



June 2010

REPORT ON

FOUNDATION INVESTIGATION AND DESIGN REPORT MONTREAL STREET UNDERPASS AND APPROACHES HIGHWAY 401, KINGSTON, ONTARIO

G.W.P. 78-99-00

W.P. 4016-06-01



Submitted to:

McCormick Rankin Corporation
1145 Hunt Club Road, Suite 300
Ottawa, Ontario
K1V 0Y3

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PART A

**FOUNDATION INVESTIGATION REPORT
MONTREAL STREET UNDERPASS AND APPROACHES
HIGHWAY 401, KINGSTON, ONTARIO
G.W.P. 78-99-00
W.P. 4016-06-01**



1.0 INTRODUCTION

Golder Associates Ltd. was initially retained by the Ministry of Transportation, Ontario (MTO) in 2000 to carry out a foundation investigation at the proposed Montreal Street Underpass at Highway 401 (Site 7-068) near Kingston, Ontario under assignment G.W.P. 78-99-00 and W.P. 4016-06-01 (Agreement No. 4005-A-000069). This investigation formed part of the overall project which involved the widening of Highway 401 from 2.7 km west of Highway 38 easterly to Highway 15. Golder Associates Ltd. (Golder) has since been retained by McCormick Rankin Corporation (MRC) on behalf of the MTO to update the original foundation investigation associated with the replacement of the Montreal Street Underpass and to provide additional foundation investigations for the widening of the CNR bridge and Highway 401 embankments within the Cataraqui Wetlands (G.W.P. 78-99-00).

This report addresses the proposed Montreal Street Bridge and its approaches within 20 m of the structure.

The purpose of the 2000 foundation investigation was to determine the subsurface conditions at the site of the proposed bridge structure by drilling boreholes, carrying out structural mapping of the exposed rock cut, and performing in-situ tests and laboratory tests on selected samples, where appropriate. Additional structural mapping of the exposed rock cuts was subsequently carried out in 2009. Based on our interpretation of the data obtained, recommendations on the foundation aspects of design of the proposed works are provided. Comments are also provided on anticipated construction constraints where they may affect the design of the proposed bridge structure.

The terms of reference for the current scope of work are outlined in our proposal letter P81-1402, dated July, 2008.



2.0 SITE DESCRIPTION

The bridge site is located adjacent to and immediately east of the existing Montreal Street Underpass at Highway 401, near Kingston, Ontario.

The existing Montreal Street Underpass is a single-span concrete structure which carries four-lanes of traffic for Montreal Street over Highway 401. The existing Highway 401 is four-lane and divided, and runs east-west within the project limits.

The topography of the site area is generally level with a regional trend sloping down to the south towards Lake Ontario. Within the area of the existing Montreal Street Underpass, Highway 401 has been constructed in cut with near vertical exposed rock side walls up to 9 m high. The existing ground surface above the cut (i.e., the tableland) varies from about Elevation 104 m on the north side to between about Elevation 101.5 m and Elevation 102.5 m on the south side. Based on available information, the approximate existing grade of Highway 401 at the proposed Montreal Street Underpass is about Elevation 95 m.

The lands in the vicinity of site are mainly agricultural to the north and residential to the south. The tableland on the north side of Highway 401 is essentially bare of vegetation. The vegetation cover on the tableland to the south of Highway 401 generally consists of grass, bushes, and a few mature trees.



3.0 INVESTIGATION PROCEDURES

3.1 Drilling Investigations

The field work for the original investigation was carried out on April 17 and 19, 2000. At that time seven boreholes were put down at the site. Boreholes 3-1 and 3-2 were put down near the limits of the proposed north abutment. Boreholes 3-4 and 3-6 were put down near the limits of the proposed south abutment. Boreholes 3-3 and 3-5 were advanced in the area of the proposed north and south approaches, respectively, and Borehole 3-7 was located within the median near the proposed center pier. The boreholes were extended to depths of between 0.1 m and 9.2 m below the existing ground surface.

The investigation was carried out using a track-mounted CME 55 drill rig supplied and operated by Marathon Drilling Co. Ltd. of Ottawa, Ontario. In the boreholes, samples of the overburden were generally obtained at regular intervals of depth of 0.75 m using 50 mm outside diameter split-spoon samplers in accordance with the Standard Penetration Test (SPT) procedures. Bedrock was cored in NQ size in Boreholes 3-2, 3-4 and 3-7. The open boreholes were backfilled with bentonite mixed with auger cuttings to provide an adequate seal in accordance with MOE requirements. Groundwater conditions in the open boreholes were observed throughout the drilling operation and upon completion of drilling. Piezometers were installed in Boreholes 3-2 and 3-4 to permit monitoring of the groundwater levels at these locations. The piezometers consisted of a 200 mm long slotted tip threaded into 12 mm diameter PVC rigid tubing.

The field work was supervised on a full-time basis by a member of our engineering staff who located the boreholes in the field, directed the drilling, sampling and in-situ testing operations, and logged the boreholes. The soil samples and bedrock core were identified in the field, placed in labelled containers and boxes, respectively, and transported to our laboratory in Mississauga for further examination. Water contents were determined on selected samples of the recovered soil. Point load testing was carried out on selected samples of the recovered rock core.

The borehole locations were surveyed and staked in the field by Transenco Ltd., who have professional land surveyors on staff. Based on the information provided, the northing and easting co-ordinates of the borehole locations are given in UTM, and the borehole elevations are referenced to Geodetic Datum. The co-ordinates of the boreholes are indicated on the Record of Borehole sheets and the locations of the boreholes are shown on Drawing 1.

3.2 Structural Mapping

Structural mapping of the exposed rock faces on the north and south sides of the Highway 401 cut within the general limits of the proposed bridge abutments was carried out on April 22, 2000 by a member of our rock engineering staff and again by our staff in September, 2009. The purpose of the mapping was to characterize the rock and identify any potential failure mechanisms or rockfall hazards evident on the existing cuts and relate these to the proposed cuts to be excavated during the widening of Highway 401 at Montreal Street and construction of the new Montreal Street E-N/S and N/S-W ramps. Each of the rock cuts were photographed and selectively mapped. The geological mapping consisted of measuring the dip and dip direction of the major joint sets exposed on the abutments above the Montreal Street Underpass and on the faces of the existing ramp rock cuts. In addition, joint roughness, shape, condition and spacing were recorded to assess the frictional characteristics of the joints. A general visual assessment was conducted in the field to describe the rock and assess any potential stability issues including the potential for wedge, sliding and toppling failure.



The discontinuity orientation data collected during structural mapping was analyzed statistically using the software DIPS®, distributed by Rocscience, to obtain the major discontinuity sets from stereographic projections which provide a 3D representation of structural data. Analyses were carried out using lower hemisphere and equal area projections, and a Fisher distribution. Peak joint set orientations were identified from the contours and a kinematic stability analysis was conducted with this data.



4.0 GENERAL SITE GEOLOGY AND STRATIGRAPHY

4.1 Site Geology

The site is located in the physiographic region of southern Ontario known as the Napanee plain (The Physiographic of Southern Ontario, Chapman and Putnam, 3rd Edition, 1984). The overburden is typically shallow. The Napanee plain, which is generally flat to undulating, has been stripped of most of its overburden during the late Wisconsinian glaciation period some 11,000 years ago.

Geologic mapping (Map 2544, Ministry of Northern Development and Mines, 1991) indicates the bedrock at the site consists of Paleozoic rock of the middle Ordovician age. The predominant bedrock type in the area is limestone of the Gull River Formation. The local bedrock is generally located at or near the ground surface. Within the area of the Montreal Street Underpass, the existing Highway 401 has been constructed in cut, exposing the limestone bedrock on both sides of the highway.

4.2 Site Stratigraphy

The detailed subsurface soil, bedrock, and groundwater conditions encountered in the boreholes, together with the results of the laboratory tests carried out on selected soil and rock samples, are given on the attached Record of Borehole sheets following the text of this report. The stratigraphic boundaries shown on the borehole sheets are inferred from non-continuous sampling and, therefore, represent transitions between soil types rather than exact planes of geological change. Subsurface conditions will vary between and beyond the borehole locations. In summary, the subsoils at the site generally consist of a surficial layer of topsoil underlain by shallow thicknesses of sand and gravel fill and / or silty clay. Bedrock was encountered or inferred from refusal to further auger penetration at about Elevation 104 m on the tableland on the north side of Highway 401 and at about Elevations 100.5 m to 101.5 m on the tableland on the south side of Highway 401. Within the existing median (i.e., at the base of the rock cut) the bedrock was encountered below the granular fills at about Elevation 93 m. A detailed description of the subsurface conditions encountered in the boreholes for this investigation is provided in the following sections.

4.2.1 Topsoil

An 80 mm thick surficial layer of topsoil was encountered in Boreholes 3-4 to 3-6.

4.2.2 Asphalt and Granular Fills

About 50 mm of asphalt was encountered surficially in Borehole 3-7.

About 130 mm, 690 mm and 100 mm of sand and gravel fill with traces of silt was encountered surficially in Boreholes 3-1 and 3-3, respectively, and below the asphalt in Borehole 3-7. Standard Penetration testing (SPT) carried out within the sand and gravel fill measured an 'N' value of 38 blows per 0.3 m of penetration, which indicates a dense state of packing. Below the sand and gravel fill in Borehole 3-7 is a layer of silty sand fill about 690 mm in thickness. The silty sand fill is compact with an SPT 'N' value of 22 blows per 0.3 m of penetration measured within the layer. The water content for a selected sample of the silty sand fill was measured at about 5 percent. A 1.1 m thick layer of rock fill consisting of a mixture of sand, gravel and limestone fragments was encountered below the silty sand fill in Borehole 3-7. This layer may be shattered rock as part of the original construction or it may be rock fill placed to raise the grade after over-excavation.



4.2.3 Silty Clay

A deposit of silty clay, between 0.3 m and 0.9 m thick, was encountered below the sand and gravel fill in Borehole 3-3 and below the topsoil in Boreholes 3-4 to 3-6. Traces of sand and gravel, and occasional organics, consisting primarily of fine rootlets, were noted within the silty clay. Standard Penetration testing (SPT) carried out within the silty clay measured 'N' values of between 7 blows and 15 blows per 0.3 m of penetration, which indicates a firm to stiff consistency. The natural water contents for two selected samples of the silty clay were measured at about 35 percent.

4.2.4 Bedrock

The rock exposed on the rock cuts at the Hwy 401-Montreal Street Underpass is a fresh to faintly weathered, thinly to medium bedded, light tannish grey to buff, fossiliferous limestone. The north face east of Montreal Street, which can be seen on Figure 2, is approximately 8.2 m high and extends for approximately 320 m to the east of the Montreal Street Underpass. The south face is approximately 6.1 m high at the underpass and extends for approximately 280 m to the east (refer to Figure 1), increasing in height to between 8 and 10 m. The north and south faces west of Montreal Street extend westward from the underpass for approximately 95 m and 165 m respectively (see Figures 4 and 3).

The rock exposed along the existing ramp cuts is fresh to faintly weathered, thinly to medium bedded, interbedded light tannish grey to buff and grey, blocky, fossiliferous limestone. The crest of the N/S-W ramp's north face typically ranges between 3.2 m and 6.5 m in height. The profile of the cut typically consists of a near vertical cut face above which lies a graded overburden slope (see Figure 5). There are numerous locations along the slope where seepage is visible and as can be seen on Figure 5, the seepage comes from the crest and from a thin shaley layer at approximately 3m above the road level. The south face of the E-N/S ramp typically ranges between 5.0m to 5.5m high and can be seen on Figure 6.

On the tableland to the north of Highway 401, bedrock was encountered at about Elevation 103.9 m (at ground surface) in Borehole 3-2 and was inferred from refusal to further auger penetration in Boreholes 3-1 and 3-3 at Elevation 104.0 m (0.1 m depth) and Elevation 103.6 m (1.2 m depth), respectively. On the tableland to the south of Highway 401, bedrock was encountered at about Elevation 101.6 m (about 1 m depth) in Borehole 3-4 and was inferred from refusal to further auger penetration in Boreholes 3-5 and 3-6 at Elevation 100.4 m (1.1 m depth) and Elevation 100.9 m (0.4 m depth), respectively. Within the median of Highway 401, bedrock was encountered at about Elevation 93.2 m (about 2 m depth) in Borehole 3-7.

Boreholes 3-2, 3-4 and 3-7 were advanced about 9.2 m, 6.9 m and 3.1 m, respectively, into the bedrock by coring in NQ size. The rock core samples consist of grey, fresh, fine-grained, thinly to medium bedded limestone of the Gull River formation. The fracture index ranges from 0 to greater than 30 fractures per 0.3 m. In general, the fracture frequency is between 1 and 3 fractures per 0.3 m, and decreases with depth. The Rock Quality Designation (RQD) measured on the core samples ranged from about 30 percent to greater than 90 percent, indicating the rock mass is poor to excellent quality. In general, the rock mass is good to excellent with RQD values measured at about 75 percent to greater than 90 percent, and generally increased with depth. The rock is classified as moderately strong to strong; Grade 3 to 4, according to the Canadian Engineering Foundation Manual (CFEM, 4th Edition, 2006). Strength testing carried out on select samples of the recovered core gave diametral point load indices of between 3.7 MPa to greater than 11 MPa. Based on these point load values, the unconfined compressive strength (UCS) of the rock is generally estimated to be greater than 80 MPa.



