FOUNDATION INVESTIGATION AND DESIGN REPORT
BERNARD CREEK BRIDGE ON ROBINS ROAD
HIGHWAY 11 BURK'S FALLS TO SOUTH RIVER
G.W.P. 742-93-00, SITE: 44-92

Geocres Number: 31E-266

Report to

Marshall Macklin Monaghan

Thurber Engineering Ltd.
2010 Winston Park Drive, Suite 103
Oakville, Ontario
L6H 5R7
Phone: (905) 829 8666
Fax: (905) 829 1166

December 13, 2006
File: 19-1423-12

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TABLE OF CONTENTS

Part 1

Part 2

THURBER

Appendices

FOUNDATION INVESTIGATION AND DESIGN REPORT
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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 the Bernard Creek Bridge on Robins Road. This bridge will be constructed as part of the proposed four-laning of Highway 11 in the Township of Strong, Ontario.

The purpose of this 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 investigation.

Reference was also made to the results of a preliminary borehole investigation by Golder Associates. The location of this borehole is shown on the Borehole Locations and Soil Strata drawing in Appendix G and the borehole log is included in Appendix C.

Thurber carried out the investigation as a sub-consultant to Marshall Macklin Monaghan, under the Ministry of Transportation Ontario (MTO) Agreement Number 5005-A-000188.

2 SITE DESCRIPTION

The site is located approximately 700 m east of the existing Highway 11 in the Township of Strong and approximately 75 m south of the present structure carrying Robins Road over Bernard Creek. The site lies approximately 10 km north of the town of Burk's Falls.

The general 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 where exposed.

Bedrock outcrops are apparent nearby and the immediate area is generally wooded, but giving way to farmland closer to the highway. Bernard Creek flows in a generally southward direction at the site and the stream is comparatively fast flowing with a bed composed of gravel with cobbles and boulders. There are numerous boulders at the ground surface. Bedrock is exposed at the northeast corner of the existing structure.

At the site of the proposed crossing, the valley slopes are comparatively gentle and wooded. At the existing structure, the stream flows in a well defined channel but at the proposed site the edge of water is poorly defined and the width is estimated to be 10 to 12 m and the depth is less than 1 m.

There are isolated dwellings in the vicinity of the bridge site.

Photograph 1 in Appendix H shows the existing structure and Photograph 2 shows the site of the proposed structure.

3 SITE INVESTIGATION AND FIELD TESTING

The site investigation and field testing for this project were carried out between the periods of January 12 to January 13, and March 22 to 23, 2005. Eight boreholes numbered 92-1, 92-2, 92-4, 92-6, 92-9, 92-11, 92-13 and 92-14 were drilled to depths ranging from 0.6 m to 5.8 m. The approximate locations of the boreholes are shown on the attached Borehole Locations and Soil Strata Drawing in Appendix G.

As described later in the report, the investigation encountered shallow bedrock at both sides of the creek. Under the Terms of Reference, a total of six sampled boreholes are required at each foundation element. At this site, it was not possible to drill that number of boreholes within each of the foundation footprints due to the fact that the foundations are essentially in the creek. An ice platform was constructed to allow drilling at the outer boreholes of the east abutment. The outer edge of the west abutment was also accessible.

As shown on "Record of Borehole Sheets" and the "Borehole Location and Soil Strata" drawing, the proven rock elevations across the two foundation locations ranged from 324.1 to 325.4. In two boreholes where rock was not proved by coring, auger refusal was encountered at elevations of 324.9 and 325.7. This data is interpreted as indicating a comparatively low risk of large or unexpected variations in bedrock elevation.

A combination of hollow-stem auger drilling techniques and diamond coring methods were used to advance the boreholes. Samples were obtained from the overburden at selected intervals using a split spoon sampler in conjunction with Standard Penetration Testing (SPT). Overburden sampling was continued until auger refusal was encountered and at two boreholes in each foundation element the boreholes were advanced 2.8 m to 3.2 m into bedrock by NQ size diamond coring techniques.

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 Oakville 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.

Since the site stratigraphy consists of shallow deposits of non-cohesive soils over bedrock and the groundwater is presumed to be controlled by the creek, no piezometers were installed at this site.

4 LABORATORY TESTING

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

5 DESCRIPTION OF SUBSURFACE CONDITIONS

Reference is made to the Record of Borehole sheets in Appendix A. Details of the encountered soil and rock stratigraphy are presented in this appendix and on the "Borehole Locations and Soil Strata" drawing in Appendix G. 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 0.6 m to 2.7 m of overburden soils overlying Pre-Cambrian bedrock. The overburden soils generally consist of sands or sand and gravel.

5.1 Topsoil

Topsoil was encountered only under the east approach. A 300 mm thick layer of organic topsoil with roots was recorded at that location.

5.2 Sand

All boreholes encountered a layer of sand at the surface, except at the east approach where the sand is overlain by topsoil. The composition of the sand is somewhat variable but it is described as sand, trace silt, trace gravel, occasional to frequent cobbles and boulders. The soil is brown and moist to wet, with natural moisture contents ranging from 14 to 26%. Two higher values were recorded (44 and 59%) but these may be due to sampling problems or very localized changes in the sand gradation.

The thickness of the sand layer ranged from 0.6 to 2.1 m and it extended to depths of 0.6 to 2.1 m from the ground surface. The underside of the sand layer lay at Elevation 327.5 at the west approach and 326.8 at the east approach dropping to a low of 324.7 at the west abutment.

SPT 'N' values at the abutments and west approach generally ranged from 25 to more than 50 blows for 0.3 m penetration and the soil is classed as dense. Occasional SPT values exceeding 100 blows for 0.3 m of penetration are attributed to contact with contact with cobbles, boulders or the bedrock. At the east approach, the sand is classed as compact, based on a SPT value of 17 blows for 0.3 m of penetration.

Seven selected sample from this deposit was subjected to grain size distribution tests and the results are presented in Appendix B. Figure B1 shows the data from the west approach and abutment and Figure B2 shows data from the east abutment. The results indicate that the soil contains 2 to 12% gravel, 58 to 69% sand and 11 to 29% silt.

5.3 Sand and Gravel

The layer of dense sand was found to be underlain by a layer of sand and gravel with occasional to frequent cobbles and boulders except at the west approach. The soil is brown and wet, with natural moisture contents of 16 to 21%.

The thickness of the sand and gravel deposit ranged from 0.5 to 0.8 m and it extended to depths of 1.7 to 2.7 m below ground surface. The underside of this soil layer lay at elevation 325.0 at the north end of the west abutment to 324.0 at the south end.

SPT N values ranged from 74 to values greater than 100 blows for 0.3 m of penetration and the soil is classed as very dense.

There was insufficient sample recovery in this deposit to conduct a representative grain size analysis. The soil was classified on the basis of visual examination.

5.4 Bedrock

The overburden soils described above are underlain by gneiss bedrock. Bedrock was proved by coring in two boreholes at each of the west and east abutments. In the remaining boreholes, auger refusal was assumed to indicate that bedrock had been encountered. Table 5.1 summarizes the bedrock depth and the elevations to the top of bedrock.

TABLE 5.1 – Depth to Bedrock

Location	BH Number	Depth to Bedrock (m)	Top of Bedrock Elevation (m)	Proved by
West approach	92-1	1.5	327.5	Refusal
West abutment	92-2	1.7	325.0	Cored
	92-4	1.7	324.9	Refusal
	92-6	2.7	324.1	Cored
East abutment	92-9	1.7	324.8	Cored
	92-11	0.6	325.7	Refusal
	92-13	1.0	325.4	Cored
East approach	92-14	1.5	326.7	Refusal

The gneiss bedrock is generally described as fresh to slightly weathered and massive to thinly banded. Its colour is generally black and white.

Core recovery in the bedrock was generally between 83% and 100%. The RQD values generally ranged from 72% 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 4.

The unconfined compressive strength of most of the rock cores is estimated to range between 78 and 119 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

Due to the highly pervious nature and small thickness of the soils overlying bedrock at this site, piezometers were not installed.

Groundwater levels at the foundations will be the same as the creek level, which was determined to be at Elevation 325.9 in August 2002, and will fluctuate with the creek level.

6 MISCELLANEOUS

All-Terrain Drilling of Waterloo, Ontario supplied a track mounted CME 75 drill rig and conducted the drilling, sampling and in-situ testing operations.

Full time supervision of the drilling and sampling program was provided by Mr. George Azzopardi of Thurber.

Layout of the boreholes was carried out by a survey crew from Marshall Macklin Monaghan.

Mr Alastair E. Gorman, P.Eng. provided overall direction the field investigation and prepared the report.

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

Thurber Engineering Ltd.
Alastair E. Gorman, P.Eng.
Senior Foundations Engineer.

REGISTERED PROFESSIONAL ENGINEER
A. E. GORMAN
DEC 13/06
PROVINCE OF ONTARIO

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

LICENSED PROFESSIONAL ENGINEER
P. K. CHATTERJI
Dec 13/06
PROVINCE OF ONTARIO

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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 Robins Road will be realigned approximately 75 m south of the present crossing over Bernard Creek. The crossing will be carried on an 18.8 m, single span concrete structure.

At the west abutment, the finished grade of Robins Road will be at Elevation 329.4 and the existing ground surface averages Elevation 326.7, resulting in an approach embankment approximately 2.7 m high. At the east abutment, the finished grade of Robins Road will be at Elevation 330.6 and the existing ground surface averages Elevation 326.4, resulting in an approach embankment approximately 4.2 m high.

The existing structure, as shown in Photo 1 in Appendix H, consists of a timber deck supported on two abutments and a timber bent in the middle of the creek. There is no information regarding how the bent is founded. At the abutments, armour stone and a timber abutment or ballast wall are visible but it is unclear whether the abutments are supported on concrete foundations or on a rock-filled timber crib. The foundations and approaches of this structure appear to be performing satisfactorily for the level of service the Robins Road provides.

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.

