



January 2013

FOUNDATION INVESTIGATION AND DESIGN REPORT

**Culvert Replacement, Station 17+730
Site No. 35-514/C
Highway 89 Structure Replacements and Rehabilitation
From 6.0 km west of Mount Forest to Shelburne
GWP 3049-08-00
Ministry of Transportation, West Region**

Submitted to:

Mr. Edward Li, P.Eng.
Vice-President, Transportation and Civil Structures
Morrison Hershfield Limited
600 - 235 Yorkland Boulevard
Toronto, Ontario
M2J 1T1

REPORT



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LIST OF ABBREVIATIONS

LIST OF SYMBOLS

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PART A

FOUNDATION INVESTIGATION REPORT

**CULVERT REPLACEMENT, SITE NO. 35-514/C, STATION 17+730
HIGHWAY 89 STRUCTURE REPLACEMENTS AND REHABILITATIONS
FROM 6.0 KM WEST OF MOUNT FOREST TO SHELBURNE
GWP 3049-08-00
MINISTRY OF TRANSPORTATION - 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 detail design of 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 culvert at Station 17+730 (Site 35-514/C).

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 our 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 DESCRIPTION

The subject culvert is situated at Station 17+730 on Highway 89, approximately 195 metres west of Southgate Township Road 15 near the boundary of Wellington County and Grey County, Ontario. The villages of Egerton and Signet, are 3.7 kilometres west and 1.5 kilometres east of the site, respectively. The location of the culvert 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 flow direction in the culvert is from north to south beneath Highway 89. The construction date of the culvert is unknown. The existing culvert is open-footing non-rigid frame (NRFO) structure with the following characteristics:

Dimensions (m)	Obvert Elevation (m)		Construction
	Lt ¹	Rt ¹	
3.05 x 1.22 x 23.20	494.235	494.130	NRFO

NOTE: 1. When facing the direction of increasing chainage, Lt and Rt are defined as Left and Right of centreline, respectively.

The banks of the drainage channel upstream and downstream of the culvert are grass covered and the channel flows through fields adjacent to Highway 89. Site photographs are provided in Appendix B.

The culvert is situated in a rural agricultural area with low relief. Ground surface elevations in the vicinity of the culvert range from about 493 to 495 metres.

2.1 Site Geology

The project area is located on a glacial till plain within the Dundalk Till Plain physiographic region. This region is characterized by swamps or bogs and poorly drained depressions. Surficial silt deposits less than 0.6 metres deep cover most of the area.¹ The quaternary geological mapping indicates that surficial soils consist of sandy silt to silty sand till.²

The geological mapping indicates that the underlying bedrock consists of tan or brown dolostone of the Guelph Formation.³ The mapped bedrock surface lies between elevation 456 metres west of the site and 470 metres east of the site.⁴ The overburden thickness in the local area surrounding the site has been mapped as being between 32 and 37 metres.⁵

¹ Chapman, L.J., and Putnam, D.F., 1984: Physiography of Southern Ontario; Ontario Geological Survey, Special Volume 2, 270p. Accompanied by Map. P.2715 (coloured), scale 1:600,000.

² Gwyn, Q.H.J. 1972: Quaternary Geology of the Dundalk Area, Southern Ontario; Ontario Department of Mines and Northern Affairs, Preliminary Map P.727, Geological Series, scale 1:50,000. Geology 1971.

³ Liberty, B.A., Telford, P.G., and Bond, I.J., 1976: Paleozoic Geology Dundalk Sheet, Southern Ontario; Ontario Division of Mines Map 2340, Paleozoic Geology Series, Scale 1:50,000.

⁴ Gwyn, Q.H.J., and Frazer, J.Z., 1975: Bedrock Topography, Dundalk Area, Southern Ontario, Ontario Department of Mines, Preliminary Map P.306 (Revised), Bedrock Topography Series. Scale 1:50,000. Geological compilation 1975.

⁵ Gwyn, Q.H.J., and Frazer, J.Z., 1975: Drift Thickness of the Dundalk Area, Southern Ontario, Ontario Department of Mines, Preliminary Map P.1023, Drift Thickness Series. Scale 1:50,000. Geological compilation 1975.



3.0 INVESTIGATION PROCEDURES

The field work for the investigation was carried out between August 23 and September 5, 2012, during which time 5 boreholes were drilled at the locations shown on the Borehole Location Plan, Drawing 1.

The boreholes were drilled using track-mounted drilling equipment supplied and operated by a specialist drilling contractor. Samples of the overburden were typically obtained at depth intervals of 0.75 metres using 50 millimetre outside diameter split spoon sampling equipment in accordance with the Standard Penetration Test procedures (ASTM D1586).

The samplers used in the investigations limit the maximum particle size that can be sampled and tested to about 40 millimetres. Therefore, particles or objects that may exist within the soils that are larger than this dimension will not be sampled or represented in the grain size distributions. Larger particle sizes, including cobbles and boulders, are known to be present in the glacial till deposits as discussed in the text of this report.

Groundwater conditions in the boreholes were observed throughout the drilling operations and a groundwater observation well was installed in borehole 201 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 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.

The as-drilled borehole locations and ground surface elevations are shown on the Record of Borehole sheets and on Drawing 1. The table below summarizes the coordinates, ground surface elevations and depths of the boreholes.

Borehole	Location (m)		Ground Surface Elevation (m)	Borehole Depth (m)
	Northing	Easting		
201	4 875 796	226 583	493.59	8.08
202	4 875 827	226 607	493.53	7.77
203	4 875 815	226 590	494.91	9.42
204	4 875 810	226 614	494.76	5.94
205	4 875 799	226 575	494.81	5.94



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 granular fill used for the pavement structure or topsoil overlying variable embankment fill materials then in sequence, clayey silt till and sandy silt till. The sandy silt till was interlayered with silt near elevation 490 metres.

The locations and elevations of the boreholes, together with the interpreted stratigraphic profile, are shown on Drawing 1. A detailed description of the subsurface conditions encountered in the boreholes is provided on the Record of Borehole sheets and is summarized in the following sections.

4.2 Soil Conditions

4.2.1 Topsoil

A layer of topsoil was encountered at the ground surface in boreholes 201 and 202. The topsoil was 180 to 430 millimetres thick.

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

4.2.2 Pavement Structure

Pavement granular base materials were encountered at the roadway shoulder surface in boreholes 203, 204 and 205. The granular base materials were about 150 to 340 millimetres thick.



4.2.3 Fill

Fill materials were encountered beneath the pavement granular base materials in boreholes 203, 204 and 205 from elevations 494.5 to 494.8 metres. The fill consisted of topsoil, silt, sandy silt, sand, sand and gravel, and clayey silt. The fill materials were 1.2 to 2.0 metres thick with standard penetration test N^6 values of 5 to 24 blows per 0.3 metres. Samples of the fill had water contents of 5 to 37 per cent.

4.2.4 Clayey Silt

Layers of stiff clayey silt were encountered beneath the fill in boreholes 204 and 205 at elevations 493.4 and 493.1 metres, respectively. The clayey silt layers were 0.4 to 0.8 metres thick with a measured N value of 15 blows per 0.3 metres.

4.2.5 Clayey Silt Till

Layers of stiff to hard clayey silt glacial till were encountered between elevations 490.5 and 493.4 metres beneath the topsoil in boreholes 201 and 202, below the fill in borehole 203, beneath the clayey silt in borehole 205, and beneath the sandy silt till in borehole 203. The clayey silt till was 0.8 to 2.6 metres thick. Cobbles and/or boulders were not encountered within the clayey silt till at the boreholes. However, the presence of cobbles and boulders should be anticipated due to its depositional history.

The clayey silt till had measured N values of 12 to 37 blows per 0.3 metres and water contents of 10 to 11 per cent. The clayey silt till is of low plasticity based on one Atterberg limits determination carried out on a sample obtained during standard penetration testing. The plastic limit was 17 per cent, the liquid limit was 32 per cent, and the plasticity index was 15 per cent. The Atterberg limits data for the clayey silt till are presented on Figure A-4. Grain size distribution curves for samples of the clayey silt till are provided on Figure A-1.

4.2.6 Sandy Silt Till

Layers of compact to very dense sandy silt glacial till were encountered in all boreholes. An upper layer of sandy silt till was encountered below the clayey silt or clayey silt till between elevations 490.8 and 492.6 metres and a lower layer was encountered between elevations 489.0 and 489.6 metres. Boreholes 201 to 204 were terminated in the sandy silt till after exploring it for 0.2 to 3.9 metres. The upper sandy silt till layers were 0.5 to 2.3 metres thick.

⁶ 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.



The sandy silt till had measured N values of 20 to over 100 blows per 0.3 metres in the upper layer and 24 to over 100 blows per 0.3 metres in the lower layer. Water contents of the samples ranged from 6 to 11 per cent. Grain size distribution curves for samples of the sandy silt till recovered from the standard penetration testing are provided on Figure A-2. Atterberg limits determination carried out on a sample of the sandy silt till obtained from borehole 201 indicated a plastic limit of 12 per cent, liquid limit of 19 per cent and a plasticity index of 6 per cent. The Atterberg limits data are presented on Figure A-4. Cobbles were encountered in the sandy silt till in borehole 204 and a boulder was encountered in borehole 201. As such, the presence of cobbles and boulders should be anticipated within the sandy silt till.

4.2.7 Silt

Layers of compact to dense silt were encountered between elevations 489.2 and 490.4 metres beneath the clayey silt till in borehole 203 and beneath the sandy silt till in boreholes 201, 202, 204 and 205. Where fully penetrated, the silt layers were 0.3 to 1.3 metres thick. Borehole 205 was terminated in silt with clayey silt layers after exploring some 0.3 metres. The silt had measured N values of 28 to 45 blows per 0.3 metres and water contents of 16 to 20 per cent with an average water content of about 18 per cent. Grain size distribution curves for samples of the silt are provided on Figure A-3.

