



January 2013

FOUNDATION INVESTIGATION AND DESIGN REPORT

**Replacement of Entrance Culvert, Site No. 35-593/C
Highway 89 Structure Replacements and Rehabilitation
From 6.0 km West of Mount Forest to Shelburne
GWP 3049-08-00
Ministry of Transportation, Ontario - West Region**

Submitted to:

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REPORT



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LIST OF SYMBOLS

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PART A

FOUNDATION INVESTIGATION REPORT

**REPLACEMENT OF ENTRANCE CULVERT, SITE NO. 35-593/C
HIGHWAY 89 STRUCTURE REPLACEMENT 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 entrance culvert at Station 23+150 (Site 35-593/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, 2008.

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



2.0 SITE DESCRIPTION

The culvert site 35-593/C is situated at Station 23+150 on the north side of Highway 89. The entrance culvert leads to the Misty Meadows Country Market, about 80 metres west of the intersection of County of Grey Road 14 and Wellington County Road 14. The location of the culvert site 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 culvert spans a drainage ditch which flows from east to west. The existing culvert is an open-footing non-rigid frame (NRFO structure) reported by the MTO as being constructed in 1943. It has a 3.05 metre span, a 1.54 metre high opening and is 3.50 metres long. There is a wooden railing connected to the culvert deck with metal posts. Site photographs are provided in Appendix B.

Adjacent land use is typically rural agricultural and residential with some commercial development at the intersection to the east. The village of Conn, Ontario is located east of the site. The local topography is relatively flat to gently undulating. Ground surface elevations in the vicinity of the subject culvert vary from 472 to 475 metres.

2.1 Site Geology

The project area is located on a drumlinized 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 the site is situated on the Elma Till sheet which comprise the Teeswater drumlin field and overlies the Mornington and Tavistock tills. The Elma Till is a stony sandy silt to silt till formed during a major re-advance of the Georgian Bay lobe during the Port Bruce Stadial.²

The geological mapping indicates that the underlying bedrock consists of cream and brown coloured, fine to medium, crystalline dolomite of the Guelph Formation.³ The mapped bedrock surface lies between the approximate elevations of 465 metres, west of the site, and 470 metres near the intersection of Highway 89 and County of Grey Road 14/Wellington County Road 14.⁴

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 Cowan, W.R., 1976. Quaternary Geology of the Palmerston Area, Southern Ontario; Ontario Division of Mines Preliminary Map P.1185, Geological Series, scale 1:50,000. Geology 1972, 1973.

³ Sanford, B.V., 1969. Geology Toronto-Windsor Area, Ontario, Geological Survey of Canada, Map 1263A, Scale 1:250,000.

⁴ Karrow, P.F., Davies, L.L., and McClymont, W.R. 1962. Bedrock Topography, Palmerston Sheet, Ontario Department of Mines, Preliminary Map P.166, Bedrock Topography Series, Scale 1:50,000.



3.0 INVESTIGATION PROCEDURES

The field work for the investigation was carried out on August 22 and 27, 2012, during which time three (3) boreholes were drilled at the locations shown on the Borehole Location Plan, Drawing 1. The table below summarizes the borehole locations, ground surface elevations at the borehole locations and borehole depths.

Borehole	Location (m)		Ground Surface Elevation (m)	Borehole Depth (m)
	Northing	Easting		
301	4 873 678.0	218 951.6	474.31	8.99
302	4 873 691.6	218 952.6	474.96	8.99
303	4 873 678.7	218 959.3	474.36	8.99

The investigation was carried out using track-mounted CME 45 drilling equipment supplied and operated by a specialist drilling contractor. Samples of the overburden were obtained in the boreholes at 0.76 metre intervals of depth using 50 millimetre outside diameter split spoon sampling equipment in accordance with the standard penetration test (SPT) procedures as described in ASTM D 1586. The recorded SPT N values are noted on the Record of Borehole sheets. According to ASTM D1586, the SPT resistance, or 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 samplers used in the investigations limit the maximum particle size that can be sampled and tested to about 38 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.

The boreholes were terminated 9.0 metres below the existing ground surface at the borehole locations. Groundwater conditions in the boreholes were observed throughout the drilling operations and a 50 millimetre diameter groundwater observation well was installed in borehole 303 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 an experienced member 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 labeled containers and transported to our London laboratory for further examination and testing. Index and classification tests, consisting of water content determinations, grain size distribution analyses and Atterberg limits determinations, 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.

In summary, the boreholes drilled at the site encountered topsoil overlying in sequence clayey silt, clayey silt till, and sandy silt glacial till.

The locations and elevations of the boreholes, together with the interpreted stratigraphic profile, are shown on the attached Drawing 1. It should be noted that the interpreted stratigraphic profiles have been simplified for clarity. 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

Brown silty topsoil was encountered at the ground surface of each of the boreholes. The topsoil was 150 to 520 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 Clayey Silt

Brown firm clayey silt was encountered underlying the topsoil in boreholes 301 and 303 below elevations 474.2 and 473.8 metres, respectively. The clayey silt was 1.2 and 0.9 metres thick in boreholes 301 and 303, respectively. Standard penetration testing N values of 4 blows per 0.3 metres were recorded. The water content of a sample of the brown clayey silt was 14 per cent.



4.2.3 Clayey Silt Till

Brown clayey silt glacial till was encountered beneath the clayey silt in boreholes 301 and 303, and the topsoil in borehole 302. In boreholes 301 and 303 the clayey silt till was encountered at approximate elevation 473.0 metres and was 0.8 metres thick. In borehole 302 the clayey silt till was encountered at elevation 474.7 metres and was 3.1 metres thick.

The clayey silt till is stiff to hard based on measured N values of 10 to over 100 blows per 0.3 metres, but N values were generally less than 21 blows per 0.3 metres. Water contents of 11 to 13 per cent were measured for selected samples. Based on two (2) Atterberg limits determinations carried out on samples obtained during standard penetration testing, the clayey silt till is of low plasticity. The plastic limits were 13 and 14 per cent, the liquid limits were 22 and 23 per cent, and the plasticity indices were 8 and 10 per cent. The Atterberg limits data for the clayey silt till are presented on Figure A-3. Grain size distribution curves for two samples of the clayey silt till are presented on Figure A-1.

Cobbles were encountered in the clayey silt till. Although boulders were not specifically encountered within the clayey silt till, the presence of both cobbles and boulders should be expected in this deposit.

