

GEOCRES No. _____

DIST. 52 REGION _____

W.P. No. 454-93-00 (D)

CONT. No. _____

W. O. No. _____

STR. SITE No. 42-316

HWY. No. 11

LOCATION East Service Rd. / Big
East River

No of PAGES -

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OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT. _____

REMARKS: _____

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FINAL REPORT

**FOUNDATION INVESTIGATION AND DESIGN REPORT
FOR EAST SERVICE ROAD
CROSSING BIG EAST RIVER
W.P. 454-93-00, SITE 42-316
HIGHWAY 11, DISTRICT 52
HUNTSVILLE, ONTARIO**

Submitted to:

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Ministry of Transportation

1.0 INTRODUCTION

Golder Associates Ltd. has been retained by Cole, Sherman & Associates (Cole, Sherman) on behalf of the Ministry of Transportation, Ontario (MTO) to carry out a geotechnical investigation at the site of the proposed bridge carrying Muskoka Road 3 Service Road (East Service Road) over Big East River. The East Service Road bridge over Big East River is part of the Four Laning of Highway 11 project, which extends from 2.2 km north of Highway 60 in Huntsville and northerly 4.5 km. This report addresses the proposed bridge and its approaches within 20 m of the structure, which is designated as Site 42-316.

The purpose of this investigation is to determine the subsurface conditions at the site of the proposed bridge structure by means of a limited number of boreholes, in-situ tests and laboratory tests on selected samples. Based on our interpretation of the data obtained, recommendations on the geotechnical aspects of design of the proposed works are provided. Comments are also provided on anticipated construction problems where they may affect design of the proposed bridge and approach embankments.

The subsurface information obtained during a previous investigation carried out by the MTO Foundation Design Section, for the then proposed detour structure over Big East River, contained in a report prepared by MTO, dated March 26, 1997, has been utilized in the preparation of this report.

The terms of reference for the scope of work are outlined in our proposal letter P71-8053, dated June 18, 1997 and the work was carried out in accordance to our Quality Control Plan for Foundation Design Services, dated August 1, 1997. During the course of the field work, the number of boreholes and extent of testing was revised slightly to accommodate the subsoil and site conditions as encountered.

2.0 SITE DESCRIPTION

The site is located approximately 6.7 km north of Highway 60 at Huntsville within the MTO District 52 in Huntsville, Ontario. The proposed alignment of the East Service Road is some 38 m to 45 m east of the existing Highway 11 and the East Service Road bridge is designated as Site 42-316.

The alignment of the East Service Road crosses the parkland adjacent to Arrowhead Provincial Park and the existing remains of the old bridge abutments. The topography of the site is undulating with ground surface between approximately Elevation 288 m and Elevation 291.5 m (the latter at the old bridge abutments). The Big East River banks at the proposed Service Road crossing are approximately 2 m to 4 m in height with relatively steep slopes. The north bank located in the vicinity of the north abutment is oversteepened with the river flowing directly at the toe of the slope. There has been a recent failure of the upper portion of the bank leaving overhanging tree roots at the crest and failure debris at the toe. The slope failure has exposed sandy soils along the slope face. The south bank of the river is approximately 2 m high. Vegetation cover on the tableland consists of shrubs and grass to the west of the road centreline and young and mature trees to the east of the road centreline. The water level in the river was at about Elevation 285.5 m at the time of our current investigation.

3.0 INVESTIGATION PROCEDURES

The field work for this investigation was carried out between August 26 and September 4, 1997. At this time, one borehole was put down at each of the two proposed bridge abutments and two boreholes were drilled along the approach embankments. The investigation was carried out using bombardier mounted CME 55 drill rigs supplied and operated by Marathon Drilling Inc. of Ottawa.

In each boring, samples were obtained at regular intervals of depth using 50 mm outside diameter split spoon samplers, in accordance with Standard Penetration Test (SPT) procedures. Groundwater conditions in the open boreholes were observed throughout the drilling operations, and two 12.5 mm diameter plastic piezometers were installed in one of the four boreholes to permit monitoring of the groundwater levels. The water levels in the piezometers were monitored during our investigation with the last readings obtained on September 12, 1997.

The field work was supervised on a full-time basis by a member of our technical staff who located the boreholes in the field, directed the drilling, sampling and in-situ testing operations, and logged the boreholes. The soil samples were identified in the field, placed in containers and transported back to our laboratory in Sudbury for further examination. Index and classification tests were carried out on selected samples.

The as-drilled borehole locations were determined by our field personnel based on the chainages as staked in the field. Surveyed borehole locations and elevations were provided by Cole, Sherman and we understand that the elevations are referenced to Geodetic Datum. The northing and easting co-ordinates of the boreholes as surveyed are shown on the Record of Borehole sheets and on Drawing M8033001, attached.

4.0 GENERAL SITE GEOLOGY AND STRATIGRAPHY

4.1 Site Geology

From published geologic information, the site is located in the physiographic region known as the Canadian Precambrian Shield. The shield terrain comprises large expanses of intrusive rocks such as gneisses and gneissic or massive granitic rocks. The rocks are geologically complex with considerable folding, intrusive activity, regional metamorphism and faulting. Pleistocene lacustrine / fluvial deposits and recent swamp sediments have been laid down in depressions and are associated with the Glacial Lake Algonquin. The local physiography is characterized by the overburden consisting mainly of discontinuous glaciolacustrine deposits and irregular, variable bedrock surface with frequent rock outcrops and shallow bedrock. Since irregular bedrock surface is typical in the area, organic terrain is widespread.

