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FINAL REPORT ON

PIT SLOPE DESIGN CRITERIA FOR THE PORTAGE AND GOOSE ISLAND DEPOSITS MEADOWBANK PROJECT, NUNAVUT VOLUME 1

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EXECUTIVE SUMMARY

Meadowbank Mining Corporation (MMC) is preparing an "A Water License" application for submission to the Nunavut Water Board (NWB).

As part of the license application, the Nunavut Impact Review Board (NIRB) provided a list of certain requirements to be completed along with the license application. The following report responds to Item #17b from this list which requested the following:

> *Cumberland shall undertake a detailed technical review of all pit wall designs and shall include the detailed technical review in the application to the NWB for a water license.*

The request was part of a broader scope requiring the submission of detailed designs for dikes constructed in water depths greater than 10 m. The following report presents the results of the detailed stability analyses of the pit slopes in the Portage and Goose Island areas, where water depths exceed 10 m locally, and where the dike structures are located behind the pit walls.

The project area is covered extensively by lakes and portions of the gold-bearing deposits trend off-shore, beneath the lakes. As a consequence, it will be necessary to construct a series of dewatering dikes to allow mining of the deposits where they extend off-shore. The interaction between the open pits and the dewatering dikes is a critical design aspect of the project particularly from the perspective of the safety of personnel working in the open pits.

This document provides a summary of the slope design criteria for the initial open pits, proposed for the project, and includes additional stability analyses to assess the setback distance between pit crest and dike toe for the proposed tailings dike and the proposed dewatering dikes, along with predictions of slope deformation along the most critical dike section.

The slope design criteria were presented previously, and used in the economic Feasibility Study for the project. The design criteria were based on a kinematic analysis of the main structural features in the deposit area. The main structural controls are the foliation and stratigraphic contacts, which dip variably to the west at angles of up to 70 degrees, and systematic jointing. The foliation and stratigraphic contacts are considered to be persistent, while the jointing is considered to be non-persistent. The foliation and stratigraphic contacts may control multiple bench stability and potentially overall slope stability, whereas the non-persistent minor joint sets are more likely to result in local bench scale failures.

It is typical for mining projects to apply a factor of safety of 1.2 to 1.3 in the assessment of pit slopes. In the case of the Meadowbank Project, and specifically the Portage and Goose Island deposits, the design must consider the risks associated with the presence of the de-watering dike structures directly above the pits and a higher factor of safety is appropriate. Hence a factor of safety of 1.3 has been adopted for the assessment of the overall pit slopes while the minimum factor of safety against failures that may intersect the dikes has been set as 1.5 (consitent with guidelines for the dams according to Canadian Dam Association, 1999). The following table summarizes the minimum factors of safety for the assessment of pit slopes.

TABLE 1.1: Minimum Factors of Safety for Slope Stability

Location or Scale	Minimum Factor of Safety
Bench Scale Kinematic Assessment	1.2
Overall Pit Slope	1.3
Dike Toe	1.5
Pseudo-static	1.1

Additional limit equilibrium analyses, and distinct element analyses have been carried out to assess the overall stability of the pit slopes as these relate to the stability of the toe region of the dewatering dikes and tailings dike. The results of the analyses indicate that under conservative assumptions of continuous rock structure, uninterrupted by intact rock bridges, the required factors of safety for failures within the pit slope and for those intersecting the dike toe at the current minimum design setback distance of 80 m (70 m for the southeast wall of the Goose Island Pit) are achieved for all slopes with the exception of the Goose Southeast slope, the Goose West slope, and the Portage Southeast. Additional studies were directed towards those slopes.

Sensitivity analyses carried out for the Goose Southeast section, which is the critical design section, indicate that if groundwater in the slope is depressurized, and assuming the most conservative condition of 0% rock bridge, the required safety factors are achieved. Furthermore, the analyses show that if a reasonable percentage of rock bridge (20% for the Goose Southeast slope and 5% for both the Goose West and Portage Southeast slopes) is included in the analyses, then the required safety factors are achieved without depressurization. It is prudent, however to plan for slope depressurization for this slope because although 5 to 20% rock bridging is a reasonable assumption based upon the available data, it cannot be verified until mining begins and rock exposures can be mapped.

Based on a distinct element stress analysis along the Goose 12+00SE section representing the narrowest setback distance from pit crest to dike toe, and the deepest dike section, displacements at the inside toe of the dewatering dike are predicted to be less than 2.5 cm for the case of the pit slope with natural slope drainage, and less than 1 cm for the case of the depressurized pit slope. Consequently, deformations in the pit slope are not predicted to impact the stability or integrity of the de-watering dikes or tailings dike.

During operations a program of detailed geotechnical mapping should be implemented immediately. If the mapping can confirm whether a sufficient percentage of rock bridge is occurring in a given slope, then a decision could be made to reduce slope depressurization measures.

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LIST OF ABBREVIATIONS

2PL ..... Second Portage Lake
3PL ..... Third Portage Lake
ARD ..... Acid Rock Drainage
CDA ..... Canadian Dam Association
EM31 ..... Electromagnetic Geophysical Survey
IF ..... Iron Formation Rock
IV ..... Intermediate Volcanic Rock
m.A.S.L.: ..... Metres above Sea Level
NIRB ..... Nunavut Impact Review Board
NWB ..... Nunavut Water Board
PAG ..... Potential for Acid Generation
PHGA ..... Peak Horizontal Ground Acceleration
QTZ ..... Quartzite
UM ..... Ultramafic Rock
UTM ..... Universal Trans Mercator

Acidic – Any substance with a pH below 7.

Active layer – The zone above the permafrost that thaws in the summer and freezes in the winter.

Aggrade – To build up the level of (any land surface) by the deposition of sediment.

Alkaline – A substance with a pH greater than 7.

Azimuth – The horizontal component of a direction measured clockwise from north and measured in degrees.

Berm – A narrow path or ledge at the edge of a slope, road, or canal.

Borrow pit – An excavation dug to provide fill to make up ground elsewhere.

Cryoturbation – Churning and heaving of the ground and subsoil by frost action.

Degrade – To reduce or be reduced by erosion or down-cutting, as a land surface or bed of a river.

Dike – An embankment constructed to prevent flooding.

Dip – The geometric orientation, or inclination, of a plane, measured in degrees.

Dip Direction – Referring to the horizontal projection of a line on a plane marking the steepest inclination.

Effluent – Liquid discharged as waste, as from an industrial plant or sewage works.

Fault – A Fracture zone in which there has been movement or displacement of rocks, relative to each other, on either side.

Frost mound – Any mound-shaped landform produced by ground freezing combined with groundwater movement or the mitigation of soil moisture.

Glaciomarine – The processes, sediments and landforms associated with meltwater streams in contact with the sea.

Ground ice – A term used to describe all bodies of ice in the ground surface of the permafrost layer.

Hydraulic conductivity – A measure of the ability of a fluid to flow through the ground, determined by the size and shape of the pore spaces of the various rocks and soils.

Hydraulic Gradient – The slope of the groundwater level or water table.

Hydraulic head – The pressure exerted by a liquid as a result if the difference in its surface level between two points

Ice lens – Horizontal accumulation of permanently frozen ground ice.

Ice wedges – Wedge-shaped, ice body composed of vertically oriented ground ice that extends into the top of a permafrost layer.

Lithology – The physical characteristics of a rock, including colour, composition, and texture

Overburden – Earth overlying a useful deposit of rock or other useful material.

Patterned ground – Term used to describe a number of surface features found in periglacial environments. These features can resemble circles, polygons, nets, steps, and stripes. The development of some of these shapes is thought to be the result of freeze-thaw action.

Permeability – The ease of which liquids can pass through a rock or soil.

pH – A numerical measure of the acidity or alkalinity of water ranging from 0 to 14. Neutral waters have pH near 7. Acidic waters have pH less than 7 and alkaline waters have pH greater than 7.

Recharge area – An area of land where the groundwater moves downward and water infiltrates from the surface into the geological formations below.

Seismicity – Seismic activity; the phenomenon of earthquake activity.

Subaerial – Beneath the sky; in the open air, specifically (Geol) taking place on the earth's surface.

Subaqueous – Occurring, appearing, formed, or used under water.

Tailings – Waste left over after certain milling processes, such as from an ore-crushing plant.

Talik – An area of permanently unfrozen ground in regions of permafrost.

Thalweg – Line of deepest water in a stream channel as seen from above. Normally associated with the zone of greatest velocity in the stream.

Thermistor – (thermal resistor) A semiconductor whose resistance varies sharply in a known manner with temperature.

Thermokarst – A landscape dominated by depressions, pits, and caves that is created by the thawing of ground ice in high latitude locations.

Till – Unsorted sediment deposited by a glacier.

1.0 INTRODUCTION

The Meadowbank Mining Corporation (MMC) is currently planning the development of their Meadowbank Gold Project located in Nunavut. The project is located some 70 km north of the community of Baker Lake (see Figure 1.1) and consists of a series of gold deposits in close proximity to one another. It is planned to mine the deposits in a series of open pits using standard truck and shovel open pit mining methods.

Meadowbank Mining Corporation (MMC) is preparing an "A Water License" application for submission to the Nunavut Water Board (NWB).

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> *Cumberland shall undertake a detailed technical review of all pitwall designs and shall include the detailed technical review in the application to the NWB for a water license.*

The request is part of a broader scope requiring the submission of detailed designs for dikes constructed in water depths greater than 10 m. The following report presents the stability analyses of the proposed pit slopes in the Portage and Goose Island areas.

The project area is covered extensively by lakes. Portions of the gold-bearing deposits trend off-shore, beneath the lakes and as a consequence, it will be necessary to construct a series of dewatering dikes to allow mining of the deposits where they extend off-shore. The water depths in which the dewatering dikes are to be constructed are typically shallow, on the order of 4 m or less. In certain areas of the dike alignment, water depths may reach upwards of 18 m to 20 m, such as to the southeast of the Goose Island pit.

The interaction between the open pits and the dewatering dikes is a critical design aspect of the project, particularly from the perspective of the safety of personnel working in the open pits. This document provides the slope design criteria for the various open pits and includes additional stability analyses to assess the interaction between pit crest and dike toe for the proposed tailings dike and the proposed dewatering dikes.

There are three main deposit areas being considered for mining: Goose Island, Portage, and Vault. The Portage Deposit is the main deposit at the project. It is located on a peninsula, and extends northward under Second Portage Lake and southward under Third Portage Lake. The Goose Island Deposit lies some 1000 m to the south of the Third Portage Deposit and beneath Third Portage Lake. The Vault Deposit is located some 5 km to the northeast of the Portage Deposit area. Figure 1.2 illustrates the site plan for Meadowbank.

The Vault Deposit pit slope designs are not part of this study because the dike required at the confluence of Vault Lake and Walley Lake is through very shallow water, is approximately 700 m from the proposed pit, and is not expected to be technically challenging to construct.

2.0 BACKGROUND

2.1 Gold Mineralization

The gold mineralization at the Meadowbank Project is stratiform, and is hosted predominantly within the iron formation rock at the Portage and Goose Island deposit areas and the intermediate volcanic rock at the Vault deposit area. In the Portage area, gold mineralization appears to be generally associated with the upper fold limb of the recumbent, isoclinal synform forming the main structural feature of the Portage area.

2.2 Mining Method and General Development Sequence

It is planned to mine the deposits as open pits using truck and shovel open pit mining methods.

Figures 2.1 to 2.5 illustrate the currently proposed mining development sequence progression for the Portage and Goose Island Pits. The sequence is discussed in the following paragraphs.

Mining in the project area will begin at the Portage deposit. A starter pit will be developed on the Third Portage peninsula (see Figure 2.1). The initial cut will not be back to the ultimate pit crest. Waste rock mined from the Portage starter pit will be used to construct the East Dike, extending northward from the Portage deposit to the north shore of Second Portage Lake, and the Bay Zone dike, constructed as a wrap around dike at the south end of the Portage Deposit. Following construction of the East Dike and the Bay Zone Dike, the northwest arm of Second Portage Lake will be dewatered to allow construction of a tailings dam. Tailings will be deposited into the natural basin of the de-watered lake.

Mining of the south end of the Portage Pit will continue after dewatering has been completed (see Figure 2.2). During this time, waste rock from the pit will be used for construction of the Goose Island Dike.

The Goose Pit will be developed once the Goose Island dike has been constructed and the area behind the dike has been de-watered. The initial cut on the east side of the Goose Pit will be at the ultimate pit crest. The ramp for the Goose Pit will be developed as a series of switch-backs on the west wall of the pit to allow the east wall of the pit to be developed steeply. The east side of the pit will consist of a benched wall with no ramp. Goose will be excavated starting in Year 2 and will be completed in Year 5 (see Figure 2.3).

The final depth of the Goose Pit will be approximately 166 m, to an elevation of approximately 132 m above sea level. The pit will be roughly oval in shape and approximately 630 m in a north-south direction and 430 m in an east-west direction.

The remaining waste rock mined from the Portage Pit will be directed to the Portage rock storage facility (RSF). Mining of the south end of the pit is expected to take approximately four years. The currently planned final pit depth at the south end of the Portage Pit will be 138 m, or at an elevation of 132 m above sea level.

Mining of the north end of the Portage Pit will begin in approximately Year 4 of the mining operations. Mining will consist of a northward progression from the main deposit area (see Figure 2.3). The access ramp will be located along the west pit wall. The region between the Portage and the North Portage Deposit, the Connector Zone, consists of a saddle, to the north and south of which the deposit plunges. As mining of the Portage pit extends northward, the deposit will occur at its shallowest point adjacent to the Tailings Dike. The depth of the pit in this area will be on the order of 80 m deep. Mining will follow the plunge of the deposit to the north. Waste rock mined from the north end of the Portage Pit will be end dumped to an in-pit dump at the completed south end of the pit (see Figure 2.4).

The final Portage Pit will be on the order of 1,900 m in a north-south direction and 300 to 500 m in an east west direction, depending on which area of the pit is being considered (see Figure 2.5).

2.3 Seismicity and Design Earthquake

The Meadowbank Project site is located in Seismic Zone '0' according to the Canadian Foundation Engineering Design Manual (1992), which is a region that historically experiences very low seismic activity. The range in Peak Ground Acceleration (PGA) values for Seismic Zone '0' is between 0g and 0.04g based on the manual.

For the assessment of slope stability under seismic loading, it is common geotechnical practice to adopt a hazard probability of occurrence of 10% in 50 years for the Maximum Design Earthquake (MDE). This is equivalent to a one in 475 year recurrence criteria. However, it is necessary to consider the impact of the design earthquake event as it relates to the stability of the de-watering dikes. According to Canadian Dam Safety Guidelines (CDA, 1999) dams are to be designed to withstand ground motions associated with the MDE without release of the reservoir. Therefore, the selection of the MDE must be predicated on the consideration of the consequence of failure of the de-watering dikes, rather than of the pit slope, as shown in the following table.

TABLE 2.1: Classification of Dams in Terms of Consequences of Failure, with Maximum Design Earthquake (after CDA 1999, p.1-12, 5-2)

Consequence Category	**Potential for incremental consequences of failure**		**Maximum Design Earthquake (MDE)**
	Loss of Life	**Financial**	**Probabilistically Determined (Annual exceedance Probability)**
Very High	Large increase	Excessive Increase	1/10,000
High	Some Increase	Large Increase	1/1,000 to 1/10,000
Low	No Fatalities	Moderate Damages	1/100 to 1/1,000
Very Low	No fatalities	Minor Damages beyond owner's property	

The dewatering dikes fall into the High consequence category. Dike failure could result in loss of life relative to the same earthquake with no dike in place. The maximum design earthquake (MDE) for the dewatering dikes is the 1 in 10,000 year event

The stabilities of the dewatering and tailings dikes have been assessed separately from the pit slopes, and are reported in Golder (2007a, 2007b).

A site specific study done by Pacific Geoscience Centre in Victoria in 2003 for the Meadowbank project area indicated the following data.

TABLE 2.2: Peak Horizontal Ground Accelerations for Meadowbank Site

Return Period of Seismic Event (years)	**Peak Horizontal Ground Acceleration (g)**
100	0.02
200	0.03
475	0.03
975	0.04
10,000	0.07[D]

Source: Seismic Risk Calculation for Meadowbank Project Site, Geological Survey of Canada, Natural Resources Canada, Sidney, BC, July, 2003.

[D] Design event extrapolated from Geological Survey of Canada (2003) data

3.0 SUMMARY OF PREVIOUS STUDIES

The geotechnical studies on which the respective pit slope and underground design criteria have been formulated are based on the collection of both oriented and non-oriented geotechnical data from drill core. The collection of non-oriented geotechnical data from exploration drill holes was implemented during the 1995 summer drilling program, and has continued through to 2006. Cumberland geologists and geotechnicians under the direction of Golder have undertaken the collection of geotechnical data from non-oriented drill holes. Golder personnel trained Cumberland staff in geotechnical data collection procedures, and their work has been reviewed periodically. The geotechnical data have been used to develop slope design criteria for the project based on kinematic analysis and pseudo-probabilistic studies. The development of the design criteria are described in greater detail later in this report.

A summary of the geotechnical investigations carried out on an annual basis is contained in Appendix I.

3.1 Geotechnical and Engineering Studies

Field geotechnical and engineering related studies have involved the following activities:

- Geotechnical drilling;
- Oriented coring;
- Laboratory testing;
- Thermistor installations;
- Hydraulic conductivity testing of bedrock and soils;
- Bathymetry surveys of lake systems;
- Lake bottom seismic profiling along dike alignments;
- Electro-magnetic surveys;
- Geomorphological mapping;
- Bedrock and soil laboratory testing and materials characterization;
- Infrastructure design;
- Surface hydrology studies;
- Water balance;
- Waste and water management studies;
- Geochemical characterization of waste rock, ore, and soils; and
- Water quality predictions.

Additional information for these studies can be obtained from individual reports. A list of reports is contained in the reference section at the end of this report.

The following table lists the specific geotechnical boreholes drilled during the various field investigations and considered as part of the current study. The corresponding deposit area in which each borehole was drilled is also indicated. The locations of the boreholes are shown on Figure 3.1.

TABLE 3.1: List of Geotechnical Boreholes Relevant to Current Study

Hole	Year	Location	UTM (83) Zone 14 Northing	UTM (83) Zone 14 Easting	El. (m)	Az (deg)	Dip (deg)
TP96-154	1996	Third Portage	7213573.01	639058.11	145.3	55	61
TP96-155		Third Portage	7213731.12	639012.49	141.27	90	60
TP97-194	1997	Third Portage	7213398.98	638984.36	137.97	90	64
TP97-196		Third Portage	7213223.55	638851.67	137.32	90	87
TP97-202		Third Portage	7213308.15	639000.72	139.31	90	64
TP98-312	1998	Third Portage	7213829.23	639031.35	139.01	90	54
TP98-313		Third Portage	7213460.52	639082.62	142.07	55	48.5
GT-NP02-01	2002	North Portage	7214523.02	639009.26	138.87	231.31	64
GT-NP02-02		North Portage	7214653.64	638943.12	136.87	272.04	50
GT-NP02-03		North Portage	7214721.32	639050.17	146.84	60.51	50
02GT-01*		East Dike	7214323*	639354*	132.5*	-	90
02GT-02A		East Dike	7214118.763	639382.430	132.478	-	90
02GT-02B		East Dike	7214118.763	639382.430	132.478	-	90
02GT-03		East Dike	7214485.544	639321.961	135.071	175	51
02GT-04		East Dike	7213818.697	639255.863	136.036	22	50
02GT-05*		Tailings Dike	7214823*	638598*	132.5*	-	90
02GT-06		Tailings Dike	7214639.370	638462.461	133.000	-	90
02GT-11		Tailings Dike	7214556.759	638394.374	132.782	-	90
NP02-401		North Portage	7214453.225	638956.244	132.911	114	66
NP02-412		North Portage	7214154.633	639073.478	133.237	109	68
GT02-NP-1		North Portage	7214523.01	639009.26	251.38	139	60
GT02-NP-2		North Portage	7214653.64	638943.12	292.12	137	50
GT02-NP-3		North Portage	7214721.32	639050.17	80.59	147	50
GT02-TP-1		Third Portage	7213739.65	638936.58	295.40	138	49
GT02-TP-2		Third Portage	7213350.33	638964.11	271.52	138	51
03GT-BZ-1	2003	Bay Zone Dike	7213129.491	639153.134	135.779	192	58
03GT-BZ-2		Bay Zone Dike	7212931	639000	134	-	90
03GT-BZ-3		Bay Zone Dike	7212991	638793	134	-	90
03GT-BZ-4		Bay Zone Dike	7213201.65	638646.00	137.311	150	50

Hole	Year	Location	UTM (83) Zone 14 Northing	UTM (83) Zone 14 Easting	El. (m)	Az (deg)	Dip (deg)
03GT-BZ-5	2003	Bay Zone Dike	7213096	638703	134	-	90
03GT-BZ-6		Bay Zone Dike	7212952	638906	134	-	90
03GT-GI-1		Goose Island Dike	7212775	638924	134	-	90
03GT-GI-2		Goose Island Dike	7212535	638938	134	-	90
03GT-GI-3		Goose Island Dike	7212073	638912	134	-	90
03GT-GI-4		Goose Island Dike	7211999	638676	134	-	90
03GT-GI-5		Goose Island Dike	7212091	638502	134	-	90
03GT-GI-6		Goose Island Dike	7212236	638398	134	-	90
03GT-GI-7		Goose Island Dike	7212245.771	638251.184	135	86	61
03GT-GI-8		Goose Island Dike	7212300	638967	134	-	90
03GT-GPIT-1		Goose Island Deposit	7212419.597	638677.886	133	52	62
03GT-GPIT-2		Goose Island Deposit	7212324.228	638761.620	134	271	69
03GT-GPIT-3		Goose Island Deposit	7212225.781	638653.143	133	134	61
03GT-GPIT-4		Goose Island Deposit	7212321.867	638699.236	130	291	58
03GT-SE-1		East Dike	7214176	639458	133	-	90
03GT-SE-2		East Dike	7213945	639527	133	-	90
03GT-TD-1		Tailings Dike	7214846.638	638841.538	134	223	56
03GT-TD-2		Tailings Dike	7214712	638772	133	-	90
03GT-TD-3		Tailings Dike	7214516	638752	133	-	90
03GT-TD-4		Tailings Dike	7214353	638775	133	-	90
03GT-TD-6		Tailings Dike	7214515.522	638751.681	133	-	90
03GT-Spec-F2		East Dike	7213853.491	639415.404	132	232	58.19
06GT-TD1	2006	Tailings Dike	7214678.0	638718.4	133	-	-89.4
06GT-TD2	Hole Abandoned						
06GT-TD2A	2006	Tailings Dike	7214467.1	638804.5	133.2	-	-89.3
06GT-TD3		Tailings Dike	7214396.6	638738.9	133.1	-	-89.6

Note: *All coordinates surveyed except 02GT-01 and 02GT-05.

3.2 Permafrost Investigations

A detailed description of the permafrost regime at the site is contained in:

- Golder Associates Ltd., 2003a. Report on Permafrost Thermal Regime Baseline Studies, Meadowbank Project, December 18, 2003

- Golder Associates Ltd., TM-2005g. Technical Memorandum on “Item #17 – Meadowbank Gold Project – Annotation of Permafrost Cross Section”, 12 September 2005

The Meadowbank Project area is located within the zone of continuous permafrost (see Figure 3.2) and with an annual average air temperature of -11.1ºC, is underlain by continuous permafrost except for lake induced taliks and thaw bulbs.

Thermal studies at the site were initiated during the 1996 summer exploration drilling program, with the installation of two thermistor cables in exploration boreholes drilled on Third Portage Peninsula. These studies continued with the installation of additional thermistor cables during field investigations in 1997, 1998, 2002, 2003, and 2006. To date, twenty three thermistor cables have been installed to characterize and monitor the thermal conditions and permafrost at the project site. The thermistors have been located to characterize the thermal regime at the project site both inland (away from the influence of deep lakes), as well as adjacent to lakes. The locations of the thermistors are shown on Figure 3.3.

The thermistors were installed in open holes which were subsequently allowed to freeze around the strings. All boreholes were drilled using Diamond Rotary Coring methods. In general, heated water was used to flush cuttings from the boreholes during drilling. In some cases, calcium chloride was added to the drilling water to prevent freezing of the water in the boreholes. Thermistor boreholes that were completed prior to 2002 and the background thermistor (THERM-1) installed in 2003 were drilled with NQ size drilling equipment. All thermistor boreholes drilled after 2002 with the exception of the background thermistor, were drilled with HQ size drilling equipment. Resistivity measurements for each point or node on the thermistor string were measured after installation and over increasing time intervals. Data have been collected manually from the thermistors during the subsequent field seasons. No data loggers have been installed, although this may be considered for future continuous monitoring.

An electromagnetic survey (EM31) carried out by Golder (2003) in specific areas of the project site indicated that in general, the majority of the areas covered by the survey are underlain by dry permafrost (upland areas). There is little evidence of ground ice, patterned ground, thermokarst, frost mounds, cryoturbation, and other permafrost features, although these may be found in limited areas of the project site such as lowlands, marshy areas, and poorly drained areas. The extensive areas of felsenmeer or block fields in some areas are indicative of intense frost action. An assessment of surficial geomorphology and periglacial processes by Golder (2004e) provides additional information relating to typical hazards to be expected in permafrost terrain.

3.3 Hydraulic Conductivity Testing

The following reports and technical memoranda provide additional information relating to the hydrogeological conditions at the site:

- Golder Associates Ltd., TM-2005c. Technical Memorandum on "Items #24A and 37, Predictions of Regional Groundwater Flow Directions After Mine Closure, Meadowbank Project", October 5, 2005

- Golder Associates Ltd., TM-2004a. Technical Memorandum on "Meadowbank 2004 Baseline Groundwater Quality", October 20, 2004

- Golder Associates Ltd., TM-2004d. Technical Memorandum on "Meadowbank Baseline Groundwater Quality", April 26, 2004

- Golder Associates Ltd., 2004i. Report on *Hydrogeology Baseline Studies, Meadowbank Gold Project*, February 3, 2004

- Golder Associates Ltd., 2002a. Letter on *Meadowbank Project – Groundwater Characterization*, December 13, 2002

- Golder Associates Ltd., 2004c. Report on *Predictions of Brackish Water Upwelling in Open Pits, Meadowbank Project, Nunavut*, March 12, 2004

The locations of the boreholes from which data have been collected are shown on Figure 3.4.

The data used to develop the hydrogeological model for the site were collected from hydraulic conductivity testing conducted in the geotechnical boreholes during the geotechnical drilling investigation. The testing consisted of falling head and constant head tests to provide estimates of the hydraulic conductivity of the materials that may form the foundations and abutments for the proposed dewatering dykes and causeway and the structural features that may be exposed in the pit walls.

3.4 Laboratory Testing

The results of the laboratory testing are contained in Appendix II, and are summarized in the subsequent relevant Sections.

3.4.1 Rock Strength Testing

To date, a total of 36 rock samples from the project area have been tested for compressive strength, 8 samples of which had secant modulus and Poisson's ratio measured at 50% of the ultimate strength. The samples were tested at Golder Associates Ltd. Burnaby laboratory testing facilities. Wet and dry densities were determined for the samples.

The results of the rock strength laboratory testing are discussed in relevant sections later in the report.

3.4.2 Fault Gouge Characterization

A sample of clay fault gouge was preserved by first wrapping in cellophane followed by double bagging using "Zip-Loc" bags. The sample was tested to determine grain size gradation, Atterberg Limits, and moisture content.

3.5 Data Interpretation Methods

The data interpretation methods used to assess rock mass quality and to develop slope design criteria were based on standard internationally accepted methodologies for characterizing rock mass and for designing open pit slopes. The data collected from the on-going exploration drilling programs have been used to classify the rock quality in the project area on the basis of the Norwegian Geotechnical Institute (NGI) rock mass rating method. A description of the system is provided in Appendix III.

The RQD values determined from the 1996 exploration drill holes, based incorrectly on a 15cm core index length, have been excluded from all the analyses.

For the purposes of rock mass classification, Q´ is calculated assuming a Stress Reduction Factor (SRF) equal to 1, and Joint Water Reduction number (Jw) equal to 1. Therefore, Q´ is defined as:

- Q´ = RQD/Jn x Jr/Ja where,
- RQD = rock quality designation;
- Jn = joint set number;
- Jr = joint roughness number;
- Ja = joint alteration number;

Q and Q´ are based on a logarithmic scale. Q´ can be converted to Bieniawski's Rock Mass Rating (RMR), which is based on a linear scale, using the following relationship:

$$RMR = 9*\ln(Q') + 44$$

The RMR rock mass ratings and classifications are shown in the following table.

TABLE 3.2: RMR Rating Classification

RMR Rating		Rock Mass Classification
Greater Than	**Less Than**	
0	20	Very Poor Rock
20	40	Poor Rock
40	60	Fair Rock
60	80	Good Rock
80	100	Very Good Rock

As part of the geotechnical drilling investigations, qualitative estimates of rock strength at the Meadowbank Project were made based on International Society of Rock Mechanics (ISRM) standard field identification techniques (ISRM, 1976).

Oriented Geotechnical Data

The clay imprint method was used to collect orientation data from exploration and geotechnical boreholes relating to discontinuities. The orientation data were entered into electronic spreadsheets and then imported into the RocScience program DIPS. The DIPS program takes oriented core data and converts it to the true dip and dip direction for the discontinuities. The data was then used to plot stereonets, or graphical representations of the orientation of the data. This information was then used to carry out kinematic stability assessments of potential planar and wedge instability mechanisms and used as input to limit equilibrium and distinct element modeling programs.

3.6 Data Management

The data collected from the oriented and non-oriented geotechnical drilling programs were input into electronic spreadsheet files. In 2007, the data from the previous geotechnical data collection programs were combined into a single AcQuire database for data management. A series of algorithms, based on the algorithms developed for the spreadsheet files, have been written to calculate Q' and RMR within the database as well as to extract relevant data.

3.7 Data Integrity

Quality control (QC) was performed on approximately 5% of the data used to calculate Q' and RMR. This consisted of a check between selected original paper logs and the electronic logs, as well as a manual calculation of Q' and RMR to compare with the calculated values. In cases where original logs were not available, it was not possible to validate the data entry. A summary of the checked boreholes, as well as any adjustments made, is included in Appendix IV.

3.8 Formulation of Slope Designs

Non-oriented geotechnical data collection from exploration drill holes was initiated at the Third Portage site during 1995. The results of the preliminary non-oriented data collection program formed the basis for preliminary pit slope configurations presented in early 1996 (Golder, 1996c).

Preliminary oriented geotechnical data collection was initiated at the Third Portage project site during the 1996 summer drilling investigations. Drill core from drill holes TP96-154 and TP96-155 was oriented and geotechnically logged in detail in the field by a Golder staff member. The results of the preliminary oriented data collection program formed the basis for revised pit slope configurations in 1996 (Golder, 1996a).

Additional oriented geotechnical data were collected in 1997 (Golder, 1997a) and 1998 (Golder, 1998b) and were used to update the engineering geological model. The data formed the basis the pre-feasibility slope design criteria for the Portage project area (Golder, 1999d). The data were also used to develop preliminary pit slope design criteria for the North Portage Deposit (Golder, 1999c) and the Goose Island Deposit (Golder, 1999a). The slope design criteria were developed by carrying out a kinematic assessment and pseudo-probabilistic analysis of the main structural features in the deposit area.

During 2002 a series of oriented geotechnical boreholes were drilled in the North Portage deposit area. In 2003, a series of oriented geotechnical boreholes were drilled in the Goose Island deposit. The boreholes were used to confirm the assumptions on which the preliminary pit slope design criteria were based.

In 2004, Amec Americas Ltd (Amec) began a bankable Feasibility Study for the project (Amec, 2005). As part of that study, Golder undertook to review and update the engineering geological model based on the additional oriented data collected in 2002 and

2003. A series of Technical Memoranda were issued between December 2003 and May 2004 presenting the updated slope design criteria for the Goose Pit and Portage Pit (Golder, TM-2003a, TM-2003b, TM-2004b) to be used in the Feasibility Study by Amec.

4.0 SITE PHYSIOGRAPHY AND REGIONAL GEOLOGY

The Meadowbank project is located in the Canadian Shield, the largest physiographic region of Canada. The Shield hosts the largest area of Archean rocks (>2.5 billion years old) in the world including the oldest rocks, dated at 4 billion years. All rock units at the project site are of Archean age.

4.1 Topography and Bathymetry

The general site area consists of low, rolling hills with numerous small lakes. The topography in the immediate vicinity of the main deposits (Portage and Goose Island Deposits) is of generally low relief with a range in elevation of about 70 m. Elevations vary from about 133 m A.S.L. along the Second Portage Lake shoreline and 134 m A.S.L. along the Third Portage Lake shoreline, to maximum elevations of approximately 200 m A.S.L. to the northwest of the Portage area deposits.

Bathymetric surveys were conducted by Golder in 2002, 2003 and 2006 (Golder, 2002c, 2003h, 2006a, respectively) for lake areas adjacent to and over the main deposits at Meadowbank. The lake bottom has a similar topography to the adjacent land. Water depths reach 38 m in Second Portage Lake while at the south end of Third Portage Lake, water depths close to 64 m were measured during the bathymetric surveys.

4.2 Geomorphology and Surficial Processes

The surficial geology is dominated by discontinuous thin veneers of organic material, till and/or weathered parent material overlying undulating to hummocky bedrock. Blockfields of weathered parent material interspersed with thin veneers of moraine or organics are common. The observed periglacial geomorphic processes are typical of areas underlain by continuous permafrost, although their expression is subdued by the relative thin cover of overburden and the relatively dry site conditions. Terrain features and geomorphic processes associated with excess ground ice and generally wet conditions exist in limited areas, generally associated with low lying bogs and areas of poor drainage. Colluvial aprons at the toe of steep rocky bluffs indicate that frost action on exposed bedrock and the resultant rockfall and rock displacement are active processes, although rare within the project area.

Physical weathering (frost wedging and frost shattering) of *in situ* materials typically occurs on exposed bedrock and in coarse grained block fields. Freezing induced displacements of soils (frost creep, frost heave, frost jacking and frost sorting) are ubiquitous across the site. Evidence of cryoturbation of the morainal, weathered bedrock and organic veneers occurs in the form of sorted stripes and muted patterned ground.

Thaw induced displacement of soils (solifluction and thaw consolidation/settlement) associated with the active layer is pervasive throughout the area. Solifluction typically occurs where the slope is greater than sub-horizontal and the parent bedrock has at least a veneer of material. Thaw settlement and consolidation of soils following development or surface disturbance would be expected to occur to a limited extent within portions of the project area.

Within the Meadowbank Project area, surficial weathering processes, such as frost shattering of bedrock, is typically limited to the upper 2 m to 3 m of bedrock or less. This is based on a review of rock quality data from drill core in the upper 20 m of the borehole. Furthermore, observations made at a rock quarry currently being used to supply aggregate on site indicate that the depth of frost shattering is on the order of 1.5 m in that specific area.

4.3 Permafrost

The Meadowbank Gold project is located within the Low Arctic ecoclimatic zone, one of the coldest and driest regions of Canada. The topography of the area is of generally low relief with an elevation range of 70 m. The surficial geology is dominated by discontinuous thin veneers of organic material, till, and/or weathered parent material overlying undulating to hummocky bedrock. Block fields of weathered rock interspersed with thin veneers of moraine or organics are common.

The ground ice content in the region is expected to be between 0% and 10% (dry permafrost) based on regional scale compilation data (International Permafrost Association, 1997).

The observed periglacial geomorphic processes are typical of areas underlain by continuous permafrost, although their expression is subdued by the relatively thin cover of overburden and the relatively dry site conditions. Terrain features and geomorphic processes associated with excess ground ice and generally wet conditions exist in limited areas commonly associated with low-lying bogs and areas of poor drainage. Colluvial aprons at the toe of steep rocky bluffs indicate that frost action on exposed bedrock and the resultant rockfall and rock displacement are active processes, although rare within the project area. In general, the geomorphology and soils observed within the area do not present any features or processes that prohibit the development of the proposed mine. Continuous permafrost to depths of between 450 and 550 m underlies most of the Meadowbank project area. Based on the current site thermistor instrumentation, the depth of the active layer in the project area ranges from about 1.3 m in areas of shallow overburden and away from the influence of lakes, up to 4 m adjacent to lakes, and up to 6.5 m beneath the stream connecting Third Portage and Second Portage lakes. Taliks extending through the permafrost will exist beneath circular lakes having a minimum diameter of 570 m, and elongate lakes having a minimum width of 320 m. Based on this, Second Portage Lake and Third Portage Lake will have taliks extending through the permafrost. The talik beneath Vault Lake is considered to be isolated, not extending through the permafrost.

The shallow groundwater flow regime at the Meadowbank project has little to no hydraulic connection with the groundwater regime located below the deep permafrost. On a regional scale, deep groundwater in the Meadowbank project area will flow either to the northwest or to the southeast from Third Portage Lake. This is due to the project being located near the drainage divide between the Back River Basin, which flows north and northwest to the Arctic Ocean, and the Thelon River Basin, which flows east to southeast towards Hudson Bay. The northwest portion of Second Portage Lake, however, is a discharge zone with water flowing upwards from the deep groundwater regime. This is due to large and higher elevation lakes located to the east of Second Portage Lake.

The relationship between the permafrost and the deep groundwater regime is discussed in greater detail in Section 7.

4.4 Surficial Geology

The project area is covered by laterally extensive deposits of glacial till. In general terms, the till can be described as a sandy till, having a fines (silt plus clay) content between about 30% and 40% based on laboratory grain size analyses.

Glaciofluviual sand and gravel deposits have reportedly been observed in three areas: on the north shore of Second Portage Lake, the north shore of the eastern arm of Turn Lake, and to the south of the Vault Deposit (Cumberland, 2003).

In general, the material that has been recovered from beneath the lakes during geotechnical drilling along the proposed dike alignments has been described as cobbles and gravel with traces of sand, silt, and clay. Locally, samples of sand have been obtained.

4.5 Regional Geology

Figure 4.1 shows the regional geology of the Meadowbank area. The following description of the general geology of the project area is based on reports by MMC and by Amec (2005).

The deposits that make up the Meadowbank Project lie in the Rae sub-province of the western Churchill Province of the Canadian Shield. The host unit is the Archean Woodburn Lake Group, which occurs as a narrow neck of structurally complex supracrustal rocks sandwiched between granite plutons. Rocks of the Woodburn Lake Group have been correlated with units of the Prince Albert Group to the northeast towards Committee Bay. The Paleo-proterozoic Baker Lake Basin unconformably overlies the Woodburn Lake Group to the south.

The Woodburn Lake Group consists of quartzites, komatiites, iron formation, felsic to intermediate volcaniclastic rocks and related sedimentary rocks. These units are variably deformed and metamorphosed at greenschist to granulite facies. The regional metamorphic history is characterized by amphibolite facies metamorphic assemblages to the south of the Meadowbank Project. To the north, chloritoid-bearing greenschist facies assemblages prevail. This suggests that the Meadowbank gold deposits lie near the greenschist/amphibolite transition. A low-grade Hudsonian thermal metamorphic overprint is indicated by 1.75 Ga K-Ar ages of micas, and Hudsonian magmatic activity documented by 1.8 Ga monzonite in an undeformed granite dike south of Meadowbank.

Three principal deformation events are preserved throughout the Meadowbank region. These entail an early tight to isoclinal folding and profound transposition (D1), subsequent mesoscopic to macroscopic kink folding (D2) of D1-related fabrics, and a gentler crenulation overprint (D3), which weakly modifies D1-D2 fabrics. The morphologies of the D1-D3 structural fabric elements, and their relative timing relations, are consistent throughout the area.

4.6 Regional Faulting

Two main faults exist in the Portage and Goose Island deposit areas; the Second Portage Lake Fault, and the Bay Fault. The locations of these along a section through the Portage Deposit are shown on Figure 4.2. The orientations are tabulated in the following table

TABLE 4.1: Database Entry Quality Control Checks

Fault	Dip	Dip Direction	Age (Ga)
Bay Fault	70°	270°	1.85 (Pehrsson et al., 2005, Sherlock et al., 2005)
Second Portage Lake Fault	70°	235°	1.72 (Rainbird et al., 2005)

The Second Portage Fault is located just north of the Third Portage Peninsula area, and trends northwest, parallel to the northwest arm of Second Portage Lake. Its projected surface trace would intersect the proposed tailings dike as well as the east dewatering dike. The Second Portage Fault is interpreted as a discrete, narrow focused brittle-ductile fault.

The Third Portage peninsula is flanked on the west by the north-south trending Bay Zone Fault, which roughly parallels the western shoreline of Third Portage peninsula and extends northward along the western flank of the North Portage deposit and south-southeastward between the eastern shore of the Bay Zone Island and the Third Portage peninsula. A splay trending off the Bay Zone Fault begins south of the narrows separating the Third Portage peninsula from the mainland and trends south along the western side of the Bay Zone Island. Both pass beneath the proposed Bay Zone dike, but whether one or both pass beneath the Goose Island dike is unknown. The Bay Zone fault is interpreted as a ductile shear, across which stratigraphic continuity is transposed but maintained.

The potential for fault reactivation is considered to be very low. The project area is in a zone of low seismic activity, Seismic Zone '0' (Canadian Foundation Engineering Manual, 1992). The Second Portage fault and Bay fault are on the order of 1.7 to 1.9 billion years in age (Pehrsson, 2001; Rainbird, 2005). Pehrsson (2005) reports that the structures in the deposit area have not been demonstrably reactivated by orogenic events in the area. McMartin et al (2004) report that these structures do not localize post-glacial rebound. The potential for fault reactivation is low due to the type, or character of the faults, the absence of evidence for reactivation despite known tectonism, and the low seismic activity of the area. Consequently, there is a very low risk for potential damage to the tailings dike and loss of tailings pond integrity associated with reactivation of the Second Portage Lake Fault.

4.6.1 Second Portage Lake Fault

The Second Portage Lake Fault was intersected in geotechnical borehole NP02-412, which was drilled towards 109 degrees azimuth and remained within the fault to a down hole depth of about 67 m. The borehole was characterized by very poor quality rock with clay fault gouge associated with fracture surfaces. The projected extension of the Second Portage Lake fault to the southeast would pass beneath the proposed east dewatering dyke at its southern end. Two geotechnical boreholes (02GT-04 and 03GT-SPEC-F2) were drilled in the area of the East Dike to attempt to intersect the fault. Borehole 02GT-04 was drilled as part of the dyke foundation investigations while 03GT-SPEC-F2 was drilled specifically to attempt to intersect the projected southeast extension of the fault. While each borehole intersected occasional small (<0.5m wide) shear zones along foliation, neither borehole intersected broad zones of poor quality rock associated with major structures. This is consistent with previous observations and interpretations at the site. Borehole 03GT-SPEC-F2 intersected the contact between mafic to ultramafic rock and iron formation at a depth of about 140 m. The contact zone was characterized by low RQD and low Solid Core Recovery over a down hole interval of about 4 m.

4.6.2 Bay Fault

Areas of lower RQD and rock mass quality are generally associated with the major high-angle Bay Fault trend and Second Portage Lake Fault trend. A review of the basic and detailed non-oriented geotechnical data was presented previously in the pre-feasibility study (Golder, 1999d) and indicated the Bay Fault to be characterized by intervals of up to 15 metres of low RQD (<50%) in areas where the fault is near to the ground surface (at depths less than approximately 60 metres). However, a review of the geotechnical data from several holes indicated that at depths below approximately 60 metres, the degree to which the Bay Fault affects the surrounding rock mass quality is generally limited to one to two drill runs, or 3 to 6 metres. Consequently, where the Bay Fault trend is intersected in the pit walls at depths greater than about 60 metres, limited zones of lower rock mass quality will be encountered and only limited ravelling is expected. These observations are consistent with the interpretation of both the Bay Fault and the Second Portage Lake fault having narrow widths, possibly on the order of 5 m or less. Cumberland geologists have indicated that these faults are interpreted to be narrow and the associated discontinuities, structure on the order of 3 m to 5 m width.

4.7 Geology of the Portage Deposit Area

The project area has undergone a series of regional deformational events resulting in typical ‘dome and basin’ fold structures, which is evident in the geology plan of Figure 4.3 for the Portage area. “Dome and basin” folding, are illustrated on Figure 4.4. The dominant structural feature of the Third Portage area is a gently to steeply inclined, tightly folded north/south trending anticline within the Third Portage peninsula. This structure has resulted in the iron formation, interbedded volcaniclastic, and metasedimentary rocks being folded around a core of ultramafic volcanics (see Figure 4.5). The limbs of the fold dip to the west at variable angles. Superimposed on the east-west deformational even, but less pronounced, is a north-south deformational event that has resulted in secondary folding of stratigraphy about east-west trending axes. Consequently, while the general trend of stratigraphy and foliation is north-south striking and westward dipping, the superposition of folding events results in complexly folded stratigraphy. The axial plane of this fold is inclined to the west and south-west at angles of around 20 degrees in the north to 50 to 60 degrees, or greater, through the central and southern portion of the deposit area (see Figure 4.6). A variable, but locally intense, S1 axial plane foliation is associated with this folding.

Axial plane and layer-parallel ductile shearing, faulting, and intense mylonization are commonly associated with the main stratigraphic contacts. This shearing is likely associated with the weaker lithologic units; the ultramafic complex and the sericite schist. Brittle deformation features such as fault gouge and strongly broken core are reportedly rare in the immediate Third Portage area.

Bedding parallel flexural slip surfaces are associated with the major stratigraphic contacts rather than with individual beds within stratigraphic units, which are likely to be erratically folded. Within the fold limbs these will tend to be parallel, or sub-parallel to the axial plane, while through the nose region these will tend to be steeply dipping and perpendicular to the axial plane.

Where the major stratigraphic contacts are sheared and faulted, these will control the bench scale and pit wall stability. Beneath the Third Portage Peninsula, these contacts will be inclined westward at steep angles (>60 degrees) along the eastern side of the deposit. Through the central portion of the deposit, the contacts will dip at shallow angles (<30 degrees) to the west and to the east within the lower limbs of the deposit.

Directly adjacent to the main north-south trending Bay Fault the stratigraphic contacts are inclined westward at moderate to steep angles, generally about 60 degrees. The dip of the stratigraphy and foliation decreases with increasing distance westward and eastward, away from the Bay Fault. Based on the geological cross sections and on the oriented drilling the most significant drag folding and down-warping of stratigraphy occurs within about 15 m into the hanging wall and in the footwall of the fault.

4.8 Geology of the Goose Island Area

The Goose Island Deposit is a steeply dipping, stratiform gold bearing iron formation that is part of a sequence of Archean ultramafic and mafic flow sequences, volcaniclastic sediments, felsic to intermediate flows and tuffs, and sediments. The ultramafic rocks are variably altered and contain serpentinite, chlorite, actinolite, and talc. The geology of the Goose Island Area is shown on Figure 4.7. A typical cross section through the deposit is shown on Figure 4.8.

The deposit trends northward and southward from Goose Island and dips at steep angles, generally greater than about 55 to 60 degrees to the west.

Axial plane and bedding-parallel ductile shearing are common due to the intense regional deformation events. This shearing is most commonly associated with the weaker lithologic units such as the ultramafic volcanic rock. Brittle deformation features such as fault gouge and strongly broken core are reportedly rare in the immediate Third Portage and Goose Island area. Axial planar and bedding parallel foliation, which is pervasive throughout the rock mass, occurs commonly as healed fractures, rather than open fractures within the rock.

4.9 Deposit Characteristics

Gold mineralization in the ore is closely associated with low levels of sulphide mineralization, dominantly pyrrhotite and pyrite. In the main Third Portage deposit and Goose Island deposit, these sulphides dominantly occur as a replacement of magnetite in the oxide iron formations. They also occur as a fracture fill silica and disseminations in both the iron formation and surrounding volcaniclastic units.

The bulk of the gold mineralization is contained within the iron formations, with mineralization occurring in the clastic units representing remobilization and secondary enrichment by gold-bearing fluids. The gold tends to be concentrated along the lower limb and in the hinge areas of a recumbent fold and shows excellent continuity both along strike and down dip through the deposits.

The gold mineralization at the Portage Deposit and Goose Island Deposit is stratiform, generally associated with the iron formation rock.

5.0 ENGINEERING GEOLOGY OF THE GOOSE ISLAND AND PORTAGE DEPOSITS

5.1 Lakebed Sediments

Soft, lakebed sediments are expected to be on the order of 0.2 to 0.25 m in thickness based on estimates made from the 2006 bathymetric survey (Golder, 2006a).

5.2 Till

The project area is covered by laterally extensive deposits of glacial till. In general terms, the till can be described as a sandy till, having a fines (silt plus clay) content between about 30% and 40% based on laboratory grain size analyses.

In addition to till deposits local occurrences of glaciofluvial sand and gravel have been noted in boreholes.

5.3 Bedrock Geology

In the Third Portage Lake area, the supracrustal stratigraphy consists (from oldest to youngest) of:

(1) ultramafic volcanics;

(2) felsic to intermediate volcaniclastic and/or greywacke;

(3) interbedded magnetite-chert iron formation and associated pelitic schists; and,

(4) quartzite.

In the Portage - Goose Island area, the package of rocks occur in a recumbent fold geometry with the volcaniclastic/clastic units and interbedded pelitic and chemical sediments isoclinally folded about an ultramafic core. This geometry is best developed in the central part of Third Portage, where the fold closure can be seen. To the north and south of Third Portage, erosion has removed the closure and only the lower limb of the fold structure remains. A detailed description of the lithological units is provided below.

5.3.1 Ultramafic Volcanics/Komatiites (UMV)

Ultramafic volcanics at Meadowbank are a pale blue-grey to blue-green, fine-grained to aphanitic, talc with greater or lesser amounts of serpentine and minor amounts of chlorite schist. Numerous blebs and stringers of calcite that tend to be aligned parallel to the dominant foliation commonly cut the units. A variant of this unit, UMA, contains abundant coarse, randomly oriented actinolite/tremolite blades of apparent hydrothermal as it tends to occur at UMV/iron formation contacts.

5.3.2 Iron Formation (IFMQ or IFQM)

Oxide facies iron formations are the most common host of mineralization at Meadowbank. These units consist of banded magnetite (brown) and chert (grey/white). The bands are vary in width from a few millimetres to several centimeters and often display impressive evidence of strong folding that is related to regional stresses. The magnetite often contains radiating masses of acicular grunerite crystals, a regional metamorphic effect. Locally, where mineralized, the iron formation can be moderately to strongly silicified.

5.3.3 Intermediate Volcaniclastic (IV)

Intermediate volcaniclastic units are the most voluminous rocks in the Meadowbank area and generally consist of light to medium green, fine to medium grained chlorite schist. Where intimate with or peripheral to gold mineralization, these units have been subject to weak to moderate sericite alteration and locally weak to moderate biotite alteration imparting yellow-green and brown-green colors to the rock respectively. The IV units are often cut by swarms of S1 parallel grey to white quartz veinlets.

A main subtype of IV that has been identified at Meadowbank is IVchl, an aphanitic, chlorite rich unit that is best described as a pelitic schist. At higher metamorphic grades the IVchl commonly contains significant biotite and lesser garnet porphyroblasts.

5.3.4 Quartzite/Chert Pebble Conglomerate (QTZ/CPC)

This is a fine to medium grained unit, which contains minor muscovite and sericite on foliation planes. Often contains minor fuchsite and disseminated pyrite. Quartzite often grades into or is interbedded with chert pebble conglomerate. The chert pebble conglomerate generally contains sub-rounded clasts (1 cm to 5 cm in size) of chert and/or quartz.

5.3.5 Mineralization

Gold mineralization in the Portage and Goose deposits at Meadowbank is intimately associated with sulphides, dominantly pyrite and pyrrhotite, which occur in two main habits. Most predominant is as replacement of magnetite in the oxide iron formations where the sulphides tend to be concentrated along S0/S1 planes and possibly S2 in fold limbs. Also important is sulphide occurring as fracture fill with or without silica and disseminations in both the iron formation and surrounding clastic units. Total sulphide content generally varies from 1% to 2% up to approximately 10%. Locally over very short widths sulphide content, the proportions of pyrrhotite versus pyrite and replacement versus fracture fill can be higher and variable. In the Goose and Third Portage areas pyrrhotite replacement is dominant while in North Portage pyrite replacement is dominant. Gold grades do generally increase with increasing sulphide content however there does not appear to be a specific correlation with either pyrrhotite or pyrite.

The bulk of the gold mineralization in the deposits is contained within the iron formations (wrapped around a core of ultramafic rocks). However some is contained within the IV units occurring as very fine to fine disseminations and to a lesser extent as fracture fill.

The gold tends to be concentrated along the lower limb and in the hinge areas of the recumbent fold, and shows excellent continuity both along strike and down dip through the deposits. The concentration of sulphides and gold along S1 and S2 in the deposits indicates that the bulk of the mineralization must have occurred during the D1-D2 deformational event. Later concentrations of pyrite with or without pyrrhotite and gold are associated with local quartz veins that appear to occur along the axial planes of F3 folds. This style of mineralization may be related to remobilization of gold during the D3 deformational event.

5.4 Main Structural Features

The following sections describe characteristics of the main structural features at the Portage and Goose deposit areas. Overall pit slope stability will be controlled by the orientation and persistence of the main stratigraphic contacts and major fault structures in the Portage and Goose Deposit areas, while bench stability will be influenced by the persistent structures as well as the non-persistent structures.

5.4.1 Persistence of Discontinuities

The potential for movement (shearing) to occur along a discontinuity is a function of the persistence of the discontinuity, the small scale roughness and large scale waviness of the discontinuity, the presence or absence of rock bridging within the rock mass, the presence of water, and the confining stress across the potential shear plane.

The presence of rock bridges within the rock mass is a function of the persistence of the joint discontinuities. Persistence is the trace length of a discontinuity as observed in an exposure and is sometimes referred to as continuity. It is reduced when a discontinuity terminates in solid rock or against other discontinuities. Discontinuities can be classified as:

1. persistent, where the discontinuity is a continuous plane in the geotechnical unit;
2. abutting, where the discontinuity terminates against other discontinuities; and
3. non-persistent, where the discontinuities terminate in intact rock.

The main stratigraphic contacts are expected to be persistent, through-going features with an associated reduction of rock quality immediately adjacent to these contacts. The areas of lower rock quality are generally confined to narrow intervals.

Major fault structures, such as the Bay Fault and the Second Portage Lake Fault, are considered to be through-going and persistent and will therefore influence pit slope stability at the scale of the pit wall height and at the scale of the bench height.

Stratigraphic Contacts, Bedding Parallel and Axial Planar Faulting

The Bay Fault zone trends northward along the west side of the Portage deposit area, extending beyond the north shore of Second Portage Lake. The southward extension of the Bay Fault zone, associated with the possible westward dipping axial plane of a tight isoclinal fold, has been intersected in geotechnical and exploration boreholes to the west of Goose Island.

Bedding parallel flexural slip surfaces are likely to be most commonly associated with the major stratigraphic contacts rather than with individual beds within stratigraphic units. The main stratigraphic contacts are potentially zones of weakness, but are expected to be undulating.

The stratigraphic contacts and bedding parallel flexural slip surfaces are considered to be peristent and through going structures. Therefore, pit slope configurations will be predominantly controlled by the orientations of these features.

Foliation

Based on the geotechnical studies at the Meadowbank Project, slickensiding and shearing can be associated with the foliation surfaces. The foliation surfaces tend to be slightly altered with occasional coatings.

Bedding parallel foliation will be pervasive throughout the deposit area and will commonly occur as competent, solid rock with the exception of major stratigraphic contacts and where flexural slip has occurred along bedding surfaces. Within the ultramafic rock, it is likely that the bedding parallel foliation surfaces will be chloritic, serpentinitic, talcose, or actinolitic. Within the more felsic units, sericitic coatings will likely be common. Within the iron formation, bedding parallel foliation surfaces are likely to be fresh and unaltered.

The detailed analyses of the bedding parallel foliation orientations indicate that, locally, the foliation can vary considerably in orientation, and that this local variation is random due to the two different deformational events. This high degree of local variability may contribute to the overall stability of the bench faces and overall pit slopes in that potential sliding failure along these features will be limited in extent, as they will be irregular and wavy, rather than planar, smooth features. However, bench face ravelling will likely still occur.

The axial planar surfaces within the iron formation and other rock types tend to be healed, discontinuous fractures. It is likely that well developed, through-going axial planar surfaces will only be developed as localized structural zones. These surfaces are likely to be moderately close (0.3 m to 1 m) to widely spaced (1 m to 3 m), based on the analyses of the recent geotechnical drilling.

Jointing

The joint surfaces at the site are expected to be non-persistent on the basis of joint surface characteristics determined in drill core. Consequently, both planar and wedge failure mechanisms formed by these discontinuous surfaces are themselves expected to be discontinuous and widely spaced.

Orthogonal Joints

Orthogonal joints will strike in approximately the same direction as the main foliation and stratigraphic contacts, but will dip at approximately 90 degrees to the dip of the stratigraphy. Due to the relatively consistent, westward inclined orientation of the stratigraphy the orthogonal joints will generally be inclined to the east.

The orthogonal joints are likely to be non-persistent, widely (1 m to 3 m) to very widely spaced (>3 m) based on the detailed oriented geotechnical data collection. These will likely be confined to individual stratigraphic horizons, and terminate within rock, or against main stratigraphic surfaces or bedding parallel foliation. The orthogonal joint surfaces are expected to be rough and planar to rough and wavy and may be infilled, or healed, by secondary minerals such as quartz and carbonate.

Conjugate Joints

Conjugate joints are systematic joints that have developed during flexural-slip folding of stratigraphic layers. These joints are oblique joints that form as a response to shortening perpendicular to the axial surface of a fold.

These features are expected to be widely spaced (1 m to 3 m) to very widely spaced (>3 m) and non-persistent over vertical distances greater than an operating bench height, based on the results of oriented drilling. Due to their wide spacing and relatively steep dip, these joints are expected to be non-persistent, and will present a limited or low risk of large scale or multi-bench failure.

Cross and Shallow Joints

Cross joints are typically oriented perpendicular to the axis of folding. These joints are expected to be discontinuous and very widely spaced (4 m average spacing for the cross joints and > 8 m average spacing for the shallow joints).

The shallow oriented joint set may present an adversely oriented structure that could potentially contribute to overall slope instability in the Goose Island area. In the Goose Island area, this joint set dips at angles of up to about 34 degrees to the west, and so may form a potential basal release surface.

The small scale (drill core scale) joint roughness, Jr, for the shallow sets varies on average from about 1.8 to 2.2 indicating rough surfaces while the joint alteration, Ja, is on the order of 1.1 to 1.2 indicating clean unaltered joint surfaces. The fact that these joints are very rough and have negligible alteration suggests that they have not been subjected to any movement and are therefore likely to be non-persistent, terminating within rock. The wide spacing of these features suggests that there is a low likelihood of encountering such features in a given bench. On a 24 m high final bench, only 3 such discontinuous joints might be encountered. Therefore, the influence of this particular joint set on the bench scale and overall stability of the slopes is expected to be minimal. This is discussed in greater detail before.

5.5 Structural Domains

Geologic deposits are generally separated into Structural Domains for rock slope design purposes. These Domains are generally defined by significant structural boundaries, such as fault zones, by changes in structural orientations that may control pit slope configurations or by changes in rock mass quality that may control bench scale or overall slope stability.

The development of Structural Domains for pit slope design purposes at the Meadowbank Project Site have been based on the interpretative geological cross sections, the oriented geotechnical drilling, and the general understanding of the engineering geology of the project that has been developed through many years of data collection and analysis.

5.5.1 Goose Island Structural Domains

The Goose Island deposit dips at reasonably consistent angles of about 55 to 60 degrees towards the west. Based on the interpretative cross sections and on the recent oriented geotechnical drilling, at depths of less than about 35 m vertically the stratigraphy and foliation may dip at shallower angles, on the order of 45 to 55 degrees. Furthermore, at deeper depths the stratigraphy and foliation steepens, and in some cases may reverse dip direction, dipping eastward at high angles of 70 degrees or greater. Finally, to the west of the Bay Fault trending north to northeast just west of Goose Island, the dip of stratigraphy decreases substantially to very shallow westward dipping, and in some cases eastward dipping, angles.

Based on the above, the Goose Island deposit has been subdivided into four Structural Domains. These are shown on Figure 5.1 as projected on to the open pit design.

Structural Domain 1 (East of approximately 0+75E, north of approximately 12+25S, and elevation less than about 100 m A.S.L.)

The upper slopes of the east wall and portions of the north wall of the Goose Island Pit will be developed within this Domain. Based on the interpretative geological cross sections and on the oriented geotechnical drilling at depths less than about 35m, the foliation and main stratigraphic contacts within this Domain will dip at angles on average between about 45 and 55 degrees to the west, west-northwest, and west-southwest. The conjugate joint sets within this Domain dip at angles averaging between 62 and 73 degrees. The northeast dipping conjugate joint is absent within this Domain while the cross joints dip at about 56 degrees towards the northwest. The orthogonal joint set dips at shallow angles towards the east and a shallow dipping joint set is present dipping at about 20 degrees to the west to northwest.

The following table summarizes the major structural orientations within this Domain.

TABLE 5.1: Measured Structural Orientations – Domain 1

Joint Set	Type	Dip	Dip Direction	Interpreted Persistence
1	Foliation and Bedding	49°	252°	Persistent
	Foliation	44°	296°	
2	Orthogonal Joint Set	12°	129°	Non-persistent
3	Conjugate Joint CJ1	62°	212°	
	Conjugate Joint CJ2	Absent		
	Conjugate Joint CJ3	71°	147°	Non-persistent
	Conjugate Joint CJ4	75°	340°	
4	Cross Joint CX1	43°	357°	
5	Shallow Joints	20°	279°	

The orientations in the above table are shown on Figure 5.2.

Structural Domain 2 (Elevations below about 100 m, and above about 64 m A.S.L., northeast and southeast transition walls)

The mid-portions of the east pit wall and the mid to upper portions of the northeast and southeast transition walls will be developed within this Domain. At depths between about 35 m and 70 m, the dip of the foliation and stratigraphic contacts increases, dipping at angles greater than 60 degrees to the west based on the interpretative geological cross sections and on the oriented geotechnical drilling. Conjugate joint sets and the cross jointing within this Domain dip at angles greater than 60 degrees. The orthogonal joint set dips at moderate angles of about 42 degrees towards the east. A shallow joint set dips at about 34 degrees towards the west to northwest.

The following table summarizes the major structural orientations within this Domain.

TABLE 5.2: Measured Structural Orientations – Domain 2

Joint Set	Type	Dip	Dip Direction	Interpreted Persistence
1	Foliation and Bedding	74°	280°	Persistent
2	Orthogonal Joint Set	39°	097°	Non-persistent
3	Conjugate Joint CJ1	65°	221°	
	Conjugate Joint CJ2	63°	055°	
	Conjugate Joint CJ3	65°	142°	
	Conjugate Joint CJ4	60°	335°	
4	Cross Joint CX1	60°	008°	
5	Shallow Joints	33°	305°	

These orientations are shown on Figure 5.3.

Structural Domain 3 (Elevations below about 64 m A.S.L.)

The lower portions of the east and west pit walls and the north and south end walls will be developed within this Domain. At pit depths below about 70 m, the dip of the foliation and stratigraphic contacts is variable. Based on the oriented drilling, the orientation of bedding within this Domain differs from that of the foliation to a larger degree than in other Domains. The bedding dips at high angles of about 74 degrees to the west while the foliation dips at lower angles around 54 degrees to the west. In some cases the foliation and bedding reverse dip direction to dip steeply at about 71 degrees to the east. It is not possible to resolve the variability of the reverse dipping orientation into specific depths or locations.

Conjugate joint sets and the cross jointing within this Domain dip at angles greater than 60 degrees. The southeast dipping conjugate joint set is absent within this Domain while the orthogonal joint set dips at shallow angles of about 21 degrees towards the west. A shallow joint set dips at about 24 degrees towards the north.

The following table summarizes the major structural orientations within this Domain.

TABLE 5.3: Measured Structural Orientations – Domain 3

Joint Set	Type	Dip	Dip Direction	Interprete d Persistence
1	Foliation	55°	292°	Persistent
	Bedding	73°	273°	
	Foliation and Bedding	71°	099°	
2	Orthogonal Joint Set	23°	286°	Non-Persistent
3	Conjugate Joint CJ1	61°	231°	
	Conjugate Joint CJ2	62°	041°	
	Conjugate Joint CJ3	Absent		
	Conjugate Joint CJ4	73°	326°	Non-Persistent
4	Cross Joint CX1	60°	002°	
5	Shallow Joints	24°	008°	

These orientations are shown on Figure 5.4.

Structural Domain 4 (Mid to upper west pit wall, west of about 0+50 east)

The mid to upper portions of the west pit and portions of the southwest and northwest transition walls will be developed within this Domain. The orientation of stratigraphy and foliation within this Domain is indicated to generally dip at shallow angles of about 20 degrees towards the west, although it may also dip in some cases at shallow angles to the east to southeast. The southwest and northeast dipping conjugate joint sets dip at moderate angles of about 43 degrees and 36 degrees respectively. The southeast dipping conjugate set dips steeply at about 73 degrees while the northwest dipping set is absent. The orthogonal joint set dips steeply to the east at about 66 degrees. The cross joint set is interpreted to be dipping steeply at about 60 degrees to the northeast. A shallow joint set dips at about 21 degrees towards the southeast.

The following table summarizes the major structural orientations within this Domain.

TABLE 5.4: Measured Structural Orientations – Domain 4

<table>
<tr><th>Joint Set</th><th>Type</th><th>Dip</th><th>Dip Direction</th><th>Interpreted Persistence</th></tr>
<tr><td>1</td><td>Foliation and Bedding</td><td>20°</td><td>255°</td><td>Persistent</td></tr>
<tr><td>2</td><td>Orthogonal Joint Set</td><td>66°</td><td>100°</td><td rowspan="4">Non-persistent</td></tr>
<tr><td rowspan="4">3</td><td>Conjugate Joint CJ1</td><td>43°</td><td>201°</td></tr>
<tr><td>Conjugate Joint CJ2</td><td>36°</td><td>043°</td></tr>
<tr><td>Conjugate Joint CJ3</td><td>73°</td><td>148°</td></tr>
<tr><td>Conjugate Joint CJ4</td><td colspan="3">Absent</td></tr>
<tr><td>4</td><td>Cross Joint CX1</td><td>60°</td><td>031°</td><td>Non-persistent</td></tr>
<tr><td>5</td><td>Shallow Joints</td><td>21°</td><td>144°</td><td>Non-persistent</td></tr>
</table>

These orientations will be the dominant controls over the pit slope design configurations within this Domain and are shown on Figure 5.5.

5.5.2 Portage (Main) Deposit Structural Domains

The Portage (Main) area is that portion of the portage deposit that lies south of Second Portage Fault.

The Portage (Main) Deposit has been subdivided into seven Structural Domains based on the results of the oriented drilling, rock mass quality assessments, and on the geologic cross sections. Due to the interpretation of a north trending splay fault located between the west shoreline of the Bay Zone island and the east shoreline of the mainland, Domain TP-4 has been subdivided further into an additional sub-domain. In the north part of the Portage area, the boundaries of the Domains are defined by the north-south Bay Fault trend bounding the deposit to the west and by the northwest-southeast trending Second Portage Lake Fault more south. The Domain boundaries are shown on Figure 5.6.

Based on geological cross sections and on oriented geotechnical drilling, the orientation of the stratigraphy directly adjacent to the Bay Fault is steeply inclined at angles of about 60 degrees westward or greater due to drag-folding. Similar observations are made at Goose Island. The influence of the fault on the orientation of the stratigraphy (*i.e.*, the steepest inclination) appears to be generally limited to within about 15 m into the hangingwall and the footwall of the fault. However, in some cases this influence may extend further. This forms a transition between steeply westward dipping stratigraphy and stratigraphy dipping at shallower angles of less than about 50 degrees, with increasing distance away from the fault. To the east, the mineralized fold limbs generally dip westward at shallow inclinations of less than about 30 degrees. In the northern section of the pit, moving westward away from the fault, the orientation of stratigraphy is interpreted to gradually decrease to shallower inclinations. This interpretation is substantiated by the drilling along the proposed Tailings Dyke to the west of the northwestern section of the Portage pit crest.

The Structural Domains for the Portage Deposit are described below.

Structural Domain TP-1 (North of approximately 2+50N)

The fold axial plane of the dominant fold trends at approximately 022 degrees azimuth in this Domain. The fold nose within this domain has been eroded leaving only the lower recumbent limb of the fold.

Based on the geologic cross sections and on the oriented geotechnical drilling, the foliation and stratigraphy within Structural Domain TP-1 are inclined at shallow angles generally to the west. Foliation measurements from oriented boreholes indicate the foliation to be generally inclined to the west at angles less than about 20 to 25 degrees. The average dip of stratigraphic contacts, based on the oriented drilling, is approximately 13 degrees towards 310 degrees azimuth. Due to the flat lying foliation and stratigraphic contacts within this Domain, the dip azimuth of these features is somewhat variable. In general, these features dip to the west at shallow angles. Occasionally, these dip to the east at very shallow angles. Because of the variability in the orientation of the foliation and stratigraphic contacts, the orientation of the orthogonal joint set is also variable as it is oriented generally perpendicular to the foliation and stratigraphic contacts. Consequently, the orthogonal joint set may dip at steep angles to the east and to the west, depending on the orientation of the foliation.

The following table summarizes the average structural orientations for this Domain.

TABLE 5.5:.Third Portage Deposit Structural Domain TP-1 – Structural Orientations

Joint Set	Type	Dip	Dip Direction	Interpreted Persistence
1	Foliation and Contact Surfaces	12°	307°	Persistent
2	Orthogonal Joint Set (East Dipping)	75°	093°	Non-peristent
	Orthogonal Joint Set (West Dipping)	69°	279°	
3	Conjugate Joint 1	76°	238°	
	Conjugate Joint 2	71°	046°	
	Conjugate Joint 3 (Rare)	86°	148°	
	Conjugate Joint 4 (Rare to Absent)	Absent		
4	Cross Joint 1	26°	191°	Non-peristent
5	Bay Fault Trend and Associated Splays	70°	270°	Persistent
	Second Portage Lake Fault	70°	235°	

Contact and foliation orientations determined from the oriented drilling are summarized in the following table.

TABLE 5.6: Summary of Measured Contact Orientations – Structural Domain TP-1

Type	Dip	Dip Direction	Hole Number	Down-Hole Depth (m)
Measured Contacts	05°	112°	TP98-312	65.65
Measured Contacts	13°	271°	TP98-312	66.68
Measured Contacts	21°	299°	TP98-312	67.96
Measured Contacts	07°	291°	TP98-312	68.10
Measured Contacts	33°	007°	02GT-TP-1	34.80
Measured Contacts	35°	263°	02GT-TP-1	54.70
Measured Contacts	14°	336°	02GT-TP-1	63.14
Measured Contacts	11°	049°	02GT-TP-1	64.55
Measured Contacts	35°	298°	02GT-TP-1	65.17

TABLE 5.7: Summary of Measured Fault and Shear Surfaces – Structural Domain TP-1

Type	Dip	Dip Direction	Hole Number	Down-Hole Depth (m)
Measured Shears	19°	253°	02GT-TP-1	53.14
Measured Shears	31°	299°	02GT-TP-1	53.30
Measured Shears	31°	299°	02GT-TP-1	53.37

These orientations are shown on Figure 5.7.

At the southern limit of this domain, a transition zone is present where the foliation and stratigraphic contacts begin to steepen in orientation along the eastern margins of the deposit and the dip direction of these features changes from west to northwest dipping, to west to southwest dipping. Consequently, the division between Structural Domain TP-1 and Structural Domain TP-2 is defined by this transition area. This occurs at approximately 2+50N and generally corresponds to a re-orientation of the fold axial plane to a trend of about 340º azimuth.

Structural Domain TP-2 (approximately 0+00N to 2+50N)

The axial plane of the dominant folding direction trends at approximately 340 degrees azimuth through this Domain and is inclined to the south-west. Data collected from oriented boreholes TP96-154, TP96-155, and TP98-313 were used to evaluate the structural orientations within this Domain.

Domain TP-2 has been subdivided into an East Sector and a West Sector based on the general orientation of the foliation and stratigraphy as interpreted in the plan geology and also from the oriented drilling. Within the East Sector, the foliation and stratigraphy dip to the west at angles less than about 15 degrees to steeper than 60 degrees. Within the West Sector, the foliation is generally flat lying, or may dip to the east at shallow angles less than about 20 to 30 degrees.

Directly adjacent to the Bay Fault, stratigraphy becomes down-warped into the orientation of the fault and will dip at high angles of greater than 60 degrees to the west.

It is expected that the upper slopes of the east walls of the Starter Pit and Ultimate Pit in this Domain will be developed within the steeply dipping stratigraphy in the East Sector, while the lower slopes and the base of the pit will be developed in the more shallowly westward dipping, or flat lying lower limbs.

The current interpretation suggests that the west wall of the proposed Starter Pit will be developed entirely within the shallowly inclined to flat lying westward dipping limbs of the fold. As mining of the deposit progresses beyond the Starter Pit phase, the west wall will be pushed back and the pit will be deepened. Depending on the final pit plan, it is expected that the base of the pit and a small portion of the final lower west wall of the ultimate pit will be developed within the shallow east dipping foliation and stratigraphy. It is expected that the middle to upper portions of the wall will be developed in more steeply westward dipping stratigraphy.

The following table summarizes the general structural orientations for this Domain.

TABLE 5.8: Third Portage Deposit Structural Domain
TP-2 – Structural Orientations

<table>
<tr><th>Joint Set</th><th>Type</th><th>Dip</th><th>Dip Direction</th><th>Interpreted Persistence</th></tr>
<tr><td>1</td><td>Foliation/Contact Surfaces (Steep)</td><td>64°</td><td>263°</td><td>Persistent</td></tr>
<tr><td>2</td><td>Associated Orthogonal Joint Set (Shallow)</td><td>14°</td><td>067°</td><td>Non-persistent</td></tr>
<tr><td>1</td><td>Foliation/Contact Surfaces (Shallow)</td><td>25°</td><td>298°</td><td>Persistent</td></tr>
<tr><td>2</td><td>Associated Orthogonal Joint Set (Steep)</td><td>67°</td><td>097°</td><td>Non-persistent</td></tr>
<tr><td rowspan="4">3</td><td>Conjugate Joint 1</td><td>56°</td><td>221°</td><td rowspan="2">Non-persistent</td></tr>
<tr><td>Conjugate Joint 2 (Rare)</td><td>73°</td><td>035°</td></tr>
<tr><td>Conjugate Joint 3 (Absent)</td><td colspan="3">Absent</td></tr>
<tr><td>Conjugate Joint 4</td><td>63°</td><td>330°</td><td rowspan="2">Non-persistent</td></tr>
<tr><td>4</td><td>Cross Joint</td><td>62°</td><td>170°</td></tr>
</table>

Contact orientations determined from the oriented drilling are summarized in the following table.

TABLE 5.9: Summary of Measured Contact Orientations – Structural Domain TP-2

Type	Dip	Dip Direction	Hole Number	Down-Hole Depth (m)
Measured Contacts	59°	346°	TP98-261	31.75
Measured Contacts	81°	277°	TP98-261	46.67
Measured Contacts	57°	202°	TP98-261	65.03
Measured Contacts	51°	292°	TP98-261	94.87
Measured Contacts	89°	283°	TP98-261	112.02
Measured Contacts	25°	19°	TP98-261	120.63
Measured Contacts	28°	95°	TP98-313	26.60
Measured Contacts	16°	272°	TP98-313	32.83
Measured Contacts	19°	242°	TP98-313	33.30
Measured Contacts	40°	325°	TP98-313	37.70
Measured Contacts	51°	329°	TP98-313	39.30
Measured Contacts	56°	268°	TP98-313	46.30
Measured Contacts	36°	277°	TP98-313	52.15
Measured Contacts	88°	214°	TP98-313	86.80

The following table summarizes the measured fault and shear orientations based on the oriented geotechnical drilling.

TABLE 5.10: Summary of Measured Fault and Shear Surfaces – Structural Domain TP-2

Type	Dip	Dip Direction	Hole Number	Down-HoleDepth (m)
Measured Shears	89°	318°	TP97-194	44.97
Measured Shears	26°	350°	TP97-194	104.13
Measured Shears	31°	145°	TP97-194	104.18
Measured Shears	17°	347°	TP97-194	104.28
Measured Shears	21°	57°	TP97-194	108.00
Measured Shears	26°	303°	TP98-261	33.54
Measured Shears	71°	304°	TP98-261	120.08

The orientations presented in the previous tables are shown on Figure 5.8.

Structural Domain TP-3 (South of 0+00N)

The division between Domain TP-2 and Domain TP-3 results from re-orientation of the main fold axial plane at around 0+00N as indicated by geologic sections and by oriented drilling. The foliation and stratigraphy dip towards the west and northwest.

As with Structural Domain TP-2, Domain TP-3 has been sub-divided into an East Sector and West Sector based on the general orientation of the foliation and stratigraphy as interpreted in the plan geology, and also from the oriented drilling. Within the East Sector, the foliation and stratigraphy dip to the west at angles less than about 15 degrees to steeper than 60 degrees. Within the West Sector, the foliation is generally flat lying, or may dip to the east at shallow angles less than about 20 to 30 degrees. Within this Structural Domain, the fold axial plane trends at about 015 degrees azimuth and is inclined to the west.

Boreholes TP97-194, TP97-202, and 03GT-BZ-1 were collared within the East Region of this Domain. Borehole 03GT-BZ-1 was drilled as part of the dyke geotechnical field investigations and the oriented structural data obtained from this borehole has been used to confirm the interpreted structural orientations within this region. Geotechnical borehole GT02-TP-2 was collared within the West Region of Domain TP-3, and was drilled towards the west to investigate east dipping structures.

Drill holes TP98-258 and TP98-265 were collared west of the Bay Fault, drilled through the fault, and terminated within Domain TP-3. Thus, the lower portions of these holes within Domain TP-3 have been used in assessing the structural trends within this Domain, adjacent to the Bay Fault.

Within the West Sector, the foliation and stratigraphy will be generally flat lying with variable dip azimuth based on the contoured pole densities of foliation data collected from TP97-202 and from foliation measurements from TP98-258 and TP98-265 in the footwall of the Bay Fault. The foliation and stratigraphy will dip at shallow angles of less than about 30 degrees to the north, or less than about 10 degrees to the west. Foliation and stratigraphy may also dip at shallow angles less than about 30 degrees towards the east.

Within the East Sector and in the hangingwall and footwall of the Bay Fault, stratigraphy will be inclined steeply to the west at inclinations greater than 60 degrees to 70 degrees. Oriented structural data from borehole 03GT-BZ-1, drilled approximately 100 m east of the proposed pit crest, confirms that foliation and stratigraphy dip at steep angles towards the west.

The upper slopes of the east pit walls of the proposed Starter Pit will likely be developed within the steeply westward inclined stratigraphy. The west pit wall of the proposed Starter Pit will be developed entirely within the shallow westward inclined fold limbs. Expansion of the Starter Pit will result in a push-back of the west wall into Structural Domain TP-4.

The following table summarizes the general structural orientations for this Domain for the East and West sectors, respectively. These have been updated from the pre-feasibility study based on boreholes GT02-TP-2 and 03GT-BZ-1 to reflect the current understanding of the deposit area. These orientations have been used in subsequent analyses of plane and wedge failure modes.

TABLE 5.11: Structural Domain TP-3 East and Bay Fault - Structural Orientations

<table>
<tr><th>Joint Set</th><th>Type</th><th>Dip</th><th>Dip Direction</th><th>Interpreted Persistence</th></tr>
<tr><td>1</td><td>Foliation and Contact Surfaces</td><td>73°</td><td>266°</td><td>Persistent</td></tr>
<tr><td>2</td><td>Orthogonal Joint Set</td><td>28°</td><td>093°</td><td rowspan="8">Non-Persistent</td></tr>
<tr><td rowspan="4">3</td><td>Conjugate Joint 1 (Rare to absent)</td><td>79°</td><td>237°</td></tr>
<tr><td>Conjugate Joint 2 (Rare to absent)</td><td>81°</td><td>068°</td></tr>
<tr><td>Conjugate Joint 3</td><td>72°</td><td>138°</td></tr>
<tr><td>Conjugate Joint 4</td><td>69°</td><td>316°</td></tr>
<tr><td rowspan="3">4</td><td>Cross Joint 1</td><td>47°</td><td>178°</td></tr>
<tr><td>Cross Joint 2</td><td>46°</td><td>023°</td></tr>
<tr><td>Cross Joint 3</td><td>88°</td><td>349°</td></tr>
<tr><td rowspan="2">5</td><td>Bay Fault Trend and Associated Splays</td><td>70°</td><td>270°</td><td rowspan="2">Persistent</td></tr>
<tr><td>Second Portage Lake Fault</td><td>70°</td><td>235°</td></tr>
</table>

TABLE 5.12: Structural Domain TP-3 West – Structural Orientations

Joint Set	Type	Dip	Dip Direction	Interpreted Persistence
1	Foliation and Contact Surfaces	26°	340°	Persistent
	Foliation and Contact Surfaces	08°	281°	
2	Orthogonal Joint Set (West Dipping)	69°	262°	Non-Persistent
	Orthogonal Joint Set (East Dipping)	76°	103°	
3	Conjugate Joint 1 (Rare to Absent)	72°	221°	
	Conjugate Joint 2	Absent		
	Conjugate Joint 3	85°	147°	
	Conjugate Joint 4 (Rare)	58°	320°	
4	Cross Joint 1	47°	171°	
	Cross Joint 2	61°	010°	
	Cross Joint 3	Absent		
5	Bay Fault Trend and Associated Splays	70°	270°	Persistent
	Second Portage Lake Fault	70°	235°	

TABLE 5.13: Measured Contact Orientations – Structural Domain TP-3

Type	Dip	Dip Direction	Hole Number	Down-Hole Depth (m)
Measured Contacts	21°	247°	TP98-258	159.5
Measured Contacts	16°	350°	TP98-258	161.6
Measured Contacts	36°	351°	TP98-258	170.8
Measured Contacts	22°	089°	TP98-258	170.6
Measured Contacts	75°	244°	TP98-258	81.8
Measured Contacts	28°	292°	GT02-TP-2	44.4
Measured Contacts	87°	143°	GT02-TP-2	58.6

TABLE 5.14: Summary of Measured Fault and Shear Surfaces – Structural Domain TP-3

Type	Dip	Dip Direction	Hole Number	Down-Hole Depth (m)
Measured Shears	26°	350°	TP97-194	104.13
Measured Shears	31°	145°	TP97-194	104.18
Measured Shears	17°	347°	TP97-194	104.28
Measured Shears	21°	57°	TP97-194	108.00
Measured Shears	53°	218°	TP98-258	82.33
Measured Shears	37°	228°	TP98-258	82.40

The stereographic projections of orientation data in Domain TP-3 are shown in Figure 5.9 and 5.10.

Structural Domain TP-4 (West of Bay Fault)

The Bay Zone mineralization is located beneath a small island to the south-west of the Third Portage peninsula, and occurs within an interbedded sequence of iron formation and intermediate volcanic rock. The depth to mineralization at the Bay Zone may vary between about 20 m below surface near the shoreline of the peninsula to about 70 m beneath the bay itself.

The Bay Zone island is flanked on the east by the Bay Fault and on the west by a splay of the main Bay Fault. Both faults are inclined steeply to the west at angles between 60 and 80 degrees. Structural Domain TP-4 has been sub-divided into Domains TP-4 and TP-4A.

Drill holes TP98-258 and TP98-265 were collared in Domain TP-4, west of the Bay Fault. The two holes were drilled eastward to intersect the fault. Foliation orientations measured in TP98-258 vary from steeply dipping to the west to moderate to shallow inclinations to the east within the hangingwall of the Bay Fault. Foliation orientations measured in borehole TP98-265 indicate generally moderate to shallow inclinations to the north to depths of around 30 m. Between 30 m and 80 m, closer to the Bay Fault, inclinations increase to around 40 degrees to the north-east.

Measurements of contact orientations within the immediate hangingwall of the Bay fault indicate these to be steeply inclined (greater than 70 degrees) to the west. Stratigraphic contact orientations measured in drill-hole TP98-265 are indicated to be steeply inclined to the north (between 60 and 70 degrees) and to the east. The steeply inclined east dipping orientations are associated directly with the fault zone and likely are a result of local drag-folding of stratigraphy due to faulting.

Based on oriented data collected from borehole TP98-258, foliation and stratigraphy within the immediate hangingwall of the Bay Fault dips steeply to the west, which is consistent with the engineering geology model for the deposit. In the footwall of the fault, the stratigraphic contacts tend to flatten resulting in variable dip azimuths.

Foliation orientations measured in borehole TP98-261 remain reasonably constant to a depth of 130 m, and are steeply inclined at angles greater than about 60 to 65 degrees to the west and north-west and possibly the south-east (undifferentiated fracture set). At depths below 130 m, the orientation of the foliation changes to shallow to moderate inclinations around 30 degrees to the north.

Additional oriented geotechnical data collected from borehole 03GT-BZ-4 and GT02-TP-2 have been used to update and confirm the structural interpretation of Structural Domain TP-4. Borehole 03GT-BZ-4 was drilled approximately 100 m west of the proposed pit crest at the southern end of the deposit. Foliation orientations measured in borehole 03GT-BZ-4 dip at about 24 degrees towards 316 degrees azimuth. Foliation orientations measured in borehole GT02-TP-2 dip at generally shallow angles (less than about 30 degrees) to the northwest and southwest.

The following tables summarize the structural orientations within this Domain.

TABLE 5.15: Structural Domain TP-4 – Structural Orientations

Joint Set	Type	Dip	Dip Direction	Interpreted Persistence
1	Foliation and Contact Surfaces (Adjacent to Bay Fault)	73°	271°	Persistent
	Foliation west of Bay Fault (03GT-BZ-4)	21°	325°	
2	Orthogonal Joint Set	24°	089°	Non-Persistent
3	Conjugate Joint 1 (weakly present)	59°	234°	
	Conjugate Joint 2	44°	025°	
	Conjugate Joint 3	75°	142°	
	Conjugate Joint 4	67°	315°	
4	Cross Joint 1	46°	173°	
	Cross Joint 2	81°	349°	
5	Bay Fault Trend and Associated Splays	70°	270°	Persistent
	Second Portage Lake Fault	70°	235°	

TABLE 5.16: Average Contact Orientations – Structural Domain TP-4

Type	Dip	Dip Direction	Hole Number	Down-Hole Depth (m)
Measured Contacts	75°	256°	TP98-258	72.04; 81.80
Measured Contacts	62°	352°	TP98-261, TP98-265	12.78; 31.75; 44.50
Measured Contacts	65°	101°	TP98-265	57.40; 75.00

TABLE 5.17: Summary of Measured Fault and Shear Surfaces – Structural Domain TP-4

Type	Dip	Dip Direction	Hole Number	Down-Hole Depth (m)
Measured Shears	34°	312°	TP98-258	33.97
Measured Fault	53°	218°	TP98-258	82.33
Measured Fault	37°	228°	TP98-258	82.40
Measured Fault	26°	303°	TP98-261	33.54
Measured Shears	35°	255°	GT-02-TP-2	118.63
Measured Shears	14°	272°	GT-02-TP-2	118.72
Measured Shears	27°	302°	GT-02-TP-2	118.80

The orientations presented in the previous tables will be the dominant controls over the inter-ramp pit slope design configurations within this Domain, and are shown on Figure 5.11.

5.5.3 Portage (North) Deposit Structural Domains

The Portage (North) area is that portion of the Portage Deposit that is north of the Second Portage Lake Fault.

