PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT PROPOSED HIGHWAY 402 WB TRUCK LANES/ HIGHWAY 40 OVERPASS STRUCTURE HIGHWAY 402, GWP 3038-03-00 AGREEMENT NUMBER 3005-A-000394

Submitted to:

URS Canada Inc.
75 Commerce Valley Drive East
Markham, Ontario
L3T 7N9

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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 which will convey two lanes of westbound truck traffic from the future S-W ramp and Highway 402 over Highway 40 (Modeland Road) immediately north of the existing Highway 402 overpass at the Highway 40 Interchange.

The purpose of the foundation investigation is to determine the subsurface conditions at the site of the proposed new bridge by utilizing existing borehole data. 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 14-581. The new two-span structure will be a precast concrete girder bridge with integral abutments. The design drawings indicate that the abutments and piers will be supported on HP 310 x 110 steel piles. The average elevation of the future bridge deck is 187.9 metres.

2.0 SITE DESCRIPTION

The project area covered by this report is located on Highway 402, approximately 6.2 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 30 metres north of the centerline of the existing westbound Highway 40 Overpass. The subject site is adjacent to agricultural fields. During recent visits to the site, it was noted that the fields are cultivated. The site location is shown on Figure 1.

The approximate elevation of Highway 40 at this location is 180.7 metres.

3.0 INVESTIGATION PROCEDURES

No site specific field work was carried out for this investigation. However, the report utilizes the results of a previous investigation which was carried out for the adjacent Highway 40 Overpass structures, known as Sites 14-338/1 and 14-338/2. The results of this investigation are contained within MTO Report Geocres No. 40J16-40 entitled “Foundation Investigation Report for the Proposed Highway No. 402 Overpass at Modeland Road, District No. 1 (Chatham), W.O. 70-11046 -- W.P. 122-65-03 & 04” dated July 1970. At that time, sixteen boreholes were put down at the site of the existing structures. Information from twelve of the boreholes closest to the proposed structure was used to compile this report. Boreholes 1 to 8, 11, 13, 15 and 16 were drilled and sampled to depths of 6.9 to 33.9 metres with dynamic cone penetration testing carried out in the upper portion of two of the boreholes. The boreholes are shown on the Department of Highways, Ontario Drawing No. 70-11046A dated July 7, 1970. The boreholes were drilled during the periods November 18 to 22, 1969 and June 4 to 19, 1970.

The borehole locations are shown in plan on Drawing 1. The subsurface conditions encountered in the boreholes drilled along the north edge of the westbound structure are shown on Drawing 1 in metric units. The results of the boreholes drilled for the existing Highway 40 Overpass structures are provided in Appendix A in their original imperial units and format. Corresponding depth and elevations in metric units have been added to the Records of Boreholes.

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 boreholes together with the results of the field and laboratory testing are shown on the Record of Borehole sheets from the original investigation which are attached to this report in Appendix A. The stratigraphic boundaries shown on the borehole sheets are inferred from non-continuous sampling and, therefore, may represent transitions between soil types rather than exact planes of geological change. Subsoil conditions will vary between and beyond the borehole locations.

In summary, the subsoils at the site originally consisted of about 3 metres of generally soft peat and other organic deposits which are underlain by as much as 13 metres of hard to firm clayey silt. In the area of the interchange, the original soils were removed and replaced with compacted clayey fill. The clayey silt was underlain by a layer of generally stiff silty clay with an average

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

thickness of 17 metres. Seven of the twelve boreholes were advanced into the shale bedrock, the surface of which was encountered beneath the silty clay some 32 to 33 metres below the ground surface or between elevation 147 and 148 metres.

The Record of Borehole sheets for the original investigation did not note the presence of fill or topsoil. However, fill materials associated with the existing embankments for the E-S ramp and construction of the southbound lanes of Highway 40 are present at this site.

Locations and elevations of the borings are shown on the attached Drawing 1 together with the interpreted stratigraphical profile. A detailed description of the subsurface conditions encountered in the boreholes for this investigation is provided on the Record of Borehole sheets and is summarized in the following sections.

4.2.1 Surficial Peat, Organic Silt and Organic Clay

With the exception of boreholes 2 and 3, all of the boreholes encountered surficial layers of black fibrous peat, organic silt and/or organic clay which contained seams or pockets of peat, sand, organic silt, organic clay, shells or other organics. The surficial organic layers were 0.5 to 5 metres thick at the borehole locations with an average thickness of 3 metres. Department of Highways Ontario Drawing 69-F-119A dated April 15, 1970 indicates that the average depth of organic material in the area of the east abutment for the proposed structure was less than 1.5 metres. The depth of organic material then increased to over 4.6 metres at the central pier and west abutment area of the existing structure.

Standard penetration test N values of 1 to 15 blows per 0.3 metres with an average of 5 blows per 0.3 metres were recorded in the organic materials. The measured shear strength of the organic deposits ranged from 6 to 31 kilopascals (kPa) with an average of 22 kPa. Several samples were obtained by manually advancing a thin walled sampling tube. The water contents varied from 15 to 242 per cent. The organic deposits were highly plastic with an average plastic and liquid limits of 59 and 94 per cent, respectively, with an average plasticity index of 35 per cent.

The original report stated that the organic content of the samples were as high as 25 to 29 per cent. Analyses of the particle size distribution of selected samples indicated that the sand seams or pockets contained a trace to some fines.

The bulk density of the organic deposits ranged from 1.2 to 1.9 megagrams per cubic metre with an average of 1.4 megagrams per cubic metre based on the testing of five samples.

4.2.2 Granular Fill

Granular roadbase fill material was encountered only at the surface of borehole 3 to a depth of 1.8 metres. The granular fill had an N value of 6 blows per 0.3 metres and a water content of 16 per cent.

4.2.3 Clayey Silt

Clayey silt was encountered from the ground surface in borehole 2, beneath the granular fill in borehole 3 and beneath the surficial organics in the remaining boreholes. The clayey silt was intercepted at elevation 174.4 to 180.1 metres and extended for 9 to 23 metres, where fully penetrated. Boreholes 3, 8, 13, 15 and 16 were terminated in clayey silt. In borehole 2, trace amounts of organics were noted in the upper 1.5 metres.

Generally a very stiff to hard clayey silt crust was identified above elevation 174 metres. The clayey silt had standard penetration test N values from 7 to 46 blows per 0.3 metres. The shear strength of the stratum was measured by in situ vane testing and unconfined and quick triaxial testing of thin wall samples. In situ vane testing indicated undrained shear strengths ranging from 28 to greater than 105 kilopascals (kPa) with an average of 79 kPa. Confined and unconfined triaxial testing yielded shear strength values of 29 to 260 kPa. In situ and laboratory testing confirmed a firm to hard but generally stiff consistency.

