October 2010

50 YEARS in 2010

REPORT ON

FOUNDATION INVESTIGATION AND DESIGN PROPOSED CULVERT REPLACEMENT TAYSIDE MUNICIPAL DRAIN CULVERT HIGHWAY 417 G.W.P. 4059-01-00

Submitted to:
D.M. Wills Associates Ltd.
452 Charlotte Street
Peterborough, Ontario
K9J 2W3

REPORT

Report Number: 09-1121-1004-3000

Geocres Number: 31G-239

Distribution:

1 copy - D.M. Wills Associates Ltd.

5 copies - Ministry of Transportation, Ontario, Kingston, Ontario

1 copy - MTO Foundations Section, Downsview, Ontario

2 copies - Golder Associates Ltd., Ottawa, Ontario

A world of capabilities delivered locally

Golder Associates

Table of Contents

TABLES

DRAWINGS

FIGURES

APPENDICES

APPENDIX A

PART A

FOUNDATION INVESTIGATION
PROPOSED CULVERT REPLACEMENT
TAYSIDE MUNICIPAL DRAIN CULVERT
HIGHWAY 417
G.W.P. 4059-01-00

1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by D.M. Wills Associates Ltd. on behalf of the Ministry of Transportation, Ontario (MTO) to carry out foundation investigations for two existing structures along Highway 17 and Highway 417 in Ontario.

Foundation investigation services are required on this project for the following components:

- Petawawa River Bridge structural rehabilitation and widening; and,
- Tayside Municipal Drain Culvert replacement.

This report addresses the replacement of the Tayside Municipal Drain Culvert on Highway 417 under G.W.P. 4059-01-00 and W.P. 4009-08-01.

The terms of reference for the original scope of work are outlined in the MTO's Request for Proposal (RFP) dated August 2009 and in Section 6.8 (Foundations Engineering) of the *Technical Proposal* for this assignment as well as Addendums 1 & 2 dated October 2, 2009.

The work was carried out in accordance with Golder's Quality Control Plan dated March 2010.

2.0 SITE DESCRIPTION

The existing Tayside Municipal Drain Culvert is located on Highway 417 at about Station 14+810 within the eastbound lanes and about Station 14+880 within the westbound lanes, approximately 2 km east of Highway 138 on Lot 7, Concession 10 of the Township of Roxborough in the County of Prescott and Russell, Ontario. The culvert is aligned approximately northeast-southwest with a 45 degree skew and carries the Tayside Municipal Drain under the westbound and eastbound lanes of Highway 417. The existing culvert is a corrugated steel pipe arch approximately 4370 mm wide by 2870 mm high and about 96 m in length. The culvert inverts are at about Elevations 62.7 m and 62.5 m, at the south and north ends, respectively. The existing embankment slopes at the south and north ends of the culvert are generally 3H:1V, however the upper portion of the embankment slope at the north end of the culvert is sloped at about 2H:1V. There is no evidence of inlet/outlet protection at the culvert ends however the embankment slopes are well vegetated. The flow in the culvert is from south to north. The water level within the culvert at the time of the investigation was between about 0.2 and 0.3 metres in depth. The existing pavement grades of the eastbound and westbound lanes above the culvert are at about Elevation 66.3 m and 66.5 m, respectively.

There are no signs of pavement distress within the eastbound and westbound lanes above the culvert. However, D.M. Willis Associates Ltd. observed during their culvert inspection that there are indications of structural distress of the culvert in the form of localized deformations at the crown of the pipe arch as well as severe corrosion along the base of the culvert below the water line.

3.0 INVESTIGATION PROCEDURES

The subsurface investigation was carried out for the culvert replacement in April 2010, at which time three boreholes (number 10-1 to 10-3, inclusive) were advanced at the locations shown on Drawing 1.

The boreholes were advanced using 108 mm inside diameter continuous flight hollow stem augers on a track-mounted drill rig, supplied and operated by Marathon Drilling Ltd. of Ottawa, Ontario. The boreholes were advanced to depths of between about 12.0 and 14.3 m below the existing ground surface.

Soil samples were obtained at intervals ranging from 0.75 m to 1.5 m of depth, using a 50 mm outer diameter split-spoon sampler in accordance with Standard Penetration Test (SPT) procedures. In-situ vane testing (using an N-size vane) was carried out within the cohesive deposits where possible. Six relatively undisturbed, 73 mm diameter thin-walled Shelby tube samples of the clay were retrieved using a fixed piston sampler.

A standpipe piezometer was installed in Borehole 10-1 to monitor the groundwater level at the site. The standpipe consists of a 50 mm diameter rigid PVC pipe with a 1.5 m long slotted screen section, installed within silica sand backfill and sealed by a 0.8 m long section of bentonite pellet backfill. The water level in the standpipe piezometer was measured on April 30 and May 11, 2010.

The boreholes were backfilled with bentonite pellets, mixed with native soils, and the site conditions restored following completion of work. The standpipe piezometer will be decommissioned following construction, unless instructed otherwise by the Ministry.

The field work was supervised throughout by a member of Golder's technical staff, who located the boreholes, supervised the drilling, sampling and in-situ testing operations, logged the boreholes, and examined and cared for the soil samples. The samples were identified in the field, placed in appropriate containers, labelled, and transported to Golder's laboratories in Ottawa and Mississauga for further examination. Index and classification tests consisting of grain size distribution, water content, and Atterberg limit testing were carried out on selected soil samples at the Ottawa laboratory. An oedometer (consolidation) test was carried out on one sample of the silty clay from Borehole 10-1. This testing was carried out at the Mississauga laboratory. All of the laboratory tests were carried out to MTO and/or ASTM standards as appropriate.

The borehole elevations and locations were surveyed by Golder Associates Ltd. using a Trimble R8 GPS unit. The borehole locations, including MTM NAD83 northing and easting coordinates and ground surface elevations referenced to Geodetic datum are summarized in the following table and are shown on Drawing 1.

Borehole No.	Borehole Location	MTM NAD83 Northing (m)	MTM NAD83 Easting (m)	Ground Surface Elevation (m)
10-1	North end of the culvert	5022159.0	190502.8	66.1
10-2	Centre of the culvert in the median	5022136.5	190485.5	66.0
10-3	South end of the culvert	5022096.8	190439.8	66.1

4.0 SITE GEOLOGY AND STRATIGRAPHY

4.1 Regional Geological Conditions

The study area for this assignment is within the *Winchester Clay Plain*, as delineated in *The Physiography of Southern Ontario*[1] that lies within the major physiographic region of the Ottawa-St. Lawrence Lowland.

The Winchester Clay Plain lies between the Glengarry Till Plain and the sand plains of the United Counties of Prescott and Russell and composes an area of 580 square kilometres. It is a flat lying area located almost entirely within the drainage basin of the South Nation River.[1] The Winchester Clay Plain is characterized by relatively thick deposits of sensitive marine clay, silt and silty clay that overlie relatively thin, commonly reworked glacial till and glacial fluvial deposits that in turn overlie bedrock. This region is underlain by sedimentary rock, consisting of limestone interbedded with shale.

4.2 Site Stratigraphy

As part of the subsurface investigation at this site, three boreholes were advanced along the alignment of the existing Tayside Municipal Drain Culvert. The borehole locations, ground surface elevations and an interpreted stratigraphic profile are shown on Drawing 1.

The detailed subsurface soil, bedrock and groundwater conditions as encountered in the boreholes advanced during this investigation for the culvert, together with the results of the laboratory tests carried out on selected soil samples, are given on the attached Record of Borehole sheets and on Figures 1 to 7.

The stratigraphic boundaries shown on the Record of Borehole sheets are inferred from non-continuous sampling and, therefore, represent transitions between soil types rather than exact planes of geological change. The subsoil conditions will vary between and beyond the borehole locations.

In general, the subsurface conditions at the culvert consist of up to 4.1 metres of fill, underlain by a deposit of sensitive clay which extends to depths ranging from about 10.8 m to 11.9 m below the ground surface. The upper 1.6 m of the clay deposit in the median has locally been weathered to a grey brown crust. The clay beneath the fill material at the culvert ends and beneath the weathered portion in the median is unweathered and grey in colour and has a soft to stiff consistency. The silty clay deposit is underlain by silty sand and gravel till which was proven to depths of up to 14.3 m relative to the ground surface level (i.e., Elevation 51.8 m).

A more detailed description of the subsurface conditions encountered in the boreholes is provided in the following sections.

4.2.1 Embankment Fill

Embankment fill was encountered at ground surface at Boreholes 10-1, 10-2 and 10-3, with thicknesses of about 4.1 m (to Elevation 62.0 m), 1.5 m (to Elevation 64.5 m), and 3.4 m (to Elevation 62.7 m), respectively. The embankment fill consists of crushed stone overlying sand and gravel. The fill in the median consists of topsoil, overlying silty clay and sand and gravel.

---

[1] Chapman, L.J. and D.F. Putnam. *The Physiography of Southern Ontario*, Ontario Geological Survey Special Volume 2, Third Edition, 1984. Accompanied by Map P.2715, Scale 1:600,000.

Standard penetration test (SPT) "N" values in the granular fill typically ranged from 4 to 13 blows per 0.3 m of penetration, indicating a loose to compact state of packing. The results of grain size distribution testing on one sample of the sand and gravel fill are shown on Figure 1.

4.2.2 Sensitive Clay

The fill is underlain by a deposit of clay. The deposit was fully penetrated at depths ranging between about 10.8 m and 11.9 m below the ground/embankment surface level, at about Elevations 54.2 m to 55.2 m.

At Borehole 10-2 in the median, the upper 1.6 m of the clay has been weathered to a grey brown crust. Two measured SPT "N" values in this material were 6 and 4 blows per 0.3 m of penetration, indicating a very stiff to stiff consistency of the weathered crust.

The results of grain size distribution testing on one sample of the weathered clay are shown on Figure 2. The results of Atterberg limit testing on one sample of the weathered material indicate a plasticity index value of 35 percent and a liquid limit value of 62 percent, as shown on Figure 3, indicating a clay soil of high plasticity. Two measured natural water contents of the weathered material were 34 to 44 percent.

The clay below the depth of weathering at Borehole 10-2 and below the embankment fill at Boreholes 10-1 and 10-3 is grey in colour and about 8 metres in thickness.

The results of in situ vane testing carried out in this material indicate undrained shear strengths which range from 17 to 52 kPa indicating a soft to firm consistency.

The results of grain size distribution testing on one sample of the unweathered clay are shown on Figure 4. The results of Atterberg limit testing on samples of the unweathered clay indicate plasticity index values which range from 28 to 51 percent and liquid limit values that range from 53 to 80 percent, as shown on Figure 5, indicating a clay of high plasticity. The measured natural water content of the unweathered material ranges from 52 to 86 percent. These natural water contents are generally near or above the measured liquid limits.

Oedometer consolidation testing was carried out on one sample of the unweathered grey clay from Borehole 10-1 at Elevation 60.5 m. The results of that testing, which are provided on Figure 6 are summarized in the table below and indicate that this material is normally consolidated, with a preconsolidation pressure of 75 kPa and overconsolidation ratio of 1.0.

Borehole/ SAMPLE NUMBER	Sample Depth/Elev. (m)	Unit Weight (kN/m^3)	σ_P' (kP)	σ_{VO}' (kP)	$\sigma_P' - \sigma_{vo}'$ (kPa)	Cc	Cr	e_o	OCR
10-1 / 6	5.6 / 60.5	15.1	75	75	0	1.58	0.060	2.29	1.0

Notes:

σ_P' - Apparent preconsolidation pressure
σ_{VO}' - Computed existing vertical effective stress
Cc - Compression index
Cr - Recompression index
e_o - Initial void ratio
OCR - Overconsolidation ratio

4.2.3 Silty Sand and Gravel Till

The clay is underlain by glacial till. The surface of the glacial till ranges from about Elevation 54.2 to 55.2 m. The glacial till was proven to depths of about 12.3 m, 12.0 m, and 14.3 m, at Boreholes 10-1, 10-2 and 10-3, respectively. The till is at least 0.4 to 3.0 m thick at these locations.

