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**PRELIMINARY FOUNDATION INVESTIGATION  
AND DESIGN REPORT  
PROPOSED EAST END TRANSFER STRUCTURE  
HIGHWAY 402, GWP 3038-03-00  
AGREEMENT NUMBER 3005-A-000394**

Submitted to:

URS Canada Inc.  
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**PART A**

**PRELIMINARY FOUNDATION INVESTIGATION REPORT  
PROPOSED EAST END TRANSFER STRUCTURE  
HIGHWAY 402, GWP 3038-03-00  
AGREEMENT NUMBER 3005-A-000394**

## **1.0 INTRODUCTION**

Golder Associates Ltd. (Golder) has been retained by URS Canada Inc. (URS) on behalf of the Ministry of Transportation, Ontario (MTO) to carry out foundation investigations at various sites along Highway 402 in conjunction with GWP 3038-03-00 which extends from the Bluewater Bridge Authority plaza east for 16 kilometres to Lambton Road 26 (Mandaumin Road) in Sarnia, Ontario. This report addresses the proposed structure at the East End Transfer Facility.

The purpose of the foundation investigation is to determine the subsurface conditions at the site of the proposed structure. The terms of reference for the scope of work are outlined in Golder's Total Project Management (TPM) proposal P31-3109, dated December 2003 and amended by our letter dated June 22, 2005. The work was carried out in accordance with our Quality Control of TPM Services Plan, Agreement No. 3005-A-000394, dated May 2004.

URS provided Golder with a general arrangement drawing for the future East End Transfer Facility where the west bound traffic will be separated into local, U.S. bound truck and U.S. car traffic. This report focuses on the single lane overpass structure which will convey U.S. bound truck traffic over two lanes of west bound local traffic.

## **2.0 SITE DESCRIPTION**

The project area covered by this report is located on the north side of Highway 402, approximately 11.8 kilometres east of the east end of the Blue Water Bridge over the St. Clair River. The subject site is situated about 500 metres east of Brigden Road and 125 metres west of Pulse Creek North in a rural agricultural area. The surrounding area is generally flat with cultivated fields with scrub vegetation bordering Pulse Creek. The ground surface elevation in the vicinity of the proposed structure varies between 180 and 182 metres. The site location is shown on Figure 1.

### 3.0 INVESTIGATION PROCEDURES

The field work for this investigation was carried out between August 3 and 8, 2005. Two boreholes, EB-1 and EB-2, were drilled at the west and east sides of the proposed structure, respectively. The boreholes were advanced to depths of 36.0 metres at EB-1 and 36.6 metres at EB-2. The borehole locations are shown in plan on Drawing 1.

The investigation was carried out using a Nodwell track mounted CME 75 drill rig supplied and operated by Aardvark Drilling. The boreholes were drilled using mud rotary drilling techniques with an N-sized tricone bit being used to penetrate the underlying bedrock. In the boreholes, samples of the overburden were obtained using 50 millimetre outside diameter split-spoon samplers in accordance with the Standard Penetration Test (SPT) procedures. Samples were obtained at 0.76 metre intervals to a depth of 4.6 metres, then at 1.5 metre intervals to a depth of 15.2 metres, then at 3.0 metre intervals thereafter. Groundwater conditions in the open boreholes were observed throughout the drilling operations. Both of the boreholes were backfilled using MTO recommended procedures and as required by Ontario Regulation 903 (amended by Ontario Regulation 128/03).

The field work was supervised on a full-time basis by members of our engineering staff who located the boreholes in the field, obtained utility locates, directed the drilling, sampling and in-situ testing operations, and logged the boreholes. The soil samples were identified in the field, placed in labeled containers and transported to our laboratory in London, Ontario for further examination. Index and classification tests, consisting of grain size analyses and water content determinations were carried out on selected samples. Consolidation testing was carried out on a thin walled tube sample from borehole EB-1 at our Mississauga laboratory. The results of the field and laboratory testing are given on the Record of Borehole sheets and in Appendix A.

The as-drilled borehole locations and elevations were surveyed by J.D. Barnes Limited. The elevations at the boreholes are referenced to geodetic datum.

The borehole locations are shown in plan on Drawing 1. The subsurface conditions encountered in the boreholes are shown in profile on Drawing 1.

The borehole locations and ground surface elevations are summarized as follows:

BOREHOLE NUMBER	BOREHOLE LOCATION		GROUND SURFACE ELEVATION (m)
	NORTHING (m)	EASTING (m)	
EB-1	4761663.28	323542.69	181.35
EB-2	4761659.70	323663.93	180.52

## **4.0 GENERAL SITE GEOLOGY AND STRATIGRAPHY**

### **4.1 Geology**

The area of the site is located in the physiographic region known as the St. Clair Till Plain<sup>1</sup>. Geological information indicates that the general soil conditions for the area consist of glacial lacustrine deposits overlying deep lacustrine till deposits.

The surficial glaciolacustrine deposits represent the shoreline and near shores of former Lakes Algonquin and Nipissing. These deposits consist of sand, silt and minor amounts of gravel. The lacustrine tills underlying the surficial deposits are referred to as the St. Joseph Tills and generally consist of silty clay to clayey silt materials deposited in glacial Lake Whittlesey or Lake Warren during the Wisconsin period of glaciation. The upper 3 to 5 metres of the till deposit has been desiccated and oxidized forming a crust, the lower extent of which corresponds to the long-term groundwater level in the deposit. The St. Joseph Tills are commonly separated from the underlying black shale bedrock by massive to laminated lacustrine sandy silt to clay.

The average overburden thickness is 34 metres and generally varies from about 30 to 40 metres in the area of the site. The bedrock belongs to the Kettle Point Formation. It is black bituminous shale with greenish grey silty shale interbeds. Beneath the Kettle Point Formation, the bedrock reportedly consists of a sequence of shale, limestone and dolomite of the Hamilton and Port Lambton Groups.

### **4.2 Site Stratigraphy**

The detailed subsurface soil and groundwater conditions encountered in the boreholes are shown on the Record of Borehole sheets. The results of the field and laboratory testing are shown on the Record of Borehole sheets and are also presented in Appendix A. The stratigraphic boundaries shown on the borehole sheets are inferred from non-continuous sampling and, therefore, may represent transitions between soil types rather than exact planes of geological change. Subsoil conditions will vary between and beyond the borehole locations. Locations and elevations of the borings are shown on the attached Drawing 1 along with the interpreted stratigraphic profile along the centerline of the proposed structure.

In summary, the subsurface conditions encountered at the site consist of approximately 33 to 34 metres of overburden consisting of surficial topsoil and fill overlying extensive deposits of silty clay till, silty clay and occasionally clayey silt till. Bedrock was encountered at elevations 146 and 148 metres.

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<sup>1</sup> L.J. Chapman and D.F. Putnam: The Physiography of Southern Ontario, Third Edition. Ontario Geological Survey, Special Volume 2, 1984.



A detailed description of the subsurface conditions encountered in the boreholes for this investigation is provided on the Record of Borehole sheets and is summarized in the following sections.

#### **4.2.1 Topsoil**

Topsoil was encountered at the surface of both boreholes. The topsoil thicknesses were 240 and 270 millimetres.

#### **4.2.2 Fill**

Fill was encountered beneath the topsoil at elevation 180.3 metres in borehole EB-2. The fill layer was about 1 metre thick and consisted of firm clayey silt with some topsoil. The fill had an N value of 4 blows per 0.3 metres and a water content of 30 per cent.

#### **4.2.3 Silty Clay Till**

Silty clay till was encountered in both boreholes. An upper silty clay till layer was found beneath the topsoil at elevation 181.1 metres in borehole EB-1 and beneath the fill at elevation 179.3 metres in borehole EB-2. A lower silty clay till layer was encountered beneath the silty clay layers at elevations 166.8 and 151.8 metres in borehole EB-1 and at elevations 157.7 metres in borehole EB-2. The upper silty clay till layers were 7.6 to 9.7 metres thick. A stiff to very stiff desiccated crust was noted above approximate elevation 177 metres. The lower silty clay till layers were 3.4 to 13.9 metres thick.

The silty clay till is comprised of an average of 42 percent silt, 32 per cent clay, 22 per cent sand and 4 per cent gravel based on grain size analyses conducted on four representative samples of silty clay till. The grain size distribution curve is shown on Figure A-1.

The upper silty clay till was firm to very stiff with standard penetration test N values of 5 to 24 blows per 0.3 metres. The shear strengths measured in the softer zones using in-situ vane testing ranged from 103 to over 144 kilopascals indicating a very stiff material. Based on the remoulded strengths, sensitivities varied between 1.5 and 2.3 with an average of 1.9. The lower silty clay was firm to very stiff with standard penetration test N values of 4 to 24 blows per 0.3 metres. The shear strengths measured in the softer zones using in-situ vane testing were 75 and 106 kilopascals indicating a stiff to very stiff material. The sensitivity of the lower silty clay till was 1.4 and 1.7.