8 STRUCTURE FOUNDATIONS

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

The stratigraphy encountered at the foundation elements consist of 0.6 m to 2.7 m of overburden soils overlying bedrock. The overburden consists of topsoil, sand and sand and gravel with occasional to frequent cobbles and boulders, underlain by bedrock. The water level to be considered in design will be the creek level, which was at Elevation 325.9 in August 2002. The 50-year flood elevation is understood to lie at Elevation 326.4.

Based on the proximity of the bedrock surface to the founding levels, consideration was given to the following foundation types:

- Spread footings on bedrock
- Augered Caissons (drilled shafts)

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

Anecdotal information from local residents indicates that there was once a sawmill operating at or close to this location. There is, therefore, the possibility of encountering remnants of earlier construction that may present obstructions to the planned construction activities.

The contract documents must contain a notice alerting Bidders to the presence of cobbles and boulders in the overburden soil and to the possible presence of obstructions in the form of remnants from previous construction activity at or near the site.

8.1 Spread Footings on Bedrock

The preliminary General Arrangement drawing indicates that the underside of the footings will lie a short distance above or below the top of bedrock, depending on the variations in the top of bedrock across the site.

Two design options that can be considered for the support of footings on bedrock are:

- Design the footing to bear directly on bedrock
- Design the footing to bear at an elevation appropriate to the structure and place mass concrete fill between the underside of the footing and the bedrock.

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.

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.

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.

8.2 Spread Footing Construction

In either of the above cases for a spread footing, all overburden must be stripped from the bedrock surface within the footprint of the footing or the mass concrete fill and any broken, detached fragments of rock must be removed. The excavation must be unwatered prior to placing concrete.

The footings will be constructed in close proximity to, or partially in the creek and the overburden soil at the abutment locations is highly permeable. Cleaning and unwatering the footing excavation will necessitate the implementation of procedures to control both the groundwater and the surface water from the creek. Procedures for controlling water from the creek must take account of the possibility of flood conditions in the creek. The selection, design and implementation of these controls must be the responsibility of the Contractor, but for the purposes of assessing constructability two possible systems are:

1. Excavation within a closed sheet pile cofferdam
2. Construction of a sand bag cofferdam within an initially oversized excavation.

From a foundations perspective, it is acceptable to leave the cofferdam in place permanently.

8.2.1 Sheet Pile Cofferdam

If a sheet pile cofferdam is selected, the procedure will be generally as follows:

1. Strip the boulders from the ground surface
2. Install the sheet piling to bedrock
3. Excavate within the cofferdam, installing bracing as necessary
4. Unwater the excavation, placing seal or filter material at the toe of the sheeting as necessary (this may take the form of sandbags or similar material to prevent infiltration of soil and to reduce water inflow as far as is practicable)

Disadvantages associated with this method include:

- Difficulties associated with installing and supporting sheet piling through shallow deposits of cohesionless soil (contractor may elect to build an earth berm around the excavation and drive sheeting through this berm)
- Difficulties in obtaining sufficient seal at the toe of the sheeting to allow unwatering

The main advantage of this method is that the support can be installed in advance of excavation.

8.2.2 Sandbag Cofferdam

If a sandbag cofferdam is selected, the procedure will be generally as follows:

1. Excavate to bedrock
2. Construct a cofferdam using sandbags (from a foundations perspective, these can be filled with sand or filled with dry concrete mix)
3. Unwater the excavation, placing additional sandbags if necessary
4. Clean out the remaining disturbed material in the base of the excavation.

The main disadvantages of this method are:

- The requirement for an oversized excavation in saturated, cohesionless soils below the water table
- The impact on the creek channel as a result of the large excavation.

Some contractors may prefer to sandbag an oversize excavation rather than attempt to install sheeting.

8.2.3 Unwatering

The excavation must be unwatered prior to placing structural concrete. At this site, however, unwatering may be difficult due to the problems in obtaining a seal between the cofferdam and the underlying bedrock. Contractors bidding the work may conclude that it will be necessary to tremie a mud slab in place to control the inflow of water prior to attempting to unwater the excavation. From a foundations perspective, this is acceptable provided they allow for the extra depth of excavation necessary to accommodate the thickness of the mud slab. In presenting any proposal to tremie a mud-slab, the contractor must also demonstrate how he will satisfactorily clean the bearing surface prior to placing the tremie concrete.

8.3 Caissons

Given the proximity of the bedrock surface at this site, caissons would not normally be considered to be an economical alternative. However, in view of the location of the foundations in or close to the edge of the creek and the potential difficulty in unwatering an excavation, consideration can be given to a caisson foundation as a means of solving constructability issues.

The Contractor must determine the details of the method of construction, having regard to his equipment and experience. However, for the purposes of design and evaluation, the following methodology is suggested:

1. Advance a steel liner with an internal diameter equal to or greater than the required diameter of the caisson shaft
2. Drill the liner into the top of the bedrock to exclude, as much as feasible, infiltration of soil
3. Advance a socket of the required diameter into the bedrock to the required depth
4. Remove as much drill debris from the socket as is feasible
5. Place the concrete for the caisson by tremie methods.

Unless the Contractor brings forward an alternate proposal that is acceptable to the Ministry, the methodology described above inherently requires that:

- The steel liner be left in place permanently and be cut off at the top of the caisson shaft
- That the liner be fitted with a drill shoe that will be left in place with the liner and thus will be a consumable.

Since the caisson excavation will not be unwatered and it will not necessarily be possible to clean the base of the excavation, the caisson must be designed on the basis of the bond strength between the concrete and the bedrock. The ultimate geotechnical resistance should be computed on the basis of the shaft area in the socket and a bond strength of 5,000 kPa. A resistance factor of 0.4 must be applied to the ultimate resistance when design for shaft adhesion.

8.4 Frost Cover

The design depth of frost penetration at this site is 1.8 m.

Frost penetration is not an issue for footings bearing on bedrock or mass concrete fill on bedrock.

8.5 Preferred Foundation

From a foundations technical perspective, a spread footing bearing on bedrock/mass concrete fill is the preferred option.

9 ABUTMENT TYPE

The shallow bedrock and the foundation systems considered feasible for this site, i.e. footing on bedrock or caissons socketed into bedrock, are not suitable for the design of an integral abutment structure.

From a foundations perspective, the site is suitable for the design of conventional abutments or semi-integral abutments.

10 EXCAVATION AND BACKFILL

10.1 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 above the water table and Type 4 soils below the water table. Excavation below the groundwater level/creek level is not recommended without the use of a cofferdam.

10.2 Foundations

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

Bidders must be alerted to the fact that excavation must be carried out through cohesionless soils, including deposits of cobbles and boulders, under the groundwater table and terminate on an uneven bedrock surface.

The methods used to excavate, control groundwater and maintain a stable excavation must be selected by the Contractor. However, when different options are evaluated, it must be recognized that there may be difficulties in unwatering an excavation and control the inflow of soil under the toe of the cofferdam.

11 CONTROL OF WATER INFLOW

At this site, the control of groundwater and surface water from the creek will potentially present a serious constructability issue.

The design of any system to control the water is the responsibility of the Contractor. However, suitable systems that might be considered include a steel sheet pile cofferdam or drilled in circular liner as discussed elsewhere in this report.

12 APPROACH EMBANKMENTS

The approach embankments will be in the order of 2.7 m high at the west and 4.2 m at the east.

Due to the site location and the possibility of high velocity flows under flood conditions, it is recommended that the approaches be constructed using rock fill.

From the point of view of slope stability, a rock fill embankment will be internally stable if constructed with side slopes no steeper than 1.25H:1V.

The global stability of a rock fill embankment was analyzed using the commercially available slope stability program GSLOPE® developed by Mitre Software Inc. The Bishop's simplified method for stability analysis was employed and drained analysis was carried out, considering the permeable nature of the overburden soils. The analysis was also carried out taking account of potential seismic acceleration.

The following factors of safety obtained are shown in Table 12.1.

Table 12.1 – Results of Stability analysis

Location	Factors of Safety	
	Normal Case	Seismic Case
West approach	1.6	1.3
East approach	1.6	1.2

Output from the analyses is included in Appendix E.

It is recommended that all topsoil, organics, loose soils and other deleterious material be removed from the footprint of the approach fills. Embankment construction should be in accordance with the most recent version of SP 206S01.

At the anticipated heights, mid-height berms will not be required.

13 RETAINED SOIL SYSTEMS

Given the conditions prevailing at this site, the RSS would have to be founded on cohesionless soil below the creek level. Construction of a "high Performance", "High Appearance" RSS wall under these circumstances is not considered to be practicable or economic.

Accordingly, RSS walls are not recommended for this site.

14 BACKFILL TO ABUTMENTS

In the case of semi-integral abutments, backfill to the abutment must be granular material. In the case of a conventional abutment, granular backfill is recommended but rock backfill can be permitted. A NSSP is required to limit rock fill used as abutment backfill to fragments no greater than 150 mm and including adequate spalls to fill voids in the rock fill.

In all cases where the approach embankment consists of rock fill and granular backfill to the abutment wall is used, the granular backfill must consist of OPSS Granular "B" Type II.

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, and rock backfill should be placed to the extents shown in OPSD 3505.000 and the design must incorporate a subdrain as shown in these drawings.

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.

15 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 respective OPSD, i.e. a line projected up at 1.5H:1V for granular backfill and 1.25H:1V for rock 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. The amount of wall movement required for the development of active, passive and at-rest earth pressures may be interpreted using Figure C6.9.1(a) in the Commentary to the CHBDC.

For a fully drained condition, the static 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 15.1.

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) might be preferred 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) might be 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 15.1 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 15.1 – Earth Pressure Coefficients

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

16 SEISMIC CONSIDERATIONS

16.1 Seismic Design Parameters

The site is treated as lying in Seismic Zone 2. The following seismic parameters should be used for design:

- Velocity Related Seismic Zone 2
- Zonal Velocity Ratio 0.1
- Acceleration Related Seismic Zone 2
- Zonal Acceleration Ratio 0.1
- Peak Horizontal Acceleration 0.11

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.

