FOUNDATION INVESTIGATION AND DESIGN REPORT
MURDOCK RIVER BRIDGE REPLACEMENT
HIGHWAY 607, TOWNSHIP OF BIGWOOD, SUDBURY AREA
W.P. 33-78-01, SITE: 46-208

Geocres Number: 41A-266

Report to

MMM Group

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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 proposed replacement bridge that will carry a re-aligned Highway 607 over Murdock River in the Township of Bigwood, Ontario. The existing bridge is located to the west of the proposed alignment and consists of a single-span steel bailey bridge that carries a single lane of traffic.

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 investigation.

Thurber carried out the investigation as a sub-consultant to MMM Group, under the Ministry of Transportation Ontario (MTO) Agreement Number 5008-E-0013.

2 SITE DESCRIPTION

The site is located approximately 5 km south of the intersection of Highway 64 and Highway 607 near Alban, Ontario. At the site, Murdock River flows on a relatively gentle gradient towards the east in a channel that is approximately 15 m wide at the existing bridge and 35 m wide at the location of the proposed bridge. The water level in the river was recorded as Elevation 182.50 in May 2009.

The south river bank consists mainly of bedrock outcrops and some small swamp areas vegetated with shrubs and trees. The north river bank is vegetated with shrubs and trees and there is a pile of rock fill east of the proposed alignment on the north river bank.

Geologically, the site lies within the Canadian Shield, which is characterized by Pre-Cambrian bedrock. Locally, however, Murdock River flows across deposits of sand and silt overlying the bedrock. There are several private residences set back from the south bank of the river, both east and west of the proposed alignment.

Photographs of the site are included in Appendix D and show the existing bridge, the approaches and the site terrain.

3 SITE INVESTIGATION AND FIELD TESTING

The site investigation and field-testing for this project was carried out on March 8 and from May 31 to June 3, 2010 and consisted of drilling ten boreholes identified as MR10-03 to MR10-8 and MR10-11 to MR10-14. Proposed Boreholes MR10-01 and MR10-02 (located at the south abutment) could not be drilled due to cobbles and boulders at the surface. However, bedrock was observed at surface approximately 2.5 m northeast of the proposed location of MR10-02. Proposed Boreholes MR10-09 and MR10-10 (located at the north abutment) could not be drilled due to the proximity of overhead power lines. At the location of Borehole MR10-13 (located along the south approach) bedrock is at surface.

Four boreholes were drilled at the approximate locations of each of the proposed north and south abutments and one borehole was drilled along each of the south approach (MR10-13) and north approach (MR10-14). The depths of the boreholes ranged from 0.7 m to 14.8 m, with the deeper boreholes located at the north abutment due to thicker overburden deposits. The Record of Borehole sheets for these boreholes are included in Appendix A. The approximate locations of the boreholes, are shown on the Borehole Locations and Soil Strata Drawing included in Appendix E.

Prior to commencing the site investigation, clearance was obtained from utility companies having plant in the area.

A combination of hollow-stem auger drilling and NQ-sized 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.

At least 3 m of bedrock was cored in Boreholes MR10-04 to MR10-08, MR10-11, and MR10-12. The rock cores were logged and total core recovery, solid core recovery and Rock Quality Designation (RQD) was determined for each core.

A standpipe piezometer consisting of 19 mm PVC pipe with a slotted screen was installed in Borehole MR10-12 and enclosed in filter sand to permit groundwater level monitoring. The locations and completion details of the piezometer and all other boreholes are shown in Table 3.1.

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

Table 3.1 – Borehole Completion Details

Borehole Location	Borehole ID	Piezometer Tip Depth/ Elevation (m)	Completion Details
South Approach	MR10-13	None Installed	Borehole not drilled. Bedrock at surface.
South Abutment	MR10-03	None Installed	Borehole caved to surface.
	MR10-04	None Installed	Borehole backfilled with bentonite to surface.
	MR10-05	None Installed	Borehole backfilled with bentonite to surface.
	MR10-06	None Installed	Borehole backfilled with bentonite to surface.
North Abutment	MR10-07	None Installed	Borehole backfilled with bentonite to 1.8 m, then cuttings to surface.
	MR10-08	None Installed	Borehole backfilled with bentonite to 1.8 m, then cuttings to surface.
	MR10-11	None Installed	Borehole backfilled with bentonite to 2.3 m, then cuttings to surface.
	MR10-12	11.6 / 168.9	Piezometer with 1.5 m slotted screen installed with sand filter to 9.3 m, bentonite seal from 9.3 m to 0.2 m, then cuttings to surface.
North Approach	MR10-14	None Installed	Borehole caved to 1.2 m, then backfilled with bentonite to surface.

4 LABORATORY TESTING

All of the recovered soil samples were subjected to Visual Identification (VI) and to natural moisture content determination in the laboratory. Selected samples were also subjected to gradation analysis (hydrometer and sieve), the results of which are summarized on the Record of Borehole Sheets in Appendix A. Grain size distribution curves for these samples are presented on the figures included in Appendix B.

Point load tests were carried out in the laboratory on selected samples of intact bedrock to assist in evaluation of the compressive strength of the bedrock. The results of the point load tests are tabulated in Table 1 in Appendix B and on the Record of Borehole sheets in Appendix A.

5 DESCRIPTION OF SUBSURFACE CONDITIONS

A detailed description of the soil stratigraphy encountered at each borehole location is presented in Appendix A and on the "Borehole Locations and Soil Strata" drawing in Appendix E. An overall description of the stratigraphy is given in the following paragraphs. However, the factual data presented in the Record of Borehole Sheets (Appendix A) governs any interpretation of the site conditions.

In general, the site is underlain by organic material overlying cohesionless deposits of sand and silt and silty sand till overlying granite bedrock.

5.1 Organic Material

Fibrous, peaty, organic material mixed with sand, gravel and roots was encountered surficially in Boreholes MR10-03 to MR10-06, MR10-11 and MR10-12 and was encountered below a surficial layer of fill in Borehole MR10-08. The thickness of the organic material ranged from 200 to 600 mm (underside elevation 180.3 to 180.9 m). Natural moisture contents of the organic material samples ranged from 8 to 254%. The thickness of organic material may vary between and beyond the boreholes.

SPT N-values recorded in the organic material generally ranged from 6 to 10 blows per 0.3 m penetration, indicating a loose condition. An SPT N-value of 53 blows for 0.225 m of penetration was recoded in the organic material overlying bedrock in Borehole MR10-04.

5.2 Sand Fill

Sand fill was encountered surficially in Boreholes MR10-07 and MR10-08. The sand fill contained some gravel and occasional cobbles and boulders and was brown and moist. The thickness of the granular fill ranged from 0.6 m in Borehole MR10-08 to 0.7 m in Borehole MR10-07 (underside elevation 180.6 to 180.9 m).

5.3 Sand to Sand and Silt

A layer of wet water bearing sand to sand and silt was encountered below the organic material in Boreholes MR10-03, MR10-05, MR10-08, MR10-11, and MR10-12, below the sand fill in Borehole MR10-07, and below the ice and water in Borehole MR10-14. The sand and silt also contains trace clay, trace gravel, and occasional cobbles and is brown to grey. This layer encountered in Boreholes MR10-03 and MR10-05 (located at the south abutment) was 100 to 200 mm thick (underside elevations 180.6 and 180.7 m, respectively), overlying bedrock. The sand and silt layer encountered in Boreholes MR10-07, MR10-08, MR10-11, MR10-12, and MR10-14 (located at the north abutment and north approach) was 5.2 to 10.9 m thick (underside elevations 169.4 to 175.8 m).

A 0.6 m thick layer of gravel and cobbles was encountered within the sand to sand and silt layer in Borehole MR10-14 at 0.6 m depth (elevation 180.7 m).

SPT N-values recorded in the sand and silt layer generally ranged from 1 to 44 blows per 0.3 m of penetration, indicating a very loose to dense relative density. An SPT N-value of 100 blows for 0.08 m of penetration was recorded in Borehole MR10-03 at 0.6 m, just above bedrock. Typically, the sand and silt layer had a very loose to compact relative density. Natural moisture contents of samples collected from the sand and silt layer ranged from 10 to 36%.

Selected samples from the sand to sand and silt layer were subjected to gradation analysis, the results of which are summarized below.

Soil Particles	Percentage
Gravel	0 to 8
Sand	33 to 91
Silt	21 to 65
Clay	2
Silt and Clay	2 to 14

The grain size distribution curves for these samples are presented in Figures B1 and B2 of Appendix B and the results are summarized on the corresponding Record of Borehole sheet in Appendix A.

5.4 Silty Sand Till

In Boreholes MR10-07, MR10-11, MR10-12, and MR10-14 a layer of silty sand till was encountered below the sand and silt layer. The silty sand till contained trace to some gravel and trace clay and was grey. Although not recovered in the SPT samples, glacial tills inherently contain cobbles and boulders. The thickness of the silty sand till layer ranged from 0.6 to 1. 5 m (underside elevation 168.8 to 174.5 m).

SPT N-values recorded in the silty sand till ranged from 48 blows for 0.3 m of penetration to 100 blows for 0.275 m penetration, indicating a dense to very dense relative density. Natural moisture contents of samples of the silty sand till ranged from 11 to 13%.

Selected silty sand till samples were subjected to gradation analysis, the results of which are summarized below. The grain size distribution curves for these samples are presented in Figure B3 of Appendix B and the results are summarized on the appropriate Record of Borehole sheet in Appendix A.

Soil Particles	Percentage
Gravel	3 to 11
Sand	56 to 82
Silt	29
Clay	4
Silt and Clay	15

5.5 Bedrock

The overburden soils described above are underlain by granite bedrock. The bedrock was generally grey with occasional pink and white bands visible in most cores. Occasional mechanical breaks and sub-vertical fractures were observed in the rock cores.

Bedrock was encountered at various depths and was proved by coring in a number of boreholes. Table 5.1 summarizes the depths and elevations to the top of bedrock in the boreholes. Where coring was not carried out, bedrock was inferred from auger refusal.

Table 5.1 – Depths and Elevations of Top of Bedrock

Borehole	Location	Top of Bedrock	
		Depth (m)	Elevation (m)
MR10-03	South Abutment	0.7*	180.6
MR10-04		0.4	180.8
MR10-05		0.5	180.7
MR10-06		0.3	180.8
MR10-07	North Abutment	8.8	172.5
MR10-08		10.4	171.1
MR10-11		10.0	Not surveyed
MR10-12		11.7	168.8
MR10-13	South Approach	0.0	Not surveyed
MR10-14	North Approach	6.8*	174.5

* Auger refusal on probable bedrock.

Core recovery in the bedrock generally ranged from 85% to 100%. The RQD values generally ranged from 65% to 100%, indicating fair to excellent rock quality. RQD values of 0% were recorded in Borehole MR10-05 Run 2 and MR10-12 Run 1. The Fracture Index (FI) of the rock, expressed as fractures per 0.3 m of core, generally ranged from 0 to 5. In some bedrock cores the Fracture Index ranged from 5 to 10.

The estimated unconfined compressive strength of the rock cores generally ranges from 76 MPa to 287 MPa, indicating a strong to extremely strong rock. These estimated rock strength values are interpreted from 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 1 immediately following the text of this report.

5.6 Groundwater Conditions

A 19 mm standpipe piezometer was installed in Borehole MR10-12, located at the north abutment. A water level of 0.13 m below ground surface (elevation 180.4 m) was measured on June 3, 2010. This is a short-term water level reading only and the water table will fluctuate seasonally. The river level was at Elevation 182.5 in May 2009.

