

**PRELIMINARY FOUNDATION INVESTIGATION
AND DESIGN REPORT
PROPOSED FRONT STREET/CNR OVERPASS
STRUCTURE WIDENING
HIGHWAY 402, GWP 3038-03-00
AGREEMENT NUMBER 3005-A-000394**

Submitted to:

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PART A

**PRELIMINARY FOUNDATION INVESTIGATION REPORT
PROPOSED FRONT STREET/CNR OVERPASS
STRUCTURE WIDENING
HIGHWAY 402, GWP 3038-03-00
AGREEMENT NUMBER 3005-A-000394**

1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by URS Canada Inc. (URS) on behalf of the Ministry of Transportation, Ontario (MTO) to carry out preliminary foundation investigations at various sites along Highway 402 in conjunction with GWP 3038-03-00 which extends from the Bluewater Bridge Authority plaza easterly for 16 kilometres to Lambton Road 26 (Mandaumin Road) in Sarnia, Ontario. This report addresses widening and rehabilitation of the structures conveying Highway 402 over Front Street and the adjacent Canadian National Railway (CNR) right of way.

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 general arrangement drawings for the existing Highway 402 overpass structures and proposed widenings and rehabilitation works. It has been proposed to widen the existing westbound structure in order to provide two additional lanes and a shoulder on the south side. The new structure will be founded on piles and will feature semi-integral abutments. The remainder of the existing westbound structure will be rehabilitated. The eastbound structure will also be rehabilitated with the construction of new approach slabs and semi-integral abutments.

2.0 SITE DESCRIPTION

The project area covered by this report is located on Highway 402, approximately 1.1 kilometres east of the east end of the Blue Water Bridge over the St. Clair River, at the crossings of Front Street and former CN Rail tracks immediately east of Front Street in Sarnia, Ontario. The subject site contains two structures, one for the eastbound lanes and one for the westbound lanes. Both structures were constructed in 1981 and are concrete with post tensioned deck slabs and five spans. Each structure has two through lanes and one ramp lane. The eastbound and westbound structure numbers are 14-363/1 and 14-363/2, respectively. The site location is shown on Figure 1.

The approximate elevation of the existing bridge decks is about 187.5 metres. The approximate elevations of Front Street and the former railway right of way are at 177.5 and 178.3 metres, respectively.

Site photographs are provided in Appendix B.

3.0 INVESTIGATION PROCEDURES

No site specific intrusive field work was carried out for this investigation. However, the report utilizes the results of the original investigation which was carried out for the existing structures. The results of that investigation is contained within MTO Report Geocres No. 40J16-58 entitled “Foundation Investigation Report for Front Street Overpass and CNR Overhead, W.P. 347-65-02/03, Site 14-363, Highway 402, District 1, Chatham” dated August 1977. At that time, nine boreholes were put down at the site of the structures. The boreholes were drilled and sampled to depths of 8.5 to 41.1 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. 14-363A-2 dated August 7, 1977. Boreholes 1, 2, 3, 4, 6, 7, 8, 9 and 11 were drilled during the period April 28, 1977 to May 11, 1977.

The inferred borehole locations are shown in plan on Drawing 1. The subsurface conditions encountered in the boreholes are shown on the Soil Strata Drawing, Drawing 1, in metric units. The records of the boreholes drilled for the existing structures are provided in Appendix A in their original imperial units and format. Corresponding depths 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 of southwestern Ontario¹ known as the Huron Fringe. Geological information indicates that the general soil conditions for the area consist of glacial lacustrine deposits overlying deep lacustrine till deposits.

The surficial glaciolacustrine deposits represent the shoreline and near shores of the 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 elevation 130 and 140 metres. The bedrock belongs to the Kettle Point Formation and consists of 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 generally consist of about 3 metres of very loose to compact sand which are underlain by as much as 14 metres of hard to firm clayey silt. The clayey silt is underlain by a 20 metre thick layer of stiff silty clay over about 2 metres of black sand which

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

overlies black shale bedrock. Four of the boreholes were advanced into the shale bedrock which was encountered some 37 to 40 metres below the ground surface or between elevations 139 and 141 metres. The Record of Borehole sheets for the original investigation did not note the presence of fill and/or topsoil. However, fill and topsoil materials associated with the existing embankments are now present at this site. Much of the embankment fill is reported to be fly ash and bottom ash from the Lambton Generating Station.

The locations and elevations of the borings are shown on the attached Drawing 1 and the interpreted stratigraphical profile is provided on Drawing 2. 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 Sand

All of the previous boreholes encountered surficial layers of sand. The sand layers were about 1.8 to 3.0 metres thick and extended to about elevation 175.0 to 176.8 metres.

The sand had standard penetration test N values of 2 to 26 blows per 0.3 metres and water contents of 11 to 32 per cent.

4.2.2 Clayey Silt

Beneath the surficial deposits, from elevations 175.0 to 176.8 metres, all of the boreholes encountered an extensive deposit of clayey silt. The clayey silt deposit typically had a stiff to hard crust extending to about elevation 173 to 174 metres. Where fully penetrated, the clayey silt deposits were about 13 to 17 metres thick.

The clayey silt crust had standard penetration test N values of 9 to 32 blows per 0.3 metres with an average of 19 blows per 0.3 metres indicating a very stiff consistency. The clayey silt below the crust had standard penetration test N values from 5 to 55 blows per 0.3 metres with an average of 12 blows per 0.3 metres. The shear strength of the stratum was measured by in situ vane testing as well as unconfined and quick triaxial testing of thin wall samples obtained from boreholes 4, 6, 8 and 9. In situ vane testing indicated undrained shear strengths ranging from 31 to greater than 105 kilopascals (kPa) with an average of 64 kPa. The vane sensitivities ranged from 1.4 to 3.3 with an average of 2.0. Confined and unconfined triaxial testing yielded shear strength values of 34 to 115 kPa. In situ and laboratory testing confirmed a firm to very stiff consistency.

The water contents of the clayey silt samples of the crust ranged from about 10 to 20 per cent with an average water content of 16 per cent. Below the crust, the water contents of the clayey silt samples ranged from about 12 to 28 per cent with an average water content of 22 per cent.

The clayey silt deposits were of low plasticity with average plastic and liquid limits of 15 and 30 per cent, respectively, with an average plasticity index of 15.

