



**August 2013**

## **PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT**

**South Saugeen River Bridge Replacement, Site No. 35-5  
Highway 89 Structure Replacements and Rehabilitation  
From 6.0 Km West of Mount Forest to Shelburne  
GWP 3035-11-00  
Ministry of Transportation, Ontario - West Region**

**Submitted to:**

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REPORT



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**PART A**

**FOUNDATION INVESTIGATION REPORT**

**SOUTH SAUGEEN RIVER BRIDGE REPLACEMENT, SITE NO. 35-5  
HIGHWAY 89 STRUCTURE REPLACEMENTS AND REHABILITATIONS  
FROM 6.0 KM WEST OF MOUNT FOREST TO SHELBURNE  
GWP 3035-11-00  
MINISTRY OF TRANSPORTATION, ONTARIO - WEST REGION**





## **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 carry out foundation investigations as part of the preliminary design work for GWP 3035-11-00. The project involves the replacement and rehabilitation of several structures along Highway 89 from 6.0 kilometres west of Mount Forest to Shelburne, Ontario. This report addresses the proposed replacement of the South Saugeen River Bridge at Station 32+010 (Site No. 35-5).

The purpose of the foundation investigation is to explore the subsurface conditions at the location of the proposed structure 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's proposal P1-1132-109-P01 dated November 3, 2011. The work was carried out in accordance with Golder's Quality Control Plan for Foundation Engineering dated March 8, 2012.

Golder was provided with digital copies of preliminary drawings prepared by MH for this project.





## **2.0 SITE AND PROJECT DESCRIPTION**

The South Saugeen River Bridge is located on Highway 89 approximately 1.5 kilometres west of Grey Road 6/Wellington Road 6. Highway 89 forms the boundary between Wellington County and Grey County, Ontario in this area. The location of the project is shown on the Key Plan, Figure 1.

This section of Highway 89 is currently a two lane undivided highway with gravel shoulders. It is generally oriented east-west in the vicinity of the subject site. The existing single-span bridge was constructed in 1953. The 36.6 metre long pony truss bridge has a deck width of 7.8 metres. The pavement surface on the existing bridge deck is at approximate elevation 383.6 metres. A Saugeen Valley Conservation Area bio-monitoring sampling site is located in the northwest quadrant of the project site. The existing bridge replaced a previous structure with a short span at the same location. The exact configuration of that bridge is not known. Photographs taken at the site on October 20, 2011 are provided in Appendix B.

Adjacent land use is typically rural agricultural and residential. The local topography is relatively flat with ground surface elevation in the tablelands adjacent to river at approximately 390 metres with elevation in the valley area ranging from 380 to 390 metres. The South Saugeen River at this location flows north in a broad channel with grassy banks flanked by wooded areas.

### **2.1 Site Geology**

The subject site is located in the physiographic region of southern Ontario known as the Teeswater Drumlin Field and is adjacent to the Dundalk Till Plain<sup>1</sup>. The former region is a drumlinized till plain with the drumlins oriented to the southeast. The Teeswater Ice Field was traversed by large meltwaters draining ice fronts to the north of west of the former "Ontario Island". Branches of the Saugeen River occupy broad valleys cut into the till plain by the meltwater channels. These valleys are associated with broad terraces of sand and gravel which fill much of the low ground. The physiographic mapping indicates that a former glacial spillway is situated just west of the bridge crossing. The latter region is gently undulating and has few low drumlins located in the area adjacent to the Teeswater Drumlin Field where the Saugeen River originates.

The surficial soils in the South Saugeen River channel consists of recent alluvial materials such as silt, sand and gravel. An aggregate pit was mapped east of the site. According to the quaternary geology mapping, the east valley walls comprise the Catfish Creek Till, a stony sandy silt till. The west valley walls were mapped as glaciofluvial outwash consisting of sand and minor gravel. The adjacent tablelands to the west and north were mapped to consist of ice-contact stratified drift consisting of sand, gravel, silt and minor till.<sup>2</sup> The tablelands to

<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> Cowan, W.R. and assistants, 1973: Palmerston, Southern, Ontario, Quaternary Geology. Ministry of Natural Resources, Ontario Geological Survey Map 2383, scale 1:50,000.





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the east and south consist of glaciofluvial outwash consisting of gravel and gravelly sand. Bedrock topographic mapping of the area indicates that the bedrock surface in the vicinity of this site lies between elevation 350.5 and 358.1 metres or some 20 to 40 metres below the ground surface.<sup>3</sup> The bedrock is reported to be tan dolomite of the 'E' member of the Salina Formation. The dolomite contains lenses of anhydrite or gypsum.<sup>4</sup>

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<sup>3</sup> Davies, L.L., McClymont, W.R. and Karrow, P.F., 1962: Bedrock Topography Series, Palmerston Sheet, Ontario Department of Mines, Preliminary Map No. P.166, scale 1:50,000.

<sup>4</sup> Sanford, B.V., 1969: Geology, Toronto-Windsor Area, Ontario. Geological Survey of Canada, Map 1263A, scale 1:250,000.

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### 3.0 INVESTIGATION PROCEDURES

The field work for the investigation was carried out on October 1 and 2, 2012, during which time 2 boreholes were drilled at the 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		
1	4 871 073	201 350	379.97	9.17
2	4 871 100	201 387	381.24	10.76

The investigation was carried out using track-mounted drilling equipment supplied and operated by a specialist drilling contractor. In the boreholes, samples of the overburden were obtained at suitable intervals of depth using 50 millimetre outside diameter split spoon sampling equipment with an automatic trip hammer in accordance with the standard penetration test (SPT) procedures as described in ASTM D1586. The SPT 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 into the soil after having first penetrated 150 millimetres. The samplers used in the investigations limit the maximum particle size that can be sampled and tested to about 40 millimetres. Therefore, particles or objects 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 native till materials as discussed in the text of this report.

The boreholes were terminated at depths of 9.2 and 10.8 metres below the existing ground surface. Groundwater conditions in the boreholes were observed throughout the drilling operations and a standpipe was installed in borehole 2 as indicated on the corresponding Record of Borehole sheet. 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 members of our engineering staff who 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 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.

A search of the MTO Geocres database indicated that there are no existing Foundation Investigation or Design reports for this site. Additionally, a review of the previous design drawings for this site provided with the Request for Proposal package indicated that geotechnical information including design bearing capacities were not provided on the drawings.





## **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 given 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 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 topsoil and surficial fill underlain by an upper sand and gravel layer. A sandy silt till and sandy silt layer lies between the upper and lower sand and gravel layers. The boreholes were terminated in sandy silt till or silt layers found below the lower sand and gravel deposits.

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 summarized in subsequent report sections.

#### **4.1.1 Topsoil**

Topsoil layers 250 and 610 millimetres thick were found at the ground surface at boreholes 2 and 1, respectively. A buried topsoil layer 210 metres thick was encountered beneath the fill in borehole 2 from elevation 380.8 metres. Cobbles and boulders were noted within the topsoil at borehole 1.

Materials designated as topsoil in this report were classified solely based on visual and textural evidence. Testing of organic content or for other nutrients was not carried out. Therefore, the use of materials classified as topsoil cannot be relied upon for support and growth of landscaping vegetation.

#### **4.1.2 Fill**

The topsoil in borehole 2 was underlain by a 0.2 metre thick layer of silty sand fill from elevation 381.0 metres. Boreholes 1 and 2 were drilled outside the roadway as the scope of work for this assignment did not necessitate investigation of the embankments. Fill materials associated with the existing approach embankments and pavement structure should be expected. It is possible that remnants of structures associated with construction of the present and pre-1953 structure may be also buried in the fill.





### **4.1.3 Sand and Gravel**

Layers of sand and gravel were encountered in both boreholes beneath the topsoil layers near ground surface and between the silt and sandy silt till layers at depth. The upper sand and gravel layers were encountered at elevation 379.4 metres in borehole 1 and elevation 380.6 metres in borehole 2. The upper sand and gravel layers were 0.8 and 1.3 metres thick. The upper sand and gravel layer contained cobbles at borehole 1 and was found to be silty at borehole 2. The upper sand and gravel was loose to compact with measured N values of 8 to 13 blows per 0.3 metres.

The lower sand and gravel layers were encountered at elevation 374.8 metres between sandy silt till layers in borehole 1 and at elevation 374.7 metres below the sandy silt layer in borehole 2. The lower sand and gravel layers were 1.8 and 2.0 metres thick. The lower sand and gravel contained cobbles at borehole 1 and the gravel fraction was predominant in borehole 2. The gradations of two samples of the lower sand and gravel are presented on Figure A-1 in Appendix A. The lower sand and gravel is dense to very dense based on measured N values of 46 to over 100 blows per 0.3 metres. Water contents of 7 and 11 per cent were measured in samples retrieved from the SPT testing.

### **4.1.4 Clayey Silt Till**

The upper silty sand and gravel in borehole 2 was underlain by a 0.8 metre thick layer of stiff clayey silt till at elevation 379.3 metres. The results of the grain size analysis carried out on a sample of clayey silt till is presented on Figure A-2. The presence of cobbles and boulders should be anticipated within the clayey silt till based on the depositional history of glacial till even though cobbles and boulders were not specifically encountered in this layer.

An N value of 13 blows per 0.3 metres was measured in the clayey silt till which had a water content of 15 per cent. The clayey silt till is of low plasticity based on a plastic limit of 13 per cent, a liquid limit of 21 per cent and a plasticity index of 8 per cent. The results of the Atterberg limits determination are presented on Figure A-5.

