REPORT

March 2013

FOUNDATION INVESTIGATION AND DESIGN REPORT

FLETCHER'S CREEK BRIDGES
HIGHWAY 401 WIDENING FROM HIGHWAY 403/410 INTERCHANGE TO THE CREDIT RIVER
CITY OF MISSISSAUGA, REGION OF PEEL
GWP 2150-01-00

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1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by AECOM Canada Ltd. (AECOM) on behalf of the Ministry of Transportation, Ontario (MTO) to provide detail foundation engineering services for the proposed widening of Highway 401 from the Highway 403/410 Interchange to the Credit River in the City of Mississauga, Region of Peel, Ontario.

This report addresses the foundation investigation carried out for the proposed replacement of the existing Fletcher's Creek culvert with two bridge structures (designated North Bridge and South Bridge on Drawing 1) to accommodate the proposed Highway 401 widening. The purpose of this investigation is to establish the subsurface conditions at the location of the proposed culvert replacement with the two bridge structures, by borehole drilling and laboratory testing on selected samples.

The Terms of Reference (TOR) and the scope of work for the foundation engineering services are outlined in MTO's Request for Proposal dated October 2010 and the associated MTO Clarification Packages No.1 to 3 issued between October and November 2010, which forms part of the Consultant's Agreement Number 2010-E-0003 for this project. The work has been carried out in accordance with Golder's Supplementary Specialty Plan for this project, dated April 2011. Foundation engineering services for the proposed Fletcher's Creek bridges, which were not addressed in MTO's original Request for Proposal, are outlined in Golder's Scope Change Request No. 2 dated January 18, 2012 and Revised Scope Change Request No. 2 dated May 11, 2012; accepted by MTO on April 11, 2012 and June 19, 2012, respectively.

2.0 SITE DESCRIPTION

The existing Fletcher's Creek culvert carries the eastbound (EBL) and westbound lanes (WBL) of Highway 401 over Fletcher's Creek in the City of Mississauga, within the Regional Municipality of Peel, Ontario. Fletcher's Creek is located approximately 1 km west of the intersection of Highway 401 and Mavis Road, and approximately 1 km east of the Credit River, in the City of Mississauga. Highway 401 in this area is a three lane freeway in both eastbound and westbound directions. The existing culvert structure consists of a double 6.0 m wide x 3.6 m high x 48 m long concrete box culvert with associated north and south concrete cantilever wing walls. The northeast and northwest wing walls are skewed at about a 30^{o} angle and are about 5.9 m long. The southeast and southwest wing walls are parallel to the long axis of the culvert and are about 5.1 m long. Referring to the *General Plan* design drawing (Drawing No. D-4003-1, dated Nov. 19, 1957, WP No. 160-57) for the existing culvert, Fletcher's Creek was previously designated Meadowvale Creek which flowed in a northwest to southeast direction. The 1957 drawing indicated the creek was realigned to cross perpendicular to the Highway 401 alignment in more of a north-south direction.

The Fletcher's Creek culvert is located within the flat floodplain of a naturally occurring valley where the ground surface varies from about Elevation 164 m to 163 m. Immediately west and east of the culvert site, the natural ground surface rises out of the Meadowvale valley to between approximately Elevation 167 m and 166 m. The creek bed is approximately 8 m to 15 m wide at the site, and is at approximately Elevation 161.5 m. During the MTO start-up meeting for this project, the Fletcher's Creek site was identified as having areas of "quicksand" (i.e. loose sands with high groundwater pressures). Based on several visits to the site prior to and during the investigation, groundwater seepage and loose soil conditions were observed in the low-lying areas adjacent to the Fletcher's Creek. The creek high water level of approximately Elevation 164.5 m and the normal water level of approximately Elevation 162.9 m are indicated on the 1957 drawing. Vegetation in the adjacent areas (east and west) of the creek is densely treed with small shrubs and trees present near the highway.

