



November 2012

## FOUNDATION INVESTIGATION AND 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**

**Submitted to:**

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DRAFT REPORT

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**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 Plans, 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 monitoring 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 471.7 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 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 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 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 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 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.

#### **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.

### **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 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:



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Borehole	Ground Surface Elevation (m)	Encountered Groundwater Elevation (m)
1	481.36	479.5
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 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 October 1 and 24, 2012, the water level in the groundwater observation well installed in borehole 1 was at about elevation 479.6 metres and 479.9 metres, respectively.

Based on the measured and encountered groundwater levels, the inferred groundwater level is at elevation 479.5 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 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, Ph.D., 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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**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 preliminary design information provided by MH, the replacement structure will be a 10.4 metre long, single span, precast concrete structure. The abutments are to be founded on 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 approaches were provided.

### **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 be approximately at elevation 479.5 metres.

Pile foundations are not considered economical for this bridge or suitable due to the dense to very dense nature of the granular soils and glacial till as well as the presence of cobbles and boulders. It is understood that the foundation option being considered for the bridge replacement are relatively shallow CIP concrete spread footings.



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

The abutments for the replacement bridge can be founded on spread/strip footings bearing on the compact to very dense sand and/or sand and gravel at or below elevation 480.0 metres at the east abutment, and at an elevation of 477.5 metres at the west abutment. Otherwise, supporting the west foundations at a higher elevation on the approximately 1.5 metres thick silt deposit (encountered in borehole 3) would likely lead to unacceptable differential settlement. A factored geotechnical resistance at Ultimate Limit States (ULS) of 450 kilopascals and a geotechnical resistance at Serviceability Limit States (SLS) of 300 kilopascals should be used for designing the spread foundations supported on the native sand and/or sand and gravel. The SLS value corresponds to an estimated total settlement of 25 millimetres.

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.

It is understood that the bridge abutments are planned to be constructed with precast concrete units. The recommended maximum foundation elevations provided above result in an approximately 2.5 metre difference in bearing elevation from east to west. Furthermore, 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 volume of 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 and the founding soils and corresponding unfactored coefficient of friction,  $\tan \delta$ , may be used:

Footings on sand/sand and gravel	angle of friction	32°
	$\tan \delta$	0.62



## ***Frost Protection***

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

## **6.4 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.

If the replacement bridge is a lifeline bridge, then the Seismic Performance Zone (SPZ) is 2; otherwise, the zone is SPZ 1. 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.

## **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





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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 III</u>
Soil unit weight:	22 kN/m <sup>3</sup>	21 kN/m <sup>3</sup>
Coefficients of lateral earth pressure:		
Active, $K_a$	0.27	0.31
At rest, $K_o$	0.43	0.47

- 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 III 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)	0.31
Coefficient of friction between granular backfill and founding soils	0.62

## 6.7 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 or vacuum well points in the silty fine sand or wells in the sand and gravel installed within a sheet pile enclosure around the perimeter of the excavation. It should be noted; however, that installation of



sheet piles for groundwater control and potentially for excavation support will be difficult because of the soil density and presence of cobbles and boulders. However, the sheet piling would control the water inflow if sufficient penetration could be obtained with all the sheets. It is understood that evaluation to determine the need for a Permit to Take Water and the associated groundwater control methods is 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.

## **6.8 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.

## **6.9 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 support systems could consist of soldier piles and lagging installed using temporary casings. Cantilever sheet pile walls are not considered feasible due to the likely limited and irregular toe penetration depths that might results when attempting to install these in the dense to very dense nature of the granular soils and glacial till that includes cobbles and boulders. Temporary shoring may have a height of about 5 metres above the excavation base.



## FOUNDATION INVESTIGATION AND DESIGN REPORT SOUTH SAUGREEN 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 area, line or point 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 pressure distribution acting over an equivalent width equal to three times the pile 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 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, Ph.D., 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**

Storer J. Boone, Ph.D., P.Eng.  
Associate

TP/SJB/FJH/cr

**ORIGINAL SIGNED**

Fintan J. Heffernan, P.Eng.  
MTO Designated Contact

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n:\active\2011\1132-geo\1132-0100\11-1132-0109 mh-po 3011-e-0001-hwy 89\ph 1000-prelim& detail fdns gwp 3049-08-00\rvts\rv01\1111320109-1000-r01 nov 2 12 (draft) fdns part a&b south saugeen river bridge.docx

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> </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 ground 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>• N/A</li> </ul>	<ul style="list-style-type: none"> <li>• Possible pile tip damage if piles are not adequately protected while driving through very dense/hard 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>• Not suitable</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 &amp; risks of damage to adjacent works compared to H-piles</li> </ul>	<ul style="list-style-type: none"> <li>• N/A</li> </ul>	<ul style="list-style-type: none"> <li>• Possible pile tip damage if piles are not adequately protected while driving through very dense/hard soils</li> </ul>

- NOTES:
1. Costs are order of magnitude estimates in 2012 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 = (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 11-1132-0109-1000

W.P. 3049-08-00

LOCATION N 4875164.0 :E 224292.5

ORIGINATED BY MA

DIST West 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										WATER CONTENT (%)				
								○ UNCONFINED	+	FIELD VANE	● QUICK TRIAXIAL	×						LAB VANE				
481.36	GROUND SURFACE						20	40	60	80	100	10	20	30								
0.00	TOPSOIL, sandy, trace gravel Brown																					
480.96																						
0.40	SILTY SAND, trace clay, trace gravel, with topsoil Loose Brown		1	SS	17								○									
480.32													○									
1.04	SAND AND GRAVEL, trace silt Compact Brown																					
479.99			2	SS	51								○									
1.37	SAND, some silt, some gravel Compact to very dense Brown																					
			3	SS	20								○									
478.46																						
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								○									
			7	SS	81								○									
475.42																						
5.94	SAND, trace gravel, with sand and gravel layers Very dense Brown		8	SS	64								○									
474.65																						
6.71	SAND AND GRAVEL, trace to some silt Very dense Brown		9	SS	76								○									
			10	SS	57								○									
			11	SS	101/ 250mm								○									
471.30																						
10.06	SANDY SILT TILL, some gravel, some clay Very dense Grey		12	SS	106/ 250mm								○									

LDN\_MTO\_06 11-1132-0109.GPJ LDN\_MTO.GDT 26/10/12



PROJECT <u>11-1132-0109-1000</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>West</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 _____			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC NATURAL LIQUID LIMIT MOISTURE LIMIT CONTENT			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	
								<div><div>20406080100</div><div>○ UNCONFINED + FIELD VANE</div><div>● QUICK TRIAXIAL × LAB VANE</div></div>					<div><div>102030</div><div>W<sub>P</sub> W W<sub>L</sub></div></div>								
	SANDY SILT TILL, some gravel, trace to some clay Dense to very dense Grey		15	SS	80/ 150mm		467														
							466														
			16	SS	30/ 25mm		465														
463.87			17	SS	58/ 75mm		464														
18.50	END OF BOREHOLE																				
	Groundwater encountered at about elev. 479.5m during drilling on September 10, 2012.																				

