FINAL
REPORT ON

QUARRY BENCH HEIGHT EVALUATION
PROPOSED FLAMBOROUGH QUARRY
ST MARYS CEMENT INC.

Submitted to:

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November 2008 08-1117-0009

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1.0 INTRODUCTION

The following report provides a summary of the geotechnical evaluation of the planned bench faces for a proposed quarry project located on Part of Lots 1, 2 and 3, Concession 11 in the geographic Township of East Flamborough, now the City of Hamilton. Specifically, this report evaluates the stability of the proposed quarry bench faces which are planned as single vertical faces up to 38 m high.

In order to carry out the stability assessment, use was made of previous geotechnical investigations. The available geotechnical data included a geotechnical report with borehole logs for 12 boreholes to depths of 35 m to 45 m and a geophysical investigation which included acoustic televiewer data collected in 8 boreholes. Golder Associates interpreted the televiewer data in order to determine the orientation of the discontinuities encountered in the boreholes. This discontinuity data was then used in the kinematic analysis to determine the potential for structurally controlled instabilities along the proposed quarry bench faces.

This assessment was carried out by Mr. Mark Telesnicki, P.Eng., a Principal with Golder Associates, who has been involved in rock mechanics and rock slope stability projects for over 20 years. Mr. Telesnicki, whose curriculum vitae is found in Appendix B, has been involved with a number of open pit and quarry bench design projects throughout North and South America, Asia, and Africa.

2.0 BACKGROUND DATA

The following geotechnical data was made available for the current study:

1. Report by John Emery Geotechnical Engineering Ltd. entitled “Geological Investigation, Proposed Dolostone Quarry, Part of Lot 1 and All of Lots 2 and 3, Concession 11, Geographic Township of East Flamborough, City of Hamilton”, dated July 16, 2004.

2. Report by Lotowater Technical Services Inc. entitled “Flamborough Quarry, Geophysical Logging and Testing Results for Monitoring and Test Wells” dated July 24, 2007.

The borehole data from the geological investigation was used to assess of the rock lithology and rock quality and the geophysical logging was used to determine the orientation of the major discontinuities which could have an impact on the stability of the final bench faces.

3.0 SITE CONDITIONS

The proposed quarry property is situated approximately 6 km north of Carlisle and about 10 km west of the Town of Milton, Ontario in the Township of East Flamborough, now the City of Hamilton, as seen in Figure 1. The area being considered for quarry development is bounded by Milburough Line to the east and 11th Concession Road East to the south (see Figure 2).

The general topography of the proposed quarry property consists of gently rolling terrain with bedrock outcrops evident at several locations around the site. Based on previous geotechnical investigations the bedrock at the site consists of Amabel Formation dolostone with an average thicknesses ranging from 27 to 40 m.

4.0 PROPOSED QUARRY EXTRACTION

The dolostone bedrock would be excavated to approximately the base of the Amabel Formation, at an approximate elevation of 247 – 250 m. As presently proposed, the dolostone would be extracted in a single bench ranging in height from 35 to 38 m. The quarry is anticipated to operate throughout the year with an annual maximum production of 3 million tonnes.

Bedrock excavation within the proposed Flamborough quarry would commence with a 16 m to 20 m deep sinking cut in the east quadrant of the quarry, shown adjacent to Area 1 on Figure 2. Once the area had been opened up and a ramp established a second sinking cut would be taken out to the final floor elevation of the quarry.

5.0 GEOTECHNICAL DATA

5.1 Lithology

The Amabel is described in the previous borehole logs and in the geotechnical report as light brownish grey, fine crystalline, thin to thickly bedded, fossiliferous dolostone in the upper part (above elevation 270 masl) and light to dark bluish grey, fine crystalline, medium to massive bedded, fossiliferous dolostone in the lower part of the formation.

5.2 Rock Quality

The Rock Quality Designation (RQD) shown on the borehole logs ranges from 0% to 100% with an average RQD value of 81%. In general the RQD values are good to very good with typical RQD values in the range from 80% to 100% with some localized poor to very poor zones (RQD as low as 0%) in each of the boreholes.

5.3 Major discontinuities

The major discontinuities were determined from acoustic televiewer data collected in 8 of the boreholes. The acoustic televiewer produces images of the borehole wall that are based on the amplitude and travel time of acoustic beams reflected from the formation wall. When an acoustic beam is transmitted, part of the energy is lost in voids or fractures, producing dark bands on the amplitude log. Travel time measurements allow a reconstruction of the borehole shape, essentially producing a three-dimensional caliper representation of the borehole. Planar features that intersect the borehole wall, such as joints and bedding, produce sinusoidal traces in the "unwrapped" televiewer image. Using the reference direction recorded during logging, sinusoids can be analyzed to produce dip and dip directions of structural features.

Based on the interpreted televiewer data, a stereoplot of the joint orientations was prepared as shown on Figure 3 which is representative of the discontinuities encountered across the site in the boreholes. It should be noted that all of the boreholes were vertical and as such vertical to sub-vertical discontinuities may be under-represented in the stereoplots. In order to compensate for this a Terzaghi correction has been applied to the data. The Terzaghi correction applies a weighting to the discontinuity data such that joints with a dip direction closest to the direction of the borehole receive more weighting than those with a dip direction at right angles to the borehole. Based on the joint data shown on Figure 3, there are at least four main joint sets as well as sub-horizontal bedding joints. Joint set J1 is a low angle joint set assumed to be bedding joints, joint set J2 dips to the NNE at between 40 and 80 degrees; joint set J3 dips to the NNW at between 40 and 80 degrees; joint set J4 dips to the west at between 40 and 60 degrees and joint set J5 dips to the SSE at 40 to 50 degrees. No information is available from the borehole logs on the roughness, shape or infilling of the joints. For the purpose of this assessment it has been assumed, based on prior experience, that the joints are planar and slightly rough to smooth. The actual joint conditions will need to be confirmed during subsequent mapping of the interim or temporary excavations.

6.0 STABILITY ASSESSMENT OF THE PROPOSED BENCHES

6.1 Slope Design Definitions

A pit slope has three major components: bench configuration, inter-ramp slope and overall slope, as illustrated on Figure 4. The bench configuration is defined by vertical bench separation (or bench height), catch bench width (or berm width) and bench face angle (or batter). The inter-ramp slope is formed by a series of uninterrupted benches and the overall slope is formed by a series of inter-ramp slopes separated by haul roads.

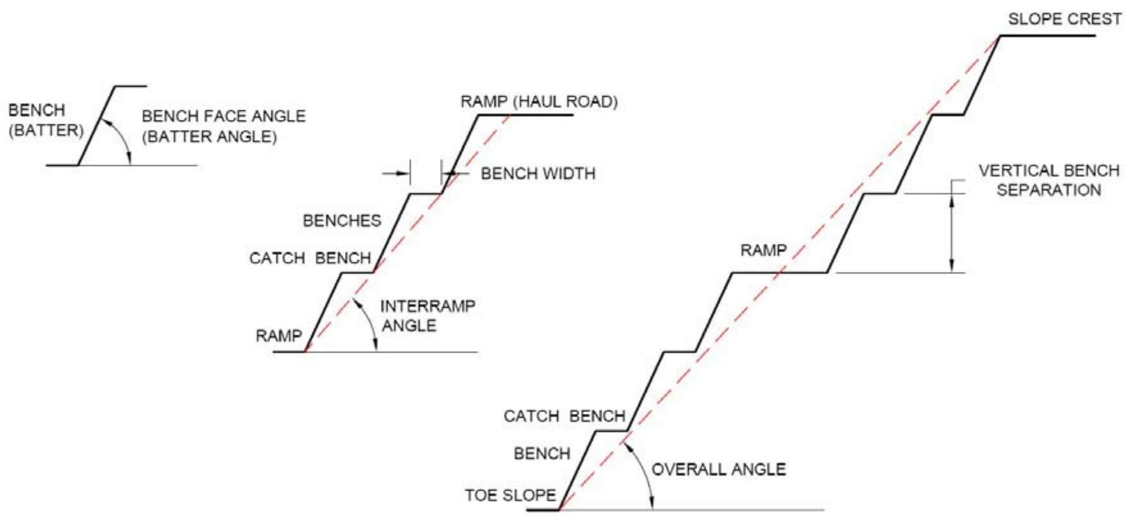

Figure 4: Schematic representation of bench face angle (BFA) and inter-ramp angle (IRA).

6.2 Mechanisms of Instability

The stability of the quarry benches will largely be governed by the rock mass structure and on a more localized scale by ongoing weathering processes such as ice jacking due to freeze thaw cycles in the winter months. Rock mass structure refers to the nature and occurrence of discontinuities within a defined rock mass. The spacing, orientation and continuity (persistence) of discontinuities govern the size and shape of blocks within the rock mass. Discontinuities refer to all fractures occurring within the rock mass, including; joints, bedding planes and faults. These discontinuities represent surfaces of weakness, the larger and more closely spaced they are the more influential they become in reducing the effective strength of the rock mass. The discontinuity wall roughness, aperture size, infilling materials, and water conditions define the nature of the discontinuity – these factors control the shear resistance along the discontinuity.

6.2.1 Structurally Controlled Failure Mechanisms

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

A **plane failure** may occur when a geological discontinuity dips out of a rock slope at an angle that is shallower than the inclination of the slope and steeper than the effective angle of friction on the discontinuity. Plane failures will generally only develop to a significant extent if the strike of the geologic discontinuity is within ***±20°*** of the strike of the rock slope.

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

Toppling failures may develop when a rock mass contains multiple, parallel, steeply dipping continuous geologic structures, such as faults or continuous joint/foliation planes, that strike nearly parallel to the strike of the face of the rock slope. Toppling failure will generally only develop when the strike of the structures is within ***±20°*** of the azimuth of the slope face. Kinematically, the potential for toppling failure is determined by the spacing (separation), inclination and continuity of the toppling blocks and the slope angle. Wide spacing and/or discontinuous structures will mitigate the potential for toppling. At a bench scale, this failure mechanism is controlled by berm width and/or the inclusion of mid-slope catch berms, both which improve stability by reducing the effective length of the toppling blocks.