### **4.1.5 Sandy Silt Till to Sandy Silt**

The sand and gravel layers in borehole 1 and the clayey silt till and lower sand and gravel layers in borehole 2 are underlain by compact to very dense, but typically very dense deposits of sandy silt and sandy silt till. Collectively these deposits likely constitute the Catfish Creek Till which texturally ranges from sand and silt to sandy silt.<sup>5</sup> The Catfish Creek Till generally overlies bedrock and is the oldest till unit mapped in this area. It is possible that the very dense silt and sandy silt which underlie the glacial till may also belong to the Catfish Creek

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<sup>5</sup> Cowan, W.R., 1979: Quaternary Geology of the Palmerston Area, Southern Ontario; Ontario Geological Survey Report 187, 64p. Accompanied by Maps 2383, 2384, scale 1:50,000.





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Till deposit. However, upon visual examination of the samples and comparison of the gradations, where obtained, the sandy silt till, silt and sandy silt have been identified as distinct strata.

The sand and gravel in borehole 1 and the clayey silt till in borehole 2 was underlain by compact to very dense sandy silt till from approximately elevation 378.5 metres. These upper sandy silt till layers were 1.7 and 3.8 metres thick. The lower 1.1 metres of the till contained less gravel and was classified as sandy silt. Borehole 1 was terminated in the lower very dense sandy silt till layer after exploring some 2.0 metres below elevation 372.8 metres. The gradations of two samples of sandy silt till are provided on Figure A-3. Cobbles were encountered in the lower sandy silt till in borehole 1. Cobbles and boulders should be expected in the sandy silt till based on the depositional history of glacial materials.

Measured N values in the sandy silt till ranged from 25 to over 100 blows per 0.3 metres. Water contents varied from 7 to 11 per cent.

The sandy silt till in borehole 2 was underlain by a 1.1 metre thick layer of compact to very dense sandy silt from elevation 376.8 metres. The result of the grain size analysis carried out on a sample of sandy silt is presented on Figure A-4. Measured N values in the sandy silt ranged from 15 to over 100 blows per 0.3 metres. The water content of the sandy silt was 16 per cent.

### 4.1.6 Silt

The lower sand and gravel in borehole 2 was underlain by silt at elevation 372.9 metres. Borehole 2 was terminated in the silt after exploring some 2.4 metres. The silt was very dense with N values greater than 100 blows per 0.3 metres and a water content of 12 per cent.

## 4.2 Groundwater Conditions

Groundwater conditions were observed during and on completion of drilling and sampling and a standpipe was installed in borehole 2. Installation details are provided on the corresponding Record of Borehole sheet following the text of this report. Groundwater was encountered in boreholes 1 and 2 at depths of 1.1 and 1.2 metres or elevations 380.2 and 378.8 metres, respectively. A summary of the encountered and measured groundwater levels are provided in the following table:

Borehole	Ground Surface Elevation (m)	Encountered Groundwater (m)		Measured Groundwater (m)			
				October 2, 2012		October 24, 2012	
		Depth	Elevation	Depth	Elevation	Depth	Elevation
1	379.97	1.2	378.8	-	-	-	-
2	381.24	1.1	380.2	0.00	381.24	0.91	380.33





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The above-noted encountered water levels are not considered to be representative of the long-term, stabilized groundwater conditions. The corresponding water level in the watercourse was measured at elevation 378.88 metres on October 2, 2012 and was reported at elevation 379.15 metres in April 1988. The most recent groundwater measurement at borehole 2 was obtained on October 24, 2012 shortly before the standpipe was decommissioned. On this date, the water level in the standpipe was at about elevation 380.3 metres.

Based on the measured and encountered groundwater levels, the surrounding topography, and the water level in the South Saugeen River, the inferred groundwater level ranges between elevation 379.0 and 380.5 metres in the area where foundation construction is anticipated. The groundwater levels are expected to fluctuate seasonally and are expected to be higher during periods of sustained precipitation or during spring melt conditions and will be influenced by water surface elevations in the South Saugeen River.





## **5.0 MISCELLANEOUS**

The investigation was carried out using equipment supplied and operated by Aardvark Drilling Inc., which is an Ontario Ministry of Environment licensed well contractor. The field operations were supervised by Mr. Michael Arthur under the direction of Mr. David J. Mitchell. The laboratory testing was carried out at Golder's 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. Dirka U. Prout, P.Eng. under the direction of the Team Leader, Dr. Storer J. Boone, P.Eng. This report was reviewed by Mr. Fintan J. Heffernan, P.Eng., the Designated MTO Contact and Quality Control Auditor for this assignment.

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**PART B**

**FOUNDATION DESIGN REPORT**

**SOUTH SAUGEEN RIVER BRIDGE REPLACEMENT, SITE NO. 35-5  
HIGHWAY 89 STRUCTURE REPLACEMENTS AND REHABILITATIONS  
FROM 6.0 KM WEST OF MOUNT FOREST TO SHELBURNE  
GWP 3035-11-00  
MINISTRY OF TRANSPORTATION, ONTARIO - WEST REGION**





## **6.0 ENGINEERING RECOMMENDATIONS**

### **6.1 General**

This section of the report provides our Preliminary Design recommendations for the foundation aspects of the design of the replacement of the existing South Saugeen River Bridge (Site No. 35-5). 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 in order 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 and scheduling.

### **6.2 Bridge Foundations**

Three replacement options were under consideration at the time of preparation of this report. The replacement bridge alternatives involve designs with a 1500 millimetre deep steel plate girder, a 1900 millimetre depth CPCI girder and an NU girder (size to be determined). The steel plate girder option is the preferred technical solution from a structural engineering perspective. The Structural Section of the MTO has requested that semi-integral abutments also be considered. The General Arrangement drawings for each replacement alternative features full depth retaining wall abutments constructed on shallow foundations.

The preferred staging option identified by MH involves a permanent, single lane shift of the alignment with the existing bridge replaced in two stages. The first stage involves constructing the south side of the new bridge structure adjacent to the existing bridge. Once construction of the south half is complete, the existing structure will be demolished and the north half of the replacement structure built in the second stage. This alternative allows the crossing location to be open throughout construction. The existing approach embankments in this area will be widened approximately 7 metres to the south. Some modification of the roadway vertical profile may be carried out to move the low point away from the bridge but it is anticipated that grade raises in this area will be minimal.

Based on a review of a 1952 drawing entitled "The County of Wellington, Holliday Bridge, Road No. 79A, Lot 4, Concession XIV, Township of Minto" the existing bridge abutments are of the full retaining wall type with turned back wingwalls. The existing abutment and wingwall footings were to be 2.74 metres wide and founded approximately 5.33 metres below the deck or near elevation 378.3 metres. The drawings indicate that piles were to be used if necessary but based on the soil conditions encountered in boreholes 1 and 2, it is unlikely that piles were installed. The 1952 structure replaced a previous structure with a shorter span at this site.





The subsurface soil conditions at the borehole locations typically consist of surficial topsoil and fill underlain in sequence by compact sand and gravel to elevation 379 metres then predominantly compact to very dense sandy silt till to elevation 375 metres. On the east side, borehole 2 encountered compact to very dense sandy silt between elevation 377 and 375 metres. Dense to very dense sand and gravel to gravel was encountered at elevation 375 metres. The sand and gravel was underlain from elevation 373 metres to approximate elevation 370.5 metres by very dense sandy silt till on the north side and very dense silt on the south side. The prevailing groundwater level was inferred to vary between approximately elevation 379 and 380.5 metres. The river elevation at the Highway 89 South Saugeen River Bridge (Site 35-5) is 379 metres.

Shallow foundations are preferred for the replacement structure and associated wingwalls and retaining walls since competent dense to very dense granular soils are near the ground surface. Shallow foundations are compatible with conventional abutments such as those of the perched, partial and full depth retaining wall types and for semi-integral abutments. Spread footings supported on the compact to dense sandy silt till and sand and gravel has been selected as the preferred technical alternative since this foundation option is compatible with the full depth retaining wall type abutment designs currently under consideration. It should be noted that if perched or partial depth retaining wall type abutments are considered, spread footings supported on Granular A pads would be the most suitable choice from a foundations engineering perspective. Deep foundations can be used for any abutment type. However it should be noted, that depending on the pile cap location and the type of abutment selected, the piles may be short (less than 5 metres). Use of driven piles may not be economical particularly at the west abutment where, depending on the abutment design, pile lengths may be limited to 3 metres.

A comparison of foundation alternatives is presented in Table I. Actual costs have not been provided due to the preliminary nature of the design. Rather qualitative descriptions (low, moderate, high, very high) have been given to provide a relative comparison of the alternatives for foundation engineering purposes and should not be considered to be indicative of actual construction costs which are dependent on various factors including the staging requirements, and the selected design.

### **6.2.1 Shallow Foundations**

New abutment footings may be constructed in the native sandy silt till at or below elevation 378.5 metres. The footings may be designed using a factored geotechnical resistance of 525 kilopascals at Ultimate Limit States (ULS), and a geotechnical reaction of 350 kilopascals at Serviceability Limit States (SLS). The SLS value corresponds to 25 millimetres of settlement.

Alternatively, if a perched abutment is used, the footings can be constructed on a compacted Granular A pad built within the approach embankment fill. Footings for perched type abutments may be designed for a factored geotechnical resistance at ULS of 900 kilopascals and a geotechnical reaction at SLS of 350 kilopascals. The advantage of this latter alternative is that foundations can be constructed above the groundwater level.





### **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 Canadian Highway Bridge Design Code (CHBDC). Assuming that the founding soils are not loosened/disturbed during excavation and footing construction, the following angle of friction between the mass cast-in-place (CIP) concrete (e.g., levelling pad for precast foundations) and the founding soils and corresponding unfactored coefficient of friction,  $\tan \delta$ , may be used:

Footings on sandy silt till	angle of friction	29°
	$\tan \delta$	0.55
Footings on Granular A pad	angle of friction	31°
	$\tan \delta$	0.60

Where precast concrete footings are to be placed directly on a prepared soil subgrade, the coefficient of friction and angle of friction as provided above should be reduced by 25 to 30 per cent.

