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**PRELIMINARY FOUNDATION INVESTIGATION
AND DESIGN REPORT
CNR OVERPASS STRUCTURE REPLACEMENT
HIGHWAY 402, GWP 3038-03-00
AGREEMENT NUMBER 3005-A-000394**

Submitted to:

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TABLE OF CONTENTS

<u>SECTION</u>	<u>PAGE</u>
PART A - PRELIMINARY FOUNDATION INVESTIGATION REPORT	
1.0 INTRODUCTION.....	1
2.0 SITE DESCRIPTION.....	2
3.0 INVESTIGATION PROCEDURES.....	3
4.0 GENERAL SITE GEOLOGY AND STRATIGRAPHY.....	4
4.1 Geology	4
4.2 Site Stratigraphy.....	4
4.2.1 Surficial Organics and Silty Sand or Sandy Silt.....	5
4.2.2 Clayey Silt	5
4.2.3 Silty Clay	6
4.2.4 Bedrock	6
4.3 Groundwater Conditions.....	6
5.0 MISCELLANEOUS.....	8
PART B - PRELIMINARY FOUNDATION DESIGN REPORT	
6.0 ENGINEERING RECOMMENDATIONS.....	9
6.1 General	9
6.2 Bridge Foundations	9
6.3 Shallow Foundations.....	10
6.4 Deep Foundations.....	10
6.4.1 Geotechnical Axial Resistance – Driven Steel Piles.....	10
6.4.2 Downdrag Load (Negative Skin Friction).....	10
6.4.3 Resistance to Lateral Loads	11
6.4.4 Frost Protection	12
6.5 Lateral Earth Pressures.....	12
6.6 Embankments	13
6.7 Excavations and Temporary Cut Slopes	14
7.0 CLOSURE	15

In Order
Following
Page 15

TABLE OF CONTENTS CONTINUED

TABLE I - Comparison of Foundation Alternatives

FIGURE 1 - Site Location Map

DRAWING 1 - C.N.R. Overpass Structure Replacement, Borehole Locations and Soil
Strata

LIST OF ABBREVIATIONS

LIST OF SYMBOLS

APPENDIX A - Records of Previous Boreholes

APPENDIX B - Site Photographs

PART A

**PRELIMINARY FOUNDATION INVESTIGATION REPORT
CNR OVERPASS STRUCTURE REPLACEMENT
HIGHWAY 402, GWP 3038-03-00
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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 preliminary foundation investigations at various sites along Highway 402 in conjunction with GWP 3038-03-00 which extends from the Bluewater Bridge Authority plaza east for 16 kilometres to Lambton Road 26 (Mandaumin Road) in Sarnia, Ontario. This report addresses the Canadian National Rail (CNR) overpass structure for the westbound lanes (Site 14-337/2).

The purpose of the foundation investigation is to determine the subsurface conditions at the site of the proposed new bridge which will replace the current westbound structure 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. The work was carried out in accordance with our Quality Control of TPM Services Plan, Agreement No. 3005-A-000394, dated May 2004.

The report is provided as part of the preliminary design phase of the project. URS provided Golder with a general arrangement drawing for the existing Highway 402 overpass structures and the future westbound replacement structure.

2.0 SITE DESCRIPTION

The project area covered by this report is located on Highway 402 at the crossing of the former CN Rail 1.3 kilometres to the west of Modeland Road in Sarnia, Ontario. The east and westbound structure numbers are 14-337/1 and 14-337/2, respectively. The site location is shown on Figure 1.

Residential developments are situated west of the former railway. East of the right-of-way, the land use is predominantly agricultural adjacent to Highway 402.

The existing westbound Highway 402 structure is a three span, prestressed concrete box girder bridge which was erected in 1976 and rehabilitated in 2000. The overall width is 13.1 metres with a roadway width of about 12.1 metres. The existing westbound structure accommodates two lanes of traffic.

The existing bridge decks are at elevation 190.3 metres and the former railway elevation is about 183 metres. The former railway right-of-way has been abandoned and currently serves as a pathway. In the future, it is designated to become the Rapids Parkway.

Site photographs are provided in Appendix B.

3.0 INVESTIGATION PROCEDURES

No site specific field work was carried out for this investigation. However, the report utilizes the results of the original investigation for the existing structures. The investigation was conducted by the Department of Highways Ontario, a precursor to the MTO. The results were presented in Geocres report No. 40J16-39 entitled "Foundation Investigation Report for the Proposed CNR Overhead of Highway 402, Near Sarnia, District No. 1 (Chatham), W.O. 70-11045 - W.P. 346-65-01 & 02", dated July 17, 1970. At that time, twelve boreholes were put down at the site of the proposed bridges. The boreholes were drilled and sampled to depths of 6 to 35 metres with dynamic cone penetration testing carried out in the upper portion of five of the boreholes and at one additional location. The boreholes and dynamic cone penetration testing locations are shown on the Department of Highways, Ontario Drawing No. 70-11045A dated July 21, 1970. The boreholes were drilled between November 1969 and June 1970 for two possible alignments for Highway 402.

The approximate borehole locations are shown in plan on Drawing 1. The subsurface conditions encountered in the boreholes drilled along the east and west abutments are shown on Drawing 1 in metric units. The results 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 known as the St. Clair Clay Plain. Geological information indicates that the general soil conditions for the area consist of glacial lacustrine deposits overlying deep lacustrine till deposits.

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

4.2 Site Stratigraphy

The detailed subsurface soil and groundwater conditions encountered in the boreholes together with the results of the field and laboratory testing are shown on the Record of Borehole sheets 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 1.5 metres of organics and silty sand or sandy silt which are underlain by up to 17 metres of hard to firm clayey silt, with a 2 to 7 metre thick crust. Below the clayey silt, a 14 to 22 metre thick deposit of generally firm to stiff silty clay was encountered extending to bedrock. Three of the boreholes were advanced to the shale bedrock which was encountered some 34 metres below ground surface at elevation 147 metres.

Locations and elevations of the borings are shown on the attached Drawing 1 and 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 Organics and Silty Sand or Sandy Silt

All of the previous boreholes encountered surficial layers of organics and very loose to loose silty sand or sandy silt. The surficial deposits were about 1.4 to 2.9 metres thick and extended to about elevation 178.3 to 179.3 metres.

The silty sand or sandy silt had standard penetration test N values of 2 to 7 blows per 0.3 metres and water contents of from 15 to 81 per cent. The organic sandy silt/silty sand had average plastic and liquid limits of 17 and 24 per cent, respectively, with an average plasticity index of 7. Grain size distribution testing was carried out on the surficial organics and sandy silt materials encountered in boreholes 3, 5 and 7 and the results are reported on the Record of Boreholes.

4.2.2 Clayey Silt

Beneath the surficial deposits, all of the boreholes encountered an extensive deposit of clayey silt. Based on the original investigation data, the clayey silt deposit had a stiff to hard crust ranging from about 2 to 7 metres in thickness, extending to about elevation 174 to 175 metres. Where fully penetrated, the clayey silt deposits were about 9 to 17 metres thick and extended to elevation 168 to 161 metres.

The clayey silt crust had standard penetration test N values of 9 to 43 blows per 0.3 metres. The clayey silt below the crust had standard penetration test N values from 6 to 30 blows per 0.3 metres. In situ vane testing indicated undrained shear strengths ranging from 38 to greater than 105 kilopascals (kPa), with an average shear strength of about 65 kPa. The vane sensitivities shown on the Record of Boreholes ranged from 1.6 to 2.4 with an average sensitivity of 2.1.

The water contents of the clayey silt samples of the crust ranged from about 14 to 22 per cent with an average water content of 18 per cent. Below the crust, the water contents of the clayey silt samples ranged from about 17 to 31 per cent with an average water content of 21 per cent. The clayey silt deposits had an average plastic and liquid limits of 16 and 30 per cent, respectively, with an average plasticity index of 14.