During structural mapping, tight near vertical joints and nearly horizontal bedding were identified on the rock cuts at both the Montreal Street Underpass and the E-N/S and N/S-W Ramps. The joints observed were typically planar to undulating in shape with smooth to rough, and clean to iron and calcite stained surfaces. Based on the jointing data that was collected from the Hwy 401-Montreal Street Underpass and the Montreal Street E-N/S and N/S-W Ramps in 2000 and 2009, and the structural assessment conducted using DIPS©, there appear to be three main sub-vertical joint sets plus nearly horizontal bedding at this site (See Figures 7 and 8). Joint set J1 is the dominant joint set. The J1 set typically dips towards the northwest, J2 is generally vertical, J3 dips to the north, and J4 (bedding) dips slightly (1 degree) towards the northeast (but is essentially flat-lying). This jointing together with closely spaced bedding creates a blocky rock mass. Because the mapping was conducted on the current rock faces, it is possible that additional structure exists that is not currently observable. Note all joint orientations are with respect to True North (TN).

The physical attributes of the discontinuity sets are summarized in the table below:

Discontinuity Set	No. Poles	Ave. Dip/Dip Dir.*	Spacing (Ave.)**	Roughness	Filling	Aperture	Continuity**
J1	34	79/312 (NW)	0.3-1.5 (0.6)	Smooth-Rough Planar-Undulating	Generally clean, minor calcite	Tight	1.0-5.0
J2	15	89/224 (SE)	1.0-3.0 (1.5)	Smooth-Rough Planar-Undulating	1-3mm calcite, minor FeO	1-5 mm	0.5-10
J3	11	87/005 (N)	0.5-4 (1.5)	Smooth-Rough Planar-Undulating	Generally clean, minor calcite	Tight	1.0-5.0
J4 (bedding)	10	01/050 (NE)	0.1-0.5 (0.4)	Smooth-Rough Planar-Undulating	Generally clean, minor calcite	Tight	2.0->20.0

* Relative to True North

** In Metres

Photo-mosaics of the north and south rock cut faces are presented on Figures 1 through 4 to illustrate the potential rock structure and failure mechanisms that might be encountered during construction of the new Montreal Street Underpass. Photo-mosaics illustrating potential failure mechanisms for the current N/S-W Ramp and the E-N/S Ramp rock faces are presented on Figures 5 and 6 respectively. Based on the observed and mapped structure, there appears to be very little risk of sliding, wedge or toppling type failures on the new cuts (Figures 7 and 8). As can be seen on Figure 8, the south side cut face has a low risk of very steep narrow wedges, which can often be removed by mechanical scaling.



4.3 Groundwater Conditions

The water level in the open boreholes was observed during and upon completion of the drilling operation. A piezometer was installed in both Boreholes 3-2 and 3-4 to permit monitoring of the groundwater level at these locations. Details of the piezometer installations and water level measurements are shown on the attached Record of Borehole sheets.

A summary of the water level monitoring results for the subject site is provided in the following table.

Borehole	In Open Boreholes at Completion of Drilling		In Piezometer May 21, 2000	
	Depth (m)	Elevation (m)	Depth (m)	Elevation (m)
3-1	Dry	-	N/A	N/A
3-2	6.5*	97.4	6.2	97.7
3-3	Dry	-	N/A	N/A
3-4	3.0*	99.5	2.5	100.0
3-5	Dry	-	N/A	N/A
3-6	Dry	-	N/A	N/A
3-7	2.0	93.1	N/A	N/A

*Water Level Measured in Piezometer

The above results indicate that the groundwater table is influenced by the existing rock cut and generally slopes downward from the north and the south toward the Highway 401 cut. In addition, minor seepage from the exposed rock faces was noted at the time of the structural mapping.

It should be noted that groundwater levels are expected to fluctuate seasonally and are expected to be higher during wet periods of the year.



5.0 CLOSURE

This report was originally prepared in 2000 by Mr. Dan K. Breeze, under the direction of the Project Manager, Ms. Anne. S. Poschmann, P. Eng. Sections pertaining to bedrock were subsequently updated by Erin O'Neill, P. Eng., under the direction of Mr. Mark Telesnicki, P. Eng. and Mr. Michael Snow, P. Eng. upon the completion of additional structural mapping in 2009. Golder's Designated MTO Contact for this project, Mr. Fin Heffernan, conducted a technical and independent quality control review of the report.

Yours truly,

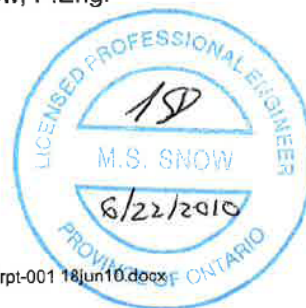
GOLDER ASSOCIATES LTD.

Erin O'Neill, P.Eng.
Geotechnical Engineer

Fin Heffernan, P.Eng.
Designated MTO Contact



Michael Snow, P.Eng.
Principal



ESO/MSS/cg/am

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PART B

**FOUNDATION DESIGN REPORT
MONTREAL STREET UNDERPASS AND APPROACHES
HIGHWAY 401, KINGSTON, ONTARIO
G.W.P. 78-99-00
W.P. 4016-06-01**



6.0 ENGINEERING RECOMMENDATIONS

6.1 General

This section of the report provides our recommendations on the geotechnical aspects of design of the proposed Montreal Street Underpass structure at the realigned Montreal Street, based on our interpretation of the factual information obtained during the investigation. It should be noted that the interpretation and recommendations are intended for use only by the design engineer. Where comments are made on construction they are provided only in order to highlight those aspects which could affect the design of the project. Those requiring information on aspects of construction should make their own interpretation of the factual information provided as it may affect equipment selection, proposed construction method and scheduling.

It is understood that Highway 401 will be widened from Montreal Street west to the Cataraqui wetlands. The widening will include a new bridge structure at the Montreal Street Underpass, the widening of the bridge over the CN Rail right-of-way, and widening of embankments within the Cataraqui wetlands. The works described in this report are associated with the proposed Montreal Street Underpass and its approaches within 200 m of the structure.

It is understood that Montreal Street will be realigned to the east of the existing alignment in the area of the proposed bridge and will carry two-lanes in each direction over Highway 401. It is also understood that, along with the additional lanes for Highway 401, the existing E-NS ramp and the NS-W ramps on the north side of Highway 401) will be realigned and will extend to within the proposed underpass. The proposed bridge will be a two-span structure about 98 m in length with the finished deck at about Elevation 105 m at the north limit and about Elevation 104 m at the south limit. Based on the information provided, the final grade of Highway 401 at the bridge structure will remain unchanged at about Elevation 95 m and the proposed grade of Montreal Street will be approximately 1 m and 2.5 m above the existing ground surface on the north and south sides of the structure, respectively. The west limit of the proposed bridge will be about 6 m from the east limit of the existing bridge. As such, the potential impact on the existing structure during construction of the proposed structure has been reviewed to assess if measures are necessary to maintain the integrity of the existing foundations and embankments.

Shallow (less than 1.2 m) fill layers and overburden deposits overlie the bedrock at this site, except at the location of the proposed centre pier where about 2 m of granular fill was encountered above the bedrock surface. The bedrock surface is at about Elevation 104 m on the tableland on the north side of Highway 401 and at about Elevations 100.5 m to 101.5 m on the tableland on the south side of Highway 401. Bedrock was encountered at about Elevation 93 m (2 m depth) within the floor of the rock cut in the median area. The bedrock encountered in the boreholes consists of fresh, fine-grained, moderately strong limestone of generally good to excellent quality. The groundwater table was encountered at about Elevation 97.7 m (6.2 m depth) on the north side of Highway 401 and at about Elevation 100 m (2.5 m depth) on the south side of Highway 401, and at about Elevation 93 m (2 m depth) in the median area.



6.2 Bridge Foundation Options

The following options have been considered for the foundations of the new bridge abutments and piers:

- Shallow foundations (i.e., spread footings) bearing on or within the limestone bedrock; and,
- Deep foundations (cast-in-place concrete caissons) embedded or socketted in the limestone bedrock (at pier locations only).

Geotechnical recommendations for the design of the foundations for bridge abutments and piers are presented in the following sections. A summary comparison of the advantages, disadvantages, relative costs, and risks associated with the foundation options is presented in Table 1 following the text of this report.

6.2.1 Shallow Foundations

The use of spread footings placed on or within the fresh limestone bedrock at the site is considered appropriate for support of the abutments and centre piers. The north and south abutment footings may be founded at or below Elevations 103 m and 100 m, respectively. In the case of the centre pier, the footing may be placed directly on the bedrock surface after excavation of the overlying fill and any loose or fractured rock at about Elevation 93 m.

It is understood that the preferred foundation concept is to maintain the founding level of the bridge abutments above the Highway 401 grade in a 'perched' condition. For the 'perched' abutment configuration (i.e., founding level maintained above the grade of the widened Highway 401 at about Elevation 95 m) rock excavation will be carried out below and in proximity to the founding levels. The rock excavation for the general cut should be made before construction of the footings. In order to maintain the integrity of the bedrock underlying the new and existing foundation, special precautions including controlled blasting (pre-shearing or cushion blasting) will be required for the general road cut where it is in close proximity to the footings. Good perimeter blasting control is a key to achieve the required competence of the foundations for the abutments, and as such, a non-standard special provision has been included for perimeter wall control blasting near new and existing bridge foundations.

Bedrock excavation is required for the footing construction at both abutments and may be required at the central piers. The extent of excavation will depend on the chosen founding level. The excavation could be carried out using drilling and hoe ramming techniques where relatively shallow depths of cut into the bedrock are required, however this procedure can result in a very uneven founding surface. Line drilling and pre-shearing techniques would be the preferred approach where deeper excavation into the bedrock is required for footing construction.

All loose or fractured bedrock at the founding level should be removed prior to placing concrete. In addition, the design for the pier footing should be flexible enough to allow for some variation in the bedrock surface elevation. The supply and placement of a working slab to raise the grade to the founding level and protect the founding bedrock after exposing the bedrock and removing any loosened/fractured bedrock, if required, should be in accordance with the Non-Standard Special Provision in Appendix B.

The contract documents should contain the MTO Special Provision SP902S01 - Excavation and Backfilling - which contains reference to the use of a Quality Verification Engineer to inspect the foundation area prior to footing construction. All footing excavations should be inspected prior to placing concrete to ensure that the base has been adequately cleaned and that the bedrock conditions as exposed at the founding level are consistent with the design assumptions. All loose or shattered rock within the footprint of the footings should be removed from the base of the excavation and replaced with concrete.



6.2.1.1 Limit States Factored Geotechnical Resistance

Spread footings placed on the limestone bedrock at this site may be designed for a factored geotechnical resistance at Ultimate Limit States (ULS) of 5 MPa. This value is for vertical concentric loads only. Effects of load inclination and eccentricity need to be taken into account as appropriate in accordance with Section 6.7.4 of the *Canadian Highway Bridge Design Code (CHBDC)* using the curve for "cohesive soil or rock". Serviceability Limit States (SLS) conditions do not apply to footings placed on the limestone bedrock which is classified as non-yielding.

The factored geotechnical resistance value given above assumes that controlled blasting for road widening and footing construction is used, that the bedrock at and below the founding level has not been fractured by the blasting and that no adverse jointing is present below the footings. As such, an inspection of the widened rock cut should be carried out prior to construction. If the abutment footings are located within the zone defined by a line drawn at 1 horizontal to 1 vertical upwards from the base of the cut, the cut slopes should be inspected by a qualified rock engineer. Additional rock reinforcement in the form of rock bolts or dowels and/or protection in the form of shotcrete may be required before the footings are constructed in order to ensure the integrity of the rock mass. In all cases, a minimum edge distance between the crest of the rock cut and the abutment footing of 2 m should be maintained.

6.2.1.2 Resistance to Lateral Loads

Resistance to lateral forces / sliding resistance between the concrete footings and bedrock should be calculated in accordance with Section 6.7.5 of the *CHBDC*. The coefficient of friction, $\tan \delta$, may be taken as 0.7 for cast-in-place concrete footings constructed on bedrock. This represents an unfactored value; in accordance with the *CHBDC*, a resistance factor of 0.8 is to be applied in calculating the horizontal resistance.