16.2 Liquefaction Potential

The potential for liquefaction of the foundations soils was assessed using the Seed and Idriss (1971) method[1]

Using this method and assuming an earthquake of magnitude 7.5, it is estimated that under the existing conditions there is negligible potential for liquefaction of the foundation soils below the approach fills. It is recommended elsewhere in this report that the approach embankments be constructed using rock fill. Accordingly there is no danger of liquefaction of the embankment material itself.

The foundations will bear directly on bedrock and soil liquefaction is not an issue in that case.

16.3 Retaining Wall Dynamic Earth Pressures

In accordance with Clause 4.6.4 of the CHBDC, the design of retaining structures must take account of potential seismic loadings. Seismic events generally result in increased loading of the structure and the design should be checked using the active (K_{AE}) and passive (K_{PE}) earth pressure coefficients that incorporate the effects of earthquake loading. These coefficients are given in Table 16.1. 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.

[1] Seed, H.B. and Idriss, I.M. 1971, "Simplified Procedure for Evaluating Soil Liquefaction Potential" *Journal of Soil Mechanics and Foundations Division,* ASCE, Vol. 101, No. SM9, September, pp. 1249-1273.

Table 16.1 – Earth Pressure Coefficient for Earthquake Loading

Earth Pressure Coefficient (K) for Earthquake Loading						
	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	
Wall Condition	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

17 CONSTRUCTION CONCERNS

Potential construction concerns include, but are not necessarily limited to problems associated with constructing foundations that will lie partially or wholly in the creek, such as:

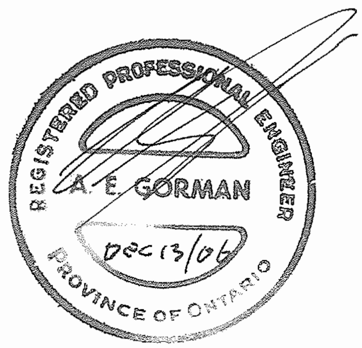
- The stability of excavations and the need for shoring or liners to retain the soil
- Difficulty in unwatering temporary excavations
- Problems associated with controlling the inflow of soil with the water prior to placing concrete
- Placement of concrete by tremie methods.

18 CLOSURE

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

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

Thurber Engineering Ltd.



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

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

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

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

$\underline{\nabla}$ 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

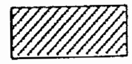
MAJOR DIVISIONS		GROUP SYMBOL	TYPICAL DESCRIPTION
COARSE GRAINED SOILS	GRAVEL AND GRAVELLY SOILS	GW	Well-graded gravels or gravel-sand mixtures, little or no fines.
		GP	Poorly-graded gravels or gravel-sand mixtures, little or no fines.
		GM	Silty gravels, gravel-sand-silt mixtures.
		GC	Clayey gravels, 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			

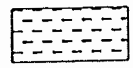
EXPLANATION OF ROCK LOGGING TERMS

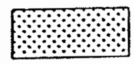
ROCK WEATHERING CLASSIFICATION

Fresh (FR)	No visible signs of weathering.
Fresh Jointed (FJ)	Weathering limited to the surface of major discontinuities.
Slightly Weathered (SW)	Penetrative weathering developed on open discontinuity surfaces, but only slight weathering of rock material.
Moderately Weathered (MW)	Weathering extends throughout the rock mass, but the rock material is not friable.
Highly Weathered (HW)	Weathering extends throughout the rock mass and the rock is partly friable.
Completely Weathered (CW)	Rock is wholly decomposed and in a friable condition, but the rock texture and structure are preserved.

SYMBOLS


CLAYSTONE


SILTSTONE


SANDSTONE


COAL


Bedrock (general)

DISCONTINUITY SPACING

Bedding	**Bedding Plane Spacing**
Very thickly bedded	Greater than 2m
Thickly bedded	0.6 to 2m
Medium bedded	0.2 to 0.6m
Thinly bedded	60mm to 0.2m
Very thinly bedded	20 to 60mm
Laminated	6 to 20mm
Thinly Laminated	Less than 6mm

TERMS

Total Core Recovery: (TCR)	Core recovered as a percentage of total core run length.
Solid Core Recovery: (SCR)	Percent Ratio of solid core of full cylindrical shape recovered. Expressed with respect to the total length of core run.
Rock Quality Designation: (RQD)	Total length of sound core recovered in pieces 0.1m in length or larger as a percentage of total core run length.
Uniaxial Compressive Strength (UCS)	Axial stress required to break the specimen
Fracture Index: (FI)	Frequency of natural fractures per 0.3m of core run.

STRENGTH CLASSIFICATION

Rock Strength	**Approximate Uniaxial Compressive Strength** (MPa)	(psi)	**Field Estimation of Hardness***
Extremely Strong	Greater than 250	Greater than 36,000	Specimen can only be chipped with a geological hammer
Very Strong	100-250	15,000 to 36,000	Requires many blows of geological hammer to break
Strong	50-100	7,500 to 15,000	Requires more than one blow of geological hammer to break
Medium Strong	25.0 to 50.0	3,500 to 7,500	Breaks under single blow of geological hammer.
Weak	5.0 to 25.0	750 to 3,500	Can be peeled by a pocket knife with difficulty
Very Weak	1.0 to 5.0	150 to 750	Can be peeled by a pocket knife, crumbles under firm blows of geological pick.
Extremely Weak (Rock)	0.25 to 1.0	35 to 150	Indented by thumbnail

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RECORD OF BOREHOLE No 92-1

METRIC

W.P. 480-93-00	LOCATION N 5 063 332.0 E 310 005.0		ORIGINATED BY GA
HWY 11	BOREHOLE TYPE Hollow Stem Augers		COMPILED BY WM
DATUM Geodetic	DATE 12.01.05 - 12.01.05		CHECKED BY MA

ELEV / DEPTH	SOIL PROFILE – DESCRIPTION	STRAT PLOT	SAMPLES – NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE) 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
329.0 / 0.0	**SAND**, some silt to silty, trace to some gravel Dense Brown Damp to Dry augers grinding on probable cobble or gravel		1	SS	31		329		○ (approx. 24)		
			2	SS	25		328		○ (approx. 6)		5 66 25 3
327.5 / 1.5	END OF BOREHOLE AT 1.52 m. AUGER REFUSAL AT 1.52 m ON PROBABLE BEDROCK OR BOULDER. BOREHOLE OPEN TO 1.52 m AND DRY UPON COMPLETION. BOREHOLEBACKFILLED WITH DRILL CUTTINGS.										

ONTMT4S 2316.GPJ 12/04/05

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

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

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RECORD OF BOREHOLE No 92-2

METRIC

W.P. 480-93-00 | LOCATION N 5 063 338.0 E 310 019.0 | ORIGINATED BY GA
HWY 11 | BOREHOLE TYPE Hollow Stem Augers | COMPILED BY WM
DATUM Geodetic | DATE 12.01.05 - 12.01.05 | CHECKED BY MA

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
326.7 / 0.0	**SAND**, some silt to silty, trace to some gravel, trace rootlets Compact Brown Wet	1	SS	29		326			4 76 18 3
325.5 / 1.2	**SAND and GRAVEL**, trace silt Very Dense Brown Wet	2	SS	50/ .150					
324.9 / 1.7	END OF SOIL SAMPLING AT 1.73 m. CORING STARTED AT 1.73 m. Slightly weathered to fresh, coarse grained, grey, white/black, strong to very strong **GNEISS**	1	RUN			325 324		FI 0 0 2 0	RUN 1# TCR=100%, SCR=100%, RQD=100%, UCS=106.2MPa
		2	RUN			323		0 0 0 0 2	RUN 2# TCR=100%, SCR=97%, RQD=97%, UCS=113.8MPa
322.2 / 4.5	END OF BOREHOLE AT 4.47 m. BOREHOLE OPEN TO 4.47 m AND WATER LEVEL AT 0.30 m UPON COMPLETION. BOREHOLE GROUTED TO SURFACE.							1	

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

+³, ×³: Numbers refer to Sensitivity

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

RECORD OF BOREHOLE No 92-4

METRIC

W.P. 480-93-00
LOCATION N 5 063 332.2 E 310 019.0
ORIGINATED BY GA
HWY 11
BOREHOLE TYPE Hollow Stem Augers
COMPILED BY WM
DATUM Geodetic
DATE 12.01.05 - 12.01.05
CHECKED BY MA

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE) (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
326.6											
0.0	**SAND**, some silt to silty, trace to some gravel, occasional cobbles and boulders Very Dense Grey Wet		1	SS	57		326		~20		19 66 13 2
325.6 0.9	**SAND and GRAVEL**, trace silt Very Dense Grey Wet more frequent cobbles and boulders below 1.37 m		2	SS	74				~15		
324.9 1.7	END OF BOREHOLE 1.68 m. AUGER REFUSAL AT 1.68 m ON PROBABLE BEDROCK OR BOULDER. BOREHOLE OPEN TO 1.68 m AND WATER LEVEL AT SURFACE. BOREHOLE BACKFILLED WITH DRILL CUTTINGS.						325				

+ 3, × 3: Numbers refer to Sensitivity

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

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RECORD OF BOREHOLE No 92-6

METRIC

W.P. 480-93-00 LOCATION N 5 063 327.0 E 310 018.1 ORIGINATED BY GA

HWY 11 BOREHOLE TYPE Hollow Stem Augers COMPILED BY WM

DATUM Geodetic DATE 13.01.05 - 13.01.05 CHECKED BY MA

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
326.8 / 0.0	**SAND**, some silt to silty, trace to some gravel, occasional cobbles and boulders. Dense to Very Dense Brown Wet	1	SS	45				
		2	SS	50	326			
	frequent cobbles and boulders	3	SS	50/ .100	325			7 80 11 2
324.7 / 2.1	**SAND and GRAVEL**, trace silt Very Dense Brown Wet	4	SS	50/ .150			FI	
324.0 / 2.7	END OF SOIL SAMPLING AT 2.74 m. CORING STARTED AT 2.74 m. Slightly weathered to fresh, coarse grained, massive, black and white, laminated to thinly banded, strong to very strong **GNEISS**	1	RUN		324, 323		2, 2, 1, 3, 0	RUN 1# TCR=97%, SCR=97%, RQD=72%, UCS=90.2MPa
		2	RUN		322		0, 1, 2, 2, 1	RUN 2# TCR=100%, SCR=100%, RQD=85%, UCS=118.8MPa
321.0 / 5.8	END OF BOREHOLE AT 5.79 m. BOREHOLE OPEN TO 5.79 m. BOREHOLE GROUTED TO SURFACE.							