The clayey silt had bulk densities ranging from 1.9 to 2.2 megagrams per cubic metre. A single grain size determination was carried out on a sample of the clayey silt in borehole 9. The grain size analysis indicated 38 per cent clay, 46 per cent silt and 16 per cent sand. A boulder was encountered in the base of the clayey silt deposit in borehole 3.

4.2.3 Silty Clay

Where the clayey silt deposits were penetrated in boreholes 1, 2, 3, 8 and 9, silty clay with traces of sand and gravel were encountered. At boreholes 1, 2 and 3, the silty clay materials were about 14 to 22 metres thick and extended to bedrock at about elevation 145 to 147 metres. The silty clay was only 3 metres thick in borehole 8 and borehole 9 was terminated in the silty clay deposit after exploring it for about 4 metres. In situ vane testing carried out in the silty clay materials indicated undrained shear strengths ranging from 38 to 67 kPa, with an average shear strength of about 52 kPa. The vane sensitivities shown on the Record of Boreholes ranged from 1.8 to 3.5 with an average sensitivity of 2.6. Laboratory testing indicated that the silty clay materials are normally consolidated or slightly over consolidated with shear strengths of 40 to 45 kPa. The laboratory testing indicated that the silty clay materials have a firm to stiff consistency. In borehole 3, the surface of the silty clay had a single standard penetration N value of 34 blows per 0.3 metres, indicating a hard consistency.

The water contents of the silty clay samples ranged from about 22 to 35 per cent with an average water content of 26 per cent. The silty clay deposits had average plastic and liquid limits of 19 and 38 per cent, respectively, with an average plasticity index of 19.

The silty clay materials had bulk densities ranging from 1.9 to 2.0 megagrams per cubic metre. Two grain size analyses carried out on samples of the silty clay from borehole 1 indicated 21 and 46 per cent clay, 44 and 53 per cent silt, 8 and 24 per cent sand and about 2 per cent gravel.

4.2.4 Bedrock

The bedrock surface was encountered some 33.6 to 33.8 metres below ground surface, or at elevation 147 metres, in boreholes 1, 2 and 3. The top 1.2 to 1.6 metres of the bedrock was cored in boreholes 1 and 3 and it was identified to be black shale of the Kettle Point formation. The rock core recoveries reported on the borehole logs were 85 to 95 per cent.

4.3 Groundwater Conditions

Water levels were noted in three of the boreholes during the June 1970 drilling and are reported to be 0.2 to 0.4 metres below ground surface, or at about elevation 180 metres. These levels are shown on the attached Record of Borehole sheets in Appendix A and are summarized below.

BOREHOLE NUMBER	GROUND SURFACE ELEVATION (m)	ENCOUNTERED WATER LEVEL ELEVATION (m)
2	180.59	180.41
8	180.66	180.43
12	180.17	179.79

It should be noted that the reported groundwater level does not indicate the long term stable groundwater elevation and that the groundwater level is subject to seasonal fluctuations.

5.0 MISCELLANEOUS

This report was written by Mr. Azmi M. Hammoud, P.Eng. an Associate with Golder Associates Ltd. under the direction of Mr. Philip R. Bedell, P.Eng., who is the Project Manager and 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.

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AMH/PRB/FJH/cr
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PART B

**PRELIMINARY FOUNDATION DESIGN REPORT
CNR OVERPASS STRUCTURE REPLACEMENT
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 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 ETL will be accommodated by replacing the existing 13.1 metre wide westbound structure with a new 25.6 metre wide structure. The existing three span structures for Highway 402 have decks at elevation 190.4 metres and the former railway elevation is 183 metres. The bridges are founded on HP 12x74 piles extending to bedrock and the existing bases of the pile caps are at elevation 186.5 metres at the abutments and at elevation 179.7 metres at the piers. The new westbound structure will also have three spans with a deck elevation of 190.0 metres. It is anticipated that the new bridge will also be founded on piles which extend to bedrock.

6.2 Bridge Foundations

The subsurface conditions encountered in the boreholes put down during the investigation for the existing structures typically consisted of surficial deposits of organics and silty sand or sandy silt to about elevation 179 metres which are underlain by a hard to firm clayey silt crust to about elevation 170 metres. The underlying hard to firm clayey silt extends to about elevation 161 metres. Below the clayey silt, a deposit of firm to stiff silty clay extends to the surface of the shale bedrock encountered at about elevation 147 metres. The groundwater level was encountered in the boreholes at about elevation 180 metres, or some 0.5 to 0.6 metres below ground surface.

Based on the subsurface information noted above and the understanding that the proposed structure will be built with piled foundations similar to the existing structures, consideration may be given to supporting the proposed bridge on deep foundations such as steel piles driven to practical refusal on bedrock. A comparison of foundation alternatives is given on Table I. The recommended foundation alternative is steel H-piles driven to rock.

6.3 Shallow Foundations

Based on the subsurface conditions encountered in the boreholes drilled at the site, spread footings are not considered appropriate for the new structure as the underlying clayey silt soils will offer low bearing resistance.

6.4 Deep Foundations

Steel piles driven to practical refusal on bedrock are considered suitable to support the abutments and piers for the proposed westbound 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.

6.4.1 Geotechnical Axial Resistance – Driven Steel Piles

For preliminary design, the factored axial geotechnical resistance at Ultimate Limit States (ULS) for HP 310 x 110 piles driven to refusal in the shale bedrock at about elevation 146.5 metres may be taken as 2,000 kilonewtons (kN). This value takes into account the structural capacity limitation of the pile and potential difficulties that the pile may have seating into the bedrock. Vertically driven pile tips should be equipped with flange reinforcement in accordance with current MTO practice (Standard Ontario Provincial Standard Drawing (OPSD) 3301.00) and battered piles should be equipped with suitable driving points (such as Titus Ejector or equivalent) 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 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, some widening of the approach embankments will be required. The increased loading will cause consolidation settlement of the underlying extensive clayey deposits as a result of increased vertical grades. The consolidation settlement is time-dependent and, depending on the sequencing of construction, may not completely occur during the construction period. That is, post-construction settlement of the clayey deposits may take place and settlement of the clayey soils relative to the piles will result in the development of negative skin friction acting on the piles. Therefore, negative skin friction, or downdrag loads, will need to be taken into account during design of the piles supporting the abutments. It is

expected that additional fill will be placed in order to widen the existing westbound embankment somewhat to the south into the median and 4.6 metres north in order to accommodate the wider replacement bridge structure. The extent of filling is expected to be relatively minor in the median with no increase in the height of the embankment. The existing abutment piling adjacent to the filling will also be affected by downdrag loads. If the approach embankment widenings are constructed well in advance of the piling or if the additional fills are minimal, the downdrag loads may be 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. Based on the embankment modifications described above, and the limited extent of filling, the negative skin friction load on a single end bearing pile may be taken as 65 kN for preliminary design. The actual negative skin friction will depend on the extent of the additional 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 detail 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. If integral abutments are under consideration, 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	90	35

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

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

6.4.4 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 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/m ³
Coefficients of lateral earth pressure:	
Active, K_a	0.33
At rest, K_o	0.50

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

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

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

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

6.6 Embankments

It is understood that widening of the existing embankment fills will be required to accommodate the larger westbound replacement structure. The preliminary design drawings indicate that the existing embankments will be widened approximately 4.6 metres to the north and to the south to infill the slightly depressed median. The side slopes will remain at 2 horizontal to 1 vertical. The new embankments may be up to 10 metres in height and will need to be benched into the existing embankments in accordance with OPSD 208.010. 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 with loose thickness not exceeding 300 millimetres and be compacted to at least 95 per cent of the material's standard Proctor maximum dry density.

