FOUNDATION INVESTIGATION AND DESIGN REPORT
HAWKEYE CREEK BRIDGE REPLACEMENT
HIGHWAY 589, NORTH OF LAPPE, ONTARIO
THUNDER BAY UNORGANIZED DISTRICT
G.W.P. 6045-08-00, SITE 48W-241

Geocres Number: 52A-150

Report to

GENIVAR

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PART 1: FACTUAL INFORMATION

1 INTRODUCTION

This report presents the factual findings obtained from a foundation investigation conducted at the location of a proposed bridge replacement crossing Hawkeye Creek. The existing bridge carries Highway 589 over the Hawkeye Creek, approximately 20 Km north of Lappe, Ontario, in Thunder Bay Unorganized District.

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, laboratory test results and written descriptions 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 GENIVAR, under the Ministry of Transportation Ontario (MTO) Agreement Number 6010-E-0012.

2 SITE DESCRIPTION

The Hawkeye Creek bridge is located on Highway 589 (Dog Lake Road), between Paul Lake Road and Mary Lake Road, approximately 20 Km north of Lappe, Ontario.

Higwhay 589 is an unpaved two-lane road. The existing bridge consists of a single span bridge with timber stringers and deck. The length and width of the existing bridge are 7.9 m and 6.75 m, respectively.

At this location, the Hawkeye Creek flows from west to east.

The lands immediately surrounding the bridge site consist of forested areas.

A Photograph in Appendix C shows the general nature of the surrounding land.

The site is underlain by Precambrian rocks and is covered with Pleistocene and recent deposits. These deposits consist of clays, silts and sands.

3 SITE INVESTIGATION AND FIELD TESTING

The site investigation and field testing for this project were carried out on July 15, 17 to 19 and 21 to 23, 2011 and consisted of drilling and sampling six boreholes (identified as HCB-01 to HCB-06), at the existing bridge location through the existing highway embankments. Boreholes HCB-01 to HCB-04 were drilled near the north and south abutments and advanced within the overburden to depths ranging from 20.1 m to 24.1m (elevations 72.9 to 77.2), where the auger encountered refusal. Bedrock was proved in Boreholes HCB-01 and HCB-03 by NQ size diamond coring. Borehole HCB-01 was advanced 3.0 m into bedrock and terminated at 27.1 m depth (Elevation 69.9). Borehole HCB-03 was advanced 3.6 m into bedrock and terminated at 25.5 m depth (Elevation 71.8).

Boreholes HCB-05 and HCB-06 were drilled at the south and north approaches, respectively, and terminated at 12.8 m and 11.3 m depth (elevations 84.6 and 85.5).

Boreholes HCB-01 to HCB-04 were supplemented by dynamic cone penetration testing (DCPT) conducted adjacent to each borehole. The depths to the DCPT ranged from 18.6 m to 22.7 m (elevations 74.2 to 78.6).

The approximate borehole locations are shown on the attached Borehole Locations and Soil Strata Drawing included in Appendix F.

The borehole locations were marked in the field and utility clearances were obtained prior to drilling.

The drilling was carried out from the highway grade using a CME75 truck-mounted drill rig. A combination of hollow-stem auger drilling techniques and NQ coring methods were used to advance the boreholes. Overburden samples were obtained at selected intervals using a split spoon sampler in conjunction with Standard Penetration Testing (SPT).

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 samples for transport to Thurber's laboratory for further examination and testing.

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

Groundwater conditions were observed in the open boreholes during and upon completion of the drilling operations. One standpipe piezometer consisting of 19 mm diameter PVC pipe with a slotted screen was installed at each abutment and enclosed in filter sand to permit longer term groundwater level monitoring. The boreholes were backfilled with bentonite holeplug in general accordance with O.Reg. 903 upon completion. The locations and completion details of the boreholes are shown in Table 3.1.

Table 3.1 – Borehole Abandonment Details

Location	Borehole	Piezometer Tip Depth/ Elevation (m)	Abandonment Details
North Abutment	HCB-01	18.9 / 78.1	Piezometer with 1.5 m slotted screen installed with sand filter to 15.5 m, holeplug from 15.5 m to 14.9 m, grout from 14.9 m to 1.8 m, holeplug from 1.8 m to 0.15 m, then concrete to surface. Flushmount installed.
	HCB-02	None installed	Backfilled with bentonite holeplug from 20.4 m to 2.1 m, then sand and gravel to surface.
South Abutment	HCB-03	25.5/71.8	Piezometer with 1.5 m slotted screen installed with sand filter from 25.5 m to 21.8 m, holeplug from 21.8 m to 0.6 m, then sand and gravel to surface.
	HCB-04	None installed	Backfilled with holeplug from 10.9 m to 1.5 m, then sand and gravel to surface.
South Approach	HCB-05	None installed	Backfilled with holeplug from 6.7 m to 1.5 m, then sand and gravel to surface.
North Approach	HCB-06	None installed	Backfilled with holeplug from 7.0 m to 1.5 m, then sand and gravel to surface.

4 LABORATORY TESTING

The recovered soil samples were subjected to Visual Identification (VI) and to natural moisture content determination. Selected samples were also subjected to gradation analysis. The results of these tests are summarized on the Record of Borehole sheets included in Appendix A and are presented on the figures included in Appendix B.

Point load tests were carried out on selected samples of intact bedrock upon arrival at the laboratory to assist in evaluation of the compressive strength of the bedrock. Results of point load tests on the rock core samples are included in Appendix B and on the Record of Borehole sheets in Appendix A.

5 DESCRIPTION OF SUBSURFACE CONDITIONS

Reference is made to the Record of Borehole sheets included in Appendix A. Details of the encountered soil and rock stratigraphy are presented in these sheets and on the “Borehole Locations and Soil Strata” drawing included in Appendix F. An overall description of the stratigraphy is given in the following paragraphs. However, the factual data presented in the Record of Borehole sheets governs any interpretation of the site conditions.

In general terms, the overburden soil stratigraphy encountered at this site consists of sand and gravel fill over native layers of sand and gravel, sand, sandy silt and silt. Cobbles and boulders were encountered at some locations within the sand and sand and gravel layers. A 1.1 m to 2.9 m thick layer of organic silt was contacted at the north approach and north abutment. The overburden is underlain by highly weathered to fresh diorite bedrock. More detailed descriptions of the individual strata are presented below.

5.1 Fill

Granular fill was encountered surficially in all the boreholes. The granular fill consists of layers of brown sand and sand and gravel containing some silt and trace to some clay. The thickness of the cohesionless fill ranges from 1.5 m to 4.1 m.

The elevations to the base of the fill range from 93.3 to 95.5.

SPT N-values ranging from 4 to 35 blows for 0.3 m penetration were recorded in the sand fill and sand and gravel fill, indicating a loose to dense relative density.

The moisture contents of the sand fill and sand and gravel fill range from 5% to 19%.

Grain size distribution curves for selected fill samples are presented on the Record of Borehole sheets and on Figures B1 and B2 of Appendix B. The results of the laboratory tests are summarized as follows:

Soil Particles	Sand Fill (%)	Sand and Gravel Fill (%)
Gravel	7 to 8	28
Sand	67 to 86	56
Silt	23	-
Clay	3	-
Silt and Clay	6	16

5.2 Sand

Native brown to grey sand containing trace to some gravel, clay and silt was encountered in the boreholes at depths and elevations indicated in Table 5.1. Cobbles and boulders were encountered within the sand layer near elevation 77.3 in Borehole HCB-02.

Table 5.1 – Depths and Elevations of Native Sand

Foundation Unit	Borehole	Depth below existing ground surface (m)	Elevation (m)	Thickness (m)
North Abutment	HCB-01	2.3 to 4.6	94.7 to 92.4	2.3
	HCB-02	1.5 to 7.6 9.0 to 13.7 15.2 to 21.0 (borehole termination depth)	95.4 to 89.3 88.0 to 83.2 81.7 to 75.9	6.1 4.7 5.8
South Abutment	HCB-03	2.9 to 4.6 13.7 to 15.2	94.3 to 92.6 83.5 to 82.0	1.7 1.5
	HCB-04	8.5	88.7	Less than 300 mm
South Approach	HCB-05	4.1 to 12.8 (borehole termination depth)	93.3 to 84.6	8.7
North Approach	HCB-06	2.9 to 6.1	93.9 to 90.7	3.2

SPT 'N' values measured in the sand ranged from 3 to 12 blows per 0.3 m of penetration indicating a very loose to compact relative density. In Borehole HCB-02, SPT 'N' values measured in sand below elevation 81.7, ranged from 22 to 28 blows per 0.3 m of penetration, indicating a compact relative density. A SPT 'N' of 50 blows without any penetration was measured in Borehole HCB-02 near elevation 77.3, which corresponds to the zone where cobbles and boulders were encountered.

The natural moisture contents generally lay in the range of 17% to 58%.

Grain size distribution curves for selected native sand samples are presented on the Record of Borehole sheets and on Figures B3 and B4 of Appendix B. The results of the laboratory tests are summarized as follows:

Soil Particles	(%)
Gravel	0 to 28
Sand	52 to 93
Silt	13
Clay	3
Silt and Clay	3 to 20

5.3 Organic silt

Dark brown to black organic silt containing some sand to sandy, trace clay and occasional roots was contacted below the native sand at 4.6 m and 6.1 m depth (elevations 92.4 and 90.7) in Boreholes HCB-01 and HCB-06, respectively. The thickness of the organic silt was 2.9 m and 1.1 m.

A layer of organic silt, approximately 800 mm thick, was encountered in Borehole HCB-01 drilled at the south abutment, at 1.5 m depth (elevation 95.5).

The depths to the base of the organic silt layer were 7.5 m and 7.2 m (elevations 89.5 and 89.7).

SPT N-values of 3 to 4 blows for 0.3 m penetration were recorded in the organic silt layer, indicating a very loose to loose relative density.

The moisture content of organic silt ranged from 81% to 159%.

A grain size distribution curve for an organic silt sample is presented on the Record of Borehole sheets and on Figure B7 of Appendix B. The results of the laboratory tests are summarized as follows:

Soil Particles	(%)
Gravel	0
Sand	24
Silt	71
Clay	5

5.4 Sand and Gravel

Layers of dark brown to grey sand and gravel were encountered in Boreholes HCB-01, HCB-03 and HCB-04 at depths and elevations indicated in Table 5.2.

Table 5.2 – Depths and Elevations of Native Sand and Gravel

Foundation Unit	Borehole	Depth below existing ground surface (m)	Elevation (m)	Thickness (m)
North Abutment	HCB-01	20 to 24.1	77.0 to 72.9	4.1
South Abutment	HCB-03	4.6 to 13.7 15.2 to 21.9	92.6 to 83.5 82.0 to 75.3	9.1 6.7
	HCB-04	2.3 to 20.1	94.9 to 77.2	17.8

Cobbles and boulders were encountered at various depths through the sand and gravel layers. Frequent cobbles and boulders were encountered below elevation 83.0 in Borehole HCB-04, drilled at the south abutment. Cobbles and boulders were also contacted below elevation 78.0 in Boreholes HCB-02 and HCB-03.

SPT N-values in the sand and gravel layer, ranged from 3 to 55 blows per 0.3 m of penetration indicating very loose to dense relative density. In Borehole HCB-04, below elevation 83.0 generally where cobbles and boulders were contacted, the SPT 'N' values were 100 blows per 0.25 m of penetration.

The moisture contents of samples of the sand and gravel range from 8% to 20%. A sample from Borehole HCB-04, near elevation 82.5, revealed a moisture content of 3%.

Grain size distribution curves for selected samples are presented on the Record of Borehole sheets and on Figure B6 of Appendix B. The results of the laboratory tests are summarized as follows:

Soil Particles	(%)
Gravel	18 to 64
Sand	28 to 79
Silt and Clay	1 to 14

5.5 Sandy Silt

A layer of grey sandy silt containing trace clay was contacted below the organic silt layer at 7.5 m depth (elevation 89.5) in Borehole HCB-01. The thickness of the sandy silt layer was 12.5 m.

The depth to the base of the sandy silt was 20.0 m (elevation 77.0).

SPT 'N' values of the sandy silt layer ranged from 2 to 27 blows per 0.3 m of penetration, indicating a very loose to compact relative density. Near elevation 77.5, a SPT 'N' value of 44 blows per 0.3 m of penetration, indicating a dense relative density.

The moisture content in the sandy silt ranged from 16% to 30%, and one sample was 62%.

Grain size distribution curves for selected sandy silt samples are presented on the Record of Borehole sheets and on Figure B5 of Appendix B. The results of the laboratory tests are summarized as follows:

Soil Particles	(%)
Gravel	0
Sand	24 to 33
Silt	63 to 72
Clay	4 to 5

5.6 Silt

Grey silt containing trace to some sand, trace clay and occasional roots was contacted in Boreholes HCB-02 and HCB-06 at depths and elevations indicated in Table 5.3.

Table 5.3 – Depths and Elevations of Native Silt

Foundation Unit	Borehole	Depth below existing ground surface (m)	Elevation (m)	Thickness (m)
North Abutment	HCB-02	7.6 to 9.0 13.7 to 15.2	89.3 to 88.0 83.2 to 81.7	1.4 1.5
South Approach	HCB-06	7.2 to 11.3 (borehole termination depth)	89.7 to 85.5	4.1

SPT 'N' values measured in the silt ranged from 2 to 16 blows per 0.3 m of penetration indicating a very loose to compact relative density.

The natural moisture contents generally lay in the range of 17% to 19%. One sample from Borehole HCB-02 revealed moisture content of 78%.

Grain size distribution curves for selected silt samples are presented on the Record of Borehole sheets and on Figure B8 of Appendix B. The results of the laboratory tests are summarized as follows:

Soil Particles	(%)
Gravel	0
Sand	2 to 14
Silt	82 to 94
Clay	4

5.7 Bedrock

The overburden soils described above are underlain by grey diorite bedrock. Occasional mechanical breaks and sub-vertical fractures were noted throughout the bedrock cores. The bedrock is generally described as moderately weathered to fresh. The cores obtained from Runs 1 and 2 of Borehole HCB-01 were described as highly weathered.

Bedrock was proved by coring in Boreholes HCB-01 and HCB-03 drilled at the north and south abutment, respectively. Table 5.4 summarizes depths and elevations to the top of bedrock or depth to auger refusal in the boreholes.

Table 5.4 – Depths and Elevations of Top of Bedrock / Auger Refusal

Location	Borehole	Top of Bedrock/Refusal	
		Depth (m)	Elevation (m)
North Abutment	HCB-01*	24.1	72.9
	HCB-02	21.0	75.9
South Abutment	HCB-03*	21.9	75.3
	HCB-04	20.1	77.2

*Bedrock proved by coring

Total core recovery (TCR) in the bedrock was 100% in most of the cores and 76% in Borehole HCB-03 Run 1. The RQD values ranged from 25% to 68%, indicating poor to fair rock quality. The Fracture Index (FI) of the rock, expressed as fractures per 0.3 m of core, was generally 0 to 8. The FI was greater than 10 in cores from Borehole HCB-01.

The estimated unconfined compressive strength of the rock cores generally ranges from 50 MPa to 193 MPa, indicating a strong to very strong rock. Lower estimated unconfined compressive strength of 12 MPa was measured in Borehole HCB-01 Run1, indicating a weak 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 Appendix B.

5.8 Water Levels

Water levels were observed in the open boreholes upon completion of drilling operations. Two standpipe piezometers were installed in Boreholes HCB-01 and HCB03 to monitor water levels after completion of drilling. The water levels measured in the open boreholes and piezometers are summarized in Table 5.5.

Table 5.5 – Water Level Measurements

Location	Borehole	Date	Water Level (m)		Comment
			Depth	Elevation	
North Abutment	HCB-01	August 17, 2011	1.4	95.6	Piezometer
South Abutment	HCB-03	August 17, 2011	1.6	95.6	Piezometer
	HCB-04	July 21, 2011	1.8	95.4	Open borehole
South Approach	HCB-05	July 23, 2011	2.3	95.1	Open borehole
North Approach	HCB-06	July 23, 2011	1.7	95.1	Open borehole

The piezometric readings reveal that the groundwater level is at elevation 95.6, 1.4 m to 1.6 m below ground surface.

GA drawing indicates the following water levels/elevations of the Hawkeye Creek at the bridge location at various dates:

- 95.8 on April 14, 2011
- 95.2 on July 19, 2011
- 95.4 on August 15, 2011

The above values are short-term readings and seasonal fluctuations of the groundwater level are to be expected. In particular, the groundwater level may be at a higher elevation after the spring snowmelt or after periods of heavy rainfall.

6 MISCELLANEOUS

Borehole locations were selected in the field by Thurber Engineering Ltd. Borehole elevations and coordinates were provided by Genivar.

Thurber obtained utility clearances for the borehole locations prior to drilling.

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

The field program was supervised on a full time basis by Mr. Jason Mei and Mr. George Azzopardi of Thurber.

Routine laboratory testing was carried out by Thurber Engineering Ltd.

