

TECHNICAL MEMORANDUM

TO: Mr Chris McShane
(McIntosh Perry Consulting Engineers Ltd.)
FR: Phil de Graaf/Mark Telesnicki
RE: **ROCK CUT STABILITY ASSESSMENT**
1000 ISLAND PARKWAY BRIDGE OVER HWY 137
W.P. 254-98-00

DATE: February 12, 2004
JOB NO: 041-117-016

Golder Associates Ltd. were retained by McIntosh Perry to carry out a geotechnical assessment of the rock cut along the east side of Highway 137 at the 1000 Island Parkway Bridge overpass. The scope of work included:

- a) a site visit to collect the required geotechnical data
- b) identification of potential instabilities and identification of potential failure modes
- c) stability analysis of any unstable blocks of rock
- d) development of remedial recommendations

Site Visit

On Monday 12 January, 2004, Golder Associates carried out a site investigation at the east abutment of the 1000 Islands Parkway bridge over Highway 137 (Site 16-173). Mr. Larry O'Grady (McIntosh Perry Consulting Engineers Ltd.) met with Mr. Philip de Graaf (Golder Associates Ltd.) and identified in the field the previously noted tension cracks and areas of rockfall hazard concern in the vicinity of the bridge.

Site Description

Highway 137 runs north south and links Highway 401 with the 1000 Islands Bridge to the USA. Highway 137 passes below the 1000 Islands Parkway at an overpass bridge at marker chainage 12+542 (Ref. Figure 1). The east bridge abutment comprises a steep (75° dip angle) rock cut ranging in height from less than 1 m to the south, up to more than 10 m to the north of the bridge. In the immediate vicinity of the bridge, the rock cut height is approximately 4.6 m high, except for a localised area near the southern end of the bridge deck which is about 5.4 m high. The clear zone, measured from the edge of pavement to the toe of the rock cut (at road elevation) is at least 10 m.

Rockfall Hazard Assessment

The abutment rock cut comprises pinkish grey-brown, coarse grained, fresh to slightly weathered, moderately to widely jointed, very strong, granitic rock. In the vicinity of the bridge, the overall impression of the rock cut, is of a clean, well blasted slope (since greater than 90% half barrels or drillhole traces are evident) excavated into good to very good quality rock. However, on closer inspection a number of potentially adverse joint intersections were noted as discussed in the following sections (Ref. Plate 1).

Two separate tension cracks were observed along the slope crest, directly below the bridge deck. These tension cracks run sub-parallel to the slope crest and daylight along the top of the cut, the full width of the bridge deck. Based on observed (and inferred) discontinuity intersections of at least three sub-vertical joints (providing lateral 'release' planes) and a series of shallow dipping basal sliding planes, two kinematically feasible sliding blocks were identified (Ref. Plate 2).

North & South Blocks

The tension crack at the rear of the larger of the two blocks; the south block; has an aperture (crack opening) ranging between 0.25 to 0.75 cm. For the north block, the aperture is less than 0.5 cm. Portions of these tension cracks appear to have preferentially developed along pre-existing joints, however, there is also evidence of tensile failure through intact rock. Although it is difficult to ascertain, for certain, whether these tension cracks developed during construction or are a post-construction phenomenon (i.e. due to weathering such as ice jacking), it is likely they were at least partly developed during initial construction (i.e. during blasting).

A number of shallow dipping, planar and rough to undulating and rough joints daylight in the cut slope. These joints could form basal sliding planes for potential plane failure type block sliding. Although a series of shallow dipping joints occur throughout the rock face, and probably represent a stepped basal plane, in order to simplify analysis and in the interest of a conservative assessment, the lowest plane has been assumed to be continuous and defines the potential sliding plane. Given that these joints are clean, planar-rough to undulating-rough; joint strength values of 30° for the friction angle (ϕ) and 0 kPa for the cohesion (c) have been assumed for the stability

analysis (note that no laboratory testing was carried out but that these values are considered to be conservative).

The south block has an inferred calculated volume of about 190 m³ with a weight of just over 500 tonnes. The volume of the north block is about 15 m³ with a weight of less than 40 tonnes. Measured and calculated block geometrical information for both blocks are summarized below:

Table 1 – Summary of Potentially Unstable Block Characteristics

Block Geometry	Measured Parameters				Calculated Parameters	
	Block Height [H] (m)	Tension Crack to Crest Length [b] (m)	Block Width (m)	Failure Plane Angle [ϕ_o] (°)	Block (m ³)	Block Weight (t)
South Block	2.45 to 3.20	1.5 to 5.1	23.0	10 to 15	193	503
North Block	2.00 to 2.45	1.5	4.6	10 to 15	15	38

A small zone of blocky, potentially unstable material was identified to the north of the north block (Ref. Section A on Plate 2). This zone comprises about 2 to 3 m³ and would probably be contained by the roadside catch ditch should it become dislodged. This material is not considered a significant hazard to the roadway nor to the bridge structure. The rock could be removed using a back-hoe to minimize future ditch maintenance requirements.