**RECORD OF BOREHOLE No 3**

1 OF 2

**METRIC**

PROJECT 11-1132-0109-1000

W.P. 3049-08-00

LOCATION N 4875132.7 ; E 224274.1

ORIGINATED BY DB

DIST West 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 LIQUID LIMIT CONTENT			UNIT WEIGHT  γ  kN/m³	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>		GR	SA	SI	CL		
								20	40	60	80	100	WATER CONTENT (%)									
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									o				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									o					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							o					12	19	54	15
							471															
			12	SS	43		470															
			13	SS	68		469															
							468															
			14	SS	100/ 200mm		467							o					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 11-1132-0109-1000 W.P. 3049-08-00 LOCATION N 4875132.7 ; E 224274.1 ORIGINATED BY DB  
DIST West 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  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 (%)		
								20 40 60 80 100											10 20 30		
465.82			15	SS	100/ 75mm		466														
15.33	BOULDER																				
15.61	SANDY SILT TILL, some gravel, trace clay Very dense Grey						465														
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 11-1132-0109-1000

W.P. 3049-08-00

LOCATION N 4875158.0 ; E 224275.5

ORIGINATED BY DB

DIST West HWY 89




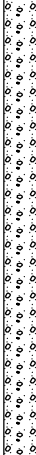

BOREHOLE TYPE POWER AUGER, HOLLOW STEM

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  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)										
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									WATER CONTENT (%)			GR	SA	SI	CL				
								○ UNCONFINED	+ FIELD VANE																		
								● QUICK TRIAXIAL	× LAB VANE																		
481.24	GROUND SURFACE							20	40	60	80	100															
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																				
							469																				
			13	SS	100/ 250mm																						
							468																				
			14	SS	100/ 275mm		467																				

Continued Next Page

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

## METRIC

PROJECT 11-1132-0109-1000

W.P. 3049-08-00

LOCATION N 4875158.0 ;E 224275.5

ORIGINATED BY DB

DIST West HWY 89BOREHOLE 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/m³	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>		WATER CONTENT (%)			
								○ UNCONFINED	+ FIELD VANE	● QUICK TRIAXIAL	× LAB VANE	20	40	60	80		100	10	20	30
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													

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



**RECORD OF BOREHOLE No 5**

1 OF 1

**METRIC**

PROJECT 11-1132-0109-1000

W.P. 3049-08-00

LOCATION N 4875143.2 ; E 224256.1

ORIGINATED BY MA

DIST West 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 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									
							20 40 60 80 100										
							○ UNCONFINED + FIELD VANE										
							● QUICK TRIAXIAL × LAB VANE										
							20 40 60 80 100					WATER CONTENT (%)					
												10 20 30					
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		3	SS	9		480										
	Loose to compact Brown		4	SS	23												
478.77							479									0 1 86 13	
3.66	SILTY FINE SAND, trace clay		5	SS	15												
478.01	Compact Brown															0 50 44 6	
4.42	SAND, trace gravel, trace silt, with clayey silt layers		6	SS	24		478										
477.25	Compact Brown																
5.18	SAND AND GRAVEL, trace silt		7	SS	61		477										
	Dense to very dense Brown																
			8	SS	42		476										
475.88	END OF BOREHOLE																
6.55	Groundwater encountered at about elev. 480.3m during drilling on August 28, 2012.																

**RECORD OF BOREHOLE No 6**

1 OF 1

**METRIC**

PROJECT 11-1132-0109-1000

W.P. 3049-08-00

LOCATION N 4875151.1 ; E 224314.7

ORIGINATED BY MA

DIST West HWY 89

BOREHOLE TYPE POWER AUGER, HOLLOW STEM

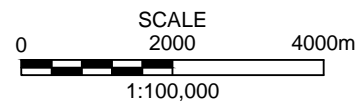
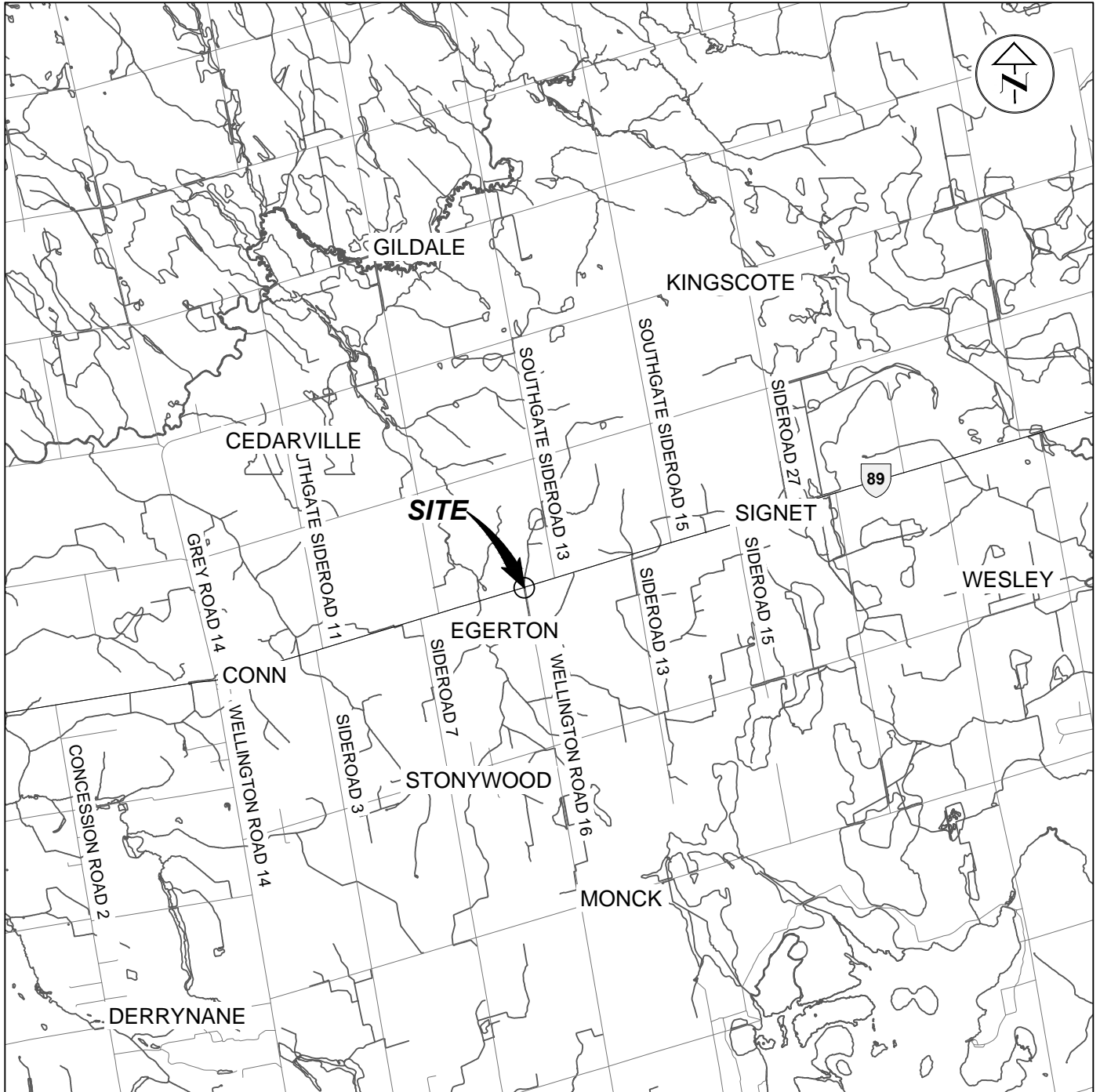
COMPILED BY LMK/AMG