4.3 Groundwater Conditions

Groundwater conditions were observed during and on completion of drilling and sampling and a groundwater observation well was installed in borehole 201. Installation details are provided on the corresponding Record of Borehole sheet following the text of this report. Groundwater was encountered in boreholes 202, 203 and 205 at depths of 4.0 to 5.2 metres or between elevation 489.5 and 490.4 metres. A summary of the encountered measured groundwater levels is provided in the table below:

Borehole	Ground Surface Elevation (m)	Encountered Groundwater Elevation (m)	Measured Groundwater Level Elevation (m)		
			Oct. 1, 2012	Oct. 24, 2012	Dec. 28, 2012
201	493.59	dry	492.12	492.83	493.10
202	493.53	489.5	-	-	-
203	494.91	489.7	-	-	-
204	494.76	dry	-	-	-
205	494.81	490.4	-	-	-



FOUNDATION INVESTIGATION AND DESIGN REPORT CULVERT REPLACEMENT, STATION 17+730

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 492.40 and 492.64 metres on August 24 and December 28, 2012. The watercourse was dry during drilling on September 5, 2012. On October 24 and December 28, 2012, the water level in the groundwater observation well installed in borehole 201 was about 0.76 and 0.49 metres below ground surface or at about elevations 492.83 and 493.10 metres, respectively.

Based on the observed groundwater levels, the surrounding topography, and water levels in the drain, the groundwater level is inferred to typically vary from about elevation 492.2 to 493.2 metres. The groundwater levels are expected to fluctuate seasonally and are expected to be higher during periods of sustained precipitation or during spring snow melt conditions.



5.0 MISCELLANEOUS

The investigation was carried out using equipment supplied and operated by Aardvark Drilling Inc., an Ontario Ministry of Environment licensed well contractor. The field operations were supervised by Mr. Michael Arthur under the direction of Mr. David J. Mitchell. The laboratory testing was carried out at Golder Associates' London laboratory under the direction of Mr. Chris M. Sewell. The laboratory is an accredited participant in the MTO Soil and Aggregate Proficiency Program and is certified by the Canadian Council of Independent Laboratories for testing Types C and D aggregates. This report was prepared by Ms. Dirka U. Prout, P.Eng. under the direction of the Team Leader, Dr. Storer J. Boone, P.Eng. This report was reviewed by Mr. Fintan J. Heffernan, P.Eng., the Designated MTO Contact and Quality Control Auditor for this assignment.

GOLDER ASSOCIATES LTD.

ORIGINAL SIGNED

Dirka U. Prout, P.Eng.
Project Engineer

ORIGINAL SIGNED

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

ORIGINAL SIGNED

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

TP/DUP/SJB/FJH/cr

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**FOUNDATION INVESTIGATION AND DESIGN REPORT
CULVERT REPLACEMENT, STATION 17+730**

PART B

FOUNDATION DESIGN REPORT

**CULVERT REPLACEMENT, SITE NO. 35-514/C, STATION 17+730
HIGHWAY 89 STRUCTURE REPLACEMENTS AND REHABILITATIONS
FROM 6.0 KM WEST OF MOUNT FOREST TO SHELBURNE
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6.0 ENGINEERING RECOMMENDATIONS

6.1 General

This section of the report provides recommendations on the foundation aspects of the design of replacement culvert at Station 17+730 on Highway 89 on the boundary of Wellington County and Grey County (Site 35-514/C).

The recommendations are based on interpretation of the factual data obtained from the five boreholes advanced during the investigation at this site. The interpretation and recommendations are intended to provide the designers with sufficient information to design the proposed culvert foundations. As such, 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.

The existing culvert is a 23.20 metre long NRFO structure with a 3.05 metre span and a 1.22 m high opening with an invert elevation of 493.05 metres at the inlet and 492.95 metres at the outlet. Based on information provided by MH, it is understood that the culvert is to be replaced with a pre-cast concrete box culvert. Other culvert types were considered as discussed in the Structural Design Report for this site but the use of a concrete pre-cast box was considered by MH to be the preferred option which is currently being advanced for Detail Design. The proposed span of the replacement culvert is 3.5 metres with a height of 2.1 metres. The replacement culvert will be constructed in the same location as the existing culvert. Substrate consisting of washed river-run stone will be placed to a thickness of 300 to 600 millimetres along the invert within the box culvert. Erosion protection in the form of a 450 millimetre thick granular apron of river-run stone will extend 4.8 metres beyond the culvert inlet and outlets respectively. The design invert elevations will be 492.74 metres at the inlet and 492.62 metres at the outlet. The design high water level is elevation 493.93 metres. The new culvert will have wingwalls at the northeast and southwest corners. No grade raise is proposed at this location.

6.2 Replacement Culvert

6.2.1 Foundations

The subsurface conditions encountered during the investigation generally consisted of the existing pavement granular base materials or topsoil overlying variable fill materials to elevation 493 metres, stiff to hard clayey silt till to elevation 492 metres, compact to very dense upper sandy silt till to elevation 490 metres, compact to dense silt to elevation 489 metres then compact to very dense sandy silt till. The groundwater level is inferred to typically vary from about elevation 492.2 to 493.2 metres. The water level in the watercourse varied from dry to about elevation 492.6 metres.



The culvert replacement should be designed to withstand the appropriate weight of fill and traffic loading. It is not necessary to found a box culvert replacement at the standard depth for frost protection purposes as box structures are tolerant of small magnitude movements related to freeze-thaw cycles should these occur. A box culvert replacement should, however, be founded below any existing fill and surficial organic materials. Based on the soil conditions encountered at the borehole locations, and the design culvert invert elevations, the replacement box culvert may be founded on the very stiff to hard clayey silt till and/or compact sandy silt till at or below elevation 492.0 metres. Any observed fill materials or soft or loose soils should be removed to the native glacial till soils. Any low areas should be brought to design grade using lean concrete fill or well graded compacted granular materials.

Geotechnical Resistances

The very stiff to hard clayey silt till and compact to very dense sandy silt till are suitable for support of the proposed culvert replacement. A factored geotechnical resistance at Ultimate Limit States (ULS) of 375 kilopascals (kPa) and a geotechnical resistance at Serviceability Limit States (SLS) of 250 kPa may be used for design purposes. The SLS value corresponds to a maximum of 25 millimetres of total settlement for new culvert construction.

Frost Treatment

Frost treatment in the form of a frost taper symmetrical about the culvert centreline must be provided in accordance with Ontario Provincial Standard Drawing (OPSD) 803.010. The design frost penetration depth for this area is 1.6 metres below ground surface.

Resistance to Lateral Forces/Sliding Resistance

The resistance to lateral forces/sliding resistance between the culvert base and the bedding should be calculated in accordance with Section 6.7.5 of the Canadian Highway Bridge Design Code (CHBDC). The coefficient of friction, $\tan \delta$, between the pre-cast concrete slab and compacted Granular A fill may be taken as 0.45. In accordance with the CHBDC Section 6.7.5, a factor of 0.8 is applied in the equation to calculate the factored horizontal geotechnical resistance, H_{ri} , as follows:

$$H_{ri} = 0.8A'c' + 0.8V\tan\delta > H_f$$

where:

A'	-	effective contact area, square metres
c'	=	Nil
$\tan \delta$	-	as given above
V	-	unfactored vertical force, kilonewtons
H_f	-	unfactored horizontal load, kilonewtons



Other Design Considerations

No widening or grade raise of the existing Highway 89 embankment will be undertaken in conjunction of replacement of the culvert at Station 17+730. Since negligible changes are anticipated in the embankment loading, differential settlement along the replacement culvert is not anticipated and cambering of the replacement culvert will not be required.

Water flowing beneath a culvert could potentially cause undermining and scouring. Seepage flowing around the culvert barrel or walls has the potential to remove fines from the embankment fill and lead to piping and erosion. Therefore the replacement culvert must be designed with the appropriate end treatment to prevent undermining, scouring and piping. The risk of piping at this location is considered low because the foundation soils are not susceptible to piping. As required by the CHDBC, pre-cast concrete box culverts should be designed with cutoff walls, at least at the upstream end, to prevent undermining or possible collapse of the ends.

The results of the hydraulic analyses indicated that water is not ponded at this culvert for an extended period of time at the 25-year design flows and there will not be a significant difference in hydraulic head between each end of the culvert. Therefore the use of additional anti-seepage measures such as clay seals or outlet filters is not considered necessary.

If the water flow velocities are sufficiently high, provision should be made for scour protection, in the form of non-woven geotextiles and/or rip-rap, at the inlet and outlet. The requirements for and specific design of scour and erosion protection measures should be assessed by the hydraulic design engineer. However, as a minimum, it is recommended that rip-rap treatment consistent with the standard OPSD 810.010 Treatment Type A should be provided at the culvert outlet. In addition, sediment control measures such as silt fences and erosion blankets may be required during construction along with diversion/piping of the watercourse to mitigate migration of fine particles.

6.2.2 Bedding

For pre-cast box culverts, bedding should be placed above a properly prepared subgrade from which all frozen, soft, uncompacted fill, organic materials or other deleterious materials have been removed. Subexcavated material below the design subgrade elevation should be replaced with compacted OPSS Granular B, Type II. It is recommended that the box culvert units be placed on a minimum thickness of compacted 300 millimetres of Granular A bedding material. The pre-cast box units should be placed on a minimum 75 millimetre thick levelling course consisting of uncompacted Granular A or fine aggregates as specified in MTO Special Provision (SP) 422S01.



