

GEOCRES No. _____

DIST. 52 REGION _____

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

CONT. No. _____

W. O. No. _____

STR. SITE No. 42-09S

HWY. No. 11

LOCATION Hwy 11 Southbound /
Big East River

No of PAGES - 1



OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT. _____

REMARKS: _____

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FINAL REPORT

**FOUNDATION INVESTIGATION AND DESIGN REPORT
FOR HIGHWAY 11 SOUTHBOUND
CROSSING BIG EAST RIVER
W.P. 454-93-00, SITE 42-09S
HIGHWAY 11, DISTRICT 52,
HUNTSVILLE, ONTARIO**

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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 to carry the southbound lanes of Highway 11 over Big East River. The 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. The site of the project is designated as Site 42-09S.

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 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 with 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 within the MTO District 52 in Huntsville, Ontario. The centreline of the proposed alignment of the southbound lanes of Highway 11 at the bridge is located some 35 m west of the centreline of the existing Highway 11. The site is designated as Site 42-09S.

On the north side of the Big East River, the Highway 11 Southbound alignment crosses the MTO picnic area; on the south side of the river, the alignment crosses a forested area. The topography of the site is relatively level with ground surface between approximately Elevation 288.0 m and Elevation 289.5 m. The Big East River banks at the bridge crossing are approximately 2 m to 3 m in height. The north bank is oversteepened with the river flowing directly at the toe of the bank. The south bank of the river is relatively flat. Vegetation cover on the tableland consists of shrubs, grass and several young and mature trees on the north side of the river within the picnic area. On the south side, a thick forest with deciduous and coniferous trees extends some 20 m south from the river edge. Based on the information provided to us, the water level in the river was at Elevation 286.4 m in November 1996 and in August 1997.

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. One dynamic cone penetration test was carried out at the location of the south abutment. 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.

The field work was supervised on a full time basis by members 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 highway 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 are shown on the Record of Borehole sheets and on Drawing M8033003, 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 boreholes 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 97-6 to 97-9) and one dynamic cone test (CT 97-2). Boreholes 97-7 and 97-8 were put down on the south and north side of the river valley, respectively, in the general vicinity of the proposed abutment locations and advanced to 25.3 m and 35.1 m depths (Elevation 263.2 m to 253.4 m), respectively. Boreholes 97-6 and 97-7 had to be located some 15 m east of the proposed centreline of the south abutment due to initial difficult access to the site. Subsequently, a dynamic cone penetration test (CT 97-2) was carried out in the area of the proposed south abutment. Borehole 97-9 was put down along the north approach embankment to 6.7 m depth (Elevation 281.6 m).

In summary, the soils encountered in the boreholes consist of extensive glaciofluvial and/or lacustrine deposits of sand, silty sand and silty clay. A layer of cobbles, boulders and sand was encountered overlying the bedrock in the two deep boreholes. The overburden is underlain by gneiss bedrock. The surface of the bedrock was encountered at 32 m depth in Borehole 97-8 (about Elevation 256.5 m) and was inferred from resistance to auger penetration at 25.3 m depth (about Elevation 263.2 m) in Borehole 97-7, put down some 15 m east of the alignment. The surface of the bedrock inferred during the dynamic cone penetration test CT 97-2 located at the centreline of the proposed bridge was at 29.7 m depth (about Elevation 258.7 m).

The locations of the boreholes and a stratigraphic section showing the inferred subsurface conditions at the proposed bridge site are shown on Drawing M8033003. A detailed description of the subsurface conditions encountered in the boreholes is provided in the following sections.

4.2.1 Sand

Extending from the ground surface is a sand deposit containing interlayers of sandy silt and silty sand. The sand extends to the depths investigated of 5.2 m and 6.7 m in Boreholes 97-6 and 97-9, respectively. Where fully penetrated in the deeper boreholes (97-7 and 97-8), the base of the sand deposit was at 7.6 m depth on the south side and at 5.6 m on the north side of the river.

The sand is typically in a very loose to loose state of packing; the measured Standard Penetration Test (SPT) 'N' values range from 1 blow to 9 blows for 0.3 m of penetration. The measured natural water content of selected samples of this deposit range from 3 per cent to 30 per cent. A grain size distribution for one sample of the sand is shown on Figure 1.

4.2.2 Silty Sand

Underlying the sand in Boreholes 97-7 and 97-8 is a deposit of greenish grey to grey silty sand containing trace clay, trace organic matter and thin sand seams. The silty sand deposit is 2.3 m thick in Borehole 97-7, located on the south side of the river and 4.4 m thick in Borehole 97-8, on the north side of the river. The silty sand is in a very loose to loose state of packing. The measured SPT 'N' values range from 1 to 4 blows for 0.3 m of penetration. The base of the silt was encountered at about 10 m depth in both boreholes.

