



July 2015

## PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT

Highway 4 Bayfield River Bridge Replacement  
Site Number 12-195  
Highways 401, 4 and 21  
Structural Replacements  
GWP 3059-11-00, Assignment No. 3 (3011-E-0048)  
Ministry of Transportation, Ontario – West Region

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REPORT



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**PART A**

**PRELIMINARY FOUNDATION INVESTIGATION REPORT**

**HIGHWAY 4 BAYFIELD RIVER BRIDGE REPLACEMENT  
SITE NUMBER 12-195**

**HIGHWAYS 401, 4 AND 21 STRUCTURAL REPLACEMENTS  
GWP 3059-11-00, ASSIGNMENT No. 3 (3011-E-0048)**

**MINISTRY OF TRANSPORTATION, ONTARIO - WEST REGION**





## **1.0 INTRODUCTION**

Golder Associates Ltd. (Golder Associates) has been retained by Dillon Consulting Limited (Dillon) on behalf of the Ministry of Transportation, Ontario (MTO) to carry out foundation investigations as part of the preliminary design and 30% detailed design work for GWPs 3030-11-00, 3054-11-00, 3053-11-00, 3070-11-00, 3059-11-00, and 3055-11-00. The project involves the preliminary design and 30% detailed design for ten (10) bridges and two (2) culverts, including improvements at five (5) Highway 401 interchanges.

This report addresses the replacement of the Highway 4 bridge over the Bayfield River (Site 12-195) for GWP 3059-11-00.

The purpose of the foundation investigation is to explore the subsurface conditions at the location of the proposed bridge replacement by drilling boreholes and carrying out in situ testing and laboratory testing on selected samples. The terms of reference for the scope of work are outlined in the MTO's Request for Proposal and in Golder Associates' proposal P2-1132-0076-P01 dated September 10, 2012. The work was carried out in accordance with our Quality Control Plan for Foundations Engineering dated November 2012.



## **2.0 SITE DESCRIPTION**

### **2.1 General**

The Highway 4 bridge over the Bayfield River is located at the southern limit of the Town of Clinton, Ontario. The bridge site is located at the junction of the boundaries of the Municipalities of Central Huron, Huron East and Bluewater. The location of the project is shown on the Key Plan, Figure 1.

For the purposes of this report, Highway 4 and the Bayfield River in the vicinity of the site are assumed to be oriented in north-south and east-west directions, respectively. This section of Highway 4 is currently a two lane, non-divided highway. The highway surface is at approximately elevation 272 metres at the bridge location. The existing bridge was constructed in 1932 and consists of a two-span, concrete, rigid frame structure with Tee type girders. The area immediately surrounding the bridge consists of the heavily treed river valley and flood plain. Adjacent to the site and river valley is parkland to the southeast and residential properties to the north and west. It is anticipated that the existing bridge will be demolished and replaced with a new bridge at the same location.

### **2.2 Site Geology**

This project lies within the physiographic region of southern Ontario known as the Huron Slope.<sup>1</sup> Geological mapping indicates that the surficial materials at the site consist of alluvial deposits of gravel, sand and silt, glaciofluvial outwash sand with minor gravel and glaciolacustrine deep water deposits of sand, silt and clay.<sup>2</sup>

The rock formation in the area of the site is described as medium brown, microcrystalline limestone of the Dundee Formation which belongs to the Hamilton Group of Middle Devonian Age.<sup>3</sup> The bedrock surface is estimated, based on the available mapping, to be between about elevations 244 and 250 metres<sup>4</sup> or some 22 to 28 metres below ground surface at the site.

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<sup>1</sup> Chapman, L.J. and Putnam, D.F., 1984: The Physiography of Southern Ontario, Third Edition. Ontario Geological Survey, Special Volume 2.

<sup>2</sup> Cooper, A.J. and Fitzgerald, W.D., 1977: Quaternary Geology of the Goderich area, Southern Ontario; Ontario Geological Survey Prelim. Map P.1232, Geol. Ser., scale 1:50,000. Geology 1975, 1976.

<sup>3</sup> Sanford, B.V., 1969: Geology, Toronto-Windsor Area, Ontario. Geological Survey of Canada, Map 1263A, scale 1:250,000.

<sup>4</sup> Cooper, A.J., 1978: Bedrock Topography of the Goderich-Seaforth Area, Southern Ontario; Ontario Geological Survey Prelim. Map P.1974, Bedrock Topography Ser., Scale 1:50 000. Compilation 1977, 1978.



### 3.0 INVESTIGATION PROCEDURES

The field work for the investigation was carried out between June 16 and 24, 2014 during which time four boreholes were drilled at the approximate locations shown on the Borehole Location Plan, Drawing 1. The table below summarizes the borehole locations, ground surface elevations at the borehole locations and borehole depths.

Borehole	Location (m)		Ground Surface Elevation (m)	Borehole Depth (m)
	Northing	Easting		
801	4 829 918	382 782	271.87	14.2
802	4 829 935	382 776	271.91	22.9
803	4 829 990	382 762	272.36	19.1
804	4 830 016	382 760	273.57	6.6

The investigation was carried out using truck mounted drilling equipment supplied and operated by a specialist drilling contractor. In the boreholes, samples of the overburden were obtained at generally 0.75 or 1.5 metre intervals of depth using 50 millimetre outside diameter split spoon sampling equipment in accordance with the standard penetration test (SPT) procedures of ASTM D1586. According to ASTM D1586, the SPT resistance, or N value, is defined as the number of blows required by a 63.5 kilogram hammer dropped from a height of 760 millimetres to drive a split-spoon sampler a distance of 300 millimetres after an initial 150 millimetres of penetration. In cases where it was not possible to achieve a full 450 millimetres of drive, a penetration resistance representing the number of blows to drive the sampler is recorded on the Record of Borehole. The penetration resistance obtained in the first 150 millimetres is normally neglected unless the sampler could only be driven 150 millimetres or less, in which case SPT testing was terminated after 100 blows. The results of the SPT testing as presented on the Record of Borehole sheets and in Section 4 are unmodified (not standardized for hammer efficiency, borehole diameter, rod length, etc.). The samplers used in the investigation limit the maximum particle size that can be sampled and tested to about 40 millimetres. Therefore, particles that may exist within the soils that are larger than this dimension will not be sampled or represented in the grain size distributions. Larger particle sizes, including cobbles and boulders, are known to be present in the fill materials and glacial till deposits as discussed in the text of this report.

The boreholes were terminated between about 6.6 and 22.9 metres below the existing pavement surface. Groundwater conditions in the boreholes were observed throughout the drilling operations and piezometers were installed in borehole 803 as indicated on the corresponding Record of Borehole sheets. The boreholes were backfilled in accordance with current MTO procedures and Ontario Regulation 903 (as amended).

The field work was monitored on a full-time basis by experienced Golder Associates staff members who also located the boreholes in the field, monitored the drilling, sampling and in situ testing operations and logged the boreholes. The samples were identified in the field, placed in labelled containers and transported to our London laboratory for further examination and testing. Index and classification tests, consisting of water content



determinations, Atterberg limits determinations and grain size distribution analyses, were carried out on selected samples. The results of the testing are shown on the Record of Borehole sheets and in Appendix A.

The locations of the boreholes are shown on the Record of Borehole sheets and on Drawing 1, attached.

## **4.0 SUBSURFACE CONDITIONS**

### **4.1 Site Stratigraphy**

The detailed subsurface soil and groundwater conditions encountered in the boreholes, together with the results of the in situ testing and the laboratory testing carried out on selected samples, are provided on the attached Record of Borehole sheets following the text of this report and in Appendix A. The stratigraphic boundaries shown on the Record of Borehole sheets are inferred from non-continuous samples and observations of drilling resistance and, therefore, may represent transitions between soil and rock types rather than exact planes of geological change. Further, the subsurface conditions will vary between and beyond the borehole locations.

The boreholes drilled at the site generally encountered the existing pavement structure and fill materials, topsoil, overlying layers of sand and gravel, glacial till and limestone bedrock. A layer of buried topsoil was encountered beneath the fill in borehole 802.

The locations and elevations of the boreholes, together with the interpreted stratigraphic profiles, are shown on Drawing 1. Detailed descriptions of the subsurface conditions encountered in the boreholes are provided on the Record of Borehole sheets and are summarized in the following sections.

#### **4.1.1 Pavement Structure**

Each of the boreholes was advanced through the Highway 4 pavements. Approximately 150 to 270 millimetres of asphaltic concrete was encountered at the pavement surface. Beneath the asphalt, about 190 to 640 millimetres of granular base material was encountered. In boreholes 801 and 802, the granular base was underlain by 420 and 640 millimetres, respectively, of granular subbase materials. The loose granular subbase had an N value, as determined by the standard penetration testing, of 7 blows per 0.3 metres.

#### **4.1.2 Fill and Topsoil**

Between 5.0 and 6.0 metres of fill materials were encountered beneath the pavement structure in boreholes 801, 802 and 803. The fill materials varied in composition from silty sand and sandy silt to clayey silt. Layers and pockets of topsoil were encountered within the fill materials in borehole 801. Measured N values in the fill materials ranged from 1 to 13 blows per 0.3 metres. Samples of the fine grained granular fill materials had water contents of 20 to 29 per cent. Samples of the clayey silt fill materials had water contents between 18 and 45 per cent. A sample of the clayey silt fill had plastic and liquid limits of 16 and 27 per cent, respectively, and a plasticity index of 11 per cent, indicating a clay of low plasticity. Grain size distribution curves for samples of the fill materials are shown on Figure A-1. The Atterberg limits data are shown on Figure A-6.



A 0.3 metre thick layer of buried topsoil was encountered beneath the fill materials in borehole 802. The buried topsoil had a water content of about 72 per cent.

#### **4.1.3 Sand and Gravel**

Compact to dense sand and gravel was encountered beneath the fill materials in borehole 801, the buried topsoil in borehole 802 and the clayey silt in borehole 803. The sand and gravel was encountered between about elevation 265.2 and 265.5 metres. The sand and gravel was 1.5 to 3.1 metres thick at the borehole locations. Measured N values in the sand and gravel ranged from 13 to 48 blows per 0.3 metres. Samples of the sand and gravel had water contents ranging from 14 to 24 per cent. Grain size distribution curves for samples of the sand and gravel are shown on Figure A-2.

Cobbles and boulders should be expected in the sand and gravel.

#### **4.1.4 Clayey Silt Glacial Till**

Firm to hard clayey silt glacial till was encountered in boreholes 801 and 803 beneath the sand and gravel, in borehole 802 beneath a layer of sandy silt and in borehole 804 beneath the pavement structure and a layer of clayey silt. In boreholes 801, 802 and 803 the clayey silt till was encountered between elevations 262.5 and 263.0 metres and in borehole 804, the clayey silt till layers were encountered at elevations 269.9 and 272.7 metres. Where fully penetrated in boreholes 802 and 803, the clayey silt till layers were 8.8 and 3.2 metres thick, respectively. The upper layer of clayey silt till in borehole 804 was 1.7 metres thick. Boreholes 801 and 804 were terminated in the clayey silt till layers after exploring it for 5.3 and 2.9 metres, respectively. While not specifically encountered in the boreholes, cobbles and boulders should be anticipated in the glacial till deposits.

