

**DETAIL FOUNDATION INVESTIGATION AND DESIGN REPORT
WABIGOON RIVER BRIDGE REPLACEMENT
HIGHWAY 605
NORTHWESTERN REGION
SITE NUMBER 41S-40

AGREEMENT NUMBER 6007-E-0022
GWP # 454-00-00**

Submitted to:
Ministry of Transportation
Northwestern Region

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July 2008
AMEC Ref.: TT83000

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DRAWING Nos. 1, 2

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1.0 INTRODUCTION

AMEC Earth & Environmental, a Division of AMEC Americas Limited (AMEC), Consulting Geotechnical, Construction Quality Control and Environmental Engineers, was retained by the Ministry of Transportation Ontario (MTO), Northwestern Region, to conduct a detail foundation investigation and design for Wabigoon River Bridge Replacement on Highway 605 at its intersection with Wabigoon River, located approximately 5.6 km north of Highway 17 in the northwestern region of Ontario.

Two (2) 30 m deep boreholes close to the existing bridge abutments and two (2) 10 m deep boreholes in the approach areas within 20 m of the abutments, were specified by MTO in the Terms of Reference (TOR) as outlined in the Request for Quotation (Agreement Number: 6007-E-0022 dated 27 November 2007). The Scope of Work under the assignment also included the foundation investigation for Moose Creek Bailey Bridge site, the results of which were presented in a separate report (submitted 14 March 2008). Authorization to proceed with this investigation was signed by the Regional Director of MTO dated 19 December 2007. The work was carried out by AMEC according to the MTO Terms of Reference and AMEC's Proposal No. P27411 dated 12 December 2007.

A previous geotechnical report at the existing bridge site was obtained by AMEC from the MTO GEOCRES library (W.J. 62-F-116 & W.J. 62-F-117, District #20, GEOCRES No. 52F-3 and 52F-3-1). The report contained investigation reports for two bridge sites. The site in the report that was relevant to this investigation was Hwy. 605 Bridge (W.J. 62-F-117). The report described four (4) boreholes drilled at the site at the end of 1962, which included two cone penetration holes. Additional information (i.e. site profile of the existing bridges) was also forwarded to AMEC.

The investigation was carried out by means of a limited number of boreholes, in-situ tests and laboratory tests on selected samples. The factual results of the soil conditions encountered in the boreholes and laboratory tests are presented in this report.

2.0 SITE DESCRIPTION

The site for the foundation investigation was the existing bridge over Wabigoon River located on Highway 605, approximately 5.6 km north of Highway 17 in the northwestern region of Ontario, as shown in the site location plan (Drawing No. 1). At the time of investigation, the existing bridge was a 3-span Steel Stringer Bridge with concrete deck and curbs, founded on two sets of concrete piers in Wabigoon River and two concrete abutments on the river banks. The bridge was about 40 m long and 6 m wide. The bridge deck was approximately 4.4 m above the river. At the time of investigation, the river was covered with snow and ice.

3.0 GEOLOGY

According to Northern Ontario Engineering Geology Terrain Study (Ontario Geological Survey, 1980), the Wabigoon River area is described as peaty organic terrain plain (minor sandy alluvial plain) with low planar local relief and mixed wet and dry surface conditions.

4.0 INVESTIGATION PROCEDURES

4.1 FIELD INVESTIGATION

In accordance with the Terms of Reference for this investigation, four (4) borehole locations (BH 1-2008 to BH 4-2008) were to be investigated for foundation design. BH 2-2008 and BH 4-2008 were to be located close to the existing bridge abutments. BH 1-2008 and BH 3-2008 were to be located in the approach areas within 20 m of the bridge abutments.

The fieldwork was performed from 10 to 13 March 2008, starting with stake out of the borehole locations, followed by underground utilities clearance, acquiring all necessary permits for road occupancy, and drilling the boreholes using a truck-mounted drilling rig. The drilled borehole locations were surveyed in the field by AMEC.

The borehole locations and the general soil stratigraphy at the bridge site are presented on Drawing No. 2 (two sheets). The ground surface elevations at the specified borehole locations were surveyed by AMEC after drilling, with reference to a benchmark (BM) location near the bridge provided by MTO. The BM (Elevation 100.206 m) was marked in a Poplar in the northwest area of the bridge at station 10+306.81 (17.2 LT).

The borehole investigation was carried out under the full-time supervision of experienced geotechnical personnel from AMEC. The boreholes were drilled using tri-cone augers with casing advanced through the borehole.

Boreholes BH 1-2008 and BH 3-2008, both of which were located in the approach areas, were each advanced to a depth of 9.6 m (Elevations 91.8 m and 91.0 m respectively). Borehole BH2-2008 was advanced to a depth of 32.2 m (Elevation 68.9 m), where bedrock was encountered. The bedrock was cored down to a depth of 33.7 m (Elevation 67.3 m). Similarly, borehole BH4-2008 was advanced to a depth of 24.4 m (Elevation 76.2 m), before bedrock was encountered and was cored down to a depth of 27.4 m (Elevation 73.2 m). Rock core samples were obtained from both boreholes. Photographs of the core samples are presented in Appendix G.

Soil samples were taken at 0.76 m intervals in the upper 10 m and 1.5 m intervals thereafter up to a depth of approximately 20 m. Sampling interval was increased to 3.0 m beyond 20 m depth. Soil sampling was done during the performance of Standard Penetration Test (SPT) in accordance with ASTM D1586. In-situ field MTO shear vane tests and collection of undisturbed samples using thin walled tubes were carried out intermittently. The SPT consisted of freely dropping a 63.5 kg (140 lbs.) hammer for a vertical distance of 0.76 m (30 inches) to drive a 51 mm (2 inches) diameter O.D. split-barrel (split spoon) sampler into the ground. The number of blows of the hammer required to drive the sampler into the relatively undisturbed ground by a vertical distance of 0.30 m (12 inches) was recorded as SPT 'N' value of the soil which indicated the consistency of cohesive soils or the relative density of non-cohesive soils.

Soil samples were collected for each soil layer exposed in the boreholes for laboratory inspection and testing.

The collected soil samples were screened by a gas detector for the total organic vapours (TOV), the results of which are shown in the Record of Boreholes in Appendix A.

Upon completion of drilling, the boreholes were backfilled with bentonite in accordance with the general requirements of Ministry of the Environment Regulation 903 as indicated in the Record of Boreholes.

The soil samples were transported to AMEC's Advanced Soil Laboratory in Scarborough (Toronto) for further examination and laboratory soil testing. The program of laboratory testing included, where applicable, grain size analysis, Liquid and Plastic Limits, in-situ water content determination, consolidation test and soil corrosivity analysis.

The results of the in-situ and laboratory tests are presented in the corresponding Record of Boreholes (Appendix A) and Laboratory Test Results (Appendix B).

4.2 LABORATORY TESTS

Representative soil samples were subject to laboratory testing in AMEC's Advanced Soil Laboratory in Scarborough (Toronto) for soil classification. The following tests were conducted:

- In-situ water content determination (56);
- Grain size distribution analysis (10);
- Liquid and Plastic Limits (10);
- Soil Corrosivity test (1); and
- One-Dimensional Consolidation Test (1).

The results of the laboratory tests are included in the Record of Boreholes in Appendix A. The grain size distribution curves, Liquid / Plastic Limits, Soil Corrosivity test results and the results of the One-Dimensional Consolidation Test are shown in Appendix B.

5.0 SUB-SURFACE CONDITIONS

In brief, the soil profile consisted of surficial sand, with some silt and gravel, or gravelly sand, underlain by fill soils. The fill was underlain by very soft silty clay, except in Borehole BH 4-2008, where clayey silt soil, with organic matters and wood pieces, was encountered over the silty clay. **Bedrock was encountered in both deep boreholes, BH 2-2008 and BH 4-2008, at depths of 32.2 m and 24.4 m respectively.**

Groundwater was not encountered in both the shallow boreholes in the approach areas (BH 1-2008 and BH 3-2008). As coring water was used, the groundwater level could not be established with certainty. Water level was measured at 6.3 m and 9.1 m below the ground surface in BH 2-2008 and BH 4-2008 respectively at the completion of drilling.

The stratigraphic units and groundwater conditions at the borehole locations are discussed in the following sections. Detailed information is provided in the Record of Boreholes (Appendix A).

The following summary is to assist the designers of the project with an understanding of the

anticipated soil conditions across the site. However, it should be noted that the soil and groundwater conditions may vary between the borehole locations.

5.1 SAND / SILTY CLAY / CLAYEY SILT FILL

The boreholes were drilled at the edge of the driving road surface, about 1.0 to 1.5 m from the edge of the road. At the time of the drilling, the ground was frozen to a maximum depth of about 2.2 m.

The surficial fill comprised a surficial sand fill with thickness ranging from about 1.5 m to 2.0 m (to lowest Elevation 99.0 m), underlain by silty clay / clayey silt fill up to a depth of about 2.3 m to 2.9 m (lowest Elevation 97.7 m). The sand fill included varying amounts of gravel and silt. Trace to some sand was noted in the silty clay / clayey silt fill.

The SPT 'N' values of the silty clay / clayey silt fill varied from 9 to 28 blows per 0.3 m, indicating a stiff to very stiff consistency. A SPT 'N'-value of 46 blows per 0.3 m was recorded in borehole BH 4-2008. However, the soil sample obtained was frozen. The sand fill samples in all boreholes were obtained as auger samples, as they were frozen.

The results of laboratory tests conducted on selected soil samples are as follows:

Natural moisture content (%):	Sand Fill:	3.6 to 10.2	
	Clay Fill:	23.1 to 36.9	
Plastic Limit (2 samples):	22		
Liquid Limit (2 samples):	53 and 55		
Grain size (2 samples):	Gravel (%):	7 and 25 <1 to 2	(Sand Fill) (Clay Fill)
	Sand (%):	61 and 63 ~ 3	(Sand Fill) (Clay Fill)
	Silt (%):	12 and 22 36 and 37	(Sand Fill) (Clay Fill)
	Clay (%):	0 and 10 58 and 61	(Sand Fill) (Clay Fill)

The plasticity indices are plotted in Figure No. B1 and the grain size distribution curves are presented in Figure Nos. B3 and B4 in Appendix B.

5.2 CLAYEY SILT / SILT

Clayey silt / silt deposit was encountered in BH 4-2008 underlying the surficial fill from a depth of about 2.3 m to 6.6 m (Elevation 98.3 m to 94.0 m). The clayey silt deposit was mixed with organic matters and wood fragments were noted in some samples. A pocket of wet sand and gravel was noted in the undisturbed thin wall sample (TW 5) extracted from a depth of about 3.8 m.

The SPT 'N' values of the clayey silt / silt deposit ranged from 3 to 10 blows per 0.3 m (firm to stiff consistency).

The results of a MTO field vane test carried out within the soil deposit in BH 4-2008 at a depth of about 4.1 m are as follows:

Undrained shear strength (kPa):	22
Remoulded undrained shear strength (kPa):	17
Soil sensitivity:	1.3

The results of laboratory tests conducted on soil samples are as follows:

Natural moisture content (%):	18 to 42
Plastic Limit (1 sample):	19
Liquid Limit (1 sample):	33
Grain size (1 sample):	Gravel (%): <1
	Sand (%): ~9
	Silt (%): 68
	Clay (%): 23

The plasticity index is plotted in Figure No. B2 and the grain size distribution curve is presented in Figure No. B5 in Appendix B.

5.3 SILTY CLAY / SILT AND CLAY / CLAY (VARVED)

The fill soils and clayey silt / silt deposit were underlain by high-plasticity silty clay / silt and clay / clay (varved) deposit from a depth between about 2.9 m (highest Elevation 98.5 m) in BH 1-2008 to BH 3-2008 and 6.6 m (Elevation 94.0 m) in BH 4 to a depth of about 32.2 m (Elevation 68.9 m) in BH 2-2008 and 24.4 m (Elevation 76.2 m) in BH 4-2008. Boreholes BH 1-2008 and BH 3-2008 were terminated at a depth of about 9.6 m (Elevations 91.8 m and 91.0 m respectively). The SPT 'N' values of the silty clay / silt and clay / clay deposit varied from 0 to 13 blows per 0.3 m, indicating a very soft to stiff consistency. All SPT 'N' values higher than 3 (soft to stiff consistency) were at the upper 2.0 m portion of the deposit.

The results of the MTO field vane tests carried out within the soft silty clay / silt and clay / clay are as follows:

Undrained shear strength (kPa)	11 to 106
Remoulded undrained shear strength (kPa)	6 to 44
Soil sensitivity	2 to 4

The results of laboratory tests conducted on soil samples are as follows:

Natural moisture content (%):	27 to 79
Plastic Limit (4 samples):	17 to 19
Liquid Limit (4 samples):	41 to 66

Grain size (5 samples):	Sand (%):	0 to 2
	Silt (%):	30 to 56
	Clay (%):	42 to 70

The plasticity indices are plotted in Figure No. B1 and B2 and the grain size distribution curves are presented in Figure No. B6 in Appendix B.

A one-dimensional consolidation test was carried out on one undisturbed thin wall sample (TW 9) from BH 2-2008. The void ratio and coefficient of consolidation (C_v) plotted versus log of pressure are presented in Figure No. B7 in Appendix B. The estimated consolidation parameters are follows:

p_0'	= effective overburden pressure	=	108 kPa
p_c	= preconsolidation pressure	=	89 kPa
OCR	= overconsolidation ratio	=	0.82 (underconsolidated / disturbed)
C_v	= coefficient of consolidation	=	0.023 m ² /day at p_0'
C_c	= compression index	=	0.27
C_r	= recompression index	=	0.05
C_s	= swelling index	=	0.03

5.4 BEDROCK

Bedrock was encountered in both deep boreholes near the abutments, BH 2-2008 and BH 4-2008, at depths of 32.2 m (Elevation 68.9 m) and 24.4 m (Elevation 76.2 m) respectively. BH 2-2008 was drilled near the south abutment and BH 4-2008 was drilled near the north abutment. Fragments of weathered rock (sand and gravel size) were noted at a joint in the rock core sample for BH 4-2008 at a depth of about 25.2 m (Elevation 75.5 m), followed by sound rock core sample. The rock core consisted of granite. The Sample Core Recovery (SCR) in both boreholes was 100 %. The Rock Quality Designation (RQD) measured in the cores obtained were 88.4 % and 78.0 % in BH 2-2008 and BH 4-2008 respectively, indicating good quality bedrock. Weathered joints were present within the rock cores. Photographs of the rock cores are included in Appendix G.

5.5 GROUNDWATER

At the end of drilling, the water level was measured at 6.3 m and 9.1 m below ground surface in BH 2-2008 and BH 4-2008 respectively. Groundwater was not encountered in BH 1-2008 and BH 3-2008, which were both drilled to a depth of about 9.6 m below ground level. However, due to the use of coring water, groundwater levels measured may not be representative and may be different from the actual groundwater level.

As per the previous report prepared in 1963, obtained from MTO, (refer to Section 1.0) – “When contact was made with the bedrock in borehole 3, artesian water was observed. A maximum head of 2.25 feet above ground level, corresponding to an elevation of 1146.25, was recorded.” The water levels, as measured during the previous investigation in 1963, were 10 feet (3.0 m) and 11 feet (3.4 m) in BH 1 and BH 2 respectively. Copies of the borehole logs from the previous investigation at the bridge site are attached in Appendix D.

It should be noted that the groundwater at the site would fluctuate seasonally and can be expected to be somewhat higher during the spring months and in response to major weather events / water levels in the river.

6.0 DISCUSSIONS AND RECOMMENDATIONS

Based on the information provided by MTO, the existing, about 40 m long, bridge would be demolished and replaced by a new bridge. Details of the new replacement bridge were not available at the time of preparation of this report. As per the RFQ, the replacement structure is to be designed by the MTO internally. According to MTO, the new bridge could be approximately 45 m long and single-span, with the existing bridge abutments and piers left in-place.

As noted in the previous investigation report, the bridge foundations are supported by concrete caissons. The size and depth of the caissons were unknown. The existing bridge abutments and piers would be demolished down to the ground surface. The remaining existing caissons may provide additional resistance to potential slope movement. **It is therefore imperative that the as-installed caisson details (i.e., location, size and depth) be verified and the new piles/caissons be driven/installed at a sufficient distance away from the existing caissons.**

It should be noted that, according to the 1963 report obtained from GEOCREs as mentioned in Section 6.1, the existing bridge built in 1940 had experienced some movements at the abutments.

Based on the soil conditions encountered in the boreholes drilled for at the existing bridge abutments, deep foundations using steel piles driven to bedrock would be the preferred foundation type. Because of the considerable depth to bedrock (about 26 m to 33 m deep) and the very soft clayey soil strata, cast-in-situ caissons will be difficult to be installed. Shallow foundations (spread/strip footings) should not be used due to the low bearing capacity of the very soft clayey soil subgrade.

Due to the quite abrupt change in the soil profile from soft clayey soils to sound bedrock, driven steel piles should either be HP piles equipped with rock shoe or pipe piles equipped with proper shoe. It should be noted that driven steel piles alone may not be able to penetrate sufficiently into the bedrock for good pile-end anchorage. As such, steel pipe piles may be considered and the contract may provide a provisional item for rock sockets at the bottom end of the steel pipe piles if the steel pipe piles can not penetrate sufficiently into the bedrock.

From the facts that the existing caisson details and the integrity of the caissons after more than 65 years of service are not known, the new bridge may not be able to depend on the existing caissons to enhance the slope stability of the new bridge abutments and the associated river bank slopes. Without the contribution of the existing caissons to improve slope stability, the new river bank slope under the new bridge should be at a slope of 15H:1V (average) or flatter (refer to Table 3) to prevent deep-seated slope instability and the new bridge approach

embankment should be at a slope of 2.5H:1V or flatter (refer to Table 4). Based on the recommended slope inclinations, the length of the new bridge would be about 180 m to 220 m, with abutments located about 80 to 100 m away from the top of bank of the river, depending on the slope used.

However, if the existing caissons are considered to be intact and to be able to contribute to enhance slope stability, a new, about 45 m long, single-span bridge may be constructed to replace the existing one while maintaining the existing features (i.e., road profile grades, caissons and river bank slopes). The new piles should however be installed behind and close to the existing caissons for the existing abutments, i.e., locating the new piles between the existing abutments and the river, in order to use the existing abutments with caissons to prevent lateral loading onto the new piles due to potential slope movement.

All new bank slopes under the new bridge and new bridge approach embankments, if required, should be constructed by compacted engineered fill or equivalent.

6.1 SUMMARY OF THE PREVIOUS REPORT

The following paragraphs provide some excerpts from the previous geotechnical report (dated 04 April 1963) at the existing bridge site obtained from the MTO GEOCRE library (W.J. 62-F-116 & W.J. 62-F-117, District #20, GEOCRE No. 52F-3 and 52F-3-1):

The Wabigoon River flows from Wabigoon Lake to English Lake and is approximately 150 feet wide at the bridge site. The site is underlain by Precambrian rock of the Keewatin period and is covered with Pleistocene and recent deposits consisting of clays, silts and sands.

The Wabigoon bridge on Highway 605 was constructed in 1940 by the Department of Highways for the local municipality. Signs of failure of the bridge were observed at the end of the first winter after completion of the structure. The investigation was carried out to determine the cause of the movement of the bridges at the site. Two sampled boreholes and two dynamic cone penetration tests were carried out at the site during the field work.

The approach fills varied in thickness from 6.5 feet on south bank to 10 on north bank. The fill material was underlain by a layer of clayey silt with organic matter, followed by a stratum of varved clay which extended to the bedrock.

The fill material consisted of silty clay with dispersed sand and gravel. Standard Penetration Test (SPT) values ('N' values) ranged from 8 to 12.

The underlying clayey silt stratum with organic material was about 10 feet thick, with SPT values varying from 2 to 14. One in-situ vane shear strength test carried out in this layer yielded a shear strength of 600 P.S.F. (28.7 kPa).

The stratum of varved clay, found below the clayey silt, extended to the bedrock. The varved clay consisted of clayey silt varves. The in-situ vane test carried out in this layer varied from 400 P.S.F. (19.2 kPa) to 1400 P.S.F. (67 kPa). The shear strength varied between 200 P.S.F. (9.5 kPa) to 630 P.S.F. (30.2 kPa), when measured by the unconfined compression test in the laboratory.

The bedrock, encountered in the boreholes consisted of granite pegmatite and mica gneiss.

Artesian water was observed in one of the boreholes (BH 3), when contact was made with the bedrock. A maximum head of 2.25 feet above ground level was recorded. Other water level readings are shown in the borehole logs of the report (refer to Appendix D).

6.2 PILED FOUNDATION

Deep foundations (low-displacement, driven, steel H-piles or open-ended steel pipe piles), should be used to support the new bridge structures. Based on the information available, the piles should be driven into the bedrock. **A heavy section such as HP 310 x 110 equipped with rock point as per Ministry of Transportation of Ontario's Standard requirements or Titus's rock injector "H" bearing pile point should be used. The piles should be driven sufficiently into the bedrock, preferably at least 1 m into the bedrock. However, due to the high RQD of the bedrock, driving the piles into the bedrock in order to secure the pile tips may not be achievable. In this case, driven steel pipe piles (open-ended), e.g., 324 mm diameter, 11 mm thick, steel pipe piles with rock anchor sockets or equivalent may be required. The rock anchor sockets may be installed after driving the steel pipe piles at the pile locations where sufficient pile embedment into the bedrock can not be achieved. A special provisional item in the contract may be provided if rock sockets are required.**

6.2.1 Resistance to Axial Loads

For piled foundations, HP 310 x 110 steel piles or 324 mm diameter by 11 mm thick steel pipe piles may be considered with the following axial geotechnical forces:

Factored axial resistance at Ultimate Limit States	= 1,200 kN
Axial reaction at Serviceability Limit States	= 900 kN

Based on the results of the boreholes drilled in the vicinity of the existing bridge abutments, the following Table 1 summarizes the approximate pile tip elevations that should be used for design purposes.

