FOUNDATION INVESTIGATION AND DESIGN REPORT
CHIPPEWA RIVER BRIDGE REHABILITATION
Highway 17, Site 38S-008
G.W.P. 5141-08-00
Township of Tilley

Geocres Number: 41K-89

Report to

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PART 1: FACTUAL INFORMATION

1. INTRODUCTION

This report presents the factual findings obtained from a foundation investigation conducted at the site of the Chippewa River bridge located on Highway 17 in the Township of Tilley, Ontario. This investigation was completed for the proposed rehabilitation of the bridge.

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, a stratigraphic profile, 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 McCormick Rankin Corporation (MRC), under the Ministry of Transportation Ontario (MTO) Agreement Number 5009-E-0032.

2. SITE DESCRIPTION

The Chippewa River Bridge is located on Highway 17 in the Township of Tilley, Ontario, approximately 42.3 km north of Sault Ste. Marie.

At present, the highway crosses the Chippewa River on a reinforced concrete arch. The concrete arch is covered with granular fill from the top of concrete arch to the base of the asphalt. A concrete pedestrian sidewalk and an observation deck are located on the upstream edge of the bridge. The total length of the bridge is 47.5 m. The widths of the bridge and the pedestrian sidewalk are is approximately 11.4 m and 2.0 m, respectively.

The Chippewa River flows south and discharges near Batchawana Bay of Lake Superior.

The Chippewa Falls are located and visible on the north side of Highway 17. At this location, the Chippewa River descends about 20 m over several sets of waterfalls. The river is up to 32 m wide near the bridge.

The lands on the south side of the bridge are relatively flat and heavily treed. A low-rise building is located on the southeast side of the bridge. A picnic/rest area occupies the northeast side of the bridge.

Photographs of the site included in Appendix F show the general nature of the surrounding land:

1 and 2	-	General view of the south side of the Chippewa River Bridge (downstream)
3 and 4	-	Existing conditions of west and east slopes/approaches on the south side of the bridge
5 and 6	-	General view of the north side of the Chippewa River Bridge (upstream)
7 and 8	-	Upstream and downstream of the Chippewa River

Physiographically, the site lies within the Canadian Shield which is characterized by Pre-Cambrian igneous and metamorphic bedrock. Locally, the bedrock is mantled by deposits of sand and gravel.

3. SITE INVESTIGATION AND FIELD TESTING

The site investigation and field testing for this project was carried on October 19 and 20, 2010, and consisted of drilling and sampling four boreholes numbered CHIP-1 to CHIP-4. The boreholes were drilled on the existing Highway 17 lanes near each foundation element.

Boreholes CHIP-1 and CHIP-4 were drilled near the west and east abutments, respectively. Both boreholes were initially advanced through overburden soils to 8.6 m and 7.4 m depth, and then further advanced into the bedrock by coring to 12.0 m and 10.6 m.

Boreholes CHIP-2 and CHIP-3 were drilled near the existing buttresses and were terminated at 3.7 m depth.

Eight Dynamic Cone Penetration Tests (DCPTs) were also performed in close proximity of the four boreholes. DCPTs extended to depths ranging from 4.4 m to 8.5 below the existing highway grade.

The borehole and DCPT locations and termination depths are indicated in Table 3.1.

Table 3.1 – Borehole and DCPTs locations and termination depths

Location relative to the existing bridge	Borehole	DCPT	Borehole/ DCPT termination depth (m)	Borehole/ DCPT termination elevation (m)
West abutment	CHIP-1	-	12.0	179.7(1)
	-	1A	8.5	183.1
	-	1B	8.2	183.4
West buttress/ end of arch	CHIP-2	-	3.7	187.9
	-	2A	4.6	187.0
	-	2B	6.1	185.5
East buttress/ end of arch	CHIP-3	-	3.7	187.8
	-	3A	5.5	186.0
	-	3B	4.6	186.9
East abutment	CHIP-4	-	10.6	180.8(1)
	-	4A	7.3	184.2
	-	4B	4.4	187.1

(1) Depth and elevation include coring into bedrock.

The approximate locations of the four boreholes and eight DCPTs are shown on the attached Borehole Locations and Soil Strata Drawing in Appendix G. The coordinates and elevations of the boreholes are given on these drawings and on the individual Record of Borehole Sheets in Appendix A.

Prior to commencement of drilling, utility clearances were obtained for all borehole locations. Road occupancy permits were obtained for boreholes drilled on the existing Highway 17 platform.

Hollow stem augers were used to advance the boreholes in the overburden. Samples were obtained at selected intervals using a split spoon sampler in conjunction with Standard Penetration Testing (SPT). NQ rock coring equipment was used to recover core samples of the underlying bedrock in selected boreholes. A minimum 3.0 m of rock cores were recovered from Boreholes CHIP-1 and CHIP-4.

A member of Thurber's engineering staff supervised the drilling and sampling operations on a full time basis. The supervisor logged the boreholes, visually examined the recovered samples, and transported them 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 Index (FI) were determined.

Groundwater conditions in the open boreholes were observed throughout the drilling operations. Two standpipe piezometers consisting of 19 mm PVC pipes with slotted screens were installed in Boreholes CHIP-1 and CHIP-4 to permit monitoring of groundwater levels. Details of the piezometer installations and other borehole completion details are as shown in Table 3.2.

Table 3.2 – Borehole Completion Details

Location relative to the existing bridge	Borehole	Details	
		Piezometer Tip Depth/ Elevation (m)	Completion Details
West abutment	CHIP-1	11.8/179.8	Piezometer with 1.5 m slotted screen installed with sand filter to 8.7 m, bentonite from 8.7 m to 5.8 m, cuttings from 5.8 m to 0.5 m, bentonite from 0.5 m to 0.15 m, sand from 0.15 m to 0.08 m, and asphalt to surface. Flushmount cover installed.
West buttress	CHIP-2	None installed	Backfilled with cuttings to 0.05 m, then asphalt to surface.
East buttress	CHIP-3	None installed	Backfilled with cuttings to 0.1 m, then asphalt to surface.
East abutment	CHIP-4	7.3/184.2	Piezometer with 1.5 m slotted screen installed with sand filter to 5.4 m, bentonite from 5.4 m to 2.4 m, cuttings from 2.4 m to 0.3 m, bentonite from 0.3 m to 0.08 m, and asphalt to surface. Flushmount cover installed.

4. GEOTECHNICAL LABORATORY TESTING

All recovered soil and rock samples were subjected to Visual Identification (VI) and geological logging. Moisture content determinations were carried out on all soil samples. At least 25% of the recovered samples of soil were also subjected to grain size distribution analyses (sieve and hydrometer). The results of this testing program are shown on the Record of Borehole sheets in Appendix A and on the figures contained in Appendix B.

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 selected rock core samples are shown in Point Load Test Sheets included in Appendix B and on the Record of Borehole sheets in Appendix A.

5. CHLORIDE CONTENT TESTING

Selected soil samples from the boreholes were submitted to a qualified, CAEAL accredited laboratory (AGAT Laboratories Limited) in Mississauga, Ontario for analytical testing to assess for presence of chloride content.

The results of analytical analyses are presented on Appendix C, 'Certificate of Analysis'.

6. DESCRIPTION OF SUBSURFACE CONDITIONS

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

In general terms, the soil stratigraphy encountered at this site consists of pavement structure overlying fill which is underlain by native deposits of sand, silty sand and gravelly sand. Granitic bedrock was contacted below the native cohesionless deposits. More detailed descriptions of the individual strata are presented below.

6.1 Pavement Structure

Pavement structure consisting of approximately 50 mm to 75 mm of asphalt overlying granular (sand and gravel) fill was encountered in all the boreholes drilled on existing Highway 17 lanes.

6.2 Fill

Granular fill was contacted below the asphalt in all the boreholes. The fill generally consists of brown sand containing trace gravel to gravelly, trace silt and clay and occasional cobbles.

The thickness of the fill was about 3.6 m at the west and east buttress locations (Boreholes CHIP-2 and CHIP-3). At the west and east abutments, the thickness of the fill was 1.2 m and 1.7 m, respectively.

The depths to the base of the fill at the east and west abutments are 1.3 m and 1.8 m (elevations 190.4 and 189.6), respectively.

Boreholes CHIP-2 and CHIP-3 were terminated on top of the concrete arch contacted below the fill at 3.7 m depth (elevations 187.9 and 187.8).

Within the upper 1.5 m of the cohesionless fill layer, the SPT ‘N’ values ranged from 24 to 143 blows per 0.3 m of penetration, indicating a compact to very dense relative density. Below elevation 190.4, the measured SPT ‘N’ values ranged from 3 to 19 blows per 0.3 m of penetration, indicating a very loose to compact relative density.

The moisture content of the fill ranged from 2% to 8%.

Grain size distribution curves for samples of sand fill and gravelly sand fill tested are presented on the Record of Borehole sheets and on Figure B1 of Appendix B. The results of the laboratory test are summarized as follows:

Soil Particles	Sand fill (%)	Gravelly sand fill (%)
Gravel	17	27 to 34
Sand	77	56 to 66
Silt & Clay	6	5 to 12

Based on the grain size distribution curves, the coefficient of permeability of the sand, gravelly sand fill is estimated to range from 2.0×10^{-2} cm/sec to 6.0×10^{-3} cm/sec.

6.3 Sand

Native sand was contacted below the fill at 1.3 m depth (elevation 190.4) in Borehole CHIP-1. The sand was brown in colour and contained trace to some gravel, trace silt and trace clay.

The thickness of the sand was 7.3 m.

The depth to the base of the sand was 8.6 m (elevation 183.1).

SPT ‘N’ values measured in the sand ranged from 2 to 17 blows for per 0.3 m penetration, indicating a very loose to compact relative density.

The natural moisture contents of samples recovered from the sand layer ranged from 2% to 21%.

Grain size distribution curves for samples of sand tested are presented on the Record of Borehole sheets and on Figure B2 of Appendix B. The results of the laboratory test are summarized as follows:

Soil Particles	Sand fill (%)
Gravel	5 to 12
Sand	82 to 90
Silt & Clay	3 to 8

Based on the grain size distribution curves, the coefficient of permeability of the sand, gravelly sand fill is estimated be 2.0 x 10^{-2} cm/sec.

6.4 Silty Sand

Native brown silty sand containing trace gravel and clay was contacted below the fill at 1.8 m depth (elevation 189.6) in Borehole CHIP-4, drilled at the east abutment.