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  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 ● QUICK TRIAXIAL	+ FIELD VANE × LAB VANE	20							40	60
482.82	PAVEMENT SURFACE					▽												
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 Loose Brown		1	SS	7		482											
			2	SS	9		481							○		0 13 65 22		
480.69																		
2.13	SAND, some silt, some gravel Compact Brown		3	SS	24		480											
479.92																		
2.90	SAND AND GRAVEL, trace to some silt, trace clay, with cobbles Compact to dense Brown		4	SS	28		479					○				55 34 8 3		
			5	SS	31													
478.40																		
4.42	SILTY SAND AND GRAVEL, with cobbles Dense Brown		6	SS	37		478											
			7	SS	46													
476.88							477											
5.94	SAND, medium to coarse, trace gravel 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.																	



## REFERENCE

PLAN BASED ON CANMAP STREETFILES V.2008.5.

## NOTE

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

PROJECT

HIGHWAY 89 STRUCTURE REPLACEMENTS  
SOUTH SAUGEEN RIVER BRIDGE, SITE No. 35-27  
GWP 3049-08-00

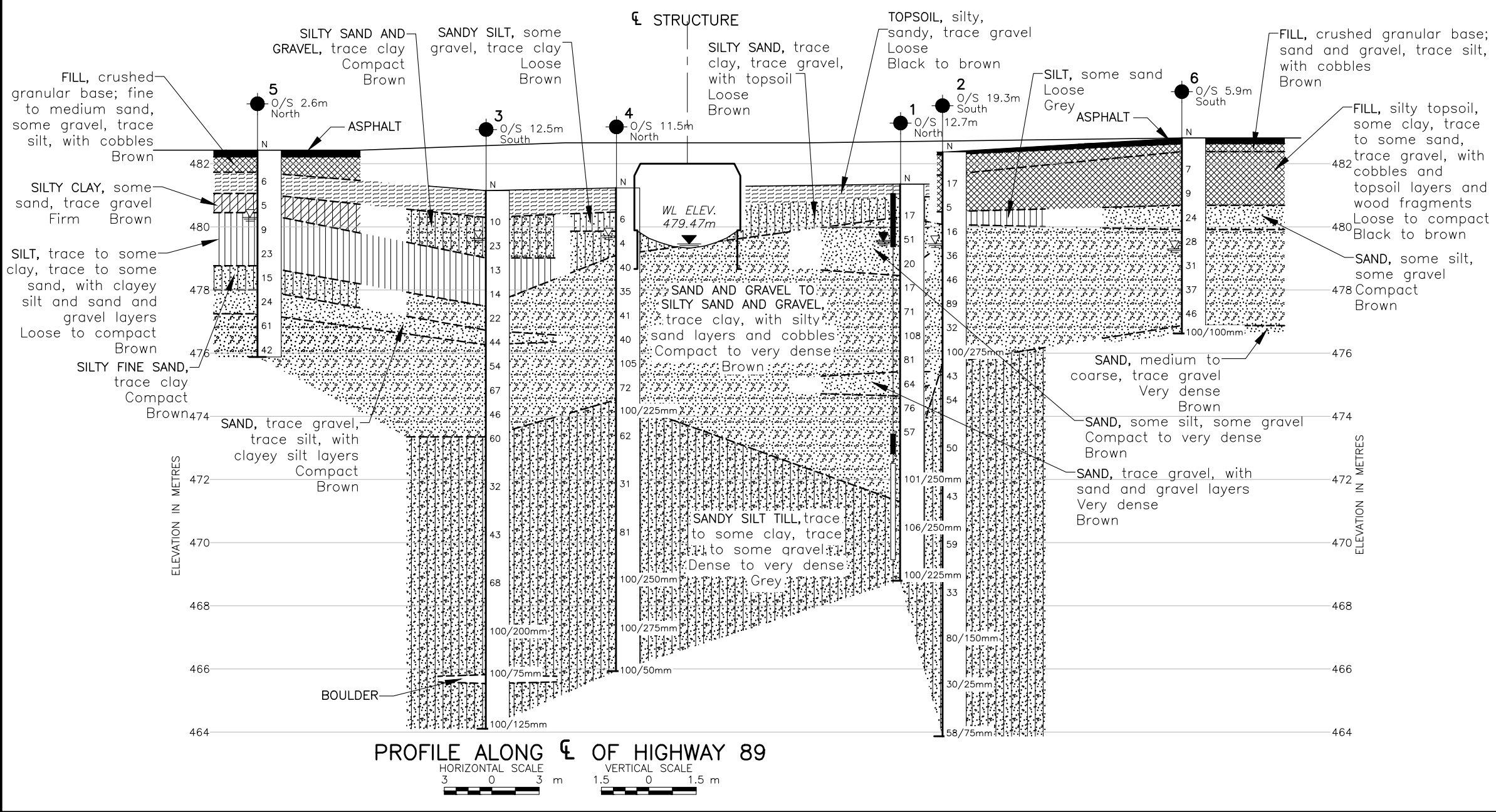
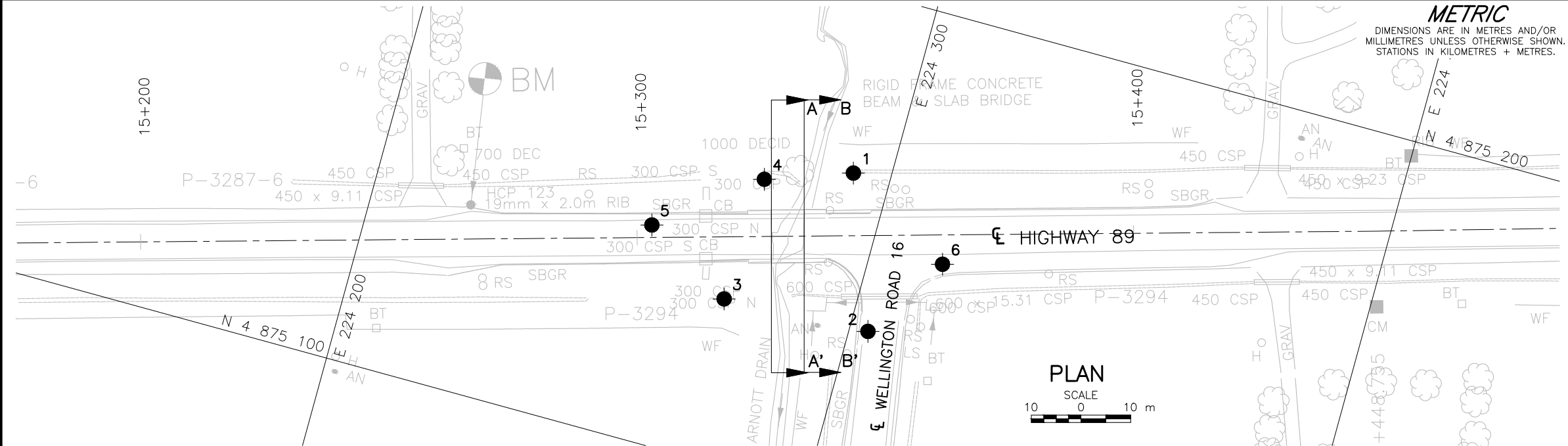
TITLE

