September 17, 2009

FOUNDATION INVESTIGATION AND DESIGN REPORT

KENDALL CREEK BRIDGE REPLACEMENT
HIGHWAY 11, SITE NO. 39W-035
TOWNSHIP OF KENDALL, ONTARIO
MINISTRY OF TRANSPORTATION, ONTARIO
GWP 5413-04-00

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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 LEA Consulting Ltd. (LEA) on behalf of the Ministry of Transportation, Ontario (MTO) to provide foundation engineering services for the detail design of the Kendall Creek Bridge on Highway 11 in the Township of Kendall, east of the town of Hearst, Ontario.

The terms of reference for the scope of work are outlined in Golder's proposal P7-1191-0007, dated February 26, 2007, which forms part of the Consultant's Agreement (P.O. Number 5006-E-0015) for this project. The work was carried out in accordance with the Quality Control Plan for this project dated September 18, 2007. The General Arrangement drawing (GA) for the bridge structure was provided to Golder by LEA in April 2009 and updated in June 2009.

The purpose of this investigation is to establish the subsurface conditions at the proposed replacement and detour structure locations by borehole drilling and in situ testing and laboratory testing on selected samples. The boreholes were located in the field by Golder relative to the centreline and offset stakes laid out at the site by LEA. The location of the investigated area is shown in plan on Drawing 1.

2.0 SITE DESCRIPTION

The site is situated in the Township of Kendall on Highway 11 crossing Kendall Creek, approximately 3 km east of the town of Hearst, Ontario. The surrounding land is mainly comprised of scattered residences, residential farms and businesses. Grass and tree cover extend beyond the limits of the site, while the banks adjacent to the creek are vegetated with grass and small shrubs. Boulders are visible within the creek bed. The river is up to 1.5 m deep, as indicated in the GA provided to us, and is mainly used for recreation. The river is about 15 m wide at the existing bridge location.

We understand that the existing Kendall Creek Bridge was constructed in 1939. The existing single span bridge has an overall deck length of about 15 m and overall width of about 10 m. We understand from LEA that no rehabilitations or repairs have been made to the bridge. The water level of Kendall Creek was measured at approximately Elevation 232.4 m in August 2008.

3.0 INVESTIGATION PROCEDURES

A total of eight (8) boreholes were advanced at the site between April 16 and 26, 2009. Six boreholes (KC09-1 to KC09-4, KC09-6 and KC09-7) were advanced for the proposed main bridge abutments and approaches and two boreholes (KC09-5 and KC09-8) were advanced for the proposed detour bridge. The locations and elevations of the boreholes are shown on Drawings 1 and 3.

Boreholes KC09-1 to KC09-4 were drilled using a track mounted CME 45-C drill rig supplied and operated by Downing Drilling Ltd. of Grenville-Sur-La-Rouge, Quebec. Boreholes KC09-5 to KC09-8 were drilled using portable equipment that was supplied and operated by OGS Inc. of Almonte, Ontario

The machine drilled boreholes were advanced using 108 mm inside diameter (I.D.) continuous flight hollow stem augers, with NW casing with wash boring, as necessary. The boreholes advanced using portable equipment were advanced using NW and/or BW casing with wash boring. A combination of continuous and non-continuous (at intervals of depth of about 0.75 m to 2.5 m) sampling methods were used in obtaining soil samples. Samples were obtained using a 50 mm outer diameter (O.D.) split-spoon sampler in accordance with Standard Penetration Test (SPT) procedures (ASTM D1586-99). NQ size rock core barrel was used to advance through the lower portion of the borehole containing cobbles and boulders at KC09-1 and KC09-2.

The boreholes were advanced to depths ranging from 4.6 m to 21.6 m below the existing ground surface where they were terminated due to either casing, spoon or auger refusal.

The fieldwork was supervised throughout by members of our engineering and technical staff, who located the boreholes, arranged for the clearance of underground service locations, supervised the drilling and sampling operations, logged the boreholes, and examined and cared for the soil and rock core samples. The samples were identified in the field, placed in appropriate containers, labelled and transported to our Sudbury geotechnical laboratory where the samples underwent further visual examination and laboratory testing. All of the laboratory tests were carried out to MTO and/or ASTM Standards, as appropriate. Classification testing (water content, Atterberg limits and grain size distribution) was carried out on selected soil samples.

The proposed boreholes were laid out in the field by Golder relative to the proposed centreline alignment and offset stakes surveyed by LEA and based on the dimensions shown on the GA supplied by LEA in April 2009. The northings and eastings in MTM NAD 83 were determined by plotting the station and offset of the boreholes (relative to the stakes) on the April 2009 GA and converting to the coordinate system.

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

Published literature indicates that the site is located in the Quetico Subprovince of the Superior Province (Geology of Ontario; OGS Special Volume 4)[1]. The bedrock of this domain consists of muscovite-bearing granitic rocks (peraluminous), and may include biotite granite. Igneous rocks (white to grey muscovite leucogranite) are the most abundant type of rock in the Quetico Subprovince.

Based on terrain mapping by the Ontario Geological Survey[2], the subsurface soils in the vicinity of the site consist of ground moraine deposits comprising of stony sandy till overlying bedrock.

4.2 Subsurface Conditions

The detailed subsurface soil and groundwater conditions, as encountered in the boreholes advanced during this investigation, together with the results of the laboratory tests carried out on selected soil samples, are given on the Record of Borehole sheets attached in Appendix A. The stratigraphic boundaries shown on the Record of Borehole sheets are inferred from non-continuous sampling and observations of drilling progress and cuttings. These boundaries, therefore, represent transitions between soil types rather than exact planes of geological change. Further, subsurface conditions will vary between and beyond the borehole locations. The inferred soil stratigraphy based on the results of the boreholes at the bridge locations are shown on Drawings 1 to 3.

In general, the subsoils at the main structure site generally consist of embankment fill (sandy and/or clayey) and alluvium/peat underlain by silty clay to clayey silt and/or sandy silt to sand, overlying a very dense sandy silt to silt and sand till deposit. At the detour structure site, native silty clay, peat and/or sandy silt was encountered overlying the till deposit.

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

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[1] Geology of Ontario, 1991. Ontario Geological Survey, special Volume 4, Part 1. Eds P.C. Thurston, H.R. Williams, R.H. Sutcliffe and G.M. Stott, Ministry of Northern Development and Mines, Ontario.

[2] Northern Ontario Engineering Geology Terrain Study, OGS Electronic Map

4.2.1 Fill

Asphalt was encountered in Boreholes KC09-1 to KC09-3 at the ground surface of Elevation 236.4 m. The thickness of the asphalt layer varied from 150 mm to 410 mm, being thickest closest to the bridge abutments.

A layer of sand to gravelly sand fill was encountered in Boreholes KC09-1 to KC09-4, KC09-6 and KC09-7. The sand to gravelly sand fill layer varied in thickness from 0.4 m to 1.5 m. Where asphalt was encountered at the ground surface (Borehole KC09-1 to KC09-3), it was underlain by sand to gravelly sand fill; otherwise, the fill was encountered at ground surface. The fill was noted to contain trace clay, trace to some silt and trace organics, and Borehole KC09-6 contained trace asphalt. Between 1.8 m and 3.0 m of frozen fill was encountered from ground surface in Boreholes KC09-1 to KC09-3 and KC09-8, drilled between April 15 and 26, 2009.

The surface of the sand to gravelly sand fill was encountered between Elevation 235.4 m to 236.3 m and the surface of the silty clay to clayey silt fill was encountered between Elevation 234.7 m and 236.4 m.

Below the sand to gravelly sand fill layer, a layer of silty clay to clayey silt fill, containing trace sand, trace gravel and trace organics was encountered. The fill in Borehole KC09-2 contained trace asphalt. The silty clay to clayey silt fill layer varied in thickness from 0.7 m to 1.8 m.

The total thickness of fill in the boreholes in which it was encountered varied from 1.1 m to 3.0 m.

SPT 'N' values measured within the encountered sand to gravelly sand fill ranged from 7 blows to greater than 100 blows (below the roadway) per 0.3 m of penetration suggesting a loose to very dense relative density. SPT 'N' values measured with the silty clay to clayey silt fill varied from 6 to 32 blows per 0.3 m of penetration suggesting a firm to hard consistency.

One grain size distribution test from the sand fill is shown on Figure B-1 in Appendix B.

Atterberg limits testing carried out on samples of the silty clay to clayey silt fill layer indicate liquid limits ranging from about 32 percent to 41 percent and plastic limits ranging from 16 percent to 22 percent, yielding plasticity indices ranging from about 16 percent to 20 percent. The results of the Atterberg limits testing are shown on the plasticity chart on Figure B-2, and indicate that the fill material is classified as a clayey silt of low plasticity to a silty clay of intermediate plasticity.

The natural water content measured on the samples of the cohesionless fill are 5 and 10 percent, and in the cohesive fill, are between 24 and 32 percent.

4.2.2 Alluvium

A deposit of alluvium was encountered below the cohesive fill in Boreholes KC09-1, KC09-2, KC09-6 and KC09-7. The alluvium consisted of silty clay containing trace sand to sandy silt or sand containing trace gravel. The deposit also contained various quantities of organics ranging from trace to layered. The surface of the 1.7 m to 2.0 m thick layer was encountered between Elevation 233.3 m to 233.6 m.

SPT 'N' values measured within the alluvium ranged from 6 blows to 17 blows per 0.3 m indicating a loose to compact relative density where the alluvium is mainly cohesionless to a firm to very stiff consistency where the alluvium is mainly cohesive.

Atterberg limits testing carried out on one sample of the silty clay alluvium indicate liquid limit of about 36 percent, a plastic limit of about 22 percent, yielding a plasticity index of about 14 percent. The results of the Atterberg limits testing are shown on the plasticity chart on Figure B-3, and indicate that this sample of the alluvium material is classified as a silty clay of intermediate plasticity. One grain size distribution test from the silty clay alluvium is shown on Figure B-4 in Appendix B.

The natural water content measured on samples of alluvium are between 19 and 35 percent.

4.2.3 Peat

Below the existing silty clay to clayey silt fill in Borehole KC09-3, within the alluvium layer in Boreholes KC09-2 and at the existing ground surface in Borehole KC09-8, a deposit of moist to wet, brown to black, fibrous peat was encountered. The sample retrieved at the existing ground surface in Borehole KC09-8 was found to contain trace sand and pieces of wood. The top of this deposit ranged from Elevation 233.0 m to 234.1 m and the thickness varied from 0.3 m to 1.8 m.

SPT 'N' values measured within the peat deposit ranged from 8 to 20 blows per 0.3 m of penetration indicating a firm to very stiff consistency.

The natural water content measured in this stratum varied from 29 to 85 percent.

4.2.4 Silty Clay to Clayey Silt

A deposit of moist to wet, brown to grey, silty clay to clayey silt was encountered beneath the sand in Borehole KC09-3, the silty clay fill in Borehole KC09-4 and at the existing ground surface in Borehole KC09-5. This deposit contained trace to some sand and trace gravel. Trace organics were noted in Borehole KC09-5. The silty clay was noted to be layered in Borehole KC09-4. The top of the deposit was encountered from Elevation 232.6 m to 235.2 m and the thickness ranged from 0.5 m to 2.3 m.

SPT 'N' values measured within the silty clay to clayey silt deposit ranged from 4 to 22 blows per 0.3 m of penetration indicating a firm to very stiff relative consistency.

Atterberg limits testing carried out on three samples of the silty clay to clayey silt deposit indicate liquid limits ranging from about 22 to 50 percent and plastic limits ranging from about 13 to 23 percent, yielding plasticity indices ranging from about 8 to 27 percent. The results of the Atterberg limits testing are shown on the plasticity chart on Figure B-5 in Appendix B, and indicate that the deposit ranges from a clayey silt of low plasticity to a silty clay of intermediate plasticity, on the border of becoming a clay.

The natural water content measured on samples of this deposit range from about 15 percent in the low plasticity clayey silt to 41 percent in the silty clay, which is below their respective liquid limits.

4.2.5 Sandy Silt to Sand

A deposit of moist to wet, brown to grey to black, sandy silt to sand was encountered beneath the alluvium in Borehole KC09-2, beneath the peat deposit in Boreholes KC09-3 and KC09-8 and beneath the silty clay to clayey silt deposit in Borehole KC09-5. This deposit was found to contain some gravel and clay, and in Borehole KC09-8, the deposit contained some organics and cobbles. The top of the deposit ranged from about Elevation 231.6 m to 233.0 m and the thickness of the deposit ranged from about 0.4 m to 0.9 m.

SPT 'N' values measured within the sandy silt to sand deposit range from 15 blows to 63 blows per 0.3 m of penetration indicating a compact to very dense relative density. Based on observations during drilling of Boreholes KC09-2 and KC09-3, it became difficult to advance the augers (hard drilling and/or auger refusal) through the deposit and, in some cases, it was necessary to switch to casing to advance the boreholes.

Grain size distribution tests were carried out on two samples of the sandy silt and the results are shown on Figure B-6.

The natural water content measured on samples of the sandy silt to sand deposit are between 9 and 13 percent.

4.2.6 Sandy Silt to Silt and Sand (Till)

A deposit of moist to wet, brown to grey, silty sand to silt and sand till was encountered at the base of all the boreholes. The till typically contained trace to some clay and trace to some gravel as well as cobbles and boulders. The top of this deposit was encountered between Elevation 230.8 m to 232.9 m and the encountered thickness of the deposit varied from 1.2 m and 16.0 m. All boreholes terminated within this deposit on either auger, casing or spoon refusal between Elevation 214.8 m and 231.7 m, which is between 4.6 m and 21.6 m below the ground surface.

SPT ‘N’ values measured within the till deposit ranged from 11 blows per 0.3 m of penetration to 130 blows per 0.1 m of penetration suggesting a compact to very dense relative density which is also indicative of the presence of cobbles and boulders. Difficult auger and/or casing advance was observed during drilling through the till deposit and, in Boreholes KC09-1 and KC09-2, soil coring using an NQ core barrel had to be used to advance the borehole further.

Grain size distribution tests were carried out on several samples of the sandy silt to silt and sand till deposit and the results are shown on Figure B-7.

The natural water content measured on samples of the till ranged from 8 to 15 percent.

4.2.7 Groundwater Conditions

The water levels were noted during and after the drilling operations in the boreholes. In general, the soil samples taken in the boreholes were noted to be moist. The groundwater level ranged from 0.4 m to 5.1 m below the ground surface, with Borehole KC09-4 being dry upon completion of drilling. The groundwater level ranged from about Elevation 231.3 m to 234.1 m and was generally between 232 m and 233 m. The water level in Kendall Creek was surveyed by others in August 2008 at Elevation 232.4 m. Therefore, the water levels measured in the boreholes were typically within 1.4 m below to 1.6 m above the previous water level measured in Kendall Creek. The 100 year flood level of Kendall Creek is Elevation 234.0 m.

The water levels may not represent the stabilized water level at the site and that the groundwater elevation will fluctuate seasonally depending on precipitation and local soil permeability and should be expected to rise during wet periods of the year.

5.0 CLOSURE

The field personnel supervising the drilling program was Mr. Ed Savard and Mr. Mat Riopelle. This report was prepared by Mr. Luigi Gianfrancesco, EIT and the technical aspects were reviewed by Ms. Sarah E. M. Coyne, P.Eng., an Associate with Golder. A quality control review of the report was provided by Mr. Fintan J. Heffernan, P.Eng., Golder’s Designated MTO Contact for this project.

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PART B

FOUNDATION DESIGN REPORT

KENDALL CREEK BRIDGE REPLACEMENT

HIGHWAY 11, SITE NO. 39W-035

TOWNSHIP OF KENDALL, ONTARIO

MINISTRY OF TRANSPORTATION, ONTARIO

GWP 5413-04-00

6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

This section of the report provides design recommendations on the foundation aspects of the proposed Highway 11 Bridge structure over Kendall Creek. The recommendations are based on interpretation of the factual data obtained from the boreholes advanced during the subsurface investigation at the site. The interpretation and recommendations presented 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.

6.1 General

The existing bridge carrying Highway 11 over Kendall Creek is a single span structure with an overall deck length of about 15 m and width of about 10 m. The structure, erected in 1939, is supported on shallow foundations at about Elevation 230.2 m. Significant asphalt padding, up to about 400 mm, was encountered near the bridge abutments suggesting settlement has occurred.

We understand the new bridge will be located on the existing alignment with traffic diverted onto a single lane modular detour bridge on the north side of the existing bridge. The proposed main bridge will have one 21 m long span with the abutments located slightly behind the existing bridge abutments. The width of the bridge will be 13.5 m. The proposed grade will be at approximately Elevation 237 m, about 0.6 m higher than the current road grade. Kendall Creek is less than 15 m wide at the crossing location with a water level of about Elevation 232.4 m, measured in August 2008. The new approach embankments will be up to 4.6 m high closest to the creek. The temporary modular detour bridge will have an overall length of about 30 m and a grade of Elevation 236.4 m.