6 MISCELLANEOUS

George Downing Estate Drilling Ltd. of Hawkesbury, Ontario supplied a track mounted CME 55 drill rig and conducted the drilling, sampling and in-situ testing operations for Boreholes MR10-03 to MR10-08, MR10-11, and MR10-12. OGS Inc. of Almonte, Ontario supplied portable drilling equipment and conducted the drilling, sampling and in-situ testing operations for Borehole MR10-14.

The drilling and sampling operations in the field were supervised on a full time basis by Mr. Stephane Loranger and Mr. Jason Mei of Thurber, under the direction of Mr. Tony Harte, M.Sc..

The coordinates for the boreholes and the ground surface elevations were provided by MMM Group Limited.

Mrs. Lindsey Blaine, E.I.T. and Mr. Alastair E. Gorman, P.Eng prepared the Foundation Investigation 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 preliminary 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 607 will cross the Murdock River on a single span structure that will lie on a new alignment immediately east of the existing Bailey bridge.

Based on the preliminary General Arrangement (GA) drawing provided by MMM Group Limited, a structure is proposed that will have span will have a single 27 m long span. The proposed finished grade at the structure will be about Elevation 187.5 m at the south abutment and the original ground surface is near Elevation 181.2 m, resulting in an approach embankment of 6.3 m high. At the north abutment, the finished grade will be at Elevation 186.3 m and the original ground surface is near Elevation 180.8 m, resulting in an approach embankment up to 5.5 m high.

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 structure is a one-span bridge with two abutments.

The stratigraphy encountered is presented on the Record of Borehole sheets and on the Borehole Locations and Soil Strata drawing in Appendix A and E, respectively. In general terms, within the footprint of the south abutment, the ground condition consists of shallow organic soil overlying bedrock. At the north abutment, bedrock is overlain by 8.8 to 11.7 m of very loose to dense sand and very dense silty sand till. The groundwater level is high and lies near ground surface at Elevation 180.3 m as recorded in borehole MH10-12 and as a result most of the overburden sand is below the groundwater level.

Initial consideration was given to the following foundation types:

- Spread footings bearing on native soil or bedrock
- Spread footings on engineered fill
- Driven Piles

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

8.1 Spread Footings on Native Soil or Bedrock

Spread footings bearing on native soil or bedrock generally are the least expensive form of construction.

8.1.1 South Abutment

At the south abutment, the near surface soils consist of 300 to 700 mm of organic material overlying bedrock. Footings should not be founded on organic material and footings should bear on the underlying bedrock layer.

Spread footings founded on the undisturbed bedrock at or below elevations given in Table 8.1 may be designed for the following geotechnical resistance:

- Factored geotechnical resistance of 5,000 kPa at Ultimate Limit States (ULS)
- The SLS condition will not govern design for footings founded on bedrock

Table 8.1 - Depth and Elevation of Top of Bedrock

Foundation Unit	Borehole	Top of Bedrock	
		Depth (m)	Elevation (m)
South Abutment	MR10-03	0.7	180.6
	MR10-04	0.4*	180.8
	MR10-05	0.5*	180.7
	MR10-06	0.3*	180.8

* Bedrock proved by coring

The geotechnical resistances quoted above are for concentric, vertical loads only. In the case of eccentric or inclined loading, the geotechnical resistance must be calculated as illustrated in the CHBDC Clause 6.7.3 and Clause 6.7.4.

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.

The bearing surface should be prepared by removing all loose/disturbed material and shattered/loosened rock fragments. The base of the foundation should be inspected by a geotechnical engineer to confirm that the exposed surface conforms to the design requirements and has been adequately prepared to receive concrete. Areas where the final bedrock surface lies beneath the underside of footing should be backfilled with the same class of concrete as used in the footing.

8.1.2 North Abutment

At the north abutment, the near surface soils consist of 6.6 to 10.9 m of very loose to dense sand. As a result, spread footings on native soil are not recommended for the following reasons:

1. The near surface soils are cohesionless and the relative density was found to be highly variable, being very loose in some boreholes. Accordingly, the geotechnical resistance is low and uneven settlement is also anticipated.
2. Shallow foundations placed near the edge of the river would be at risk from erosion during the design life of the structure.
3. Founding spread footings at sufficient depth to resist erosion would require excavation in permeable, cohesionless soils below the water table. Such an excavation would require extensive dewatering and yet would remain at risk of becoming destabilized due to the inflow of unbalanced groundwater heads.

8.2 Spread Footings on Engineered Fill

The available geotechnical resistance could be improved by founding the footing on a pad of Granular "A" engineered fill. If an engineered fill pad is used at this site, all organics, fill or other deleterious materials must be stripped from the footprint of the engineered fill to expose competent material. Typically, spread footings on pads of engineered granular fill may be designed for the following geotechnical resistances:

- Factored geotechnical resistance of 900 kPa at Ultimate Limit States (ULS)
- Geotechnical resistance of 350 kPa at Serviceability Limit States (SLS)

However, the use of spread footings on engineered fill at the south abutment is not recommended due to the close proximity of the bedrock to the original ground surface. In addition, a spread footing on engineered fill at the north abutment is also not recommended for the following reasons:

1. The engineered fill pad could be subject to erosion or undermining during high river flows.
2. Founding the engineered fill on suitably dense, uniform subgrade soils may require excavation in permeable, cohesionless soils below the water table. Such an excavation would require extensive dewatering and yet would remain at risk of becoming destabilized due to the inflow of unbalanced groundwater heads.

8.3 Driven Piles

The high bedrock elevation at the south abutment is considered unsuitable for support of foundations on driven steel piles.

The soil stratigraphy encountered at the north abutment is considered to be suitable for the design of foundations supported on steel H-piles driven to bedrock.

8.3.1 Axial Resistance

The following, factored ULS resistances may be used for the design of piles driven to bedrock:

Table 8.2 - Axial Resistance of Pile Founded on Bedrock

Pile Section	Resistance (kN) ULS (Factored)
HP 310 X 110	2,000
HP 310 X 152	2,700

These resistances are based on the limits placed on the axial resistance of an HP 310 X 110 pile by the Bridge Office Memo dated April 15, 1998.

The geotechnical resistance would exceed the structural resistance and will not govern at this site.

8.3.2 Pile Tips

Since the piles will be driven to bedrock, they must be fitted with cast steel H-section rock points from an approved manufacturer such as Titus Steel (Standard H-points) or approved equivalent.

8.3.3 Pile Installation

Pile installation should be in accordance with OPSS 903.

8.3.4 Pile Driving

The appropriate pile driving note is “Piles to be driven to bedrock”.

An NSSP should require the QVE to terminate driving before the pile is damaged by overdriving.

To facilitate pile installation, embankment fill through which piles will be driven must not contain oversize material, i.e. no particles exceeding 75 mm in size.

8.3.5 Downdrag

The soils at this site are non-cohesive and settlements induced in the foundation soils by construction of the approach fills will be substantially complete as construction of the embankment is completed and downdrag on the piles is not considered to be an issue at this site.

8.4 Abutment Considerations

The ground conditions are suitable for the design and construction of conventional or semi-integral abutments.

8.4.1 South Abutment

At the south abutment, it is anticipated that the distance from the finished grade to the top of bedrock will not be sufficient to accommodate the height of the abutment stem and the required length of pile to provide flexibility without excavating bedrock. Accordingly, integral abutment is not recommended at this location.

8.4.2 North Abutment

The ground conditions on the north abutment at this site are considered suitable for an integral abutment design. The use of H-piles at the abutment allows for the design of an integral abutment structure.

The integral abutment design requires that the piles possess flexibility in the upper 3 m of the pile length. The near surface, native soils at this site are very loose to compact and these soils will provide the pile with the required flexibility.

8.5 Lateral Resistance

The lateral resistance of the pile may be calculated using a value for the coefficient of horizontal subgrade reaction (k_s) and ultimate lateral resistance (p_{ult}) as follows:

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

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

where z = depth of embedment of pile in metres
D = pile width in metres
n_h = value from Table 8.3
γ = unit weight (Table 8.3)

K_p = passive earth pressure coefficient (Table 8.3)

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 should not exceed the ultimate lateral resistance.

The spring constant, K, for analysis may be obtained by the expression, $K = k_s*L*D$ (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 on any one segment of pile, P_{ult}, may be obtained from the expression, $P_{ult} = p_{ult}*L*D$. 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 in one pile be limited to no more than 150 kN at ULS and 50 kN at SLS. Parameters for lateral pile resistance are shown in Table 8.3.

Table 8.3 - Parameters for Lateral Pile Resistance

Location	Elevation	n_h (kN/m^3)	K_p	Unit Weight (kN/m^3)	Soil Conditions
South Abutment	Not applicable				
North Abutment	OGS to 175.0	3,000	3.0	11*	Very loose to compact sand and silt
	175.0 to bedrock	5,000	3.2	11*	Compact to very dense sand and silt and sand to silt till

- Submerged unit weight.

Pile interaction should be considered with reference to CHBDC Clause 6.8.9.2.

For lateral soil/pile group interaction analysis, the modulus of subgrade reaction (k_s) may have to be reduced based on the pile spacing.

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

Pile Spacing Perpendicular to Direction of Loading	Horizontal Subgrade Reaction Reduction Factor, R
4 D*	1.00
1 D*	0.50

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

Intermediate values may be obtained by interpolation.

For conventional abutments, the lateral resistance may be provided by battered piles

8.6 Recommended Foundation

From a geotechnical perspective, the recommended foundation consists of:

- South abutment - spread footings on bedrock
- North abutment - steel H-piles driven to bedrock

8.7 Frost cover

The design depth of frost penetration at this site is 1.9 m.

The pile cap at the north abutment must be provided with 1.9 m of earth cover as frost protection.

Frost protection is not an issue for spread footings founded on bedrock.

9 EXCAVATION

All excavation must be carried out in accordance with the Occupational Health and Safety Act (OHSA). For the purposes of the OHSA, the native soils within the probable depth of excavation at this site may be classed as Type 3 soils above the water table and Type 4 soils below the water table. This classification is based on the lack of cohesion in the soils and the resulting possibility that excavation slopes will slough if excavated vertically for the lower 1.2 m. Excavation slopes must not exceed 1H:1V above the groundwater level

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

Excavation below the groundwater level without prior dewatering is not recommended since the inflow of groundwater will cause boiling and sloughing of the soil below the water table making it difficult to maintain a dry, sound base on which to work.

Prior to excavation below the natural groundwater level, the groundwater must be depressed to a level below the deepest excavation level sufficient to maintain a stable base and prevent soil disturbance by construction traffic.

Bidders must be alerted to the fact that excavation must be carried out through cohesionless soils, which may include cobbles and boulders

10 UNWATERING

The piezometer installed in borehole MR10-12 revealed that the groundwater level is near the ground surface. It must be assumed that any excavation at the north abutment will require dewatering and protection of the work area from inundation by the river. The contract documents should provide a high water level in the river against which the Contractor is required to provide protection. Placement of concrete must be done in the dry. Unwatering must remain operational and effective until the foundation is constructed and backfilled.

The design of foundation on the south abutment bearing on bedrock will not be influenced by the groundwater, but the Contractor must make provisions to control the groundwater seepage or inundation from the river and to remove any accumulated water from the footing base prior to placing the concrete in the dry.

The Contract Documents should also contain a NSSP alerting the Contractor to the risks associated with excavation of cohesionless soils submerged below the groundwater level without prior dewatering. Suggested wording is included in Appendix F.

The design of the dewatering system and any system to control inundation by the river, e.g. a cofferdam, that may be required is the responsibility of the Contractor and the Contract Documents must alert him to this responsibility and the need to engage a dewatering specialist. While the responsibility for dewatering remains with the Contractor, suitable systems that might be employed include pumping from filtered sumps for penetration of no more than 0.5 m below the groundwater level and the use of vacuum wellpoints for deeper penetration below the groundwater level. Vacuum wellpoints in conjunction with sheetpiled cofferdam may be required due to the proximity of the river.