Block Stability Assessment

The occurrence of sub-vertical tension cracks, basal plane joints roughly parallel to the slope crest, and sub-vertical end joints, define classic planar type potential sliding blocks (Ref. Figure 2). The plane failure mechanism involves sliding on a planar discontinuity which strikes sub-parallel to and is undercut by the slope face (i.e.: the failure plane “daylights” in the slope face). For sliding to occur, the shear strength along the failure surface must be exceeded and release surfaces must exist normal to or at a steep angle to the failure plane. This usually means that the dip of the failure plane for the ‘dry’ slope must be steeper than the angle of friction (ϕ) applicable to that plane, however; where water-filled tension crack and sliding plane conditions exist, water pressure and /or ice jacking induced failure may occur even where the sliding plane angle is lower than the friction angle.

The stability of the identified blocks has been evaluated by limit equilibrium techniques such as those described by Hoek and Bray (1977). Refer to Figure 2 for a graphical representation of the plane failure mechanism, and explanation of the limit equilibrium stability assessment methodology. Results of the plane failure analysis are summarized on Figure 2. Three typical cross-sectional block geometries were used to evaluate the stability of the blocks (refer to Plate 2a for locations of these sections). Due to variations in block height and tension crack location two sections (Sections A & B) were taken to represent the south block, and a single section (Section C) for the north block. Most input parameters can be fairly well defined through actual field measurements, however, some uncertainty remains as to the quantification of key parameters such as:

- sliding plane slope angle, and
- water condition of the tension and sliding plane.

These uncertainties arise since only limited sliding plane exposure is available for mapping, and water conditions may vary seasonally.

In order to address these uncertainties, a limited parametric or sensitivity study of the factor of safety (FOS) with respect to changes in basal sliding plane angle (ϕ_p) and tension crack water condition was undertaken. Field measurements indicate that the basal sliding planes dip between 10° and 15° , therefore best, average and worst case sliding plane angles of 10° , 15° and 20° are considered reasonable, and were used in the analysis. At the time of the investigation some evidence of water seepage from the basal sliding planes was observed (accumulated frozen ground water flow on the slope face), however this flow is probably fairly limited and the sliding plane appears to be free draining. In terms of water condition, dry, fully saturated and half full tension crack conditions were used for best, worst and average case conditions.

As summarised in the tables on Figure 2, the stability assessment for the average and best case block conditions indicates that the blocks are stable (Factor of safety, $FOS > 1.5$). This analysis suggests that planar block failure could occur should worst case conditions for both sliding plane angle and tension crack water condition occur simultaneously. Although this scenario is considered somewhat conservative, the limit equilibrium analysis indicates that by installing a

total of 27 (twenty seven) 25 mm diameter untensioned shear dowels, or a total of 10 (ten) tensioned (100 kN) rock bolts, would provide sufficient reinforcement to achieve a FOS of at least 2 for the worst case scenario for both the south and north blocks.

Although safety factors in excess of 1.5 are considered acceptable for many civil engineering applications, in some instances slightly higher factors of safety are necessary, in particular where repercussions of failure will be severe or where some uncertainty exists over in-put parameter values. In these cases a safety factor in excess of 2 are usually used. For the average case scenario a total of 5 (five) untensioned shear dowels or a total of 3 (three) tensioned (100 kN) rock bolts will provide an FOS of 2 for both the south and north blocks.

Remediation Options

Upon careful evaluation of the potentially unstable blocks, the proximity of the slope crest to the bridge deck, the slope height, and the block volume in relation to the catch-ditch capacity we propose four alternative options for consideration:

1. Do nothing,
2. Install rock reinforcement and/ or slope drainage,
3. Excavate or remove blocks,
4. Install instrumentation and monitor the rock blocks for evidence of movement.

Option 1 – Do Nothing

The identified plane failure type blocks are not considered to pose an imminent rockfall hazard in the short to medium term. These blocks have negligible influence on the adjacent bridge abutment. Based on information contained in the east abutment 1000 Islands Parkway Structure Synopsis Reports (1995, 1997 and 1999), when the tension cracks were first noticed, the blocks have remained stable for at least 9 years. Further, the limit equilibrium analysis indicates that for average and best case conditions the identified plane failure blocks are stable. However, ice jacking may continue to cause some tension crack dilation. Should these blocks become dislodged, it is likely that the monolithic blocks would remain intact and would probably be retained in the catch-ditch and not reach the roadway.

Option 2 – Rock Reinforcement and/ or Drainage

Based on the worst case limit equilibrium analysis, at least 27 No., untensioned, 4m long shear dowels (if considering 25 mm diameter standard rebar) or 10 No. tensioned (100 kN) rock bolts of similar length, would be required to achieve a factor of safety of 2. These dowels could fairly easily be installed either from scaffolding, sky-jack or rope access techniques. Construction costs could range from \$25,000.00 to \$30,000.00 and would probably require that north bound traffic be restricted to one lane for the duration of the work (probably 4 to 5 days). Construction supervision and other engineering fees have not been included in the construction cost estimate. Scaling of potentially loose blocks to the north of the north block could also be carried out in conjunction with the rock reinforcement works.