Embankment settlements will be dependent on the extent of the additional filling required. Assuming that the existing 10 metre high embankment will be modified as described above, preliminary estimates of total embankment settlements are in the order of 100 millimetres. Settlements could be reduced by using lightweight fill and the effects reduced by constructing the fills well in advance of structure construction. More detailed settlement analyses 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 topsoil materials and the surficial organics and 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 will be above 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 Provision (NSSP) should be included in the contract documents.

All excavations should be carried out in accordance with the guidelines outlined in the latest edition of the Ontario Occupational Health and Safety Act and Regulations For Construction Projects. The surficial organics and 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 CLOSURE

This report was written by Mr. Azmi M. Hammoud, P.Eng. an Associate with Golder Associates Ltd. under the direction of Mr. Philip R. Bedell, P.Eng., who is the Project Manager and 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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AMH/PRB/FJH/cr
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TABLE I

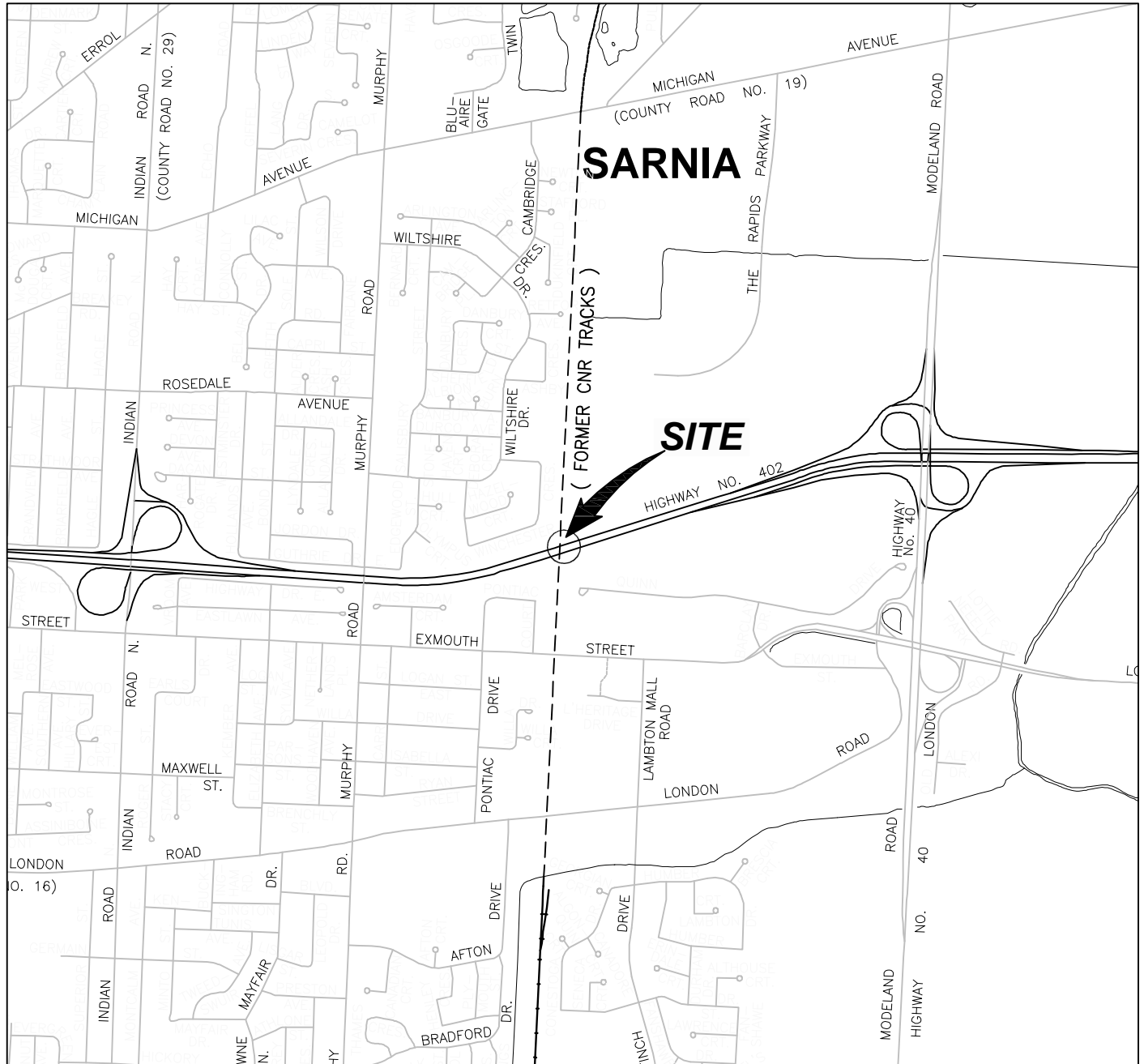
COMPARISON OF FOUNDATION ALTERNATIVES

CNR Overpass Structure Replacement
Highway 402, GWP 3038-03-00

FOUNDATION OPTION	FEASIBILITY	ADVANTAGES	DISADVANTAGES	RELATIVE COSTS	RISKS/CONSEQUENCES
Spread footings supported on native clayey silt	<ul style="list-style-type: none"> Not considered feasible due to low allowable bearing value 	<ul style="list-style-type: none"> Cost 	<ul style="list-style-type: none"> 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 	<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
Steel H-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"> \$380,000 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

NOTES:

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



PROJECT				CNR OVERPASS STRUCTURE REPLACEMENT WP No. 3038-03-00 HWY. 402			
TITLE				SITE LOCATION PLAN			
 Golder Associates LONDON, ONTARIO		PROJECT No.		FILE No.		SCALE	
		041-130099-1		041130099-1F001		AS SHOWN	
		CADD	DCH	Dec. 20/05	CHECK	REV.	0
		FIGURE 1					

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

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

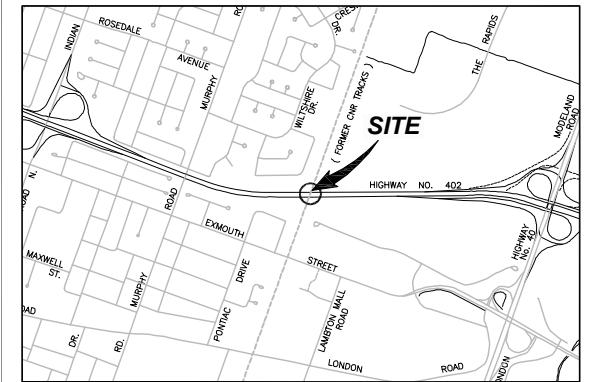


C.N.R. OVERPASS STRUCTURE REPLACEMENT BOREHOLE LOCATIONS & SOIL STRATA

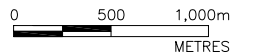
SHEET








Golder Associates Ltd.
LONDON, ONTARIO, CANADA



KEY PLAN



LEGEND

- | | |
|---|--|
|  | Borehole (BY OTHERS,
Geocres Report No. 40J16-39) |
|  | CONE PENETRATION TEST |
|  | Blows/0.3m (Std. Pen. Test, 475 j/blow) |
|  | WL during drilling |
|  | Borehole dry during drilling |

No.	ELEVATION (metres)	CO—ORDINATES (UTM NAD83, ZONE 17)	
		NORTHING	EASTING
1	180.62	4760881.67	316319.94
2	180.59	4760903.35	316323.21
3	180.41	4760870.49	316354.85
4	180.59	4760879.54	316347.97
5	180.96	4760895.12	316334.63
6	180.44	4760903.13	316332.11
7	181.08	4760863.37	316332.80
8	180.65	4760871.11	316321.58
9	180.44	4760895.84	316347.81
10	180.47	4760910.83	316348.64
11	180.50	4760882.66	316359.66
12	180.17	4760908.98	316361.15
14	180.29	4760919.31	316358.34