The existing Highway 401 grade in the general area of the Fletcher's Creek culvert structure varies between about Elevation 168.6 m and 168.5 m. The existing Highway 401 embankments are up to about 5 m high at the east and west approaches; with the embankment side slopes oriented approximately between 2.3H:1V and 2.8H:1V (i.e. about 2.5H:1V).

3.0 INVESTIGATION PROCEDURES

3.1 Previous Investigation by Others

A previous foundation investigation was conducted near the Fletcher's Creek culvert as part of the investigation for the existing Second Line underpass structure (previously referred to as Concessions II / III gravel road) by the Materials and Research Branch of the Department of Highways, Foundation Section of the MTO in June 1957. The results of the investigation conducted at Fletcher's Creek were contained in the Foundation Report for the Second Line underpass structure, titled "Foundation Report on Underpass Bridge at Highway 401 and Road allowance between Concessions II and III crossing, Toronto Twp, Lot 8, MTO WP 74-57, Geocres No. 30M12-31", dated August 14, 1957. The Second Line underpass structure is located approximately 150 m east of the Fletcher's Creek culvert. As part of the previous investigation, two boreholes were advanced in the area of Fletcher's Creek using continuous flight augers (with no in-situ testing); however, no borehole location plan was available in the Geocres information.

The subsurface conditions encountered in the two boreholes advanced near the creek during the previous investigation are described as consisting of clay till deposits underlain by sandy loam till, with the upper section of the layer observed to be soft and partially saturated by infiltration water. Considering the locations of these boreholes are not known and that the sampling method during the borehole drilling did not conform to the current MTO standards, these boreholes are not included in the preparation of this report.

3.2 Current Investigation

The field work for the current foundation investigation at the Fletcher's Creek site was carried out between May 1 and 15, 2012 and between September 4 and 6, 2012, during which time a total of fourteen (14) sampled boreholes were advanced at the proposed North and South Bridge sites as follows: four (4) boreholes near the outer corners of the proposed bridge structures; two (2) boreholes between the North and South Bridge structures near the abutment locations; two (2) boreholes at the mid width of the North Bridge abutments; four (4) approach boreholes for the widened Highway 401 EBL and WBL collector lanes and an additional two (2) boreholes to monitor the ground water level at the site. The boreholes (designated as Boreholes FC-1 to FC-13 and FC-13A) were advanced to depths up to about 17.8 m below ground surface and their locations are shown on Drawing 1. While drilling Borehole FC-13, the casing broke at a depth of about 10.3 m below ground surface, and Borehole FC-13A was advanced about 1.5 m west of Borehole FC-13 in order to continue sampling the overburden.

The field investigation was carried out using a Diedrich D50 track–mounted and Acker Renegade track-mounted drill rigs supplied and operated by Walker Drilling Ltd. of Utopia, Ontario, and a CME 55 track-mounted drill rig supplied and operated by Geo-Environmental Drilling Inc. of Halton Hills, Ontario. The boreholes were advanced through the overburden using 108 mm inside diameter (I.D.) hollow stem augers and/or HW and NW casing using Tricone and wash boring techniques to depths ranging from 5.2 m to 17.8 m. The overburden was cored using a HQ-size core barrel in one borehole (FC-1) due to auger and HW casing refusal. Soil samples were obtained in the boreholes at 0.75 m and 1.5 m intervals of depth using a 50 mm outer diameter split-spoon sampler driven by an automatic hammer in accordance with Standard Penetration Test (SPT) procedures (ASTM D1586)[1].

Dynamic Cone Penetration Testing (DCPT) was conducted in Borehole FC-1 to a depth up to 15.9 m below ground surface. This test consists of continuously driving into undisturbed ground a 50 mm diameter cone (60° vertex angle) attached to a drill rod, with a driving energy of 475 J per blow (63.5 kg automatic hammer dropping freely a vertical distance of 0.76 m). The number of blows for each 300 mm of penetration is recorded and this provides an indication of the relative changes in the soil density/consistency with depth.