6.2.3 Backfill and Cover

Backfill, cover and construction of the frost taper (backfill transition) for concrete box culverts should be completed in accordance with SP 422S01 and OPSD 803.010. The excavation for the culvert replacement should exceed the culvert dimensions by at least one metre on each side to promote good workmanship and effective compaction of the fill.

Based on the results of the boreholes, the existing fill is not suitable for backfill. Backfill should consist of free-draining, non-frost susceptible granular materials such as OPSS Granular A or Granular B Type II placed and compacted in accordance with SP105S21 but with less than 5 per cent passing the No. 200 sieve. All bedding, backfill and cover materials should be placed in accordance with SP105S21, OPSS 902 and SP422S01.

Heavy compaction equipment should not be used immediately adjacent to the walls and roof of the culvert. The height of backfill adjacent to the culvert walls should be maintained equal on both sides of the structure during all stages of backfill placement with one side not exceeding the other by more than 500 millimetres.

6.3 Retaining Walls/Wingwalls

The wingwalls for the culvert replacement could consist of reinforced concrete gravity or cantilever walls, concrete toe walls or reinforced soil system (RSS) walls. The concrete gravity wall could consist of pre-cast elements or cast-in-place (CIP). Pre-cast wingwalls are preferred since the culvert will also be pre-cast. The wingwalls will be approximately 2 metres long.

6.3.1 Wingwall Options

Reinforced Concrete Gravity and Cantilever Walls

Construction of reinforced concrete gravity or cantilever walls is geotechnically feasible. Compared to a concrete toe wall or RSS walls, footings for gravity and cantilever walls must be constructed with a frost cover of 1.6 metres. This may result in a longer foundation construction time compared to a concrete toe or RSS wall particularly if CIP walls are constructed. Groundwater control/dewatering will be required as footing excavations will be advanced below the groundwater level.

Concrete Toe Walls

A concrete toe wall is geotechnically feasible for use as a wingwall provided the wall has a maximum height of less than 1.8 metres above the final ground surface. Concrete toe walls 0.8 to 1.8 metres in height require a minimum embedment depth of 450 millimetres and should be constructed to bear on undisturbed soil having a minimum factored geotechnical resistance of 200 kilopascals. The embedment depth is defined as the distance from the underside of the toe wall foundation to the top of finished grade in front of the wall. The concrete toe



wall should be designed in accordance with the requirements for a Type II Concrete Toe Wall as shown on OPSD 3120.100. Compared to concrete gravity and cantilever walls, construction costs and time may be reduced if a concrete toe wall is used.

RSS Walls

The height of the wingwalls will be relatively low. Therefore, an RSS wall utilizing an interlocking block system and geogrid reinforcement is a geotechnically feasible alternative. Retained Soil System walls are proprietary systems which are to be designed by the supplier and constructed in accordance with their specifications. The internal stability of the mechanically-reinforced soil walls should be verified by the RSS supplier/designer. If an RSS block system wall is selected, the geotechnical aspects of the global stability of the detailed retaining wall design should be reviewed prior to construction. Depending on the design approach selected, an embedment depth equivalent to the frost depth may not be required for foundations of an RSS block system wall. This wall type can be constructed relatively quickly and inexpensively using small equipment.

6.3.2 Foundations – Wingwalls

Reinforced concrete gravity and cantilever walls and RSS walls founded on concrete strip footings must be provided with a frost cover of 1.6 metres below the adjacent ground or thermal equivalent. Assuming the adjacent ground is at the average culvert invert elevation of 492.7 metres, foundations for these wall types must be founded at or below elevation 491.1 metres in the hard clayey silt till or dense sandy silt till. A factored geotechnical resistance at ULS of 450 kilopascals and a geotechnical reaction at SLS of 300 kilopascals may be used for design. The SLS value corresponds to 25 millimetres of settlement. The wingwalls should be structurally separate from the box culvert to accommodate some differential settlement.

Concrete toe walls must be embedded a minimum depth of 450 millimetres beneath the adjacent ground. Assuming the adjacent ground is at the average culvert invert elevation of 492.7 metres, foundations for concrete toe walls must be founded at or below elevation 492.3 metres in the stiff to very stiff clayey silt till. A factored geotechnical resistance at ULS of 375 kilopascals and a geotechnical reaction at SLS of 250 kilopascals may be used for design. The SLS value corresponds to 25 millimetres of settlement. The pre-cast concrete toe wall units should be founded on a 75 millimetre thick levelling pad consisting of uncompacted Granular A.

The RSS walls may be designed such that the facing blocks are constructed on a levelling pad. The levelling pad should be constructed with Granular A to a minimum thickness of 300 millimetres. As noted previously, depending on the design selected by the RSS supplier, it may not be necessary to provide 1.6 metres of earth cover or thermal equivalent for frost protection. However the foundations must have adequate embedment to provide a stable structure. Typically the embedment depth, defined as the distance between the top of the levelling pad to the top of the adjoining finished grade, is a minimum of 500 millimetres. The geotechnical resistances recommended for concrete toe walls are applicable to RSS walls founded on a granular levelling pad.



6.3.3 Resistance to Lateral Forces

The lateral pressures acting on the wingwalls will depend on the backfill soils, the type and method of placement of the backfill materials behind the walls and the subsequent lateral movement of the structures. The resistance to lateral forces/sliding resistance between the compacted granular backfill (assumed to be Granular B Type II) of RSS block system walls or concrete footings for all other wall types and the subgrade soils should be calculated in accordance with Section 6.7.5 of the CHBDC. Each retaining wall shall be checked for overturning. Assuming that the founding soils are not loosened/disturbed during excavation and footing construction, the following angles of friction and corresponding unfactored coefficient of friction, $\tan \delta$, may be used for the interaction between the base of the wall and the founding soil:

Wall type	Interaction	Angle of Friction, δ (degrees)	Coefficient of Friction, $\tan \delta$
Reinforced Concrete Gravity or Cantilever Wall and RSS Block System Wall on concrete strip footings	Concrete footing on sandy silt till	31	0.60
	Concrete footing on clayey silt till	30	0.58
Concrete Toe Wall	Concrete footing on clayey silt till	30	0.58
RSS Block System Wall	Granular A levelling pad on clayey silt till	30	0.58

6.4 Liquefaction Potential and Seismic Analysis

6.4.1 Seismic Parameters

The site is located near the Community of Mount Forest in southwestern 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. 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.

The importance category of the replacement culvert is “other” based on the current version of the CHBDC. The corresponding seismic performance zones (SPZ) to this importance category is 1. Structural culverts situated in SPZ 1 need not be analyzed for seismic loads. However, design forces for restraining elements and support lengths must meet the minimum requirements as outlined in CHBDC Clause 4.4.5.1. It should be noted that the MTO views culverts with spans greater than 3 metres as being similar to bridges. The designer should ensure that the selected culvert design meets the seismic requirements for buried structures as outlined in Clause 7.5.5 of the CHBDC.



6.4.2 Seismic Hazard Assessment

A preliminary screening of the soil stratigraphy was conducted using the procedure outlined in the Federal Highway Administration recommended procedures⁷ and Canadian Foundation Engineering Manual (CFEM). The soils at this site are not considered to be susceptible to liquefaction or cyclic mobility. Therefore, a detailed evaluation of the liquefaction potential of the foundation soils is not considered warranted.

6.5 Lateral Earth Pressures for Design

Lateral pressures acting on the proposed replacement culvert and wingwalls will depend on the type and method of placement of the backfill materials, the nature of the soil behind the backfill, the magnitude of surcharge including construction loadings, the freedom of lateral movement of the structure and the drainage conditions behind the walls.

The following recommendations are made concerning the design of the walls in accordance with the current CHBDC. It should be noted that these design recommendations and parameters assume full removal of the existing poor quality fill and level backfill and ground surface behind the walls. Where there is sloping ground behind the walls, the coefficient of lateral earth pressure must be adjusted to account for the slope as described in this report.

- Select free-draining granular fill meeting the specifications of OPSS Granular A or Granular B Type II, but with less than 5 per cent passing the No. 200 sieve, should be used as backfill behind the culvert and wingwalls. This fill should be compacted in accordance with MTO's SP105S21. Longitudinal drains and weep holes should be installed within any cast-in-place concrete walls to provide positive drainage of the granular backfill. Other aspects of the granular backfill requirements with respect to subdrains and frost taper should be in accordance with OPSD 803.010, 3101.150, 3121.150 and 3190.100.
- 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 slope of 1 horizontal to 1 vertical extending up and back from the rear face of the foundation (Case (b) from Commentary on CHBDC Figure C6.20).
- A minimum compaction surcharge of 12 kilopascals should be included in the lateral earth pressures for the structural design of the culvert / wall stem, in accordance with CHBDC Figure C6.6. Compaction equipment should be used in accordance with SP105S21. Other surcharge loadings should be accounted for in the design, as required.

⁷ Federal Highway Administration (FHWA). (1997). "Design Guidance: Geotechnical Earthquake Engineering For Highways. Volume I – Design Principles." *Geotechnical Engineering Circular No. 3: FHWA-SA-97-076*, Washington, D.C.