The clayey silt had bulk densities ranging from 2.0 to 2.1 megagrams per cubic metre. The results of five grain size analyses indicated 26 to 44 per cent clay, 39 to 46 per cent silt, 14 to 30 per cent sand and up to 6 per cent sand.

4.2.3 Silty Clay

Silty clay with trace amounts of sand was encountered in boreholes 1, 2, 4 and 7 between elevations 161.5 and 161.8 metres after penetrating the clayey silt deposits. All four boreholes fully penetrated the silty clay deposits after exploring them for some 18 to 21 metres.

In situ vane testing carried out in the silty clay materials indicated undrained shear strengths ranging from 55 to 101 kPa with an average of 74 kPa. The vane sensitivities ranged from 1.3 to 2.0 with an average of 1.6. The shear strengths estimated from confined and unconfined triaxial testing conducted on thin wall samples from borehole 4 were 69 to 79 kPa. The laboratory testing indicated that the silty clay materials have a stiff consistency. The standard penetration testing profile within the silty clay deposit was complete in borehole 1 only with limited testing done in the remaining three deep boreholes. The silty clay had N values ranging from 9 to 16 blows per 0.3 metres, with an average of 13 blows per 0.3 metres.

The water contents of the silty clay samples ranged from about 17 to 36 per cent with an average water content of 27 per cent. The silty clay deposits were of intermediate plasticity with average plastic and liquid limits of 20 and 39 per cent, respectively, with an average plasticity index of 19.

The silty clay materials had a bulk density of 1.9 megagrams per cubic metre based on testing of a single sample.

4.2.4 Sand

Black sand was encountered beneath the silty clay layers in boreholes 1, 2, 4 and 7 at elevation 141.9 to 143.8 metres. The sand layers were 1.5 to 1.9 metres thick.

4.2.5 Bedrock

The bedrock surface was encountered some 37 to 40 metres below ground surface, or at elevation 139.2 to 141.6 metres, in boreholes 1, 2, 4 and 7. All four boreholes were terminated in bedrock. The Record of Borehole for boreholes 2 and 7 indicate refusal on an inferred bedrock surface. The top 1.7 to 2.1 metres of the bedrock was cored in boreholes 1 and 4 and it was identified as

black shale of the Kettle Point formation. The total rock core recoveries reported on the borehole logs were 77 to 100 per cent indicating good to excellent recovery.

4.3 Groundwater Conditions

Groundwater was encountered in eight of the nine boreholes during drilling. The encountered groundwater levels were reported to be 0.8 to 2.3 metres below ground surface, or at about elevation 176 to 178 metres. No groundwater level was established for borehole 7. The encountered water levels are shown on the attached Record of Borehole sheets in Appendix A and are summarized below.

BOREHOLE NUMBER	GROUND SURFACE ELEVATION (m)	ENCOUNTERED GROUNDWATER LEVEL ELEVATION (m)
1	178.6	177.5
2	178.5	177.5
3	178.1	176.4
4	178.1	176.4
6	178.1	175.9
7	178.2	-
8	178.6	177.4
9	178.8	178.0
11	178.5	176.2

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.

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.

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PART B

**PRELIMINARY FOUNDATION DESIGN REPORT
PROPOSED FRONT STREET/CNR OVERPASS STRUCTURE
WIDENING
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 the existing Highway 402 bridges in the area. Based on the currently available information, it is understood that the proposed ETL will be accommodated at this site by widening the westbound structure in order to provide two westbound lanes and a shoulder on the south side of the structure. The remainder of the existing westbound structure and the eastbound structure will be rehabilitated. The existing five span structures for Highway 402 have decks at about elevation 187.5 metres. Front Street and the railway are at approximately elevation 177.5 and 178.3 metres, respectively. The bridges are reportedly founded on concrete filled steel pipe piles 325 millimetres in diameter which were driven to bedrock.

6.2 Bridge Widening Foundations

The subsurface conditions encountered in the boreholes put down during the investigation for the existing structures typically consisted of surficial deposits of sand to about elevation 176 metres which are underlain by a generally very stiff clayey silt crust to about elevation 174 metres. The underlying firm to very stiff clayey silt extends to about elevation 162 metres. Below the clayey silt, a deposit of stiff silty clay extends to top of an approximately 2 metre thick sand layer which lies between the silty clay and the underlying bedrock. The surface of the shale bedrock was encountered at about elevation 139 to 141 metres. The groundwater level was encountered in the boreholes at about elevation 176 to 178 metres, or some 0.8 to 2.3 metres below ground surface.

Based on the subsurface information noted above and the understanding that the proposed widening will be built with pile foundations similar to the existing structures, consideration may be given to supporting the proposed widening 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 these options are compared in Table I.

6.3 Shallow Foundations

Based on the subsurface conditions, high groundwater levels and heavy loading of the proposed structure widening, the use of spread footings bearing on the surficial sands or upper portion of the clayey silt on the clayey silt crust are not considered appropriate. Higher bearing values could be achieved for footings for the abutment by perching spread footings on granular pads in the embankments. However, shallow footings at the pier locations can only be provided in the surficial deposits which offer limited bearing resistance. Therefore, the use of perched abutment footings were not considered feasible. Further, the construction of granular pads for the abutment widenings would necessitate additional traffic protection and would require the appropriate disposal of the excavated fly ash and bottom ash waste materials. The clayey silt soils have a limited bearing capacity and will be subject to consolidation settlements. In addition, the existing structure is on piles and differential settlement is likely to occur if the widening is founded on shallow footings.

6.4 Deep Foundations

Steel tube piles driven closed ended to practical refusal on bedrock and filled with concrete are considered suitable to support the abutments and piers for the proposed widening. H-piles may be a suitable option for use with integral abutments. While steel H-piles would more easily penetrate the clayey deposits and minimize the amount of disturbance during driving, steel tube piles are recommended because they will provide the same horizontal deformation characteristics as the existing piles. Noting that semi-integral abutments, which are tolerant of still foundation elements, have been proposed, the use of driven steel tube 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 325 millimetre diameter steel tube piles filled with concrete and driven to refusal in the shale bedrock at about elevation 140 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 practices (Ontario Provincial Standard Drawing (OPSD) 3302.000) 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 since the SLS value will be much higher than 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)

As will be discussed in a later section, only very minor modification of the existing approach embankments will be required and, as a result, it is not anticipated that the new piles will be subject to negative skin friction loads.