4.2.4 Sandy Silt Till

Sandy silt till was encountered beneath the clayey silt till in each of the boreholes from elevations 472.2 to 471.6 metres. The boreholes were terminated in the sandy silt till after exploring some 5.6 to 6.9 metres.

Measured N values from the sandy silt till ranged between 18 and greater than 100 blows per 0.3 metres indicating a compact to very dense relative density. Water contents of 6 to 9 per cent were measured in selected samples. Grain size distribution curves for samples of the sandy silt till recovered from the standard penetration testing are presented on Figure A-2.

Cobbles were found in the sandy silt till. Although boulders were not specifically encountered in the boreholes, the presence of both cobbles and boulders should be anticipated throughout the sandy silt till deposit.

4.3 Groundwater Conditions

Groundwater conditions were observed during and on completion of drilling and sampling. Also, a groundwater observation well was installed in borehole 303 and groundwater levels were measured on October 1 and 24 and December 28, 2012. A summary of the encountered and measured groundwater levels is provided in the table below.



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Borehole	Ground Surface Elevation (m)	Encountered Groundwater Elevation (m)	Installation	Measured Groundwater Elevation (m)		
				Oct. 1, 2012	Oct. 24, 2012	Dec. 28, 2012
301	474.31	Dry to 465.3	-	-	-	-
302	474.96	Dry to 466.0	-	-	-	-
303	474.36	Dry to 465.4	Observation well	473.19	472.94	473.05

The water level in the drain was measured at elevation 473.0 and 473.26 metres on October 24 and December 28, 2012. 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 472.9 to 473.3 metres. Groundwater levels are expected to fluctuate seasonally and are expected to be higher during periods of sustained precipitation or during spring snow melt conditions and will be influenced by the water levels in the municipal drain.



5.0 MISCELLANEOUS

The investigation was carried out using equipment supplied and operated by Aardvark Drilling, 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. Nicole A. Gould, P.Eng. under the direction of the Project Engineer Ms. Dirka U. Prout, P.Eng. and Team Leader, Dr. Storer J. Boone, P.Eng. This report was reviewed by Mr. Fintan J. Heffernan, P.Eng., the Designated MTO Contact and Quality Control Auditor for this assignment.

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PART B

FOUNDATION DESIGN REPORT

**REPLACEMENT OF ENTRANCE CULVERT, SITE NO. 35-593/C
HIGHWAY 89 STRUCTURE REPLACEMENT 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 our recommendations on the foundation aspects of the design of the replacement entrance culvert at Site 35-593/C.

The recommendations are based on interpretation of the factual data obtained from the three boreholes advanced during the Foundation Investigation at this site. The interpretation and recommendations are intended to provide the designers with sufficient information to assess the feasible foundation alternatives and to design the foundations of the proposed culvert replacement. 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.

MH has indicated that several culvert types were considered during design but a concrete pre-cast box culvert was identified in the Structural Design Report to be the preferred option to advance for Detail Design. It has been proposed to replace the existing culvert with a new pre-cast 4.77 metre long box culvert with an internal opening 3.0 metres wide and 2.1 metres high. Substrate consisting of washed river-run stone is to be placed to a thickness of 300 to 600 millimetres along the invert. The new culvert will have wingwalls in each quadrant and will be erected 3.0 metres west of the existing culvert on a new entrance alignment. The design high water level in the drain is 474.47 metres.

6.2 Replacement Culvert

6.2.1 Foundation Design

The subsurface conditions encountered during the investigation generally consisted of the topsoil overlying firm clayey silt to about elevation 473 metres, then stiff to hard clayey silt till to about elevation 472 metres, then compact to very dense sandy silt till. The groundwater level is inferred to typically vary from about elevation 472.9 to 473.2 metres. The water level in the watercourse ranged from elevation 473.0 to 473.3 metres on October 24 and December 28, 2012, respectively.

The culvert replacement should be designed to withstand the appropriate weight of fill and traffic loading. It is not necessary to found the box culvert replacement at the standard depth for frost penetration protection purposes as pre-cast box culvert 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 assuming the design culvert invert elevations will be near those of the existing approximate invert elevation of 473.0 metres, the replacement box culvert may be founded on the stiff to very stiff clayey silt till at or below elevation



472.5 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 compacted Granular A or Granular B Type II fill.

Geotechnical Resistances

The stiff to very stiff clayey silt till is suitable for support of the proposed culvert replacement. A factored geotechnical resistance at Ultimate Limit States (ULS) of 300 kilopascals (kPa) and a geotechnical reaction at Serviceability Limit States (SLS) of 200 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 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	-	factored horizontal load, kilonewtons

Other Design Considerations

Considering the relatively low cover for the proposed replacement culverts and the presence of very dense soils below the founding elevation, the provision of camber for the replacement culvert is not required.

Water flowing beneath a culvert could potentially cause undermining and scouring. Seepage flowing around the culvert 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.

It is anticipated that water is not ponded at this culvert for an extended period of time at the 25-year design flows and that 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 material have been removed. Subexcavated material below the design subgrade elevation should be replaced with compacted OPSS Granular 'B', Type II fill material. It is recommended that the box culvert units be placed on a minimum thickness of compacted 300 millimetres of OPSS 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.

The 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 number 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 of the culvert. The height of backfill adjacent to the culvert walls should be maintained as equal as possible on both sides of the walls 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, reinforced soil system (RSS) walls, or gabion walls. The concrete gravity wall could consist of pre-cast elements or be cast-in-place (CIP), however, pre-cast wingwalls are preferred for compatibility with the pre-cast culvert. The wingwalls will be approximately 3.2 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 pre-cast concrete toe wall or RSS wall, particularly if CIP walls are erected. Groundwater control will be required since footing excavations will be advanced below the groundwater and drain water 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. 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 pre-cast concrete toe wall is used.

RSS Walls

The heights 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.

Gabion Walls

Construction of a gabion wall is geotechnically feasible at the site. Gabion walls require the least amount of space behind the wall. Gabion walls do not require an embedment depth equivalent to the frost depth provided it is founded on a granular pad of 300 millimetres compacted thickness, and the foundations have adequate embedment to provide a stable structure. Advantages of gabion walls compared to more rigid structures include the ability to accommodate differential settlements, dissipation of the energy of flowing water, and are free-draining provided an adequate filter is placed behind the wall. Gabion walls can be constructed relatively quickly with minimal equipment and materials. The life expectancy of a gabion wall can be extended by utilizing PVC-coated galvanized steel baskets. Gabion walls are to be constructed in accordance with OPSS 512 if greater than 2 metres in height or SP512S03 for structures less than 2 metres in height.