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- For Case (a), the restrained case which is typical for box culvert walls, the pressures are based on the existing embankment fill materials and the following parameters (unfactored) may be used:

Soil unit weight:	20 kN/m ³
Coefficients of lateral earth pressure:	
Active, K_a	0.33
At rest, K_o	0.50
Passive, K_p	3.0

- For walls backfilled using granular materials in accordance with Case (b), the following parameters (unfactored) may be assumed:

	<u>GRANULAR A</u>	<u>GRANULAR B</u> (Type II)
Fill unit weight:	22 kN/m ³	21kN/m ³
Coefficients of static lateral earth pressure:		
'active' or unrestrained, K_a	0.27	0.27
'at rest' or restrained, K_o	0.43	0.43
'passive', K_p	3.7	3.7

- If the wall support allows lateral yielding (unrestrained structure), active earth pressures may be used in the geotechnical design of the structure. If the wall support does not allow lateral yielding (which is typically the case for a rigid concrete box culvert), at-rest earth pressures should be assumed for geotechnical design.

6.6 Construction Considerations

Care should be taken during construction to avoid disturbance of the subgrade prior to constructing foundations for the replacement culvert and wingwalls. All topsoil, organics and soft or loose soils should be removed from below the proposed founding elevation and wasted or reused as landscaping fill, as required. Subgrade preparation should be performed and monitored in accordance with OPSS 902.

The cleaned excavation base should be inspected by a geotechnical QVE and a working slab placed immediately after inspection to protect the founding materials. It is recommended that the footing excavation be carried out such that the final 0.5 metres of excavation is completed with the QVE on site with construction of the working slab commencing immediately after inspection. A Non Standard Special Provision (NSSP) should be added to the Contract Documents specifying protection of the founding soil through use of a working slab.

Erosion and scour protection for the culvert inlet and outlet should be provided, as appropriate. Consideration could be given to using suitable non-woven geotextile and rip rap, as required, to provide erosion protection based on hydraulic requirements. Rip-rap treatment at the culvert outlet should be provided in accordance with



OPSD 810.010. In addition, sediment control such as silt fences and erosion control blankets may be required during construction and diversion/piping of the watercourse to mitigate migration of fine soil particles.

MH has indicated that in addition to placing washed river-run stone along the culvert inlet, scour protection in the form of river-run stone pads 4.8 metres wide, 5 metres long and 0.45 metres thick will be constructed at the inlet and outlet. River-run stone will be used in place of rip-rap to address fisheries concerns. The use of river-run stone for scour protection is geotechnically feasible provided that the pad thickness is equivalent to 1.5 times the median stone diameter and the resulting velocities meet the hydraulic and fish passage requirements of the MTO Drainage Management Manual.

6.7 Excavations and Groundwater Control

Excavations will extend through the existing pavement structure, topsoil, fill and into the underlying clayey silt till, silt and sandy silt till. Contractors should also be prepared for the presence of cobbles and boulders within the fill and till strata.

It is anticipated that the excavations will not extend significantly below the inferred groundwater levels ranging from elevation 492.2 to 493.2 metres unless strip footings are constructed for CIP culvert or wingwalls. It is considered that groundwater can be controlled by pumping from properly constructed and filtered sumps located at the base of the excavations. A Permit To Take Water is not considered necessary at this time. Sumps should be maintained outside of the actual foundation limits.

Surficial water seepage into the excavations should be expected and will be heavier during periods of sustained precipitation. Surface water runoff should be directed away from the excavations at all times. The existing culvert flows will need to be diverted/piped during construction. The appropriate NSSP should be included in the contract documents to alert the contractor about the need for adequate control of surface and groundwater flows.

Temporary open cut slopes within the fill materials should be maintained no steeper than 1 horizontal to 1 vertical and localized sloughing and ground movements should be expected. All excavations should be carried out in accordance with the latest edition of the Ontario Occupational Health and Safety Act and Regulations for Construction Projects. The fill and any cohesionless materials below the groundwater level would be classified as Type 3 soils. The cohesionless materials above the groundwater level and glacial till would be classified as Type 2 soils.

6.8 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 culvert will need to be conducted in stages using a signalized single lane.



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Temporary support systems could consist of soldier piles and lagging or steel sheet piles. A soldier pile and lagging system is preferred for constructability reasons and for dealing with cobbles or boulders in the till. The temporary shoring may have a maximum height of 3 metres above the excavation base.

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.

Where the support to the wall is provided by anchors or rakers, the wall design should be based on a triangular earth pressure distribution using the design parameters given below. The raker/anchor support must be designed to accommodate the loads applied from pressures and surcharge pressures from area, line or point loads as well as the impact of sloping ground behind the system. Passive toe restraint to the soldier piles may be determined using a triangular pressure distribution acting over an equivalent width equal to three times the pile socket diameter.

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

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

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

K_a = active coefficient of earth pressure

γ = soil unit weight

q = surcharge for traffic and other loading

The support systems may be designed using the parameters provided in the table below. These parameters are provided to assist with design for the unfactored ultimate resistance and loading conditions and may not result in a temporary support design that adequately controls ground and structure displacements. Achieving adequate displacement control in accordance with the MTO performance criteria may require designs that result in a system that is stiffer than might otherwise be required based on the soil parameters provided in the table below.

Soil Type	Coefficient of Earth Pressure			Internal Angle of Friction (degrees)	Unit Weight (kN/m^3)
	Active, K_a	At Rest, K_o	Passive, K_p		
Fill	0.36	0.53	2.8	28	19
Clayey Silt	0.33	0.50	3.0	30	19
Clayey Silt Till	0.31	0.47	3.3	32	20
Silt/Sandy Silt	0.33	0.50	3.0	30	19
Sandy Silt Till	0.30	0.46	3.4	33	21



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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 fill materials and glacial till and the appropriate NSSP should be provided.



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7.0 MISCELLANEOUS

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

GOLDER ASSOCIATES LTD.

ORIGINAL SIGNED

Dirka U. Prout, P.Eng.
Project Engineer

ORIGINAL SIGNED

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

ORIGINAL SIGNED

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

TP/DUP/SJB/FJH/cr

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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 cm² 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 ¹
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement ¹
D_R	relative density (specific gravity, G_s)
DS	direct shear test
M	sieve analysis for particle size
MH	combined sieve and hydrometer (H) analysis
MPC	Modified Proctor compaction test
SPC	Standard Proctor compaction test
OC	organic content test
SO_4	concentration of water-soluble sulphates
UC	unconfined compression test
UU	unconsolidated undrained triaxial test
V	field vane (LV-laboratory vane test)
γ	unit weight

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

LIST OF SYMBOLS

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

I. General

π	3.1416
$\ln x$,	natural logarithm of x
\log_{10}	x or log x, logarithm of x to base 10
g	acceleration due to gravity
t	time
F	factor of safety
V	volume
W	weight

II. STRESS AND STRAIN

γ	shear strain
Δ	change in, e.g. in stress: $\Delta \sigma$
ϵ	linear strain
ϵ_v	volumetric strain
η	coefficient of viscosity
ν	poisson's ratio
σ	total stress
σ'	effective stress ($\sigma' = \sigma - u$)
σ'_{vo}	initial effective overburden stress
$\sigma_1, \sigma_2, \sigma_3$	principal stress (major, intermediate, minor)
σ_{oct}	mean stress or octahedral stress $= (\sigma_1 + \sigma_2 + \sigma_3)/3$
τ	shear stress
u	porewater pressure
E	modulus of deformation
G	shear modulus of deformation
K	bulk modulus of compressibility

III. SOIL PROPERTIES

(a) Index Properties

$\rho(\gamma)$	bulk density (bulk unit weight*)
$\rho_d(\gamma_d)$	dry density (dry unit weight)
$\rho_w(\gamma_w)$	density (unit weight) of water
$\rho_s(\gamma_s)$	density (unit weight) of solid particles
γ'	unit weight of submerged soil ($\gamma' = \gamma - \gamma_w$)
D_R	relative density (specific gravity) of solid particles ($D_R = \rho_s / \rho_w$) (formerly G_s)
e	void ratio
n	porosity
S	degree of saturation

(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
σ'_p	pre-consolidation pressure
OCR	over-consolidation ratio $= \sigma'_p / \sigma'_{vo}$

(d) Shear Strength

τ_p, τ_r	peak and residual shear strength
ϕ'	effective angle of internal friction
δ	angle of interface friction
μ	coefficient of friction $= \tan \delta$
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 ρ . Unit weight symbol is γ where $\gamma = \rho g$ (i.e. mass density x acceleration due to gravity)

RECORD OF BOREHOLE No 201

1 OF 1

METRIC

PROJECT 11-1132-0109
W.P. 3049-08-00 LOCATION N 4875796.4 ; E 226583.2 ORIGINATED BY MA
DIST HWY 89 BOREHOLE TYPE POWER AUGER, HOLLOW STEM COMPILED BY LMK
DATUM GEODETIC DATE September 4, 2012 CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	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														
493.59	GROUND SURFACE							20	40	60	80	100										
0.00	TOPSOIL, silty, trace sand, trace gravel Brown																					
493.16	CLAYEY SILT TILL, some sand, trace to some gravel, with sandy silt layers Stiff Brown, mottled																					
0.43			1	SS	12																	
492.07	SANDY SILT TILL, some gravel, some clay Compact to dense Grey and brown																					
1.52			2	SS	24																	
			3	SS	42																	
490.27	SILT, trace sand, with clayey silt layers Dense Grey																					
3.32			4	SS	32																	
489.17	SANDY SILT TILL, some clay, trace to some gravel Dense to very dense Grey																					
4.42			6	SS	37																	
				7	SS	100/ 250mm																
				8	SS	81																

RECORD OF BOREHOLE No 202

1 OF 1

METRIC

PROJECT 11-1132-0109
W.P. 3049-08-00 LOCATION N 4875826.7 ; E 226607.3 ORIGINATED BY MA
DIST HWY 89 BOREHOLE TYPE POWER AUGER, HOLLOW STEM COMPILED BY LMK
DATUM GEODETIC DATE September 4, 2012 CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	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	× LAB VANE	WATER CONTENT (%)					
493.53	GROUND SURFACE						20	40	60	80	100						
0.00	TOPSOIL, silty, trace sand, trace gravel Brown																
0.18	CLAYEY SILT TILL, trace to some sand, trace gravel Very stiff to hard Brown		1	SS	15												
			2	SS	26												
			3	SS	37												
490.79																	
2.74	SANDY SILT TILL, some gravel, trace clay Very dense Brown		4	SS	100/ 200mm												
490.02																	
3.51	SILT, trace sand, with clayey silt seams and layers Compact Grey		5	SS	28												
489.26																	
4.27	SANDY SILT TILL, trace to some gravel, trace clay Dense to very dense Grey		6	SS	34												
			7	SS	98												
			8	SS	100												
			9	SS	100/ 50mm												
485.76			10	SS	114												
7.77	END OF BOREHOLE																
	Groundwater encountered at about elev. 489.5m during drilling on September 4, 2012.																