4.2.3 Silt

The silty sand in both boreholes is underlain by 3.2 m and 4.8 m of silt with trace clay and thin sand seams. The silt is very loose and extends to 13.1 m depth in Borehole 97-7 and to 14.8 m depth in Borehole 97-8.

4.2.4 Clayey Silt and Silty Clay

The silt is underlain by a clayey silt / silty clay deposit. In Borehole 97-7, the upper 1.7 m of the deposit consists of stiff clayey silt with a liquid limit of 28 per cent and plasticity index of 5 per cent.

Underlying the clayey silt in Borehole 97-7 and the silt in Borehole 97-8 at 14.8 m depth in both boreholes is a silty clay deposit. The base of the silty clay deposit was at depths of 20.9 m and 24.1 m in Borehole 97-7 and Borehole 97-8, respectively. The measured SPT 'N' values in the silty clay generally range from 6 to 13 blows per 0.3 m of penetration indicating a firm to stiff consistency. The measured natural water content of the samples of this deposit varies from 28 per cent to 32 per cent. Atterberg Limit tests indicated liquid limits between 29 per cent and 35 per cent and plasticity indices between 8 per cent and 10 per cent. The grain size distribution test results for two samples of the silty clay are shown on Figure 2.

4.2.5 Sand, Cobbles and Boulders

About 1.6 m of very loose silty sand was encountered below the silty clay in Borehole 97-7. A 2.7 m to 8 m thick layer consisting of cobbles and boulders with sand and gravel underlies the silty sand deposit at 22.5 m depth in Borehole 97-7 and the silty clay deposit at 24.1 m depth in Borehole 97-8. The presence of cobbles and boulders within this layer was inferred based on auger resistance with persistent grinding and bouncing of the augers observed during drilling.

4.2.6 Bedrock

Refusal to further auger penetration was met in Boreholes 97-7 and 97-8 at 25.3 m and 32.0 m depth, respectively. Bedrock was cored in Borehole 97-8 commencing at 32 m depth confirming that the auger refusal met at this location was on the bedrock surface at about Elevation 256.5 m on the north side of the river. The dynamic cone penetration test carried out in the area of the south abutment encountered abrupt refusal at 29.7 m depth, about Elevation 258.7 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 probably indicates the bedrock surface.

Rock coring was carried out in Borehole 97-8 for a length of 3 m. The Rock Quality Designation (RQD) measured on the core samples ranged from 78 per cent to 87 per cent. Based on the rock core obtained, the bedrock consists of a Biotite-Hornblende Gneiss.

Based on the results of boreholes put down along the proposed northbound lanes and service road bridges to the east of the southbound lanes bridge, the bedrock surface slopes steeply downwards to the west (perpendicular to the road alignment).

4.2.7 Groundwater Conditions

No piezometers were installed in the boreholes put down along the Highway 11 Southbound bridge. The water levels in the open boreholes varied from 3.1 m to 3.4 m depth below the existing ground surface 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 Highway 11 southbound bridge 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.

The works described in this report are associated with the proposed bridge and its approaches within 20 m of the structure. The site of the bridge is designated as Site 42-09S. It is understood that the Highway 11 southbound bridge 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 293.6 m on the north side of the crossing. The proposed road grade requires an embankment height of up to 5.5 m for the south approach and about 5 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 vertical 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 open-ended pipe piles or steel H-piles driven to the bedrock surface which was inferred to be at about Elevation 259 m on the south side of the river and was encountered at about Elevation 256.5 m on the north side of the river. The above bedrock surface elevations may be assumed for design; the actual pile lengths required will have to be confirmed based on the driving records during installation.

Based on the general arrangement drawing provided, the base of pile cap at the piers will be at approximately Elevation 284 m and will be 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.

A layer of cobbles and boulders up to 8 m thick directly overlies the bedrock in the boreholes put down on both sides of the river. There is a possibility of encountering refusal to pile penetration on this bouldery layer. Steel H-piles may have a better chance of penetrating this type of cobble/boulder layer although the pile could be deflected during drilling. Open-end pipe piles give the opportunity of churn drilling from inside the casing through the boulders if the pile "hangs up". If the H-piles or pipe piles "hang up" above the bedrock surface and churn drilling is not carried out, a reduced pile capacity may have to be used and additional piles be 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 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.

