Inclined pre-splits reduce back-break at the crest, and facilitate development of clean toes.

Recommended slope design configurations are illustrated in Figure 16.

8.3.2 <u>Northeast Sector</u>

Available information indicates that the Sladen Fault/greywacke-porphyry contact dips ~45°-60°S in this sector. For feasibility-level slope designs, bench faces in this sector should be designed at 60° to reduce back-break along structure related to the Sladen Fault, and to allow development of effective catch benches.

Some south-dipping structure in this sector is indicated to dip flatter than the indicated dip of the Sladen Fault/greywacke-porphyry contact and the Sladen-parallel underground mapping data. Average dips are on the order of 40°, but the dip is variable in the underground mapping data and the oriented core data, ranging from 25° to 70°. Although these are strong concentrations of poles, particularly in the televiewer data, as discussed previously structures identified in the televiewer images may not always represent open discontinuities in the core. Where these south-dipping structures are represented by planes of mechanical weakness, they can be expected to control bench stability in the west half of the sector where they strike within 20° of the strike of the pit slopes, and dip steeper than the discontinuity friction angle. When encountered they may cause local loss of catch benches due to back-break. These effects will be decreased as the slope orientation changes toward the east; the highest risk to the slope design is in the west half of the sector.

A double bench configuration should be used, with 8-m catch benches at 20 m vertical intervals, for an inter-ramp slope angle of 46° as illustrated in Figure 16. Careful blasting will be required to break the bench faces cleanly to the structure. In view of the indicated variability of the dips of the structures, actual slope configuration may vary from this nominal design and should be optimized in the field. Where the mineralized contact flattens in the lower portion of the slope, flatter bench face angles may develop as the dip of predominant structure decreases, but overall slope angles are relatively flat due to grade control and the incorporation of the haul road, and so flatter bench face angles are not expected to be a constraint on slope design.

8.3.3 Overburden, All Areas

Overburden is less than 10 m thick and consists of glaciolacustrine deposits, till, and local soft sediment deposits. Overburden should be excavated at 2(H):1(V). The first bench, which will be excavated partially or entirely in overburden, should be a single bench and should incorporate an 8 m catch bench.

8.4 Summary of Recommended Bedrock Slope Designs

The following design slope configurations are recommended for feasibility-level pit slope designs.

RECOMMENDED SLOPE DESIGNS IN BEDROCK					
Wall	**Operating Practices**	**Bench Configuration and Height (m)**	**Catch Bench Width (m)**	**Bench Face Angle (°)**	**Design Inter-Ramp Slope Angle (°)**
All sectors except Northeast sector	Buffer Blasting	Double Bench 2 x 10 m 20 m between catch benches	9	69	50
All sectors except Northeast sector	Pre-Split	Double Bench 2 x 10 m 20 m between catch benches	8.5	75	55
Northeast sector	Controlled Blasting to Break Cleanly Along Structure	Double Bench 2 x 10 m 20 m between catch benches	8	60	46

Incorporate a haul road or wide bench below slopes 200 m to 250 m high to de-couple the slopes.

8.5 Water Pressures in Slopes

Rock mass strength appears to be sufficient to preclude rock mass failure even if high water pressures are present within the slopes, so natural draindown that will develop in the slopes due to the dewatering necessary for operating purposes will likely be adequate for most slopes. However, structurally controlled failures that could develop in the Northeast sector would be sensitive to the presence of groundwater pressures, and these slopes should be effectively de-pressurized in the vicinity of the pit wall to enhance stability. Piezometers should be used to verify that slopes in this sector are effectively de-pressurized.

Also, maintaining a high hydraulic head in the pit walls is not recommended, as it can exacerbate damage related to freeze/thaw and problems caused by local adversely-oriented structure, and tends to amplify blast energy in the pit walls. A hydrogeological program to include monitoring of water pressures in the vicinity of the pit slopes during pit development should be used as a basis for assessing potential impacts of groundwater pressures on pit slopes and the need to enhance dewatering for geotechnical purposes.

8.6 Risks

Bench scale failures can be expected locally where structural conditions are unfavourable or blasting practices are poor. Additionally, seasonal variations in temperature such as winter freeze and spring thaw may adversely affect bench stability and increase rockfall hazards. More aggressive slope designs always result in increased risks of instability. Our current understanding of the site geology indicates little risk of large-scale slope failures due to increased slope angles. However, in a geological environment, local structural conditions can always vary from the average. This can result in local instability, typically at a bench or multi-bench scale, and the risk of such local instability developing increases with steeper slope angles.

Existing structural data include some south-dipping structures in the Northeast sector that dip at angles flatter than the recommended design bench face angles. While current data indicate that these will not be widespread controls of bench stability due to their orientations or distributions, these structures represent the potential for local structural control where they are strongly developed. Conflicting data in the Northeast sector of the pit currently indicate that the majority of structures will be steeply-dipping or will strike such that they do not control bench stability, but that local adjustments to slope design or orientation may be required, particularly in the west half of the sector, if unfavourably oriented structures are determined to be well-developed during mining. The west half of the Northeast sector should be considered a higher risk area because structural conditions are not well understood and available data are conflicting; careful monitoring and documentation of this area is critical for maximizing stability and optimizing design during slope development.

Stopes in the underground mines up to 50 m high and 80 m wide intersect the north wall in a number of places and the south wall in several places. Local modification to the slope design may be required in these areas, such as moving the wall back behind the stope or drift. Care must be taken during drilling and excavation of these areas to avoid injury to personnel and damage to equipment due to collapsing underground workings.

The risks associated with slope development increase with inter-ramp slope height. It is common practice to break steep inter-ramp slopes with a ramp or stability bench at intervals of 200 m to 250 m to mitigate these risks. Higher slopes designed without a break have an increased risk of having to incorporate an unplanned stability bench due to unfavourable geological conditions or operating practices. Where tolerance to the risk of failures is low, such as slopes near community facilities or tailings impoundments, we recommend mitigating the risks by incorporating a haul road to reduce inter-ramp slope heights and to reduce the overall slope angle.

8.7 Opportunities

Steeper slope angles may be feasible by optimizing these slope designs based on the documented geological conditions and actual performance achieved in the field. Excellent field performance may warrant increasing the design bench face angle, or slightly reducing the design catch bench width. However, we do not recommend assuming steeper design slope angles without field verification of slope performance.

There is potential for increasing the bench face angle in the Northeast sector to greater than 60° if the south-southeast-dipping structures are not well developed and benches do not break back along them. During phase pit development, if these structures are determined to be poorly-developed, bench face angles can be increased as part of a systematic slope optimization process, potentially matching design slope angles in other pit sectors.

Triple benching may be possible if clean and stable bench faces are developed that would enable safe drilling and excavation of a third bench. However, it would likely be impractical to implement a mix of double and triple benches, so this is most likely to be practical locally where slope geometry conditions are suitable, such as above ramps.

9.0 OPERATING CONSIDERATIONS

9.1 Double Benching Practices

Double benching operations differ from single bench operations primarily because drilling and charging of the second bench must be conducted closer to the overlying bench face, and excavation of the second bench must be completed directly below the upper bench. This increases potential rockfall hazards during drilling, charging, and excavation operations. Safe double benching therefore requires more care in drilling, blasting, and scaling operations to mitigate the potential increase in risks of rockfall hazards. A safe double benching operation generally requires that the following be implemented:

- Controlled blasting plan to reduce damage due to blasting, particularly in the upper bench;
- Blast optimization program to determine the optimum blasting procedures for site conditions;
- Thorough bench face scaling to reduce risks of rockfall using equipment that can safely reach the top of the bench to scale loose rock;
- Inspection and monitoring program to ensure that conditions are safe for initiating drilling and loading of blastholes below the upper bench;
- Geological documentation and geotechnical evaluation program to ensure that the conditions assumed for the double bench design are met in the field;
- Operator awareness training to train operators in safe practices, and to educate operators regarding potential hazards of double benching.

9.2 Controlled Blasting

9.2.1 General

Where geological structure does not control slope and bench stability, the achievable inter-ramp slope angle depends on the stable bench face angles that can be developed. In the absence of structural control, achievable bench face angles are largely a function of drilling, blasting, and operating practices. Poor blasting that damages and disturbs the rock behind the design bench face, and poor scaling or excavation of the bench faces, will result in flatter bench face angles and flatter inter-ramp slopes. Blasting and operating practices that develop steep, stable bench faces enable the safe development of steeper inter-ramp slopes. The costs of the improved drilling and blasting practices result in benefits of improved operator safety and reduced stripping from more stable benches and steeper slope angles.

A careful wall control blasting program will be essential to develop steep, stable bench faces and to maximize inter-ramp slope angles at Canadian Malartic. In massive, unfractured to lightly fractured rock, pre-split blasting using standard size production drillholes or slightly reduced drillhole diameters on reduced spacing is generally effective. However, as the intensity of fracturing increases, it is generally necessary to reduce the diameter and spacing of pre-split holes. In more fractured rock, cushion blasting is generally more cost effective, but cushion blasting rarely achieves the steep bench face angles assumed for the steep slope designs in fresh rock at Canadian Malartic.

Bedrock at Canadian Malartic is generally characterized by a Good quality rock mass with average RQDs in the geotechnical drillholes ranging from 77% to 94%. Pre-split blasting is generally effective in good quality rock. Buffer blasting is generally more effective in poor quality rock, such as where the RQD is less than about 50%, and also generally produces satisfactory results where flatter bench face angles are controlled by geological structure.

9.2.2 Controlled Blasting Recommendations

9.2.2.1 Pre-Split Blast Design

Pre-split blasting should be used to maximize stable bench face and inter-ramp angles along final walls. Pre-split blasting consists of drilling a row of closely-spaced holes along the design excavation limit, charging them lightly, and then detonating them simultaneously or in groups separated by short delays. Firing the pre-split row creates a crack that forms the excavation limit and helps to prevent wall rock damage by venting explosive gases and reflecting shock waves. The pre-split row is fired in advance of the adjacent trim blast, which must be designed to limit damage beyond the pre-split line. The trim blast includes a “Buffer” row adjacent to the pre-split row to break back to, but not beyond, the pre-split, by ensuring that the Buffer row is fired with good horizontal relief.

Optimizing the Buffer row location and charge is often the most challenging task when implementing a pre-split, since it must be close enough to the pre-split to break the toe, but should not create excessive crest back-break. The Buffer row must have a reduced spacing to enable clean breaking along the pre-split line, and a reduced burden so that it can easily push its burden away from the pre-split. Also, the Buffer row must be fired with effective horizontal relief; otherwise, instead of moving the broken ground forward to the free face, if it is fired in a choked condition it will break equally in both directions and will cause excessive back-break across the pre-split line due to flexural rupture and block heave.

We understand that most drilling will be 8-inch diameter blastholes on a 6 m x 6.6 m pattern, but 4.5-inch to 5-inch blastholes will be used in some sensitive areas. While every pre-split design should be optimized based on site conditions, the following general guidelines provide a starting point for initial design of a pre-split in competent rock using 8-inch and 5-inch diameter blastholes.

- Spacing between blastholes in the pre-split line is typically about 12 blasthole diameters in hard rock, or about 2.4 m with 8-inch pre-split holes (1.5 m with 4.5- to 5-inch pre-split holes).
- Charge weight per pre-split hole = 1 kg/m^2 of wall area created, or 24 kg for 10 m holes on 2.4 m spacing, and 15 kg for 10 m holes on 1.5 m spacing.
- Distributed charges will give better results than toe charges, but toe charges have produced satisfactory results at many other operations where geological conditions are favorable, and can likely produce satisfactory results for 10 m bench heights. However, distributed charges would likely be necessary if pre-split holes are drilled to double bench height (20 m).
- If de-coupled distributed charges are used, load to about 8 blasthole diameters from the drillhole collar (1.6 m for 8-inch blastholes, and 1 m for 5-inch blastholes).
- Inclining the pre-split line 10° to 20° off vertical will reduce back-break and improve toe breakage where there is no prominent structure for the pre-split to break along or if it is inclined with the pre-split row. Vertical pre-splits should be used if there is prominent vertical structure.
- Pre-split holes should be detonated simultaneously, or nearly so, and at least 50 ms before the first holes of the adjacent trim rows
- The Buffer row (adjacent to the pre-split row) should be located ⅓ to ½ the normal burden distance in front of the pre-split line. For a nominal burden of 6 m between production rows, this indicates a standoff distance from the toe of the pre-split row of 2 m to 3 m.
- The burden and spacing of the Buffer row is typically about ½ to ⅔ of normal production holes – 50% is often used to facilitate pattern tie-in. This suggests about 3 m burden and a 3.3 m spacing, given the planned 6 m x 6.6 m production pattern.
- Powder factor for the Buffer row should be the same as for the production holes (0.31 kg/t). Charge weights of the Buffer holes are reduced to account for their reduced burden and spacing. The charge weight in Buffer row blastholes with 3 m burden and 3.3 m spacing should be reduced to about 25% of the charge of a 6 m x 6.6 m production pattern
- No sub-grade drilling above final benches or in the vicinity of bench crests.

