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PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT PROPOSED HIGHWAY 40 N/S-W RAMP OVERPASS STRUCTURE HIGHWAY 402, GWP 3038-03-00 AGREEMENT NUMBER 3005-A-000394

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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 construction of a new structure at the Highway 40 (Modeland Road) interchange which will convey two lanes of westbound local Highway 402 traffic over the future U.S. bound trucks only Highway 40 N/S-W Ramp.

The purpose of the foundation investigation is to determine the subsurface conditions at the site of the proposed bridge. 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 proposed overpass structure, Site 14-582. The design drawings indicate that a new single span structure on HP 310 x 110 steel piles has been proposed. The average elevation of the future bridge deck is 187.0 metres.

2.0 SITE DESCRIPTION

The project area covered by this report is located on Highway 402, approximately 6.1 kilometres east of the east end of the Blue Water Bridge over the St. Clair River, at the crossing of Highway 40 (Modeland Road) in Sarnia, Ontario. The subject site is approximately 90 metres west of the of the centerline of Modeland Road and northwest of Highway 40 and Highway 402. The ground surface elevation in the vicinity of the site is between 178 and 181 metres. The site is adjacent to an agricultural field which was under cultivation during recent visits to the site and Highway 402. The site location is shown on Figure 1.

3.0 INVESTIGATION PROCEDURES

The field work for this investigation was carried out between July 25 and July 27, 2005. One borehole was drilled near the north end of the proposed structure. The borehole penetrated the overburden and was advanced into the underlying bedrock to a depth of 34.0 metres. The borehole location is shown in plan on Drawing 1.

The investigation was carried out using a track mounted CME 750 drill rig supplied and operated by Lantech Drilling Services Inc. The borehole was drilled using mud rotary drilling techniques with an N-sized tricone bit being used to penetrate the bedrock. 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 for laboratory consolidation testing. Further, in situ vane shear testing was carried out in the softer cohesive soils to determine their undrained shear strength. Groundwater conditions were observed in the open borehole throughout the drilling operations. The borehole was 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 borehole in the field, obtained utility locates, directed the drilling, sampling and in-situ testing operations, and logged the borehole. 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 in 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 location and elevation was surveyed by J.D. Barnes Limited. The elevation at the borehole is understood to be referenced to geodetic datum.

The borehole location is shown in plan on Drawing 1. The subsurface conditions encountered in the borehole are shown on the Record of Borehole sheet.

The borehole location and ground surface elevation is summarized as follows:

BOREHOLE NUMBER	BOREHOLE LOCATION		GROUND SURFACE ELEVATION (m)
	NORTHING (m)	EASTING (m)	
1	4761274.97	317291.19	179.45

4.0 GENERAL SITE GEOLOGY AND STRATIGRAPHY

4.1 Geology

The area of the site is located in the physiographic region known as the Huron Fringe[1]. Geological information indicates that the general soil conditions for the area consist of glacial lacustrine deposits overlying deep lacustrine till deposits. An extensive deposit of peat and muck lies between the approximate boundaries of Highway 7 to the south, Lakeshore Road to the north, Highway 40 to the west and Bridgen Road to the east.

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. A previous study of bedrock in the area indicated that the elevation of the bedrock surface in the vicinity of the site was between 150 and 152 metres. 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 borehole together with the results of the field and laboratory testing are shown on the Record of Borehole sheets. 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 beyond the borehole location.

In summary, the subsoils at the site originally consisted of topsoil and a surficial layer of loose silty fine sand and stiff clayey silt. Beneath the clayey silt was a 31 metre thick layer of silty clay till. Bedrock was encountered at a depth of 33 metres or at approximate elevation 146 metres.

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

Fill materials were not encountered at the borehole location; however, fill associated with the existing embankments for the existing N/S-W ramp should be expected.

The location and elevation of the borehole is shown on the attached Drawing 1. A detailed description of the subsurface conditions encountered in the borehole for this investigation is provided on the Record of Borehole sheets and is summarized in the following sections.

4.2.1 Topsoil and Organics

A 450 millimetre thick layer of topsoil was encountered at the surface of borehole I. Although organics were not encountered in this borehole, Department of Highways – Ontario Drawing 69-F-119A, indicates that the proposed structure is in an area where the surficial depth of organic material is typically 1.5 metres.

4.2.2 Silty Fine Sand

Beneath the topsoil at elevation 179.0 metres, a 0.9 metre thick layer of silty fine sand was encountered. The silty fine sand had an N value of 6 blows per 0.3 metres and a water content of 15 per cent.

4.2.3 Clayey Silt

The silty fine sand was underlain by a 0.8 metre thick clayey silt layer from elevation 178.1 metres. The clayey silt had an N value of 12 blows per 0.3 metres.

4.2.4 Silty Clay Till

The clayey silt was underlain by an extensive deposit of silty clay till from elevation 177.3 metres. The silty clay till layer was found to be 30.9 metres thick at the borehole location with a desiccated crust above elevation 176 metres. The grain size distribution curves for the silty clay till samples obtained from the standard penetration testing and thin walled tube samples are shown on Figure A-1. Shale fragments were noted in the till below elevation 150.4 metres.

In situ vane testing carried out in the silty clay till material indicated undrained shear strengths ranging from 62 to over 144 kilopascals (kPa) indicating a stiff to very stiff consistency. The silty clay had N values ranging from 5 to 26 blows per 0.3 metres.

The water contents of the silty clay till samples ranged from about 15 to 34 per cent. The silty clay till is an inorganic clay of low to intermediate plasticity with average plastic and liquid limits of 18 and 34 per cent, respectively and an average plasticity index of 16 per cent. The results of the Atterberg Limit testing are shown in the Plasticity Chart, Figure A-2.

The results of the laboratory consolidation testing carried out on sample 8 from borehole I are provided on Figures A-3 and A-4. The results indicate that the silty clay till is slightly preconsolidated by about 65 kilopascals beyond the existing overburden pressure. The following table summarizes the relevant oedometer test results:

BOREHOLE AND SAMPLE	DEPTH (m)	σ'_{po} (kPa)	σ'_{p} (kPa)	OCR	e_o	C_r	C_c	C_v cm²/sec
1 SA 8	7.1	95	160	1.68	0.60	0.039	0.19	6.2×10^{-3}

4.2.5 Bedrock

The bedrock surface was encountered some 33.0 metres below ground surface, or at elevation 146.4 metres, in borehole I. The borehole was terminated in shale bedrock of the Kettle Point formation after exploring it for 1 metre.

4.3 Groundwater Conditions

Groundwater was not encountered in borehole I. However, several boreholes advanced for the existing Highway 402 overpass at Modeland Road intercepted the groundwater level at depths of 0.5 to 1.1 metres below ground surface, or at about elevation 178 to 180 metres.[2]

It should be noted that the encountered groundwater levels reported do not indicate the long term stable groundwater elevations and that the groundwater levels are subject to seasonal fluctuations.

Based on soil colour changes and the encountered groundwater levels, the long term groundwater level has been inferred to be within the silty clay till deposits at approximate elevation 177 metres.