6.3.2 Foundations – Wingwalls

Cast-in-place reinforced concrete gravity and cantilever 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 473.0 metres, foundations for these wall types must be founded at or below elevation 471.4 metres in the compact to very dense sandy silt till. A factored geotechnical resistance at ULS of 475 kilopascals and a geotechnical reaction at SLS of 325 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.

Pre-cast 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 473.0 metres, foundations for concrete toe walls must be founded at or below elevation 472.5 metres in the stiff to hard clayey silt till. A factored geotechnical resistance at ULS of 300 kilopascals and a geotechnical reaction at SLS of 200 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 above for pre-cast concrete toe walls are applicable to RSS walls founded on a granular levelling pad.



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Gabion walls may be founded directly on a 300 millimetre thick compacted Granular 'A' pad placed on the stiff to hard clayey silt till at or below about elevation 472.5 metres. The geotechnical resistances recommended above for design of pre-cast concrete toe walls may be used for design of gabion walls. Alternatively, gabion baskets can be placed at the frost depth on the sandy silt till subgrade, near elevation 471.4 metres and the geotechnical resistances recommended for gravity walls as provided above. If required, a granular levelling course approximately 75 millimetres in thickness may be placed on the founding strata for gabion walls. Non-woven geotextile is to be placed between the gabions and the backfill in accordance with OPSS 512, OPSS 1860, and the manufacturer's specifications.

All wingwall foundations must be protected against scour as noted in the CHBDC Section 1.9.5.

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	Cast-in-place concrete footing on sandy silt till	34	0.67
Concrete Toe Wall	Pre-cast concrete footing on Granular A levelling pad	33	0.65
	Pre-cast concrete footing on sandy silt till	32	0.62
	Pre-cast concrete footing on clayey silt till	27	0.51
RSS Block System	Pre-cast concrete block facing units on Granular A levelling pad	33	0.65
Gabion Wall	Gabion cage on Granular A pad	35	0.70
	Gabion cage on sandy silt till	34	0.67
	Gabion cage on clayey silt till	30	0.58



6.4 Liquefaction Potential and Seismic Analysis

6.4.1 Seismic Parameters

The site is located in the Hamlet of Conn 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. 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.

The effects of site conditions on the response of the culvert are to be included in the determination of the seismic loads. The site stratigraphy generally consists of clayey and sandy silt till. The underlying bedrock is located approximately 5 to 11 metres below the ground surface in the vicinity of the site. 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.

6.4.2 Seismic Hazard Assessment

A preliminary screening of the soil stratigraphy was conducted using the procedure outlined by the Federal Highway Administration⁵ and the 2006 version of the Canadian Foundation Engineering Manual (CFEM).

The culvert and associated wingwalls will likely be founded on the stiff to very stiff clayey silt till or very dense sandy silt till. Although the sandy silt till deposit is saturated, the likelihood of these soils liquefying is very low. Considering the foregoing, the liquefaction potential is considered to be low. A detailed evaluation of the liquefaction potential of the foundation soils and the impact of liquefaction on the proposed culvert foundations is not considered warranted.

⁵ 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.



6.5 Lateral Earth Pressures for Design

The lateral pressures acting on the proposed culvert and associated wingwalls will depend on the type and method of placement of the backfill materials, on the nature of the soil behind the backfill, on the magnitude of surcharge including construction loadings, 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 walls in accordance with the current CHBDC. It should be noted that these design recommendations and parameters assume 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.

- 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 walls. Longitudinal drains and weep holes should be installed 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, 3121.50, and 3190.100.
- 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 Section 6.9.1 and Figure C6.6. Compaction equipment should be used in accordance with SP105S21. Other surcharge loadings should be accounted for in the design, as required.
- 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' or unrestrained, K_a	0.33
'at rest' or restrained, K_o	0.50
'passive', K_p	3.0

- If the wall support allows lateral yielding (unrestrained structure, such as typically the case for wingwalls), active earth pressures may be used in the geotechnical design of the structure. The granular fill should be placed in a wedged shaped zone with a width equal to at least 1.6 metres at the footing level against a cut slope which begins at the footing level and extends upwards at a maximum inclination of 1 horizontal to 1 vertical (case (b) from commentary on CHBDC Figure C6.20).
- If the wall support does not allow lateral yielding (such is typically the case for a rigid concrete box culvert), at-rest earth pressures should be assumed for geotechnical design. The granular fill should be placed in a zone with a width equal to at least 1.6 metres behind the culvert walls (case (a) from commentary on CHBDC Figure C6.6).



- 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> <u>(Type II)</u>
Fill unit weight:	22 kN/m ³	21 kN/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

6.6 Construction Considerations

6.6.1 General

Care should be taken during construction to avoid disturbance of the subgrade prior to constructing foundations for the replacement culvert and wingwalls. All existing fill and any topsoil, organics, and soft or loose soils should be stripped from the proposed founding area prior to placement of base materials. Subgrade preparation should be performed and monitored in accordance with OPSS 902.

It is recommended that the footing excavation be carried out such that the final 0.5 metres of excavation is completed with a geotechnical QVE on site. The cleaned excavation base should be inspected by the QVE and granular base materials or a working slab should be placed immediately after inspection to protect the founding materials. If a working slab is to be used, 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.

6.6.2 Erosion and Scour Protection

Erosion and scour protection for the culvert backfill and stream bank should be provided to protect the roadway, approach embankments and culvert, as appropriate. 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 against migration of fine particles. 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. 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 3.5 metres wide, 2 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 for the replacement of the culverts will encounter surficial topsoil, firm clayey silt, very stiff to hard clayey silt till and compact to very dense sandy silt till. Temporary open cut slopes within these materials should be maintained no steeper than 1 horizontal to 1 vertical and localized sloughing and ground movements should be expected. All debris, cobbles and boulders should be removed from slope surfaces. Where excavations extend below the groundwater level in granular soils, it may be necessary to use flatter slopes. 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. Any existing embankment fill materials that may be encountered and the saturated native granular soils at the site would be classified as Type 3 soils. The clayey silt and clayey silt till would be classified as Type 2 soils.