Overall planning and supervision of the field program was conducted by Mr. Mark Farrant, P. Eng. Interpretation of the data and preparation of this report were carried out by Ms. R. Palomeque Reyna, P.Eng.

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

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PART 2: ENGINEERING DISCUSSION AND RECOMMENDATIONS

7 INTRODUCTION

This report presents interpretation of the geotechnical data in the factual report and presents geotechnical recommendations for design of a new bridge to replace the existing bridge that carries Highway 589 over the Hawkeye Creek, approximately 20 Km north of Lappe, Ontario, in Thunder Bay Unorganized District.

Highway 589 is an unpaved two-lane road. The existing bridge consists of a single span bridge with timber stringers and deck supported on two abutments. The north and south abutments are founded on rock-filled timber cribs. The length and width of the bridge are 7.9 m and 6.75 m, respectively. The embankments are approximately 2.0 m to 3.0 m high. It is understood the existing bridge and the timber cribs supporting the existing abutments will be removed.

Based on the General Arrangement (GA) drawing provided by Genivar, the proposed bridge consists of a two lane, single span structure with a deck of prestressed voided concrete girders supported on a single row of driven steel H-piles. A sheet pile wall will be driven just behind the H-piles to retain the approach fill. The proposed length of the bridge is 10.4 m with a width of 7.2 m. It is anticipated that the replacement structure will be constructed along the existing horizontal alignment. The bridge approach will be raised approximately 0.5 m at the bridge location.

The discussions and recommendations presented in this report are based on the factual data obtained during the course of the investigation. The plans and profiles used for preparation of this report were provided by Genivar.

8 STRUCTURE FOUNDATIONS

The stratigraphy encountered in the six boreholes drilled at the north and south approaches and abutments revealed 1.5 m to 4.1 m of loose to dense sand fill and sand and gravel fill. Below the fill, layers of native sand and gravel, sand, sandy silt and silt were encountered. The relative density of the native cohesionless soils varied from very loose to dense. Cobbles and boulders were encountered at some locations within the native sand/sand and gravel layers. A 1.1-m to 2.9-m thick layer of organic silt was contacted at the north approach and north abutment. Moderately weathered to fresh diorite bedrock was encountered directly below the sand and gravel layer at 21.9 m and 24.1 m depth. Auger refusal on probable bedrock was met at 21.0 m and 20.1 m depth.

The piezometric readings reveal that the groundwater level is at elevation 95.6. GA shows that water level of Hawkeye Creek at the bridge location varies from elevation 95.8 to 95.2 from April to August.

Geotechnical recommendations for design of the proposed H-pile foundation system are presented in the following sections. Foundation alternatives together with corresponding geotechnical design parameters for feasible options are also presented in the event that the foundation concept changes.

A comparison of the technical advantages and disadvantages of alternative foundation schemes (driven steel H-piles, spread footings on native soil, and caissons/drilled shafts) is presented in Appendix D. A foundation scheme preferred from a foundations perspective is recommended.

8.1 Steel H-Pile Foundations

The ground conditions at the site are considered to be suitable for the support of foundations on steel H-piles driven to bedrock or dense sand and gravel. In general, it is anticipated that the piles will encounter refusal at the bedrock surface. However at the south abutment (Borehole HCB-04), the piles may encounter refusal above the bedrock surface in very dense/dense sand and gravel with cobbles and boulders.

Recommendations are provided for various pile tip elevations for piles driven to dense sand and gravel and piles driven to bedrock.

The anticipated pile tip elevations, soil conditions, vertical, factored geotechnical resistance at Ultimate Limit States (ULS_f) and geotechnical resistance at Serviceability Limit States (SLS) for H-piles founded on bedrock or on very dense sand and gravel are presented in Table 8.1.

Table 8.1 – Estimated Pile Tip Elevation and Recommended Pile Resistance Values

Soil Conditions at Pile Tip	Pile Tip Elevation	Pile Section HP 310 x 110	
		Factored Geotechnical Resistance at ULS	Geotechnical Resistance at SLS
Dense sand and gravel	77*	900	750
Bedrock	72.9 to 75.3	2,000	Does not govern

* Possible presence of cobbles and boulders.

The factored structural resistance of the piles at ULS must be checked by the structural designer as per Section 6.8.8 of the CHBDC.

Oversize materials (e.g. greater than 75 mm nominal diameter) must not be used in any fills through which the piles will be driven.

8.1.1 Pile Tips

The tips of all piles should be fitted with cast steel, H-section rock points from an approved manufacturer such as Titus Steel (Standard H-point) or approved equivalent. Some piles may achieve refusal on large boulders, which will require the same pile tip protection and reinforcement as founding on bedrock.

8.1.2 Pile Installation

Pile installation should be in accordance with OPSS 903, November 2009.

Pile installation should consider the following subsurface factors:

- The presence of cobbles and boulders in the native sands and gravels, particularly at the south abutment. This has been discussed in Section 5.4 of the Factual Report.
- The possibility that piles may encounter refusal in cobbles and boulders at different elevations and/or above the anticipated bedrock elevation.

We understand that the proposed bridge design may require that the deviation at the top of the pile be limited to 12 mm. Use of a driving template or other means may be required to achieve the specified maximum deviation.

Recommendations are provided for installation of piles driven to refusal/bedrock and piles driven to very dense soils.

8.1.2.1 Pile driven to refusal or bedrock

For piles installed for the tolerances shown in Clause 903.07.05.01 of the Specification, the foundation drawing should include the note "Piles to be driven to bedrock".

To reduce the potential for misalignment resulting from hard driving to confirm bedrock, it is recommended that the pile driving note on the foundation drawing be modified as follows:

"Piles to be driven to bedrock without any damage". Upon initial contact with the bedrock:

1. Apply 10 blows at 10% of the hammer energy. Record the penetration.
2. Apply 10 blows at 50% of the hammer energy. If the penetration under 10 blows is less than 12.5 mm, the pile is set.
3. If the penetration under 10 blows is greater than 12.5 mm, refer the issue to the design team for resolution."

8.1.2.2 Pile driven to very dense soils

For piles driven to very dense soil, pile driving must be controlled by the Hiley Formula and an ultimate pile resistance to be specified by the designer in accordance with Clause 3.3.2 (b) Construction Stage of the Structural Manual. The Hiley formula need not be used until the piles are within 2.0 m of the bearing stratum, near Elevation 77.0. The appropriate pile driving note is "Piles to be driven in accordance with Standard SS 103-11 using an ultimate resistance of "R" kN per pile". "R" must have a minimum value of twice the design load at ULS.

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.

8.1.3 Downdrag

A layer of loose organic silt was encountered at 4.6 m depth (elevation 92.4) in Borehole HCB-01, drilled at the northwest side of the north embankment. The organic silt layer is 2.9 m thick.

Downdrag forces will develop along the length of pile embedded in the organic silt layer, and overlying sand due to increased approach embankment loads. For design purposes, an unfactored downdrag load of 180 kN per pile is recommended to evaluate the impact of downdrag at the northwest side of the north abutment. This downdrag load should be multiplied by a load factor of 1.25 as per CHBDC Commentary Clause C8.6.4 to obtain a factored downdrag load.

In accordance with Section 6.8.4 of the CHBDC and clause C6.8.4 of the Commentary to CHBDC, at the north abutment, in the structural design of a pile, the factored downdrag load should be added to the factored permanent loads to assess the effects of downdrag.

In geotechnical analysis of downdrag, live load effects should not be considered. The location of the neutral plane for a pile or groups of piles should be determined by using unfactored loads and unfactored geotechnical parameters.

8.1.4 Lateral Resistance

The lateral resistance of a pile in the predominantly cohesionless soils encountered at this site may be calculated using a value for the coefficient of horizontal subgrade reaction (k_s) and ultimate lateral resistance (p_{ult}) as follows:

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

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

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 assumed in one pile be limited to no more than 120 kN at ULS and 50 kN at SLS.

Table 8.3 – Parameters for Lateral Pile Resistance

Location	Elevation	K_p	n_h (kN/m^3)	Unit Weight* (kN/m^3)	Soil Conditions
North abutment	OGL to 95.0	3.0	2,500	21	Sand fill, sand and gravel fill, loose to dense
	95.0 to 92.4	3.0	2,500	11*	Sand, loose to compact
	92.4 to 89.5	2.7	1,000	9*	Organic silt, loose
	89.5 to 77.0	3.0	2,500	11*	Sand and silt, very loose to dense
	77.0 to 73.9	3.3	6,500	11*	Sand and gravel, dense
South abutment	OGL to 94.5	3.0	2,500	21	Sand fill, sand and gravel fill, loose to dense
	94.5 to 83.0	3.0	3,000	11*	Sand, sand and gravel, very loose to compact
	82.0 to 75.5	3.3	8,000	11*	Sand and gravel, cobbles and boulders, compact to vey dense

*Buoyant unit weight below the water table.

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 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

Where a pile group is oriented ***parallel*** 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 Parallel to Direction of Loading	Horizontal Subgrade Reaction Reduction Factor, R
8 D	1.00
6 D	0.70
4 D	0.40
3 D	0.25

Intermediate values may be obtained by interpolation.

8.2 Spread Footings on Native Soils

Spread footings founded on native soils are not recommended at this site due to the following reasons:

- Very low geotechnical capacities are available in the overburden cohesionles soils at this site.
- Groundwater levels at the site are high. Unwatering/groundwater control will be difficult for construction of footings.

8.3 Caissons

Augered caissons founded on bedrock were also considered for the support of the structure. However, the use of augered caissons is not recommended in view of the depth to bedrock or refusal which is in the order of 20.1 m to 24.1 m below existing ground surface and the presence of water-bearing cohesionless overburden soils at this site. The base of the caissons will be well below the groundwater level, resulting in difficulties in dewatering, base cleaning and base inspection. Construction of caissons will require the use of a liner sealed below the sand/sand and gravel layers and/or slurry methods to control ground water, support the sidewalls of the shaft.

Installation of deep caissons to bedrock is also expected to be a more expensive option than driven piles.

For these reasons, the use of a caisson foundation is not recommended.

8.4 Recommended Foundation

From a geotechnical perspective and based on the subsurface conditions steel H-pile foundations driven to bedrock or refusal in dense sand and gravel are considered the most cost effective foundation option for supporting the bridge at this site.

8.5 Frost Cover

The depth of frost penetration at this site is 2.3 m. The base of pile caps, if employed, must be provided with a minimum of 2.3 m of earth cover as protection against frost action.

9 SHEET PILE WALLS

Steel sheet pile walls will be driven adjacent to the H-pile foundations at each abutment. The sheet piles will provide containment and resistance to lateral earth pressures from the approach fill.

Driving of the sheet piles through the existing approach fill and into the underlying very loose to compact sands and silts is considered feasible based on the borehole data.

Backfill to the sheet pile walls should be in accordance with OPSS 902. Granular backfill should be placed to the extents shown in OPSD 3101.150. All granular material should meet the specifications of OPSS 1010 as amended by Special Provision 110S13. Compaction equipment to be used adjacent to retaining structures should be restricted in accordance with OPSS 501.

Earth pressures acting on the sheet pile walls may be assumed to be triangular and to be governed by the characteristics of the abutment backfill and the existing sand and silt. 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 (see Table 9.1)

γ = unit weight of retained soil (see Table 9.1)

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

q = value of any surcharge (kPa)

Earth pressure coefficients for backfill to the abutment wall are dependent on the material used as backfill. Typical values are shown in Table 9.1.

Table 9.1 – 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		Existing Sand and Gravel Fill, native sand and gravel and OPSS Granular B Type I $\phi = 32°, \gamma = 21.2$ kN/m^3		Native Sand/Silt $\phi = 30°$ $\gamma = 20$ kN/m3	
	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)	Horizontal Surface Behind Wall	Horizontal Surface Behind Wall	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)
Active (Unrestrained Wall)	0.27	0.40*	0.31	0.48*	0.33	0.57*
At rest (Restrained Wall)	0.43	-	0.47	-	0.5	-
Passive (Movement Towards Soil Mass)	3.7	-	3.3	-	3.0	-

* For wing walls.

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.

The factors in Table 9.1 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 (a) in the Commentary to the Canadian Highway Bridge Design Code.

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.

10 EXCAVATION AND GROUNDWATER CONTROL

Excavation for removal of the existing timber cribs and backfill is expected to be limited to the existing sand and gravel approach fill, above the groundwater and creek water level.

All excavation must be carried out in accordance with the Occupational Health and Safety Act (OHSA). For the purposes of the OHSA, the sand and gravel fill forming the existing approach embankment and the underlying native sand/sand and gravel may be classified as Type 3 soil above the water table and Type 4 below the water table.

The excavation and backfilling must be carried out in accordance with OPSS 902, November 2010.

The selection of the method of excavation is the responsibility of the contractor and must be based on his equipment, experience and interpretation of the site conditions. Excavations should be inspected regularly for evidence of instability.

The Contractor should be prepared to pump from sumps to remove any seepage water or surface water collecting in an excavation. Excavation below the creek level, if required for an alternate foundation system, would require dewatering within a cofferdam to lower the water level below the base of the excavation.

11 APPROACH EMBANKMENTS

Based on a contour drawing provided by Genivar, it was estimated that the existing approach embankment is up to 2.0 m to 3.0 m high. The foundation soils governing stability of the approach embankments consist generally of existing native very loose to compact sand, silt and sand and gravel layers.

Proposed highway profiles provided by Genivar indicates that the existing Highway 589 grade will be raised approximately 0.5 m at the bridge location.

The embankment foundation soils are considered to provide adequate stability to earth fills inclined at 2H:1V and rockfills inclined at 1.25H:1V.

11.1 Slope stability

The global, internal and surficial stability of the approach embankment fills depends on the slope geometry and also to a large degree on the material used to construct the embankment.

The existing embankment bearing on the foundation soils present at this site has performed well under the existing conditions. Since the ground raise is minimal (330 mm) and foundation soils are largely granular, it is expected that the raised embankment will perform satisfactorily.

11.2 Settlement

The existing embankments have been in place for some years and it is anticipated that settlements of the native soils are completed.

However, placement of approximately 0.5 m of fill will induce some settlement in the existing sand fill and layers of native sand and organic silt.

It is estimated that at the north abutment, settlements in the order of 20 mm to 30 mm will occur in the foundation soils under the loading imposed by the new fill. At the south abutment, the estimated settlement of the native cohesionless soils range from 10 mm to 15 mm.

Due to the non-cohesive nature of the foundation soils, this settlement will be immediate and essentially complete when construction of the fill is completed.

For placement of new fill, the existing slope surfaces should be appropriately benched, as per OPSD 208.010, after stripping of vegetation, topsoil, organics, soft soils or otherwise unsuitable overburden materials.

12 EROSION PROTECTION

Erosion protection should be provided along the toe of any slopes that may be in contact with the creek flow.

A vegetation cover should be established on all other exposed earth surfaces to protect against surficial erosion, in general accordance with OPSS 804, November 2010.

13 ROADWAY PROTECTION

During the new bridge construction, temporary excavation of existing embankments will be required. The bridge construction will be done in stages in order to keep at least one highway lane operational. Roadway protection will be required to facilitate staging of removals and support the existing Highway589 adjacent to the excavation.

Roadway protection should be provided in accordance with OPSS 539 and designed for Performance Level 2.

Conventional steel soldier pile and timber lagging walls or continuous sheet pile wall are two options to provide temporary support to the roadway during excavation. Timber lagging boards should be installed as soon as the soil face is exposed and properly prepared.

The following parameters apply for design of the temporary shoring system:

γ = 21 kN/m^3 (bulk unit weight)
γ_w = 11 kN/m^3 (submerged unit weight under groundwater table)
K_a = 0.33 (Active pressure coefficient for road embankment fill)
= 0.33 (Active pressure coefficient for native sand/silt)
K_p = 3.0 (Passive pressure coefficient for road embankment fill)
= 3.0 (Passive pressure coefficient for native sand/silt)
h_w = 0 (assuming that the groundwater is maintained below the base of the excavation and that there is no hydrostatic pressure build-up behind a presumably permeable wall)

h_w = 95.6 (elevation for hydrostatic pressure build-up behind wall)

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.

Temporary groundwater and surface water control measures may be required during construction.

The design of roadway protection should be the responsibility of the Contractor. All shoring systems should be designed by a Professional Engineer experienced in such designs.

14 SEISMIC CONSIDERATIONS

14.1 Seismic Design Parameters

The following seismic parameters should be used for design:

• Velocity Related Seismic Zone	0
• Zonal Velocity Ratio	0.0
• Acceleration Related Seismic Zone	0
• Zonal Acceleration Ratio	0.0
• Peak Horizontal Acceleration	0.02

The soil profile type at this site has been classified as Type III. Therefore, according to Table 4.4.6.1 of the CHBDC, a Site Coefficient "S" (ground motion amplification factor) of 1.5 should be used in seismic design.

14.2 Liquefaction Potential

The site overlies very loose to dense cohesionless deposits and a high water table is present at this site.

