January 21, 2013

FOUNDATION INVESTIGATION AND DESIGN REPORT

ROADWAY PROTECTION
REHABILITATION OF AIDIE CREEK BRIDGE
SITE 47-023 ON HIGHWAY 573
TOWNSHIP OF SAVARD, ONTARIO
MINISTRY OF TRANSPORTATION, ONTARIO
GWP 5302-05-00

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REPORT

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1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by Morrison Hershfield Limited (MH) on behalf of the Ministry of Transportation, Ontario (MTO) to provide foundation engineering services for the temporary roadway protection associated with the rehabilitation of the Highway 573 bridge crossing Aidie Creek in the Township of Savard, north of Charlton, Ontario.

The Terms of Reference and the Scope of Work for the foundation investigation are outlined in MTO's Request for Proposal (RFP), dated October 2011, and Request for Clarification Letter, dated December 12, 2011. Golder's proposal P1-1191-0032, dated December 2011, for foundation engineering services associated with this project is contained in Section 6.8 of MH's Technical Proposal that forms part of the Consultant's Agreement Number 5011-E-0009 for this project. The work has been carried out in accordance with Golder's Supplementary Specialty Quality Control Plan for foundation engineering services for this project, dated March 22, 2012. The General Arrangement (GA) drawing for the bridge was provided to Golder by MH.

The purpose of this investigation is to establish the subsurface conditions within the vicinity of the proposed roadway protection for the Aidie Creek 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 MH. The approximate location of the Highway 573 Bridge over Aidie Creek is shown on the Key Plan on Drawing 1.

2.0 SITE DESCRIPTION

The site is situated in the Township of Savard on Highway 573 crossing Aidie Creek, approximately 10 km south of the junction with Highway 11. The bridge was constructed around 1944 and last rehabilitated in 2008. The single-span rigid frame structure is 20 m long and 7 m wide, constructed of reinforced cast-in-place concrete. Based on the GA drawing provided to us by MH, the bridge abutments are supported on footings founded on bedrock on the south abutment and firm clayey silt on the north abutment.

In general, the topography in the vicinity of the bridge is flat with Aidie Creek located about 4 m below the bridge deck. The banks of the river and the approach embankment side slopes are vegetated with grass. The area in the vicinity of the site is landscaped with trees. The surrounding land is mainly used for recreational activities, with grass and tree cover extending beyond the limits of the site. The surface of the roadway at the bridge is at about Elevation 269 m and at the time of the subsurface investigation the creek water level was at about Elevation 264.8 m.

3.0 INVESTIGATION PROCEDURES

The fieldwork for the investigation associated with the proposed temporary roadway protection for the rehabilitation of the Aidie Creek Bridge was carried out between May 29 and 31, 2012. A total of four (4) boreholes were advanced as part of the investigation, one each at the north and south approach embankments (Boreholes AC-2 and AC-4, respectively) and one each at the north and south abutments (Boreholes AC-3 and AC-1, respectively), at approximately the locations shown on Drawing 1.

The field investigation was carried out using a track-mounted D-50 drilling rig supplied and operated by Walker Drilling of Utopia, Ontario.

The boreholes were advanced through the overburden using 108 mm inside diameter (I.D.) 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 (O.D.) 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.

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. All boreholes were backfilled with bentonite upon completion in accordance with Ontario Regulation 903-Wells (as amended).

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 11 centerline were installed by MH 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 MH 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 MH. 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		
AC-1	5304839.6	378227.0	268.8	4.0
AC-2	5304886.2	378230.4	268.6	14.3
AC-3	5304873.2	378226.4	268.9	12.8
AC-4	5304826.8	378222.9	268.5	3.4

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

Published literature[1] indicates that the site is located in the transition zone between the Western Abitibi Subprovince of the Superior Province (to the north) and the Huronian Supergroup (to the south). The bedrock geology follows the river valley and consists of mafic metavolcanic rock (Geology of Ontario, OGS Special Volume 4) [1].

Terrain mapping by the Ontario Geological Survey[2], describes the soils in the vicinity of the site as silty colluvial slopewash and debris creep sheet with minor talus.

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. The results of the laboratory tests are also presented 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.

In general, the subsurface stratigraphy encountered at the site consists of pavement structure (asphalt underlain by embankment fill comprised of gravelly sand and clayey silt) underlain by alternating layers of clayey silt, sand and silt, and silt. Refusal to further auger and split-spoon sampling was recorded at a depth of 4 m or less in the two boreholes advanced at the south abutment/approach embankment.

4.2.1 Asphalt

Approximately 30 mm of asphalt was encountered at ground surface in all boreholes. In Boreholes AC-2 to AC-4 between approximately 100 mm and 240 mm of gravelly sand fill was encountered underlying the surficial asphalt, in turn underlain by a 100 mm thick layer of asphalt. The ground surface at these boreholes ranges from Elevation 268.9 m to 268.5 m.

4.2.2 Fill

A layer of fill comprised of brown, gravelly sand to sand and gravel containing trace silt, between 0.4 m and 3.0 m thick was encountered underlying the asphalt in all of the boreholes. The top of the fill was encountered between Elevation 268.7 m and 268.2 m. In the approach embankment Boreholes AC-2 and AC-4, between 1.5 m and 1.1 m, respectively, of brown to grey clayey silt fill containing trace sand was encountered underlying the gravelly

[1] Geology of Ontario, 1991. Ontario Geological Society Special Volume 4, Part 1. Ministry of Northern Development and Mines, Ontario.
[2] Northern Ontario Engineering Geology Terrain Study, Ontario Geological Society, Map 5020 and 5021.

sand to sand and gravel fill. The top of the clayey silt fill was encountered at Elevation 267.8 m and 267.6 m at the respective boreholes.

SPT 'N'-values measured within the gravelly sand to sand and gravel fill range from 5 blows to 29 blows per 0.3 m of penetration, indicating a loose to compact relative density. SPT 'N'-values measured within the clayey silt fill range from 4 blows to 9 blows per 0.3 m of penetration, indicating a firm to stiff consistency.

The grain size distribution for one sample of the sand and gravel fill is shown on Figure B-1 in Appendix B.

An Atterberg limits test carried out on a sample of the clayey silt fill yielded a liquid limit of about 34 per cent, a plastic limit of about 18 per cent and a plasticity index of about 16 per cent. The result of the Atterberg limits testing is shown on the plasticity chart on Figure B-2 in Appendix B and indicates that the fill consists of clayey silt of low plasticity.

The natural water content measured on two samples of the gravelly sand to sand and gravel fill is about 2 per cent. The natural water content measured on two samples of the clayey silt fill is about 25 per cent and 28 per cent.

4.2.3 Peat

A 0.4 m and 0.1 m thick layer of black amorphous peat was encountered underlying the fill in Boreholes AC-1 and AC-4, respectively. The top of the peat was encountered at Elevation 266.0 m and 266.5 m at the respective boreholes.

The natural water content measured on a sample of the peat is about 40 per cent.

4.2.4 Clayey Silt

A deposit comprised of individual or alternating layers of clayey silt and sand and silt were encountered below the fill in Boreholes AC-2 and AC-3 and underlying the peat in Boreholes AC-1 and AC-4. The clayey silt layers are brown to grey and contain trace sand. The layers are between 0.2 m and 4.0 m thick and the uppermost layer was encountered between Elevation 265.6 m and 266.4 m. In Borehole AC-2 the top layer of clayey silt contained trace organics.

SPT 'N'-values measured within the clayey silt portion of the deposit ranges from 0 blows (weight of hammer) to 11 blows per 0.3 m of penetration. In situ field vane testing carried out within this stratum measured undrained shear strengths ranging from about 22 kPa to 47 kPa. The SPT 'N'-values together with the field vane test results suggest that the deposit has a soft to firm consistency.

Atterberg limits testing carried out on four (4) samples of the clayey silt deposit yielded liquid limits ranging from about 23 per cent to 32 per cent, plastic limits ranging from about 15 per cent to 19 per cent and plasticity indices ranging from about 8 per cent to 14 per cent. The results of the Atterberg limits testing are shown on the plasticity chart on Figure B-3 in Appendix B and indicate that the deposit consists of clayey silt of low plasticity.

The grain size distributions for two (2) samples of the clayey silt are shown on Figure B-4 in Appendix B.

The natural water content measured on five (5) samples of the clayey silt ranges between about 23 per cent and 34 per cent.