Structural Domain NP-1

The interpreted Bay Fault trend and corresponding drag-folding of stratigraphy defines the main north-south domain boundary through the deposit area, separating Domain NP-1 west of the boundary from Domain NP-2 east of the boundary. Domain NP-1 is characterized by north trending and westward dipping stratigraphy interpreted as being inclined at angles less than about 30 degrees. This interpretation is similar through all deposits at the site, from Goose Island northward through the Portage Deposit area. The orientation of stratigraphy within Domain NP-1 and within the Transition Zone, and the orientation of the major fault trends, are favourable for the east facing walls that will be excavated along the west side of the pit, as these structures will be inclined into the walls, and for north and south facing walls, as the structures will intersect these walls at high angles.

Near the northern end of Domain NP-1, stratigraphy begins to trend westward, dipping to the south and southeast as it is folded around the hinge of a synform in this area.

Due to the similarity in interpretation of stratigraphic orientation to the west of the main Bay Fault trend, oriented structural data collected from the following boreholes were combined to provide a more statistically representative data set on which to base the pit slope design criteria.

- 03GT-TD-1
- 03GT-TD-5A
- GT02-TP-1
- TP98-261
- TP98-265

The following table summarizes the structural orientations within this Domain on which the pit slope designs are based.

TABLE 5.18: Structural Domain NP-1 - Structural Orientations

<table>
<tr><th>Joint Set</th><th>Type</th><th>Dip</th><th>Dip Direction</th><th>Interpreted Persistence</th></tr>
<tr><td rowspan="2">1</td><td>Foliation and Contact Surfaces</td><td>16°</td><td>284°</td><td rowspan="2">Persistent</td></tr>
<tr><td>Foliation and Contact Surfaces</td><td>17°</td><td>107°</td></tr>
<tr><td>2</td><td>Orthogonal Joint Set</td><td>75°</td><td>263°</td><td rowspan="6">Non-persistent</td></tr>
<tr><td rowspan="4">3</td><td>Conjugate Joint 1</td><td>68°</td><td>222°</td></tr>
<tr><td>Conjugate Joint 2</td><td>71°</td><td>038°</td></tr>
<tr><td>Conjugate Joint 3</td><td>76°</td><td>143°</td></tr>
<tr><td>Conjugate Joint 4</td><td>68°</td><td>313°</td></tr>
<tr><td>4</td><td>Cross Joint 1</td><td>77°</td><td>350°</td></tr>
<tr><td rowspan="2">5</td><td>Bay Fault Trend and Associated Splays</td><td>70°</td><td>280°</td><td rowspan="2">Persistent</td></tr>
<tr><td>Second Portage Lake Fault</td><td>70°</td><td>235°</td></tr>
</table>

The above orientations are shown on Figure 5.12 and 5.13.

Structural Domain NP-2

Structural Domain NP-2 is interpreted as the area east of the Bay Fault trend and corresponding drag-folding of stratigraphy. Domain NP-2 is characterized by westward gently dipping stratigraphy.

Based on the oriented geotechnical drilling and on interpretative geological cross sections, the main stratigraphic contacts within this Domain will be inclined at less than about 20 degrees westward. Again, these orientations are favourable for east, north, and south facing walls, as the orientations are inclined into the east facing walls, and strike into the north and south facing walls. However, these orientations will present some risk of slope failure for west facing (east) pit walls where these orientations are undercut by either bench faces or inter-ramp angles, but only if these surfaces dip greater than about 30 to 35 degrees.

The following table summarizes the structural orientations within this Domain on which the preliminary pit slope designs will be based. These orientations are taken from the detailed oriented drilling at the Third Portage site.

TABLE 5.19: Structural Domain NP-2 - Structural Orientations

<table>
<tr><th>Joint Set</th><th>Type</th><th>Dip</th><th>Dip Direction</th><th>Interpreted Persistence</th></tr>
<tr><td>1</td><td>Foliation and Contact Surfaces</td><td>14°</td><td>278°</td><td>Persistent</td></tr>
<tr><td>2</td><td>Orthogonal Joint Set</td><td>73°</td><td>092°</td><td rowspan="6">Non-persistent</td></tr>
<tr><td rowspan="4">3</td><td>Conjugate Joint 1</td><td>73°</td><td>210°</td></tr>
<tr><td>Conjugate Joint 2</td><td>69°</td><td>036°</td></tr>
<tr><td>Conjugate Joint 3</td><td>72°</td><td>125°</td></tr>
<tr><td>Conjugate Joint 4</td><td>68°</td><td>299°</td></tr>
<tr><td>4</td><td>Cross Joints</td><td>62°</td><td>341°</td></tr>
<tr><td rowspan="2">5</td><td>Bay Fault Trend and Associated Splays</td><td>70°</td><td>280°</td><td rowspan="2">Persistent</td></tr>
<tr><td>Second Portage Lake Fault</td><td>70°</td><td>235°</td></tr>
</table>

The orientations presented will be the dominant controls over the inter-ramp pit slope design configurations within this Domain, and are shown on Figure 5.14 and 5.15.

Structural Domain NP-3

Structural Domain NP-3 corresponds to the area adjacent to the Bay Fault trend where stratigraphy has been interpreted to be drag-folded into steeper orientations. The degree to which the fault zone influences the orientation of stratigraphy varies with lateral distance away from the fault zone. In some cases, the width of Domain NP-3 may be less than 15 m while in other cases it may reach widths up to about 50 m into the hangingwall and footwall based on interpretative geological cross sections and on oriented geotechnical data collection from rock core.

Domain NP-3 may be exposed over significant portions of the west pit wall. The orientation of stratigraphy within Domain NP-3 is favourable to the development of the west and north pit walls, dipping into the west wall at moderate to steep inclinations while intersecting the north pit wall at high angles. However, toppling failure along the west pit wall within this Domain could potentially occur due to the orientation of stratigraphy.

Based on experience at the Meadowbank Project, it is known that the geometric relationships between the dominant discontinuity types are well preserved across the project area. In general terms, the northeast and northwest trending conjugate joint sets and east-west trending cross joint sets are relatively consistent between Domains. Furthermore, the geometric relationship between the orthogonal set and the foliation/stratigraphy is also reasonably consistent; as the foliation/stratigraphy steepens, the orthogonal joint set tends to flatten and as the foliation/stratigraphy flattens, the orthogonal set steepens. Consequently, while no specific oriented data are available for this Domain, the following structural orientations have been inferred for Domain NP-3 based on the general known orientations for the conjugate and cross joint sets within Domain NP-1 and based on inferred moderate to steeply westward dipping foliation and stratigraphic contacts and associated eastward dipping orthogonal jointing.

TABLE 5.20: Domain NP-3 – Inferred Structural Orientations

Joint Set	Type	Dip	Dip Direction	Interpreted Persistence
1	Foliation and Contact Surfaces	60°	285°	Persistent
2	Orthogonal Joint Set	30°	105°	Non-persistent
3	Conjugate Joint 1	68°	222°	
	Conjugate Joint 2	71°	038°	
	Conjugate Joint 3	76°	143°	
	Conjugate Joint 4	68°	313°	
4	Cross Joints	77°	350°	
5	Bay Fault Trend and Associated Splays	70°	280°	Persistent
	Second Portage Lake Fault	70°	235°	

The above orientations are shown on Figure 5.16.

5.6 Summary of the General Stratigraphic Orientations of the Goose Island and Portage Deposits

The stratigraphic contacts and general foliation orientations for the Goose Island and Portage deposits will dip at steep angles (>60 degrees) to the west at the eastern and western margins of these deposits. They will dip at shallower angles (<30 degrees) to the west through the central portion of the Portage deposit through to the north end.

Layer parallel shearing due to tectonic activity is demonstrated in the project area, and is indicated by additional fracturing at stratigraphic boundaries. The stratigraphic contacts are considered persistent structures that will control bench scale and pit wall stability. A high degree of variability in the orientation of the stratigraphy and foliation on a local scale is expected due to the intense dome and basin type folding in the deposit areas.

6.0 CHARACTERIZATION OF MATERIALS

6.1 Lakebed Sediments

The results of laboratory testing are contained in Appendix II. Based on laboratory testing, the Specific Gravity of the lakebed sediments was 1.69 with organics present and 2.52 without. Grain size analysis test results on two samples are presented in Figure 6.1 and indicated that the lakebed sediments are primarily sand, silt, and clay sized particles.

6.1.1 Lakebed Sediments Strength

Seabed Terminal Impact Newton Gradiometer (STING) tests results for dynamic bearing strength profile of lake sediments from Golder (2007b) are presented in the following table. A 1 m long steel shaft and foot instrumented for vertical acceleration and water pressure measurements are dropped onto the lakebed. Results indicate a limited thickness of soft sediments, underlain by a harder material, typically a rocky till or bedrock.

TABLE 6.1: STING Penetrometer Testing Results

Lake	Site Name	Easting	Northing	Thickness Soil (m)	Dynamic Bearing Strength (kPa)
Second Portage	2-A	639416	7213795	0.2	<20
Second Portage	2-B	639345	7213916	0.1	20-50
Second Portage	2-C	638708	7214795	0.2	Increasing with depth to 300 kPa
Third Portage	3-A	638312	7212033	0.25	Penetration depths of 0.25 m at 300 kPa
Third Portage	3-B	638641	7213032	0.15	20 - 200
Third Portage	3-C	638799	7212833	0.23	20 - 100

6.2 Till Gradation

Descriptions of the disturbed soils that have been recovered from beneath the lakes during geotechnical drilling in proximity to the dike alignments generally are cobbles and gravel with traces of sand, silt, and clay. Locally, samples of sand have been obtained. Samples of clayey sand material have been recovered using split spoon sampling methods.

A grain size gradation showing the results of laboratory testing is shown on Figure 6.2. Laboratory test data for till gradation are contained in Appendix II. Average till consistency for 45 samples included 17% gravel, 52% sand, and 31% fines. The till had 25% silt and 6% clay sizes for 42 samples tested by hydrometer analyses. Boulders were noted in 19 of 51 boreholes.

6.2.1 Till –Density Related Properties

Values of Specific Gravity for three till samples were 2.63, 2.67, and 2.71.

Till moisture content and Plasticity Indices are presented in the following table.

TABLE 6.2: Till Moisture Content and Plasticity Indices

Borehole	Sample	Moisture Content (%)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	Reference
02GT-07	3	10.5	16.8	11.3	5.5	Golder 2002b
	5	13.1	15.9	12.7	3.2	
	6	11.1	16.9	13	3.9	
2nd Portage Trenches Area	Till Spoil 1	4.4	15	15.1	Non-plastic	Golder 2006 Lab Testing – Unreported
	Till Spoil 2	5.7				
	Till Spoil 3	5.9	17.2	16	1.2	
	Till. Grassland 4	14.8	18.8	18	0.8	
	Till Spoil 5	5.3	16.3	15.8	0.5	
	Till Spoil 6 & 7	5				
	Till Spoil 8	4.2				
	Till Spoil 9-10	7				
	Till Spoil 11	10.9	17.5	16.8	0.7	
	Till Spoil 12	8.2	19.4	17.6	1.8	
Average		8.2	17.1	15.1	2.2*	

* calculated for plastic results

6.2.2 Till Strength

Strength testing of till samples indicated shear angles shown in the following table.

TABLE 6.3: Strength Testing for Till

Borehole	Sample	Friction Angle (degrees)	Cohesion (kPa)	Test Type	Reference
02GT-07	3,5,6	32	22	Consolidated Drained Direct Shear	Golder (2002b)
Third Portage Trench	59644, 59645, 59646	37	17	Consolidated Drained Direct Shear	Golder (2002b)
Third Portage Trench	3,6,7	33	0	Undrained Triaxial Test	2006 Laboratory Testing

6.2.3 Till Hydraulic Conductivity and Consolidation Response

The following information was presented in the final report on detailed design of dewatering dikes (Golder, 2007a).

Till hydraulic conductivity testing included:

- Borehole 02GT-03 interval El. 132 to 133 m A.S.L. within bouldery till: 3x10-4 m/s
- Borehole 02GT-07 interval El. 126 to 127 m A.S.L.: 1x10-7 m/s
- Laboratory consolidation testing: 2x10-9 m/s

The laboratory hydraulic conductivity value was derived from progress of consolidation of a large diameter (152 mm diameter) sample of reconstituted fine till. However, the sample was noted to be unsaturated.

6.2.4 Ice Rich Soils

Observations in 2006 indicated that near surface soils on land flowed when thawed. The behaviour indicates near surface ice contents that are wet of the liquid limit.

Where the soils and the depth of the active layer extend below the lake level, it is assumed that the soils will be sealed, or that a cut-off trench will be installed in the soils to prevent high groundwater inflows into the pit. The stability of these soils will be largely dependent upon the groundwater pressures that will exist within the soils, which will in turn be dependent upon the method of sealing the groundwater inflows. It is likely that relatively steep slopes can be excavated in these tills, provided that seepage is adequately cut-off and that the overburden slopes are adequately drained.

6.3 Field Estimates of Intact Rock Strength - Goose Island Deposit

As part of the geotechnical drilling investigations, qualitative estimates of rock strength at the Meadowbank Project were made based on ISRM standard field identification techniques. The following tables summarize the results of the field estimates of rock strength based on the previous geotechnical data collection by Cumberland and Golder.

TABLE 6.4: Field Estimates of Intact Rock Strength – Goose Island Deposit

Rock Type	Mean ISRM Grade		ISRM Description	Range in Uniaxial Compressive Strength (MPa)
	CRL Estimate	GAL Estimate (2003)		
Iron Formation	R4-R5	R4	Strong to Very Strong	50 – 250
Intermediate Volcanic	R4-R5	R4	Strong to Very Strong	50 – 250
Ultramafic Volcanic	R2-R3	R2	Weak to Medium Strong	5.0 – 50
Volcanic Interbeds	R4-R5	No Data	Strong to Very Strong	50 – 250
Chert Pebble Conglomerate	R5 (1996)	No Data	Very Strong	100 – 250
Volcanic Dykes	R5 (1996)	No Data	Very Strong	100 – 250
Quartzite	R5 (1996)	R4	Strong to Very Strong	50 – 250
Fault Zones	R2-R3	R1	Very Weak to Medium Strong	1.0 - 50
Contact Zones	R4-R5	R4	Strong to Very Strong	50 – 250

The field estimates of intact rock strength for the various rock types at the Goose Island Deposit are considered to be consistent with estimates throughout the Meadowbank Project area. Very little variation in the rock strength estimates is noted between the various deposits. The rock is generally strong to very strong, and competent.

6.4 Field Estimates of Intact Rock Strength – Portage Deposit

The following tables summarize the results of the field estimates of rock strength based on geotechnical data collection by Golder Associates and by Cumberland Resources Ltd. for the Portage Deposit areas.

TABLE 6.5: Field Estimates of Intact Rock Strength by Golder and Cumberland – North Portage Deposit

Rock Type	Mean ISRM Grade	ISRM Description	Range in Uniaxial Compressive Strength (MPa)
Iron Formation	R5	Very Strong Rock	100 – 250
Intermediate Volcanic	R3-R4	Medium Strong Rock	25 – 100
Ultramafic Volcanic	R2	Weak Rock	5 – 25
Faulting - General	R3	Medium Strong Rock	25 – 50
Faulting - IV	R3	Medium Strong Rock	25 – 50
Faulting - UM	R2	Weak Rock	5 – 25
Faulting - Contacts	R3-R4	Medium to Strong Rock	25 - 100

TABLE 6.6: Field Estimates of Intact Rock Strength by Cumberland (1997 - 1998) - North Portage Deposit

Rock Type	Mean ISRM Grade	ISRM Description	Range in Uniaxial Compressive Strength (MPa)
Iron Formation	R4-R5	Strong to Very Strong	50 – 250
Intermediate Volcanics	R3-R4	Medium to Strong	25 – 100
Mafic Volcanics	R3	Medium Strong	25 - 50
Ultramafic Volcanics	R2-R3	Weak to Medium Strong	5.0 – 50
Contact Zones	R3-R4	Medium to Strong	25 – 100

TABLE 6.7: Field Estimates of Intact Rock Strength (1995 to 1998) – Third Portage Peninsula

Rock Type	Mean ISRM Grade	ISRM Description	Range in Uniaxial Compressive Strength (MPa)	Number of Samples
Iron Formation	R4-R5	Strong to Very Strong	50 – 250	328
Intermediate Volcanics	R3-R4	Medium to Strong	25 - 100	269
Ultramafic Volcanics	R2-R3	Weak to Medium Strong	5.0 - 50	149
Serpentinized Ultramafic	R2-R3	Weak to Medium Strong	5.0 - 50	22
Felsic Volcanics	R2-R3	Weak to Medium Strong	5.0 - 50	19
Felsic Dykes	R2-R3	Weak to Medium Strong	5.0 - 50	23
Volcanic Interbeds	R3	Medium Strong	25 - 50	14
Fault Zones	R2-R3	Weak to Medium Strong	5.0 - 50	22
Contact Zones	R3-R4	Medium to Strong	25 - 100	254

TABLE 6.8: Field Estimates of Intact Rock Strength (1996 to 1998) – Bay Zone

Rock Type	Median ISRM Grade	ISRM Description	Range in Uniaxial Compressive Strength (MPa)	Number of Samples
Iron Formation	R4-R5	Strong to Very Strong	50 – 250	217
Intermediate Volcanics	R3-R4	Medium to Strong	25 - 100	135
Ultramafic Volcanics	R2-R3	Weak to Medium Strong	5.0 - 50	134
Serpentinized Ultramafic	R2	Weak	5.0 - 25	42
Fault Zones	R2-R3	Weak to Medium Strong	5.0 – 50	37
Contact Zones	R3-R4	Medium to Strong	25 - 100	95

TABLE 6.9: Field Estimates of Intact Rock Strength (1998) – Third Portage Mainland

Rock Type	Mean ISRM Grade	ISRM Description	Range in Uniaxial Compressive Strength (MPa)	Number of Samples
Iron Formation	R4-R5	Strong to Very Strong	50 – 250	24
Intermediate Volcanics	R4-R5	Strong to Very Strong	50 – 250	86
Ultramafic Volcanics	R2-R3	Weak to Medium Strong	5.0 - 50	60
Quartzite	R5	Very Strong	100 - 250	9
Fault Zones	R2-R3	Weak to Medium Strong	5.0 - 50	15
Contact Zones	R3-R4	Medium to Strong	25 - 100	30

The field estimates of intact rock strength for the various rock types at the North Portage Deposit are considered to be consistent with estimates throughout the Meadowbank Project area. Very little variation in the rock strength estimates is noted between the various deposits. Consequently, the tables have been combined and updated with more recent drilling information. The following table summarizes the field intact rock strength estimates.

TABLE 6.10: Field Estimates of Intact Rock Strength (1995 to 2006) – Portage Deposit

Rock Type	Mean ISRM Grade	ISRM Description	Range in Uniaxial Compressive Strength (MPa)	Number of samples
UM/UMV	R2	Weak Rock	5-25	244
UMS	R2	Weak Rock	5-25	48
UMA	R2	Weak Rock	5-25	25
Iron Formation	R4-R5	Strong to Very Strong Rock	50-250	608
IV IVT IVbio	R3	Medium Strong Rock	25-50	387
IVcs IVsc IVchl	R3	Medium Strong Rock	25-50	751
Felsic Volcanics (incl. dykes)	R3	Medium Strong Rock	25-50	29
Quartzite	R4	Strong Rock	50-100	79
Volcanic Interbeds		Insufficient Data		
Faults		Insufficient Data		
FV_IF contact	R3	Medium Strong Rock	25-50	52
IF_IV Contact	R3-R4	Medium Strong to Strong Rock	25-100	95
IF_IVcs and IF_IVchl Contact	R3-R4	Medium Strong to Strong Rock	25-100	220
IF_UM Contact		Insufficient Data		
IV_UM Contact	R2-R3	Weak Rock to Medium Strong Rock	5-50	42

6.5 Unconfined Compressive Strength Testing and Density Determinations

A total of 37 rock core samples were collected from the various phases of geotechnical drilling investigations at the project site for laboratory unconfined compressive strength testing. The testing was carried out in Golder Associates Ltd. Burnaby laboratory. Photographs were taken of the core before and after testing, the results of which are contained in Appendix II and are summarized in the following table.

TABLE 6.11: Unconfined Compressive Strengths

Borehole No.	Sample	Hole Location	Unconfined Compressive Strength (MPa)	Mode of Failure*	Density (kg/m3) Wet/Dry (If single value then Dry)	Point Load Index (Dia.) (MPa)	Point Load Index (Ax.) (Mpa)
02GT-02	1	East Dike	119.3	3	2786/2784		
	2	East Dike	115.9	1	2787/2785		
02GT-04	1	East Dike	111.0	4	2680/2677		
	2	East Dike	148.3	2	2671/2669		
02GT-10	1	Stormwater	51.0	2	2811/2810		
	2	Stormwater	43.6	4	2853/2851		
GT02-VLT-2	1	Vault	86.0	2			
GT02-VLT-3	1	Vault	128.6	5			
GT02-NP-3	1	North Portage	62.2	1			
TP98-261	4	Third Portage	117.3 80.0	1 1	2775	8.6	6.3
03GT-PS-8	1	Plant Site	68.9	2	2708/2706		
03GT-GPIT-3	2	Goose Island	106.7	1	2808/2807		
03GT-PS-1	1	Plant Site	99.8	1	2711/2710		
03GT-PS-2		Plant Site	49.5	5	2721/2719		
03GT-PS-7		Plant Site	62.0	4	2712/2711		
TP98-263	1	Third Portage	64.8 67.7	1 1	2850	3.9	5.4
TP98-258	1	Third Portage	248.3	2	3117	10.2	16.7
TP98-261	1	Third Portage	137.1 138.4	2 2	3298	7.5	9.9
02GT-07	1	Stormwater	67.5	4	2668/2666		
	3	Stormwater	112.7	5	2666/2665		
02GT-08	1	Stormwater	140.1	1-5	2662/2660		
03GT-TD-1		Tailings Dike	88.4	5	2653/2652		
02GT-09	1	Stormwater	69.5	1	2647/2645		
02GT-11 (UMA)	1	Tailings Basin	67.9	4	2721/2716		
GT02-NP-2 (UMA)	2	North Portage	12.6	4		0.93 1.14	3.22
03GT-BZ-5	1	Bay Zone	9.2	4	2863/2862		
03GT-GPIT-3	1	Goose Island	40.2	1	2920/2917		
03GT-GPIT-1	1	Goose Island	21.6	4	2899/2892		
03GT-GPIT-1	2	Goose Island	33.7	4	2910/2904		

Borehole No.	Sample	Hole Location	Unconfined Compressive Strength (MPa)	Mode of Failure*	Density (kg/m3) Wet/Dry (If single value then Dry)	Point Load Index (Dia.) (MPa)	Point Load Index (Ax.) (Mpa)
TP98-261 (UMV)	3	Third Portage	91.6	1	2736	5.7	9.1
TP98-261 (UMS)	2	Third Portage	24.4 24.0	4 4	2838	1.9	3.9
TP98-258 (UMV/UMS)	2	Third Portage	30.5 27.3	4 4	2831	3.2	2.3

\+ Samples that failed along foliation have been excluded from average rock type UCS calculations
* Failure Mode: 1 – Single diagonal shear plane
2 – multi-vertical fracture
3 – vertical splitting
4 – shear along foliation or joint
5 – conical

The average compressive strength was calculated for each of the major rock types, based on the results of the laboratory testing. The following table presents a summary of the average strengths. The values presented in the table include only tests that were considered to be valid, based on the observations of the failure mode.

TABLE 6.12: Summary of Average Compressive Rock Strength

Rock Type	Minimum Unconfined Compressive Strength (MPa)	Maximum Unconfined Compressive Strength (MPa)	Average Unconfined Compressive Strength (MPa)	Equivalent ISRM Grade	Range in Uniaxial Compressive Strength (MPa)
Intermediate Volcanic	51.0	153.6	98 (94*)	R4	50 - 100
Iron Formation	137.1	248.3	175	R5	100 - 250
Quartzite	88.4	140.1	114 (107*)	R5	100 - 250
Ultramafic	67.9	91.6	80 (66*)	R4	50 - 100

*Bracket values are previously reported values, and are updated in this table to include results from modulus testing.

The pit walls in the Portage and Goose Island deposit areas will generally be excavated within the intermediate volcanic rock types. The lower portions of the Goose Island Pit will be excavated in ultramafic rock, while the upper portions will be excavated in intermediate volcanic and quartzite.

6.6 Shear Strength of Discontinuities

A direct shear testing program was undertaken in early 2007 to determine joint strength characteristics for the different rock types on the Meadowbank property. The direct shear testing was undertaken on the dominant rock types that are expected to form the majority of the pit walls. In general terms, the eastern wall of the Portage and Goose Island pits will be composed of Intermediate Volcanic (IV) rock. The south and north walls of the pit are expected to be composed primarily of IV rock, Ultramafic (UM) rock, or Iron Formation (IF) rock while the west walls are expected to be composed primarily of IV, UM, and minor amounts of Quartzite (QTZ). IV will form the greatest proportion of rock type exposed in the pit walls.

The following table summarizes the direct shear testing results for both peak and residual joint strength properties. Not all samples were tested for peak strength as the number of available samples was limited and a minimum of three unaltered samples are required to determine peak strength. The shear stress vs. normal stress plots for each rock type are shown in Figures 6.3 and 6.4.

TABLE 6.13: Direct Shear Testing Results

Rock Type	**Friction Angle**	
	Peak	Residual
IV	42	32
IVbio	-	27
IVcl	62	32
IVcs	42	33
UMA	-	27
UMS	47*	30
UMV	-	34

*the results of the 500 kpa test are questionable.

One of the direct shear tests in the UMS series resulted in a questionable value, specifically the test performed at 500 kPa. Consequently this result was excluded from the evaluation of the data.

The values determined for peak friction angle are generally as would be expected for such surfaces tested. The peak friction angle of 62 degrees for the IVcs sample, however, is higher than would be expected and so has not been considered in subsequent analyses. The values determined from residual friction angle are within a reasonable range to be expected.

6.6.1 Field Estimates of Discontinuity Shear Strength

During the 2002 and 2003 geotechnical engineering investigations, field estimates of the basic frictional strength parameters for joint surfaces in the various rock types were estimated using a simple field procedure. This was done by stacking three pieces of drill core in a core box such that a base was formed by two of the pieces, with the third piece resting on top. The core box was tilted until sliding of the top core piece occurred. The angle at which sliding occurred was recorded. This angle provides a reasonable estimate of the basic frictional characteristics of a smooth joint surface having few or no surface asperities. The following table summarizes the field estimates of the frictional strength parameters based on the sliding core test.

TABLE 6.14: Average Angle of Drill Core Sliding

Rock Type	Average Angle of Sliding	Range of Residual Friction Angles from Laboratory Testing
Iron Formation	34 degrees	No Data
Intermediate Volcanic	36 degrees	27 degrees to 33 degrees
Sericitic Intermediate Volcanic	27 degrees	
Ultramafic Volcanic	24 degrees	27 degrees to 34 degrees

A comparison of the average angle of sliding with the ranges of residual shear strength determined from laboratory testing indicates a good correlation for the Intermediate Volcanic rock type, while the laboratory residual strengths measured for the Ultramafic Volcanic rock type are higher than the average angles of sliding estimated in the field.

In addition to the measurements from drill core, joint roughness profile mapping of a limited number of joint surfaces exposed in the Third Portage trenches was carried out in 2002. The following table summarizes the field estimates of the joint roughness profile mapping at the Third Portage trenches.

TABLE 6.15: Average Small and Large Scale Joint Roughness Characteristics from Third Portage Trench Mapping

	Average Small Scale Amplitude (mm)	Average Small Scale Joint Roughness Coefficient (JRC)	Average Large Scale Amplitude (cm)	Wavelength[1](m)	Average Large Scale Joint Roughness (Barton's Jr)
Foliation Joints	3.2	11	2.4	0.3	2.5
Orthogonal Joints	4.2	15	7.0	0.2	3.0
Conjugate Joints	3.2	11	2.0	0.3	2.5
Contact	3.3	11	10.0	1	2.3

1. Wavelength estimated from charts by Hack (2001) relating large scale amplitude and profiles to estimates of wavelength.

Average first order and second order asperity angles were estimated for each of the joint roughness profiles. The asperity angles were estimated from the joint surface profiles by drawing lines to approximate the angle of the asperity and measuring the angle of this line relative to the estimated mid-line of the profile. First order asperities represent larger scale (cm to m scale) undulations or dilation angles along a joint surface, while second order asperities represent smaller scale (mm to cm scale) variations along the surface.

TABLE 6.16: First and Second Order Asperity Angles

	1^o Asperity Angle (degrees)	2^o Asperity Angle (degrees)
Foliation Joints	9	28
Orthogonal Joints	15	31
Conjugate Joints	7	30
General	7	30

At low stress levels, such as those that might be encountered at the scale of a pit bench, the surface asperities will generally remain intact and will result in a higher shear strength along the discontinuity surface. At higher stress levels, the contribution of the large scale undulations to the shear strength is difficult to assess. At higher confining stresses, such as those that would be experienced deeper in the slope, the strength along the foliation and contact surfaces will be higher due to the undulations along the surfaces.

6.6.2 Discussion of the Selection of Shear Strength Parameters for the Foliation

The average of the residual shear strength testing of the foliation is 31 degrees, as derived from the small scale laboratory testing of naturally and mechanically broken foliation surfaces within the Intermediate Volcanic rock. Peak friction angles from the testing on the order of 42 degrees were also obtained. Consequently, a friction angle some point between residual and peak strengths is reasonable to assume, as it is known that the foliation at the deposit is often irregular, uneven, and rough and large scale waviness along foliation and contacts is observed in surface trenching. Field measurements indicate large scale waviness to be on the order of 11 degrees. First order asperity angles for foliations were measured in the field and have an average angle of 9 degrees. Large scale amplitude measurements made on foliation in the field are consistent with dilation angles of about 4 degrees. Therefore, a dilation angle somewhere between 4 and 11 degrees would be reasonable to add to the residual strength to estimate the shear strengths for displacements that fall somewhere in between. Since residual strength is not mobilized until large displacements have occurred, then large scale waviness is a realistic addition to shear strength. The use of residual strengths for foliation discontinuities would only be reasonable for completely planar discontinuities or pre-sheared discontinuities where there is no additional dilation possible. Based on observations and measurements made from trench outcrops in the Third Portage area, this is not the case at the Meadowbank Project due to the intensely folded strata. Therefore, a friction angle of 37 degrees, mid-way between residual and peak for the foliation surfaces, would be reasonable and justifiable on the basis of the large scale waviness known to exist at the project. Such estimates and assumptions are consistent with ISRM suggested methods for estimating shear strength of discontinuities.

6.6.3 Joint Persistence and Rock Bridging

Typically, discontinuities of one master joint set will be more persistent than those of the minor cross joint set, so that the minor set generally terminates at the master set, or within solid rock, as illustrated on Figure 6.5. Therefore, the degree to which joint discontinuities persist within the rock mass without terminating against other joints or within the rock mass itself determines the degree to which failure of the intact rock will occur, and the mechanism by which it will occur. The persistence or lack thereof, determines whether the stability along a given structure is controlled by the strength of a persistent surface, or by a combination of non-persistent surfaces and intact rock. This also has a bearing on the potential for the development of step-path failure mechanisms. Where non-persistent discontinuities exist, there can only be movement along them if there is shearing through the intact rock itself.

At the Meadowbank project, it is recognized that the foliation and stratigraphic contacts are likely to represent persistent features, along which plane shear could potentially occur (not accounting for the dilation angle associated with the large scale waviness of the foliation surfaces). These surfaces are typically smooth and planar at the scale of the drill core. By comparison, the other joint sets in the project area are interpreted to be abutting, or non-persistent discontinuities. This interpretation is supported by data collected from drill core relating to the surface shape and planarity. Typically, the minor joint sets are characterized by rough to very rough surfaces with little to no alteration. This is suggestive of abutting or non-persistent discontinuities along which no movement has occurred.

Miller, Whyatt, and McHugh (2004) have studied the application of statistical point estimation methods to evaluate the probability distribution for various rock slope failure modes. In their evaluation of step path failure mechanisms, they suggest that when the fraction of intact rock is greater than about 8%, the probability of sliding is negligible. There are obviously other factors to consider in such a statement. However, this serves to illustrate the importance of the effective tensile strength of a rock mass in the stability of a rock slope.

Persistence is a large scale feature. Consequently, it cannot be quantitatively assessed from drill core. The persistence of joint continuities can only be estimated by taking measurements of trace lengths in rock outcrop exposures. Since there are currently no mining activities at the site and the large scale outcrop exposure is poor, it is not possible to quantify estimates of potential intact rock bridging. However, given the current understanding of the various discontinuity sets at the project site, it is reasonable to expect the main foliation to be persistent and the minor joint sets to be abutting or non-persistent and widely spaced. Consequently, it is reasonable to expect there will be a relatively high percentage of intact rock for the joint surfaces that are not co-planar with the foliation.

In order to arrive at a reasonable estimate of the percentage of intact rock that could contribute to overall stability of the slopes, a review of core samples that had been tested in uniaxial compression, and that had failed through a step-path mechanism of plane shear parallel to the foliation and through intact rock, was undertaken. Examples are shown on Figures 6.6 and 6.7. The percentage of intact rock through which failure occurs has been calculated for each sample. It is recognized that there are scale effects not accounted for in comparing the results of the laboratory specimens to a larger scale slope. However, the example serves to illustrate visually what the relative percentage of intact rock would look like. Estimates of persistence are based on engineering judgment and are necessarily conservative to account for the significant contribution of the intact rock strength to the strength of the overall failure surface. Based on the current level of understanding of the project that has been developed over 10 years, it is therefore reasonable to expect a minimum rock bridge percentage on the order of 10% and possibly higher.

6.7 Summary of Modulus Testing

A total of 8 core samples were submitted for compressive strength and modulus testing. The purpose of the testing was to determine estimates of the elastic modulus and Poisson’s ratio to be used as apart of the plant site foundation designs. There is no core available from previous drilling completed at the plant site. However, the rock at the plant site is similar to the rock in other project areas on the site from which samples are available. The rock type found at the plant site is typically intermediate volcanic. No modulus testing has been carried out on iron formation or ultramafic rock. The results of the modulus testing are contained in Appendix II, and are summarized in the following table.

TABLE 6.17: Summary of Modulus Testing Results

Borehole No.	Rock Type	Sample Depth (m)	Dry Density (kg/m3)	σ (MPa)	Secant Modulus (GPa)	Poisson's Ratio
NP02-434	IVcs	16	2724	78.52	69.0	0.36
NP02-425	IVcs	24.10	2748	96.34	64.1	0.29
NP02-432	IVchl	21.20	2781	66.25	60.7	0.25
NP96-148	IVchl	14.20	2690	60.87	22.5	0.25
NP98-309	IV	10.20	2707	52.22	48.2	0.27
TP99-345	IV	10.17	2742	106.76	55.7	0.18
TP99-338	IV	8.41	2772	102.48	67.8	0.29
TP98-265	IV	23.89	2712	153.62	75.4	0.24

6.8 Summary of Fault Gouge Testing

A sample of fault gouge material collected from drill hole TP98-261 was tested to determine its grain size gradation, Atterberg Limits, and moisture content associated with this gouge.

The following table summarizes the results of the laboratory testing. The size gradation curve for this sample is contained in Appendix II.

TABLE 6.18: Fault Gouge Grain Size Gradation

Hole Number	Sample Number	Depth From (m)	Depth To (m)	% Gravel	% Sand	% Silt	% Clay	% Moisture Content
TP98-261	Sa #5	33.32	33.54	N/D	24	71	5	8.0

Based on the results of the limits testing, the gouge material is classified as a low plasticity sandy silt.

6.9 Rock Quality and Rock Mass Rating

The strength of an overall rock mass, and consequently its response to loading, is controlled by:

- the strength of the intact rock;
- the degree of fracturing of the rock mass; and
- the nature of the fracture surfaces.

The overall strength of a rock mass is usually characterized by various rock mass classification methods on the basis of core logging data or surface mapping data.

6.9.1 Rock Mass Classification Based on Drill Core

The data collected from the on-going exploration drilling programs have been used to classify the rock quality in the project area on the basis of the Norwegian Geotechnical Institute (NGI) rock mass rating method, which is commonly used for the design of underground openings.

For the purposes of rock mass classification, Q′ is calculated assuming a Stress Reduction Factor (SRF) equal to 1 and Joint Water Reduction number (Jw) equal to one.

Goose Island Deposit

The results of the statistical analyses of the geomechanical data, and the rock quality ratings for the various rock types in the Goose Island Deposit, based on geotechnical data collection by Cumberland personnel, are summarized in the following table. The assessment of the rock mass quality of the quartzite has been based on RQD values obtained from quartzite intersected during drilling in the Third Portage Deposit.

TABLE 6.19: Summary of Rock Quality Rating (based on data collected by Cumberland) – Goose Island Deposit

Rock Type	RQD (1)		Jr (1)		Ja (1)		Jn (2)	Q′ (3)		Rating
	Mean	Med.	Mean	Med.	Mean	Med.		Mean	Med.	
Iron Formation	97	98	2.2	2.3	1.4	1.2	9	17	21	Good
Intermediate Volcanic	97	99	2.2	2.2	1.3	1.2	9	18	20	Good
Ultramafic Volcanic	86	92	2.1	2.1	2.0	1.5	12	8	11	Fair to Good
Volcanic Interbeds	96	97	2.6	2.7	1.2	1.2	12	17	18	Good
Chert Pebble Conglomerate	I/D	I/D	2.9	3.0	1.3	1.1	12	I/D	I/D	Fair
Quartzite	84**	89**	2.8	2.9	1.4	1.1	12	14	20	Good
Faulting	61	56	1.9	2.0	2.0	1.7	15	4	4	Poor
Contacts	92	97	2.3	2.3	1.4	1.2	12	13	16	Good

1. Mean and Median geomechanical values based on statistical analysis of geotechnical data collection. Jw and SRF assumed to be 1.0 for rock classification.
2. Jn has been adjusted from drill core estimates based on stereographic projections from oriented core data.
3. Q′ calculated based on mean and median geomechanical properties.
   I/D = Insufficient Data available at this time.

** Values for RQD assumed from quartzite at Third Portage.

The results of the statistical analyses of the geomechanical data, and the rock quality ratings for the various rock types in the Goose Island Deposit, based on geotechnical data collection by Golder personnel, are summarized in the following table.

TABLE 6.20: Summary of Rock Quality Rating Based on 2003 Oriented Drilling – Goose Island Deposit

Rock Type	RQD (1)		Jr (1)		Ja (1)		Jn (2)	Q′ (3)		Rating
	Mean	Med.	Mean	Med.	Mean	Med.		Mean	Med.	
Iron Formation	95	100	2.5	2.3	1.5	1.3	4	40	44	Good to Very Good
Intermediate Volcanic	93	100	2.0	1.5	1.3	1.0	4	36	38	Good
Ultramafic Volcanic	81	87	1.8	1.7	1.7	1.5	6	14	16	Good
Faulting	25	10	1.4	1.3	4.4	5.0	12	0.7	0.2	Very Poor

1. Mean and Median geomechanical values based on statistical analysis of geotechnical data collection. Jw and SRF assumed to be 1.0 for rock classification.
2. Jn based on geotechnical logging of oriented core.
3. Q' calculated from mean geomechanical values.

Based on the stereographic interpretation, four distinct joint sets (the foliation/stratigraphic orientations, the orthogonal joint set, and the northeast and northwest trending conjugate joint pairs) and two random sets (the cross joint and shallow set) are indicated. This would imply that a higher value for the joint set number, Jn, should be used. As a minimum, two joint sets are present; these are the foliation and the orthogonal joint sets. However, the wide spacing of both the conjugate sets and of the cross joint set, based on the oriented drilling, suggest that these may more appropriately be termed random sets. Therefore, a Jn in the range of 6 to 9 could potentially be expected.

Goose Island NGI Rating

Based on the results of the detailed geotechnical investigations from 1996 to 1998 and the recent oriented geotechnical drilling investigations in 2003, the major rock types that will be encountered during the underground development at the Goose Island Deposit can be classified based on their NGI ratings as follows.

Very Poor to Poor Rock

The statistical analyses of the data collected from the Goose Island project area indicate the various rock types associated with ***Faulting*** can be classified as ***Very Poor to Poor Quality rock (Q´<=4)***. Faulting in the Goose Island area is generally characterized by rubbly core and fault gouge. The faulting may be chloritic to talcose. Based on the orientations recorded on the geologic logs and on knowledge of the Meadowbank Project area, these zones are steeply dipping.

The average intact rock strength associated with the various rock types influenced by the faulting is indicated to range from very weak to medium strong (R1-R3).

Fair Rock

There is insufficient data to re-classify the ***Chert Pebble Conglomerate*** in the Goose Island area. Therefore, the previous classification as ***Fair Rock (4<=Q´<=10)*** is considered appropriate at this time. The average intact rock strength for the chert pebble conglomerate is indicated to be very strong (R5) based on the 1996 geotechnical logging. No chert pebble conglomerate was intersected in the 2003 oriented geotechnical drilling.

Fair to Good Rock

Ultramafic volcanic units are commonly altered and may contain varying contents of talc, serpentine, chlorite, and actinolite. They exhibit a lower average intact strength (R2-R3) than the other major rock types in the project area. The higher degree of alteration, and to some extent higher degree of fracturing than the other rocks, is manifested by lower RQD values in the range of 80% to 90%. These rocks are classified as ***Fair to Good Quality (4<=Q´<=40)***.

Good to Very Good Quality Rock

The remainder of the rocks encountered in the Goose Island area can be rated as Good Quality. These include the ***Iron Formation, Intermediate Volcanic, and Volcanic Interbeds. Quartzite*** has been classified as ***Good Quality Rock (10<=Q´<=100)*** based on mean and median RQD values recorded for quartzite intersected in the Third Portage area. Only minor intervals of quartzite were encountered in the oriented geotechnical drilling. No volcanic interbeds were identified in the 2003 oriented geotechnical drilling.

Portage Deposit

The results of the analyses of the geomechanical data, and the rock quality ratings for the various rock types in the Third Portage area, based on the geotechnical data collection by Cumberland Resources Ltd and by Golder Associates are summarized in the following table.

TABLE 6.21: Summary of NGI Rock Quality Rating of the Major Rock Types - Third Portage Deposit (1995 to 2002)

Rock Type	RQD (1)		Jr (1)		Ja (1)		Jn (2)		Jn (3)	Q´ (4)		Rating
	Mean	Med.	Mean	Med.	Mean	Med.	Mean	Med.	Bench Scale	Mean	Med.	
Iron Formation	91	97	2.2	2.1	1.5	1.3	5	3	9	15	17	Good
Intermediate Volcanics	87	96	2.3	2.1	1.4	1.2	6	4	9	16	19	Good
Ultramafic Volcanics	80	89	1.7	1.8	2.0	1.6	7	6	9	8	11	Fair to Good
Felsic Dykes	94	98	2.5	2.5	1.3	1.0	4	3	9	20	24	Good
Volcanic Interbeds	73	89	2.4	2.3	1.1	1.0	6	6	9	18	23	Good
Quartzite	84	89	2.2	2.1	1.5	1.5	5	6	9	14	14	Good
Faulting (General)	52	55	2.2	1.8	1.9	1.6	15	15	15	4	4	Poor to Fair
Faulting - IV	46	47	3.1	4.0	1.0	0.8	15	15	15	10	16	Good
Faulting - UM	58	60	1.5	1.4	2.3	2.1	15	15	15	3	3	Poor
Faulting - IF	51	51	1.8	1.7	1.9	1.5	14	15	15	3	4	Poor to Fair
Contacts	81	92	2.3	2.3	1.5	1.2	6	6	9	14	12	Good

1. Mean and Median geomechanical values based on statistical analysis of geotechnical data collection. Jw and SRF assumed to be 1.0 for rock classification.
2. Small scale Jn based on geotechnical logging of core.
3. Bench scale Jn based on sterographic interpretations
4. Q' calculated using statistical mean and median values for RQD, Jr, and Ja, and bench scale Jn.

The results of the statistical analyses of the geomechanical data, and the rock quality ratings for the various rock types in the North Portage Deposit area, based on geotechnical data collection by Cumberland Resources Ltd. and by Golder Associates personnel, are summarized in the following table.

TABLE 6.22: Summary of Rock Quality Rating Based on Drill Core – North Portage Deposit

Rock Type	RQD (1)		Jr (1)		Ja (1)		Jn (2)		Jn (3)	Q' (4)		Rating
	Mean	Med.	Mean	Med.	Mean	Med.	Mean	Med.	Bench Scale	Mean	Med.	
Iron Formation	93	96	2.2	1.9	1.4	1.3	4	3	9	16	16	Good
Intermediate Volcanic (IV/IVCL)	86	91	1.7	1.6	1.6	1.5	6	4	9	10	11	Good
Intermediate Volcanic (IV/IVCS)	92	97	2.3	1.5	1.3	1.3	4	3	9	18	12	Good
Ultramafic Volcanic	64	73	1.4	1.4	2.6	2.5	6	4	9	4	5	Fair
Faulting - General	62	68	1.6	1.5	2.1	2.0	15	15	15	3	3	Poor
Faulting - IV	86	94	1.6	1.4	1.6	1.1	15	15	15	6	8	Fair
Faulting – UM	40	38	1.3	1.3	2.4	2.3	14	15	15	2	1	Poor
Faulting - Contacts	58	52	1.8	1.9	2.5	2.6	15	15	15	3	3	Poor

1. Mean and Median geomechanical values based on statistical analysis of geotechnical data collection. Jw and SRF assumed to be 1.0 for rock classification.
2. Jn based on geotechnical logging of core.
3. Jn based on sterographic interpretations
4. Q' calculated using statistical mean and median values for RQD, Jr, and Ja, and bench scale Jn.

The major rock types that will be exposed in the pit walls at the Portage Deposit can therefore be classified based on their NGI ratings.

Poor Rock

The statistical analyses of the data collected from the Portage project area indicate the various rock types associated with ***Faulting*** and ***Faulted Contacts*** can be classified as ***Poor Quality Rock (Q´<=4))***. Faulting may take the form of high angle, cross cutting structures such as the Bay Fault, or bedding-parallel structures relating to flexural slip along the main stratigraphic contacts.

The average intact rock strength associated with the various rock types within the influence of the faulting is indicated to range from weak to medium strong (R2-R3). The mean and median RQD values recorded through the fault zones are generally between 50% and 60%, indicating a significant degree of rock fracturing associated with these zones. The detailed geotechnical logging suggests that where the faulting extends to surface, broad zones of rock fracturing may occur. Conversely, where the faulting is intersected at depth within the drill holes, the zones of higher rock fracturing attenuate rapidly into the hangingwall and footwall of the fault.

Fair to Good Rock

Ultramafic volcanic units are commonly altered and may contain varying contents of talc, serpentine, chlorite, and actinolite. They exhibit a lower average intact strength (R2-R3) than the other major rock types in the project area. The higher degree of alteration, and in some cases the higher degree of fracturing than the other rocks is manifested by lower RQD values in the range of 80% to 90%. These ***Ultramafic*** rocks are classified as ***Fair to Good Quality (4<=Q´<=40)***.

The review of the basic and detailed geotechnical logging suggests that the overall rock mass quality of the ultramafic rocks may be better where these rocks are not located within the fold nose region, or the fold core. Where the ultramafics are not located directly within the fold nose region, or within the fold core, the RQD is indicated to be around 85% to 90%.

A review of the 1998 detailed geotechnical logging of the controlled (oriented) drill holes by Golder indicate mean RQD values within the ultramafic rocks of 95%. Drill holes TP98-265 and TP98-258 intersected ultramafic rock sequences within the core of the fold, and adjacent to the major fault structures. Recorded RQD values from these holes are generally greater than 95% within the ultramafic rocks. Hence, where the west and east pit walls are developed in these better quality ultramafic rocks and barring any other structural influences, only minor ravelling of material is expected.

Good to Very Good Quality Rock

The remainder of the rocks encountered in the Third Portage area can be rated as ***Good Quality***. These include the ***Iron Formation, Intermediate Volcanics, Felsic Dykes, Volcanic Interbeds, and Quartzite***. In addition to the major rock types, ***Minor Stratigraphic Contacts*** within the major stratigraphic units can be classified as ***Good Quality ((10<=Q´<=100)).***

The rock quality deteriorates somewhat at the north-west corner of the property (boreholes TP95-93, TP95-94, TP95-95, TP98-312), where the top 20 m of iron formation is interbedded closely with intermediate volcanics and falls into the range of Fair to Good Rock Mass Quality. However, this may be a local phenomenon as rock quality within TP98-279 and TP98-285 is generally good within 20 metres of surface. A deterioration of rock quality is also noted adjacent to the major fault structures where these zones project to surface. This deterioration in rock quality is characterized by generally broad zones of highly fractured rock, as indicated by drill holes TP97-167 and TP98-236. This is likely attributable to the effects of surficial weathering processes (freeze/thaw) where the fault zones are exposed. Where the fault zones are intersected at depth, the areas of poorer rock quality tend to be confined to relatively discrete intervals within the hangingwall and footwall to the faults (TP98-258, TP98-261, TP98-265, TP98-280, and TP98-284).

6.10 Surface Characteristics and Spacing of Discontinuities

6.10.1 Goose Island Deposit Area

In addition to the orientation of the rock fabric and major structures relative to the orientation of the proposed pit walls, the actual spacing of the features will affect the ability for wedge and plane failure mechanisms to actually form within the walls of the pit. Deere (1963) and Bieniawski (1974) present a classification of joints based on spacing.

TABLE 6.23: Rock Mass Description Based on Joint Spacing after Deere (1963) and Bieniawski (1974)

Description	Joint Spacing	Rock Mass Description
Very Wide	>= 3 m	Solid
Wide	1 to 3 m	Massive
Moderately Close	30 cm to 1 m	Blocky/Seamy
Close	5 cm to 30 cm	Fractured
Very Close	<5 cm	Crushed

The following table summarizes the expected spacing and surface characteristics of the discontinuities at the Goose Island Deposit area based on the oriented geotechnical data collection program.

TABLE 6.24: Surface Characteristics and Spacing of the Main Discontinuity Types – Goose Island

Type	Average Joint Roughness, Jr	Avg. Joint Alteration, Ja	Jr_{ave}/Ja_{ave}	Approx. Spacing (m)	Description	Grade
Foliation	1.6	1.7	0.9	1	Moderately Close to Wide	Blocky to Massive
Orthogonal	1.8	1.4	1.3	12	Very Wide	Solid
CJ1	1.9	1.7	1.1	3	Wide to Very Wide	Massive to Solid
CJ2	2.0	1.7	1.2	3	Wide to Very Wide	Solid
CJ3	2.2	1.1	2.0	4	Very Wide	Solid
CJ4	1.7	2.2	0.8	1	Moderately Close to Wide	Blocky to Massive
Cross	1.7	1.6	1.1	4	Very Wide	Solid
Shallow sets	1.8 – 2.2	1.2 – 1.1	1.5 - 2.0	>8	Very Wide	Solid

- Description and Grade descriptions based on Deere (1963)

6.10.2 Portage Deposit Area

The mean and median surface characteristics of the main discontinuity types recorded from the non-oriented geotechnical logging during 1995, 1996, 1997, 1998, and 2002 at Third Portage are summarized in the following table.

TABLE 6.25: Summary of Discontinuity Surface Characteristics Based on Non-oriented Drilling (1995, 1996, 1997, 1998, 2002)

Discontinuity Type	Joint Roughness	Joint Alteration	Jr_{ave}/Ja_{ave}	Number
	Mean	Mean		
Contacts	2.1	1.4	1.5	124
Faulting	1.8	3.4	0.5	81
Sheared Surfaces	1.1	1.6	0.7	454
Foliation	1.6	1.6	1.0	2890
Undifferentiated Joints	2.1	1.8	1.2	9767
Conjugate Joints*	1.9	1.4	1.4	33
Orthogonal Joints	1.7	1.7	1.0	116
Veins	2.0	1.7	1.2	559

*2002 data only

The average spacing and surface characteristics of the various structural features intersected by the oriented geotechnical boreholes have been estimated and are summarized in the following table. Spacing was determined for each joint set based on the average dip of each set as determined by stereographic analysis and the total length of core that was oriented.

TABLE 6.26: Surface Characteristics and Average Spacing of the Main Discontinuities Based on Oriented Drilling – Third Portage Deposit

Type	Mean Joint Roughness, Jr	Mean Joint Alteration, Ja	Jr_{ave}/Ja_{ave}	Approx. Spacing (m)	Description*	Grade*
Contacts	1.6	1.4	1.1	5 to 20	Very Wide	Solid
Foliation	1.6	1.5	1.1	0.5 to 1.5	Moderately Close To Wide	Blocky to Massive
Orthogonal	1.8	1.5	1.2	1.4 to 6.0	Wide to Very Wide	Massive to Solid
CJ1	1.7	1.6	1.1	2.5 to 11.4	Wide to Very Wide	Massive to Solid
CJ2	1.9	1.3	1.5	2.5 to 8.3	Wide to Very Wide	Massive to Solid
CJ3	1.7	1.6	1.1	1.2 to 9.5	Wide to Very Wide	Massive to Solid
CJ4	1.8	1.3	1.4	1.4 to 7.0	Wide to Very Wide	Massive to Solid
Cross Joints	1.6	1.4	1.1	0.4 to 19.4	Moderately Close to Very Wide	Blocky to Massive

*Description and Grade based on Deere (1963) and Bieniawski (1974).

7.0 HYDROGEOLOGY

7.1 Regional Setting

The Meadowbank Gold Project is located in an area of continuous permafrost reaching depths up to 550 m thick based on extrapolation of data from thermistor cables installed at the site and on thermal modelling (Golder, 2003a). As shown schematically on Figure 7.1, in areas of continuous permafrost there are two groundwater flow regimes: a deep groundwater regime beneath the permafrost and within the taliks beneath large lakes and a shallow groundwater flow regime located in the active, seasonally thawed layer near the ground surface. Permafrost is considered to be virtually impermeable, which results in the isolation of the deep groundwater flow regime from infiltrating precipitation. Consequently, the regional groundwater flow is generally controlled by the water levels in lakes with taliks (unfrozen ground) that extend down through the deep permafrost and into the deep regional flow system.

Taliks are formed beneath lakes that do not freeze to the bottom in winter. If a lake is large enough, the talik extends down to the deep groundwater flow regime. The elevation of water levels in lakes that have these deep taliks provide the driving force (hydraulic head) for the deep groundwater flow. At the Meadowbank project, results of thermistor data and thermal modelling indicate that the taliks beneath Third Portage and Second Portage Lakes likely extend down to the regional groundwater flow regime (Golder, 2003a).

The Portage pit will be excavated both on land and in dewatered portions of Second Portage and Third Portage Lakes, while the Goose pit will be excavated entirely from a dewatered portion of Third Portage Lake. Beneath the areas of the pit that are excavated into the dewatered portions of the lakes, taliks extend down to the regional groundwater flow regime while beneath the portion of the pit excavated on land, permafrost is present.

Groundwater conditions in the areas of both the planned pits are strongly influenced by the presence of Third and Second Portage Lakes, which act as extensive constant head boundaries, while the presence of permafrost at the northern extent and in the southeast of the Portage pit will act as an impermeable boundary. The presence of permafrost in some areas of the Portage pit will likely result in lower pressures in the pit walls in the talik near the permafrost boundary while in areas where the pit is excavated in dewatered and diked areas, pressures in the pit walls will be greatest due to the influence of the lakes.

7.2 Available Data

Golder has conducted hydrogeological field investigations at the site from 1998 to the present. The results of these investigations together with data collected by others were compiled into a hydrogeological database. This database contains:

- Permeability testing at the Meadowbank Project site in 38 boreholes over intervals ranging from approximately 1 m to 130 m with an average interval length of 29 m;
- Permeability testing in three boreholes which intersect the fractured rock zone associated with Second Portage Fault;
- Permeability testing in three boreholes which intersect the Bay Zone Fault or Fault Splay; and
- Two permeability tests conducted in shallow till sediments.

Prior to the 2006 field season, the combined hydrogeological database consisted of over 80 permeability tests. With the addition of the data from the 2006 field season, the database now consists of nearly 100 tests.

7.3 Hydrogeologic Properties

7.3.1 Equivalent Hydraulic Conductivity in Competent Country Rock

The analyses of over 90 tests in the competent rock indicates that the bulk hydraulic conductivity of different rock types (Intermediate Volcanic, Iron Formation, Ultramafic Volcanic) is essentially the same at a given depth. Although there are an insufficient number of deep tests to evaluate reductions in hydraulic conductivity with depth, such a reduction in hydraulic conductivity, due to a reduction in fracture aperture as a result of increased vertical loading, is assumed. Reductions in hydraulic conductivity with depth have been observed at other locations in the Canadian Shield (Stevenson et al., 1996 a & b, Ophori et al., 1996, Ophori and Chan, 1994). The hydraulic conductivity of the bedrock over 30 m depth intervals was calculated as a geometric mean of packer tests conducted within each interval as shown in Figure 7.2.

7.3.2 Equivalent Hydraulic Conductivity of Fractured Rock Zones

There are two main fault zones in the North Portage, Third Portage, and Goose Island deposit areas – the Second Portage Lake Fault and the Bay Fault and associated Fault Splay. The Second Portage Lake Fault trends to the northwest through Second Portage Lake and the Bay Fault and Fault Splay trend in a north-south direction along the western margin of the North Portage, Third Portage and Goose Island Deposits. Although limited data are available for each of the fault zones, the available data indicate that the hydraulic conductivity of the fractured rock zone associated with the Second Portage Fault is likely of greater hydraulic conductivity than the surrounding competent rock while the Bay Zone Fault and Fault Splay are likely of similar hydraulic conductivity to the competent bedrock.

Six packer tests have been conducted in Second Portage Fault in boreholes NP02-412, 03GT-TD-06, and 06GT-TD-01 and the results of these tests indicate that this feature has higher hydraulic conductivity than the surrounding bedrock. The dominant flow pattern within this enhanced permeability feature will be parallel to this structure. Although the geometric mean of packer tests is suitable for the characterization of average hydraulic conductivity of the surrounding more competent bedrock, in this feature, the average hydraulic conductivity is better characterized with an arithmetic average of small scale measurements (Freeze and Cherry, 1979). Use of the arithmetic mean assumes that flow is parallel to continuous, higher hydraulic conductivity fractures within a matrix of lower hydraulic conductivity material. The arithmetic mean of these testing results indicates that the hydraulic conductivity of the Second Portage Fault is approximately 1×10^{-5} m/s.

Hydraulic conductivities measured in the Bay Fault and its Splay ranged from 2×10^{-9} m/s to 3×10^{-8} m/s, and were similar or lower than the ones measured in bedrock. This testing, in holes TP98-258, TP98-261, and TP98-265, suggests that the Bay Fault and its Splay may not be hydrogeologically significant. Because both of these fault zones are relatively narrow, at approximately 5 m width, it is considered unlikely that these fractured zones are well connected over large distances. Instead, it is more likely that portions of these faults would be well sealed with low hydraulic conductivity values and highly permeable portions would be of limited areal extent.

7.3.3 Equivalent Hydraulic Conductivity of Till Overburden

The till unit overlying the bedrock was characterized by two falling head tests conducted in 2002 in boreholes 02GT-03 and 02GT-07. The test in 02GT-03 was conducted in a boulder till and yielded a value of 3×10^{-4} m/s while the test conducted in 02GT-07 was conducted in a sandy clay till and yielded a much lower value of 1×10^{-7} m/s. The test in 02GT-03 was conducted from through surface sediments on the shore of Second Portage Lake while the test in 02GT-07 was conducted within the sediments underneath the lake; therefore, the test from 02GT-07 is likely more representative of the hydraulic conductivity of the till unit which will be present beneath the dikes constructed around the dewatered areas surrounding the pits.

Grain size analysis conducted on 45 samples collected from the till unit (Golder, 2007a) indicates the till is on average composed of 17% gravel, 52% sand, and 31% fines. Although equations for determining hydraulic conductivity from grain-size analysis cannot be directly applied to this material, as the proportion of silt and clay sized particles is too great, these descriptions of the proportions of material types support the conclusion that the hydraulic conductivity of the till unit is less than 1 x 10^{-5} m/s.

7.4 Conceptual Model

A conceptual model is a representation of the groundwater regime that organizes and simplifies the site hydrogeology, so that it can be modeled more readily. This conceptual model must retain enough complexity so that the numerical model developed from it adequately reproduces or simulates, to the degree required to meet the project objectives, the real groundwater behavior. Based on the data summarized above, the following presents the conceptual model of the groundwater regime at the site.

7.4.1 Hydrostratigraphy

The conceptual hydrogeologic model consists of three main hydrostratigraphic units: weakly fractured bedrock (occurring in Intermediate Volcanic, Iron Formation, Ultramafic Volcanic), fractured rock zones associated with faults, and shallow sediments. The hydraulic conductivities of each of these units, based primarily on the results of the permeability tests, are summarized in Table 7.1.

TABLE 7.1: Hydraulic Conductivities of Hydrostratigraphic Units

Hydrostratigraphic Unit	Depth Interval (m)	Horizontal Hydraulic Conductivity (m/s)
Till	0-5	1.E-05
Competent Rock	5-30	7.E-07
Competent Rock	30-60	2.E-07
Competent Rock	60-90	8.E-08
Competent Rock	90-120	4.E-08
Competent Rock	120 - 1000	3.E-08
Second Portage Fault (5 m wide)	0 - 500	1.E-05
Second Portage Fault (5 m wide)	500 - 1000	1.E-06

The relatively competent bedrock comprises the majority of the rock domain. The hydraulic conductivity/depth profile for this unit consists of five zones with the hydraulic conductivity uniform over each zone, but decreasing with depth. The hydraulic conductivity of this unit is assumed to remain constant below 120 m depth. Although the calculated geometric mean of tests over the depth interval from 60 m to 90 m was lower than the value calculated for the test interval from 90 m to 120 m (3×10^{-8} m/s for the interval from 60 m to 90 m compared to 4×10^{-8} m/s for the interval from 90 to 120 m depth), this may be the result of the limited data for the deeper portions of the rock. In the model the hydraulic conductivity was assumed to decrease with depth over these two intervals (Table 7.1).

The Bay Zone Fault and Fault Splay have been found to have virtually identical hydraulic conductivity to that of the adjacent competent rock; therefore, they are considered as part of the competent bedrock hydrostratigraphic unit. Because a high conductivity feature would cause greater drainage of the slope and lower pore pressures, this assumption for the Bay Zone Fault and Fault Splay is conservative for slope stability.

The fractured rock zone associated with the Second Portage Fault has been found to have a higher hydraulic conductivity than the surrounding less disturbed bedrock.

The hydraulic conductivity (K) of this till is assumed to be 1×10^{-5} m/s. Grain size analysis indicates that this is the upper limit for the conductivity of this unit and that the actual hydraulic conductivity of this unit is less than 1×10^{-5} m/s. This value is approximately two orders of magnitude greater than the value calculated from the single test conducted in the lakebed till sediments, but approximately 30 times lower that the single test in a boulder till conducted on the shore of Second Portage Lake. Although the test performed in the lakebed till is likely more representative of the till that will be present beneath the dikes the higher K value for the till unit assumed in the analyses is considered conservative as it will result in greater shallow flows and higher pore pressures in the underlying rock units. Low permeability cut-off walls installed underneath the dikes, will limit the impact of this unit on inflows and pore pressures.

7.4.2 Permafrost

The mine site lies within the region of continuous permafrost, which is considered to be virtually impermeable.

Two cross sections presenting the inferred permafrost extent and groundwater flow directions prior to mining were prepared previously (Golder, 2005g). Figure 7.3 presents conceptual cross sections through the Second and Third Portage Lakes. Through taliks are present beneath Second and Third Portage Lake and permafrost is present to depths in excess of 500 m. Although a thawed zone occurs in the warmer months through a thin layer near the surface, referred to as the active layer, this flow is considered to be negligible.

7.4.3 Groundwater Flow

Prior to mining of the North, Third, and Goose deposits, groundwater flow in the deep flow system is predominantly governed by the water levels in large lakes with through taliks. The Meadowbank Project is located close to the surface water divide between the Back River basin, which flows north to northwest towards the Arctic Ocean and the Thelon River which flows east to the southeast into Hudson Bay. Regional scale groundwater flow is in a northwest direction from the northwestern end of Third Portage Lake and in a southeast direction from the southeast end of Third Portage and Second Portage Lakes. On a local scale, the northwest portion of Second Portage Lake has the lowest water level in the area and consequently it forms a discharge zone (groundwater flows from high elevation water levels to lower elevation water levels) with groundwater flowing from a higher elevation lake located to the east, through the deep groundwater flow system, and then up into this portion of the lake. The southern portion of Second Portage Lake forms a recharge zone which discharges to Tehek Lake to the southeast.

During mining of the deposits, the Portage and Goose open pits act as sinks for groundwater flow. The presence of the Second and Third Portage Lakes behind the dikes acts as strong constant head boundaries for groundwater flow. Seepage faces will be present on the pit walls. Water will be induced to flow from the lakes through the lakebed sediments, beneath the low permeable core of the dikes, through the bedrock and into the open pits. Bedrock between the dikes and the pit crests will be partially de-saturated and the water table will slope steeply from the base of the dike core to the open pits. The presence of permafrost in some areas of the Portage pit will result in lower pressures in the pit walls in the talik near the permafrost boundary while in areas where the pit is excavated in dewatered and diked areas, pressures in the pit walls will be greater due to the influence of the lakes behind the dikes.

Most of the groundwater seeping into the mine will originate from the lakes, but some flow from deep bedrock is also expected. Groundwater flow will draw deep-seated brackish water up towards the mine. Consequently, the average quality of mine inflow will be a result of mixing of fresh groundwater flowing from the lakes, and brackish water flowing from deep bedrock.

7.5 Numerical Model

7.5.1 General

A site-scale numerical hydrogeologic model was developed for mining of the North Portage, Third Portage, and Goose Deposits in 2004 (Golder, 2004i) based on the conceptual model and all hydrogeologic data available at the time. This site-scale model was developed using MODFLOW. MODFLOW is a finite difference model code developed by the United States Geological Survey (McDonald and Harbaugh, 1988), to simulate transient groundwater flow in three-dimensions in a continuous porous medium (*i.e.*, continuum model). The model has been revised and refined based on the results of additional hydrogeologic testing in the 2006 field season and on changes to mine plan and dike design.

7.5.2 Model Grid

The original model grid in MODFLOW that was developed to simulate both groundwater inflow quantity and quality to the open pits (Golder, 2004i) was refined for the present analysis. This refinement includes smaller grid sizes in the pit walls to more accurately represent the open pit mines and predict pore pressures. This refined model consists of 143 columns (lateral extent of 5,300 m), 313 rows (lateral extent of 5,500 m), and 23 layers (vertical extent of 1,000 m) for a total of over 400,000 grid blocks. The location and extent of the model grid is presented in Figures 7.4 and 7.5. Uniform grid blocks that are 13 m by 13 m are used in and around the mine workings at the center of the model domain. Outside of the immediate vicinity of the mine workings, a horizontal grid spacing of 25 is used to a minimum distance of approximately 600 m from the pit crests. At greater distances, horizontal grid spacing is gradually increased up to 250 m. The two uppermost layers of the model are 7.5 m and 10 m thick, respectively. Beneath these layers, the thickness of model layers are expanded to 30 m down to a depth of 200 m below ground surface with thicker layers at depths of greater than 200 m.

7.5.3 Boundary Conditions

Three types of boundary conditions were used in the groundwater flow model: constant head boundaries, no-flow boundaries, and head dependent boundaries. The locations of these boundaries are shown in Figure 7.6.

Specified heads set to the average water level in Third Portage Lake of 134 m A.S.L. were assigned in layer 1 to represent all lakes present within the model domain. Minor lakes have closed taliks that are not hydraulically connected to the regional groundwater flow system.

Specified head boundaries were used to simulate the height of tailings throughout the mine life. The head assigned to these boundaries was set to increase as the height of tailings increased over the mine life.

A no-flow boundary was applied along the bottom of the model at a depth of 1,000 m below the ground surface. Due to the observed decrease in hydraulic conductivity with depth, flow of groundwater from greater depths is expected to be negligible. No-flow boundaries were also assigned along the bottom and edges of the permafrost; as the permafrost is considered to essentially impermeable. The thickness of permafrost (depth of the no-flow boundary) assigned to the model was based on thermistor data. Permafrost was assumed to extend vertically down from the shoreline to a depth of 500 m and beneath Goose Island to a depth of 75 m. Although thermistor data from the site suggests that the permafrost profile extends out beneath the lakes as shown in Figure 7.4, the assumption that permafrost extends vertically beneath the shoreline is considered conservative as this will result in predicted pore pressures in the pit walls being greater than the actual pore pressures. This is because the no-flow boundary representing the permafrost will be further away from the pit walls than the actual location of permafrost. No-flow boundaries were also assigned in the outermost rows and columns in all remaining layers. These outermost boundaries are located sufficiently far from the mine to minimize their influence on model predictions.

Head dependent boundaries constrained to inflow only were used to simulate the mine workings including the open pits. At the model cells that are assigned these boundaries, the flow into the boundary is proportional to the hydraulic head calculated at each individual cell and the specified elevation of the boundary assigned to that cell with the flow rate controlled by a boundary conductance. If the hydraulic head in the cell is less than the specified elevation of the boundary in the cell, the cell becomes a no-flow boundary during the simulation (i.e., the model assumes the cell is dry and groundwater flow does not occur).

Head-dependent boundaries were assigned to cells corresponding to the outline of the open pits. During simulation of the mine schedule, these boundaries were adjusted automatically to reflect the advance of the mine plan over time. The conductance assigned to grid blocks representing the open pits was based on the cross-sectional area of the grid block and the hydraulic conductivity of the surrounding rock.

7.5.4 Model Parameters

The model parameters have been updated based on the results of hydrogeologic testing conducted in the 2006 field season. The parameters used in the model are summarized in Table 7.2 and Figure 7.7. Where available parameter values were based on site specific data, otherwise they were based on published data.

TABLE 7.2: Model Parameter Values

Hydrostratigraphic Unit	Depth Interval (m)	Horizontal Hydraulic Conductivity (m/s)	Ratio of Vertical Hydraulic Conductivity to Horizontal Hydraulic Conductivity	Specific Storage (1/m)*	Specific Yield (-)*	Effective Porosity (-)*
Till	0-5	1.E-05	1:1	1.E-04	0.2	0.3
Competent Rock	5-30	7.E-07	1:1	1.E-05	0.0006	0.003
Competent Rock	30-65	2.E-07	1:1	1.E-05	0.0006	0.003
Competent Rock	65-105	8.E-08	1:1	1.E-05	0.0006	0.003
Competent Rock	105-135	4.E-08	1:1	1.E-05	0.0006	0.003
Competent Rock	135 - 1000	3.E-08	1:1	1.E-05	0.0006	0.003
Second Portage Fault (5 m wide)	10 - 500	1.E-05	1:1	4.E-05	0.002	0.01
Second Portage Fault (5 m wide)	500 - 1000	1.E-06	1:1	4.E-05	0.002	0.01
Tailings	-	5.E-07	1:1	1.E-04	0.2	0.54
Central Dike (Bituminous Liner)	0-5	1.E-09	1:1	-	-	-
East Dike (Soil-Bentonite)	0-5	1.E-09	1:1	-	-	-
Bay Zone Dike and Goose Dike - Medium Depth Portion (Cement-Soil-Bentonite)	0-5	1.E-08	1:1	-	-	-
Goose Dike Deep Portion (Jet Grout)	0-5	1.E-07	1:1	-	-	-
All Dikes Grout Curtain in Bedrock	5-30	1.E-07	1:1	-	-	-

Notes:
For competent rock based on the values by Ophori et al. 1996
Values of hydraulic conductivity assigned to competent rock units and second portage fault units are based on statistical analyses of results from packer testing.

Based on the results of additional packer testing data available from the 2006 field season (Golder, 2006b), the hydraulic conductivity of the competent rock was increased somewhat from previous values (Golder, 2004i) at all depths except the interval between 60 m and 90 m.

To simulate shallow flow underneath the dikes, a till layer was added to the model with hydraulic conductivity of 1×10^{-5} m/s and specific yield of 0.2. Tailings were represented with a hydraulic conductivity of 5×10^{-7} m/s. Although the porosity of the tailings is expected to be quite high (0.54), the pore space available for drainage is expected to be much lower due to the fine-grained nature of the tailings. Therefore, a specific yield of 0.2 was assigned based on literature values for fine grained materials (Maidment, 1992).

The properties of each of the cut-off walls beneath the dikes and the grout curtain extending into the bedrock were based on the dike designs summarized in Golder (2007a) and Golder (2007b). All cut-off walls and grout curtains were assumed to be approximately 1 m thick.

The bituminuous liner installed along the tailings dike is expected to have a very low hydraulic conductivity and seepage through the liner has been estimated based only on assumed defects in the liner (Golder, 2007b), this liner was assumed to be essentially impermeable in this investigation.

7.5.5 Model Calibration

Due to the relatively small differences in hydraulic head across the site, it is not possible to calibrate the hydrogeologic model at this time. At mining sites where permafrost is absent groundwater models are calibrated to hydraulic heads and base flows to creeks and rivers. This is not possible at Meadowbank as permafrost eliminates the driving force for groundwater flow by cutting off recharge from precipitation. If possible, models are also calibrated to long-duration pumping tests and inflows to engineered structures such as exploratory declines. The latter does not exist at Meadowbank. Furthermore, pumping tests under the current conditions at the site would not be meaningful as they would need to be conducted beneath the existing lakes, whereas during mining the portions of the lakes overlying the pits would be dewatered. Once mining commences at the site, calibration of the model to mine inflow quantity and quality and hydraulic heads in the pit walls will be possible.

7.5.6 Simulation of Mine Schedule

A simplified schedule for mining at the North, Third, and Goose Deposits was developed based on the open pit mine schedule and tailings deposition plan outlined in Golder (2005a). Although the tailings deposition plan has been updated recently (2007b), the general concept for the mine schedule and deposition plan remains unchanged. The mine plan was simulated in four intervals and the simplified schedule is as follows.

Year 1 – Dikes are constructed around the areas of the Portage and Goose pits and the area within the dikes is dewatered. The Portage Pit is begun and mined down to 115 m elevation.

Years 1 to 3 – Mining in the Portage Pit begins and the bottom of the Portage pit is at approximately 84 m elevation at the end of Year 3. Mining of the Goose Pit progresses to 45 m elevation. The elevation of tailings are assumed to increase to 127 m by the end of Year 3.

Years 3 to 5 – Mining of the Portage Pit progresses to 48 m elevation. Mining of the Goose Pit reaches the ultimate pit depth with the pit bottom at -34 m elevation. The maximum elevation of tailings is assumed to increase to 142 m by the end of Year 5. The elevation of the water table in the tailings is assumed to be 140 m.

Years 5 to 8 - Mining of the Portage Pit reaches the ultimate pit depth at -6 m elevation. Mining of the Goose Pit is complete and the former pit is used as an attenuation pond. The area within the dikes around the Goose Pit remains dewatered.

The tailings deposition sequence has been updated since the 2005 assessment. However, the general timing and elevations of the tailings and reclaim pond are similar to those presented above. The differences in elevations are not expected to result in significant changes to the model.

7.6 Model Results

The results of the modelling have been used to develop pore pressures used in the limit equilibrium and UDEC analyses. This is described in the relevant sections below.

8.0 PIT DESIGN BASIS

The following summarizes design considerations for each of the deposit areas.

8.1 Goose Island and Portage Pits

The previous stability analyses took the following into consideration.

- The Portage pit will be on the order of 120 m to 150 m deep at its southern end, based on contour pit plans for the Feasibility Study. It will be approximately 50 m deep through a 'saddle' separating the north end of the pit from the south end, where the mineralized zone rises near to surface. The pit will be on the order of 70 m deep directly adjacent to the tailings dike. At the north end, the pit will be on the order of 100 m deep. The Portage pit will be on the order of 1900 m in a north-south direction, and 400 m in an east-west direction.

- The Goose Island pit will be on the order of 130 m deep. The pit will be about 650 m in a northeast-southwest direction, and 420 m in an east-west direction.

- Bench and overall slope configurations will be controlled by structural features.

- Faulting, shearing, stratigraphic contacts and foliation are considered to be continuous. The stratigraphy and foliation are expected to be highly variable, wavy, and undulating.

- Joints are considered to have a limited continuity (less than one bench height).

- A Caterpillar 5130 (or equivalent) shovel, having a bucket capacity of about 11 m^3 and a reach of about 14.5 m will be used for loading.

- Caterpillar 777 haul trucks will be used to transport waste and ore.

- Mining of ore and waste will be on 6-m-high benches. A multiple-bench configuration within waste can likely be developed on most walls.

- Final bench heights will be on the order of 24 m. In areas of poorer quality rock, such as adjacent to fault zones, increased ravelling will be controlled by limiting bench face angles to shallower dips and by increasing catch bench widths. In certain areas, double benching (12 m high) may be required.

- Where Factors of Safety for plane and wedge instability are reported, a base friction angle of ϕ=42º has been assumed for discontinuous joint surfaces, ϕ=37º has been assumed for foliation and stratigraphic contacts, and ϕ=26º for fault surfaces.

Typically for mining projects, pit slopes having a factor of safety of 1.2 or better are acceptable. However, in the case of the Meadowbank Project, and specifically the Portage and Goose Island deposits, a higher design factor of safety is required due to the presence of the de-watering dike structures directly above the pits. The following table summarizes the minimum factors of safety for the assessment of pit slopes.

TABLE 8.1: Minimum Factors of Safety for Slope Stability

Location or scale	Minimum Factor of Safety
Bench scale	1.2
Pit Slope	1.3
Dike Toe	1.5
Pseudo-static	1.1

8.2 Rock Slope Failure Mechanisms

Rock slope design procedures are intended to establish design criteria based on the anticipated failure modes as determined by the analysis of the engineering geological model. In this context, instability in rock slopes can generally be classified according to two principal failure mechanisms, namely:

- Rock mass strength failures; and,
- Structurally controlled failures (kinematic failures).

These principal failure mechanisms are briefly discussed below.

8.2.1 Rock Mass Strength Failure Mechanisms

A rock mass failure mechanism involves the development of a failure surface through randomly jointed rock, in which the failure surface is generally quasi-circular in form. In rock that has preferentially oriented, relatively continuous structure, such as bedding, then the failure surface may take the form of linear segments with failure through randomly fractured rock between segments.

Through the laboratory strength testing and rock mass classification of the rock units at the Meadowbank Project area, it has been concluded that the rock mass strength is relatively high and thus rock mass failure is unlikely. However, overall slope stability has been assessed nonetheless.

8.2.2 Structurally Controlled Failure Mechanisms

Structurally controlled failure in rock occurs as the result of sliding along pre-existing geologic discontinuities. The three basic mechanisms of structurally controlled failure in rock slopes are plane failures, wedge failures, and toppling failures. The scale of failure depends to a large degree on the continuity, or persistence, of the discontinuities, on the large scale waviness of the discontinuities, and on the confining stresses that must be overcome in order for dilation to occur.

A plane failure may occur when a geologic discontinuity dips out of a rock slope at an angle that is shallower than the inclination of the slope. Plane failures will generally only develop to a significant extent if the strike of the geologic discontinuity is within +/-20 degrees of the strike of the rock slope. However, this strike range can be significantly increased in the presence of structures that form release surfaces along the ends of the potential planar failure.

Wedge failures may occur when two geologic discontinuities intersect to form a wedge and when the line of intersection of the wedge dips out of the slope at an inclination that is shallower than the inclination of the slope. Wedge failures will only develop to a significant extent if the azimuth of the line of intersection is within +/-40 degrees of the azimuth of the slope face.

Toppling failures may develop when a rock mass contains steeply dipping continuous geologic structures, such as fault zones, that strike nearly parallel to the strike of the face of the rock slope. Toppling failures can be sensitive to water pressures within the slope.

The orientation and persistence are the two characteristics of geologic structures that are of primary importance to structurally controlled rock slope stability. As previously discussed, structurally controlled failure will only occur where geologic structures are unfavourably oriented with respect to the orientation of the rock slope. The magnitude of a structurally controlled failure is directly related to the persistence of the structures along which sliding occurs. Geologic structures that exhibit limited persistence, such as joint discontinuities, will result in small bench scale failures that are rarely of consequence to overall slope stability and to the safety of the mining operations. Conversely, larger scale failures can occur along continuous structures, such as faulted contacts or continuous bedding planes. Therefore, it is the more continuous structures that are of primary concern.

8.3 Constraints on Pit Design

The design crest location of the Portage and Goose open pit is controlled by the location of the water retention dike. For the current study, a minimum pit crest to dike toe offset of 80 m (70 m at the southeast wall of Goose Island Pit) is currently assumed. This offset distance is particularly critical in the portions of the pits adjacent to the deepest dike cross section, and the deepest water, such as the southeast portion of the Goose Island pit wall.

8.4 In-Situ Stress

There have been no in-situ stress measurements made at the project site. It is anticipated that the maximum horizontal stress will be oriented approximately perpendicular to the trend of the ore zone, or in an approximately east-west direction. For the purposes of this study, a review of North American stress measurements tabulated by Hoek (1980) was undertaken. The ratio of horizontal to vertical stress is estimated to be about 2:1.

8.5 Design Approach

A kinematic analysis of potential bench scale planar and wedge instability was undertaken in order to develop bench design criteria. A pseudo-probabilistic approach was used, whereby the probability of wedge or plane instability is represented by a cumulative frequency distribution of failure surfaces. Typically for mining projects, a probability of failure within a range of about 20% to 30% is considered acceptable. Such analyses do not account for variations in spacing, continuity, and persistence of discontinuity features, and hence are not truly probabilistic. The results are generally considered to be conservative, because if additional information on the probability of occurrence of other geometric properties are considered, this will tend to reduce the overall probability of instability.

Once bench design criteria are developed, the overall slope stability is assessed using limit equilibrium methods in which critical joint sets are represented as angular ranges over which anisotropic strength functions are applied. Limit equilibrium methods are limited in that the joint surfaces represented in the model are considered continuous and through-going structures. The limit equilibrium models are limited in that they assume that the joint surfaces represented in the model are continuous planar structures on the order of hundreds of metres in length and so do not account for the natural continuity and termination of joint systems within a rock mass. Hence, the models do not account for the influence of an effective cohesive strength that would result if a failure surface were forced to break through intact rock.

8.5.1 Rock Bridge and Effective Cohesion

Rock bridge percentages can be used to develop scaled values of joint cohesion and friction angles based on the following formulae:

$$\textit{Scaled Cohesion} = (\%\text{Rock Bridge}) \times \text{Rock Mass Cohesion} + (1\text{-}\%\text{Rock Bridge}) \times \text{Joint Cohesion}$$

$$\textit{Scaled}\ \Phi = \tan^{-1}[(\%\text{Rock Bridge}) \times \tan(\Phi_{\text{Rock Mass}}) + (1\text{-}\%\text{Rock Bridge}) \times \tan(\Phi_{\text{Joint}})]$$

In addition, the models do not explicitly account for large scale undulations, or waviness, of features such as foliation and contacts, although such conditions are known to exist at the Meadowbank Project. In this case, these large scale effects can be accommodated in the model by increasing the joint frictional strength of these surfaces by an angle equivalent to the waviness angle of the surface. This is a reasonable approach to take provided the additional frictional strength characteristics are quantifiable. For the stability analyses, the frictional strength of the continuous foliation and stratigraphic contacts has been increased by a value of 6 degrees based on field measurements of small scale and large scale joint roughness and waviness.

Distinct element models were used as a method to confirm the factors of safety estimated by the limit equilibrium models and to estimate the potential magnitude of deformations within the pit slope and in the toe region of the water retention dike for the critical southeast section of the Goose Island pit.

Groundwater flow models provide the groundwater pressures used in the slope stability analyses. These models are run for different stages of the of pit development and should be repeated once the final pit design is completed for the Meadowbank Project.

9.0 DETERMINISTIC AND PSEUDO-PROBABILISTIC STABILITY ASSESSMENT

The Structural Domains were sub-divided into Design Sectors on the basis of wall orientations within each of the Domains. The bench face and inter-ramp angles within each of the Design Sectors were formulated to minimize significant structurally controlled failures based on deterministic and pseudo-probabilistic analyses.

Pit slope design reports have been previously provided to Cumberland for each deposit area by Golder Associates Ltd. The following lists these reports:

- Golder Associates Ltd., Report on Preliminary Review of Pit Slope Stability Considerations for the Third Portage Deposit, Meadowbank Gold Project, NWT, January 10, 1996

- Golder Associates Ltd., Report on Revised Pit Slope Stability Considerations, Third Portage, Meadowbank Gold Project, NWT, October 9, 1996

- Golder Associates Ltd., Report on Geotechnical Assessment of Underground Mining Methods, Meadowbank Gold Project, Goose Island Deposit, Northwest Territories, September 25, 1996

- Golder Associates Ltd., Report on Pre-Feasibility Geotechnical Studies, Third Portage Deposit, Meadowbank Gold Project, Northwest Territories, February 1999

- Golder Associates Ltd., "Goose Island Pit Slope Design Criteria", Technical Memorandum to Cumberland Resources, Ltd., August 21, 2003

- Golder Associates Ltd., "Third Portage Pit Slope Design Criteria", Technical Memorandum to Cumberland Resources, Ltd., December 15, 2003

- Golder Associates Ltd., "Vault Pit Slope Design Criteria", Technical Memorandum to Cumberland Resources, Ltd., January 9, 2004

- Golder Associates Ltd., "North Portage Pit Slope Design Criteria", Technical Memorandum to Cumberland Resources, Ltd., May 14, 2004

The designs presented in these reports were based on kinematic assessments of the structure within each Structural Domain.

Both deterministic and probabilistic plane and wedge stability analyses have been undertaken based on the main structural orientations in each of the Structural Domains.

A probabilistic study was then carried out using an in-house Visual Basic program that runs within Excel and uses the processed DIPS files. The program analyses the dip and dip direction of planes and the trend and plunge of wedges for various wall orientations. Factors of safety are calculated for kinematically possible plane and wedge instability based on the input parameters for friction angle, wall orientation, and kinematic window. A cumulative frequency distribution is then generated based on the probability of failure for a given bench face angle.

The following lists basic assumptions and criteria for the analyses.

- In order for failure to occur, two criteria must be met: the structure must be undercut and the structure must be dipping at greater than its friction angle, assuming dry or depressurized conditions. Hence, where "Probability of a Bench Face Failure" is indicated in the following tables, this represents the percentage probability of failure of a structure dipping greater than its friction angle AND undercut by a specific bench face angle. For example, if the probability of a bench face failure is indicated to be 5% for a bench face angle of 60 degrees and 39 structures out of a possible 39 are indicated to fail (*i.e.*, 100% failure), then the total number of failures at that bench face angle can be expected to be 5% x 100% x 39 or 2 structures. Similarly, if the probability of a bench face failure is indicated to be 100% for a bench face angle of 60 degrees, and 4 structures out of a possible 26 are indicated to fail (*i.e.*, 15% failure), then the total number of failures at that bench face angle can be expected to be 100% x 15% x 26 or four structures. Therefore, a 100% probability of failure indicates only that 100% of those structures undercut and dipping greater than their friction angle will fail.

- For the purposes of the analysis, a friction angle of ϕ=42° was assumed for non-persistent joint surfaces. This value has been based on the peak shear strength value obtained from rock shear strength testing. A friction angle of ϕ=37° has been assumed for the foliation and stratigraphic contacts. A friction angle of ϕ=26° has been assumed for fault surfaces. This has been based on the residual shear strength values from laboratory testing, with 6 degrees added to account for joint waviness and the roughness of surface asperities.

The results of the analyses have been presented previously as part of the reports and technical memoranda listed above and are contained in Appendix VI. The analyses have formed the basis of the bench design criteria for the project. The bench design criteria have been developed to minimize undercutting of potential plane or wedge failure modes.