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

RECORD OF BOREHOLE No 203

1 OF 1

METRIC

PROJECT 11-1132-0109

W.P. 3049-08-00

LOCATION N 4875815.4 ; E 226590.3

ORIGINATED BY MA

DIST HWY 89

BOREHOLE TYPE POWER AUGER, HOLLOW STEM

COMPILED BY LMK

DATUM GEODETIC

DATE September 5, 2012

CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _P	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa										WATER CONTENT (%)		
494.91	GROUND SURFACE						20	40	60	80	100									
0.00	FILL, crushed granular base, trace silt																			
0.16	Brown																			
0.27	FILL, silt, trace sand																			
0.52	Brown																			
494.00	FILL, sand and gravel, trace silt		1	SS	24															
0.91	Brown																			
	FILL, topsoil, silty																			
493.26	Black																			
1.65	FILL, sand and gravel, trace silt, trace topsoil		2	SS	8															
492.78	Compact Brown																			
2.13	FILL, clayey silt, trace sand, trace gravel, with topsoil layers		3	SS	15															
492.01	Firm Brown and grey																			
2.90	CLAYEY SILT TILL, some sand, trace gravel		4	SS	28															
	Stiff Brown and grey																			
	SANDY SILT TILL, some clay, trace to some gravel		5	SS	53															
490.49	Very dense to compact Brown																			
4.42	CLAYEY SILT TILL, some sand, trace gravel		6	SS	21															
489.73	Very stiff Brown																			
5.18	SILT, with clayey silt layers																			
489.42	Compact Brown		7	SS	28															
5.49	SANDY SILT TILL, some clay, trace to some gravel																			
	Compact to very dense Grey		8	SS	24															
			9	SS	67															
			10	SS	100/ 225mm															

RECORD OF BOREHOLE No 204

1 OF 1

METRIC

PROJECT 11-1132-0109
W.P. 3049-08-00 LOCATION N 4875809.8 ; E 226613.6 ORIGINATED BY MA
DIST HWY 89 BOREHOLE TYPE POWER AUGER, HOLLOW STEM COMPILED BY LMK
DATUM GEODETIC DATE August 23, 2012 CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	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 × LAB VANE									
								WATER CONTENT (%)									
494.76	GROUND SURFACE						20	40	60	80	100						
0.00	FILL, crushed granular base, trace silt																
0.24	Brown																
493.72	FILL, topsoil, silty																
1.04	Brown		1	SS	5												
493.39	FILL, sand, fine to medium, some gravel, trace silt, with cobbles																
1.37	Loose																
492.63	Brown																
	FILL, topsoil, silty, with silt, trace sand layers		2	SS	15												
	Loose																
	Brown																
2.13	CLAYEY SILT, some gravel, trace sand, with silt layers		3	SS	20											3 28 49 20	
	Stiff																
	Brown and grey mottled																
	SANDY SILT TILL, some clay, trace gravel, with cobbles		4	SS	48												
	Compact to very dense																
	Brown																
			5	SS	96												
490.34																	
4.42	SILT, trace sand, with clayey silt layers		6	SS	45											0 3 89 8	
	Dense																
	Brown																
489.03			7	SS	52												
5.73	SANDY SILT TILL, some gravel																
5.94	Very dense																
	Brown																
	END OF BOREHOLE																
	Borehole dry during drilling on August 23, 2012.																

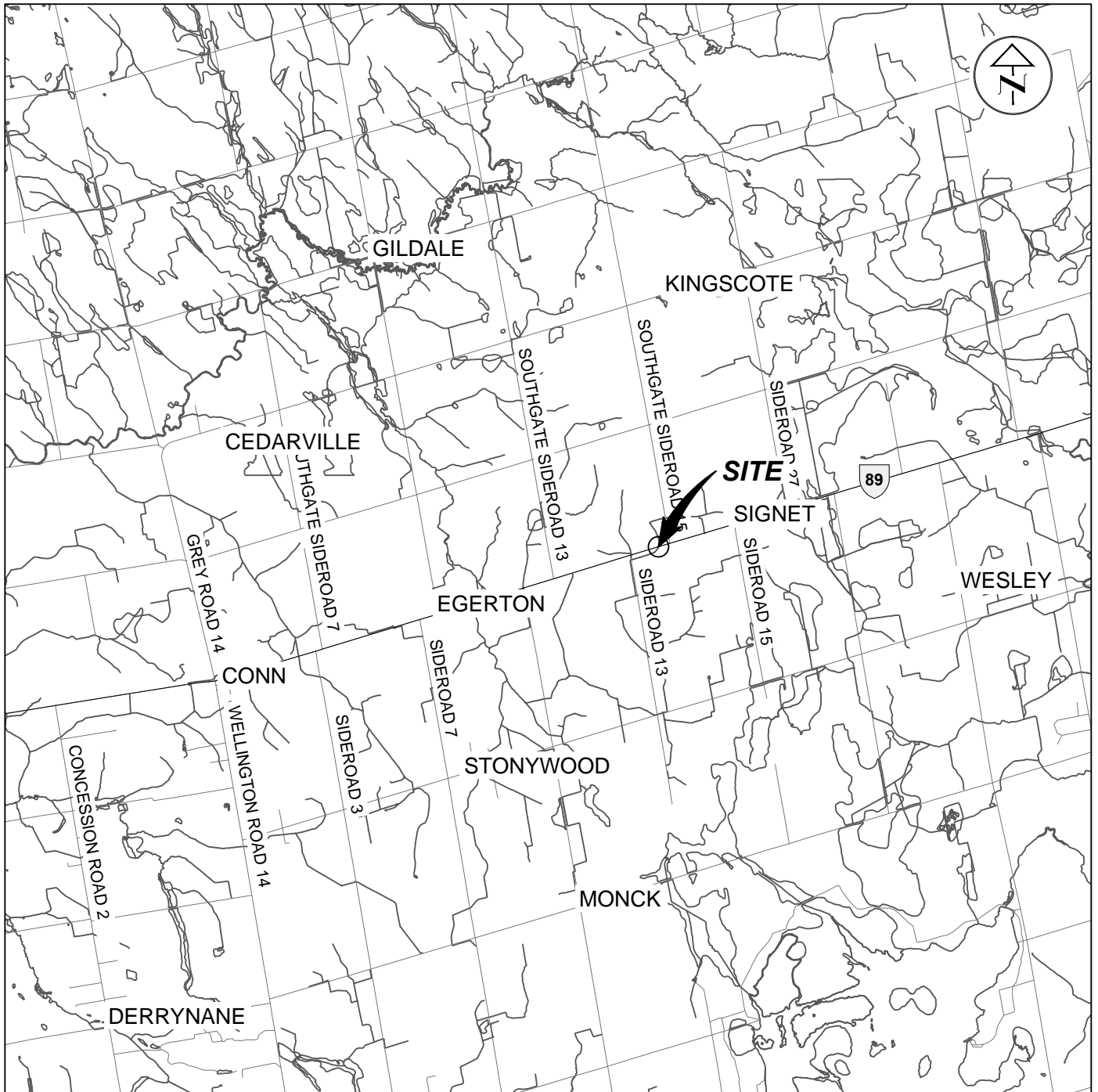
RECORD OF BOREHOLE No 205

1 OF 1

METRIC

PROJECT 11-1132-0109
W.P. 3049-08-00 LOCATION N 4875798.8 ; E 226575.2 ORIGINATED BY MA
DIST HWY 89 BOREHOLE TYPE POWER AUGER, HOLLOW STEM COMPILED BY LMK
DATUM GEODETIC DATE August 24, 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						
								20 40 60 80 100		W _p	W	W _L		
								○ UNCONFINED + FIELD VANE						
								● QUICK TRIAXIAL × LAB VANE						
494.81	GROUND SURFACE							20 40 60 80 100		10	20	30		GR SA SI CL
0.00	FILL, crushed granular base, trace silt													
494.47	Brown													
0.34	FILL, topsoil, silty, trace sand, trace gravel													
494.08	Brown		1	SS	6		494							
0.73	FILL, sand, fine to medium, some gravel, trace silt													
0.98	Loose Brown													
493.07	FILL, sandy silt, trace to some gravel, some clay, trace topsoil		2	SS	9		493							
1.74	Loose Brown													
492.68	CLAYEY SILT, trace sand, trace gravel		3	SS	24		492							
2.13	Stiff Brown and grey mottled													
491.91	CLAYEY SILT TILL, trace sand, trace gravel		4	SS	38		491							
2.90	Very stiff Brown													
	SANDY SILT TILL, some clay, trace to some gravel, with cobbles		5	SS	37		490			○			5	22 57 16
490.39	Dense Brown													
4.42	SILT, trace sand, trace clay, with clayey silt layers		6	SS	34		490				○		0	2 90 8
489.63	Dense Brown													
5.18	SANDY SILT TILL, some gravel, trace clay		7	SS	38		489							
489.17	Dense Brown													
5.64	SILT, with clayey silt layers													
488.87	Dense Brown													
5.94	END OF BOREHOLE													
	Groundwater encountered at about elev. 490.4m during drilling on August 24, 2012.													