4.2.5 Sand and Silt

Sand and silt layers were encountered interlayered below the uppermost clayey silt layer in Boreholes AC-2 to AC-4. The sand and silt layers are brown to grey and contain trace to some clay. The layers are between 0.2 m and 2.5 m thick and the uppermost layer was encountered at Elevation 264.0 m in Boreholes AC-2 and AC-3 and at Elevation 266.1 m in Borehole AC-4. In Borehole AC-4, the bottom of this deposit was defined by refusal to split-spoon and auger penetration.

SPT 'N'-values measured within the sand and silt deposit range from 0 blows (weight of hammer) to 12 blows per 0.3 m of penetration and a SPT 'N'-value of 14 blows per 0.15 m of penetration, indicating a very loose to compact relative density.

The grain size distributions of two (2) samples of the sand to silt layers are shown on Figure B-5 in Appendix B.

The natural water content measured on samples of the sand to silt ranges between about 13 per cent and 24 per cent.

4.2.6 Silt

A deposit of grey silt containing some clay and trace to some sand was encountered underlying the clayey silt deposit in Boreholes AC-2 and AC-3. The top of the silt deposit was encountered at Elevation 257.2 m and Elevation 255.5 m and the thickness of the deposit is 1.1 m and 1.2 m thick at the respective boreholes. This silt deposit was not fully penetrated in either of these boreholes.

SPT 'N'-values measured within the silt deposit range from 4 blows to 8 blows per 0.3 m of penetration, indicating a loose relative density.

The grain size distribution of a sample of the silt is shown on Figure B-6 in Appendix B.

The natural water content measured on a sample of the silt is about 26 per cent.

4.2.7 Gravel

A 0.2 m thick layer of pink gravel was encountered underlying the clayey silt deposit in Borehole AC-1. The top of the gravel deposit was encountered at Elevation 265.0 m and the thickness of the deposit is 0.2 m. Refusal to split-spoon and auger advancement was encountered within this layer at a depth of 4.0 m below ground surface.

An SPT 'N'-value of 22 blows per 0.15 m of penetration was measured in this layer although some of the blows did not achieve any penetration, in this case suggesting a compact relative density.

4.2.8 Refusal

In Boreholes AC-1 and AC-4, refusal to further auger advancement and split-spoon penetration was encountered at depths of 4.0 m and 3.4 m, respectively, below ground surface, corresponding to Elevation 264.8 m and 265.1 m. A DCPT was advanced 1.0 m north of Borehole AC-1. Refusal was encountered at a 4.1 m depth below ground surface, corresponding to Elevation 264.7 m.

4.2.9 Groundwater Conditions

In general, the soil samples obtained were moist to wet. Groundwater levels were measured in the open boreholes upon completion of drilling and are summarized below.

Borehole	Depth to Groundwater Level Below Ground Surface (m)	Groundwater Elevation (m)
AC-1	3.8	265.0
AC-2	3.7	264.9
AC-3	3.4	265.5
AC-4	3.1	265.4

The water level of the river at the location of the bridge was measured at Elevation 264.8 m in May 2012, and the high water level is Elevation 267.2 m, as provided by MH.

Groundwater levels encountered in the boreholes during and shortly after drilling may not be representative of static levels since the groundwater levels in the boreholes may not have stabilized on completion of drilling. Groundwater and river water levels in the area are subject to seasonal fluctuations and to fluctuations after precipitation events and snowmelt.

5.0 CLOSURE

The field personnel supervising the drilling program was Mr. Ed Savard. This report was prepared by Mr. Evan Childerhose, P.Eng. The technical aspects were reviewed by 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.

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PART B

FOUNDATION INVESTIGATION REPORT
ROADWAY PROTECTION
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SITE 47-023 ON HIGHWAY 573
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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 roadway protection for the bridge rehabilitation 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 MH to provide recommendations on temporary roadway protection for the rehabilitation of the Highway 573 bridge over Aidie Creek in the Township of Savard, Ontario. 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 existing structure consists of a rigid frame single-span, 20 m long and 7 m wide reinforced cast-in-place concrete bridge. The roadway surface at the bridge is at about Elevation 269 m and the water level in Aidie Creek was about 4 m below the bridge deck, at about Elevation 264.8 m in May 2012.

The subsurface conditions at the south approach embankment, at Boreholes AC-1 and AC-4, generally consist of 2.0 m and 2.8 m of pavement structure (asphalt and embankment fill comprised of gravelly sand to sand and gravel and clayey silt), underlain by a stratum of peat 0.1 m and 0.4 m thick and a clayey silt deposit 0.3 m and 0.6 m thick. A 1.0 m thick deposit of sand and silt underlies the clayey silt in Borehole AC-4 and a 0.2 m thick layer of gravel underlies the clayey silt in AC-1. Refusal was encountered at depths of from 4.0 m to 3.4 m below ground (pavement) surface. The non-stabilized water level upon completion of drilling was measured at depths of 3.8 m and 3.1 m below the roadway surface corresponding to Elevation 265.0 m and 265.4 m, respectively.

The subsurface conditions at the north approach embankment, at Boreholes AC-2 and AC-3, generally consist of between 2.3 m and 3.2 m of pavement structure (asphalt and embankment fill comprised of gravelly sand and clayey silt), underlain by a deposit of alternating layers of clayey silt and sand and silt. The clayey silt layers range between 0.2 m and 4.0 m thick and the sand and silt layers range between 0.3 m and 2.5 m thick. The total thickness of this deposit is 10.8 m and 8.5 m thick at the respective boreholes. Underlying the clayey silt/sand and silt deposit is a stratum of silt between 1.2 m and 1.1 m thick to the termination of these Boreholes at Elevation 254.3 m and 256.1 m, respectively. The non-stabilized water level upon completion of drilling was measured at depths of 3.7 m and 3.4 m below the roadway surface, corresponding to Elevation 264.9 m and 265.5 m, respectively.

Excavations will be required to expose the existing abutments. Based on the GA drawing provided by MH, the underside of the existing abutments footings is about Elevation 263 m at the south abutment and Elevation 262 m at the north abutment, corresponding to about 6 m and 7 m below the roadway surface.

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 the excavations, groundwater may be encountered, as the unstabilized water level at the approach embankments is higher than the underside of the existing abutment foundations. The groundwater level is subject to fluctuations and the depth of excavation below the groundwater will depend on the time of year that the rehabilitation works are carried out. Also, perched groundwater may be present within the granular fill layers. Pumping from properly filtered sumps located at the base of the excavations will likely be required to provide groundwater control, but should be located outside of the actual excavation limits required for the rehabilitation works. Surficial water seepage into the excavations should be expected and will be greater during periods of sustained precipitation. Therefore, 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 soft to stiff clayey silt, very loose to compact sand and silt, loose silt and peat 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 (1H:1V). Flatter side slopes may be necessary in areas with saturated or loose granular soils.

6.3 Temporary Excavation Support Systems

Temporary protection systems are required to support embankment fills and native sand to sandy silt, silt and clayey silt deposits during rehabilitation of the abutments. Assuming the depth of excavation is required only to the top of the footings and assuming 1 m thick footings, the GA drawing suggests that the excavations would extend to about Elevation 264 m and Elevation 263 m at the south and north abutments, respectively, approximately 5 m and 6 m below the existing roadway surface. 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. If soldier piles and lagging is selected, pile installation should be in accordance with OPSS 903 (Deep Foundations). At the south abutment/approach embankment, the H-piles will likely have to be installed into pre-drilled sockets into the refusal stratum (inferred to be bedrock) to provide for adequate toe embedment for lateral support. A comparison of the different applicable temporary support systems based on advantages, disadvantages, risks and relative costs is provided in Table 1 following the text of this report.

Support to the system could be in the form of rakers or anchors. The pull-out resistance (P_{ar} in kPa/m), for tremie grouted anchors in cohesionless soils can be estimated from the following equation as per the Canadian Foundation Engineering Manual (CFEM, 2006):

$$P_{ar} = \sigma'_z A_s L_s \alpha_g$$

where
σ'_z = effective vertical stress at the midpoint of the load carrying length (in kPa)
A_s = effective unit surface area of the anchor bond zone (m^2)
L_s = effective length of the anchor bond zone (m)
α_g = anchorage coefficient dependent on the soil type and conditions (as given in Table 26.5 of CFEM, 2006)

The computation of the pull-out resistance (P_{ar} in kPa/m) in stiff to very hard clay soils can be estimated from the following equation (CFEM, 2006):

$$P_{ar} = \alpha_c A_s L_s s_u$$

where α_c = reduction factor related to the undrained shear strength (as given on Figure 26.17 of CFEM, 2006)

A_s = effective unit surface area of the anchor bond zone (m^2)

L_s = effective length of the anchor bond zone (m)

α_g = average undrained shear strength of the clay over the anchor length (kPa)

On the north abutment/approach embankment, grouted soil anchors would have their fixed anchor length formed within the alternating soft to stiff clayey silt and loose sand and silt zones. Typically, anchors are not suitable in soft to firm clays, however in this case assuming that the nominal diameter of the anchor is 150 mm, the recommended ultimate bond stress for design of anchors is approximately 25 kPa/m. It should be noted, however, that the anchor capacity will not increase for bond lengths greater than about 8 m, and therefore, the length of the bond zone should be restricted to 8 m.

