



May 9, 2013

FOUNDATION INVESTIGATION AND DESIGN REPORT

**KAIBUSKONG RIVER BRIDGE REHABILITATION
ROADWAY PROTECTION, SITE 43-113
REHABILITATION OF HIGHWAY 17, FROM 0.3 KM
WEST OF HIGHWAY 94 EASTERLY 12.8 KM
BONFIELD TOWNSHIP, ONTARIO
MINISTRY OF TRANSPORTATION, ONTARIO
GWP 174-98-00, WP 5628-04-01**

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REPORT





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PART A

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1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by MMM Group Limited (MMM) on behalf of Ministry of Transportation, Ontario (MTO) to provide foundation engineering services for the temporary roadway protection associated with the rehabilitation of the Highway 17 Bridge over Kaibuskong River in Bonfield Township, east of North Bay, Ontario. The proposed work is part of the rehabilitation of Highway 17 from 0.3 km west of Highway 94 easterly 12.8 km. The approximate location of the Highway 17 Bridge over Kaibuskong River is shown on the Key Plan on Drawing 1.

The Terms of Reference and the Scope of Work for the foundation investigation for the Kaibuskong River Bridge are outlined in Addendum No. 2 of MTO's Request for Proposal, dated September 2010. Golder's proposal for foundation engineering services associated with this project is contained in Section 6.8 of MMM's Technical Proposal for this assignment. The work has been carried out in accordance with Golder's Supplemental Specialty Quality Control Plan for foundation engineering services for this project, dated January 5, 2011. The General Arrangement (GA) drawing for the bridge was provided to Golder by MMM.

This report addresses the investigation carried out for the temporary roadway protection for the Kaibuskong River Bridge only. Separate reports will be submitted detailing the foundation investigations for the culverts, lane extensions and the proposed Highway 17/94 Roundabout.

The purpose of this investigation is to establish the subsurface conditions within the vicinity of the proposed roadway protection for the Kaibuskong River Bridge by methods of borehole drilling, in situ testing and laboratory testing on selected soil samples. The boreholes were located in the field by Golder relative to stakes installed by MMM.

2.0 SITE DESCRIPTION

The Highway 17 bridge crossing Kaibuskong River is located approximately 220 m west of Highway 531. The bridge was constructed in 1960 and is a single span structure 36.6 m long. The bridge deck is supported by steel girders and the abutments are founded on approximately 7 m long steel H-piles driven to bedrock.

In general, the topography in the vicinity of the bridge is flat with the Kaibuskong River incised into the plain about 4 m and flowing in a southerly direction. The banks of the river and the approach embankment side slopes are vegetated with grass. The area in the vicinity of the site is generally residential, landscaped with trees and shrubs. The surface of the roadway at the bridge is at about Elevation 215 m and the water level surveyed by MMM in June 2011 was at Elevation 208.1 m. The river is about 12 m wide at the bridge and the water is about 2 m deep.

3.0 INVESTIGATION PROCEDURES

The fieldwork for the investigation associated with the proposed temporary roadway protection for the rehabilitation of Kaibuskong River Bridge was carried out on July 5 and 6, 2011. A total of two (2) boreholes were advanced through the existing approach embankments (designated as Boreholes KB-1 and KB-2) at approximately the locations shown on Drawing 1.



The field investigation was carried out using a track mounted CME 55 drilling rig supplied and operated by Landcore Drilling of Sudbury, Ontario.

The boreholes were advanced through the overburden using 108 mm inside diameter hollow-stem augers. In general, soil samples were obtained at intervals at depths of about 0.75 m and 1.5 m, using a 50 mm outer diameter split-spoon sampler operated by automatic hammers on the drill rig, performed in accordance with Standard Penetration Test (SPT) procedures (ASTM D1586). Field vane shear tests were carried out in cohesive soils for determination of undrained shear strengths (ASTM D2573) using an MTO Standard 'N' size vane. All boreholes were backfilled with bentonite upon completion in accordance with Ontario Regulation 903 Wells (as amended).

Boreholes KB-1 and KB-2 were advanced to auger refusal encountered at depths of 11.6 m and 12.6 m, respectively, below the existing roadway surface. The refusal to further auger advancement was likely a result of the augers being on or in proximity to the bedrock surface.

The groundwater conditions and water levels in the open boreholes were observed during the drilling operations and are described on the Record of Borehole sheets in Appendix A. Groundwater elevations as encountered in the boreholes may not be representative of static groundwater levels since the groundwater levels in the boreholes may not have stabilized on completion of drilling. Furthermore, groundwater elevations will vary depending on seasonal fluctuations, precipitation and local soil permeability. A piezometer was installed in Borehole KB-2 to permit monitoring of the groundwater level at this location. The piezometer consists of a 19 mm diameter PVC pipe with a 1.5 m long slotted screen sealed within the silt and silty sand deposits. The borehole annulus surrounding the piezometer screen was backfilled with sand and the remainder of the borehole was backfilled with bentonite. On October 23, 2012, the piezometer was found to be blocked at ground surface with bentonite and a water level could not be recorded. The piezometer was decommissioned on October 23, 2012.

The fieldwork was observed by members of our engineering and technical staff who: located the boreholes; arranged for the clearance of underground services; observed the drilling, sampling and in situ testing operations; logged the boreholes; and examined and cared for the soil samples. The samples were identified in the field, placed in appropriate containers, labelled and transported to our Sudbury Geotechnical Laboratory where the samples underwent further visual examination and laboratory testing. All of the laboratory tests were carried out to MTO Laboratory Standards and/or ASTM Standards, as appropriate. Classification testing (water content, Atterberg limits and grain size distribution) was carried out on selected samples.

Survey stakes offset from the Highway 17 centerline were installed by MMM prior to the commencement of drilling. The as-drilled borehole locations, in stations and offsets, were measured in reference to the applicable stakes installed by MMM and were subsequently converted into MTM NAD 83 coordinates in AutoCAD. The ground surface elevation at the borehole locations was surveyed by a member of our technical staff in reference to the ground surface elevations at applicable stakes installed by MMM. The borehole locations shown on Drawing 1 are positioned relative to MTM NAD 83 northing and easting coordinates and the ground surface elevations are referenced to Geodetic datum. The borehole locations, ground surface elevations, and drilled depths are as follows:



Borehole	Location (m)		Ground Surface Elevation (m)	Borehole Depth (m)
	Northing	Easting		
KB-1	5125612.6	332319.9	215.2	11.6
KB-2	5125622.5	332384.1	215.0	12.6

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

Based on terrain mapping by the Ontario Geological Survey, the soils in the vicinity of the Highway 17 bridge crossing at Kaibuskong River consist of glaciolacustrine plain silts and sands.

The bedrock in the area typically consists of gneisses of the Powassan or Tilden Lake Domain, both within the Central Gneiss Belt, a subdivision of the Grenville Structural Province, as described in Geology of Ontario, OGS Special Volume 4.