NOTES

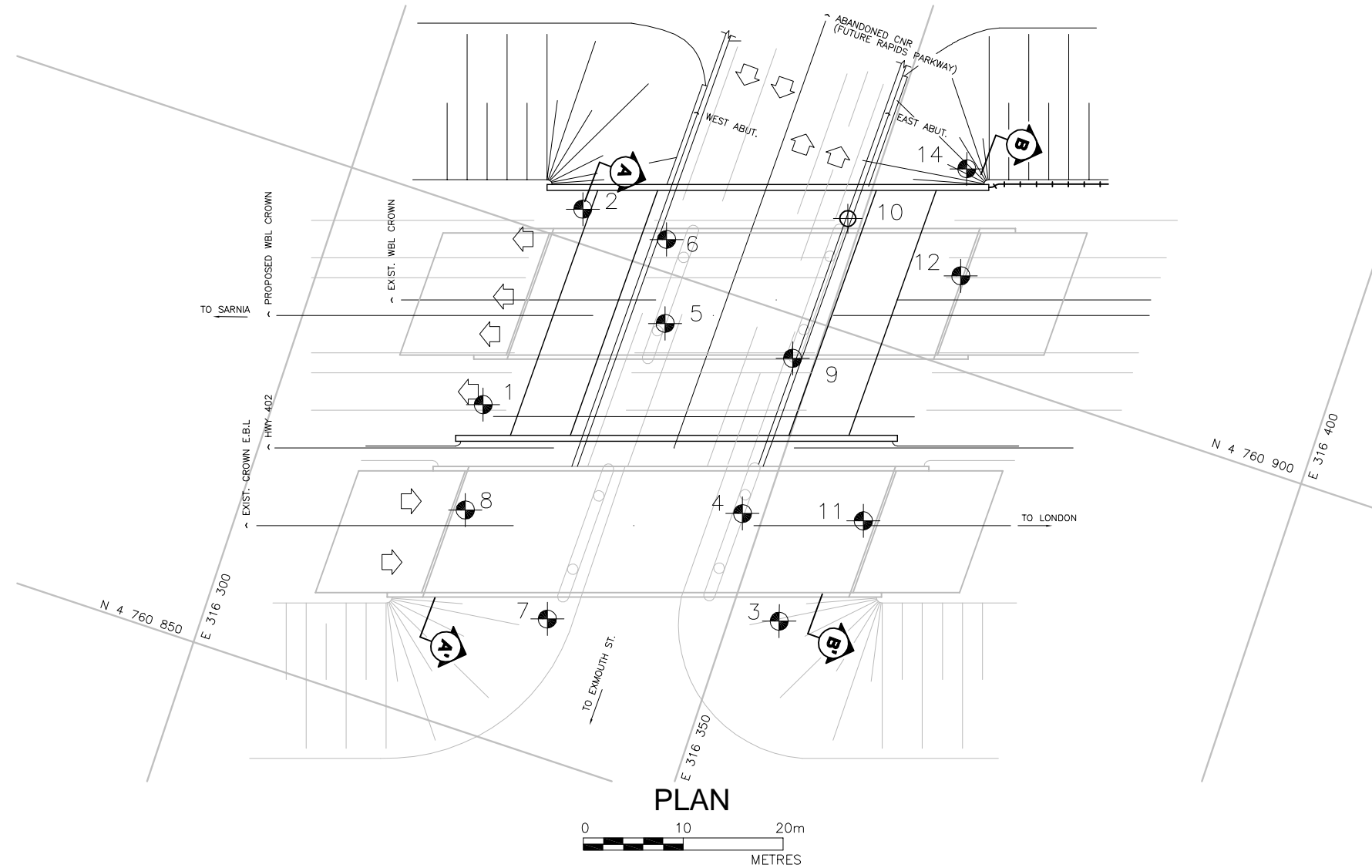
The boundaries between soil strata have been established only at Borehole locations. Between Boreholes the boundaries are assumed from geological evidence.

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

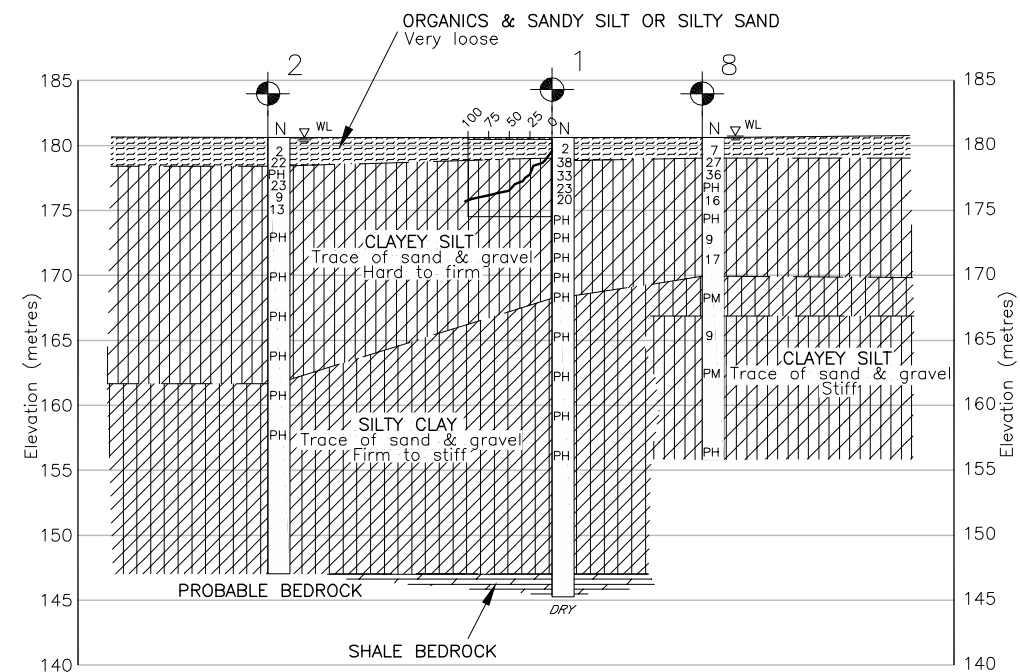
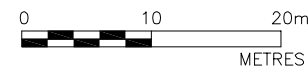
REFERENCE

DRAWING SUPPLIED BY: URS CANADA INC.
ENTITLED: C.N.R. OVERHEAD W.B.L. PRELIMINARY GENERAL ARRANGEMENT
SITE 14-337/2 DWG. 1
DATED: OCT. 2005.

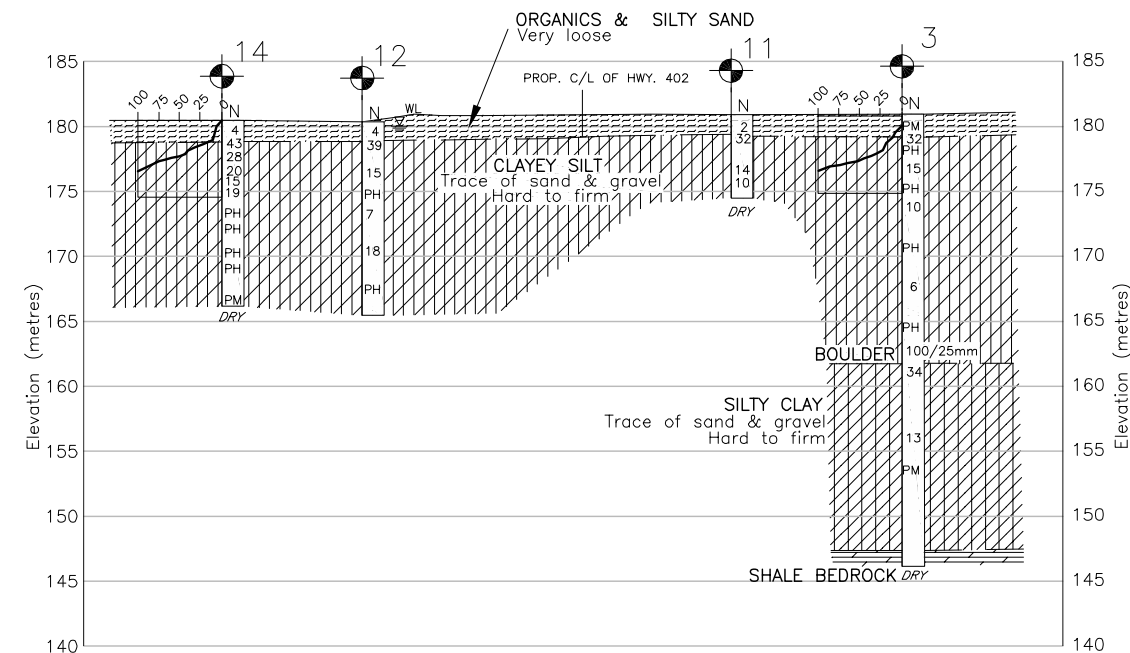
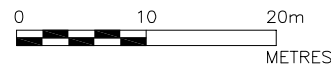
NO.	DATE	BY	REVISION	
Geocres No. 40J16-64				
HWY. No. 402		PROJECT NO.: 041-130099-1		
SUBM'D. —	CHKD:	DATE: Dec. 20/05		
DRAWN: BG/DCH	CHKD:	APPD.	DWG. 1	