On the south abutment/approach embankment, grouted rock anchors could be installed into bedrock below about Elevation 264.8 m. The fixed anchor length (bond zone) should be a minimum of 2 m long. Assuming a minimum grout strength of 50 MPa (as given in the Contract Drawings provided by MH) the ultimate bond stress of 1,700 kPa/m may be used. All anchors should be proof tested.

The above ultimate bond stress values are unfactored values. . In accordance with the Canadian Highway Bridge Design Code (CHBDC, 2006) Section 6.6.2, a factor of 0.4 should be applied to the ultimate resistance to obtain the factored geotechnical resistance at Ultimate Limit States (ULS).

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 CFEM (2006) Subsection 26.16, structural loads may be carried by either direct underpinning of the foundations or by providing additional support to the excavation face.

It is recommended that the existing structure, as well as the temporary support structure, be monitored for settlement and lateral movement while excavation, pile/sheet driving or other construction activities are carried out at the site. The type, location and number of settlement monitoring points and lateral movement points and frequency of readings should be developed by the bridge designer to ensure that the magnitude of tolerable movements is not exceeded.

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 rakers or anchors, 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	K_a	=	active coefficient of earth pressure
	H	=	the depth of the excavation at any point (m)
	γ	=	soil unit weight (kN/m^3)
	q	=	surcharge for traffic and other loading (kN/m^2)

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:

	p	=	$Ka(0.65 \gamma H + q)$
where	K_a	=	active coefficient of earth pressure
	H	=	the total depth of the excavation (m)
	γ	=	soil unit weight (kN/m^3)
	q	=	surcharge for traffic and other loading (kN/m^2)

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

	p	=	0 at ground surface increasing linearly to a depth of 0.25 H_T to:
	p	=	$\gamma H_T - m\, 4\, S_u$ at 0.25 H_T and from 0.25 H_T to H_T below ground surface
where	H_T	=	the total depth of the excavation (m)
	γ	=	soil unit weight (kN/m^3)
	q	=	surcharge for traffic and other loading (kN/m^2)
	m	=	0.4 if an extensive soft clay layer underlies the excavation
			1.0 if more resistant layer is present at the excavation base
	S_u	=	undrained shear strength (kN/m^2).

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^2)	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 Fill	0.38	0.55	2.7	27	17	-
Clayey Silt*	0.38	0.55	2.7	27	17	-
	1.0	1.0	1.0	0	17	20
Sand and Silt/Silt	0.36	0.53	2.8	28	18	-

Note: *Design Temporary Protection System on the more conservative (higher) earth pressure value.

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.

7.0 CLOSURE

This report was prepared by Mr. Evan Childerhose, 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 and conducted an independent quality control audit of the report.

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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
OPSS 903	Construction Specification for Deep Foundations

Ontario Water Resources Act:

Ontario Regulation 372/97	Amendment to Ontario Regulation 903, Wells (as Amended)

Table 1: Evaluation of Temporary Support System Alternatives

Support System Type	Rank	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Driven Steel Sheet Piling	1	■ Straightforward installation ■ Suitable for north side system as adequate overburden thickness is available for embedment	■ Limited load bearing capacity in shallow soils and with stiff cohesive strata may be inadequate for tieback anchors for north side system ■ Varying depth to bedrock may not provide adequate protection at/along base/bottom of system at bedrock contact on south side ■ Likely not suitable for south side due to lack of adequate overburden thickness for adequate embedment - will require a tieback system of anchors or rakers	■ Lower cost of construction	■ Installation may be difficult or not possible if obstructions are encountered (in the fill)
Soldier Pile and Lagging Wall	2	■ Appropriate for installations in both shallow and deep overburden (can be used on north and south sides).	■ Additional installation effort required overall but is also a straight forward installation ■ Will require bedrock sockets on south side for adequate toe embedment ■ May require tieback anchors for both north and south side systems depending on depth of embedment into overburden (north side) or rock socket (south side)	■ Shallow refusal on south side requiring sockets into bedrock makes this less cost effective	■ Potential for loss of soil as lagging is installed

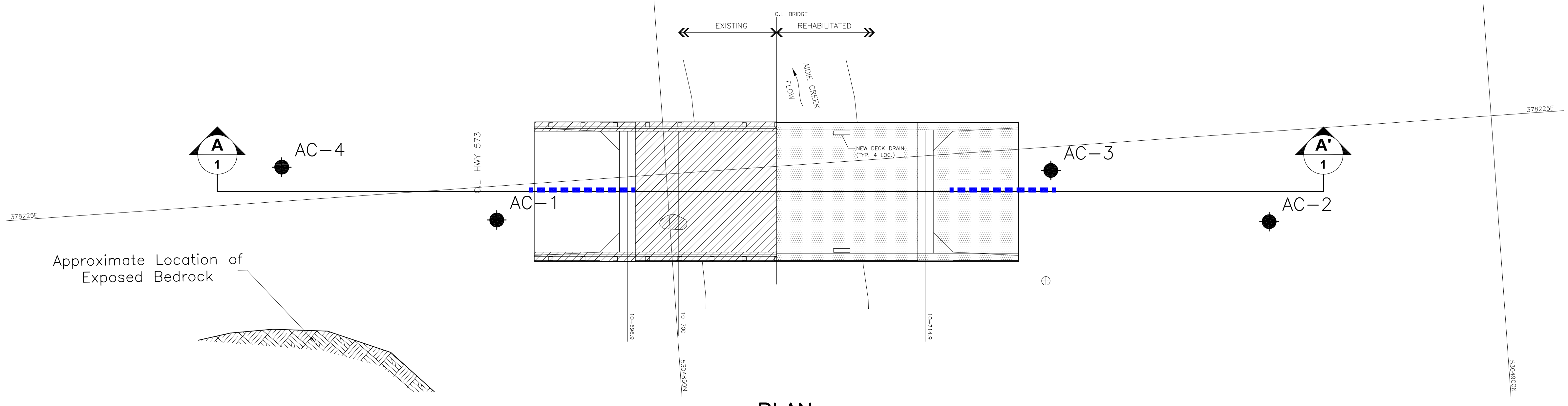
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