The thickness of the silty sand was 3.1 m.

The depth to the base of the silty sand was 4.9 m (elevation 186.6).

SPT 'N' values measured in the silty sand ranged from 3 to 8 blows for per 0.3 m penetration, indicating a very loose to loose relative density.

The natural moisture contents of samples recovered from the silty sand layer ranged from 17% to 19%.

Grain size distribution curves for samples of silty sand tested are presented on the Record of Borehole sheets and on Figure B3 of Appendix B. The results of the laboratory test are summarized as follows:

Soil Particles	Silty sand (%)
Gravel	1 to 5
Sand	63 to 65
Silt	28 to 34
Clay	2

Based on the grain size distribution curves, the coefficient of permeability of the sand, gravelly sand fill is estimated to be 3.0 x 10^{-4} cm/sec.

6.5 Gravelly Sand

A layer of gravelly sand with trace silt and clay and occasional cobbles was encountered below the silty sand at 4.9 m depth (elevation 186.6) in Borehole CHIP-4.

The thickness of the gravelly sand was 2.5 m.

The depth to the base of the gravelly sand was 7.4 m (elevation 184.1).

SPT 'N' values measured in the gravelly sand ranged from 5 to 10 blows per 0.3 m of penetration indicating a loose to compact relative density.

The natural moisture content in the gravelly sand ranged from 7% to 22%.

Grain size distribution curve for a sample of gravelly sand tested is presented on the Record of Borehole sheets and on Figure B4 of Appendix B. The results of the laboratory test are summarized as follows:

Soil Particles	Gravelly sand (%)
Gravel	24
Sand	70
Silt & Clay	6

6.6 Bedrock

The soils described in Boreholes CHIP-1 and CHIP-4 were found to be underlain by granitic bedrock. The bedrock encountered in the boreholes is generally described as moderately weathered. The bedrock in Borehole CHIP-1 was generally grey with occasional pink and white bands visible in most cores. In Borehole CHIP-4, the bedrock was red with grey bands. Occasional mechanical breaks and sub-vertical fractures were observed in the rock cores.

Table 5.1 summarizes depths and elevations to the top of bedrock in the boreholes and DCPTs. Where coring was not carried out, bedrock was inferred from cone refusal in DCPTs.

Table 5.1 – Depth and Elevation of Top of Bedrock

Location relative to the existing bridge	Borehole/DCPT	Depth to Bedrock (m)	Top of Bedrock Elevation (m)
West abutment	CHIP-1	8.6[(2)]	183.1
	1A[(1)]	8.5	183.1
	1B[(1)]	8.2	183.4
East abutment	CHIP-4*	7.4[(2)]	184.1
	4A[(1)]	7.3	184.2
	4B[(1)]	4.4	187.1

(1) DCPT
(2) Proved by coring below augered depth

Total core recovery (TCR) in the bedrock ranged from 90% to 100% in all boreholes, except in Borehole CHIP-1 Run 1 where a TCR of 50% was observed.

Rock quality designation (RQD) values of 54% and 57% were recorded in Borehole CHIP-4 Runs 1 and 2, indicating a fair rock quality. Higher RQD values ranging from

83% to 90%, indicating a good rock quality, were obtained in Borehole CHIP-1 Runs 2 and 3 and Borehole CHIP-4 Run 3.

RQD value of 0% was noted in Borehole CHIP-1 Run 1.

The Fracture Index (FI) of the rock, expressed as fractures per 0.3 m of core, generally ranged from 1 to greater than 10 in most of the cores.

The estimated uniaxial compressive strength of the rock cores generally ranges from 57 MPa to 219 MPa, indicating a strong to very strong rock. Low values of unconfined compressive strength, 6.2 MPa and 31 MPa, were measured in Borehole CHIP-1 Run 2. These low values indicate a weak to medium 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 Point Load Test Sheets included in Appendix B.

6.7 Water Levels

Water levels were monitored in the boreholes during and upon completion of drilling. A standpipe piezometer was installed in boreholes drilled at the east and west abutments to monitor water levels after completion of drilling. The water levels measured in the piezometers are summarized in Table 5.2.

Table 5.2 – Water Level Measurements

Location relative to the existing bridge	Borehole	Date	Water Level (m)	
			Depth	Elevation
West abutment	CHIP-1	November 28, 2010	6.9	184.7
East abutment	CHIP-4	November 28, 2010	7.1	184.4

Piezometric reading indicates that water level is near elevations 184.4 to 184.7.

Preliminary GA drawing indicates that water level in the Chippewa River is near Elevation 183.5.

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.

6.8 Chloride Content Results

Chloride analyses were conducted on four soil samples (fill and native soils). Table 5.3 shows the chemical testing results. The results are presented on the attached Certificate of Analysis in Appendix C.

Table 5.3 – Results of Chloride Testing

Borehole	Sample	Depth (m)	Elevation	Soil	Chloride concentration (µg/g)
CHIP-1	SS1	0.15	191.4	Sand, some gravel, FILL	169
CHIP-2	SS3	1.2	190.4	Sand, some gravel, trace silt and clay, FILL	657
CHIP-3	SS4	1.8	189.7	Gravelly SAND, trace to some silt and clay, FILL	329
CHIP-4	SS5	2.4	189.1	Silty SAND, trace gravel and clay	2270

7. MISCELLANEOUS

Borehole locations were selected and established in the field by Thurber Engineering Ltd. Surveyors from McCormick Rankin Corporation obtained the co-ordinates and the ground surface elevations at each borehole.

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

Eastern Ontario Diamond Drilling of Hawkesbury, Ontario supplied a truck-mounted CME75 drill rig and conducted the drilling, sampling and in-situ testing operations.

The drilling and sampling operations in the field were supervised on a full time basis by Ms. Eckie Siu of Thurber.

Routine laboratory testing was carried out by Thurber Engineering Ltd.

Overall supervision of the field program was conducted by Mr. Alastair E. Gorman, P.Eng. and Mr. Lukasz Gilarski, E.I.T. Interpretation of the data and preparation of the report were carried out by Mr. Lukasz Gilarski, E.I.T. and 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

8. GENERAL

This report presents interpretation of the geotechnical data in the factual report and geotechnical recommendations related to the rehabilitation of the existing Chippewa River bridge in the Township of Tilley, Ontario.

The Chippewa River bridge is located on Highway 17, approximately 42.3 km north of Sault Ste. Marie, Ontario. The existing bridge structure consists of a reinforced concrete arch which is covered with granular fill. The bridge is supported on two abutments. Additional support for the bridge is provided by concrete buttresses built at each end of the concrete arch. The total length of the bridge is 47.5 m. Rock filled gabion walls are placed at each corner of the bridge abutments. Recent bridge inspections revealed water seeping through the base of the arch structure.

The bridge was constructed in 1939 and rehabilitated in 1989.

The rehabilitation program conducted in 1989 involved:

- Installation of rock fill gabion retaining walls at each corner of the bridge abutments.
- Extension of the buttresses.
- Construction of new abutments, including extending footings to bedrock.

Leakage and water seepage control measures are required as part of the proposed rehabilitation scheme.

The purpose of the present geotechnical investigation is to provide recommendations for the proposed rehabilitation to control leakage and water seepage above the concrete arch.

The discussion and recommendations presented in this report are based on information provided by McCormick Rankin Corporation and on the factual data obtained in the course of this investigation.

9. EXISTING CONDITIONS

The stratigraphy encountered in the four boreholes drilled during the present investigation at the east and west abutments revealed pavement structure overlying 1.2 m to 1.7 m of compact to very dense sand fill. Native deposits of very loose to compact sand, silty sand and gravelly sand were encountered below the fill. Granitic bedrock was contacted below the native cohesionless deposits at 7.4 m to 8.6 m depth. Boreholes drilled near the buttresses revealed that the fill used to cover the concrete arch, consist of 3.7 m of very dense to compact sand fill.

Piezometric reading indicates that water level is near elevation 184.7.

Preliminary GA drawing indicates that water level in the Chippewa River is near Elevation 183.5.

At this site, the granular fill above the arch and the native cohesionless soils are considered to be relatively permeable. Based on grain size distribution curves, the coefficients of permeability of the soils encountered at this site are:

Soil	Permeability, k (cm/sec)
Sand, gravelly sand Fill above the arch	2.0×10^{-2} to 6.0×10^{-3}
Native Sand	2.0×10^{-2}
Native Silty sand	3.0×10^{-4}

10. LEAKAGE/ WATER SEEPAGE CONTROL

It is understood that water seepage is noted through the base of the concrete arch.

Leakage and water seepage control measures are required as part of the rehabilitation scheme.

Initial consideration was given to the following seepage control measures for this site:

- Provision of a waterproof membrane on top of the concrete arch structure.
- Provision of a waterproof membrane at a shallow depth below the pavement.
- Pressure grouting of the granular backfill above the arch.

A comparison of the leakage/water seepage control alternatives based on advantages and disadvantages of each one is included in AppendixD.

10.1 Waterproof membrane on top of the concrete arch structure

This alternative will involve the following general construction sequence.

1. Remove existing pavement and granular fill to expose concrete arch.
2. Repair any cracks or delamination of the concrete in the arch
3. Apply a waterproofing membrane
4. Backfill the arch with compacted granular material
5. Reinstate the pavement.

This remedial measure should incorporate a drainage system at the base of the arch to keep the granular backfill drained and to avoid water pressure build up above the arch. Surface drainage measures such as curbs with gutters (OPSD 600.010) should be provided to direct surface run off away from the bridge structure.

The following issues need to be considered for this option:

1. This option will be labour intensive and expensive since the granular fill will have to be removed within confined spaces between the arch buttresses.
2. The success of this alternative will depend upon good workmanship.

If the membrane installation is not done properly, infiltration will occur through the membrane joints which will cause continuing deterioration of the concrete arch.

However, this option will allow a thorough inspection of the concrete arch and facilitate repair of arch before a waterproofing membrane is applied.

A detour will likely be required for constructing this option as discussed in Section 11 of this report.

10.2 Waterproof membrane just beneath the pavement structure

A HDPE (high density polyethylene) or Geosynthetic clay liner (such as Bentofix available from Terrafix or similar product) may be installed just beneath the pavement structure/concrete slab.

If properly designed and installed, the liner would reduce/limit the flow of water into the granular backfill above the arch. Surface drainage measures such as curbs with gutters (OPSD 600.010) should be provided to direct surface run off away from the bridge structure.

