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**PRELIMINARY FOUNDATION INVESTIGATION
AND DESIGN REPORT
PROPOSED CHRISTINA STREET UNDERPASS
REPLACEMENT
HIGHWAY 402, GWP 3038-03-00
AGREEMENT NUMBER 3005-A-000394**

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LIST OF ABBREVIATIONS

LIST OF SYMBOLS

RECORDS OF BOREHOLES

FIGURE 1 - Site Location Plan

DRAWING 1 - Borehole Location Plan and Soil Strata

APPENDIX A - Laboratory Test Results

APPENDIX B - Site Photographs

PART A

**PRELIMINARY FOUNDATION INVESTIGATION REPORT
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Golder Associates

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 preliminary 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 replacement of the Christina Street underpass structure. The location of the site is shown on the site location plan, Figure 1.

The purpose of the foundation investigation is to determine the subsurface conditions at the site of the proposed new bridge by drilling two boreholes at the site. The terms of reference for the scope of work are outlined in Golder's Total Project Management (TPM) proposal P31-3109, dated December 2003. 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 general arrangement drawings for the existing Highway 402 underpass structure and the proposed replacement structure. The original structure drawing was prepared by the Department of Highways, Ontario and was dated August 25, 1950.

2.0 SITE DESCRIPTION

The project area covered by this report is located on Christina Street at the crossing of Highway 402 south of the Village of Point Edward and east of Front Street within the City of Sarnia, Ontario. The site location is shown on the site location plan, Figure 1. The site is located near the harbour area of Sarnia. The ground surface is relatively flat at an elevation of about 6 metres above the St. Clair River.

The existing bridge structure is a two span rigid frame bridge. The existing bridge deck is at elevation 188.9 metres and the top of pavement elevation at Highway 402 is about 181.7 metres.

Selected site photographs are presented in Appendix B.

3.0 INVESTIGATION PROCEDURES

The field work for this investigation was carried out between July 12 and 20, 2004. At that time two boreholes were put down at the site of the proposed bridge abutments. The boreholes were drilled and sampled to depths of 53.5 to 55.2 metres. The borehole locations are shown in plan on Drawing 1.

The investigation was carried out using a track mounted CME 75 drill rig supplied and operated by Aardvark Drilling. The boreholes were advanced using a combination of 159 millimetre outside diameter continuous flight hollow stem augers and mud rotary drilling techniques. In the boreholes, samples of the overburden were obtained at suitable intervals of depth using 50 millimetre outside diameter split-spoon samplers in accordance with the Standard Penetration Test (SPT) procedures. In addition, thin walled tube samples were obtained from the clayey soil for consolidation testing. 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 a member 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 grains 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 2 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 understood to be referenced to geodetic datum.

4.0 GENERAL SITE GEOLOGY AND STRATIGRAPHY

4.1 Geology

The area of the site is located in the physiographic region of Southwestern Ontario¹ known as the St. Clair Clay Plain. Geological information indicates that the general soil conditions for the area consist of glacial lacustrine deposits overlying deep lacustrine till deposits.

The surficial deposits are referred to as Lake Algonquin-Nipissing deposits. Consisting of sand, silt and minor gravel, these soils were laid down in the shoreline and near shoreline areas of former glacial Lakes Algonquin and Nipissing.

The total overburden thickness 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 together with the results of the field and laboratory testing are shown on the Record of Borehole sheets, Drawing 1 and Appendix A attached to this report. The stratigraphic boundaries shown on the borehole sheets and profile 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.

In summary, the subsoils at the site generally consist of surficial topsoil and fill between 1.2 and 1.4 metres thick underlain by compact to very loose sand which is then underlain by an extensive deposit of cohesive strata consisting mainly of firm to stiff clayey silt and silty clay. The lower part of the silty clay is interlayered with seams of sand and gravel or silt below approximately elevation 137 metres. Both boreholes were terminated in very dense glacial till between elevations 126 and 128 metres.

The locations of the boreholes are shown on Drawing 1. The interpreted stratigraphical profile is also shown on Drawing 1. 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.

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

4.2.1 Topsoil

A surficial layer of sandy topsoil was encountered at the ground surface of each borehole. The topsoil layers were approximately 0.4 metres thick.

4.2.2 Fill

Fill materials consisting of fine to medium sand underlay the topsoil layers at each borehole. The fill layers were between 0.8 and 1.0 metres thick and extended to about elevation 180.1 metres. The fill had standard penetration test N values of 9 and 21 blows per 0.3 metres of penetration. The water contents of the two fill samples collected from the boreholes averaged 7 per cent.

4.2.3 Sands

A stratum of fine uniform sand between 3.7 and 4.6 metres thick was encountered under the fill, at elevations 179.9 to 180.4 metres in the boreholes. N values from standard penetration tests conducted in the sand ranged from 1 to 26 blows per 0.3 metres. The values in the upper 3 metres were 9 blows per 0.3 metres or above indicating a generally compact density and loose to very loose below that depth. The water contents of the retrieved sand samples varied between 24 and 30 per cent with an average of about 24 per cent. Figure A-1 in Appendix A shows gradation curves for two samples recovered from the sand deposit. The deposit consists mainly of fine sand with some silt and trace of clay.

4.2.4 Peat

A 0.15 metre thick layer of fibrous peat was encountered beneath the sand and a 0.16 metre thick layer of clayey silt in borehole 1 at elevation 175.2 metres. The peat has a standard penetration test N value of 4 blows per 0.3 metres with a water content of 110 per cent.

4.2.5 Clayey Silt

A layer of clayey silt was encountered at elevation 175.4 metres between the sand and peat layers in borehole 1. More extensive clayey silt deposits, some 11.2 to 17.7 metres thick, were intercepted at elevation 175.0 metres below the peat layer in borehole 1 and at elevation 176.7 metres below the sand layer at borehole 2. An approximately 0.9 metre thick layer was encountered at elevation 132.8 metres above the sand and gravel layer the base of borehole 2.

The upper clayey silt layer in borehole 1 had a measured N value of 4 blows per 0.3 metres indicating a firm consistency. The clayey silt had a water content of 23 per cent.

The N values from standard penetration testing in the lower clayey silt deposits in boreholes 1 and 2 were between 4 and 32 blows per 0.3 metres with an average of 13 blows per 0.3 metres. In situ shear strength testing conducted in boreholes 1 and 2 indicated shear strengths of 80 kilopascals or greater. Remoulded shear strengths gave sensitivity values between 1.3 and 4.5 corresponding to insensitive to sensitive.

Figure A-2 in Appendix A shows the gradation curves for three samples retrieved from the clayey silt deposits. The deposits consist mainly of silt and clay size material with some sand and a trace of fine gravel. Water contents in the lower clayey silt ranged from 12 to 26 per cent with an average of 19 per cent. The average plastic and liquid limits for the clayey silt, based on the five samples tested, are 15 and 31 per cent respectively, with an average plasticity index of 16. The results are plotted on the plasticity chart, Figure A-3, which generally shows the deposit to be an inorganic clay of low plasticity.

The results of consolidation testing carried out on sample 13 from borehole 2 are provided on Figure A-4 in Appendix A. The results indicate that the clayey silt is slightly overconsolidated by about 10 kilopascals beyond the existing overburden pressure.

4.2.6 Silty Clay

Extensive deposits of silty clay were encountered in each borehole below the clayey silt layers between elevations 163.9 and 159.0 metres. The silty clay layers were between 26.2 and 31.9 metres thick. Figure A-5 in Appendix A shows the gradation curves for two samples of silty clay retrieved from boreholes 1 and 2.

Standard penetration test N values within these deposits ranged from 4 to 22 blows per 0.3 metres with an average of 9 blows per 0.3 metres indicating a firm to very stiff but generally stiff consistency. Water contents determined from selected samples varied between 27 and 33 per cent with an average of 30 per cent. The average plastic and liquid limits for the silty clay, based on the three samples tested, are 21 and 43 per cent respectively, with an average plasticity index of 23. The results are plotted on the plasticity chart, Figure A-6, which generally shows the deposit to be an inorganic clay of intermediate plasticity.

4.2.7 Silt

At elevation 131.9 metres, a 0.9 metre thick layer of silt underlays the silty clay layer at borehole 1. The measured N value in this layer was 25 blows per 0.3 metres with a corresponding water content of 26 per cent.

4.2.8 Sand and Gravel

A layer of sand and gravel approximately 0.6 metres thick was intercepted at elevation 131.9 metres in borehole 2 between an approximately 0.9 metre thick clayey silt layer and an underlying sandy silt till deposit. A single N value exceeding 100 blows per 0.3 metres and a water content of 7 per cent were measured in this deposit.

4.2.9 Sandy Silt Till

Boreholes 1 and 2 were terminated in a deposit of sandy silt till after exploring it for some 4.8 and 3.2 metres to elevation 126.1 metres and 128.1 metres respectively. Shale fragments were noted in the till which had recorded N values over 100 blows per 0.3 metres. The water contents measured in seven samples of sandy silt till were between 7 and 26 per cent and averaged 12 per cent.

Figure A-7 in Appendix A shows the gradation curve for a sample of sandy silt till retrieved from a standard penetration test in borehole 1.

4.2.10 Bedrock

The available geological information indicated that the anticipated depth to bedrock in the general area of the site was 30 to 40 metres below the ground surface. Boreholes 1 and 2, which were advanced to depths of 55.2 and 53.5 or elevation 126.1 and 128.1 metres, respectively, without encountering bedrock, may be located within a trough in the bedrock. It should be noted that shale fragments were observed in the sandy silt till near the bottom of both boreholes.

4.2.11 Groundwater Conditions

Water levels were noted in the boreholes during and upon completion of the drilling operations. Groundwater was encountered at elevations 179.5 and 178.7 metres at boreholes 1 and 2, respectively, as indicated on the Record of Borehole sheets. However, based on water content and colour changes within the sand layers, the inferred long-term groundwater level is expected to fluctuate seasonally between 178 and 180 metres.