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 abutments is not generally prone to liquefaction. Localized liquefaction during a seismic event may result in local toe failure or minor embankment settlement, but this is expected to be readily repairable.

If the structure is supported on steel piles, the foundation loads will be transferred by the steel piles to bedrock. In this case, it is not considered likely that the vertical geotechnical resistance of the piles will be compromised.

14.3 Retaining Wall Dynamic Earth Pressures

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

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[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.

of earthquake loading. The coefficients of horizontal earth pressure for seismic loading presented in Table 14.1 may be used:

Table 14.1 – Earth Pressure Coefficients for Earthquake Loading

Condition	Earth Pressure Coefficient (K)		
	OPSS Granular A or OPSS Granular B Type II $\phi = 35°, \gamma = 22.8$ **kN/m³**	**Existing Sand and Gravel Fill, native sand and gravel and OPSS Granular B Type I** $\phi = 32°, \gamma = 21.2$ **kN/m³**	**Native Sand/Silt** $\phi = 30°$ $\gamma = 20$ **kN/m³**
Active (K_{AE})*	0.28	0.32	0.34
Passive (K_{PE})	3.7	3.2	3.0
At Rest (K_{OE})**	0.45	0.50	0.52

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

15 CONSTRUCTION CONCERNS

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

- The potential variability of pile lengths since some piles particularly at the south abutment may reach refusal in a cobble/boulder layer above the bedrock.
- Pile tips must be protected with H-section rock points and driving must be terminated before pile is damaged.
- The side embankment slopes should be inspected after construction for surficial disturbance. Where necessary, erosion control measures must be implemented.

16 CLOSURE

Engineering analysis and preparation of the report were carried out by Ms. R. Palomeque Reyna, P.Eng.

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

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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		Field Estimation of Hardness*
	(MPa)	(psi)	
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

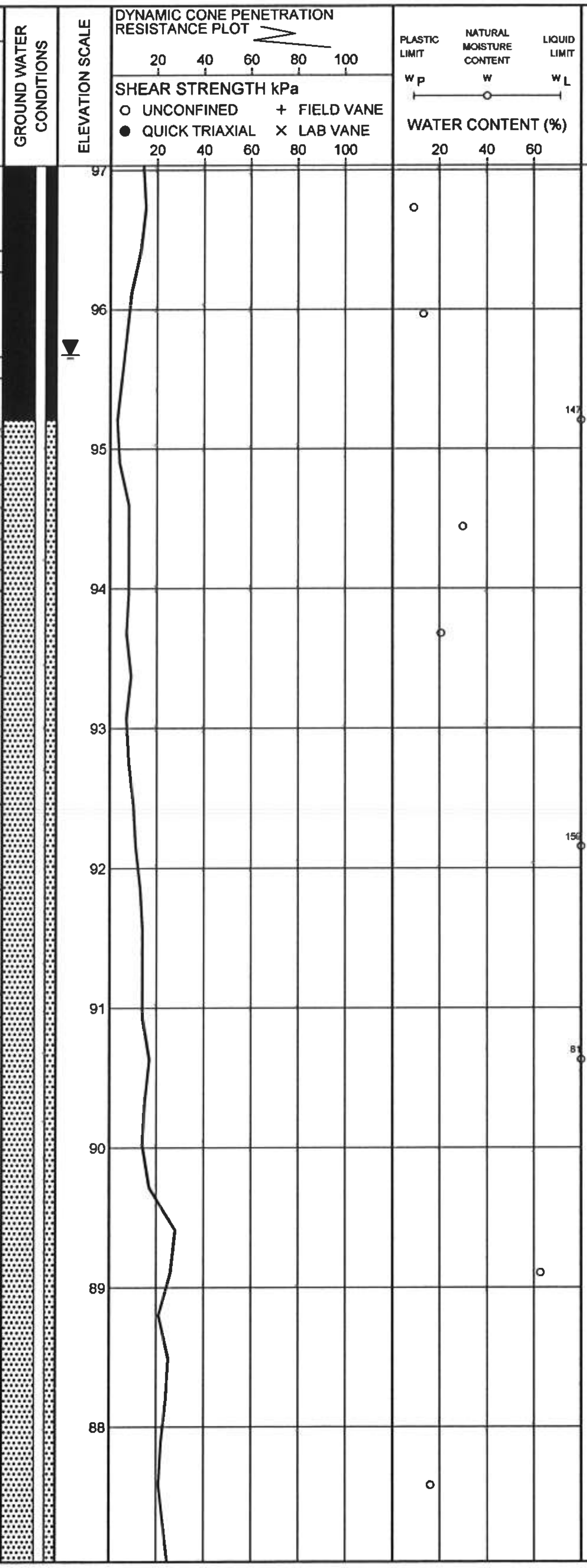
RECORD OF BOREHOLE No HCB-01

1 OF 3

METRIC

W.P.	6045-08-00	LOCATION	N 101 17.0 E 9 976.5 Hawkeye Creek Bridge	ORIGINATED BY	GA
HWY	589	BOREHOLE TYPE	Hollow Stem Augers/NQ Coring	COMPILED BY	AN
DATUM	Geodetic	DATE	2011.07.15 - 2011.07.15	CHECKED BY	LRB

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL
97.0 0.0	**SAND**, some silt, trace clay, trace gravel Compact Brown Damp (FILL)	1	SS	20				
		2	SS	16	7	67	23	3
95.5 1.5	**ORGANIC SILT**, occasional rootlets Loose Dark Brown to Black	3	SS	4				
94.7 2.3	**SAND** Loose to Compact Brown Wet	4	SS	6				
		5	SS	12				
92.4 4.6	**ORGANIC SILT**, sandy, trace clay Loose Dark Brown to Black	6	SS	4				
		7	SS	4				
89.5 7.5	Sandy **SILT**, trace clay Very Loose to Loose Grey Damp	8	SS	2	0	24	71	5
	Compact	9	SS	9				



UNIT WEIGHT γ kN/m³

REMARKS & GRAIN SIZE DISTRIBUTION (%)

Continued Next Page

+³, ×³: Numbers refer to Sensitivity

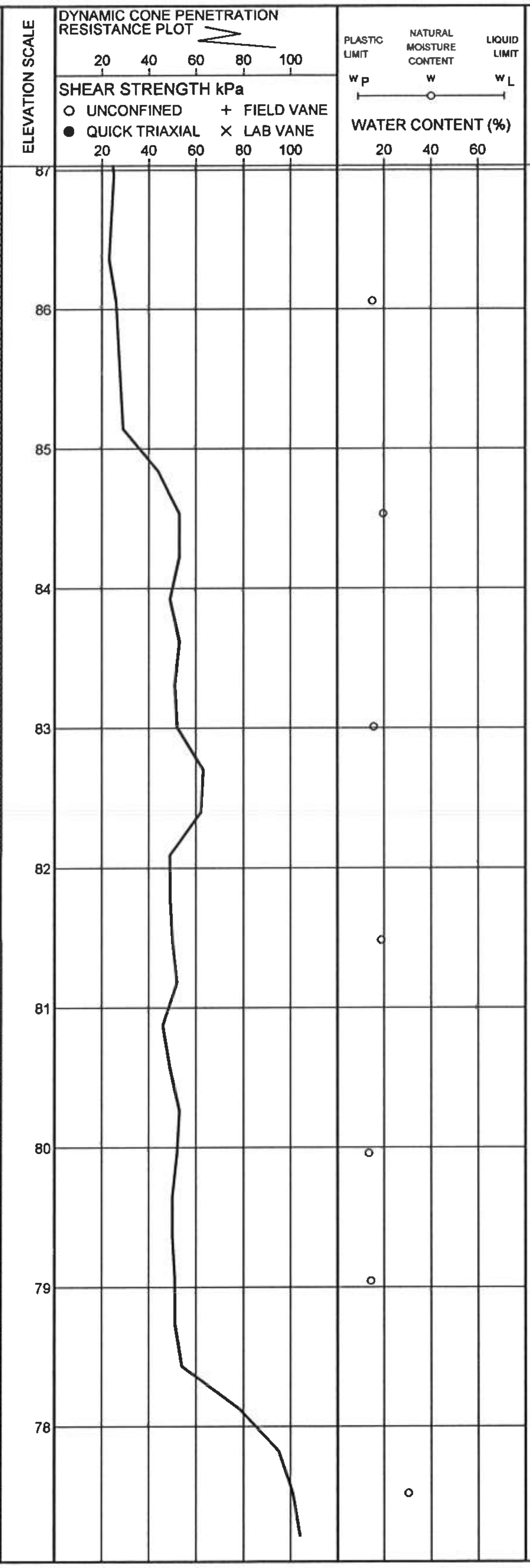
20 / 15 / 5 / 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No HCB-01

2 OF 3 **METRIC**

W.P.	6045-08-00	LOCATION	N 101 17.0 E 9 976.5 Hawkeye Creek Bridge	ORIGINATED BY	GA
HWY	589	BOREHOLE TYPE	Hollow Stem Augers/NQ Coring	COMPILED BY	AN
DATUM	Geodetic	DATE	2011.07.15 - 2011.07.15	CHECKED BY	LRB

SOIL PROFILE		SAMPLES			REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL
	Continued From Previous Page							
	Sandy **SILT**, trace clay Compact Grey Wet							
		10	SS	16	0	24	72	4
		11	SS	15				
		12	SS	16				
		13	SS	13	0	33	63	4
		14	SS	27				
		15	SS	23				
	Dense	16	SS	44				
77.0								



UNIT WEIGHT γ kN/m³

Continued Next Page

+³, ×³: Numbers refer to Sensitivity

20
15 ⊕ 5
10
(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No HCB-01

3 OF 3

METRIC

W.P. 6045-08-00
LOCATION N 101 17.0 E 9 976.5 Hawkeye Creek Bridge
ORIGINATED BY GA
HWY 589
BOREHOLE TYPE Hollow Stem Augers/NQ Coring
COMPILED BY AN
DATUM Geodetic
DATE 2011.07.15 - 2011.07.15
CHECKED BY LRB

ELEV DEPTH	SOIL PROFILE: DESCRIPTION	SAMPLES: NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	UNIT WEIGHT kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	Continued From Previous Page							
20.0	**SAND** and **GRAVEL**, some silt and clay Dense Grey Wet				77			
		17	SS	42	76	o		51 35 14 (SI+CL)
					75			
					74			
		18	SS	50/ 0.00				
72.9					73		FI	
24.1	**BEDROCK DIORITE**, highly weathered, grey, occasional mechanical and sub-vertical breaks Coring started at 24.1m						>5 >10	
	Highly broken zone from 24.2m to 24.6m and 24.9m to 25.2m Horizontal joints at 24.1m	1	RUN		72		3 >10 0	RUN #1 TCR=100% SCR=30% RQD=25% UCS=12MPa (Average)
	Highly broken zone from 25.6m to 25.9m and 25.9m to 26.0m				71		>10 >5	
	Fresh	2	RUN				0 0 0	RUN #2 TCR=100% SCR=68% RQD=68% UCS=52MPa (Average)
69.9					70			
27.1	END OF BOREHOEL AT 27.1m. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 3.05m slotted screen.							

STRAT PLOT; 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

WATER LEVEL READINGS:

DATE	DEPTH (m)	ELEV. (m)
Aug.17/11	1.4	95.6

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

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

Ministry of Transportation
Ontario

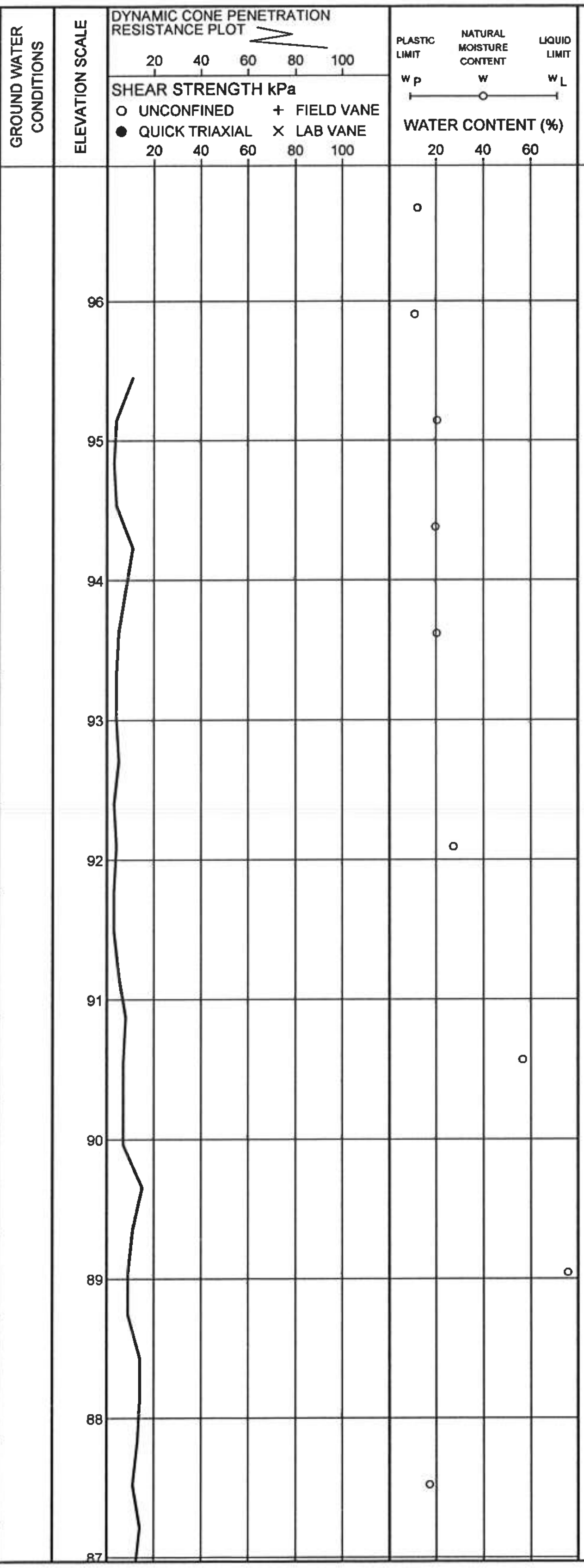
RECORD OF BOREHOLE No HCB-02

1 OF 3

METRIC

W.P. 6045-08-00	LOCATION N 101 17.4 E 9 981.1 Hawkeye Creek Bridge		ORIGINATED BY GA
HWY 589	BOREHOLE TYPE Hollow Stem Augers		COMPILED BY AN
DATUM Geodetic	DATE 2011.07.17 - 2011.07.17		CHECKED BY LRB

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLES NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR	SA	SI	CL
96.9								
0.0	**SAND and GRAVEL** Dense to Compact Brown Damp (FILL)	1	SS	35				
		2	SS	11				
95.4 1.5	**SAND**, some gravel, some silt and clay Loose Grey Wet	3	SS	9	28	52	20	(SI+CL)
	Occasional wood fibres	4	SS	8				
		5	SS	3				
	Occasional cobbles	6	SS	4	0	90	10	(SI+CL)
	Layer of organics from 6.0m to 6.7m	7	SS	4				
89.3 7.6	**SILT**, trace sand, trace clay Very Loose Grey Wet	8	SS	2				
88.0 9.0	**SAND** Loose Grey Wet	9	SS	8				



UNIT WEIGHT γ kN/m³

Continued Next Page

+³, ×³: Numbers refer to Sensitivity

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

RECORD OF BOREHOLE No HCB-02

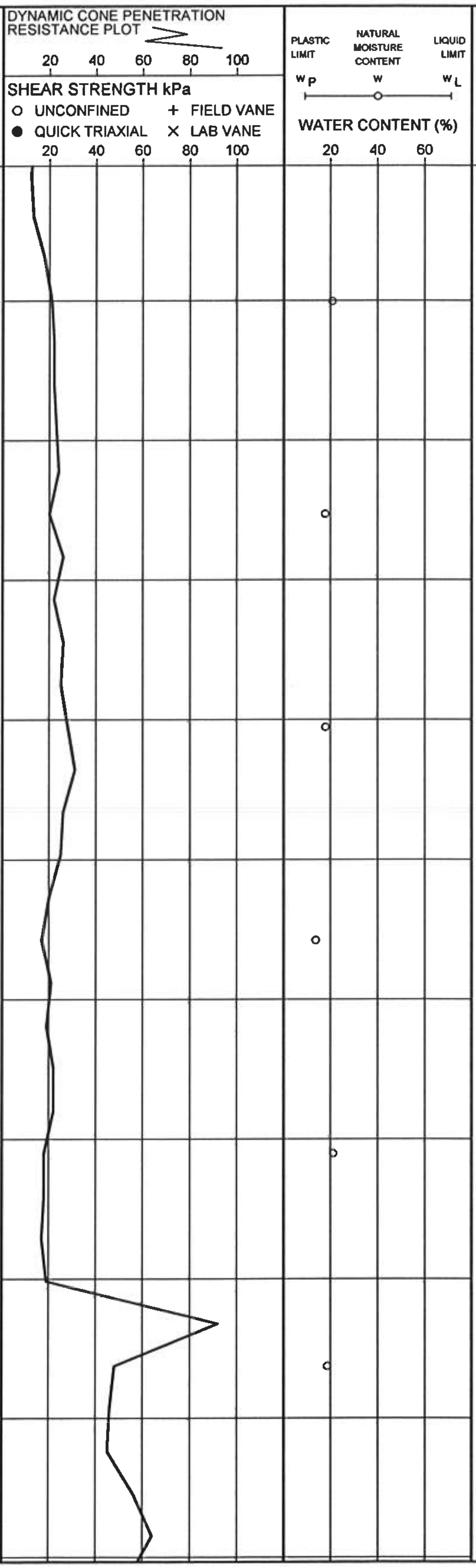
2 OF 3

METRIC

W.P. 6045-08-00	LOCATION N 101 17.4 E 9 981.1 Hawkeye Creek Bridge		ORIGINATED BY GA
HWY 589	BOREHOLE TYPE Hollow Stem Augers		COMPILED BY AN
DATUM Geodetic	DATE 2011.07.17 - 2011.07.17		CHECKED BY LRB