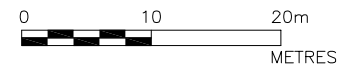
PLAN



SECTION A - A'



SECTION B - B'



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

 $1 = 1 \text{ metric}$

041130099-1D002.dwg

LIST OF ABBREVIATIONS

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

I. SAMPLE TYPE

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

III. SOIL DESCRIPTION

(a) Cohesionless Soils

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

II. PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:

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

Consistency

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

(b) Cohesive Soils

Dynamic Cone Penetration Resistance; N_d :

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

PH: Sampler advanced by hydraulic pressure

PM: Sampler advanced by manual pressure

WH: Sampler advanced by static weight of hammer

WR: Sampler advanced by weight of sampler and rod

Piezo-Cone Penetration Test (CPT)

A electronic cone penetrometer with a 60° conical tip and a project end area of 10 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)

APPENDIX A

RECORDS OF PREVIOUS BOREHOLES

" Note: This Drawing has been Reduced and is in Imperial Units - metric conversion also shown "

DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 1

FOUNDATION SECTION

JOB 70-11045

LOCATION Sta. 20 + 39 Offset 3 Lt.

ORIGINATED BY H.S.

W.P. 346-65-01802

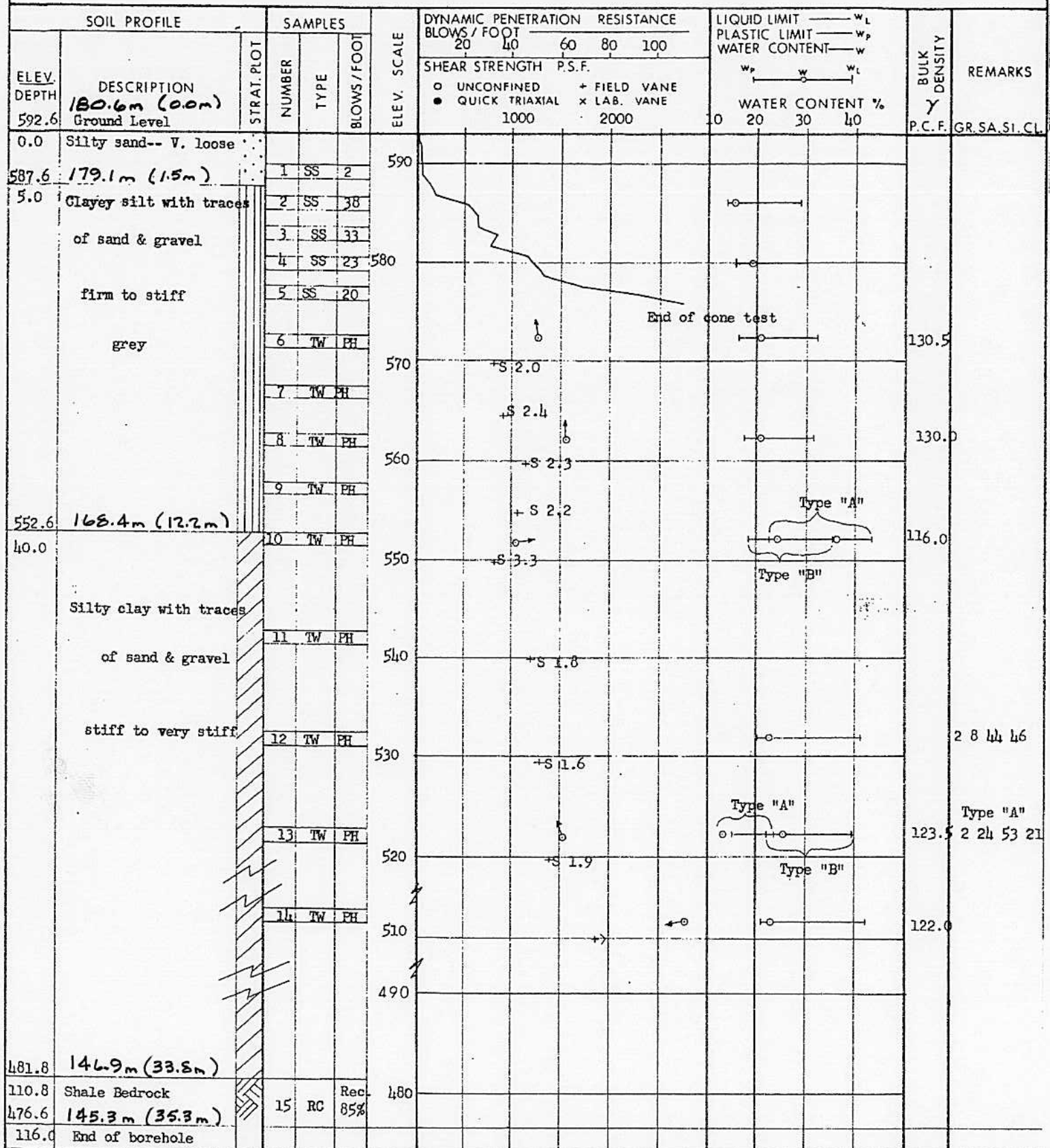
BORING DATE Nov. 26, 27, Dec. 1 & 2, 1969

COMPILED BY: G.A.

DATUM Geodetic

BOREHOLE TYPE Washboring, NX, BX casing, BXL core

CHECKED BY



" Note: This Drawing has been Reduced and is in Imperial Units - metric conversion also shown "

DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 2

FOUNDATION SECTION

JOB 70-11045

LOCATION STA. 20 + 71. 69.5 Ft. Lt. of \emptyset

ORIGINATED BY T.P.

W.P. 346-65-01 & 02

BORING DATE June 24, 1970

COMPILED BY A.K.B.

DATUM Geodetic

BOREHOLE TYPE C.M.E. Auger

CHECKED BY LL

SOIL PROFILE			SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION BLOWS / FOOT		RESISTANCE	LIQUID LIMIT — w_L PLASTIC LIMIT — w_p WATER CONTENT — w			BULK DENSITY γ P.C.F.	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FOOT		SHEAR STRENGTH P.S.F.			WATER CONTENT %				
592.5	180.6m (0.0m) Ground Level						\circ UNCONFINED \bullet QUICK TRIAXIAL	+ FIELD VANE x LAB. VANE		w_p — w — w_L				
							1000	2000			10	20	30	
0.0	Black organics & sandy silt--V. loose		1	SS	2	590								
585.5	178.5m (2.1m)		2	SS	22									
7.0	Clayey silt with traces of sand & gravel hard to firm		3	TW	PH				4000				136	
			4	SS	23	580								
			5	SS	9									
			6	SS	13									
			7	TW	PH	570							130	
			8	TW	PH	560							134	
										2200			134.5	
			9	TW	PH	550							120	
			10	TW	PH	540							130	
530.0		161.5m (19.1m)					530							
62.5	Silty clay with traces of sand & gravel firm to stiff		11	TW	PH	520							120.5	
			12	TW	PH									
481.5	147.1m (33.5m) Probable Bedrock					490								
110.0	End of borehole													

" Note: This Drawing has been Reduced and is in Imperial Units - metric conversion also shown "

DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No.3

FOUNDATION SECTION

JOB 70-11045

LOCATION STA 21 + 37, 69.5 Ft. Rt. of E

ORIGINATED BY T.P.