4.2 Subsurface Conditions

The detailed subsurface soil and groundwater conditions, as encountered in the boreholes advanced during this investigation, together with the results of the laboratory tests carried out on selected soil samples, are presented on the Record of Borehole sheets in Appendix A and the laboratory test sheets in Appendix B. The stratigraphic boundaries shown on the Record of Borehole sheets are inferred from non-continuous sampling, observations of drilling progress and in situ testing. These boundaries, therefore, represent transitions between soil types rather than exact planes of geological change. Further, subsurface conditions will vary between and beyond the borehole locations.

The inferred soil stratigraphy, as encountered in the boreholes, is shown in profile on Drawing 1. The orientation (i.e. north, south, east, west) stated in the text of the report is typically referenced to project north. For the purposes of this report, the Highway 17 corridor alignment is in an east-west direction, therefore, the directions indicated in the text may differ from those shown on the drawing.

In general, the subsurface stratigraphy encountered within the two boreholes generally consists of pavement structure (asphalt and granular fill) underlain by deposits of clayey silt to silty clay, silt and silty sand.

Embankment Fill

Boreholes KB-1 and KB-2 were advanced through the existing roadway and encountered 150 mm and 260 mm of asphalt, respectively, at the highway surface, corresponding to Elevation 215.2 m and 215.0 m. Below the asphalt, the boreholes penetrated a deposit of granular fill comprised of sand to sand and gravel and silty sand with a thickness of about 2.8 m and 2.7 m at the respective boreholes.

SPT 'N'-values measured within the granular fill range from 8 blows to 60 blows per 0.3 m of penetration, indicating a loose to very dense relative density.



The grain size distributions of two samples of the granular fill are presented on Figure B1 in Appendix B.

The water content measured on two samples of the granular fill is 6 per cent and 11 per cent.

Clayey Silt to Silty Clay

A deposit of brown to grey, clayey silt to silty clay was encountered underlying the granular fill in both boreholes. The clayey silt to silty clay deposit was noted to contain silt seams/layers. The top of this deposit is at Elevation 212.3 m and 212.0 m in Boreholes KB-1 and KB-2, respectively, and the thicknesses of the deposit is 4.3 m and 6.6 m.

The SPT 'N'-values recorded within this deposit range from 0 blows (weight of hammer) to 8 blows per 0.3 m of penetration. In situ field vane testing carried out within this stratum measured undrained shear strengths ranging from about 28 kPa to greater than 100 kPa. The SPT tests together with the field vane test suggest the deposit is generally firm to very stiff.

Atterberg limits testing carried out on four samples of the clayey silt to silty clay deposit yielded liquid limits ranging from about 31 per cent to 39 per cent, plastic limits ranging from about 20 per cent to 23 per cent and plasticity indices ranging from about 11 per cent to 16 per cent. The results of the Atterberg limits testing are shown on the plasticity chart on Figure B2 in Appendix B, and indicate that the deposit consists of clayey silt of low plasticity to silty clay of intermediate plasticity.

The natural water content measured on five samples of the clayey silt to silty clay deposit is between about 28 per cent and 44 per cent.

Silt

A deposit of grey silt was encountered underlying the clayey silt to silty clay stratum in both boreholes. The top of the silt deposit is at Elevation 208.0 m and 205.4 m, and the thickness of the deposit is 4.4 m and 2.1 m at Boreholes KB-1 and KB-2, respectively.

SPT 'N'-values recorded within the silt deposit range from 4 blows to 12 blows per 0.3 m of penetration, indicating a loose to compact relative density.

The grain size distributions for two samples of the silt deposit are shown on Figure B3, in Appendix B.

The natural water content measured on two samples of the silt deposit is about 28 per cent and 34 per cent. An Atterberg limits test was attempted on a sample of the silt deposit from Borehole KB-1 and the results of the test indicated that the sample was non-plastic.

Silty Sand

A 0.9 m thick stratum of grey silty sand was encountered underlying the silt in Borehole KB-2 at Elevation 203.3 m.



The SPT 'N'-value recorded within the silty sand deposit is 6 blows per 0.3 m of penetration, indicating a loose relative density.

Refusal

Refusal to further auger advancement was encountered in Boreholes KB-1 and KB-2 at depths of 11.6 m and 12.6 m, respectively, below ground surface, corresponding to Elevation 203.6 m and 202.4 m. These refusal depths, while they do not confirm bedrock elevations, may be likely on or in proximity to the bedrock surface.

Groundwater Conditions

The depth to the water level measured below ground surface in Boreholes KB-1 and KB-2 upon completion of drilling is 7.8 m and 3.8 m, respectively, corresponding to Elevation 207.4 m to 211.2 m.

Groundwater levels in the area are subject to seasonal fluctuations and variations due to precipitation events.

5.0 CLOSURE

The field personnel supervising the drilling program were Mr. Luigi Gianfrancesco, EIT and Mr. Matthew Thibeault, EIT. This report was prepared by Mr. Matthew Thibeault, EIT. The technical aspects were reviewed by Mr. André Bom, P.Eng., and Mr. Jorge M. A. Costa, P.Eng., Principal and Golder's Designated MTO Contact for this project, who also carried out a quality control review of the report.

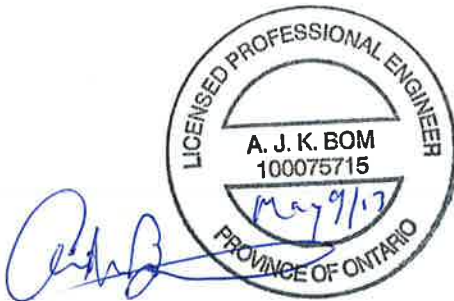


Report Signature Page

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PART B

FOUNDATION DESIGN REPORT
KAIBUSKONG RIVER BRIDGE REHABILITATION
ROADWAY PROTECTION, SITE 43-113
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6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

This section of the report provides an interpretation of the geotechnical data obtained during the investigation and recommendations on the foundation aspects of design of the proposed works. The recommendations provided are intended for the guidance of the design engineer. Where comments are made on construction, they are provided to highlight aspects of construction that could affect the design of the project. Those requiring information on aspects of construction must make their own interpretation of the subsurface information provided as it affects their proposed construction methods, costs, equipment selection, scheduling and the like.

6.1 General

Golder was retained by MMM to provide recommendations on temporary roadway protection for the rehabilitation of the Highway 17 Bridge over Kaibuskong River in Bonfield Township, Ontario. The roadway protection will be required during the construction works to convert the existing bridge to a semi-integral structure. As the structure will be rehabilitated in stages, with traffic reduced to one lane in the vicinity of the bridge, the excavations at the abutments and approach embankments will be supported by a temporary support structure to maintain the stability of the existing roadway embankment.

