FOUNDATION INVESTIGATION AND DESIGN REPORT

ONTARIO STREET OVERPASS HIGHWAY 11 NBL

HIGHWAY 11 FOUR-LANING AT BURK'S FALLS

W.P. 485-93-01, SITE: 44-398N

Geocres Number: 31E-259

Report to

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ADDENDUM

To:

FOUNDATION INVESTIGATION AND DESIGN REPORT

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Date: December 13, 2006

All references in this report to "G.W.P. 473-93-00" or to "W.P. 473-93-00" shall be read as "W.P. 485-93-01"

End.

TABLE OF CONTENTS

THURBER

Appendices

THURBER

FOUNDATION INVESTIGATION AND DESIGN REPORT

ONTARIO STREET OVERPASS HIGHWAY 11 NBL

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PART 1: FACTUAL INFORMATION

1   INTRODUCTION

This report presents the factual findings obtained from a foundation investigation conducted at the site of a proposed highway structure at Burk's Falls, Ontario. The proposed single-span structure will carry the northbound lanes (NBL) of the future four-laned Highway 11 across Ontario Street at the proposed interchange.

A previous foundation investigation was carried out by AGRA Earth and Environmental Ltd. for a certain interchange and structure configuration. The design was subsequently changed and additional boreholes were drilled to reflect these changes. The factual data from both investigations has been used in preparing this report.

The purpose of the investigation was to explore the subsurface conditions at the site and, based on the data obtained, to provide a borehole location plan, records of boreholes, stratigraphic profile and cross-sections, laboratory test results and a written description of the subsurface conditions. A model of the subsurface conditions was developed from the data obtained in the course of the present and previous investigations.

Thurber carried out the investigation as a sub-consultant to Marshall Macklin Monaghan, under the Ministry of Transportation Ontario (MTO) Agreement Number 5005-E-0028.

2   SITE DESCRIPTION

The site of the investigation is that of a proposed interchange between the four-laned Highway 11 and realigned Ontario Street to serve the north end of Burk's Falls. The site lies immediately east of the existing highway, encroaching onto the NB shoulder, and approximately 300 m north of the existing intersection of Highway 11 and Ontario Street.

There is some industrial development on Ontario Street a short distance south and east of the site but no development in the immediate vicinity of the site. A cleared area through the trees that is aligned with Ontario Street to the south suggests that the site lies close to the point where the

THURBER

current alignment of Highway 11 deviates from the earlier alignment that took Highway 11 through the town of Burk's Falls.

Adjacent to the site of the proposed structure, existing Highway 11 passes through a rock cut approximately 1.5 m deep.

Geologically, the site area is located within the physiographic region known as the Canadian Shield, characterized by Pre-Cambrian bedrock typically occurring as rounded knobs and ridges with muskeg deposits developed in poorly drained depressions. The site lies on a low knob and bedrock outcrop is exposed over part of the site.

Photographs of the site are included in Appendix G.

3   SITE INVESTIGATION AND FIELD TESTING

The site investigation and field testing for this project were carried out in the period of July 24 and 25, 2006. Nine boreholes numbered 06-11 to 06-16 and 06-18 to 06-20 were drilled at the south and north abutments of the single-span structure to depths ranging from 0.0 m to 18.8 m. The approximate locations of all of the boreholes are shown on the attached Borehole Locations and Soil Strata Drawing in Appendix F. The location of an additional borehole numbered 06-17 is also shown on the drawing, however it was not drilled since it is located adjacent to the existing highway within the rock cut and bedrock can be inferred at this location.

The borehole locations were marked in the field by surveyors from Marshall Macklin Monaghan Ltd. who also provided Thurber with the coordinates and geodetic elevations. Thurber obtained utility clearances prior to drilling.

A combination of hand excavation, hollow-stem auger drilling and diamond coring techniques were used to advance the boreholes. Samples were obtained at selected intervals using a split spoon sampler in conjunction with Standard Penetration Testing (SPT) in the overburden soils. In all of the boreholes refusal was observed on inferred bedrock. Boreholes 06-11, 06-12, 06-16 and 06-19 were advanced by hand excavation and encountered inferred bedrock at very shallow depths ranging from 0.0 m (rock exposed at surface) to 0.5 m. During preliminary discussions with MTO, it was decided that one borehole at each abutment would be cored at least 3.0 m into bedrock and one of these boreholes would be cored an additional 15 m into bedrock to investigate the rock quality to the full depth of the proposed rock cut. At the south abutment, Borehole 06-13 was advanced 3.3 m into bedrock and at the north abutment, Borehole 06-18 was advanced 18.2 m into bedrock by NQ size diamond coring techniques.

Groundwater conditions in the open boreholes were observed throughout the drilling operations. At each abutment a standpipe piezometer consisting of 19 mm PVC pipe with a slotted screen was installed and enclosed in filter sand to permit longer term groundwater level monitoring. The locations and completion details of the piezometers are shown in Table 3.1. The boreholes in which no piezometers were installed were grouted with bentonite or backfilled with cuttings when the boreholes were less than 3.0 m deep. The borehole completion details are shown in Table 3.1.

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Table 3.1 – Borehole Completion Details

Borehole Location	Piezometer Tip Depth/ Elevation (m)	Completion Details
06-11 South Abutment	None Installed	Backfilled with cuttings to ground surface (< 3.0 m deep).
06-12 South Abutment	None Installed	Backfilled with cuttings to ground surface (< 3.0 m deep).
06-13 South Abutment	5.7/316.8	Piezometer with 1.5 m slotted screen installed with sand filter to 3.1 m, bentonite seal from 3.1 m to 2.4 m, grout from 2.4 m to 0.6 m and auger cuttings from 0.6 m to ground surface.
06-14 South Abutment	None Installed	Bentonite grout to ground surface.
06-15 South Abutment	None Installed	Bentonite grout to ground surface.
06-16 North Abutment	None Installed	Bedrock outcrop at ground surface.
06-18 North Abutment	18.8/305.9	Piezometer with 1.5 m slotted screen installed with sand filter to 17.1 m, bentonite seal from 17.1 m to 16.5 m, grout from 16.5 m to 0.6 m and bentonite seal from 0.6 m to ground surface.
06-19 North Abutment	None Installed	Backfilled with cuttings to ground surface (< 3.0 m deep).
06-20 North Abutment	None Installed	Backfilled with cuttings to ground surface (< 3.0 m deep).

The drilling and sampling operations were supervised on a full time basis by a member of Thurber's technical staff. The supervisor logged the boreholes and processed the recovered soil and rock samples for transport to Thurber's laboratory for further examination and testing.

All rock cores were logged, and the Total Core Recovery (TCR), Rock Quality Designation (RQD) and the Fracture Indices (FI) were determined.

4 LABORATORY TESTING

The recovered soil samples were subjected to Visual Identification (VI) and to natural moisture content determination. The results of this testing are shown on the Record of Borehole sheets in Appendix A. Selected samples were also subjected to gradation analysis and the results of this testing program are shown on the Record of Borehole sheets in Appendix A and on the figures contained in Appendix B. The results of point load tests on rock cores retrieved from the boreholes are shown in Table B1 in Appendix B.

5 DESCRIPTION OF SUBSURFACE CONDITIONS

Reference is made to the Record of Borehole sheets in Appendix A. Details of the encountered soil and rock stratigraphy are presented in this appendix and on the "Borehole Locations and Soil Strata" drawing in Appendix F. An overall description of the stratigraphy is given in the following

THURBER

paragraphs. However, the factual data presented in the Record of Borehole Sheets governs any interpretation of the site conditions.

In general, the site is underlain by 0.0 m to 4.3 m of overburden soils overlying Pre-Cambrian bedrock. The overburden soils generally consist of topsoil, sands, silts, clay and occasional gravel and cobbles.

5.1   Topsoil

With the exception of Borehole 06-16 where bedrock was exposed at the ground surface, 125 mm to 175 mm of topsoil was encountered across the site, extending to elevations ranging from 324.5 m to 321.3 m.

The moisture content of a sample of this topsoil was approximately 13%.

5.2   Silty Sand to Sand and Gravel

Underlying the topsoil, a deposit of sand exists across the site. This deposit generally consists of silty sand, but varies to sand with some silt and sand and gravel. Cobbles and boulders were encountered within the sand and gravel deposit. The sand deposit was found predominantly overlying bedrock, extending to depths ranging from 0.4 m to 4.3 m or to elevations from 324.1 to 316.4 m.

Two selected samples from this deposit were subjected to grain size distribution tests and the results are presented in Figure B1.

Standard penetration tests (SPT) in this deposit gave 'N' values generally ranging from 16 to 48 blows per 0.3 m penetration indicating a compact to dense relative density, although occasional very loose and very dense zones were encountered, as evidenced by SPT 'N' values ranging from 3 to 90 blows per 0.3 m penetration. The sand and gravel layer containing cobbles and boulders is a dense layer.

The moisture content of samples from this material ranged from approximately 5% to 35%.

5.3   Silt to Silty Clay

Occasional zones of silt with some clay ranging to silty clay were encountered within or underlying the sand deposit. These zones were encountered to depths ranging from 0.5 m to 3.7 m or to elevations varying from 321.8 m to 317.0 m.

Two selected samples from these zones were subjected to grain size distribution tests and the results are presented in Figure B2. An Atterberg Limit test was also conducted on one sample of the silty clay and the result is presented in Figure B3.

SPT 'N' values ranged from 31 to 38 blows for 0.3 m penetration indicating a dense / hard relative density.

The moisture content of samples from these zones ranged from 5% to 27%.

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5.4   Bedrock

The overburden soils described above are underlain by granodiorite and granitic gneiss bedrock. Bedrock was proved by coring at the north and south abutments. Table 5.1 summarizes the bedrock depth and the elevations to the top of bedrock at the foundation elements where rock was cored.

TABLE 5.1 – Depth to Bedrock at Foundation Elements

Location	BH Number	Depth to Bedrock (m)	Top of Bedrock Elevation (m)
South Abutment	06-11	0.4	322.0
South Abutment	06-12	0.4	322.4
South Abutment	06-13	2.4	320.0
South Abutment	06-14	3.7	317.7
South Abutment	06-15	3.5	320.3
North Abutment	06-16	0.0	323.8
North Abutment	06-18	0.6	324.1
North Abutment	06-19	0.5	321.7
North Abutment	06-20	0.5	321.8

The granodiorite bedrock was encountered in Borehole 06-18 to a depth of 10.7 m or an elevation of 313.9 m. It is described as fresh. Its colour is black and white and it contains occasional quartz intrusions.

The granitic gneiss bedrock was encountered in Borehole 06-13 and underlying the granodiorite bedrock in Borehole 06-18. It is generally described as fresh to faintly weathered. Its colour is pink, white and black with occasional black banding.

Core recovery in the bedrock was generally between 96% and 100%. The RQD values generally ranged from 57% to 96% indicating fair to excellent rock quality.

The Fracture Index (FI) of the rock, expressed as fractures per 0.3 m of core, was generally low ranging from 0 to 5. Fracture Indices greater than 5 were obtained in some core runs indicating the presence of rubble zones within the rock mass. Sub-vertical joints were encountered and they were mostly tight with little to no infilling or secondary weathering material.

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The unconfined compressive strength of most of the rock cores is estimated to range between 113 and 204 MPa indicating a very strong intact rock. These estimated rock strength values are based on point load tests that were conducted on rock cores recovered from the boreholes. A summary of the Point Load Test Results is presented in Table B1 in Appendix B.

5.5   Water Levels

A standpipe piezometer was installed at each foundation element in a selected borehole and water levels were measured after completion of drilling prior to demobilization from the site and during a return site visit at a later date. The water level readings are presented in Table 5.2.

Table 5.2: Water Level Measurements

Date	BH 06-13		BH 06-18	
	Depth (m)	Elev. (m)	Depth (m)	Elev. (m)
July 26, 2006	2.3	320.3	4.5	320.2
August 20, 2006	Dry to 5.7	Dry to 316.8	4.5	320.2

Based on these observations, local groundwater levels exist at Elevations 320.2 m to below 316.8 m. All groundwater observations at this site are short term and the levels are expected to fluctuate seasonally and after severe weather events.

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6   MISCELLANEOUS

Eastern Ontario Diamond Drilling Ltd. of Hawkesbury, Ontario supplied a track mounted CME 75 drill rig and conducted the drilling, sampling and in-situ testing operations.

The drilling and sampling operations in the field were supervised on a full time basis by Mr. Stephane Loranger of Thurber.

Mr. Alastair E. Gorman, P.Eng. and Mr. Mark E. Farrant, P.Eng. directed the field operations and prepared the report.

Dr. P.K. Chatterji, P.Eng., a Designated Principal Contact for MTO Foundations projects, reviewed the report.

Thurber Engineering Ltd.
Mark E. Farrant, P.Eng.,
Geotechnical Engineer

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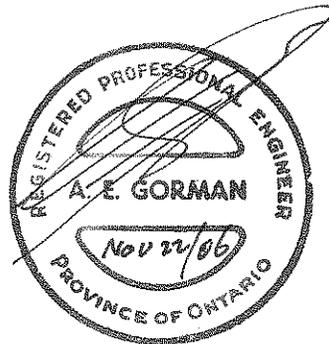
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PART 2: ENGINEERING DISCUSSION AND RECOMMENDATIONS

7   GENERAL

This report presents interpretation of the geotechnical data in the factual report and presents geotechnical design recommendations to assist the design team to select and design a suitable foundation system and approach embankments for the proposed structure.

It is understood that Highway 11 NBL will cross over Ontario Street via a single-span structure. The preliminary GA drawing indicates an approximate span of 52 m between the abutments. Ontario Street will be realigned north of its existing alignment to pass under the new Highway 11 NBL at Sta. 21+119.

At the south abutment, the finished grade of Highway 11 will be at Elevation 318.3 and the existing ground surface lies at Elevation 321.5 to 323.8, average Elevation 322.6. These elevations result in Highway 11 lying in a cut approximately 4.3 m deep.

At the north abutment, the finished grade of Highway 11 will be at Elevation 319.4 and the existing ground surface lies at Elevation 322.1 to 324.7, average Elevation 323.3. These elevations result in Highway 11 lying in a cut approximately 3.9 m deep.

The grade of Ontario Street will lie approximately at Elevation 310.5, resulting in a further cut of approximately 9 m below the highway grade.

The discussion and recommendations presented in this report are based on our understanding of the project and on the factual data obtained in the course of this investigation. Reference has also been made to the boreholes drilled in a previous investigation by AGRA.

8   STRUCTURE FOUNDATIONS

The proposed bridge is a single-span overpass structure with two abutment foundation elements.

At the south abutment the stratigraphy consists of 0.4 to 3.7 m of generally compact to dense, cohesionless overburden soils overlying bedrock. Occasional higher SPT values indicating very dense conditions are believed to be due to the presence of cobbles or boulders. The overburden consists of soils ranging from sand to clayey silt.

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At the north abutment, the stratigraphy ranges from exposed bedrock to a maximum depth of 0.5 m of overburden overlying bedrock. The northwest corner of the north abutment foundation encroaches on the NB shoulder of the existing highway. No drilling was carried out at that location, but site inspection makes it apparent that the stratigraphy consist of granular fill, probably overlying rock shatter and intact bedrock.

An apparent groundwater level exists at approximate Elevation 320.2 at the north abutment. At the south abutment, groundwater was recorded in one borehole at Elevation 320.4 on completion of drilling however the piezometer installed at the south abutment was dry approximately one month later.

In the preparation of geotechnical design recommendations, consideration was given to the following foundation types:

- Spread footings bearing on bedrock, the most obvious choice given that the entire structure will be founded within a rock cut

- Steel H-piles, a requirement if an integral abutment design is implemented.

A comparison of the foundation alternatives based on advantages and disadvantages of each is included in Appendix D.

8.1 Spread Footings on Bedrock

The top of bedrock elevations established in the course of the investigation are shown in Table 5.1. Based on these elevations, it is estimated that abutment footings will be founded at 4 to 10 m below the surface of the bedrock (8 m below the present highway surface at the northwest corner of the north abutment footing).

Footings bearing directly on the bedrock may be designed on the basis of a factored geotechnical resistance at ULS of 10,000 kPa. The SLS condition will not govern for a footing bearing on bedrock.

All rock shatter and other loose material must be removed from the bearing surface prior to placement of concrete. In the case of over-excavation, mass concrete fill may be used to reinstate the bearing surface to the design founding elevation.

Uneven rock surfaces or over-excavation may be made up using mass concrete fill. Footings bearing on mass concrete fill may be designed on the basis of a factored geotechnical resistance at ULS of 10,000 kPa, provided the concrete fill will safely support this loading. It is recommended that the fill consist of 30 MPa concrete and that the plan dimensions of the fill be at least 0.6 m larger than the footing dimensions in all directions to mitigate stress concentrations in the unreinforced concrete. The SLS condition will not govern for a footing bearing on mass concrete as described herein.

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A NSSP governing the placement of the mass concrete fill is included in Appendix E.

The stated bearing resistance is for vertical, concentric loads. In the case of eccentric or inclined loading, the geotechnical resistance must be calculated as illustrated in the CHBDC, 2000 Clause 6.7.3 and Clause 6.7.4.