REFERENCE

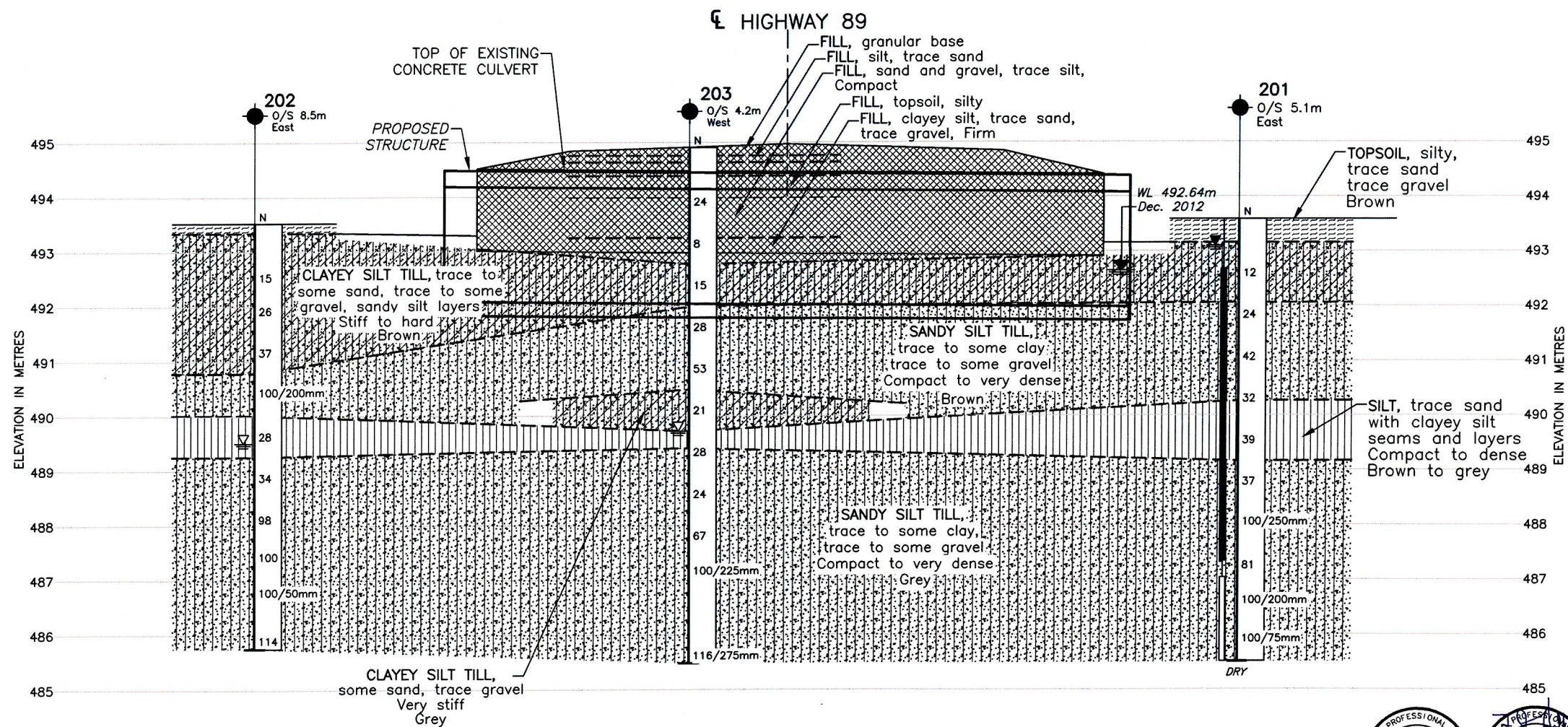
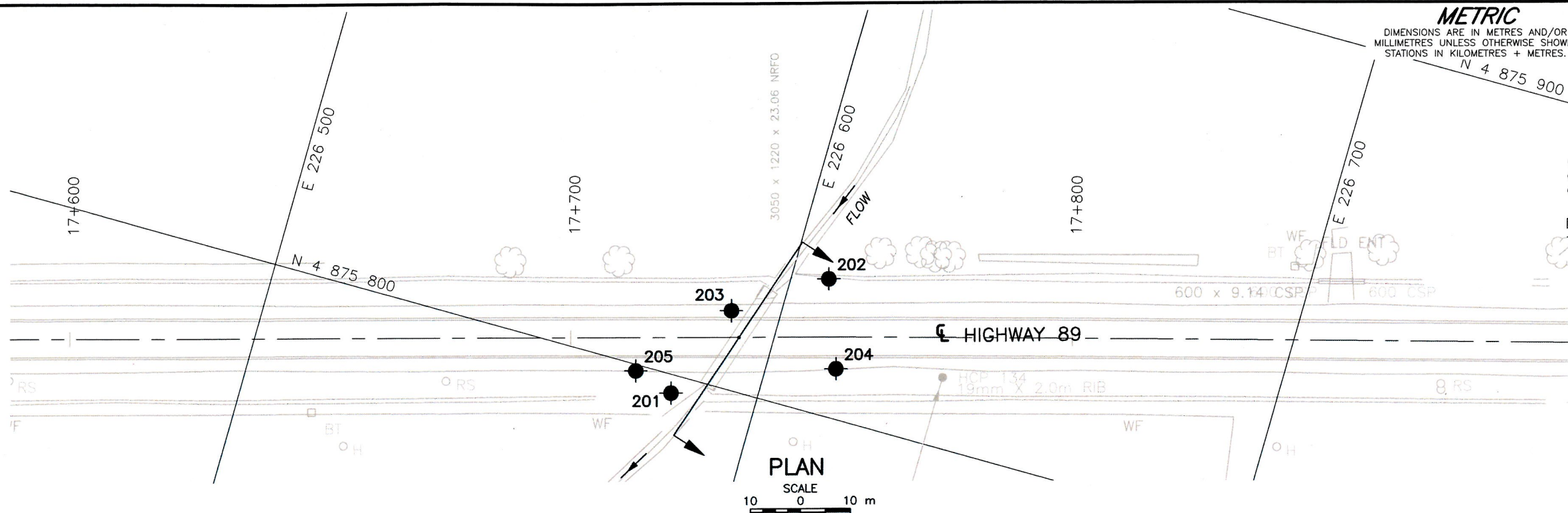
PLAN BASED ON CANMAP STREETFILES V.2008.5.

NOTE

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

PROJECT		CULVERT STATION 17+730, SITE 35-514/C HIGHWAY 89 STRUCTURE REPLACEMENTS GWP 3049-08-00	
TITLE			
KEY PLAN			
PROJECT No. 11-1132-0109		FILE No. 1111320109-1000-F04001	
CADD	LMKWDF	Dec. 29/12	SCALE AS SHOWN REV. 0
CHECK			FIGURE 1





PROFILE ALONG CL OF CULVERT

HORIZONTAL SCALE
2 0 2 m

VERTICAL SCALE
1 0 1 m

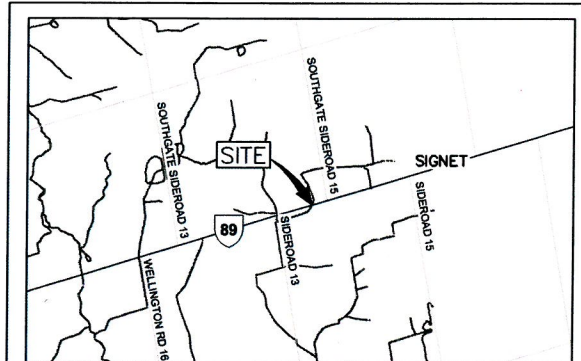
CONT No.
WP No. 3049-08-00

CULVERT, Station 17+730
HIGHWAY 89 STRUCTURE REPLACEMENTS

BOREHOLE LOCATIONS AND 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
- DRY Borehole dry during drilling

No.	ELEVATION	CO-ORDINATES (MTM ZONE 10)	
		NORTHING	EASTING
201	493.59	4 875 796.4	226 583.2
202	493.53	4 875 826.7	226 607.3
203	494.91	4 875 815.4	226 590.3
204	494.76	4 875 809.8	226 613.6
205	494.81	4 875 798.8	226 575.2

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.