The following table shows the groundwater observations:

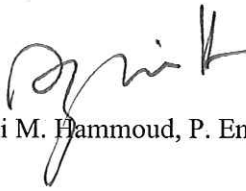
BOREHOLE NUMBER	BOREHOLE LOCATION		GROUND SURFACE ELEVATION (m)	ENCOUNTERED WATER LEVEL ELEVATION (m)
	NORTHING (m)	EASTING (m)		
1	4760934.8	313086.3	181.29	178.70
2	4760884.5	313100.0	181.59	179.46


5.0 MISCELLANEOUS


The investigation was carried out using equipment supplied and operated by Aardvark Drilling (Aardvark). Aardvark is an Ontario Ministry of Environment licensed well contractor. Field operations were supervised by Mr. Mike Arthur under the direction of Mr. David J. Mitchell. All routine laboratory testing was conducted at Golder's London laboratory. The consolidation testing was conducted at Golder's 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.

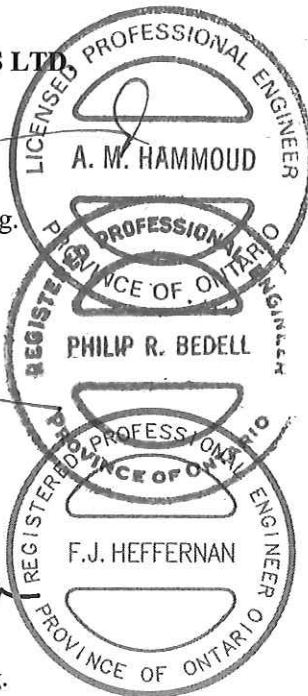
This report was written by Mr. Azmi M. Hammoud, P. Eng., an Associate with Golder under the direction of the Project Manager, Mr. Philip R. Bedell, P. Eng., a Principal with Golder. The report was reviewed by Mr. Fintan J. Heffernan, P. Eng., the Designated MTO Contact and Quality Control Auditor.

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PART B

**PRELIMINARY FOUNDATION DESIGN REPORT
PROPOSED CHRISTINA ST. UNDERPASS REPLACEMENT
HIGHWAY 402, GWP 3038-03-00
AGREEMENT NUMBER 3005-A-000394**

6.0 ENGINEERING RECOMMENDATIONS

6.1 General

This section of the draft report provides our recommendations on the foundation aspects of the preliminary design of the proposed Christina Street Underpass replacement. 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 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 widening and/or replacement of existing bridges associated with Highway 402 in the area. The existing Christina Street underpass structure will be demolished and replaced by a multi-span structure. The new structure will consist of two spans, a 24 metre long south span and a 35 metre long north span. Only minor profile grade revisions are proposed for Christina Street; however, Highway 402 is to be lowered by 1.5 to 1.8 metres. A 1.3 metre centreline shift to the west is proposed for Christina Street.

6.2 Bridge Foundations

The subsoils encountered in the boreholes put down during this investigation typically consist of surficial topsoil and fill overlying a surficial layer of compact to very loose sand which in turn overlies extensive deposits of firm to very stiff but generally stiff clayey silt and silty clay to about elevation 132 metres. A thin layer of peat was encountered near elevation 175 metres in borehole 1 only. Deposits of very dense sand and gravel and sandy silt till were present below elevation 132 metres to the termination depth of the boreholes. Shale fragments were noted in the lower portion of the till deposit. A groundwater level of about elevation 180 metres, or some 2.6 to 3.6 metres below ground surface, was encountered during drilling.

Various foundation options are presented. The options were evaluated for geotechnical feasibility, relative costs and risks and consequences. A comparison of the foundation alternatives is presented in Table I.

Design drawings for the existing Christina Street Underpass indicate that the existing structure is founded on spread footings located in the surficial sand near elevation 179.8 metres.

6.3 Shallow Foundations

The new bridge structure pier and abutments might be supported by spread footings founded in the native sand layers. However, it should be noted that a relatively thin layer of peat was encountered below the sand near elevation 175 metres at borehole 1 and only low bearing resistances are available in the sand. If the new structure is to be founded on spread footings, drilling of several additional boreholes during detail design is required to further delineate the extent of the peat deposits.

6.3.1 Axial Geotechnical Resistance

Based on the results of this investigation, spread footings may be constructed at about elevation 180 metres on the compact sand. Factored geotechnical design values of 150 kilopascals at Ultimate Limit States (ULS) and 100 kilopascals at Serviceability Limit States (SLS) for an assumed 4 metre wide footing can be used for preliminary design purposes. These values may not be sufficient for the support of the bridge structure. The stated values assume that the sand is properly dewatered and that the groundwater level is maintained a minimum of 1.0 metre below the base of the footing during construction. The geotechnical resistances also assume that appropriate construction procedures are adopted during footing construction to ensure that the excavations are properly dewatered and the founding soils are not loosened/disturbed prior to concrete placement.

The settlement(s) of these footings will be dependent on the actual footing size, configuration, and applied loads. Additional settlement of the footings will occur due to consolidation of the founding soils under the minor embankment modifications. However, modifications to the existing approach embankments could be constructed well in advance to reduce the footing settlements. In addition to the suggested additional investigation to delineate the extent of the peat layer(s), settlements should be confirmed at the detail design stage, once the footing size, configuration and loads are known, to assess whether the spread footing option is feasible. Additional field and laboratory testing should be carried out to determine the compressibility characteristics of the subsoils to refine the settlement predictions.

Alternatively, perched abutments on compacted Granular A constructed within the approach embankment fill may be designed for a factored geotechnical resistance at ULS of 450 kPa and a SLS value of 300 kPa. Although the abutments can be perched on Granular A pads, it is preferred that the pier be founded on piles.

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 loosened/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 sand	angle of friction	32°
	$\tan \delta$	0.62

6.3.3 Frost Protection

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

6.3.4 Construction Considerations

The founding soils are sensitive to disturbance and loosening due to water seepage and/or ponding. Placement of a mud coat will be required at the base of the excavation for the footing area. Exposure without protection of the mud coat will result in loosening of the founding soils. The cleaned excavation base should be inspected by qualified geotechnical personnel prior to placing the mud coat. It is recommended that the footing excavation be carried out such that the final 0.5 metres of excavation is completed with the geotechnical personnel on site and the mud coat be placed immediately after the footing is inspected.

6.4 Deep Foundations

End bearing piles driven to practical refusal on the very dense sandy silt till below elevation 132 metres or friction piles embedded in very stiff silty clay are considered suitable options to support the abutments and piers for the proposed widening. Steel H-piles and open pipe piles are considered to be viable options as they will more readily penetrate the clayey deposits and minimize the amount of disturbance given their shape and small cross-sectional area. In the case of a design incorporating integral abutments, augering and placement of a corrugated steel pipe (CSP) liner around the upper 3 metres of the pile is required.

6.4.1 Geotechnical Axial Resistance – Driven Steel Piles (End Bearing)

For preliminary design, the factored axial geotechnical resistance at ULS for HP 310 x 110 piles driven into the very dense sandy silt till to about elevation 130.7 metres may be taken as 1,800 kilonewtons (kN). The geotechnical resistance at SLS maybe taken as 1,300 kN. This value takes into account the structural capacity limitation of the pile and potential difficulties that the

pile may have seating into the till. The flanges of HP sections should be equipped with flange reinforcement as per current MTO practice (Ontario Provincial Standard Drawing (OPSD) 3301.00). This is the preferred foundation option for this bridge.

Provision should be made to re-tap the piles to confirm the set after adjacent piles have been driven in accordance with Special Provision 903S01.

6.4.2 Geotechnical Axial Resistance – Driven Steel Piles (Friction)

For preliminary design, the factored axial geotechnical resistance at ULS for 324 millimetre outside diameter steel tube piles driven into the stiff silty clay deposit to about elevation 154 metres may be taken as 350 kN. The geotechnical resistance at SLS maybe taken as 225 kN. The capacity of the pile groups will need to be checked during detail design.

Provision should be made to re-tap the piles to confirm the set after adjacent piles have been driven in accordance with Special Provision 903S01.

6.4.3 Downdrag Load (Negative Skin Friction)

As noted previously, only minor modification to the cross-section of the approach embankments is required and significant amounts of existing embankment are to be removed. As a result, no negative skin friction or downdrag loads will need to be taken into account during design of the piles supporting the abutments or pier.

6.4.4 Resistance to Lateral Loads

The lateral loading could be resisted fully or partially by the use of battered piles. If vertical piles are used, the resistance to lateral loading will have to be derived from the soil in front of the piles.

The abutment piles will be driven through embankment fill and the underlying granular and cohesive soils and the pier piles will be driven through the cohesive deposits. The resistance to lateral loading may be based on the following assessed values:

SOIL TYPE	HORIZONTAL RESISTANCE VALUES (kN) PER PILE	
	Factored ULS	SLS
Embankment fill (granulars)	110	40
Embankment fill (cohesive)	160	65
Clayey silt and silty clay deposits	90	35

Group action for lateral loading should be considered when the pile spacing in the direction of the loading is less than six to eight pile diameters. Group action can be evaluated by reducing the coefficient of horizontal 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

6.4.5 Frost Protection

The pile caps should be provided with a minimum of 1.2 metres of soil cover for frost protection.

