

GEOCRES No. 31C-158DIST. 41 REGION W.P. No. 76-99-01CONT. No. W. O. No. STR. SITE No. HWY. No. 401LOCATION CP RAILWAYOVERHEAD

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OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT.

REMARKS:

**Singuliano, Tony (MTO)**

**To:** Baird, Mike (MTO)

**Subject:** CP Rail Overhead Recreational Culvert - Hwy 401 - WP 76-99-01

Mike:

We have reviewed Golder's final report for the abovementioned structure. The report addresses our comments previously submitted in our memorandum dated June 05, 2000. We have no further comments.

Tony

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REPORT ON

GEOLRES # 31C-158

**FOUNDATION INVESTIGATION AND DESIGN  
CP RAIL OVERHEAD RECREATIONAL CULVERT  
HIGHWAY 401 FROM 2.7 KM WEST OF HIGHWAY 38,  
EASTERLY TO HIGHWAY 15  
DISTRICT 41, KINGSTON, EASTERN REGION  
WP: 76-99-01  
AGREEMENT NO. 4005-A-000069**

Submitted to:

Ministry of Transportation, Ontario  
Geotechnical Section, Engineering Office  
355 Counter Street  
P.O. Box #4000  
Kingston, Ontario  
K7L 5A3

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**PART A – FIELD INVESTIGATION**  
**CP RAIL OVERHEAD RECREATIONAL CULVERT**  
**HIGHWAY 401 FROM 2.7 KM WEST OF HIGHWAY 38,**  
**EASTERLY TO HIGHWAY 15**  
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List of Abbreviations and Symbols

Record of Borehole Sheets (Boreholes 1-1 to 1-4)

Drawing 1

**LIST OF DRAWINGS**

Drawing 1      Borehole Locations and Soil Strata – Highway 401 at CP Rail Overhead

## 1.0 INTRODUCTION

Golder Associates Ltd. has been retained by the Ministry of Transportation, Ontario (MTO) to carry out a foundation investigation at the abandoned CP Rail overhead along Highway 401 (Site 7-058) near Kingston, Ontario. This investigation forms part of the overall project which involves the widening of Highway 401 from 2.7 km west of Highway 38 easterly to Highway 15. The foundation design component includes the placement of a recreational culvert within the CP Rail overhead opening, the widening of the existing culvert at Little Cataraqui Creek and a new bridge structure at the Montreal Street underpass. This report addresses the proposed recreational culvert structure to be located within the existing opening of the CP Rail overhead.

The purpose of the foundation investigation is to determine the subsurface conditions at the site of the proposed culvert structure by drilling boreholes, and carrying out in-situ tests and laboratory tests on selected samples, where appropriate. 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 problems where they may affect the design of the proposed culvert.

A plan and profile of the existing CP Rail overhead structure were provided to us at 1:1000 scale by MTO. The profile drawing also indicates the proposed Highway 401 grade in the area of the proposed structure.

The terms of reference for the scope of work are outlined in our original proposal letter P01-1008, dated January 27, 2000 and our revised proposal letter P01-1008, dated March 21, 2000.

## 2.0 SITE DESCRIPTION

The site is located some 500 m to the east of the Highway 38 underpass along Highway 401, near Kingston, Ontario.

The topography of the site area is generally level with a regional trend sloping down to the south towards Lake Ontario. The ground surface at the site varies locally from about Elevations 110.5 m to 112 m, and locally slopes down towards the north. The existing Highway 401 in the area is four-lane and divided and runs east-west within the project limits. Based on available information, the approximate existing grade of Highway 401 at the CP Rail overhead is about Elevation 121 m. It is understood that the existing opening of the abandoned CP Rail overhead is part of the Trans-Canada Trail system. The trail appears to be in partial cut south of Highway 401.

The CP Rail overhead of Highway 401 consists of a concrete arch culvert that runs below the Highway, and is about 9 m high and about 15 m wide. Drainage to the culvert is provided by weep holes near the base of the culvert on both the north and south sides. Water was flowing from the weep holes at the time of the investigation, in particular at the south end of the culvert where the trail is in partial cut.

Within the project limits, the vegetation cover generally consists of grass, bushes, and mature trees.



### 3.0 INVESTIGATION PROCEDURES

The field work for this investigation was carried out on April 17, 2000. At this time four boreholes were put down at the site. Boreholes 1-1 and 1-2 were put down near the limits of the proposed structure on the north side of Highway 401. Boreholes 1-3 and 1-4 were advanced near the limits of the proposed structure on the south side of Highway 401. The boreholes were extended to depths of between 0.2 m and 4.7 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 1-1 and 1-4. The open boreholes were backfilled with a mixture of auger cuttings and bentonite. Groundwater conditions in the open boreholes were observed throughout the drilling operation and upon completion of drilling. Piezometers were installed in Boreholes 1-1 and 1-4 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 labeled 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.

## **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 Physiography of Southern Ontario, Chapman and Putnam, 3<sup>rd</sup> 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 Phanerozoic 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; a few bedrock outcroppings were noted in the area of the subject site.

### **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 silty clay or sand and gravel. Bedrock was encountered or inferred from refusal to further auger penetration at about Elevation 110 m on the north side of Highway 401 and at about Elevation 111 m on the south side of Highway 401. A detailed description of the subsurface conditions encountered in the boreholes for this investigation is provided in the following sections.

#### **4.2.1 Topsoil**

A 150 mm thick surficial layer of topsoil was encountered in Borehole 1-1.