Measured N values in the clayey silt till were between 7 and greater than 100 blows per 0.3 metres. Water contents of samples of the clayey silt till ranged from 10 to 13 per cent. The clayey silt till had liquid limits ranging from 22 to 25 per cent, plastic limits ranging from 13 to 14 per cent and plasticity indices ranging from 9 to 11 per cent indicating a clay of low plasticity. Grain size distribution curves for samples of the clayey silt till are shown on Figure A-3 and results of the Atterberg limits determinations are shown on Figure A-6.

#### **4.1.5 Clayey Silt**

A 0.3 metre thick layer of stiff clayey silt was encountered beneath the fill materials in borehole 803 at elevation 265.8 metres and a 1.0 metre thick layer of hard clayey silt was encountered in borehole 804 within clayey silt till at elevation 271.0 metres. The clayey silt in borehole 803 had an N value of 15 blows per 0.3 metres and the corresponding sample had a water content of 31 per cent. The clayey silt in borehole 804 had an N value of 46 blows per 0.3 metres with a water content of 12 per cent. A grain size distribution curve for a sample of the clayey silt is provided on Figure A-4.



#### 4.1.6 Silty Clay

A 2.3 metre thick layer of silty clay was encountered at elevation 259.3 metres in borehole 803 beneath the clayey silt till. The silty clay had N values of 36 and 62 blows per 0.3 metres.

#### 4.1.7 Sandy Silt

Layers of sandy silt were encountered in borehole 802 beneath the sand and gravel at elevation 263.7 metres and in borehole 803 beneath the silty clay at elevation 256.9 metres. In borehole 802, the sandy silt was 1.2 metres thick and was inferred to contain cobbles as based on the N value and the presence of rock fragments in the sampler. Borehole 803 was terminated in a layer of very dense sandy silt after exploring it for 3.7 metres. Measured N values in the sandy silt ranged from 85 to greater than 100 blows per 0.3 metres. Samples of the sandy silt had water contents of 7 to 17 per cent. The sandy silt in borehole 803 contained clayey zones as evidenced by the grain size distributions and an Atterberg limits determination. The sample had plastic and liquid limits of 13 and 20 per cent, respectively, and a plasticity index of 7 per cent. Grain size distribution curves for samples of the sandy silt are shown on Figure A-5. The results of the Atterberg limits determinations are shown on Figure A-6.

#### 4.1.8 Silty Clay Glacial Till

A 1.5 metre thick layer of hard silty clay glacial till was encountered at elevation 253.6 metres in borehole 802 beneath the clayey silt till. The silty clay till had an N value of 50 blows per 0.3 metres. Cobbles and boulders should be expected in the silty clay till.

#### 4.1.9 Bedrock

The bedrock surface was encountered in borehole 802 at elevation 252.1 metres. The bedrock was explored for 3.1 metres by tricone drilling. The rock chip samples were examined and described as grey, limestone bedrock.

### 4.2 Groundwater Conditions

Groundwater conditions were observed in the boreholes during drilling and two piezometers were installed in borehole 803 with the piezometer tips at elevations 262.7 and 253.5 metres as shown on the Record of Borehole sheets. The encountered and measured groundwater levels are summarized in the following tables.

Borehole	Ground Surface Elevation (m)	Encountered Groundwater Level	
		Depth (m)	Elevation (m)
801	271.9	6.7	265.2
802	271.9	6.8	265.1
803	272.4	6.9	265.5
804	273.6	*	*

\* Borehole 804 dry during drilling on June 19, 2014.



The encountered water levels are not considered to be representative of the long-term, stabilized groundwater conditions as the readings were taken only during the relatively short duration of the drilling program.

Installation	Measured Groundwater Elevation (m)	
	June 24, 2014 (following installation)	February 19, 2015
BH 803 shallow piezometer	267.94	265.55
BH 803 deep piezometer	261.84	265.04

Based on the measured and encountered groundwater levels, the groundwater level is inferred to be perched in the sand and gravel at elevation 265 metres. The original contract drawing, Department of Highways, Ontario Drawing Number D2181-1, dated August 1, 1931 indicates high and low river water levels of 268.8 and 265.8 metres, respectively. Groundwater levels should be expected to fluctuate seasonally and are expected to be higher during periods of sustained precipitation or during spring melt conditions.





## 5.0 MISCELLANEOUS

This investigation was carried out using equipment supplied and operated by Henderson Drilling Inc., an Ontario Ministry of Environment licensed well contractor. The field operations were supervised by Mr. Michael Arthur under the direction of the field investigation manager, Mr. David J. Mitchell.

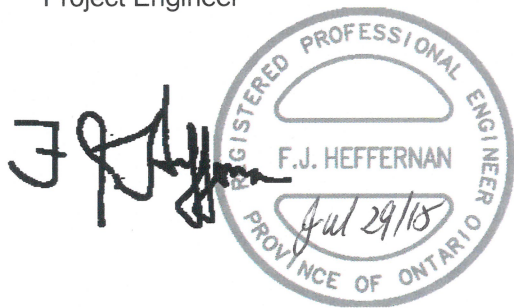
The laboratory testing was carried out at Golder Associates' London laboratory under the direction of Mr. Chris M. Sewell. The laboratory is an accredited participant in the MTO Soil and Aggregate Proficiency Program and is certified by the Canadian Council of Independent Laboratories for testing Types C and D aggregates. This report was prepared by Ms. Nicole A. Gould, P.Eng., under the direction of the Project Engineer, Ms. Dirka U. Prout, P.Eng. This report was reviewed by Mr. Michael E. Beadle, P.Eng. an Associate and Senior Geotechnical Engineer with Golder Associates. Mr. Fintan J. Heffernan, P.Eng., the Designated MTO Contact and Quality Control Auditor for this assignment, conducted an independent quality review of the report.

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**PART B**

**PRELIMINARY FOUNDATION DESIGN REPORT**

**HIGHWAY 4 BAYFIELD RIVER BRIDGE REPLACEMENT  
SITE NUMBER 12-195**

**HIGHWAYS 401, 4 AND 21 STRUCTURAL REPLACEMENTS  
GWP 3059-11-00, ASSIGNMENT No. 3 (3011-E-0048)**

**MINISTRY OF TRANSPORTATION, ONTARIO - WEST REGION**



## 6.0 ENGINEERING RECOMMENDATIONS

This section of the report provides our recommendations on the foundation aspects of the design of the replacement of the existing Highway 4 bridge over Bayfield River (Site 12-195). The recommendations are based on our interpretation of the factual information obtained during the investigation. It should be noted that the interpretation and recommendations are intended for use only by the design engineer. Where comments are made on construction, they are provided only to highlight those aspects which could affect the design of the project. Those requiring information on aspects of construction should make their own interpretation of the factual information provided as it may affect equipment selection, proposed construction methods, scheduling and the like.

It is understood that the existing bridge structure, constructed in 1932, is a two-span bridge over the Bayfield River located in the Town of Clinton, Ontario. The existing bridge is a rigid frame structure with two spans of 15.2 metres each (total 30.4 metres) and a width of 11.53 metres.

Based on the available information, the abutments, wing walls and centre pier are founded on shallow footings. Based on DHO Drawing No. D-2181-1 dated September 22, 1931, the underside of the footing elevation for the structural elements and design founding materials are summarized as follows:

Bridge Element	Founding Elevation (m)	Founding Material
North Abutment and Wing Walls	264.13	Clay and gravel
Central Pier	263.45	Very hard clay and gravel
South Abutment and Wing Walls	263.99	Gravel and clay

It has been indicated by Dillon that one and two span options are being considered, including a 35 metre long bridge with spread footings located at about the location of the existing footings, and a 46 metre long bridge with integral abutments on H-piles, both centred on the existing pier. It is understood that an integral abutment design is preferred. It is anticipated that minimal grade raise of the Highway 4 platform will be required. The RFP indicates that this section of Highway 4 is to remain a two-lane cross section. The replacement structure will be widened by about 2 metres.

### 6.1 Bridge Foundations

The subsurface soil conditions at the site typically consist of the existing pavement structure and fill materials, overlying sand and gravel, glacial till and limestone bedrock. Native soils were encountered in boreholes 801, 802 and 803 between about elevations 265.2 and 265.8 metres and in borehole 804, advanced approximately 36 metres north of the bridge structure, at about elevation 272.7 metres. The elevation of Highway 4 is at about elevation 272 metres at the bridge site. The inferred groundwater level is at about elevation 265 metres.

Integral abutments are typically founded on steel H-piles with lateral loads applied in the direction of the weak axis. Alternatively, integral abutments may be founded on concrete filled steel tube piles provided the additional resistance to lateral loads is adequately considered during structural design. Driven steel H-piles, closed-end



concrete filled steel tube piles or drilled shafts (caissons) are considered appropriate for support of semi-integral or conventional abutments at the site.

If a multi-span structure is designed for the new bridge, pier(s) may be supported on shallow spread footings, driven piles or drilled shafts. For preliminary design purposes, the subsoil conditions at the location of a pier may be extrapolated from the borehole information available at the abutments, though additional investigation would be required for detailed design stages if a pier is to be constructed. Construction for a central pier will require the construction of a cofferdam in the Bayfield River.

Recommendations for each of these foundation systems are provided below. A comparison of foundation alternatives is presented in Table I following the text of this report. The relative costs are compared using the most economical foundation option (shallow foundations) as the base cost. The estimated relative costs are meant to provide an order of magnitude comparison amongst the alternatives and are not indicative of actual construction costs. For an integral abutment design the preferred technical alternative from a foundation engineering perspective is to found the structure on driven H-piles or concrete filled steel tube piles. It may be cost-effective to support the pier, if required, on a shallow foundation.

### **6.1.1 Shallow Foundations**

#### ***Geotechnical Axial Resistance***

The abutments, pier and retaining walls for the replacement bridge may be founded on conventional spread/strip footings. Assuming the new footings are constructed at similar elevations as the existing footings, a factored geotechnical resistance at Ultimate Limit States (ULS) of 500 kilopascals (kPa) and a geotechnical reaction at Serviceability Limit States (SLS) of 350 kPa may be used for footings founded on the compact to dense sand and gravel at or below elevation 265 metres. The SLS value corresponds to an estimated total settlement of 25 millimetres. A footing width of 4 to 6 metres has been assumed.

#### ***Resistance to Lateral Forces***

Resistance to lateral forces/sliding between the concrete spread/strip footings and the native, undisturbed subsoil should be calculated in accordance with Section 6.7.5 of the 2006 Canadian Highway Bridge Design Code (CHBDC). Assuming that the founding soils are not loosened/disturbed during excavation and footing construction, an angle of friction between the mass cast-in-place concrete and the founding soils of 35 degrees and corresponding unfactored coefficient of friction,  $\tan \delta$ , of 0.7 may be used.

#### ***Frost Protection and Scour Protection***

All footings should be provided with a minimum of 1.2 metres of soil cover or thermal equivalent for frost protection. Footings should be adequately protected against scour as noted in Section 1.9.5.2 of the CHBDC.



### 6.1.2 Deep Foundations

The abutments and pier for the replacement structure could be founded on 324 millimetre outside diameter (OD) concrete filled steel tube piles with 9.5 millimetre wall thicknesses, steel HP 310 x 110 piles or drilled shafts filled with concrete. Integral abutments could be founded on concrete filled steel tube piles or steel HP 310 x 110 piles. Piles supporting integral abutments require pre-augering and placement of a corrugated steel pipe (CSP) liner filled with loose, uniform sand around the upper 3 metres of the pile to reduce resistance to lateral movement. Integral abutments are typically founded on steel H-piles with lateral loads applied in the direction of the weak axis bending (i.e. lateral loads applied parallel to the pile flange). Alternatively, integral abutments may be founded on concrete filled steel tube piles provided the additional resistance to lateral loads is adequately considered during structural design.