Column headings: SOIL PROFILE (ELEV DEPTH, DESCRIPTION, STRAT PLOT); SAMPLES (NUMBER, TYPE, "N" VALUES); GROUND WATER CONDITIONS; ELEVATION SCALE; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ 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

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

Ministry of Transportation Ontario

THURBER

RECORD OF BOREHOLE No 92-9

METRIC

W.P. 480-93-00 LOCATION N 5 063 336.0 E 310 043.1 ORIGINATED BY SL

HWY 11 BOREHOLE TYPE Hollow Stem Augers/NQ Coring COMPILED BY WM

DATUM Geodetic DATE 22.03.05 - 22.03.05 CHECKED BY MA

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
326.5								
0.0	**SAND**, some silt to silty, trace to some gravel. Dense Brown Moist							
		1	SS	33	326	o (~13)		
		2	SS	37		o (~57)		9 67 22 2
324.9 / 1.7	END OF SAMPLING AT 2.64 m. Coring started at 2.64 m. **GRANITIC GNEISS**, thinly bedded				325		FI	
		1	RUN				2, 1	RUN 1# TCR=83%, SCR=83%, RQD=83%, UCS=101.6MPa
		2	RUN		324		0, 3, 0, 3	RUN 2# TCR=100%, SCR=100%, RQD=100%, UCS=96.0MPa
		3	RUN		323		0, 0, 1, 4	RUN 3# TCR=100%, SCR=100%, RQD=97%, UCS=81.9MPa
321.8 / 4.7	END OF BOREHOLE AT 4.72 m.				322			

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; GROUND WATER CONDITIONS; 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

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

RECORD OF BOREHOLE No 92-11

METRIC

W.P.	480-93-00	LOCATION	N 5 063 330.0 E 310 042.1	ORIGINATED BY	SL
HWY	11	BOREHOLE TYPE	Hollow Stem Augers	COMPILED BY	WM
DATUM	Geodetic	DATE	23.03.05 - 23.03.05	CHECKED BY	MA

ELEV / DEPTH	SOIL PROFILE: DESCRIPTION	STRAT PLOT	SAMPLES: NUMBER	SAMPLES: TYPE	"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 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
326.3											
0.0	**SAND**, some silt to silty, trace to some gravel. Dense Brown Moist		1	SS	36		326		○		14 63 21 2
325.7 / 0.6	END OF BOREHOLE AT 0.61 m. AUGER REFUSAL AT 0.61 m ON PROBABLE BEDROCK OR BOULDER. BOREHOLE BACKFILLED WITH DRILL CUTTINGS.										

+ 3, × 3: Numbers refer to Sensitivity

20 15 5 10 (%) STRAIN AT FAILURE

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RECORD OF BOREHOLE No 92-13

METRIC

W.P. 480-93-00
LOCATION N 5 063 325.0 E 310 042.0
ORIGINATED BY SL
HWY 11
BOREHOLE TYPE Hollow Stem Augers/NQ Coring
COMPILED BY WM
DATUM Geodetic
DATE 23.03.05 - 23.03.05
CHECKED BY MA

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"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)	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
326.4											
0.0	**SAND**, some silt to silty, trace to some gravel, occasional cobbles Dense Brown Wet		1	SS	50/ .050				○		
			2	SS	37		326				23 56 19 2
325.4 1.0	END OF BOREHOLE AT 2.59 m. CORING STARTED AT 2.69 m. **GRANITIC GNEISS**, thinly bedded									FI	
			1	RUN			325			2 0	RUN 1# TCR=100%, SCR=100%, RQD=81%, UCS=77.8MPa
			2	RUN			324			0 0 0 0 0	RUN 2# TCR=100%, SCR=100%, RQD=100%, UCS=94.8MPa
			3	RUN			323			0 0 1	RUN 3# TCR=100%, SCR=100%, RQD=100%, UCS=116.5MPa
322.2 4.2	END OF BOREHOLE AT 4.19 m. BOREHOLE BACKFILLED WITH BENTONITE BENSEAL.										

ONTMT4S 2316.GPJ 12/04/05

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

15 20 5 10 (%) STRAIN AT FAILURE

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RECORD OF BOREHOLE No 92-14

METRIC

W.P. 480-93-00 LOCATION N 5 063 328.0 E 310 056.1 ORIGINATED BY SL

HWY 11 BOREHOLE TYPE Hollow Stem Augers COMPILED BY WM

DATUM Geodetic DATE 23.03.05 - 23.03.05 CHECKED BY MA

ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"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 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
328.2											
0.0	**ORGANICS**, with rootlets Black										
328.0 0.3	**SAND**, some silt to silty, trace to some gravel, occasional cobbles, occasional wood fragments Compact Brown						328				
			1	SS	17				○ (approx. 45)		
326.8 1.5	END OF BOREHOLE AT 1.74 m. AUGER REFUSAL AT 1.74 m ON PROBABLE BEDROCK OR BOULDER. BOREHOLE OPEN TO 1.07 m AND WATER LEVEL AT 1.07 m UPON COMPLETION. BOREHOLE BACKFILLED WITH DRILL CUTTINGS.						327				

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

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

Appendix B

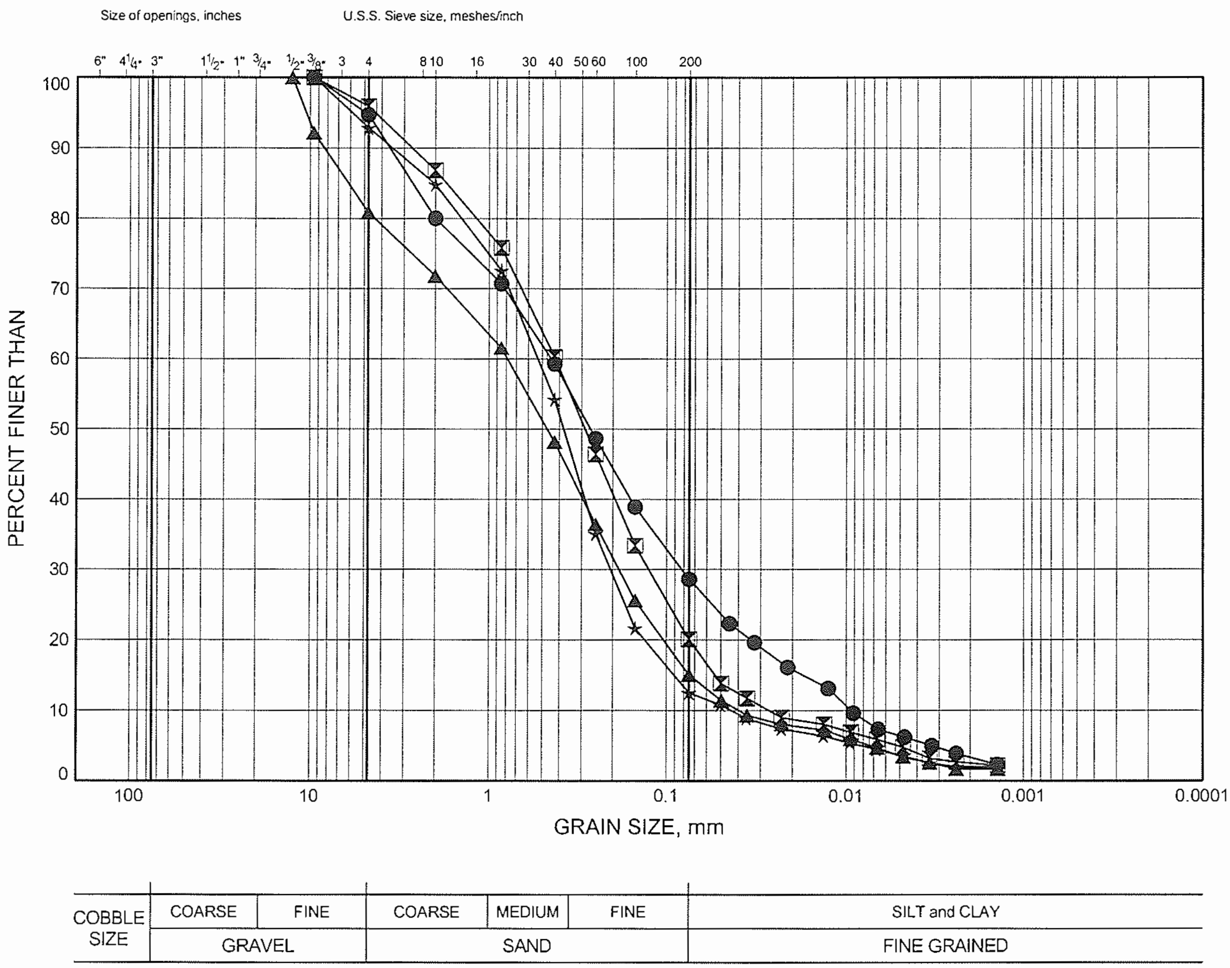
Laboratory Test Results

Hwy 11 Katrine

GRAIN SIZE DISTRIBUTION

FIGURE B1

SAND, SOME SILT TO SILTY, TRACE GRAVEL



SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	92-1	1.07	327.97
⧖	92-2	0.76	325.91
▲	92-4	0.46	326.09
★	92-6	1.83	324.96