6.5 Lateral Earth Pressures

The lateral pressures acting on the bridge abutment additions and associated retaining walls will depend on the type and method of placement of the backfill materials, on the nature of the soils behind the backfill, on the freedom of lateral movement of the structure, and on the drainage conditions behind the walls. The following recommendations are made concerning the design of the abutments, in accordance with the CHBDC:

- Select, free-draining granular fill meeting the specifications of OPSS Granular A or Granular B but with less than 5 per cent passing the 200 sieve should be used as backfill behind the abutments and walls. Longitudinal drains and weep holes should be installed to provide positive drainage of the granular backfill. Other aspects of the abutment granular backfill requirements with respect to subdrains and frost taper should be in accordance with OPSD 3501.00 and 3504.00.
- A compaction surcharge equal to 12 kilopascals should be included in the lateral earth pressures for the structural design of the abutment wall, in accordance with CHBDC Figure 6.9.3. Compaction equipment should be used in accordance with OPSS 501.06.
- The granular fill may be placed either in a zone with a width equal to at least 1.5 metres behind the back of the stem (Case i from Commentary on CHBDC Figure C6.9.1(l) or within the wedge-shaped zone defined by a line drawn at 1.5 horizontal to 1 vertical extending up and back from the rear face of the footing (Case ii from Commentary on CHBDC Figure C6.9.1(l)).

- For Case i, the pressures are based on the proposed embankment fill materials and the following parameters (unfactored) may be assumed for granular fill:

Soil unit weight: 21 kN/m³

Coefficients of lateral earth pressure:

Active, K_a 0.33
At rest, K_o 0.50

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

	<u>GRANULAR A</u>	<u>GRANULAR B</u>
Soil unit weight:	22 kN/m ³	21 kN/m ³
Coefficients of lateral earth pressure:		
Active, K_a	0.27	0.31
At rest, K_o	0.43	0.47

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

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.

6.6 Embankments

It is understood that, with the increased bridge length, the existing embankment fills will be shortened to accommodate the proposed ETL. 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 the existing embankments constructed with suitable native or borrow materials. If the new fill slopes exceed 8 metres in height, a 2 metre wide mid-height bench should be provided.

Only minor, localized embankment settlements are expected as a result of the minor centreline shift for Christina Street.

6.7 Excavations and Temporary Cut Slopes

Excavations for pile cap construction and/or spread footings will extend through existing fill and sand deposits and will encounter the clayey silt crust. Based on the subsurface conditions encountered in the boreholes, excavations below 180 metres will require dewatering. Temporary open cut slopes should be maintained no steeper than 1 horizontal to 1 vertical.

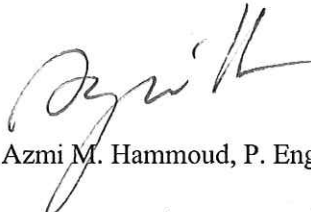
Surficial water seepage into the excavations should be expected, and will be heavier during periods of sustained precipitation. Pumping from well filtered sumps located at the base of the excavations may be required to provide groundwater control during foundation excavations. Sumps should be maintained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

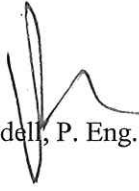
All excavations should be carried out in accordance with the guidelines outlined in the latest edition of the Ontario Occupational Health and Safety Act and Regulations For Construction Projects. The surficial topsoil, fill, peat, and native sands below the groundwater table at this site would be classified as Type 3 soils. Properly dewatered sands and the underlying cohesive deposits would be classified as Type 2. Roadway protection should conform to Performance Level 2, SP No. 539S01.


7.0 CLOSURE

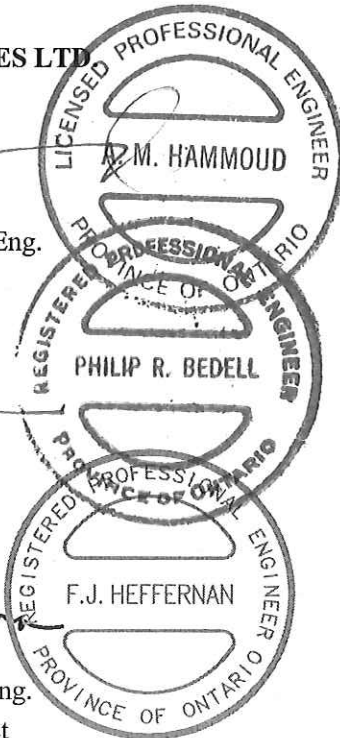
This report was written by Mr. Azmi M. Hammoud, P. Eng., an Associate with Golder Associates under the direction of the Project Manager, Mr. Philip R. Bedell, P.Eng., a Principal with Golder Associates. The report was reviewed by Mr. Fintan J. Heffernan, P.Eng., the Designated MTO Contact and Quality Control Auditor.

GOLDER ASSOCIATES LTD


Azmi M. Hammoud, P. Eng.


Philip R. Bedell, P. Eng.
Principal


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



October 2005

041-130099-2

TABLE I
COMPARISON OF FOUNDATION ALTERNATIVES
CHRISTINA STREET OVERPASS REPLACEMENT
GWP 3038-03-00

Foundation Option	Feasibility	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Spread footings supported on native sand	<ul style="list-style-type: none"> Not considered feasible due to low bearing resistance and presence of peat at depth 	<ul style="list-style-type: none"> Cheapest option 	<ul style="list-style-type: none"> Time and cost of settlement mitigation measures Even if mitigation adopted, settlement of shallow foundations could still take place 	<ul style="list-style-type: none"> \$225,000 Less expensive than deep foundation options 	<ul style="list-style-type: none"> Even if mitigation in place, shallow foundations will still be affected by settlement of silty clay and sporadic peat deposits Loosening of subsoils if groundwater level is high at the time of footing construction
Steel H-pile foundations founded in very dense sandy silt till	<ul style="list-style-type: none"> Feasible for support of all foundation elements 	<ul style="list-style-type: none"> High bearing resistance Negligible settlement 	<ul style="list-style-type: none"> Possibility of encountering cobbles or boulders while pile driving in till 	<ul style="list-style-type: none"> \$480,000 More expensive than shallow foundations but preferred technical solution 	<ul style="list-style-type: none"> Possible pile tip damage if cobbles and boulders encountered in till, reinforcement of pile tips required
Steel pipe friction piles	<ul style="list-style-type: none"> Feasible for support of all foundation elements 	<ul style="list-style-type: none"> N/A 	<ul style="list-style-type: none"> Low bearing resistance Some settlement under design loading 	<ul style="list-style-type: none"> \$600,000 More expensive than steel H-pile option 	<ul style="list-style-type: none"> Close pile spacing, with reduction for pile groups

NOTE: Costs are very preliminary estimates and are intended to provide a comparison between alternatives rather than actual construction costs.

LIST OF ABBREVIATIONS

The abbreviations commonly employed on each "Record of Borehole", on the figures and in the text of the report, are as follows:

I. SAMPLE TYPES

<i>AS</i>	auger sample
<i>CS</i>	chunk sample
<i>DO</i>	drive open
<i>DS</i>	Denison type sample
<i>FS</i>	foil sample
<i>RC</i>	rock core
<i>ST</i>	slotted tube
<i>TO</i>	thin-walled, open
<i>TP</i>	thin-walled, piston
<i>WS</i>	wash sample
<i>SS</i>	split spoon

II. PENETRATION RESISTANCES

Dynamic Penetration Resistance:

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 0.3 m (12 in.).

Standard Penetration Resistance, 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 0.3 m (12 in.).

<i>WH</i>	sampler advanced by static weight-weight, hammer
<i>PH</i>	sampler advanced by hydraulic force
<i>PM</i>	sampler advanced by manual force

III. SOIL DESCRIPTION

(a) Cohesionless Soils

Relative Density	"N" Blows/0.3 m or Blow/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	"Cu" = "Su" kPa	psf.
Very soft	0 to 12	0 to 250
Soft	12 to 25	250 to 500
Firm	25 to 50	500 to 1000
Stiff	50 to 100	1000 to 2000
Very stiff	100 to 200	2000 to 4000
Hard	over 200	over 4000

IV. SOIL TESTS

<i>C</i>	consolidation test
<i>H</i>	hydrometer analysis
<i>M</i>	sieve analysis
<i>MH</i>	combined analysis, sieve and hydrometer ¹
<i>Q</i>	undrained triaxial ²
<i>R</i>	consolidated undrained triaxial ²
<i>S</i>	drained triaxial
<i>U</i>	unconfined compression
<i>V</i>	field vane test
<i>Chem</i>	chemical analysis

NOTES:

1. Combined analyses when 5 to 95 per cent of the material passes the No. 200 sieve.
2. Undrained triaxial tests in which pore pressures are measured are shown as Q or R.

LIST OF SYMBOLS

I. GENERAL

π	= 3.1416
e	= base of natural logarithms 2.7183
$\log_e a$ or $\ln a$	natural logarithm of a
$\log_{10} a$ or $\log a$	logarithm of a to base 10
t	time
g	acceleration due to gravity
V	volume
W	weight
m	mass
M	moment
F	factor of safety

II. STRESS AND STRAIN

u	pore pressure
σ	normal stress
σ'	normal effective stress (σ is also used)
τ	shear stress
ϵ	linear strain
ϵ_{sy}	shear strain
ν	Poisson's ration (μ is also used)
E	modulus of linear deformation (Young's modulus)
G	modulus of shear deformation
K	modulus of compressibility
η	coefficient of viscosity

III. SOIL PROPERTIES

(a) Unit weight

γ	unit weight of soil (bulk density)
γ_s	unit weight of solid particles
γ_w	unit weight of water
γ_d	unit dry weight of soil (dry density)
γ'	unit weight of submerged soil
G_s	specific gravity of solid particles $G_s = \gamma_s/\gamma_w$
e	void ratio
n	porosity
w	water content
S_r	degree of saturation

(b) Consistency

w_L	liquid limit
w_P	plastic limit
I_P	plasticity index
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
D_r	relative density = $(e_{max} - e)/(e_{max} - e_{min})$

(c) Permeability

h	hydraulic head or potential
q	rate of discharge
v	velocity of flow
i	hydraulic gradient
κ	coefficient of permeability
j	seepage force per unit volume

(d) Consolidation (one-dimensional)

m_v	coefficient of volume change = $-\Delta e/(1+e)\Delta\sigma'$
C_c	compression index = $-\Delta e/\Delta\log_{10}\sigma'$
c_v	coefficient of consolidation
T_F	time factor = $c_v t/d^2$ (d , drainage path)
U	degree of consolidation

(e) Shear strength

τ_f	shear strength	in terms of effective stress $\tau_f = c' + \sigma' \tan \phi$
c'	effective cohesion intercept	
ϕ'	effective angle of shearing resistance, or friction	
S_u	apparent cohesion*	
ϕ_u	apparent angle of shearing resistance, or friction	in terms of total stress $\tau_f = c_u + \sigma \tan \phi_u$
μ	coefficient of friction	
S_t	sensitivity	

*For the case of a saturated cohesive soil, $\phi_u = 0$ and the undrained shear strength $\tau_f = S_u$ is taken as half the undrained compressive strength.