- No stemming is preferred in pre-split holes; if stemming is necessary for noise control, stem only at collar using minimum stemming.
- Air deck Buffer row to reduce confinement and limit crest damage, or alternatively reduce stemming if oversize rock from the upper portion of the bench is a concern.
- The trim shot in front of pre-split line, and particularly the Buffer row, must not be confined but must fire to a free face to prevent damage behind the pre-split line.

For double benching with limited height production benches, drilling and developing a double height pre-split often increases efficiency, enables development of steeper bench face angles, and eliminates potential ledges between lifts. However, distributed charges become necessary for longer blastholes.

The design of the pre-split should be optimized in the field based on performance. A successful pre-split will include:

- Stable bench faces with the pre-split barrels visible over most of the bench height
- Clean toes
- Well defined and linear bench crests at close to the design crest line
- Effective catch benches at close to the design width.

9.3 Scaling

Effective scaling must remove potential rockfall from bench faces and crests before drilling activates are initiated on the underlying bench. The bench crest should be inspected from the crest level bench to identify potential rockfall that should be removed. The purpose of scaling is to remove any loose rock that could represent a rockfall hazard to operations below the bench, but no more. When pre-split blasting is used, bench faces should be defined by the pre-split line and not by excavation.

The crest of 10 m high production benches is easily within the reach of the RH 340B excavators for scaling. However, these powerful shovels can readily excavate into un-blasted rock, so will have to be operated delicately when used for scaling. It is commonly not cost-efficient to use these expensive machines for extended periods for scaling. Over-excavation is particularly a risk for excavators configured as face shovels because they can easily lift and loosen tight rock in the vicinity of the bench crests; excavators configured as backhoes are more suited to scaling because they do not lift the rock and open fractures at the bench face. The bench crest will be close to the maximum height reachable by the 994F wheel loader, and effective scaling of the crest may be difficult without lifting and damaging the crest.

For good pre-splits, it will likely be preferable to have the large excavators rough scale the bench faces, and then fine scale the benches using smaller and more specialized equipment such as a rock pick on an extended boom, possibly in combination with alternative methods such as chaining the upper slope and crest using a dozer on the crest level bench, or trimming bench crests with a dozer in advance of excavating the shot rock.

9.4 Planning Around Underground Workings

Local modifications to the slope design will be required for safe and stable excavations in areas where stopes intersect the pit wall or floor, or drifts run parallel to the pit wall. Slopes in these areas should be developed with care to ensure the safety of personnel and prevent equipment damage due to collapsing stopes and drifts. Modifications may include excavating the final wall behind stopes and drifts; re-grading overhanging slopes to safe, stable angles; or backfilling the stopes with cemented backfill.

10.0 MONITORING AND FUTURE GEOTECHNICAL WORK

The slope design recommendations provided herein are based on a geotechnical model developed from underground mapping, exploration core drilling, and geotechnical core drilling and testing. The level of confidence in the geotechnical model and the slope design recommendations is therefore good compared to many feasibility-level projects, where there is often not the level of historic data that is available at Canadian Malartic. However, there are still significant uncertainties in the extent to which geological structures will be developed, and the manner in which they will affect slope stability, particularly in the Northeast sector of the pit. The risks associated with these uncertainties should be managed by a program of ongoing geotechnical documentation and monitoring.

The geotechnical model and the implications for slope design should be reviewed if additional relevant information becomes available prior to mining, as through additional drilling completed for development purposes, or geological model revisions. In addition, we consider it essential that a systematic program of pit documentation and slope monitoring should be implemented during mining. These programs should provide information to support the following:

- Development of safe operational systems and procedures;
- Confirmation/revision of the geotechnical model discussed in Section 5;
- Evaluation of controlled blasting effectiveness;
- Optimization of pit slope design through identification of structural controls, and measurement of achieved inter-ramp angles, catch bench widths, and bench face angles;
- Assessing the performance of the slope design and identification of slope displacements.

Elements of this program are described in the following sections.

10.1 Pit Documentation

Phase and final walls should be mapped after they are mined, but before access is affected by subsequent mining. The complete mapping format will depend in part on geological requirements, but for engineering purposes, it should include:

- Location, orientation, and description (infilling type and thickness, continuity) of all faults;
- Location and orientation of joints that are continuous on a bench scale, or with orientations that affect achieved bench face angles;
- Rock types and alteration;
- Achieved bench configuration, including overall bench face angle, bench width, and factors that control bench performance;
- Occurrence of perched groundwater (location, controls, flow rate and duration);
- Slope failures (limits, crack mapping, timing, contributing factors, photographs).
- Locations of underground workings

Mapping should be completed bench-by-bench, in an electronic format (AutoCAD, GIS-based, etc.). However, it should ultimately be compiled into a “master” map of each phase pit. As part of the compilation of a “master” map from maps of individual benches, discrete continuous structures will require correlation bench-to-bench.

In addition to general confirmation/revision of the geotechnical model, some specific aspects of the model should be given particular attention in the field because of their effect on slope designs, and potential impacts on stability. These include:

- Evaluate the occurrence, orientation, continuity, and effect on slope performance of south-dipping structures in the Northeast sector;
- Given the apparent flattening of the porphyry contact along the east side of the pit, track the orientation of this contact, and evaluate potential effects on slope stability;
- Evaluate the occurrence and effect on orientation of structure that affects slope performance throughout the pit. Our pit slope design recommendations are based on data that indicate that potential structural control of slope angles is limited to the Northeast sector. Verification of this characterization will be required in order to optimize design bench configurations, as well as design sector boundaries.
- Track the change in orientation of bedding related to the drag anticline in the southwest area of the deposit and evaluate potential effects on slope stability
- Verify the assumed high rock mass strength;
- Verify the assumed good geotechnical characteristics of the schists (metamorphosed mafic dikes).

10.2 Monitoring

A comprehensive slope monitoring program that includes surface displacement monitoring, subsurface displacement monitoring, and visual inspection is an essential part of pit slope engineering. Some of the objectives of this program are generic in that they apply to many open pit operations, but others are specific to pit development at Canadian Malartic. The program should provide the following:

- A basis for maintaining safe operational systems and procedures;
- General coverage of the pit in order to provide “background” data, and early indications of slope displacements;
- Focused coverage along the north wall, where the pit is adjacent to a residential area;
- Focused coverage to delineate, track, and manage displacing slopes;
- Input to short- and long-term planning with respect to areas of potentially unstable ground.
- Support for investigation of unstable slopes.

10.2.1 Visual Inspection

Regular visual inspection of working areas is an important responsibility of all involved in a mining operation. The occurrence of rockfalls, crack observations, etc. should be reported to pit supervisors and engineering staff for evaluation. In addition, visual reconnaissance of pit crests, active benches, and overall walls should be conducted on a regular basis by engineering staff. Observations from these inspections should be compiled in a consistent format to establish a formal record of pit slope performance with time during pit development.

Visual observations should be augmented with photographs taken during inspections. In addition to photographs of specific slope areas of interest during a given inspection, photographs should also be taken from fixed stations around the pit. These photographs will also contribute to the record of pit slope performance with time.

10.2.2 Surface Displacement Monitoring

A method of monitoring for surface displacements throughout the pit, and behind the crest of the north wall, will be required. Alternatives for this type of monitoring include:

- Geodetic surveying methods (i.e., prism monitored by precise theodolite/EDM or total station);
- GPS monitoring;
- Laser scanning;
- Radar scanning.

All of these are functional systems that have been demonstrated to be effective at open pit mines. However, there are distinctions between the systems in terms of accuracy and precision, cost, data processing, hardware, and operation. For most operations, a widespread prism array, augmented by more closely-spaced prisms in critical areas, monitored with an electronic total station provides adequate surface monitoring data. However, as the number of prisms increases with continued mining, or circumstances for frequent monitoring of certain prisms develop, the manpower requirements for data acquisition increase. Robotic total stations can be used in such cases as a means of automating the data collection and compilation. As an example, given the proximity of residential development to the north wall, and the potential for occurrence of adverse structure along the east half of this wall, it may be preferable to provide for “continuous” monitoring of the north wall prism array.

GPS monitoring also involves an array of monitoring discrete monitoring points, and in this respect it is similar to the prism monitoring described above. However, this type of monitoring requires that stations are accessible for monitoring by surveyors with a portable GPS unit; or that a dedicated GPS unit be installed at each monitoring station. The cost of dedicated units, and the access-related risk of monitoring fixed stations with portable GPS units, typically makes conventional geodetic surveying a better alternative than GPS monitoring for coverage of a pit area.

Laser and radar scanning can be used to generate more complete and detailed coverage of a slope, often in a shorter amount of time than monitoring an array of prisms or GPS points. They are also more costly, and more specialized in terms of data acquisition and interpretation. The cost and complexity notwithstanding, there are circumstances in which advantages of these systems make them clearly superior to other monitoring methods. The proximity of the north wall to residential development may justify the use of these technologies, although prism arrays and geodetic surveying have been used effectively to monitor pit wall stability in similar circumstances at other open pit operations.

The methods outline above should constitute a “primary” method of monitoring surface displacements. In addition, “secondary” methods, such as crack measuring pins and wireline extensometers, should be used as required to monitor displacement across discrete cracks.

10.2.3 Subsurface Displacement Monitoring

Subsurface displacement monitoring should be used to augment the surface monitoring data along the north wall. Available methods of subsurface displacement monitoring include inclinometers, time domain reflectometry (TDR) cables, and various types of extensometers. All require drillholes for installation.

The value of these installations is that they can be used to establish whether displacement is occurring at depth, and if this is the case, the combination of surface and subsurface monitoring is needed to develop an understanding of the failure mode. They are also more sensitive than surface survey systems, and so are commonly used to monitor critical or sensitive structures.

10.2.4 Water Level Monitoring

While Osisko plans to dewater the pit slopes during mining, water levels throughout the pit should be monitored to verify de-pressurization. In the Northeast sector, where stability is particularly sensitive to groundwater pressures, piezometers should be installed and read regularly.

10.3 Blasting-Related Monitoring

Blast vibration monitoring will be required both to limit vibration levels in the adjacent residential area and to keep blast vibration levels and frequencies within a range that will minimize wall damage. With regard to the adjacent residential area, a pre-mining survey to document the conditions of structures may be appropriate. Also prior to mining, targets for allowable vibration levels at structures should be established, and these should be used as design criteria for initial blasts. As mining proceeds, blast designs should be reviewed consistently in light of measured vibration levels vs. allowable levels; achieved slope conditions; and production considerations.