### **Frost and Scour Protection**

All footings should be provided with a minimum of 1.6 metres of earth cover or thermal equivalent for frost protection purposes. Scour protection should be provided in accordance with Section 1.9.5 of the 2006 CHBDC.

## **6.2.2 Deep Foundations**

Steel H-piles are suitable for both conventional and integral abutments whereas steel tube piles are generally not used for integral abutments due to their stiffness. Driven H-piles are preferred over steel tube piles since they are better suited to driving through coarse granular deposits containing cobbles and boulders. It has been assumed that pile caps for a full depth retaining wall type abutment will be constructed at or near the location of the existing footings. Therefore a cut-off elevation of 379.0 metres has been assumed. This abutment type was selected because it is fairly common but it yields the shortest pile lengths. The resulting pile lengths suggest that use of deep foundations require some pre-augering/pre-drilling to obtain sufficient pile length. If the design features integral abutments, the piles will need to be installed in pre-bored holes at depths deeper than indicated in the table below in order to develop sufficient fixity to effectively resist lateral forces.

### **Driven Steel H-Piles**

End bearing HP 310 x 110 piles, if used, would need to be driven to practical refusal in the very dense sandy silt till or sand and gravel to the depths noted in the following table. A factored geotechnical resistance of 1800 kilonewtons at ULS and a geotechnical reaction of 1500 kilonewtons at SLS may be used for design.





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Pile Location	BH	Proposed Tip Elevation (m)	Proposed Pile Length (m)	Founding Material
West Abutment	1	376.0	3.0	Sandy Silt Till
East Abutment	2	373.5	5.5	Sand and Gravel to Gravel

Piles supporting integral abutments would likely 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.

### *Driven Steel Tube Piles*

End bearing steel tube piles, 324 millimetres outer diameter (O.D.) x 9.5 millimetre wall thick, if used, would need to be driven to practical refusal in the very dense sandy silt till or sand and gravel to the depths noted in the following table. A factored geotechnical resistance of 1350 kilonewtons at ULS and a geotechnical reaction of 1125 kilonewtons at SLS may be used for design. As discussed above, these pile lengths are impractically short and, therefore, may not be appropriate for this project.

Pile Location	BH	Proposed Tip Elevation (m)	Proposed Pile Length (m)	Founding Material
West Abutment	1	376.5	2.5	Sandy Silt Till
East Abutment	2	375.0	4.0	Sandy Silt Till to Sand and Gravel to Gravel

### *Frost and Scour Protection*

All pile caps for conventional abutments should be provided with a minimum of 1.6 metres of earth cover or thermal equivalent for frost protection purposes. Scour protection should be provided in accordance with Section 1.9.5 of the 2006 CHBDC.

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

The overburden materials at this site are granular in nature. If, as assumed in Section 6.2, there will be no change in grade or widening of the road platform, then consideration of downdrag loads are not applicable for piles at this location. Downdrag loads must be considered in the unlikely case of significant grade raises and platform widenings which comprise cohesive fills.





### **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. Preliminary estimates of lateral resistances for driven steel HP 310 x 110 piles were derived from interpolating the table of assessed values, Table C6.4 in the Commentary to the 2006 CHBDC and are given in the table below. The geotechnical reaction at SLS corresponds to a lateral movement of 10 millimetres. As noted above, pre-drilled holes would likely be required for H pile supported integral abutments to develop appropriate fixity and driven steel tube piles would not be feasible for integral abutments.

Pile Type	Lateral Resistance at Factored ULS (kN)	Lateral Reaction at SLS (kN)
Pre-drilled HP 310 x 110	110	40

## **6.3 Wingwalls**

Geotechnically feasible alternatives for the wingwalls of the replacement structure are reinforced concrete gravity or cantilever walls or reinforced soil system (RSS) walls. The cantilever walls could be CIP or comprise precast elements. In addition, in areas where the wingwall segments are relatively low, they could consist of precast concrete toe walls. Based on the results of the foundation investigation, shallow foundations are suitable and use of deep foundations is not warranted since competent materials are near the ground surface.

### **6.3.1 Comparison of Wall Types**

CIP concrete gravity and cantilever walls will require the longest construction time and deepest excavation compared to concrete toe or RSS walls. Construction time can be reduced if precast elements are utilized. RSS wingwalls typically may be rapidly constructed using geogrid or metallic strip reinforcement and facings consisting of precast concrete panels or modular block. Construction of RSS walls is less labour intensive and can be accomplished with smaller equipment in less time than gravity or cantilever walls. In addition, depending on the proprietary design selected by the manufacturer, it is not necessary to found the footings at or below the frost penetration depth. This will result in a decrease in excavation depth and dewatering/groundwater control effort. The design of the existing bridge is such that the wingwalls are approximately 5 metres high. If an alternative design has wingwall sections that are up to 1.8 metres in height, then a Type II concrete toe wall as per Ontario Provincial Standard Drawing (OPSD) 3120.100 may be used. Concrete toe walls may be founded above the frost depth provided that the subgrade can supply a factored geotechnical resistance of 300 kilopascals at ULS at a minimum embedment depth of 450 millimetres. The embedment depth is defined as the distance from the underside of the toe wall foundation to the top of finished grade in front of the wall.





### 6.3.2 Foundations

Reinforced concrete gravity and cantilever walls should be founded at an elevation such that frost cover in the form of soil to a depth 1.6 metres or thermal equivalent is provided. The existing wingwalls are founded at the same elevation as the abutments or at approximate elevation 378.3 metres. Compact to dense sandy silt till was encountered at this elevation at boreholes 1 and 2. New reinforced concrete gravity and cantilever wingwalls may be founded in the sandy silt till at or below elevation 378.3 metres.

Concrete strip footings or a 300 millimetre thick levelling pad consisting of compacted Granular A may be used for RSS footings. The RSS walls may be founded on the compact sand and gravel near elevation 379.4 metres for the west abutment and 380.6 metres for the east abutment. If higher geotechnical resistances or a greater depth of embedment is required for stability, the footings may be founded in the dense sandy silt till or stiff clayey silt till that underlies the upper sand and gravel layers. Concrete toe walls are to be founded in the sandy silt till and clayey silt till layers in order to achieve the geotechnical resistance required by this standardized design.

Geotechnical resistances for each wall type are presented in the following table:

Wall Type	Founding Material	Maximum Founding Elevation (m)	Factored Geotechnical Resistance at ULS (kPa)	Geotechnical Reaction at SLS (kPa)
Reinforced concrete gravity and cantilever walls	Sandy Silt Till	378.3	450	300
RSS wall				
- West abutment	Sand and Gravel	379.4	300	200
- East abutment	Sand and Gravel	380.6	300	200
Concrete toe wall				
- West abutment	Sandy Silt Till	378.6	450	300
- East abutment	Clayey Silt Till	379.3	300	200





### **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 CHBDC. Assuming that the founding soils are not loosened/disturbed during excavation and footing construction, the following angle of friction between the mass CIP concrete (e.g., levelling pad for precast foundations) and the founding soils and corresponding unfactored coefficient of friction,  $\tan \delta$ , may be used:

Wall Type	Interaction	Friction Angle, $\delta$	Coefficient of Friction, $\tan \delta$
Reinforced concrete gravity and cantilever walls	CIP concrete footing on sandy silt till	29	0.55
RSS wall	Precast concrete footing on sand and gravel	30	0.58
	Granular A levelling pad on sand and gravel	35	0.70
Concrete toe wall - West abutment - East abutment	Precast concrete footing on Sandy Silt Till	27	0.51
	Clayey Silt Till	26	0.49

## **6.4 Lateral Earth Pressures**

The lateral pressures acting on the bridge abutments and wingwalls will depend on the type and method of placement of the backfill materials, on the nature of the soils behind the backfill, on the freedom of lateral movement of the structure and on 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 Provincial Standard Specification (OPSS) Granular A or Granular B Type II but with less than 5 per cent passing the No. 200 sieve should be used as backfill behind the abutments and wingwalls. This fill should be compacted in loose lifts not greater than 200 millimetres in thickness. Longitudinal drains and weep holes should be installed to provide positive drainage of the granular backfill.
- A compaction surcharge equal to 12 kPa should be included in the lateral earth pressures for the structural design of the abutment wall in accordance with CHBDC Figure 6.6.
- The granular fill may be placed either in a zone with a width equal to at least 1.6 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).





## PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN SOUTH SAUGREEN RIVER BRIDGE, SITE NO. 35-5

- 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>

Coefficients of lateral earth pressure:

Active,  $K_a$  0.33

At rest,  $K_o$  0.50

Passive,  $K_p$  3.0

- 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 II</u>
Soil unit weight:	22 kN/m <sup>3</sup>	22 kN/m <sup>3</sup>
Coefficients of lateral earth pressure:		
Active, $K_a$	0.27	0.27
At rest, $K_o$	0.43	0.43

- 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.
- Integral and semi-integral abutment structures are designed to allow for movement of the ends of the bridge deck. In this case, passive earth pressure 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. The movement to allow passive pressures to develop within the backfill may be taken as:
  - Rotation of approximately 0.1 about the base of a vertical wall;
  - Rotation of approximately 0.02 about the top of a vertical wall;
  - Horizontal translation of 0.05 times the height of the wall; or,
  - A combination of the above.

Rotation is assumed to take place at a fixed point either at the top or bottom of the wall and is defined as the ratio of the horizontal displacement to the height of the wall. A resistance factor equal to 0.5 should be applied to the calculated total passive resistance in accordance with Table 6.1 of the CHBDC.

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.





## **6.5 Embankment Stability and Settlement**

It is anticipated that the existing embankment will be widened some seven metres to the south to accommodate the alignment shift and widened replacement structure. The embankment subgrade primarily consists of compact to very dense granular materials. The existing approach embankments exhibited no signs of distress or settlement. For these reasons, a conventional slope geometry of 2 horizontal to 1 vertical can be established for the sideslopes of the reconstructed embankment. No problems with embankment settlement or instability are foreseen provided they are constructed of engineered fill materials on subgrades prepared in accordance with applicable OPSS requirements and under the inspection of qualified geotechnical personnel.