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

+³, ×³: Numbers refer to Sensitivity ○^{3%} STRAIN AT FAILURE

PROJECT 041-130099-2 RECORD OF BOREHOLE No **BH1** 3 OF 4 **METRIC**
 G.W.P. 3038-03-00 LOCATION N 4760934.8 ; E 313086.3 ORIGINATED BY MA
 DIST 1 HWY 402 BOREHOLE TYPE POWER AUGER/HOLLOW STEM & MUD ROTARY COMPILED BY BG
 DATUM GEODETIC DATE July 12, 2004 - July 15, 2004 CHECKED BY [Signature]

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	
								SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL X LAB VANE	WATER CONTENT (%)			GR SA SI CL
	SILTY CLAY, trace sand, trace gravel, Firm to stiff, Grey		25	SS	7		151					
							150					
			26	SS	6		149					5 53 42
							148					
			27	SS	7		147					
							146					
			28	SS	7		145					
							144					
			29	SS	7		143					
							142					
			30	SS	9		141					
							140					
			31	SS	9		139					
							138					
			32	SS	11		137					
			33	SS	10							
138.01 43.28	SILTY CLAY, trace sand, Firm to stiff, Grey		34	SS	10							

ON_MTO 041-130099-2.GPJ ON_MOT.GDT 10/12/05

Continued Next Page

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

RECORD OF BOREHOLE No BH1

4 OF 4

METRIC

PROJECT 041-130099-2

G.W.P. 3038-03-00

LOCATION N 4760934.8 E 313086.3

ORIGINATED BY MA

DIST 1 HWY 402

BOREHOLE TYPE POWER AUGER/HOLLOW STEM & MUD ROTARY

COMPILED BY BG

DATUM GEODETIC

DATE July 12, 2004 - July 15, 2004

CHECKED BY *[Signature]*

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									WATER CONTENT (%)		
								20 40 60 80 100											
								20 40 60 80 100											

ON_MTO 041-130099-2.GPJ ON_MOT.GDT 10/12/05

PROJECT 041-130099-2

RECORD OF BOREHOLE No BH2

1 OF 4

METRIC

G.W.P. 3038-03-00

LOCATION N 4761884.5; E 313100.0

ORIGINATED BY MA

DIST 1 HWY 402

BOREHOLE TYPE POWER AUGER/HOLLOW STEM & MUD ROTARY/TRI-CONE

COMPILED BY BG

DATUM GEODETIC

DATE July 19, 2004 - July 20, 2004

CHECKED BY *[Signature]*

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa				
								○ UNCONFINED	+ FIELD VANE	● QUICK TRIAXIAL × LAB VANE		
181.59	GROUND SURFACE						20 40 60 80 100					
0.00	TOPSOIL, sandy, some gravel, Brown											
181.16												
0.43	FILL, fine to medium sand, some gravel, trace silt, Loose, Brown		1	SS	9		181					
180.37												
1.22	SAND, fine, trace to some silt layered Compact, Brown becoming grey at about elev. 178.0m		2	SS	10		180					
			3	SS	21		179					
			4	SS	26		178					
			5	SS	16		177					
			6	SS	12		176					
176.71												
4.88	CLAYEY SILT, trace to some sand, trace gravel, Firm to very stiff, Brown becoming grey at about elev. 173.4m		7	SS	20		175					
			8	SS	28		174					
			9	SS	32		173					
			10	SS	27		172					
							171					
			11	SS	10		170					
							169					
			12	SS	9		168					
							167					
			13	TO	PH							
			14	TO	PH							

Continued Next Page

+ 3, × 3: Numbers refer to Sensitivity

○ 3% STRAIN AT FAILURE

ON_MTO 041-130099-2.GPJ ON_MOT.GDT 10/12/05

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

PROJECT 041-130099-2

RECORD OF BOREHOLE No BH2

4 OF 4

METRIC

G.W.P. 3038-03-00

LOCATION N 4761884.5; E 313100.0

ORIGINATED BY MA

DIST 1 HWY 402

BOREHOLE TYPE POWER AUGER/HOLLOW STEM & MUD ROTARY/TRI-CONE

COMPILED BY BG

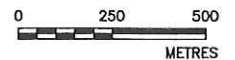
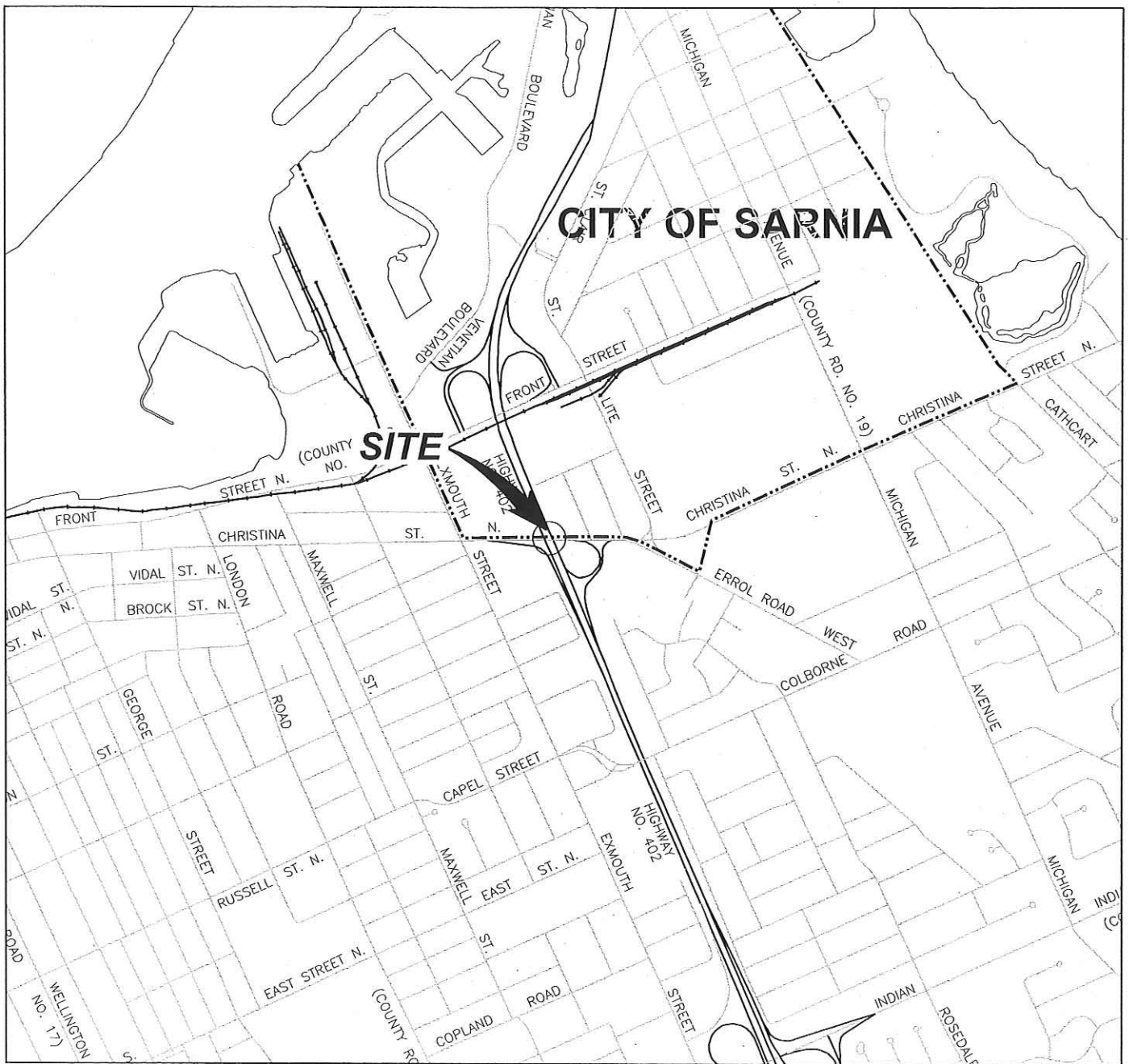
DATUM GEODETIC


DATE July 19, 2004 - July 20, 2004

CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa										WATER CONTENT (%)		
								○ UNCONFINED + FIELD VANE										○		
								● QUICK TRIAXIAL × LAB VANE												
							20	40	60	80	100									
132.82	SILTY CLAY, trace sand, trace gravel, Firm to very stiff, Grey		25	SS	9		136													
							135													
							134													
							133													
48.77	CLAYEY SILT, with silt seams Very stiff, Grey		26	SS	22		132													
131.91							131													
49.68	SAND AND GRAVEL, some silt Very dense, Grey		27	SS	250/ 150mm		130													
131.30							129													
50.29	SANDY SILT TILL, some clay, trace gravel, with shale fragments, Very dense, Grey		28	SS	109/ 230mm															
128.10			29	SS	120															
53.49	END OF BOREHOLE																			
	Groundwater encountered in borehole at elev. 179.9m during drilling July 19-20, 2004																			