#### **4.2.2 Sand and Gravel**

About 230 mm of sand and gravel, trace silt was encountered surficially in Borehole 1-2.

#### **4.2.3 Silty Clay**

A deposit of silty clay, between 0.8 m and 1.2 m thick, was encountered below the topsoil in Borehole 1-1 and surficially in Boreholes 1-3 and 1-4. Trace sand and gravel, and occasional organics consisting primarily of fine rootlets were noted within the silty clay. Standard Penetration testing carried out within the silty clay measured 'N' values of 2 blows and 10 blows per 0.3 m of penetration, which indicates a soft to stiff consistency. The natural water contents for two selected samples of the silty clay were measured at about 40 percent.

#### **4.2.4 Bedrock**

Bedrock was encountered at about Elevation 110.0 m (about 1.0 m depth) in Borehole 1-1 and at about Elevation 110.8 m (about 1.2 m depth) in Borehole 1-4. Bedrock was inferred from refusal to further auger penetration in Boreholes 1-2 and 1-3 at Elevation 110.4 m (0.2 m depth) and Elevation 111.3 m (0.8 m depth), respectively. Boreholes 1-1 and 1-4 were advanced about 3.2 m and 3.4 m, respectively, into the bedrock by coring in NQ size. The rock core samples consist of grey, fresh, thinly bedded, fine grained to micritic limestone. The Rock Quality Designation (RQD) measured on the core samples ranged from about 75 percent to greater than 90 percent, indicating the rock mass is good to excellent quality. In general, the measured RQD values increased with depth. The rock is classified as moderately strong; Grade 3 to 4, according to the Canadian Engineering Foundation Manual (CFEM, 3<sup>rd</sup> Edition, 1992). Strength testing carried out on three sections of the recovered core gave diametral point load indices of between 2.5 MPa to greater than 11 MPa.

#### 4.2.5 Groundwater Conditions

The water level in the open boreholes was observed during and upon completion of the drilling operation. A piezometer was installed in Boreholes 1-1 and 1-4 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 Borehole at Completion of Drilling		In Piezometer			
			April 26, 2000		May 21, 2000	
	Depth (m)	Elevation (m)	Depth (m)	Elevation (m)	Depth (m)	Elevation (m)
1-1	0.3*	110.7*	0.5	110.5	1.2	109.8
1-2	At ground surface	110.6**	N/A	N/A	N/A	N/A
1-3	At ground surface	112.0**	N/A	N/A	N/A	N/A
1-4	0.1*	111.9*	0.1	111.9	0.2	111.8


\* water level measured in piezometer


\*\* ground surface elevation

The above results indicate that the groundwater table generally follows the ground surface topography and slopes downward toward the north.


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

**GOLDER ASSOCIATES LTD.**

  
for: Dan K. Breeze, B.Sc.

  
Anne S. Poschmann, P.Eng.  
Principal



  
Fintan J. Heffernan, P.Eng.  
Designated MTO Contact



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**PART B – FOUNDATION DESIGN**  
**CP RAIL OVERHEAD RECREATIONAL CULVERT**  
**HIGHWAY 401 FROM 2.7 KM WEST OF HIGHWAY 38,**  
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## **5.0 ENGINEERING RECOMMENDATIONS**

### **5.1 General**

This section of the report provides our recommendations on the geotechnical aspects of design of the proposed culvert structure to be located within the abandoned CP Rail overhead 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 2.7 km west of Highway 38 easterly to Highway 15. This widening will involve the placement of a recreational culvert within the existing CP Rail overhead opening, an extension of the existing rigid frame structure at the Little Cataraqui Creek crossing, an a new bridge structure at the Montreal Street underpass. The works described in this report are associated with the proposed recreational culvert to be located within the existing opening at the abandoned CP Rail overhead at about Station 18+550. Based on the information provided, the proposed grade of Highway 401 at the culvert will be lowered about 1.5 m from the current pavement grade and the final grade will be at about Elevation 119.5 m. It is understood that consideration is being given to replacing the existing structure with either a single span concrete culvert or a large diameter corrugated steel pipe. It is further understood that utility conduits may be installed below the invert of the culvert.

The drainage of the existing culvert is by weep holes which were flowing in mid-April 2000, in particular at the south end where the trail is in partial cut.

### **5.2 Culvert Foundations**

Shallow (less than 1.2 m) overburden deposits overlie the bedrock at this site. The bedrock surface varies by about 1.3 m at the four borehole locations and is at about Elevation 110 m on the north side of Highway 401 and at about Elevation 111 m on the south side of Highway 401. The bedrock

consists of fresh, fine grained to micritic, moderately strong limestone of good to excellent quality. The groundwater table is at about Elevation 110 m (1.2 m depth) on the north side of Highway 401 and at about Elevation 112 m (0.2 m depth) on the south side of Highway 401.

Shallow spread footings are considered to be the most feasible founding alternative for support of the proposed single span concrete culvert given the shallow depth of bedrock at this site.

The footings may be placed below the bedrock surface or may be placed directly on the exposed limestone bedrock surface after cleaning any loose or fractured rock. A design founding level of Elevation 110 m or lower may be assumed over the full length of the culvert for both abutments. This founding level will require variable bedrock excavation depths of up to at least 1.3 m but could be less than 0.5 m over a large proportion of the footing area. With this shallow depth of bedrock excavation, it can be difficult to achieve a uniform excavation base particularly with limestone bedrock which typically breaks along stratifications.