<u>HP 310x110</u>	<u>HP 310x79</u>	<u>324 mm dia. pile</u>
2800 kN/pile	2000 kN/pile	1900 kN/pile

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.

Embankment construction at the site will induce consolidation settlement of the clayey silt / silty clay deposit. At the abutments, this consolidation will induce a downward movement of the soils adjacent to the piles and negative skin friction will develop along the portion of the pile shaft embedded within the clayey silt / silty clay deposit and within the sands and silts above the clayey silt. The following values of negative skin friction on a single pile should be assumed for design (developed as a result of consolidation):

HP 310x110
400 kN/pile

HP 310x79
400 kN/pile

324 mm dia. Pile
370 kN/pile

To minimize the amount of negative skin friction that may develop on a pile, consideration should be given to placing the approach embankment fill as soon as possible to permit as much of the settlement to occur prior to the driving of the piles. The embankment could also be surcharged to increase the magnitude of settlement. Surplus material after final grading could be used elsewhere on the site.

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 through the bouldery layer. Based on the results of the boreholes located immediately to the east, the bedrock surface slopes steeply 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 used 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 (m) (approx.)</i>	<i>Soil Type</i>	<i>$z \times n_h$ (MPa)</i>
289.0 to 276.0	Sands and Silts, very loose to loose	$z \times 1.0$
276.0 to 268.0 (south side) 276.0 to 264.5 (north side)	Clayey Silt to Silty Clay, firm to stiff	2.0 to 3.0 (constant with depth)
268.0 to 259* (south side) 264.5 to 256.5* (north side)	Silty sand and cobbles and boulders, very loose to loose	$z \times 1.0$

* approximate bedrock surface

Group action for lateral loading should be considered when the pile spacing in the direction of loading is less than six to eight 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 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

Based on the general arrangement drawing the bases of the pile caps at the abutments will be at about Elevation 289 m, at or above the existing grade; at the piers the bases of the pile caps will be at about Elevation 284 m, about 2 m below the river bed.

At the piers, the excavation may extend possibly to about 2.5 m below the river water level. Based on the information provided to us, the river water level was at Elevation 286.4 m in November 1996 and in August 1997. The excavations will be through loose sand deposits and some form of groundwater control will be required to permit pile installation and pile cap construction in the dry. In general, the sands are sensitive to disturbance and where they form the excavation base below the groundwater level, the soils undergo rapid loosening due to upward water seepage. Closed steel sheetpiling could be used as a cut-off to groundwater flow and to provide temporary support to the excavation. The cut-off wall must extend to sufficient depth below the base of the excavation to minimize piping of the sands forming the base. In order to provide a groundwater cut-off, sheetpiling should be extended into the clayey silt/silty clay stratum which was encountered at about Elevation 275 m.

The actual sheetpile depth requirements must be established by the contractor to ensure adequate base stability and overall stability of the sheetpile cofferdam. An undrained shear strength of 50 kPa may be assumed for the clayey silt / silty clay deposit.

Provision for pumping through properly filtered sumps at the base of excavation must be made. Alternatively, the excavation may be made in the wet within closed steel sheetpiling and a tremie plug placed at the base of the excavation.

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 approach embankments approximately 5 m high. Based on the subsurface information, the subsoils underlying the approach embankment consist generally of about 13 m to 15 m of very loose to loose sands and silts underlain by firm to stiff clayey silt and silty clay.

Given the above subsoil conditions, stability of the proposed embankments is not a concern with respect to deep seated failure through the founding soils. Settlement of the embankment due to consolidation of the upper sand and silt deposits will occur during embankment construction. Some long term settlement of the embankments will occur due to consolidation of up to 11 m thick clayey silt / silty clay deposit encountered below 13 m to 15 m depth. It is estimated that up to 150 mm of settlement may occur as a result of embankment construction.

// In order to minimize post-construction settlement of the approach embankments (and minimize differential settlement), consideration could be given to preloading. Preloading for about 9 months with the final 5 m to 5.5 m embankment height should induce approximately 100 mm to 120 mm of consolidation settlement. //

5.6 Subgrade Preparation and Embankment Construction

Topsoil and organic deposits should be stripped from below the fill embankment areas and the exposed subgrade soils should be proof-rolled prior to 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 the material available to the project. 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.

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.

GOLDER ASSOCIATES LTD.