If necessary, sliding resistance can be supplemented by doweling the footings into bedrock. The horizontal resistance of the dowels will be dependent on the strength of the bedrock, grout and steel. The dowels may be designed based on a factored lateral resistance for the rock mass at ULS of 5 MPa. The dowels should have a minimum embedded length of 1.0 m within the bedrock, and the structural strength of the dowel and of the grout should not be exceeded. If required, a Non-Standard Special Provision for dowels in rock should be included in the contract documents. A sample has been included in Appendix B of this report

For uplift of the dowels, a factored value of 400 kPa may be assumed for the grout-to-rock bond stress for ULS design. The actual bond stress along the rock-grout interface may vary from the design value given and it should therefore be verified in the field by pull-out testing. In this case, a Special Provision will have to be included in the Contract Documents to cover this testing.

6.2.1.3 Frost Protection

For spread footings placed on fresh limestone bedrock or mass concrete, frost protection cover is not required.

6.2.2 Caisson Foundations

Caissons founded on or socketed into the bedrock may be used for support of the centre bridge piers where site constraints limit the size or positioning of conventional spread footings.

To advance the caissons with minimal ground loss, a liner or casing may be used within the upper 2 m of overburden soils. It will be difficult to clean the bedrock surface, even with the use of liners, unless the liner extends into the bedrock to prevent the flow of overburden soils under the casing at the interface with the



bedrock. It may therefore be more practical to socket the caissons into the rock rather than founding them directly on the bedrock surface.

If socketing the caissons into the bedrock is required, sockets will have to be advanced by rock coring or churn drilling. Drilling operations for each caisson should be monitored by qualified personnel to record the rate of penetration, nature of the rock cuttings, quality of the sidewalls and bottom, and placement of the concrete into the drilled shaft.

The upper 0.3 m of the bedrock may be more fractured than the remainder of the rock mass due to excavation and therefore, it is recommended that the upper 0.3 m of the bedrock be discounted when calculating the required socket length. Caissons should bear on slightly weathered to fresh bedrock. Socketing the caisson a minimum of 1 m into rock will also ensure good development of lateral bearing resistance and higher axial resistance.

6.2.2.1 Axial Geotechnical Resistance

Caissons founded on the surface of the limestone bedrock, or socketed nominally (less than 1 m) into the bedrock, should be designed based on end-bearing resistance and a factored geotechnical resistance at ULS of 5 MPa. Where caissons extend to depths greater than 1 m into bedrock, a factored geotechnical resistance at ULS of 7 MPa may be used. SLS resistances do not apply to caissons founded on or socketed in the limestone bedrock, as the SLS resistance for 25 mm of settlement is greater than the factored axial geotechnical resistance at ULS. End bearing for the caisson relies solely on the quality of the rock surface at the base of the excavation. As such, it is imperative that the rock surface be adequately cleaned of loose soils, rock, and debris prior to construction of the caisson.

In such cases where the basal rock surface cannot be adequately cleaned, the caissons at these locations can be designed solely on shaft resistance within the bedrock. Where suitable end bearing cannot be achieved, a factored geotechnical side-wall shaft resistance at ULS of 500 kPa in competent bedrock (i.e., RQD greater than 75 percent) may be used for compression. For uplift, a factored geotechnical side-wall shaft resistance at ULS of 375 kPa may be used.

Minimal additional loading (i.e., site fills) is expected at the location of caissons and, as such, no downdrag forces are expected at this site.

6.2.2.2 Resistance to Lateral Loads

The factored lateral resistance (ULS) for caissons in slightly weathered to fresh limestone is 5 MPa using a passive horizontal resistance factor of 0.5. The resistance to lateral loading from the granular fills in front of the caissons should be ignored considering their limited thickness and stiffness contrast with the underlying bedrock.

Group action for lateral loading should be considered when the spacing in the direction of loading is less than six to eight caisson diameters. Group action can be evaluated by reducing the lateral resistance in the direction of loading by a reduction factor as follows:



Caisson Spacing in Direction of Loading (d = Caisson Diameter)	Reduction Factor
8d	1.0
6d	0.7
4d	0.4
3d	0.25

6.2.2.3 Frost Protection

Below grade caissons with pier columns socketed directly into fresh limestone bedrock do not require frost protection cover.

6.3 Feasibility of Integral and Semi-Integral Abutments

6.3.1 Integral Abutments

As outlined in MTO's report SO-96-01, integral abutment bridges are single span or multiple span continuous deck type bridges with a movement system composed primarily of abutments on flexible integral foundations and approach slabs, in lieu of movable deck expansion joints and bearings at abutments. The feasibility of integral abutments is influenced by a number of factors including geometry and subsurface conditions. The primary criterion is the need to support the abutments on relatively flexible piles. Where the load bearing stratum is near the surface or where the use of short piles or caissons (less than 5 m in length) is planned, the site is not considered suitable for integral abutment bridges. Geometric constraints on the use of integral abutments are also applicable and include: overall bridge length less than 150 m; skew angle less than 35°; and abutment wall heights less than 6 m without a retained soil system.

Given the proximity of bedrock to surface at both the north and south abutments and the overall skew of the bridge, integral abutments are not considered to be feasible at this site.

6.3.2 Semi-Integral Abutments

As outlined in MTO's report BO-99-03, semi-integral abutment bridges are single or multiple span structures of less than 150 m in length with rigid foundations (spread footings) where the concrete deck is continuous with the approach slabs. Expansion joints are eliminated at the end of the deck and the superstructure is supported on movable bearings and is almost independent of the abutment. A expansion joint is provided at the end of the approach slab that is detailed to slide between or over the wingwalls. Unlike integral abutment bridges, there is no limit on skew angle for semi-integral abutments provided that lateral restraint is incorporated in the bridge design to prevent rotation of the superstructure caused by eccentric lateral earth pressures in the horizontal plane acting on both ends of the superstructure and that the movement system at the end of the approach can accommodate deformations associated with skew.

The use of semi-integral abutments would be feasible for the north and south abutments, where the ground conditions allow for support of the structure on rigid spread footings founded on bedrock. If lateral forces due to earth pressures cause undue pressure on the abutments, consideration could be given to the installation of retained soil systems (geogrids). Given that the grades of the approaches are relatively close to existing ground surface and that abutment retaining walls are expected to be less than 3 m in height, we do not expect that retained soil systems will be required.



We understand that lateral movement in the order of +/- 15 mm can be expected at the back of the deck diaphragm due to thermal expansion and contraction. In order to accommodate this movement, we recommend the installation of a minimum of 1.5 m width of compacted granular material behind the deck diaphragm.

6.4 Site Coefficient

For seismic design purposes, the Site Coefficient, S , for the site in accordance with Section 4.4.6 of the CHBDC may be taken as 1.0, consistent with Soil Profile Type 1.

6.5 Lateral Earth Pressures for Design

The lateral pressures acting on the bridge abutments and piers will depend on the type and method of placement of the backfill materials, on the nature of the soils/bedrock behind the backfill, on the magnitude of surcharge including construction loading, on the freedom of lateral movement of the structure, and on the drainage conditions behind the walls. Seismic (earthquake) loading must also be taken into account in the design.

The following recommendations are made concerning the design of the abutments and piers in accordance with CHBDC:

- Select free-draining granular fill meeting the specifications of Ontario Provincial Standard Specifications (OPSS) Granular 'A' or Granular 'B', Type II but with less than 5 per cent passing the 200 sieve should be used as backfill behind the walls. This fill should be compacted in accordance with MTO's Special Provision 105S10;
- Longitudinal drains and weep holes should be installed to provide positive drainage of the granular backfill. Other aspects of the abutment granular backfill requirements with respect to sub-drains and frost taper should be in accordance with OPSD-3101.150 and 3121.150;
- The granular fill may be placed either in a zone with width equal to at least 1.5 m behind the back of the stem (Case (a) in Figure C6.20 of the Commentary to the CHBDC) or within the wedge-shaped zone defined by a line drawn at 1.5 horizontal to 1 vertical (1.5H:1V) extending up and back from the bottom of the rear face of the footing (Case (b) in Figure C6.20 of the Commentary to the CHBDC) in soil cuts;
- Where walls abut a vertical face of bedrock, the granular fill may be placed in a zone with width equal to at least 1.5 m behind the back of the stem (Case (a) in Figure C6.20 of the Commentary to the CHBDC).
- A minimum compaction surcharge equal to 12 kPa should be included in the lateral earth pressures for the structural design of the abutment walls, in accordance with CHBDC Section 6.9.3 and Figure 6.6. Compaction equipment should be used in accordance with MTO's Special Provision 105S10. Other surcharge loadings should be accounted for in the design, as required;

6.5.1 Static Lateral Earth Pressures for Design

- For Case I, the pressures are based mainly on the overburden materials. For Case II, the pressures are based on granular fill;
- The following unfactored lateral earth pressure parameters may be assumed for static design:

**STATIC LATERAL EARTH PRESSURE COEFFICIENTS**

	Case I	Case II	
	Overburden	Granular 'A'	Granular 'B' Type II
Soil Unit Weight:	21 kN/m³	22 kN/m³	21 kN/m³
Active, K_a	0.33	0.27	0.27
At rest, K_0	0.50	0.43	0.43
Passive, K_p	3.0	3.7	3.7

- If the wall support and superstructure allow lateral yielding or where the abutments are expected to move away from the retained soils as the superstructure contracts due to decreases in ambient temperature, active earth pressures may be used in the geotechnical design of the structure. The movement to allow active pressures to develop within the backfill, and thereby assume an unrestrained structure, may be taken as:
 - Rotation of approximately 0.002 about the base of a vertical wall;
 - Horizontal translation of 0.001 times the height of the wall; or,
 - A combination of both.
- If the abutment support does not allow lateral yielding (i.e., restrained structure where the rotational or horizontal movement is not sufficient to mobilize an active earth pressure condition), at-rest pressures (plus any compaction surcharge) should be assumed for geotechnical design.
- Where abutments allow lateral yielding into the retained soils, such as at semi-integral abutments where increases in ambient temperature cause expansion of the superstructure, passive earth pressures should be used in the geotechnical design. The movements required to fully mobilize passive pressure or resistance are much larger than those required to mobilize active pressure. In practice, movements may not be sufficient to mobilize the full passive resistance. The movement to allow passive pressures to develop within the backfill may be taken as:
 - Rotation of approximately 0.100 about the base of the vertical wall;
 - Rotation of approximately 0.020 about the top of a vertical wall;
 - Horizontal translation of 0.05 times the height of the wall; or,
 - A combination of the above.
- Where movements are not great enough to mobilize full passive resistance, K_p may be determined in accordance with Figure C6.16 of the *CHBDC Commentary* based on the amount of displacement.



6.5.2 Seismic Lateral Earth Pressure Design

Seismic (earthquake) loading must be taken into account in the design in accordance with Section 4.6 of the CHBDC. In this regard, the following should be included in the assessment of lateral earth pressures:

- Seismic loading will result in increased lateral earth pressures acting on the walls. The walls should be designed to withstand the combined lateral loading for the appropriate static pressure conditions given above, plus the earthquake-induced dynamic earth pressure. According to the CHBDC, the site-specific zonal acceleration ratio for Kingston is 0.1. Based on experience, for the subsurface conditions at this site, no significant amplification of the ground motion is expected. The seismic lateral earth pressure coefficients given below have been derived based on a design zonal acceleration ratio of $A = 0.1$;
- In accordance with Sections 4.6.4 and C.4.6.4 of the CHBDC and its *Commentary*, for structures which do not allow lateral yielding, the horizontal seismic coefficient, k_h , used in the calculation of the seismic active pressure coefficient is taken as 1.5 times the zonal acceleration ratio (i.e., $k_h = 0.15$). For structures which allow lateral yielding, k_h is taken as 0.5 times the zonal acceleration ratio (i.e., $k_h = 0.05$). The vertical seismic coefficient, k_v , used in the calculation is assumed to range from -0.5 to 0.5 times the horizontal seismic coefficient, k_h ;

The following seismic active pressure coefficients (K_{AE}) for the two backfill cases (Case I and Case II) may be used in design.

SEISMIC ACTIVE EARTH PRESSURE COEFFICIENTS, K_{ae}

	Case I	Case II	
		Granular 'A'	Granular 'B' Type II
Yielding wall	0.34	0.28	0.28
Non-yielding wall	0.46 ⁽¹⁾	0.38 ⁽¹⁾	0.38 ⁽¹⁾

¹ For non-yielding walls only static earth pressures need to be considered for this low seismicity ($A=0.1$) location.