TABLE 1 - APPROXIMATE PILE TIP LEVELS

Support Location	Reference Borehole	Approximate Pile Tip Level (m)		Founding Stratum
		Elevation*	Depth Below Existing Grade	
		(m)	(m)	
North Abutment	BH 1 (1962)	75.8 ^(a)	24.8	Bedrock
North Abutment	BH 2 (1962)	74.1 ^(a)	26.5	Probable Bedrock
North Approach	BH 3-2008			Bedrock was not reached.
North Abutment	BH 4-2008	74.4	26.2	Bedrock
South Approach	BH 1-2008			Bedrock was not reached.
South Abutment	BH 2-2008	67.9	33.2	Bedrock
South Abutment	BH 3 (1962)	68.4 ^(b)	32.7	Probable Bedrock
South Abutment	BH 4 (1962)	69.0 ^(b)	32.1	Bedrock

* Based on the Bench Mark (BM) provided by MTO.

^(a) Assuming same elevations as BH 3-2008.

^(b) Assuming same elevations as BH 2-2008.

It should be noted that no boreholes have been drilled in the Wabigoon River. The pile tip elevations within Wabigoon River could be significantly different from those shown in Table 1.

The elevations for boreholes drilled in 2008 shown in Table 1 were surveyed by AMEC with reference to the bench mark (BM) near the bridge site identified in the MTO drawing.

The elevations for the previous boreholes drilled in 1962 (included in the MTO report dated 04 April 1963) were calculated by assuming the top of the boreholes to be the same elevations as the new boreholes (as noted in Table 1), since the elevations provided in the 1962 borehole logs do not match with the BM.

The piles should be driven with a suitably heavy hammer capable of delivering a rated capacity of at least 50 kJ per blow. The energy should, however, be restricted to not more than 60 kJ per blow in order to prevent damage to the piles.

If necessary, the pile capacity may be verified by conducting field tests, i.e., Pile Driving Analyzer with CAPWAP and/or static pile load testing in accordance with ASTM procedures.

Provided that the piles are designed and installed as recommended in this report, the Serviceability Limit State value provided should correspond to not more than 15 mm of

settlement of the pile group. If necessary, the pile group settlement should be calculated using the design pile group configuration.

Oversize materials (e.g., greater than 75 mm nominal diameter) should not be used in any new fill soils through which piles would be driven.

During the pile driving process, piles which have already been driven should be monitored to determine if they are heaving due to the effects of driving adjacent piles. If this phenomenon occurs, the heaved piles should be re-driven. It is recommended that not less than 10 % of the piles (at least two piles) in each abutment be re-tapped no sooner than 24 hours after initial installation, as a precaution against relaxation. If relaxation occurs, all piles in that abutment should be re-tapped.

6.2.2 Resistance to Lateral Loads

Laterally-applied loads on piles can be resisted, from the geotechnical consideration, by the driven piles through passive earth pressures developed in the soils in which the piles are embedded. The pile tip elevations recommended in Table 1 indicate that the piles will be approximately 24 to 33 m in length, depending on the design of the bridge abutment and the actual pile penetration into the bedrock. The recommended horizontal resistances for driven piles at this site are as follows:

Factored Horizontal Resistance at Ultimate Limit States = 100 kN

Horizontal Reaction at Serviceability Limit States = 40 kN for HP 310 x 110
(for 7 mm horizontal movement – fixed head condition)

Horizontal Reaction at Serviceability Limit States = 40 kN for 324 mm pipe pile
(for 5 mm horizontal movement – fixed head condition)

In accordance with the Ministry of Transportation of Ontario's requirements (Ministry of Transportation of Ontario Structural Office Standard), piles for integral abutments require a 3.0 m long flexible zone. The typical Ministry of Transportation of Ontario's standards for the flexible zone consist of a space in between the driven pile and a concentric corrugated steel pipe. The corrugated steel pipe surrounding the H-pile has a typical diameter of 600 mm. After the pile is driven, the space between the H-pile and the corrugated steel pipe is to be filled with uniform sand.

For lateral soil-pile interaction analysis, the horizontal subgrade reaction to the pile may be calculated from the following expression:

In cohesionless soils:

$$k_s = n_h \times z / d$$

In cohesive soils:

$$k_s = 67 \times c_u / d$$

where k_s = coefficient of horizontal subgrade reaction,
 n_h = coefficient related to soil density,
 c_u = undrained shear strength of soil,
 d = pile width,
 z = depth.

Based on the results of the boreholes drilled in the vicinity of the new bridge abutments, Table 2 summarizes the estimated values of internal friction angle, bulk unit weight and coefficient related to soil density.

If conventional abutments on pile groups are to be built instead of integral abutments, the unbalanced horizontal forces may be partially resisted by battered piles.

TABLE 2
SOIL PARAMETERS RECOMMENDED FOR DESIGN

REFERENCE BOREHOLE	APPLICABLE DEPTH FROM EXISTING GROUND SURFACE (m)	APPROX. ELEVATION (m)	SIMPLIFIED SOIL TYPE	ESTIMATED BULK UNIT WEIGHT (kN/m ³)	ESTIMATED ANGLE OF INTERNAL FRICTION ϕ (DEGREES)	ESTIMATED UNDRAINED SHEAR STENGTH (kPa)	ESTIMATED n_h VALUE (MN/m ³)
South Approach							
BH 1-2008	0 to 1.5	101.4 to 99.9	Sand fill	18	30	-	5
	1.5 to 2.9	99.9 to 98.5	Stiff silty Clay/Clayey Silt fill	18	-	25	-
	2.9 to 9.6	98.5 to 91.8	Very soft silty Clay / silt and clay / clay	18	-	15	-
South Abutment							
BH 2-2008	0 to 2.0	101.1 to 99.1	Sand fill	18	30	-	5
	2.0 to 2.9	99.1 to 98.2	Stiff silty Clay/Clayey Silt fill	18	-	25	-
	2.9 to 16.1	98.2 to 85.0	Very soft silty Clay / silt and clay / clay	18	-	15	-
	16.1 to 32.2	85.0 to 68.9		18	-	35	-
North Approach							
BH 3-2008	0 to 1.5	100.6 to 99.0	Sand fill	18	30	-	5
	1.5 to 2.9	99.0 to 97.7	Stiff silty Clay/Clayey Silt fill	18	-	25	-
	2.9 to 9.6	97.7 to 91.0	Very soft silty Clay / silt and clay / clay	18	-	15	-

Table 2 (contd....)

REFERENCE BOREHOLE	APPLICABLE DEPTH FROM EXISTING GROUND SURFACE (m)	APPROX. ELEVATION (m)	SIMPLIFIED SOIL TYPE	ESTIMATED BULK UNIT WEIGHT (kN/m ³)	ESTIMATED ANGLE OF INTERNAL FRICTION ϕ (DEGREES)	ESTIMATED UNDRAINED SHEAR STRENGTH (kPa)	ESTIMATED n_h VALUE (MN/m ³)
North Abutment							
BH 4-2008	0 to 1.5	100.6 to 99.1	Sand fill	18	30	-	5
	1.5 to 2.3	99.1 to 98.3	Stiff silty Clay/Clayey Silt fill	18	-	25	-
	2.3 to 6.6	98.3 to 94.0	Firm to stiff clayey silt / silt	18	-	20	-
	6.6 to 15.6	94.0 to 85.0	Very soft silty Clay / silt and clay / clay	18	-	15	-
	15.6 to 24.4	85.0 to 76.2	Very soft silty Clay / silt and clay / clay	18	-	35	-

6.3 RETAINING / WING WALLS

To support the approach embankment, stable slopes and/or retaining wall structures can be used. Several soil retaining wall structures typically used are integral abutment with wing walls and conventional abutments with retaining wall structure (e.g., reinforced-concrete retaining wall, gabion retaining wall, retained soil system (RSS), etc.). The retaining wall structures are required to support the approach embankment typically in two directions – one where the retaining wall structure is underneath the bridge deck and the other where the retaining wall structure length is parallel to the road alignment. Where space is available, slopes would be preferred to retaining wall structures due generally to the lower construction costs of slopes.

In the case of the Wabigoon River Bridge Replacement where space is available for constructing slopes, it is preferable to construct a slope along the river bank underneath the new bridge deck to support the approach embankment. Based on slope stability analyses (Section 6.4), for a new bridge that does not include the contribution of the existing caissons to improve slope stability, the new bank slope should be constructed at an average slope of 15H:1V or flatter, which would increase the length of the new bridge significantly to about 180 m to 200 m, depending on the height of the embankment. Alternatively, for a new bridge that considers the contribution of the existing caissons to improve slope stability, the existing bank slope underneath the existing bridge and the existing caissons should be maintained without increasing the existing road grade, slope height and slope inclination.

As for the support of the approach embankment in the direction parallel to the road alignment, wing walls supported by the integral abutment or retaining wall structures may be used. The reinforced-concrete wing walls to be extended from the abutment would support the approach embankment immediately adjacent to the bridge deck, together with embankment slopes.

The wing walls or retaining structures should be designed using the following soil parameters and relevant considerations.

Backfill materials behind abutments and wing (retaining) walls should consist of non-frost susceptible, free-draining granular materials in accordance with the Ministry of Transportation of Ontario's standards.

Free-draining backfill materials (i.e., Granular 'A' or Granular 'B') and the provision of drain pipes and weep holes, etc., should be provided to prevent hydrostatic pressure build-up. Computation of earth pressures should be in accordance with Section 6.9.2 – Lateral Pressures of Canadian Highway Bridge Design Code (CAN/CSA-S6-00). For design purposes, the following parameters (unfactored) should be used.

Compacted Granular 'A'

- Unit weight = 22 kN/m^3
- Coefficient of Lateral Earth Pressures:
 - $K_a = 0.27$ (active condition)
 - $K_o = 0.43$ (at-rest condition)
 - $K_p = 1.20$ (passive condition)

Compacted Granular 'B'

- Unit weight = 21 kN/m^3
- Coefficient of Lateral Earth Pressures:
 - $K_a = 0.31$ (active condition)
 - $K_o = 0.47$ (at-rest condition)
 - $K_p = 1.10$ (passive condition)

It should be noted that the above design parameters assume relatively-level ground surface and backfill behind the retaining structure. The coefficient of Lateral Earth Pressure for the passive condition has been reduced in order to limit the wall movement that is necessary to mobilize the passive resistance.

The earth pressure coefficient to be used in the design will depend on whether the retaining structure is restrained or movements are allowed such that the active and passive states of earth pressure can develop. If the abutment is restrained and does not allow lateral yielding, the at-rest pressures should be used for design. Similarly, the effects of compaction should also be taken into account in the selection of the appropriate earth pressure coefficients.

Vibratory equipment for use behind abutments and retaining walls should be restricted in size as per current Ministry of Transportation of Ontario's practice.

6.4 SLOPE STABILITY ANALYSES OF APPROACH EMBANKMENT AND BANK SLOPE

Plans for the new replacement bridge were not available at the time of this report preparation. Preliminary slope stability analyses were carried out for the most critical river bank slopes and approach embankments, based on the drawings provided by MTO for the existing bridge site conditions. The results of the slope stability analyses, along with slope configurations and soil

parameters, are presented in Appendix E. The cross sections for slope stability analyses are shown in Figure No. E1 of Appendix E.

6.4.1 River Bank Slope

a. Without Existing Bridge Caissons

The stability of the new river bank slope (located underneath the new bridge deck) was analysed by considering Cross Section 1-1 in Figure No. E1. The objective of the slope stability analysis carried out was to determine the inclination of the stable river bank slope without the effects of the existing bridge caissons. This is based on the assumption that the as-built construction of the existing bridge caissons are not known and they are old structures (over 65 years old). As such, their contribution to enhance slope stability of the river bank slope may not be relied upon.

The topographical profile of the river and the water level in the river shown in MTO's drawings (Appendix C) were used in the analysis. The soil parameters used in the slope stability analysis (as shown in the figures in Appendix E) were selected from the borehole information. The slope stability analyses were carried out by the computer program, SLOPE/W, using the simplified Bishop method. A 12 kPa traffic load was considered in the analyses.

The results of the slope stability analyses (Section 1-1) are summarized in Table 3. Only deep-seated critical slip surfaces are considered.

Table 3 – Results of Slope Stability Analyses for River Bank Slope

Figure No.	Slope Inclination	Final Road Surface Grade	Minimum Calculated Factor of Safety	Approximate Horizontal Distance of Critical Slip Surface from River Bank to Top of Bank*, m
River Bank Slope Without Existing Bridge Caissons – Deep-Seated Critical Slip Surface				
E2.1a	15H:1V	Similar to the existing road grade	1.4	70
E2.1b	20H:1V	Similar to the existing road grade	1.7	90
E2.2a	15H:1V	1 m raise above the existing road grade	1.4	80
E2.2b	20H:1V	1 m raise above the existing road grade	1.8	110
River Bank Slope With Existing Bridge Caissons – Deep-Seated Critical Slip Surface				
E3.1	About 1.7H:1V	Similar to the existing road grade	> 1.4	70

* "River bank" is defined as the point where the water level in the river meets the river bank.

"Top of Bank" is defined as the point where the river bank above the river water level meets the final road surface.

The slope stability analyses did not consider Bjerrum's correction factor to the field vane shear strength which for the clayey soils at the site should be about 0.9. As such, a minimum factor of safety against slope failure should be 1.4, provided that a minor slope movement is acceptable. Higher minimum factor of safety will reduce the potential slope movement.

The calculated factors of safety should be higher when considering the three-dimensional effect. For the slope under the bridge, the effect of new piles and abutment would have additional stabilising effect.

Based on the results of slope stability analyses shown in Table 3 and assuming no slope-stabilising effects from the existing bridge caissons, the new bridge abutments should be located at least 100 m beyond the "river bank". This would lead to a new bridge with 180 m to 220 m in length, if the existing road grade is maintained or raised by about 1 m. If the new bridge abutment is located within the critical slip surface, i.e., less than 100 m from the "river bank", it should be designed against possible movement of the river bank slope.

The settlement of the new river bank, with a 1 m increase in height above the existing road surface grade, should be between 200 mm and 300 mm in 5 years to 30 years.

If the new river bank slopes are significantly different in dimensions from those shown in Figures E2.1 (a and b) and E2.2 (a and b) in Appendix E, the slope stability and settlement of the new river banks should be analysed during the detail design.

b. With Existing Bridge Caissons

If the existing bridge caissons are considered to improve the slope stability of the new bridge river bank and the existing road grades are not substantially raised, the calculated factor of safety against slope instability is 0.9 without the existing caissons and higher than 1.4 with the existing caissons as shown in Figure E3.1 and Table 3. This is based on the following assumptions:

- The existing caissons are still structurally intact such that they will not deteriorate during the service life of the new bridge.
- The existing caissons extend to the bedrock or sufficiently deep below the critical slip surface as shown in Figure E3.1.
- The existing caisson dimensions below the existing ground surface are approximately 1.0 m by 1.5 m in plan area and built solidly with reinforced-concrete or equivalent.
- At each new bridge abutment, there are two existing bridge caissons that will enhance the river bank slope stability, i.e., one at the existing bridge abutment and the other at the existing river pier located close to the existing bridge abutment.

If the assumptions mentioned above are acceptable or have been verified, the new replacement bridge can be built approximately with the same dimensions and river bank profiles as the existing bridge, i.e., about 45 m in length supported by new piled foundations while maintaining the existing river bank slope profiles. In order to prevent/minimize the new piles from potential slope movement, the piles should be

located between the existing abutment caissons and the existing river piers, and close to the existing abutment caissons. The new river bank may have to be flatter than the existing one, or stabilized, in order to improve the river bank stability according to the locations of the new piles.

6.4.2 Approach Embankment

The slope stability of the approach embankment was evaluated using the same methodology as described in Section 6.4.1. Section 2-2 in Figure E1 was used in the slope stability analyses. The results of the slope stability analyses are summarized in Table 4.

Table 4 – Results of Slope Stability Analysis for Approach Embankment

Figure No.	Slope Inclination	Final Road Surface Grade	Minimum Calculated Factor of Safety	Mode of Failure
E4.1	2.75H:1V	Similar to the existing road grade. The embankment height is about 1.5 m.	2.5	Through embankment
E4.2	2.5H:1V	1 m rise in grade above the existing road grade. The embankment height is about 2.5 m.	1.8	Deep-seated critical slip surface

From the results shown in Table 4, the approach embankment should be constructed at 2.5H:1V or flatter. The height of the approach embankment should be limited to be less than 2.5 m high, otherwise slope instability and/or high embankment movements may occur. The settlement of the approach embankment, with a 1 m increase in height above the existing road surface grade, could be between 200 mm and 300 mm in 5 years to 30 years.

6.5 CONSTRUCTION OF RIVER BANK/APPROACH EMBANKMENT

The river bank slope below the existing bridge may have to be reconstructed for the new bridge, unless the new bridge abutments have been designed for the potential slope movement.

The existing bridge approach embankments would likely be excavated, in whole or in part, during the excavation for the installation of piles and the construction of the new bridge abutments. New approach embankments should be constructed with compacted engineered fill with 2.5H:1V (or flatter) side slopes.

For widening the existing embankments, the new fill should be constructed with benching according to OPSD – 208.010 (Benching of Earth Slopes). The additional slope to the existing slope (if not removed and reconstructed) should be properly benched into the existing slope and the new fill should be properly compacted. Settlement of the new extended slope could be significant due to the presence of soft subgrade. The existing approach embankment would form the main part of the road surface. As such, the settlement of the new road surface should be considered in the design. If the existing bridge approach embankment is removed and a new bridge approach embankment is constructed to the same elevation as the existing one without widening, settlement of the new approach embankment should not be high.

If the new bridge embankments are significantly different in dimensions from the existing embankments, the stability and settlement of the new embankments should be analysed during the detail design.

All organic matters and other unsuitable soils should be removed, as per Ministry of Transportation of Ontario's current practice, with an envelope given by a gradient not steeper than 1H:1V away from the toe of the embankment. For above water construction, the exposed subgrade should be inspected, approved and properly compacted from the surface, to a minimum of 95 % Standard Proctor Maximum Dry Density (SPMDD), under the supervision of qualified geotechnical personnel. Any soft spots identified during stripping and/or recompacting should be sub-excavated and replaced with compacted engineered fill. Care should be exercised to minimize disturbance to the subgrade during preparation and the construction of embankment. For under water construction, all organic matters and /or soft / loose soils should be removed and backfilled with rock / granular soils until the backfill level is above the water level. Otherwise, dewatering is required for placing an engineered fill.

For an engineered fill, the fill soils used for construction of the conventional earth fill embankment, or for the purposes of backfilling, should consist of approved, clean earth fill (e.g. Select Subgrade Materials – Ontario Provincial Standards Specifications Number: 1010). The fill may be imported for this purpose or the excavated soils may be reused provided that they do not contain organic matters and can be compacted to the specifications. However, the existing very soft clayey soils may not be suitable for reuse unless their high water contents have been reduced. The fill soils should be placed in accordance with Ontario Provincial Standards Specifications Number: 501. Each lift should not exceed 300 mm before compaction and each lift should be uniformly compacted to at least 95 % of the Standard Proctor Maximum Dry Density (SPMDD) of the materials. The degree of compaction within the top 0.6 m of the fill (i.e., the subgrade immediately beneath the granular sub-base) should be increased to 98 % SPMDD. The selection, placement and compaction of the fill should be carried out under a geotechnical control program.

For the fill embankment at this site, using properly compacted and acceptable inorganic fill soils, the side slopes should not be steeper than 2.5H:1V for earth fill embankment. Proper erosion control measures should be implemented both during construction and on a permanent basis. This can be achieved by immediate seeding or sodding (Ontario Provincial Standards Specification Number: 572) or equivalent.

6.6 CONSTRUCTION STAGING AND DETOUR

To construct the new bridge, the existing bridge structure and part of the existing bridge approach embankment will have to be demolished. Due to the narrow width of the existing bridge, a detour may be constructed through the river, during the construction of the new bridge. This will however necessitate temporary crossing of the river and working in-stream, which may need to be avoided due to the stringent requirements for environmental protection.

As an alternative, the road alignment at the bridge site may be shifted slightly to the east or west of the existing bridge and the new bridge be constructed, while the old bridge is used for

maintaining traffic flow. The existing bridge may be demolished after completion of the new bridge construction.

If a detour through the river or re-alignment of the road is undesirable, part of the existing bridge structure will have to be demolished while keeping the remaining part of the bridge open for traffic. This will reduce the traffic lane to be a single lane and all necessary traffic protection will have to be provided. A roadway protection system (i.e., temporary shoring system) will be required, if necessary, along the bridge approach alignment and adjacent to the existing bridge to support the partially-excavated existing bridge approach embankment.