Continued From Previous Page

ELEV DEPTH	SOIL PROFILE DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	SAMPLES TYPE	SAMPLES "N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR	SA	SI	CL
	SAND, trace to some silt, trace clay Compact Grey Wet											
			10	SS	11		86					
							85					
			11	SS	12							
							84					
83.2 13.7	**SILT**, trace sand, trace clay Compact Grey Wet		12	SS	16		83		0	2	94	4
							82					
81.7 15.2	**SAND**, trace gravel, trace silt and clay Compact Grey Wet		13	SS	22							
							81					
			14	SS	28		80		6	84	10 (SI+CL)	
							79					
			15	SS	22							
							78					
	Cobbles and boulders from 19.5m to 21.0m		16	SS	50/		77					



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+³, ×³: Numbers refer to Sensitivity

20
15-⊕-5 (%) STRAIN AT FAILURE
10

Ministry of Transportation
Ontario

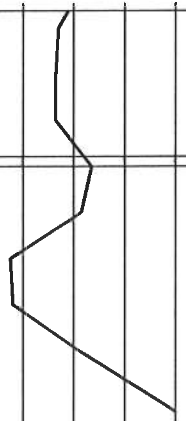
RECORD OF BOREHOLE No HCB-02

3 OF 3

METRIC

W.P. 6045-08-00	LOCATION N 101 17.4 E 9 981.1 Hawkeye Creek Bridge		ORIGINATED BY GA
HWY 589	BOREHOLE TYPE Hollow Stem Augers		COMPILED BY AN
DATUM Geodetic	DATE 2011.07.17 - 2011.07.17		CHECKED BY LRB

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 (%)			
ELEV DEPTH	DESCRIPTION Continued From Previous Page	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE					GR	SA	SI	CL
	SAND, trace gravel, trace clay, cobbles and boulders Compact Grey Wet				0.00										
75.9 21.0	END OF BOREHOLE AT 21.0m UPON AUGER REFUSAL ON PROBABLE BEDROCK OR BOULDERS. BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG FROM 20.4m TO 2.1m, THEN SAND AND GRAVEL TO SURFACE.						76								



+³, ×³: Numbers refer to Sensitivity

20
15 ⊕ 5
10
(%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

RECORD OF BOREHOLE No HCB-03

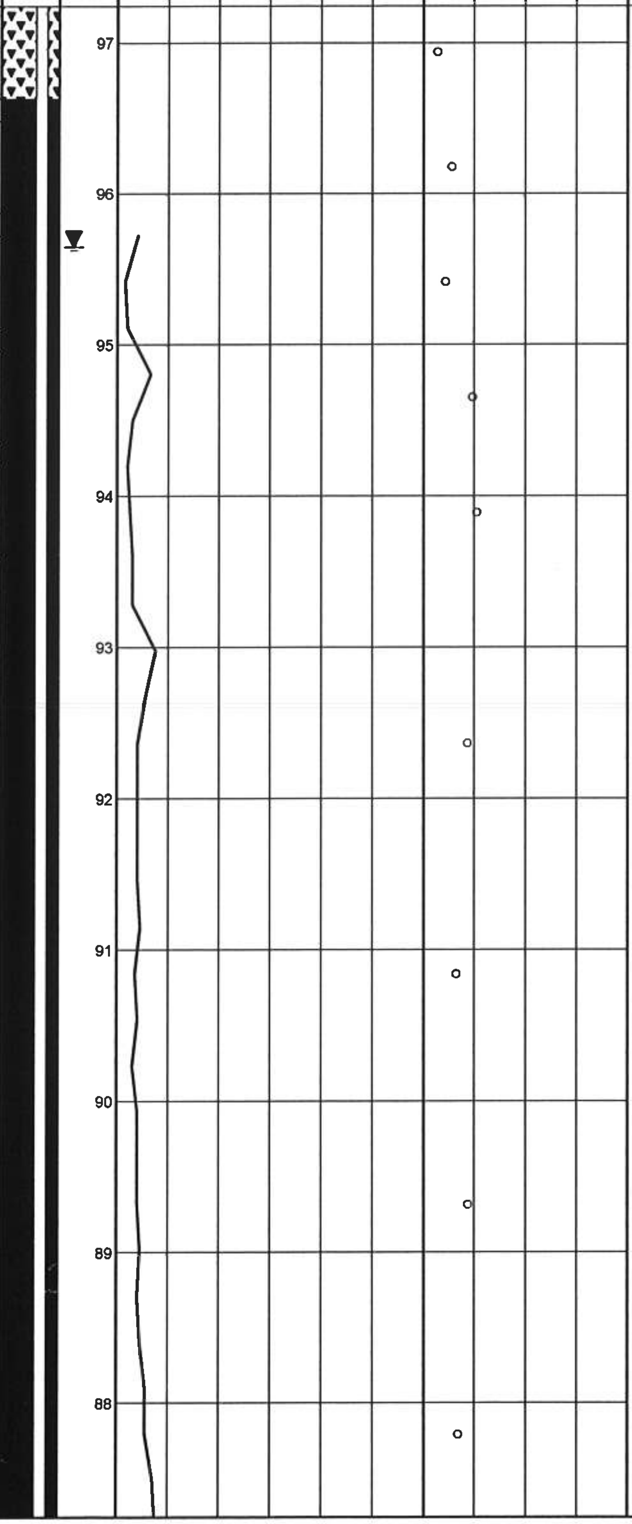
1 OF 3

METRIC

W.P.	6045-08-00	LOCATION	N 101 05.7 E 9 981.3 Hawkeye Creek Bridge	ORIGINATED BY	JM
HWY	589	BOREHOLE TYPE	Hollow Stem Augers/NQ Coring	COMPILED BY	AN
DATUM	Geodetic	DATE	2011.07.18 - 2011.07.19	CHECKED BY	LRB

ELEV DEPTH	SOIL PROFILE DESCRIPTION
97.2 0.0	**SAND** and **GRAVEL** Loose to Compact Brown Moist (FILL) Trace silt Brown to Dark Brown Moist to Wet
94.3 2.9	**SAND**, trace gravel, some silt, trace clay, occasional organics Very Loose Dark Brown Wet
92.6 4.6	**SAND** and **GRAVEL**, trace silt and clay Loose to Compact Dark Brown Wet Grey

SAMPLE NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	AS		
1	SS	12	
2	SS	4	
3	SS	8	
4	SS	3	7 77 13 3
5	SS	8	
6	SS	12	45 54 1 (SI+CL)
7	SS	5	
8	SS	5	



UNIT WEIGHT γ kN/m³

Continued Next Page

+³, ×³: Numbers refer to Sensitivity

20 15-⊕-5 10 (%) STRAIN AT FAILURE

ONTMT4S 0840.GPJ 9/29/11

Ministry of Transportation
Ontario

RECORD OF BOREHOLE No HCB-03

2 OF 3 **METRIC**

W.P. 6045-08-00	LOCATION N 101 05.7 E 9 981.3 Hawkeye Creek Bridge		ORIGINATED BY JM
HWY 589	BOREHOLE TYPE Hollow Stem Augers/NQ Coring		COMPILED BY AN
DATUM Geodetic	DATE 2011.07.18 - 2011.07.19		CHECKED BY LRB

Continued From Previous Page

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLES NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	SAND and **GRAVEL**, trace silt and clay Loose to Compact Grey Wet	9	SS	7	63 34 3 (SI+CL)
		10	SS	10	
83.5 13.7	**SAND**, trace gravel, trace silt Compact Grey Wet	11	SS	11	
82.0 15.2	**SAND** and **GRAVEL**, trace silt and clay Compact to Very Dense Grey Wet	12	SS	24	
	Cobbles and boulders	13	SS	55	64 28 8 (SI+CL)
		14	SS	47	
	Cobbles and boulders from 19.8m to 21.1m	15	SS	100/	

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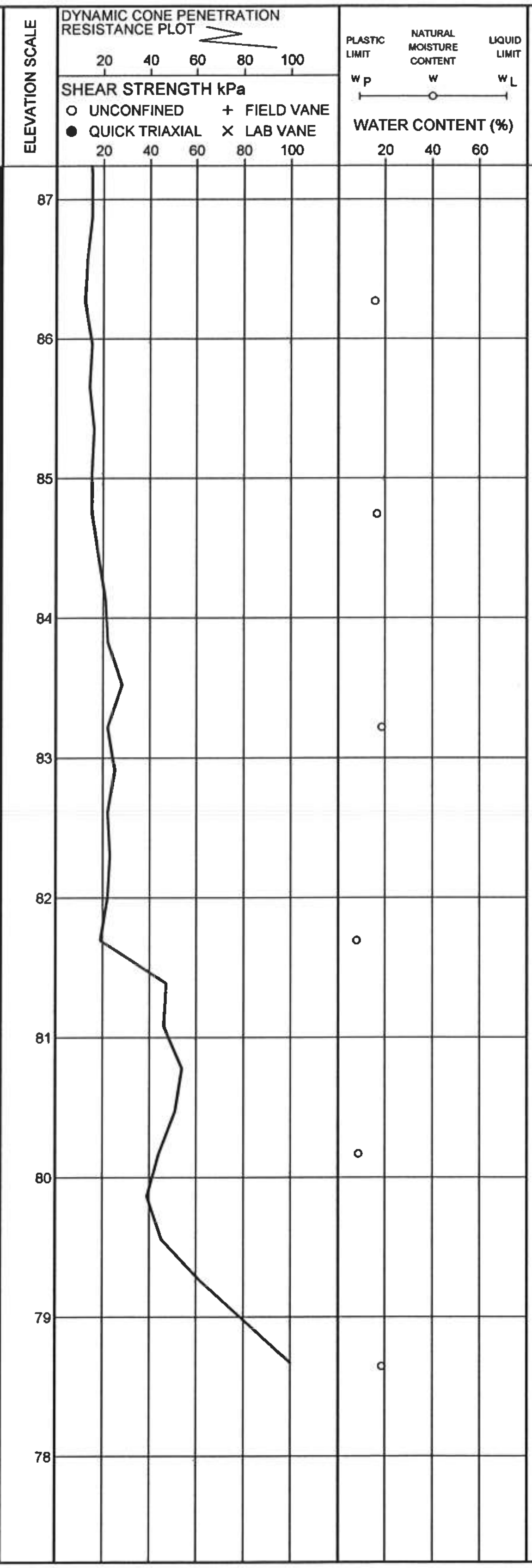


UNIT WEIGHT γ kN/m³

+³, ×³: Numbers refer to Sensitivity

20 15-⊕-5 10 (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

RECORD OF BOREHOLE No HCB-03

3 OF 3

METRIC

W.P.	6045-08-00	LOCATION	N 101 05.7 E 9 981.3 Hawkeye Creek Bridge
HWY	589	BOREHOLE TYPE	Hollow Stem Augers/NQ Coring
DATUM	Geodetic	DATE	2011.07.18 - 2011.07.19

ORIGINATED BY JM
COMPILED BY AN
CHECKED BY LRB

ELEV DEPTH	DESCRIPTION (Continued From Previous Page)	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	SAND and GRAVEL, trace silt and clay Compact Grey Wet			0.025	77			
	Cobbles and boulders from 19.8m to 21.1m				76			
75.3		16	SS	23		o		
21.9	**BEDROCK DIORITE**, moderately to slightly weathered, grey, occasional mechanical and sub-vertical breaks Coring started at 2.2m Horizontal breaks at 22.3m, 22.7m, 22.8m, 22.9m, 23.1m, 23.2m, 23.4m Sub-vertical breaks: 150mm at 22.5m 100mm at 22.6m 100mm at 23.4m	1	RUN		75 74		FI 8 6 5	RUN #1 TCR=76% SCR=71% RQD=33% UCS=163MPa (Average)
	Sub-vertical breaks (25mm to 50mm thick) at 23.8m, 23.9m, 24.0m, 24.1m, 24.4m and 24.5m Bedrock, granite at 24.0m, white and pink, slightly weathered to fresh Horizontal breaks at 25.0m, 25.1m, 25.2m and 25.3m	2	RUN		73		4 3 4 1	RUN #2 TCR=100% SCR=100% RQD=63% UCS=105MPa (Average)
71.8	125mm thick sub-vertical breaks at 24.9m	3	RUN		72		3 8 4	RUN #3 TCR=100% SCR=87% RQD=44% UCS=102MPa (Average)
25.5	END OF BOREHOLE AT 25.5m. BOREHOLE OPEN TO 25.5m AND WATER LEVEL AT 2.1m. 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 ● QUICK TRIAXIAL + FIELD VANE × 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)
Aug.17/11	1.6	95.6

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

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

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No HCB-04

1 OF 3

METRIC

W.P.	6045-08-00	LOCATION	N 101 05.3 E 9 978.1 Hawkeye Creek Bridge	ORIGINATED BY	GA
HWY	589	BOREHOLE TYPE	Hollow Stem Augers	COMPILED BY	AN
DATUM	Geodetic	DATE	2011.07.21 - 2011.07.23	CHECKED BY	LRB

ELEV DEPTH	SOIL PROFILE: DESCRIPTION	STRAT PLOT	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: "N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
97.2 0.0	**SAND and GRAVEL** Brown Moist (FILL)	☒	1	AS			
	Compact to Loose	☒	1	SS	13		
		☒	2	SS	9	∇	
94.9 2.3	**SAND and GRAVEL**, trace silt and clay, occasional organics Loose Dark Brown Wet	⁘	3	SS	9		32 61 7 (SI+CL)
		⁘	4	SS	9		
	Very Loose	⁘	5	SS	4		
	Compact	⁘	6	SS	21		
	Grey	⁘	7	SS	13		
	Layer of sand, some gravel at 8.5m	⁘	8	SS	14		18 79 3 (SI+CL)

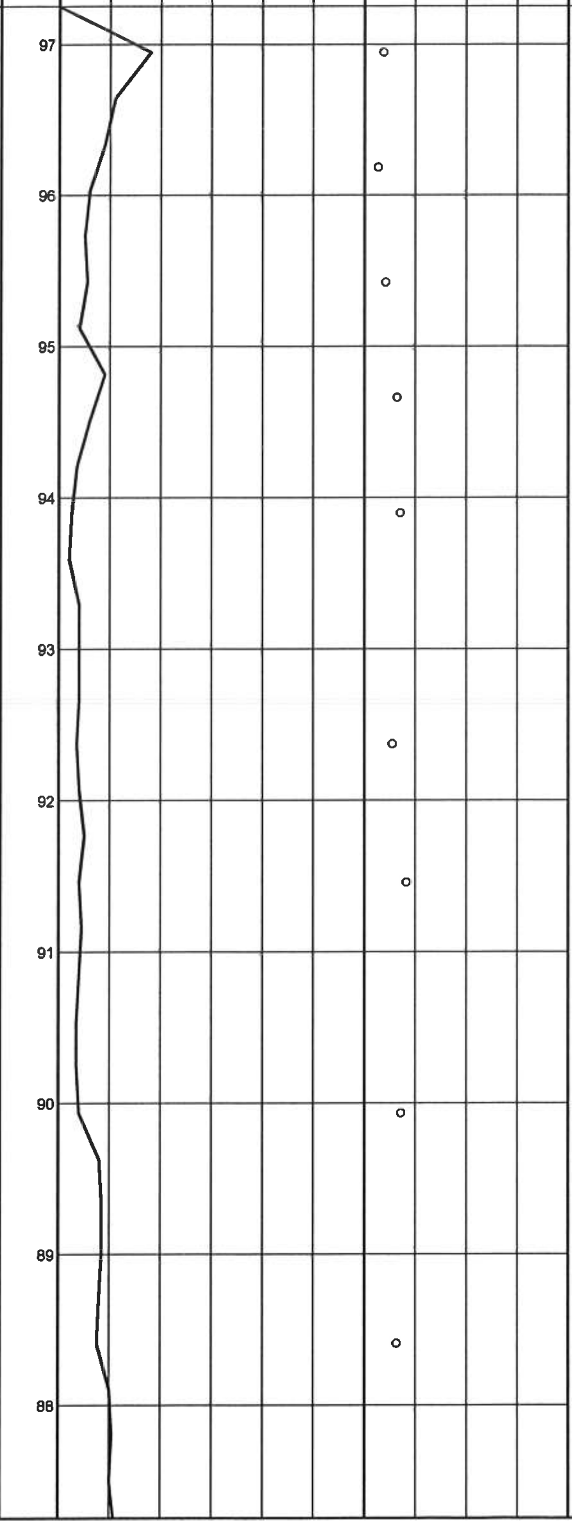
DYNAMIC CONE PENETRATION RESISTANCE PLOT
20 40 60 80 100

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



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+³, ×³: Numbers refer to Sensitivity

