FOUNDATION INVESTIGATION AND DESIGN REPORT
ONTARIO STREET OVERPASS SBL
HIGHWAY 11 FOUR-LANING AT BURK'S FALLS
G.W.P. 473-93-00, SITE: 44-398S

Geocres Number: 31E-263

Report to

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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 southbound lanes (SBL) 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 west of the existing highway, encroaching onto the SB 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 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 10 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. Photograph #1 shows the general site area from the existing highway, looking south. Photograph #2 shows the ground conditions at the site, in particular the shallow soil cover over the bedrock.

3 SITE INVESTIGATION AND FIELD TESTING

The site investigation and field testing for this project were carried out in the period July 25and 26, 2006. Eight boreholes numbered 06-01 to 06-03 and 06-06 to 06-10 were drilled at the south and north abutments of the single-span structure to depths ranging from 0.1 m to 17.7 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 two additional boreholes numbered 06-04 and 06-05 are also shown on the drawing, however they were not drilled since they are 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 06-02, 06-03, 06-06, 06-08, 06-09 and 06-10 were advanced by hand excavation and encountered inferred bedrock at very shallow depths ranging from 0.1 m to 0.7 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 north abutment, Borehole 06-07 was advanced 3.0 m into bedrock and at the south abutment, Borehole 06-01 was advanced 17.4 m into bedrock by NQ size diamond coring techniques.

Groundwater conditions in the open boreholes were observed throughout the drilling operations. At the north 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 piezometer 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.

Table 3.1 – Borehole Completion Details

Location	Details	
	Tip Depth/ Elevation (m)	**Completion Details**
06-01	None Installed	Bentonite grout to ground surface.
06-02	None Installed	Backfilled with cuttings to ground surface (<3.0 m deep).
06-03	None Installed	Backfilled with cuttings to ground surface (<3.0 m deep).
06-07 North Abutment	3.7/325.4	Piezometer with 1.5 m slotted screen installed with sand filter to 1.8 m, bentonite seal from 1.8 m to 1.5 m, grout from 1.5 m to 0.6 m and bentonite seal from 0.6 m to ground surface.
06-08	None Installed	Backfilled with cuttings to ground surface (<3.0 m deep).
06-09	None Installed	Backfilled with cuttings to ground surface (<3.0 m deep).
06-10	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 of the rock cores retrieved from the boreholes were subjected to point load tests and the results of these tests are shown on the Record of Borehole sheets in Appendix A and 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 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 0.1 m to 0.7 m (1.5 m recorded in the AGRA investigation) of overburden soils overlying Pre-Cambrian bedrock. The overburden soils generally consist of topsoil and sands with occasional gravel, cobbles and boulders. Some asphalt and fill were also encountered during the AGRA investigation in boreholes located on the lane and shoulder of existing Highway 11 near the base of the rock cut.

5.1 Topsoil

Approximately 125 mm to 300 mm of topsoil was encountered across the site, extending to elevations ranging from 332.3 m to 326.7 m.

5.2 Silty Sand to Sand

Underlying the topsoil, a deposit of sand exists across the site. This deposit generally consists of silty sand, but varies to sand with some gravel, cobbles and boulders and trace silt. The sand deposit was found overlying bedrock, extending to depths ranging from 0.3 m to 0.7 m or to elevations from 332.3 to 328.4 m.

A Standard penetration test (SPT) in an AGRA borehole in this deposit gave an 'N' value of 22 blows per 0.3 m penetration indicating a compact relative density.

The moisture content of a sample from this material was approximately 18%.

5.3 Asphalt and Fill

Approximately 100 mm of asphalt was encountered in an AGRA borehole (OS 2) located on the lane of existing Highway 11. Underlying the asphalt and at another AGRA borehole located on the highway shoulder, sand fill containing some gravel was encountered extending to depths of 0.2 m to 0.8 m or to elevations of 321.9 m to 321.0 m.

5.4 Bedrock

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

TABLE 5.1 – Depth to Bedrock at Foundation Elements

Location	BH Number	Depth to Bedrock (m)	Top of Bedrock Elevation (m)
South Abutment	06-01	0.3	332.3
South Abutment	06-02	0.2	332.3
South Abutment	06-03	0.2	331.5

South Abutment	06-06	0.7	328.6
North Abutment	06-07	0.7	328.4
North Abutment	06-08	0.5	327.9
North Abutment	06-09	0.2	326.8
North Abutment	06-10	0.1	326.7

The bedrock is described as faintly weathered to fresh. Its colour is pink, white and black.

Core recovery in the bedrock was generally between 83% and 100%. The RQD values generally ranged from 60% to 100% 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.

The unconfined compressive strength of most of the rock cores is estimated to range between 64 and 175 MPa indicating a strong to 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 the north abutment 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 06-07**	
	Depth (m)	**Elev. (m)**
July 26, 2006	1.3	327.8
August 20, 2006	3.6	325.5

Based on these observations, the local groundwater level exists at an approximate elevation of 325.5 m. The groundwater observations at this site are short term and the level is expected to fluctuate seasonally and after severe weather events.

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.

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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 SBL 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 SBL at approximately Sta. 21+148.

At the south abutment, the finished grade of Highway 11 will be at Elevation 321.3 and the existing ground surface lies at Elevation 331.7 to 332.5, with an average Elevation 332.2. These elevations result in Highway 11 lying in a cut approximately 10.9 m deep.

At the north abutment, the finished grade of Highway 11 will be at Elevation 322.6 and the existing ground surface lies at Elevation 326.8 to 329.3, average Elevation 328.4. These elevations result in Highway 11 lying in a cut approximately 5.8 m deep.

The grade of Ontario Street will lie approximately at Elevation 312, resulting in a further cut of approximately 10 to 11 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 0.2 to 0.3 m of topsoil and sand containing cobbles and boulders overlying bedrock. The east end of the south abutment foundation encroaches on the SB shoulder of the existing highway. No drilling was carried out at that location, but site inspection makes it apparent that the stratigraphy consists of granular fill, probably overlying rock shatter and intact bedrock.

At the north abutment, the stratigraphy consists of 0.1 to 0.7 m of topsoil and sand containing cobbles and boulders overlying bedrock.

An apparent groundwater level exists at approximate Elevation 325.5 at the north abutment. The groundwater level was not established at the south abutment, but will lie below the surface of the bedrock.

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 15 to 20 m below the surface of the bedrock (8 m below the present highway surface at the east edge of the south 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 or an uneven bedrock surface, mass concrete fill may be used to reinstate the bearing surface to the design founding elevation.

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. A typical NSSP governing the placement of mass concrete 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. The edge of the foundation closest to the edge of the rock face must lie behind a line projected up from the toe of the excavation at an inclination of 1H:4V and, for this site, not closer than 1.5 m..