The subsurface conditions at the west approach embankment at Borehole KB-1 consist of 2.9 m of embankment fill, underlain by a stratum of varved clayey silt to silty clay approximately 4.3 m thick and a deposit of silt approximately 4.4 m thick. Auger refusal, likely on or in the vicinity of bedrock, was encountered at a depth of 11.6 m below the existing roadway surface, corresponding to Elevation 203.6 m. The non-stabilized water level upon completion of drilling was measured at a depth of 7.8 m below the roadway surface corresponding to Elevation 207.4 m.

At the east approach embankment at Borehole KB-2, the subsurface conditions consist of 2.8 m of embankment, fill underlain by a 6.6 m thick deposit of varved clayey silt to silty clay and a 3.0 m thick deposit of silt to silty sand. Auger refusal was encountered at a depth of 12.6 m below the existing roadway surface corresponding to Elevation 202.4 m. The non-stabilized water level in the borehole upon completion of drilling was measured at a depth of 3.8 m below existing ground surface, corresponding to Elevation 211.2 m.

Excavations will be required to expose the existing abutments. Based on the GA drawing provided by MMM, the underside of the existing abutments are at about Elevation 208 m. The river water level was surveyed by MMM in June 2011 at Elevation 208.1 m.

6.2 Excavations and Temporary Cut Slopes

The proposed works will require excavations through the embankment fill behind the existing abutments in order to rehabilitate the existing abutments and other components of the bridge. Depending on the depth of excavation, groundwater may be encountered, as the stabilized water level at the approach embankments is likely at or near the elevation of the river water level. The groundwater level is subject to fluctuations and the depth of excavation below the groundwater will depend on the time of year of construction. Also, perched groundwater may be present within the granular fill layers. Surficial water seepage into the excavations should be expected and will be greater during periods of sustained precipitation. Pumping from properly filtered sumps



located at the base of the excavations may be required to provide groundwater control but should be located outside of the actual excavation limits required for the rehabilitation works. Surface water runoff should be directed away from the excavations at all times.

All excavations should be carried out in accordance with the latest edition of the Ontario Occupational Health and Safety Act and Regulations for Construction Projects. The fill materials at this site would be classified as Type 3 soils. The native firm to stiff clayey silt to silty clay and loose to compact silt to silty sand materials would be classed as Type 3 soils. Temporary open cut slopes within the fill materials and native soils should be maintained no steeper than 1 horizontal to 1 vertical. Flatter side slopes may be necessary in areas with saturated or loose granular fills.

6.3 Temporary Excavation Support Systems

Temporary protection systems are required to support embankment fills and native clayey silt to silty clay during rehabilitation. Assuming the depth of excavation is required to the underside of the pile cap, which is at about Elevation 208 m, an excavation in the order of 7 m deep below existing roadway surface will be required. The temporary support systems could consist of either driven steel sheet piling or soldier piles and lagging where the H-piles would be driven to a suitable depth and horizontal lagging installed as the excavation proceeds. Support to the system could be in the form of struts and walers or rakers and anchors.

Where necessary, adequate support must be provided for structures such as existing foundations or utilities which may be present adjacent to the excavations. In accordance with the Canadian Foundation Engineering Manual (2006) Subsection 26.16, structural loads may be carried by either direct underpinning of the foundations or by providing additional support of the excavation face.

6.3.1 Lateral Earth Pressures

The temporary excavation support system should be designed and constructed in accordance with Ontario Provincial Standard Specification (OPSS) 539 (Temporary Protection Systems). The lateral movement of the temporary shoring system should meet Performance Level 2 as specified in OPSS 539. The contractor is responsible for the complete detailed design of the protection system.

The design of braced soldier pile and lagging walls should be based on a rectangular earth pressure distribution using the design parameters given below. Where the support to the wall is provided by anchors or rakers, the wall design should be based on a triangular earth pressure distribution using the design parameters given below. The raker/anchor support must be designed to accommodate the loads applied from pressures and surcharge pressures from area, line or point loads as well as the impact of sloping ground behind the system. Passive toe restraint to the soldier piles may be determined using a triangular pressure distribution acting over an equivalent width equal to three times the pile socket diameter.



The unfactored triangular earth pressure distribution (p in kN/m^2 ; increasing with depth), can be calculated as follows:

$$p = K_a (\gamma H + q)$$

where H = the depth of the excavation at any point (m)

$$K_a = \text{active coefficient of earth pressure}$$

$$\gamma = \text{soil unit weight (kN/m}^3\text{)}$$

$$q = \text{surcharge for traffic and other loading (kN/m}^2\text{)}$$

For a braced excavation in granular fill and native cohesionless soils, the unfactored rectangular earth pressure distribution (p in kN/m^2 ; constant with depth), can be calculated as follows (CFEW 2006; NAVFAC 1982):

$$p = 0.65 K_a (\gamma H + q)$$

where H = the total depth of the excavation (m)

$$K_a = \text{active coefficient of earth pressure}$$

$$\gamma = \text{soil unit weight (kN/m}^3\text{)}$$

$$q = \text{surcharge for traffic and other loading (kN/m}^2\text{)}$$

For a braced excavation in cohesive soil, the unfactored rectangular earth pressure distribution (p in kN/m^2 ; varying with depth), can be calculated as follows (CFEW 2006; NAVFAC 1982):

$$p = 0 \text{ at ground surface increasing linearly to a depth of } 0.25 H_T \text{ to:}$$

$$p = \gamma H_T - 4mS_u \text{ at } 0.25 H_T \text{ and from } 0.25 H_T \text{ to } H_T \text{ below ground surface}$$

where H_T = the total depth of the excavation (m)

$$\gamma = \text{soil unit weight (kN/m}^3\text{)}$$

$$q = \text{surcharge for traffic and other loading (kN/m}^2\text{)}$$

$$m = \begin{cases} 0.4 & \text{if an extensive soft clay layer underlies the excavation} \\ 1.0 & \text{if a more resistant layer is present at the excavation base} \end{cases}$$

$$S_u = \text{undrained shear strength (kN/m}^2\text{)}.$$

The support systems may be designed using the following parameters:

SOIL TYPE	COEFFICIENT OF EARTH PRESSURE			INTERNAL ANGLE OF FRICTION (ϕ , degrees)	UNIT WEIGHT (γ , kN/m^3)	UNDRAINED SHEAR STRENGTH (S_u , kPa)
	Active, K_a	At Rest, K_o	Passive, K_p			
Existing Granular Fill	0.33	0.50	3.0	30	20	-
Clayey Silt/Silty Clay	0.37	0.55	2.7	27	17	30
Silt to Silty Sand	0.36	0.53	2.8	28	18	-



The earth pressure coefficients noted above are based on a horizontal surface adjacent to the excavation. If sloped surfaces are present, the coefficients should be adjusted accordingly. Further, hydrostatic pressures must be added to the earth pressure where groundwater is not fully covered to below the excavation level.

It should be noted that the pressure distributions given above are the minimum for the ultimate stress condition. A stiffer design may be required than predicted by these distributions in order to maintain displacements within an acceptable range.