All structurally controlled failure modes are aggravated by water pressures within the slope, particularly toppling failures.

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

6.3 Stability Analyses

The joint data from the televiewer surveys has been used in a kinemtic analyses in order to understand the potential failure mechanisms for each of the walls or benches of the proposed quarry. In general, the final quarry faces can be divided into the following four orientations:

North-west faces – faces on the north-west side of the quarry with a 45 degree strike;

North-east faces – faces on the north-east side of the quarry with a 135 degree strike;

South-east faces – faces on the south-east side of the quarry with a 225 degree strike; and

South-west faces - faces on the south-west side of the quarry with a 315 degree strike.

For the analyses of planar failures, each of the joints in the data set was checked for planar sliding assuming a friction angle of approximately 40 degrees. The results are plotted as a cumulative frequency plot of failures (values less than a factor of safety of 1) against the face angle on Figure 5. In general, 14% to 26% of the joints could result in planar failures for a vertical bench face.

A kinematic wedge analysis was also carried out in a similar manner using combinations (pairs) of all of the joints in the data set to check for potential wedges. The results of the wedge analysis indicate that between 1% and 4% of the joint combinations could form unstable wedges. Given the relatively low percentage of wedge failures, this type of failure mechanism is considered unlikely to affect the overall bench stability but this will need to be confirmed by mapping the exposed joints in the interim walls during initial development.

Toppling failure is not expected to affect the overall bench face stability but could affect localized areas of the face.

7.0 CONCLUSIONS AND RECOMMENDATIONS

Based on the kinematic analyses carried out using the borehole televiewer data, there is a risk of both planar and wedge type failures on all four of the main quarry wall orientations. The risk of wedge type failures is considered to be relatively low based on the statistical results, however, the frequency and continuity of the joints forming potential wedges will need to be confirmed by mapping of the exposed joints in the interim benches (i.e. during the sinking cuts) during initial development.

The risk of planar failures is a greater concern for the overall stability of the quarry faces, particularly if the walls are excavated full face (i.e. a single bench face) to a height of approximately 38 m. In this case rather large planar slides involving hundreds of cubic metres of rock are theoretically possible. The size and therefore impact of any planar type failures will depend largely on the continuity of the joints forming the planar sliding surfaces (i.e. for these planar failures to affect the overall face they would need to have a continuity of 30 m or more). At present there is no data on the continuity of the joints. Therefore, geotechnical mapping of the exposed interim bench faces, for example during the first initial sinking cut, will be required to make an assessment of the likelihood of encountering joints which would be continuous enough to affect the overall wall stability. If continuous joints are noted in the initial sinking cut, then a benched final face will be required on some or all of the final walls to decrease the size and likelihood of large (full wall height) failures. The walls requiring benching and the final bench design will depend on the orientation of the continuous joints.

Alternatively, if the quarry excavation is designed with between 15 to 20 m high bench faces and a 6 m to 8 catch bench width (i.e. an effective inter-ramp angle of approximately 65 to 75 degrees) the probability of overall, large scale planar failures will decrease significantly (approximately 50% reduction compared to the full face excavation based on the kinematic analysis of the borehole data). The catch bench will also serve to contain small wedge and planar failures from the upper bench as well as ravelling type failures due to ongoing weathering and blast damage. There will still be a risk of smaller scale wedge and planar failures at a bench scale, however, this is the case for most quarry faces where there are inclined joints present. These bench scale instabilities would need to be addressed through careful inspections and scaling after blasting.

Stability of the final bench faces can be improved through the use of good quality controlled blasting methods and thorough scaling of loose rock from the bench faces after blasting.

GOLDER ASSOCIATES LTD.

LICENSED PROFESSIONAL ENGINEER
M. J. TELESNICKI
PROVINCE OF ONTARIO

Mark Telesnicki, P. Eng.
Principal

Joe Carvalho, P.Eng.
Principal

MJT/JLC /ms

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FIGURES

Golder Associates Mississauga, Ontario, Canada	SCALE: AS SHOWN	TITLE	
	DATE: July 23, 2008	**PROPOSED QUARRY** **SITE LOCATION PLAN**	
	DESIGN: MVB		
	CAD: JFC		
FILE No. 0811170009AB001.dwg	CHECK: MVB		FIGURE
PROJECT No. 08-1117-0009 / REV. A	REVIEW: MVB		1

PLOT DATE: August 11, 2008
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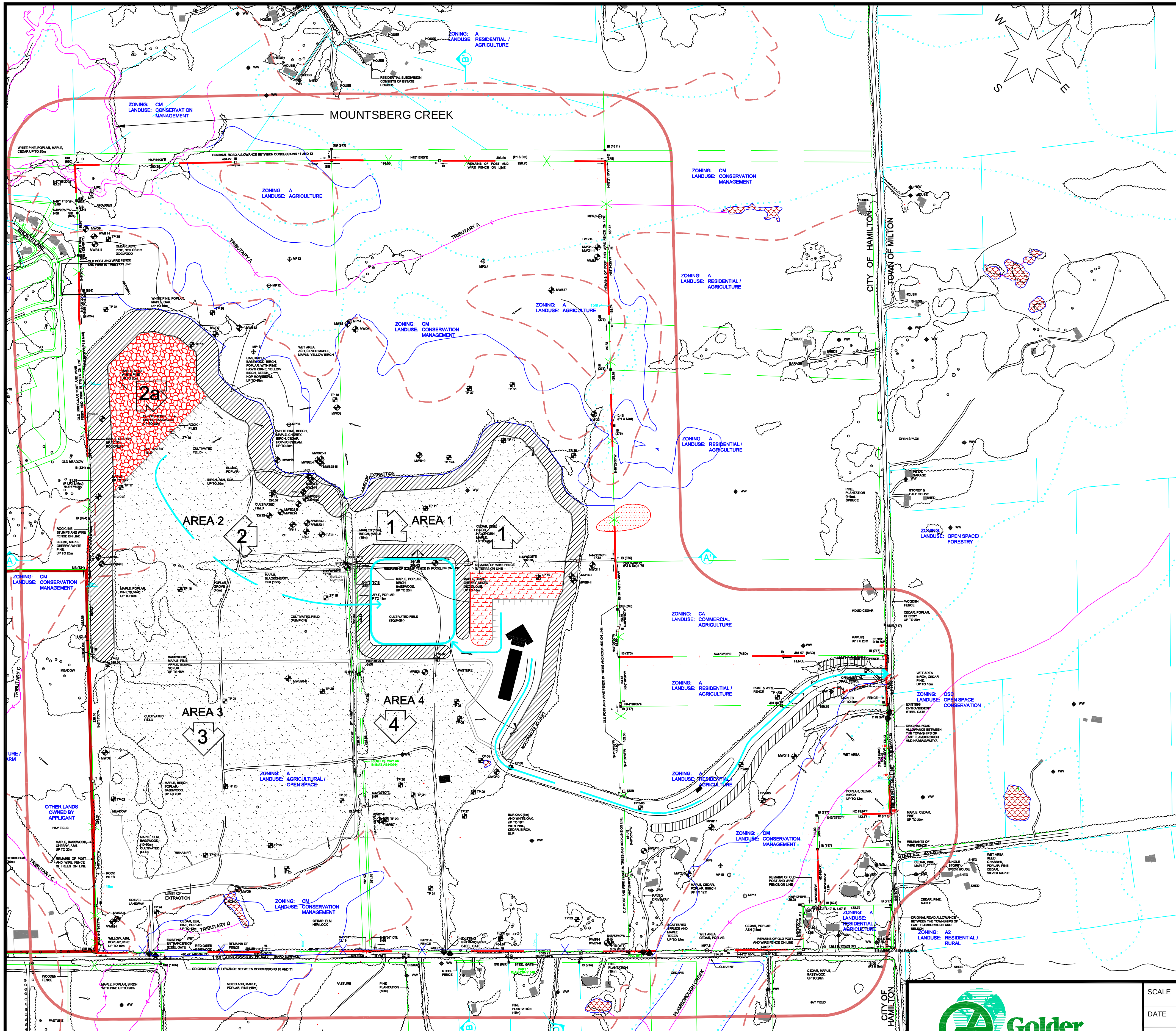


LEGEND:

- BOUNDARY OF AREA TO BE LICENCED
- BOUNDARY OF AREA TO BE EXCAVATED
- LIMIT OF DEVELOPMENT
- LIMIT OF EXTRACTION
- REGULATORY SETBACK LINE
- 120m INFORMATION BOUNDARY
- PROPERTY / LOT/ CONCESSION LINE
- ZONING / LANDUSE BOUNDARY
- PROVINCIALLY SIGNIFICANT WETLAND (LOWER MOUNTSBERG CREEK SWAMP COMPLEX)
- 345.00 — EXISTING 5m CONTOUR LINE
- 344.00 — EXISTING 1m CONTOUR LINE
- X359.73 — EXISTING SPOT ELEVATION
- EXISTING FENCE
- EXISTING TRIBUTARY / RIVER
- EXISTING TRAIL
- EXISTING LANEWAY
- EXISTING VEGETATION
- EXISTING POND
- DIRECTION OF SURFACE WATER DRAINAGE
- EXISTING BUILDING
- TP# — TEST PIT # PREPARED BY: (ZOLTON KATONA), NOV.13, 2003
- BH# ELEV. (2002) — BOREHOLE #, ELEVATION, DATE DRILLED (JEGEL CONSULTING ENGINEERS)
- TW # ELEV. — PRODUCTION WELL, ELEVATION
- WW — WATER WELL

150 0 150 300

SCALE 1:7500 METRES

REFERENCES:

1. BASE MAPPING BASED ON DIGITAL FILE PROVIDED BY HARRINGTON AND HOYLE LTD. LANDSCAPERS AND ARCHITECTS, TITLED "OPERATIONS PLAN" DRAWING No. 2 OF 5, PROJECT No. 06-25, RECEIVED NOVEMBER 2008.

