OCTOBER 2009

FOUNDATION INVESTIGATION AND DESIGN REPORT

EXTENSION OF CULVERT 11-338/C
HIGHWAY 62 REHABILITATION
FROM HICKEY ROAD WEST TO HIGHWAY 127, MAYNOOTH
MINISTRY OF TRANSPORTATION, ONTARIO
G.W.P. 187-99-00

Submitted to:
GENIVAR
69 Cleak Avenue, Box 187
Bancroft, Ontario
K0L 1C0

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REPORT

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FOUNDATION INVESTIGATION REPORT
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1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by GENIVAR on behalf of the Ministry of Transportation, Ontario (MTO) to provide foundation engineering services for the proposed rehabilitation of Highway 62 from 13.7 km north of Bancroft at Hickey Road West to Highway 127 in Maynooth, Ontario.

This report addresses the foundation investigation carried out for the east extension of Culvert 11-338/C. The location of this culvert site is shown in the key plan on Drawing 1.

The terms of reference for the foundation engineering services are provided in the Request for Proposal for MTO Assignment No. 4008-E-0004, dated July 14, 2008, and the scope of work is outlined in Section 6.8 of GENIVAR's *Technical Proposal* for this assignment (Golder's Proposal No. P81-1431, dated July 30, 2008). The work has been carried out in accordance with Golder's Supplemental Specialty Quality Control Plan for foundation engineering services for this project, dated February 2009.

2.0 SITE DESCRIPTION

The existing culvert (MTO Structure Site No. 11-338/C) passes beneath Highway 62 approximately 50 m south of North Baptiste Lake Road in the Township of Monteagle, in the County of Hastings.

In general, the terrain in the area of the culvert is relatively flat, poorly drained and swampy. The overall surface topography along Highway 62 slopes downwards towards the south. The road surface in the immediate vicinity of the culvert is at about Elevation 418.3 m, and the natural ground surface in the area of the culvert is at about Elevation 415.3 m.

The existing Highway 62 embankment is about 3 m high relative to the surrounding natural ground surface in the area of this culvert. The existing embankment slopes on the east and west sides of Highway 62 are oriented at approximately 1.7 to 2.5 horizontal to 1 vertical (1.7H:1V to 2.5H:1V) in the vicinity of the culvert. No evidence of distress or instability was observed on the highway embankment shoulders or side slopes at the time of the borehole investigations at the culvert site, with the exception of slight surficial erosion observed on the east side of the embankment. The existing Highway 62 pavement over the culvert area was observed to be in fair condition, with few longitudinal and transverse cracks observed at the time of the borehole investigation.

The existing culvert is a rigid frame, open footing structure, approximately 20 m long, with a span and height of 3.7 m and 1.55 m, respectively. Twin corrugated steel pipe (CSP) culvert extensions, each 1.2 m in diameter and approximately 5 m in length, are present at the east end of the concrete culvert. According to the *Drainage Review Report* completed by Totten Sims Hubicki Associates (TSH), dated October 2007, the twin CSP culvert extensions were not suitably sized to convey flows for a ten-year (or greater) storm event. The field inspection conducted by TSH indicated the CSP culverts were in fair condition with some rusting, and the concrete culvert was in good condition, at the time of the assessment.

3.0 INVESTIGATION PROCEDURES

The borehole investigation for the east extension of Culvert 11-338/C was carried out on May 5 and 6, 2009, at which time two boreholes were advanced to investigate the subsurface conditions for the east culvert extension/ replacement. Borehole 09-101 was drilled through the east shoulder of Highway 62 and extended to a depth of 12.5 m, and Borehole 09-102 was drilled at the east toe of the Highway 62 embankment and extended to a depth of 9.8 m. The borehole locations are shown on Drawing 1.

The boreholes were drilled using a track-mounted D-55 drill rig, supplied and operated by Walker Drilling Ltd. of Utopia, Ontario. The boreholes were advanced through the overburden using hollow stem augers, and soil samples were obtained at 0.75 m and 1.5 m intervals of depth, using a 50 mm outer diameter split-spoon sampler driven by an automatic hammer in accordance with Standard Penetration Test (SPT) procedures (ASTM D1586-99).

The groundwater conditions in the open boreholes were observed during the drilling operations. Borehole 09-101 was backfilled with bentonite upon completion, in accordance with Ontario Regulation 903 (as amended by Ontario Regulation 372). A standpipe piezometer was installed in Borehole 09-102 to permit monitoring of the groundwater level at the site. The piezometer consisted of 30 mm diameter PVC pipe, with a slotted screen sealed within a sand filter pack at a selected depth interval within the borehole. Above the sand filter pack and piezometer screen, the annulus surrounding the piezometer pipe was backfilled to the ground surface with bentonite pellets/grout. The piezometer installation details and water level readings are indicated on the record for Borehole 09-102 contained in Appendix A. This standpipe piezometer was decommissioned on June 19, 2009, in accordance with Ontario Regulation 903.

The field work was observed by a member of Golder's technical staff, who located the boreholes, arranged for the clearance of underground services, observed the drilling, sampling and in situ testing operations, logged the boreholes, and examined the recovered samples. The samples were identified in the field, placed in appropriate containers, labelled and transported to Golder's Mississauga geotechnical laboratory where the samples underwent further detailed visual examination and geotechnical classification testing (water contents and grain size distribution tests). All of the laboratory tests were carried out to MTO and/or ASTM Standards, as appropriate.

The borehole locations were measured in the field relative to existing site features, and the ground surface elevation at the borehole locations was surveyed relative to existing site features and then converted to geodetic elevation based on the survey information for this area. The borehole locations shown on Drawing 1 and on the borehole records are given relative to MTM NAD 83 northing and easting coordinates, and the ground surface elevations are referenced to the geodetic datum.

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

The study area for this assignment lies within the physiographic region known as the Algonquin Highlands, as delineated in *The Physiography of Southern Ontario* (Chapman and Putnam, 1984). The Algonquin Highlands region is characterized by frequent outcrops of granite and other strong Precambrian bedrock, which can extend as high as 160 m above the surrounding land. The thickness of soils over the bedrock can vary greatly over short distances, with many of the valleys between the bedrock outcrops floored with outwashed sand, silt and gravel. Several areas within this region have deeper deposits of glacial till with few bedrock outcrops.

4.2 Subsurface Conditions

The detailed subsurface soil and groundwater conditions as encountered in the boreholes advanced at this site are shown on the borehole records contained in Appendix A, and the results of the laboratory tests carried out on selected soil samples are shown on these borehole records as well as in Appendix B. The stratigraphic boundaries shown on the borehole records are inferred from non-continuous sampling, observations of drilling progress and the results of Standard Penetration Tests. These boundaries, therefore, represent transitions between soil types rather than exact planes of geological change.

The interpreted stratigraphic conditions at the culvert extension area are shown on Drawing 1. This stratigraphic profile represents a simplification of the subsurface conditions as encountered in the boreholes. Variation in the stratigraphic boundaries and properties of the soil deposits will occur between and beyond the borehole locations.

In general, the subsurface conditions at the site of the proposed eastward culvert extension/replacement consist of approximately 3.2 m of existing sand to sand and gravel embankment fill in Borehole 09-101 (drilled through the Highway 62 embankment shoulder), underlain by strata of compact to dense sandy silt to silt, sand and gravel, and sand. The boreholes were terminated in deposits of hard clayey silt and dense silty sand. A more detailed description of the subsurface conditions encountered in the boreholes is provided in the following sections.

4.2.1 Sand and Gravel to Sand Fill

Fill was encountered below the asphalt roadway surface in Borehole 09-101, which was advanced through the east shoulder of the existing Highway 62 embankment, extending from ground surface to a depth of 3.2 m (Elevation 414.8 m).

The encountered fill varies in composition from sand and gravel containing trace to some silt, to sand containing trace silt; cobbles were observed in the lower layer of the sand and gravel, between a depth of about 0.8 m and 3.2 m (Elevation 417.2 m to 414.8 m). The result of a grain size distribution test completed on one sample of the sand and gravel fill is shown on Figure B1, contained in Appendix B. This sample, obtained with a standard 50 mm outside diameter split-spoon sampler, is not representative of the coarse gravel and cobbles that are present within the fill.

The measured Standard Penetration Test (SPT) “N” values within the fill range from 11 to 19 blows per 0.3 m of penetration, indicating that the fill has a compact relative density.

4.2.2 Topsoil

Approximately 0.6 m of topsoil was encountered immediately below ground surface in Borehole 09-102, which was located east of the east toe of the Highway 62 embankment, near the existing CSP culvert extensions.

4.2.3 Sandy Silt to Silt

A layer of sandy silt to silt was encountered below the embankment fill in Borehole 09-101 and below the topsoil in Borehole 09-102. This layer was about 0.6 m thick, extending from Elevation 414.8 m to 414.2 m, in Borehole 09-101, and about 1.9 m thick, extending from about Elevation 414.5 m to 412.6 m, in Borehole 09-102.

The deposit varies from sandy silt containing trace clay and trace gravel, to silt containing trace to some sand, trace clay and trace gravel; organic materials and rootlets were also observed in the recovered samples. The results of grain size distribution tests completed on two selected samples of the sandy silt to silt are shown on Figure B2, contained in Appendix B.