4.2 Site Stratigraphy

The detailed subsurface soil and groundwater conditions encountered in the boreholes, together with the results of the laboratory tests carried out on selected soil samples, are given on the attached Record of Borehole sheets, following the text of this report. The stratigraphic boundaries shown on the borehole logs 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.

Relevant information on subsurface conditions was obtained from four boreholes (Boreholes 97-1 to 97-4) put down during the current investigation and two boreholes (Boreholes 1 and 2) drilled during a previous investigation carried out by MTO in 1996 for the detour structure over Big East River, documented in a report titled "Foundation Investigation Report for Detour Structure, Big East River, W.P. 207-93-01, Site 42-09, Highway 11, District 52, Huntsville", dated March 26, 1997.

Boreholes 97-2 and 97-3 were put down on the south and north side of the river valley, respectively, in the vicinity of the proposed abutment locations and advanced to about 12.9 m and 14.4 m depths (Elevation 278.5 m to 277.2 m), respectively. Boreholes 97-1 and 97-4 were put down along the south and north approach embankments and advanced to 5.2 m and 6.7 m depths (Elevation 285.7 m to 284.6 m). MTO Boreholes 1 and 2 indicated on our drawing as Boreholes 96-1 and 96-2 were put down on the east side of the existing Highway 11, some 20 m to 25 m west of the proposed East Service Road alignment. These boreholes were advanced to depths of 21.5 m and 20.9 m.

In summary, the soils encountered in the boreholes consist of an extensive glaciofluvial and / or lacustrine origin deposits of sand, silty sand and clayey silt. A layer of cobbles, boulders and sand, about 1.6 m in thickness, was encountered overlying the bedrock in one borehole. The overburden is underlain by a gneiss bedrock. The inferred surface of the bedrock was encountered at depths of 12.9 m (Elevation 278.5 m) and 14.4 m (Elevation 277.2 m) below the existing ground surface in the two deeper boreholes. In the MTO Boreholes 1 and 2, the bedrock surface was found at 19.1 m and 19.3 m depth (at Elevations 269.4 m and 268.9 m), respectively.

The locations of the boreholes and stratigraphic section showing the inferred subsurface conditions at the proposed bridge site are shown on Drawing M8033001. A detailed description of the subsurface conditions encountered in the boreholes is provided in the following sections.

4.2.1 Fill Materials

Borehole 97-3 was put down on the north side of the river, in the vicinity of the existing old bridge abutment and encountered some 2.2 m of fill materials consisting of gravelly sand with some fragments of asphalt and concrete. The fill is generally loose to compact. It is considered that the fill materials were generated during construction of the old bridge abutment and / or approach embankment.

4.2.2 Sand

Extending from the ground surface and / or underlying the fill is a sand deposit. The sand extends to the depths investigated of 5.2 m and 6.7 m in Boreholes 97-1 and 97-4, respectively. Where fully penetrated in the deeper boreholes (97-2 and 97-3), the base of the sand deposit was at 10.2 m depth on the south side and at 7.9 m on the north side of the river.

The sand is typically in a very loose to compact state of packing; the measured Standard Penetration Test (SPT) "N" values range from 1 blow to 10 blows for 0.3 m of penetration. The measured natural water contents of selected samples of this deposit range from 4 per cent to 24 per cent.

The MTO Boreholes 1 and 2 encountered very loose to compact sand to 7.2 m and 4.4 m depth. These boreholes were put down in the low lying tableland where the ground surface is some 3 m to 3.5 m below the ground surface at the locations of the current boreholes.

4.2.3 Silty Clay

A layer of silty clay about 2.4 m thick was encountered between 7.9 m and 10.3 m depth, underlying the sand, in Borehole 97-3, on the north side of the river. One measured SPT "N" value in the silty clay was 4 blows for 0.3 m of penetration. The measured natural water content of one sample of this deposit was 38 per cent. The Atterberg Limit test indicated liquid limit of about 34 per cent and plasticity index of about 12 per cent.

The MTO boreholes encountered silt (or clayey silt) with low plasticity indices (between 2 per cent and 5 per cent) underlying the sand at 4.4 m and 7.2 m (Elevations 283.8 m and 281.3 m). The silt was encountered to 13.2 m and 16.2 m depth (Elevations 275.3 m and 272.0 m). The silt had a soft to very stiff consistency.

4.2.4 Silty Sand

Underlying the sand in Borehole 97-2 and silty clay in Borehole 97-3 is a layer about 2.5 m thick of silty sand. On the north side of the river, the silty sand underlies the silty clay and is in a very loose to loose state of packing. Two measured SPT "N" values were 6 blows for 0.3 m of penetration and weight of hammer. On the south side, the silty sand is in a compact state of packing with measured "N" values of 20 and 23 blows for 0.3 m of penetration. The silty sand extends to the inferred bedrock surface in Borehole 97-2 and to the cobbles and boulders layer at 12.8 m depth in Borehole 97-3.

The natural water content of one sample of this deposit was 15 per cent.

4.2.5 Layer of Cobbles and Boulders

Underlying the silty sand deposit in Borehole 97-3 is a 1.6 m thick layer consisting of cobbles and boulders with sand, some gravel and silty clay. The cobbles and boulders were inferred based on auger resistance during drilling. No samples were recovered of this deposit. Persistent grinding and bouncing of the augers was observed during drilling; practical refusal to further auger penetration was met at 14.4 m depth (Elevation 277.2 m), probably on bedrock.

