



January 2013

## FOUNDATION INVESTIGATION AND DESIGN REPORT

**South Saugeen River Bridge Replacement, Site No. 35-27  
Highway 89 Structure Replacements and Rehabilitation  
From 6.0 Km West of Mount Forest to Shelburne  
GWP 3049-08-00  
Ministry of Transportation, Ontario - West Region**

**Submitted to:**

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REPORT



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## Table of Contents

### **PART A - FOUNDATION INVESTIGATION REPORT**

<b>1.0 INTRODUCTION.....</b>	<b>1</b>
<b>2.0 SITE AND PROJECT DESCRIPTION .....</b>	<b>2</b>
2.1 Site Geology .....	2
<b>3.0 INVESTIGATION PROCEDURES .....</b>	<b>3</b>
<b>4.0 SUBSURFACE CONDITIONS.....</b>	<b>4</b>
4.1 Site Stratigraphy .....	4
4.1.1 Topsoil .....	4
4.1.2 Pavement Structure .....	4
4.1.3 Fill .....	5
4.1.4 Silty Clay .....	5
4.1.5 Silt .....	5
4.1.6 Sand to Silty Sand .....	5
4.1.7 Sand and Gravel .....	6
4.1.8 Sandy Silt Till .....	6
4.2 Groundwater Conditions .....	6
<b>5.0 MISCELLANEOUS .....</b>	<b>8</b>

### **PART B - FOUNDATION DESIGN REPORT**

<b>6.0 ENGINEERING RECOMMENDATIONS.....</b>	<b>9</b>
6.1 General .....	9
6.2 Existing Structure.....	9
6.3 Bridge Foundations.....	9
6.3.1 Shallow Foundations.....	10
6.3.2 Deep Foundations.....	12
6.4 Liquefaction Potential and Seismic Analysis.....	14
6.4.1 Seismic Parameters .....	14
6.4.2 Seismic Hazard Assessment .....	14
6.5 Lateral Earth Pressures .....	15
6.6 Retaining Walls.....	16



## FOUNDATION INVESTIGATION AND DESIGN REPORT SOUTH SAUGEEN RIVER BRIDGE, SITE NO. 35-27

6.7	Construction Considerations.....	17
6.7.1	Shallow Foundations.....	17
6.7.2	Driven H-Piles .....	17
6.7.3	Driven Sheet Piles .....	18
6.8	Excavations .....	18
6.9	Embankments.....	19
6.10	Staging and Temporary Roadway Protection .....	19
<b>7.0</b>	<b>MISCELLANEOUS .....</b>	<b>22</b>
TABLE I - Comparison of Foundation Alternatives Replacement Structure		
LIST OF ABBREVIATIONS		
LIST OF SYMBOLS		
RECORD OF BOREHOLE SHEETS		
FIGURE 1 - Key Plan		
DRAWING 1 - Borehole Locations and Soil Strata		
DRAWING 2 – Soil Strata		
<b>APPENDICES</b>		
<b>APPENDIX A</b>		
Laboratory Test Data		
<b>APPENDIX B</b>		
Site Photographs		



**PART A**

**FOUNDATION INVESTIGATION REPORT**

**SOUTH SAUGEEN RIVER BRIDGE REPLACEMENT, SITE NO. 35-27  
HIGHWAY 89 STRUCTURE REPLACEMENTS AND REHABILITATIONS  
FROM 6.0 KM WEST OF MOUNT FOREST TO SHELBURNE  
GWP 3049-08-00  
MINISTRY OF TRANSPORTATION, ONTARIO - WEST REGION**



## **1.0 INTRODUCTION**

Golder Associates Ltd. (Golder Associates) 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 and detail design work for GWP 3049-08-00. 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 15+330 (Site No. 35-27).

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 Associates' 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 Associates 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 immediately west of Wellington Road 16 and is on the boundary of Wellington County and Grey County, Ontario. 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 bridge was constructed in 1930 and consists of a single span concrete "tee-beam" structure. Site photographs are provided in Appendix B.

Adjacent land use is typically rural agricultural and residential. The local topography is relatively flat with ground surface elevations in the vicinity of the bridge ranging from 480 to 483 metres.

It is understood that the existing structure will be demolished and replaced with a 10.4 metre long single span, precast concrete structure. The abutments are to be founded on cast-in-place (CIP) concrete spread footings. The replacement structure will be erected at the same location as the existing bridge. No details of a highway profile grade change at the abutments were provided.

### **2.1 Site Geology**

This project lies within the physiographic region of southern Ontario known as the Dundalk Till Plain<sup>1</sup>. This region is a drumlinized till plain characterized by swamps or bogs and poorly drained depressions.

Based on the Ministry of Natural Resources Map P.1556 entitled "Quaternary Geology, Durham Area, Southern Ontario", the site lies in an area mapped as consisting primarily of Tavistock Till consisting of silt to clayey silt glacial till. Although the predominant sedimentary geologic deposit is the extensive Tavistock Till, the South Saugeen River has modified the near-surface soils through erosion and redeposition of alluvium that will be of softer and looser character than the underlying glacial till. The Geologic Survey of Canada Map 1263A entitled "Geology, Toronto-Windsor Area, Ontario" indicates that the bedrock underlying the glacial deposits in the area of the site is dolomite of the Guelph Formation of Middle Silurian age. Based on the Ministry of Natural Resources Preliminary Map P.1836 entitled "Bedrock Topography, Durham Area, Southern Ontario", the bedrock surface at the site may be at about elevation 462 metres or some 20 metres below ground surface.

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



### 3.0 INVESTIGATION PROCEDURES

The field work for the investigation was carried out between August 28 and September 10, 2012, during which time 6 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	4 875 164	224 293	481.36	12.56
2	4 875 134	224 304	482.37	18.50
3	4 875 133	224 274	481.15	17.04
4	4 875 158	224 276	481.24	15.30
5	4 875 143	224 256	482.43	6.55
6	4 875 151	224 315	482.82	6.19

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. The boreholes were terminated between 6.2 and 18.5 metres below the existing pavement or ground surface. Groundwater conditions in the boreholes were observed throughout the drilling operations and a groundwater observation well was installed in borehole 1 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.



## **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 the existing pavement structure or topsoil overlying silty clay, silt, sand, sand and gravel, and sandy silt till. A boulder was encountered within the sandy silt glacial till. As a result of the glacial origin of the soils at this site, cobbles and boulders should be anticipated within all these deposits.

The locations and elevations of the boreholes, together with the interpreted stratigraphic profiles, are shown on Drawings 1 to 2. 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**

A layer of topsoil, approximately 400 to 850 millimetres in thickness, was encountered at the ground surface in boreholes 1, 3 and 4. A 670 millimetre thick layer of loose buried topsoil was encountered beneath the fill in borehole 5 at elevation 481.7 metres. The buried topsoil had a standard penetration testing N value<sup>2</sup> of 6 blows per 0.3 metres.

#### **4.1.2 Pavement Structure**

Asphaltic concrete pavement was encountered at the ground surface in boreholes 2, 5 and 6. The asphaltic concrete was about 90 to 215 millimetres thick at the borehole locations.

Pavement granular base course materials were encountered beneath the asphalt in boreholes 2, 5 and 6. The granular base materials were about 90 to 180 millimetres thick.

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<sup>2</sup> 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.



Pavement granular subbase materials were encountered beneath the granular base course materials in boreholes 2, 5 and 6. The granular subbase materials were about 130 to 460 millimetres thick.

#### **4.1.3 Fill**

Fill materials were encountered beneath the pavement structure in boreholes 2 and 6. The fill consisted of topsoil and silt with sand and trace gravel and contained cobbles. The fill materials were 1.2 and 1.7 metres thick with measured N values of 5 to 17 blows per 0.3 metres. A sample of the fill had a water content of 34 per cent. Grain size distribution curves for samples of the fill recovered from the standard penetration testing are provided on Figure A-1.

#### **4.1.4 Silty Clay**

A 0.7 metre thick layer of firm silty clay was encountered beneath the buried topsoil in borehole 5 at elevation 481.1 metres. The silty clay had a measured N value of 5 blows per 0.3 metres.

#### **4.1.5 Silt**

Layers of loose to compact silt were encountered beneath the fill in borehole 2, beneath the silty sand and gravel in borehole 3 and beneath the silty clay in borehole 5 between elevations 479.0 and 480.5 metres. The silt layers were 0.4 to 1.7 metres thick, had measured N values of 9 to 23 blows per 0.3 metres and water contents of 18 and 24 per cent. Grain size distribution curves for samples of the silt are provided on Figure A-2.

A 0.6 metre thick layer of loose sandy silt was encountered beneath the topsoil in borehole 4 at elevation 480.5 metres. The sandy silt had an N value of 6 blows per 0.3 metres.

#### **4.1.6 Sand to Silty Sand**

Layers of compact to very dense sand were encountered beneath the fill in borehole 6, beneath the silt in borehole 5, beneath the silty sand in borehole 5, and beneath the sand and gravel in boreholes 1 and 3 between elevations 476.7 and 480.7 metres. Where fully penetrated, the sand layers were 0.5 to 1.5 metres thick. The sand had measured N values of 15 blows to 51 blows per 0.3 metres and water contents of 9 to 15 per cent with an average water content of about 12 per cent.



Layers of loose to compact silty sand were encountered beneath the topsoil in borehole 1, beneath the sandy silt in borehole 4 and beneath the silt in borehole 5 between elevations 478.8 and 481.0 metres, respectively. The silty sand layers were 0.6 to 0.8 metres thick with N values of 4 and 15 blows per 0.3 metres and water contents of 13 to 28 per cent. A grain size distribution curve for a sample of the silty sand is provided on Figure A-3.

#### **4.1.7 Sand and Gravel**

Layers of compact to very dense sand and gravel were encountered beneath the topsoil in borehole 3, beneath the silt in boreholes 2 and 3, beneath the sands in boreholes 1, 3, 4, 5 and 6, and beneath the sand and gravel in borehole 6 between elevations 477.5 and 480.3 metres. The sand and gravel layers in borehole 3 at elevations 477.5 and 480.3 metres and in borehole 6 at elevation 478.4 metres were described as silty. Borehole 5 was terminated in the sand and gravel after exploring it for about 1.4 metres. Where fully penetrated, the sand and gravel layers were 0.3 to 4.6 metres thick. The sand and gravel had measured N values of 10 blows per 0.3 metres to 101 blows per 250 millimetres and water contents of 5 to 12 per cent with an average water content of about 7 per cent. Grain size distribution curves for samples of the sand and gravel are provided on Figure A-4.

Cobbles were encountered in the sand and gravel in boreholes 1 and 4. The presence of both cobbles and boulders should be anticipated in the sand and gravel deposits.

#### **4.1.8 Sandy Silt Till**

Layers of dense to very dense sandy silt glacial till were encountered beneath the sand and gravel in boreholes 1 to 4 between elevations 471.3 and 475.7 metres. Boreholes 1 to 4 were terminated in the sandy silt till after exploring it for 2.5 to 11.8 metres. The sandy silt till had measured N values of 31 blows per 0.3 metres to 100 blows per 50 millimetres and water contents of 7 to 8 per cent. Grain size distribution curves for samples of the sandy silt till recovered from the standard penetration testing are provided on Figure A-5. A boulder was encountered within the sandy silt till in borehole 3 at about elevation 465.8 metres.