Date December 2006

Project 5404-04-01

THURBER

Prep'd JHL

Chkd. AEG

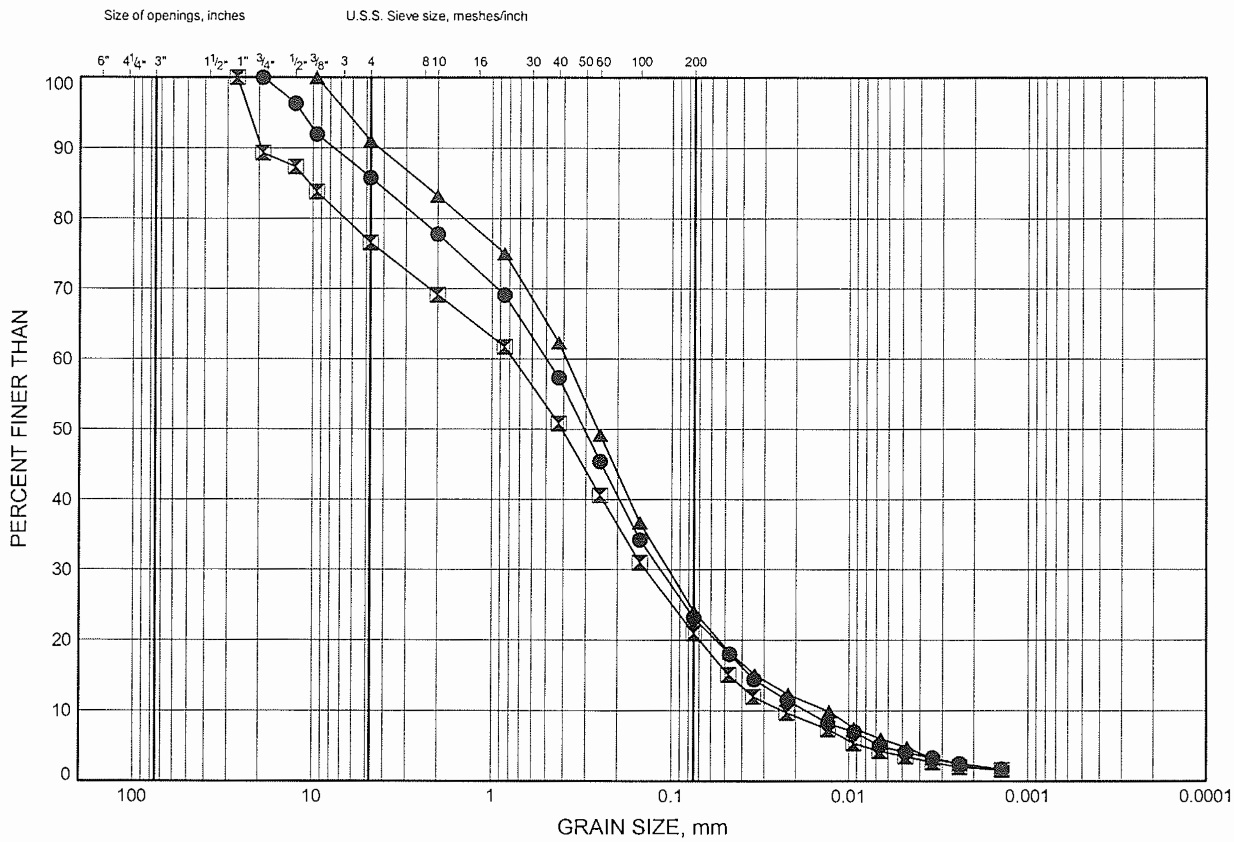
THURBGSD 2316.GPJ 12/12/06

Hwy 11 Katrine

GRAIN SIZE DISTRIBUTION

FIGURE B2

SAND, SILTY, TRACE TO SOME GRAVEL



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

SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	92-11	0.30	326.00
⊠	92-13	0.51	325.87
▲	92-9	1.39	325.14

THURBGSD 2316.GPJ 12/12/06

Date December 2006

Project 5404-04-01

THURBER

Prep'd JHL

Chkd. AEG

Robins Road Bridge
Point Load Test Results

Depth feet	Depth Inches	m	Is50	UCS (MPa)
92-2				
7	0	2.13	4.80	115.08
8	0	2.44	3.76	90.20
9	0	2.74	3.59	86.05
10	0	3.05	4.36	104.71
11	0	3.35	5.62	134.78
12	0	3.66	3.76	90.20
13	0	3.96	4.23	101.60
14	0	4.27	6.18	148.26
15	0	4.57	4.80	115.08

Total Rock Core

Average	Minimum	Maximum	
110	86	148	MPa

Run #	Average
1	106.17
2	113.79

Depth feet	Depth Inches	m	Is50	UCS (MPa)
92-6				
11	4	3.45	3.11	74.65
12	4	3.76	2.94	70.50
13	4	4.06	3.54	85.02
14	4	4.37	3.76	90.20
15	4	4.67	5.44	130.63
16	4	4.98	5.10	122.34
17	4	5.28	6.78	162.77
18	4	5.59	3.50	83.98
19	4	5.89	5.10	122.34
20	4	6.20	4.28	102.64

Total Rock Core

Average	Minimum	Maximum	
105	71	163	MPa

Run #	Average
1	90.20
2	118.81

Depth feet	Depth Inches	m	Is50	UCS (MPa)
92-9				
8	11	2.72	4.41	105.75
10	2	3.10	4.06	97.46
11	0	3.35	3.80	91.24
11	10	3.61	4.49	107.82
13	0	3.96	3.37	80.87
14	3	4.34	2.59	62.21
15	2	4.62	5.75	137.89
16	0	4.88	4.80	115.08
17	0	5.18	3.28	78.80
18	0	5.49	2.16	51.84

Total Rock Core

Average	Minimum	Maximum	
93	52	138	MPa

Run #	Average
1	101.60
2	96.01
3	81.91

Depth				UCS
feet	Inches	m	Is50	(MPa)
92-13				
9	6	2.90	2.64	63.24
10	6	3.20	3.84	92.27
11	2	3.40	3.37	80.87
12	2	3.71	3.15	75.68
13	2	4.01	4.88	117.16
14	2	4.32	4.19	100.57
15	2	4.62	4.15	99.53
16	2	4.93	5.18	124.41
17	2	5.23	4.36	104.71
18	2	5.54	5.01	120.27

Total Rock Core

Average	Minimum	Maximum	
98	63	124	MPa

Run #	Average
1	77.76
2	94.76
3	116.46

Appendix C

Previous Information

RECORD OF BOREHOLE No 5-A

METRIC

PROJECT 991-1193			
W.P. 335-98-00	LOCATION N 5063361.34; E 310051.51	ORIGINATED BY SB
DIST 54 HWY 11	BOREHOLE TYPE 108mm I.D. HOLLOW STEM AUGERS	COMPILED BY DKB
DATUM GEODETIC	DATE March 1/00	CHECKED BY ASP

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE
330.46	GROUND SURFACE				
0.00	Sandy Silt with organics, Loose, Blackish brown, Moist	1	SS	8	330
329.77 / 0.69	Sand and Gravel, trace silt, Very dense, Brown, Wet	2	SS	50/.10	
329.24 / 1.22	Slightly weathered to fresh, grey-white with black streaks, moderately to widely jointed, lightly foliated (30°), coarse-grained, strong BIOTITE GNEISS. Bedrock cored from 1.22m to 4.28m depth. For bedrock coring details refer to Record of Drillhole 5-A				329, 328, 327
326.18 / 4.28	END OF HOLE				

Plot headings: DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED (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.

Note:
1. Water level measured in piezometer at 1.0m depth (El.329.5m) upon completion of installation.
2. Water level measured in piezometer at 0.8m depth (El. 329.7m) on March 8 and 26, 2000.

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

PROJECT: 991-1193

RECORD OF DRILLHOLE: 5-A

LOCATION: N 5063343.50; E 310034.55

INCLINATION: -90° AZIMUTH: —

DRILLING DATE: Mar.1/00

DRILL RIG: CME 55 Bombardier

DRILLING CONTRACTOR: Marathon

DATUM: Geodetic

FR-FRACTURE, CL-CLEAVAGE, SH-SHEAR, VN-VEIN, F-FAULT, J-JOINT, P-POLISHED, S-SLICKENSIDED, SM-SMOOTH, R-ROUGH, ST-STEPPED, PL-PLANAR, FL-FLEXURED, UE-UNEVEN, W-WAVY, C-CURVED, BC-BROKEN CORE, MB-MECH. BREAK, B-BEDDING

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	PENETRATION RATE (m/min)	COLOUR % RETURN / FLUSH	RECOVERY TOTAL CORE %	RECOVERY SOLID CORE %	R.Q.D. %	FRACT. INDEX PER 0.3	DISCONTINUITY DATA DIP w.r.t. CORE AXIS	DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION	HYDRAULIC CONDUCTIVITY k, cm/sec	DIAMETRAL POINT LOAD INDEX (MPa)	NOTES WATER LEVELS INSTRUMENTATION
		GROUND SURFACE		328.58												
1	NQ RC	Slightly weathered, grey-white with black streaks, moderately to widely jointed, lightly foliated (30°), coarse-grained, strong BIOTITE GNEISS.		1.22	1											
2–3					2								J,R,P J,R,P J,R,P (x2)			
				325.52 4.28												
		END OF HOLE														

DEPTH SCALE 1 : 50

Golder Associates

LOGGED: SB

CHECKED: PD

Appendix D

Foundation Comparison

THURBER

COMPARISON OF FOUNDATION ALTERNATIVES FOR EACH FOUNDATION ELEMENT

Foundation Element	Footings on Bedrock	Caissons
East Abutment **And** **West Abutment**	***Advantages*** i. High geotechnical resistance available ii. Conventional construction practices. ***Disadvantages*** i. Uneven or sloping bedrock surface may be encountered. ii. Difficulties with excavation control and unwatering. iii. May require mass concrete fill.	***Advantages:*** i. High bearing resistances available using socket into bedrock. ii. May be a method of controlling excavation and unwatering problems at this site. ***Disadvantages*** i. May be impossible to seal liner into bedrock, requiring placement of concrete by tremie methods. ii. More costly than conventional footing construction in normal situations.