Installing drainage holes to intersect the basal and tension cracks would also improve the block safety factors, by preventing water pressure build-up on the sliding planes. The efficiency or success rate of drain hole depressurisation is sometimes difficult to predict, and is usually applied in conjunction with rock reinforcement. Drain hole installation may permit the level of rock reinforcement levels to be slightly reduced in view of the lower likelihood of excess water pressures, and may mitigate ice jacking to some extent.

Option 3 – Excavate Blocks

By excavating and removing potentially unstable planar blocks (north & south blocks) as well as the blocky zone to the north of the north block, any potential for these materials contributing to rockfall hazard in the vicinity of the bridge east abutment would be prevented. Although removal of the 2 to 3 m³ blocky material to the north of the bridge could be carried out quite easily, the bulk of the excavation, about 200 m³ (for the north and south blocks), would be rather challenging considering that there is only about 0.75 m to 1 m clearance below the bridge deck, and there may be some potential to cause damage to the bridge during excavation. Non-explosive techniques would need to be adopted to minimise damage to the bridge. The excavation works could cost between \$60,000.00 (\$300/m³) and \$100,000.00 (\$500/m³) and probably require complete closure of the north bound portion of Highway 137. Construction supervision and other engineering fees have not been included in these construction cost estimates.

Option 4 – Monitor and Re-Assess Later

Since the blocks have apparently remained stable for about 9 years already, and considering that the average case limit equilibrium stability calculations suggest that the blocks will remain stable, it is our opinion that it would be prudent to install some rudimentary geotechnical instrumentation and monitor crack dilation over a period (say several years) and reassess the situation once block displacement has definitively been determined. Two monitoring options are available;

- Option A – involving intermittent manual reading of Tell-Tale Crackmeters, and
- Option B – vibrating wire strain gauges (joint meters) which could be frequently monitored via satellite or cellular phone.

Option A - On of the simplest, and cheapest, crack dilation monitoring techniques is the Tell-Tale Crackmeter which comprises two overlapping plates; one plate is calibrated in millimetres and the overlapping transparent plate has a cross hair. As the crack opens or closes the plates move relative to each other, and the relationship of the cross hair to the scale represents the crack movement. Two Tell-Tales could be installed across the south block tension crack and a single Tell-Tale across the north block crack. The Tell-Tales can be purchased for a few hundred dollars, and could be installed in a couple of hours, for a total cost of about \$3,500.00. Golder, McIntosh Perry or MTO personnel could make periodic instrument readings (say monthly for the first year).

Option B - Vibrating wire strain gauges (joint meters) provide an absolute measure of joint dilation. Since these instruments have electronic output, these instruments offer the advantage of allowing remote data acquisition. This could include either a cellular or satellite data transfer system. An advantage of this system is that multiple readings (daily with weekly uploads) could be taken each month, and the results could be made available on a secure, access controlled, web based, data management system, such as Golder's GIDIE (Golder Instrumentation Data Interpretation and Evaluation) system. The VW gauges and telemetry system could be purchased and installed for a total cost of about \$8,500.00, and a monthly data transfer fee of less than \$30.00 per month.

Both Options A and B will allow future joint dilation to be quantified, however, in our opinion, Option B is preferable since the system is fully automated and multiple readings can be easily

Memo to: Mr. Chris McShane
From: P. de Graaf/M. Telesnicki

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February 12, 2004
041-117-016

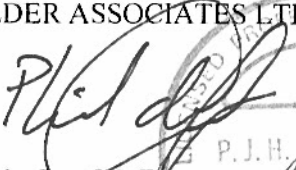
obtained permitting seasonal trends and possible ongoing movements to be accurately assessed without having to dispatch field staff.

If after a few years, no noticeable movement has occurred, this would provide greater confidence in opting for the 'do nothing' option. However, if significant dilation occurs then this will substantiate the need for additional rock reinforcement.

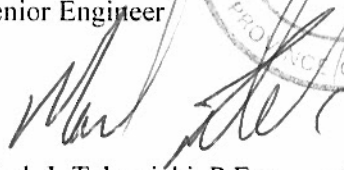
We trust this Technical Memorandum addresses your concerns. If you require any additional information please do not hesitate to contact us.