- The above K_{AE} values for yielding walls are applicable provided that the wall can move up to 250 A (mm), where A is the design zonal acceleration ratio of 0.10 during the seismic event. This corresponds to displacements of up to approximately 25 mm at this site; and,
- The earthquake-induced dynamic pressure distribution, which is to be added to the static earth pressure distribution, is a linear distribution with maximum pressure at the top of the wall and minimum pressure at its toe (i.e., an inverted triangular pressure distribution). The total pressure distribution (static plus seismic) may be determined as follows:

$$\sigma_h(d) = K_a \gamma d + (K_{AE} - K) \gamma (H-d)$$

Where: $\sigma_h(d)$ Is the (static plus seismic) lateral earth pressure at depth, d, (kPa);

K Is either the static active earth pressure K_a , or the static at-rest earth pressure coefficient (K_0);

K_{AE} Is the seismic active earth pressure coefficient;

γ Is the unit weight of the backfill soil (kN/m^3), as given previously;



d Is the depth below the top of the wall (m); and,

H Is the total height of the wall (m).

The following dynamic increment of passive pressure ($K_{PE}-K_P$) may be used in the design of the bridge deck ends for semi-integral abutments. These coefficients represent the maximum ($K_{PE}-K_P$) value obtained using the k_h and three values of k_v as described above and assuming that movements are sufficient to mobilize full passive resistance.

SEISMIC PASSIVE EARTH PRESSURE COEFFICIENTS, ($K_{PE}-K_P$)

	Case I	Case II	
		Granular 'A'	Granular 'B' Type II
Yielding wall	0.1	0.1	0.1
Non-yielding wall	0.3	0.3	0.3

- Where seismic movements are not great enough to mobilize full passive resistance, K_P may be determined in accordance with Figure C6.16 of the CHBDC Commentary based on the amount of displacement. The dynamic increment of passive pressure ($K_{PE}-K_P$) should be reduced to the same extent as K_P .
- The earthquake-induced dynamic passive lateral pressure distribution, which is to be subtracted from the static earth pressure distribution, is a linear distribution with maximum pressure at the base of the wall and minimum pressure at its top (i.e., a triangular pressure distribution). The total pressure distribution (static plus seismic) may be determined as follows:

$$\sigma_h(d) = K_P \gamma d - (K_{PE} - K_P) \gamma (H-d)$$

Where: $\sigma_h(d)$ is the total lateral earth pressure at depth, d, (kPa);

K_P is the static passive earth pressure K_P ;

$K_{PE} - K_P$ is the dynamic increment of passive earth pressure coefficient;

γ is the unit weight of the backfill soil (kN/m^3), as given previously;

d is the depth below the top of the wall (m); and,

H is the total height of the wall (m).

It should be noted that these seismic earth pressure coefficients assume that the back of the wall is vertical and the ground surface behind the wall is flat. Where sloping backfill is present above the top of the wall, the lateral earth pressures under seismic loading conditions should be calculated by treating the weight of the backfill located above the top of the wall as a surcharge.



6.6 Design and Construction Considerations

6.6.1 Temporary Excavations

The excavations for abutment footing construction will extend through surficial layers of sand and gravel fill and / or silty clay, and into the limestone bedrock. If the central piers are supported by conventional spread footings, the excavation will extend about 2 m through granular fills ranging from silty sand to sand and gravel with crushed limestone and terminate on the bedrock surface.

The groundwater level in the piezometers installed in Boreholes 3-2 at the north abutment was measured at a depth of 6.2 m (Elevation 97.7 m) and in Borehole 3-4 at the south abutment at a depth of 2.5 m (Elevation 100.0 m), on May 21, 2000. Where the abutment founding level is below the groundwater table, groundwater inflow to the excavation will be controlled by the presence of fractures within the bedrock. The groundwater level was observed at about 2 m depth (Elevation 93 m) in Borehole 3-7 at the centre pier upon completion of drilling, which corresponds to the top of bedrock. Groundwater inflow to the excavation should be expected at the overburden / bedrock interface. Some form of groundwater control may be required in order to construct the footings in the dry; however, it is considered that the anticipated inflow can be handled by conventional sump pumping at the base of the excavation. Sumps should be maintained outside the footing area. Surficial water should be directed away from the excavations. An NSSP for the control of overburden soils, surface water and groundwater during foundation installation should be included in the contract documents and a sample is included in Appendix B.

Excavations extending through the soils which will be open for a relatively short period of time can be made using temporary unsupported cut with side slopes within the overburden deposits maintained not steeper than 1 horizontal to 1 vertical. Temporary excavations for footing construction extending through the bedrock may be completed using vertical sides. Adjacent to semi-integral abutments, rock cuts should be offset a minimum of 1.5 m from the end of the diaphragm to allow for sufficient contraction (i.e.; 15 mm) of granular materials.

For the pier footing construction, temporary excavation support may be required due to space restrictions. It is considered that the temporary support system could consist of internally braced soldier piles and lagging; the internal bracing may reduce the need for sockets and tie-backs which would have to be extended into the bedrock. Support to the caisson toes, if required, could be provided with the use of rock bolts. The recommendations given in Section 6.2.1.2 may be used for the design of the rock bolts. A coefficient of lateral earth pressure of 0.28 and a soil unit weight of 21 kN/m³ with a rectangular pressure distribution may be used for the design of the temporary support system. Traffic loading should be included as a surcharge.

Roadway protection should be included in the tender documents. Temporary excavation support systems should be designed and constructed in accordance with OPSS 539. The lateral movement of the temporary shoring system should meet Performance Level 2.

All excavations should be carried out in accordance with the guidelines outlined in the latest edition of the Ontario Occupational Health & Safety Act. The native soils at this site would be classified as Type I soil.



6.6.2 Permanent Cut Slopes

For permanent cut slopes through the bedrock, such as those required for the approach E-N/S and N/S-W ramps, the overall slope to the cut face may be formed vertical to near vertical (i.e., 1 horizontal to 12 vertical) and constructed in accordance with MTO's Special Provision 206S03 for Rock Faces. The use of carefully controlled drill and blast excavation techniques will be required in order to ensure a neat excavation line and minimize face instabilities and long-term maintenance problems. Alternatively, the rock faces could be excavated mechanically using large hydraulic rock breakers as was done for the section of widening along Highway 401 to the west of Montreal Street. Line drilling of the rock face prior to mechanical excavation could be used to produce a neat face with minimal overbreak. Regardless of the method of excavation, mechanical scaling will be required to remove loose rock on the face which may be created due to the blocky nature of the rock mass and the presence of joint sets sub-parallel to the cut face. It is also likely that there will be some overbreak associated with the rock faces due to the joint sets that strike sub-parallel or obliquely to the faces.

In terms of kinematic stability, wedge and planar type failures are unlikely due to the typically sub-vertical joint sets. During excavation of the new Highway 401 road cuts there is a slight possibility of toppling failure since the J3 set is nearly parallel to the orientation of the cut. However, it is likely that most unstable blocks would be dislodged during blasting, and that prominent J3 sets will control overbreak where present. Stability of the ultimate rock slopes will be affected by any blasting induced damage.

The main mechanisms for instability on the existing rock faces are ravelling of loose surficial blocks of rock and the creation of overhangs which can eventually result in undercut blocks of rock falling. The rock falls on the northeast and southeast cuts are mainly the result of poor blasting practices during original construction which have damaged (fractured) the rock face and ongoing weathering processes, predominantly ice jacking due to freeze thaw cycles in the winter months which tends to loosen blocks on the face. When the loosened blocks eventually fall, they sometimes create an overhang above. Eventually the overhanging blocks may also fall due to further weathering. There are also numerous 25mm to 200mm diameter tree stumps above the crests of the rock cuts. The roots of these trees can grow inside the joints in the rock mass forcing the joints open and in some cases eventually creating unstable blocks.

To minimize the risk of undercutting the toe of rock cuts, we recommend that a 0.5 m offset be maintained between the base of the rock cut and the outside edge of the ditch. Alternatively, if space limitations preclude this offset, consideration could be given to lining the ditch with shotcrete.

6.6.3 Blasting Considerations

The use of controlled blasting techniques in accordance with OPSS 120 may be used for bedrock excavation for the new ramps and road widening along this section of Highway 401. Within 50 m of the new foundation areas and existing bridge structure, a separate non-standard special provision should be included to reinforce the need to minimize damage to the rock face, overbreak and fly rock adjacent to the existing and new structures. A sample non-special standard provision had been prepared and is included in Appendix B.

Above and beyond OPSS 120, the Special Provision includes requirements for:

- Submission of a separate perimeter wall control blast design by the blasting contractor or their blast consultant in accordance with OPSS 120 detailing the proposed blast methodology for perimeter wall control blasting within 50 m of new and existing bridge foundations;



- Separate trial blasts using perimeter wall control blast procedures prior to blasting within 50 m of new and existing structures; and,
- Acceptance of the perimeter wall control blasting methodology by the Contract Administrator following demonstration that the blast design is adequate to minimize damage to the rock face, overbreak and fly rock.

Inspection of the rock cut face immediately after blasting should be carried out by qualified geotechnical personnel retained by the contract administrator in order to assess where scaling / loosened rock removal should be carried out adjacent to the footings and where additional rock bolting may be required. The rock bolts, if required, should be 25 mm diameter, galvanized, fully grouted deformed bars, generally 3 m in length.

6.6.4 Approach Embankments

Based on the information provided, the proposed Montreal Street grade will be at about Elevation 105 m on the north side and about Elevation 104 m on the south side. An approach embankment of about 2.5 m in height will be required on the south side and about 1 m on the north side.

Based on the subsurface information obtained, the subsurface materials along the south approach embankment consists of up to 1 m of firm to stiff silty clay, which is underlain by bedrock at about Elev. 101 m. At the north abutment, the subsurface materials consist of up to 0.7 m of granular fills overlying 0.6 m of stiff silty clay, which in turn is underlain by bedrock.

Given the above, stability of the proposed embankments is not a concern with respect to deep seated failure through the founding soils. It is recommended that the surficial soils which contain organics will be removed where present at ground surface prior to placement of embankment fill. Settlement of the embankment due to consolidation of the silty clay deposit encountered along the south approach embankment is expected to be minimal.

6.6.5 Subgrade Preparation and Embankment Construction

Any surficial topsoil, organic matter and softened/loosened soils should be stripped from within the limits of fill embankment areas. All subgrade soils should be proof-rolled prior to fill placement in accordance with MTO's Special Provision 105S10.

Construction of the embankment above the prepared subgrade may be carried out using clean earth or rock fill (in accordance with OPSS 212) or Select Subgrade Material (in accordance with OPSS 1010) or rock fill, depending on material availability. Embankment fill (clean earth fill or SSM) should be placed in regular lifts with a loose thickness not exceeding 300 mm, and be compacted to at least 95 percent of the material's Standard Proctor maximum dry density in accordance with MTO's Special Provision 105S10. The final lift prior to placement of the granular subbase or base courses should be compacted to 100 percent of the Standard Proctor maximum dry density. Inspection and field density testing should be carried out by qualified geotechnical personnel during all fill placement operations to ensure that appropriate materials are used and that adequate levels of compaction have been achieved. Rock embankments should be constructed and compacted in accordance with SP206S03.



For semi-integral abutments, rock fill should not be placed within the active wedge zone. Rock fill contains numerous voids into which finer material can migrate due to water action and/or repeated loading. To limit potential settlement resulting from the migration of finer material, a filter material is required at the transition between the rock fill and the abutment backfill or other earth embankment fill. Granular B Type II (OPSS 1010) meets the criteria for filtration and drainage and therefore could be used as backfill to the abutment to transition between rock fill and earth fill embankment.

The permanent slopes of the embankment should be maintained not steeper than 2 horizontal to 1 vertical (2H:1V) if constructed of earth fill or 1.25H:1V if constructed with rock fill.

To reduce surface water erosion on the embankment side slopes constructed of earth fill, placement of topsoil and seeding or pegged sod is recommended.



7.0 CLOSURE

This report was originally prepared in October, 2000 by Mr. Dan K. Breeze, B.Sc., under the direction of the Project Manager, Ms. Anne S. Poschmann, P. Eng.. Mr. Fintan J. Heffernan, Golder's Designated MTO Contact for this project, conducted a technical and independent quality control of the report. As requested by MRC, Golder has updated this report to reflect changes to the CHBDC, to include potential changes in the methodology for installation of centre piers, a discussion on semi-integral abutments, and seismic design parameters. Updates to this report were prepared by Ms. Erin S. O'Neill, P. Eng., under the direction of Project Manager, Mr. Michael Snow, P. Eng.. Mr. Fintan J. Heffernan, Golder's Designated MTO Contact for this project, also conducted a technical and independent quality control of the updated report.

Yours truly,

GOLDER ASSOCIATES LTD.

Erin S. O'Neill, P.Eng.
Geotechnical Engineer

Fin H. Heffernan, P.Eng.
Designated MTO Contact

ESO/MSS/cg/am

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Michael S. Snow, P.Eng.
Principal





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Yours truly,

GOLDER ASSOCIATES LTD.