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

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No HCB-04

2 OF 3

METRIC

W.P.	6045-08-00	LOCATION	N 101 05.3 E 9 978.1 Hawkeye Creek Bridge	ORIGINATED BY	GA
HWY	589	BOREHOLE TYPE	Hollow Stem Augers	COMPILED BY	AN
DATUM	Geodetic	DATE	2011.07.21 - 2011.07.23	CHECKED BY	LRB

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

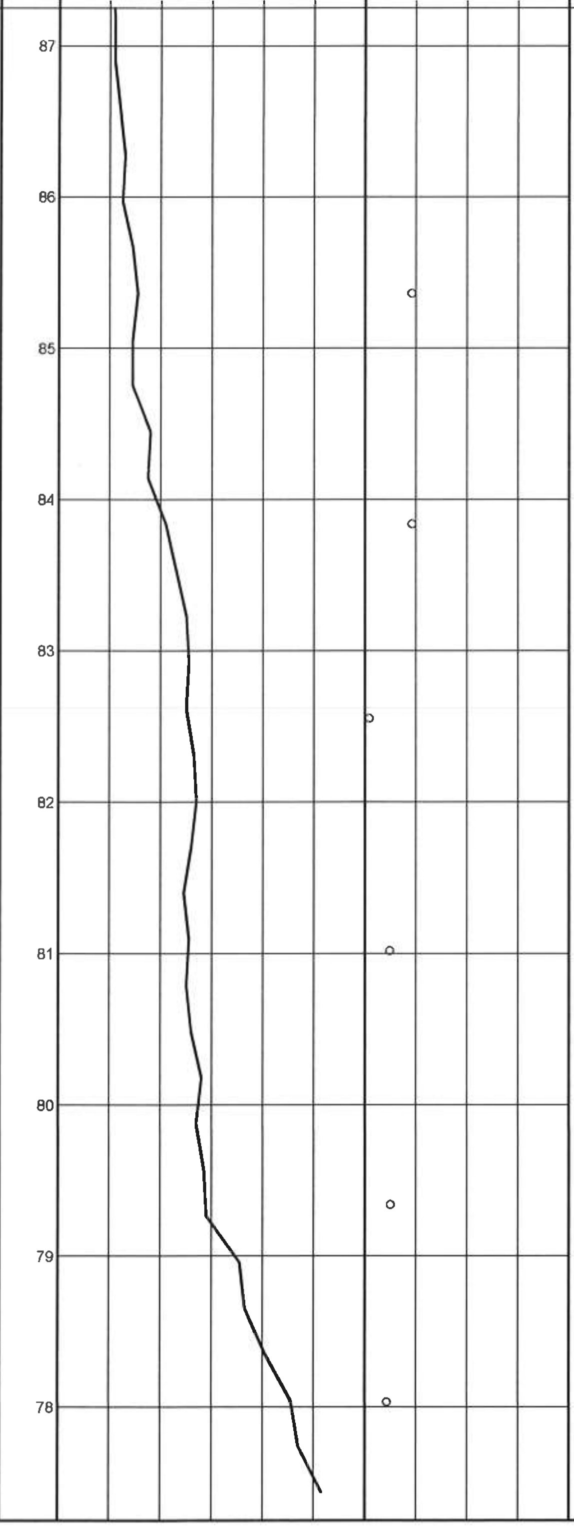
PLASTIC LIMIT w_P; NATURAL MOISTURE CONTENT w; LIQUID LIMIT w_L; WATER CONTENT (%): 20 40 60

UNIT WEIGHT γ kN/m³

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

Continued From Previous Page

DESCRIPTION	NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL
SAND and **GRAVEL**, trace silt and clay Compact to Loose Grey Wet	9	SS	10				
	10	SS	5				
Very Loose	11	SS	3				
Cobble at 14.0m Cobble at 14.3m Boulder at 14.6m	12	SS	100/ 0.125				
Boulder at 15.2m Boulder at 15.5m	13	SS	100/0.150	37	53	10 (SI+CL)	
Boulder at 17.1m	14	SS	100/ 0.150				
Boulder at 18.6m	~~15~~	~~SS~~	~~100/~~ 0.025				



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+³, ×³: Numbers refer to Sensitivity

20
15-⊕-5 (%) STRAIN AT FAILURE
10

Ministry of Transportation
Ontario

RECORD OF BOREHOLE No HCB-04

3 OF 3

METRIC

W.P. 6045-08-00	LOCATION N 101 05.3 E 9 978.1 Hawkeye Creek Bridge	ORIGINATED BY GA
HWY 589	BOREHOLE TYPE Hollow Stem Augers	COMPILED BY AN
DATUM Geodetic	DATE 2011.07.21 - 2011.07.23	CHECKED BY LRB

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 (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE				GR	SA	SI	CL
	Continued From Previous Page													
77.2 20.1	END OF BOREHOLE AT 20.1m UPON REFUSAL ON PROBABLE BEDROCK OR BOULDER. BOREHOLE OPEN TO 10.9m AND WATER LEVEL AT 1.8m. BOREHOLE BACKFILLED WITH HOLEPLUG FROM 10.9m TO 1.5m, THEN SAND AND GRAVEL TO SURFACE.						77							

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

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

Ministry of Transportation
Ontario

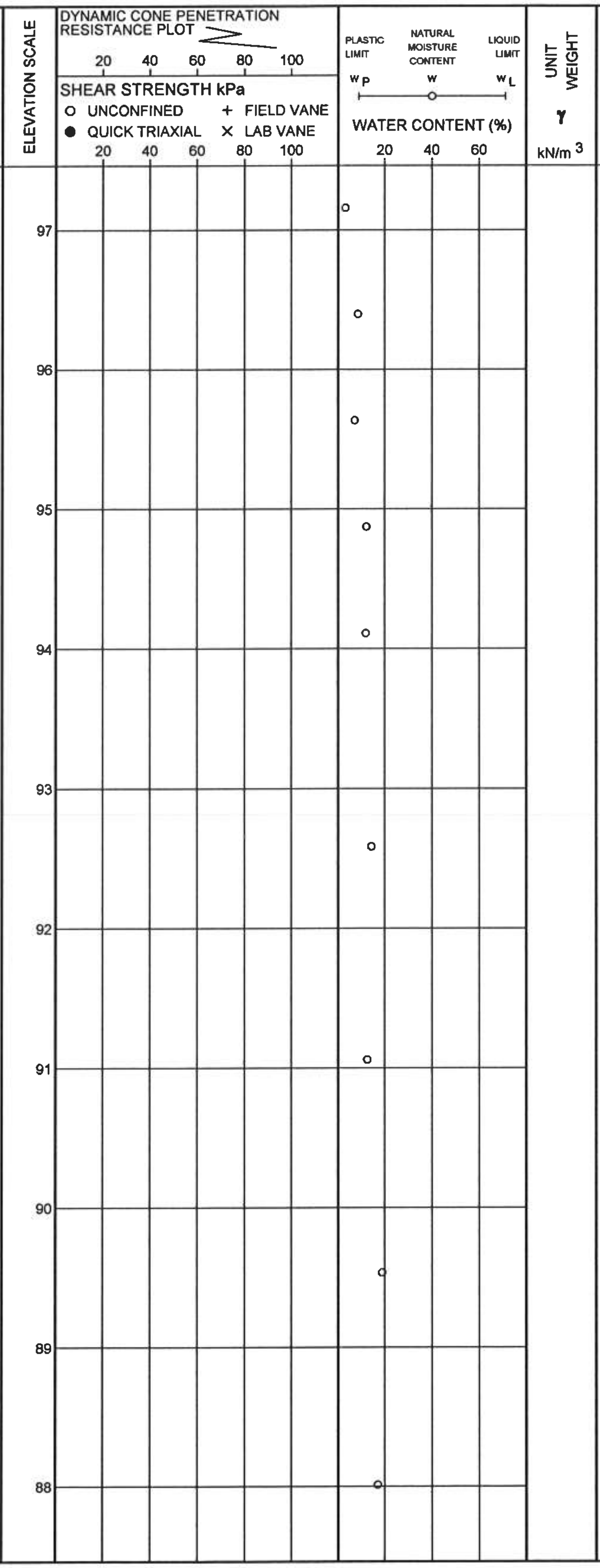
RECORD OF BOREHOLE No HCB-05

1 OF 2

METRIC

W.P.	6045-08-00	LOCATION	N 100 97.8 E 9 983.1 Hawkeye Creek Bridge	ORIGINATED BY	JM
HWY	589	BOREHOLE TYPE	Hollow Stem Augers	COMPILED BY	AN
DATUM	Geodetic	DATE	2011.07.23 - 2011.07.23	CHECKED BY	LRB

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR	SA	SI	CL
97.4									
0.0	**SAND** and **GRAVEL**, some silt and clay Brown Moist (FILL)	1	AS						
	Loose to Compact	1	SS	8					
		2	SS	17		28	56	16 (SI+CL)	
					∇				
		3	SS	20					
		4	SS	8					
93.3 4.1	**SAND**, trace gravel, trace silt and clay Loose Grey Wet								
		5	SS	8					
		6	SS	6		15	81	4 (SI+CL)	
		7	SS	9					
		8	SS	4		0	93	7 (SI+CL)	



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+ 3, × 3: Numbers refer to Sensitivity

20 15 5 10 (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

RECORD OF BOREHOLE No HCB-05

2 OF 2

METRIC

W.P. 6045-08-00	LOCATION N 100 97.8 E 9 983.1 Hawkeye Creek Bridge	ORIGINATED BY JM
HWY 589	BOREHOLE TYPE Hollow Stem Augers	COMPILED BY AN
DATUM Geodetic	DATE 2011.07.23 - 2011.07.23	CHECKED BY LRB

SOIL PROFILE			SAMPLES									
ELEV DEPTH	DESCRIPTION Continued From Previous Page	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
	SAND, trace silt and clay Loose Grey Wet						87				
			9	SS	7				o		
							86				
			10	SS	8		85				
84.6 12.8	END OF BOREHOLE AT 12.8m. BOREHOLE OPEN TO 6.7m AND WATER LEVEL AT 2.3m. BOREHOLE BACKFILLED WITH HOLEPLUG FROM 6.7m TO 1.5m, THEN SAND AND GRAVEL TO SURFACE.										

ONTMT4S 0840.GPJ 9/29/11

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

20 15 5 10 (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

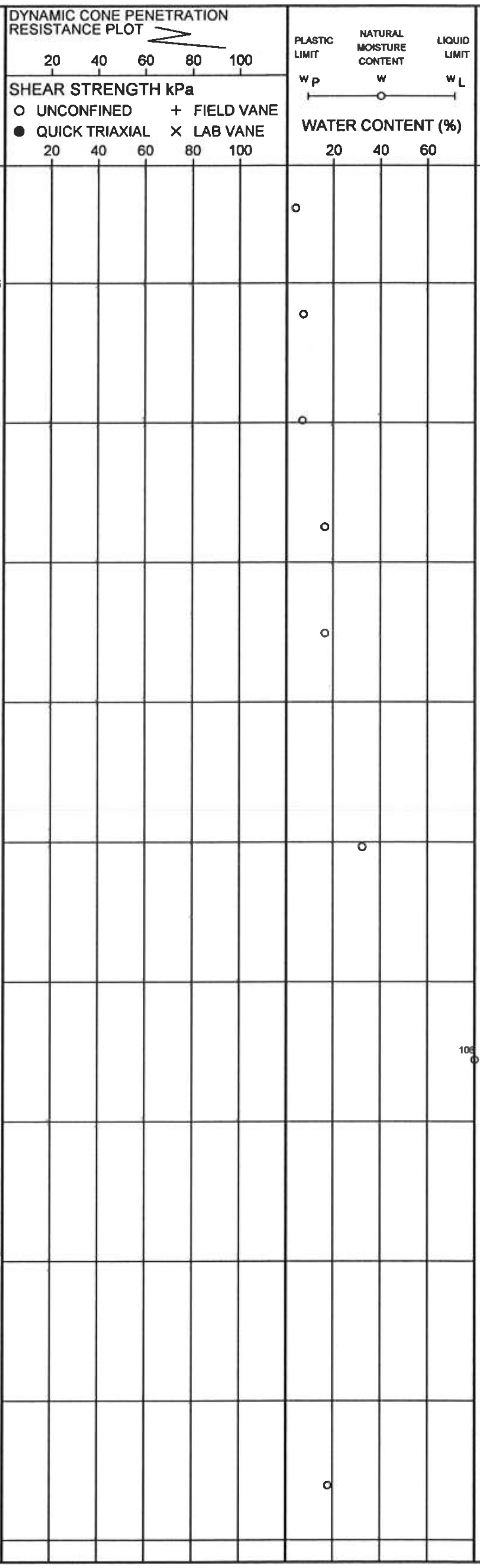
RECORD OF BOREHOLE No HCB-06

1 OF 2

METRIC

W.P. 6045-08-00	LOCATION N 101 25.1 E 9 976.4 Hawkeye Creek Bridge		ORIGINATED BY GA
HWY 589	BOREHOLE TYPE Hollow Stem Augers		COMPILED BY AN
DATUM Geodetic	DATE 2011.07.23 - 2011.07.23		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
96.8									
0.0	**SAND**, trace gravel, trace silt and clay Compact to Dense Brown Moist (FILL)		1	AS					
			1	SS	17		96		
			2	SS	34	▽	95		
	Loose		3	SS	9				8 86 6 (SI+CL)
93.9							94		
2.9	**SAND**, trace gravel, trace silt and clay Loose Grey Wet		4	SS	9				
							93		
			5	SS	5		92		
							91		
90.7									
6.1	**ORGANIC SILT**, sandy, trace clay, occasional roots Very Loose Dark Brown to Black Wet		6	SS	3				0 24 71 5
							90		
89.7									
7.2	**SILT**, trace to some sand, trace clay, occasional roots Very Loose to Loose Grey Wet		7	SS	3		89		
							88		
			8	SS	9				
86.9							87		



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+³, ×³: Numbers refer to Sensitivity

20 15 5 10 (%) STRAIN AT FAILURE

ONTMT4S 0840.GPJ 10/12/11

Ministry of Transportation
Ontario

RECORD OF BOREHOLE No HCB-06

2 OF 2

METRIC

W.P. 6045-08-00	LOCATION N 101 25.1 E 9 976.4 Hawkeye Creek Bridge		ORIGINATED BY GA
HWY 589	BOREHOLE TYPE Hollow Stem Augers		COMPILED BY AN
DATUM Geodetic	DATE 2011.07.23 - 2011.07.23		CHECKED BY LRB

SOIL PROFILE			SAMPLES					DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) SHEAR STRENGTH kPa O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE (20 40 60 80 100)	PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L WATER CONTENT (%) (20 40 60)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION Continued From Previous Page	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE				GR	SA	SI	CL
9.9	**SILT**, some sand, trace clay Loose Grey Wet													
			9	SS	8		86		○		0	14	82	4
85.5 11.3	END OF BOREHOLE AT 11.3m. BOREHOLE OPEN TO 7.0m AND WATER LEVEL AT 1.7m. BOREHOLE BACKFILLED WITH HOLEPLUG FROM 7.0m TO 1.5m, THEN SAND AND GRAVEL TO SURFACE.													