The excavation for the proposed structural culvert footings will extend below the inferred groundwater levels ranging from elevation 472.9 to 473.3 metres. Unwatering of excavations made into the saturated sandy silt till will be required in order to construct the footings in the dry. Groundwater at the site may be controlled by using properly filtered sumps at the base of the excavation. Sumps should be maintained outside the actual footing limits. The volume of seepage from the sandy silt till is expected to be low due to the low hydraulic conductivity. The estimated hydraulic conductivity of the sandy silt till based on empirical methods is about 2.3×10^{-6} centimetres per second. Based on a preliminary analysis, a Ministry of the Environment Permit to Take Water (PTTW) is not required.

The existing culvert flows will need to be diverted/piped during construction. Surficial water seepage into the excavation should be expected and will be heavier during periods of sustained precipitation. Surface water should be directed away from the excavation at all times. 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.

The exposed sandy silt glacial till will be sensitive to disturbance from construction activities once exposed to moisture. Therefore, it will be essential to redirect surface water flow away from the excavations and construct all granular levelling pads or cast-in-place concrete footings during the same day the excavation is completed to the planned subgrade elevation.



7.0 MISCELLANEOUS

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

GOLDER ASSOCIATES LTD.

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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 301

1 OF 1

METRIC

PROJECT 11-1132-0109
W.P. 3049-08-00 LOCATION N 4873678.0 ; E 218951.6 ORIGINATED BY MA
DIST HWY 89 BOREHOLE TYPE POWER AUGER, HOLLOW STEM COMPILED BY LMK
DATUM GEODETIC DATE August 22, 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												
474.31	GROUND SURFACE						20	40	60	80	100									
0.00	TOPSOIL, silty, trace gravel Brown																			
0.15	CLAYEY SILT, trace sand, trace gravel Firm Brown		1	SS	4									○						
472.94	CLAYEY SILT TILL, trace sand, trace gravel Stiff Brown		2	SS	10									○						
472.18	SANDY SILT TILL, some clay, trace to some gravel, with cobbles Compact to very dense Brown		3	SS	51									○						
2.13			4	SS	38									○						
			5	SS	21									○					5 38 41 16	
			6	SS	46									○						
			7	SS	22									○						
			8	SS	25									○					15 30 39 16	
			9	SS	100/ 150mm									○						
			10	SS	104									○						
			11	SS	66									○					22 33 33 12	
465.32	END OF BOREHOLE																			
8.99	Borehole dry during drilling on August 22, 2012.																			

RECORD OF BOREHOLE No 302

1 OF 1

METRIC

PROJECT 11-1132-0109

W.P. 3049-08-00

LOCATION N 4873691.6 ; E 218952.6

ORIGINATED BY MA

DIST HWY 89

BOREHOLE TYPE POWER AUGER, HOLLOW STEM

COMPILED BY LMK

DATUM GEODETIC

DATE August 27, 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	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
								○ UNCONFINED	+	FIELD VANE							
474.96	GROUND SURFACE						20	40	60	80	100						
0.00	TOPSOIL, silty Brown																
0.27	CLAYEY SILT TILL, some sand, trace gravel, with cobbles Very stiff to hard Brown		1	SS	15												
			2	SS	21												
			3	SS	18												
471.61			4	SS	100/ 25mm												
3.35	SANDY SILT TILL, trace to some gravel, some clay Compact to very dense Brown		5	SS	44												
			6	SS	55												
			7	SS	18												
			8	SS	34												
			9	SS	27												
			10	SS	64												
			11	SS	59												
465.97	END OF BOREHOLE																
8.99	Borehole dry during drilling on August 27, 2012.																