ON_MTO 041-130099-2.GPJ ON_MOT.GDT 10/12/05



PROJECT			
CHRISTINA STREET UNDERPASS REPLACEMENT			
GWP No. 3038-03-00			
HWY. 402			
TITLE			
SITE LOCATION PLAN			
PROJECT No. 041-130099-2		FILE No. 041130099-2F001	
CADD	DCH	Oct. 12/05	SCALE AS SHOWN
CHECK	BY	Oct. 13/05	REV. 0
 Golder Associates LONDON, ONTARIO			FIGURE 1



CHRISTINA STREET
UNDERPASS REPLACEMENT
BOREHOLE LOCATIONS & SOIL STRATA

SHEET



Golder Associates Ltd.
LONDON, ONTARIO, CANADA



LEGEND

- Borehole
- Seal
- Piezometer
- N Blows/0.3m (Std. Pen. Test, 475 j/blow)
- WL in piezometer
- WL during drilling

No.	ELEVATION (metres)	CO-ORDINATES	
		NORTH	EAST
1	181.29	4 760 934.8	313 086.3
2	181.59	4 760 884.5	313 100.0

NOTES

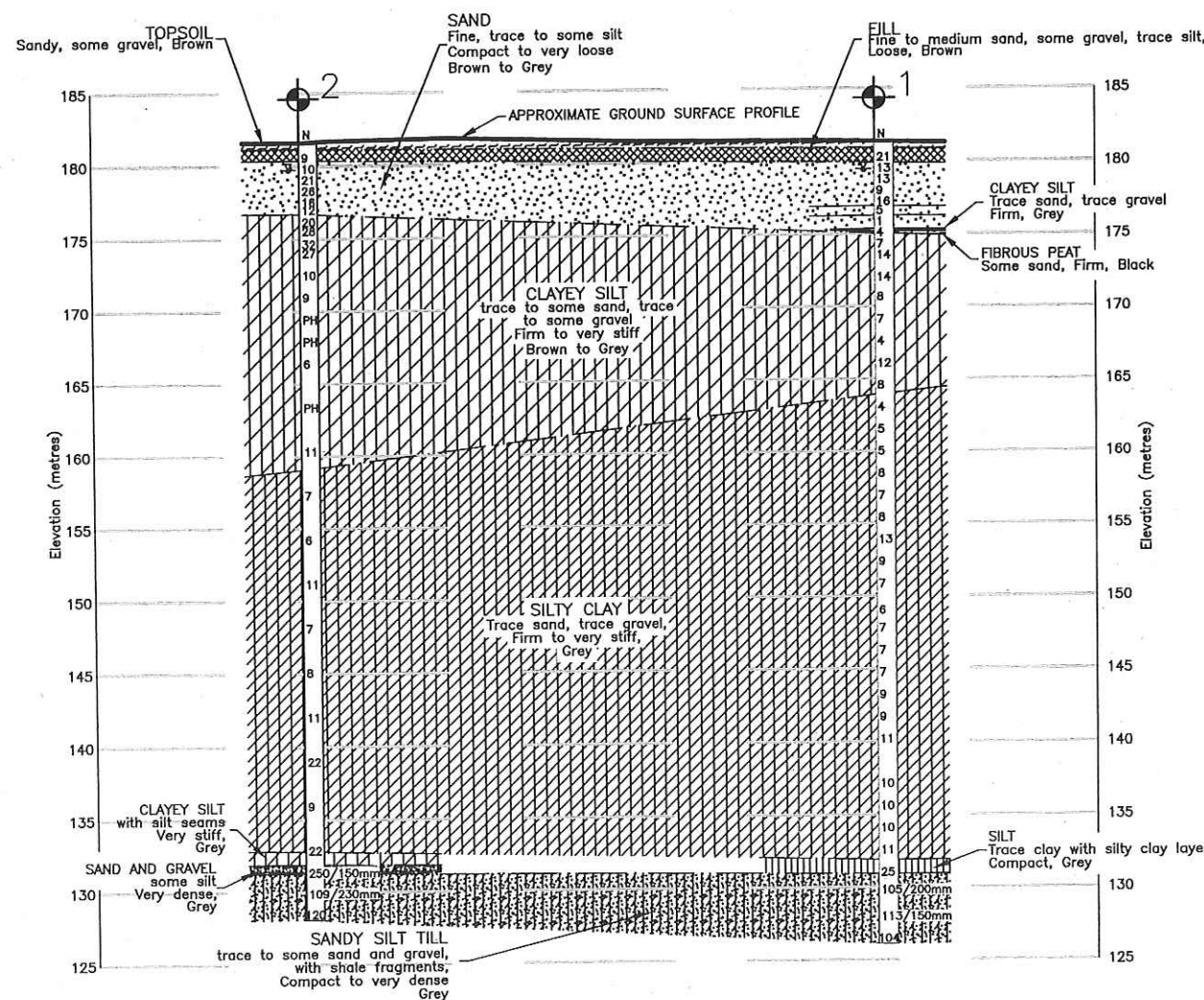
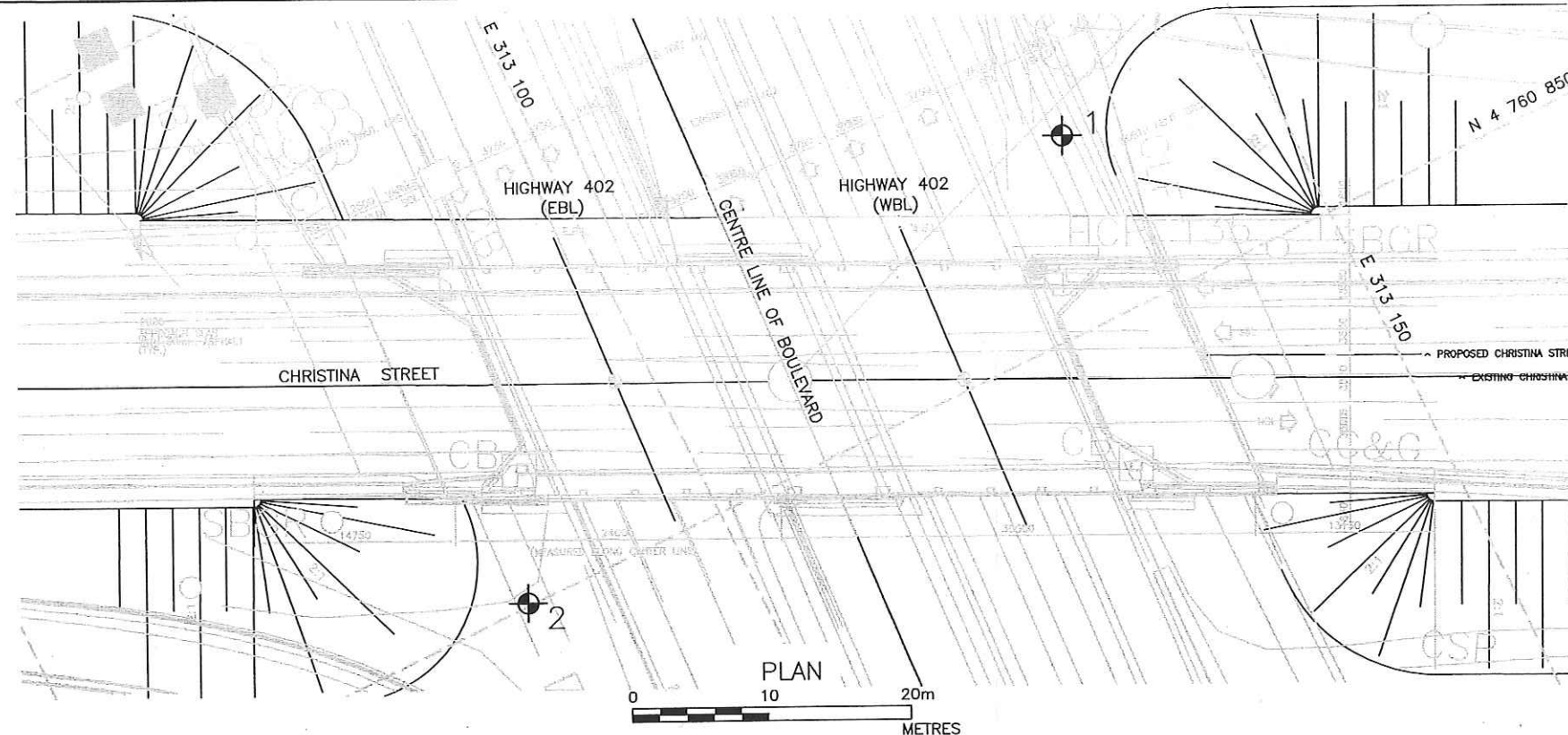
The boundaries between soil strata have been established only at Borehole locations. Between Boreholes the boundaries are assumed from geological evidence.