Based on the boreholes drilled at the existing bridge approach embankments (BH 2-2008 and BH 4-2008), temporary cantilever/braced sheet piles or soldier piles with wood lagging may be used to support the partially-removed approach embankment. The temporary shoring system should be designed using the following soil parameters and surcharge:

Soil bulk unit weight	=	18 kN/m ³
Coefficient of lateral soil earth pressure	=	0.4
Traffic load as surcharge	=	12 kPa

Groundwater levels shown in the Record of Boreholes (Appendix A) should be considered in the design of the temporary shoring system.

6.7 GENERAL CONSTRUCTION COMMENTS

All excavations should be carried out in accordance with the latest edition of the Ontario Occupational Health and Safety Act and its regulations (i.e. Occupational Health and Safety Act O.Reg. 213/91).

The boreholes show that the excavation for the structure foundations (including abutment and wing walls) should encounter mainly the fill soils (sand/silty clay/ clayey silt) and possibly some native clayey silt/silty clay/silt/clay. These soils should be classified as Type 3 and according to the Occupational Health and Safety Act, temporary open cut slopes should not be steeper than 1H:1V.

Groundwater was not encountered in the shallow boreholes BH 1-2008 and BH 3-2008 (up to 9.6 m below surface). The previous report prepared for MTO (1963) indicated groundwater at about 3.0 m to 3.3 m below ground surface. Therefore, some groundwater seepage, if any, should be expected. Other water sources could be river water (if excavated close to or in the river), perched groundwater and/or surface runoff. Dewatering in the excavation within the existing road surface could be carried out by gravity drainage and/or pumping from a properly filtered sump. The base of the excavation should be graded towards a sump pump in order to drain any surface water inflow into the excavation and from severe weather events in order to avoid excessive softening of the base.

No major excavation difficulties are foreseen but allowance should be made for boulders and cobbles which could be present in fill soils and native soil deposits.

The demolition of the existing bridge abutment should be limited such that the remaining existing bridge abutment and its existing caisson foundation would not interfere with the performance of the new bridge and would enhance the slope stability of the river bank slope of the bridge.

Allowance should be made to place an approximately 150 mm thick layer of lean concrete on the subgrade surface, i.e. the excavation base for the new bridge abutment, within four hours of preparation and acceptance of the bearing soil. It should be pointed out that if the foundation soil is disturbed, excessive settlements could occur after structural loads are applied.

Compaction of the granular backfill soils against the new bridge abutment should be carried out with small compacting equipment in order to prevent potential damage to the new bridge abutment. Both sides of the bridge abutment should be backfilled to approximately the same level at the same time in order to minimize the lateral load due to compaction.

6.8 CONSTRUCTION INSPECTION

It is recommended that a quality control programme of inspection and testing be carried out during the construction phase of the project to confirm that the conditions encountered are consistent with design assumptions; and to confirm that the various project specifications and material requirements and handling are followed.

7.0 LIMITED ENVIRONMENTAL INVESTIGATION

In accordance with the Terms of Reference, soil samples obtained during the geotechnical field drilling program were field screened for evidence of environmental impact.

The field screening activities included visually inspecting the soil samples for evidence of environmental impact (i.e., staining, etc.) and measuring the total organic vapours (TOV) in the headspace of samples with a portable hydrocarbon surveyor instrument (Gastechtor 1238ME).

The results of the field screening program did not identify any visual or olfactory evidence of environmental impact in the fill and/or native soil samples recovered from the boreholes. The measured TOV concentrations in all soil samples were non-detect, as shown in the Record of Boreholes in Appendix A. The TOV results are semi-quantitative at best and are generally only used for relative sample comparison purposes when selecting samples for laboratory analysis. Based on the field screening results, chemical analyses of the soil samples were not deemed necessary.

8.0 SOIL CORROSIVITY

One soil sample (BH 2-2008 / SS 5) was analysed by AMEC's chemical laboratory in Mississauga to determine the soil corrosivity potential with respect to concrete and steel. The results are presented in Table 5. The Certificate of Analysis is attached in Appendix B.

Table 5 – Soil Corrosivity Test Results

Soil Sample No.	pH	Resistivity (ohms-cm)	Chloride (µg/g)	Sulphate (µg/g)
BH 2-2008 / SS 5	7.3	1400	324	11

The test results have shown a sulphate content of 11 ppm (µg/g) and according to Table 3 - "Additional Requirements for Concrete Subject to Sulphate Attack", Clause 4.1.1.6, of CSA Standard Specification A23.1-04, any soil which has a sulphate content below 1000 ppm is not considered corrosive with respect to concrete. The soil resistivity measured at BH 2-2008 was "severe", while the pH value was close to neutral. The soil corrosivity on concrete and steel should be studied, if necessary, by a corrosivity expert.

9.0 CLOSURE

The sub-soil information and recommendations contained in this report should be used solely for the purpose of foundation assessment of this site.

AMEC should be retained to review the recommendations provided in this report, once the details of the project are finalized and prior to the final design stage of the project.

The attached Report Limitations is an integral part of this report.

Sincerely,

**AMEC Earth & Environmental,
a Division of AMEC Americas Limited**

Prapote Boonsinsuk, Ph.D., P.Eng.
Project Manager

George Chow, P.Eng.
Designated Principal Contact and
Project Reviewer

AMEC Earth & Environmental, a division of AMEC Americas Limited

REPORT LIMITATIONS

The conclusions and recommendations given in this report are based on information determined at the testhole locations. The information contained herein in no way reflects on the environmental aspects of the project, unless otherwise stated. Subsurface and groundwater conditions between and beyond the testholes may differ from those encountered at the testhole locations, and conditions may become apparent during construction, which could not be detected or anticipated at the time of the site investigation. It is recommended practice that the Geotechnical Engineer be retained during the construction to confirm that the subsurface conditions across the site do not deviate materially from those encountered in the testholes.

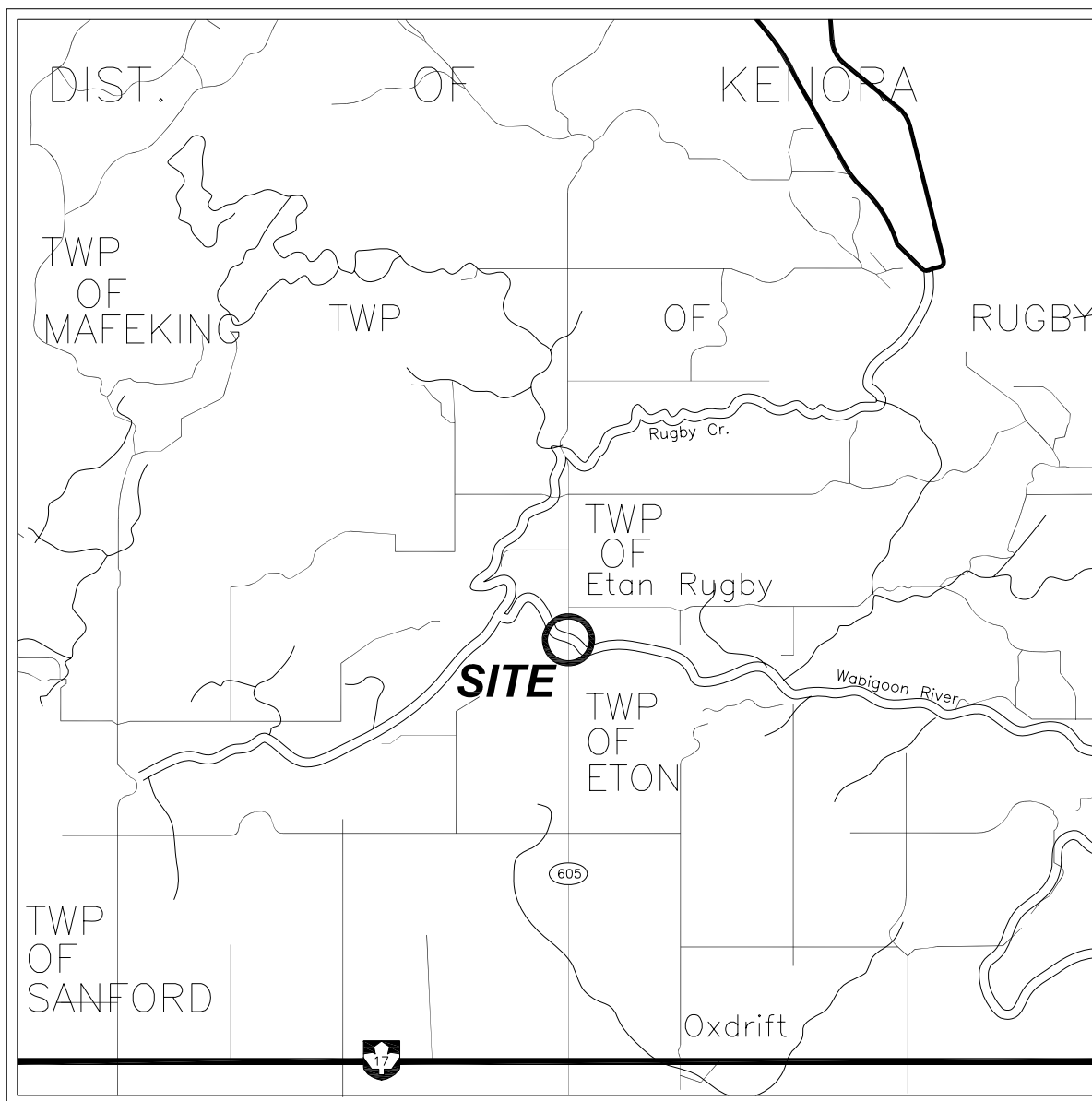
The design recommendations given in this report are applicable only to the project described in the text, and then only if constructed substantially in accordance with the details stated in this report. Since all details of the design may not be known, we recommend that we be retained during the final design stage to verify that the design is consistent with our recommendations, and that assumptions made in our analysis are valid.

The comments made in this report relating to potential construction problems and possible methods of construction are intended only for the guidance of the designer. The number of testholes may not be sufficient to determine all the factors that may affect construction methods and costs. For example, the thickness of surficial topsoil or fill layers may vary markedly and unpredictably. The contractors bidding on this project or undertaking the construction should, therefore, make their own interpretation of the factual information presented and draw their own conclusions as to how the subsurface conditions may affect their work. This work has been undertaken in accordance with normally accepted geotechnical engineering practices. No other warranty is expressed or implied.

The benchmark and elevations mentioned in this report were obtained strictly for use by this office in the geotechnical design of the project. They should not be used by any other party for any other purpose.

Any use which a third party makes of this report, or any reliance on or decisions to be made based on it, are the responsibility of such third parties. AMEC Earth & Environmental accepts no responsibility for damages, if any, suffered by any third party as a result of decisions made or actions based on this report.

DRAWINGS



SCALE

500m 0 1000 2000 3000 4000m

**AMEC Earth & Environmental,
a Division of AMEC Americas Limited**



CLIENT LOGO



CLIENT

**MINISTRY OF
TRANSPORTATION ONTARIO**

TITLE
SITE MAP

DWN BY:
KW

DATUM:
-

DATE: April 2008

PROJECT
**FOUNDATION INVESTIGATION REPORT
WABIGOON RIVER BRIDGE ON HIGHWAY 605 REPLACEMENT
NORTHWESTERN ONTARIO - SITE NUMBER 41S-40**

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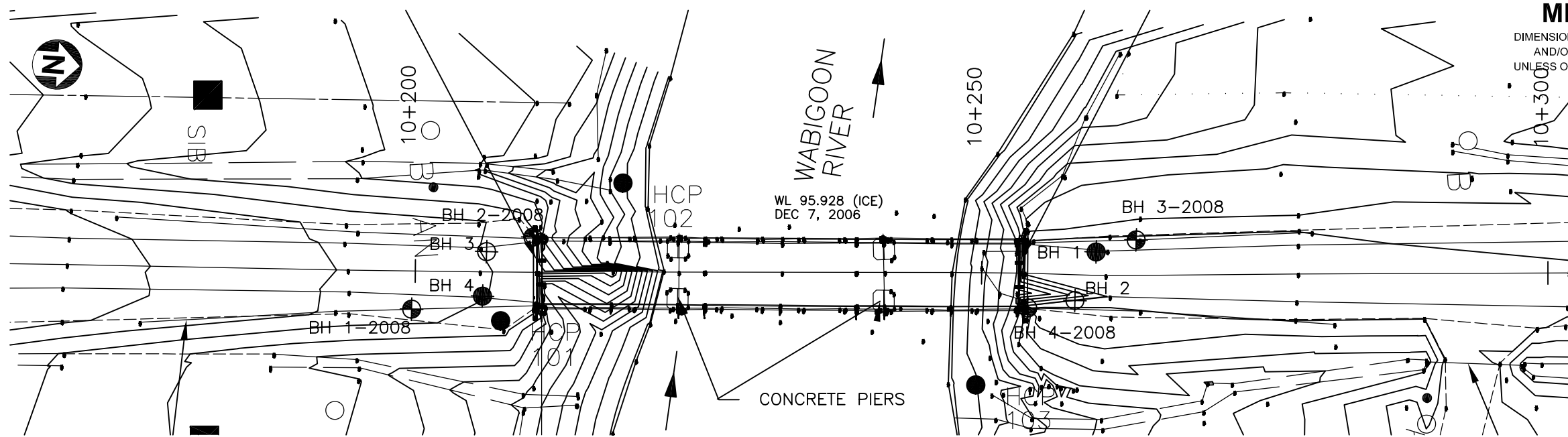
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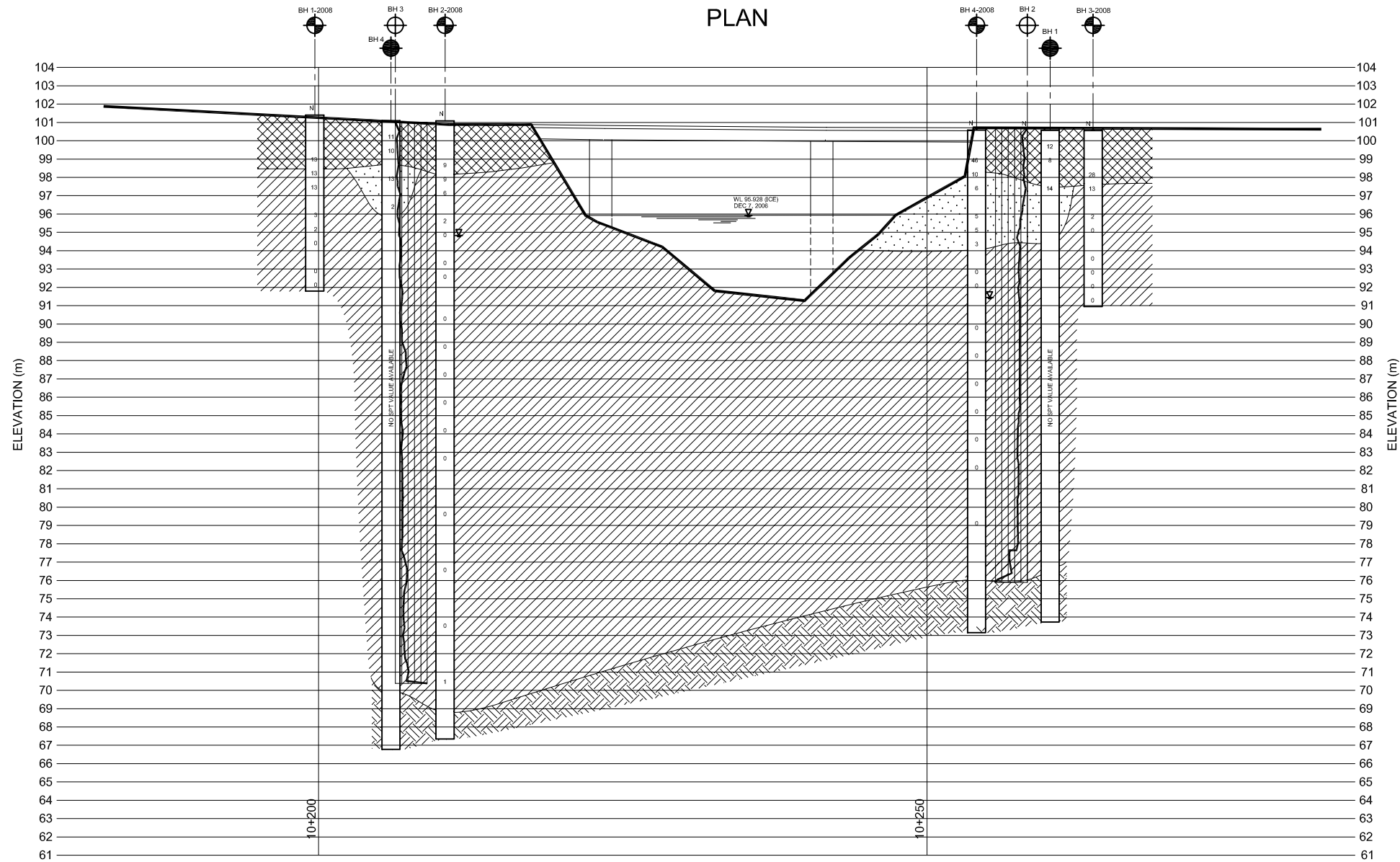
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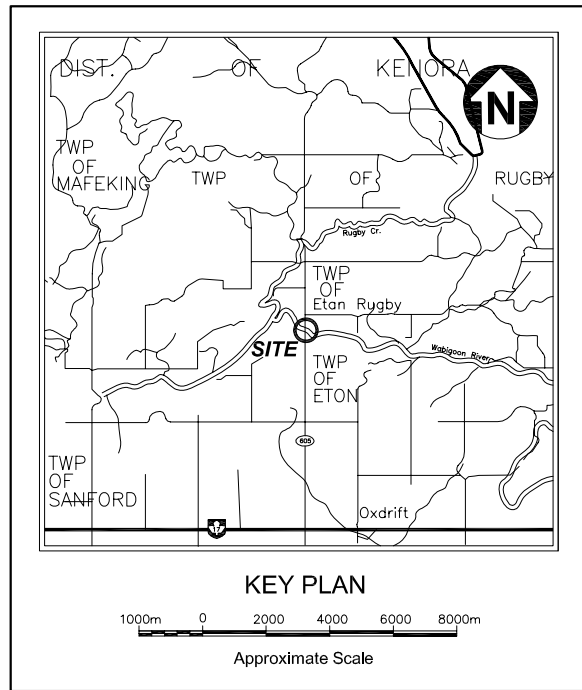


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DETAIL FOUNDATION INVESTIGATION REPORT WABIGOON RIVER BRIDGE ON HIGHWAY 605 REPLACEMENT SITE NUMBER 41S-40	
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PROFILE



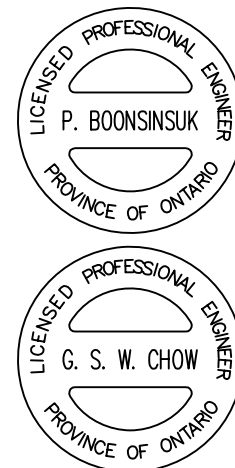
LEGEND

- BOREHOLE LOCATION (2008)
- BOREHOLE LOCATION (1963)
- CONE PENETRATION HOLE (1963)
- GROUND WATER LEVEL AT TIME OF INVESTIGATION

NOTES:
The boundaries between soil strata have been established only at borehole locations. Between boreholes, the boundaries are assumed from geological evidence and may be subject to considerable error.