4.2.6 Bedrock

Refusal to further auger penetration was met in Borehole 97-2 at 12.9 m depth (Elevation 278.5 m) and in Borehole 97-3 at 14.4 m depth (Elevation 277.2 m). Based on the nature of the auger refusal and on the results of other boreholes in the area, it is considered that the refusal indicates the bedrock surface although no rock coring was carried out in these boreholes. The MTO Boreholes 1 and 2, which are located some 20 m to 25 m to the west of the road alignment encountered bedrock at 19.1 m and 19.3 m depth (Elevations 269.4 m and 268.9 m). Bedrock coring was carried out in these boreholes for lengths of 1.6 m and 2.4 m. The Rock Quality Designation (RQD) measured on the core samples ranged from 21 per cent to 93 per cent. Based on the rock core obtained, the bedrock consists of a Biotite-Hornblende Gneiss.

Based on the results of boreholes for the adjacent bridge sites, the bedrock surface appears to be relatively level in the north-south direction along the road alignment but slopes steeply downwards to the west.

4.2.7 Groundwater Conditions

The water level in the piezometers installed into Borehole 97-2 located on the south side of the river was measured at 7.2 m depth (Elevation 284.2 m) in both the shallow and deep piezometers.

Borehole 97-1 was dry on completion of drilling operations. The water levels in the open Boreholes 97-2 to 97-4 were between 5.8 m and 6.4 m depth during drilling operations. It should be noted that the water levels are subject to seasonal and river water level fluctuations.

5.0 ENGINEERING RECOMMENDATIONS

5.1 General

This section of the report provides our recommendations on the geotechnical aspects of design of the East Service Road bridge based on our interpretation of the factual information obtained during our recent investigation and the 1996 field investigation carried out by MTO. 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.

The works described in this report are associated with the proposed bridge and its approaches within 20 m of the structure. It is understood that the East Service Road will be carried over Big East River by a three span structure about 50 m in length. As shown on the profile drawings (revised), the final road grade will be at about Elevation 293.3 m on the south side of the crossing and at about Elevation 293.8 m on the north side of the crossing. The proposed road grade requires an embankment height of up to 5 m for the south approach and about 3 m for the north approach to the bridge.

The proposed horizontal and vertical alignment for Highway 11 and the locations of the bridges were provided to us on 1:2000 plan drawings and section prepared by Cole, Sherman. Revisions to the vertical alignment have been made and the revised data were received on August 21, 1997.

5.2 Bridge Foundations

The subsoils encountered in the boreholes put down at this site are not suitable for support of shallow spread footings; therefore, deep foundations are recommended for the support of the abutments and the piers. Consideration could be given to the use of closed-end or open-end pipe piles and steel H-piles driven to the bedrock surface which was inferred to be at about Elevation 277 m and 278.5 m on the north and south sides of the river, respectively.

For the purpose of this discussion, the base of pile cap at the piers has been assumed to be at approximately Elevation 285 m and at Elevation 289 m at the abutments. All pile caps should be provided with at least 1.8 m of soil cover for frost protection.

Some 1.6 m thickness of cobbles and boulders directly overlies the bedrock in Borehole 97-3 put down on the north side of the river. There is a possibility of encountering refusal to pile penetration on this bouldery layer. Steel H-piles would have a better chance of penetrating this type of cobble / boulder layer. Closed-end pipe piles would be most prone to "hanging up" on the boulders and a reduced pile capacity may have to be used and additional piles installed.

5.2.1 Factored Geotechnical Resistance

The geotechnical resistance at Ultimate Limit States (ULS) for piles driven to practical refusal on the Biotite – Hornblende Gneiss bedrock at this site will be greater than the structural capacity of the piles. In addition, the geotechnical resistance at Serviceability Limit States (SLS) of 25 mm of settlement is not applicable to piles driven to refusal on the bedrock since the stresses required to induce 25 mm of settlement exceed those at ULS.

For this site, therefore, the structural resistance of the piles will govern and the following values may be assumed:

HP 310X110
2,800 kN / pile

HP 310x79
2,000 kN / pile

324 mm diameter pipe
1,900 kN / pipe

Where piles "hang up" in the bouldery layer, an axial capacity at ULS of 1,100 kN and at SLS of 950 kN may be assumed.

Based on the borehole results, the abutment piles are anticipated to be about 10 m to 11 m in length; the pier piles are anticipated to be about 5 m to 6 m in length. These lengths may be assumed for design; however, the actual lengths must be confirmed in the field based on the driving records.

The piles should be equipped with suitable rock points to ensure penetration to and seating into the bedrock and other stiffening as appropriate in anticipation of heavy driving. Based on the results of the boreholes located to the west, the bedrock surface slopes downward to the west and rock points as well as appropriate driving procedures are essential to bite into the bedrock and prevent sliding along the surface.

The H-piles should be driven to an initial set equal to or greater than 10 blows per 12 mm of penetration (unless abrupt peaking occurs) using a hammer with rated energy of about 50 kilojoules but not exceeding 60 kilojoules. On reaching the required set, the hammer energy should be reduced by about 75 per cent and the pile should then be re-driven by increasing the hammer energy slowly up to the maximum rated energy over about 40 blows. This procedure is intended to improve the process of seating of the pile on the sloping bedrock surface.

A final set of no less than 10 blows per 12 mm of penetration should be obtained at the maximum hammer energy. Provision should be made to re-tap all piles to confirm the set after adjacent piles have been driven. The above set criteria should be reviewed at the time of construction in light of the constructor's proposed equipment, so that over-driving and possible damage to the piles is avoided.

The pipe piles should be driven to the same set criteria, with the exception that the hammer should have a rated energy of about 65 kilojoules and not exceed 80 kilojoules.