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

DRAWING SUPPLIED BY: URS CANADA INC. GENERAL ARRANGEMENT
ENTITLED: CHRISTINA STREET UNDERPASS,
SITE: 14-37
DATED: AUGUST 2005

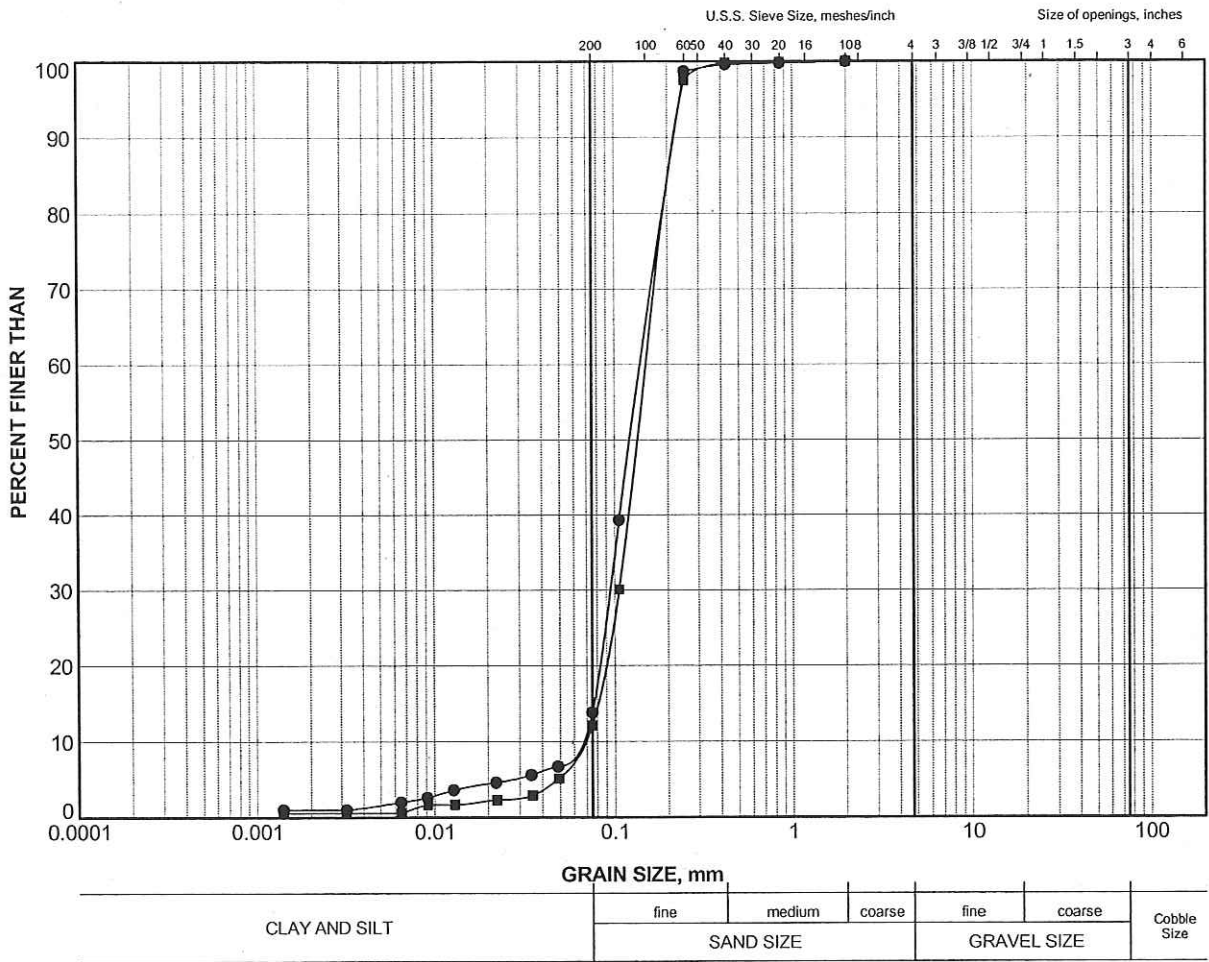
NO.	DATE	BY	REVISION
Geocres No.	40J16-65		
HWY. No.	402	PROJECT NO.:	041-130099-2
SUBM'D.		CHKD:	DATE: Aug 15/05
DRAWN: BG/DCH		CHKD:	APPD.
			DWG. 1






PROFILE ALONG CENTRELINE OF CHRISTINA STREET

APPENDIX A

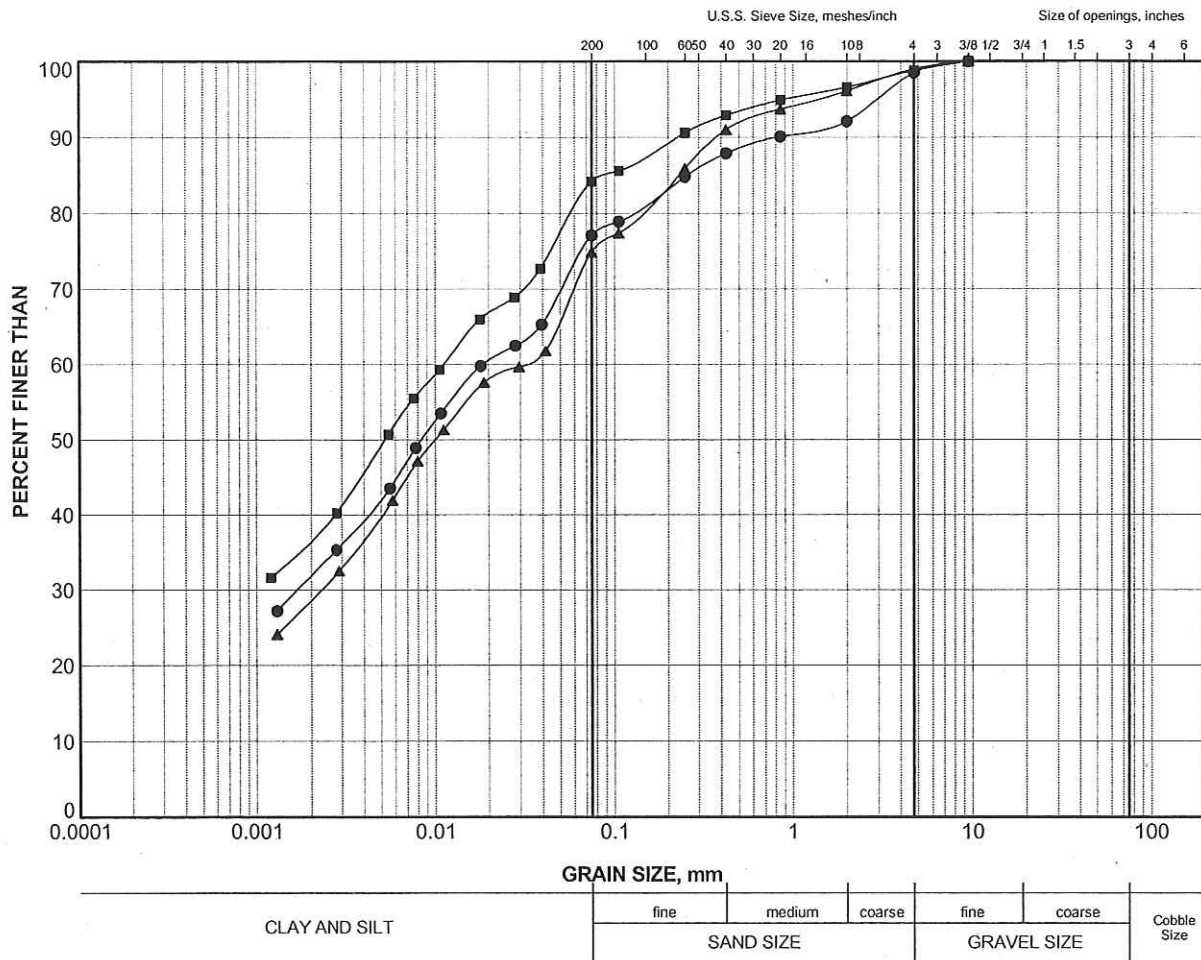
LABORATORY TEST RESULTS



LEGEND			
SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	BH1	4	178.0
■	BH2	2	179.8

PROJECT				CHRISTINA STREET UNDERPASS REPLACEMENT GWP 3038-03-00 HWY 402			
TITLE				GRAIN SIZE DISTRIBUTION FINE SAND			
PROJECT No.		041-130099		FILE No.		041-130099-2.GPJ	
DRAWN		WDF		SCALE		N/A	
CHECK		Sep 14/04		REV.			
 Golder Associates LONDON, ONTARIO		 		FIGURE A-1			