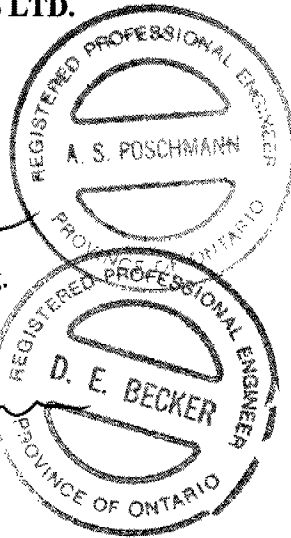
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AMP/ASP/amp/dh/clg
WORD S/FINALDAT/OTHPRT/971-8033/78033LR3

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_6 :

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 (L.V.-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

M8033006 BHS

PROJECT: 971-8033

RECORD OF BOREHOLE 97-6

SHEET 1 OF 1

LOCATION: N 5026589.788; E 326538.239

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.	NUMBER	TYPE	BLOWS/0.3m	SHEAR STRENGTH				WATER CONTENT, PERCENT					
				DEPTH (m)				nat V - + rem V - ⊕	Q - ● U - ○	Wp	W	Wi					
0	BOMBARDIER CME-55 HOLLOW STEM AUGERS	GROUND SURFACE		288.41 0.00													
		Sand, fine with some silt Very loose to loose Brown to grey Moist, becoming wet at 3.0m depth -trace organics noted to approx. 0.6m depth		1	50 DO	2											
1				2	50 DO	4											
				3	50 DO	4											
2				4	50 DO	2											
				5	50 DO	1											
3																	
4																	
5					6	50 DO	4										
		END OF BOREHOLE		283.23 5.18													
6																	
7																	
8																	
9																	
10																	

NOTE:
Water level in
open borehole at
3.1m depth on
completion of
drilling.

NOTE:
Water level in
open borehole at
3.1m depth on
completion of
drilling.

DATA INPUT: PS dec 18/97

SOIL M6

DEPTH SCALE

1 to 50

Golder Associates

LOGGED: MSB

CHECKED: DC

PROJECT: 971-8033

RECORD OF BOREHOLE 97-7

SHEET 2 OF 3

LOCATION: N 5026615.518; E 326524.276

BORING DATE: AUG.27/97

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 - ⊕ U - ○	Q - ● U - ○	WATER CONTENT, PERCENT Wp — W — Wi					
				DEPTH (m)												
10	BOMBARDIER CME-55 HOLLOW STEM AUGERS	CONTINUED FROM PREVIOUS PAGE														
11		Silt with trace clay and thin sand seams Very loose Dark greenish grey to grey Wet			10	50 DO	3									
12																
13																
14		Clayey Silt, trace sand Stiff Grey Wet		275.35 13.10	11	50 DO	WH									
15																
16		Silty Clay with trace sand Firm to stiff Brown to grey Wet		273.65 14.80	12	50 DO	9									
17																
18																
19																
20																
CONTINUED ON NEXT PAGE																

DEPTH SCALE

1 to 50

Golder Associates

LOGGED: MSB

CHECKED: DC

M8033007.BHS

DATA INPUT: PS dec.18/97

SOILM6

M603007.BHS

DATA INPUT: PS dec 18/97

SOILM6

PROJECT: 971-8033

RECORD OF BOREHOLE 97-7

SHEET 3 OF 3

LOCATION: N 5026615.518; E 326524.276

BORING DATE: AUG.27/97

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 — Wt
				DEPTH (m)									
20	BOMBARDIER CMES-55 HOLLOW STEM AUGERS	CONTINUED FROM PREVIOUS PAGE											
				267.55	16	50 DO	9						
21				20.90									
		Silty Sand Very loose Grey Wet			17	50 DO	2						
22				265.89 22.56									
23		Cobbles and Boulders with some sand, gravel and silty clay (inferred from resistance to augering and observations of auger cuttings during drilling operations).			18	50 DO	28/ .12						
24													
25		-predominantly silty sand with some gravel below 24.7 depth			19	AS							
		END OF BOREHOLE Refusal to augering Probably on bedrock		263.15 25.30									
26													
27													
28													
29													
30													

NOTE:
Water level in
open borehole at
3.4m depth on
completion of
drilling.

NOTE:
Water level in
open borehole at
3.4m depth on
completion of
drilling.