The measured SPT ‘N’ values in the sandy silt to silt range from 11 to 14 blows per 0.3 m of penetration, indicating that this stratum has a compact relative density.

4.2.4 Sand and Gravel

A 2.1 to 2.3 m thick layer of sand and gravel was encountered below the sandy silt to silt in the boreholes, extending from a depth of 3.8 m to 6.1 m (Elevation 414.2 m to 411.9 m) in Borehole 09-101, and from a depth of 2.5 m to 4.6 m (Elevation 412.6 m to 410.5 m) in Borehole 09-102.

The sand and gravel contains trace to some silt and trace clay; cobbles were also observed in the soil cuttings during drilling through this layer, and were inferred based on a high SPT 'N' value (30 blows for 0.08 m of penetration) for one sample. The results of grain size distribution tests completed on two selected samples of the sand and gravel are shown on Figure B3, contained in Appendix B.

The measured SPT 'N' values in the sand and gravel range from 10 to 46 blows per 0.3 m of penetration, indicating that this deposit has a compact to dense relative density.

4.2.5 Sand to Silty Sand

A 4.1 m to 5.2 m thick layer of sand to silty sand was encountered below the sand and gravel in the boreholes, extending from a depth of 6.1 m to 10.2 m (Elevation 411.9 m to 407.8 m) in Borehole 09-101, and from a depth of 4.6 m to 9.8 m (Elevation 410.5 m to 405.3 m) in Borehole 09-102. The borehole was terminated within this deposit in Borehole 09-102.

The deposit varies in composition from sand containing some gravel, trace silt and trace clay, to silty sand containing some gravel and trace clay; this stratum also contains cobbles, which were both observed in the auger cuttings and inferred based on observations of drilling progress. The results of grain size distribution tests completed on three selected samples of the sand to silty sand are shown on Figure B4, contained in Appendix B.

The measured SPT 'N' values in the sand to silty sand range from 10 to 32 blows per 0.3 m of penetration, indicating that this deposit has a compact to dense relative density.

4.2.6 Silt to Clayey Silt Till

A till deposit was encountered below the sand to silty sand in Borehole 09-101. The surface of the till was encountered at a depth of 10.2 m (Elevation 407.8 m), and the borehole was terminated within the deposit after penetrating it for 2.3 m (Elevation 405.5 m).

The till deposit varies in composition from non-plastic (cohesionless) silt containing some sand, trace gravel and trace to some clay, to low plasticity clayey silt containing some sand and trace gravel; cobbles were observed or inferred within the till deposit. The result of a grain size distribution test completed on one selected sample of the till is shown on Figure B5, contained in Appendix B. Atterberg limits testing was carried out on one sample of the till, and measured a plastic limit of 17 per cent, a liquid limit of 20 per cent, and a plasticity index of 3 per cent. This result, which is plotted on a plasticity chart on Figure B6 contained in Appendix B, confirms that the tested sample is a non-plastic or low plasticity silt till.

The measured SPT 'N' values in the silt to clayey silt till deposit were 44 and 75 blows per 0.3 m of penetration, indicating that this deposit has a dense relative density/hard consistency.

4.3 Groundwater Conditions

A standpipe piezometer was installed in Borehole 09-102 to permit monitoring of the water level at this site. Details of the piezometer installation are shown on the borehole record contained in Appendix A. The groundwater level measured in the piezometer installation is summarized in the following table.

Borehole No.	Ground Surface Elevation	Depth to Groundwater Level	Groundwater Elevation (m)	Date of Measurement
09-102	415.1 m	0.5 m	414.6 m	June 19, 2009

The water level measured in the piezometer is near the natural ground surface, similar to that encountered in Borehole 09-101 during drilling (encountered at a depth of 3.0 m below the Highway 62 shoulder grade, corresponding to Elevation 415.0 m). The groundwater level in the area should be expected to be subject to seasonal fluctuations and precipitation events, and should be expected to be higher during wet periods of the year.

5.0 CLOSURE

This Foundation Investigation Report was prepared by Mr. Jordan Lee and reviewed by Ms. Lisa Coyne, P.Eng., a geotechnical engineer and Associate with Golder. Mr. Fin Heffernan, P.Eng., Golder's Designated MTO Contact for this project, conducted an independent quality control review of the report.

for: Jordan T.W. Lee, B.A.Sc.
Geotechnical Group

Lisa C. Coyne, P. Eng.
Associate, Geotechnical Engineer

LICENSED PROFESSIONAL ENGINEER
L. C. COYNE
23 OCT 2009
PROVINCE OF ONTARIO

REGISTERED PROFESSIONAL ENGINEER
F. J. HEFFERNAN
PROVINCE OF ONTARIO

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

JTWL/LCC/FJH/jl/lcc

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PART B

FOUNDATION DESIGN REPORT
EXTENSION OF CULVERT 11-338/C
HIGHWAY 62 REHABILITATION
FROM HICKEY ROAD WEST TO HIGHWAY 127, MAYNOOTH
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6.0 FOUNDATION ENGINEERING RECOMMENDATIONS

6.1 General

This section of the report provides foundation design recommendations for the detail design of the replacement of the existing CSP culvert extensions at the east end of Culvert 11-338/C on Highway 62, located approximately 50 m south of North Baptiste Lake Road in the Township of Monteagle, in the County of Hastings. The recommendations are based on interpretation of the factual data obtained from the boreholes advanced during this subsurface investigation. The discussion and recommendations presented are intended to provide the designers with sufficient information to assess the feasible foundation alternatives and to carry out the design of the structure foundations.

Where comments are made on construction, they are provided to highlight those aspects that could affect the design of the project, and for which special provisions or operational constraints may be required in the contract documents. Those requiring information on the aspects of construction should make their own interpretation of the factual information provided as such interpretation may affect equipment selection, proposed construction methods, scheduling and the like.

6.2 Foundation Options

The existing culvert consists of a 3.7 m wide by 1.55 m high rigid frame open footing structure, with twin 5 m long, 1.2 m diameter CSP culvert extensions on the east end. The existing concrete structure is in good condition, but the CSP culvert extensions on the east end do not satisfy drainage and hydrology requirements and require replacement with a concrete culvert extension.

The creek channel and culvert invert level are at approximately Elevation 414.2 m at the upstream (west) end of the culvert, and approximately Elevation 414.0 m at the downstream (east) end of the existing culvert.

Either a box culvert or an “open footing” (shallow foundation) culvert is feasible for the replacement of the existing CSP extensions at the east end of this culvert; deep foundations are not required for the extension/ replacement, as shallow foundations will provide sufficient bearing resistance and acceptable settlement performance. Both pre-cast concrete elements (box culvert segments or footing elements) and cast-in-place concrete elements are also feasible from a foundations perspective.

The advantages and disadvantages associated with both an open footing and a box culvert extension/ replacement are summarized in Table 1 following the text of this report; this table also includes comments on the use of pre-cast concrete box culvert segments or pre-cast concrete footing sections versus cast-in-place concrete. From a foundations perspective, a pre-cast box culvert extension is preferred over a cast-in-place open footing extension in terms of minimizing the depth of excavation and groundwater control requirements compared with open footings; in addition, pre-cast box culvert segments can often be installed more expeditiously than cast-in-place open footing culverts, resulting in shorter durations for dewatering and surface water pumping. However, a box culvert extension/replacement may not satisfy fisheries requirements, or may result in compatibility problems with the existing open footing culvert channel. Since both foundations options are geotechnically feasible, an open footing extension/replacement option is considered an acceptable alternative to a box culvert.

Recommendations for both a box culvert extension and a shallow foundation (open footing) culvert extension are provided in the following sections.

6.3 Box Culvert Extension Replacement

6.3.1 Founding Elevation

A box culvert extension/replacement can be founded below the topsoil and existing fill, supported on the sand and gravel and the sandy silt to silt deposits. The invert/creek bed of the existing culvert, and that for the proposed extension/replacement, is at approximately Elevation 414.0 m. Assuming that the surface of the concrete base slab for a box culvert extension would be constructed to match the existing creek bed, and assuming that the base slab for a box culvert extension has a thickness of 300 mm, the base slab for the east extension/replacement would be founded at Elevation 413.7 m on dense sand and gravel and compact sandy silt to silt. It is recommended that a minimum 150 mm thick layer of Ontario Provincial Standard Specification (OPSS) 1010 Granular A be placed below the base slab on the subgrade to form a working/bedding layer for the box culvert segments, and to limit the degradation of the siltier portion of the subgrade.

Excavations for the box culvert extension would extend approximately 1 m to 1.4 m below the groundwater level at the site, as measured immediately after completion of Borehole 09-101, and approximately six weeks after completion of drilling in the standpipe piezometer in Borehole 09-102. Groundwater and surface water control will be required for construction of the box culvert; further discussion on this aspect is provided in Section 6.8 (Construction Considerations).

As noted above, the siltier portion of the subgrade will be susceptible to degradation on exposure to water and construction traffic. As an alternative to the placement of a minimum 150 mm thick layer of Granular A, a 100 mm thick layer of mass concrete could be placed on the subgrade within the culvert extension footprint to form a working mat for construction of the culvert extension, to protect the subgrade from degradation. This aspect is discussed further in Section 6.8 (Construction Considerations). In this case, a 75 mm thick layer of OPSS 1010 Granular A or concrete fine aggregate (meeting the gradation requirements set out in OPSS 1002) should be placed on top of the concrete mat to provide a “levelling pad” for the box culvert extension replacement.