5.0 MISCELLANEOUS

The investigation was carried out using equipment supplied and operated by Lantech Drilling Services Inc. (Lantech). Lantech is an Ontario Ministry of Environment licensed well contractor. Field operations were supervised by Mr. Robert Cotnam under the direction of Mr. David J. Mitchell. All routine laboratory testing was conducted at Golder Associates' London laboratory. The consolidation testing was conducted at Golder Associates' Mississauga laboratory. Both

[2] Department of Highways Ontario 1970, Foundation Investigation Report For The Proposed Hwy. #402 Overpass at Modeland Road, District No. 1 (Chatham), W.O. 70-11046 – W.P. 122-65-03 & 04 (Geocres No. 40J16-40).

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 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 HIGHWAY 40 N/S-W RAMP
OVERPASS 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 and existing structures 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 the widening and/or replacement of existing Highway 402 bridges in the area. Based on the currently available information, a new overpass structure will be constructed to convey local traffic traveling westbound on Highway 402 over the redesigned Highway 40 N/S-W ramp for U.S. bound trucks only at the Highway 40 (Modeland Road) interchange. The future 15 metre single span structure will be founded on steel H-piles with the underside of the pile cap near elevation 179.0 metres. An approximate deck elevation of 187.0 metres has been proposed.

6.2 Bridge Foundations

The subsurface conditions at the site of the proposed Highway 40 N/S-W ramp overpass structure generally consists of surficial deposits of topsoil approximately 0.5 metres thick. Previous investigations conducted by the MTO in this area noted that surficial organic deposits typically 1.5 metres in depth in the vicinity of the proposed structure. Although fill materials were not encountered in the borehole, fills associated with the existing N/S-W ramp should be expected. The topsoil is underlain by loose silty fine sand from elevation 179 metres. Beneath the sandy deposits at elevation 178 metres is a layer of stiff clayey silt. The clayey silt is underlain by stiff to very stiff silty clay till from elevation 177 metres. Shale bedrock of the Kettle Point Formation was encountered near elevation 146 metres after penetrating 33 metres of overburden. The groundwater table is expected to fluctuate between elevations 176 and 179 metres.

As noted above, substantial deposits of organic soils were noted in previous investigations conducted by the MTO in the surrounding area. It has been assumed for the purposes of this report that these materials were removed from the Highway 40 interchange area during construction of the existing N/S-W ramp and Highway 40 overpass structure and replaced with compacted inorganic clayey fill. Fill depths up to 1.5 metres can be expected in areas where organic materials were removed. However, it is possible that localized deposits of organic soils still remain in previously undeveloped areas.

Based on the subsurface information noted above and the understanding that the proposed overpass structure will be built with piled foundations, consideration may be given to supporting the proposed structure on deep foundations such as steel piles driven to practical refusal on bedrock. Various shallow and deep foundation alternatives have been considered and the risks, consequences, costs and feasibility of the foundation options are compared in Table I.

6.3 Shallow Foundations

The new overpass structure could be supported by spread footings founded in the native clayey silt or silty clay till crust below any fill or organic layers. If the new structure is to be founded on spread footings, drilling of several additional boreholes for detail design will required in order to ascertain if organic materials and/or fill materials are present within the footprint of the proposed structure, and, if found, to what extent. The silty clay till soils are expected to provide limited bearing resistance and the groundwater table is relatively shallow. Settlement of the footings due to consolidation of the underlying clayey deposits and differential settlement of the embankment fills relative to the footings should be expected. Various measures, involving additional time and costs, could be taken to minimize the amount of the settlement(s). However, the use of shallow footings is not the preferred founding option due to limited bearing resistance of the clayey silt, the potential for excessive settlement, and the additional construction costs that could be incurred if organic materials or unsuitable fills are encountered or settlement mitigation measures such as preloading for several years are implemented.

6.3.1 Axial Geotechnical Resistance – Spread Footings

Based on the results of this investigation, spread footings at about elevation 177 metres on the stiff to very stiff silty clay till could be utilized. A factored geotechnical resistance of 225 kPa at the Ultimate Limit States (ULS) and a geotechnical resistance 150 kPa 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 actual settlements of these footings will be dependent on the footing size, configuration and applied loads. Additional settlement of the footings will occur due to consolidation of the founding soils under the new embankments in the abutment areas. However, the embankments could be constructed well in advance to reduce the footing settlements. In addition to the additional investigation to delineate the extent any organic and/or fill layers, settlements should be confirmed at the 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 after preloading of the embankments have been carried out.

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 δ, may be used:

Footings on silty clay till	angle of friction	27°
	tan δ	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 piles driven to practical refusal on bedrock are considered suitable to support the abutments and the pier for the proposed structure. H-piles are recommended because they will easily penetrate the clayey deposits and minimize the amount of disturbance given their shape and small cross-sectional area. They will also have the necessary flexibility required for use with integral abutments should they be part of the design. Use of driven steel H-piles is the preferred foundation alternative.

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 146 metres may be taken as 2,000 kilonewtons (kN). This value takes into account the structural capacity limitation of the pile. Vertically driven piles 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. 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 that the ULS value.

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

6.4.2 Downdrag Load (Negative Skin Friction)

Consolidation settlement of the underlying extensive clayey deposits due to the increased loading imposed by the embankments should be expected. The consolidation settlement is time-dependent and, depending on the sequencing of construction, may not completely occur during the construction period. That is, post-construction settlement of the clayey deposits may take place and settlement of the clayey soils relative to the piles will result in the development of negative skin friction acting on the piles. Therefore, negative skin friction or downdrag loads will need to be taken into account during design of the piles supporting the abutments. If the new embankments are constructed well in advance of the piling or lightweight fills are utilized, the downdrag loads may be minimized or eliminated.

The magnitude of the downdrag load acting on a pile is a function of the adhesion (skin friction) that develops between the pile and the clay, the surface area of the pile within the clay deposit and the embankment loading. The load calculated in this manner is a nominal (unfactored) load. The structural engineer needs to multiply this load by a load factor of 1.25, as defined in the Canadian Highway Bridge Design Code (CHBDC), and include it as part of the load effects acting on the pile as described in the CHBDC. For preliminary design, the negative skin friction load on a single end bearing pile may be taken as 250 kN. This value was computed using methods described in the Canadian Foundation Engineering Manual and is based on our experience with piles founded in similar moderately compressible soils in the area. The actual negative skin friction will depend on the extent of filling and construction sequencing, both of which are currently unknown. If the embankments are not constructed well in advance of the piling, the downdrag load will have to be reassessed during the detailed design stage by the foundation engineer.

6.4.3 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. Since integral abutments are under consideration, there is a requirement for the piles to move sufficiently to accommodate deflections of the bridge deck.