## **6.6 Construction Considerations**

This discussion presents construction related comments and recommendations pertaining to shallow foundations, and driven steel piles and sheeting.

### **6.6.1 Shallow Foundations**

The elevation of the inferred groundwater level ranges between 379 and 380.5 metres. Dewatering will be required to construct footings in the sand and gravel, clayey silt till and sandy silt till deposits. Due to the high permeability of the sand and gravel and the proximity of the footing to the watercourse, it is anticipated that a suitable enclosure of sheeting, soldier pile and lagging or sand-bag or pre-fabricated cofferdam systems may be constructed around each section of footing. The sheets or lagging should terminate in the lower permeability sandy silt till so that a groundwater cutoff results. It would then be necessary to dewater inside the enclosure only. Sheet pile and soldier pile and lagging enclosures will both provide temporary excavation support with sheeting being the most effective option for groundwater cut-off. While some limited dewatering should be expected within a completed sheet-pile enclosure during footing excavation and construction, soldier pile and lagging enclosures may only be constructed in the presence of active dewatering operations. Soldier pile and lagging enclosures are permeable, therefore extensive dewatering must continue during footing excavation, construction and backfilling operations. Sand-bag and pre-fabricated fabric cofferdams, are only suitable for isolation of the footings from the nearby Saugeen River. Cofferdams constructed with sand-bags or lagging are prone to leakage. Due to the increased potential for ground-loss, cofferdams constructed with sand-bags or lagging must be carefully constructed and regularly inspected for signs of damage.





### **6.6.2 Driven Steel Piles**

Driven steel H piles, if used, should be equipped with Type I bearing shoes since cobbles and boulders are expected in both the sand and gravel and glacial till deposits. The planned replacement will be the second known for this site. There may be remnants of former temporary or permanent works associated with either the pre- or post-1953 structure which are buried in the fill and would present an obstruction for piles driven in this area.

### **6.6.3 Sheet piling**

Sheet piling, if used during construction as a groundwater cutoff or for temporary excavation support may be difficult to advance below elevation 378 metres on the west side of the river due to the presence of very dense sandy silt till. In addition, obstructions such as cobbles and boulders in the sand and gravel and glacial till deposits and remnants of previous works, as noted in Section 6.6.2, should be expected in the native soils and fills, respectively.

## **6.7 Excavations**

Excavations for spread/strip footings or pile caps for full-depth retaining wall type abutments will extend through the existing pavement structure, fill materials, upper sand and gravel layer and terminate in the glacial till. Groundwater seepage into the excavations, particularly from the saturated upper sand and gravel layer should be anticipated. It may be necessary to use flatter slopes in the sand and gravel to enhance stability. If the founding elevations for footings or pile caps are such that they will be below the groundwater level, use of an enclosure cofferdam to permit groundwater cut-off is required. Sheet piling used for the enclosure cofferdam must fully surround the excavation and have sufficient penetration to permit control of groundwater inflow with an effective cutoff. As noted in Section 6.6.3, the Contractor should be prepared for difficulties which may occur during sheet pile installation due to natural and potential man-made obstructions. Extraction of the sheet piles may cause disturbance to the surrounding ground. Consideration could be given to leaving the sheet piles in place after construction to protect the footings from scour and erosion. Dewatering systems should be installed at least 1 metre from the perimeter of the actual footing limits. Surface water runoff should be directed away from the excavations at all times.

Temporary open cut slopes within the granular materials should be maintained no steeper than 1 horizontal to 1 vertical. 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 and granular materials below the groundwater level would be classified as Type 3 soils. The clayey silt till and properly dewatered cohesionless materials including the sandy silt till would be classified as Type 2 soils.





## **7.0 COMMENTS FOR DETAIL DESIGN**

Shallow foundations have been identified as the preferred foundation type from the perspectives of cost and foundations engineering. A Foundation Investigation and Design Report will need to 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 this site. Specifically, a minimum of four boreholes should be advanced as follows:

- 1 at each abutment opposite to existing boreholes 1 and 2. These two boreholes shall be advanced to 3 metres below refusal, defined as material for which the Standard Penetration Test (SPT) N value is greater than 100 blows per 0.3 metres. It may be necessary to drill deeper, particularly at the west abutment, if integral abutments are under consideration. These two additional boreholes are recommended because of the potential variability in subsurface conditions at this site, particularly as related to water control issues.
- 1 at each approach embankment within 20 metres of the abutment. These two boreholes are to be drilled through the embankment to a minimum depth of 100 per cent of the embankment height below the base of the embankment fill. The embankment fill may be up to 4 metres high.
- A minimum of 2 supplementary boreholes may be required if the proposed construction staging requires temporary embankment widening.

Sampling and SPT testing should be carried out at intervals of 0.75 metres. Routine soil testing consisting of grain size analyses, water contents and Atterberg limits, where applicable, is considered appropriate particularly if there will be an increase in the road grade or change in alignment.

The preliminary 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. Emphasis should be placed on provision of detailed recommendations for foundations for the abutments and wingwalls. The ground conditions are such that if the final design must feature integral abutments, additional analyses may be required to determine the minimum pile length required to develop adequate fixity to withstand horizontal displacement of the superstructure. Since the approach embankments are to be modified, the stability of the altered embankments should be confirmed and the resulting settlement evaluated. If the bridge is to be replaced using staged construction and temporary roadway protection, the discussion on conceptual shoring alternatives must be extended to include lateral earth pressures and detailed impact of ground conditions on shoring construction and design.

Effective dewatering will be critical to successful construction of shallow foundations or pile caps below the groundwater level. An evaluation of the need for an Ontario Ministry of the Environment Permit To Take Water (PTTW) and estimation of the hydraulic conductivity of the shallow soils should be conducted during detailed design and the results included in the Foundation Design Report.





## **8.0 MISCELLANEOUS**

This report was prepared by Ms. Dirka U. Prout, P.Eng. under the direction of the Team Leader, Dr. Storer J. Boone, P.Eng. This report was reviewed by Mr. Fintan J. Heffernan, P.Eng., the Designated MTO Contact and Quality Control Auditor for this assignment.

**GOLDER ASSOCIATES LTD.**

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TABLE I

**COMPARISON OF FOUNDATION ALTERNATIVES – REPLACEMENT STRUCTURE**

South Saugeen River Bridge, Site No. 35-5  
Highway 89  
GWP 3035-11-00

<b>FOUNDATION OPTION</b>	<b>FEASIBILITY</b>	<b>ADVANTAGES</b>	<b>DISADVANTAGES</b>	<b>RELATIVE COSTS</b>	<b>RISKS/ CONSEQUENCES</b>
Spread footings supported on compact to dense sandy silt till/sand and gravel	<ul style="list-style-type: none"> <li>• Feasible</li> <li>• Preferred technical alternative</li> </ul>	<ul style="list-style-type: none"> <li>• Ease of construction</li> <li>• Less expensive than deep foundation options</li> <li>• Compatible with full depth retaining wall type abutments which are featured in all three bridge design options selected by the structural designers</li> </ul>	<ul style="list-style-type: none"> <li>• Not compatible with integral abutments</li> <li>• Dewatering required</li> <li>• Shallow foundations are more susceptible to scour and erosion than deep foundations</li> </ul>	<ul style="list-style-type: none"> <li>• Low to Moderate</li> </ul>	<ul style="list-style-type: none"> <li>• Relatively low risk provided groundwater is adequately controlled</li> </ul>
Spread footings supported on Granular A pad	<ul style="list-style-type: none"> <li>• Feasible</li> </ul>	<ul style="list-style-type: none"> <li>• Ease of construction</li> <li>• Reduced excavation and dewatering effort compared to spread footings on native ground</li> <li>• Less expensive than deep foundation options</li> <li>• Reduced potential for damage/undermining by scour</li> </ul>	<ul style="list-style-type: none"> <li>• Not compatible with integral abutments or full depth retaining wall type abutments</li> <li>• Shallow foundations are more susceptible to scour and erosion than deep foundations</li> <li>• Settlement performance dependent on care with which Granular A pad is constructed.</li> </ul>	<ul style="list-style-type: none"> <li>• Low</li> </ul>	<ul style="list-style-type: none"> <li>• Relatively low risk provided Granular A pad is properly constructed</li> </ul>



**COMPARISON OF FOUNDATION ALTERNATIVES – REPLACEMENT STRUCTURE**

<b>FOUNDATION OPTION</b>	<b>FEASIBILITY</b>	<b>ADVANTAGES</b>	<b>DISADVANTAGES</b>	<b>RELATIVE COSTS</b>	<b>RISKS/ CONSEQUENCES</b>
End bearing steel H-pile foundations driven to practical refusal into very dense sandy silt till/sand and gravel to gravel	<ul style="list-style-type: none"> <li>• Feasible but impractical</li> </ul>	<ul style="list-style-type: none"> <li>• High bearing resistance</li> <li>• Negligible settlement</li> <li>• Only solution compatible with integral abutments</li> <li>• Low displacement</li> <li>• More suitable than tube piles for hard driving conditions</li> </ul>	<ul style="list-style-type: none"> <li>• Difficulty driving through very dense sand and gravel and glacial till containing cobbles to appropriate embedment depths</li> <li>• More expensive than shallow foundations; cost competitive with tube piles</li> <li>• If integral abutments selected, the piles may have to be installed in pre-drilled holes to achieve sufficient fixity particularly at the west abutment.</li> <li>• Cannot be visually inspected at depth</li> <li>• Integrity inspection requires specialty dynamic testing</li> <li>• Impractical driven lengths (less than 3 to 5 metres)</li> </ul>	<ul style="list-style-type: none"> <li>• High</li> </ul>	<ul style="list-style-type: none"> <li>• Possible pile tip damage if piles are not adequately protected while driving through very dense soils</li> <li>• Variation in pile tip elevations</li> </ul>