RECORD OF BOREHOLE No 303

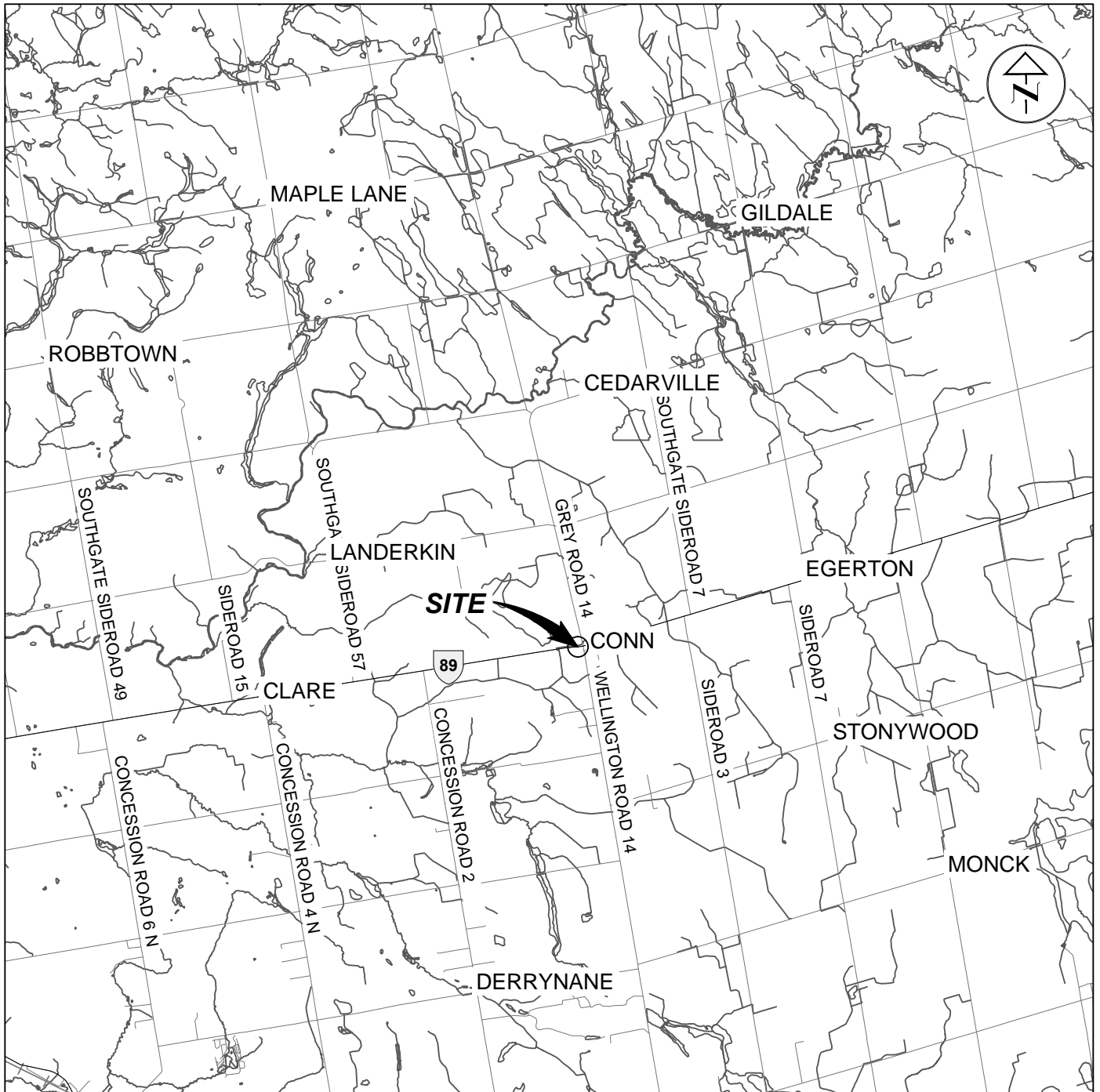
1 OF 1

METRIC

PROJECT 11-1132-0109
W.P. 3049-08-00 LOCATION N 4873678.7 : E 218959.3 ORIGINATED BY MA
DIST HWY 89 BOREHOLE TYPE POWER AUGER, HOLLOW STEM COMPILED BY LMK
DATUM GEODETIC DATE August 27, 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								
								20	40	60	80					
						○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE										
474.36	GROUND SURFACE															
0.00	TOPSOIL, silty, trace sand, trace gravel Brown						Concrete									
473.84																
0.52	CLAYEY SILT, trace sand, trace gravel Firm Brown		1	SS	4											
472.99																
1.37	CLAYEY SILT TILL, some sand, trace gravel Very stiff Brown		2	SS	16										9	16 46 29
472.23																
2.13	SANDY SILT TILL, some gravel, some clay, with cobbles Dense to very dense Brown		3	SS	49											
			4	SS	34		Granular Bentonite								12	35 38 15
			5	SS	76											
			6	SS	71											
			7	SS	78											
			8	SS	100/ 250mm											
			9	SS	62		Filter Sand								13	34 39 14
			10	SS	87		Well									
			11	SS	67											
465.37	END OF BOREHOLE															
8.99	Borehole dry during drilling on August 27, 2012. Water level measured at elev. 473.19m on October 1, 2012. Water level measured at elev. 472.94m on October 24, 2012. Water level measured at elev. 473.05m on December 28, 2012.															

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



REFERENCE

PLAN BASED ON CANMAP STREETFILES V.2008.5.

NOTE

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

PROJECT

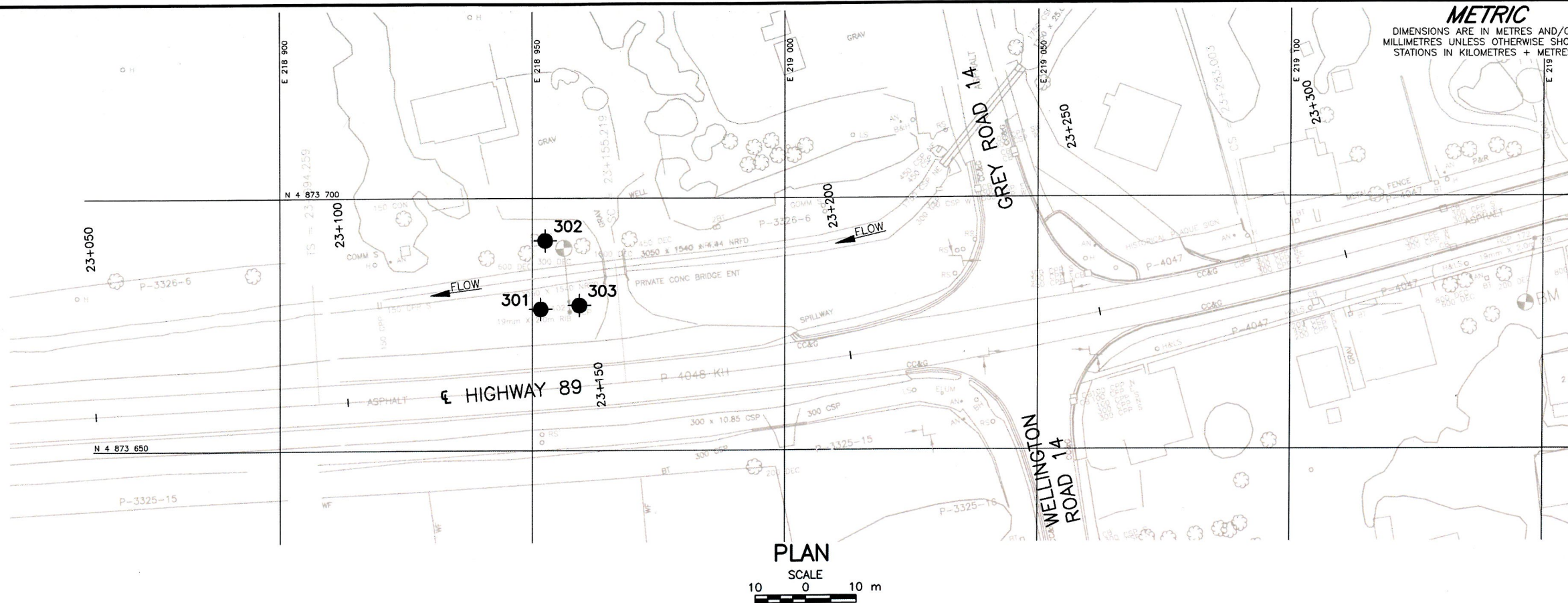
ENTRANCE CULVERT, SITE 35-593/C
HIGHWAY 89 STRUCTURE REPLACEMENTS
GWP 3049-08-00