The water contents of the clayey silt samples ranged from about 10 to 25 per cent with an average water content of 18 per cent. The clayey silt deposits were of low plasticity with average plastic and liquid limits of 16 and 29 per cent, respectively, and an average plasticity index of 14 per cent.

The clayey silt had bulk densities ranging from 2.0 to 2.3 megagrams per cubic metre with an average of 2.1 megagrams per cubic metre. The results of three grain size analyses of samples obtained in the clayey silt deposits indicated an average of 34 per cent clay, 47 per cent silt, 17 per cent sand and 2 per cent gravel. The results of a fourth sample suggest that a sand and gravel seam was intercepted near elevation 174 metres in borehole 6.

4.2.4 Silty Clay

In the deeper boreholes, stiff to very stiff silty clay was encountered beneath the clayey silt at elevation 157.1 to 166.5 metres. Boreholes 4 and 6 were terminated in silty clay after exploring it for some 4.9 to 8.8 metres, respectively. Where fully penetrated, the silty clay layer was found to be 9.4 to 18.9 metres thick.

In situ vane testing carried out in the silty clay materials indicated undrained shear strengths ranging from 22 to over 122 kPa with an average of 70 kPa. The shear strengths measured from confined and unconfined triaxial testing conducted on several thin wall samples were 26 to 120 kPa with an average of 86 kPa. The field and laboratory testing indicated that the silty clay materials generally have a stiff consistency. The silty clay had N values ranging from 5 to 36 blows per 0.3 metres.

The water contents of the silty clay samples ranged from about 18 to 36 per cent with an average water content of 25 per cent. The silty clay deposits had average plastic and liquid limits of 20 and 37 per cent, respectively, with an average plasticity index of 17 per cent.

The silty clay materials had a bulk density of 1.8 to 2.1 megagrams per cubic metre with an average of 2.0 megagrams per cubic metre based on the testing of ten samples.

The grain size distribution obtained from a single sample of silty clay indicated 43 per cent clay, 35 per cent silt, 18 per cent sand and 4 per cent gravel.

4.2.5 Bedrock

The bedrock surface was encountered some 31.9 to 32.6 metres below ground surface, or at elevation 146.8 to 147.9 metres, in boreholes 1, 2, 5, 7 and 11. All of these boreholes were terminated in bedrock. The top 1.5 metres of the bedrock was cored in borehole 7. The bedrock surface was inferred by refusal at the remaining borehole locations. The surface of the bedrock dips slightly to the west. The original report classified the rock as sound black shale of the Kettle Point formation. The rock core recovery reported on the log for borehole 7 was 92 per cent indicating excellent recovery of the AXT sized core.

4.3 Groundwater Conditions

Groundwater was encountered in seven of the twelve boreholes during drilling and are reported to be 0.5 to 1.1 metres below ground surface, or at about elevation 178 to 180 metres. The groundwater table was not established during drilling at boreholes 2, 8, 11, 13 and 16. The encountered water levels are shown on the attached Record of Borehole sheets in Appendix A and are summarized below.

It should be noted that the encountered groundwater levels reported do not indicate the long term stable ground water elevations and that the groundwater levels are subject to seasonal fluctuations. The groundwater table may also be influenced by the water level in the adjacent St. Clair River which can vary as much as 1.6 metres. On August 10, 2005, the Canadian Hydrographic Service water level gauge on the St. Clair River at nearby Point Edward, Ontario recorded a high water level of 176.175 metres and a low water level of 175.829 metres as referenced to the International Great Lakes Datum 1985 (IGLD).

BOREHOLE	GROUND SURFACE ELEVATION (m)	ENCOUNTERED WATER LEVEL ELEVATION (m)
1	180.2	179.1
2	179.9	-
3	180.6	179.6
4	179.4	178.8
5	179.0	178.2
6	179.5	178.9
7	179.4	178.6
8	180.1	-
11	179.0	-
13	179.4	-
15	179.8	179.3
16	179.7	-

5.0 MISCELLANEOUS

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.

GOLDER ASSOCIATES LTD.

Dirka U. Prout, P. Eng.

Philip R. Bedell, P. Eng.
Principal

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

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PART B

PRELIMINARY FOUNDATION DESIGN REPORT
PROPOSED HIGHWAY 402 WB TRUCK LANES/
HIGHWAY 40 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 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, it is understood that the proposed westbound ETL will be accommodated by a new overpass structure over Highway 40. The approximate elevation of Modeland Road is 180.7 metres. The existing structures are founded on steel H-piles driven to rock.

6.2 Bridge Foundations

The subsurface conditions encountered in the boreholes put down during the investigation for the design of the existing Highway 40 Overpass structures to the south typically consisted of surficial deposits of generally soft peat and organic silt or clay to about elevation 177 metres. The organic deposits are underlain by stiff clayey silt which overlies generally stiff silty clay from about elevation 165 metres. Black shale bedrock of the Kettle Point formation was encountered at depths of 32 to 33 metres below the ground surface or at elevation 147 to 148 metres. Where encountered, the groundwater level in the boreholes was reported to be at about elevation 178 to 180 metres, or some 0.5 to 1.1 metres below ground surface.

Substantial deposits of peat and other organic soils were noted in previous investigations conducted by the MTO for this site. 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 E-S ramp, the existing overpass structure and previous Highway 40 widening and replaced with compacted inorganic clayey fill. Fill depths in excess of 4.6 metres can be expected in areas where organic materials were removed. However, it is possible that deposits of organic soils still remain in localized previously undeveloped areas. In addition, fills not reported in the boreholes but associated with the existing ramps, roads and overpass structures should be anticipated.

Based on the subsurface information noted above and the understanding that the proposed structure will be built with piled foundations similar to the existing Highway 402 overpass structures, 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 featured 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 layers below any fill or organic layers. If the new structure is to be founded on spread footings, drilling of several additional boreholes 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. Also, the additional boreholes should determine the strength of the overburden soil to a significant depth below proposed footing level. The clayey silt 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 founded at about elevations 174 to 178 metres on the stiff native clayey silt could be utilized. A factored geotechnical resistance of 200 kilopascals at the Ultimate Limit States (ULS) and a geotechnical resistance of 150 kilopascals at Serviceability Limit States (SLS) for an assumed 6 metre wide footing can be used for preliminary design purposes. These values may not be sufficient for the support of the bridge structure.

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).