## KEY PLAN



PROJECT No. 11-1132-0109		FILE No. 1111320109-1000-F01001	
CADD	LMK	Sept. 25/12	SCALE AS SHOWN
CHECK			REV. 0

**FIGURE 1**



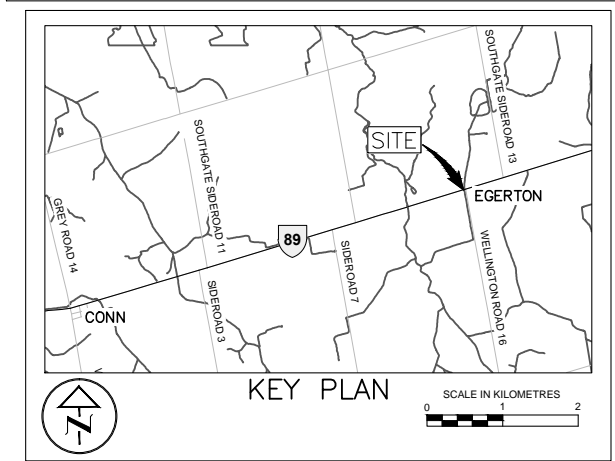
CONT No.  
WP No. 3049-08-00

SOUTH SAUGEE RIVER BRIDGE  
HIGHWAY 89 STRUCTURE REPLACEMENTS

BOREHOLE LOCATIONS AND SOIL STRATA

SHEET

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



LEGEND

- Borehole - Current Investigation
- Seal
- Standpipe
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- WL in standpipe, measured on October 1, 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