If footings bear on a bedrock bench, the rock mass below the footing must be sound and not subject to sliding or toppling. Suggested wording to be included in the contract documents is given in Appendix E.

The forward edge of the footing must also lie behind a line projected upward at 1H:4V from the toe of the bench.

The excavation must be unwatered prior to placing concrete.

8.1.1 Sliding Resistance

Initial calculations of the horizontal resistance may be carried out using a value of 0.7 for the ultimate friction factor of concrete poured on rock.

If the frictional component is insufficient, the horizontal resistance may be increased by dowelling into the rock mass. Dowels are considered to be comparatively short steel bars that may be assumed to provide only shear resistance. If vertical resistance in tension is required, rock anchors should be included in the design.

The dowel may be considered as acting as a fully embedded pile in the rock and hence will fail when the ultimate lateral resistance of the rock is exceeded. Using lower bound values for the strength of the rock, an ultimate horizontal resistance of 2.6 MN may be assumed for a 50 mm steel dowel embedded 500 mm into the rock. The depth of embedment is measured below the bearing surface prepared to receive the concrete footing.

The shearing resistance of the selected dowel must be checked structurally.

A Special Provision governing the installation and testing of dowels in rock is included in Appendix E.

8.2 Steel Piles Supported on Bedrock

The foundations may be supported on steel H-piles bearing on the bedrock. At this site, a piled foundation system is recommended only in support of an integral abutment design.

The stratigraphy encountered at the site consists of relatively thin overburden deposits overlying the bedrock. The elevations of the bedrock surface are given in Table 5.1 and the undersides of the abutment stems will lie 4 to 10 m below the original bedrock surface.

If an integral abutment design is pursued at this site, it will be necessary to excavate bedrock to provide sufficient pile length. The elevation to which the bedrock must be excavated can be determined from structural analysis by considering the required flexibility and the depth of embedment in concrete required to provide fixity.

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It is anticipated that the minimum length of pile will consist of a free length of 3 m to provide flexibility plus the length required to provide fixity in the bedrock. Two options that can be considered to provide fixity in the bedrock are:

- Concrete the piles into individual sockets drilled into the bedrock

- Concrete the piles into a common trench excavated across the width of the abutment.

Individual sockets should be of sufficient diameter to allow the piles to be placed in the specified location and to allow the socket to be filled with 30 MPa concrete. Socket diameters of 500 to 600 mm are expected to be appropriate. Typical depths of sockets will be 1.0 m, measured from the base of rock shatter. The method of constructing the socket must be such that the sides are not shattered and such that drill cuttings or broken rock can be removed from the base of the socket, leaving a base in undisturbed bedrock.

If a trench is selected to embed the piles into the bedrock, it must be excavated to a depth of 1 m below the base of rock shatter. The trench must be completed by removing all shatter from the sides and base of the trench to expose undisturbed bedrock. The piles can then be placed in the specified locations and the trench backfilled with 30 MPa concrete.

A NSSP governing trench excavation in bedrock is included in Appendix E.

The relative advantages and disadvantages of these two methods of installing the piles in bedrock are as shown in Table 8.1.

Table 8.1

Method of Installation	Advantages	Disadvantages
Individual drilled sockets	i. Good control over the dimensions of the hole ii. Good probability of achieving a hole with minimal disturbance to walls, hence readily achieving fixity. iii. Reduces volume of excavation and of concrete.	i. Possibly higher cost than trench excavation. ii. Requires specialized equipment and operations.
Trench excavated in bedrock	i. Can be carried out by equipment already on site for bulk rock excavation.	i. More difficult to control dimensions of excavation. ii. Requires some

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	ii. Possibly less expensive than individually drilled sockets.	specialized techniques such as pre-splitting in order to achieve neat trench sides.
		iii. Larger volume of excavation and concrete placement.

In view of the anticipated difficulties in achieving a neat trench excavation in bedrock, drilled sockets are the recommended option for embedding the piles in bedrock.

8.2.1   Axial Resistance

Four steel pile sections normally available in the market have been considered for use in the proposed foundations. The factored, vertical, concentric, geotechnical resistances at ULS for these pile sections, when founded on bedrock, are as follows:

- 2,000 kN for HP 310 x 110

- 2,400 kN for HP 310 x 132

- 2,750 kN for HP 310 x 152

- 2, 400 kN for HP 360 x 132

The SLS condition will not govern for piles founded on bedrock.

The structural resistance of the pile must be checked by the structural designer.

8.2.2   Downdrag

Downdrag on the piles is not an issue at this site.

8.2.3   Abutment Design Considerations

From a geotechnical perspective, the conditions at this site are considered to favour the design of conventional or semi-integral abutments. However, integral abutments can be designed if other considerations warrant.

8.2.3.1   Conventional Abutment

Conventional abutments can be supported on spread footings bearing on the bedrock. To minimize the length of the bridge, by avoiding the set-back on a rock bench, the footings can be founded at the elevation of Ontario Street. Typically, such high abutment stems would not be recommended because of the high horizontal loads from the retained soil.

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However, at this site, the abutment is in a rock cut and the intact rock mass will not exert pressure on the abutment. Drainage must be provided at the back of the concrete to relieve hydrostatic pressures.

8.2.3.2   Semi-Integral Abutment

The foundations for semi-integral abutments can be designed and constructed as described for the conventional abutment. However, since the ballast wall is integral with the bridge deck, attention must be paid to the rock cut geometry and backfill behind the ballast wall. A suitable geometry is illustrated in Figure 1 (at the end of the text).

8.2.3.3   Integral Abutment

For an integral abutment design, it is recommended that the piles be concreted into sockets in the bedrock below the level of Ontario Street. The socket should be sufficiently deep to provide fixity of the pile. Typically, a depth of 1 m below the base of rock shatter, with 30 MPa concrete grout, provides fixity.

Due to the ground conditions at this site, the piles supporting the abutment must be surrounded by concentric CSP's. Typically, 600 mm and 800 mm CSP's should be suitable, with the 600 mm CSP filled with sand as specified in the Special Provision governing integral abutments, see Appendix E.

Figure 2 (at the end of the text) illustrates suggested geometry for the rock cut behind an integral abutment though the final geometry must meet the requirements of all other applicable standards. The rock cut should extend to, or beyond, the line shown in order to allow the passive pressure wedge to develop within the granular backfill. If the zone of granular material is more restricted, and the passive wedge cannot develop, higher pressures may be developed on the abutment wall during passive loading conditions.

The following three abutment treatments are considered to be feasible for encasing the CSP's at this site:

1. The CSP's may be encased in concrete as follows:

    *   Cut the rock to a vertical face approximately 200 to 300 mm behind the CSP's

    *   Install the piles and CSP's

    *   Place formwork approximately 200 to 300 mm in front of the CSP's

    *   Pour a concrete stem between the formwork and the vertical rock face.

    This arrangement can only be used if the 100 mm annular space between the CSP's will accommodate all probable movement of the bridge deck.

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Drainage must be provided behind the concrete to relieve hydrostatic pressures.

2. A RSS false abutment may be constructed to encase the CSP's. This arrangement is frequently used for false abutment design. However, at this site, it will require the excavation of sufficient rock behind the abutment to accommodate the length of the RSS reinforcement.

3. As an alternative to (2), the quantity of rock excavation could be reduced by anchoring the reinforcement to the rock face rather than providing the length required to develop resistance through friction. However, this would be a modification to a proprietary design and must be carried out and approved by the RSS supplier. Therefore, it is not recommended as the primary design but could be an acceptable alternative if proposed by the Contractor.

8.2.4   Lateral Resistance of Piles

It is anticipated that the piles will be partially embedded in bedrock and that the balance of the length will be surrounded by concentric CSPs. If, however, the pile is partially embedded in backfill below the CSPs, the lateral resistance of that portion may be calculated using a value for the coefficient of horizontal subgrade reaction (k_s) and ultimate lateral resistance (p_{ult}) as follows:

$$k_s = n_h . z / D \quad (kN/m^3)$$

$$p_{ult} = 3 . \gamma . z . K_p \quad (kPa)$$

where
z = depth of embedment of pile in metres

D = pile width in metres

n_h = coefficient of horizontal subgrade reaction (Table 8.5)

γ = unit weight (Table 8.5)

K_p = passive earth pressure coefficient (Table 8.5)

The above equations and recommended parameters may be used to analyze the interaction between a pile and the surrounding soil. The lateral pressures obtained from the analysis must not exceed the ultimate lateral resistance.

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Table 8.5 – Recommended Soil Parameters

Material	n_h (kN/m^3)	K_p	Unit Weight* (kN/m^3)	Soil Conditions
Granular B-I Fill	15,000	3.3	21.3	Compacted fill.
Granular B-II Fill	20,000	3.7	22.8	Compacted fill.
Granular A Fill	20,000	3.7	22.8	Compacted fill.

The spring constant, K_s, for analysis may be obtained by the expression, $K_s = k_s \times L \times D$ (kN/m), where k_s is the coefficient of horizontal subgrade reaction (kN/m^3), D is the pile width (m) and L is the length (m) of the pile segment or element used in the analysis. The ultimate lateral resistance, P_{ult}, may be obtained from the expression, $P_{ult} = p_{ult} \times L \times D$. This represents the ultimate load at which the pile fails and will not support any additional load at greater displacements. It is recommended, however, that the total lateral resistance assumed in one pile be limited to no more than 150 kN at ULS and 50 kN at SLS.

Since the piles are end bearing on rock, the vertical resistance will not be significantly affected by the pile spacing. Pile interaction should be considered with reference to CHBDC Clause 6.8.9.2.

For lateral soil/pile group interaction analysis, the equation for k_s and p_{ult} quoted above may be used in conjunction with appropriate reduction factors.

Where a pile group is oriented *perpendicular* to the direction of loading, group action may be considered by reducing values for k_s and p_{ult} by a reduction factor R as follows:

Pile Spacing Perpendicular to Direction of Loading	Horizontal Subgrade Reaction Reduction Factor, R_s	Ultimate Resistance Reduction Factor; R_r
4 D*	1.00	1.0
1 D*	0.50	0.33

* D is the width of the pile, and spacing is measured centre to centre

Intermediate values may be obtained by interpolation.

8.2.5   Pile Installation

Pile installation should be in accordance with Special Provision No. 903S01.

8.3   Recommended Foundation

From a geotechnical perspective, the recommended foundation is a spread footing bearing on bedrock.

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However, if other considerations warrant an integral abutment design, this can be accomplished by following the recommendations provided for piles with their tips embedded in bedrock, as described earlier in the report.

8.4 Frost Cover

The depth of earth cover for frost protection at this site is 1.8 m. Frost penetration is not required for footings bearing on bedrock or mass concrete fill or for pile caps surrounded by free-draining backfill and placed above the groundwater level.

9 EXCAVATION AND BACKFILL

9.1 General

All excavations must be carried out in accordance with the Occupational Health and Safety Act (OHSA). For the purposes of the OHSA, the native soils at this site may be classified as Type 3 soils and temporary excavations may be sloped at 1H:1V.

Excavation in sound bedrock may be carried out to vertical slopes, though the resulting face may be prone to toppling or sliding wedge failures. Post-excavation inspection of the excavated slopes must be carried out and any potential unstable rock that is detected must be removed or stabilized. The Contract Administrator should hire a rock specialist to inspect the rock excavation. The Contract Documents must instruct the Contractor to cooperate with this inspection and to implement the resulting recommendations.

If blasting is used to remove rock, it must be carried out in accordance with the Amendment to OPSS 120, August 1994. In addition, a NSSP should be included in the Contract to provide direction regarding neat trench excavation in bedrock. Suggested wording to be included is shown in Appendix E.

9.2 Foundations

The excavation and backfilling for foundations must be carried out in accordance with SP 902S01.

Bidders must be alerted to the fact that soil stripping at the site may include cobbles and/or boulders.

10 GROUNDWATER CONTROL

Short term groundwater levels were recorded at depths of 2.3 to 4.5 m, corresponding to Elevation 320.2 to 322.2. These levels are within or at the surface of the bedrock.

Prior dewatering of the site is not considered necessary but groundwater seepage or surface water may enter the open excavation, which may have to be unwatered to allow construction to proceed.

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The design of the groundwater control system is the responsibility of the Contractor. However, suitable systems that might be considered include pumping from sumps in the base of the rock excavation.

Any accumulation of water from the base of the excavation should be removed prior to placing concrete or compacting granular fill. Placement of concrete or compacting of fill must be done in the dry.

11   BRIDGE APPROACHS

The available information indicates that the approaches to the bridge will lie in rock cuts and that no embankments will be required.

The geotechnical conditions at the site are considered to be suitable for the construction of false abutments using RSS walls, thus eliminating the need for forward slopes in front of the abutments.

12   RETAINED SOIL SYSTEMS

Retained soil system (RSS) walls may be used subject to the requirements presented in this section.

RSS walls must be specified to be "High Performance" and "High Appearance". The contract drawings must include information on the longitudinal alignment of the wall in plan, the top and base elevations of the wall in profile, cross-sectional space constraints and an NSSP for the RSS wall.

12.1   Foundation

The performance of an RSS is dependent, among other factors, on the characteristics of its foundation. Failure to provide an adequate foundation may lead to settlement and distortion of the RSS and, in severe cases, to possible failure of the system. The foundation of the entire RSS mass must be considered, i.e. from the face of the wall to the furthest extent of the reinforcement.

At this site, the founding stratum will be the base of the rock cut. In order to provide a stable foundation, all rock shatter must be removed from the area under the RSS mass. The levelling pad may be poured directly on the cleaned rock surface, or the rock surface may be levelled using mass concrete up to the underside of the levelling pad. Any requirement for free drainage through the rock shatter should be taken into consideration, as it may be partially impeded by this form of construction.

If the designers determine that the rock shatter must be left in place, then the foundation for the RSS may be formed as follows:

-   Carefully chink the surface of the shatter

-   Cover the chinking with filter cloth

-   Place a 500 mm layer of compacted Granular "A"

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The following parameters may be used for the design of the RSS over fill as described above:

- Factored geotechnical resistance of 900 kPa at Ultimate Limit States (ULS)

- Geotechnical resistance of 350 kPa at Serviceability Limit States (SLS)

- Ultimate coefficient of sliding resistance of cast in-situ concrete levelling pad on Granular A or Granular B Type II fill = 0.7

- Ultimate coefficient of sliding resistance of RSS mass on Granular A or Granular B Type II fill = 0.6

Settlement under a RSS mass constructed as outlined above is expected to be minimal and to occur essentially as the RSS is constructed.

The RSS is a proprietary system and the supplier must design for internal, sliding and overturning stability and for any other failure modes identified by the supplier.

12.2   Global Stability

The global stability of a RSS wall constructed at this site, as described above, will not govern the design.

13   BACKFILL TO ABUTMENTS

It is recommended that only granular backfill be used within the immediate approaches to the structure.

The backfill to the abutment walls should be in accordance with OPSS 902 as amended by Special Provision 902S01. Granular backfill should be placed to the extents shown in OPSD 3501.000.

All granular material should meet the specifications of Special Provision 110F13 "Amendment to OPSS 1010, March 1993". Compaction equipment to be used adjacent to retaining structures should be restricted in accordance with SSP 105S10.

Settlement within the mass of the backfill is expected to be approximately 0.5% of the fill depth and to be essentially complete at the end of construction.

The design of the abutment should incorporate a subdrain as shown in OPSD 3501.000 or OPSD 3505.000, as applicable.

14   EARTH PRESSURE

For cases where backfill to the abutment is placed in accordance with OPSD 3501.000 or OPSD 3505.000, as recommended, the lateral earth pressure will be governed by the properties of

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the material within the backfill limits shown in the OPSD, i.e. a line projected up at 1.5H:1V for granular backfill.

If the support system allows yielding of the wall (unrestrained system), active horizontal earth pressure may be used in the geotechnical design of the structure. If the support system does not allow yielding (restrained system), at-rest horizontal earth pressures should be used.

Earth pressures acting on the structure should be computed in accordance with Clause 6.9 of the CHBDC but generally are given by the expression:

$$P_h = K(\gamma h + q)$$

P_h = horizontal pressure on the wall (kPa)

K = earth pressure coefficient (see table below)

γ = unit weight of retained soil (see table below)

h = depth below top of fill where pressure is computed (m)

q = value of any surcharge (kPa)

In accordance with Clause 6.9.3 of the CHBDC, a compaction surcharge should be added. The magnitude should be 12 kPa at the top of fill and decreasing to 0 kPa at a depth of 2.0 m for Granular B Type I or at a depth of 1.7 m for Granular A or Granular B Type II.

Earth pressure coefficients for backfill to the abutment wall are dependent on the material used as backfill. Typical values are given in Table 14.1.

The use of a material with a high friction angle and low active pressure coefficient (e.g. Granular A, Granular B Type II) will result in lower earth pressures acting on the wall. The use of a material with a low friction angle and lower passive pressure coefficient (e.g. Granular B Type I) will result in lower passive pressures acting on the wall. The designer must make a selection based on the design requirements of the structure.