#### *Geotechnical Axial Resistance – Driven Piles*

For design, the factored geotechnical axial resistances at ULS and geotechnical reactions at SLS for HP 310 x 110 piles and concrete filled steel tube piles driven to refusal at or below the maximum anticipated elevations shown are provided in the following table. SLS values are not provided since the founding condition is considered to be effectively unyielding. The steel tube piles should be driven closed-ended and filled with concrete.

Pile Type and Location	Assumed Cut-off Elevation (m)	Maximum Tip Elevation (m)	Factored Geotechnical Resistance at ULS (kN)
HP 310 x 110			
- South Abutment	267.0		
- Central Pier	263.5	252.0	2,000
- North Abutment	267.0		
324 mm OD x 9.5 mm concrete filled steel tube			
- South Abutment	267.0		
- Central Pier	263.5	255.0	1,600
- North Abutment	267.0		



The above cut-off elevations have been assumed based on the existing footing elevations at the site plus an above ground height of 3 metres for integral abutments. Lower cut-off elevations will likely be used if other abutment types are selected, resulting in shorter piles.

The actual pile penetration and pile set characteristics will be dependent, to some extent, upon the driving equipment selected by the contractor. The pile driving hammer should be selected such that the energy delivered to the piles is sufficient to achieve the termination criteria (ultimate geotechnical resistances and tip elevations) stated in the Contract Documents. It is recommended that, following the selection of the driving equipment, the piling contractor submit for review the proposed pile driving criteria based on the characteristics of the hammer and equipment intended for use.

### Geotechnical Axial Resistance – Drilled Shafts (Caissons)

For preliminary design, the vertical load carrying capacity of the caissons derived from skin friction may be calculated using the following equation.

$$Q_s = \pi B \Delta z f_{SN}$$

Where  $Q_s$  is the nominal skin friction in kilonewtons (kN),  $B$  is the shaft diameter in metres,  $\Delta z$  is the thickness of the soil layer over which resistance is calculated in metres and  $f_{SN}$  is the nominal unit skin friction in kPa. The upper 1.2 metres below the ground surface should be neglected to account for frost action. Any portion of the caisson within fill materials should also be neglected.

Assuming that caissons greater than 1 metre in diameter will be used, the component of the vertical load carrying capacity that may be derived from end bearing in the native soils/bedrock may be calculated using the following equation:

$$Q_b = q_{BN} A_t$$

where  $Q_b$  is the toe resistance in kN,  $q_{BN}$  is the nominal unit base resistance in kPa and  $A_t$  is the cross-sectional area of the caisson in square metres. Caissons founded in the native soils/bedrock may be designed using the nominal unit side and base resistances provided in the following table. The stratigraphy presented in the table below has been simplified for the purposes of this report.

Soil Type	Elevation (m)	$f_{SN}$ (kPa)	$q_{BN}$ (kPa)	Unit Weight (kN/m <sup>3</sup> )
Fill	Where applicable	-	-	19.0
South Abutment				
- Sand and gravel	264.0 to 265.0	50	500	22.0
- Sandy silt	263.0 to 264.0	400	2,000	21.0
- Clayey silt till	254.0 to 263.0	150	1,800	21.0
- Silty clay till	252.0 to 254.0	150	1,800	20.0
- Bedrock	252.0	-	4,000	-



Soil Type	Elevation (m)	$f_{SN}$ (kPa)	$q_{BN}$ (kPa)	Unit Weight (kN/m <sup>3</sup> )
North Abutment				
- Sand and gravel	262.5 to 265.0	50	500	22.0
- Clayey silt till	259.5 to 262.5	150	1,800	21.0
- Silty clay	257.0 to 259.5	150	1,800	20.0
- Sandy Silt	253.5 to 257.0	500	2,000	20.0

The ultimate resistance  $Q_u$  is the sum of  $Q_b$  and  $Q_s$ . A resistance factor of 0.5 should be applied to  $Q_u$  to obtain the factored axial resistance at ULS. A 1.2 metre diameter caisson founded in the clayey silt till or sandy silt at elevation 257 metres may be designed using a factored geotechnical axial resistance at ULS of 2,800 kilonewtons.

### ***Frost Protection and Scour Protection***

The pile caps should be provided with a minimum of 1.2 metres of soil cover or thermal equivalent for frost protection. Pile caps should be adequately protected against scour as noted in Section 1.9.5.2 of the CHBDC.

### ***Downdrag Load (Negative Skin Friction)***

Considering the presence of compact to dense granular soils and very stiff to hard cohesive till, negative skin friction can be neglected.

### ***Resistance to Lateral Loads***

Lateral loading could be resisted fully or partially by the use of battered piles. In the case of integral abutments, the vertical piles must provide the resistance to the lateral loading. The stratigraphy presented in the table below has been simplified for the purposes of this report.

The horizontal reaction to the pile can be estimated using the following equations, undrained shear strengths, and ranges in subgrade reaction coefficient where:

$$\begin{aligned}
 k_h &= \text{coefficient of horizontal subgrade reaction (MPa/m)} &= n_h (z/d) &\text{for cohesionless soils} \\
 & &= \frac{67 S_u}{d} &\text{for cohesive soils}
 \end{aligned}$$

where:

- $d$  = pile width or diameter (m)
- $n_h$  = constant of horizontal subgrade reaction (MPa/m)
- $S_u$  = undrained shear strength of the soil (MPa)
- $z$  = depth below ground surface grade (m)



The range in values reflects the variability in subsurface conditions as well as the two extremes of design; the requirement for flexibility if integral abutments are selected and the requirement for lateral support in the cases of non-integral abutments or pier foundations.

Location	Soil Type	Elevation (m)	$n_h$ (MPa/m)	$S_u$ (kPa)
CSPs for integral abutments	Granular backfill	Where applicable	5 - 10	-
South Abutment	Compact sand and gravel	264.0 to 265.0	6 - 11	-
	Very dense sandy silt	263.5 to 264.0	18 - 22	-
	Very stiff to hard clayey silt till	254.0 to 263.5	-	200
	Hard silty clay till	252.0 to 254.0	-	200
North Abutment	Compact to very dense sand and gravel	262.5 to 265.0	9 - 16	-
	Very stiff to hard clayey silt till	259.5 to 262.5	-	200
	Hard silty clay	256.5 to 259.5	-	200
	Very dense sandy silt	253.5 to 256.5	15 - 19	-

The lateral resistances for the various foundation options are summarized in the following table.

Pile Type	Lateral Resistance	
	Factored ULS (kN)	SLS (kN)
Integral abutments		
- HP 310 x 110, weak axis bending	265	250
- 324 mm OD x 9.5 mm tube	315	270
Semi-Integral or Conventional abutments		
- HP 310 x 110, strong axis bending	430	300
- 324 mm OD x 9.5 mm tube	315	270

The lateral resistances are based on Brom's Method from "Design and Construction of Driven Pile Foundations Workshop Manual – Volume 1, FHWA, Pub. No. FHWA H1 97-013, Revised November 1998". Fixed-head conditions were assumed. For the steel H-pile and concrete filled steel tube pile alternatives, the horizontal load for semi-integral or conventional abutments was assumed to be applied at the anticipated footing elevation and for integral abutments at a height of 3 metres above the existing ground surface. The SLS values are based on 10 millimetres of deflection at the ground surface. Based on these assumptions, the steel H-piles and concrete



filled steel tube piles are expected to exhibit long pile behaviour, while the caissons will exhibit short pile behaviour.

Group action for lateral loading should be considered when the pile spacing in the direction of the loading is less than six to eight pile diameters. Group action can be evaluated by reducing the coefficient of horizontal subgrade reaction in the direction of loading by a reduction factor,  $R$ , as follows:

<i>Pile Spacing in Direction of Loading, <math>d</math> = Pile Diameter</i>	<i>Subgrade Reaction Reduction Factor <math>R</math></i>
8d	1.00
6d	0.70
4d	0.40
3d	0.25

## 6.2 Liquefaction Potential and Seismic Analysis

### 6.2.1 Seismic Parameters

The site is located in the Town of Clinton, Ontario. According to Table A.3.1.1 of the CHBDC, the zonal acceleration ratio,  $A$ , applicable to this site is 0.00. The corresponding acceleration-related seismic zone,  $Z_a$  is 0.

The replacement bridge is in Seismic Performance Zone (SPZ) 1, based on a CHBDC classification as an "Emergency Route Bridge". Based on the site stratigraphy, the soil profile type is categorized as Type I with a seismic site response coefficient,  $S$ , of 1.0 based on the CHBDC criteria. Analysis of bridges in SPZ 1 is not a requirement of the CHBDC. However, design forces for restraining elements and bridge support lengths must meet the minimum requirements as outlined in CHBDC Clause 4.4.5.1.

### 6.2.2 Seismic Hazard Assessment

A preliminary screening of the soil stratigraphy was conducted using the procedure outlined in the FHWA recommended procedures.<sup>5</sup> The liquefaction potential is considered to be low based on the soil profile type, age of the deposits, relative density and the historically low regional seismicity; therefore, a detailed evaluation of the liquefaction potential of the foundation soils, impact of liquefaction on the bridge foundations and the effect of seismic forces on embankment stability is not considered warranted unless the structure is deemed to be a lifeline bridge.

<sup>5</sup> FHWA, 1997: "Design Guidance: Geotechnical Earthquake Engineering For Highways. Volume I – Design Principles." *Geotechnical Engineering Circular No. 3: FHWA-SA-97-076*, Washington, D.C.





### 6.3 Lateral Earth Pressures for Design

The lateral pressures acting on the bridge abutments and associated wing/retaining walls will depend on the type and method of placement of the backfill materials, the nature of the soils behind the backfill, the freedom of lateral movement of the structure and the drainage conditions behind the walls. The following recommendations are made concerning the design of the abutments, in accordance with the CHBDC:

- Select, free-draining granular fill meeting the specifications of Ontario Standard Specifications (OPSS) Granular A or Granular B Type III but with less than 5 per cent passing the No. 200 sieve should be used as backfill behind the abutments and walls. This fill should be compacted in loose lifts not greater than 200 millimetres in thickness in accordance with OPSS.PROV 501. Longitudinal drains and weep holes should be installed to provide positive drainage of the granular backfill. Other aspects of the abutment granular backfill requirements with respect to subdrains and frost taper should be in accordance with Ontario Provincial Standard Drawing (OPSD) 3101.150 and 3190.100.
- A compaction surcharge equal to 12 kPa should be included in the lateral earth pressures for the structural design of the abutment walls in accordance with CHBDC Figure 6.6. Compaction equipment should be used in accordance with OPSS.PROV 501.
- The granular fill may be placed either in a zone with a width equal to at least 1.2 metres behind the back of the stem (Case a from Commentary on CHBDC Figure C6.20) or within the wedge-shaped zone defined by a line drawn at a maximum 1 horizontal to 1 vertical extending up and back from the rear face of the footing (Case b from Commentary on CHBDC Figure C6.20).
- For Case a, the restrained case, the pressures are based on the proposed embankment fill materials and the following parameters (unfactored) may be used assuming the use of Select Subgrade Material (SSM).

