

GEOCRES No. 30M12-249DIST. CR REGION W.P. No. 613-89-00CONT. No. W. O. No. STR. SITE No. 24-354HWY. No. 403LOCATION MATHESON BLVD BRIDGE
§ Hwy 403=====OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT. REMARKS:

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**FOUNDATION INVESTIGATION AND DESIGN REPORT
MATHESON BOULEVARD OVERPASS
HIGHWAY 403 WIDENING BETWEEN
HIGHWAYS 407 AND 401, MISSISSAUGA
G.W.P. 613-89-00, AGREEMENT NO. 2005-A-000201**

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January 2001

001-1131B

TABLE OF CONTENTS

<u>SECTION</u>	<u>PAGE</u>
PART A - FOUNDATION INVESTIGATION REPORT	
1.0 INTRODUCTION.....	1
2.0 SITE DESCRIPTION	2
3.0 INVESTIGATION PROCEDURES	3
4.0 GENERAL SITE GEOLOGY AND STRATIGRAPHY	5
4.1 Site Geology.....	5
4.2 Site Stratigraphy	5
4.2.1 Topsoil.....	6
4.2.2 Fill.....	6
4.2.3 Bedrock	6
4.3 Groundwater Conditions	8
PART B - FOUNDATION DESIGN REPORT	
5.0 ENGINEERING RECOMMENDATIONS	9
5.1 General	9
5.2 Bridge Foundations.....	9
5.2.1 Spread Footings.....	10
5.2.2 Factored Geotechnical Resistance	11
5.2.3 Resistance to Lateral Loads	11
5.2.4 Frost Protection	11
5.3 Lateral Earth Pressures	11
5.4 Excavations	12
5.4.1 Temporary Excavations for Spread Footings.....	12
5.4.2 Temporary Support Systems for Roadway Protection	13
5.4.2.1 Earth Pressure Distribution for Temporary Shoring Design	14
5.4.2.2 Temporary Soil Anchor Design.....	15
5.5 Embankment Construction.....	15
5.5.1 Embankment Configuration and Settlement.....	15
5.5.2 Subgrade Preparation and Fill Placement	17

In Order
Following
Page 17

TABLE OF CONTENTS (continued)

Lists of Abbreviations and Symbols
Records of Borehole Sheets (Boreholes 1 to 5)
Drawing 1
Appendix A

LIST OF DRAWINGS

Drawing 1 Matheson Boulevard Overpass, Highway 403 – Borehole Locations and Soil Strata

LIST OF APPENDICES

Appendix A Records of Boreholes – 1975 MTO Investigation

January 2001

001-1131B

PART A

**FOUNDATION INVESTIGATION REPORT
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TABLE OF CONTENTS

<u>SECTION</u>	<u>PAGE</u>
PART A - FOUNDATION INVESTIGATION REPORT	
1.0 INTRODUCTION.....	1
2.0 SITE DESCRIPTION	2
3.0 INVESTIGATION PROCEDURES	3
4.0 GENERAL SITE GEOLOGY AND STRATIGRAPHY	5
4.1 Site Geology.....	5
4.2 Site Stratigraphy	5
4.2.1 Topsoil.....	6
4.2.2 Fill.....	6
4.2.3 Bedrock	6
4.3 Groundwater Conditions	8

1.0 INTRODUCTION

Golder Associates Ltd. has been retained by The Greer Galloway Group Inc. (Greer Galloway) on behalf of the Ministry of Transportation, Ontario (MTO) to provide foundation investigation and design services for the widening of Highway 403 between Highways 401 and 407 in Mississauga, Ontario. The foundations engineering component of the project includes widening of both the Mullet Creek bridge and the Matheson Boulevard overpass structures within the median area of Highway 403. This report addresses the Matheson Boulevard Overpass widenings.

The purpose of the foundation investigation is to determine the subsurface conditions at the site of the proposed structure by drilling boreholes, and carrying out in-situ tests and laboratory tests on selected samples. Existing subsurface data from a 1975 investigation carried out by the MTO ("Foundation Investigation Report for Highway 403 Eastbound at Matheson Boulevard, Bridge 61, W.P. 36-74-03, Site 24-354C and Highway 403 Westbound at Matheson Boulevard, Bridge 62, W.P. 36-74-02, Site 24-354D", dated February 1977) was used to supplement the data obtained in the current investigation.

The terms of reference for the scope of work are outlined in Golder Associates Ltd. Proposal No. P01-1108, dated April 2000. The work was carried out in accordance with the Quality Control Plan for Foundation Design Services, submitted to MTO on June 26, 2000.

The General Arrangement plan showing the proposed configuration for the widening of the eastbound and westbound core structures at Matheson Boulevard was provided by Greer Galloway in digital format on October 23, 2000.

2.0 SITE DESCRIPTION

The existing Highway 403 bridges over Matheson Boulevard are located about 1.5 km south of Highway 401 and 1 km north of Eglinton Avenue, in the City of Mississauga, Regional Municipality of Peel.

The original ground surface at the site varied from Elevation 167 m to 164 m, generally declining southward and eastward, toward the Little Etobicoke Creek located immediately south of the site. Currently, the Highway 403 grade in the core lanes at Matheson Boulevard is at about Elevation 166.5 m. Within the approach embankment limits, the median ditch is at about Elevation 165 m to 165.5 m, approximately 1 m to 1.5 m lower than the adjacent Highway 403 grade. Matheson Boulevard is constructed in cut and the road grade is at about Elevation 159.5 m under the Highway 403 core lanes. The median area close to the bridge has been sloped downward toward Matheson Boulevard. Retaining walls are present adjacent to these slopes in the median area.

The existing Highway 403 eastbound and westbound bridges over Matheson Boulevard are single-span structures supported on spread footings which are founded on shale bedrock. The eastbound Highway 403 overpass structure is founded at about Elevation 157.7 m, while the westbound structure is founded at about Elevation 158 m. The footings step up to about Elevation 163 m at the ends of the median retaining walls. This information was determined from the General Arrangement and Foundation Layout drawings for the existing eastbound and westbound structures (Sheets 49/51 and 64/66 of Contract 77-21, respectively).

3.0 INVESTIGATION PROCEDURES

The field work for this investigation was carried out in October and November, 2000. At this time, five boreholes were advanced to obtain subsurface information in the vicinity of the north and south abutments and at the approach embankments. Also referenced in this report are boreholes advanced during the 1975 subsurface investigation carried out by the MTO (as referenced in Section 1.0) for the original construction of the Highway 403 structures over Matheson Boulevard.