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TABLES

TABLE 1
ISRM STRENGTH CLASSIFICATION

ISRM Strength	Description	Field Identification	UCS Range (MPa)	
			Min	Max
R0	Extremely Weak Rock	Indented with thumbnail	0.25	1.0
R1	Very Weak Rock	Crumbles under firm blows with point of geological hammer, can be peeled by a pocket knife	1.0	5.0
R2	Weak Rock	Can be peeled by a pocket knife with difficulty, shallow indentations made by firm blow with point of geological hammer	5.0	25
R3	Medium Strong Rock	Cannot be scraped or peeled with a pocket knife, specimen can be fractured with single firm blow of geological hammer	25	50
R4	Strong Rock	Specimen requires more than one blow of geological hammer to fracture it	50	100
R5	Very Strong Rock	Specimen required many blows of geological hammer to fracture it	100	250
R6	Extremely Strong Rock	Specimen can only be chipped with geological hammer	>250	

TABLE 2
SUMMARY OF ROCK MASS RATING SYSTEM (BIENIAWSKI, 1976)

Rock Mass Rating							
Parameter	**Range of Values**						
UCS **Rating**	>200Mpa **(15)**	100-200 MPa (12)	50-100 Mpa **(7)**	25-50Mpa **(4)**	10-25 **(2)**	<3-10 **(1)**	1-3 **(0)**
RQD **Rating**	90-100% **(20)**	75-90% **(17)**	50-75% **(13)**	25-50% **(8)**	<25% **(3)**		
Joint Spacing **Rating**	>3m **(30)**	1-3m **(25)**	0.3-1m **(20)**	50-300mm **(10)**	<50mm **(5)**		
Joint Condition **Rating**	Very rough No separation Hard wall rock **(25)**	Slightly rough Separation<1mm Hard wall rock **(20)**	Slightly rough Separation<1mm Soft wall rock **(12)**	Slickensides, Separation or gouge <5mm **(6)**	Soft gouge or Separation >5mm **(0)**		
Groundwater **Rating**	Completely Dry **(10)**		Moist **(7)**	Mod. Pressure **(4)**	Severe **(0)**		
Total RMR Value (Sum of Ratings for 5 Items) =							
Rating	100 – 81	80 – 61	60 – 41	40 - 21	20 - 0		
Description	I – Very Good	II – Good	III – Fair	IV - Poor	V - Very Poor		

Rating values are shown in parentheses.

TABLE 3
SUMMARY OF TRIAXIAL TESTING RESULTS

Lithology	Drillhole ID	From (m)	To (m)	σ_3 (MPa)	σ_1 (MPa)
AGR	GT07-03	221.40	222.05	2	98.3[1]
AGR	GT07-03	221.40	222.05	5	154.0[1]
AGR	GT07-03	221.40	222.05	10	227.1
AGR	GT07-02	160.71	161.32	14	444.5[2]
AGR	GT07-02	213.50	214.28	14	196.1[1]
AGR	GT07-02	160.71	161.32	25	470.7
AGR	GT07-02	213.50	214.28	25	296.5[1]
AGR	GT07-02	160.71	161.32	35	422.6
AGR	GT07-02	213.50	214.28	35	276.2[1]
APO	GT07-02	115.07	115.79	14	396.5
APO	GT07-02	115.07	115.79	14	395.8
APO	GT07-02	115.07	115.79	25	512.0
APO	GT07-02	115.07	115.79	25	468.6
APO	GT07-02	115.07	115.79	35	526.0
CGR	GT07-01	135.11	135.80	14	216.1
CGR	GT07-05	363.34	363.82	14	249.6
CGR	GT07-01	135.11	135.80	25	261.4[2]
CGR	GT07-05	363.34	363.82	25	271.0[2]
CGR	GT07-01	135.11	135.80	35	280.5
CGR	GT07-05	363.34	363.82	35	202.5[2]
CPO	GT07-02	106.86	107.27	14	318.1
CPO	GT07-02	184.78	185.54	14	202.7
CPO	GT07-02	106.86	107.27	25	503.7[2]
CPO	GT07-02	184.78	185.54	25	416.9[2]
CPO	GT07-02	106.86	107.27	35	544.2
CPO	GT07-02	184.78	185.54	35	310.9[1]
REMGR	GT07-05	43.61	44.37	14	224.9[2]
REMGR	GT07-05	43.61	44.37	14	285.5
REMGR	GT07-05	43.61	44.37	25	573.3
REMGR	GT07-05	43.61	44.37	25	348.1[1]
REMGR	GT07-05	43.61	44.37	35	479.8[2]
SCH	GT07-04	216.71	217.13	14	113.3[1]
SCH	GT07-04	216.71	217.13	25	159.4[1]
SCH	GT07-04	216.71	217.13	35	174.4[1]
SGR	GT07-01	94.60	94.97	14	375.1
SGR	GT07-01	95.11	95.35	14	443.2
SGR	GT07-01	94.60	94.97	25	394.3[2]
SGR	GT07-01	94.60	94.97	35	478.6[1]
SPO	GT07-02	91.21	92.08	14	482.8
SPO	GT07-02	91.21	92.08	14	382.1
SPO	GT07-02	91.21	92.08	25	423.1[2]
SPO	GT07-02	91.21	92.08	25	576.2[2]
SPO	GT07-02	91.21	92.08	35	393.2
SPO	GT07-02	91.21	92.08	35	470.1[2]

[1]Sample failed along pre-existing fracture
[2]Sample failed partially along pre-existing fracture

TABLE 4
ROCK QUALITY DESIGNATION DESCRIPTION

Description of Rock Quality	RQD (%)
Very Poor	0 – 25
Poor	25 – 50
Fair	50 – 75
Good	75 – 90
Excellent	90 – 100

TABLE 5
VALUES OF THE CONSTANT m_i FOR INTACT ROCK, BY ROCK GROUP

Rock Type	Class	Group	Texture			
			Coarse	**Medium**	**Fine**	**Very fine**
SEDIMENTARY	Clastic		*Conglomerates* (21 ± 3) *Breccias* (19 ± 5)	*Sandstones* 17 ± 4	*Siltstones* 7 ± 2 *Greywackes* (18 ± 3)	*Claystones* 4 ± 2 *Shales* (6 ± 2) *Marls* (7 ± 2)
SEDIMENTARY	Non-clastic	Carbonates	*Crystalline Limestone* (12 ± 3)	*Sparitic Limestones* (10 ± 2)	*Micritic Limestone* (9 ± 2)	*Dolomites* (9±3)
SEDIMENTARY	Non-clastic	Evaporites		*Gypsum* 8 ± 2	*Anhydrite* 12 ± 2	
SEDIMENTARY	Non-clastic	Organic				*Chalk* 7 ± 2
METAMORPHIC	Non-foliated		*Marble* 9 ± 3	*Hornfels* (19 ± 4) *Metasandstone* (19 ± 3)	*Quartzites* (20 ± 3)	
METAMORPHIC	Slightly foliated		*Migmatite* (29 ± 3)	*Amphibolites* 26 ± 6		
METAMORPHIC	Foliated*		*Gneiss* 28 ± 5	*Schists* 12 ± 3	*Phyllites* (7 ± 3)	*Slates* 7 ± 4
IGNEOUS	Plutonic	Light	*Granite* 32 ± 3 *Granodiorite* (29 ± 3)	*Diorite* 25 ± 5		
IGNEOUS	Plutonic	Dark	*Gabbro* 27 ± 3 *Norite* 20 ± 5	*Dolerite* (16 ± 5)		
IGNEOUS	Hypabyssal		*Porphyries* (20 ± 5)		*Diabase* (15 ± 5)	*Peridotite* (25 ± 5)
IGNEOUS	Volcanic	Lava		*Rhyolite* (25 ± 5) *Andesite* 25 ± 5	*Dacite* (25 ± 3) *Basalt* (25 ± 5)	*Obsidian* (19 ± 3)
IGNEOUS	Volcanic	Pyroclastic	*Agglomerate* (19 ± 3)	*Breccia* (19 ± 5)	*Tuff* (13 ± 5)	

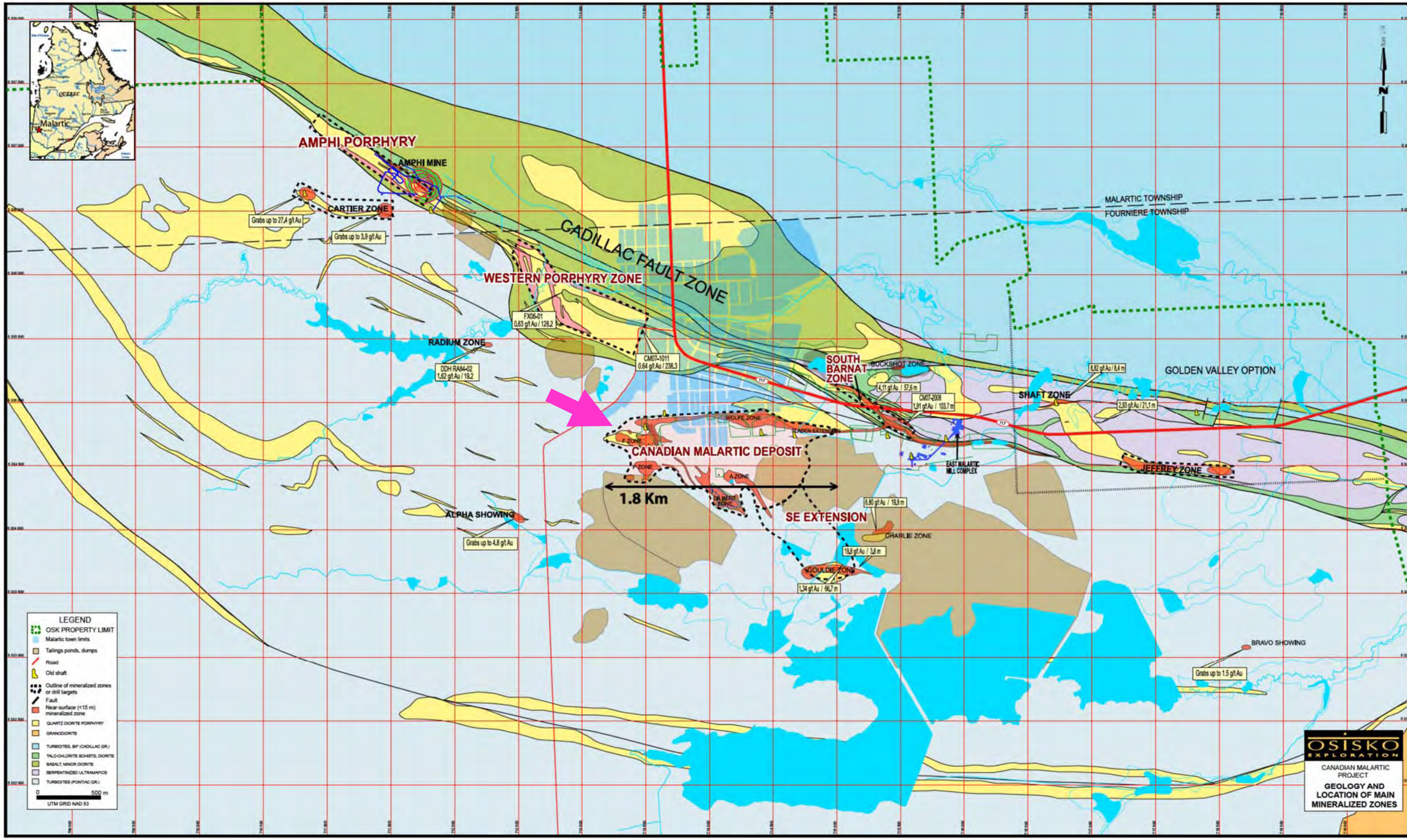
Values in parenthesis are estimates

*Values are for intact rock specimens tested normal to bedding or foliation. Value of m_i will be significantly different if failure occurs along weakness plane.

Source: Hoek, E., and Karzulovic, A., 2000. Rock Mass Properties for Surface Mines, *in* Slope Stability in Surface Mining: Society for Mining, Metallurgy, and Exploration.