Soil unit weight: 20 kN/m<sup>3</sup>

Coefficient of lateral earth pressure:  
At rest,  $K_o$  0.50

- For Case b, the unrestrained case, the pressures are based on the granular fill as placed and the following parameters (unfactored) may be assumed:

	<u>GRANULAR A</u>	<u>GRANULAR B</u> <u>Type III</u>
Soil unit weight:	22 kN/m <sup>3</sup>	21 kN/m <sup>3</sup>
Coefficients of lateral earth pressure:		
Active, $K_a$	0.27	0.31
At rest, $K_p$	3.7	3.3

- If the wall support and superstructure allow lateral yielding of the stem, active earth pressures may be used in the geotechnical design of the structure. If the wall support does not allow lateral yielding, at-rest earth pressures should be assumed for geotechnical design.



- For integral abutments, passive earth pressures may be used in the geotechnical design of the structure. The movements required to fully mobilize passive pressure or resistance are much larger than those required to mobilize active pressure. In practice, movements may not be sufficient to mobilize the full passive resistance. A resistance factor equal to 0.5 should be applied to the calculated total passive resistance in accordance with Table 6.1 of the CHDBC.

It should be noted that the above design parameters assume level backfill and ground surface behind the wall. The lateral earth pressure coefficients should be adjusted if there is sloping ground at the back of the wall.

## **6.4 Construction Considerations**

### **6.4.1 Shallow Foundations**

The cleaned excavation base should be inspected by a Quality Verification Engineer (QVE) qualified in geotechnical engineering prior to placing the working slab. It is recommended that the footing excavations be carried out such that the final 0.5 metres of excavation is completed with the geotechnical personnel on site and the working slab be placed immediately after footing inspection.

### **6.4.2 Deep Foundations**

Cobbles and boulders should be expected in the soils at the site and may impact pile driving/caisson drilling operations. A non-standard special provision (NSSP) should be added to the Contract Documents to alert the Contractor to the need for special procedures to deal with cobbles, boulders and other obstructions during pile installation. If piles are to be driven through the existing embankment fill near the existing abutments, they may encounter remnants of temporary works buried in the fill. Slurry based or casing based methods will be necessary to advance drilled shafts into the sand and gravel in order to maintain caisson stability.

Deep foundations should be installed and monitored in accordance with OPSS 903, as well as OPSD 3000.150, 3001.150, and SS103-11 (Pile Driving Control) for the driven piles. The H-piles and steel tube piles should to be equipped with Type I driving shoes as shown in OPSD 3000.100 and 3001.100, respectively.

## **6.5 Embankments**

It is anticipated that if raising of the embankments is required to accommodate repositioned abutments, the grade raise will be minimal. Further, it is understood that the highway cross section will remain two lanes; therefore approach embankment widening is not expected. However, if the design of the replacement bridge does require grade raising or embankment widening the following should be considered. All surficial topsoil, organic, loose, soft, and/or otherwise deleterious materials should be stripped from areas of proposed embankment widenings. Prior to placement of embankment fill material, the exposed subgrade should be proofrolled under the direction of a geotechnical QVE. The embankment fills should consist of an approved granular borrow such as SSM, except for the top approximately 0.5 to 1.0 metres where pavement base and subbase materials will be placed. Embankment fill materials should be placed in maximum 300 millimetre thick loose lifts properly benched into the existing embankments and compacted. Upon completion of filling to the



pavement subgrade level, the embankment side slopes should be trimmed to a final inclination of two horizontal to one vertical or flatter. Embankment modifications constructed in this manner are expected to have an adequate factor of safety against instability. Embankments greater than 8 metres in height should be constructed with a 2 metre wide bench at mid-height.

## **6.6 Excavations**

Excavations for the pile caps or shallow foundations will penetrate the existing fill and organic materials into the underlying sand and gravel, sandy silt and clayey silt till. The groundwater level is expected to be at about elevation 265 metres and will fluctuate seasonally. The excavations are expected to extend below the groundwater level. Groundwater flow from the sand and gravel should be expected. It is considered that adequate groundwater control could be achieved by pumping from properly constructed and filtered sumps in the base of the excavation. Sumps should be maintained outside of the actual pile cap and/or abutment limits. Depending on the time of year, aggressive pumping may be required. Surface water runoff should be directed away from the excavations at all times.

All excavations should be carried out in accordance with the guidelines outlined in 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 as would any cohesionless soils below the groundwater level. The native clayey materials and properly dewatered cohesionless soils would be classified as Type 2.

### **6.6.1 Temporary Roadway Protection**

To support the sides of the excavation and permit the use of vertical cuts, temporary road protection systems may be required where space is restricted and will not permit open cuts. The design and limits of the systems are to be determined by the contractor.

Temporary support systems could consist of soldier piles and lagging, where the H-piles would be driven to a suitable depth and horizontal lagging installed as the excavation proceeds, or driven steel sheet piling. Support to the system(s) could be in the form of struts and walers or rakers and anchors. The support system must be designed to accommodate the loads applied from pressures and surcharge pressures from area, line or point loads as well as address the impact(s) of sloping ground behind the system.

### **6.6.2 Temporary Cofferdams**

Cofferdams will be required to permit construction of the pier footing in the dry. The design and limits of the cofferdams are to be determined by the Contractor.

An internally braced interlocking steel sheet pile cofferdam may be used to enclose the footing excavation. The sheeting should be driven to sufficient depth in the sand and gravel to control the water inflow. In that case, the water inflow could be handled by pumping from sumps. Alternatively, the sheeting could be driven into the clayey silt till to effect a cut-off to permit construction in the dry. The footings should also be anchored to internal seals. The design and construction of braced sheet pile cofferdams is to be carried out by the Contractor. It



should be conducted in accordance with OPSS 539 with permissible lateral deflections meeting Performance Level 2.

Cofferdam construction should be carried out during low flow periods and adhere to the temporary erosion protection and sediment control requirements of OPSS 805 and OPSS.PROV 182. Cofferdams should be monitored daily for leaks and other defects with more detailed inspections before and after periods of high precipitation. All cofferdams should be sized to accommodate additional footing excavation, as necessary. Inclusion of an NSSP to address cofferdam construction for shallow foundation construction should be included in the contract documents.



## **7.0 RECOMMENDATIONS FOR DETAIL DESIGN**

Driven steel H or concrete filled steel tube piles are considered to be the preferred foundation alternative for the design of integral abutments for the replacement structure. A Foundation Investigation and Design Report should be prepared during a future assignment to provide appropriate information for future Detail Design. A standard MTO foundation investigation for a bridge structure is considered appropriate for the site. Specifically, a minimum of two boreholes should be advanced, one at each abutment opposite to existing boreholes 802 and 803, and at the proposed central pier. The bedrock in these two boreholes should be cored in NQ size for a minimum depth of 3 metres. If the Detail Design involves realigning the replacement structure from its current location, additional boreholes may be required at the approaches and/or abutments in addition to those noted above.

The recommendations given in this Preliminary Foundation Design Report should be expanded upon and updated in the Foundation Design Report for detail design in accordance with MTO's standard requirements for foundation engineering assignments. Detailed recommendations should be provided for foundations for the abutments, wingwalls and any proposed central pier(s). If the vertical alignments of the highway or the approach embankments are to be significantly altered, embankment stability and settlement should be evaluated. Further, if staged construction is to be used for the construction of the replacement bridge, the discussion on temporary roadway protection should include lateral earth pressures and effect of ground conditions on shoring construction design.



## 8.0 MISCELLANEOUS

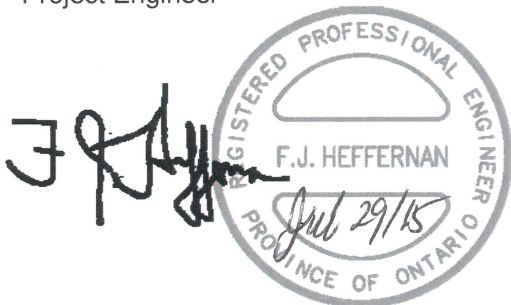
This report was prepared by Ms. Nicole A. Gould, P.Eng. under the direction of the Project Engineer, Ms. Dirka U. Prout, P.Eng. This report was reviewed by Mr. Michael E. Beadle, P.Eng. an Associate and Senior Geotechnical Engineer with Golder Associates. Mr. Fintan J. Heffernan, P.Eng., the Designated MTO Contact and Quality Control Auditor for this assignment, conducted an independent quality review of the report.

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TABLE I

**COMPARISON OF FOUNDATION ALTERNATIVES – BRIDGE REPLACEMENT**

Highway 4 Bayfield River Bridge  
 Highways 401, 4 and 21 Structural Replacements  
GWP 3059-11-00

<b>FOUNDATION OPTION</b>	<b>FEASIBILITY</b>	<b>ADVANTAGES</b>	<b>DISADVANTAGES</b>	<b>RELATIVE COSTS<sup>1</sup></b>	<b>RISKS/ CONSEQUENCES</b>
Spread footings supported on compact to very dense sand and gravel.	<ul style="list-style-type: none"> <li>• Feasible for abutments and central pier(s).</li> </ul>	<ul style="list-style-type: none"> <li>• Least expensive option.</li> <li>• Ease of construction.</li> </ul>	<ul style="list-style-type: none"> <li>• Not compatible with integral abutments.</li> <li>• More settlement expected than with deep foundations.</li> <li>• Larger work area required compared to caissons or driven piles.</li> </ul>	<ul style="list-style-type: none"> <li>• Low</li> </ul>	<ul style="list-style-type: none"> <li>• Relatively low risk.</li> <li>• Deeper excavations required if soil at founding elevation is unsuitable.</li> <li>• Dewatering requirements.</li> </ul>
End bearing steel H-pile or steel tube pile foundations driven to refusal.	<ul style="list-style-type: none"> <li>• Preferred technical alternative for abutments.</li> <li>• May be considered for central pier(s).</li> </ul>	<ul style="list-style-type: none"> <li>• High bearing resistance.</li> <li>• Negligible settlement.</li> <li>• Compatible with integral abutments.</li> </ul>	<ul style="list-style-type: none"> <li>• More expensive than shallow foundations.</li> <li>• Can be damaged and deflected by cobbles and boulders within glacial till deposits.</li> <li>• More construction noise and vibration compared to shallow foundations or caissons.</li> <li>• H-Piles cannot be visually inspected at depth.</li> </ul>	<ul style="list-style-type: none"> <li>• Moderate</li> </ul>	<ul style="list-style-type: none"> <li>• Possible pile tip damage if piles are not adequately protected while driving through till and sand and gravel deposits.</li> </ul>

**COMPARISON OF FOUNDATION ALTERNATIVES – BRIDGE REPLACEMENT**

<b>FOUNDATION OPTION</b>	<b>FEASIBILITY</b>	<b>ADVANTAGES</b>	<b>DISADVANTAGES</b>	<b>RELATIVE COSTS<sup>1</sup></b>	<b>RISKS/ CONSEQUENCES</b>
<ul style="list-style-type: none"> <li>• Concrete caissons drilled into hard clayey silt till.</li> </ul>	<ul style="list-style-type: none"> <li>• Feasible for abutments but not preferred.</li> <li>• May be preferred for pier(s).</li> </ul>	<ul style="list-style-type: none"> <li>• Negligible settlement.</li> <li>• Less construction noise and vibration compared to driven piles.</li> <li>• Faster construction and less work space required compared to shallow foundations.</li> <li>• Less potential for caissons to be impeded by cobbles in native till deposits, compared to driven piles.</li> </ul>	<ul style="list-style-type: none"> <li>• Potential for greater settlement compared to driven piles.</li> <li>• Not compatible with integral abutments.</li> <li>• Cannot be visually inspected at depth due to health and safety regulations.</li> </ul>	<ul style="list-style-type: none"> <li>• High</li> </ul>	<ul style="list-style-type: none"> <li>• Cleaning of base could be problematic or overlooked during construction.</li> </ul>

- NOTES:
1. The estimated relative costs are intended to provide a comparison between alternatives rather than actual construction costs, and do not take into consideration additional costs such as traffic control, staging and shoring that may influence the total cost.
  2. Table to be read in conjunction with accompanying report.