TITLE

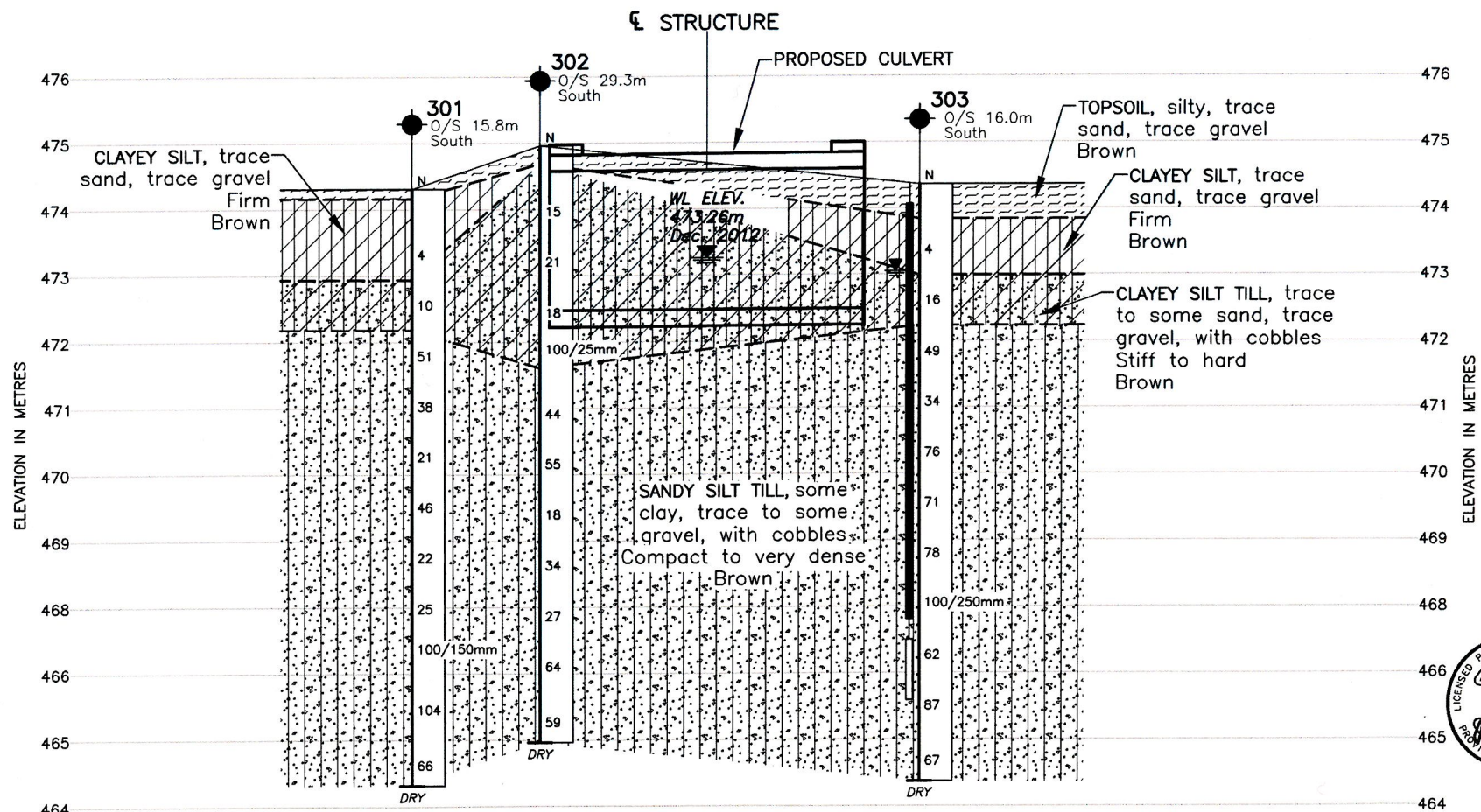
KEY PLAN



PROJECT No. 11-1132-0109			FILE No. 1111320109-1000-F02001	
CADD	LMK	Dec. 29/12	SCALE AS SHOWN	REV. 0
CHECK			FIGURE 1	



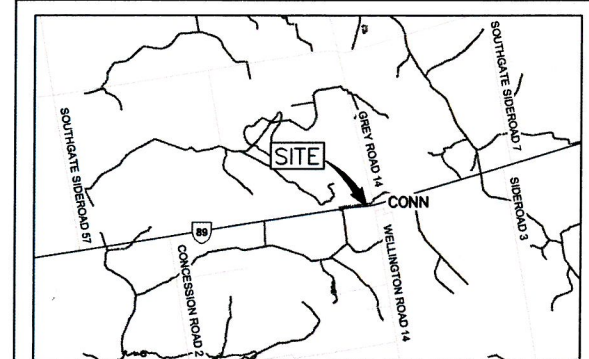
PLAN

SCALE
10 0 10 m

PROFILE ALONG CL OF HIGHWAY 89

SCALE
1 0 1 mCONT No.
WP No. 3049-08-00ENTRANCE CULVERT
HIGHWAY 89 STRUCTURE REPLACEMENTS
BOREHOLE LOCATIONS AND SOIL STRATA

SHEET

Golder Associates Ltd.
LONDON, ONTARIO, CANADA

KEY PLAN

SCALE IN KILOMETRES
0 1 2

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
- DRY Borehole dry during drilling

No.	ELEVATION	CO-ORDINATES (MTM ZONE 10)	
		NORTHING	EASTING
301	474.31	4 873 678.0	218 951.6
302	474.96	4 873 691.6	218 952.6
303	474.36	4 873 678.7	218 959.3

NOTES

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

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

REFERENCE

Base plans provided in digital format by Morrison Hershfield Limited.