11 BRIDGE APPROACHES AND EMBANKMENTS

Approach embankment construction using either earth fill or rock fill is feasible on the foundation soils encountered at this site. The immediate approach fill at the south abutment will be constructed over bedrock therefore settlement and stability are not issues. At the north abutment approach, the embankment will be constructed over very loose to compact sands. Settlements may approach 100 mm but in the cohesionless soils at the north abutment but will be immediate in nature and are anticipated to be completed by the end of construction.

Earth fill embankment slopes must be provided with erosion protection in accordance with OPSS 572.

12 BACKFILL TO ABUTMENTS

In the case of integral or semi-integral abutments, backfill to the abutment should 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 specify grading limits for the rock fill. The rock fill used as backfill to the abutment should be limited to fragments no greater than 150 mm.

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 must be in accordance with OPSS 902. Granular backfill must be placed to the extents shown in OPSD 3101.150, and rock backfill must be placed to the extents shown in OPSD 3101.200. All granular material should meet the requirements of SP 110F13 Amendment to OPSS 1010, March 1993.

Compaction equipment to be used adjacent to retaining structures must be restricted in accordance with SSP 105S10.

The design of the abutment must incorporate a subdrain as shown in OPSD 3101.150 or OPSD 3101.200, as applicable.

13 EARTH PRESSURE

Earth pressures acting on the structure may be assumed to be triangular and to be governed by the characteristics of the abutment backfill. For a fully drained condition, the pressures should be computed in accordance with the CHBDC but generally are given by the expression:

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

Where:

P_h = horizontal pressure on the wall at depth h (kPa)

K = earth pressure coefficient (Table 13.1)

γ = unit weight of retained soil (Table 13.1)

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 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 shown in Table 13.1.

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 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.

The factors in Table 13.1 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 C6.16 in the Commentary to the Canadian Highway Bridge Design Code.

Table 13.1 - Earth Pressure Coefficient (K)

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 (Limited to 300 mm size) $\phi = 42°$, $\gamma = 19$ 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*	0.2	0.28*
At rest (Restrained Wall)	0.43	-	0.47	-	0.33	-
Passive (Movement Towards Soil Mass)	3.7	-	3.3	-	5.0	-

* For wing walls.

14 SEISMIC CONSIDERATIONS

14.1 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

At the south abutment, the soil profile type has been classified as Type I. Therefore, according to Table 4.4 of the CHBDC, a Site Coefficient "S" (ground motion amplification factor) of 1.0 should be used in seismic design.

At the north abutment, the soil profile type has been classified as Type III. Therefore, according to Table 4.4 of the CHBDC, a Site Coefficient "S" (ground motion amplification factor) of 1.0 should be used in seismic design.

14.2 Liquefaction Potential

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

Using this method, it is estimated that under the existing conditions the foundation soils at the north abutment is not prone to liquefaction. At the abutments, the approach embankments will increase the effective stress on the soil under the embankment and around the piles and as a result, liquefaction at the foundation is not considered to be likely.

If the structure is supported on steel piles, the foundation loads will be transferred by the steel piles to very dense sand with cobbles and boulders, or possibly to bedrock. In either case, it is not considered likely that the vertical geotechnical resistance of the piles will be compromised.

The embankments themselves will be constructed above the groundwater level and are not considered to be in danger of undergoing liquefaction. Some toe failure may occur but it is expected to be of limited nature and readily repairable.

14.3 Retaining Wall Dynamic Earth Pressure

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 14.1 may be used:

---

[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 14.1 - Pressure Coefficient for Earthquake Loading

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 (Max. size: 300 mm) $\phi = 42°$; $\gamma = 19.0$ 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)	0.30	0.47*	0.34	0.58*	0.22	0.31*
At rest (Restrained Wall)	0.53	-	0.58	-	0.44	-
Passive (Movement Towards Soil Mass)	3.58	-	3.15	-	4.92	-

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

15 ROADWAY PROTECTION

North Abutment

Due to the proximity of the new north abutment to the existing structure, it is possible that roadway protection will be required.

An item titled "Protection System" as per OPSS 539 should be included in the contract documents. It is recommended that Performance Level 2 as per Clause 539.04.01.01 and the alignment of the shoring be specified on the contract drawings.

The design of roadway protection should be the responsibility of the contractor. The temporary shoring for roadway protection must be designed by a Professional Engineer experienced in such designs and must be retained by the Contractor.

One option for roadway protection is provision of conventional steel soldier pile and timber lagging walls. Timber lagging boards should be installed as soon as the soil face is exposed and properly prepared. If piles are used, they will be installed into very loose to compact sand to sand and silt.

For a temporary cantilevered shoring system consisting of a soldier pile and lagging wall, the following parameter values may be used:

γ = 20 kN/m^3 (bulk unit weight)
γ_w = 10 kN/m^3 (submerged unit weight under groundwater table)
K_a = 0.33 (Active pressure coefficient for road embankment fill)
= 0.33 (Active earth pressure coefficient for sand)
K_p = 3.0 (Passive earth pressure coefficient for road embankment fill)
= 3.0 (Passive earth pressure coefficient for sand)

The actual pressure distribution acting on the shoring system is a function of the construction sequence and the relative flexibility of the wall and these factors must be considered when designing the shoring system.

South Abutment

At the south, there is a greater distance between the new and the existing abutments and it is not anticipated that there will be a requirement for roadway protection.

If roadway protection is required, the design must take account of the fact that bedrock lies at the ground surface. The active forces may be calculated using the parameters given above. If the height of retained soil is low, a toe wall may provide sufficient resistance but if a higher wall is required then it will have to be socketed into the bedrock.

16 CONSTRUCTION CONCERNS

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

- Unwatering in the case of excavations that must penetrate below the groundwater level
- Inundation of the site due to rising river levels.

17 CLOSURE

Engineering analysis and preparation of the Foundation Design 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
APR 13/11
PROVINCE OF ONTARIO

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

LICENSED PROFESSIONAL ENGINEER
PKChatterji
P. K. CHATTERJI
April 13/11
PROVINCE OF ONTARIO

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

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

▼ 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

THURBER

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Ontario

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RECORD OF BOREHOLE No MR10-03

1 OF 1

METRIC

W.P. 33-78-00
LOCATION N 5 103 851.1 E 218 791.3
ORIGINATED BY SLL
HWY 607
BOREHOLE TYPE Solid Stem Augers
COMPILED BY AN
DATUM Geodetic
DATE 2010.06.03 - 2010.06.03
CHECKED BY LRB

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 ○ 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
181.3						▽								
0.0	**ORGANICS**, mixed with clayey silt, trace cobbles, with roots and rootlets Stiff Black Wet		1	SS	10		181		62					
180.7 180.6 0.6 0.7	**SAND**, trace gravel, occasional cobbles Very Dense Brown Wet		2	SS	100/ 0.075									
	END OF BOREHOLE AT 0.7m UPON AUGER REFUSAL ON PROBABLE BEDROCK. BOREHOLE CAVED TO SURFACE AND WATER LEVEL AT SURFACE.													

+³, ×³: Numbers refer to Sensitivity

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

Ministry of Transportation
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THURBER

RECORD OF BOREHOLE No MR10-04

1 OF 1

METRIC

W.P. 33-78-00
LOCATION N 5 103 853.7 E 218 793.7
ORIGINATED BY SLL
HWY 607
BOREHOLE TYPE NW Casing/NQ Coring
COMPILED BY AN
DATUM Geodetic
DATE 2010.06.03 - 2010.06.03
CHECKED BY LRB

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 (%) 20 40 60	UNIT WEIGHT γ kN/m^3	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV / DEPTH 181.2	DESCRIPTION	NUMBER	TYPE	"N" VALUES						
0.0	**ORGANICS**, mixed with sand, trace gravel, with roots and rootlets Very Dense Brown Wet	1	SS	53/ 0.225		181		o		
180.8 0.4	**GRANITE BEDROCK**, with quartzite veins, very strong to extremely strong Start coring at 0.4m 100mm vertical joints at 0.7m 50mm rubble zone at 1.1m 100mm sub-vertical joints at 0.8m	1	RUN			180			FI 3 4	RUN 1# TCR=100%, SCR=92%, RQD=70% UCS=199MPa
		2	RUN			179			3 1 2 6	RUN 2# TCR=100%, SCR=100%, RQD=72% UCS=108MPa
	—	3	RUN			178			4 4 2	RUN 3# TCR=100%, SCR=100%, RQD=69% UCS=266MPa
177.4 3.8	END OF BOREHOLE AT 3.8m. BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG TO SURFACE.								5	

ONTMT4S 6158.GPJ 10/8/10

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

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

Ministry of Transportation
Ontario

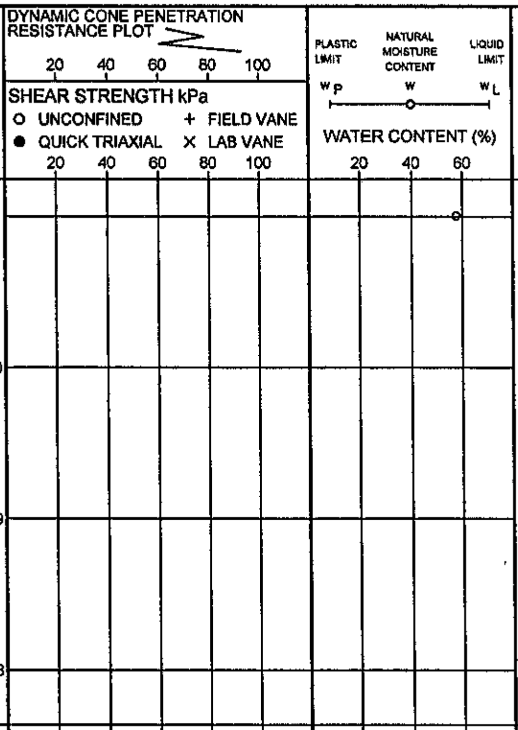
RECORD OF BOREHOLE No MR10-05

1 OF 1

METRIC

W.P. 33-78-00
LOCATION N 5 103 847.9 E 218 794.9
ORIGINATED BY SLL
HWY 607
BOREHOLE TYPE NW Casing/NQ Coring
COMPILED BY AN
DATUM Geodetic
DATE 2010.06.02 - 2010.06.02
CHECKED BY LRB

ELEV DEPTH	SOIL PROFILE DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
181.2									
0.0	**ORGANICS**, peaty, fibrous		1	SS	6		181		
180.9 0.3	**SAND**, trace gravel, occasional cobbles Loose Wet								
180.7 0.5	**GRANITE BEDROCK**, with pink quartizite, strong to extremely strong Start coring at 0.5m 100mm rubble zone at 1.2m Vertical joints at: 100mm at 1.1m 500mm at 1.3m		1	RUN				FI 0 1 8	RUN 1# TCR=85%, SCR=85%, RQD=85% UCS=178 (Average)MPa
			2	RUN			180	5 5	RUN 2# TCR=87%, SCR=37%, RQD=0%
			3	RUN			179	2 5 4 3	RUN 3# TCR=100%, SCR=100%, RQD=76% UCS=195 (Average)MPa
177.6			4	RUN			178	4	RUN 4# TCR=100%,
3.6	END OF BOREHOLE AT 3.6m. BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG TO SURFACE.								SCR=100%, RQD=69% UCS=155 (Average)MPa



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

20 15 5 10 (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No MR10-06

1 OF 1 **METRIC**

W.P. 33-78-00 LOCATION N 5 103 850.5 E 218 797.2 ORIGINATED BY SLL
HWY 607 BOREHOLE TYPE NW Casing/NQ Coring COMPILED BY AN
DATUM Geodetic DATE 2010.06.02 - 2010.06.02 CHECKED BY LRB