Prepared By: NG  
 Checked By: MEB



## 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

### III. SOIL DESCRIPTION

#### (a) Cohesionless Soils

Density Index (Relative Density)	N 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

### 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.) split spoon sampler for a distance of 300 mm (12 in.)

#### Consistency

	<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 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

#### (b) Cohesive Soils

#### 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 cm<sup>2</sup> 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.

### 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 <sup>1</sup>
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement <sup>1</sup>
$D_R$	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)
$\gamma$	unit weight

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

## LIST OF SYMBOLS

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

### I. General

$\pi$	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
F	factor of safety
V	volume
W	weight

### II. STRESS AND STRAIN

$\gamma$	shear strain
$\Delta$	change in, e.g. in stress: $\Delta \sigma$
$\epsilon$	linear strain
$\epsilon_v$	volumetric strain
$\eta$	coefficient of viscosity
$\nu$	poisson's ratio
$\sigma$	total stress
$\sigma'$	effective stress ( $\sigma' = \sigma - u$ )
$\sigma'_{vo}$	initial effective overburden stress
$\sigma_1, \sigma_2, \sigma_3$	principal stress (major, intermediate, minor)
$\sigma_{oct}$	mean stress or octahedral stress $= (\sigma_1 + \sigma_2 + \sigma_3)/3$
$\tau$	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
$\gamma'$	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
$w_l$	liquid limit
$w_p$	plastic limit
$I_p$	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
$C_a$	coefficient of secondary consolidation
$m_v$	coefficient of volume change
$c_v$	coefficient of consolidation
$T_v$	time factor (vertical direction)
U	degree of consolidation
$\sigma'_p$	pre-consolidation pressure
OCR	over-consolidation ratio $= \sigma'_p / \sigma'_{vo}$

#### (d) Shear Strength

$\tau_p, \tau_r$	peak and residual shear strength
$\phi'$	effective angle of internal friction
$\delta$	angle of interface friction
$\mu$	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

- Notes:** 1  $\tau = c' + \sigma' \tan \phi'$   
 2 shear strength = (compressive strength)/2  
 \* density symbol is  $\rho$ . Unit weight symbol is  $\gamma$  where  $\gamma = \rho g$  (i.e. mass density x acceleration due to gravity)

**RECORD OF BOREHOLE No 801**

1 OF 1

**METRIC**

PROJECT 12-1132-0076  
W.P. 3059-11-00 LOCATION N 4829917.9 , E 382781.6 ORIGINATED BY BT  
DIST HWY 4 BOREHOLE TYPE POWER AUGER, HOLLOW STEM COMPILED BY LMK  
DATUM GEODETIC DATE June 16, 2014 CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL LIQUID LIMIT MOISTURE CONTENT CONTENT W <sub>P</sub> W      W <sub>L</sub>			UNIT WEIGHT  γ  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)				GR	SA	SI	CL	
								○ UNCONFINED      + FIELD VANE ● QUICK TRIAXIAL      × LAB VANE													
271.87	PAVEMENT SURFACE							20	40	60	80	100									
0.00	ASPHALTIC CONCRETE																				
0.15	FILL, sand and gravel, crushed																				
0.34	FILL, sand, fine to coarse, trace gravel, silt																				
271.11	Brown																				
0.76	FILL, silty fine sand, trace gravel		1	SS	13		271														
0.91	FILL, clayey silt, some sand, gravel																				
	Stiff to firm																				
	Brown and grey		2	SS	5		270														
269.98																					
1.89	FILL, topsoil, silty																				
2.13	Brown																				
	FILL, clayey silt, trace to some sand, gravel, with topsoil pockets		3	SS	3		269														
	Soft to stiff																				
	Brown and black		4	SS	5		268											1	19	58	22
			5	SS	8																
267.45																					
4.42	FILL, clayey silt, trace sand, gravel, topsoil		6	SS	6		267														
	Firm																				
	Brown and grey		7	SS	4		266														
			8	SS	7																
265.16																					
6.71	SAND AND GRAVEL, some silt		9	SS	13		265														
	Compact to dense																				
	Brown		10	SS	30		264											21	67	(12)	
263.03																					
8.84	CLAYEY SILT TILL, trace to some sand, trace gravel		11	SS	46		263														
	Hard																				
	Brown																				
			12	SS	42		261											3	6	62	29
			13	SS	39		260														
							259														
			14	SS	41		258											2	4	63	31
257.70																					
14.17	END OF BOREHOLE																				
	Groundwater encountered at about elev. 265.2m during drilling on June 16, 2014.																				

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

PROJECT <u>12-1132-0076</u>		<b>RECORD OF BOREHOLE No 802</b>		1 OF 2	<b>METRIC</b>
W.P. <u>3059-11-00</u>	LOCATION <u>N 4829935.0 , E 382776.1</u>	ORIGINATED BY <u>MA</u>			
DIST <u>          </u> HWY <u>4</u>	BOREHOLE TYPE <u>POWER AUGER, HOLLOW STEM, TRICONE</u>	COMPILED BY <u>LMK</u>			
DATUM <u>GEODETIC</u>	DATE <u>June 17 - 19, 2014</u>	CHECKED BY <u>          </u>			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT  γ  kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100	W <sub>p</sub> W W <sub>L</sub>	WATER CONTENT (%)			GR SA SI CL							
271.91	PAVEMENT SURFACE					▽															
0.00	ASPHALTIC CONCRETE																				
0.27	FILL, sand and gravel, with cobbles Brown																				
271.18																					
0.73	FILL, silty fine sand, trace gravel Loose Brown		1	SS	7			271													
270.54																					
1.37	FILL, sandy silt to silty sand, trace to some clay, topsoil layers and pockets, trace gravel Very loose to loose Brown and black		2	SS	4			270													
				3	SS		7		269												
				4	SS		3		268												
				5	SS		7		267												
				6	SS		5		266												
			7	SS	7		265														

Continued Next Page

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity      ○ 3% STRAIN AT FAILURE

LDN\_MTO\_06 12-1132-0076-3001-R02.GPJ LDN\_MTO\_GDT 29/05/15

PROJECT <u>12-1132-0076</u>		<b>RECORD OF BOREHOLE No 802</b>		2 OF 2	<b>METRIC</b>
W.P. <u>3059-11-00</u>	LOCATION <u>N 4829935.0 , E 382776.1</u>	ORIGINATED BY <u>MA</u>			
DIST <u>          </u> HWY <u>4</u>	BOREHOLE TYPE <u>POWER AUGER, HOLLOW STEM, TRICONE</u>	COMPILED BY <u>LMK</u>			
DATUM <u>GEODETIC</u>	DATE <u>June 17 - 19, 2014</u>	CHECKED BY <u>          </u>			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT   NATURAL LIMIT   MOISTURE   CONTENT   LIQUID LIMIT			UNIT WEIGHT  γ  kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)  GR   SA   SI   CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)				
								○ UNCONFINED   + FIELD VANE ● QUICK TRIAXIAL   × LAB VANE					w <sub>p</sub> w   w <sub>L</sub>				
								20   40   60   80   100	20   40   60   80   100	10   20   30							
	CLAYEY SILT TILL, trace sand, gravel Very stiff to hard Brown		16	SS	24												
254.99 16.92	CLAYEY SILT TILL, trace sand, some gravel Hard Grey		17	SS	100/ 150mm												
253.62 18.29	SILTY CLAY TILL, trace sand, gravel Hard Grey		18	SS	50												
252.10 19.81	LIMESTONE BEDROCK Grey		19	SS	100/ 0mm												
			20	SS	100/ 0mm												
			21	WS													
249.05 22.86	END OF BOREHOLE  Groundwater encountered at about elev. 265.1m during drilling on June 17-19, 2014.																

LDN\_MTO\_06 12-1132-0076-3001-R02.GPJ LDN\_MTO\_GDT 03/06/15

**RECORD OF BOREHOLE No 803**

1 OF 2

**METRIC**

PROJECT 12-1132-0076  
W.P. 3059-11-00 LOCATION N 4829989.5, E 382761.8 ORIGINATED BY MA  
DIST HWY 4 BOREHOLE TYPE POWER AUGER, HOLLOW STEM, TRICONE COMPILED BY LMK  
DATUM GEODETIC DATE June 24, 2014 CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT  w <sub>p</sub>	NATURAL MOISTURE CONTENT  w	LIQUID LIMIT  w <sub>L</sub>	UNIT WEIGHT  γ  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)  GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa										WATER CONTENT (%)		
								○ UNCONFINED	+ FIELD VANE	● QUICK TRIAXIAL	× LAB VANE									
								20 40 60 80 100												
272.36	PAVEMENT SURFACE																			
0.00	ASPHALTIC CONCRETE																			
0.20	FILL, sand and gravel, trace silt																			
271.72	Brown																			
0.64	FILL, clayey silt, trace to some sand, trace gravel, topsoil pockets, silt layers Very soft to stiff Brown		1	SS	10															
			2	SS	7															
			3	SS	1															
			4	SS	5															
			5	SS	2															
			6	SS	7															
			7	SS	4															
265.81	CLAYEY SILT, trace sand		8	SS	15															
6.55	Very stiff																			
265.50	Brown																			
6.86	SAND AND GRAVEL, some silt		9	SS	25															
	Compact to dense																			
	Brown																			
			10	SS	48															
262.45	CLAYEY SILT TILL, trace sand, gravel		11	SS	64															
9.91	Hard																			
	Brown																			
			12	SS	39															
259.25	SILTY CLAY, trace sand, with silt seams		13	SS	62															
13.11	Hard																			
	Brown																			