Cobbles and boulders should be anticipated based on the depositional history of glacial tills.

### **4.2 Groundwater Conditions**

Groundwater conditions were observed during and on completion of drilling and sampling and a groundwater observation well was installed in borehole 1. Installation details are provided on the corresponding Record of Borehole sheet following the text of this report. Groundwater was encountered in boreholes 1 to 6 at depths of



## FOUNDATION INVESTIGATION AND DESIGN REPORT SOUTH SAUGEEN RIVER BRIDGE, SITE NO. 35-27

1.5 to 3.5 metres or between elevation 479.3 and 480.3 metres. A summary of the encountered groundwater levels is provided in the following table:

Borehole	Ground Surface Elevation (m)	Encountered Groundwater Elevation (m)	Measured Groundwater Elevation (m)		
			Oct. 1, 2012	Oct. 24, 2012	Dec. 28, 2012
1	481.36	479.5	479.58	479.86	480.29
2	482.37	479.5	-	-	-
3	481.15	479.6	-	-	-
4	481.24	479.7	-	-	-
5	482.43	480.3	-	-	-
6	482.82	479.3	-	-	-

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 479.47 metres on September 10, 2012 and at elevation 479.8 metres on December 28, 2012. The most recent groundwater measurement was obtained on December 28, 2012. On this date, the water level in the groundwater observation well installed in borehole 1 was at about elevation 480.3 metres.

Based on the colour change from brown to grey, measured and encountered groundwater levels, the surrounding topography, and the water level in the South Saugeen River, the inferred groundwater level ranges between elevation 479.5 and 480.0 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 and Mr. Dan Babcock, P.Eng. under the direction of Mr. David J. Mitchell. The laboratory testing was carried out at Golder Associates' London laboratory under the direction of Mr. Chris M. Sewell. The laboratory is an accredited participant in the MTO Soil and Aggregate Proficiency Program and is certified by the Canadian Council of Independent Laboratories for testing Types C and D aggregates. This report was prepared by Mr. Tyson Pitt, 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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TP/DUP/SJB/FJH/cr

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**PART B**

**FOUNDATION DESIGN REPORT**

**SOUTH SAUGEEN RIVER BRIDGE REPLACEMENT, SITE NO. 35-27  
HIGHWAY 89 STRUCTURE REPLACEMENTS AND REHABILITATIONS  
FROM 6.0 KM WEST OF MOUNT FOREST TO SHELBURNE  
GWP 3049-08-00  
MINISTRY OF TRANSPORTATION, ONTARIO - WEST REGION**



## **6.0 ENGINEERING RECOMMENDATIONS**

### **6.1 General**

This section of the report provides our recommendations on the foundation aspects of the design of the replacement of the existing South Saugeen River Bridge (Site No. 35-27). 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.

Based on the information provided by MH, the replacement structure will be a 10.4 metre long, single span, precast concrete structure. The replacement structure will be erected at the same location as the existing bridge with no change in the pavement grade. The abutments are to be founded on 1.80 metre wide pre-cast concrete spread footings founded at elevation 478.64 metres which is the same elevation as the existing footings. The new foundations will abut the existing footings which are to be left in place. The replacement will be carried out using staged construction. A temporary protection system will be offset 1.0 metre from the bridge centreline. The portion of the existing superstructure in front of the temporary protection system will be demolished then replaced while traffic is maintained on the remaining portion behind the temporary protection system. Once the new portion is complete and in service, the remaining portion of the existing superstructure will be demolished and the new structure completed.

### **6.2 Existing Structure**

The South Saugeen River Bridge was built in 1930. The structure consists of a single span, concrete "tee-beam" structure. The overall structure is approximately 6.7 metres long. No as-built drawings were provided; however, based on later drawings, dated 1959, the existing bridge was founded on spread footings at about elevation 478.6 metres. It is unlikely that there will be a conflict between the old and new foundations as the length of the new bridge will be 10.4 metres; however, this should be confirmed during removal of the existing bridge and related works.

### **6.3 Bridge Foundations**

The subsurface soil conditions typically consist of the existing pavement structure or topsoil overlying silts, sands, sand and gravel, and sandy silt till. The prevailing groundwater level was inferred to vary between approximately elevation 479.5 and 480.0 metres.



## FOUNDATION INVESTIGATION AND DESIGN REPORT SOUTH SAUGEN RIVER BRIDGE, SITE NO. 35-27

Deep foundations such as driven steel H-piles are geotechnically feasible but installation may be difficult due to the presence of dense to very dense sand and gravel and sandy silt till materials which contain cobbles and boulders. Pile foundations are not considered economical for this bridge since competent foundation materials are available near the surface. Shallow foundations consisting of pre-cast concrete spread footings have been selected by the design team as the preferred foundation system.

A comparison of foundation alternatives is presented in Table I. The costs provided are estimates meant to provide an order of magnitude comparison for the alternatives for foundation engineering purposes and should not be considered to be indicative of actual construction costs. It should be noted, however, that traffic control and structural issues that may either complicate or simplify construction staging will significantly influence the actual costs.

### 6.3.1 Shallow Foundations

Footings constructed at the design elevation of 478.64 metres will encounter dense sand and gravel and compact silt along the west abutment footing and compact to dense sand to sand and gravel along the east abutment footing. The silt thickness beneath the footing is anticipated to be about 1.2 metres based on borehole 3. The silt is underlain by compact to very dense sand and gravel. The silt foundation subgrade will be very sensitive to disturbance during foundation preparation particularly if exposed to moist or wet conditions or freezing temperatures. Special precautions including dewatering are to be taken during construction to protect the silty subgrade as noted in Section 6.7. The following factored geotechnical resistances at Ultimate Limit States (ULS) and geotechnical reactions at Serviceability Limit States (SLS) should be used for designing the spread foundations erected at the design elevation of 478.64 metres. The SLS value corresponds to an estimated total settlement of 25 millimetres.

Location	Borehole	Founding Material	Factored Geotechnical Resistance at ULS (kPa)	Geotechnical Reaction at SLS (kPa)
West Abutment				
- North Side	4	Dense sand and gravel	375	250
- South Side	3	Compact silt	375	250
East Abutment				
- North Side	1	Compact sand to sand and gravel	450	300
- South Side	2	Dense sand and gravel	450	300

Alternatively, footings for both abutments can be constructed at elevation 479.5 metres using a factored geotechnical resistance at ULS of 375 kilopascals and a geotechnical reaction of 250 kilopascals at SLS. The SLS value corresponds to an estimated total settlement of 25 millimetres. The advantage of this alternative is that the footings will be at or near the inferred groundwater level and a reduced depth of dewatering of 0.5



## FOUNDATION INVESTIGATION AND DESIGN REPORT SOUTH SAUGREEN RIVER BRIDGE, SITE NO. 35-27

metres versus 1.3 metres will be required to lower the groundwater level to a minimum of 0.5 metres below the footing elevation. As shown in the table below, subexcavation of approximately 0.4 metres of loose silty sand to the underlying dense sand and gravel at approximate elevation 479.1 metres will be required in the northwest quadrant based on borehole 4. The subgrade in this area can be brought up to grade with lean concrete.

Location	Borehole	Founding Material
West Abutment		
- North Side	4	Loose silty sand <sup>1</sup>
- South Side	3	Compact silty sand and gravel
East Abutment		
- North Side	1	Compact to very dense sand
- South Side	2	Compact to dense sand and gravel

Note: 1 – The loose silty sand is to be removed to competent subgrade and replaced to underside of footing elevation with lean concrete fill.

These recommended maximum foundation elevations do not account for any potential scour. Evaluation of additional foundation embedment or scour protection is outside the scope of work for this report and may require separate evaluation by a hydrologist or hydraulic engineer qualified in river engineering. Scour protection and the minimum footing embedment required for scour protection should meet the requirements of the MTO's Structural Manual and Section 1.9.5 of the CHBDC.

Given that this bridge is to be constructed over a watercourse that has likely meandered in the past, some degree of additional excavation and replacement of unsuitable silt soils should be anticipated. Where additional excavation is required to remove unsuitable soils or where it is necessary to raise the subgrade level to an elevation suitable for accommodating the precast concrete elements, the material required beneath the foundations should consist of concrete of 20 megapascal strength. Use of compacted granular materials could be problematic because of wet conditions.

### **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 cast-in-place 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 silty sand/sand/sand and gravel	angle of friction	32°
	$\tan \delta$	0.62
Footings on silt	angle of friction	25°
	$\tan \delta$	0.47

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 30 per cent.



### **Frost Protection**

All footings should be provided with a minimum of 1.6 metres of earth cover or thermal equivalent for frost protection purposes.

### **6.3.2 Deep Foundations**

While the design is based on use of precast concrete shallow foundation elements, recommendations are provided within this report for a deep foundation alternative in keeping with past MTO practices. If necessary and an alternate bridge design is selected, the bridge can be founded on deep foundations such as driven steel HP 310 x 110 piles driven to practical refusal in the very dense sandy silt till. Steel H-piles are suitable for both conventional and integral abutments whereas steel tube piles cannot be used for integral abutments due 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.

End bearing HP 310 x 110 piles driven to practical refusal in the very dense sand and gravel or sandy silt till to the depths noted in the following table may be designed using a factored geotechnical resistance at ULS of 1600 kilonewtons and a geotechnical reaction at SLS of 1350 kilonewtons. A cut-off elevation equivalent to the underside of spread footing elevation or elevation 478.6 metres has been assumed.

<b>Pile Location</b>	<b>BH</b>	<b>Proposed Tip Elevation (m)</b>	<b>Proposed Pile Length (m)</b>	<b>Founding Material</b>
West Abutment				
- north side	4	468.0	10.7	Sandy silt till
- south side	3	466.0	12.7	Sandy silt till
East Abutment				
- north side	1	471.0	7.7	Sandy silt till
- south side	2	466.0	12.7	Sandy silt till

Piles supporting integral abutments require pre-augering and placement of a corrugated steel pipe (CSP) liner filled with loose uniform sand around the upper 3 metres of the pile to reduce resistance to lateral movement. A Non-Standard Special Provision (NSSP) for CSP integral abutments detailing the sand gradation should be included in the Contract Documents.

The steel H-piles should be installed and monitored in accordance with Ontario Provincial Standard Drawing (OPSD) 3000.100, OPSD 3000.150 and OPSS 903. The piles should be equipped with Type I bearing shoes as per OPSD 3000.100. The maximum ultimate resistance of two times the factored ULS value shown in the above table should be noted on the foundation drawing. The foundation drawings should have a note stating that each pile is to be driven in accordance with Standard SS103-11 using the ultimate resistances and below the depths indicated in the above table. Construction considerations specific to piles are outlined in Section 6.7.



### Frost 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.

### Downdrag Load (Negative Skin Friction)

Downdrag loads are not applicable for piles at this location since there no grade change or platform widening will be undertaken and the overburden materials are granular in nature.