6.3.2 Geotechnical Resistance

A box culvert extension/replacement placed on the properly prepared subgrade, at or below the elevation identified above, should be designed based on the following factored geotechnical resistance at Ultimate Limit States (ULS) and geotechnical resistance at Serviceability Limit States (SLS):

Culvert Span	Factored Geotechnical Resistance at ULS	Geotechnical Resistance at SLS*
3.7 m	200 kPa	150 kPa

* For 25 mm of total settlement, assuming the box culvert extension has a width of approximately 3.7 m to match the existing concrete culvert.

The ULS resistance and settlement are dependent on the footing size, configuration and applied loads; the geotechnical resistances should, therefore, be reviewed if the culvert span or founding elevation differs significantly from those given above.

The geotechnical resistances provided above are based on loading 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).*

6.3.3 Resistance to Lateral Loads

Resistance to lateral forces / sliding resistance between the base slab for the culvert extension/replacement and the subgrade should be calculated in accordance with Section 6.7.5 of the CHBDC. For design, the coefficient of friction (tan δ) between a pre-cast concrete box culvert extension/replacement and the granular bedding should be taken as 0.5. If used, the coefficient of friction (tan φ') between the cast-in-place concrete mat and the underlying sand and gravel or sandy silt to silt deposits should be taken as 0.6. These values are unfactored; in accordance with the *CHBDC*, a factor of 0.8 is to be applied in calculating the horizontal resistance.

6.4 Open Footing Culvert Extension Replacement

6.4.1 Founding Elevation

An open footing culvert extension/replacement, and any associated retaining walls, can be supported on strip footings founded below the topsoil and existing fill on the sand and gravel deposit. Strip footings should be founded at a minimum depth of 1.8 m below the lowest surrounding grade, to provide adequate protection against frost penetration. The following founding elevation is recommended for strip footings for support of the east culvert extension replacement and any associated retaining walls:

Channel Invert Elevation	Maximum Founding Elevation
414.0 m	412.2 m

The maximum founding level identified above will require excavation to a depth of up to about 2.4 m below the groundwater level at the site, as measured immediately after completion of Borehole 09-101, and approximately six weeks after completion of drilling in the standpipe piezometer in Borehole 09-102. Groundwater and surface water control will be required for construction of the footings; further discussion on this aspect is provided in Section 6.8 (Construction Considerations).

The subgrade for the culvert extension footings will be susceptible to loosening and degradation on exposure to water and construction traffic. It is recommended that a 100 mm thick layer of mass concrete be placed on the subgrade to form a working mat for construction of the extension footings, to protect the subgrade from degradation; this aspect is discussed further in Section 6.8.

6.4.2 Geotechnical Resistance

Strip footings placed on the properly prepared subgrade, at or below the maximum founding elevation identified above, should be designed based on the following factored geotechnical resistances at ULS and geotechnical resistances at SLS.

Footing Width	Factored Geotechnical Resistance at ULS	Geotechnical Resistance at SLS*
0.6 m	300 kPa	250 kPa
0.9 m	325 kPa	250 kPa

* For 25 mm of total settlement for the given footing width.

The ULS resistance and settlement are dependent on the footing size, configuration and applied loads; the geotechnical resistances should, therefore, be reviewed if the selected footing width or founding elevation differs significantly from those given above.

The geotechnical resistances provided above 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 *CHBDC.*

6.4.3 Resistance to Lateral Loads

Resistance to lateral forces / sliding resistance between the concrete footings for the culvert extension replacement and the subgrade should be calculated in accordance with Section 6.7.5 of the *CHBDC*. The following values for the coefficient of friction, tan ϕ' or tan δ, can be used for cast-in-place and pre-cast concrete footings founded on a mass concrete mat and/or on the properly prepared, compact sand and gravel deposit:

Footing Type	Coefficient of Friction
Cast-in-place concrete footing on mass concrete mat	tan δ = 0.55
Cast-in-place concrete footing on compact sand and gravel	tan ϕ' = 0.60
Pre-cast concrete footing on compact sand and gravel	tan δ = 0.45

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

6.5 Culvert Bedding, Backfill and Erosion Protection

For a box culvert extension, the bedding levelling pad and backfill requirements should be in accordance with OPSS 422 for pre-cast rigid frame culverts. Box culvert extensions or replacements should be provided with at least 150 mm of OPSS 1010 Granular A material for bedding purposes.

Backfill and cover for concrete culverts should be completed in accordance with Ontario Provincial Standard Drawing (OPSD) 803.010. Backfill to both box culvert and open footing culvert walls should consist of granular fill meeting the requirements of OPSS 1010 Granular A or Granular B Type II, but with less than 5 per cent passing the No. 200 sieve. The backfill should be placed and compacted in accordance with MTO's Special Provision SP105S10. 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. The culvert extension should be designed for the full overburden pressure and live load, assuming an embankment fill unit weight of 22 kN/m^3 for Granular A, and 21 kN/m^3 for Granular B Type II or select earth fill above and/or surrounding the culvert.

If the creek flow velocities are sufficiently high, provision should be made for scour and erosion protection (suitable non-woven geotextiles and/or rip-rap) in the culvert extension area. 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 end of the culvert extension. If a clay seal is adopted, the clay material should meet the requirements of OPSS 1205. The clay seal 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 opening, and a minimum vertical height equivalent to the high water level including treatment of the adjacent side slopes. Alternatively, a clay blanket may be constructed, extending upstream to a distance equal to three times the culvert height, and extending along the adjacent side slopes to a height of two times the culvert height or the high water level, whichever is higher.

The requirements for and design of erosion protection measures for the inlet of the culvert extension/ replacement should be assessed by the hydraulic design engineer. As a minimum, rip-rap treatment for the culvert extension outlet should be consistent with the standard presented in OPSD 810.010 Rip-Rap Treatment Type A. Erosion protection for the existing culvert inlet (west end) should follow the standard presented in OPSD 810.010, similar to Rip-Rap Treatment Type A 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, including the creek side slopes and embankment fill slope adjacent to the culverts.

6.6 Lateral Earth Pressures for Design

The lateral earth pressures acting on the culvert extension walls and on any associated head walls/retaining 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 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 1010 Granular A or Granular B Type II but with less than 5 percent passing the 200 sieve should be used as backfill behind the 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 sub drains and frost taper should be in accordance with OPSD 3101.150 and OPSD 3121.150.

- 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. 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: Active, K_a At rest, K_o	 0.33 0.50

- For Case B, where the pressures are based on OPSS 1010 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: Active, K_a At rest, K_o	 0.27 0.43	 0.27 0.43

Where the culvert wall support does not allow lateral yielding, at-rest earth pressures should be assumed for the geotechnical design. Where associated retaining wall support allows lateral yielding of the stem, active earth pressures should be used in the geotechnical design of the structure. The movement required to allow active pressures to develop within the backfill, and thereby assume an unrestrained structure for design, should be calculated in accordance with Section C6.9.1 and Table C6.6 of the *Commentary* to the *CHBDC*.

6.6.1 Seismic Considerations

Seismic (earthquake) loading must be considered in the design in accordance with Section 4.6.4 of *CHBDC,* as significant seismic loading will result in increased lateral earth pressures acting on the abutment stem and retaining 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:

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

where
- K is either the static active earth pressure coefficient (K_a) or the static at rest earth pressure coefficient (K_o);
- K_{AE} is the seismic active earth pressure coefficient;
- γ' is the effective unit weight of the soil (kN/m^3)
  - taken as soil unit weights given above for fill materials
  - taken as 20 kN/m^3 for the native materials
- 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 1, and the site specific zonal acceleration ratio for the Bancroft area is 0.10. For the thicknesses and type of overburden soils at this site, a site coefficient of 1.0 and an amplification factor of 1.33 are recommended. Therefore, the recommended ground surface acceleration is 0.133g.

The seismic lateral earth pressure coefficients given below have been derived based on a design zonal acceleration ratio of A = 0.133. These coefficients have been determined in accordance with Sections 4.6.4 and C4.6.4 of the *CHBDC* and its *Commentary,* 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
Yielding Wall	0.32	0.29	0.29
Non-Yielding Wall	0.44	0.40	0.40

6.7 Settlement and Culvert Connection Requirements

If additional fill is placed to widen Highway 62, some settlement of the foundation soils will occur below the culvert extension/replacement. Assuming placement of a 3 m thickness of fill for the widening, it is predicted that approximately 10 mm of settlement will occur under the widened embankment shoulder, decreasing to less than 5 mm of settlement below the existing embankment shoulder and new embankment toe. This settlement will be completed relatively quickly during and immediately following any embankment widening work. It is recommended that the structural designer determine, based on this predicted magnitude of settlement and the actual change in embankment geometry and loading, whether a rigid connection or an articulation is required between the existing concrete open footing culvert and the culvert extension/replacement.