The investigation was carried out using drill rigs supplied and operated by Master Soil Investigations Ltd. of North York, Ontario. Boreholes 2, 3 and 4 were advanced to between 2.7 m and 7.3 m depth in the Highway 403 median area, using a bombardier-mounted CME-45. Because of the steep forward slope, it was not possible to drill Boreholes 2 and 3 at the abutment foundation footprint; also, because of the location of the Peel fibre optic line, drilling was not permitted on the south sidewalk of Matheson Boulevard. Borehole 5 was advanced to 3.8 m depth on the north sidewalk of Matheson Boulevard using a truck-mounted D-90. Borehole 1, at the north approach embankment in the Highway 403 median area, was advanced to 1.2 m depth using a power hand auger operated by Golder Associates personnel. In Borehole 1, samples were retrieved from the augers, and in Boreholes 2 to 5, samples of the overburden were obtained at 0.75 m to 1.5 m intervals of depth using 50 mm outside diameter split-spoon samplers in accordance with the Standard Penetration Test (SPT) procedure. The bedrock was cored in Boreholes 3 and 5 using NQ-size coring equipment. The groundwater conditions in the open boreholes were observed throughout the drilling operations.

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, coring and in-situ testing operations, and logged the boreholes. The soil and rock samples were identified in the field, placed in labelled containers and transported to our laboratory in Mississauga for further examination. Index and classification tests consisting of water content determinations and Atterberg Limits tests were carried out on selected soil samples.

The boreholes were located in the field relative to the existing structures, and the UTM northing and easting coordinates for these boreholes were determined from the digital files provided by Greer Galloway. The ground surface elevations at the borehole locations were surveyed relative to the existing bridge abutments, and were further compared to digital survey information provided by Greer Galloway; the borehole elevations are referenced to the geodetic datum. The borehole locations are shown on Drawing 1, and northing and easting coordinates are indicated on the Record of Borehole sheets.

4.0 GENERAL SITE GEOLOGY AND STRATIGRAPHY

4.1 Site Geology

The site is located on the margin of the physiographic regions known as the Peel Plain and the Trafalgar Moraine portion of the South Slope. The surficial soils generally consist of clayey silt to silty clay till. This cohesive till overlies shale bedrock, with interbedded limestone layers, of the Georgian Bay Formation (Chapman and Putnam, "The Physiography of Southern Ontario", 3rd Edition, 1984).

4.2 Site Stratigraphy

The detailed subsurface soil and groundwater conditions encountered in the boreholes, together with the results of the laboratory testing carried out on selected soil samples, are given on the attached Record of Borehole sheets. The detailed subsurface conditions encountered in the 1975 boreholes are given on the Record of Borehole sheets in Appendix A. The stratigraphic boundaries shown on the borehole records are inferred from non-continuous sampling and, therefore, represent transitions between soil types rather than exact planes of geological change. Subsoil conditions will vary between and beyond the borehole locations.

Boreholes 1 and 4 were advanced for the north and south approach embankments, respectively, while Boreholes 2 and 3 were advanced in the vicinity of the south abutments, and Borehole 5 was advanced at the north abutment of the eastbound (northbound) bridge. The locations and elevations of the boreholes, together with the interpreted stratigraphic profile and sections at the bridge sites, are shown on Drawing 1.

In summary, the subsoils encountered at the site typically consist of up to 4 m of fill overlying shale bedrock of the Georgian Bay Formation. A detailed description of the subsurface conditions encountered in the boreholes for the current and 1975 investigations is provided in the following sections.

4.2.1 Topsoil

In the boreholes drilled as part of the current investigation, trace organics were encountered within the upper 100 mm to 300 mm of the fill. A surficial layer of topsoil, typically 450 mm in thickness, was encountered in the boreholes drilled during the 1975 investigation.

4.2.2 Fill

Fill material was placed during construction of the existing Highway 403 and the bridge structures. The fill as encountered in Boreholes 2 and 3 in the vicinity of the south abutment is about 4 m thick. It is expected that there will be a greater thickness of fill immediately adjacent to the abutments. The fill encountered in these boreholes generally consists of red-brown to grey silty clay containing trace to some sand, gravel and shale fragments. In Borehole 5, extended through the sidewalk on Matheson Boulevard, sand and gravel fill was encountered below 100 mm of concrete.

The silty clay fill is typically very stiff, although it varies from stiff to hard. The measured Standard Penetration Test (SPT) "N" values ranged from 10 to 75 blows per 0.3 m of penetration, but were typically greater than 15 blows per 0.3 m of penetration. The natural water contents measured on selected samples of the fill were between 9 and 23 per cent. Atterberg limits measured on a selected, representative sample of the fill indicated a plastic limit of about 19 per cent, a liquid limit of 31 per cent, and a plasticity index of about 12 per cent. These test results indicate that the silty clay fill is inorganic and of low plasticity.

4.2.3 Bedrock

Grey shale bedrock of the Georgian Bay Formation was encountered in Boreholes 2, 3 and 5, drilled in the vicinity of the abutments for the proposed widenings, and in the boreholes drilled during the 1975 investigation. The original surface of the weathered shale bedrock was at about Elevation 166 m to 164.5 m in Boreholes 75-3 to 75-6. On the Record of Borehole logs for these previous boreholes, there is a distinction given between "weathered" and "sound" bedrock; the delineation between the two is indicated to vary between Elevations 164.5 m and 161.5 m. The distinction between "weathered" and "sound" shale bedrock appears to be related to the depth

where coring of the bedrock was commenced. This may be the point where refusal to further auger penetration was encountered; however, this is not specifically stated in the report.

The original bedrock surface, as encountered in the previous four boreholes, generally dips downward both to the east and the south. The surface of what is referred to in the previous report as sound bedrock also slopes downward towards both the east and the south, generally following the ground surface topography. Matheson Boulevard was constructed in cut with existing road grade some 5 m to 6 m below the bedrock surface. The abutment footings are founded at about Elevation 158 m (some 6.5 m to 7.5 m below the bedrock surface) and the retaining walls are founded as high as Elevation 163 m.

During the recent investigation, the bedrock surface was encountered at about Elevations 162.7 m and 161.7 m in Boreholes 2 and 3, respectively, located behind the south abutment. The upper approximately 1 m of the bedrock in these two boreholes is completely to highly weathered; the split-spoon samples exhibited the shale bedrock texture but were essentially weathered to a soil consistency. The bedrock surface as encountered in Borehole 5 located adjacent to the north abutment is at about Elevation 159 m.

Bedrock coring was carried out in Boreholes 75-4 and 75-5 from the 1975 investigation, and in Boreholes 3 and 5 advanced near the south and north abutments as part of the current investigation. Limestone and siltstone interbeds were evident throughout the recovered core, and were between 20 mm and 400 mm in thickness. Rock Quality Designation (RQD) values measured on the core samples ranged from 0 to 10 per cent in Boreholes 3 and 5; no RQD values were recorded on the Record of Borehole logs from the 1975 investigation. The total core recovery (TCR) ranged from about 80 to 100 per cent in the boreholes advanced during both investigations, with the exception of Borehole 5 advanced in the north sidewalk of Matheson Boulevard where the TCR ranged from about 40 to 90 per cent.