The factors in the Table 14.1 are "ultimate" values and require certain movements for the respective conditions to be mobilized. The values to use in design can be estimated from Figure C6.9.1 (a) in the Commentary to the CHBDC, 2000.

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Table 14.1 – Earth Pressure Coefficients

Wall Condition	Earth Pressure Coefficient (K)					
	OPSS Granular A or OPSS Granular B Type II $\phi = 35°$; $\gamma = 22.8$ kN/m³		OPSS Granular B Type I $\phi = 32°$; $\gamma = 21.2$ kN/m³		Rock Fill $\phi = 42°$; $\gamma = 19.0$ kN/m³	
	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)
Active (Unrestrained Wall)	0.27	0.40*	0.31	0.48*	0.20	0.28*
At rest (Restrained Wall)	0.43	-	0.47	-	0.33	-
Passive (Movement Towards Soil Mass)	3.70	-	3.30	-	5.0	-

- For wing walls.

-

15 SEISMIC CONSIDERATIONS

15.1 Seismic Design Parameters

The site is treated as lying in Seismic Zone 1. The following seismic parameters should be used for design:

- Velocity Related Seismic Zone   1

- Zonal Velocity Ratio   0.05

- Acceleration Related Seismic Zone   1

- Zonal Acceleration Ratio   0.05

- Peak Horizontal Acceleration   0.08

The soil profile type at this site has been classified as Type I. Therefore, according to Table 4.4.6.1 of the CHBDC, a Site Coefficient "S" (ground motion amplification factor) of 1.0 should be used in seismic design.

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15.2   Liquefaction Potential

The fill in the immediate approaches will consist of a limited extent of granular material over bedrock, all in a drained condition.

There is not considered to be any potential for liquefaction at this site under a seismic event.

15.3   Retaining Wall Dynamic Earth Pressures

In accordance with Clause 4.6.4 of the CHBDC, retaining structures should be designed using active (K_{AE}) and passive (K_{PE}) earth pressure coefficients that incorporate the effects of earthquake loading.

In calculating the active, passive and at rest earth pressure coefficients the angle of friction between the wall and backfill material is assumed to be 0.5 ϕ. For the design of retaining walls, the coefficients of horizontal earth pressure in Table 15.1 may be used.

Table 15.1 – Earth Pressure Coefficient for Earthquake Loading

Earth Pressure Coefficient (K) for Earthquake Loading						
Wall Condition	Granular A or Granular B Type II $\phi = 35°$; $\delta = 17.5°$ $\gamma = 22.8$ kN/m^3		OPSS Granular B Type I $\phi = 32°$; $\delta = 16°$ $\gamma = 21.2$ kN/m^3		Rock Fill $\phi = 42°$; $\delta = 21°$ $\gamma = 19.0$ kN/m^3	
	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)
Active (K_{AE})*	0.3	0.45	0.33	0.54	0.23	0.31
Passive (K_{PE})	6.3	6.3	5.4	5.4	12.0	12.0
At Rest (K_{OE})**	0.59		0.63		0.33	

* After Mononobe and Okabe, passive case assumes a horizontal surface in front of the wall.
** After Woods

16   CONSTRUCTION CONCERNS

Potential construction concerns include, but are not necessarily limited to:

• Control of the installation of piles to provide adequate fixity in the bedrock

• Control of rock excavation to mitigate damage to founding surface

• Preparation of the founding surface for any RSS walls

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- Steep rock excavation faces may be prone to toppling, sliding wedge or other modes of instability. The Contract Administrator should engage a recognized rock slope specialist to advise on any remedial measures that may be necessary. The Contractor must cooperate with the inspection and implement the recommendations.

17 CLOSURE

Engineering analysis and preparation of the report were carried out by Mr. Alastair E. Gorman, P.Eng.

The report was reviewed by Dr. P.K. Chatterji, P.Eng., a Designated Principal Contact for MTO Foundations Projects.

Thurber Engineering Ltd.

Alastair E. Gorman, P.Eng.,
Senior Foundations Engineer
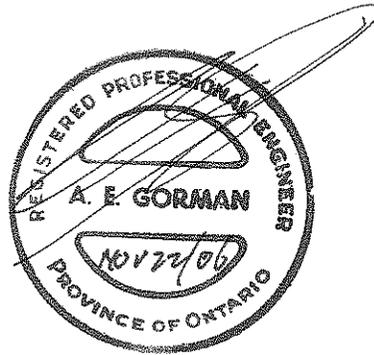
P. K. Chatterji, P.Eng.,
Review Principal

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℄ SOUTH ABUT.
BRGS.

ORIGINAL GROUND

℄ REALIGNED
ONTARIO ST.
(EAST SIDE)

1m min.

2*

1

Rock Subgrade as Determined By:
Highway / Pavement Design

Rock Line

320

315

310

305
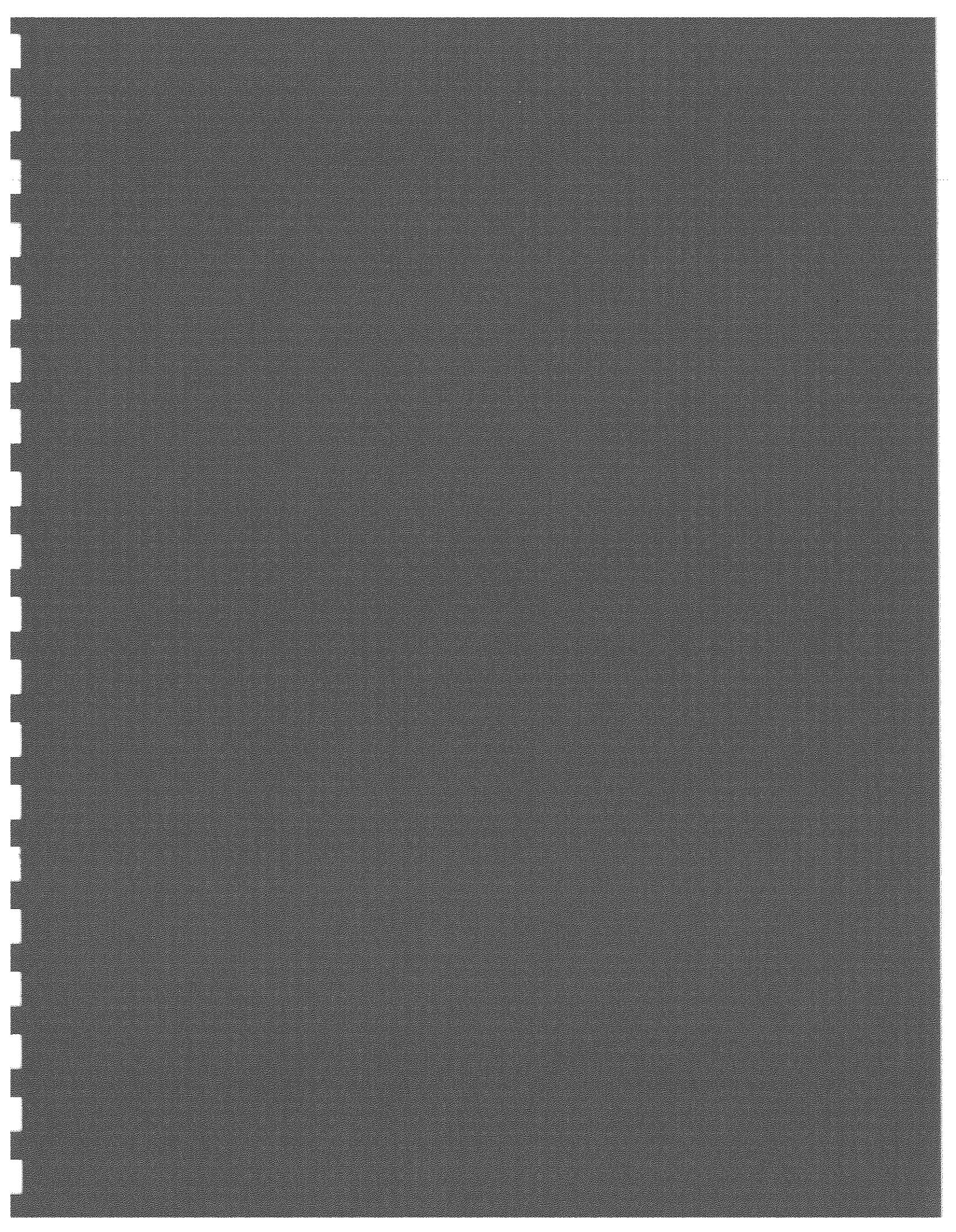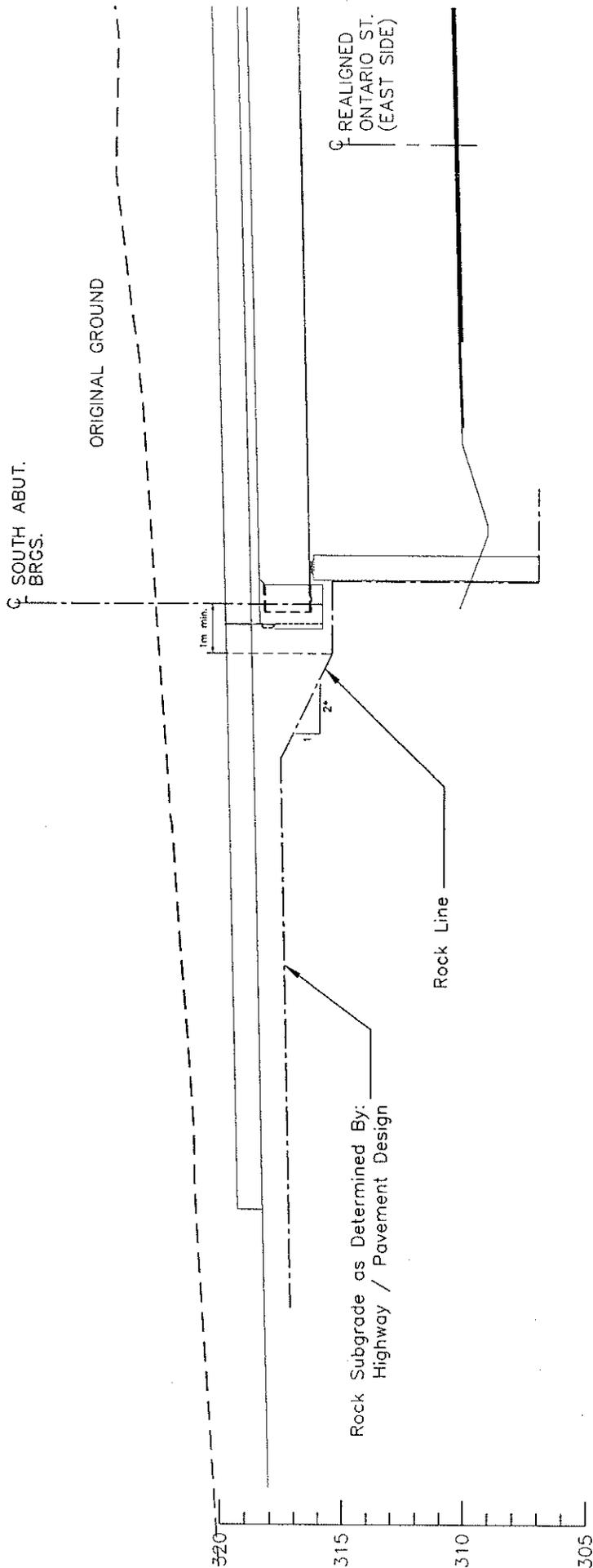
* Slope at 2h:1v or flatter to conform
to transition requirements

Marshall Macklin Monaghan

THURBER ENGINEERING LTD.
GEOTECHNICAL • ENVIRONMENTAL • MATERIALS

**ONTARIO STREET INTERCHANGE
HIGHWAY 11
Burk's Falls, Ontario**

| ENGINEER: AEG | DRAWN: MFA | APPROVED: PKC |
| DATE: NOVEMBER 2006 | SCALE: NTS | DRAWING No. FIGURE1 |

19-1423-31   Burk's Falls, ON

Figure2.DWG   THIS DRAWING IS PART OF A THURBER "REPORT" AND ITS USE IS SUBJECT TO THURBER'S STATEMENT OF GENERAL CONDITIONS   August 11, 2006

ORIGINAL GROUND

℄ SOUTH ABUT. BRGS.

℄ REALIGNED ONTARIO ST. (EAST SIDE)

1m min.

Rock Line Determined By:
1. Flexibility of pile
2. Highway design requirements,
   —eg. Depth of shatter in ditch

Rock Subgrade as Determined By:
Highway / Pavement Design

2″
1

Rock Line

(VERTICAL FACE MAY BE DETERMINED BY RSS REQUIREMENTS)

* Slope at 2h:1v or flatter to conform to transition requirements

320
315
310
305

Marshall Macklin Monaghan

ONTARIO STREET INTERCHANGE
HIGHWAY 11
Burk's Falls, Ontario

19-1423-31   Burk's Falls, ON

THURBER ENGINEERING LTD.
GEOTECHNICAL • ENVIRONMENTAL • MATERIALS

| ENGINEER: AEG | DRAWN: JHL | APPROVED: PKC |
| DATE: AUGUST 2006 | SCALE: NTS | DRAWING No. FIGURE2 |

Appendix A

Record of Borehole Sheets

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SYMBOLS, ABBREVIATIONS AND TERMS USED ON RECORDS OF BOREHOLES

1. TEXTURAL CLASSIFICATION OF SOILS

CLASSIFICATION	PARTICLE SIZE	VISUAL IDENTIFICATION
Boulders	Greater than 200mm	same
Cobbles	75 to 200mm	same
Gravel	4.75 to 75mm	5 to 75mm
Sand	0.075 to 4.75mm	Not visible particles to 5mm
Silt	0.002 to 0.075mm	Non-plastic particles, not visible to the naked eye
Clay	Less than 0.002mm	Plastic particles, not visible to the naked eye

2. COARSE GRAIN SOIL DESCRIPTION (50% greater than 0.075mm)

TERMINOLOGY	PROPORTION
Trace or Occasional	Less than 10%
Some	10 to 20%
Adjective (e.g. silty or sandy)	20 to 35%
And (e.g. sand and gravel)	35 to 50%

3. TERMS DESCRIBING CONSISTENCY (COHESIVE SOILS ONLY)

DESCRIPTIVE TERM	UNDRAINED SHEAR STRENGTH (kPa)	APPROXIMATE SPT[1] 'N' VALUE
Very Soft	12 or less	Less than 2
Soft	12 to 25	2 to 4
Firm	25 to 50	4 to 8
Stiff	50 to 100	8 to 15
Very Stiff	100 to 200	15 to 30
Hard	Greater than 200	Greater than 30

NOTE: Hierarchy of Soil Strength Prediction
1) Laboratory Triaxial Testing
2) Field Insitu Vane Testing
3) Laboratory Vane Testing
4) SPT value
5) Pocket Penetrometer

4. TERMS DESCRIBING DENSITY (COHESIONLESS SOILS ONLY)

DESCRIPTIVE TERM	SPT "N" VALUE
Very Loose	Less than 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very Dense	Greater than 50

5. LEGEND FOR RECORDS OF BOREHOLES

SYMBOLS AND ABBREVIATIONS FOR SAMPLE TYPE

SS Split Spoon Sample   WS Wash Sample   AS Auger (Grab) Sample
TW Thin Wall Shelby Tube Sample   TP Thin Wall Piston Sample
PH Sampler Advanced by Hydraulic Pressure   PM Sampler Advanced by Manual Pressure
WH Sampler Advanced by Self Static Weight   RC Rock Core   SC Soil Core

$$\text{Sensitivity} = \frac{\text{Undisturbed Shear Strength}}{\text{Remoulded Shear Strength}}$$

$\underline{\mathbf{Y}}$ Water Level

C_{pen} Shear Strength Determination by Pocket Penetrometer

(1) SPT 'N' Value   Standard Penetration Test 'N' Value – refers to the number of blows from a 63.5kg hammer free falling a height of 0.76m to advance a standard 50 mm outside diameter split spoon sampler for 0.3 m depth into undisturbed ground.

(2) DCPT   Dynamic Cone Penetration Test – Continuous penetration of a 50 mm outside diameter, 60° conical steel point attached to "A" size rods driven by a 63.5 kg hammer free falling a height of 0.76 m. The resistance to cone penetration is the number of hammer blows required for each 0.3 m advance of the conical point into undisturbed ground.