5.2.2 Resistance to Lateral Loads

The lateral loading could be resisted fully or partially by the use of battered piles. If vertical piles are to resist the lateral loading, the horizontal reaction to the pile can be calculated from the expression:

$$k_s = z \times n_h / d,$$

Where

- k_s = coefficient of horizontal subgrade reaction (MPa/m),
- d = pile diameter (m),
- n_h = constant of horizontal subgrade reaction (MPa/m),
- z = depth (m).

The constant of horizontal subgrade reaction depends on the soil type and soil density / consistency around the pile shaft. For design of resistance to lateral loads, the values (or range of values) indicated in the tables below may be assumed:

<i>Elevation (approximate) (m)</i>	<i>Soil Type</i>	<i>$z \times n_h$ (MPa)</i>
from 288.0 to 279.0	Sand to Silty Sand, very loose to loose and silty clay, firm	$z \times 1.0$
from 279.0 to bedrock*	Silty Sand and Cobbles and Boulders, compact	$z \times 4.0$

* Bedrock at Elevation 277 m on the north side and Elevation 278.5 m on the south side.

Group action for lateral loading should be considered when the pile spacing in the direction of loading is less than 6 to 8 pile diameters. Group action can be evaluated by reducing the coefficient of lateral subgrade reaction in the direction of loading by a reduction factor R as follows:

<i>Pile Spacing in Direction of loading d = Pile Diameter</i>	<i>Subgrade Reaction Reduction Factor R</i>
8d	1.00
6d	0.70
4d	0.40
3d	0.25

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 and the retaining walls in accordance with OHBDC:

- Select free-draining non-frost susceptible 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 walls. All granular fill should be compacted in lifts of loose thickness not greater than 200 mm 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.
- The granular fill may be placed either in a zone with width equal to at least 1.8 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).
- If the wall support allows lateral yielding of the stem (unrestrained structure), active earth pressures may be used in the geotechnical design of the structure. If the abutment support does not allow lateral yielding (restrained structure), at-rest pressures should be assumed for geotechnical design.
- 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 in-situ soils / embankment fill materials and the following parameters (unfactored) may be assumed:

Soil unit weight (assuming the in-situ soils and/or clean earth fill)	20 kN/m ³
Coefficients of lateral earth pressure:	
'active'	0.33
'at rest'	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
Soil Unit Weight	22 kN/m ³	21 kN/m ³
Coefficient of Lateral Earth Pressure		
'active'	0.27	0.31
'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 Excavations

The excavations for the pile cap construction at the abutments could be extended to about 1 m to 2 m below the grade (assuming base of pile cap at about Elevation 289 m). Excavations will therefore be through loose granular deposits and the base will most likely be above the groundwater level. Excavations which will be open for relatively short period of time and above the groundwater table can be made in temporary unsupported cut with side slopes maintained not steeper than 1.5 horizontal to 1 vertical.

At the piers, excavations for pile cap construction at about Elevation 285 m may extend up to about 2 m below the existing river channel base and possibly to about 3.4 m below the river water level as shown to be at Elevation 286.4 m in November 1996 and August 1997. Some form of groundwater control will be required to permit pile installation and pile cap construction in the dry conditions. In general, the sands are sensitive to disturbance and where they form the excavation base or side walls below the groundwater level, the soils undergo rapid loosening due to upward water seepage. Closed steel sheetpiling could be used as cut-off to groundwater flow and to form temporary support to the excavation. The cut-off wall must extend to sufficient depth below the base of the excavation to minimize the piping of the sands forming the base.

The inferred bedrock surface and the overlying boulder / cobble layer are at relatively shallow depth below the inferred base of the pile cap excavation (about 5 m or less). It may not be possible to obtain an adequate cut-off to groundwater flow using driven sheeting since a seal at the overburden / bedrock surface would require penetration into the hard and sloping bedrock. Consideration could be given to making the excavation in the wet within the closed steel sheetpiling driven as deep as possible and placing a tremie plug at the base of the excavation. The stability of the sheetpile cofferdam must be checked given the relatively shallow penetration depth that is feasible.

All excavations should be carried out in accordance with the current Occupational Health and Safety Act.

5.5 Approach Embankments

The proposed road grade at the crossing will involve an approach embankment approximately 3 m high on the north side of the river and 5 m high on the south side of the river. Based on the previous and recent subsurface information, the subsoils underlying the approach embankment consist generally of very loose to loose sands, silty sands and silts.

Given the above, stability of the proposed embankments is not a concern with respect to deep seated failure through the founding soils. No long-term settlement of the road embankments is expected due to consolidation of the subsoils given the composition of the granular founding materials. Some settlement of the embankments will occur during construction and within a relatively short time following construction due to consolidation of the very loose sands and silts underlying the site; however, the amount of settlement following construction is expected to be minimal.

Given the current condition of the valley bank on the north side of the river in proximity to the proposed north abutment with evidence of on-going erosion and surficial slope stability, some rip-rap placement at the slope toe may be required depending on the final layout and configuration of the abutments and piers.

5.6 Subgrade Preparation and Embankment Construction

Topsoil, organic deposits and fill materials should be stripped from below the fill embankment areas and the exposed subgrade soils should be proof-rolled prior to embankment fill placement. The subgrade consists of sand and silty sand.

Construction of the embankment above the prepared subgrade may be carried out using clean earth fill (in accordance with OPSS 212) or Select Subgrade Material (in accordance with OPSS 1010) depending on material available. 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 all fill placement operations to ensure that appropriate materials are used and that adequate levels of compaction have been achieved. The permanent slopes of the embankment should be maintained not steeper than 2 horizontal to 1 vertical. Vegetation cover should be established on all slopes to protect embankment fill against surficial erosion. Depending on the final layout and configuration of the abutments, some rip-rap placement at the slope toe may be required.