7.0 CLOSURE

This report was prepared by Mr. André Bom, P.Eng. Mr. Jorge M. A. Costa, P.Eng., Golder's Designated MTO Contact for this project and a Principal with Golder, reviewed the technical aspects of and conducted an independent quality control review of the report



Report Signature Page

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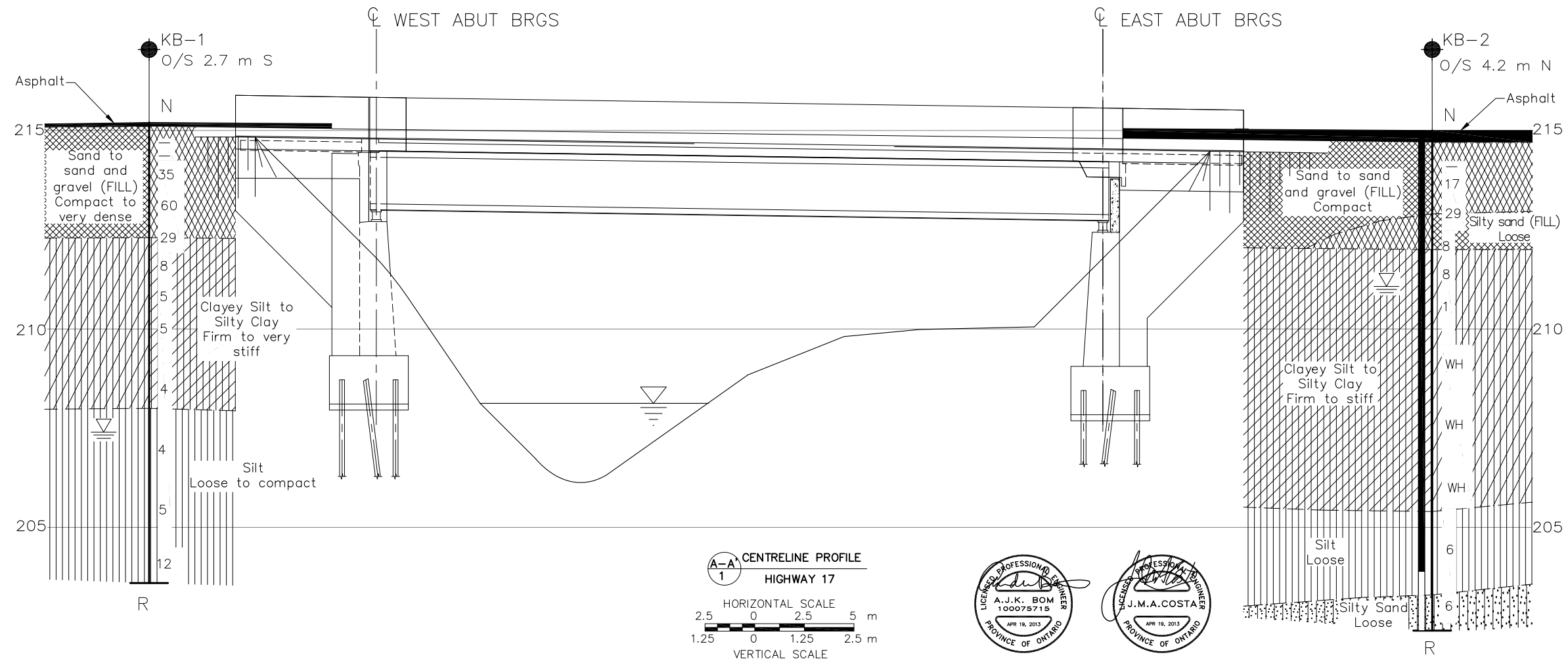
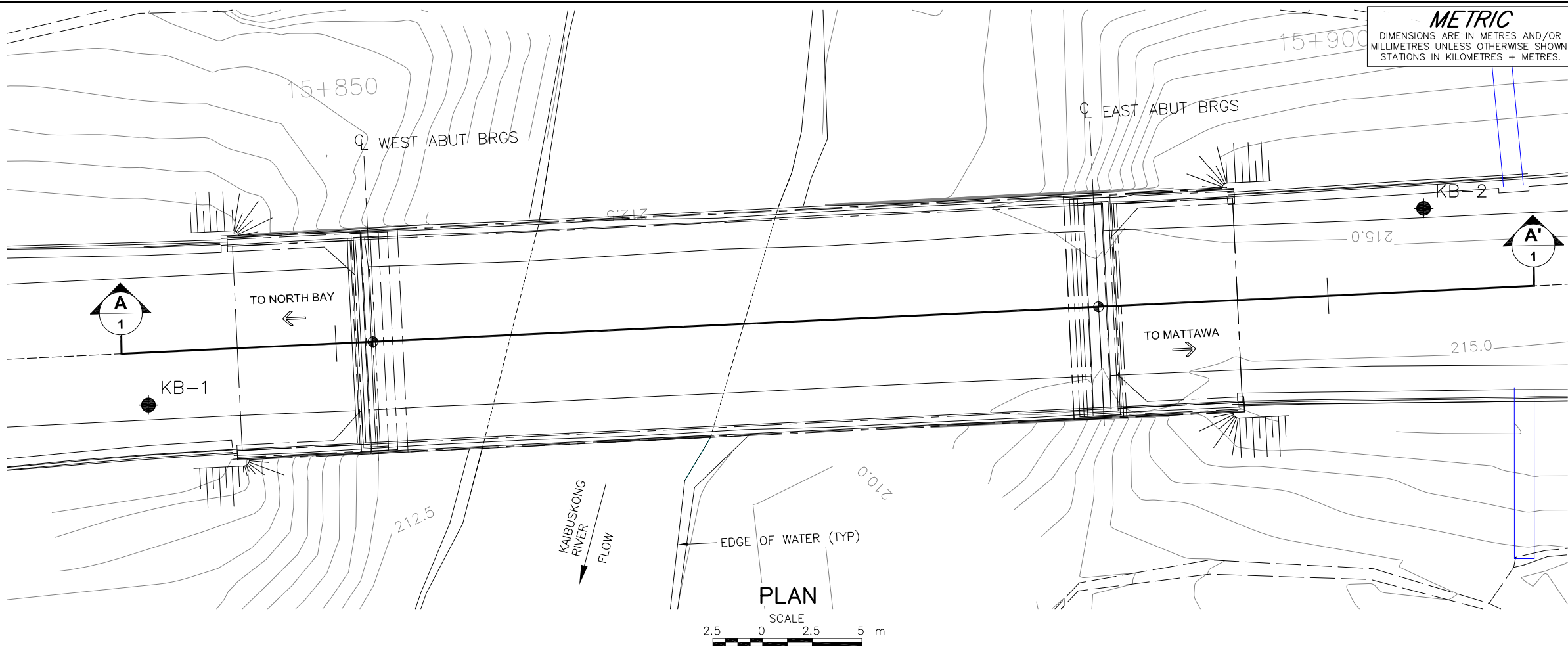
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- Geology of Ontario, 1991. Ontario Geological Society, Special Volume 4, Part 2. Eds. P.C. Thurston, H.R. Williams, R.H. Sutcliffe and G.M. Stott. Ministry of Northern Development and Mines, Ontario.
- NAVFAC Design Manual, DM-7.2. Soil Mechanics, Foundation and Earth Structures. U.S. Navy, 1982, Alexandria, Virginia.
- Northern Ontario Engineering Geology Terrain Study, Ontario Geological Society, Map 5044.
- ASTM International:
- | | |
|------------|---|
| ASTM D1586 | Standard Test Method for Standard Penetration Test (SPT) and Split-Barrel Sampling of Soils |
| ASTM D2573 | Standard Test Method for Field Vane Shear Test in Cohesive Soil |
- Ontario Occupational Health and Safety Act:
- Ontario Regulation 213/91 Construction Projects as amended by O. Reg. 443/09
- Ontario Provincial Standard Specification:
- | | |
|----------|---|
| OPSS 539 | Construction Specification for Temporary Protection Systems |
|----------|---|
- Ontario Water Resources Act:
- Ontario Regulation 372/97 Amendment to Ontario Regulation 903, Wells (as Amended)