The abutment piles will be driven through embankment fill and the underlying cohesive soils. 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 (cohesive)	120	35
Sandy deposits	100	25
Clayey silt and silty clay deposits	160	65

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:

Pile Spacing in Direction of Loading, d = Pile Diameter	***Subgrade Reaction Reduction Factor R***
8d	1.00
6d	0.70
4d	0.40
3d	0.25

6.4.4 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 and retaining walls in accordance with the CHBDC:

- Select, free-draining granular fill meeting the specifications of Ontario Provincial Standard Specifications (OPSS) Granular A or Granular B Type III 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 Ontario Provincial Standard Drawing (OPSD) 3501.00 and 3504.00.
- A compaction surcharge equal to 12 kPa 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 1.2 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:

	GRANULAR A	GRANULAR B TYPE III
Soil unit weight:	22 kN/m³	21 kN/m³
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 and OPSD 3504.00.

6.6 Embankments

The embankments for the proposed overpass are expected to be up to 6 metres in height. 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.

The topsoil and any organic materials encountered 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. Preliminary estimates of total embankment settlement, using granular or earth fill, are in the order of 350 millimetres assuming an embankment height of 6 metres and a width of 16 metres with 2 horizontal to 1 vertical side slopes. Settlements could be reduced by up to 50 per cent by using lightweight fill. More detailed settlement analyses should be conducted once the construction sequencing is known and the design has been finalized.

6.7 Retaining Walls

6.7.1 Foundations

Retaining walls, have been proposed to retain embankment fills at the approaches. The exact length and heights of these walls are not known, but the wall height is expected to vary with maximum heights of 5 to 6 metres. The proposed retaining wall design is a retained soil system (RSS) wall. The settlements resulting from the embankment loadings may be too large for an RSS wall or for a concrete cantilever wall alternative unless embankment preloading is employed. A soldier pile and lagging wall with decorative facings may be a suitable alternative to an RSS system.

The subsurface conditions encountered in the borehole typically consist of surficial topsoil over loose silty fine sand from elevation 179 metres. Beneath the sand is a layer of stiff clayey silt from elevation 178 metres underlain by an extensive deposit of stiff to very stiff silty clay till from elevation 179 metres. Shale bedrock of the Kettle Point formation was encountered at elevation 146 metres. The groundwater table is expected to be near 177 metres.

Based on the subsurface information and effective embankment preloading, RSS and cantilever walls may be supported on spread footings. Alternatively, an RSS wall may be founded on a 0.3 metre thick compacted Granular A leveling pad constructed on the surface of the clayey silt. Otherwise deep foundations would be considered warranted.

6.7.2 Geotechnical Resistance

Spread footings placed on the stiff native clayey silt near elevation 178 metres are suitable for founding the proposed retaining wall provided that embankment preloading is employed. A factored geotechnical resistance at ULS of 150 kilopascals and 100 kilopascals at SLS may used for preliminary design purposes assuming a 2 metre wide footing. Care should be taken during construction to avoid disturbance of the clayey silt subgrade prior to concrete placement.

6.7.3 Resistance to Lateral Forces

The lateral pressures acting on the proposed retaining wall will depend on the backfill soils, the type and method of placement of the backfill materials behind the wall, provision of reinforcement grids, as well as the subsequent lateral movement of the structure. Recommendations concerning the design of retaining walls in accordance with the CHBDC were provided in Section 6.5. In addition to the lateral earth pressures stated in Section 6.5, the following unfactored values may be assumed for the passive earth pressure coefficient, $K_{p:}$

	GRANULAR A	GRANULAR B TYPE III
Coefficients of lateral earth pressure:		
Passive, K_p	3.6	3.3

The design recommendations and parameters stated in Section 6.5 and this section assume that the backfill and ground surface behind the walls are level. Where there is sloping ground behind the walls, the coefficient of lateral earth pressure must be adjusted accordingly.

6.8 Excavations and Temporary Cut Slopes

Excavations for pile cap construction will extend through any surficial fill and/or organics and will encounter the loose silty fine sand. Based on the subsurface conditions encountered in the boreholes, the base of the pile cap excavations may intercept the long term groundwater level. 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 particularly if the groundwater table is high at the time of construction. 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 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. The organic deposits, silty fine sand and any fills encountered at this site would be classified as Type 3 soils. The underlying cohesive 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

Proposed Highway 40 N/S-W Ramp Overpass Structure
Highway 402, GWP 3038-03-00
Agreement Number 3005-A-000394

FOUNDATION OPTION	FEASIBILITY	ADVANTAGES	DISADVANTAGES	ESTIMATED COSTS	RISKS/CONSEQUENCES
Spread footings supported on native clayey silt	• Not considered feasible due to low geotechnical resistance, potential for high consolidation settlements and possible need to remove organic materials if encountered	• Cost	• Time and cost of settlement mitigation measures • Even if mitigation measures adopted, settlement of shallow foundations could still take place • If encountered, additional time and costs will be incurred to replace unsuitable organic materials with clean fill	• Expected to be less expensive than deep foundation options • Approximate cost $55,000 assuming four 4 m wide strip footings	• Even if mitigation in place, shallow foundations will still be affected by settlement of clayey silt and silty clay till deposits • Probability of encountering at footing locations organic materials which are still in place and will require removal
Spread footings perched on granular pad in embankments	• Potential for high consolidation settlements and possible need to remove organic materials if encountered	• Cost • Greater bearing resistance compared to spread footings on native clayey silt	• Time and cost of settlement mitigation measures • Even if mitigation measures adopted, settlement of shallow foundations could still take place	• Similar in costs to spread footings on native soils and less expensive than deep foundations • Approximate cost $45, 000 assuming two 2.5 metre wide strip footings; cost of preloading not included	• Even if mitigation in place, shallow foundations will still be affected by settlement of clayey silt and silty clay till deposits • Probability of encountering at footing locations organic materials which are still in place and will require removal

FOUNDATION OPTION	FEASIBILITY	ADVANTAGES	DISADVANTAGES	ESTIMATED COSTS	RISKS/CONSEQUENCES
Steel H pile foundations founded on shale bedrock	• Feasible for support of all foundation elements	• High bearing resistance • Negligible settlement	• Possibility of damage to tip while pile driving in bedrock • Care must be taken with driving of battered piles to ensure that the piles do not deflect along the bedrock surface	• Approximate cost $265, 000 • More expensive than shallow foundations but preferred technical solution	• Possible pile tip damage if tip is not suitably protected while driving in rock • Probability of encountering at pile cap locations organic materials which are still in place and will require removal

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

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.)

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, 60º 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^2 pushed through ground at a penetration rate of 2 cm/s. Measurements of tip resistance (Q_t), porewater pressure (PWP) and friction along a sleeve are recorded electronically at 25 mm penetration intervals.