ELEV DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLES – NUMBER	SAMPLES – TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
181.1								
0.0 180.8	**ORGANICS**, with roots and rootlets, trace gravel	1	SS	10		181		
0.3	**GRANITE BEDROCK**, with micaceous layers and quartzite veins, strong to extremely strong Start coring at 0.3m	1	RUN			180	FI 3 1 1 2	RUN 1# TCR=100%, SCR=100%, RQD=97% UCS=81 (Average)MPa
		2	RUN			179	0 1 1 1	RUN 2# TCR=100%, SCR=98%, RQD=85% UCS=65 (Average)MPa
177.5	75mm mechanicl breaks at 2.7m	3	RUN			178	1 3	RUN 3# TCR=100%, SCR=100%, RQD=70%
3.6	END OF BOREHOLE AT 3.6m. BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG TO SURFACE.							UCS=280MPa

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

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

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

Ministry of Transportation
Ontario

RECORD OF BOREHOLE No MR10-07

1 OF 2 **METRIC**

W.P. 33-78-00
LOCATION N 5 103 873.5 E 218 805.2
ORIGINATED BY SLL
HWY 607
BOREHOLE TYPE Hollow Stem Augers/NW Casing/NQ Coring
COMPILED BY AN
DATUM Geodetic
DATE 2010.06.01 - 2010.06.02
CHECKED BY LRB

SOIL PROFILE		SAMPLES						
ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	UNIT WEIGHT γ kN/m 3	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
181.3								
0.0	**SAND**, some gravel, occasional cobbles Dense Brown Moist (FILL)							
180.6 0.7	**SAND** and **SILT**, trace clay Loose to Compact Brown to Grey Wet	1	SS	14				
		2	SS	5				
		3	SS	6			0 53 45 2	
		4	SS	8				
		5	SS	5				
		6	SS	9			0 33 65 2	
174.0 7.3	Silty **SAND**, trace gravel Very Dense Grey Moist (TILL)	7	SS	77				
172.5 8.8	**GRANITE BEDROCK**, very strong Start coring at 8.8m	1	RUN			FI 1 1 1	RUN 1# TCR=100%, SCR=100%, RQD=100% UCS=198MPa	

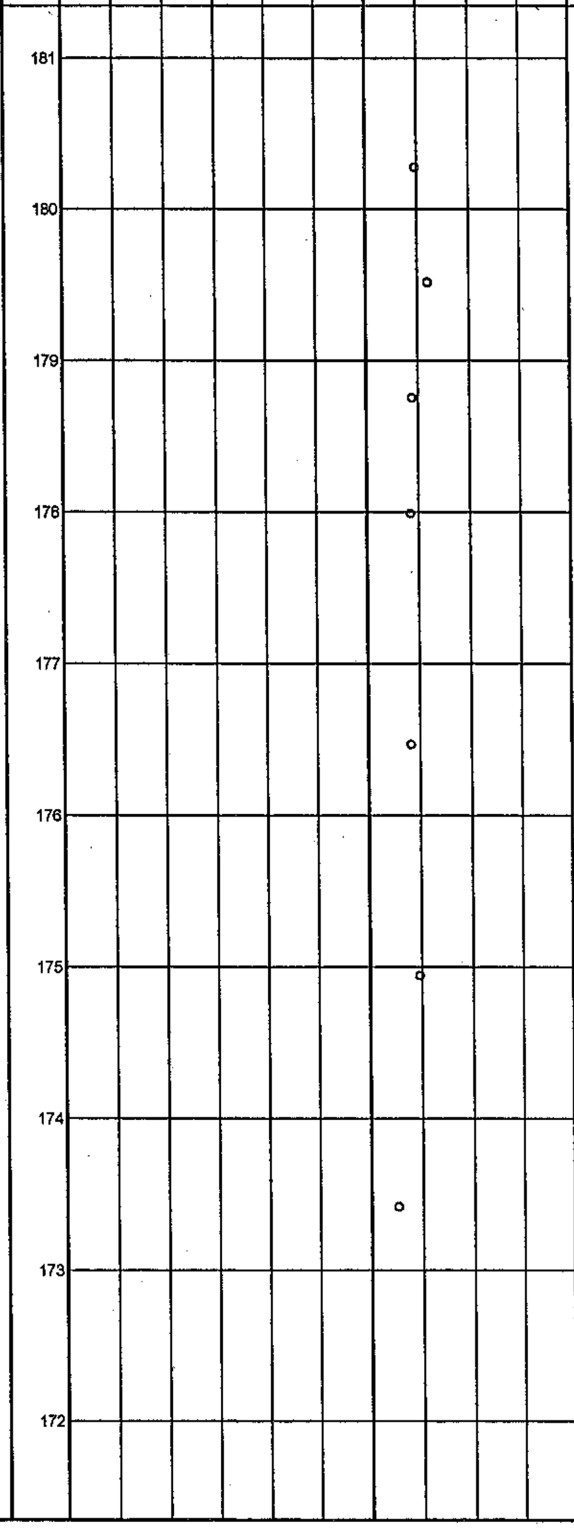
ELEVATION SCALE

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



Continued Next Page

+ 3, × 3 : Numbers refer to Sensitivity

15 20 5 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No MR10-07

2 OF 2

METRIC

W.P. 33-78-00
LOCATION N 5 103 873.5 E 218 805.2
ORIGINATED BY SLL
HWY 607
BOREHOLE TYPE Hollow Stem Augers/NW Casing/NQ Coring
COMPILED BY AN
DATUM Geodetic
DATE 2010.06.01 - 2010.06.02
CHECKED BY LRB

SOIL PROFILE: ELEV DEPTH	SOIL PROFILE: DESCRIPTION Continued From Previous Page	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 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
										0	
							171				
										0	RUN 2# TCR=100%,
										2	SCR=100%, RQD=98%
			2	RUN						1	UCS=190MPa
							170			1	
										0	
										1	RUN 3# TCR=100%,
169.0			3	RUN							
12.3	END OF BOREHOLE AT 12.2m. BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG TO 1.8m, THEN CUTTINGS TO SURFACE.										SCR=100%, RQD=86% UCS=235MPa

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

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

Ministry of Transportation
Ontario

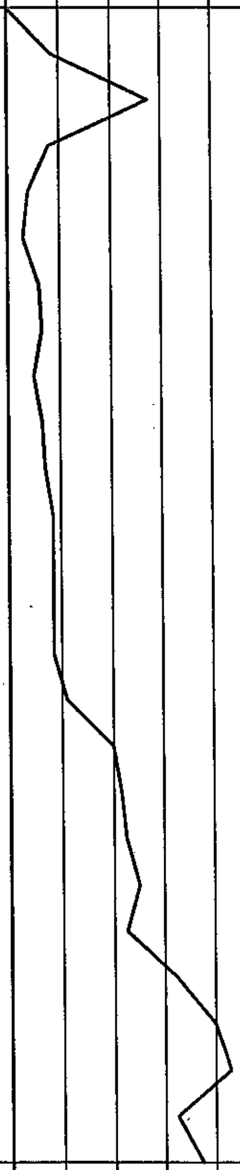
RECORD OF BOREHOLE No MR10-07D

1 OF 1

METRIC

W.P. 33-78-00
LOCATION
ORIGINATED BY SLL
HWY 607
BOREHOLE TYPE DCPT
COMPILED BY AN
DATUM Geodetic
DATE 2010.11.17 - 2010.11.17
CHECKED BY TH

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				
0.0	Start DCPT from surface										
7.6	END OF DCPT AT 7.6m.										



+³, ×³: Numbers refer to Sensitivity

20 15 5 10 (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

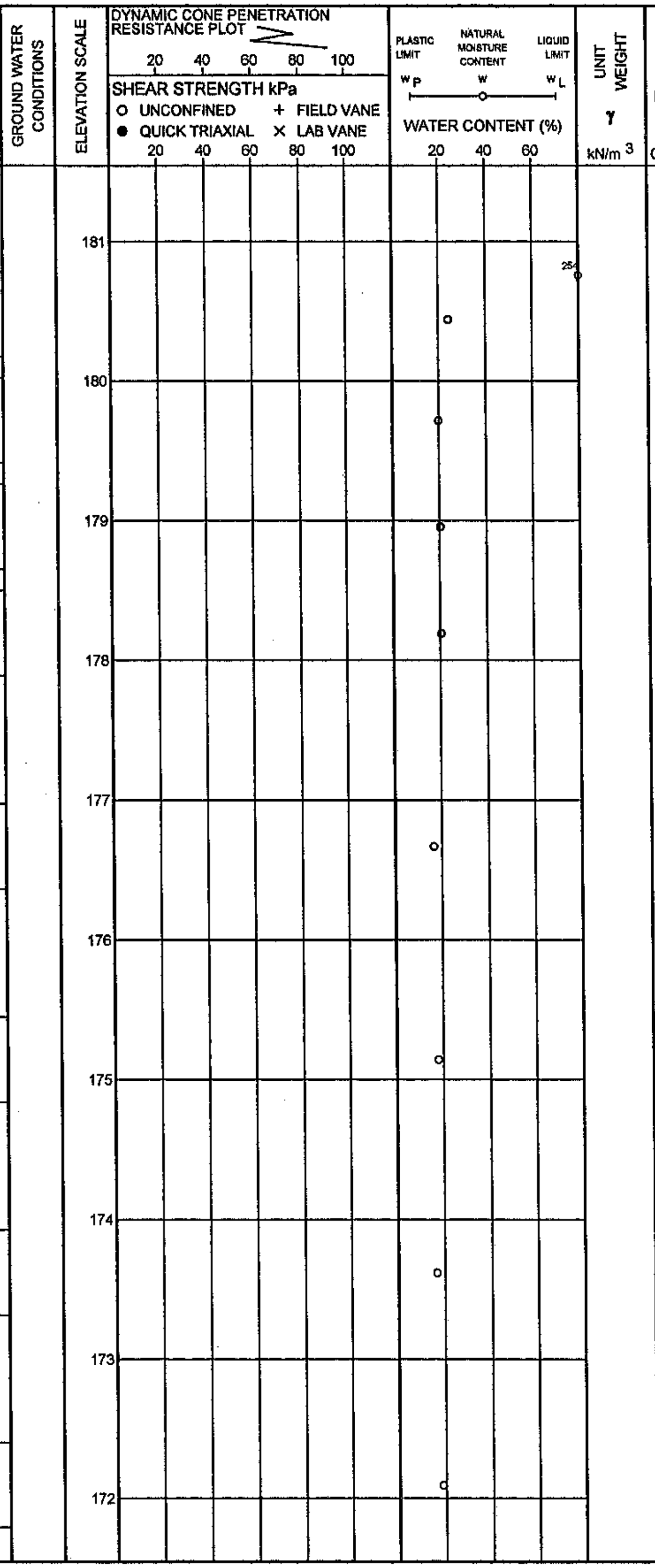
RECORD OF BOREHOLE No MR10-08

1 OF 2

METRIC

W.P.	33-78-00	LOCATION	N 5 103 876.2 E 218 807.6	ORIGINATED BY	SLL
HWY	607	BOREHOLE TYPE	Hollow Stem Augers/NW Casing/NQ Coring	COMPILED BY	AN
DATUM	Geodetic	DATE	2010.06.01 - 2010.06.01	CHECKED BY	LRB

ELEV / DEPTH	SOIL PROFILE: DESCRIPTION	SAMPLES: NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%): GR	SA	SI	CL
181.5								
0.0	**SAND**, some gravel, occasional cobbles and boulders (FILL)							
180.9 0.6	**ORGANICS**, with roots and rootlets Black							
180.7 0.8	**SAND and SILT**, trace clay Very Loose to Dense Brown to Grey Moist to Wet	1	SS	4				
		2	SS	1	0	57	41	2
		3	SS	2				
		4	SS	2				
		5	SS	7				
		6	SS	15				
	Trace gravel	7	SS	38	2	66	30	2
		8	SS	26				