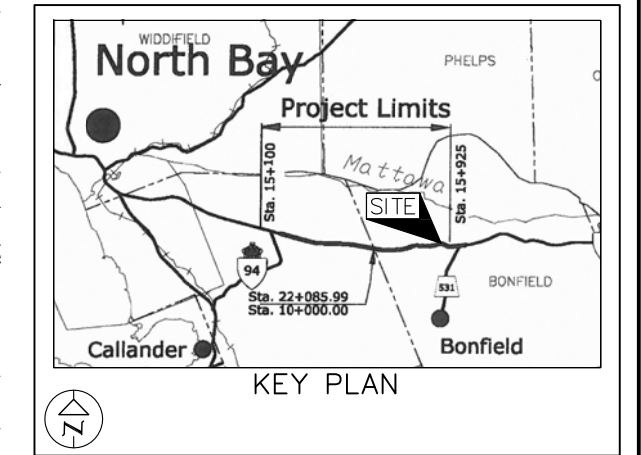
METRIC
DIMENSIONS ARE IN METRES AND/OR
MILLIMETRES UNLESS OTHERWISE SHOWN.
STATIONS IN KILOMETRES + METRES.

CONT No.
WP No. 5628-04-01

HIGHWAY 17
KAIBUSKONG RIVER BRIDGE
BOREHOLE LOCATIONS AND
SOIL STRATA

Golder Associates

Golder Associates Ltd.
SUDBURY, ONTARIO, CANADA



LEGEND

- Borehole - Current Investigation
- Seal
- Piezometer
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- WL upon completion of drilling
- R Refusal

BOREHOLE CO-ORDINATES			
No.	ELEVATION	NORTHING	EASTING
KB-1	215.2	5125612.6	332319.9
KB-2	215.0	5125622.5	332384.1

NOTES

This drawing is for subsurface information only. The proposed structure details/works are shown for illustration purposes only and may not be consistent with the final design configuration as shown elsewhere in the Contracts Documents.

The boundaries between soil strata have been established only at borehole locations. Between boreholes the boundaries are assumed from geological evidence.

The complete Foundation Investigation and Design Report for this project and other related documents may be examined at the Materials Engineering and Research Office, Downsview. Information contained in this report and related documents is specifically excluded in accordance with Section GC 2.01 of OPS General Conditions.

REFERENCE

Base plans provided in digital format by MMM, drawing file nos. BASE_ALL.dwg and Alignment.dwg received NOV 10, 2011. Keyplan received DEC 7, 2011. Contours received JAN 4, 2012. General arrangement received FEB 17, 2012



NO.	DATE	BY	REVISION
Geocres No. 31L-159			
HWY. 17	PROJECT NO. 10-1191-0041		DIST.
SUBM'D. MT	CHKD. AB	DATE: APR 2013	SITE: 43-113
DRAWN: JJJ	CHKD. JMAG	APPD.	DWG. 1



APPENDIX A

Record of Boreholes



LIST OF ABBREVIATIONS

The abbreviations commonly employed on Records of Boreholes, on figures and in the text of the report are as follows:

I. SAMPLE TYPE

AS	Auger sample
BS	Block sample
CS	Chunk sample
SS	Split-spoon
DS	Denison type sample
FS	Foil sample
RC	Rock core
SC	Soil core
ST	Slotted tube
TO	Thin-walled, open
TP	Thin-walled, piston
WS	Wash sample

II. PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:

The number of blows by a 63.5 kg. (140 lb.) hammer dropped 760 mm (30 in.) required to drive a 50 mm (2 in.) drive open sampler for a distance of 300 mm (12 in.)

Dynamic Cone Penetration Resistance; N_d :

The number of blows by a 63.5 kg (140 lb.) hammer dropped 760 mm (30 in.) to drive uncased a 50 mm (2 in.) diameter, 60° cone attached to "A" size drill rods for a distance of 300 mm (12 in.).

PH:	Sampler advanced by hydraulic pressure
PM:	Sampler advanced by manual pressure
WH:	Sampler advanced by static weight of hammer
WR:	Sampler advanced by weight of sampler and rod

Piezo-Cone Penetration Test (CPT)

A electronic cone penetrometer with a 60° conical tip and a project end area of 10 cm² pushed through ground at a penetration rate of 2 cm/s. Measurements of tip resistance (Q_t), porewater pressure (PWP) and friction along a sleeve are recorded electronically at 25 mm penetration intervals.

III. SOIL DESCRIPTION

(a) Cohesionless Soils

Density Index	N
Relative Density	Blows/300 mm or Blows/ft
Very loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very dense	over 50

(b) Cohesive Soils Consistency

	C_u, S_u	
	kPa	psf
Very soft	0 to 12	0 to 250
Soft	12 to 25	250 to 500
Firm	25 to 50	500 to 1,000
Stiff	50 to 100	1,000 to 2,000
Very stiff	100 to 200	2,000 to 4,000
Hard	over 200	over 4,000

IV. SOIL TESTS

w	water content
w_p	plastic limit
w_l	liquid limit
C	consolidation (oedometer) test
CHEM	chemical analysis (refer to text)
CID	consolidated isotropically drained triaxial test ¹
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement ¹
D_R	relative density (specific gravity, G_s)
DS	direct shear test
M	sieve analysis for particle size
MH	combined sieve and hydrometer (H) analysis
MPC	Modified Proctor compaction test
SPC	Standard Proctor compaction test
OC	organic content test
SO ₄	concentration of water-soluble sulphates
UC	unconfined compression test
UU	unconsolidated undrained triaxial test
V	field vane (LV-laboratory vane test)
γ	unit weight

Note: 1 Tests which are anisotropically consolidated prior to shear are shown as CAD, CAU.