Alternatively, perched abutments on compacted Granular A constructed within the approach embankment fill may be designed for a factored geotechnical resistance at ULS of 400 kPa and a

SLS value of 300 kPa. Although abutments can be perched on Granular A pads, it is preferred that the pier be founded on piles.

The SLS values provided are based on 25 millimetres of structure settlement. The 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 suggested 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.

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 clayey silt	angle of friction	28°
	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 which have been proposed for this site. 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 147 to 148 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 abutment additions. The design drawings indicated that embankment fills for the proposed Highway 402 WB truck lanes/Highway 40 overpass will encroach onto the embankments of the existing Highway 40 Overpass. Abutment piling for the existing structure adjacent to the fills for the new embankments will also be affected by downdrag loads. 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 is based on our experience with piles founded in similar soil conditions in the area and was estimated using methods outlined in the Canadian Foundation Engineering Manual. 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 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 (cohesive)	120	35
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, in accordance with the CHBDC:

- Select, free-draining granular fill meeting the specifications of Ontario Provincial Standard Specifications (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 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
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 8 metres in height. The new embankments will encroach onto the existing embankments for the Highway 40 overpass. Therefore, it may be necessary to bench the new embankments into the existing embankments in accordance with OPSD 202.010. 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. For fill slopes in excess of 8 metres in height, a 2 metre wide mid-height bench should be provided.

The topsoil and organic materials should be removed from within the area of the embankment and the exposed subgrade soils should be proofrolled and benched prior to fill placement.

Construction of the embankment widening above the prepared subgrade may be carried out using clean earth fill (in accordance with OPSS 212) or select subgrade material (in accordance with OPSS 1010) depending on material availability. All embankment fill should be placed in regular lifts and compacted.

Embankment settlements will be dependent on the extent of the additional filling required. Preliminary estimates of total embankment settlement, using granular or earth fill, are in the order of 350 millimetres at the centre and 50 millimetres at the toe assuming an embankment height of 7 metres and a width of 15 metres. Settlements could be reduced by up to 50 per cent by using lightweight fill. Primary settlements prior to paving can be further reduced by preloading, possibly in conjunction with wick drains and a drainage blanket. A more detailed settlement analysis should be conducted once the construction sequencing is known and the design has been finalized.

6.7 Excavations and Temporary Cut Slopes

Excavations for pile cap construction will extend through the surficial fill and/or organics and will encounter the clayey silt crust. Based on the subsurface conditions encountered in the boreholes, the base of the pile cap excavations for the central pier will likely encounter 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. 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 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.

GOLDER ASSOCIATES LTD.

Dirka U. Prout, P. Eng.

Philip R. Bedell, P. Eng.
Principal

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

DUP/PRB/FJH/jk
n:\active\2004\130000\041-130099 urs - gwp 3038 - sarnia\reports\041-130099-4 hwy 402 wb trucks\july 26 06 - prelim fdn inv report - hwy 402 wb truck lanes.doc

TABLE I

COMPARISON OF FOUNDATION ALTERNATIVES

Proposed Highway 402 WB Truck Lanes/Highway 40 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 allowable bearing resistance, potential for high consolidation settlements and possible need to remove organic materials if found	• 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 $80,000 assuming three 6 m wide by 1 m thick by 4 m long strip footings	• Even if mitigation in place, shallow foundations will still be affected by settlement of clayey silt and silty clay 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	• Not considered feasible due to low allowable bearing resistance, potential for high consolidation settlements and possible need to remove organic materials if found	• 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 • Due to space limitations, perched footings can only be utilized for the abutment piers	• More expensive than spread footings on native soils and less expensive than deep foundations • Approximate cost $105, 000 assuming three 4 metre wide strip footings	• Even if mitigation in place, shallow foundations will still be affected by settlement of clayey silt and silty clay 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 $220, 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.
3) Cost estimates based on information provided to Golder by others and 2006 Ontario Heavy Construction Costs (www.get-a-quote.net).
4) Cost estimates include labour, materials and equipment but exclude excavation and site preparation costs.

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

II. STRESS AND STRAIN

γ	shear strain
Δ	change in, e.g. in stress: $\Delta\ \sigma$
ε	linear strain
ε_v	volumetric strain
η	coefficient of viscosity
v	poisson's ratio
σ	total stress
σ'	effective stress ($\sigma' = \sigma$-u)
σ'_{vo}	initial effective overburden stress
$\sigma_1, \sigma_2, \sigma_3$	principal 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

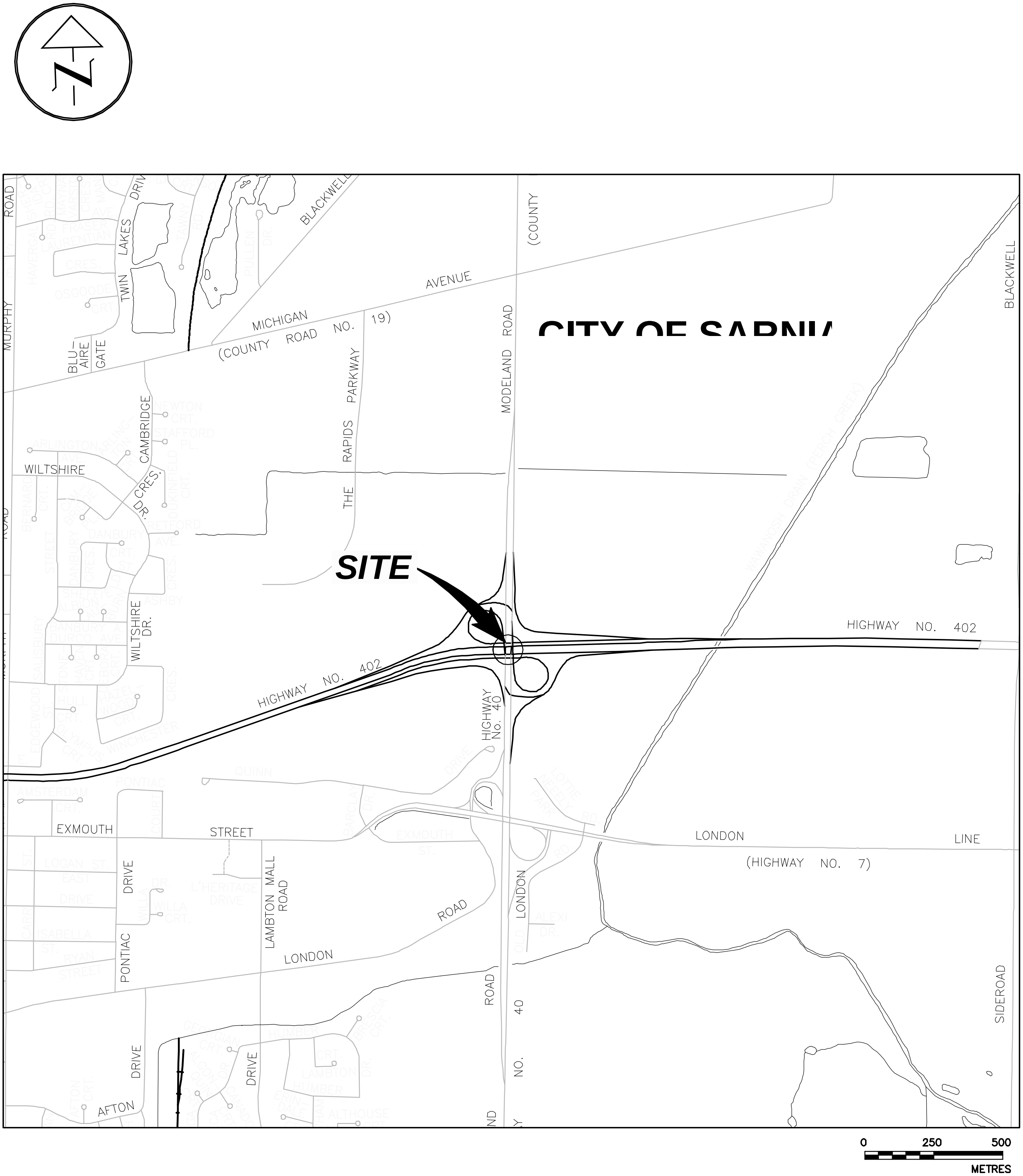
(c) Consolidation (one-dimensional)