### Resistance to Lateral Loads

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

The horizontal reaction to the pile can be estimated using the following equation and ranges in subgrade reaction coefficient where:

- $k_s$  = coefficient of horizontal subgrade reaction (MPa/m) =  $n_h (z/d)$  for cohesionless soils
- $d$  = pile width or diameter (m)
- $n_h$  = constant of horizontal subgrade reaction (MPa/m)
- $z$  = depth below ground surface grade (m)

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

Soil Type	Elevation (m)		$n_h$ (MPa/m)
	From	To	
Granular backfill around piles and CSPs (for integral abutments)	Where applicable		5 – 10
Compact to dense sand and gravel, sand, sandy silt and silt (saturated)			
- West abutment	480	473	4 – 10
- East abutment	480	471	4 – 10
Dense to very dense sandy silt till (saturated)			
- West abutment	473	467	10 – 12
- East abutment	471	467	10 – 12

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



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

The lateral resistance for a single HP 310 x 110 pile of 125 kilonewtons at factored ULS and 55 kilonewtons at SLS may be used for design. The lateral resistances were calculated using Brom's hand calculation method as described in Federal Highways Administration (FHWA) Publication No. FHWA HI 97-013.<sup>3</sup> A free-headed pile was assumed with the vertical load applied at the ground surface. The SLS values are based on 10 millimetres of deflection at the ground surface.

## 6.4 Liquefaction Potential and Seismic Analysis

### 6.4.1 Seismic Parameters

The site is located near Mount Forest, Ontario. According to Table A.3.1.1 of the CHBDC, the zonal acceleration ratio,  $A$ , applicable to this site is 0.05. The corresponding acceleration related seismic zone,  $Z_a$  is 1.

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

### 6.4.2 Seismic Hazard Assessment

It is considered that the soils at the site are not susceptible to liquefaction and therefore a detailed evaluation of the liquefaction potential of the foundation soils is not considered warranted.

<sup>3</sup> Federal Highway Administration, 1998: *Design and Construction of Driven Pile Foundations, Workshop Manual – Volume 1*. Publication No. FHWA HI 97-013.



## 6.5 Lateral Earth Pressures

The lateral pressures acting on the bridge abutments 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 OPSS Granular A or Granular B Type III but with less than 5 per cent passing the No. 200 sieve should be used as backfill behind the abutments and walls. This fill should be compacted in loose lifts not greater than 200 millimetres in thickness in accordance with SP 105S10. Longitudinal drains and weep holes should be installed to provide positive drainage of the granular backfill. Other aspects of the abutment granular backfill requirements with respect to subdrains and frost taper should be in accordance with Ontario Provincial Standard Drawing (OPSD) 3101.150 and 3190.100.
- A compaction surcharge equal to 12 kPa should be included in the lateral earth pressures for the structural design of the abutment wall in accordance with CHBDC Figure 6.6. Compaction equipment should be used in accordance with SP 105S10.
- 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).
- 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.

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.

## 6.6 Retaining Walls

The wingwalls for the bridge replacement could consist of reinforced concrete gravity or cantilever walls or reinforced soil system (RSS) walls. Based on the results of the foundation investigation, the wingwalls may be founded on the native, undisturbed sand and/or sand and gravel at or below elevations 477.5 and 480.0 metres at the west and east abutments, respectively. For design purposes, a factored geotechnical resistance at ULS of 300 kPa and a geotechnical resistance at SLS of 200 kPa may be used for pre-cast and CIP walls as well as RSS walls on the sand and/or sand and gravel. The SLS value corresponds to 25 millimetres of settlement.

Select, free draining granular fill, in accordance with OPSS Granular B Type II gradation specifications should be used as backfill immediately adjacent to the retaining wall. As a minimum requirement, the granular backfill should be placed in a wedge-shaped zone defined by a 45 degree line extending up and back from the bottom of the rear face of the excavation.

Filtered longitudinal drains should be installed behind the wall(s) to provide positive drainage of the granular backfill. All granular backfill should be placed in maximum 200 millimetre thick loose lifts and uniformly compacted to at least 95 per cent of standard Proctor maximum dry density (SPMDD). Heavy compaction equipment, however, should not be used within the lateral distance behind any structure equal to the current height of the fill above the base of the structure.

Any proprietary RSS walls should be designed by the supplier and constructed in accordance with their specifications. The geotechnical aspects of the global stability of the detailed retaining wall design should be reviewed prior to construction.

The recommended geotechnical design parameters for the retaining walls are as follows:

Unit weight of granular backfill	22 kN/m <sup>3</sup>
Unit weight of water	9.8 kN/m <sup>3</sup>
Active earth pressure coefficient, $K_a$ (based on horizontal ground)	0.27
Coefficient of friction between granular backfill and founding soils	0.70



## **6.7 Construction Considerations**

The discussion presents construction related comments and recommendations pertaining to shallow foundations, and driven H-piles and driven sheet piles.

### **6.7.1 Shallow Foundations**

All footings are to be constructed and inspected in accordance with OPSS 902. A portion of the footing for the west abutment will be founded on the compact silt which is sensitive to disturbance. The silt subgrade should be carefully prepared as described in this section. In general, the final 300 millimetres of the foundation excavation should be carried out with equipment fitted with a smooth cutting edge, rather than conventional bucket 'teeth' since these will gouge the silt and allow pooling of water. The silt subgrade within the foundation footprint should not be walked upon or otherwise disturbed. The cleaned excavation base should be inspected by a QVE qualified in geotechnical engineering. It is recommended that the footing excavation be carried out such that the final 0.5 metres of excavation is completed with the geotechnical QVE on site. A 100 millimetre thick working slab consisting of concrete having a minimum 28 day compressive strength of 20 megapascals should be placed immediately after the foundation excavation has been prepared and inspected particularly if the footing will not be constructed right away. A Non-Standard Special Provision (NSSP) relating to the working slab and protection of the silt subgrade should be added to the Contract Documents. Footing construction should not be carried out during winter conditions.

Dewatering of the sand and gravel to elevation 478.2 metres or 0.5 metres below the underside of design footing elevation will be required to permit construction of the footings and avoid disturbance of the silt subgrade where present within the footprint of the west abutment footing. Due to the high permeability of the sand and gravel and the proximity of the footing to the watercourse, it is anticipated that a sheet pile enclosure may be constructed around each section of footing.

### **6.7.2 Driven H-Piles**

As previously discussed, the abutments for an alternative bridge design could be founded on driven H-piles. It should be noted that the piles will be driven through sand and gravel and sandy silt till layers which contain cobbles and boulders that may interfere with advancement of the piles. Hard driving conditions can be expected in these deposits, particularly if very dense zones with SPT N values in excess of 100 blows per 0.3 metres are encountered. Such zones were encountered between elevation 476.0 and 476.5 metres in the area of the east abutment and between elevation 474.5 and 476.0 metres in the northwest quadrant. An NSSP should be added to the Contract Documents to alert the Contractor to the need for special procedures to deal with cobbles and boulders during pile installation.



### **6.7.3 Driven Sheet Piles**

It is anticipated that sheet piles are to be driven to provide partial groundwater cut-off during construction of abutment foundations. Ideally, the sheet piles should extend to the underlying lower permeability sandy silt glacial till interface encountered between the elevations of 471 and 476 metres. However, installation of the sheet piles will be very challenging due to the generally dense to very dense nature of the saturated sand and gravel deposit which contains cobbles and boulders. It is unlikely that the piles will penetrate as deep as or into the underlying till. Therefore the sheet piles should be driven to practical refusal in the sand and gravel. Use of a vibratory hammer is preferred as it may facilitate sheet driving in the dense to very dense granular soils and should minimize the effect of large-amplitude impact-hammer vibrations on the existing structure. Special methods may be required to facilitate installation of the sheet piles such as pre-drilling and reinforcement of the toes of the sheet piles. Consideration should be given to test-driving to select the correct sheet pile section. An NSSP should be added to the Contract Documents to alert the Contractor to the hard driving conditions and presence of cobbles and boulders within the sand and gravel deposits. Extraction of the sheet piles will be difficult and must be carefully executed to avoid disturbance of the excavation and footings. In general, it is recommended that the sheet piles be left in place and cut-off at an appropriate level below the ground surface or watercourse bottom. Leaving the sheet piles in place should assist in protecting the footings from scour and erosion and avoid creation of voids during sheet extraction. The NSSP for sheet piles should specify that the sheet piles are to be cut-off and left in place at the end of construction.

## **6.8 Excavations**

Excavations for spread/strip footing construction will extend primarily through the existing pavement structure, fill materials, silt, sandy silt, silty sand, sand, and sand and gravel. Groundwater seepage into the excavations should be anticipated. It is considered that adequate groundwater control will not be achieved solely by pumping from properly constructed and filtered sumps in the base of the excavations, especially at the west abutment. Based on the encountered subsurface conditions, recommended maximum foundation elevations and the river water levels, it is anticipated that additional groundwater control methods will be necessary. Such methods may consist of eductors, deep wells or vacuum well points in the silty fine sand or sand and gravel within a sheet pile enclosure around the perimeter of the excavation.

It should be noted that installation of sheet piles for groundwater control and potentially for excavation support will be difficult. In order to achieve full groundwater cut-off within the sheet pile enclosure, the sheet piles would have to extend into the lower permeability sandy silt till; however, this will not likely be possible because of the soil density and presence of cobbles and boulders. While sheet piling will assist with control of the water inflow if sufficient penetration cannot be obtained, supplementary dewatering measures will also be required within the sheet pile enclosure. It is understood that assessments of the need for a Permit to Take Water and the associated groundwater control methods are being undertaken by others.



Temporary open cut slopes within the granular materials should be maintained no steeper than 1 horizontal to 1 vertical. 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. Appropriate Non Standard Special Provisions (NSSP) should be included in the contract documents to address groundwater control.

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, silts and granular materials below the groundwater level would be classified as Type 3 soils. Properly dewatered cohesionless materials would be classified as Type 2 soils.

### 6.9 Embankments

All surficial topsoil, organic, loose, soft and/or otherwise deleterious materials should be stripped from the areas requiring embankment widening. The exposed subgrade should be proofrolled prior to fill placement under the direction of qualified geotechnical personnel. Grading and embankment construction should be conducted in accordance with MTO Special Provision 206S03.

The embankment fills should consist of an approved granular borrow such as SSM or Granular B Type I or Type III. Embankment fill materials should be placed in maximum 300 millimetre thick loose lifts and properly benched into the existing embankments in accordance with Ontario Provincial Standard Drawing (OPSD) 208.010 and compacted. Upon completion of filling to the pavement subgrade level, the embankment side slopes should be trimmed to a final inclination of 2 horizontal to 1 vertical or flatter.

### 6.10 Staging and Temporary Roadway Protection

It is understood that a single lane is to remain open to traffic during construction therefore, replacement of the existing South Saugeen River Bridge will need to be conducted in stages using a signalized single lane. Temporary shoring may have a height of about 5 metres above the excavation base. Temporary support systems might consist of driven sheet pile walls. Cantilever sheet pile walls are not considered feasible due to the likely limited and irregular toe penetration depths that might result when attempting to install these in the dense to very dense granular soils that include cobbles and boulders. Depending on the contractor's selected means and methods, a system of braced or anchored sheet piles may be feasible at this site. As discussed above, for driven sheet piles use of a vibratory hammer is preferred, special methods may be required to facilitate adequate sheet pile penetration, and toe reinforcement should be implemented. Extraction of any sheet piles will be difficult and must be carefully executed to avoid disturbance of the excavation and footings and it is recommended that the sheet piles be left in place and cut-off at an appropriate level below the ground and pavement surface.