+ ³, × ³ : Numbers refer to Sensitivity

20 15-⊕-5 10 (%) STRAIN AT FAILURE

Appendix B

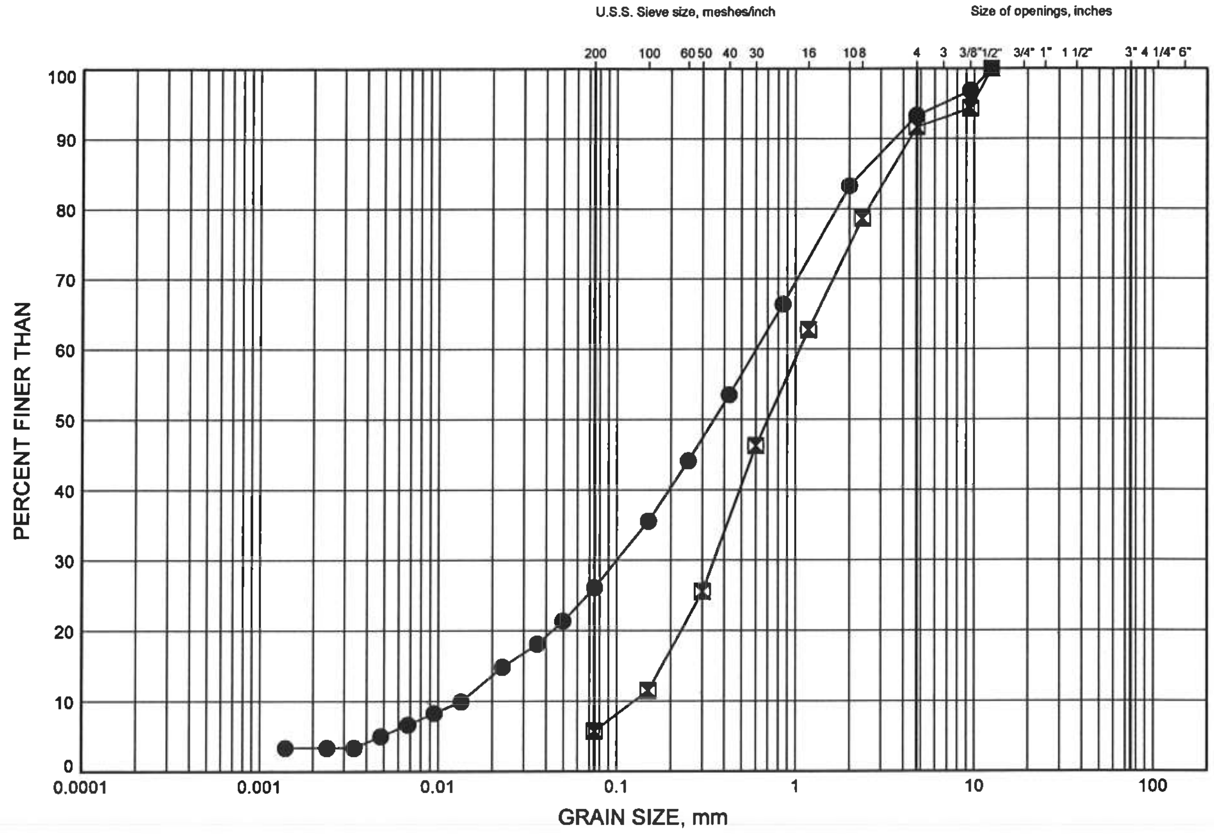
Laboratory Test Results

NWR HWY 11 Bridge

GRAIN SIZE DISTRIBUTION

FIGURE B1

SAND FILL



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	HCB-01	1.07	95.94
☒	HCB-06	2.59	94.23

W.P.# 6045-08-00
Prepared By AN
Checked By RPR

THURBER

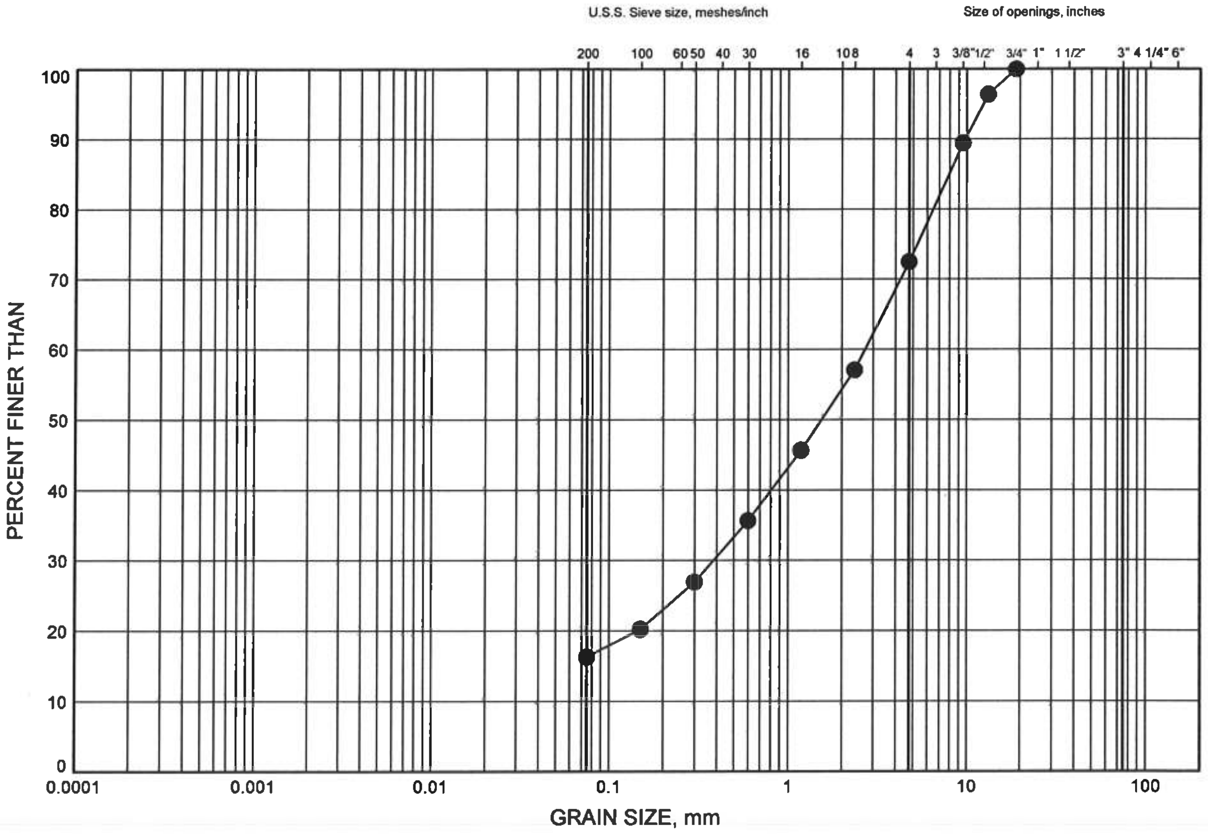
GRAIN SIZE DISTRIBUTION - THURBER 0840.GPJ 10/5/11

NWR HWY 11 Bridge

GRAIN SIZE DISTRIBUTION

FIGURE B2

SAND & GRAVEL FILL



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	HCB-05	1.83	95.61

THURBER

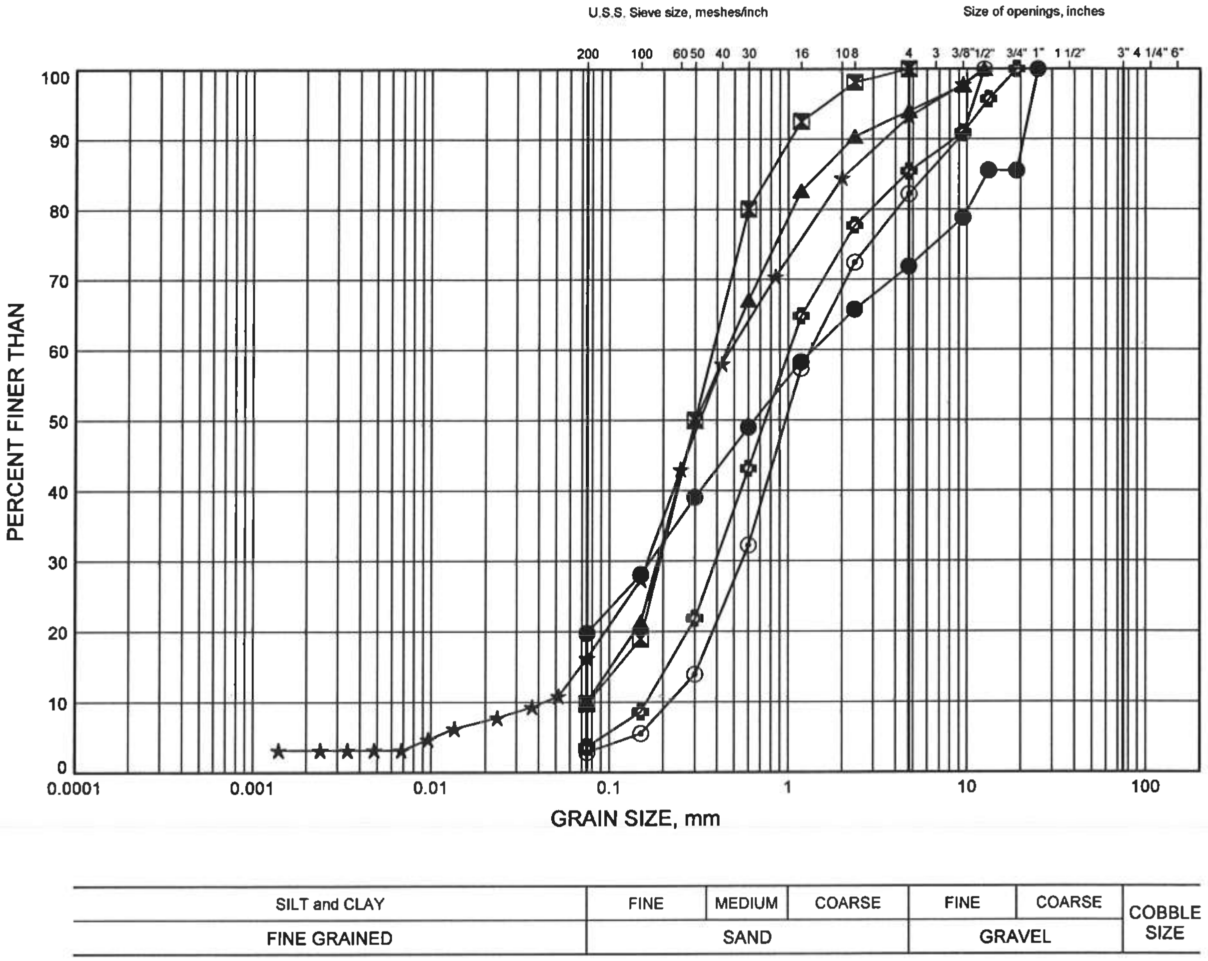
W.P.# 6045-08-00
Prepared By AN
Checked By RPR

NWR HWY 11 Bridge

GRAIN SIZE DISTRIBUTION

FIGURE B3

SAND



LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	HCB-02	1.83	95.12
☒	HCB-02	4.88	92.07
▲	HCB-02	17.07	79.88
★	HCB-03	3.35	93.87
⊙	HCB-04	8.84	88.39
✜	HCB-05	6.40	91.04

W.P.# 6045-08-00

Prepared By AN

Checked By RPR

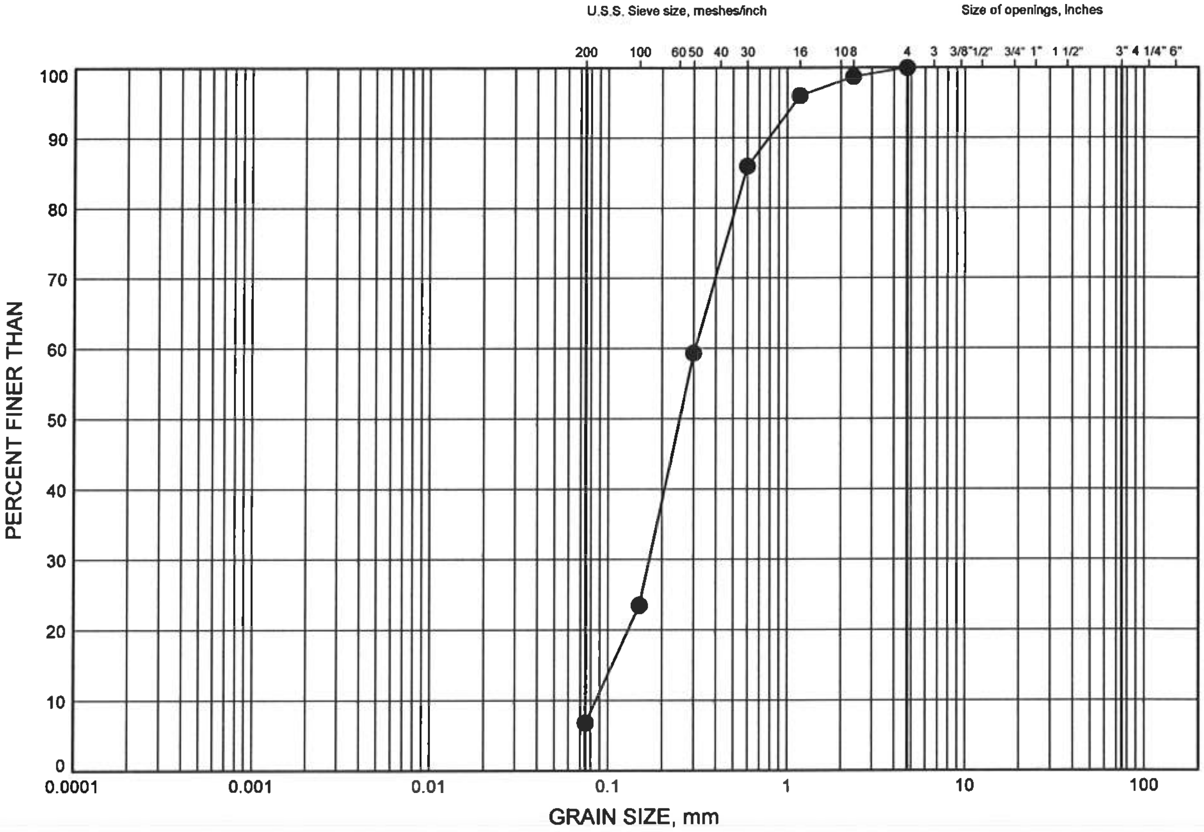
THURBER

NWR HWY 11 Bridge

GRAIN SIZE DISTRIBUTION

FIGURE B4

SAND



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	HCB-05	9.45	87.99

W.P.# 6045-08-00

Prepared By AN

Checked By RPR

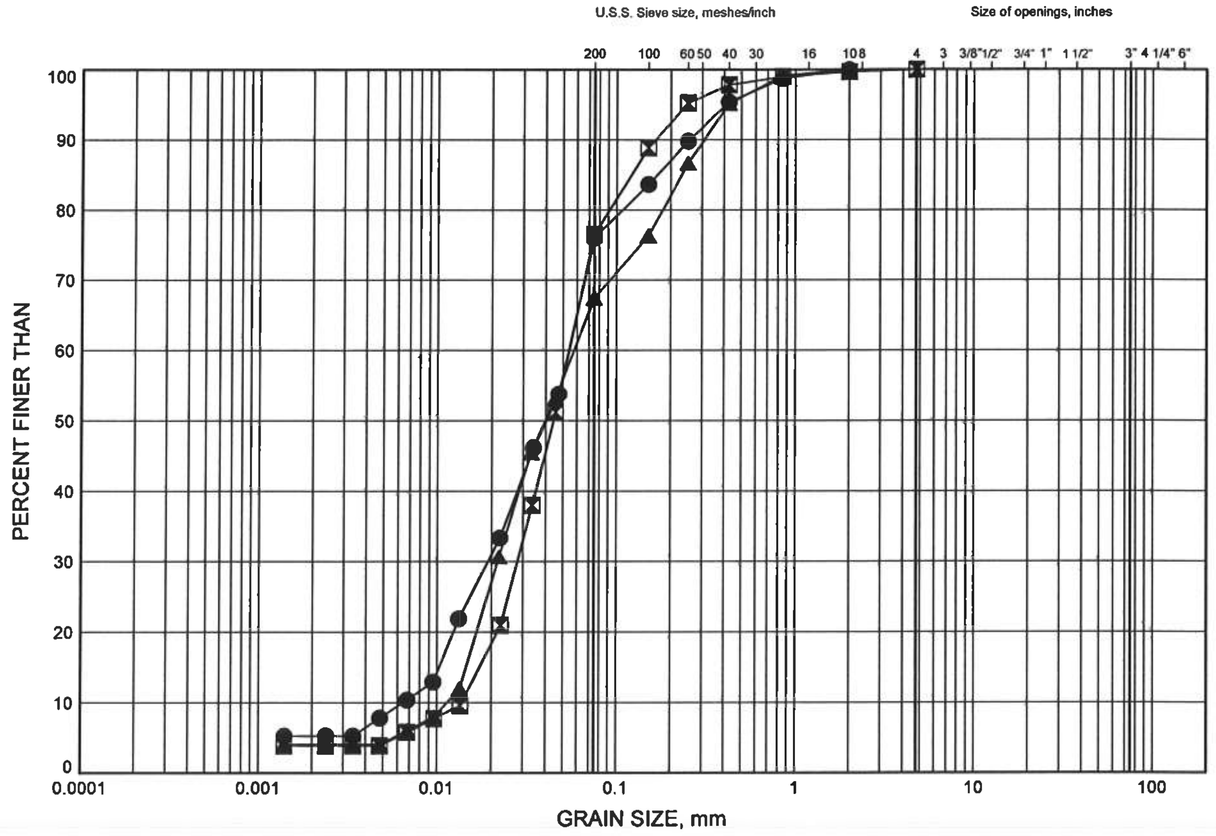
GRAIN SIZE DISTRIBUTION - THURBER 0840.GPJ 10/5/11