The glacial till is considered to be a heterogeneous mixture of gravel, cobbles, and boulders in a matrix of sand and silt with trace clay. However, those samples were retrieved using a 50 mm external diameter sampler and therefore the test results do not reflect the cobble and boulder portions of the deposit.

SPT "N" values of between 9 and 26 blows per 0.3 metres of penetration in the glacial till indicate a loose to compact state of packing.

The results of grain size distribution testing on samples of the glacial till are shown on Figure 7. The measured natural water content of the glacial till ranges from 7 to 9 percent.

4.2.4 Groundwater Conditions

The groundwater level in the piezometer in Borehole 10-1 was measured on April 30 and May 11, 2010. The groundwater levels in the piezometer are summarized in the table below:

Borehole	Ground Surface Elevation (m)	Water Level Depth (m)	Water Level Elevation (m)	Date
10-1	66.1	2.5	63.6	April 30, 2010
		2.6	63.5	May 11, 2010

It should be noted that groundwater levels in the area are subject to fluctuations both seasonally and with precipitation events.

5.0 CLOSURE

This report was prepared by Ms. Susan Trickey, P.Eng. and reviewed by Mr. Fintan Heffernan P.Eng., the designated MTO contact for this project.

Yours truly,

GOLDER ASSOCIATES LTD.

LICENSED PROFESSIONAL ENGINEER
S. A. TRICKEY
100104579
PROVINCE OF ONTARIO

Susan Trickey, P.Eng.
Geotechnical Engineer

Fintan Heffernan, P.Eng.
Designated MTO Contact

REGISTERED PROFESSIONAL ENGINEER
F. J. HEFFERNAN
PROVINCE OF ONTARIO

SAT/WC/FJH/am

n:\active\2009\1121 - geotechnical\09-1121-1004 dm wills petawawa bridge\foundations\tayside culvert\09-1121-1004-3000 rpt-001 october2010.docx

PART B

FOUNDATION DESIGN
PROPOSED CULVERT REPLACEMENT
TAYSIDE MUNICIPAL DRAIN CULVERT
HIGHWAY 417
G.W.P. 4059-01-00

6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

6.1 General

This section of the report provides foundation design recommendations for the proposed replacement of the Tayside Municipal Drain Culvert on Highway 417. The recommendations are based on interpretation of the factual data obtained from the boreholes advanced during the subsurface investigation at this site. The interpretation and recommendations provided are intended only to provide the designers with sufficient information to assess the feasible foundation alternatives and to design the proposed structure 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, scheduling and the like.

The existing culvert consists of a 4370 mm wide by 2870 mm high corrugated steel pipe arch which is approximately 96 m in length.

The replacement of the culvert will be along the existing alignment as shown on Drawing 1. The proposed invert levels will vary depending on the type of replacement being considered. However, it is understood that the existing grades of the Highway 417 eastbound and westbound lanes will remain the same.

It is also understood that the preferred culvert replacement methods and types that are being considered are as follows:

- A shored excavation with a pre-cast concrete box culvert using 'cut and cover' construction methods;
- A shored excavation with an aluminum pipe arch using 'cut and cover' construction methods; or,
- Relining the existing culvert using light weight grout.

However, other foundation types are also considered in the following section and a comparison of the foundation alternatives is provided in Table 1.

6.2 Culvert Foundations

The subsurface conditions at this site generally consist of up to about 4 metres of fill, underlain by sensitive clay, followed by silty sand and gravel till at depth.

A significant concern for design of the replacement culvert is compression and settlement of the underlying clay soil and the impacts that those settlements could have on the performance of the culvert. The clay soils at this site have very limited capacity to accept any increase in load. As discussed below in Section 6.2.1.2, it is considered generally feasible to support the culvert on or within the native clay subgrade but it is important to limit the magnitude of the foundation stresses since higher stresses will result in increased magnitudes of settlement.

A pre-cast concrete closed box culvert is considered to be feasible for this site since the foundation loads are distributed over a larger area, resulting in lower foundation stress levels, and therefore reduced settlement magnitudes. A pre-cast culvert is preferred over a cast-in-place culvert because it will be more accommodating to the expected settlements at this site and take less time to install.

A pipe arch culvert may also be considered for this location and is considered to be feasible. However, the potential exists for significant deformation of the culvert itself over time due to the high loadings at the culvert haunches required to maintain the culvert's shape. The preparation and placement of the culvert bedding and backfill, if not carried out to a high standard of care, could also result in deformation and potentially failure of the pipe arch culvert.

The settlements for a rigid frame open box culvert would be larger than those for a closed box culvert due to the higher concentration of foundation stresses and is therefore not considered suitable for this site.

Similarly, relining the culvert with conventional grout would add additional loading (i.e., the weight of the liner as well as the grout) on the underlying clay deposits resulting in larger than tolerable settlements. Relining the culvert with conventional grout is therefore also not considered to be a suitable option at this site. However, it is considered feasible to reline the culvert using a light weight grout, provided the additional loading applied on the underlying clay deposit is minimized (i.e., to limit the resulting settlements).

It is also not considered to be a practical option to support the culvert on deep foundations since the available subsurface information indicates the bedrock surface is at a depth of 10 m or more below founding level and would result in long and potentially expensive piles. Therefore detailed design guidelines are not provided for deep foundations since they would not be economical or practical.

Based on the above, a closed box culvert is the preferred design option, from a foundation design perspective, due to the greater distribution of the foundation loads which would result in lower foundation stress levels and reduced settlement magnitudes. A pipe arch culvert is also considered to be feasible but is less preferred from a foundations perspective. Relining the culvert using light weight grout is also considered a feasible option from a foundation design perspective.

Recommendations for the box culvert, aluminum pipe arch and culvert relining using light weight grout options are presented in the following sections.

6.2.1 Box Culvert

6.2.1.1 Geotechnical Resistance

It is understood that the box culvert will be founded on or within the soft to firm unweathered clay subgrade at about Elevations 62.1 m and 62.0 m at the centrelines of the eastbound and westbound lanes, respectively. It is also understood that an approximately 200 mm thick layer of substrate will be placed within the culvert. It is recommended that a minimum 300 mm thick layer of Ontario Provincial Standard Specification (OPSS) 1010 *(Material Specification for Aggregates – Base, Subbase, Select Subgrade and Backfill Material)* Granular A be placed below the base slab on the subgrade to form a bedding layer for the culvert segments, and to limit the degradation of the sensitive clay subgrade.

The geotechnical resistance at Serviceability Limit States (SLS) is controlled by the compressibility of the soft clay deposit present at a shallow depth below founding level. In considering the height and weight of the existing embankment fills beneath the eastbound and westbound lanes, the available information indicates that the clay deposit is normally consolidated (i.e., the existing effective pressure at depth is about equal to the deposit's preconsolidation pressure). Therefore, beneath the existing lanes, any increase in stress above the existing value can be expected to result in settlements that could be significant.

Therefore, it is considered that the SLS resistance at the founding level of the culvert should be selected as a value that does not exceed the existing stress level at the founding level, due to the weight of the adjacent soils. On that basis, the foundations should be designed based on a geotechnical resistance at Serviceability Limit States (SLS) of 65 kPa. This value is somewhat lower than the measured preconsolidation pressure of 75 kPa to account for possible variations in the proposed loading conditions (i.e., the unit weights of the existing soils, the thickness of the existing fill etc.) as well as the existing stress condition at the proposed founding level of the culvert (which is at a shallower depth than that of the consolidation test sample).

If the bearing pressure is slightly less than the existing stress level in the adjacent ground, then the culvert settlements will be essentially consistent with (and no more than) the adjacent ground settlements, as discussed below in Section 6.2.1.2 of this report.

The factored geotechnical resistance at Ultimate Limit States (ULS) for the culvert will be controlled by the shear strength of the underlying soft grey clay as well as by the depth of embedment below ground surface level. A factored ULS geotechnical resistance of 100 kPa may be used, provided the culvert is founded at least 4 m below the pavement surface.

These geotechnical resistances are given under the assumption that the loads will be applied perpendicular to the surface of the footings. Where the load is not applied perpendicular to the surface of the footing, inclination of the load should be taken into account in accordance with Section 6.7.2 of the *Canadian Highway Bridge Design Code (CHBDC)*.

6.2.1.2 Estimates of Culvert Settlement

If the SLS geotechnical resistance for the replacement culvert is limited to 65 kPa (i.e., does not closely approach or exceed the existing stress level in the adjacent ground) then the total and differential culvert settlements should be minimal (i.e., less than about 25 and 15 millimetres, respectively). However, since the clay deposit is normally consolidated, any increase in stress beyond the existing condition (i.e., due to increased loading from the replacement culvert) will result in significant settlement. Based on the proposed founding elevations and loading of the replacement box culvert, the stress increase on the underlying silty clay will likely be in the order of 5 kPa which would result in about 50 to 75 mm of primary consolidation settlement. In addition to the primary consolidation settlement, an additional 60 to 80 mm of secondary (creep) settlement would result following the installation of the culvert (i.e., within the next 25 years).

Similar settlements to those mentioned above will also result adjacent to the culvert due to the increased weight of the granular materials which will be used as backfill alongside the culvert. This could result in 'sags' in the pavement profile leading up to and away from the culvert.

Expanded polystyrene (EPS) fill could be provided to reduce the settlement magnitudes, if required, immediately adjacent to the culvert. The required thickness of EPS would depend on the founding level elevation. If the new culvert is founded at an elevation 1 metre below the existing culvert founding level, then it is considered that a layer of EPS about 0.35 metres in thickness would be sufficient to reduce the settlements to within tolerable limits. The EPS should extend at least 5 metres from the edge of the culvert along the roadway.

Alternatively, the settlement due to the increased weight of backfill could be allowed to occur but increased pavement maintenance alongside the culvert (i.e., padding) should be expected.

It should be noted that since the clay at this site is normally consolidated, small changes in loading can result in significant settlement magnitudes. Increases in loading could occur during construction due to a number of factors that are outside the control of the designer (i.e., such as over-excavation resulting in increased thicknesses of granular material or increased concrete thicknesses). The risk of larger than tolerable settlements, due to unforeseeable construction or site circumstances that result in modest load increases, should be considered during the design. A sample Non-Standard Special Provision (NSSP) has been included in Appendix A which warns the contractor of this issue.

6.2.1.3 Resistance to Lateral Loads

The resistance to lateral forces/sliding for the culvert should be calculated in accordance with Section 6.7.5 of the *CHBDC*. The resistance values will also depend on the founding levels/strata.

The culvert will be constructed on a granular pad on the unweathered clay. For this case, the following parameters should be used:

Interface and Loading Condition	Parameter
Concrete – granular pad: short or long term loading	Effective friction angle = 33 degrees
Granular A pad – clay subgrade: short term loading	Undrained cohesion = 20 kPa
Granular A pad – clay subgrade: long term loading	Effective friction angle = 28 degrees

These values are unfactored; in accordance with the *CHBDC*, a factor of 0.8 is to be applied in calculating the horizontal resistance.

6.2.1.4 Culvert Bedding, Backfill and Erosion Protection

For a box culvert replacement, the bedding levelling pad and backfill requirements should be in accordance with OPSS 422 *(Construction Specification for Precast Reinforced Concrete Box Culverts and Box Sewers in Open Cut)* for pre-cast rigid frame culverts. Box culvert replacements should be provided with at least 300 mm of OPSS 1010 Granular A material for bedding purposes.

Backfill and cover for the concrete culvert should be completed in accordance with Ontario Provincial Standard Drawing (OPSD) 803.010 *(Backfill and Cover for Concrete Culverts)*. Backfill for the box culvert walls should consist of granular fill meeting the requirements of OPSS Granular A or Granular B Type II, but with less than 5 percent passing the No. 200 sieve. The backfill should be placed and compacted in accordance with MTO's Special Provision SP105S10 (Amendment to OPSS 501). The fill depth during placement should be maintained equal on both sides of the culvert walls, with one side not exceeding the other by more than 500 mm in height. The culvert replacement should be designed for the full overburden pressure and live loads, assuming an embankment fill unit weight of 22 kN/m^3 for Granular A and 21 kN/m^3 for Granular B Type II. The performance of the box culvert is dependent on the construction procedures. Therefore it is suggested that if a box culvert is selected as the method of replacement, that a Quality Verification Engineer be retained to verify the construction procedures.