Continued Next Page

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

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

ONTMT4S 6158.GPJ 10/8/10

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No MR10-08

2 OF 2

METRIC

W.P. 33-78-00
LOCATION N 5 103 876.2 E 218 607.6
ORIGINATED BY SLL
HWY 607
BOREHOLE TYPE Hollow Stem Augers/NW Casing/NQ Coring
COMPILED BY AN
DATUM Geodetic
DATE 2010.06.01 - 2010.06.01
CHECKED BY LRB

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						
	Continued From Previous Page										
	SAND and **SILT**, trace clay Compact Grey Moist									FI	
171.1 10.4	**GRANITE BEDROCK**, with thin quartz seam, very strong Start coring at 10.4m		1	RUN			171 170			1 0 1 0	RUN 1# TCR=100%, SCR=100%, RQD=100% UCS=221MPa
			2	RUN			169			0 1 0 0 0	RUN 2# TCR=100%, SCR=100%, RQD=100% UCS=195MPa
168.0			3	RUN						0	RUN 3# TCR=100%, SCR=100%,
13.5	END OF BOREHOLE AT 13.5m. BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG TO 1.8m, THEN CUTTINGS TO SURFACE.										RQD=100% UCS=189 (Average)MPa

ONTMT4S 6158.GPJ 10/8/10

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

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

Ministry of Transportation
Ontario

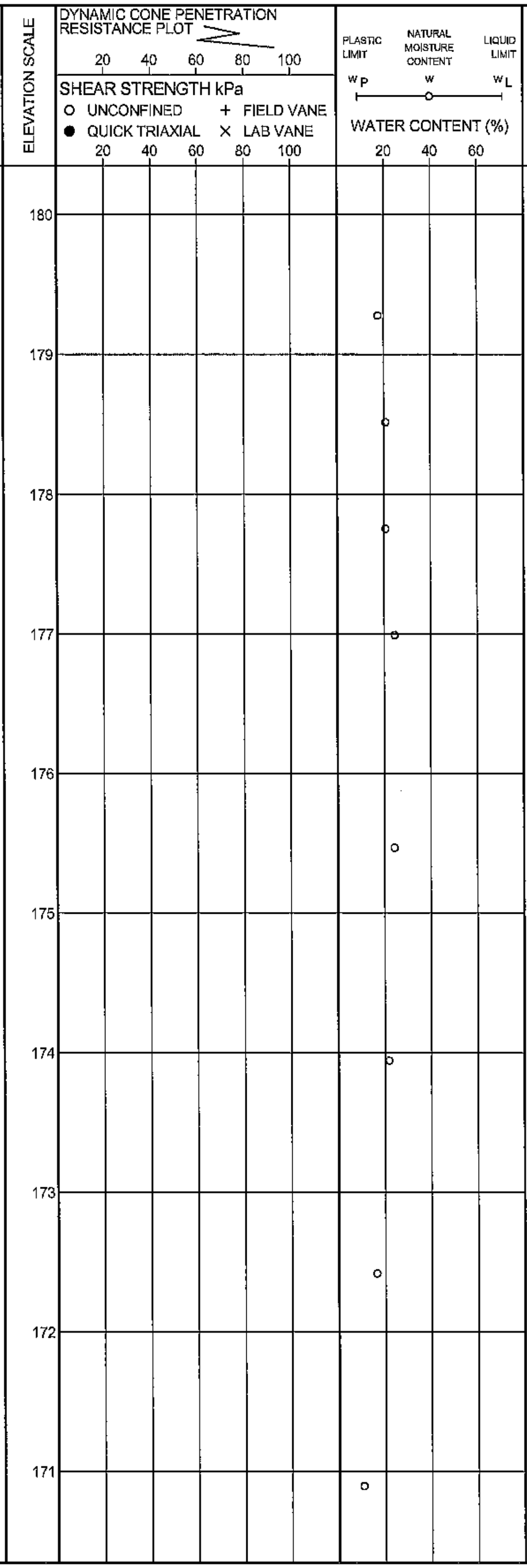
RECORD OF BOREHOLE No MR10-11

1 OF 2

METRIC

W.P. 33-78-00
LOCATION
ORIGINATED BY SLL
HWY 607
BOREHOLE TYPE Hollow Stem Augers/NQ Coring
COMPILED BY AN
DATUM Geodetic
DATE 2010.05.31 - 2010.05.31
CHECKED BY LRB

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
180.3										
0.0	**ORGANICS**, with roots and rootlets									
0.2	**SAND** and **SILT**, trace clay, occasional cobbles Very Loose to Compact Grey Moist to Saturated									
		1	SS	13						
		2	SS	1						
		3	SS	1						
		4	SS	2			0	77	21	2
		5	SS	1						
		6	SS	22						
		7	SS	4						
171.5 8.8	Silty **SAND**, trace to some gravel, trace clay Very Dense Grey Moist (TILL)									
		8	SS	82			11	56	29	4
170.3										



Continued Next Page

+³, ×³: Numbers refer to Sensitivity

20 15 5 10 (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No MR10-11 2 OF 2 METRIC

W.P. 33-78-00
LOCATION
ORIGINATED BY SLL
HWY 607
BOREHOLE TYPE Hollow Stem Augers/NQ Coring
COMPILED BY AN
DATUM Geodetic
DATE 2010.05.31 - 2010.05.31
CHECKED BY LRB

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 Continued From Previous Page	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
10.0	**GRANITE BEDROCK**, thinly beded, strong to very strong Start coring at 10.0m	▨	1	RUN						FI	RUN 1# TCR=100%, SCR=100%, RQD=100% UCS=120MPa
							170			1 2	
	Sub-vertical joints at: 87.5mm at 10.5m 75mm at 11.3m	▨	2	RUN			169			4 3 2 0	RUN 2# TCR=100%, SCR=98%, RQD=90% UCS=108 (Average)MPa
167.1		▨	3	RUN			168			0 0 1 2	RUN 3# TCR=100%, SCR=100%, RQD=96% UCS=73 (Average)MPa
13.2	END OF BOREHOLE AT 13.1m. BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG TO 2.3m, THEN CUTTINGS TO SURFACE.										

ONTMT4S 6158.GPJ 4/6/11

+³, ×³: Numbers refer to Sensitivity

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

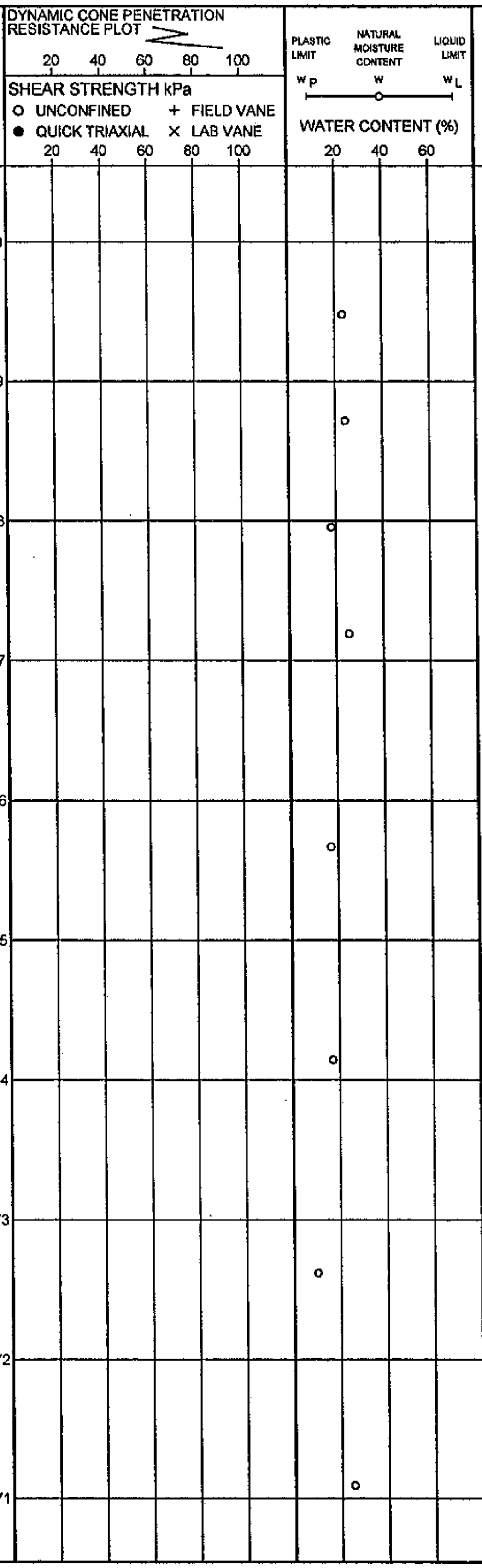
RECORD OF BOREHOLE No MR10-12

1 OF 2

METRIC

W.P. 33-78-00
LOCATION N 5 103 869.8 E 218 814.7
ORIGINATED BY SLL
HWY 607
BOREHOLE TYPE Hollow Stem Augers/NW Casing/NQ Coring
COMPILED BY AN
DATUM Geodetic
DATE 2010.06.01 - 2010.06.01
CHECKED BY LRB

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	GR	SA	SI	CL
180.5 0.0	**ORGANICS**, with roots and rootlets Black								
0.2	**SAND** and **SILT**, trace clay, occasional cobbles Loose to Dense Brown to Grey Wet				180				
		1	SS	15					
					179				
		2	SS	6		0	58	40	2
		3	SS	14	178				
		4	SS	17		0	60	38	2
					177				
					176				
		5	SS	13					
					175				
		6	SS	44					
					174				
					173				
		7	SS	43					
					172				
		8	SS	36	171				



UNIT WEIGHT γ kN/m³

REMARKS & GRAIN SIZE DISTRIBUTION (%)

Continued Next Page

+³, ×³: Numbers refer to Sensitivity

15-20-5-10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No MR10-12

2 OF 2

METRIC

W.P. 33-78-00
LOCATION N 5 103 869.8 E 218 814.7
ORIGINATED BY SLL
HWY 607
BOREHOLE TYPE Hollow Stem Augers/NW Casing/NQ Coring
COMPILED BY AN
DATUM Geodetic
DATE 2010.06.01 - 2010.06.01
CHECKED BY LRB

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	UNIT WEIGHT γ kN/m 3	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	Continued From Previous Page						
169.4	**SAND** and **SILT**, trace clay Loose to Dense Grey Wet	9	SS	4	170		
11.1 168.8	Silty **SAND**, trace gravel, trace clay Grey Moist (TILL)				169		3 82 15 (SI+CL)
11.7 165.7	**GRANITE BEDROCK**, very strong Start coring at 11.7m 50mm sub-vertical joints at 12.2m	1 2 3	RUN RUN RUN		168 167 166	FI 1 5 2 3 1 2 2 1 5 6	RUN 1# TCR=100%, SCR=100%, RQD=0% RUN 2# TCR=100%, SCR=100%, RQD=88% UCS=133 (Average)MPa RUN 3# TCR=100%, SCR=100%, RQD=65% UCS=182 (Average)MPa
14.8	END OF BOREHOLE AT 14.8m. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen.						