The settlement analyses were carried out using the commercially-available program Unisettle (Version 3.0), using the elastic deformation moduli given below based on correlations (Bowles, 1982) with the SPT "N" values and engineering judgement from experience with similar soils in this region of Ontario.

Soil Unit	***Bulk Unit Weight***	***Elastic Modulus***
Embankment fill	20 kN/m^3	-
Compact sandy silt to silt	20 kN/m^3	5-10 MPa
Compact to dense sand and gravel	21 kN/m^3	30-40 MPa
Compact sand	20 kN/m^3	15-20 MPa

6.8 Construction Considerations

6.8.1 Groundwater and Surface Water Control for Foundation Excavation

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

Depending on the creek 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 foundation subgrade soils; further discussion on this aspect is provided in Section 6.8.3.

As discussed previously, excavations will extend approximately 1 m to 1.4 m below the groundwater level for a box culvert extension, and approximately 2.4 m below the groundwater level for an open footing extension. Appropriate dewatering of the water-bearing sandy silt to silt and sand and gravel deposits will be required to draw the water level down to at least 0.3 m below the founding level for the culvert extension replacement. A wellpoint system, designed and installed by a specialist dewatering contractor, is expected to be necessary for dewatering at this site. As discussed in the next section, an interlocking sheetpile system could also be used at the site to control groundwater seepage through the excavation side walls, in addition to providing temporary excavation support where needed. A sheetpile system would still have to be supplemented with wellpoints or educators to draw the groundwater level down to below the excavation base.

It is recommended that a Non-Standard Special Provision (NSSP) be included in the Contract Documents to warn the Contractor of the soil conditions and the requirement for design and installation of a groundwater control system for this culvert extension replacement. An example NSSP is given in Appendix C.

6.8.2 Excavation and Temporary Roadway Protection

Temporary excavations for the culvert extension/replacement will be made through the existing embankment fill and compact sandy silt to silt, generally terminating in the compact to dense sand and gravel deposit. Excavation works must be carried out in accordance with the guidelines outlined in the Occupational Health and Safety Act and Regulations for Construction Projects. The existing fill would be classified as Type 3 soil, according to the OHSA, assuming that proper groundwater control is in place to dewater the cohesionless soil deposits prior to excavation. Where space permits, temporary open-cut excavations through these materials should be made with side slopes formed no steeper than 1H:1V, assuming proper groundwater control is in place to lower the groundwater level to below the excavation base.

It is expected that a temporary protection system will be required along the east side of Highway 62 to facilitate the culvert extension/replacement. The temporary excavation support systems should be designed and constructed in accordance with MTO's Special Provision 105S19. The lateral movement of the temporary shoring system should meet Performance Level 2 as specified in SP105S19, provided that any adjacent utilities can tolerate this magnitude of deformation.

It is considered that a driven, interlocking sheetpile system would be suitable, based on the subsurface soil and groundwater conditions encountered at the site, as a sheetpile system would contribute to both ground and groundwater control, particularly if a closed sheetpile system is used. Cobbles are present within the fill and native soils, which could cause problems in driving the sheetpiles. A soldier pile and lagging system (using rakers to provide lateral support as necessary) could also be adopted for the temporary protection system; however, groundwater control would be critical to control groundwater seepage and associated loss of soil particles through the lagging boards.

The selection and design of the protection system will be the responsibility of the Contractor. However, conceptually, it is anticipated that sheetpiles or soldier piles through the existing Highway 62 embankment fill would have to extend to a depth of approximately 2 m to 3 m below the base of strip footing excavations.

6.8.3 Subgrade Preparation

The soils exposed at the footing subgrade level will be susceptible to disturbance from construction traffic and/or ponded water. To limit this degradation, it is recommended that a working mat of mass concrete be placed on the subgrade within four hours after preparation, inspection and approval of the footing subgrade. This requirement can be addressed with a note on the General Arrangement drawing and/or with an NSSP. A sample NSSP is included in Appendix C.

7.0 CLOSURE

This report was prepared by Mr. Jordan Lee, B.A.Sc., and reviewed by Ms. Lisa Coyne, P.Eng., a geotechnical engineer and Associate with Golder. Mr. Fin Heffernan, P.Eng., Golder's Designated MTO Contact for this project, conducted an independent quality control review of the report.

for: Jordan T.W. Lee, B.A.Sc.
Geotechnical Group

Lisa C. Coyne, P. Eng.
Associate, Geotechnical Engineer

LICENSED PROFESSIONAL ENGINEER
L. C. COYNE
23OCT2009
PROVINCE OF ONTARIO

REGISTERED PROFESSIONAL ENGINEER
F. A. HEFFERNAN
PROVINCE OF ONTARIO

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

JTWL/LCC/FJH/jl/lcc

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TABLE 1
COMPARISON OF FOUNDATION ALTERNATIVES – EAST EXTENSION OF CULVERT 11-338/C
HIGHWAY 62 REHABILITATION FROM HICKEY ROAD WEST TO HIGHWAY 127, MAYNOOTH
G.W.P. 187-99-00

Option	Advantages	Disadvantages	Risks/Consequences
Box culvert extension replacement	• Minimizes depth of excavation and dewatering requirements compared to open footing option • Pre-cast box sections may allow faster construction than cast-in-place open footings, with shorter time duration for dewatering and surface water pumping	• Excavation would extend to a depth of about 1 m to 1.4 m below groundwater level, and dewatering would be required • Compatibility of box culvert extension with existing open footing culvert channel; box culvert may not satisfy fisheries requirements	• Small risk related to effective dewatering in fine-grained sand silt to silt deposits at this site
Open footing culvert extension replacement	• Matches existing culvert foundation type and would satisfy any fisheries requirements, if applicable • May be feasible to build culvert extension/replacement on pre-cast footing sections, to accelerate construction schedule and reduce time for dewatering and surface water pumping	• Excavation would extend to a depth of about 2.4 m below groundwater level; dewatering would be required • Cast-in-place footings may require a longer duration for construction, including dewatering and surface water pumping, as compared with pre-cast footing elements	• Greater risk associated with effective dewatering in fine-grained sandy silt to silt deposit and underlying sand and gravel deposit as compared to box culvert extension/replacement, due to greater drawdown required

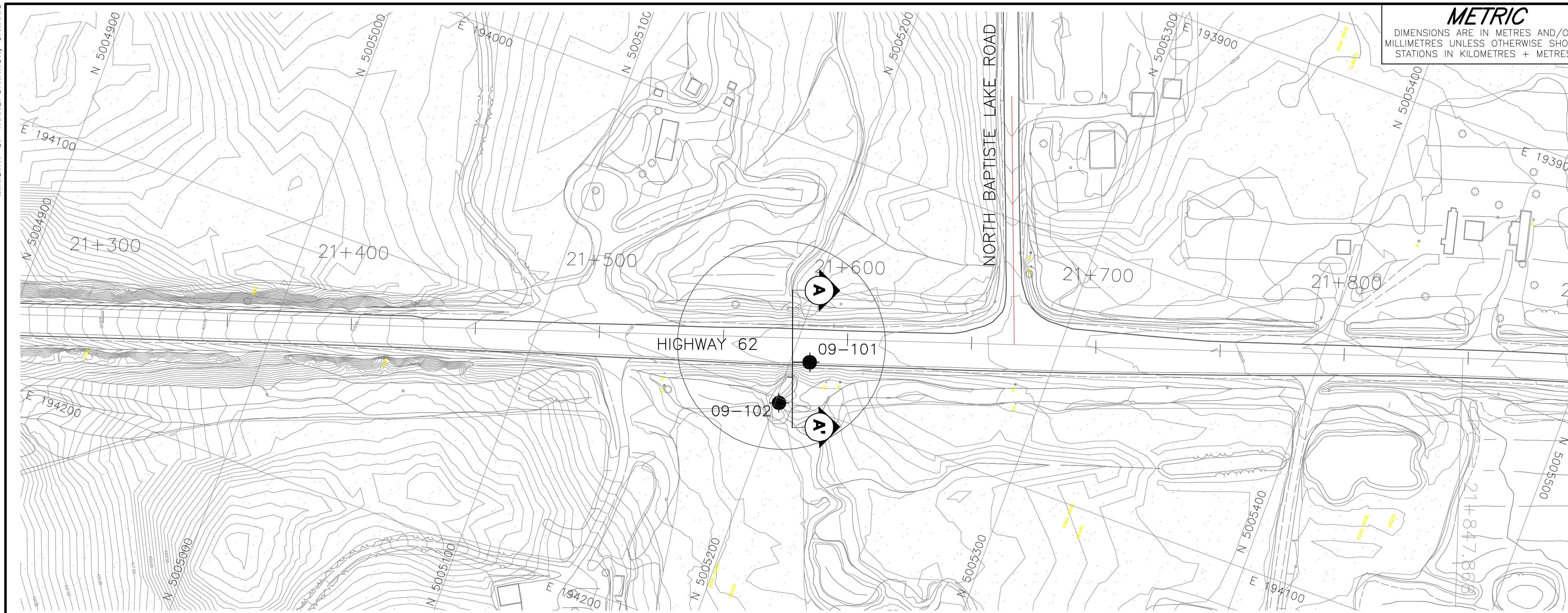
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