**COMPARISON OF FOUNDATION ALTERNATIVES – REPLACEMENT STRUCTURE**

<b>FOUNDATION OPTION</b>	<b>FEASIBILITY</b>	<b>ADVANTAGES</b>	<b>DISADVANTAGES</b>	<b>RELATIVE COSTS</b>	<b>RISKS/ CONSEQUENCES</b>
End bearing concrete filled steel tube pile foundations driven to practical refusal into very dense sandy silt till/sand and gravel to gravel	<ul style="list-style-type: none"> <li>• Impractical</li> </ul>	<ul style="list-style-type: none"> <li>• High bearing resistance</li> <li>• Negligible settlement</li> <li>• Inspection for damage possible prior to concrete filling</li> </ul>	<ul style="list-style-type: none"> <li>• Not compatible with integral abutments</li> <li>• Difficulty driving through very dense sand and gravel containing cobbles to appropriate embedment depths</li> <li>• More expensive than shallow foundations; cost competitive with H-piles</li> <li>• Impractical driven lengths (less than 3 to 5 metres)</li> </ul>	<ul style="list-style-type: none"> <li>• High especially if piles are less than 5 metres in length</li> </ul>	<ul style="list-style-type: none"> <li>• Possible pile tip damage if piles are not adequately protected while driving through very dense soils</li> <li>• Variation in pile tip elevations</li> </ul>

- NOTES:
1. Relative costs are intended only to provide a comparison between alternatives rather than actual construction costs.
  2. Table to be read in conjunction with accompanying report.

Prepared By: DUP  
Checked By: SJB



## 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 1**

1 OF 1

**METRIC**

PROJECT 11-1132-0109-2000

W.P. 3035-11-00

LOCATION N 4871073.1, E 201350.4

ORIGINATED BY MA

DIST HWY 89

BOREHOLE TYPE POWER AUGER, HOLLOW STEM

COMPILED BY LMK

DATUM GEODETIC

DATE October 1, 2012

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
379.97	GROUND SURFACE					▽	379										14 36 37 13			
0.00	TOPSOIL, silty to sandy with gravel, cobbles and boulders																			
379.36																				
0.61	SAND AND GRAVEL, trace silt, with cobbles Compact Brown		1	SS	11															
378.60																				
1.37	SANDY SILT TILL, some clay, some gravel Dense to very dense Brown		2	SS	33															
			3	SS	75									○						
			4	SS	112															
			5	SS	100/ 250mm															
			6	SS	100/ 225mm							○								
374.79																				
5.18	SILTY SAND AND GRAVEL, cobbles, trace clay Very dense Brown		7	SS	100/ 250mm															
			8	SS	100/ 250mm															
372.81																				
7.16	SANDY SILT TILL, some clay, trace gravel, cobbles Very dense Grey		9	SS	64															



# RECORD OF BOREHOLE No 2

1 OF 1

**METRIC**

PROJECT 11-1132-0109-2000

W.P. 3035-11-00

LOCATION N 4871100.2, E 201386.7

ORIGINATED BY MA

DIST HWY 89

BOREHOLE TYPE POWER AUGER, HOLLOW STEM

COMPILED BY LMK

DATUM GEODETIC

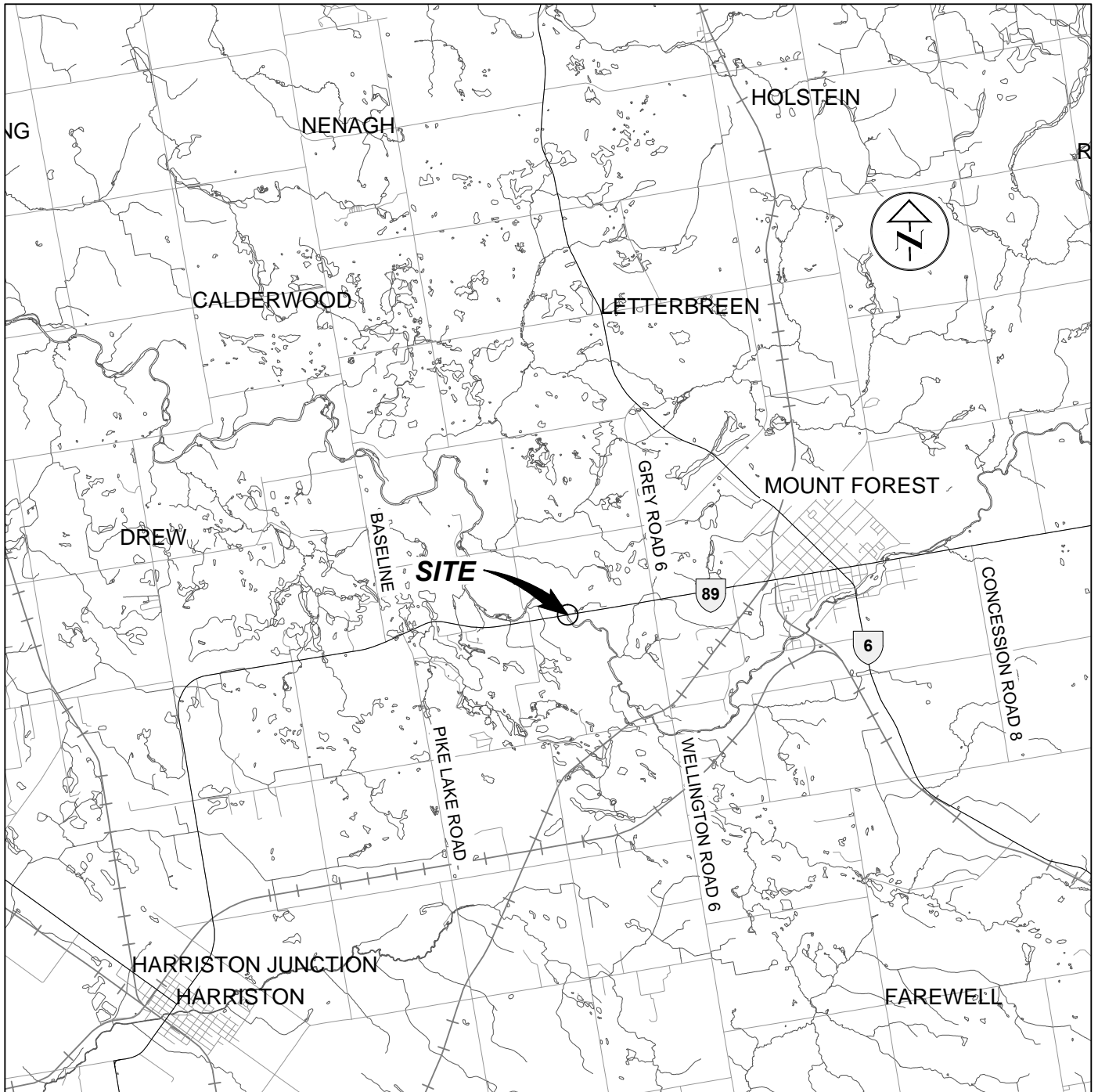
DATE October 2, 2012

CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT  γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					W <sub>P</sub>	W	W <sub>L</sub>		
381.24	GROUND SURFACE							20	40	60	80	100					
0.00	TOPSOIL, silty, trace sand Brown																
0.25	FILL, silty sand, trace gravel, trace clay																
0.46	Brown																
0.67	TOPSOIL, silty Brown		1	SS	13												
	SILTY SAND AND GRAVEL, trace clay, trace topsoil																
	Loose to compact Brown		2	SS	8												
379.26																	
1.98	CLAYEY SILT TILL, some sand, trace gravel		3	SS	13												2 15 46 37
	Stiff Brown																
378.50																	
2.74	SANDY SILT TILL, some gravel Compact Brown		4	SS	25												
			5	SS	25												
376.82																	
4.42	SANDY SILT, trace to some clay, trace gravel		6	SS	29												1 38 51 10
	Compact to very dense Brown																
			7	SS	15												
			8	SS	100/ 200mm												
374.69																	
6.55	SAND AND GRAVEL TO GRAVEL, trace clay, trace silt		9	SS	46												47 39 9 5
	Dense to very dense																
			10	SS	105/ 250mm												
372.86																	
8.38	SILT, trace sand, trace gravel Very dense Brown																
			11	SS	100/ 175mm												
370.48																	
10.76	END OF BOREHOLE		12	SS	100/ 75mm												
	Groundwater encountered at about elev. 380.2m during drilling on October 2, 2012.																
	Water level measured at elev. 381.24m after installation on October 2, 2012.																
	Water level measured at elev. 380.33m on October 24, 2012.																
	Installation decommissioned following measurement on October 24, 2012.																

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





0 SCALE 2000 4000m  
1:100,000

## REFERENCE

PLAN BASED ON CANMAP STREETFILES V.2008.5.