Suggested wording regarding inspection of the rock excavation to be included in the contract documents is given in Appendix E.

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 11 to 15 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.

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 in the highway subgrade. A "neat" trench is required, meaning that undisturbed bedrock should be exposed in the sides and base of the trench and that all shatter and partially dislodge rock fragments must be removed. The piles can then be placed in the specified locations and the trench backfilled with 30 MPa concrete.

Excavation of a neat trench in bedrock will require carefully consideration of the blast pattern and charges and will probably required pre-splitting. The removal of all shatter and displaced rock from the sides and base of the trench is expected to be an arduous, time consuming procedure that will be labour intensive. Drilling of sockets, on the other hand is highly mechanized and rock cuttings can be cleaned from the bottom of the socket either by hand or by other means, such as by a vacuum truck.

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 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:

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

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

where z = depth of embedment of pile in metres
D = pile width in metres
n_h = coefficient of horizontal subgrade reaction (Table 8.2)
γ = unit weight (Table 8.2)
K_p = passive earth pressure coefficient (Table 8.2)

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.

Table 8.2 – 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, $K_s = k_s \text{ x } L \text{ x } D$ (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, $P_{ult} = p_{ult} \text{ x } L \text{ x } D$. 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.

8.2.4 Pile Installation

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

8.3 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.

8.3.1 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. 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.

8.3.2 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).

8.3.3 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.

    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.

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.

8.4 Recommended Foundation

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

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.

8.5 Frost Cover

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

Frost cover need not be provided at this site.

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 Elevation 325.5. This level is below 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.

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

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.

Backfill behind the abutment will settle after construction has been completed. It is recommended that settlement of 0.5% of the height of backfill be assumed and be accommodated in the bridge design.

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 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.

In conventional design, the use of a material with a high friction angle and low active pressure coefficient (e.g. Granular A, Granular B Type II) is considered preferable as it results in lower earth pressures acting on the wall. In the case of integral or semi-integral abutments, material with a lower passive pressure coefficient (e.g. Granular B Type I) is preferred as it results in lower forces acting on the ballast wall as the wall moves toward the soil mass. However, the use of Granular "B" Type I may be restricted if the approach embankment consists of rock fill.

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.

Table 14.1 – Earth Pressure Coefficients

Wall Condition	**Earth Pressure Coefficient (K)**					
	OPSS Granular A or OPSS Granular B Type II $\phi = 35^\circ$; $\gamma = 22.8\ \mathrm{kN/m^3}$		OPSS Granular B Type I $\phi = 32^\circ$; $\gamma = 21.2\ \mathrm{kN/m^3}$		Rock Fill $\phi = 42^\circ$; $\gamma = 19.0\ \mathrm{kN/m^3}$	
	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 (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.

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
- Steep rock excavation faces may be prone to toppling, sliding wedge or other modes of instability. The Contract Administrator must engage a recognized rock slope specialist to advise on any remedial measures that may be necessary.

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.

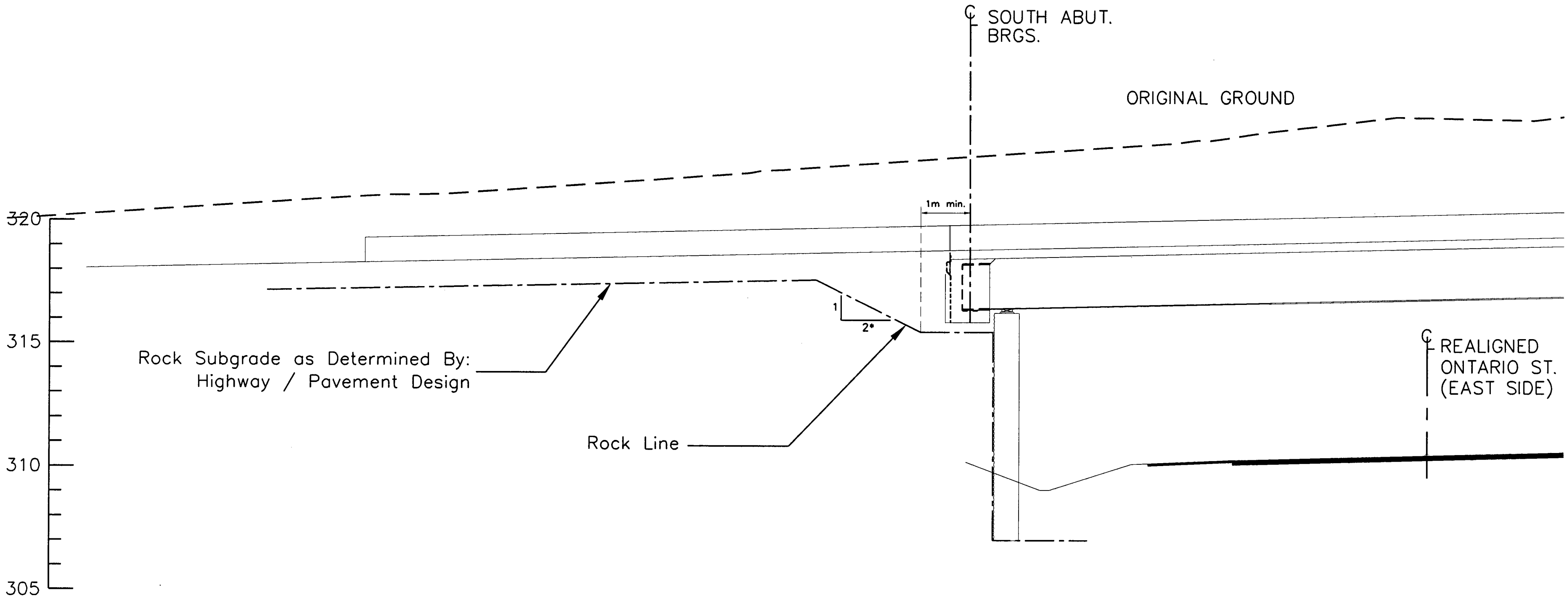
REGISTERED PROFESSIONAL ENGINEER
A. E. GORMAN
NOV 22/06
PROVINCE OF ONTARIO

Alastair E. Gorman, P.Eng.,
Senior Foundations Engineer

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

P. K. Chatterji, P.Eng.,
Review Principal

Figure1.DWG THIS DRAWING IS PART OF A THURBER "REPORT" AND ITS USE IS SUBJECT TO THURBER'S STATEMENT OF GENERAL CONDITIONS November 20, 2006