CONT No.
WP No. 187–99–00

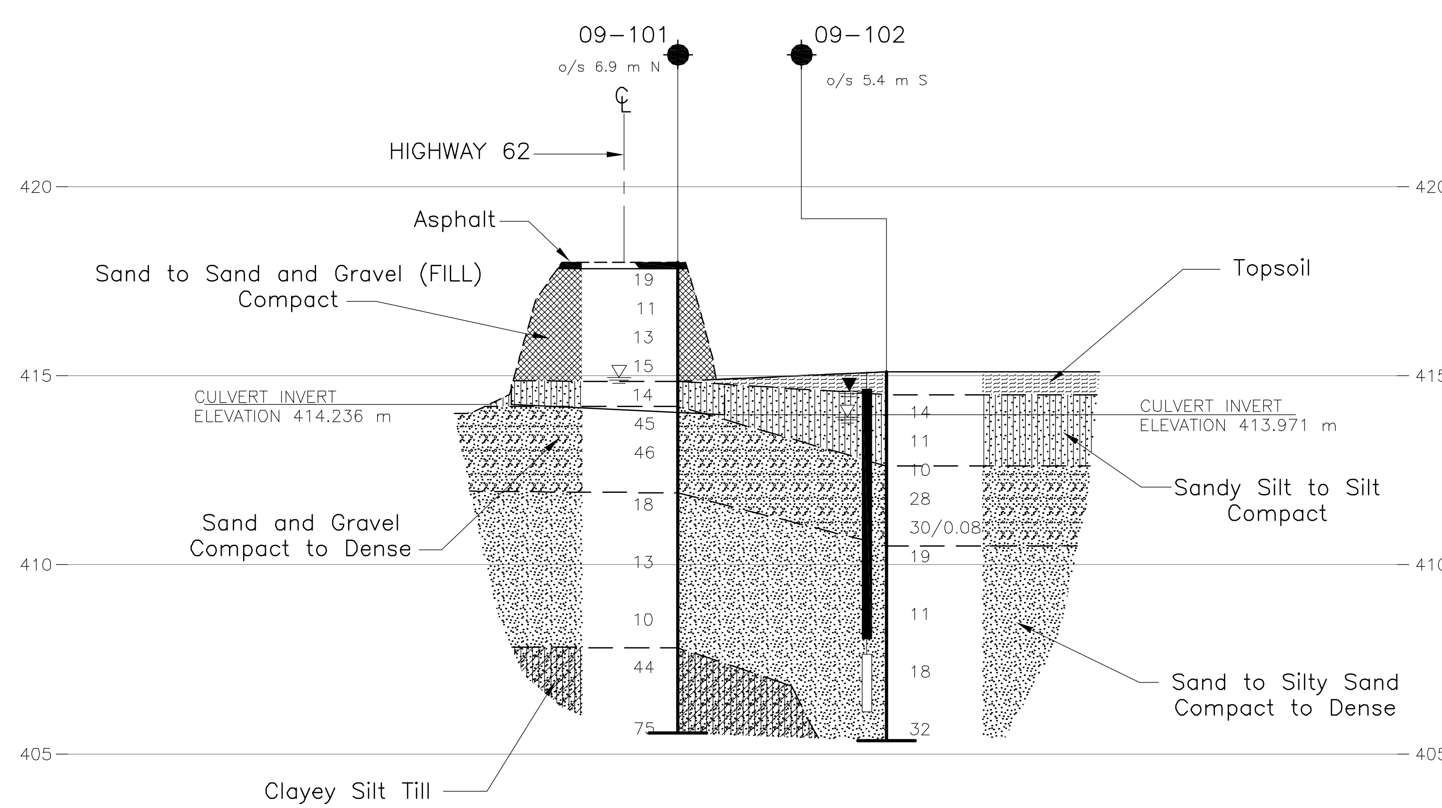
HIGHWAY 62 CULVERT EXTENSION
STATION 21+580
BOREHOLE LOCATION AND SOIL STRATA

SHEET

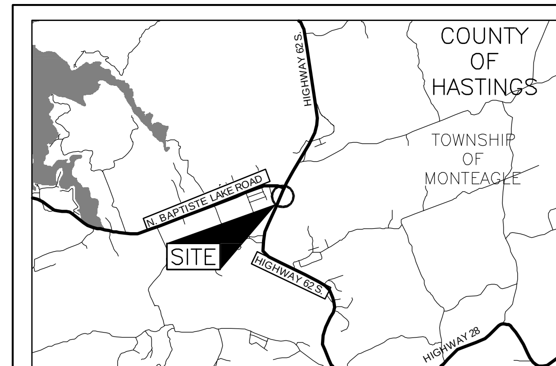
Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



PLAN
SCALE 20 0 20 40 m



SECTION A–A'
HORIZONTAL SCALE 10 0 10 20 m
2 0 2 4 m VERTICAL SCALE



KEY PLAN
SCALE 2 0 2 4 km

LEGEND

- Borehole – Current Investigation
- Seal
- Piezometer
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- WL in piezometer, measured on June 18, 2009
- WL upon completion of drilling

No.	ELEVATION	CO–ORDINATES NORTHING	CO–ORDINATES EASTING
09–101	418.0	5005207.5	194082.4
09–102	415.1	5005201.5	194102.0

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 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 plans provided in digital format by GENIVAR (Drawing file "Hwy 62N 62405.dwg", received May 14, 2009 and file "Plan–Existing.dwg, received June 5, 2009.

NO.	DATE	BY	REVISION

Geocres No. 31C–193

HWY. 62		PROJECT NO. 09–1111–0005	DIST.
SUBM'D. PKS	CHKD. JL	DATE: 21–Oct–2009	SITE:
DRAWN: JFC	CHKD. JL	APPD. LCC	DWG. 1

PLOT DATE: October 21, 2009
FILENAME: T:\Projects\2009\09-1111-0005 (TRANSENCO,CO,Hasting)\-AA-\09111100054A001.dwg

APPENDIX A

Borehole Records

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

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.)

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, 60º cone attached to "A" size drill rods for a distance of 300 mm (12 in.).

PH: Sampler advanced by hydraulic pressure
PM: Sampler advanced by manual pressure
WH: Sampler advanced by static weight of hammer
WR: Sampler advanced by weight of sampler and rod

Piezo-Cone Penetration Test (CPT)
A electronic cone penetrometer with a 60° conical tip and a project end area of 10 cm^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, s_u kPa	c_u, s_u 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 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
in 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
υ	poisson's ratio
σ	total stress
σ'	effective stress ($\sigma' = \sigma - \mu$)
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
μ	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

* Density symbol is ρ. Unit weight symbol is γ where $\gamma = \rho g$ (i.e. mass density multiplied by acceleration due to gravity)

(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

RECORD OF BOREHOLE No 09-101

 METRIC

PROJECT: 09-1111-0005			
G.W.P.: 187-99-00	LOCATION: N 5005207.5 ;E 194082.4	ORIGINATED BY: MR	
DIST: Eastern	HWY: 62	BOREHOLE TYPE: D55 Track Mount, 200 mm O.D. Hollow Stem Augers, H Casing, Wash Boring	COMPILED BY: AT
DATUM: Geodetic	DATE: May 5 & 6, 2009	CHECKED BY: LCC	

SOIL PROFILE

ELEV / DEPTH	DESCRIPTION
418.0	GROUND SURFACE
0.0	ASPHALT
0.3	Sand and gravel, trace silt (FILL) Compact Brown Moist
417.2 / 0.8	Sand, trace silt, trace gravel (FILL) Compact Brown Moist
	Sand and gravel, some silt, containing cobbles (FILL) Compact Brown Moist
414.8 / 3.2	Sandy SILT, trace gravel, trace clay, containing organics Compact Brown to grey Wet
414.2 / 3.8	SAND and GRAVEL, trace silt, trace clay, containing cobbles and boulders Dense Brown to grey Wet
411.9 / 6.1	SAND, some gravel, trace silt Compact Brown Wet. Becoming grey below a depth of 9.1 m
407.8 / 10.2	SILT, trace to some clay, to CLAYEY SILT, some sand, trace gravel, containing cobbles (TILL) Dense / Hard Grey Moist
405.5 / 12.5	END OF BOREHOLE

SAMPLES

NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR	SA	SI	CL
1	SS	19				
2	SS	11				
3	SS	13	31	58	11	0
4	SS	15				
5	SS	14	7	19	72	2
6	SS	45				
7	SS	46	52	40	6	2
8	SS	18	12	83	5	0
9	SS	13				
10	SS	10				
11	SS	44	9	19	57	15
12	SS	75/0.10				

Column headings: GROUND WATER CONDITIONS; ELEVATION SCALE (417 to 406); 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 (%) (10, 20, 30); UNIT WEIGHT γ kN/m³.