10.0 BENCH DESIGN CRITERIA

Pit slope design reports have been previously provided to Cumberland for each deposit area by Golder Associates Ltd.. The following lists these reports:

- Golder Associates Ltd., Report on Preliminary Review of Pit Slope Stability Considerations for the Third Portage Deposit, Meadowbank Gold Project, NWT, January 10, 1996

- Golder Associates Ltd., Report on Revised Pit Slope Stability Considerations, Third Portage, Meadowbank Gold Project, NWT, October 9, 1996

- Golder Associates Ltd., Report on Geotechnical Assessment of Underground Mining Methods, Meadowbank Gold Project, Goose Island Deposit, Northwest Territories, September 25, 1996

- Golder Associates Ltd., Report on Pre-Feasibility Geotechnical Studies, Third Portage Deposit, Meadowbank Gold Project, Northwest Territories, February 1999

- Golder Associates Ltd., "Goose Island Pit Slope Design Criteria", Technical Memorandum to Cumberland Resources, Ltd., August 21, 2003

- Golder Associates Ltd., "Third Portage Pit Slope Design Criteria", Technical Memorandum to Cumberland Resources, Ltd., December 15, 2003

- Golder Associates Ltd., "Vault Pit Slope Design Criteria", Technical Memorandum to Cumberland Resources, Ltd., January 9, 2004

- Golder Associates Ltd., "North Portage Pit Slope Design Criteria", Technical Memorandum to Cumberland Resources, Ltd., May 14, 2004

Kinematic analysis and pseudo-probabilistic studies form the basis for the bench design criteria. The results of the pseudo-probabilistic analyses are presented in Appendix VI and are discussed in the following sections in the context of the slope design criteria.

10.1 On-Land Overburden Slopes

The on-land overburden at the Third Portage site consists of a silty sand and gravel till based on grain size analyses. The on-shore thicknesses will range from 0 metres to about 5 m based on casing depths. It will likely be possible to excavate slopes between 1.5H:1V and 1H:1V within these relatively competent materials, provided they are well drained.

A 10 m wide bench should be left between the crest of the pit and the toe of the overburden slopes to allow for the excavation of a drainage ditch at the toe of the slope and to retain any instability within the overburden slopes. Un-benched slope heights within the overburden materials should be restricted to a maximum of 10 m. Overburden slopes higher than 10 m should incorporate an intermediate catch-bench. A minimum catch-bench width of 3 m to 4 m can be assumed where slope heights are greater than 10 m.

10.2 Off-Shore Overburden Slopes

The thicknesses of overburden off-shore will be variable. Based on geotechnical drilling along the proposed Bay Zone dyke alignment and the Second Portage Lake east dyke alignment, overburden thicknesses are indicated to be on the order of 2 m to 5 m. Locally, thicknesses of up to 18 m to 20 m may be encountered. Based on geotechnical drilling along the proposed dyke alignments, the overburden will consist of silt, sand, and gravel till with boulders. Local sand and gravel deposits possibly of glacio-fluvial origin were encountered in some boreholes.

Where the soils and the depth of the active layer extend below the lake level, it is assumed that the soils will be sealed or that a cut-off trench will be installed in the soils to prevent high groundwater inflows into the pit. The stability of these soils will be largely dependent upon the groundwater pressures that will exist within the soils, which will in turn be dependent upon the method of sealing the groundwater inflows. It is likely that relatively steep slopes can be excavated in these tills provided that seepage is adequately cut-off and that the overburden slopes are adequately drained.

10.3 Design Sectors

The Structural Domains have been sub-divided into Design Sectors on the basis of wall orientations within each of the Domains. The bench face and inter-ramp angles within each of the Design Sectors were formulated to minimize significant structurally controlled failures based on kinematic and probabilistic analyses

10.3.1 Goose Island Design Sectors

The north, south, and west wall stability will be controlled predominantly by the orientation of discontinuous and widely spaced joint features resulting in bench scale wedge or planar failure mechanisms which are expected to be limited to localized small scale occurrences. The east wall stability will be controlled predominantly by the west dipping foliation, stratigraphic contacts, and faulting, which are interpreted to be relatively continuous and through-going structures. It has been assumed that failure along these features could involve multiple benches.

The recommended bench configurations for the respective design sectors are summarised in the following tables. Figure 10.1 shows the Design Sectors in relation to the potential location of the Goose Island Pit. A cross section of the Goose Island Deposit showing the conceptual pit slope configurations within the Design Sectors is shown on Figure 10.2.

Design Sector 1

Design Sector 1 will form the north pit wall. Within Design Sector 1, local instability will occur associated with wedges formed by the southeast and southwest dipping conjugate joint sets. These joint sets are expected to be relatively discontinuous and widely spaced and failures are therefore expected to be limited to localized occurrences.

It is expected that the majority of the rock exposed in the north pit wall will be competent iron formation. However, a thin band of poor to very poor quality ultramafic rock is expected along the western margin of this Sector where the fault/axial plane forming the boundary with Domain 4 is located. Due to the narrowness of this band of poorer quality rock, it will not be practical to adopt separate slope design criteria, such as triple-benching, for this portion of the wall. Therefore, in order to adequately transition with the multi-bench configurations of the east and west pit walls, a quadruple bench configuration with an increased catch bench within the ultramafic rock is recommended.

The following slope designs are presented for Sector 1.

TABLE 10.1: Design Sector 1 - Wall Azimuth 160° to 220°

Slope Component	Design Sector 1	
	UM	IV/IF
Vertical Bench Separation (m)	24	24
Bench Face Angle (degrees)	60	65
Catch Bench Width (m)	10	8
Inter-Ramp Angle (degrees)	45	51

Design Sector 2

Design Sector 2 will form the east pit wall. Within Design Sector 2, the pit slope design criteria are predicated on avoiding undercutting of the west dipping stratigraphy and foliation, as these discontinuities are considered to be continuous and through-going structures. Hence, bench face angles will need to be excavated parallel to the orientation of these structures. The following slope design criteria are recommended.

TABLE 10.2: Design Sector 2 - Wall Azimuth 220° to 300°

Slope Component	2A	2B	2C
	IV/IF	IV/IF	IV/IF
Vertical Bench Separation (m)	24	24	24
Bench Face Angle (degrees)	55	65	55
Catch Bench Width (m)	8	8	8
Inter-Ramp Angle (degrees)	44	51	44

Design Sector 3

Design Sector 3 will form the south pit wall. Within this Design Sector, potential wedge failure mechanisms will develop where the northeast and northwest dipping conjugate joints and cross joints intersect. The line of intersection of these planes will plunge northward out of the slope. In addition to wedge failure mechanisms, plane failure may occur along the north dipping cross joint sets. Both the conjugate joint sets and the cross joint sets are expected to be discontinuous and widely spaced. Consequently, wedge and plane failures are anticipated to be localized occurrences and relatively steep slopes can therefore be excavated for this wall.

It is expected that the majority of this wall will be excavated within good quality intermediate volcanic and iron formation rock, with a narrow band of ultramafic rock exposed at the west end of this pit wall. An increased catch bench width is recommended within the ultramafic rock as it is expected this rock will be in contact with the Domain 4 fault/axial plane boundary and hence will be of poorer quality.

TABLE 10.3: Design Sector 3 - Wall Azimuth 300º to 060º

Slope Component	Design Sector 3	
	UM	IV/IF
Vertical Bench Separation (m)	24	24
Bench Face Angle (degrees)	60	65
Catch Bench Width (m)	10	8
Inter-Ramp Angle (degrees)	45	51

Design Sector 4

Design Sector 4 will form the lower portions of the west pit wall, below the Domain 4 boundary that will exposed in the pit wall. Planar failure may occur along the east dipping orthogonal joint set where this dips at angles greater than about 42 degrees. Wedge failure may occur along the line of intersection formed by the southeast and southwest conjugate joint sets. These wedges will plunge at high angles to the east. Both the orthogonal joint sets and the conjugate joint sets are expected to be relatively widely spaced and discontinuous features and failures along these surfaces are expected to be limited to localized occurrences. However, it is expected that the general rock quality directly adjacent to the Domain 4 boundary will be poor, particularly within the ultramafic rock where this is exposed in the pit walls. Furthermore, the pit walls within this Sector will be down-slope of the ramp. Consequently, increased ravelling of material is expected through this sector and wider catch bench widths will be required.

TABLE 10.4: Design Sector 4 - Wall Azimuth 060° to 160°

Slope Component	Design Sector 4	
	UM	IV/IF
Vertical Bench Separation (m)	24	24
Bench Face Angle (degrees)	60	65
Catch Bench Width (m)	10	10
Inter-Ramp Angle (degrees)	45	49

Design Sector 5

Design Sector 5 will form the upper west pit wall, west of the Domain 4 fault/axial plane boundary through the central portion of the deposit. The Sector will be excavated primarily within ultramafic, intermediate volcanic, and quartzite rock types. Within this Sector, the foliation and stratigraphy will dip to the west at shallow to moderate angles. The orthogonal joint set will dip to the east at high angles of about 74 degrees. This set is expected to be non-persistent and of limited continuity. Consequently, relatively steep slopes can be excavated.

TABLE 10.5: Design Sector 5 - Wall Azimuth 020° to 200°

Slope Component	**Design Sector 5**
	All Rock Types
Vertical Bench Separation (m)	24
Bench Face Angle (degrees)	70
Catch Bench Width (m)	8
Inter-Ramp Angle (degrees)	55

10.3.2 Portage (Main) Pit Design Sectors

The following Design Sectors for the Portage Pit are presented:

- Design Sector 1 – Lower West Wall, East of Bay Fault, South of Approximately 0+50S
- Design Sector 2 – Lower West Wall, East of Bay Fault, North of Approximately 0+50S and South of Approximately 1+75N
- Design Sector 3 – East Wall North of Approximately 1+75N and south of the Second Portage Lake Fault
- Design Sector 4 – East Wall South of Approximately 1+75N and North of Approximately 0+00N
- Design Sector 5 – East Wall South of Approximately 0+00N
- Design Sector 6 – Southeast to South Wall East of Bay Fault

- Design Sector 7 – South to Southwest Wall, Domain TP-4A West of Bay Fault and East of Bay Fault Splay
- Design Sector 8 – West Wall West of Bay Fault splay and south of the Second Portage Lake Fault

The Design Sectors and the potential location of the Ultimate Pit wall are shown conceptually in plan on Figure 10.3. Cross sections of the Third Portage Deposit showing the conceptual pit slope configurations within the Design Sectors are shown on Figures 10.4 and 10.5.

The mining of the Third Portage Deposit will involve the development of a Starter Pit through the central portion of the Third Portage Peninsula, located where the ore zone sub-crops at surface. This Starter Pit will be fully contained on-land by the Third Portage Peninsula and will likely be less than 65 m in depth, based on information provided by Cumberland Resources Ltd. The north, south, east, and west walls of the Starter Pit will be located east of the Bay Fault, within Structural Domains TP-1, TP-2, and TP-3, and Design Sectors 1 through 5.

As mining progresses, the base of the Starter Pit will be deepened and the west pit wall will be pushed back further to the west. The Bay Fault will be exposed in the west wall of the Ultimate Pit.

Design Sectors 1 and 2 Lower West Wall East of Bay Fault

Sectors 1 and 2 are located at the south end of the proposed pit within the eastern sectors of Domains TP-3 and TP-2, respectively. The lower west wall of the Ultimate Pit will likely be excavated within these Design Sectors and will trend in an approximately north/south direction, dipping to the east. Stratigraphy will either be flat lying or will dip out of the west wall at shallow angles generally less than about 20 to 30 degrees. Directly adjacent to the Bay Fault, the stratigraphy will dip into the lower pit wall at angles up to about 60 degrees.

The results of the pseudo-probabilistic analyses, which are contained in Appendix VI, indicate that there is a low likelihood of planar and wedge failure for east facing walls within these Design Sectors. Where planar and wedge failures do occur, these are expected to be limited to local occurrences where bench faces undercut these features.

Where the west wall intersects the Bay Fault, the stratigraphy immediately adjacent to the fault zone is interpreted to be down-warped into the fault zone resulting in moderately to steeply westward dipping stratigraphy within about 15 m into the hangingwall and footwall of the fault. As described previously, the fault zone is expected to be characterized by poor quality rock. However, the fault is interpreted to be a relatively discrete structure and the quality of the rock is expected to improve substantially within a relatively short distance from the structure. The Bay Fault will intersect the west wall at favourable orientations (≅70 degrees towards 270 degrees azimuth). Where the west wall is intersected by the Bay Fault trend, increased ravelling and toppling failure may occur. The degree to which ravelling will occur will likely be controlled by the location within the west wall that the fault trend is exposed. If the fault is intersected by the wall at depths of greater than about 60 m, only limited ravelling is expected. This is based on the indication that, at depth, lower quality rock associated with the Bay Fault is limited to one to two run lengths, as discussed previously. If the fault is intersected at depths less than about 60 m, increased ravelling may occur. Where the west wall extends beyond the Bay Fault, the recommended pit slope configurations for Design Sector 7 will apply.

Based on the oriented drilling, the orthogonal joint surfaces will dip out of the west pit wall at shallow inclinations less than about 30 degrees where stratigraphy is inclined steeply to the west, and steep inclinations greater than about 70 degrees where stratigraphy is flat lying. Bench scale planar failures kinematically could occur along these surfaces with the conjugate and cross joints acting as release surfaces. Wedges formed by the intersection of the conjugate, cross joints, and the orthogonal joints will plunge to the northeast, east, and southeast at inclinations of about 40 degrees. Based on the oriented geotechnical drilling, these surfaces will be widely spaced. The surface characteristics of these discontinuities are suggestive of non-persistence. Wedge failures are therefore expected to be limited to bench scale failures, where these wedges plunge at inclinations greater than about 40 degrees and where the direction of plunge is within 45 degrees of the orientation of the pit wall face. Bench face angles have been designed to minimize the potential for undercutting of potential wedge and planar structures. Where potential wedge and planar structures are undercut, the catch benches will provide sufficient catchment to retain the ravelled material.

Based on the oriented drilling and on the interpretative cross sections, the inclination of the stratigraphy will likely be less than about 30 degrees. Where stratigraphy dips to the east at angles less than about 35 degrees, a benched configuration can be used and planar failure is not expected. The pseudo-probabilistic analyses indicate a limited likelihood of planar failure for these surfaces. If stratigraphy dips to the east at angles greater than about 35 degrees, a footwall design configuration should be adopted whereby bench faces dip parallel to stratigraphy. The footwall design philosophy is described in greater detail for Design Sectors 3, 4, and 5, below.

The lower portions of the west pit wall are expected to be excavated within the ultramafic rocks while the middle to upper pit wall are expected to be excavated within competent iron formation and intermediate volcanic rock. Within Design Sectors 1 and 2, the steeply eastward dipping orthogonal joint set will dip at inclinations of 65 to 70 degrees. It is likely that during development of the west pit wall, the wall will break back to the orientation of the orthogonal set so that bench face angles will be governed by this. Within the ultramafic rock and where the Bay Fault is exposed in the pit wall, a wider catch bench width will be necessary to accommodate the potential for increased ravelling of material from the poorer quality rock. It is conceivable that during actual development of the west pit wall in the ultramafic rock that this rock type is determined to be competent and that the catch bench width can be reduced.

The following pit slope configurations are recommended for the west pit wall within Design Sectors 1 and 2, south of about 1+75N. Separate slope designs are provided for the ultramafic rocks where these will be exposed over intervals greater than a bench height. Separate slope design criteria are presented for areas where the Bay Fault intersects the west pit wall over intervals greater than a bench height.

TABLE 10.6: Design Sector 1 and 2 Lower West Wall East of Bay Fault – Pit Slope Configurations

Sectors 1 and 2 Slope Configurations	Applicable Range in Wall Sector Azimuth 225º to 315º	
Slope Component	**Iron Fm., Volcanics**	**Ultramafics**
Bench Height	24 m	24 m
Bench Face	70 deg	65 deg
Catch Bench	8 m	10 m
Inter-Ramp	55 deg	49 deg

Design Sectors 1 and 2 – Intermediate Pit Wall

During push-back of the west pit walls, the intermediate west pit wall will likely expose the stratigraphy associated with the West regions of Structural Domains TP-2 and TP-3. Within these Domains, the stratigraphy will be inclined to the east in the middle to lower intermediate pit walls. Based on the oriented geotechnical drilling, and on the geological cross sections, stratigraphy will dip out of these intermediate walls at inclinations generally less than 20 to 30 degrees. Where the stratigraphy dips out of the intermediate walls at inclinations of less than 30 to 35 degrees, the following pit slope design criteria will apply. However, if stratigraphy dips at angles greater than 30 to 35 degrees, a footwall design approach should be applied, whereby the bench face is excavated parallel to the dip of stratigraphy. This is shown conceptually on Figure 10.6.

Where the orientation of stratigraphy is changing rapidly over short distances, it may be necessary to limit bench heights to 12 m in these areas.

TABLE 10.7: Design Sectors 1 and 2 – Intermediate Pit Wall

Sectors 1 and 2 Slope Configurations	**Applicable Range in Wall Sector Azimuth 225º to 315º**	
Dip of Faulted Contacts	**Slope Configuration**	
<30° to 35°	Unbenched Footwall Slope	Parallel to Bedding/Stratigraphy/ Sheared and Faulted Contacts
>30° to 35°	Bench Face Angle	Parallel to Bedding/Stratigraphy/ Sheared and Faulted Contacts to a maximum 70°
	Bench Height	12 metres
	Catch Bench Width	8 metres
	Inter-Ramp Angle	35° to 55° dependent on bench face

Design Sector 3 – East Pit Wall

Within this Design Sector, the east pit wall will be inclined to the west and the base of the pit will be excavated to the footwall of the ore. Maximum pit depths in this sector will likely be less than 50 m.

The oriented data obtained from drill hole TP98-312 and GT02-TP-1 indicate the axial plane foliation/banding, and consequently the fold limbs in this domain, dip westward at shallow angles less than about 25 degrees. Based on the geotechnical studies, the average orientation of the foliation and stratigraphy is inclined at around 21 degrees towards 301 degrees azimuth, while the orientation of the stratigraphic contacts are indicated to be inclined at angles of about 13 degrees towards 287 degrees azimuth.

Due to the relatively shallow inclination of the ore zone within this Design Sector, the east wall will likely be excavated along the ore contact and the wall will likely dip to the west at maximum inclinations of between 25 degrees and 35 degrees. It is anticipated that either intermediate volcanic rocks or iron formation will be exposed in the footwall.

The stability of the slopes in this sector will be controlled by the westward dipping sheared contacts, fold limbs, axial planar surfaces, and bedding planes. Therefore, an unbenched slope configuration, following the footwall of the ore zone, is recommended within this Design Sector. This is shown conceptually on Figure 10.5.

TABLE 10.8: Design Sector 3 – Pit Slope Configurations

<table>
<tr><th>Sector 3 Slope Configurations</th><th colspan="2">Applicable Range in Wall Sector Azimuth
045° to 135°</th></tr>
<tr><th>Dip of Faulted Contacts</th><th colspan="2">Slope Configuration</th></tr>
<tr><td><30° to 35°</td><td>Unbenched Footwall Slope</td><td>Parallel to Bedding/Stratigraphy/ Sheared and Faulted Contacts</td></tr>
<tr><td rowspan="4">>30° to 35°</td><td>Bench Face Angle</td><td>Parallel to Bedding/Stratigraphy/ Sheared and Faulted Contacts to a maximum 70°</td></tr>
<tr><td>Bench Height</td><td>24 metres</td></tr>
<tr><td>Catch Bench Width</td><td>8 metres</td></tr>
<tr><td>Inter-Ramp Angle</td><td>35° to 55° dependent on bench face</td></tr>
</table>

Design Sectors 4 and 5 – East Pit Wall

It is anticipated that the east pit wall through these Design Sectors will be excavated within good quality intermediate volcanic rock.

The east pit wall within Design Sector 4 will likely trend around 350 degrees azimuth, facing towards the south-west. The east pit wall within Design Sector 5 will trend toward 015 degrees azimuth, facing west. It is anticipated that intermediate volcanic rock will form most of the east pit wall within this sector. Iron Formation may also form portions of the east pit wall.

It is likely that within these Design Sectors, the upper slopes of the east pit wall (<70 to 100 m depth) will be developed within the steeply inclined (>60 degrees westward) stratigraphy east of the surface trace of the fold axial plane. These areas are shown on Figure 10.3 as Design Sectors 4A and 5A. The lower slopes at depths greater than about 70 m to 100 m will be developed within the more shallowly dipping fold limb west of the fold axial trace. These are shown on Figure 10.3 as Design Sectors 4B and 5B.

Design Sectors 4 and 5

Within Design Sectors 4 and 5, the pit slope configurations of the Starter Pit and Ultimate Pit east wall will largely be controlled by planar failure along the westward dipping sheared and faulted stratigraphic contacts. Based on the geologic cross sections, at depths greater than about 100 m, the sheared and faulted contacts will dip at inclinations greater than 35 to 40 degrees towards the west. At depths less than about 70 m, the sheared and faulted contacts will dip at inclinations greater than 60 degrees towards the west. Hence, pit slope configurations will be predicated on not undercutting these through-going structures.

Potential wedges may form where north-west dipping conjugate joint surfaces intersect south-west dipping conjugate joints. These wedges will plunge at inclinations up to about 70 degrees to the west, and therefore are not expected to be undercut by the bench face angles. Additional wedges formed by the intersection of cross joints and conjugate joints will plunge to the southwest and northwest at angles of less than 30 degrees, and therefore are expected to be stable as the plunge of the wedges is less than the frictional resistance along the plane surfaces forming the wedge.

The following pit slope configurations are recommended for Design Sectors 4 and 5 that will comprise the east pit wall south of about 2+50N, and are designed so as not to undercut the west dipping stratigraphic contacts:

TABLE 10.9: Design Sectors 4 and 5 – East Pit Slope Configurations Wall Azimuth 045° to 135°

Sectors 4 and 5 Slope Configurations	**Applicable Range of Wall Sector Azimuth 045° to 135°**	
Dip of Faulted Contacts	**Slope Configuration**	
<30° to 35°	Unbenched Footwall Slope	Parallel to Bedding/Stratigraphy/ Sheared and Faulted Contacts
>30° to 35°	Bench Face Angle	Parallel to Bedding/Stratigraphy/ Sheared and Faulted Contacts to a maximum 70°
	Bench Height	24 metres
	Catch Bench Width	8 metres
	Inter-Ramp Angle	35° to 55° dependent on bench face

Design Sector 6 – Ultimate Pit Southeast through South End Wall

The southeast to south end wall of the Ultimate Pit, east of the Bay Fault, will be oriented approximately east/west, and will face toward the north-west to north.

The current geologic interpretation of the south end of the deposit area, based on the oriented drilling and on geologic cross sections and recent geologic plan maps, is that the main north-south trend of the stratigraphy is tightly folded at the southern end of the deposit area but still trends in a north-south direction to intersect the south pit wall.

Based on the pseudo-probabilistic analyses, there is a low likelihood of planar failure where the bench face angle is less than about 65 degrees. This likelihood increases to greater than 30% for a bench face angle of 70 degrees, particularly for the southeast wall. While it is expected that the majority of these planar failures will be associated with non-persistent and widely spaced joint features, some may also be associated with a re-orientation of stratigraphy due to tight folding. Bench face angles should therefore be limited to 65 degrees through this Design Sector to minimize the undercutting of potential planar surfaces.

Potential plane failures could occur along the locally occurring north dipping foliation and stratigraphic contact surfaces where these are tightly folded around the hinge line of the north-south trending fold hinge line. Based on the oriented borehole data, the average dip of these surfaces, where present, will be less than about 26 degrees northward. Consequently, while local planar failures could potentially occur where these are inclined at angles greater than about 30 to 35 degrees, failures are unlikely for inclinations less than 30 degrees.

The intersection of the west dipping foliation and the east and northeast dipping joint sets will plunge at about 30 degrees to 50 degrees to the north. The east and northeast dipping joint sets are interpreted to be non-persistent and widely spaced. Bench face angles have been designed to minimize undercutting of potential wedges. However, where this is impractical and wedge failures occur, catch benches will retain the ravelled material.

Where the orthogonal set is steeply inclined to the west, it will form wedges with the north and northeast dipping joint sets. These wedges will plunge towards the northwest at angles greater than about 60 degrees. Bench face angles have been designed to minimize undercutting of these wedges, and catch benches designed to retain any ravelled material.

Wedges formed by the intersection of the north to north-west dipping conjugate joint surfaces and the east dipping orthogonal joint surfaces will plunge at about 60 degrees towards the northeast. As these joint sets are interpreted to be non-persistent and widely spaced, ravelling will likely be minimal.

Wedges formed by the steeply east dipping orthogonal joint set and north dipping joint sets will plunge at approximately 60 degrees towards the northeast. These wedges are formed by widely spaced and discontinuous joint sets. Failures will therefore be limited to local occurrences.

Where the south end wall intersects the Bay Fault trend, wedges will potentially form along the intersection of the fault zones with north-west and north dipping conjugate joint surfaces. These wedges will plunge at around 60 degrees towards the north-west. As the Bay Fault trend is considered a continuous feature, these wedges may result in increased ravelling where exposed in the south pit wall.

The following slope configurations are recommended for the southeast through to south pit wall. Separate slope designs are provided for the ultramafic rocks if exposed in the wall over significant thicknesses (greater than a bench height). An increased catch bench width is suggested to accommodate the potential for increased ravelling of material from the poorer quality ultramafic rock.

TABLE 10.10: Design Sector 6 South End Wall – Pit Slope Configurations

Sector 6 Slope Configurations	**Applicable Range of Wall Sector Azimuths 135º to 185º**	
Slope Component	**Iron Fm., Volcanics**	**Ultramafics**
Bench Height	24 m	24 m
Bench Face	65 deg	65 deg
Catch Bench	8 m	10 m
Inter-Ramp	51 deg	49 deg

<u>Design Sector 7 – Ultimate Pit South to Southwest End Wall – Structural Domain TP-4A West of Bay Fault</u>

The south to southwest end wall of the Ultimate Pit will be oriented approximately east/west and will face toward the north to northeast.

Design Sector 7 lies within Domain TP-4A, west of the Bay Fault and east of the Bay Fault splay. Within this Design Sector, stratigraphy is expected to strike into the south pit wall at high angles. Near the Bay Fault and its splay, the foliation and stratigraphy will dip at steep angles (about 60 to 70 degrees) westward and strike in a north-south direction. Further to the west of the Bay Fault, the dip of the foliation and stratigraphy becomes more gently inclined to the west and northwest at angles less than about 40 degrees to 50 degrees and as low as 20 degrees to the west of the Bay Fault splay.

The results of pseudo-probabilistic analyses, contained in Appendix VI, indicate that there is a low likelihood of planar failure where the bench face angle is less than about 65 degrees. While it is expected that the majority of these planar failures will be associated with non-persistent and widely spaced joint features, some may also be associated with a re-orientation of stratigraphy due to tight folding. Based on the analysis bench face angles should be limited to 65 degrees through this Design Sector.

Design criteria for the south end wall within Domain TP-4 will be controlled by northeast dipping joint sets. These joint sets dip at average angles of about 44 degrees to the northeast. However, based on the oriented drilling these will be widely to very widely spaced joints and non-persistent. Hence, while local failures may occur, large scale bench failures are not anticipated.

The design criteria for the southwest end wall will also be controlled by wedge failure mechanisms formed by the intersection of southeast dipping joints and north dipping joints. These wedges plunge at angles of about 55 degrees towards the northeast. However, the factor of safety associated with this wedge is about 2 and hence wedge failure is not expected.

The ultramafic rocks may form a significant portion of the Ultimate Pit south end wall. In addition to this, the Bay Fault and splay will intersect the south pit wall at a high angle to the wall. It is expected that the overall rock quality of the Bay Fault will be poorer than the surrounding rock. Consequently, there is a risk of ‘gullying’ developing within the Bay Fault and splay where these are exposed in the pit wall. The potential for ‘gullying’ could be compounded as a result of seepage along these structures which trend southward and will intersect the Bay Zone Dyke. Furthermore, the potential for on-going ravelling of material due to exposure to weather and freeze-thaw processes could result in erosion of this feature within the pit wall. As a consequence, it may be necessary to establish wider berms in this area to control ravelling. There is also the potential for glaciation to occur in association with this feature if it begins acting as a conduit for seepage. This is a common occurrence in open pit mines in the Arctic. It is anticipated that mitigation of erosion and/or glaciation of this feature can be accomplished as required on an operational basis.

The following slope configurations are recommended for the south through to southwest pit walls. Separate slope designs are provided for the ultramafic rocks, if exposed in the west wall over significant thicknesses (greater than a bench height). An increased catch bench width is suggested to accommodate the potential for increased ravelling of material from the poorer quality ultramafic rock.

TABLE 10.11: Design Sector 7 South End Wall – Pit Slope Configurations

Sector 7 Slope Configurations	**Applicable Range of Wall Sector Azimuths 185º to 225º**	
Slope Component	**Iron Fm., Volcanics**	**Ultramafics and Bay Fault and Splay**
Bench Height	24 m	24 m
Bench Face	65 deg	60 deg
Catch Bench	10 m	10 m
Inter-Ramp	49 deg	45 deg

<u>Design Sector 8 – Ultimate Pit West Wall – Wall Sector Azimuth 225º to 315º</u>

Design Sector 8 will comprise the west pit wall of the Ultimate Pit, east of the Bay Fault. The Sector has been sub-divided into sub-sectors 8A and 8B. Sub-sector 8A consists of that area of the west pit wall between the Bay Fault, and its splay, within Structural Domain TP-4A. Sub-sector 8B consists of that area of the west pit wall west of the Bay Fault splay within Structural Domain TP-4. The pit slope design configurations for this wall are based on the geological cross sections and on the detailed oriented drilling of holes TP98-258, TP98-261, TP98-265, GT-TP02-02 for sections of these holes located west of the Bay Fault and 03GT-BZ-4 located along the east shoreline of the Bay Zone bay.

The foliation, contacts, and faults will dip into the west wall at angles ranging from about 70 degrees down to about 20 degrees with increasing distance from the Bay Fault and its splay. Hence, planar failure along these through-going and continuous structures is not expected. However, toppling failures could potentially occur. Based on the oriented drilling and on the geological cross sections, the spacing of the main stratigraphic contacts is on the order of 10 m to 20 m while the average spacing of the orthogonal joint set is on the order of 6 m. Consequently, the ratio of the base to the height of the blocks formed by these structures is such that the blocks will be stable or that stability will be governed by sliding along the east dipping orthogonal joint set. Increased ravelling of material can therefore be expected.

The orthogonal joints are expected to be inclined at low angles (<30 degrees) towards the east directly adjacent to the Bay Fault and its splay and at steeper angles further west of these faults. Where these joints are inclined at greater than about 40 degrees and where the dip direction of these joints is within about 20 degrees of the dip direction of the wall, small scale planar failures are anticipated where these are undercut by bench faces.

Conjugate joints are expected to dip at angles greater than about 70 degrees towards the southeast and about 45 degrees to the northeast. Cross joints are expected to be inclined at moderate angles (45 degrees) to the north and south, and at high angles (>70 degrees) to the north. Wedges formed by the intersection of the southeast dipping conjugate joints and the north dipping cross joints will plunge at about 48 degrees towards the east, but will have a factor of safety greater than 2.

Sub-sector 8A will form the lower to middle regions of the west pit wall, down-slope of the Bay Fault splay. Consequently, increased ravelling of material could occur from the fault zone exposed in the slope.

The following slope configurations are recommended for the east facing Ultimate Pit west wall within this Design Sector. Separate slope designs are provided for the ultramafic rocks, if exposed in the west wall over significant thicknesses (greater than a bench height). Within the ultramafic rock, an increased catch bench width is suggested to accommodate the potential for increased ravelling of material from the poorer quality rock. It is conceivable that during actual development of the west pit wall in the ultramafic rock that this rock type is determined to be competent and that the catch bench width can be reduced.

TABLE 10.12: Design Sector 8 West Wall Including Bay Fault and Splay

Sector 8 Slope Configurations	**Applicable Range of Wall Sector Azimuths 225º to 315º**				
	Sector 8A Middle to Lower Slopes		**Bay Fault Zone and Splay**	**Sector 8B Upper Slopes**	
Slope Component	**Iron Fm., Volcanics**	**Ultramafics**		**Iron Fm., Volcanics**	**Ultramafics**
Bench Height	24 m	24 m	24 m	24 m	24 m
Bench Face	70 deg	65 deg	60 deg	70 deg	70 deg
Catch Bench	10 m	10 m	10 m	8 m	10 m
Inter-Ramp	49 deg	49 deg	45 deg	55 deg	52 deg

10.3.3 North Portage Design Sectors

The Design Sectors north of the Second Portage Fault are as follows:

- Design Sector 9 – West Wall – Domains NP-1, NP-2, and NP-3
- Design Sector 10 – Northwest and North Wall – Domains NP-1, and NP-3
- Design Sector 11 – North Wall – Domain NP-2
- Design Sector 12 – North and Northeast Wall – Domain NP-2
- Design Sector 13 – East Wall – Domain NP-2

The north and west wall design criteria will be controlled predominantly by the orientation of discontinuous and widely spaced joint features resulting in bench scale wedge or planar failure mechanisms which are expected to be limited to localized small scale occurrences. The east wall design criteria will be controlled predominantly by the west dipping foliation, stratigraphic contacts, and faulting, which are interpreted to be relatively continuous and through-going structures. It has been assumed that failure along these features could involve multiple benches where these surfaces are inclined at angles greater than about 30 to 35 degrees, assuming base friction angles and asperity angles as described previously.

West Wall Design Sectors

Pit slope design criteria are presented below for Design Sectors for the west pit wall, which will expose all three Structural Domains. The wall will strike north/south (Wall Azimuth 240° through 310°, approximately) and will face to the east.

Sector 9A – Domain NP-1 Upper Elevation West Wall - Sector Azimuth 240° to 310°

The upper elevation west pit wall lies within Structural Domain 9. Based on the interpretative geological cross sections and on geological plan maps, the upper west pit wall within this Design Sector will expose competent quartzite and intermediate volcanic rock. The crest of the pit wall in this area is between approximately 70 and 120 m east of the toe of the tailings dike.

It is anticipated that the foliation, and hence the major stratigraphic contacts, will dip into this wall at shallow to moderate angles. Consequently, large scale multi-bench failures are not anticipated.

Within this Design Sector minor wedges formed by the intersection of the northeast and southeast dipping conjugate sets will plunge at about 64 degrees towards 083 degrees azimuth. These will have a factor of safety of less than 1. As the joint sets forming these wedges are expected to be widely spaced and non-persistent, wedge failures are expected to be limited to localized bench scale occurrences.

Wedges formed by the intersection of the north dipping cross joint and southeast dipping conjugate set will plunge at about 44 degrees towards 067 degrees azimuth and will have a factor of safety greater than about 2.

Planar failure could potentially occur along the steeply inclined orthogonal joint set where it dips to the east. Based on the oriented geotechnical drilling, the orthogonal joint set will be inclined steeply either to the west, or to the east, depending on the orientation of the foliation and stratigraphy. Where the orthogonal set dips to the east, it is likely that bench faces will break back to the inclination of the orthogonal joint set.

While the rock exposed in the upper west wall is expected to be competent, the geotechnical studies to date indicate that the bedding parallel flexural shearing is more prevalent in the North Portage Deposit area than in the Third Portage Deposit area. As a result, increased ravelling of material may occur where the main contacts are exposed in the pit walls. Therefore, catch bench widths of 10 m are recommended to accommodate the increased ravelling that is expected.

Sector 9B – Domain NP-3 Mid-Elevation West Wall - Sector Azimuth 240° to 310°

The middle elevation west pit wall lies within Structural Domain NP-3. Based on the interpretative geological cross sections and on geological plan maps, the upper west pit wall within this Design Sector will expose primarily quartzite and intermediate volcanic rock and possibly some ultramafic rock. It is expected that the northern extension of the Bay Fault trend will be exposed within this Sector.

It is anticipated that Domain NP-3 and the Bay Fault trend will be exposed within this pit wall. It is expected that the rock quality immediately in contact with the fault and adjacent footwall and hangingwall zones will be of poorer quality than the general rock mass quality within Domains NP-1 and NP-2. Hence, ravelling of material could be more extensive than in other areas. Within Domain NP-3, stratigraphy and faulting will dip into the west wall at steep angles of 60 to 70 degrees. Consequently, large scale multi-bench failures are not anticipated. However, toppling failure may occur, particularly directly adjacent to the fault zone. Where the fault zone is exposed in the pit wall, double benching with alternating catch bench widths of 5 m and 8 m are recommended.

Sector 9C – Domain NP-2 Lower Elevation West Wall - Sector Azimuth 240° to 310°

The lower elevation west pit wall lies within Structural Domain NP-2, east of the Bay Fault trend. Based on the interpretative geological cross sections, predominantly intermediate volcanic rock, and quartzite, and variable thicknesses of ultramafic rock and iron formation will be exposed in these walls.

Through this Sector, pit slope design criteria will be controlled primarily by wedge failure mechanisms along the intersection formed by the northeast and southeast dipping conjugate joint sets, which will plunge at about 63 degrees towards 076 degrees azimuth, and by the east dipping orthogonal joint set and the southwest dipping conjugate set which will plunge at about 59 degrees towards 151 degrees azimuth. Both wedge failures mechanisms have factors of safety less than 1. However, the joint sets forming these wedges are expected to be widely space and non-persistent based on the evaluation of their surface characteristics. Consequently, wedge failures resulting from the intersection of these joints is expected to be limited to local occurrences.

The upper portions of the wall within this Sector will run parallel to the Bay Fault Zone, which will dip into this wall at angles of 60 or 70 degrees. Consequently, increased ravelling and toppling may occur down-slope of this fault zone. However, based on borehole data, the influence of faulting on the rock mass quality of the adjacent rock attenuates rapidly with distance from the fault so that fault zones are characterized by discrete intervals of poorer quality rock rather than broad zones of fracturing. Therefore, while poorer quality rock may be encountered immediately adjacent to the fault zone, it is also expected that better quality rock will be encountered rapidly. Within the fault zone, benching should be restricted to double benches (12 m) with alternating 5 m and 8 m catch bench widths.

Separate slope design criteria are provided for the ultramafic rock which may be exposed in the lower portions of the wall, where these are exposed over a bench height. It is anticipated that increased ravelling of material will occur due to the poorer quality of these rocks.

West Wall Design Sector – Recommended Slope Configurations

The recommended slope configurations for West Wall Design Sector are summarized below in the following table.

TABLE 10.13: West Wall Design Sector – Pit Slope Configurations

West Wall Slope Configurations	**Wall Sector**				
	9A	**9B**		**9C**	
Slope Component	**Upper West Wall 240° to 310°**	**Middle West Wall 240° to 310°**		**Lower West Wall 240° to 290°**	
		Sound Rock	**Bay Fault Zone**	**IF, IV, QTZ**	**UM**
Bench Height	24 m	24 m	12 m	24 m	24 m
Bench Face	70 deg	70 deg	65 deg	70 deg	65 deg
Catch Bench	10 m	10 m	5m/8m	10 m	10 m
Inter-Ramp	52 deg	52 deg	45 deg	52 deg	49 deg

Middle to Upper Elevation North Wall Design Sectors

Pit slope design criteria are presented below for Design Sectors for the middle to upper elevation north pit wall. The middle to upper north pit wall will expose Domain NP-1 and Domain NP-3 which will trend generally into the wall face. The wall will strike east/west (wall azimuth approximately 310° through 050°) and will face to the southeast through southwest.

Sector 10A – Domain NP-1 Upper Elevation Northwest Wall - Sector Azimuth 310° to 345°

Based on the interpretative geological cross sections, the upper elevation northwest end wall within this Design Sector will expose primarily quartzite and intermediate volcanic rock. Minor occurrences of ultramafic rock and iron formation may be exposed in the wall.

Based on the oriented geotechnical data collection and geological interpretation there appear not to be significant structural controls on pit slope stability through this wall sector. The southeast dipping conjugate joint set dips at about 76 degrees towards 143 degrees azimuth, and hence will not be undercut by bench face angles.

Sector 10B – Domain NP-3 Middle Elevation North Wall - Sector Azimuth 310° to 345°

Through this Design Sector, the Bay Fault trend and associated stratigraphic orientations will trend roughly parallel to sub-parallel to the trend of the wall. Stratigraphy and high-angle faults will dip into the wall at steep angles. It is expected that intermediate volcanic rock and ultramafic rock will be exposed in this Sector.

Increased raveling of material can be expected due to the poorer quality of rock associated with faulting. Consequently, double benching with alternating catch benches of 5 m and 8 m may be required to reduce the bench height over which ravelling occurs.

Sector 10C – Domain NP-1 Upper Elevation North Wall - Sector Azimuth 345° to 050°

Within this wall sector, bench scale slope design criteria will be controlled by potential wedge failure and planar failure mechanisms. Wedges formed by the intersection of the southwest dipping conjugate set and the steeply westward dipping orthogonal set will plunge at about 68 degrees towards 214 degrees azimuth, with a factor of safety of less than 1. Wedges formed by the intersection of the southwest and southeast dipping conjugate sets will plunge at about 66 degrees towards 199 degrees azimuth and have a factor of safety of less than 1. Wedges formed by the intersection of the west to southwest dipping orthogonal joint set and the southeast dipping conjugate joint plunge at about 63 degrees towards 204 degrees azimuth and have a factor of safety of less than 1. The joint sets forming these wedges are expected to be widely spaced and non-persistent based on the oriented geotechnical drilling and geotechnical mapping carried out to date. Hence, wedge failures are expected to be limited to small scale local occurrences.

Wedges may also form by the intersection of the southeast dipping conjugate set and the west dipping Bay Zone Fault trend, where present. These will plunge at about 55 degrees towards 212 degrees azimuth. The factor of safety associated with these wedges is less than 1. However, since the Bay Zone Fault is interpreted to be a relatively narrow, discrete feature, and the conjugate set is interpreted to be non-persistent, it is expected that any failures will be limited in extent and will be confined to a narrow portion of wall where the fault may be exposed.

Sector 10C forms a small segment of the pit slope up-slope of Sector 10D and the intersection of the Bay Fault Zone with the north pit wall. While it is anticipated that this segment of the slope will be stable at steeper bench face angles, in practice it will be difficult to apply separate slope design criteria to Sector 10C and to Sector 10D. Consequently, the proposed slope design criteria for Sector 10D should be adopted for this Design Sector.

Sector 10D – Domain NP-3 Middle and Upper Elevation North Wall - Sector Azimuth 345° to 050°

This Design Sector is characterized by the Bay Fault trend and associated stratigraphic orientations of Domain NP-3 which will trend into the north end wall at a high angle to the wall. It is expected that the overall rock quality Bay Fault trend will be poorer than the surrounding rock and there is a risk of 'gullying' developing within the Bay Fault trend where the fault intersects the north end wall. On-going ravelling of material due to exposure to weather and freeze-thaw processes may result in erosion of this feature within the pit wall. As a consequence, it may be necessary to establish wider berms in this area to control ravelling and to limit bench face angles to 65 degrees. However, it is anticipated that mitigation of erosion of this feature can be accomplished as required on an operational basis.

Middle to Upper Elevation North Wall Design Sectors – Recommended Slope Configurations

The recommended slope configurations for the North Wall Design Sectors are summarized below in the following table.

TABLE 10.14: Middle to Upper North Wall Design Sectors – Pit Slope Configurations

Middle to Upper North Wall Slope Configurations	Wall Sector 310° to 050°			
	10A	**10B**	**10C**	**10D**
Slope Component	**Upper Northwest Wall**	**Middle Northwest Wall (Bay Fault Intersection)**	**Upper North Wall**	**Mid to Upper North and Northeast Wall**
	310° to 345°	310° to 345°	345° to 050°	345° to 050°
Bench Height	24 m	12 m	24 metres	24 metres
Bench Face	70 deg	65 deg	70 degrees	70 degrees
Catch Bench	10 m	5 m/8 m	12 metres	12 metres
Inter-Ramp	52 deg	45 deg	49 degrees	49 degrees

Sector 11 Lower Elevation North Wall Design Sectors

Pit slope design criteria are presented below for Design Sectors for the lower elevation north pit wall. The lower north pit wall will expose Domain NP-2 which will trend generally into the wall face. The wall will strike east/west (wall azimuth approximately 310° through 050°) and will face to the southeast through southwest.

Northeast through Northwest Lower Elevation Walls - Sector Azimuth 290° to 050°

The lower north wall design sectors will apply to pit walls facing generally towards the southeast through to southwest (wall sector azimuth 290° to 050°) and within Domain NP-2, to the east of Domain NP-3. Based on the geological cross sections, predominantly intermediate volcanic rock and quartzite will be exposed in these walls. Some iron formation and ultramafic rock may be exposed.

Through this Sector, pit slope design criteria will be controlled primarily by wedge failure mechanisms along the intersection formed by the east dipping orthogonal set and the southwest dipping conjugate set, which will plunge at about 59 degrees towards 151 degrees azimuth. The factor of safety associated with this wedge is less than 1. However, the orthogonal and conjugate joints are considered to be non-persistent and widely spaced, and wedge failures resulting from the intersection of these two discontinuity surfaces are expected to be limited to local bench scale failures.

Potential wedge failures may form along the line of intersection formed by the east dipping orthogonal joint set and the southwest dipping Second Portage Lake Fault trend. Where these two discontinuities intersect, wedges will plunge at about 43 degrees towards 165 degrees azimuth and will have a factor of safety greater than 1.5 so that failures are not anticipated. Wedge failures may also occur where faulting parallel to the Second Portage Lake Fault trend is intersected by the southwest dipping conjugate joint set. These wedges will plunge at about 59 degrees towards 182 degrees azimuth. Both the orthogonal joint set and the conjugate joint set are expected to be non-persistent and widely spaced, based on the oriented geotechnical drilling and geotechnical mapping carried out to date, while the faulting parallel to the Second Portage Lake Fault trend is expected to be through-going and continuous. Wedge failures are therefore expected to be restricted to local occurrences and only where the parallel fault trends are present.

Lower Elevation North Wall Design Sectors – Recommended Slope Configurations

The recommended slope configurations for the lower north wall Design Sectors are summarized below in the following table.

TABLE 10.15: Lower North Wall Design Sectors – Pit Slope Configurations

Lower North Wall Slope Configurations	**Wall Sector 11 290° to 050°**
Slope Component	**Northeast to Northwest Wall**
Bench Height	24 m
Bench Face	70 deg
Catch Bench	10 m
Inter-Ramp	52 deg

East Wall Design Sector – Wall Sector Azimuth 050° to 140°

The foliation and major stratigraphic contacts within this Design Sector will be inclined to the west at less than about 20 degrees based on the current geologic interpretation, and on the oriented geotechnical drilling. These contacts will strike approximately north/south.

Due to the orientation of the faulted stratigraphic contacts (less than about 20 degrees westward), the approach to mining of the mineralization within this Sector will result in a non-benched configuration with mining to the footwall contact of the mineralization.

The frictional strength of the foliation and contact surfaces is expected to be on the order of about 37 degrees when waviness angles are considered. Therefore, where these surfaces exceed about 30 degrees to 35 degrees, the bench face angles will be predicated on not undercutting these contacts where the strike of the pit wall is within 20 to 30 degrees of the strike of the contacts. In this circumstance, bench face angles will be excavated parallel to the dip of the stratigraphic contacts to a maximum of 70 degrees. This will ensure that the inter-ramp angles will not undercut the contacts.

The recommended design bench configurations are summarized in the following table.

TABLE 10.16: East Wall Design Sector – Pit Slope Configuration

Dip of Faulted Contacts	Slope Configuration	
<30° to 35°	Unbenched Slope	Parallel to Bedding/Stratigraphy/ Faulted Contacts
>30° to 35°	Bench Face Angle:	Parallel to Bedding/Stratigraphy/ Faulted Contacts to a maximum 70°
	Bench Height:	24 metres
	Catch Bench Width:	10 metres
	Inter-Ramp Angle:	35° to 52° dependent on bench face

10.3.4 Second Portage Lake Fault Structural Zone

The Second Portage Lake Fault Zone trends in a northwest to southeast direction and will intersect east facing and west facing walls at the southern end of the North Portage Pit. The fault forms a Domain Boundary separating the North Portage Pit from the Third Portage Pit. The fault zone width is expected to be narrow on the order of 5 to 10 m. However, it could potentially be up to 40 m to 50 m in width near surface based on the spacing of boreholes bounding the fault zone.

The fault is interpreted as dipping at vertically to sub-vertically at high angles to the southwest and will intersect the east and west pit walls at relatively high angles. Consequently, the orientation of the fault is favourable for the stability of the east and west pit walls. However, the rock quality within the fault zone will be poor and considerable ravelling of material and potential gullying may therefore occur where the fault intersects the pit walls. It is recommended that the bench heights for the west pit wall within this Sector be limited to double bench heights (12 m) through this structural zone, with alternating 5 m and 8 m catch bench widths, and that bench face angles be limited to 65 degrees to minimize ravelling of material. During mining, if it is determined that the fault zone, where exposed, behaves favorably in terms of stability and is of limited lateral extent, then it is conceivable that full 24 m high final benches would be achievable with a 10 m catch bench.

The recommended bench configurations for the pit slopes within the Second Portage Lake Fault area are summarized in the following tables.

TABLE 10.17: Second Portage Lake Fault Zone – West Pit Wall Slope Configurations

Component	Wall Sector Azimuth 250° to 320°
Bench Height	12 m
Bench Face	65 deg
Catch Bench	Alternating 5 m/8 m
Inter-Ramp	45 deg

TABLE 10.18: Second Portage Lake Fault Zone – East Pit Wall Slope Configuration

Dip of Faulted Contacts	Slope Configuration	
<30° to 35°	Unbenched Slope	Parallel to Bedding/Stratigraphy/ Faulted Contacts
>30° to 35°	Bench Face Angle:	Parallel to Bedding/Stratigraphy/ Faulted Contacts to a maximum 65°
	Bench Height:	12 metres (double bench)
	Catch Bench Width:	Alternating 5 and 8 metres (10 metres for first bench below ground surface)
	Inter-Ramp Angle:	35° to 45° dependent on bench face

11.0 OVERALL SLOPE STABILITY ASSESSMENT AND PIT-DIKE SETBACK ASSESSMENT

Once bench design criteria were established for the various pits and design sectors, the overall slope stability was assessed using two-dimensional limit equilibrium methods for specific sections of the Goose Island and Portage Pits. The analysis sections were chosen to assess representative areas of the pit, with a focus on the most significant areas in regards to pit-dike set-back and unfavourable discontinuity orientations.

Numerical modeling techniques were also employed to assess the overall slope stability and to evaluate the stability of the foundation materials beneath the de-watering dikes along the analysis sections. Distinct element methods were used to analyse stability and deformations along this section.

The cross sections used in the analyses were developed from a geological block model provided by Meadowbank Mining Corporation. The pit designs were formulated by AMEC and were provided to Golder by MMC.

The methods used for the analyses, and the results of the analyses, are described below.

11.1 Analysis Sections

The following describes the analysis cross sections that were selected for slope stability assessment using limit equilibrium methods. The analysis sections are shown on Figure 11.1.

11.1.1 Goose Island Pit

Four analysis cross sections were selected for the Goose Island Pit at azimuths 065, 110, 190 and 290.

- Goose Southeast – 12+00S (azimuth 110) is located along the southeast wall of the pit, through the deepest section of the Goose Island dewatering dike at its closest point to the Goose Island Pit. The section represents the Goose Island pit at the end of its life cycle. The pit is approximately 165 m deep along this section. The toe of the de-watering dike is approximately 80 m back from the crest of the pit and the height of the dike above the lake floor is approximately 24 m at its crest mid-point. This section has been identified as being the most critical section for the pit slope stability analyses. The dike along this section has the smallest setback from the crest of the pit and is the highest due to it being built in the deepest section of water. Most of the pit slope along this section is expected to be composed of intermediate volcanic rock. This section is shown on Figure 11.2.

- Goose – South (azimuth 190) is located at the south end of the Goose Island Pit (ultimate). The section was selected to intersect the Goose Island Pit at its deepest level. The pit is approximately 155 m deep along this section. The toe of the de-watering dike is approximately 90 m back from the crest of the pit and the height of the dike above the lake floor is approximately 6 m at its mid-point. Most of the pit slope along this section is expected to be composed of ultramafic rock with intermediate volcanic and iron formation rock at the toe of the slope. The Bay Fault may be exposed in this slope, but has not been modeled explicitly because, with current interpretations of its location and orientation, it would have no bearing on the slope stability of this section. This section is shown on Figure 11.3.

- Goose – Northeast (azimuth 065) is located along the northeast wall of the pit. The pit is approximately 144 m deep along this section. The toe of the de-watering dike is approximately 105 m from the crest of the pit and the height of the dike above the lake floor is approximately 8 m at its crest mid-point. Most of the slope along this section is composed of intermediate volcanic rock, with a small amount of iron formation at the toe of the slope. Figure 11.4 illustrates this section.

- Goose West – 11+00S (azimuth 290) is located along the west wall of the pit. The section represents the Goose Island pit at the end of its life cycle. The pit is approximately 145 m deep along this section. The slope along this section is expected to be composed of ultramafic volcanics with bands of quartzite, intermediate volcanic and iron formation at the top of the slope. This analysis section is shown on Figure 11.5.

11.1.2 Portage Pit

Two analysis cross sections were selected for the Portage Pit at azimuths 144 and 270.

- Portage – Southeast (azimuth 144) is located at the southeast end of the Portage Pit (ultimate). The section was selected to intersect the Portage Pit at its deepest level and nearest to the Bay Zone de-watering dike. The pit is approximately 125 m deep along this section. The toe of the de-watering dike is approximately 80 m back from the bedrock crest of the pit and the height of the dike above the lake floor is approximately 8 m at its mid-point of the crest. Most of the pit slope along this section is composed of intermediate volcanics. Based on the block model from Cumberland, there may be a lens of iron formation rock within the slope. The

Bay Fault and the Bay Fault Splay Faults run through the floor of the pit. This analysis section is shown on Figure 11.6.

- Portage/Tailings – Northwest (azimuth 270) is located along the west wall of the Portage Pit at its north end, directly adjacent to the Central Dike. The section cuts through the Central Dike at its closest point to the Portage Pit. The pit is approximately 75 m deep along this section. The Central Dike toe is approximately 75 m back from the crest of the pit. This section was chosen to assess the pit slope stability with respect to the Central Dike. The geological block model supplied by Meadowbank does not extend sufficiently far enough West to provide a geological interpretation of this area. Therefore, a geological interpretation was developed based on a review of borehole geotechnical logs that have been drilled approximately within the footprint area of the tailings dike and on the known geological interpretation in the deposit area. This interpretation was then reviewed by MMC personnel (personal communication with Andrew Hamilton, February 15, 2007) to confirm the interpretation based on the available information. The analysis section is interpreted to cut through banded intermediate volcanics, quartzite, and ultramafic stratigraphy dipping into the slope. Figure 11.7 illustrates this analysis section.

11.1.3 Discontinuity Orientations

The discontinuity orientations that were used in the modeling were determined from the kinematic analysis of the stereographic projections included in Section 5.0 and were chosen based on the major sets. In order to assess the most likely range of foliation surfaces to potentially problematic, the data were filtered to exclude any foliation surface with a joint roughness parameter, Jr, greater than 1.0. The remaining data set, with joint roughness of 1 or less, are expected in general to represent the more persistent foliation surfaces as these are indicated to be smooth planar surfaces along which there may have been movement in the past. These are a subset of the more broadly distributed foliation set including Jr greater than 1, which are expected to be of lower persistence and with higher friction angles due to a combination of joint roughness and rock bridging.

In the limit equilibrium modeling, anisotropic ranges were used to represent ranges of discontinuity dip. These ranges were based either on two standard deviations of the average dip of the major set or, where this approach was not applicable the ranges were based on visual assessment of the extents of the major sets. The distinct element modeling incorporates the average dip of the major sets only.

The discontinuity orientations used in the slope stability analyses are described in the following paragraphs.

<u>Goose Island Pit</u>

The main structural features that are expected to influence overall pit slope stability along the Goose Southeast (12+00S) section are the moderately to steeply dipping foliation (persistent), and the shallow dipping joint set (non-persistent).

For the Goose – South, section the major sets affecting slope stability are a moderately steep conjugate joint set and a moderately dipping cross joint set. Both sets are interpreted to be discontinuous, non-persistent features such that large scale slope instability is not expected to be developed.

For the Goose – Northeast section, the major set affecting slope stability is the moderately steep foliation.

For the Goose West (11+00S) section, the major discontinuity sets are a steep orthogonal joint set and a shallow dipping joint set. Both sets are interpreted to be discontinuous, non-persistent features.

<u>Portage Pit</u>

For the Portage – Southeast section, the major sets that will influence slope stability are the persistent foliation, a non-persistent conjugate joint set, and a non-persistent orthogonal set.

For the Portage/Tailings Northwest section, the main discontinuity sestets are the persistent foliation dipping sub-horizontally, along with a non-persistent orthogonal set dipping out of the slope at about 74 degrees.

The following table summarizes the discontinuity orientations and associated dip ranges used in the slope stability analyses.

TABLE 11.1: Discontinuity Orientations Used in Stability Analyses

Analysis Section	Structural Domain	Rock Type	Mean Dip Out of Slope	Discontinuity Type	Persistence	Dip Range Around Mean (degrees)	Range Based on Two Standard Deviations (SD) or Visual Assessment (VA)	Stereographic Projection of All Poles (All) or Poles with Jr less than or equal to 1.0 (Jr) and/or only good reliability data (Good)
Goose Southeast - 12+00S	1	IV	20	Joint	NP	+7,-5	VA	All
			61	Foliation	P	+9,-6	VA	Jr
	2	IV	33	Joint	NP	+10,-11	SD	All
			74	Foliation	P	+16,-24	VA	Jr
	3	IV and IF	18	Joint	NP	+7,-8	VA	All
			55	Foliation	P	+20,-10	VA	Jr
Goose - South	2	IV, IF, UMV	60	Cross Joint	NP	+12,-12	SD	All
	3	IV, IF, UMV	40	Cross Joint and Shallow	NP	+38,-35	SD	All
Goose - Northeast	1	IV	52	Foliation	P	+8,-16	VA	Jr
	2	IV	77	Foliation	P	+8,-7	VA	Jr
	3	IV and IF	55	Foliation	P	+15,-15	VA	Jr
Goose West - 11+00S	4	IF, UMV, QZ	12	Shallow	NP	+8,-7	VA	Jr
			67	Orthogonal	NP	+6,-12	VA	Jr
Portage - SE	3 West	IV and IF	21	Foliation	P	+12,-15	VA	Jr
			87	Conjugate Joint	NP	+3,-2	VA	Jr
	3 East	IV, IF, UMV	12	Orthogonal	NP	+13,-12	VA	All
			69	Conjugate Joint	NP	+9,-7	VA	Jr
Portage/Tailings - NW	1	IV, UMV, QZ	8	Foliation	P	+7,-8	VA	Jr - Good
	2	IV, UMV, QZ	-14 (into the slope)	Foliation	P	+26,-26	SD	All
			73	Orthogonal	NP	+14,-13	VA	All
	3	IV, UMV, QZ	8	Foliation	P	+7,-8	VA	Jr - Good

P = Persistent, or continuous discontinuity

NP = Non-persistent, or discontinuous discontinuity

11.2 Methodology

Two dimensional slope stability limit equilibrium analyses on the analysis sections were completed using the software SLIDE. Models were run for 30,000 surfaces. In addition, two dimensional distinct element slope stability analyses were performed on the Goose Southeast – 12+00S wall section, representing the section of pit wall closest to the deepest section of the de-watering dike, using the software Universal Distinct Element Code (UDEC).

SLIDE utilizes the vertical method of slices technique to determine factors of safety along potential instability surfaces. The SLIDE results presented in this report were determined using the General Limit Equilibrium (GLE) – Morgenstern Price method of slices. The main distinction between different methods is how each approaches which equations of statics to consider which interslice normal and shear forces to include and how to assess the relationship between the interslice forces. The GLE method satisfies moment equilibrium equations, vertical and horizontal force equilibrium equations, and considers both interslice normal and shear forces while other methods only satisfy some of these requirements. The GLE method is useful for non-circular sliding surfaces.

In the UDEC analyses, the factors of safety were calculated using a strength-reduction technique. This technique involves successive reduction of both joint and rock mass strength parameters, in steps of 5% of their original values, until slope instability occurs. For example, if the strength parameters have to be decreased by 50% to cause instability, the resultant factor of safety is equal to 1.5. If the in-situ strength parameters need only be decreased by 20% to cause instability, then the resultant factor of safety is 1.2. The displacements of selected points along the slope are monitored during each factor of safety iteration (*i.e.,* each step of strength reduction). The factor of safety for the slope is chosen based upon the percentage of strength reduction at the instant that the rate at which the slope deforms becomes asymptotic.

The input parameters used for both the SLIDE and UDEC analyses are presented in the following section. The results from both types of analyses are also presented and compared in the subsequent sections.

11.3 Limit Equilibrium and Distinct Element Input Parameters

11.3.1 Rock Mass Strength Input Parameters

The rock mass strength parameters used in the SLIDE and UDEC analyses were derived using the program RocLab, which provides Mohr-Coulomb failure criterion based on UCS values, RMR values, intact m (m_i) values and estimates of blasting disturbance (D).

The m_i value is an empirical constant used in the Hoek-Brown criterion. Values for m_i are either determined using the results of triaxial testing, or are estimated based on tabulated values. In the case of the Meadowbank project, the values of m_i have been based on tabulated values.

The Disturbance Factor (D) used in RocLab depends upon the degree of disturbance to which a rock mass will be subjected to by blast damage and stress relaxation (Hoek et al, 2002). A D of 1.0 is used when modeling a rock that will undergo significant disturbance due to heavy production blasting in very large open pit mines. Using a D of 1.0 for the relatively small pit slopes at Meadowbank, which will be mined with careful blasting due to the proximity and sensitivity of the dewatering dikes, is expected to result in a conservative design approach. Mining at the Meadowbank project will require careful blasting practices due to the proximity of the walls to the dewatering dikes. Consequently, a D value of 0.7 might be more appropriate. However, the geometry of the main structural features, the potential for stress relaxation of the rock leading to possible separation of foliation surfaces, and the presence of pore water pressures suggests a more conservative approach should be taken until mining is underway and more site specific data are available regarding the behaviour of the rock mass in the slopes. Hence the D value has been maintained 1.0 for the current study.

The following table summarizes input parameters to RocLab used to calculate the Hoek-Brown failure criterion.

TABLE 11.2: Rockmass Strength Input Parameters

Rock Type	Pit	UCS (MPa)	RMR (from Q')		m_{intact}	Disturbance Factor, D
			Mean	Median		
IF	Goose	137 to 248 (avg. 175)	69/75*	71/78*	20	1
	Third		68	69		
	North		69	69		
	Vault		69	68		
IV	Goose	51 to 148 (avg. 94)	70/76*	71/77	18	1
	Third		66	66		
	North		65/70**	66/66**		
	Vault		66	66		
UMV	Goose	40 to 92 (avg. 66) Serpentinized avg. 32	63/68*	66/69*	10	1
	Third		64	68		
	North		56	58		
	Vault		-	-		
Quartzite	Goose	70 to 140 (avg. 107)	-	-	20	1
	Third		68	68		
	North		-	-		
	Vault		-	-		

*(Data collected by Cumberland)/(Data collected by Golder Associates Ltd. in 2003)
**(IV/IVcl) / (IV/IVcs)

RocLab uses intact rock strength input parameters to develop linear Mohr-Coulomb rock mass strength parameters based on curve fitting of the non-linear Hoek-Brown criterion for rock mass. The curve fitting results are presented in Appendix VII. The confining stresses used to determine the RocLab failure criterion were calculated within the program using the “custom” function. The confining stress used to determine the failure criterion was 1.4 MPa, which is a reasonable representation of the range of confining stresses over which failure could potentially occur. The rock mass strength parameters that were derived for the limit equilibrium analyses summarized in the following table.

TABLE 11.3: Limit Equilibrium and UDEC Rock Mass Input Parameters

Material	Unit Weight kN/m3[1]	Cohesion, c (MPa)[3]	Friction Angle (degrees)[3]
IF (RMR 75%)	30.616[1]	2.681	60
IV (RMR 76%)	25.721[1], 24.475[1]	1.771	56
UMV (RMR 68%)	25.899[1]	0.898	46
Quartzite (RMR 68%)	24.03[1]	1.151	55
Dike[1]	18.64[2]	0	38
Till[1]	19.40[2]	0	32

1. Cumberland Resources Ltd, 2005.

2. Golder Associates Ltd. 2007a/2007b.

3. Value calculated in RocLab

11.3.2 Discontinuity Strength Input Parameters

The discontinuity strength input parameters used in SLIDE and UDEC were based on direct shear lab testing results, as summarized in Section 6.6. All discontinuity types were modeled without cohesion.

The foliation for each rock type was modeled using the residual friction angle determined by laboratory testing with an additional 6 degrees of frictional strength added to account for undulations as described in Section 6.6,.

The joint sets were modeled using the peak friction angle. This is because the joint sets are described in drillcore as being generally rough to irregular.

The following table summarizes the joint strength input parameters used in SLIDE.

TABLE 11.4: Limit Equilibrium Discontinuity Strength Input Parameters

Discontinuity	Cohesion (kPa)	Friction Angle (degrees)
IF, IV – Foliation	0	37
IF, IV – Joint	0	42
UMV – Foliation	0	37
UMV – Joint	0	42
Fault Zone	0	25

Note: Friction angles based on peak and residual angles from shear testing.

Although the discontinuity strength parameters have assumed zero cohesion for joint and discontinuity surfaces, it is reasonable to expect some cohesion along the planes. It is known that many of the foliation and joint surfaces at the project site are intact and strongly bonded and are described as rough. Furthermore, based on the current interpretation the joint systems are expected to be non-persistent, and so are expected to have a certain percentage of rock bridging associated with them, which will impart an effective cohesion along certain step-path failure modes. Therefore, the assumption of zero cohesion is conservative and some effective cohesion due to rock bridging should be expected. Although it is not currently possible to quantify what the percentage rock bridging may be, it is reasonable to expect something on the order of 10%, and potentially as high as 20%.

Sensitivity analyses have been performed to estimate the effect of different rock bridge percentages along the shallow set will have on the overall pit slope stability. The effect of rock bridging was considered in both SLIDE and UDEC modeling. The sensitivity analyses started with 5% rock bridging along the shallow set and then, if resultant effective joint cohesion was not sufficient to meet the required factors of safety, the rock bridge percentage was progressively increased by 5% until the required safety factors were reached. The following equations were used to derive the scaled joint strength properties:

$$\textit{Scaled Cohesion} = (\%\text{Rock Bridge}) \times \text{Rock Mass Cohesion} + (1\text{-}\%\text{Rock Bridge}) \times \text{Joint Cohesion}$$

$$\textit{Scaled } \Phi = \tan^{-1}[(\%\text{Rock Bridge}) \times \tan(\Phi_{\text{Rock Mass}}) + (1\text{-}\%\text{Rock Bridge}) \times \tan(\Phi_{\text{Joint}})]$$

The discontinuity strength input parameter values used in the UDEC modeling are summarized in the following table. The UDEC modeling used the residual friction angle from lab testing without the "waviness factor" added. This provides a conservative evaluation of the stability of the slopes.

TABLE 11.5: UDEC Modeling Discontinuity Strength Input Parameters

Discontinuity	Scaled Cohesion (kPa)	Scaled Friction Angle (degrees)
IV-Foliation	0	31
IV-Joints	88.5	42.9
IF-Foliation	0	31
IF-Joint	134	43.3

11.4 Groundwater Conditions

For the SLIDE and UDEC analyses, water pressure values determined in the ModFLOW® hydrogeological model were used. In SLIDE, a water pressure grid with spacing suitable for the software's interpolation method was used. The grid interpolation method used in the SLIDE modeling was the "Modified Chugh" method. For the UDEC models, water pressure values were input at points along every joint.

11.4.1 Prediction of Pressure Heads and Pit Inflows

Base Case

The ModFLOW numerical model was used to predict the hydraulic heads and water pressures within the walls of the Portage and Goose Pits. The mine plan was simulated in the model as described above. Pressure heads were predicted for three sections in the pit walls of the Portage Pit and four sections in the pit walls of the Goose Pit. The locations of these sections are presented in Figure 11.1. The pressures along these sections were provided for the end of Year 5 for the Goose Pit and the End of Year 8 for the Portage Pit. These times correspond to the ultimate depths for each of the pits.

The initial pit slope stability analyses indicate that depressurization along three of the seven sections, Portage Southeast, the Goose Southeast 11+00 and Southeast 12+00, may be required to meet an acceptable factor of safety. Mitigated measures, consisting of horizontal drainholes drilled from the pit walls as the pit advanced, were simulated and slope stability analyses conducted on the resulting pore pressure profiles to assess if the factor of safety criteria could been met along these three sections through depressurization. Based on the results of these analyses, a configuration of depressurization wells considered to be adequate for slope stability purposes was determined. Information on the portions of slope requiring depressurization is discussed in Section 13.

The areas of the pit walls where the proposed depressurization system will be installed are presented in Figure 7.9. To depressurize the pit walls along the portions of the pit walls represented by the Portage Southeast, the Goose Southeast 11+00, and the Goose Southeast 12+00, horizontal drainholes spaced approximately 15 m apart would be installed at the elevations summarized in Table 11.6. An equivalent spacing could be achieved if the drilling stations were located 30 m apart and two boreholes were drilled from a fan pattern from each station. A total of approximately 115 drainholes would be necessary as indicated in Table 11.6. This depressurization system will be installed with a phased approach and initially, approximately only half of these drainholes would be installed at twice the spacing that was simulated in the model. This phased approach would be adopted because, unlike the model, the actual rock mass is expected to be anisotropic and inhomogeneous and twice the spacing simulated in the model might be sufficient in some areas of the wall. Alternatively, other areas of the wall may require that a closer spacing of the drainholes be installed than was predicted in the model.

Drainholes along the Portage Southeast and the western extent of the Goose Southeast 11+00 would be approximately 60 m long, while drainholes along eastern extent of the Goose Southeast 11+00 and the Goose Southeast 12+00 sections would be approximately 90 m long. Depressurization wells were assumed to be HQ size, although PQ size would be preferred. The depressurization plan was initially based on that recommended for low conductivity formations in CANMET (1977) and was refined during initial model simulations.

TABLE 11.6: Specifications for Drainholes installed for Pit Wall Depressurization

Section	Elevation	Length	Number
Portage Southeast	24 m, 42 m, 66 m	60 m	25
Goose Southeast 11+00 and 12+00 (east side of the pit)	-34 m, -12 m, 12 m, 36 m, 60 m*	90 m	60
Goose 11+00 (west side of the pit)	-12 m, 12 m, 36 m, 60 m	60 m	30

*Depressurization wells were not assumed to be installed at 60 m elevation beneath widest section of Goose Island where permafrost would likely be present over the entire length of the hole.

Assuming this depressurization system is not installed until each of the ultimate pits is reached, inflows to the drainholes are predicted to be approximately 500 m^3/day for the Goose Pit by the end of Year 5 and approximately 200 m^3/day for the Portage Pit at the end of Year 8. By Year 8, the Goose Pit is planned to be in operation as an attenuation pond and the drainholes will no longer be operational. Predicted inflows to the Goose and Portage Pits, and the associated depressurization system, are summarized in the table below.

TABLE 11.7: Predicted Inflows to the Meadowbank Mines Base Case

Year	Predicted Inflow (m^3/day)			
	Portage Pit	**Goose Pit**	**Drainholes**	**Total**
End of Year 05	1300	700	500	2500
End of Year 08	1200	800	200	2200

Sensitivity to Anisotropy

A sensitivity simulation was conducted in which the vertical hydraulic conductivity of the bedrock was assumed to be three times greater than the horizontal hydraulic conductivity to account for the near vertical foliation in the rock. This adjustment in the model was made such that the geometric mean of the hydraulic conductivity of the bedrock with depth remained the same as in the base case simulation.

Predicted pore pressures resulting from this simulation were found to be somewhat higher in the pit walls, but lower beneath the toe. Predicted inflows were found to be up to 15% less than those predicted in the Base Case simulation. Predicted pore pressures determined during this simulation were used as input to slope stability analyses.

Predicted inflows to the pits and depressurization system are summarized in Table 11.8.

TABLE 11.8: Predicted Inflows to the Meadowbank Mines Sensitivity Scenario with Anisotropy

Year	Predicted Inflow (m^3/day)			
	Portage Pit	Goose Pit	Drainholes	Total
End of Year 05	1200	600	400	2200
End of Year 08	1100	700	100	1900

Sensitivity to Weathered Bedrock

Although the hydraulic conductivity data collected to date for the shallow bedrock does not indicate that a highly weathered zone with greater hydraulic conductivity exists in the uppermost 25 m of bedrock, at some other sites in the Canadian Shield (Diavik, 2005) such a zone has been found to exist. Consequently, a sensitivity simulation was performed to determine the effect of a high hydraulic conductivity zone in the shallow weathered/exfoliated bedrock on pit inflows and the effectiveness of the planned depressurization system. In this sensitivity simulation, the hydraulic conductivity of the uppermost 25 m of the bedrock was increased from 7×10^{-7} m/s to 5×10^{-6} m/s (Golder, 2007a).

The results of this simulation indicate that inflows to the Meadowbank mines would be up to two times greater if the high K zone was present. Predicted inflows to the pit and depressurization system are summarized in the following table. Predicted pore pressures determined during this simulation were used as input to slope stability analyses.

TABLE 11.9: Predicted Inflows to the Meadowbank Mines Sensitivity Scenario with Weathered Bedrock

Year	Predicted Inflow (m^3/day)			
	Portage Pit	Goose Pit	Drainholes	Total
End of Year 05	2700	1500	600	4800
End of Year 08	2600	1800	200	4600

11.5 Additional UDEC Considerations

11.5.1 Boundary Conditions

The UDEC models were designed to avoid boundary effects, especially in areas of particular interest such as the region beneath the water retention dike. The boundaries are located approximately three times the half-width of the pit away from the pit crest and the toe. The bottom and side boundaries of the models were designed to represent zero displacements boundaries allowing movements parallel to the boundary but not in a direction perpendicular to it. The open pit side boundary of the models is representative of an axis of symmetry. The models are two dimensional, but not axisymmetric, meaning that they simulate a long high wall rather than the true circular pit. As a result, the models neglect the effects of lateral confining stresses for a circular pit, such as at Goose pit or at the south end wall of the Portage pit, and so are conservative in those cases.

11.5.2 In Situ Stress

In the absence of measured pre-mining in situ stresses, based on available literature (ref: Hoek and Brown, 1981) a horizontal to vertical stress ratio (K) of 2:1 was used for the UDEC modeling.

11.5.3 Modeling Limitations

<u>SLIDE</u>

Limit equilibrium methods consider statics, but not strains nor displacements. As a consequence, strain and displacements cannot be evaluated using these methods. It is therefore possible to achieve an acceptable factor of safety for a given slope using limit equilibrium methods, but predict unacceptable deformations from a load-deformation analysis. This limitation is overcome by the use of finite element and distinct element modeling software that allows strain and deformation to be assessed.

<u>UDEC</u>

Limitations of distinct element modeling are associated with difficulty in simulating discontinuity geometry that is representative of the actual spacing and persistence of discontinuities within a rock mass, particularly highly jointed rock masses. Often limited data on discontinuity properties are available, especially with regards to joint stiffness, which is a parameter that is not well understood and is not easily quantified. Sensitivity analysis can be difficult to perform because models typically require substantial computing time to complete. Also, the computational time that is necessary to realistically model pore pressures in UDEC is very restrictive. To model the pore pressure change along the joint surfaces once the joint surfaces have opened would require substantial computing time and a complexity in the code that would be difficult to achieve.

11.6 Slope Stability Analyses Results

The required factor of safety is 1.3 for overall slope stability and 1.5 at the toe of the dewatering and tailings dikes under static conditions. Under pseudo-static conditions a design factor of safety of 1.1 is adopted for use with the 1 in 10,000 year earthquake event.

The analyses were run assuming various rock bridge percentages to develop effective cohesion strength parameters. Where the required Factor of Safety was not achieved with natural drainage of a slope, slope depressurization was modeled.

The following sections present the results of the slope stability modeling for all of the analysis sections.

11.6.1 Goose Southeast – 12+00S

The Goose Southeast – 12+00S analysis section was modeled in both SLIDE and UDEC. This section intersects the deepest section of the Goose Island Pit dewatering dike and the pit along this section has the closest setback from the dike. The slope stability of this section will be controlled by the relatively steep, wavy, persistent foliation and the shallow non-persistent shallow dipping joint set.

Initially, analyses for this section were completed assuming only natural slope depressurization. Under these conditions, the required factors of safety were not met in either SLIDE or UDEC analyses.

Sensitivity Analysis 1 – SLIDE Analysis with 20% Rock Bridge, No Depressurization

As discussed in Section 6.6.3 drilling data and laboratory testing suggest that the joint set is discontinuous and widely spaced. It is therefore reasonable to expect a relatively high rock bridge percentage along the joint surfaces. Sensitivity analyses were performed with SLIDE to determine what rock bridge percentage was sufficient to achieve the required safety factors for the slope. No depressurization measures were modeled in these analyses. A rock bridge percentage of 20% along the joint set was assumed. This resulted in a predicted factor of safety of 1.3 for the overall slope and 2.3 for potential failure surface developing back to the toe of the dewatering dike. Figure 11.8 shows the SLIDE results.

Sensitivity Analysis 2 – UDEC Analysis with 20% Rock Bridge, No Depressurization

Results from the UDEC analyses suggest that a 20% rock bridge percentage with no slope depressurization is insufficient to meet the required safety factors. The disagreement between UDEC and SLIDE results is related to the different algorithms used to solve equations of static equilibrium and force-displacement. UDEC uses a force-displacement law specifying interaction between deformable, joint bounded blocks and Newton’s second law of motion, providing displacement induced within the rock slope. Stress and strain are computed at each step of the numerical analysis. The high pore pressure values modeled along discontinuity surfaces, combined with the decreasing horizontal stresses after the excavation, induce tensile stresses on the steep foliation. All discontinuities were modeled under the conservative assumption of zero tensile strength and tensile stresses on the foliation surfaces results in the opening of these features and very low predicted factor of safety. Figure 11.9 illustrates UDEC modeling results for this scenario.

Sensitivity Analysis 3 – SLIDE and UDEC Analysis with 0% Rock Bridge and Slope Depressurization

An analysis was carried out using SLIDE and UDEC, assuming cohesionless discontinuities but with slope depressurization measures installed. The depressurization assumes 90 m long horizontal drainholes spaced 24 m vertically and 15 m horizontally.

The results of the analyses indicate that the required factors of safety for the pit slope and the dike toe are achievable through the use of slope depressurization and under the assumption of no effective cohesive strength imparted to the failure surfaces due to rock bridging.

The maximum displacement of the pit slope is predicted to be about 25 cm; the maximum displacement at the toe of the dewatering dike is predicted to be about 1 cm. Figures 11.10 to 11.13 show the results of the SLIDE and UDEC analyses for the depressurized slope for Goose Southeast (12+00S).

Sensitivity Analysis 4 – UDEC Analysis with 5% Rock Bridge and Slope Depressurization

A further UDEC model was run to analyse the compositive effect of slope depressurization and rock bridging along the shallow non-persistent joint set. The results of the analyses indicate that with depressurization and a 5% rock bridge along the shallow joint set, a factor of safety of 1.5 for the overall slope is achieved, but with up to 50 cm of displacement. These results are shown in Figures 11.14 to 11.16.

Sensitivity Analysis 5 – SLIDE Analysis for Groundwater Anisotropic Flow Model

Anisotropy was assessed in the hydrogeological model during sensitivity analysis and was modelled concurrently with depressurization. In this sensitivity analysis, a case was considered in which hydraulic conductivity is three times greater along the steep foliation. The effect of the anisotropy on the effectiveness of the depressurization was small, resulting in slightly higher safety factors in the SLIDE modeling as shown on Figure 11.17.

Summary of Results for Goose Southeast 12+00S

The following table presents the results of the SLIDE and UDEC modeling for the various scenarios previously discussed in this section.

TABLE 11.10: SLIDE Results for Goose Southeast (12+00S) Analysis Section

Groudwater Conditions	Rock Bridge Percentage	SLIDE FOS (GLE)	SLIDE FOS (GLE) at dike toe	Scale of Instability Predicted in SLIDE	UDEC FOS	UDEC Maximum Pit Slope Displacement (cm)	Scale of Instability Predicted in UDEC	UDEC Maximum Dyke Toe Displacement (cm)
No depressurization - ver3 model (March 2)	20%	1.3	2.3	Full Slope	<1	500	Multiple Benches	2.5
Depressurized - ver2 model (March 2)	0%	1.3	1.6	Full Slope	1.3	25	Multiple Benches	< 1
Depressurized – ver1 model	5%	-	-	-	1.5	50	Multiple Benches	< 1
Depressurized - ver2 model (March 2) - anisotropic flow	0%	1.3	1.6	Full Slope	-	-	-	-

The results of the analyses indicate that with expected artificial slope depressurization levels the required factors of safety for the pit wall and for the dike toe are achievable for the most conservative assumptions of 0% rock bridge. It is reasonable to expect significant rock bridging to occur within the pit slopes that will contribute further to the stability of the slopes. During development of the pit, it will be critical to carry out detailed geotechnical mapping of the early benches to confirm the assumptions on which the current designs are based. If rock bridging is proven by field observation, it may be possible to reduce the degree to which slope depressurization is required.

The maximum pit slope displacements predicted in UDEC for the depressurized scenarios are 25 cm for 0% rock bridging with a 1.3 safety factor and 50 cm for 5% rock bridging with a 1.5 safety factor. These values of displacement are considered reasonable as they do not effect or amplify deformations at the dewatering dike, which are predicted to be less than 1 cm for the same models. Typically, it is acceptable for displacements of the predicted magnitudes to occur in pit slopes provided that the interaction of other aspects of the mine with these displacements is not unfavourable, namely the interactions with safety and with infrastructure. Slope stability monitoring and instrumentation will be required to be installed in the walls and around the crest of the Goose and Portage Pits. The slope displacements can then be monitored to insure that they are not exceeding the predicted displacements and that they are meeting with required operational safety guidelines.

11.6.2 Goose – South

The Goose – South analysis section was assessed in SLIDE. The analysis section is for the south pit wall, where the foliation and stratigraphy at the Goose Island deposit will strike into the wall and the pit wall has a very small radius of curvature. Consequently, it is expected that additional confinement due to the development of tangential, or hoop, stresses within the pit walls will contribute to overall slope stability in this area. Nonetheless, a stability analysis was completed for the wall section.

The controlling discontinuity sets for this section are a cross joint set in Domain 2 that dips at approximately at 60 Degrees towards the north, and both a shallow set and a cross joint set that together dip at an average of 40 degrees towards the north. Based on the current engineering geological model for the project, it is expected that both the cross set and the shallow set are discontinuous, non-persistent joint sets such that rock bridging between coplanar joints will result in significant increased effective cohesion along any potential failure planes. However, the analyses for this section were run first with no cohesion along any of the joint sets and no slope depressurization to represent the most conservative conditions.

The required factors of safety were achieved under these conservative assumptions. Figure 11.18 shows the results of the SLIDE modeling of the Goose – South section. The predicted factor of safety for overall slope instability is approximately 2.0. The predicted factor of safety for instability extending back to the toe of the dewatering dike is 2.8. The following table summarizes the SLIDE results for the Goose – South section.

TABLE 11.11: SLIDE Results for Goose - South Analysis Section

Groudwater Conditions	Rock Bridge Percentage	FOS (GLE)	FOS (GLE) at dike toe	Scale of Instability Predicted in SLIDE
No depressurization - ver3 model (March 2)	0%	2.0	2.8	Upper Slope, Upper 5 Benches

11.6.3 Goose – Northeast

The Goose – Northeast analysis section was assessed in SLIDE. The northeast pit wall is expected to be comprised primarily of intermediate volcanic rock. The structural control for pit slope stability for this section is the steep foliation, which is interpreted to dip at 52, 77, and 55 towards the Southwest for Domain 1, 2 and 3 respectively. The analysis was run for conservative assumptions of 0% rock bridge and no slope depressurization. The resultant factor of safety for this section is 2.8 for overall slope instability and 4.2 for instability extending back to the toe of the dewatering dike. Figure 11.19 presents the SLIDE results for Goose – Northeast and the following table summarizes the results.

TABLE 11.12: SLIDE Results for Goose - Northeast Analysis Section

Groudwater Conditions	Rock Bridge Percentage	FOS (GLE)	FOS (GLE) at dike toe	Scale of Instability Predicted in SLIDE
No depressurization - ver3 model (March 2)	0%	2.8	4.2	Full Slope

11.6.4 Goose West – 11+00S

The Goose West (11+00S) analysis section was modeled in SLIDE. The analysis section is for the west wall of the Goose Pit where the foliation and stratigraphy will dip into the wall at moderate angles. It is expected that banded quartzite, iron formation and ultramafic rock will form the upper portions of the wall, while only ultramafic rock will form the lower portions.

The slope stability of this section will be controlled by the relatively steep orthogonal joint set, dipping at approximately 67 degrees towards southeast and a shallower joint set dipping at approximately 12 degrees towards the southeast. Based on the current engineering geological model, these joints are interpreted to be discontinuous, non-persistent sets.

As discussed in Section 6.5.3 drilling data and laboratory testing suggest that rock bridging along joint sets is very likely. It is therefore likely to expect a relatively high rock bridge percentage along the joint surfaces in the shallow and orthogonal joint sets. Sensitivity analyses were performed with SLIDE to determine what rock bridge percentage would result in the required safety factor for the slope. A rock bridging of 5% along the joint sets was predictable to be sufficient to achieve the required design criteria, resulting in a safety factor of 1.3 for the overall slope. Figure 11.20 illustrates the SLIDE results for the 5% rock bridge model.

Alternatively, if slope depressurization is carried out and the percentage of rockbridge is assumed to be zero, the required safety factor is still achievable. As for Goose Southeast (12+00S), the depressurization incorporated into the hydrogeological model for the Goose West (11+00S) section consisted of 90 m long horizontal drainholes spaced 24 m vertically and 15 m horizontally. The factor of safety for the depressurized analysis is 1.6. Figure 11.21 shows the SLIDE depressurized runs for Goose West (11+00S).

The following table presents the results of the SLIDE modeling for the various scenarios aforementioned in this section.

TABLE 11.13: SLIDE Results for Goose West (11+00S) Analysis Section

Groudwater Conditions	Rock Bridge Percentage	FOS (GLE)	FOS (GLE) at dike toe	Scale of Instability Predicted in SLIDE
No depressurization - ver3 model (March 2)	5%	1.3	No Dike	Multiple Benches
Depressurized - ver2 model (March 2)	0%	1.6	No Dike	Multiple Benches

11.6.5 Portage – Southeast

The Portage – Southeast analysis section was assessed using SLIDE. The southeast analysis section is for the southeast wall of the pit, adjacent to the de-watering dike. The toe of the de-watering dike is approximately 80 m back from the crest along this section. It is expected that intermediate volcanic rock will form the pit wall.

The controlling discontinuity sets for this section in Domain 3 East are a steep conjugate joint set dipping approximately 69 degrees towards the northwest and a shallow joint set dipping at approximately 12 degrees towards the northwest. The controlling discontinuity sets for this section in Domain 3 West are a steep conjugate joint set dipping approximately 87 degrees towards the northwest and demonstrating a non-distinct, or dispersed, pole pattern as well as a shallow foliation dipping at approximately 21 degrees towards the northwest,

The analyses for this section were completed assuming conservative parameters of no effective cohesion due to rock bridging along any of the joint sets and no slope depressurization. The required factors of safety were not achieved for these conservative assumptions, but based on the current engineering geological model, these joints are interpreted to be discontinuous, non-persistent sets

If rock bridging is considered along the joint sets, the required safety factors are achievable with a minimal rock bridge percentage of 5%. The resultant safety factors are 1.4 for the overall slope and 2.1 for instability back to the toe of the dewatering dike. Figure 11.22 illustrates the SLIDE results for the 5% rock bridge model.