DEPTH SCALE

1 to 50

Golder Associates

LOGGED: MSB

CHECKED: DC

M8033008 BHS

PROJECT: 971-8033

RECORD OF BOREHOLE 97-8

SHEET 1 OF 5

LOCATION: N 5026643.683; E 326488.919

BORING DATE: AUG.28/97

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	
								nat V - + rem V - ⊗	Q - ● U - ○			Wp	W
0		GROUND SURFACE		288.50 0.00									
1	BOMBARDIER CME-55 HOLLOW STEM AUGERS	Sand, fine to medium with some silt Very loose to loose Brown to grey Moist becoming wet below 3m depth		1	50 DO	2							
2				50 DO	2								
3				50 DO	2								
4				50 DO	3								
5				50 DO	4								
6				50 DO	3								
7				50 DO	1								
8				50 DO	1								
9				50 DO	4								
10				50 DO									
		-becoming coarse below 4.5m depth		282.90 5.60									
		Silty Sand with trace organics Very loose Dark grey Wet											
		-trace clay and thin sand seams noted below 8.53m depth											
		CONTINUED ON NEXT PAGE		278.50 10.00									

DATA INPUT: PS dec. 18/97

SOIL/M8

DEPTH SCALE

1 to 50

Golder Associates

LOGGED: MSB

CHECKED: DC

PROJECT: 971-8033

RECORD OF BOREHOLE 97-8

SHEET 2 OF 5

LOCATION: N 5026643.683; E 326488.919

BORING DATE: AUG.28/97

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 - + Q - ● rem V - ⊕ U - ○			WATER CONTENT, PERCENT Wp — W — Wl 10 20 30 40
				DEPTH (m)								
10	BOMBARDIER CME-55 HOLLOW STEM AUGERS	CONTINUED FROM PREVIOUS PAGE		278.50 10.00								
11		Silt with trace clay and thin sand seams Very loose Dark greenish grey to grey Wet		10	50 DO	3						
12												
13												
14												
15												
16												
17												
18												
19												
20												

DEPTH SCALE

1 to 50

Golder Associates

LOGGED: MSB

CHECKED: DC

DATA INPUT: PS dec 18/97

SOLM6

M8033006 BHS

DATA INPUT: PS dec 18/97

SOIL M6

PROJECT: 971-8033

RECORD OF BOREHOLE 97-8

SHEET 3 OF 5

LOCATION: N 5026643.683; E 326488.919

BORING DATE: AUG.28/97

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	nat V - + rem V - @ U - O	WATER CONTENT, PERCENT Wp — W — Wi		
20	BOMBARDIER CME-55 HOLLOW STEM AUGERS	CONTINUED FROM PREVIOUS PAGE										
21		Silty Clay, trace sand Firm to stiff Grey Wet		16	50 DO	7						
22				17	50 DO	9						
23				18	50 DO	8						
24				264.42 24.08								
25		Cobbles and Boulders with clayey silt, sand and gravel (inferred from resistance to augering and observations of auger cuttings during drilling operations).										
26												
27												
28												
29												
30												
		CONTINUED ON NEXT PAGE										

DEPTH SCALE

1 to 50

Golder Associates

LOGGED: MSB

CHECKED: DC

PROJECT: 971-8033

RECORD OF BOREHOLE 97-8

SHEET 4 OF 5

LOCATION: N 5026643.683; E 326488.919

BORING DATE: AUG.28/97

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		
30	BOMBARDIER CME-55 HOLLOW STEM AUGERS	CONTINUED FROM PREVIOUS PAGE									
31		Cobbles and Boulders with clayey silt (inferred from resistance to augering and observations of auger cuttings during drilling operations).									
32		AUGER REFUSAL ON BEDROCK BOREHOLE CONTINUED									
33		For bedrock coring description refer to sheet 5.									
34											
35											
36											
37											
38											
39											
40		CONTINUED ON NEXT PAGE									

DEPTH SCALE

1 to 50

Golder Associates

LOGGED: MSB

CHECKED: DC

M8033008 BHS

DATA INPUT: PS dec.18/97

SOIL M6

PROJECT: 971-8033

RECORD OF DRILLHOLE: 97-8

SHEET 5 OF 5

LOCATION: N5026643.683; E 326488.919

DRILLING DATE: AUG.28/97

DATUM: GEODETIC

INCLINATION: AZIMUTH:

DRILL RIG: BOMBARDIER CME-55

DRILLING CONTRACTOR: MARATHON DRILLING



DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	PENETRATION RATE (m/min)	FLUSH & RETURN	FR-FRACTURE CL-CLEAVAGE SH-SHEAR VN-VEIN	F-FAULT J-JOINT P-POLISHED S-SLICKENSIDED	SM-SMOOTH R-ROUGH ST-STEPPED PL-PLANAR	FL-FLEXURED UE-UNEVEN W-WAVY C-CURVED	BC-BROKEN CORE MB-MECH. BREAK B-BEDDING	DIAMETRAL POINT LOAD INDEX (MPa)	NOTES WATER LEVELS INSTRUMENTATION
31		CONTINUED FROM PREVIOUS PAGE												
32				256.50 32.00										
33		BIOTITE-HORNBLLENDE GNEISS, crystalline, light grey, medium grained. (Bedrock)			1									
34					2									
35		END OF DRILLHOLE		253.40 35.10										
36														
37														
38														
39														
40														
41														