NOTES:

1. Water level in open borehole at a depth of 3.0 m below ground surface (Elev. 415.0 m) upon completion of drilling.

2. Borehole caved at a depth of 4.0 m below ground surface (Elev. 414.0 m) upon completion of drilling.

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

Golder Associates

RECORD OF BOREHOLE No 09-102

 METRIC

PROJECT 09-1111-0005			
G.W.P. 187-99-00	LOCATION N 5005201.5 ;E 194102.0		ORIGINATED BY MR
DIST Eastern HWY 62	BOREHOLE TYPE D55 Track Mount, 200 mm O.D. Hollow Stem Augers, H Casing, Wash Boring		COMPILED BY AT
DATUM Geodetic	DATE May 6, 2009		CHECKED BY LCC

SOIL PROFILE | SAMPLES | GROUND WATER CONDITIONS | ELEVATION SCALE | 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 (%) 10, 20, 30 | UNIT WEIGHT γ kN/m³ | REMARKS & GRAIN SIZE DISTRIBUTION (%)

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL
415.1	GROUND SURFACE							
0.0	TOPSOIL	1	SS	1				
414.5 / 0.6	Sandy SILT to SILT, trace to some sand, some gravel, trace clay, containing organics and rootlets; Compact; Dark brown to brown; Wet	2	SS	14				
		3	SS	11	15	11	70	4
412.6 / 2.5	SAND and GRAVEL, trace to some silt, containing cobbles; Compact; Brown to grey; Wet	4	SS	10				
		5	SS	28	33	55	12	0
		6	SS	30/0.08				
410.5 / 4.6	SAND, some gravel, trace silt, containing cobbles; Compact; Brown; Wet	7	SS	19				
		8	SS	11	12	80	8	0
		9	SS	18				
406.0 / 9.1	Silty SAND, some gravel, trace clay, containing cobbles; Dense; Grey; Wet	10	SS	32	13	65	18	4
405.3 / 9.8	END OF BOREHOLE							

Elevation scale: 415, 414, 413, 412, 411, 410, 409, 408, 407, 406

NOTES:

1. Water level in open borehole at a depth of 1.2 m below ground surface (Elev. 413.9 m) upon completion of drilling.

2. Borehole caved at a depth of 9.0 m below ground surface (Elev. 406.1 m) upon completion of drilling.

3. Water level in piezometer measured at a depth of 0.5 m (Elev. 414.6 m) on June 19, 2009.

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

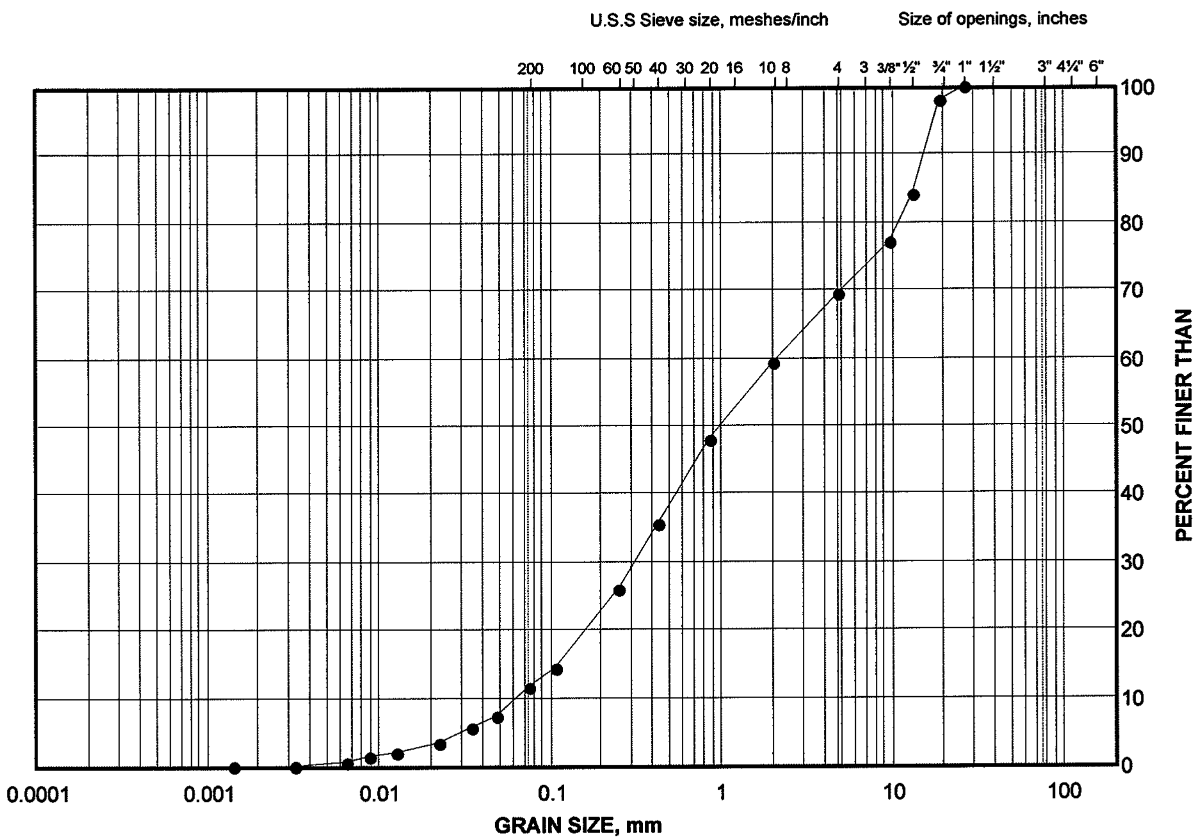
APPENDIX B

Laboratory Test Results

GRAIN SIZE DISTRIBUTION TEST RESULT

Sand and Gravel Fill

FIGURE B1



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	09-101	3	416.2

Project Number: 09-1111-0005

Checked By: [signature]

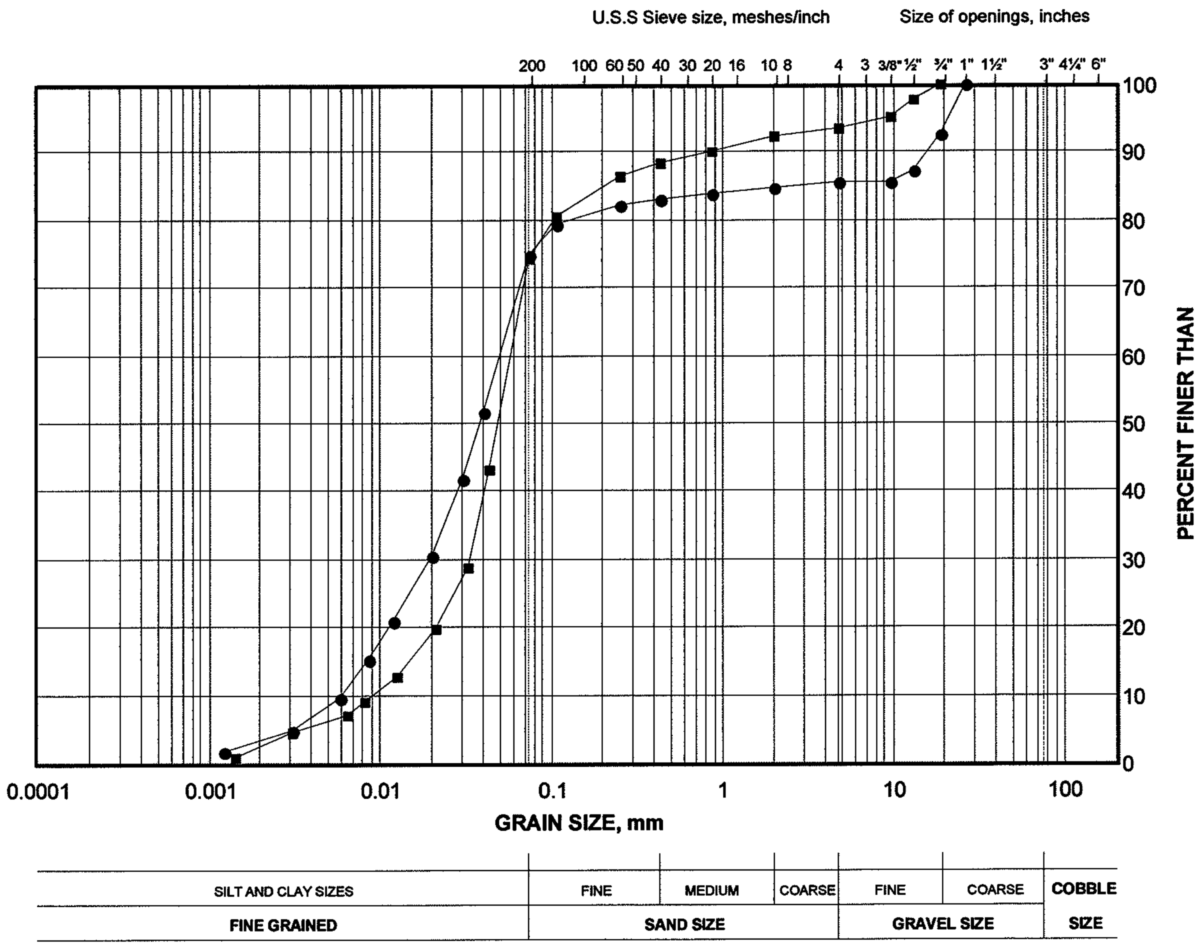
Golder Associates

Date: 13-Jul-09

GRAIN SIZE DISTRIBUTION TEST RESULTS

Sandy Silt to Silt

FIGURE B2



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	09-102	3	413.3
■	09-101	5A	414.6