## FOUNDATION INVESTIGATION AND DESIGN REPORT SOUTH SAUGEN RIVER BRIDGE, SITE NO. 35-27

Excavation support systems should be designed and constructed in accordance with OPSS 539 and the design should limit the lateral movement of the temporary shoring system to meet Performance Level 2. The Contractor is responsible for the complete detailed design of the protection system.

The design of a soldier pile and lagging wall may be designed based on a triangular earth pressure distribution using the design parameters given below. The support system must be designed to accommodate the loads applied from earth, water and surcharge pressures from wide area, traffic and equipment loads as well as the effects of any sloping ground behind the system. Passive toe restraint to the soldier piles may be determined using a triangular passive pressure distribution acting over an equivalent width equal to three times the pile socket diameter provided that the pile sockets are separated by a distance of at least three times the socket diameter.

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

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

where  $H$  = the height of the excavation at any point in metres

$K_a$  = active coefficient of earth pressure

$\gamma$  = soil unit weight (total unit weight above groundwater, buoyant unit weight below groundwater)

$q$  = surcharge for traffic and other construction loading

The support systems may be designed using the following parameters:

Soil Type	Coefficient of Earth Pressure for Horizontal Ground Surfaces			Internal Angle of Friction (degrees)	Unit Weight ( $\text{kN/m}^3$ )
	Active, $K_a$	At Rest, $K_o$	Passive, $K_p$		
Fill	0.36	0.53	2.8	28	19
Silt	0.36	0.53	2.8	28	19
Sands	0.31	0.47	3.3	32	21
Sand and Gravel	0.31	0.47	3.3	32	22
Sandy Silt Till	0.31	0.47	3.3	32	21

The earth pressure coefficients identified above may be applied assuming a horizontal ground surface behind the retaining structure. Where the ground surface behind the retaining structure is sloped, the earth pressure coefficients provided in the table above must be increased. Contractors should be prepared for the presence of cobbles and boulders within the sand and gravel strata and the appropriate NSSP should be provided.

The design parameters provided above are considered to be appropriate for defining the minimum structural and geometric requirements to limit the potential for ultimate failure conditions and designs based solely on these parameters may not result in a support system that limits ground or roadway displacements to acceptable



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## FOUNDATION INVESTIGATION AND DESIGN REPORT SOUTH SAUGEEN RIVER BRIDGE, SITE NO. 35-27

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performance requirements. Depending on the displacement performance requirements, stiffer support system components may be required.



## **7.0 MISCELLANEOUS**

This report was prepared by Mr. Tyson Pitt, 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.**

**ORIGINAL SIGNED**

**ORIGINAL SIGNED**

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TABLE I

**COMPARISON OF FOUNDATION ALTERNATIVES – REPLACEMENT STRUCTURE**

South Saugeen River Bridge, Site No. 35-27  
 Highway 89  
 GWP 3049-08-00

<b>FOUNDATION OPTION</b>	<b>FEASIBILITY</b>	<b>ADVANTAGES</b>	<b>DISADVANTAGES</b>	<b>ESTIMATED COSTS</b>	<b>RISKS/ CONSEQUENCES</b>
Spread footings supported on compact to very dense sand/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>• Least expensive option</li> <li>• Ease of construction</li> <li>• Less expensive than deep foundation options</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>• Estimated cost \$10,000 per abutment exclusive of groundwater control</li> </ul>	<ul style="list-style-type: none"> <li>• Relatively low risk provided groundwater is adequately controlled</li> </ul>
End bearing steel H-pile foundations driven to refusal into very dense sandy silt till	<ul style="list-style-type: none"> <li>• Feasible</li> <li>• Not preferred technical alternative</li> </ul>	<ul style="list-style-type: none"> <li>• High bearing resistance</li> <li>• Negligible settlement</li> <li>• Less vibration related damage to adjacent works compared to steel tube piles</li> <li>• Only solution compatible with integral abutments</li> </ul>	<ul style="list-style-type: none"> <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 tube piles</li> <li>• Cannot be visually inspected at depth</li> <li>• Integrity inspection requires specialty dynamic testing</li> </ul>	<ul style="list-style-type: none"> <li>• Each pile will cost approximately \$270 per metres plus \$275 per driving shoe.</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**

<b>FOUNDATION OPTION</b>	<b>FEASIBILITY</b>	<b>ADVANTAGES</b>	<b>DISADVANTAGES</b>	<b>ESTIMATED COSTS</b>	<b>RISKS/ CONSEQUENCES</b>
End bearing concrete filled steel tube piles driven into very dense sandy silt till	<ul style="list-style-type: none"> <li>• Feasible.</li> <li>• Not preferred technical alternative.</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>• Difficulty driving through very dense sand and gravel containing cobbles to appropriate embedment depths</li> <li>• More costly than shallow footings</li> <li>• Not compatible with integral abutments</li> <li>• Cost competitive with steel H-piles</li> <li>• Greater vibrations and risks of damage to adjacent works compared to H-piles</li> </ul>	<ul style="list-style-type: none"> <li>• Each pile will cost approximately \$275 per metre plus \$600 per pile shoe.</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> </ul>

- NOTES:
1. Costs are order of magnitude estimates in 2013 dollars and 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: TP  
 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  $= (\text{compressive strength})/2$
  - \* 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  
W.P. 3049-08-00 LOCATION N 4875164.0 :E 224292.5 ORIGINATED BY MA  
DIST HWY 89 BOREHOLE TYPE POWER AUGER, HOLLOW STEM COMPILED BY LMK/AMG  
DATUM GEODETIC DATE September 6, 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									
								○ UNCONFINED      + FIELD VANE ● QUICK TRIAXIAL    x LAB VANE									
481.36	GROUND SURFACE							20	40	60	80	100					
0.00	TOPSOIL, sandy, trace gravel Brown						Concrete										
480.96							481										
0.40	SILTY SAND, trace clay, trace gravel, with topsoil Loose Brown		1	SS	17		Granular Bentonite						○				
480.32							480						○				
1.04	SAND AND GRAVEL, trace silt Compact Brown		2	SS	51								○				
479.99																	
1.37	SAND, some silt, some gravel Compact to very dense Brown		3	SS	20								○				
478.46							479										
2.90	SAND AND GRAVEL, some silt, trace clay, with cobbles Compact to very dense Brown		4	SS	17								○				
			5	SS	71								○				
			6	SS	108		Backfill						○				37 48 11 4
			7	SS	81		476						○				
475.42																	
5.94	SAND, trace gravel, with sand and gravel layers Very dense Brown		8	SS	64		475						○				
474.65																	
6.71	SAND AND GRAVEL, trace to some silt Very dense Brown		9	SS	76		474						○				40 44 13 3
			10	SS	57								○				
							Granular Bentonite										
							Filter Sand										
			11	SS	101/ 250mm		472						○				
471.30																	
10.06	SANDY SILT TILL, some gravel, some clay Very dense Grey		12	SS	106/ 250mm		Well 71						○				
							470										
							Filter Sand										
468.80			13	SS	100/ 225mm		469						○				10 28 47 15
12.56	END OF BOREHOLE																
	Groundwater encountered at about elev. 479.5m during drilling on September 6, 2012.  Water level measured at elev. 479.58m, October 1, 2012.  Water level measured at elev. 479.86m, October 24, 2012.  Water level measured at elev. 480.29m, December 28, 2012.																

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

**RECORD OF BOREHOLE No 2**

1 OF 2

**METRIC**

PROJECT 11-1132-0109  
W.P. 3049-08-00 LOCATION N 4875134.0 ; E 224303.8 ORIGINATED BY MA  
DIST HWY 89 BOREHOLE TYPE POWER AUGER, HOLLOW STEM COMPILED BY LMK/AMG  
DATUM GEODETIC DATE September 10, 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  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa ○ UNCONFINED      + FIELD VANE ● QUICK TRIAXIAL    × LAB VANE					WATER CONTENT (%) W <sub>P</sub> W                      W <sub>L</sub>					GR	SA	SI
482.37	PAVEMENT SURFACE					▽														
0.09	ASPHALT						482													
0.27	FILL, crushed granular base, Brown																			
481.64	FILL, sand and gravel, trace silt Brown																			
0.73	FILL, topsoil, silty, trace sand, with sand and gravel layers Loose to compact Black		1	SS	17															
480.48			2	SS	5															0 65 31 4
1.89	SILT, some sand Loose Grey																			
480.08																				
2.29	SAND AND GRAVEL, trace to some silt, trace clay, with silty sand layers Compact to very dense Brown		3	SS	16															
			4	SS	36															
			5	SS	46															
			6	SS	89															
			7	SS	32															
			8	SS	100/ 275mm															
475.66																				
6.71	SANDY SILT TILL, some gravel, trace to some clay Dense to very dense Grey		9	SS	43														20 35 35 10	
			10	SS	54															
			11	SS	50															
			12	SS	43															
			13	SS	59															
			14	SS	33														15 37 36 12	

Continued Next Page

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

PROJECT <u>11-1132-0109</u>		<b>RECORD OF BOREHOLE No 2</b>		2 OF 2	<b>METRIC</b>
W.P. <u>3049-08-00</u>		LOCATION <u>N 4875134.0 ; E 224303.8</u>		ORIGINATED BY <u>MA</u>	
DIST <u>          </u> HWY <u>89</u>		BOREHOLE TYPE <u>POWER AUGER, HOLLOW STEM</u>		COMPILED BY <u>LMK/AMG</u>	
DATUM <u>GEODETIC</u>		DATE <u>September 10, 2012</u>		CHECKED BY <u>          </u>	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT   NATURAL MOISTURE CONTENT   LIQUID LIMIT			UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																										
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)				GR	SA	SI	CL																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																							
													20	40	60						80	100	W <sub>p</sub>	W	W <sub>L</sub>																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																		

**RECORD OF BOREHOLE No 3**

1 OF 2

**METRIC**

PROJECT 11-1132-0109  
W.P. 3049-08-00 LOCATION N 4875132.7 ; E 224274.1 ORIGINATED BY DB  
DIST HWY 89 BOREHOLE TYPE POWER AUGER, HOLLOW STEM COMPILED BY LMK/AMG  
DATUM GEODETIC DATE September 6, 2012 - September 7, 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  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)				
								20	40	60	80	100	W <sub>P</sub>	W	W <sub>L</sub>		
481.15	GROUND SURFACE						481										
0.00	TOPSOIL, silty, Brown																
480.30							480										
0.85	SILTY SAND AND GRAVEL, trace clay Compact Brown		1	SS	10												
			2	SS	23												
479.02							479										
2.13	SILT, some sand, trace clay, with sand and gravel layers Compact Brown		3	SS	13											0 10 82 8	
			4	SS	14		478										
477.49																	
3.66	SILTY SAND AND GRAVEL Compact Brown		5	SS	22		477										
476.73																	
4.42	SAND, trace gravel Dense Brown		6	SS	44												
476.27							476										
4.88	SAND AND GRAVEL, some silt Dense to very dense Brown		7	SS	54											37 37 12 4	
			8	SS	67		475										
			9	SS	46		474										
473.35																	
7.80	SANDY SILT TILL, some clay, trace to some gravel Dense to very dense Grey		10	SS	60		473										
			11	SS	32		472									12 19 54 15	
							471										
			12	SS	43		470										
			13	SS	68		469										
							468										
			14	SS	100/ 200mm		467									6 32 45 17	

Continued Next Page

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

**RECORD OF BOREHOLE No 3**

2 OF 2

**METRIC**

PROJECT 11-1132-0109  
W.P. 3049-08-00 LOCATION N 4875132.7 ; E 224274.1 ORIGINATED BY DB  
DIST HWY 89 BOREHOLE TYPE POWER AUGER, HOLLOW STEM COMPILED BY LMK/AMG  
DATUM GEODETIC DATE September 6, 2012 - September 7, 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  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)  GR SA SI CL				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									WATER CONTENT (%)			
								○ UNCONFINED      + FIELD VANE ● QUICK TRIAXIAL    × LAB VANE												
							20	40	60	80	100									
465.82			15	SS	100/ 75mm															
15.33	BOULDER																			
15.61	SANDY SILT TILL, some gravel, trace clay Very dense Grey																			
464.11			16	SS	100/ 125mm															
17.04	END OF BOREHOLE																			
	Groundwater encountered at about elev. 479.6m during drilling on September 6 and 7, 2012.																			