Erin S. O'Neill, P.Eng.
Geotechnical Engineer

Fin H. Heffernan, P.Eng.
Designated MTO Contact

ESO/MSS/cg/am

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Michael S. Snow, P.Eng.
Principal





View looking east from the south side of the Hwy 401-Montreal Street Underpass



Example of overbreak of the face due to sub-parallel joint sets



Example of blocky rock face and ravelling failures



Example of overhangs resulting in gravity falls



Example of minor seepage and small overhangs



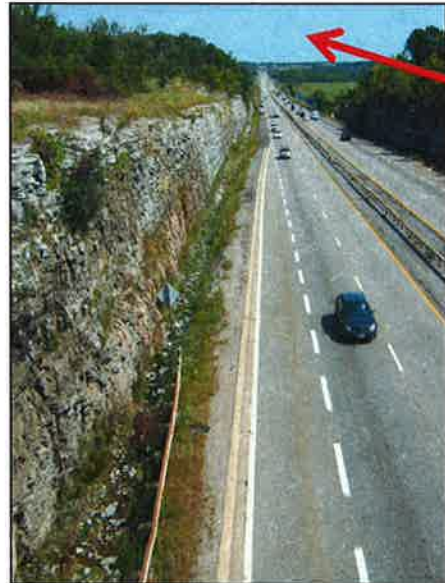
Example of tree stumps jacking blocks open on the crest of the face



Example of seepage from a layer about 4m above the highway; blocky and ravelling face, minor overhangs.



Significant overhangs (1-2m)



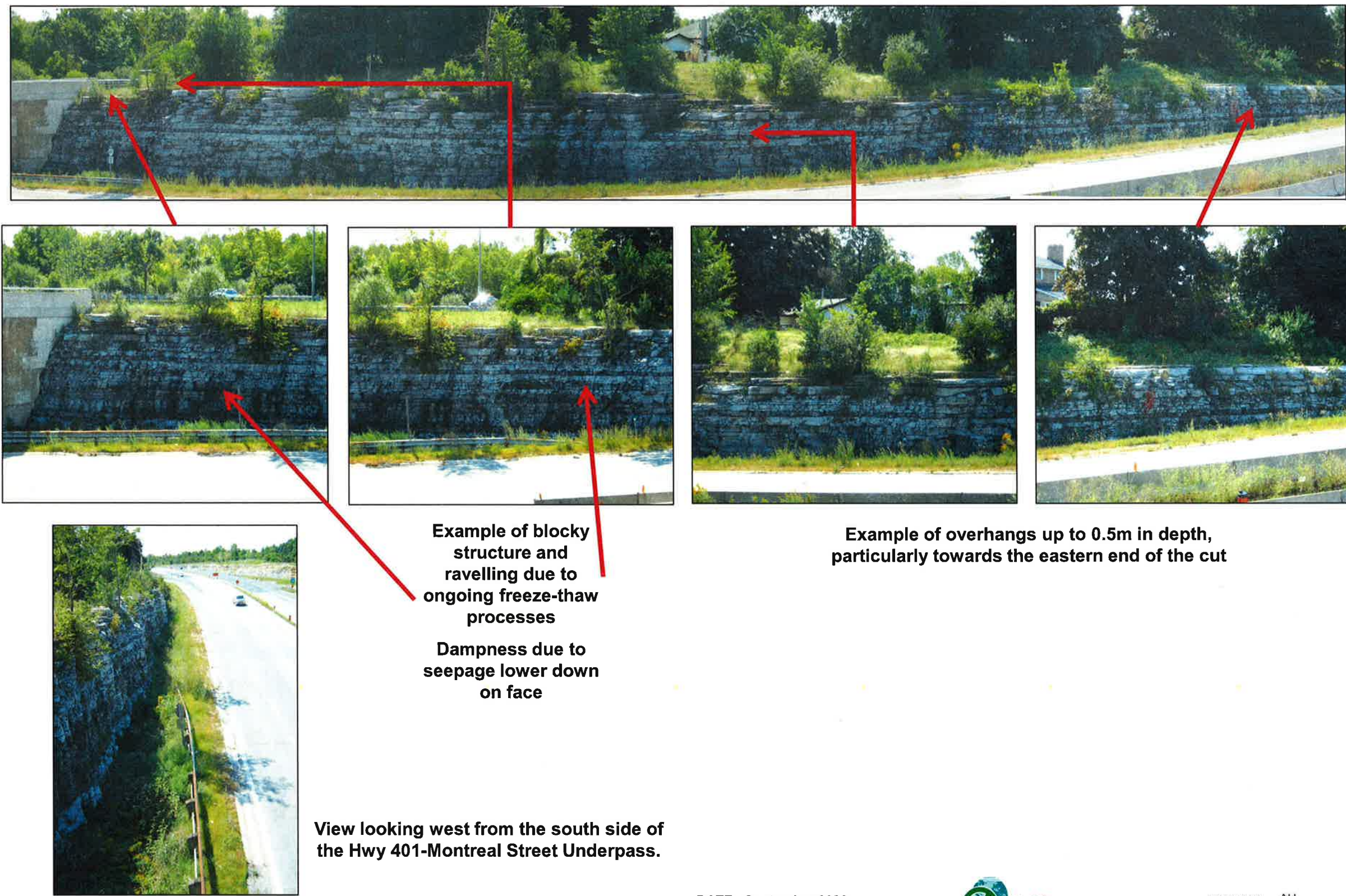
View looking east from the north side of the Hwy 401-Montreal Street Underpass.

East end of Cut: example of ravelling due to ongoing freeze-thaw processes



Looking west from east end of cut: Note overhangs due to gravity falls







Example of blocky structure and ravelling due to ongoing freeze-thaw processes



View looking west from the north side of the Hwy 401-Montreal Street Underpass.



Example of saw tooth profile of rock face due to overbreak along sub-parallel joint sets Minor overhangs



Three near vertical joint sets clearly visible on the crest above the cut. Along with near horizontal bedding, defines the blocky structure of the rock mass.

G.W.P 78-99-00

Highway 401 Expansion
Montreal Street Ramp N/S-W

Figure 5



Looking West toward
Hwy 401



DATE September 2009
PROJECT 08-1111-0044



DRAWN...AH.....
CHKD...MJT.....



Example of very blocky face, minor ravelling, overhangs < 0.5m



5-5.5m
typical crest
height

Example of very Blocky, minor ravelling, and overhangs < 0.5m



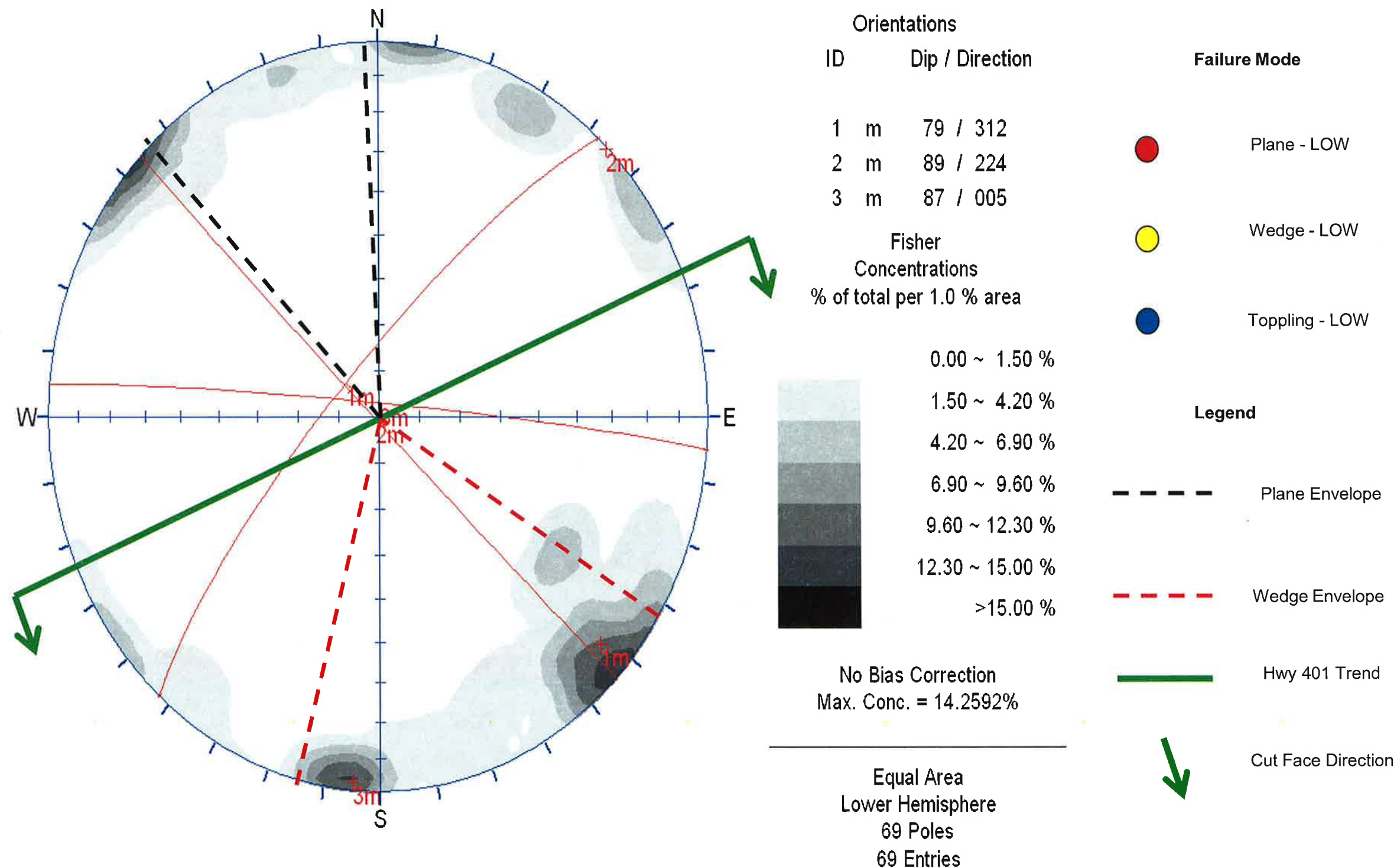
Example of blocky face due to poor blasting practices resulting in ravelling and minor overhangs



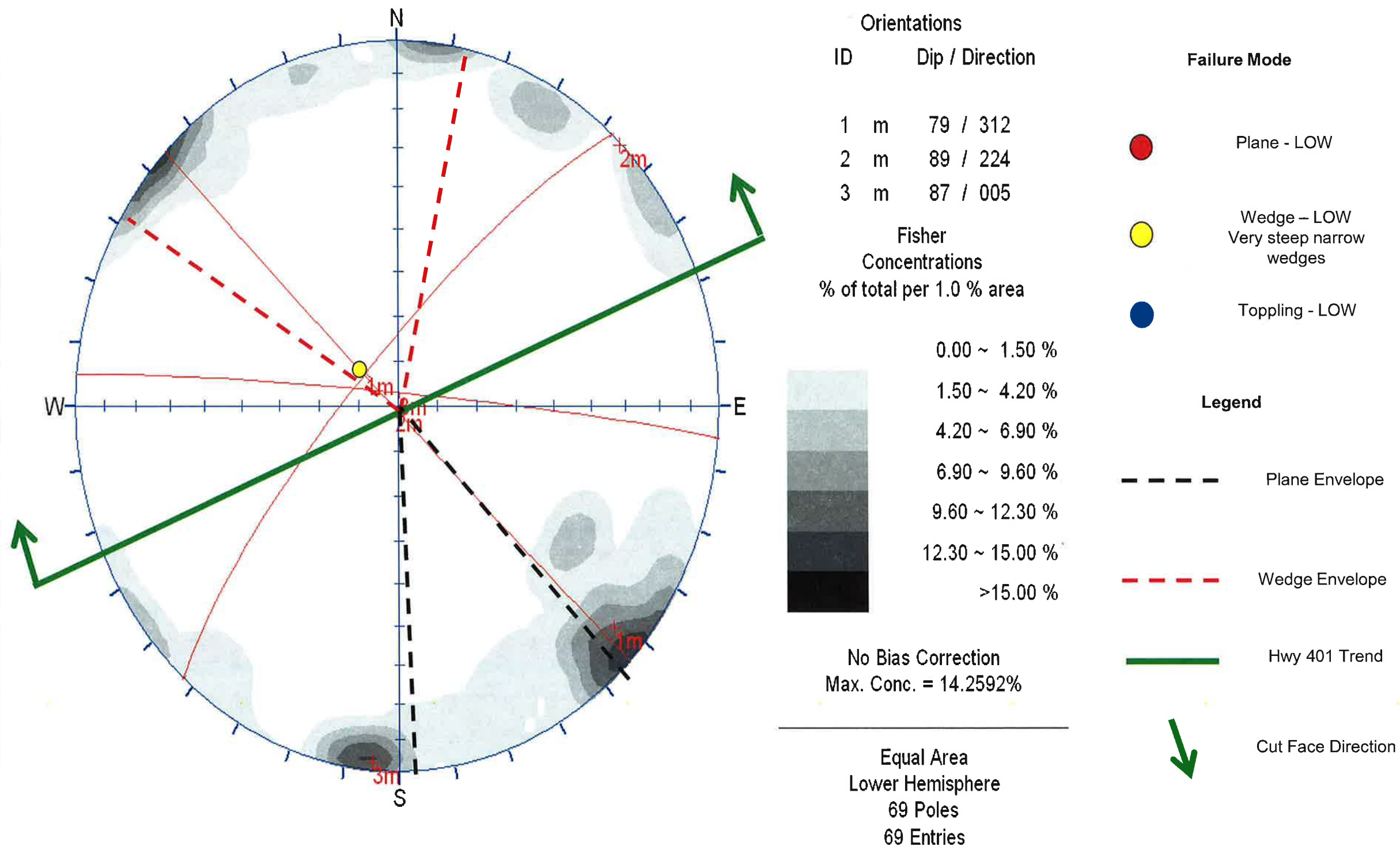
Looking West toward Hwy 401



Looking east toward Montreal Street



NOTE: NEAR HORIZONTAL BEDDING POINTS NOT PLOTTED;
DATA POINTS ARE REFERENCED WITH RESPECT TO TRUE
NORTH





APPENDIX A

List of Abbreviations and Symbols
Lithological and Geotechnical Rock Description Terminology
Record of Borehole and Drillhole Sheets
(Boreholes 3-1 to 3-7 and Drillholes 3-2, 3-4 and 3-7)
From Previous Investigations