Alternatively, a design founding level varying from Elevation 110.0 m to 111.3 m may be assumed which would be associated with placing the footing directly on the bedrock surface. This alternative would minimize the amount of shallow bedrock excavation required.

For the variable founding level alternative, it should be noted that there may be loose / fractured bedrock at the founding level that should be removed prior to placing concrete. For the constant founding level alternative the design should be flexible enough to allow for some further variation in the bedrock surface elevation with both an allowance for placement of mass concrete to raise the grade to the founding level after exposing the bedrock and an allowance for removing additional bedrock if required.

#### **5.2.1 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 3,000 kPa. This value is for vertical concentric loads only. Serviceability Limit States (SLS) conditions do not apply to footings placed on the limestone bedrock which is classified as non-yielding. The factored resistance at ULS



should be reduced for inclined loading according to the OHBDC Clause 6-8.4.2 using the relationship for rock.

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.

### **5.2.2 Horizontal Resistance**

Resistance to lateral forces / sliding resistance between the concrete footings and bedrock should be calculated in accordance with Clause 6-8.4.3 of the OHBDC assuming an unfactored angle of friction of 35 degrees. If necessary, sliding resistance can be supplemented by doweling into bedrock.

A value of 350 kPa may be assumed for the grout-to-rock bond stress for ULS design. This value refers to the rock-grout interface and can be used for tension design. The actual bond stress along the rock-grout interface may vary from the typical design value given and should therefore be verified in the field. The dowels should be a minimum of 1.0 m long within the rock (embedded length in the rock) and the structural strength of the dowel and the compressive strength of the grout should not be exceeded. If a utility trench is provided below the culvert invert, it is recommended that it be installed prior to foundation construction and this trench be backfilled with concrete to maintain the horizontal resistance of the footings.

### **5.2.3 Frost Protection**

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

## **5.3 Lateral Earth Pressures – Rigid Frame Structure**

The lateral pressures acting on the culvert 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. For the proposed rigid frame culvert, it is assumed that lateral yielding of the abutment support will not be allowed and as such, at-rest pressures will apply.

The following recommendations are made concerning the design of the rigid frame culvert abutments in accordance with OHBDC:

- Select free-draining granular fill meeting the specifications of OPSS Granular 'A' or Granular 'B', Type II but with less than 5 percent passing the 200 sieve should be used as backfill behind the walls. All granular fill should be compacted in lifts of loose thickness not greater than 200 mm to 95 percent of the material's Standard Proctor maximum dry density.
- Longitudinal drains should be installed behind the base of the walls to provide positive drainage of the granular backfill towards the north. In addition, weep holes should be provided through the walls to guard against hydrostatic pressures during wet periods.
- 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 degree line extending up and back from the bottom of the rear face of the footing (Case II).
- 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 OHBDC Figure 6-7.4.3.
- For Case I, the pressures are based on the embankment fill materials and the following parameters (unfactored) may be assumed:

Soil unit weight (assuming clean earth fill)	21 kN/m <sup>3</sup>
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Coefficient of lateral earth pressure: 'at rest'	0.47
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- 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 <sup>3</sup>	21 kN/m <sup>3</sup>
Coefficient of Lateral Earth Pressure 'at rest'	0.43	0.47

It should be noted that the above design parameters assume level backfill and ground surface behind the wall. Other aspects of the abutment granular backfill requirements with respect to sub-drains and frost taper should be in accordance with OPSD-3501.00.

#### **5.4 Arch Culvert**

It is understood that consideration is also being given to the use of a CSP or multi-plate type of structure. The overburden materials are generally weak and highly variable and as such are not considered suitable for the support of the culvert. The structure should be founded on the bedrock surface. The factored capacity at ULS of the limestone bedrock as provided in Section 5.2.1 should be used for design. The pipe arch should be provided with at least 400 mm of OPSS Granular 'A' bedding shaped to the underside of the pipe arch. The OHBDC Clause 7-8.4 requires that a 200 mm layer of that portion of the bedding which is in direct contact with the pipe arch be left uncompacted to allow proper embedment of the corrugation profile. The remaining portion of the bedding should be compacted to at least 95 percent of the Standard Proctor maximum dry density using suitable vibratory compaction equipment. Specifications provided by the pipe manufacturer with respect to bedding requirements should be adhered to if different from the above general requirements. The minimum depth of bedding will depend on the size of the pipe and should adhere to the manufacturer's specifications. The utility trench should be installed before this construction and preferably backfilled with concrete or well compacted Granular 'A'.

The backfill to the culvert should be free-draining granular fill meeting the specifications of OPSS Granular 'A' or Granular 'B', Type II but with less than 5 percent passing the 200 sieve. Adequate and careful compaction of the backfill under the haunches is essential for the performance of the culvert. The backfill should be placed in lifts not exceeding 200 mm loose thickness and compacted to 95 percent Standard Proctor dry density. The fill depth during placement should be maintained equal on both sides of the culvert with one side not exceeding the other by more than 400 mm.

Since the performance of the arch culvert is highly dependant on the compaction of the haunches, equipment restrictions exist that do not apply for the rigid frame culvert option. The suppliers' specifications should be followed with respect to compaction equipment tolerances in close proximity to the culvert. As a guide, small air operated tampers or vibrating pads are typically

required for the compaction of the fill within 300 mm of the culvert sides while hand rollers or packers are typically specified for distances of up to 1.5 m from the culvert sides. Heavier compactors may generally be used for compaction of the remaining areas.