SOIL STRATIGRAPHY

- Sand / Silty Clay / Clayey Silt FILL
- CLAYEY SILT / SILT
- CLAY / SILTY CLAY / SILT AND CLAY (varved)
- Bedrock



REVISIONS	DESIGN	CHK	CODE	CL	DATE
	PB	PB	CHBDC-06	CL 625-ONT	APR. 2008
	KW	GC	SITE	41S-40	DWG 2

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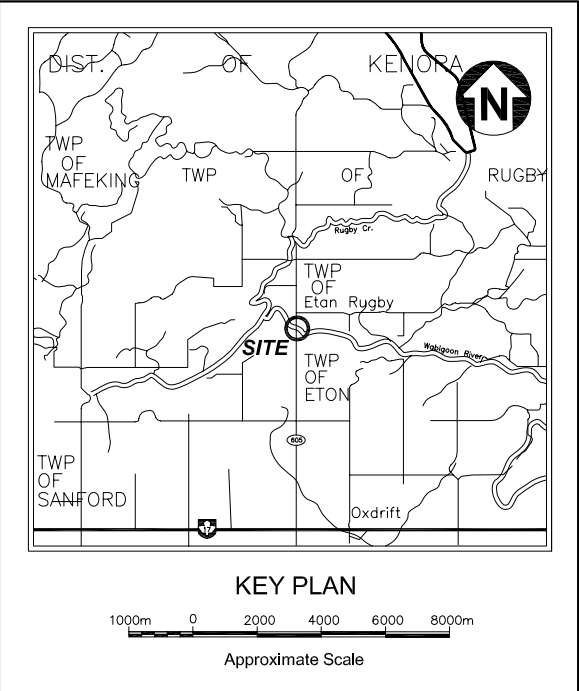
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DETAIL FOUNDATION INVESTIGATION REPORT
WABIGOOON RIVER BRIDGE ON HIGHWAY 605
REPLACEMENT
SITE NUMBER 41S-40

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SHEET
2 of 2



LEGEND

BOREHOLE LOCATION (2008)

BOREHOLE LOCATION (1963)

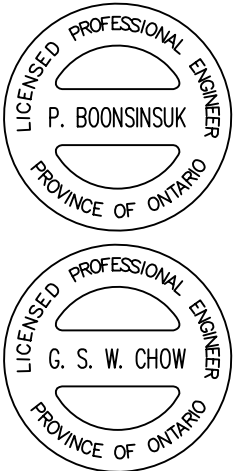
CONE PENETRATION HOLE (1963)

GROUND WATER LEVEL AT TIME OF INVESTIGATION

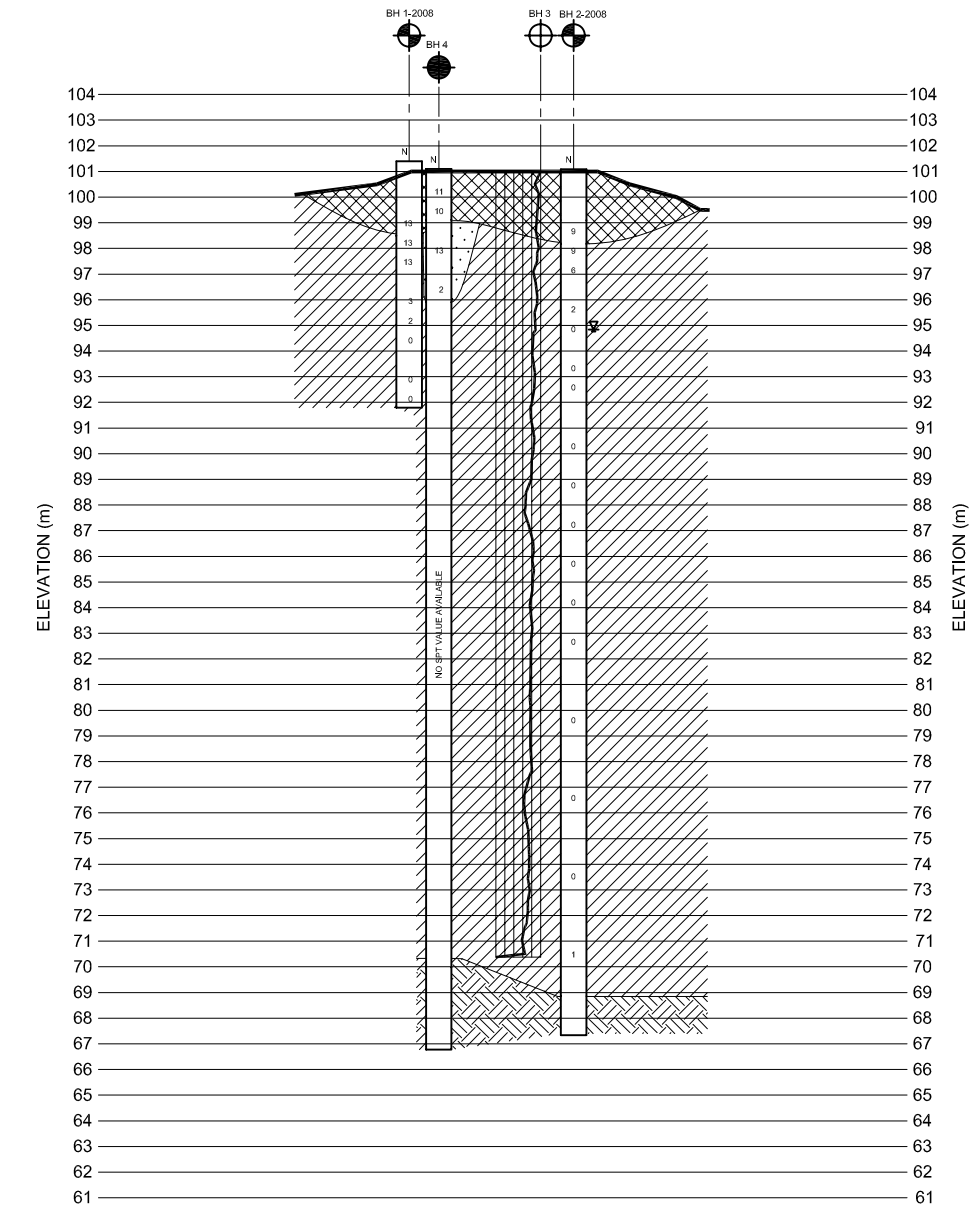
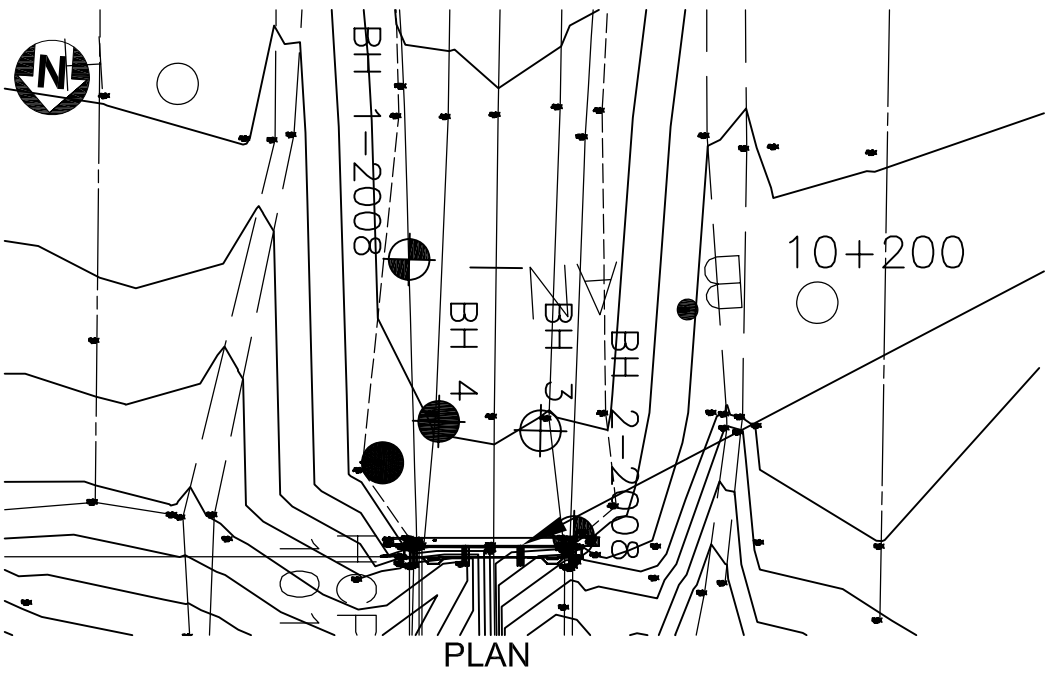
NOTES:
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SOIL STRATIGRAPHY

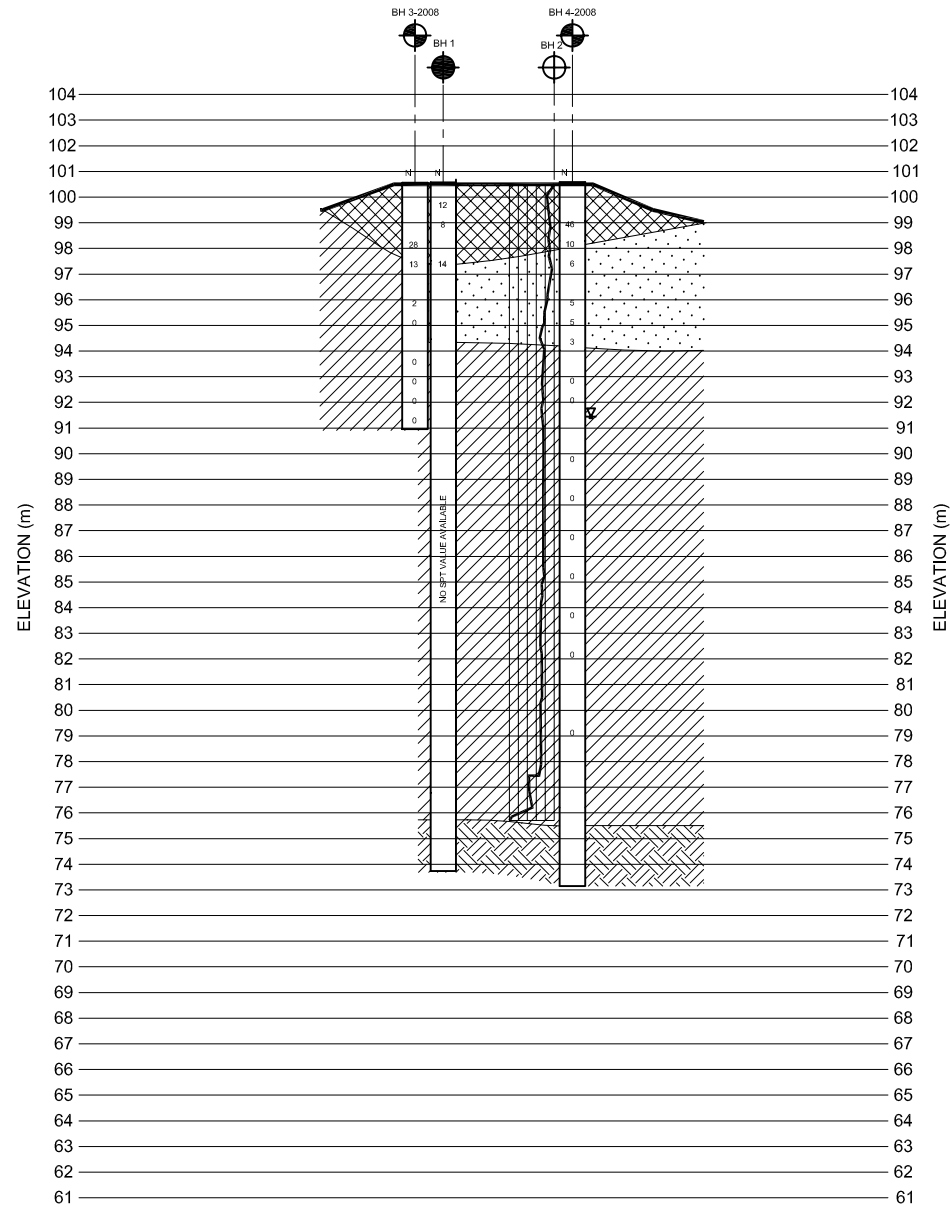
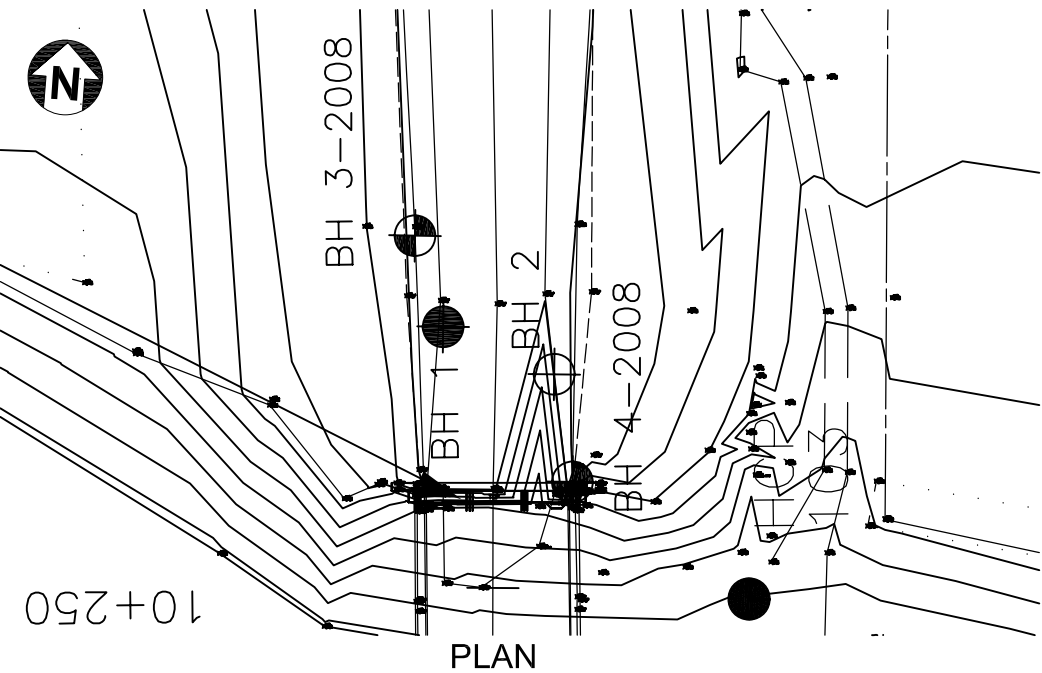
-
- Sand / Silty Clay / Clayey Silt FILL



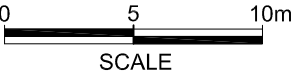
REVISIONS				
DESIGN PB	CHK PB	CODE CHBDC-06	CL 625-ONT	DATE APR. 2008
DRAWN KW	CHK GC	SITE 41S-40		DWG 2 (cont'd)



SOUTH ABUTMENT



NORTH ABUTMENT



P:\GEO\Projects\2007\TZ71046 - Hwy 417, Ottawa Drawings\TZ71046 - BH07-1 - BH07-8.DWG

APPENDIX A
RECORD OF BOREHOLES

EXPLANATION OF BOREHOLE LOG

This form describes some of the information provided on the borehole logs, which is based primarily on examination of the recovered samples, and the results of the field and laboratory tests. Additional description of the soil/rock encountered is given in the accompanying geotechnical report.

GENERAL INFORMATION

Project details, borehole number, location coordinates and type of drilling equipment used are given at the top of the borehole log.

SOIL LITHOLOGY

Elevation and Depth

This column gives the elevation and depth of inferred geologic layers. The elevation is referred to the datum shown in the Description column.

Lithology Plot

This column presents a graphic depiction of the soil and rock stratigraphy encountered within the borehole.

Description

This column gives a description of the soil strata, based on visual and tactile examination of the samples augmented with field and laboratory test results. Each stratum is described according to the *Modified Unified Soil Classification System*.

The compactness condition of cohesionless soils (SPT) and the consistency of cohesive soils (undrained shear strength) are defined as follows (*Ref. Canadian Foundation Engineering Manual*):

Compactness of	
<u>Cohesionless</u>	<u>SPT N-Value*</u>
<u>Soils</u>	
Very loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very Dense	> 50

Consistency of		<u>Undrained Shear Strength</u>
<u>Cohesive Soils</u>	<u>kPa</u>	<u>psf</u>
Very soft	0 to 12	0 to 250
Soft	12 to 25	250 to 500
Firm	25 to 50	500 to 1000
Stiff	50 to 100	1000 to 2000
Very stiff	100 to 200	2000 to 4000
Hard	Over 200	Over 4000

* For penetration of less than 0.3 m, N-values are indicated as the number of blows for the penetration achieved (e.g. 50/25: 50 blows for 25 centimeter penetration).

Soil Sampling

Sample types are abbreviated as follows:

SS	Split Spoon	TW	Thin Wall Open (Pushed)	RC	Rock Core	GS	Grab Sample
AS	Auger Sample	TP	Thin Wall Piston (Pushed)	WS	Washed Sample	AR	Air Return Sample

Additional information provided in this section includes sample numbering, sample recovery and numerical testing results.

Field and Laboratory Testing

Results of field testing (e.g., SPT, pocket penetrometer, and vane testing) and laboratory testing (e.g., natural moisture content, and limits) executed on the recovered samples are plotted in this section.

Instrumentation Installation

Instrumentation installations (monitoring wells, piezometers, inclinometers, etc.) are plotted in this section. Water levels, if measured during fieldwork, are also plotted. These water levels may or may not be representative of the static groundwater level depending on the nature of soil stratum where the piezometer tips are located, the time elapsed from installation to reading and other applicable factors.


Comments

This column is used to describe non-standard situations or notes of interest.

MODIFIED * UNIFIED CLASSIFICATION SYSTEM FOR SOILS						
*The soil of each stratum is described using the Unified Soil Classification System (Technical Memorandum 36-357 prepared by Waterways Experiment Station, Vicksburg, Mississippi, Corps of Engineers, U.S Army. Vol. 1 March 1953,) modified slightly so that an inorganic clay of "medium plasticity" is recognized.						
MAJOR DIVISION			GROUP SYMBOL	TYPICAL DESCRIPTION	LABORATORY CLASSIFICATION CRITERIA	
COARSE GRAINED SOILS (MORE THAN HALF BY WEIGHT LARGER THAN 75µm)	GRAVELS MORE THAN HALF THE COARSE FRACTION LARGER THAN 4.75mm	CLEAN GRAVELS (TRACE OR NO FINES)	GW	WELL GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES	$C_u = \frac{D_{60}}{D_{10}} > 4; C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}} = 1 \text{ to } 3$	
			GP	POORLY GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES	NOT MEETING ABOVE REQUIREMENTS	
		DIRTY GRAVELS (WITH SOME OR MORE FINES)	GM	SILTY GRAVELS, GRAVEL-SAND- SILT MIXTURES	ATTERBERG LIMITS BELOW "A" LINE OR P.I MORE THAN 4	
			GC	CLAYEY GRAVELS, GRAVEL-SAND-CLAY MIXTURES	ATTERBERG LIMITS BELOW "A" LINE OR P.I MORE THAN 7	
	SANDS MORE THAN HALF THE COARSE FRACTION SMALLER THAN 4.75mm	CLEAN SANDS (TRACE OR NO FINES)	SW	WELL GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES	$C_u = \frac{D_{60}}{D_{10}} > 6; C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}} = 1 \text{ to } 3$	
			SP	POORLY GRADED GRAVELS, GRAVEL- SAND MIXTURES, LITTLE OR NO FINES	NOT MEETING ABOVE REQUIREMENTS	
		DIRTY SANDS (WITH SOME OR MORE FINES)	SM	SILTY SANDS, SAND-SILT MIXTURES	ATTERBERG LIMITS BELOW "A" LINE OR P.I MORE THAN 4	
			SC	CLAYEY SANDS, SAND-CLAY MIXTURES	ATTERBERG LIMITS BELOW "A" LINE OR P.I MORE THAN 7	
FINE-GRAINED SOILS (MORE THAN HALF BY WEIGHT SMALLER THAN 75µm)	SILTS BELOW "A" LINE NEGLIGIBLE ORGANIC CONTENT	$W_L < 50\%$	ML	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY SANDS OF SLIGHT PLASTICITY	CLASSIFICATION IS BASED UPON PLASTICITY CHART (SEE BELOW)	
		$W_L < 50\%$	MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS, FINE SANDY OR SILTY SOILS		
	CLAYS ABOVE "A" LINE NEGLIGIBLE ORGANIC CONTENT	$W_L < 30\%$	CL	INORGANIC CLAYS OF LOW PLASTICITY, GRAVELLY, SANDY OR SILTY CLAYS, LEAN CLAYS		
		$30\% < W_L < 50\%$	CI	INORGANIC CLAYS OF MEDIUM PLASTICITY, SILTY CLAYS		
		$W_L < 50\%$	CH	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS		
	ORGANIC SILTS & CLAYS BELOW "A" LINE	$W_L < 50\%$	OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY	WHENEVER THE NATURE OF THE FINES CONTENT HAS NOT BEEN DETERMINED, IT IS DESIGNATED BY THE LETTER "F", E.G SF IS A MIXTURE OF SAND WITH SILT OR CLAY	
		$W_L < 50\%$	OH	ORGANIC CLAYS OF HIGH PLASTICITY		
	HIGH ORGANIC SOILS			Pt	PEAT AND OTHER HIGHLY ORGANIC SOILS	STRONG COLOUR OR ODOUR, AND OFTEN FIBROUS TEXTURE

SOIL COMPONENTS					
FRACTION	U.S STANDARD SIEVE SIZE		DEFINING RANGES OF PERCENTAGE BY WEIGHT OF MINOR COMPONENTS		
GRAVEL	COARSE	PASSING	RETAINED	PERCENT	DESCRIPTOR
		76 mm	19 mm	35-50	AND
	FINE	19 mm	4.75 mm	20-35	Y/EY
SAND	COARSE	4.75 mm	2.00 mm	10-20	SOME
	MEDIUM	2.00 mm	425 µm	1-10	TRACE
	FINE	425 µm	75 µm		
FINES (SILT OR CLAY BASED ON PLASTICITY)		75 µm			
OVERSIZED MATERIAL					
ROUNDED OR SUBROUNDED: COBBLES 76 mm TO 200 mm BOULDERS > 200 mm				NOT ROUNDED: ROCK FRAGMENTS > 76 mm ROCKS > 0.76 CUBIC METRE IN VOLUME	

Plasticity Chart for Soil Passing 425 Micron Sieve

AMEC Earth & Environmental 104 Crockford Boulevard Scarborough, ON M1R 3C3 Ph: (416) 751-6565 Fax: (416) 751-7592 www.amec.com		Note 1: Soils are classified and described according to their engineering properties and behaviour. Note 2: The modifying adjectives used to define the actual or estimated percentage range by weight of minor components are consistent with the Canadian Foundation Engineering Manual (4th Edition, Canadian Geotechnical Society, 2006.)
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RECORD OF BOREHOLE No BH 1-2008

1 OF 2

G.W.P. 454-00-00 LOCATION Wabigoon River Bridge ORIGINATED BY PE
 DIST HWY 605 BOREHOLE TYPE Tri-cone with casing COMPILED BY SN
 DATUM Geodetic DATE 12 March 2008 - 12 March 2008 CHECKED BY PB
 PROJECT Wabigoon River Bridge Replacement at HWY 605 JOB NO. TT83000

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	DEPTH m	ELEVATION SCALE # ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _P	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	SOIL VAPOUR READING	REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH (m)	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES				SHEAR STRENGTH kPa							WATER CONTENT (%)	
									○ UNCONFINED ● QUICK TRIAXIAL	+ FIELD VANE × LAB VANE							
101.4								20 40 60 80 100	20 40 60 80 100	10 20 30			PPM	GR SA SI CL			
0.0	grey Sand FILL trace to some silt and gravel moist		1	AS			101						0	Frozen soil			
99.9			2	AS			100						0	Frozen soil			
1.5	brown Silty Clay / Clayey Silt FILL trace to some sand and gravel		3	SS	13		99						0	Frozen soil			
98.5			4	SS	13		98						0				
2.9	brown CLAY / SILTY CLAY / SILT AND CLAY (varved) trace sand stiff to very soft oxidized stains		5	SS	13		97						0				
			6	TW			96						0				
			7	SS	3		95						0				
			8	SS	2		94						0				
			9	SS	0		93						0				
			10	TW									0				
			11	SS	0								0				

Continued Next Page

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

RECORD OF BOREHOLE No BH 1-2008

2 OF 2

G.W.P. 454-00-00 LOCATION Wabigoon River Bridge ORIGINATED BY PE
 DIST HWY 605 BOREHOLE TYPE Tri-cone with casing COMPILED BY SN
 DATUM Geodetic DATE 12 March 2008 - 12 March 2008 CHECKED BY PB
 PROJECT Wabigoon River Bridge Replacement at HWY 605 JOB NO. TT83000

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	DEPTH m	ELEVATION m	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	SOIL VAPOUR READING PPM	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH (m)	DESCRIPTION	STRAT PLOT	NUMBER	TYPE				"N" VALUES	SHEAR STRENGTH kPa								
						○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE					WATER CONTENT (%)						
						20	40	60	80	100							
91.8	grey CLAY / SILTY CLAY / SILT AND CLAY (varved) very soft		12	SS	0											49	
9.6	End of Borehole																
	No groundwater in open borehole on completion																
	Borehole was backfilled with bentonite at the completion of drilling.																