W.P. 346-65-01 & 02

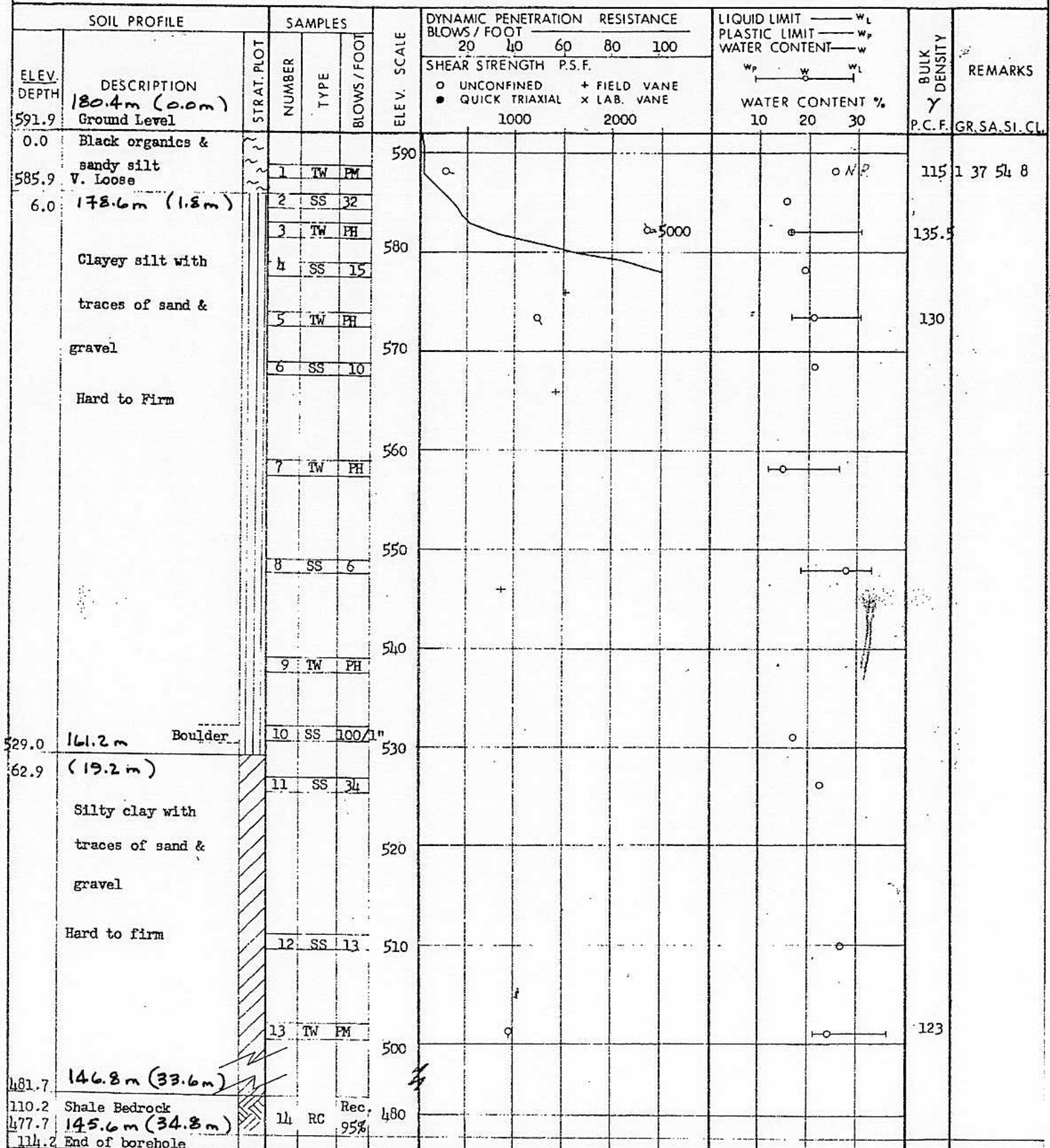
BORING DATE June 11-15, 1970

COMPILED BY A.K.B.

DATUM Geodetic

BOREHOLE TYPE C.M.E. Auger & Washboring, BX casing

CHECKED BY



FOUNDATION SECTION

CHECKED BY *[Signature]*

[illegible]

(Geocres Report No. 40J16-39)

" Note: This Drawing has been Reduced and is in Imperial Units - metric conversion also shown "

DEPARTMENT OF HIGHWAYS- ONTARIO

RECORD OF BOREHOLE No. 6

FOUNDATION SECTION

MATERIALS & TESTING OFFICE

JOB 70-11045

LOCATION STA 20 + 98 Offset 60 Lt.

ORIGINATED BY G.A.