Golder Associates
Mississauga, Ontario, Canada

SCALE	AS SHOWN
DATE	Nov. 24, 2008
DESIGN	
CAD	JFC
CHECK	MVB
REVIEW	MVB
FILE No.	081117009AD002.dwg
PROJECT No.	08-1117-0009
REV.	A

TITLE

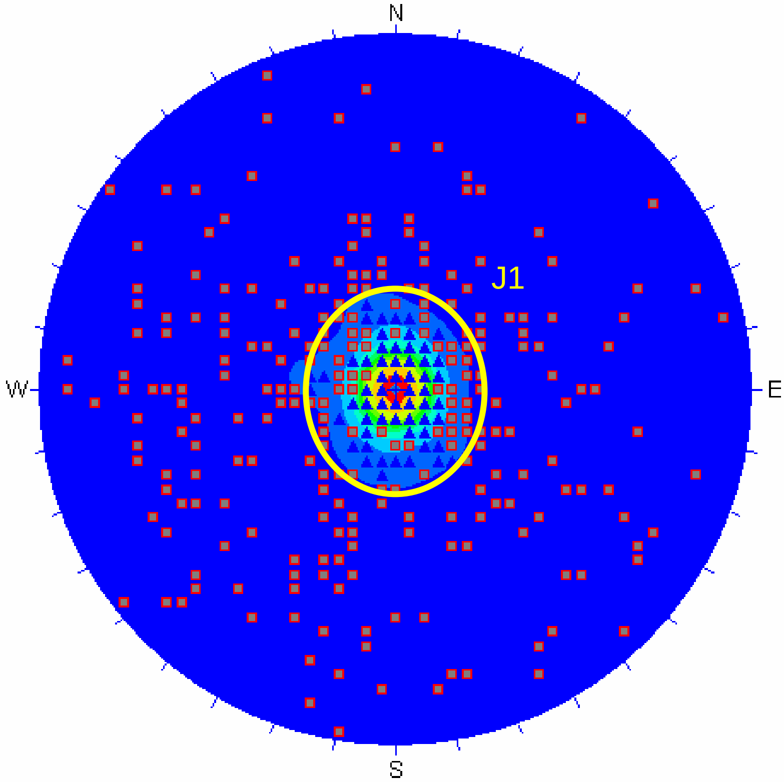
PROPOSED FLAMBOROUGH QUARRY OPERATIONAL PLAN

FLAMBOROUGH QUARRY

FIGURE 2

PLOT DATE: November 26, 2008
FILENAME: T:\Projects\2008\08-1117-0009 (LOWNDES, Flamborough)\-AD-\081117009AD002.dwg

Figure 3A - Contoured Stereoplot of all poles

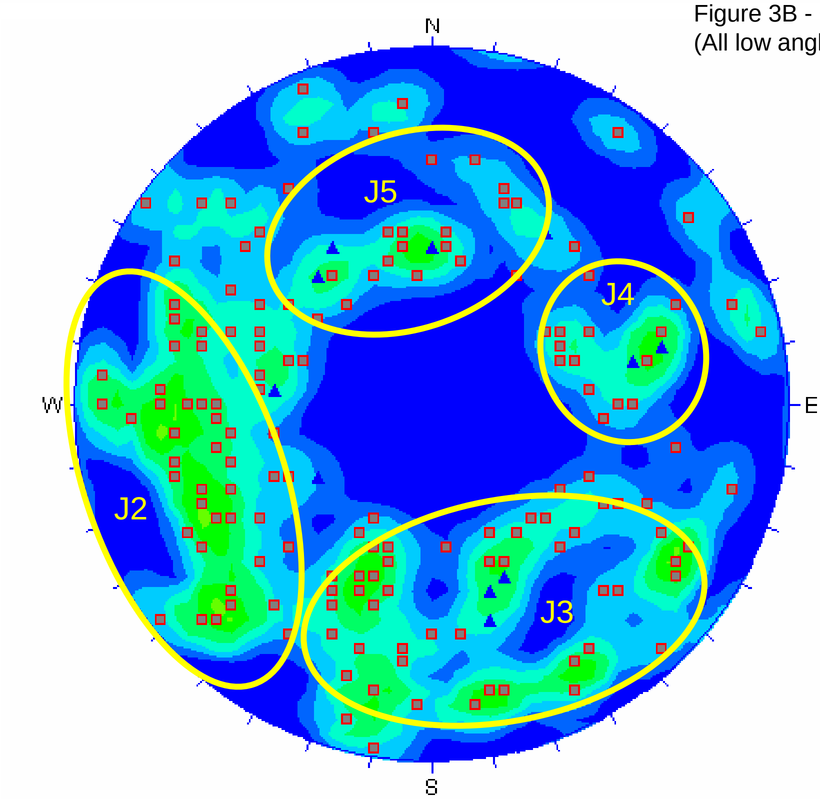


Number of Poles

- 1 pole
- 2 to 13 poles
- 14 to 25 poles
- 26 to 37 poles
- 38 to 49 poles
- 50 to 61 poles
- 62 to 73 poles
- 74 to 85 poles
- 86 to 97 poles
- 98 to 109 poles

Equal Area
Lower Hemisphere
592 Poles
592 Entries

Figure 3B - Contoured Stereoplot of all poles >30 deg. Dip
(All low angle joints excluded)



Number of Poles

- 1 pole
- 2 poles

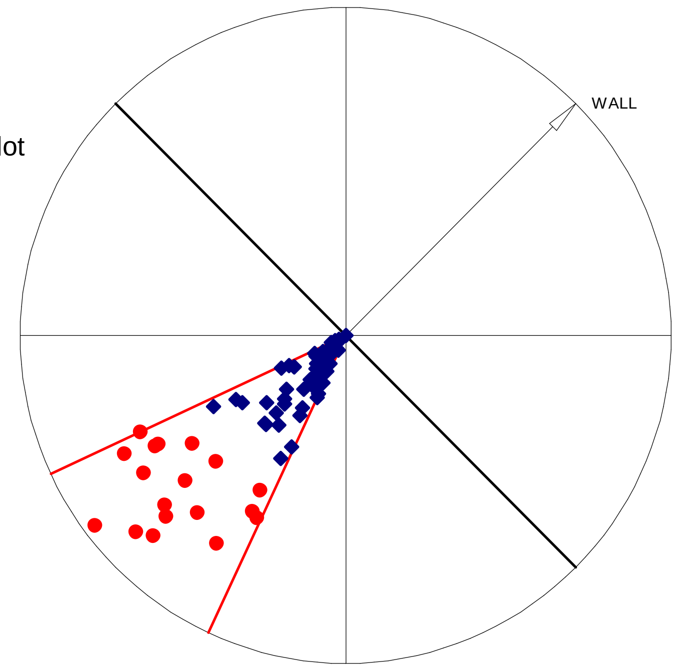
Equal Area
Lower Hemisphere
173 Poles
173 Entries

Stereoplot of the Acoustic Televiewer Data	FIGURE 3
Golder Associates	08-1117-0009
	Oct. 2. 2008

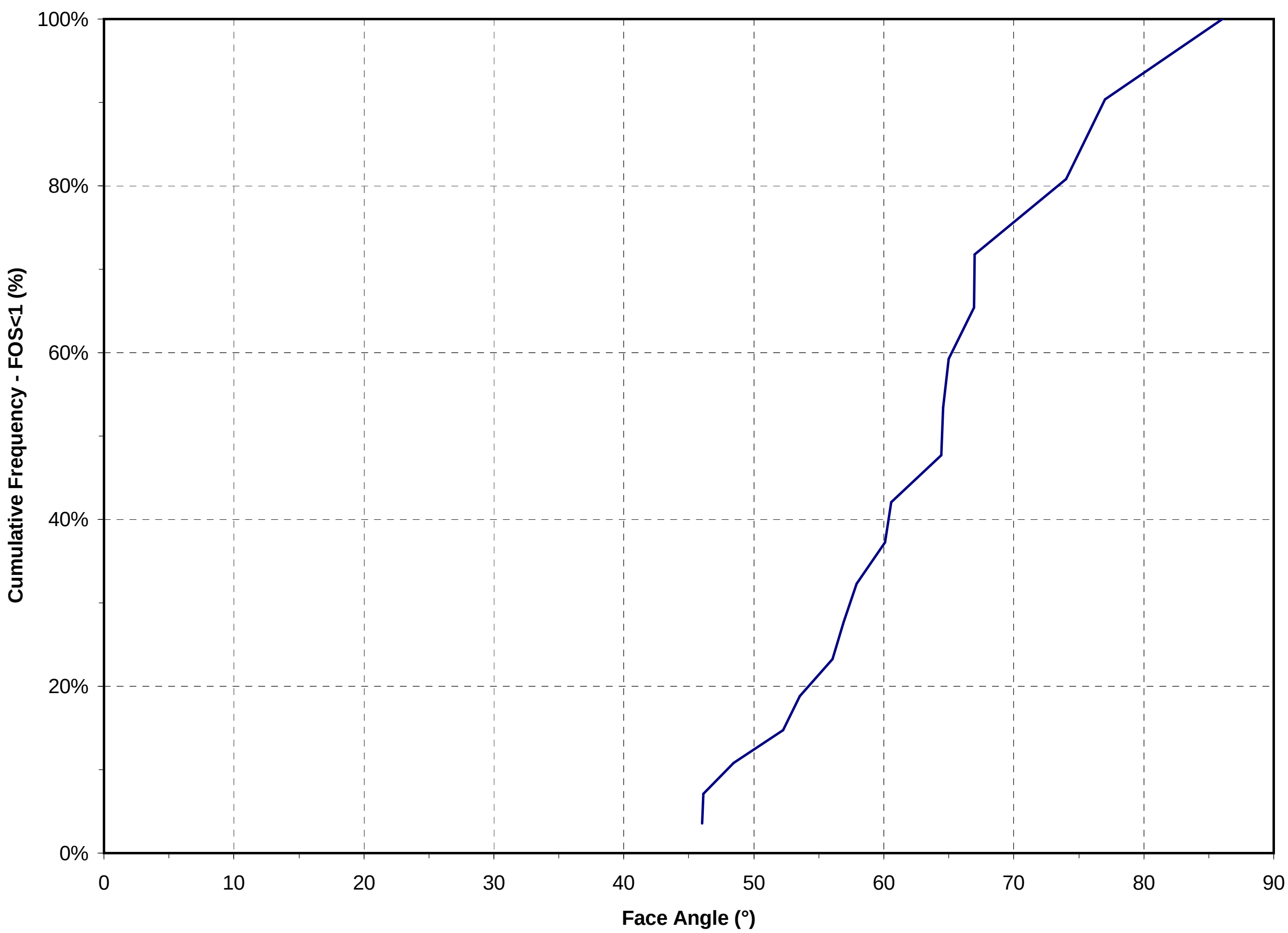
APPENDIX A
KINEMATIC ANALYSES RESULTS

PLANAR BENCH ANALYSIS	FIGURE A1

PIT WALL PARAMETERS:
Slope Direction: 45 °
Kinematic Window: 40 °
Friction Angle (ϕ): 40 °
69 Planes included in the window analysis
18 Failed Planes in the Cumulative Frequency Plot
26% Failure
Quantity: FALSE
Terzaghi Weight: TRUE