For granular backfill placed and compacted as specified above, a factored capacity at ULS of 700 kPa may be assumed where the granular fill bears against the limestone bedrock. Where the granular fill bears against the silty clay overburden, a factored capacity at ULS of 250 kPa should be assumed. The capacity at SLS for the latter case may be taken as 100 kPa. This value may be increased by providing a thrust beam to transfer the load at the haunches down to the bedrock whereby SLS design will not apply. Alternatively, the granular fill thickness at the haunches should be increased to provide improved bearing resistance. The extent of granular will be dependent on the size and configuration of the culvert.

Positive drainage for the haunches should be provided by weep holes placed along the length of the culvert near the base.

The culvert should be designed for the full overburden pressure and live load assuming a soil unit weight of 21 kN/m<sup>3</sup>.

## **5.5 Excavations**

### ***Footings***

Excavations for footing construction will extend through the surficial deposits of silty clay and sand and gravel and will be terminated on / within the limestone bedrock.

The groundwater level in the piezometer installed in Borehole 1-1 on the north side of Highway 401 was measured at Elevation 109.8 m which is below the bedrock surface. The water level in the piezometer in Borehole 1-4 on the south side of the highway was at Elevation 111.8 m which is above the bedrock surface. It is anticipated that groundwater inflow through the clayey native soils into the excavation will be minimal; however, some form of groundwater control may be required in order to construct the footing in the dry particularly for seepage at the

overburden / bedrock interface. It is considered that this inflow can be handled by conventional sump pumping at the base of the excavation. Sumps should be maintained outside the footing area.

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 1.5 horizontal to 1 vertical through the sand and gravel and 1 horizontal to 1 vertical through the silty clay.

### ***Removal of Existing Structure***

In order to remove the existing concrete arch structure, temporary cuts through the existing embankment will be required. It is assumed that the removal of the existing culvert will be carried out in stages to permit traffic to flow during construction (i.e. one direction of traffic closed at a time).

It is assumed that the embankment material is comprised of granular material. As such, temporary support in the form of a soldier pile and lagging system could be used. The soldier pile and lagging system would be required on either side of the culvert and along the median to allow for staged construction. The soldier piles should be socketed into the limestone bedrock or rock bolts installed to provide support to the soldier piles. Soldier piles should be installed in augered holes and temporary liners will be required to support the excavation. Support to the soldier pile and lagging wall can be provided by bracing across the excavation if the configuration allows and / or by rakers, as required. To avoid excessive loss of ground, lagging boards should be installed as soon as possible during excavation. The contract documents should require that any voids behind the lagging boards be backfilled and that a provision be made to filter fines such that there is no loss of ground from behind the lagging boards.

The design of the soldier pile and lagging wall should be based on the earth pressure distribution shown on Figure 1, assuming an average earth pressure coefficient (K) equal to 0.3 and unit weight of soil ( $\gamma$ ) of 21 kN/m<sup>3</sup>. A design groundwater level of Elevation 110 m and a construction surcharge of 16 kPa should be assumed. Unfactored socket resistances, can be determined based on the following relationship:

Where:	$R_p$	= $1.5 k_p \gamma H^2 D$
	$R_p$	= unfactored passive resistance of socket (kPa)
	$K_p$	= coefficient of passive resistance
	$\gamma$	= unit weight of soil (kN/m <sup>3</sup> )
	H	= rock socket depth (m)
	D	= diameter of pile (m)

A coefficient of passive resistance of 6.0, and a unit weight of soil of 20 kN/m<sup>3</sup> may be assumed for design. The resistance of the upper 1 m of the embedded length of the pile should be ignored. As per OHBDC requirements a resistance factor of 0.5 should be applied to obtain the factored ultimate geotechnical resistance.

As an alternative, consideration could be given to the use of an open cut with unsupported side slopes within the embankment to be maintained no steeper than 1.5 horizontal to 1 vertical. Soldier pile and lagging would be required along the median to allow for staged construction, as outlined above.

The proposed shoring system should be reviewed by a geotechnical engineer prior to installation. Stability and design checks should be made for each stage of the excavation.

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.

The contract documents should contain the MTO Special Provision 902501 – Excavation and Backfilling.

Roadway protection should be included in the contract documents as per current MTO end result specifications.

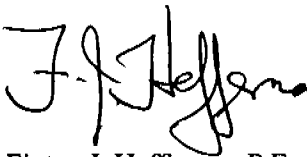
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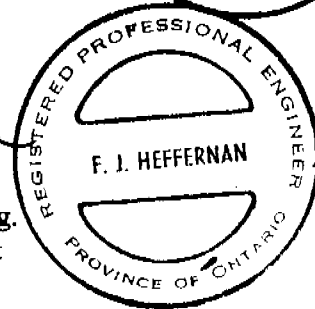
for: Dan K. Breeze, B.Sc.



Anne S. Poschmann, P.Eng.  
Principal



Fintan J. Heffernan, P.Eng.  
Designated MTO Contact



DKB/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

	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

#### (b) Cohesive Soils

#### 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 cm<sup>2</sup> 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<sub>p</sub> plastic limit  
w<sub>l</sub> liquid limit  
C consolidation (oedometer) test  
CHEM chemical analysis (refer to text)  
CID consolidated isotropically drained triaxial test<sup>1</sup>  
CIU consolidated isotropically undrained triaxial test with porewater pressure measurement<sup>1</sup>  
D<sub>R</sub> 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<sub>4</sub> 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.