III. SOIL DESCRIPTION

(a) Cohesionless Soils

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

(b) Cohesive Soils

Consistency	c_u, s_u kPa	c_u, s_u psf
Very soft	0 to 12	0 to 250
Soft	12 to 25	250 to 500
Firm	25 to 50	500 to 1,000
Stiff	50 to 100	1,000 to 2,000
Very stiff	100 to 200	2,000 to 4,000
Hard	over 200	over 4,000

IV. SOIL TESTS

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

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

LIST OF SYMBOLS

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

I. General

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

II. STRESS AND STRAIN

γ	shear strain
Δ	change in, e.g. in stress: $\Delta\ \sigma$
ε	linear strain
ε_v	volumetric strain
η	coefficient of viscosity
v	poisson's ratio
σ	total stress
σ'	effective stress ($\sigma' = \sigma$-u)
σ'_{vo}	initial effective overburden stress
σ_1, σ_2, σ_3	principal stress (major, intermediate, minor)
σ_{oct}	mean stress or octahedral stress $= (\sigma_1+\sigma_2+\sigma_3)/3$
τ	shear stress
u	porewater pressure
E	modulus of deformation
G	shear modulus of deformation
K	bulk modulus of compressibility

III. SOIL PROPERTIES

(a) Index Properties

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

(a) Index Properties (continued)

w	water content
w_l	liquid limit
w_p	plastic limit
l_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)

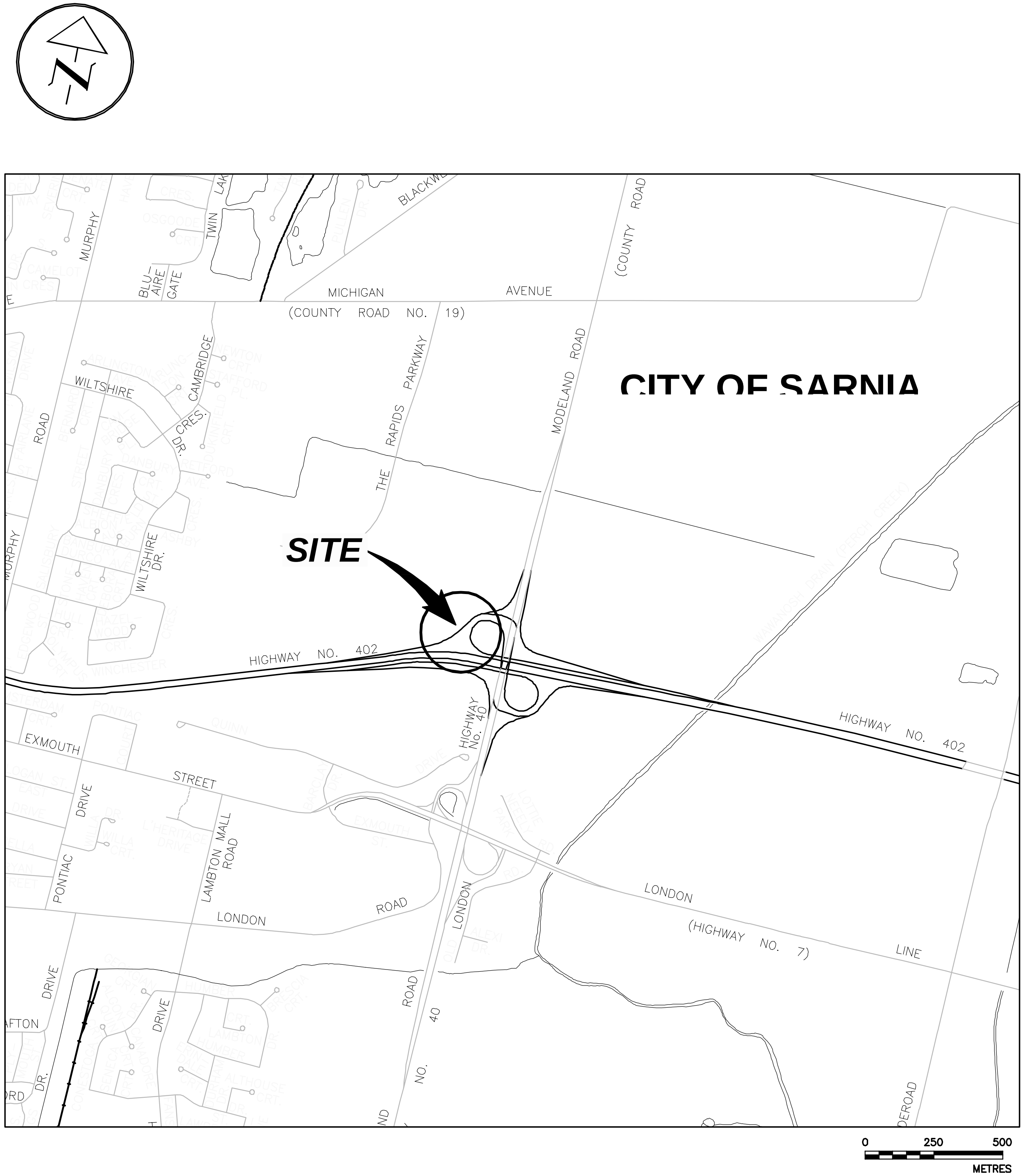
C_c	compression index (normally consolidated range)
C_r	recompression index (over-consolidated range)
C_s	swelling index
C_a	coefficient of secondary consolidation
m_v	coefficient of volume change
c_v	coefficient of consolidation
T_v	time factor (vertical direction)
U	degree of consolidation
σ'_p	pre-consolidation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

(d) Shear Strength

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

Notes:
1. $\tau = c' + \sigma' \tan \phi'$
2. shear strength = (compressive strength)/2

* density symbol is ρ. Unit weight symbol is γ where $\gamma = \rho g$ (i.e. mass density x acceleration due to gravity)