If this option is selected, it is recommended that discussions be held with a liner supply company to address installation details. Good workmanship is critical for this option for the liner to function effectively.

The advantage of this option is that it will require shallow excavation to install the liner and no roadway protection is required.

10.3 Pressure Grouting

Pressure grouting is another remedial measure for controlling seepage above the arch. There are two options in this alternative:

1. Pressure grouting of the entire granular backfill above the arch to create a practically impervious barrier above the arch.
2. Pressure grouting of the upper 1.0 m of the granular fill above the arch to create an impervious layer below the pavement.

For this option, curbs and gutters will have to be incorporated to carry surface run off away from the bridge. Both grouting options will add weight on top of the arch. Option 2 will add less weight and can be done with relatively minor repair to the bridge pavement surface.

Permeation grouting under pressure is possible for gravel or coarse to medium sands. For fines content greater than 20%, permeation grouting will not be possible. Gradation testing on five samples of granular fill that exists above the arch indicates a fines content (silt and clay) of 5 to 12% and permeation grouting of the fill may be possible.

Cement based grout or chemical grouts are two options for this grouting alternative. Chemical grouting may provide a more rapid and robust solution at this site.

Poly-Mor resins (Poly-Mor Canada Inc.) or similar products could be used as chemical grout at this site. Poly-Mor resins/polymers are reported to have superior sealing qualities, are reportedly unaffected by freeze-thaw actions and provide a long-lasting seal. The suppliers indicate that the product is generally light in weight and cost-effective and may be injected with minimum disruption.

Pressure grouting is generally customized for each project. If this option is to be explored further, it will be necessary to discuss the grouting scheme with several local specialty grouting contractors (such as Poly-Mor Canada, Geo-Foundations Contractors, Deep Foundations) in order to develop an appropriate grouting plan and specifications including spacing of the grout injection holes, type of grout, grouting pressures, grouting control plan, etc.

One advantage of carrying out the grouting of the bridge from the top is that relatively minor repair to the bridge surface (pavement/concrete slab) would be required.

One disadvantage with the cement based grouting option is the potential of the cracking of the grout with time and due to freeze-thaw effects.

10.4 Recommended seepage control alternative

On the basis of existing site conditions, the use of waterproof membrane is a better option for this site.

11. ROADWAY PROTECTION/DETOUR

For construction staging to keep part of the bridge open to traffic, applying a roadway protection such as driving conventional soldier piles and providing lagging is not possible at this site and an alternate method must be selected.

A modular Bailey bridge detour will likely be required for the option involving provision of a waterproof membrane on top of the concrete arch structure. No roadway protection is required for a membrane installed at shallow depth or for the grouting option.

12. BACKFILL TO ABUTMENTS OR ABOVE THE ARCH

Backfill to the abutments or above the arch should consist of Granular A or Granular B Type II material meeting the requirements of Special Provision 110S13 "Amendment to OPSS 1010, April 2004". The backfill must be in accordance with OPSS 902, November 2010. Any abutment backfill should be placed to the extents shown in OPSD 3101.150.

Compaction equipment to be used above the arch or adjacent to retaining structures must be restricted in accordance with OPSS 501 dated November 2010. It is assumed that there is a functioning subdrain at the base of the embankment.

13. EARTH PRESSURE

Earth pressure acting on the structure abutments may be assumed to be triangular and to be governed by the characteristics of the abutment backfill.

For fully drained conditions, earth pressures acting on the structure should be computed in accordance with Clause 6.9 of the CHBDC but generally are given by the expression:

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

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

K = earth pressure coefficient (see table 13.1)

γ = unit weight of retained soil (see 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 at a depth of 1.7 m for Granular A or Granular B Type II.

Earth pressure coefficients for backfill to the abutment wall are dependent on the material used as backfill. Typical values are given in Table 13.1.

The coefficients in the 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 C6.9.1 (a) in the Commentary to the CHBDC, 2006.

Table 13.1 – Earth Pressure Coefficients

Wall Condition	**Earth Pressure Coefficient (K)**			
	OPSS Granular A or OPSS Granular B Type II $\phi = 35°; \gamma = 22.8$ kN/m^3		**OPSS Granular B Type I $\phi = 32°; \gamma = 21.2$ kN/m^3**	
	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*
At rest (Restrained Wall)	0.43	-	0.47	-
Passive (Movement Towards Soil Mass)	3.70	-	3.30	-

* For wing walls.

14. TEMPORARY EXCAVATION

Temporary excavation will be required at this site to conduct rehabilitation operations. Excavation will be required if the selected method for seepage control consists of installing a waterproof membrane at the top of the concrete arch. Excavation of granular fill will extend more than 3.7 m, where boreholes were terminated. The excavation will be labour intensive since the granular backfill will have to be removed within confined spaces between the arch and the buttresses.

All excavations must be carried out in accordance with the Occupational Health and Safety Act (OHSA) and in accordance with OPSS 902, November 2010. For the purposes of the OHSA, the fill in the existing abutments at this site may be classified as Type 3 soils.

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. If required, remedial actions must be taken to ensure the stability of the excavation and the safety of workers.

15. GROUNDWATER AND SURFACE WATER CONTROL

Piezometers installed in Boreholes CHIP-1 and CHIP-4 revealed that groundwater is approximately 6.9 m to 7.1 m below ground surface, near elevations 184.4 and 184.7. Preliminary GA drawing indicates that water level in the Chippewa River is near Elevation 183.5. The base of the arch is above the groundwater level. However, seepage may be experienced from perched zones in the granular fill above the arch. The level of perched water within the fill will vary between locations.

The Contractor should be prepared to pump from sumps to remove any seepage water or surface water collecting in an excavation. Unwatering must remain operational and effective until the structure is backfilled.

The design of the dewatering system that may be required is the responsibility of the Contractor and the Contract Documents must alert him to this responsibility.

16. STRUCTURE APPROACHES

The existing structure has some minor sliding embankment. The abutment footing of the pedestrian bridge is also undermined at one corner.

The GA drawing indicates that the bridge rehabilitation will include restoration of the existing embankments and slope protection, repair of erosion at the northwest corner of the west abutment and erosion repair at sidewalk.

Discussions with MRC indicate that restoration of the abutments will generally consist of re-establishing the slope to its original state, placing minor fill where necessary and creating an even and uniform surface.

The restoration of the existing slopes to its original state may be accomplished using granular fill or rock fill. For rock fill, the slope inclinations should not be steeper than 1.25H:1V. For granular fill the slope inclinations should not be steeper than 2H:1V.

Repair of undermining of the abutment footing of the pedestrian bridge should involve grouting of the void under the footing followed by placement of rock protection.

If rock fill is placed against an existing earth fill or granular fill embankment, the existing embankment should be benched as per OPSD 208.010, after stripping of vegetation, topsoil, organics, soft soils or otherwise unsuitable overburden materials.

After bridge rehabilitation is completed, disturbed, regraded earth slopes or exposed earth surfaces must be covered with vegetation to protect against surficial erosion, in general accordance with OPSS 804, November 2010.

17. CHLORIDE CONTENT TESTING

Results of chloride content testing is presented in Section 6.8 of this report.

The chloride values reflect the effects of road de-icing salt. Salt/chloride migrates from the road surface to the existing fill and native soils. Penetration of chloride can accelerate concrete arch deterioration and corrode the reinforcing steel.

Overlays, membranes and sealers are protective systems that help to decrease ingress of chlorides.

If pressure grouting is selected as a remedial option for seepage, the levels of chloride in the soils will dictate the type of cement/grout to be used for this purpose at the site.

18. CONSTRUCTION CONCERNS

Potential construction concerns include, but are not necessarily limited to the issues discussed below:

1. **Design of grouting of backfill above the arch**

   If the grouting option is to be pursued, it is recommended that a specialty grouting contractor be contacted to develop a grouting plan for this site. All factual data included in this report (grain size analysis, permeability and chloride testing results) must be provided to the grouting contractor.

2. **Staging construction**

Conventional roadway protection during construction staging is not possible at this site. A modular Bailey bridge detour may be required for the option of providing a waterproof membrane on top of the concrete arch.

3. **Existing slopes**

The side embankment slopes should be inspected after construction for surficial disturbance. Where necessary, erosion control measures must be implemented.

4. **Construction timing**

The provision of a water proofing membrane or grouting should be carried out in non-freezing and dry weather conditions and not during winter.

19. 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

RECORD OF BOREHOLE No CHIP-1

METRIC

W.P. 93-89-00 LOCATION N 5 198 839.4 E 272 311.5 (Chippewa River Bridge) ORIGINATED BY ES

HWY 17 BOREHOLE TYPE Hollow Stem Augers COMPILED BY AN

DATUM Geodetic DATE 2010.10.19 - 2010.10.19 CHECKED BY JL

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
191.6						
0.0 / 0.1	**ASPHALT:** (63mm)					
	SAND, some gravel Very Dense to Dense Brown Moist (FILL)	1	SS	93		
		2	SS	38	191	
190.4 / 1.3	**SAND**, trace to some gravel, trace silt, trace clay Compact to Loose Brown Moist	3	SS	10	190	5 87 8 (SI+CL)
		4	SS	4		
		5	SS	6	189	
	Wet	6	SS	6	188	11 86 3 (SI+CL)
		7	SS	4		
		8	SS	4	187	
		9	SS	4	186	12 82 6 (SI+CL)
		10	SS	9		
		11	SS	17	185	
		12	SS	6		
	Very Loose to Loose	13	SS	2	184	5 90 5 (SI+CL)
		14	SS	4		
183.1 / 8.6	**BEDROCK**, granitic, moderately weathered, grey, with red bands, mechanical breaks Coring started at 8.6m 75mm sub-vertical fractures at 8.7m, 8.8m Sub-horizontal fractures at 8.8m Sub-vertical fractures between 25mm to 100mm at 9.5m, 10.1m 300mm at 8.9m	1	RUN		183	FI: >5, 7 RUN #1 TCR=50% SCR=50% RQD=0% UCS=188MPa (Average)
		2	RUN		182	FI: 2, 2, 2 RUN #2 TCR=100% SCR=97% RQD=83% UCS=93MPa

Column headings: SOIL PROFILE; STRAT PLOT; 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

Continued Next Page

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

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

Ministry of Transportation Ontario

RECORD OF BOREHOLE No CHIP-1

METRIC

W.P.	93-89-00	LOCATION	N 5 198 839.4 E 272 311.5 (Chippewa River Bridge)	ORIGINATED BY	ES
HWY	17	BOREHOLE TYPE	Hollow Stem Augers	COMPILED BY	AN
DATUM	Geodetic	DATE	2010.10.19 - 2010.10.19	CHECKED BY	JL

ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, X 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										
	BEDROCK, granitic, moderately weathered, grey, red bands, mechanical breaks 50mm soft zone at 10.3m									>5	(Average)
							181			0	RUN #3 TCR=90% SCR=90% RQD=90% UCS=93MPa (Average)
	Sub-vertical fractures between 25mm to 75mm at 10.6m, 11.2m, 11.3m, 11.5m		3	RUN						1	
										2	
										2	
179.7	Quartz vein at 10.5m, 11.1m						180			1	
12.0	END OF BOREHOLE AT 12.0m. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen. WATER LEVEL READINGS: DATE DEPTH (m) ELEV. (m) 2010.11.28 6.9 184.7										

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

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

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Ontario

THURBER

RECORD OF BOREHOLE No CHIP-2

METRIC

W.P.	93-89-00	LOCATION	N 5 198 836.2 E 272 327.3 (Chippewa River Bridge)	ORIGINATED BY	ES
HWY	17	BOREHOLE TYPE	Hollow Stem Augers	COMPILED BY	AN
DATUM	Geodetic	DATE	2010.10.19 - 2010.10.19	CHECKED BY	JL

ELEV DEPTH	DESCRIPTION (SOIL PROFILE)	STRAT PLOT	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH kPa	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
191.6											
0.0 / 0.1	**ASPHALT:** (75mm)										
	SAND, some gravel, trace silt and clay Very Dense to Compact Brown Moist (FILL)		1	SS	143				o		
							191				
			2	SS	24				o		17 77 6 (SI+CL)
			3	SS	19				o		
							190				
	Becoming gravelly		4	SS	11				o		32 63 5 (SI+CL)
							189				
			5	SS	22				o		
	Loose		6	SS	5				o		
187.9 / 3.7	END OF BOREHOLE AT 3.7m. BOREHOLE BACKFILLED WITH CUTTINGS TO 0.05m, THEN ASPHALT TO SURFACE.						188				

Legend: DCPT scale 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

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

Ministry of Transportation Ontario

RECORD OF BOREHOLE No CHIP-3

METRIC

W.P. 93-89-00 | LOCATION N 5 198 815.4 E 272 353.4 (Chippewa River Bridge) | ORIGINATED BY ES

HWY 17 | BOREHOLE TYPE Hollow Stem Augers | COMPILED BY AN

DATUM Geodetic | DATE 2010.10.20 - 2010.10.20 | CHECKED BY JL

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
191.5 / 0.0	**ASPHALT:** (50mm)						
	Gravelly **SAND**, trace to some silt and clay, occasional cobbles, Very Dense to Compact, Brown, Moist (FILL)	1	SS	141	191	o	27 61 12 (SI+CL)
		2	SS	44		o	34 56 10 (SI+CL)
		3	SS	10	190	o	
	Loose	4	SS	5		o	
		5	SS	9	189	o	
187.8 / 3.7	Very Loose	6	SS	3	188	o	29 66 5 (SI+CL)
	END OF BOREHOLE AT 3.7m. BOREHOLE BACKFILLED WITH CUTTINGS TO 0.1m, THEN ASPHALT TO SURFACE.						

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

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

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

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Ministry of Transportation
Ontario

RECORD OF BOREHOLE No CHIP-4

METRIC

W.P.	93-89-00	LOCATION	N 5 198 815.6 E 272 363.8 (Chippewa River Bridge)	ORIGINATED BY	ES
HWY	17	BOREHOLE TYPE	Hollow Stem Augers	COMPILED BY	AN
DATUM	Geodetic	DATE	2010.10.19 - 2010.10.19	CHECKED BY	JL

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

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

UNIT WEIGHT γ kN/m³

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

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	FI	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
191.5							
0.0 / 0.1	**ASPHALT:** (63mm)						
	SAND, some gravel, occasional cobbles Very Dense Brown Moist (FILL)	1	SS	121	191		
		2	SS	59			
	Compact	3	SS	14	190		
189.6 / 1.8	Silty **SAND**, trace gravel, trace clay Very Loose to Loose Brown Moist	4	SS	8			1 63 34 2
		5	SS	6	189		
		6	SS	4	188		5 65 28 2
		7	SS	3			
		8	SS	8	187		
186.6 / 4.9	Gravelly **SAND**, trace silt and clay, occasional cobbles Loose Brown Moist	9	SS	5	186		
		10	SS	9			24 70 6 (SI+CL)
		11	SS	5	185		
	Compact	12	SS	10			
184.1 / 7.4		13	SS	50/		FI	
	BEDROCK, granitic, moderately weathered, red, with grey bands, mechanical breaks Coring started at 7.4m Sub-vertical fractures between 25mm to 50mm at 7.7m, 7.9m, 8.0m, 8.1m	1	RUN	0.025	184	>5 >5 1 >5	RUN #1 TCR=100% SCR=100% RQD=54% UCS=196MPa (Average)
	Highly weathered Sub-vertical fractures between 25mm to 50mm at 8.9m, 9.2 175mm at 8.7m 150mm at 9.0m	2	RUN		183 182	>10 >10 >10 1	RUN #2 TCR=100% SCR=70% RQD=57% UCS=127MPa (Average)

Continued Next Page

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

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

Ministry of Transportation Ontario

RECORD OF BOREHOLE No CHIP-4

 METRIC

W.P. 93-89-00 LOCATION N 5 198 815.6 E 272 363.8 (Chippewa River Bridge) ORIGINATED BY ES

HWY 17 BOREHOLE TYPE Hollow Stem Augers COMPILED BY AN

DATUM Geodetic DATE 2010.10.19 - 2010.10.19 CHECKED BY JL

ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE) 20 40 60 80 100	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L — WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m^3	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	Continued From Previous Page										
180.8	Sub-vertical fractures between 25mm to 50mm at 9.9m, 10.3m, 10.5m	+ +	3	RUN			181			4 >5	RUN #3 TCR=100% SCR=100% RQD=89% UCS=206MPa (Average)
10.6	END OF BOREHOLE AT 10.6m. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen.										

WATER LEVEL READINGS:

DATE	DEPTH (m)	ELEV. (m)
2010.11.28	7.10	184.4

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

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

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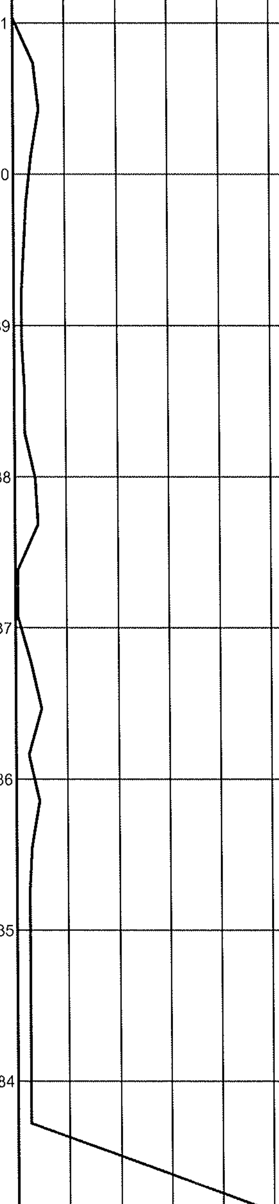
RECORD OF BOREHOLE No DCPT-1A

METRIC

W.P.	5198-06-00	LOCATION	N 5 198 839.1 E 272 311.9 (Chippewa River Bridge)	ORIGINATED BY	ES
HWY	17	BOREHOLE TYPE	Dynamic Cone Penetration Test	COMPILED BY	AN
DATUM		DATE	2010.10.20 - 2010.10.20	CHECKED BY	JL

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/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
191.6 / 0.0	Auger to 1.0m, then start DCPT						191, 190, 189, 188, 187, 186, 185, 184				
183.1 / 8.5	END OF DCPT AT 8.5m UPON CONE REFUSAL. BOREHOLE BACKFILLED WITH HOLEPLUG TO 0.1m THEN ASPHALT TO SURFACE.										

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

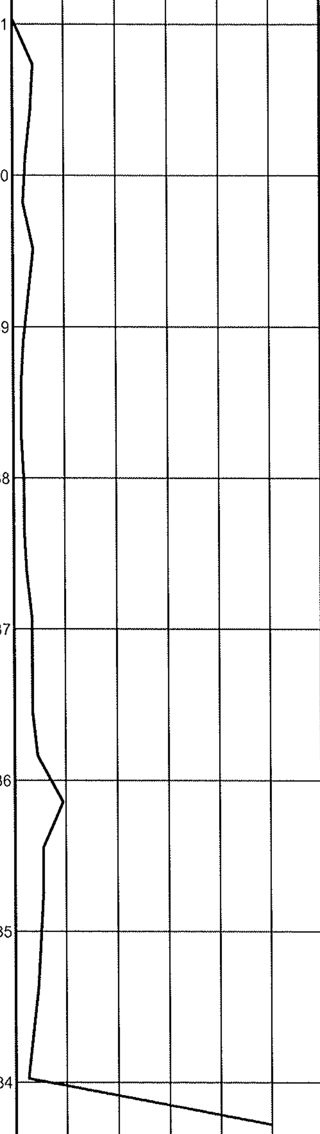
20 / 15 – 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

RECORD OF BOREHOLE No DCPT-1B

METRIC

W.P. 5198-06-00	LOCATION N 5 198 841.6 E 272 312.7 (Chippewa River Bridge)	ORIGINATED BY ES
HWY 17	BOREHOLE TYPE Dynamic Cone Penetration Test	COMPILED BY AN
DATUM	DATE 2010.10.20 - 2010.10.20	CHECKED BY JL

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, X 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						
191.6 / 0.0	Auger to 1.0m, then start DCPT						191, 190, 189, 188, 187, 186, 185, 184				
183.4 / 8.2	END OF DCPT AT 8.2m UPON CONE REFUSAL. BOREHOLE BACKFILLED WITH HOLEPLUG TO 0.1m THEN ASPHALT TO SURFACE.										