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

Marshall Macklin Monaghan

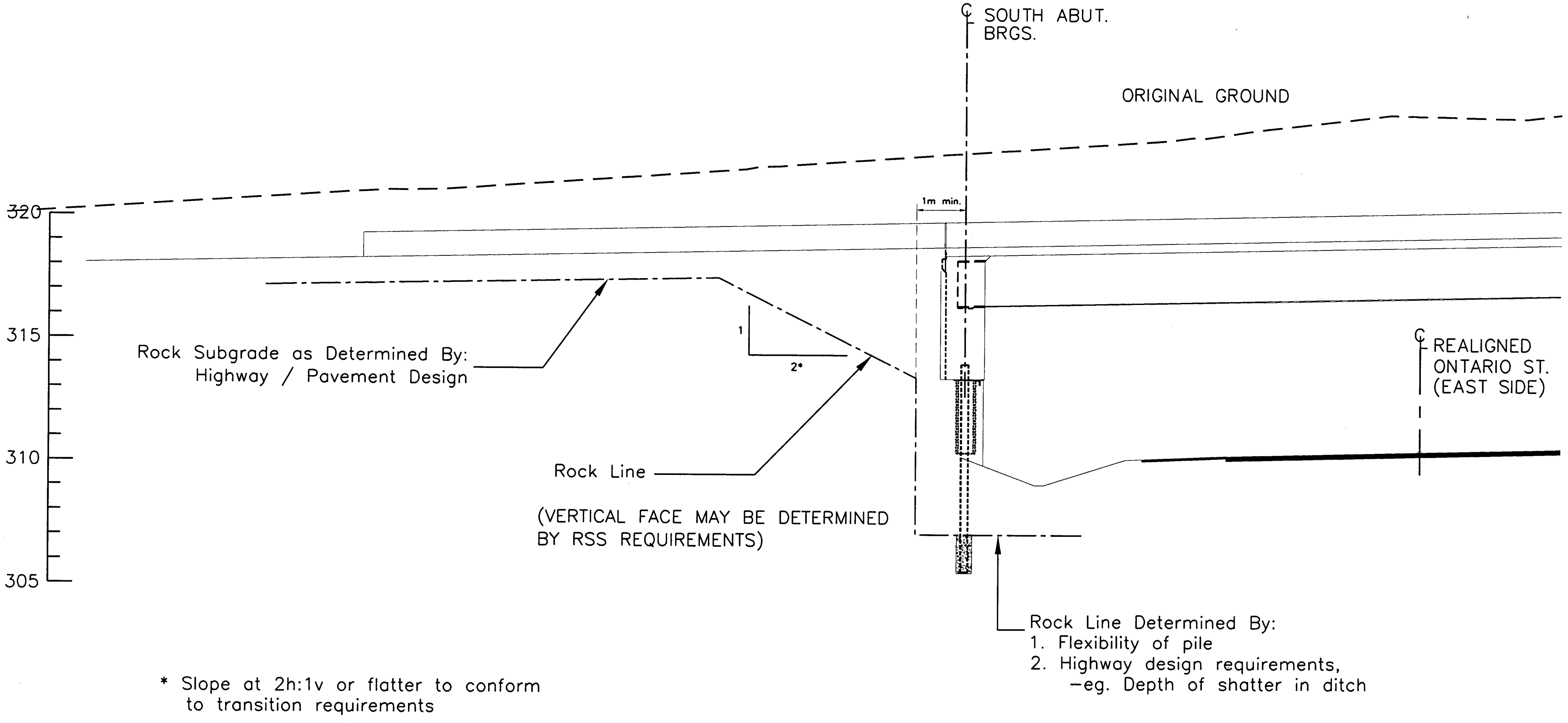
ONTARIO STREET INTERCHANGE
HIGHWAY 11
Burk's Falls, Ontario

19-1423-31

Burk's Falls, ON

THURBER ENGINEERING LTD.
GEOTECHNICAL ▪ ENVIRONMENTAL ▪ MATERIALS

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



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

Marshall Macklin Monaghan

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AEG	JHL	PKC
DATE:	SCALE:	DRAWING No.
AUGUST 2006	NTS	FIGURE2

Appendix A

Record of Borehole Sheets

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

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

▽ 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 06-01

1 OF 2

METRIC

W.P. 473-93-00
LOCATION N 5 054 699.25 E 311 526.61 Ontario Street Overpass (SBL)
ORIGINATED BY SLL
HWY 11
BOREHOLE TYPE NQ Core Barrel
COMPILED BY JHL
DATUM Geodetic
DATE 25.07.06 - 25.07.06
CHECKED BY AEG

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	UNIT WEIGHT γ kN/m³ FI	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
332.5								
0.0	**TOPSOIL**: (150 mm)							
332.3 0.1	Silty **SAND** Brown Moist							
0.3	**BEDROCK** Pink, white and black, crystalline, faintly weathered to fresh, thinly banded, occasional mica veins, strong to very strong, **GRANITIC GNEISS**	1	RUN			332	0 1 4	RUN 1# TCR=100%, SCR=98%, RQD=80%, UCS=73MPa
	Subvertical joint from 0.64 to 0.74 m Subvertical joint from 1.42 to 1.50 m Subvertical joint from 1.68 to 1.83 m Slightly weathered, rough joint surfaces	2	RUN			331	4 2 7 1	RUN 2# TCR=98%, SCR=90%, RQD=95%, UCS=114MPa
		3	RUN			330 329	1 2 2 4 3	RUN 3# TCR=100%, SCR=90%, RQD=60%, UCS=138MPa
	Subvertical joint from 4.32 to 4.42 m Subvertical joint from 4.98 to 5.23 m Slightly weathered, rough joint surfaces	4	RUN			328	1 2 1 2 1	RUN 4# TCR=95%, SCR=93%, RQD=83%, UCS=68MPa
	Subvertical joints from 5.79 to 6.10 m, and from 6.60 to 6.74 m Slightly weathered, rough joint surfaces	5	RUN			327 326	0 1 1 1 2	RUN 5# TCR=98%, SCR=98%, RQD=94%, UCS=64MPa
		6	RUN			325	5 0 2 1 0	RUN 6# TCR=98%, SCR=98%, RQD=95%, UCS=109MPa
		7	RUN			324 323	0 2 0 1 1	RUN 7# TCR=100%, SCR=100%, RQD=98%, UCS=133MPa

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L

WATER CONTENT (%): 20 40 60

Continued Next Page

$+^3, \times^3$: Numbers refer to Sensitivity

$15 \overset{20}{\underset{10}{\oplus}} 5$ (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 06-01