## NOTE

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

PROJECT

SOUTH SAUGREEN RIVER BRIDGE, SITE 35-5  
HIGHWAY 89 STRUCTURE REHABILITATIONS  
GWP 3035-11-00

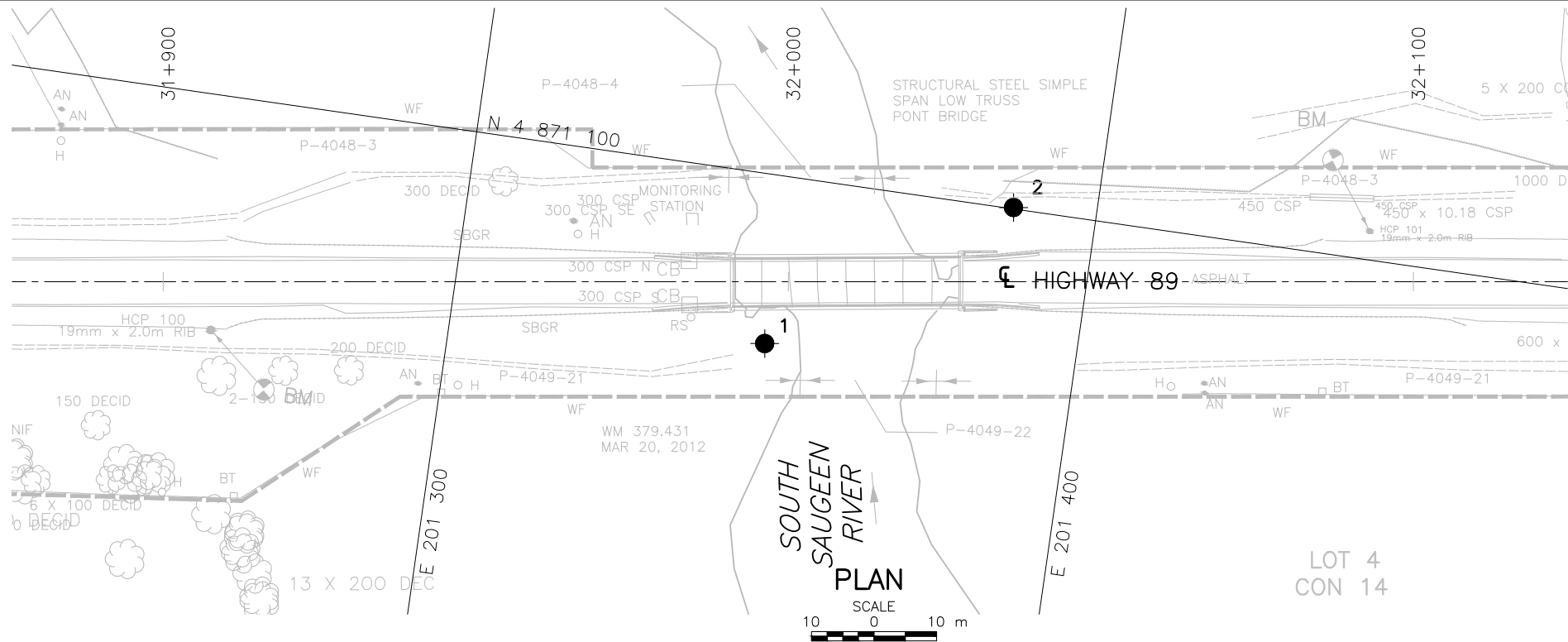
TITLE

## KEY PLAN



PROJECT No. 11-1132-0109			FILE No. 1111320109-2000-F01001	
CADD	LMK	Dec. 11/12	SCALE AS SHOWN	REV. 0
CHECK			<b>FIGURE 1</b>	





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

CONT No.  
WP No. 3035-11-00

SOUTH SAUGEEN RIVER BRIDGE  
HIGHWAY 89 STRUCTURE REPLACEMENTS

BOREHOLE LOCATIONS AND SOIL STRATA

SHEET

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

KEY PLAN

LEGEND

Borehole - Current Investigation

Seal

Standpipe

Standard Penetration Test Value

Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)

WL in standpipe, measured on October 24, 2012

WL encountered during drilling

No.	ELEVATION	CO-ORDINATES (MTM ZONE 10)	
		NORTHING	EASTING
1	379.97	4 871 073.1	201 350.4
2	381.24	4 871 100.2	201 386.7

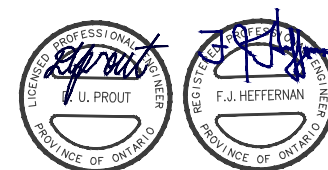
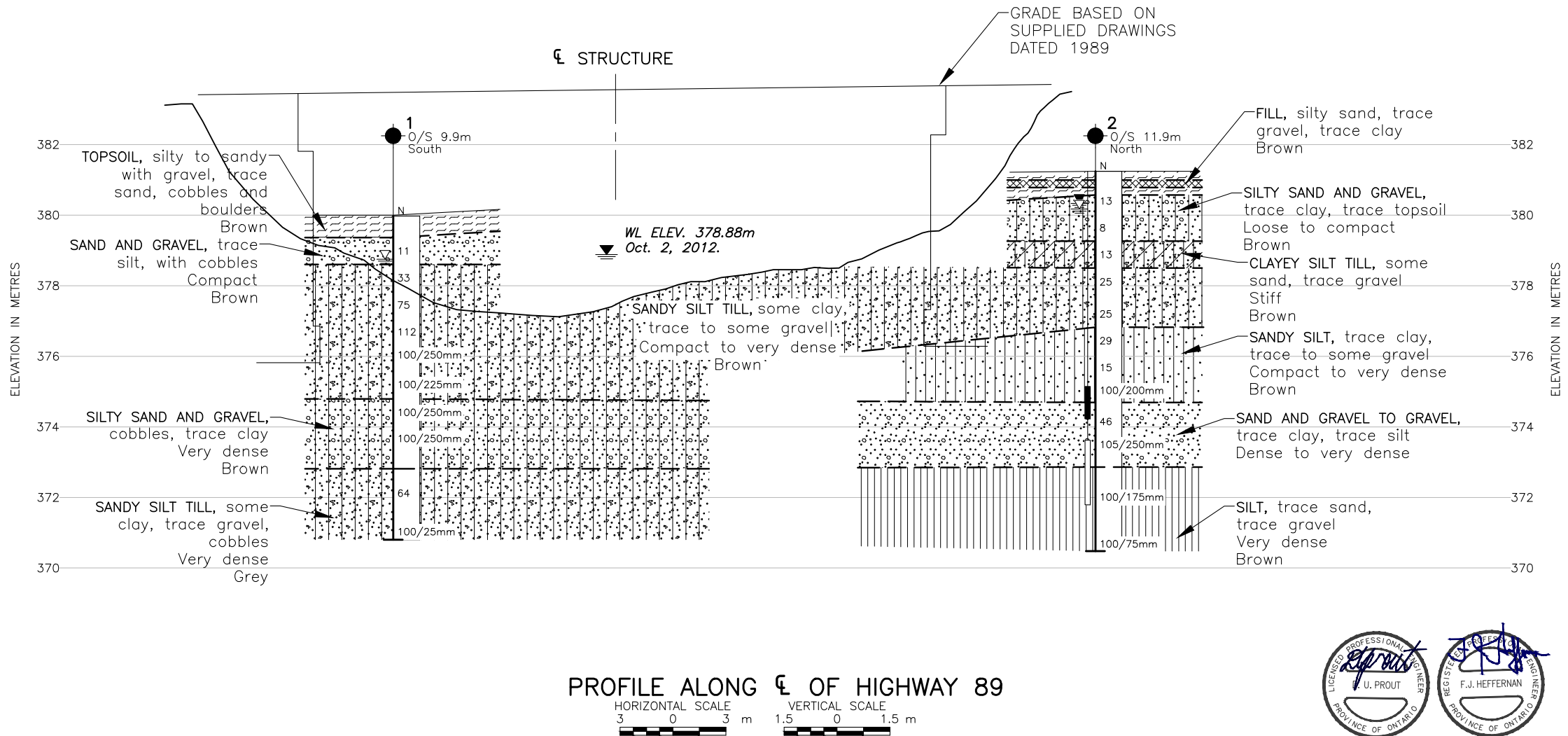
**NOTES**

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 Morrison Hershfield Limited.



NO.	DATE	BY	REVISION
Geocres No.	40P15-46		
HWY.	89	PROJECT NO.	11-1132-0109
SUBM'D.	DUP	CHKD.	DUP
DRAWN:	LMK	CHKD.	FJH
DATE:	Dec. 13/12	DIST.	SITE: 35-5
APPD.		DWG.	1