V. MINOR SOIL CONSTITUENTS

Percent by Weight	Modifier	Example
0 to 5	Trace	Trace sand
5 to 12	Trace to Some (or Little)	Trace to some sand
12 to 20	Some	Some sand
20 to 30	(ey) or (y)	Sandy
over 30	And (cohesionless) or With (cohesive)	Sand and Gravel Silty Clay with sand / Clayey Silt with sand



LIST OF SYMBOLS

Unless otherwise stated, the symbols employed in the report are as follows:

I. GENERAL

π	3.1416
$\ln x$,	natural logarithm of x
\log_{10}	x or log x, logarithm of x to base 10
g	acceleration due to gravity
t	time

II. STRESS AND STRAIN

γ	shear strain
Δ	change in, e.g. in stress: $\Delta \sigma$
ε	linear strain
ε_v	volumetric strain
η	coefficient of viscosity
ν	Poisson's ratio
σ	total stress
σ'	effective stress ($\sigma' = \sigma - u$)
σ'_{vo}	initial effective overburden stress
$\sigma_1, \sigma_2, \sigma_3$	principal stress (major, intermediate, minor)
σ_{oct}	mean stress or octahedral stress $= (\sigma_1 + \sigma_2 + \sigma_3)/3$
τ	shear stress
u	porewater pressure
E	modulus of deformation
G	shear modulus of deformation
K	bulk modulus of compressibility

III. SOIL PROPERTIES

(a) Index Properties

$\rho(\gamma)$	bulk density (bulk unit weight)*
$\rho_d(\gamma_d)$	dry density (dry unit weight)
$\rho_w(\gamma_w)$	density (unit weight) of water
$\rho_s(\gamma_s)$	density (unit weight) of solid particles
γ'	unit weight of submerged soil ($\gamma' = \gamma - \gamma_w$)
D_R	relative density (specific gravity) of solid particles ($D_R = \rho_s / \rho_w$) (formerly G_s)
e	void ratio
n	porosity
S	degree of saturation

(a) Index Properties (continued)

w	water content
w_l or LL	liquid limit
w_p or PL	plastic limit
I_p or PI	plasticity index = $(w_l - w_p)$
w_s	shrinkage limit
I_L	liquidity index = $(w - w_p) / I_p$
I_C	consistency index = $(w_l - w) / I_p$
e_{max}	void ratio in loosest state
e_{min}	void ratio in densest state
I_D	density index = $(e_{max} - e) / (e_{max} - e_{min})$ (formerly relative density)

(b) Hydraulic Properties

h	hydraulic head or potential
q	rate of flow
v	velocity of flow
i	hydraulic gradient
k	hydraulic conductivity (coefficient of permeability)
j	seepage force per unit volume

(c) Consolidation (one-dimensional)

C_c	compression index (normally consolidated range)
C_r	recompression index (over-consolidated range)
C_s	swelling index
C_α	secondary compression index
m_v	coefficient of volume change
c_v	coefficient of consolidation (vertical direction)
c_h	coefficient of consolidation (horizontal direction)
T_v	time factor (vertical direction)
U	degree of consolidation
σ'_p	pre-consolidation stress
OCR	over-consolidation ratio = σ'_p / σ'_{vo}

(d) Shear Strength

τ_p, τ_r	peak and residual shear strength
ϕ'	effective angle of internal friction
δ	angle of interface friction
μ	coefficient of friction = $\tan \delta$
c'	effective cohesion
c_u, s_u	undrained shear strength ($\phi = 0$ analysis)
p	mean total stress $(\sigma_1 + \sigma_3)/2$
p'	mean effective stress $(\sigma'_1 + \sigma'_3)/2$
q	$(\sigma_1 - \sigma_3)/2$ or $(\sigma'_1 - \sigma'_3)/2$
q_u	compressive strength $(\sigma_1 - \sigma_3)$
S_t	sensitivity

* Density symbol is ρ . Unit weight symbol is γ where $\gamma = \rho g$ (i.e. mass density multiplied by acceleration due to gravity)

Notes: 1
2

$$\tau = c' + \sigma' \tan \phi'$$

$$\text{shear strength} = (\text{compressive strength})/2$$

PROJECT 10-1191-0041			RECORD OF BOREHOLE No KB-1			1 OF 1 METRIC															
W.P. 5628-04-01			LOCATION N 5125612.6; E 332319.9			ORIGINATED BY LG															
DIST _____ HWY 17			BOREHOLE TYPE 108mm ID Continuous Flight Hollow Stem Augers			COMPILED BY MT															
DATUM Geodetic			DATE July 5, 2011			CHECKED BY AB															
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC NATURAL LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)						
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa					WATER CONTENT (%)			γ			GR SA SI CL		
215.2	GROUND SURFACE							20 40 60 80 100	20 40 60 80 100	20 40 60	W _p W W _L										
0.0	ASPHALT (150 mm)		1	AS	-		215														
	Sand to sand and gravel, trace to some silt (FILL) Compact to very dense Brown Moist		2	AS	-																
			3	SS	35		214													1 89 (10)	
	Gravel 0.6 m thick at 1.5 m depth.		4	SS	60		213														
			5	SS	29																
212.3	CLAYEY SILT, silt seams/layers Firm to very stiff Brown to grey Moist to wet		6	SS	8		212														
2.9			7	SS	5		211														
			8	SS	5		210														
			9	SS	4		209														
208.0	SILT, trace to some clay Loose to compact Grey Wet		10	SS	4		208														
7.2																					
			11	SS	5		206														
			12	SS	12		204														
203.6	END OF BOREHOLE AUGER REFUSAL																				
11.6	Note: 1. Water level at a depth of 7.8 m below ground surface (Elev. 207.4 m) upon completion of drilling.																				

SUD-MTO 001 1011910041.GPJ GAL-MISS.GDT 11/03/13 DATA INPUT:

PROJECT		10-1191-0041		RECORD OF BOREHOLE No KB-2		1 OF 1 METRIC								
W.P.		5628-04-01		LOCATION		N 5125622.5; E 332384.1								
DIST		HWY 17		BOREHOLE TYPE		108mm ID Continuous Flight Hollow Stem Augers								
DATUM		Geodetic		DATE		July 6, 2011								
				ORIGINATED BY		MT								
				COMPILED BY		MT								
				CHECKED BY		AB								
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT		UNIT WEIGHT		REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV	DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa		WATER CONTENT (%)			
215.0	0.0	GROUND SURFACE							20 40 60 80 100	W _p W W _L	20 40 60	γ	GR SA SI CL	
214.7	0.3	ASPHALT (280 mm)							20 40 60 80 100			kN/m ³		
		Sand to sand and gravel, trace to some silt (FILL) Compact Brown Moist		1	AS	-		214						
				2	SS	17								
				3	SS	29		213						
212.9	2.1	Silty sand, trace gravel (FILL) Loose Brown Moist		4	SS	8								
212.0	3.0	CLAYEY SILT to SILTY CLAY, silt seams/layers Firm to stiff Brown to grey Moist to wet		5	SS	8		212						
				6	SS	1		211						
								210						
				7	SS	WH		209						
								208						
				8	SS	WH		207						
								206						
				9	SS	WH		205						
								204						
205.4	9.6	SILT, trace to some clay, sand, gravel Loose Grey Wet		10	SS	6								
								203						
203.3	11.7	Silty SAND, trace to some clay Loose Grey Wet		11a	SS	6								
				11b										
202.4	12.6	END OF BOREHOLE AUGER REFUSAL												
		Notes: 1. Water level at a depth of 3.8 m below ground surface (Elev. 211.2 m) upon completion of drilling. 2. Piezometer blocked at ground surface on October 23, 2012 - unable to measure depth to water level.												