RECORD OF BOREHOLE No BH 2-2008

1 OF 4

G.W.P. 454-00-00 LOCATION Wabigoon River Bridge ORIGINATED BY PE
 DIST HWY 605 BOREHOLE TYPE Tri-cone with casing COMPILED BY SN
 DATUM Geodetic DATE 12 March 2008 - 13 March 2008 CHECKED BY PB
 PROJECT Wabigoon River Bridge Replacement at HWY 605 JOB NO. TT83000

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	DEPTH m	ELEVATION SCALE ELEVATION m	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _P	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	SOIL VAPOUR READING PPM	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV. DEPTH (m)	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES				SHEAR STRENGTH kPa							WATER CONTENT (%)
									○ UNCONFINED ● QUICK TRIAXIAL	+ FIELD VANE × LAB VANE						
101.1									20 40 60 80 100							
0.0	grey Sand FILL trace to some silt, some to with gravel wet		1	AS			101							0	Frozen soil 25 63 (12)	
							1									
							100									
99.1			2	AS			2							0	Frozen soil	
2.0	brown Silty Clay / Clayey Silt FILL trace sand and gravel		3	SS	9		99							0		

Continued Next Page

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

RECORD OF BOREHOLE No BH 2-2008

2 OF 4

G.W.P. 454-00-00 LOCATION Wabigoon River Bridge ORIGINATED BY PE
 DIST HWY 605 BOREHOLE TYPE Tri-cone with casing COMPILED BY SN
 DATUM Geodetic DATE 12 March 2008 - 13 March 2008 CHECKED BY PB
 PROJECT Wabigoon River Bridge Replacement at HWY 605 JOB NO. TT83000

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	DEPTH m	ELEVATION m	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	SOIL VAPOUR READING PPM	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH (m)	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES				SHEAR STRENGTH kPa						
	grey CLAY / SILTY CLAY / SILT AND CLAY (varved) very soft		12	TW			92		3.5						No sample
							91								
			13	SS	0		90						58	0	
							89						68	0	
			14	SS	0		88						58	0	
							87						58	0	
			15	SS	0		86						48	0	
							85								
			16	SS	0		84						58	0	
	grey to reddish brown		17	SS	0								58	0	

Continued Next Page

+ 3, X 3: Numbers refer to Sensitivity O 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No BH 2-2008

3 OF 4

G.W.P. 454-00-00 LOCATION Wabigoon River Bridge ORIGINATED BY PE
 DIST HWY 605 BOREHOLE TYPE Tri-cone with casing COMPILED BY SN
 DATUM Geodetic DATE 12 March 2008 - 13 March 2008 CHECKED BY PB
 PROJECT Wabigoon River Bridge Replacement at HWY 605 JOB NO. TT83000

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	DEPTH m	ELEVATION m	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	SOIL VAPOUR READING PPM	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH (m)	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES				SHEAR STRENGTH kPa									
						○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE					WATER CONTENT (%)							
						20 40 60 80 100 20 40 60 80 100					10	20	30					
	reddish brown CLAY / SILTY CLAY / SILT AND CLAY (varved) very soft		18	SS	0		83											
							82											
							81											
							80											
	grey		19	SS	0		79											
							78											
							77											
			20	SS	0		76											
							75											

Continued Next Page

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

[illegible]

RECORD OF BOREHOLE No BH 3-2008

2 OF 2

G.W.P. 454-00-00 LOCATION Wabigoon River Bridge ORIGINATED BY PE
 DIST HWY 605 BOREHOLE TYPE Tri-cone with casing COMPILED BY SN
 DATUM Geodetic DATE 12 March 2008 - 12 March 2008 CHECKED BY PB
 PROJECT Wabigoon River Bridge Replacement at HWY 605 JOB NO. TT83000

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	DEPTH m	ELEVATION m	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			SOIL VAPOUR READING	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH (m)	DESCRIPTION	STRAT PLOT	NUMBER	TYPE				"N" VALUES	SHEAR STRENGTH kPa					WATER CONTENT (%)			
							20	40	60	80	100	W _p	W	W _L			
							○ UNCONFINED	+	FIELD VANE	×	LAB VANE						
							● QUICK TRIAXIAL										
							20	40	60	80	100	10	20	30	PPM	GR SA SI CL	
91.0	grey CLAY / SILTY CLAY / SILT AND CLAY (varved) very soft		12	SS	0												
9.6	End of Borehole																
	No groundwater in open borehole on completion																
	Borehole was backfilled with bentonite at the completion of drilling.																

RECORD OF BOREHOLE No BH 4-2008

1 OF 4

G.W.P. 454-00-00 LOCATION Wabigoon River Bridge ORIGINATED BY PE
 DIST HWY 605 BOREHOLE TYPE Tri-cone with casing COMPILED BY SN
 DATUM Geodetic DATE 10 March 2008 - 11 March 2008 CHECKED BY PB
 PROJECT Wabigoon River Bridge Replacement at HWY 605 JOB NO. TT83000

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	DEPTH m	ELEVATION SCALE m	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			SOIL VAPOUR READING	REMARKS & GRAIN SIZE DISTRIBUTION (%)																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																															
ELEV. DEPTH (m)	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES				SHEAR STRENGTH kPa					W _P	W	W _L		PPM	GR	SA	SI	CL																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																											
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100.6									20	40	60	80	100																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																				

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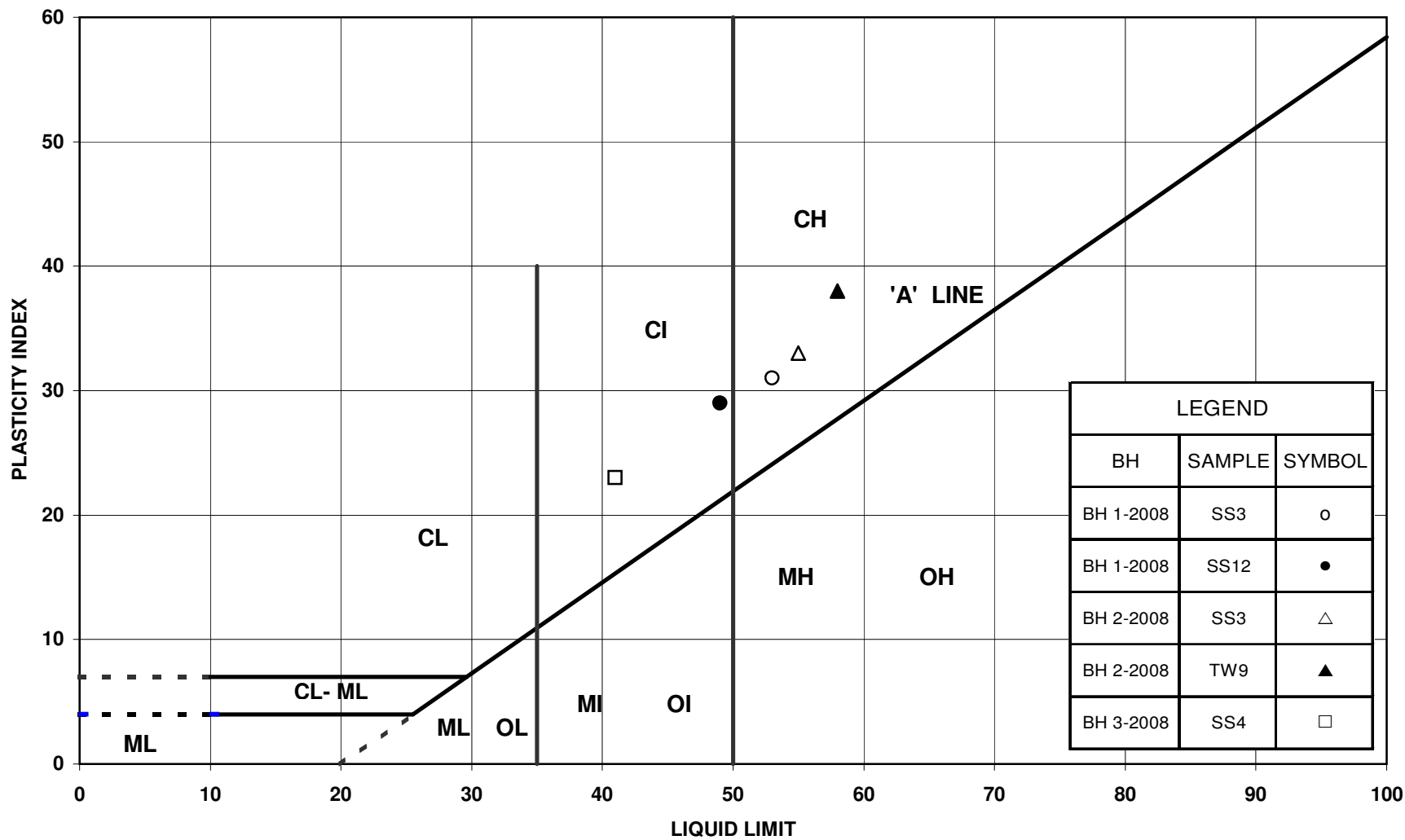
+³, ×³: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

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[illegible]

APPENDIX B

LABORATORY TEST RESULTS



Ontario

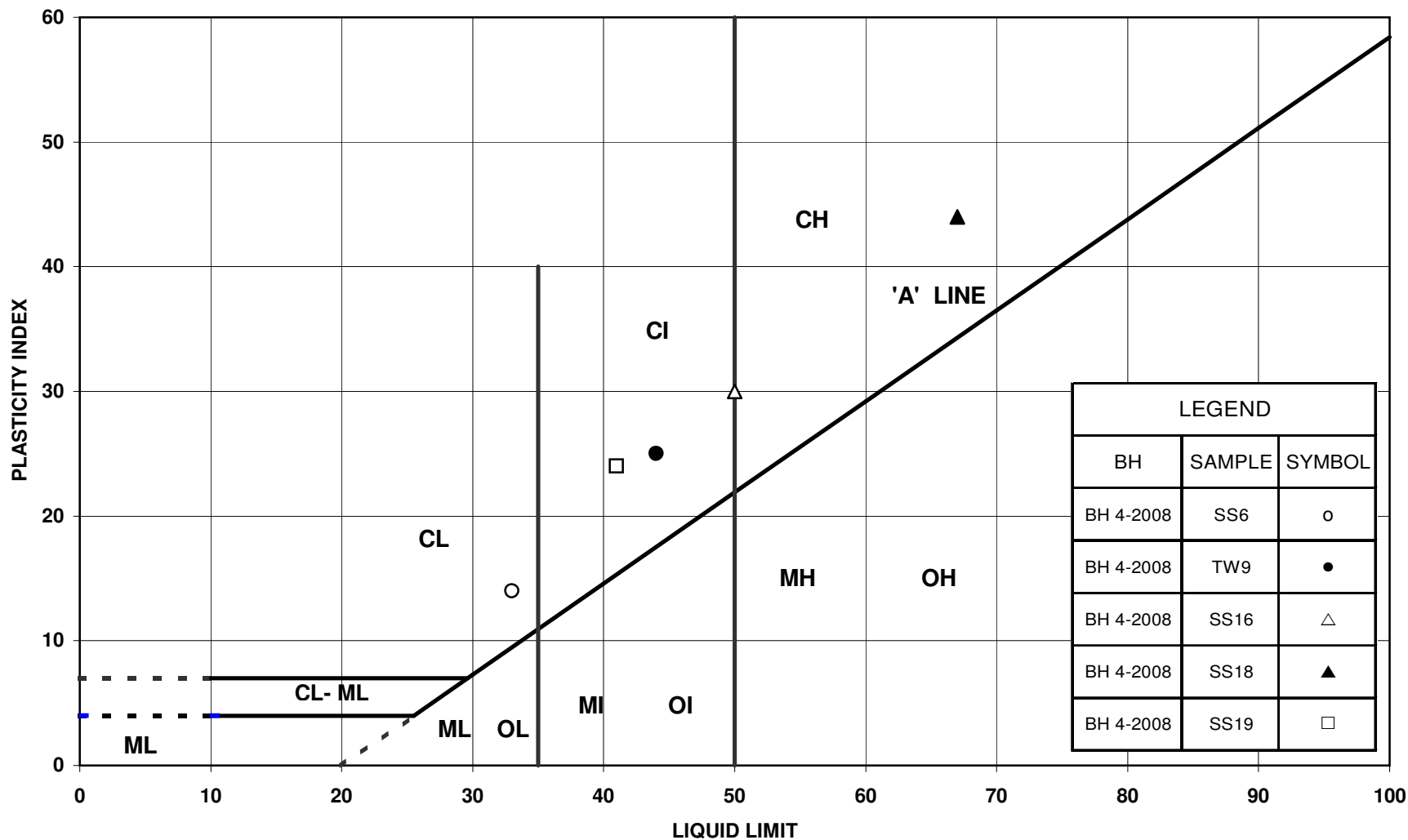
Ministry of
Transportation

PLASTICITY CHART

BH 1-2008, BH 2-2008 and BH 3-2008

FIG No. B1

G.W.P. 454-00-00



Ontario

Ministry of
Transportation

PLASTICITY CHART

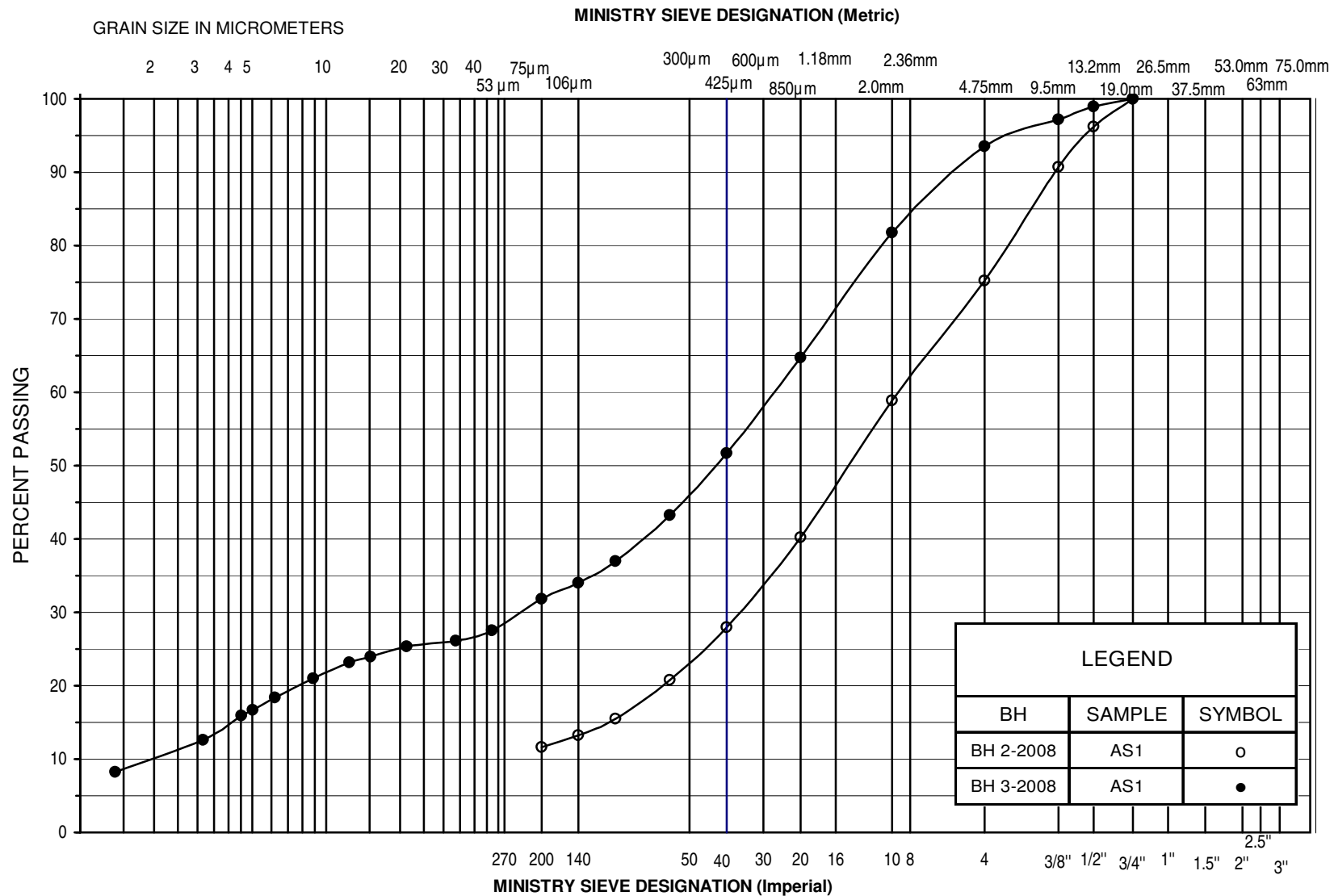
BH 4-2008

FIG No. B2

G.W.P. 454-00-00

UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY & SILT	SAND			GRAVEL	
	Fine	Medium	Coarse	Fine	Coarse