2 OF 2

METRIC

W.P. 473-93-00	LOCATION N 5 054 699.25 E 311 526.61 Ontario Street Overpass (SBL)	ORIGINATED BY SLL
HWY 11	BOREHOLE TYPE NQ Core Barrel	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 20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100	PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES						
		8	RUN			322			1 3 0 0	RUN 8# TCR=100%, SCR=100%, RQD=98%, UCS=139MPa
		9	RUN			321			0	RUN 9# TCR=83%, SCR=83%, RQD=83%, UCS=158MPa
		10	RUN			320			2 0 2 0 0	RUN 10# TCR=95%, SCR=95%, RQD=80%, UCS=151MPa
		11	RUN			319 318			5 2 0 1 3	RUN 11# TCR=100%, SCR=100%, RQD=100%, UCS=94MPa
	Subvertical joint from 15.04 to 15.14 m Subvertical joint from 15.63 to 15.74 m Slightly weathered, rough joint surface	12	RUN			317			1 1 2 2 4	RUN 12# TCR=100%, SCR=100%, RQD=100%, UCS=175MPa
314.9		13	RUN			316 315			3 4 4 0 0	RUN 13# TCR=100%, SCR=98%, RQD=86%, UCS=143MPa
17.7	END OF BOREHOLE AT 17.68 m. BOREHOLE GROUTED WITH BENTONITE TO SURFACE.									

ONTMT4S 2331.GPJ 20/11/06

$+^3, \times^3$: Numbers refer to Sensitivity

$15 \overset{20}{\underset{10}{\oplus}} 5$ (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No 06-02

1 OF 1

METRIC

W.P.	473-93-00	LOCATION	N 5 054 703.59 E 311 526.55 Ontario Street Overpass (SBL)	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 20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100	PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m^3	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
332.5											
0.0	**TOPSOIL**: (150 mm)										
0.2	END OF BOREHOLE AT 0.15 m. REFUSAL AT 0.15 m ON PROBABLE BEDROCK OR BOULDERS. BORCHOLE BACKFILLED WITH CUTTINGS TO SURFACE.						332				

ONTMT4S 2331.GPJ 20/11/06

$+^3, \times^3$: Numbers refer to Sensitivity

$15 \overset{20}{\underset{10}{\oplus}} 5$ (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No 06-03

1 OF 1

METRIC

W.P. 473-93-00
LOCATION N 5 054 701.77 E 311 531.05 Ontario Street Overpass (SBL)
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 20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100	PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
331.7											
0.0	**TOPSOIL**: (200 mm)										
0.2	END OF BOREHOLE AT 0.20 m. REFUSAL AT 0.20 m ON PROBABLE BEDROCK OR BOULDERS. BOREHOLE BACKFILLED WITH CUTTINGS TO SURFACE.						331				

ONTMT4S 2331.GPJ 20/11/06

$+^3, \times^3$: Numbers refer to Sensitivity

$15 \oplus 5$ (20 above, 10 below) (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No 06-06

1 OF 1

METRIC

W.P. 473-93-00
LOCATION N 5 054 750.62 E 311 521.62 Ontario Street Overpass (SBL)
ORIGINATED BY SLL
HWY 11
BOREHOLE TYPE Hand Excavation
COMPILED BY JHL
DATUM Geodetic
DATE 26.07.06 - 26.07.06
CHECKED BY AEG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100	PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
329.3											
0.0 329.0	**TOPSOIL**: (300 mm)										
0.3 328.6	Silty **SAND** Brown Moist						329				
0.7	END OF BOREHOLE AT 0.69 m. REFUSAL AT 0.69 m ON PROBABLE BEDROCK OR BOULDERS. BOREHOLE BACKFILLED WITH CUTTINGS TO SURFACE.										

$+^3, \times^3$: Numbers refer to Sensitivity

$15 \overset{20}{\underset{10}{\oplus}} 5$ (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No 06-07

1 OF 1

METRIC

W.P. 473-93-00	LOCATION N 5 054 755.26 E 311 520.99 Ontario Street Overpass (SBL)	ORIGINATED BY SLL
HWY 11	BOREHOLE TYPE Hollow Stem Augers / NQ Core Barrel	COMPILED BY JHL
DATUM Geodetic	DATE 26.07.06 - 26.07.06	CHECKED BY AEG

ELEV DEPTH	SOIL PROFILE DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	SAMPLES TYPE	SAMPLES "N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH kPa ○ 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	UNIT WEIGHT γ kN/m3	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
329.1											
0.0	**TOPSOIL**: (125 mm)		1	SS	56/.200		329		○	FI	
0.1 328.7	Silty **SAND**, trace roots Brown Moist		1	RUN						0	
0.4 328.4	**SAND**, some gravel, cobbles and boulders Brown		2	RUN						2	RUN 2# TCR=98%, SCR=80%, RQD=65%, UCS=115MPa
0.7	**BEDROCK** Pink, white and black, crystalline, faintly weathered to fresh, very strong, **GRANITIC GNEISS** Subvertical joint from 0.86 to 1.17 m Slightly weathered, rough joint surface									10 2 0 3	
	Subvertical joint from 3.23 to 3.33 m Slightly weathered, smooth joint surface		3	RUN		▼				1 0 0 4 0	RUN 3# TCR=98%, SCR=94%, RQD=95%, UCS=157MPa
325.4 3.7	END OF BOREHOLE AT 3.66 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	1.28	327.82
08/20/06	3.58	325.52

+3, ×3: Numbers refer to Sensitivity

20 15 ⊕ 5 10 (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No 06-08

1 OF 1

METRIC

W.P. 473-93-00	LOCATION N 5 054 754.17 E 311 529.77 Ontario Street Overpass (SBL)		ORIGINATED BY SLL
HWY 11	BOREHOLE TYPE Hand Excavation		COMPILED BY JHL
DATUM Geodetic	DATE 26.07.06 - 26.07.06		CHECKED BY AEG

SOIL PROFILE			SAMPLES					DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100	PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE				
328.3 0.0	**TOPSOIL**: (225 mm)										
327.9 0.2	Silty **SAND** Brown Moist						328				
0.5	END OF BOREHOLE AT 0.46 m. REFUSAL AT 0.46 m ON PROBABLE BEDROCK OR BOULDERS. BOREHOLE BACKFILLED WITH CUTTINGS TO SURFACE.										

+ 3, × 3: Numbers refer to Sensitivity

20
15 ⊕ 5
10
(%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No 06-09

1 OF 1

METRIC

W.P. 473-93-00
LOCATION N 5 054 752.24 E 311 539.51 Ontario Street Overpass (SBL)
ORIGINATED BY SLL
HWY 11
BOREHOLE TYPE Hand Excavation
COMPILED BY JHL
DATUM Geodetic
DATE 26.07.06 - 26.07.06
CHECKED BY AEG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100	PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
327.0											
0.0	**TOPSOIL** (150 mm)						327				
0.2	END OF BOREHOLE AT 0.15 m. REFUSAL AT 0.15 m ON PROBABLE BEDROCK OR BOULDERS. BOREHOLE BACKFILLED WITH CUTTINGS TO SURFACE.										