The subsurface conditions generally consist of sandy and or clayey fill overlying a compact to dense sandy silt to sand layer followed by a very dense sandy silt to silt and sand till at an Elevation of 231 m to 233 m, corresponding to up to about 6 m below the proposed final grade. Peat/alluvium was encountered below the fill in some of the boreholes. The water level at the time of the field investigation was between Elevation 231.3 m and 234.1 m. The stabilized water level was generally between Elevation 232 m and 233 m, generally about the same as the creek water level.

The recommendations on the foundation design aspects of the new structure presented in this report take into consideration the impact of the detour bridge foundations and approach embankments on the existing bridge foundations and approach embankments during construction as well as the removal of the existing bridge.

Shallow spread footings founded on the surface of the very dense till or on a granular pad are recommended for founding the proposed bridge structures. Deep foundations, as discussed below, are feasible at this site but not considered to be practical. Table 1 (attached), summarizes the advantages, disadvantages, relative costs and risks/consequences of the foundation alternatives. Discussion on the alternatives is given in the sections below.

6.2 Shallow Foundations

Spread footings placed directly on the native very dense till material are recommended for support of the main bridge foundations. For the detour bridge, spread footings placed on a compacted Granular 'A' fill pad over the very dense till are recommended.

Main Bridge

The lowest surface of the native sandy silt to silt and sand till is Elevation 230.8 m and 231.4 m at the east and west abutment locations, respectively. Consideration must be given to the proximity and depth (Elevation 230.2 m) of the existing footings when determining the founding elevation of the proposed footings. The feasible founding elevation alternatives are:

- Elevation 230.8 m to 231.7 m at the east abutment and 231.4 m at the west abutment. Since the adjacent existing footings are founded at Elevation 230.2 m, and given that little is known about the construction, it is possible that some fill may be encountered below the proposed founding level. This fill, if encountered, would be less than 1.2 m and would have to be removed and filled with mass concrete. This assumes that the existing footing would be left in place during and after construction of the new bridge foundations.
- Elevation 230.2 m or at the same elevation of the existing footings. This would require a slightly deeper excavation below the water level. In this case, the new footing would be immediately adjacent to the existing footing, which would have to be exposed during construction.

Another feasible founding alternative from a foundations perspective is to found the new bridge on the existing bridge footings. This assumes that the existing footings are to be left in place and that they have been constructed on the native very dense till. The designers would need to determine if this alternative is feasible structurally.

Due to the possibility of encountering fill below the surface of the till in the area closest to the existing footings, we recommend founding the spread footings at the same level as the existing bridge footings, i.e. about Elevation 230.2 m.

Detour Bridge

We recommend founding the detour bridge footings on a granular pad extending to the surface of the very dense till. The surface of the native sandy silt to silt and sand till is Elevation 231.0 m at the east and west detour abutment locations, which is about 1.4 m below the water level in Kendall Creek (August 2008). The overburden should be removed to this elevation and the granular pad constructed to the underside of footings, a minimum of 2 m above the native till. Based on the approximate level of the proposed underside of footings, the pad will be about 3.4 m thick.

Alternatively, the native material could be excavated to the surface of the native sandy silt at Elevation 231.6 m, which in the case of Borehole KC09-8 (east abutment) contains some organics. This would decrease the quantity of excavation and filling below the water level to about 1 m and the total pad thickness would be about 2.8 m. Some settlement of the founding soils below the granular pad should be anticipated and is discussed in Section 6.6.3.

In both cases, the thickness of the pad is greater than the frost depth of 2.6 m, as given in Section 6.2.3, although the detour is not expected to be in use for more than one construction season (spring to fall).

6.2.1 Geotechnical Axial Resistance

Spread footings placed directly on or below the surface of the properly prepared sandy silt to silt and sand till may be designed based on a factored geotechnical axial resistance at Ultimate Limit States (ULS) of 500 kPa. A geotechnical resistance at Serviceability Limit States (SLS) of 350 kPa may be used for design, based on 25 mm of settlement and assuming a 2 m to 3 m wide footing.

If the existing footings are being considered for support of the new bridge, then the designer should check that their current size is appropriate for the above values for geotechnical resistance and the new bridge loading. Enlargement of the existing footings may be feasible from a foundations perspective provided that the new concrete is dowelled into the existing concrete and other structural considerations have been addressed.

If the very dense till is not encountered at the founding level, the excavation must be extended to the surface of the till and backfilled to the founding level with mass concrete as discussed in Section 6.7.

For spread footings placed (or perched) within the approach embankments on a compacted Granular 'A' core (extending to Elevation 231.6 m or 231.0 m), a factored geotechnical axial resistance at ULS of 850 kPa may be used. A corresponding SLS value of 350 kPa may be used assuming a 2 m to 3 m wide footing. These values assume a minimum 2 m thick granular pad placed below the base of the footing placed directly over the surface of the native till in dry conditions. The granular pad should extend at least 1 m beyond the plan limits of the footing and be sloped no steeper than 1 Horizontal:1 Vertical (1H:1V) in general accordance with MTO guidelines and Figure 1. The granular pad should be constructed in accordance with MTO Special Provision 105S10 (Compaction). MTO SP 902S01 (Excavation and Backfilling) should also be included in the Contract Documents.

For either detour founding elevation, the pad should be constructed out of Granular 'B' Type II to 0.6 m above the water level (i.e. to Elevation 233.0 m), if dewatering is not carried out. It has been observed in the field that Granular 'B' Type II compacts to an adequate degree below the water level without any additional compactive effort provided it is not placed in more than 2 m of water and provided that the surface above the water is compacted properly. After placement and compaction of the Granular 'B' Type II, Granular 'A' can be placed to the underside of the footing. In this case (without dewatering), the factored axial geotechnical resistance at ULS and the axial geotechnical resistance at SLS to be used for design is 650 kPa and 350 kPa, respectively.

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 Sections 6.7.2 and 6.7.4 of the *Canadian Highway Bridge Design Code (CHBDC)* and its *Commentary.*

6.2.2 Resistance to Lateral Loads

Resistance to lateral forces/sliding resistance between the base of the mass concrete and the subgrade should be calculated in accordance with Section 6.7.5 of the *CHBDC*. The coefficient of friction, tan δ, may be taken as 0.55 between the base of the concrete footings and the native sandy silt to silt and sand till and as 0.70 between the concrete and the compacted granular pad, constructed in-the-dry. This value represents an unfactored value; in accordance with the *CHBDC*, a factor of 0.8 is to be applied in calculating the horizontal resistance.

6.2.3 Frost Protection

All footings should be provided with a minimum of 2.6 m of soil cover or equivalent thickness of insulation for frost protection (OPSD 3090.100).

6.3 Deep Foundations

Given that very dense bouldery material is present at shallow depth at this site, the use of driven steel H-piles or caissons socketted into the very dense bouldery till are not considered to be practical. Driving piles to the surface of the till is not feasible due to the shallow depth and the requirement for minimum pile lengths. Since driven piles would "hang-up" on boulders within the till, the geotechnical axial resistance would be difficult to determine. The only way to achieve the required design capacity and minimum length for driven piles on till would be to pre-auger in advance of pile driving. However, when driving below the pre-augered depth, the piles would encounter boulders and would tend to be out of alignment and/or location. Caissons are typically many times more costly than the pile alternative.

6.4 Site Coefficient

For seismic design purposes, the Site Coefficient, S, for this site, in accordance with Section 4.4.6 of the *CHBDC* may be taken as 1.0, consistent with Soil Profile Type I.

6.5 Lateral Earth Pressures for Design

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

6.5.1 Static

The following recommendations are made concerning the design of the walls. It should be noted that these design recommendations and parameters assume level backfill and ground surface behind the walls. Where there is sloping ground behind the walls, the coefficient of lateral earth pressure must be adjusted to account for the slope.

- Select free-draining granular fill meeting the specifications of Ontario Provincial Standard Specifications (OPSS) 1010 Granular 'A' or Granular 'B' Type II but containing less than 5 percent passing the No. 200 sieve size should be used as backfill behind the walls. This fill should be compacted in loose lifts not greater than 200 mm in thickness to 95 percent of the material's Standard Proctor maximum dry density in accordance with OPSS 501. 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 Ontario Provincial Standard Drawings (OPSD) 3101.150 and 3121.150.

- For structures that are not comprised of integral or semi-integral abutments, rock fill may be used as backfill behind the walls and the material should meet the specifications as outlined in the Northern Region Directive for backfill to structures adjacent to rock fill embankments, dated November 2002. Other aspects of rock backfill requirements should be in accordance with OPSD 3101.200.

- A minimum compaction surcharge of 12 kPa should be included in the lateral earth pressures for the structural design of the wall stem, in accordance with *CHBDC* Section 6.9.3 and Figure 6.6. Compaction equipment should be used in accordance with OPSS 501.06 or SP 105S10. Other surcharge loadings should be accounted for in the design, as required.

- The granular fill may be placed either in a zone with width equal to at least 2.6 m behind the back of the wall stem (as outlined on Figure C6.20(a), Case I, 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 base of the footing/pile cap (as outlined in Figure C6.20(b), Case II, of the Commentary to the *CHBDC*).

- For Case I, the pressures are based on the proposed embankment fill materials and the existing overburden soils and the following parameters (unfactored) may be used assuming the use of earth fill or rock fill:

	Earth Fill	Rock Fill
Soil unit weight:	21 kN/m^3	19 kN/m^3
Coefficients of static lateral earth pressure:		
Active, K_a	0.31	0.22
At rest, K_o	0.47	0.36

- For Case II, the pressures are based on the rock fill as above or on the granular fill as placed and 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	0.27	0.27
At rest, K_o	0.43	0.43

If the wall support and superstructure allow lateral yielding of the stem, active earth pressures may be used in the geotechnical design of the structure. If the abutment support does not allow lateral yielding, at rest earth pressures should be assumed for geotechnical design. The movement to allow active pressures to develop within the backfill, and thereby assume an unrestrained structure, may be taken as the following (in accordance with Section C6.9.1 and Table C6.6 of the Commentary to the *CHBDC*):

- rotation (i.e. ratio of wall movement to wall height) of approximately 0.002 about the base of a vertical wall;
- horizontal translation of 0.001 times the height of the wall; or
- a combination of both.

A restrained structure is typically a concrete box culvert or a rigid frame bridge where the rotational and/or horizontal movement is not sufficient to mobilize the active earth pressure condition. For this condition, an at-rest pressure plus any compaction surcharge should be included in the design of the structure.

6.5.2 Dynamic

The potential for seismic (earthquake) loading must also be considered for the design of abutment stems/retaining walls in accordance with Section 4.6 of the *CHBDC*. In this regard, the following should be taken into account in the lateral earth pressures.

- Seismic loading may 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 earthquake-induced dynamic earth pressure. According to Table A3.1.7 of the *CHBDC*, this site is located in Seismic Zone 0. The site-specific zonal acceleration ratio for the Hearst area is 0.00. Based on experience, for the subsurface conditions at this site, no amplification of the ground motion will occur (i.e. Site Coefficient, S=1.0).

We understand that this highway route/bridge is not designated as a lifeline bridge. As such, based on Section 4.4.4 of the *CHBDC*, this bridge structure is assigned to Seismic Performance Zone 1. Given this, and in accordance with Section 4.4.5.1 of the *CHBDC,* structures located in Seismic Performance Zone 1 need not be analysed for seismic loads.

6.6 Approach Embankment Design

The new bridge will be replaced along the same alignment as the existing bridge with a final grade of about Elevation 237 m, resulting in a grade raise of about 0.6 m. The new embankment will be approximately 4.5 m above the existing grade adjacent to the river. Further, the new bridge will be widened by a total of 3.5 m resulting in embankment widening on both sides by approximately 1.5 m. The existing embankment side slopes are at slope angles between approximately 2H:1V and 2.5H:1V; the front slopes are at about 1H:1V.

For the detour structure, the approach embankments will be at a final grade of Elevation 236.4 m, which is approximately 4 m above the existing ground in this area. The front face of the proposed abutments will be between 3 m and 4 m back from the creek bank.

The soils encountered below the proposed approach embankments consisted primarily of fill, alluvium, and/or peat underlain by a thin layer of silty clay to clayey silt (cohesive soils) overlying cohesionless soils consisting of sandy silt to sand and very dense till. Based on field observations at the site and the results of our subsurface investigation, settlement of the existing fill materials has occurred in the past. Up to 410 mm of asphalt padding was measured at the abutments. This is likely the result of settlement of the silty clay fill and the presence of alluvium and/or peat below the highway fill.

At all areas, the analyses assume that prior to construction of the new embankments, all surficial organic soils, peat and alluvium will be removed below the new embankment footprint. Since at the main approach embankments alluvium was encountered below the existing fill, the existing fill and the alluvium and/or peat must be removed within a distance of 20 m back from the abutment. This involves sub-excavation to Elevation 231.4 m at the west abutment to Elevation 235.2 m, 20 m behind the west abutment (up to 5.0 m below the existing roadway surface). At the east abutment, this involves sub-excavation to Elevation 231.7 m to Elevation 233.0 m, 20 m behind the east abutment (up to 4.7 m below the existing ground surface). Details of the sub-excavation limits are given in Section 6.7.

The piezometric conditions were assessed based on the water levels observed in Kendall Creek in August 2008 (Elevation 232.4 m) and the groundwater levels noted during drilling of the boreholes in and immediately adjacent to this area. For design purposes in our analysis, the groundwater level has been assumed to be consistent with the adjacent creek level, at Elevation 232.4 m. The design high water level is Elevation 234.0 m.

For the purpose of analysis, granular fill has been considered for the construction of the approach embankments as indicated below using side slopes at 2H:1V. For the small volumes of fill required at this site, rock fill has not been considered in the analysis.

The following sections present the stability and settlement analysis that was carried out for the proposed approach embankments for the main bridge and detour. Recommendations for settlement and stability mitigation measures, if required, will also be addressed. The proximity of the new, existing and detour bridges and approach embankments has also been taken into consideration.

6.6.1 Stability

Analyses were performed on the critical sections of the proposed approach embankments to assess the stability and liquefaction potential for the proposed embankment height and geometry and soil stratigraphy. The critical embankment sections at this site were located at the proposed abutments (detour, main and combined, where appropriate) and include both the front slopes (into the river) and side slopes of the new approaches. The geometry of the proposed approach embankments, existing ground surface and existing riverbed included in the analyses is based on the information obtained from the GA. The analyzed geometry for the east and west abutments is similar to that shown on Figures 2 and 3, respectively (for the final bridge configuration).

6.6.1.1 Methodology

Limit equilibrium slope stability analyses were performed using the commercially available program GeoStudio 2004 (Version 6.20), produced by Geo-Slope International Ltd., employing the Morgenstern-Price method of analysis. For all analyses, the Factor of Safety of numerous potential failure surfaces was computed in order to establish the minimum Factor of Safety. The Factor of Safety is defined as the ratio of the forces tending to resist failure to the driving forces tending to cause failure. A target minimum Factor of Safety of 1.3 is normally adopted for the design of embankment slopes under static conditions. This Factor of Safety is considered adequate for the embankments at this site considering the design requirements and the field data available. The stability analyses were performed to check that the target minimum Factor of Safety was achieved for the design embankment height and geometries. In general, circular slip surfaces were analysed in the design.

6.6.1.2 Parameter Selection

For the cohesionless and cohesive layers, effective stress parameters were employed in the analysis assuming drained conditions. The effective stress parameters (effective friction angle and cohesion) for these soils were estimated from empirical correlations using the results of in situ SPTs and Atterberg limits, in conjunction with engineering judgement considering experience in similar soil conditions.

Summarized below are the simplified stratigraphy and the associated friction angle, cohesion and unit weights employed for the different soil types in the proposed approach/abutment areas.

Soil Type	Unit Weight (kN/m^3)	Angle of Internal Friction	Cohesion (kPa)
New Granular Fill	21	35^o	0
Peat	12	27^o	0
Alluvium	17	27^o	0
Silty Clay to Clayey Silt	17	27^o	0
Sandy Silt to Sand	19	30^o	0
Sandy Silt to Silt to Sand (Till)	21	35^o	0

6.6.1.3 Results of Analysis - Stability

Limit equilibrium analysis indicates a Factor of Safety of greater than 1.8 for the front slopes and cross-sections for both the detour and main bridges assuming front and side slopes no steeper than 2H:1V. The minimum Factor of Safety is based on a deep-seated, global trial failure surface that would impact the operation of the roadway. Therefore, stability mitigation is not required at this site.

The results of stability analysis for the final bridge configuration are shown on Figures 2 and 3 for the east and west approaches, respectively.