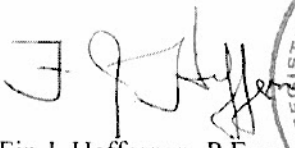
Sincerely,

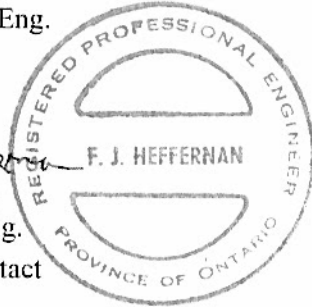
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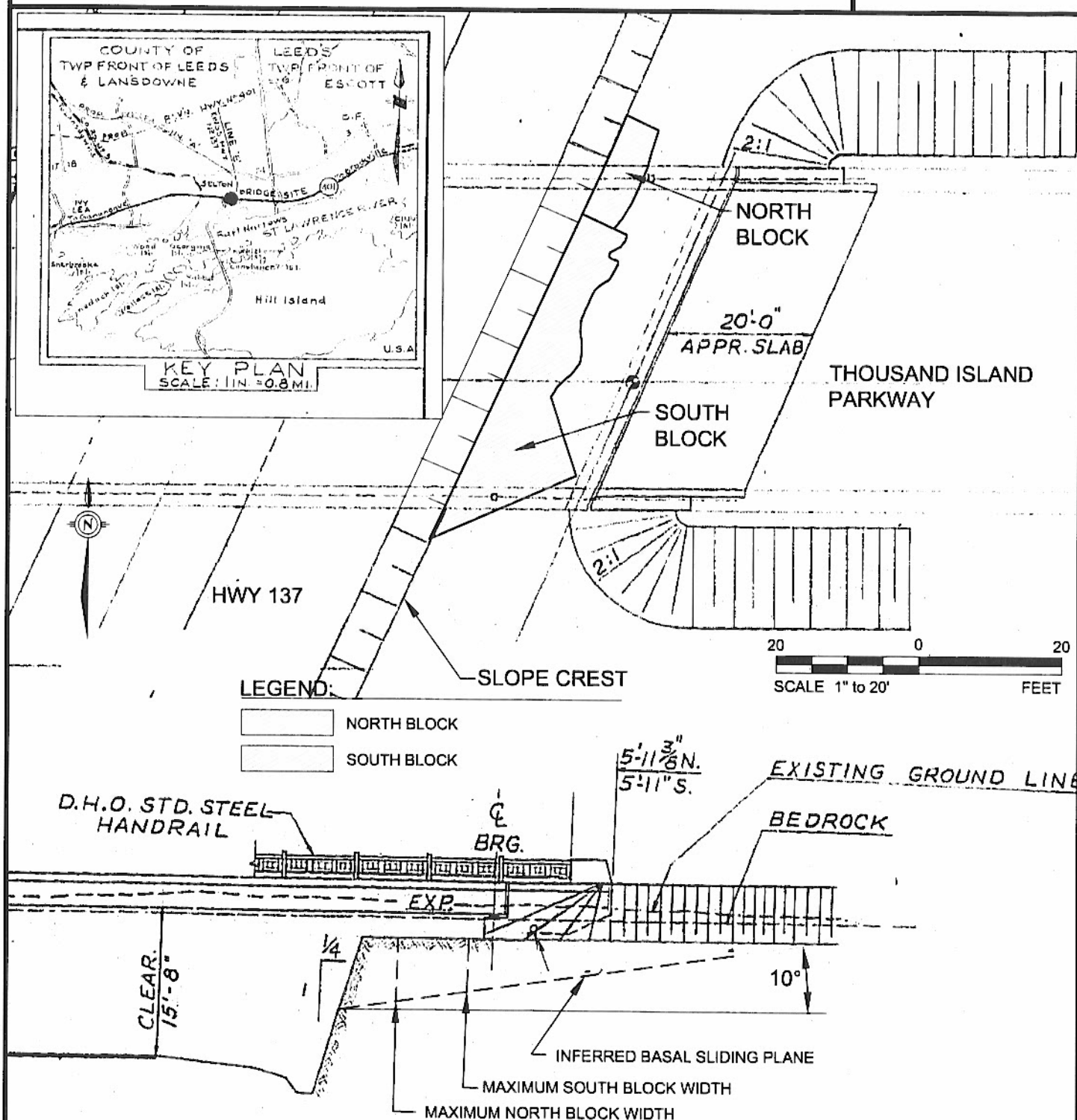


PdG/MJT/

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1000 ISLAND PARKWAY BRIDGE AND HWY 137 LOCATION PLAN & SKETCH OF POTENTIAL SLIDING BLOCKS FOR EAST BRIDGE ABUTMENT

FIGURE 1



REFERENCE:

DRAWING ADAPTED FROM "OLD KING'S HWY 401 UNDERPASS AT THE INTERSECTION OF OLD KING'S HWY 401 & 137 (NEW ROAD TO IVY LEA BRIDGE)". FROM DRAWING No. D-5585-1.