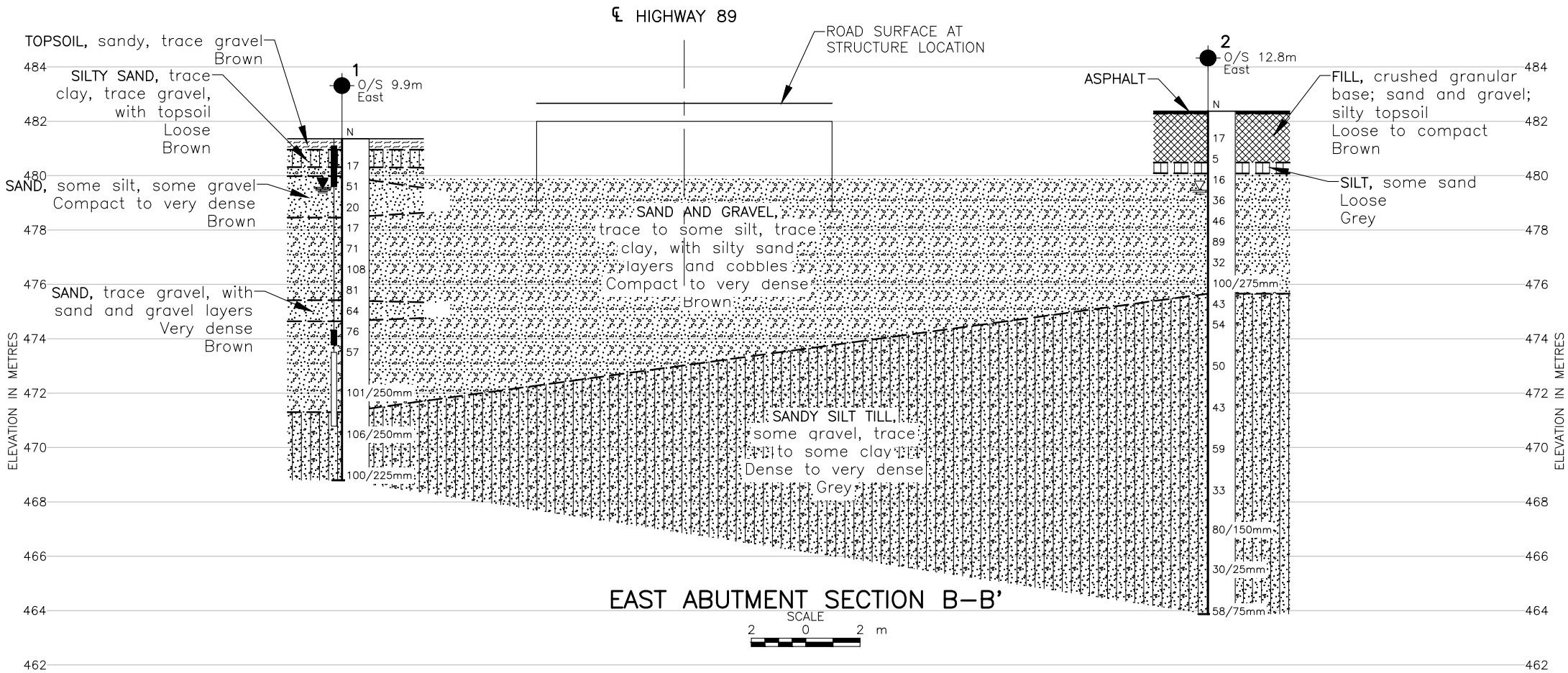
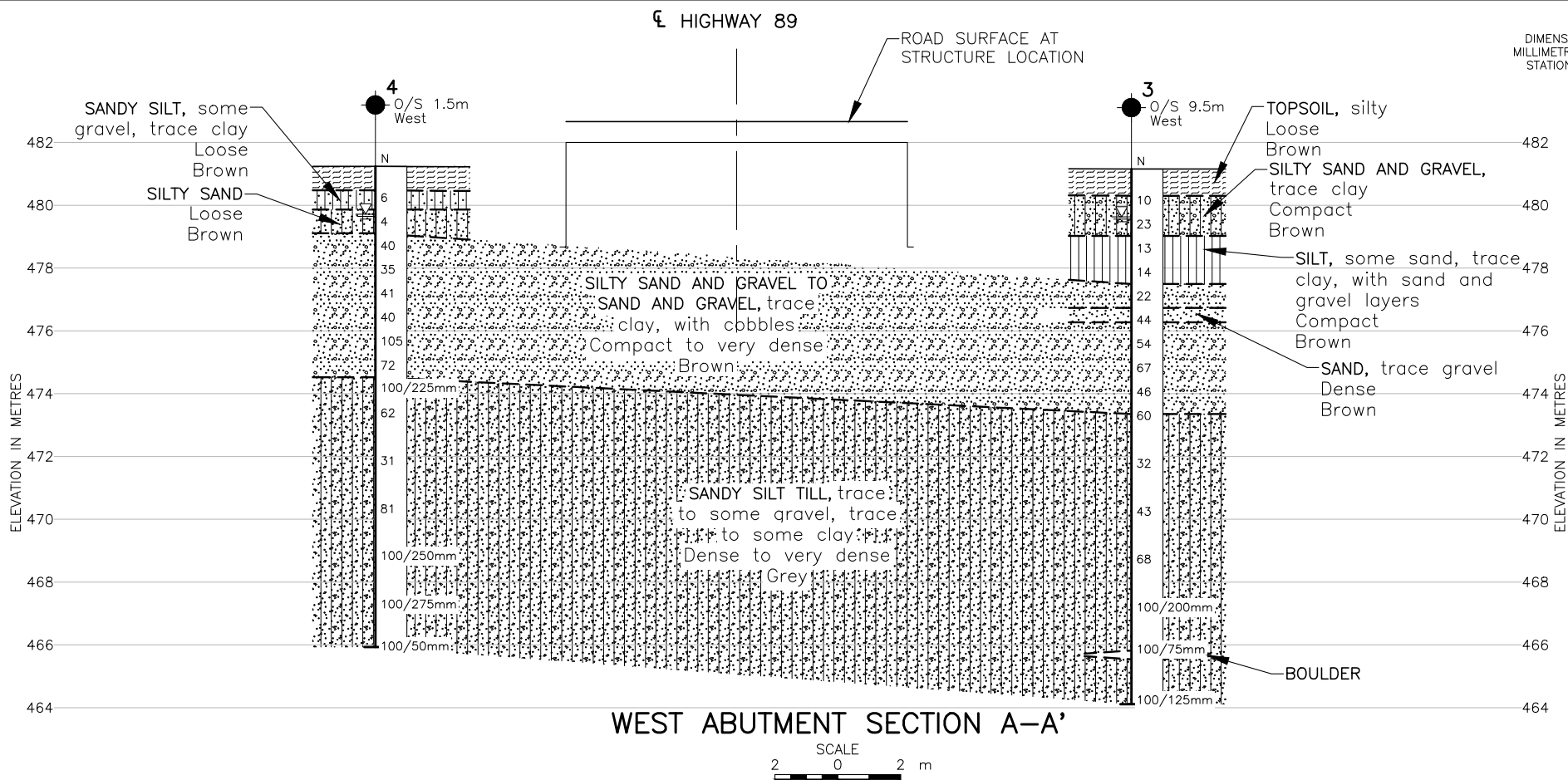
This drawing is for subsurface information only. Surface details and features are for conceptual illustration.

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.			
HWY.	89	PROJECT NO.	11-1132-0109
SUBM'D.	TP	CHKD.	DATE: Oct. 16/12
DRAWN:	LMK	CHKD.	APPD.
DIST.		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

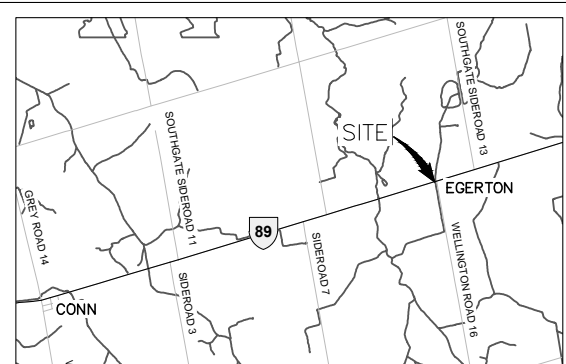
SOUTH SAUGEEN RIVER BRIDGE  
HIGHWAY 89 STRUCTURE REPLACEMENTS

SOIL STRATA

SHEET



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



LEGEND

- Borehole – Current Investigation
- Seal
- Standpipe
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- WL in standpipe, measured on October 1, 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**