CONT No.
WP No. 5302-05-00

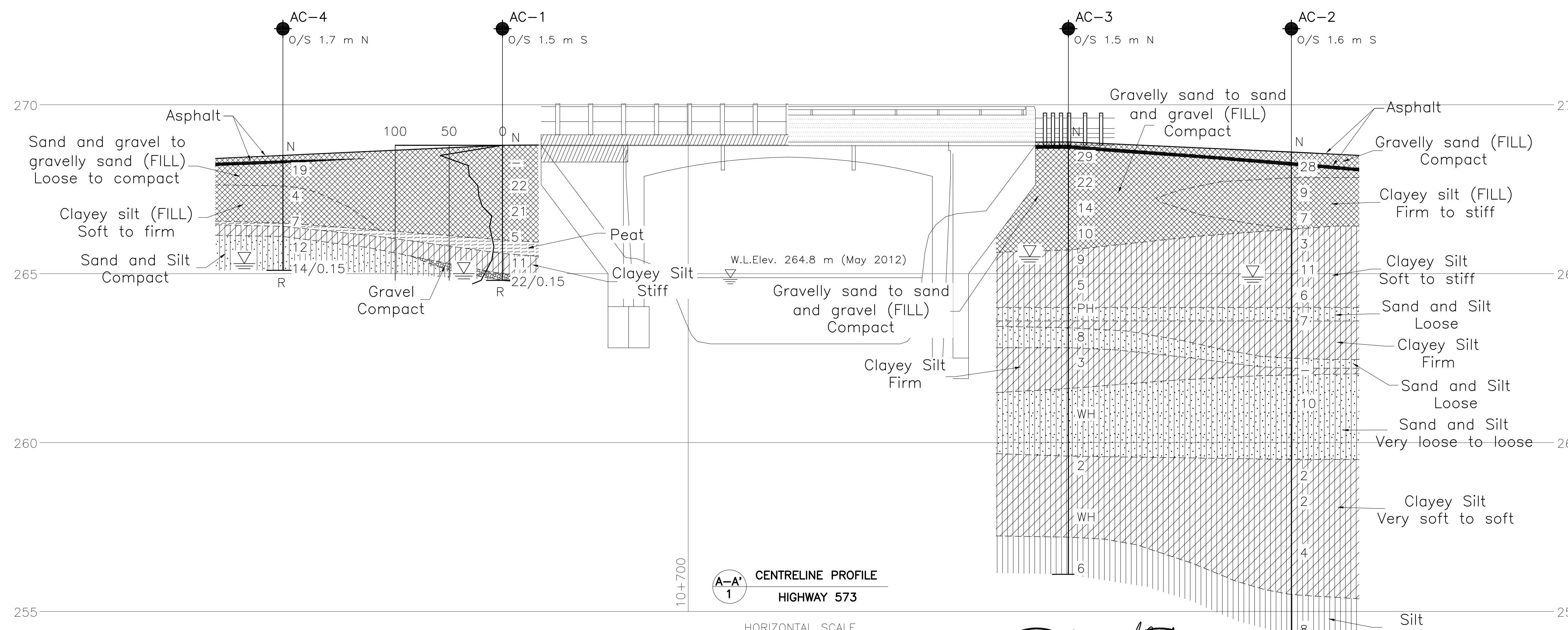
SHEET

AIDIE CREEK
HIGHWAY 573 BRIDGE 47-023
BOREHOLE LOCATIONS AND SOIL STRATA

Golder Associates Ltd.
SUDBURY, ONTARIO, CANADA



PLAN
SCALE
3 0 3 6 m

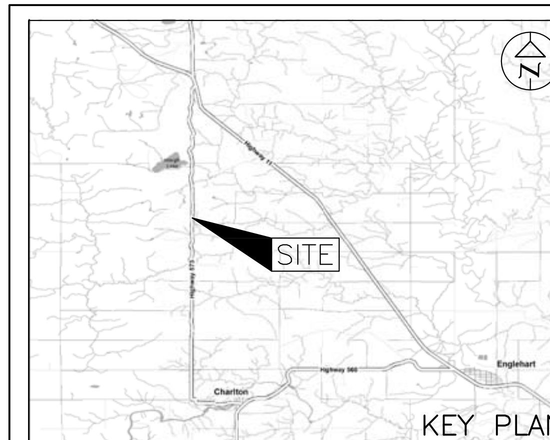


A-A' 1 CENTRELINE PROFILE
HIGHWAY 573

HORIZONTAL SCALE
3 0 3 6 m
1.5 0 1.5 3 m
VERTICAL SCALE

PROFESSIONAL ENGINEER
E.R. CHILDERHOSE
100128289
PROVINCE OF ONTARIO

PROFESSIONAL ENGINEER
J.M.A.COSTA
PROVINCE OF ONTARIO



LEGEND

(symbol)	Borehole – Current Investigation
N	Standard Penetration Test Value
16	Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
R	Refusal
▽	WL upon completion of drilling
▪▪▪▪▪	Temporary Protection System

BOREHOLE CO-ORDINATES

No.	ELEVATION	NORTHING	EASTING
AC-1	268.8	5304839.6	378227.0
AC-2	268.6	5304886.2	378230.4
AC-3	268.9	5304873.2	378226.4
AC-4	268.5	5304826.8	378222.9

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 Morrison Hershfield, drawing file nos. 47023-01.dwg, received July 23, 2012.

NO.	DATE	BY	REVISION

Geocres No. 41P-51

HWY. 573	PROJECT NO. 11-1191-0032		DIST.
SUBM'D. EC	CHKD.	DATE: DEC 2012	SITE: 47-023
DRAWN: TB	CHKD.	APPD.	DWG. 1

PLOT DATE: January 18, 2013
FILENAME: \\sud1-s-cad01\Golder\CAD\Projects\2011\11-1191-0032 3 Englehart Bridges HWY 573\47-023 Aidie Creek\11119100320A001_47-023.dwg

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, 60º 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^2 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 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

(b) Cohesive Soils
Consistency

	c_u, s_u kPa	c_u, s_u 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
υ	Poisson's ratio
σ	total stress
σ'	effective stress ($\sigma' = \sigma$ - u)
σ'_{vo}	initial effective overburden stress
σ_1, σ_2, σ_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

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

(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 $= \sigma'_p / \sigma'_{vo}$

(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 δ
c'	effective cohesion
c_u, s_u	undrained shear strength (ϕ = 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

WEATHERINGS STATE

Fresh: no visible sign of weathering

Faintly weathered: weathering limited to the surface of major discontinuities.

Slightly weathered: penetrative weathering developed on open discontinuity surfaces but only slight weathering of rock material.

Moderately weathered: weathering extends throughout the rock mass but the rock material is not friable.

Highly weathered: weathering extends throughout rock mass and the rock material is partly friable.

Completely weathered: rock is wholly decomposed and in a friable condition but the rock and structure are preserved.

BEDDING THICKNESS

Description	Bedding Plane Spacing
Very thickly bedded	Greater than 2 m
Thickly bedded	0.6 m to 2 m
Medium bedded	0.2 m to 0.6 m
Thinly bedded	60 mm to 0.2 m
Very thinly bedded	20 mm to 60 mm
Laminated	6 mm to 20 mm
Thinly laminated	Less than 6 mm

JOINT OR FOLIATION SPACING

Description	Spacing
Very wide	Greater than 3 m
Wide	1 m to 3 m
Moderately close	0.3 m to 1 m
Close	50 mm to 300 mm
Very close	Less than 50 mm

GRAIN SIZE

Term	Size*
Very Coarse Grained	Greater than 60 mm
Coarse Grained	2 mm to 60 mm
Medium Grained	60 microns to 2 mm
Fine Grained	2 microns to 60 microns
Very Fine Grained	Less than 2 microns

Note: * Grains greater than 60 microns diameter are visible to the naked eye.

CORE CONDITION

Total Core Recovery (TCR)
The percentage of solid drill core recovered regardless of quality or length, measured relative to the length of the total core run.

Solid Core Recovery (SCR)
The percentage of solid drill core, regardless of length, recovered at full diameter, measured relative to the length of the total core run.

Rock Quality Designation (RQD)
The percentage of solid drill core, greater than 100 mm length, recovered at full diameter, measured relative to the length of the total core run. RQD varied from 0% for completely broken core to 100% for core in solid sticks.

DISCONTINUITY DATA

Fracture Index
A count of the number of discontinuities (physical separations) in the rock core, including both naturally occurring fractures and mechanically induced breaks caused by drilling.

Dip with Respect to Core Axis
The angle of the discontinuity relative to the axis (length) of the core. In a vertical borehole a discontinuity with a 90^o angle is horizontal.

Description and Notes
An abbreviation description of the discontinuities, whether naturally occurring separations such as fractures, bedding planes and foliation planes or mechanically induced features caused by drilling such as ground or shattered core and mechanically separated bedding or foliation surfaces. Additional information concerning the nature of fracture surfaces and infillings are also noted.

Abbreviations

JN	Joint	PL	Planar
FLT	Fault	CU	Curved
SH	Shear	UN	Undulating
VN	Vein	IR	Irregular
FR	Fracture	K	Slickensided
SY	Stylolite	PO	Polished
BD	Bedding	SM	Smooth
FO	Foliation	SR	Slightly Rough
CO	Contact	RO	Rough
AXJ	Axial Joint	VR	Very Rough
KV	Karstic Void		
MB	Mechanical Break		

RECORD OF BOREHOLE No AC-1

 METRIC

PROJECT 11-1191-0032			
W.P. 5302-05-00	LOCATION N 5304839.6; E 378227.0		ORIGINATED BY EHS
DIST	HWY 573	BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers	COMPILED BY AC
DATUM GEODETIC	DATE May 29, 2012		CHECKED BY EC

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
268.8	GROUND SURFACE					
0.0	ASPHALT (30 mm)					
0.1	Sand and gravel to gravelly sand, trace silt (FILL) Loose to compact Brown Dry to moist	1	AS	-		
		2	SS	22		50 45 (5)
		3	SS	21		
		4	SS	5		
266.0	PEAT (Amorphous) Black Moist					
265.6 / 3.2	CLAYEY SILT, trace sand Stiff Grey Wet	5	SS	11	▽	
265.0 / 4.0	GRAVEL, some sand Compact Pink Wet	6	SS	22/0.15		
	END OF BOREHOLE SPLIT SPOON AND AUGER REFUSAL					