DATE: JANUARY 2004

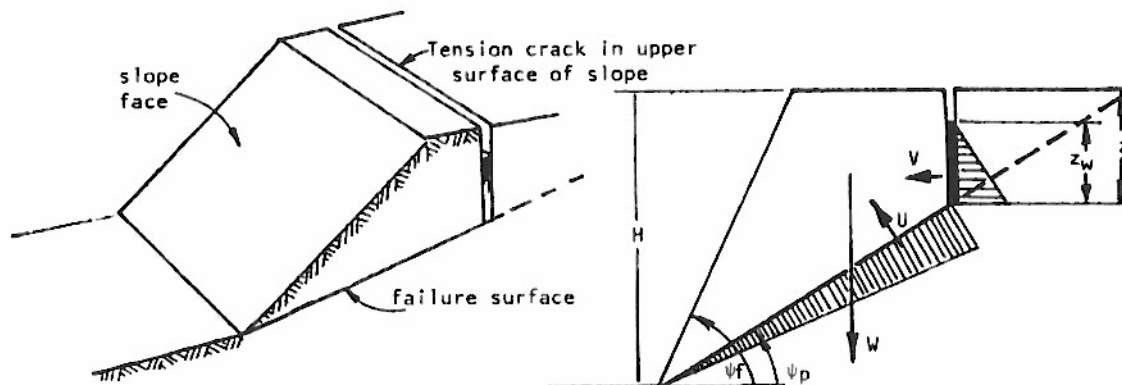
PROJECT: 04-1117-016



CAD: JFC

CHK: PdG

FIGURE 2



$$F = \frac{cA + (W \cos \psi_p - U - V \sin \psi_p) \tan \phi}{W \sin \psi_p + V \cos \psi_p}$$

where

$$z/H = (1 - \cot \psi_f \cdot \tan \psi_D)$$

$$W = \frac{1}{2} \gamma H^2 \left((1 - (z/H)^2) \cot \psi_p - \cot \psi_f \right)$$

$$A = (H - z) \cdot \operatorname{Cosec} \psi_D$$

$$U = \frac{1}{2} \gamma_w \cdot z_w (H - z) \cdot \text{Cosec} \psi_p$$

$$V = \frac{1}{2} \gamma_w \cdot z_w^2 \quad (\text{after Hoek and Bray, 1977})$$

Plane Failure Mechanism Limit Equilibrium Stability Assessment Summary

Section A (South Block)	Basal Sliding Plane Angle ϕ_p	Tension Crack Water Condition (Z_w/Z)	FOS	No. Shear Dowels for FOS>2*	No. 100kN Rock Bolts for FOS>2*
Best Case	10	0 (Dry)	3.28	NA	NA
Average	15	0.5 (half full)	1.86	2	1
Worst Case	20	1 (full tension crack)	1.21	12	5

In-put Parameters

H = 3.2 m b = 5.1 m Section A approximately 11 m wide

Section B (South Block)	Basal Sliding Plane Angle ϕ_p	Tension Crack Water Condition (Z_w/Z)	FOS	No. Shear Dowels for FOS>2*	No. 100kN Rock Bolts for FOS>2*
Best Case	10	0 (Dry)	3.28	NA	NA
Average	15	0.5 (half full)	1.70	2	1
Worst Case	20	1 (full tension crack)	0.93	9	4

In-put Parameters

H = 2.4 m b = 2.1 m Section B approximately 12 m wide

Section C (North Block)	Basal Sliding Plane Angle ϕ_p	Tension Crack Water Condition (Z_w/Z)	FOS	No. Shear Dowels for FOS>2*	No. 100kN Rock Bolts for FOS>2*
Best Case	10	0 (Dry)	2.44	NA	NA
Average	15	0.5 (half full)	1.67	1	1
Worst Case	20	1 (full tension crack)	0.88	6	1

In-put Parameters

H = 2.0 m b = 1.5 m Section C approximately 4.6 m wide

cohesion (c) = 0 kPa $\phi = 30^\circ$

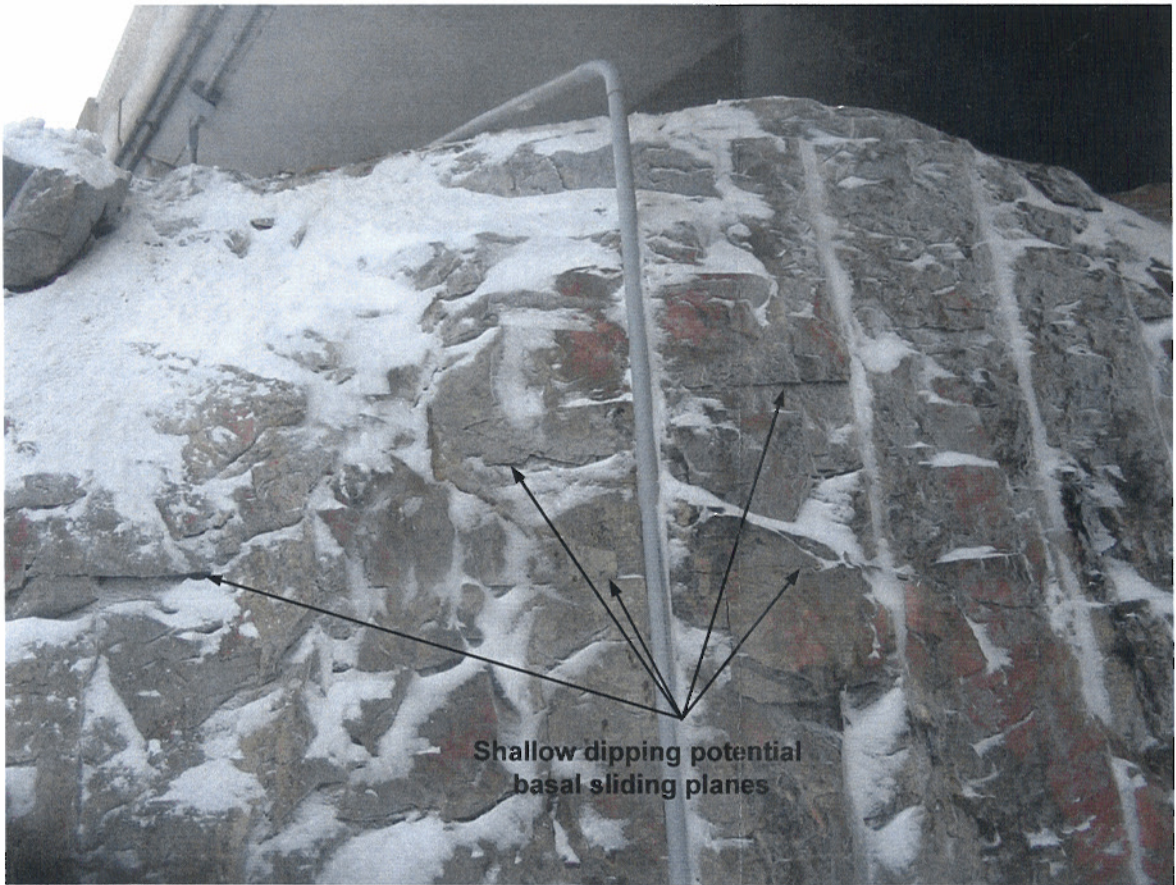
* 25 mm diameter shear dowels Refer to Plate 2a for locations of Cross-Sections A, B & C