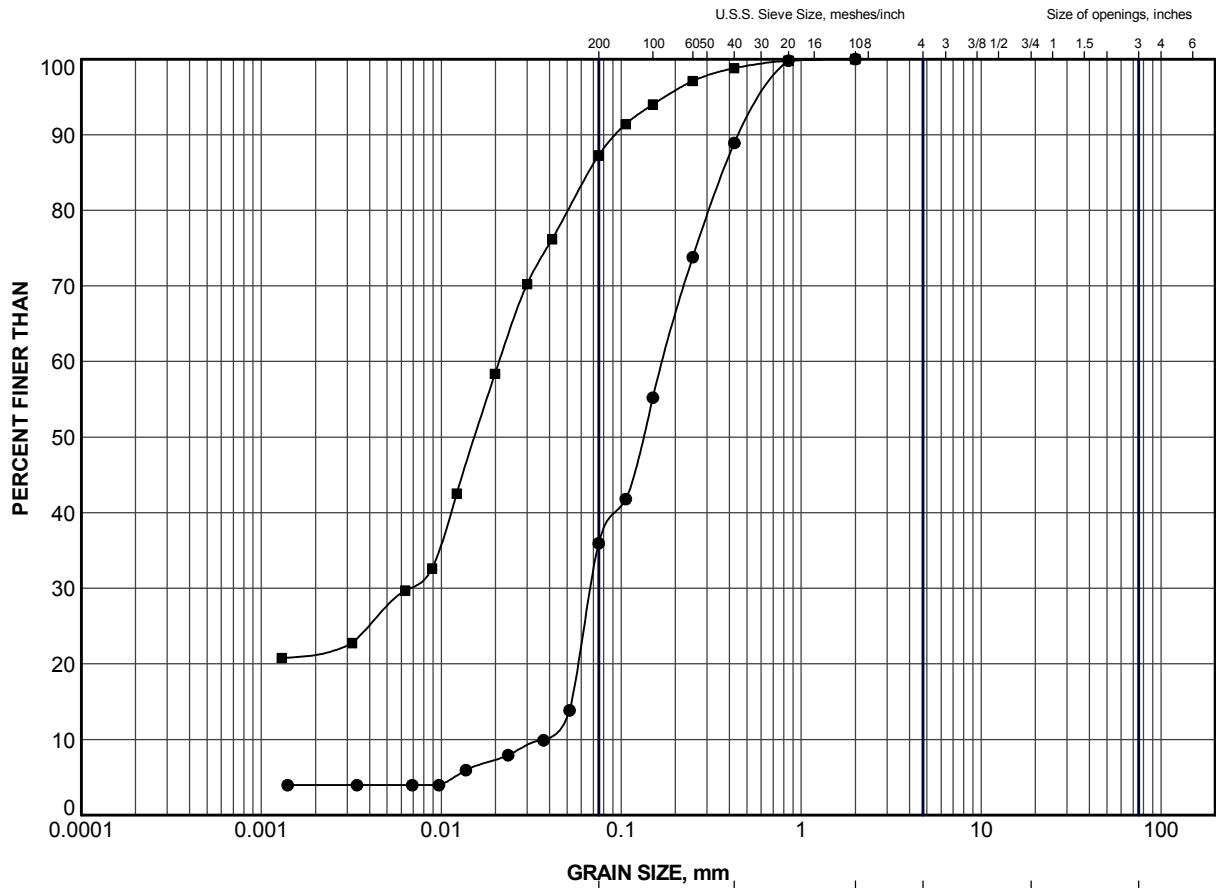
Base plans provided in digital format by Morrison Hershfield Limited.

NO.	DATE	BY	REVISION
Geocres No.			
HWY. 89		PROJECT NO. 11-1132-0109	DIST.
SUBM'D. TP	CHKD.	DATE: Oct. 16/12	SITE: 35-27
DRAWN: LMK	CHKD.	APPD.	DWG. 2



# **APPENDIX A**

## **Laboratory Test Data**

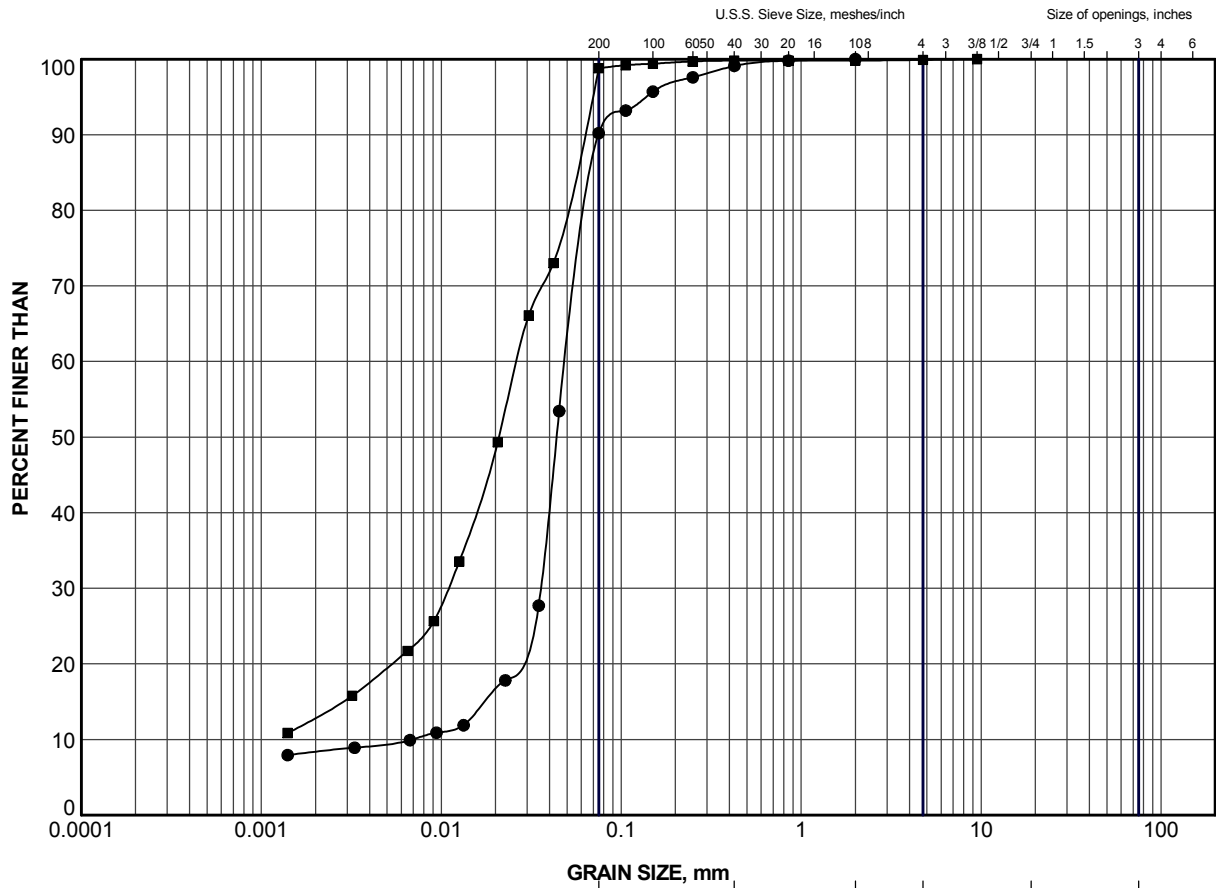


### LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	4	2	479.5
■	6	2	481.1

PROJECT				HIGHWAY 89 STRUCTURE REPLACEMENTS SOUTH SAUGEEN RIVER BRIDGE, SITE No. 35-27 GWP 3049-08-00			
TITLE				GRAIN SIZE DISTRIBUTION FILL			
PROJECT No:11-1132-0109-1000				FILE No. 1111320109-1000-F010A1			
DRAWN LMK Oct 15/12				SCALE N/A REV.			
CHECK				FIGURE A-1			