To prevent surface water from flowing either beneath the culvert (potentially causing undermining and scouring) or around the culvert (creating seepage through the embankment fill, and potentially causing erosion and loss of fine soil particles), a clay seal or concrete cut-off wall should be provided at the upstream and downstream ends of the culvert replacement. If clay seals are adopted, the clay material should meet the requirements of OPSS 1205 *(Material Specification for Clay Seal)*. The clay seals should have a thickness of 1 m, and the seal should extend from a depth of 1 m below the scour level to a minimum horizontal distance of 2 m on either side of the culvert inlet/outlet opening, and a minimum vertical height equivalent to the maximum 100 year water level including treatment of the adjacent side slopes. Alternatively, clay blankets may be constructed, extending upstream/downstream to a distance equal to three times the culvert height. Normally, a clay blanket would extend along the adjacent embankment side slopes to a height of two times the culvert height or the high water level, whichever is higher; however, at this site where the cover over the culvert is relatively thin, it is recommended that a clay blanket, if adopted, extend to the top of the embankment side slope. If a cast-in-place concrete cut-off wall is adopted it should extend the full width of the culvert. The concrete cut-off should have a thickness of 400 mm and extend to a depth of 1.2 m below the scour level. The cut-off walls should be earth formed within trenches cut for their construction or precast and backfilled with compactable clay to maintain intimate contact between the concrete and the native low permeability soils.

If the municipal drain flow velocities are sufficiently high, a provision should be made for scour and erosion protection (suitable non-woven geotextiles and/or rip-rap) at the culvert inlet and outlet. The requirements for and design of erosion protection measures for the culvert inlet should be assessed by the hydraulic design engineer. As a minimum, rip-rap treatment for the culvert outlet should be consistent with the standard Treatment Type A presented in OPSD 810.010 *(Rip-Rap Treatment for Sewer and Culvert Outlets)*, with the rip-rap placed up to the toe of slope level, in combination with the cut-off measures noted above. Similarly, rip-rap should be provided over the full extent of the clay blanket if adopted, including the drain side slopes and embankment fill slope adjacent to the culvert.

6.2.1.5 Lateral Earth Pressures for Design

The lateral earth pressures acting on the box culvert walls will depend on the type and method of placement of the backfill materials, the nature of the soils behind the backfill, the magnitude of the surcharge including construction loadings, the freedom of lateral movement of the structure, and the drainage conditions behind the walls. Seismic (earthquake) loading must also be taken into account in the design.

The following recommendations are made concerning the design of the walls. 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 percent passing the No. 200 sieve should be used as backfill behind the walls. Longitudinal drains and weep holes should be installed as necessary to provide positive drainage of the granular backfill.
- A minimum compaction surcharge of 12 kPa should be included in the lateral earth pressures for the structural design of the culvert walls, in accordance with *CHBDC* Section 6.9.3 and Figure 6.6. Compaction equipment should be used in accordance with MTO's Special Provision SP105S10 *(Amendment to OPSS 501)*. Other surcharge loadings should be accounted for in the design as required.

- The granular fill may be placed either in a zone with the width equal to at least 1.8 m behind the back of the walls (see Case A in Figure C6.20(a) of the *Commentary* to the *CHBDC*), or within the wedge shaped zone defined by a line drawn at 1.5 horizontal to 1 vertical (1.5H:1V) extending up and back from the rear face of the footing (see Case B in Figure C6.20(b) of the *Commentary* to the *CHBDC*).

- For Case A, the pressures are based on the existing embankment fill materials and the existing overburden soils and the following parameters (unfactored) may be used:

	Existing Fill
Soil unit weight:	20 kN/m^3
Coefficients of static lateral earth pressure: At rest, K_o	0.50

For Case B, where the pressures are based on OPSS Granular A or Granular B Type II fill behind the wall, the following parameters (unfactored) may be assumed:

	Granular A	**Granular B Type II**
Soil unit weight:	22 kN/m^3	21 kN/m^3
Coefficients of static lateral earth pressure: At rest, K_o	0.43	0.43

Because the culvert walls do not allow lateral yielding, at-rest earth pressures should be assumed for the geotechnical design.

6.2.1.6 Seismic Considerations

Seismic (earthquake) loading should be assessed in the design in accordance with Section 4.6.4 of *CHBDC,* as significant seismic loading would result in increased lateral earth pressures acting on the culvert walls. The walls should be designed to withstand the combined lateral loading for the appropriate static pressure conditions given above, plus the applicable earthquake-induced dynamic earth pressure. The earthquake-induced dynamic pressure distribution is a linear distribution with maximum pressure at the top of the wall and minimum pressure at its toe (i.e., an inverted triangular pressure distribution). The total pressure distribution (static plus seismic) may be determined as follows:

$$P = K_o \gamma d + (K_{AE} - K_o) \gamma (H - d)$$

where K_o is the static at-rest earth pressure coefficient (K_o);

K_{AE} is the seismic active earth pressure coefficient;

γ is the unit weight of the backfill soil (kN/m^3) as given previously;

d is the depth below the top of the wall (m); and

H is the height of the wall above the toe (m).

According to Table C4.2 of the *Commentary* to the *CHBDC*, this site is located in Seismic Zone 3, and the site-specific zonal acceleration ratio for the Ottawa area is 0.2 which is also applicable for the culvert site. Based on experience, for the subsurface conditions at this site, a 10 percent amplification of the ground motion could occur, resulting in an increase in the ground surface acceleration to 0.22 g.

The seismic lateral earth pressure coefficients given below have been derived based on a design zonal acceleration ratio of A = 0.22.

In accordance with Sections 4.6.4 and C.4.6.4 of the *CHBDC* and its *Commentary*, for structures which do not allow lateral yielding (i.e., culvert walls), the horizontal seismic coefficient, k_h, used in the calculation of the seismic active pressure coefficient is taken as 1.5 times the zonal acceleration ratio (i.e., $k_h = 0.33$).

These following seismic active pressure coefficients (K_{AE}) for the two backfill cases (Case A and Case B) may be used in design. These values include the effect of wall friction and assume that the back of the wall is vertical and the ground surface behind the wall is essentially flat.

SEISMIC ACTIVE PRESSURE COEFFICIENTS, K_{AE}

	Case A	Case B	
	Earth Fill	Granular A	Granular B Type II
Non-Yielding Wall	0.65	0.50	0.50

6.2.2 Pipe Arch

6.2.2.1 Geotechnical Resistance

It is understood that the pipe arch culvert will be founded on the soft unweathered clay subgrade. It is recommended that a minimum 500 mm thick layer of OPSS Granular A be placed below the culvert (as discussed in further detail below in Section 6.2.2.3) to form a bedding layer for the culvert segments and to limit the degradation of the sensitive clay subgrade.

It is understood that the haunch pressures for a pipe arch culvert at this location would be in the order of about 180 kPa. Based on the limited information available at the time of this report, this haunch pressure would result in a sustained loading at the surface of the underlying clay of about 65 kPa which is very near to the preconsolidation pressure of the clay. A slight increase in haunch loading could result in significant deformations of the culvert over time. If a pipe arch culvert is considered for this location, the design should be reviewed in detail to assess the loadings from the pipe arch haunches on the underlying clay.

6.2.2.2 Estimates of Culvert Settlement

If the SLS geotechnical resistance for the replacement culvert is limited to 65 kPa (i.e., does not closely approach or exceed the existing stress level in the adjacent ground) then the total and differential culvert settlements should be minimal (i.e., less than about 25 and 15 millimetres, respectively). However, since the clay deposit is normally consolidated, any increase in stress beyond the existing condition (i.e., due to increased loading from the replacement culvert) will result in significant settlement. For example, a stress increase of 5 kPa would result in about 50 to 75 mm of primary consolidation settlement and a stress increase of 10 kPa would result in about 100 to 150 mm of primary consolidation settlement. In addition to the primary consolidation settlement an additional 60 to 80 mm of secondary (creep) settlement could result following the installation of the culvert (i.e., within the next 25 years) if the imposed loading exceeds the existing stress level within the clay deposit.

The reinforcing trenches specified in Section 7.6.4.2 of the *CHBDC* (see Section 6.2.2.3 below) would be excavated into the underlying clay and would be backfilled with granular material. The granular material has higher unit weight than the clay and this would result in an increase in loading on the clay. Depending on the depth of the trenches, this increase in stress could result in settlement magnitudes that are greater than tolerable.

These settlements due to the increased weight of backfill will likely result in increased pavement maintenance alongside the culvert (i.e., padding).

6.2.2.3 Culvert Bedding, Backfill and Erosion Protection

It is understood that a 4370 mm wide by 2870 mm high pipe arch is being considered for the culvert replacement (i.e., a pipe arch section equivalent to the existing culvert and will be founded between Elevations 62.5 m and 62.7 m at the existing invert levels). For a pipe arch replacement, the bedding levelling pad and backfill requirements should be in accordance with OPSS 421 *(Construction Specification for Pipe Culvert Installation in Open Cut)*. The pipe arch should be provided with at least 500 mm of OPSS Granular A material for bedding purposes shaped to the underside of the pipe arch as per Section 7.6.5.4 of the *CHBDC*. A 200 mm thickness of the bedding layer that is in direct contact with the invert should be left uncompacted to allow proper embedment of the corrugation profile. The remaining portion of the bedding should be compacted to at least 95 percent of the standard Proctor maximum dry density using suitable vibratory equipment. In addition, as per Section 7.6.4.2 of the *CHBDC,* reinforcement of the haunches of pipe-arches should be provided as shown in Figure 7.4 of the *CHBDC*. The material placed in the trench reinforcement should also consist of OPSS Granular A material compacted to at least 95 percent of the standard Proctor dry density.

Backfill and cover for the pipe arch should consist of OPSS Granular A. The backfill and cover should be placed in maximum 300 mm thick lifts and should be compacted to at least 95 percent of the standard Proctor dry density. The backfill should extend transversely at least 2.2 metres on each side beyond the spring lines of the pipe and vertically up to at least 900 mm above the pipe. The fill depth during placement should be maintained equal on both sides of the culvert walls, with one side not exceeding the other by more than 200 mm in height. To limit compaction induced stresses in the wall of the structure, heavy equipment should not be allowed within 1 m of the wall during compaction of the backfill. The performance of the pipe arch is greatly dependent on the construction procedure. If inadequate compaction is provided around the pipe walls, particularly in the haunch areas, the culvert could fail. Therefore it is suggested that if a pipe arch is selected as the method of replacement, that a Quality Verification Engineer be retained to verify the construction procedures.

Frost tapers should be provided within the zone of frost penetration (i.e., from the pavement grade to 1.8 metres below the pavement grade) as shown on OPSD 803.031 *(Frost Treatment – Pipe Culverts Frost Penetration Line Between Top of Pipe and Bedding Grade)*.

The culvert replacement should be designed for the full overburden pressure and live loads, assuming an embankment fill unit weight of 22 kN/m^3 for Granular A.

To prevent surface water from flowing either beneath the culvert (potentially causing undermining and scouring) or around the culvert (creating seepage through the embankment fill, and potentially causing erosion and loss of fine soil particles), a clay seal or concrete cut-off wall should be provided at the upstream and downstream ends of the culvert replacement. If clay seals are adopted, the clay material should meet the requirements of OPSS 1205 *(Material Specification for Clay Seal)*. The clay seals should have a thickness of 1 m, and the seal should extend from a depth of 1 m below the scour level to a minimum horizontal distance of 2 m on either side of the culvert inlet/outlet opening, and a minimum vertical height equivalent to the high water level including treatment of the adjacent side slopes. Alternatively, clay blankets may be constructed, extending upstream/downstream to a distance equal to three times the culvert height. Normally, a clay blanket would extend along the adjacent embankment side slopes to a height of two times the culvert height or the high water level, whichever is higher; however, at this site where the cover over the culvert is relatively thin, it is recommended that a clay blanket, if adopted, extend to the top of the embankment side slope. If a concrete cast-in-place cut-off wall is adopted it should be designed in accordance with OPSD 812.010 (*Cut Off Wall for Structural Plate Pipe Arch and Circular CSP*). The cut-off walls should be earth formed within trenches cut for their construction or precast and backfilled with compactable clay to maintain intimate contact between the concrete and the native low permeability soils.