4.3 Groundwater Conditions

The groundwater conditions were observed in the open boreholes following drilling operations in both the current and 1975 investigations. In both investigations, the boreholes were dry on completion of overburden drilling, prior to commencement of coring operations.

It should be noted that some seepage of "perched" water from within the fill is expected; this is expected to fluctuate seasonally.

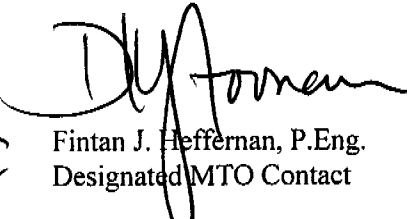
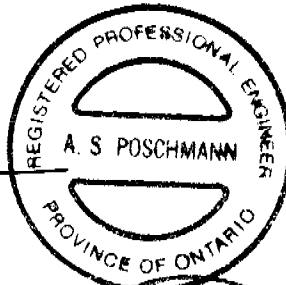
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January 2001

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PART B

**FOUNDATION DESIGN REPORT
MATHESON BOULEVARD OVERPASS
HIGHWAY 403 WIDENING BETWEEN
HIGHWAYS 407 AND 401, MISSISSAUGA
G.W.P. 613-89-00, AGREEMENT NO. 2005-A-0002**

5.0 ENGINEERING RECOMMENDATIONS

5.1 General

This section of the report provides recommendations on the geotechnical aspects of design of the proposed widening of the Highway 403 structures over Matheson Boulevard, based on interpretation of the factual information obtained during the current and 1975 investigations. 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 the existing eastbound and westbound Highway 403 structures will be widened approximately 3.5 m into the median area. The current ground surface in the median area immediately adjacent to the core lanes is at about Elevation 165.5 m, sloping down toward the Matheson Boulevard grade of about Elevation 159.5 m. Within the approach embankment limits, therefore, it will be necessary to raise the median grade by about 1 m to 7 m to match the existing grade of the core lanes. In addition, partial removal of the existing retaining walls along the median area will be required in order to construct the proposed widening structures.

Based on the general and foundation layout drawings for the existing structures, from Contract No. 77-21, the single-span bridges over Matheson Boulevard are supported on spread footings which are founded on shale bedrock. The eastbound and westbound bridge abutments are founded at about Elevation 157.7 m and 158 m, respectively; the median retaining wall footings step upward from this level to about Elevation 163 m at the ends of the retaining walls.

5.2 Bridge Foundations

The subsoils encountered in the boreholes put down during the current investigation consist of very stiff to hard silty clay fill overlying shale bedrock. In the 1975 investigation, carried out before the construction of Highway 403, the bedrock was covered by topsoil. The original bedrock surface was at about Elevation 166 m to 164.5 m. Given the site grades, it is recommended that the

proposed widened structures be supported on spread footings founded on the bedrock at the same elevations as the existing structures.

5.2.1 Spread Footings

It should be noted that the bedrock surface elevations and the condition of the bedrock as encountered in the recent boreholes raises concerns with respect to the excavation works which were carried out during construction of the Highway 403 bridges and retaining walls. The bedrock surface was encountered at about Elevation 162.7 m and 161.7 m in Boreholes 2 and 3, respectively, near the south end of the existing median retaining walls. These elevations of the bedrock surface are about 2.5 m to 3 m lower than the inferred original bedrock surface in this area. In addition, the current bedrock surface at these two borehole locations is 0.3 m and 1.3 m lower than the design founding elevation for the south limits of the retaining walls.

It is known that there are existing utility trenches in the area between the two existing bridges. It is possible that the bedrock excavation for the construction of the works has resulted in additional fracturing and / or disturbance of the bedrock. In this regard, it is recommended that the design for the widened structure assume that the founding level for the widened abutments will be consistent with the existing; however, there should be some flexibility in the contract to adjust the founding level based on conditions actually encountered in the field.

For design of the bridge abutments, spread footings placed on the shale bedrock may be assumed at about Elevation 157.7 m for the eastbound structure and at about Elevation 158 m for the westbound structure. The retaining wall footings should be stepped upward away from the abutments such that a minimum soil cover of 1.2 m is maintained above the underside of the footings but should also be maintained at or below the bedrock surface. It is recommended that the design founding elevation for the south limits of the retaining walls be taken at or below Elevation 161.7 m.

As noted above, it is essential that there be some flexibility in the design and the contract such that sub-excavation and replacement with mass concrete can be carried out if poor quality bedrock is encountered at the design founding elevations.

5.2.2 Factored Geotechnical Resistance

Spread footings placed on properly prepared shale bedrock at the design elevations given above may be designed using a geotechnical resistance at Ultimate Limit States (ULS) of 1,200 kPa. Provided that the bedrock is properly prepared, Serviceability Limit States (SLS) conditions do not apply.

5.2.3 Resistance to Lateral Loads

Sliding resistance between the concrete spread footings and the shale bedrock should be calculated in accordance with Section 6-8.4.3 of the Ontario Highway Bridge Design Code (OHBD). An unfactored coefficient of friction equal to 0.45 may be assumed between the concrete footings and the prepared shale bedrock.

5.2.4 Frost Protection

The footings should be provided with a minimum of 1.2 m of soil cover for frost protection.

5.3 Lateral Earth Pressures

The lateral pressures acting on the bridge abutments will depend on the type and method of placement of the backfill materials, on the nature of the soils behind the backfill and on the subsequent lateral movement of the structure. The following recommendations are made concerning the design of the abutments, in accordance with the OHBD:

- Select free-draining granular fill meeting the specifications of OPSS Granular "A" or Granular "B" but with less than 5 per cent passing the 200 sieve should be used as backfill behind the abutments. This fill should be compacted in loose lifts not greater than 200 mm in thickness to 95 per cent of the material's Standard Proctor maximum dry density. 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 3501.00.
- A compaction surcharge equal to 16 kPa should be included in the lateral earth pressures for the structural design of the abutment wall, in accordance with OHBD Figure 6-7.4.3. Compaction equipment should be used in accordance with OPSS 501.06.

- The granular fill may be placed either in a zone with width equal to at least 1.2 m behind the back of the stem (Case I) or within the wedge-shaped zone defined by a 60° line extending up and back from the bottom of the rear face of the footing (Case II).
- For Case I, the pressures are based on the existing and proposed embankment fill materials and the following parameters (unfactored) may be assumed:

Soil unit weight:	20 kN/m ³
Coefficients of lateral earth pressure:	
Active, K_a	0.35
At rest, K_o	0.50

- For Case II, the pressures are based on the granular fill as placed and the following parameters (unfactored) may be assumed:

	Granular "A"	Granular "B" Type II
Soil unit weight:	22 kN/m ³	21 kN/m ³
Coefficients of lateral earth pressure:		
Active, K_a	0.27	0.31
At rest, K_o	0.43	0.47

- If the wall support allows lateral yielding of the stem, active earth pressures may be used in the geotechnical design of the structure. If the abutment support does not allow lateral yielding, at-rest earth pressures should be assumed for geotechnical design.