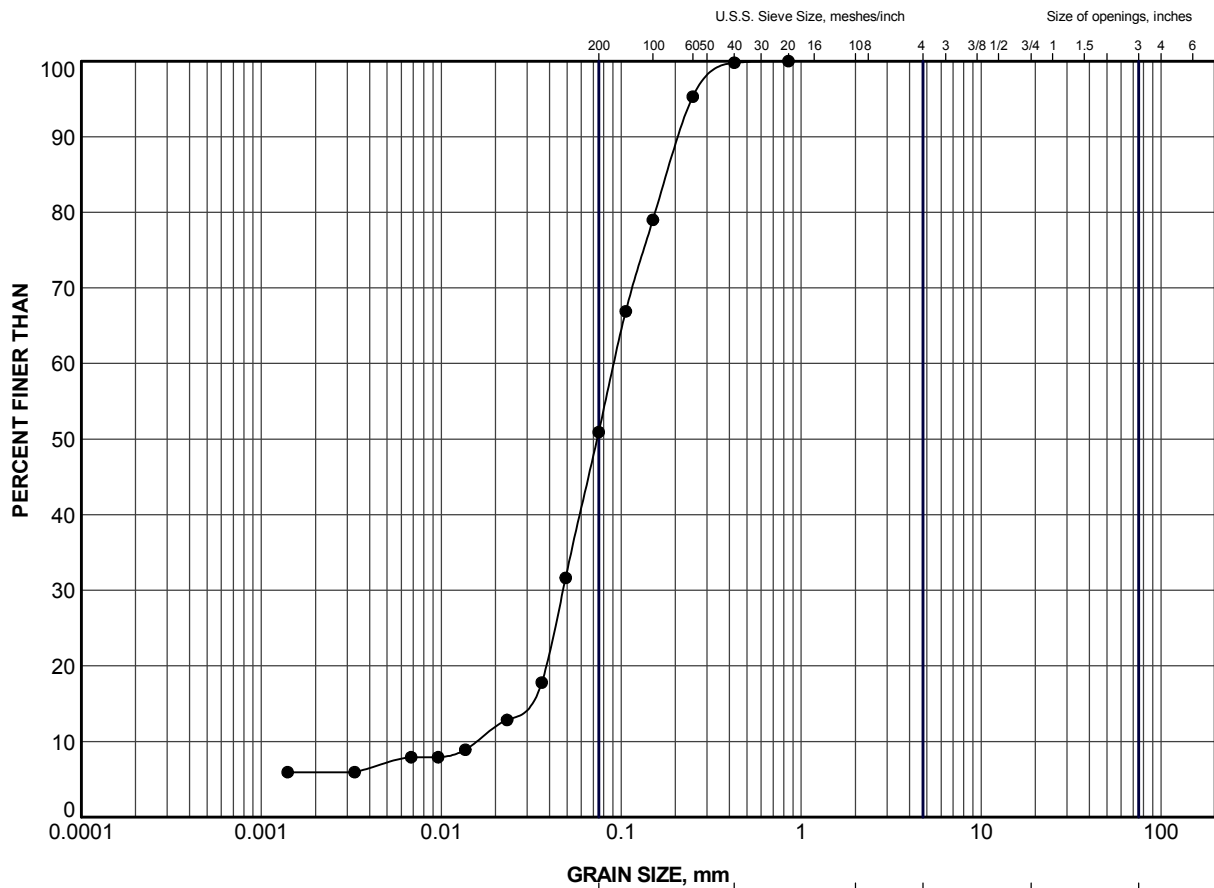
CLAY AND SILT	SAND SIZE, mm					Cobble Size
	fine	medium	coarse	fine	coarse	
	SAND SIZE			GRAVEL SIZE		

### LEGEND

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

PROJECT				HIGHWAY 89 STRUCTURE REPLACEMENTS SOUTH SAUGEEN RIVER BRIDGE, SITE No. 35-27 GWP 3049-08-00			
TITLE				GRAIN SIZE DISTRIBUTION SILT			
PROJECT No:11-1132-0109-1000				FILE No. 1111320109-1000-F010A2			
DRAWN LMK Oct 15/12				SCALE N/A REV.			
CHECK				FIGURE A-2			
 <b>Golder Associates</b> LONDON, ONTARIO							