Appendix E

Slope Stability Output

	Gamma kN/m3	C kPa	Phi deg	Piezo Surf.
Rock Fill	20	0	45	1
Overburden	21	0	30	1
Bedrock	24	10000	50	1

Thurber Engineering Ltd. - Toronto
19-1423-12
Bernard Creek on Robins Road
April 7, 2005
Rock Fill, West Approach

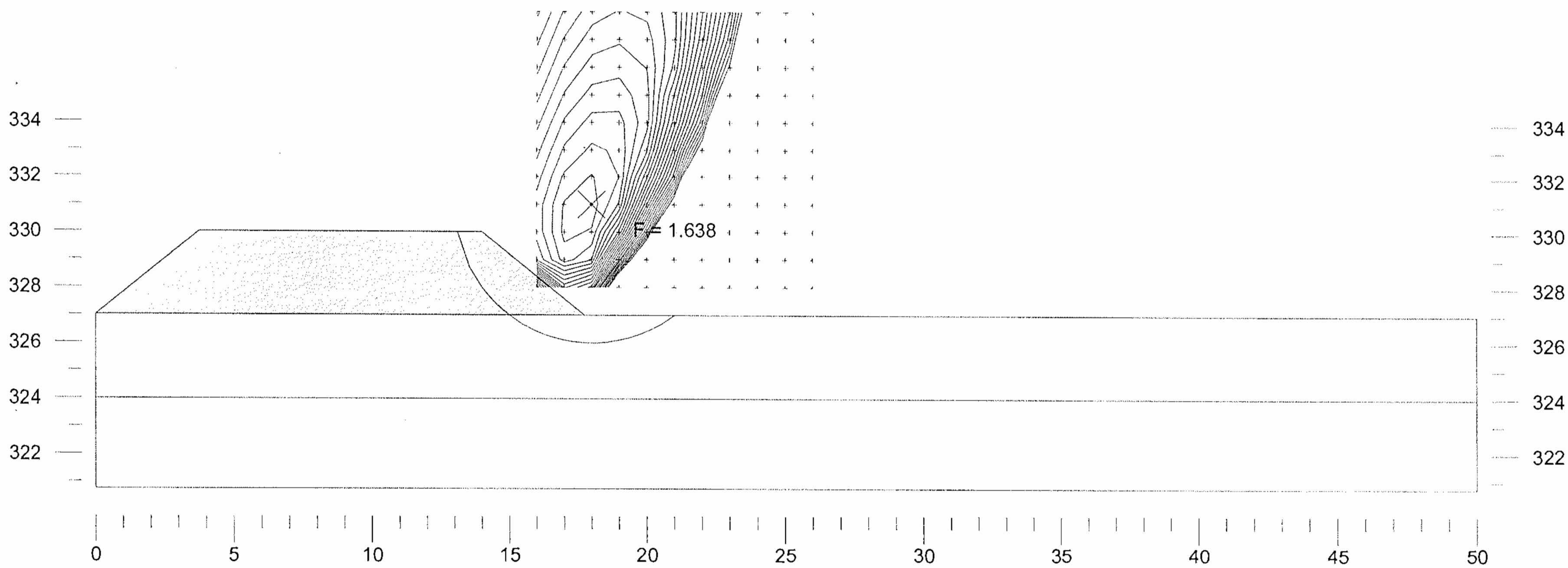


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Figure E1

	Gamma kN/m3	C kPa	Phi deg	Piezo Surf.
Rock Fill	20	0	45	1
Overburden	21	0	30	1
Bedrock	24	10000	50	1

Seismic coefficient = 0.11

Thurber Engineering Ltd. - Toronto
19-1423-12
Bernard Creek on Robins Road
April 7, 2005
Rock Fill, West Approach
Seismic 0.11

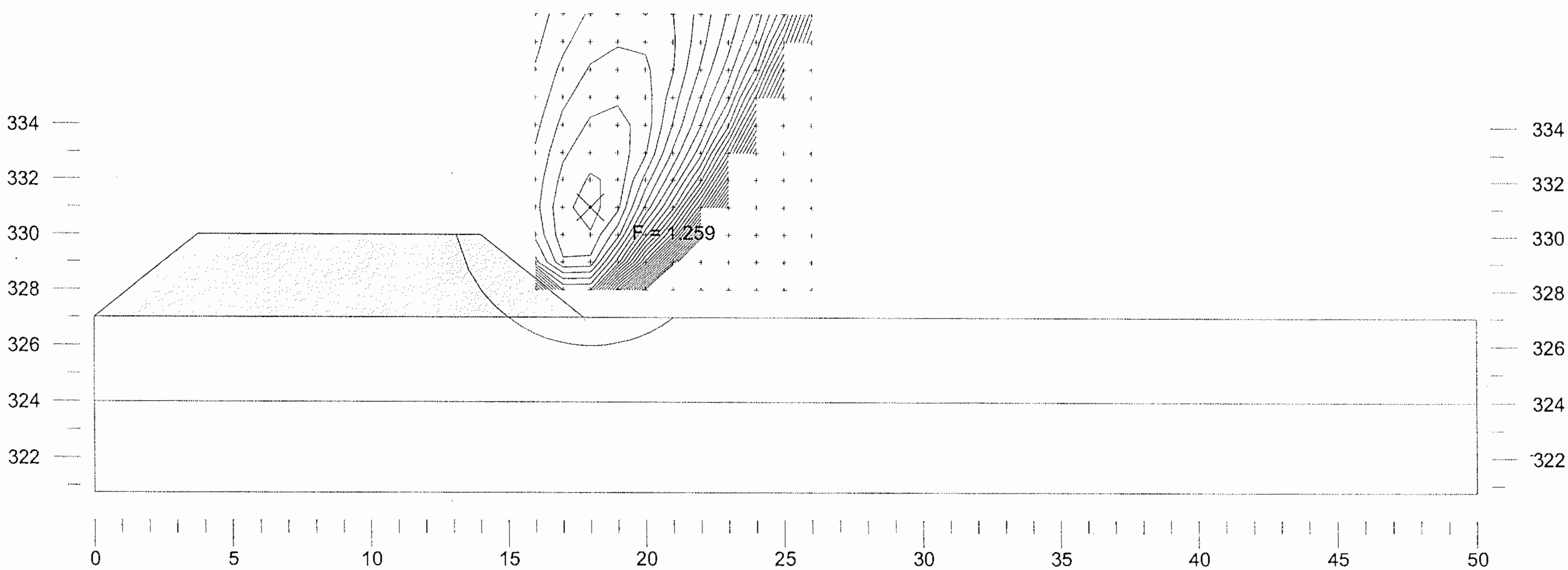

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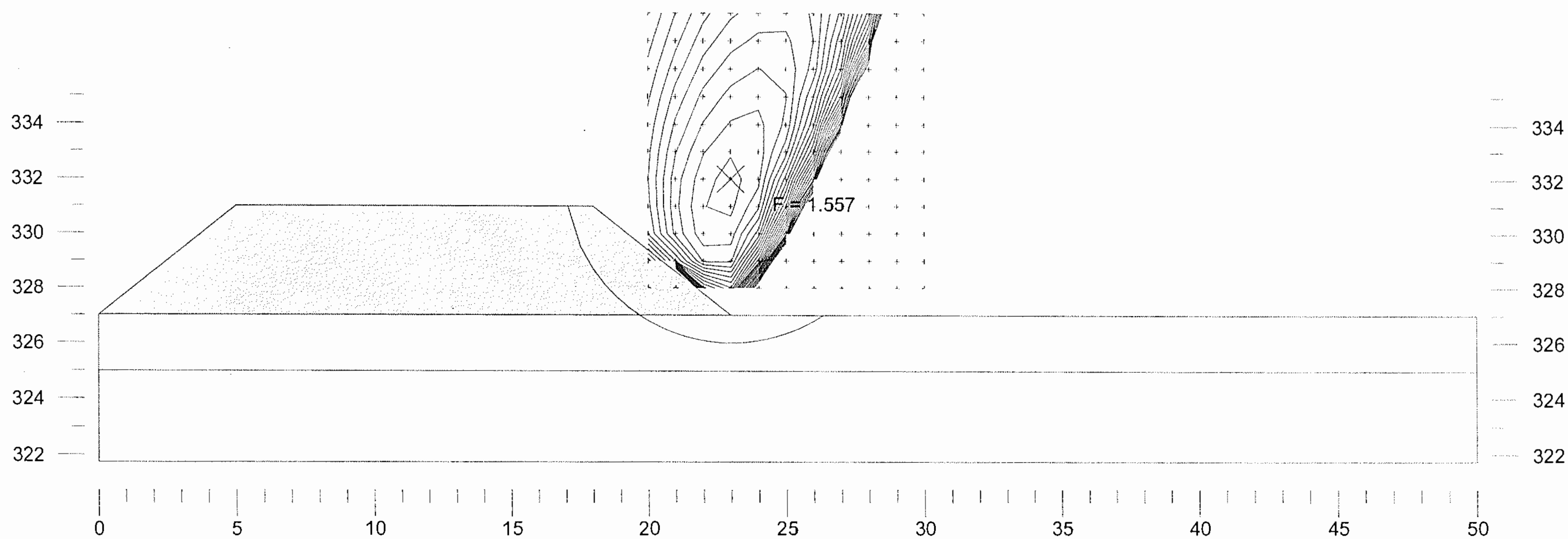
Figure E2

	Gamma kN/m3	C kPa	Phi deg	Piezo Surf.
Rock Fill	20	0	45	1
Overburden	21	0	30	1
Bedrock	24	10000	50	1

Thurber Engineering Ltd. - Toronto
19-1423-12
Bernard Creek on Robins Road
April 7, 2005
Rock Fill, East Approach



4/27/2005 10:49:23 AM C:\PROJ\19\1423\12HWY1~1\BRIDGES\92BERN~1\EARF01.GSL Thurber Engineering Ltd. - Toronto F = 1.557