Alternatively, the approach embankments could be constructed using rockfill if available to the project. The permanent side slopes of the rockfill embankments should be maintained not steeper than 1.25 horizontal to 1 vertical.

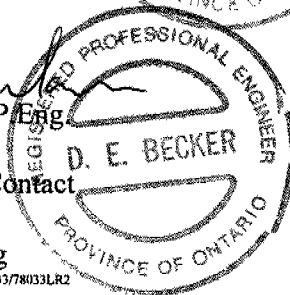
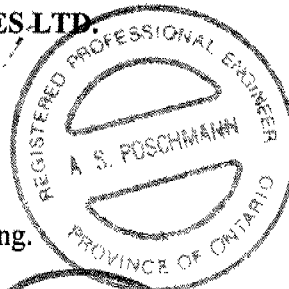
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WORD S/FINAL/DAT/OTHPRT/971-8033/78033LR2



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
DO	Drive open
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

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.).

Dynamic 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):

An electronic cone penetrometer with a 60° conical tip and a projected 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.

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

(b) Cohesive Soils

Consistency	c_u, 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

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 test ¹
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement ¹
D _R	relative density (specific gravity, G _s)
DS	direct shear test
M	sieve analysis for particle size
MH	combined sieve and hydrometer (H) analysis
MPC	Modified Proctor compaction test
SPC	Standard Proctor compaction test
OC	organic content test
SO ₄	concentration of water-soluble sulphates
UC	unconfined compression test
UU	unconsolidated undrained triaxial test
V	field vane test (LV-laboratory vane test)
γ	unit weight

Note:

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

LIST OF SYMBOLS

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

I GENERAL

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

II STRESS AND STRAIN

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

III SOIL PROPERTIES

(a) Index Properties

$\rho(\gamma)$	bulk density (bulk unit weight*)
$\rho_d(\gamma_d)$	dry density (dry unit weight)
$\rho_w(\gamma_w)$	density (unit weight) of water
$\rho_s(\gamma_s)$	density (unit weight) of solid particles
γ'	unit weight of submerged soil ($\gamma' = \gamma - \gamma_w$)
D_R	relative density (specific gravity) of solid particles ($D_R = \rho_s / \rho_w$) (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 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_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 = (Compressive strength)/2

M8033001 BHS

PROJECT: 971-8033

RECORD OF BOREHOLE 97-1

SHEET 1 OF 1

LOCATION: N 5026561.113; E 326591.988

BORING DATE: AUG.26/97

DATUM: GEODETIC

SAMPLER HAMMER, 63.5kg; DROP, 760mm

PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm



DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE		SAMPLES		DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m	HYDRAULIC CONDUCTIVITY, k, cm/s		ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	NUMBER TYPE BLOWS/0.3m		SHEAR STRENGTH Cu, kPa	WATER CONTENT, PERCENT Wp — W — Wi		
0		GROUND SURFACE		290.85 0.00						
1	BOMBARDIER CME-55 HOLLOW STEM AUGERS	Sand, fine to medium with some silt and trace organics at the surface. Very loose to loose Brown to grey Moist	[Pattern]	1	50 DO	6				
2				50 DO	4					
3				50 DO	3					
4				50 DO	3					
5				50 DO	3					
6				50 DO	9					
5		END OF BOREHOLE		285.67 5.18						
6										
7										
8										
9										
10										

NOTE:
Open borehole dry
on completion of
drilling.

DEPTH SCALE

1 to 50

Golder Associates

LOGGED: MSB

CHECKED: DC

DATA INPUT PS DEC 18/97

SOIL M6

MS033092.BHS

PROJECT: 971-8033

RECORD OF BOREHOLE 97-2

SHEET 1 OF 2

LOCATION: N 5026620.435; E 326583.688

BORING DATE: SEPT.4/97

DATUM: GEODETIC

SAMPLER HAMMER, 63.5kg; DROP, 760mm

PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm



DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE		SAMPLES		DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m		HYDRAULIC CONDUCTIVITY, k, cm/s		ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION	
		DESCRIPTION	STRATA PLOT	ELEV.	NUMBER	TYPE	BLOWS/0.3m	SHEAR STRENGTH Cu, kPa	nat V - + rem V - ⊕ Q - ● U - ○			WATER CONTENT, PERCENT Wp — W — Wi 10 20 30 40
				DEPTH (m)								
0	BOMBARDIER CME 55 HOLLOW STEM AUGERS	GROUND SURFACE		291.36								
		Sand and Gravel, trace silt, trace organics and wood fragments Loose Brown Moist (FILL)		0.00	1	50 DO	7				BENTONITE SEAL	
1				2	50 DO	10		○				
			289.96									
		Sand, fine to coarse with trace silt Very loose to loose Brown Moist, becoming wet below 6.1m		1.40								NATIVE BACKFILL
2				3	50 DO	1						
				4	50 DO	2		○				
3				5	50 DO	3						
4												
5				6	50 DO	4		○				
			7	50 DO	1							
8				8	50 DO	2			○			
9				9	50 DO	4						
10												
CONTINUED ON NEXT PAGE												