PROJECT

HIGHWAY 40 N/S-W RAMP OVERPASS STRUCTURE
WP No. 3038-03-00
HWY. 402

TITLE

SITE LOCATION PLAN

PROJECT No.	041-130099-9	FILE No.	041130099-9F001.
		SCALE	AS SHOWN REV. 0
CADD	DCH	July 19/06	
CHECK			**FIGURE 1**

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APPENDIX A

RESULTS OF LABORATORY TESTING

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

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

SHEET

HIGHWAY 40 N/S-W RAMP OVERPASS STRUCTURE
BOREHOLE LOCATION & SOIL STRATA

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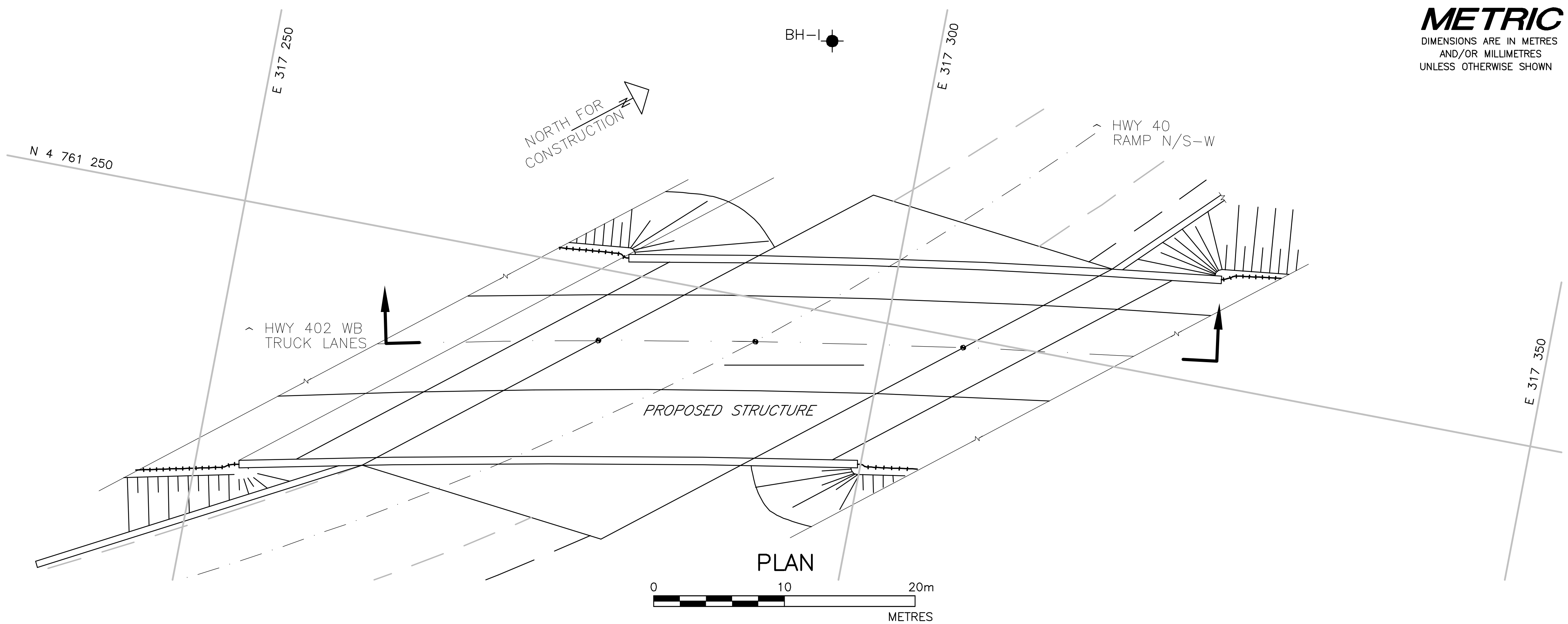
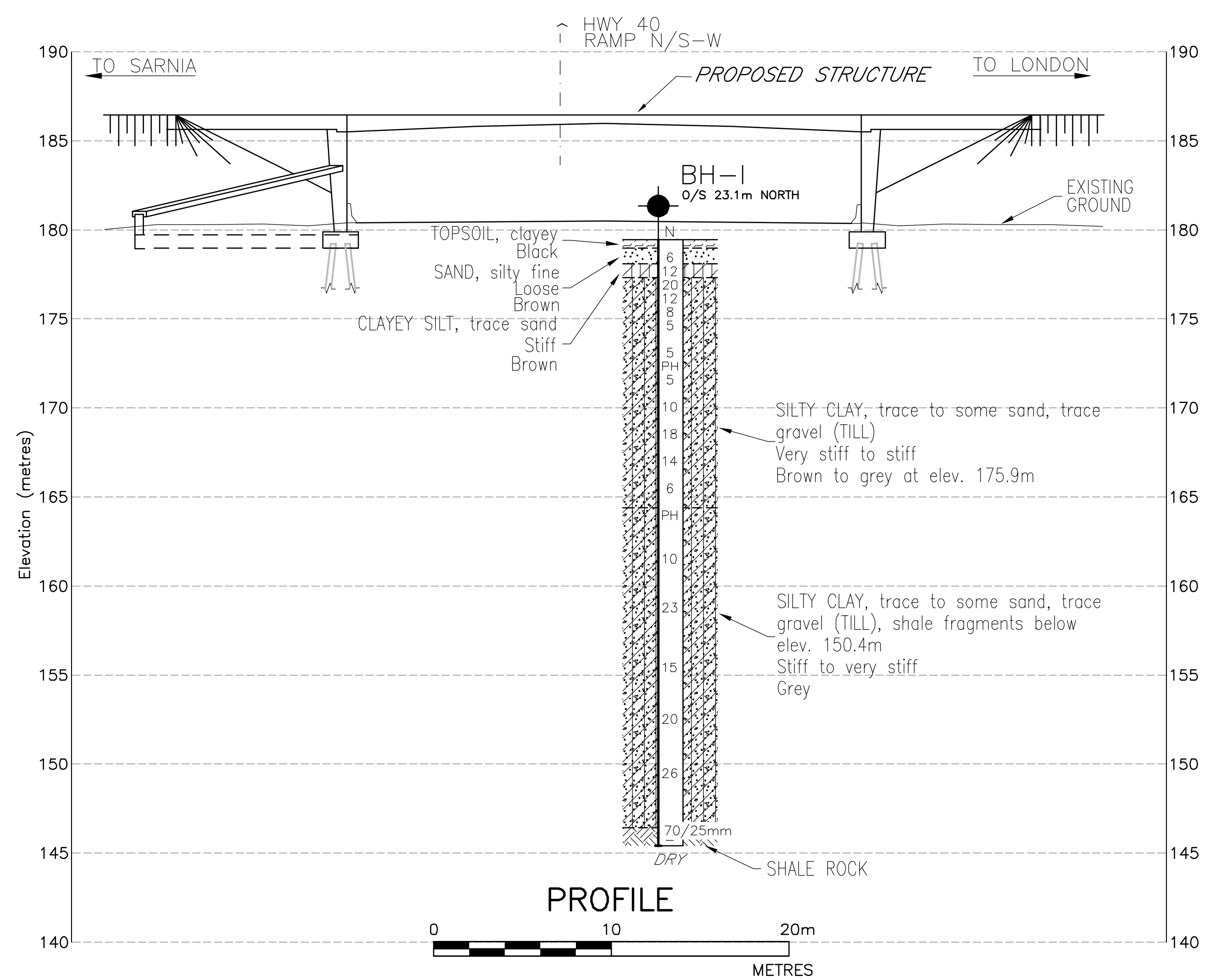
LONDON, ONTARIO, CANADA



PLAN



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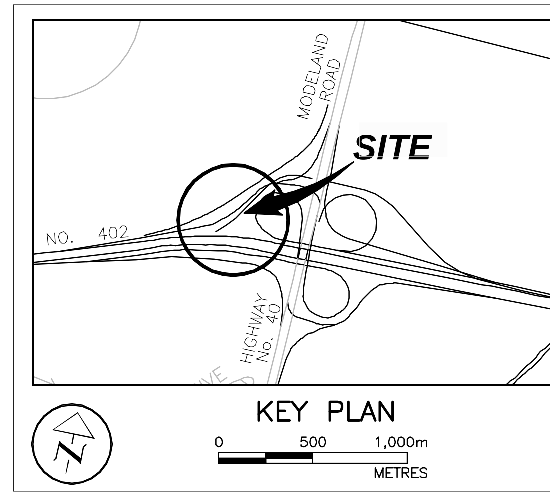


KEY PLAN

LEGEND

● Borehole

N Blows/0.3m (Std. Pen. Test, 475 j/blow)

DRY Borehole dry during drilling

No.	ELEVATION (metres)	CO-ORDINATES NORTH	CO-ORDINATES EAST
BH-I	179.45	4 761 275.0	317 291.2

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

REFERENCES

1) REFERENCE : URS
ENTITLED: HIGHWAY 40 NS-W RAMP OVERPASS
SITE NO: 14-582
DATED: AUGUST 2005

2) BOREHOLE STRATIGRAPHY AS NOTED ON RECORDS OF BOREHOLES FOR MINISTRY OF TRANSPORTATION ONTARIO, REPORT GEOCRES No.