NWR HWY 11 Bridge

GRAIN SIZE DISTRIBUTION

FIGURE B5

SANDY SILT



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	HCB-01	7.92	89.08
☒	HCB-01	10.97	86.04
▲	HCB-01	15.54	81.46

W.P.# 6045-08-00

Prepared By AN

Checked By RPR

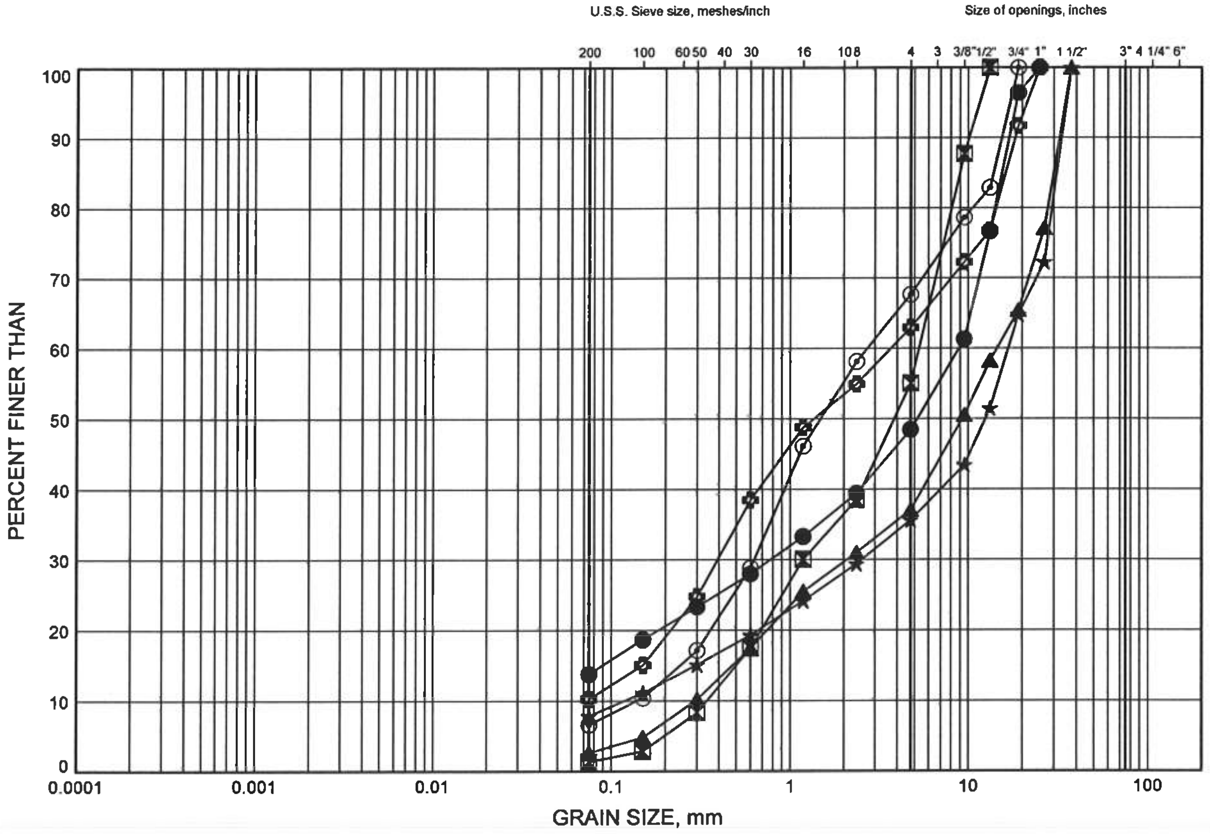
THURBER

NWR HWY 11 Bridge

GRAIN SIZE DISTRIBUTION

FIGURE B6

SAND & GRAVEL



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	HCB-01	21.03	75.98
☒	HCB-03	6.40	90.82
▲	HCB-03	10.97	86.25
★	HCB-03	17.07	80.15
⊙	HCB-04	2.59	94.64
✠	HCB-04	16.23	81.00

THURBER

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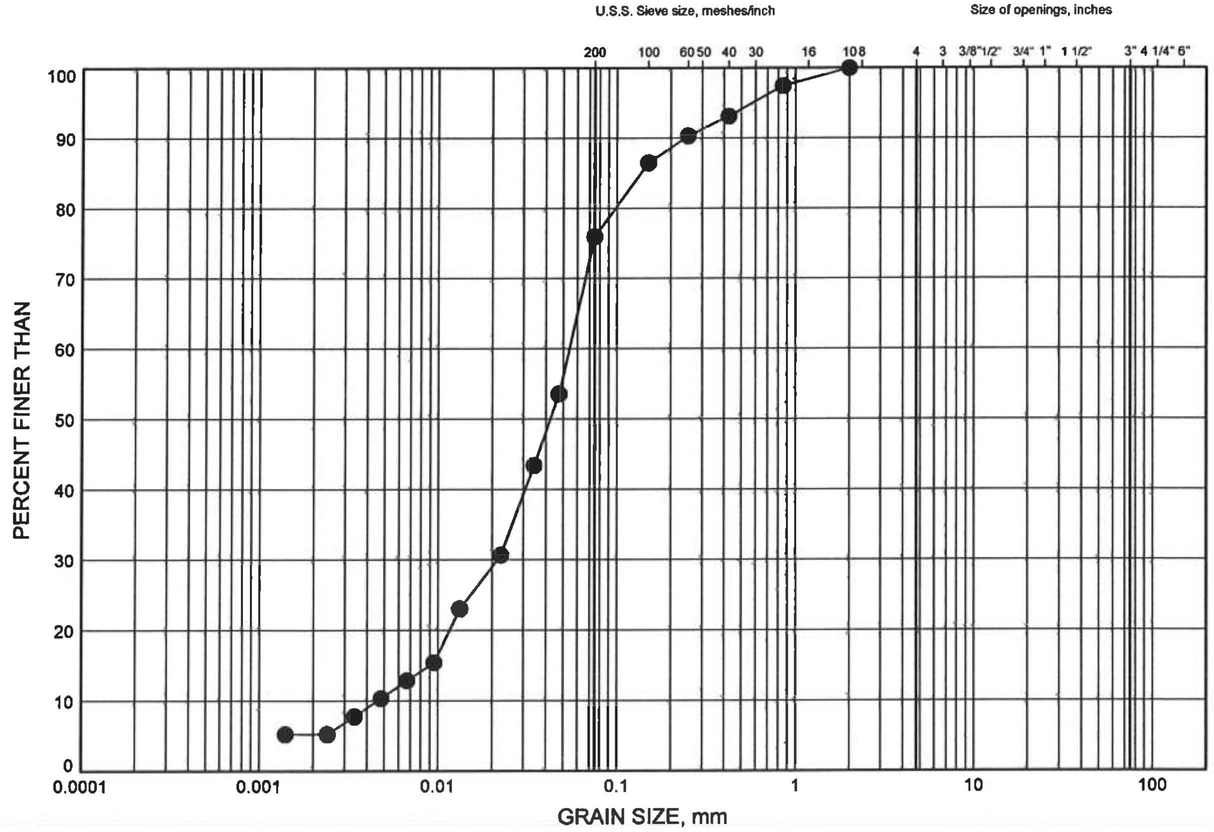
GRAIN SIZE DISTRIBUTION - THURBER 0840.GPJ 10/5/11

NWR HWY 11 Bridge

GRAIN SIZE DISTRIBUTION

FIGURE B7

ORGANIC SILT



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	HCB-06	6.40	90.42

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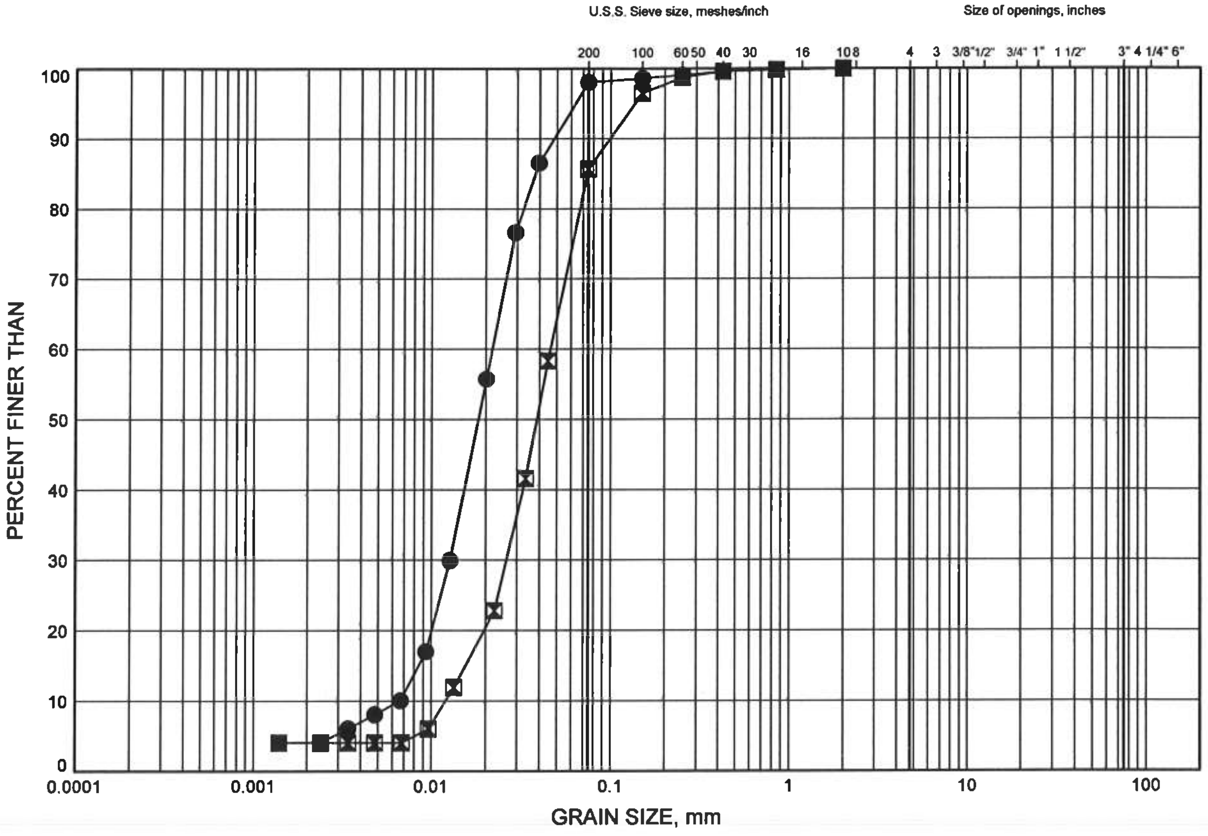
GRAIN SIZE DISTRIBUTION - THURBER 0840.GPJ 10/12/11

NWR HWY 11 Bridge

GRAIN SIZE DISTRIBUTION

FIGURE B8

SILT



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	HCB-02	14.02	82.93
☒	HCB-06	10.97	85.85

W.P.# 6045-08-00
Prepared By AN
Checked By RPR

THURBER

THURBER ENGINEERING LTD.
GEOTECHNICAL • ENVIRONMENTAL • MATERIALS

POINT LOAD TEST SHEET

Job No :	19-5308-40	**Client :**	GENIVAR
		Date Drilled :	July 15,2011
Project Name :	Hawkeye Creek Bridge	**Date Tested :**	9/8/2011
Core Size :	NQ **BH No :** HCB-01	**Tester :**	DB

Test No.	Run No.	Depth (m)	Axial or Diametral	Force (kN)	Diameter (mm)	Length (mm)	UCS (MPa)	Rock Type	Notes
1	1	25.5	A	1.5	47.1	50.8	12.5	Diorite	Weak
2	2	26.4	D	4.3	47.2	69.8	44.8	Diorite	Medium Strong
3	2	26.9	D	5.7	47.2	59.5	59.9	Diorite	Strong
4									
5									
6									
7									
8									
9									
10									
11									
12									
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16									
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20									
21									
22									
23									
24									
25									
26									
27									
28									
29									
30									

* It is ideal to perform axial test on core specimens with D/L ratio of 1.1 ± 0.1
Long pieces of core can be tested diametrically to produce suitable lengths for axial testing

* Diametral Test should have 0.7 x D on either side of test point.

THURBER ENGINEERING LTD.
GEOTECHNICAL • ENVIRONMENTAL • MATERIALS

POINT LOAD TEST SHEET

Job No : 19-5308-40
Client : GENIVAR
Date Drilled : July 19,2011
Project Name : Hawkeye Creek Bridge
Date Tested : 7/29/2011
Core Size : NQ **BH No :** HCB-03
Tester : MAT

Test No.	Run No.	Depth (m)	Axial or Diametral	Force (kN)	Diameter (mm)	Length (mm)	UCS (MPa)	Rock Type	Notes
1	1	22.9	D	18.6	47.5	76.4	193.7	Diorite	Very Strong
2	1	23.3	A	20.0	47.4	66.0	133.8	Diorite	Very Strong
3	2	23.6	D	4.4	47.5	94.2	46.1	Diorite	Medium Strong
4	2	23.7	D	11.4	47.6	78.0	118.1	Diorite	Very Strong
5	2	24.4	D	14.5	47.6	81.2	149.8	Diorite	Very Strong
6	3	24.8	D	8.3	47.0	68.8	87.3	Diorite	Strong
7	3	25.3	A	7.3	47.1	48.3	62.3	Diorite	Strong
8	3	25.3	D	14.9	47.1	48.8	157.3	Diorite	Very Strong
9									
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30									

* It is ideal to perform axial test on core specimens with D/L ratio of 1.1 ± 0.1
Long pieces of core can be tested diametrically to produce suitable lengths for axial testing

* Diametral Test should have 0.7 x D on either side of test point.

Appendix C

Site Photographs

Photograph 1 – Hawkeye Creek Bridge

Appendix D

Foundation Comparison

COMPARISON OF FOUNDATION ALTERNATIVES FOR EACH FOUNDATION ELEMENT

Footings on Native Soil	Caissons	Driven H-Piles to Bedrock or Refusal
Advantages: i. Generally less costly construction than deep foundation elements.	***Advantages:*** i. Construction of caissons could continue in freezing weather. ii. High geotechnical resistance available for units founded on bedrock.	***Advantages:*** i. High geotechnical resistance available by driving piles to achieve resistance on the bedrock or refusal. ii. Installation of piles could continue in freezing weather. iii. Foundation construction may require less volume of excavation than footings. iv. Readily installed.
Disadvantages: ii. Low available geotechnical resistance in native cohesionless deposits. iii. Dewatering will be required due to the high groundwater levels. iv. Potential disturbance of creek during excavation.	***Disadvantages:*** i. Higher cost than spread footings ii. Specialized installation measures such as temporary liners and drilling mud will be required to install caissons under the water table. iii. Potential difficulty in cleaning and inspecting bases.	***Disadvantages:*** i. Higher unit costs than footings. ii. Pile lengths required to achieve design resistance may vary.
NOT RECOMMENDED	**NOT RECOMMENDED**	**RECOMMENDED**