Ontario

Ministry of
Transportation

GRAIN SIZE DISTRIBUTION

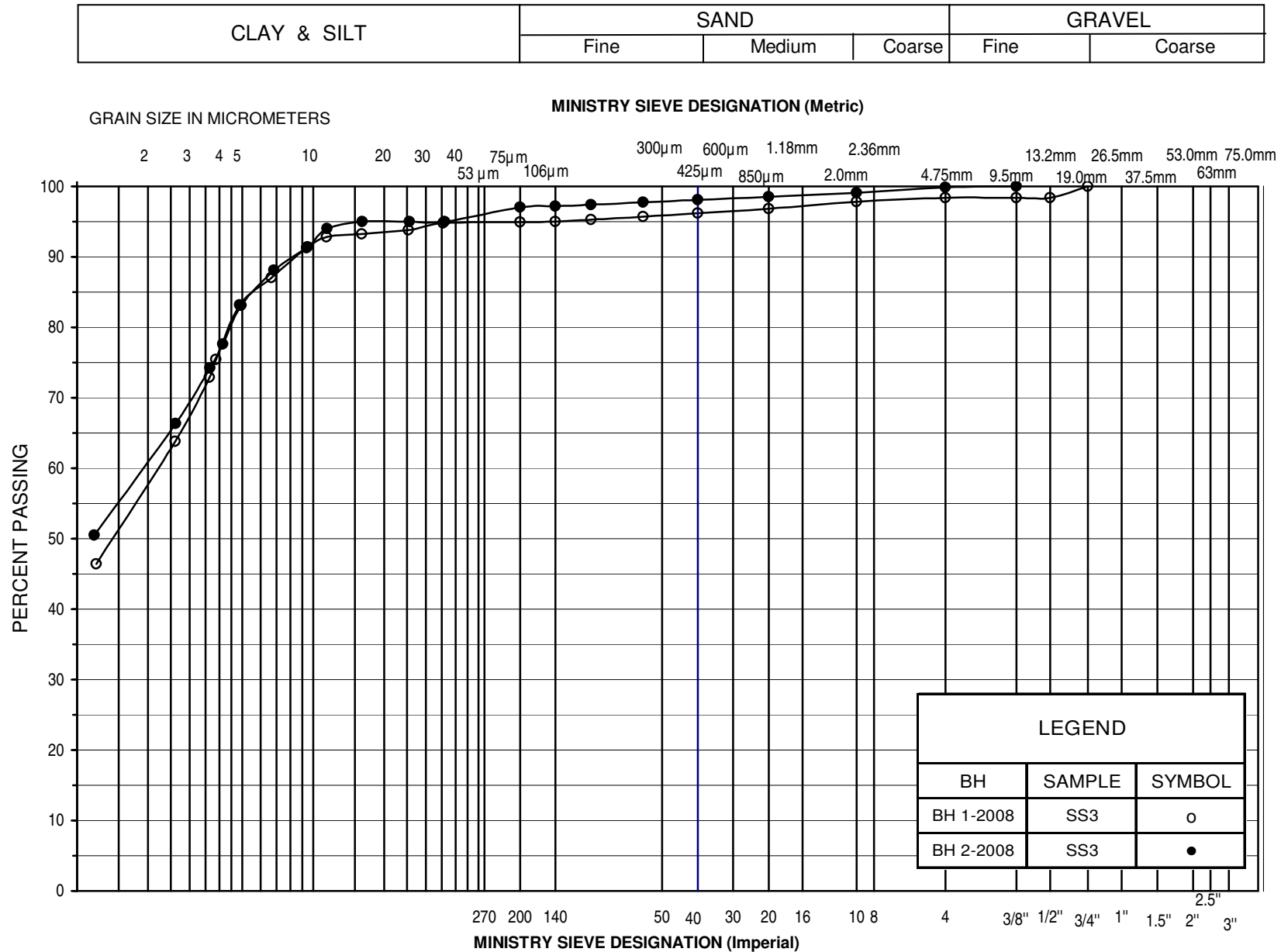
SAND

trace clay, some to with silt, trace to with gravel

FIG. NO. B3

G.W.P.454-00-00

UNIFIED SOIL CLASSIFICATION SYSTEM



Ontario

Ministry of
Transportation

GRAIN SIZE DISTRIBUTION

SILTY CLAY

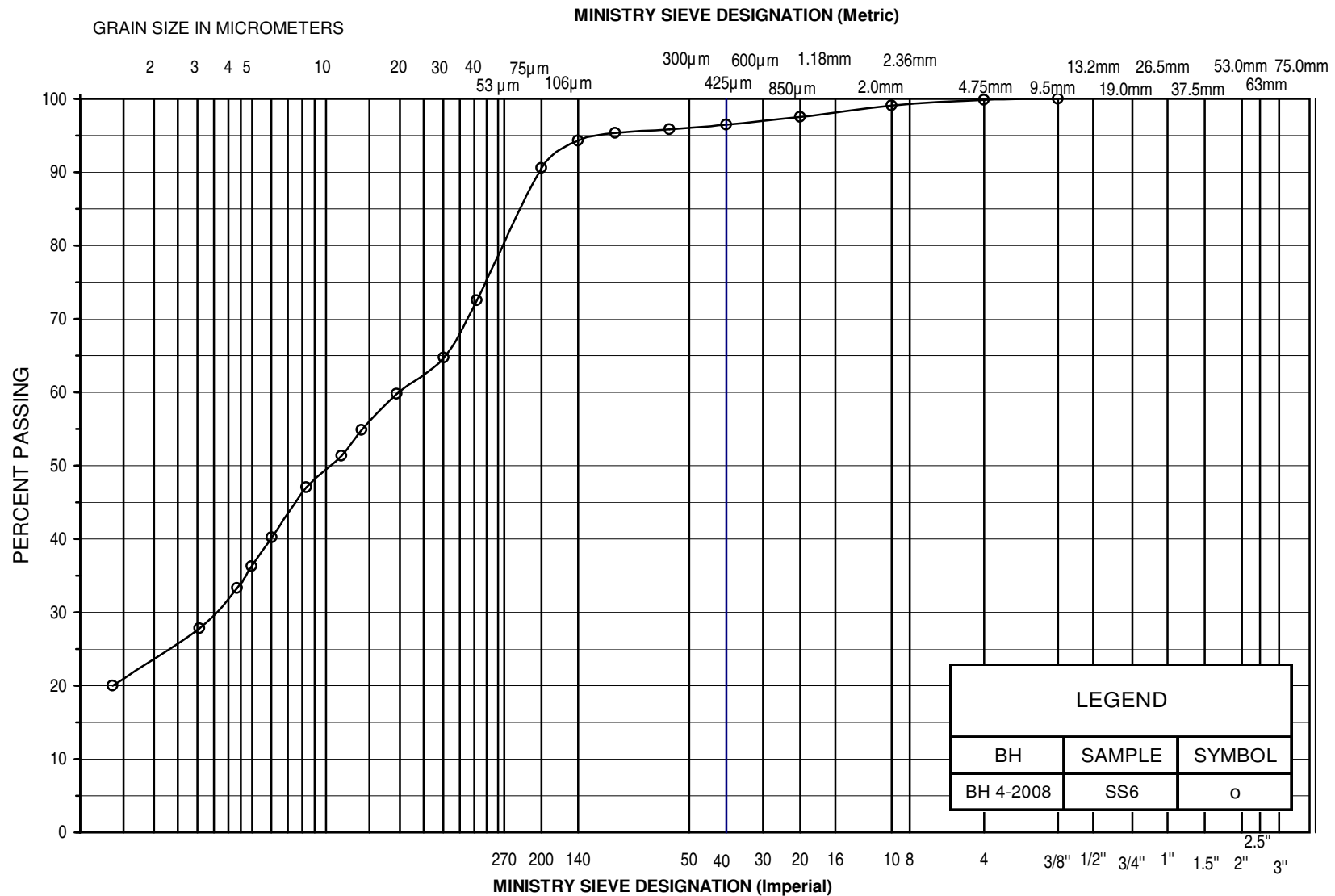
trace sand and gravel

FIG. NO. B4

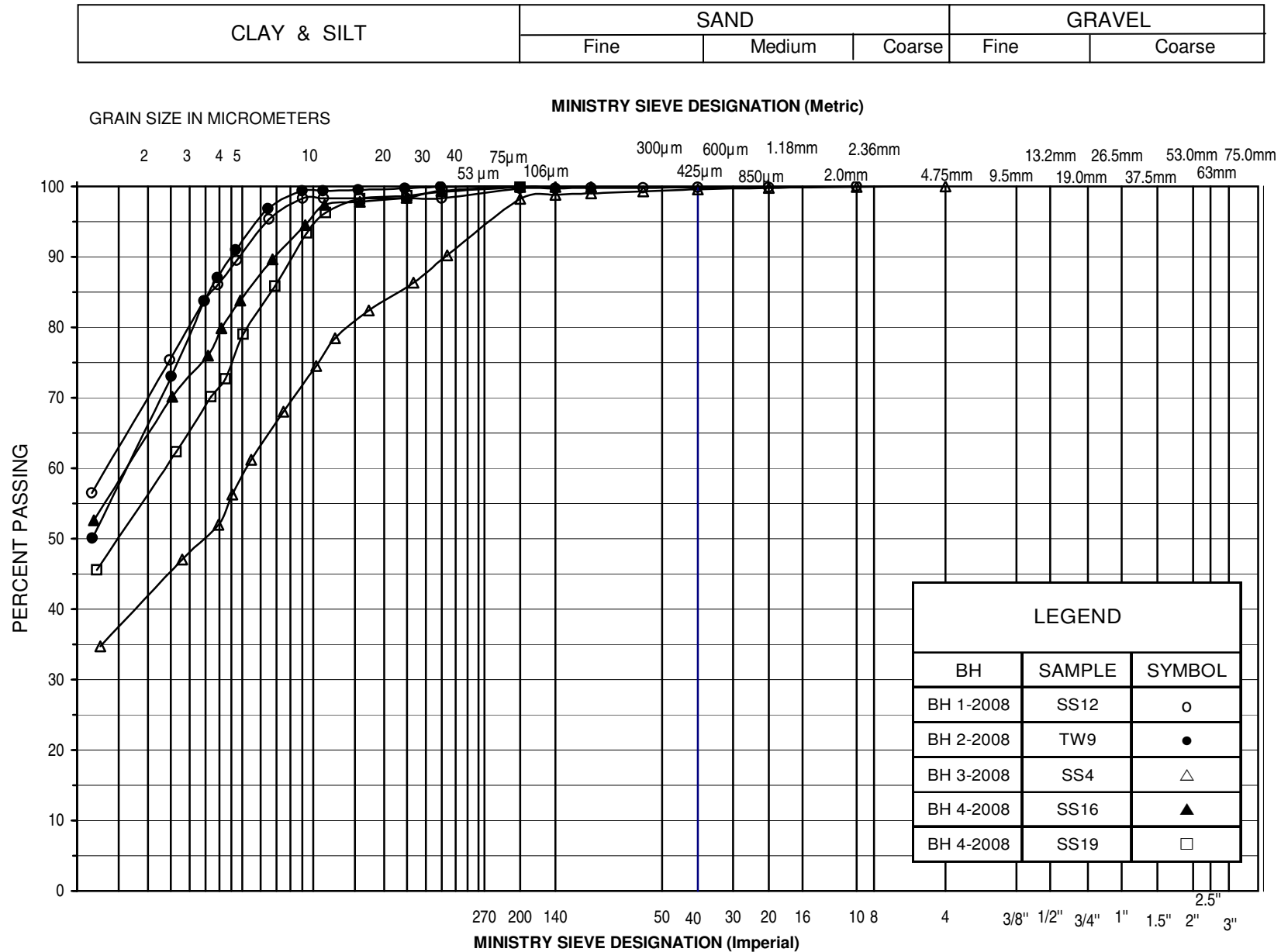
G.W.P.454-00-00

UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY & SILT	SAND			GRAVEL	
	Fine	Medium	Coarse	Fine	Coarse



UNIFIED SOIL CLASSIFICATION SYSTEM



Ontario

Ministry of
Transportation

GRAIN SIZE DISTRIBUTION
SILTY CLAY / SILT AND CLAY
trace sand and gravel

FIG. NO. B6

G.W.P.454-00-00



ONE DIMENSIONAL CONSOLIDATION TEST (ASTM D 2435)

Project:

Job No.: **TT83000**

Client:

Date:

Sample ID: **BH 2-2008 TW9**

Depth(m): **6.1**

σ'_v versus e and c_v

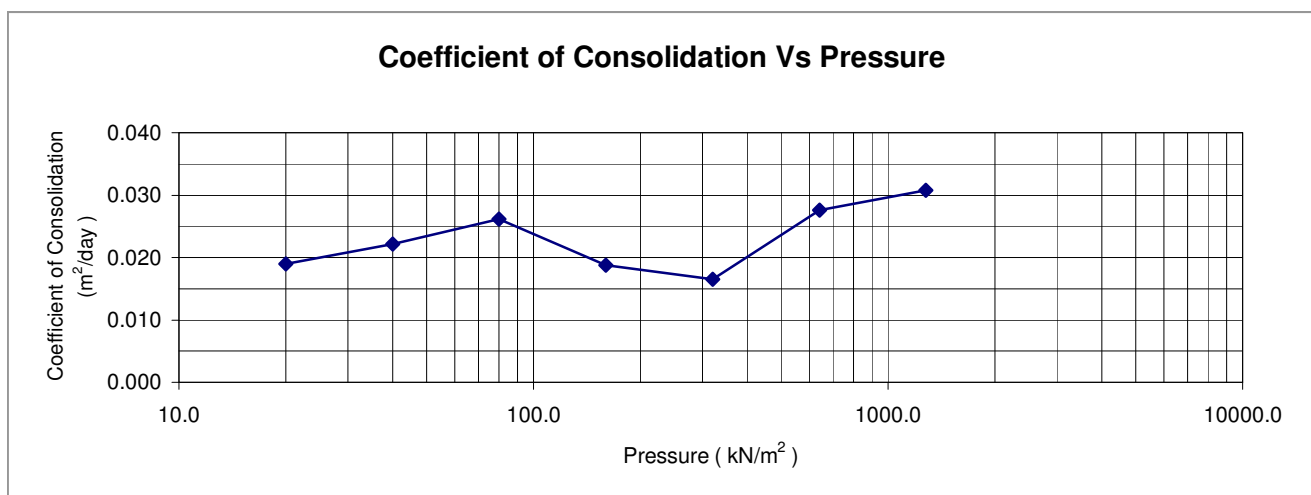
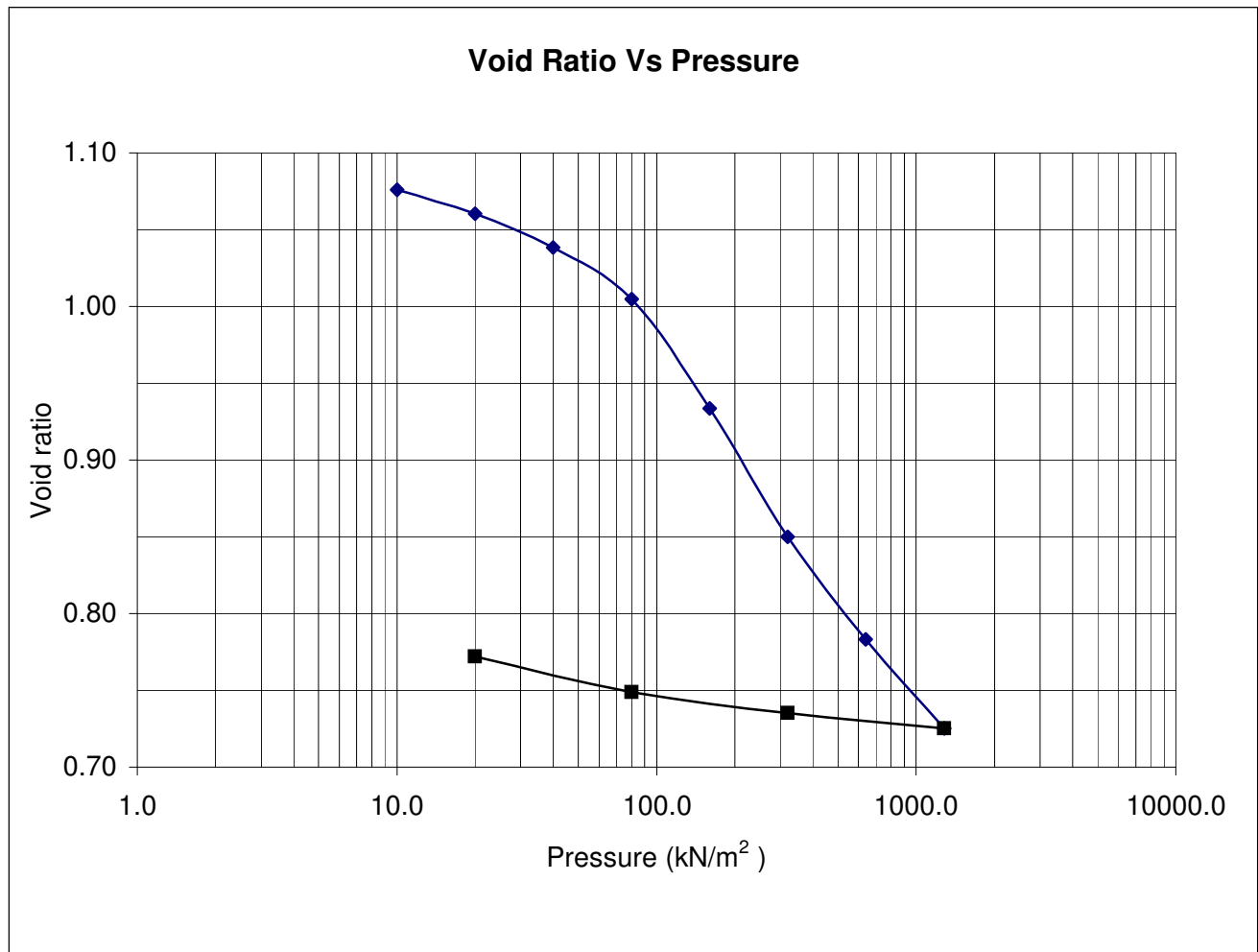


FIGURE B7

Client: AMEC Earth and Environmental,
a division of AMEC Americas Limited
104 Crockford Boulevard
Scarborough, Ontario M1R 3C3

Report Date: April 01, 2008
Received Date: March 25, 2008

Page: 1 of 2

Project Name: Wabigoon River Bridge

Sample Type: Soil

Project Number: TT 83000

Lab Ref.: F2008-0457

Contact: Shami Malla


Final

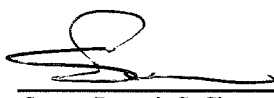
CERTIFICATE OF ANALYSIS

Corrosivity Package

Lab Number Sample ID Date Collected			S2008-03449 BH-2-08 SS5 25-Mar-08	Lab Blank	Q. C. Standard Actual	Q. C. Standard Expected	Date of Analysis
Parameters	Unit	MDL					
Chloride	(µg/g)	1	324	<1	4.2	4.2	27-Mar-08
pH	-	-	7.3	6.3	6.0	6.0	01-Apr-08
Resistivity	(ohmscm)	-	1400	-	-	-	01-Apr-08
Sulphate	(µg/g)	1	11	<1	24.4	24.0	27-Mar-08
Sulphide	-	-	Not Detected	-	-	-	01-Apr-08

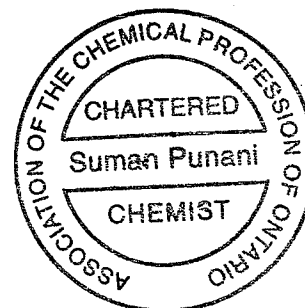
			Method References
Parameters	Unit	MDL	
Chloride	(µg/g)	1	MOE 3013, APHA 4110 C
pH	-	-	MOE 9045
Resistivity	(ohmscm)	-	MOE 3137
Sulphate	(µg/g)	1	MOE 3013, APHA 4110 C
Sulphide	-	-	APHA 4500 S ²⁻ , F


Cynthia Ridge, C. Chem.
Q.A./Q.C. Officer


Suman Punani, C. Chem.
Laboratory Manager

/bpj

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~ GENERAL COMMENTS ~

MDL	Method Detection Limit
RDL	Reporting Detection Limit
ANR	Analysis not required
NA	Analysis not applicable
NP	Not Provided
NR	No Lab Replicate

Result in (brackets) represents Lab Replicate.
Results relate only to the items tested.

APPENDIX C

**DRAWING FOR EXISTING BRIDGE SITE
(As received from MTO)**

HWY 605 CONT No WP No	
WABIGOOON RIVER PLAN and PROFILE	SHEET

UNLESS OTHERWISE SHOWN

WABIGOON RIVER
PLAN and PROFILE

Crest Engineering
Transfer Eng. Office

10+050 10+100 10+150 10+200 10+250 10+300 10+350 10+400

500 x 12.0m CSP

CONCRETE ABUTMENT

RIVER

WL 95.928 (ICE)
DEC 7, 2006

WABIGOON

CONCRETE ABUTMENT

CONCRETE PIERS

500 x 13.0m CSP

SBGR with CHANNEL (typ)