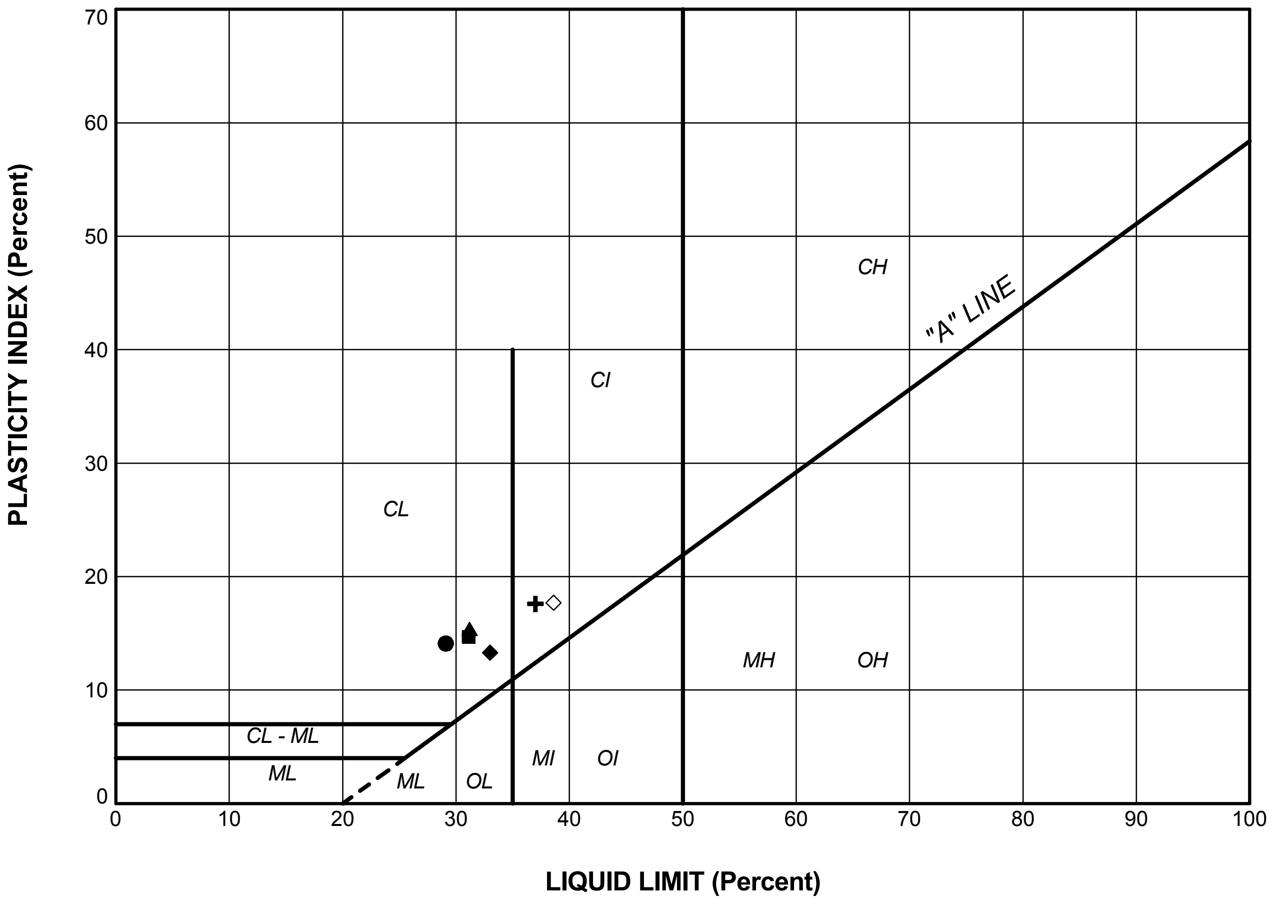
NO.	DATE	BY	REVISION

Geocres No. 40J16-69

HWY. No. 402	PROJECT NO.: 041-130099-9		
SUBM'D. –	CHKD:	DATE: July 19/06	
DRAWN: DCH	CHKD.	APPD.	DWG. 1

041130099-9D001.dwg

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



SOIL TYPE
C = Clay
M = Silt
O = Organic

PLASTICITY
L = Low
I = Intermediate
H = High

LEGEND

SYMBOL	BOREHOLE	SAMPLE	LL(%)	PL(%)	PI
●	BH-I	3	29.1	15.0	14.1
■	BH-I	6	31.1	16.4	14.7
▲	BH-I	8	31.2	15.8	15.4
✚	BH-I	13	37.0	19.4	17.6
◆	BH-I	15	33.0	19.7	13.3
◇	BH-I	19	38.6	20.9	17.7