## LIST OF SYMBOLS

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

### I. GENERAL

$\pi$	= 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

$\gamma$	shear strain
$\Delta$	change in, e.g. in stress: $\Delta \sigma$
$\epsilon$	linear strain
$\epsilon_v$	volumetric strain
$\eta$	coefficient of viscosity
$\nu$	Poisson's ratio
$\sigma$	total stress
$\sigma'$	effective stress ( $\sigma' = \sigma - u$ )
$\sigma'_{vo}$	initial effective overburden stress
$\sigma_1, \sigma_2, \sigma_3$	principal stresses (major, intermediate, minor)
$\sigma_{oct}$	mean stress or octahedral stress $= (\sigma_1 + \sigma_2 + \sigma_3)/3$
$\tau$	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
$\gamma'$	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 $\rho$ . Unit weight symbol is $\gamma$ where $\gamma = \rho g$ (i.e. mass density x 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
$C_\alpha$	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
$\sigma'_p$	pre-consolidation pressure
OCR	Overconsolidation ratio $= \sigma'_p / \sigma'_{vo}$

#### (e) Shear Strength

$\tau_p, \tau_r$	peak and residual shear strength
$\phi'$	effective angle of internal friction
$\delta$	angle of interface friction
$\mu$	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 = (Compressive strength)/2

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

ON\_MOT 001-1119.GPJ ON\_MOT.GDT 6/6/00

PROJECT: 001-1110

## RECORD OF DRILLHOLE: 1-1

SHEET 1 OF 1

LOCATION: N 4904818.13; E 299680.88

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-VEN	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 PORT LOAD INDEX (MPa)	NOTES WATER LEVELS INSTRUMENTATION
				110.00										
				1.01										
					1		100							
2														
					2		100							
3														
4					3		100							
				108.81										
				4.20										
		END OF HOLE												
5														
6														
7														
8														
9														
10														
11														

DEPTH SCALE

1:50



LOGGED: SB

CHECKED: MR

DRILLHOLE 1115ROCK.GPJ GLDR CAN.GDT 12/1000 PS

PROJECT 001-1119			RECORD OF BOREHOLE No 1-2			1 OF 1			METRIC				
W.P. 78-99-01			LOCATION N 4904807.01; E 299595.29			ORIGINATED BY SB							
DIST 41 HWY 401			BOREHOLE TYPE Hand Dug			COMPILED BY DKB							
DATUM Geodetic			DATE April 17, 2000			CHECKED BY ASP							
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC NATURAL LIQUID UNIT WEIGHT REMARKS & GRAIN SIZE DISTRIBUTION (%)				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	TV VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	20 40 60 80 100	PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	UNIT WEIGHT Y	REMARKS & GRAIN SIZE DISTRIBUTION (%)
110.60	GROUND SURFACE		1	GRAB									GR SA SI CL
110.89 0.23	Sand and Gravel, trace silt Brown Wet END OF HOLE Refusal to further shovel penetration; probable bedrock  Note: Water level in open borehole at ground surface upon completion of drilling.						110						

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

PROJECT <u>001-1119</u>		<b>RECORD OF BOREHOLE No 1-3</b>		1 OF 1	<b>METRIC</b>
W.P. <u>78-99-01</u>		LOCATION <u>N 4904758.20, E 299738.84</u>		ORIGINATED BY <u>SB</u>	
DIST <u>41</u> HWY <u>401</u>		BOREHOLE TYPE <u>108mm I.D. Hollow Stem Augers</u>		COMPILED BY <u>DKB</u>	
DATUM <u>Geodetic</u>		DATE <u>April 17, 2000</u>		CHECKED BY <u>ASP</u>	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	w <sub>L</sub> VALUES			SHEAR STRENGTH kPa										WATER CONTENT (%)		
								20	40	60	80	100						UNCONFINED	FIELD VANE	QUICK TRIAXIAL
112.03	GROUND SURFACE																			
0.00	Silty Clay, trace sand and gravel, occ. rootlets Soft		1	SS	3															
111.27	Brown Molai																			
0.76	END OF BOREHOLE Refusal to further auger penetration; probable bedrock  Note: Water level in open borehole at ground surface upon completion of drilling.																			

PROJECT 001-1119		RECORD OF BOREHOLE No 1-4				1 OF 1		METRIC								
W.P. 78-99-01		LOCATION N 4904749.17; E 299745.20				ORIGINATED BY SB										
DIST 41 HWY 401		BOREHOLE TYPE 108mm I.D. Hollow Stem Augers				COMPILED BY DKB										
DATUM Geodetic		DATE April 17, 2000				CHECKED BY ASP										
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC NATURAL LIQUID UNIT			REMARKS & GRAIN SIZE DISTRIBUTION (%)				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	T <sub>v</sub> VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	20 40 60 80 100	PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	GR	SA	SI	CL
111.99	GROUND SURFACE															
0.00	Silty Clay, trace sand and gravel, occ. rootlets to 0.6m depth. Soft to stiff. Brown. Moist.		1	SS	2											
110.80			2	SS	10		111									
1.19	Fresh, thinly bedded, fine grained to micritic, gray moderately strong LIMESTONE, occasionally mottled. Styolitic.						110									
							109									
							108									
107.32	Bedrock cored from 1.27m to 4.67m depth. For bedrock coring details refer to Record of Drillhole 1-4.															
4.67	END OF HOLE															
<p>Note:</p> <p>1. Water level measured in piezometer at 0.1m depth (El. 111.9m) upon completion of installation.</p> <p>2. Water level measured in piezometer at 0.1m depth (El. 111.9m) on April 28, 2000.</p> <p>3. Water level measured in piezometer at 0.2m depth (El. 111.8m) on May 21, 2000.</p>																