EQUAL AREA STEREONET

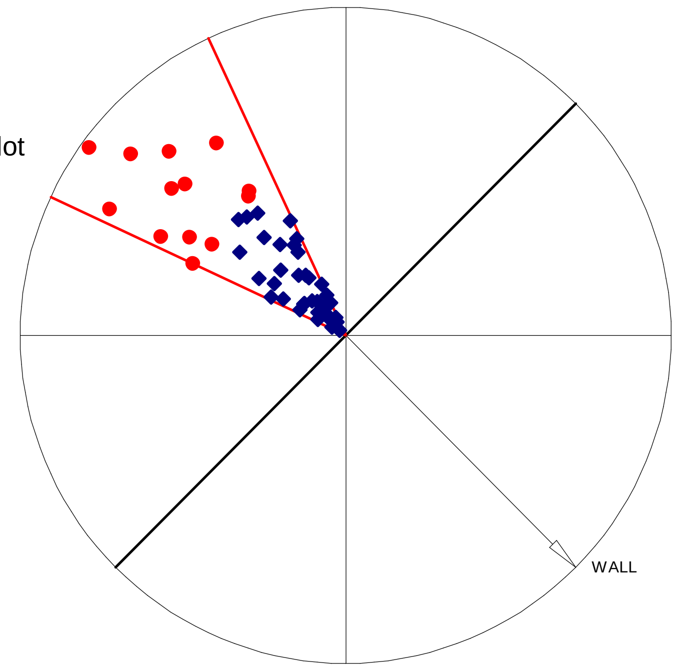


DATE: October-2008 **Golder Associates** Project No. 08-1117-0009

PLANAR BENCH ANALYSIS	FIGURE A2

PIT WALL PARAMETERS:

Slope Direction:		135 °
Kinematic Window:		40 °
Friction Angle (ϕ):		40 °
52	Planes included in the window analysis	
13	Failed Planes in the Cumulative Frequency Plot	
25%	Failure	
Quantity:	FALSE	
Terzaghi Weight:	TRUE	



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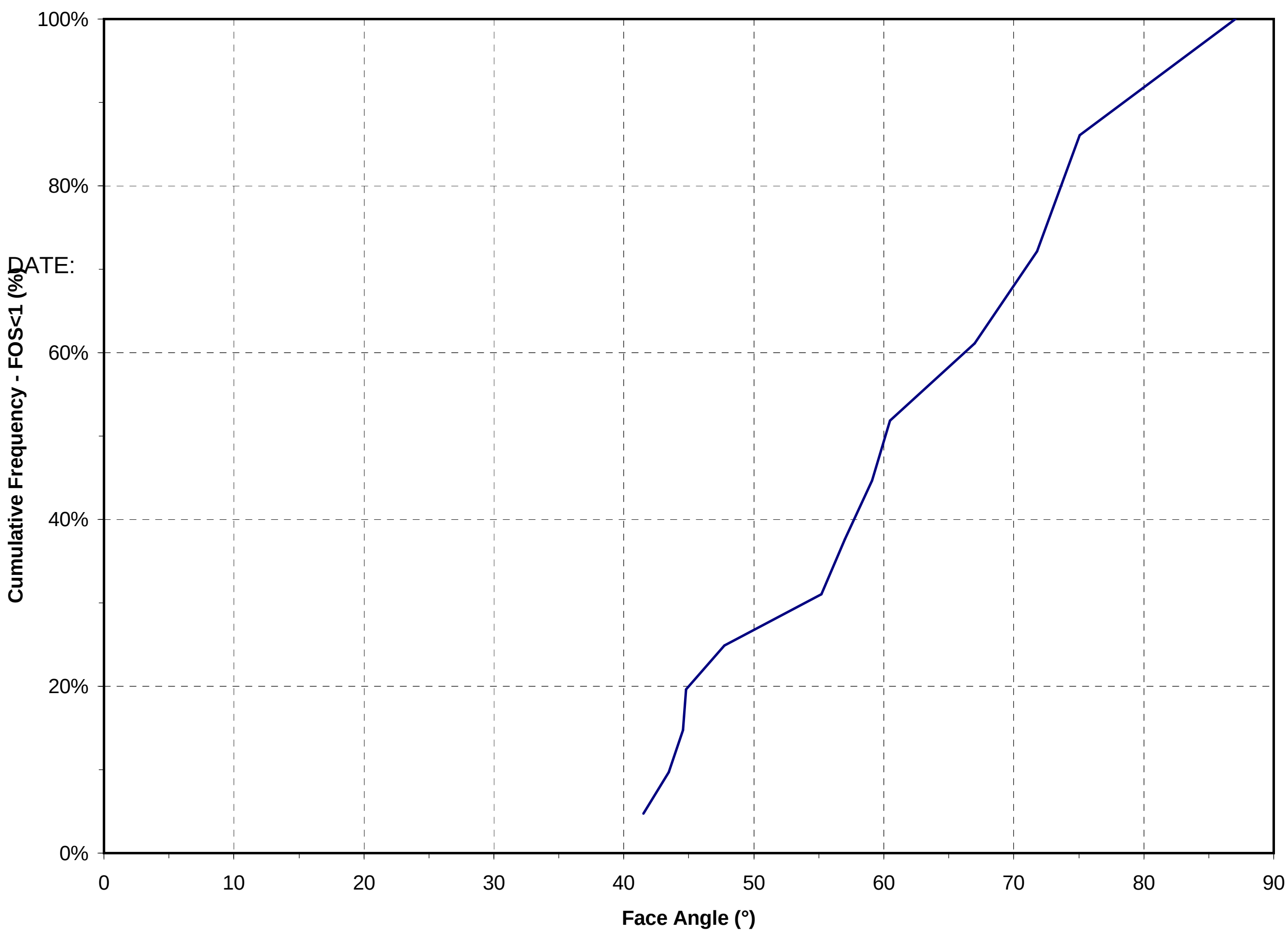


DATE:

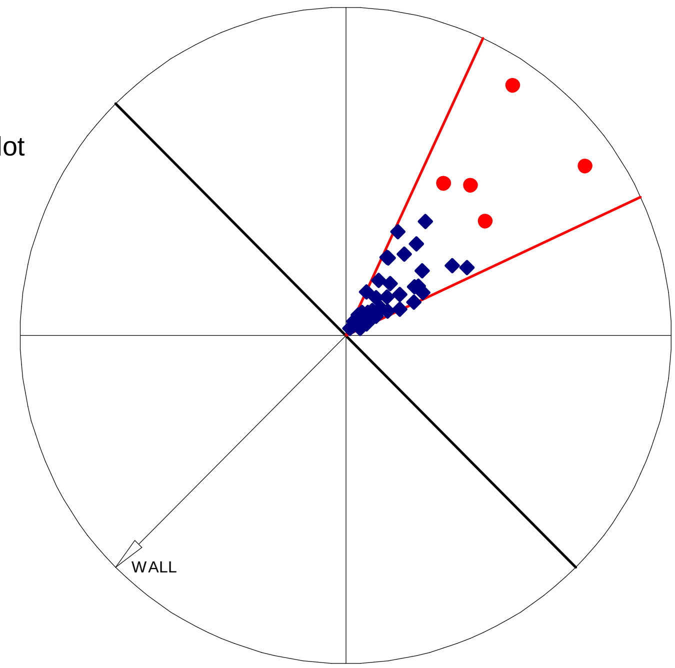
October-2008

Golder Associates

Project No. 08-1117-0009

PLANAR BENCH ANALYSIS	FIGURE A3

PIT WALL PARAMETERS:
Slope Direction: 225 °
Kinematic Window: 40 °
Friction Angle (ϕ): 40 °
43 Planes included in the window analysis
6 Failed Planes in the Cumulative Frequency Plot
14% Failure
Quantity: FALSE
Terzaghi Weight: TRUE