6.6.2 Liquefaction Potential and Seismic Analysis

As noted in Section 6.6.2, this site is located in Seismic Zone 0 with a PHA=0.00. Further, the bridge structure is not a lifeline structure. As such, based on Section 4.4.4 of the *CHBDC*, the site is assigned to Seismic Performance Zone 1 and, therefore, in accordance with Section 4.4.5.1 of the *CHBDC*, no liquefaction analysis is required.

6.6.3 Settlement

Settlement of the new approach embankments can be expected as a result of the loading from the new fills on the foundation soils at this site. In addition, depending on the type of fill materials employed in the construction, settlements may also occur due to compression of the embankment fill itself.

To estimate the magnitude of the expected settlements, analyses were carried out on the critical sections of the proposed main bridge and detour approach embankments using hand and spreadsheet calculations.

6.6.3.1 Parameter Selection

The immediate compression of the subsoils were assessed by estimating an elastic modulus of deformation based on the SPT ‘N’-values and empirical correlations found in literature by Bowles (1984) and Kulhawy and Mayne (1990).

The following simplified stratigraphy, unit weights and deformation parameters have been employed in the settlement analysis of the proposed approach embankments. The geometry used in the analysis is shown on Figures 2 and 3 for the east and west approaches, respectively.

Soil	Location	Relevant Boreholes	Range of Elevation/Maximum Thickness (m)	Unit Weight (kN/m^3)	Estimated Deformation Properties
New Granular Fill	East Abutment (Main) West Abutment (Main) East and West Abutment (Detour)	KC09-2, KC09-7 KC09-1, KC09-6 KC09-5, KC09-8	El. 237.0 – 231.7 = 5.3 El. 236.9 – 231.4 = 5.5 El. 234.4 – 231.0 = 3.4	21	-
Silty Clay to Clayey Silt	East Approach (Main) West Approach (Main) West Abutment (Detour)	KC09-3 KC09-4 KC09-5	El. 232.6 – 232.1 = 0.5 El. 235.2 – 232.9 = 2.3 El. 233.4 – 231.6 = 1.8	18	E’ = 5 MPa

Soil	Location	Relevant Boreholes	Range of Elevation/Maximum Thickness (m)	Unit Weight (kN/m^3)	Estimated Deformation Properties
Sandy Silt to Sand	East Approach (Main) West and East Abutment (Detour)	KC09-3 KC09-5, KC09-8	El. 233.0 – 232.6 = 0.4 El. 231.6 – 231.0 = 0.6	19	E' = 7 MPa
Sandy Silt to Silt and Sand (Till)	East and West Approach (Main) East and West Approach (Detour)	KC09-2, KC09-7 KC09-1, KC09-6 KC09-5, KC09-8	Below El. 231.7 = 7* Below El. 231.0 = 5*	21	E' = 30 MPa

* Within the zone of influence below the foundation.

The maximum estimated settlement of the foundation soils in these areas (due to the loading imposed by the new approach embankment fill) is presented below and a discussion on the rate of settlement is included.

6.6.3.2 Results of Analysis - Settlement

We recommend the use of granular fill for the new embankment construction at this site. In this case, the additional settlement from the properly compacted granular fill is expected to be less than 25 mm and will occur during construction for both the main and detour approach embankments. It is recommended that the fines content of the granular fill used for embankment construction be minimized to avoid long-term settlement and maintenance issues.

The estimated magnitude of settlement of the native foundation soils below the new embankment fill (main bridge and detour bridge for base of pad at Elevation 231.0 m) is expected to be less than about 50 mm provided that the existing fill and organic soils (peat and alluvium) have been removed. This settlement will occur rapidly, during construction.

The total estimated settlement of the fills and native subsoils at both the main and detour sites is anticipated to be less than 75 mm. Post-construction settlement is not anticipated and, therefore, mitigation of settlement is not required.

If the higher founding level of the granular pad (i.e. Elevation 231.6 m) is chosen for the detour bridge, settlement of the sandy silt containing organics will occur and is estimated to be less than 50 mm (total settlement of 100 mm). This magnitude of settlement should be considered over the life span of the detour, which is expected to be one construction season (spring to fall) and some maintenance may be required. Further, since organics were not encountered in the sandy silt on the west side of the detour footings, this settlement could be differential between the east and west abutments. Maintenance of the detour roadway during this time period may be required in this case.

6.7 Subgrade Preparation and Embankment Construction

Prior to embankment construction, all topsoil/peat/vegetation/organic soils must be removed below the footprint of the proposed main and detour bridge embankments. Due to the minor grade raise anticipated at the main bridge approaches, the existing fill may be left in place. One exception is the immediate east abutment area where peat was encountered below the fill. This peat (and therefore overlying fill) should be removed to a distance of 10 m behind the abutment and backfilled with new granular fill. The native subsoils are considered to be an appropriate subgrade; however, all softened/loosened soils should be stripped from below the approach embankments, prior to placement of new fill.

At the main east approach embankment, the existing fill and the alluvium and/or peat must be removed between STA 16+409 (abutment) and 16+429 (20 m back from the abutment). The base of the sub-excavation will be Elevation 231.7 m at the abutment rising to Elevation 233.0 m, 20 m behind the abutment. This will result in excavations up to 4.7 m below the existing ground surface.

At the main west approach embankment, the existing fill and/or peat must be removed between STA 16+388 (abutment) and 16+368 (20 m back from the abutment). The base of the sub-excavation will be Elevation 231.4 m at the abutment rising to Elevation 235.2 m, 20 m behind the abutment. This will result in excavations up to 5.0 m below the existing ground surface.

For the detour, the peat/alluvium is only required to be sub-excavated below the granular pad as shown on Figure 1. The base of the sub-excavation for the detour abutment granular pads is 231.6 m or 231.0 m, depending on which founding elevation is chosen. This will result in excavations up to 2.4 m below the existing ground surface.

Granular fill materials and placement should be carried out in accordance with the requirements as outlined in Special Provision SP206S03 above the water level. If filling below the water level is required at the detour for construction of the footing pad, then granular should be used to backfill the excavation, without compaction, to 0.6 m above the water level in accordance with OPSS209. All granular fill should be placed in regular lifts with loose thickness not exceeding 300 mm and compacted to at least 95 percent of the standard Proctor maximum dry density. Side slopes for granular fill embankments should be no steeper than 2H:1V.

The final lift of fill prior to placement of the granular subbase and base courses should be compacted to 100 percent of the Standard Proctor maximum dry density. Inspection and field density testing should be carried out by qualified personnel during placement operations to ensure that appropriate materials are used and that adequate levels of compaction have been achieved.

In order to minimize differential settlement between the existing embankment slopes and the newly placed embankment fill, as at the west approach, the new fill should be keyed into the existing embankment side slope per the requirements of OPSD 208.010.

The abutment front slopes and side slopes adjacent to the river require erosion protection in accordance with SP511S01. Erosion protection should be placed on the slopes to at least 0.5 m above the design high water level. Erosion protection could consist of a minimum 0.6 m thick layer of rip rap (300 mm diameter), rock protection or concrete slope paving. The designer should address the potential for scour below the pile caps in the design of the bridge foundations.

To reduce surface water erosion on the embankment side slopes, topsoil and seeding should be carried out as soon as possible after construction where earth fill is used. If this slope protection is not in place before winter, then alternate protection measures, such as covering the slope with straw or gravel sheeting to prevent erosion, will be required to reduce the potential for remedial works on the side slopes in the spring prior to topsoil and seeding.

6.8 Design and Construction Considerations

6.8.1 Excavations

As noted in Section 6.2, excavations for construction of the footings may extend to Elevation 230.2 m at the east and west main bridge abutments and to Elevation 231.0 m at the east and west detour abutments. In the area of the main bridge, this requires an excavation of up to 6.2 m below the existing roadway. At the detour, this corresponds to an excavation about 3 m below the existing ground surface. The main and detour bridge excavations will be up to 2.2 m and 1.4 m below the water level, respectively, measured in Kendall Creek in August 2008 at Elevation 232.4 m.

Excavations for removal of organic material below the fill under the new approach embankments for the main and detour bridges will be up to 5.0 m at the west approach and 4.7 m at the east approach, corresponding to 1.4 m and 0.7 m below the water level measured in Kendall Creek in August 2008.

Open cut excavations through the existing fill and native soils are feasible, provided they are no steeper than 2H:1V below the water level and 1H:1V above the water level. However, for any footing or granular pad construction adjacent to the creek, the excavations would also have to be supported by a temporary shoring system that controls groundwater inflows, limits the excavation extent and supports and maintains the stability of the existing adjacent roadway embankment. This could be accomplished using a soldier pile and lagging cut-off wall or cofferdam.

Conventional excavation equipment should be suitable for excavation through the on-site soils; however, the contractor shall be made aware of the potential for obstructions in the foundation strata as discussed in Section 6.8.5.

All excavations must be carried out in accordance with the latest edition of the Ontario Occupational Health and Safety Act (OHSA) and Regulations for Construction Projects and good construction practice. The existing fill materials and the native soils (other than till) should be classified as Type 3 soil, according to the OHSA. The very dense till below the water level should be classified as a Type 2 Soil.

6.8.2 Subgrade Protection

The very dense sandy silt to silt and sand till subgrade is susceptible to disturbance from construction traffic (machine and foot), ponded water, etc., once exposed. During construction of the granular pad for the detour footings (if constructed within a cofferdam) and for the main bridge abutment footings, the exposed subgrade (i.e. till) should be protected from machine and foot traffic and weather by the use of a 5 MPa lean concrete "mudslab" placed within 4 hours of first exposure and after review by the Quality Verification Engineer (QVE). The mudslab should be a minimum of 100 mm thick to limit the disturbance and to provide a platform for construction of the spread footing. The contractor should be aware that trafficking over the exposed silty material may not be possible and an NSSP for placement of the mudslab and protection of the subgrade should be contract specifications; an example is included in Appendix C.

6.8.3 Groundwater and Surface Water Control

At the abutments for both the main and detour bridges, the footprint of the foundation elements is located adjacent to Kendall Creek. Further, the base of the excavation may be up to 2.2 m below the water level measured in the creek. Construction of the main bridge footings should be carried out in-the-dry. An NSSP stating this should be included in the contract specifications; an example is included in Appendix C.

In order to construct the footings, groundwater inflow should be expected and controlled dewatering within the temporary shoring/cofferdam will be required in accordance with the special provision and performance level specified below. The shoring should be advanced to an appropriate depth to control groundwater inflow and to minimize ground loss during excavation, backfilling and concrete placement. The Contractor is responsible to ensure that appropriate construction procedures and equipment are used for construction. Surface water should be directed away from the excavation at all times.

To avoid the use of dewatering and/or cofferdams to construct the granular pad for the detour footings, the recommended Granular 'B' Type II can be placed below the water level without compaction, in accordance with OPSS209.

6.8.4 Temporary Shoring

Given the depth of the excavation required to construct the footing pad for the detour bridge and the proximity of the excavation to the existing roadway, temporary roadway protection will be required between the existing and detour bridges on both the east and west sides of the creek. Further, during construction of the foundations for the new bridge after traffic has moved to the detour, temporary shoring will be required to protect the footings of the detour bridge. It is possible that the same shoring could be used for both excavations. In order to get penetration into the very dense till, soldier pile and lagging would be required with the piles installed through pre-augered holes.

Given the proximity of the footing excavations for both the main bridge and detour to the creek, it is unlikely that open (i.e. unsupported) cuts can be utilized on the creek side. Therefore, a temporary cut-off wall (cofferdam) may be required at all abutment locations. Given that the new bridge footing will be immediately adjacent to the existing footing, temporary cofferdams may be required that encompass the existing and new bridge footings in the same excavation. This shoring could consist of soldier pile and lagging or a pre-fabricated box installed within an excavation. In either case, it will be difficult to excavate/drill through the very dense till.

Temporary excavation support systems should be designed and constructed in accordance with Special Provision SP105S19. The lateral movement of the temporary shoring system should meet Performance Level 2.

6.8.5 Obstructions

Due to the proximity of the new bridge footings to the existing, careful consideration should be given to the location of the older abandoned shoring or cofferdam elements (i.e. used to construct the existing bridge footings in 1939) when determining the location of new cofferdams. An Operational Constraint (OC) alerting the contractor to the potential for encountering old shoring should be included in the contract; an example is given in Appendix C.

As observed during drilling of the boreholes at this site, the very dense sandy silt to silt and sand till contains cobbles and boulders. In some cases, casing and/or a rock core barrel was required to penetrate the deposit. Further, boulders and/or erosion protection (i.e. rip-rap) was observed on the side slopes of the existing embankment and into the river and may require removal during construction of the bridge. The existing fill material may also contain cobbles and boulders and/or obstructions. The contractor should be alerted to the presence of cobbles and boulders and obstructions in an OC; an example is included in Appendix C.

Due to the presence of cobbles and boulders as well as any erosion protection, the contractor may experience difficulties installing shoring systems that are to extend below the surface of the very dense till. The contractor should select an appropriate type and method of installation.

7.0 CLOSURE

This report was prepared by Ms. Sarah E. M. Coyne, P.Eng., an Associate and senior geotechnical engineer with Golder. Mr. Fintan Heffernan, P.Eng., the Designated MTO Contact, reviewed the technical aspects and conducted a quality control review of the report.

Report Signature Page

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REFERENCES

Bowles, J.E., 1984. Physical and Geotechnical Properties of Soils, 2nd Edition. McGraw-Hill Book Company, New York.

Canadian Foundation Engineering Manual, 1992. Third Edition. Canadian Geotechnical Society, Technical Committee on Foundations, 512p.

Canadian Highway Bridge Design Code and its Commentary, 2006. CAN/CSA-S6-06, Canadian Standards Association.

Kulhawy, F.H. and Mayne, P.W., 1990. Manual on Estimating Soil Properties for Foundation Design. EL-6800, Research Project 1493-6. Prepared for Electric Power Research Institute, Palo Alto, California.

Ontario Provincial Standards Specifications (OPSS) and Drawings (OPSD).

Table 1: Evaluation of Foundation Alternatives

Options	Ranking	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Spread Footings at same founding level as existing footings (on very dense till)	1 (main bridge)	■ Straightforward construction	■ Cofferdam construction required for spread footing construction adjacent to river ■ Extra excavation required (and therefore depth of cofferdam) compared to footings placed on the surface of the till ■ Interference with existing footing or old cofferdam elements possible	■ Lower cost relative to deep foundations ■ Slightly more costly than the higher foundation elevation	■ Dewatering required ■ Risk of encountering old cofferdam
Spread Footings founded on surface of very dense till	2 (main bridge)	■ Straightforward construction	■ Cofferdam construction required for spread footing construction adjacent to river ■ Fill material may be encountered at the founding level immediately adjacent to the existing footing requiring over-excavation and replacement with structural concrete to bring to the founding level ■ Interference with existing footing or old cofferdam elements possible	■ Lower cost relative to deep foundations	■ Risk of encountering old cofferdam
Spread Footings on granular pad over very dense till	1 (detour bridge)	■ Straightforward construction ■ Cofferdam may not be required to construct granular pad	■ Granular fill placement for footing pad could be below the water level; proper compaction of fill above the water level will be required ■ Dewatering may be required	■ Lower cost relative to deep foundations or footings on native material	■ Risk of minor settlement of granular pad below footing

Table 1: Evaluation of Foundation Alternatives (Continued)

Options	Ranking	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Spread Footings on granular pad over sandy silt deposit	2 (detour bridge)	■ Straight forward construction ■ Less excavation required than for granular pad constructed on very dense till ■ Proper compaction of granular pad more likely without cofferdam	■ Settlement of sandy silt deposit containing organics will result in settlement over the life of the detour and differential settlement between the east and west abutments	■ Lower cost relative to spread footings on a granular pad over the till	■ Risk of minor settlement of sandy silt deposit below granular pad; maintenance of roadway would be required
Steel H-Piles driven into very dense till (through pre-augered holes)	3	■ Integral abutment bridge possible	■ Pre-augered holes though bouldery till deposit required to extend piles to sufficient minimum depth ■ High possibility of piles “hanging up” on a cobble/boulder within the till deposit and alignment and/or location concerns ■ Cofferdam construction be required for pile cap construction adjacent to river	■ Lower relative costs compared with caisson option	■ Difficulty achieving pile capacity if hung-up on a boulder
Caissons socketted into very dense till	4	■ Reduced number of deep elements compared to steel H-piles	■ Temporary liners may be required for ground support during caisson advance ■ Concrete for caissons would have to be placed by tremie methods below the water level ■ Difficulty advancing caissons through bouldery till deposit ■ Cofferdam construction required for caisson cap construction adjacent to river	■ Cost many times higher than for piles	■ Risk of difficulties penetrating the bouldery till deposit

PLOT DATE: September 17, 2009
FILENAME: N:\Active\2007\1190 Sudbury\1191\07-1191-0007 LEA Frederickhouse & Kendall\5000 Drawings\Kendall Creek Dwgs\CPS Stamped\Dwg2-3--KCFND1-2.dwg