Water contents of the silty clay till samples ranged from 13 to 23 per cent with an average of 15 per cent. The average plastic and liquid limits were 14 and 29 per cent with an average plasticity index of 15 per cent based on four samples. The results of the Atterberg Limits determination shown on Figure A-2 indicate that the silty clay till is of low plasticity.

The results of the laboratory consolidation testing carried out on sample 8 from borehole EB-1 are provided on Figures A-3 and A-4. The results indicate that the silty clay till is slightly preconsolidated by about 45 kilopascals beyond the existing overburden pressure. The following table summarizes the relevant oedometer test results.

<u>BOREHOLE AND SAMPLE</u>	<u>DEPTH</u> (m)	<u><math>\sigma'_{po}</math></u> (kPa)	<u><math>\sigma'_p</math></u> (kPa)	<u>OCR</u>	<u><math>e_o</math></u>	<u><math>C_r</math></u>	<u><math>C_c</math></u>	<u><math>C_v</math></u> cm <sup>2</sup> /sec
EB-1 SA 8	7.9	160	205	1.28	0.58	0.057	0.133	1.2x10 <sup>-3</sup>

#### 4.2.4 Silty Clay

Silty clay was intercepted beneath the silty clay till at elevations 171.4 and 152.9 metres in borehole EB-1, and at elevation 171.7 metres in borehole EB-2. The thickness of the upper silty clay layer near elevation 172 metres was 4.5 to 14.0 metres. The lower silty clay layer in borehole EB-1 was 1.1 metres thick.

The silty clay had standard penetration test N values between 1 and 8 blows per 0.3 metres with an average of 3 blows per 0.3 metres. The shear strength of the softer zones in the upper silty clay stratum was measured by in situ vane testing which indicated undrained shear strengths ranging from 46 to 103 kilopascals (kPa) indicating a firm to very stiff consistency. Sensitivities of the silty clay ranged from 1.6 to 2.3 with an average of 1.8.

The water contents of silty clay samples ranged from about 24 to 38 per cent with an average of 33 per cent. Based on three samples, the silty clay deposits were of low to intermediate plasticity with average plastic and liquid limits of 17 and 34 per cent, respectively, and an average plasticity index of 17 per cent. The results of the Atterberg Limit testing are shown on Figure A-2.

The results of three grain size analyses of samples obtained in the clayey silt deposits indicated an average of 50 per cent clay, 44 per cent silt, 5 per cent sand and 1 per cent gravel. The grain size distributions are shown on Figure A-5 in Appendix A.

#### 4.2.5 Clayey Silt Till

Clayey silt till was found at elevation 147.2 metres and below the lower silty clay till layer in borehole EB-2. The stiff clayey silt till layer was about 1 metre thick with a standard penetration test N value of 29 blows per 0.3 metres.

#### **4.2.6 Bedrock**

The bedrock surface was encountered at elevations 148.3 and 146.2 metres in boreholes EB-1 and EB-2, respectively. The bedrock was probed with a tricone bit for 2.3 to 2.9 metres to the termination depth of the boreholes. Fragments of black shale of the Kettle Point formation were retrieved from the rotary drilling techniques.

The report for a proposed Highway 402 bridge, which was never constructed over Pulse Creek, approximately 50 metres east of borehole EB-2 stated that the surface of the shaley limestone bedrock at that location was proven at elevation 147.9 metres and inferred based on refusal to augering at elevations 147.3 to 148.3 metres.<sup>2</sup> It was also noted in the previous investigation report that natural gas was observed in the boreholes near the bedrock.

#### **4.3 Groundwater Conditions**

Boreholes EB-1 and EB-2 were dry during and upon completion of drilling. The 1969 Department of Highways Ontario report noted that groundwater was encountered between elevation 179.2 to 179.7 metres near Pulse Creek during the period October 31 to November 6, 1969. Pulse Creek was noted to be dry in September 1969.

It should be noted that the encountered groundwater elevations reported do not indicate the long term stable ground water levels and that the groundwater levels are subject to seasonal fluctuations. Based on soil colour changes and the encountered and measured groundwater levels, the long term groundwater level has been inferred to be within the upper silty clay till deposits between elevations 178 and 180 metres.

#### **5.0 MISCELLANEOUS**

The investigation was carried out using equipment supplied and operated by Aardvark Drilling, Inc. (Aardvark). Aardvark is an Ontario Ministry of Environment licensed well contractor. Field operations were supervised by Mr. Lubomir Kosc under the direction of Mr. David J. Mitchell. All routine laboratory testing was conducted at Golder Associates' London laboratory. The consolidation test was conducted at Golder Associates' Mississauga laboratory. Both laboratories are accredited participants in the MTO's Soil and Aggregate Proficiency program and are certified for full quality testing of Types C and D Aggregates by the Canadian Council of Independent Laboratories. The Mississauga laboratory is registered in the specialty of Soil and Rock Including Testing for Foundation Engineering – Low and High Complexity.

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<sup>2</sup> Department of Highways Ontario 1969, Foundation Investigation Report For Proposed Crossing at Pulse Creek and C.A.H. #402, Line 'C', Lot 5, Concession VII, Twp. Of Sarnia – Co. of Lambton, District #1 (Chatham, Ont.) W.J. 69-F-94 – W.P. 43-66-07 & 08.

This report was written by Ms. Dirka U. Prout, P. Eng., a geotechnical engineer under the direction of the Project Manager, Mr. Philip R. Bedell, P. Eng., a Principal with Golder Associates Ltd. The report was reviewed by Mr. Fintan J. Heffernan, P. Eng., the Designated MTO Contact and Quality Control Auditor.

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**PART B**

**PRELIMINARY FOUNDATION DESIGN REPORT  
PROPOSED EAST END TRANSFER STRUCTURE  
HIGHWAY 402, GWP 3038-03-00  
AGREEMENT NUMBER 3005-A-000394**

## **6.0 ENGINEERING RECOMMENDATIONS**

### **6.1 General**

This section of the report provides our recommendations on the foundation aspects for the preliminary design phase of the project. It should be noted that the interpretation of the factual information obtained during the investigation for the proposed structure and recommendations made 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 methods and scheduling.

It is understood that the existing Highway 402 will be widened by adding an Exclusive Truck Lane (ETL) in advance of the toll facility for the Blue Water Bridge. The ETL will require the widening and/or replacement of existing Highway 402 bridges in the area. New structures such as the east end transfer facility will also be erected in order to separate U.S. bound trucks and cars from the local westbound traffic. The east end transfer facility will include construction of a new structure to convey U.S.-bound truck traffic over the local west bound traffic lanes.

The approximate elevation of the deck of the proposed east end transfer structure is 188.9 metres. The existing ground surface elevation in the vicinity of the structure varies between 181 and 182 metres. The structure will be approximately 122 metres long with an interior width of 14.2 metres. The general arrangement drawings indicate that the structure will be founded on strip footings approximately 4 metres wide at about elevation 180.4 metres on the west side of the structure and 181.2 metres on the east side.

### **6.2 Bridge Foundations**

Based on the results of the soil investigation, the subsurface conditions in the area of the proposed structure typically consisted of firm clayey silt fill and surficial topsoil up to 1.2 metres thick. The topsoil and fill is underlain by a firm to very stiff upper silty clay till layer. The upper silty clay till layer is underlain by firm to stiff silty clay from about elevation 172 metres. Beneath the silty clay is a lower layer of firm to very stiff silty clay till from approximate elevation 167 and 158 metres on the west and east sides of the structure respectively. The lower silty clay till was noted to contain seams of silty clay and clayey silt till. Black shale bedrock of the Kettle Point formation was encountered at elevations 148 metres at the west side and 146 metres on the east side after penetrating 33 to 34 metres of cohesive overburden. The long term groundwater elevation is believed to be between 178 and 180 metres.

Various shallow and deep foundation alternatives have been assessed based on the subsurface information noted above. After comparing spread footings, perched abutments and end-bearing

piles, it was determined that strip footings are the optimal founding option. The risks, consequences, costs and feasibility of the featured options are compared in Table I. The feasibility of the various founding options are discussed below.