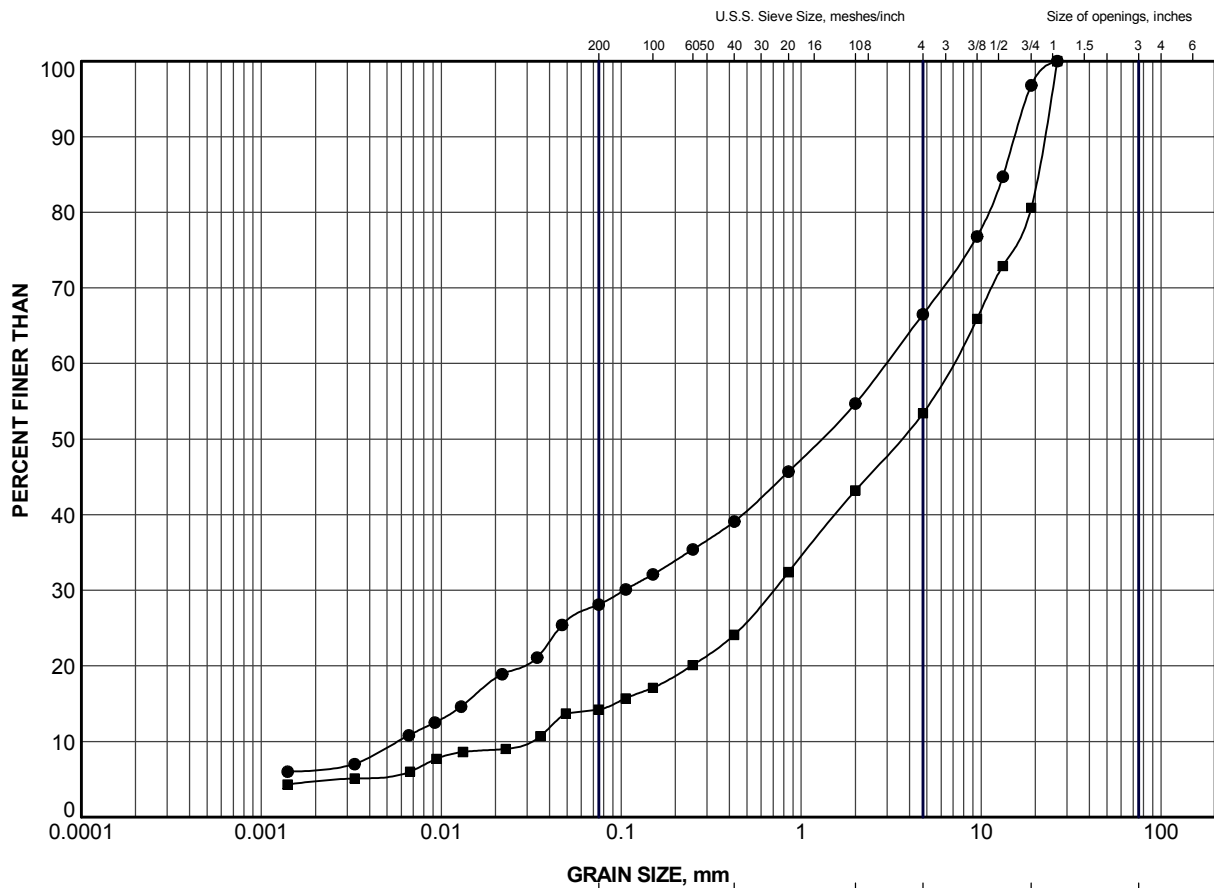




# **APPENDIX A**

## **Laboratory Test Data**






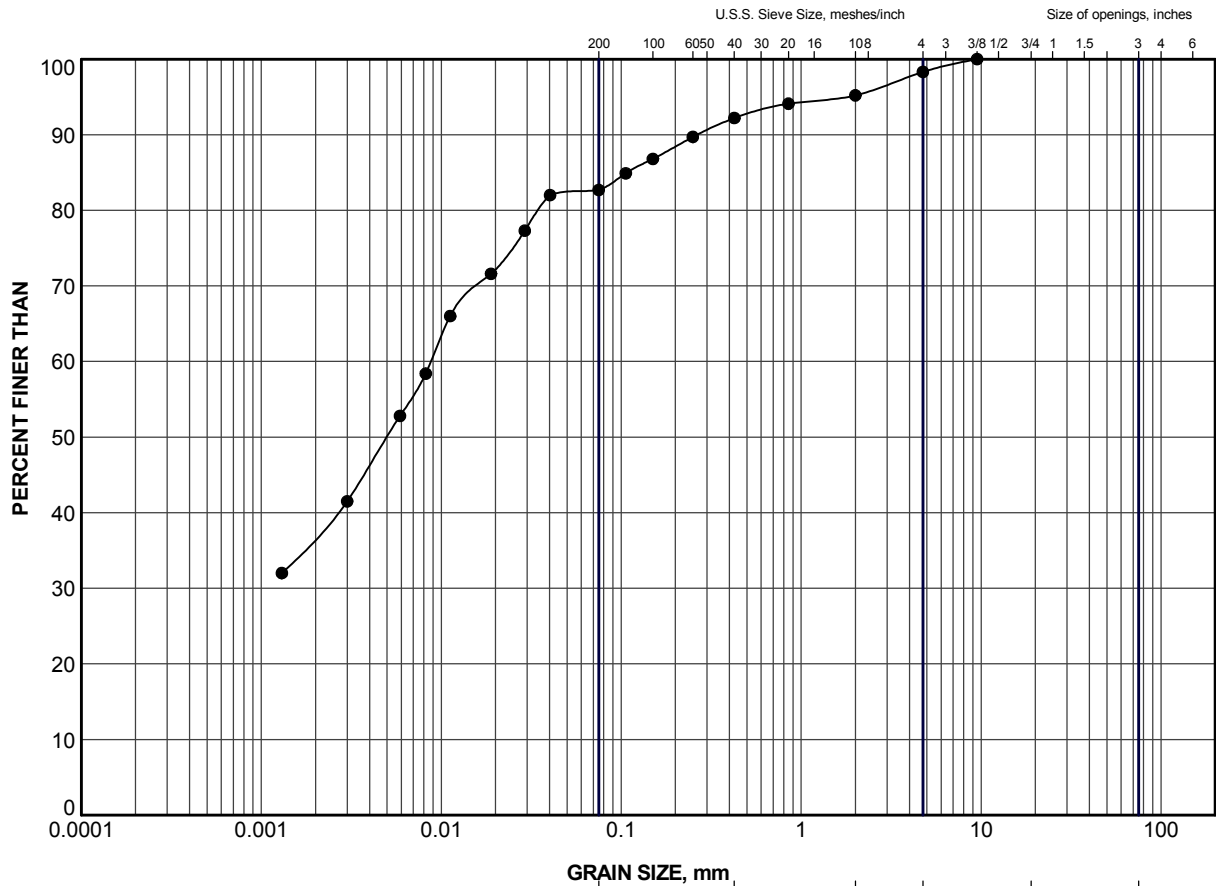
GRAIN SIZE, mm						
CLAY AND SILT	fine	medium	coarse	fine	coarse	Cobble Size
	SAND SIZE			GRAVEL SIZE		

#### LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	1	8	373.7
■	2	9	374.2


PROJECT				SOUTH SAUGREEN RIVER BRIDGE, SITE 35-5 HIGHWAY 89 STRUCTURE REPLACEMENTS GWP 3035-11-00			
TITLE				GRAIN SIZE DISTRIBUTION SAND AND GRAVEL			
PROJECT No:11-1132-0109-2000				FILE No. 1111320109-2000-F010A1			
DRAWN		LMK		Dec 12/12		SCALE N/A REV.	
CHECK						FIGURE A-1	
 <b>Golder Associates</b> LONDON, ONTARIO							



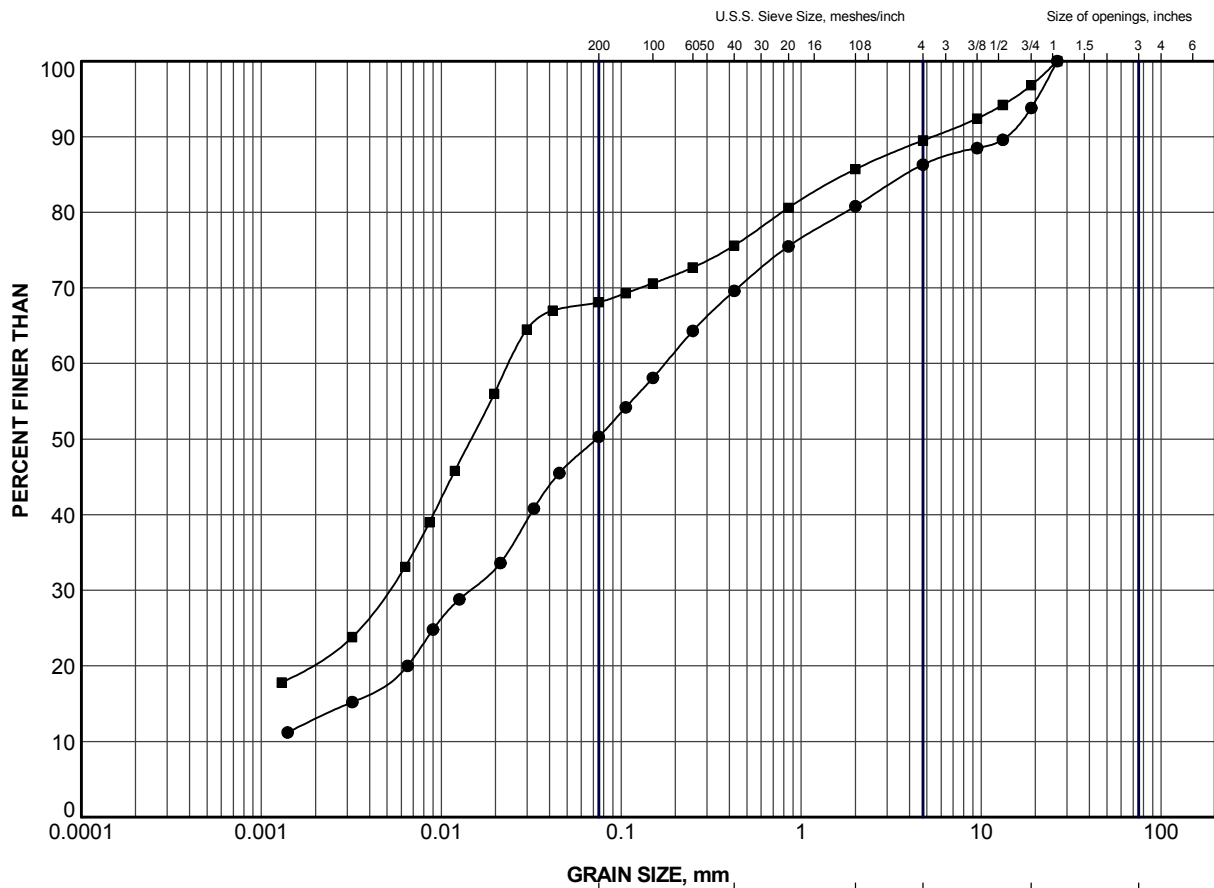


CLAY AND SILT	fine	medium	coarse	fine	coarse	Cobble Size
	SAND SIZE			GRAVEL SIZE		

LEGEND			
SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	2	3	378.9

PROJECT				SOUTH SAUGEE RIVER BRIDGE, SITE 35-5 HIGHWAY 89 STRUCTURE REPLACEMENTS GWP 3035-11-00					
TITLE				GRAIN SIZE DISTRIBUTION CLAYEY SILT TILL					
				PROJECT No:11-1132-0109-2000		FILE No. 1111320109-2000-F010A2			
				DRAWN	LMK	Dec 12/12	SCALE	N/A	REV.
				CHECK			FIGURE A-2		