DATA INPUT: PS dec 18/97

SOILM6

DEPTH SCALE

1 to 50

Golder Associates

LOGGED: MSB

CHECKED: DC

M8033002 BHS

PROJECT: 971-8033

RECORD OF BOREHOLE 97-2

SHEET 2 OF 2

LOCATION: N 5026620.435; E 326583.688

BORING DATE: SEPT.4/97

DATUM: GEODETIC

SAMPLER HAMMER, 63.5kg; DROP, 760mm

PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm



DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE		SAMPLES		DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m		HYDRAULIC CONDUCTIVITY, k, cm/s		ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		DESCRIPTION	STRATA PLLOT ELEV. DEPTH (m)	NUMBER TYPE	BLOWS/0.3m	SHEAR STRENGTH Cu, kPa nat V - + Q - ● rem V - ⊕ U - ○		WATER CONTENT, PERCENT Wp — W — Wl 10 20 30 40			
10	BOMBARDIER CME-55 HOLLOW STEM AUGERS	CONTINUED FROM PREVIOUS PAGE	281.18 10.20								
11		Silty Sand with some gravel Compact Grey Wet		10	50 DO	20					SAND
12											
13				11	50 DO	23					
13		END OF BOREHOLE Refusal to auger penetration Probably on bedrock	278.48 12.88								
14											
15											
16											
17											
18											
19											
20											

NOTE:
Water level in
open borehole at
6.1m depth on
completion of
drilling.

Water level in
shallow and
deep piezometer
at Elevation
284.2m on
Sept.12/97.

DEPTH SCALE

1 to 50

Golder Associates

LOGGED: MSB

CHECKED: DC

DATA INPUT: PS dec.18/97

SOIL M6

PROJECT: 971-8033

RECORD OF BOREHOLE 97-3

SHEET 1 OF 2

LOCATION: N 5026669.442; E 326573.079

BORING DATE: SEPT.2/97

DATUM: GEODETIC

SAMPLER HAMMER, 63.5kg; DROP, 760mm

PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm



DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE		SAMPLES		DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m		HYDRAULIC CONDUCTIVITY, k, cm/s		ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		DESCRIPTION	STRATA PLOT ELEV. DEPTH (m)	NUMBER	TYPE	SHEAR STRENGTH Cu, kPa	WATER CONTENT, PERCENT Wp, W, Wi				
0		GROUND SURFACE	291.57 0.00								
1		Gravelly sand with some asphalt and concrete fragments Loose to compact Brown Moist (FILL)		1	50 DO	4					
	2			50 DO	8						
	2a			50 DO	16						
2		Sand, fine to coarse with trace silt and gravel, occ. silt lenses Grey Very loose Moist becoming wet at 6.4m depth		3	50 DO	1					
	4			50 DO	1						
	5			50 DO	3						
	6			50 DO	1						
	7			50 DO	6						
8		Silty Clay with trace sand Firm Grey Wet		7	50 DO	6					
	8			50 DO	4						
10		CONTINUED ON NEXT PAGE									

DEPTH SCALE

1 to 50

Golder Associates

LOGGED: MSB

CHECKED: DC

DATA INPUT: PS DEC 18/97

SOIL M6

PROJECT: 971-8033

RECORD OF BOREHOLE 97-3

SHEET 2 OF 2

LOCATION: N 5026669.442; E 326573.079.

BORING DATE: SEPT.2/97

DATUM: GEODETIC

SAMPLER HAMMER, 63.5kg; DROP, 760mm

PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm



DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE		SAMPLES			DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m				HYDRAULIC CONDUCTIVITY, k, cm/s				ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION	
		DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	NUMBER	TYPE	BLOWS/0.3m	SHEAR STRENGTH				WATER CONTENT, PERCENT					
								Cu, kPa		c _u , kPa		W _p		W			
10	BOMBARDIER CM-55 HOLLOW STEM AUGERS	CONTINUED FROM PREVIOUS PAGE															
				281.27													
				10.30													
11		Silty Sand with trace clay and gravel Very loose to loose Grey Wet		10	50 DO	WH											
12				11	50 DO	6											
13		Cobbles and Boulders with sand, some gravel and silty clay (inferred from resistance to augering and observations of auger cuttings during drilling operations).															
14																	
15		END OF BOREHOLE Refusal to auger penetration Probably on bedrock															
16																	
17																	
18																	
19																	
20																	

NOTE:
Water level in
open borehole at
6.4m depth during
drilling.

DEPTH SCALE

1 to 50

Golder Associates

LOGGED: MSB

CHECKED: DC

DATA INPUT: PS DEC 18/97

SOILM6

PROJECT: 971-8033

RECORD OF BOREHOLE 97-4

SHEET 1 OF 1

LOCATION: N 5026686.555; E 326568.492

BORING DATE: AUG.29/97

DATUM: GEODETIC

SAMPLER HAMMER, 63.5kg; DROP, 760mm

PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm



DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE		SAMPLES		DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m		HYDRAULIC CONDUCTIVITY, k, cm/s		ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	NUMBER	TYPE	BLOWS/0.3m	SHEAR STRENGTH Cu, kPa	WATER CONTENT, PERCENT Wp — W — Wi		
0		GROUND SURFACE		291.25 0.00							
1	BOMBARDIER CME-S5 HOLLOW STEM AUGERS	Sand, fine to medium to silty Sand Very loose to loose Brown Moist becoming grey at 5.8m depth -trace organics noted to approx. 3m depth			1	50 DO	7				
				2	50 DO	5					
				3	50 DO	3					
				4	50 DO	3					
				5	50 DO	5					
				6	50 DO	8					
				7	50 DO	3					
6		-coarse below 6.1m depth									
7		END OF BOREHOLE		284.55 6.70							
8											
9											
10											

NOTE:
Water level in
open borehole at
5.8m depth during
drilling.