**RECORD OF BOREHOLE No 4**

1 OF 2

**METRIC**

PROJECT 11-1132-0109

W.P. 3049-08-00

LOCATION N 4875158.0 ; E 224275.5

ORIGINATED BY DB

DIST HWY 89

BOREHOLE TYPE POWER AUGER, HOLLOW STEM

COMPILED BY LMK/AMG

DATUM GEODETIC

DATE September 10, 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>				
								○ UNCONFINED                      + FIELD VANE ● QUICK TRIAXIAL                      × LAB VANE	20    40    60    80    100			10    20    30					
481.24	GROUND SURFACE																
0.00	TOPSOIL, silty Brown						481										
480.48																	
0.76	SANDY SILT, some gravel, trace clay Loose Brown		1	SS	6		480										
479.87																	
1.37	SILTY SAND Loose Brown		2	SS	4								○				
479.11																	
2.13	SAND AND GRAVEL, some silt, trace clay, with cobbles Dense to very dense Brown		3	SS	40		479										
			4	SS	35		478										
			5	SS	41		477										
			6	SS	40								○				
			7	SS	105		476										
			8	SS	72		475										
474.53																	
6.71	SANDY SILT TILL, some gravel, trace to some clay Dense to very dense Grey		9	SS	100/ 225mm		474										
			10	SS	62		473						○				
			11	SS	31		472										
							471										
			12	SS	81		470										
			13	SS	100/ 250mm		469										
							468										
			14	SS	100/ 275mm		467										

Continued Next Page

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

**RECORD OF BOREHOLE No 4**

2 OF 2

**METRIC**

PROJECT 11-1132-0109 W.P. 3049-08-00 LOCATION N 4875158.0 ; E 224275.5 ORIGINATED BY DB  
DIST            HWY 89 BOREHOLE TYPE POWER AUGER, HOLLOW STEM COMPILED BY LMK/AMG  
DATUM GEODETIC DATE September 10, 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	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100					
465.94 15.30	END OF BOREHOLE  Groundwater encountered at about elev. 479.7m during drilling on September 10, 2012.		15	SS	100/ 50mm		466										

**RECORD OF BOREHOLE No 5**

1 OF 1

**METRIC**

PROJECT 11-1132-0109  
W.P. 3049-08-00 LOCATION N 4875143.2 ; E 224256.1 ORIGINATED BY MA  
DIST HWY 89 BOREHOLE TYPE POWER AUGER, HOLLOW STEM COMPILED BY LMK/AMG  
DATUM GEODETIC DATE August 28, 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>		
482.43	PAVEMENT SURFACE																
0.00	ASPHALT																
0.30	FILL, crushed granular base, Brown						482										
481.73	FILL, sand, fine to medium, some gravel, trace silt, with cobbles, Brown																
0.70	TOPSOIL, silty, trace sand		1	SS	6												
481.06	Loose Black						481										
1.37	SILTY CLAY, some sand, trace gravel		2	SS	5												
480.45	Firm Brown																
1.98	SILT, trace to some clay, trace sand, with clayey silt layers Loose to compact Brown		3	SS	9		480										
			4	SS	23												
478.77							479									0 1 86 13	
3.66	SILTY FINE SAND, trace clay Compact Brown		5	SS	15											0 50 44 6	
478.01																	
4.42	SAND, trace gravel, trace silt, with clayey silt layers Compact Brown		6	SS	24		478										
477.25																	
5.18	SAND AND GRAVEL, trace silt Dense to very dense Brown		7	SS	61		477										
			8	SS	42		476										
475.88																	
6.55	END OF BOREHOLE																
	Groundwater encountered at about elev. 480.3m during drilling on August 28, 2012.																

**RECORD OF BOREHOLE No 6**

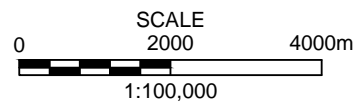
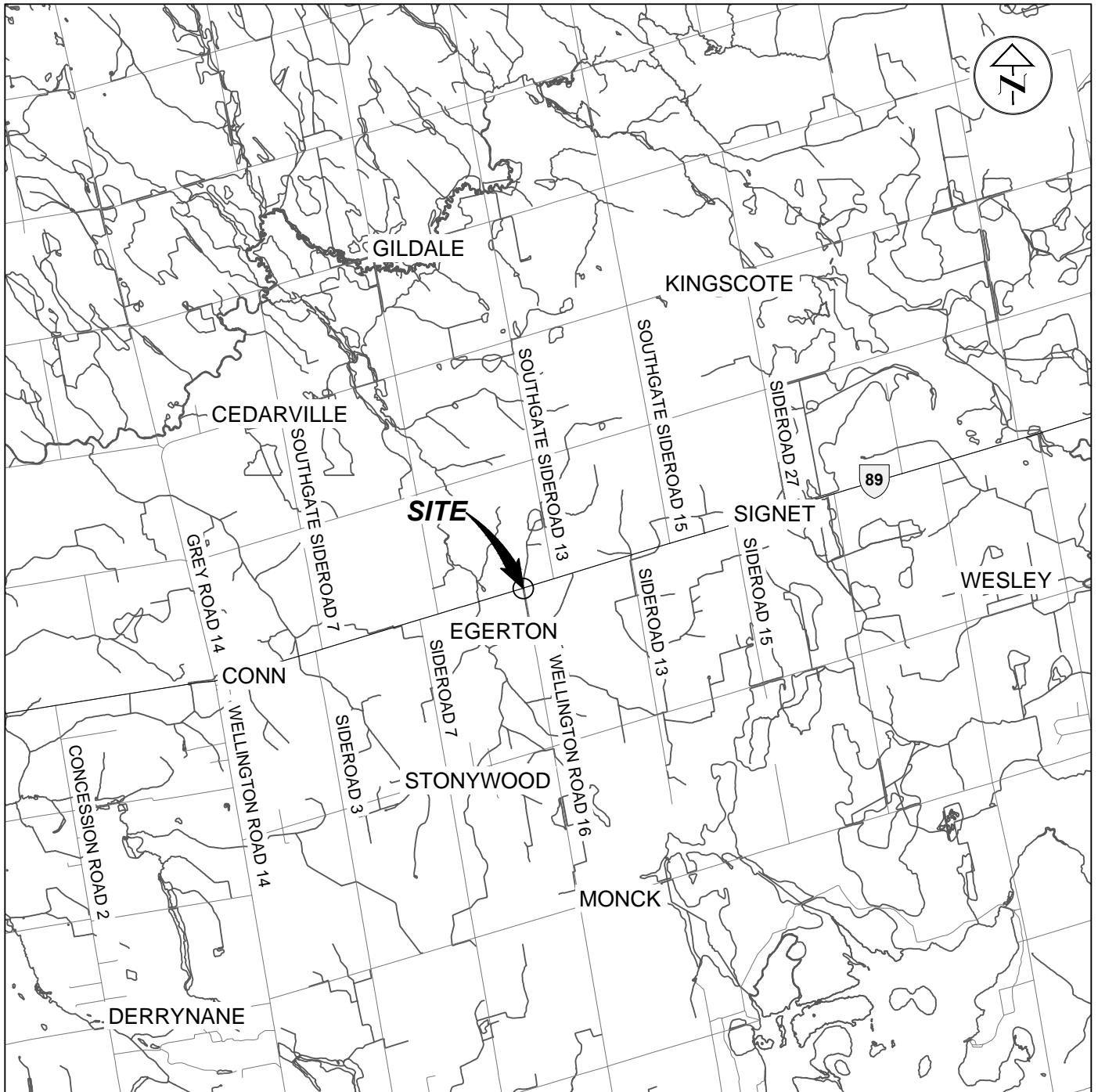
1 OF 1

**METRIC**

PROJECT 11-1132-0109  
W.P. 3049-08-00 LOCATION N 4875151.1 ; E 224314.7 ORIGINATED BY MA  
DIST HWY 89 BOREHOLE TYPE POWER AUGER, HOLLOW STEM COMPILED BY LMK/AMG  
DATUM GEODETIC DATE September 10, 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	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa										WATER CONTENT (%)		
								○ UNCONFINED      + FIELD VANE ● QUICK TRIAXIAL    × LAB VANE												
482.82	PAVEMENT SURFACE							20	40	60	80	100								
0.00	ASPHALT																			
0.18	FILL, crushed granular base, Brown																			
0.30	FILL, sand and gravel, trace silt, with cobbles, Brown																			
0.43	FILL, silt with topsoil, some clay, some sand, trace gravel with cobbles and topsoil layers and wood fragments		1	SS	7		482													
	Loose Brown		2	SS	9		481								○		0 13 65 22			
480.69	SAND, some silt, some gravel																			
2.13	Compact Brown		3	SS	24		480													
479.92	SAND AND GRAVEL, trace to some silt, trace clay, with cobbles																			
2.90	Compact to dense Brown		4	SS	28		479							○			55 34 8 3			
			5	SS	31															
478.40	SILTY SAND AND GRAVEL, with cobbles																			
4.42	Dense Brown		6	SS	37		478													
			7	SS	46															
476.88	SAND, medium to coarse, trace gravel						477													
5.94	Very dense Brown		8	SS	100/100mm															
6.19	END OF BOREHOLE																			
	Groundwater encountered at about elev. 479.3m during drilling on September 10, 2012.																			

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



## REFERENCE

PLAN BASED ON CANMAP STREETFILES V.2008.5.