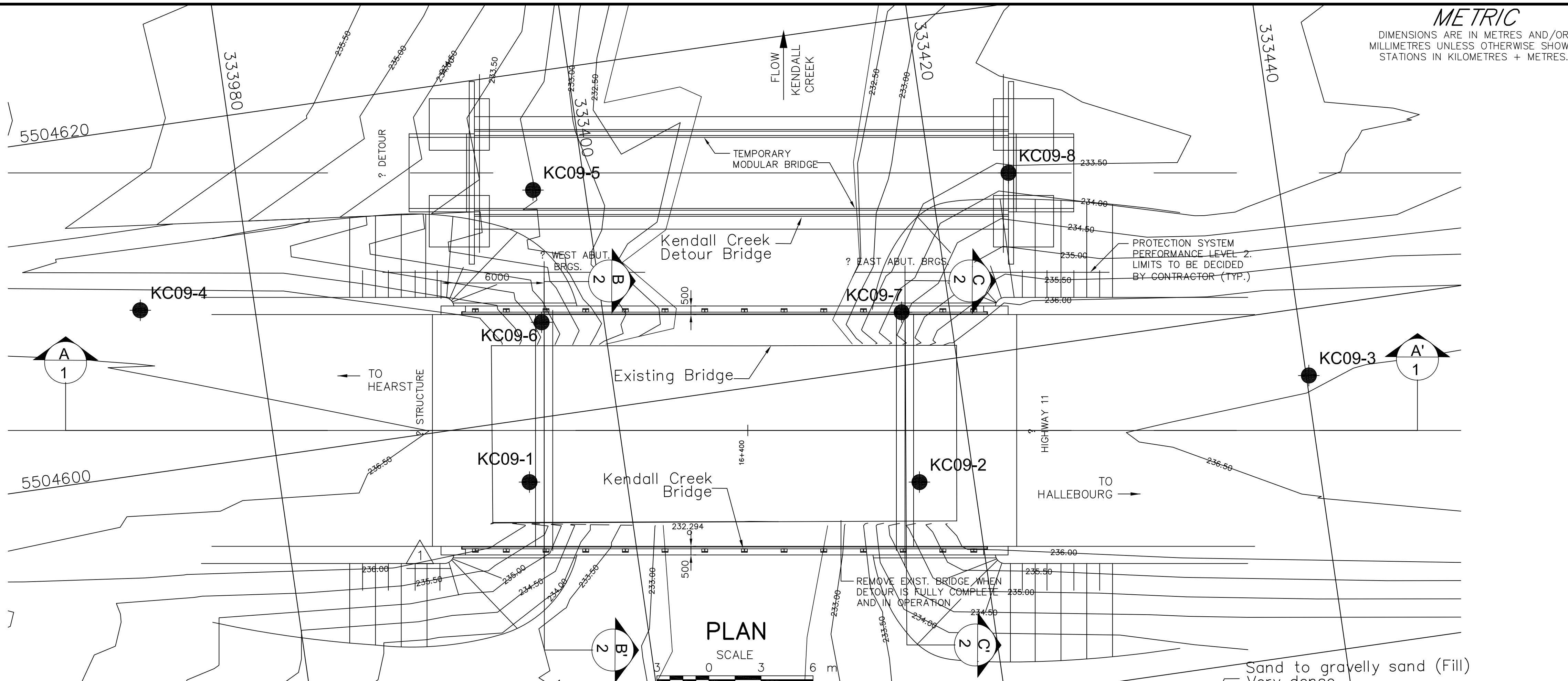
CONT No.
WP No. 5413-04-00

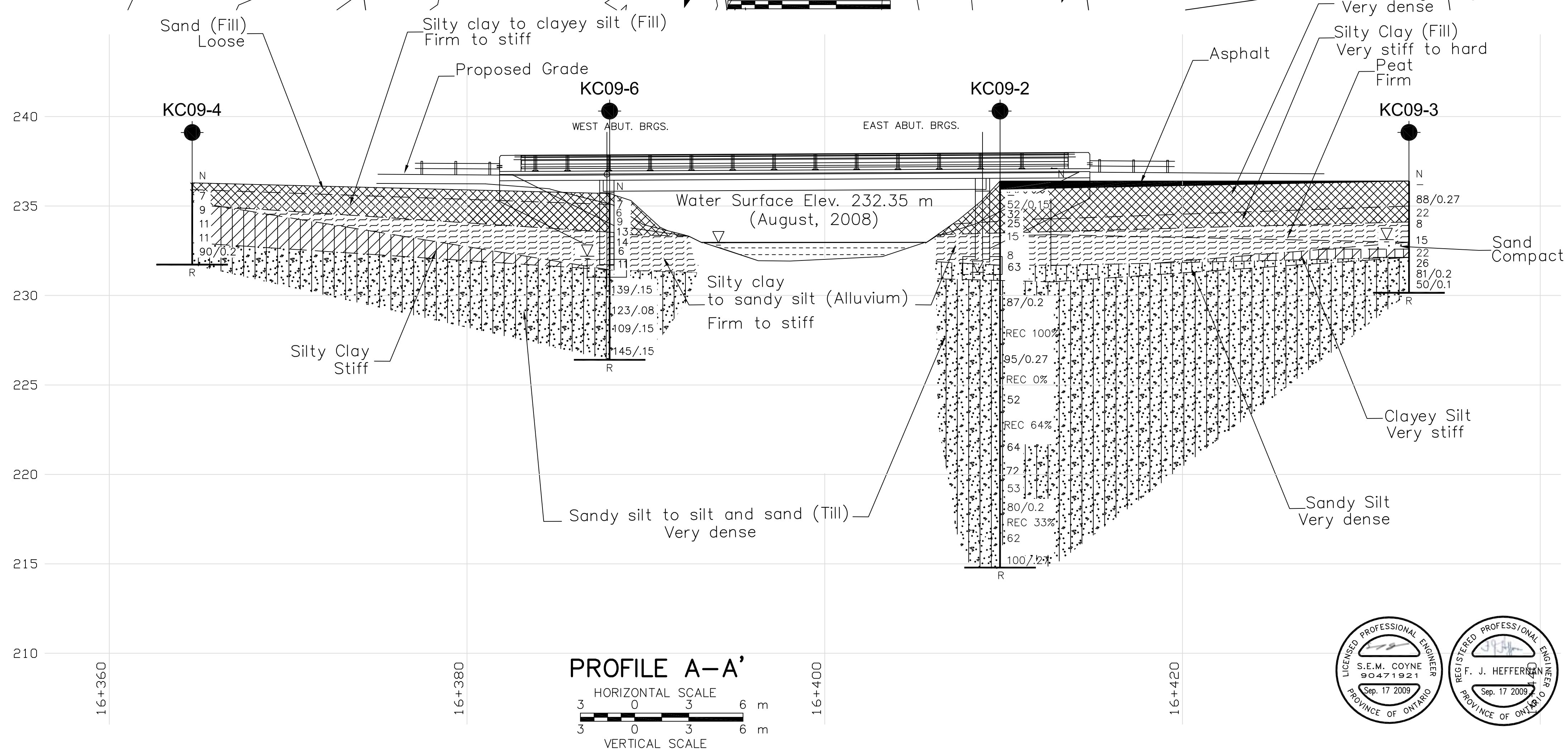
SHEET

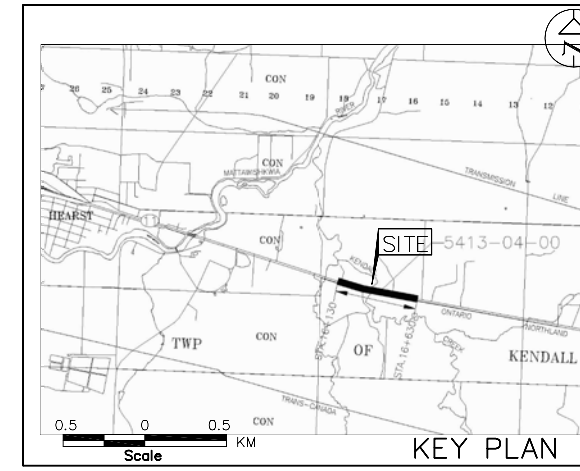
HIGHWAY 11 CROSSING
KENDALL CREEK BRIDGE
BOREHOLE LOCATION AND
SOIL STRATA

Golder Associates
Golder Associates Ltd.
SUDBURY, ONTARIO, CANADA

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







KEY PLAN

LEGEND

- Borehole – Current Investigation
- Dynamic Cone Penetration Test
- N Standard Penetration Test Value
- 4 Blows/0.3 m unless otherwise stated (Std. Pen. Test, 475j/blow)
- WL upon completion of drilling
- R Refusal

No.	ELEVATION(m)	CO-ORDINATES NORTHING	CO-ORDINATES EASTING
KC09- 1	236.4	5504596.6	333994.4
KC09- 2	236.4	5504592.8	334016.8
KC09- 3	236.4	5504596.1	334040.2
KC09- 4	236.3	5504609.5	333973.4
KC09- 5	233.4	5504613.2	333997.0
KC09- 6	235.7	5504605.5	333996.4
KC09- 7	235.4	5504602.9	334017.2
KC09- 8	233.4	5504610.2	334024.5

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 Contracts Documents.

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 plan provided in digital format by LEA Group Ltd., drawing file no. 8461-KEN-P01.dwg (received June, 2009) and key plan, drawing file no. 2007-032 Key Plan-Kendall Creek.dwg (received June, 2009).

NO.	DATE	BY	REVISION

Geocres No. 42G-30

HWY. 11	PROJECT NO. 07-1191-0007		DIST.
SUBM'D. LG	CHKD. AB	DATE: Sep. 2009	SITE: 39W-035
DRAWN: MM	CHKD. SEMC	APPD. FJH	DWG. 1

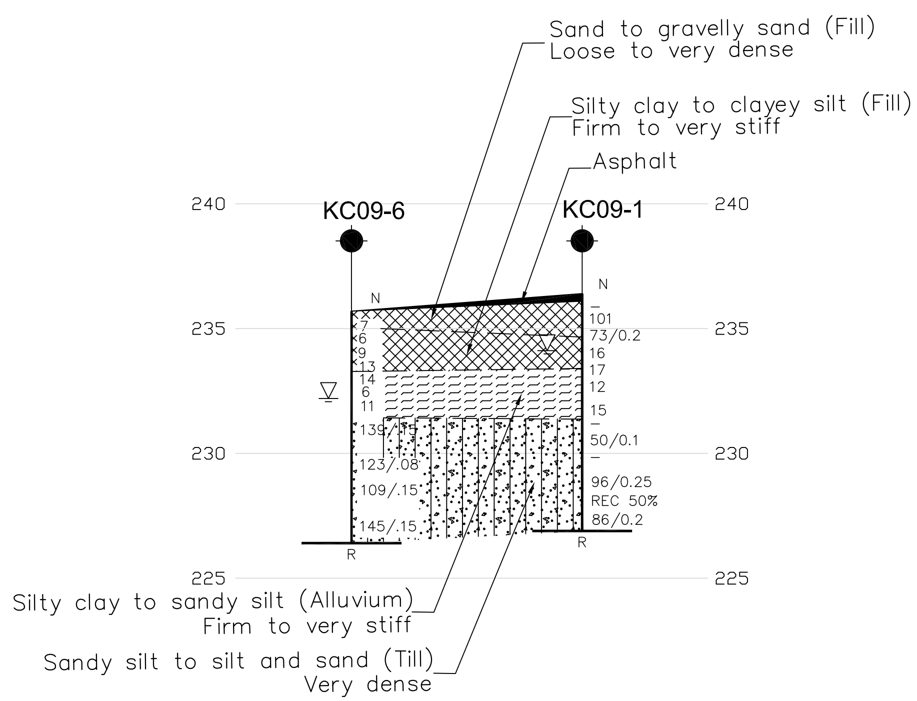
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

CONT No.
WP No. 5413-04-00

HIGHWAY 11 CROSSING
KENDALL CREEK BRIDGE
SOIL STRATA

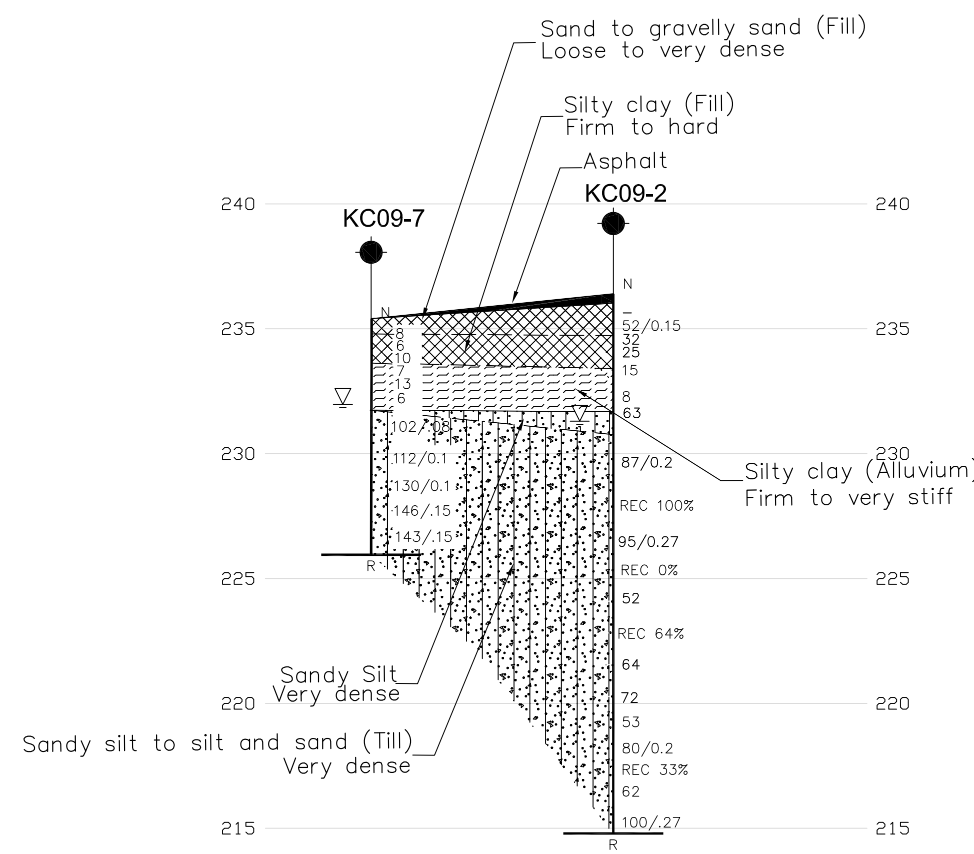
SHEET

Golder Associates Ltd.
SUDBURY, ONTARIO, CANADA



SECTION B–B'

HORIZONTAL SCALE 3 0 3 6 m
3 0 3 6 m VERTICAL SCALE



SECTION C–C'

HORIZONTAL SCALE 3 0 3 6 m
3 0 3 6 m VERTICAL SCALE

LEGEND

- Borehole – Current Investigation
- Dynamic Cone Penetration Test
- N — Standard Penetration Test Value
- 4 — Blows/0.3 m unless otherwise stated (Std. Pen. Test, 475j/blow)
- ∇ — WL upon completion of drilling
- R — Refusal

No.	ELEVATION(m)	CO-ORDINATES	
		NORTHING	EASTING
KC09- 1	236.4	5504596.6	333994.4
KC09- 2	236.4	5504592.8	334016.8
KC09- 3	236.4	5504596.1	334040.2
KC09- 4	236.3	5504609.5	333973.4
KC09- 5	233.4	5504613.2	333997.0
KC09- 6	235.7	5504605.5	333996.4
KC09- 7	235.4	5504602.9	334017.2
KC09- 8	233.4	5504610.2	334024.5

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 Contracts Documents.

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 plan provided in digital format by LEA Group Ltd., drawing file no. 8461-KEN-P01.dwg (received June, 2009) and key plan, drawing file no. 2007-032 Key Plan-Kendall Creek.dwg (received June, 2009).