If the municipal drain flow velocities are sufficiently high, a provision should be made for scour and erosion protection (suitable non-woven geotextiles and/or rip-rap) at the culvert inlet and outlet. The requirements for and design of erosion protection measures for the culvert inlet should be assessed by the hydraulic design engineer. As a minimum, rip-rap treatment for the culvert outlet should be consistent with the standard Treatment Type A presented in OPSD 810.010 *(Rip-Rap Treatment for Sewer and Culvert Outlets),* with the rip-rap placed up to the toe of slope level, in combination with the cut-off measures noted above. Similarly, rip-rap should be provided over the full extent of the clay blanket if adopted, including the drain side slopes and embankment fill slope adjacent to the culvert.

6.2.3 Relining with Light Weight Grout

6.2.3.1 General

Relining of the existing culvert with a grouted in place metal liner is another alternative that is being considered. The metal liner would be installed within the existing culvert and the annular space between the existing culvert and the liner would be filled with grout. In order to limit the stress increase on the underlying compressible silty clay, this option is only viable if a light weight grout such as a CEMATRIX Cellular Concrete or equivalent is used.

6.2.3.2 Estimates of Culvert Settlement

As mentioned previously, since the clay deposit at this site is normally consolidated, any increase in stress beyond the existing condition (i.e., due to increased loading from the liner and grout) will result in significant settlement. For example, a stress increase of 5 kPa would result in about 50 to 75 mm of primary consolidation settlement and a stress increase of 10 kPa would result in about 100 to 150 mm of primary consolidation settlement. In addition to the primary consolidation settlement an additional 60 to 80 mm of secondary (creep) settlement could result following the installation of the culvert (i.e., within the next 25 years) if the imposed loading exceeds the existing stress level within the clay deposit.

6.2.3.3 *Erosion Protection*

Erosion protection measures as described in Section 6.2.2.3 should also be carried out for the culvert relining.

6.2.4 Construction Considerations

6.2.4.1 *Groundwater and Surface Water Control*

Control of the surface water and groundwater will be necessary for the construction of the culvert replacement, to allow excavation and foundation construction to be carried out in dry conditions.

Depending on the municipal drain flow at the time of construction, the surface water flow could be passed through the culvert area by means of a temporary pipe, or diverted by pumping from behind a temporary cofferdam. Surface water should be directed away from the excavation areas to prevent ponding of water that could result in disturbance and weakening of the sensitive clay subgrade soils; further discussion on this aspect is provided in Section 6.2.4.3.

A sample NSSP for groundwater and surface water control is provided in Appendix A.

6.2.4.2 *Excavations and Temporary Roadway Protection*

Temporary excavations for the culvert replacement will be made through the existing fill and are expected to terminate or extend into the soft to firm unweathered clay deposit. Excavation works must be carried out in accordance with the guidelines outlined in the Occupational Health and Safety Act (OHSA) and Regulations for Construction Projects. The existing fill would be classified as Type 3 soil and the underlying soft to firm clay would be classified as Type 4 soil, based on the OHSA. According to OHSA, excavations that extend to, or into, Type 4 soils should be made with side slopes no steeper than 3 horizontal to 1 vertical (3H:1V). However, stability analyses carried out for excavations up to about 4.8 m depth (i.e., the maximum proposed depth for the culvert replacement) below the existing ground/pavement surface, with excavation side slopes of 3H:1V, indicate a factor of safety of 1.0, against slope instability for short term excavation side slopes. Therefore flatter excavation side slopes may be necessary. A sample NSSP regarding this issue is provided in Appendix A.

If 3H:1V open cut excavation side slopes cannot be accommodated, then roadway protection (i.e., temporary excavation shoring) will be required. It is understood that this is the preferred option for the culvert replacement if a box culvert or pipe arch is selected as the design alternative. Where shoring is required, the support system should be designed and constructed, by the contractor, in accordance with SP105S19. The lateral movement of the temporary shoring system should meet Performance Level 2 as specified in SP105S19.

A conventional shoring system for these conditions would consist of interlocking steel sheet piling supported against lateral movement using whalers, tie backs (into the glacial till or underlying bedrock) and/or internal struts/braces. The design of that system should be entirely the responsibility of the contractor. However the design of that shoring must consider the soft clay deposit at depth and the potential for basal instability of the excavation. Basal instability occurs when the soil beneath the sheeting is sheared by the unbalanced weight between the soil outside and inside the excavation. Calculations indicated that a conventional shored excavation with full depth vertical walls to the base of the excavation would not have an adequate factor of safety against basal instability (i.e., have a factor of safety of less than 1.5). A basal instability failure could lead to the flow of sheared/disturbed clay into the excavation, significant loss of ground (settlement and ground slumping) behind the sheeting, and possible collapse of the shoring system. Therefore, the shoring system would need to extend below the excavation floor level into the native glacial till (i.e., about 11 to 12 metres below the existing pavement grade) to prevent basal instability. In addition, the design of the sheeting projection would also need to resist the lateral loading imposed by the clay. In accordance with the U.S. Department of Navy, 1971 NAVFAC DM-7[2] method, the force per unit length of the buried sheet pile would likely be in the range of 200 to 400 kN/m. This may require a very heavy/strong sheeting section.

As a further guideline, excavated soils should not be stockpiled adjacent to the crest of the excavation side slopes (or above shoring) due to the potential to reduce the factor of safety against side slope or basal instability.

A sample NSSP regarding basal heave is provided in Appendix A.

6.2.4.3 Subgrade Protection

The clay that is exposed at the founding/subgrade level will be susceptible to disturbance from construction traffic and ponded water.

Trafficking over the soft clay subgrade will not be possible. An Operational Constraint or a Non-Standard Special Provision should be included in the contract in this regard, which directs the contractor to not travel on the subgrade surface with equipment.

The box culvert should be provided with a minimum 300 mm of OPSS Granular A bedding and the pipe arch should be provided with a minimum of 500 mm of OPSS Granular A bedding (as mentioned in Sections 6.2.1.4 and 6.2.2.3).

A sample NSSP for subgrade protection is provided in Appendix A.

[2] U.S. Department of the Navy (1971). "Design Manual – Soil Mechanics, Foundations, and Earth Structures," NAVFAC DM-7, Washington, D.C.

7.0 CLOSURE

This report was prepared by Ms. Susan Trickey, P.Eng. and reviewed by Mr. Fintan Heffernan P.Eng., the designated MTO contact for this project.

Yours truly,

GOLDER ASSOCIATES LTD.

LICENSED PROFESSIONAL ENGINEER
S. A. TRICKEY
100104579
PROVINCE OF ONTARIO

Susan Trickey, P.Eng.
Geotechnical Engineer

REGISTERED PROFESSIONAL ENGINEER
F. J. HEFFERNAN
PROVINCE OF ONTARIO

Fintan Heffernan, P.Eng.
Designated MTO Contact

SAT/WC/FJH/am

n:\active\2009\1121 - geotechnical\09-1121-1004 dm wills petawawa bridge\foundations\tayside culvert\09-1121-1004-3000 rpt-001 october2010.docx

TABLE 1

COMPARISON OF FOUNDATION ALTERNATIVES
G.W.P. 4059-01-00

Foundation Option	Feasibility	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Option 1 Concrete Box	▪ Feasible	▪ Can be designed to reduce settlement magnitudes. ▪ Long design life	▪ Need to optimize culvert dimensions to minimize the increase in foundation stresses ▪ Deeper founding levels required for hydraulic design could result in higher settlement magnitudes adjacent to culvert	▪ Moderate cost	▪ Generally low risk option
Option 2 Aluminum Pipe Arch	▪ Feasible	▪ Can be designed to maintain existing stress levels (at the culvert location)	▪ Requires careful placement and compaction of backfill materials to provide adequate support to the culvert walls to prevent failure ▪ May have shorter design life than a concrete structure ▪ Reinforcing trenches may increase stress levels and settlement magnitudes at those locations adjacent to the culvert	▪ Moderate cost	▪ Moderate risk of long term deformations
Option 3 Relining with Light Weight Grout	▪ Feasible	▪ Will minimize traffic disruption ▪ Roadway protection would not be required	▪ May have a shorter design life than a concrete structure	▪ Moderate cost	▪ Low risk option
Option 4 Rigid Frame Open Footing	▪ Not feasible	▪ Desirable option for culvert flow	▪ Large settlements due to higher concentration stresses from narrow foundations	▪ Moderate cost	▪ High risk option
Option 5 Deep Foundations	▪ Feasible but not required/practical	▪ Would not result in culvert settlement	▪ Would require piles	▪ Expensive option	▪ Low risk option
Option 6 Relining with Conventional Grout	▪ Not feasible	▪ Will minimize traffic disruption ▪ Roadway protection would not be required	▪ Increased loading due to grout and liner would result in large culvert settlements	▪ Least expensive option	▪ High risk option

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
DO	Drive open
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
DT	Dual Tube sample

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.) drive open Sampler for a distance of 300 mm (12 in.)
DD- Diamond Drilling

Dynamic 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, 60^0 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

Peizo-Cone Penetration Test (CPT):
An electronic cone penetrometer with a 60^0 conical tip and a projected end area of 10 cm^2 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.

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

(b) Cohesive Soils

Consistency	**C_u or S_u** Kpa	Psf
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

IV. SOIL TESTS

w	water content
w_p	plastic limited
w_l	liquid limit
C	consolidaiton (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 test (LV-laboratory vane test)
γ	unit weight

Note:
1. Tests which are anisotropically consolidated prior 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 stresses (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 ($D_R= p_s/p_w$) formerly (G_s)
e	void ratio
n	porosity
S	degree of saturation

* Density symbol is p. Unit weight symbol is γ where γ=pg(i.e. mass density x acceleration due to gravity)

(a) Index Properties (cont'd.)

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 (overconsolidated 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	Overconsolidation ratio=σ'_p/σ'_{vo}

(d) Shear Strength

$\tau_p\tau_r$	peak and residual shear strength
ϕ'	effective angle of internal friction
δ	angle of interface friction
μ	coefficient of friction=tan δ
c'	effective cohesion
c_u,S_u	undrained shear strength (ϕ=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. τ=c'σ' tan |'

2. Shear strength=(Compressive strength)/2

Golder Associates

RECORD OF BOREHOLE No 10-1

 METRIC

PROJECT	09-1121-1004		
G.W.P.	4059-01-00	LOCATION	N 5022159.0 ;E 190502.8
DIST		HWY	417
BOREHOLE TYPE	Power Auger, 200mm Diam. Hollow Stem		
DATUM	Geodetic	DATE	April 14, 2010
ORIGINATED BY	HEC	COMPILED BY	JM
CHECKED BY	SAT		

ELEV / DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
66.1	GROUND SURFACE				
0.0	Crushed stone (FILL) Grey				
65.3 / 0.8	Sand and gravel, some silt (FILL) Loose to compact Grey-brown to grey Moist	1	SS	4	
		2	SS	6	
		3	SS	12	42 46 12 0
		4	SS	12	
62.0 / 4.1	CLAY Soft to firm Grey Wet	5	SS	5	
		6	TP	PH	
		7	TP	PH	

Column headings: SOIL PROFILE (ELEV DEPTH, DESCRIPTION, STRAT PLOT); SAMPLES (NUMBER, TYPE, "N" VALUES); GROUND WATER CONDITIONS; ELEVATION SCALE (66, 65, 64, 63, 62, 61, 60, 59, 58, 57); DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100; PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L, WATER CONTENT (%) 25 50 75; UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

Continued Next Page

$+^3, \times^3$: Numbers refer to Sensitivity ○ $^{3\%}$ STRAIN AT FAILURE

RECORD OF BOREHOLE No 10-1

 METRIC

PROJECT 09-1121-1004
G.W.P. 4059-01-00
LOCATION N 5022159.0 ;E 190502.8
ORIGINATED BY HEC
DIST
HWY 417
BOREHOLE TYPE Power Auger, 200mm Diam. Hollow Stem
COMPILED BY JM
DATUM Geodetic
DATE April 14, 2010
CHECKED BY SAT

-- CONTINUED FROM PREVIOUS PAGE --

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE
	CLAY Soft to firm Grey Wet	8	SS	WR	56
		9	SS	WH	55
54.2 11.9	Silty SAND and GRAVEL, trace clay (TILL) Compact Grey Wet	10	SS	26	54
53.8 12.3	End of Borehole Note: Water level in piezometer at 2.6 m depth (Elev. 63.5 m) on May 11, 2010.				