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 $= \sigma'_p/\sigma'_{vo}$

(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 402 WB TRUCK LANES / HIGHWAY 40 OVERPASS WP No. 3038-03-00, HWY. 402
TITLE	SITE LOCATION PLAN

PROJECT No.	041-130099-4	FILE No.	041130099-4F001.
		SCALE	AS SHOWN
CADD	DCH	Aug 23/05	REV. 0
CHECK			FIGURE 1

Golder Associates
LONDON, ONTARIO

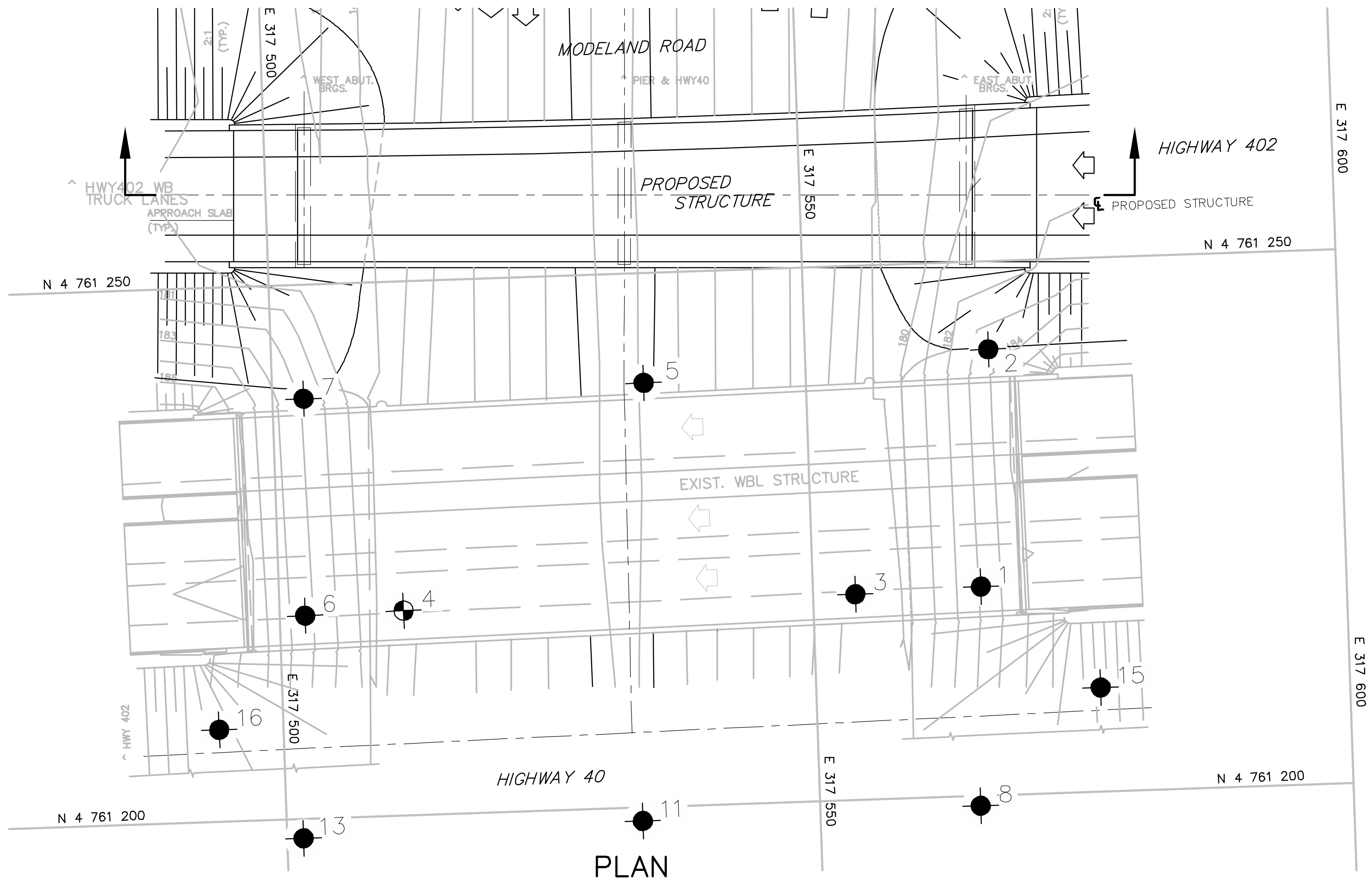
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN

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

HIGHWAY 402 WB TRUCK LANES / HIGHWAY 40 OVERPASS
BOREHOLE LOCATIONS & SOIL STRATA

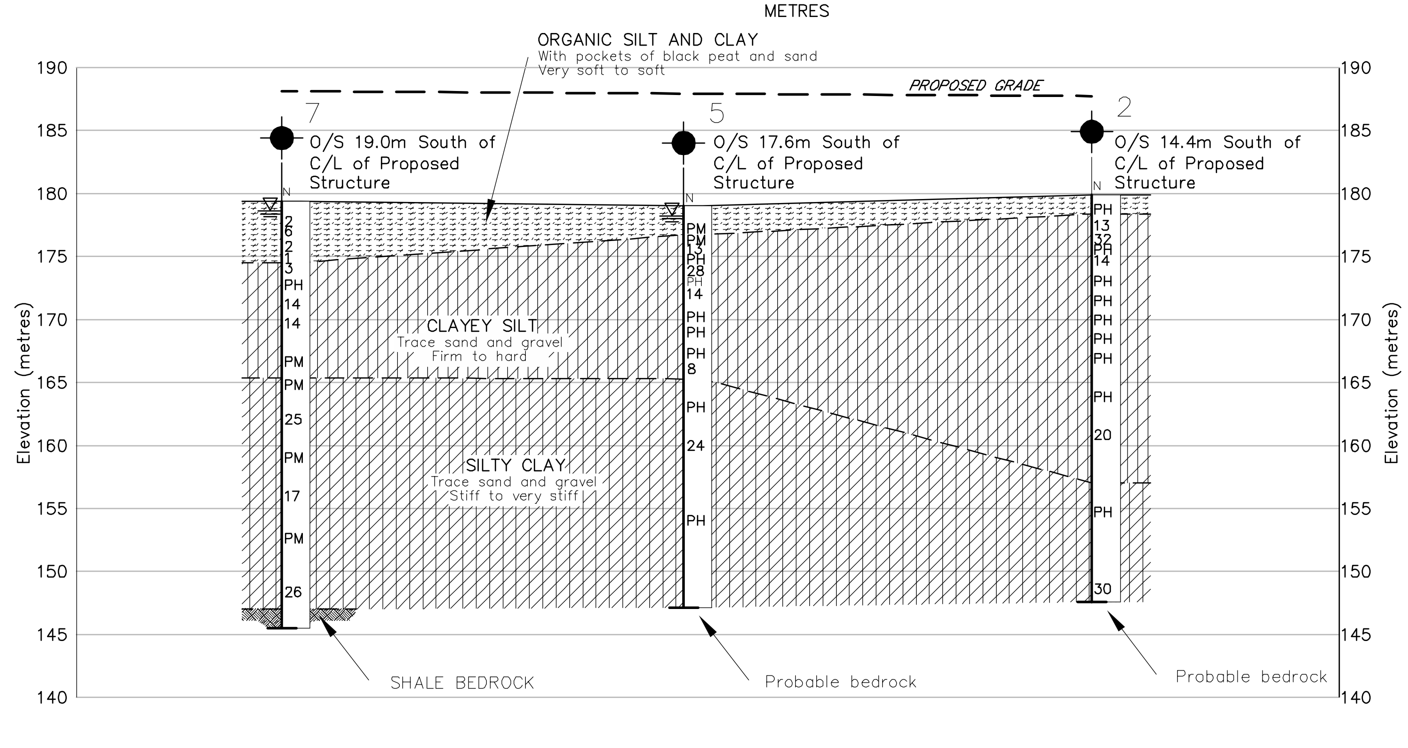
SHEET

Golder Associates
Golder Associates Ltd.
LONDON, ONTARIO, CANADA



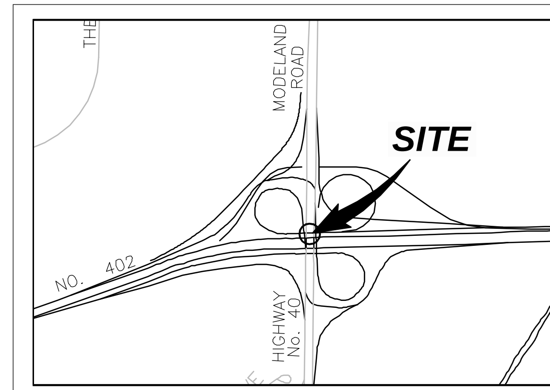
PLAN

0 10 20m
METRES



PROFILE ALONG CENTRELINE OF PROPOSED STRUCTURE

0 10 20m
METRES



KEY PLAN

0 250 500m
METRES

LEGEND

●	Borehole (Previous Investigation by Others)
◐	Borehole and Cone (Previous Investigation by Others)
N	Blows/0.3m (Std. Pen. Test, 475 j/blow)
▽	WL during drilling

No.	ELEVATION (metres)	CO-ORDINATES NORTH	EAST
1	180.2	4761217.5	317565.7
2	179.9	4761239.7	317567.2
3	180.6	4761217.2	317554.0
4	179.4	4761217.1	317511.6
5	179.0	4761237.7	317534.8
6	179.5	4761217.0	317502.4
7	179.4	4761237.3	317502.9
8	180.1	4761197.0	317565.0
11	179.0	4761196.7	317533.3
13	179.4	4761196.1	317501.5
15	179.8	4761207.7	317576.6
16	179.7	4761206.6	317494.0

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 suburface 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: HWY 402 WB TRUCK LANES/HWY 40 OVERPASS
GENERAL ARRANGEMENT
SITE: 14-581, DATED: JULY 2005
2) REFERENCE : DEPARTMENT OF HIGHWAYS - ONTARIO
ENTITLED: BOREHOLE LOCATIONS AND SOIL STRATA
DRAWING No. 70-11046A
W.P. No: 122-65-03, DATED : JULY 1970
3) BOREHOLE STRATIGRAPHY AS NOTED ON RECORDS OF BOREHOLES FOR MINISTRY OF TRANSPORTATION ONTARIO, REPORT GEOCRES No. 40J16-40

NO.	DATE	BY	REVISION

Geocres No. 40J16-70

HWY. No. 402		PROJECT NO.: 041-130099-4	
SUBM'D. –	CHKD:	DATE: Aug 23/05	
DRAWN: DCH	CHKD.	APPD.	DWG. 1

1 = 1 metric D size dwg 22" x 34" 11" x 17" plot half scale

041130099-4D001.dwg

APPENDIX A

RECORDS OF PREVIOUS BOREHOLES

DEPARTMENT OF HIGHWAYS- ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 1

FOUNDATION SECTION

JOB 70-11046 LOCATION STA. 62 + 88, 33 Ft. Lt. of Ø ORIGINATED BY A.K.B.
W.P. 122-65-03604 BORING DATE June 17-18, 1970 COMPILED BY A.K.B.
DATUM Geodetic BOREHOLE TYPE Auger CHECKED BY [illegible]