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. As semi-integral abutments have been proposed, there may also be 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 granular and cohesive soils and the pier piles will be driven through the cohesive deposits. The resistance to lateral loading may be based on the following assessed values:

SOIL TYPE	HORIZONTAL RESISTANCE VALUES (kN) PER PILE	
	Factored ULS	SLS
Embankment fill (granulars)	110	40
Embankment fill (cohesive)	160	65
Clayey silt and silty clay deposits	180	90

Group action for lateral loading should be considered when the pile spacing in the direction of the loading is less than six to eight pile diameters. Group action can be evaluated by reducing the coefficient of horizontal subgrade reaction in the direction of loading by a reduction factor R as follows:

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

6.4.4 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 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 1.2 horizontal to 1 vertical extending up and back from the rear face of the footing (Case ii from Commentary on CHBDC Figure C6.9.1(l)).
- For Case i, the pressures are based on the proposed embankment fill materials and the following parameters (unfactored) may be assumed for granular fill:

Soil unit weight: 21 kN/m³

Coefficients of lateral earth pressure:

Active, K_a 0.33
At rest, K_o 0.50

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

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

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

It should be noted that the above design parameters assume level backfill and ground surface behind the wall. Other aspects of the abutment granular backfill requirements with respect to subdrains and frost taper should be in accordance with OPSD 3501.00 and OPSD 3504.00.

6.6 Embankments

The proposed widening will extend into the area between the existing bridge structures where the configuration of the existing embankment fills are such that only minimal filling will be required to match the final grades of the future embankment. The existing embankment fills are approximately 8.5 to 9.0 metres in height. The additional embankment fill should be benched into the existing embankments in accordance with OPSD 202.010. For fill slopes in excess of 8 metres in height, a 2 metre wide mid-height bench should be provided. Embankment side slopes formed no steeper than 2 horizontal to 1 vertical are considered suitable for this site.

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 with loose thicknesses not exceeding 300 millimetres and should be compacted to at least 95 per cent of the material's standard Proctor maximum dry density. Since the existing embankments are expected to be modified only slightly, the resulting settlement is expected to be minimal. The magnitude of the settlements can be determined at the time of final design.

It should be noted that since much of the existing embankments are comprised of fly and bottom ash waste materials, proper disposal of any excess excavated materials will be required.

6.7 Excavations and Temporary Cut Slopes

Excavations for pile cap construction will extend through topsoil materials and the surficial sandy deposits and will encounter the clayey silt crust. Based on the subsurface conditions encountered in the boreholes, the base of the pile cap excavations may 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 excavation. Sumps should be maintained outside of the actual footing limits. Surface water runoff should be directed away from the excavations at all times. The appropriate Non Standard Special Provision (NSSP) should be included in the contract documents.

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

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

7.0 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.

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TABLE I

COMPARISON OF FOUNDATION ALTERNATIVES

Proposed Front Street/CNR Overpass Structure Widening
 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	<ul style="list-style-type: none"> Not considered feasible due to low allowable bearing resistance and differential settlement between existing and proposed structure 	<ul style="list-style-type: none"> Cost 	<ul style="list-style-type: none"> Potential for differential settlement as existing structure founded on piles Time and cost of settlement mitigation measures Even if mitigation measures adopted, settlement of shallow foundations could still take place 	<ul style="list-style-type: none"> Expected to be less expensive than deep foundation options Approximate cost \$126,000 assuming six 5 m wide strip footings 	<ul style="list-style-type: none"> Even if mitigation in place, shallow foundations will still be affected by settlement of clayey silt and silty clay deposits Proposed bridge structure is settlement intolerant to existing structure
Spread footings perched on granular pad in embankments	<ul style="list-style-type: none"> Not considered feasible due to low allowable bearing resistance and differential settlement between existing and proposed structure 	<ul style="list-style-type: none"> Cost Greater bearing resistance compared to spread footings on native clayey silt 	<ul style="list-style-type: none"> Potential for differential settlement as existing structure founded on piles 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 	<ul style="list-style-type: none"> More expensive than spread footings on native soils and less expensive than deep foundations Approximate cost \$231, 000 assuming six 5 metre wide strip footings 	<ul style="list-style-type: none"> Even if mitigation in place, shallow foundations will still be affected by settlement of clayey silt and silty clay deposits Proposed bridge structure is settlement intolerant to existing structure

FOUNDATION OPTION	FEASIBILITY	ADVANTAGES	DISADVANTAGES	ESTIMATED COSTS	RISKS/CONSEQUENCES
Steel pipe pile foundations founded on shale bedrock	<ul style="list-style-type: none"> Feasible for support of all foundation elements 	<ul style="list-style-type: none"> High bearing resistance Negligible settlement 	<ul style="list-style-type: none"> Possibility of 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 Adjacent existing abutment piles may be subjected to downdrag loads 	<ul style="list-style-type: none"> Approximate cost \$418, 000 More expensive than shallow foundations but preferred technical solution 	<ul style="list-style-type: none"> Possible pile tip damage if tip is not suitably protected while driving in rock
H-pile foundations founded on shale bedrock	Feasible for support of all foundation elements	<ul style="list-style-type: none"> High bearing resistance Negligible settlement 	<ul style="list-style-type: none"> Possibility of 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 Adjacent existing abutment piles may be subjected to downdrag loads 	<ul style="list-style-type: none"> Approximate cost \$399, 000 More expensive than shallow foundation May be more suitable for use with integral abutments as more flexible than a concrete filled pipe pile 	<ul style="list-style-type: none"> Possible pile tip damage if tip is not suitably protected while driving in rock

NOTES: 1) Costs are very preliminary estimates and are intended to provide a comparison between alternatives rather than actual construction costs.

2) Table to be read in conjunction with accompanying report.

LIST OF ABBREVIATIONS

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

I. SAMPLE TYPE

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

III. SOIL DESCRIPTION

(a) Cohesionless Soils

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

II. PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:

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

(b) Cohesive Soils

Consistency	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

Dynamic Cone Penetration Resistance; N_d :

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

PH: Sampler advanced by hydraulic pressure

PM: Sampler advanced by manual pressure

WH: Sampler advanced by static weight of hammer

WR: Sampler advanced by weight of sampler and rod

Piezo-Cone Penetration Test (CPT)

A electronic cone penetrometer with a 60° conical tip and a project end area of 10 cm² pushed through ground at a penetration rate of 2 cm/s. Measurements of tip resistance (Q_t), porewater pressure (PWP) and friction along a sleeve are recorded electronically at 25 mm penetration intervals.