+ 3, × 3: Numbers refer to Sensitivity

20 / 15 ⊕ 5 / 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 06-10

1 OF 1

METRIC

W.P. 473-93-00
LOCATION N 5 054 757.02 E 311 539.19 Ontario Street Overpass (SBL)
ORIGINATED BY SLL
HWY 11
BOREHOLE TYPE Hand Excavation
COMPILED BY JHL
DATUM Geodetic
DATE 26.07.06 - 26.07.06
CHECKED BY AEG

SOIL PROFILE			SAMPLES					DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100	PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE				
326.8											
0.0	**TOPSOIL** (125 mm)										
0.1	END OF BOREHOLE AT 0.13 m. REFUSAL AT 0.13 m ON PROBABLE BEDROCK OR BOULDERS. BOREHOLE BACKFILLED WITH CUTTINGS TO SURFACE.						326				

$+^3, \times^3$: Numbers refer to Sensitivity

$15 \oplus 5$ (20 above, 10 below) (%) STRAIN AT FAILURE

Appendix B

Laboratory Test Results

TABLE B1 - Point Load Test Results
Ontario Street Overpass SBL

Depth Feet	Depth Inches	m	Is50	UCS (MPa)
06-1				
1	4	0.41	2.52	60.50
2	7	0.79	3.57	85.56
3	7	1.09	6.00	143.98
5	3	1.60	2.22	53.21
6	6	1.98	4.52	108.50
7	8	2.34	6.26	150.24
9	10	3.00	3.91	93.91
10	10	3.30	6.61	158.59
11	11	3.63	5.65	135.65
12	9	3.89	6.87	164.86
14	8	4.47	2.61	62.59
16	3	4.95	3.04	73.03
19	4	5.89	1.74	41.74
20	4	6.20	1.83	43.82
21	4	6.50	4.43	106.42
22	6	6.86	2.17	52.18
23	3	7.09	5.13	123.12
24	5	7.44	3.48	83.50
25	3	7.70	5.96	142.94
26	3	8.00	6.04	145.03
27	4	8.33	3.39	81.38
28	4	8.64	6.43	154.42
30	4	9.25	6.30	151.30
31	4	9.55	5.52	132.50
32	6	9.91	6.09	146.06
33	7	10.24	5.65	135.65
34	9	10.59	6.17	148.15
35	8	10.87	5.52	132.50
36	7	11.15	5.83	139.80
37	8	11.48	6.56	157.54
39	0	11.89	3.04	73.03
40	0	12.19	6.65	159.62
41	2	12.55	8.04	193.03
42	0	12.80	7.48	179.45
44	3	13.49	4.13	99.12
44	10	13.67	3.17	76.18
46	0	14.02	5.43	130.42
47	0	14.33	2.96	70.94
48	6	14.78	7.74	185.71
50	8	15.44	8.91	213.89
51	11	15.82	5.57	133.56
52	10	16.10	7.00	167.98
54	1	16.48	8.26	198.24
55	7	16.94	2.96	70.94
56	4	17.17	5.43	130.42
57	1	17.40	7.30	175.30
58	2	17.73	5.78	138.77

Total Rock Core

Average	Minimum	Maximum	
124	42	214	MPa

Run #	Average
1	73.03
2	113.98
3	138.25
4	67.81
5	63.99
6	109.35
7	133.13
8	139.03
9	157.54
10	151.28
11	94.16
12	175.28
13	142.73

Depth				UCS
Feet	Inches	m	Is50	(MPa)
06-7				
2	4	0.71	4.35	104.33
4	0	1.22	4.35	104.33
5	0	1.52	4.78	114.77
6	0	1.83	5.65	135.65
7	6	2.29	7.83	187.80
8	6	2.59	6.09	146.06
9	6	2.90	7.39	177.36
10	6	3.20	5.65	135.65
11	6	3.51	5.65	135.65

Total Rock Core

Average	Minimum	Maximum	
138	104	188	MPa
Run #	Average		
1	114.77		
2	156.50		

Appendix C

Factual Information from the AGRA Report

AGRA

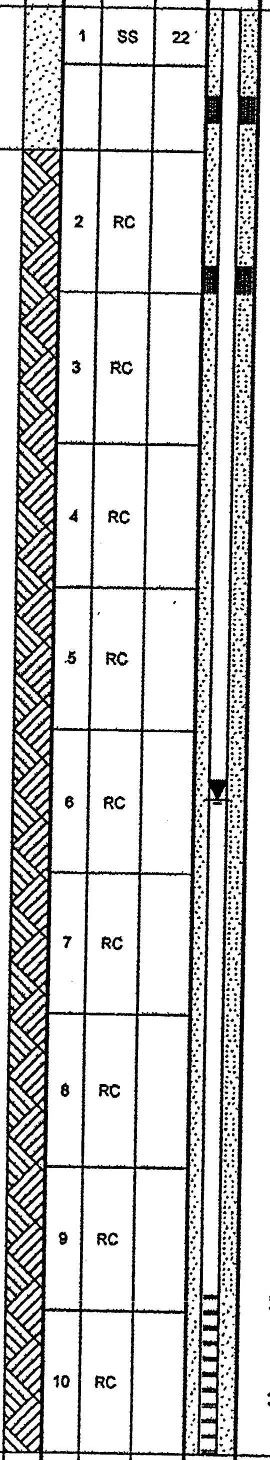
Foundation Design

RECORD OF BOREHOLE No OS1

1 OF 1 **METRIC**

W.P. 486-93-01
LOCATION Site No. 44-398S N5054757 E311524
ORIGINATED BY AD
DIST 52 HWY 11
BOREHOLE TYPE Hollow Stem
COMPILED BY AD
DATUM Geodetic
DATE 5 May 1999
CHECKED BY EYC

SOIL PROFILE		SAMPLES			ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES		
328.6						
0.0	brown brown,**SAND** with cobbles and boulders trace Silt, compact, damp	1	SS	22	328	STATION 21+180 8 LT SBL C/L
327.1 1.5	**GNEISS BEDROCK** massive, occasional Mica inclusions closely to moderately closely jointed	2	RC		327 326	RC2: REC=100% RQD=100%
		3	RC		325	RC3: REC=100% RQD=100%
		4	RC		324 323	RC4: REC=100% RQD=97%
		5	RC		322	RC5: REC=100% RQD=95%
		6	RC		321 320	RC6: REC=100% RQD=88%
		7	RC		319	RC7: REC=100% RQD=90%
		8	RC		318 317	RC8: REC=100% RQD=86%
		9	RC		316	RC9: REC=100% RQD=99%
		10	RC		315 314	RC10: REC=100% RQD=90%
313.4 15.2	END OF BOREHOLE Water Level in PIEZOMETER July 9/99: 8.3m					