Continued Next Page

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

LDN\_MTO\_06 12-1132-0076-3001-R02.GPJ LDN\_MTO\_GDT 29/05/15

**RECORD OF BOREHOLE No 803**









2 OF 2

**METRIC**

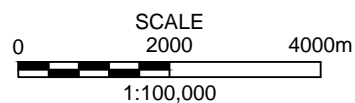
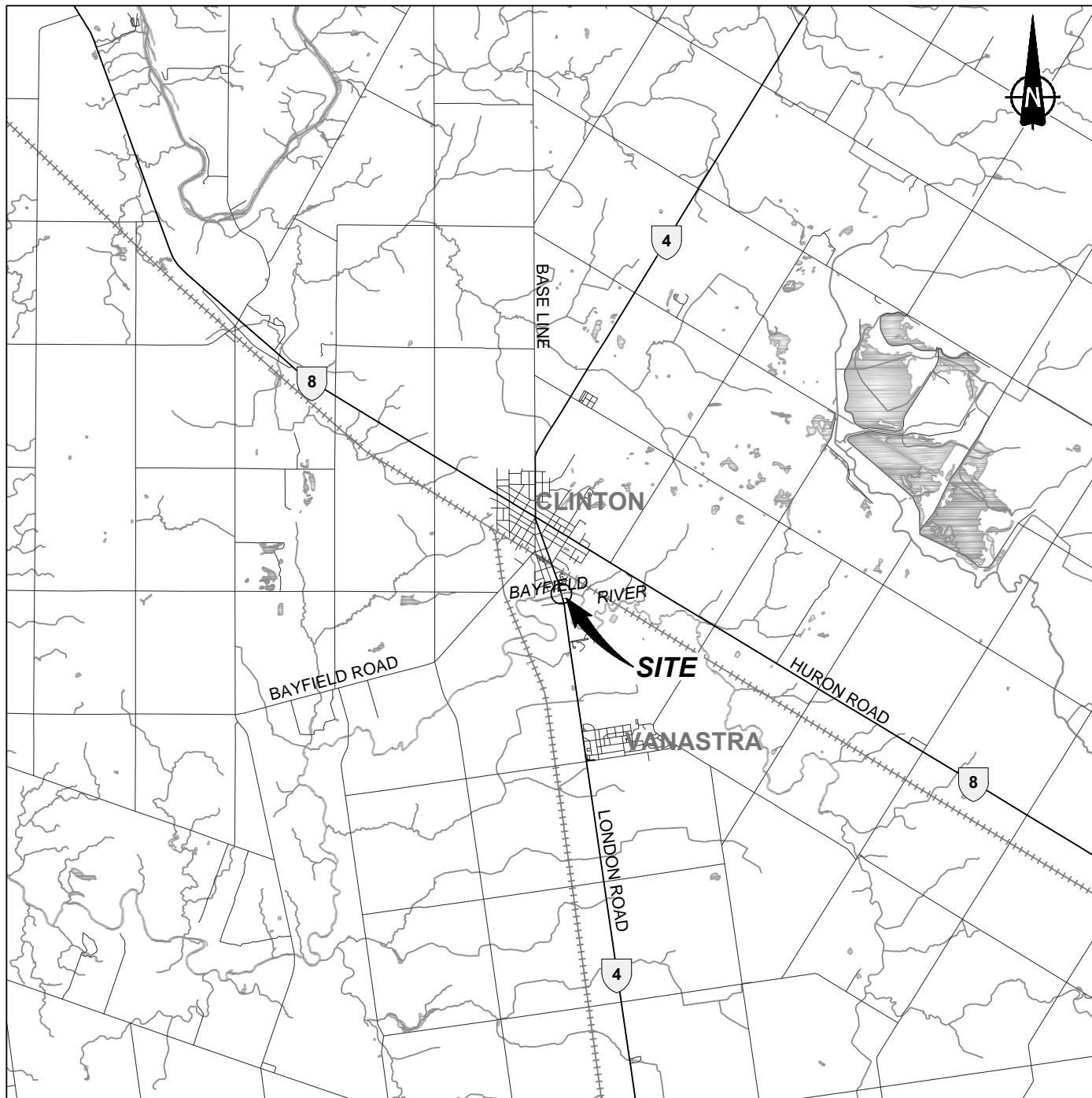
PROJECT 12-1132-0076  
W.P. 3059-11-00 LOCATION N 4829989.5 , E 382761.8 ORIGINATED BY MA  
DIST HWY 4 BOREHOLE TYPE POWER AUGER, HOLLOW STEM, TRICONE COMPILED BY LMK  
DATUM GEODETIC DATE June 24, 2014 CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT  w <sub>p</sub>	NATURAL MOISTURE CONTENT  w	LIQUID LIMIT  w <sub>L</sub>	UNIT WEIGHT  γ  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							WATER CONTENT (%)			
								20 40 60 80 100										
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE										
						20 40 60 80 100												
256.94	SANDY SILT, some clay, gravel Very dense Brown		14	SS	36		257							15 34 39 12				
15.42								256										
			15	SS	100/ 225mm			255										
			16	SS	85			254										
253.22	END OF BOREHOLE		17	SS	100/ 100mm													
19.14	Groundwater encountered at about elev. 265.5m during drilling on June 24, 2014.  Water level measured in shallow piezometer at elev. 267.94m on June 24, 2014.  Water level measured in deep piezometer at elev. 261.84m on June 24, 2014.  Water level measured in shallow piezometer at elev. 265.55m on February 19, 2015.  Water level measured in deep piezometer at elev. 265.04m on February 19, 2015.																	

PROJECT <u>12-1132-0076</u>		<b>RECORD OF BOREHOLE No 804</b>		1 OF 1		<b>METRIC</b>	
W.P. <u>3059-11-00</u>		LOCATION <u>N 4830016.4 , E 382759.7</u>		ORIGINATED BY <u>MA</u>			
DIST <u>          </u> HWY <u>4</u>		BOREHOLE TYPE <u>POWER AUGER, HOLLOW STEM</u>		COMPILED BY <u>LMK</u>			
DATUM <u>GEODETIC</u>		DATE <u>June 19, 2014</u>		CHECKED BY <u>          </u>			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT  γ  kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							WATER CONTENT (%)			
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE										
273.57 0.00	PAVEMENT SURFACE ASPHALTIC CONCRETE						20	40	60	80	100	Wp	W	WL	GR	SA	SI	CL
0.27 272.66 0.91	FILL, sand and gravel, trace silt, with cobbles Compact Brown		1	SS	11													
	CLAYEY SILT TILL, trace sand, gravel Firm to stiff Brown		2	SS	7													
270.95 2.62	CLAYEY SILT, trace sand Hard Brown		3	SS	32													
			4	SS	46								○		0	2	65	33
269.91 3.66	CLAYEY SILT TILL, trace sand, gravel Hard Brown to grey at about elev. 269.0m		5	SS	33													
			6	SS	33													
267.02 6.55	END OF BOREHOLE  Borehole dry during drilling on June 19, 2014.		7	SS	23								○	—	2	10	53	25





## REFERENCE

PLAN BASED ON CANMAP STREETFILES V.2008.

## NOTE

THIS DRAWING IS TO BE READ IN CONJUNCTION WITH ACCOMPANYING TEXT.

ALL LOCATIONS ARE APPROXIMATE.

PROJECT HIGHWAY 4 / BAYFIELD RIVER BRIDGE REPLACEMENT  
SITE 12-195  
HIGHWAYS 401, 4 AND 21 STRUCTURAL REPLACEMENTS  
GWP 3059-11-00

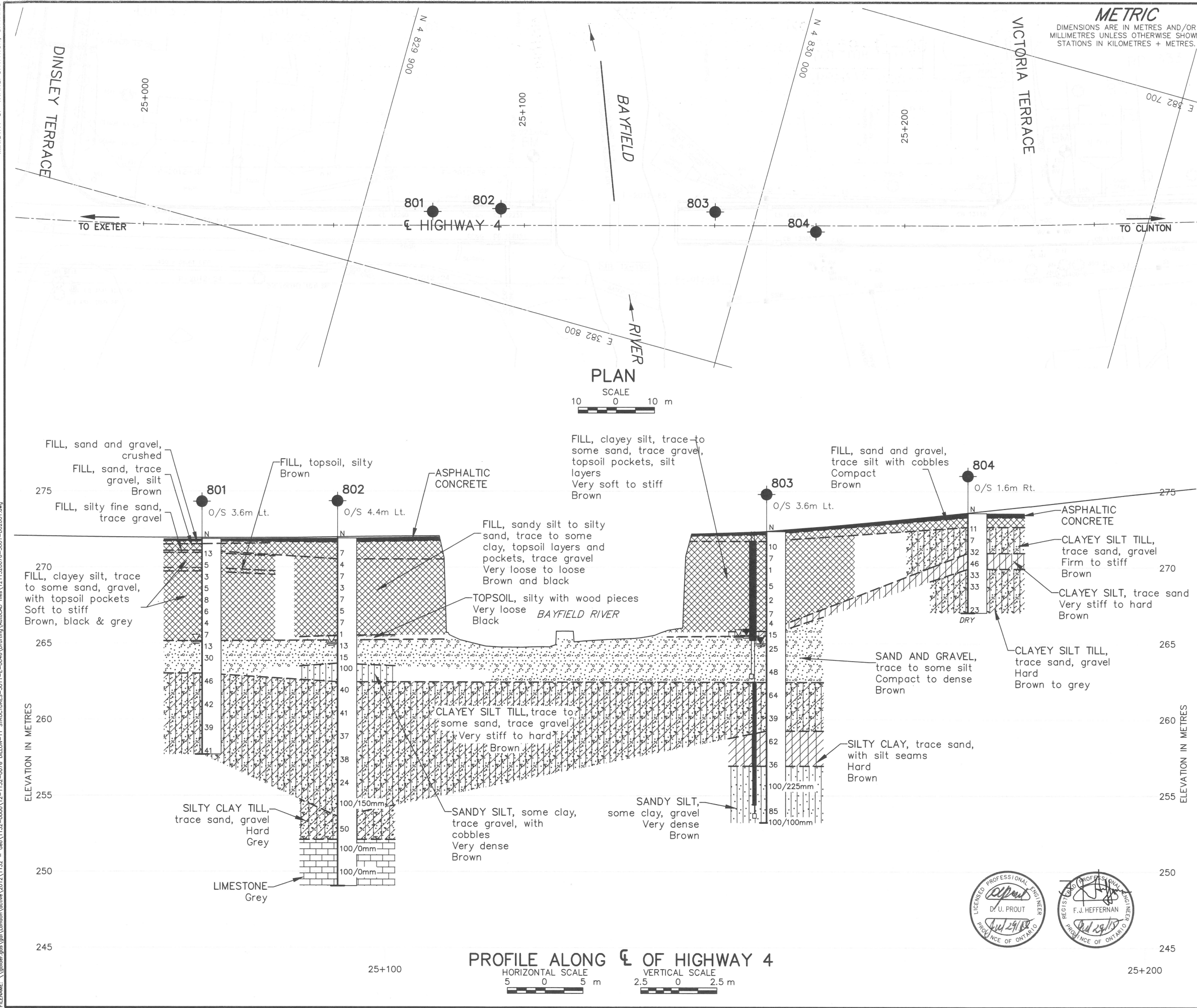
TITLE

## KEY PLAN



PROJECT No. 12-1132-0076		FILE No. 1211320076-3001-F02001	
CADD	LMK	Dec. 15/14	SCALE AS SHOWN REV. 0
CHECK			

**FIGURE 1**



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

CONT No.  
WP No. 3059-11-00

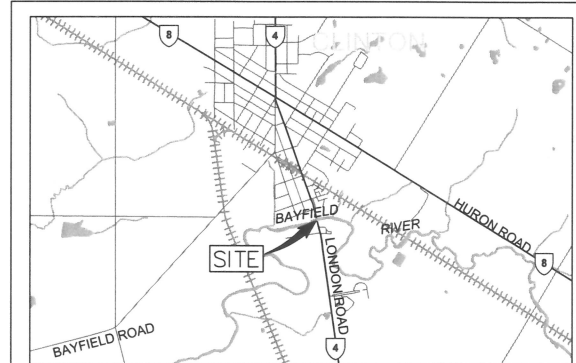


**BAYFIELD RIVER BRIDGE**  
HIGHWAYS 401, 4 AND 21 STRUCTURAL  
REPLACEMENTS  
BOREHOLE LOCATIONS AND SOIL STRATA

SHEET



**Golder Associates Ltd.**  
LONDON, ONTARIO, CANADA



KEY PLAN

**LEGEND**

- Borehole - Current Investigation
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- ⬮ Seal
- ⬮ Standpipe
- ⬮ WL upon completion of drilling
- ⬮ WL in Piezometer (February 19, 2015)
- DRY Borehole dry during drilling

No.	ELEVATION	CO-ORDINATES (MTM ZONE 10)	
		NORTHING	EASTING
801	271.87	4 829 917.9	382 781.6
802	271.91	4 829 934.9	382 776.1
803	272.36	4 829 989.5	382 761.8
804	273.57	4 830 016.4	382 759.7

**NOTES**

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

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

**REFERENCE**

Base plans provided in digital format by Dillon.