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
ν	poisson's ratio
σ	total stress
σ'	effective stress ($\sigma' = \sigma - u$)
σ'_{vo}	initial effective overburden stress
$\sigma_1, \sigma_2, \sigma_3$	principal 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
I_p	plasticity index $= (w_l - w_p)$
w_s	shrinkage limit
I_L	liquidity index $= (w - w_p) / I_p$
I_C	consistency index $= (w_l - w) / I_p$
e_{max}	void ratio in loosest state
e_{min}	void ratio in densest state
I_D	density index $= (e_{max} - e) / (e_{max} - e_{min})$ (formerly relative density)

(b) Hydraulic Properties

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

(c) Consolidation (one-dimensional)

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

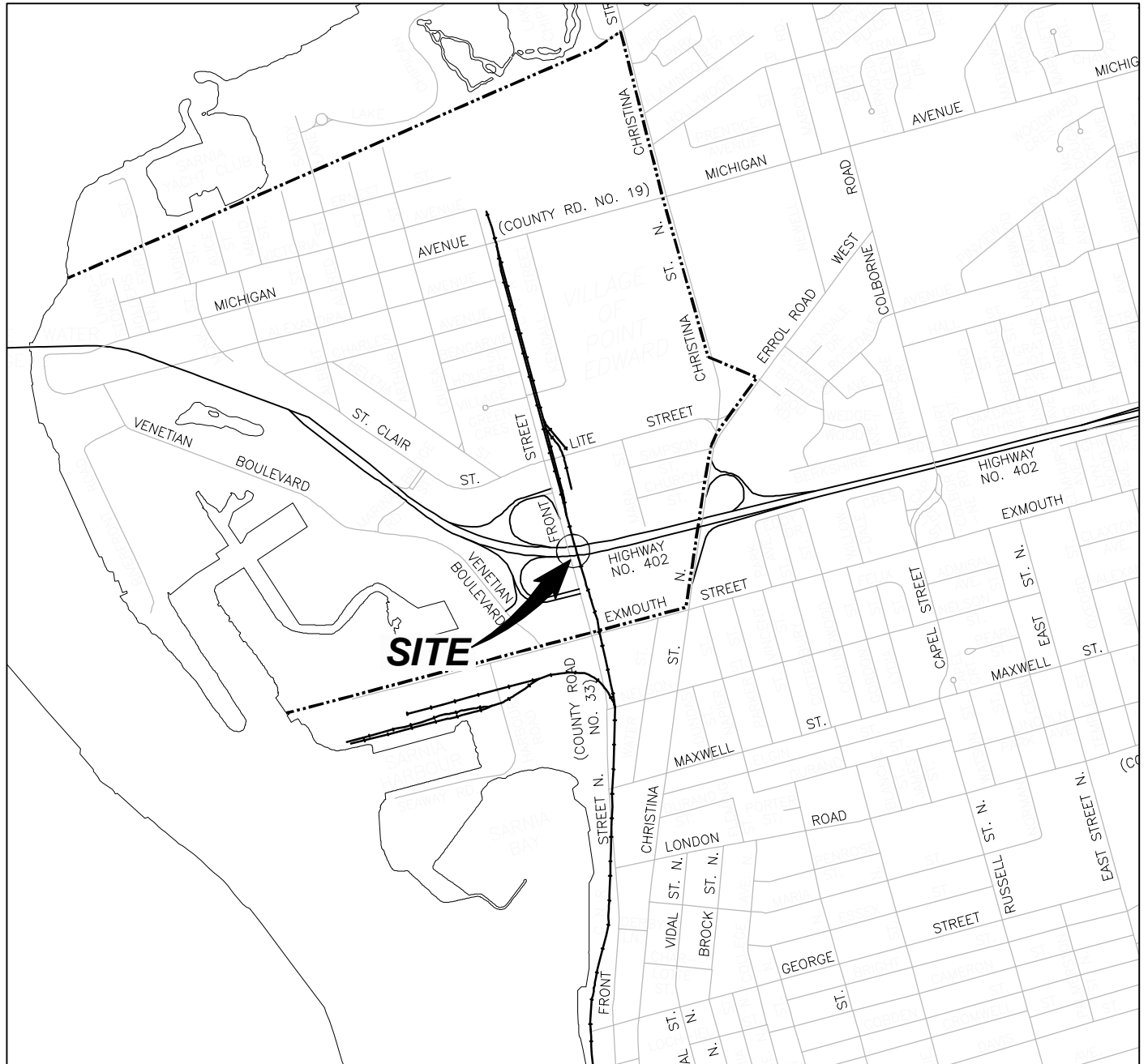
(d) Shear Strength

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

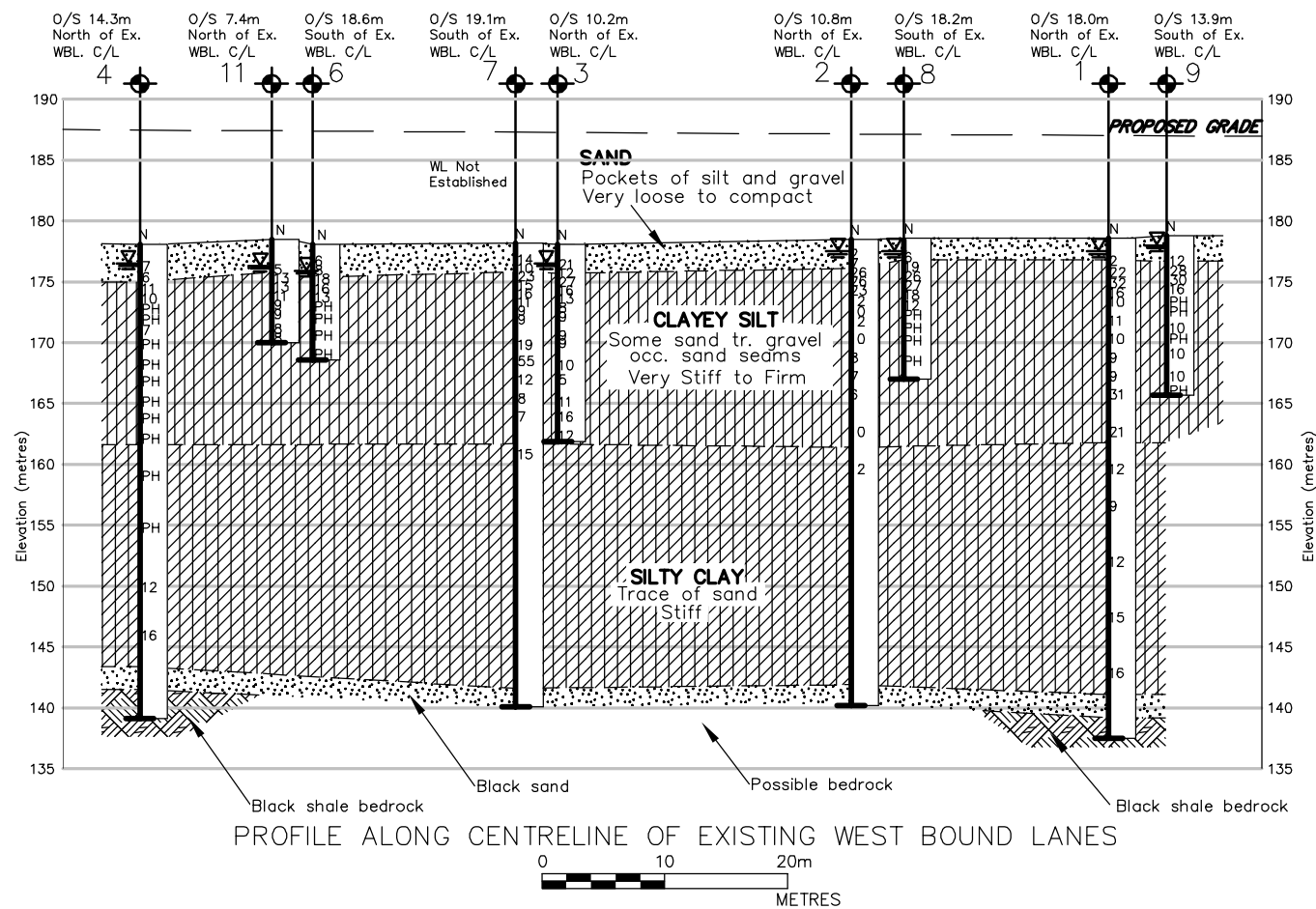
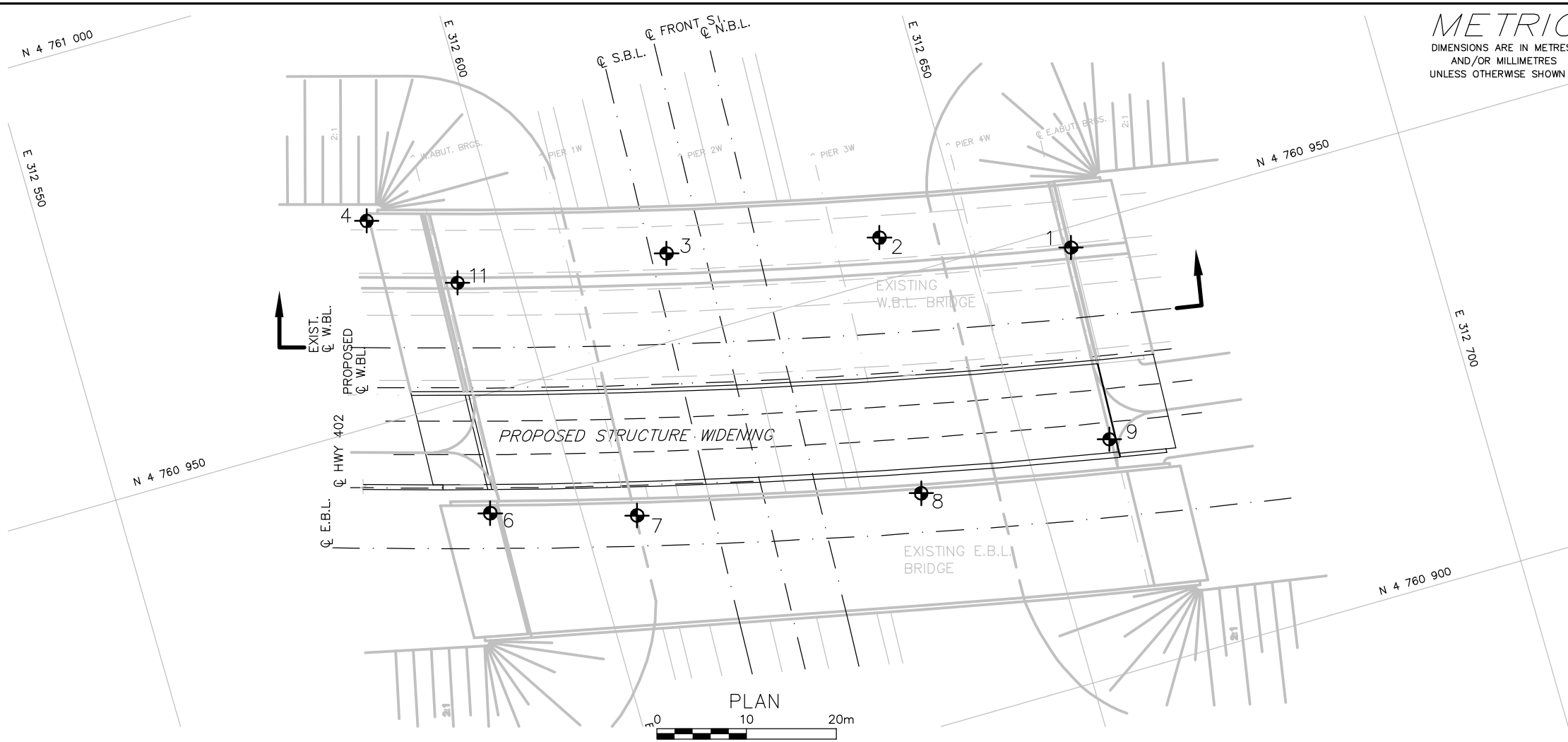
- Notes:**
- 1 $\tau = c' + \sigma' \tan \phi'$
 - 2 shear strength = (compressive strength)/2
 - * density symbol is ρ . Unit weight symbol is γ where $\gamma = \rho g$ (i.e. mass density x acceleration due to gravity)