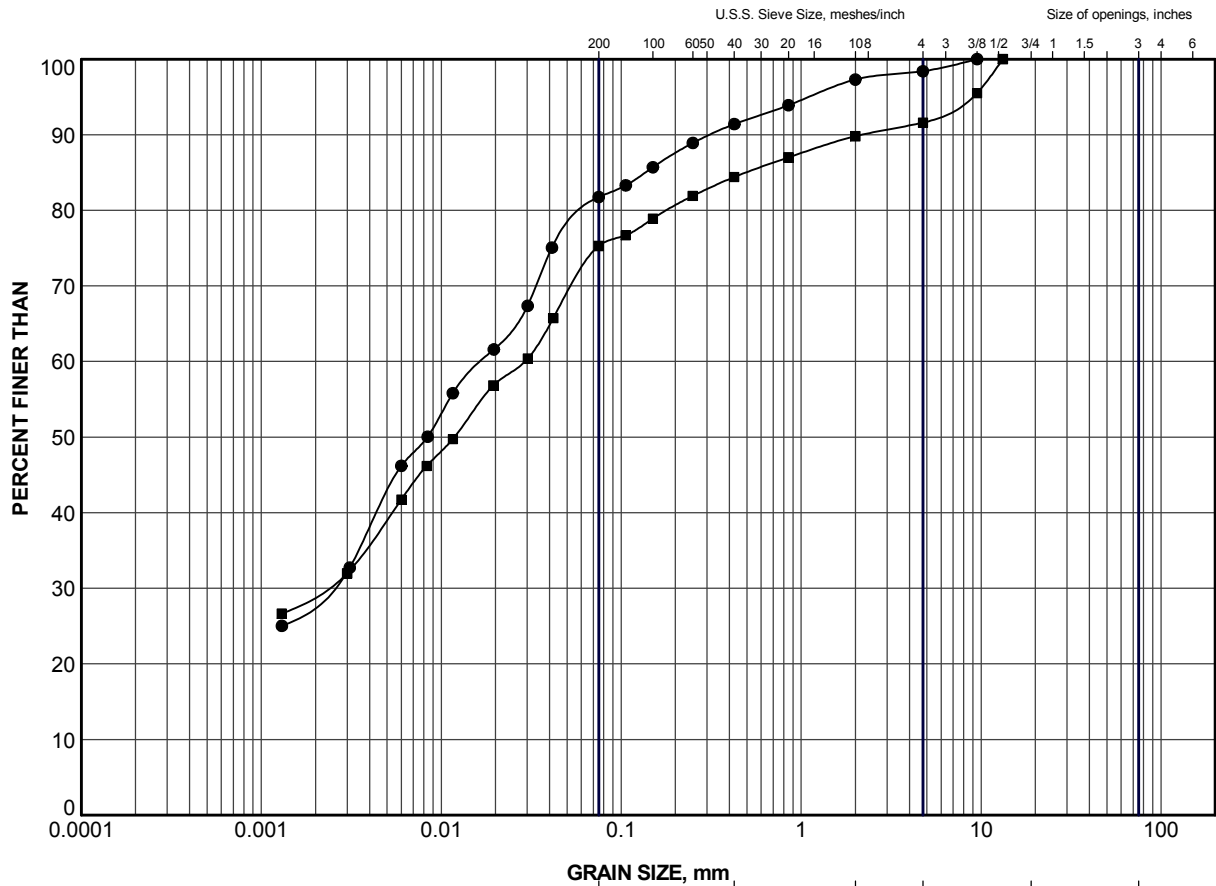


NO.	DATE	BY	REVISION
Geocres No.	40P15-45		
HWY.	89	PROJECT NO.	11-1132-0109
SUBM'D.	TP	CHKD.	DUP
DRAWN:	LMK\WDF	CHKD.	SJB
DATE:	Dec. 29/12	DATE:	Dec. 29/12
APPD.	F.J.H.	DATE:	Dec. 29/12
DWG.	1		




APPENDIX A

Laboratory Test Data

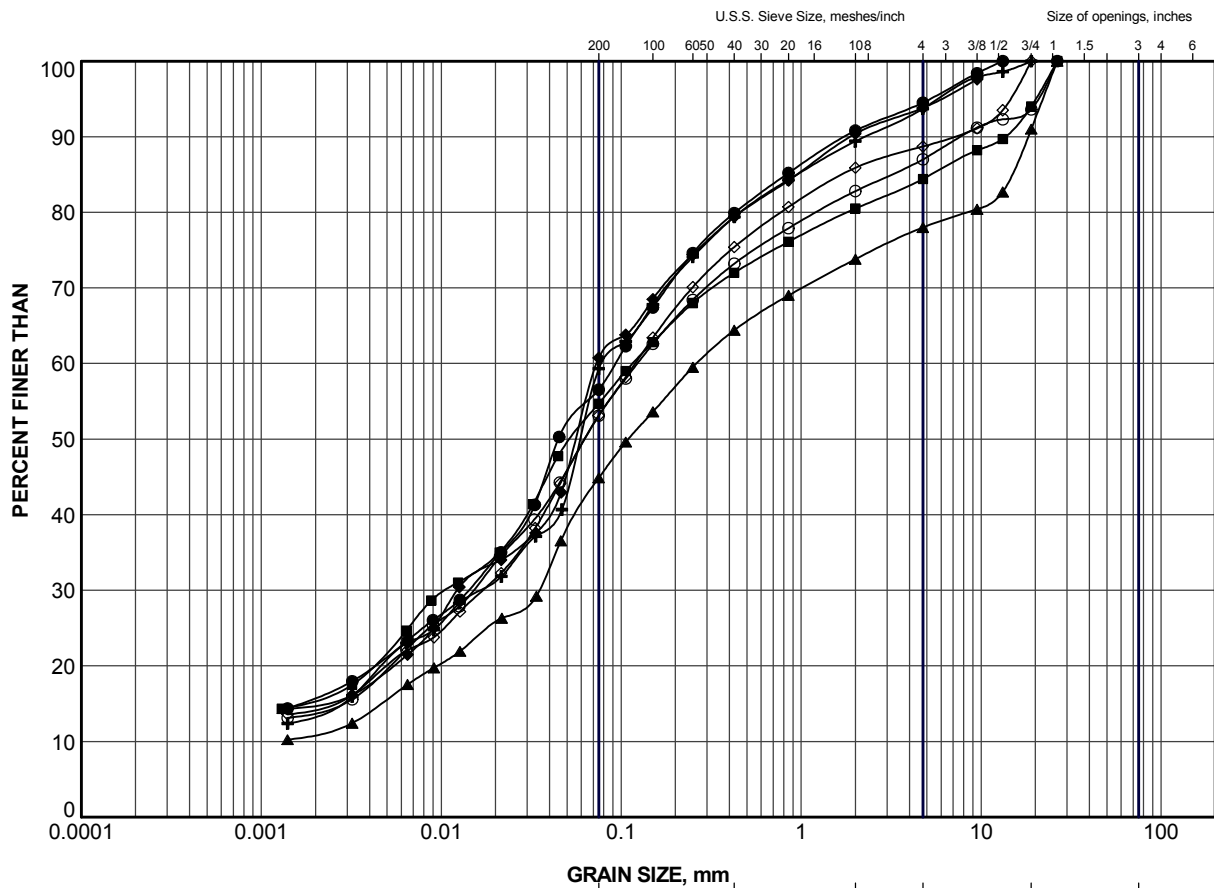


CLAY AND SILT	fine	medium	coarse	fine	coarse	Cobble Size
	SAND SIZE			GRAVEL SIZE		