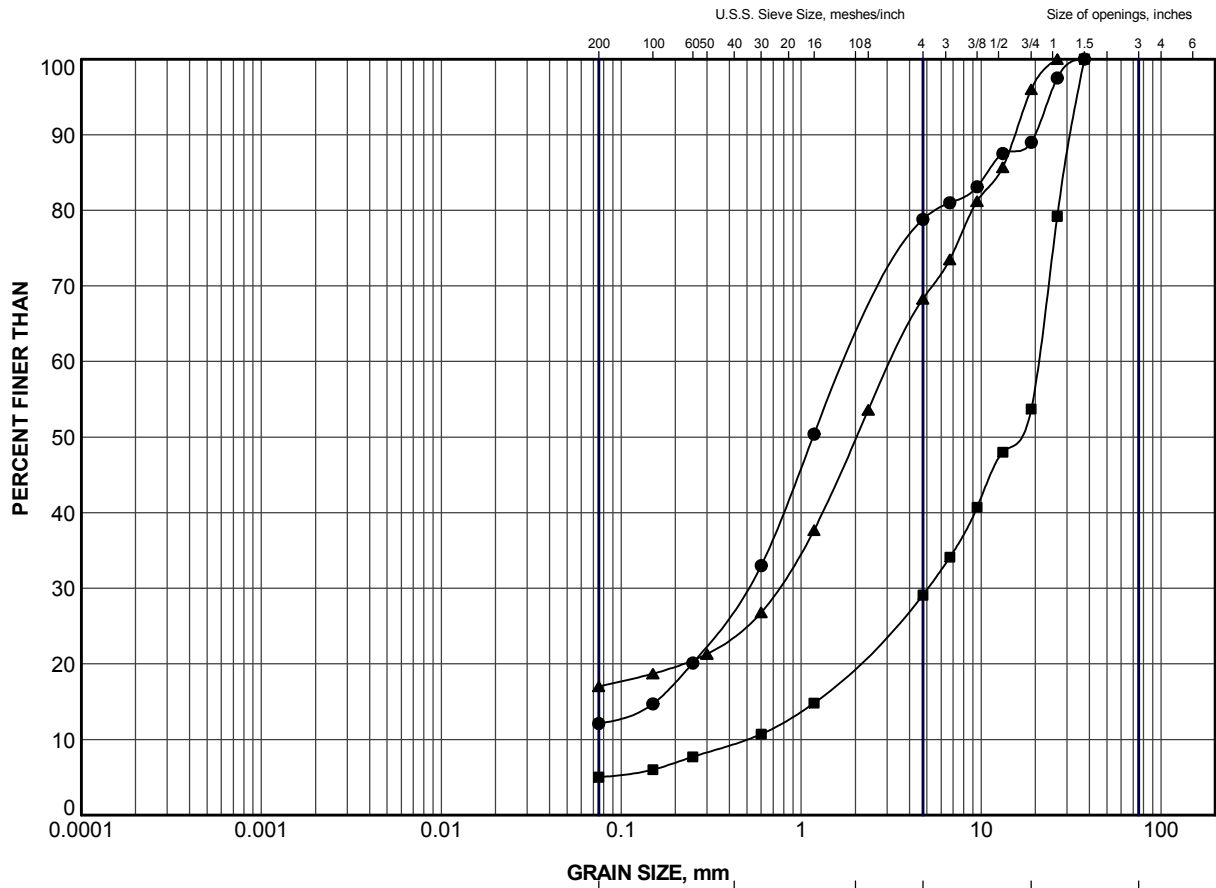
NO.	DATE	BY	REVISION
Geocres No.	40P12-33		
HWY.	4	PROJECT NO.	12-1132-0076
SUBM'D.	NAG	CHKD.	NAG
DRAWN:	LMK\WDF	CHKD.	APPD.
DIST.		DATE:	Dec. 24/14
SITE:	12-195	DWG.	1



# APPENDIX A

## Laboratory Test Data





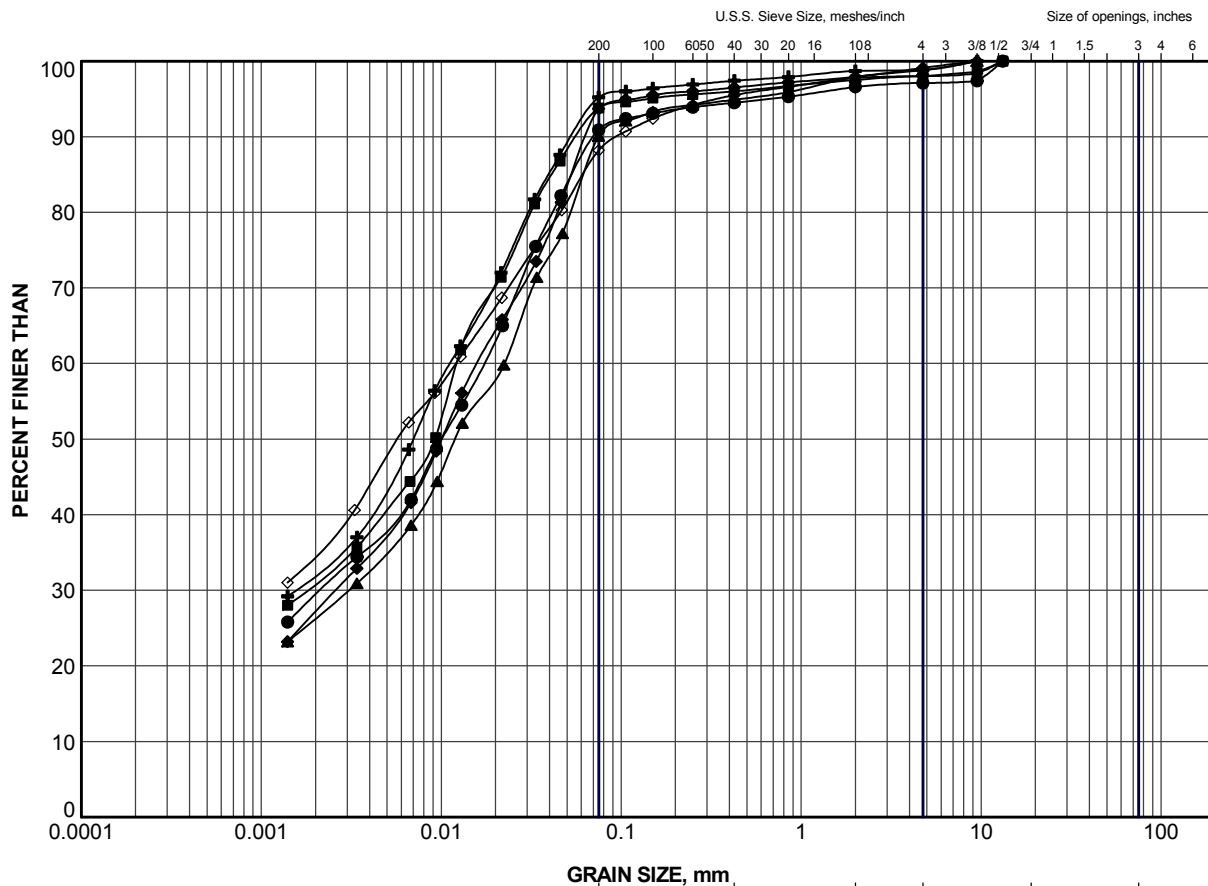
CLAY AND SILT	SAND SIZE, mm					Cobble Size
	fine	medium	coarse	fine	coarse	
	SAND SIZE			GRAVEL SIZE		

#### LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	801	10	264.0
■	802	9	264.8
▲	803	9	264.7

PROJECT HIGHWAY 4 / BAYFIELD RIVER BRIDGE REPLACEMENT HIGHWAYS 401, 4 AND 21 STRUCTURAL REPLACEMENTS GWP 3059-11-00			
TITLE <b>GRAIN SIZE DISTRIBUTION SAND AND GRAVEL</b>			
PROJECT No. 12-1132-0076		FILE No. 1211320076-3001-F020A2	
DRAWN	LMK	Dec 12/14	SCALE N/A REV.
CHECK			<b>FIGURE A-2</b>





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

### LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	801	12	261.0
■	801	14	257.9
▲	802	12	261.9
+	802	15	257.4
◆	803	12	260.1
◇	804	7	267.2

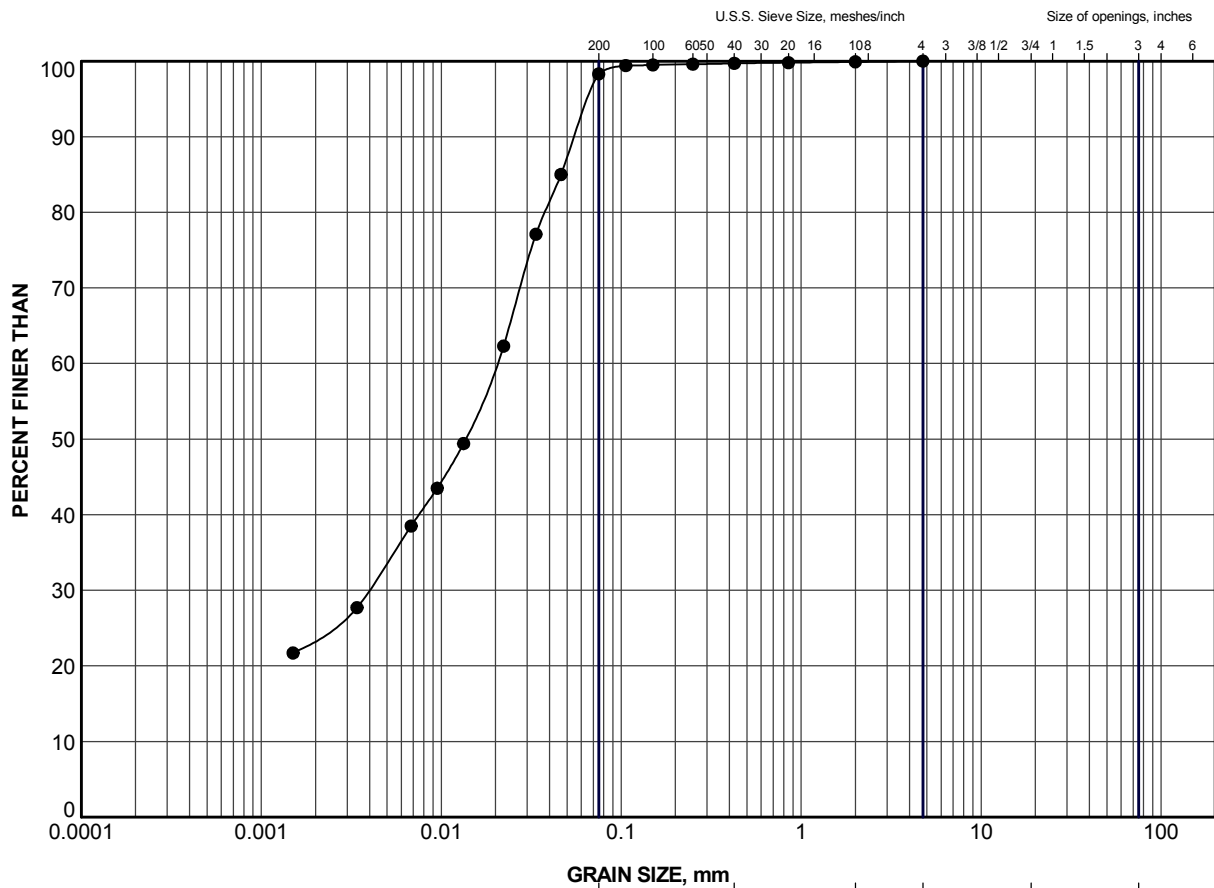
PROJECT  
HIGHWAY 4 / BAYFIELD RIVER BRIDGE REPLACEMENT  
HIGHWAYS 401, 4 AND 21 STRUCTURAL REPLACEMENTS  
GWP 3059-11-00