UNIFIED SOILS CLASSIFICATION

MAJOR DIVISIONS		GROUP SYMBOL	TYPICAL DESCRIPTION
COARSE GRAINED SOILS	GRAVEL AND GRAVELLY SOILS	GW	Well-graded gravels or gravel-sand mixtures, little or no fines.
		GP	Poorly-graded gravels or gravel-sand mixtures, little or no fines.
		GM	Silty gravels, gravel-sand-silt mixtures.
		GC	Clayey gravels, gravel-sand-clay mixtures.
	SAND AND SANDY SOILS	SW	Well-graded sands or gravelly sands, little or no fines.
		SP	Poorly-graded sands or gravelly sands, little or no fines.
		SM	Silty sands, sand-silt mixtures.
		SC	Clayey sands, sand-clay mixtures.
FINE GRAINED SOILS	SILTS AND CLAYS $W_L < 50\%$	ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity.
		CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays. ($W_L < 30\%$).
		CI	Inorganic clays of medium plasticity, silty clays. ($30\% < W_L < 50\%$).
		OL	Organic silts and organic silty-clays of low plasticity.
	SILTS AND CLAYS $W_L > 50\%$	MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts.
		CH	Inorganic clays of high plasticity, fat clays.
		OH	Organic clays of medium to high plasticity, organic silts.
HIGHLY ORGANIC SOILS		Pt	Peat and other highly organic soils.
CLAY SHALE			
SANDSTONE			
SILTSTONE			
CLAYSTONE			
COAL			

RECORD OF BOREHOLE No 06-11   1 OF 1   METRIC

W.P.	473-93-00	LOCATION	N 5 054 668.96 E 311 568.53 Ontario Street Overpass (NBL)	ORIGINATED BY	SLL
HWY	11	BOREHOLE TYPE	Hand Excavation	COMPILED BY	JHL
DATUM	Geodetic	DATE	24.07.06 - 24.07.06	CHECKED BY	AEG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT							PLASTIC LIMIT W_P	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W_L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100							WATER CONTENT (%) 20 40 60			kN/m³	GR SA SI CL
322.4 0.0	TOPSOIL: (150 mm)																		
0.2 322.0	Silty SAND, trace roots Brown														○				
0.4	Moist END OF BOREHOLE AT 0.40 m. REFUSAL AT 0.40 m ON PROBABLE BEDROCK OR BOULDERS. BOREHOLE BACKFILLED WITH CUTTINGS TO SURFACE.						322												

$+^3, \times^3$: Numbers refer to Sensitivity $15\text{-}\underset{10}{\overset{20}{\underset{}{\oplus}}}\text{-}5$ (%) STRAIN AT FAILURE

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W.P.	473-93-00	LOCATION	N 5 054 672.62 E 311 568.46 Ontario Street Overpass (NBL)	ORIGINATED BY	SLL
HWY	11	BOREHOLE TYPE	Hand Excavation	COMPILED BY	JHL
DATUM	Geodetic	DATE	24.07.06 - 24.07.06	CHECKED BY	AEG

ELEV DEPTH	SOIL PROFILE DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH kPa (O UNCONFINED, ● QUICK TRIAXIAL, + FIELD VANE, × LAB VANE) 20 40 60 80 100	WATER CONTENT (%) PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
322.8 0.0	TOPSOIL: (150 mm)										
0.2 322.4 0.4	Silty SAND, trace gravel Brown Moist								o		
	END OF BOREHOLE AT 0.35 m. REFUSAL AT 0.35 m ON PROBABLE BEDROCK OR BOULDERS. BOREHOLE BACKFILLED WITH CUTTINGS TO SURFACE.						322				

+ 3 , × 3   Numbers refer to Sensitivity

20
15 ⊕ 5
10   (%) STRAIN AT FAILURE

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W.P. 473-93-00	LOCATION N 5 054 671.35 E 311 577.62 Ontario Street Overpass (NBL)	ORIGINATED BY SLL
HWY 11	BOREHOLE TYPE Hollow Stem Augers / NQ Core Barrel	COMPILED BY JHL
DATUM Geodetic	DATE 24.07.06 - 24.07.06	CHECKED BY AEG

SOIL PROFILE / SAMPLES / Remarks

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
322.5									
0.0	TOPSOIL: (125 mm)								
0.1	Silty SAND, trace roots		1	SS	16		322		
	Compact to Dense								
	Brown								
	Moist		2	SS	37				
320.9							321		
1.5	SILT, some clay, trace sand		3	SS	31				0 5 83 11
	Dense								
	Brown								
	Moist								
320.0									
2.4	BEDROCK		1	RUN			320	FI	RUN 1# TCR=100%,
	Pink, white and black, crystalline,							4	SCR=83%, RQD=0%,
	faintly weathered to fresh, very							2	UCS=MPa
	strong, GRANITIC GNEISS		2	RUN			319	3	RUN 2# TCR=100%,
								4	SCR=93%, RQD=65%,
	Subvertical joint from 3.71 to 3.89 m							10	UCS=131MPa
	Faintly weathered, smooth joint surface							10	
	Rubble zone from 3.89 to 3.96 m							1	RUN 3# TCR=100%,
							318	0	SCR=98%, RQD=93%,
	Subvertical joint from 4.74 to 4.92 m		3	RUN				2	UCS=140MPa
	Slightly weathered, rough joint surface							0	
316.8							317	2	
5.7	END OF BOREHOLE AT 5.64 m.								

Piezometer installation consists of 19
mm diameter Schedule 40 PVC pipe
with a 1.52 m slotted screen.

WATER LEVEL READINGS:
DATE   DEPTH(m)   ELEV.(m)
07/26/06   2.25   320.25
08/20/06   Dry

DYNAMIC CONE PENETRATION RESISTANCE PLOT

20 40 60 80 100

SHEAR STRENGTH kPa
O UNCONFINED   + FIELD VANE
● QUICK TRIAXIAL   × LAB VANE
20 40 60 80 100

PLASTIC LIMIT wP — NATURAL MOISTURE CONTENT w — LIQUID LIMIT wL
WATER CONTENT (%)
20 40 60

+³ ,×³ ; Numbers refer to Sensitivity   15 20 5 10 (%) STRAIN AT FAILURE

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W.P.	473-93-00	LOCATION	N 5 054 670.05 E 311 586.85 Ontario Street Overpass (NBL)	ORIGINATED BY	SLL
HWY	11	BOREHOLE TYPE	Hollow Stem Augers	COMPILED BY	JHL
DATUM	Geodetic	DATE	24.07.06 - 24.07.06	CHECKED BY	AEG

ELEV DEPTH	DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION / SHEAR STRENGTH	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
321.5 / 0.0	TOPSOIL: (150 mm)										
0.2	Silty SAND, trace gravel, Loose to Very Dense, Brown, Moist		1	SS	5		321				
			2	SS	90						
320.1 / 1.4	Silty CLAY, trace silt seams, trace sand, Hard, Grey (CI)		3	SS	38		320				0 5 42 52
318.7 / 2.8			4	SS	32		319				
	Silty SAND, trace clay, trace gravel, Compact, Brown, Moist		5	SS	22		318				3 71 22 3
317.7 / 3.7	END OF BOREHOLE AT 3.74 m. AUGER REFUSAL AT 3.74 m ON PROBABLE BEDROCK OR BOULDERS. BOREHOLE OPEN TO 3.35 m AND DRY UPON COMPLETION. BOREHOLE GROUTED WITH BENTONITE TO SURFACE.										

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

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RECORD OF BOREHOLE No 06-15   1 OF 1   METRIC

W.P. 473-93-00   LOCATION N 5 054 673.77 E 311 586.58 Ontario Street Overpass (NBL)   ORIGINATED BY SLL

HWY 11   BOREHOLE TYPE Hollow Stem Augers   COMPILED BY JHL

DATUM Geodetic   DATE 24.07.06 - 24.07.06   CHECKED BY AEG

SOIL PROFILE			SAMPLES				ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				NATURAL MOISTURE CONTENT		UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS		SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE				PLASTIC LIMIT wP — WATER CONTENT (%)	LIQUID LIMIT wL	γ	GR SA SI CL
323.8								20 40 60 80 100				20 40 60		kN/m³	
0.0	TOPSOIL: (175 mm)														
0.2	Silty SAND Loose Brown		1	SS	7							○ (≈40)			
323.1	Moist														
0.7	SAND, some silt, trace clay Dense to Compact Brown Moist		2	SS	31		323					○			
	trace gravel, occasional cobbles very dense below 1.8 m		3	SS	20		322					○			0 80 18 2
			4	SS	62		321					○			
			5	SS	50/ .125							○			
320.3															
3.5	END OF BOREHOLE AT 3.51 m. AUGER REFUSAL AT 3.51 m ON PROBABLE BEDROCK OR BOULDERS. BOREHOLE OPEN AND WATER LEVEL AT 3.43 m UPON COMPLETION. BOREHOLE GROUTED WITH BENTONITE TO SURFACE.														

+³, ׳: Numbers refer to Sensitivity   20 15-⊕-5 10 (%) STRAIN AT FAILURE

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Ontario

THURBER

RECORD OF BOREHOLE No 06-16   1 OF 1   METRIC

W.P.	473-93-00	LOCATION	N 5 054 719.98 E 311 568.67 Ontario Street Overpass (NBL)	ORIGINATED BY	SLL
HWY	11	BOREHOLE TYPE	Hand Excavation	COMPILED BY	JHL
DATUM	Geodetic	DATE	24.07.06 - 24.07.06	CHECKED BY	AEG

SOIL PROFILE			SAMPLES					DYNAMIC CONE PENETRATION RESISTANCE PLOT			REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa	PLASTIC LIMIT / NATURAL MOISTURE CONTENT / LIQUID LIMIT — WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	GR SA SI CL
323.8 0.0	REFUSAL AT 0.0 m ON EXPOSED BEDROCK OUTCROP.						323				

$+^3$, $\times^3$: Numbers refer to Sensitivity $15\text{-}\overset{20}{\underset{10}{\diamond}}\text{-}5$ (%) STRAIN AT FAILURE

ONTMT4S 2331.GPJ 18/08/06

Ministry of
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THURBER

RECORD OF BOREHOLE No 06-18   1 OF 2   METRIC

W.P.	473-93-00	LOCATION	N 5 054 723.00 E 311 574.66 Ontario Street Overpass (NBL)	ORIGINATED BY	SLL
HWY	11	BOREHOLE TYPE	NQ Coring	COMPILED BY	JHL
DATUM	Geodetic	DATE	25.07.06 - 25.07.06	CHECKED BY	AEG

SOIL PROFILE			SAMPLES					DYNAMIC CONE PENETRATION RESISTANCE PLOT	REMARKS & GRAIN SIZE DISTRIBUTION (%)

SOIL PROFILE: ELEV DEPTH / DESCRIPTION / STRAT PLOT
SAMPLES: NUMBER / TYPE / "N" VALUES
GROUND WATER CONDITIONS
ELEVATION SCALE

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH kPa
O UNCONFINED   + FIELD VANE
● QUICK TRIAXIAL   × LAB VANE
WATER CONTENT (%) 20 40 60 80 100

PLASTIC LIMIT wP — NATURAL MOISTURE CONTENT w —O— LIQUID LIMIT wL
WATER CONTENT (%) 20 40 60

UNIT WEIGHT γ kN/m³

REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

ELEV/DEPTH	DESCRIPTION	NUM	TYPE	γ	REMARKS
324.7 / 0.0	TOPSOIL: (150 mm)				
0.2	Silty SAND / Brown				
324.1	Moist			FI	
0.6	BEDROCK / White and black, crystalline, fresh, occasional quartz intrusions, very strong, GRANODIORITE	1 RUN		3 / 5 / 2	RUN 1# / TCR=96%, SCR=64%, RQD=96%, UCS=165MPa
				3	
		2 RUN		3 / 1 / 0	RUN 2# / TCR=100%, SCR=100%, RQD=93%, UCS=142MPa
				1 / 1	
	Quartz intrusion at 3.79 m	3 RUN		5 / 1 / 0	RUN 3# / TCR=98%, SCR=98%, RQD=85%, UCS=142MPa
				2 / 0	
		4 RUN		1 / 2 / 2	RUN 4# / TCR=100%, SCR=100%, RQD=92%, UCS=155MPa
				2	
	Quartz vein from 7.18 to 7.26 m / Subvertical joints from 7.26 to 7.37 m, and from 7.64 to 7.87 m / Slightly weathered, rough joint surface	5 RUN		2 / 1 / 7	RUN 5# / TCR=100%, SCR=100%, RQD=92%, UCS=204MPa
				5 / 2	
		6 RUN		1 / 4 / 2	RUN 6# / TCR=100%, SCR=100%, RQD=85%, UCS=141MPa
				1 / 6	
	Subvertical joint from 9.45 to 9.70 m / Slightly weathered, rough joint surfaces			6	RUN 7# / TCR=100%,

Elevation scale values: 324, 323, 322, 321, 320, 319, 318, 317, 316, 315

Continued Next Page

+³ ,×³ : Numbers refer to Sensitivity   20 / 15—O—5 / 10   (%) STRAIN AT FAILURE

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Ministry of Transportation
Ontario

THURBER

W.P. 473-93-00	LOCATION N 5 054 723.00 E 311 574.66 Ontario Street Overpass (NBL)
HWY 11	BOREHOLE TYPE NQ Coring
DATUM Geodetic	DATE 25.07.06 - 25.07.06

ORIGINATED BY SLL
COMPILED BY JHL
CHECKED BY AEG

SOIL PROFILE		SAMPLES					DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH kPa	NATURAL MOISTURE / WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE			GR SA SI CL

Legend in header:
- DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
- SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE : 20 40 60 80 100
- PLASTIC LIMIT wP, NATURAL MOISTURE CONTENT w, LIQUID LIMIT wL
- WATER CONTENT (%): 20 40 60

ELEV DEPTH	DESCRIPTION	SAMPLE No	TYPE	ELEV SCALE	UNIT WEIGHT	REMARKS
		7	RUN		1	SCR=95%, RQD=57%, UCS=148MPa
				314	4	
313.9					3	
10.7	BEDROCK GRANITIC GNEISS				10	
	Pink, white and black, crystalline, fresh, occasional black banding, very strong, GRANITIC GNEISS				10	RUN 8# TCR=100%, SCR=85%, RQD=58%, UCS=113MPa
	Subvertical joint from 10.92 to 10.95 m	8	RUN		2	
	Slightly weathered, smooth joint surface			313		
	Rubble zone from 11.16 to 11.28 m				2	
	Subvertical joint from 12.12 to 12.29 m Slightly weathered, rough joint surface				5	
	Rubble zone from 12.42 to 12.50 m				10	
	Rubble zones from 12.68 to 12.77 m, and from 12.85 to 12.90 m	9	RUN	312	10	RUN 9# TCR=100%, SCR=90%, RQD=77%, UCS=131MPa
					2	
				311	1	
					2	
					1	
		10	RUN		1	RUN 10# TCR=100%, SCR=100%, RQD=95%, UCS=187MPa
				310	3	
					0	
					4	
	Subvertical joint from 15.44 to 15.49 m Fresh, rough joint surface			309		RUN 11# TCR=100%, SCR=98%, RQD=82%, UCS=151MPa
		11	RUN		1	
					0	
	Rubble zone from 16.66 to 16.71 m			308	0	
					4	
					6	
	Subvertical joints from 17.35 to 17.58 m, and from 17.88 to 17.95 m Slightly weathered, rough joint surfaces Mica zone from 17.74 to 18.12 m			307	3	RUN 12# TCR=100%, SCR=100%, RQD=89%, UCS=186MPa
		12	RUN		6	
					1	
					0	
305.9				306	1	
18.8	END OF BOREHOLE AT 18.77 m. Piezometer installation consists of 19 mm diameter Schedule 40 PVC pipe with a 1.52 m slotted screen. WATER LEVEL READINGS:					

WATER LEVEL READINGS:
DATE   DEPTH(m)   ELEV.(m)
07/26/06   4.50   320.2
08/20/06   4.50   320.2

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+³, ׳: Numbers refer to Sensitivity
15-⊕-5 / 20 / 10 (%) STRAIN AT FAILURE

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Ontario

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RECORD OF BOREHOLE No 06-19   1 OF 1   METRIC

W.P. 473-93-00	LOCATION N 5 054 721.00 E 311 583.83 Ontario Street Overpass (NBL)	ORIGINATED BY SLL
HWY 11	BOREHOLE TYPE Hand Excavation	COMPILED BY JHL
DATUM Geodetic	DATE 25.07.06 - 25.07.06	CHECKED BY AEG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT	PLASTIC LIMIT wp — NATURAL MOISTURE CONTENT w — LIQUID LIMIT wL	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100	WATER CONTENT (%) 20 40 60		GR SA SI CL
322.1								20 40 60 80 100			
0.0	TOPSOIL: (150 mm)										
0.2	Clayey SILT						322				
321.7	Brown										
0.5	Moist										
	END OF BOREHOLE AT 0.46 m. REFUSAL AT 0.46 m ON PROBABLE BEDROCK OR BOULDERS. BOREHOLE BACKFILLED WITH CUTTINGS TO SURFACE.										

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+³, ׳ :   Numbers refer to Sensitivity   15 ⊕ 5 / 20 / 10   (%) STRAIN AT FAILURE

W.P.	473-93-00	LOCATION	N 5 054 725.88 E 311 583.39 Ontario Street Overpass (NBL)	ORIGINATED BY	SLL
HWY	11	BOREHOLE TYPE	Hollow Stem Augers	COMPILED BY	JHL
DATUM	Geodetic	DATE	25.07.06 - 25.07.06	CHECKED BY	AEG

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH kPa					PLASTIC LIMIT WP — NATURAL MOISTURE CONTENT w — LIQUID LIMIT WL / WATER CONTENT (%)			UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
								20	40	60	80	100	20	40	60		
322.4 0.0	TOPSOIL: (150 mm)																
0.2	Clayey SILT, some sand seams		1	SS	56/								○				
321.8	Hard				225		322										
0.5	Brown																
	Moist																
	END OF BOREHOLE AT 0.53 m.																
	REFUSAL AT 0.53 m ON PROBABLE BEDROCK OR BOULDERS.																
	BOREHOLE BACKFILLED WITHCUTTINGS TO SURFACE.																