LICENSED PROFESSIONAL ENGINEER
S.E.M. COYNE
90471921
Sep. 17 2009
PROVINCE OF ONTARIO

REGISTERED PROFESSIONAL ENGINEER
F. J. HEFFERNAN
Sep. 17 2009
PROVINCE OF ONTARIO

NO.	DATE	BY	REVISION

Geocres No. 42G-30

HWY. 11		PROJECT NO. 07-1191-0007	DIST.
SUBM'D. LG	CHKD. AB	DATE: Sep. 2009	SITE: 39W-035
DRAWN: MM	CHKD. SEMC	APPD. FJH	DWG. 2

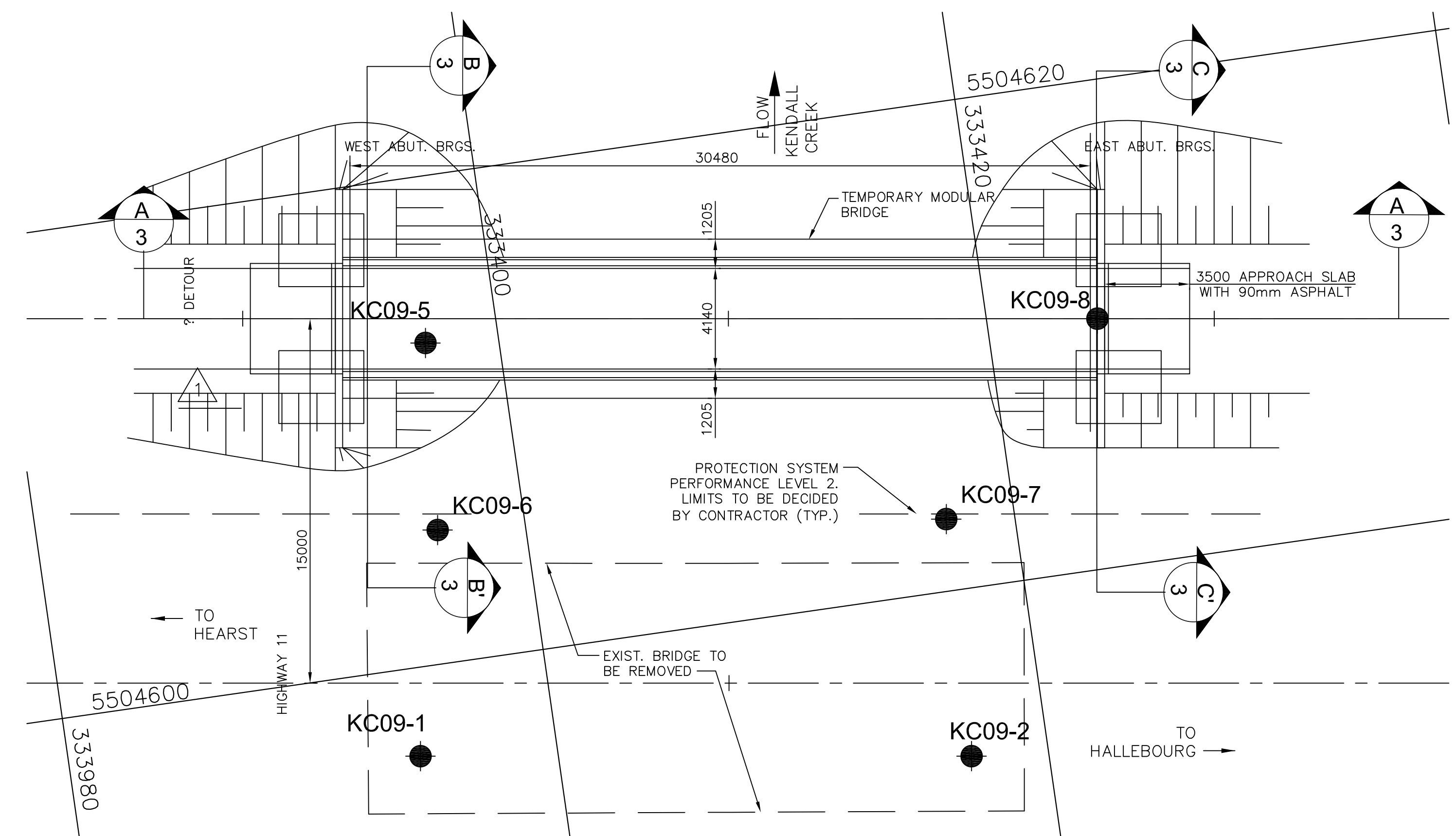
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

CONT No. WP No. 5413-04-00

SHEET

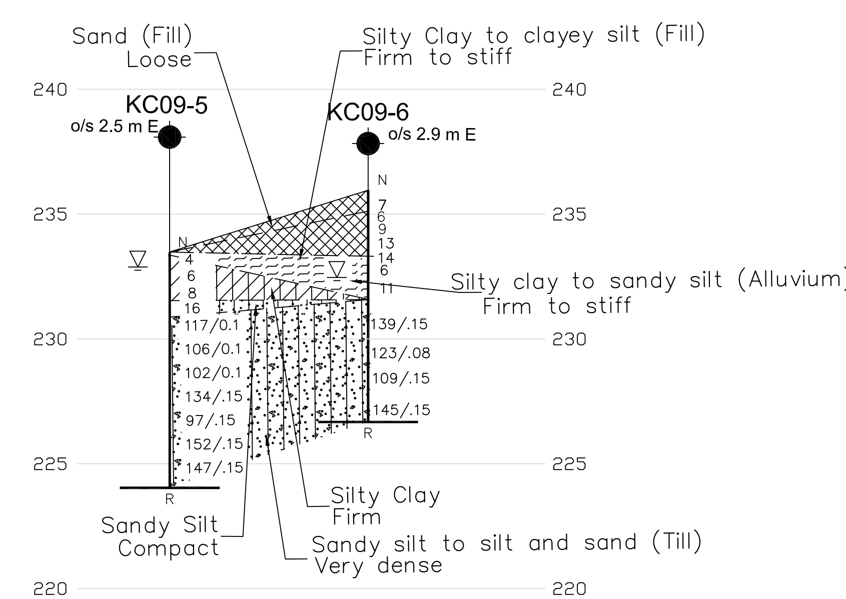
HIGHWAY 11 CROSSING
KENDALL CREEK DETOUR BRIDGE
BOREHOLE LOCATION AND
SOIL STRATA

Golder Associates
Golder Associates Ltd.
SUDBURY, ONTARIO, CANADA



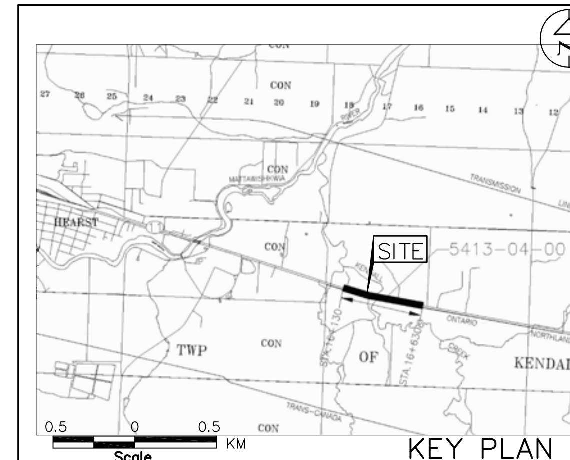
PLAN

SCALE
3 0 3 6 m



SECTION B–B'

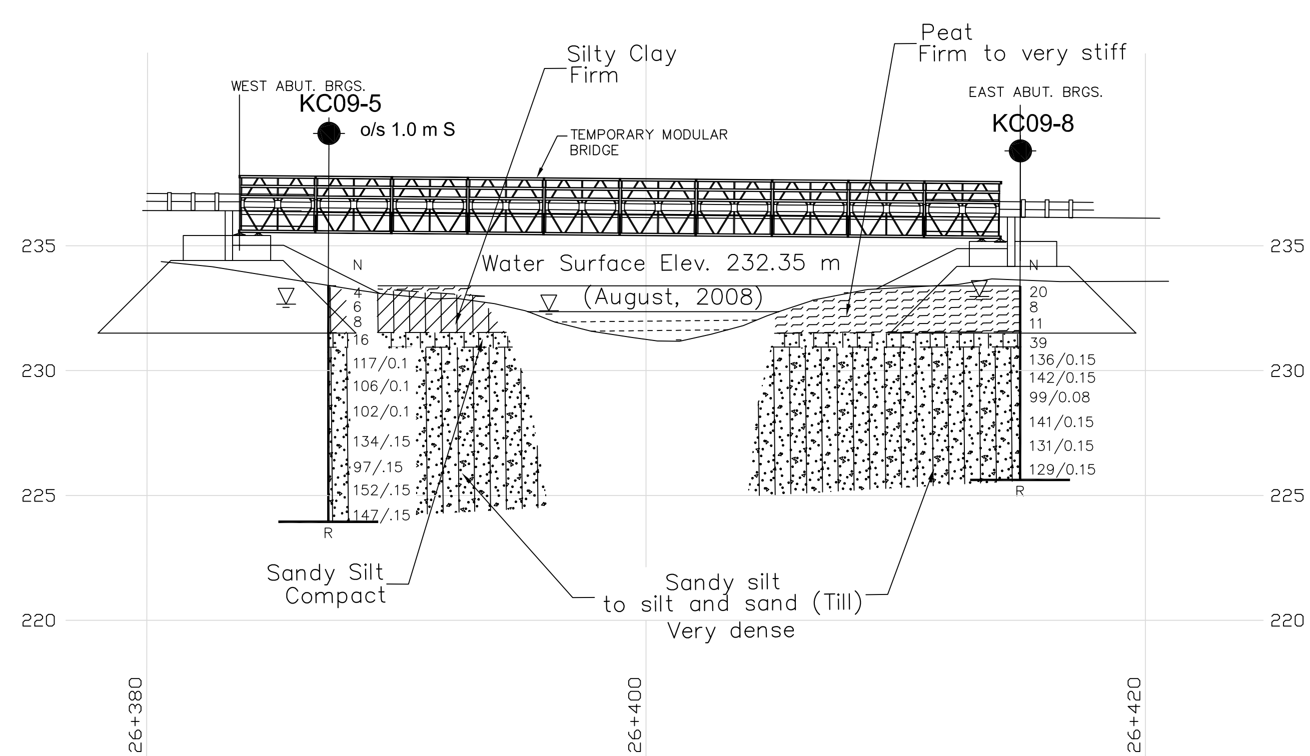
HORIZONTAL SCALE
3 0 3 6 m
3 0 3 6 m
VERTICAL SCALE



LEGEND

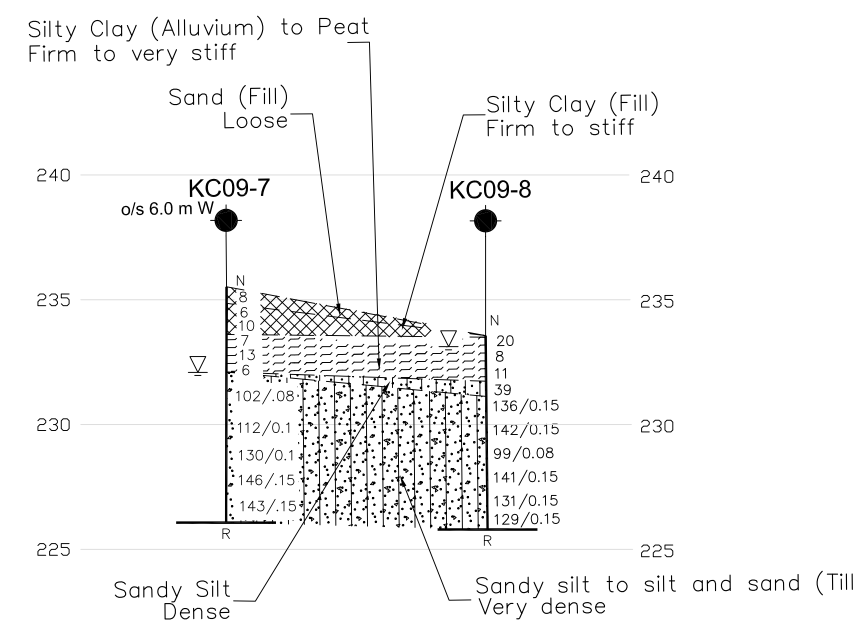
- Borehole – Current Investigation
- Dynamic Cone Penetration Test
- N Standard Penetration Test Value
- 4 Blows/0.3 m unless otherwise stated (Std. Pen. Test, 475j/blow)
- WL upon completion of drilling
- R Refusal

No.	ELEVATION(m)	CO-ORDINATES NORTHING	CO-ORDINATES EASTING
KC09– 1	236.4	5504596.6	333994.4
KC09– 2	236.4	5504592.8	334016.8
KC09– 3	236.4	5504596.1	334040.2
KC09– 4	236.3	5504609.5	333973.4
KC09– 5	233.4	5504613.2	333997.0
KC09– 6	235.7	5504605.5	333996.4
KC09– 7	235.4	5504602.9	334017.2
KC09– 8	233.4	5504610.2	334024.5



PROFILE A–A'

HORIZONTAL SCALE
3 0 3 6 m
3 0 3 6 m
VERTICAL SCALE



SECTION C–C'

HORIZONTAL SCALE
3 0 3 6 m
3 0 3 6 m
VERTICAL SCALE

NOTES

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

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REFERENCE

Base plan provided in digital format by LEA Group Ltd., drawing file no. 8461-KEN-DETOUR-B01.dwg (received June, 2009) and key plan, drawing file no. 2007-032 Key Plan-Kendall Creek.dwg (received June, 2009).

LICENSED PROFESSIONAL ENGINEER
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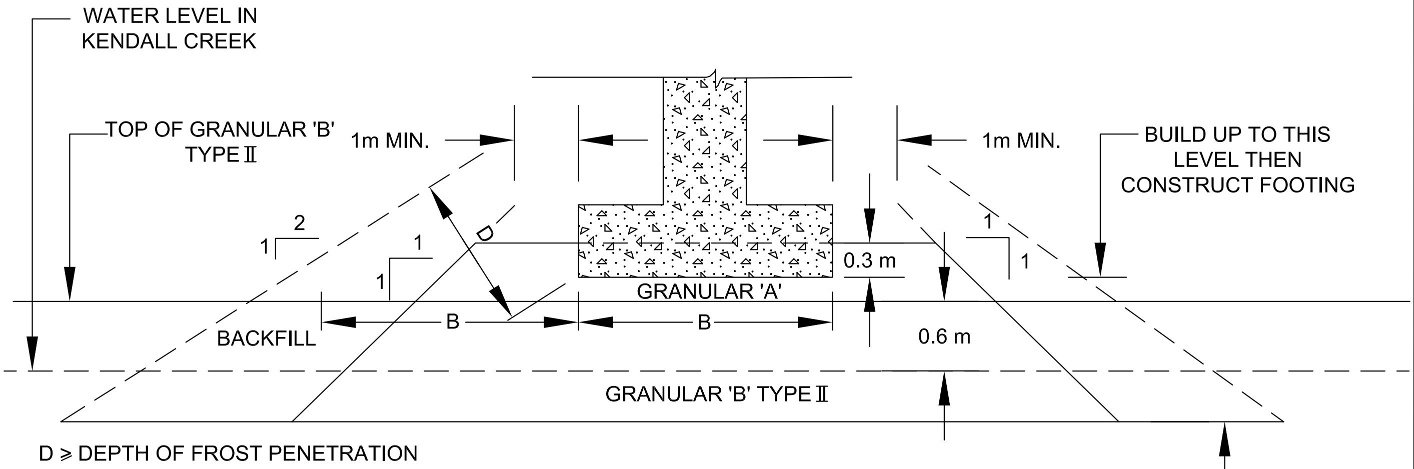
90471921
Sep. 17 2009
PROVINCE OF ONTARIO

REGISTERED PROFESSIONAL ENGINEER
F. J. HEFFERNAN
Sep. 17 2009
PROVINCE OF ONTARIO

NO.	DATE	BY	REVISION

Geocres No. 42G-30			
HWY. 11		PROJECT NO. 07-1191-0007	DIST.
SUBM'D. LG	CHKD. AB	DATE: Aug. 2009	SITE: 39W-035
DRAWN: MM	CHKD. SEMC	APPD. FJH	DWG. 3



SECTION

CONSTRUCTION SEQUENCE:

1. REFER TO ACCOMPANYING FOUNDATION DESIGN REPORT, SECTION 6.2.
2. REMOVE SUBSOILS UNDER FOOTPRINT OF COMPACTED GRANULAR CORE TO ELEVATION SPECIFIED.
3. PLACE GRANULAR 'B' TYPE Ⅱ TO 0.6 m ABOVE THE GROUND WATER LEVEL IN ACCORDANCE WITH OPSS 209.
4. COMPACT GRANULAR 'B' TYPE Ⅱ IN ACCORDANCE WITH SP105 510.
5. PLACE AND COMPACT GRANULAR 'A' TO UNDER SIDE OF FOOTING LEVEL.
6. CONSTRUCT CONCRETE FOOTING.
7. PLACE REMAINDER OF GRANULAR 'A' AND BACKFILL AS REQUIRED.
8. SOURCE M.T.C 1982.