DYNAMIC CONE PENETRATION RESISTANCE PLOT
20 40 60 80 100

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

PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L

WATER CONTENT (%)
10 20 30

UNIT WEIGHT γ kN/m³

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

AGRA

RECORD OF BOREHOLE No OS2

1 OF 1 **METRIC**

W.P. 486-93-01
LOCATION Site No. 44-398S N 5054710 E 311546
ORIGINATED BY AD
DIST 52 HWY 11
BOREHOLE TYPE Hollow Stem
COMPILED BY AD
DATUM Geodetic
DATE 5 May 1999
CHECKED BY EYC

SOIL PROFILE			SAMPLES			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 20 40 60 80 100	PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
322.7 0.0	0.1m ASPHALTIC CONCRETE brown **SAND FILL** with Gravel, damp compact	▩	1	SS	59				○		STATION 21+130 10RT SBL C/L
321.9 0.8	Highly fractured ------------ **GNEISS BEDROCK** massive, occasional Mica inclusions closely to moderately closely jointed	▨	2	RC			322				RC2: REC=40% RQD=13%
		▨	3	RC			321 320				RC3: REC=90% RQD=50%
		▨	4	RC			319				RC4: REC=100% RQD=90%
		▨	5	RC			318 317				RC5: REC=100% RQD=83%
316.8 5.9	END OF BOREHOLE No water in borehole before coring										

+3, ×3: Numbers refer to Sensitivity

○3% STRAIN AT FAILURE

RECORD OF BOREHOLE No OS3

1 OF 1 **METRIC**

W.P. 486-93-01
LOCATION Site No. 44-398S N 5054710 E 311535
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 20 40 60 80 100 SHEAR STRENGTH kPa O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100	PLASTIC LIMIT W_p NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE				
330.7											
0.0 330.4	brown SAND & GRAVEL trace Organics, Silt damp		1	GS							
0.3 330.1	fractured GNEISS BEDROCK										
0.6	END OF TESTPIT ON BEDROCK Water Level on Completion: dry						330				STATION 21+131 2 LT SBL C/L

$+^{3}, \times^{3}$: Numbers refer to Sensitivity

$O^{3\%}$ STRAIN AT FAILURE

RECORD OF BOREHOLE No OS4

1 OF 1 **METRIC**

W.P. 486-93-01
LOCATION Site No. 44-398S N 5054743 E 311535
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 20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100	PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE				
328.2 0.0	0.4m **TOPSOIL** with cobbles and boulders						328				
327.7 0.6	0.15m **SAND & GRAVEL** damp										
	END OF TESTPIT ON BEDROCK Water Level on Completion: dry										STATION 21+165 1 RT SBL C/L

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

RECORD OF BOREHOLE No OS5

1 OF 1 **METRIC**

W.P. 486-93-01	LOCATION Site No. 44-398S N 5054739 E 311539	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 20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100	PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE				
327.5											
327.3 0.2	brown, damp SAND & GRAVEL		1	GS							
	END OF TESTPIT ON BEDROCK Water Level on Completion: dry						327				STATION 21+160 5RT SBL C/L

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

AGRA

RECORD OF BOREHOLE No OS6

1 OF 1

METRIC

W.P. 486-93-01
LOCATION Site No. 44-398S N 5054762 E 311532
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 20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100	PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
327.6											
0.0	0.1m TOPSOIL										
327.1 0.5	red-brown **SILTY SAND** with cobbles and boulders, damp		1	GS			327				STATION 21+184 SBL C/L
326.7 0.9	brown **SAND & GRAVEL** damp										

END OF TESTPIT
ON BEDROCK

Water Level on Completion:
dry

$+^{3}, \times^{3}$: Numbers refer to Sensitivity

$\circ^{3\%}$ STRAIN AT FAILURE

AGRA

Foundation Design

RECORD OF BOREHOLE No OS7

1 OF 1 **METRIC**

W.P. 486-93-01

LOCATION Site No. 44-398S N 5054688 E 311540

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 20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100	PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
321.2											
321.0 0.2	brown **Sand with Gravel FILL** some Organics, damp	▩	1	GS			321		○		
	END OF TESTPIT ON BEDROCK Water Level on Completion: dry										STATION 21+112 2 RT SBL C/L

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

Appendix D

Foundation Comparison

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**

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.

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.

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.

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

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.

3 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 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.

- 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

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

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 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.

- 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.

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.