+ ³, × ³: Numbers refer to Sensitivity

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

Ministry of Transportation Ontario

RECORD OF BOREHOLE No DCPT-2A

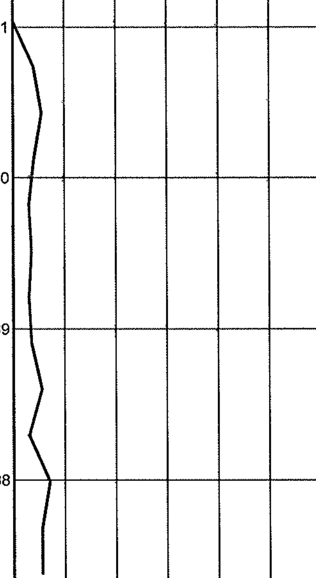
METRIC

W.P. 93-89-00 | LOCATION N 5 198 836.4 E 272 326.9 (Chippewa River Bridge) | ORIGINATED BY ES

HWY 17 | BOREHOLE TYPE Dynamic Cone Penetration Test | COMPILED BY AN

DATUM Geodetic | DATE 2010.10.20 - 2010.10.20 | CHECKED BY JL

ELEV / DEPTH	DESCRIPTION (SOIL PROFILE)	STRAT PLOT	NUMBER (SAMPLES)	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, X 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
191.6 / 0.0	Auger to 1.0m, then start DCPT						191, 190, 189, 188				
187.0 / 4.6	END OF DCPT AT 4.6m. BOREHOLE BACKFILLED WITH HOLEPLUG TO 0.1m THEN ASPHALT TO SURFACE.										



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

15-○-5 (20, 10) (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

RECORD OF BOREHOLE No DCPT-2B

METRIC

W.P. 93-89-00	LOCATION N 5 198 834.5 E 272 326.3 (Chippewa River Bridge)	ORIGINATED BY ES	
HWY 17	BOREHOLE TYPE Dynamic Cone Penetration Test	COMPILED BY AN	
DATUM Geodetic	DATE 2010.10.20 - 2010.10.20	CHECKED BY JL	

ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE) 20 40 60 80 100	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L — WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
191.6 / 0.0	Auger to 1.0m, then start DCPT						191, 190, 189, 188, 187, 186				
185.5 / 6.1	END OF DCPT AT 6.1m. BOREHOLE BACKFILLED WITH HOLEPLUG TO 0.1m THEN ASPHALT TO SURFACE.										

ONTMT4S 1185.GPJ 7/21/11

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

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

Ministry of Transportation
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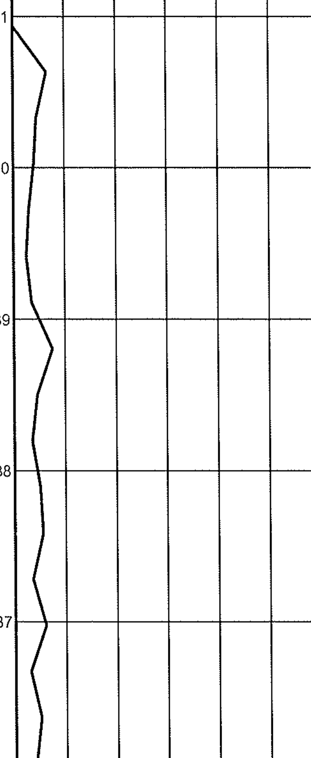
RECORD OF BOREHOLE No DCPT-3A

METRIC

W.P. 5198-06-00	LOCATION N 5 198 815.1 E 272 353.9 (Chippewa River Bridge)	ORIGINATED BY ES
HWY 17	BOREHOLE TYPE Dynamic Cone Penetration Test	COMPILED BY AN
DATUM	DATE 2010.10.20 - 2010.10.20	CHECKED BY JL

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	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
191.5											
0.0	Auger to 1.0m, then start DCPT						191, 190, 189, 188, 187				
186.0 / 5.5	END OF DCPT AT 5.5m ON CONCRETE ARCH. BOREHOLE BACKFILLED WITH HOLEPLUG TO 0.1m THEN ASPHALT TO SURFACE.										

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

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

ONTMT4S 1185.GPJ 3/23/11

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Ontario

RECORD OF BOREHOLE No DCPT-3B

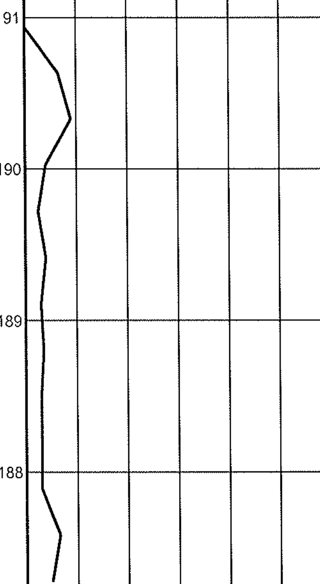
METRIC

W.P. 5198-06-00 | LOCATION N 5 198 817.1 E 272 354.4 (Chippewa River Bridge) | ORIGINATED BY ES

HWY 17 | BOREHOLE TYPE Dynamic Cone Penetration Test | COMPILED BY AN

DATUM | DATE 2010.10.20 - 2010.10.20 | CHECKED BY JL

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH kPa	WATER CONTENT (%)	UNIT WEIGHT γ kN/m3	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
191.5											
0.0	Auger to 1.0m, then start DCPT						191, 190, 189, 188				
186.9 4.6	END OF DCPT AT 4.6m ON CONCRETE ARCH. BOREHOLE BACKFILLED WITH HOLEPLUG TO 0.1m THEN ASPHALT TO SURFACE.						187				



Legend: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE; PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L

+ 3, × 3: Numbers refer to Sensitivity

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

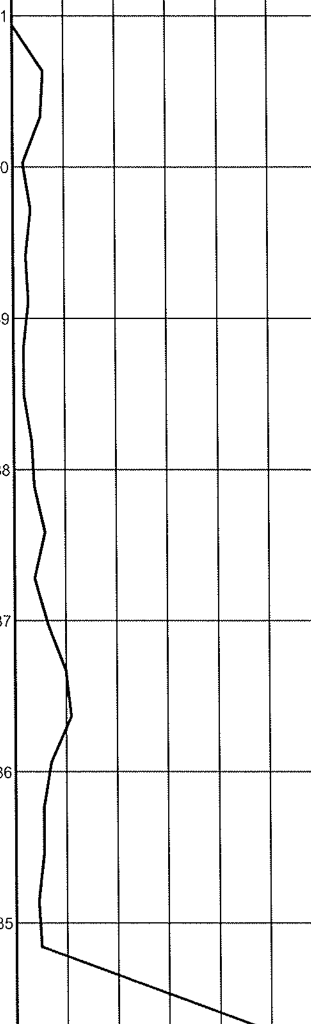
RECORD OF BOREHOLE No DCPT-4A

METRIC

W.P. 5198-06-00 LOCATION N 5 198 814.6 E 272 365.5 (Chippewa River Bridge) ORIGINATED BY ES

HWY 17 BOREHOLE TYPE Dynamic Cone Penetration Test COMPILED BY AN

DATUM DATE 2010.10.20 - 2010.10.20 CHECKED BY JL

ELEV / DEPTH	SOIL PROFILE: DESCRIPTION	STRAT PLOT	SAMPLES: NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE (20 40 60 80 100)	PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L — WATER CONTENT (%) (20 40 60)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
191.5 0.0	Auger to 1.0m, then start DCPT						191 190 189 188 187 186 185				
184.2 7.3	END OF DCPT AT 7.3m UPON CONE REFUSAL. BOREHOLE BACKFILLED WITH HOLEPLUG TO 0.1m THEN ASPHALT TO SURFACE.										

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

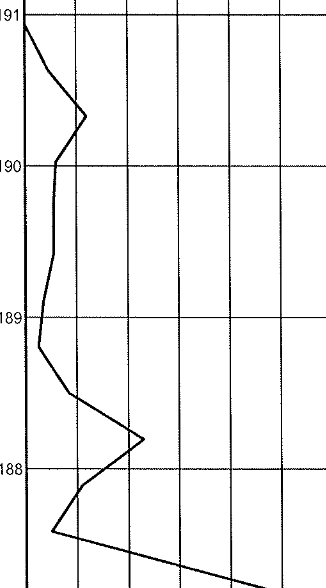
15-⊕-5 (20 above, 10 below) (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

RECORD OF BOREHOLE No DCPT-4B

METRIC

W.P. 5198-06-00	LOCATION N 5 198 814.3 E 272 363.1 (Chippewa River Bridge)		ORIGINATED BY ES
HWY 17	BOREHOLE TYPE Dynamic Cone Penetration Test		COMPILED BY AN
DATUM	DATE 2010.10.20 - 2010.10.20		CHECKED BY JL

ELEV / DEPTH	SOIL PROFILE: DESCRIPTION	STRAT PLOT	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: "N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, X 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
191.5 / 0.0	Auger to 1.0m, then start DCPT						191, 190, 189, 188				
187.1 / 4.4	END OF DCPT AT 4.4m UPON REFUSAL. BOREHOLE BACKFILLED WITH HOLEPLUG TO 0.1m THEN ASPHALT TO SURFACE.										