ELEV. DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	BLOWS/FOOT	ELEV. SCALE	SHEAR STRENGTH P.S.F.	WATER CONTENT %	BULK DENSITY γ P.C.F.	REMARKS GR.SA.SI.CL.
591.3	180.2m (0.0m) Ground Level								
0.0	Black peat, seams of organic sand, silt	1	SS	5	590				
588.	Sand clay, firm,	2	TW	PM					
5.5	178.5m (1.7m)	3	SS	21					
	Clayey silt, Traces of sand & gravel stiff to hard grey	4	SS	19	580				
		5	SS	24					
		6	SS	10					
		7	TW	PM	570	+ > 2000		133	
		8	SS	46	560				
		9	SS	22	550				
546.3	166.5m (13.7m)								
45.0	Silty clay, traces of sand and gravel stiff to hard grey	10	SS	15	540	+ >2000			
		11	TW	PM	530			128	
		12	SS	36	520				
		13	SS	25	510				
					490				
484.2	147.6m (32.6m) Probable bedrock								
107.1	End of borehole								

DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT
SHEAR STRENGTH P.S.F.: ○ UNCONFINED, ● QUICK TRIAXIAL, + FIELD VANE, × LAB VANE; 1000, 2000
LIQUID LIMIT — w_L; PLASTIC LIMIT — w_P; WATER CONTENT — w; WATER CONTENT %: 10, 20, 30

20 / 15 – 5 / 10 % STRAIN AT FAILURE

DEPARTMENT OF HIGHWAYS- ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 2 (69-F-104)

FOUNDATION SECTION

JOB 70-11046 | LOCATION STA 62+ 94 105' Lt | ORIGINATED BY A.P.

W.P. 122-65-03+04 | BORING DATE Nov. 18-20, 1969 | COMPILED BY A.K.B.

DATUM Geodetic | BOREHOLE TYPE Auger | CHECKED BY [illegible]

SOIL PROFILE

ELEV. DEPTH	DESCRIPTION
590.3	Ground Level 179.9m (0.0m)
	Clayey silt Traces of organics
585.3 5.0	178.4m (1.5m)
	Clayey silt with some sand & traces of gravel firm to very stiff
515.3 75.0	157.0m (22.9m)
	Silty clay with some sand traces of gravel very stiff
484.3	Prob. Bedrock 147.6m (32.3m)
106.0	End of borehole

SAMPLES

NUMBER	TYPE	BLOWS / FOOT	ELEV. SCALE	BULK DENSITY γ P.C.F.	REMARKS GR.SA.SI.CL.
1	TW	PH	590		
2	SS	13			
3	SS	32	580		2-20-47-31
4	TW	PH			
5	SS	14			
6	TW	PH	570	129	2-16-48-34
7	TW	PH			
8	TW	PH	560		
9	TW	PH			
10	TW	PH	550	134	1-16-47-36
11	TW	PH	540		
12	SS	20	530		
			520		
13	TW	PH	510	127	1-9-50-40
14	SS	30	490		

DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT

SHEAR STRENGTH P.S.F.
o UNCONFINED + FIELD VANE
● QUICK TRIAXIAL × LAB. VANE
1000 2000

LIQUID LIMIT —— w_L
PLASTIC LIMIT —— w_P
WATER CONTENT —— w

WATER CONTENT %
10 20 30

20
15 –o– 5 % STRAIN AT FAILURE
10

DEPARTMENT OF HIGHWAYS- ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 3

FOUNDATION SECTION

JOB 70-11046 | LOCATION STA 62 + 46, 33Ft. Lt of ℄ | ORIGINATED BY A.K.B.
W.P. 122-65-03-04 | BORING DATE June 19, 1970 | COMPILED BY A.K.B.
DATUM Geodetic | BOREHOLE TYPE Auger | CHECKED BY

ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FOOT	ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT; SHEAR STRENGTH P.S.F. (○ UNCONFINED, ● QUICK TRIAXIAL, + FIELD VANE, × LAB. VANE) 1000 2000	LIQUID LIMIT w_L, PLASTIC LIMIT w_P, WATER CONTENT w; WATER CONTENT % 10 20 30	BULK DENSITY γ P.C.F.	REMARKS GR.SA.SI.CL.
592.5	180.6m (0.0m) Ground Level									
0.0	Gravelly sand (Road Base)		1	SS	6	590		○		
586.5 6.0	178.8m (1.8m) Clayey silt with traces of sand & gravel Hard to stiff		2	SS	32			○		
			3	SS	39	580		○		
			4	SS	28			├○──┤		
570.0	173.7m (6.9m)		5	SS	13	570		├○──┤		
22.5	End of borehole									

20
15 -○- 5 % STRAIN AT FAILURE
10

DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 4

FOUNDATION SECTION

JOB 70-11046	LOCATION Sta. 61 + 11 34 FT. Lt. of Ø	ORIGINATED BY A.K.B.
W.P. 122-65-03504	BORING DATE June 4-8, 1970	COMPILED BY A.K.B.
DATUM Geodetic	BOREHOLE TYPE C.M.E. Auger	CHECKED BY

SOIL PROFILE

ELEV. / DEPTH	DESCRIPTION
588.6	179.4m (0.0m) Ground Level
0.0	Seams of black peat, organic sand silt and clay. numerous shells very soft
573.6	174.8m (4.6m)
15.0	Clayey silt, traces of sand & gravel, stiff to very stiff. Brown and Grey
543.6	165.7m (13.7m)
45.0	Silty clay, traces of sand and gravel stiff, grey
527.6	160.8m (18.6m)
61.0	End of borehole

SAMPLES

NUMBER	TYPE	BLOWS / FOOT	ELEV. SCALE	WATER CONTENT / LIMITS	BULK DENSITY γ P.C.F.	REMARKS GR.SA.SI.CL.
1	SS	1				0-90-(10)
2	TW	Pm	580		118	
3	SS	1		W 95 Wp 35 Wl 77		
4	TW	PH		W 172	73	
5	SS	11			77	
6	TW	PH	570		132 / 133	
7	SS	11				
8A	TW	PH	560			
8	SS	28				
9	TW	PH	550		133	
10	SS	5	540			
11	TW	PH	530		127 / 128	

DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT 20 40 60 80 100

SHEAR STRENGTH P.S.F. 1000 2000
o UNCONFINED + FIELD VANE
• QUICK TRIAXIAL x LAB. VANE

0.3040

LIQUID LIMIT — w_L
PLASTIC LIMIT — w_P
WATER CONTENT — w

WATER CONTENT % 10 20 30

% STRAIN AT FAILURE

DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 5

FOUNDATION SECTION

JOB 70-11046 LOCATION STA 61 + 87 100 Ft. Lt of Ø ORIGINATED BY T.P.
W.P. 122-65-03404 BORING DATE June 9-10, 1970 COMPILED BY A.K.B.
DATUM Geodetic BOREHOLE TYPE C.M.E. Auger CHECKED BY