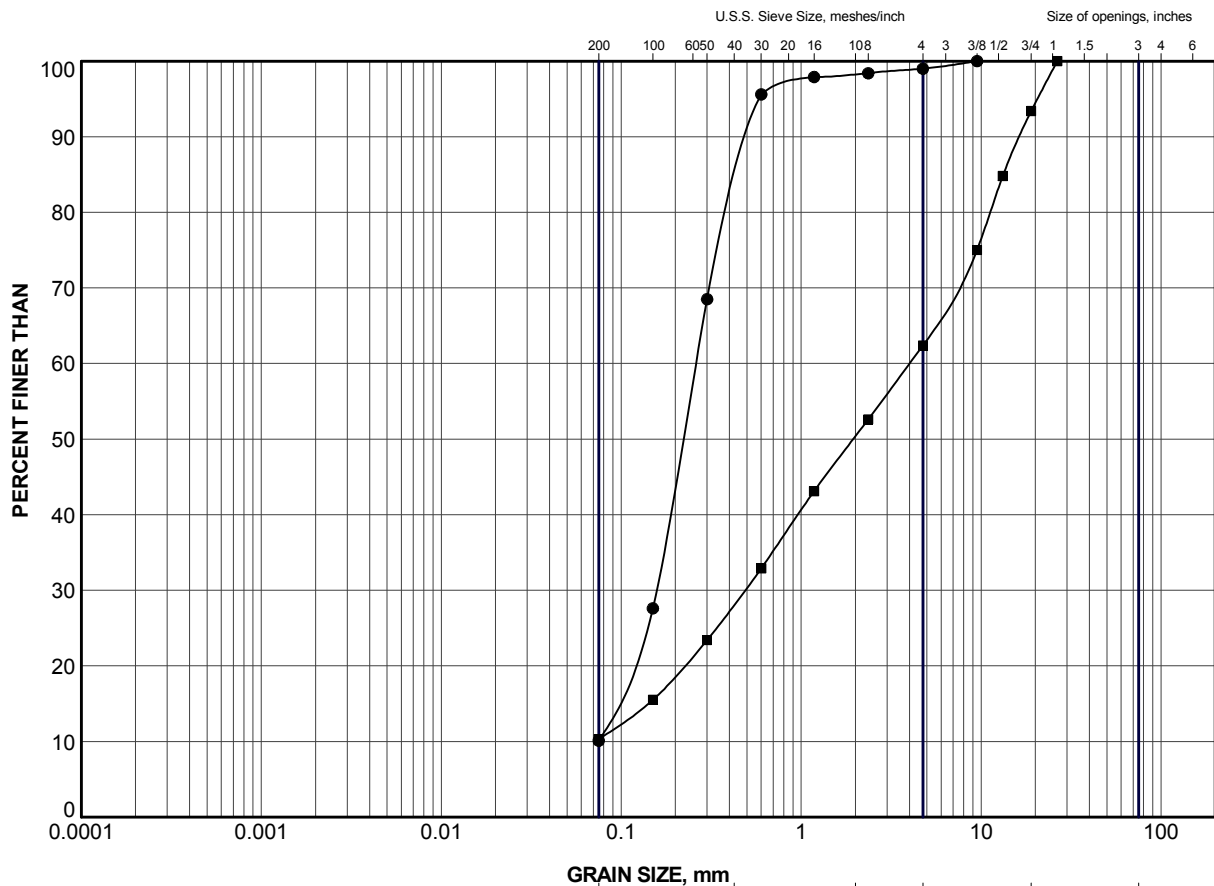
SUD-MTO 001 1011910041.GPJ GAL-MISS.GDT 15/04/13 DATA INPUT:

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



APPENDIX B


Laboratory Test Results



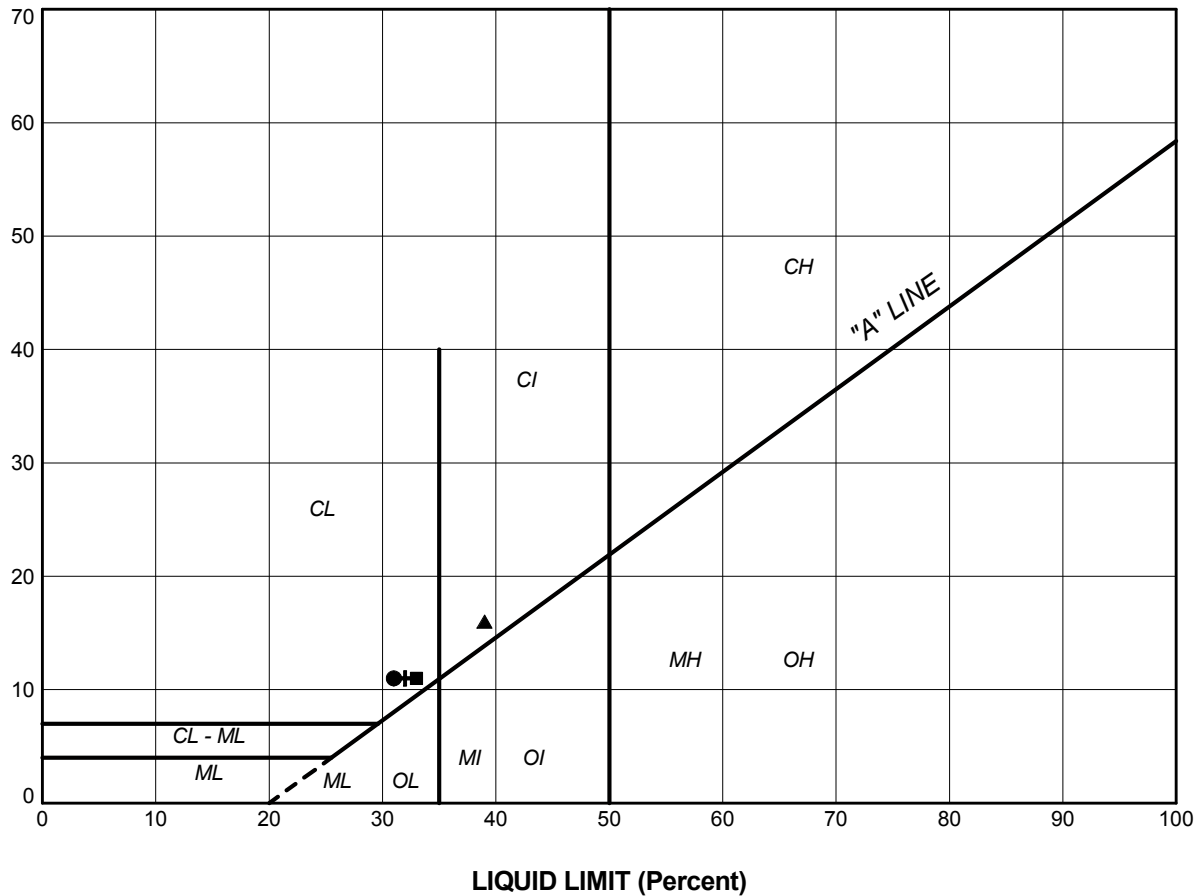
GRAIN SIZE, mm						
CLAY AND SILT	fine	medium	coarse	fine	coarse	Cobble Size
	SAND SIZE			GRAVEL SIZE		