It should be noted that the above design recommendations and parameters assume level backfill and ground surface behind the abutment and retaining walls.

5.4 Excavations

5.4.1 Temporary Excavations for Spread Footings

Excavation for the abutment footings will extend about 2 m below the existing Matheson Boulevard grade and will be through silty clay fill materials and fractured bedrock. Excavations for retaining wall footings will be as much as 4 m below the existing ground surface and will generally be through the existing fill materials. Excavations should be carried out in accordance with the guidelines outlined in the latest edition of the Occupational Health and Safety Act for Construction Activities. The very stiff to hard silty clay fill soils at this site would be

classified as Type 2 soil. The bedrock is expected to be weathered and / or fractured within the excavation depth and should be treated as Type 1 soil.

Temporary open-cut slopes through the fill and fractured rock should be maintained no steeper than 1 horizontal to 1 vertical (1H:1V). Recommendations for temporary excavation support systems as will be required adjacent to Highway 403 are provided in Section 5.4.2.

The abutment and retaining wall footings will be founded between Elevation 157.7 m and 163 m, on the shale bedrock. The shale is extremely sensitive to disturbance from ponded water, construction traffic and frost. It is recommended that a lean concrete mud coat be placed once excavation to the founding level is complete to protect the subgrade from such disturbance. The excavation base should be inspected by qualified geotechnical personnel prior to placement of the mud coat.

Groundwater seepage into the footing excavations may occur from perched water within the fill, although the quantity is expected to be minor. Pumping from properly-filtered sumps or a filtered drain placed at the base of the excavation, but maintained outside of the footing area, should provide sufficient groundwater control during foundation works. Surface water run-off, which is expected to be more significant than groundwater seepage, should be directed away from the footing excavations.

5.4.2 Temporary Support Systems for Roadway Protection

Temporary shoring will be required to support the excavation adjacent to the travelled lanes of eastbound and westbound Highway 403 to allow excavation to the footing level for the abutments and retaining walls. The temporary support system could consist of a soldier pile and lagging wall. The soldier piles should be placed in pre-augered holes extended below the excavation base and generally socketted into the bedrock. Lateral support to a soldier pile and lagging wall system could be in the form of struts and walers, rakers, or temporary soil anchors.

The silty clay fill through which the soldier piles would be installed may contain boulders or other obstructions, although none were encountered during the current borehole investigation and none

were listed on the 1975 borehole records. In addition, the shale bedrock contains limestone interbeds. Provision should be made in the contract to ensure that the Contractor's method of installation can handle these obstructions and / or hard layers.

5.4.2.1 Earth Pressure Distribution for Temporary Shoring Design

The design of strutted soldier pile and lagging walls should be based on a rectangular earth pressure distribution, while the design of shoring walls supported by anchors or rakers should be based on a triangular earth pressure distribution; appropriate distributions and design parameters are provided below. Surcharge loadings, as for traffic on Highway 403, must be added to these distributions.

The unfactored triangular earth pressure distribution can be calculated as follows:

$$p = K_a (\gamma H - p_w) + p_w$$

where

$$H = \text{the height of the excavation at any point in metres}$$
$$K_a = 0.3 \text{ for level ground behind excavation}$$
$$\gamma = \text{soil unit weight} = 21 \text{ kN/m}^3$$
$$p_w = \text{porewater pressure} = 0 \text{ for this site}$$

The unfactored rectangular earth pressure distribution can be calculated as follows:

$$p = K \gamma H$$

where

$$H = \text{the total height of the excavation}$$
$$K = 0.3 \text{ for level ground behind excavation}$$
$$\gamma = \text{soil unit weight} = 21 \text{ kN/m}^3$$

The passive toe restraint to the soldier piles may be determined using a triangular earth pressure distribution acting over an equivalent width equal to three times the pile width or pile socket diameter. The coefficient of passive lateral earth pressure, K_p , may be taken as follows:

In very stiff to hard silty clay fill:	$K_p =$	3.3
In shale bedrock:	$K_p =$	7.5

5.4.2.2 Temporary Soil Anchor Design

The working load on temporary soil or rock anchors should not be greater than 60 per cent of the ultimate tensile strength of the anchor tendons or bars. The anchors may be sized based on the following ultimate bond stresses acting between the grout and the soil / rock; the ultimate anchor capacity calculated from these adhesion values should be reduced by a factor of safety of at least 1.5:

Very stiff to hard silty clay fill:	50 kPa
Shale / limestone bedrock:	300 kPa

Because the ground-to-anchor bond is highly dependent upon the installation technique, the Contractor should be held to an anchor performance specification enforced by proof tests on all anchors. The Contract Documents should incorporate a Non-Standard Special Provision (NSSP) to this end. Temporary anchor installation and testing should be carried out under the full-time inspection of a geotechnical engineer. A performance test should be carried out, to at least 150 per cent of the design working load, on at least one anchor to confirm the design and the Contractor's installation method. In addition, each anchor should be proof-tested to 125 per cent of the working load. The tensile stress in the anchor bar or strands during test loading should not exceed 80 per cent of the guaranteed ultimate tensile strength of the bar or strands. Anchor installation and preloading should be complete before the excavation proceeds below the anchor elevation.

5.5 Embankment Construction

Placement of additional embankment fill material will be required in the median area of Highway 403; a grade raise of up to about 7 m will be required within the limits of the approach embankments.

5.5.1 Embankment Configuration and Settlement

Provided that the approach embankment subgrade is properly prepared, as outlined below, settlement of the existing silty clay fill is expected to be negligible; any settlement is likely to occur within the new embankment fill itself. In order to minimize differential settlement between the

widened portion of Highway 403 and the existing embankments, the use of granular fill is recommended for the widening. The majority of settlement of granular fills will occur during construction whereas the majority of settlement of cohesive fills, if used, would occur post-construction. Fill placement should be controlled as outlined in Section 5.5.2.

In addition, keying of the new embankment fill into the existing side slopes along the eastbound and westbound lanes of Highway 403 would help to reduce the impact of differential settlement. Such benching should be carried out in accordance with OPSD 208.01.

Outside of the new median retaining wall areas, the approach embankments for the widening should be constructed with side slope gradients of 2 horizontal to 1 vertical (2H:1V). Placement of topsoil and seeding, or pegged sod, will be required to reduce surface water erosion on the side slopes.

5.5.2 Subgrade Preparation and Fill Placement

Topsoil and softened or disturbed fill soils should be stripped from below the approach embankment areas, and the subgrade proof-rolled to delineate any softened or disturbed areas; such areas will require subexcavation and replacement with compacted granular soil prior to placement of the embankment fill. As noted above, construction of the embankment above the prepared subgrade should be carried out using Select Subgrade Material or other granular fill meeting the specifications of OPSS 1010. All embankment fill should be placed in regular lifts with loose thickness not exceeding 300 mm, and be compacted to at least 95 per cent of the material's Standard Proctor maximum dry density. The final lift prior to placement of the granular subbase or base course should be compacted to 100 per cent of the Standard Proctor maximum dry density. Inspection and field density testing should be carried out by qualified geotechnical personnel during fill placement operations to ensure that appropriate materials are used and that adequate levels of compaction have been achieved.