DEPTH SCALE:

1 to 50

LOGGED: MSB

DATE:

CHECKED: DC

Golder Associates

PROJECT: 971-8033

RECORD OF BOREHOLE 97-9

SHEET 1 OF 1

LOCATION: REFER TO PLAN

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 PLAT	ELEV. DEPTH (m)	NUMBER	TYPE	BLOWS/0.3m	SHEAR STRENGTH				WATER CONTENT, PERCENT	
								nat V - + Cu, kPa	rem V - ⊗ U - ○			Wp	W
0		GROUND SURFACE		288.35 0.00									
1	BOMBARDIER CME-55 HOLLOW STEM AUGERS	Sand, fine to medium with trace to some silt and trace gravel Very loose to loose Brown, becoming grey below 5m depth Moist becoming wet below 3.3m depth		1	50 DO	9							
2				50 DO	6								
3				50 DO	5								
4				50 DO	7								
5				50 DO	4								
6				50 DO	1								
7				50 DO	1								
7		END OF BOREHOLE		281.64 6.71									
8													
9													
10													

NOTE:
Water level in
open borehole at
3.3m depth during
drilling.

DEPTH SCALE

1 to 50

Golder Associates

LOGGED: MSB

CHECKED: DC

DATA INPUT: PS dec.18/97

SOIL M6

PROJECT: 971-8033

RECORD OF CONE TEST CT97-2

SHEET 1 OF 3

LOCATION: N 5026615.109; E 326511.135

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. DEPTH (m)	NUMBER	TYPE	BLOWS/0.3m	SHEAR STRENGTH				WATER CONTENT, PERCENT					
								nat V - Cu, kPa	rem V - U - O	+	-	Q - ● U - ○	Wp			W	Wi
0		GROUND SURFACE		288.44 0.00													
1		For soil stratigraphy refer to Borehole 97-7.															
2																	
3																	
4																	
5																	
6																	
7																	
8																	
9																	
10																	
		CONTINUED ON NEXT PAGE															

DEPTH SCALE

1 to 50

Golder Associates

LOGGED: MB

CHECKED: AMP

MB033CT2 BHS
DATA INPUT: PS DEC 18/97
SM PROBE

PROJECT: 971-8033

RECORD OF CONE TEST CT97-2

SHEET 2 OF 3

LOCATION: N 5028615.109; E 326511.135

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. DEPTH (m)	NUMBER	TYPE	BLOWS/0.3m	SHEAR STRENGTH Cu, kPa				WATER CONTENT, PERCENT					
								nat V - +	Q - ●	rem V - ⊕	U - ○	Wp	W	Wi			
10		CONTINUED FROM PREVIOUS PAGE															
11																	
12																	
13																	
14																	
15																	
16																	
17																	
18																	
19																	
20		CONTINUED ON NEXT PAGE															

DEPTH SCALE

1 to 50

Golder Associates

LOGGED: MB

CHECKED: AMP

PROJECT: 971-8033

RECORD OF CONE TEST CT97-2

SHEET 3 OF 3

LOCATION: N 5026615.109; E 326511.135

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. DEPTH (m)	NUMBER TYPE	BLOWS/0.3m	30 60 90 120	30 60 90 120	WATER CONTENT, PERCENT Wp — W — Wl 10 20 30 40		
20	BOMBARDIER CMES55	CONTINUED FROM PREVIOUS PAGE									
21											
22											
23											
24											
25											
26											
27											
28											
29											
30			END OF CONE TEST		258.72 29.72						