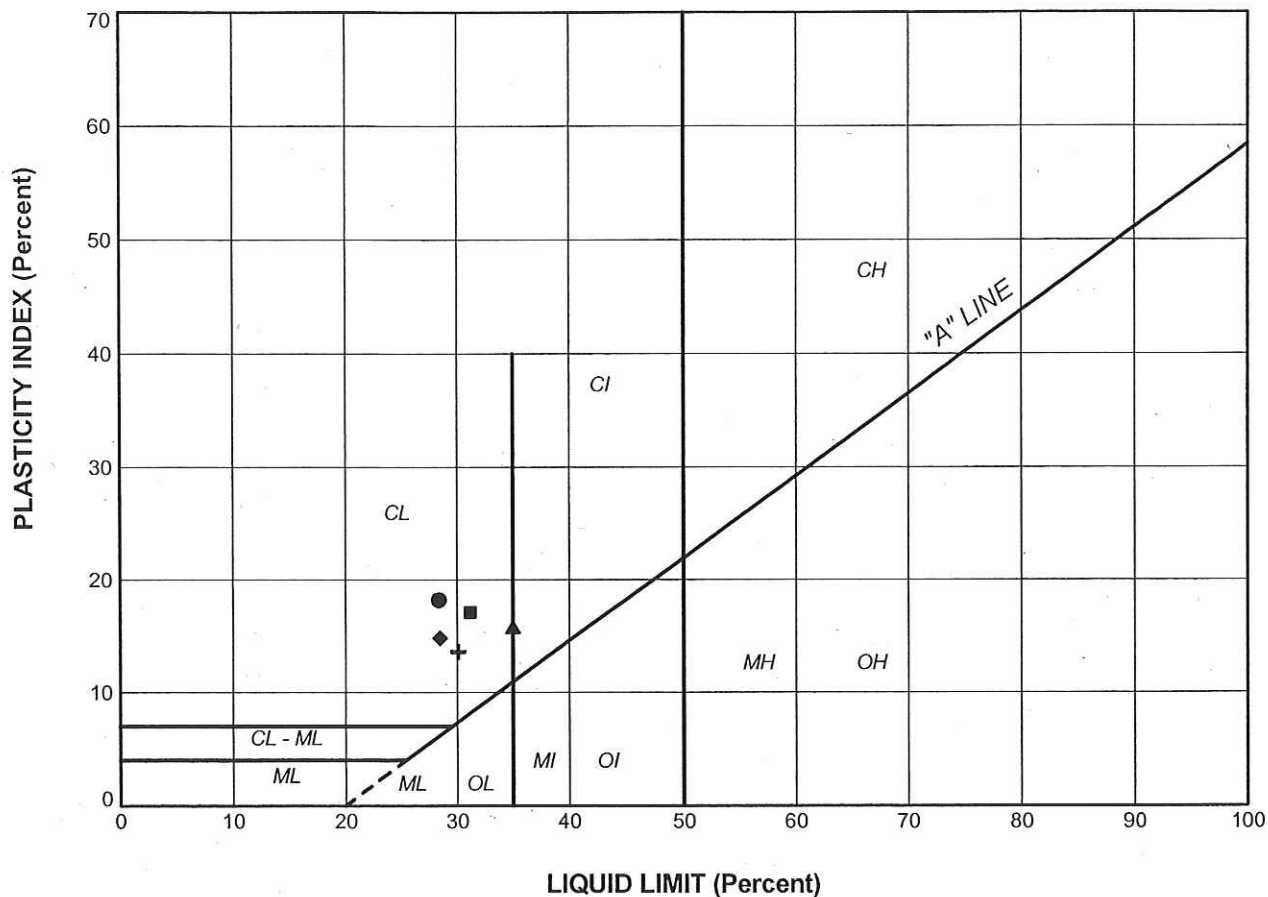
LDN_MTO_NEW GLDR_LDN.GDT




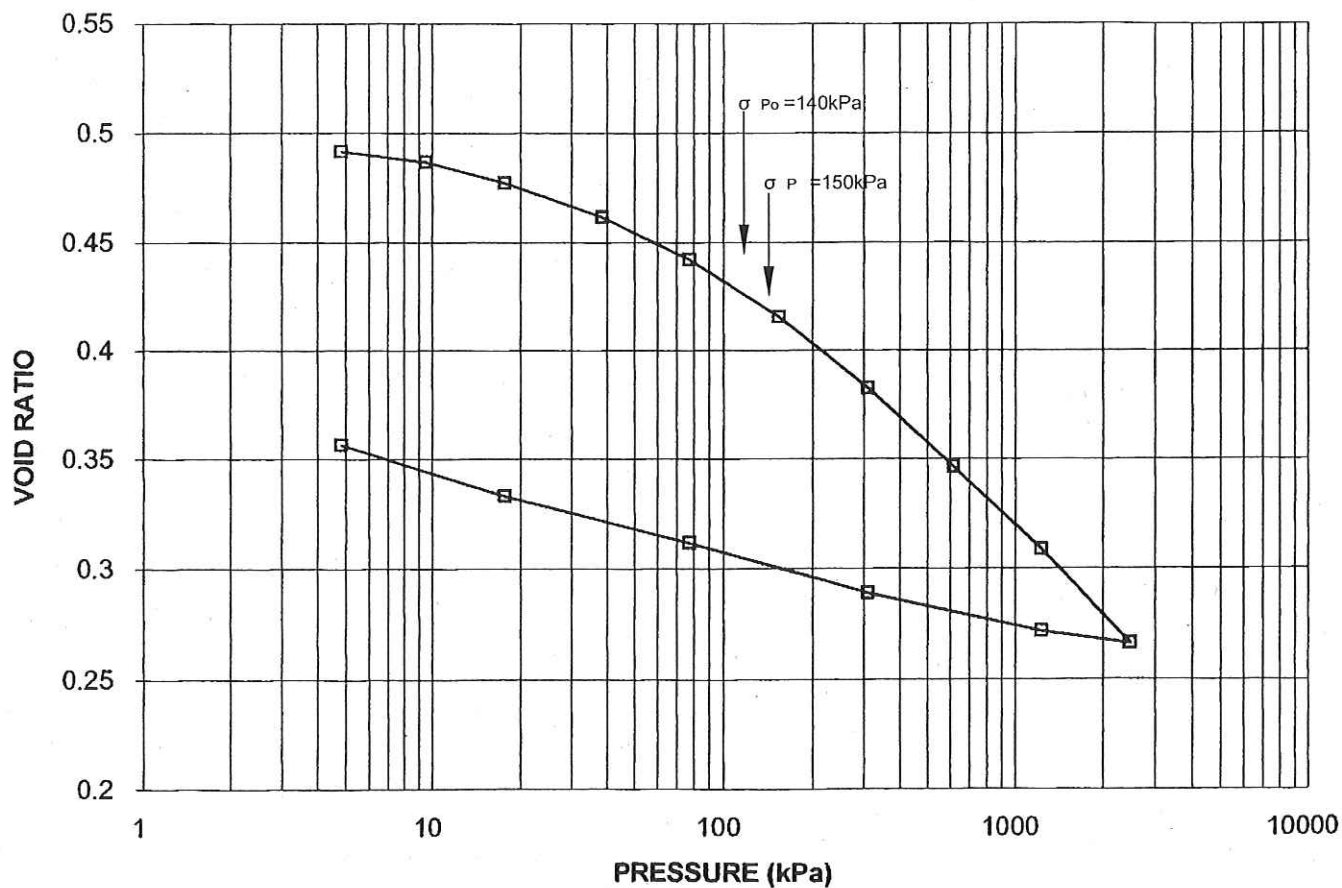
LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	BH1	11	172.0
■	BH1	14	167.6
▲	BH2	9	174.6

PROJECT					
CHRISTINA STREET UNDERPASS REPLACEMENT					
GWP 3038-03-00					
HWY 402					
TITLE					
GRAIN SIZE DISTRIBUTION					
CLAYEY SILT					
PROJECT No.		041-130099		FILE No. 041-130099-2.GPJ	
DRAWN		WDF		SCALE N/A	
CHECK		WDF		REV.	
		Sep 14/04			
		Oct 13/05			
				FIGURE A-2	



PROJECT				CHRISTINA STREET UNDERPASS REPLACEMENT GWP 3038-03-00 HWY 402			
TITLE							
PLASTICITY CHART (Clayey Silt)							
PROJECT No.		041-130099-2		FILE No.		041-130099-2.GPJ	
DRAWN		BG		SCALE		N/A	
CHECK		ep		REV.		13/05	
 Golder Associates LONDON, ONTARIO				FIGURE A-3			



BOREHOLE 2, SAMPLE 13, ELEV. 169.5m

NOTE

- 1) THIS DRAWING IS TO BE READ IN CONJUNCTION WITH ACCOMPANYING TEXT.

PROJECT
CHRISTINA STREET UNDERPASS REPLACEMENT
GWP 3038-03-00
HWY 402

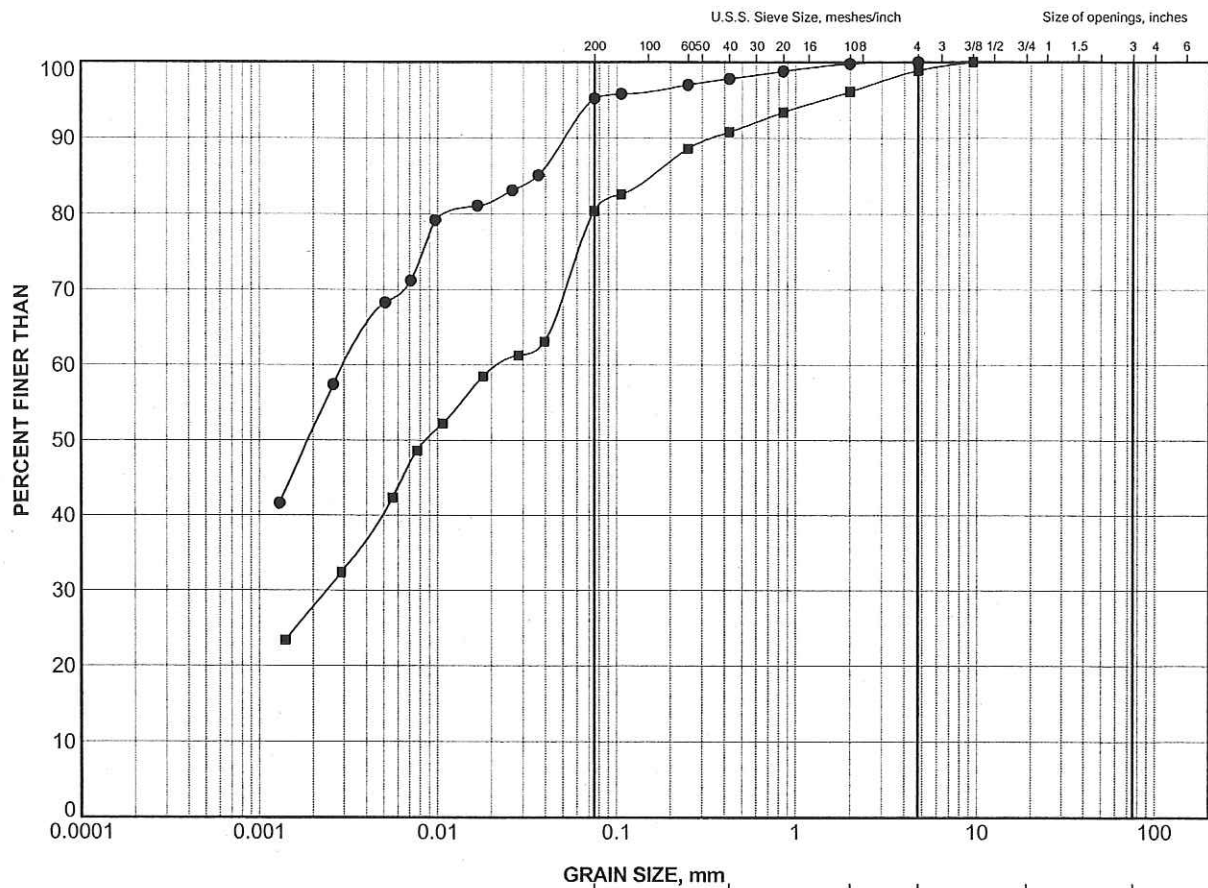
TITLE

CONSOLIDATION TEST VOID RATIO VS. LOG PRESSURE




PROJECT No. 041-130099-2			FILE No. 041130099-2D0A4	
CADD	WDF	Sept 10/04	SCALE	AS SHOWN
CHECK	<i>[Signature]</i>	<i>Oct. 13/04</i>	REV.	0