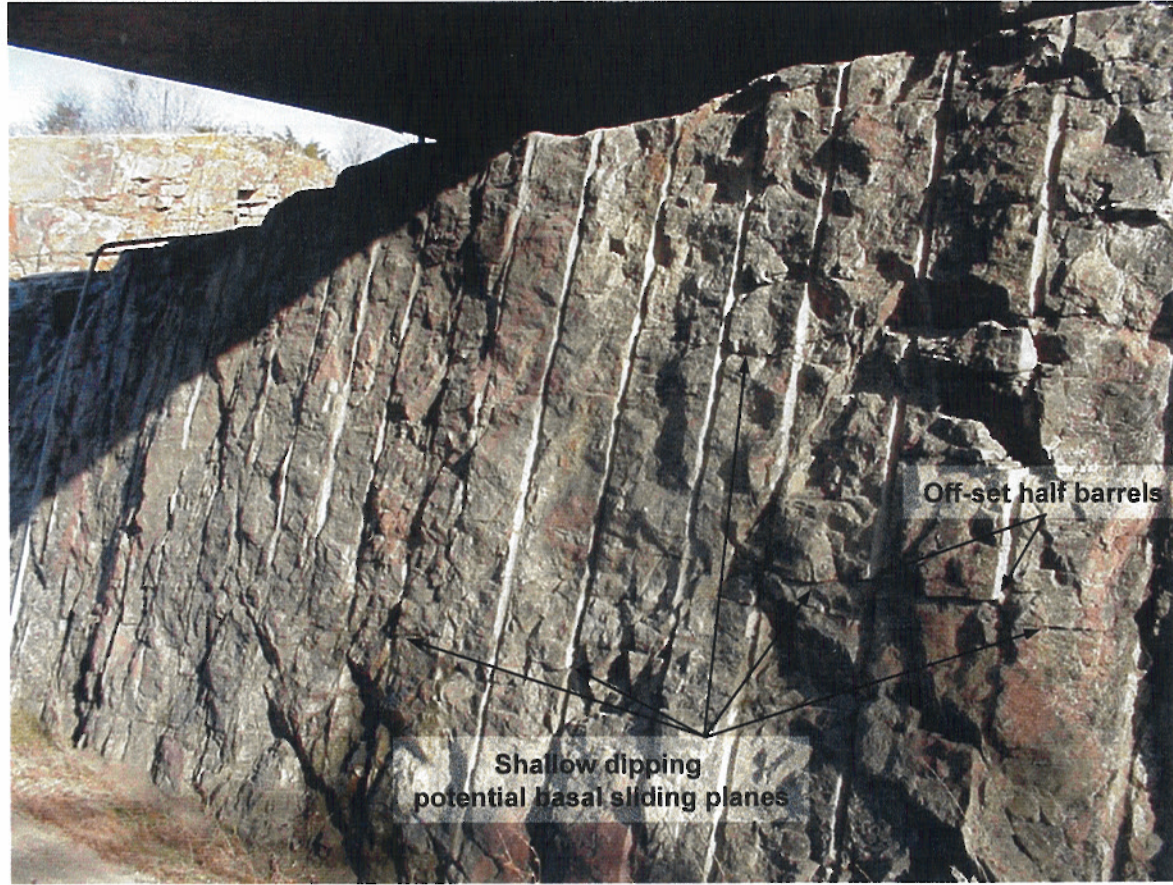
A) Slope crest looking north



B) Slope crest looking south



C) Slope face north side of bridge



D) Slope face south side of bridge (photo provided by Tony Sangiuliano)