The groundwater conditions in the open boreholes were typically observed during and immediately following the drilling operations. In some boreholes where casing and wash boring techniques were used (i.e. water was pumped into the borehole), the groundwater level was not measured. High hydrostatic pressures (artesian conditions) were observed in many of the boreholes advanced near the Fletcher's Creek valley for the proposed abutments and as a result, the boreholes located on the Highway 401 pavement were strategically advanced to avoid penetrating into the artesian aquifer zone(s). In order to permit monitoring of the stabilized groundwater level at the site, Boreholes FC 13/13A were advanced with NW casing into the underlying aquifer to measure the static hydrostatic head at this site. The water level readings are described on the borehole records presented in Appendix A. All boreholes were backfilled to ground surface using bentonite or with cement grout (i.e. where artesian conditions were encountered) upon completion, in accordance with Ontario Regulation 903 (as amended). The boreholes advanced through the Highway 401 asphalt were sealed at the surface with cold patch asphalt, approximately 0.2 m thick.

During the course of the fieldwork, the ground surface in the area of the Borehole FC-1 was observed to exhibit groundwater seepage, as discussed during the MTO start-up meeting. However, upon completion of the borehole abandonment, groundwater seepage at the ground surface was not observed at the actual borehole location but increased groundwater seepage was observed in localized areas adjacent to the borehole. The area surrounding FC-1 and the areas surrounding other boreholes drilled at this site were visually monitored by Golder personnel daily for about one (1) week and weekly for about one (1) month after borehole abandonment. The groundwater seepage in the area of Borehole FC-1 appeared to decrease over the monitoring period and no visual signs of erosion or significant ground loss were evident during our last visit to the site on September 6, 2012.

As mentioned above, high hydrostatic pressures (artesian conditions) were encountered within the cohesionless till when the drill casing was advanced to about Elevation 150.8 m in the first borehole (Borehole FC-1) drilled at this site. The borehole was advanced to refusal on competent soil below the artesian groundwater level (aquifer layer) by penetrating 3 m into the 100 blow soil, in order to fulfill the MTO standards for exploration for a bridge structure. However, difficulties (over the course of two days) were experienced when the borehole was abandoned in accordance with Ontario Regulation 903 (as amended)) as the cement grouts pumped into the borehole (using tremie techniques) were "blown out" twice by the high hydrostatic pressures within the aquifer layer. As a result, and upon further consultation with the Ministry of Transportation, as recorded in our email correspondences up to May 14, 2012 and as detailed in our Revised Scope Change Request No. 2, dated

[1] ASTM D1586 – Standard Test Method for Standard Penetration Tests and Split Barrel Sampling of Soils.

May 11, 2012; the remaining boreholes advanced at this site (especially the boreholes advanced within the abutment footprints and/or boreholes advanced through the Highway 401 pavement grade) were strategically advanced to prove the presence of very dense (100 blows) cohesionless till deposit, and then terminated within the competent soil, without penetrating the aquifer layer. This action was taken to prevent flooding of the highway caused by the high hydrostatic pressures present at the site which could disrupt traffic along Highway 401; and to minimize disruptions during future abutment construction resulting from groundwater flowing upward through the boreholes.

The field work was monitored on a full-time basis by a member of Golder's technical staff who located the boreholes in the field, directed the sampling and in situ testing operations, logged the boreholes and examined and cared for the soil samples.

The recovered soil samples were identified in the field, placed in labelled containers and transported to Golder's laboratory in Mississauga for visual identification. Selected samples were subjected to a laboratory testing program consisting of natural moisture content, Atterberg limits and grain size distribution analyses in accordance with MTO and/or ASTM Standards as applicable. The results of this testing program are shown on the Record of Borehole sheets in Appendix A and the laboratory figures contained in Appendix B.

The borehole locations were staked/marked in the field by Golder personnel relative to the existing culvert and on-site features shown on the digital terrain model for the site, provided by AECOM. The ground surface elevations at the borehole locations were surveyed by J.D. Barnes Ltd., a licensed surveying company retained by AECOM. The borehole locations (referenced to MTM NAD83 northing and easting coordinates), ground surface elevations (referenced to geodetic datum) and the borehole depths are shown on Drawing 1 and are summarized below.