CITY OF SARNIA



PROJECT		FRONT STREET/CNR OVERPASS WP No. 3038-03-00 HWY. 402			
TITLE		SITE LOCATION PLAN			
 Golder Associates LONDON, ONTARIO		PROJECT No. 041-130099-3		FILE No. 041130099-3D001.	
		CADD	DCH	Aug 05/05	SCALE AS SHOWN
		CHECK			REV. 0
		FIGURE 1			



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

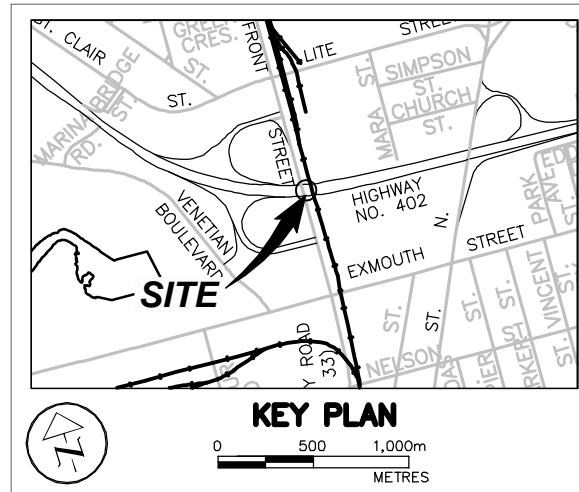
**FRONT STREET/CNR OVERPASS
STRUCTURE WIDENING**

BOREHOLE LOCATIONS & SOIL STRATA

SHEET



Golder Associates Ltd.
LONDON, ONTARIO, CANADA



LEGEND

- Borehole (Previous Investigation by Others)
- Seal
- Piezometer
- Blows/0.3m (Std. Pen. Test, 475 j/blow)
- WL during drilling

No.	ELEVATION (metres)	CO-ORDINATES	
		NORTH	EAST
1	178.6	4 760 947.7	312 661.1
2	178.5	4 760 954.7	312 640.7
3	178.1	4 760 959.6	312 617.2
4	178.1	4 760 972.4	312 585.8
6	178.1	4 760 937.0	312 590.1
7	178.2	4 760 933.2	312 605.9
8	178.6	4 760 925.8	312 637.3
9	178.8	4 760 925.8	312 659.3
11	178.5	4 760 962.9	312 593.7

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

- DRAWING SUPPLIED BY: URS G.A PLAN
ENTITLED: FRONT STREET/CNR OVERPASS HWY 402 WBL
- BOREHOLE STRATIGRAPHY AS NOTED ON RECORDS OF BOREHOLES FOR
MINISTRY OF TRANSPORTATION ONTARIO, REPORT GEOCREs No. 40J16-58

NO.	DATE	BY	REVISION
Geocres No. 40J16-66			
HWY. No.	402	PROJECT NO.:	041-130099-3
SUBM'D.	-	CHKD:	DATE: Aug 9/05
DRAWN: BG/DCH		CHKD.	APPD.
			DWG. 1

APPENDIX A
RECORDS OF PREVIOUS BOREHOLES

RECORD OF BOREHOLE NO 1

WP 347-65-02/03

LOCATION Co-ords N 15 619 207 E 1 025 870

ORIGINATED BY EJS

DIST 1 HWY 402

BORING DATE April 28 1977

COMPILED BY EJS

DATUM Geodetic

BOREHOLE TYPE Hollow Stem Auger

CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER ELEV	DYNAMIC CONE PENETRATION RESISTANCE PLOT					LIQUID LIMIT w_L PLASTIC LIMIT w_p WATER CONTENT w WATER CONTENT % w_p w w_L	UNIT WEIGHT γ	REMARKS
ELEV DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	N VALUES		20	40	60	80	100			
585.9	Ground Level													
0.0	Sand Pockets of Silt and Gravel Very Loose 178.6m (0.0m)		1	SS	2	580								9.77 (14)
579.9	Loose 176.8m (1.8m)		2	SS	22									0 24.46 30
6.0	Clayey Silt Some Sand Trace of Gravel Occasional Sand Seams Very Stiff to Firm		3	SS	32									
			4	SS	16									
			5	SS	10									
			6	SS	11									
			7	SS	10									
			8	SS	9									
			9	SS	9									
			10	SS	31									
			11	SS	21									0 14.42 44
530.9	161.8m (16.8m)													
55.0	Silty Clay Trace of Sand		12	SS	12									
			13	SS	9									
			14	SS	12									
			15	SS	15									
481.9	146.9m (31.7m)													
104.0	Continued													

20
15 ϕ 5 % STRAIN AT FAILURE
10

RECORD OF BOREHOLE No 1 cont

WP 347-65-02103

LOCATION Co-ords N 15 519 207 E 1 025 870

ORIGINATED BY EJS

DIST 1 HWY 402

BORING DATE April 28, 1977

COMPILED BY EJS

DATUM Geodetic

BOREHOLE TYPE Hollow Stem Auger

CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER	DYNAMIC CONE PENETRATION RESISTANCE PLOT					LIQUID LIMIT — w_L PLASTIC LIMIT — w_p WATER CONTENT — w			UNIT WEIGHT γ	REMARKS
ELEV DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	WV VALUES		20	40	60	80	100	w_p	w	w_L		
481.9	Continued					ELEV										
104.0	Silty Clay Trace of Sand Stiff					480										
			16	SS	16	470										
146.9	146.9m (31.7m)															
123.0	Black Sand					460										
139.2	139.2m (39.4m)															
129.3	Black Shale Bedrock															
137.4	137.4m (41.1m)		17	RC	77%											
135.0	End of Borehole															

RECORD OF BOREHOLE NO 2

WP 347-65-02/03

LOCATION Co-ords N 15 619 230 E 1 025 803

ORIGINATED BY EJS

DIST 1 HWY 402

BORING DATE May 5, 1977

COMPILED BY PJS

DATUM Goodale

BOREHOLE TYPE Hollow Stem Augers

CHECKED BY _____

[illegible]