SOIL PROFILE ELEV. DEPTH	DESCRIPTION	SAMPLES NUMBER	TYPE	BLOWS / FOOT	ELEV. SCALE	SHEAR STRENGTH P.S.F. / WATER CONTENT %	BULK DENSITY γ P.C.F.	REMARKS GR.SA.SI.CL.
587.4	179.0m (0.0m) Ground Level							
0.0	Organic Clay, Pockets of black Peat seams of sand	1	TW	PM		W 85		7-85-(8)
579.9	V. Soft 176.7m (2.3m)	2	TW	PM	580	W 148 W_L 220	82	
7.5	Clayey silt with traces of sand & Gravel--hard to firm Brown and Grey	3	SS	13		W_p 99		
		4	TW	PH		5375	133	
		5	SS	28				
		6	TW	PH	570		130 131	
		7	SS	14				
		8	TW	PH	560			
		9	TW	PH				
		10	TW	PH	550		127 129	
		11	SS	8				
542.4	165.3m (13.7m)							
45.0	Silty clay with traces of sand & gravel very stiff grey				540			
		12	TW	PH		> 2000	128	
					530			
		13	SS	24				
					520			
					510			
		14	TW	PH				4-18-35-43
					490			
482.9	147.2m (31.9m) Probably bedrock							
104.5	End of borehole							

DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT; SHEAR STRENGTH P.S.F.: o UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB. VANE (1000, 2000). LIQUID LIMIT — w_L, PLASTIC LIMIT — w_P, WATER CONTENT — w; WATER CONTENT % (10, 20, 30).

20 / 15 3 / 10 % STRAIN AT FAILURE

DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 6

FOUNDATION SECTION

JOB 70-11046	LOCATION STA 60 + 80, 34 Ft. Lt. of ℄	ORIGINATED BY T.P.
W.P. 122-65-03¢04	BORING DATE June 8-9, 1970	COMPILED BY A.K.B.
DATUM Geodetic	BOREHOLE TYPE C.M.E. Auger	CHECKED BY

DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT

SHEAR STRENGTH P.S.F. — o UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB. VANE (1000, 2000)

LIQUID LIMIT — w_L; PLASTIC LIMIT — w_P; WATER CONTENT — w; WATER CONTENT % (10, 20, 30)

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	BLOWS / FOOT	ELEV. SCALE	SHEAR STRENGTH / WATER CONTENT	BULK DENSITY γ P.C.F.	REMARKS GR.SA.SI.CL.
588.8	Ground Level 179.5m (0.0m)							
0.0	Organic Silt & Clay with pockets of black peat & sand numerous shells very soft to stiff	1	SS					0-89-(11)
		2	SS	5				
		3	SS	3	580			
		4	TW	PH		W 83 139; W_p 110; W_L 170	91, 72	
572.8 / 16.0	174.6m (4.9m) Clayey silt with traces of sand & gravel stiff to very stiff brown & grey	5	SS	11				
		6	SS	17	570			38-53-(9)
		7	SS	16				
		8	TW	PH			132	
		9	SS	17	560			
		10	TW	PH		+>2000	140, 141	
		11	SS	29	550			
543.8 / 45.0	165.8m (13.7m) Silty clay with traces of sand & gravel. Firm to very stiff grey	12	TW	PH	540		116, 115	
529.3 / 59.5	161.4m (18.1m) End of borehole	13	SS	28	530			

20 / 15 — 5 / 10 % STRAIN AT FAILURE

DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 7

FOUNDATION SECTION

JOB 70-11046
W.P. 122-65-03604
DATUM Geodetic

LOCATION STA 60 + 83, 100 FT Lt. of Ø
BORING DATE June 4-5, 1970
BOREHOLE TYPE Pendrill & Washboring BX casing

ORIGINATED BY A.K.B.
COMPILED BY A.K.B.
CHECKED BY [illegible]

ELEV. DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	BLOWS/FOOT	REMARKS
588.6	179.4m (0.0m) Ground level				
	Organic silt & clay numerous shells pockets of black peat and sand V. Soft to firm	1	SS	2	
		2	SS	6	0-95-(5)
		3	SS	2	
		4	SS	1	
572.6 / 16.0	174.5m (4.9m)	5	SS	3	
	Clayey silt with traces of sand & gravel Stiff to very stiff Brown and Grey	6	TW	PH	
		7	SS	14	
		8	SS	14	
		9	TW	PM	
542.6 / 46.0	165.4m (14.0m)	10	TW	PM	
	Silty clay with traces of sand and gravel stiff to very stiff grey	11	SS	25	
		12	TW	PM	
		13	SS	17	
		14	TW	PM	
		15	SS	26	
482.3 / 106.3	147.0m (32.4m) Shale Bedrock	16	RC	Rec. 92%	
477.3 / 111.3	145.5m (33.9m) End of borehole				

ELEV. SCALE: 580, 570, 560, 550, 540, 530, 520, 510, 490, 480

DYNAMIC PENETRATION RESISTANCE BLOWS/FOOT 20 40 60 80 100
SHEAR STRENGTH P.S.F. 1000 2000
○ UNCONFINED + FIELD VANE
● QUICK TRIAXIAL × LAB. VANE

LIQUID LIMIT — w_L
PLASTIC LIMIT — w_P
WATER CONTENT — w
WATER CONTENT % 10 20 30

W 94, W_P 210, W_L 320
W 198, W_P 106, W_L 109

BULK DENSITY γ P.C.F.: 130, 129, 126, 125, 125

REMARKS GR, SA, SI, CL

20
15 -+- 5 % STRAIN AT FAILURE
10

DEPARTMENT OF HIGHWAYS- ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 8

FOUNDATION SECTION

JOB 70-11046 LOCATION STA. 62 & 85, 34Ft. RT. of ℄ ORIGINATED BY A.K.B.
W.P. 122-65-03+04 BORING DATE June 19, 1970 COMPILED BY A.K.B.
DATUM Geodetic BOREHOLE TYPE Auger CHECKED BY [illegible]

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	BLOWS / FOOT	ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT; SHEAR STRENGTH P.S.F. (O UNCONFINED, ● QUICK TRIAXIAL, + FIELD VANE, × LAB. VANE) 1000 2000	LIQUID LIMIT w_L, PLASTIC LIMIT w_P, WATER CONTENT w; WATER CONTENT % 10 20 30	BULK DENSITY γ P.C.F.	REMARKS GR.SA.SI.CL.
590.8	180.1m (0.0m) Ground Level									
0.0	Sand with some					590				
586.8	black peat		1	SS	15					
4.0	178.9m (1.2m) Clayey silt with		2	SS	34					
	Traces of sand & Gravel.		3	SS	39	580				
	Hard to stiff		4	SS	12					
	Brown & Grey									
567.8	173.1m (7.0m)		5	SS	13	570				
23.0	End of borehole									

20
15 -◇- 5 % STRAIN AT FAILURE
10

DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 11

FOUNDATION SECTION

JOB 70-11046 | LOCATION STA 61 + 81, 34 Ft. Rt of ℄ | ORIGINATED BY T.P.