Borehole No.	Location (MTM NAD 83)		Ground Surface Elevation (m)	Borehole Depth (m)
	Northing (m)	Easting (m)		
FC-1	4,830,867.9	287,311.4	163.7	15.9
FC-2	4,830,851.3	287,283.0	163.8	10.4
FC-3	4,830,871.4	287,325.3	165.9	5.2
FC-4	4,830,837.9	287,276.8	164.4	5.8
FC-5	4,830,834.4	287,327.5	168.6	13.9
FC-6	4,830,817.2	287,306.2	168.3	14.2
FC-7	4,830,814.6	287,338.1	168.5	14.2
FC-8	4,830,790.6	287,315.9	164.0	6.7
FC-9	4,830,805.8	287,360.1	164.3	5.2
FC-10	4,830,774.0	287,308.1	164.1	5.2
FC-11	4,830,795.8	287,352.4	164.4	12.7
FC-12	4,830,778.0	287,318.6	163.9	6.6
FC-13	4,830,881.8	287,336.5	167.1	10.8
FC-13A	4,830,881.8	287,335.0	167.1	17.8

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

This section of Highway 401 is located in the Peel Plain close to the border of the South Slope physiographic region, as delineated in *The Physiography of Southern Ontario* (Chapman and Putnam, 1984)[2].

The Peel Plain physiographic region covers the central portions of the Regional Municipalities of York, Peel and Halton. The general topography of this region consists of level to gently rolling terrain, sloping gradually southward toward Lake Ontario. A surficial till sheet, which generally follows the surface topography, is present throughout much of this area. The till, which is mapped in this area as the Halton Till, typically consists of clayey silt to silty clay, with occasional sand to silt zones. Shallow, localized deposits of loose sand and silt and/or soft clay can overlie this uppermost till sheet, and these represent relatively recent deposits, formed in small glacial melt water ponds scattered throughout the Peel Plain and concentrated near river valleys. The recent sand, silt and clay and uppermost till deposits in this area overlie and are interbedded with stratified deposits of sand, silt and clay. The overburden within the majority of the Peel Plain area is underlain by shale bedrock of the Georgian Bay Formation which contains limestone interlayers.

4.2 Subsurface Conditions

The detailed subsurface soil and groundwater conditions encountered in the boreholes advanced as part of the current investigation and the results of the laboratory testing are provided on the borehole records contained in Appendix A and the results of the geotechnical laboratory testing are presented on Figures B1 to B10 contained in Appendix B.

The stratigraphic boundaries shown on the borehole records are inferred from non continuous sampling, observations of drilling progress and the results of Standard Penetration Tests. These boundaries, therefore, represent transitions between soil types rather than exact planes of geological change and the stratigraphy shown on the profiles and cross sections on Drawings 1 to 3 are interpretations of the subsurface conditions. Variation in the stratigraphic boundaries between and beyond boreholes will exist and is to be expected, however, the factual data presented in the Record of Borehole sheets governs any interpretation of the site conditions.

In general, the subsurface conditions at the site consist of a surficial layer of asphalt or topsoil over a deposit of fill associated with the Highway 401 embankments. The fill is underlain by a deposit of sand and silt in places, which in turn is underlain by either a deposit of clayey silt to clayey silt with sand or clayey silt with sand to clayey silt till. The cohesive/cohesive till deposit is underlain by a cohesionless till deposit consisting predominantly of silt and sand, silty sand and gravel to sand and gravel. Gravelly sand to silty sand interlayers are present between the cohesive and cohesionless till deposits at the southeast quadrant of the site.

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

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

4.2.1 Asphalt

Boreholes FC-5 to FC-7 were drilled through the surficial pavement on Highway 401. An approximately 200 mm thick layer of asphalt was encountered at these locations.

4.2.2 Topsoil

A 200 mm thick surficial layer of topsoil was encountered in Boreholes FC-1, FC-2 and FC-4 advanced in the valley bordering the creek at the north side of the site.