### **6.3 Shallow Foundations**

#### **6.3.1 Axial Geotechnical Resistance – Strip Footings**

Shallow foundations are the preferred founding option due to the ease of and lower cost of construction. Foundations alternatives such as use of strip footings on the native silty clay till or perched on granular pads in the embankments were considered. For preliminary design of strip footings founded on the silty clay till crust below all fill and topsoil layers and at approximately elevations of 179 to 180 metres, a factored geotechnical resistance of 300 kilopascals at Ultimate Limit States (ULS) and a geotechnical resistance of 200 kilopascals at the Serviceability Limit States (SLS) can be used for an assumed 4 metre wide footing.

The SLS value provided is based on 25 millimetres of structure settlement. Settlement of the footings due to consolidation of the extensive underlying clayey deposits and differential settlement of the embankment fills relative to the footings should be expected. The magnitude of the settlements will depend on the actual footing size, configuration and applied loading. Various measures, such as staged embankment construction, involving additional time and costs, could be taken to minimize the amount of the settlement(s). Prior to final design, additional laboratory testing should be carried out to further define the compressibility characteristics of the subsoils to refine settlement predictions.

The geotechnical resistances provided are given under the assumption that the loads will be applied perpendicular to the surface of the footings; where the load is not applied perpendicular to the surface of the footing, inclination of the load should be taken into account in accordance with the current Canadian Highway Bridge Design Code (CHBDC).

#### **6.3.2 Resistance to Lateral Forces**

Resistance to lateral forces/sliding resistance between the concrete spread footings and the subsoil should be calculated in accordance with Section 6.7.5 of the CHBDC. Assuming that the founding soils are not disturbed during excavation and footing construction, the following angle of friction between the concrete and the founding soils, and corresponding coefficient of friction,  $\tan \delta$ , may be used:

Footings on silty clay till	angle of friction	28°
	$\tan \delta$	0.53

### **6.3.3 Frost Protection**

All footings should be provided with a minimum of 1.2 metres of earth cover for frost protection purposes.

## **6.4 Deep Foundations**

Steel H-piles or concrete filled tube piles driven to practical refusal on bedrock are an alternative foundation option. However due to the length of the structure and the practicality of founding on shallow footings, deep foundations are not the preferred foundation option.

### **6.4.1 Geotechnical Axial Resistance – Driven Steel Piles**

For preliminary design, the factored axial geotechnical resistance at Ultimate Limit States (ULS) for HP 310 x 110 piles driven to refusal in the shale bedrock at about elevation 143 metres may be taken as 2,000 kilonewtons (kN). This value takes into account the structural capacity limitation of the pile. Vertically driven pile tips should be equipped with Type I driving shoes in accordance with current MTO practice (Standard Ontario Provincial Standard Drawing (OPSD) 3301.00) and battered piles should be equipped with Type II driving shoes to ensure adequate seating of the piles on the bedrock. If this option is chosen, the surface elevation and quality of the bedrock should be confirmed in the investigation for the final design.

A Serviceability Limit States (SLS) value is not provided because the shale bedrock is considered to be an unyielding material. Under such conditions, SLS values (for 25 millimetres of settlement) do not govern design because the SLS value is much higher than the ULS value.

The pile driving note to be added to the drawings is: “Piles to be driven to bedrock”.

Post-construction settlement of the underlying cohesive deposits may induce downdrag loads on the piles. These loads can be minimized by constructing the embankments well in advance of the pile operations, embankment pre-loading together with surcharging or by use of lightweight fills. If the embankments are not constructed well before the piles are installed, negative skin friction must be considered during design.

Further comments on pile design can be provided should piled foundations be selected as the founding option at the final design stage.

## **6.5 Embankments**

The embankments for the proposed structure are expected to be up to 6 metres in height. The actual extent of the additional fills required is currently unknown. Embankment side slopes formed no steeper than 2 horizontal to 1 vertical are considered suitable for this site. A Factor of



Safety against deep seated failure of greater than 1.3 is available for embankments constructed with suitable native or borrow materials.

All topsoil and any organic or otherwise deleterious materials should be removed from within the area of the embankment and the exposed subgrade soils should be proofrolled and benched prior to fill placement.

Construction of the embankment widening 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 availability. All embankment fill should be placed in regular lifts and compacted.

Embankment settlements will be dependent on the extent of the additional filling required and settlements could be reduced by preloading or using lightweight fill. Preliminary estimates of total embankment settlements within the cohesive deposits are about 325 millimetres in the centre of the embankments and 100 millimetres in the toe areas.

Settlements could be reduced further by up to 40 per cent by using lightweight fill. Primary settlements subsequent to paving can be reduced by preloading and surcharging. A more detailed settlement analysis should be conducted once the construction sequencing is known and the design has been finalized.

## **6.6 Excavations and Temporary Cut Slopes**

Excavations for footings will likely encounter the surficial fill and granular deposits above the groundwater table. Temporary open cut slopes should be maintained no steeper than 1 horizontal to 1 vertical.

Surficial water seepage into the excavations should be expected, and will be heavier during periods of sustained precipitation. Surface water runoff should be directed away from the excavations at all times. Pumping from properly filtered sumps located at the base of the excavations may be required to provide groundwater control during foundation excavations. Sumps should be maintained outside of the actual footing limits. The appropriate Non Standard Special Provisions (NSSP) should be included in the contract documents.

All excavations should be carried out in accordance with the guidelines outlined in the latest edition of the Ontario Occupational Health and Safety Act and Regulations For Construction Projects. Any fills encountered at this site would be classified as Type 3 soils. The silty clay till and silty clay deposits would be classified as Type 2 soils.

Roadway protection should conform to Performance Level 2, SP No. 539S01.

## **7.0 CLOSURE**

This report was written by Ms. Dirka U. Prout, P. Eng., a geotechnical engineer under the direction of the Project Manager, Mr. Philip R. Bedell, P. Eng., a Principal with Golder Associates Ltd. The report was reviewed by Mr. Fintan J. Heffernan, P. Eng., the Designated MTO Contact and Quality Control Auditor.

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**TABLE I**  
**COMPARISON OF FOUNDATION ALTERNATIVES**  
**EAST END TRANSFER STRUCTURE**  
**G.W.P. 3038-03-00**

<b>Foundation Option</b>	<b>Feasibility</b>	<b>Advantages</b>	<b>Disadvantages</b>	<b>Estimated Costs</b>	<b>Risks/Consequences</b>
Strip footings supported on crust of upper silty clay till	<ul style="list-style-type: none"> <li>• Feasible for support of all foundation elements</li> </ul>	<ul style="list-style-type: none"> <li>• Least Expensive option</li> <li>• Preferred technical solution</li> <li>• Ease of construction</li> </ul>	<ul style="list-style-type: none"> <li>• Time and cost of settlement mitigation measures; if time is available, preloading, possibly with surcharging would be relatively inexpensive</li> <li>• Even if mitigation adopted, settlement of shallow foundations could still take place</li> </ul>	<ul style="list-style-type: none"> <li>• Estimated cost \$195, 000 assuming two 4 m wide strip footings</li> <li>• Expected to be less expensive than deep foundation options</li> <li>• Cost of settlement mitigation measures not included</li> </ul>	<ul style="list-style-type: none"> <li>• Even if mitigation in place, shallow foundations may still be affected by settlement of underlying clayey deposits</li> </ul>
End bearing steel H-pile foundations driven to shale bedrock	<ul style="list-style-type: none"> <li>• Feasible for support of all foundation elements however loadings could be adequately carried by shallow foundations</li> </ul>	<ul style="list-style-type: none"> <li>• High bearing resistance</li> <li>• Negligible settlement</li> </ul>	<ul style="list-style-type: none"> <li>• Possibility of pile tip damage during driving in rock</li> <li>• Care must be taken with driving of battered piles to ensure that they do not deflect along the bedrock surface</li> <li>• More costly than shallow footings and not essential for required loading</li> </ul>	<ul style="list-style-type: none"> <li>• Estimated cost \$360, 000</li> <li>• More expensive than shallow foundations</li> </ul>	<ul style="list-style-type: none"> <li>• Possible pile tip damage if piles are not adequately protected while driving in bedrock</li> <li>•</li> </ul>

**NOTES:**

- 1) Costs are very preliminary estimates and are intended to provide a comparison between alternatives rather than actual construction costs.
- 2) Table to be read in conjunction with accompanying report.

## LIST OF ABBREVIATIONS

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

### I. SAMPLE TYPE

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

### III. SOIL DESCRIPTION

#### (a) Cohesionless Soils

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

### II. PENETRATION RESISTANCE

#### Standard Penetration Resistance (SPT), N:

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

#### Consistency

	<u>kPa</u>	<u>psf</u>
Very soft	0 to 12	0 to 250
Soft	12 to 25	250 to 500
Firm	25 to 50	500 to 1,000
Stiff	50 to 100	1,000 to 2,000
Very stiff	100 to 200	2,000 to 4,000
Hard	over 200	over 4,000

#### (b) Cohesive Soils

#### Dynamic Cone Penetration Resistance; $N_d$ :

The number of blows by a 63.5 kg. (140 lb.) hammer dropped 760 mm (30 in.) to drive uncased a 50 mm (2 in.) diameter, 60° cone attached to "A" size drill rods for a distance of 300 mm (12 in.).