PROJECT: 001-1119

## RECORD OF DRILLHOLE: 1-4

SHEET 1 OF 1

LOCATION: N 4904749.17; E 299745.20

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)	CUTTING % RETURN	FR-FRACTURE		F-FAULT		SM-SMOOTH		FL-FLEXURED		BC-BROKEN CORE		DIAMETRAL POINT LOAD INDEX (MPa)	NOTES WATER LEVELS INSTRUMENTATION
								CL-CLEAVAGE	SH-SHEAR	J-JOINT	P-POLISHED	R-ROUGH	ST-STEPPED	UE-UNEVEN	W-WAVY	MB-MECH. BREAK	B-BEDDING		
								VAL-VEIN	S-SUCKEN/SIDED	PL-PLANAR	RECOVERY	R.O.D. %	FRACT. INDEX PER 0.3	DISCONTINUITY DATA		HYDRAULIC CONDUCTIVITY k, cm/sec			
TOTAL CORE %		SOLID CORE %		TYPE AND SURFACE DESCRIPTION															
		Fresh, thinly bedded, fine grained to micritic, grey, moderately strong LIMESTONE, occasionally mottled. Styolitic.		119.73 1.27	1		100												
2					2		100												
3	NO Core																		
4					3		100												
5		END OF HOLE		107.32 4.67															
6																			
7																			
8																			
9																			
10																			
11																			

DEPTH SCALE

1 : 50



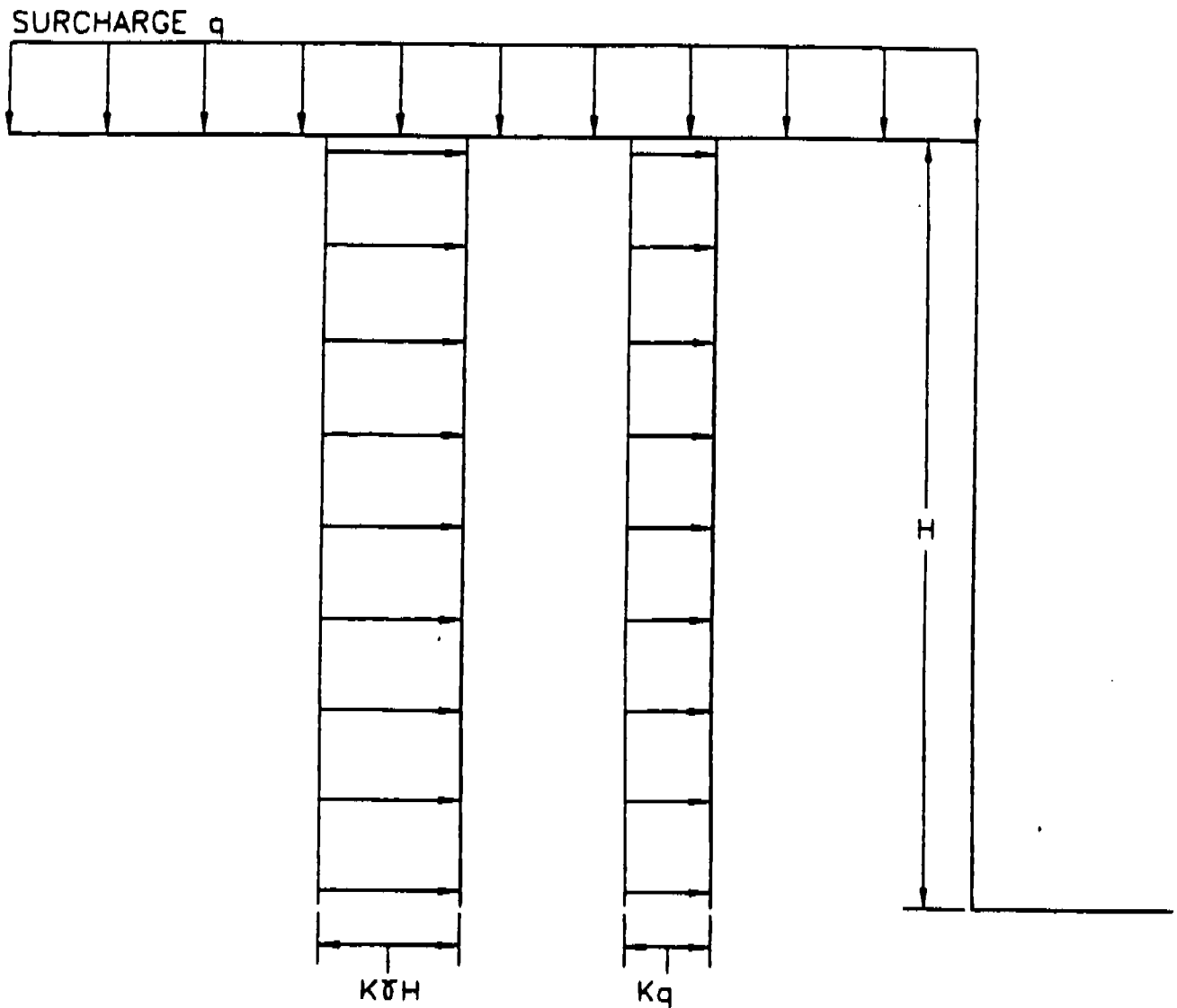
LOGGED: SB

CHECKED: MR

DRILLHOLE 1119ROCK.GPJ GLDR CAN.GOT 12/1000 PS

# DESIGN LATERAL EARTH PRESSURES OR BRACED EXCAVATION

FIGURE 1



$\gamma$  = UNIT WEIGHT OF SOIL

$K$  = EARTH PRESSURE COEFFICIENT

Date OCTOBER, 2000

Project 001-1119-1

**Golder Associates**

Drawn R.J.