### GRAIN SIZE DISTRIBUTION CLAYEY SILT TILL



PROJECT No.	12-1132-0076	FILE No. 1211320076-3001-F020A3
DRAWN	LMK	Dec 12/14
CHECK		
SCALE	N/A	REV.

**FIGURE A-3**



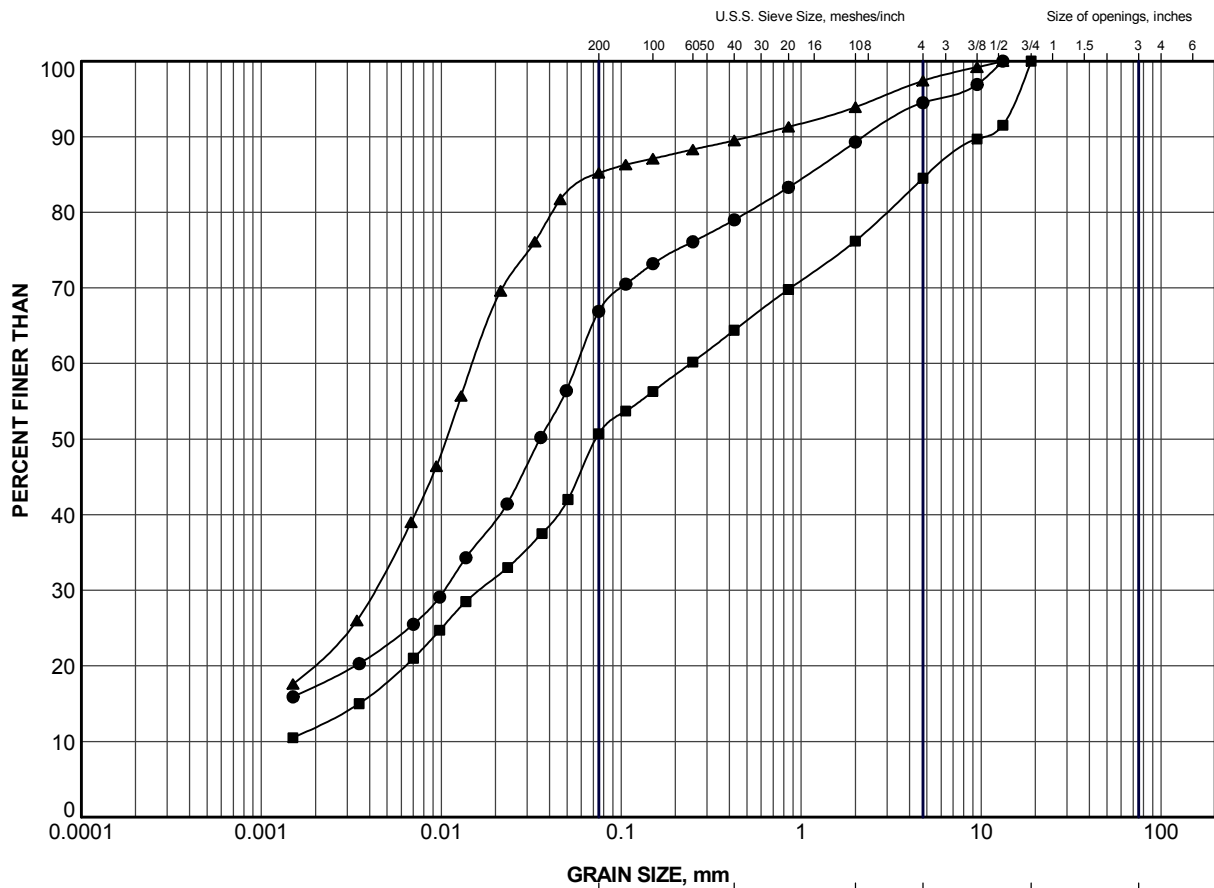
GRAIN SIZE, mm						
CLAY AND SILT	fine	medium	coarse	fine	coarse	Cobble Size
	SAND SIZE			GRAVEL SIZE		

### LEGEND

SYMBOL	BOREHOLE	SAMPLE	DEPTH (m)
●	804	4	270.3

PROJECT			
HIGHWAY 4 / BAYFIELD RIVER BRIDGE REPLACEMENT HIGHWAYS 401, 4 AND 21 STRUCTURAL REPLACEMENTS GWP 3059-11-00			
TITLE			
GRAIN SIZE DISTRIBUTION CLAYEY SILT			
PROJECT No.		12-1132-0076	
FILE No.		1211320076-3001-F020A4	
SCALE		N/A	
REV.			
DRAWN	LMK	Dec 12/14	
CHECK			
		FIGURE A-4	






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

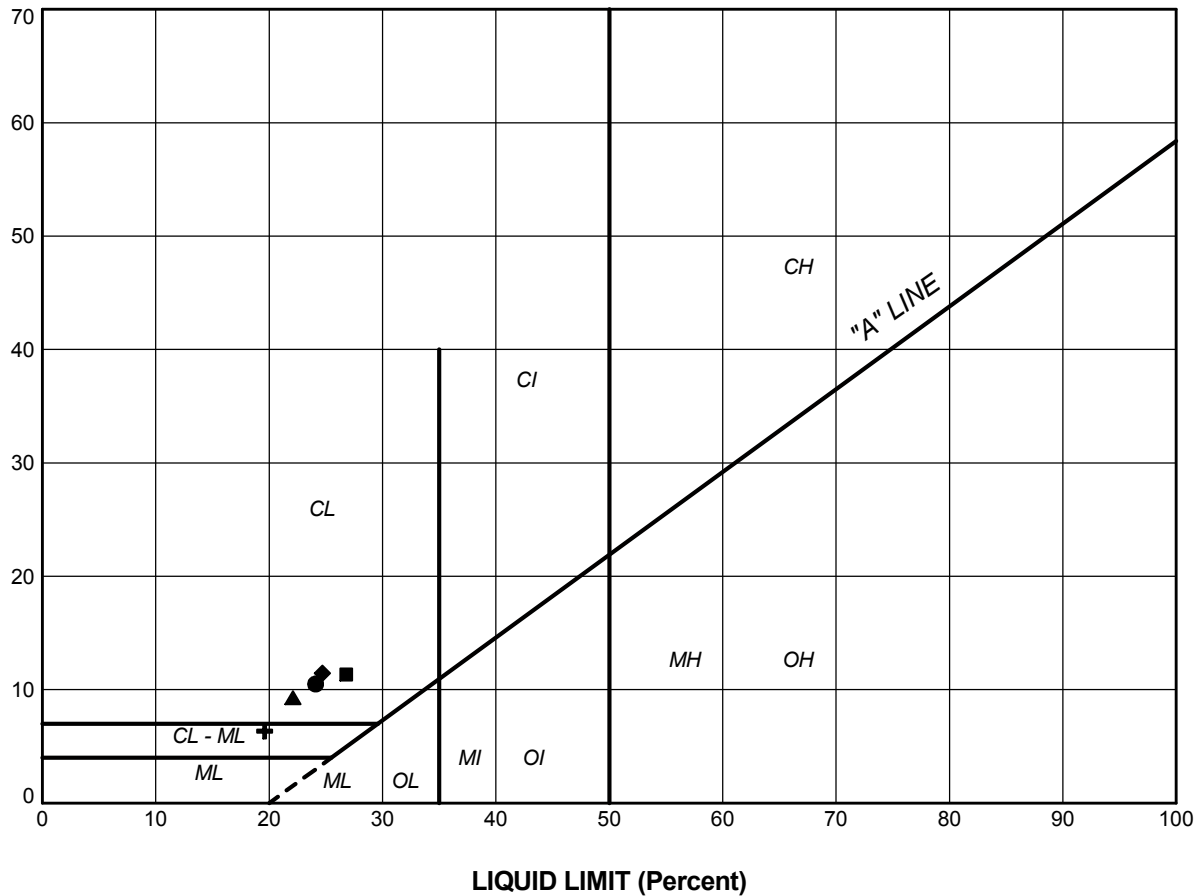
#### LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	802	11	263.3
■	803	15	255.6
▲	803	16	254.0

PROJECT HIGHWAY 4 / BAYFIELD RIVER BRIDGE REPLACEMENT HIGHWAYS 401, 4 AND 21 STRUCTURAL REPLACEMENTS GWP 3059-11-00				
TITLE <b>GRAIN SIZE DISTRIBUTION</b> <b>SANDY SILT</b>				
PROJECT No. 12-1132-0076		FILE No. 1211320076-3001-F020A5		
DRAWN	LMK	Dec 12/14	SCALE	N/A
CHECK			REV.	
			<b>FIGURE A-5</b>	




PLASTICITY INDEX (Percent)



### LEGEND

SYMBOL	BOREHOLE	SAMPLE	LL(%)	PL(%)	PI
●	802	15	24.1	13.6	10.5
■	803	4	26.8	15.5	11.3
▲	803	12	22.1	12.8	9.3
+	803	16	19.6	13.3	6.3
◆	804	7	24.7	13.3	11.4

PROJECT			
HIGHWAY 4 / BAYFIELD RIVER BRIDGE REPLACEMENT HIGHWAYS 401, 4 AND 21 STRUCTURAL REPLACEMENTS GWP 3059-11-00			
TITLE			
PLASTICITY CHART			
PROJECT No. 12-1132-0076		FILE No. 1211320076-3001-F020A6	
DRAWN	LMK	Dec 12/14	SCALE N/A REV.
CHECK			
			FIGURE A-6

At Golder Associates we strive to be the most respected global group of companies specializing in ground engineering and environmental services. Employee owned since our formation in 1960, we have created a unique culture with pride in ownership, resulting in long-term organizational stability. Golder professionals take the time to build an understanding of client needs and of the specific environments in which they operate. We continue to expand our technical capabilities and have experienced steady growth with employees now operating from offices located throughout Africa, Asia, Australasia, Europe, North America and South America.

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