## NOTE

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

PROJECT

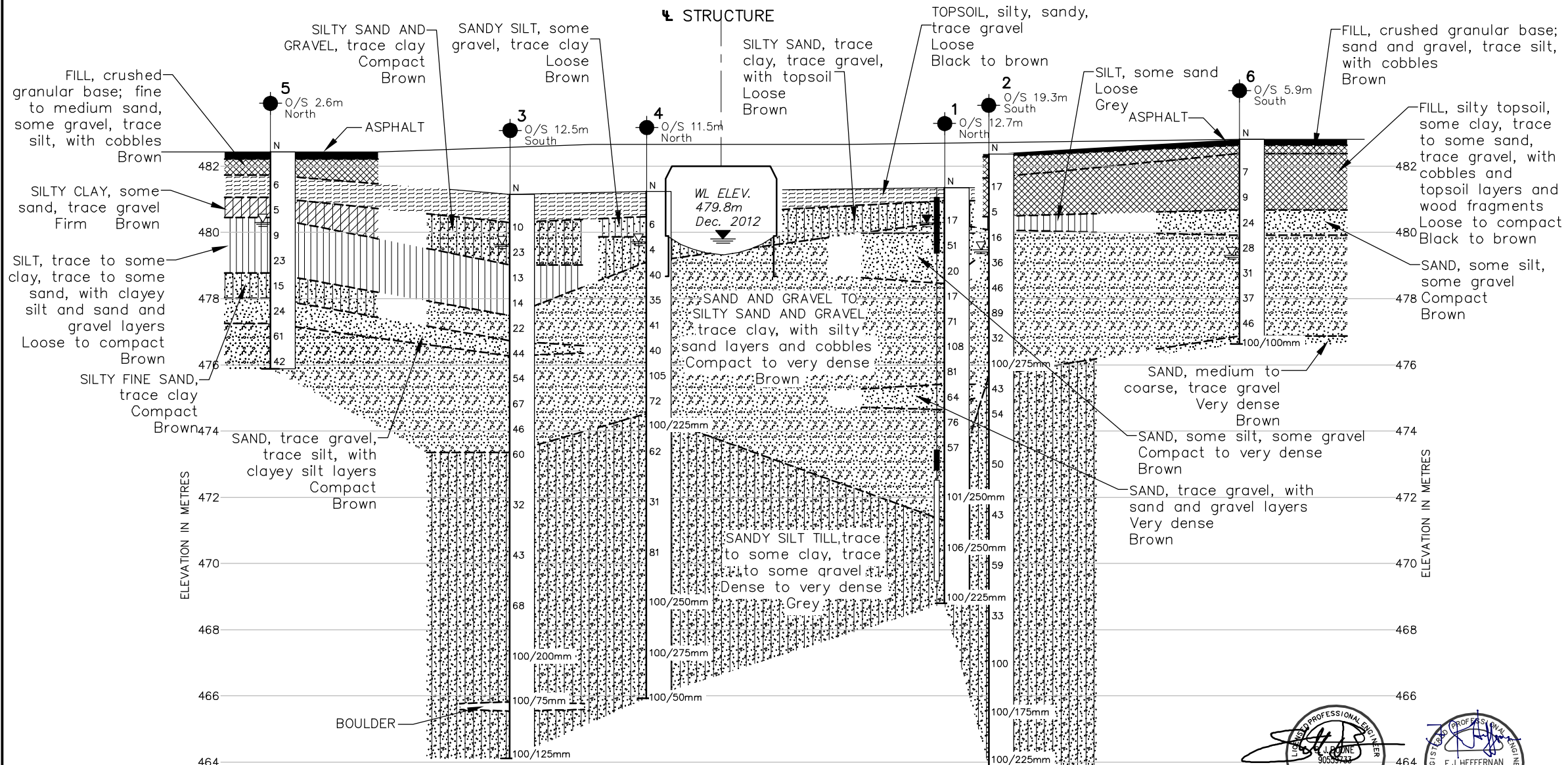
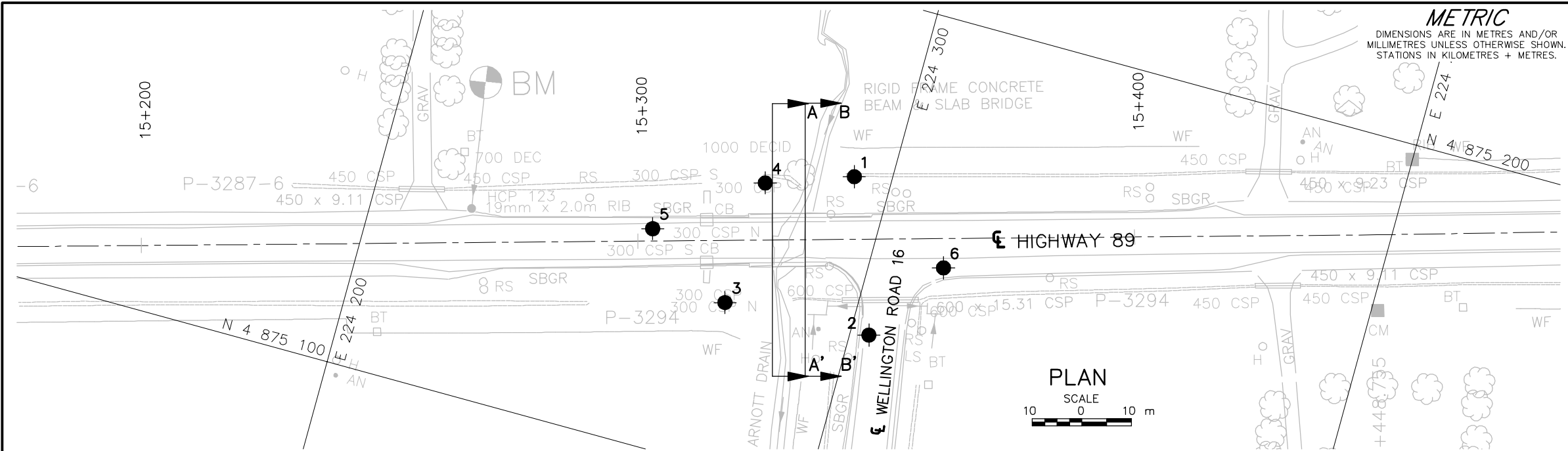
SOUTH SAUGEEN RIVER BRIDGE, SITE 35-27  
HIGHWAY 89 STRUCTURE REPLACEMENTS  
GWP 3049-08-00

TITLE

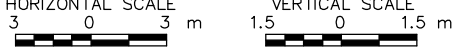
## KEY PLAN



PROJECT No. 11-1132-0109			FILE No. 1111320109-1000-F01001	
CADD	LMK	Dec. 29/12	SCALE AS SHOWN	REV. 0
CHECK			<b>FIGURE 1</b>	



PROFILE ALONG CL OF HIGHWAY 89



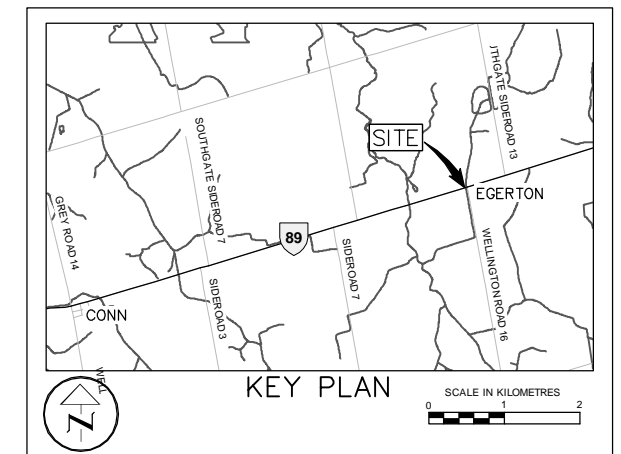
CONT No.  
WP No. 3049-08-00

SOUTH SAUGEEN RIVER BRIDGE  
HIGHWAY 89 STRUCTURE REPLACEMENTS

BOREHOLE LOCATIONS AND SOIL STRATA

SHEET

Golder Associates Ltd.  
LONDON, ONTARIO, CANADA



LEGEND

- Borehole - Current Investigation
- Seal
- Observation well
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- WL in observation well measured on Dec. 28, 2012
- WL encountered during drilling

No.	ELEVATION	CO-ORDINATES (MTM ZONE 10)	
		NORTHING	EASTING
1	481.36	4 875 164.0	224 292.5
2	482.37	4 875 134.0	224 303.8
3	481.15	4 875 132.7	224 274.1
4	481.24	4 875 158.0	224 275.5
5	482.43	4 875 143.2	224 256.1
6	482.82	4 875 151.1	224 314.7

NOTES

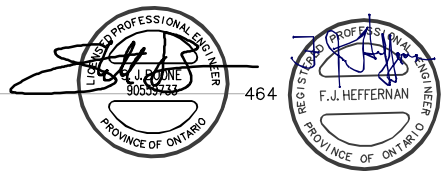
This drawing is for subsurface information only. Surface details and features are for conceptual illustration.