**PH:** Sampler advanced by hydraulic pressure

**PM:** Sampler advanced by manual pressure

**WH:** Sampler advanced by static weight of hammer

**WR:** Sampler advanced by weight of sampler and rod

#### Piezo-Cone Penetration Test (CPT)

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

### IV. SOIL TESTS

w	water content
$w_p$	plastic limit
$w_l$	liquid limit
C	consolidation (oedometer) test
CHEM	chemical analysis (refer to text)
CID	consolidated isotropically drained triaxial test <sup>1</sup>
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement <sup>1</sup>
$D_R$	relative density (specific gravity, $G_s$ )
DS	direct shear test
M	sieve analysis for particle size
MH	combined sieve and hydrometer (H) analysis
MPC	Modified Proctor compaction test
SPC	Standard Proctor compaction test
OC	organic content test
$SO_4$	concentration of water-soluble sulphates
UC	unconfined compression test
UU	unconsolidated undrained triaxial test
V	field vane (LV-laboratory vane test)
$\gamma$	unit weight

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

## LIST OF SYMBOLS

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

### I. General

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

### II. STRESS AND STRAIN

$\gamma$	shear strain
$\Delta$	change in, e.g. in stress: $\Delta \sigma$
$\epsilon$	linear strain
$\epsilon_v$	volumetric strain
$\eta$	coefficient of viscosity
$\nu$	poisson's ratio
$\sigma$	total stress
$\sigma'$	effective stress ( $\sigma' = \sigma - u$ )
$\sigma'_{vo}$	initial effective overburden stress
$\sigma_1, \sigma_2, \sigma_3$	principal stress (major, intermediate, minor)
$\sigma_{oct}$	mean stress or octahedral stress $= (\sigma_1 + \sigma_2 + \sigma_3)/3$
$\tau$	shear stress
u	porewater pressure
E	modulus of deformation
G	shear modulus of deformation
K	bulk modulus of compressibility

### III. SOIL PROPERTIES

#### (a) Index Properties

$\rho(\gamma)$	bulk density (bulk unit weight*)
$\rho_d(\gamma_d)$	dry density (dry unit weight)
$\rho_w(\gamma_w)$	density (unit weight) of water
$\rho_s(\gamma_s)$	density (unit weight) of solid particles
$\gamma'$	unit weight of submerged soil ( $\gamma' = \gamma - \gamma_w$ )
$D_R$	relative density (specific gravity) of solid particles ( $D_R = \rho_s / \rho_w$ ) (formerly $G_s$ )
e	void ratio
n	porosity
S	degree of saturation

#### (a) Index Properties (continued)

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

#### (b) Hydraulic Properties

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

#### (c) Consolidation (one-dimensional)

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

#### (d) Shear Strength

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

- Notes:**
- 1  $\tau = c' + \sigma' \tan \phi'$
  - 2 shear strength  $= (\text{compressive strength})/2$
  - \* density symbol is  $\rho$ . Unit weight symbol is  $\gamma$  where  $\gamma = \rho g$  (i.e. mass density x acceleration due to gravity)

<b>PROJECT</b> 041-130099-8		<b>RECORD OF BOREHOLE No EB-1</b>		1 OF 3	<b>METRIC</b>
<b>G.W.P.</b> 3048-03-00		<b>LOCATION</b> N 4761663.3 ; E 323542.7		<b>ORIGINATED BY</b> LK	
<b>DIST</b> 1 <b>HWY</b> 402		<b>BOREHOLE TYPE</b> MUD ROTARY / N TRICONE		<b>COMPILED BY</b> DCH	
<b>DATUM</b> GEODETIC		<b>DATE</b> August 5, 2005 - August 8, 2005		<b>CHECKED BY</b>	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE LIQUID LIMIT			UNIT WEIGHT  $\gamma$  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)  GR SA SI CL							
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa											w <sub>p</sub>	w	w <sub>L</sub>
181.35	GROUND SURFACE																				
0.00	TOPSOIL, clayey																				
181.08	Black																				
0.27	SILTY CLAY, trace sand, trace gravel (TILL) Firm to very stiff Brown to grey at about elevation 178.3m																				
			1	SS	24																
			2	SS	24																
			3	SS	20																
			4	SS	11																
			5	SS	7																
			6	SS	5																
			7	SS	7																
			8	SS	PH																
			9	TO	6																
171.35	SILTY CLAY, trace sand, trace gravel Firm to stiff Grey		10	SS	6																
10.00																					
			11	TO	PH																
			12	SS	2																
166.82																					
14.53																					

ONL\_MTO 041-130099-8.GPJ ON MOT.GDT 7/29/06

Continued Next Page

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity      ○ 3% STRAIN AT FAILURE



PROJECT <u>041-130099-8</u>		<b>RECORD OF BOREHOLE No EB-1</b>		3 OF 3	<b>METRIC</b>
G.W.P. <u>3048-03-00</u>		LOCATION <u>N 4761663.3 ; E 323542.7</u>		ORIGINATED BY <u>LK</u>	
DIST <u>1</u> HWY <u>402</u>		BOREHOLE TYPE <u>MUD ROTARY / N TRICONE</u>		COMPILED BY <u>DCH</u>	
DATUM <u>GEODETIC</u>		DATE <u>August 5, 2005 - August 8, 2005</u>		CHECKED BY _____	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC NATURAL LIQUID LIMIT MOISTURE LIMIT CONTENT			UNIT WEIGHT  $\gamma$  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)				GR	SA	SI	CL																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																													
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# RECORD OF BOREHOLE No EB-2

1 OF 3

**METRIC**

PROJECT 041-130099-8

G.W.P. 3048-03-00

LOCATION N 4761659.7 ; E 323663.9

ORIGINATED BY LK

DIST 1 HWY 402

BOREHOLE TYPE MUD ROTARY / N TRICONE

COMPILED BY DCH

DATUM GEODETIC

DATE August 3, 2005 - August 4, 2005

CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT		NATURAL MOISTURE CONTENT		LIQUID LIMIT		UNIT WEIGHT  γ  kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)					
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)					GR	SA	SI	CL	
180.52	GROUND SURFACE							20	40	60	80	100										
0.00	TOPSOIL, clayey Black																					
0.24	FILL, clayey silt, some topsoil, trace sand Firm Brown and black																					
179.30			1	SS	4																	
1.22	SILTY CLAY, trace sand, trace gravel (TILL) Firm to very stiff Brown to grey at about elevation 177.5m																					
			2	SS	9																	
			3	SS	10																	
			4	SS	7																	
			5	SS	7																	
			6	SS	7																	
			7	SH	PH																	
			8	SS	5																	
171.69																						
8.83	SILTY CLAY, trace sand Firm to stiff Grey		9	SS	2																	
			10	SS	1																	
			11	SS	1																	
			12	SS	2																	
166.19																						
14.33	SILTY CLAY, trace sand, trace gravel Very stiff to firm Grey																					