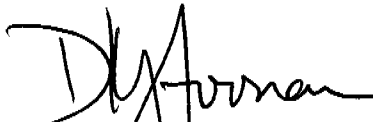
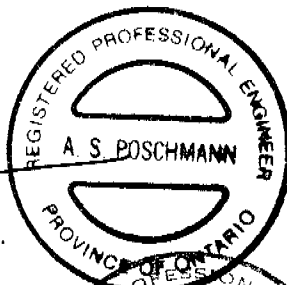
GOLDER ASSOCIATES LTD.



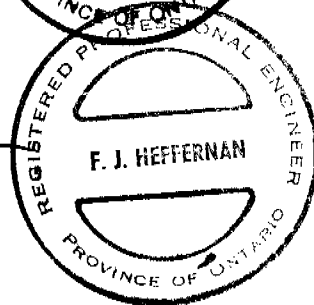
Lisa C. Coyne, P.Eng.
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LCC/ASP/FJH/clg

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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

AS Auger sample
BS Block sample
CS Chunk sample
SS Split-spoon
DS Denison type sample
FS Foil sample
RC Rock core
SC Soil core
ST Slotted tube
TO Thin-walled, open
TP Thin-walled, piston
WS Wash sample

III. SOIL DESCRIPTION

(a) Cohesionless Soils

Density Index (Relative Density)	N Blows/300 mm or Blows/ft.
Very loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very dense	over 50

II. PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:

The number of blows by a 63.5 kg. (140 lb.) hammer dropped 760 mm (30 in.) required to drive a 50 mm (2 in.) drive open sampler for a distance of 300 mm (12 in.)

Consistency

	c_{us}	s_u
	kPa	psf
Very soft	0 to 12	0 to 250
Soft	12 to 25	250 to 500
Firm	25 to 50	500 to 1,000
Stiff	50 to 100	1,000 to 2,000
Very stiff	100 to 200	2,000 to 4,000
Hard	over 200	over 4,000

Dynamic Cone Penetration Resistance; N_d :

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
PM: Sampler advanced by manual pressure
WH: Sampler advanced by static weight of hammer
WR: Sampler advanced by weight of sampler and rod

Piezo-Cone Penetration Test (CPT)

A electronic cone penetrometer with a 60° conical tip and a project end area of 10 cm² pushed through ground at a penetration rate of 2 cm/s. Measurements of tip resistance (Q_t), porewater pressure (PWP) and friction along a sleeve are recorded electronically at 25 mm penetration intervals.

IV. SOIL TESTS

w water content
 w_p plastic limit
 w_l liquid limit
C consolidation (oedometer) test
CHEM chemical analysis (refer to text)
CID consolidated isotropically drained triaxial test¹
CIU consolidated isotropically undrained triaxial test with porewater pressure measurement¹
 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 (LV-laboratory vane test)
 γ unit weight

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

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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 ($D_R = \rho_s / \rho_w$) (formerly G_s)
e	void ratio
n	porosity
S	degree of saturation
* Density symbol is ρ . Unit weight symbol is γ where $\gamma = \rho g$ (i.e. mass density \times acceleration due to gravity)	

(a) Index Properties (con't.)

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)

(c) 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

(d) Consolidation (one-dimensional)

C_c	compression index (normally consolidated range)
C_r	recompression index (overconsolidated range)
C_s	swelling index
C_α	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 $= \sigma'_p / \sigma'_{vo}$

(e) Shear Strength

τ_p, τ_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

Description	Bedding Plane Spacing
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

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

GRAIN SIZE

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

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

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 to 100% for core in solid sticks.

DISCONTINUITY DATA

Fracture Index

A count of the number of discontinuities (physical separations) in the rock core, including both naturally occurring fractures and 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 of fracture surfaces and infillings are also noted.

Abbreviations

B - Bedding	P - Polished
FO - Foliation/Schistosity	S - Slickensided
CL - Cleavage	SM - Smooth
SH - Shear Plane/Zone	R - Ridged/Rough
VN - Vein	ST - Stepped
F - Fault	PL - Planar
CO - Contact	FL - Flexured
J - Joint	UE - Uneven
FR - Fracture	W - Wavy
MF - Mechanical Fracture	C - Curved
- Parallel To	
⊥ - Perpendicular To	

PROJECT 001-1131-B				RECORD OF BOREHOLE No 1				1 OF 1		METRIC				
W.P. 613-89-00				LOCATION N 4832283.5; E 292856.1				ORIGINATED BY GM						
DIST HWY 403				BOREHOLE TYPE 108mm Dia. Power Hand Auger				COMPILED BY LCC						
DATUM Geodetic				DATE Nov.29/00				CHECKED BY LCC						
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						
164.90	GROUND SURFACE							20 40 60 80 100						
0.00	Silty Clay, trace shale, sand and gravel (Fill) Grey Moist		1	AS										
			2	AS										
163.68			3	AS										
1.22	END OF BOREHOLE		4	AS										
	Note: Borehole dry upon completion of augering.													

ON MOT. 0011131B.GPJ ON MOT.GDT 6/12/00

PROJECT <u>001-1131-B</u>				RECORD OF BOREHOLE No 2				1 OF 1				METRIC				
W.P. <u>613-89-00</u>				LOCATION <u>N 4832230.7; E 292897.9</u>				ORIGINATED BY <u>GM</u>								
DIST <u> </u> HWY <u>403</u>				BOREHOLE TYPE <u>108mm Solid Stem Augers</u>				COMPILED BY <u>LCC</u>								
DATUM <u>Geodetic</u>				DATE <u>Oct. 3/00</u>				CHECKED BY <u>LCC</u>								
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 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL X REMOULDED 20 40 60 80 100								
185.70	GROUND SURFACE															
0.00	Silty Clay, trace sand and shale fragments (Fill) Stiff to hard Brown to grey-brown Moist		1	SS	12											
			2	SS	75											
			3	SS	60											
			4	SS	14											
182.65																
3.05	Shale with limestone interbeds (Georgian Bay Formation) Weathered Grey		5	SS	54											
			6	SS	65/15											
180.98																
4.72	END OF BOREHOLE Note: Borehole dry on completion of drilling.		7	SS	63/15											