15 $\frac{20}{10}$ 5 % STRAIN AT FAILURE

15 ϕ -5 20 % STRAIN AT FAILURE

RECORD OF BOREHOLE NO 3

WP 367-65-02/03

LOCATION Co-ords N 15 619 246 E 1 025 726

ORIGINATED BY PJS

DIST 1 HWY 402

BORING DATE May 6, 1977

COMPILED BY PJS

DATUM Geodetic

BOREHOLE TYPE Hollow Stem Auger

CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER ELEV	DYNAMIC CONE PENETRATION RESISTANCE PLOT					LIQUID LIMIT w_L PLASTIC LIMIT w_p WATER CONTENT w w_p — w — w_L WATER CONTENT % 10 20 30	UNIT WEIGHT γ	REMARKS
ELEV DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	N VALUES		20	40	60	80	100			
584.2	Ground Level													
0.0														
576.2	178.1m (0.0m)		1	SS	21	580								
8.0	Sand Pockets of Silt and Sand Loose to Compact		2	SS	12									
	175.6m (2.4m)		3	SS	27									
	Clayey Silt Some Sand		4	SS	16									
	Trace of Gravel		5	SS	13									
	Occasional Sand Seams		6	SS	8	570								
	Very stiff to firm		7	SS	9									
			8	SS	9									
			9	SS	9	560								
			10	SS	10									
			11	SS	5	550								
			12	SS	11									
			13	SS	16	540								
			14	SS	12									
533.2	161.9m (16.2m)													
53.0	End of Borehole													

RECORD OF BOREHOLE NO 4

WP 347-65-02/03

LOCATION Corcoran H 15 619 288 R 1 025 623

ORIGINATED BY EJS

DIST 1 HWY A02

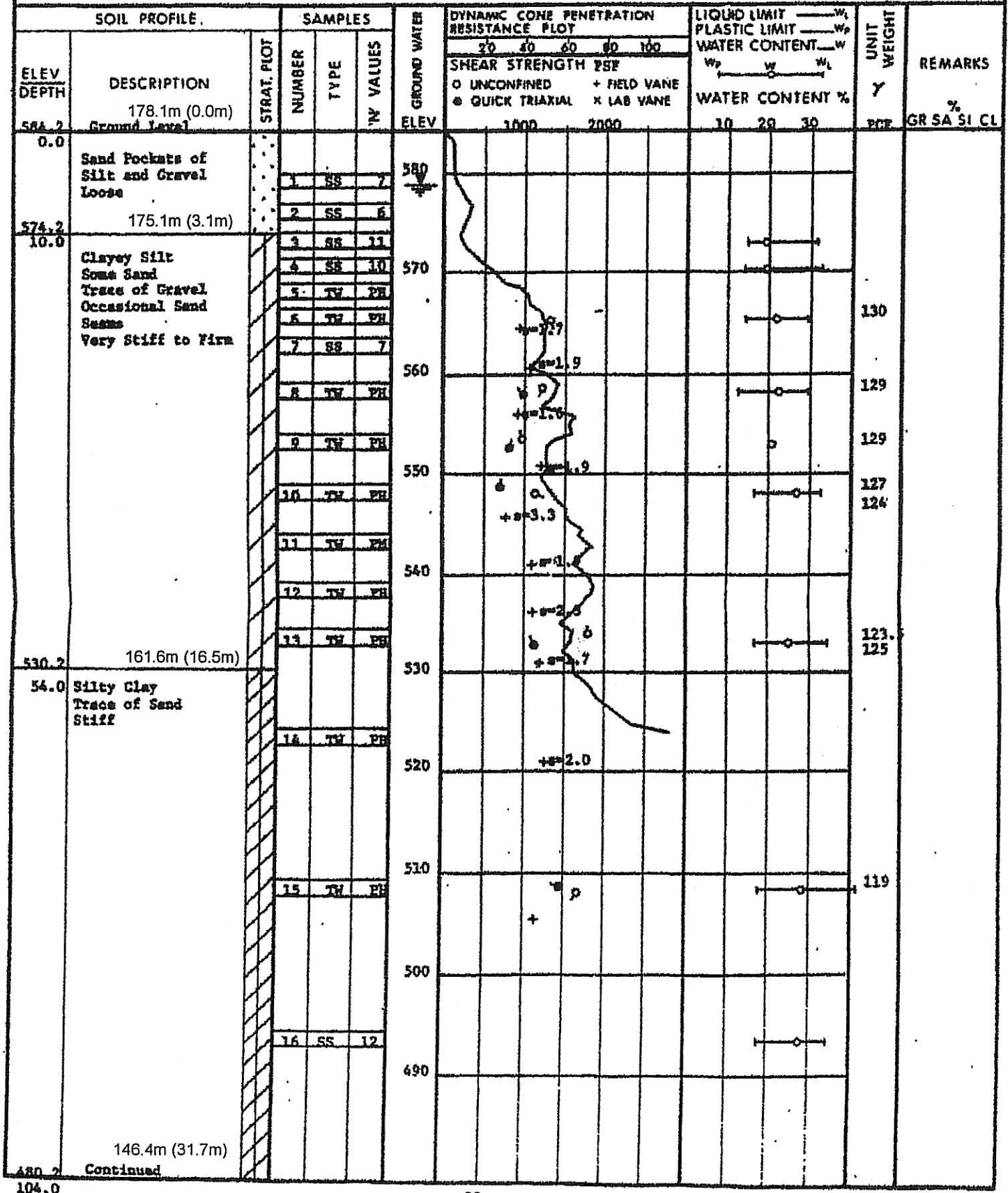
BORING DATE May 2, 1977

COMPILED BY EJS

DATUM Geodetic

BOREHOLE TYPE Hollow Stem Auger

CHECKED BY



WP 347-65-02103 LOCATION Co-ordin W 15 629 288 E 1 025 623 ORIGINATED BY PJS
DIST 1 HWY 402 BORING DATE MAY 2, 1977 COMPILED BY PJS
DATUM Geodetic BOREHOLE TYPE Roller Stem Auger CHECKED BY _____