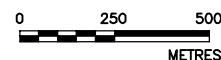
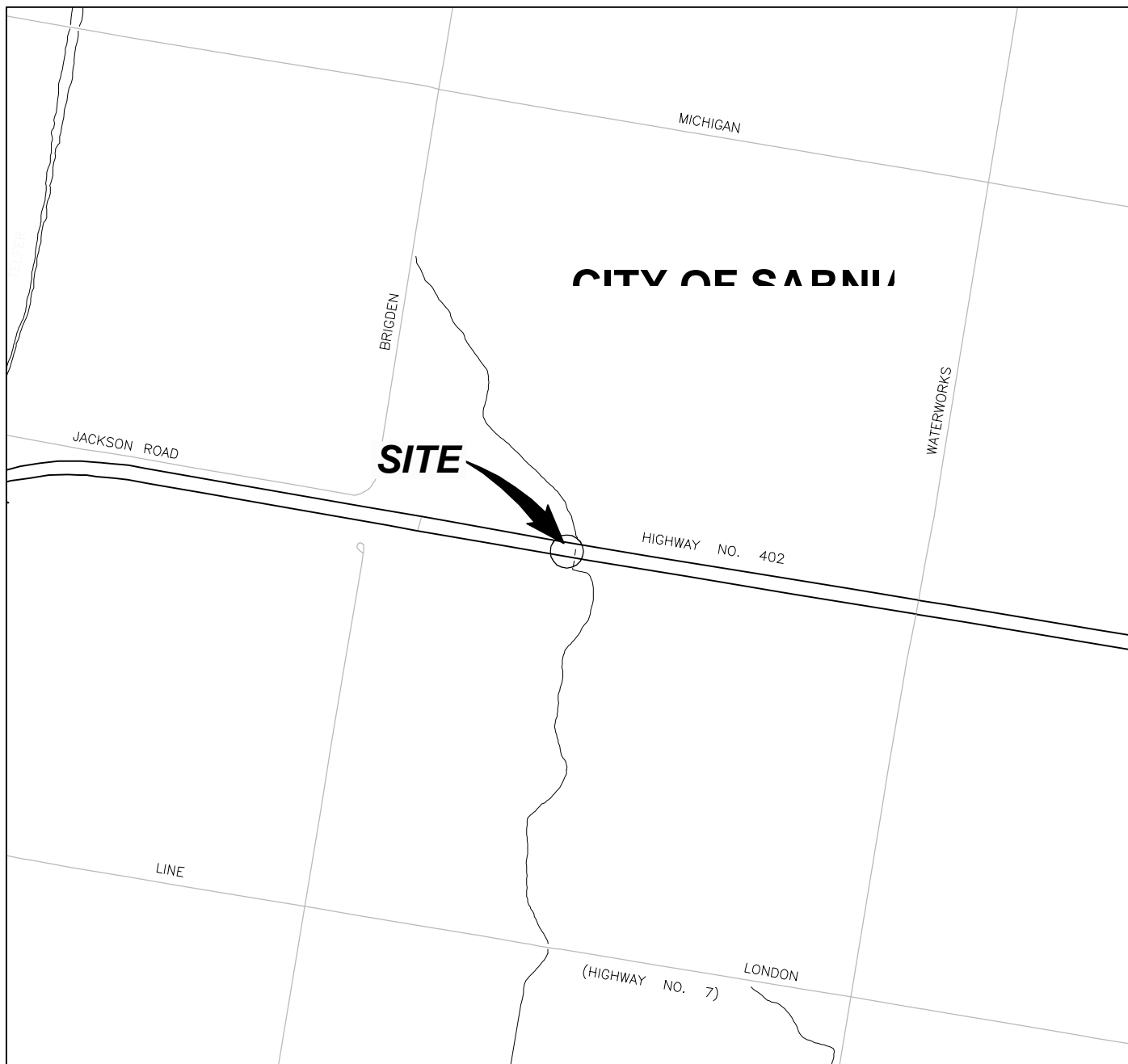
Continued Next Page


+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE



PROJECT <u>041-130099-8</u>		<b>RECORD OF BOREHOLE No EB-2</b>		3 OF 3	<b>METRIC</b>
G.W.P. <u>3048-03-00</u>	LOCATION <u>N 4761659.7 ; E 323663.9</u>	ORIGINATED BY <u>LK</u>			
DIST <u>1</u> HWY <u>402</u>	BOREHOLE TYPE <u>MUD ROTARY / N TRICONE</u>	COMPILED BY <u>DCH</u>			
DATUM <u>GEODETIC</u>	DATE <u>August 3, 2005 - August 4, 2005</u>	CHECKED BY _____			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT   NATURAL LIMIT   MOISTURE   LIQUID CONTENT   LIMIT			UNIT WEIGHT  γ  kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)				GR	SA	SI	CL	
								○ UNCONFINED   + FIELD VANE ● QUICK TRIAXIAL   × LAB VANE					w <sub>p</sub> w   w <sub>L</sub>								
							20	40	60	80	100										
147.24																					
33.28																					
146.23																					
34.29																					



PROJECT		EAST END TRANSFER STRUCTURE WP No. 3038-03-00 HWY. 402							
TITLE		SITE LOCATION PLAN							
 <b>Golder Associates</b> LONDON, ONTARIO		PROJECT No.		041-130099-8		FILE No.		041130099-8F001	
						SCALE		AS SHOWN	
		CADD		DCH		July 19/06		REV.	
		CHECK						0	
		FIGURE 1							

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

DIST 1 HWY. 402  
CONT. No.  
WP No. 3038-03-00

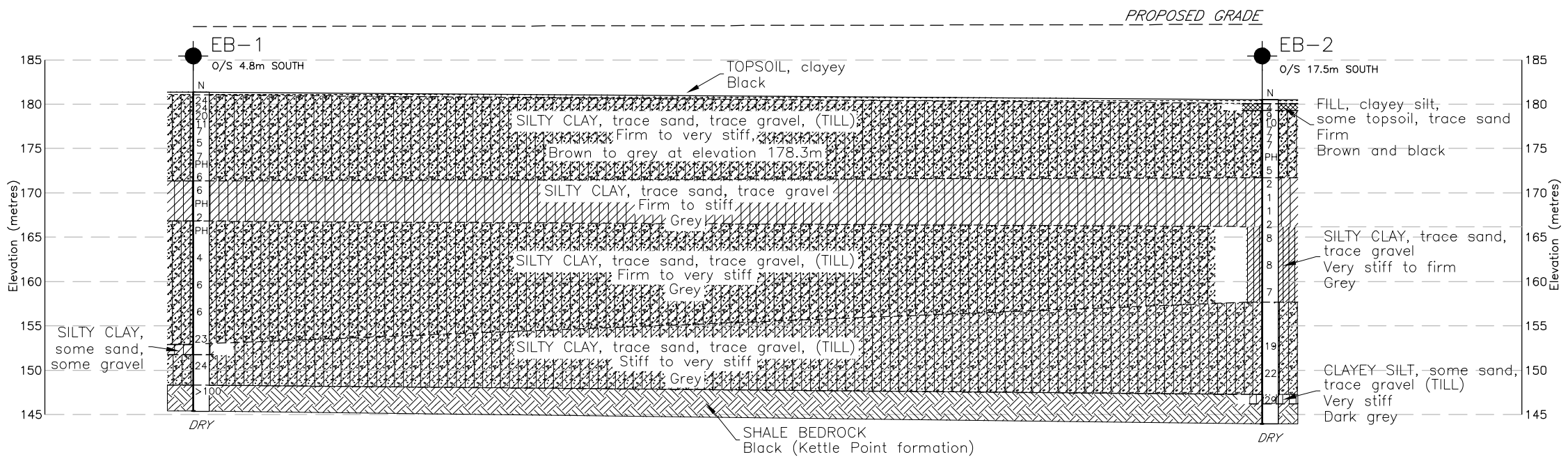
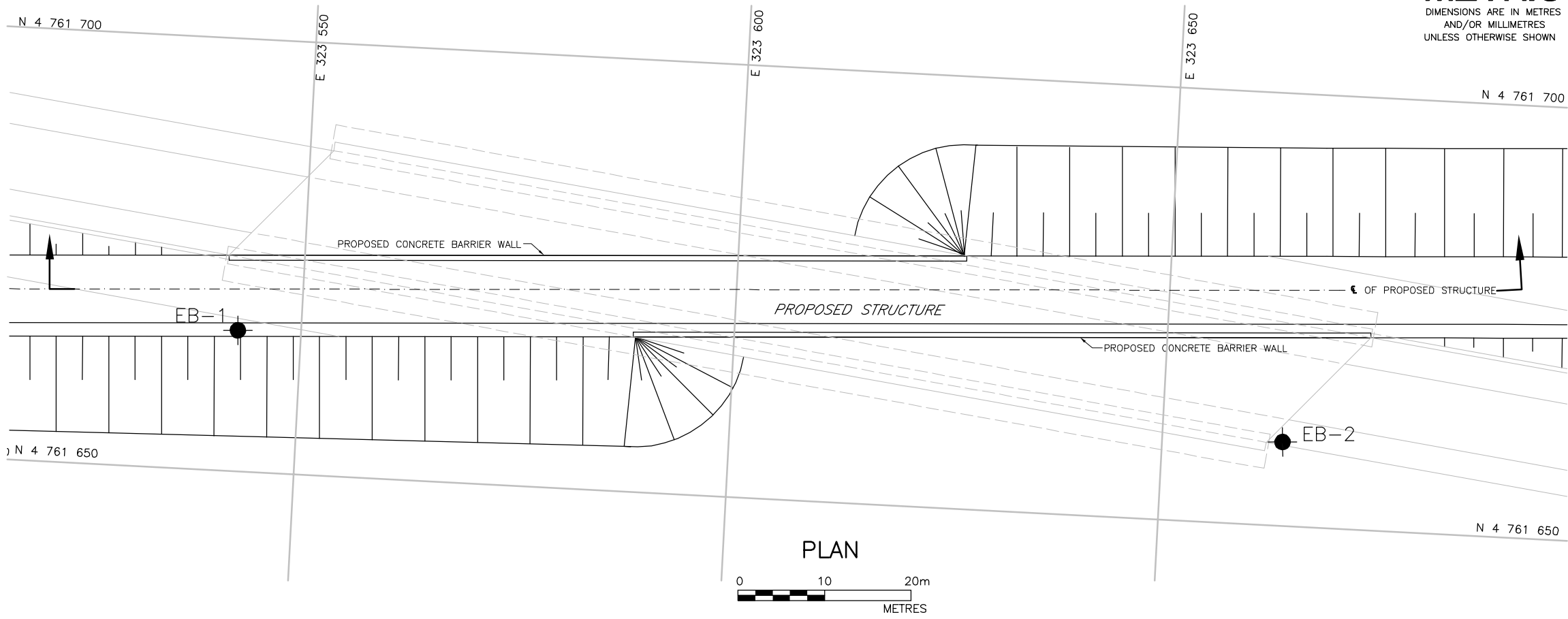
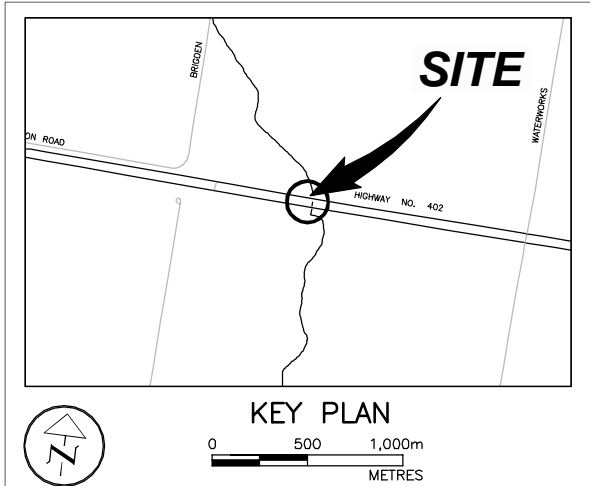