ON MOT 0011131B.GPJ ON MOT.GDT 6/12/00

PROJECT <u>001-1131-B</u>				RECORD OF BOREHOLE No 3				1 OF 1		METRIC							
W.P. <u>613-89-00</u>				LOCATION <u>N 4832239.8; E 292906.8</u>				ORIGINATED BY <u>GM</u>									
DIST <u> </u> HWY <u>403</u>				BOREHOLE TYPE <u>108mm Solid Stem Augers</u>				COMPILED BY <u>LCC</u>									
DATUM <u>Geodetic</u>				DATE <u>Oct.3/00</u>				CHECKED BY <u>LCC</u>									
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									
<div style="display: flex; justify-content: space-around; font-size: small;"> 20 40 60 80 100 20 40 60 80 100 </div> <div style="display: flex; justify-content: space-around; font-size: x-small;"> ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x REMOULDED </div>																	
165.50	GROUND SURFACE																
0.00	Silty Clay, trace sand and gravel, trace organics (Fill)		1	SS	7												
165.04	Firm																
0.46	Brown																
	Moist																
	Silty Clay, trace to some sand, gravel and shale fragments (Fill)		2	SS	25												
	Very stiff to hard																
	Grey-brown to grey																
	Moist																
			3	SS	26												
			4	SS	65												
			5	SS	70/15												
161.69																	
3.81	Shale with limestone interbeds (Georgian Bay Formation) Weathered to slightly weathered Grey		6	SS	95/23												
			7	SS	75/15												
	Bedrock cored between 4.7m and 7.3m depth For bedrock coring details, refer to Record of Drillhole 3.																
158.18																	
7.32	END OF BOREHOLE																
	Note: Borehole dry on completion of drilling, prior to commencing rock coring.																

ON MOT 0011131B.GPJ ON MOT GDT 6/12/00

PROJECT: 001-1131-B

RECORD OF DRILLHOLE: 3

SHEET 2 OF 2

LOCATION: N 4832239.6; E 292906.8

DRILLING DATE: Oct.3/00

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: —

DRILL RIG: CME-45

DRILLING CONTRACTOR: MSI

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	PENETRATION RATE (mm/min)	FLUSH % RETURN	CORRECTION												DIAMETRAL POINT LOAD INDEX (MPa)	NOTES WATER LEVELS INSTRUMENTATION	
								FR-FRACTURE				F-FAULT		SM-SMOOTH		FL-FLEXURED		BC-BROKEN CORE				
								CL-CLEAVAGE				J-JOINT		R-ROUGH		UE-UNEVEN		MB-MECH. BREAK				
								SH-SHEAR				P-POLISHED		ST-STEPPED		W-WAVY		B-BEDDING				
								VN-VEIN				S-SLICKENSIDED		PL-PLANAR		C-CURVED						
RECOVERY				R.Q.D. %		FRACT. INDEX PER 0.3		DISCONTINUITY DATA				HYDRAULIC CONDUCTIVITY k, cm/sec										
TOTAL CORE %				SOLID CORE %						DIP w.r.t. CORE AXIS				TYPE AND SURFACE DESCRIPTION								
8888				8888		8888		8888		8888				8888				8888				
		CONTINUED		180.93																		
5		Shale with limestone interbeds (Georgian Bay Formation) Weak to very weak Moderately to slightly weathered Grey Limestone interlayers: 4.72m - 4.78m 5.23m - 5.28m 5.82m - 5.87m 6.20m - 6.35m 6.53m - 6.55m 6.91m - 7.32m		4.57	1		100															
6					2		100															
7				158.18																		
		END OF BOREHOLE		7.32																		
8																						
9																						
10																						
11																						
12																						
13																						
14																						

DRILLHOLE 1131BROC.GPJ GLDR CAN GDT 6/12/00 PS

DEPTH SCALE

1 : 50



LOGGED: GM

CHECKED: LCC

PROJECT 001-1131-B				RECORD OF BOREHOLE No 4				1 OF 1		METRIC									
W.P. 613-89-00				LOCATION N 4832220.4; E 292916.5				ORIGINATED BY GM											
DIST HWY 403				BOREHOLE TYPE 108mm Solid Stem Augers				COMPILED BY LCC											
DATUM Geodetic				DATE Oct. 3/00				CHECKED BY LCC											
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			20	40	60	80	100						20	40
165.10 0.00	GROUND SURFACE																		
	Silty Clay, trace to some sand, gravel and shale fragments (Fill) Stiff to hard Brown to grey Moist		1	SS	52														
	Samples 1 and 4 contain trace organics and rootlets		2	SS	10														
			3	SS	55 100														
			4	SS	25														
162.36 2.74	END OF BOREHOLE																		
	Note: Borehole dry on completion of drilling.																		

ON MOT 0011131B.GPJ ON MOT.GDT 6/12/00

PROJECT 001-1131-B				RECORD OF BOREHOLE No 5				1 OF 1		METRIC					
W.P. 613-89-00				LOCATION N 4832267.1; E 292883.5				ORIGINATED BY GM							
DIST HWY 403				BOREHOLE TYPE 108mm Solid Stem Augers				COMPILED BY LCC							
DATUM Geodetic				DATE Oct 24/00				CHECKED BY LCC							
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 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL X REMOULDED							
159.80	GROUND SURFACE														
8.98 0.10	Concrete Sand and Gravel (Fill)		1	SS	10										
159.19 0.63	Silty Clay, some shale fragments, trace sand and gravel Stiff Grey Moist Shale with limestone interbeds (Georgian Bay Formation) Weathered Grey Bedrock cored between 1.2m and 3.8m depth For bedrock coring details, refer to Record of Drillhole 5.		2	SS	123/23										
							159								
							158								
							157								
155.99 3.81	END OF BOREHOLE Note: Borehole dry on completion of drilling, prior to commencing rock coring.						156								

ON MOT 0011131B.GPJ ON MOT.GDT 7/12/00

PROJECT: 001-1131-B

RECORD OF DRILLHOLE: 5

SHEET 2 OF 2

LOCATION: N 4832267.1; E 292883.5

DRILLING DATE: Oct.24/00

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: —

DRILL RIG: D-90

DRILLING CONTRACTOR: MSI

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-SUCKENSIDED	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
		CONTINUED		158.58										
		Shale with limestone interbeds (Georgian Bay Formation) Very weak to weak Thinly to very thinly bedded Moderately to slightly weathered Grey		1.22										
2	Diamond Drilling NQ-Size Core				1		50							
					2		50							
3					3		60							
					4		60							
4		END OF BOREHOLE		155.99 3.81										
5														
6														
7														
8														
9														
10														
11														

DEPTH SCALE

1 : 50



LOGGED: GM

CHECKED: LCC

DRILLHOLE 1131BROC.GPJ GLDR CAN.GDT 8/12/00 PS

OVERSIZE DRAWING

January 2001

001-1131B

APPENDIX A
RECORDS OF BOREHOLES – 1975 MTO INVESTIGATION

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS-ONTARIO
ENGINEERING SERVICES BRANCH-GEOTECHNICAL OFFICE-SOIL MECHANICS SECTION

RECORD OF BOREHOLE NO 3

WP 36-74-02/03 LOCATION Co-ords. 15,853,023 N; 960,775 E. ORIGINATED BY VK
DIST 6 HWY 403 BORING DATE December 1, 1975 COMPILED BY VK
DATUM Geodetic BOREHOLE TYPE CME (5.1) M.V.H.S. CHECKED BY N.J.