4.2.3 Fill

Fill soils were encountered underlying the topsoil and asphalt in all the boreholes drilled at this site, with the exception of Boreholes FC-13 and FC-13A. The thickness of the fill is variable across the site. In Boreholes FC-5 to FC-7, which were drilled through the existing Highway 401 embankment near the median and on the eastbound right shoulder, respectively, the fill extends to depths of between about 5.4 m and 7.0 m below ground surface (Elevations 162.9 m and 161.1 m). At the other borehole locations, the fill extends to depths ranging from about 0.7 m to 3.0 m below ground surface (Elevations 165.0 m to 161.0 m).

The fill material is variable in composition. In general, a layer of cohesionless fill comprised of sand and gravel was encountered below the asphalt layer in Boreholes FC-5 to FC-7; and a layer of sandy silt fill containing trace to some clay, trace gravel, organics, rootlets and wood fragments was encountered in Boreholes FC-8 and FC-11. Underlying the sand and gravel fill (FC-5 to FC-7), sandy silt fill (FC-8 and FC-11), topsoil (FC-1, FC-2 and FC-4) and at the ground surface in the remaining boreholes (FC-3, FC-9, FC-10 and FC-12), a deposit of cohesive fill consisting of clayey silt to clayey silt with sand or trace to some sand and trace to some gravel was encountered. The upper portion of the cohesive fill typically contains rootlets, organic materials and/or wood fragments.

The Standard Penetration Test (SPT) “N”-values measured within the cohesionless portion of the fill range from 5 blows to 14 blows per 0.3 m of penetration, indicating a loose to compact relative density. SPT “N”-values measured within the cohesive fill generally range from 3 blows to 32 blows per 0.3 m of penetration, suggesting that the clayey silt to clayey silt with sand fill has a soft to hard consistency. One “N”-value measured 0 blows (weight of hammer) per 0.3 m of penetration in FC-10; however this sample was obtained near the ground surface.

Grain size distribution tests were carried out on three (3) samples of the clayey silt to clayey silt with sand fill and the results are provided on Figure B1 in Appendix B.

Atterberg limits tests were carried out on six (6) samples of the clayey silt to clayey silt with sand fill. The liquid limits range from about 24 per cent to 31 per cent, the plastic limits range from about 15 per cent to 18 per cent, and the plasticity indices range from about 8 per cent to 13 per cent. The results of the Atterberg limits tests are shown on a plasticity chart on Figure B2 in Appendix B, and indicate that the fill material consists of clayey silt of low plasticity.

The natural water content measured on one (1) sample of the sandy silt fill is 20 percent, and the natural water content measured on thirteen (13) samples of the clayey silt to clayey silt with sand fill material ranges from about 11 per cent to 31 per cent.

4.2.4 Sand and Silt

A deposit of brown to grey sand and silt containing some gravel, trace to some clay and clayey silt seams was encountered underlying the fill material in Boreholes FC-5 and FC-7, advanced near the proposed new east abutment. The thickness of this deposit is about 1.5 m and 2.3 m at the borehole locations, and the deposit extends to depths of about 7.3 m and 7.9 m below ground surface (Elevations 161.3 m and 160.6 m) in Boreholes FC-5 and FC-7, respectively.

The SPT "N"-values measured within this deposit range between 4 blows and 14 blows per 0.3 m of penetration, indicating that the sand and silt has a loose to compact relative density.

Grain size distribution tests were carried out on two (2) samples of the sand and silt deposit and the result is shown on Figure B3 in Appendix B. The natural water content measured on three (3) samples of this deposit ranges from about 14 per cent to 24 per cent.

4.2.5 Clayey Silt to Clayey Silt with Sand

A cohesive deposit of brown to grey clayey silt to clayey silt with sand was encountered below the fill and sand and silt deposit or below the ground surface in all the boreholes drilled at this site with the exception of Boreholes FC-10 and FC-12. The thickness of this deposit generally ranges from about 1.7 m to 3.8 m, except at Borehole FC-13/13A (located furthest away from the creek) which was measured to be 6.6 m thick. The base of the cohesive deposit extends to depths ranging from about 4.5 m to 10.4 m below ground surface (Elevation 160.7 m to 158.1 m). Borehole FC-3 was terminated within this deposit at about Elevation 160.7 m.