FIGURES

FIGURES



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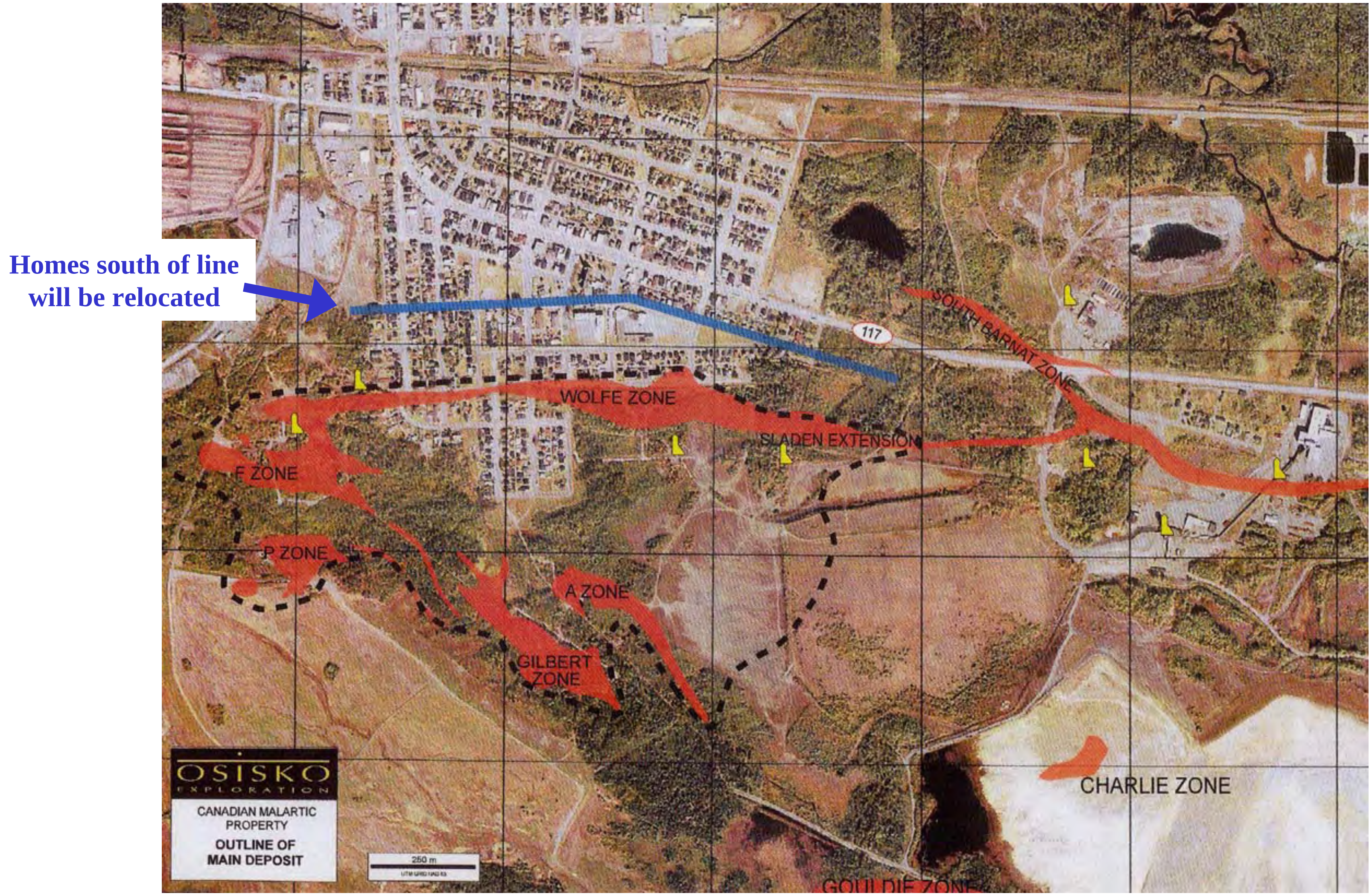
TITLE	LOCATION MAP		
CLIENT/PROJECT: OSISKO — CANADIAN MALARTIC PROJECT	DRAWN: RLK	DATE: 07/06/08	JOB NO. 07-1221-0028 2000 2300
	CHECKED: GM	SCALE	DWG. NO./REV. NO.
	REVIEWED	FILE NO.	FIGURE 1



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TITLE

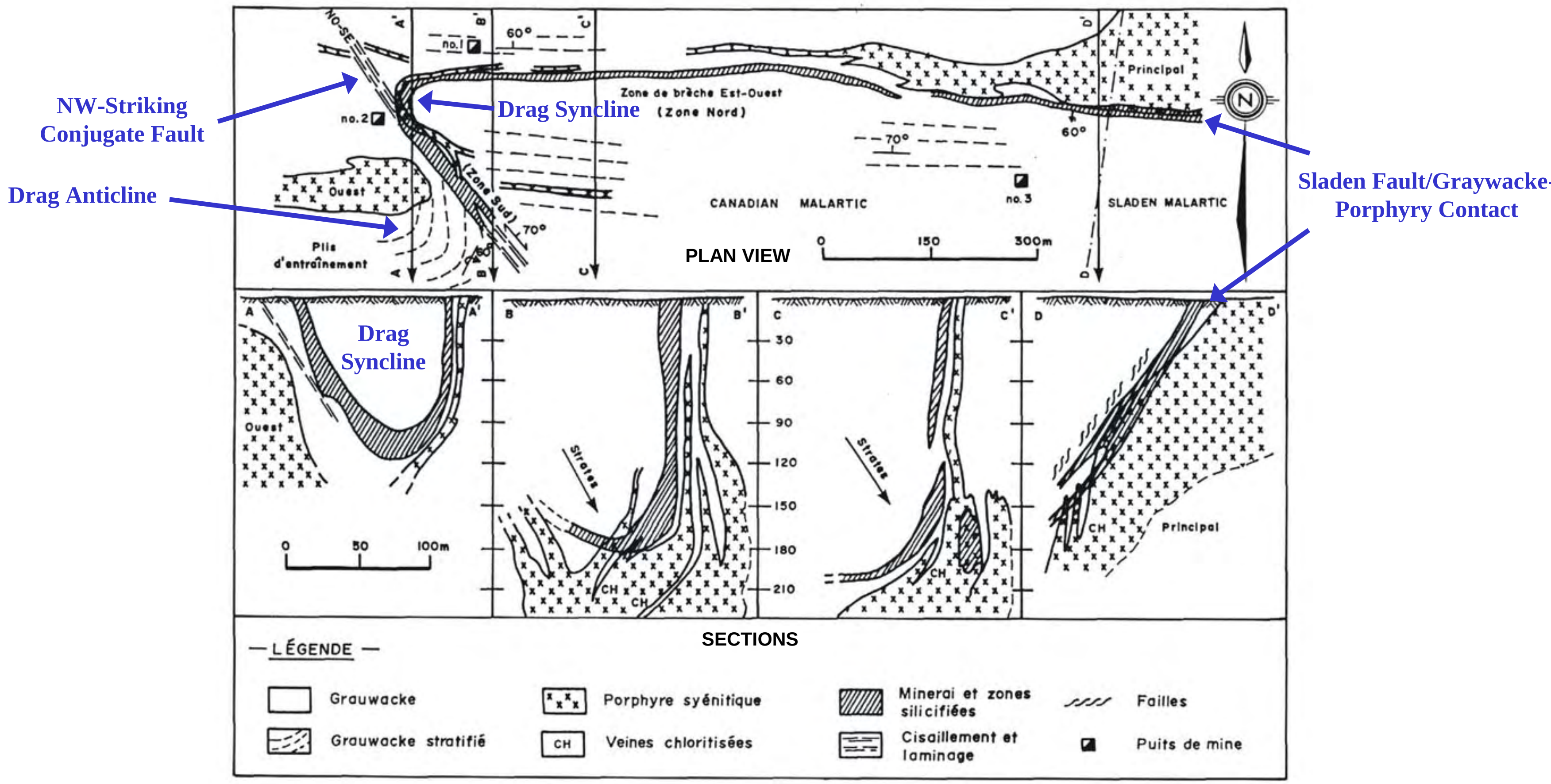
OUTLINE OF MAIN DEPOSIT

CLIENT/PROJECT

OSISKO

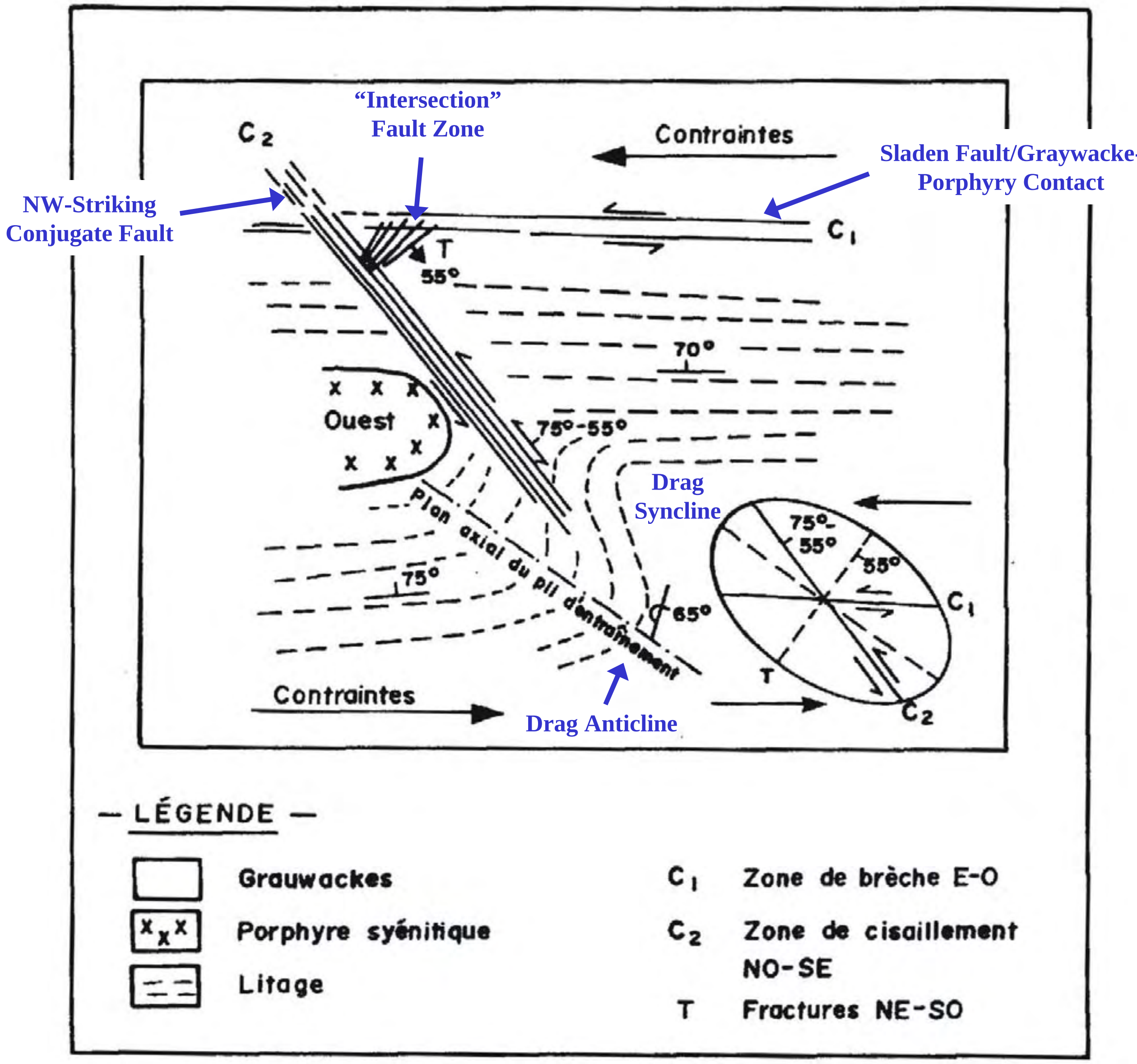
CANADIAN MALARTIC PROJECT

DRAWN	RLK	DATE	11/16/07	JOB NO.	07-1221-0028
CHECKED	GM	SCALE		DWG. NO./REV. NO.	
REVIEWED		FILE NO.			FIGURE 2



From: (Sansfacon and others, 1987a;Derry, 1939)

Golder Associates	6165 RIDGEVIEW CT, SUITE G RENO, NEVADA 89519 PHONE: (775) 828-9604 FAX: (775) 828-9645	TITLE **GENERAL GEOLOGY OF THE CANADIAN MALARTIC MINE AREA**	
CLIENT/PROJECT OSISKO	**CANADIAN MALARTIC PROJECT**	DRAWN RLK CHECKED GM REVIEWED	DATE 06/28/08 SCALE FILE NO.
		JOB NO. 07-1221-0028 2000 2300	DWG. NO./REV. NO. **FIGURE 3**



From: (Sansfacon and others, 1987a;Derry, 1939)