SOIL PROFILE			SAMPLES			GROUND WATER ELEV	DYNAMIC CONE PENETRATION RESISTANCE PLOT					LIQUID LIMIT w_L PLASTIC LIMIT w_p WATER CONTENT w			UNIT WEIGHT γ	REMARKS
ELEV DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	'N' VALUES		20 40 60 80 100					w_p w w_L				
							SHEAR STRENGTH					WATER CONTENT %				
544.5	Ground Level															
	Topsoil															
1.5	Weathered															
539.5			1	SS	102	540										
5.0	Shale Sound															
536.4			2	SS	100.1"											
8.1	End of Borehole															
														</		

RECORD OF BOREHOLE NO 4

WP 36-74-02/03 LOCATION Co-ords. 15,853,080.N; 960,723 E. ORIGINATED BY VK
 DIST 6 HWY 403 BORING DATE November 28, 1975 COMPILED BY VK
 DATUM Geodetic BOREHOLE TYPE CME (5.1) M.V.H.S. CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER ELEV	DYNAMIC CONE PENETRATION RESISTANCE PLOT					LIQUID LIMIT w_L PLASTIC LIMIT w_p WATER CONTENT w			UNIT WEIGHT γ	REMARKS % GR S A S
ELEV DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	'N' VALUES		20	40	60	80	100	w_p	w	w_L		
546.1	Ground Level															
	Topsoil															
1.5	Weathered		1	SS	100%	540										
537.1																
9.0	Sound		2	BXL	Rec 100%	530										
			3	BXL	Rec 100%											
	Shale Bedrock with limestone bands		4	BXL	Rec 100%	520										
			5	BXL	Rec 100%											
517.1																
29.0	End of Borehole					510										

ENGINEERING SERVICES BRANCH-GEOTECHNICAL OFFICE-SOIL MECHANICS SECTION


RECORD OF BOREHOLE NO 5

WP 36-74-02/03 LOCATION Co-ords. 15,853,174 N; 960,970 E. ORIGINATED BY VK
 DIST 6 HWY 403 BORING DATE December 1, 1975 COMPILED BY VK
 DATUM Geodetic BOREHOLE TYPE CME (5.1) M.V.H.S. CHECKED BY *MS*

SOIL PROFILE			SAMPLES			GROUND WATER	DYNAMIC CONE PENETRATION RESISTANCE PLOT					LIQUID LIMIT — w_L PLASTIC LIMIT — w_p WATER CONTENT — w			UNIT WEIGHT γ	REMARKS
ELEV DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	'N' VALUES		20	40	60	80	100	w_p	w	w_L		
540.4	Ground Level					ELEV										
	Topsoil					540										
1.5																
	Weathered		1	SS	91											
530.4			2	SS	100	530										
10.0			3	BXL	100%											
	Sound		4	BXL	Rec 80%											
	Shale Bedrock with limestone bands		5	BXL	Rec 80%	520										
			6	BXL	Rec 95%											
510.4						510										
30.0	End of Borehole															

RECORD OF BOREHOLE No 6

WP 36-74-02/03 LOCATION Co-ords. 15,853,030 N; 960,930 E. ORIGINATED BY VK
 DIST 6 HWY 403 BORING DATE December 1, 1975 COMPILED BY VK
 DATUM Geodetic BOREHOLE TYPE CME (5.1) M.V.H.S. CHECKED BY WJ

SOIL PROFILE			SAMPLES			GROUND WATER ELEV	DYNAMIC CONE PENETRATION RESISTANCE PLOT					LIQUID LIMIT w_L PLASTIC LIMIT w_p WATER CONTENT w			UNIT WEIGHT γ	REMARKS	
ELEV DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	'N' VALUES		20	40	60	80	100	SHEAR STRENGTH O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL X LAB VANE					w_p
542.0	Ground Level																
	Topsoil																
1.5	Weathered Shale					540											
537.0																	
5.0	End of Borehole					530											

DOCUMENT MICROFILMING IDENTIFICATION

GEOCRES No. 30M12-249

DIST. CR REGION

W.P. No. 613-89-00

CONT. No.