Date: February, 2004

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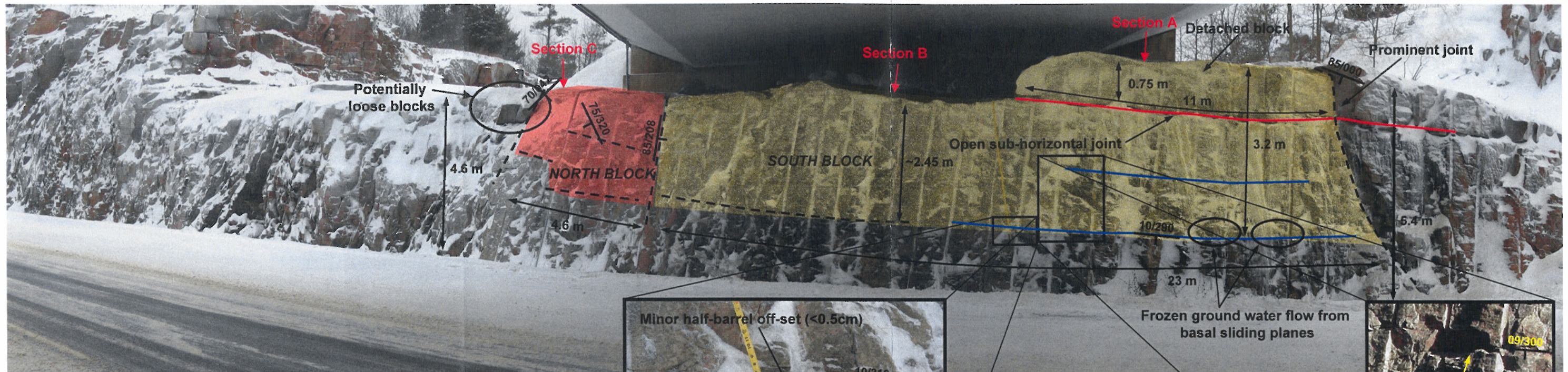
Golder Associates