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

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

Appendix B

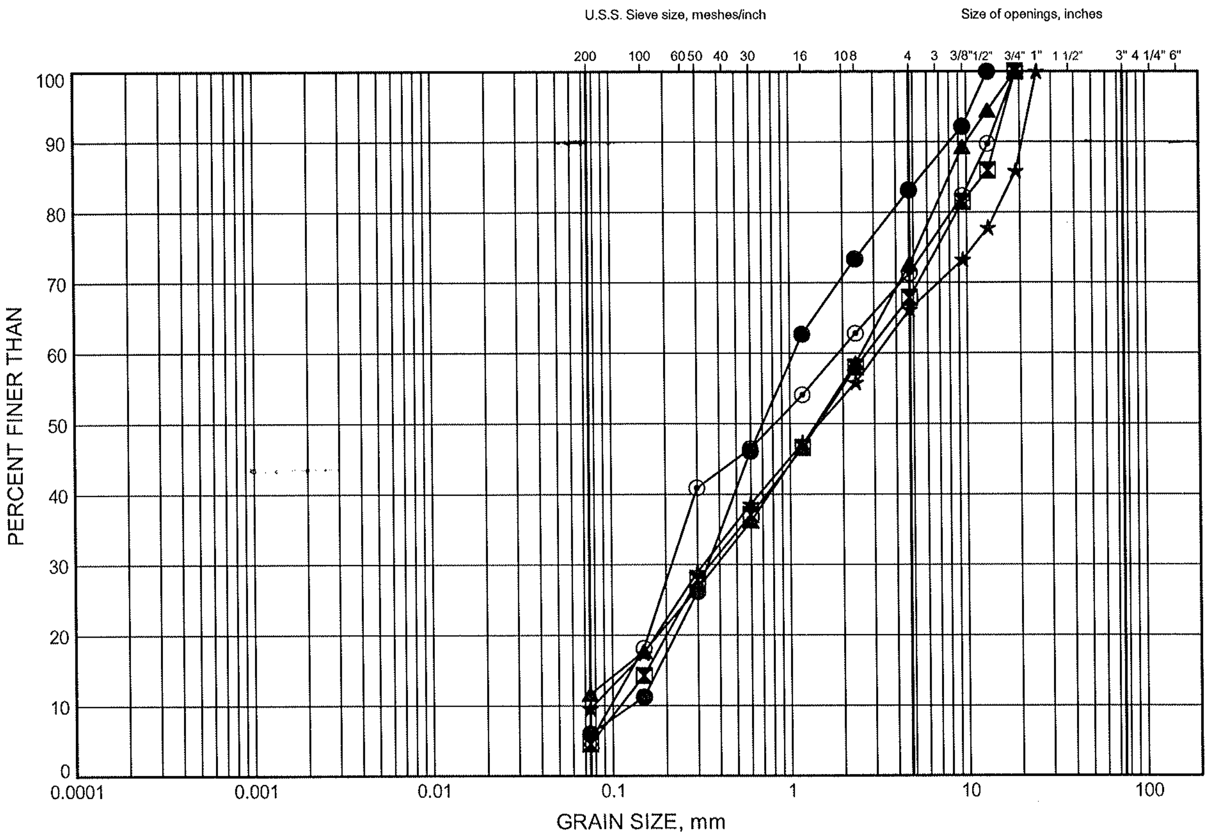
Laboratory Test Results

Ten Bridge Rehabilitations and Two Bridge Replacements

GRAIN SIZE DISTRIBUTION

FIGURE B1

FILL (SAND, GRAVELLY SAND)



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	CHIP-02	0.91	190.65
⧗	CHIP-02	2.13	189.43
▲	CHIP-03	0.38	191.12
★	CHIP-03	0.91	190.59
⊙	CHIP-03	3.35	188.15

W.P.# 5198-06-00

Prepared By AN

Checked By RPR

THURBER

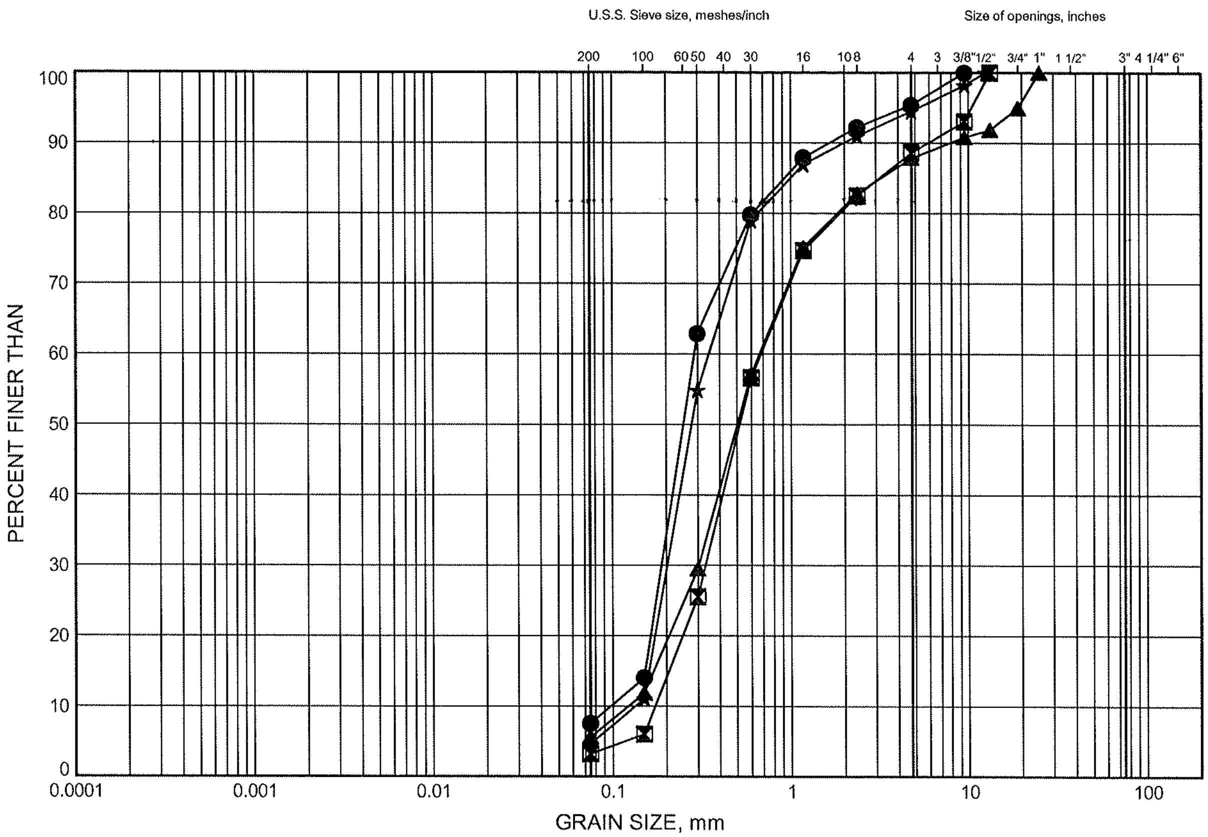
GRAIN SIZE DISTRIBUTION - THURBER 1185.GPJ 2/25/11

Ten Bridge Rehabilitations and Two Bridge Replacements

GRAIN SIZE DISTRIBUTION

FIGURE B2

SAND



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	CHIP-01	1.52	190.11
⊠	CHIP-01	3.35	188.29
▲	CHIP-01	5.18	186.46
★	CHIP-01	7.62	184.02

W.P.# 5198-06-00

Prepared By AN

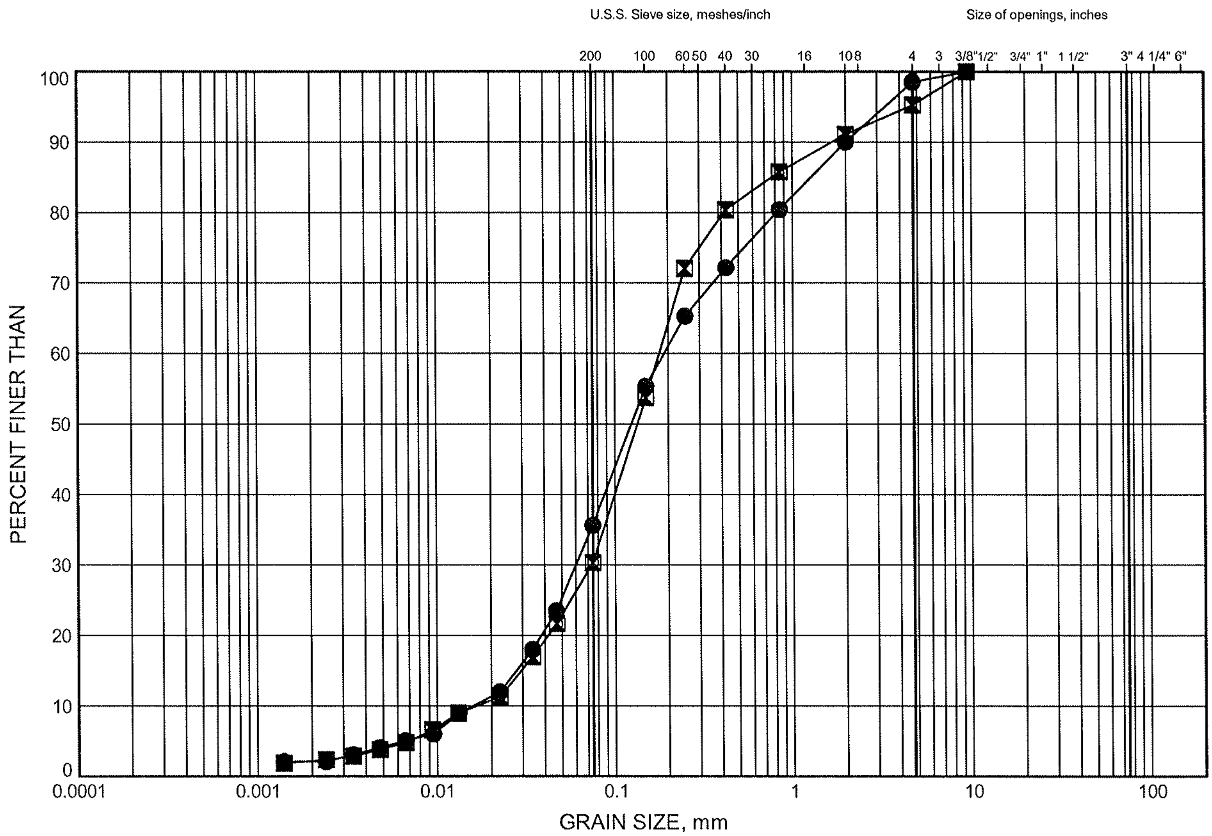
Checked By RPR

THURBER

GRAIN SIZE DISTRIBUTION - THURBER 1185.GPJ 2/25/11

Ten Bridge Rehabilitations and Two Bridge Replacements GRAIN SIZE DISTRIBUTION	FIGURE B3

SILTY SAND



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	CHIP-04	2.13	189.34
☒	CHIP-04	3.35	188.12