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED: 20 40 60 80 100

PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L

WATER CONTENT (%): 25 50 75

UNIT WEIGHT γ kN/m³

REMARKS & GRAIN SIZE DISTRIBUTION (%): GR SA SI CL

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

Golder Associates

RECORD OF BOREHOLE No 10-2

 METRIC

PROJECT	09-1121-1004		
G.W.P.	4059-01-00	LOCATION	N 5022136.5 ;E 190485.5
DIST	HWY 417	BOREHOLE TYPE	Power Auger, 200mm Diam. Hollow Stem
DATUM	Geodetic	DATE	April 13, 2010

ORIGINATED BY HEC
COMPILED BY JM
CHECKED BY SAT

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	W (%)	GR	SA	SI	CL
66.0	GROUND SURFACE										
0.0 / 65.8	Topsoil (FILL)										
0.2	Silty clay, some sand and gravel, trace organic matter (FILL) Grey-brown										
65.2 / 0.8	Sand and gravel (FILL) Compact Brown Moist		1	SS	13						
64.5 / 1.5	CLAY (Weathered Crust) Very stiff to stiff Grey-brown Moist		2	SS	6			0	1	29	70
			3	SS	4						
62.9 / 3.1	CLAY Soft to firm Grey Moist to wet		4	SS	WH			0	0	28	72
						∇					
			5	TP	PH						
			6	SS	WH						
			7	TP	PH						
			8	SS	WH						

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH kPa: O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED; 20 40 60 80 100
PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 25 50 75
UNIT WEIGHT γ kN/m³
REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEVATION SCALE: 65, 64, 63, 62, 61, 60, 59, 58, 57

Continued Next Page

$+^3, \times^3$: Numbers refer to Sensitivity $O^{3\%}$ STRAIN AT FAILURE

MIS-MTO 001 0911211004-3000.GPJ GAL-MISS.GDT 10/22/10 JM

RECORD OF BOREHOLE No 10-2

 METRIC

PROJECT 09-1121-1004
G.W.P. 4059-01-00
LOCATION N 5022136.5 ;E 190485.5
ORIGINATED BY HEC
DIST HWY 417
BOREHOLE TYPE Power Auger, 200mm Diam. Hollow Stem
COMPILED BY JM
DATUM Geodetic
DATE April 13, 2010
CHECKED BY SAT

-- CONTINUED FROM PREVIOUS PAGE --

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	CLAY Soft to firm Grey Moist to wet					
55.2 10.8	Silty SAND and GRAVEL, trace clay (TILL) Compact Grey Wet	9	SS	20	55	53 36 9 2
		10	SS	24		
54.0 12.0	End of Borehole Note: Water level in open borehole at 3.8 m depth below ground surface upon completion of drilling				54	

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH kPa: O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED; 20 40 60 80 100
PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 25 50 75
UNIT WEIGHT γ kN/m³

$+^3, \times^3$: Numbers refer to Sensitivity $O^{3\%}$ STRAIN AT FAILURE

Golder Associates

RECORD OF BOREHOLE No 10-3

 METRIC

PROJECT 09-1121-1004			
G.W.P. 4059-01-00	LOCATION N 5022096.8 ;E 190439.8	ORIGINATED BY HEC	
DIST	HWY 417	BOREHOLE TYPE Power Auger, 200mm Diam. Hollow Stem	COMPILED BY JM
DATUM Geodetic	DATE April 12, 2010	CHECKED BY SAT	

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES
66.1	GROUND SURFACE			
0.0	Crushed stone (FILL) Grey			
65.5 / 0.6	Sand, some gravel and silty clay, trace organic matter (FILL) Loose Brown Moist	1	SS	6
		2	SS	6
63.8 / 2.3	Sand and gravel (FILL) Compact Brown Moist	3	SS	12
62.7 / 3.4	CLAY Soft to stiff Grey Wet	4	SS	3
		5	SS	WR
		6	TO	PH
		7	SS	WR
		8	TO	PH
		9	SS	WR

Column headings: SOIL PROFILE; SAMPLES; GROUND WATER CONDITIONS; ELEVATION SCALE (66 to 57); DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100; PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 25 50 75; UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

Continued Next Page

$+^3, \times^3$: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

MIS-MTO 001 09112110004-3000.GPJ GAL-MISS.GDT 10/22/10 JM

Golder Associates

RECORD OF BOREHOLE No 10-3

 METRIC

PROJECT: 09-1121-1004			
G.W.P.: 4059-01-00	LOCATION: N 5022096.8 ;E 190439.8	ORIGINATED BY: HEC
DIST:	HWY: 417	BOREHOLE TYPE: Power Auger, 200mm Diam. Hollow Stem	COMPILED BY: JM
DATUM: Geodetic	DATE: April 12, 2010	CHECKED BY: SAT	

— CONTINUED FROM PREVIOUS PAGE —

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	SHEAR STRENGTH / WATER CONTENT	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	CLAY Soft to stiff Grey Wet				56		
		10	SS	WR	55		
54.8 11.3	Silty SAND and GRAVEL, trace clay (TILL) Loose to compact Grey Wet	11	SS	9	54		
		12	SS	18	53		47 33 18 2
		13	SS	24			
		14	SS	24	52		26 32 36 6
51.8 14.3	End of Borehole Note: Water level in open borehole at 3.1 m depth below ground surface upon completion of drilling						

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED — 20 40 60 80 100

PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L — WATER CONTENT (%) 25 50 75

UNIT WEIGHT γ kN/m³

MIS-MTO 001 0911211004-3000.GPJ GAL-MISS.GDT 10/22/10 JM

$+^3, \times^3$: Numbers refer to Sensitivity $O^{3\%}$ STRAIN AT FAILURE

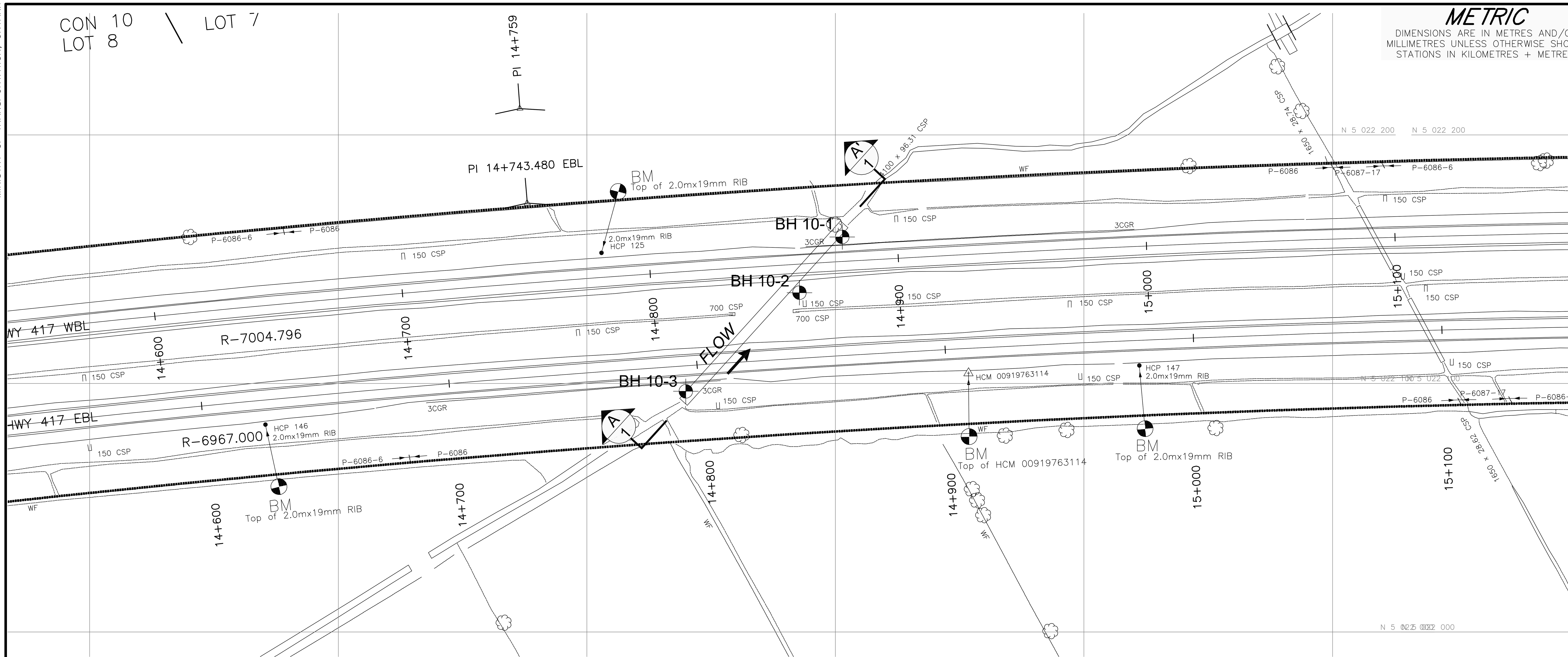
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