Conversely, if slope depressurization is considered for this section, the required safety factors can also be met with no rock bridging along the joint surfaces. The factor of safety for the depressurized slope is 1.4 and 2.1 for instability extending back to the toe of the dewatering dike. Figure 11.23 show the SLIDE depressurized runs for Portage – Southeast.

TABLE 11.14: SLIDE Results for Portage - Southeast Analysis Section

Groudwater Conditions	Rock Bridge Percentage	FOS (GLE)	FOS (GLE) at dike toe	Scale of Instability Predicted in SLIDE
No depressurization - ver3 model (March 2)	5%	1.4	2.1	Full Slope
Depressurized - ver2 model (March 2)	0%	1.4	2.1	Full Slope

As for the other sections modeled with depressurization measures, depressurization assumes 90 m long horizontal drainholes spaced 24 m vertically and 15 m horizontally.

11.6.6 Portage/Tailings – Northwest

The Portage – Northwest analysis section was assessed in SLIDE. The northwest analysis section extends west through the west pit wall of the Portage pit at its north end and through the Central Dike at it highest point. The toe of the dike is also at its closest to the pit in this area. It is expected that this wall will be composed of banded intermediate volcanics, quartzite, and ultramafic rock, with stratigraphy and foliation dipping generally westward at shallow angles into the wall.

The controlling discontinuity sets for this section are a sub-horizontal foliation dipping at most only 15 degrees towards the east and, at the toe of the slope, a steep orthogonal joint set dipping at approximately 73 degrees. The analyses for this section were run with no cohesion along any of the joint sets and no slope depressurization. The required factors of safety were met under these conservative assumptions: the safety factor for the overall slope is 1.7 and for instability through to the toe of the Central Dike, is 2.0.

Figure 11.24 presents the results for the Portage – Northwest SLIDE modeling. The following table summarizes the results.

TABLE 11.15: SLIDE Results for Portage/Tailings - Northwest Analysis Section

Groudwater Conditions	Rock Bridge Percentage	FOS (GLE)	FOS (GLE) at dike toe	Scale of Instability Predicted in SLIDE
No depressurization - ver3 model (March 2)	0%	1.7	2.0	Upper Two Benches

11.7 Summary and Conclusions

The following table presents the results of the SLIDE and UDEC modeling. Rock bridge sensitivity analyses were performed for certain sections. Also, depressurization measures were modeled for certain sections.

TABLE 11.16: Summary of Stability Analyses Results

Section	Groudwater Conditions	Rock Bridge Percentage	SLIDE FOS (GLE)	SLIDE FOS (GLE) at dike toe	UDEC FOS	UDEC Maximum Pit Slope Displacement (cm)	UDEC Maximum Dyke Toe Displacement (cm)
Goose Southeast – 12+00S	No depressurization - ver3 model (March 2)	20%	1.3	2.3	<1	500	2.5
	Depressurized - ver2 model (March 2)	0%	1.3	1.6	1.3	25	< 1
	Depressurized – ver1 model	5%	-	-	1.5	50	< 1
	Depressurized - ver2 model (March 2) - anisotropic flow	0%	1.3	1.6	-	-	-
Goose – South	No depressurization - ver3 model (March 2)	0%	2.0	2.8	-	-	-
Goose - Northeast	No depressurization - ver3 model (March 2)	0%	2.8	4.2	-	-	-
Goose West – 11+00S	No depressurization - ver3 model (March 2)	5%	1.3	No Dike	-	-	-
	Depressurized - ver2 model (March 2)	0%	1.6	No Dike	-	-	-

Section	Groudwater Conditions	Rock Bridge Percentage	SLIDE FOS (GLE)	SLIDE FOS (GLE) at dike toe	UDEC FOS	UDEC Maximum Pit Slope Displacement (cm)	UDEC Maximum Dyke Toe Displacement (cm)
Portage - Southeast	No depressurization - ver3 model (March 2)	5%	1.4	2.1	-	-	-
	Depressurized - ver2 model (March 2)	0%	1.4	2.1	-	-	-
Portage/Tailings - Northwest	No depressurization - ver3 model (March 2)	0%	1.7	2.0	-	-	-

The results of the analyses indicate that under conservative assumptions of 0% rock bridge, equivalent to no effective cohesive strength along foliation and joint surfaces, the required factors of safety for the pit slope and for the dike toe at the current design setback distance are achievable for all slopes with the exception of the Goose Southeast slope, the Goose West slope, and the Portage Southeast.

The Goose Southeast slope refers to the east wall of the Goose Pit, directly adjacent to the deepest dewatering dike section, and the shortest dike toe to pit crest distance. The Goose West slope refers to the west wall of the Goose Pit, directly across the pit floor from the Goose Southeast slope. The Portage Southeast section refers to the southeast wall of the Portage Pit, adjacent to the dewatering dike for this area of the pit. Sensitivity analyses carried out for these slope sections indicate that if the slopes are depressurized, and assuming the most conservative condition of 0% rock bridge, the required safety factors are achievable. Furthermore, the analyses show that if a reasonable percentage of rock bridge (20% for the Goose Southeast slope and 5% for both the Goose West and Portage Southeast slopes) is included, then the required safety factors are achievable even without depressurization. It is prudent to recommend slope depressurization for this slope because although 5 to 20% rock bridging is a reasonable assumption for rock bridge percentage, it cannot be quantified at this time. During operations, as the pit is advanced a program of detailed geotechnical mapping should be implemented immediately. If the mapping can confirm whether a sufficient percentage of rock bridge is occurring in a given slope, then a decision could be made to reduce or potentially eliminate slope depressurization measures.

11.7.1 Stability of Slopes Under Seismic Loading

The stability of the slopes under seismic loading conditions was assessed using limit equilibrium methods. Analyses were run using General Limit Equilibrium (GLE) method of slices. The analyses were run on 30,000 surfaces. The maximum design earthquake (MDE) for the dewatering dikes is the 1 in 10,000 year event and this has been adopted for the assessment of the pit slopes under seismic loading. This introduces conservatism into the analyses. Extrapolation of the design event from the site specific study done by Pacific Geoscience Centre in Victoria in 2003 for the Meadowbank project area indicates the 1 in 10,000 year peak horizontal ground acceleration (PHGA) is 0.07g. The stability under pseudo-static conditions has been assessed using 50% of the PHGA, or in this case 0.035g. Under pseudo-static conditions a design factor of safety of 1.1 is adopted for use with the 1 in 10,000 year earthquake event. The percentage of rock bridge in the analyses was assumed to be 0%.

The following table summarizes the results of this assessment. The results of the assessment are illustrated in Appendix VII.

TABLE 11.17: Summary of Slope Stability for Pseudo-Static Conditions

Section	Groudwater Conditions	Rock Bridge Percentage	FOS (GLE)	FOS (GLE) at dike toe
Goose - 12+00SE	No depressurization - ver3 model (March 2)	0%	0.9	1.1
	Depressurized - ver2 model (March 2)	0%	1.3	1.4
Goose - South	No depressurization	0%	1.9	2.7
Goose – NE	No depressurization	0%	2.6	3.8
Goose - 11+00West	No depressurizaiton - ver3 model (March 2)	0%	1.1	No Dike
	Depressurized - ver2 model (March 2)	0%	1.5	No Dike
Portage – SE	No depressurization	0%	1.0	1.3
	Depressurized - ver2 model (March 2)	0%	1.4	1.5
Portage/Tailings - NW	No depressurization	0%	1.5	1.5

The results of the analyses indicate that under the conservative assumptions and for the case of naturally drained slopes (no depressurization) all sections with the exception of Goose 12+00SE and Portage SE are predicted to achieve the required factors of safety for the pit wall and for the dike toe. For the Goose 12+00SE and Portage SE sections, depressurization of the pit walls in these areas is predicted to result in achievement of the required factors of safety. The analyses do not account for rock bridging which will result in an increase in the predicted factors of safety.

12.0 BLASTING CONSIDERATIONS WITH RESPECT TO DIKE OFFSET

Two blast design reports have been produced previously for the project:

- Golder Associates Ltd., 2004h. Report on *Blast Design, Meadowbank Gold Project, Nunavut*, February 10, 2004

- Addendum: Blasting Report Addendum, Golder Associates, May 25, 2004

- Golder Associates Ltd., TM-2005d. Technical Memorandum on "Item #85 and 85A – Meadowbank Gold Project – Blasting Addendum", 06 October 2005

The reader is directed to review the two previous reports which describe in greater detail the development of assumptions on which the evaluations have been based, the procedures used to carry out the evaluations, and other parameters used in the evaluations that may be presented below but are not described. This assessment has been carried out based on general assumptions for blasting agents that may be used at the mine in order to provide guidance for acceptable setback distances. A program of blast evaluation will need to be carried out during operations to optimize blast design.

The previous report, and the report addendum, included consideration of blast induced vibration from the perspective of the stability of the perimeter dikes and tailings dike and from the perspective of the effect of blast induced vibration on fish and fish habitat. Estimates of blast induced vibration and instantaneous pressure change were presented for various charge weights based on initial evaluation of blast design. The feasibility study by Amec Americas Ltd. recommended that a charge weight of 77 kg for a bench height of 6 m be used (AMEC, 2005). The previously completed blast designs were modified to reflect this recommendation and are presented below.

12.1 Instantaneous Pressure Change for Canadian Fisheries Guidelines

The following sections are based on the previous work, and on new analyses to address additional concerns presented by Department of Fisheries and Oceans (DFO) and presented during the meetings in Baker Lake.

12.1.1 Blast Induced Vibration

Blast induced vibrations have the potential to reduce the stability and performance of nearby earthen structures such as dikes. Where saturated conditions exist within the foundation materials and within the earthen structural fills of the dewatering dikes and the tailings dike, blast induced vibrations could result in the development of increased pore water pressures within the foundation and structural fill materials. This could lead to potential settlement of the structures and consequently impact to the water retaining capacity of the dikes.

12.1.2 Estimates of Peak Particle Velocity

The effects of blasting are typically assessed in terms of Peak Particle Velocity (PPV). The preliminary estimates of PPV are based on the current understanding of the site layout, mine plan, and blast design. Changes to the current site layout, mine plan, and blast design will result in changes to the estimates of PPV. Certain site specific factors that are required to calculate PPV have been estimated based on published values. However, site specific parameters can only be determined by site vibration monitoring of actual blasts. Consequently, the actual PPV values may differ from those presented here.

The US Bureau of Mines has established that the peak particle velocity, PPV, is related to the scaled distance by the following relationship:

$$PPV = k * (R/W^{0.5})^{-b}$$

Where:

- PPV = Peak Particle Velocity, mm/s
- R = Distance from blast to point of concern, m
- W = Charge weight per delay, kg
- k = confinement factor – specific to site
- b = site factor

The constants k and b are specific to the site, and can be determined by blast vibration monitoring.

For this evaluation, a value of b = 1.6 was assumed. The PPV was evaluated for a range of values of confinement, ‘k’, of 400, 800, and 1500, for down hole blasting. This range in values is considered to be reasonable for the site and to provide an estimate of the sensitivity of PPV to different values of confinement.

Based on the current understanding of site conditions and blast monitoring experience at two other northern sites, the confinement value of 800 is expected to be the most likely representative value for average conditions at the site. The actual value for confinement can only be determined through a detailed field monitoring program.

12.1.3 Minimum Setback Distance for Canadian Fisheries Guidelines

Design guidelines governing the use of explosives adjacent to Canadian fisheries waters (Guidelines for the Use of Explosives in or Near Canadian Fisheries Waters; Wright and Hopky, 1998) indicate that no explosive is to be detonated that produces a peak particle velocity greater than 13 mm/s in a spawning bed during the period of egg incubation.

The PPV's were evaluated for the Second Portage Lake East Dike, the Third Portage Peninsula east shoreline, the Bay Dike, and the Goose Island east shoreline.

12.1.4 Setback Distance for Peak Particle Velocity

The minimum setback distances to achieve a Peak Particle Velocity, PPV, of 13 mm/s have been estimated for various values of confinement, 'k', and for four potential charge weights per delay. The following table summarizes the estimates of minimum setback required to achieve a PPV value of 13 mm/s.

TABLE 12.1: Minimum Setback Distance for 13 mm/s Peak Particle Velocity Guideline

k	**12 kg charge weight per delay (3 m bench, 76 mm hole)**	**77 kg charge weight per delay (6 m bench, 165 mm hole)**	**250 kg charge weight per delay, (12 m bench, decked charge, 229 mm hole)**
	Minimum Setback Distance to Achieve PPV = 13 mm/s		
400	30 m	75 m	135 m
800	46 m	115 m	208 m
1500	67 m	171 m	308 m

The relationships presented in the above table are shown on Figure 12.1 for the three different confinement values.

The proposed charge weight of 77 kg per hole on 6 m benches will result in PPV less than the required 13 mm/s.

For the portions of the dike or shoreline where the 13 mm/s guideline is exceeded, modified blast designs consisting of lower charge weights on lower bench heights have been shown to result in PPV that meet the guideline requirement. For example, the figure indicates that a charge weight of 12 kg would result in acceptable PPV in the area of concern. Alternatively, additional fill materials could be placed along the shoreline or dike upstream (lake side) face to increase the distance from the blasting area.

12.1.5 Minimum Setback Distance for Threshold Damage Levels

General guidelines for blasting nears dams indicate vibration damage thresholds on the order of 50 mm/s to be reasonable for dams having medium to dense sand or silts within the dam or foundation materials. The following table summarizes the estimates of minimum setback required to achieve a PPV value of 50 mm/s for the charge weights considered.

TABLE 12.2: Comparison of Minimum Setback Distance for a Peak Particle Velocity of 50 mm/s for Various Blast Configurations

k	12 kg charge weight per delay (3 m bench, 76 mm hole)	77 kg charge weight per delay (6 m bench, 165 mm hole)	250 kg charge weight per delay, (12 m bench, decked charge, 229 mm hole)
	Minimum Setback Distance to Achieve PPV = 50 mm/s		
400	13 m	32 m	58 m
800	20 m	50 m	89 m
1500	29 m	74 m	133 m

The analysis indicates that for the proposed 70 to 80 m toe setback for the dikes at the Meadowbank Project, and the proposed 77 kg charge weight, PPV of 50 mm/s will not be exceeded in the toe areas of the perimeter dikes or tailings dike for confinement values of 400 and 800, and may be exceeded for a confinement value of 1500 (see Figure 12.2). If necessary, blast designs can be modified so as not to exceed the maximum PPV of 50 mm/s for the toe region of these critical structures. During initial development at the project, a program of blast monitoring and blast optimization will be required to determine in-situ blasting parameters to re-assess the offset distances. It may be necessary in certain areas directly adjacent to structures to reduce the charge weight to achieve the desired PPV.

12.1.6 Instantaneous Pressure Change for Canadian Fisheries Guidelines – 100 kPa Criteria

The required setback distance for confined explosives to achieve the 100 kPa instantaneous pressure change guideline can be estimated from relationships presented in

"Guidelines for the Use of Explosives in or Near Canadian Fisheries Waters" (Wright and Hopky, 1998).

The following properties were used to assess the minimum setback distance.

TABLE 12.3: Properties Used to Assess Setback Distance for Instantaneous Pressure Change

Medium	Density, g/cm^3	Compressional Wave Velocity, cm/s
Water	1	146,300[1]
Rock (Intermediate Volcanic)	2.8	457,200[1]

1. Guidelines for the Use of Explosives in or Near Canadian Fisheries Waters; Wright and Hopky, 1998

Based on the above properties, the range of potential charge weights, and the range in confinement value, k, the following minimum setback distances, below which the 100 kPa overpressure guideline will not be exceeded, are estimated.

TABLE 12.4: Minimum Setback Distance for Instantaneous Pressure Change Guideline (<100kPa)

Charge Weight per Delay (kg)	Minimum Setback Distance (m)		
	k=400	k=800	k=1500
12	10 m	15 m	22 m
77	25 m	38 m	57 m
250	45 m	69 m	102 m

The results in the above table are presented on Figure 12.3 for the three confinements. For the proposed charge weight of 77 kg, the instantaneous pressure change will not exceed the guideline of 100 kPa on the outside of the dikes.

12.1.7 Instantaneous Pressure Change for Ice Covered Waters – 50 kPa Criteria

In addition to the legislated criteria, Department of Fisheries and Oceans has requested that Cumberland assess the effect of blast induced vibration and instantaneous overpressure resulting from blasting adjacent to waters during ice cover periods, although this is not currently legislated. For these conditions, Department of Fisheries has recommended an additional evaluation to consider an instantaneous pressure change of 50 kPa.

For the range of potential charge weights and for a range in confinement value, k, the following minimum setback distances, below which the 50 kPa overpressure guideline will not be exceeded, are estimated.

TABLE 12.5: Minimum Setback Distance for Instantaneous Pressure Change Guideline (50 kPa Ice Covered Water)

Charge Weight per Delay (kg)	Bench Height (m)	Hole Diameter (mm)	Minimum Setback Distance (m)		
			k=400	k=800	k=1500
12	3 m (ore)	76	15	23	34
77	6 m (ore and waste)	165	38	59	87
250	6 m (waste)	165	69	106	157

The relationships presented in the above table are shown on Figure 12.4 for the three confinement values. Based on the analysis, an instantaneous pressure change of 50 kPa will not be exceeded for the proposed 77 kg charge weight per hole and for a confinement value, ‘k’, of 800.

12.1.8 Conclusions

The following summarizes the conclusions of the previous and current assessment.

- With the exception of a short segment of shoreline adjacent to the southeast wall of the Portage Pit, the Peak Particle Velocity of 13 mm/s will not be exceeded for the proposed 77 kg charge weight per hole. This relates to fisheries guidelines.

- The Peak Particle Velocity of 50 mm/s will not be exceeded in the toe region of the perimeter dikes or tailings dike for the proposed 77 kg charge weight per hole for confinement values of 400 and 800, but may be exceeded for a confinement value of 1500. If necessary, the blast designs can be modified to achieve the required PPV. This relates to the structural stability of the dikes.

- The instantaneous pressure change along the upstream (lake side) face of the East Dike, Bay Zone Dike, and Goose Dike is predicted to be less than the 100 kPa guideline for the proposed 77 kg charge weight per hole. This relates to fisheries guidelines.

- The instantaneous pressure change along the upstream (lake side) face of the dikes during periods of ice cover is predicted to be less than 50 kPa for the proposed 77 kg charge weight per hole. This relates to an additional request by DFO to assess instantaneous pressure change for ice covered water conditions.

- For the Vault deposit, Peak Particle Velocity and instantaneous pressure change guidelines along the Vault Dike face will not be exceeded for any of the proposed blast designs or charge weights. The Vault Dike lies about 750 m from the nearest crest of the Vault Pit.

During initial development at the project, a program of blast monitoring and blast optimization will be required to determine in-situ blasting parameters in order to re-assess the offset distances. It may be necessary in certain areas directly adjacent to structures to reduce the charge weight to achieve the desired PPV.

13.0 OPERATIONAL CONSIDERATIONS

13.1 Blasting

The slope design criteria presented in this report are considered achievable, provided a high degree of care and attention is taken with respect to blast design and blasting practices. Uncontrolled blasting will result in damage behind design lines and will result in excessive damage to the rock mass which may contribute to local bench scale instability and excessive ravelling of material. This is particularly critical in regions of permafrost, where uncontrolled blast damage and fracturing of the bedrock could potentially result in increased ravelling of material due to seasonal freeze-thaw processes resulting in a deep active layer forming within the pit walls. Furthermore, gas pressures from bulk blasting operations can result in opening of ice filled joints, which could contribute to overall bench scale instability and ravelling. A program of carefully controlled blasting must be implemented in order to achieve the design criteria presented in this report. It will also be essential to carefully scale the bench faces in order to minimize subsequent ravelling. It is expected that the rocks at the Meadowbank Project will be amenable to pre-split and trim and buffer blasting techniques.

The analyses have shown that peak particle velocities and instantaneous overpressure can be effectively managed through the use of lighter charge weights, decreased blasthole diameters, and decreased operating bench heights, or a combination of these mitigative measures. Recommendations regarding controlled blasting procedures and preliminary designs for the purposes of costing and to provide a basis for initial test blasting can be provided if required.

13.2 Blast Monitoring

As part of the mine development, a vibration monitoring program will be required in order to measure the response of the dewatering dikes and tailings dike to pit blasting. The data from this program would be assessed in conjunction with continuous measurements from piezometers that would be installed in the dikes and within the dike foundation materials. From this analysis, the blasting could be adjusted to minimize the impact on the dikes. Mitigative measures to the blast design to minimize the development of blast induced vibration could include modifications to the blasthole patterns, reduction in blasthole size and hence charge weight in critical areas of the pit walls within a certain distance from the proposed dewatering and tailings dike, single blasthole initiation per delay, reduction in operating bench height in critical areas, or a combination of all these measures.

A more comprehensive program of blast vibration modelling and test blasting may be required during operations if blast vibration levels remain high and their frequency (cycles per second) is low.

13.3 Excavation and Scaling

In order to minimize ravelling and the development of shallow-seated plane and wedge instabilities on the benches, it is important to prevent undercutting of the slope during excavation by the equipment. Over-excavation, or “plucking” of blocks at the toe of the bench faces, may adversely affect the bench stability. Any loose blocks should be removed in a controlled manner by using equipment appropriate to the task of scaling the bench faces on the ultimate pit walls. This procedure will minimize ravelling and should enable safe access along benches for subsequent long-term slope monitoring.

13.4 Slope Depressurization

As discussed in this report, the slope stability modeling results indicate depressurization to be required in order to stabilize the southeast wall of the Goose Island Pit. Two piezometer installations with 4 transducers per installation are suggested immediately after stripping. To achieve a depressurization target of at least 20 m horizontally from the bench toe, 90 m long sub-horizontal drainholes (inclination of about 20 degrees) would be required. The areas requiring slope depressurization are shown on Figure 13.1. Piezometers installed into the slope will be used to monitor the actual depressurization requirements.

14.0 MONITORING AND INSTRUMENTATION

14.1 Adaptive Management and the Rational Pit Slope Design Process

The pit slope design process is a dynamic process that involves iterative steps of feedback loops and re-designs during two of the key stages of the project development, the feasibility stage and operating stage. Adaptive management strategies are implemented to continually evaluate and improve designs when the opportunity to collect new data and further refine design assumptions is available. During open pit excavation, the opportunity to collect new data is presented as the rock mass is exposed. Diligent geotechnical pit wall mapping of the first benches that are excavated can provide additional information about the rock mass that was not already known and therefore, would not have been used in the initial design. Furthermore, mapping will also provide confirmation of the assumptions made to assess the stability and deformation of the pit slopes. Examples of such data that would be collected for the Meadowbank project are joint persistence and termination criteria as well as any additional geotechnical mapping information such as small scale joint shape and large scale joint waviness. Information on the persistence and termination of critical joint sets may indicate that rock bridging is sufficient along these joints so that depressurization may not be necessary, or that the degree to which depressurization is required may be reduced. This could have a significant economic impact on the operations budget if the information can be incorporated into the operational phase of the project at an early stage.

14.2 Geotechnical Pit Wall Mapping

During development of the open pits, a program of systematic geotechnical pit wall mapping should be implemented to confirm the assumptions on which the designs have been based. The mapping programs should be initiated once stripping of the mining area has been completed. This will be of critical importance at the start of mining, as the assumptions made relating to the shallow joint set and to the orientation, continuity, and waviness of foliation and stratigraphy will need to be confirmed early in the mining process.

The data collected during these mapping programs should be compiled and reviewed on a regular basis so that adjustments may be made to the proposed slope design criteria as needed. Geotechnical mapping should include conventional mapping methods, and should consider alternative mapping methods, such as photogrammetric mapping, particularly in areas where access is limited or difficult or where safety concerns restrict accessibility. Particular aspects of the geotechnical mapping that will be of importance to long-term assessment of the stability of the pit will be the collection of large scale and small scale joint asperity information, rock bridge and joint spacing data, trace length data, and joint termination data. This information will not only be important to the assessment of the stability of the slopes, but will also provide valuable information for the on-going assessment of the hydrogeological aspects of the pit.

In addition to geotechnical mapping of the pit walls, a program of systematic mapping of any seepage within the pit should be implemented. Mapping of seepages should be undertaken at least twice annually so that the effect of seasonal temperature and hydrological changes to the pit inflows can be assessed.

14.3 Dike Instrumentation and Monitoring

Plans for instrumentation and monitoring of the dewatering dikes have been presented in the following report:

- Golder Associates Ltd. 2007a. Final Report Detailed Design of Dewatering Dikes, Meadowbank Gold Project. Submitted to Meadowbank Mining Corporation, March 2007

Plans for instrumentation and monitoring of the tailings dike have been presented in the following report:

- Golder Associates Ltd. 2007b. Final Report Detailed Design of Central Dike, Meadowbank Gold Project. Submitted to Meadowbank Mining Corporation, March 2007

The reader is referred to the above reports for details on the plans.

14.4 Open Pit Slope Monitoring and Instrumentation

- A series of slope monitoring instrumentation should be installed around the crest of the pit prior to, and during, excavation of the slopes.

- Geotechnical instrumentation is installed to monitor the behaviour of the pit slopes and pit crest during operations.

- Confirmation that the performance of the pit slopes and pit crest area is consistent with the predictions made during the design studies with respect to stability, deformation, seepage and stability analyses.

- Early warning of the development of potentially adverse trends such as slope movements or accelerations, excessive pore water pressures in response to blasting or other possible activities, seepage and/or deformation.

Several types of instruments have been selected to collect the required information, including vibrating wire piezometers, thermistors, and inclinometers.

TABLE 14.1: Geotechnical Instrumentation Summary for Open Pits

Instrumentation	Goose Pit	Portage Pit
Multi-level Piezometers and Thermistors	4	3
Slope Movement Indicators	8	15
Surface Prisms (30m spacing)	35	50
Surface Monuments (30m spacing)	35	50

A plan to install monitoring instrumentation prior to, and during, excavation of the slopes should be developed in collaboration with similar instrumentation plans for the proposed dikes.

Instrumentation should include the following:

- Prisms;
- Slope wireline extensometers and inclinometers;
- Vibrating wire piezometers; and
- Thermistors.

It is anticipated that the focus of the instrumentation program will be along the east, southeast, and south portions of the Goose Pit Wall, the south through southeast portion of the Portage Pit wall, and the west portion of the Portage Pit wall directly adjacent to the Tailings Dike. It is expected that shallow instrumentation behind the pit crest will be part of the dike monitoring program.

It is recommended that eight deep boreholes, completed as nested piezometers, be installed within 50 m of the crests of the Portage and Goose Pits. Seven of the boreholes would be drilled along the sections evaluated in the slope stability analysis, namely the Portage Northwest, Portage Southeast, Goose Northeast, Goose Southeast 11+00 (both east and west sides), Goose Southeast 12+00, and the Goose South sections

(Figure 14.1). One of the boreholes will be drilled in the Southwest of the Portage Pit, for data set completeness. The boreholes should be inclined sub-parallel to the pit walls and drilled to the depths summarized in the following table. Each borehole should be completed with transducers installed along the length of the borehole at the approximate elevations summarized in the table.

TABLE 14.2: Monitoring and Instrumentation

Analysis Section Or Pit Area	Depth of Monitoring Well below Ground Surface	Recommended Elevations for Transducer Installation
Portage Northwest	80 m	40 m, 80 m
Southwest of Portage Pit	125 m	6 m, 40 m, 80 m
Portage Southeast	125 m	6 m, 33 m, 54 m, 80 m
Goose Northeast	140 m	-10 m, 30 m, 70 m
Goose Southeast 11+00 (east and west side of pit)	140 m	-23 m, 0 m, 24 m, 48 m
Goose Southeast 12+00	160 m	-23 m, 0 m, 24 m, 48 m
Goose South	160 m	-30 m , 10 m, 50 m, 80m

Each piezometer drillhole should also have a TDR cable in it.

An array of survey monuments and survey prisms will be installed at 30 m to 50 m intervals around the crest of the pit and on selected benches to measure the vertical and lateral movements associated with possible wall deformations. Accurate measurements will require the use of high resolution surveying equipment and a network of stable control monuments. The crest prisms should be installed as soon as practical after completing the first 10 m bench.

Individual slope extensometers and inclinometers, or arrays of these instruments, should be targeted for critical areas such as the northeast through southeast portions of the Goose Pit and the south through southeast portion of the Portage Pit. Slope inclinometer casings should be installed from the pit crest areas, approximately 30 m back from the crest. Inclinometer casings will be installed in boreholes drilled parallel to the pit walls and to the proposed depth of the final pit. The instrumentation will be monitored to measure the magnitude and rate of horizontal displacement of rock forming the pit walls. The top of the inclinometer should also be surveyed to monitor horizontal deflection. The results of the measurements will be used to calibrate the slope deformation models for the open pits.

Blast monitoring will be carried out using portable blast monitoring seismographs that will measure both blast induced velocities and accelerations at the crest and/or the toe of the dike.

14.4.1 Monitoring Frequency

The frequency of monitoring will depend, to some degree, on the data collected and on the state of operation. The Operating Manual will be required to address the procedures required to change the monitoring frequencies. The following table summarizes the routine monitoring program during construction, during dewatering and operations.

TABLE 14.3: Geotechnical Instrumentation Monitoring Frequency

Instrumentation[1]	Monitored By	Reported To	Operations Frequency
Piezometers and thermistors	• Manually during construction by Contractor • Automatically by Contractor during dewatering; and • Automatically during operations	Construction Manager or Operations Manager and Engineer	Weekly for piezometers; Monthly for thermistors
Slope Inclinometers	• Manually by Contractor during construction and during dewatering • Manually by Construction Manager during operations	Construction Manager or Operations Manager and Engineer	Weekly changing to Monthly
Surface Monuments and Surface Prisms	• Manually by Contractor during construction and during dewatering • Manually by Construction Manager during operations	Construction Manager or Operations Manager and Engineer	Weekly
Seismographs	• Manually by Contractor during construction and during dewatering • Manually by Construction Manager during operations	Construction Manager or Operations Manager and Engineer	During blasting

1 All instrumentation to be monitored at installation

15.0 CLOSING REMARKS

The reader is referred to the "Important Information and Limitations of This Report" which follows the text but forms an integral part of this document.

The preceding document has presented an evaluation and review of the slope design criteria for Meadowbank Mining Corporations Meadowbank Project located in Nunavut. The document provides a comprehensive review and reporting of the slope design criteria for the various open pits and a geotechnical evaluation of the project including additional stability analyses to assess the setback distance between pit crest and dike toe for the proposed tailings dike and the proposed dewatering dikes. The report presents the results of detailed stability analyses of the proposed pit slopes in the Portage and Goose Island areas, in those sectors where water depths exceed 10 m locally, and where the dike structures are behind the pit walls.

The project consists of a series of gold deposits in close proximity to one another. It is planned to mine the deposits in a series of open pits using standard truck and shovel open pit mining methods. The project area is covered extensively by lakes and portions of the gold-bearing deposits trend off-shore, beneath the lakes. As a consequence, it will be necessary to construct a series of dewatering dikes to allow mining of the deposits where they extend off-shore. The interaction between the open pits and the dewatering dikes is a critical design aspect of the project particularly from the perspective of the safety of personnel working in the open pits.

The slope design criteria presented in the report were used in the Feasibility Study, and were developed on the basis of kinematic analyses and pseudo-probabilistic assessment of the structural controls on bench stability. The main structural controls are the foliation and stratigraphic contacts which dip variably to the west at angles from horizontal to up to 70 degrees, to horizontal, and systematic jointing. The foliation and stratigraphic contacts are considered to be persistent, while the jointing is considered to be non-persistent. The foliation and stratigraphic contacts will control multiple bench stability and potentially overall slope stability, whereas the non-persistent minor joint sets may result in local bench scale failures.

Additional limit equilibrium analyses and distinct element analyses have been carried out to assess the overall stability of the pit slopes as these relate to the stability of the toe region of the dewatering dikes and tailings dike. The results of the analyses indicate that under conservative assumptions of 0% rock bridge, equivalent to no effective cohesive strength along foliation and joint surfaces, the required factors of safety for the pit slope and for the dike toe at the current design setback distance are achievable for all slopes with the exception of the Goose Southeast slope, the Goose West slope, and the Portage

Southeast. Sensitivity analyses carried out for these section indicate that if the slope is depressurized, and assuming the most conservative condition of 0% rock bridge, the required safety factors are achievable. Furthermore, the analyses show that if a reasonable percentage of rock bridge (20% for the Goose Southeast slope and 5% for both the Goose West and Portage Southeast slopes) is included then the required safety factors are achievable even without depressurization. It is prudent to recommend slope depressurization for this slope because although 5 to 20% rock bridging is a reasonable assumption for rock bridge percentage, it cannot be quantified until mining has commenced.

Under pseudo-static (seismic loading) conditions, and for conservative assumptions of 0% rock bridge and the 1 in 10,000 earthquake event, all sections with the exception of Goose 12+00SE and Portage SE are predicted to achieve the required factors of safety for the pit wall and for the dike toe for the case of a naturally drained slope. For the Goose 12+00SE and Portage SE sections, depressurization of the pit walls in these areas is predicted to result in achievement of the required factors of safety.

Displacements at the inside dike toe along the Goose 12+00SE section were calculated. This section was selected because the dike toe and pit crest are at their closest separation, and the de-watering dike is at its maximum section height, and hence maximum water depth on the upstream side of the dike. Displacements at the dike toe are predicted to be less than 2.5 cm for the case of a naturally drained pit slope, and less that 1 cm for the case of the depressurized pit slope. Deformations in the pit slope are not predicted to impact the stability of the de-watering dikes or tailings dike.

During operations, as the pit is advanced, a program of detailed geotechnical mapping should be implemented immediately. If the mapping can confirm whether a sufficient percentage of rock bridge is occurring in a given slope, then a decision could be made to reduce or potentially eliminate slope depressurization measures. In addition to geotechnical mapping, an integrated system of slope monitoring instrumentation will need to be installed. This system should be integrated with slope monitoring system designed for the dewatering dikes and for the tailings dike.

The stability analyses presented in this report assume that careful blasting procedures will be used to develop the pit walls in order to minimize damage or disturbance to the walls. During initial mining of the starter pit, a program of blast monitoring and optimization should be undertaken to confirm assumptions made relating to site specific blast design parameters so that the blast designs can be refined accordingly.

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IMPORTANT INFORMATION AND LIMITATIONS OF THIS REPORT

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The information, recommendations and opinions expressed in this report are for the sole benefit of the Client. No other party may use or rely on this report or any portion thereof without Golder's express written consent. If the report was prepared to be included for a specific permit application process, then upon the reasonable request of the client, Golder may authorize in writing the use of this report by the regulatory agency as an Approved User for the specific and identified purpose of the applicable permit review process. Any other use of this report by others is prohibited and is without responsibility to Golder. The report, all plans, data, drawings and other documents as well as all electronic media prepared by Golder are considered its professional work product and shall remain the copyright property of Golder, who authorizes only the Client and Approved Users to make copies of the report, but only in such quantities as are reasonably necessary for the use of the report by those parties. The Client and Approved Users may not give, lend, sell, or otherwise make available the report or any portion thereof to any other party without the express written permission of Golder. The Client acknowledges that electronic media is susceptible to unauthorized modification, deterioration and incompatibility and therefore the Client can not rely upon the electronic media versions of Golder's report or other work products.

The report is of a summary nature and is not intended to stand alone without reference to the instructions given to Golder by the Client, communications between Golder and the Client, and to any other reports prepared by Golder for the Client relative to the specific site described in the report. In order to properly understand the suggestions, recommendations and opinions expressed in this report, reference must be made to the whole of the report. Golder can not be responsible for use of portions of the report without reference to the entire report.

Unless otherwise stated, the suggestions, recommendations and opinions given in this report are intended only for the guidance of the Client in the design of the specific project. The extent and detail of investigations, including the number of test holes, necessary to determine all of the relevant conditions which may affect construction costs would normally be greater than has been carried out for design purposes. Contractors bidding on, or undertaking the work, should rely on their own investigations, as well as their own interpretations of the factual data presented in the report, as to how subsurface conditions may affect their work, including but not limited to proposed construction techniques, schedule, and safety and equipment capabilities.

Soil, Rock and Groundwater Conditions: Classification and identification of soils, rocks, and geologic units have been based on commonly accepted methods employed in the practice of geotechnical engineering and related disciplines. Classification and identification of the type and condition of these materials or units involves judgment, and boundaries between different soil, rock or geologic types or units may be transitional rather than abrupt. Accordingly, Golder does not warrant or guarantee the exactness of the descriptions.

Special risks occur whenever engineering or related disciplines are applied to identify subsurface conditions and even a comprehensive investigation, sampling and testing program may fail to detect all or certain subsurface conditions. The environmental, geologic, geotechnical, geochemical and hydrogeologic conditions that Golder interprets to exist between and beyond sampling points may differ from those that actually exist. In addition to soil variability, fill of variable physical and chemical composition can be present over portions of the site or on adjacent properties. **The professional services retained for this project include only the geotechnical aspects of the subsurface conditions at the site, unless otherwise specifically stated and identified in the report.** The presence or implication(s) of possible surface and/or subsurface contamination resulting from previous activities or uses of the site and/or resulting from the introduction onto the site of materials from off-site sources are outside the terms of reference for this project and have not been investigated or addressed.

Soil and groundwater conditions shown in the factual data and described in the report are the observed conditions at the time of their determination or measurement. Unless otherwise noted, those conditions form the basis of the recommendations in the report. Groundwater conditions may vary between and beyond reported locations and can be affected by annual, seasonal and meteorological conditions. The condition of the soil, rock and groundwater may be significantly altered by construction activities (traffic, excavation, groundwater level lowering, pile driving, blasting, etc.) on the site or on adjacent sites. Excavation may expose the soils to changes due to wetting, drying or frost. Unless otherwise indicated the soil must be protected from these changes during construction.

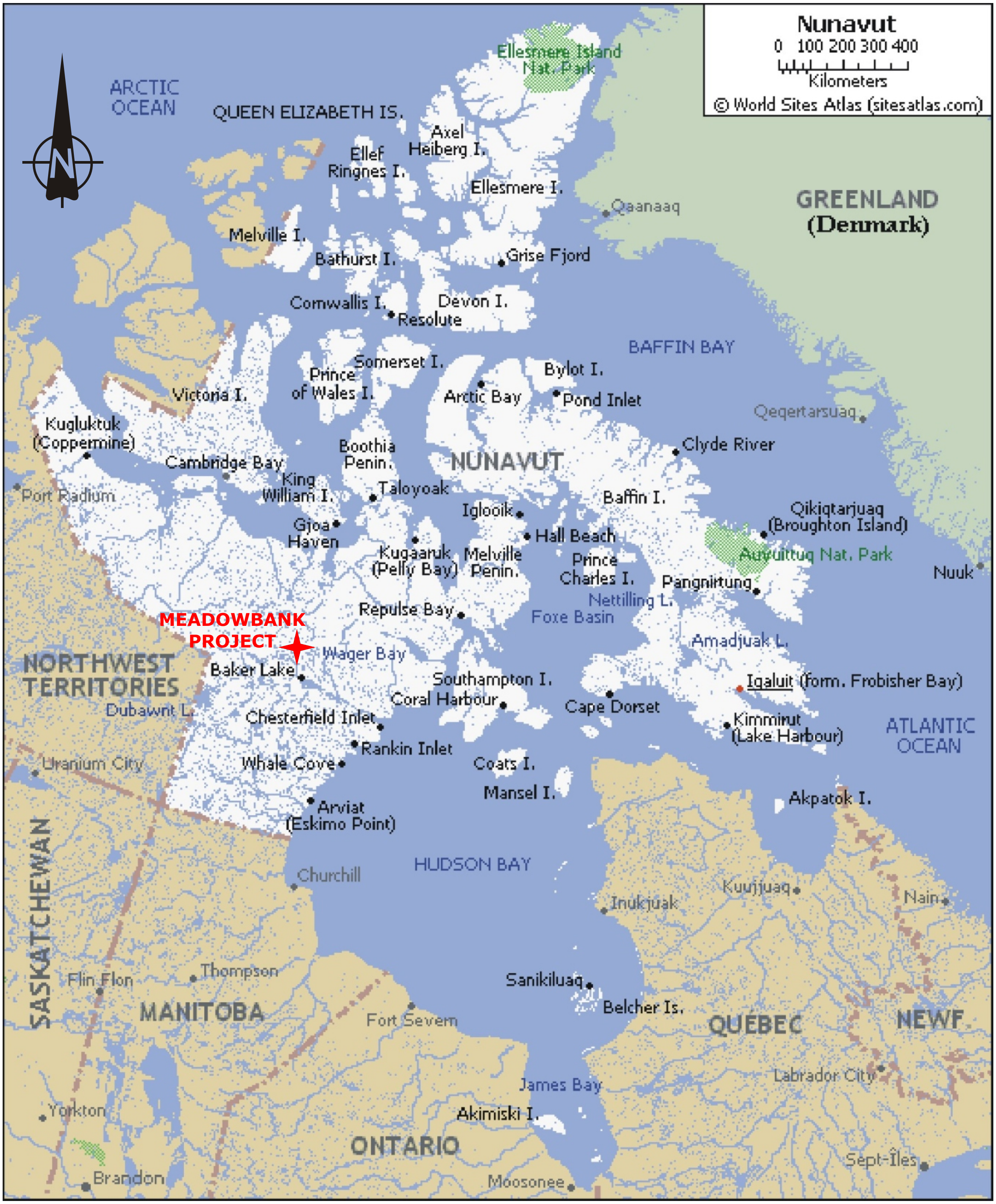
Sample Disposal: Golder will dispose of all uncontaminated soil and/or rock samples 90 days following issue of this report or, upon written request of the Client, will store uncontaminated samples and materials at the Client's expense. In the event that actual contaminated soils, fills or groundwater are encountered or are inferred to be present, all contaminated samples shall remain the property and responsibility of the Client for proper disposal.

Follow-Up and Construction Services: All details of the design were not known at the time of submission of Golder's report. Golder should be retained to review the final design, project plans and documents prior to construction, to confirm that they are consistent with the intent of Golder's report.

During construction, Golder should be retained to perform sufficient and timely observations of encountered conditions to confirm and document that the subsurface conditions do not materially differ from those interpreted conditions considered in the preparation of Golder's report and to confirm and document that construction activities do not adversely affect the suggestions, recommendations and opinions contained in Golder's report. Adequate field review, observation and testing during construction are necessary for Golder to be able to provide letters of assurance, in accordance with the requirements of many regulatory authorities. In cases where this recommendation is not followed, Golder's responsibility is limited to interpreting accurately the information encountered at the borehole locations, at the time of their initial determination or measurement during the preparation of the Report.

Changed Conditions and Drainage: Where conditions encountered at the site differ significantly from those anticipated in this report, either due to natural variability of subsurface conditions or construction activities, it is a condition of this report that Golder be notified of any changes and be provided with an opportunity to review or revise the recommendations within this report. Recognition of changed soil and rock conditions requires experience and it is recommended that Golder be employed to visit the site with sufficient frequency to detect if conditions have changed significantly.

Drainage of subsurface water is commonly required either for temporary or permanent installations for the project. Improper design or construction of drainage or dewatering can have serious consequences. Golder takes no responsibility for the effects of drainage unless specifically involved in the detailed design and construction monitoring of the system.



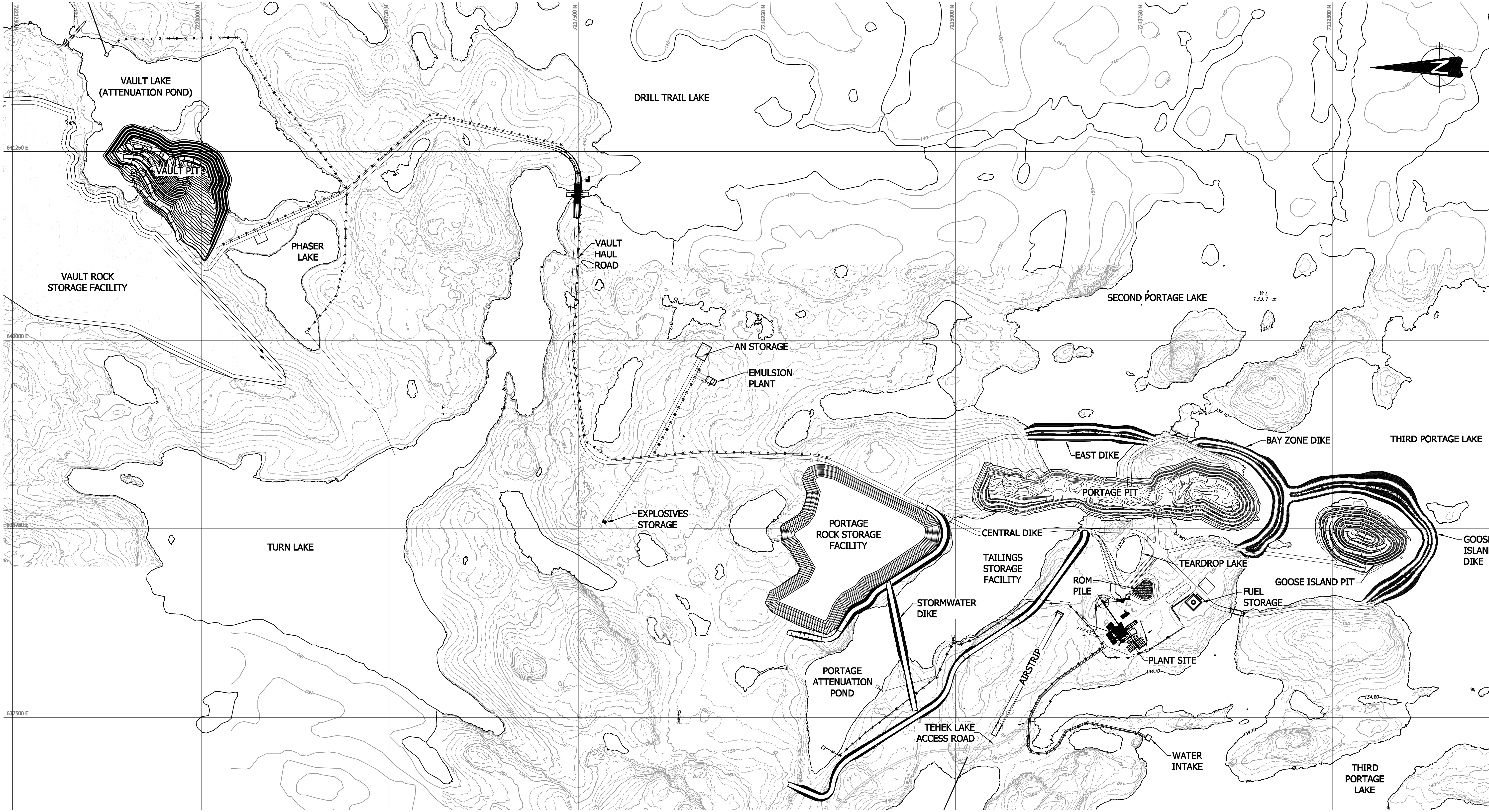
DRAWING DATE: 22-Feb-07 COREL FILE: N:\Bur-Graphics\Projects\2006\1413\06-1413-089\5000\Drafting\4000\061413089-5000-00.cdr

PROJECT	MEADOWBANK MINING CORPORATION
TITLE	LOCATION PLAN

Golder Associates

PROJECT No.	06-1413-089		FILE No.	5000-00
DESIGN	ES	22FEB07	SCALE	AS SHOWN REV.
CADD	AS	22FEB07		
CHECK	--	--		
REVIEW				

FIGURE 1.1



0 250 500 750 1000 1250

Scale in Meters

NOTES:

1) ALL DIMENSIONS ARE IN METRES UNLESS OTHERWISE NOTED.
2) ALL ELEVATIONS ARE IN METRES ABOVE SEA LEVEL (MASL), UNLESS OTHERWISE NOTED.
3) GRID REFERENCE: NAD 83, UTM ZONE 14
4) CONTOUR INFORMATION ON LAND SUPPLIED BY MEADOWBANK MINING CORPORATION.
5) CONTOURS BELOW LAKE SURFACE ARE BASED ON BATHYMETRIC SURVEYS BY GOLDER ASSOCIATES LTD., 2002, 2003, 2006.
6) LAKE CONTOURS ARE BASED ON REGIONAL PLAN MAPS OF LAKE SURFACE ELEVATIONS: 2ND PORTAGE LAKE = 133.1M, 3RD PORTAGE LAKE = 134.1M

LEGEND:

120 LAND-BASED TOPOGRAPHIC MAJOR CONTOURS

LAND-BASED TOPOGRAPHIC MINOR CONTOURS

PROJECT

MEADOWBANK
MINING CORPORATION

TITLE

MEADOWBANK GOLD PROJECT
OVERALL SITE PLAN

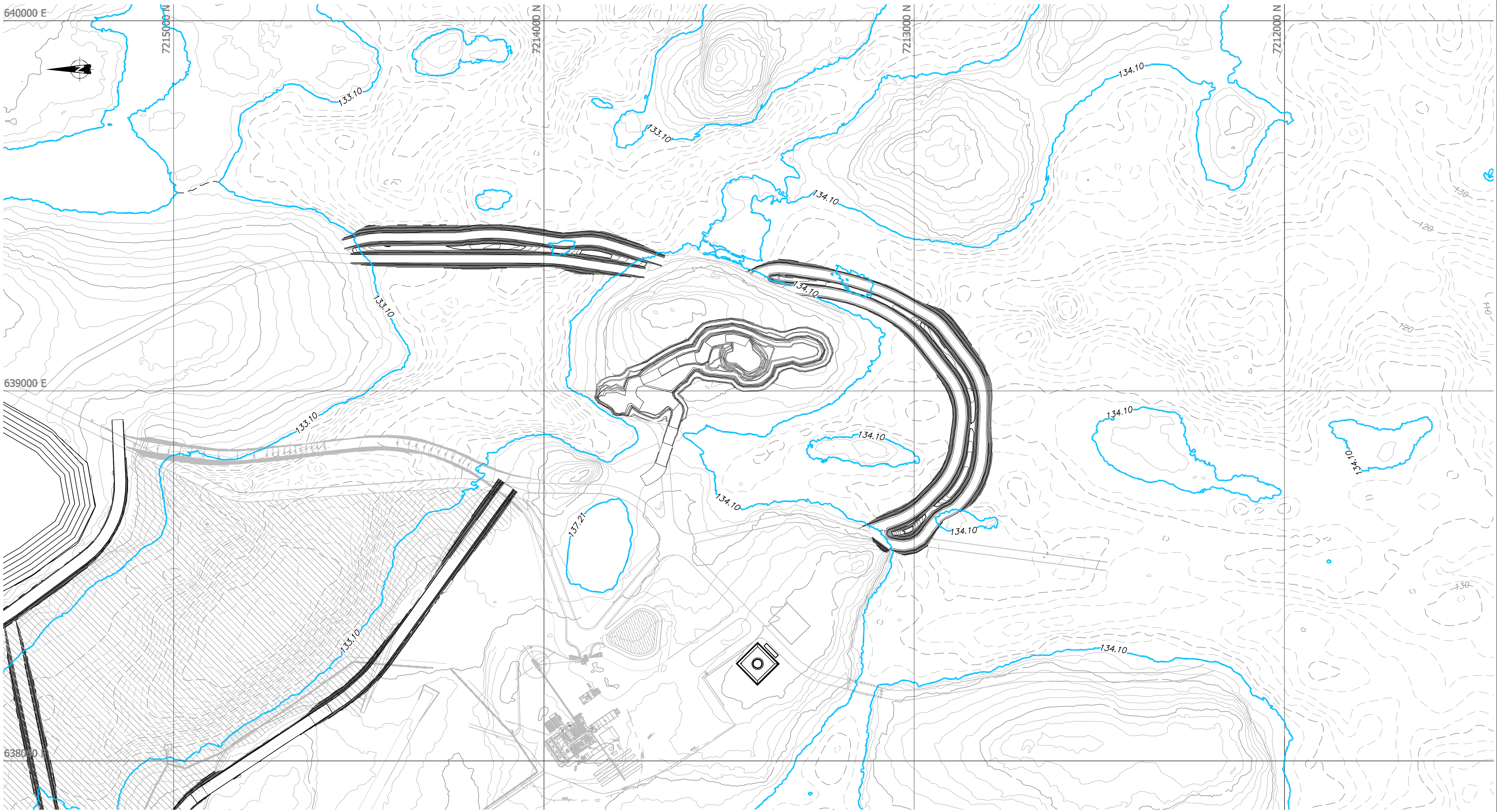
Golder Associates

PROJECT No.	06-1413-089	
DESIGN	SA	15FEB07
CADD	EA/JK	15MAR07
CHECK	-	-
REVIEW	-	-

FILE No.	061413089-5000-4000-08		
SCALE	AS SHOWN	REV.	A

FIGURE 1.2

REVISION DATE: 07/04/04 05:13PM By: ASalvador CADD FILE: N:\Bur-Graphics\Projects\2006\1413\06-1413-089\5000\Drafting\4000\061413089-5000-4000-08.dwg



LEGEND

- LAKE SHORELINE CONTOUR
- LAND - BASED MAJOR CONTOUR
- LAND - BASED MINOR CONTOUR
- PIT DESIGN MAJOR CONTOUR
- PIT DESIGN MINOR CONTOUR
- BATHYMETRY MAJOR CONTOUR
- BATHYMETRY MINOR CONTOUR

NOTES

1. ALL DIMENSIONS ARE IN METRES UNLESS OTHERWISE NOTED.
2. ALL ELEVATIONS ARE IN METERS ABOVE SEA LEVEL (MASL), UNLESS OTHERWISE NOTED.
3. GRID REFERENCE: NAD 83, UTM ZONE 14.
4. CONTOUR INFORMATION ON LAND SUPPLIED BY CUMBERLAND RESOURCES LTD.
5. CONTOUR BELOW LAKE SURFACE ARE BASED ON BATHYMETRIC SURVEYS BY GOLDER ASSOCIATES LTD., 2006. CONTOURS INTERVAL= 2m.
6. BATHYMETRY CONTOUR DATA SUBJECT TO FUTURE UPDATE.
7. LAKE CONTOURS ARE BASED ON REGIONAL PLAN MAPS OF LAKE SURFACE ELEVATIONS: 2nd PORTAGE LAKE= 133.1m, 3rd PORTAGE LAKE= 134.1m.

0 100 200 300 400 500
Scale in Meters

PROJECT
MEADOWBANK MINING CORPORATION

TITLE
MEADOWBANK GOLD PROJECT STARTER PIT YEAR (-1)

Golder Associates

PROJECT No.	06-1413-089		FILE No. 061413089-5000-3000-05
DESIGN	ES	12FEB07	SCALE AS SHOWN REV. A
CADD	EA	12FEB07	
CHECK	-	-	
REVIEW	-	-	

FIGURE 2.1

CADD FILE: N:\Bur-Graphics\Projects\2006\1413\06-1413-089\5000\Drafting\3000\061413089-5000-3000-05.dwg

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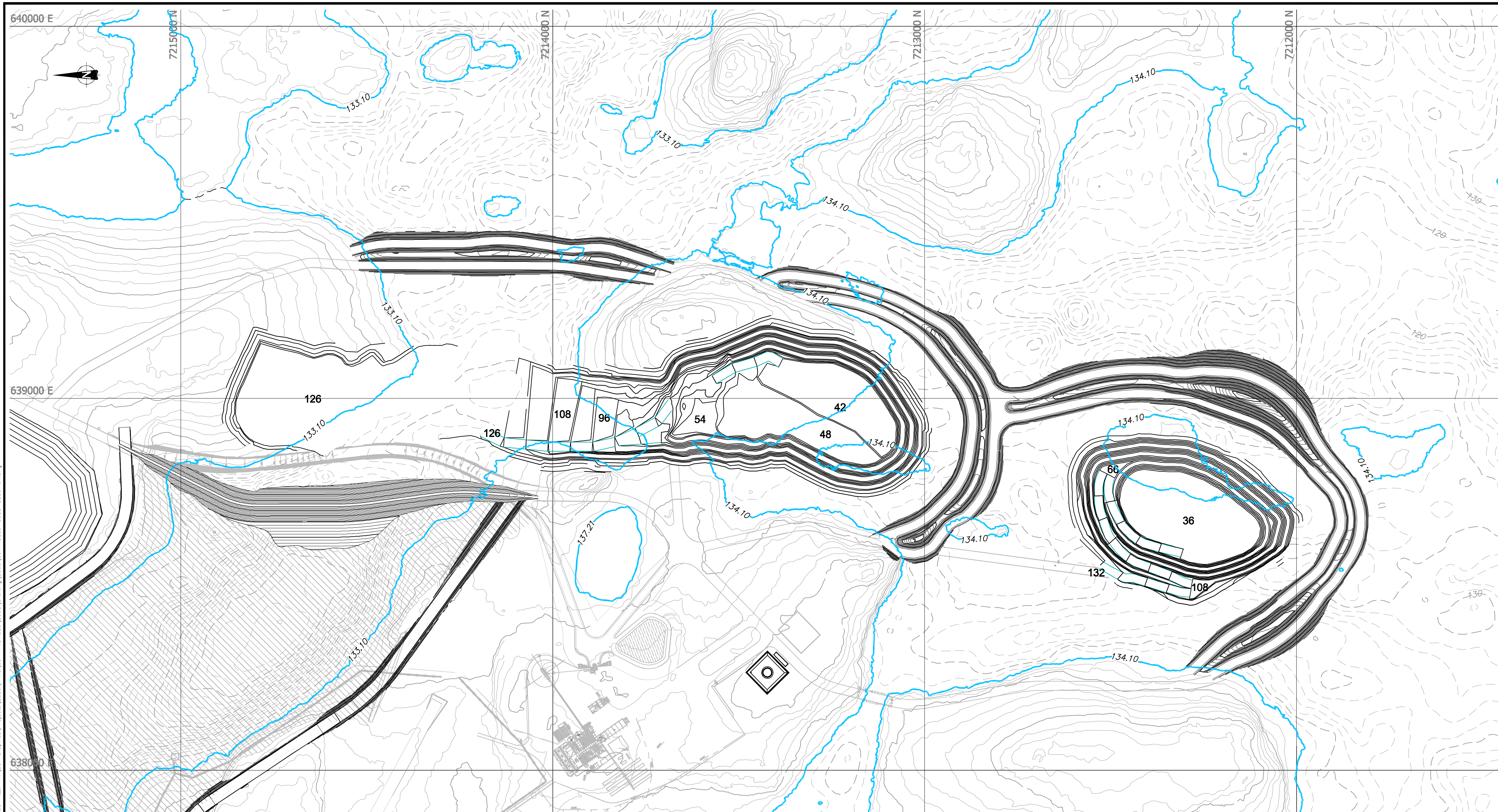


LEGEND

- LAKE SHORELINE CONTOUR
- LAND - BASED MAJOR CONTOUR
- LAND - BASED MINOR CONTOUR
- PIT DESIGN MAJOR CONTOUR
- PIT DESIGN MINOR CONTOUR
- BATHYMETRY MAJOR CONTOUR
- BATHYMETRY MINOR CONTOUR

NOTES

1. ALL DIMENSIONS ARE IN METRES UNLESS OTHERWISE NOTED.
2. ALL ELEVATIONS ARE IN METERS ABOVE SEA LEVEL (MASL), UNLESS OTHERWISE NOTED.
3. GRID REFERENCE: NAD 83, UTM ZONE 14.
4. CONTOUR INFORMATION ON LAND SUPPLIED BY CUMBERLAND RESOURCES LTD.
5. CONTOUR BELOW LAKE SURFACE ARE BASED ON BATHYMETRIC SURVEYS BY GOLDER ASSOCIATES LTD., 2006. CONTOURS INTERVAL= 2m.
6. BATHYMETRY CONTOUR DATA SUBJECT TO FUTURE UPDATE.
7. LAKE CONTOURS ARE BASED ON REGIONAL PLAN MAPS OF LAKE SURFACE ELEVATIONS: 2nd PORTAGE LAKE= 133.1m, 3rd PORTAGE LAKE= 134.1m.

PROJECT

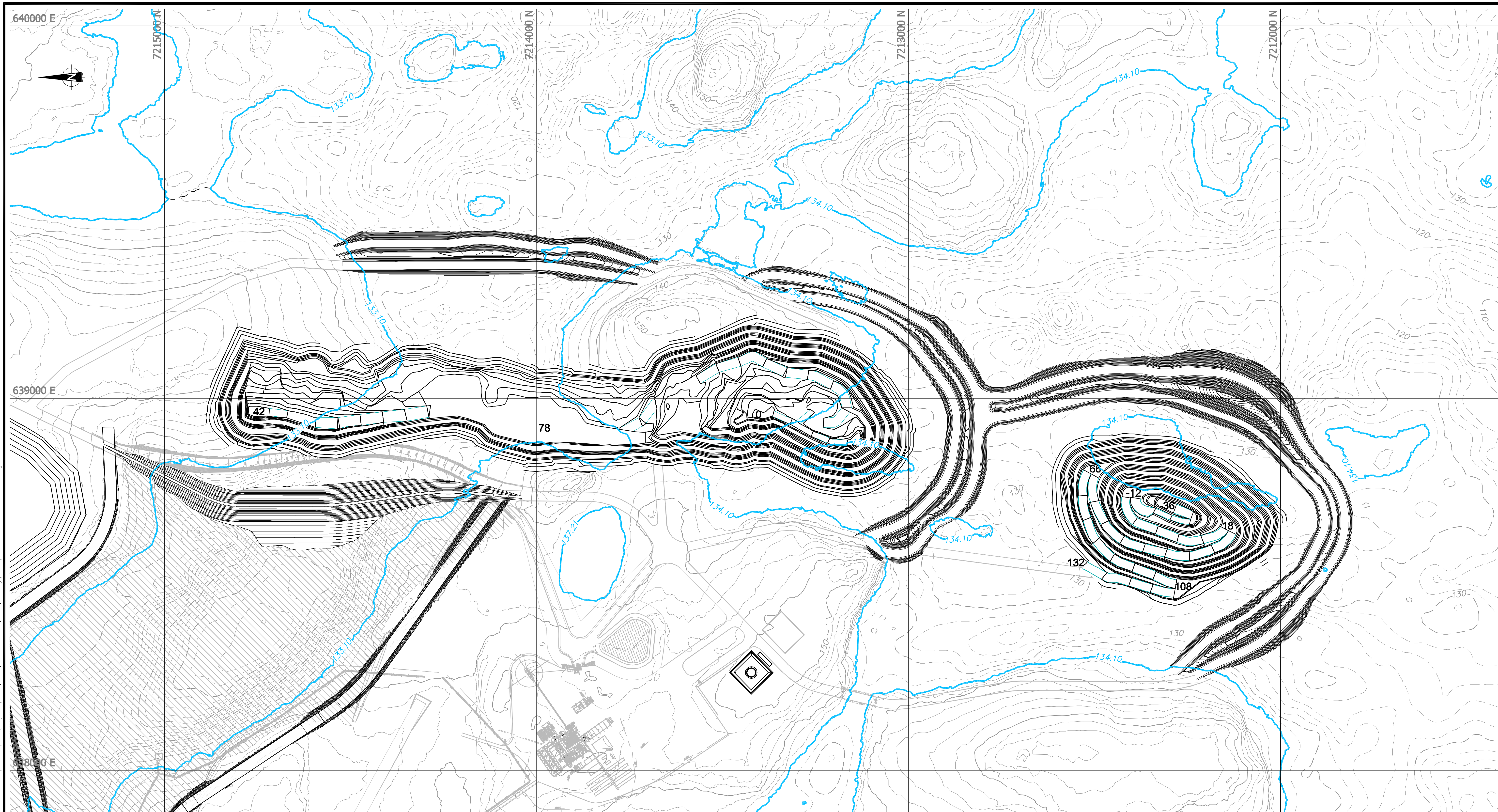
MEADOWBANK MINING CORPORATION

TITLE

MEADOWBANK GOLD PROJECT GOOSE ISLAND PIT & PORTAGE PIT - YEAR 3

Golder Associates

PROJECT No.		06-1413-089	FILE No.	061413089-5000-3000-01
DESIGN	ES	12FEB07	SCALE AS SHOWN	REV. A
CADD	EA	12FEB07	FIGURE 2.2	
CHECK	-	-		
REVIEW	-	-		



LEGEND

- LAKE SHORELINE CONTOUR
- LAND - BASED MAJOR CONTOUR
- LAND - BASED MINOR CONTOUR
- PIT DESIGN MAJOR CONTOUR
- PIT DESIGN MINOR CONTOUR
- BATHYMETRY MAJOR CONTOUR
- BATHYMETRY MINOR CONTOUR

NOTES

1. ALL DIMENSIONS ARE IN METRES UNLESS OTHERWISE NOTED.
2. ALL ELEVATIONS ARE IN METERS ABOVE SEA LEVEL (MASL), UNLESS OTHERWISE NOTED.
3. GRID REFERENCE: NAD 83, UTM ZONE 14.
4. CONTOUR INFORMATION ON LAND SUPPLIED BY CUMBERLAND RESOURCES LTD.
5. CONTOUR BELOW LAKE SURFACE ARE BASED ON BATHYMETRIC SURVEYS BY GOLDER ASSOCIATES LTD., 2006. CONTOURS INTERVAL= 2m.
6. BATHYMETRY CONTOUR DATA SUBJECT TO FUTURE UPDATE.
7. LAKE CONTOURS ARE BASED ON REGIONAL PLAN MAPS OF LAKE SURFACE ELEVATIONS: 2nd PORTAGE LAKE= 133.1m, 3rd PORTAGE LAKE= 134.1m.

0 100 200 300 400 500

Scale in Meters

PROJECT

MEADOWBANK
MINING CORPORATION

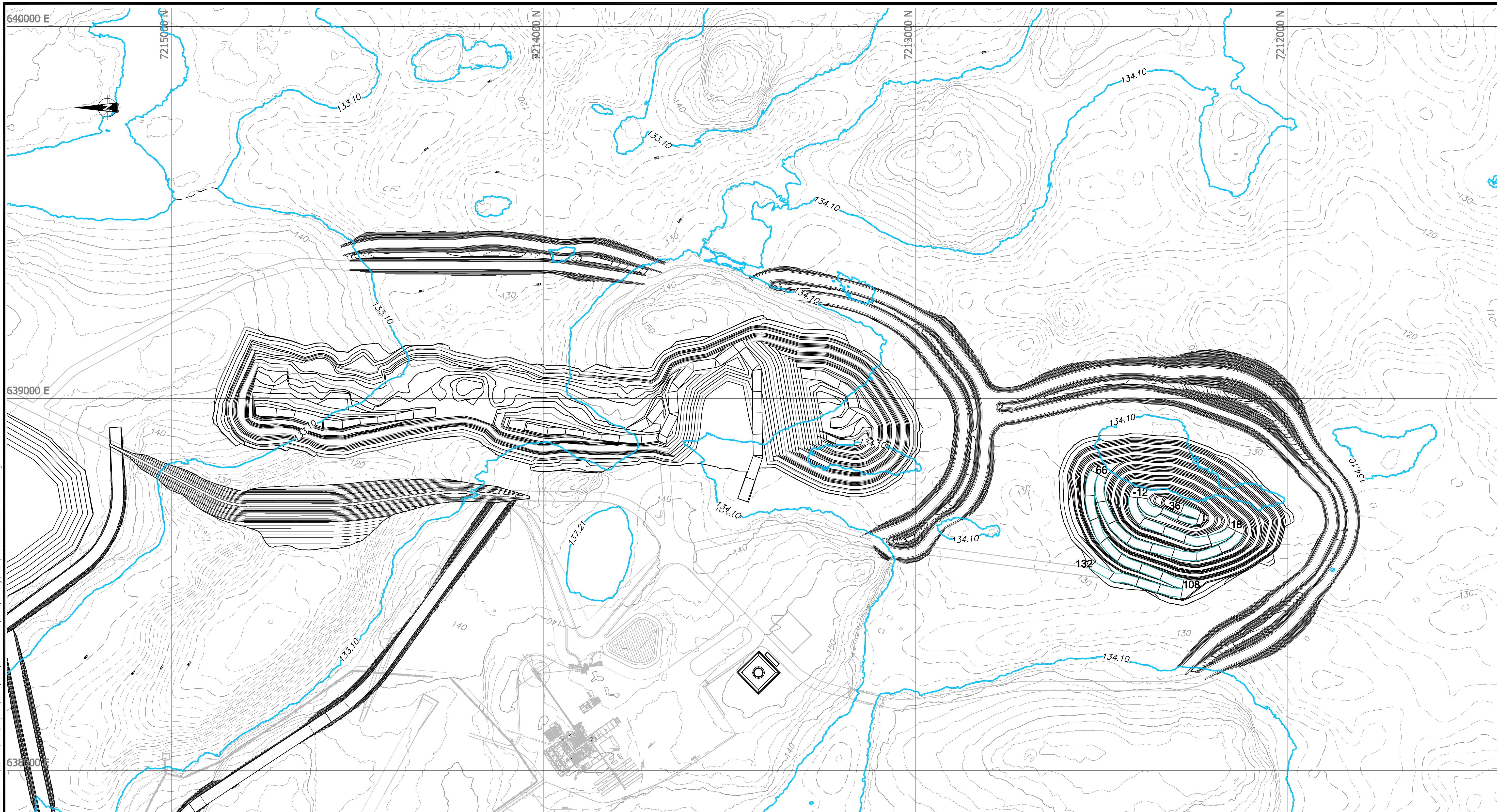
TITLE

MEADOWBANK GOLD PROJECT
GOOSE ISLAND PIT & PORTAGE PIT
- YEAR 5

Golder Associates

PROJECT No.		06-1413-089	FILE No.	061413089-5000-3000-02
DESIGN	ES	12FEB07	SCALE	AS SHOWN REV. A
CADD	EA	12FEB07		
CHECK	-	-	**FIGURE 2.3**	
REVIEW	-	-		

REVISION DATE: 07/04/05 12:44PM By: ASalvador CADD FILE: N:\Bur-Graphics\Projects\2006\1413\06-1413-089\5000\Drafting\3000\061413089-5000-3000-02.dwg



LEGEND

- LAKE SHORELINE CONTOUR
- LAND - BASED MAJOR CONTOUR
- LAND - BASED MINOR CONTOUR
- PIT DESIGN MAJOR CONTOUR
- PIT DESIGN MINOR CONTOUR
- BATHYMETRY MAJOR CONTOUR
- BATHYMETRY MINOR CONTOUR

NOTES

1. ALL DIMENSIONS ARE IN METRES UNLESS OTHERWISE NOTED.
2. ALL ELEVATIONS ARE IN METERS ABOVE SEA LEVEL (MASL), UNLESS OTHERWISE NOTED.
3. GRID REFERENCE: NAD 83, UTM ZONE 14.
4. CONTOUR INFORMATION ON LAND SUPPLIED BY CUMBERLAND RESOURCES LTD.
5. CONTOUR BELOW LAKE SURFACE ARE BASED ON BATHYMETRIC SURVEYS BY GOLDER ASSOCIATES LTD., 2006. CONTOURS INTERVAL= 2m.
6. BATHYMETRY CONTOUR DATA SUBJECT TO FUTURE UPDATE.
7. LAKE CONTOURS ARE BASED ON REGIONAL PLAN MAPS OF LAKE SURFACE ELEVATIONS: 2nd PORTAGE LAKE= 133.1m, 3rd PORTAGE LAKE= 134.1m.

PROJECT

MEADOWBANK MINING CORPORATION

TITLE

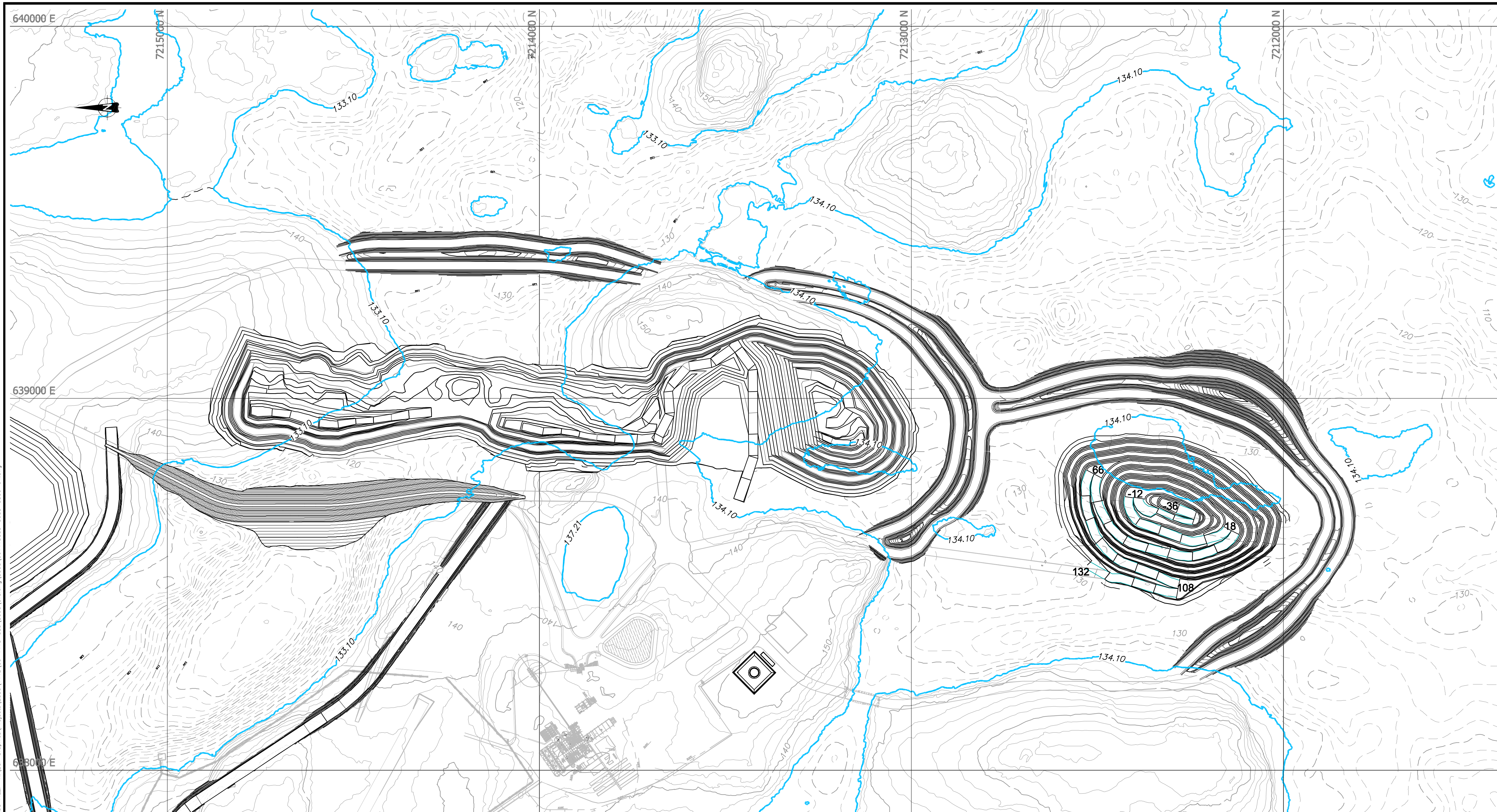
**MEADOWBANK GOLD PROJECT
GOOSE ISLAND PIT & PORTAGE PIT
- YEAR 7**

Golder Associates

PROJECT No.		06-1413-089	FILE No.	061413089-5000-3000-03
DESIGN	ES	12FEB07	SCALE	AS SHOWN REV. A
CADD	EA	12FEB07		
CHECK	-	-		
REVIEW	-	-		

FIGURE 2.4

REVISION DATE: 07/04/05 12:44PM By: ASalvador CADD FILE: N:\Bur-Graphics\Projects\2006\1413\06-1413-089\5000\Drafting\3000\061413089-5000-3000-03.dwg



LEGEND

- LAKE SHORELINE CONTOUR
- LAND - BASED MAJOR CONTOUR
- LAND - BASED MINOR CONTOUR
- PIT DESIGN MAJOR CONTOUR
- PIT DESIGN MINOR CONTOUR
- BATHYMETRY MAJOR CONTOUR
- BATHYMETRY MINOR CONTOUR

NOTES

1. ALL DIMENSIONS ARE IN METRES UNLESS OTHERWISE NOTED.
2. ALL ELEVATIONS ARE IN METERS ABOVE SEA LEVEL (MASL), UNLESS OTHERWISE NOTED.
3. GRID REFERENCE: NAD 83, UTM ZONE 14.
4. CONTOUR INFORMATION ON LAND SUPPLIED BY CUMBERLAND RESOURCES LTD.
5. CONTOUR BELOW LAKE SURFACE ARE BASED ON BATHYMETRIC SURVEYS BY GOLDER ASSOCIATES LTD., 2006. CONTOURS INTERVAL= 2m.
6. BATHYMETRY CONTOUR DATA SUBJECT TO FUTURE UPDATE.
7. LAKE CONTOURS ARE BASED ON REGIONAL PLAN MAPS OF LAKE SURFACE ELEVATIONS: 2nd PORTAGE LAKE= 133.1m, 3rd PORTAGE LAKE= 134.1m.

0 100 200 300 400 500
Scale in Meters

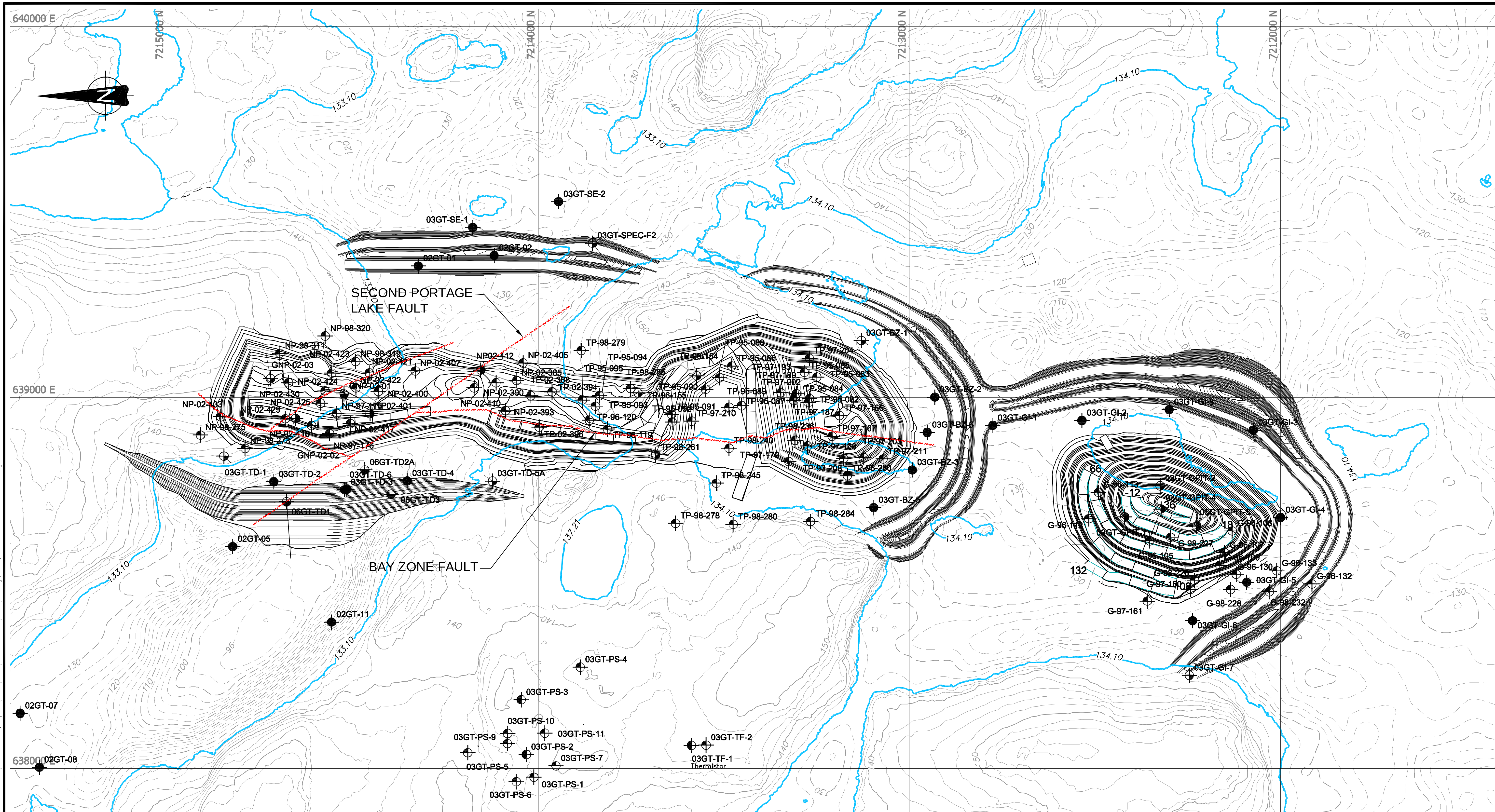
PROJECT
MEADOWBANK MINING CORPORATION

TITLE
MEADOWBANK GOLD PROJECT GOOSE ISLAND PIT & PORTAGE PIT - YEAR 9

Golder Associates

PROJECT No.		06-1413-089	FILE No. 061413089-5000-3000-04	
DESIGN	ES	12FEB07	SCALE AS SHOWN	REV. A
CADD	EA	12FEB07	FIGURE 2.5	
CHECK	-	-		
REVIEW	-	-		

REVISION DATE: 07/04/05 12:44PM By: ASalvador CADD FILE: N:\Bur-Graphics\Projects\2006\1413\06-1413-089\5000\Drafting\3000\061413089-5000-3000-04.dwg



LEGEND

Azimuth of Drill Path

Fault

TP-98-230 — Exploration Geotechnical Borehole

GNP-02-03 — Oriented Geotechnical Borehole with Thermistor

GNP-02-02 — Oriented Geotechnical Borehole with Packer Testing

GNP-02-01 — Oriented Geotechnical Borehole with Packer Testing and Thermistor

03GT-BZ-5 — Non Oriented Geotechnical Borehole with Packer Testing

02GT-06 — Lost Borehole

NOTES

1) All Pit Crest and Dike outlines are proposed.
2) Horizontal Datum NAD83
Projection: UTM Zone 14N

0 100 200 300 400 500
Scale in Meters

PROJECT

MEADOWBANK MINING CORPORATION

TITLE

INVESTIGATION GEOTECHNICAL BOREHOLES

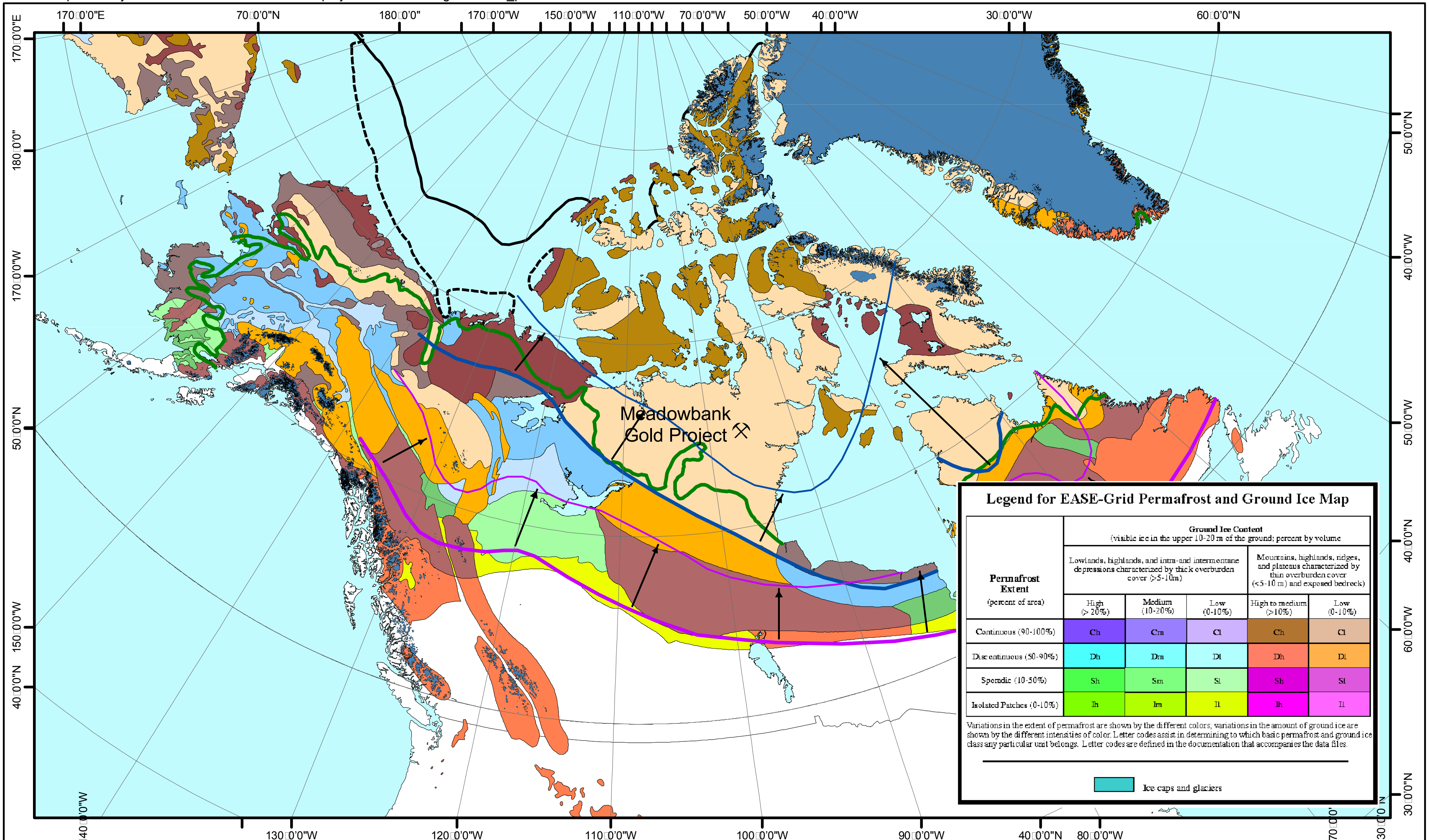
Golder Associates

PROJECT No.	06-1413-089	FILE No. 061413089-5000-3000-06	
DESIGN	JFG	22MAR07	SCALE AS SHOWN REV. A
CADD	AS	22MAR07	FIGURE 3.1
CHECK	-	-	
REVIEW	-	-	

CADD FILE: N:\Bur-Graphics\Projects\2006\1413\06-1413-089\5000\Drafting\3000\061413089-5000-3000-06.dwg

REVISION DATE: 07/04/05 07:34PM By: ASalvador

N:\Bur-Graphics\Projects\2006\1413\06-1413-089\GIS\projects\5000\4000\figure-03-02_permafrost.mxd



LEGEND

- Treeline
- Sea-ice Edge Limit
- Subsea Permafrost Limit
- Southern boundary of discontinuous permafrost - Present
- Southern boundary of discontinuous permafrost - Predicted
- Southern boundary of continuous permafrost - Present
- Southern boundary of continuous permafrost - Predicted
- Predicted movement of permafrost boundaries

0 300 600 1,200 Kilometers

Scale - 1:30,000,000

REFERENCE

Brown, J., O.J. Ferrians Jr., J.A. Heginbottom, and E.S. Melnikov. 1998. Circum-Arctic Map of Permafrost and Ground-Ice Conditions. Boulder, CO: National Snow and Ice Data Center/World Data Center for Glaciology. Digital Media.
Predicted permafrost boundaries based on Woo et al., 1992.