EAST END TRANSFER  
STRUCTURE  
BOREHOLE LOCATIONS & SOIL STRATA

SHEET



Golder Associates Ltd.  
LONDON, ONTARIO, CANADA



LEGEND

- Borehole
- N Blows/0.3m (Std. Pen. Test, 475 j/blow)
- DRY Borehole dry during drilling

No.	ELEVATION (metres)	CO-ORDINATES	
		NORTH	EAST
EB-1	182.06	4 761 663.28	323 542.69
EB-2	180.52	4 761 659.70	323 663.93

NOTES

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

This drawing is for subsurface information only. The proposed structure details are shown for illustration purposes only and may not be consistent with the final design configuration as shown elsewhere in the Contract Documents

REFERENCE

REFERENCE : URS  
ENTITLED: EAST END TRANSFER FACILITY GENERAL ARRANGEMENT  
DATED: JULY 2005

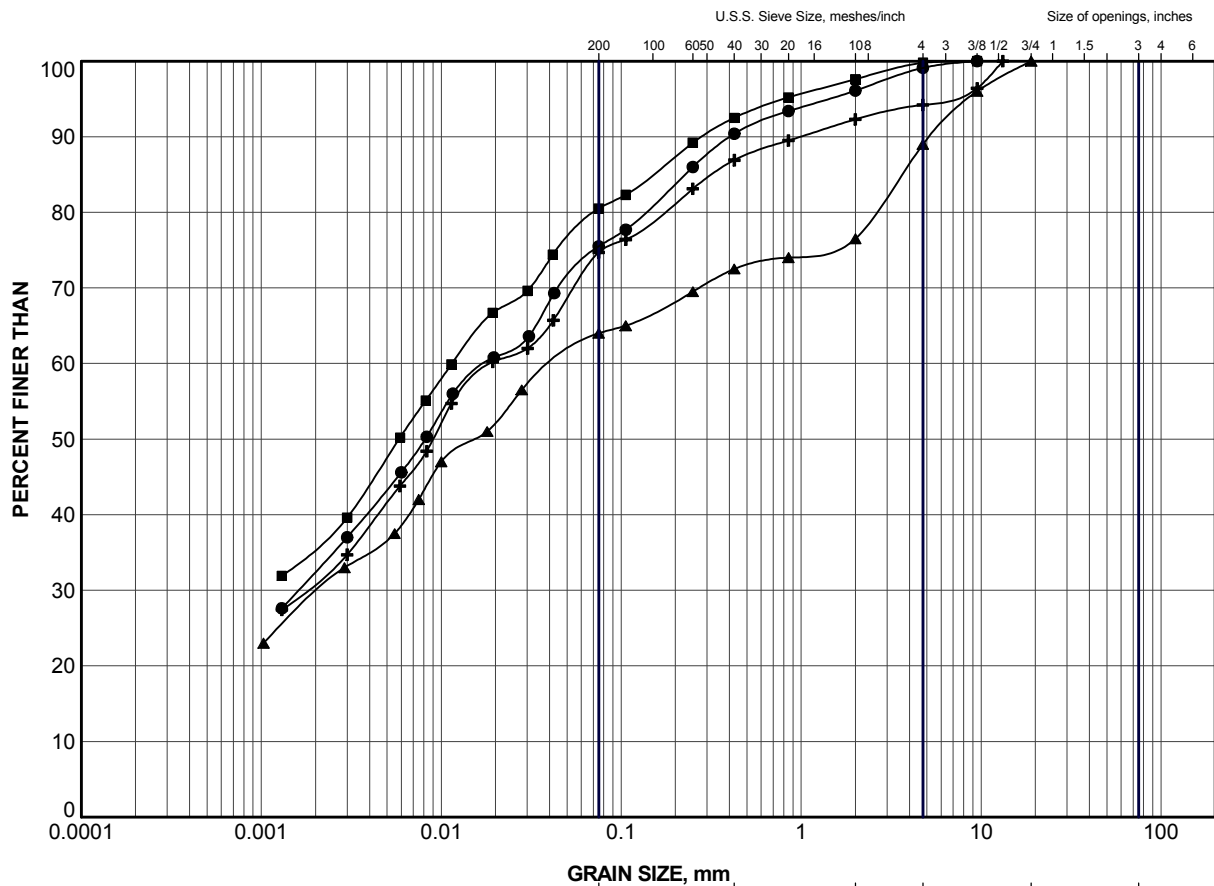
NO.	DATE	BY	REVISION
Geocres No. 40J16-67			
HWY. No.	402	PROJECT NO.:	041-130099-8
SUBM'D.	-	CHKD:	DATE: July 19/06
DRAWN:	DCH	CHKD.	APPD.
			DWG. 1

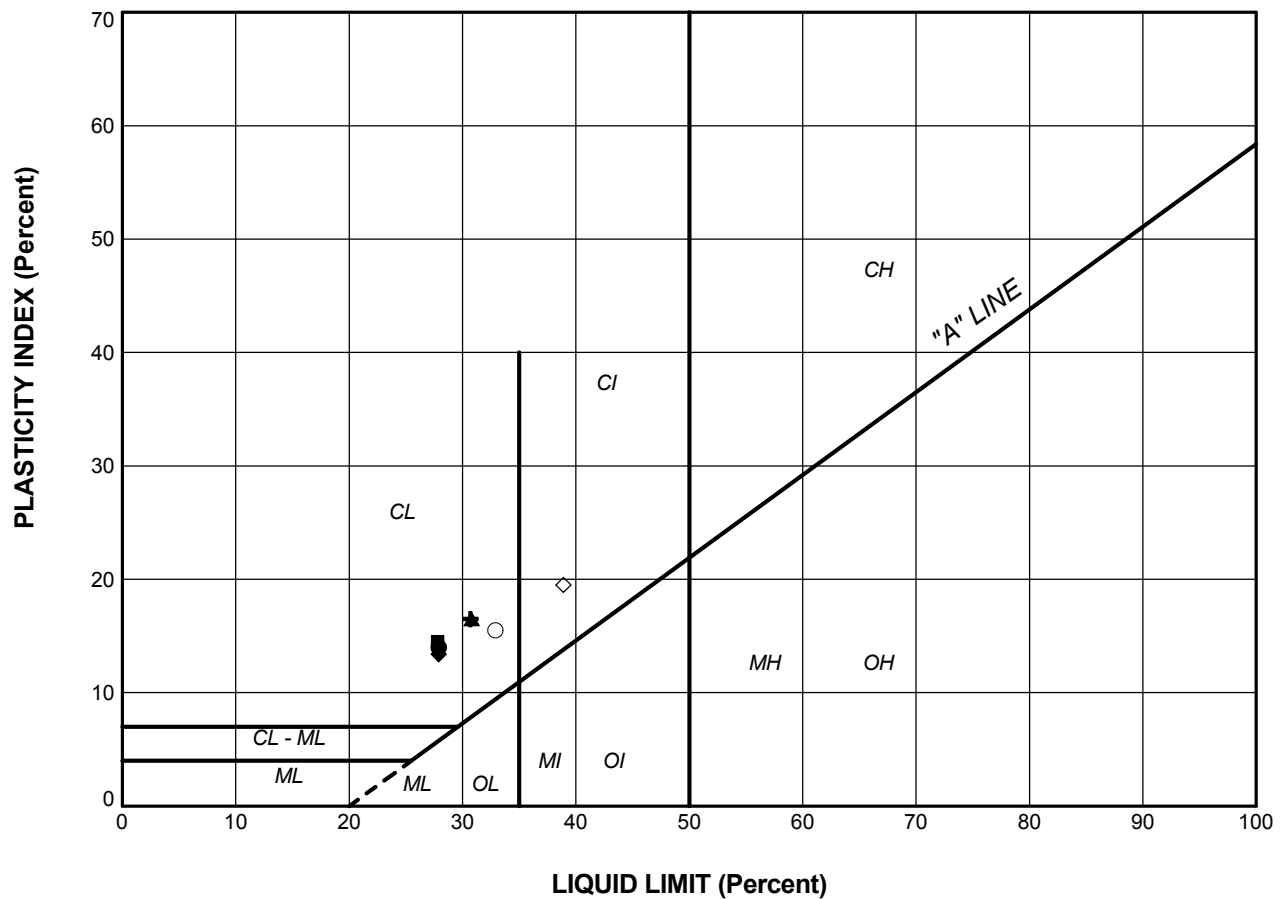
1 = 1 metric  
D size dwg 22" x 32" 11" x 17" plot half scale