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TITLE

MAJOR STRUCTURES AT THE CANADIAN MALARTIC MINE

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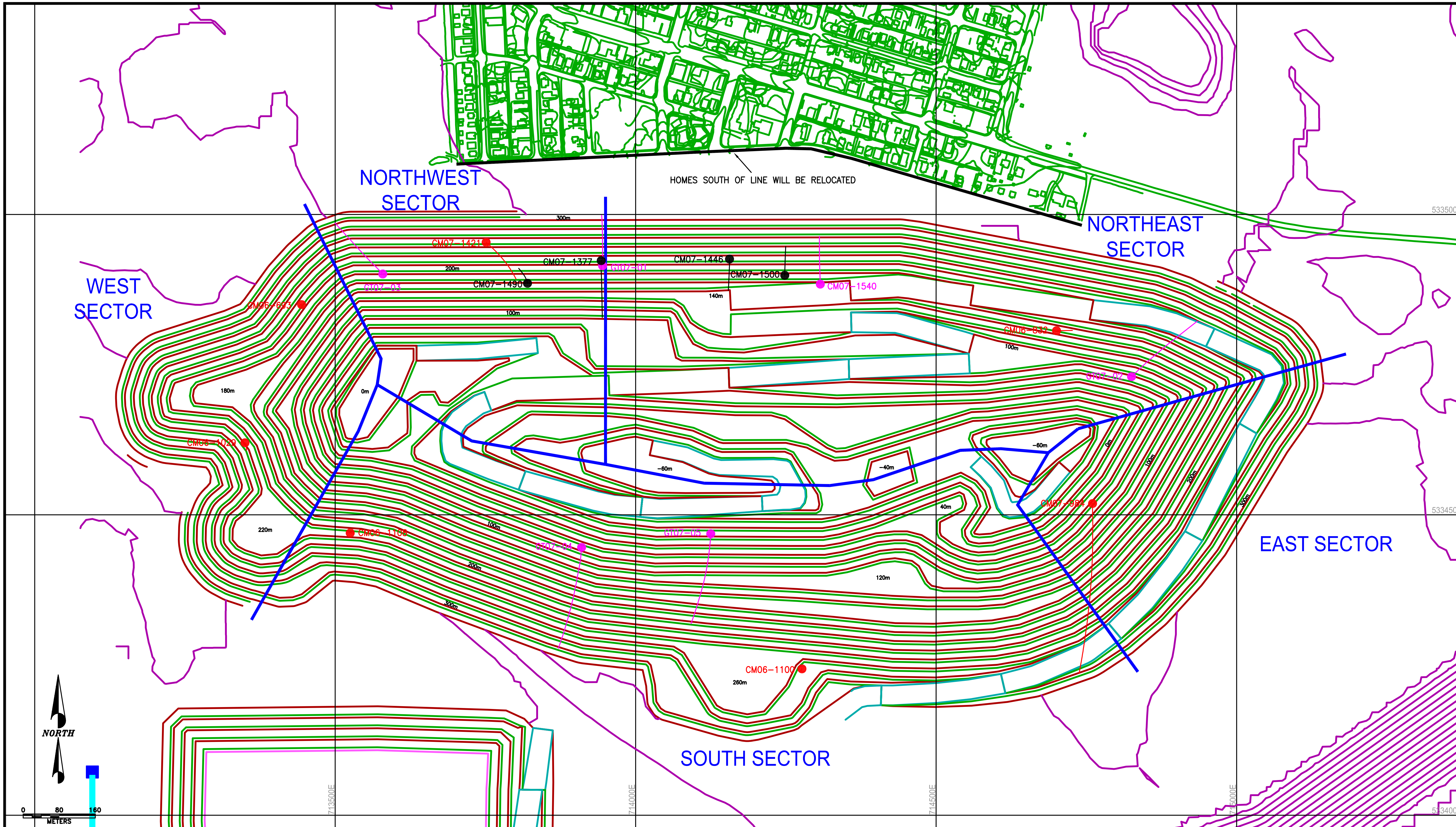
		CHECKED GM	SCALE NA	DWG. NO. / REV. NO.	
		REVIEWED GM	FILE NO.	FIGURE 4	



Explanation

GT07-02 Geotechnical Corehole

CM07-1500 Golder Televiewer Corehole

CM06-832 Terratec Televiewer Drillhole

Sector Boundary

ULTIMATE PIT PLAN SHOWING DRILLHOLE AND SECTOR LOCATIONS

FIGURE 5

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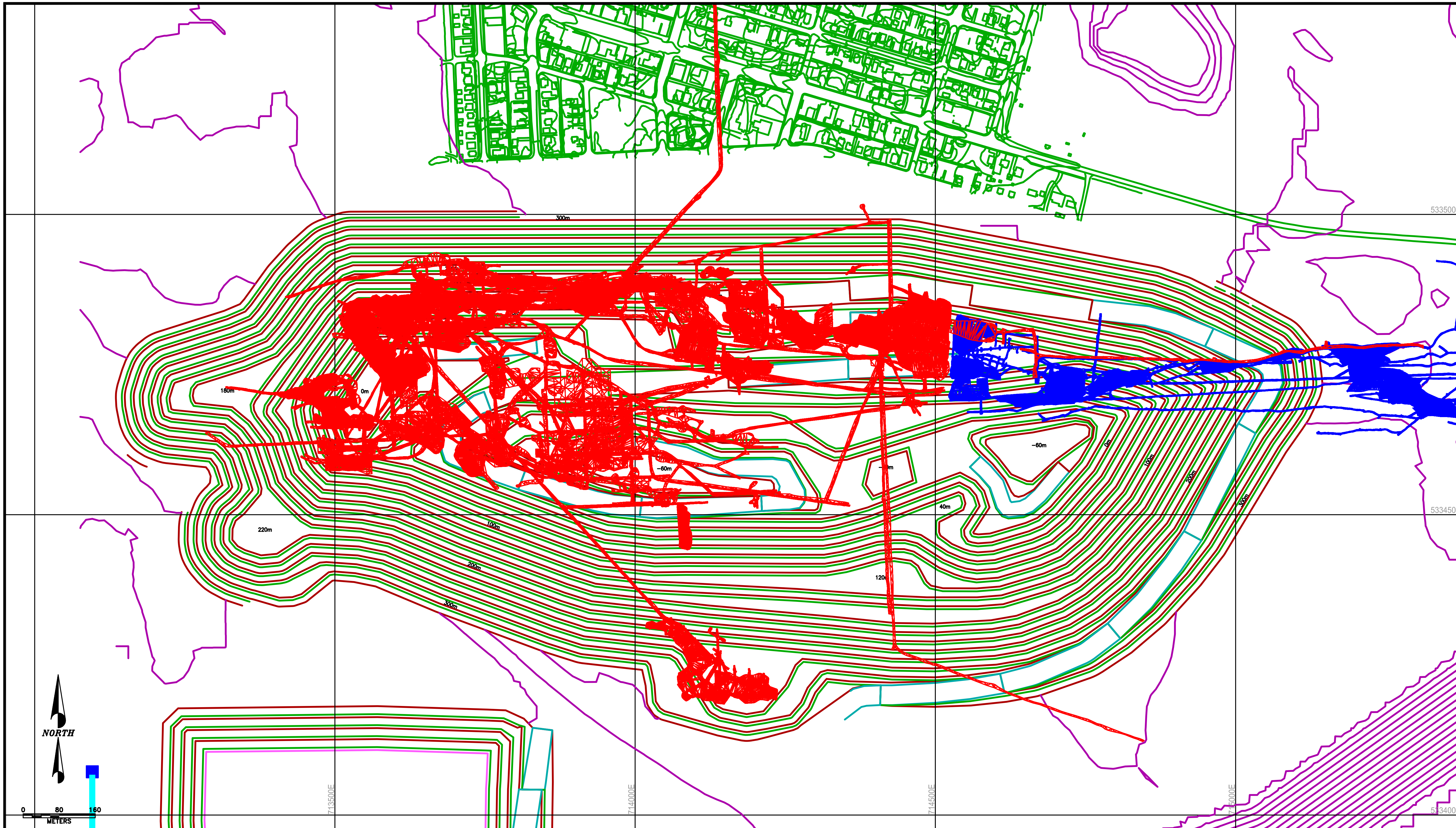
RENO, NEVADA

PROJECT No. 07-1221-0028 DATE 06/30/08



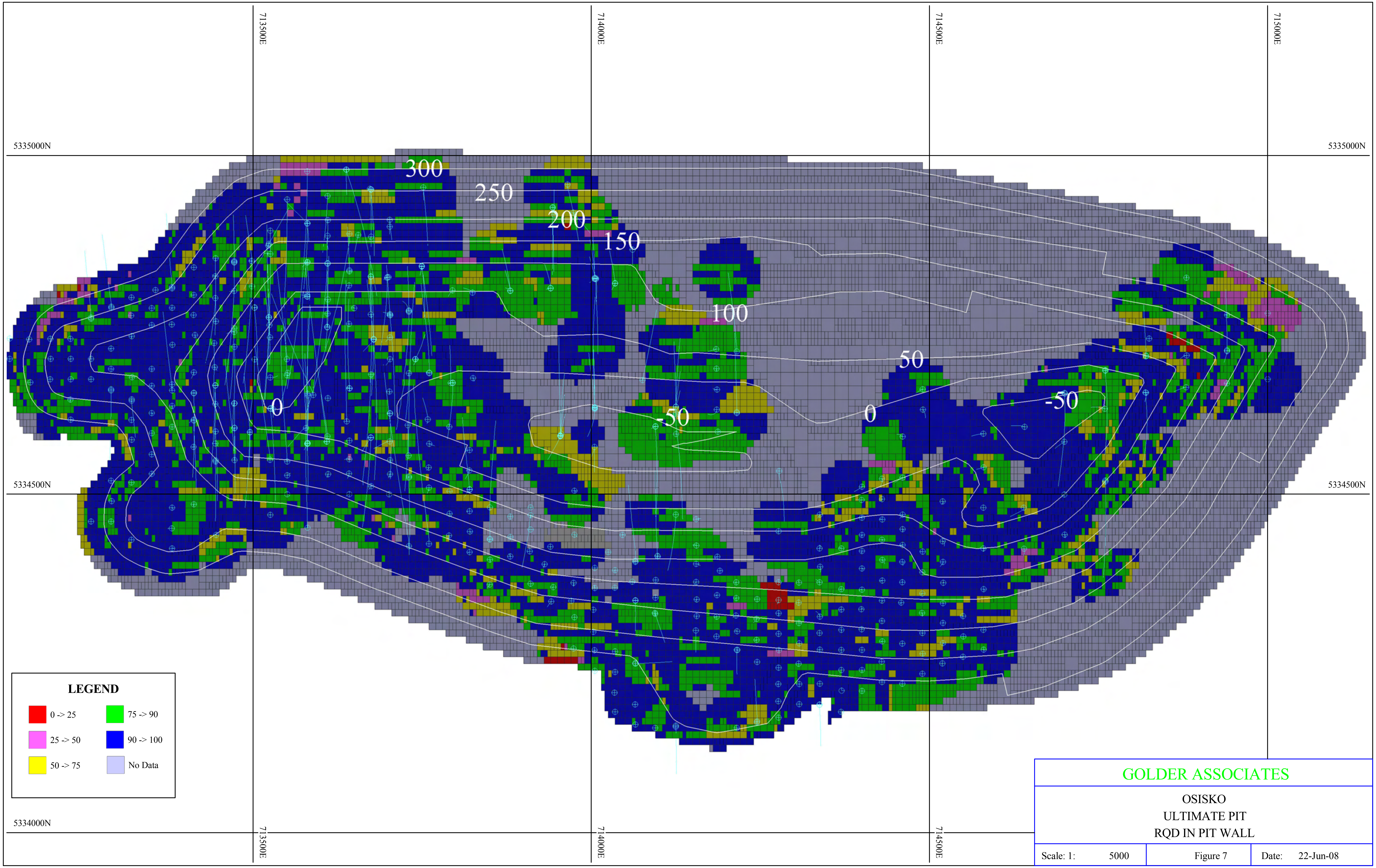
Canadian Malartic Mine shown in red, Sladen Mine shown in blue.