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

WATER LEVEL READINGS:

DATE	DEPTH (m)	ELEV. (m)
Jun03/2010	0.13	180.37

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

20 15 5 10 (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No MR10-13

1 OF 1

METRIC

W.P. 33-78-00	LOCATION N 5 103 832.6 E 218 774.5		ORIGINATED BY SLL
HWY 607	BOREHOLE TYPE Visual Inspection		COMPILED BY AN
DATUM Geodetic	DATE 2010.03.08 - 2010.03.08		CHECKED BY LRB

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						
184.3											
0.0	Bedrock at surface										

ONTMT4S 6158.GPJ 10/8/10

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

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

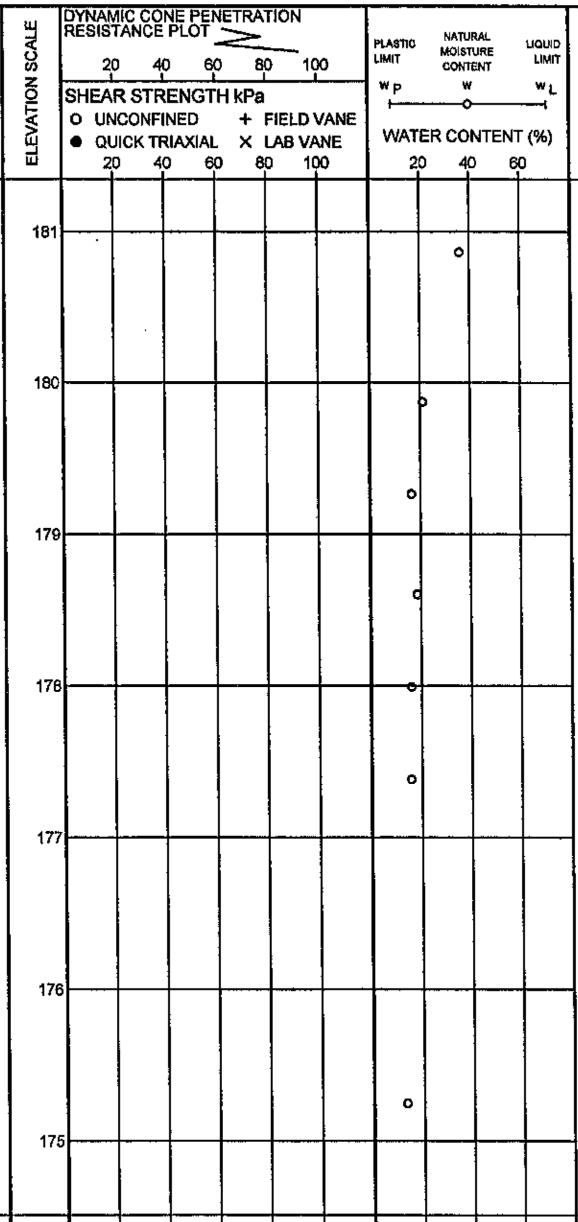
Ministry of Transportation
Ontario

RECORD OF BOREHOLE No MR10-14

1 OF 1 **METRIC**

W.P. 33-78-00
LOCATION N 5 103 882.5 E 218 826.2
ORIGINATED BY JM
HWY 607
BOREHOLE TYPE Continous Sampling/BW Casing
COMPILED BY AN
DATUM Geodetic
DATE 2010.03.08 - 2010.03.08
CHECKED BY TH

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES		GR	SA	SI	CL
181.3									
0.0	**ICE and WATER**								
181.0 0.3	**SAND**, some gravel, occasional silt, trace rootlets, occasional fibrous Compact Dark Brown Wet	1	SS	19					
180.7 0.6	**GRAVEL**, trace cobble Grey Wet								
180.1 1.2	**SAND**, trace to some silt and clay, trace gravel Compact Dark Brown to Grey Wet	2	SS	10		7	91	2 (SI+CL)	
		3	SS	12					
		4	SS	10					
		5	SS	15		8	78	14 (SI+CL)	
		6	SS	13					
175.8 5.5	Silty **SAND**, trace clay, trace gravel Dense to Very Dense Grey Moist to Wet (TILL)	7	SS	48					
		8	SS	100/ 0.275					
174.5 6.8	END OF BOREHOLE AT 6.8m UPON AUGER REFUSAL ON PROBABLE BEDROCK. BOREHOLE CAVED TO 1.2m, THEN BENTONITE HOLEPLUG TO SURFACE.								



+ 3, × 3 : Numbers refer to Sensitivity

15-⊕-5 (20 above, 10 below) (%) STRAIN AT FAILURE

Appendix B

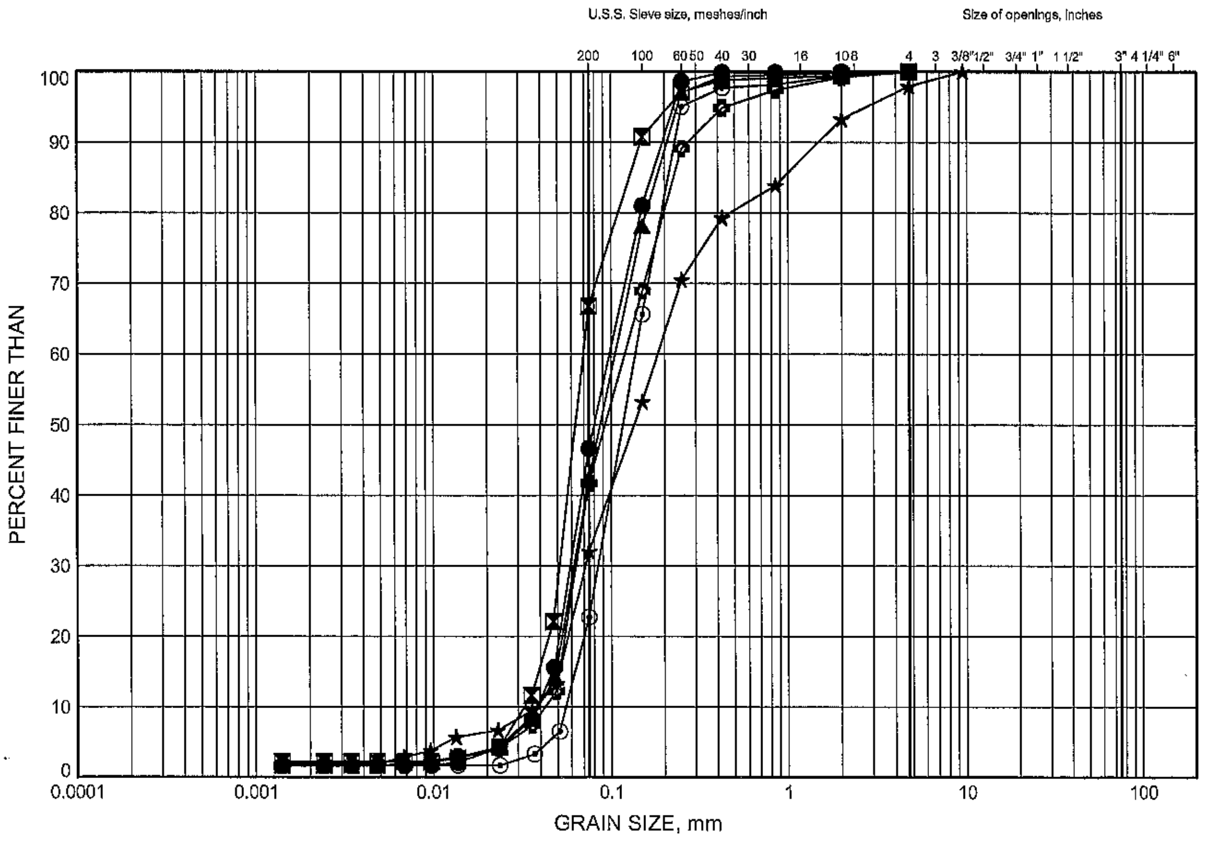
Laboratory Test Results

Murdock River and Grassy Lake Bridges

GRAIN SIZE DISTRIBUTION

FIGURE B1

SAND to SAND & SILT



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	MR10-07	2.59	178.71
⧗	MR10-07	6.40	174.90
▲	MR10-08	1.83	179.67
★	MR10-08	7.92	173.58
⊙	MR10-11	3.35	176.95
✿	MR10-12	1.83	178.67

W.P.# 33-78-00

Prepared By AN

Checked By LRB

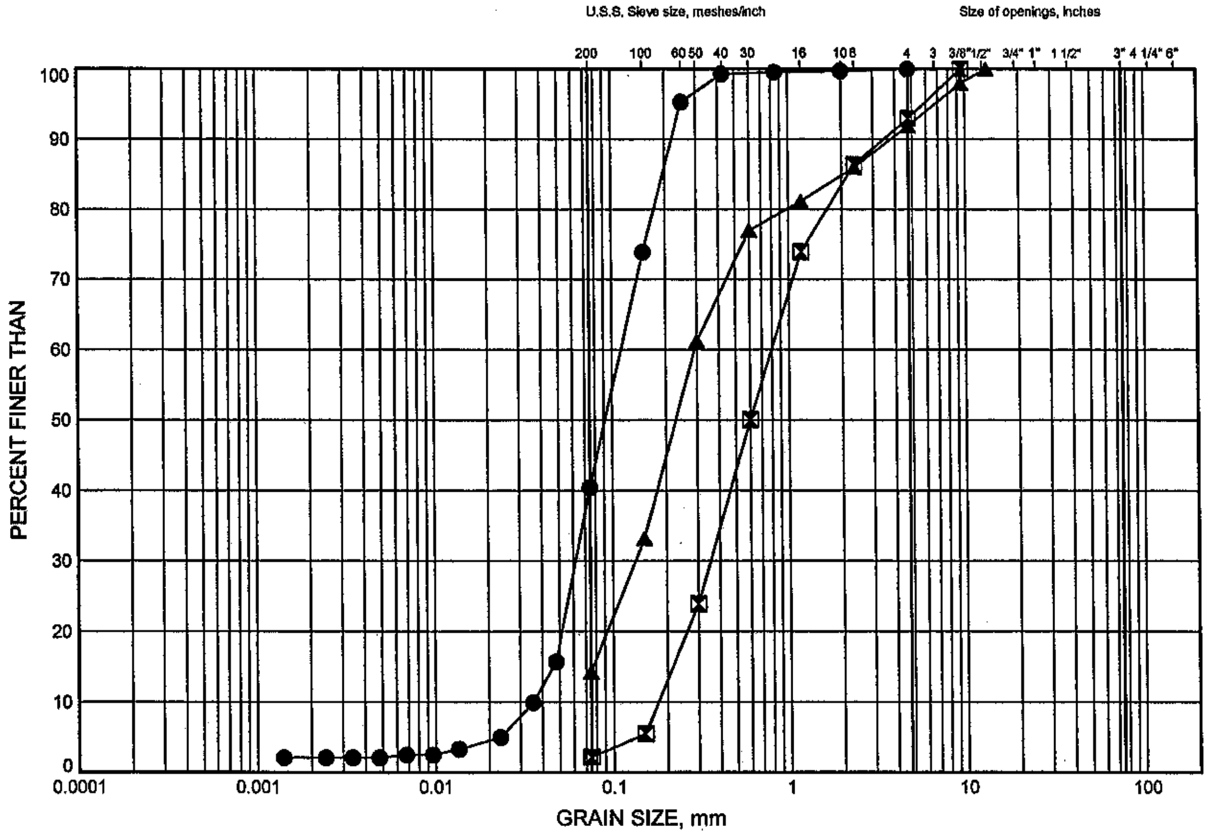
THURBER

Murdock River

GRAIN SIZE DISTRIBUTION

FIGURE B2

SAND to SAND & SILT



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	MR10-12	3.35	177.15
☒	MR10-14	1.47	179.83
▲	MR10-14	3.35	177.95