Appendix E

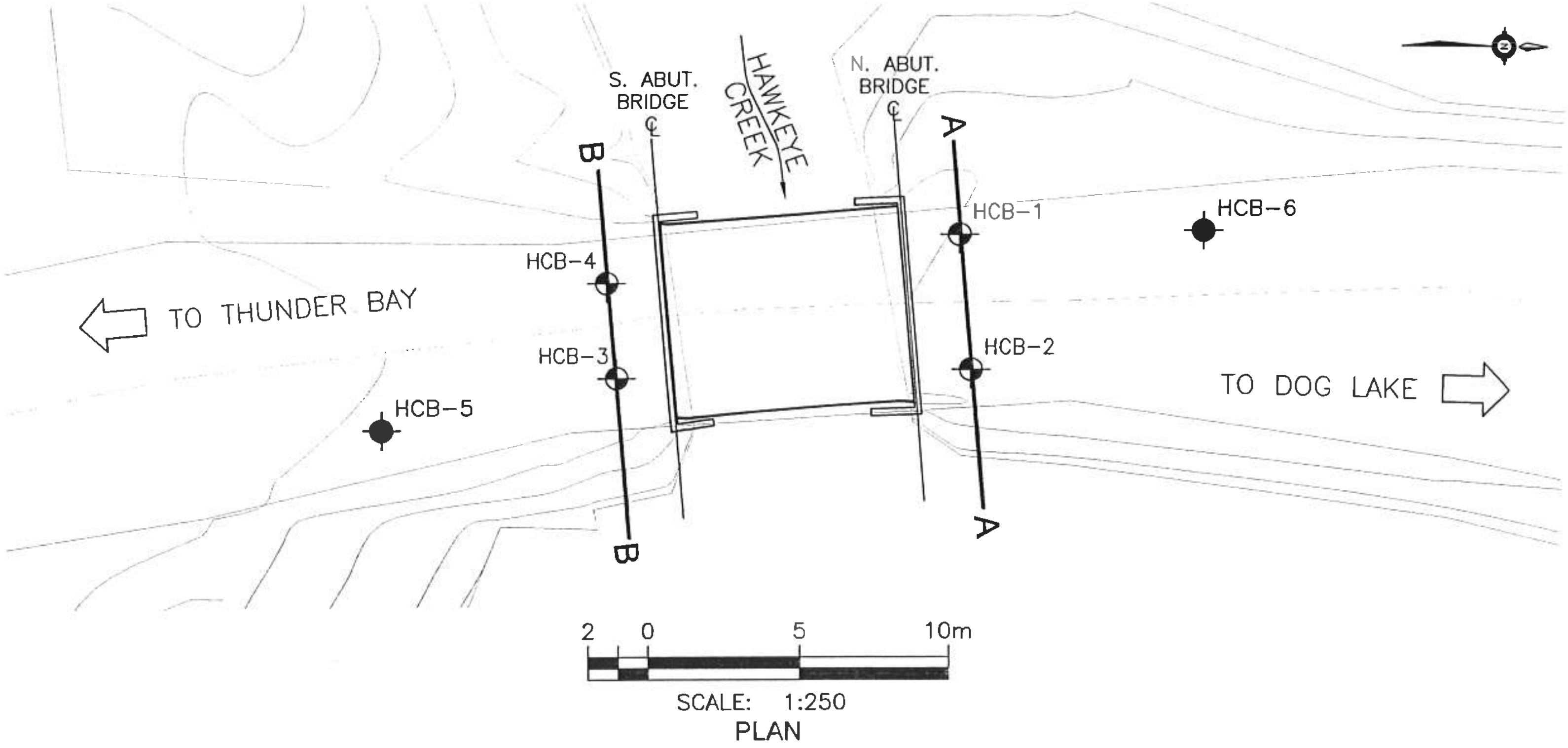
List of SPs and OPSS, and Suggested Text for NSSP

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

- OPSS 903, November 2009
- OPSS 804, November 2010
- OPSS 902, November 2010
- OPSD 3101.150.
- Special Provision 110S13 “Amendment to OPSS 1010, April 2004”.
- OPSS 539
- OPSS 501 dated November 2010
- OPSD 208.010

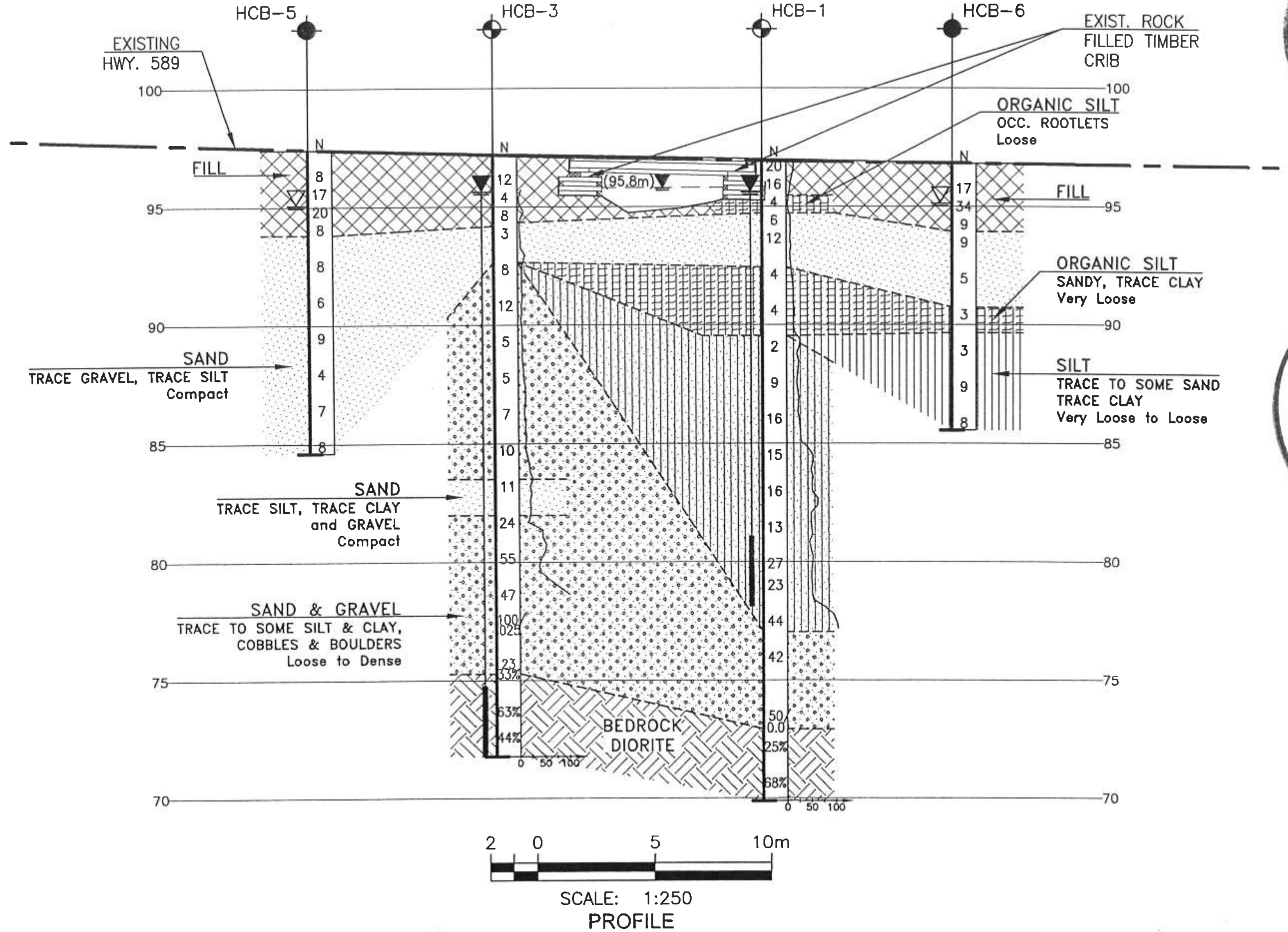
Appendix F

Borehole Locations and Soil Strata Drawings



METRIC

DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN



P. K. CHATTERJI
Jan 18/2012

R. Palomeque Reyna
100083209
Jan 18/2012

CONT No 2011-6028
WP No 6045-08-00

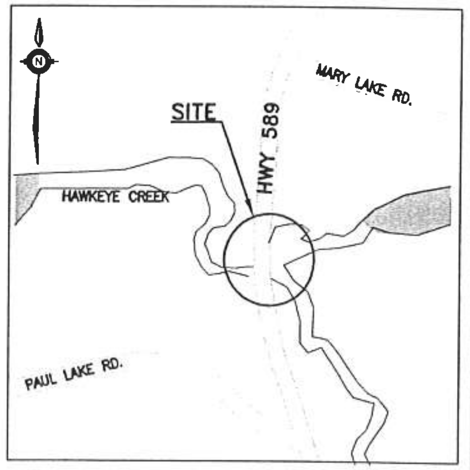
SHEET

HAWKEYE CREEK BRIDGE REPLACEMENT

BOREHOLE LOCATIONS AND SOIL STRATA

GENIVAR

THURBER ENGINEERING LTD.



KEYPLAN

LEGEND

●	Borehole
⊕	Borehole and Cone
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
HCB-1	97.0	101 17.0	9 976.5
HCB-2	96.9	101 17.4	9 981.0
HCB-3	97.2	101 05.6	9 981.3
HCB-4	97.2	101 05.3	9 978.1
HCB-5	97.4	100 97.8	9 983.0
HCB-6	96.8	101 25.1	9 976.4

-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. 52A-150

REVISIONS	DATE	BY	DESCRIPTION

DESIGN RPR	CHK RPR	CODE	LOAD	DATE JAN. 2012
DRAWN AN	CHK	SITE	STRUCT	DWG 2

FILENAME: H:\Drafting\19\5308\40\ted0840-BoreholePlan&Profile(HawkeyeCreek).dwg

MINISTRY OF TRANSPORTATION, ONTARIO

METRIC

DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN

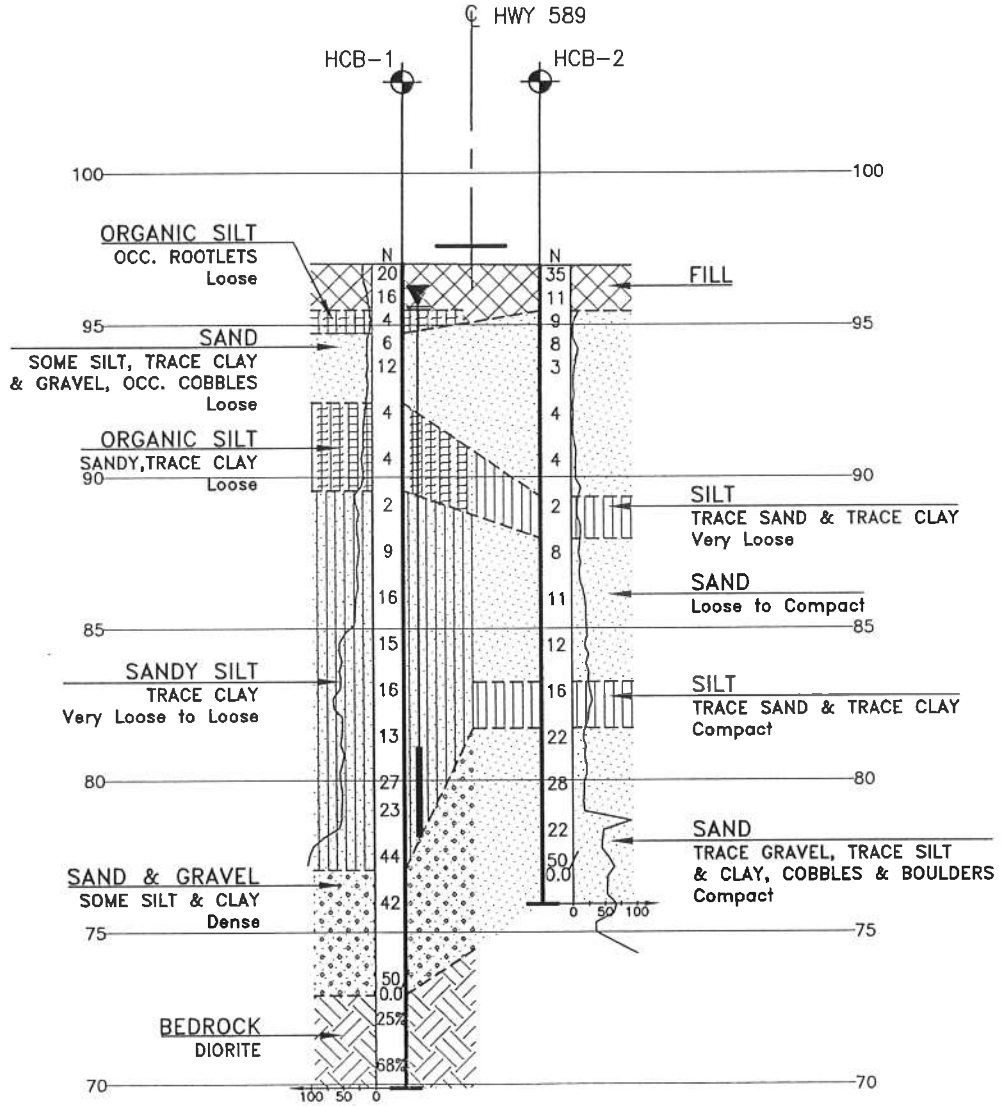
CONT No 2011-6028
WP No 6045-08-00

SHEET

HAWKEYE CREEK BRIDGE
REPLACEMENT
BOREHOLE LOCATIONS AND SOIL STRATA

GENIVAR

THURBER ENGINEERING LTD.



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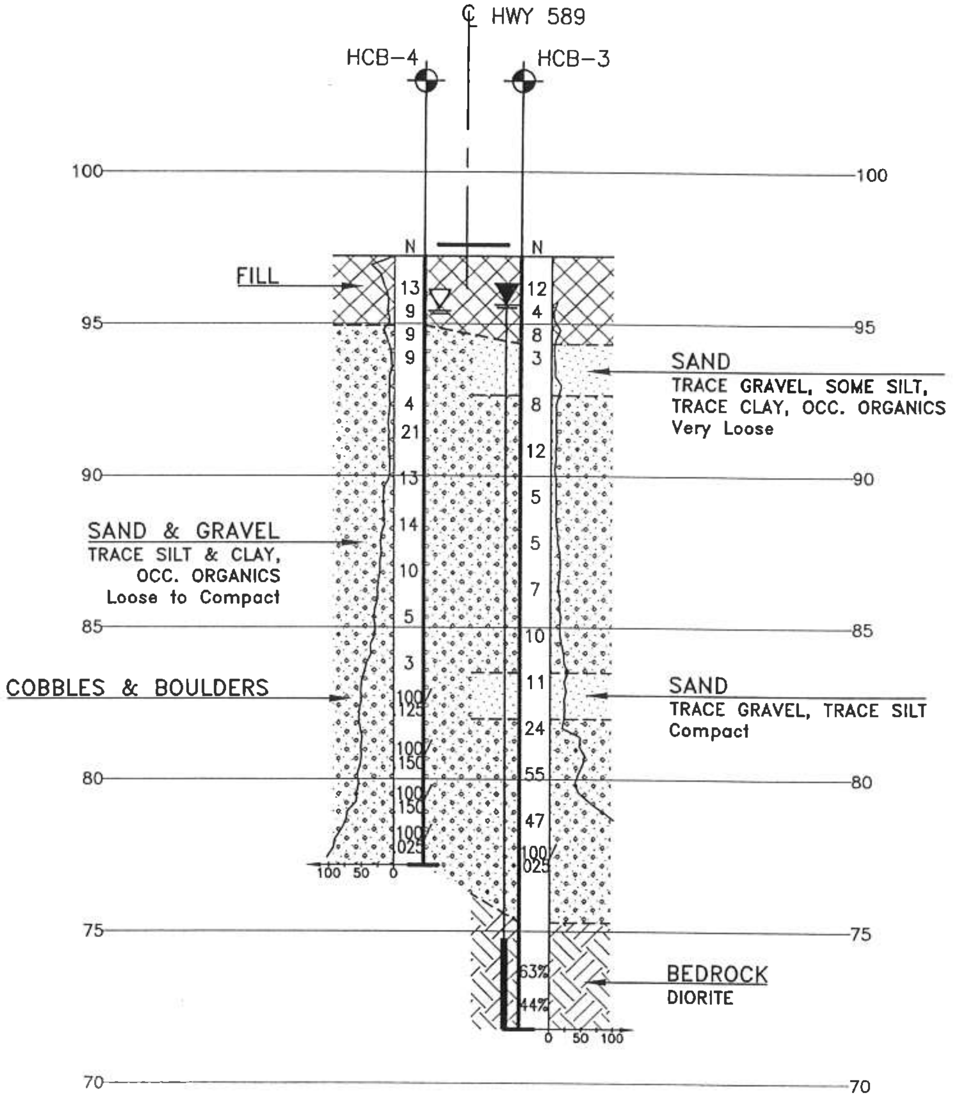

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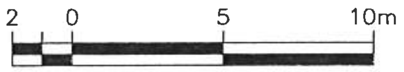

SCALE: 1:250

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Jan 18/2012

R. Palomeque Reyna
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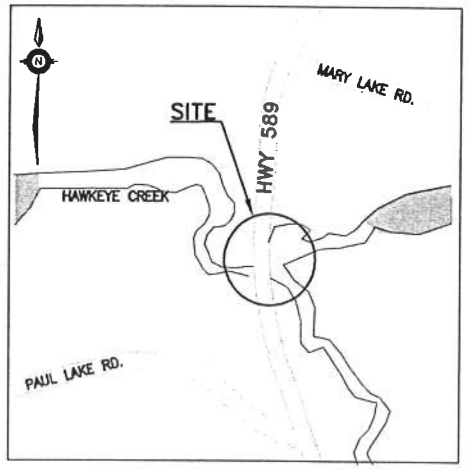
Jan/18/2012



KEYPLAN

L E G E N D

⯁	Borehole
⯁	Borehole and Cone
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
HCB-1	97.0	101 17.0	9 976.5
HCB-2	96.9	101 17.4	9 981.0
HCB-3	97.2	101 05.6	9 981.3
HCB-4	97.2	101 05.3	9 978.1
HCB-5	97.4	100 97.8	9 983.0
HCB-6	96.8	101 25.1	9 976.4

-NOTES-

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GEOCRES No. 52A-150

REVISIONS	DATE	BY	DESCRIPTION

DESIGN RPR	CHK RPR	CODE	LOAD	DATE JAN. 2012
DRAWN AN	CHK	SITE	STRUCT	DWG 3

FILENAME: H:\Drafting\19\5308\40\ted0840-BoreholePlandProfile(HawkeyeCreek).dwg