$+^{3}, \times^{3}$: Numbers refer to Sensitivity $15 \overset{20}{\underset{10}{\oplus}} 5$ (%) STRAIN AT FAILURE

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Appendix B

Laboratory Test Results

SILTY SAND TO SAND

Size of openings, inches   U.S.S. Sieve size, meshes/inch

6" 4" 4" 3"   1½" 1" ¾" ½" ⅜" 3   4   8 10 16   30 40 50 60 100   200

PERCENT FINER THAN

GRAIN SIZE, mm

COBBLE SIZE	COARSE	FINE	COARSE	MEDIUM	FINE	SILT and CLAY
	GRAVEL			SAND		FINE GRAINED

SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	06-14	3.35	318.12
⊠	06-15	1.83	321.93

Date · August 2006

Project   473-93-00

THURBER

Prep'd ...... JHL

Chkd. ...... MEF
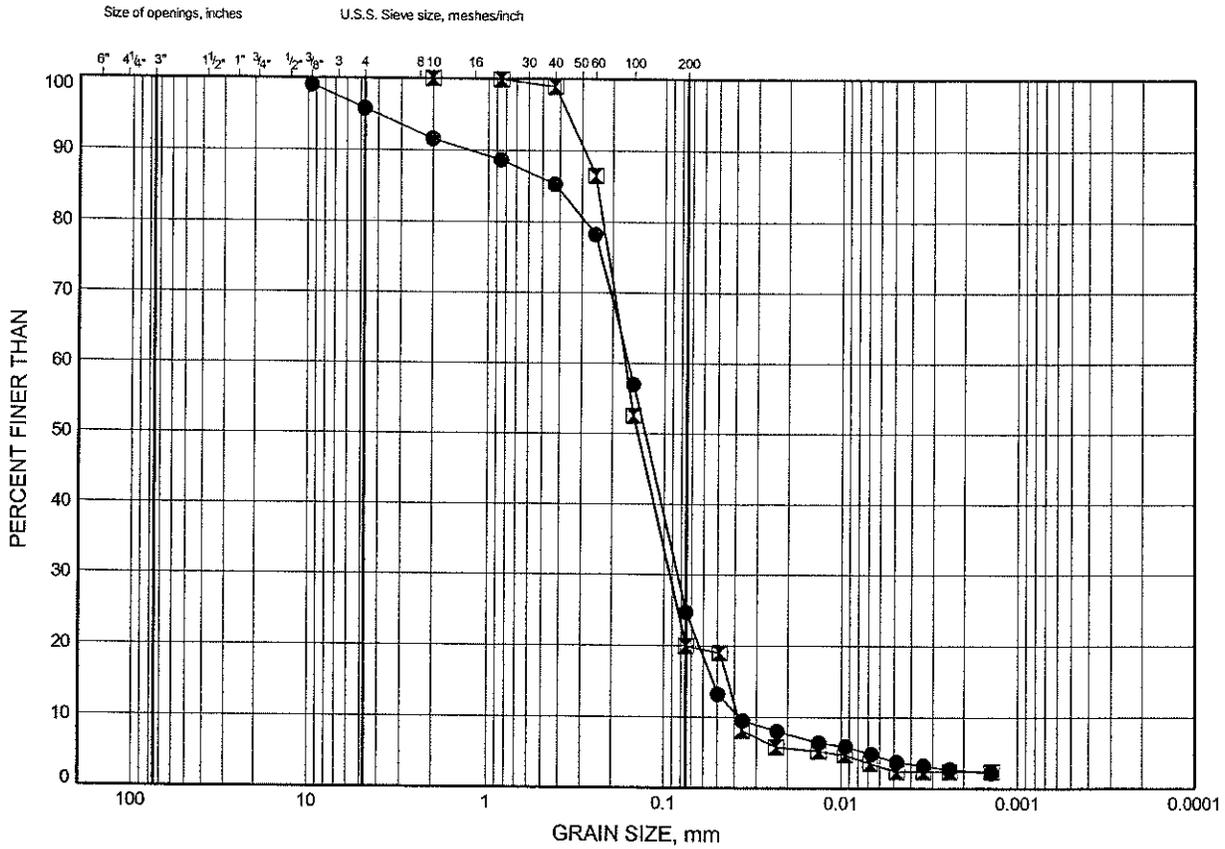
SILT, SOME CLAY TO SILTY CLAY
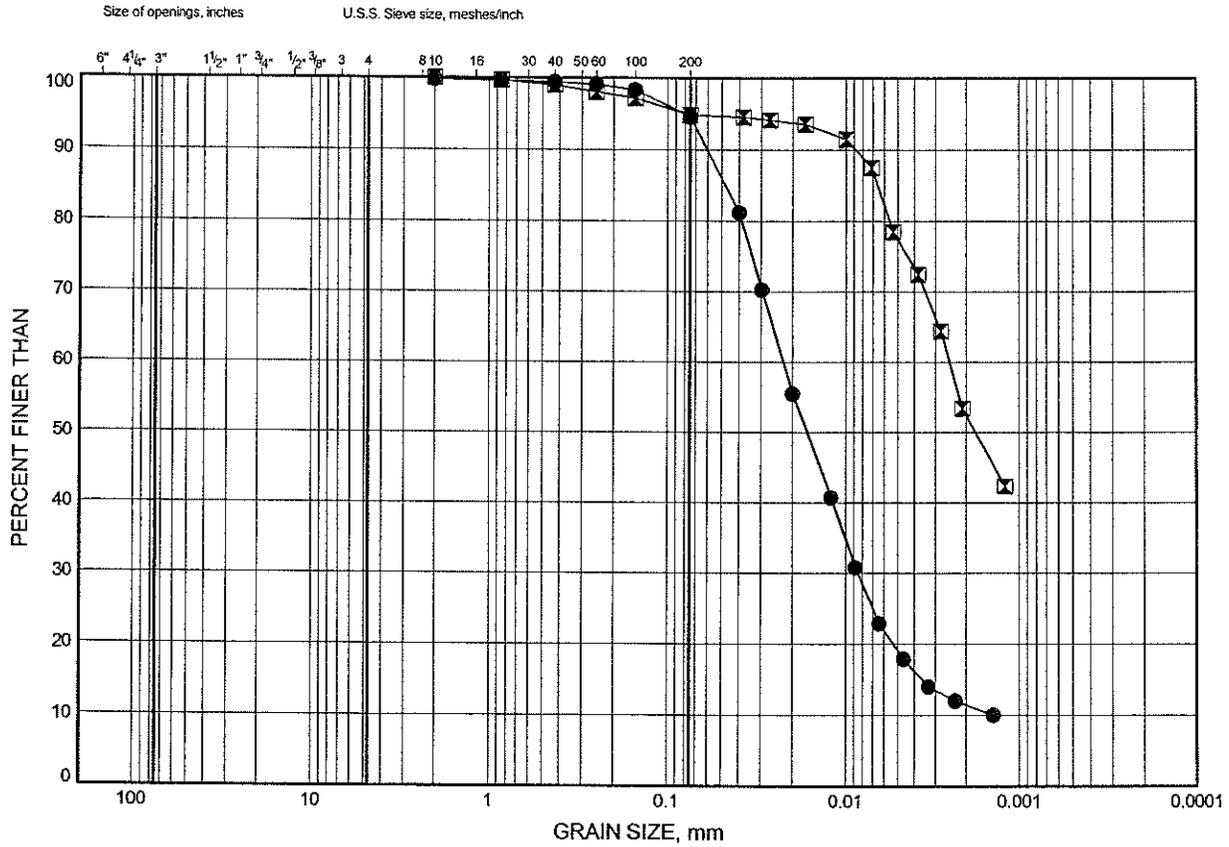
COBBLE SIZE	COARSE	FINE	COARSE	MEDIUM	FINE	SILT and CLAY
	GRAVEL		SAND			FINE GRAINED

SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	06-13	1.83	320.63
⊠	06-14	1.83	319.65

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INTERMEDIATE PLASTICITY CLAY
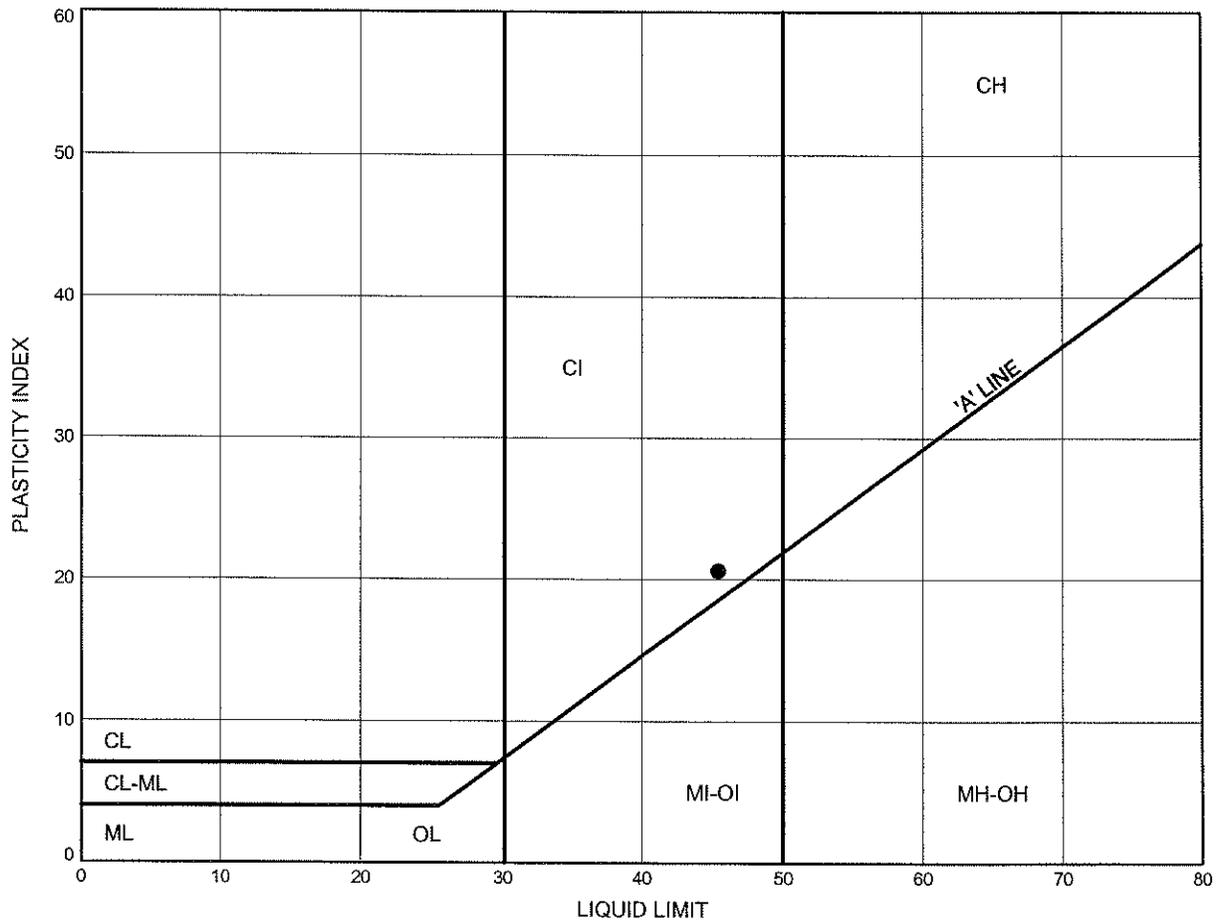
SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	06-14	1.83	319.65

Date   November 2006

Project   473-93-00

THURBER
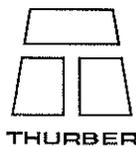
Prep'd   JHL

Chkd.   MEF

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TABLE B1 - Point Load Test Results
Ontario Street Overpass NBL

Depth Feet	Inches	m	Is50	UCS (MPa)
06-13				
9	0	2.74	3.91	93.91
10	0	3.05	6.09	146.06
10	10	3.30	6.09	146.06
12	0	3.66	6.09	146.06
13	2	4.01	5.22	125.21
14	2	4.32	5.43	130.42
15	2	4.62	6.09	146.06
17	3	5.26	5.65	135.65
18	3	5.56	6.09	146.06

Total Rock Core

Average	Minimum	Maximum	
135	94	146	MPa

Run #	Average
1	131.46
2	139.55

Depth Feet	Inches	m	Is50	UCS (MPa)
06-18				
2	6	0.76	5.64	135.26
3	8	1.12	3.30	79.25
4	5	1.35	8.01	192.34
5	5	1.65	9.03	216.62
6	3	1.91	8.37	200.78
7	6	2.29	6.08	145.82
8	5	2.57	4.05	97.22
9	4	2.84	7.09	170.14
10	5	3.18	5.90	141.60
11	4	3.45	6.52	156.41
12	8	3.86	4.40	105.67
13	8	4.17	5.81	139.49
14	8	4.47	6.91	165.91
15	9	4.80	6.56	157.46
17	0	5.18	6.16	147.94
18	3	5.56	5.33	127.87
19	4	5.89	6.30	151.10
20	6	6.25	8.01	192.34
21	6	6.55	11.32	271.58
22	6	6.86	8.45	202.90
24	6	7.47	5.76	138.24
26	8	8.13	5.28	126.82
27	10	8.48	5.06	121.54
28	11	8.81	7.05	169.08
30	0	9.14	6.12	146.88
32	5	9.88	8.37	200.78
33	4	10.16	7.18	172.25
35	3	10.74	2.99	71.66
39	0	11.89	1.98	47.54
40	3	12.27	7.40	177.53
42	6	12.95	3.04	72.91
44	3	13.49	6.43	154.27
45	6	13.87	5.46	131.04

Total Rock Core

Average	Minimum	Maximum	
156	45	315	MPa

Run #	Average
1	164.85
2	142.24
3	142.13
4	154.81
5	204.24
6	141.08
7	148.30
8	112.54
9	131.45
10	186.62
11	150.58
12	185.63

Feet	Depth Inches	m	Is50	UCS (MPa)
06-18	(cont'd)			
46	8	14.22	1.89	45.43
47	2	14.38	10.57	253.61
48	2	14.68	8.45	202.90
49	0	14.94	8.15	195.50
50	0	15.24	13.12	314.90
51	0	15.54	3.96	95.11
52	0	15.85	5.20	124.70
53	0	16.15	5.72	137.38
54	0	16.46	6.43	154.27
55	4	16.87	7.57	191.75
57	3	17.45	5.37	128.93
58	4	17.78	2.86	68.69
59	8	18.19	7.49	179.64
60	8	18.49	12.86	308.57

Total Rock Core

Average	Minimum	Maximum	
156	45	315	MPa

Run #	Average
1	164.85
2	142.24
3	142.13
4	154.81
5	204.24
6	141.08
7	148.30
8	112.54
9	131.45
10	186.62
11	150.58
12	185.63

Appendix C

Factual Information from the AGRA Report

RECORD OF BOREHOLE No ON1   1 OF 1   METRIC

W.P. 485-93-01	LOCATION Site No. 44-398N N 5054702 E 311583	ORIGINATED BY AD
DIST 52 HWY 11	BOREHOLE TYPE Hollow Stem	COMPILED BY AD
DATUM Geodetic	DATE 5 May 1999	CHECKED BY EYC

ELEV DEPTH	DESCRIPTION (SOIL PROFILE)	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION / SHEAR STRENGTH kPa	NATURAL MOISTURE CONTENT / WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
322.5 0.0	brown SAND with Gravel, trace Rootlets compact, damp		1	SS	23				o (10)		STATION 21+121 6 RT NBL C/L
321.9 0.6	brown SILTY SAND some gravel trace Rootlets very dense, damp		2	SS	62/29		322		o (15)		
321.0 1.5	brown SAND & GRAVEL frequent cobbles, some Silt very dense, moist		3	SS	50/13		321		o (10)		40 49 (11)
			4	RC							
319.8 2.7			5	RC			320				RC5: REC=65% RQD=60%
	GNEISS BEDROCK massive, with frequent mica inclusions closely to moderately closely jointed		6	RC			319				RC6: REC=100% RQD=100%
			7	RC			318				RC7: REC=100% RQD=83%
			8	RC			317				RC8: REC=100% RQD=100%
	occasional silt infillings		9	RC			316				RC9: REC=100% RQD=39%
314.9 7.6	END OF BOREHOLE No water in borehole prior to coring.						315				