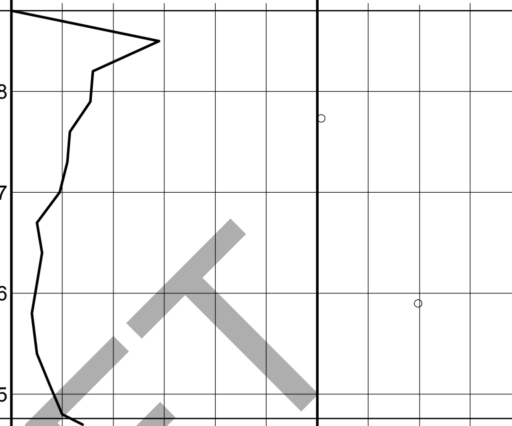
Elevation scale: 268, 267, 266, 265

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED — 20 40 60 80 100

PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L; WATER CONTENT (%) 20 40 60

UNIT WEIGHT γ kN/m³



Note:

1. Water level at a depth of 3.8 m below ground surface (Elev. 265.0 m) upon completion of drilling.

2. Advanced DCPT 1.0 m north of Borehole AC-1, refusal (hammer bouncing) at a depth of 4.1 m.

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

Golder Associates

RECORD OF BOREHOLE No AC-2

 METRIC

PROJECT 11-1191-0032
W.P. 5302-05-00
DIST ____ HWY 573
DATUM GEODETIC

LOCATION N 5304886.2; E 378230.4
BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers, NW Casing, Wash Boring
DATE May 30, 2012

ORIGINATED BY EHS
COMPILED BY AC
CHECKED BY EC

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
268.6	GROUND SURFACE				
0.0 / 268.3	ASPHALT (30 mm)	1	SS	28	
	Gravelly sand (FILL) Compact Brown Moist				
267.8 / 0.8	ASPHALT (100 mm)	2	SS	9	
	Gravelly sand (FILL) Compact Brown Moist	3	SS	7	
	Clayey silt, trace sand (FILL) Firm to stiff Brown to grey Moist				
266.3 / 2.3	CLAYEY SILT, trace organics, trace sand Soft to stiff Brown to grey Moist to wet	4	SS	3	
		5	SS	11	
		6	SS	6	
264.0 / 4.6	SAND and SILT, trace to some clay Loose Brown to grey Wet	7	SS	7	
263.6 / 5.0	CLAYEY SILT, trace sand Firm Brown to grey Wet				
262.5	SAND and SILT, trace to some clay Brown to grey Wet	8 a	TO	-	0 55 34 11
262.2		8 b			0 1 77 22
6.6	CLAYEY SILT, trace sand Brown to grey Wet				
	SAND and SILT, trace to some clay Very loose to loose Brown to grey Wet. 2.4 m of heave encountered at 9.1 m depth. Switched to NW casing at 9.1 m depth.	9	SS	10	
259.5 / 9.1	CLAYEY SILT, trace sand Very soft Brown to grey Wet	10	SS	2	
		11	SS	2	
		12	SS	4	
255.5 / 13.1	SILT, some sand, trace to some clay Loose Grey Wet	13	SS	8	0 14 75 11
254.3 / 14.3					

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; GROUND WATER CONDITIONS; ELEVATION SCALE (268–255); DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100; PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m³

Continued Next Page

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

SUD-MTO 001 11-1191-0032+BH09.GPJ GAL-MISS.GDT 26/09/12 DATA INPUT:

Golder Associates

PROJECT 11-1191-0032

RECORD OF BOREHOLE No AC-2

 METRIC

W.P. 5302-05-00 LOCATION N 5304886.2; E 378230.4 ORIGINATED BY EHS

DIST HWY 573 BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers, NW Casing, Wash Boring COMPILED BY AC

DATUM GEODETIC DATE May 30, 2012 CHECKED BY EC

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100	PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
	--- CONTINUED FROM PREVIOUS PAGE ---										
	END OF BOREHOLE Note: 1. Water level at a depth of 3.7 m below ground surface (Elev. 264.9 m) upon completion of drilling.										

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

SUD-MTO 001 11-1191-0032+BH09.GPJ GAL-MISS.GDT 26/09/12 DATA INPUT:

Golder Associates

RECORD OF BOREHOLE No AC-3

 METRIC

PROJECT	11-1191-0032		
W.P.	5302-05-00	LOCATION	N 5304873.2; E 378226.4
DIST		HWY	573
BOREHOLE TYPE	108 mm I.D. Continuous Flight Hollow Stem Augers		
DATUM	GEODETIC	DATE	May 31, 2012
ORIGINATED BY	EHS	COMPILED BY	AC
CHECKED BY	EC		

SOIL PROFILE: ELEV DEPTH	SOIL PROFILE: DESCRIPTION	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: "N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
268.9	GROUND SURFACE						
0.0	ASPHALT (30 mm)						
0.2	Gravelly sand, trace silt (FILL) Compact Brown Moist	1	SS	29			
	ASPHALT (100 mm)	2	SS	22		268	
	Gravelly sand to sand and gravel, trace silt (FILL) Compact Brown Moist	3	SS	14		267	
		4	SS	10		266	
265.7 3.2	CLAYEY SILT, trace sand Firm to stiff Brown to grey Moist to wet	5	SS	9	▽		
		6	SS	5		265	
264.0	SAND and SILT Brown to grey Wet	7	TO	PH		264	
263.6 5.5	CLAYEY SILT, trace sand Brown to grey Wet	8 a 8 b	SS	8		263	
262.8 6.1	SAND and SILT Loose Brown to grey Wet	9	SS	3			
	CLAYEY SILT, trace sand Firm Brown to grey Wet					262	
261.6 7.3	SAND and SILT Very loose Brown to grey Wet	10	SS	WH		261 260	
259.6 9.3	CLAYEY SILT, trace sand Very soft to soft Brown to grey Wet 1.2 m of heave encountered at 9.7 m depth.	11	SS	2		259	
		12	SS	WH		258	0 1 77 22
257.2 11.7	SILT, some clay, trace sand Loose Grey Wet	13	SS	6		257	
256.1 12.8	END OF BOREHOLE Note: 1. Water level at a depth of 3.4 m below ground surface (Elev. 265.5 m) upon completion of drilling.						

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: ○ UNCONFINED; + FIELD VANE; ● QUICK TRIAXIAL; × REMOULDED: 20 40 60 80 100

PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%): 20 40 60

UNIT WEIGHT γ kN/m³

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

Golder Associates

RECORD OF BOREHOLE No AC-4

 METRIC

PROJECT 11-1191-0032

W.P. 5302-05-00 LOCATION N 5304826.8; E 378222.9 ORIGINATED BY EHS

DIST HWY 573 BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers COMPILED BY AC

DATUM GEODETIC DATE May 31, 2012 CHECKED BY EC

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
268.5	GROUND SURFACE							
0.0	ASPHALT (30 mm)							
0.2	Gravelly sand, trace silt (FILL) Brown Moist	1	SS	19		268		
267.6	ASPHALT (100 mm)							
0.9	Gravelly sand, trace silt (FILL) Compact Brown Moist to wet	2	SS	4		267		
266.5	Clayey silt, some sand, trace gravel (FILL) Soft to firm Brown to grey Wet	3 a / 3 b	SS	7				
266.1 2.4	PEAT (Amorphous)	4	SS	12		266		
	CLAYEY SILT, trace sand Brown Wet	5	SS	14/0.15	▽			12 49 (39)
265.1 3.4	SAND and SILT, some gravel Compact Brown Wet							

END OF BOREHOLE
SPLIT SPOON REFUSAL (HAMMER BOUNCING) AND AUGER REFUSAL

Note:

1. Water level at a depth of 3.1 m below ground surface (Elev. 265.4 m) upon completion of drilling.

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED; 20 40 60 80 100

PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L; WATER CONTENT (%) 20 40 60