PROJECT
HIGHWAY 40 N/S-W RAMP OVERPASS STRUCTURE
WP No. 3038-03-00
HWY. 402

TITLE

PLASTICITY CHART

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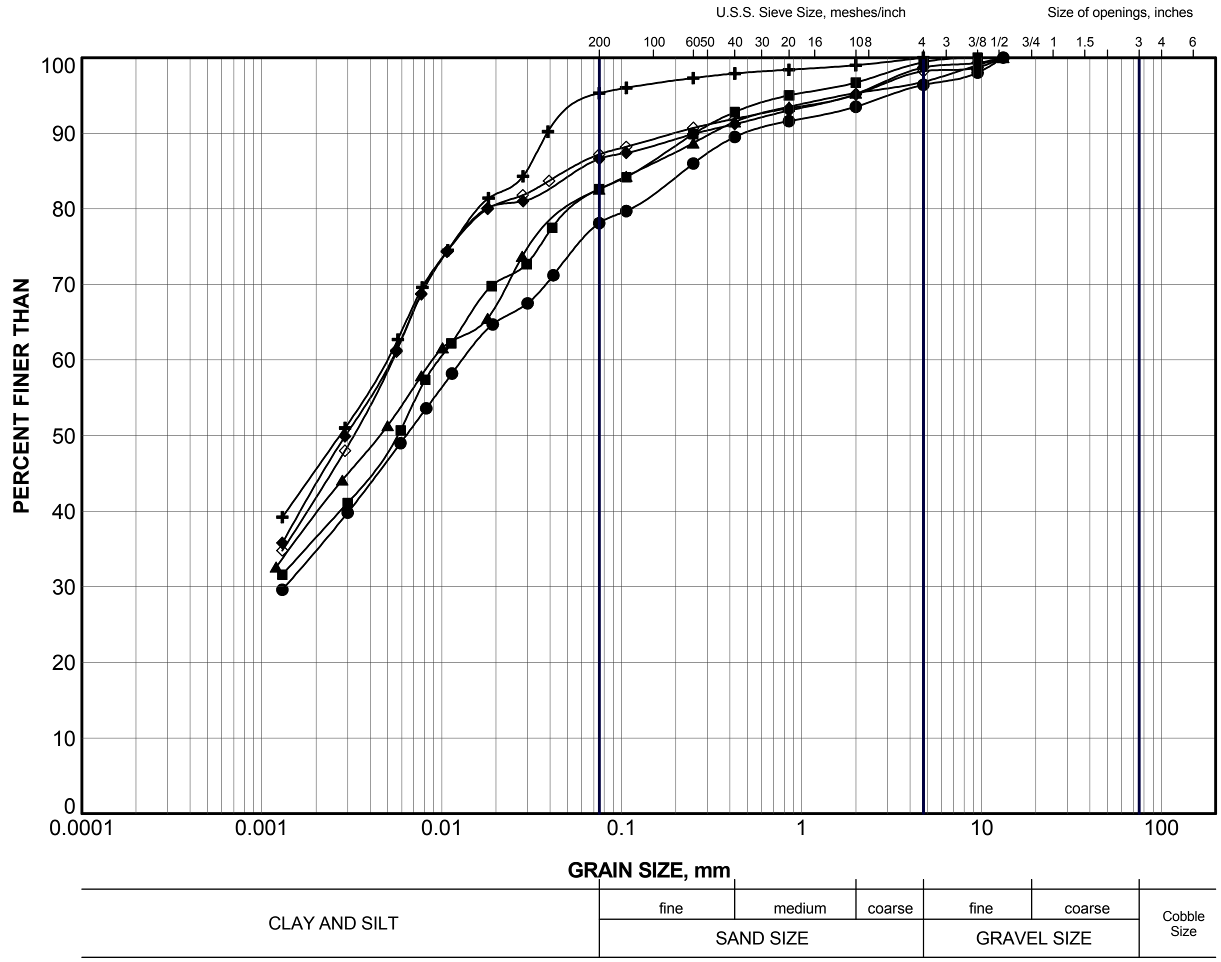
PROJECT No.	041-130099-9		FILE No.	041-130099-9.GPJ	
			SCALE	N/A	REV.
DRAWN	JPR	Sep 14/05	FIGURE A-2		
CHECK					

MTO_PI GLDR_LDN.GDT



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	BH-I	3	176.9
■	BH-I	6	174.6
▲	BH-I	8	172.3
✚	BH-I	13	165.5
◆	BH-I	17	155.4
◇	BH-I	19	149.5

PROJECT
HIGHWAY 40 N/S-W RAMP OVERPASS STRUCTURE
WP No. 3038-03-00
HWY. 402

TITLE

GRAIN SIZE DISTRIBUTION

SILTY CLAY TILL

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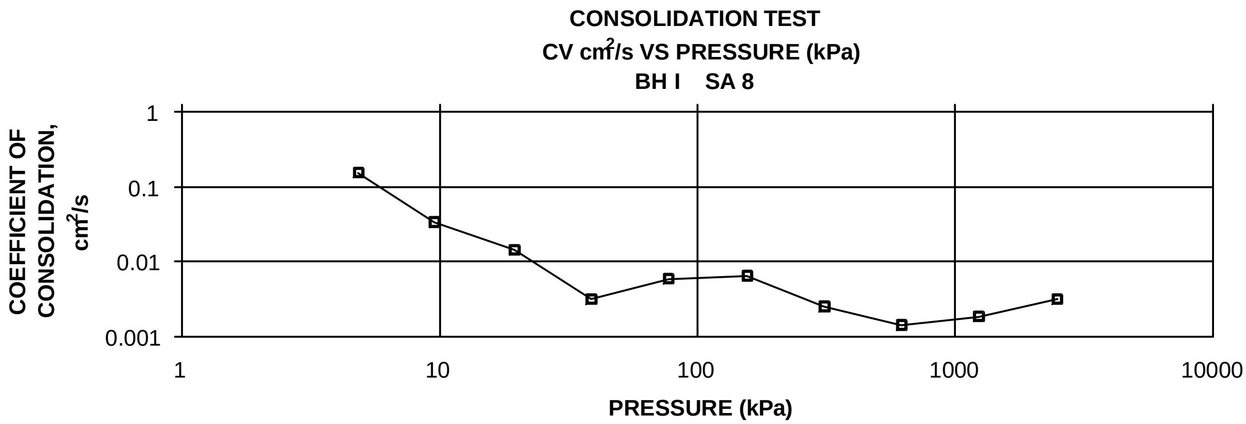
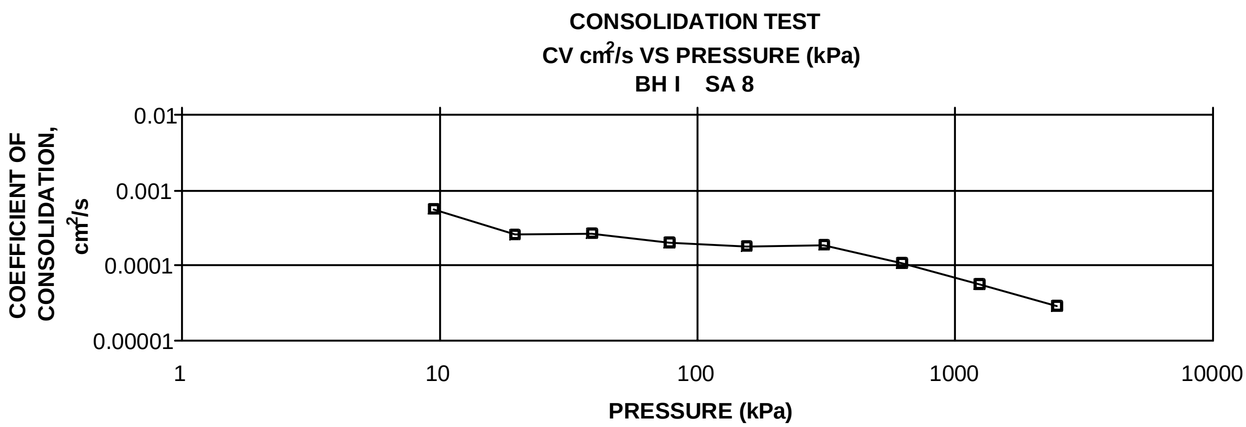
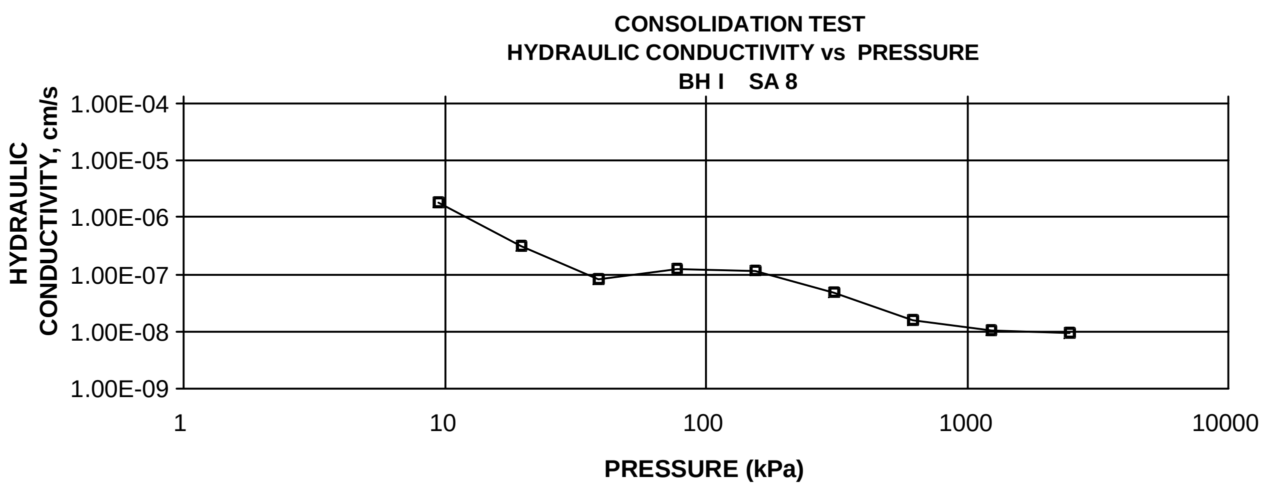
PROJECT No.	041-130099-9	FILE No.	041-130099-9.GPJ	
		SCALE	N/A	REV.
DRAWN	JPR	Sep 14/05		
CHECK				