CONT No.
WP No. 4059–01–00

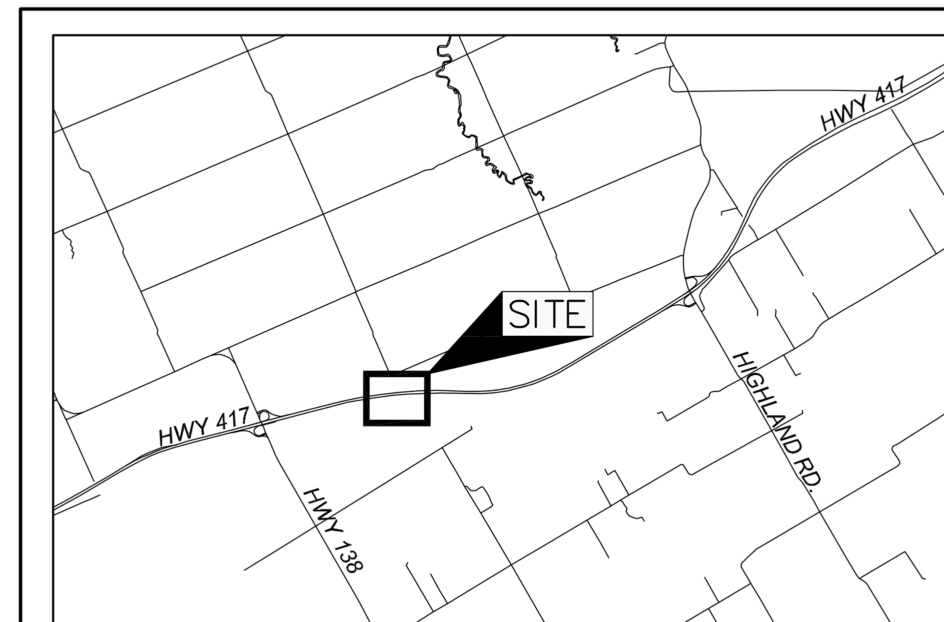
SHEET

HIGHWAY 417
TAYSIDE MUNICIPAL DRAIN CULVERT
BOREHOLE LOCATIONS AND SOIL STRATA

Golder Associates Ltd.
OTTAWA, ONTARIO, CANADA



PLAN
SCALE
20 0 20 40 m

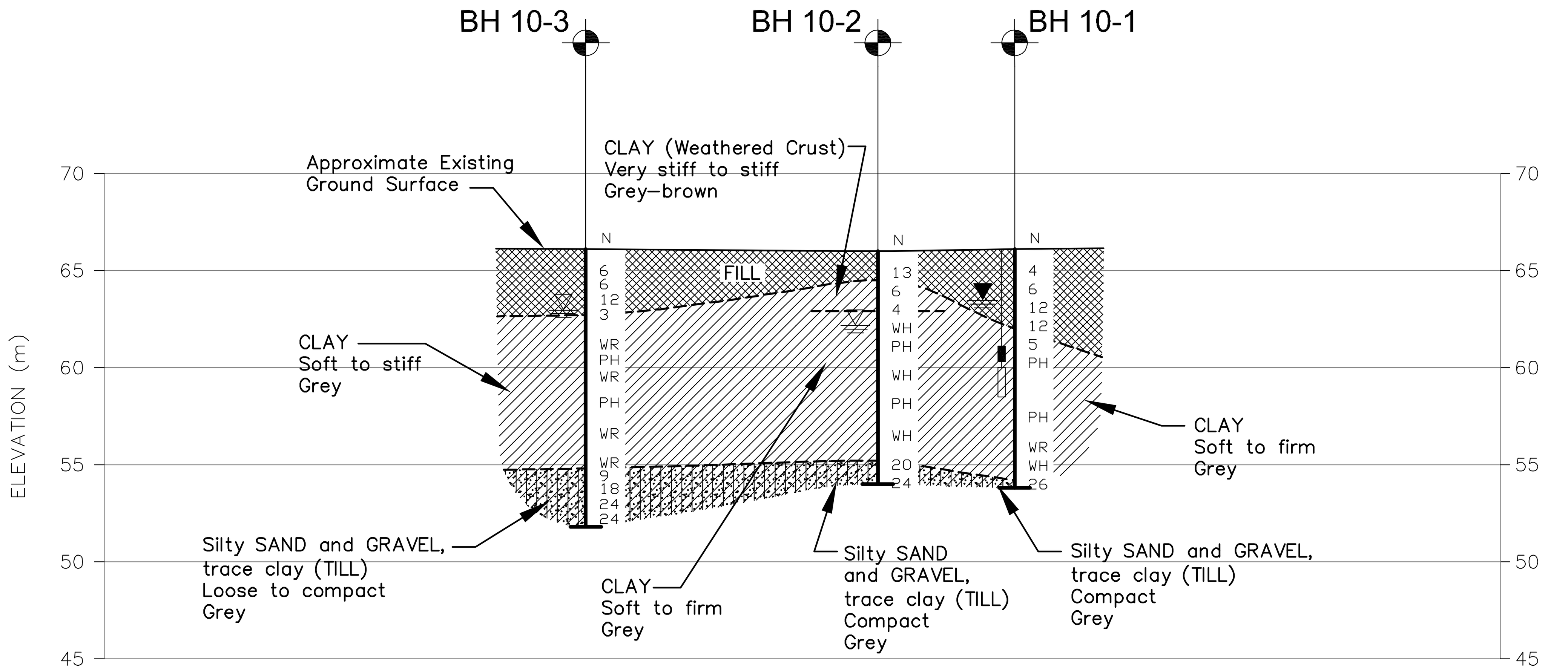


KEY PLAN
SCALE
2 0 2 4 km

LEGEND

	Borehole – Current Investigation
N	Standard Penetration Test Value
16	Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
	WL in piezometer, May 11, 2010
	WL upon completion of drilling
	Seal
	Piezometer

No.	ELEVATION	CO–ORDINATES	
		NORTHING	EASTING
10–1	66.1	5022159.0	190502.8
10–2	66.0	5022136.5	190485.5
10–3	66.1	5022096.8	190439.8



SECTION A–A'
HORIZONTAL SCALE
20 0 20 40 m
5 0 5 10 m
VERTICAL SCALE

NOTES

This drawing is for subsurface information only. The proposed structure details/works are shown for illustration purposes only and may not be consistent with the final design configuration as shown elsewhere in the Preliminary Design Report.

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

The complete Preliminary Foundation Investigation and Design Report for this project and other related documents may be examined at the Materials Engineering and Research Office, Downsview. Information contained in this report and related documents is specifically excluded in accordance with Section GC 2.01 of OPS General Conditions.

REFERENCE

Base plan provided in digital format by D.W. Wills Associates Ltd. (Drawing File No. "b–389–417–1.dwg", received May 11, 2010.

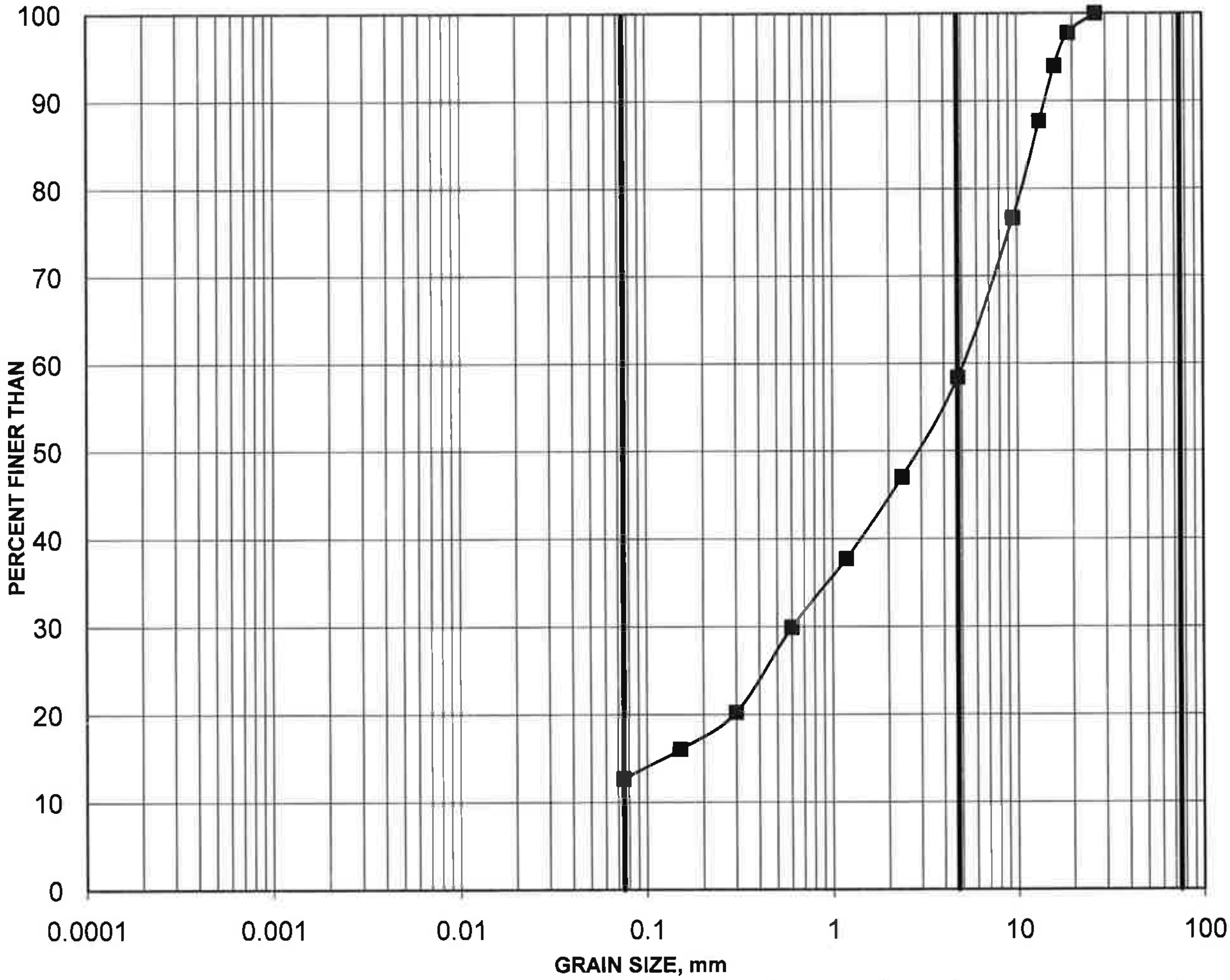
NO.	DATE	BY	REVISION

Geocres No. 31G–239

HWY. 417		PROJECT NO. 09–1121–1004	DIST.
SUBM'D. SAT	CHKD. SAT	DATE: 5/15/2010	SITE:
DRAWN: JM	CHKD. WC	APPD. FJH	DWG. 1

GRAIN SIZE DISTRIBUTION	FIGURE 1

FILL (SAND AND GRAVEL)



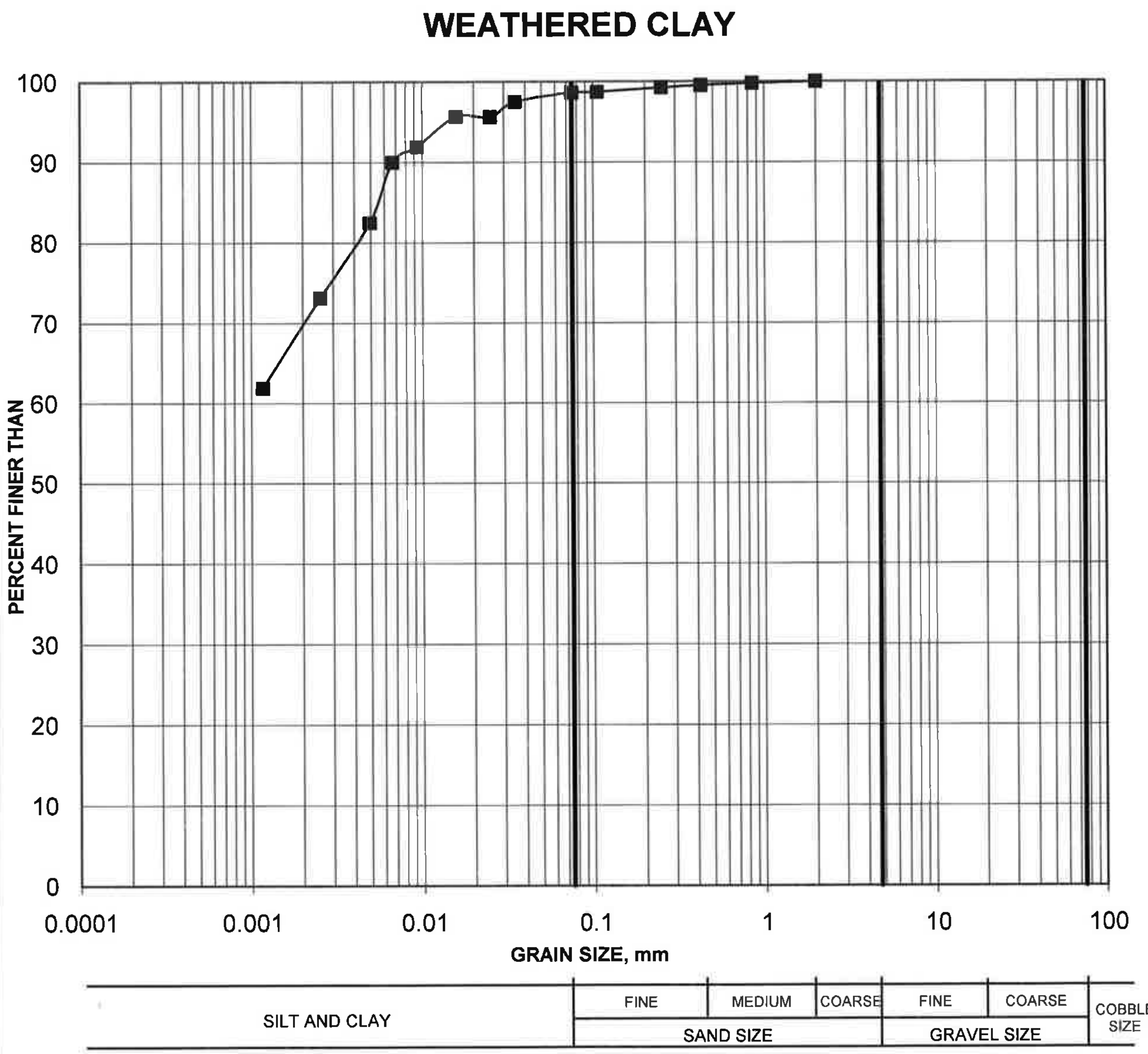
SILT AND CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
	SAND SIZE			GRAVEL SIZE		