500 x 8.5m CSP

HWY 605

B.M. 100.206m
N&W IN 0.25 POPLAR
17.2 LT 10+306.81

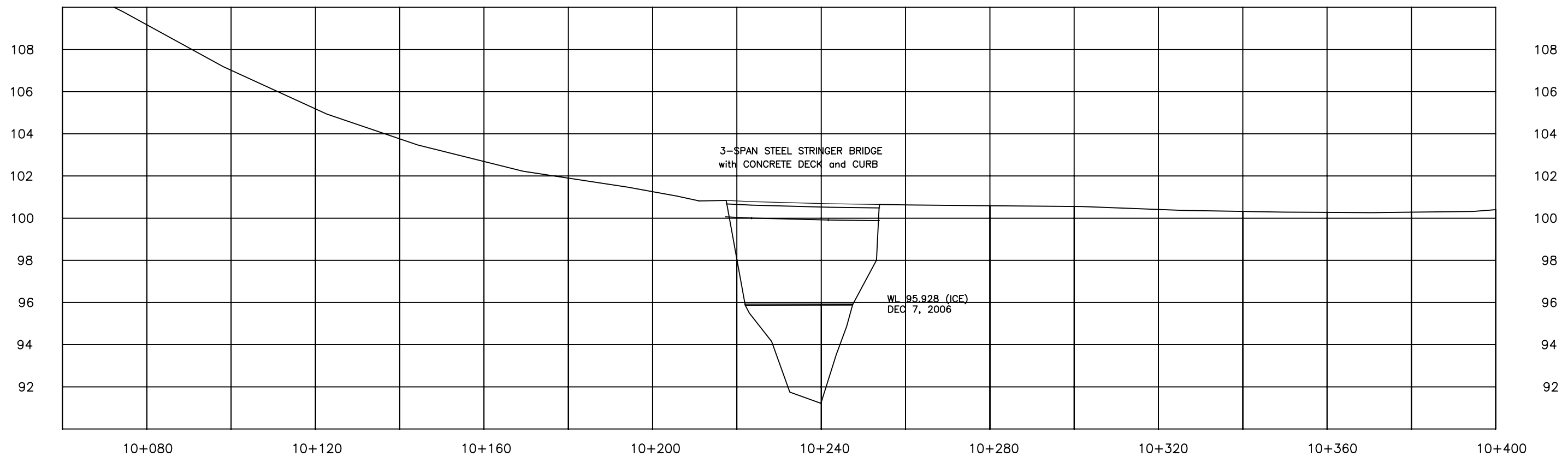
HCM 100

HCM 104

BIS

SCALE
1:1000 h
1:200 v

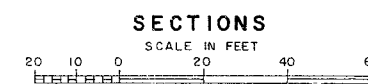
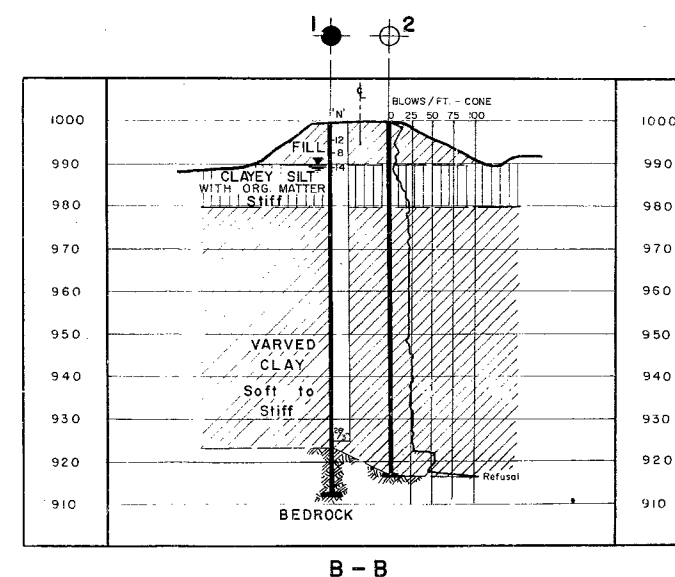
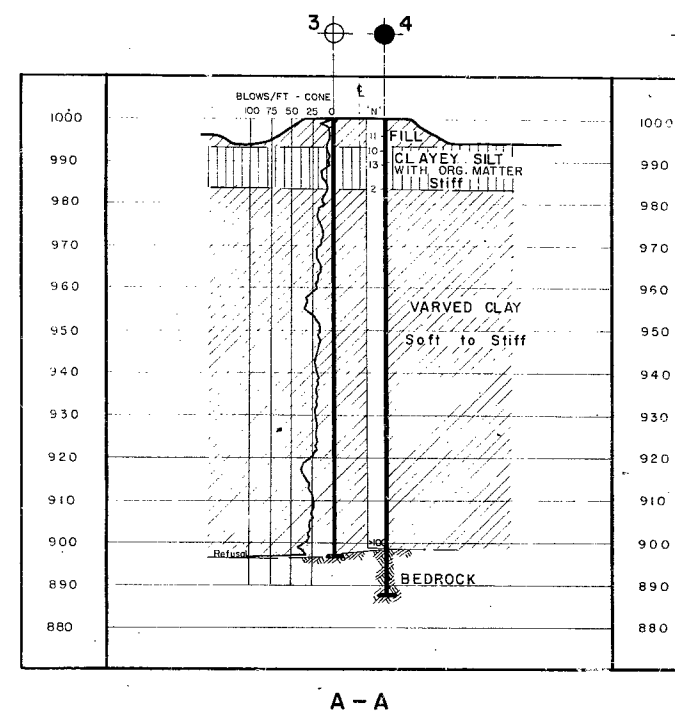
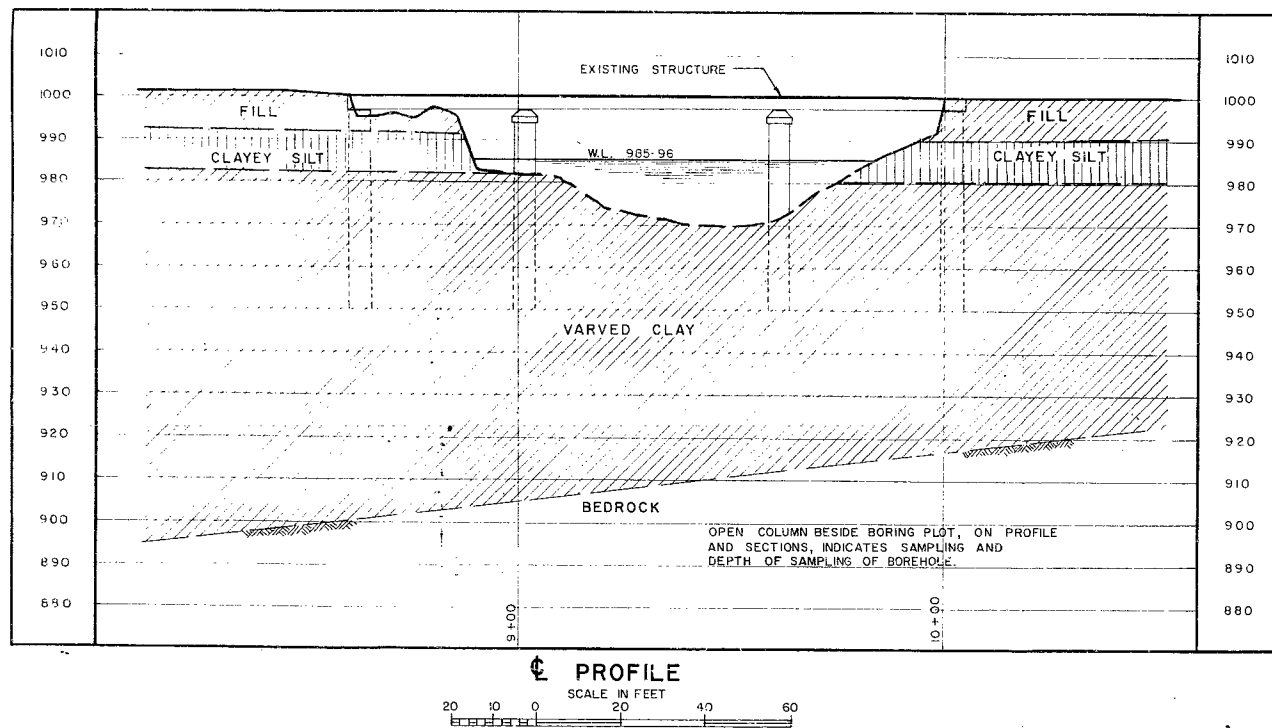
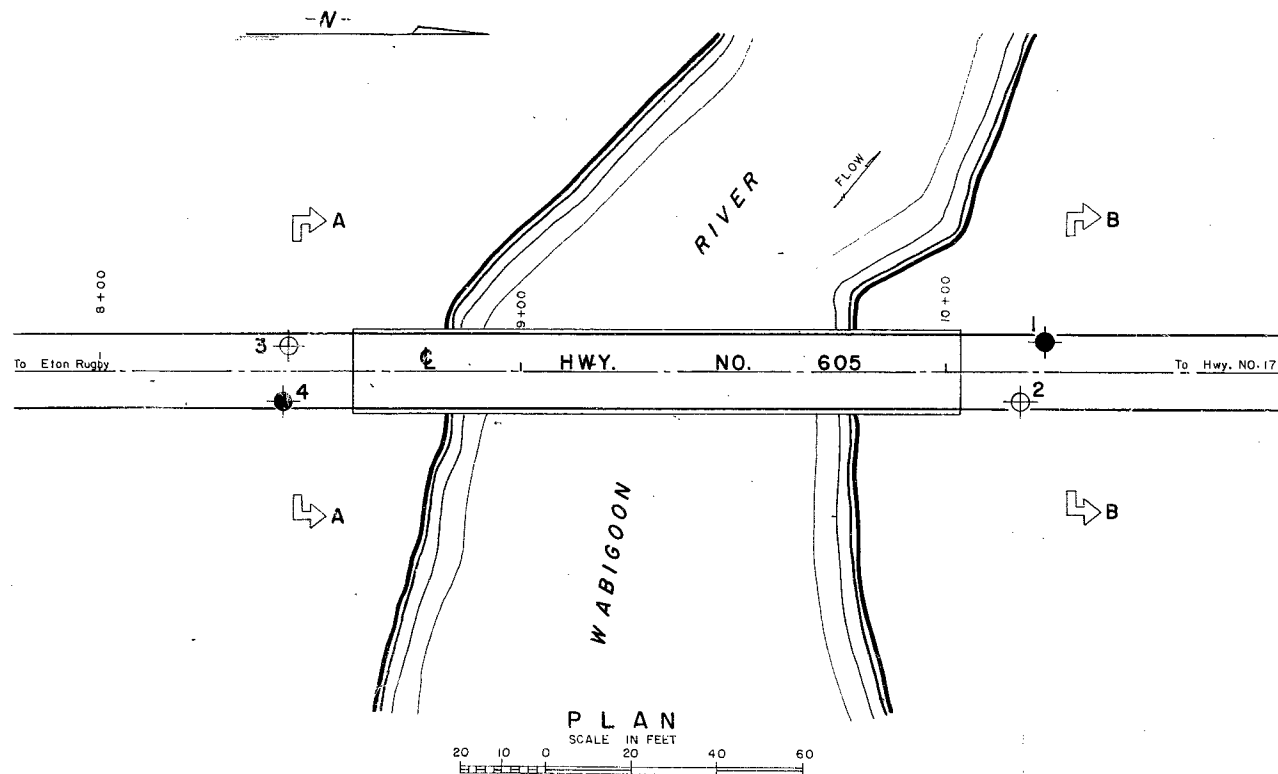
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[illegible]

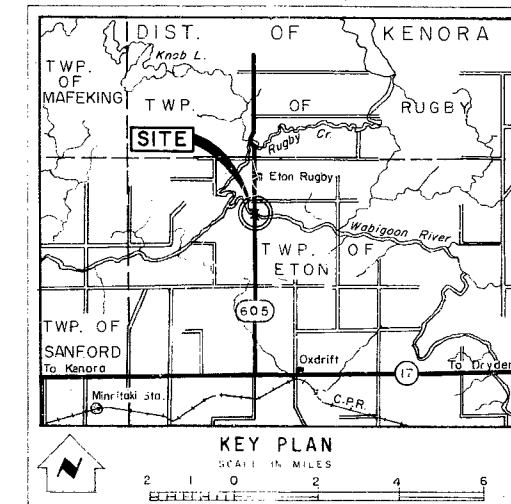
APPENDIX D

BOREHOLE LOGS EXTRACTED FROM MTO REPORT DATED 4 APRIL 1963

(W.J. 62-F-117, District #20, GEOCREs No. 52F-3 and 52F-3-1)



- NOTE -
The complete soil investigation report for this structure may be examined at the Bridge Office and Foundation Office, Downsview, and at the KENORA District Office. The Department does not guarantee the accuracy of this report or the abridged version shown on these plans.



LEGEND			
	Bore Hole		
	Cone Penetration Hole		
	Bore & Cone Penetration Hole		
	Water Levels established at time of field investigation Oct 18/63		
NO.	ELEVATION	STATION	OFFSET
1	1000	10+23	7' LT.
2	1000	10+17	7' RT.
3	1000	8+45	7' LT.
4	1000	8+43	6' RT.

- NOTE -
The boundaries between soil strata have been established only at Bore Hole locations. Between Bore Holes the boundaries are assumed from geological evidence and may be subject to considerable error.

REVISIONS	DATE	BY	DESCRIPTION

DEPARTMENT OF HIGHWAYS - ONTARIO			
MATERIALS & RESEARCH DIVISION - FOUNDATION SECTION			
WABIGOON RIVER			
KING'S HIGHWAY NO. 605		DIST. NO. 20	
CO. KENORA			
TWP. ETON	LOT 687	CON.	V
BORE HOLE LOCATIONS & SOIL STRATA			
SUBM'D T.W.	CHECKED <i>JC</i>	W.P. NO.	M.B.R. DRAWING NO.
DRAWN <i>JC</i>	CHECKED	JOB NO. 62-F-117	62-F-117A
DATE MARCH 1963	SITE NO.	BRIDGE DRAWING NO.	
APPROVED <i>[Signature]</i>	CONT. NO.		

FOUNDATION SECTION

JOB 62-F-117 LOCATION 10717 7' Rt. ORIGINATED BY T.F.W.
W P _____ BORING DATE Oct. 16, 1962. COMPILED BY T.F.W.
DATUM Assumed 1,000 BOREHOLE TYPE Dynamic Cone Penetration Test CHECKED BY _____

[illegible]

FOUNDATION SECTION

[illegible]

FOUNDATION SECTION

SOIL PROFILE			SAMPLES		ELEV SCALE		DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT	LIQUID LIMIT ——— % PLASTIC LIMIT ——— % WATER CONTENT ——— %	BULK DENSITY PCF	REMARKS
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	BLOWS / FOOT	ELEV SCALE	SHEAR STRENGTH P S F. O Unconfined Shear Strength + Field Vane X Laboratory Vane 500 1000 1500 2000	WATER CONTENT % 20 40 60		
1000	Groundlevel					1000				
0.0	Grey stiff silty clay with sand and gravel.		1	SS	11					
993.5			2	SS	10					
6.5	Yellow to grey stiff clayey silt with organic matter.		3	SS	13	990				
983.0			4	SS	2					
17.0			5	TW		980	O +x			
			6	PS			O +x			
			7	PS			+			
			8	PS		970	+ x			
			9	PS			O +x			
			10	PS		960	O +x			
			11	PS			O + x			
			12	PS		950	O +x			
			13	PS			O +x			
			14	SS		940	O +x			
			15	TW			O +x			
			16	TW		930	O x +			
			17	TW			O + x			
			18	TW		920	O x +			
			19	TW			O x +			
			20	TW		910	O x +			
			21	TW			O x +			
			22	TW		900	O x +			
898.0			23	TW						
102.0	Bedrock		24	SS						
887.5						890				
112.50	End of borehole.					880				

APPENDIX E

**RESULTS
OF
SLOPE STABILITY ANALYSES**

METRIC

DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN

AGREEMENT No.

6007-E-0022

G.W.P. No.

454-00-00

DETAIL FOUNDATION INVESTIGATION REPORT
WABIGOON RIVER BRIDGE ON HIGHWAY 605
REPLACEMENT

SITE NUMBER 41S-40



SHEET
1 of 1

amec

AMEC Earth & Environmental,
a Division of AMEC Americas Limited

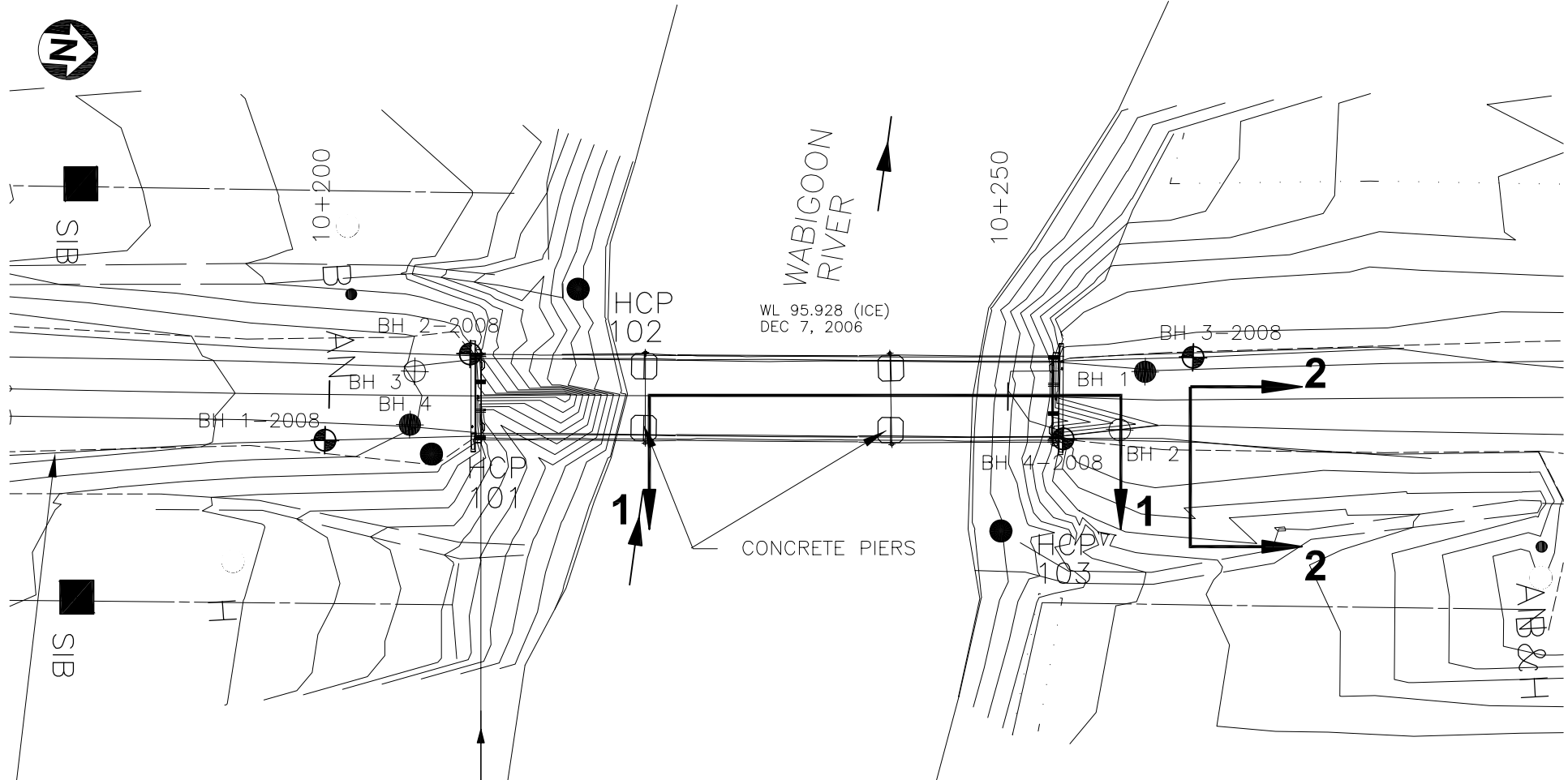
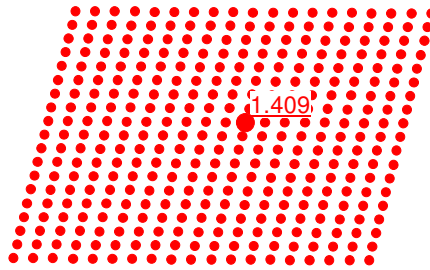


Figure E1 - Section 1-1 and 2-2 for Slope Stability Analyses

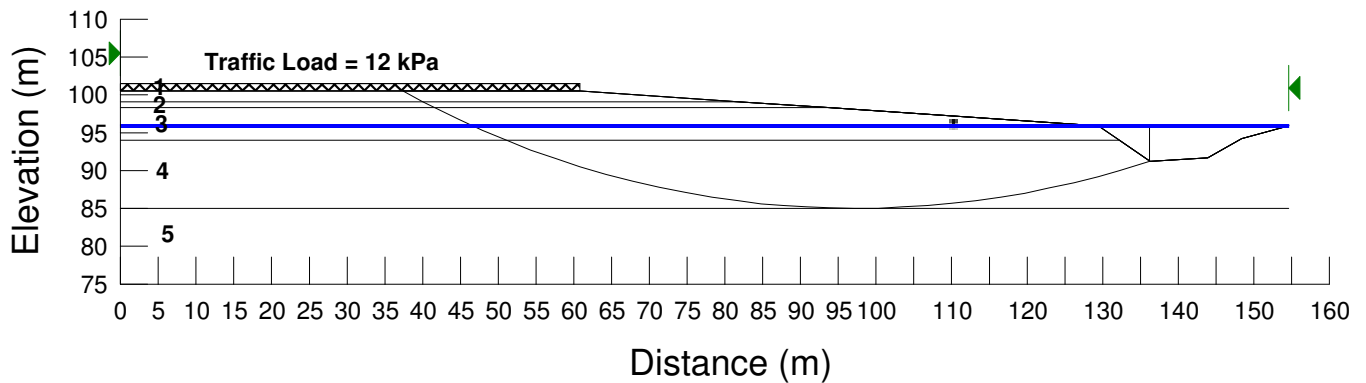
REVISIONS				
DESIGN PB	CHK PB	CODE CHBDC-06	CL 625-ONT	DATE APR. 2008
DRAWN KW	CHK GC	SITE 41S-40	FIG. E1	

Ref: TT83000
Wabigoon River Bridge on Highway 605
Section 1-1 (River Bank Slope)
with water level as of 7 December 2006



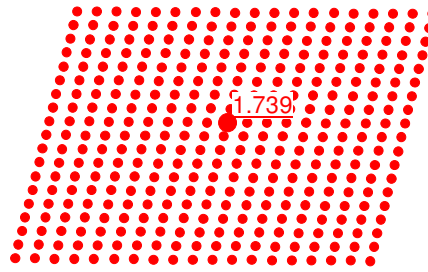
Soil Properties

Soil	Phi (deg)	Cohesion (kPa)	Unit Weight (kN / cu.m)
1 Sand Fill	30	0	18
2 Silty Clay Fill	0	25	18
3 Clayey Silt / Silt	0	20	18
4 Clay / Silty Clay / Silt and Clay	0	15	18
5 Clay / Silty Clay / Silt and Clay	0	35	18