Project Number: 09-1111-0005

Checked By: ______________

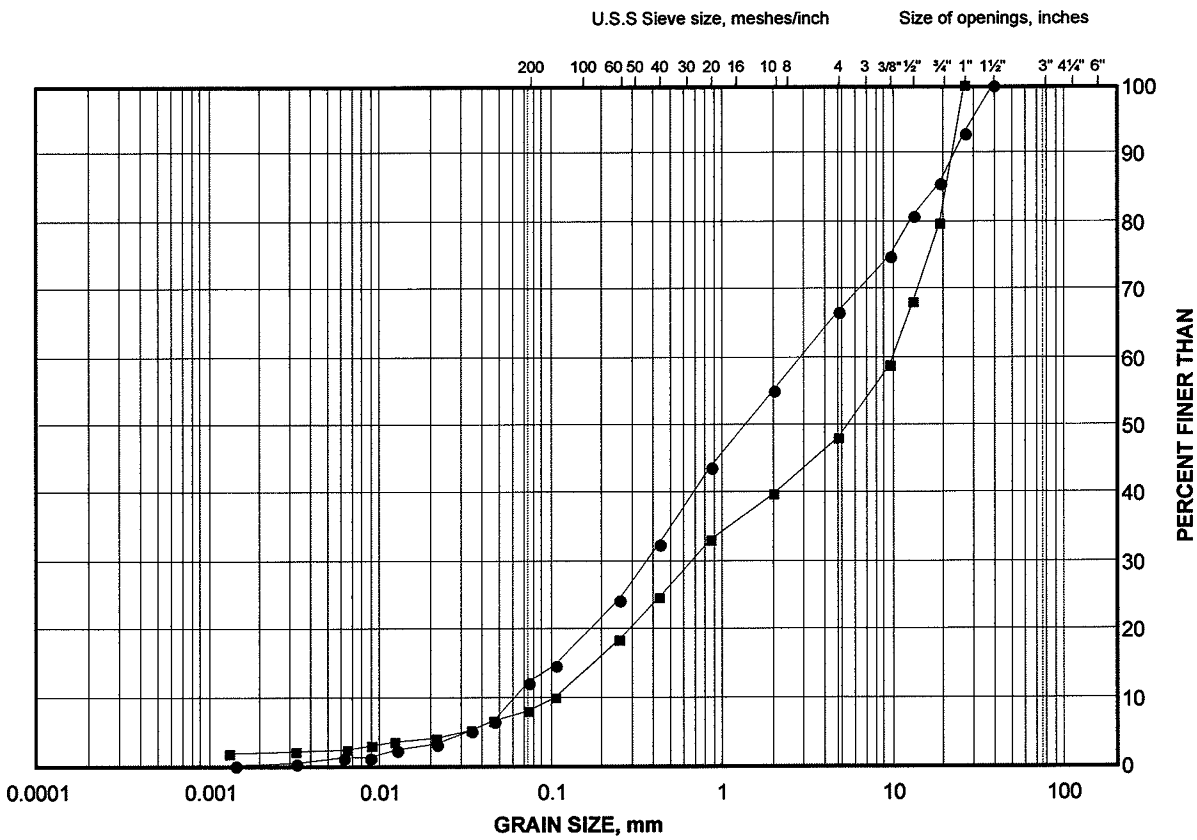
Golder Associates

Date: 13-Jul-09

GRAIN SIZE DISTRIBUTION TEST RESULTS

Sand and Gravel

FIGURE B3



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	09-102	5	411.7
■	09-101	7	413.1

Project Number: 09-1111-0005

Checked By: [signature]

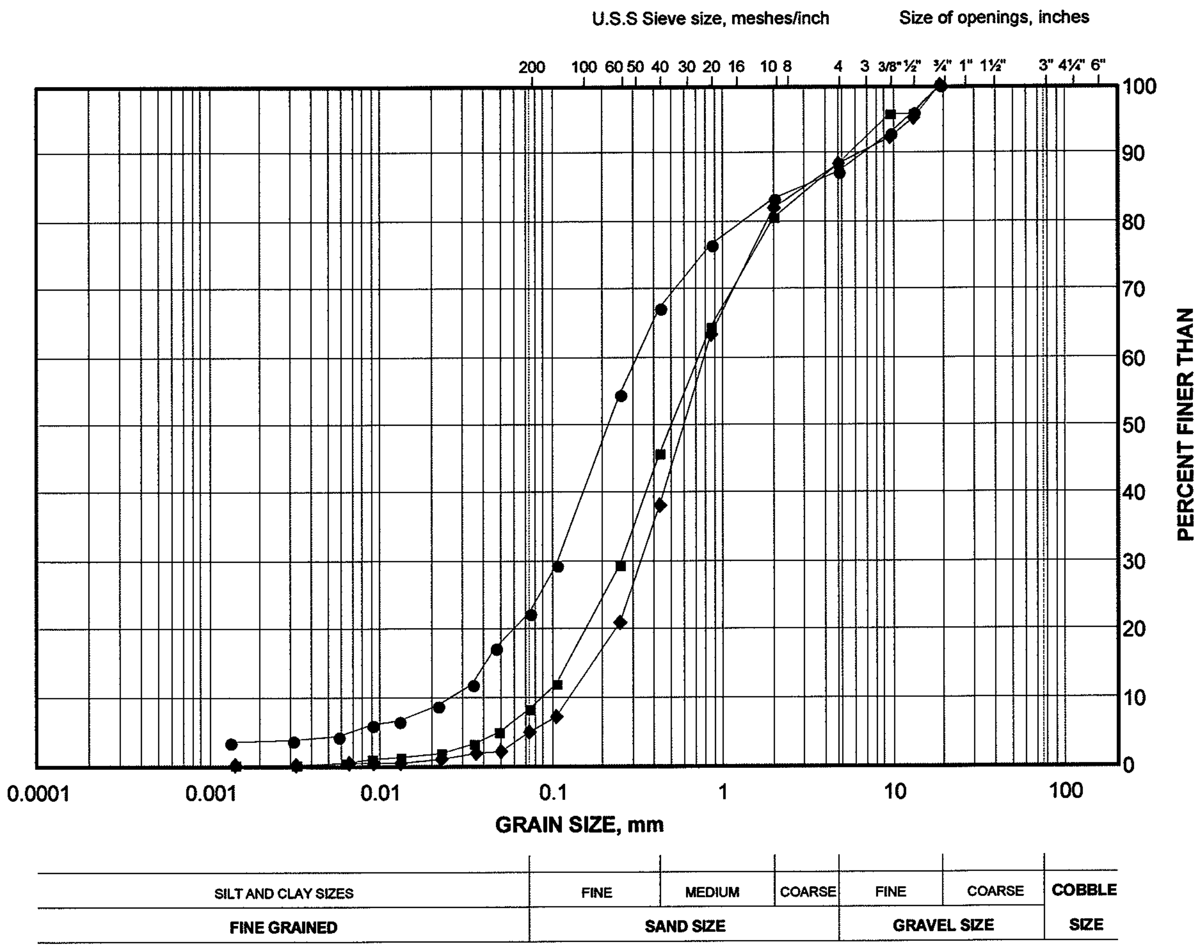
Golder Associates

Date: 13-Jul-09

GRAIN SIZE DISTRIBUTION TEST RESULTS

Sand to Silty Sand

FIGURE B4



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	09-102	10	405.7
■	09-102	8	408.7
◆	09-101	8	411.6