Golder Associates
RENO, NEVADA

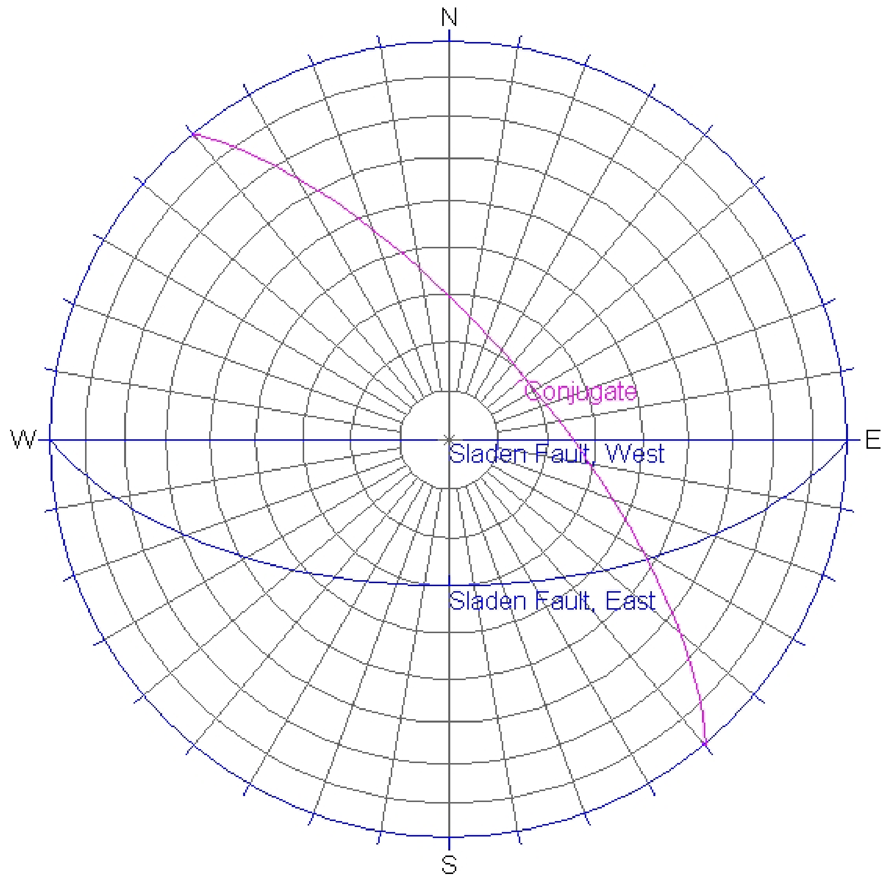
PROJECT No. 07-1221-0028 DATE 06/30/08

ULTIMATE PIT PLAN AND UNDERGROUND WORKINGS

FIGURE 6



713500E
714000E
714500E
715000E
5335000N
5334500N
5334000N
300
250
200
150
100
50
0
-50
0
-50
LEGEND
0 -> 25
25 -> 50
50 -> 75
75 -> 90
90 -> 100
No Data
GOLDER ASSOCIATES
OSISKO
ULTIMATE PIT
RQD IN PIT WALL
Scale: 1: 5000
Figure 7
Date: 22-Jun-08



Structure	Dip/Dip Direction
Sladen Fault/Greywacke-Porphyry Contact, West of 712950E	90/180
Sladen Fault/Greywacke-Porphyry Contact, East of 712950E	60/180
NW-Striking Conjugate Fault	70/050

Golder Associates

TITLE

DESIGN STEREONET, MAJOR STRUCTURES

CLIENT/PROJECT

OSISKO

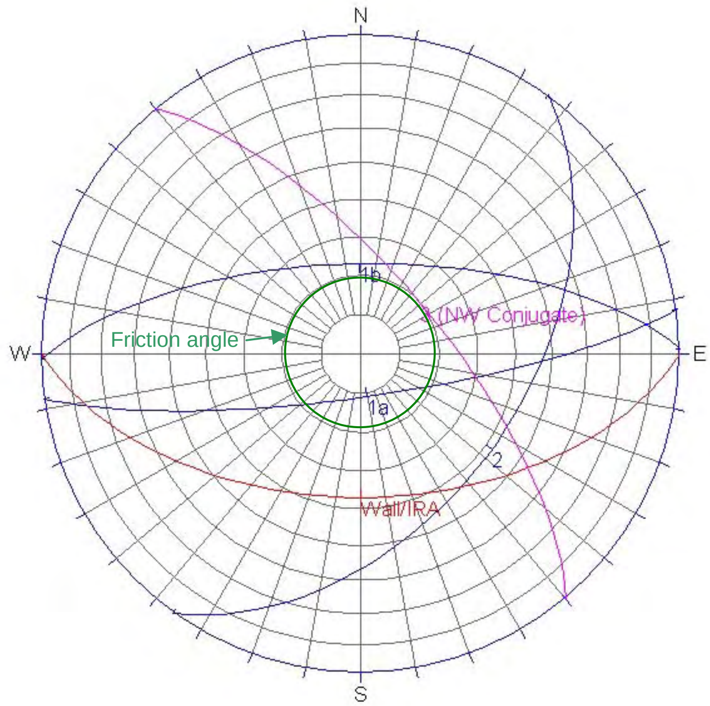
CANADIAN MALARTIC PROJECT

DRAWN	RK	DATE	04/20/08	JOB NO.	07-1221-0028
CHECKED	GM	SCALE	NA	DWG. NO. / REV. NO.	
REVIEWED	GM	FILE NO.			FIGURE 8

Northwest Sector

Set	Dip/Dip Direction
1a	79/172
1b	67/359
2	48/126
3	70/050

Pit Wall Dip Direction: 180

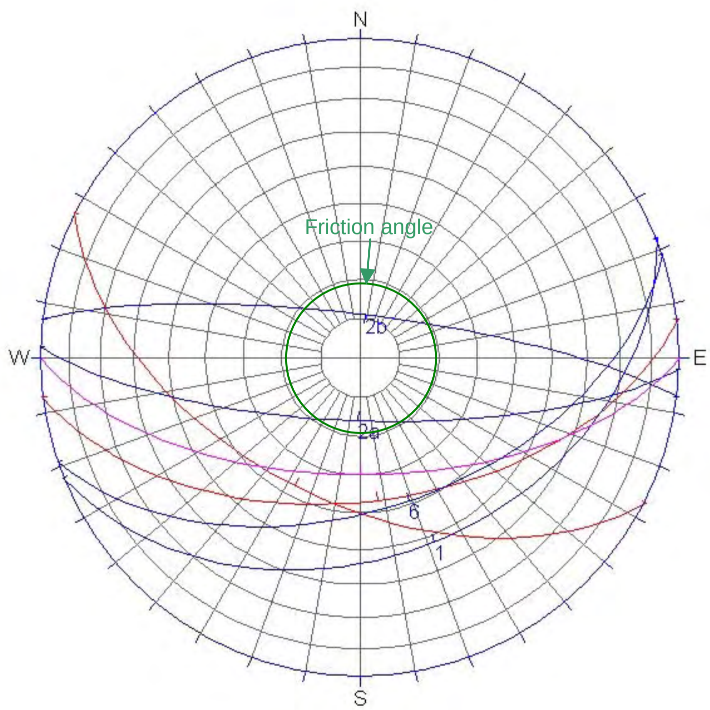


Northeast Sector

Set	Dip/Dip Direction
1	38/158
2a	77/188
2b	81/009
6	51/161

Structure	Dip/Dip Direction
Sladen Fault/Greywacke-Porphyry Contact	60/180

Pit Wall Dip Direction: 180-210



TITLE

DESIGN STEREONETS, NORTHWEST AND NORTHEAST SECTORS

Golder Associates

CLIENT/PROJECT

OSISKO

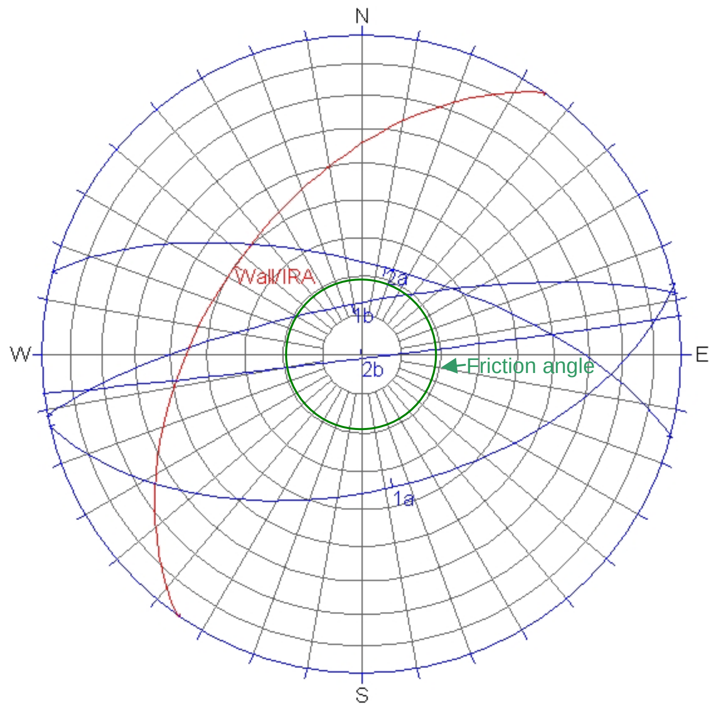
CANADIAN MALARTIC PROJECT

DRAWN	RK	DATE	04/20/08	JOB NO.	07-1221-0028
CHECKED	GM	SCALE	NA	DWG. NO. / REV. NO.	
REVIEWED	GM	FILE NO.			FIGURE 9

East Sector

Set	Dip/Dip Direction
1a	55/167
1b	77/349
2a	67/015
2b	89/173

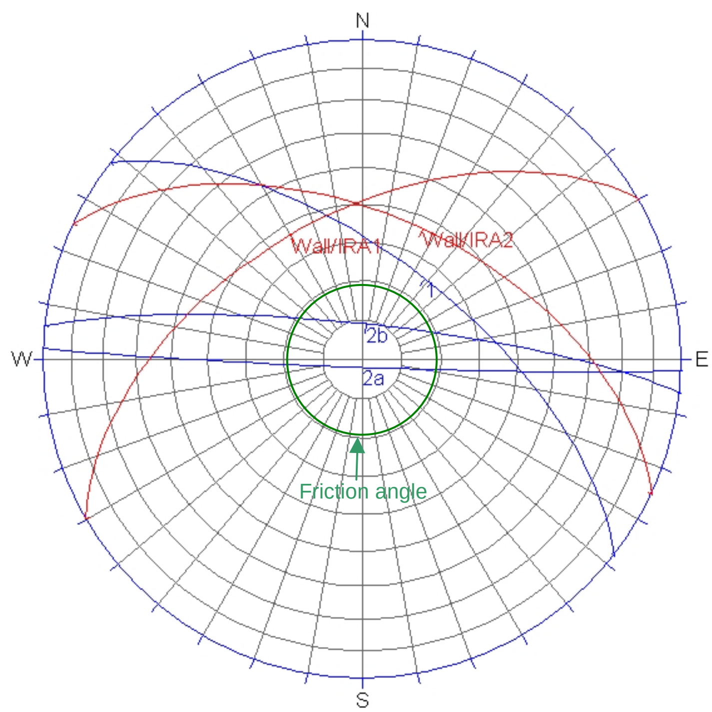
Pit Wall Dip Direction: 300-310



South Sector

Set	Dip/Dip Direction
1	64/038
2a	88/182
2b	81/006

Pit Wall Dip Direction: 330-025



Golder Associates

TITLE

DESIGN STEREONETS, EAST AND SOUTH SECTORS

CLIENT/PROJECT

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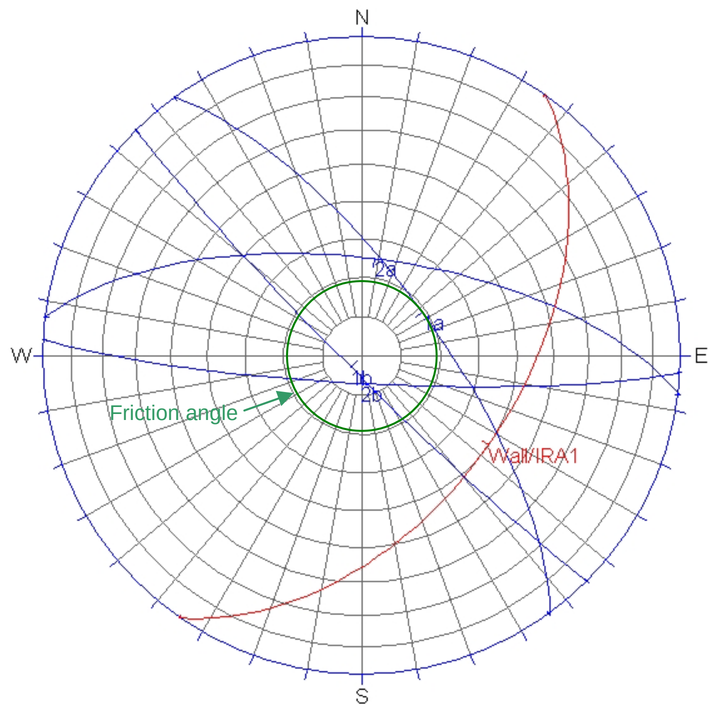
CANADIAN MALARTIC PROJECT