041130099-8D001.dwg

**APPENDIX A**


**RECORDS OF LABORATORY TEST RESULTS**





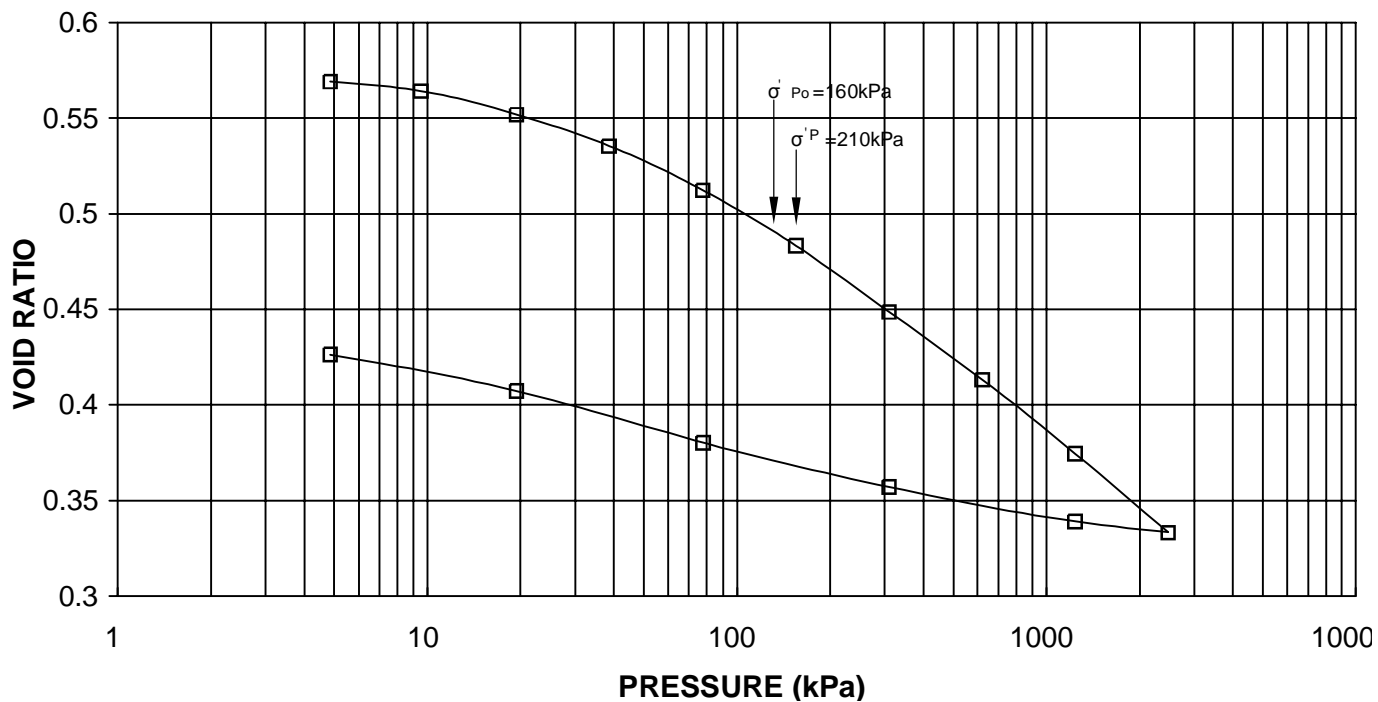
### LEGEND


	SYMBOL	BOREHOLE	SAMPLE	ELEV (m)	LL(%)	PL(%)	PI
SILTY CLAY TILL	●	EB-1	4	178.1	27.9	13.9	14.0
	■	EB-1	7	174.3	27.8	13.3	14.5
	▲	EB-1	8	173.5	30.8	14.3	16.5
	◆	EB-2	5	176.5	27.9	14.5	13.4
SILTY CLAY	+	EB-1	12	167.4	30.7	14.2	16.5
	◇	EB-2	9	171.2	38.9	19.4	19.5
	○	EB-2	13	165.1	32.9	17.4	15.5