DEPTH SCALE

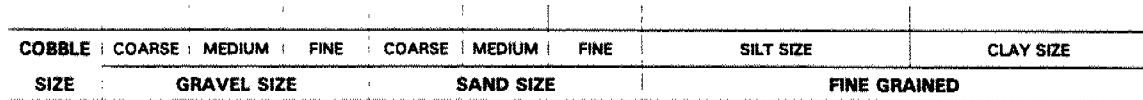
1 to 50

Golder Associates

LOGGED: MB

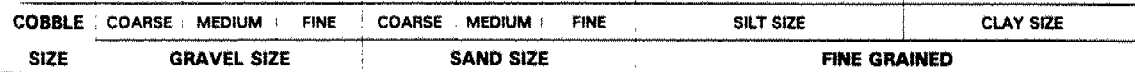
CHECKED: AMP

FIGURE 1



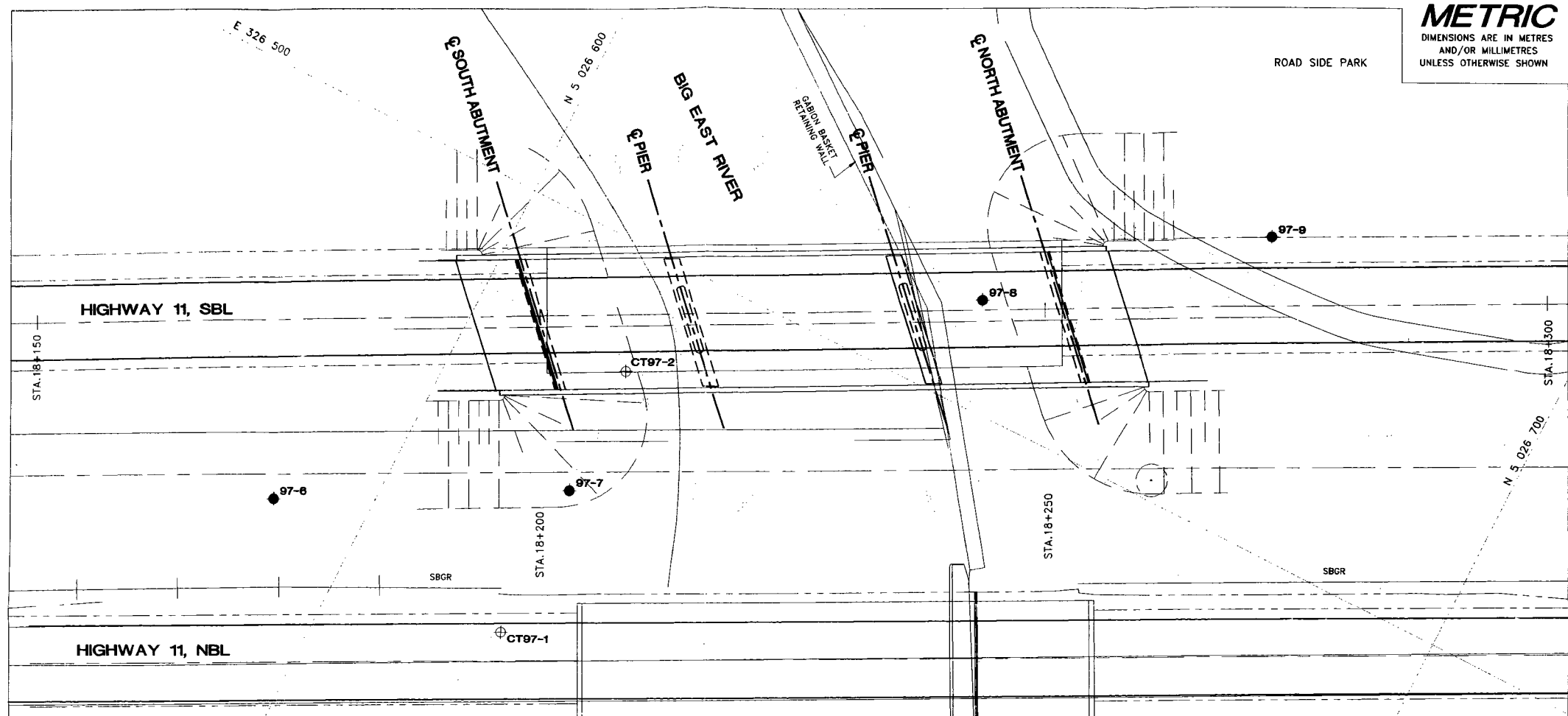
●	97-8	2	287.5
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FIGURE 2