GRAIN SIZE DISTRIBUTION - THURBER 6158.GPJ 11/12/10

W.P.# 33-78-00

Prepared By AN

Checked By LRB

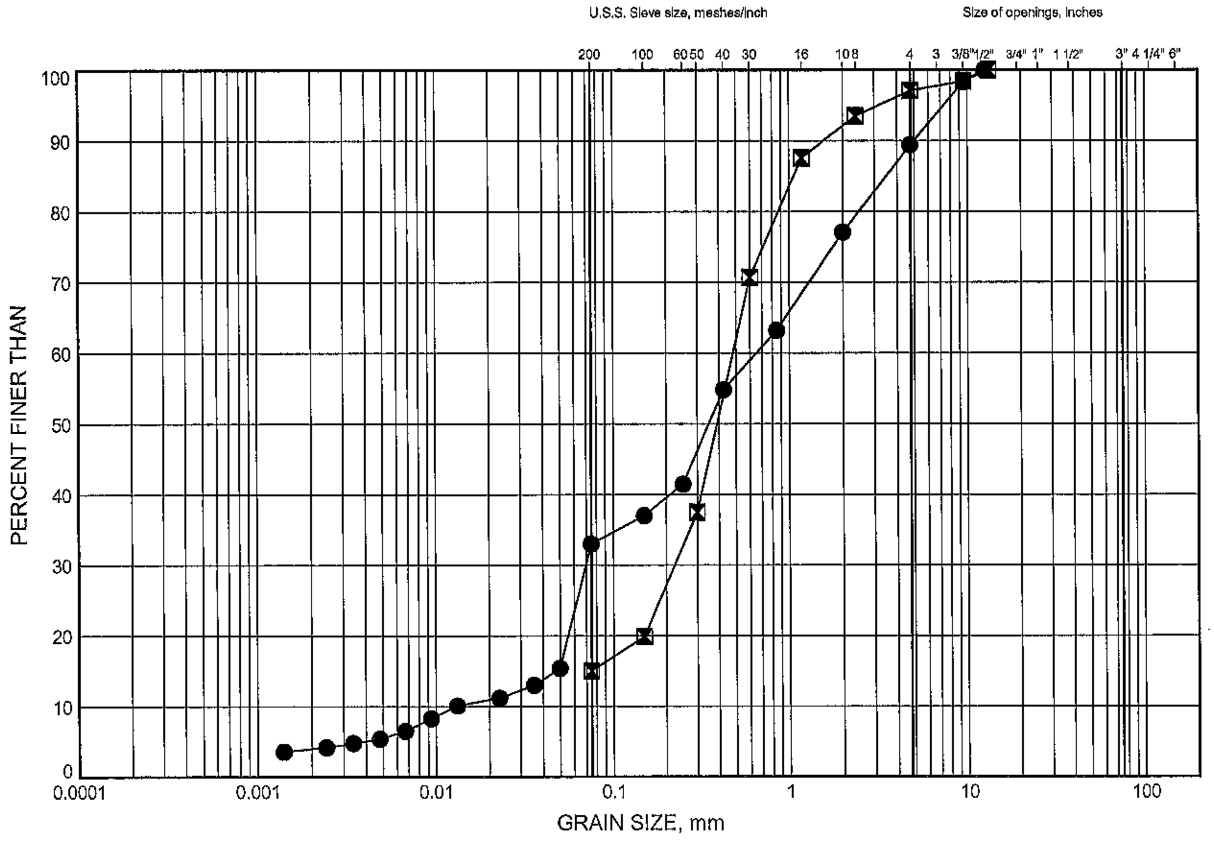
THURBER

Murdock River and Grassy Lake Bridges

GRAIN SIZE DISTRIBUTION

FIGURE B3

Silty SAND TILL



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	MR10-11	9.45	170.85
☒	MR10-12	11.20	169.30

W.P.# 33-78-00
Prepared By AN
Checked By LRB

THURBER

GRAIN SIZE DISTRIBUTION - THURBER 6153.GPJ 4/6/11

Appendix C

Foundation Comparison

COMPARISON OF FOUNDATION ALTERNATIVES FOR EACH FOUNDATION ELEMENT

Driven Piles	Footing on Native Soil/Bedrock	Footing on Engineered Fill
Advantages: i. High geotechnical resistance available by driving piles to bedrock. ii. Allows choice of conventional, integral or semi-integral abutment design. iii. Readily installed.	***Advantages:*** i. Economical to install. ii. Allows use of perched abutments. iii. Allows choice of semi-integral abutment.	***Advantages:*** i. Possibility of shortening the abutment height. ii. Higher geotechnical resistance than is available on native soil. iii. Potentially lower cost compared to deep foundations. iv. Allows use of perched abutments. v. Allows choice of semi-integral abutment.
Disadvantages: i. Higher unit cost compared to footings.	***Disadvantages:*** i. Potentially longer abutment stem. ii. Excavation and dewatering will be required.	***Disadvantages:*** i. Lower geotechnical resistance than piles. ii. High cost of constructing engineered fill. iii. Potential settlements at south abutment.
Recommended for North Abutment	***Recommended for South Abutment*** Note: To be built on near surface bedrock	***Not Recommended***

Appendix D

Site Photographs

Photo 1. South approach to the Murdock River Bridge

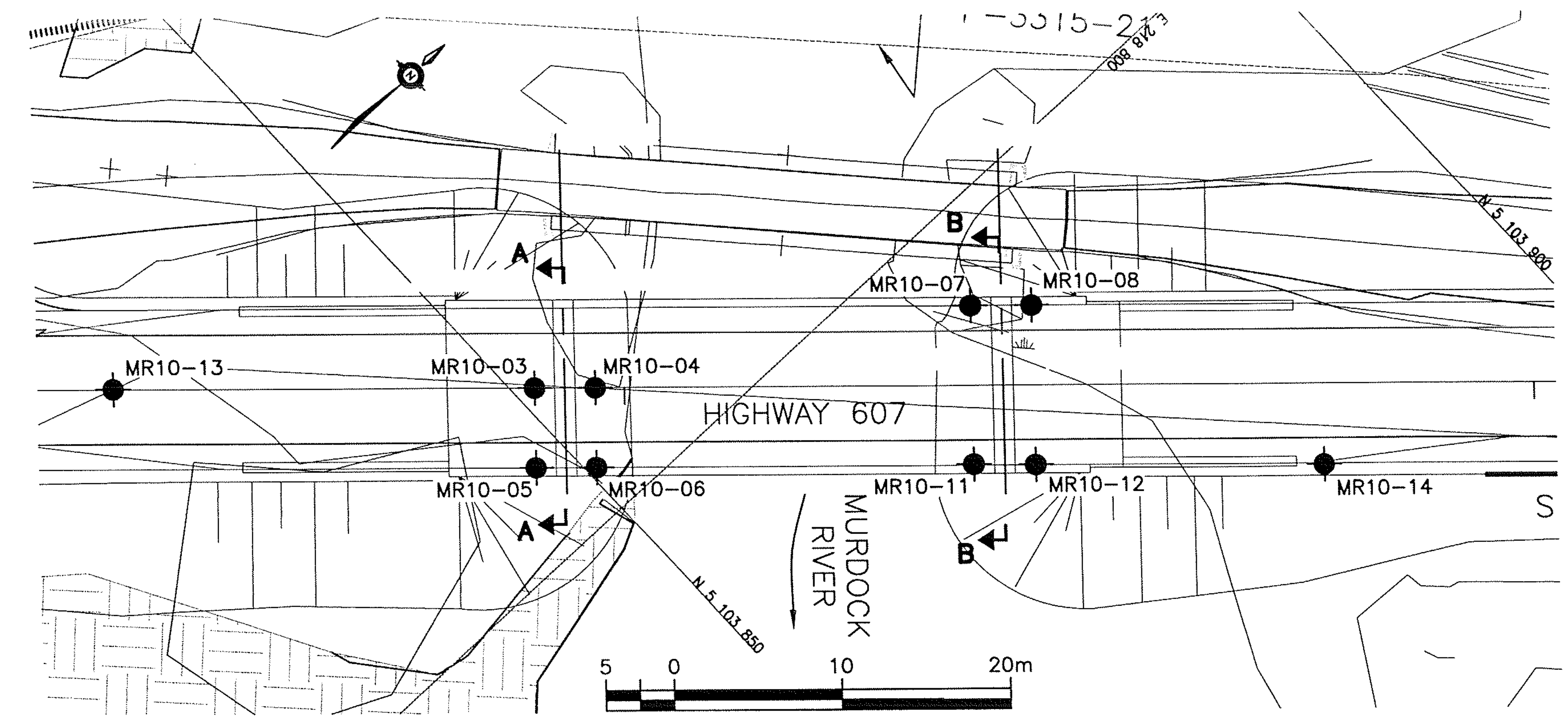
Photo 2. Looking north along the Murdock River Bridge

Photo 3. Murdock River Bridge from the southwest showing cobbles and boulders

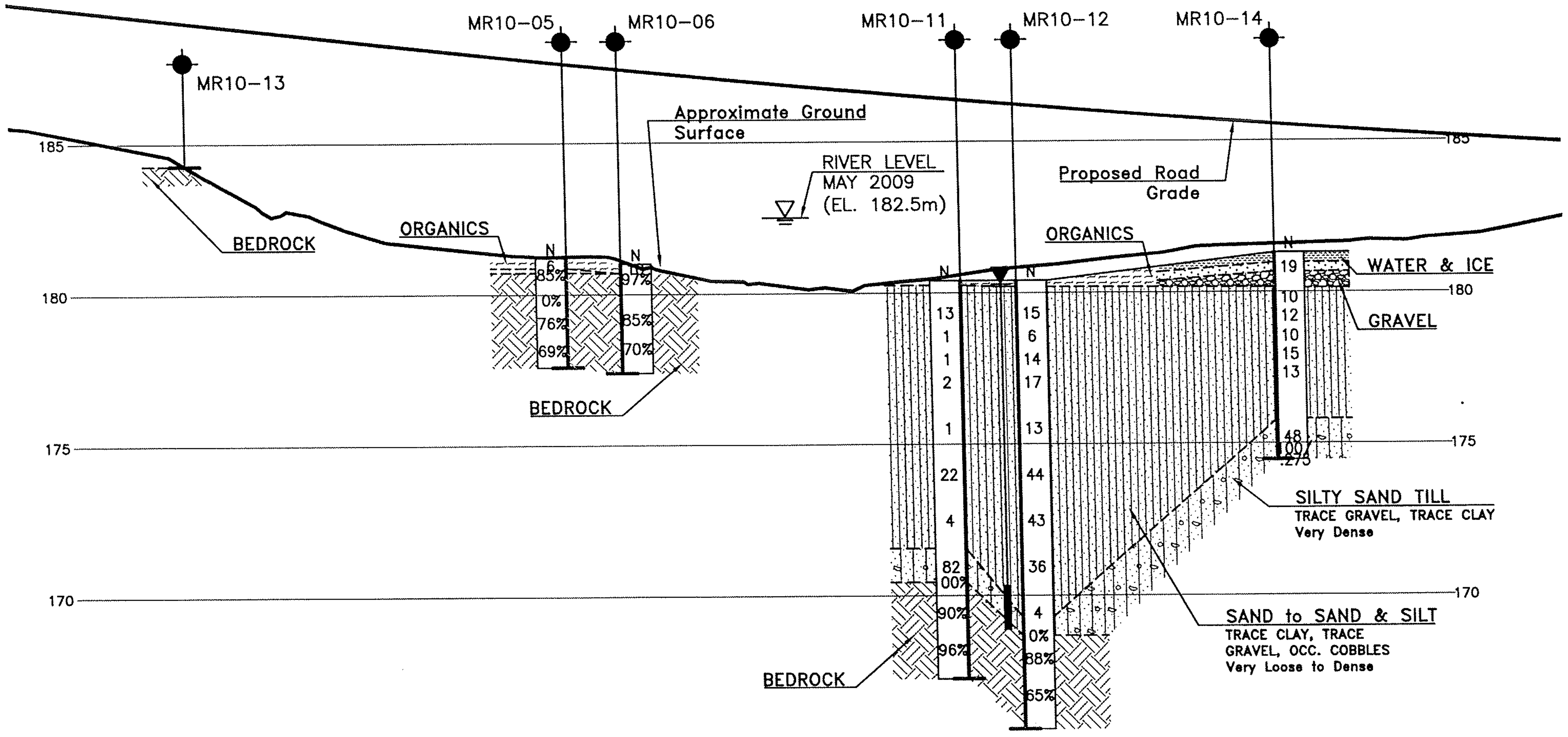
Photo 4. Rockfill pile located northeast of the existing bridge