Longitudinal Section at Abutment with existing grade and a slope of 15H:1V

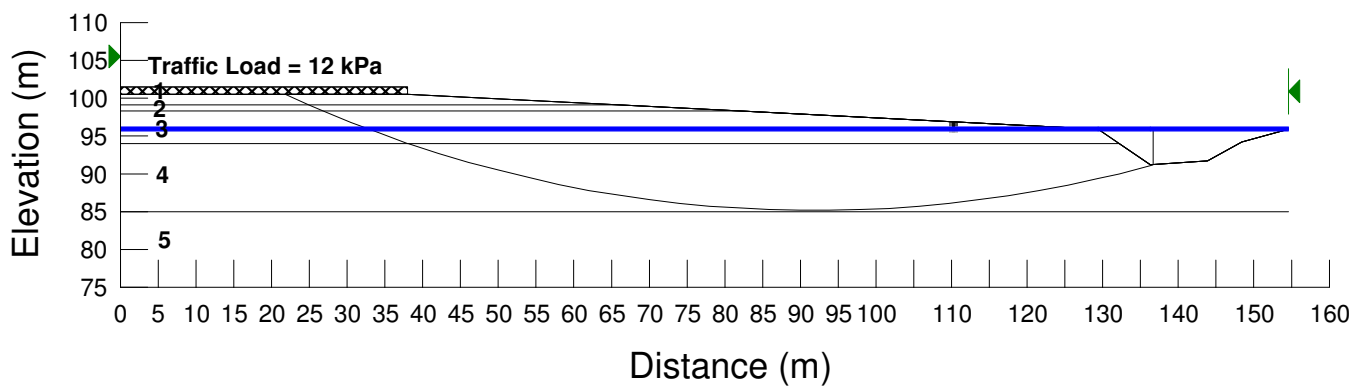
Figure E2.1a Slope Stability Analysis of Section 1-1



Ref: TT83000
Wabigoon River Bridge on Highway 605
Section 1-1 (River Bank Slope)
with water level as of 7 December 2006

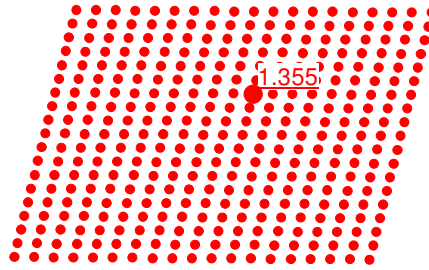
Soil Properties

Soil	Phi (deg)	Cohesion (kPa)	Unit Weight (kN / cu.m)
1 Sand Fill	30	0	18
2 Silty Clay Fill	0	25	18
3 Clayey Silt / Silt	0	20	18
4 Clay / Silty Clay / Silt and Clay	0	15	18
5 Clay / Silty Clay / Silt and Clay	0	35	18



Longitudinal Section at Abutment with existing grade and a slope of 20H:1V

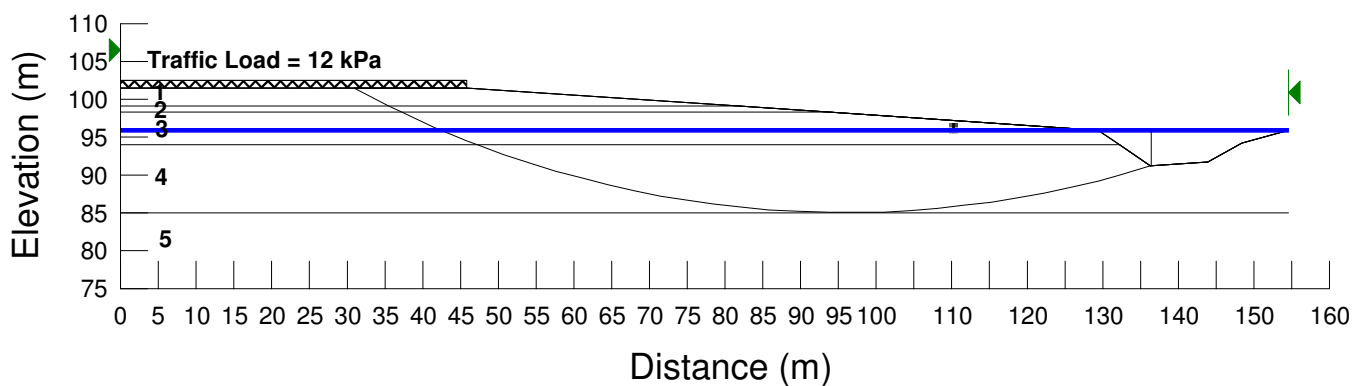
Figure E2.1b Slope Stability Analysis of Section 1-1



Ref: TT83000
Wabigoon River Bridge on Highway 605
Section 1-1 (River Bank Slope)
with water level as of 7 December 2006

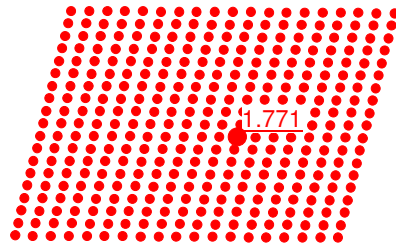
Soil Properties

Soil	Phi (deg)	Cohesion (kPa)	Unit Weight (kN / cu.m)
1 Sand Fill	30	0	18
2 Silty Clay Fill	0	25	18
3 Clayey Silt / Silt	0	20	18
4 Clay / Silty Clay / Silt and Clay	0	15	18
5 Clay / Silty Clay / Silt and Clay	0	35	18



Longitudinal Section at Abutment with 1 m raised grade and a slope of 15H:1V

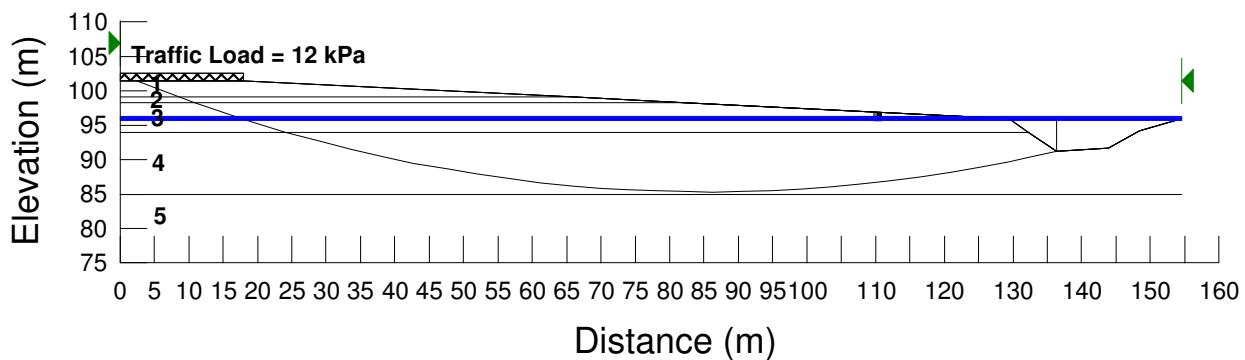
Figure E2.2a Slope Stability Analysis of Section 1-1



Ref: TT83000
 Wabigoon River Bridge on Highway 605
 Section 1-1 (River Bank Slope)
 with water level as of 7 December 2006

Soil Properties

Soil	Phi (deg)	Cohesion (kPa)	Unit Weight (kN / cu.m)
1 Sand Fill	30	0	18
2 Silty Clay Fill	0	25	18
3 Clayey Silt / Silt	0	20	18
4 Clay / Silty Clay / Silt and Clay	0	15	18
5 Clay / Silty Clay / Silt and Clay	0	35	18

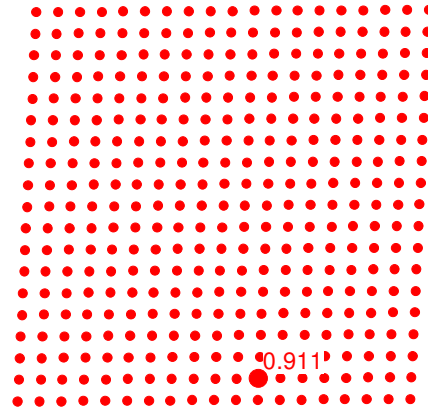


Longitudinal Section at Abutment with 1 m raised grade and a slope of 20H:1V

Figure E2.2b Slope Stability Analysis of Section 1-1

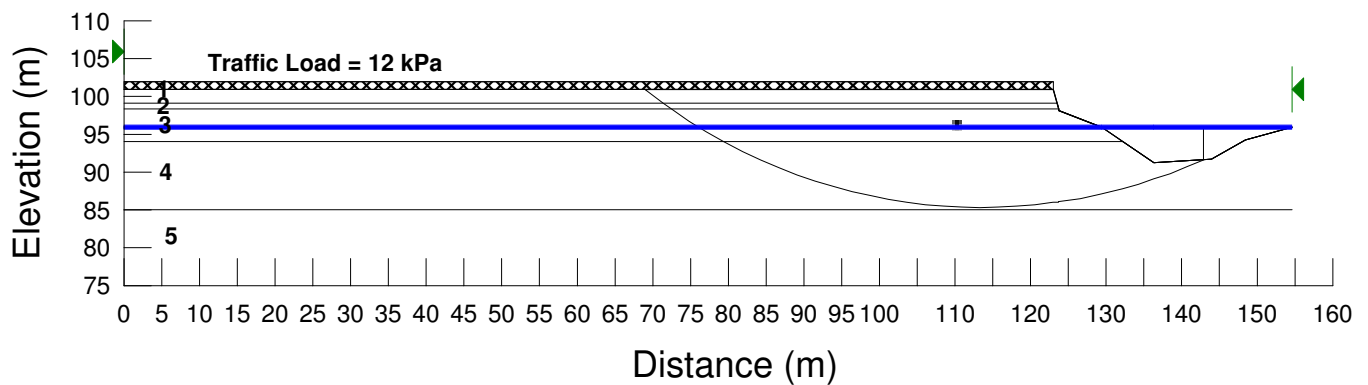
Ref: TT83000
Wabigoon River Bridge on Highway 605
Section 1-1 (River Bank Slope)
with water level as of 7 December 2006

FS without abutments/piers = 0.9
FS with abutments/piers > 1.4



Soil Properties

Soil	Phi (deg)	Cohesion (kPa)	Unit Weight (kN / cu.m)
1 Sand Fill	30	0	18
2 Silty Clay Fill	0	25	18
3 Clayey Silt / Silt	0	20	18
4 Clay / Silty Clay / Silt and Clay	0	15	18
5 Clay / Silty Clay / Silt and Clay	0	35	18



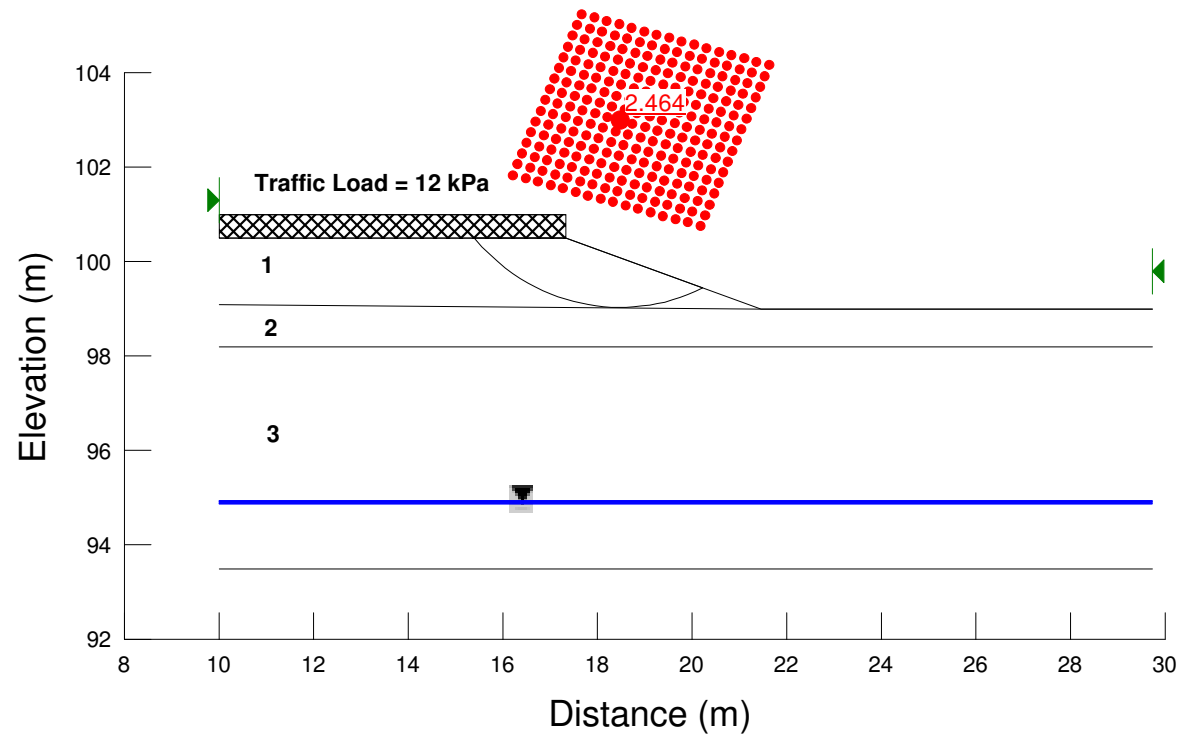
**Longitudinal Section at Abutment with existing grade
(without existing abutments and piers)**

Figure E3.1 Slope Stability Analysis of Section 1-1

Ref: TT83000
Wabigoon River Bridge on Highway 605
Section 2 - 2

Soil Properties

Soil	Phi (deg)	Cohesion (kPa)	Unit Weight (kN / cu.m)
1 Sand Fill	30	0	18
2 Silty Clay Fill	0	25	17
3 Clay / Silty Clay / Silt and Clay	0	15	18



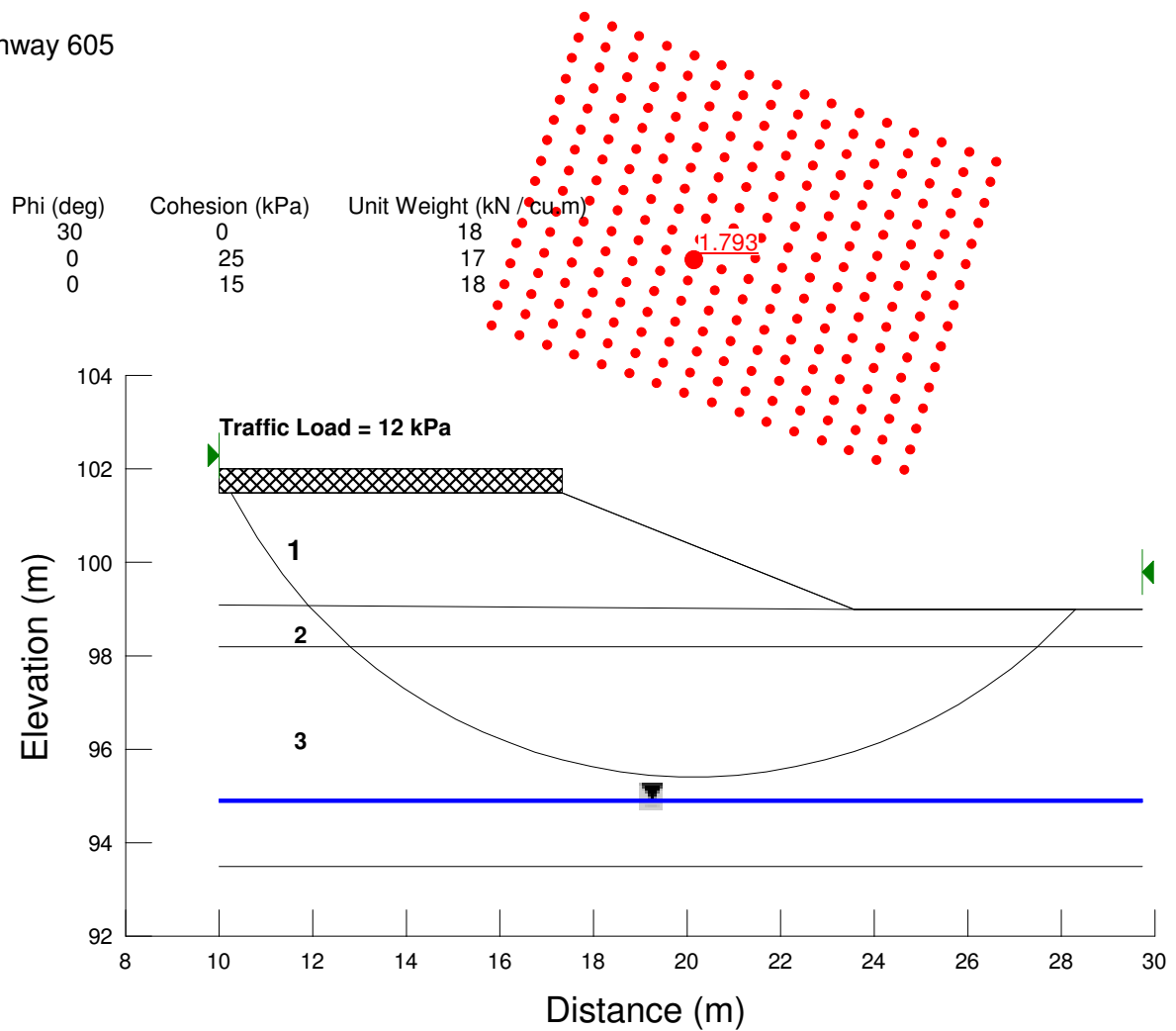
Crosss Section at Approach with existing grade and slope of 2.75H:1V

Figure E4.1 Slope Stability Analysis of Section 2 - 2

Ref: TT83000
Wabigoon River Bridge on Highway 605
Section 2 - 2

Soil Properties

Soil	Phi (deg)	Cohesion (kPa)	Unit Weight (kN / cu.m)
1 Sand Fill	30	0	18
2 Silty Clay Fill	0	25	17
3 Clay / Silty Clay / Silt and Clay	0	15	18



Cross Section at Approach with 1 m raised grade and a slope of 2.5 H:1V

Figure E4.2 Slope Stability Analysis of Section 2 - 2

APPENDIX F
SITE PHOTOGRAPHS



Photo No. 1: General view of bridge site, looking south.



Photo No. 2: General view of bridge site, looking north.



Photo No. 3: West elevation of bridge, looking south.



Photo No. 4: West elevation of bridge, looking north.



Photo No. 5: East elevation of bridge, looking south.

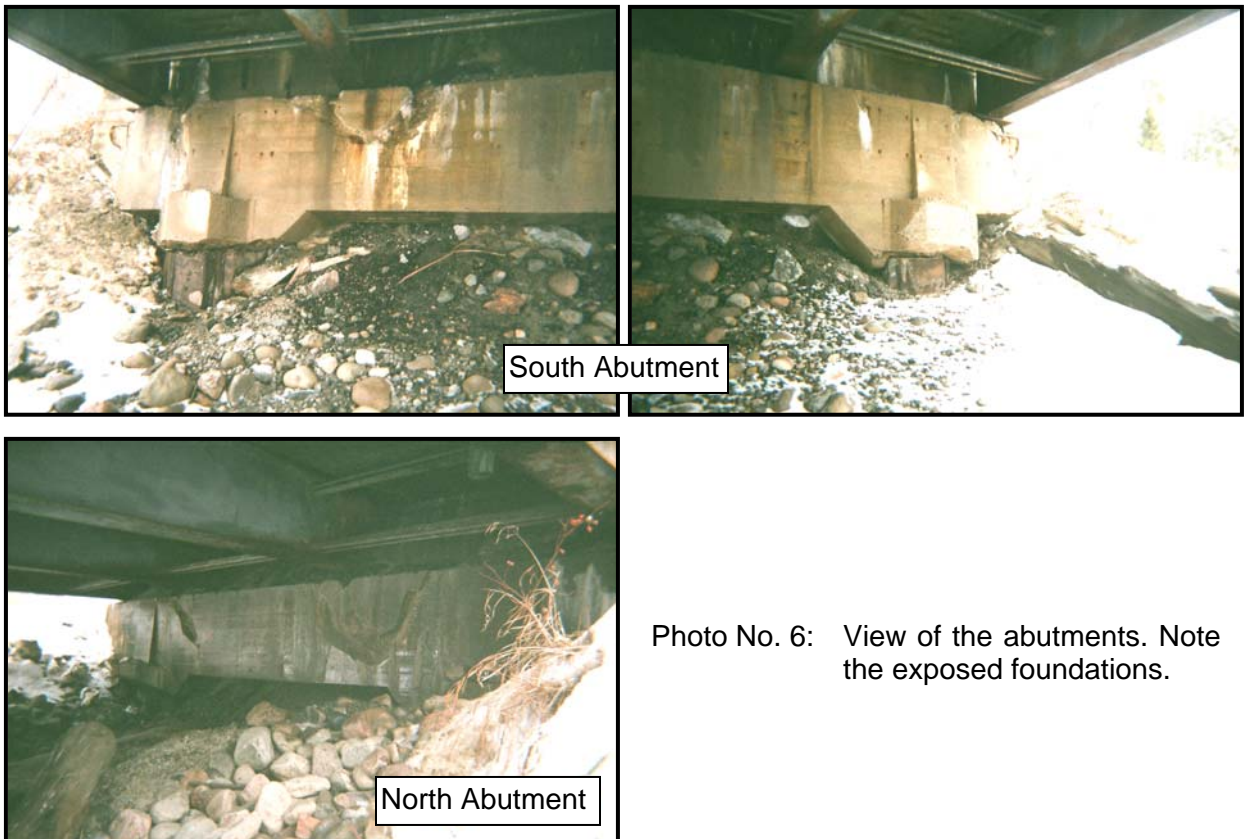


Photo No. 6: View of the abutments. Note the exposed foundations.

APPENDIX G
ROCK CORE PHOTOGRAPHS



BH 2-2008 - Rock Core (from 35.0 to 37.5 m depth)
Overall view and close-up views.

(Wabigoon River Bridge)



BH 4-2008 - Rock Core (from 35.0 to 37.5 m depth)
Overall view and close-up views.

(Wabigoon River Bridge)