PROJECTION: Lambert Azimuthal Equal Area

PROJECT

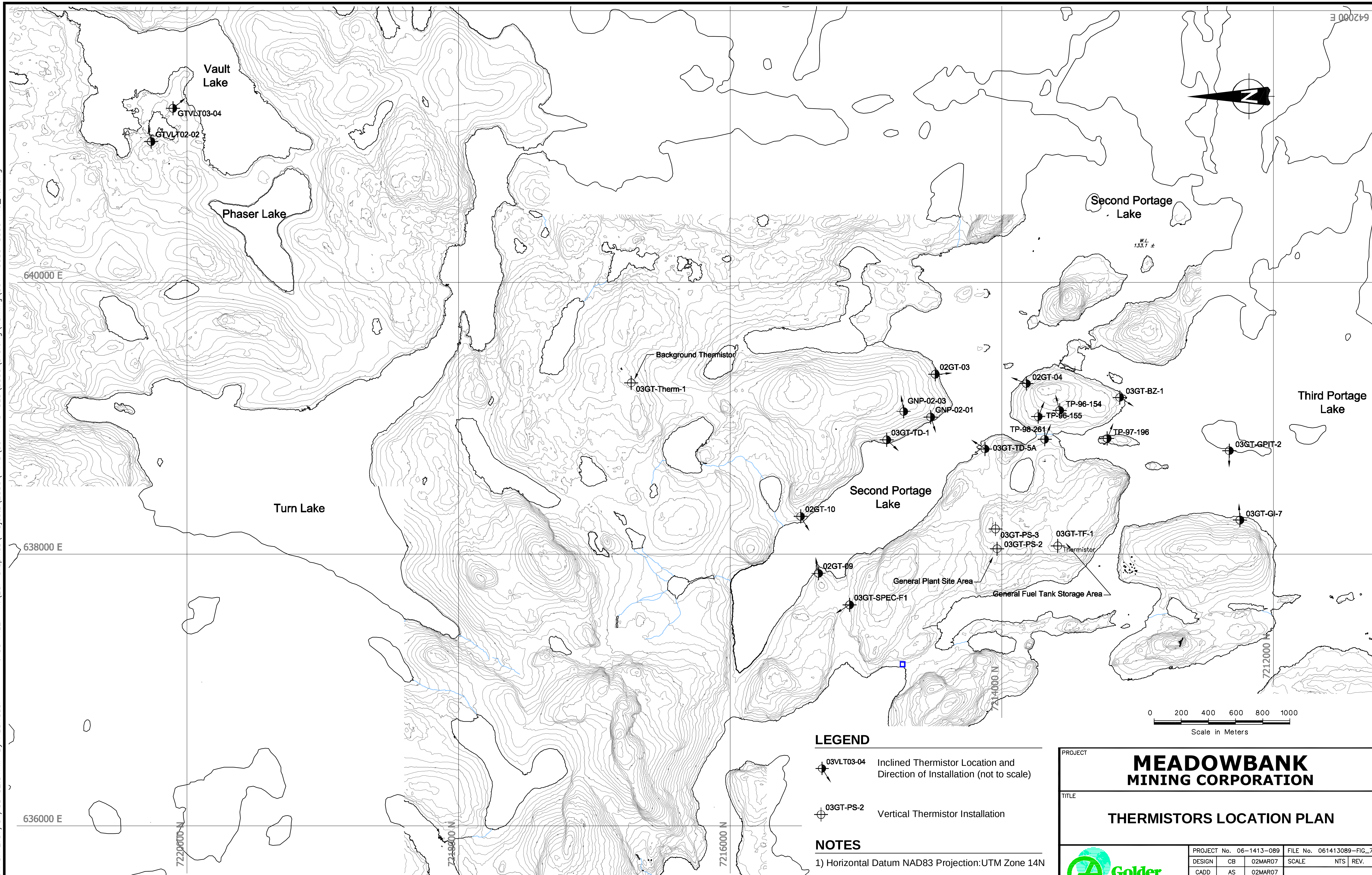
MEADOWBANK
MINING CORPORATION

TITLE

PERMAFROST MAP OF CANADA
MEADOWBANK GOLD PROJECT

Golder Associates
Burnaby, B.C.

PROJECT No. 06-1413-089			SCALE AS SHOWN	REV. 0
DESIGN	CJC	13 Mar. 2007	**FIGURE 3.2**	
GIS	CDB	13 Mar. 2007		
CHECK				
REVIEW				



LEGEND

03VLT03-04 Inclined Thermistor Location and Direction of Installation (not to scale)

03GT-PS-2 Vertical Thermistor Installation

NOTES

1) Horizontal Datum NAD83 Projection:UTM Zone 14N

PROJECT

MEADOWBANK MINING CORPORATION

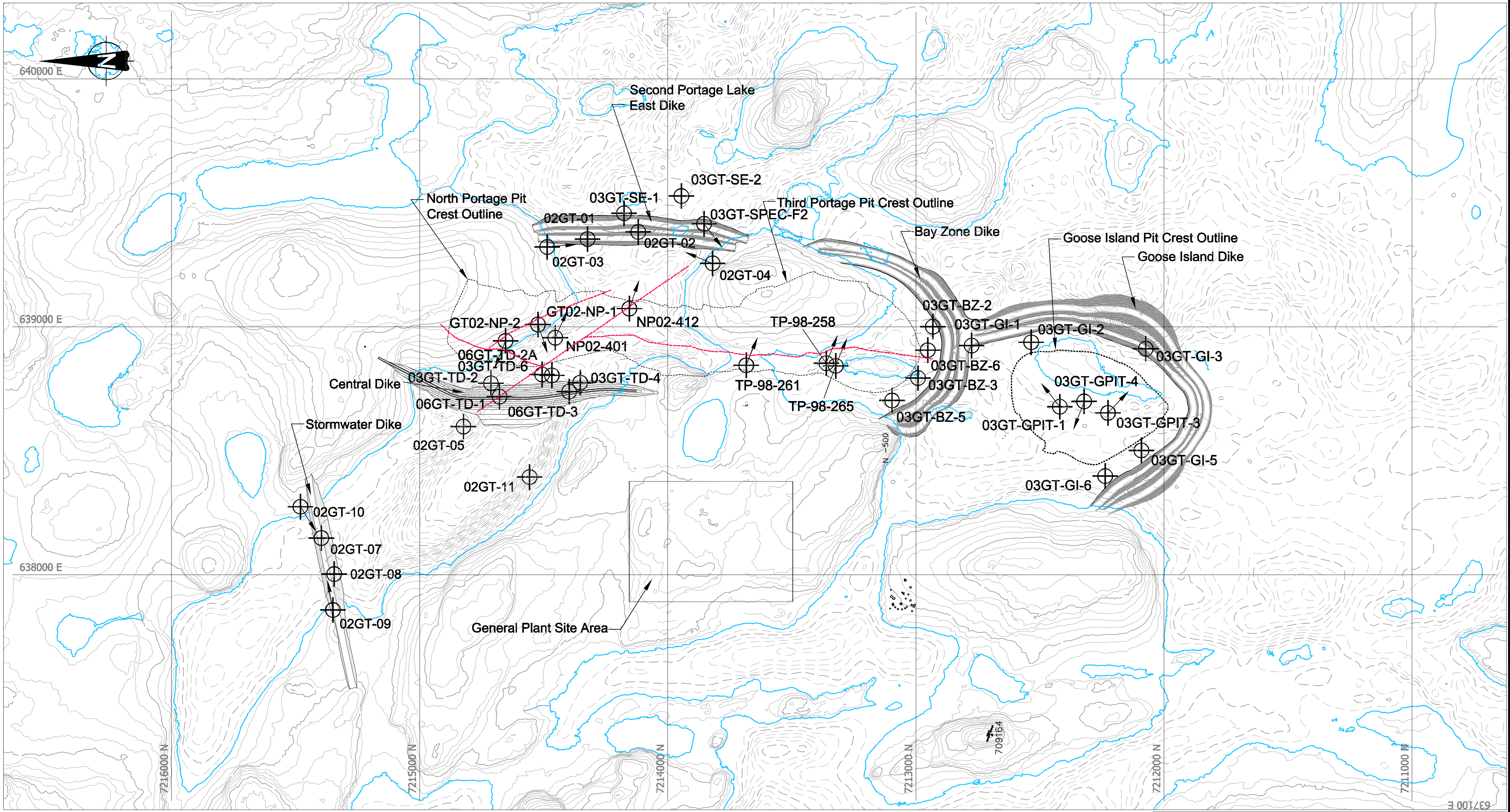
TITLE

THERMISTORS LOCATION PLAN

Golder Associates

PROJECT No. 06-1413-089		FILE No. 061413089-FIG_7.4	
DESIGN	CB	02MAR07	SCALE NTS REV.
CADD	AS	02MAR07	
CHECK			**FIGURE 3.3**
REVIEW			

REVISION DATE: 07/04/05 02:04PM By: ASalvador CADD FILE: N:\Bur-Graphics\Projects\2006\1413\06-1413-089\5000\Drafting\working\061413089-5000-FIG_7.4.dwg



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By: ASalvador
REVISION DATE: 07/04/05 02:04PM

LEGEND

- Azimuth of Drill Path
- Fault
- GNP-02-01 Borehole with Hydraulic Conductivity Testing
- Bathymetric Contour
- Topographic Contour

NOTES

1) All Pit Crest and Dike outlines are proposed.
2) Horizontal Datum NAD83
Projection: UTM Zone 14N

0 100 200 300 400 500
Scale in Meters

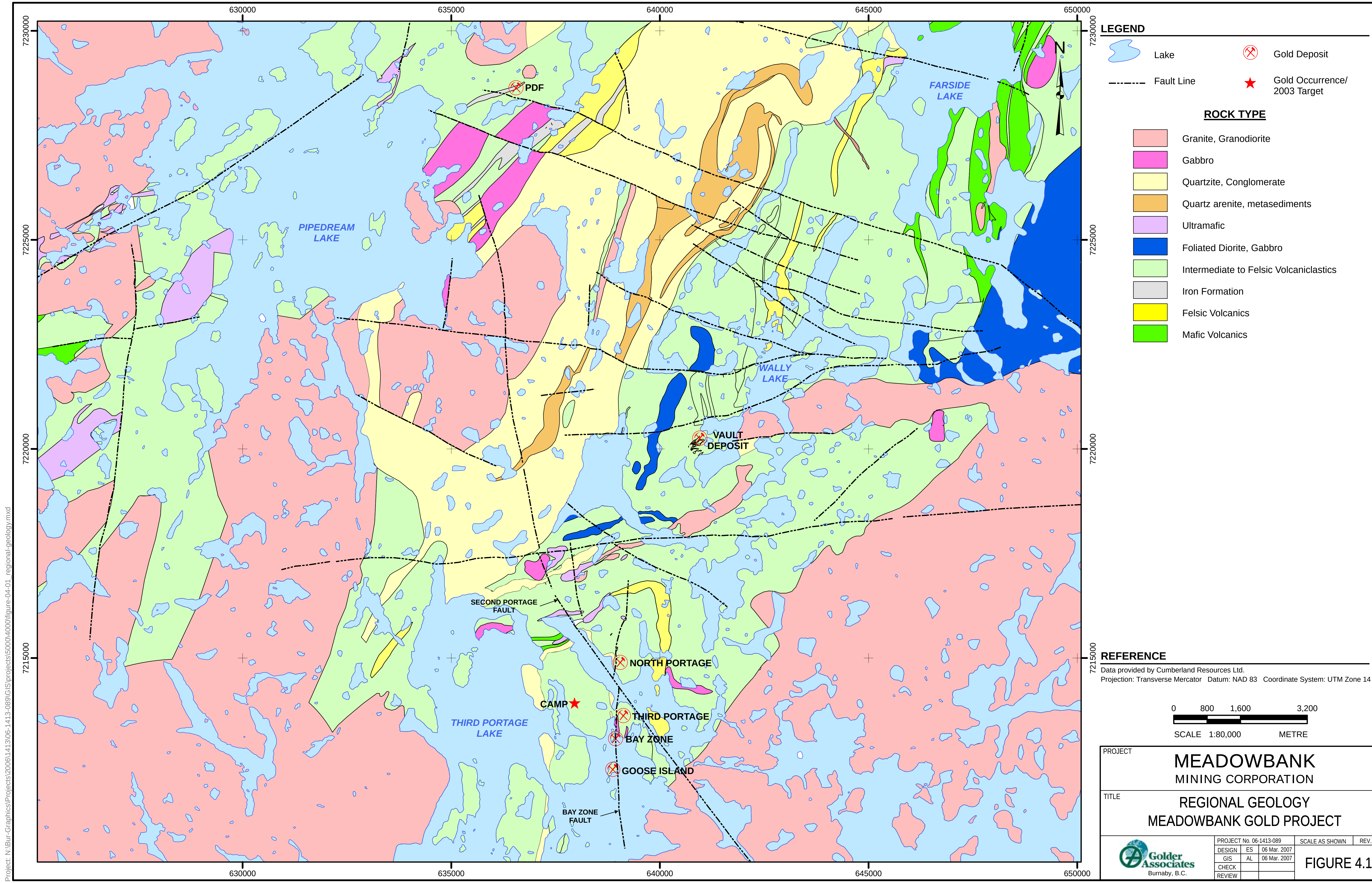
PROJECT
MEADOWBANK MINING CORPORATION

TITLE
HYDRAULIC CONDUCTIVITY TESTING BOREHOLE LOCATIONS

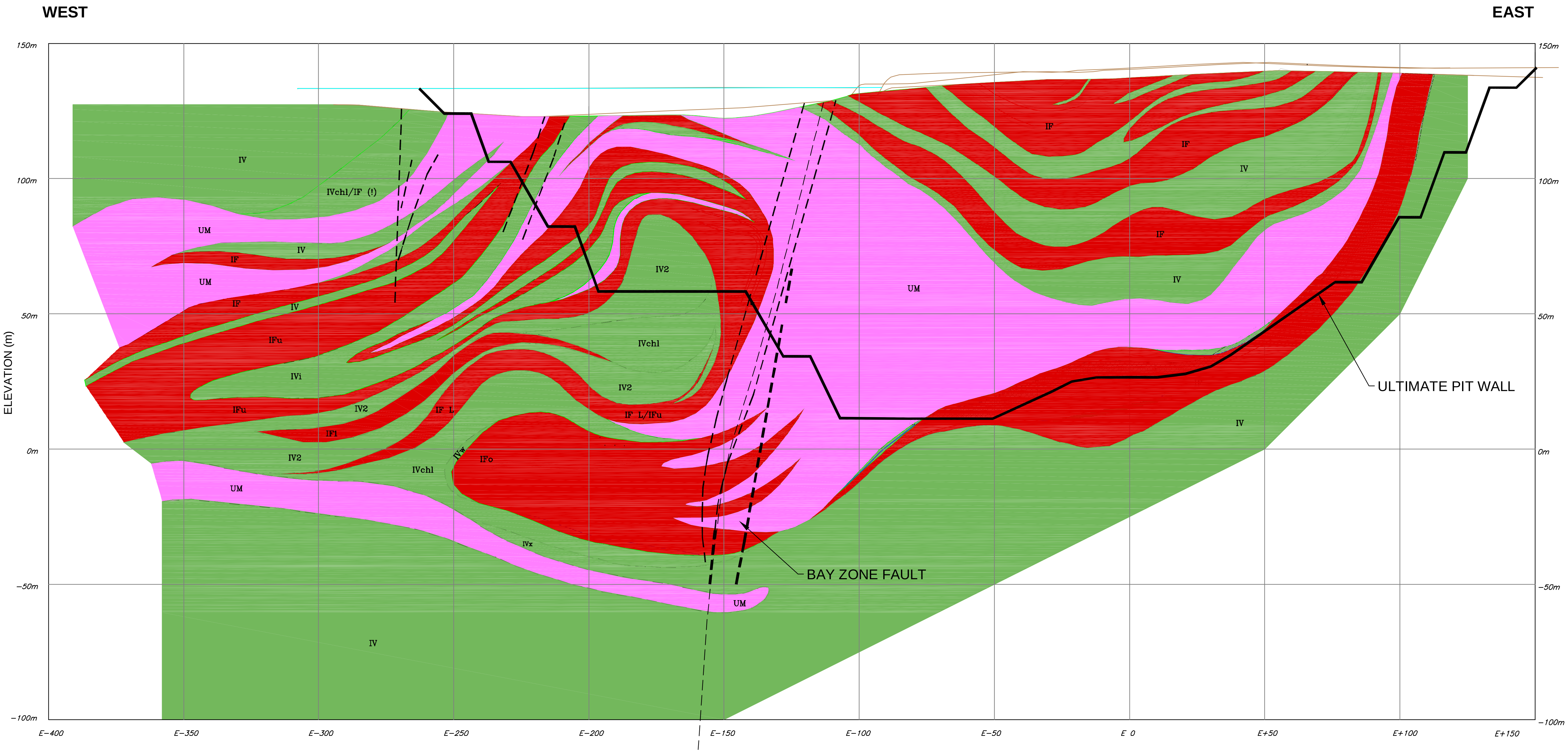
Golder Associates

PROJECT No.	06-1413-089		FILE No.	5000-FIG_3.4
DESIGN	CJC	20MAR07	SCALE	AS SHOWN
CADD	AS	20MAR07	REV.	
CHECK				
REVIEW				

FIGURE 3.4



630000
635000
640000
645000
650000
7230000
7225000
7220000
7215000
PDF
PIPEDREAM LAKE
FARSIDE LAKE
WALLY LAKE
VAULT DEPOSIT
SECOND PORTAGE FAULT
NORTH PORTAGE
CAMP
THIRD PORTAGE
THIRD PORTAGE LAKE
BAY ZONE
GOOSE ISLAND
BAY ZONE FAULT
N
LEGEND
Lake
Gold Deposit
Fault Line
Gold Occurrence/ 2003 Target
ROCK TYPE
Granite, Granodiorite
Gabbro
Quartzite, Conglomerate
Quartz arenite, metasediments
Ultramafic
Foliated Diorite, Gabbro
Intermediate to Felsic Volcaniclastics
Iron Formation
Felsic Volcanics
Mafic Volcanics
REFERENCE
Data provided by Cumberland Resources Ltd.
Projection: Transverse Mercator Datum: NAD 83 Coordinate System: UTM Zone 14
0 800 1,600 3,200
SCALE 1:80,000 METRE
PROJECT
MEADOWBANK
MINING CORPORATION
TITLE
REGIONAL GEOLOGY
MEADOWBANK GOLD PROJECT
Golder Associates
Burnaby, B.C.
PROJECT No. 06-1413-089
SCALE AS SHOWN
REV. 0
DESIGN ES 06 Mar. 2007
GIS AL 06 Mar. 2007
CHECK
REVIEW
FIGURE 4.1
Project: N:\Bur-Graphics\Projects\2006\1413\06-1413-089\GIS\projects\5000\4000\figure-04-01_regional-geology.mxd



LEGEND

- Iron Formation
- Intermediate Volcanic Rock
- Ultramafic Rock
- Fault zone

REFERENCES

1. Base geology drawing (MB_Surface.dwg) provided by Cumberland Resources Ltd.
2. Base fault drawing (3rdPortageSurfaceFaultTraces14.dwg) provided by Cumberland Resources Ltd.

0 10 20 30 40 50
Scale in Metres

PROJECT
MEADOWBANK MINING CORPORATION

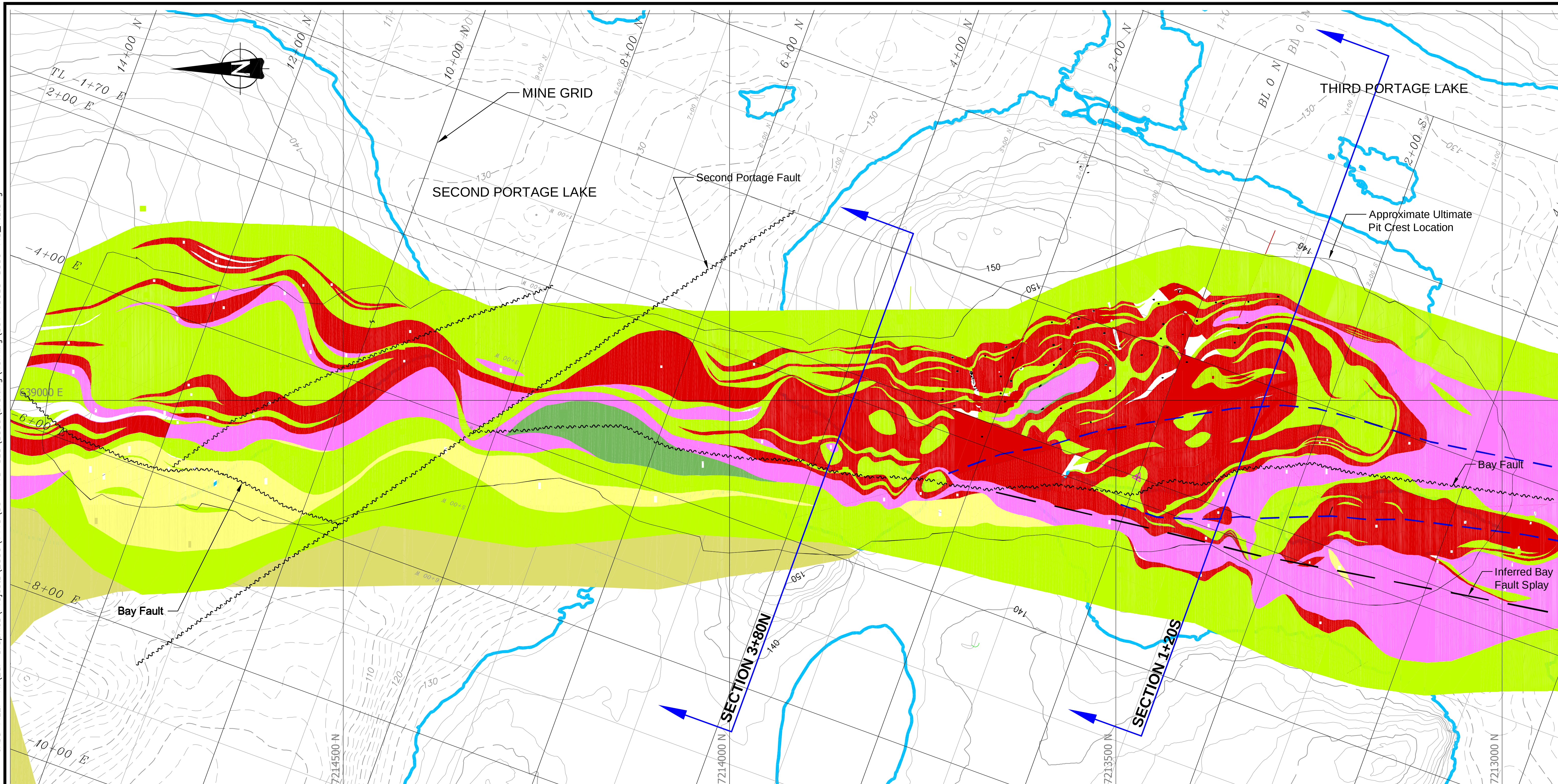
TITLE
PORTAGE DEPOSITS CROSS SECTION 3+80N SHOWING BAY ZONE FAULT EXTENTS

Golder Associates

PROJECT No.	06-1413-089		FILE No.	FIG_4.2
DESIGN	JCG	03APR07	SCALE AS SHOWN	REV. -
CADD	AS	03APR07		
CHECK				
REVIEW				

FIGURE 4.2

CADD FILE: N:\Bur-Graphics\Projects\2006\1413\06-1413-089\5000\Drafting\working\061413089-5000-FIG_4.2.dwg By: ASalvador
REVISION DATE: 07/04/05 07:09PM



CADD FILE: N:\Bur-Graphics\Projects\2006\1413\06-1413-089\5000\Drafting\working\061413089-5000-FIG_7.8.dwg

REVISION DATE: 07/04/05 05:07PM By: ASalvador

LEGEND

- Iron Formation
- Intermediate Volcanic Rock / Chloritic Intermediate Volcanic Rock
- Ultramafic Rock
- Quartzite
- Fold Hinge
- Bathymetry
- Topography

REFERENCES

1. Base geology drawing (MB_Surface.dwg) provided by Cumberland Resources Ltd.
2. Base fault drawing (3rdPortageSurfaceFaultTraces14.dwg) provided by Cumberland Resources Ltd.

0 50 100 150 200 250
Scale in Meters

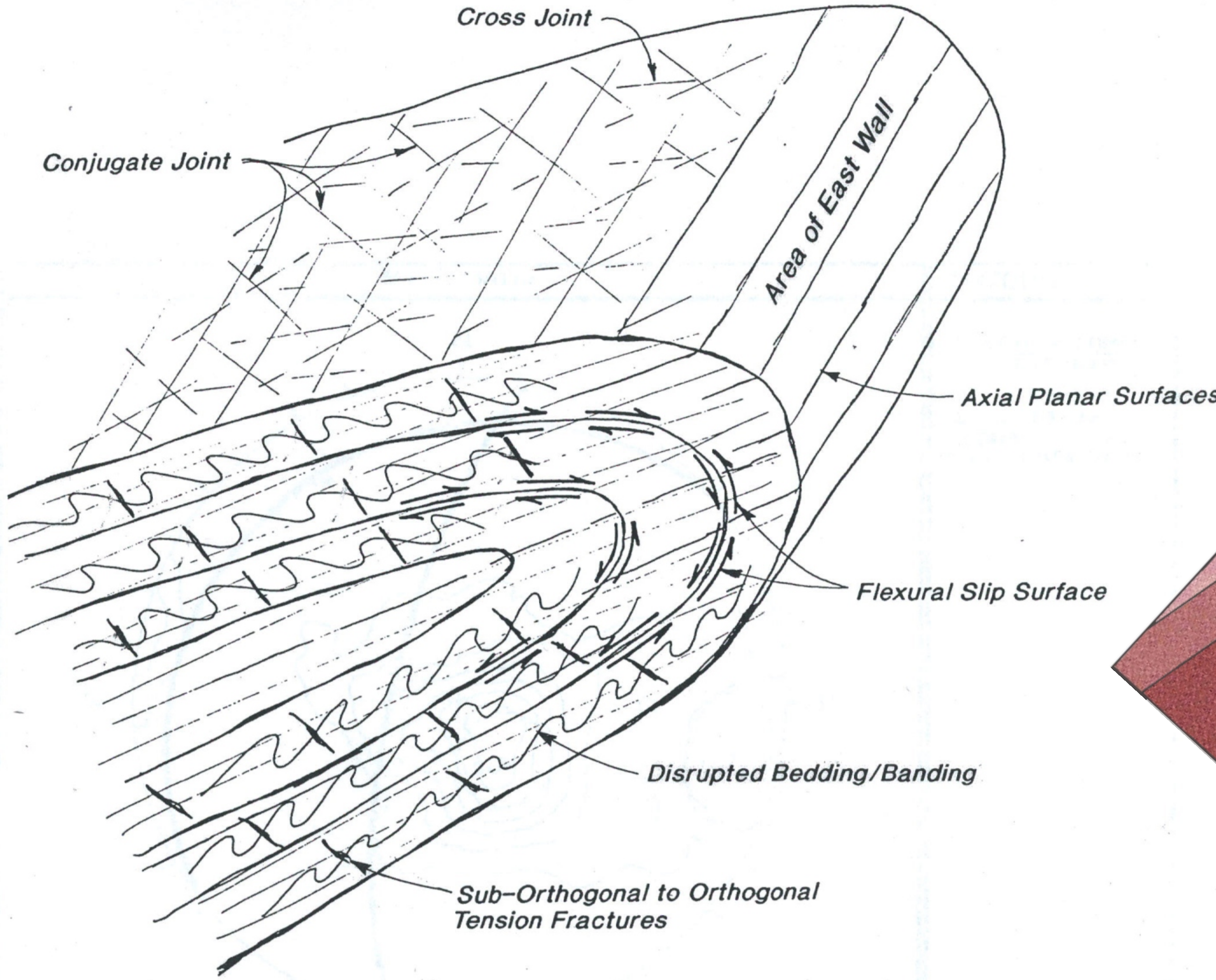
PROJECT
MEADOWBANK MINING CORPORATION

TITLE
PORTAGE DEPOSITS GENERAL GEOLOGY

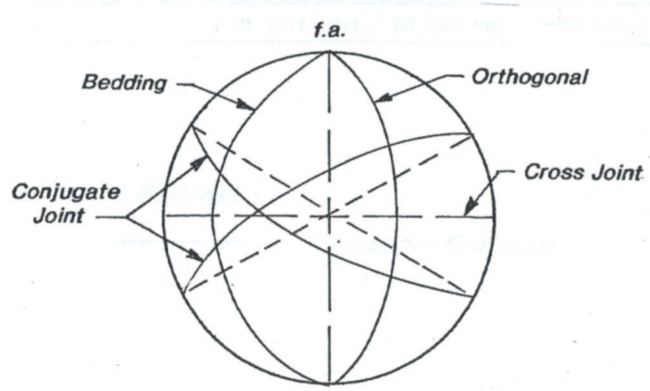
Golder Associates

PROJECT No.	06-1413-089		FILE No.
DESIGN	CJC	13MAR07	SCALE AS SHOWN REV. A
CADD	RCR	15MAR07	
CHECK			**FIGURE 4.3**
REVIEW			

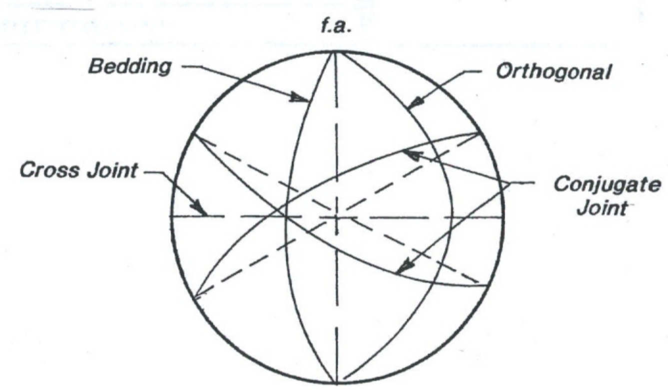
TYPICAL DISCONTINUITIES FOUND IN FOLDS



SCHEMATIC ONLY
Not to Scale

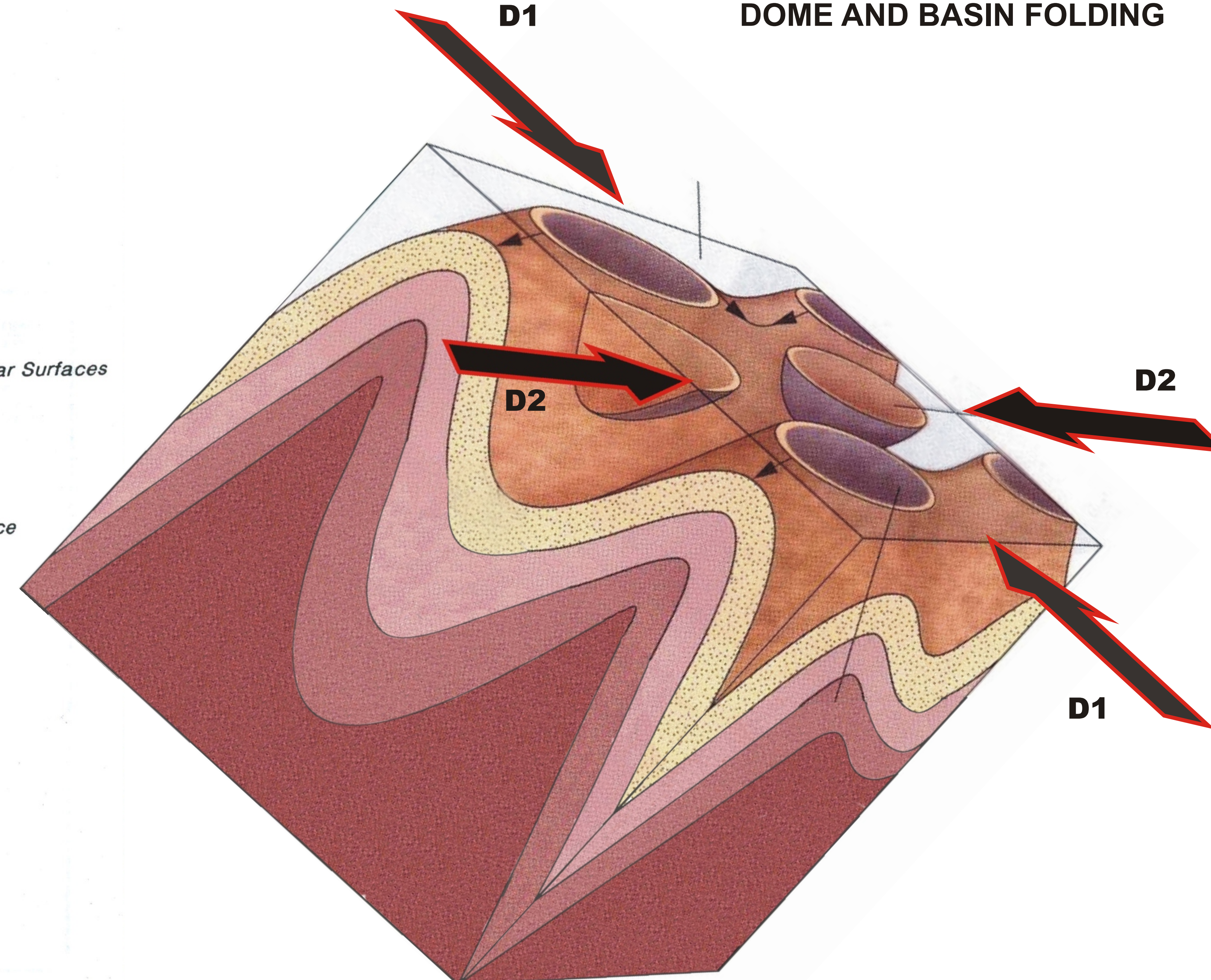


General Structure
Lower Limbs of Third Portage Anticline
at Depth to West



General Structure
Upper Limbs and Fold Nose;
Adjacent to Fault Zones

DOME AND BASIN FOLDING



EXPLANATION:

Superposition of two deformational events resulting in dome and basin style fold patterns. At Meadowbank, this pattern is a common overprint on the recumbent fold structure. Small scale sympathetic folds of bedding within the main lithostratigraphic units is commonly visible in drill core, suggesting that the main lithostratigraphic contacts, while considered to be continuous, may also be tightly folded.

REFERENCE:

Flerit Frédéric. 2006. "Folds and Thrust", Website http://flerit.free.fr/TK05/TK11-folds.ppt#301,7,Slide 7, March, 2007

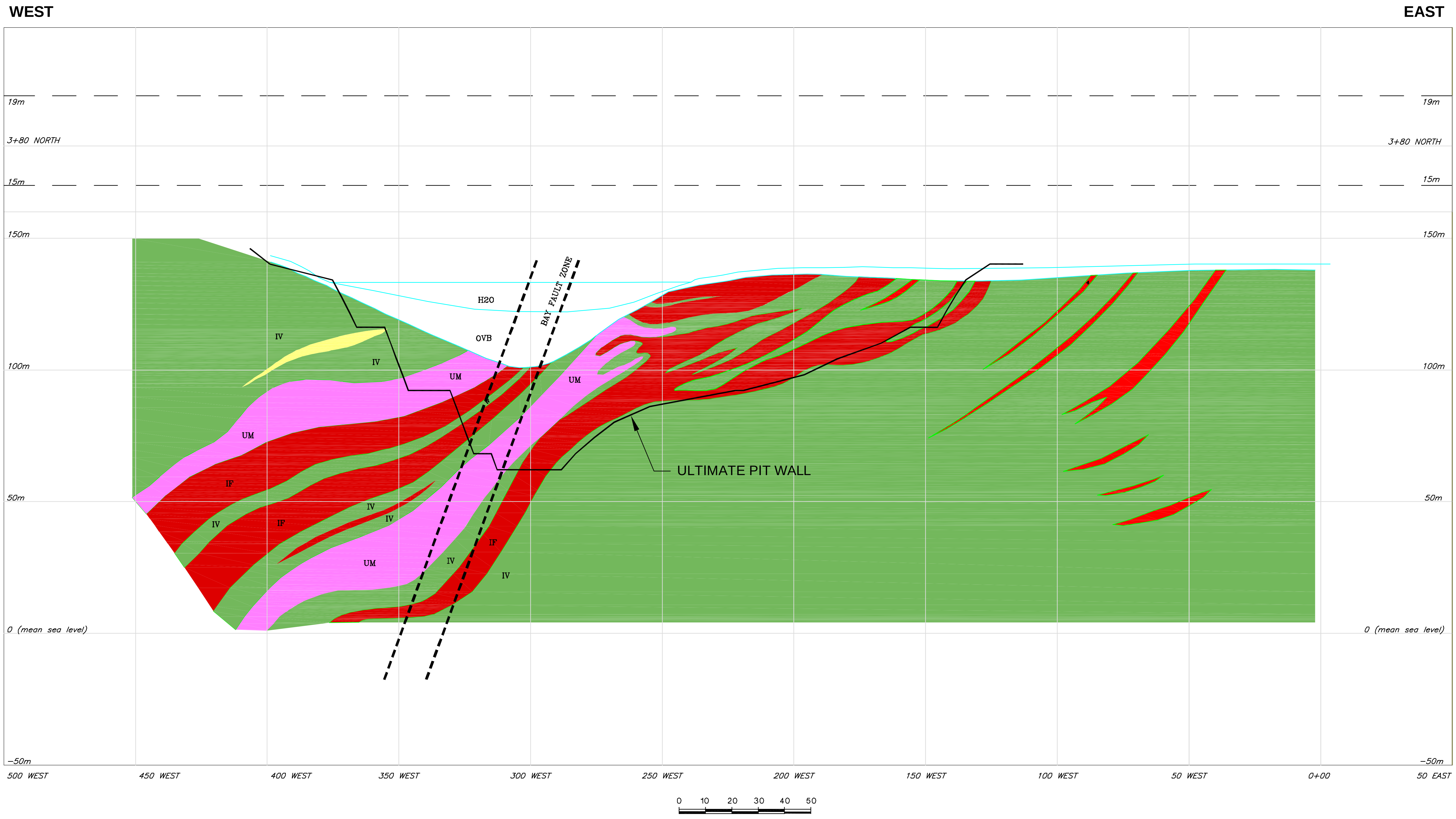
PROJECT
MEADOWBANK
MINING CORPORATION

TITLE
RELATIONSHIP BETWEEN DISCONTINUITIES AND THIRD PORTAGE ANTICLINE

PROJECT No.	06-1413-089		FILE No.	
DESIGN	CJC	21MAR07	SCALE NTS	REV.
CADD	AS/GG	21MAR07	**FIGURE 4.4**	
CHECK				
REVIEW				

Golder Associates

DRAWING DATE: 04-Apr-07 COREL FILE: N:\Bur-Graphics\Projects\2006\1413\06-1413-089\5000\Drafting\4000\Figure 7_Rev1.cdr



LEGEND

- Iron Formation
- Intermediate Volcanic Rock
- Ultramafic Rock
- Quartzite
- Fault zone

NOTES

1. Base geology plan provided by Meadowbank Mining Corporation.
2. Goose Island Pit Slope Design Criteria, August, 2003.

PROJECT

MEADOWBANK
MINING CORPORATION

TITLE

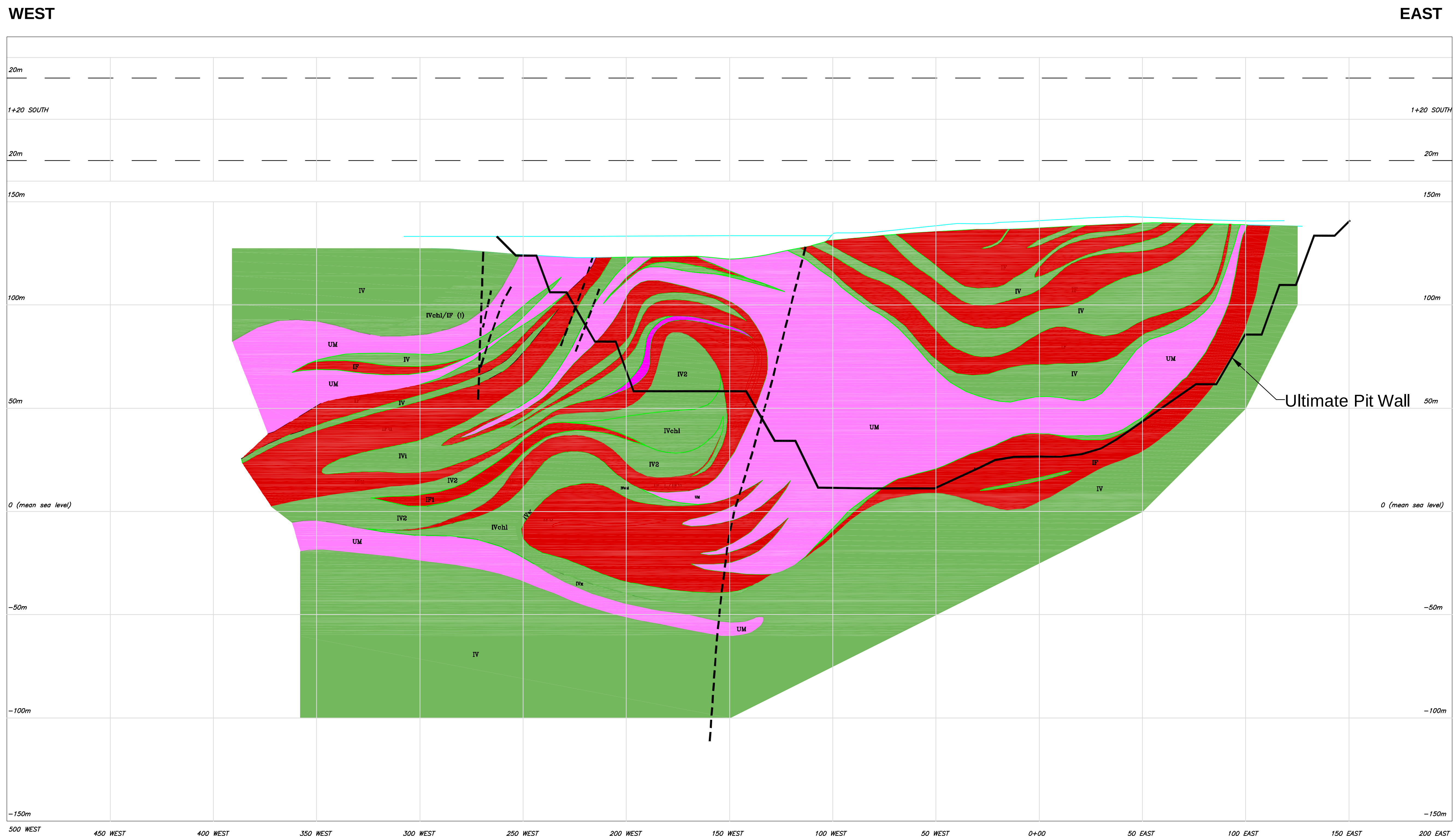
PORTAGE DEPOSIT
TYPICAL GEOLOGICAL CROSS SECTION
SECTION 3+80N (MINE GRID)

Golder Associates

PROJECT No.	06-1413-089		FILE No.	5000-4000-17
DESIGN	JFG	23MAR07	SCALE AS SHOWN	REV. -
CADD	AS	23MAR07		
CHECK			**FIGURE 4.5**	
REVIEW				

CADD FILE: N:\Bur-Graphics\Projects\2006\1413\06-1413-089\5000\Drafting\4000\061413089-5000-4000-13.dwg

REVISION DATE: 07/04/05 03:33PM By: ASalvador



LEGEND

- Iron Formation
- Intermediate Volcanic Rock
- Ultramafic Rock
- Fault zone

REFERENCES

1. Base geology drawing (MB_Surface.dwg) provided by Cumberland Resources Ltd.
2. Base fault drawing (3rdPortageSurfaceFaultTraces14.dwg) provided by Cumberland Resources Ltd.

0 20 40 60 80 100
Scale in Meters

PROJECT
MEADOWBANK
MINING CORPORATION

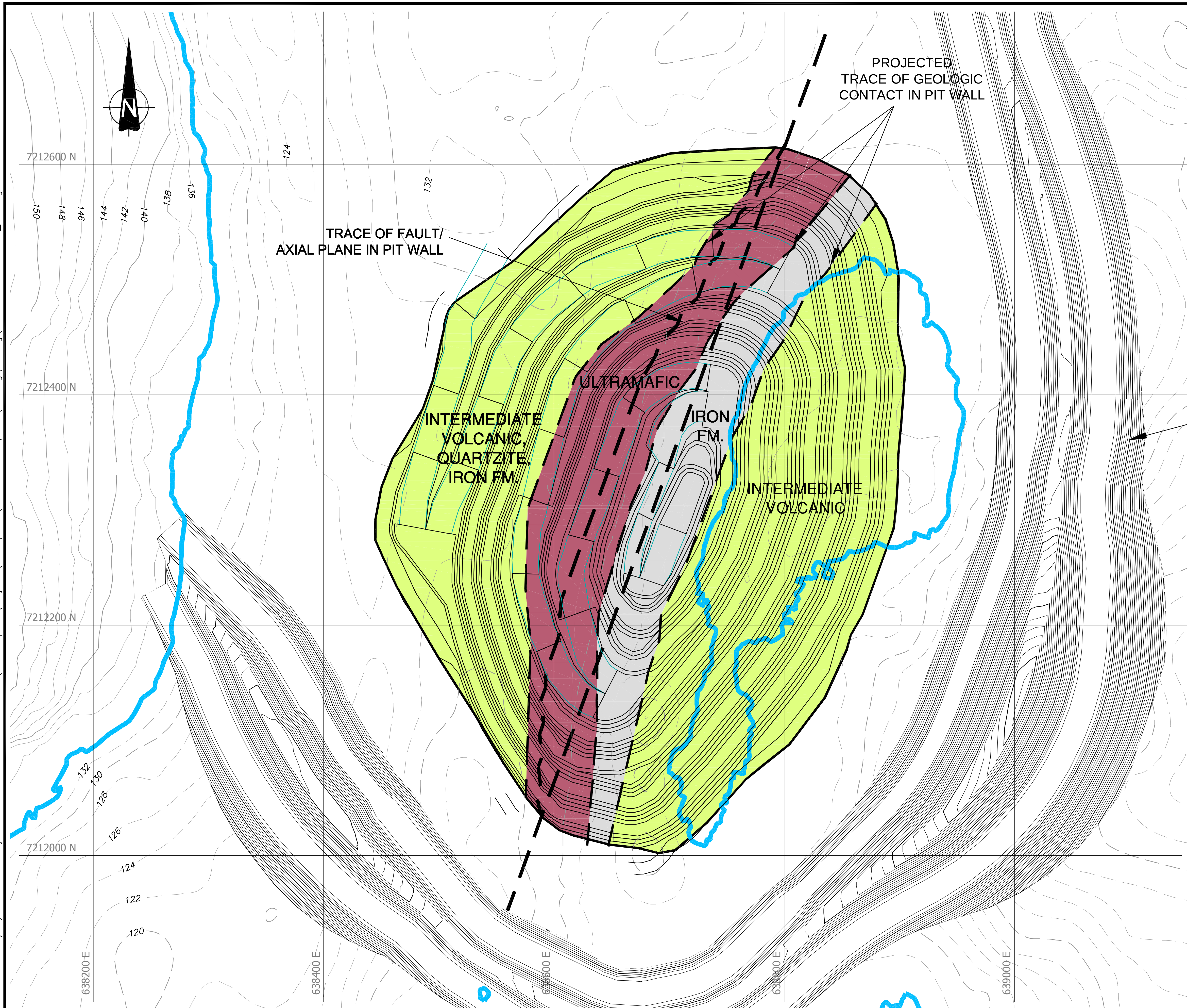
TITLE
PORTAGE DEPOSITS
TYPICAL GEOLOGICAL
CROSS SECTION 1+20S (MINE GRID)

Golder Associates

PROJECT No.	06-1413-089		FILE No.	5000-400-12
DESIGN	JFG	23MAR07	SCALE AS SHOWN	REV. -
CADD	AS	23MAR07		
CHECK				
REVIEW				

FIGURE 4.6

REVISION DATE: 07/04/05 06:53PM By: ASalvador CADD FILE: N:\Bur-Graphics\Projects\2006\1413\06-1413-089\5000\Drafting\4000\061413089-5000-4000-12.dwg



PROJECT

MEADOWBANK
MINING CORPORATION

TITLE

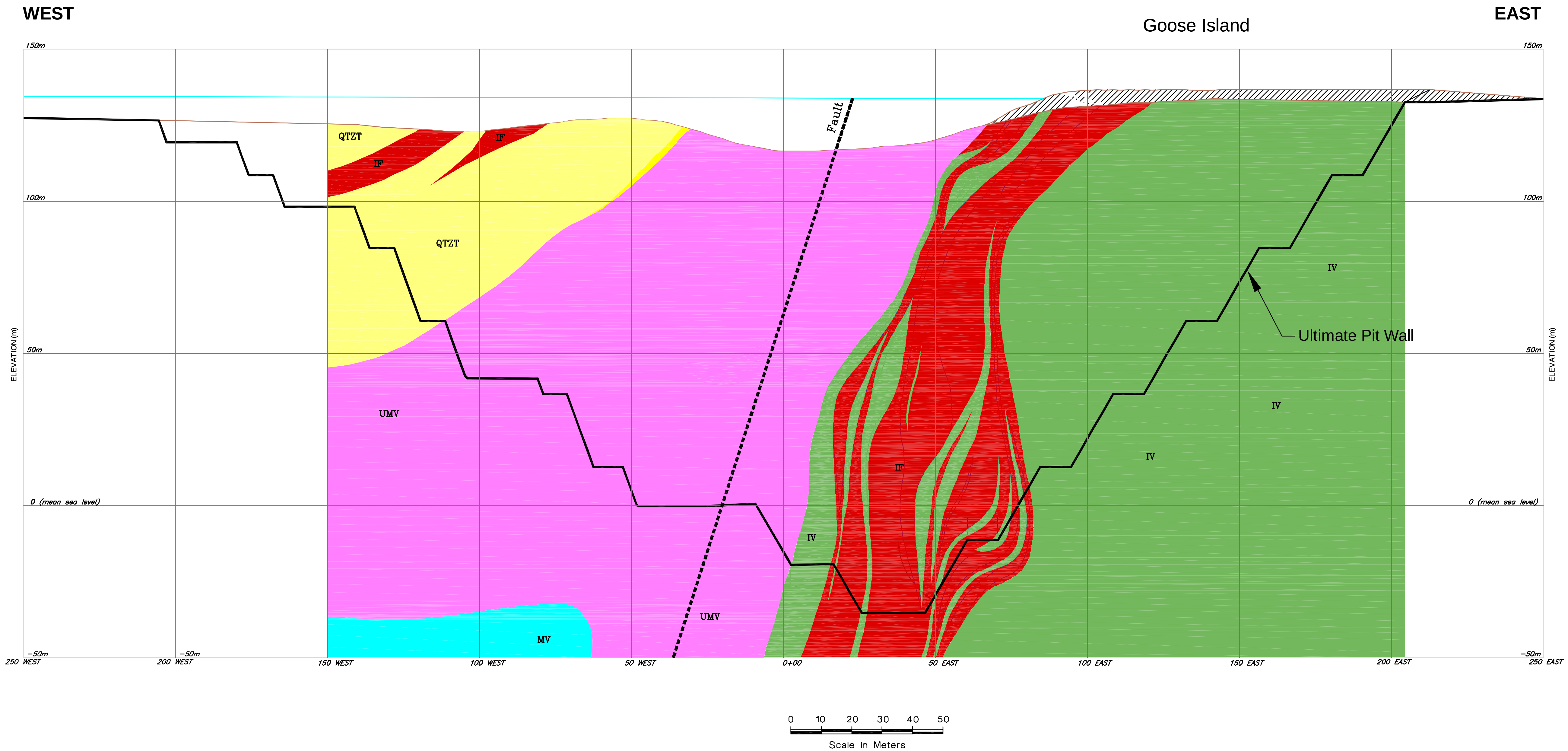
GOOSE ISLAND DEPOSIT
GENERAL GEOLOGY
PROJECTED ON PIT WALL

Golder Associates

PROJECT No.	06-1413-089		FILE No.	
DESIGN	CJC	15MAR07	SCALE AS SHOWN	REV.
CADD	CJC	15MAR07		
CHECK				
REVIEW				

FIGURE 4.7

CADD FILE: N:\Bur-Graphics\Projects\2006\1413\06-1413-089\5000\Drafting\working\061413089-5000-FIG_4.7.dwg By: ASalvador REVISION DATE: 07/04/05 03:56PM



LEGEND

- Iron Formation
- Intermediate Volcanic Rock
- Ultramafic Rock
- Quartzite
- Mafic Volcanics
- Fault zone

NOTES

1. Base geology plan provided by Meadowbank Mining Corporation.
2. Goose Island Pit Slope Design Criteria, August, 2003.

PROJECT

MEADOWBANK
MINING CORPORATION

TITLE

GOOSE ISLAND DEPOSIT TYPICAL CROSS SECTION

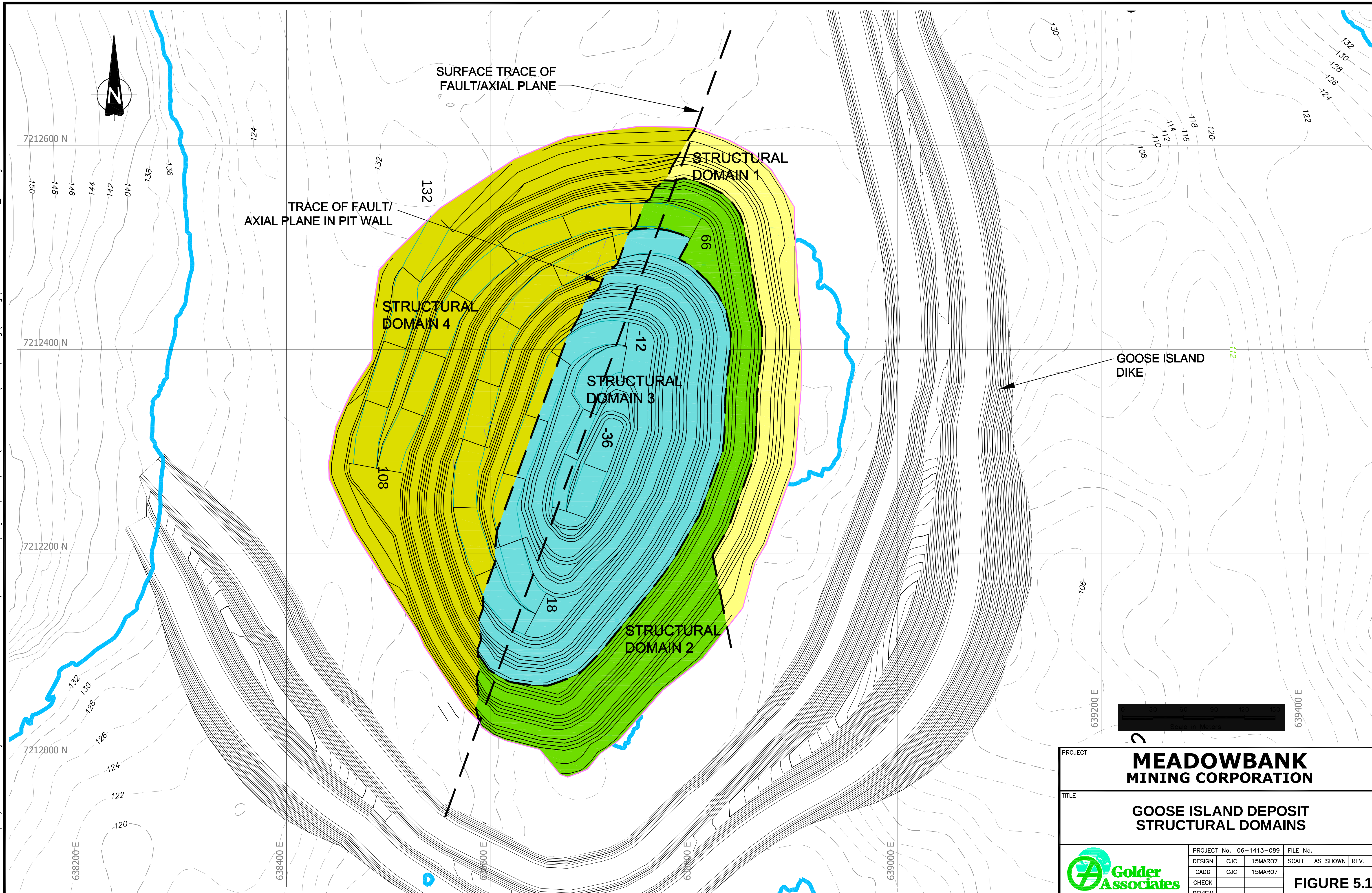
Golder Associates

PROJECT No.	06-1413-089		FILE No.	5000-4000-16
DESIGN	CJC	23MAR07	SCALE AS SHOWN	REV.
CADD	AS	23MAR07		
CHECK				
REVIEW				

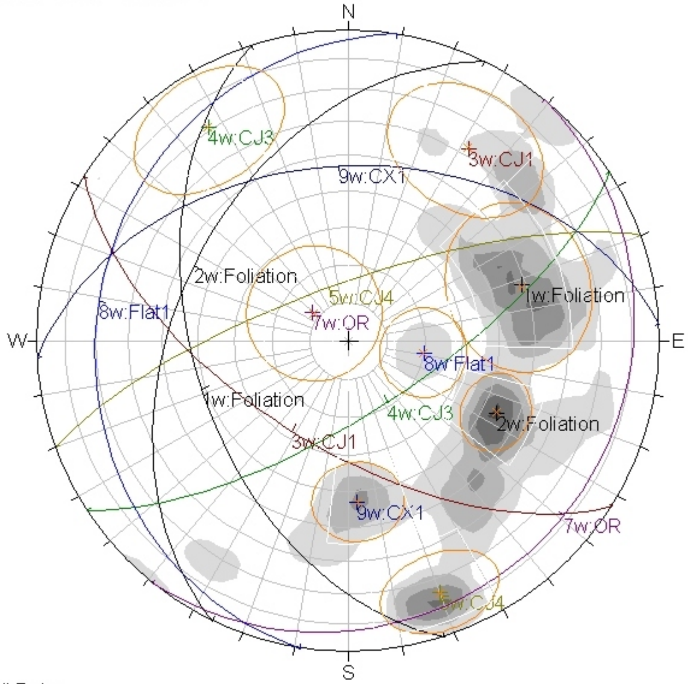
FIGURE 4.8

REVISION DATE: 07/04/05 02:03PM By: ASalvador

CADD FILE: N:\Bur-Graphics\Projects\2006\1413\06-1413-089\5000\Drafting\4000\061413089-5000-4000-16.dwg


SURFACE TRACE OF
FAULT/AXIAL PLANE
TRACE OF FAULT/
AXIAL PLANE IN PIT WALL
STRUCTURAL
DOMAIN 1
STRUCTURAL
DOMAIN 2
STRUCTURAL
DOMAIN 3
STRUCTURAL
DOMAIN 4
GOOSE ISLAND
DIKE
7212600 N
7212400 N
7212200 N
7212000 N
638200 E
638400 E
638600 E
638800 E
639000 E
639200 E
639400 E
132
108
66
-12
-36
18
PROJECT
MEADOWBANK
MINING CORPORATION
TITLE
GOOSE ISLAND DEPOSIT
STRUCTURAL DOMAINS
Golder Associates
PROJECT No. 06-1413-089
FILE No.
DESIGN CJC 15MAR07
CADD CJC 15MAR07
CHECK
REVIEW
SCALE AS SHOWN
REV.
FIGURE 5.1
CADD FILE: N:\Bur-Graphics\Projects\2006\1413\06-1413-089\5000\Drafting\working\061413089-5000-FIG_5.2.dwg
REVISION DATE: 07/04/05 04:59PM By: ASalvador

Goose Island - Domain 1



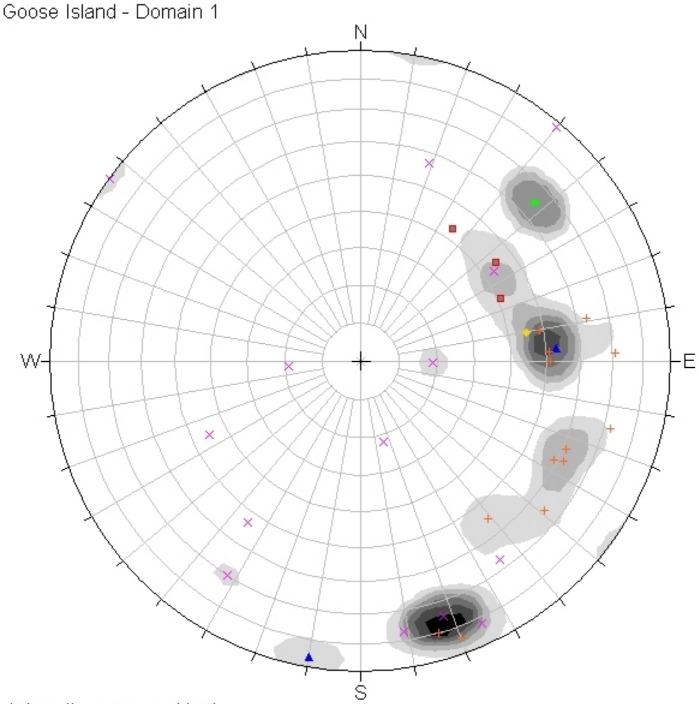
All Data

Orientations

ID		Dip / Direction
1	w	49 / 252
2	w	44 / 296
3	w	62 / 212
4	w	71 / 147
5	w	75 / 340
7	w	12 / 129
8	w	20 / 279
9	w	43 / 357

Equal Area
Lower Hemisphere
129 Poles
120 Entries

Goose Island - Domain 1



Jr less than or equal to 1

TYPE

- ■ BD [3]
- ▲ CJ [2]
- ▶ FLT [2]
- + FO [15]
- × JN [20]
- ◆ OR [1]

Equal Area
Lower Hemisphere
43 Poles
38 Entries

PROJECT

MEADOWBANK
MINING CORPORATION

TITLE

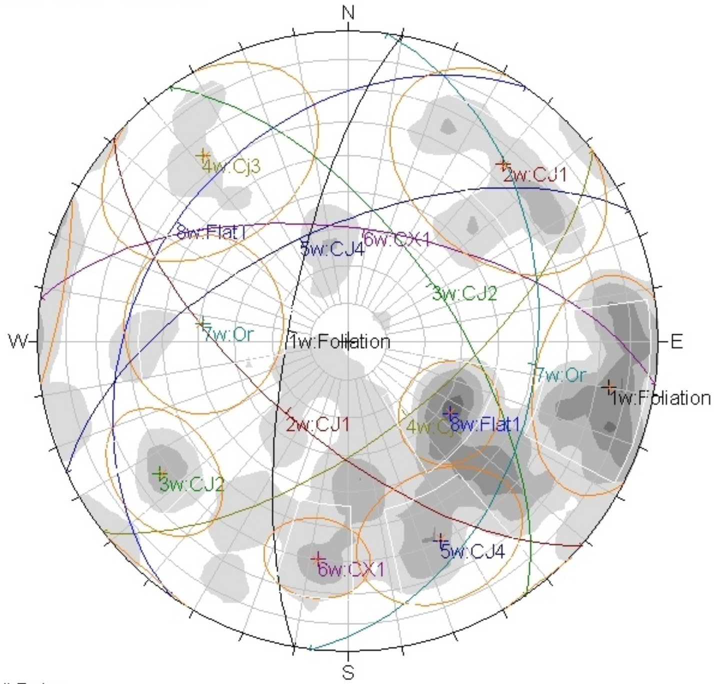
GOOSE ISLAND
STRUCTURAL DOMAIN 1

Golder Associates

PROJECT No.	06-1413-089		FILE No.	
DESIGN	JFG	09MAR07	SCALE NTS	REV.
CADD	GG	09MAR07		
CHECK			**FIGURE 5.2**	
REVIEW				

DRAWING DATE: 17-Mar-07 COREL FILE: N:\Bur-Graphics\Projects\2006\1413\06-1413-089\5000\Drafting\4000\061413089-5000-4000-B_dips.cdr

Goose Island - Domain 2



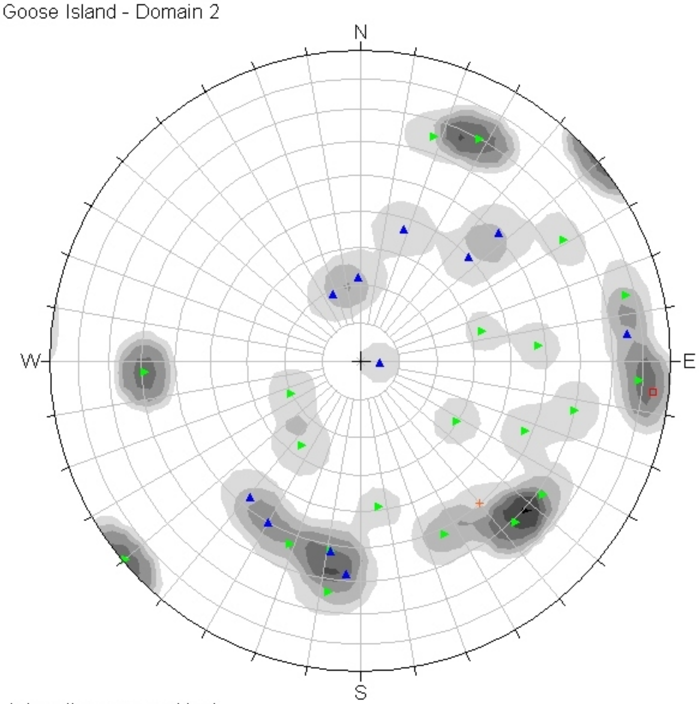
All Data

Orientations

ID		Dip / Direction
1	w	74 / 280
2	w	65 / 221
3	w	63 / 055
4	w	65 / 142
5	w	60 / 335
6	w	60 / 008
7	w	39 / 097
8	w	33 / 305

Equal Area
Lower Hemisphere
147 Poles
146 Entries

Goose Island - Domain 2



Jr less than or equal to 1

TYPE

FLT [1]
FO [11]
JN [22]
VN [1]

Equal Area
Lower Hemisphere
35 Poles
34 Entries

PROJECT

MEADOWBANK
MINING CORPORATION

TITLE

GOOSE ISLAND
STRUCTURAL DOMAIN 2

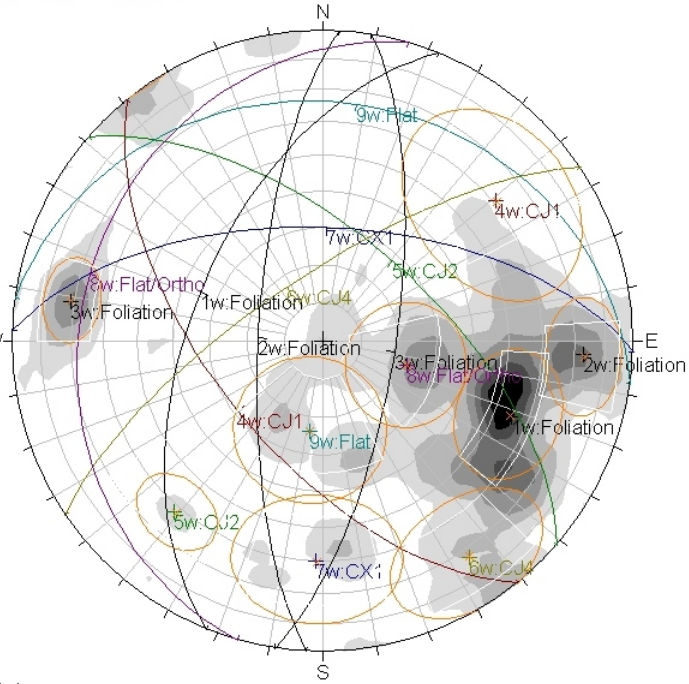
Golder Associates

PROJECT No.	06-1413-089		FILE No.	
DESIGN	JFG	09MAR07	SCALE NTS	REV.
CADD	GG	09MAR07		
CHECK				
REVIEW				

FIGURE 5.3

DRAWING DATE: 17-Mar-07 COREL FILE: N:\Bur-Graphics\Projects\2006\1413\06-1413-089\5000\Drafting\4000\061413089-5000-4000-B_dips.cdr

Goose Island - Domain 3



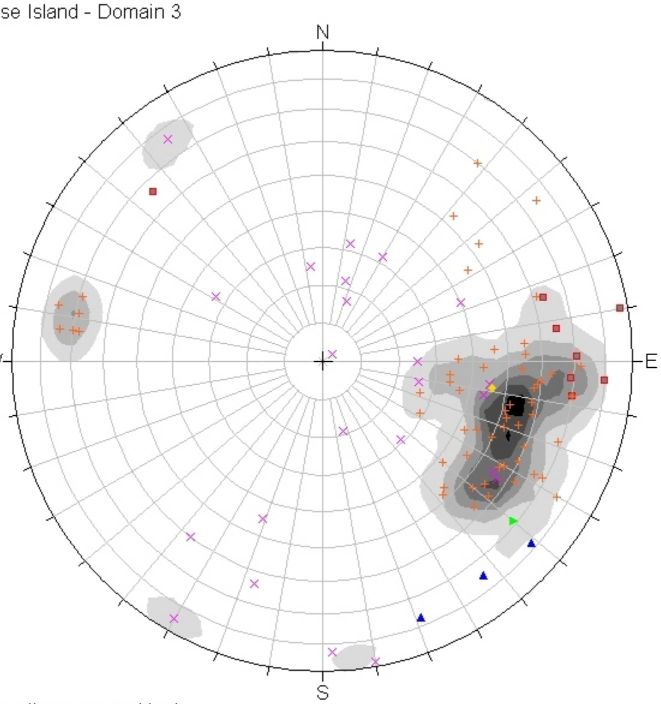
Orientations

ID		Dip / Direction
1	w	55 / 292
2	w	73 / 273
3	w	71 / 099
4	w	61 / 231
5	w	62 / 041
6	w	73 / 326
7	w	60 / 002
8	w	23 / 286
9	w	24 / 008

Equal Area
Lower Hemisphere
279 Poles
267 Entries

All Data

Goose Island - Domain 3



TYPE

- BD [8]
- CJ [3]
- FLT [1]
- FO [66]
- JN [24]
- SHR [1]

Equal Area
Lower Hemisphere
103 Poles
97 Entries

Jr less than or equal to 1

PROJECT

MEADOWBANK
MINING CORPORATION

TITLE

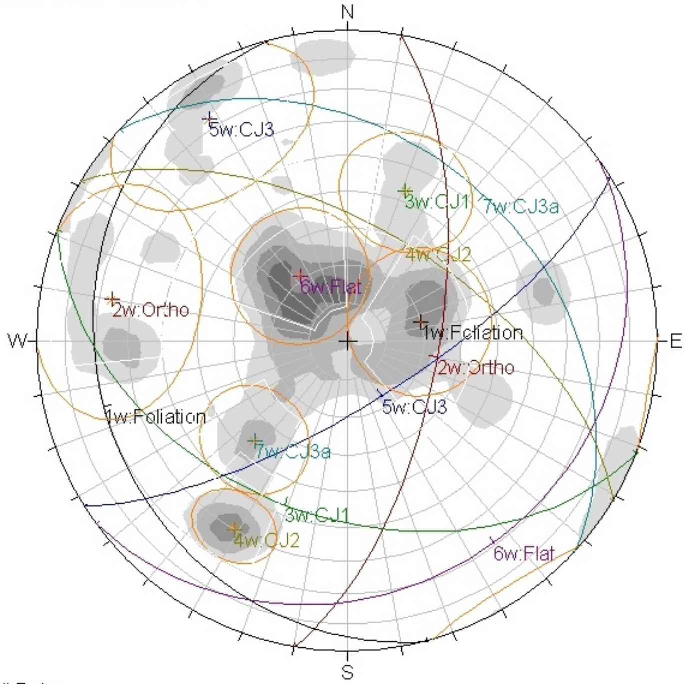
GOOSE ISLAND
STRUCTURAL DOMAIN 3

Golder Associates

PROJECT No.	06-1413-089		FILE No.	
DESIGN	JFG	09MAR07	SCALE NTS	REV.
CADD	GG	09MAR07		
CHECK			**FIGURE 5.4**	
REVIEW				

DRAWING DATE: 17-Mar-07 COREL FILE: N:\Bur-Graphics\Projects\2006\1413\06-1413-089\5000\Drafting\4000\061413089-5000-4000-B_dips.cdr

Goose Island - Domain 4



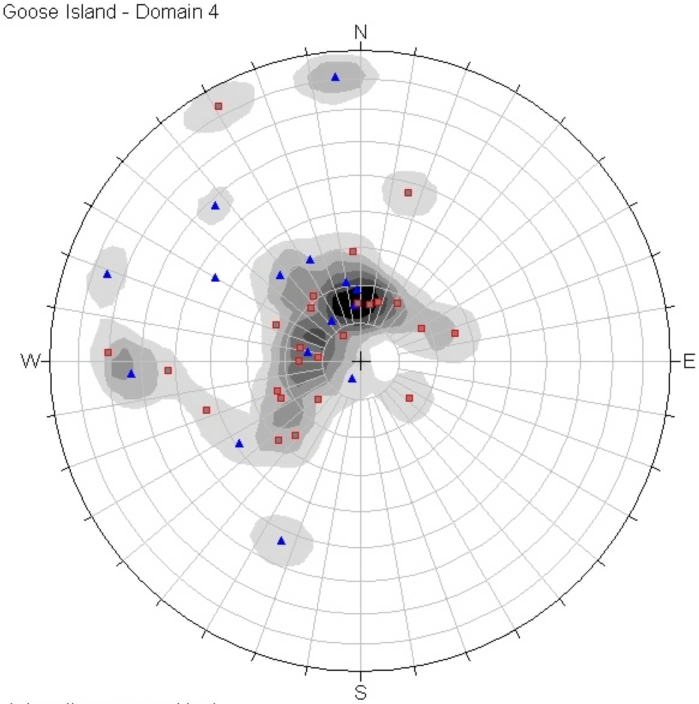
Orientations

ID		Dip / Direction
1	w	20 / 255
2	w	66 / 100
3	w	43 / 201
4	w	60 / 031
5	w	73 / 148
6	w	21 / 144
7	w	36 / 043

Equal Area
Lower Hemisphere
121 Poles
87 Entries

All Data

Goose Island - Domain 4



TYPE

FO [30]
JN [21]

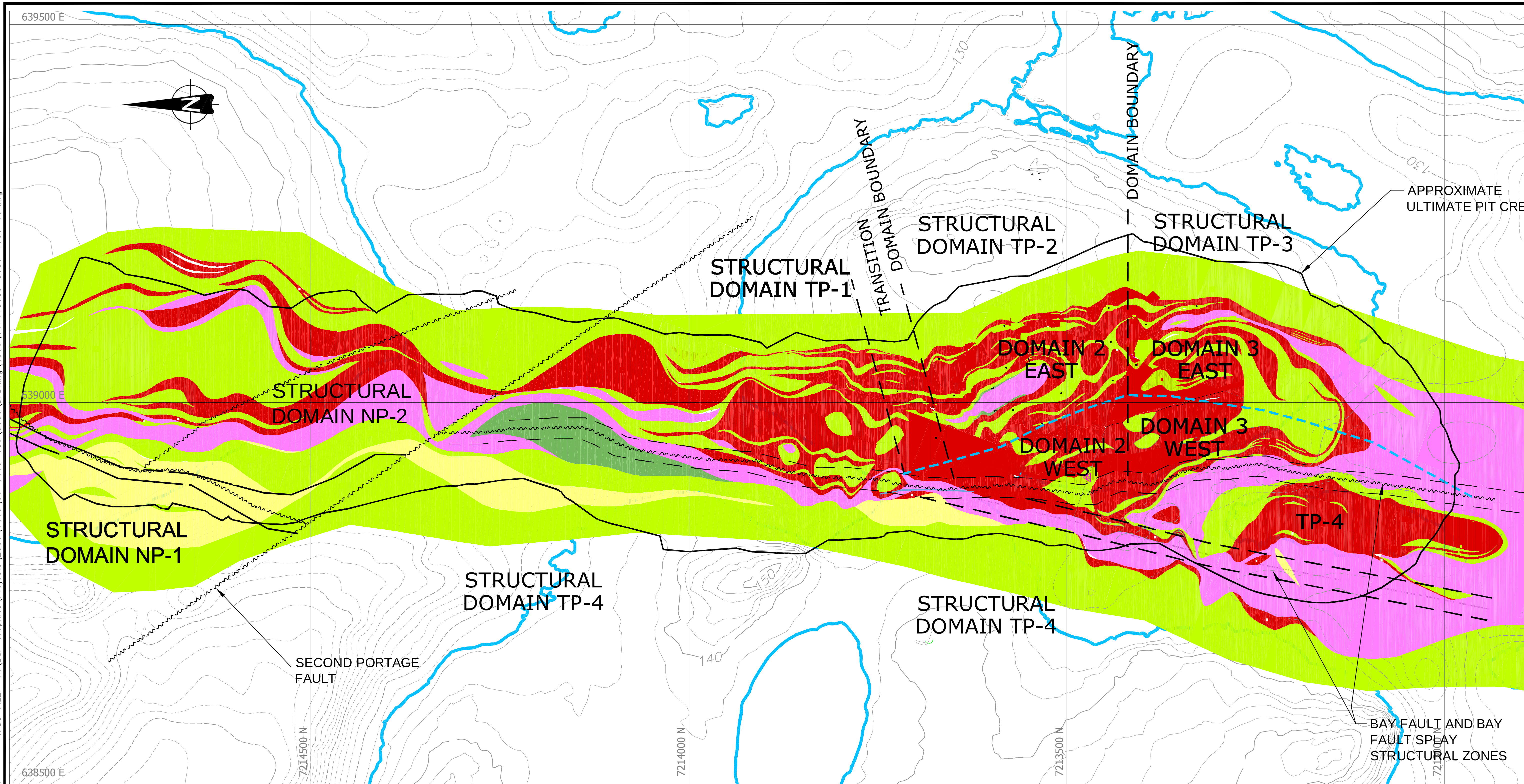
Equal Area
Lower Hemisphere
51 Poles
41 Entries

Jr less than or equal to 1

PROJECT	MEADOWBANK MINING CORPORATION		
TITLE	GOOSE ISLAND STRUCTURAL DOMAIN 4		
PROJECT No.	06-1413-089	FILE No.	
DESIGN	JFG	09MAR07	SCALE NTS / REV.
CADD	GG	09MAR07	FIGURE 5.5
CHECK			
REVIEW			

Golder Associates

DRAWING DATE: 17-Mar-07 COREL FILE: N:\Bur-Graphics\Projects\2006\1413\06-1413-089\5000\Drafting\4000\061413089-5000-4000-B_dips.cdr



LEGEND

- Iron Formation
- Intermediate Volcanic Rock / Chloritic Intermediate Volcanic Rock
- Ultramafic Rock
- Quartzite
- Fold Hinge

REFERENCES

1. Base geology drawing (MB_Surface.dwg) provided by Cumberland Resources Ltd.
2. Base fault drawing (3rdPortageSurfaceFaultTraces14.dwg) provided by Cumberland Resources Ltd.
3. Bay Fault Splay inferred from available geological cross sections.
4. Actual fault location may vary from those shown on map.