NOT TO SCALE

PROJECT
GWP 5413-04-00
KENDALL CREEK DETOUR BRIDGE

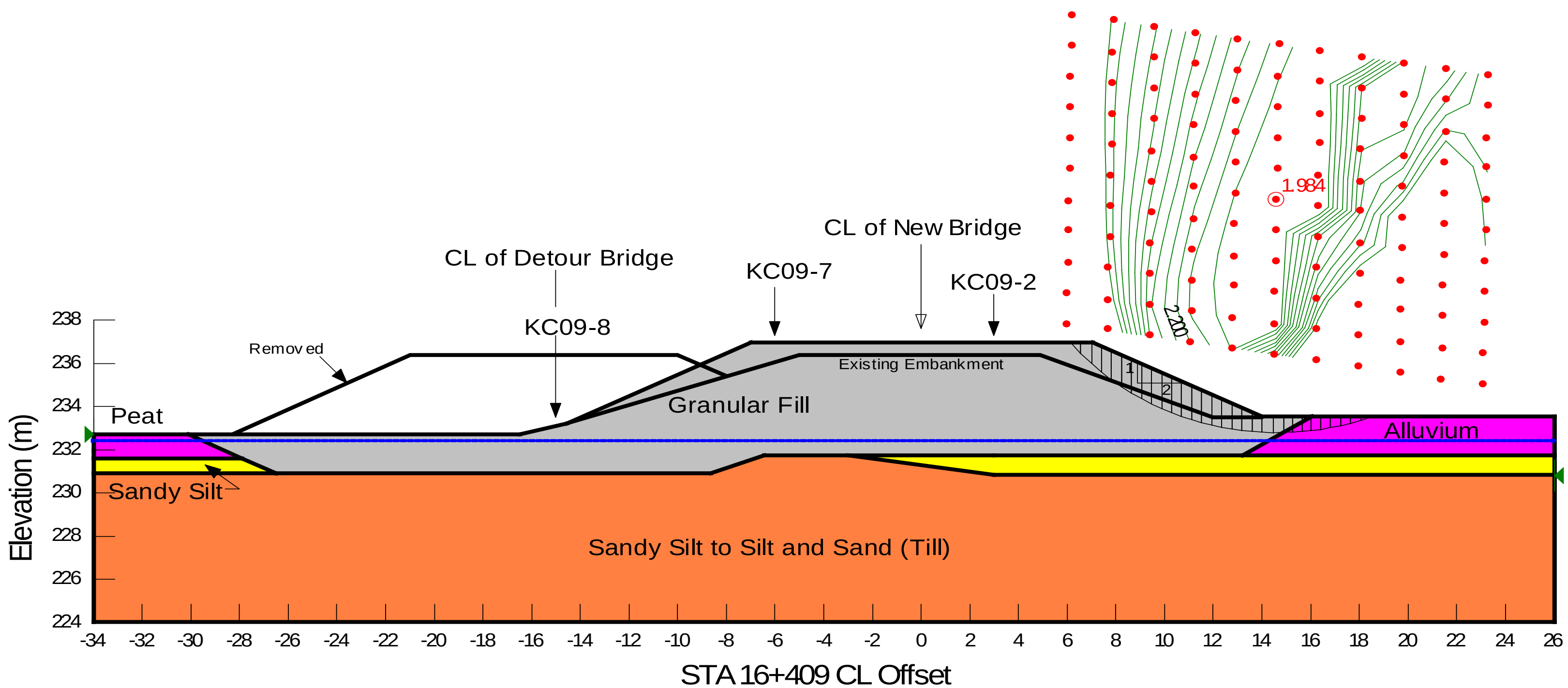
TITLE
TYPICAL DETOUR ABUTMENT ON COMPACTED FILL CORE

Golder Associates
Sudbury, Ontario, Canada

PROJECT No.		07-1191-0007	FILE No. 07-1191-0007FIG1.dwg	
DESIGN			SCALE NTS	REV.
CAD	MM	SEP 2009	FIGURE No.	
CHECK	SEMC	SEP 2009		
REVIEW	FJH	SEP 2009	1	

STABILITY ANALYSIS
East Abutment
Final Embankment Configuration

FIGURE 2



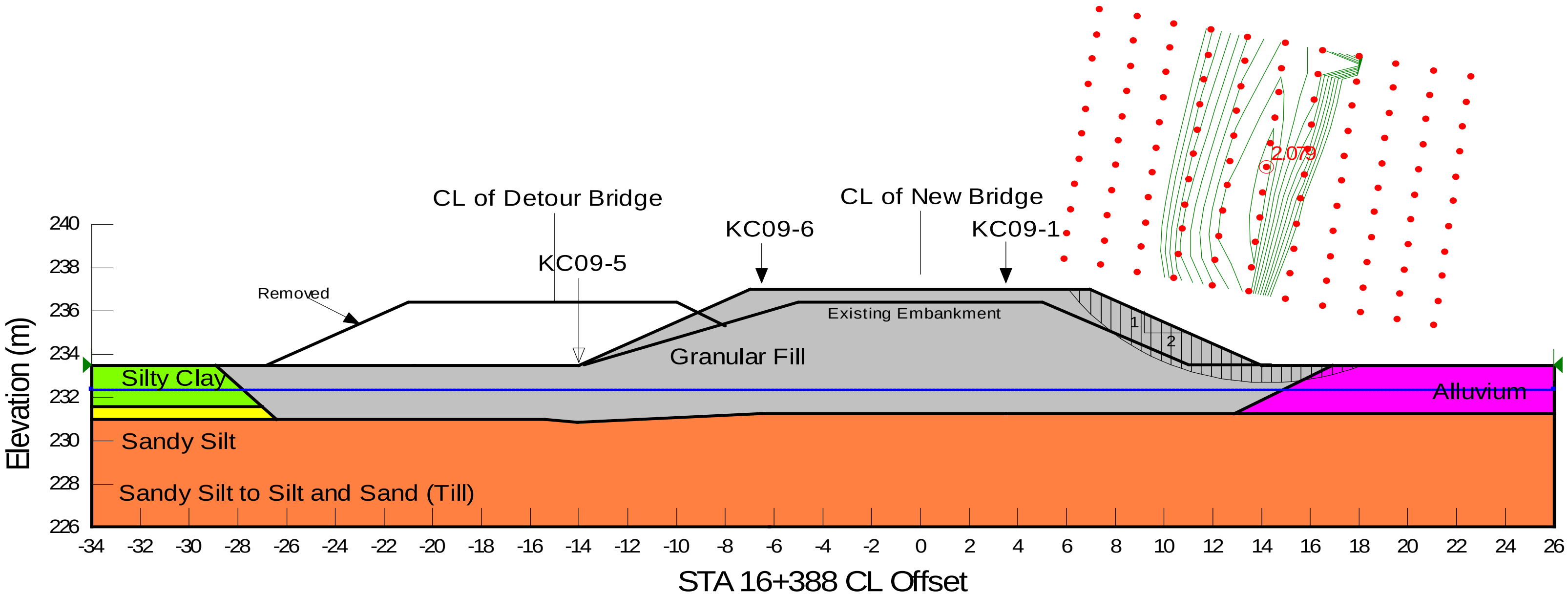
Date: September 2009
Project: 07-1191-0007-KC

Golder Associates

Drawn: EC
Checked: SEMC

STABILITY ANALYSIS
West Abutment
Final Embankment Configuration

FIGURE 3



Date: September 2009
Project: 07-1191-0007-KC

Golder Associates

Drawn: EC
Checked: SEMC

APPENDIX A

RECORD OF BOREHOLES

Golder Associates

RECORD OF BOREHOLE No KC09-1

 METRIC

PROJECT 07-1191-0007			
W.P. 5413-04-00	LOCATION N 5504596.6 ;E 333994.4		ORIGINATED BY EHS
DIST	HWY 11	BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers, NW Casing, Wash Boring	COMPILED BY MM
DATUM Geodetic		DATE April 15, 2009	CHECKED BY SEMC

SOIL PROFILE

ELEV / DEPTH	DESCRIPTION
236.4	GROUND SURFACE
0.0 / 236.1	ASPHALT
0.3	Sand to gravelly sand, some silt (FILL) Very dense Light brown Moist (Frozen)
234.6 / 1.8	Silty clay, trace sand, trace gravel (FILL) Very stiff Grey to brown Moist (Frozen)
233.4 / 3.0	Silty clay, trace sand, to sand, trace gravel, trace organics (ALLUVIUM) Stiff to very stiff Grey to dark brown to black Moist Containing cobbles below 4.5 m depth.
231.4 / 5.0	Sandy silt to silt and sand, some clay, trace gravel, containing cobbles and boulders (TILL) Very dense Grey Wet
226.9 / 9.5	End of Borehole Casing Refusal

Notes:

1. Water level at a depth of 2.3 m below ground surface (Elev. 234.1 m) upon completion of drilling.

2. Hard augering from 1.7 m to 1.8 m depth.

3. Hard drilling below 4.3 m depth. Auger refusal at 4.4 m depth. Switched to NW Casing at 5.0 m depth. Difficult casing/corebarrel advance due to cobbles and boulders.

SAMPLES

NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR	SA	SI	CL
1	AS	-				
2	SS	101	10	77	(13)	
3	SS	73/0.2				
4	SS	16				
5	SS	17				
6	SS	12	3	3	46	48
7	SS	15				
	SC	-				
8	SS	50/0.1				
	SC	-				
9	SS	96/0.25	6	32	51	13
	SC	REC 50%				
10	SS	86/0.2				

GROUND WATER CONDITIONS: water level symbol at Elev. 234.1

ELEVATION SCALE: 236, 235, 234, 233, 232, 231, 230, 229, 228, 227

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 (%): 10 20 30

41

UNIT WEIGHT γ kN/m^3

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

Golder Associates

RECORD OF BOREHOLE No KC09-2

 METRIC

PROJECT 07-1191-0007			
W.P. 5413-04-00	LOCATION N 5504592.8 ;E 334016.8	ORIGINATED BY EHS	
DIST	HWY 11	BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers, NW Casing, Wash Boring	COMPILED BY MM
DATUM Geodetic	DATE April 16, 2009	CHECKED BY SEMC	

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	GR	SA	SI	CL
236.4	GROUND SURFACE								
0.0 / 236.0	ASPHALT								
0.4	Sand to gravelly sand, trace silt (FILL) Very dense Light brown Moist (Frozen)	1	AS	-					
		2	SS	52/0.15					
234.7 / 1.7	Silty clay, trace sand, trace gravel, trace to some organics, trace asphalt (FILL) Very stiff to hard Brown and grey Moist (Frozen)	3	SS	32					
		4	SS	25					
233.4 / 3.0	Silty clay, trace sand, to sandy silt, some organics (ALLUVIUM) Firm to very stiff Brown and grey Moist Fibrous peat layer between 3.4 m and 3.7 m depth.	5	SS	15					
		6	SS	8					
231.7 / 4.7	Sandy SILT, trace to some clay, trace to some gravel Very dense Grey Moist	7	SS	63	▽	10	24	57	9
230.8 / 5.6	Sandy silt to silt and sand, trace gravel, containing cobbles and boulders (TILL) Very dense Grey Wet	8	SS	87/0.2					
			SC	REC 100%					
		9	SS	95/0.27					
			SC	REC 0%					
		10	SS	52					
			SC	REC 64%					
		11	SS	64		2	36	45	17
		12	SS	72					

Elevation scale: 236, 235, 234, 233, 232, 231, 230, 229, 228, 227, 226, 225, 224, 223, 222

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 (%): 10 20 30

84.7

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

Golder Associates

RECORD OF BOREHOLE No KC09-2

 METRIC

PROJECT 07-1191-0007
W.P. 5413-04-00
LOCATION N 5504592.8 ;E 334016.8
ORIGINATED BY EHS
DIST HWY 11
BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers, NW Casing, Wash Boring
COMPILED BY MM
DATUM Geodetic
DATE April 16, 2009
CHECKED BY SEMC

--- CONTINUED FROM PREVIOUS PAGE ---

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	GR	SA	SI	CL
	Sandy silt to silt and sand, trace gravel, containing cobbles and boulders (TILL) Very dense Grey Wet	13	SS	53	221	~10				
					220					
		14	SS	80/0.2						
			SC	REC 33%	219					
		15	SS	62	218	~8	8	41	38	13
					217					
					216					
214.8 21.6		16	SS	100/.27	215					
	End of Borehole Casing Refusal									

Notes:

1. Water level at a depth of 5.1 m below ground surface (Elev. 231.3 m) upon completion of drilling.

2. Hard drilling below 4.9 m depth. Switched to NW Casing below this depth. Difficult casing/corebarrel advance due to cobbles and boulders.

Column headings: SOIL PROFILE; SAMPLES; STRAT PLOT; GROUND WATER CONDITIONS; 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 (%) 10 20 30; UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%)

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

Golder Associates

RECORD OF BOREHOLE No KC09-3

 METRIC

PROJECT 07-1191-0007
W.P. 5413-04-00
LOCATION N 5504596.1 ;E 334040.2
ORIGINATED BY EHS
DIST
HWY 11
BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers
COMPILED BY MM
DATUM Geodetic
DATE April 17, 2009
CHECKED BY SEMC

ELEV / DEPTH (m)	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
236.4	GROUND SURFACE					
0.0	ASPHALT					
0.2	Sand, trace gravel, trace silt (FILL); Very dense; Brown; Moist (Frozen)	1	AS	-		
		2	SS	88/0.27		
234.9 / 1.5	Silty clay, trace sand, trace gravel (FILL); Very stiff; Brown; Moist (Frozen)	3	SS	22		
234.1 / 2.3	PEAT, fibrous; Firm; Brown to black; Moist	4	SS	8		
233.0 / 3.4	SAND, some gravel, trace silt; Compact; Light brown; Wet	5	SS	15	▽	
232.6 / 3.8	CLAYEY SILT, trace clay, trace to some sand, trace gravel; Very stiff; Brown; Wet	6	SS	22		
232.1 / 4.3	Sandy silt to silt, some sand, trace clay, trace gravel (TILL); Compact to very dense; Grey to brown; Wet	7	SS	26		
		8	SS	81/0.2		2 14 79 5
		9	SS	50/0.1		
230.2 / 6.2	End of Borehole; Spoon Refusal					

Notes:

1. Water level at 3.3 m depth below ground surface (Elev. 233.1 m) upon completion of drilling.

2. Hard drilling between 2.9 m and 3.8 m depth and below 5.5 m depth.

Column headings: 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); ELEVATION SCALE (236, 235, 234, 233, 232, 231); PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 10, 20, 30; UNIT WEIGHT γ kN/m³; STRAT PLOT.

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

Golder Associates

RECORD OF BOREHOLE No KC09-4

 METRIC

PROJECT 07-1191-0007
W.P. 5413-04-00
DIST HWY 11
DATUM Geodetic
LOCATION N 5504609.5 ;E 333973.4
BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers
DATE April 17, 2009
ORIGINATED BY EHS
COMPILED BY MM
CHECKED BY SEMC

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
236.3	GROUND SURFACE											
0.0	Gravelly sand, trace silt (FILL) Grey Moist					236						
235.9 0.4	Silty clay, trace sand, trace organics (FILL) Firm Brown Moist	1	SS	7								
235.2 1.1	SILTY CLAY, trace sand, layered Stiff Brown Moist to wet	2	SS	9		235	42					
		3	SS	11		234						
232.9 3.4	Sandy silt to silt and sand, some clay, trace gravel (TILL) Very dense Brown to grey Moist	4	SS	11		233			3	24	58	15
		5	SS	90/0.2		232						
231.7 4.6	End of Borehole Auger Refusal Note: 1. Borehole dry upon completion of drilling.											

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 (%) 10 20 30

REMARKS & GRAIN SIZE DISTRIBUTION (%)

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

Golder Associates

RECORD OF BOREHOLE No KC09-5

1 OF 1 **METRIC**

PROJECT 07-1191-0007
W.P. 5413-04-00 LOCATION N 5504613.2 ;E 333997.0 ORIGINATED BY MR
DIST HWY 11 BOREHOLE TYPE Portable Equipment, NW Casing, BW Casing COMPILED BY MM
DATUM Geodetic DATE April 23, 2009 CHECKED BY SEMC

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
233.4	GROUND SURFACE						
0.0	SILTY CLAY, trace sand, trace organics Firm Brown Wet	1	SS	4	▽		
		2	SS	6			
		3	SS	8		Wp–W: 50 (40.0)	
231.6 1.8	Sandy SILT, some clay, some gravel Compact Grey Wet	4	SS	16			15 28 (57)
231.0 2.4	Sandy silt to silt and sand, some clay, trace gravel, containing cobbles and boulders (TILL) Very dense Grey Wet	5	SS	117/0.1			
		6	SS	106/0.1			
		7	SS	102/0.1			
		8	SS	134/.15			0 30 52 18
		9	SS	97/.15			
		10	SS	152/.15			
224.0		11	SS	147/.15			
9.4	End of Borehole Spoon Refusal						

Notes:

1. Water level at a depth of 0.7 m below ground surface (Elev. 232.7 m) upon completion of drilling.

2. Difficult casing advance due to cobbles and boulders below 2.4 m depth. Advanced borehole with NW Casing to 2.6 m depth. Switched to BW Casing below this depth.