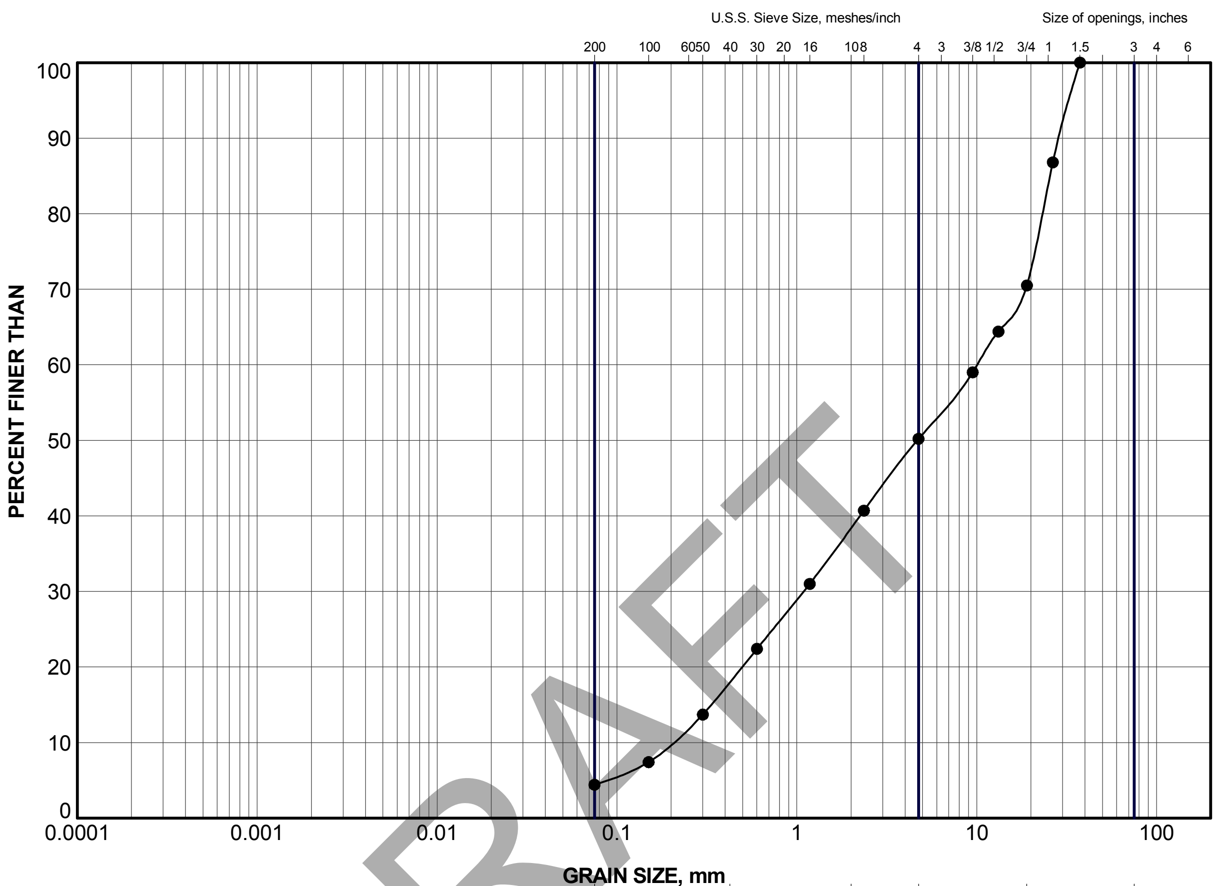
UNIT WEIGHT γ kN/m^3

STRAT PLOT

+ ^{3}, × ^{3}: Numbers refer to Sensitivity ○ $^{3\%}$ STRAIN AT FAILURE

APPENDIX B

Laboratory Test Results



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	AC-1	2	267.7

PROJECT

HIGHWAY 573
AIDIE CREEK BRIDGE 47-023

TITLE

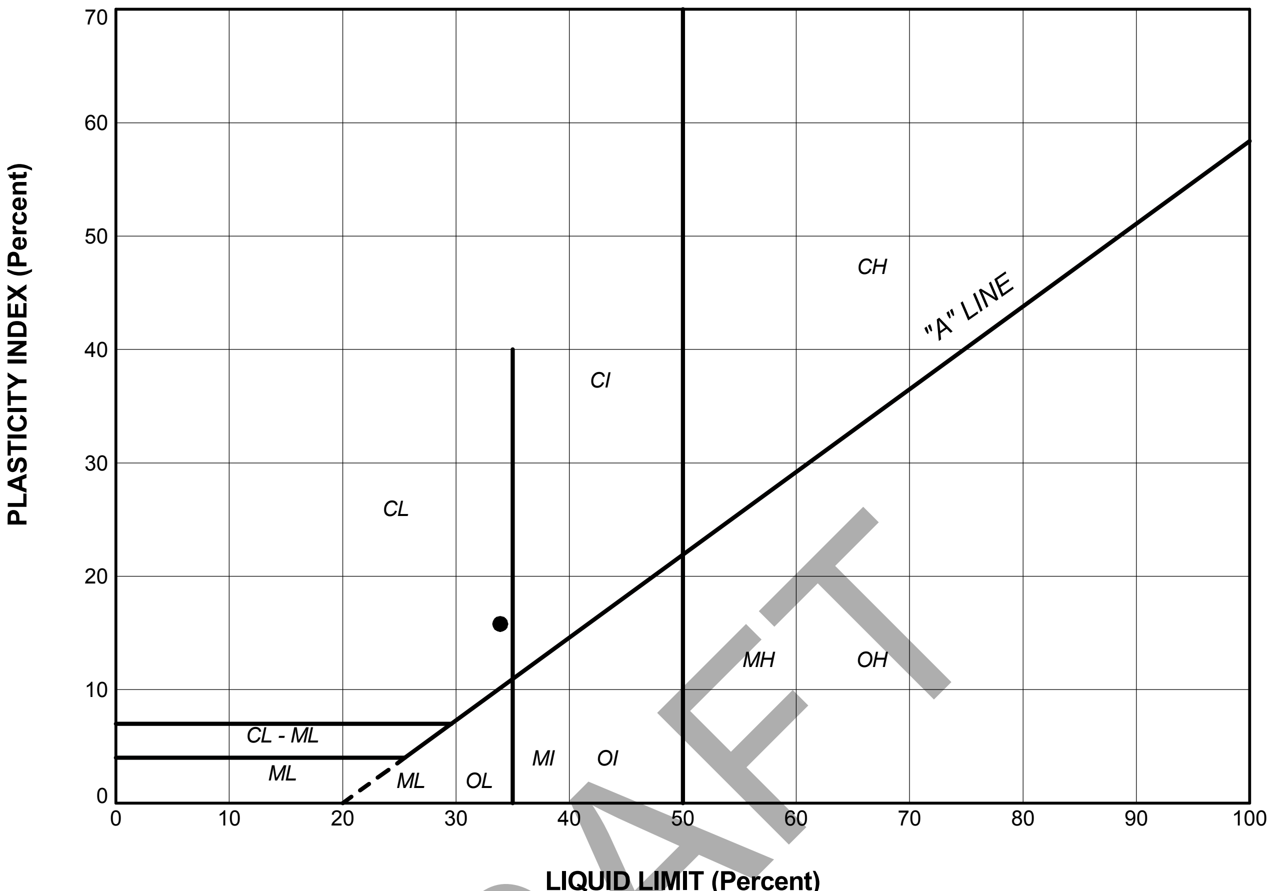
GRAIN SIZE DISTRIBUTION

Sand and Gravel (FILL)

Golder Associates
SUDBURY, ONTARIO

PROJECT No.	11-1191-0032		FILE No. 11-1191-0032+BH09.GPJ	
DRAWN	TB	Sep 2012	SCALE N/A	REV.
CHECK	EC	Sep 2012	FIGURE B-1	
APPR	JMAC	Sep 2012		

SUD-MTO GSD (NEW) GLDR_LDN.GDT



SOIL TYPE
C = Clay
M = Silt
O = Organic

PLASTICITY
L = Low
I = Intermediate
H = High

LEGEND

SYMBOL	BOREHOLE	SAMPLE	LL(%)	PL(%)	PI
●	AC-4	2	34	18	16

PROJECT

HIGHWAY 573
AIDIE CREEK BRIDGE 47-023

TITLE

PLASTICITY CHART

Clayey Silt (FILL)

PROJECT No.		11-1191-0032	FILE No. 11-1191-0032+BH09.GPJ	
DRAWN	TB	Sep 2012	SCALE N/A	REV.
CHECK	EC	Sep 2012	FIGURE B-2	
APPR	JMAC	Sep 2012		