LEGEND			
SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	302	2	473.2
■	303	2	472.6

PROJECT		ENTRANCE CULVERT, SITE 35-593/C HIGHWAY 89 STRUCTURE REPLACEMENTS GWP 3049-08-00		
TITLE		GRAIN SIZE DISTRIBUTION CLAYEY SILT TILL		
	PROJECT No:11-1132-0109-1000		FILE No. 1111320109-1000-F020A1	
	DRAWN	LMK	Nov 07/12	SCALE N/A REV.
	CHECK			
			FIGURE A-1	


LDN_MTO_GSD_GLDR_LDN.GDT

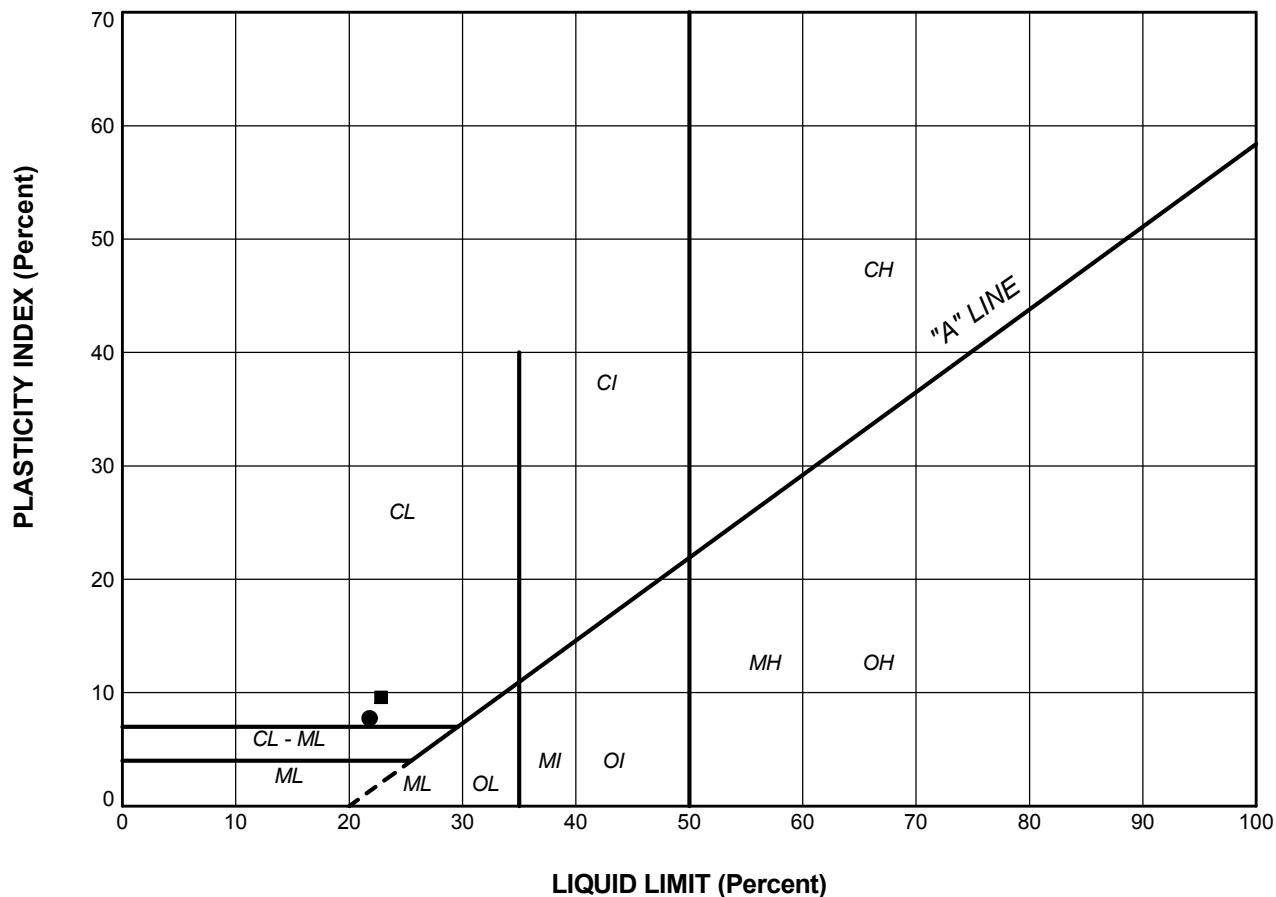


CLAY AND SILT	GRAVEL SIZE, mm					Cobble Size
	fine	medium	coarse	fine	coarse	
	SAND SIZE			GRAVEL SIZE		

LEGEND


SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	301	5	470.3
■	301	8	468.0
▲	301	11	465.6
+	302	5	470.9
◆	302	10	467.1
◇	303	4	471.1
○	303	9	467.3

PROJECT		ENTRANCE CULVERT, SITE 35-593/C HIGHWAY 89 STRUCTURE REPLACEMENTS GWP 3049-08-00			
TITLE		GRAIN SIZE DISTRIBUTION SANDY SILT TILL			
 Golder Associates LONDON, ONTARIO		PROJECT No:11-1132-0109-1000		FILE No. 1111320109-1000-F020A2	
		DRAWN	LMK	Nov 07/12	SCALE N/A REV.
		CHECK			
		FIGURE A-2			



LEGEND

SYMBOL	BOREHOLE	SAMPLE	LL(%)	PL(%)	PI
●	302	2	21.8	14.1	7.8
■	303	2	22.8	13.2	9.6

PROJECT				ENTRANCE CULVERT, SITE 35-593/C HIGHWAY 89 STRUCTURE REPLACEMENTS GWP 3049-08-00			
TITLE							
PLASTICITY CHART							
PROJECT No.11-1132-0109-1000				FILE No. 1111320109-1000-F020A3			
DRAWN	LMK	Nov 7/12	SCALE		N/A	REV.	
CHECK							
 Golder Associates LONDON, ONTARIO				FIGURE A-3			



APPENDIX B

Site Photographs



APPENDIX B PHOTOGRAPHS



Photograph 1: Culvert inlet (Photograph courtesy of Ministry of Transportation, Ontario).



Photograph 2: Culvert outlet (Photograph courtesy of Ministry of Transportation, Ontario).

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