DEPTH SCALE

1 to 50

Golder Associates

LOGGED: MSB

CHECKED: DC

DATA INPUT: PS dec 16/97

SOIL#6

OVERSIZE DRAWING(S)

December 22, 1997

971-8033-2

APPENDIX A

**RECORD OF BOREHOLE SHEETS
MINISTRY OF TRANSPORTATION**

RECORD OF BOREHOLE No 1

1 OF 1

METRIC

W.P. 207-93-01

LOCATION Co-ords.: N 5 026 632.4; E 326 556.4

ORIGINATED BY LV

DIST 52 HWY 11

BOREHOLE TYPE Hollow Stem Auger, BX Core, Cone Test

COMPILED BY KA

DATUM Geodetic

DATE 1996 08 08.09

CHECKED BY TC

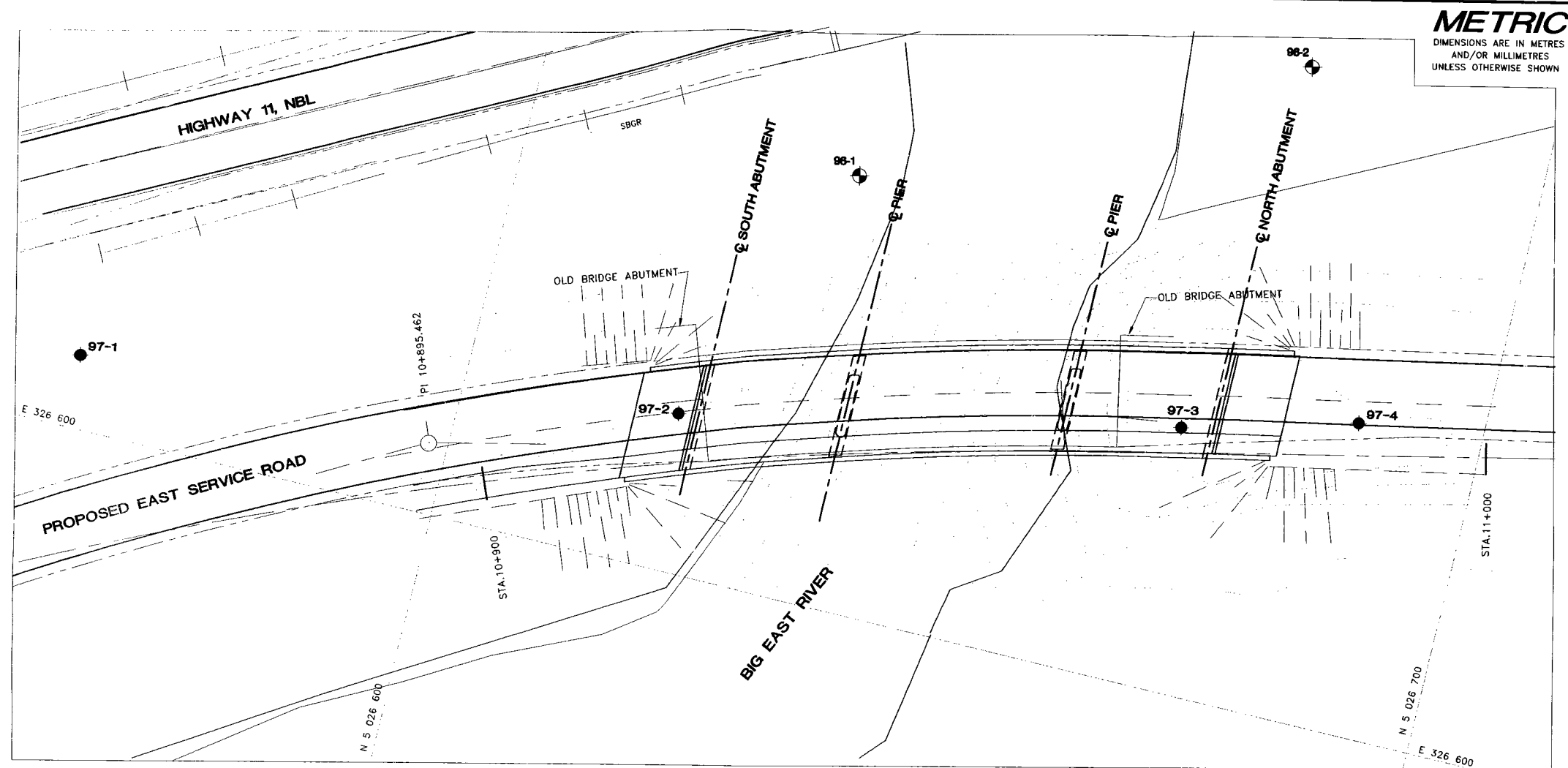
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100	PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT L	UNIT WEIGHT 7 kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES								
288.5	Ground Surface												
0.0	Silty Sand to Sand Very Loose to Loose Brown, Moist to Wet		1	SS	2		288						0 84 14 2
			2	SS	4		286					0 90 (10)	
			3	SS	6		284					0 98 (2)	
			4	SS	7								
			5	SS	5								
			6	SS	2								
			7	SS	3								
			8	SS	2								
281.3	Silt, with a trace of Clay Soft to Stiff Grey, Wet		9	SS	2		282						
7.2			10	SS	10		280						
			11	SS	11		278						
			12	SS	11		276						
275.3	Silt to Coarse Sand trace Gravel Loose to Very Dense Grey, Wet		13	SS	6		274						0 11 85 4
13.2			14	SS	54		272						
			15	SS	88		270						
			16	RC	REC	100%							
269.4	Biotite-Hornblende Gneiss Bedrock		17	RC	REC	93%							ROD 21%
19.1			18	RC	REC	73%							ROD 24%
			19	RC	REC	100%							ROD 76%
267.0			20	RC	REC	100%							ROD 48%
21.5	End of Borehole												