The cohesive deposit comprised of clayey silt, trace to some sand to clayey silt with sand containing trace to some gravel. Sandy silt interlayers were present at some locations and a silt layer (0.8 m thick) was encountered at a depth of 3.7 m below ground surface in FC-3. The upper 0.6 m and 0.7 m portion of the cohesive deposit in Boreholes FC-4 and FC-13 contains rootlets. The presence of cobbles is also inferred from difficulties advancing augers (auger grinding) in Borehole FC-4 at a depth of about 5.2 m (about Elevation 159.2 m) during the drilling operations; however the inferred cobbles may be derived from the cohesionless till underlying the clayey silt deposit.

The SPT "N"-values measured within the clayey silt to clayey silt with sand deposit generally range from 0 blows (weight of hammer) to 22 blows per 0.3 m of penetration. Typically, lower SPT "N"-values were measured at boreholes advanced within the valley on either side of the existing highway embankment, specifically at the southeast and northwest quadrants. In situ field vane tests carried out within this deposit in Boreholes FC-2 and FC-4 measured undrained shear strengths ranging from about 8 kPa to 95 kPa, with the sensitivity calculated to be between about 1 and 9. The field vane tests together with the SPT "N" value results suggest that the deposit generally has a very soft to stiff consistency. In situ field vane tests were also performed in this layer in Borehole FC-13 (located further away from the creek) and measured undrained shear strengths of 136 kPa and 144 kPa, suggesting the cohesive deposit is stiff to very stiff at this location.

Grain size distribution tests were carried out on eleven (11) samples of the clayey silt to clayey silt with sand deposit and the results are shown on Figure B4A and B4B in Appendix B.

Atterberg limits tests were carried out on sixteen (16) samples of this deposit. The liquid limits range from about 19 per cent to 31 per cent, the plastic limits range from about 13 per cent to 20 per cent and the plasticity indices range from about 5 per cent to 14 per cent. The results of the Atterberg limits tests are shown on a plasticity chart on Figure B5 in Appendix B, and indicate that this deposit consists of clayey silt of low plasticity.

The natural water content measured on nineteen (19) samples of this clayey silt to clayey silt with sand deposit ranges from about 10 per cent to 20 per cent.

In Borehole FC-3, an approximately 0.8 m thick layer of silt containing trace to some sand, trace to some clay and trace gravel was encountered within the clayey silt deposit. A SPT "N"-value of 4 blows per 0.3 m of penetration was recorded within the silt layer, indicating a loose relative density. A grain size distribution test carried out on a sample of this silt layer is shown on Figure B6 in Appendix B. The natural water content measured on one (1) sample of the silt is 26 per cent.

4.2.6 Clayey Silt to Clayey Silt with Sand Till

A cohesive till deposit of brown and grey clayey silt, some sand to clayey silt with sand containing trace to some gravel was encountered below the fill materials in Boreholes FC-10 and FC-12 advanced near the southwest quadrant of the bridge site. The measured thickness of the cohesive till deposit is about 4.5 m and 4.1 m, with the base of the deposit at about Elevation 158.9 m and 158.3 m in Boreholes FC-10 and FC-12, respectively. Borehole FC-10 did not fully penetrate the till and was terminated within this cohesive till deposit at a depth of about 5.2 m below ground surface (Elevation 158.9 m).

The SPT "N"-values recorded within the cohesive till deposit range from 6 blows to 37 blows per 0.3 m of penetration, suggesting that the clayey silt to clayey silt with sand till has a firm to hard consistency.

Grain size distribution tests were carried out on three (3) selected samples of the cohesive clayey silt to clayey silt with sand till deposit and the result is provided on Figure B7 in Appendix B.