$+^3, \times^3$: Numbers refer to Sensitivity $O^{3\%}$ STRAIN AT FAILURE

AGRA

RECORD OF BOREHOLE No ON2   1 OF 1   METRIC

W.P.	485-93-01	LOCATION	Site No. 44-398N N 5054669 E 311584	ORIGINATED BY	AD
DIST	52 HWY 11	BOREHOLE TYPE	Hollow Stem	COMPILED BY	AD
DATUM	Geodetic	DATE	8 May 1999	CHECKED BY	EYC

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH kPa	PLASTIC LIMIT — NATURAL MOISTURE CONTENT — LIQUID LIMIT / WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV/DEPTH	DESCRIPTION (STRAT PLOT)	NUMBER	TYPE	"N" VALUES						GR SA SI CL
321.8 / 0.0	0.2m TOPSOIL brown SILTY SAND very loose, damp	1	SS	3						STATION 21+058 9 RT NBL C/L
321.0 / 0.8	brown SAND some Silt compact, damp	2	SS	27		321				
		3	SS	27		320				
319.7 / 2.1	SILT brown SAND & GRAVEL frequent cobbles and boulders some Silt dense to very dense, damp	4	SS	50/4						N=50/4 denotes 50 blows for 4 cm penetration
		5	SS	46		319				37 47 (16)
318.2 / 3.6		6	SS	50/13						
3.8	0.1 m fractured					318				RC7: REC=97% RQD=90%
	GNEISS BEDROCK massive occasional Mica zones closely to moderately closely jointed	7	RC			317				
		8	RC			316				RC8: REC=89% RQD=72%
		9	RC			315				RC9: REC=96% RQD=66%
		10	RC			314 / 313				RC10: REC=97% RQD=60%
	frequent Mica zones 0.1 to 0.2 m thick	11	RC			312 / 311				RC11: REC=100% RQD=67%
		12	RC			310				RC12: REC=79% RQD=55%
		13	RC			309				RC13: REC=94% RQD=38% RC14: REC=97% RQD=69%
		14	RC			308				RC15: REC=100% RQD=100%
	occasional Sand infillings	15	RC			307				
306.5 / 15.3	END OF BOREHOLE Water Level in PIEZOMETER July 9/99: 7.3m									

+³ ×³ Numbers refer to Sensitivity   O ³% STRAIN AT FAILURE

RECORD OF BOREHOLE No ON3   1 OF 1   METRIC

W.P. 485-93-01	LOCATION Site No. 44-398N N 5054682 E 311576	ORIGINATED BY AD
DIST 52 HWY 11	BOREHOLE TYPE Backhoe	COMPILED BY AD
DATUM Geodetic	DATE 28 June 1999	CHECKED BY EYC

SOIL PROFILE		SAMPLES					DYNAMIC CONE PENETRATION RESISTANCE PLOT	NATURAL MOISTURE	UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV DEPTH 323.1	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa / WATER CONTENT (%)	γ kN/m³	GR SA SI CL
0.0	TOPSOIL with cobbles and boulders						323			STATION 21+101 1 RT NBL C/L
322.5 0.6	brown SAND with Gravel frequent cobbles, boulders						322			
321.6 damp										
1.5	END OF TESTPIT ON BEDROCK									
	Water Level on Completion: dry									

$+^3 . \times^3 .$ Numbers refer to Sensitivity $\bigcirc^{3\%}$ STRAIN AT FAILURE

AGRA

Foundation Design

W.P.	485-93-01		LOCATION	Site No. 44-398N N 5054692 E 311573		ORIGINATED BY	AD
DIST	52 HWY 11		BOREHOLE TYPE Backhoe			COMPILED BY	AD
DATUM	Geodetic		DATE	28 June 1999		CHECKED BY	EYC

SOIL PROFILE		SAMPLES					DYNAMIC CONE PENETRATION RESISTANCE PLOT							REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	20 40 60 80 100 SHEAR STRENGTH kPa O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100	PLASTIC LIMIT Wp	NATURAL MOISTURE CONTENT W	LIQUID LIMIT WL	UNIT WEIGHT γ		
324.0										WATER CONTENT (%) 10 20 30			kN/m²	GR SA SI CL
0.0	0.15 m TOPSOIL brown SILTY SAND some cobbles and boulders damp													STATION 21+112 2 LT NBL C/L
323.1 0.9	END OF TESTPIT ON BEDROCK Water Level on Completion: dry													

+³. ×³. Numbers refer to Sensitivity   O ³% STRAIN AT FAILURE

AGRA

Foundation Design

RECORD OF BOREHOLE No ON5   1 OF 1   METRIC

W.P.	485-93-01	LOCATION	Site No. 44-398N N 5054705 E 311585	ORIGINATED BY	AD
DIST	52 HWY 11	BOREHOLE TYPE	Backhoe	COMPILED BY	AD
DATUM	Geodetic	DATE	28 June 1999	CHECKED BY	EYC

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV/DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100 SHEAR STRENGTH kPa O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100					w_P	w	w_L WATER CONTENT (%) 10 20 30		γ kN/m³	GR SA SI CL
322.2 0.0	0.3m TOPSOIL brown, SILTY SAND with cobbles and boulders some organics, damp						322										STATION 21+124 11 RT NBL C/L
320.8 1.4	END OF TESTPIT ON BEDROCK Water Level on Completion: dry						321										

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

AGRA

Foundation Design

W.P. 485-93-01	LOCATION Site No. 44-398N N 5054713 E 311575	ORIGINATED BY AD
DIST 52 HWY 11	BOREHOLE TYPE Backhoe	COMPILED BY AD
DATUM Geodetic	DATE 28 June 1999	CHECKED BY EYC

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT w_p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w_L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100 SHEAR STRENGTH kPa O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100					WATER CONTENT (%) 10 20 30			kN/m³	GR SA SI CL
324.4 0.0	0.3m TOPSOIL brown SAND some Gravel, some Silt with cobbles and boulders damp		1	GS			324						o				20 62 (18)
			2	GS									o				STATION
323.2 1.2	END OF TESTPIT ON BEDROCK Water Level on Completion: dry																21+132 1 RT NBL C/L

+³,×³ Numbers refer to Sensitivity   O³% STRAIN AT FAILURE

RECORD OF BOREHOLE No ON7   1 OF 1   METRIC

W.P.	485-93-01	LOCATION	Site No. 44-398N N 5054718 E 311568	ORIGINATED BY	AD
DIST	52 HWY 11	BOREHOLE TYPE	Backhoe	COMPILED BY	AD
DATUM	Geodetic	DATE	28 June 1999	CHECKED BY	EYC

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT	PLASTIC LIMIT — NATURAL MOISTURE CONTENT — LIQUID LIMIT	UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100 SHEAR STRENGTH kPa O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL X LAB VANE 20 40 60 80 100	W_P W W_L WATER CONTENT (%) 10 20 30	γ kN/m³	GR SA SI CL
323.6 0.0	EXPOSED BEDROCK AT SURFACE					323					STATION 21+138.5 LT NBL C/L

+^{3}, x^3: Numbers refer to Sensitivity   O$^{3\%}$ STRAIN AT FAILURE

AGRA

RECORD OF BOREHOLE No ON8   1 OF 1   METRIC

W.P. 485-93-01	LOCATION Site No. 44-398N N 5054666 E 311574	ORIGINATED BY AD
DIST 52 HWY 11	BOREHOLE TYPE Backhoe	COMPILED BY AD
DATUM Geodetic	DATE 28 June 1999	CHECKED BY EYC

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH kPa — O UNCONFINED + FIELD VANE, ● QUICK TRIAXIAL × LAB VANE	PLASTIC LIMIT w_p — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L / WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100 / 20 40 60 80 100	10 20 30		
322.2 0.0	0.2m TOPSOIL brown SILTY SAND with cobbles and boulders						322				
321.7 0.5	grey/brown CLAYEY SILT varve-like structure hard		1	GS					⊢—o—⊢		STATION 21+085 1 LT NBL C/L
			2	GS			321				
							320				
319.4 2.8	END OF TESTPIT ON BEDROCK										
	Water Level on Completion: dry										

+³.×³: Numbers refer to Sensitivity   O ³% STRAIN AT FAILURE

RECORD OF BOREHOLE No ON9　　1 OF 1　　METRIC

W.P. 485-93-01	LOCATION Site No. 44-398N N 5054662 E 311587	ORIGINATED BY AD
DIST 52 HWY 11	BOREHOLE TYPE Backhoe	COMPILED BY AD
DATUM Geodetic	DATE 28 June 1999	CHECKED BY EYC

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH kPa	PLASTIC LIMIT w_P – NATURAL MOISTURE CONTENT w – LIQUID LIMIT w_L / WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
320.7 0.0	0.15m TOPSOIL										
320.3 0.4	brown SILTY SAND with cobbles and boulders										
	grey/brown CLAYEY SILT varve-like structure hard						320				STATION 21+082 12 RT NBL C/L
							319				
							318				
317.0 3.7	brown/grey SILTY SAND						317				
316.4 4.3	END OF TESTPIT ON BEDROCK

Water Level on Completion: dry | | | | | | | | | | |

$+^3, \times^3$. Numbers refer to Sensitivity　　$\bigcirc^{3\%}$ STRAIN AT FAILURE

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RECORD OF BOREHOLE No ON10   1 OF 1   METRIC

W.P.	485-93-01	LOCATION	Site No. 44-398N N 5054731 E 311574	ORIGINATED BY	AD
DIST	52 HWY 11	BOREHOLE TYPE	Backhoe	COMPILED BY	AD
DATUM	Geodetic	DATE	28 June 1999	CHECKED BY	EYC

SOIL PROFILE		SAMPLES					DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	20 40 60 80 100 SHEAR STRENGTH kPa O UNCONFINED + FIELD VANE # QUICK TRIAXIAL × LAB VANE 20 40 60 80 100		w_p	w WATER CONTENT (%) 10 20 30	w_L	γ kN/m³	GR SA SI CL

ELEV/DEPTH	DESCRIPTION	ELEVATION SCALE	REMARKS
324.5 / 0.0	0.3m TOPSOIL		
324.1 / 0.5	brown, SILTY SAND with cobbles and boulders		
	END OF TESTPIT ON BEDROCK	324	
	Water Level on Completion: dry		STATION 21+150 2 RT NBL C/L

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

▲ AGRA

W.P. 485-93-01	LOCATION Site No. 44-398N N 5054731 E 311587	ORIGINATED BY AD
DIST 52 HWY 11	BOREHOLE TYPE Backhoe	COMPILED BY AD
DATUM Geodetic	DATE 28 June 1999	CHECKED BY EYC

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT	PLASTIC LIMIT / NATURAL MOISTURE CONTENT / LIQUID LIMIT	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV/DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa — O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL X LAB VANE / WATER CONTENT (%)		kN/m³	GR SA SI CL
321.8 / 0.0	0.2m TOPSOIL brown						20 40 60 80 100 / 20 40 60 80 100 / 10 20 30			
	SILTY SAND with gravel, cobbles and boulders									
321.1 / 0.7	brown/grey **SAND & GRAVEL** with cobbles and boulders		1	GS			321			STATION 21+149 12 RT NBL C/L
	with silt						320			
319.8 / 2.0	END OF TESTPIT ON BEDROCK									
	Water Level on Completion: dry									

+³, x³: Numbers refer to Sensitivity   O³% STRAIN AT FAILURE

Appendix D

Foundation Comparison

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Ontario Street Overpass, NBL
Highway 11 Four-Laning at Burk's Falls

COMPARISON OF FOUNDATION ALTERNATIVES FOR EACH FOUNDATION ELEMENT

Foundation Element	Piles	Footings on Bedrock
Abutments	*Advantages:* i. High geotechnical resistance available by seating piles on bedrock. ii. Comparatively short abutment stem. iii Relatively short pile lengths required since structure is in a bedrock cut. iv. Will allow for the construction of an integral abutment structure. *Disadvantages:* i. Higher unit cost compared to footings, especially in view of the additional rock excavation required. NOT RECOMMENDED BUT FEASIBLE IF WARRANTED BY OTHER CONSIDERATIONS	*Advantages:* i. Lower unit cost compared to pile foundations. ii. High geotechnical resistance is available. *Disadvantages:* i. An integral abutment design is not an available option RECOMMENDED

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Appendix E

Special Provisions

The following Special Provisions are referenced in this report:

110F13
105S10
Amendment to OPSS 206, December 1993
902S01
903S01

The suggested wording for the modification of OPSS 501 is as follows:

501.08.02 Method A shall be replaced by the following:

5.0.08.02 Method a

Granular materials shall be compacted to 100% of the maximum dry density and earth materials shall be compacted to 100% of the maximum dry density.

Suggested wording regarding competence of bedrock benches is as follows:

If a footing is to be constructed on a bench or ledge in the bedrock, the bedrock shall be sound and stable. The Contractor shall cooperate with a qualified rock specialist hired by the Contract Administrator to inspect and approve the rock below the footing prior to placing any concrete. The Contractor shall provide the rock specialist with access to the site of the foundation excavation and shall assist him with the inspection, as required. The rock specialist shall verify that the bedrock below the footing is sound, free of shatter and not prone to failure or unacceptable movements due to mechanisms including, but not limited to, sliding or toppling. The Contractor shall implement the recommendations of the rock specialist.

Suggested wording regarding neat trench excavation in bedrock is as follows:

A "neat" trench shall mean a trench excavated in bedrock such that the sides and base are composed of undisturbed bedrock with no shatter and no partially dislodged fragments of bedrock protruding into the sides or base of the excavation.

The Contractor is advised that the excavation of such a trench will require the services for a blast designer familiar with such work. Special procedures such as line-drilling, pre-splitting and modified charge patterns may be required.

The walls of the trench should be effectively vertical on completion of excavation.

Other blasting and construction activities in the vicinity of the trench must be coordinated so as not to damage the neat excavation.

THURBER

MASS CONCRETE, Item No.

Non-Standard Special Provision

SCOPE

This Special Provision covers the requirements for the mass concrete pad under the pier footing. The purpose of the mass concrete pad is to provide a level working surface for the construction of the pier footing on the irregular founding rock surface as per contract drawings and documents

CONSTRUCTION

Work under this item shall follow the following requirements:

- The surface of the pier footing founding rock shall be exposed, cleaned and any loose or fractured parts removed so that sound rock is exposed.
- The mass concrete shall have a minimum 28 day strength of 30 MPa
- The mass concrete shall be placed on the exposed cleaned sound founding rock surface as per the contract drawings and documents.
- Thickness of the mass concrete pad shall depend on the slope and irregularities in the exposed founding rock surface. A nominal thickness and a footprint plan view area has been specified on the contract drawings and documents
- Unwatering of the excavation for the pier footing construction, including the construction of the mass concrete pad, might be required and is covered under separate Tender Item. The dewatering scheme shall be done in such a manner as to prevent any disturbance to the surrounding original soil.

BASIS OF PAYMENT

Payment at the contract price for this Tender Item shall include full compensation for all labour, equipment and material required to do the work.

THURBER

Rock Trench Excavation (Abutment Foundations)

Scope

This Special Provision covers additional requirements (beyond AMENDMENT TO OPSS 120, August 1994) for the excavation of rock trenches to receive abutment piles.

Construction

The sides and base of the trench shall be excavated neat and shall be free of shatter or disturbed material.

The Contractor shall design a controlled blasting procedure to achieve the required excavation. The procedure shall include line drilling and preshearing to enhance the stability of the final trench wall. All other aspects of the blasting and mucking procedures shall be designed by the Contractor.

The Contractor shall make good any damage to the founding surface or trench walls prior to constructing the foundation.

Payment

Appropriate payment terms to be filled in.

CSP FOR RSS FALSE ABUTMENT - Item No.

Special Provision   October 2000

SCOPE

This specification covers the requirements for the installation of the double CSP's, including concrete pads, sand fill and polystyrene sheets, at the RSS False Abutments.

REFERENCES

This specification refers to the following standards, specifications or publications:

Ontario Provincial Standard Specifications, General:

OPSS 180   Management and Disposal of Excess Materials

Ontario Provincial Standard Specifications, Construction:

OPSS 904   Concrete
OPSS 909   Prestressed Concrete - Precast Members

Ontario Provincial Standard Specifications, Material:

OPSS 1350   Concrete - Materials and Production
OPSS 1605   Expanded Extruded Polystyrene
OPSS 1801   Corrugated Steel Pipe Products

Canadian Standards Association Standards:

CSA G164-M   Galvanizing of Irregularly-Shaped Articles

Ministry of Transportation Publications

MTO Manual of Designated Sources of Materials

DEFINITIONS

For the purposes of this specification, the following definitions apply:

Abutment Stem: means the cast-in-place concrete component of the RSS false abutment placed over the top of the piles and forming the bearing seat for the girders.

CSP: means helical corrugated steel pipe.

Design Engineer: means the Engineer who produces the design and/or working drawings, and who has a minimum of five (5) years in the design and/or construction of bridges.

RSS: means retained soil system.

RSS False Abutment: means an abutment where the lateral earth pressure loads are carried by RSS walls and the bridge superstructure vertical loads are carried by piles; the RSS is insulated from the effects of flexure of the piles due to lateral movements of the superstructure by a system of double concentric CSP's placed over the piles.