Chkd. \_\_\_\_\_



# OVERSIZE DRAWING

DOCUMENT MICROFILMING IDENTIFICATION

GEOCRES No. 31C-158

DIST. 41 REGION \_\_\_\_\_

W.P. No. 76-99-01

CONT. No. \_\_\_\_\_

W. O. No. \_\_\_\_\_

STR. SITE No. \_\_\_\_\_

HWY. No. 401

LOCATION CP Railway

OVERHEAD

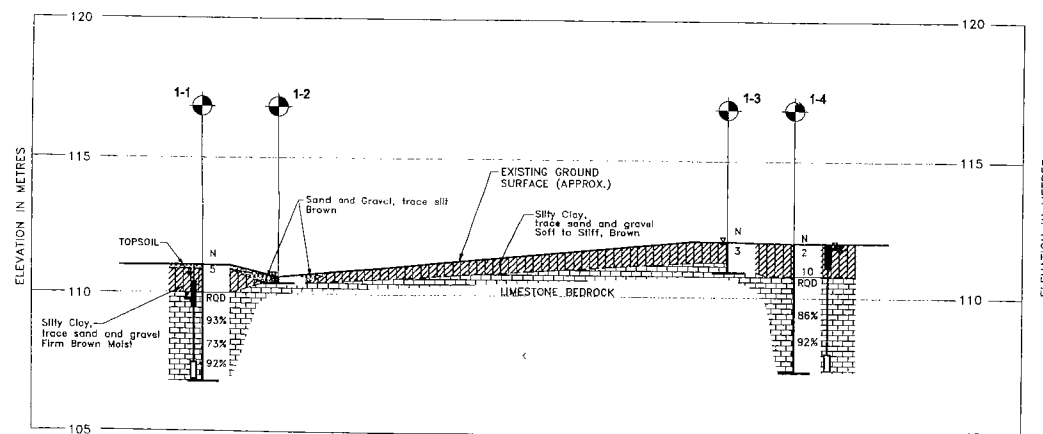
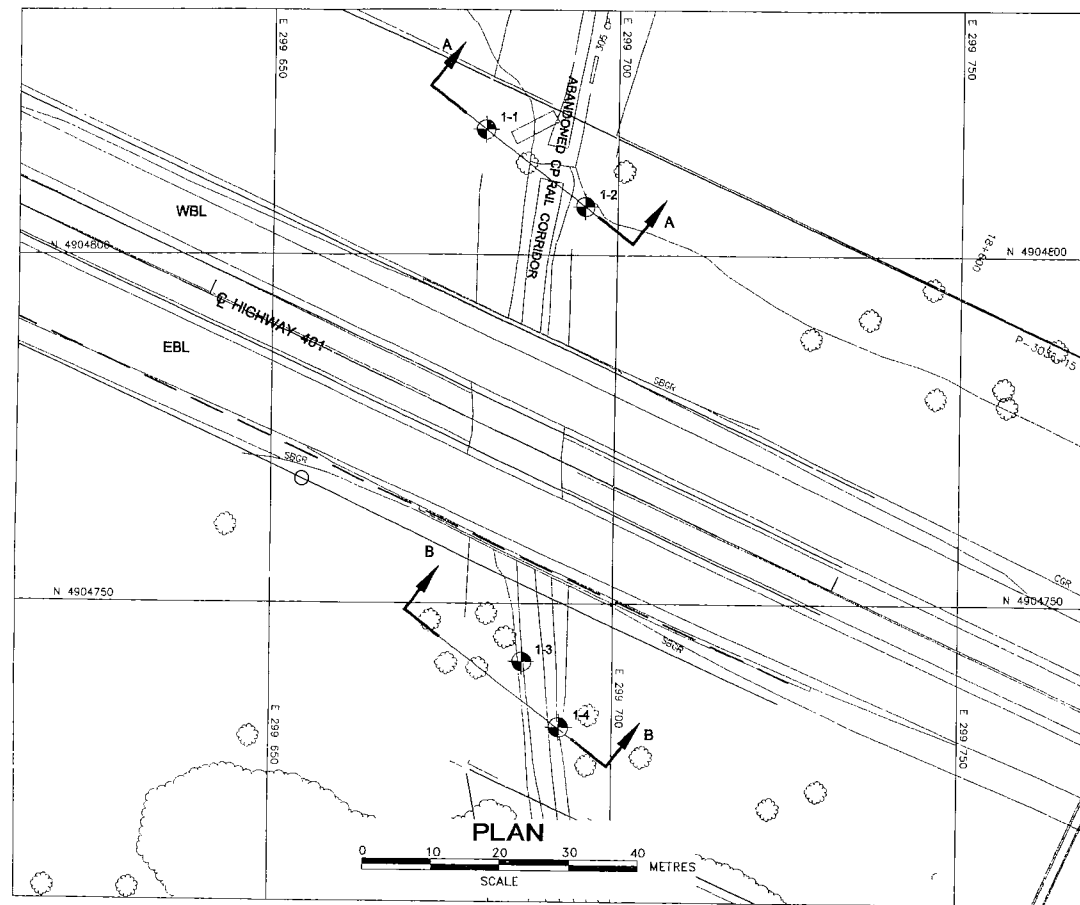
OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT. \_\_\_\_\_