Atterberg limits tests were carried out on four (4) samples of this cohesive till deposit. The liquid limits range from about 20 per cent to 25 per cent, the plastic limits range from about 13 per cent to 18 per cent and the plasticity indices range from about 6 per cent to 8 per cent. The results of the Atterberg limits tests are shown on a plasticity chart on Figure B8 in Appendix B, and indicate that this deposit consist of clayey silt of low plasticity.

The natural water content measured on six (6) samples of this clayey silt to clayey silt with sand till deposit ranges from about 11 per cent to 15 per cent.

4.2.7 Gravelly Sand and Silty Sand Interlayers

In Boreholes FC-9 and FC-11 advanced near the southeast quadrant of the bridge site, interlayers of gravelly sand (measured to be about 0.5 m thick) and silty sand (1.1 m thick) were encountered below the cohesive deposit at depths of about 4.7 m and 4.5 m below ground surface (Elevation 159.6 m and 159.9 m), respectively. The gravelly sand interlayer contains some silt and trace clay, and the silty sand interlayer contains trace gravel. Borehole FC-9 was terminated within the gravelly sand interlayer at about Elevation 159.1 m.

The SPT "N"-value recorded within the gravelly sand interlayer is 18 blows per 0.3 m of penetration, and the SPT "N"-value recorded within the silty sand interlayer is 27 blows per 0.3 m of penetration, indicating a compact relative density for the gravelly sand and silty sand interlayers.

A grain size distribution test was carried out on one (1) selected sample of the gravelly sand interlayer and the result is provided on Figure B9 in Appendix B.

The natural water content measured on (1) sample each of the gravelly sand and silty sand interlayers is about 13 per cent and 15 per cent, respectively.

4.2.8 Silty Sand to Sand and Gravel Till

A predominantly cohesionless till deposit was encountered underlying the clayey silt to clayey silt with sand deposit and/or the silty sand interlayers in Boreholes FC-1, FC-2, FC-4 to FC-8, FC-11, and FC13/13A, and below the clayey silt till deposit in Borehole FC-12. The top of the deposit was encountered at depths ranging from about 4.5 m to 10.4 m below ground surface (Elevation 160.5 m to 158.1 m). The boreholes were terminated within the cohesionless till deposit between about Elevation 158.6 m and 148.2 m, after penetrating between about 0.5 m and 11.2 m into the deposit.

The cohesionless till deposit varies in composition from silty sand containing trace to some gravel; to sand and silt containing trace to some gravel; to gravelly sand containing trace to some silt; to silty sand and gravel; to sandy silt and gravel, to sand and gravel containing trace to some silt, all containing trace to some clay. The lower portion below Elevation 157 m of the cohesionless till in Boreholes FC-1, FC-2, FC-6 and FC-13A contains inferred cobbles and shale fragments. In Borehole FC-1, refusal to advance augers or casing (possibly on cobbles or boulders) was encountered during the drilling operations at a depth of 12.9 m (Elevation 151 m). The borehole was cored between depths of 12.9 m and 15.5 m (Elevation 150.8 m and 148.2 m) and was terminated within the cohesionless till due to poor recovery of the cohesionless soil in the core barrel.

The SPT "N"-values measured within the cohesionless till generally range from 35 blows to 184 blows per 0.3 m of penetration, indicating a dense to very dense relative density. SPT "N"-values as high as 65 blows per 0.03 m of penetration to 100 blows per 0.15 m of penetration were recorded within the lower portion of the cohesionless till. A SPT "N"-value of 9 blows per 0.3 m of penetration was recorded within the till deposit in Borehole FC-13A at a depth of about 13.0 m below ground surface, it is inferred that this low value may have been a result of soil disturbance due to difficulties advancing augers/casing at this depth.

A Dynamic Cone Penetration Test (DCPT) was advanced from the bottom of the sampled Borehole FC-1 at a depth of about 15.5 m below ground surface; the DCPT was terminated on effective refusal (greater than 163 blows per 0.3 m of penetration) at a depth of about 15.9 m (Elevation 147.9 m).

Grain size distribution tests were carried out on fifteen (15) selected samples of the cohesionless till deposit and the results are shown on Figures B10A to B10C in Appendix B.