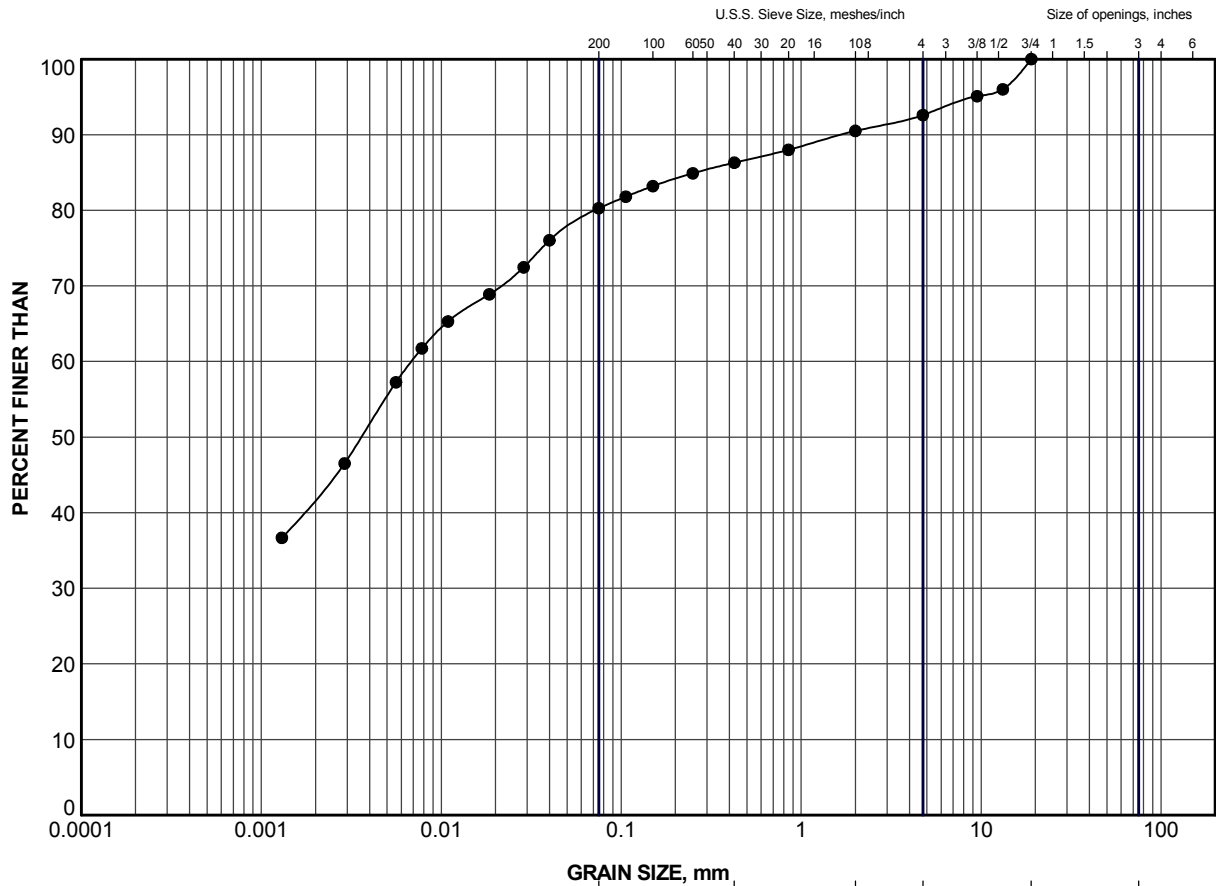


NO.	DATE	BY	REVISION
Geocres No.	41A-228		
HWY.	89	PROJECT NO.	11-1132-0109
SUBM'D. TP	CHKD. DUP	DATE	Dec. 29/12
DRAWN: LMK\WDF	CHKD. SUB	APPD. FJH	DWG. 1



APPENDIX A

Laboratory Test Data



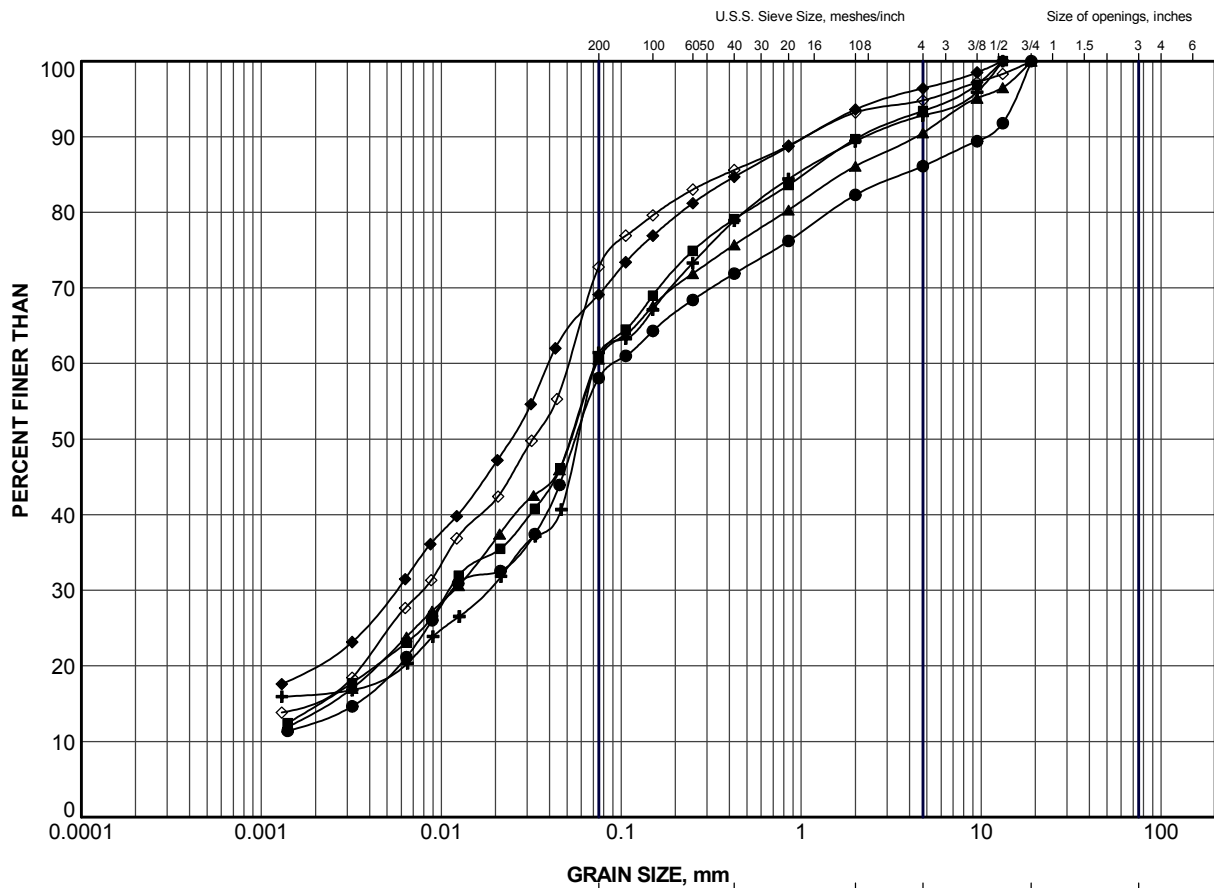
CLAY AND SILT	fine	medium	coarse	fine	coarse	Cobble Size
	SAND SIZE			GRAVEL SIZE		

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	202	3	491.2

PROJECT	CULVERT STATION 17+730, STATION 35-514/C HIGHWAY 89 STRUCTURE REPLACEMENTS GWP 3049-08-00		
TITLE	GRAIN SIZE DISTRIBUTION CLAYEY SILT TILL		
Golder Associates LONDON, ONTARIO	PROJECT No:11-1132-0109-1000		FILE No. 1111320109-1000-F040A1
	DRAWN	DCH	Nov 23/12
	CHECK		
	FIGURE A-1		

LDN_MTO_GSD_GLDR_LDN.GDT



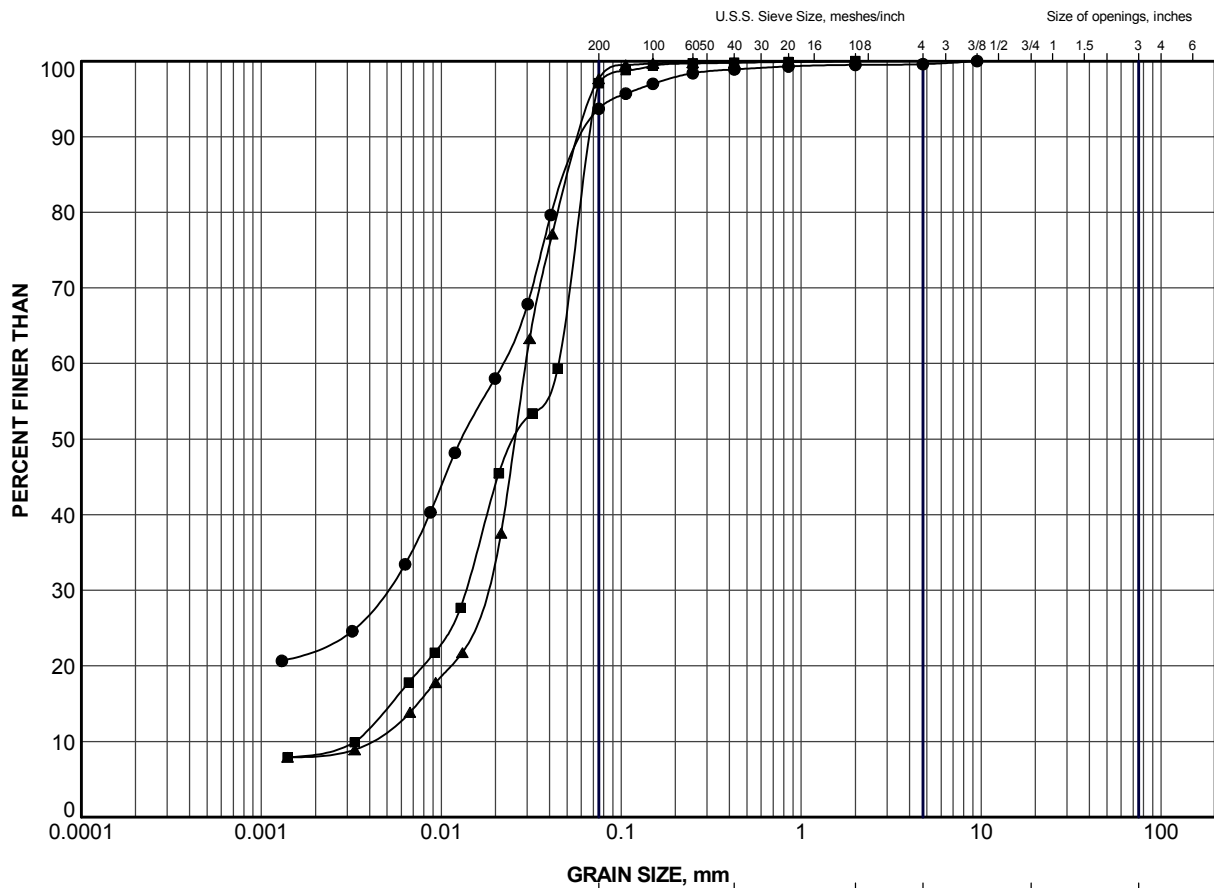
CLAY AND SILT	GRAVEL SIZE, mm						Cobble Size
	fine	medium	coarse	fine	coarse		
	SAND SIZE			GRAVEL SIZE			