Borehole	Sample	Depth (m)
10-1	3	2.29-2.90

Project: 09-1121-1004/3000 **Golder Associates**

Created by:	MI
Checked by:	CNM

GRAIN SIZE DISTRIBUTION	FIGURE 2



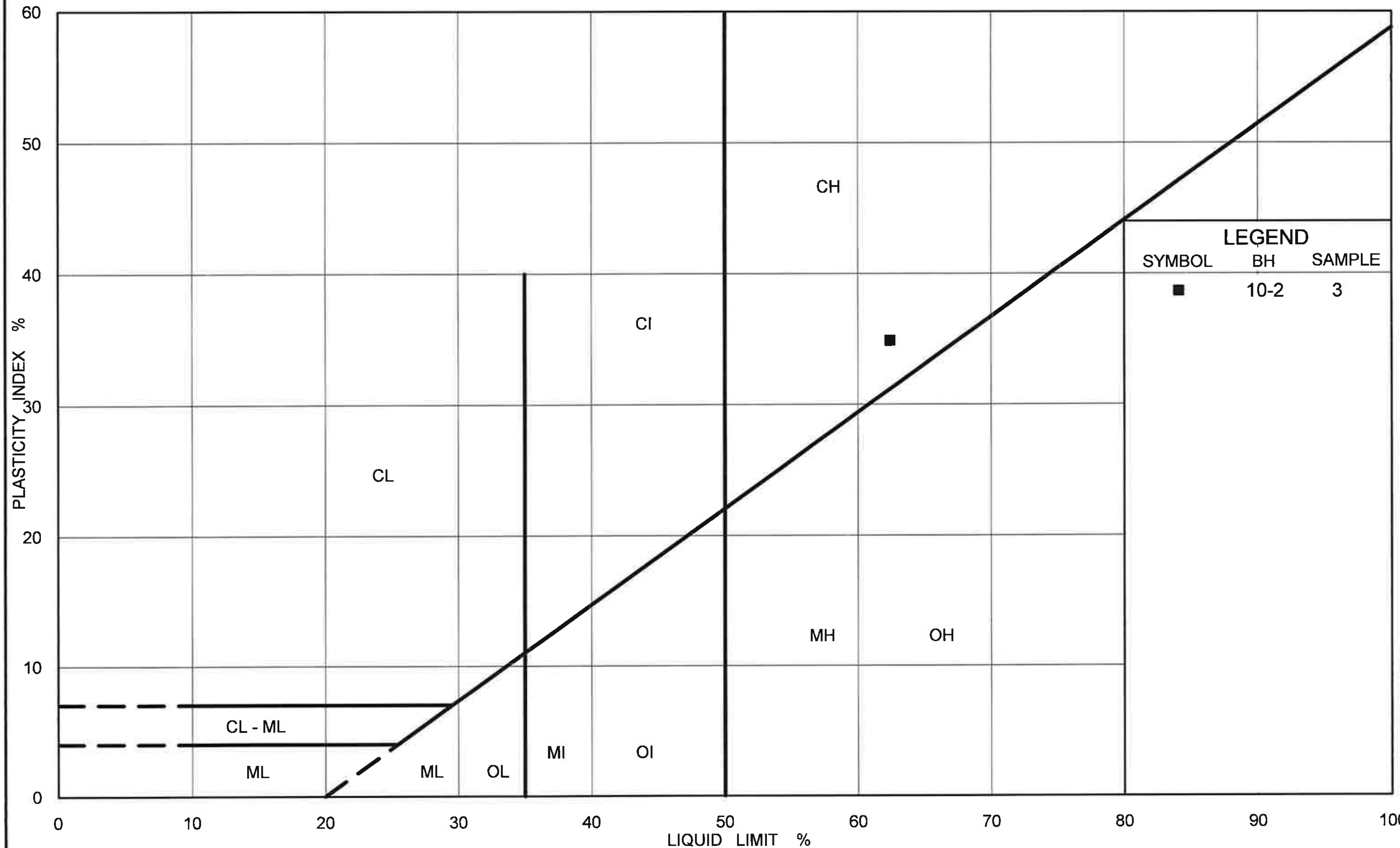
Borehole	Sample	Depth (m)
10-2	2	1.52-2.13

Project: 09-1121-1004/3000

Golder Associates

Created by: MI
Checked by: CNM

Oct 75, FF-S-21



Ministry of Transportation

Ontario

PLASTICITY CHART
Weathered Clay

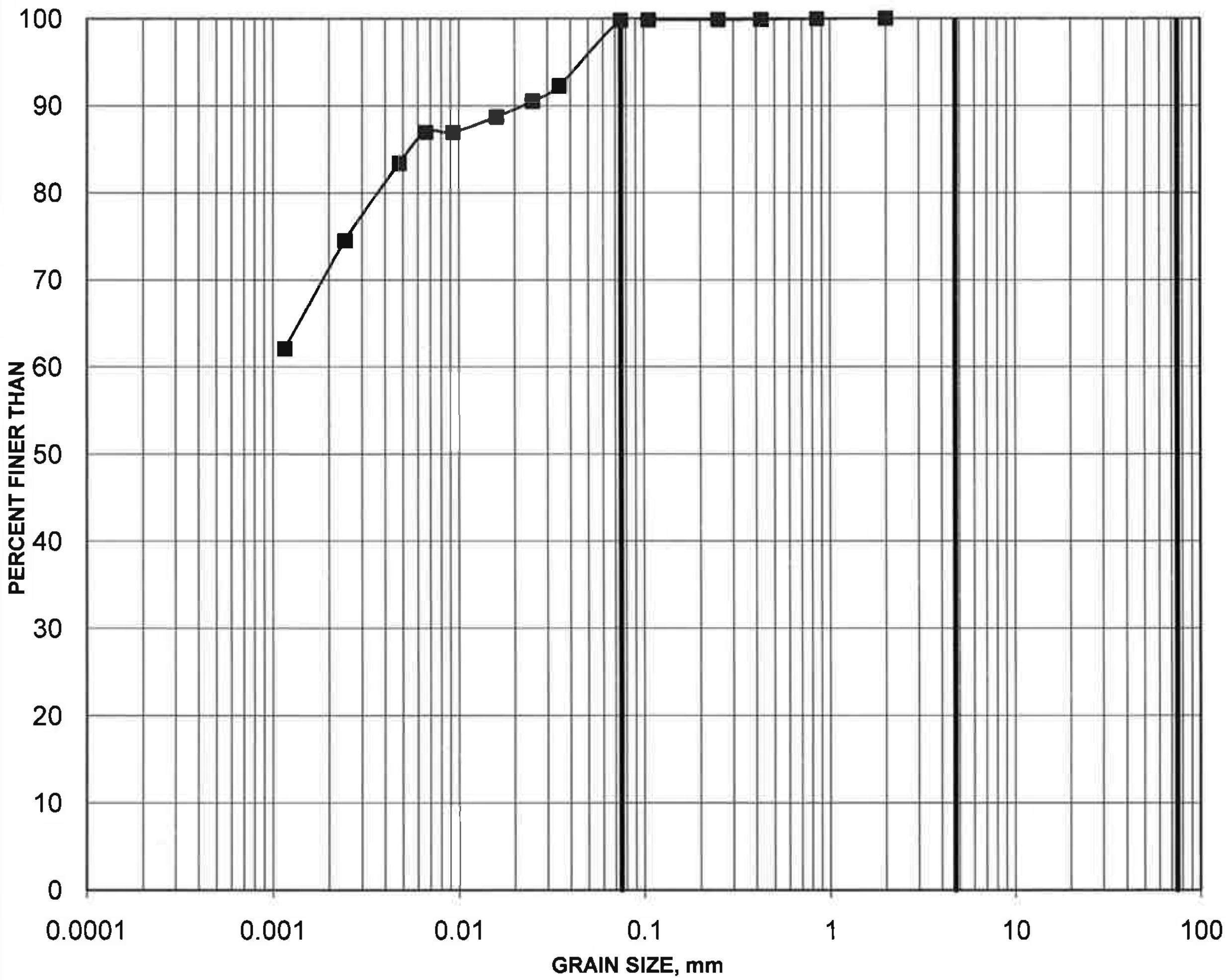
FIG No. 3

Project No. 09-1121-1004/3000

I.M.M.

GRAIN SIZE DISTRIBUTION	FIGURE 4

UNWEATHERED CLAY

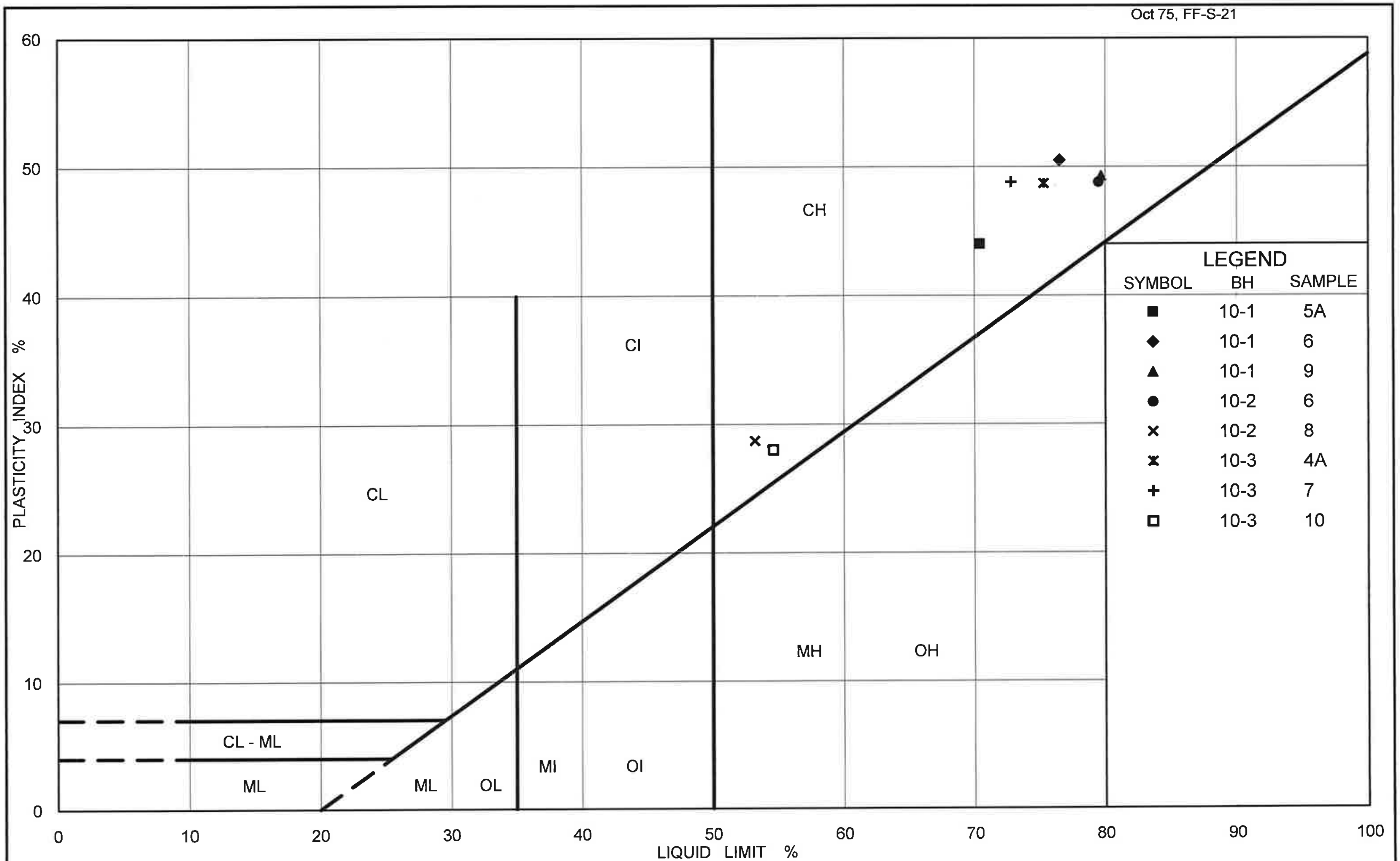


SILT AND CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
	SAND SIZE			GRAVEL SIZE		