Figure E3

	Gamma kN/m3	C kPa	Phi deg	Piezo Surf.
Rock Fill	20	0	45	1
Overburden	21	0	30	1
Bedrock	24	10000	50	1

Seismic coefficient = 0.11

Thurber Engineering Ltd. - Toronto
19-1423-12
Bernard Creek on Robins Road
April 7, 2005
Rock Fill, East Approach
Seismic 0.11

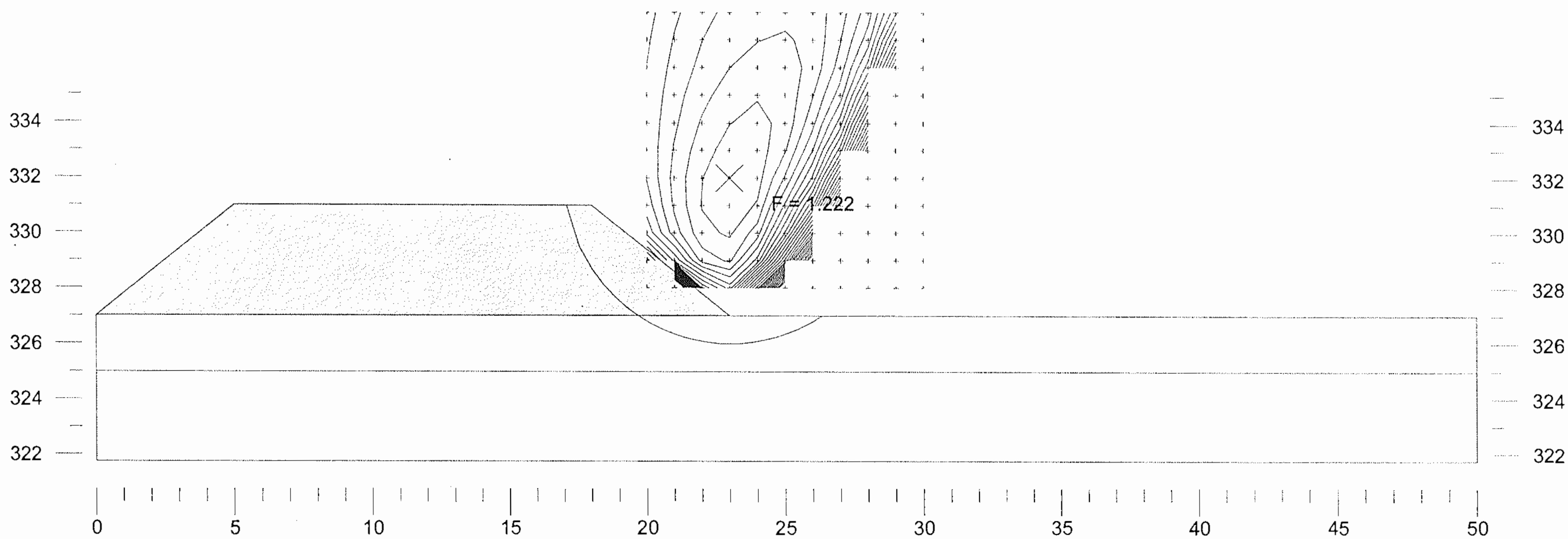


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Figure E4

Appendix F

Special Provisions

The following Special Provisions are referenced in this report:

110F13

105S10

Amendment to OPSS 206, December 1993

902S01

903S01

Appendix G

Drawings

METRIC

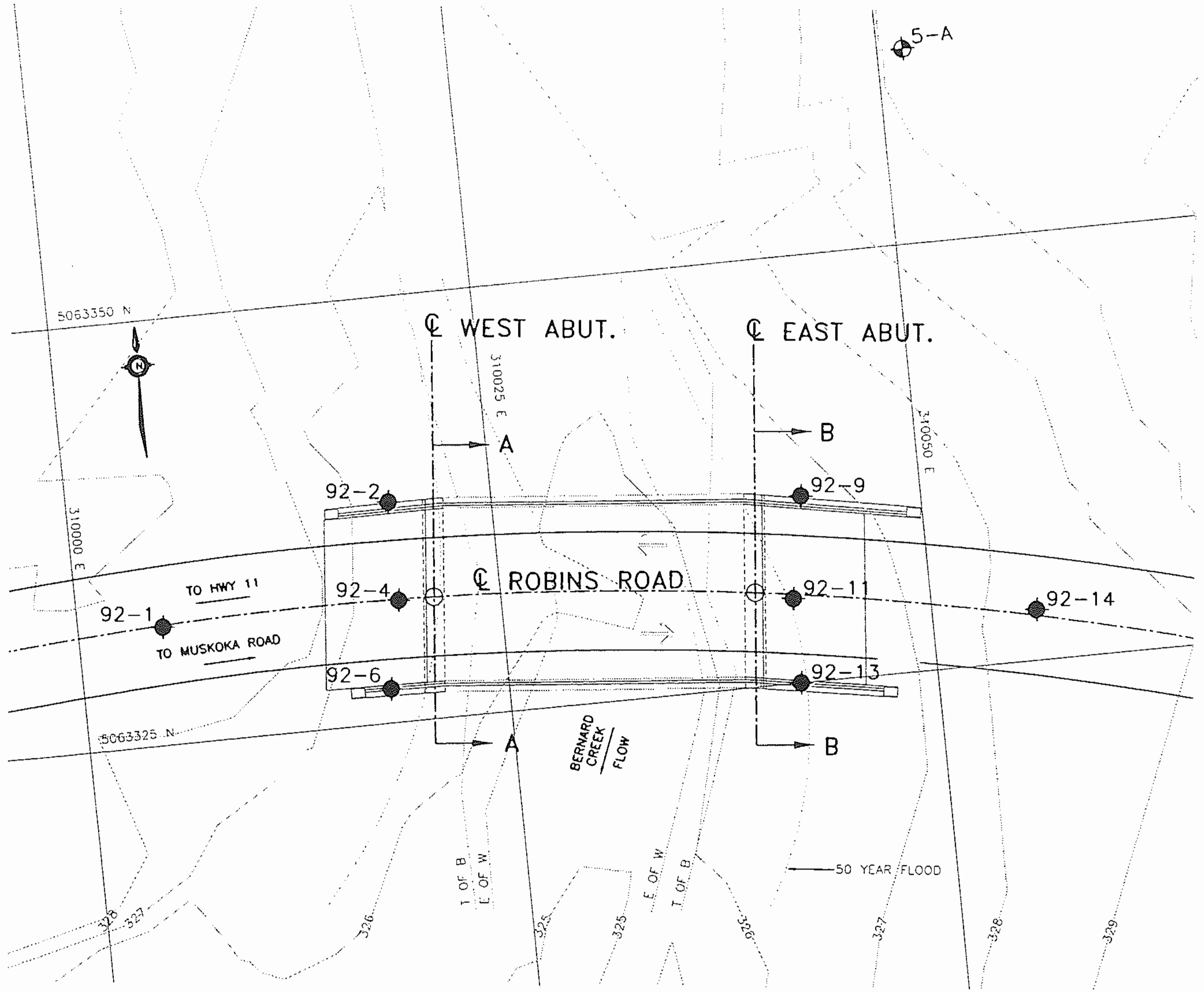
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN

HWY 11
CONT No
WP No 5042-03-01

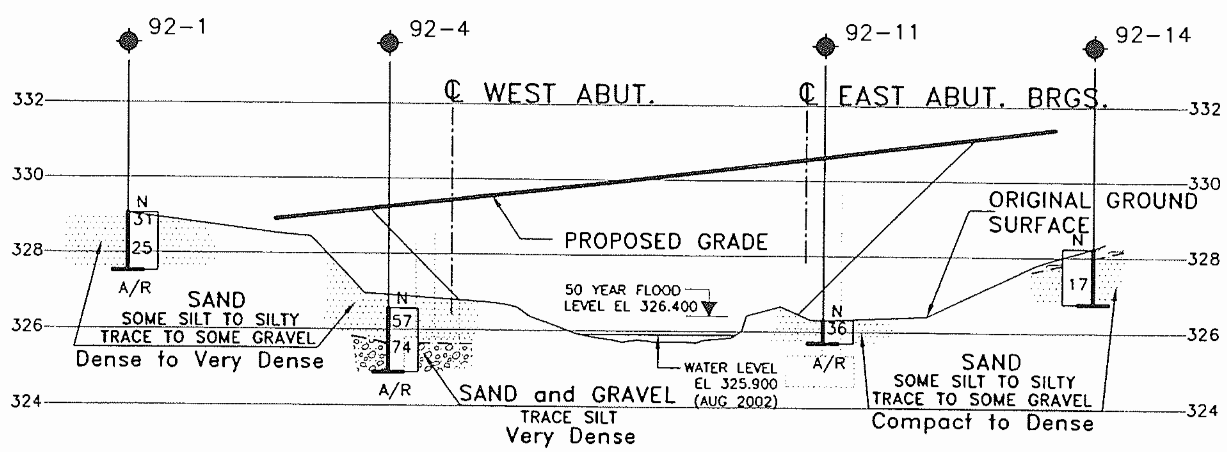
BERNARD CREEK BRIDGE ON ROBINS ROAD
BORE HOLE LOCATION & SOIL STRATA

SHEET

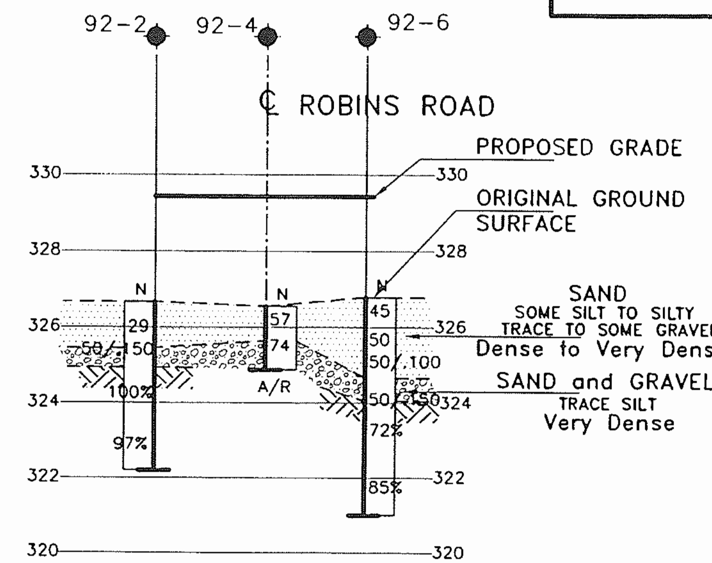
Marshall Macklin Monaghan
PROJECT MANAGERS • ENGINEERS • SURVEYORS • PLANNERS