Chart headings: SOIL PROFILE; SAMPLES; STRAT PLOT; ELEVATION SCALE (233 to 224); 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

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

Golder Associates

RECORD OF BOREHOLE No KC09-6

 METRIC

PROJECT 07-1191-0007			
W.P. 5413-04-00	LOCATION N 5504605.5 ;E 333996.4		ORIGINATED BY MR
DIST	HWY 11	BOREHOLE TYPE Portable Equipment, NW Casing, BW Casing	COMPILED BY MM
DATUM Geodetic	DATE April 24, 2009		CHECKED BY SEMC

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
235.7	GROUND SURFACE					
0.0	Sand, some organics, trace clay, trace asphalt (FILL) Loose Brown Wet	1	SS	7		
235.1 / 0.6	Silty clay to clayey silt, trace sand, trace gravel, containing sandy silt layers (FILL) Firm to stiff Brown Wet	2	SS	6		
		3	SS	9		
		4	SS	13		
233.3 / 2.4	Silty clay, trace sand, to sandy silt, trace gravel, trace organics, containing organic layers (ALLUVIUM) Firm to stiff Grey to black Wet	5	SS	14		
		6	SS	6	∇	
		7	SS	11		
231.4 / 4.3	Sandy silt to silt and sand, some gravel, some clay, containing cobbles and boulders (TILL) Very dense Grey Wet	8	SS	139/.15		
		9	SS	123/.08		25 26 36 13
		10	SS	109/.15		
		11	SS	145/.15		
226.4 / 9.3	End of Borehole Spoon Refusal					

ELEVATION SCALE: 235, 234, 233, 232, 231, 230, 229, 228, 227

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 (%): 10 20 30

UNIT WEIGHT γ kN/m³

Notes:

1. Water level at a depth of 3.5 m below ground surface (Elev. 232.2 m) upon completion of drilling.

2. Advanced borehole with NW Casing to 2.4 m depth. Switched to BW Casing below this depth. Difficult casing/corebarrel advance due to cobbles and boulders.

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

Golder Associates

RECORD OF BOREHOLE No KC09-7

 METRIC

PROJECT 07-1191-0007
W.P. 5413-04-00
LOCATION N 5504602.9 ;E 334017.2
ORIGINATED BY MR
DIST HWY 11
BOREHOLE TYPE Portable Equipment, NW Casing, BW Casing
COMPILED BY MM
DATUM Geodetic
DATE April 25, 2009
CHECKED BY SEMC

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
235.4	GROUND SURFACE					
0.0	Sand, some organics, trace clay (FILL) Loose Brown Moist	1	SS	8		
234.8 / 0.6	Silty clay, trace sand, trace gravel, trace organics, trace concrete fragments (FILL) Firm to stiff Brown Wet	2	SS	6		
		3	SS	10		
233.6 / 1.8	Silty clay, trace sand, trace gravel, trace organics (ALLUVIUM) Firm to stiff Brown Wet	4	SS	7		
		5	SS	13		
		6	SS	6	▽	
231.7 / 3.7	Sandy silt to silt and sand, some clay, trace gravel, containing cobbles and boulders (TILL) Very dense Grey Wet	7	SS	102/.08		
		8	SS	112/0.1		1 36 49 14
		9	SS	130/0.1		
		10	SS	146/.15		
226.0		11	SS	143/.15		2 38 41 19
9.4	End of Borehole Spoon Refusal					

ELEVATION SCALE: 235, 234, 233, 232, 231, 230, 229, 228, 227, 226

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 (%) 10 20 30

UNIT WEIGHT γ kN/m³

Notes:

1. Water level at a depth of 3.4 m below ground surface (Elev. 232.0 m) upon completion of drilling.

2. Advanced borehole with NW casing to 2.4 m depth. Switched to BW Casing below this depth.

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

MIS-MTO 001 KENDALL_CREEK_0711910007_METRIC.GPJ GAL-MISS.GDT 16/9/09

Golder Associates

RECORD OF BOREHOLE No KC09-8

 METRIC

PROJECT 07-1191-0007

W.P. 5413-04-00 LOCATION N 5504610.2 ;E 334024.5 ORIGINATED BY MR

DIST HWY 11 BOREHOLE TYPE Portable Equipment, NW Casing, BW Casing COMPILED BY MM

DATUM Geodetic DATE April 26, 2009 CHECKED BY SEMC

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
233.4	GROUND SURFACE											
0.0	PEAT, trace sand, containing wood fibres Firm to very stiff Dark brown Wet (Frozen)	1	SS	20	∇	233						
		2	SS	8								
		3	SS	11		232						
231.6 1.8	Sandy SILT, trace gravel, some organics, containing cobbles Dense Dark brown Wet	4	SS	39								
231.0 2.4	Sandy silt to silt and sand, some clay, some gravel, containing cobbles and boulders (TILL) Very dense Grey Wet	5	SS	136/0.15		231			21	28	46	5
		6	SS	142/0.15		230						
		7	SS	99/0.08								
		8	SS	141/0.15		229			0	33	52	15
						228						
		9	SS	131/0.15		227						
		10	SS	129/0.15		226						
225.6 7.8	End of Borehole Spoon Refusal											

Notes:

1. Water level at a depth of 0.4 m below ground surface (Elev. 233.0 m) upon completion of drilling.

2. Advanced borehole with NW Casing to 2.4 m depth. Switched to BW Casing below this depth.

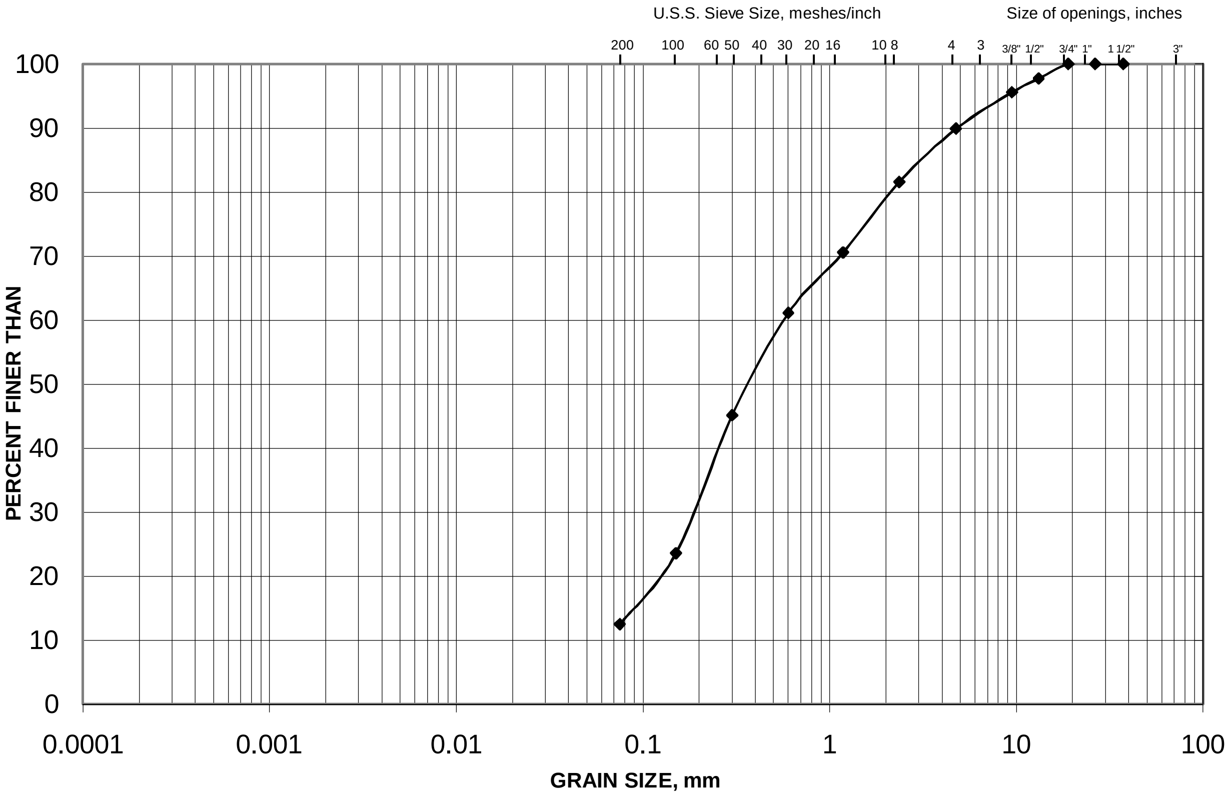
Legend: DYNAMIC CONE PENETRATION RESISTANCE PLOT (20, 40, 60, 80, 100); SHEAR STRENGTH kPa: ○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED (20, 40, 60, 80, 100); PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 10, 20, 30; REMARKS & GRAIN SIZE DISTRIBUTION (%)

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

APPENDIX B

LABORATORY TEST RESULTS

GRAIN SIZE DISTRIBUTION Sand (Fill) Kendall Creek Bridge	FIGURE B-1



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

LEGEND

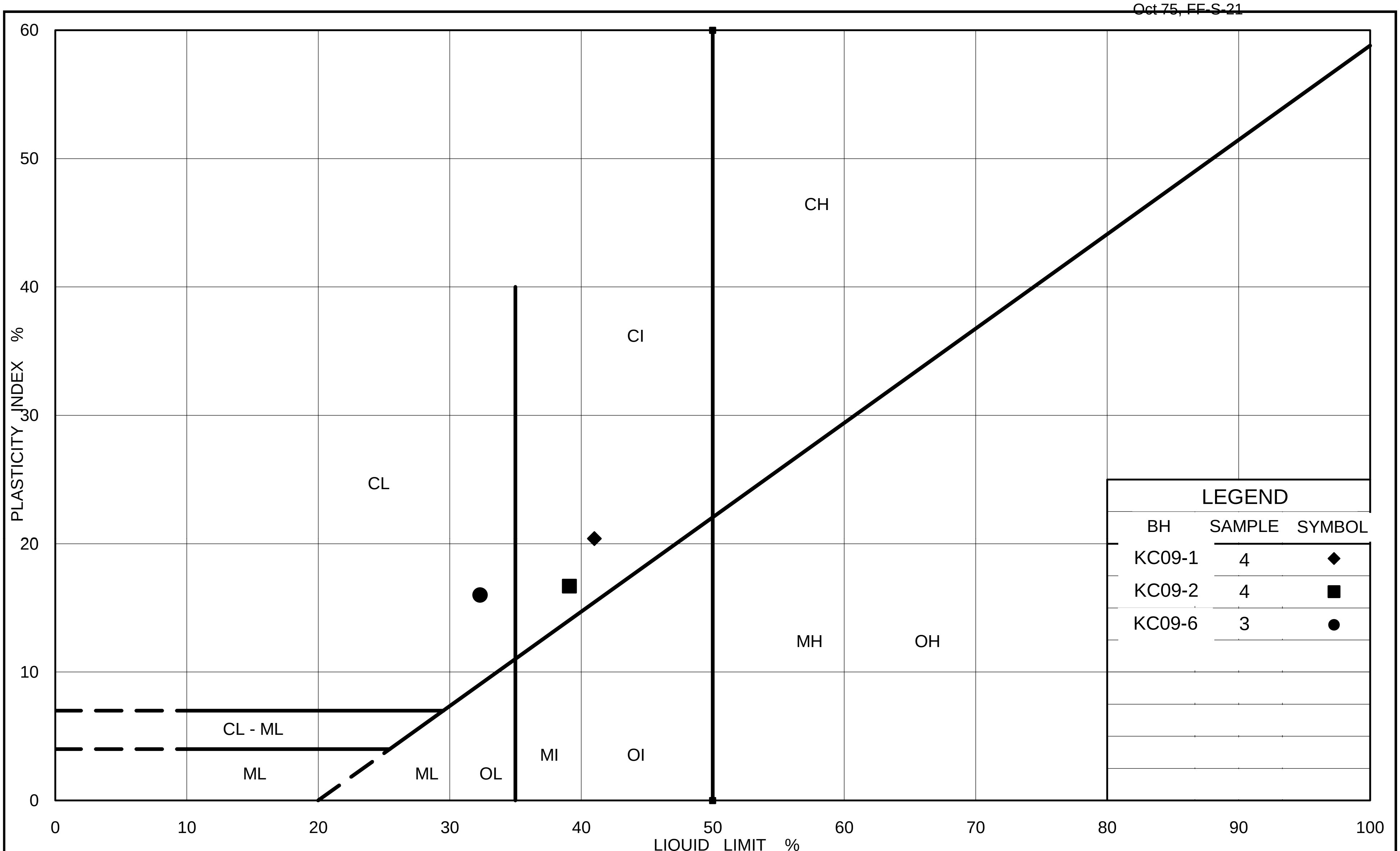
SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
—◆—	KC09-1	2	235.4

Project Number: 07-1191-0007-KC

Checked By: SEMC

Golder Associates

Date: September 2009



Ministry of Transportation

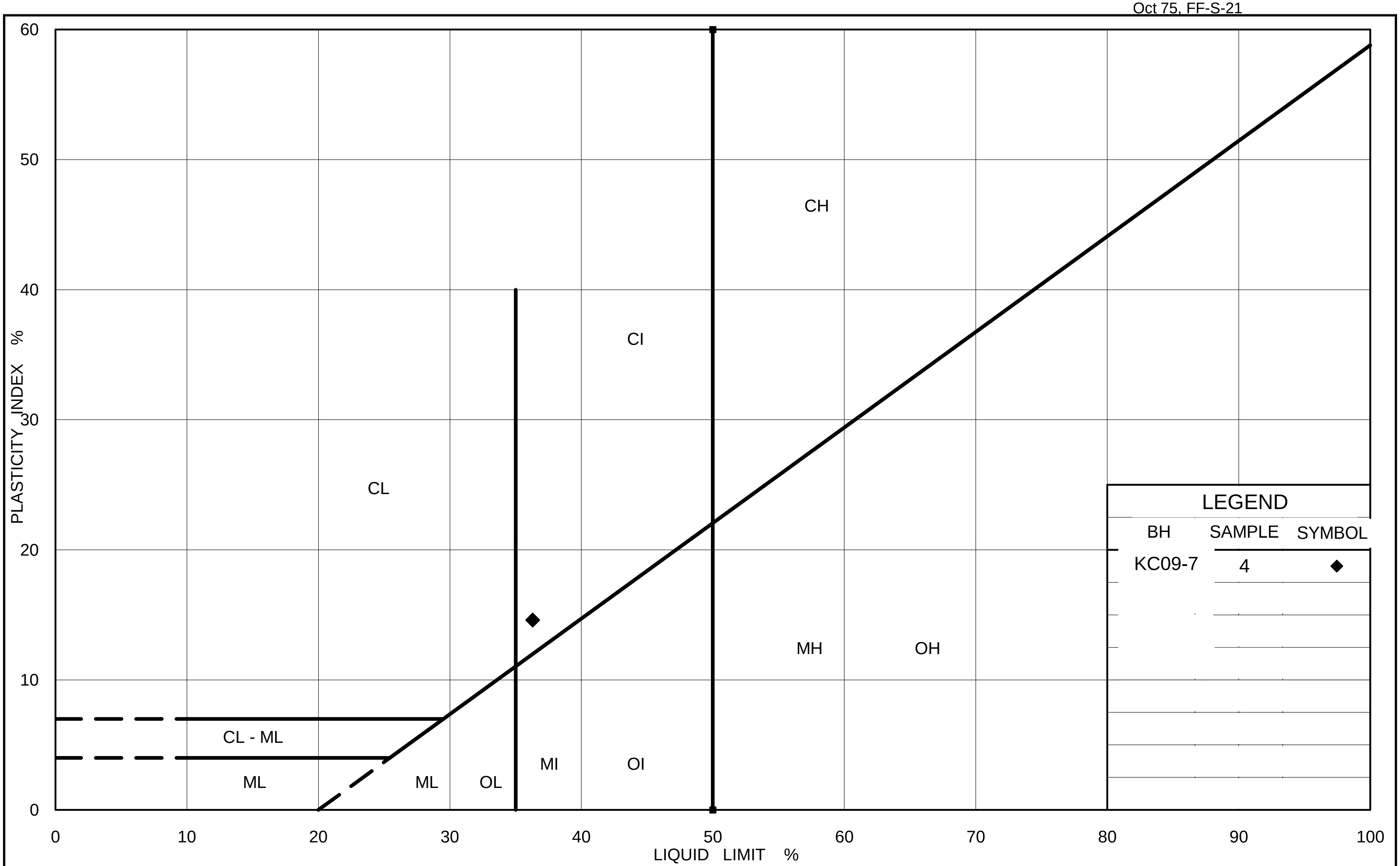
Ontario

PLASTICITY CHART
Silty Clay to Clayey Silt (Fill)