SUBMISSION AND DESIGN REQUIREMENTS

Submissions

All submissions shall bear the seal and signature of the Design Engineer.

At least two weeks prior to commencement of installation of the RSS false abutment, the Contractor shall submit to the Contract Administrator, for information purposes only, three (3) sets of the working drawings.

The Contractor shall have a copy of the submitted working drawings on site at all times.

Working Drawing Requirements

Working drawings shall include at least the following:

1. Layout and Elevations of the CSP's and concrete pads;
2. Location of reference points, and location of the centroid of each pile with respect to the reference points at the level of the concrete pad;
3. Source of the sand fill, and description of placing method and equipment;
4. Statement that the Contractor's selected RSS False Abutment is compatible with the CSP size and spacing specified on the Contract drawings;
5. Location and details of all temporary bracing, including permanent and temporary spacers, for the piles, CSP's, abutment stems, and RSS false abutments;
6. Detailed construction sequence for the work, including installation and removal of the temporary bracing.

Design Requirements

The selection of the RSS false abutment shall be as specified elsewhere in the Contract.

The Contractor shall be responsible for the complete detailed design of the construction sequence for the work, including the installation and removal of all temporary bracing. The general sequence of construction shall be as shown on the Contract drawings.

The Contractor shall be responsible for the complete detailed design of all temporary bracing, including temporary and permanent spacers, required to maintain the piles, CSP's, abutment stems, RSS false abutments, and girders in their specified positions through all stages of construction until concrete in deck has reached a compressive strength of 25 MPa. All temporary bracing, except spacers identified as permanent on the Contract drawings, shall be removed.

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Temporary bracing for prestressed, precast girders shall meet the requirements of OPSS 909.

MATERIAL

Concrete Pad

Concrete shall be in accordance with OPSS 1350.

Corrugated Steel Pipe

CSP shall be in accordance with OPSS 1801, and shall be from a supplier listed under DSM # 4.60.80. The CSP shall be of the diameter and wall thickness specified on the Contract drawings, and shall be galvanized in accordance with CSA G164-M.

Permanent Spacers and Associated Hardware

Permanent spacers and associated hardware left in place shall not consist of wood and corrodible material.

Sand Fill

The sand fill for backfilling the inner CSP shall meet the gradation requirements of Table 1 below:

Table 1 - Sand Fill Gradation Requirements

MTO Sieve Designation		Percentage Passing by Mass
2 mm	# 10	100 %
600 μm	# 30	80 % to 100 %
425 μm	# 40	40 % to 80 %
250 μm	# 60	5 % to 25 %
150 μm	# 100	0 % to 6 %

EXPANDED EXTRUDED

Expanded extruded polystyrene shall be in accordance with OPSS 1605, and shall be from a supplier listed under DSM # 3.30.30.

CONSTRUCTION

GENERAL

THURBER

The sequence of construction for installing the concrete pads, CSP's, sand fill, abutment stems, and RSS false abutment, including the installation and removal of the temporary bracing, shall be in accordance with the working drawings.

The Contractor shall not proceed with the RSS false abutment backfill above the level of the concrete pad without written permission from the Contract Administrator.

CONCRETE PAD

Concrete shall be in accordance with OPSS 904.

CORRUGATED STEEL PIPE

CSP's shall be supplied in the lengths and with the end treatments, either square or skew, as specified on the Contract drawings; field cutting and splicing of CSP's will not be permitted. Cut ends shall be neat and free of burrs. The planes defined by the end treatments of each CSP shall be parallel to each other.

Handling and storage of CSP's shall be in accordance with the manufacturer's recommendations. Damaged CSP's shall be rejected. Localized areas of damaged galvanizing on otherwise acceptable CSP's shall be repaired by two coats of zinc-rich paint.

The Contractor shall set the inner and outer CSP over each pile in the abutment into the concrete pad, following the batter of the pile, while the concrete in the concrete pad is still plastic. The CSP's shall extend at least 150 mm into the concrete pad.

The Contractor shall ensure the full perimeter of the tops of all CSP's at each abutment are at the Elevation shown on the working drawings.

After the CSP's have been set into the concrete pads, the Contractor shall take all measures necessary to prevent the ingress of water, backfill and debris into the CSP's.

SAND FILL

The sand fill shall be placed dry of optimum and free-flowing, completely filling the volume between the inner CSP and pile. No additional compaction effort other than the action of placing the sand fill itself shall be applied to the sand fill.

The placing of the sand fill shall be carried out in a manner such as to not damage and displace the CSP's.

After the sand fill has been placed to the top of each inner CSP, the Contractor shall take all measures necessary to prevent the ingress of water and other liquids into the sand fill until after the concrete in the abutment stem has been placed and cured.

EXPANDED EXTRUDED POLYSTYRENE

The expanded extruded polystyrene sheets shall completely cover the area under the abutment stem, exclusive of the RSS false abutment panels, as shown on the Contract drawings. The sheets shall be placed in one piece for the width of the abutment stem, with butt joints perpendicular to the centre-line of abutment bearings. The minimum length of sheet shall be 500 mm.

Joints between sheets within 500 mm of a pile centre-line will not be permitted. At each pile location, a minimum 1000 mm long sheet shall be centred on the pile and a 500 mm diameter hole neatly cut in the sheet so as to fit over the pile in one piece, fully spanning the annular space between the double CSP's.

The Contractor shall adjust the RSS false abutment backfill to ensure full and uniform contact of the sheets with the backfill and the full perimeter of the tops of the CSP's. The vertical step at joints between sheets shall not exceed 5 mm.

The Contractor shall protect the sheets from damage during installation of the reinforcing for the abutment stem, and shall secure the sheets from "floating" during placing of the concrete in the abutment stem. Only hardware approved by the Owner shall be used to secure the sheets. All hardware used to secure the sheets shall be installed so as not to project above the top surface of the sheets into the abutment stem.

TEMPORARY BRACING

Temporary bracing shall be installed and removed in accordance with the working drawings.

The temporary bracing shall not distort, nor pierce the walls of, the CSP's. Welding to the CSP's will not be permitted.

Concrete anchors shall be removed and the holes filled with non-shrink grout.

TOLERANCES

The CSP's at each pile shall be constructed to the following tolerances:

Criteria	Tolerance
Maximum deviation of inner and outer CSP from pile centroid.	± 25 mm
Maximum deviation from specified spacing between inner and outer CSP's.	± 25 mm
Maximum deviation of any point on the top perimeter of the CSP's from the specified Elevation.	± 10 mm

QUALITY ASSURANCE

Prior to placing concrete in the concrete pad, the Contractor shall establish reference points at each abutment and determine the location of the centroid of each pile in the abutment with respect to these reference points. The Contractor shall maintain the reference points until written permission to proceed with the RSS false abutment backfill above the level of the concrete pad has been given by the Contract Administrator.

BASIS OF PAYMENT

Payment at the contract price for the above items shall be full compensation for all labour, equipment and material required to do the work.

WARRANT: Alway with this tender item.

18   DOWELS INTO ROCK –

Special Provision	March 1,
2001	

1.0   GENERAL

1.1   Scope

The work for the above noted tender item shall be in accordance with OPSS 904, including all special provision, except as extended herein.   This document specifies additional requirements for the supply, installation and testing of Dowels into Rock for the pier footing.

1.2   Instructions to Contractor

1.2.1   These instructions are to be read in conjunction with the Contract Drawings.

1.2.2   A total of 2 test Dowels into Rock are required for the Dowels into Rock at the pier.

1.2.3   Dowels into rock at the pier shall be installed prior to unwatering the structure excavation. Dowels shall extend through tremie concrete and into sound bedrock to the specified embedment depth.

1.3   Qualifications

1.3.1   **Qualifications of Staff from Contractor or Sub-Contractor Completing Work for the Dowels into Rock:** All work shall be performed under the direction of personnel experienced with all aspects associated with the underwater installation of Dowels into Rock. Such experience shall have been obtained within the preceding five (5) years on projects of similar nature and scope to the work required for this project.

1.3.2   **Qualifications of the Quality Verification Engineer:** A resume of the work experience of the Quality Verification Engineer shall be submitted to the Contract Administrator for record purposes. The Quality Verification Engineer shall be a Professional Engineer licensed in the Province of Ontario

THURBER

having a minimum of five years of experience on projects of similar nature and scope to the work required for this project.

1.3.3 **Qualifications of the Design Engineer:** A resume of the work experience of the Design Engineer shall be submitted to the Contract Administrator for record purposes. The Design Engineer shall be a Professional Engineer licensed in the Province of Ontario having a minimum of five years of experience of projects of similar nature and scope to the work required for this project.

1.4 Responsibilities of the Contractor

1.4.1 The Contractor shall prove the allowable bond stress by tests of the Dowels into Rock on non-production Dowels into Rock.

1.4.2 The Contractor shall supply equipment, materials and skilled personnel to install production Dowels into Rock and conduct the specified acceptance tests. It shall be the responsibility of the Contractor to constantly monitor the acceptance tests, maintain specified test loads and record test measurements as specified by the Contract Administrator.

1.4.3 The Contractor is responsible for materials and workmanship. Any remedial measures, required because of defects in materials or workmanship, shall be completed by the Contractor at no cost to the Owner.

1.4.4 The Contractor shall submit 4 copies of all Working Drawings to the Contract Administrator as outlined in Section 1.6.

1.5 Definitions

1.5.1 Dowels into Rock: reinforcing steel bar and non-shrink grout.

1.5.2 Design Engineer: An Engineer who has a minimum of five (5) years experience in all aspects associated with the underwater installation of Dowels into Rock, including drilling, underwater grouting and doweling work. The Design Engineer shall be retained by the Contractor to design various components for the installation and testing for the Dowels into Rock.

1.5.3 Quality Verification Engineer: An Engineer who has a minimum of five (5) years experience in all aspects associated with the

underwater installation of Dowels into Rock, including drilling,
underwater grouting and doweling work. The Quality
Verification Engineer shall be retained by the Contractor to
ensure conformance with the contract documents and issue
certificate(s) of conformance.

1.6 Submissions and Working Drawings

1.6.1 Working Drawings shall consist of drawings, testing and
installation records, procedures and reports, and work plans.

1.6.2 The Contractor shall submit Working Drawings to the Contract
Administrator as follows:

- All Working Drawings that include drawing, testing and
installation procedures and reports, and work plans shall be
sealed and signed by the Design Engineer.

- All Working Drawings that include testing and installation results
and reports shall be signed and sealed by the Quality
Verification Engineer.

1.6.3 Upon completion of testing or installation and testing for each
component, the Contractor shall submit to the Contract
Administrator a Certificate of Conformance sealed and signed
by a Quality Verification Engineer. The Certificate shall state
that the work has been carried out in conformance with the
Working Drawings and in general conformance with the contract
documents.

1.6.4 Working Drawings consisting of testing an installation records
and reports shall be submitted four days after completion of
testing and installation. All other Working Drawings shall be
submitted two weeks prior to construction.

1.6.5 Working Drawings to be submitted include the following with
further details outlined in the remainder of this specification:

- Design calculations, specifications and shop drawings
covering all aspects of fabrication, installation and
acceptance testing of Dowels into Rock.

- Test results verifying the 28 day strength of non-shrink grout.

THURBER

- The method for constructing of the holes, maintaining the holes, and placing reinforcing steel bars, grout and other materials in the holes, including casing sizes, bit sizes and tremie grouting methods.

- The procedures to verify hole length. Records of measurements that verify the hole length.

- Records of all drilling procedures, rock conditions encountered, and installation times.

- Test procedures for Dowels into Rock.

- Drawings and design calculations for a suitable reaction system for the applied test loads.

- Records of vertical and horizontal movements of the reaction system, and elongation of the reinforcing steel bar.

- Drawings and details for reference system arrangement.

- Current calibration curves shall be provided for all gauges.

- Complete test records for all tests including plots of dowel movement versus dowel load, dowel load versus time, and dowel movement versus time.

- Remedial measures for unacceptable stressing results.

1.7 Subsurface Conditions

1.7.1 Rock and groundwater conditions are described in the Foundation Investigation Report for this Contract.

2.0 MATERIALS

The non-shrink grout shall be an approved DSM 9.10.35 non-shrink grout. The anti-washout agent shall be used with the non-shrink grout for the Dowels into Rock. The anti-washout agent shall be one of the following proprietary products:

Sikament 100 SC Anti-washout additive for underwater concrete/grouts
Sika Canada Inc.
970 Verbena Road
Mississauga, Ontario

THURBER

L5T 1T6, Canada

Mr. Greg Dolenc
Phone: 416-795-3177
Mobile: 416-573-7223

Rheomac UW 450 Liquid anti-washout admixture
Masterbuilder Technologies
1800 Clark Boulevard
Brampton, Ontario
L6T 4M7, Canada

Mr. Eliseo Conciatori
Phone: 905-792-2012
Mobile: 416-567-7665

The Contractor shall provide the following information from the manufacturer for non-shrink grout and anti-washout agent:

- Data sheets for the non-shrink grout and anti-washout agent,

- Technical information that proves that the non-shrink grout and anti-washout agent are compatible, and

- installation procedures

3.0 EQUIPMENT

3.1 General

3.1.1 All equipment for the installation of the Dowels into Rock shall be suitable for the intended purposes and capable of working on the site under the prevailing access and clearance conditions.

3.1.2 The equipment shall not cause damage to the reinforcing steel bars.

4.0 INSTALLATION

All work for the installation of Dowels into Rock shall be inspected by the Quality Verification Engineer.

4.1 Construction of Holes

THURBER

4.1.1 The sides and end of the hole shall not be disturbed. The Contractor shall submit Working Drawings to the Contract Administrator that include the method for constructing of the holes, maintaining the holes, and placing reinforcing steel bar, grout and other materials in the holes. All excavated material shall be removed from the site.

4.1.2 The hole diameters and hole length for this project are as specified on the Contract Drawings. Prior to commencing drilling operations, the Contractor shall submit Working Drawings to the Contract Administrator outlining devised procedures to verify hole length. The Contractor shall submit Working Drawings that include drilling operations records to the Contract Administrator that include the above noted records.

4.1.3 At all times, the Contractor shall keep a record of all drilling procedures, rock conditions encountered, and installation times. The Contractor shall submit Working Drawings to the Contract Administrator that include the above noted records.

4.2 Installation of Reinforcing Steel Bar

4.2.1 Reinforcing steel bar shall be installed in strict accordance with the Contract Drawings and installation procedures.

4.2.2 Centering devices shall be provided to ensure that the reinforcing steel bar is located centrally in the hole.

4.2.3 Dowels into Rock at the pier shall be installed prior to unwatering the structure excavation. Dowels shall extend through the tremie concrete for the pier footing and into sound bedrock.

4.2.4 Reinforcing steel bar shall be installed after the dowel hole has been filled with non-shrink grout.

4.3 Grout and Anti-Washout Agent

4.3.1 The non-shrink grout shall entirely fill the annular space between the reinforcing steel bar and side for the dowel hole.

4.3.2 The placement of grout for the test Dowels into Rock shall be identical to the production Dowels into Rock.

4.3.3 Anti-washout agent shall be used in accordance with the specifications of the manufacturer.

4.3.4 Non-shrink grout shall be placed into the dowel hole using tremie placement methods.

5.0 TESTING REQUIREMENTS

All work for the testing of Dowels into Rock shall be inspected by the Quality Verification Engineer.

5.1 General Testing Requirements

5.1.1 Refer to the attached Instructions to Contractor and the Contract Drawings for specific test details.

5.1.2 The Contractor shall install the number of Dowels into Rock specified in the contract documents for testing purposes. The purpose of the testing the Dowels into Rock is to prove the adequacy of the proposed anchor configuration and installation procedures under the site conditions, and to provide design parameters.

5.1.3 The equipment, labour and materials for test dowels shall be identical to Dowels into Rock at the pier. The Dowels into Rock for testing shall be 55M dowels grouted into 140 mm diameter holes filled with an approved non-shrink grout with a minimum 4,000 mm embedment into sound bedrock.

5.1.4 The Contractor shall submit Working Drawings that include proposed procedures for testing of the dowels into Rock to the Contract Administrator. Such testing shall be executed in strict accordance with the proposed procedures of the Contractor.

5.1.5 The Quality Verification Engineer shall supervise the testing of the Dowels into Rock. The Contractor will notify the Contract Administrator of the testing schedule at least 10 days prior to commencement of the testing program. Testing for Dowels into Rock shall be conducted concurrently, as scheduled by the Contract Administrator. The tests shall normally be conducted between 8:00 hrs and 20:00 hrs from Monday to Friday, unless otherwise directed by the Contract Administrator.

5.1.6 The Contractor shall supply materials and skilled personnel to conduct the tests for the Dowels into Rock. The equipment and

THURBER

materials shall be capable of stressing the Dowels into Rock to the specified loads. It shall be the responsibility of the Contractor to constantly monitor the test, maintain specified test loads and to record test measurements as specified by the Quality Verification Engineer.

5.1.7 The test site shall be restored to its pre-test condition. Reinforcing steel bars used in tests shall be cut down 25 mm below the top of the sound bedrock.