Drawn: PdG

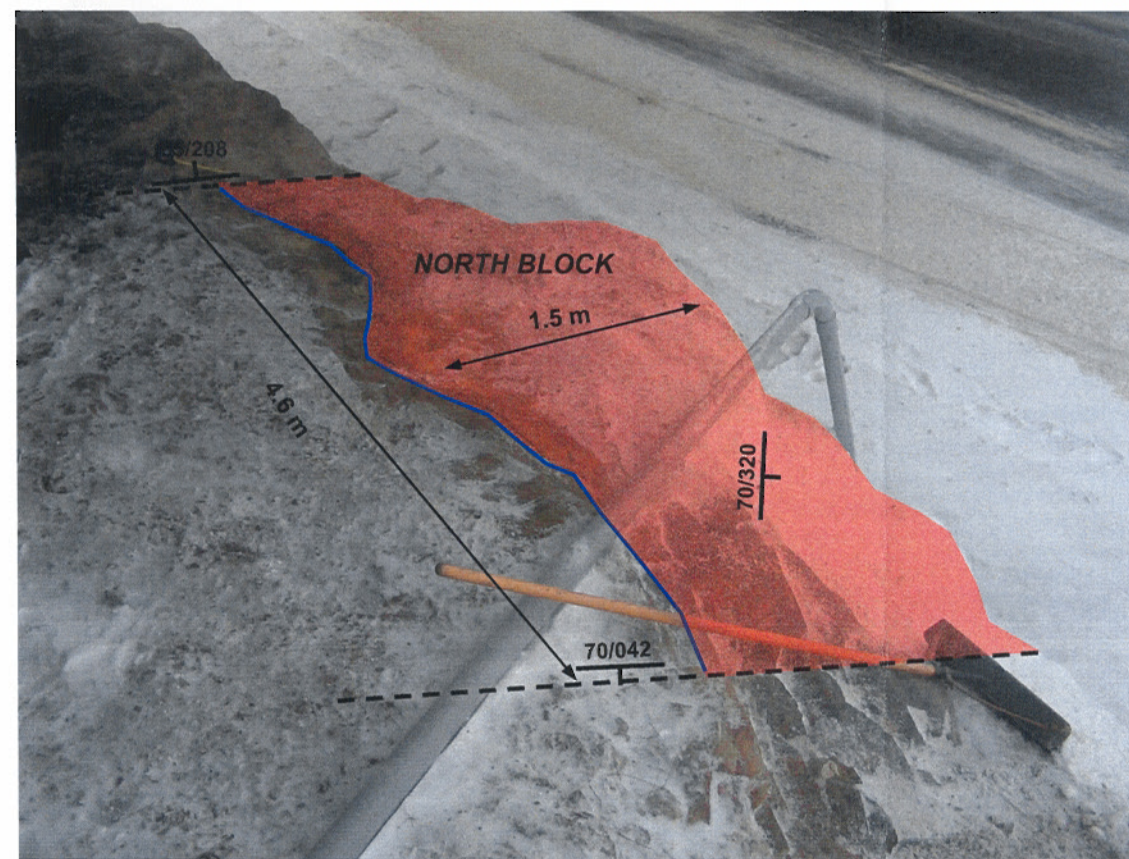
Chkd: MJT

**THOUSAND ISLAND PARKWAY BRIDGE ROCK CUT
AT HIGHWAY 137
GEOTECHNICAL MAPPING PHOTO MOSAIC**

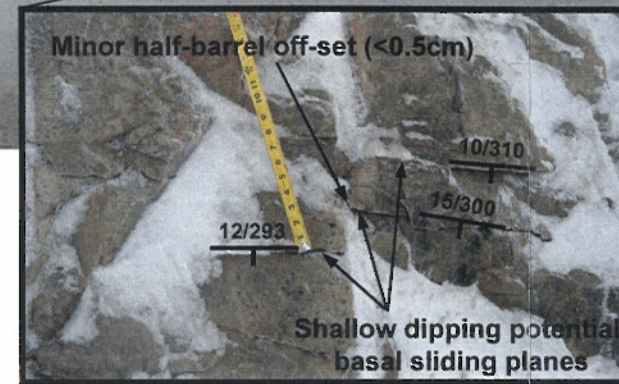
PLATE 2



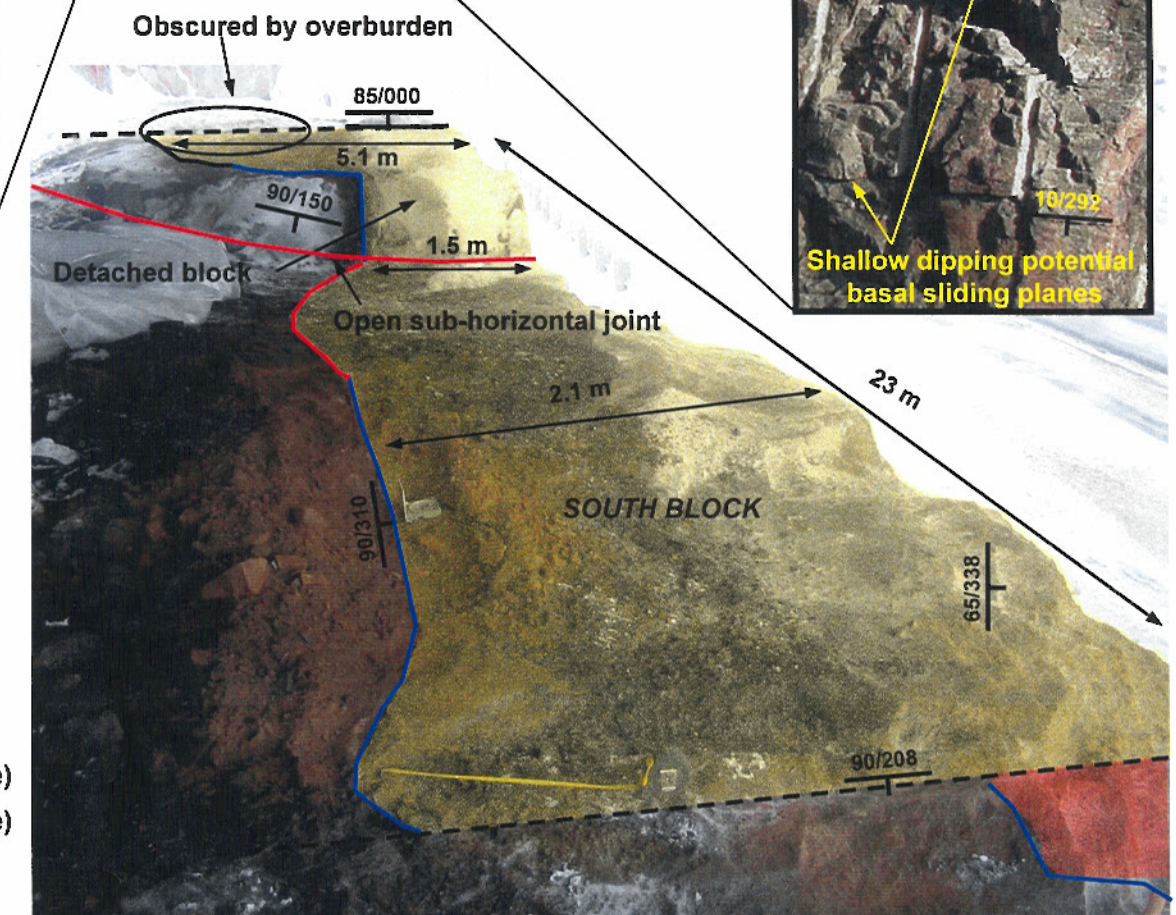
A) Overall View of East Bridge Abutment Rock Cut Face



B) North Block Crest



Basal plane zone



C) South Block Crest

Legend:

-
- 75/090
└───┘
Discontinuity orientation (strike/dip)
- Joint (aperture <0.25 cm)
- (blue)
Tension crack/ joint (0.25 to 0.5 cm wide)
- (red)
Tension crack/ joint (0.5 to 0.75 cm wide)
- (yellow)
South block
- (red)
North block

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Drawn: PdG

Chkd: MJT