0 50 100 150 200 250

Scale in Meters

PROJECT

MEADOWBANK MINING CORPORATION

TITLE

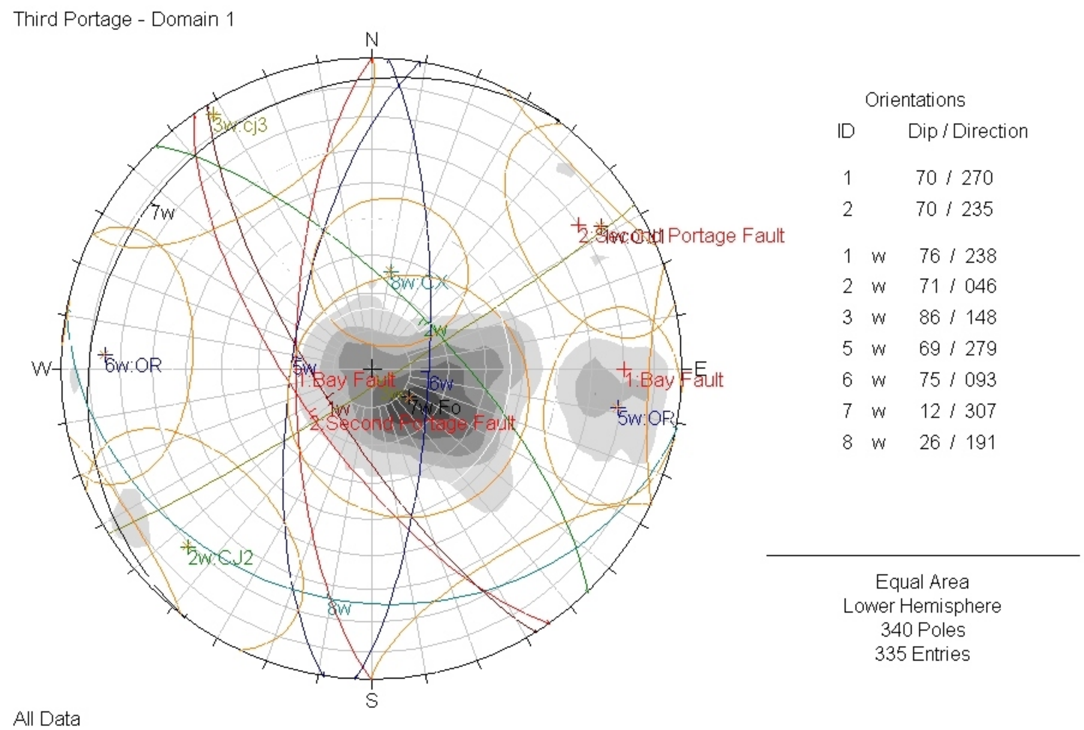
PORTAGE DEPOSITS STRUCTURAL DOMAINS

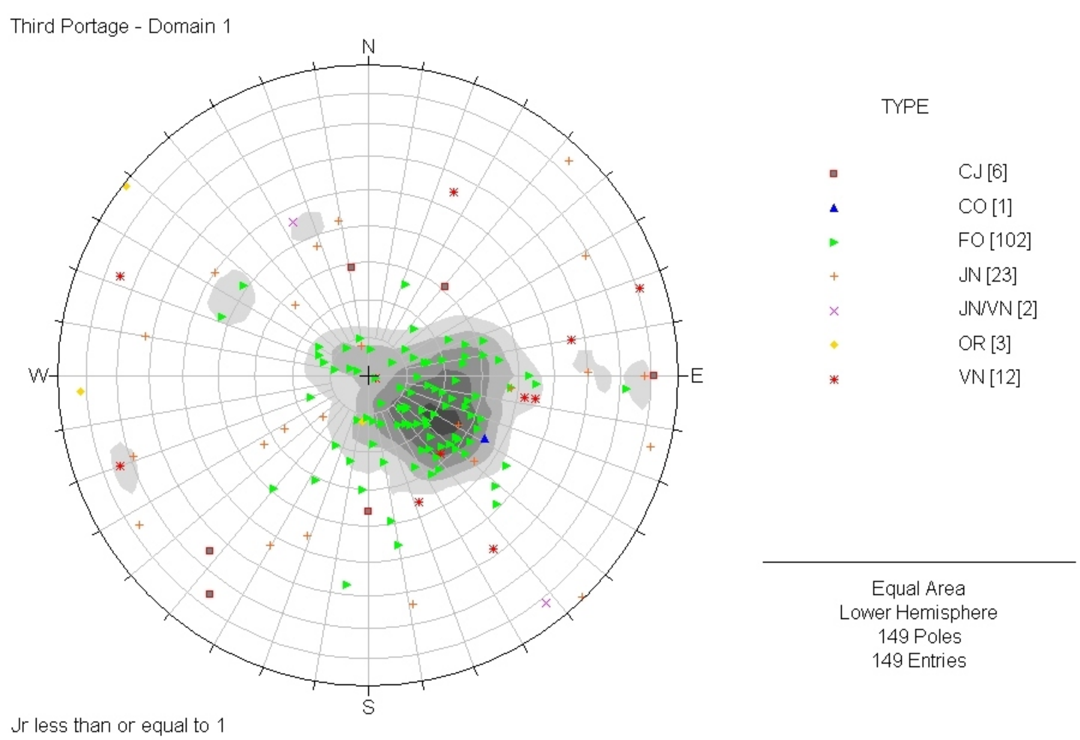
Golder Associates

PROJECT No.	06-1413-089		FILE No.	5000-4000-10
DESIGN	CJC	19MAR07	SCALE	AS SHOWN	REV.
CADD	AS	19MAR07		
CHECK			**FIGURE 5.6**	
REVIEW				

CADD FILE: N:\Bur-Graphics\Projects\2006\1413\06-1413-089\5000\Drafting\4000\061413089-5000-4000-10.dwg

REVISION DATE: 07/04/05 05:10PM By: ASalvador





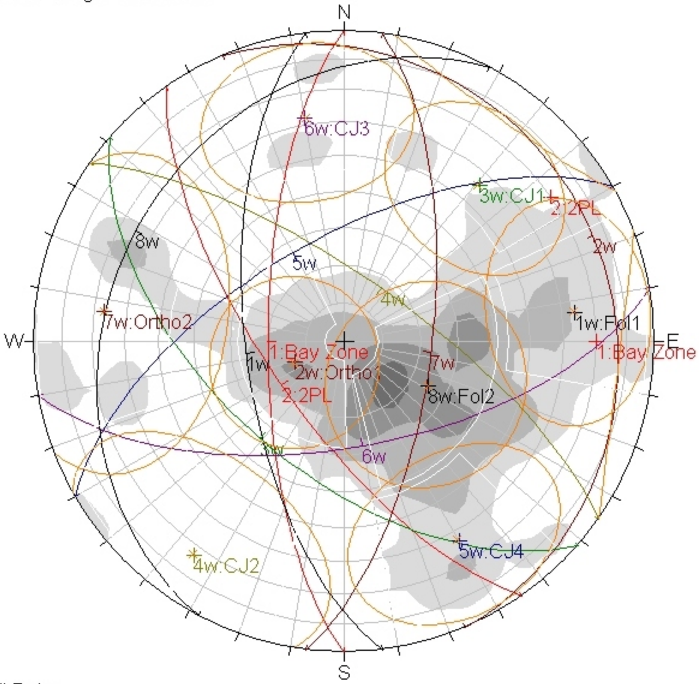
PROJECT	MEADOWBANK MINING CORPORATION
TITLE	PORTAGE STRUCTURAL DOMAIN TP-1

Golder Associates

PROJECT No.	06-1413-089		FILE No.	
DESIGN	JFG	09MAR07	SCALE NTS	REV.
CADD	GG	09MAR07	FIGURE 5.7	
CHECK				
REVIEW				

DRAWING DATE: 17-Mar-07 COREL FILE: N:\Bur-Graphics\Projects\2006\1413\06-1413-089\5000\Drafting\4000\061413089-5000-4000-B_dips.cdr

Third Portage - Domain 2



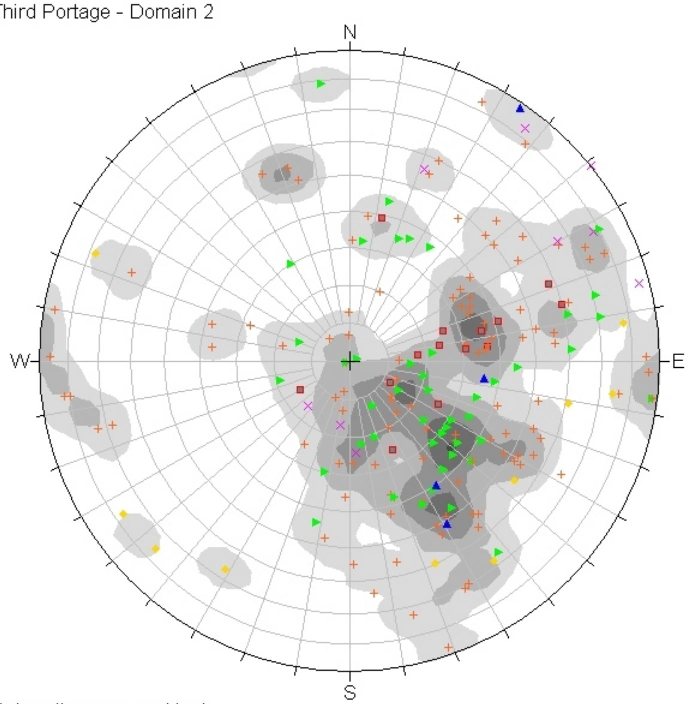
Orientations

ID		Dip / Direction
1		70 / 270
2		70 / 235
1	w	64 / 263
2	w	14 / 067
3	w	56 / 221
4	w	73 / 035
5	w	63 / 330
6	w	62 / 170
7	w	67 / 097
8	w	25 / 298

Equal Area
Lower Hemisphere
592 Poles
591 Entries

All Data

Third Portage - Domain 2



TYPE

- B [14]
- CO [4]
- FO [49]
- JN [110]
- OR [9]
- VN [10]

Equal Area
Lower Hemisphere
196 Poles
196 Entries

Jr less than or equal to 1

DRAWING DATE: 17-Mar-07 COREL FILE: N:\Bur-Graphics\Projects\2006\1413\06-1413-089\5000\Drafting\4000\061413089-5000-4000-B_dips.cdr

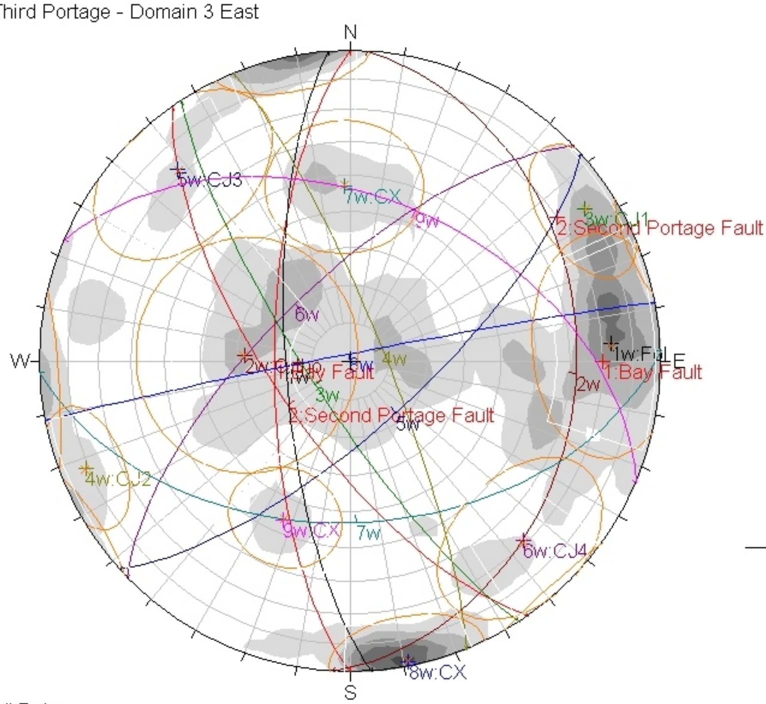
PROJECT
MEADOWBANK
MINING CORPORATION

TITLE
PORTAGE
STRUCTURAL DOMAIN TP-2

Golder Associates

PROJECT No.	06-1413-089		FILE No.	
DESIGN	JFG	09MAR07	SCALE NTS	REV.
CADD	GG	09MAR07		
CHECK			**FIGURE 5.8**	
REVIEW				

Third Portage - Domain 3 East



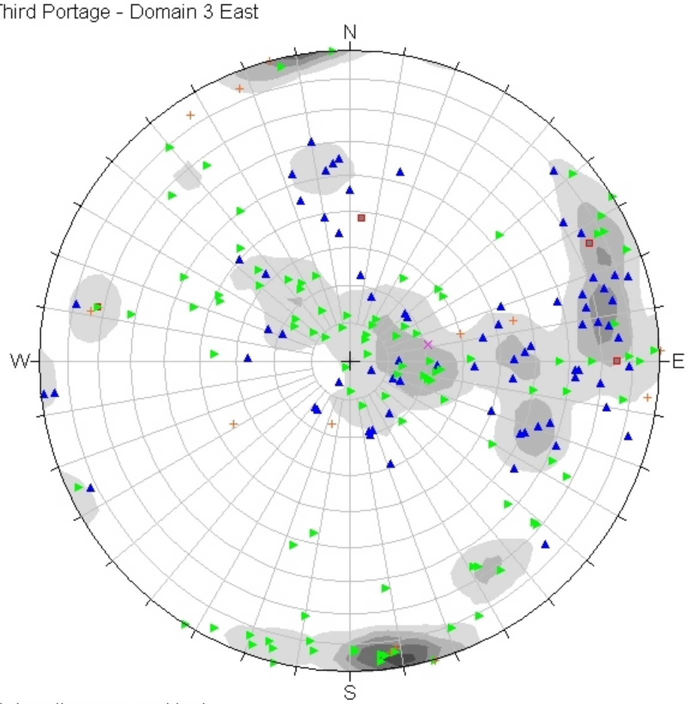
Orientations

ID		Dip / Direction
1		70 / 270
2		70 / 235
1	w	73 / 266
2	w	28 / 093
3	w	79 / 237
4	w	81 / 068
5	w	72 / 138
6	w	69 / 316
7	w	47 / 178
8	w	88 / 349
9	w	46 / 023

Equal Area
Lower Hemisphere
451 Poles
444 Entries

All Data

Third Portage - Domain 3 East



TYPE

- CO [4]
- FO [80]
- JN [117]
- OR [16]
- VN [1]

Equal Area
Lower Hemisphere
218 Poles
211 Entries

Jr less than or equal to 1

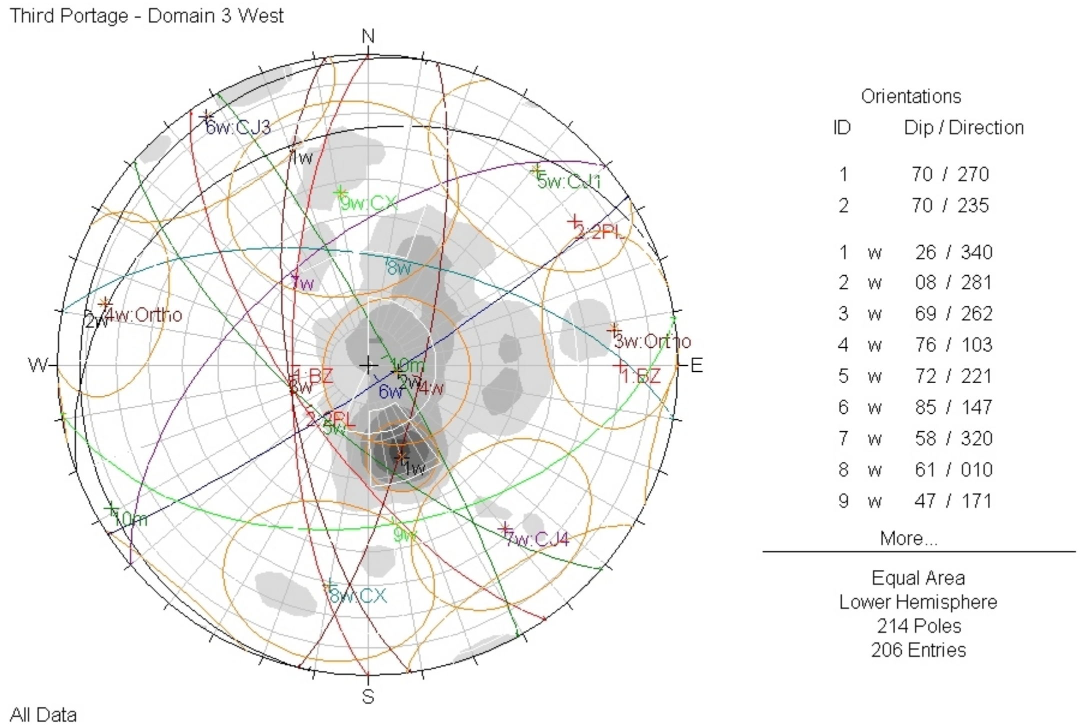
PROJECT
MEADOWBANK
MINING CORPORATION

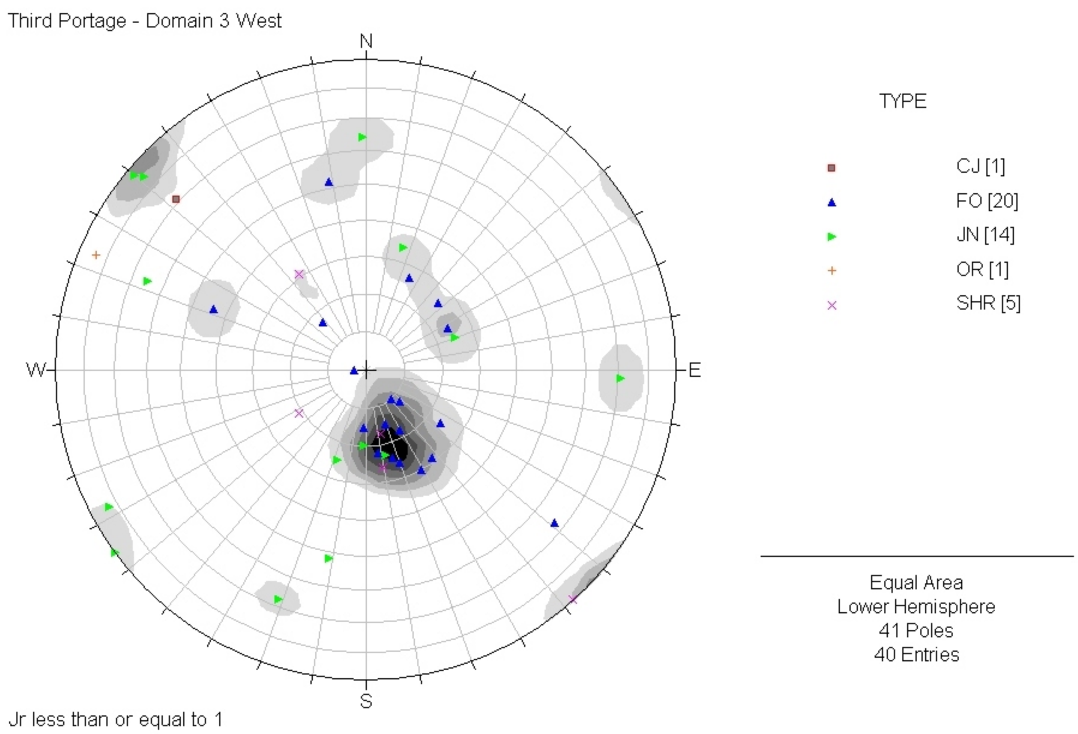
TITLE
PORTAGE
STRUCTURAL DOMAIN TP-3 EAST

Golder Associates

PROJECT No.		06-1413-089	FILE No.	
DESIGN	JFG	09MAR07	SCALE NTS	REV.
CADD	GG	09MAR07	**FIGURE 5.9**	
CHECK				
REVIEW				

DRAWING DATE: 17-Mar-07 COREL FILE: N:\Bur-Graphics\Projects\2006\1413\06-1413-089\5000\Drafting\4000\061413089-5000-4000-B_dips.cdr





PROJECT

MEADOWBANK
MINING CORPORATION

TITLE

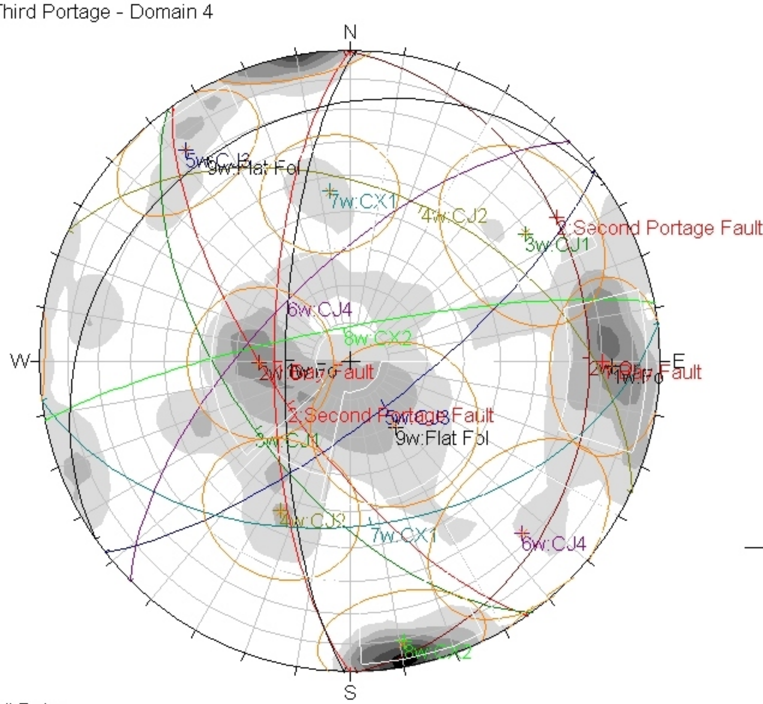
PORTAGE
STRUCTURAL DOMAIN TP-3 WEST

Golder Associates

PROJECT No.	06-1413-089		FILE No.	
DESIGN	JFG	09MAR07	SCALE NTS	REV.
CADD	GG	09MAR07		
CHECK			**FIGURE 5.10**	
REVIEW				

DRAWING DATE: 17-Mar-07 COREL FILE: N:\Bur-Graphics\Projects\2006\1413\06-1413-089\5000\Drafting\4000\061413089-5000-4000-B_dips.cdr

Third Portage - Domain 4



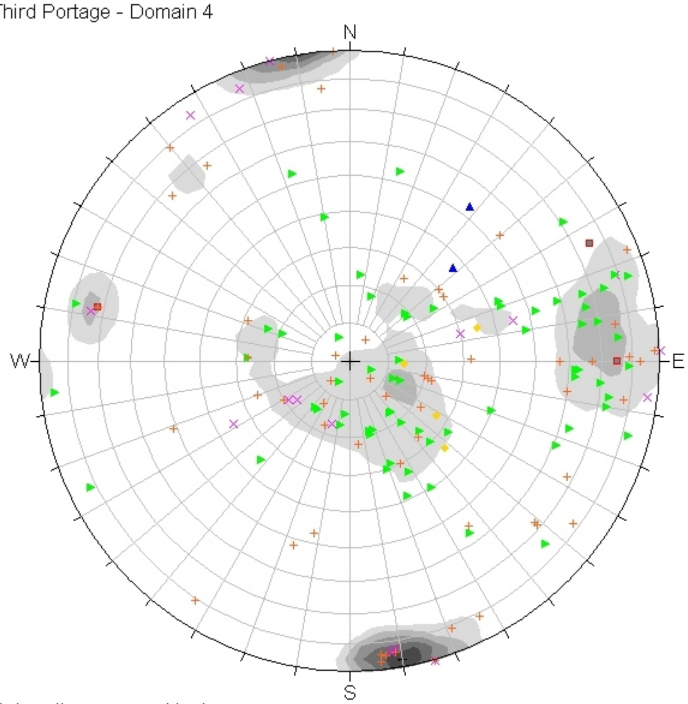
All Data

Orientations

ID		Dip / Direction
1		70 / 270
2		70 / 235
1	w	73 / 271
2	w	24 / 089
3	w	59 / 234
4	w	44 / 025
5	w	75 / 142
6	w	67 / 315
7	w	46 / 173
8	w	81 / 349
9	w	21 / 325

Equal Area
Lower Hemisphere
377 Poles
377 Entries

Third Portage - Domain 4



Jr less than or equal to 1

TYPE

- CO [3]
- FLT [2]
- FO [69]
- JN [61]
- OR [18]
- SHR [4]

Equal Area
Lower Hemisphere
157 Poles
157 Entries

PROJECT

MEADOWBANK
MINING CORPORATION

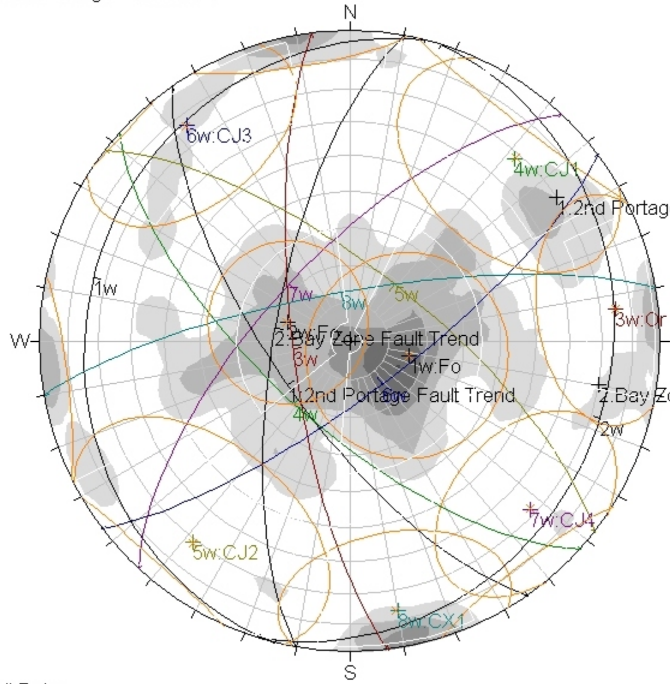
TITLE

PORTAGE
STRUCTURAL DOMAIN TP-4

Golder Associates

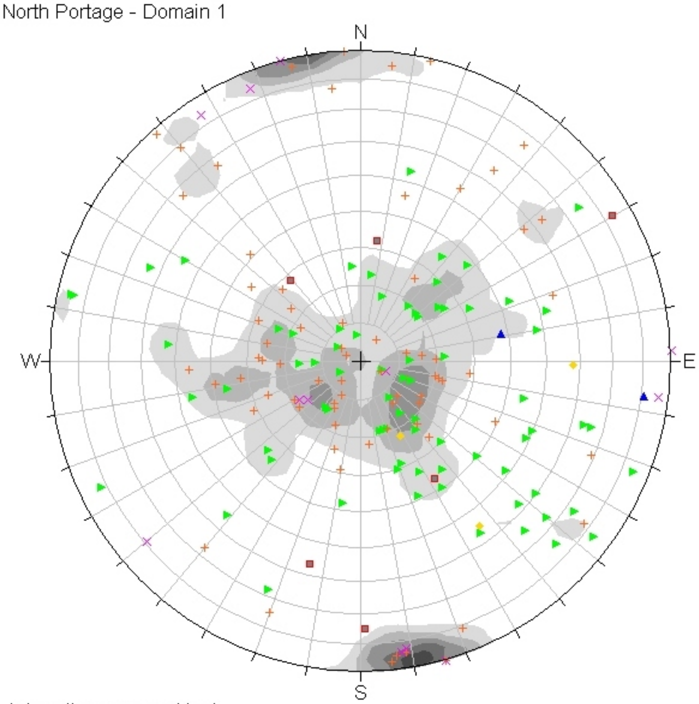
PROJECT No.	06-1413-089		FILE No.	
DESIGN	JFG	09MAR07	SCALE NTS	REV.
CADD	GG	09MAR07		
CHECK			**FIGURE 5.11**	
REVIEW				

DRAWING DATE: 17-Mar-07 COREL FILE: N:\Bur-Graphics\Projects\2006\1413\06-1413-089\5000\Drafting\4000\061413089-5000-4000-B_dips.cdr



Orientations		
ID		Dip / Direction
1		70 / 235
2		70 / 280
1	w	16 / 284
2	w	17 / 107
3	w	75 / 263
4	w	68 / 222
5	w	71 / 038
6	w	76 / 143
7	w	68 / 313
8	w	77 / 350

Equal Area
Lower Hemisphere
524 Poles
505 Entries



TYPE

- CJ [7]
- CO [2]
- FO [83]
- JN [79]
- OR [14]
- VN [3]

Equal Area
Lower Hemisphere
188 Poles
180 Entries

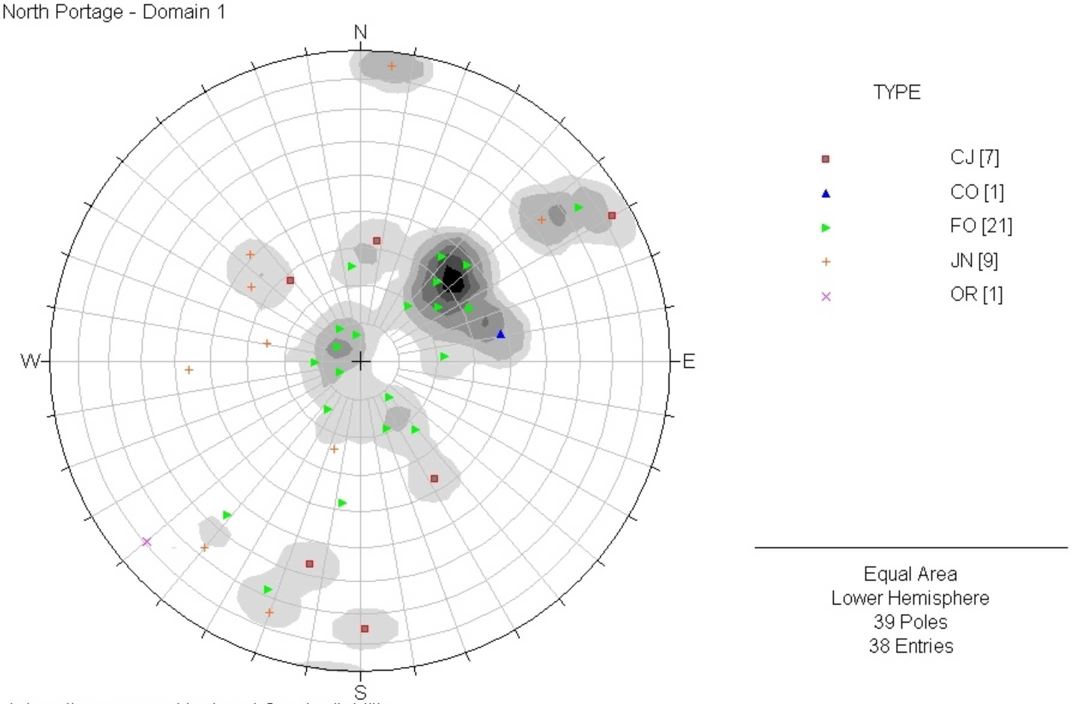
PROJECT	MEADOWBANK MINING CORPORATION
TITLE	PORTAGE STRUCTURAL DOMAIN NP-1

Golder Associates

PROJECT No.	06-1413-089		FILE No.	
DESIGN	JFG	09MAR07	SCALE NTS	REV.
CADD	GG	09MAR07	FIGURE 5.12	
CHECK				
REVIEW				

DRAWING DATE: 17-Mar-07 COREL FILE: N:\Bur-Graphics\Projects\2006\1413\06-1413-089\5000\Drafting\4000\061413089-5000-4000-B_dips.cdr

North Portage - Domain 1



Jr less than or equal to 1 and Good reliability

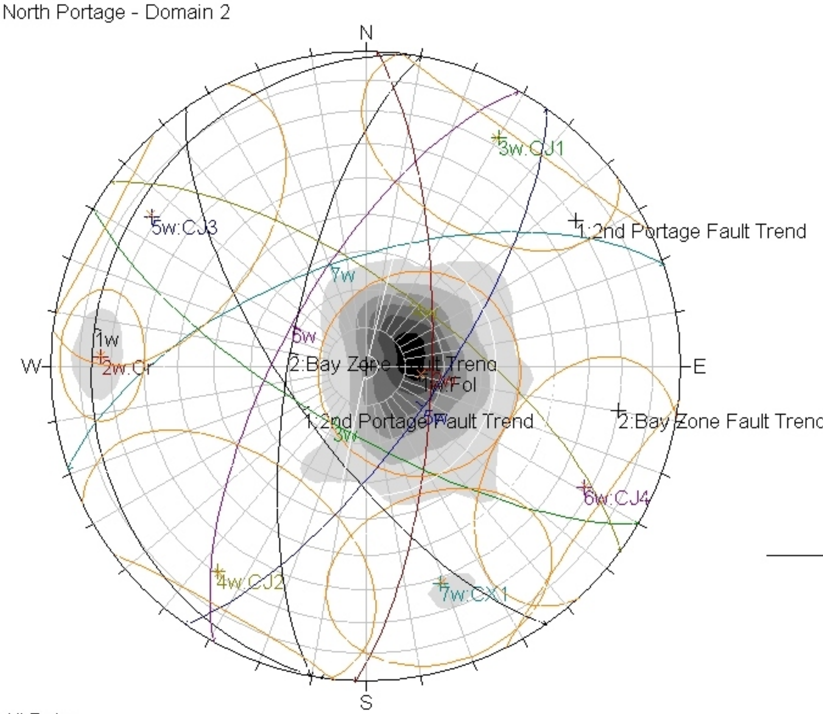
DRAWING DATE: 17-Mar-07 COREL FILE: N:\Bur-Graphics\Projects\2006\1413\06-1413-089\5000\Drafting\4000\061413089-5000-4000-B_dips.cdr

PROJECT	**MEADOWBANK MINING CORPORATION**
TITLE	**PORTAGE - STRUCTURAL DOMAIN NP-1 Jr LESS THAN OR EQUAL TO 1 AND GOOD RELIABILITY**

Golder Associates

PROJECT No.	06-1413-089		FILE No.
DESIGN	JFG	09MAR07	SCALE NTS REV.
CADD	GG	09MAR07	
CHECK			**FIGURE 5.13**
REVIEW			

North Portage - Domain 2



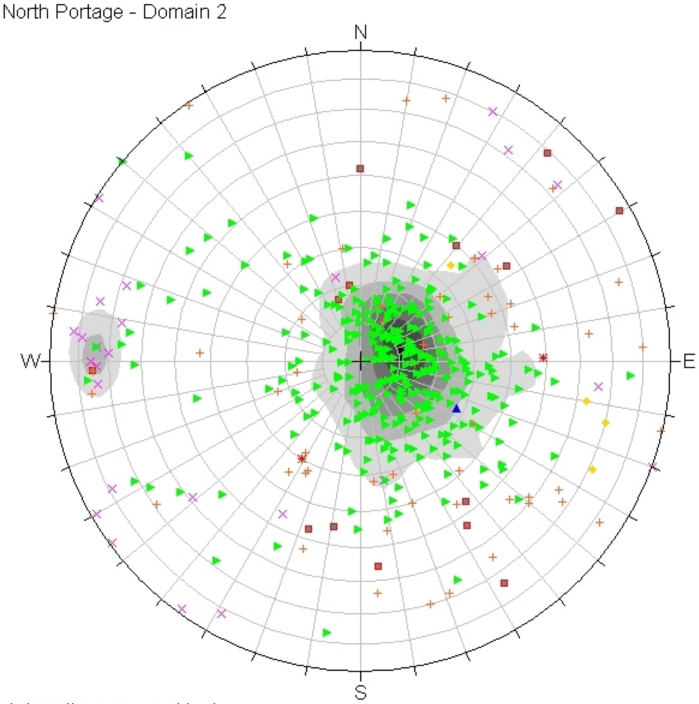
Orientations

ID		Dip / Direction
1		70 / 235
2		70 / 280
1	w	14 / 278
2	w	73 / 092
3	w	73 / 210
4	w	69 / 036
5	w	72 / 125
6	w	68 / 299
7	w	62 / 341

Equal Area
Lower Hemisphere
851 Poles
827 Entries

All Data

North Portage - Domain 2



TYPE

- CJ [14]
- CO [1]
- FO [366]
- JN [58]
- OR [28]
- SHR [4]
- VN [2]

Equal Area
Lower Hemisphere
473 Poles
460 Entries

Jr less than or equal to 1

DRAWING DATE: 17-Mar-07 COREL FILE: N:\Bur-Graphics\Projects\2006\1413\06-1413-089\5000\Drafting\4000\061413089-5000-4000-B_dips.cdr

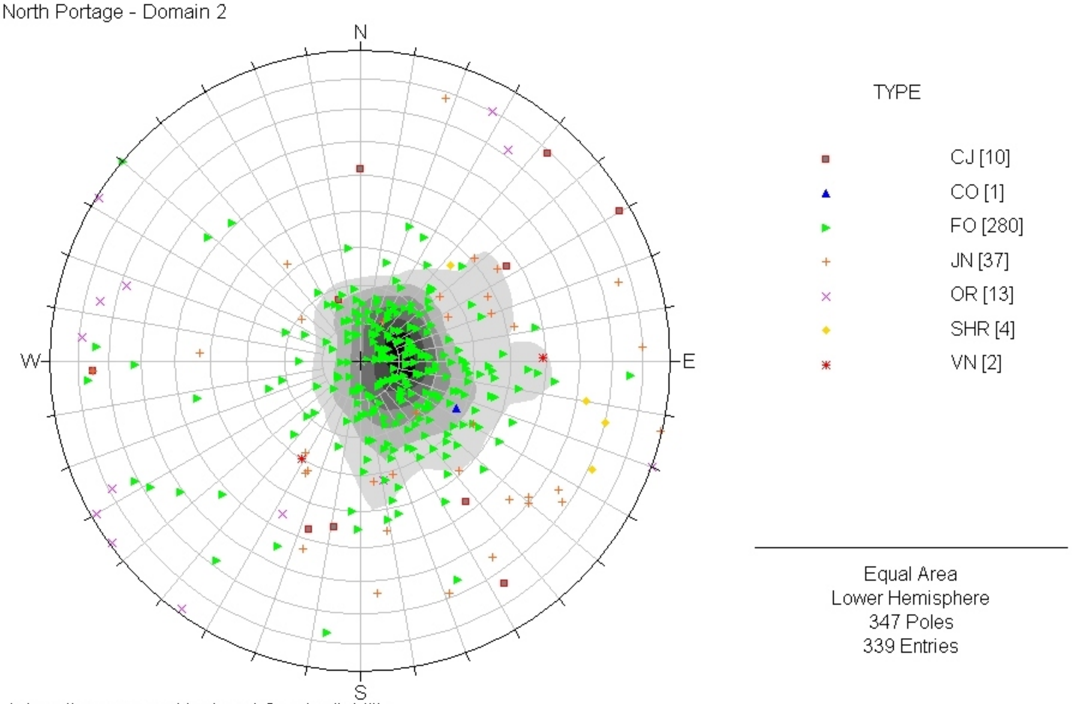
PROJECT
MEADOWBANK
MINING CORPORATION

TITLE
PORTAGE
STRUCTURAL DOMAIN NP-2

Golder Associates

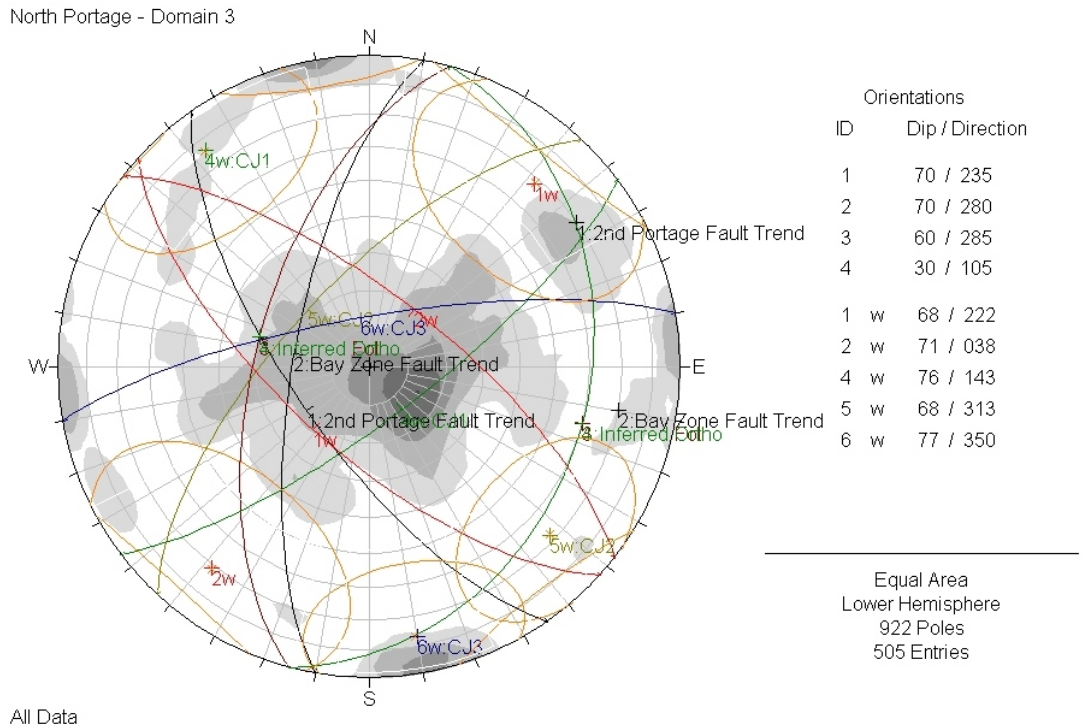
PROJECT No.	06-1413-089		FILE No.	
DESIGN	JFG	09MAR07	SCALE NTS	REV.
CADD	GG	09MAR07		
CHECK			**FIGURE 5.14**	
REVIEW				

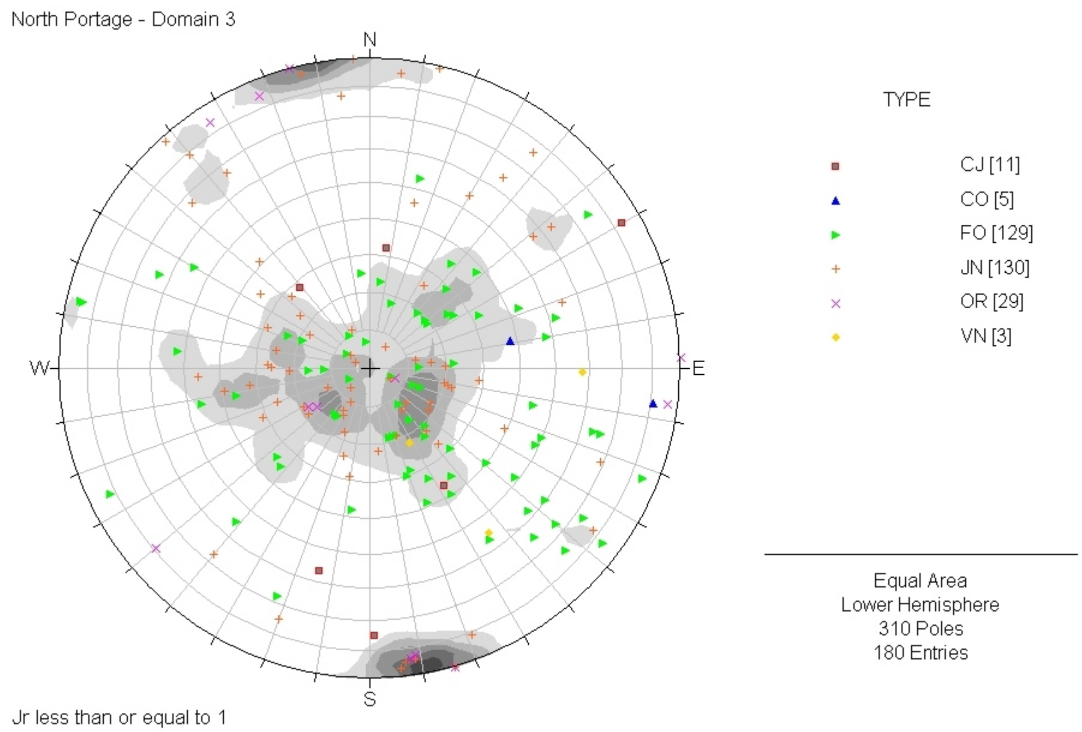
North Portage - Domain 2



Jr less than or equal to 1 and Good reliability

PROJECT	MEADOWBANK MINING CORPORATION			
TITLE	PORTAGE - STRUCTURAL DOMAIN NP-2 Jr LESS THAN OR EQUAL TO 1 AND GOOD RELIABILITY			
Golder Associates	PROJECT No.	06-1413-089		FILE No.
	DESIGN	JFG	09MAR07	SCALE NTS / REV.
	CADD	GG	09MAR07	FIGURE 5.15
	CHECK			
	REVIEW			





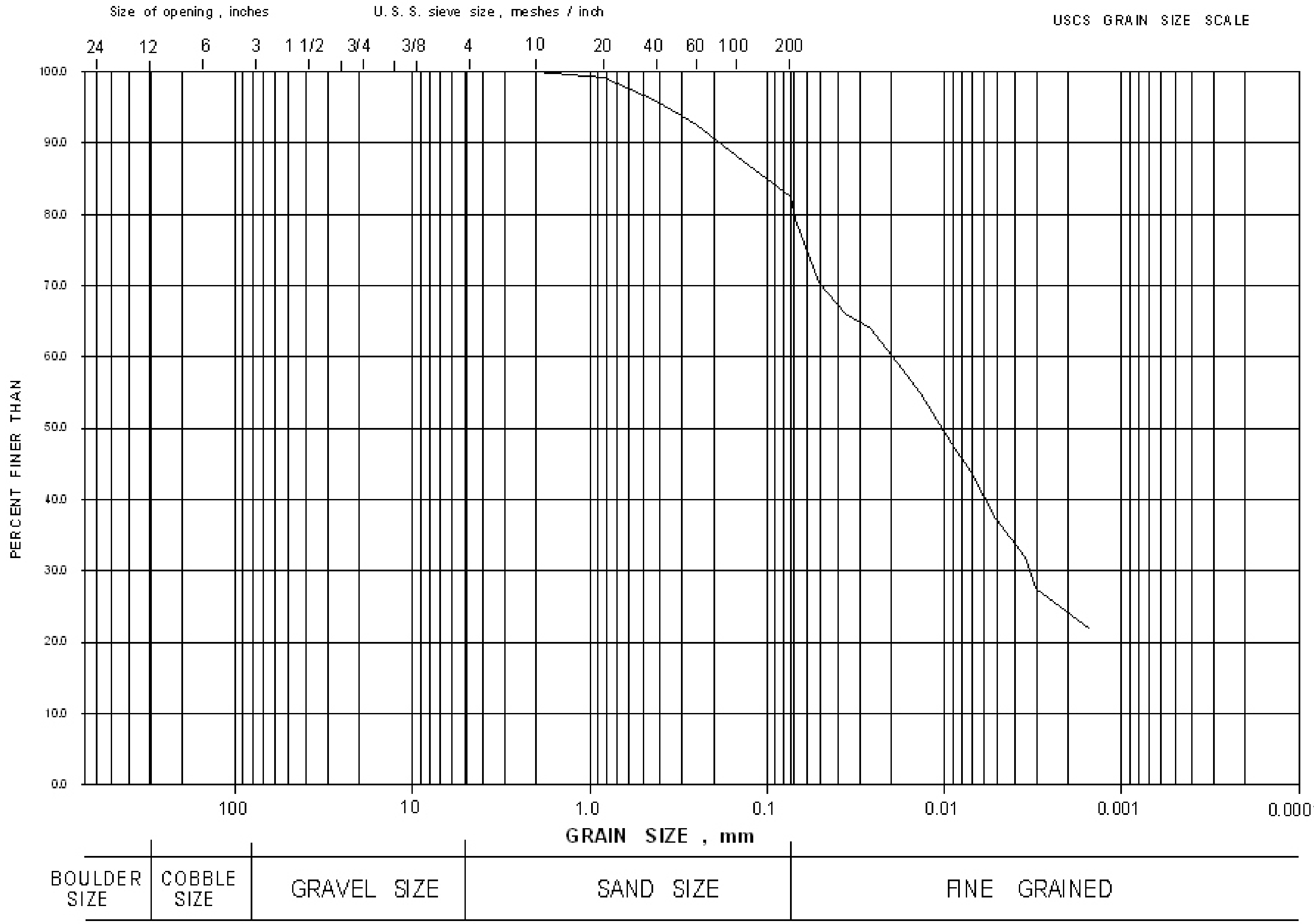
PROJECT	**MEADOWBANK MINING CORPORATION**
TITLE	**PORTAGE STRUCTURAL DOMAIN NP-3**

Golder Associates

PROJECT No.	06-1413-089		FILE No.	
DESIGN	JFG	09MAR07	SCALE NTS	REV.
CADD	GG	09MAR07		
CHECK			**FIGURE 5.16**	
REVIEW				

DRAWING DATE: 17-Mar-07 COREL FILE: N:\Bur-Graphics\Projects\2006\1413\06-1413-089\5000\Drafting\4000\061413089-5000-4000-B_dips.cdr

DRAWING DATE: 02-Apr-07 COREL FILE: N:\Bur-Graphics\Projects\2006\1413\06-1413-089\5000\Drafting\4000\Grapher.cdr

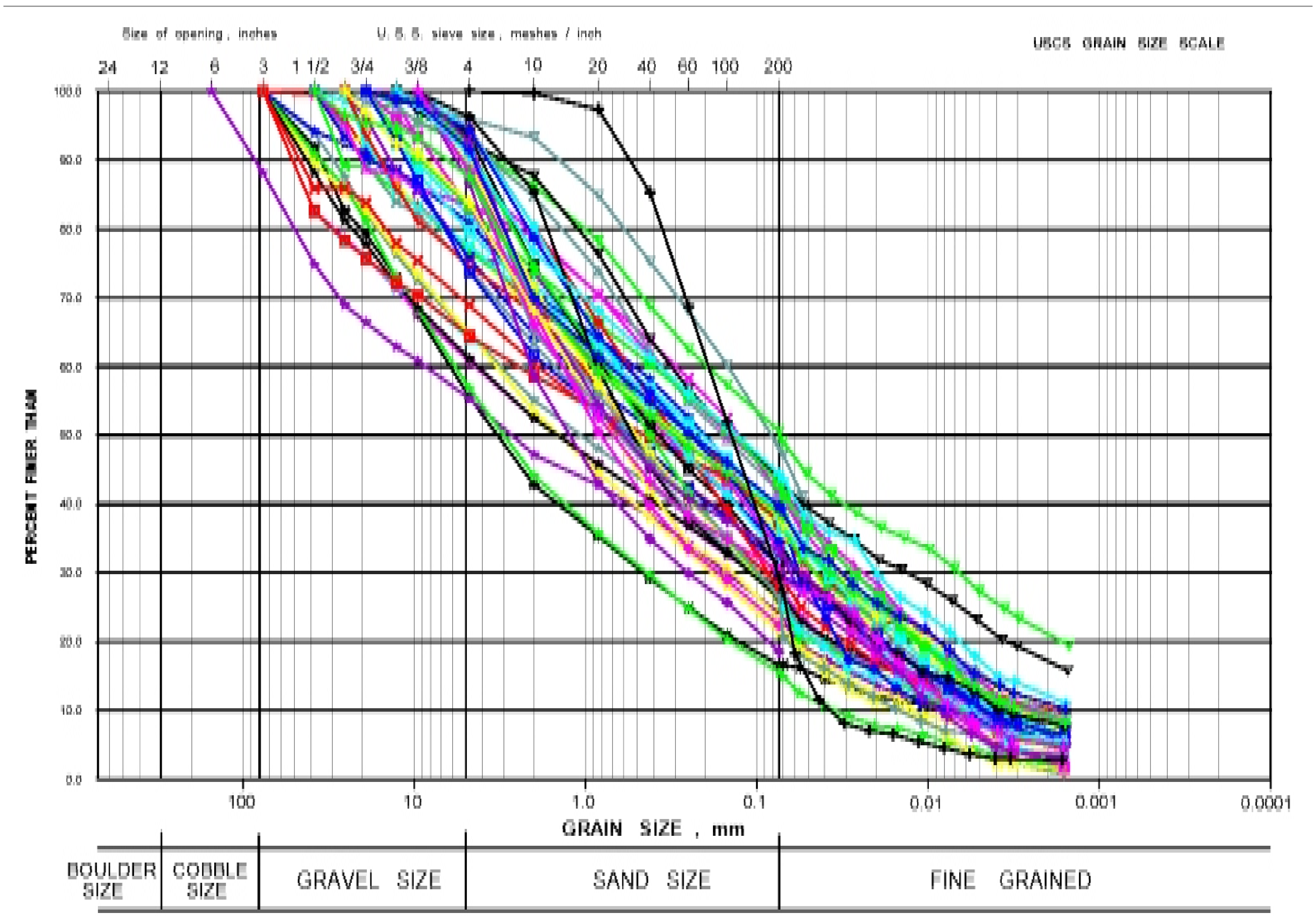


PROJECT	**MEADOWBANK MINING CORPORATION**
TITLE	**GRAIN SIZE LAKEBED SEDIMENTS**

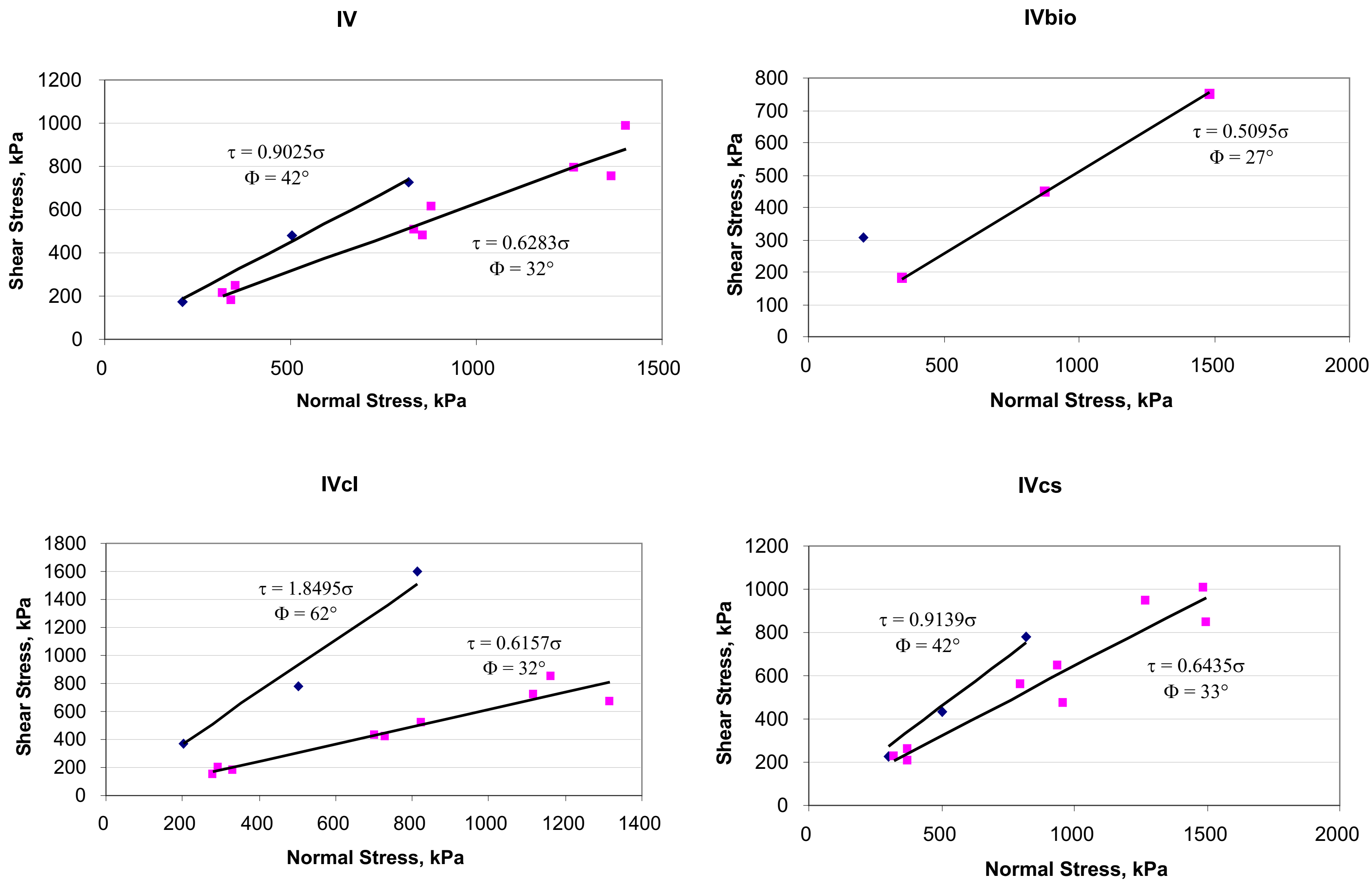
Golder Associates

PROJECT No.	06-1413-089		FILE No. Grapher	
DESIGN	JFG	02APR07	SCALE NTS	REV.
CADD	AS	02APR07	**FIGURE 6.1**	
CHECK				
REVIEW				

DRAWING DATE: 02-Apr-07 COREL FILE: N:\Bur-Graphics\Projects\2006\1413\06-1413-089\5000\Drafting\4000\Grapher.cdr



PROJECT	**MEADOWBANK MINING CORPORATION**			
TITLE	**GRAIN SIZE TILL**			
Golder Associates	PROJECT No.	06-1413-089	FILE No. Grapher	
	DESIGN	JFG	02APR07	SCALE NTS	REV.
	CADD	AS	02APR07	**FIGURE 6.2**
	CHECK			
	REVIEW			



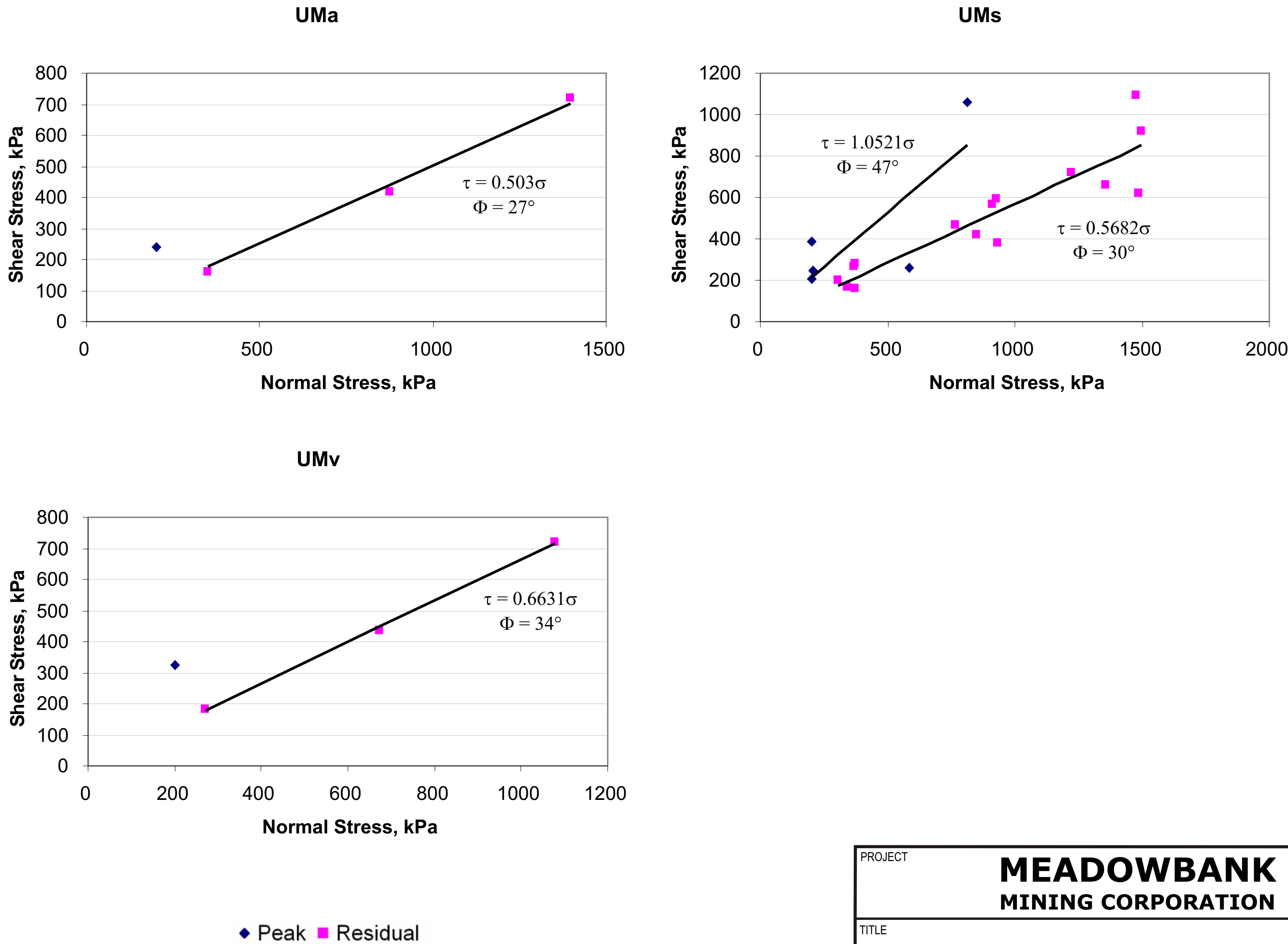
PROJECT
MEADOWBANK
MINING CORPORATION

TITLE
DIRECT SHEAR TESTING RESULTS
INTERMEDIATE VOLCANICS

Golder Associates

PROJECT No.	06-1413-089		FILE No.	
DESIGN	JFG	09MAR07	SCALE NTS	REV.
CADD	GG	09MAR07	FIGURE 6.3	
CHECK				
REVIEW				

DRAWING DATE: 09-Mar-07 COREL FILE: N:\Bur-Graphics\Projects\2006\1413\06-1413-089\5000\Drafting\4000\061413089-5000-4000-A_shears.cdr



PROJECT

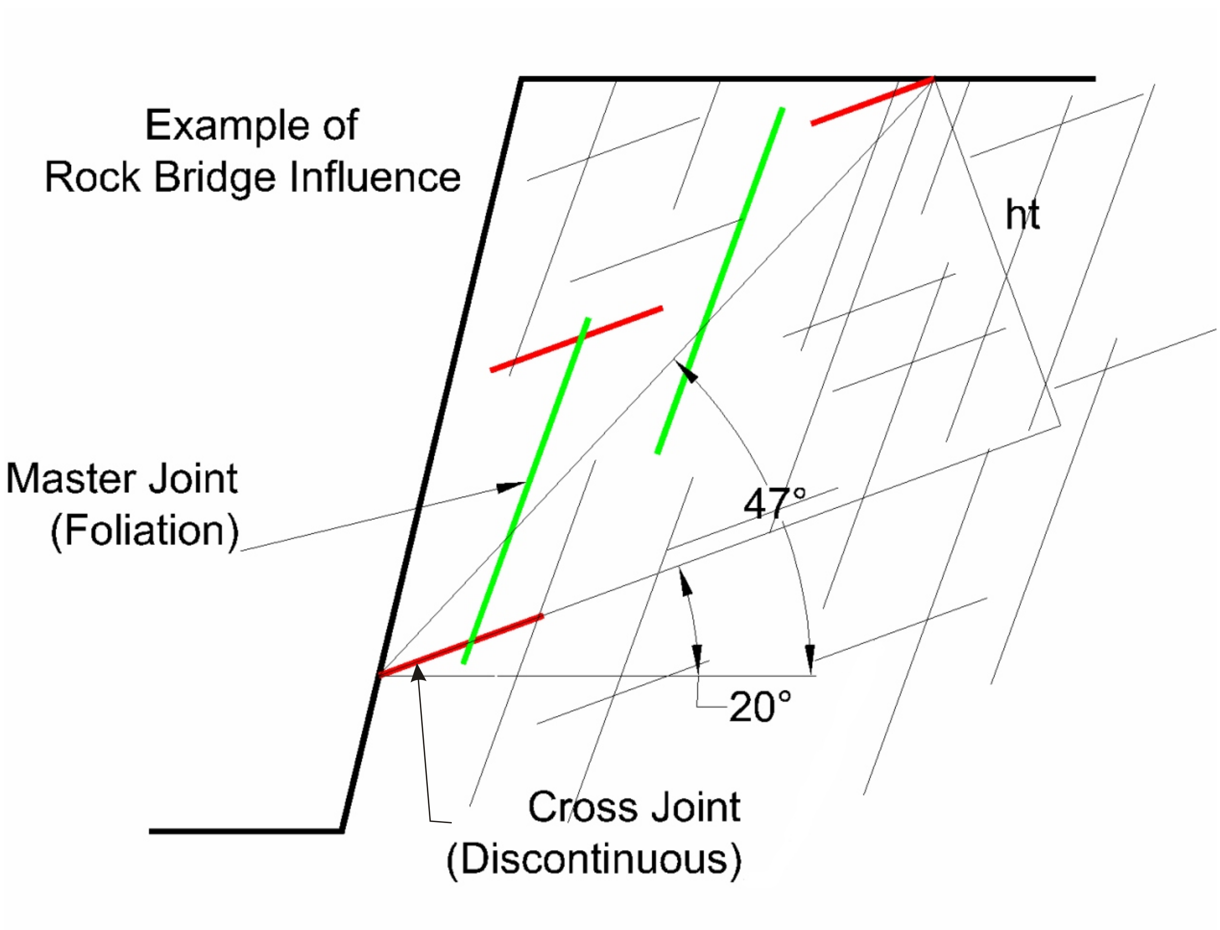
MEADOWBANK
MINING CORPORATION

TITLE

DIRECT SHEAR TESTING RESULTS
ULTRAMAFICS

Golder Associates

PROJECT No.	06-1413-089		FILE No.	
DESIGN	JFG	09MAR07	SCALE NTS	REV.
CADD	GG	09MAR07	**FIGURE 6.4**	
CHECK				
REVIEW				

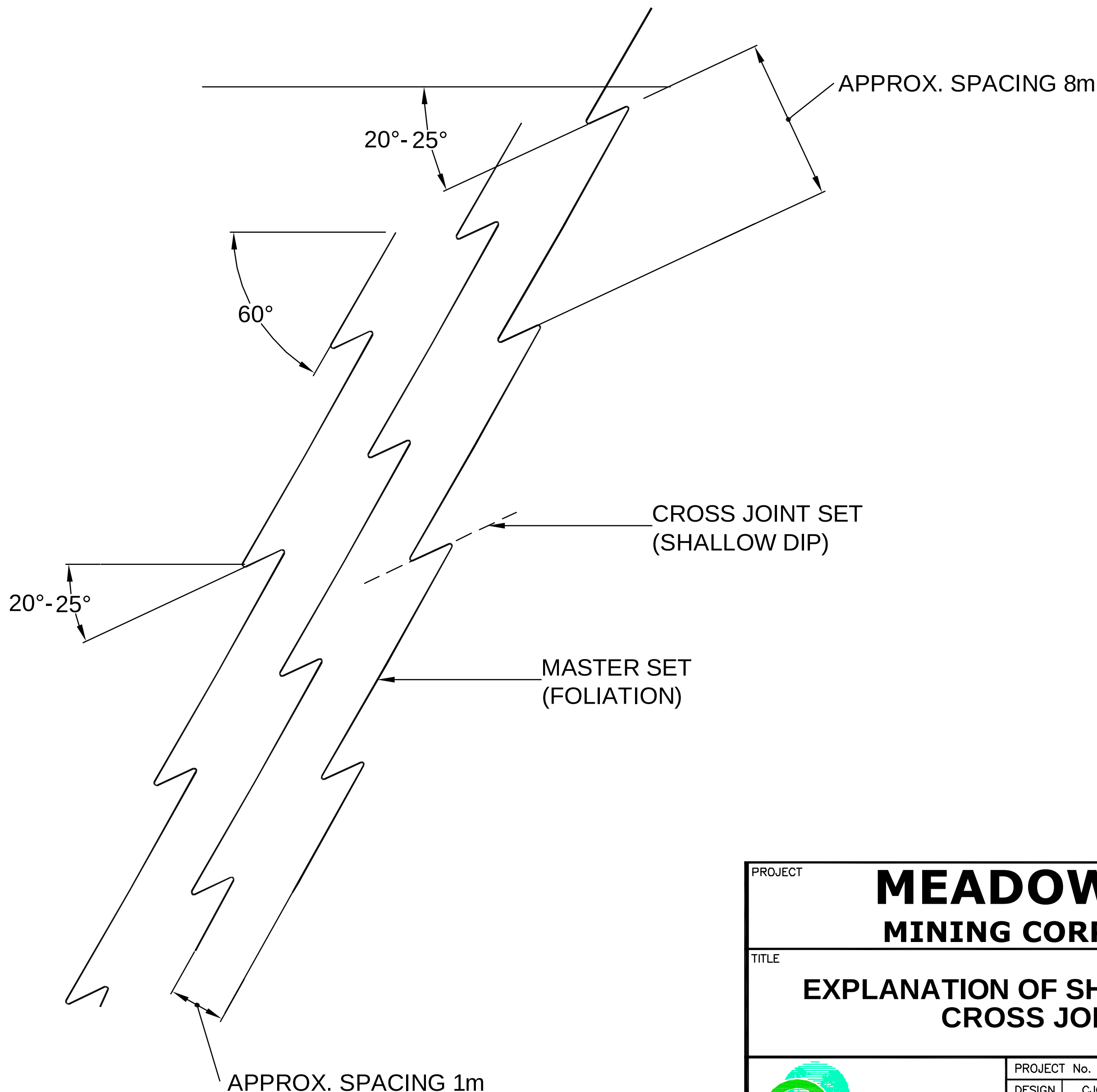


NOTE

Figure is based on drawing found in: Miller, S. M., Whyatt, J., K., and McHugh, E. L. Applications of the Point Estimation Method for Stochastic Rock Slope Engineering. Gulf Rocks 2004: Proceedings, Rock Mechanics Across Borders & Disciplines, 6th North American Rock Mechanics Conference, June 5-10, 2004, Houston, Texas. Report No. ARMA/NARMS 04-517. Alexandria, VA: American Rock Mechanics Association, 2004 Jun; :1-12

PROJECT	**MEADOWBANK MINING CORPORATION**				
TITLE	**GENERALIZED ROCK BRIDGE**				
Golder Associates	PROJECT No.	06-1413-089		FILE No.	
	DESIGN	JFG	09MAR07	SCALE NTS	REV.
	CADD	GG	09MAR07	**FIGURE 6.5**	
	CHECK				
	REVIEW				

DRAWING DATE: 15-Mar-07 COREL FILE: N:\Bur-Graphics\Projects\2006\1413\06-1413-089\5000\Drafting\4000\061413089-5000-4000-A_6.3.cdr



PROJECT

MEADOWBANK
MINING CORPORATION

TITLE

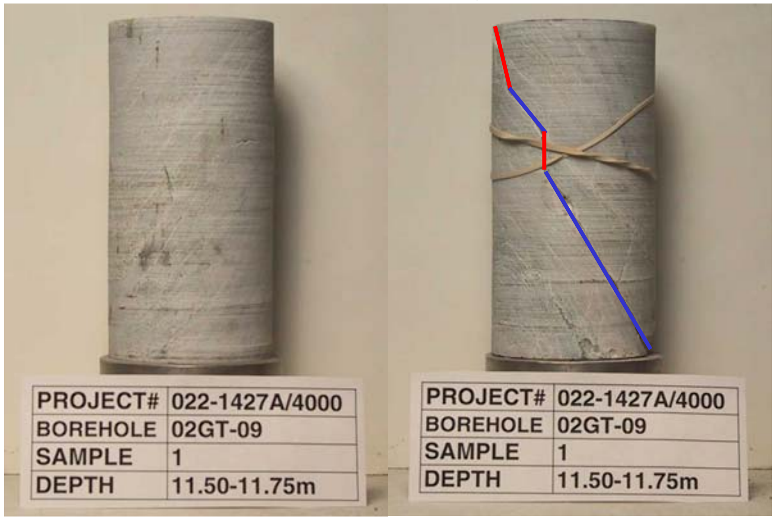
EXPLANATION OF SHALLOW DIPPING CROSS JOINT SET

Golder Associates

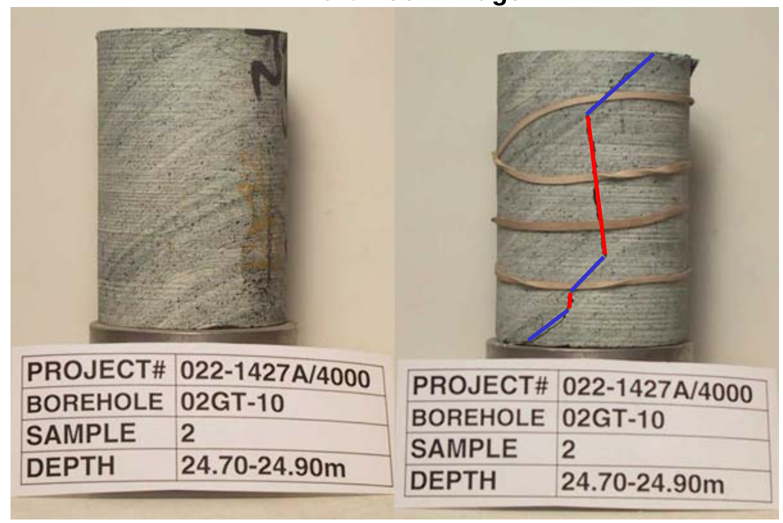
PROJECT No.	06-1413-089		FILE No. 5000-4000-11
DESIGN	CJC	19MAR07	SCALE AS SHOWN REV. –
CADD	AS	19MAR07	**FIGURE 6.6**
CHECK			
REVIEW			

DRAWING DATE: 19-Mar-07 FILE: N:\Bur-Graphics\Projects\2006\1413\06-1413-089\5000\Drafting\4000\step path failure.ppt

~20% Rock Bridge



~40% Rock Bridge



~40% Rock Bridge

LEGEND

Broken Along Natural Discontinuity

Broken Along Rock Bridge

PROJECT

MEADOWBANK MINING CORPORATION

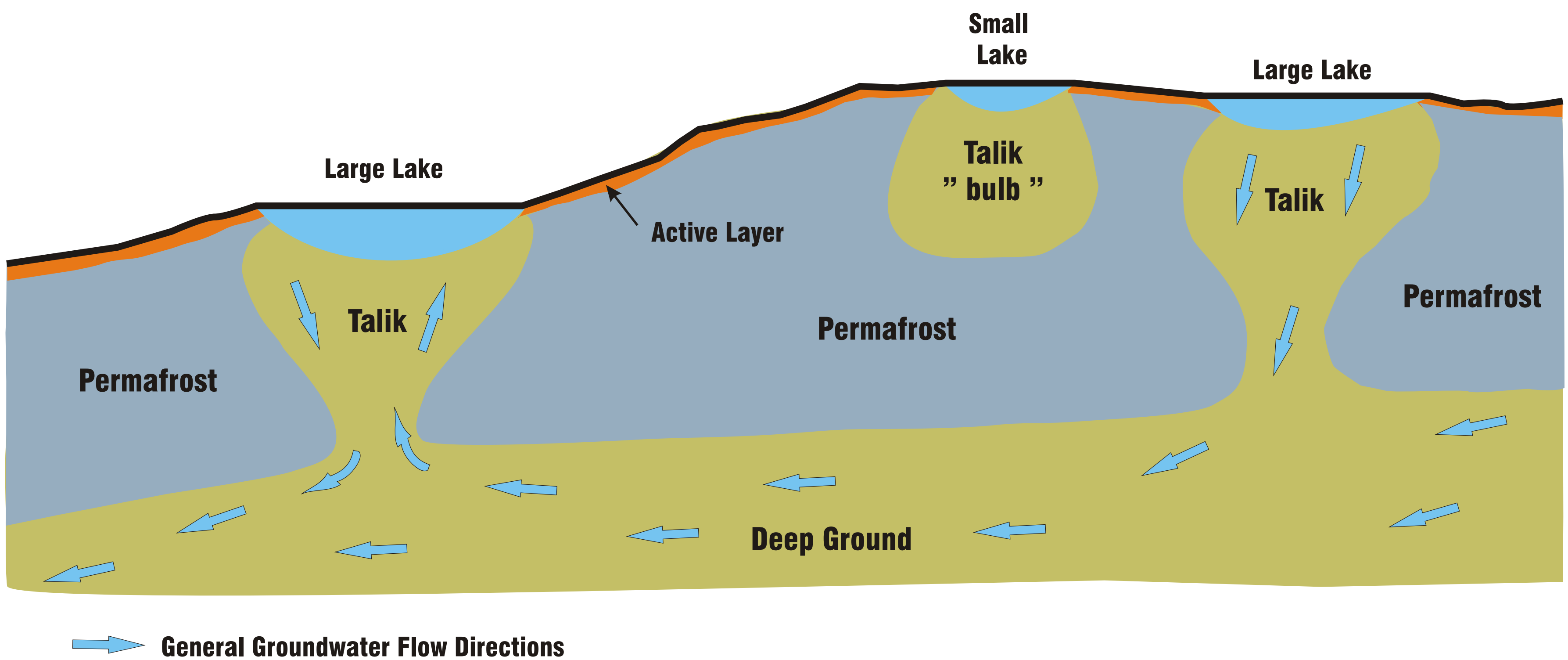
TITLE

CONCEPT OF STEP PATH FAILURE MECHANISMS

Golder Associates

PROJECT No.	06-1413-089		FILE No. ----	
DESIGN	JFG	15MAR07	SCALE NTS	REV.
CADD	JFG	15MAR07	**FIGURE 6.7**	
CHECK	--			
REVIEW				

DRAWING DATE: 12-Mar-07 COREL FILE: N:\Bur-Graphics\Projects\2006\1413\06-1413-089\061413089SK03.cdr



PROJECT

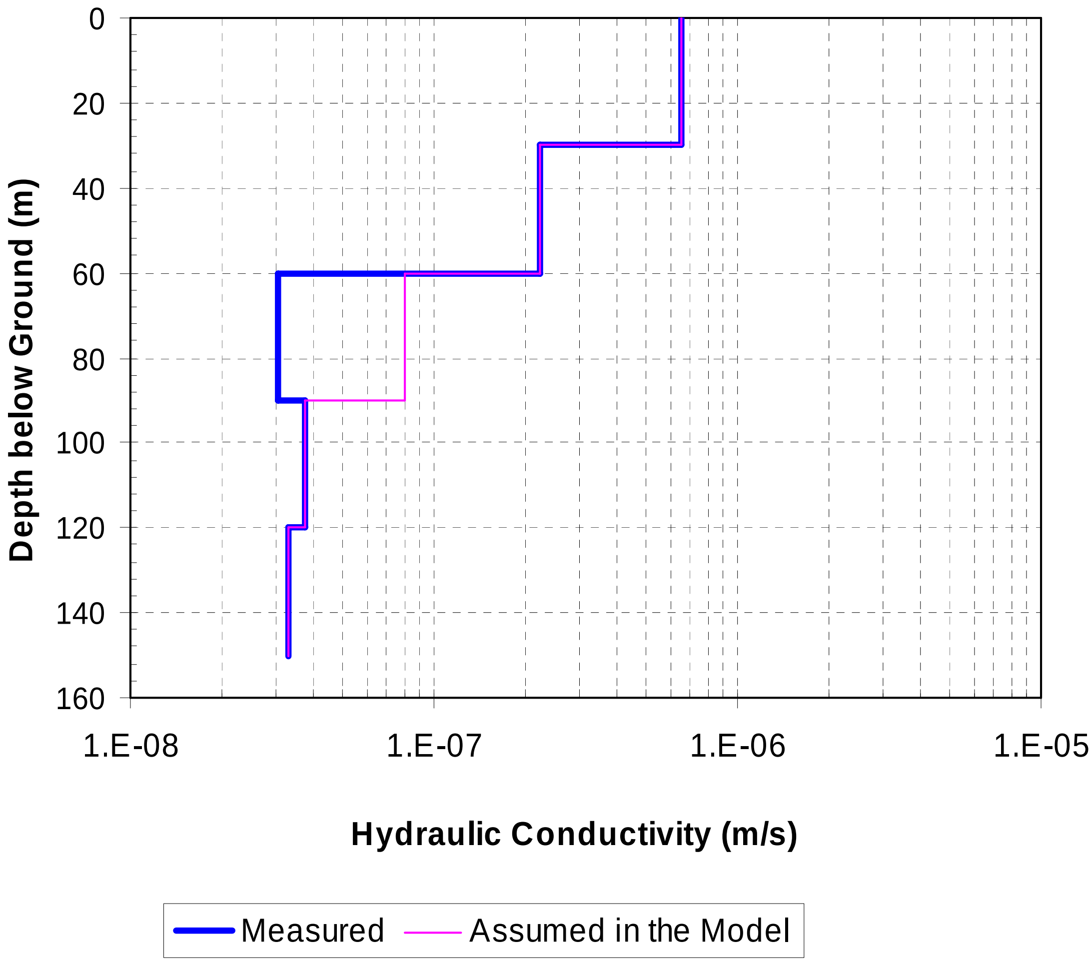
MEADOWBANK
MINING CORPORATION

TITLE

SCHEMATIC
GROUNDWATER FLOW CONDITIONS
PERMAFROST

Golder Associates

PROJECT No.	06-1413-089		FILE No. 061413089Sk03	
DESIGN	CB	02MAR07	SCALE NTS	REV.
CADD	AS	02MAR07	FIGURE 7.1	
CHECK				
REVIEW				



	Hydraulic Conductivity (m/s)	
Depth Below Ground (m)	**Measured**	**Assumed in the Model**
0-30	7.E-07	7.E-07
30-60	2.E-07	2.E-07
60-90	3.E-08	8.E-08
90-120	4.E-08	4.E-08
120-160	3.E-08	3.E-08

Note: The geometric mean of hydraulic conductivity measurements was calculated over the depth intervals in the table above.

PROJECT

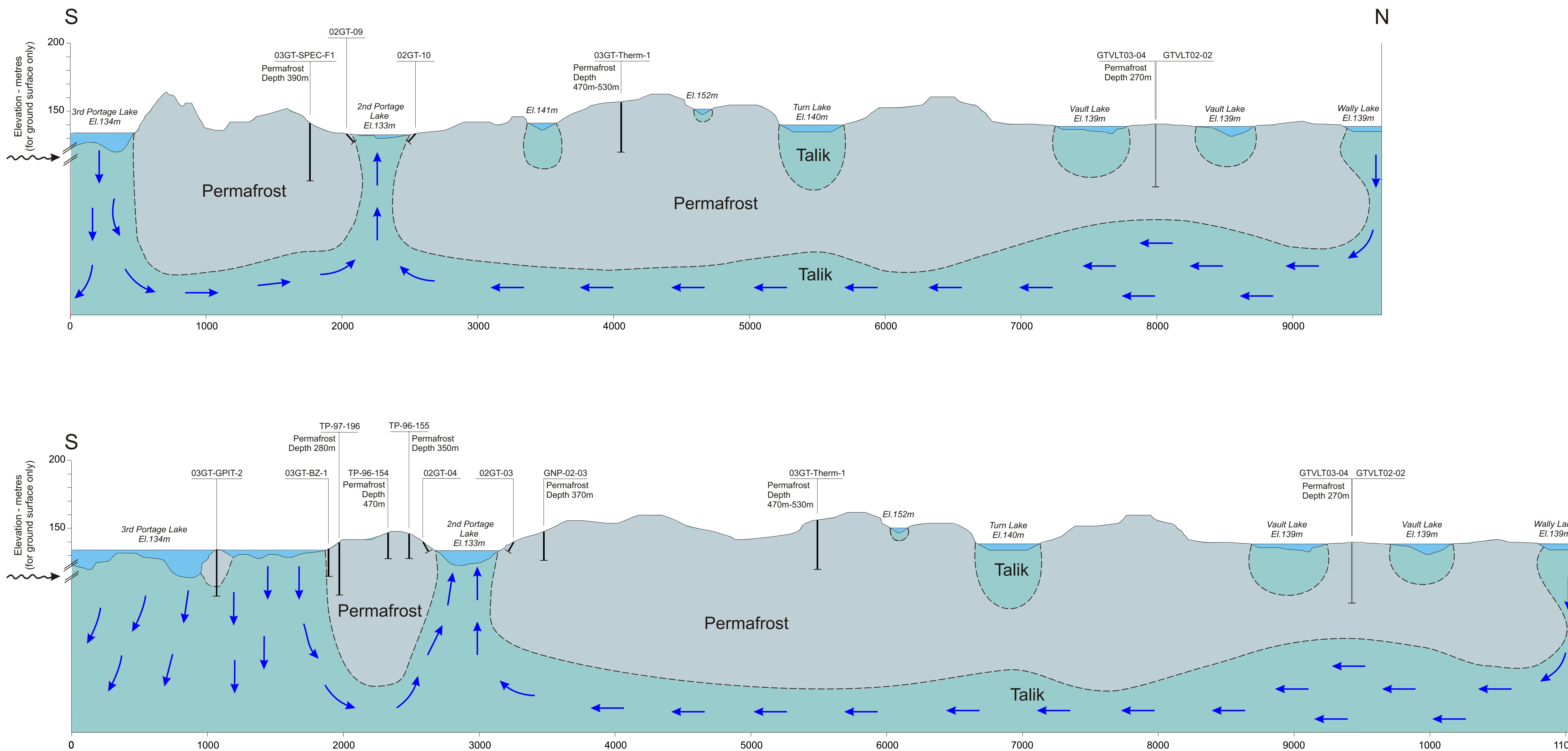
MEADOWBANK
MINING CORPORATION

TITLE

HYDRAULIC CONDUCTIVITY VS DEPTH PROFILE IN BEDROCK

Golder Associates

PROJECT No.	06-1413-089		FILE No. ----	
DESIGN	CB	01FEB07	SCALE NTS	REV.
CADD	CB	01FEB07	**FIGURE 7.2**	
CHECK	--	01FEB07		
REVIEW				



0 400 800 1200 1600 2000

Horizontal Scale (m)

0 40 80 120 160 200

Vertical Scale (m)
(above ground surface)

0 100 200 300 400 500

Depth Scale (m)
(For below ground surface.
Permafrost depth measured from ground surface.)

PROJECT

MEADOWBANK
MINING CORPORATION

TITLE

BASELINE PERMAFROST AND GROUNDWATER CONDITIONS

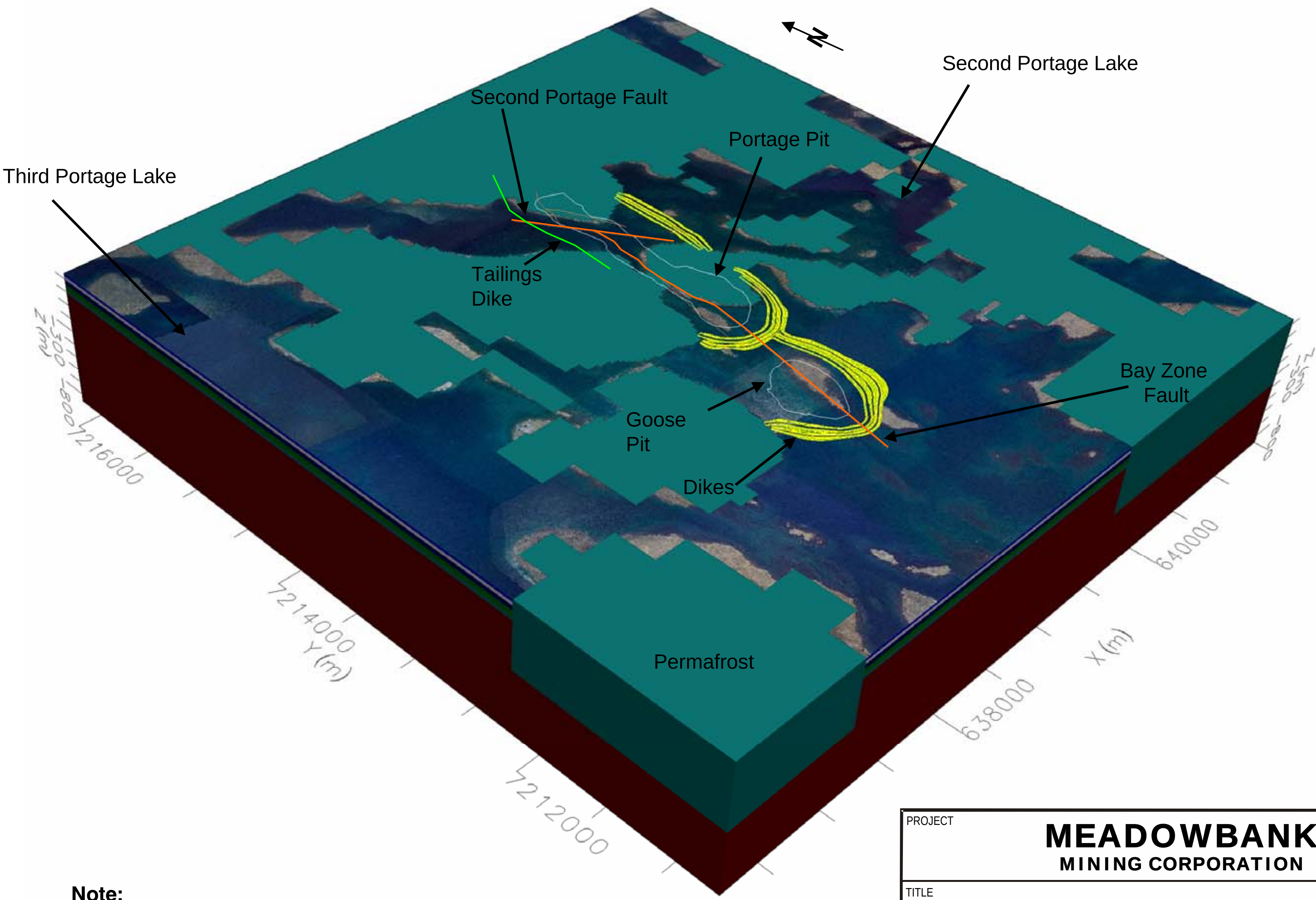
Golder Associates

PROJECT No.	06-1413-089		
DESIGN	CB	02MAR07	
CADD	AS	02MAR07	
CHECK			
REVIEW			

FILE No. Figure 7.3 | SCALE NTS | REV.

FIGURE 7.3

DRAWING DATE: 05-Apr-07 COREL FILE: N:\Bur-Graphics\Projects\2006\1413\06-1413-089\5000\Drafting\4000\Figure 7.3.cdr



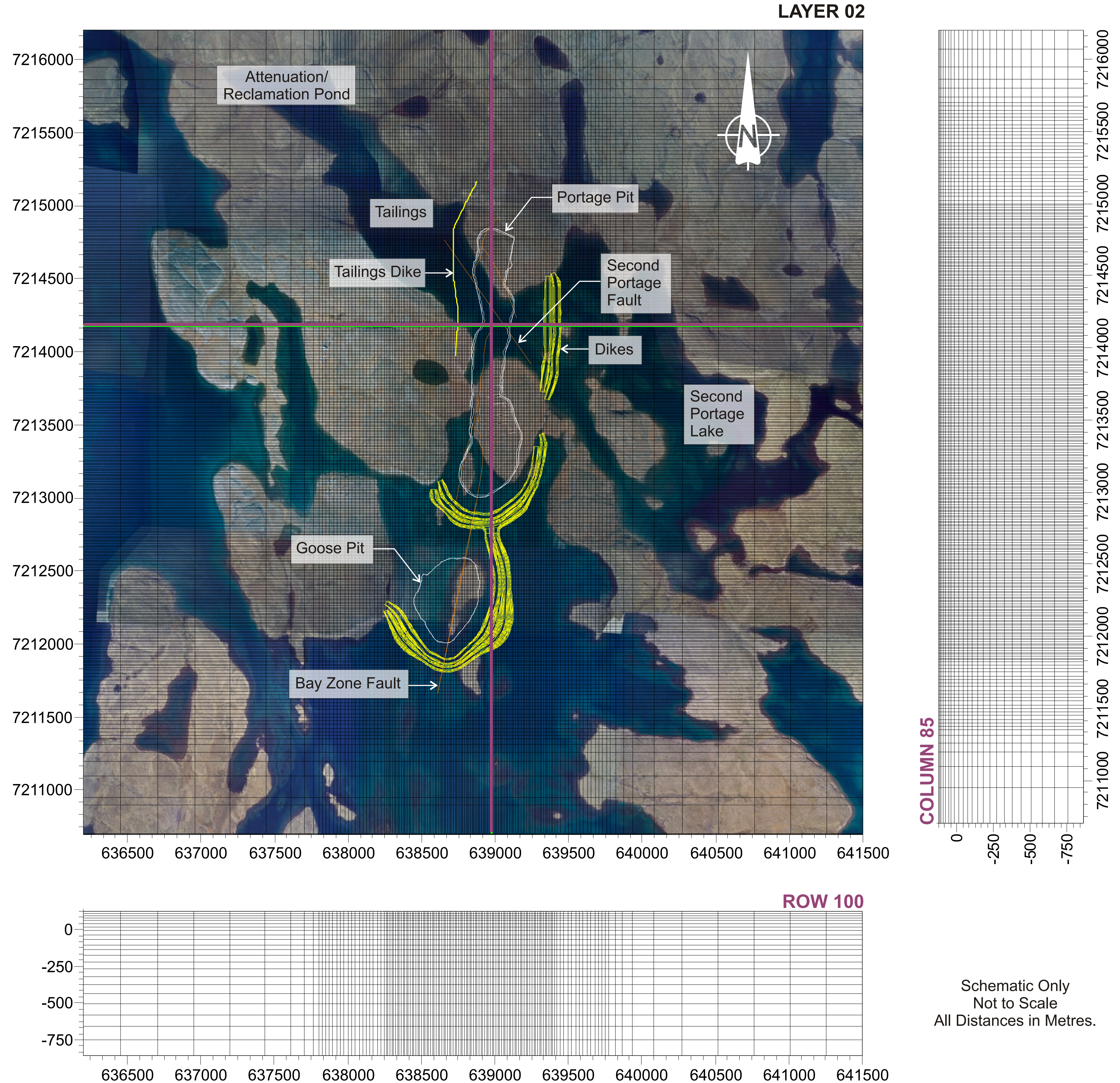
Note:
Schematic Only
NOT TO SCALE

PROJECT
MEADOWBANK
MINING CORPORATION

TITLE
MODEL EXTENTS

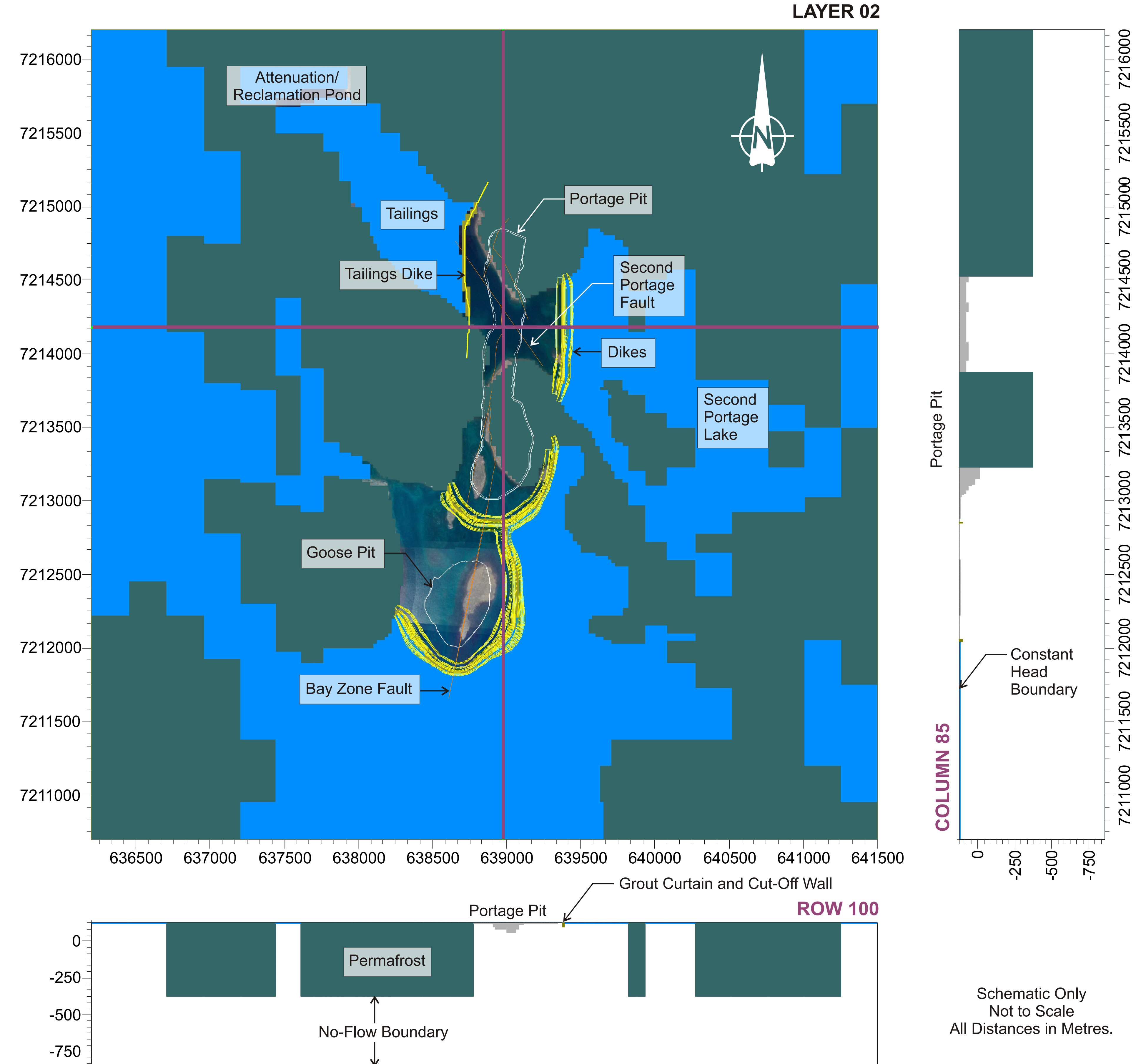
Golder Associates

PROJECT No.	06-1413-089		FILE No. ----
DESIGN	CB	06FEB07	SCALE NTS REV.
CADD	CB	06FEB07	**FIGURE 7.4**
CHECK	WZ	06FEB07	
REVIEW			



Schematic Only
Not to Scale
All Distances in Metres.