W. O. No.

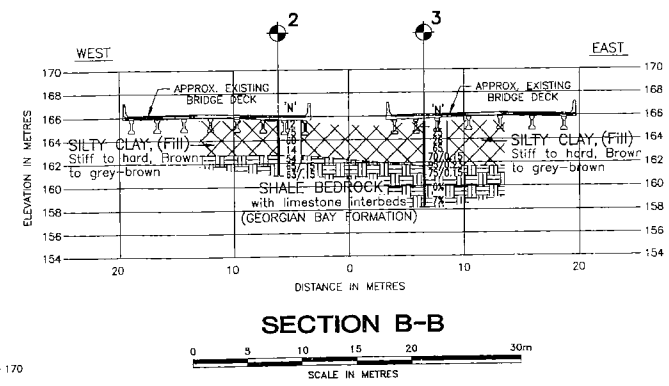
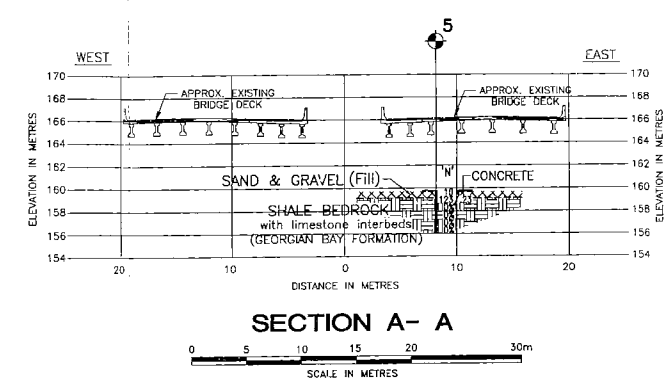
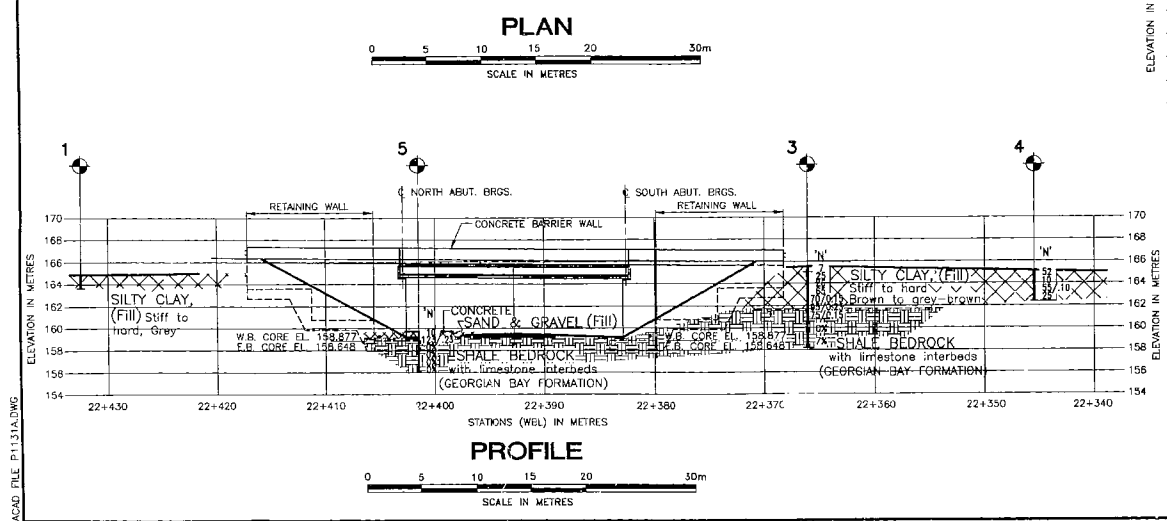
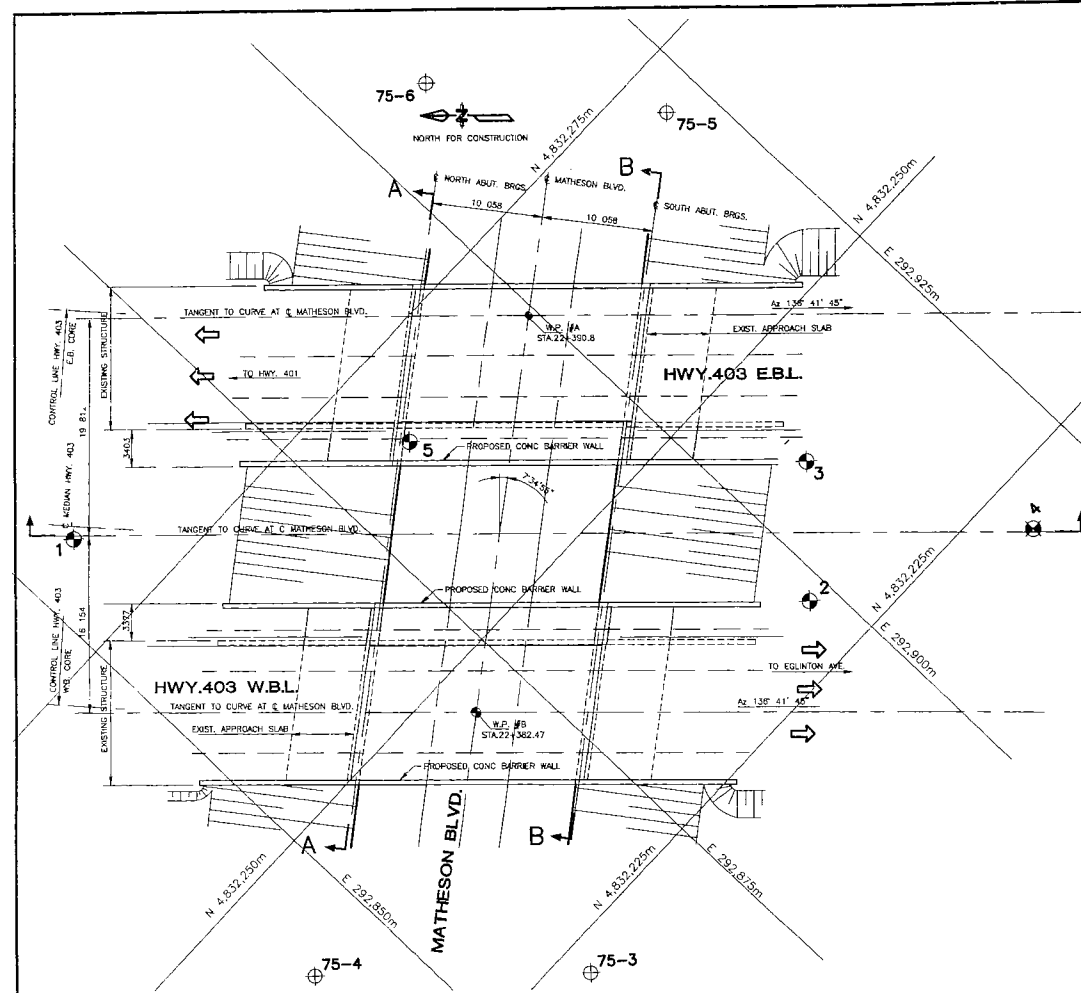
STR. SITE No. 24-354

HWY. No. 403

LOCATION MATHESON BLVD BRIDGE
4 Hwy 403

OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT.

REMARKS:



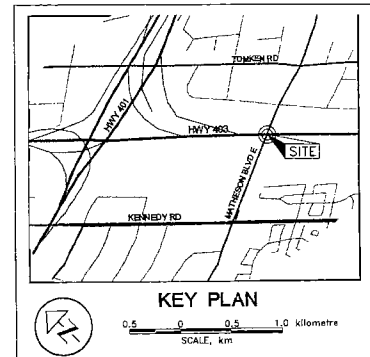
METRIC
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN

CONT. No.
WP No. 613-89-00

MATHESON BLVD. BRIDGE
HIGHWAY No. 403
BOREHOLE LOCATIONS & SOIL STRATA

Golder Associates
Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA

THE GREER GALLOWAY GROUP INC.
ENGINEERS + PLANNERS
PETERBOROUGH • OSHAWA • BANCROFT • BELLEVILLE • PEMBROKE



LEGEND			
	Borehole Location, 2000 Investigation		
	Approx. Borehole Location, 1975 Investigation		
'N'	'N' Blows/0.3m (Std. Pen. Test, 475 j/blow)		
RQD	Rock Quality Designation, %		
	WL at time of investigation		
No.	ELEVATION	CO-ORDINATES	
		NORTHING	EASTING
1	164.9m	4,832,283.54	292,856.08
2	165.7m	4,832,230.66	292,897.87
3	165.5m	4,832,239.60	292,906.77
4	165.1m	4,832,220.39	292,916.45
5	159.8m	4,832,267.07	292,893.50
75-3	166.0m	4,832,222.12	292,859.74
75-4	166.5m	4,832,240.24	292,842.29
75-5	164.7m	4,832,270.62	292,921.22
75-6	165.2m	4,832,288.46	292,908.25

NO.	DATE	BY	REVISION

Geocres No.	PROJECT NO. 001-11318
HWY. No. 403	DATE: 2001 01 12
SUB'D. LCC	DATE: 2001 01 12
DRAWN: MHW	DATE: 2001 01 12
CHKD: LCC	DATE: 2001 01 12
APPD: ASP	DATE: 2001 01 12