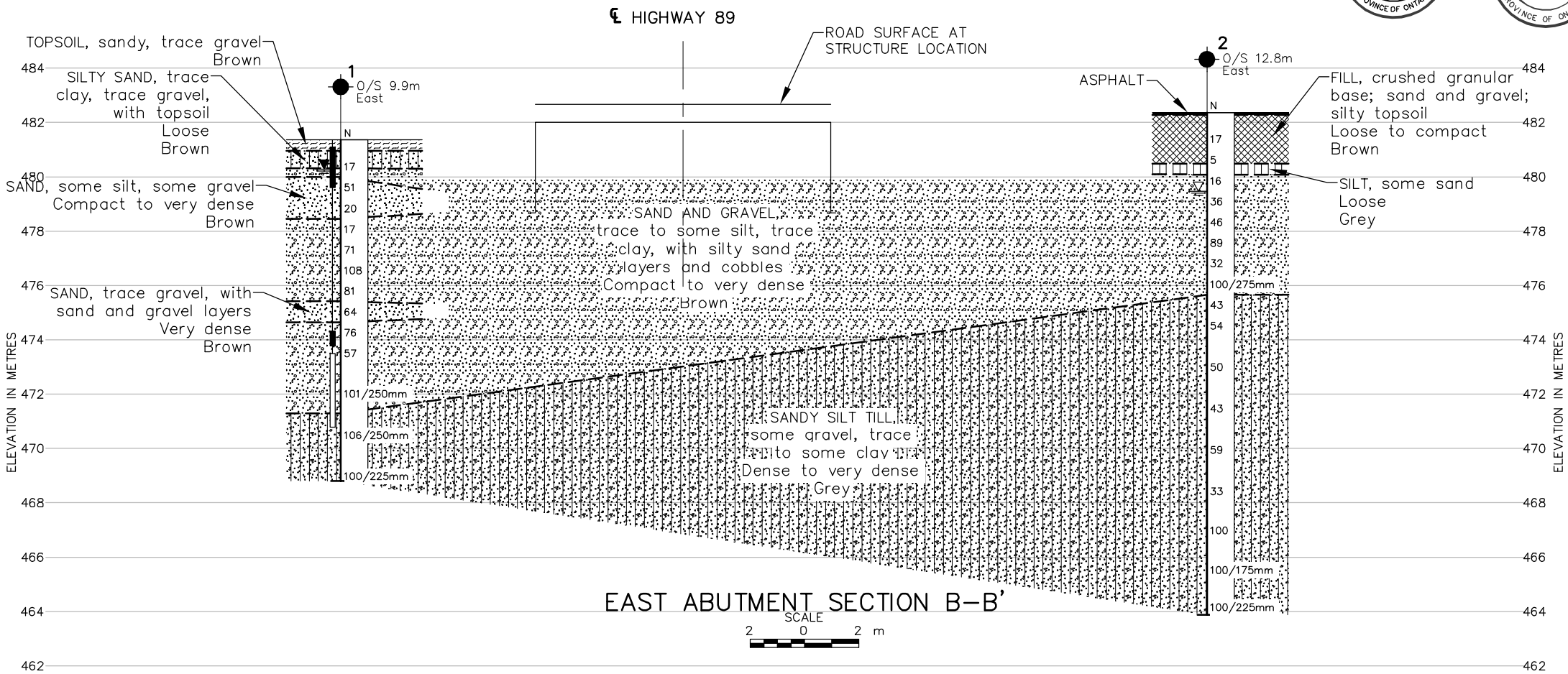
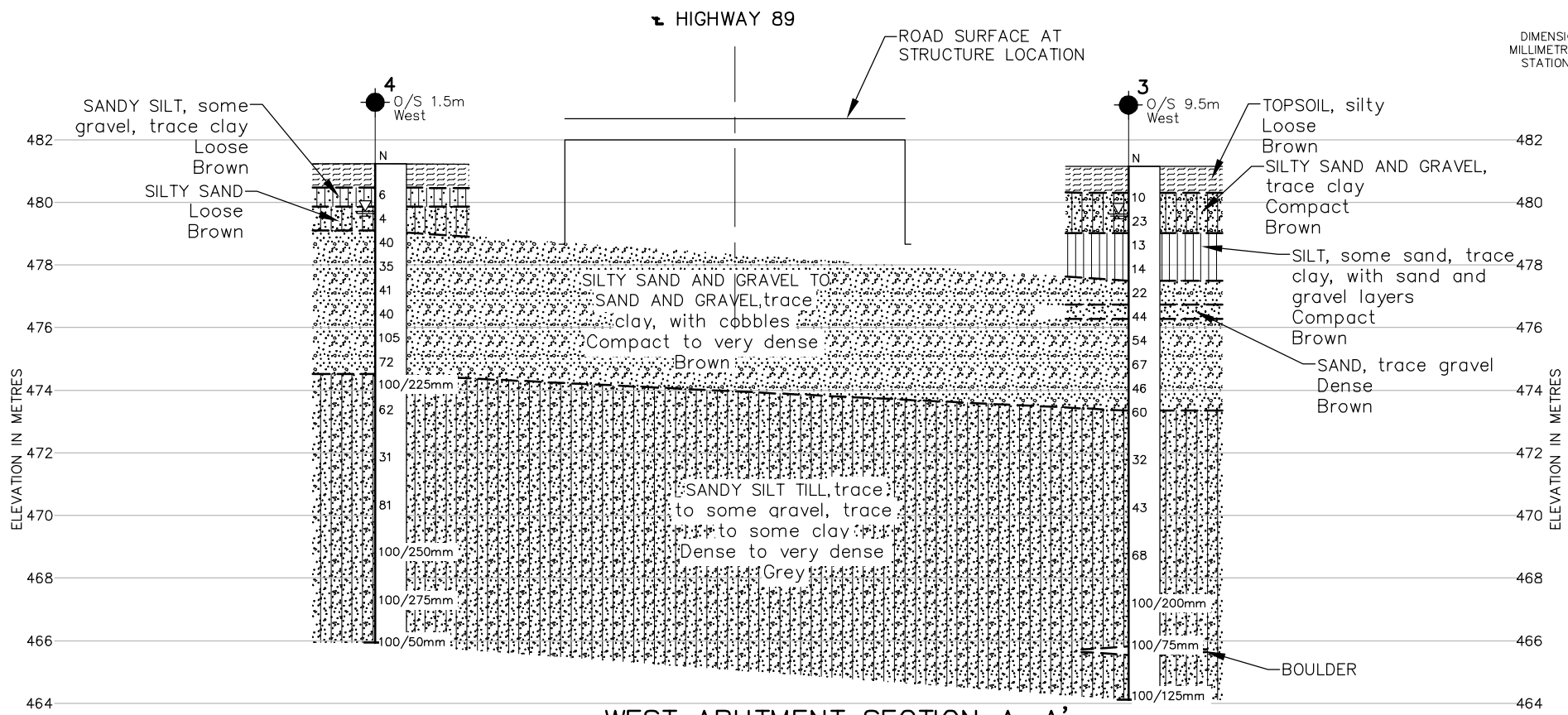
The boundaries between soil strata have been established only at borehole locations. Between boreholes the boundaries are assumed from geological evidence.

REFERENCE

Base plans provided in digital format by Morrison Hershfield Limited

NO.	DATE	BY	REVISION
Geocres No.	41A-227		
HWY.	89	PROJECT NO.	11-1132-0109
SUBM'D.	TP	CHKD.	DUP
DRAWN:	LMK\WDF	CHKD.	SJB
DATE:	Dec.29/12	APPD.	FJH
SITE:	35-27	DWG.	1





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

CONT No.  
WP No. 3049-08-00

SOUTH SAUGEEN RIVER BRIDGE  
HIGHWAY 89 STRUCTURE REPLACEMENTS

SOIL STRATA



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



LEGEND

- Borehole - Current Investigation
- Seal
- Observation well
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- WL in observation well measured on Dec. 28, 2012
- WL encountered during drilling

No.	ELEVATION	CO-ORDINATES (MTM ZONE 10)	
		NORTHING	EASTING
1	481.36	4 875 164.0	224 292.5
2	482.37	4 875 134.0	224 303.8
3	481.15	4 875 132.7	224 274.1
4	481.24	4 875 158.0	224 275.5

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.

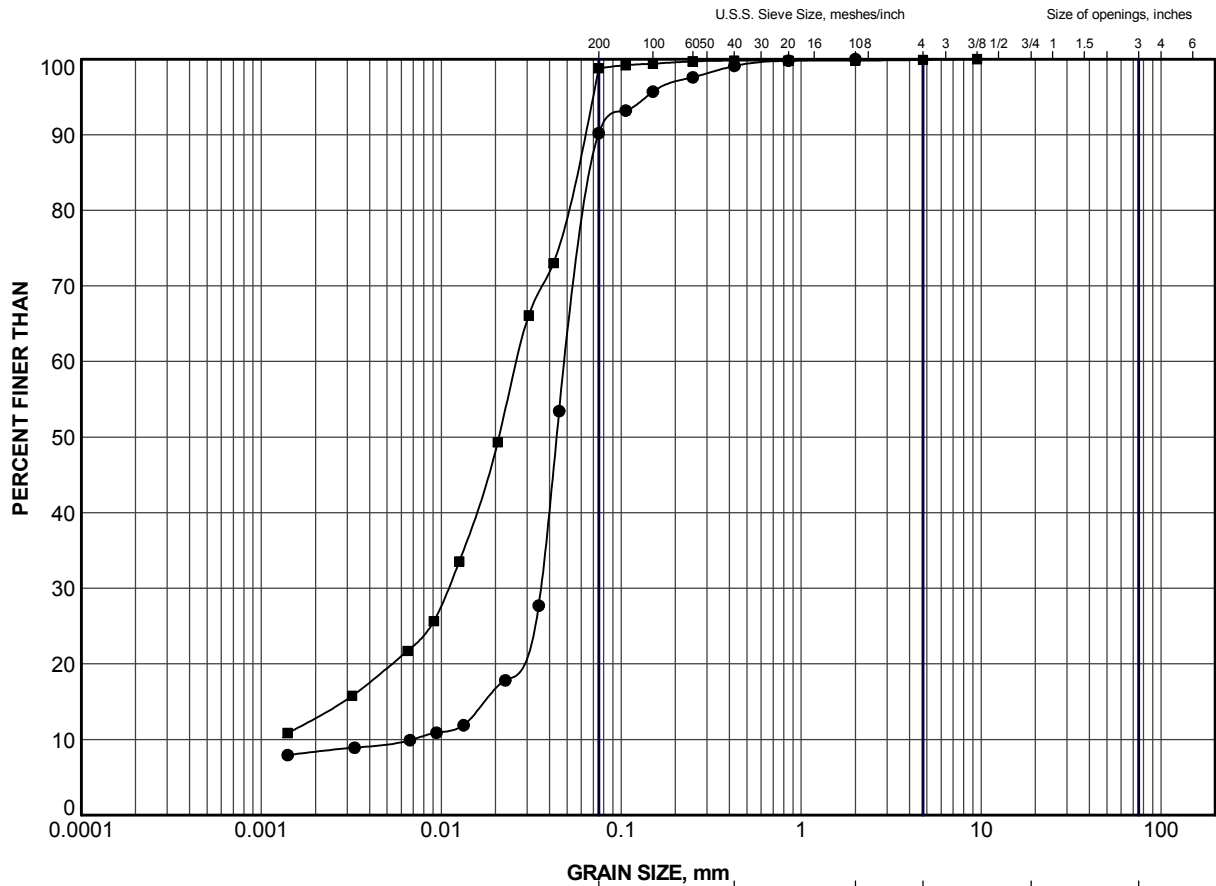
NO.	DATE	BY	REVISION
Geocres No.	41A-227		
HWY.	89	PROJECT NO.	11-1132-0109
SUBM'D.	TP	CHKD.	DUP
DRAWN:	LMK\WDF	CHKD.	SJB
DATE:	Dec. 29/12	APPD.	FJH
SITE:	35-27	DWG.	2



# **APPENDIX A**

## **Laboratory Test Data**




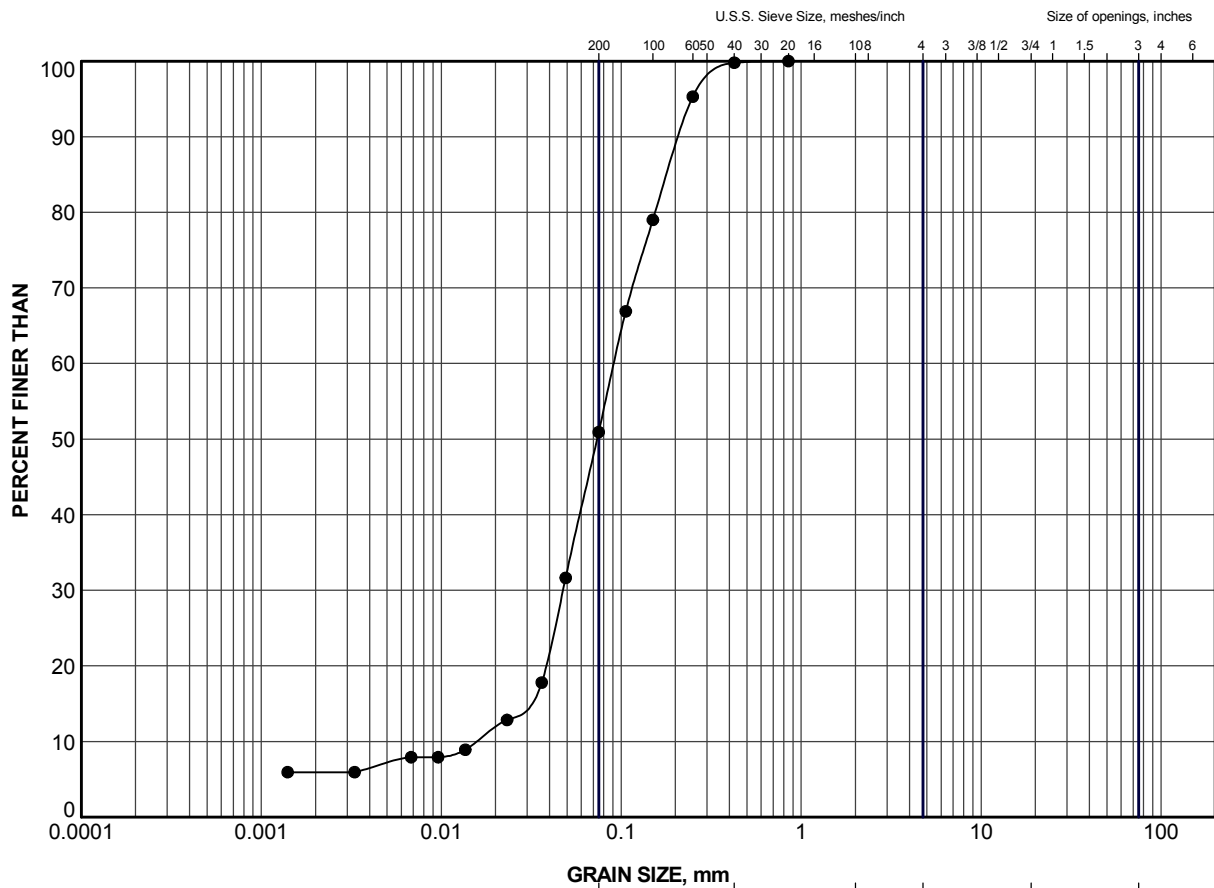


CLAY AND SILT	SAND SIZE, mm					Cobble Size
	fine	medium	coarse	fine	coarse	
	SAND SIZE			GRAVEL SIZE		

#### LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	3	3	478.6
■	5	4	479.2

PROJECT				SOUTH SAUGEE RIVER BRIDGE, SITE 35-27 HIGHWAY 89 STRUCTURE REPLACEMENTS GWP 3049-08-00			
TITLE				GRAIN SIZE DISTRIBUTION SILT			
PROJECT No: 11-1132-0109-1000				FILE No. 1111320109-1000-F010A2			
DRAWN LMK				Dec. 29/12			
CHECK				SCALE N/A REV.			
 <b>Golder Associates</b> LONDON, ONTARIO				<b>FIGURE A-2</b>			



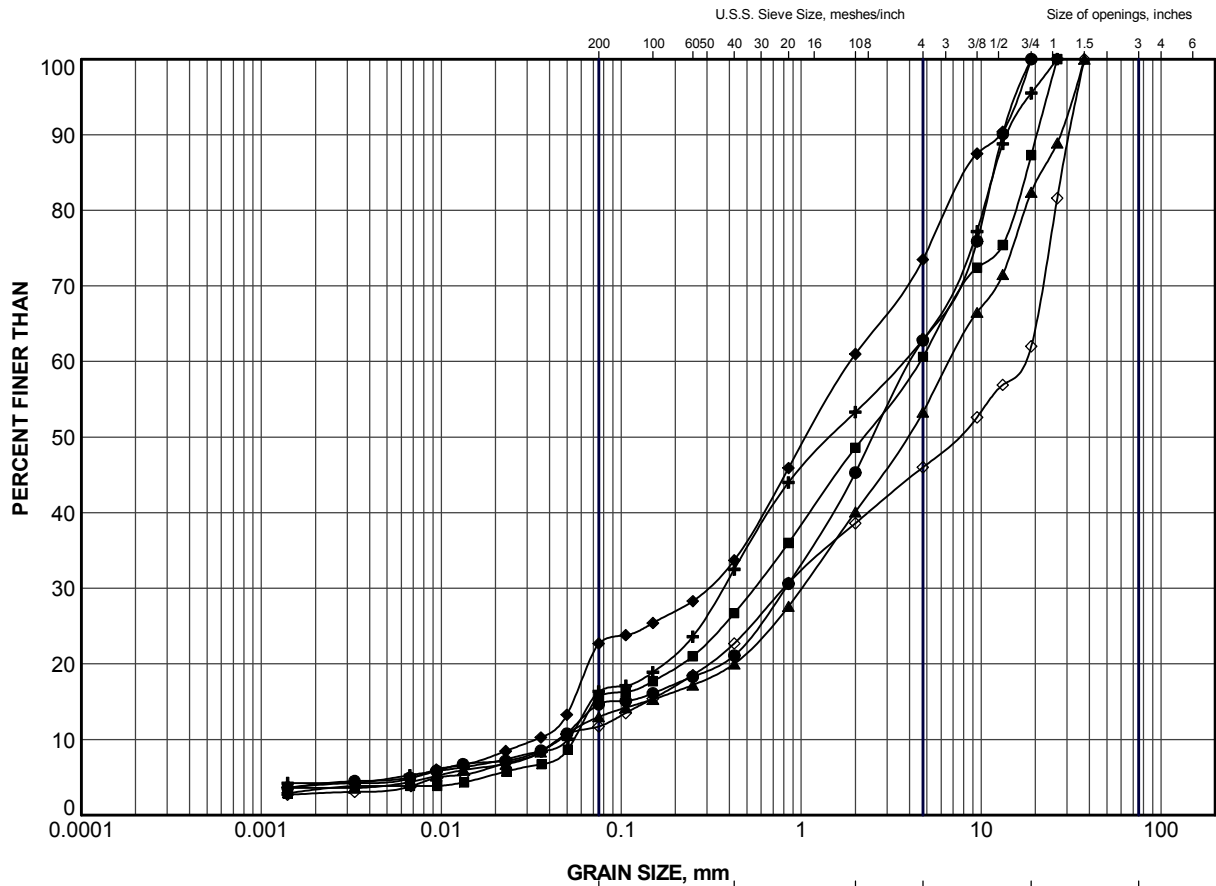
GRAIN SIZE, mm						
CLAY AND SILT	fine	medium	coarse	fine	coarse	Cobble Size
	SAND SIZE			GRAVEL SIZE		

#### LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	5	5	478.4

PROJECT				SOUTH SAUGEE RIVER BRIDGE, SITE 35-27 HIGHWAY 89 STRUCTURE REPLACEMENTS GWP 3049-08-00			
TITLE				GRAIN SIZE DISTRIBUTION SILTY FINE SAND			
PROJECT No:11-1132-0109-1000				FILE No. 1111320109-1000-F010A3			
DRAWN		LMK		Dec. 29/12		SCALE N/A REV.	
CHECK						FIGURE A-3	



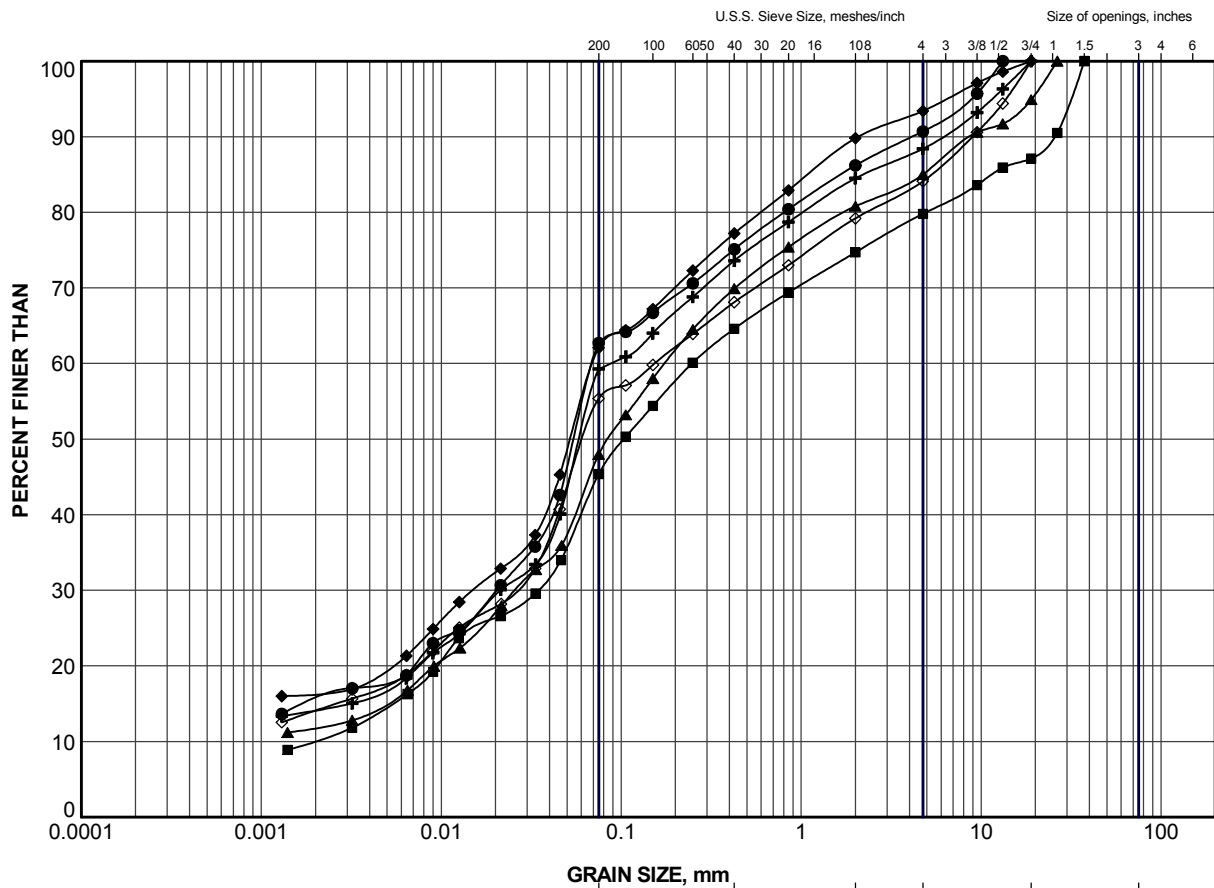


CLAY AND SILT	GRAVEL SIZE, mm					Cobble Size
	fine	medium	coarse	fine	coarse	
	SAND SIZE			GRAVEL SIZE		

#### LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	1	6	476.6
■	1	9	474.3
▲	2	4	479.1
+	3	7	475.6
◆	4	6	476.4
◇	6	4	479.5

PROJECT				SOUTH SAUGEE RIVER BRIDGE, SITE 35-27 HIGHWAY 89 STRUCTURE REPLACEMENTS GWP 3049-08-00			
TITLE				GRAIN SIZE DISTRIBUTION SAND AND GRAVEL			
PROJECT No:11-1132-0109-1000				FILE No. 1111320109-1000-F010A4			
DRAWN		LMKWDF		Dec. 29/12		SCALE N/A REV.	
CHECK						FIGURE A-4	
 <b>Golder Associates</b> LONDON, ONTARIO							



### LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	1	13	469.0
■	2	9	475.3
▲	2	14	468.4
+	3	11	471.8
◆	3	14	467.2
◇	4	10	473.4

PROJECT				SOUTH SAUGEE RIVER BRIDGE, SITE 35-27 HIGHWAY 89 STRUCTURE REPLACEMENTS GWP 3049-08-00			
TITLE				GRAIN SIZE DISTRIBUTION SANDY SILT TILL			
PROJECT No: 11-1132-0109-1000				FILE No. 1111320109-1000-F010A5			
DRAWN LMK/WDF Dec. 29/12				SCALE N/A REV.			
CHECK				FIGURE A-5			





# **APPENDIX B**

## **Site Photographs**



## APPENDIX B PHOTOGRAPHS



Photograph 1: South Saugeen River Bridge, north elevation.



Photograph 2: South Saugeen River Bridge, south elevation.



# **APPENDIX B**

## **Site Photographs**



## APPENDIX B PHOTOGRAPHS



Photograph 1: South Saugeen River Bridge, north elevation.



Photograph 2: South Saugeen River Bridge, south elevation.



## APPENDIX B PHOTOGRAPHS

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Photograph 3: South Saugeen River Bridge, looking west.

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

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