Project Number: 09-1111-0005

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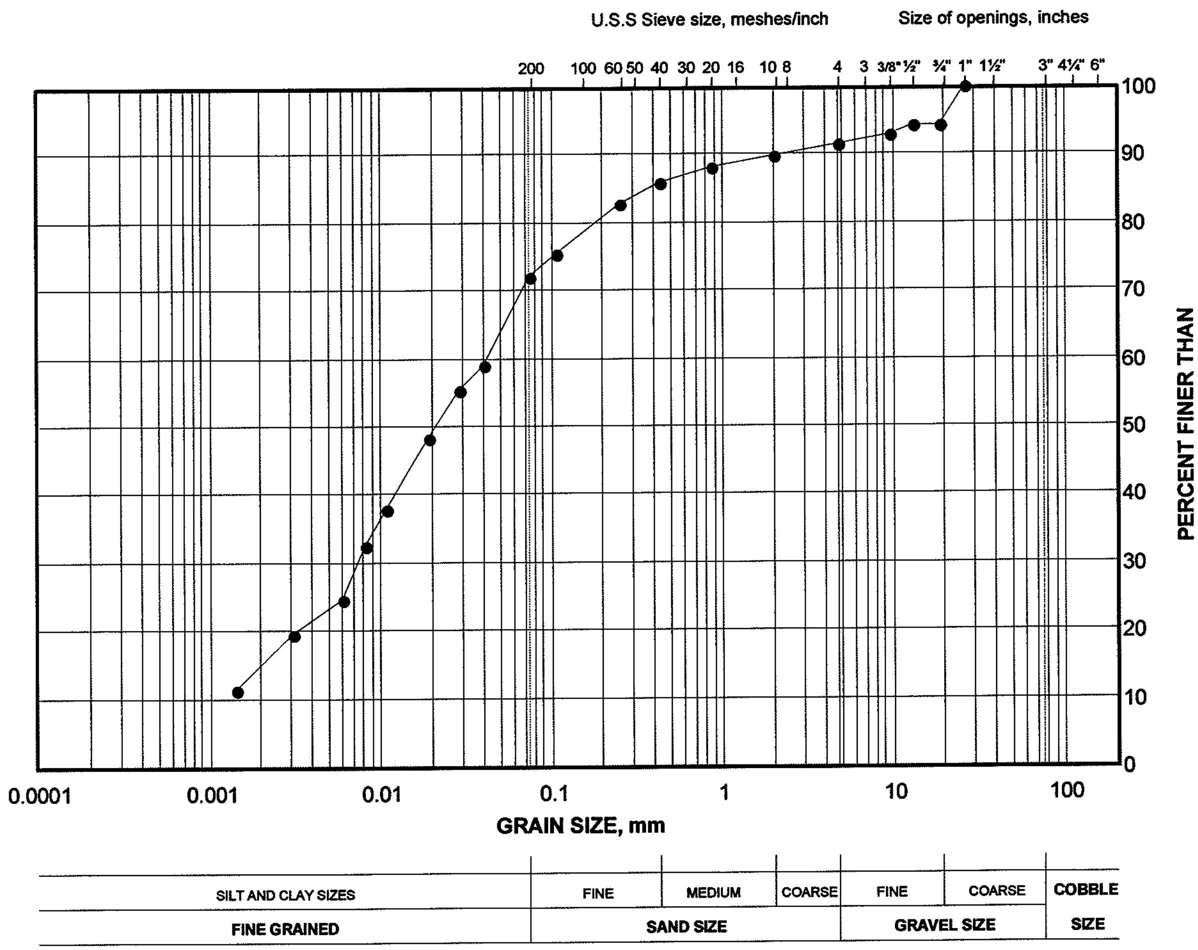
Golder Associates

Date: 13-Jul-09

GRAIN SIZE DISTRIBUTION TEST RESULT

Silt to Clayey Silt Till

FIGURE B5



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	09-101	11	407.0

Project Number: 09-1111-0005

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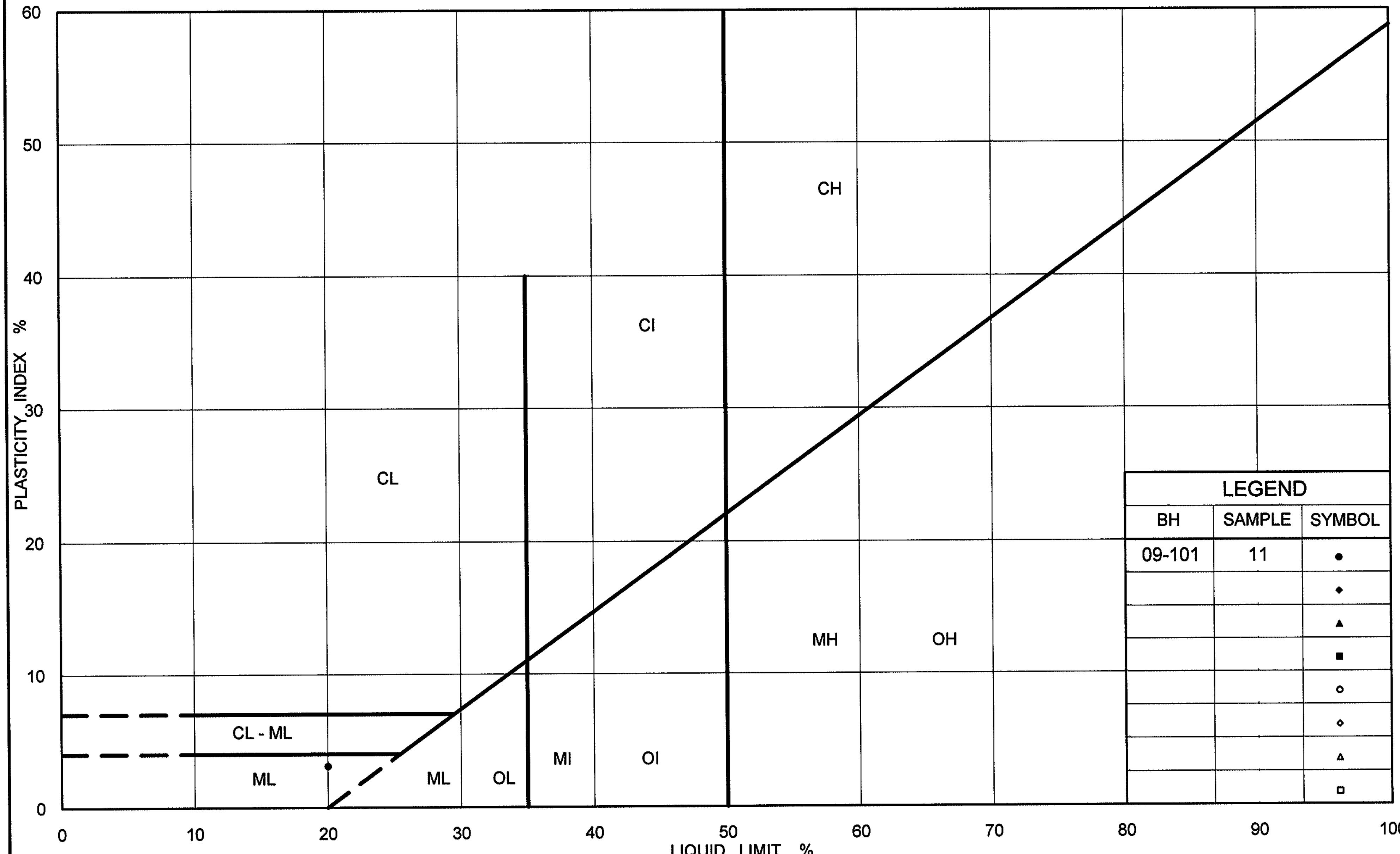
Golder Associates

Date: 13-Jul-09



Ministry of Transportation

Ontario

PLASTICITY CHART
Silt Till

Figure No. B6

Project No. 09-1111-0005

Checked By:

APPENDIX C

Non-Standard Special Provisions

DEWATERING – Item No.

Special Provision

SCOPE

The work under this item includes the design, installation, operation, maintenance and removal of temporary dewatering systems to facilitate the east extension/replacement of Culvert 11/338C at approximately Station 21+580.

Foundations for the culvert extension/replacement will require excavation into the compact sandy silt to silt and the compact to dense sand and gravel deposits, below the groundwater level at the site. The cohesionless soils below the groundwater table will be subjected to conditions of unbalanced hydrostatic head and can slough, boil and cave in during temporary excavation work.

REFERENCES

OPSS 517	Construction Specification for Dewatering of Pipeline, Utility, and Associated Structure Excavation
OPSS 518	Construction Specification for Control of Water from Dewatering Operations

SUBMISSION AND DESIGN REQUIREMENTS

Written details for the proposed dewatering system shall be submitted to the Contract Administrator for information purposes a minimum of ten business days prior to commencing dewatering operations. The Contractor shall reference borehole logs included in the contract documents as a guide in determining requirements.

CONSTRUCTION

Dewatering System

The Contractor is responsible for the design, installation, operation and maintenance of an adequate dewatering system to lower the groundwater level to at least 0.3 m below the footing founding level for the culvert extension/replacement, to allow excavation, foundation subgrade preparation and foundation construction in dry conditions.

Water pumped from trenches shall be redirected into the watercourse downstream of the work area in a manner that is not injurious to public health or safety, to property, to the environment or to any part of the work already completed or under construction.

Operation

A continuous dewatering operation shall be provided to facilitate the installation of the culvert extension/replacement at all times during the work. All components of the dewatering system shall be maintained in an effective, functioning and stable condition at all times during the work. Notwithstanding the above, the work shall be completed in accordance with the environmental and operational constraints specified elsewhere in the contract.

Restoration

All equipment and materials placed shall be removed from the right-of-way upon the completion of the work and all areas disturbed as part of this work shall be restored to their preconstruction conditions, unless specified otherwise.

BASIS OF PAYMENT

Payment at the contract price for the above tender item shall be full compensation for all labour, equipment and material to do the work.

MASS CONCRETE – Item No.

Non-Standard Special Provision

SCOPE

The work under this item addresses the supply and placement of a mass concrete working mat under the foundations for the east extension/replacement of Culvert 11/338C at approximately Station 21+580, to protect the subgrade from disturbance and loosening due to construction traffic and ponded water.

CONSTRUCTION

Following inspection and approval of the prepared subgrade, a working mat of mass concrete with a minimum thickness of 100 mm shall be placed on the foundation subgrade. The concrete shall have a compressive strength of at least 20 MPa, and be placed in accordance with OPSS 904.

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

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

Africa	+ 27 11 254 4800
Asia	+ 852 2562 3658
Australasia	+ 61 3 8862 3500
Europe	+ 356 21 42 30 20
North America	+ 1 800 275 3281
South America	+ 55 21 3095 9500

solutions@golder.com
www.golder.com

Golder Associates Ltd.
2390 Argentia Road
Mississauga, Ontario, L5N 5Z7
Canada
T: +1 (905) 567 4444