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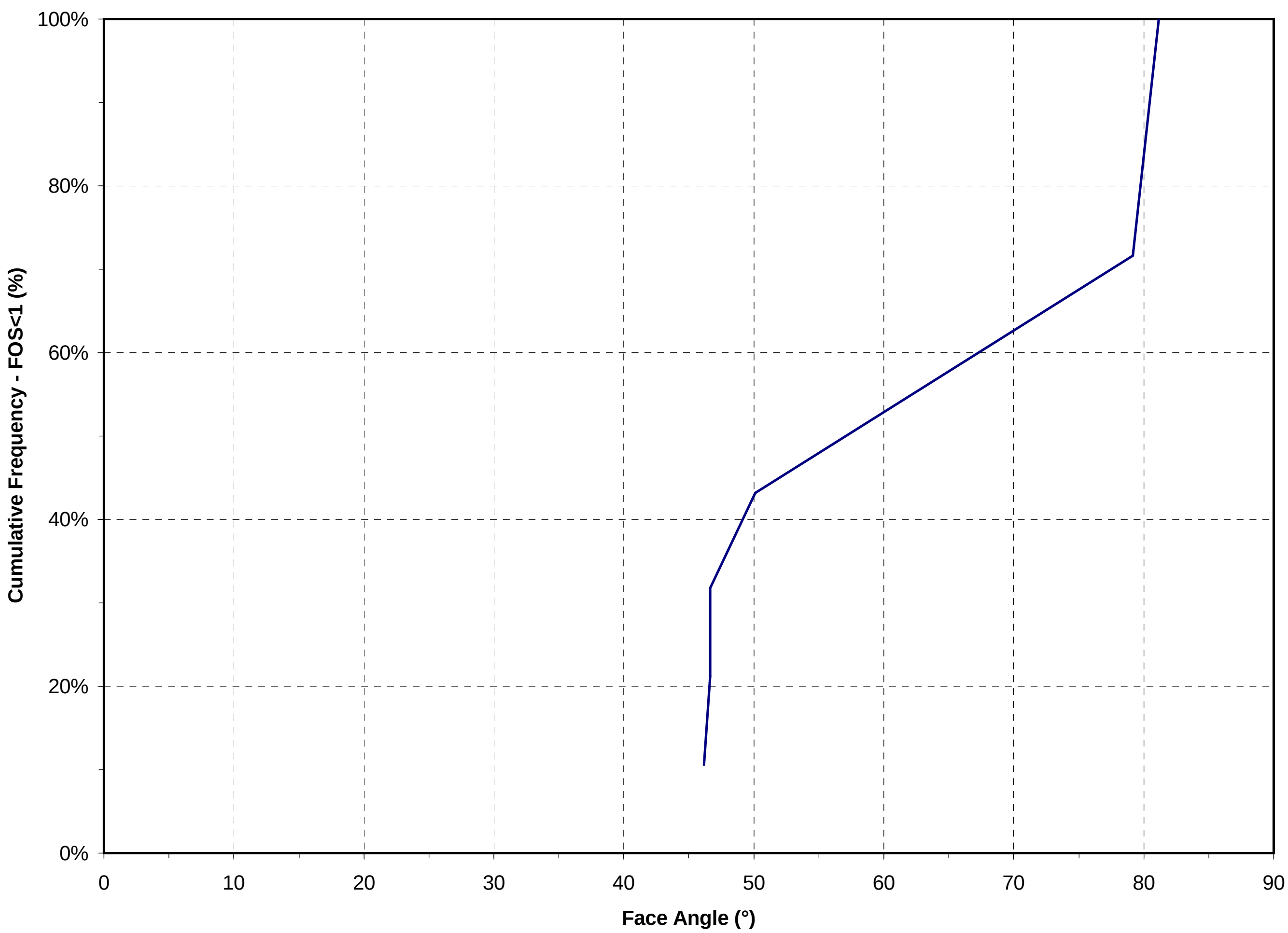
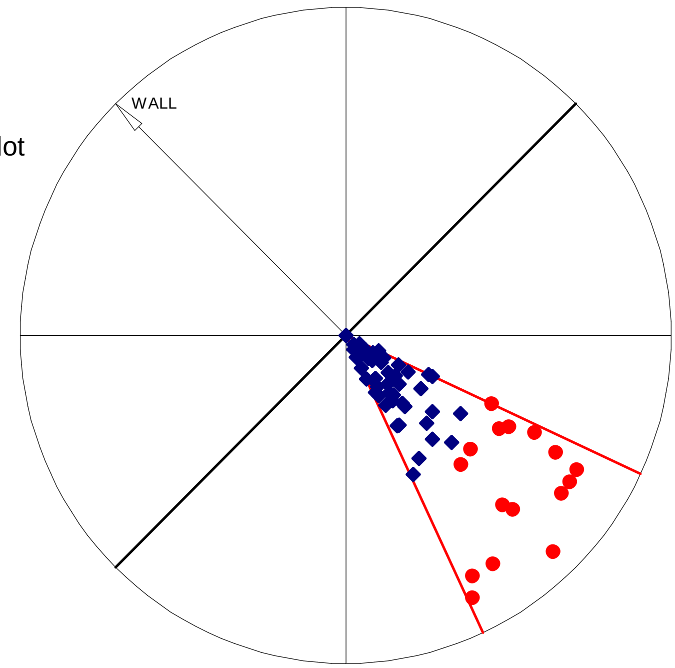


PLANAR BENCH ANALYSIS	FIGURE A4

PIT WALL PARAMETERS:
Slope Direction: 315 °
Kinematic Window: 40 °
Friction Angle (ϕ): 40 °
62 Planes included in the window analysis
16 Failed Planes in the Cumulative Frequency Plot
26% Failure
Quantity: FALSE
Terzaghi Weight: TRUE



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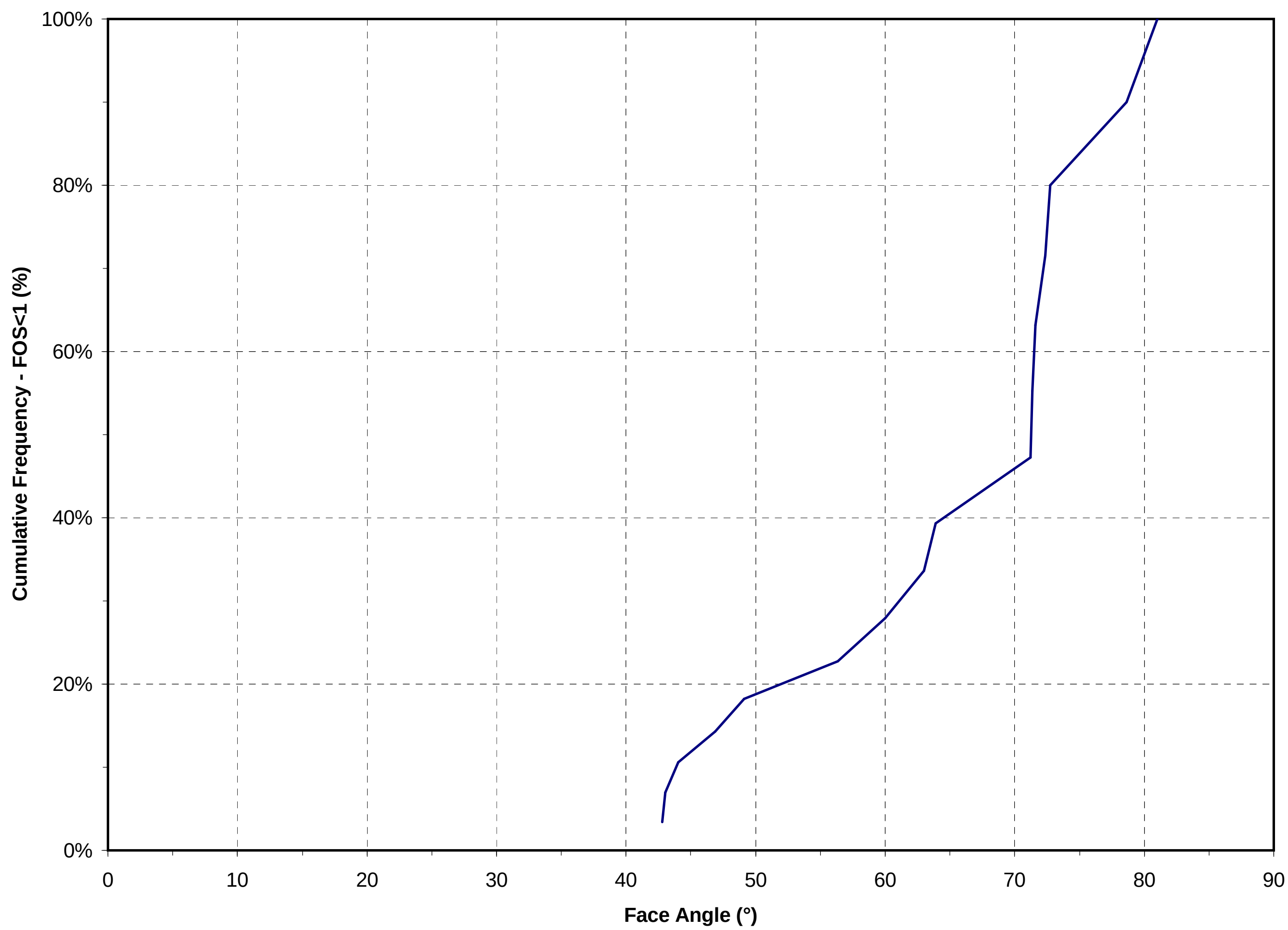


DATE: October-2008 **Golder Associates** Project No. 08-1117-0009

WEDGE BENCH ANALYSIS

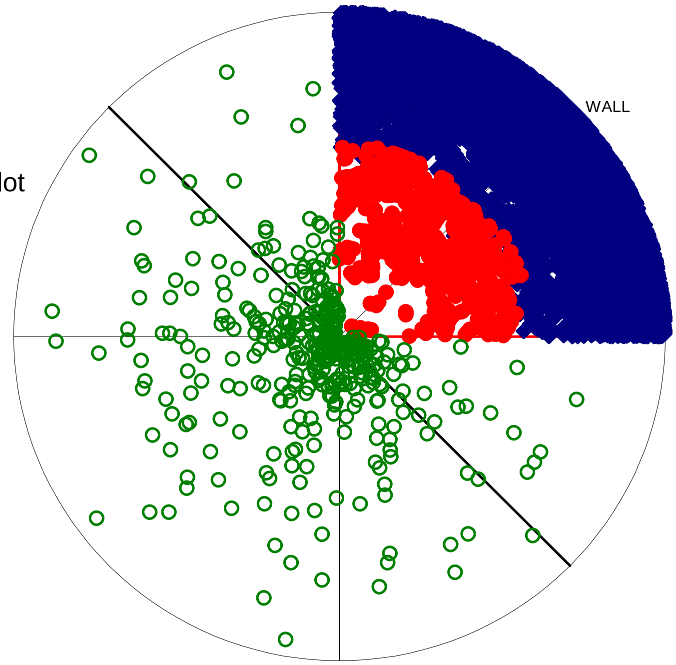
FIGURE A5

PIT WALL PARAMETERS:

Slope Direction:	45 °
Kinematic Window:	90 °
Friction Angle (ϕ):	40 °

21197 Wedges included in the window analysis
833 Failed Wedges in the Cumulative Frequency Plot
4% Failure

Quantity:	FALSE
Terzaghi Weight:	TRUE



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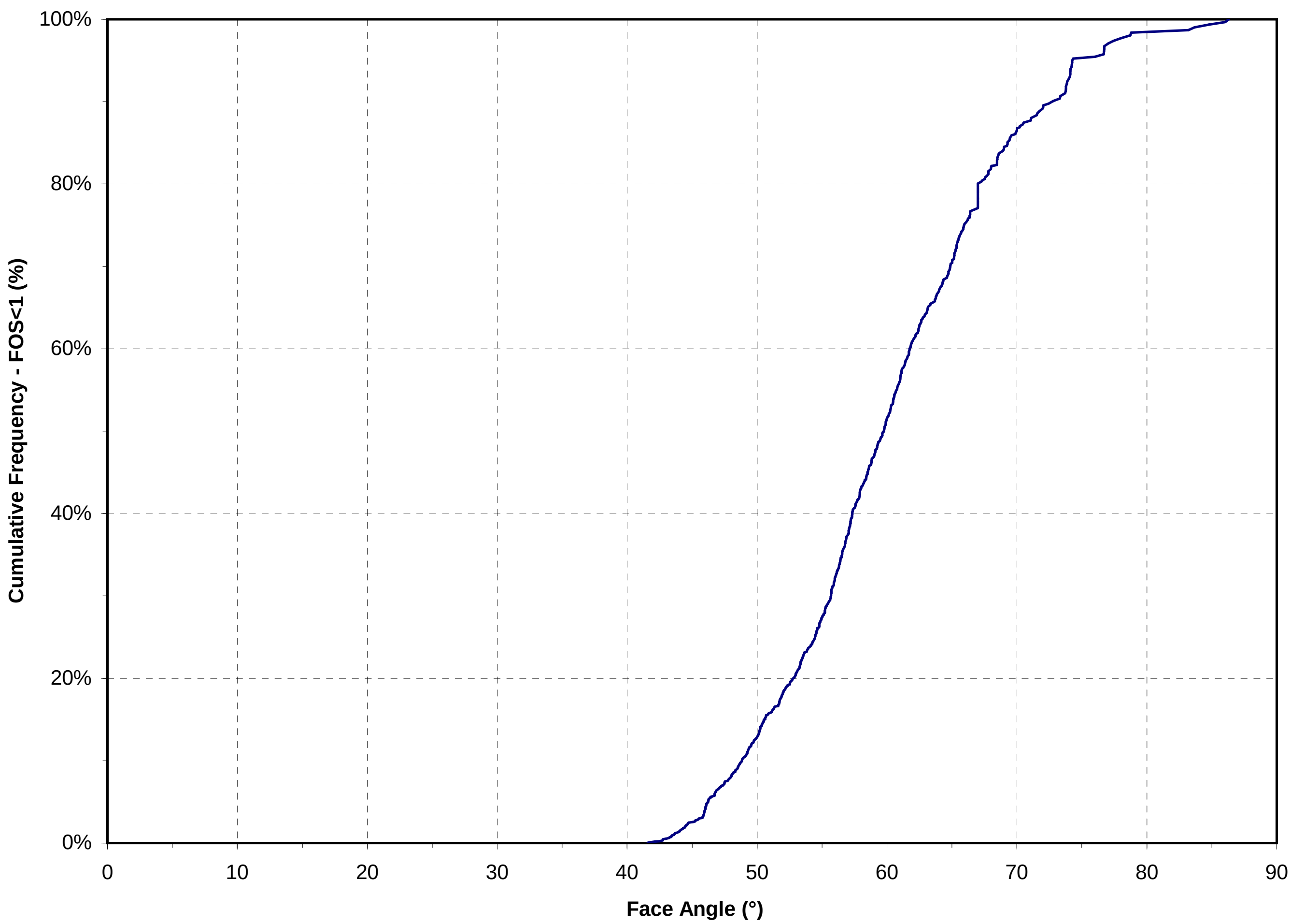


WEDGE BENCH ANALYSIS	FIGURE A6

<u>PIT WALL PARAMETERS:</u>

Slope Direction: 135 °

Kinematic Window: 90 °

Friction Angle (ϕ): 40 °

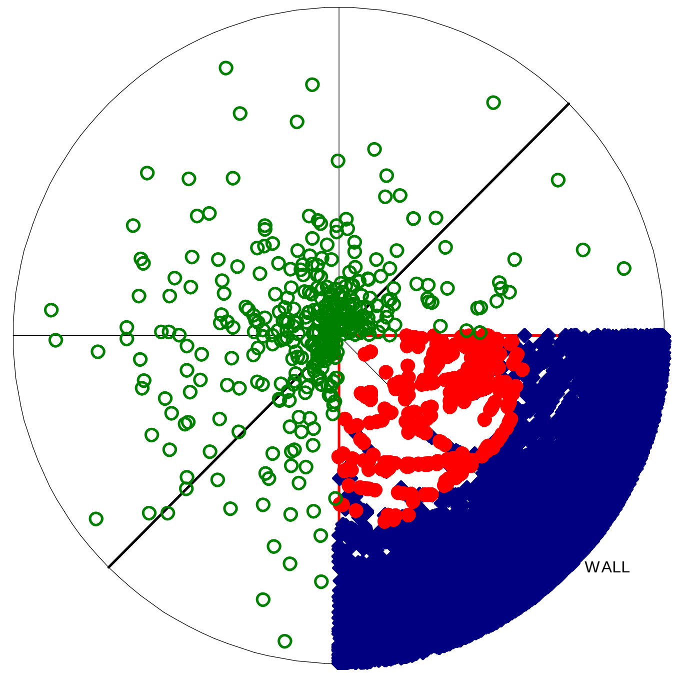
20568

359

2% Failure

Quantity:

Terzaghi Weight:



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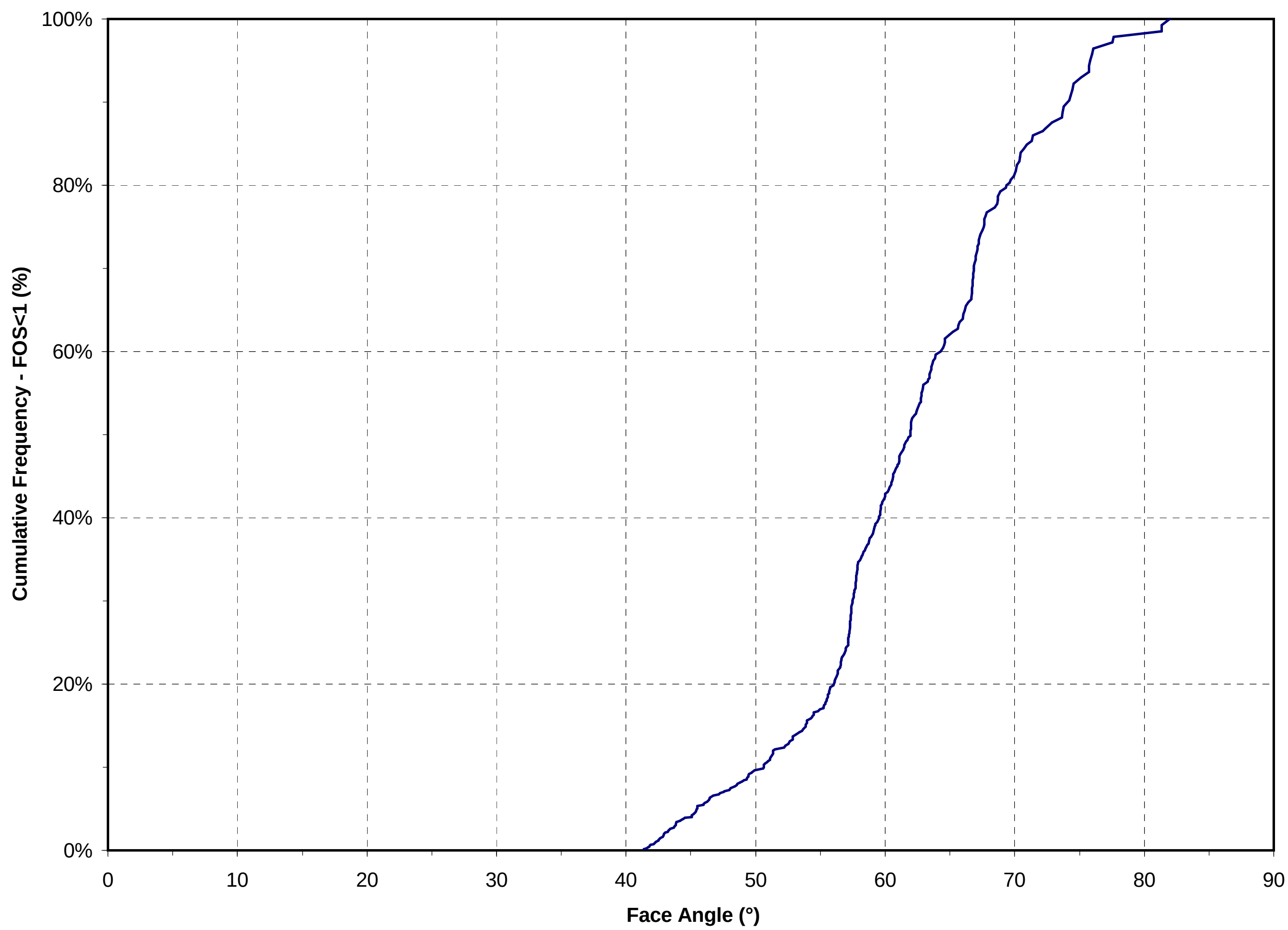


WEDGE BENCH ANALYSIS	FIGURE A7

PIT WALL PARAMETERS:

Slope Direction: 225 °

Kinematic Window: 90 °

Friction Angle (ϕ): 40 °

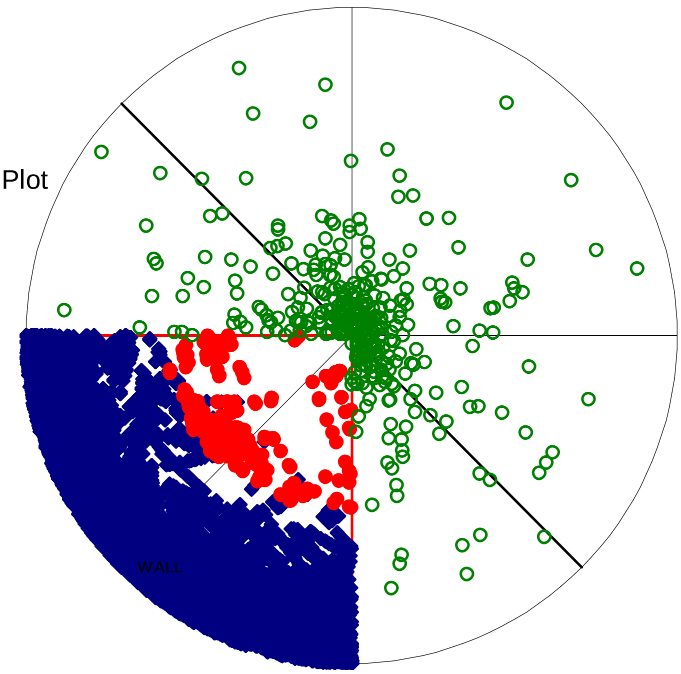
17134 Wedges included in the window analysis

199 Failed Wedges in the Cumulative Frequency Plot

1% Failure

Quantity: FALSE

Terzaghi Weight: TRUE



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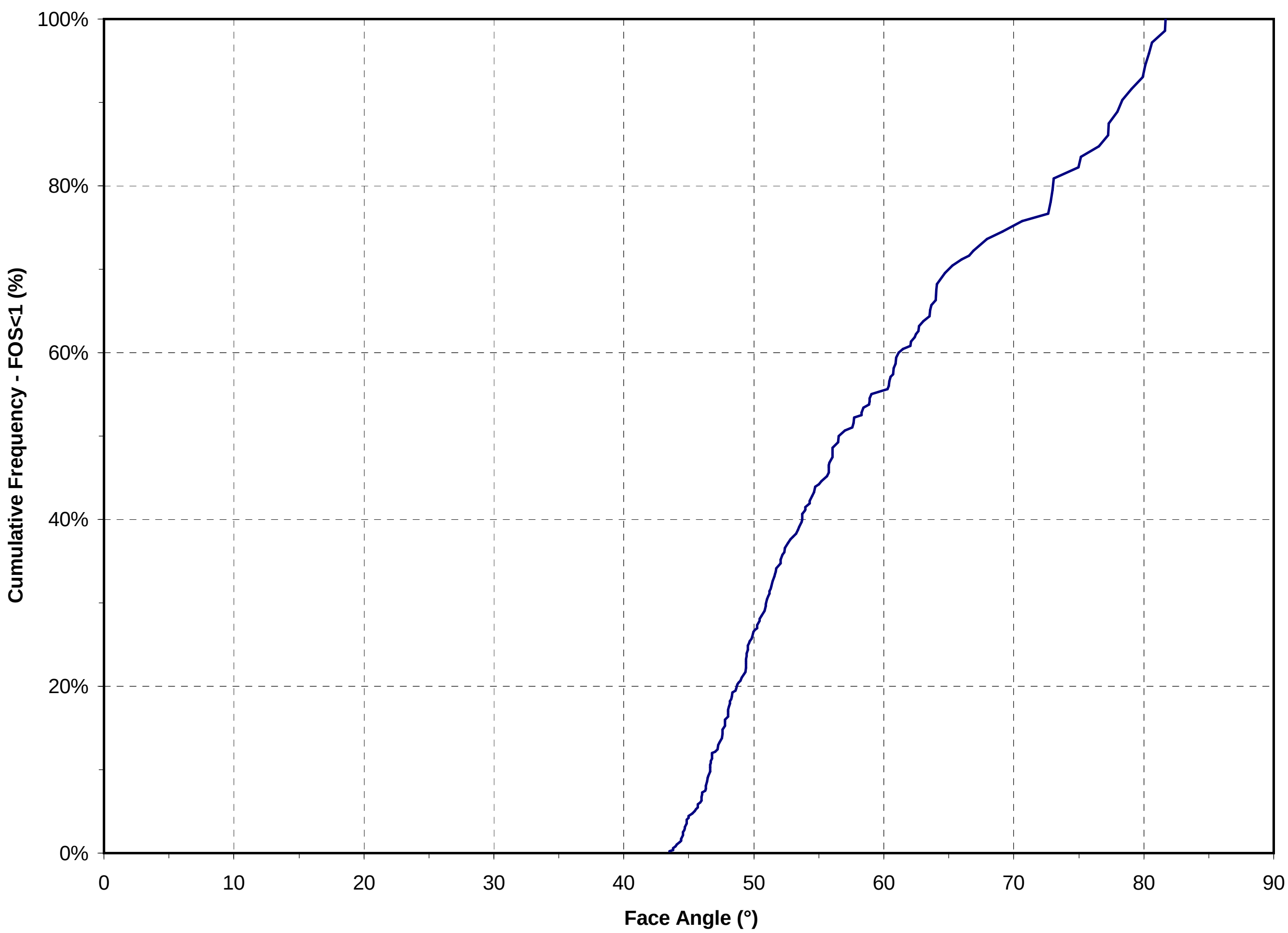


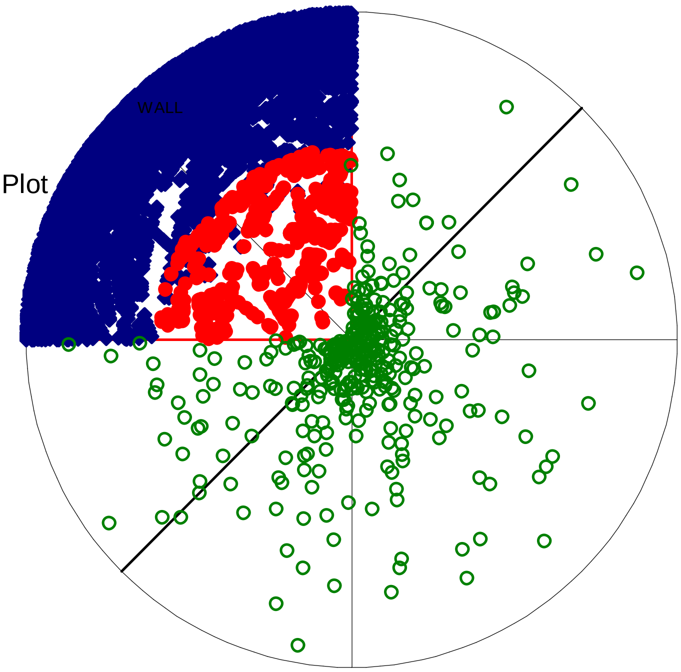
WEDGE BENCH ANALYSIS	FIGURE A8

PIT WALL PARAMETERS:

Slope Direction:	315 °
Kinematic Window:	90 °
Friction Angle (ϕ):	40 °

16563 Wedges included in the window analysis
419 Failed Wedges in the Cumulative Frequency Plot
3% Failure

Quantity:	FALSE
Terzaghi Weight:	TRUE



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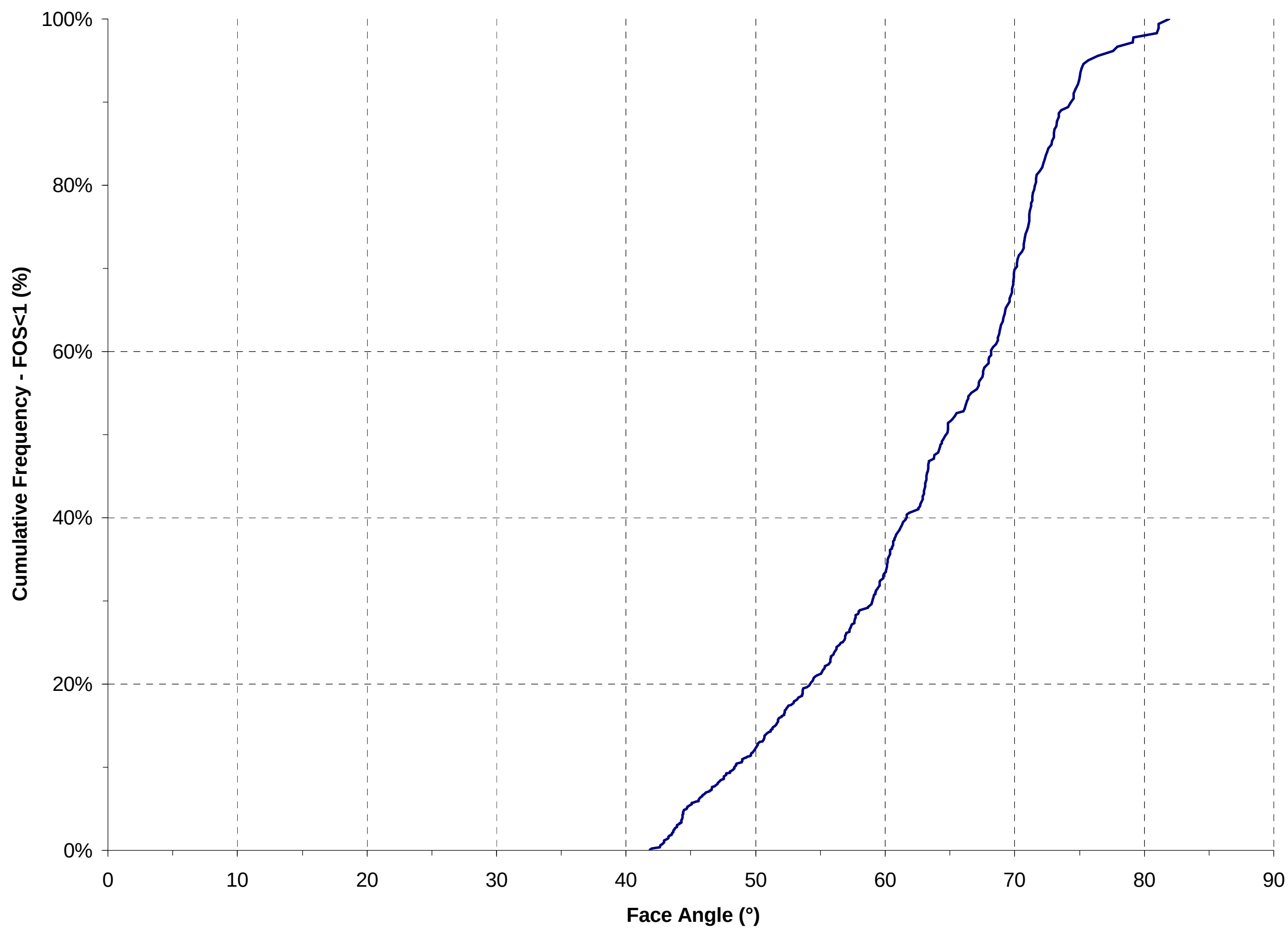

APPENDIX B
CURRICULUM VITAE, MARK TELESNICKI, P.ENG.