Borehole	Sample	Depth (m)
10-2	4	3.05-3.66

Project: 09-1121-1004/3000 **Golder Associates**

Created by: MI
Checked by: CNM



Ministry of Transportation

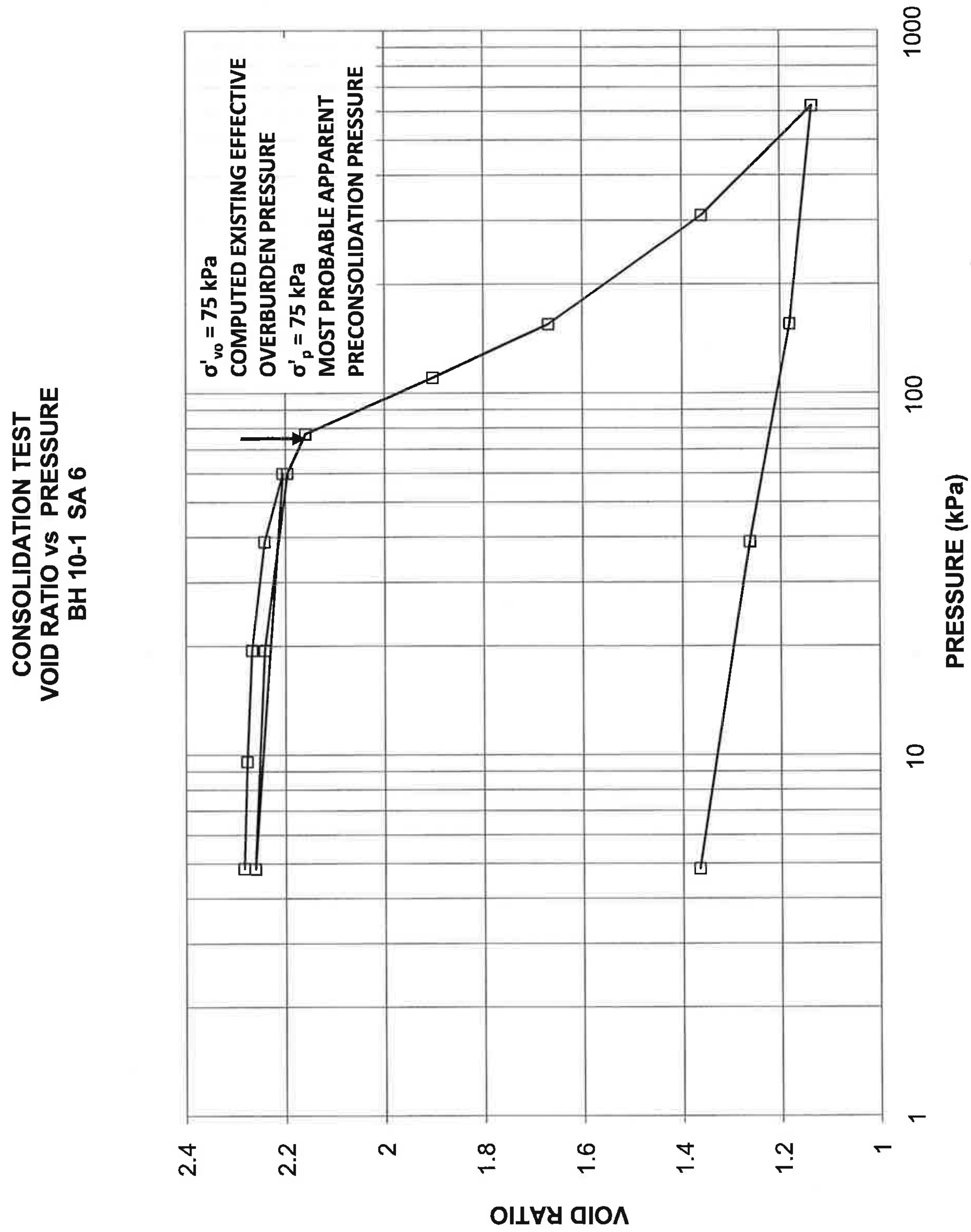
Ontario

PLASTICITY CHART
Unweathered Clay

FIG No. 5

Project No. 09-1121-1004/3000

CONSOLIDATION TEST VOID RATIO VS LOG PRESSURE	FIGURE 6



Project No. 09-1121-1004

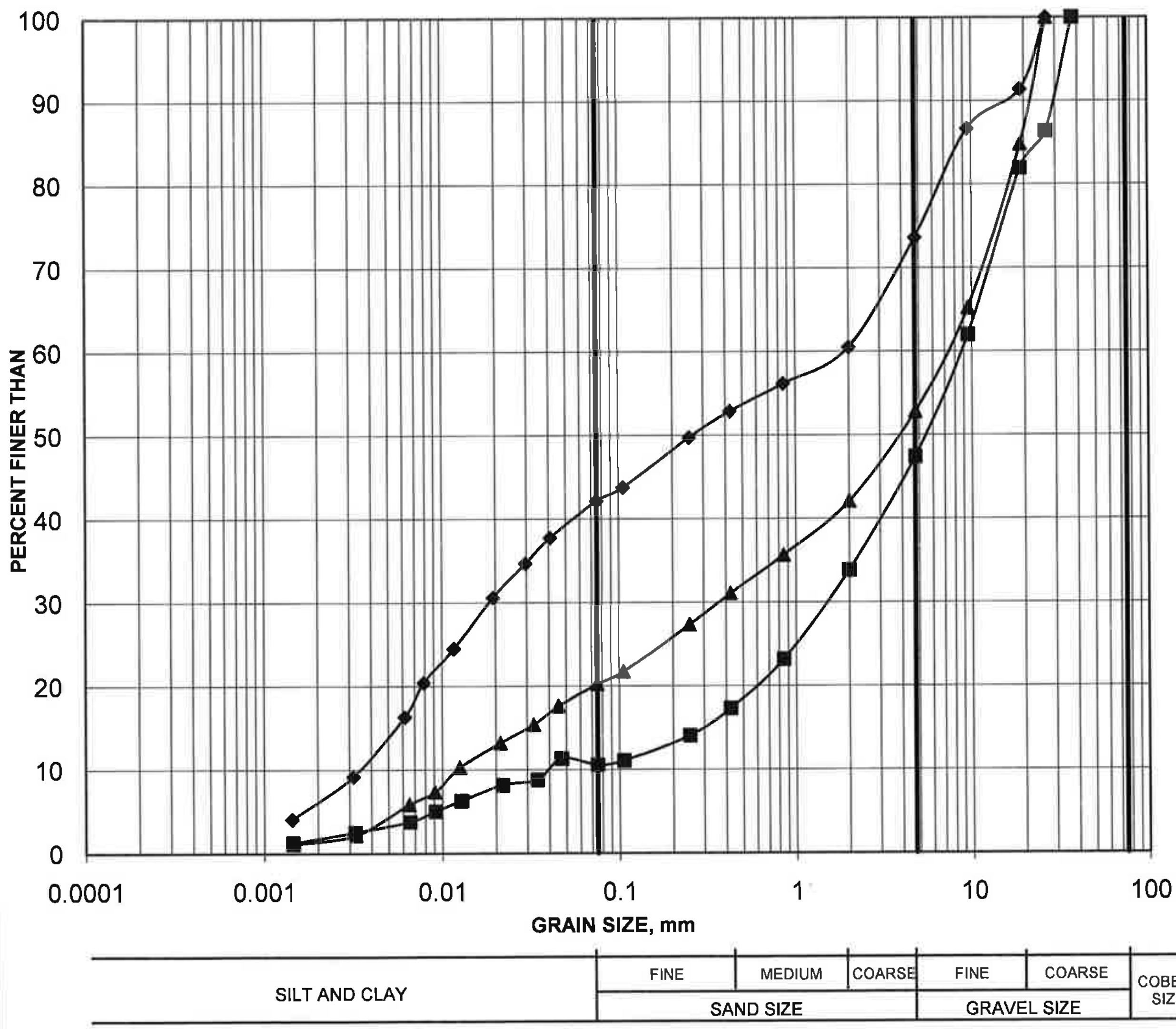
Prepared By: LFG

Golder Associates

Checked By: RO

GRAIN SIZE DISTRIBUTION	FIGURE 7

SILTY SAND AND GRAVEL (TILL)



Borehole	Sample	Depth (m)
10-2	9A	11.13-11.28
10-3	12	12.20-12.80
10-3	14	13.72-14.33

Project: 09-1121-1004/3000 **Golder Associates**

Created by:	MI
Checked by:	CNM

APPENDIX A

Sample Non-Standard Special Provisions

CULVERT SETTLEMENT – Item No.

Non-Standard Special Provision

The clay subgrade soil at this site is normally consolidated; therefore small changes in loading can result in significant settlement magnitudes. Therefore additional increases in loading during construction should be avoided such as over-excavation at the subgrade level which would result in increased thicknesses of granular material or increased concrete thicknesses.

Basis of Payment

Payment at the contract price for the above tender item shall include full compensation for all labour and materials to complete the work.

END OF SECTION

GROUND WATER AND SURFACE WATER CONTROL – Item No.

Non-Standard Special Provision

Control of the surface water and groundwater will be necessary for the construction of the culvert replacement, to allow excavation and foundation construction to be carried out in dry conditions.

The surface water flow could be passed through the culvert area by means of a temporary pipe, or diverted by pumping from behind a temporary cofferdam to the adjacent culvert crossing to the east. Surface water should be directed away from the excavation areas to prevent ponding of water that could result in disturbance and weakening of the sensitive clay subgrade soils.

Basis of Payment

Payment at the contract price for the above tender item shall include full compensation for all labour and materials to complete the work.

END OF SECTION

EXCAVATION SIDE SLOPES – Item No.

Non-Standard Special Provision

Temporary excavations for the culvert replacement will be made through the existing fill and are expected to terminate or extend into the soft to firm unweathered clay deposit. Excavation works must be carried out in accordance with the guidelines outlined in the Occupational Health and Safety Act (OHSA) and Regulations for Construction Projects. The existing fill would be classified as Type 3 soil and the underlying soft to firm clay would be classified as Type 4 soil, based on the OHSA. According to OHSA, excavations that extend to, or into, Type 4 soils should be made with side slopes no steeper than 3 horizontal to 1 vertical (3H:1V). However, due to the nature of the underlying soft sensitive clay deposit at this site flatter excavation side slopes may be necessary.

Basis of Payment

Payment at the contract price for the above tender item shall include full compensation for all labour and materials to complete the work.

END OF SECTION

BASAL INSTABILITY OF SHORED EXCAVATIONS – Item No.

Non-Standard Special Provision

The shoring system for the culvert replacement must consider the soft clay deposit at depth and the potential for basal instability of the excavation. A basal instability failure could lead to the flow of sheared/disturbed clay into the excavation, significant loss of ground (settlement and ground slumping) behind the sheeting, and possible collapse of the shoring system. Therefore, the shoring system will need to extend below the excavation floor level into the native glacial till to prevent basal instability. In addition, the design of the sheeting projection would also need to resist the lateral loading imposed by the clay. This may require a very heavy/strong sheeting section.

Basis of Payment

Payment at the contract price for the above tender item shall include full compensation for all labour and materials to complete the work.

END OF SECTION

SUBGRADE PROTECTION – Item No.

Non-Standard Special Provision

The subgrade for the culvert foundations will be very susceptible to disturbance from construction traffic and ponded water. Following inspection and approval of the prepared subgrade, a 300 mm thick layer of OPSS Granular A shall be placed on the foundation subgrade for a box culvert and 500 mm thick layer of OPSS Granular A shall be place on the foundation subgrade for a pipe arch.

The excavation for the bedding should be made using a smooth bladed bucket and the bedding should be compacted using light 'walk behind' compaction equipment in loose lifts not less than 200 mm in thickness to 95 percent of the material's Standard Proctor maximum dry density in accordance with SP105S10.

Construction traffic should not be permitted to travel on the subgrade.

Basis of Payment

Payment at the contract price for the above tender item shall include full compensation for all labour and materials to complete the work.

END OF SECTION