20
15 ϕ 5 % STRAIN AT FAILURE
10

RECORD OF BOREHOLE NO 6

WP 347-65-02/03

LOCATION Co-ords N 15 619 172 E 1 025 637

ORIGINATED BY PJS

DIST 1 HWY 402

BORING DATE May 10, 1977

COMPILED BY PJS

DATUM Geodetic

BOREHOLE TYPE Hollow Stem Auger

CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER ELEV	DYNAMIC CONE PENETRATION RESISTANCE PLOT					LIQUID LIMIT w_L PLASTIC LIMIT w_p WATER CONTENT w			UNIT WEIGHT γ PCE	REMARKS
ELEV DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	'N' VALUES		20	40	60	80	100	w_p	w	w_L		
584.4	178.1m (0.0m) Ground Level															
0.0	Sand, Pockets of Silt and Gravel Loose to Compact		1	SS	6	580										0 35 60 5
			2	SS	8											
575.4			3	SS	18											
9.0	Clayey Silt Some Sand Trace of Gravel Occasional Sand Seams Very Stiff to Firm		4	SS	16											131 134 132 130
			5	SS	13	570										
			6	TM	PH											
			7	TM	PH											
			8	TM	PH											
553.4	168.7m (9.5m)		9	TM	PH	560										
31.0	End of Borehole															

WP 347-65-02103 LOCATION Co-ordin N 15 619 156 E 1 025 689 ORIGINATED BY RJS
DIST 1 HWY 602 BORING DATE May 10, 1977 COMPILED BY RJS
DATUM Geodetic BOREHOLE TYPE Hollow Stem Auger CHECKED BY

[illegible]

20
15 ϕ 5 % STRAIN AT FAILURE
10

RECORD OF BOREHOLE No 7 cont

WP 347-65-02103 LOCATION Co-ords N 15 619 156 E 1 025 689 ORIGINATED BY EJS
 DIST 1 HWY 402 BORING DATE May 10, 1977 COMPILED BY EJS
 DATUM Geodetic BOREHOLE TYPE Hollow Stem Auger CHECKED BY _____

SOIL PROFILE			SAMPLES			GROUND WATER	DYNAMIC CONE PENETRATION RESISTANCE PLOT					LIQUID LIMIT w_L PLASTIC LIMIT w_p WATER CONTENT w			UNIT WEIGHT γ	REMARKS
ELEV DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	N° VALUES		20	40	60	80	100	w_p	w	w_L		
480.0	146.5m (31.7m) Continued					480										GR SA SI CL
104.0	Silty Clay Trace of Gravel Stiff					470										
454.5	141.6m (36.6m)															
120.0	Black Sand															
459.5	140.1m (38.1m)					460										
125.0	End of Borehole Probable Bedrock															
	NOTE Water Level not established															

RECORD OF BOREHOLE NO 8

WP 347-65-02/03 LOCATION Co-ords N 15 619 135 E 1 025 792 ORIGINATED BY EJS
 DIST 1 HWY A02 BORING DATE May 9, 1977 COMPILED BY EJS
 DATUM Geodetic BOREHOLE TYPE Hollow Stem Auger CHECKED BY _____

SOIL PROFILE			SAMPLES			GROUND WATER ELEV	DYNAMIC CONE PENETRATION RESISTANCE PLOT					LIQUID LIMIT w_L PLASTIC LIMIT w_p WATER CONTENT w			UNIT WEIGHT γ pcf	REMARKS
ELEV DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	N' VALUES		20	40	60	80	100	w_p	w	w_L		
585.8	Ground Level															
0.0	Sand, Pockets of Silt and Gravel															
579.8	Loose 176.7m (1.8m)		1	SS	6											1 38 56 5
6.0	Clayey Silt		2	SS	19											
	Some Sand		3	SS	26											
	Trace of Gravel		4	SS	27											
	Occasional		5	SS	18											
	Sand Some		6	SS	12											
	Very Stiff to Firm		7	TV	PH										131	
			8	TV	PH										131	
			9	TV	PH										129	
552.8	168.5m (11.6m)		10	TV	PH											
33.0	End of Borehole															

RECORD OF BOREHOLE NO 9

WP 347-55-02/03 LOCATION Co-ords N 15 619 135 E 1 025 864 ORIGINATED BY EJS
 DIST 1 HWY 402 BORING DATE May 9, 1977 COMPILED BY EJS
 DATUM Geodetic BOREHOLE TYPE Hollow Stem Auger CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER	DYNAMIC CONE PENETRATION RESISTANCE PLOT					LIQUID LIMIT w_L PLASTIC LIMIT w_p WATER CONTENT w			UNIT WEIGHT γ	REMARKS
ELEV	DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE		20	40	60	80	100	w_p	w	w_L		
586.5		Ground Level														
0.0		Sand, Pockets of Silt and Gravel														
579.5		Compact 176.6m (2.1m)		1	SS	12										
7.0		Clayey Silt, Some Sand		2	SS	28										
		Trace of Gravel		3	SS	30										
		Occasional Sand		4	SS	16										
		Seams		5	TV	PH										
		Very Stiff To Firm		6	TV	PH										
				7	SS	10										
				8	TV	PH										
				9	SS	10										
				10	SS	10										
				11	TV	PH										
563.5		165.7m (13.1m)														
43.0		End of Borehole														

RECORD OF BOREHOLE NO 11

WP 347-65-02/03 LOCATION Co-ords N 15 619 257 E 1 025 669 ORIGINATED BY PJS
 DIST 1 HWY 402 BORING DATE May 11, 1977 COMPILED BY PJS
 DATUM Geodetic BOREHOLE TYPE Hollow Stem Auger CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER ELEV	DYNAMIC CONE PENETRATION RESISTANCE PLOT					LIQUID LIMIT w_L PLASTIC LIMIT w_p WATER CONTENT w			UNIT WEIGHT γ	REMARKS
ELEV DEPTH	DESCRIPTION	STRAT. PLT	NUMBER	TYPE	W VALUES		20	40	60	80	100	w_p	w	w_L		
585.5	Ground Level															
0.0	Sand, Rocks of Silt and Gravel Loose to Compact		1	SS	1	580										
576.5	175.7m (2.7m)		2	SS	13											6 68 23 3
9.0	Clayey Silt Some Sand Trace of Gravel Occasional Sand Stems Very Stiff to Firm		3	SS	13											
			4	SS	11											
			5	SS	9	570										
			6	SS	9											
			7	SS	8											
			8	SS	8	560										
557.5	169.9m (8.5m)															
28.0	End of Borehole															

APPENDIX B
SITE PHOTOGRAPHS

FRONT STREET/CNR OVERPASS (OVERPASS REPLACEMENT)



Photo 1 : Front Street/CNR overpass looking north from centre line of Front Street



Photo 2 : Front Street/CNR overpass looking south from centre line of former CNR line