Golder Associates
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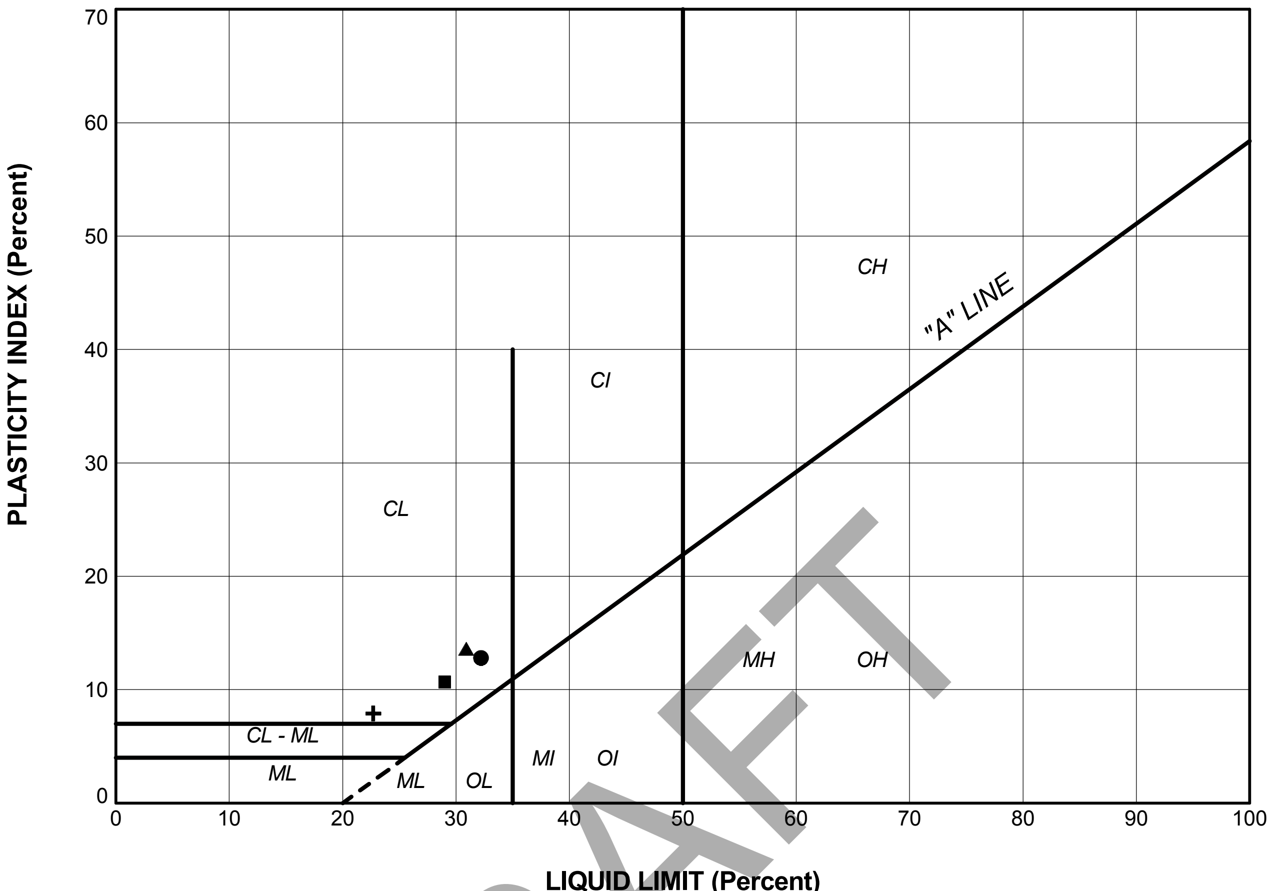
SUD-MTO PL (NEW) GLDR_LDN.GDT



SOIL TYPE
C = Clay
M = Silt
O = Organic

PLASTICITY
L = Low
I = Intermediate
H = High

LEGEND

SYMBOL	BOREHOLE	SAMPLE	LL(%)	PL(%)	PI
●	AC-2	8 b	32	19	13
■	AC-2	10	29	18	11
▲	AC-3	6	31	17	14
+	AC-3	9	23	15	8

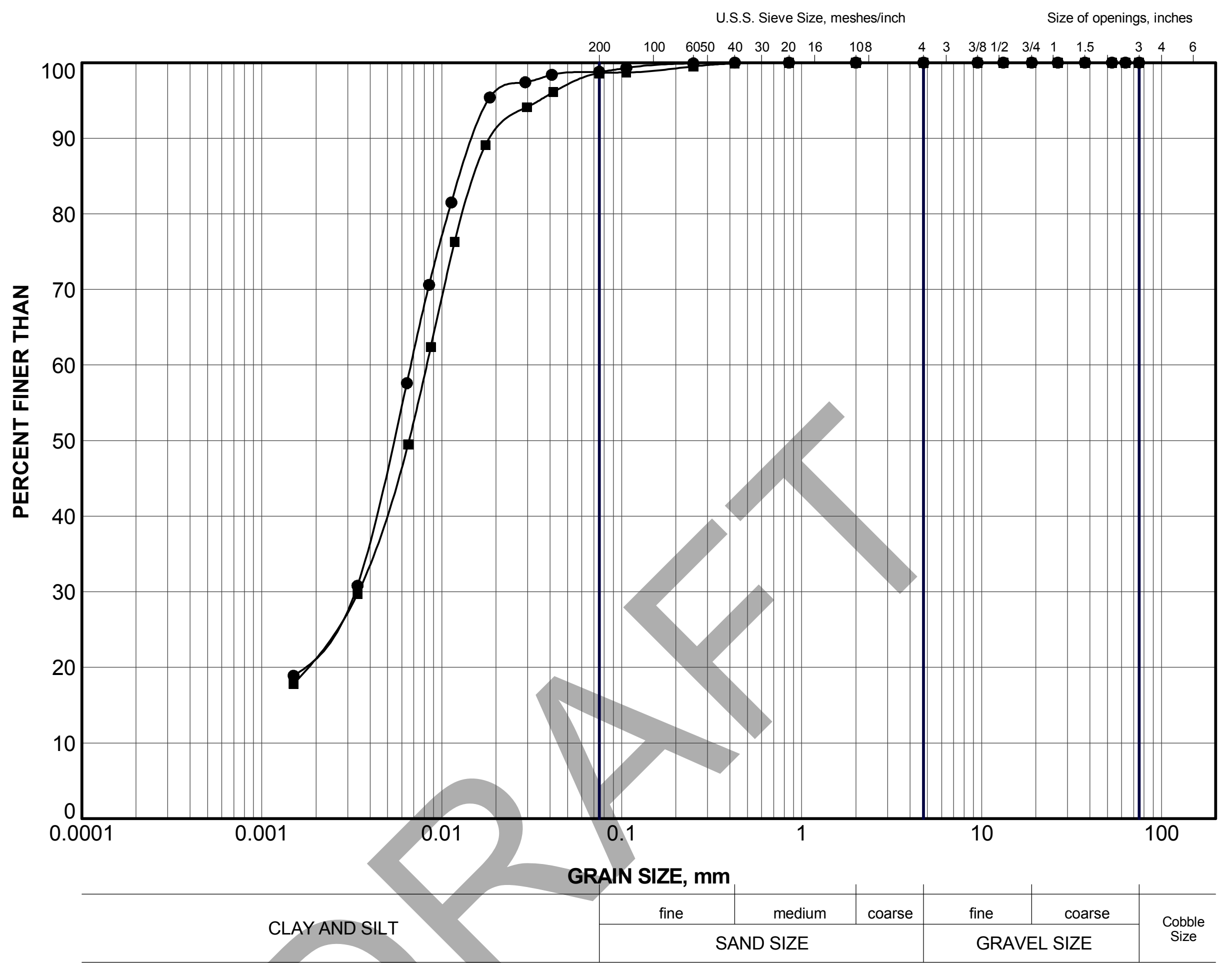
PROJECT
HIGHWAY 573
AIDIE CREEK BRIDGE 47-023

TITLE
PLASTICITY CHART
Clayey Silt

Golder Associates
SUDBURY, ONTARIO

PROJECT No.	11-1191-0032		FILE No.	11-1191-0032+BH09.GPJ
DRAWN	TB	Sep 2012	SCALE N/A	REV.
CHECK	EC	Sep 2012		
APPR	JMAC	Sep 2012	FIGURE B-3	

SUD-MTO PL (NEW) GLDR_LDN.GDT



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	AC-2	8 b	262.1
■	AC-3	12	257.9