W.P. 346-65-01 & 02

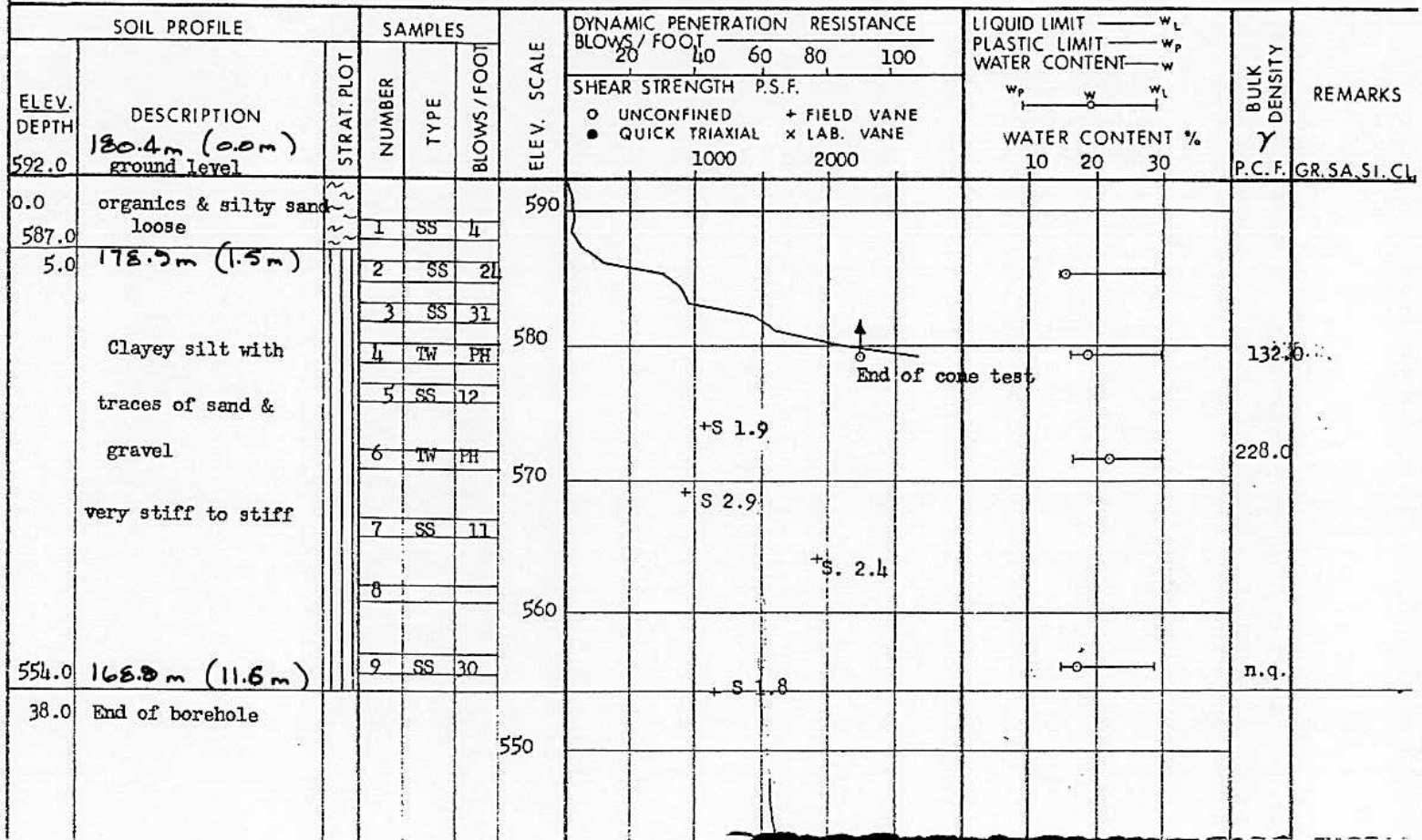
BORING DATE Dec. 4/69

COMPILED BY

DATUM Geodetic

BOREHOLE TYPE Washboring

CHECKED BY



(Geocres Report No. 40J16-39)

" Note: This Drawing has been Reduced and is in Imperial Units - metric conversion also shown "

DEPARTMENT OF HIGHWAYS- ONTARIO

RECORD OF BOREHOLE No. 8

FOUNDATION SECTION

MATERIALS & TESTING OFFICE

JOB 70-11045

LOCATION STA 20 + 34 32.5 Ft. Rt. of C

ORIGINATED BY T.P.

W.P. 346-65-01 & 02

BORING DATE June 22-23, 1970

COMPILED BY A.K.B.

DATUM Geodetic

BOREHOLE TYPE C.M.E. Auger

CHECKED BY

SOIL PROFILE			SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT		LIQUID LIMIT — w_L PLASTIC LIMIT — w_p WATER CONTENT — w			BULK DENSITY γ	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FOOT		SHEAR STRENGTH P.S.F.		w_p — w — w_L				
							\circ UNCONFINED \bullet QUICK TRIAXIAL	+ FIELD VANE \times LAB. VANE	WATER CONTENT % 10 20 30				
592.7	180.7m (0.0m) Ground Level					1000	2000					P.C.F.	GR, SA, SI, CL
0.0	Black organics & very loose sand		1	SS	3							0 = 81	
588.2	179.3m (1.4m) Clayey silt with traces of sand and gravel ; Hard to stiff		2	SS	27								
4.5			3	SS	36								
			4	TW	PH								
			5	SS	16								
			6	TW	PH								
			7	SS	9								
			8	SS	17								
557.7	169.9m (10.7m)												
35.0	Silty clay, traces of sand & gravel—firm		9	TW	PM								
547.7	166.9m (13.7m)												
45.0	Clayey silt with traces of sand & gravel stiff		10	SS	6								
			11	TW	PM								
511.2	155.8m (24.8m)		12	TW	PH								
81.5	End of boreholes												

" Note: This Drawing has been Reduced and is in Imperial Units - metric conversion also shown "

DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 9

FOUNDATION SECTION

JOB 70-11045

LOCATION STA 21 + 40 Offset 20' LT.

ORIGINATED BY G.A.