PROJECT	MEADOWBANK MINING CORPORATION			
TITLE	MODEL GRID			
Golder Associates	PROJECT No. 06-1413-089			FILE No.
	DESIGN	CB	14FEB07	SCALE NTS / REV.
	CADD	GG	14FEB07	FIGURE 7.5
	CHECK			
	REVIEW			



Schematic Only
Not to Scale
All Distances in Metres.

LEGEND

- Head Dependant Boundary
- Constant Head Boundary

NOTES

1. Grout curtain and cut-off wall were simulated using horizontal flow barrier package.

2. Initial conditions set to 134 m everywhere as small differences in lake elevations have negligible impact on model predictions.

PROJECT

MEADOWBANK MINING CORPORATION

TITLE

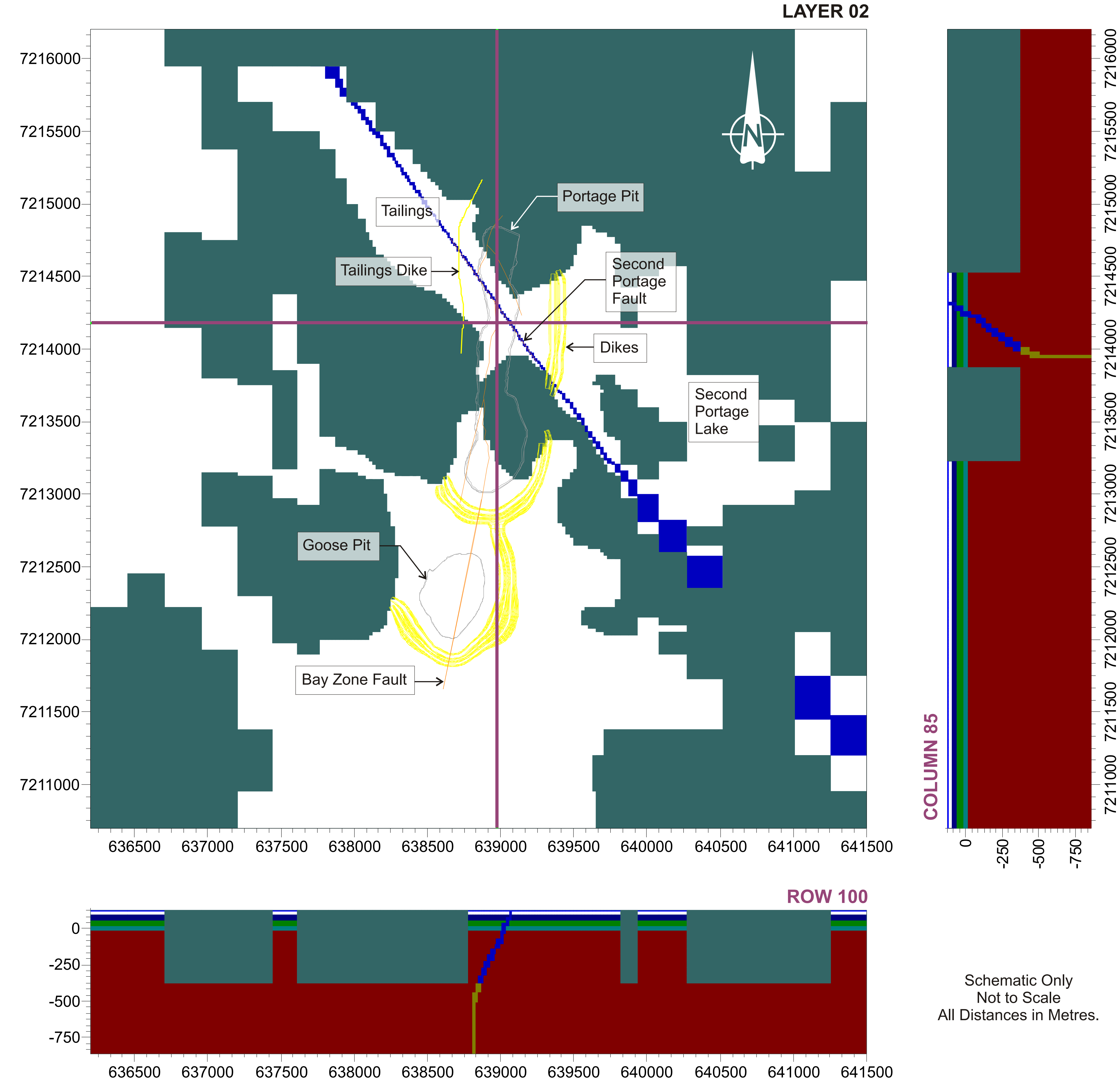
BOUNDARY CONDITIONS

PROJECT No.	06-1413-089		FILE No.
DESIGN	CB	14FEB07	SCALE NTS REV.
CADD	GG	14FEB07	
CHECK			
REVIEW			

Golder Associates

FIGURE 7.6

DRAWING DATE: 17-Mar-07 COREL FILE: N:\Bur-Graphics\Projects\2006\1413\06-1413-089\5000\Drafting\4000\Figure 7.5-7.7.cdr



LEGEND

- Permafrost
- Till Overburden Sediments
- Competent Rock (5-30 m depth)
- Competent Rock (30-65 m depth)
- Competent Rock (65-105 m depth)
- Competent Rock (105-135 m depth)
- Second Portage Fault and Fractured Rock Zone (5-500 m depth)
- Second Portage Fault and Fractured Rock Zone (500-1000 m depth)
- Competent Rock (140-1000 m depth)

NOTE

To account for the difference in width between Second Portage Fault (5 m) and the refined grid block size of 13 m, an equivalent hydraulic conductivity was assigned.

Schematic Only
Not to Scale
All Distances in Metres.

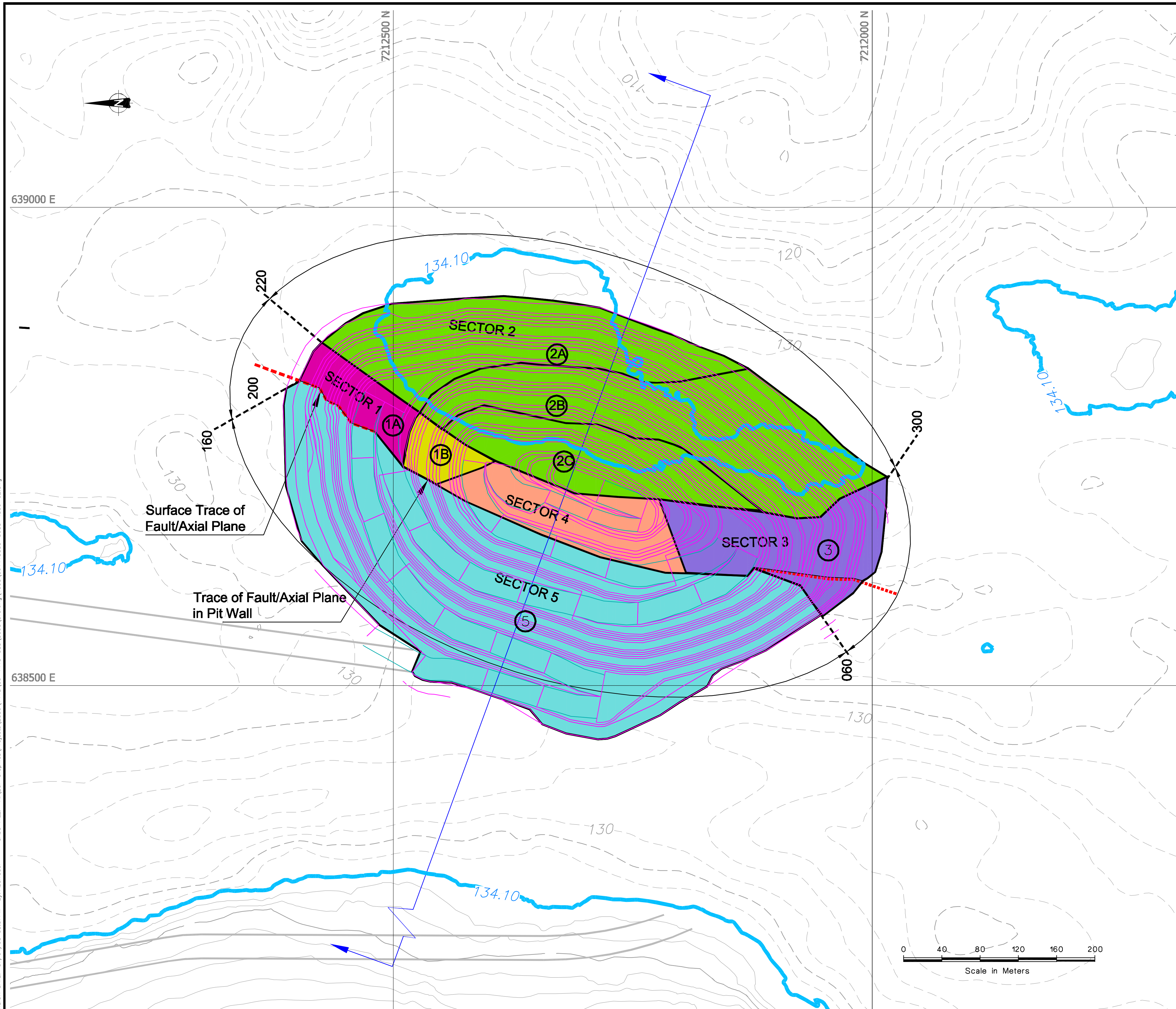
PROJECT **MEADOWBANK MINING CORPORATION**

TITLE **MODEL PARAMETERS**

Golder Associates

PROJECT No.	06-1413-089		FILE No.
DESIGN	CB	14FEB07	SCALE NTS REV.
CADD	GG	14FEB07	
CHECK			**FIGURE 7.7**
REVIEW			

DRAWING DATE: 17-Mar-07 COREL FILE: N:\Bur-Graphics\Projects\2006\1413\06-1413-089\5000\Drafting\4000\Figure 7.5-7.7.cdr



Sector 1 - Wall Azimuth 160° to 220°		
Slope Component	1	
	UM	IV/IF
Vertical Bench Separation	24	24
Bench Face Angle	60	65
Catch Bench Width	10	8
Inter-Ramp Angle	45	51

Design Sector 2 - Wall Azimuth 220° to 300°			
Slope Component	2A	2B	2C
	IV/IF	IV/IF	IV/IF
Vertical Bench Separation	24	24	24
Bench Face Angle	55	65	55
Catch Bench Width	8	8	8
Inter-Ramp Angle	44	51	44

Design Sector 3 - Wall Azimuth 300° to 060°		
Slope Component	3	
	UM	IV/IF
Vertical Bench Separation	24	24
Bench Face Angle	60	65
Catch Bench Width	10	8
Inter-Ramp Angle	45	51

Design Sector 5 - Wall Azimuth 020° to 200°	
Slope Component	Sector 5
	All Rock Types
Vertical Bench Separation	24
Bench Face Angle	70
Catch Bench Width	8
Inter-Ramp Angle	55

Design Sector 4 - Wall Azimuth 060° to 160°		
Slope Component	4	
	UM	IV/IF
Vertical Bench Separation	24	24
Bench Face Angle	60	65
Catch Bench Width	10	10
Inter-Ramp Angle	45	49

PROJECT

MEADOWBANK MINING CORPORATION

TITLE

MEADOWBANK GOLD PROJECT
GOOSE ISLAND PIT - DESIGN SECTORS

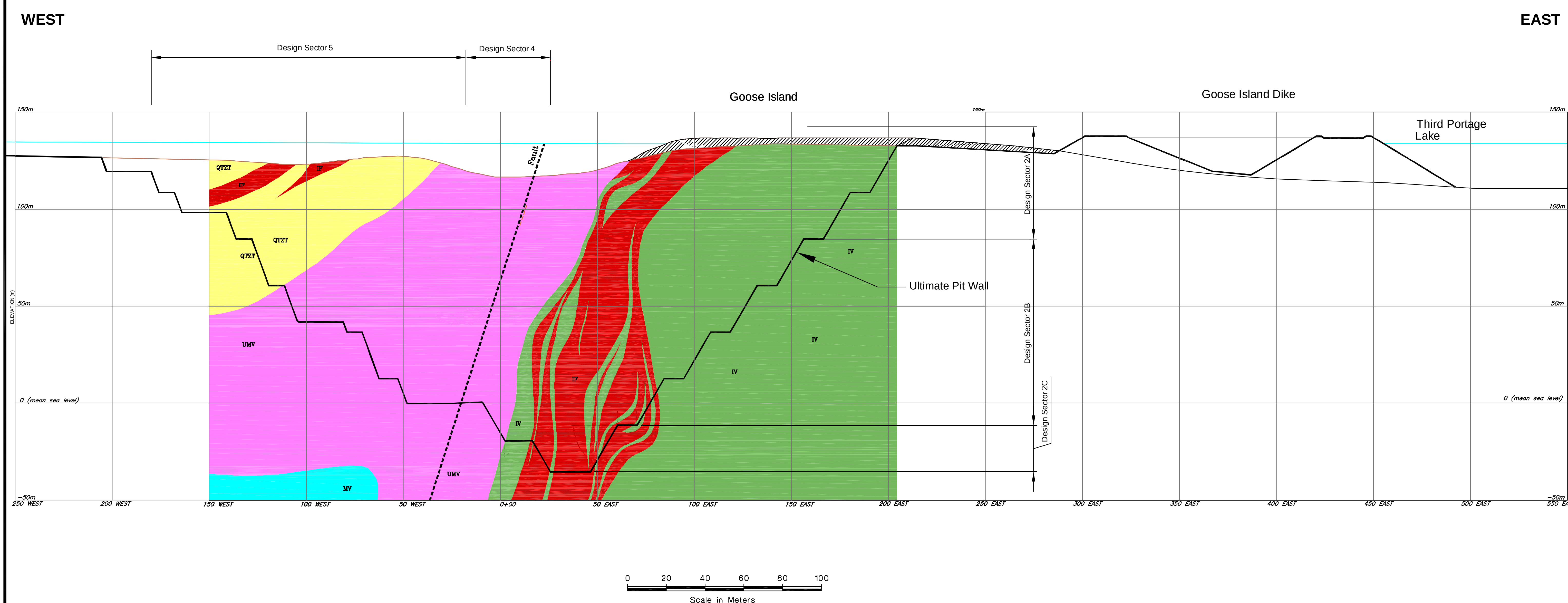
Golder Associates

PROJECT No.	06-1413-089	
DESIGN	ES	15FEB07
CADD	AS	15FEB07
CHECK	-	-
REVIEW	-	-

FILE No. 061413089-5000-4000-02
SCALE AS SHOWN REV. A

FIGURE 10.1

REVISION DATE: 07/04/05 03:07PM By: ASalvador CADD FILE: N:\Bur-Graphics\Projects\2006\1413\06-1413-089\5000\Drafting\4000\061413089-5000-4000-02.dwg



LEGEND

- Iron Formation
- Intermediate Volcanic Rock
- Ultramafic Rock
- Quartzite
- Mafic Volcanics
- Fault zone

NOTES

1. Base geology plan provided by Meadowbank Mining Corporation.
2. Goose Island Pit Slope Design Criteria, August, 2003.

PROJECT
MEADOWBANK MINING CORPORATION

TITLE
GOOSE ISLAND DEPOSIT PIT SLOPE CONFIGURATION CROSS SECTION 11+50S

Golder Associates

PROJECT No. 06-1413-089			FILE No.	5000-4000-18
DESIGN	CJC	09FEB06	SCALE AS SHOWN	REV.
CADD	AS	23MAR07		
CHECK			**FIGURE 10.2**	
REVIEW				

REVISION DATE: 07/04/05 07:34PM By: ASalvador CADD FILE: N:\Bur-Graphics\Projects\2006\1413\06-1413-089\5000\Drafting\4000\061413089-5000-4000-18.dwg

Sectors 1and 2 Slope	Applicable Range in Wall 225° to 315°	
Slope Component	Iron Fm., Volcanics	Ultramafics
Bench Height	24 m	24 m
Bench Face	70 deg	65 deg
Catch Bench	8 m	10 m
Inter-Ramp	55 deg	49 deg

Sectors 4 and 5 Slope Configurations	Applicable Range of Wall Sector Azimuth 045° to 135°	
Dip of Faulted Contacts	Slope Configuration	
<30° to 35°	Unbenched Footwall Slope	Parallel to Bedding/Stratigraphy/Sheared and Faulted Contacts
>30° to 35°	Bench Face Angle:	Parallel to Bedding/Stratigraphy/Sheared and Faulted Contacts to a maximum 70°
	Bench Height:	24 metres
	Catch Bench Width:	8 metres
	Inter-Ramp Angle:	30° to 55° dependent on bench face

Sector 3 Slope Configurations	Applicable Range in Wall Sector Azimuth 045° to 135°	
Dip of Faulted Contacts	Slope Configuration	
<30° to 35°	Unbenched Footwall Slope	Parallel to Bedding/Stratigraphy/Sheared and Faulted Contacts
>30° to 35°	Bench Face Angle:	Parallel to Bedding/Stratigraphy/Sheared and Faulted Contacts to a maximum 70°
	Bench Height:	24 metres
	Catch Bench Width:	8 metres
	Inter-Ramp Angle:	30° to 55° dependent on bench face

Sector 6 Slope Configurations	Applicable Range of Wall Sector Azimuths 135° to 185°	
Slope Component	Iron Fm., Volcanics	Ultramafics
Bench Height	24 m	24 m
Bench Face	65 deg	65 deg
Catch Bench	8 m	10 m
Inter-Ramp	51 deg	49 deg

Sector 7 Slope Configurations	Applicable Range of Wall Sector Azimuths 185° to 225°	
Slope Component	Iron Fm., Volcanics	Ultramafics and Bay Fault and Splay
Bench Height	24 m	24 m
Bench Face	65 deg	60 deg
Catch Bench	10 m	10 m
Inter-Ramp	49 deg	45 deg

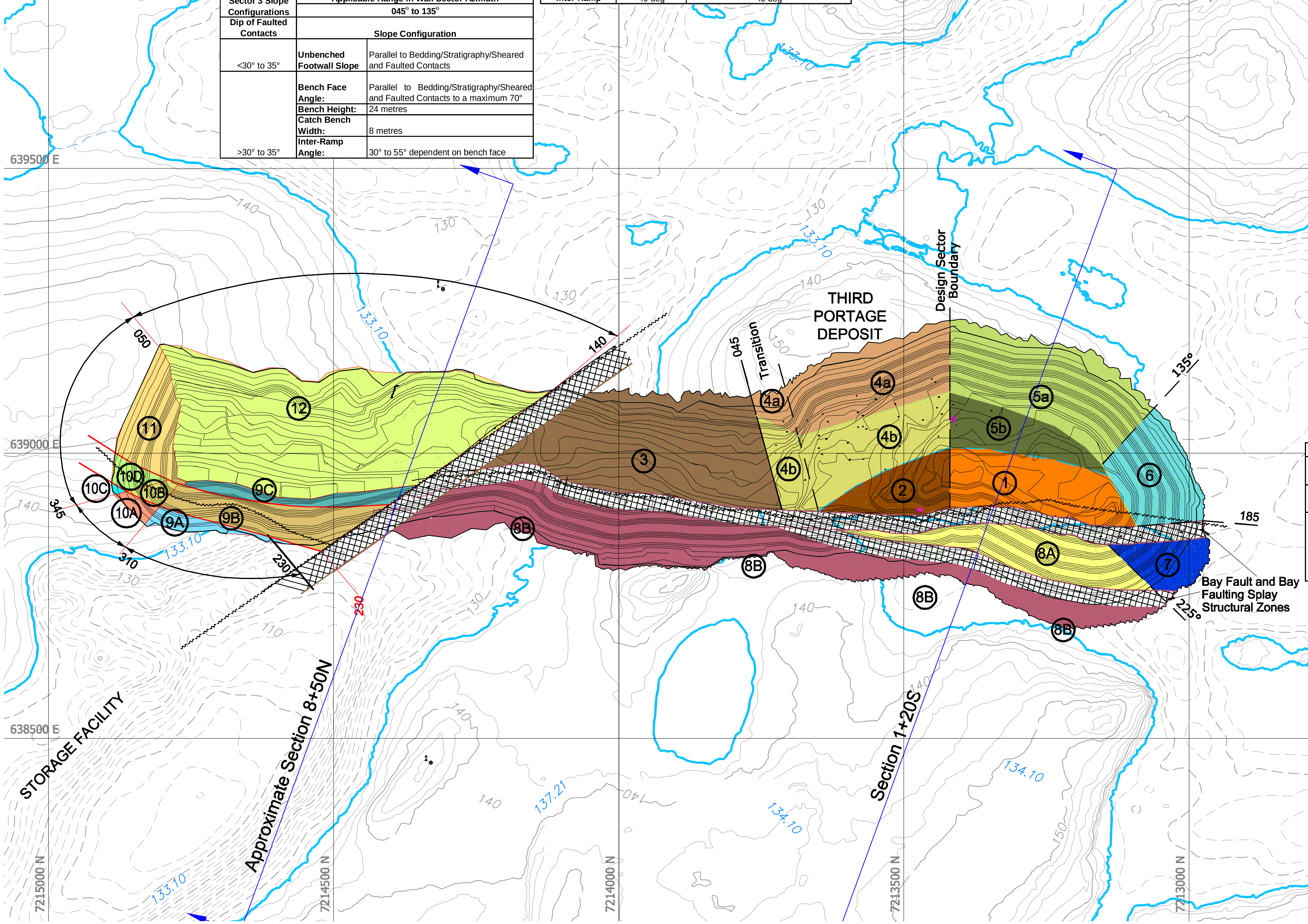
Sector 8 Slope Configurations	Applicable Range of Wall Sector Azimuths 225° to 315°				
	Sector 8A			Sector 8B	
	Middle to Lower Slopes			Upper Slopes	
Slope Component	Iron Fm., Volcanics	Ultramafics	Bay Fault Zone and Splay	Iron Fm., Volcanics	Ultramafics
Bench Height	24 m	24 m	24 m	24 m	24 m
Bench Face	70 deg	65 deg	60 deg	70 deg	70 deg
Catch Bench	10 m	10 m	10 m	8 m	10 m
Inter-Ramp	49 deg	49 deg	45 deg	55 deg	52 deg

West Wall Slope Configurations	Wall Sector				
	9A	9B		9C	
Slope Component	Upper West Wall	Middle West Wall		Lower West Wall	
	240° to 310°	240° to 310°		240° to 290°	
		Sound Rock	Bay Fault Zone	IF, IV, QTZ	UM
Bench Height	24 m	24 m	12 m	24 m	24 m
Bench Face	70 deg	70 deg	65 deg	70 deg	65 deg
Catch Bench	10 m	10 m	5m/8m	10 m	10 m
Inter-Ramp	52 deg	52 deg	45 deg	52 deg	49 deg

Middle to Upper North Wall Slope Configurations	Wall Sector 310° to 050°			
	10A	10B	10C	10D
Slope Component	Upper Northwest Wall	Middle Northwest Wall (Bay Fault Intersection)	Upper North Wall	Mid to Upper North and Northeast Wall
	310° to 345°	310° to 345°	345° to 050°	345° to 050°
Bench Height	24 m	12 m	24 metres	24 metres
Bench Face	70 deg	65 deg	70 degrees	70 degrees
Catch Bench	10 m	5 m/8 m	12 metres	12 metres
Inter-Ramp	52 deg	45 deg	49 degrees	49 degrees

Lower North Wall Slope Configurations	Wall Sector 11 290° to 050°
Slope Component	Northeast to Northwest Wall
Bench Height	24 m
Bench Face	70 deg
Catch Bench	10 m
Inter-Ramp	52 deg

Wall Sector 12 050° to 140°		
Dip of Faulted Contacts	Slope Configuration	
<30°	Unbenched Slope	Parallel to Bedding/Stratigraphy/Faulted Contacts
>30°	Bench Face Angle:	Parallel to Bedding/Stratigraphy/Faulted Contacts to a maximum 70°
	Bench Height:	24 metres
	Catch Bench Width:	10 metres
	Inter-Ramp Angle:	32° to 52° dependent on bench face



MEADOWBANK
MINING CORPORATION

PROJECT

TITLE

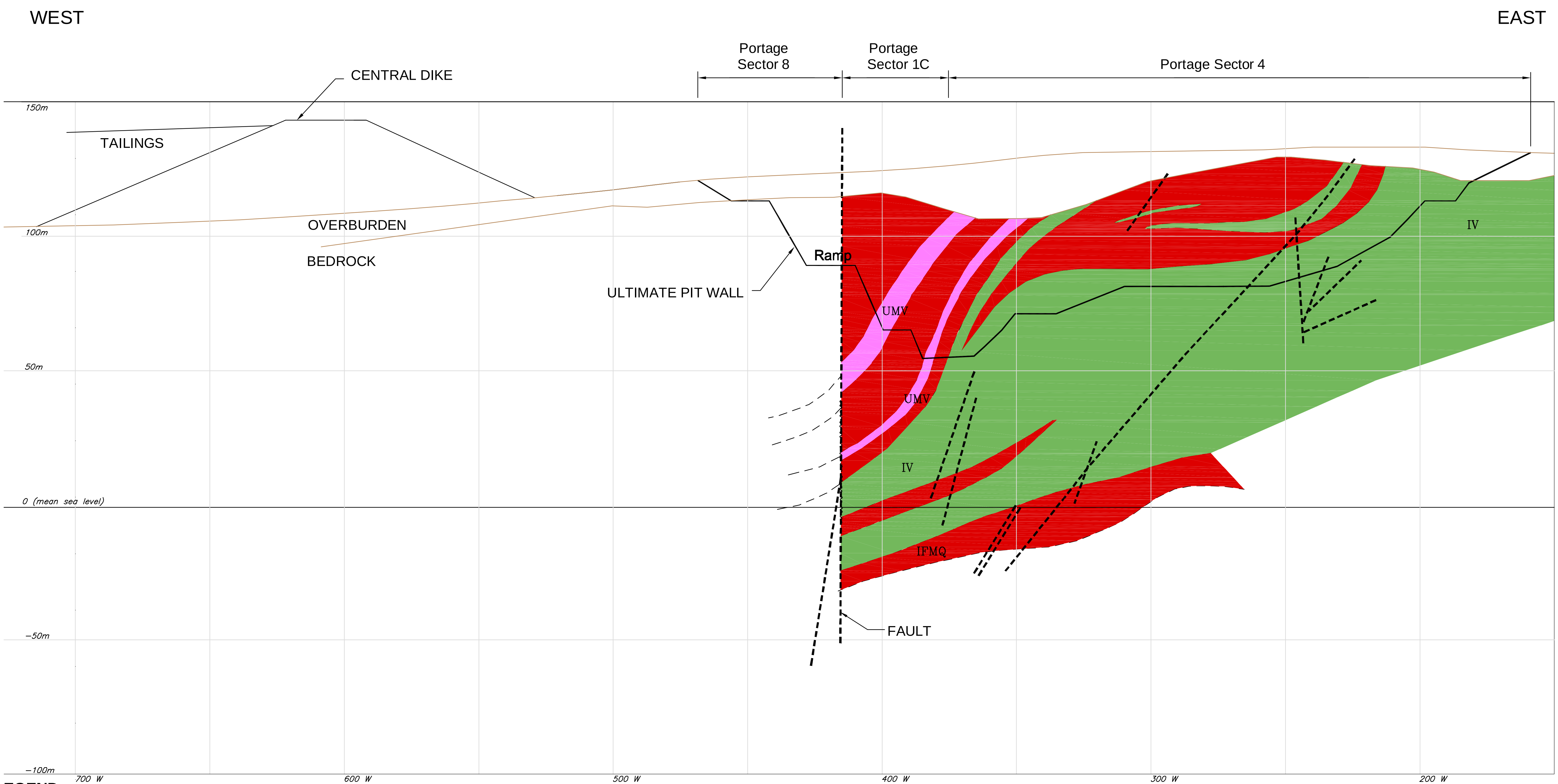
PORTAGE PIT - DESIGN SECTORS

Golder Associates

PROJECT No.	06-1413-089		FILE No. 061413089-5000-4000-01.1		
DESIGN	ES	15FEB07	SCALE	AS SHOWN	REV. A
CADD	AS	15FEB07			
CHECK	-	-			
REVIEW	-	-			

FIGURE 10.3

REVISION DATE: 07/04/05 07:08PM By: ASalvador CADD FILE: N:\Bur-Graphics\Projects\2006\1413\06-1413-089\5000\Drafting\4000\061413089-5000-4000-01.1.dwg



LEGEND

- Iron Formation
- Intermediate Volcanic Rock
- Ultramafic Rock
- Fault zone

NOTES

1. Base geology plan provided by Meadowbank Mining Corporation.
2. Goose Island Pit Slope Design Criteria, August, 2003.

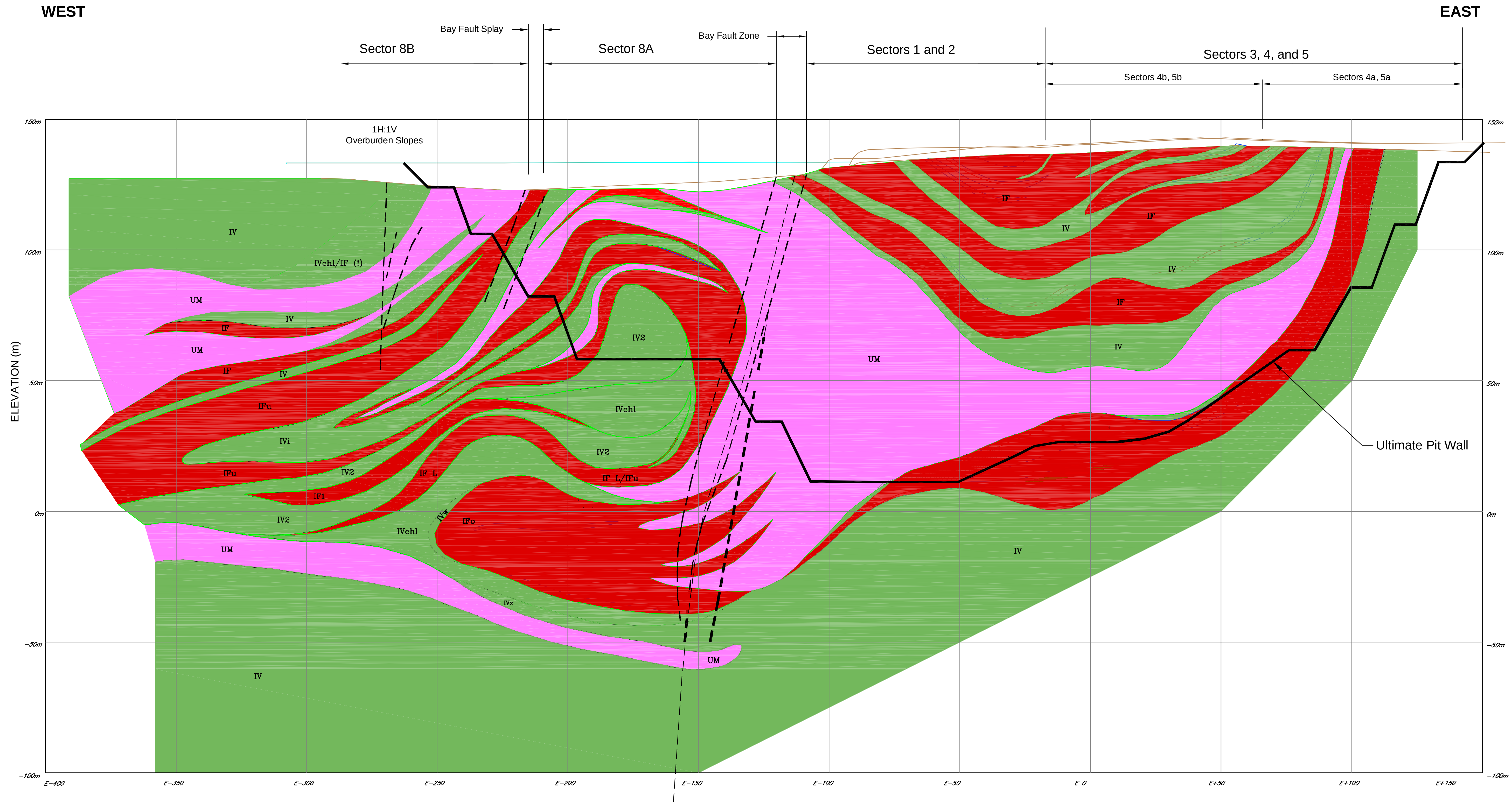
PROJECT
MEADOWBANK
MINING CORPORATION

TITLE
PORTAGE DEPOSIT
PIT DESIGN SECTORS
CROSS SECTION 8+50N (MINE GRID)

Golder Associates

PROJECT No. 06-1413-089			FILE No. 5000-4000-17	
DESIGN	JFG	23MAR07	SCALE AS SHOWN	REV. -
CADD	AS	23MAR07	**FIGURE 10.4**	
CHECK				
REVIEW				

CADD FILE: N:\Bur-Graphics\Projects\2006\1413\06-1413-089\5000\Drafting\4000\061413089-5000-4000-17.dwg
REVISION DATE: 07/04/05 03:39PM By: ASalvador



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REVISION DATE: 07/04/05 04:49PM By: ASalvador

LEGEND

- Iron Formation
- Intermediate Volcanic Rock
- Ultramafic Rock
- Fault zone

REFERENCES

1. Base geology drawing (MB_Surface.dwg) provided by Cumberland Resources Ltd.

2. Base fault drawing (3rdPortageSurfaceFaultTraces14.dwg) provided by Cumberland Resources Ltd.

0 10 20 30 40 50
Scale in Meters

PROJECT

MEADOWBANK
MINING CORPORATION

TITLE

PORTAGE DEPOSITS
PIT DESIGN SECTORS
CROSS SECTION 1+20S (MINE GRID)

Golder Associates

PROJECT No.		06-1413-089	FILE No.	FIG_10.5	
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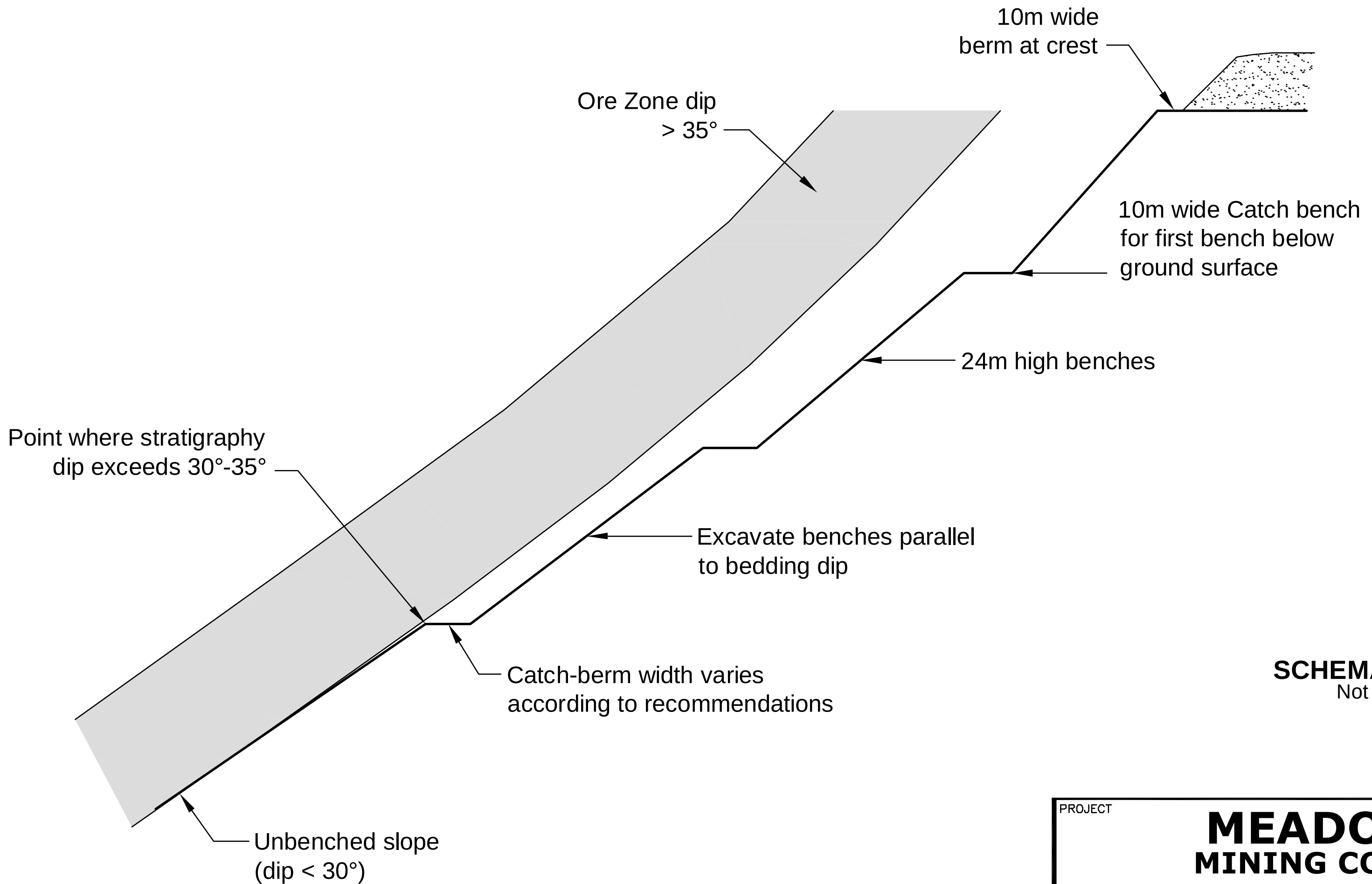
CADD	AS	03APR07			
CHECK					
REVIEW					

FIGURE 10.5



SCHEMATIC ONLY
Not to Scale

PROJECT

MEADOWBANK
MINING CORPORATION

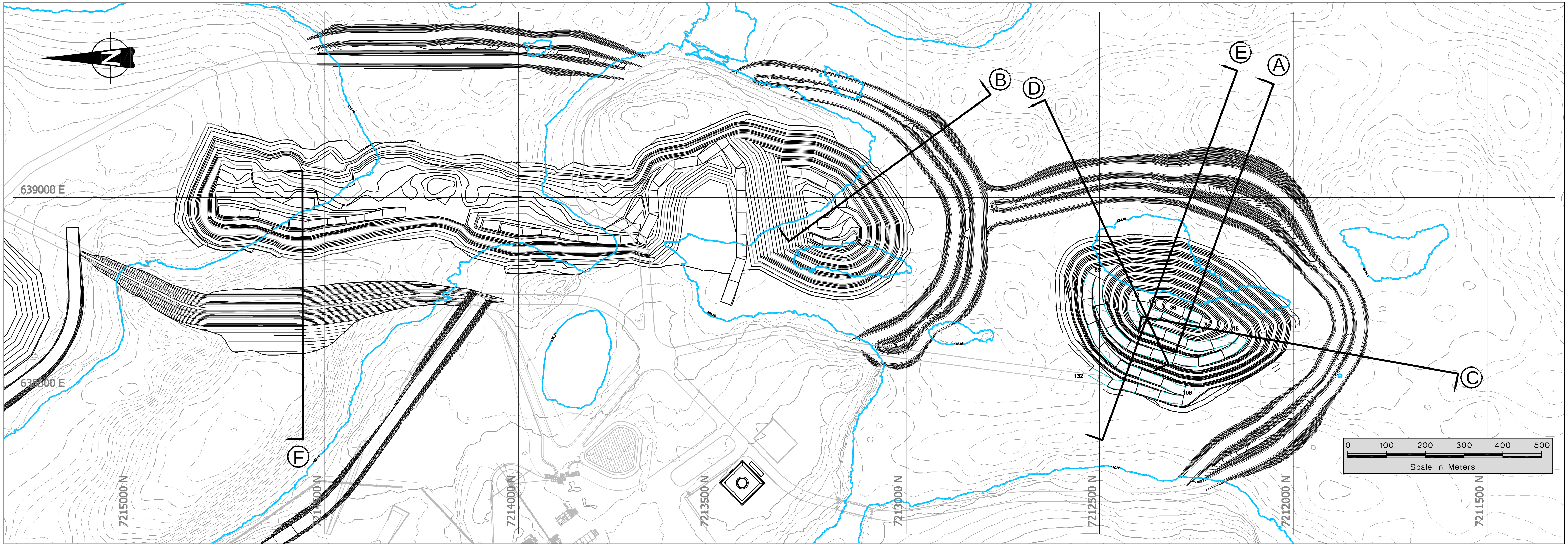
TITLE

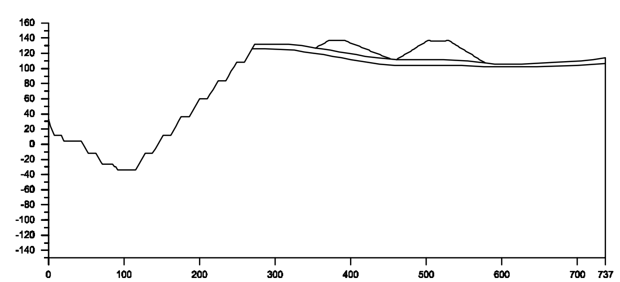
GENERAL FOOTWALL DESIGN CRITERIA

Golder Associates

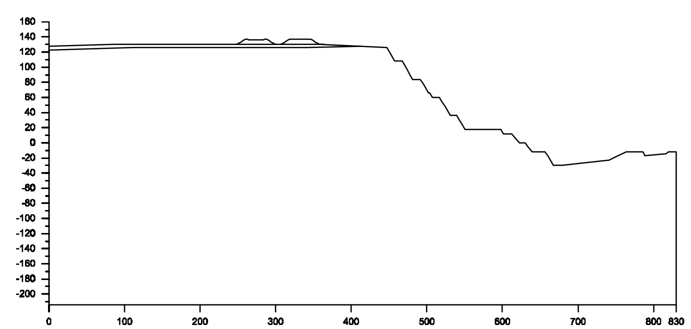
PROJECT No. 06-1413-089			FILE No.
DESIGN	CJC	15MAR07	SCALE NTS REV. 0
CADD	NV	15MAR07	
CHECK			
REVIEW			

FIGURE 10.6

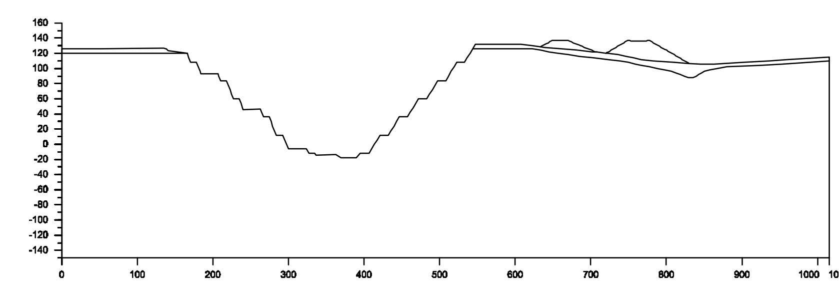




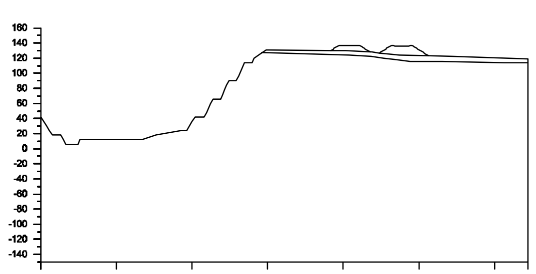
A - GOOSE SOUTHEAST - 12+00 S



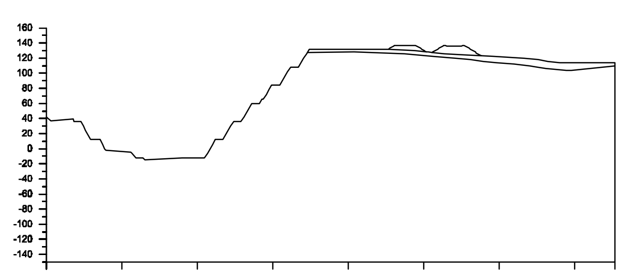
C - GOOSE SOUTH



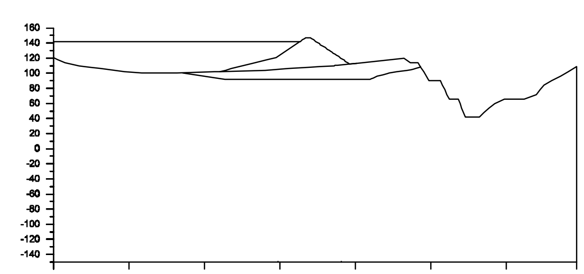
E - GOOSE SOUTHEAST - 11+00 S



B - PORTAGE SOUTHEAST



D - GOOSE NORTHEAST



F - PORTAGE/TAILINGS NORTHWEST

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Scale in Meters

PROJECT

MEADOWBANK
MINING CORPORATION

TITLE

SLOPE STABILITY ASSESSMENT ANALYSIS SECTIONS

Golder Associates

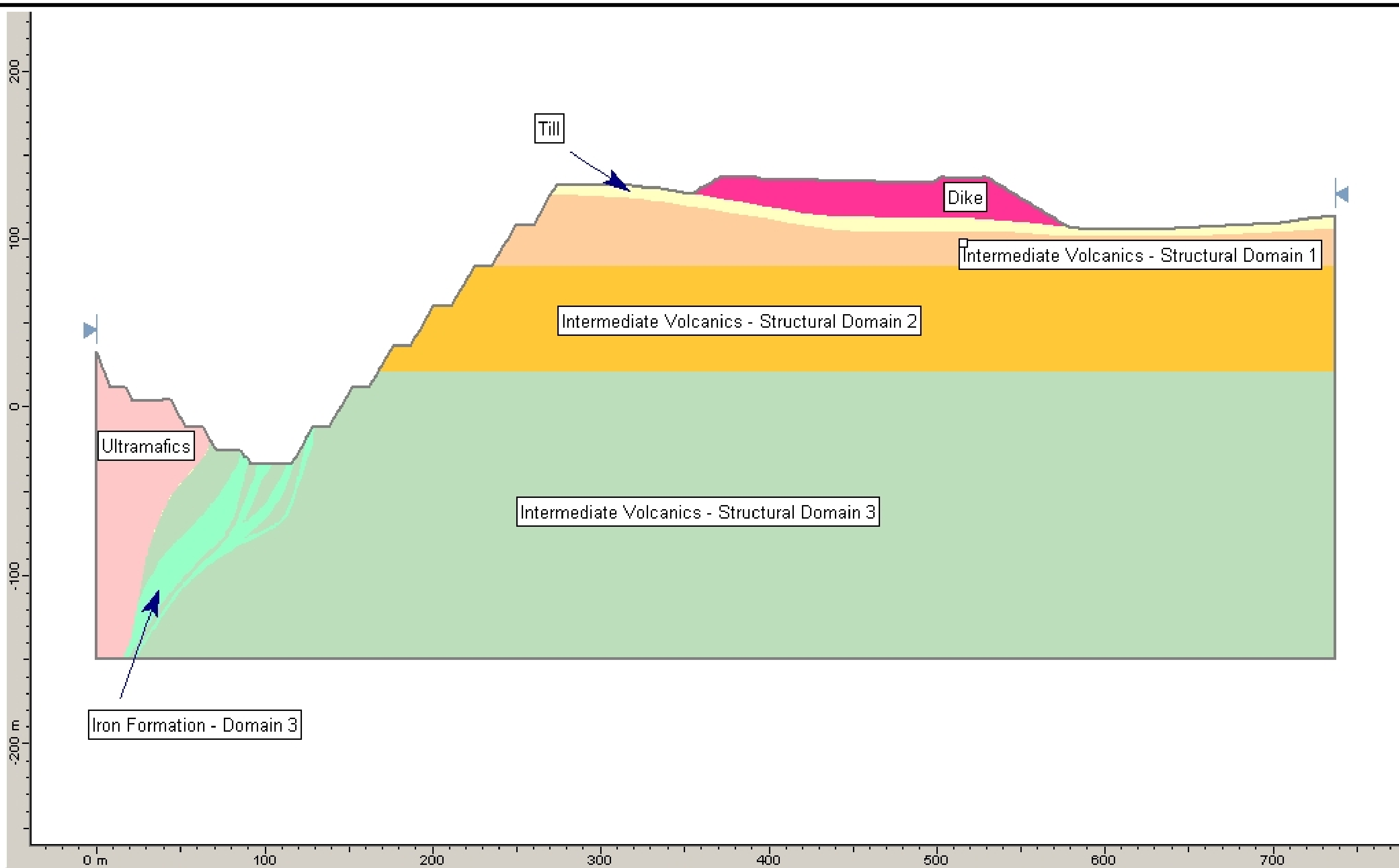
PROJECT No.	06-1413-089	FILE No.	5000-4000-04
DESIGN	ES	20FEB07	SCALE AS SHOWN	REV.
CADD	AS	20FEB07	
CHECK			
REVIEW			

FIGURE 11.1

CADD FILE: N:\Bur-Graphics\Projects\2006\1413\06-1413-089\5000\Drafting\4000\061413089-5000-4000-04.dwg

REVISION DATE: 07/04/02 02:37PM By: ASalvador

REVISION DATE: 14MAR07 BY: FILE:N:\Bur-Graphics\Projects\2006\1413\06-1413-089\5000\Drafting\4000\Appendix 11.ppt



PROJECT

MEADOWBANK
MINING CORPORATION

TITLE

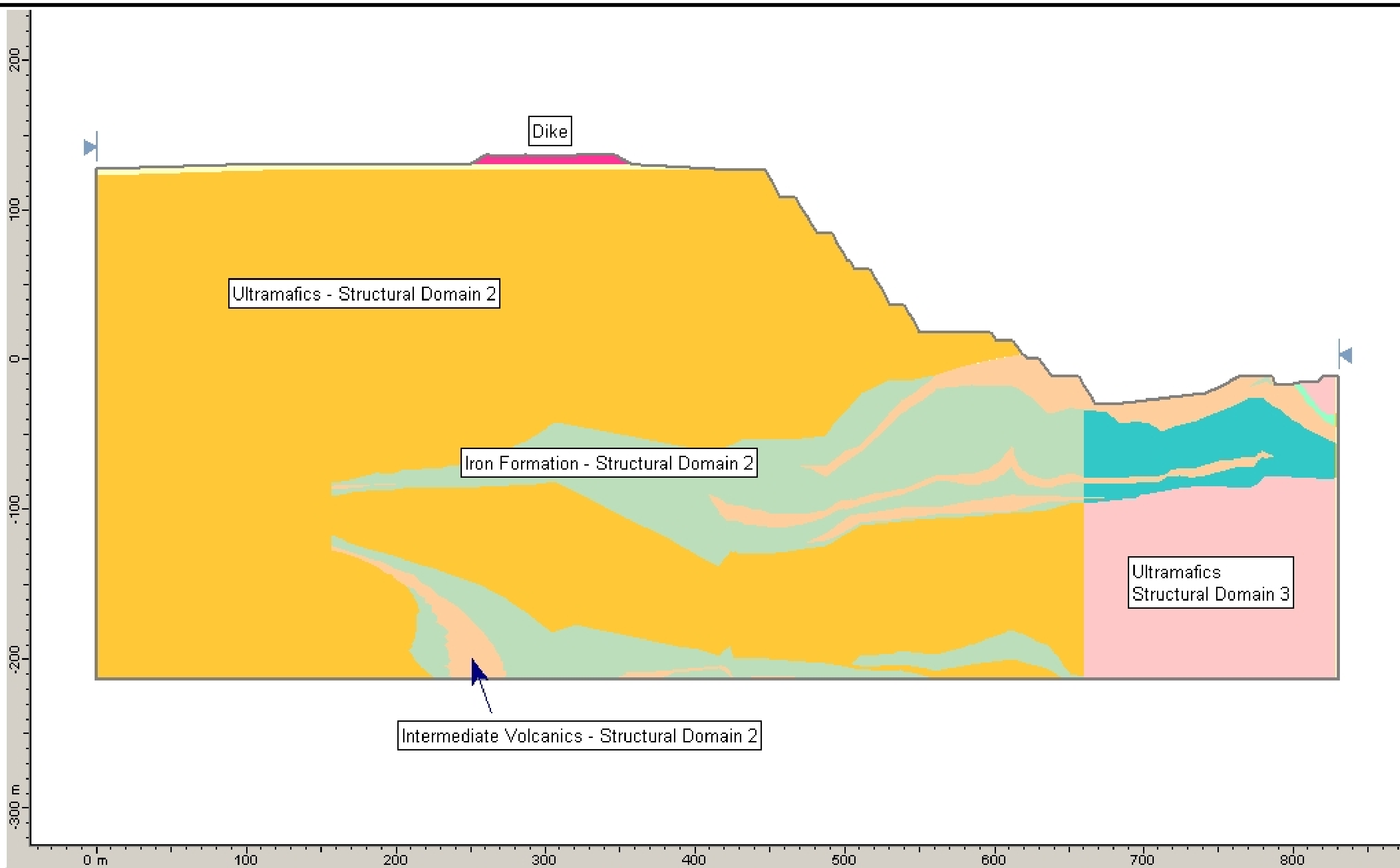
ANALYSIS SECTION
GOOSE SOUTHEAST – 12+00S

Golder Associates

PROJECT No.	06-1413-089		FILE No.	----
DESIGN	EMS	13MAR07	SCALE NTS	REV.
CADD	JFG	13MAR07		
CHECK	--	01JAN04		
REVIEW				

FIGURE 11.2

REVISION DATE: 14MAR07 BY: FILE:N:\Bur-Graphics\Projects\2006\1413\06-1413-089\5000\Drafting\4000\Appendix 11.ppt

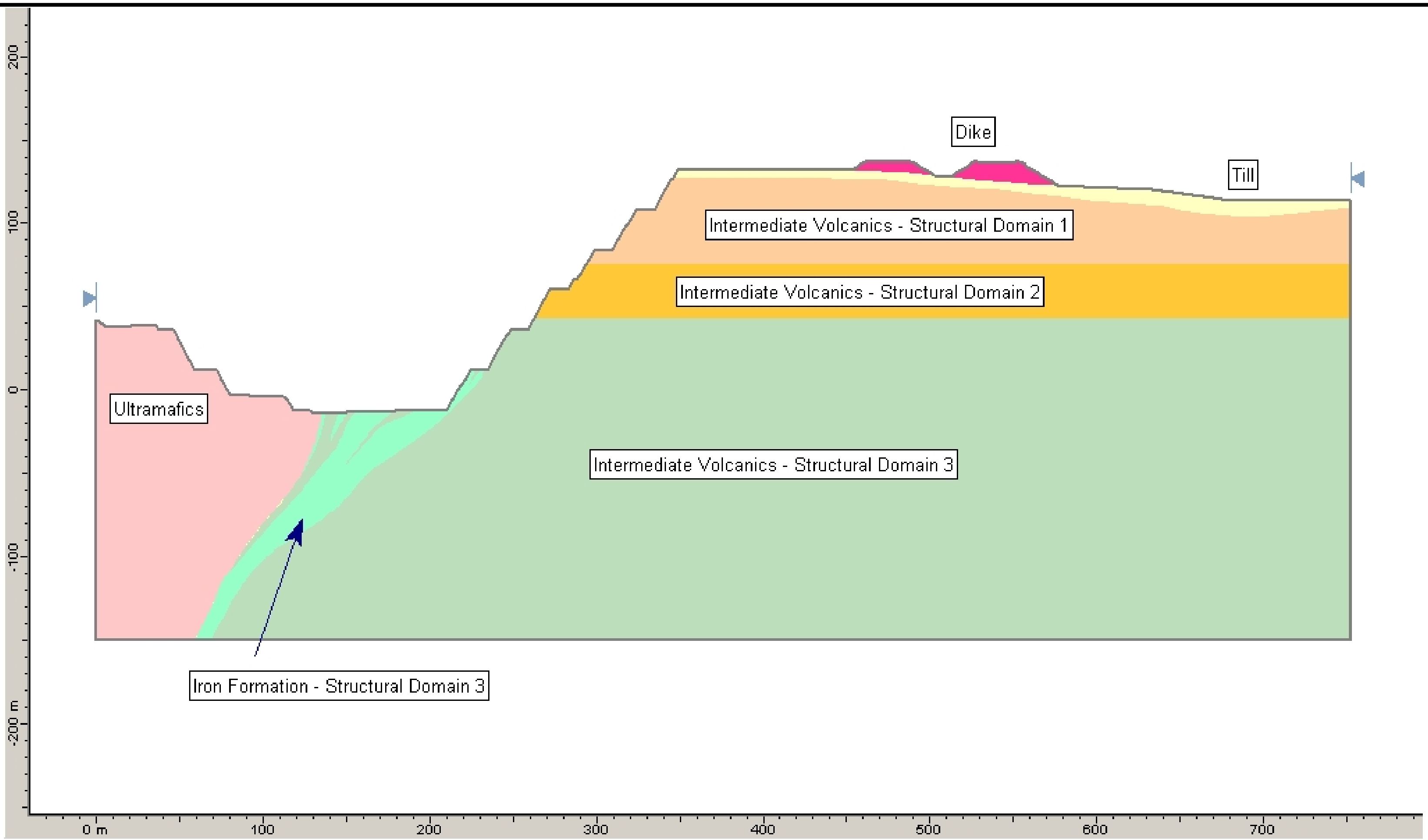


PROJECT	**MEADOWBANK** MINING CORPORATION
TITLE	**ANALYSIS SECTION GOOSE - SOUTH**

PROJECT No.	06-1413-089		FILE No. ----
DESIGN	EMS	13MAR07	SCALE NTS REV.
CADD	JFG	13MAR07	**FIGURE 11.3**
CHECK	--	01JAN04	
REVIEW			

Golder Associates

REVISION DATE: 14MAR07 BY: FILE:N:\Bur-Graphics\Projects\2006\1413\06-1413-089\5000\Drafting\4000\Appendix 11.ppt



PROJECT

MEADOWBANK
MINING CORPORATION

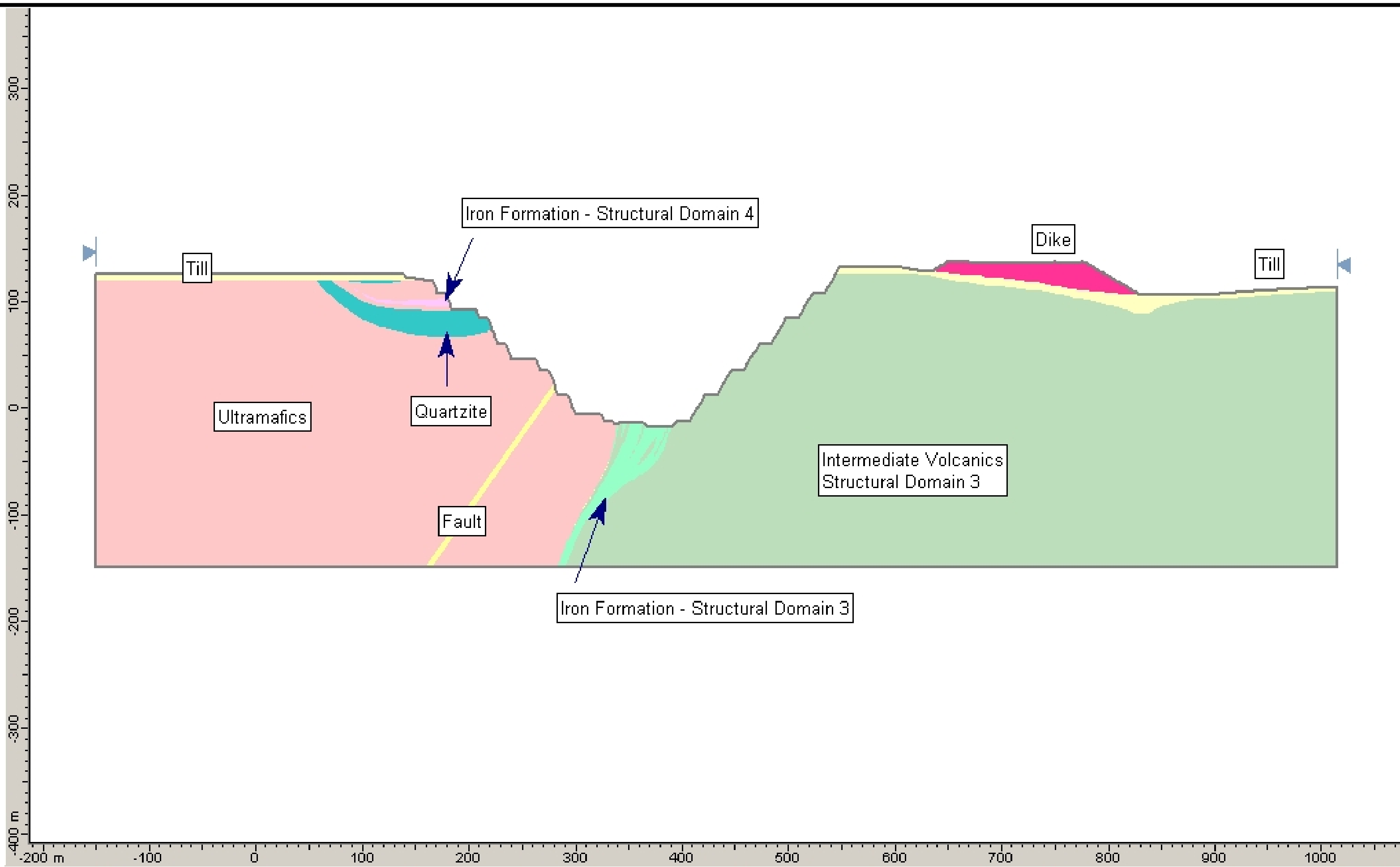
TITLE

ANALYSIS SECTION
GOOSE - NORTHEAST

Golder Associates

PROJECT No.	06-1413-089		FILE No.	----	
DESIGN	EMS	13MAR07	SCALE	NTS	REV.
CADD	JFG	13MAR07			
CHECK	--	01JAN04	**FIGURE 11.4**		
REVIEW					

REVISION DATE: 14MAR07 BY: FILE:N:\Bur-Graphics\Projects\2006\1413\06-1413-089\5000\Drafting\4000\Appendix 11.ppt



PROJECT

MEADOWBANK
MINING CORPORATION

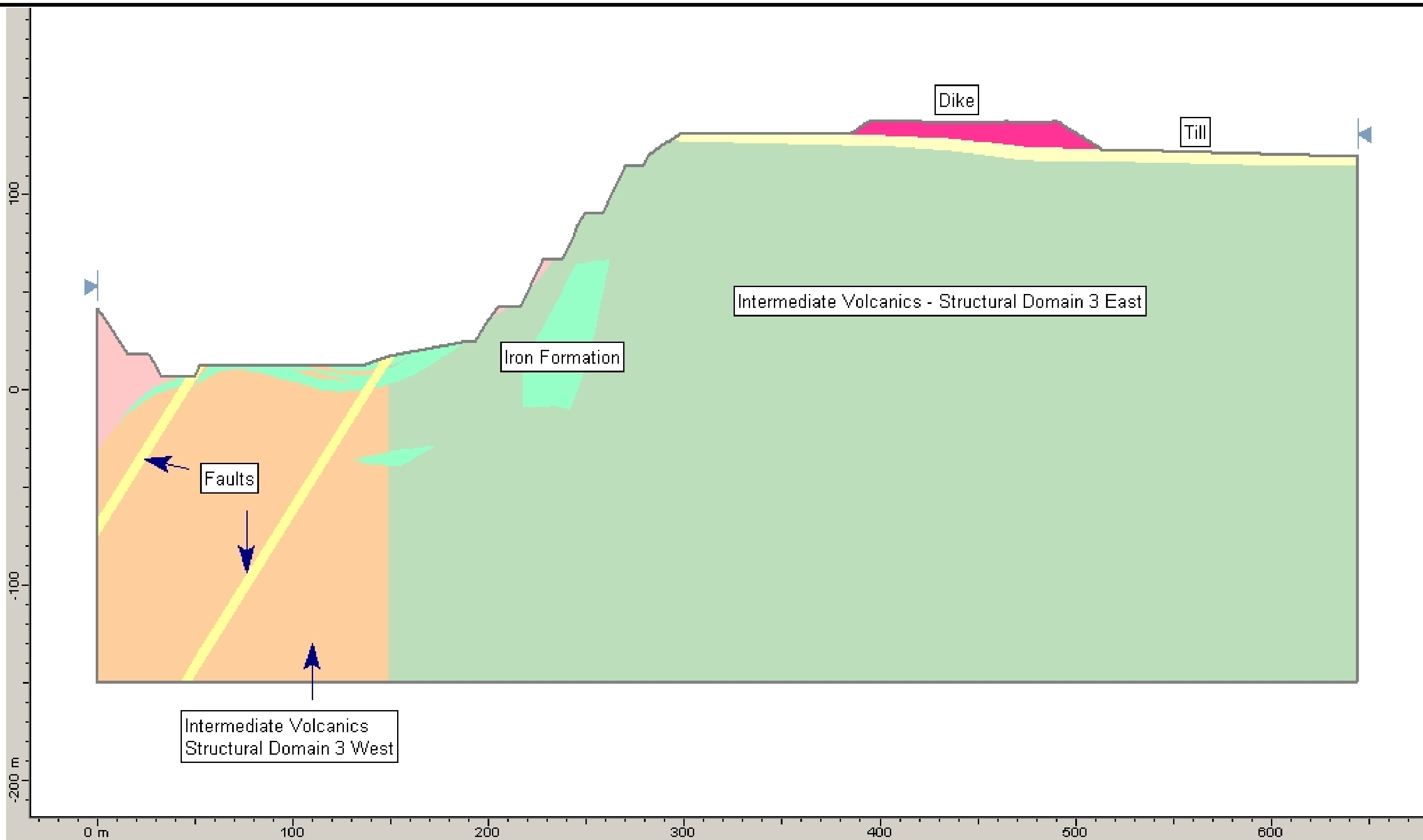
TITLE

ANALYSIS SECTION
GOOSE WEST - 11+00S

Golder Associates

PROJECT No.	06-1413-089		FILE No. ----	
DESIGN	EMS	13MAR07	SCALE NTS	REV.
CADD	JFG	13MAR07	**FIGURE 11.5**	
CHECK	--	01JAN04		
REVIEW				

REVISION DATE: 14MAR07 BY: FILE:N:\Bur-Graphics\Projects\2006\1413\06-1413-089\5000\Drafting\4000\Appendix 11.ppt



PROJECT

MEADOWBANK
MINING CORPORATION

TITLE

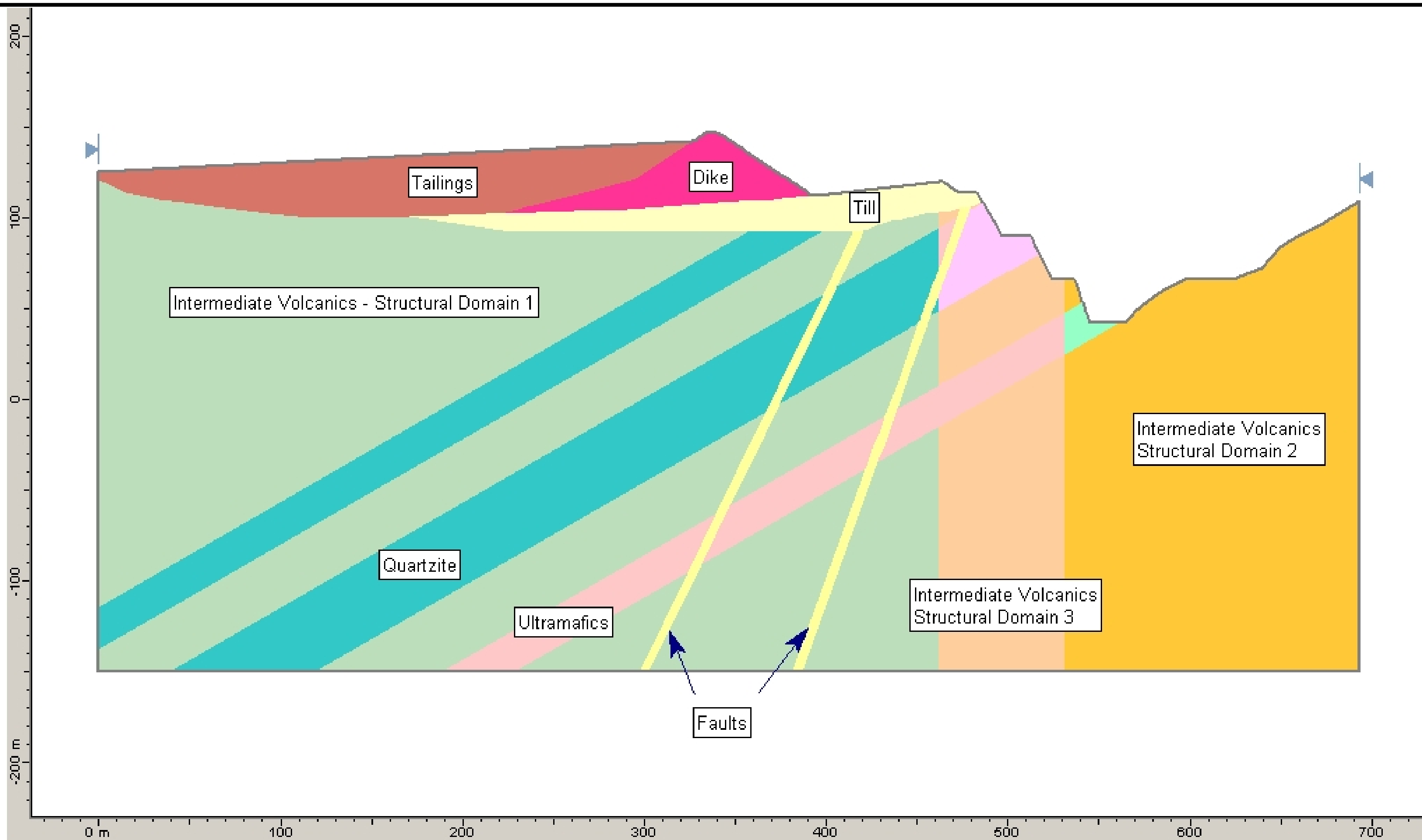
ANALYSIS SECTION
PORTAGE - SOUTHEAST

Golder Associates

PROJECT No.		06-1413-089	FILE No.	----	
DESIGN	EMS	13MAR07	SCALE	NTS	REV.
CADD	JFG	13MAR07			
CHECK	--	01JAN04			
REVIEW					

FIGURE 11.6

REVISION DATE: 14MAR07 BY: FILE:N:\Bur-Graphics\Projects\2006\1413\06-1413-089\5000\Drafting\4000\Appendix 11.ppt



PROJECT

MEADOWBANK
MINING CORPORATION

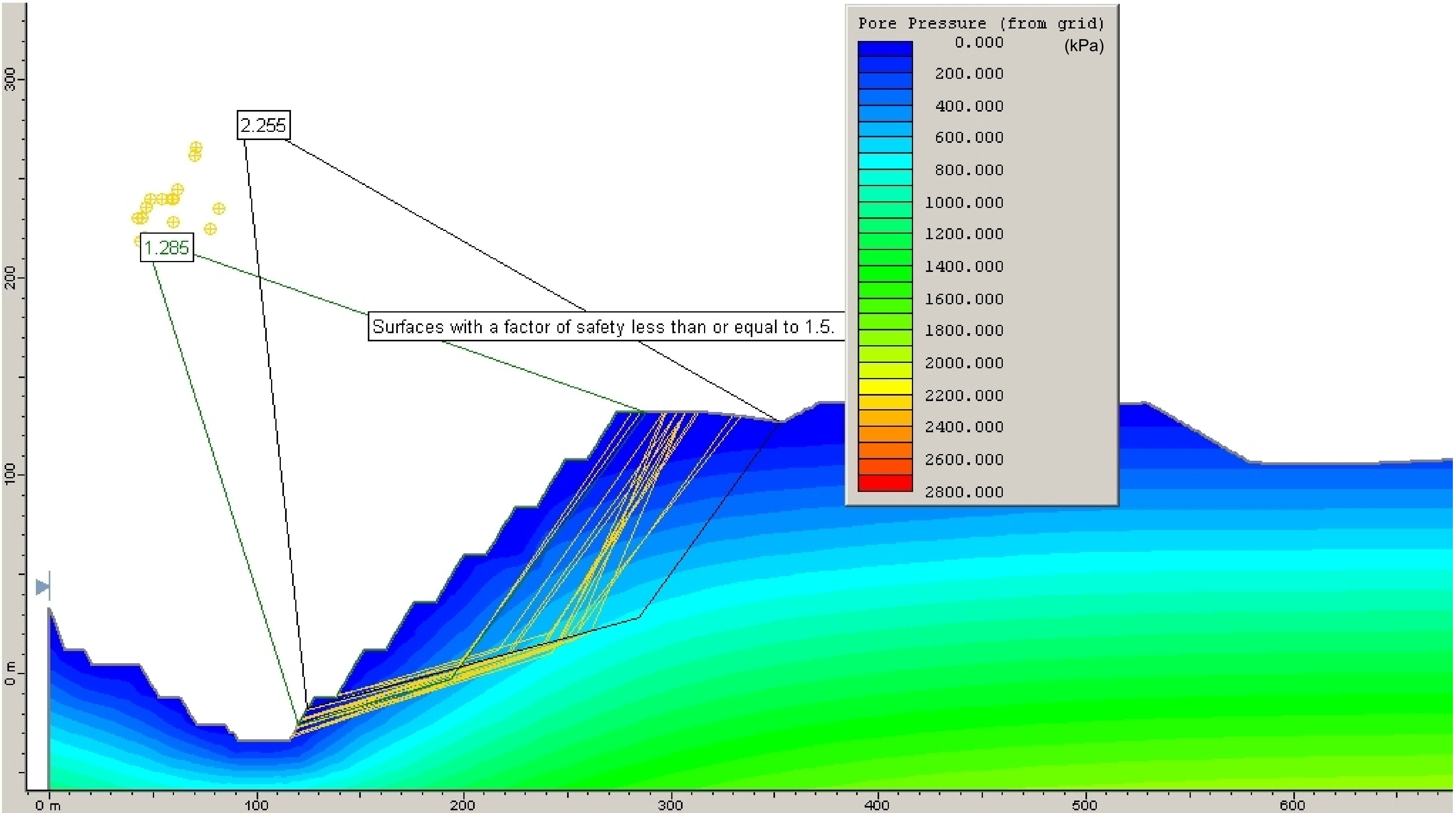
TITLE

ANALYSIS SECTION
PORTAGE/TAILINGS - NORTHWEST

Golder Associates

PROJECT No.	06-1413-089		FILE No. ----	
DESIGN	EMS	13MAR07	SCALE NTS	REV.
CADD	JFG	13MAR07	**FIGURE 11.7**	
CHECK	--	01JAN04		
REVIEW				

DRAWING DATE: 06-Mar-07 COREL FILE: N:\Bur-Graphics\Projects\2006\1413\06-1413-089\5000\Drafting\4000\Pit slopes report\FIGURES.cdr



PROJECT	**MEADOWBANK MINING CORPORATION**				
TITLE	**SLIDE ANALYSIS GOOSE SOUTHEAST - 12+00S NO DEPRESSURIZATION 20% ROCK BRIDGE**				
Golder Associates	PROJECT No.	06-1413-089		FILE No. FIGURES	
	DESIGN	JG	06MAR07	SCALE NTS	REV.
	CADD	AS	06MAR07	**FIGURE 11.8**	
	CHECK	--	--		
	REVIEW				

REVISION DATE: 14MAR07 BY: FILE:N:\Bur-Graphics\Projects\2006\1413\06-1413-089\5000\Drafting\4000\Appendix 11_1.ppt

JOB TITLE : Goose 12+00SE

UDEC (Version 4.00)

LEGEND

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X displacement contours
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-1.000E+00
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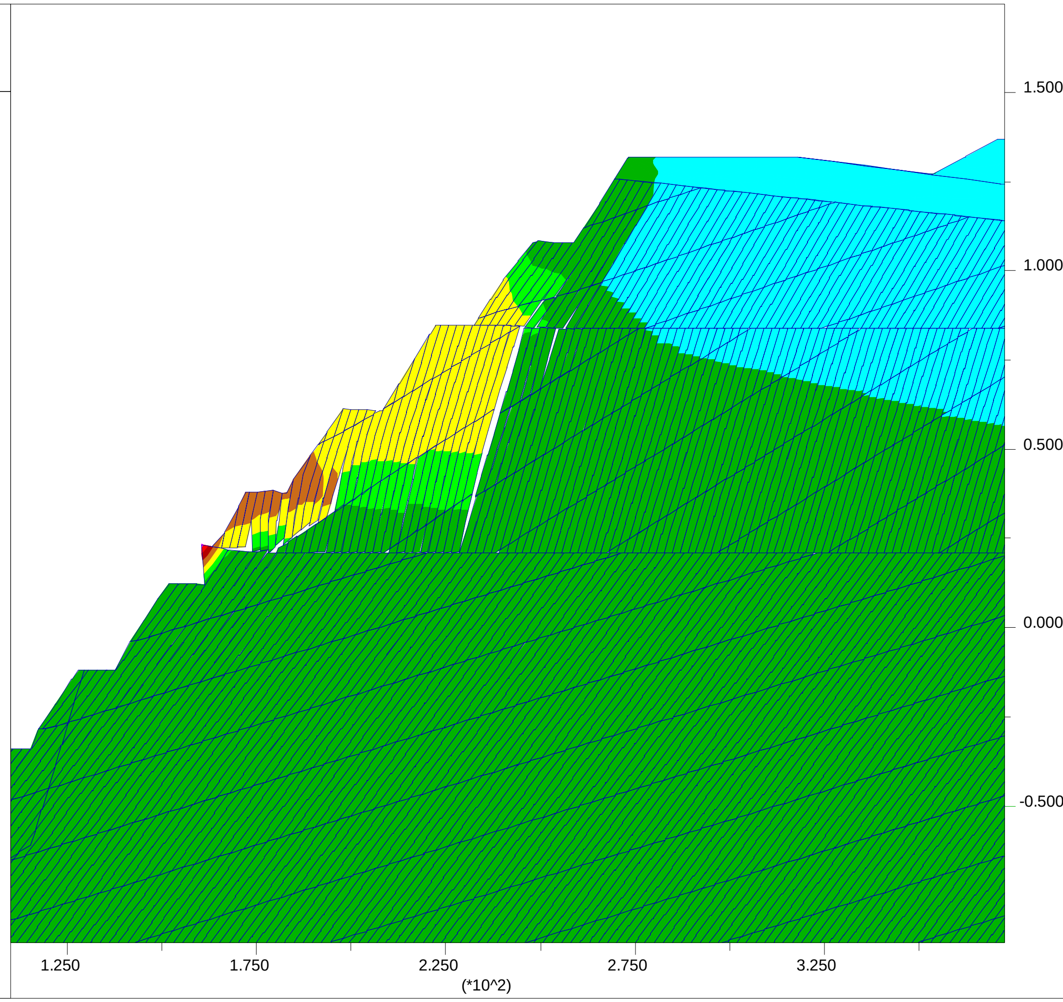
block plot



PROJECT

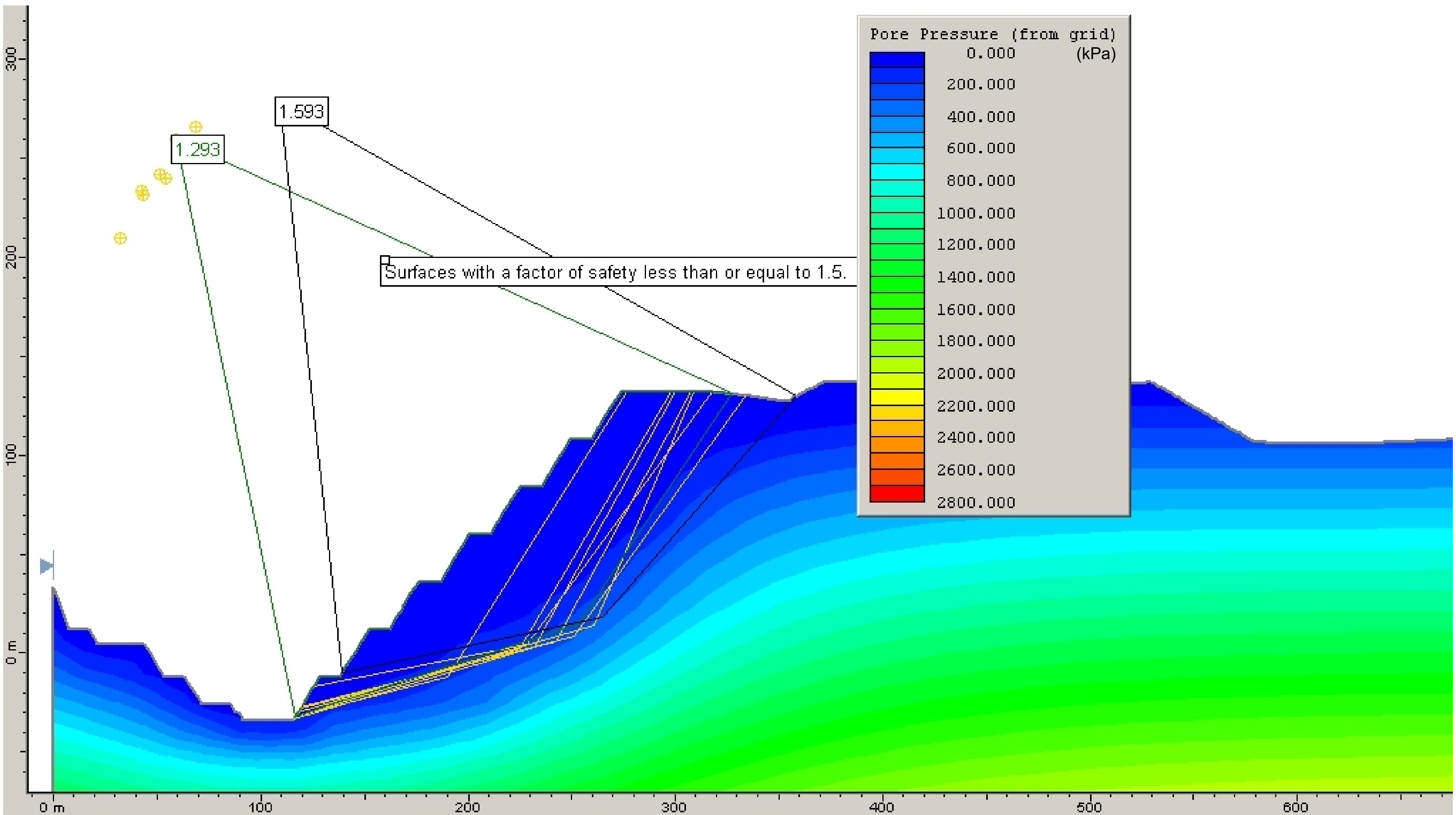
MEADOWBANK
MINING CORPORATION

TITLE **UDEC STABILITY ANALYSIS-DISPLACEMENT**
GOOSE SOUTHEAST-12+00S
20% ROCK BRIDGES ON JOINTS

Golder Associates

PROJECT No.	06-1413-089		FILE No. ----	
DESIGN	EMS	13MAR07	SCALE NTS	REV.
CADD	JFG	13MAR07	**FIGURE 11.9**	
CHECK	--			
REVIEW				

DRAWING DATE: 06-Mar-07 COREL FILE: N:\Bur-Graphics\Projects\2006\1413\06-1413-089\5000\Drafting\4000\Pit slopes report\FIGURES.cdr



PROJECT	MEADOWBANK MINING CORPORATION				
TITLE	SLIDE ANALYSIS GOOSE SOUTHEAST - 12+00S DEPRESSURIZED 0% ROCK BRIDGE				
Golder Associates	PROJECT No.	06-1413-089		FILE No. FIGURES	
	DESIGN	JG	06MAR07	SCALE NTS	REV.
	CADD	AS	06MAR07	FIGURE 11.10	
	CHECK	--	--		
	REVIEW				

REVISION DATE: 14MAR07 BY: FILE:N:\Bur-Graphics\Projects\2006\1413\06-1413-089\5000\Drafting\4000\Appendix 11_1.ppt

JOB TITLE : Goose 12+00SE

UDEC (Version 4.00)

LEGEND

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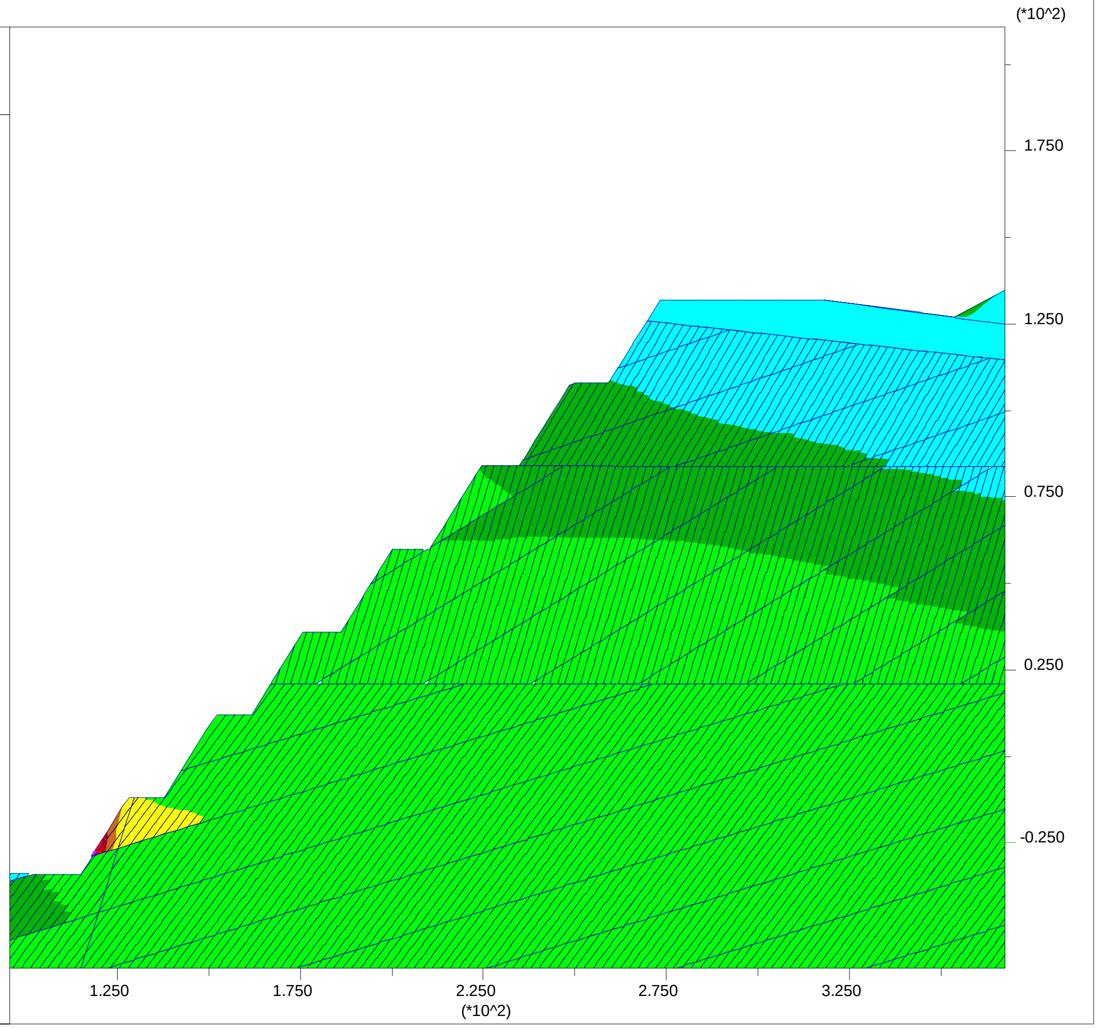
block plot