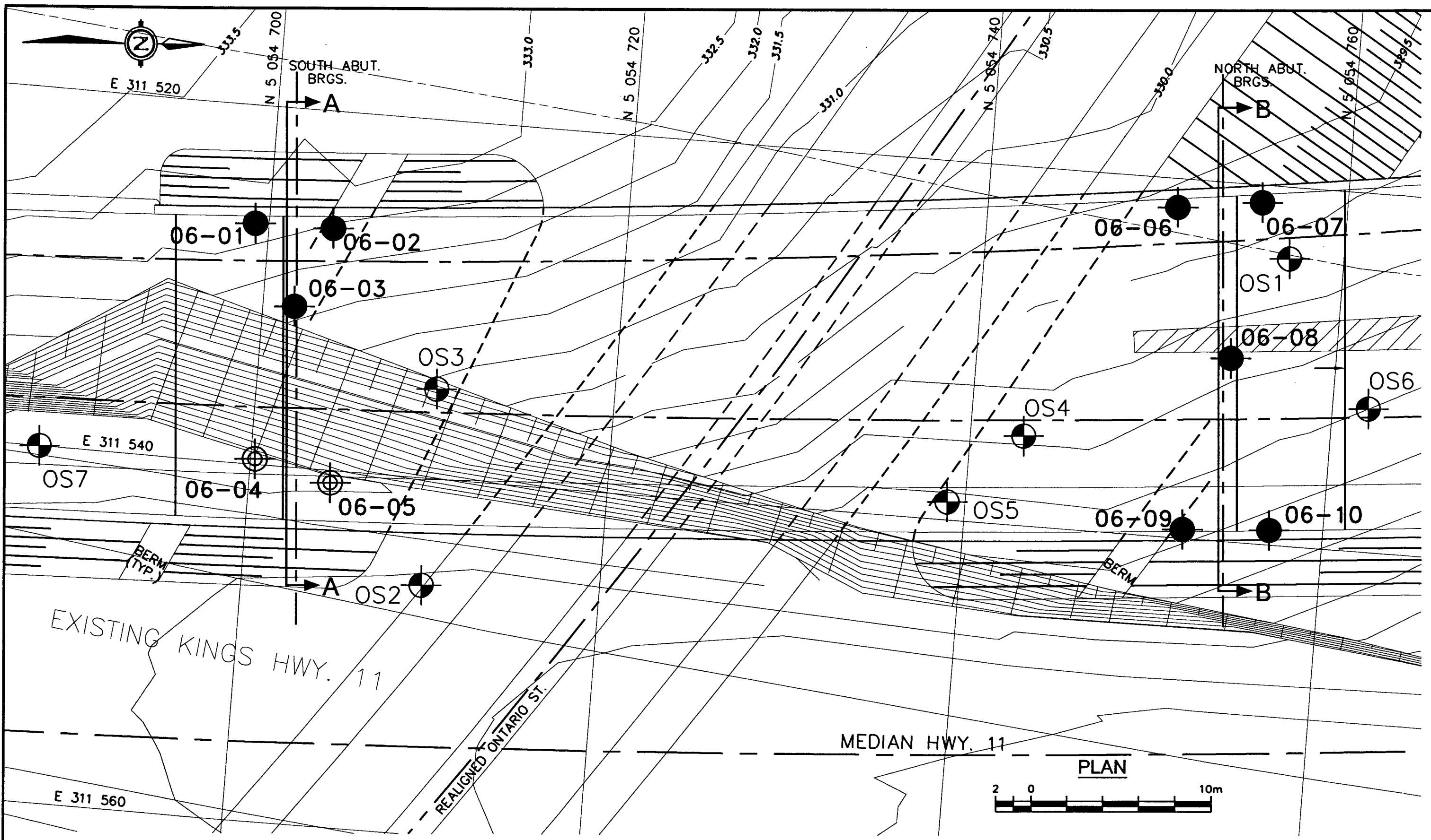
Tests for Dowels into Rock shall have a capacity of at least 1035 kN. The Quality Verification Engineer shall report on the acceptance of the tests for Dowels into Rock. The Quality Verification Engineer shall report on the testing of the Dowels into Rock including recommendations for increasing embedment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