FIGURE A-1

LDN_MTO_NEW GLDR_LDN.GDT







PROJECT
HIGHWAY 40 N/S-W RAMP OVERPASS STRUCTURE
WP No. 3038-03-00
HWY. 402

TITLE
CONSOLIDATION TEST RESULTS

Golder Associates
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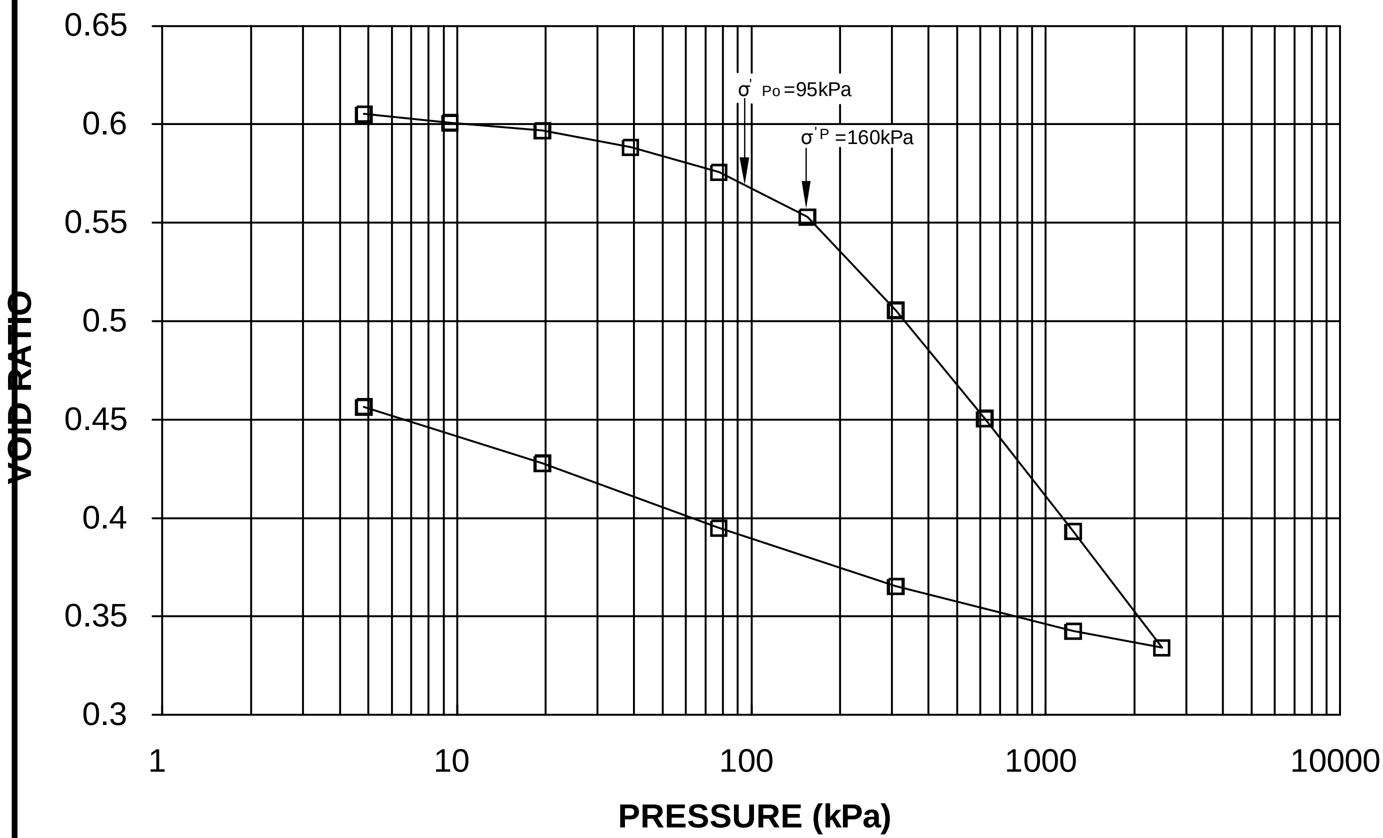
PROJECT No.	041-130099-9	FILE No.	041130099-F0A4
		SCALE	AS SHOWN REV. 0
CADD	JPR Sept. 13/05		
CHECK		**FIGURE A-4**	

Drawing file: 041130099-9F0A4.DWG Jul 29, 2006 - 11:05am

CONSOLIDATION TEST
VOID RATIO vs PRESSURE
BH I SA 8



PROJECT
HIGHWAY 40 N/S-W RAMP OVERPASS STRUCTURE
WP No. 3038-03-00
HWY. 402

TITLE

CONSOLIDATION TEST
VOID RATIO VS. LOG PRESSURE

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PROJECT No.	041-130099-9		FILE No.	041130099-F0A3
			SCALE AS SHOWN	REV. 0
CADD	DCH	Sept. 19/05		
CHECK			FIGURE A-3	

Drawing file: 041130099-9F0A3.DWG Jul 29, 2006 - 11:06am