PROJECT

MEADOWBANK
MINING CORPORATION

TITLE
UDEC STABILITY ANALYSIS-DISPLACEMENT
GOOSE SOUTHEAST-12+00S
0% ROCK BRIDGES ON JOINTS
DEPRESSURIZED

Golder Associates

PROJECT No.	06-1413-089		FILE No. ----	
DESIGN	EMS	13MAR07	SCALE NTS	REV.
CADD	JFG	13MAR07	FIGURE 11.11	
CHECK	--			
REVIEW				

JOB TITLE : Goose 12+00SE

UDEC (Version 4.00)

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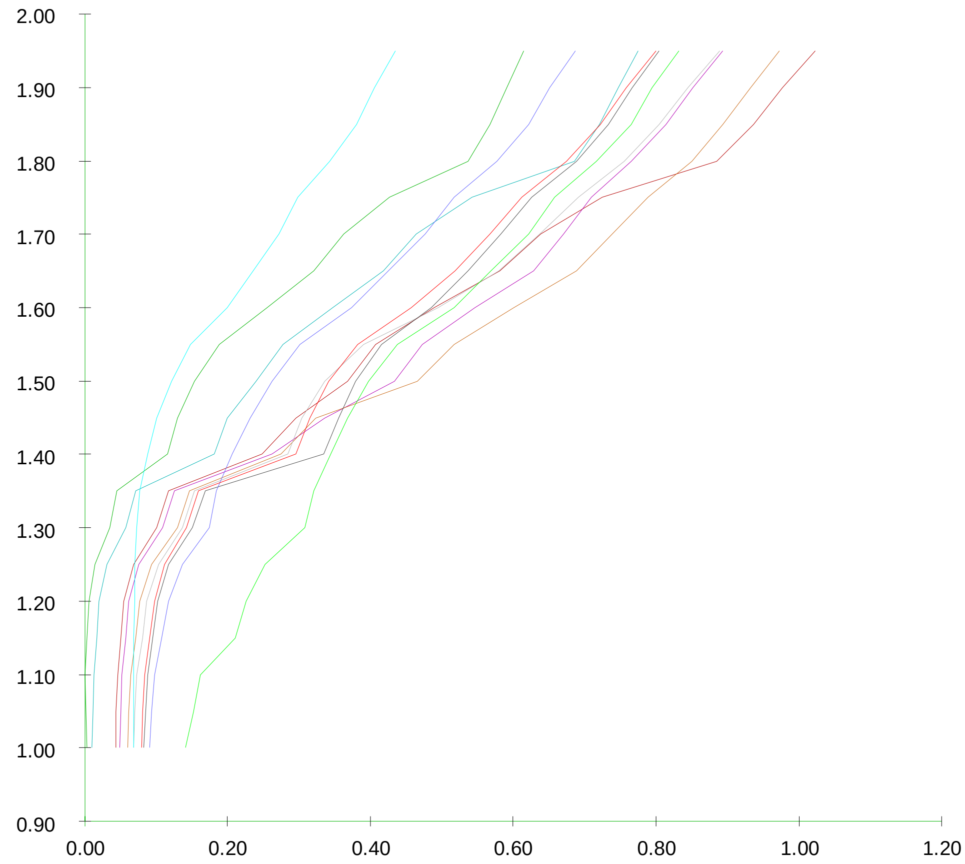

PROJECT	MEADOWBANK MINING CORPORATION			
TITLE	UDEC FOS ANALYSIS GOOSE SOUTHEAST-12+00S 0% ROCK BRIDGES ON JOINTS DEPRESSURIZED FOS vs. HORIZONTAL DISPLACEMENT OF THE PIT SLOPE			
Golder Associates	PROJECT No.	06-1413-089		FILE No. ----
	DESIGN	EMS	13MAR07	SCALE NTS REV.
	CADD	JFG	13MAR07	FIGURE 11.12
	CHECK	--		
	REVIEW			

JOB TITLE : Goose 12+00SE

UDEC (Version 4.00)

LEGEND

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X displacement contours
contour interval= 1.000E-01
-6.000E-01 to 0.000E+00

-6.000E-01
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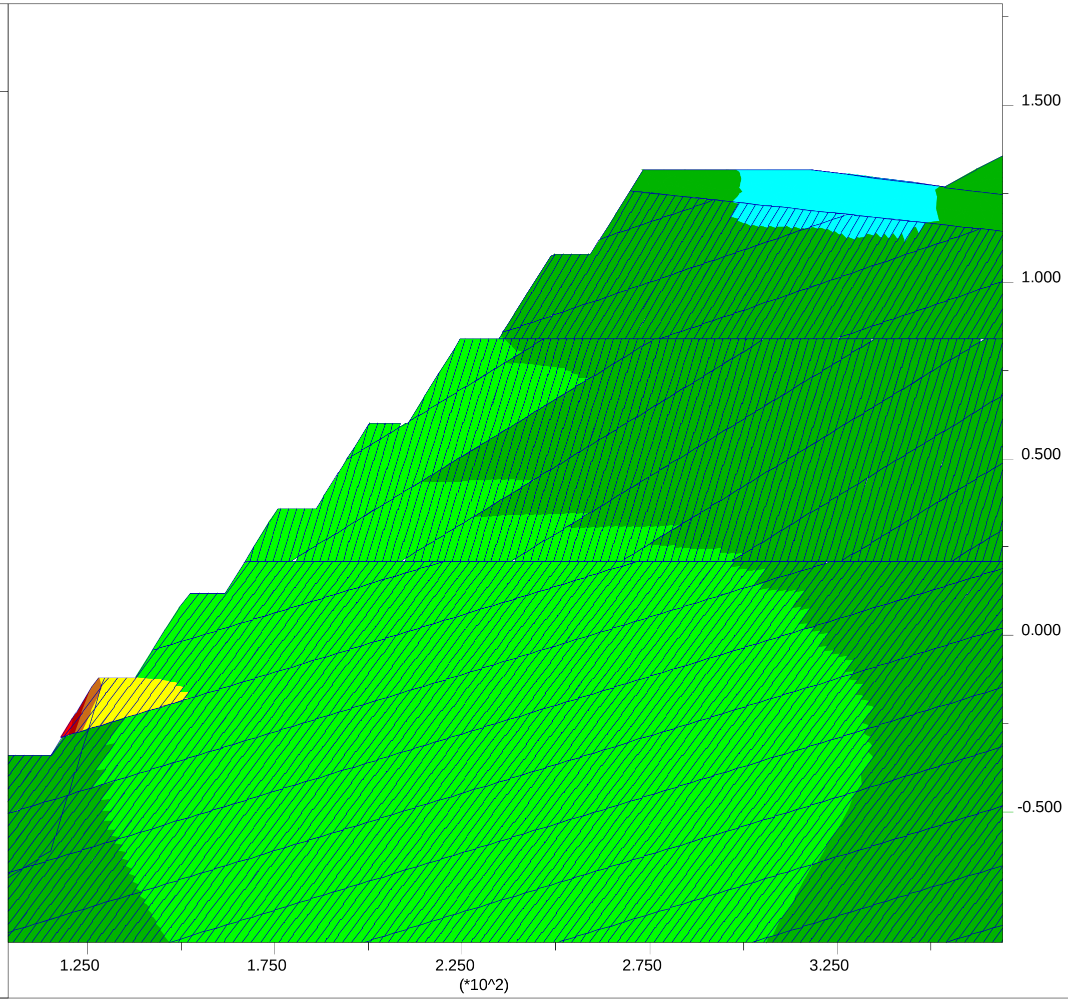
block plot



PROJECT
MEADOWBANK
MINING CORPORATION

TITLE
UDEC FOS ANALYSIS
GOOSE SOUTHEAST-12+00S
0% ROCK BRIDGES ON JOINTS DEPRESSURIZED
FOS = 1.3

PROJECT No.	06-1413-089		FILE No.	----
DESIGN	EMS	13MAR07	SCALE NTS	REV.
CADD	JFG	13MAR07	FIGURE 11.13	
CHECK	--			
REVIEW				

Golder Associates

REVISION DATE: 14MAR07 BY: FILE:N:\Bur-Graphics\Projects\2006\1413\06-1413-089\5000\Drafting\4000\Appendix 11_1.ppt

JOB TITLE : Goose 12+00SE

UDEC (Version 4.00)

LEGEND

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X displacement contours
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- -1.200E-01
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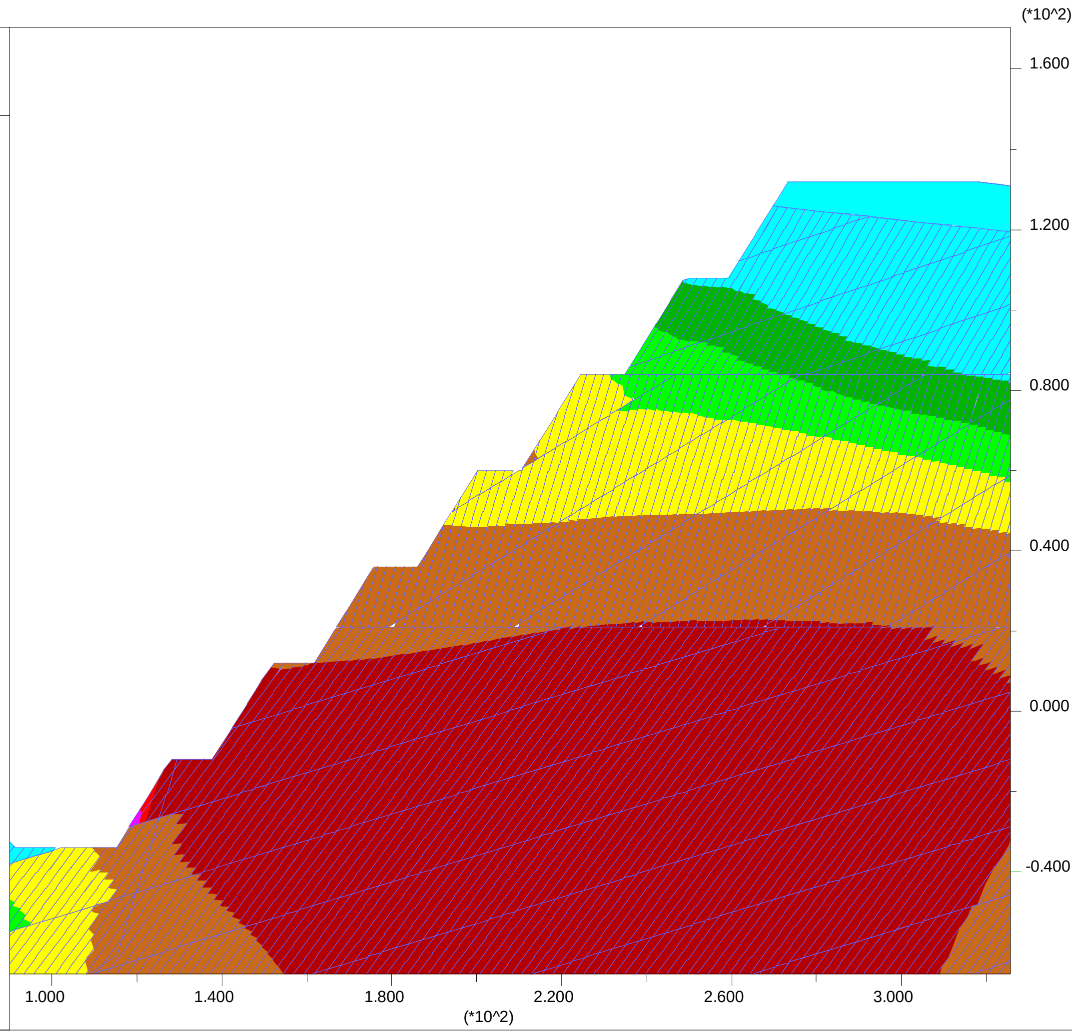
block plot



PROJECT	**MEADOWBANK MINING CORPORATION**			
TITLE	**UDEC STABILITY ANALYSIS GOOSE SOUTHEAST-12+00S 5% ROCK BRIDGES ON JOINTS DEPRESSURIZED**			
Golder Associates	PROJECT No.	06-1413-089		FILE No. ----
	DESIGN	EMS	13MAR07	SCALE NTS REV.
	CADD	JFG	13MAR07	**FIGURE 11.14**
	CHECK	--		
	REVIEW			

REVISION DATE: 14MAR07 BY: FILE:N:\Bur-Graphics\Projects\2006\1413\06-1413-089\5000\Drafting\4000\Appendix 11_1.ppt

JOB TITLE : Goose 12+00SE

UDEC (Version 4.00)

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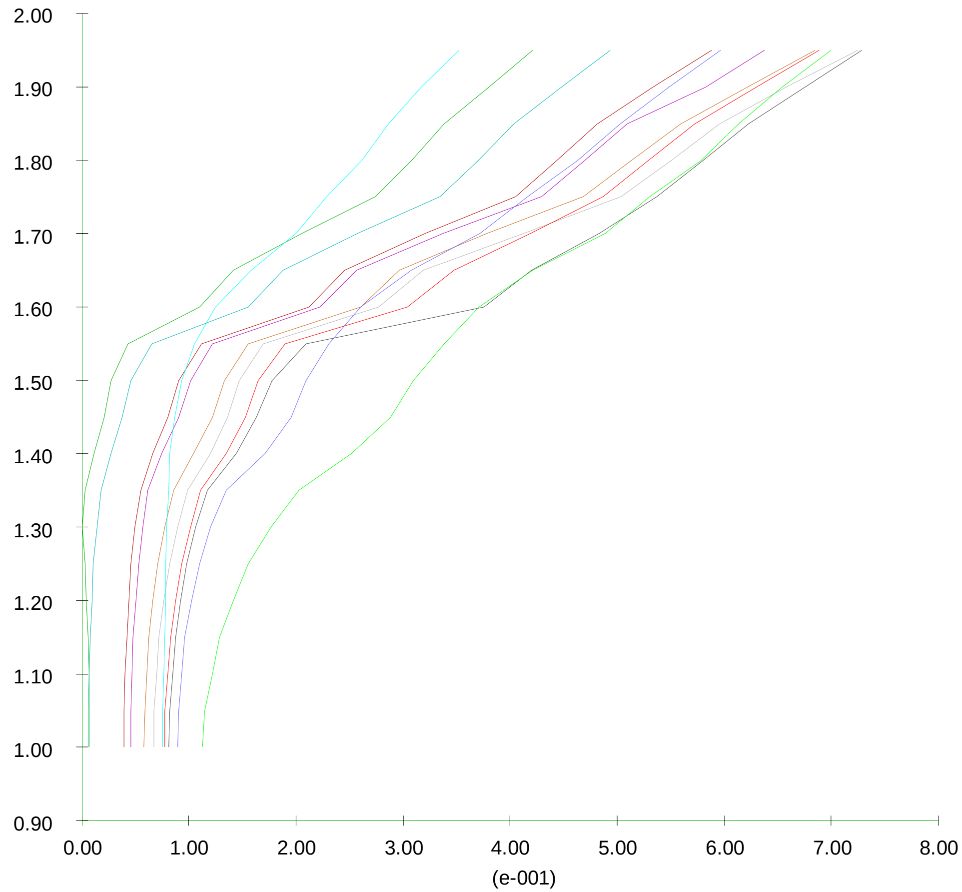


PROJECT

MEADOWBANK
MINING CORPORATION

TITLE

UDEC FOS ANALYSIS
GOOSE SOUTHEAST-12+00S
GLUED - 5% ROCK BRIDGES ON JOINTS – DEPRESSURIZED
FOS vs. HORIZONTAL DISPLACEMENT OF THE PIT SLOPE

Golder Associates

PROJECT No.	06-1413-089		FILE No. ----	
DESIGN	EMS	13MAR07	SCALE NTS	REV.
CADD	JFG	13MAR07	**FIGURE 11.15**	
CHECK	--			
REVIEW				

REVISION DATE: 14MAR07 BY: FILE:N:\Bur-Graphics\Projects\2006\1413\06-1413-089\5000\Drafting\4000\Appendix 11_1.ppt

JOB TITLE : Goose 12+00SE

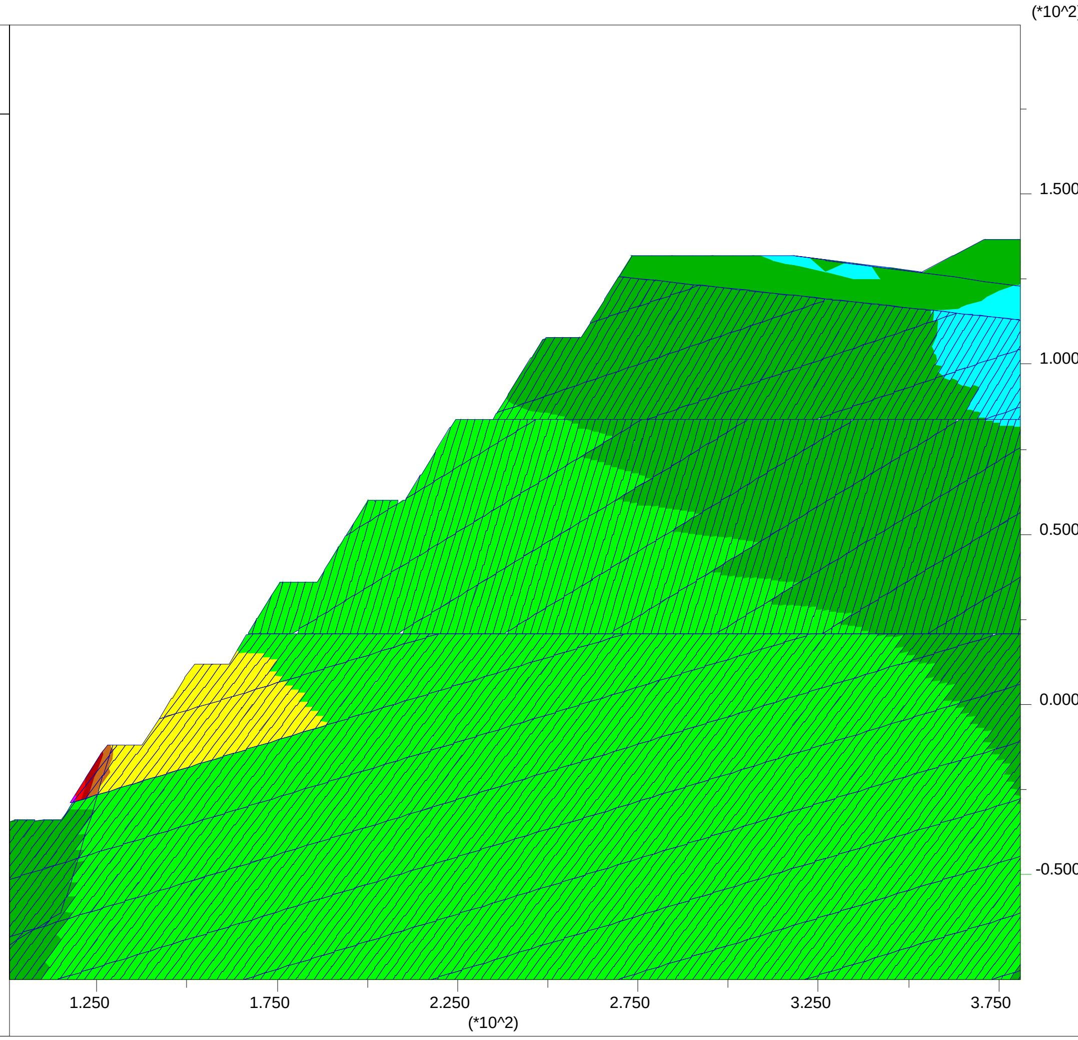
UDEC (Version 4.00)

LEGEND

3-Mar-07 3:04
cycle 65880
X displacement contours
contour interval= 1.000E-01
-6.000E-01 to 0.000E+00

-6.000E-01
-5.000E-01
-4.000E-01
-3.000E-01
-2.000E-01
-1.000E-01
0.000E+00

block plot



PROJECT

MEADOWBANK
MINING CORPORATION

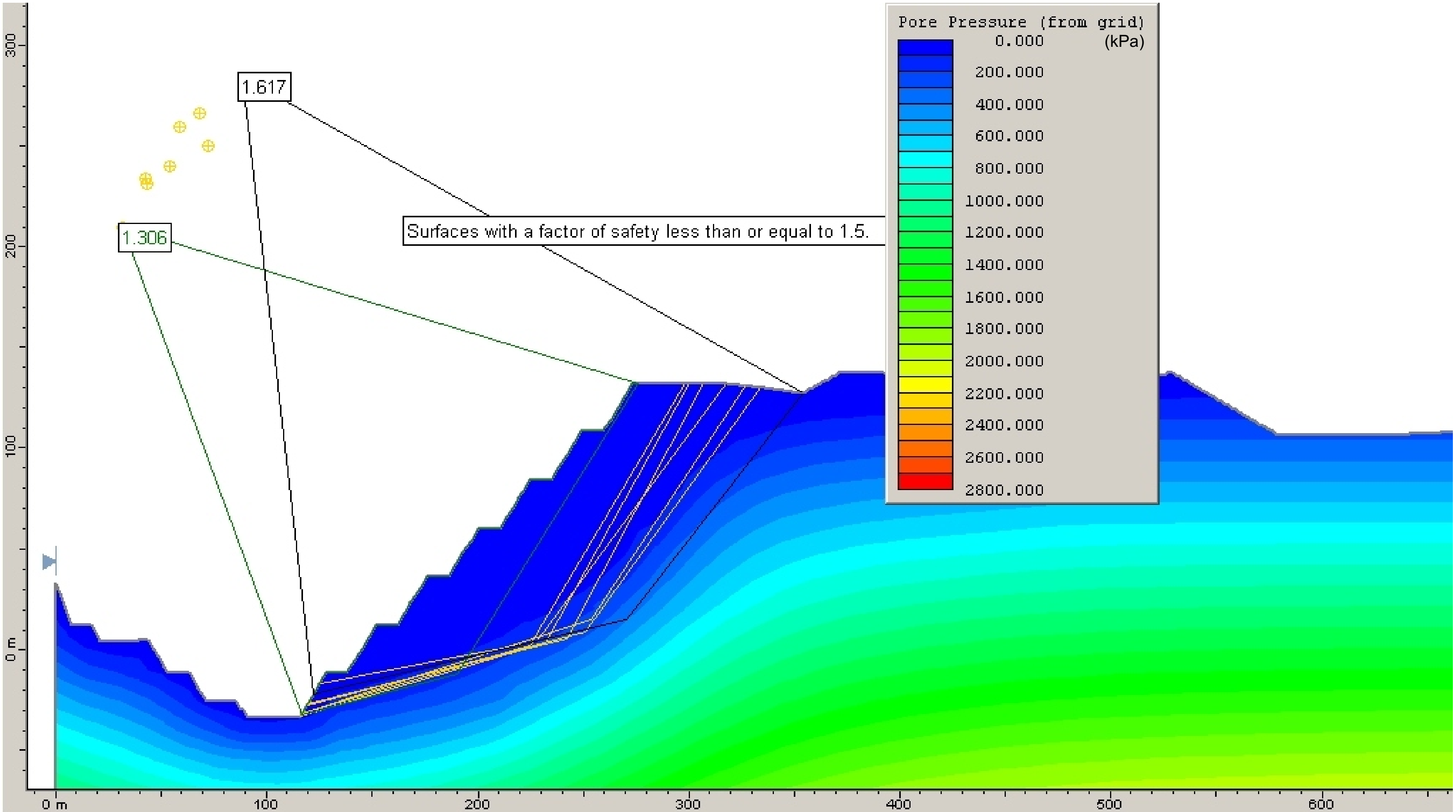
TITLE

UDEC FOS ANALYSIS
GOOSE SOUTHEAST-12+00S
5% ROCK BRIDGES ON JOINTS – DEPRESSURIZED
FOS = 1.55

Golder Associates

PROJECT No.	06-1413-089		FILE No. ----	
DESIGN	EMS	13MAR07	SCALE NTS	REV.
CADD	JFG	13MAR07	**FIGURE 11.16**	
CHECK	--			
REVIEW				

DRAWING DATE: 06-Mar-07 COREL FILE: N:\Bur-Graphics\Projects\2006\1413\06-1413-089\5000\Drafting\4000\Pit slopes report\FIGURES.cdr

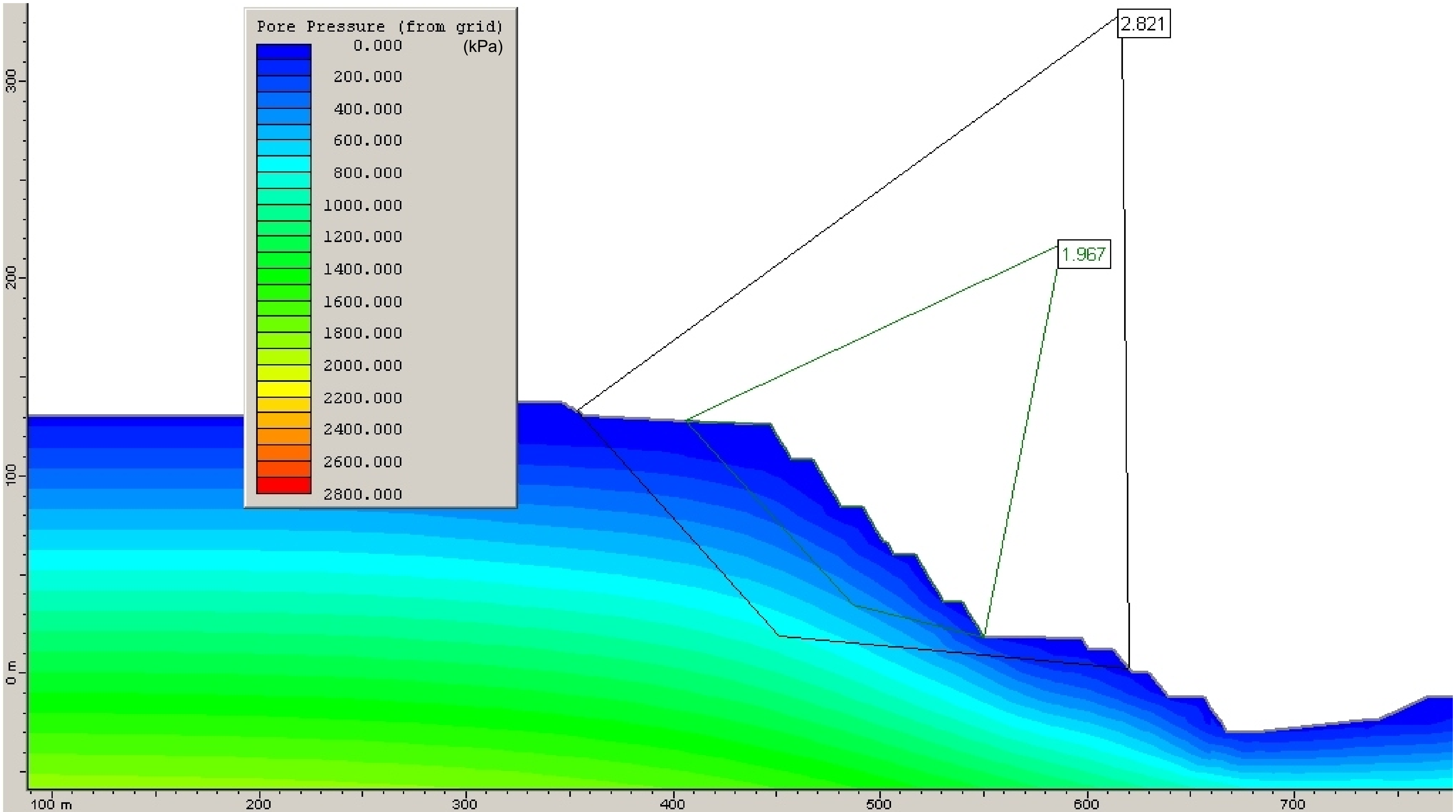


PROJECT	**MEADOWBANK MINING CORPORATION**
TITLE	**SLIDE ANALYSIS GOOSE SOUTHEAST - 12+00S DEPRESSURIZED ANISOTROPIC GROUNDWATER FLOW 0% ROCK BRIDGE**

Golder Associates

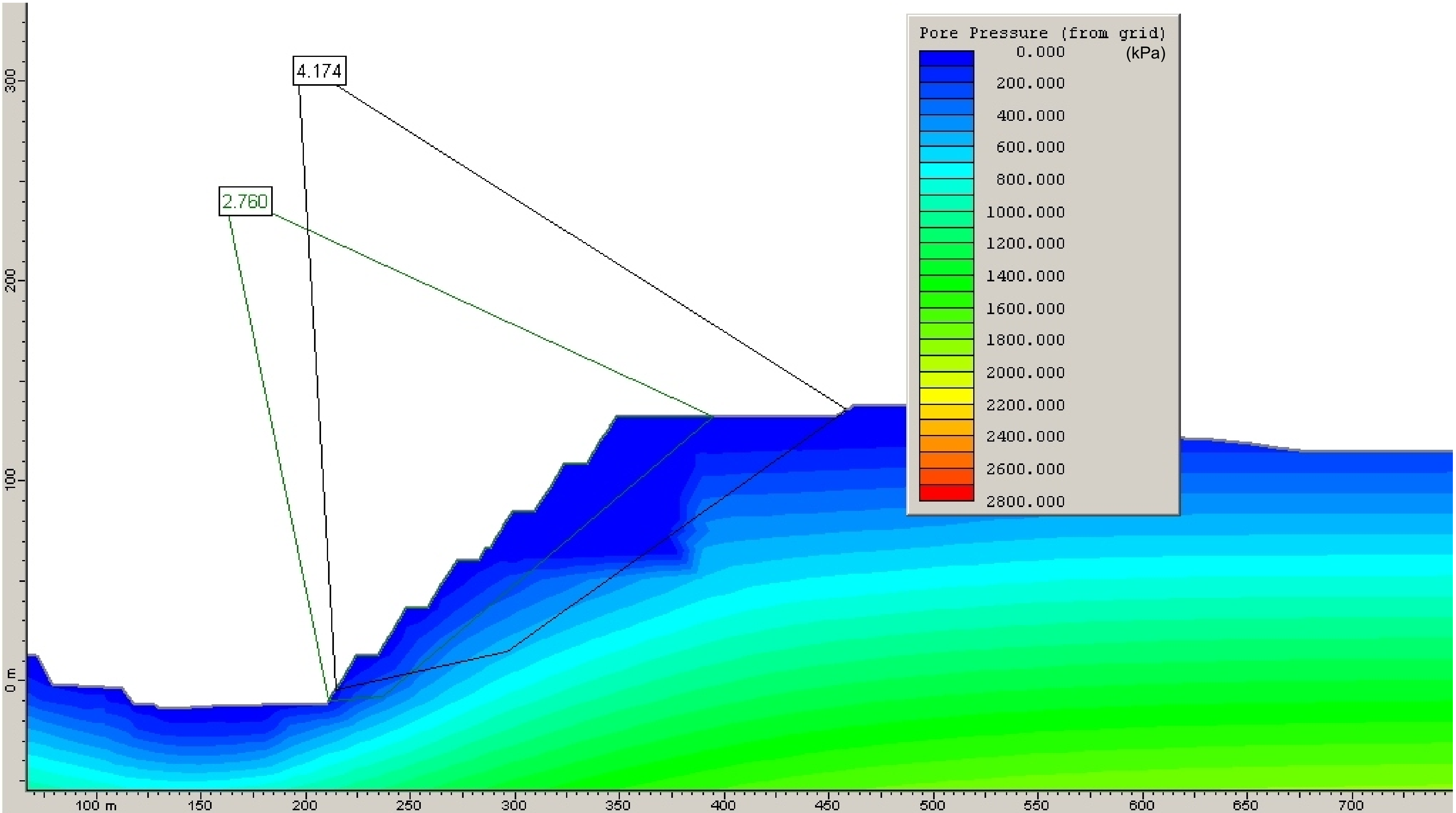
PROJECT No.	06-1413-089		FILE No.	FIGURES
DESIGN	JG	06MAR07	SCALE NTS	REV.
CADD	AS	06MAR07	**FIGURE 11.17**	
CHECK	--	--		
REVIEW				

DRAWING DATE: 06-Mar-07 COREL FILE: N:\Bur-Graphics\Projects\2006\1413\06-1413-089\5000\Drafting\4000\Pit slopes report\FIGURES.cdr



PROJECT	**MEADOWBANK MINING CORPORATION**			
TITLE	**SLIDE ANALYSIS GOOSE - SOUTH NO DEPRESSURIZATION 0% ROCK BRIDGE**			
Golder Associates	PROJECT No.	06-1413-089	FILE No. FIGURES	
	DESIGN	JG	06MAR07	SCALE NTS	REV.
	CADD	AS	06MAR07	**FIGURE 11.18**
	CHECK	--	--	
	REVIEW			

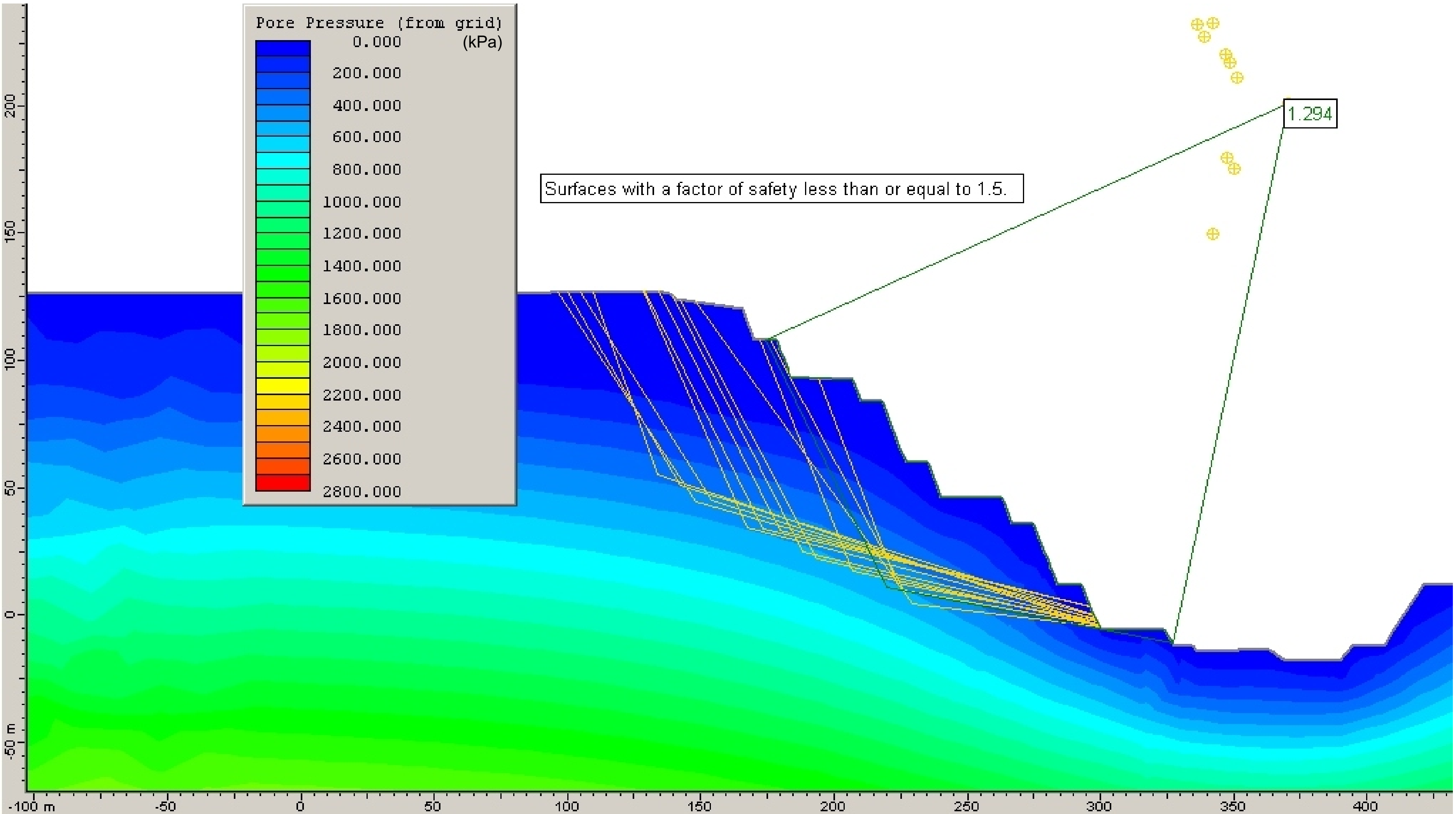
DRAWING DATE: 06-Mar-07 COREL FILE: N:\Bur-Graphics\Projects\2006\1413\06-1413-089\5000\Drafting\4000\Pit slopes report\FIGURES.cdr



PROJECT	**MEADOWBANK MINING CORPORATION**			
TITLE	**SLIDE ANALYSIS GOOSE - NORTHEAST NO DEPRESSURIZATION 0% ROCK BRIDGE**			
PROJECT No.	06-1413-089		FILE No.	FIGURES
DESIGN	JG	06MAR07	SCALE NTS	REV.
CADD	AS	06MAR07	**FIGURE 11.19**	
CHECK	--	--		
REVIEW				

Golder Associates

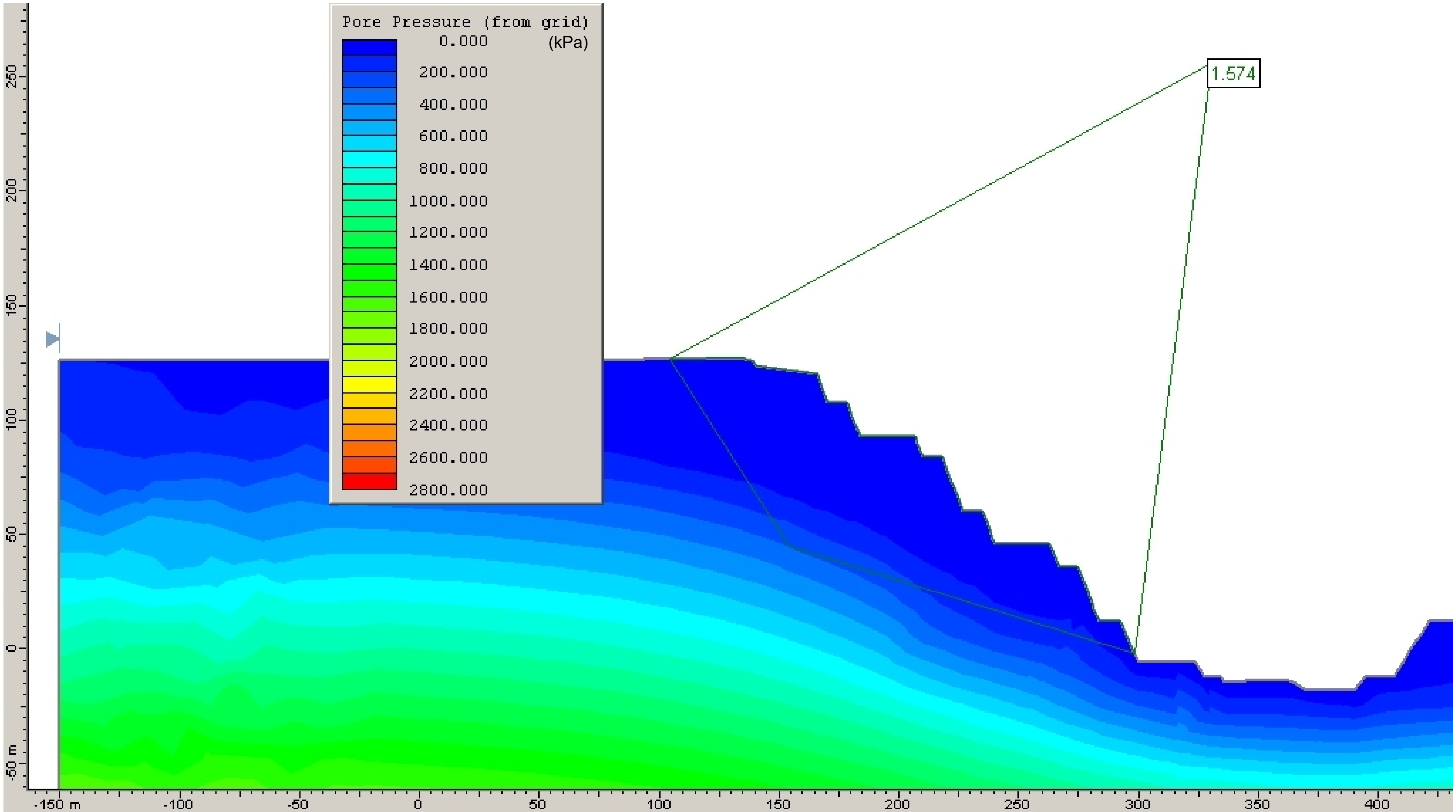
DRAWING DATE: 06-Mar-07 COREL FILE: N:\Bur-Graphics\Projects\2006\1413\06-1413-089\5000\Drafting\4000\Pit slopes report\FIGURES.cdr



PROJECT	**MEADOWBANK MINING CORPORATION**
TITLE	**SLIDE ANALYSIS GOOSE WEST - 11+00S NO DEPRESSURIZATION 5% ROCK BRIDGE**

PROJECT No.	06-1413-089		FILE No.	FIGURES
DESIGN	JG	06MAR07	SCALE NTS	REV.
CADD	AS	06MAR07	**FIGURE 11.20**	
CHECK	--	--		
REVIEW				

Golder Associates



PROJECT
MEADOWBANK
MINING CORPORATION

TITLE
SLIDE ANALYSIS
GOOSE WEST - 11+00S
DEPRESSURIZED 0% ROCK BRIDGE

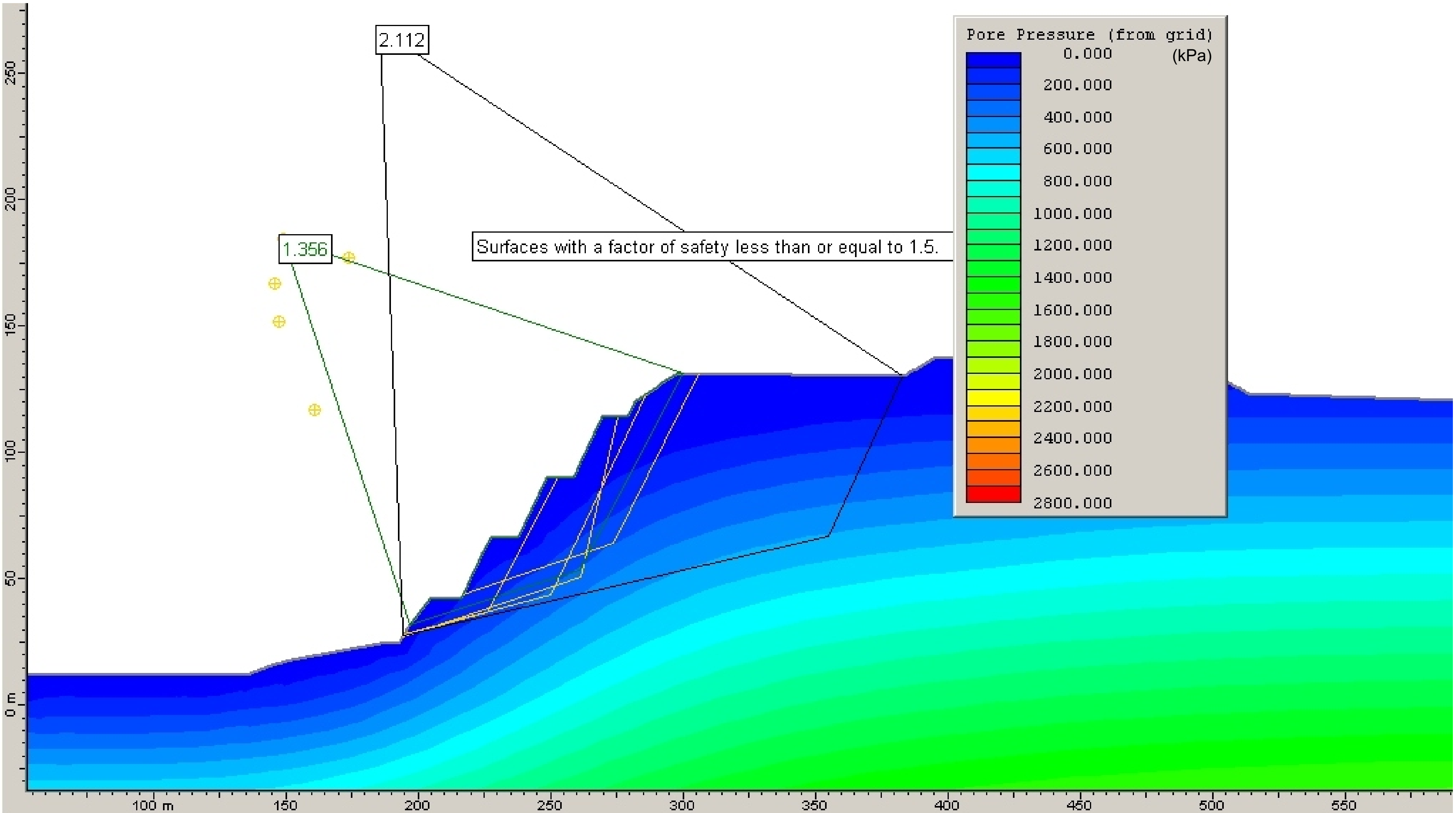
Golder Associates

PROJECT No.	06-1413-089	
DESIGN	JG	06MAR07
CADD	AS	06MAR07
CHECK	--	--
REVIEW		

FILE No. FIGURES
SCALE NTS REV.

FIGURE 11.21

DRAWING DATE: 06-Mar-07 COREL FILE: N:\Bur-Graphics\Projects\2006\1413\06-1413-089\5000\Drafting\4000\Pit slopes report\FIGURES.cdr



PROJECT

MEADOWBANK
MINING CORPORATION

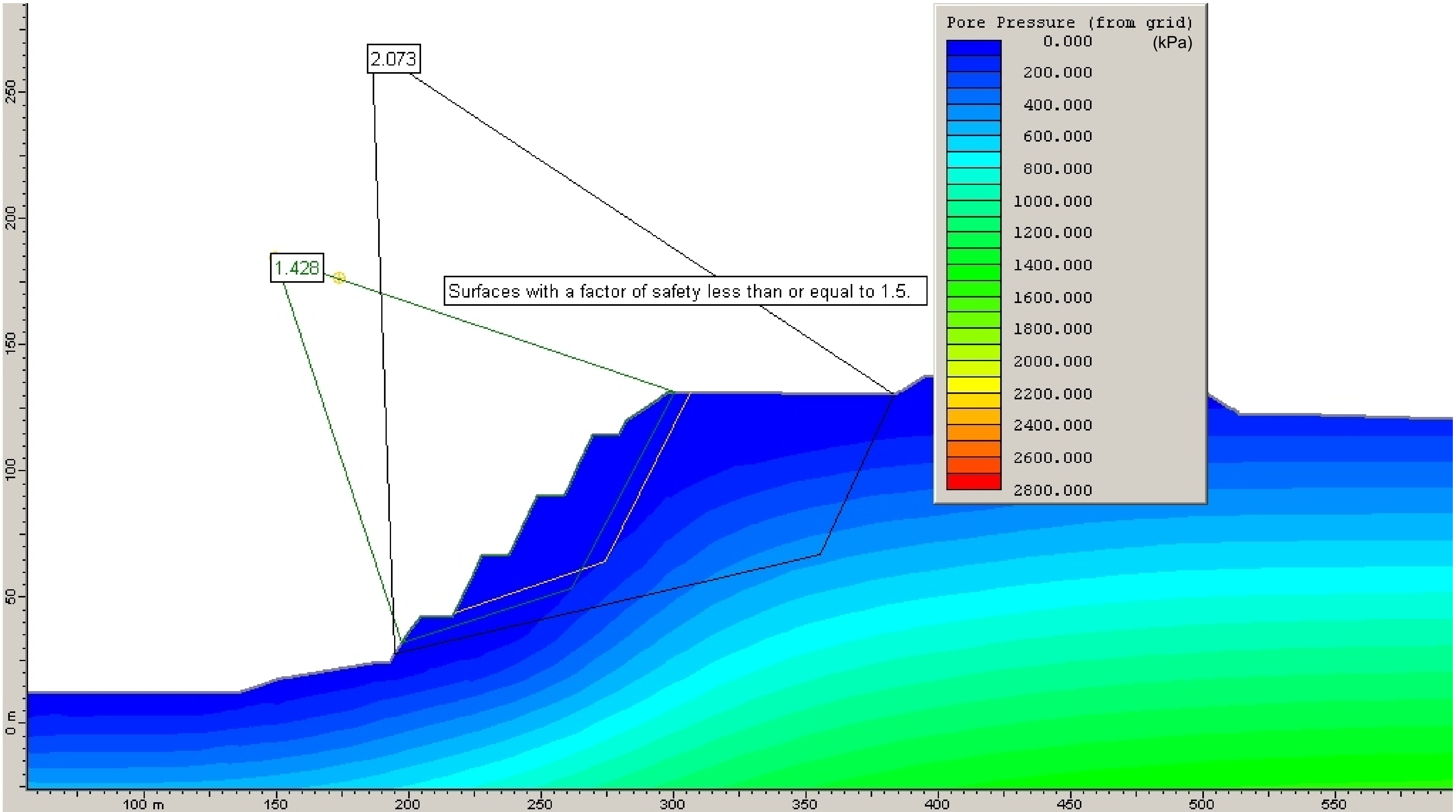
TITLE

SLIDE ANALYSIS
PORTAGE - SOUTHEAST
NO DEPRESSURIZATION 5% ROCK BRIDGE

Golder Associates

PROJECT No.		06-1413-089	FILE No.	FIGURES
DESIGN	JG	06MAR07	SCALE NTS	REV.
CADD	AS	06MAR07	**FIGURE 11.22**	
CHECK	--	--		
REVIEW				

DRAWING DATE: 06-Mar-07 COREL FILE: N:\Bur-Graphics\Projects\2006\1413\06-1413-089\5000\Drafting\4000\Pit slopes report\FIGURES.cdr



PROJECT
MEADOWBANK
MINING CORPORATION

TITLE
SLIDE ANALYSIS
PORTAGE - SOUTHEAST
DEPRESSURIZED 0% ROCK BRIDGE

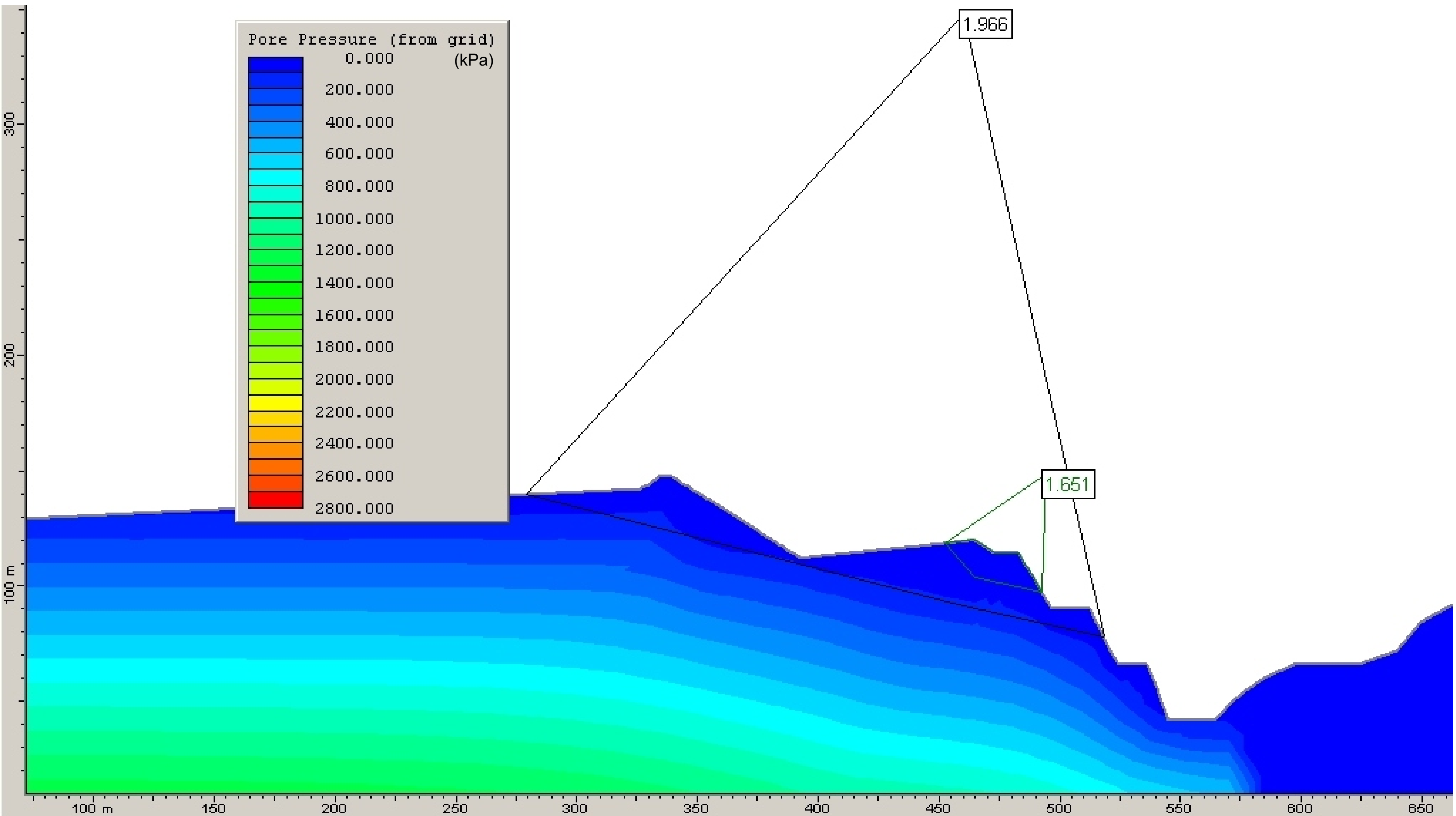
Golder Associates

PROJECT No.	06-1413-089	
DESIGN	JG	06MAR07
CADD	AS	06MAR07
CHECK	--	--
REVIEW		

FILE No. FIGURES
SCALE NTS
REV.

FIGURE 11.23

DRAWING DATE: 06-Mar-07 COREL FILE: N:\Bur-Graphics\Projects\2006\1413\06-1413-089\5000\Drafting\4000\Pit slopes report\FIGURES.cdr



PROJECT

MEADOWBANK
MINING CORPORATION

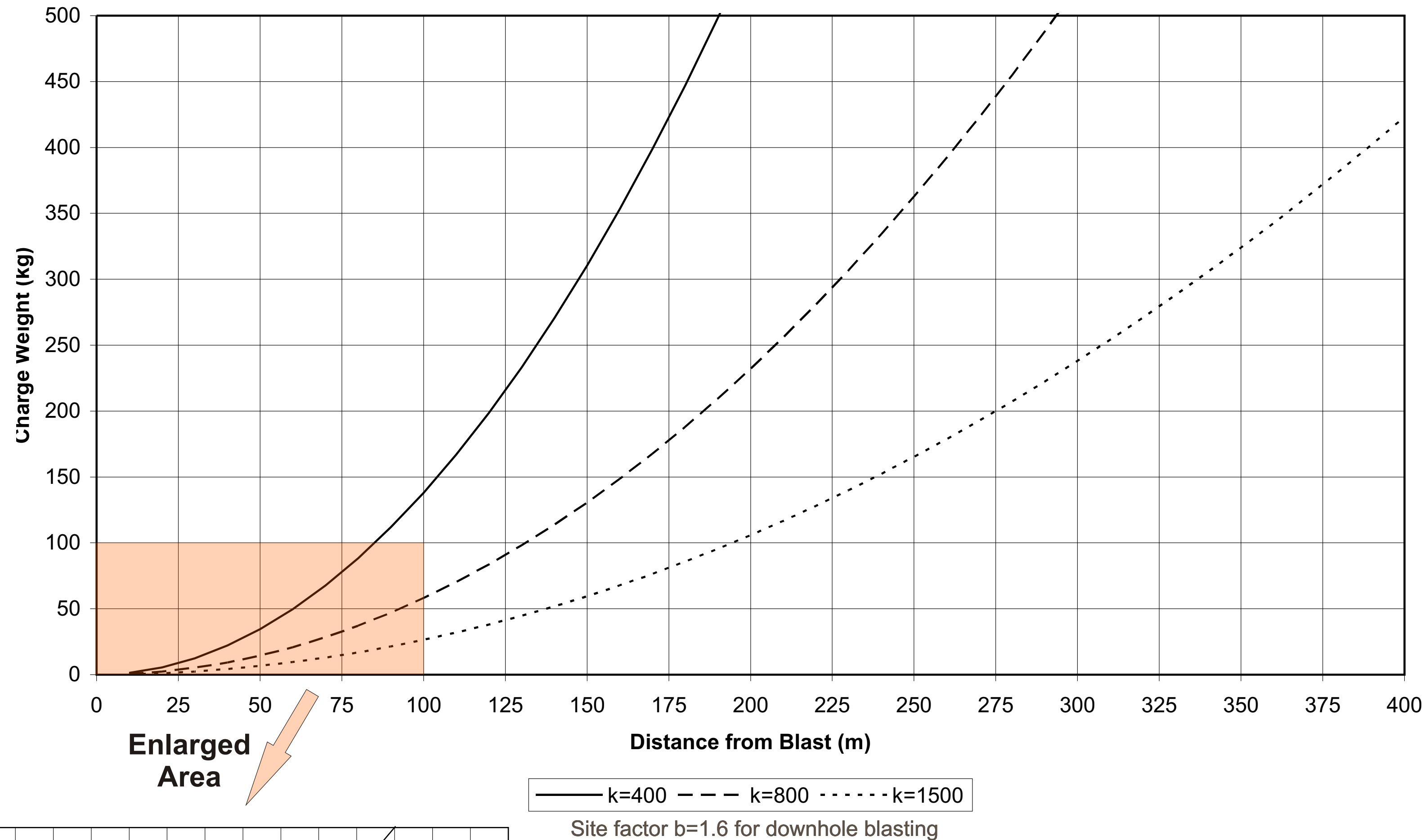
TITLE

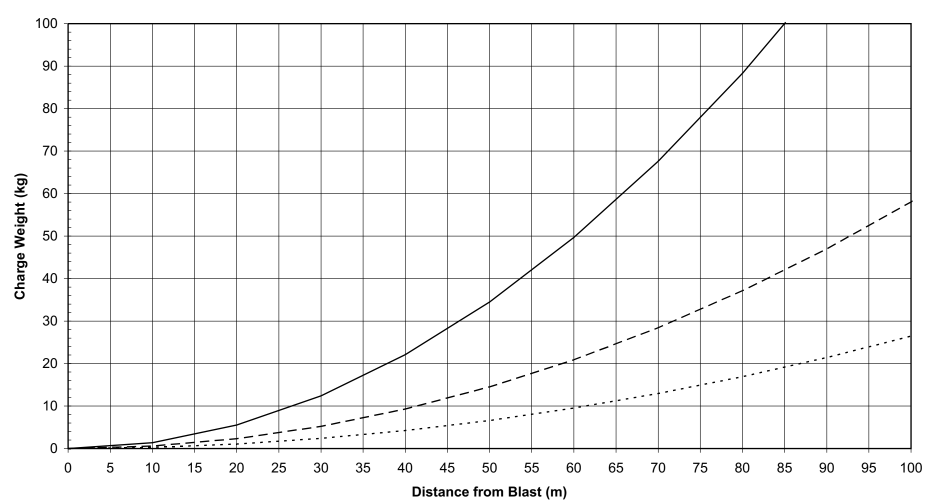
SLIDE ANALYSIS
PORTAGE/TAILINGS - NORTHWEST
NO DEPRESSURIZATION 0% ROCK BRIDGE

Golder Associates

PROJECT No.	06-1413-089		FILE No. FIGURES	
DESIGN	JG	06MAR07	SCALE NTS	REV.
CADD	AS	06MAR07	**FIGURE 11.24**	
CHECK	--	--		
REVIEW				

REVISION DATE: 15MAR07 BY: SS COREL FILE: N:\Bur-Graphics\Projects\2006\1413\06-1413-089\5000\Drafting\4000\Appendix 12.cdr





PROJECT

MEADOWBANK
MINING CORPORATION

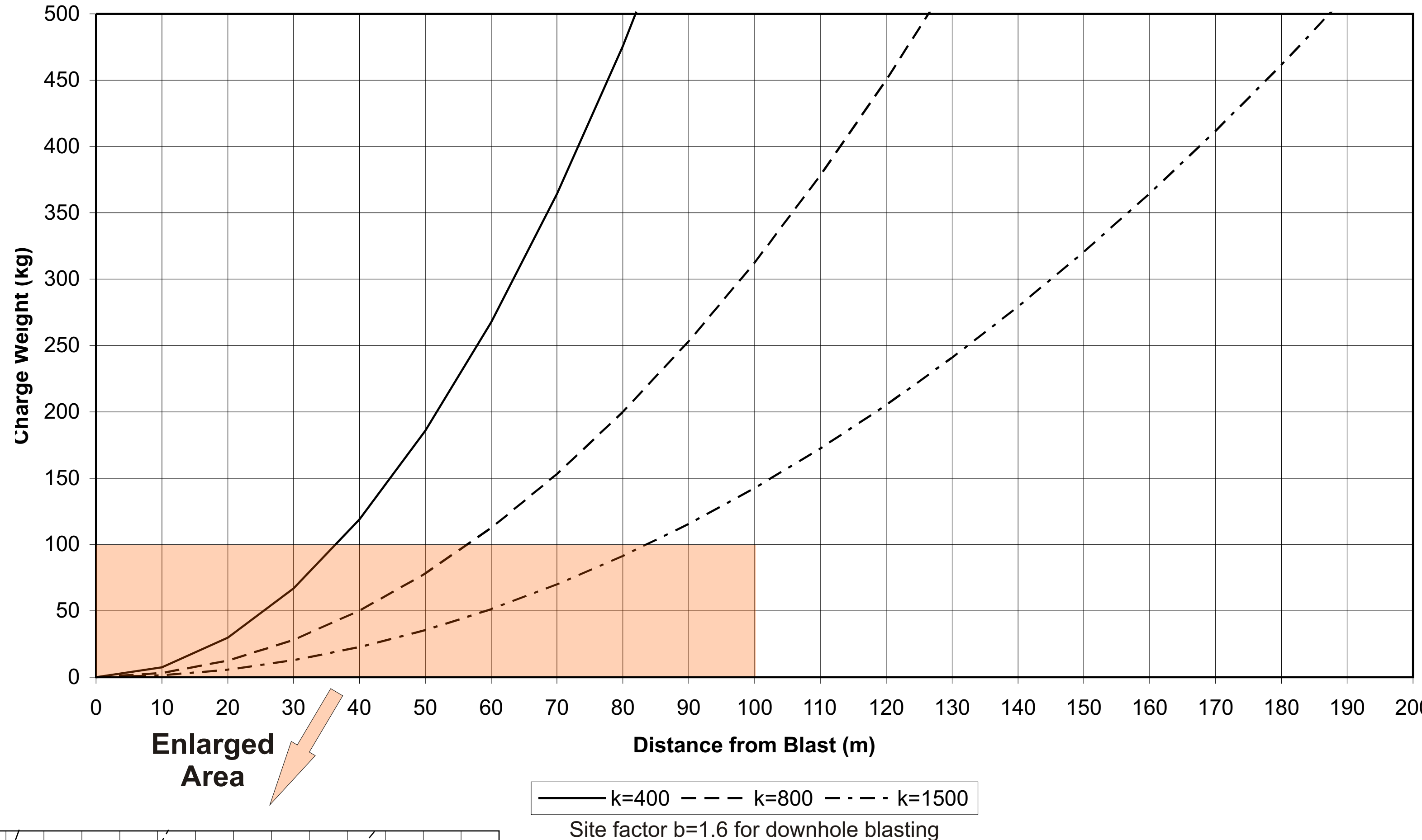
TITLE

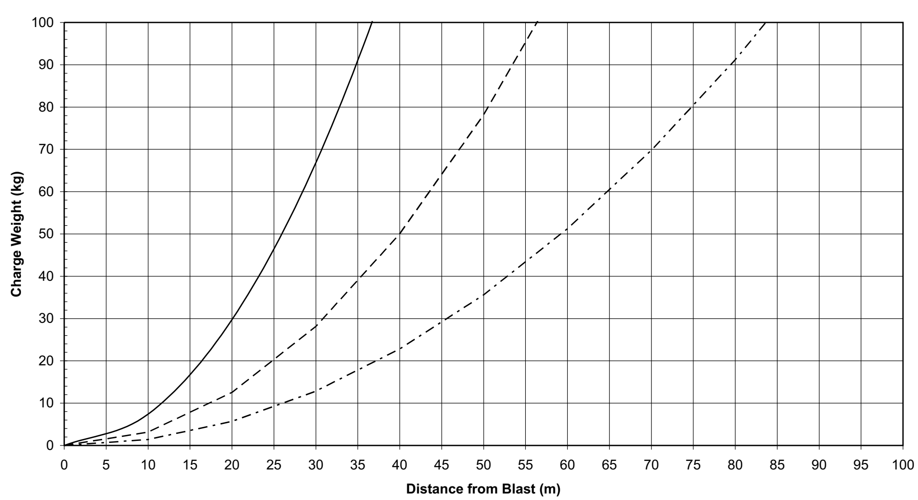
CHARGE WEIGHT vs DISTANCE FROM BLAST - PPV = 13mm/s

Golder Associates

PROJECT No.	06-1413-089		FILE No. Appendix 12	
DESIGN	CJC	28JAN04	SCALE NTS	REV.
CADD	SS	28JAN04	**FIGURE 12.1**	
CHECK				
REVIEW				

REVISION DATE: 15MAR07 BY: SS COREL FILE: N:\Bur-Graphics\Projects\2006\1413\06-1413-089\5000\Drafting\4000\Appendix 12.cdr





PROJECT

MEADOWBANK

MINING CORPORATION

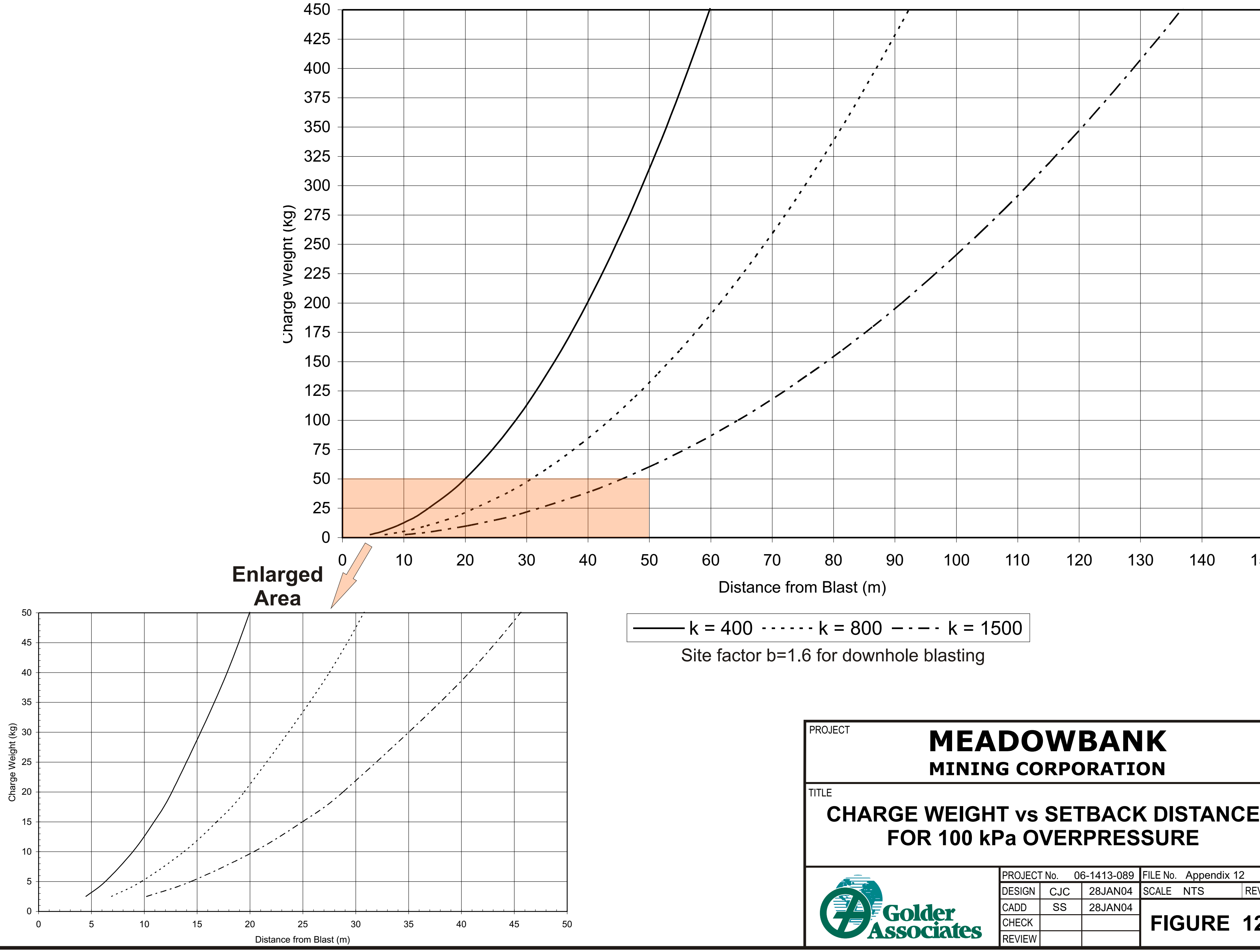
TITLE

CHARGE WEIGHT vs DISTANCE FROM BLAST - PPV = 50mm/s

Golder Associates

PROJECT No.	06-1413-089		FILE No. Appendix 12	
DESIGN	CJC	28JAN04	SCALE NTS	REV.
CADD	SS	28JAN04	**FIGURE 12.2**	
CHECK				
REVIEW				

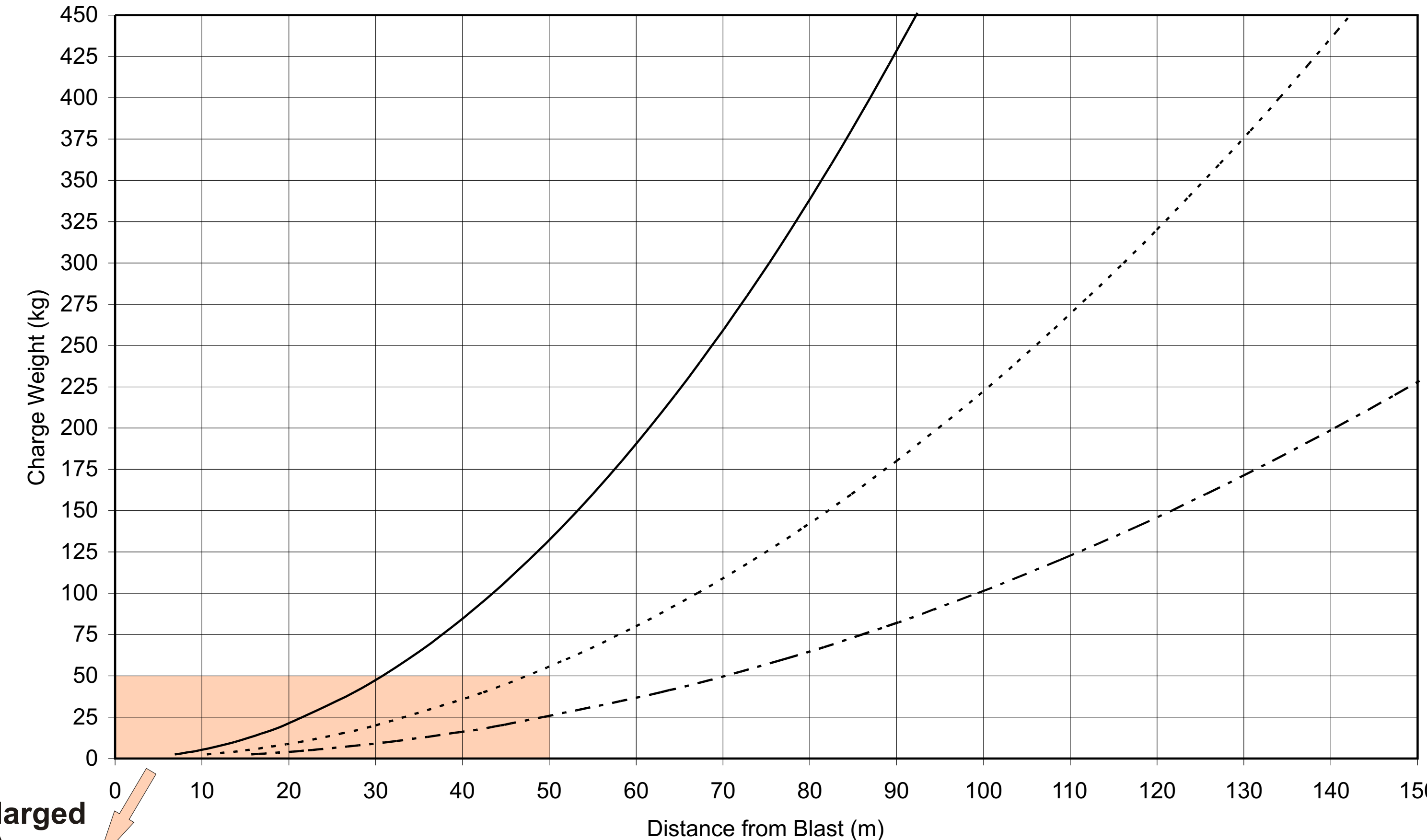
REVISION DATE: 15MAR07 BY: SS COREL FILE: N:\Bur-Graphics\Projects\2006\1413\06-1413-089\5000\Drafting\4000\Appendix 12.cdr



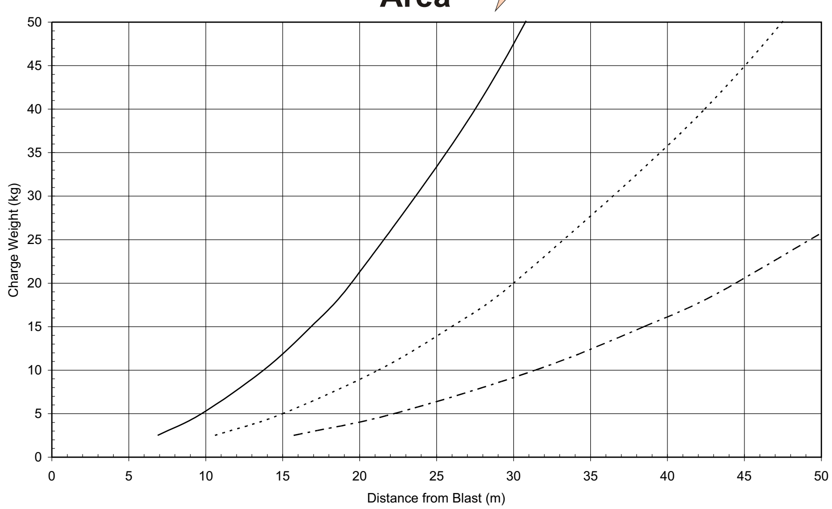
PROJECT	**MEADOWBANK MINING CORPORATION**			
TITLE	**CHARGE WEIGHT vs SETBACK DISTANCE FOR 100 kPa OVERPRESSURE**			
Golder Associates	PROJECT No.	06-1413-089		FILE No. Appendix 12
	DESIGN	CJC	28JAN04	SCALE NTS / REV.
	CADD	SS	28JAN04	**FIGURE 12.3**
	CHECK			
	REVIEW			

REVISION DATE: 15MAR07 BY: SS COREL FILE: N:\Bur-Graphics\Projects\2006\1413\06-1413-089\5000\Drafting\4000\Appendix 12.cdr

Setback Distance as a Function of Charge Weight to Achieve 50 kPa Instantantaneous Overpressure site factor b = 1.6 for downhole blasting conditions.



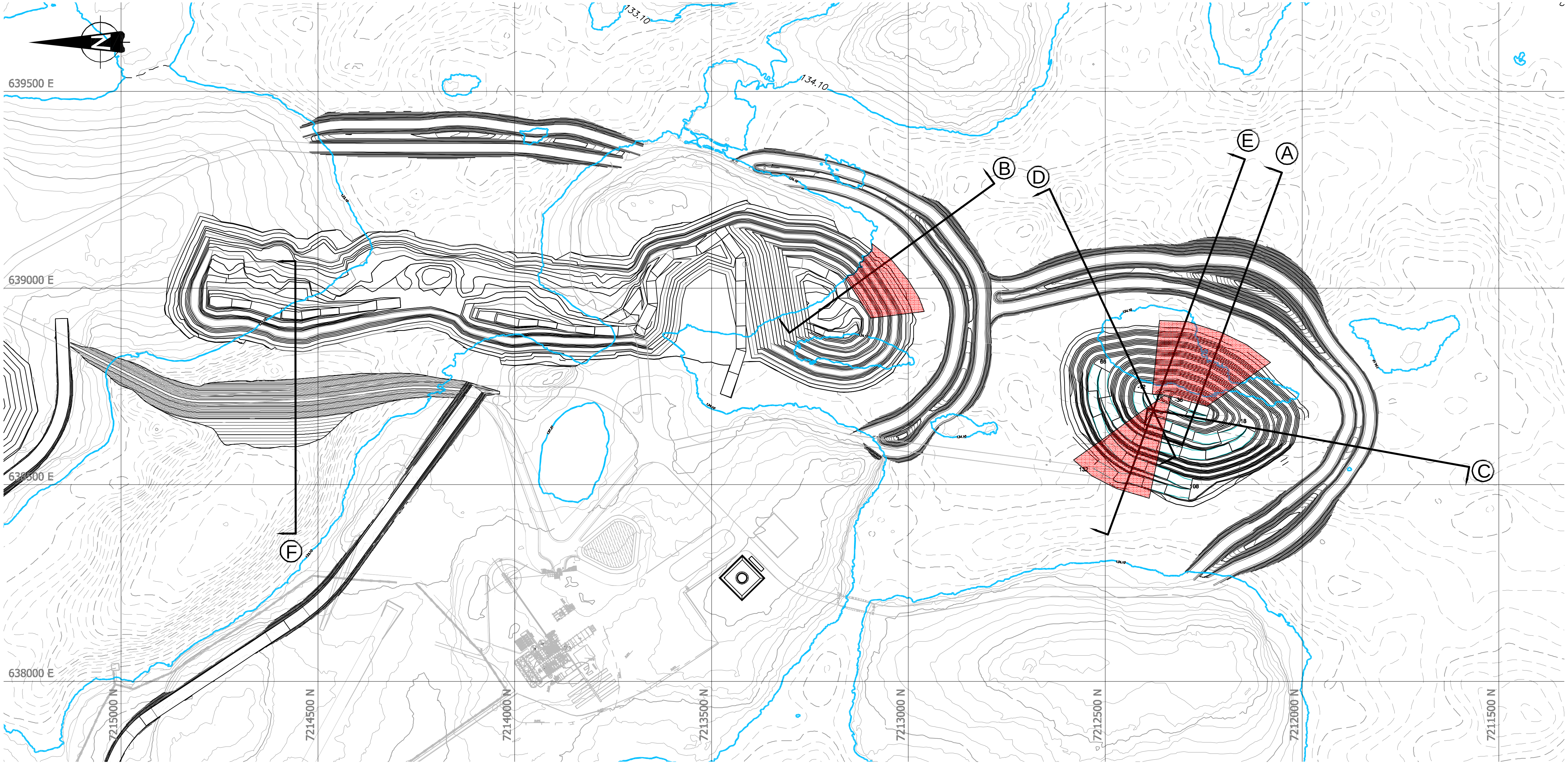
Enlarged Area



k = 400 k = 800 k = 1500

Site factor b=1.6 for downhole blasting

PROJECT	**MEADOWBANK MINING CORPORATION**				
TITLE	**CHARGE WEIGHT vs SETBACK DISTANCE FOR 50 kPa OVERPRESSURE**				
Golder Associates	PROJECT No.	06-1413-089		FILE No. Appendix 12	
	DESIGN	CJC	28JAN04	SCALE NTS	REV.
	CADD	SS	28JAN04	**FIGURE 12.4**	
	CHECK				
	REVIEW				



LEGEND

AREA REQUIRING DEPRESSURIZATION

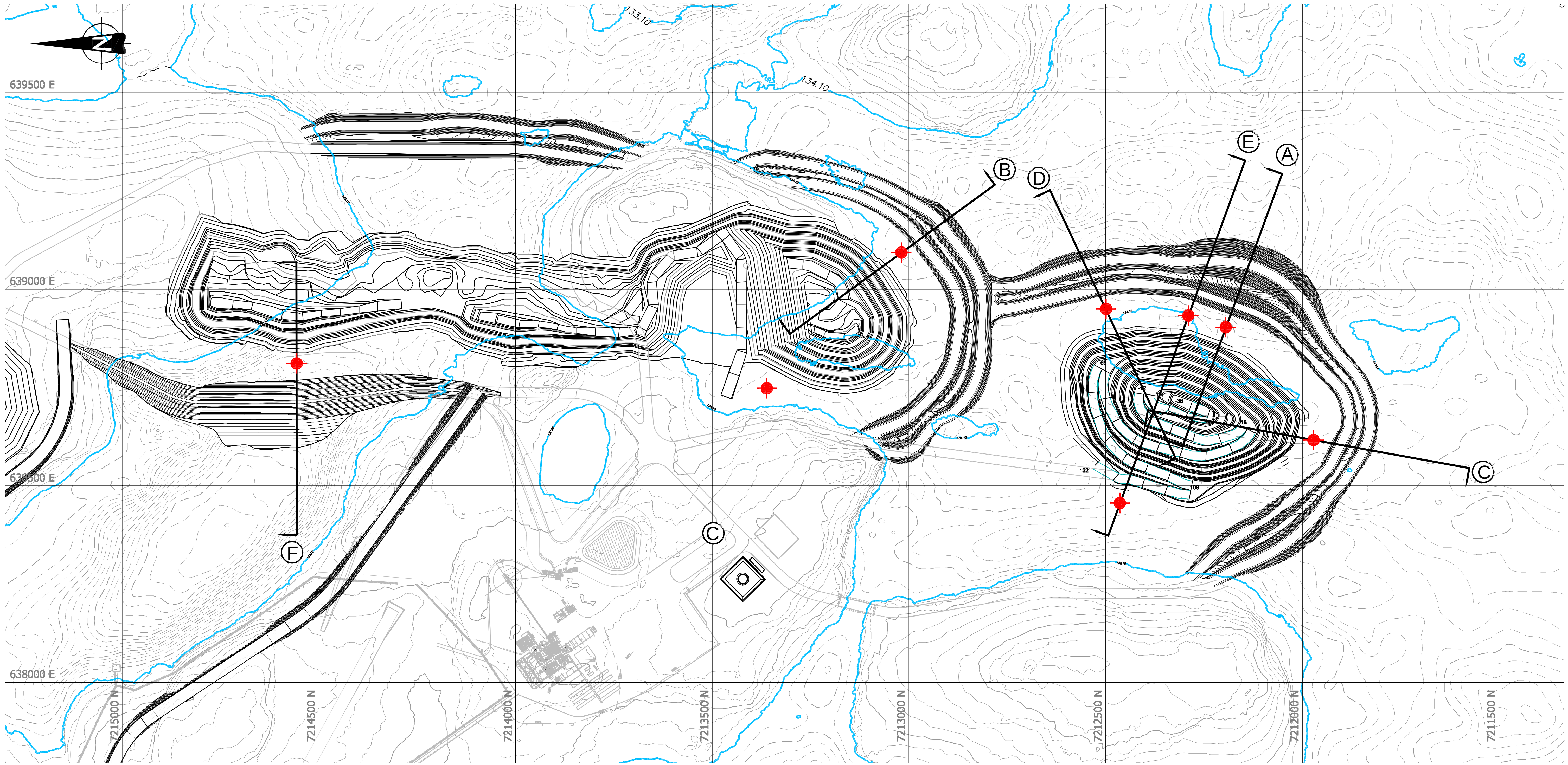
SECTION A: GOOSE SOUTHEAST - 12+00 S
SECTION B: PORTAGE SOUTHEAST
SECTION C: GOOSE SOUTH
SECTION D: GOOSE NORTHEAST
SECTION E: GOOSE SOUTHEAST - 11+00 S
SECTION F: PORTAGE/TAILINGS NORTHEAST

0 100 200 300 400 500
Scale in Meters

PROJECT	**MEADOWBANK MINING CORPORATION**			
TITLE	**AREAS OF THE PIT SLOPES REQUIRING DEPRESSURIZATION**			
PROJECT No.	06-1413-089		FILE No.	5000-4000-05
DESIGN	ES	20FEB07	SCALE	AS SHOWN
CADD	AS	20FEB07	REV.	
CHECK			**FIGURE 13.1**	
REVIEW				

Golder Associates

CADD FILE: N:\Bur-Graphics\Projects\2006\1413\06-1413-089\5000\Drafting\4000\061413089-5000-4000-05.dwg
REVISION DATE: 07/04/03 04:45PM By: ASalvador



LEGEND

MONITORING WELL LOCATION

SECTION A: GOOSE SOUTHEAST - 12+00 S
SECTION B: PORTAGE SOUTHEAST
SECTION C: GOOSE SOUTH
SECTION D: GOOSE NORTHEAST
SECTION E: GOOSE SOUTHEAST - 11+00 S
SECTION F: PORTAGE/TAILINGS NORTHEAST

0 100 200 300 400 500
Scale in Meters

PROJECT
MEADOWBANK
MINING CORPORATION

TITLE
MONITORING WELL LOCATIONS

PROJECT No.		06-1413-089	FILE No.	5000-4000-06
DESIGN	ES	20FEB07	SCALE	AS SHOWN	REV.
CADD	AS	20FEB07		
CHECK			**FIGURE 14.1**	
REVIEW				

CADD FILE: N:\Bur-Graphics\Projects\2006\1413\06-1413-089\5000\Drafting\4000\061413089-5000-4000-06.dwg

REVISION DATE: 07/04/03 04:45PM By: ASalvador