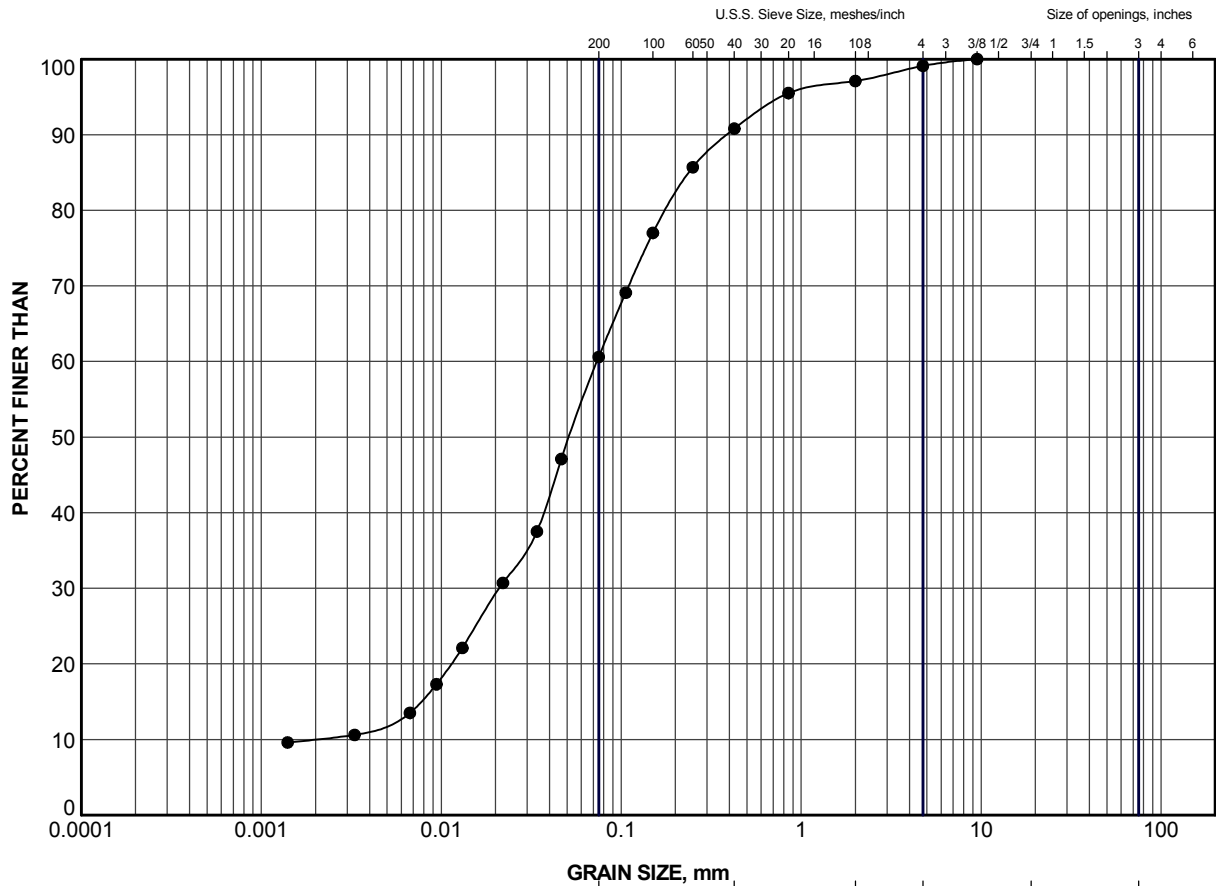
GRAIN SIZE, mm						
CLAY AND SILT	fine	medium	coarse	fine	coarse	Cobble Size
	SAND SIZE			GRAVEL SIZE		

#### LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	1	3	377.5
■	1	9	372.1


PROJECT				SOUTH SAUGEE RIVER BRIDGE, SITE 35-5 HIGHWAY 89 STRUCTURE REPLACEMENTS GWP 3035-11-00			
TITLE				GRAIN SIZE DISTRIBUTION SANDY SILT TILL			
PROJECT No:11-1132-0109-2000				FILE No. 1111320109-2000-F010A3			
DRAWN		LMK		Dec 12/12		SCALE N/A REV.	
CHECK						FIGURE A-3	
 <b>Golder Associates</b> LONDON, ONTARIO							





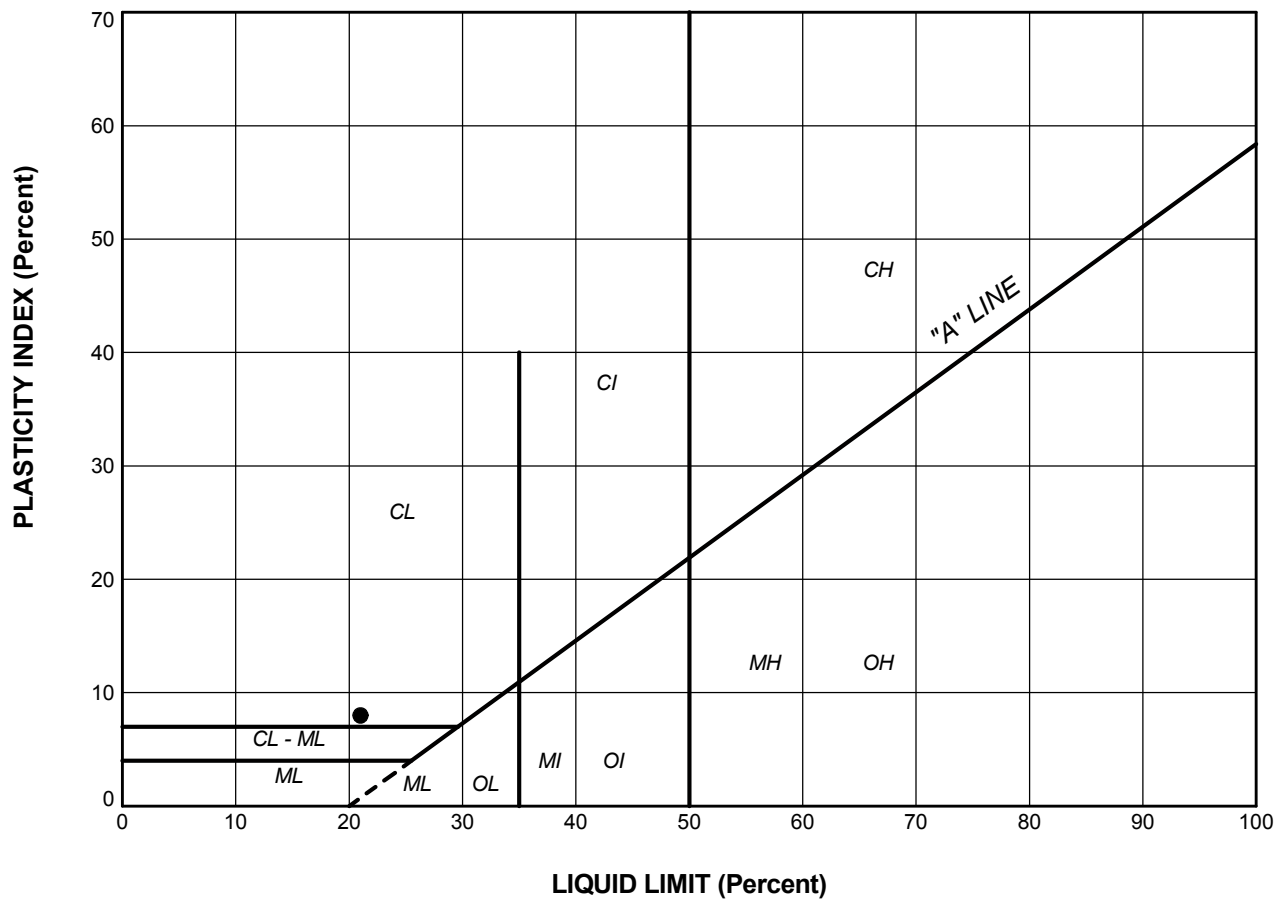
CLAY AND SILT	fine	medium	coarse	fine	coarse	Cobble Size
	SAND SIZE			GRAVEL SIZE		

LEGEND			
SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	2	6	376.4

PROJECT				SOUTH SAUGEE RIVER BRIDGE, SITE 35-5 HIGHWAY 89 STRUCTURE REPLACEMENTS GWP 3035-11-00					
TITLE				GRAIN SIZE DISTRIBUTION SANDY SILT					
 <b>Golder Associates</b> LONDON, ONTARIO				PROJECT No:11-1132-0109-2000		FILE No. 1111320109-2000-F010A4			
				DRAWN	LMK	Dec 12/12	SCALE	N/A	REV.
				CHECK			<b>FIGURE A-4</b>		

LDN\_MTO\_GSD\_GLDR\_LDN.GDT





### LEGEND

SYMBOL	BOREHOLE	SAMPLE	LL(%)	PL(%)	PI
●	2	3	21.0	13.0	8.0

PROJECT		SOUTH SAUGEEN RIVER BRIDGE, SITE 35-5 HIGHWAY 89 STRUCTURE REPLACEMENTS GWP 3035-11-00			
TITLE		PLASTICITY CHART			
PROJECT No.11-1132-0109-2000		FILE No.		1111320109-2000-F010A5	
DRAWN	LMK	Dec 11/12	SCALE	N/A	REV.
CHECK			FIGURE A-5		







# **APPENDIX B**

## **Site Photographs**





## APPENDIX B PHOTOGRAPHS



Photograph 1: North elevation as viewed from NE quadrant.



Photograph 2: Looking east from SW approach.





## APPENDIX B PHOTOGRAPHS



Photograph 3: Looking east from NW abutment at the downstream side of the bridge.



Photograph 4: Looking west from SE abutment at upstream side of bridge.



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