W.P. 122-65-03604 | BORING DATE June 16-17, 1970 | COMPILED BY A.K.B.

DATUM Geodetic | BOREHOLE TYPE C.M.E. Auger | CHECKED BY [illegible]

ELEV / DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	BLOWS/FOOT	BULK DENSITY γ P.C.F.
587.2	Ground Level 179.0m (0.0m)				
0.0	Black peat & Org. silt				
583.2 / 4.0	177.8m (1.2m)	1	SS	13	
	Clayey silt with traces of sand & gravel very stiff to firm brown & grey	2	TW	PH	136
		3	SS	19	
		4	TW	PH	131
		5	SS	12	
		6	TW	PH	131
		7	SS	10	
		8	TW	PH	138 / 140
		9	SS	31	
		10	TW	PH	131
540.2 / 47.0	164.7m (14.3m)				
	Silty clay with traces of sand & gravel: firm to very stiff, grey	11	SS	6	
		12	TW	PH	130 / 130
		13	TW	PH	
481.7	Probable bedrock 146.8m (32.2m)				
105.5	End of borehole				

ELEV SCALE: 580, 570, 560, 550, 540, 530, 520, 510, 490

DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT

SHEAR STRENGTH P.S.F.: o UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB. VANE — 1000, 2000

+ >2000

LIQUID LIMIT — w_L; PLASTIC LIMIT — w_P; WATER CONTENT — w

WATER CONTENT %: 10, 20, 30

REMARKS: GR. SA. SI. CL.

20 / 15 - 5 / 10 % STRAIN AT FAILURE

DEPARTMENT OF HIGHWAYS- ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 13

FOUNDATION SECTION

JOB 70-11046 LOCATION STA 60 + 76, 34 Ft. Rt of ¢ ORIGINATED BY T.P.
W.P. 122-65-03 & 04 BORING DATE June 18, 1970 COMPILED BY
DATUM Geodetic BOREHOLE TYPE C. M.E. Auger CHECKED BY

ELEV. DEPTH	SOIL PROFILE DESCRIPTION	STRAT PLOT	SAMPLE NUMBER	TYPE	BLOWS / FOOT	ELEV. SCALE	SHEAR STRENGTH P.S.F. (1000, 2000)	WATER CONTENT % (10, 20, 30)	BULK DENSITY γ P.C.F.	REMARKS GR.SA.SI.CL.
588.7	179.4m (0.0m) Ground Level									
0.0	Black peat with seams of organic silt clay and sand very soft to soft		1	SS	3					
			2	SS	2	580				0-91-6-3
			3	SS	4			100		
			4n	SS	5			242		
572.2	174.4m (5.0m)		5	SS	1					
16.5	Clayey silt with traces of sand & gravel Hard to stiff		6	TW	PM	570	3620		134	
			7	TW	PM					
			8	SS	27	560				
554.2	168.9m (10.5m)		9	TW	PM				139	
34.5	End of borehole									

DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT
○ UNCONFINED + FIELD VANE
● QUICK TRIAXIAL × LAB. VANE
LIQUID LIMIT w_L PLASTIC LIMIT w_P WATER CONTENT w

20 15 5 10 % STRAIN AT FAILURE

DEPARTMENT OF HIGHWAYS- ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 15

FOUNDATION SECTION

JOB 70-11046. LOCATION STA 64 + 40 ℄ ORIGINATED BY A.K.B.
W.P. 122-65-03&04 BORING DATE June 18, 1970 COMPILED BY A.K.B.
DATUM Geodetic BOREHOLE TYPE Auger CHECKED BY

ELEV. DEPTH	SOIL PROFILE DESCRIPTION	STRAT. PLOT	SAMPLES NUMBER	TYPE	BLOWS / FOOT	ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT; SHEAR STRENGTH P.S.F. ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB. VANE 1000 2000	LIQUID LIMIT w_L, PLASTIC LIMIT w_P, WATER CONTENT w; WATER CONTENT % 10 20 30	BULK DENSITY γ P.C.F.	REMARKS GR. SA. SI. CL.
589.6	Ground Level 179.8m (0.0m)									
588.3	Black Organics					▼				
1.5	179.3m (0.5m) Clayey silt with traces of sand & gravel Hard to stiff Brown to grey		1	SS	46					
			2	SS	27					
			3	SS	32	580				
			4	SS	13					
			5	SS	11	570				
567.3	172.9m (6.9m)									
22.5	End of borehole									

20 15 5 10 % STRAIN AT FAILURE

DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 16

FOUNDATION SECTION

JOB 70-11046 LOCATION STA 60 + 00 ¢ ORIGINATED BY T.P.
W.P. 122-65-03&04 BORING DATE June-19,-1970 COMPILED BY
DATUM Geodetic BOREHOLE TYPE C.M.E. Auger CHECKED BY

ELEV DEPTH	SOIL PROFILE DESCRIPTION	STRAT PLOT	SAMPLE NUMBER	SAMPLE TYPE	BLOWS/FOOT	ELEV SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS/FOOT; SHEAR STRENGTH P.S.F. (○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB. VANE) 1000 2000	LIQUID LIMIT w_L, PLASTIC LIMIT w_P, WATER CONTENT w; WATER CONTENT % 10 20 30	BULK DENSITY γ P.C.F.	REMARKS GR.SA.SI.CL.
589.5	179.7m (0.0m) Ground Level									
0.0	black peat & organic clay, soft									
585.5	178.5m (1.2m)		1	SS	4					
4.0	Clayey silt with traces of sand & gravel		2	SS	15					
			3	TW	PH	580	+> 2000			
			4	TW	PH				131	
			5	SS	11					
	Stiff		6	SS	7	570				
563.5	171.8m (7.9m)		7	TW	PH				131 133	
26.0	End of borehole									

20
15-○-5 % STRAIN AT FAILURE
10