LIST OF ABBREVIATIONS

The abbreviations commonly employed on Records of Boreholes, on figures and in the text of the report are as follows:

I. SAMPLE TYPE		III. SOIL DESCRIPTION	
AS	Auger sample	(a)	Cohesionless Soils
BS	Block sample		
CS	Chunk sample		
DO	Drive open	Density Index	N
DS	Denison type sample	(Relative Density)	Blows/300 mm
FS	Foil sample		Or Blows/ft.
RC	Rock core	Very loose	0 to 4
SC	Soil core	Loose	4 to 10
ST	Slotted tube	Compact	10 to 30
TO	Thin-walled, open	Dense	30 to 50
TP	Thin-walled, piston	Very dense	over 50
WS	Wash sample	(b)	Cohesive Soils
DT	Dual Tube sample	Consistency	C _u or S _u
II. PENETRATION RESISTANCE			
Standard Penetration Resistance (SPT), N:		<u>Kpa</u>	<u>Psf</u>
The number of blows by a 63.5 kg. (140 lb.)		Very soft	0 to 12
hammer dropped 760 mm (30 in.) required		Soft	12 to 25
to drive a 50 mm (2 in.) drive open		Firm	25 to 50
Sampler for a distance of 300 mm (12 in.)		Stiff	50 to 100
DD- Diamond Drilling		Very stiff	100 to 200
Dynamic Penetration Resistance; N_d:		Hard	Over 200
The number of blows by a 63.5 kg (140 lb.)			
hammer dropped 760 mm (30 in.) to drive			
Uncased a 50 mm (2 in.) diameter, 60° cone			
attached to "A" size drill rods for a distance			
of 300 mm (12 in.).			
PH:	Sampler advanced by hydraulic pressure	IV. SOIL TESTS	
PM:	Sampler advanced by manual pressure	w	water content
WH:	Sampler advanced by static weight of hammer	w _p	plastic limited
WR:	Sampler advanced by weight of sampler and rod	w _l	liquid limit
		C	consolidation (oedometer) test
		CHEM	chemical analysis (refer to text)
		CID	consolidated isotropically drained triaxial test ¹
		CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement ¹
		D _R	relative density (specific gravity, G _s)
		DS	direct shear test
		M	sieve analysis for particle size
		MH	combined sieve and hydrometer (H) analysis
		MPC	modified Proctor compaction test
		SPC	standard Proctor compaction test
		OC	organic content test
		SO ₄	concentration of water-soluble sulphates
		UC	unconfined compression test
		UU	unconsolidated undrained triaxial test
		V	field vane test (LV-laboratory vane test)
		γ	unit weight

Note:

1. Tests which are anisotropically consolidated prior shear are shown as CAD, CAU.

LIST OF SYMBOLS

Unless otherwise stated, the symbols employed in the report are as follows:

I. GENERAL

π	= 3.1416
$\ln x$	natural logarithm of x
$\log_{10} x$ or $\log x$	logarithm of x to base 10
g	Acceleration due to gravity
t	time
F	factor of safety
V	volume
W	weight

II. STRESS AND STRAIN

γ	shear strain
Δ	change in, e.g. in stress: $\Delta \sigma'$
ε	linear strain
ε_v	volumetric strain
η	coefficient of viscosity
ν	Poisson's ratio
σ	total stress
σ'	effective stress ($\sigma' = \sigma - u$)
σ'_{vo}	initial effective overburden stress
$\sigma_1 \sigma_2 \sigma_3$	principal stresses (major, intermediate, minor)
σ_{oct}	mean stress or octahedral stress $= (\sigma_1 + \sigma_2 + \sigma_3)/3$
τ	shear stress
u	porewater pressure
E	modulus of deformation
G	shear modulus of deformation
K	bulk modulus of compressibility

III. SOIL PROPERTIES

(a) Index Properties

$\rho(\gamma)$	bulk density (bulk unit weight*)
$\rho_d(\gamma_d)$	dry density (dry unit weight)
$\rho_w(\gamma_w)$	density (unit weight) of water
$\rho_s(\gamma_s)$	density (unit weight) of solid particles
γ'	unit weight of submerged soil ($\gamma' = \gamma - \gamma_w$)
D_R	relative density (specific gravity) of solid particles ($D_R = p_s/p_w$) formerly (G_s)
e	void ratio
n	porosity
S	degree of saturation
*	Density symbol is p . Unit weight symbol is γ where $\gamma = pg$ (i.e. mass density \times acceleration due to gravity)

(a) Index Properties (cont'd.)

w	water content
w_L	liquid limit
w_p	plastic limit
I_p	plasticity Index $= (w_L - w_p)$
w_s	shrinkage limit
I_L	liquidity index $= (w - w_p)/I_p$
I_c	consistency index $= (w_L - w)/I_p$
e_{max}	void ratio in loosest state
e_{min}	void ratio in densest state
I_D	density index $= (e_{max} - e)/(e_{max} - e_{min})$ (formerly relative density)

(b) Hydraulic Properties

h	hydraulic head or potential
q	rate of flow
v	velocity of flow
i	hydraulic gradient
k	hydraulic conductivity (coefficient of permeability)
j	seepage force per unit volume

(c) Consolidation (one-dimensional)

C_c	compression index (normally consolidated range)
C_r	recompression index (overconsolidated range)
C_s	swelling index
C_a	coefficient of secondary consolidation
m_v	coefficient of volume change
c_v	coefficient of consolidation
T_v	time factor (vertical direction)
U	degree of consolidation
σ'_p	pre-consolidation pressure
OCR	Overconsolidation ratio $= \sigma'_p / \sigma'_{vo}$

(d) Shear Strength

$\tau_p \tau_r$	peak and residual shear strength
ϕ'	effective angle of internal friction
δ	angle of interface friction
μ	coefficient of friction $= \tan \delta$
c'	effective cohesion
c_u, s_u	undrained shear strength ($\phi=0$ analysis)
p	mean total stress $(\sigma_1 + \sigma_3)/2$
p'	mean effective stress $(\sigma'_1 + \sigma'_3)/2$
q	$(\sigma_1 - \sigma_3)/2$ or $(\sigma'_1 - \sigma'_3)/2$
q_u	compressive strength $(\sigma_1 - \sigma_3)$
S_t	sensitivity

Notes: 1. $\tau = c' + \sigma' \tan \phi'$
 2. Shear strength $= (\text{Compressive strength})/2$

LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY

WEATHERING STATE

Fresh: no visible sign of weathering

Faintly Weathered: weathering limited to the surface of major discontinuities.

Slightly weathered: penetrative weathering developed on open discontinuity surfaces but only slight weathering of rock material.

Moderately weathered: weathering extends throughout the rock mass but the rock material is not friable

Highly weathered: weathering extends throughout rock mass and the rock material is partly friable.

Completely weathered: rock is wholly decomposed and in a friable condition but the rock texture and structure are preserved.

BEDDING THICKNESS

<u>Description</u>	<u>Bedding Plane Spacing</u>
Very thickly bedded	>2 m
Thickly bedded	0.6 m to 2m
Medium bedded	0.2 m to 0.6 m
Thinly bedded	60 mm to 0.2 m
Very thinly bedded	20 mm to 60 mm
Laminated	6 mm to 20 mm
Thinly laminated	<6 mm

JOINT OR FOLIATION SPACING

<u>Description</u>	<u>Spacing</u>
Very wide	>3 m
Wide	1 – 3 m
Moderately close	0.3 – 1 m
Close	50 – 300 mm
Very close	<50 mm

GRAIN SIZE

<u>Term</u>	<u>Size*</u>
Very Coarse Grained	>60 mm
Coarse Grained	2 – 60 mm
Medium Grained	60 microns - 2mm
Fine Grained	2 – 60 microns
Very Fine Grained	<2 microns

Note: *Grains >60 microns diameter are visible to the naked eye.

O:\Templates\Rock Description Terminology

CORE CONDITION

Total Core Recovery

The percentage of solid drill core recovered regardless of quality or length, measured relative to the length of the total core run.

Solid Core Recovery (SCR)

The percentage of solid drill core, regardless of length, recovered at full diameter, measured relative to the length of the total core run.

Rock Quality Designation (RQD)

The percentage of solid drill core, greater than 100 mm length, recovered at full diameter, measured relative to the length of the total core run. RQD varies from 0% for completely broken core 100% for core in solid sticks.

DISCONTINUITY DATA

Fracture Index

A count of the number of discontinuities (physical separations) in the rock core, including naturally occurring fractures but not including mechanically induced breaks caused by drilling.

Dip with Respect to (W.R.T.) Core Axis

The angle of the discontinuity relative to the axis (length) of the core. In a vertical borehole a discontinuity with a 90° angle is horizontal.

Description and Notes

An abbreviated description of the discontinuities, whether naturally occurring separations such as fractures, bedding planes and foliation planes or mechanically induced features caused by drilling such as ground or shattered core and mechanically separated bedding or foliation surfaces. Additional information concerning the nature information concerning the nature of fracture surfaces and infillings are also noted.

Abbreviations

B -	Bedding	Ca -	Calcite
FO -	Foliation/Schistosity	P -	Polished
CL -	Cleavage	S -	Slickensided
SH -	Shear Plane/Zone	SM -	Smooth
VN -	Vein	R -	Ridged/Rough
F -	Fault	ST -	Stepped
CO -	Contact	PL -	Planar
J -	Joint	FL -	Flexured
FR -	Fracture	UE -	Uneven
MF -	Mechanical	W -	Wavy
A -	Angular	C -	Curved
BP -	Bedding Plane	H -	Hackly
BL -	Blast Induced	SL -	Sludge Coated
-	Parallel To	TCA -	To Core Axis
⊥ -	Perpendicular To	STR -	Stress Induced

ON_MOT 001-1119.GPJ ON_MOT.GDT 6/6/00

+ 3, X 3: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

PROJECT 001-1119				RECORD OF BOREHOLE No 3-2				1 OF 1		METRIC				
W.P. 78-99-01		LOCATION N 4804238.00; E 306936.87		ORIGINATED BY SB										
DIST 41 HWY 401		BOREHOLE TYPE 108mm I.D. Solid Stem Augers		COMPILED BY DKB										
DATUM Geodetic		DATE April 19, 2000		CHECKED BY ASP										
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
103.94 0.00	GROUND SURFACE Fresh, light grey to grey, fine grained, moderately strong, stylonitic argillaceous LIMESTONE. Predominantly micritic, some sparitic zones.													
94.77 9.17	Bedrock cored from ground surface to 9.17m depth. For bedrock coring details refer to Record of Drillhole 3-2 END OF HOLE Notes: 1. Water level measured in piezometer at 6.5m depth (El.97.4m) upon completion of installation. 2. Water level measured in piezometer at 8.2m depth (El.97.7m) on May 21, 2000.													