FIGURE A-4



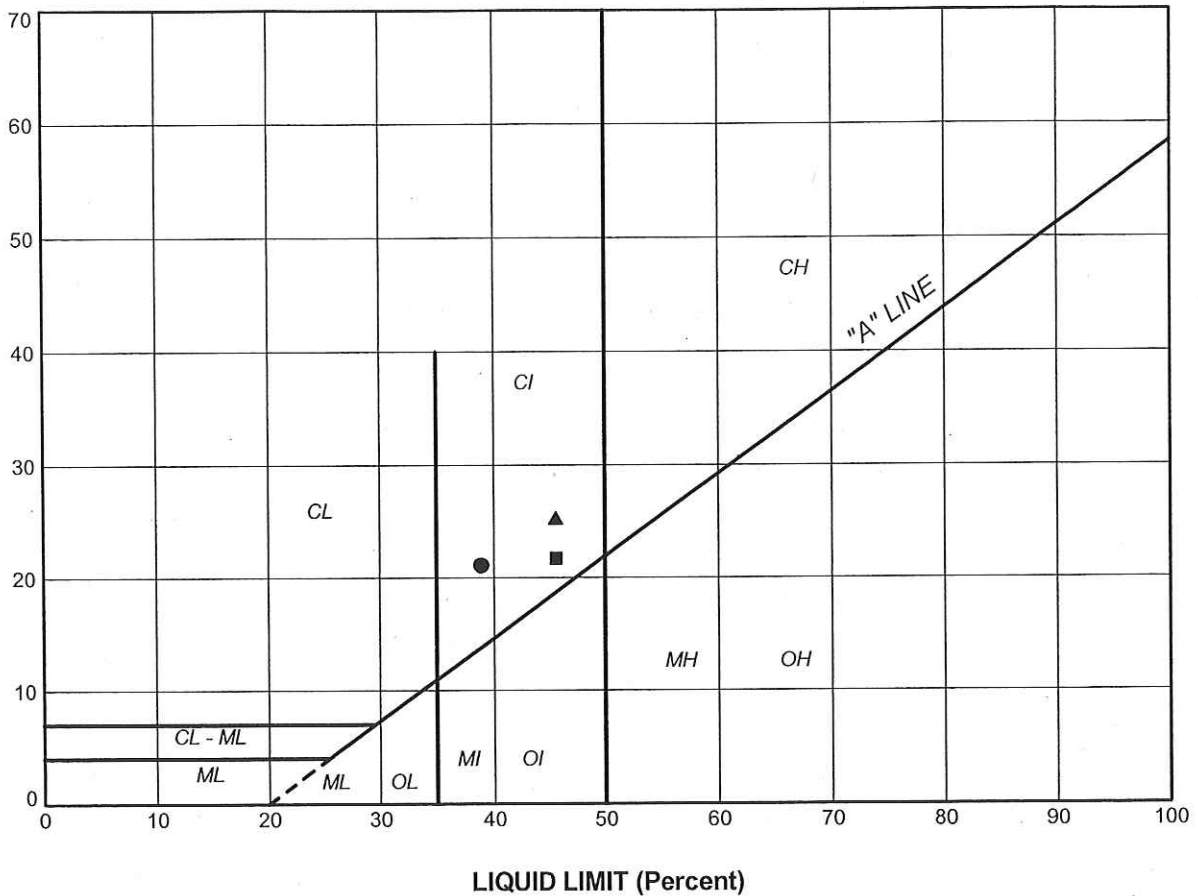
CLAY AND SILT	SAND SIZE, mm					Cobble Size
	fine	medium	coarse	fine	coarse	
	SAND SIZE			GRAVEL SIZE		

LEGEND			
SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	BH1	26	149.1
■	BH2	20	151.2

PROJECT				CHRISTINA STREET UNDERPASS REPLACEMENT GWP 3038-03-00 HWY 402			
TITLE				GRAIN SIZE DISTRIBUTION SILTY CLAY			
PROJECT No.		041-130099-2		FILE No.		041-130099-2.GPJ	
DRAWN		BG		SCALE		N/A	
CHECK		EP		REV.		16/04	
 Golder Associates LONDON, ONTARIO				FIGURE A-5			

LDN_MTO_NEW_GLDR_LDN.GDT

PLASTICITY INDEX (Percent)



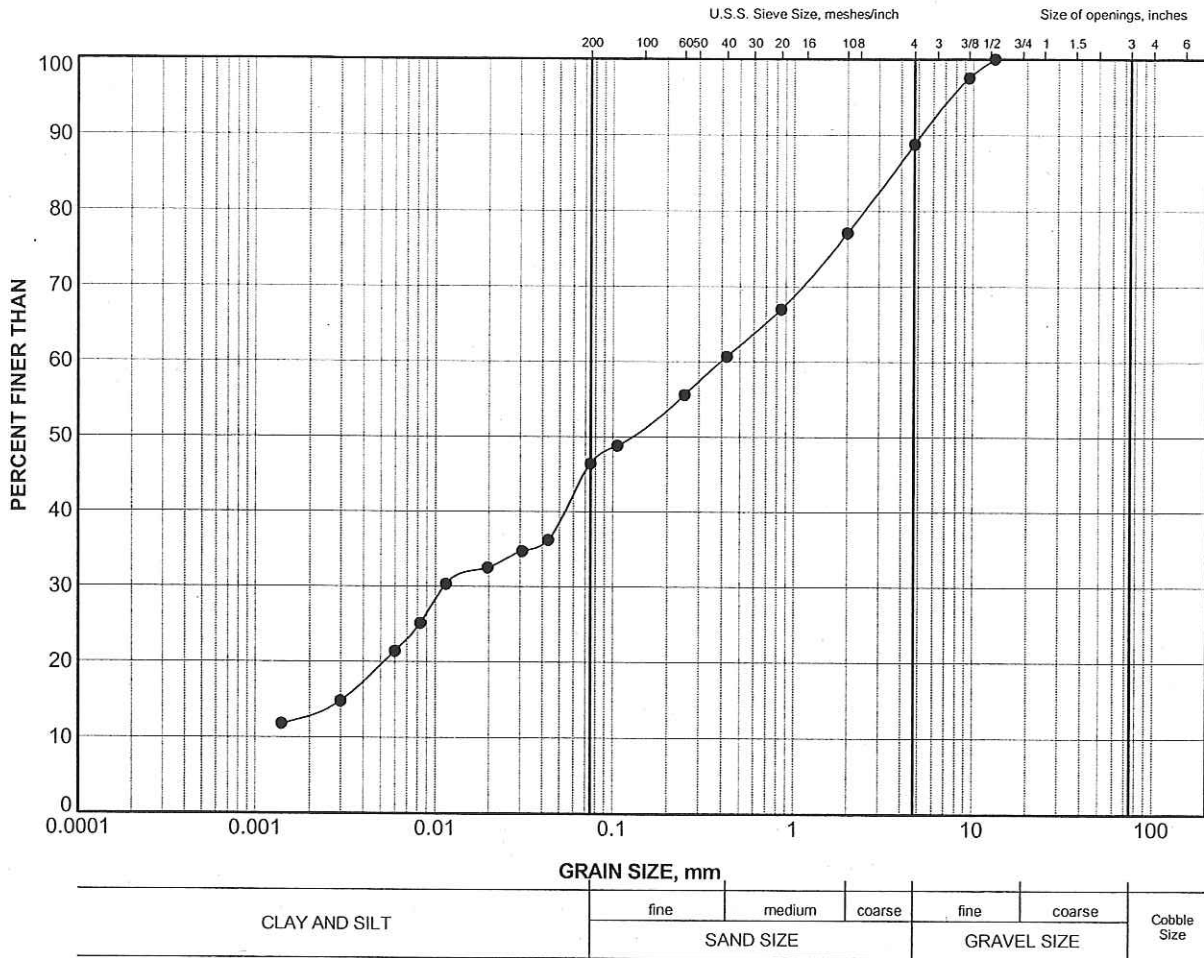
SOIL TYPE
 C = Clay
 M = Silt
 O = Organic

PLASTICITY
 L = Low
 I = Intermediate
 H = High


LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)	LL(%)	PL(%)	PI
●	BH1	18	161.8	38.9	17.8	21.1
■	BH1	28	146.5	45.6	23.9	21.7
▲	BH2	22	145.3	45.6	20.3	25.3

PROJECT CHRISTINA STREET UNDERPASS REPLACEMENT GWP 3038-03-00 HWY 402			
TITLE PLASTICITY CHART (Silty Clay)			
PROJECT No. 041-130099-2		FILE No. 041-130099-2.GPJ	
DRAWN BG		SCALE N/A	
CHECK <i>[Signature]</i>		REV.	
DATE Sep 16/04		FIGURE A-6	
Golder Associates LONDON, ONTARIO			



LEGEND			
SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	BH1	39	129.9

PROJECT					
CHRISTINA STREET UNDERPASS REPLACEMENT					
GWP 3038-03-00					
HWY 402					
TITLE					
GRAIN SIZE DISTRIBUTION					
SANDY SILT TILL					
PROJECT No.		041-130099		FILE No. 041-130099-2.GPJ	
DRAWN		WDF		SCALE N/A	
CHECK		19/		REV.	
 Golder Associates LONDON, ONTARIO		Sep 14/04 017-13/04		FIGURE A-7	

LDN MTO NEW GLDR LDN.GDT

APPENDIX B
SITE PHOTOGRAPHS

SITE PHOTOGRAPHS



Photo 1



Photo 2