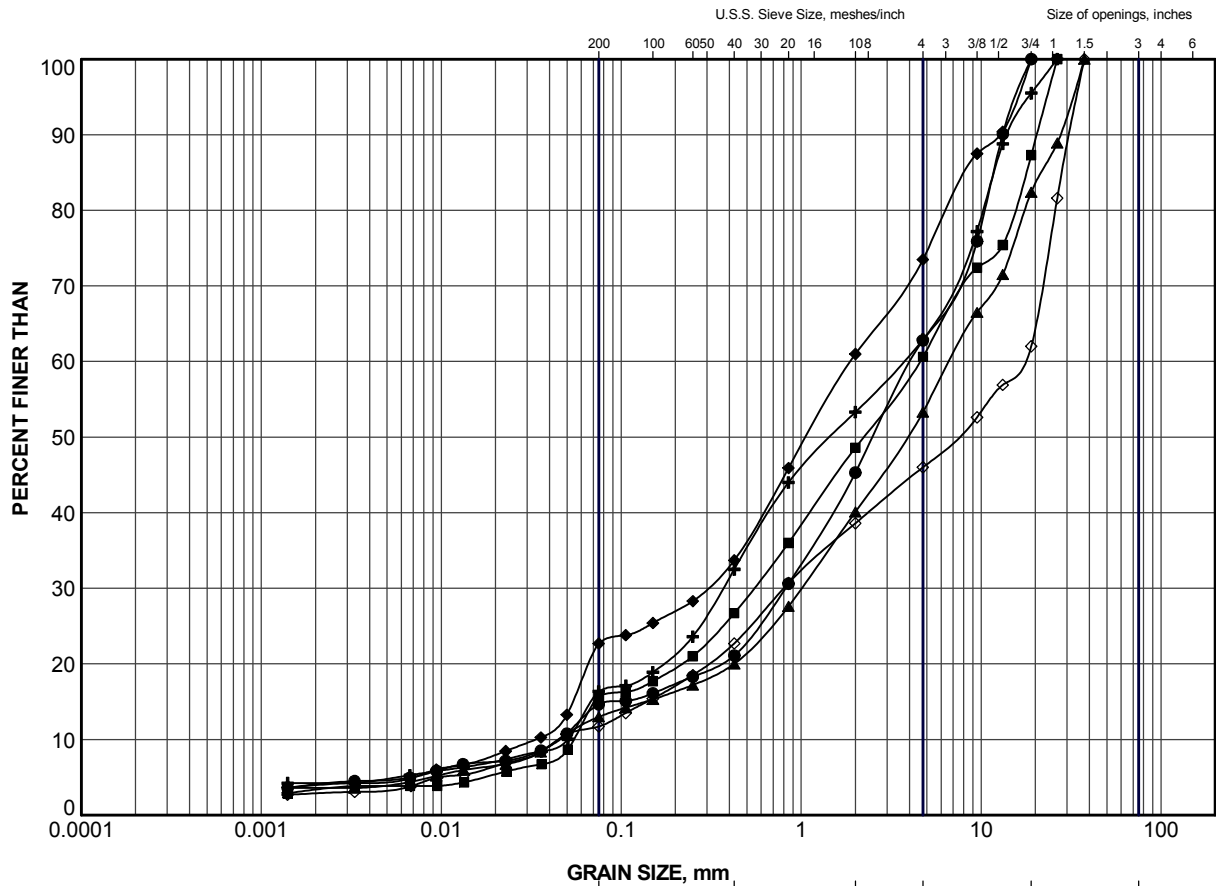


GRAIN SIZE, mm						
CLAY AND SILT	fine	medium	coarse	fine	coarse	Cobble Size
	SAND SIZE			GRAVEL SIZE		

#### LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	5	5	478.4


PROJECT				HIGHWAY 89 STRUCTURE REPLACEMENTS SOUTH SAUGEEN RIVER BRIDGE, SITE No. 35-27 GWP 3049-08-00			
TITLE				GRAIN SIZE DISTRIBUTION SILTY FINE SAND			
PROJECT No:11-1132-0109-1000				FILE No. 1111320109-1000-F010A3			
DRAWN		LMK		Oct 15/12		SCALE N/A REV.	
CHECK						FIGURE A-3	
 <b>Golder Associates</b> LONDON, ONTARIO							

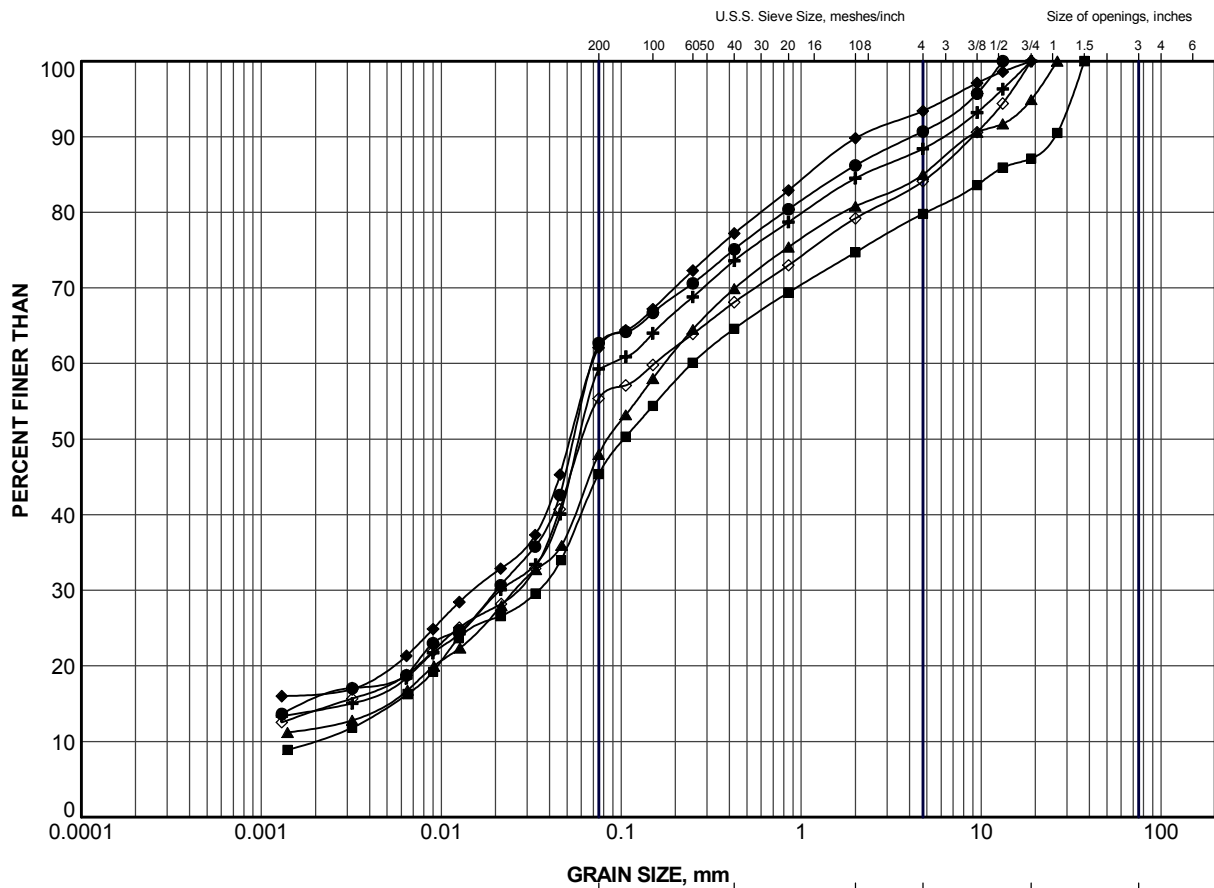


CLAY AND SILT	GRAVEL SIZE, mm					Cobble Size
	fine	medium	coarse	fine	coarse	
	SAND SIZE			GRAVEL SIZE		

#### 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				HIGHWAY 89 STRUCTURE REPLACEMENTS SOUTH SAUGEEN RIVER BRIDGE, SITE No. 35-27 GWP 3049-08-00			
TITLE				GRAIN SIZE DISTRIBUTION SAND AND GRAVEL			
PROJECT No:11-1132-0109-1000				FILE No. 1111320109-1000-F010A4			
DRAWN		LMKWDF		Oct 25/12		SCALE N/A REV.	
CHECK						FIGURE A-4	
 <b>Golder Associates</b> LONDON, ONTARIO							



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

#### 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				HIGHWAY 89 STRUCTURE REPLACEMENTS SOUTH SAUGEEN RIVER BRIDGE, SITE No. 35-27 GWP 3049-08-00			
TITLE				GRAIN SIZE DISTRIBUTION SANDY SILT TILL			
PROJECT No:11-1132-0109-1000				FILE No. 1111320109-1000-F010A5			
DRAWN		MKWDF		Oct 25/12		SCALE N/A REV.	
CHECK						FIGURE A-5	
 <b>Golder Associates</b> LONDON, ONTARIO							



# **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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