ON MOT 001-1119.GPJ ON MOT.GDT 06/00

PROJECT: 001-1119

RECORD OF DRILLHOLE: 3-2

SHEET 1 OF 1

LOCATION: N 4904239.00; E 306936.87

DRILLING DATE: April 19, 2000

DATUM: Geodetic

INCLINATION: -90°

AZIMUTH: —

DRILL RIG: CME 55 Bombardier

DRILLING CONTRACTOR: Marathon

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	PENETRATION RATE (mm/min)	FLUSH % RETURN	FR-FRACTURE CL-CLEAVAGE SH-SHEAR VN-VEIN	F-FAULT J-JOINT P-POLISHED S-SLICKENSIDED	SM-SMOOTH R-ROUGH ST-STEPPED PL-PLANAR	FL-FLEXURED UE-UNEVEN W-WAVY C-CURVED	BC-BROKEN CORE MB-MECH. BREAK B-BEDDING	DIAMETRAL POINT LOAD INDEX (MPa)	NOTES WATER LEVELS INSTRUMENTATION
0		GROUND SURFACE		103.94										
1		Fresh, light grey to grey, fine grained, moderately strong, stylonitic argillaceous LIMESTONE. Predominantly micritic, some sparitic zones.		0.00										
2														
3														
4														
5														
6														
7														
8														
9														
10		END OF HOLE		94.77										

DRILLHOLE 1119BROCK.GPJ GLDR CAN GDT 6/6/00 PS

DEPTH SCALE

1:50



LOGGED: SB

CHECKED: MR

RECORD OF BOREHOLE No 3-3										1 OF 1		METRIC		
PROJECT 001-1119			LOCATION N 4904254.49; E 306931.63			ORIGINATED BY SB								
W.P. 78-99-01			BOREHOLE TYPE 108mm I.D. Solid Stem Augers			COMPILED BY DKB								
DIST 41 HWY 401			DATE April 19, 2000			CHECKED BY ASP								
DATUM Geodetic														
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
							20 40 60 80 100	20 40 60 80 100	10 20 30					
104.87	GROUND SURFACE													
0.00	Sand and Gravel, trace silt Dense Brown and grey		1	SS	38									
104.18	Moist (Fill)													
0.69	Silty Clay, trace sand and gravel, occ. organics		2	SS	12									
103.63	Stiff Brown Moist Probable Bedrock													
1.34	END OF BOREHOLE Refusal to further auger penetration; Probable bedrock													
Note: Open borehole dry upon completion of drilling.														

ON MOT 001-1119.GPJ ON MOT GDT 6600

+3, X3: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

PROJECT: 001-1119

RECORD OF DRILLHOLE: 3-4

SHEET 1 OF 1

LOCATION: N 4904153.69; E 306891.21

DRILLING DATE: April 19, 2000

DATUM: Geodetic

INCLINATION: -90°

AZIMUTH: —

DRILL RIG: CME 55 Bombardier

DRILLING CONTRACTOR: Marathon

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV.													NOTES WATER LEVELS INSTRUMENTATION																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																	
				DEPTH (m)	RUN No.	PENETRATION RATE (mm/min)	FLUSH	COLOUR % RETURN	RECOVERY				R.Q.D. %	FRACT. INDEX PER 0.3 m	DISCONTINUITY DATA			HYDRAULIC CONDUCTIVITY k, cm/sec			DIAMETRAL POINT LOAD INDEX (MPa)																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																													
									TOTAL CORE %	SOLID CORE %	S-LICKENSIDED	PL-PLANAR			TYPE AND SURFACE DESCRIPTION	10"		15"	10"																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																															
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PROJECT 001-1119				RECORD OF BOREHOLE No 3-5				1 OF 1		METRIC						
W.P. 78-99-01				LOCATION N 4904138.25; E 306896.27				ORIGINATED BY SB								
DIST 41 HWY 401				BOREHOLE TYPE 108mm I.D. Solid Stem Augers				COMPILED BY DKB								
DATUM Geodetic				DATE April 19, 2000				CHECKED BY ASP								
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								
101.50	GROUND SURFACE															
8.88	Topsoil Silty Clay, trace sand and gravel, occ. organics Stiff Brown Moist		1	SS	8											
100.43	Probable Bedrock		2	SS	43											
1.19	END OF BOREHOLE Refusal to further auger penetration; probable bedrock Note: Open borehole dry upon completion of drilling.															

ON_MOT 001-1119.GPJ ON_MOT GDT 86/00

PROJECT 001-1119		RECORD OF BOREHOLE No 3-6				1 OF 1		METRIC					
W.P. 78-99-01		LOCATION N 4904160.99; E 306913.27				ORIGINATED BY SB							
DIST 41 HWY 401		BOREHOLE TYPE 108mm I.D. Solid Stem Augers				COMPILED BY DKB							
DATUM Geodetic		DATE April 19, 2000				CHECKED BY ASP							
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa					
101.28	GROUND SURFACE												
0.00	Topsoil												
100.91	Silty Clay, trace sand and gravel, occ. organics												
0.37	Brown Moist END OF BOREHOLE Refusal to further auger penetration; probable bedrock												
	Note: Open borehole dry upon completion of drilling.												

ON MOT 001-1119.GPJ ON MOT.GDT 6/6/00

PROJECT 001-1119				RECORD OF BOREHOLE No 3-7				1 OF 1		METRIC						
W.P. 78-99-01		LOCATION N 4904196.90; E 306911.20		ORIGINATED BY SB												
DIST 41 HWY 401		BOREHOLE TYPE 108mm I.D. Solid Stem Augers		COMPILED BY DKB												
DATUM Geodetic		DATE April 17, 2000		CHECKED BY ASP												
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC NATURAL LIQUID LIMIT MOISTURE CONTENT LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa 20 40 60 80 100 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x REMOULDED				WATER CONTENT (%) 10 20 30				
95.14	GROUND SURFACE															
0.15	Asphalt															
94.30	Sand and Gravel, trace silt Brown Moist (FILL)		1	SS	22											
0.84	Silty Sand Compact Brown Moist (FILL)															
93.16	Sand and Gravel with crushed limestone (Rock FILL)															
1.98	Fresh, grey, moderately strong, styloitic argillaceous LIMESTONE.															
	Bedrock cored from 0.86m to 5.18m For bedrock coring details refer to Drillhole 3-7.															
89.96																
5.18	END OF HOLE															
	Note: Water level measured in open borehole at 2.0m depth (El.93.1m) upon completion of drilling.															

PROJECT: 001-1119

RECORD OF DRILLHOLE: 3-7

SHEET 1 OF 1

LOCATION: N 4904196.90; E 306911.20

DRILLING DATE: April 17, 2000

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: —

DRILL RIG: CME 55 Bombardier

DRILLING CONTRACTOR: Marathon

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	PENETRATION RATE (mm/min) COL. CL. R. % RETURN	FR-FRACTURE		F-FAULT		SM-SMOOTH		FL-FLEXURED		BC-BROKEN CORE		DIAMETRAL POINT LOAD INDEX (MPa)	NOTES WATER LEVELS INSTRUMENTATION
							CL-CLEAVAGE		J-JOINT		R-ROUGH		UE-UNEVEN		MB-MECH. BREAK			
							SH-SHEAR		P-POLISHED		ST-STEPPED		W-WAVY		B-BEDDING			
							VN-VEIN		S-SUCKERSIDED		PL-PLANAR		C-CURVED					
							RECOVERY		R.Q.D. %	FRACT. INDEX PER 0.3	DISCONTINUITY DATA		HYDRAULIC CONDUCTIVITY k, cm/sec					
							TOTAL CORE %	SOLID CORE %			TYPE AND SURFACE DESCRIPTION							
							FLUSH	RECOVERY										
							100	100	100	100	100	100	100	100	100	100	100	
1		Sand and Gravel with crushed limestone (Rock FILL)		94.28 0.86	1	100												
2		Grey, fresh, moderately strong, argillaceous LIMESTONE. Stylolitic, some medium to coarse sparitic intervals (rare). Laminae are planar.		93.08 2.06	2	100												
3	NQ Core				3	100												
4					4	100												
5		END OF HOLE		89.96 5.18														
6																		
7																		
8																		
9																		
10																		

DEPTH SCALE

1:50



LOGGED: SB

CHECKED: MR

DRILLHOLE 1119BROCK GPJ GLDR CAN GDT 6/600 PS



APPENDIX B

Non-Standard Special Provisions

PERIMETER WALL CONTROL BLASTING NEAR NEW AND EXISTING BRIDGE FOUNDATIONS - ITEM NO.

Special Provision

This special provision outlines the procedure to be used where rock excavation (blasting) is required within 50 m of the new and existing bridge foundations.

All blasting shall be in accordance with OPSS 120 except as noted herein.

- **Blasting** shall be considered synonymous with Controlled Blasting and is defined as the use of explosive materials with procedures and techniques to limit ground vibration velocities, flyrock, permanent ground displacement, air concussion, and overbreak, so as to prevent damage to existing structures, services and utilities, as well as new foundation areas.
- **Perimeter Wall Control Blasting** includes line drilling along the limits of the excavation in conjunction with smooth wall blasting, cushion blasting, buffer blasting or any other approved wall control blasting technique used to provide a smooth, straight final wall.
- Perimeter Wall Control Blasting techniques shall be employed within 50 m of the new and existing bridge foundations to ensure that overbreak and damage to the final rock faces adjacent to the new and existing structure is minimized and the number of drillhole traces in the final face is maximized.
- Adequate stemming and blasting mats shall be in place prior to blasting to prevent damage to the existing structures and pavement from flyrock.
- As part of the blast design submission requirements contained in OPSS 120, the Contractor shall prepare and submit their proposed Perimeter Wall Control blast design techniques.
- Prior to blasting within 50 m of the new or existing structures, the Contractor shall carry out trial blasts using their proposed Perimeter Wall Control technique to demonstrate that the blast design is adequate to minimize damage to the rock face, overbreak and fly rock.
- Results of the trial blast shall be reviewed by the Contract Administrator and the blasting methodology must be accepted by the Contract Administrator prior to blasting within 50 m of new or existing structures.
- Acceptance by the Contract Administrator of the Perimeter Wall Control blasting plan and trial blasts shall in no way relieve the Contractor from responsibility for ensuring that the Blasting Operation is conducted in a safe and satisfactory manner, and in accordance with these specifications, nor shall the Contract Administrator assume responsibility for the adequacy of the blasting to achieve adequate breakage or acceptable results.

BASIS OF PAYMENT

Payment at the lump sum contract price for this tender item shall be full compensation for all labour, equipment and materials for completion of the work.

WORKING SLAB, ITEM NO.

Non-Standard Special Provision

SCOPE

This Special Provision covers the requirements for the supply and placement of a concrete working slab under the structure foundations. The purpose of the working slab is to protect the subgrade from disturbance and loosening due to construction traffic and ponded water and also to provide a level working surface.

CONSTRUCTION

Protection of Founding Soil

- Following inspection and approval of the prepared subgrade, a working slab with a minimum thickness of 100 mm shall be placed on the foundation subgrade as per the contract drawings and documents. The concrete shall have a minimum 28 day compressive strength of 20 MPa.

Protection of Founding Bedrock

- The surface of the footing founding rock shall be exposed, cleaned and any loose or fractured parts removed so that sound rock is exposed.
- The working slab shall have a minimum 28 day strength of 20 MPa
- The working slab shall be placed on the exposed cleaned sound founding rock surface as per the contract drawings and documents.
- Thickness of the working slab shall depend on the slope and irregularities in the exposed founding rock surface. A nominal thickness and a footprint plan view area has been specified on the contract drawings and documents

Unwatering of the excavation for the footing construction, including the construction of the working slab, might be required and is covered under separate Tender Item. The dewatering scheme shall be done in such a manner as to prevent any disturbance to the surrounding original soil.

BASIS OF PAYMENT

Payment at the contract price for this Tender Item shall include full compensation for all labour, equipment and material required to do the work.

DOWELS INTO ROCK – ITEM NO.

Special Provision

1.0 GENERAL

1.1 Scope

The work for the above noted tender item shall be in accordance with OPSS 904, including all special provisions, except as extended herein. This document specifies additional requirements for the supply, installation and testing of Dowels into Rock for the structure footings.

1.2 Instructions to Contractor

- 1.2.1 These instructions are to be read in conjunction with the Contract Drawings.
- 1.2.2 A total of 1 test Dowels into Rock are required for the Dowels into Rock at each structure footing.
- 1.2.3 Dowels shall extend through tremie concrete and into sound bedrock to the specified embedment depth.

1.3 Qualifications

- 1.3.1 **Qualifications of Staff from Contractor or Sub-Contractor Completing Work for the Dowels into Rock:** All work shall be performed under the direction of personnel experienced with all aspects associated with the installation of Dowels into Rock. Such experience shall have been obtained within the preceding five (5) years on projects of similar nature and scope to the work required for this project.
- 1.3.2 **Qualifications of the Quality Verification Engineer:** A resume of the work experience of the Quality Verification Engineer shall be submitted to the Contract Administrator for record purposes. The Quality Verification Engineer shall be a Professional Engineer licensed in the Province of Ontario having a minimum of five years of experience on projects of similar nature and scope to the work required for this project.
- 1.3.3 **Qualifications of the Design Engineer:** A resume of the work experience of the Design Engineer shall be submitted to the Contract Administrator for record purposes. The Design Engineer shall be a Professional Engineer licensed in the

Province of Ontario having a minimum of five years of experience of projects of similar nature and scope to the work required for this project.