DRAWN	RK	DATE	04/20/08	JOB NO.	07-1221-0028
CHECKED	GM	SCALE	NA	DWG. NO. / REV. NO.	
REVIEWED	GM	FILE NO.			FIGURE 10

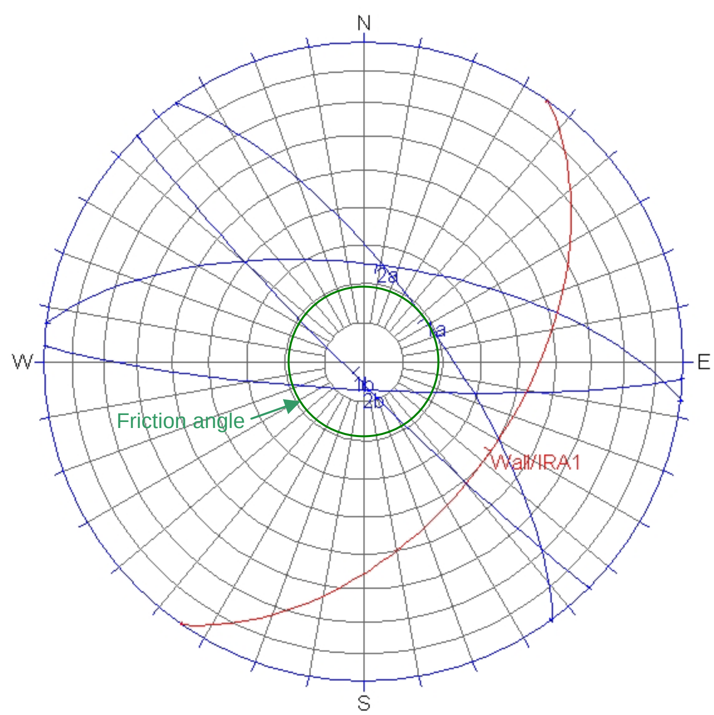
Pit Wall Dip Direction: 125



West Sector

Set	Dip/Dip Direction
1a	71/054
1b	86/225
2a	65/007
2b	83/183

Pit Wall Dip Direction: 070

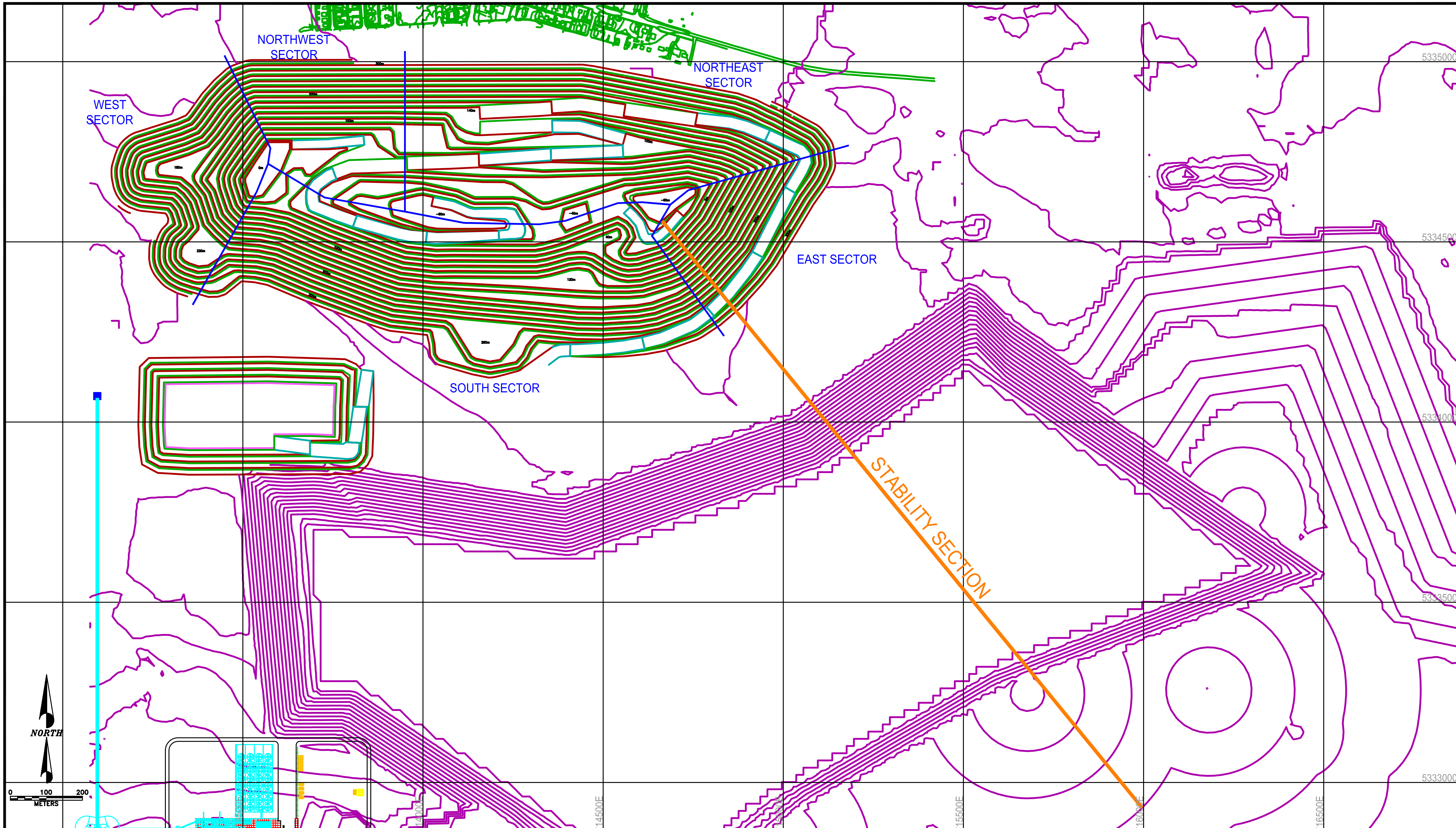


Golder Associates

TITLE

DESIGN STEREONET, WEST SECTOR

CLIENT/PROJECT					
OSISKO	**CANADIAN MALARTIC PROJECT**				
DRAWN	RK	DATE	04/20/08	JOB NO.	07-1221-0028
CHECKED	GM	SCALE	NA	DWG. NO. / REV. NO.	
REVIEWED	GM	FILE NO.			FIGURE 11



Sector Boundary

Golder Associates
RENO, NEVADA

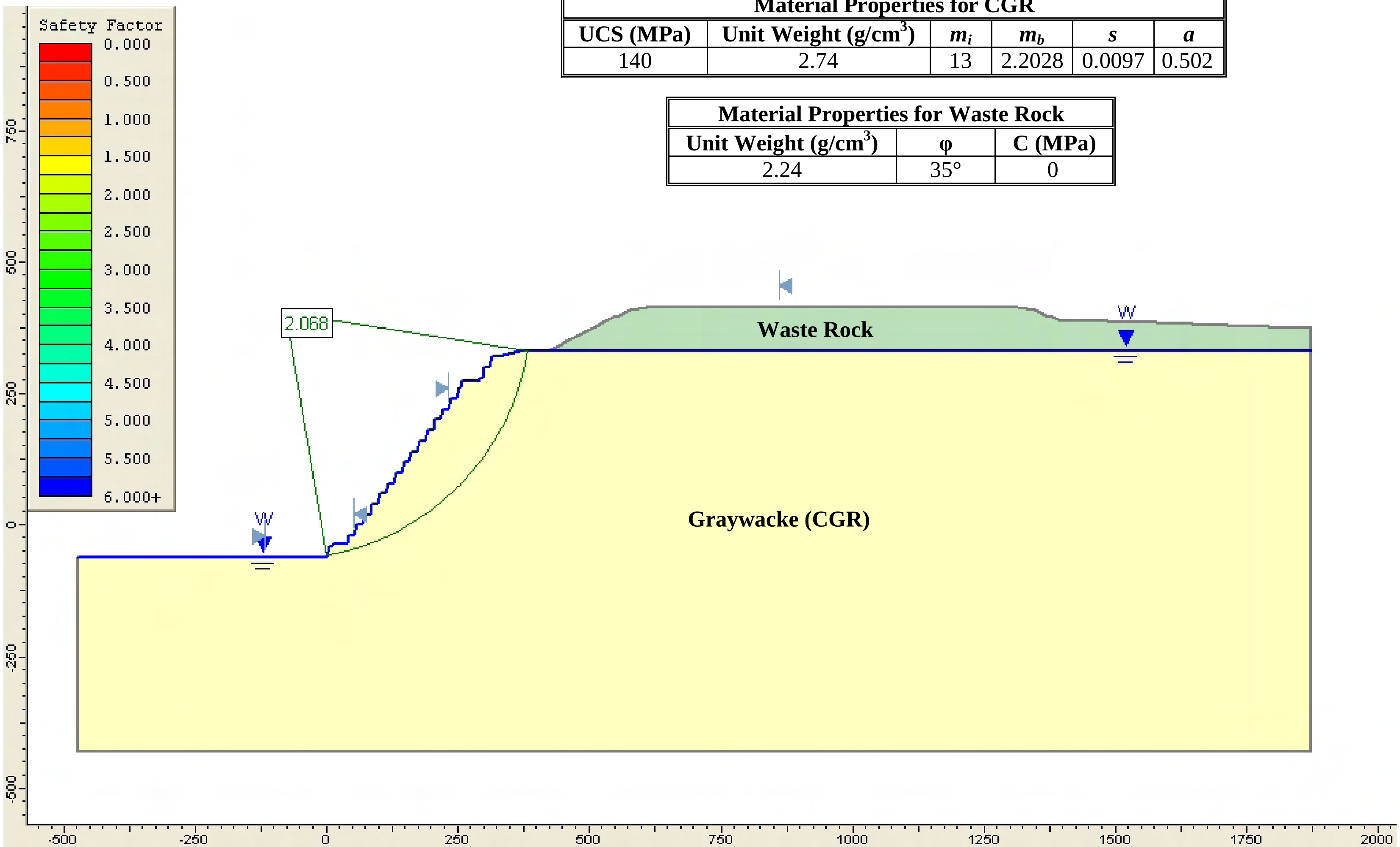
PROJECT No. 07-1221-0028 DATE 06/30/08

ULTIMATE PIT PLAN AND WASTE DUMP

FIGURE 12

Material Properties for CGR					
UCS (MPa)	Unit Weight (g/cm^3)	m_i	m_b	s	a
140	2.74	13	2.2028	0.0097	0.502