Appendix E

Drawings



SCALE: 1:400
PLAN



PROFILE ALONG ℄ HIGHWAY 607

4 0 10 20m HOR 1:400

2 0 5 10m VER 1:200

METRIC

DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN

CONT No
GWP No 33-78-00

SHEET

HIGHWAY 607
MURDOCK RIVER
BRIDGE
BOREHOLE LOCATIONS AND SOIL STRATA

MMM GROUP

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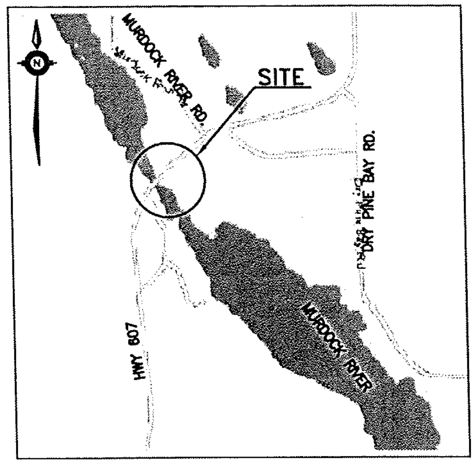


KEYPLAN

LEGEND

●	Borehole
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
MR10-03	181.3	5 103 851.1	218 791.3
MR10-04	181.2	5 103 853.7	218 793.7
MR10-05	181.2	5 103 847.9	218 794.8
MR10-06	181.1	5 103 850.5	218 797.2
MR10-07	181.3	5 103 873.5	218 805.2
MR10-08	181.5	5 103 876.2	218 807.6
MR10-11	180.3	5 103 867.2	218 812.2
MR10-12	180.5	5 103 869.8	218 814.7
MR10-13	184.3	5 103 832.6	218 774.5
MR10-14	181.3	5 103 882.5	218 826.2

A. E. GORMAN
Apr 13/11

P. K. CHATTERJI
April 13/11

-NOTES-

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

2) This drawing is for subsurface information only. Surface details and features are for conceptual illustration.

GEOCRES No. 41A-266

REVISIONS	DATE	BY	DESCRIPTION

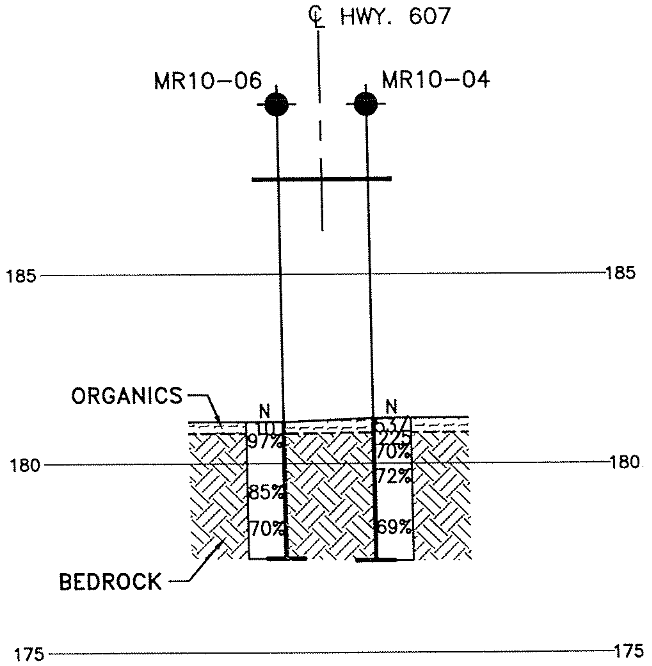
DESIGN LRB	CHK	CODE	LOAD	DATE APR. 2011
DRAWN AN	CHK	SITE	STRUCT	DWG 1

FILENAME: H:\Drafting\19\5161\58\tad5158-MurdockRiver-Plan&Profile.dwg
PLOTDATE: Apr 12, 2011 - 9:37am

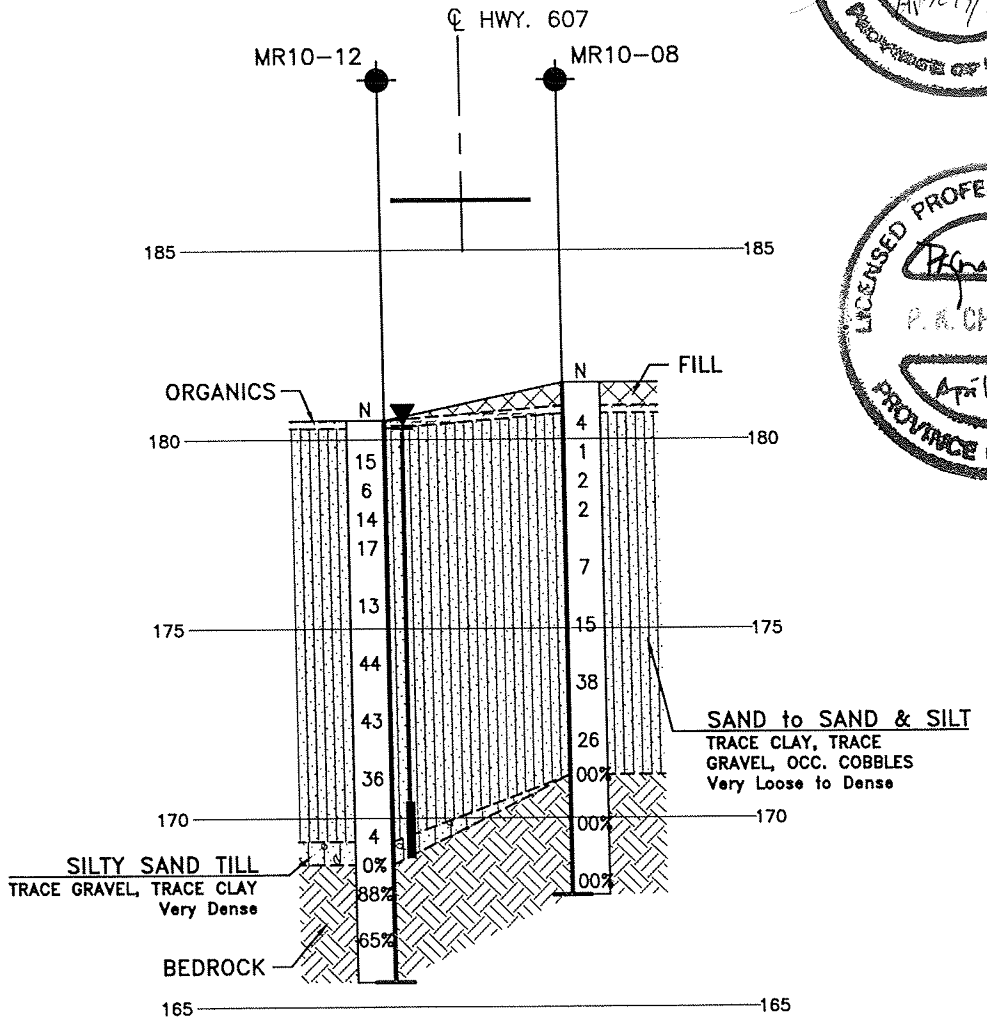
MINISTRY OF TRANSPORTATION, ONTARIO

METRIC

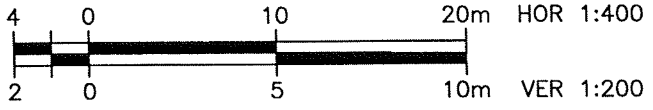
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN



SECTION A–A
SOUTH ABUTMENT



SECTION B–B
NORTH ABUTMENT



A. E. GORMAN
Apr 13/11

P. K. CHATTERJI
April 13/11

CONT No
GWP No 33–78–00

HIGHWAY 607
MURDOCK RIVER
BRIDGE
BOREHOLE LOCATIONS AND SOIL STRATA

SHEET

MMM GROUP

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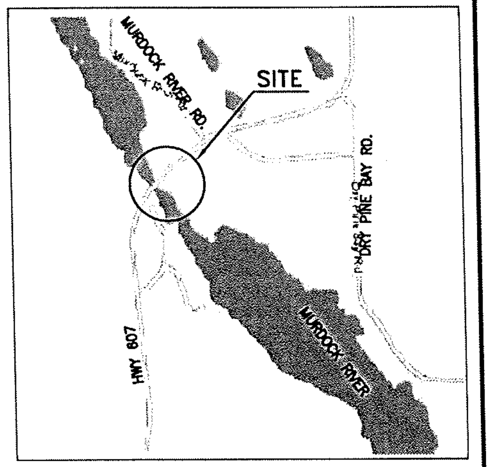

KEYPLAN

LEGEND

◆	Borehole
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
MR10–04	181.2	5 103 853.7	218 793.7
MR10–06	181.1	5 103 850.5	218 797.2
MR10–08	181.5	5 103 876.2	218 807.6
MR10–12	180.5	5 103 869.8	218 814.7

-NOTES-

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

2) This drawing is for subsurface information only. Surface details and features are for conceptual illustration.

GEOCRES No. 41A-266

REVISIONS	DATE	BY	DESCRIPTION

DESIGN LRB	CHK	CODE	LOAD	DATE APR. 2011
DRAWN AN	CHK	SITE	STRUCT	DWG 2

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PLOTDATE: Apr 12, 2011 – 9:39am

Appendix F

Technical References, NSSP Text

1. List of Special Provisions and OPSS Documents Referenced in this Report

- OPSS 902
- OPSS 903
- OPSS 572
- OPSD 3101.150
- OPSD 3101.200
- SP 110F13 Amendment to OPSS 1010, March 1993
- SP 105S10

2. List of Canadian Highway Building Design Code References in this Report

- Clause 6.7.3
- Clause 6.7.4
- Clause 6.8.9.2
- Clause 6.9.3
- Clause 4.6.4
- Figure C6.9.1(a) – CHBDC Commentary
- Table 4.4.6.1

3. Suggested text for a NSSP on Unwatering

Excavations at this site, particularly the north abutment will penetrate below the groundwater level.

The soils overlying the bedrock at this site are predominantly cohesionless and will be readily disturbed by unbalanced water heads or by flow of water.

The Contractor shall design, install and operate systems that shall:

i. Unwater the excavations
ii. Control the flow of groundwater, surface water and river water into the excavations
iii. Prevent the disturbance of the base of the excavation
iv. Prevent the sloughing of soil into the excavations.

Particular attention must be paid to the design of unwatering systems and shoring systems at the pier locations due to the proximity of the river and the cohesionless nature of the overburden.

The selection and design of suitable unwatering and shoring systems shall remain the responsibility of the Contractor. However, factors that might influence the selection and design of the unwatering system and the shoring system include, but are by no means limited to the probable level of the river during construction. The selected systems must prevent flooding of the work area due to rising river levels. It is recommended that the designs allow for a river level that will rise to Elevation [redacted].

4. Suggested text for a NSSP on Pile Driving

Steel H-piles driven at this site must be founded on bedrock. All driven piles shall be fitted with cast steel, H-section rock points from an approved manufacturer such as Titus Steel (Standard H-point) or approved equivalent.