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	201	2	491.8
■	201	6	488.8
▲	203	5	490.9
+	203	9	487.8
◆	204	3	492.2
◇	205	5	490.8

PROJECT				CULVERT STATION 17+730, STATION 35-514/C 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-F040A2			
DRAWN		DCH		Nov 23/12		SCALE N/A REV.	
CHECK						FIGURE A-2	




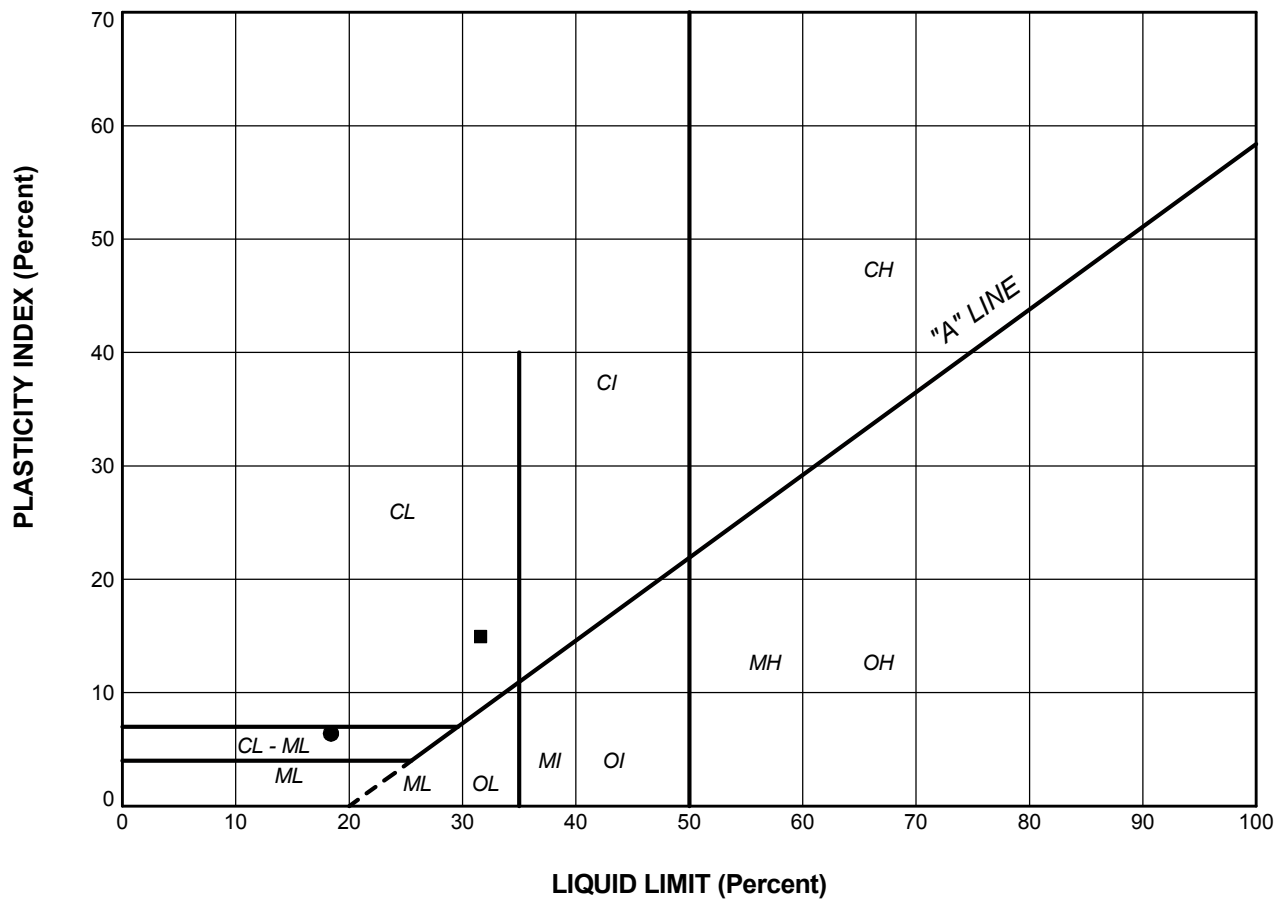


GRAIN SIZE, mm						
CLAY AND SILT	fine	medium	coarse	fine	coarse	Cobble Size
	SAND SIZE			GRAVEL SIZE		

LEGEND


SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	202	5	489.6
■	204	6	490.0
▲	205	6	490.0

PROJECT				CULVERT STATION 17+730, STATION 35-514/C HIGHWAY 89 STRUCTURE REPLACEMENTS GWP 3049-08-00			
TITLE				GRAIN SIZE DISTRIBUTION SILT			
PROJECT No:11-1132-0109-1000				FILE No. 1111320109-1000-F040A3			
DRAWN LMK Oct 15/12				SCALE N/A REV.			
CHECK				FIGURE A-3			
 Golder Associates LONDON, ONTARIO							



LEGEND

SYMBOL	BOREHOLE	SAMPLE	LL(%)	PL(%)	PI
●	201	2	18.4	12.0	6.4
■	202	3	31.6	16.7	15.0

PROJECT				CULVERT STATION 17+730, STATION 35-514/C HIGHWAY 89 STRUCTURE REPLACEMENTS GWP 3049-08-00			
TITLE							
PLASTICITY CHART							
PROJECT No.11-1132-0109-1000				FILE No. 1111320109-1000-F040A4			
DRAWN	LMK	Oct 15/12	SCALE		N/A	REV.	
CHECK							
 Golder Associates LONDON, ONTARIO				FIGURE A-4			



APPENDIX B

Site Photographs



APPENDIX B PHOTOGRAPHS



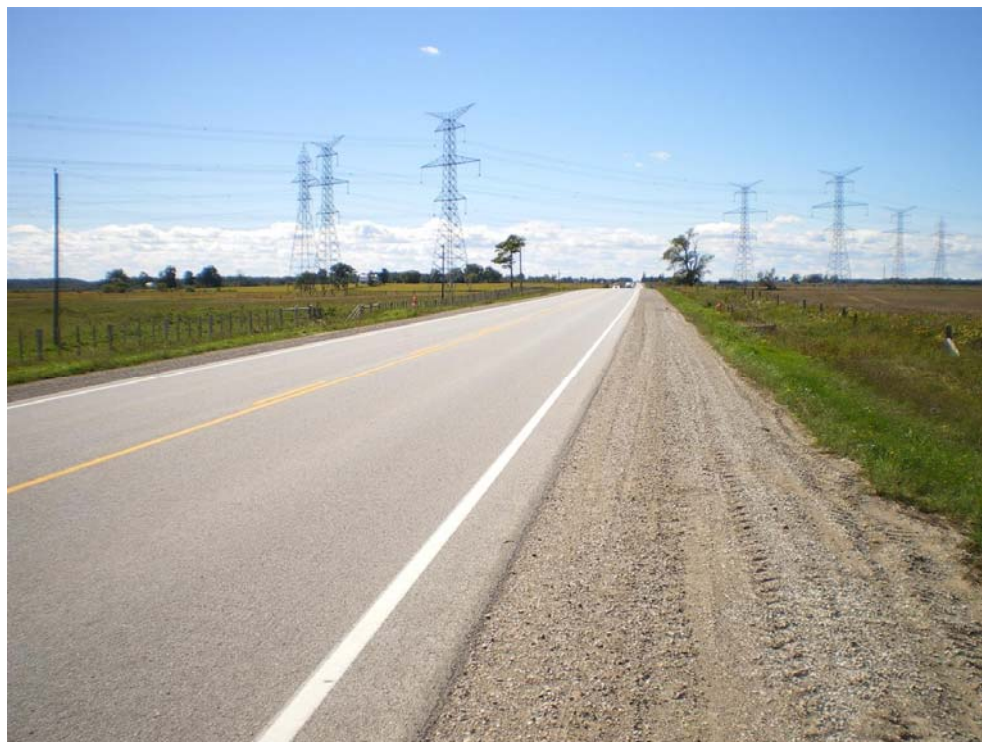
Photograph 1: North elevation (inlet) of Culvert Site 35-514/C.



Photograph 2: South elevation (outlet).



APPENDIX B PHOTOGRAPHS



Photograph 3: Highway 89 looking west from north shoulder towards Culvert Site 35-514/C.

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\vrpts\vr04\1111320109-1000-r04 jan 7 13 (final) app b - photos.docx

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Asia	+ 86 21 6258 5522
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Europe	+ 356 21 42 30 20
North America	+ 1 800 275 3281
South America	+ 55 21 3095 9500

solutions@golder.com
www.golder.com

Golder Associates Ltd.
309 Exeter Road, Unit #1
London, Ontario, N6L 1C1
Canada
T: +1 (519) 652 0099