Material Properties for Waste Rock		
Unit Weight (g/cm^3)	φ	C (MPa)
2.24	35°	0



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FAX: (775) 828-9645

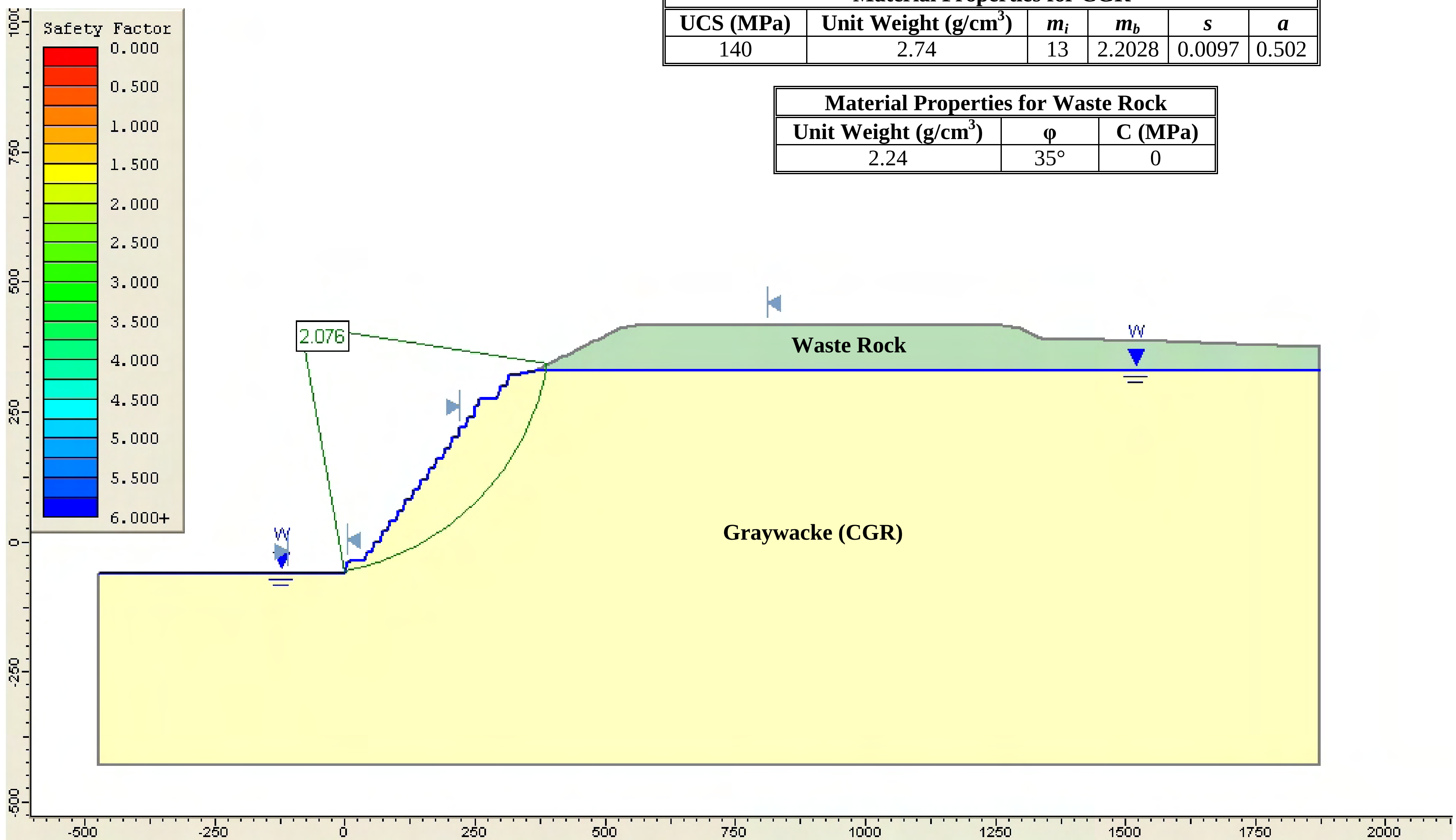
TITLE
Limit Equilibrium Analaysis, Waste Dump
100 m from Pit Crest

CLIENT/PROJECT
OSISKO
CANADIAN MALARTIC PROJECT

DRAWN	RLK	DATE	06/28/08	JOB NO.	07-1221-0028 2000 2300
CHECKED	GM	SCALE		DWG. NO./REV. NO.	
REVIEWED		FILE NO.			FIGURE 13

Material Properties for CGR					
UCS (MPa)	Unit Weight (g/cm^3)	m_i	m_b	s	a
140	2.74	13	2.2028	0.0097	0.502

Material Properties for Waste Rock		
Unit Weight (g/cm^3)	φ	C (MPa)
2.24	35°	0



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TITLE

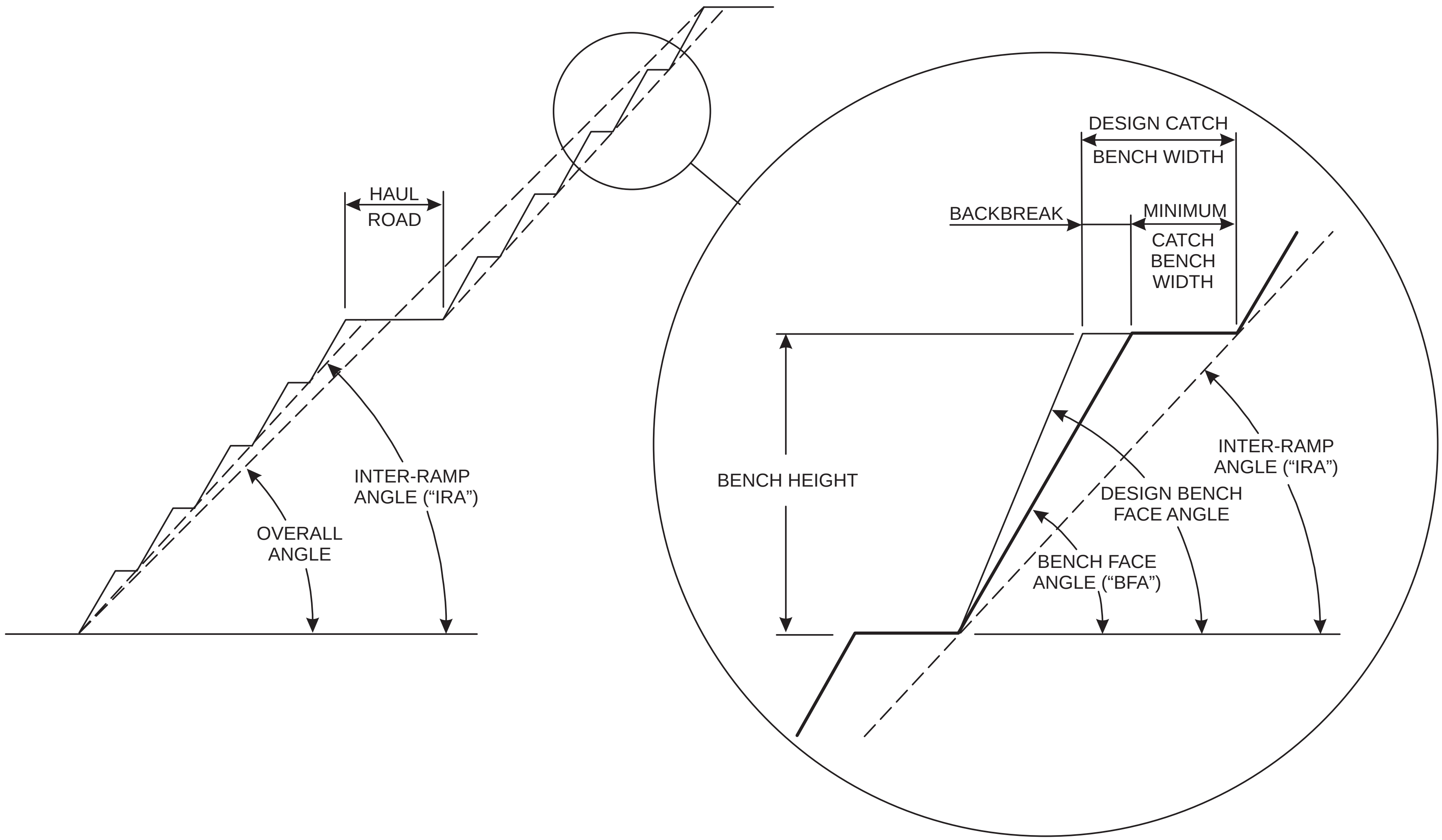
Limit Equilibrium Analaysis, Waste Dump
50 m from Pit Crest

CLIENT/PROJECT

OSISKO

CANADIAN MALARTIC PROJECT

DRAWN	RLK	DATE	06/28/08	JOB NO.	07-1221-0028 2000 2300
CHECKED	GM	SCALE		DWG. NO./REV. NO.	
REVIEWED		FILE NO.			FIGURE 14



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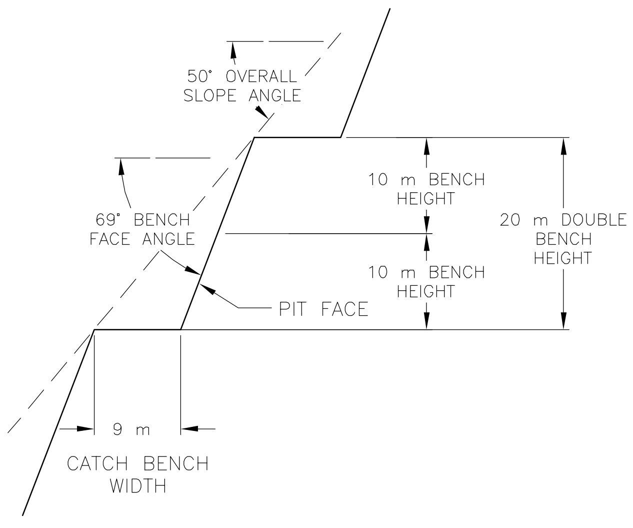
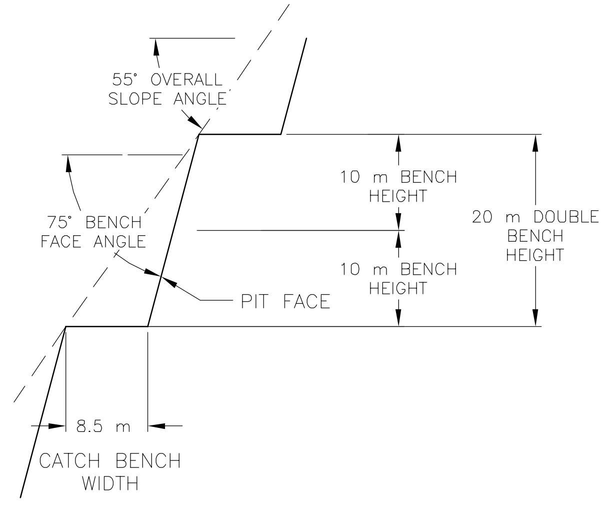
TITLE
BENCH AND INTER-RAMP SLOPE CONFIGURATION

CLIENT/PROJECT
OSISKO
CANADIAN MALARTIC PROJECT

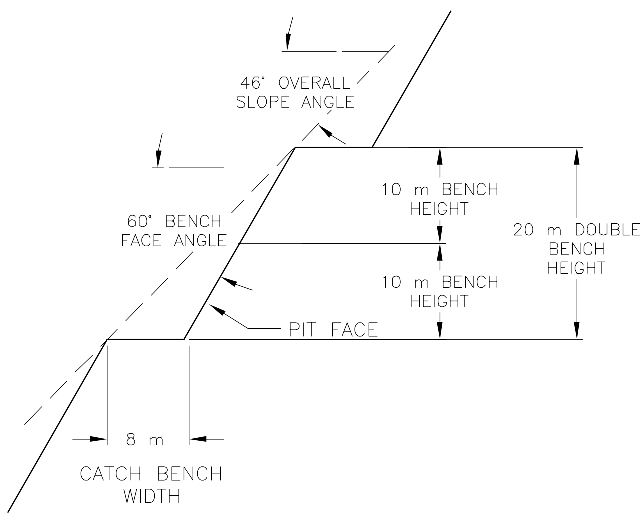
DRAWN RLK	DATE 07/06/08	JOB NO. 07-1221-0028
CHECKED GM	SCALE	DWG. NO./REV. NO.
REVIEWED	FILE NO.	FIGURE 15



ALL SECTORS EXCEPT NORTHEAST SECTOR
GOOD BUFFER BLASTING



ALL SECTORS EXCEPT NORTHEAST SECTOR
EFFECTIVE PRE-SPLIT BLASTING



NORTHEAST SECTOR

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TITLE
RECOMMENDED DESIGN SLOPE CONFIGURATIONS

CLIENT/PROJECT
OSISKO
CANADIAN MALARTIC PROJECT

DRAWN RJT	DATE 07/07/08	JOB NO. 07-1221-0028
CHECKED GM	SCALE AS SHOWN	DWG NO./REV. NO. S:\OSISKO\REPORT...\FIGURE 16.DWG
REVIEWED	FILE NO.	FIGURE 16