RECORD OF BOREHOLE No 2 1 OF 1 METRIC

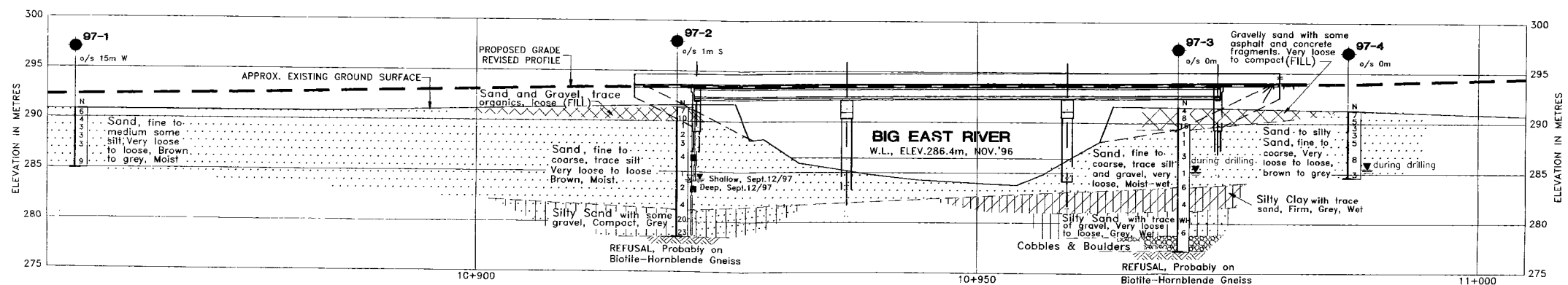
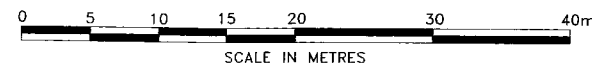
W.P. 207-93-01 LOCATION Co-ords: N 5 026 673.8; E 326 535.0 ORIGINATED BY LV
 DIST 52 HWY 11 BOREHOLE TYPE Hollow Stem Auger, BX Core, Cone Test COMPILED BY KA
 DATUM Geodetic DATE 1996 08 07 CHECKED BY TC

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							WATER CONTENT (%)			
								20 40 60 80 100										

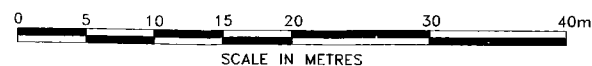
288.2	Ground Surface						288									
0.0	Silty Sand to Sand Very Loose to Compact Brown, Moist to Wet		1	SS	5		288									
			2	SS	10		286									0 82 16 2
			3	SS	9											0 95 (5)
			4	SS	3		284									
283.8			5	SS	2											
4.4	Silt with a trace of Clay Soft to Very Stiff Grey, Wet		6	SS	2		284									
			7	SS	2											
			8	SS	5		282									0 2 95 3
			9	SS	2											
			10	SS	3		280									
			11	SS	12											
			12	SS	11		278									
			13	SS	12											
			14	SS	19		276									
272.0			15	SS	0											
16.2	Silt, some sand, Tr. Clay Trace Gravel Compact, Grey, Wet		16	SS	16		272								0 8 86 6	
268.9							270									
19.3	Biotite-Hornblende Gneiss Bedrock		17	RC	REC											
267.3							268									ROD 93%
20.9	End of Borehole															



PLAN



PROFILE ALONG EAST SERVICE ROAD



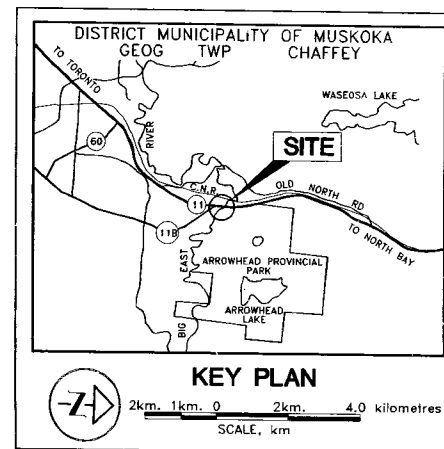
METRIC
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN

CONT. No.
WP No. 454-93-00

EAST SERVICE ROAD OVER
BIG EAST RIVER
BORE HOLE LOCATIONS & SOIL STRATA



Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



KEY PLAN

LEGEND

- Bore Hole
- ⊕ Dynamic Cone Penetration Test (Cone)
- ⊙ Bore Hole & Cone
- N Blows/0.3m (Std. Pen. Test, 475 j/blow)
- Cone Blows/0.3m (60' Cone, 475 j/blow)
- WL at time of investigation 1997 08

No.	ELEVATION	LOCATION	
		NORTHING	EASTING
97-1	290.85	5026561.11	326591.99
97-2	291.36	5026620.44	326583.69
97-3	291.57	5026669.44	326573.08
97-4	291.25	5026686.56	326568.49
96-1	288.5	5026632.4	326556.4
96-2	288.2	5026673.8	326535.0

NOTES

The boundaries between soil strata have been established only at Bore Hole locations. Between Bore Holes the boundaries are assumed from geological evidence.

A	97/09/12	AMP	ISSUED FOR REVIEW
NO.	DATE	BY	REVISION
Geocres No.			
HWY. No. 11	PROJECT NO.: 971-8033		DIST. 52
SUBM'D. AMP	CHKD: ASP	DATE: 1997 08 15	SITE 42-316
DRAWN: MHW	CHKD: AMP	APPD.	DWG. M8033001