PROJECT

HIGHWAY 573
AIDIE CREEK BRIDGE 47-023

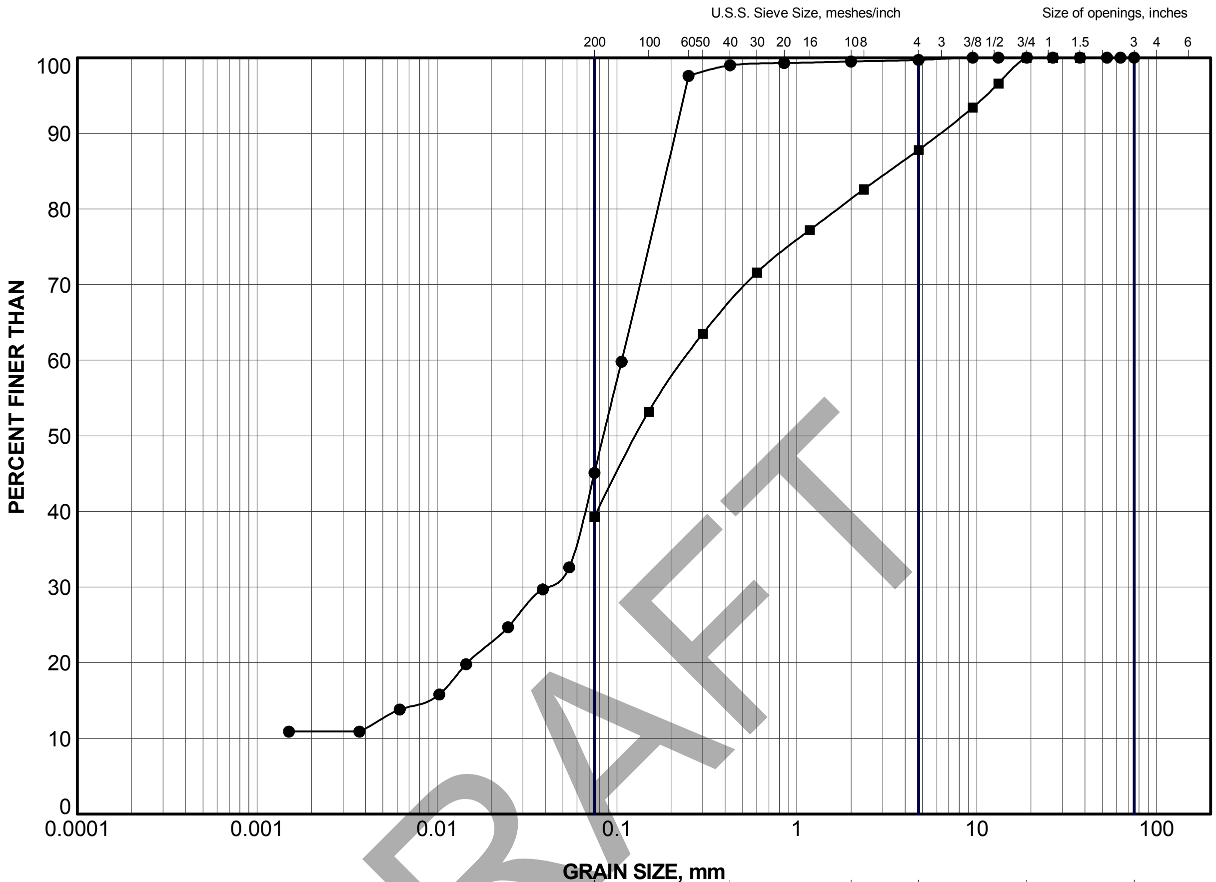
TITLE

GRAIN SIZE DISTRIBUTION

Clayey Silt

Golder Associates SUDBURY, ONTARIO	PROJECT No.	11-1191-0032	FILE No. 11-1191-0032+BH09.GPJ	
	DRAWN TB	Sep 2012	SCALE N/A	REV.
	CHECK EC	Sep 2012	FIGURE B-4	
	APPR JMAC	Sep 2012		

SUD-MTO GSD (NEW) GLDR_LDN.GDT



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	AC-2	8 a	262.3
■	AC-4	5	265.5

PROJECT

HIGHWAY 573
AIDIE CREEK BRIDGE 47-023

TITLE

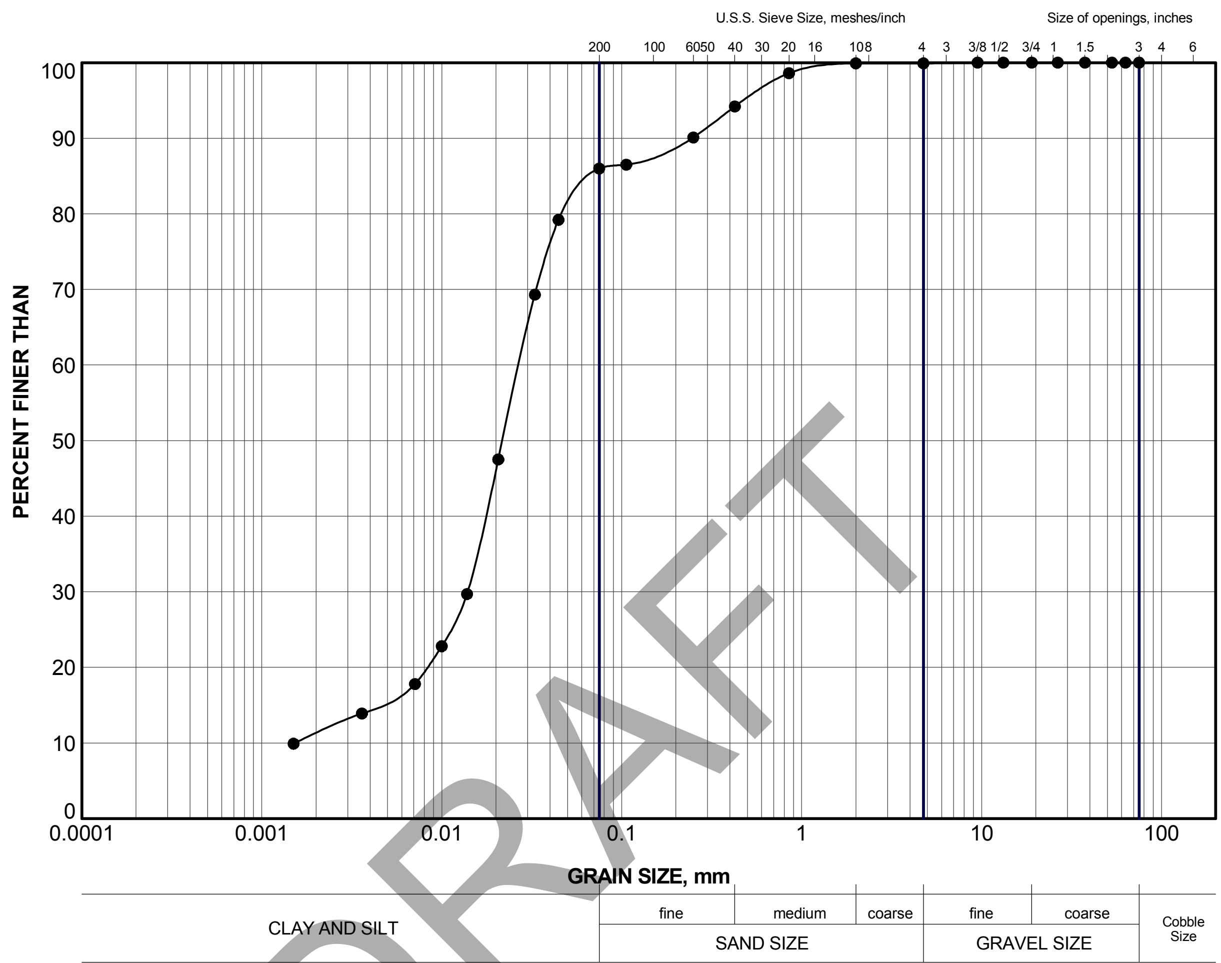
GRAIN SIZE DISTRIBUTION

Sand and Silt

Golder Associates
SUDBURY, ONTARIO

PROJECT No.	11-1191-0032		FILE No. 11-1191-0032+BH09.GPJ	
DRAWN	TB	Sep 2012	SCALE N/A	REV.
CHECK	EC	Sep 2012	**FIGURE B-5**	
APPR	JMAC	Sep 2012		

SUD-MTO GSD (NEW) GLDR_LDN.GDT



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	AC-2	13	254.6

PROJECT

HIGHWAY 573
AIDIE CREEK BRIDGE 47-023

TITLE

GRAIN SIZE DISTRIBUTION

Silt

PROJECT No.		11-1191-0032	FILE No. 11-1191-0032+BH09.GPJ
DRAWN	TB	Sep 2012	SCALE N/A REV.
CHECK	EC	Sep 2012	**FIGURE B-6**
APPR	JMAC	Sep 2012	

Golder Associates
SUDBURY, ONTARIO

SUD-MTO GSD (NEW) GLDR_LDN.GDT

At Golder Associates we strive to be the most respected global company providing consulting, design, and construction services in earth, environment, and related areas of energy. Employee owned since our formation in 1960, our focus, unique culture and operating environment offer opportunities and the freedom to excel, which attracts the leading specialists in our fields. 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 who operate from offices located throughout Africa, Asia, Australasia, Europe, North America, and South America.

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