GRAIN SIZE DISTRIBUTION - THURBER 1185.GPJ 2/25/11

W.P.# 5198-06-00
Prepared By AN
Checked By RPR

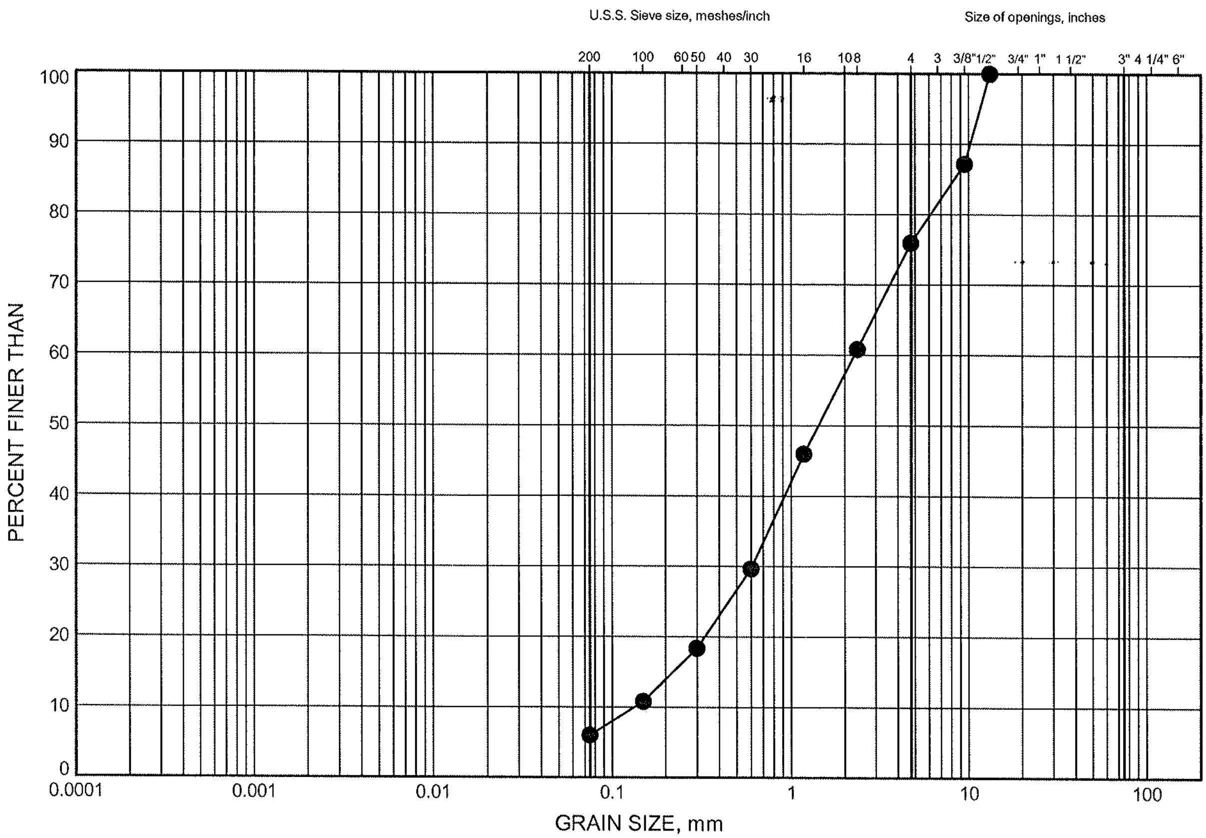
THURBER

Ten Bridge Rehabilitations and Two Bridge Replacements

GRAIN SIZE DISTRIBUTION

FIGURE B4

GRAVELLY SAND



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	CHIP-04	5.79	185.68

GRAIN SIZE DISTRIBUTION - THURBER 1185.GPJ 2/25/11

W.P.# 5198-06-00

Prepared By AN

Checked By RPR

THURBER

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GEOTECHNICAL • ENVIRONMENTAL • MATERIALS

POINT LOAD TEST SHEET

Job No : 19-1351-185
Client : McCormick Rankin Corporation
Project Name : Ten Bridge Rehabilitations and Two Bridge Replacements
Date Drilled : 19/10/2010
Date Tested : 27/10/2010
Core Size : NQ **BH No :** CHIP-01
Tester : SLL

Test No.	Run No.	Depth (m)	Axial or Diametral	Force (kN)	Diameter (mm)	Length (mm)	UCS (MPa)	Rock Type	Notes
1	1	8.6	D	18.2	47.6	103.5	188.6	GRANITE	Very Strong
2	2	9.1	D	10.8	47.6	112.4	111.9	GRANITE	Very Strong
3	2	9.3	D	14.8	47.6	126.6	153.3	GRANITE	Very Strong
4	2	9.8	D	15.5	47.6	18.6	160.6	GRANITE	Very Strong
5	2	10.0	D	0.6	47.6	109.4	6.2	GRANITE	Weak
6	2	10.2	D	3.0	47.6	106.4	31.1	GRANITE	Medium Strong
7	3	10.6	D	5.5	47.6	120.9	57.0	GRANITE	Strong
8	3	10.9	D	10.1	47.6	118.7	104.6	GRANITE	Very Strong
9	3	11.2	D	9.4	47.6	126.0	97.4	GRANITE	Strong
10	3	11.6	D	16.0	47.6	123.6	165.8	GRANITE	Very Strong
11	3	11.7	D	21.2	47.6	109.4	219.6	GRANITE	Very Strong
12									
13									
14									
15									
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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.

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POINT LOAD TEST SHEET

Job No :	19-1351-185	**Client :**	McCormick Rankin Corporation
Project Name :	Ten Bridge Rehabilitations and Two Bridge Replacements	**Date Drilled :**	19/10/2010
		Date Tested :	27/10/2010
Core Size :	NQ **BH No :** CHIP-04	**Tester :**	SLL

Test No.	Run No.	Depth (m)	Axial or Diametral	Force (kN)	Diameter (mm)	Length (mm)	UCS (MPa)	Rock Type	Notes
1	1	7.9	D	19.0	47.6	131.5	196.9	GRANITE	Very Strong
2	2	8.3	D	13.8	47.6	107.7	143.0	GRANITE	Very Strong
3	2	8.7	D	6.8	47.6	134.4	70.5	GRANITE	Strong
4	2	9.4	D	16.4	47.6	93.8	169.9	GRANITE	Very Strong
5	3	9.9	D	18.8	47.6	120.4	194.8	GRANITE	Very Strong
6	3	10.3	D	21.1	47.6	89.4	218.6	GRANITE	Very Strong
7									
8									
9									
10									
11									
12									
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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.

Appendix C

Certificate of Analysis

AGAT Laboratories

Certificate of Analysis

AGAT WORK ORDER: 11T477330

PROJECT NO: 19-1351-185

5835 COOPERS AVENUE
MISSISSAUGA, ONTARIO
CANADA L4Z 1Y2
TEL (905)712-5100
FAX (905)712-5122
http://www.agatlabs.com

CLIENT NAME: THURBER ENGINEERING LTD

ATTENTION TO: Rocio Palomeque Reyna

Chloride (Soil)

DATE SAMPLED: Oct 19, 2010 **DATE RECEIVED: Mar 09, 2011** **DATE REPORTED: Mar 17, 2011** **SAMPLE TYPE: Soil**

Parameter	Unit	G / S	RDL	CHIP-02,SS3 2293519	CHIP-03,SS4 2293520	CHIP-04,SS5 2293521	CHIP-01,SS1 2293522
Chloride (2:1)	µg/g		2	657	329	2270	169

Comments: RDL - Reported Detection Limit; G / S - Guideline / Standard

2293519-2293522 Chloride was determined on the extract obtained from the 2:1 leaching procedure (2 parts DI water:1 part soil).

Certified By: Elizabeth Polakowska

Appendix D

Leakage/Water seepage Control Alternatives Comparison

COMPARISON OF LEAKAGE/WATER SEEPAGE CONTROL ALTERNATIVES

Waterproof membrane on top of the concrete arch structure	Waterproof membrane just beneath the pavement structure	Pressure Grouting
Advantages: i. Allows repairs of the concrete arch. ii. Extends the durability of the bridge.	***Advantages:*** i. Practical to be installed at the site. ii. Extends the durability of the bridge. iii. No deep excavation required. iv. No roadway protection or detour required.	***Advantages:*** i. Less disruption to traffic. ii. Creates an impervious backfill. iii. Chemical grouting is generally lighter in weight and there is less concern about freeze-thaw effects. iv. Minor repair to the bridge pavement surface.
Disadvantages: i. Labour intensive to remove granular fill within the confined space between arch buttresses. ii. Requires high quality control and workmanship. iii. Likely, requires a detour.	***Disadvantages:*** i. This option does not allow repair of the arch bridge.	***Disadvantages:*** i. Requires a design by a specialist grouting contractor ii. Special equipment iii. Monitoring should be conducted to prevent grout to flow into the river.
FEASIBLE	**FEASIBLE**	**FEASIBLE**

Appendix E

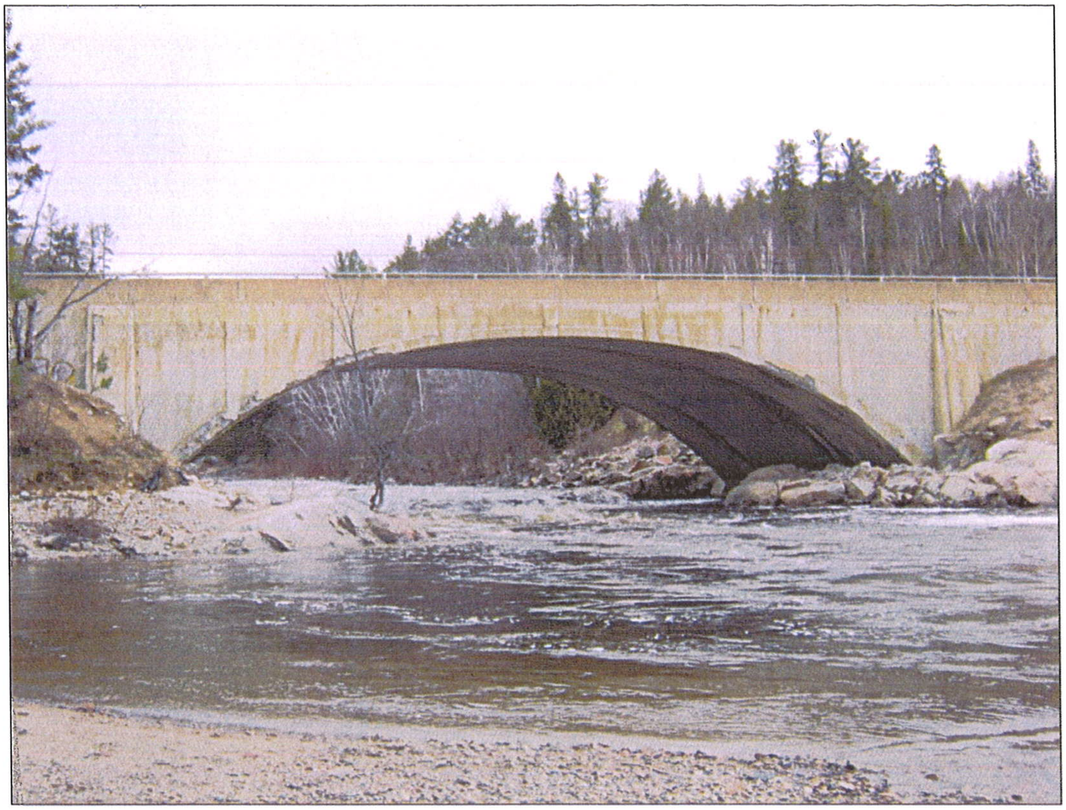
List of SPs and OPSS

The following OPSS and OPSD documents are referenced in this report:

OPSS 804, November 2010
OPSS 902, November 2010
OPSS 1010
OPSD 3101.150
OPSS 501 dated November 2010
OPSD 208.010

Appendix F

Site Photographs



Photographs 1 and 2 – General view of the south side of the Chippewa River Bridge (downstream)

D R A F T

Photograph 3 - Existing conditions of the west slopes/approaches on the south side of the bridge

Photograph 4 - Existing conditions of east slopes/approaches on the south side of the bridge

Photographs 5 and 6- General view of the north side of the Chippewa River Bridge (upstream)

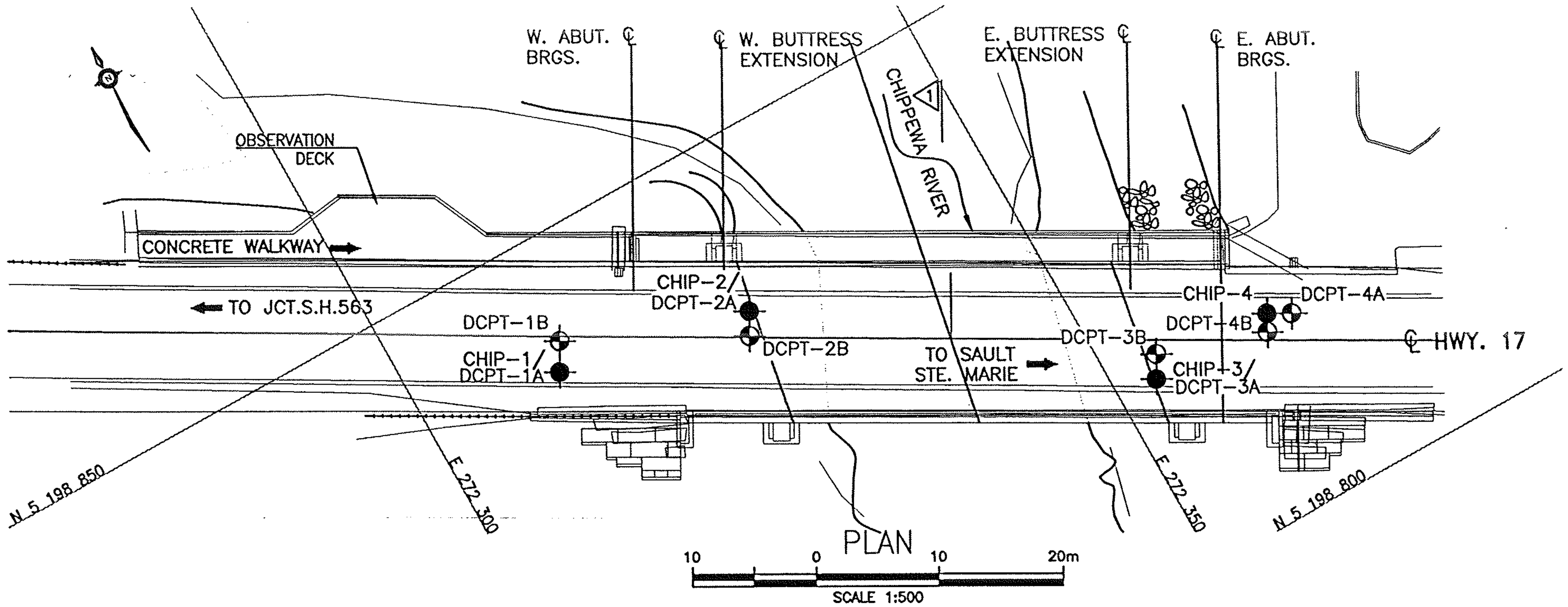
DRAFT

Photograph 7- South side of the Chippewa River Bridge (downstream)

Photograph 8- North side of the Chippewa River Bridge (upstream)

Appendix G

Drawing titled "Borehole Locations and Soil Strata"



METRIC

DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN

CONT No
WP No 5141-08-00

CHIPPEWA RIVER BRIDGE REHABILITATION HWY 17

BOREHOLE LOCATIONS AND SOIL STRATA

SHEET

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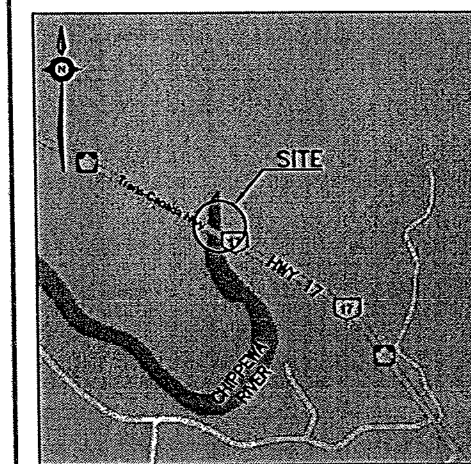
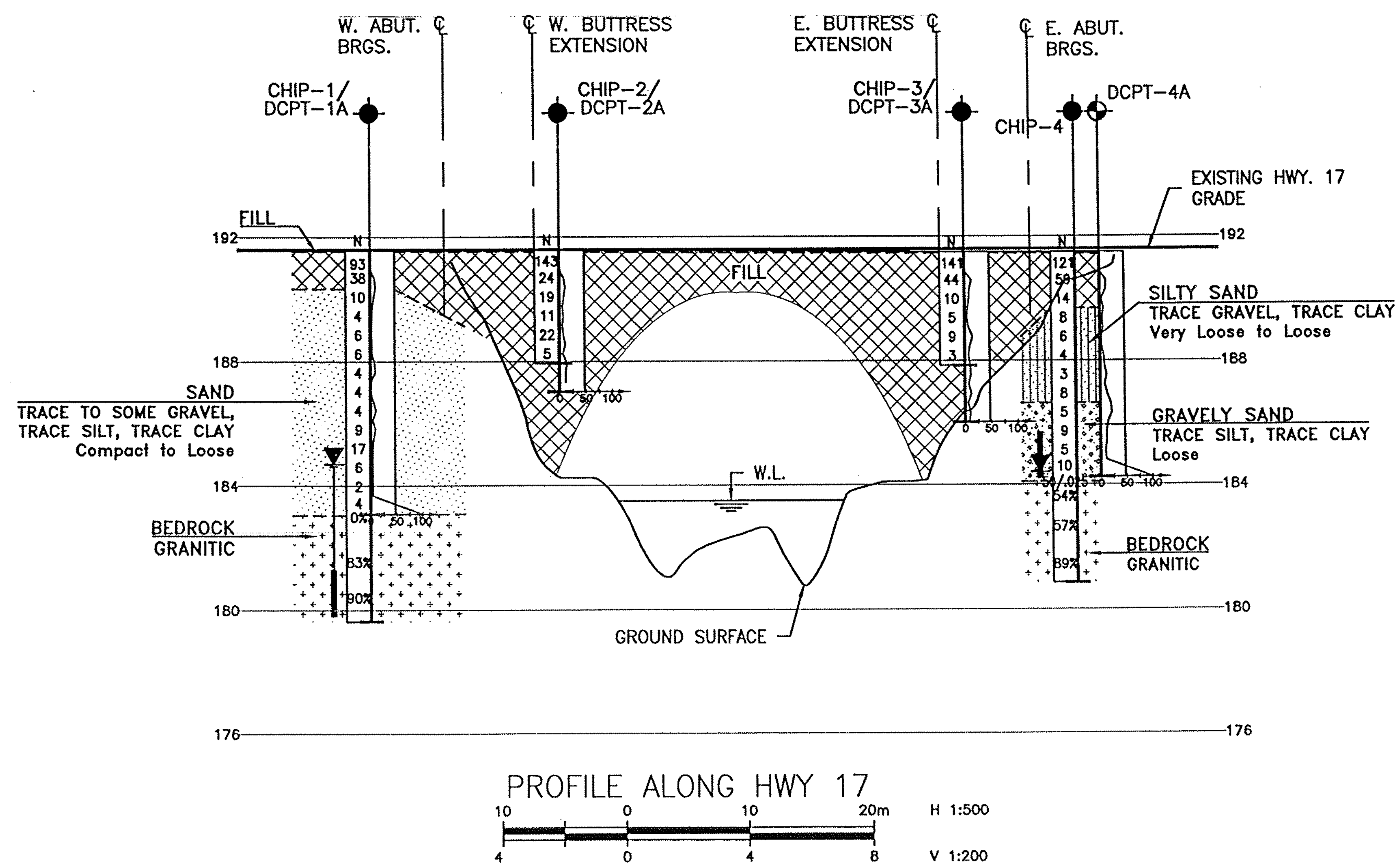


KEYPLAN

LICENSED PROFESSIONAL ENGINEER
R. Palomeque Reyna
100083209
Nov 3/2011
PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER
P. K. CHATTERJI
Nov 3/2011
PROVINCE OF ONTARIO



LEGEND

⌖	Borehole
⌖	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
CHIP-1	191.6	5 198 839.4	272 311.5
CHIP-2	191.6	5 198 836.2	272 327.3
CHIP-3	191.5	5 198 815.4	272 353.4
CHIP-4	191.5	5 198 815.6	272 363.8
DCPT-1A	191.6	5 198 839.1	272 311.9
DCPT-1B	191.6	5 198 841.6	272 312.7
DCPT-2A	191.6	5 198 836.4	272 326.9
DCPT-2B	191.6	5 198 834.5	272 326.3
DCPT-3A	191.5	5 198 815.1	272 353.9
DCPT-3B	191.5	5 198 817.1	272 354.4
DCPT-4A	191.5	5 198 814.6	272 365.5
DCPT-4B	191.5	5 198 814.3	272 363.1

-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. 41K-89

REVISIONS	DATE	BY	DESCRIPTION			
DESIGN RPR	CHK	CODE		LOAD		DATE NOV. 2011
DRAWN AN	CHK RPR	SITE		STRUCT		DWG 1