REMARKS: \_\_\_\_\_

\_\_\_\_\_

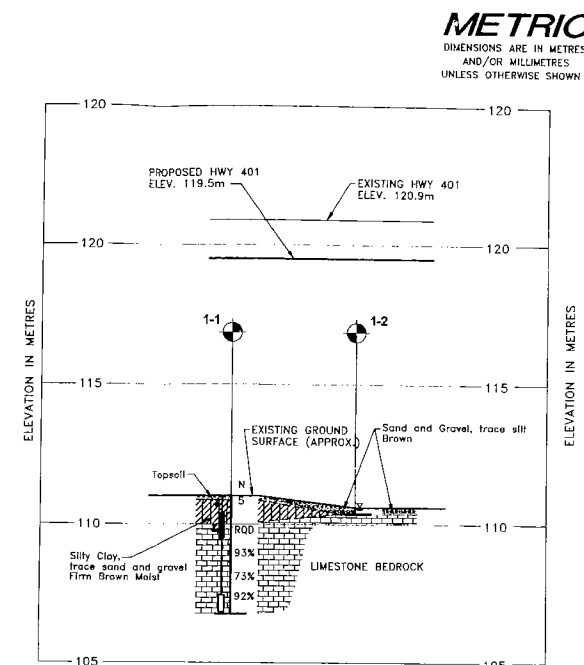
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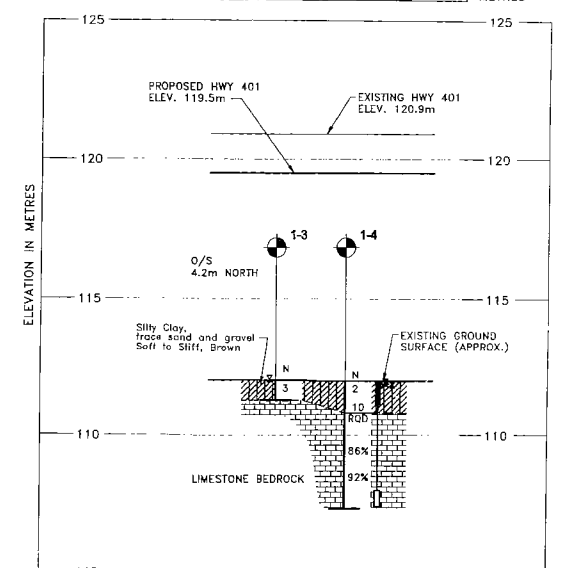
**PROFILE ALONG CP RAIL OVERHEAD**

HORIZONTAL SCALE 0 10 20 30 40 METRES  
VERTICAL SCALE 0 2 4 6 8 10 METRES



**SECTION A-A**

HORIZONTAL SCALE 0 10 20 30 40 METRES  
VERTICAL SCALE 0 2 4 6 8 10 METRES



**SECTION B-B**

HORIZONTAL SCALE 0 10 20 30 40 METRES  
VERTICAL SCALE 0 2 4 6 8 10 METRES

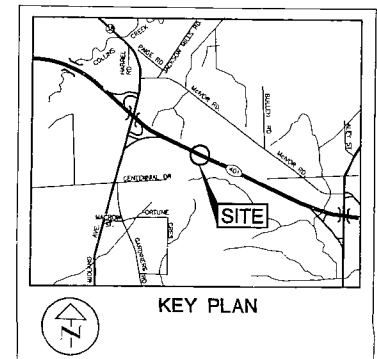
**METRIC**  
DIMENSIONS ARE IN METRES  
AND/OR MILLIMETRES  
UNLESS OTHERWISE SHOWN

DIST 41 HWY 401  
CONT No.  
WP No. 76-99-01

**HIGHWAY 401  
CP RAILWAY OVERHEAD  
BOREHOLE LOCATIONS & SOIL STRATA**



**Golder Associates Ltd.**  
MISSISSAUGA, ONTARIO, CANADA



**LEGEND**

- Borehole - Current Golder Associates Ltd. Investigation
- Seal
- Piezometer
- N Blows/0.3m (Std. Pen. Test, 475 j/blow)
- WL in piezometer on May, 21/00
- WL upon completion of drilling

No.	ELEVATION	LOCATION	
		NORTHING	EASTING
1-1	111.01	4904818.13	299680.88
1-2	110.60	4904807.01	299695.29
1-3	112.03	4904741.53	299686.82
1-4	111.99	4904731.94	299692.31

**NOTES**

The boundaries between soil strata have been established only at borehole locations. Between boreholes the boundaries are assumed from geological evidence.

**REFERENCE**

This drawing was created from digital file "P118-200.dwg" provided by MTO.

NO.	DATE	BY	REVISION

Geocres No.

HWY. No. 401	PROJECT NO.: 001-1119	DIST. 41
SUBM'D. DKB	CHKD: AMP	DATE: 2000 05 08
DRAWN: JFC	C.RKD. DKB	APPD.
		DWG. SITE 1