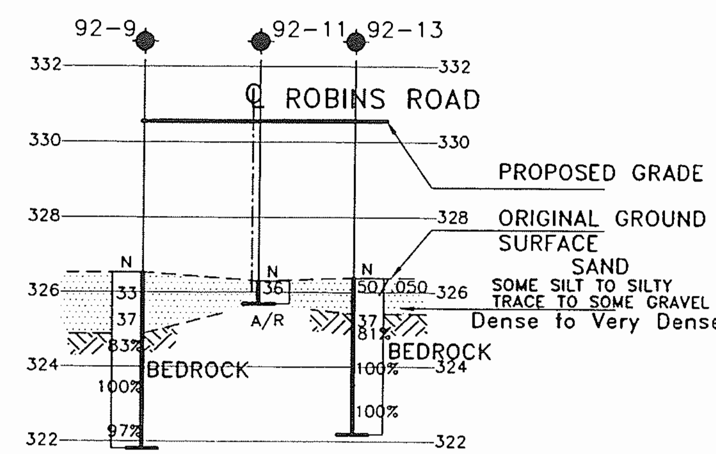
PLAN



PROFILE ℄ ROBINS ROAD



SECTION A–A



SECTION B–B

KEYPLAN

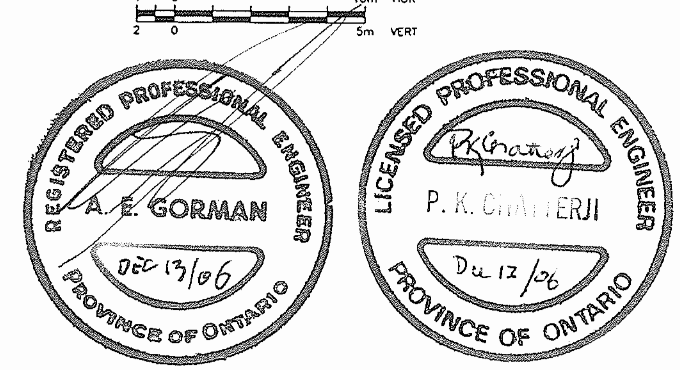
LEGEND

●	BoreHole by THURBER
⊕	BoreHole by GOLDER
N	Blows/ 0.3m (Std Pen Test, 475 J/blow)
CONE	Blows/ 0.3m (60° Cone, 475 J/blow)
PH	Pressure, Hydraulic
▽	WL at Time of Investigation
	Head Artesian Water
	Piezometer
90%	Rock Quality Designation (RQD)
A/R	Auger Refusal

NO	ELEVATION	NORTHING	EASTING
92-1	329.0	5063332.0	310005.0
92-2	326.7	5063338.0	310019.0
92-4	326.5	5063332.2	310019.0
92-6	326.8	5063327.0	310018.1
92-9	326.5	5063336.0	310043.1
92-11	326.3	5063330.0	310042.1
92-13	326.4	5063325.0	310042.0
92-14	328.2	5063328.0	310056.1
5-A	330.5	5063361.3	310051.5

– NOTE –

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



BENCHMARK
N&W IN W ROOT 0.2 TAMARACK
236.0 RT OF 14+235.5
BM EL : 316.961

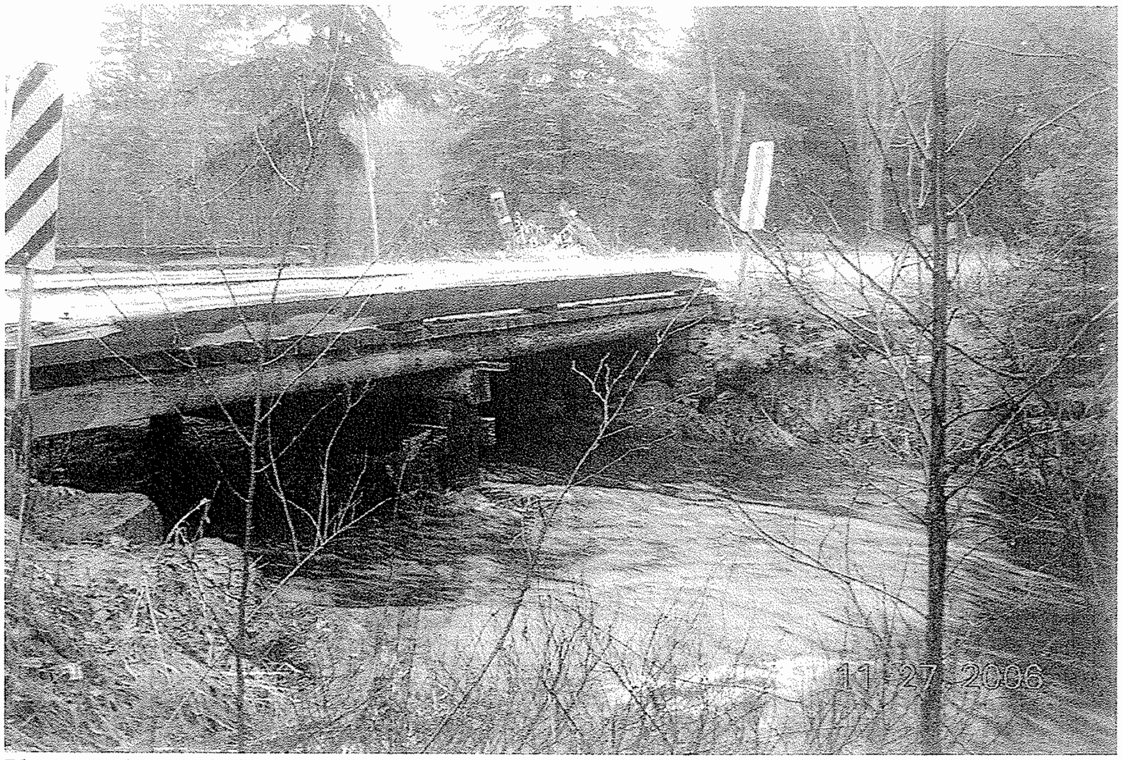
DRAWING NOT TO BE SCALED
100 mm ON ORIGINAL DRAWING

REVISIONS | DATE | BY | DESCRIPTION

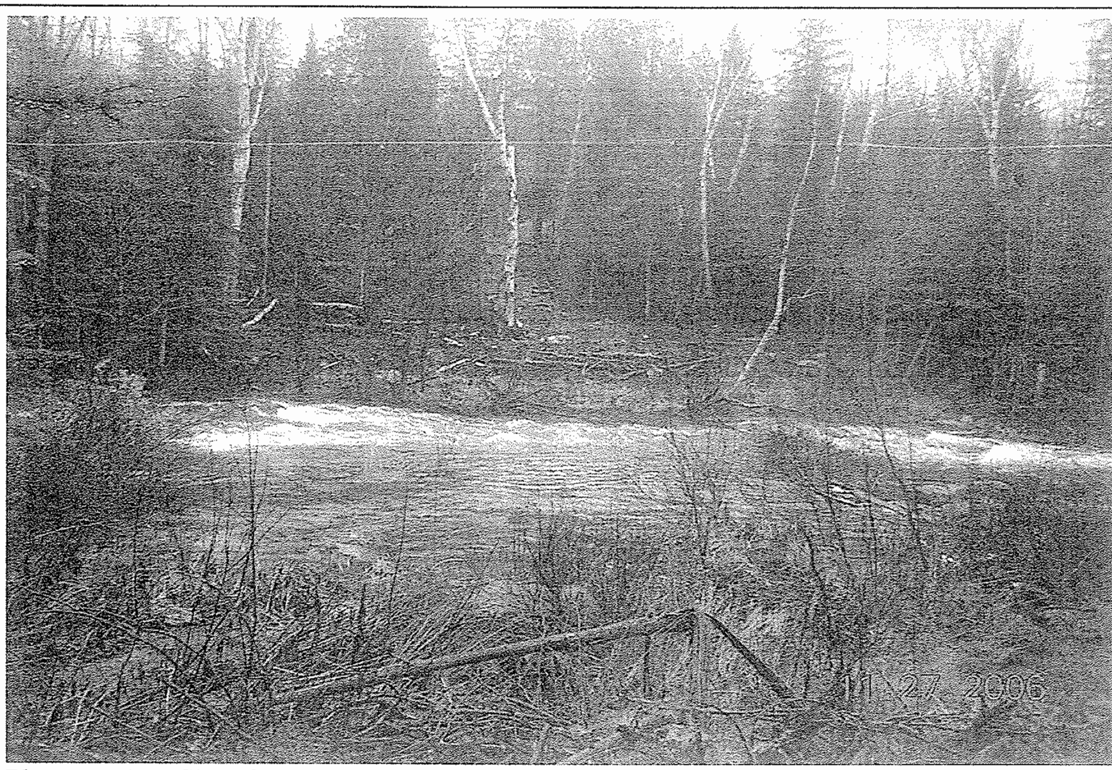
DESIGN AEG | CHK | CODE CHBDC 2000 | LOAD CL-625-ONT | DATE APR. 2005
DRAWN HS | CHK AEG | SITE 44-92 | STRUCT. | SCHEME. | DWG

Appendix H

Site Photographs



Photograph 1, Existing Robins Road over Bernard Creek Structure



Photograph 2, Site of Proposed Structure