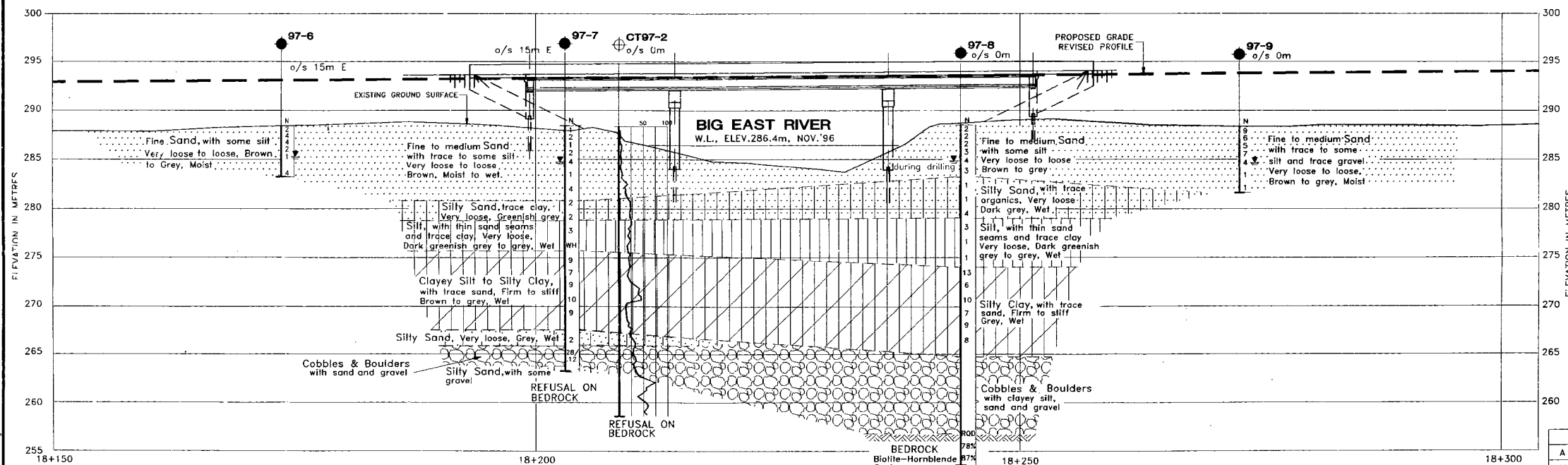
SYMBOL	BOREHOLE	SAMPLE ELEVATION(m)
1	1	1
2	2	2
3	3	3
4	4	4
5	5	5
6	6	6
7	7	7
8	8	8
9	9	9
10	10	10
11	11	11
12	12	12
13	13	13
14	14	14
15	15	15
16	16	16
17	17	17
18	18	18
19	19	19
20	20	20
21	21	21
22	22	22
23	23	23
24	24	24
25	25	25
26	26	26
27	27	27
28	28	28
29	29	29
30	30	30
31	31	31
32	32	32
33	33	33
34	34	34
35	35	35
36	36	36
37	37	37
38	38	38
39	39	39
40	40	40
41	41	41
42	42	42
43	43	43
44	44	44
45	45	45
46	46	46
47	47	47
48	48	48
49	49	49
50	50	50
51	51	51
52	52	52
53	53	53
54	54	54
55	55	55
56	56	56
57	57	57
58	58	58
59	59	59
60	60	60
61	61	61
62	62	62
63	63	63
64	64	64
65	65	65
66	66	66
67	67	67
68	68	68
69	69	69
70	70	70
71	71	71
72	72	72
73	73	73
74	74	74
75	75	75
76	76	76
77	77	77
78	78	78
79	79	79
80	80	80
81	81	81
82	82	82
83	83	83
84	84	84
85	85	85
86	86	86
87	87	87
88	88	88
89	89	89
90	90	90
91	91	91
92	92	92
93	93	93
94	94	94
95	95	95
96	96	96
97	97	97
98	98	98
99	99	99
100	100	100

October 1997



PLAN

0 5 10 15 20 30 40m
SCALE IN METRES



PROFILE ALONG HIGHWAY 11, SOUTH BOUND LANES

0 5 10 15 20 30 40m
SCALE IN METRES

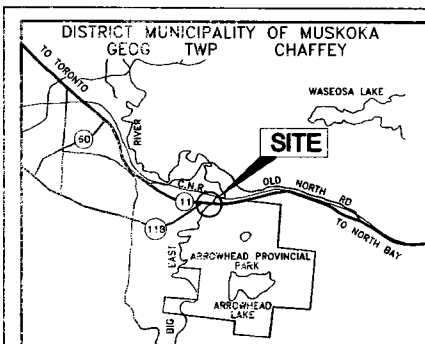
METRIC
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN

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

HIGHWAY 11 SOUTHBOUND LANES
OVER BIG EAST RIVER
BORE HOLE LOCATIONS & SOIL STRATA



Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



KEY PLAN

2km, 1km, 0, 2km, 4.0 kilometres
SCALE, km

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
97-6	288.41	5026589.79	326520.61
97-7	288.45	5026615.52	326524.28
97-8	288.50	5026643.68	326488.92
97-9	288.35	5026666.19	326470.42
CT97-2	288.44	5026615.11	326511.14

NOTES

The boundaries between soil strata have been established only of 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
DRAWN: MFC	CHKD: AMP	APPD.

SITE 42-09 S

DWG. M8033003