Mark J. Telesnicki

Education B.A.Sc., Geological Engineering, University of Waterloo, 1987.

Affiliations Registered Professional Engineer, Ontario.
Member, Tunnelling Association of Canada
Editor – Tunnelling Association of Canada Journal.

EXPERIENCE

2001 to Present **Golder Associates Ltd.** **Mississauga, Ontario**

Rock Engineering Group Manager, Principal (2006).

Responsible for managing the Rock Engineering Group in the Mississauga office, which consists of approximately 20 professional and support staff members. Primary responsibilities include: management of staff, preparation of budgets and business plans, senior technical review and project management of rock mechanics projects in Canada and abroad.

1989 to 2000 **Golder Associates Ltd.** **Mississauga, Ontario**

Project Engineer, then Associate.

Responsible for rock mechanics aspects of mining and civil engineering projects. Primarily engaged in project management, supervision of site investigations, numerical analysis, engineering design, construction supervision and technical specification preparation. Experience includes:

- detailed surface and subsurface investigations including diamond drilling, hydraulic conductivity testing, geological mapping and rock mass characterization
- rock slope investigations including stability analysis and rock reinforcement design for highway rock cuts, open pit mines, and natural slopes
- underground rock support design for a variety of mining and civil works including remediation of large underground rockfalls and support of difficult ground conditions
- instrumentation of rock slopes and underground openings for monitoring water pressures, rock movement and loads and time dependant deformations.

1996 **Golder Associates Ltd.** **Himachal Pradesh, India**

Resident Chief Engineering Geologist - Nathpa Jhakri Hydroelectric Project.

Responsible for supervision of the Engineering Geology Group of the construction joint venture. Duties included supervision of the day to day work of the project engineering geologists and technicians. Responsible for rock mass characterization, design of rock reinforcement for temporary works, evaluation of rock excavation and support drawings for tunnel works and slopes, underground blast design, selection of tunnelling methods in difficult ground conditions, co-ordinating approvals and maintaining geologic records.

1987-1989 **Golder Associates Ltd.** **Mississauga, Ontario**

Engineering Geologist.

Responsible for various aspects of geotechnical projects including field investigations and construction supervision. Involved in investigation and remediation of abandoned near surface mine workings; remedial design, cost estimating and construction supervision for reinforcement of rock slopes and rock cuts; detailed geotechnical mapping and rock mass classification for site selection, mine design and numerical modelling.

Mark J. Telesnicki

PROJECT RELATED EXPERIENCE - ROCK SLOPES

Highway Rock Cut Hazard Assessment — Eastern Ontario

Responsible for a preliminary hazard assessment of over 300 highway rock cuts for MTO Eastern Region using the Ontario Rockfall Hazard Rating System (RHRON). All significant rock cuts were logged, photographed as required and assessed a RHRON hazard rating.

Decew Power Generation Station Cliff Face — St. Catharines, Ontario

Responsible for geotechnical investigation and detailed design of remedial measures to stabilize a 60 m high rock face behind a hydro-electric power station. The investigation included detailed geotechnical mapping using rope access techniques (rappelling), and detailed surveying using a laser system. Remedial work included scaling of loose rock, rock bolting, installation of rock fall fences and draped wire mesh.

Ontario Power Generation Station Cliff Face — Niagara Falls, Ontario

Responsible for the geotechnical investigation and detailed design of remedial measures to stabilize a 30 m high rock face behind a hydro-electric power station. The investigation included detailed geotechnical mapping using a crane support basket, in situ stress measurements, diamond core drilling and detailed surveying using a laser system. Remedial work included scaling of loose rock, rock bolting, drainhole installation and cleaning, instrumentation and monitoring.

Upper Goodwood Park Rock Slope Assessment — Trinidad, BWI

Carried out a field assessment of several rock cuts for residential building sites and recommended remedial measures to control large scale planar failures and surface ravelling. Remedial work included roc excavation, bolting, sclaing and draped mesh.

Nathpa Jhakri Hydro Power Plant Dam Site — India

Responsible for review of rock slope stabilization measures for high rock slopes at the dam abutments, intake structures and diversion tunnel portals. Provided input to the design of the remedial works and assisted in the planning of the construction work which included installation of high capacity rock anchors.

Kingston Mills Rock Slope Stabilization — Kingston Mills, Ontario

Detailed geotechnical investigation using rope access techniques and development of remedial measures including rock excavation (using non-explosive methods), scaling and rock bolting.

Kerncliff Park Stabilization — Burlington, Ontario

Detailed design and implementation of rock stabilization program to stabilize the walls of an abandoned quarry for use as a new community park. Remedial measures involved extensive scaling, rock bolting and trim blasting of the walls using rock climbing/rapelling techniques.

Feasibility Study for Widening Tuen Mun Highway — Hong Kong

Field investigation and preliminary design of high cut slopes and natural slopes above an existing 6 lane highway as part of an overall feasibility assessment of alternate schemes for widening the highway by adding additional lanes.

Mark J. Telesnicki

Green Island Park **Ottawa, Ontario**

Field investigation and preliminary design of remedial measures to stablize a rock cliff face below a stone parapet wall. Investigation included detailed geological mapping, concrete coring, and visual assessment of wall and slope conditions.

Rue de la Montagne Slope **St. Nicolas, Quebec**

Field investigation and design of remedial measures to stabilize unstable blocks of rock on a 30 m high slope. Differential GPS was used to accurately locate each unstable block of rock which was subsequently removed or stabilized using concrete buttresses, rock bolts or dowels. High capacity rockfall fences were also installed along the slope in selected areas.

Highway 69 – Parry Sound Bypass **Parry Sound, Ontario**

Rock hazard assessment and rock foundation design for the twinning of Highway 69 near Parry Sound. Work included detailed geotechnical mapping of rock cuts, remedial designs, specifications and assistance during construction.

Highways 60 and62 **Bancroft, Ontario**

Rock hazard assessments for the upgrading of Highways 60 and 62 near Bancroft, Ontario. Work included geotechnical mapping and identification of rockfall hazards as well as the development of remedial measures and cost estimates.

Highway 69 – Seguin River **Parry Sound, Ontario**

Rock hazard assessment and rock foundation design for the twinning of Highway 69 north of Parry Sound near the Seguin River. Work included detailed geotechnical mapping of rock cuts, remedial design recommendations, rock foundation designs for three bridge structures and preparations of specifications.

QEW/Erin Mills Parkway Interchange **Mississauga, Ontario**

Investigation and design for a new interchange at Erin Mills Parkway/Southdown Road and QEW which included four new ramps, a QEW overpass and roadway re-alignment. Work involved design of new rock cuts and recommendations for long term stability.

Cape Breton Highlands National Park **Cape Breton, Nova Scotia**

Remediation of highway rockcuts along the Cabot Trail. Project involved field investigations including detailed mapping of slope conditions, laboratory testing, hazard rating and prioritization using the Rockfall Hazard Rating System; design and evaluation of remedial options and construction supervision.

Gros Morne National Park **Newfoundland**

Remedial stabilization of a large rockslide in metamorphic rock along Highway 430 in Gros Morne National Park. Work included investigation and back analysis of the slide, remedial design and supervision of construction activities which comprised controlled blasting, rock bolt installation and excavation of slide debris.

Signal Hill **St. John's, Newfoundland**

Stability assessment of high natural rock slopes along a public walkway in Signal Hill historic park. Work involved detailed geotechnical mapping of slope conditions and recommendations on remedial alternatives to stabilize potential rock fall areas.

Niagara River **Niagara Falls, Ontario.**

Annual inspection and stability assessment of rock slope conditions at tourist attractions along the Niagara River Gorge. Work involved the design and implementation of an instrumentation program at Niagara Falls including vibrating wire piezometers and extensometers connected to a central data logger and modem for remote retrieval of data, surveying of monitoring pins, evaluation of photographic records, geotechnical mapping, supervision of remedial rock scaling work and design of remedial measures where required.

Burleigh Hill **St. Catharines, Ontario**

Stability evaluation and design of remedial measures for a road cut through the Niagara Escarpment. Work including extensive scaling to prevent injury to pedestrians and damage to vehicular traffic, removal of overhangs created by differential weathering of shales and dolostone and preparation of specifications for drainage and shotcrete for the slope.

Cliff Park **Ottawa, Ontario**

Stability assessment of a 10 m high sedimentary rock face and remedial recommendations for development of a park above the rock slope.

Highway Road Cuts **Ashley, Ontario.**

Inspection of rock conditions and remedial recommendations to ensure the stability of rock excavations for a highway expansion. Remedial measures included trim blasting and rock bolting.

Niagara Escarpment **Wiarton, Ontario**

Inspection and detailed geotechnical assessment of part of the Niagara Escarpment near a planned water treatment plant. Recommendations were made to stabilize the slope above the proposed plant.

Parliament Hill **Ottawa, Ontario**

Investigation and stability analysis of a masonry retaining wall and underlying rock and soil slope at the Parliament Buildings. Remedial measures included wall replacement and foundation reinforcement using dental concrete and dowels.