Figure B-2

Project No. 07-1191-0007-KC

Checked By: SEMC



Ministry of Transportation

Ontario

PLASTICITY CHART
Silty Clay (Alluvium)

Figure B-3

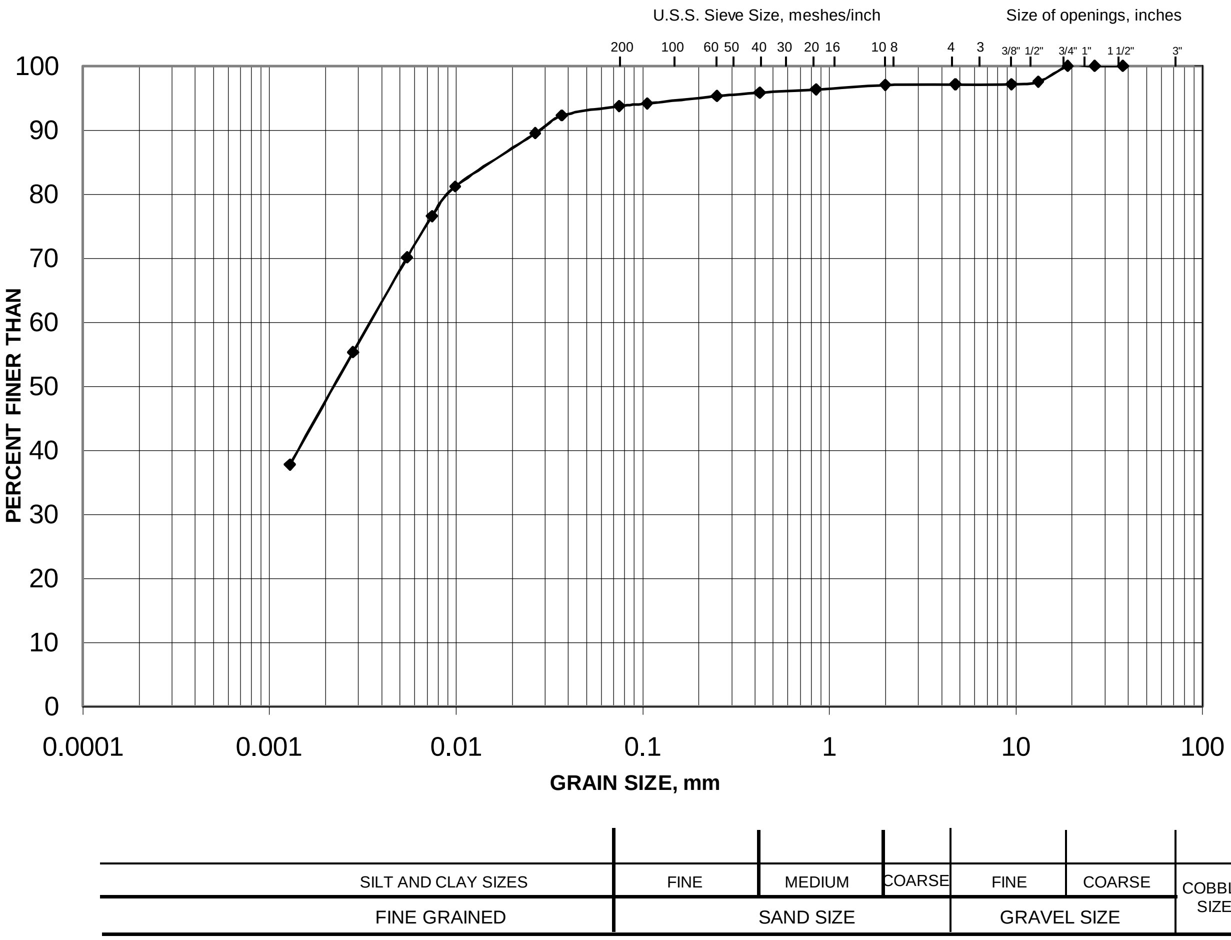
Project No. 07-1191-0007-KC

Checked By: SEMC

GRAIN SIZE DISTRIBUTION

Silty Clay (Alluvium)
Kendall Creek Bridge

FIGURE B-4



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
—◆—	KC09-1	6	232.3

Project Number: 07-1191-0007-KC

Checked By: SEMC

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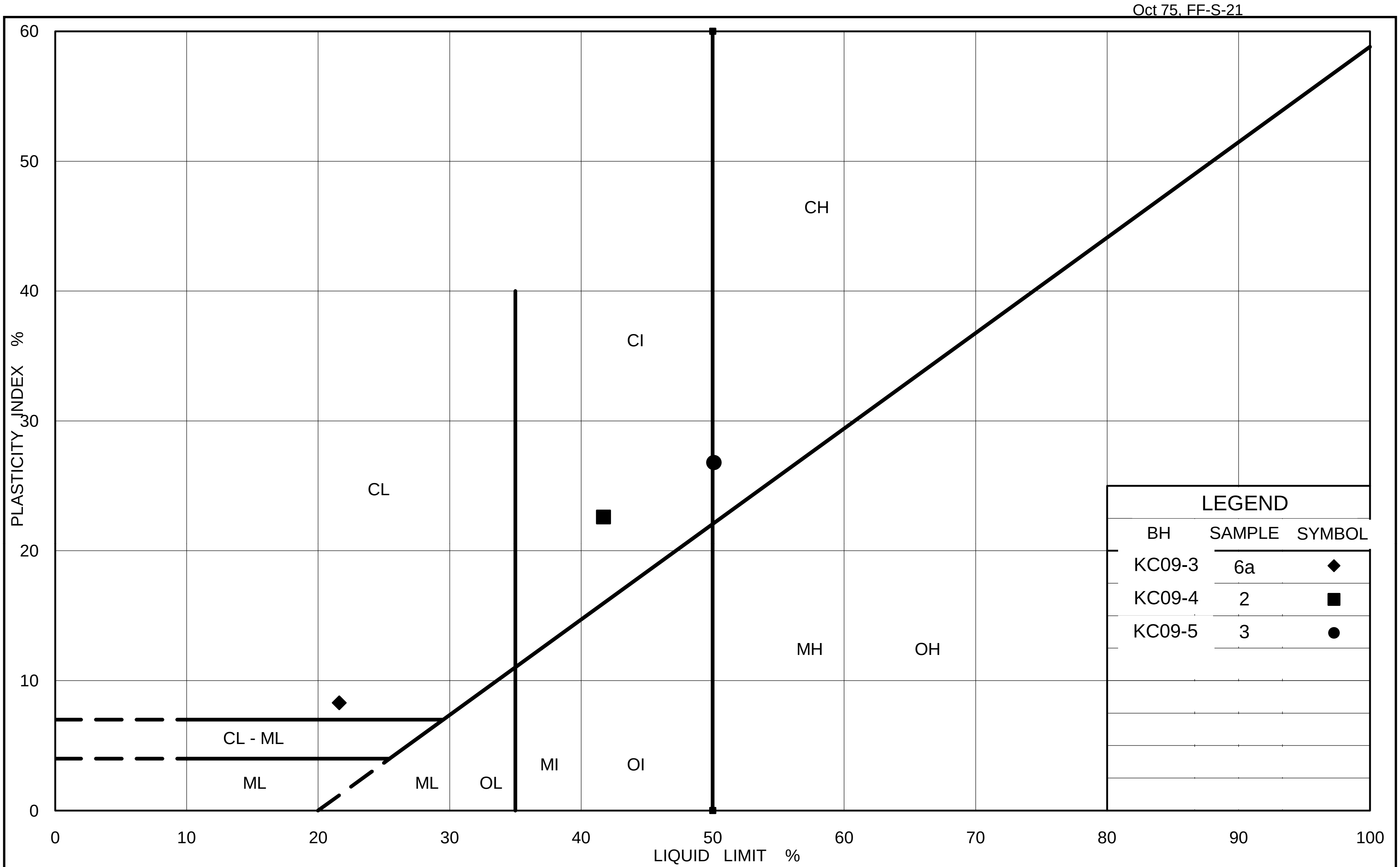
Date: September 2009



Ministry of Transportation

Ontario

PLASTICITY CHART
Silty Clay to Clayey Silt

Figure B-5

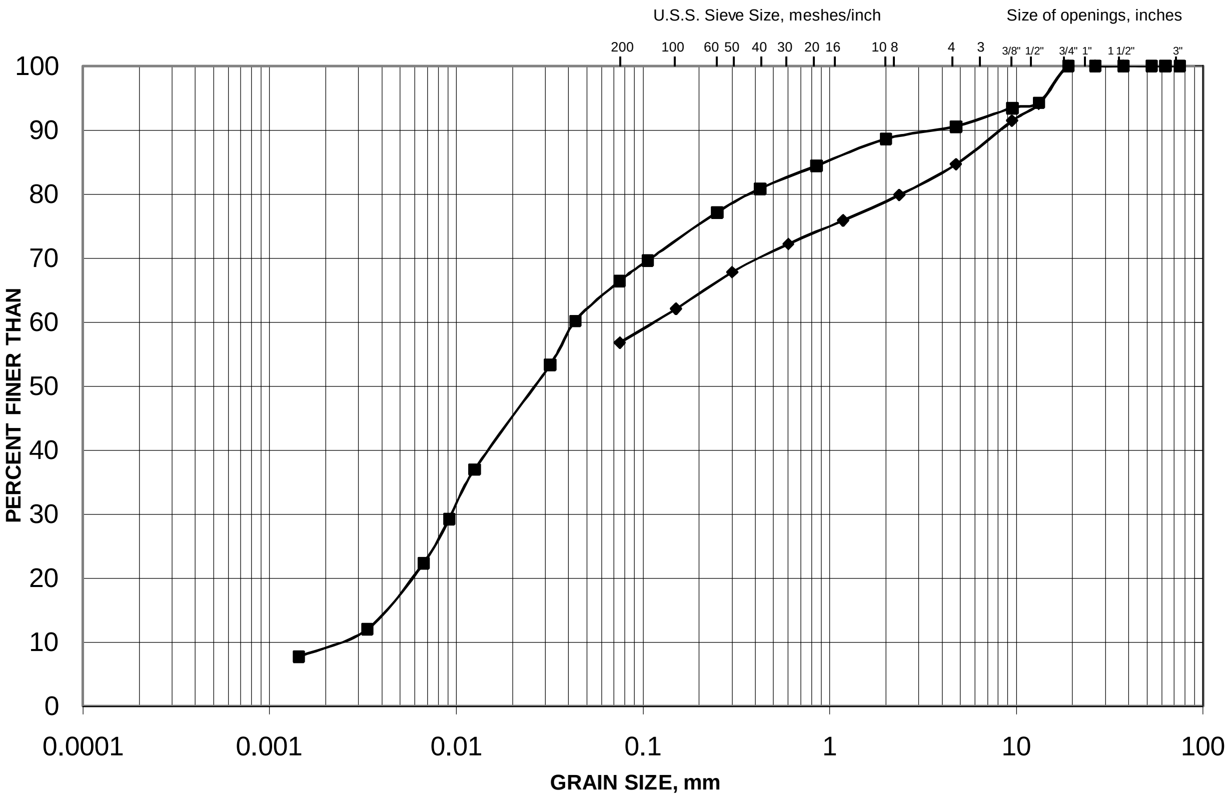
Project No. 07-1191-0007-KC

Checked By: SEMC

GRAIN SIZE DISTRIBUTION

Sandy Silt to Sand
Kendall Creek Bridge

FIGURE B-6



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
■	KC09-2	7	231.5
◆	KC09-5	4	231.3

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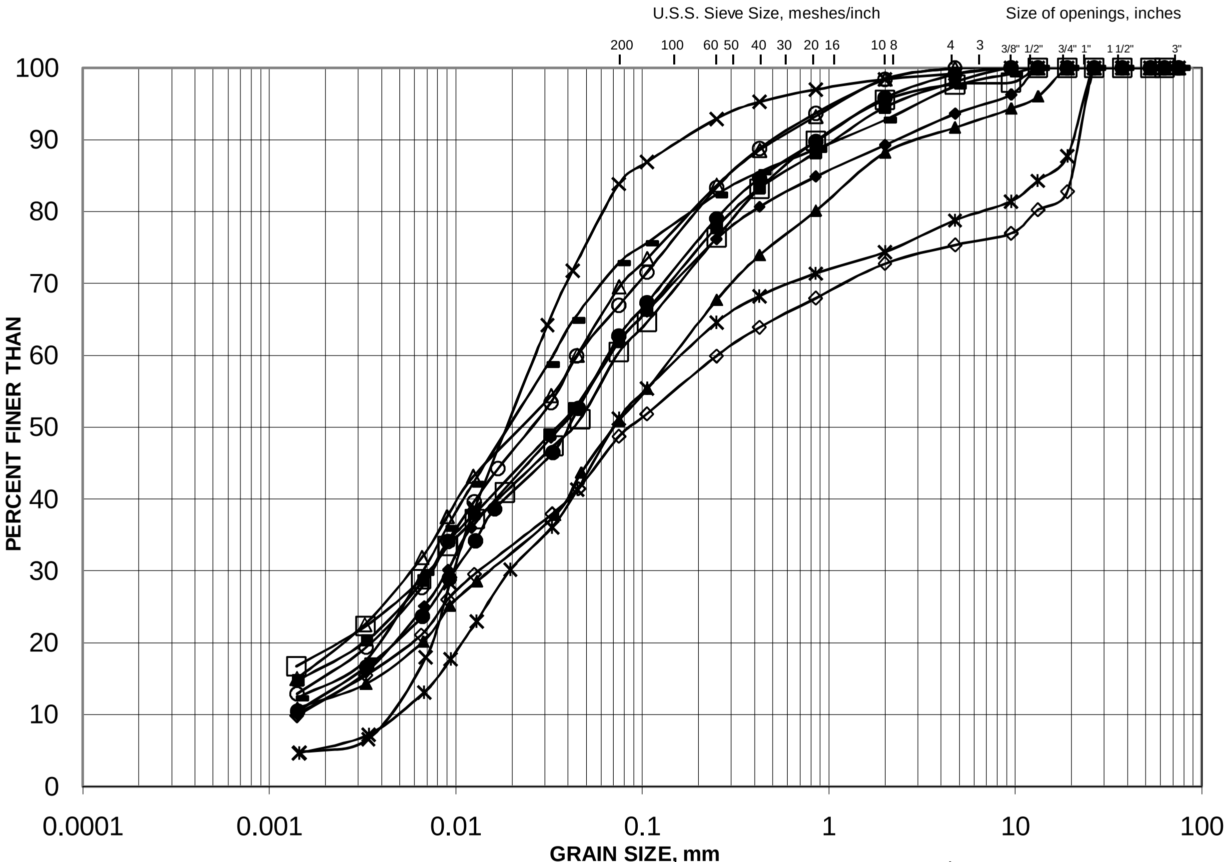
Date: September 2009

GRAIN SIZE DISTRIBUTION

Silt and Sand to Silt and Sand (Till)
Kendall Creek Bridge

FIGURE B-7



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
◆	KC09-1	9	228.6
■	KC09-2	11	233.9
▲	KC09-2	15	217.8
×	KC09-3	8	231.0
-	KC09-4	4	233.0
△	KC09-5	8	228.7
◇	KC09-6	9	229.5
●	KC09-7	8	230.8
□	KC09-7	11	226.1
*	KC09-8	5	231.3
○	KC09-8	8	228.8

Project Number: 07-1191-0007-KC

Checked By: SEMC

Golder Associates

Date: September 2009

APPENDIX C

NON-STANDARD SPECIAL PROVISIONS AND OPERATIONAL CONSTRAINTS

SUBGRADE PROTECTION – Item No.

Special Provision

Where shallow foundations are adopted, the sandy silt to silt and sand till subgrade will be susceptible to softening and degradation on exposure to water and construction traffic. If the concrete for the footings cannot be poured within four hours after inspection and approval of the subgrade, a working mat of lean concrete or mass concrete, with a minimum thickness of 100 mm, should be placed on the foundation subgrade.

Lean concrete shall have a compressive strength of at least 5 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

UNWATERING - Item No.

Non-Standard Special Provision

Spread footing construction below the groundwater and/or river water levels must be carried out in-the-dry. The excavation shall be kept stable during the work.

Operational Constraint – Obstructions – Existing Shoring

As part of the work for the installation of spread footings and/or placement of granular pads below spread footings, the Contactor shall be alerted that timbers and other old shoring elements may be present at the site.

Operational Constraint – Obstructions - Boulders

The Contractor shall be alerted that the fill and slope materials may contain cobbles and/or boulders. Cobbles and boulders will be encountered within the sandy silt to silt and sand till deposit.

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

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