1.4 Responsibilities of the Contractor

- 1.4.1 The Contractor shall prove the allowable bond stress by tests of the Dowels into Rock on non-production Dowels into Rock.
- 1.4.2 The Contractor shall supply equipment, materials and skilled personnel to install production Dowels into Rock and conduct the specified acceptance tests. It shall be the responsibility of the Contractor to constantly monitor the acceptance tests, maintain specified test loads and record test measurements as specified by the Contract Administrator.
- 1.4.3 The Contractor is responsible for materials and workmanship. Any remedial measures, required because of defects in materials or workmanship, shall be completed by the Contractor at no cost to the Owner.
- 1.4.4 The Contractor shall submit 4 copies of all Working Drawings to the Contract Administrator as outlined in Section 1.6.

1.5 Definitions

- 1.5.1 Dowels into Rock: reinforcing steel bar and non-shrink grout.
- 1.5.2 Design Engineer: An Engineer who has a minimum of five (5) years experience in all aspects associated with the installation of Dowels into Rock, including drilling, grouting and doweling work. The Design Engineer shall be retained by the Contractor to design various components for the installation and testing for the Dowels into Rock.
- 1.5.3 Quality Verification Engineer: An Engineer who has a minimum of five (5) years experience in all aspects associated with the installation of Dowels into Rock, including drilling, grouting and doweling work. The Quality Verification Engineer shall be retained by the Contractor to ensure conformance with the contract documents and issue certificate(s) of conformance.

1.6 Submissions and Working Drawings

- 1.6.1 Working Drawings shall consist of drawings, testing and installation records, procedures and reports, and work plans.

1.6.2 The Contractor shall submit Working Drawings to the Contract Administrator as follows:

- All Working Drawings that include drawing, testing and installation procedures and reports, and work plans shall be sealed and signed by the Design Engineer.
- All Working Drawings that include testing and installation results and reports shall be signed and sealed by the Quality Verification Engineer.

1.6.3 Upon completion of testing or installation and testing for each component, the Contractor shall submit to the Contract Administrator a Certificate of Conformance sealed and signed by a Quality Verification Engineer. The Certificate shall state that the work has been carried out in conformance with the Working Drawings and in general conformance with the contract documents.

1.6.4 Working Drawings consisting of testing and installation records and reports shall be submitted four days after completion of testing and installation. All other Working Drawings shall be submitted two weeks prior to construction.

1.6.5 Working Drawings to be submitted include the following with further details outlined in the remainder of this specification:

- Design calculations, specifications and shop drawings covering all aspects of fabrication, installation and acceptance testing of Dowels into Rock.
- Test results verifying the 28 day strength of non-shrink grout.
- The method for constructing of the holes, maintaining the holes, and placing reinforcing steel bars, grout and other materials in the holes, including casing sizes, bit sizes and tremie grouting methods.
- The procedures to verify hole length. Records of measurements that verify the hole length.
- Records of all drilling procedures, rock conditions encountered, and installation times.
- Test procedures for Dowels into Rock.

- Drawings and design calculations for a suitable reaction system for the applied test loads.
- Records of vertical and horizontal movements of the reaction system, and elongation of the reinforcing steel bar.
- Drawings and details for reference system arrangement.
- Current calibration curves shall be provided for all gauges.
- Complete test records for all tests including plots of dowel movement versus dowel load, dowel load versus time, and dowel movement versus time.
- Remedial measures for unacceptable stressing results.

1.7 Subsurface Conditions

- 1.7.1 Soils, rock and groundwater conditions are described in the Foundation Investigation Report for this Contract.

2.0 MATERIALS

The non-shrink grout shall be an approved DSM 9.10.35 non-shrink grout.

The **Contractor** shall provide the following information from the manufacturer for non-shrink grout:

- Data sheets for the non-shrink grout,
- installation procedures

3.0 EQUIPMENT

3.1 General

- 3.1.1 All equipment for the installation of the Dowels into Rock shall be suitable for the intended purposes and capable of working on the site under the prevailing access and clearance conditions.

- 3.1.2 The equipment shall not cause damage to the reinforcing steel bars.

4.0 INSTALLATION

All work for the installation of Dowels into Rock shall be inspected by the Quality Verification Engineer.

4.1 Construction of Holes

- 4.1.1 The sides and end of the hole shall not be disturbed. The Contractor shall submit Working Drawings to the Contract Administrator that include the method for constructing of the holes, maintaining the holes, and placing reinforcing steel bar, grout and other materials in the holes. All excavated material shall be removed from the site.
- 4.1.2 The hole diameters and hole length for this project are as specified on the Contract Drawings. Prior to commencing drilling operations, the Contractor shall submit Working Drawings to the Contract Administrator outlining devised procedures to verify hole length. The Contractor shall submit Working Drawings that include drilling operations records to the Contract Administrator that include the above noted records.
- 4.1.3 At all times, the Contractor shall keep a record of all drilling procedures, rock conditions encountered, and installation times. The Contractor shall submit Working Drawings to the Contract Administrator that include the above noted records.

4.2 Installation of Reinforcing Steel Bar

- 4.2.1 Reinforcing steel bar shall be installed in strict accordance with the Contract Drawings and installation procedures.
- 4.2.2 Centering devices shall be provided to ensure that the reinforcing steel bar is located centrally in the hole.
- 4.2.3 Dowels shall extend through the tremie concrete for the footing and into sound bedrock.
- 4.2.4 Reinforcing steel bar shall be installed after the dowel hole has been filled with non-shrink grout.

4.3 Grout

- 4.3.1 The non-shrink grout shall entirely fill the annular space between the reinforcing steel bar and side for the dowel hole.

4.3.2 The placement of grout for the test Dowels into Rock shall be identical to the production Dowels into Rock.

4.3.3 Non-shrink grout shall be placed into the dowel hole using tremie placement methods.

5.0 TESTING REQUIREMENTS

All work for the testing of Dowels into Rock shall be inspected by the Quality Verification Engineer.

5.1 General Testing Requirements

5.1.1 Refer to the attached Instructions to Contractor and the Contract Drawings for specific test details.

5.1.2 The Contractor shall install the number of Dowels into Rock specified in the contract documents for testing purposes. The purpose of the testing the Dowels into Rock is to prove the adequacy of the proposed anchor configuration and installation procedures under the site conditions, and to provide design parameters.

5.1.3 The equipment, labour and materials for test dowels shall be identical to Dowels into Rock at the each structure location.

5.1.4 The Contractor shall submit Working Drawings that include proposed procedures for testing of the dowels into Rock to the Contract Administrator. Such testing shall be executed in strict accordance with the proposed procedures of the Contractor.

5.1.5 The Quality Verification Engineer shall supervise the testing of the Dowels into Rock. The Contractor will notify the Contract Administrator of the testing schedule at least 10 days prior to commencement of the testing program. Testing for Dowels into Rock shall be conducted concurrently, as scheduled by the Contract Administrator. The tests shall normally be conducted between 8:00 hrs and 20:00 hrs from Monday to Friday, unless otherwise directed by the Contract Administrator.

5.1.6 The Contractor shall supply materials and skilled personnel to conduct the tests for the Dowels into Rock. The equipment and materials shall be capable of stressing the Dowels into Rock to the specified loads. It shall be the responsibility of the Contractor to constantly monitor the test, maintain specified test loads and to record test measurements as specified by the Quality Verification Engineer.

- 5.1.7 The test site shall be restored to its pre-test condition. Reinforcing steel bars used in tests shall be cut down 25 mm below the top of the sound bedrock.

5.2 Testing Location

- 5.2.1 The Contractor shall remove all loose rock down to sound bedrock at the test location.
- 5.2.2 The test Dowels into Rock shall be constructed at locations specified by the Contract Administrator.
- 5.2.3 If site conditions dictate, changes to the test locations will be considered. The Contractor shall provide the Contract Administrator at least 2 days notice in writing of this operation.

5.3 Testing Equipment

- 5.3.1 The dowels into rock will be carried out generally in accordance with the prevailing requirements of A.S.T.M. (Designation D1143-81) superseded where applicable by the procedures specified in this document.
- 5.3.2 The Contractor shall submit Working Drawings for a suitable reaction system for the applied test loads to the Contract Administrator. Jacks must be secured with chains to provide adequate protection for the personnel in the event of breakage of the reinforcing steel bar or stressing system.
- 5.3.3 The Contractor shall submit Working Drawings for the reference system arrangement to the Contract Administrator. All reference beams shall be as follows:
- The beams shall be independently supported with the support firmly embedded in the ground.
 - The testing device shall not apply compression to the bedrock surrounding the test for the Dowels into Rock, within a circle concentric with the dowel hole and a diameter equal to 4.0 m.
 - Reference beams shall be sufficiently rigid to support instrumentation such that variations in readings do not occur.

- 5.3.4 The Contractor shall construct suitable enclosures to provide complete protection for equipment and instruments from variations in the weather conditions and disturbances during the test program. These provisions must meet the approval of the Quality Verification Engineer and will include that the test enclosures must be weather-proof and provide a consistent temperature in order to eliminate temperature variations that could affect instrumentation.

5.4 Testing for Dowels Into Rock, and Report

- 5.4.1 At all times, the Contractor shall keep records of vertical and horizontal movements of the reaction system, elongation of reinforcing steel bar, and the record of test enclosure temperature. The movements shall be recorded with respect to an independent fixed reference point. The Contractor shall submit Working Drawings that include the above noted records to the Contract Administrator.
- 5.4.2 Dial gauges shall have at least a 76.2 mm (3.0 in.) travel. Longer gauge stems or sufficient gauge blocks shall be provided to allow for greater travel if required. Gauges shall have precision of at least 0.025 mm (0.0001 in.). The dial gauges shall be placed on smooth bearing surfaces mounted perpendicular to the direction of movement. All gauges, scales or reference points attached to the test anchor shall be mounted so as to prevent movement relative to the test anchor during the test. The Contractor shall submit Working Drawings that include details for current calibration and curves for all gauges to the Contract Administrator.
- 5.4.3 Jacks used for reinforcing steel bars shall have a minimum ram dimension of 153 mm (6.0 in.). The Contractor shall submit Working Drawings that include details for current calibration and curves for all gauges to the Contract Administrator.
- 5.4.4 Requirements for Clauses 5.4.1 to 5.4.4 shall be repeated as required at different testing locations.

5.5 Testing Loading

- 5.5.1 The testing procedures shall safely load test the Dowels into Rock in tension at a rate of approximately 100kN per minute to the specified test load. The load shall be increased by an additional 50 kN beyond this level as directed by the Quality Verification Engineer.

- 5.5.2 Each load shall be maintained for a minimum time of 15 minutes and until the rate of displacement is not greater than 0.25 mm (0.01 inches) per hour.

5.6 Acceptance Criteria

- 5.6.1 The following acceptance criteria apply:

The testing of dowels shall be carried out in advance of the instalment of Dowels into Rock at each structure location.

Tests for Dowels into Rock shall have a capacity of at least [insert value] kN.

The Quality Verification Engineer shall report on the acceptance of the tests for Dowels into Rock. The Quality Verification Engineer shall report on the testing of the Dowels into Rock including recommendations for increasing embedment depth, if necessary.

6.0 BASIS OF PAYMENT

Payment at the contract unit price for the above tender item shall include full compensation for all labour, equipment, and materials to do the work. No additional payment will be made for tests for Dowels into Rock which are deemed as included as part of the work for the above noted item.

**CONTROL OF OVERBURDEN SOILS, SURFACE WATER AND GROUNDWATER
DURING FOUNDATION EXCAVATION INSTALLATION - ITEM NO.**

Special Provision

Caisson foundation and spread footing excavations will be advanced through cohesionless soils that may be water-bearing; these soils should be expected to slough/flow into unsupported caisson holes or excavations. Perched water is also expected along the overburden/bedrock interface. Appropriate construction procedures and equipment will be required to control sloughing and flowing during drilling and concrete placement for caisson and spread footing foundations. Provision should be made for dewatering to control surface water, runoff, and perched groundwater.

BASIS OF PAYMENT

Payment at the lump sum contract price for this tender item shall be full compensation for all labour, equipment and materials for completion of the work.

END OF SECTION

At Golder Associates we strive to be the most respected global group of companies specializing in ground engineering and environmental services. Employee owned since our formation in 1960, we have created a unique culture with pride in ownership, resulting in long-term organizational stability. Golder professionals take the time to build an understanding of client needs and of the specific environments in which they operate. We continue to expand our technical capabilities and have experienced steady growth with employees now operating from offices located throughout Africa, Asia, Australasia, Europe, North America and South America.

Africa	+ 27 11 254 4800
Asia	+ 852 2562 3658
Australasia	+ 61 3 8862 3500
Europe	+ 356 21 42 30 20
North America	+ 1 800 275 3281
South America	+ 55 21 3095 9500

solutions@golder.com
www.golder.com

Golder Associates Ltd.
32 Steacie Drive
Kanata, Ontario, K2K 2A9
Canada
T: +1 (613) 592 9600