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	KB-1	3	214.1
■	KB-2	3	213.2

PROJECT					
HIGHWAY 17 KAIBUSKONG RIVER BRIDGE					
TITLE					
GRAIN SIZE DISTRIBUTION SAND AND GRAVEL AND SAND (FILL)					
PROJECT No.		10-1191-0041		FILE No. 1011910041.GPJ	
DRAWN	JJL	Mar 2013	SCALE	N/A	REV.
CHECK	AB	Mar 2013			
APPR	JMAC	Mar 2013			
 Golder Associates SUDBURY, ONTARIO			FIGURE B1		

PLASTICITY INDEX (Percent)



SOIL TYPE
 C = Clay
 M = Silt
 O = Organic

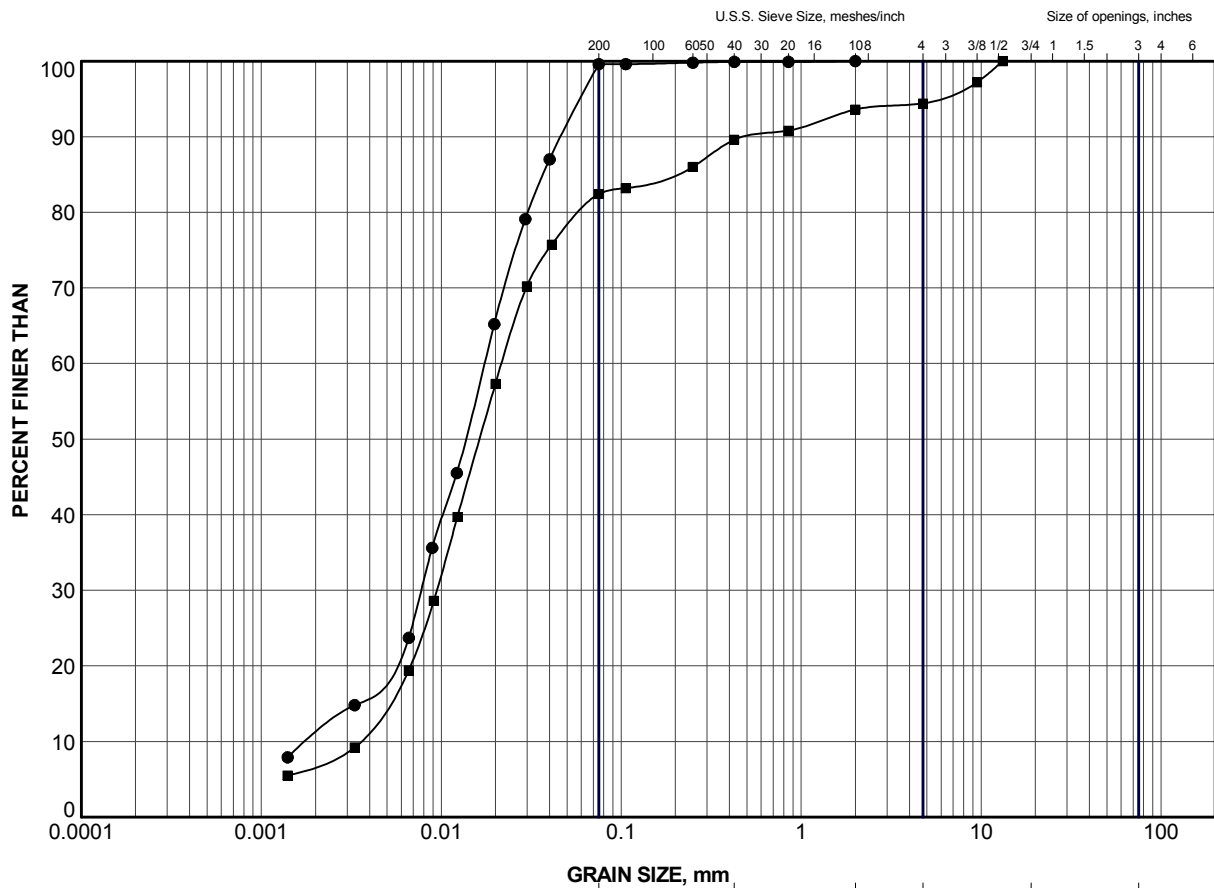
PLASTICITY
 L = Low
 I = Intermediate
 H = High

LEGEND

SYMBOL	BOREHOLE	SAMPLE	LL(%)	PL(%)	PI
●	KB-1	6	31.0	20.0	11.0
■	KB-1	8	33.0	22.0	11.0
▲	KB-2	5	39.0	23.0	16.0
+	KB-2	7	32.0	21.0	11.0

PROJECT					
HIGHWAY 17 KAIBUSKONG RIVER BRIDGE					
TITLE					
PLASTICITY CHART CLAYEY SILT TO SILTY CLAY					
PROJECT No.		10-1191-0041		FILE No.	
				1011910041.GPJ	
DRAWN	JJL	Mar 2013	SCALE	N/A	REV.
CHECK	AB	Mar 2013			
APPR	JMAC	Mar 2013			
			FIGURE B2		






GRAIN SIZE, mm						
CLAY AND SILT	fine	medium	coarse	fine	coarse	Cobble Size
	SAND SIZE			GRAVEL SIZE		

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	KB-1	10	207.3
■	KB-2	11a	203.4

PROJECT					
HIGHWAY 17 KAIBUSKONG RIVER BRIDGE					
TITLE					
GRAIN SIZE DISTRIBUTION SILT					
PROJECT No.		10-1191-0041		FILE No. 1011910041.GPJ	
DRAWN	JJL	Mar 2013	SCALE	N/A	REV.
CHECK	AB	Mar 2013			
APPR	JMAC	Mar 2013			
 Golder Associates SUDBURY, ONTARIO			FIGURE B3		

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