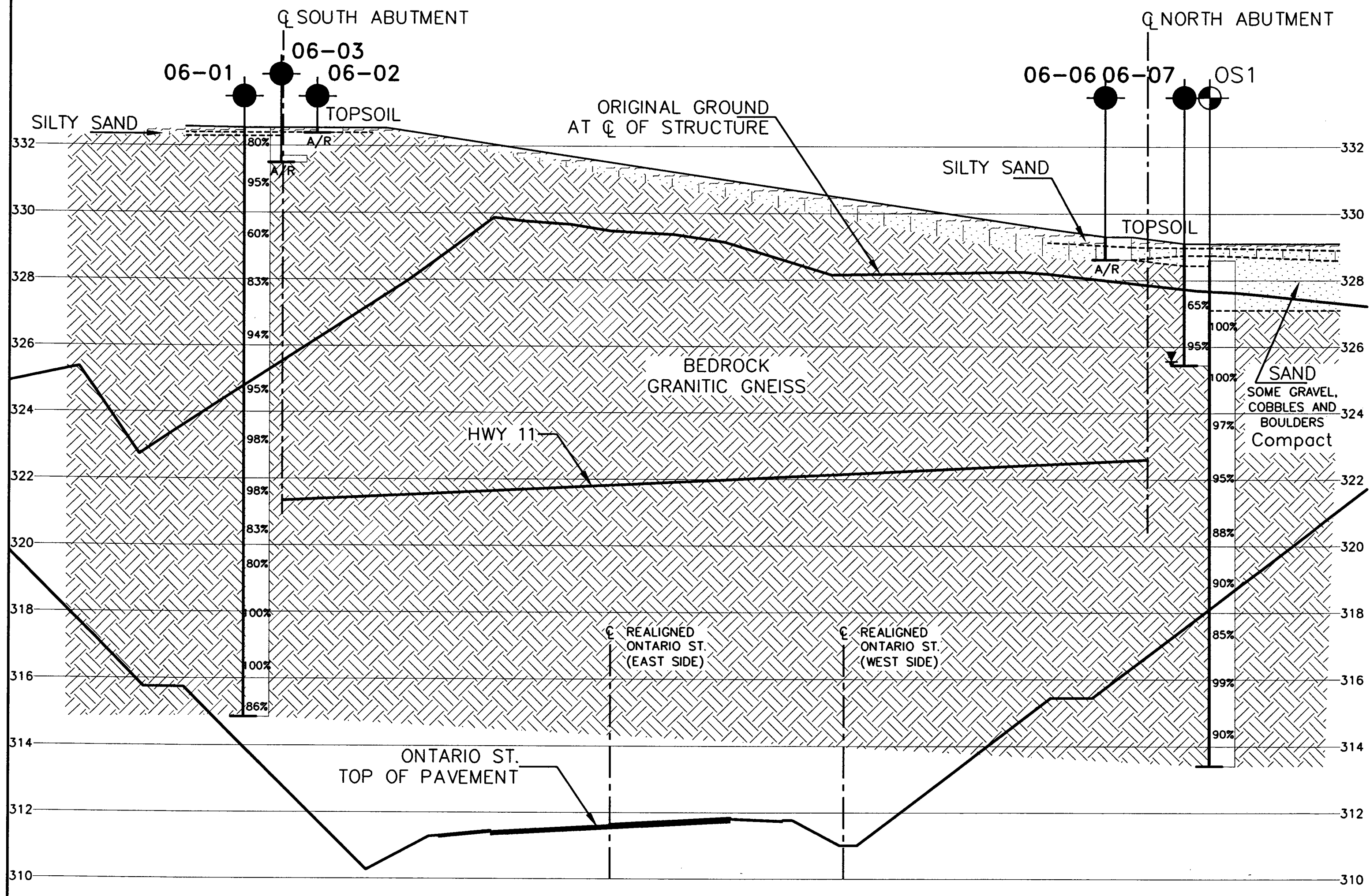
Appendix F

Drawings



PLAN

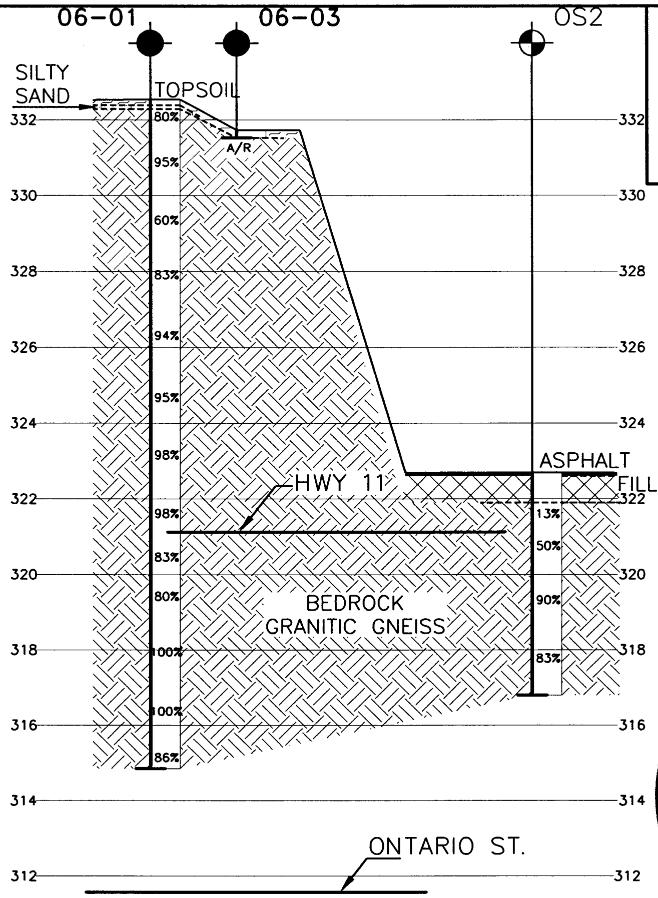
2 0 10m



PROFILE OF ℄ OF HWY 11

2 0 10m Hort 1:400

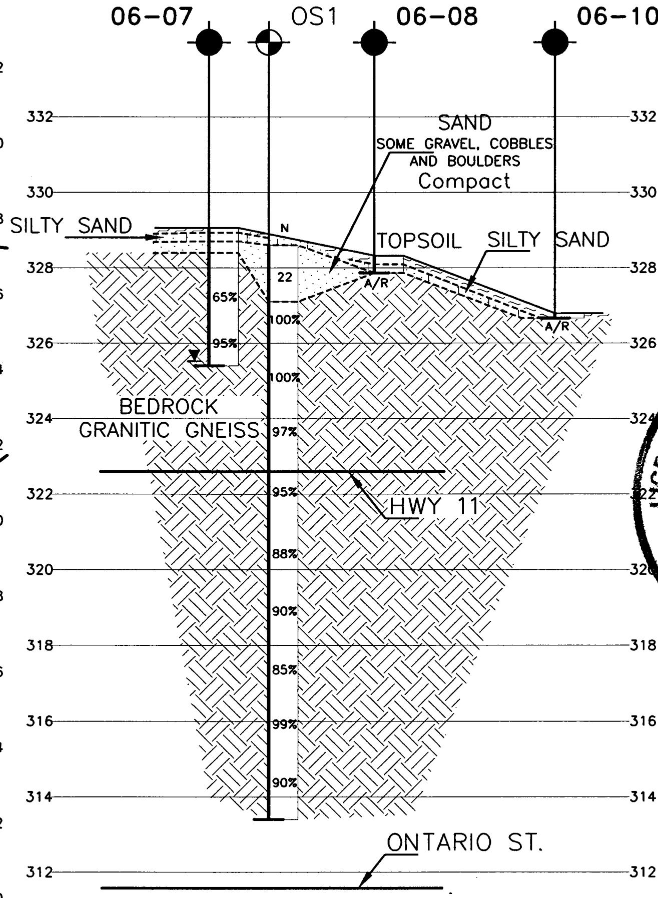
1 0 5m Vert 1:200



SECTION A-A

2 0 10m Hort 1:400

1 0 5m Vert 1:200



SECTION B-B

2 0 10m Hort 1:400

1 0 5m Vert 1:200 DRAWING NOT TO BE SCALED
100 mm ON ORIGINAL DRAWING

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

HWY 11
CONT No
WP No 473-93-00

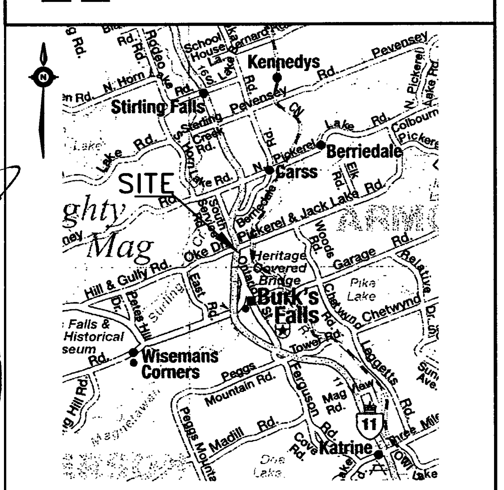
SHEET

REALIGNED ONTARIO STREET
CROSSING AT HIGHWAY 11
SBL
BOREHOLE LOCATIONS AND SOIL STRATA

Marshall Macklin Monaghan
PROJECT MANAGERS • ENGINEERS • SURVEYORS • PLANNERS

THURBER ENGINEERING LTD.
GEOTECHNICAL • ENVIRONMENTAL • MATERIALS

A. E. GORMAN
NOV 22/06



KEYPLAN

LEGEND

●	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
06-01	332.54	5 054 699.25	311 526.61
06-02	332.48	5 054 703.59	311 526.55
06-03	331.73	5 054 701.77	311 531.05
06-04	320.26	5 054 700.24	311 539.70
06-05	321.50	5 054 704.50	311 540.70
06-06	329.32	5 054 750.62	311 521.62
06-07	329.06	5 054 755.26	311 520.99
06-08	328.32	5 054 754.16	311 529.77
06-09	327.05	5 054 752.24	311 539.51
06-10	326.79	5 054 757.02	311 539.19

P. K. CHATTERJI
Nov 22/06

-NOTE-

The boundaries between soil strata have been established only at Bore Hole locations. Between Bore Holes the boundaries are assumed from geological evidence.

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

MINISTRY OF TRANSPORTATION, ONTARIO
PR-D-707
88-05
PLOT SCALE 1:1

MODIFIED:

DRAWING NAME:
CREATED:

FILENAME: E:\Job Files\19\1423\31 Burk's Falls Overpass\SHEETS\ted23313-hwy11sbl-ontariost.dwg
PLOTDATE: Nov 20, 2006 – 5:15pm

Appendix G

Site Photographs

Photograph #1, SBL Site from North in March 2006

Photograph #2, SBL Site in August 2006. Note thin cover on bedrock.