5.2 Testing Location

5.2.1 The Contractor shall remove all loose rock down to sound bedrock at the test location.

5.2.2 The test Dowels into Rock shall be constructed at locations specified by the Contract Administrator. The water depth at the location of the test shall be at least 0.5 m deep.

5.2.3 If site conditions dictate, changes to the test locations will be considered. The Contractor shall provide the Contract Administrator at least 2 days notice in writing of this operation.

5.3 Testing Equipment

5.3.1 The dowels into rock will be carried out generally in accordance with the prevailing requirements of A.S.T.M. (Designation D1143-81) superseded where applicable by the procedures specified in this document.

5.3.2 The Contractor shall submit Working Drawings for a suitable reaction system for the applied test loads to the Contract Administrator. Jacks must be secured with chains to provide adequate protection for the personnel in the event of breakage of the reinforcing steel bar or stressing system.

5.3.3 The Contractor shall submit Working Drawings for the reference system arrangement to the Contract Administrator. All reference beams shall be as follows:

- The beams shall be independently supported with the support firmly embedded in the ground.

THURBER

- The testing device shall not apply compression to the bedrock surrounding the test for the Dowels into Rock, within a circle concentric with the dowel hole and a diameter equal to 4.0 m.

- Reference beams shall be sufficiently rigid to support instrumentation such that variations in readings do not occur.

5.3.4 The Contractor shall construct suitable enclosures to provide complete protection for equipment and instruments from variations in the weather conditions and disturbances during the test program. These provisions must meet the approval of the Quality Verification Engineer and will include that the test enclosures must be weather-proof and provide a consistent temperature in order to eliminate temperature variations that could affect instrumentation.

5.4 Testing for Dowels Into Rock, and Report

5.4.1 At all times, the Contractor shall keep records of vertical and horizontal movements of the reaction system, elongation of reinforcing steel bar, and the record of test enclosure temperature. The movements shall be recorded with respect to an independent fixed reference point. The Contractor shall submit Working Drawings that include the above noted records to the Contract Administrator.

5.4.2 Dial gauges shall have at least a 76.2 mm (3.0 in.) travel. Longer gauge stems or sufficient gauge blocks shall be provided to allow for greater travel if required. Gauges shall have precision of at least 0.025 mm (0.0001 in.). The dial gauges shall be placed on smooth bearing surfaces mounted perpendicular to the direction of movement. All gauges, scales or reference points attached to the test anchor shall be mounted so as to prevent movement relative to the test anchor during the test. The Contractor shall submit Working Drawings that include details for current calibration and curves for all gauges to the Contract Administrator.

5.4.3 Jacks used for reinforcing steel bars shall have a minimum ram dimension of 152.6 mm (6.0 in.). The Contractor shall submit Working Drawings that include details for current calibration and curves for all gauges to the Contract Administrator.

THURBER

5.4.4 Requirements for Clauses 5.4.1 to 5.4.4 shall be repeated as required at different testing locations.

5.5 Testing Loading

5.5.1 The testing procedures shall safely load test the Dowels into Rock in tension at a rate of approximately 100kN per minute to the test load of 1,150 kN. The load shall be increased by an additional 50 kN beyond this level as directed by the Quality Verification Engineer.

5.5.2 Each load shall be maintained for a minimum time of 15 minutes and until the rate of displacement is not greater than 0.25 mm (0.01 inches) per hour.

5.6 Acceptance Criteria

5.6.1 The following acceptance criteria apply:

The testing of dowels shall be carried out in advance of the instalment of Dowels into Rock at the pier footing.

Tests for Dowels into Rock shall have a capacity of at least 1035 kN. The Quality Verification Engineer shall report on the acceptance of the tests for Dowels into Rock. The Quality Verification Engineer shall report on the testing of the Dowels into Rock including recommendations for increasing embedment depth, if necessary.

6.0 BASIS OF PAYMENT

Payment at the contract unit price for the above tender item shall include full compensation for all labour, equipment, and materials to do the work. No additional payment will be made for tests for Dowels into Rock which are deemed as included as part of the work for the above noted item.

Appendix F

Drawings
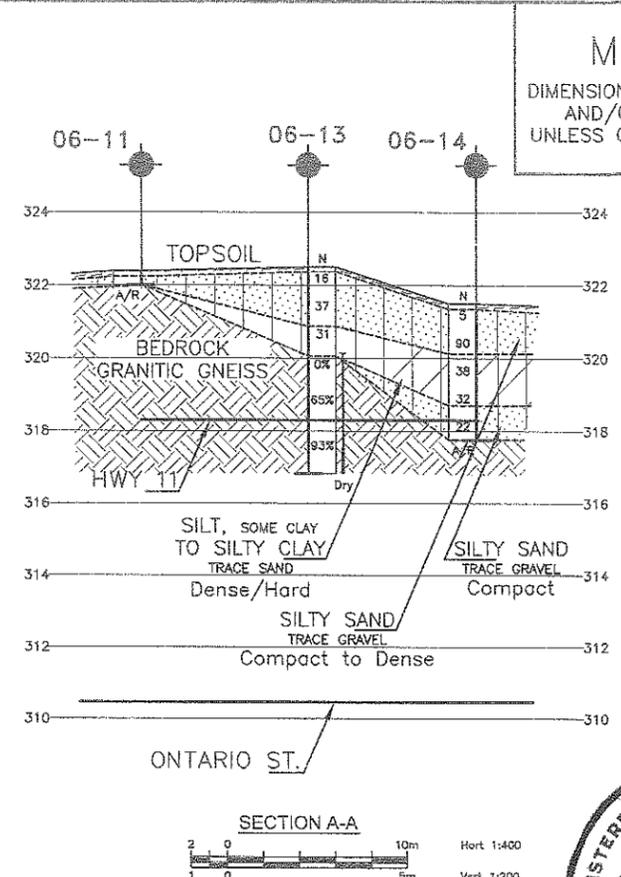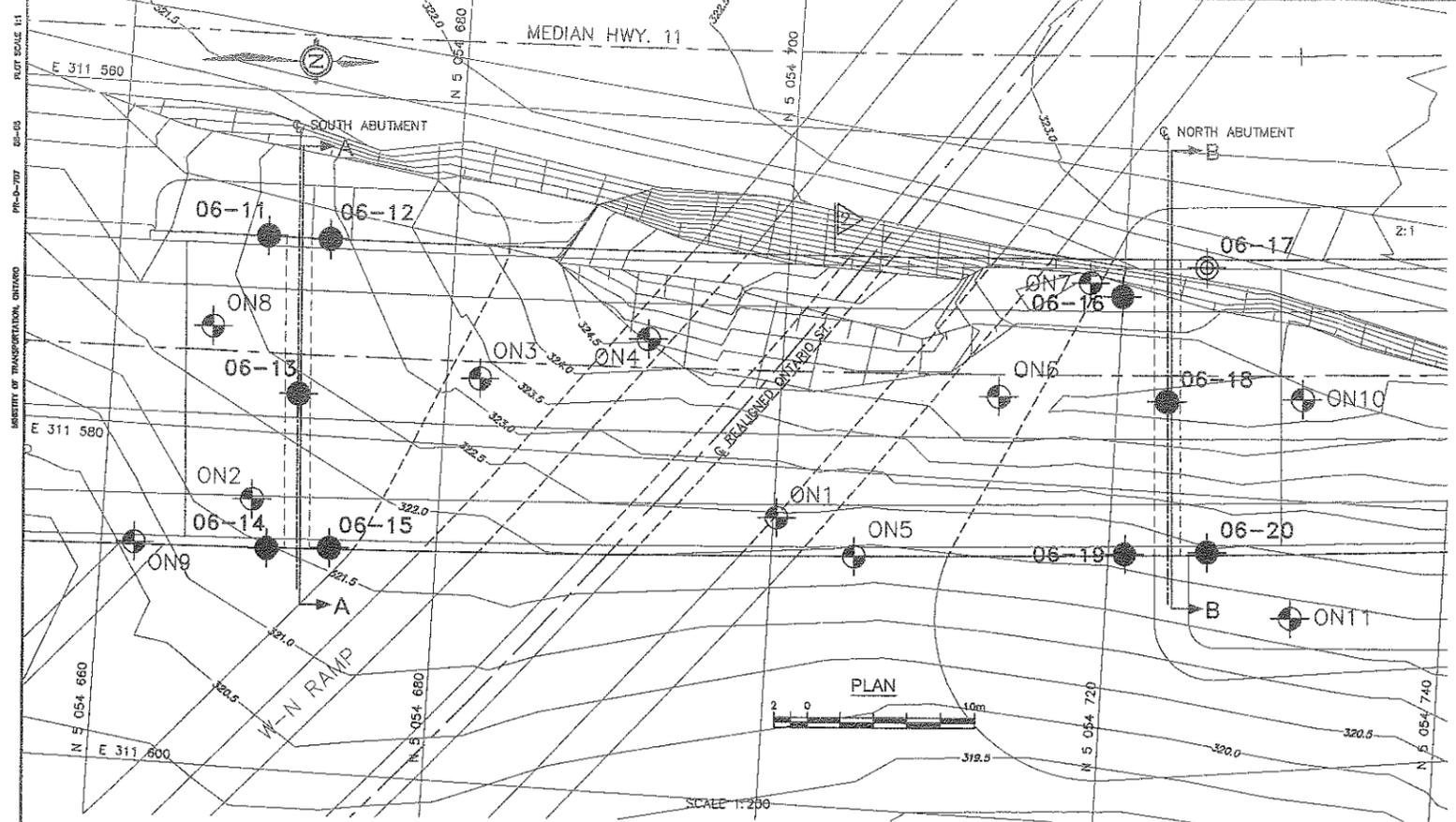
HWY 11
CONT No
WP No 473—93—00

REALIGNED ONTARIO ST.
CROSSING AT HIGHWAY 11
NBL
BOREHOLE LOCATIONS AND SOIL STRATA

SHEET

Marshall Macklin Monaghan
PROJECT MANAGERS • ENGINEERS • SURVEYORS • PLANNERS

THURBER ENGINEERING LTD.
GEOTECHNICAL • ENVIRONMENTAL • MATERIALS

KEYPLAN

PLAN

MEDIAN HWY. 11

C SOUTH ABUTMENT — A
C NORTH ABUTMENT — B

06—11
06—12
06—17
ON7
06—16
ON8
ON3 ON4
06—13
ON6
06—18
ON10
ON2
ON1
ON5
ON9
06—14
06—15
06—19
06—20
ON11

REALIGNED ONTARIO ST.
W—N RAMP

SCALE 1:200

SECTION A—A

06—11   06—13   06—14

TOPSOIL
BEDROCK
GRANITIC GNEISS
HWY 11

SILT, SOME CLAY
TO SILTY CLAY
TRACE SAND
Dense/Hard

SILTY SAND
TRACE GRAVEL
Compact

SILTY SAND
TRACE GRAVEL
Compact to Dense

ONTARIO ST.

Hort 1:400
Vert 1:200

SECTION B—B

06—16   06—18   06—19

BEDROCK OUTCROP
TOPSOIL
SILTY SAND
TRACE GRAVEL
CLAYEY SILT
TRACE SAND
HWY 11
BEDROCK
GRANODIORITE
BEDROCK
GRANITIC GNEISS
ONTARIO ST.

Hort 1:400
Vert 1:200

PROFILE OF C OF HWY 11

ON8 06—13   ON3   ON4   ON6   06—18 ON10

C SOUTH ABUTMENT   C NORTH ABUTMENT

ORIGINAL GROUND
AT C OF STRUCTURE

TOPSOIL
SILT, SOME CLAY
TO SILTY CLAY
TRACE SAND
Dense/Hard

TOPSOIL

SILTY SAND TO SAND
TRACE GRAVEL
Compact to Dense

BEDROCK
GRANITIC GNEISS

HWY 11

BEDROCK
GRANODIORITE

C REALIGNED
ONTARIO ST.
(EAST SIDE)

C REALIGNED
ONTARIO ST.
(WEST SIDE)

ONTARIO ST.
TOP OF PAVEMENT

BEDROCK
GRANITIC GNEISS

Hort 1:400
Vert 1:200

LEGEND

Symbol	Description
	Borehole by THURBER
	Borehole at base of rock cut (NOT DRILLED)
	Borehole by AGRA
N	Blows /0.3m (Std Pen Test, 475J/blow)
CONE	Blows /0.3m (60° Cone, 475J/blow)
PH	Pressure, Hydraulic
	Water Level
	Head Artesian Water
	Piezometer
90%	Rock Quality Designation (RQD)
A/R	Auger Refusal

NO	ELEVATION	NORTHING	EASTING
06—11	322.37	5 054 668.96	311 568.53
06—12	322.78	5 054 672.62	311 568.46
06—13	322.46	5 054 671.35	311 577.62
06—14	321.47	5 054 670.05	311 586.85
06—15	321.58	5 054 673.77	311 586.58
06—16	323.76	5 054 719.98	311 568.67
06—17	322.45	5 054 724.90	311 566.66
06—18	324.66	5 054 723.00	311 574.66
06—19	322.12	5 054 721.00	311 583.83
06—20	322.37	5 054 725.88	311 583.93

—NOTE—
The boundaries between soil strata have been established only at Bore Hole locations. Between boreholes the boundaries are assumed from geological evidence.
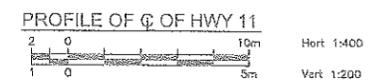
DRAWING NOT TO BE SCALED
100 mm ON ORIGINAL DRAWING

BENCHMARK

REGISTERED PROFESSIONAL ENGINEER
A. E. GORMAN
PROVINCE OF ONTARIO
NOV 22/06

LICENSED PROFESSIONAL ENGINEER
P. K. CHATTERJI
PROVINCE OF ONTARIO
NOV 22/06

DATE	BY		CODE	—	DESCRIPTION		DATE	AUG 2006
DESIGN	AEG	CHK	PKC		LOAD	—		
DRAWN	JHL	CHK	PKC	SITE	—	STRUCT	SCHEME	DWG 1

FILENAME: E:\Job Files\191142X\31 Burk's Falls Overpass\SHEETS\to023312—hwy11nbl—ontariost.dwg
PLOTDATE Nov 15, 2006 — 3:40pm

Appendix G

Site Photographs
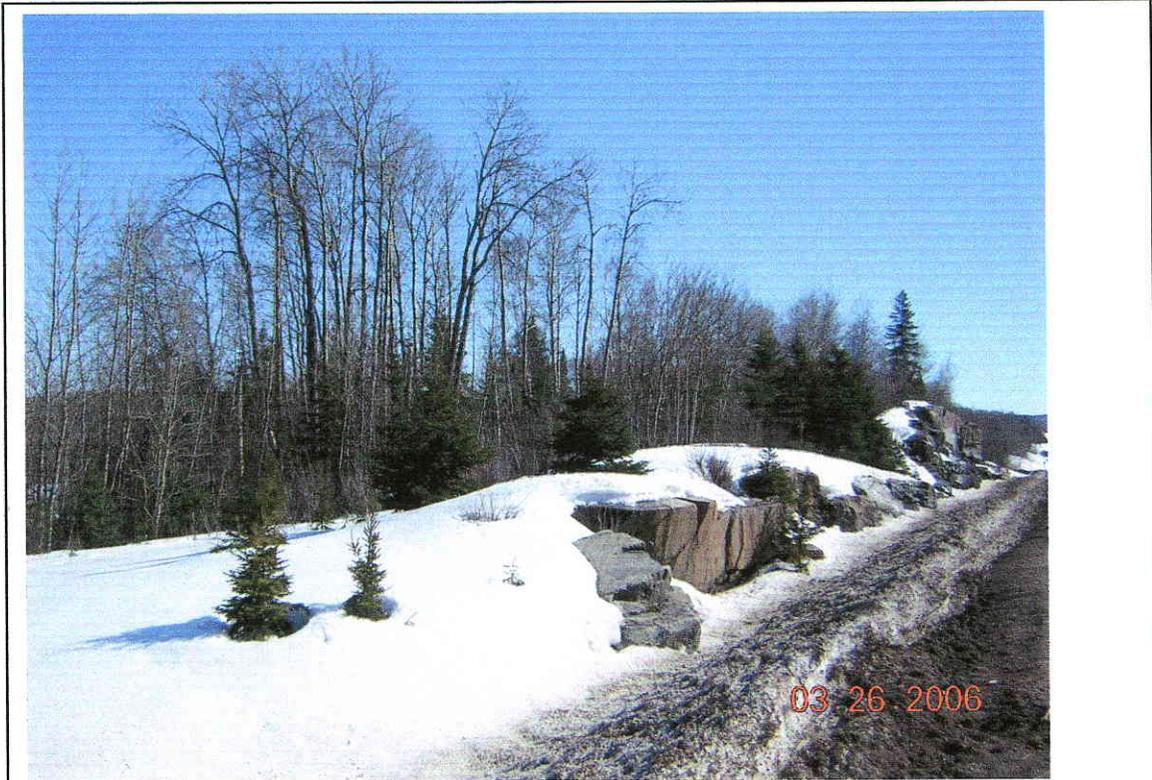
Photograph #1, NBL Site from North in March 2006

Photograph #2, NBL Site in August 2006