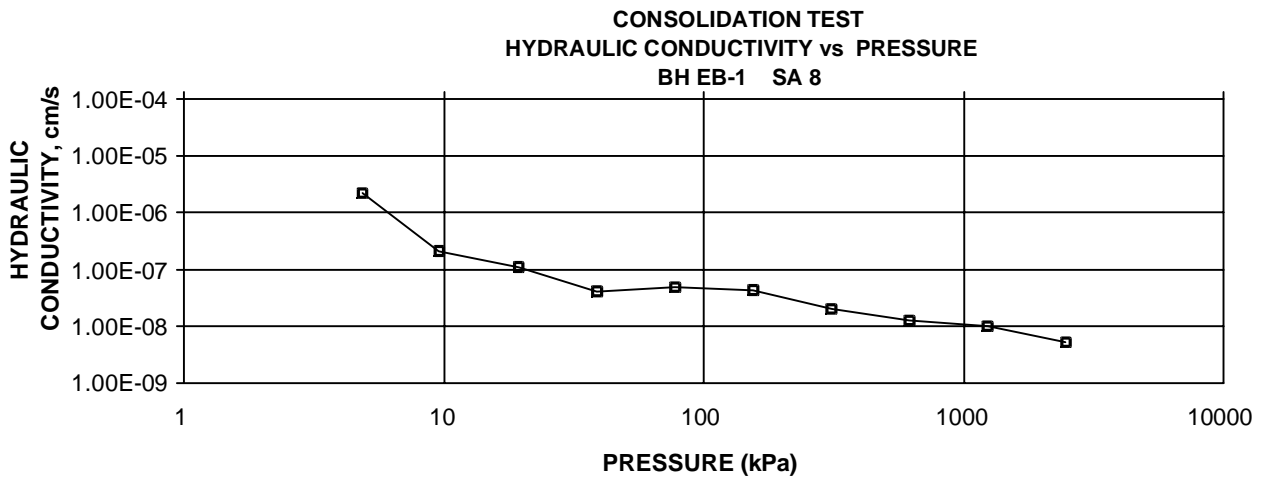
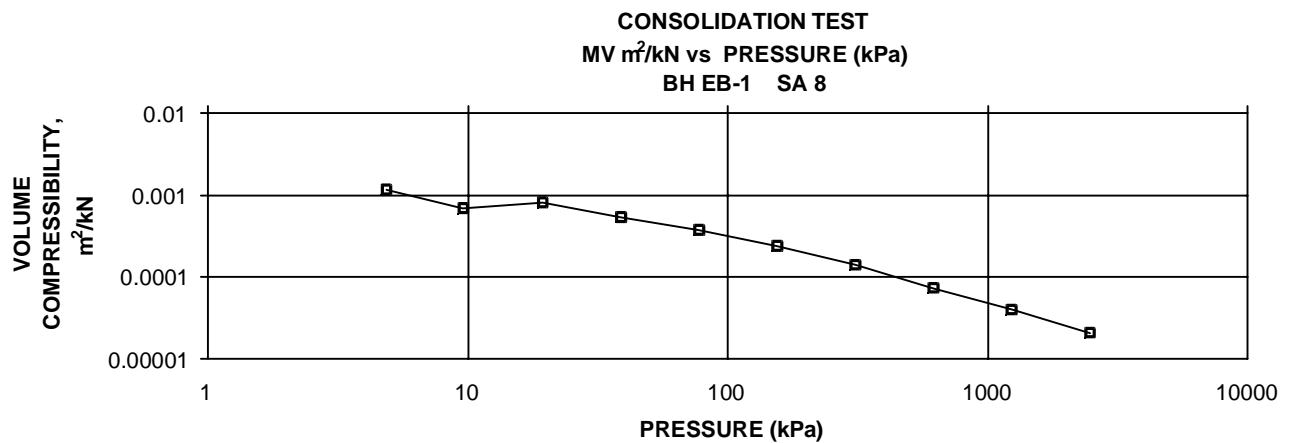
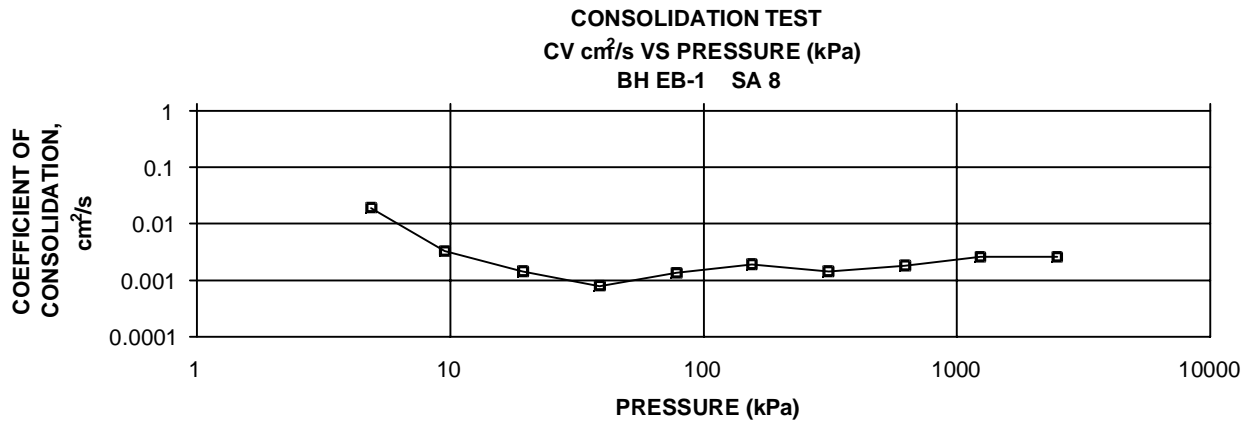
PROJECT				EAST END TRANSFER STRUCTURE WP No. 3038-03-00 HWY. 402			
TITLE				PLASTICITY CHART			
PROJECT No.		041-130099-8		FILE No.		041-130099-8.GPJ	
DRAWN	DCH	Sep 08/05		SCALE	N/A	REV.	
CHECK				FIGURE A-2			
 <b>Golder Associates</b> LONDON, ONTARIO							



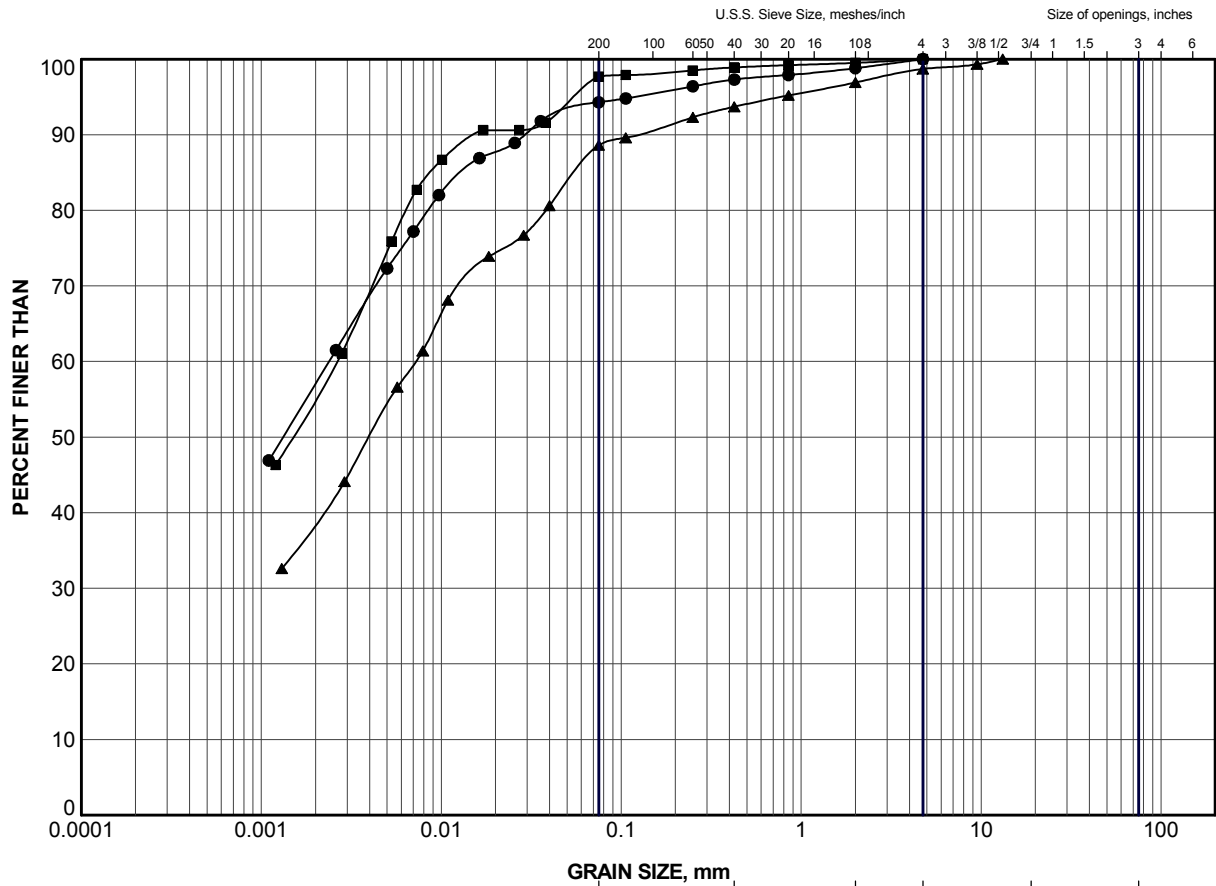
**CONSOLIDATION TEST  
VOID RATIO vs PRESSURE  
BH EB-1 SA 8**



PROJECT		EAST END TRANSFER STRUCTURE WP No. 3038-03-00 HWY. 402			
TITLE		CONSOLIDATION TEST VOID RATIO Vs. LOG PRESSURE			
 <b>Golder Associates</b> LONDON, ONTARIO		PROJECT No. 041-130099-8		FILE No. 041130099-FOA3	
		CADD	DCH	Sept. 08/05	SCALE AS SHOWN
		CHECK			REV. 0
					<b>FIGURE A-3</b>




<b>PROJECT</b>	EAST END TRANSFER STRUCTURE WP No. 3038-03-00 HWY. 402			
<b>TITLE</b>	<b>CONSOLIDATION TEST RESULTS</b>			
<b>Golder Associates</b> LONDON, ONTARIO	<b>PROJECT No.</b>	041-130099-8	<b>FILE No.</b>	041130099-FOA4
	<b>CADD</b>	DCH	Sept. 08/05	<b>SCALE</b> AS SHOWN
	<b>CHECK</b>			<b>REV.</b> 0
FIGURE A-4				



CLAY AND SILT	GRAVEL SIZE, mm					Cobble Size
	fine	medium	coarse	fine	coarse	
	SAND SIZE			GRAVEL SIZE		

#### LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	EB-1	12	167.4
■	EB-2	9	171.2
▲	EB-2	13	165.1

PROJECT		EAST END TRANSFER STRUCTURE WP No. 3038-03-00 HWY. 402			
TITLE		GRAIN SIZE DISTRIBUTION SILTY CLAY			
PROJECT No.		041-130099-8		FILE No. 041-130099-8.GPJ	
DRAWN		DCH		Sept. 08/05	
CHECK					
 <b>Golder Associates</b> LONDON, ONTARIO		<b>FIGURE A-5</b>			