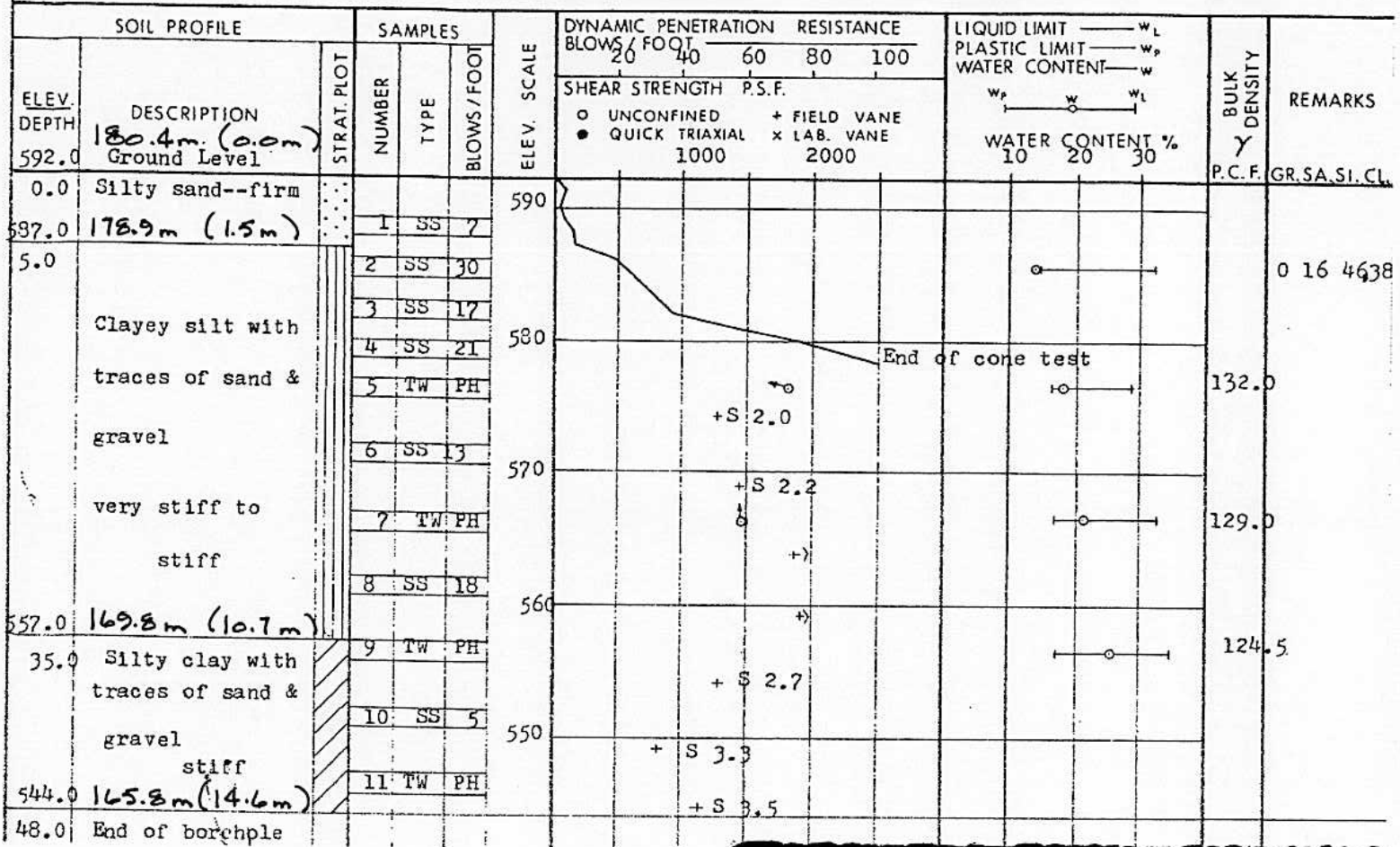
W.P. 346-65-01 & 02

BORING DATE Dec. 2, 1969

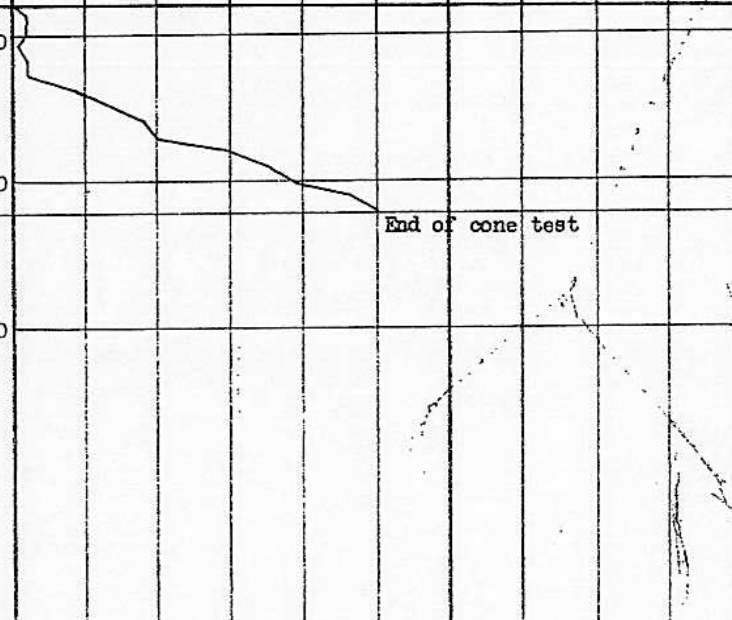
COMPILED BY G.A.

DATUM Geodetic

BOREHOLE TYPE Cont. Flt. Auger

CHECKED BY *LL*

" Note: This Drawing has been Reduced and is in Imperial Units - metric conversion also shown "

DEPARTMENT OF HIGHWAYS - ONTARIO						RECORD OF BOREHOLE No. 10								FOUNDATION SECTION							
MATERIALS & TESTING OFFICE																					
JOB <u>70-11045</u>						LOCATION <u>STA 21+57 Offset 69' LT.</u>								ORIGINATED BY <u>G.A.</u>							
W.P. <u>346-65-01 & 02</u>						BORING DATE <u>1-2-1967</u>								COMPILED BY <u>G.A.</u>							
DATUM <u>Geodetic</u>						BOREHOLE TYPE <u>Dynamic Cone Penetration</u>								CHECKED BY <u>[Signature]</u>							
SOIL PROFILE			SAMPLES			DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT						LIQUID LIMIT — w_L PLASTIC LIMIT — w_P WATER CONTENT — w			BULK DENSITY		REMARKS				
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FOOT	ELEV. SCALE	SHEAR STRENGTH P.S.F.						WATER CONTENT % $w_p \quad w \quad w_L$			Y					
							○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB. VANE														
592.1	180.5m (0.0m) Ground Level					590										P.C.F.	GR. SA. SI. CL.				
0.0	Cone Penetration Only					580															
578.1	176.2m (4.3m)					570															
14.0	End of cone test																				

(*Geocres Report No. 40J16-39*)

" Note: This Drawing has been Reduced and is in Imperial Units - metric conversion also shown "

DEPARTMENT OF HIGHWAYS- ONTARIO

MATERIALS & TESTING OFFICE

RECORD. OF BOREHOLE No. 11

FOUNDATION SECTION

JOB 70-11045 LOCATION STA 21 + 64, 34 Ft. Rt. of E

ORIGINATED BY A.K.B.

W.P. 346-65-01-02 BORING DATE June 26, 1970

COMPILED BY A.K.B.

DATUM Geodetic BOREHOLE TYPE C.M.E. Auger

CHECKED BY

[illegible]

CHECKED BY

(Geocres Report No. 40J16-39)

"Note: This Drawing has been Reduced and is in Imperial Units - metric conversion also shown"

DEPARTMENT OF HIGHWAYS- ONTARIO

MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 11

FOUNDATION SECTION

JOB 70-11045 LOCATION Sta 21 + 96 Offset 86' LT.

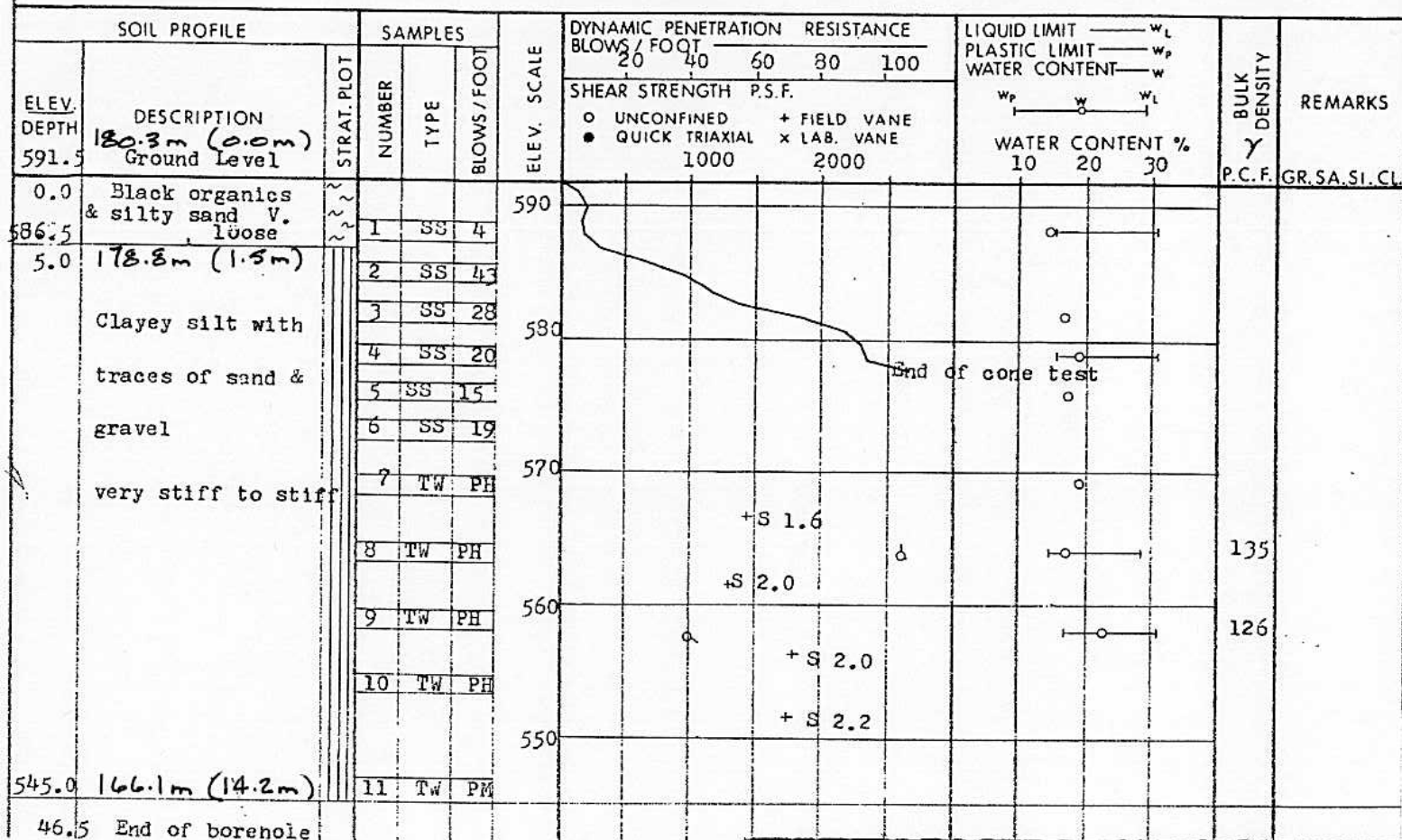
ORIGINATED BY P.P.

W.P. 346-65-01&02 BORING DATE Dec. 1 & 2, 1969

COMPILED BY G.A.

DATUM Geodetic BOREHOLE TYPE Cont. Flt. Auger

CHECKED BY J/K



APPENDIX B
SITE PHOTOGRAPHS

SITE PHOTOGRAPHS



Photo 1



Photo 2