The natural water content measured on nineteen (19) selected samples of the cohesionless till deposit ranges from about 7 per cent to 13 per cent.

4.2.9 Groundwater Conditions

In general, the overburden samples taken in the boreholes were moist to wet. The groundwater levels in the boreholes were measured during and upon completion of drilling operations as shown on the Record of Borehole sheets in Appendix A. Boreholes FC-3, FC-9 and FC-10 (all drilled to a depth of about 5 m below ground surface) were observed to be dry upon completion of drilling. Groundwater levels were not recorded in Boreholes FC-6 and FC-7 given that wash boring techniques were used and the water levels were not considered to be representative upon completion of drilling.

Artesian groundwater conditions were observed during the drilling operations in Boreholes FC-1, FC-2, FC-8, FC-11 to FC13/13A (all advanced near the proposed abutment locations) with the groundwater levels measured above the ground surface. Upon consultation with MTO regarding the high hydrostatic pressures present at this site, it was agreed not to install piezometers in any of the boreholes advanced near the proposed abutment footprints or in boreholes advanced through the Highway 401 pavement grade in order to reduce the risk of ground loss or potential flooding in these areas. Therefore, Borehole FC-13/13A was advanced approximately 45 m east of the east edge of Fletcher's Creek on higher ground (outside the floodplain and low-lying valley) and away from the foundation element footprints, to measure the stabilized hydrostatic head at the site over a short period of time. Details of the groundwater levels measured in the open boreholes, either within the casing or hollow-stem augers, the depth of casing and augers below ground surface when groundwater levels were recorded and the corresponding casing and auger elevations are presented on the Record of Borehole sheets and are summarised below.

Borehole No.	Ground Surface Elevation (m)	Depth / Elevation of Casing or Augers (m)	Depth to Water Level (m)	Water Elevation (m)	Date (Time)	Comments
FC-1[2]	163.7	12.8 / 150.9 13.0 / 150.7	1.8 -1.0	161.9 164.7**	May 01, 2012 May 02, 2012	Inside Augers Inside Casing
FC-2[2]	163.8	10.4 / 153.4	0.0	163.8**	May 08, 2012	Inside Casing
FC-4[1]	164.4	5.8 / 158.6	2.0	162.4	May 09, 2012	Inside Augers
FC-5	168.6	13.9 / 154.7	1.8	166.8	May 15, 2012	Inside Casing
FC-8	164.0	6.7 / 157.3	-0.9	164.9**	May 10, 2012	Inside Casing
FC-11[2]	164.4	12.2 / 152.2	-1.5	165.9**	May 04, 2012	Inside Casing
FC-12[2]	163.9	6.0 / 157.9	-1.2	165.1**	May 08, 2012	Inside Casing
FC-13/13A	167.1	12.2 / 154.9 12.8 / 154.3 17.8 / 149.3 17.8 / 149.3 17.8 / 149.3 17.8 / 149.3	2.7 2.1 -0.6 -2.1 -3.5 -4.8	164.4 165.0 167.7** 169.2** 170.6** 171.9**	Sep. 05, 2012 (4:16 pm) Sep. 06, 2012 (7:00 am) Sep. 06, 2012 (12:30 pm) Sep. 06, 2012 (12:36 pm) Sep. 06, 2012 (12:41 pm) Sep. 06, 2012 (1:30 pm)	Inside Augers Inside Augers Inside Casing Inside Casing Inside Casing Inside Casing

** Artesian Conditions

Notes:
1. Water level not considered stabilized given that wash boring methods were used and water was introduced into the borehole.
2. Water level not considered stabilized as water was flowing out of the top of the casing.

As previously noted, during the field investigation the ground surface in the areas adjacent to the abandoned Borehole FC-1 (drilled at the northeast quadrant of the proposed bridge site) was monitored for the presence of groundwater seepage. Groundwater seepage was observed in the area prior to drilling activities; however, additional seepage areas were noticed shortly after the borehole was abandoned by sealing the borehole with cement grout. Although groundwater was not observed emanating from the borehole, localized areas adjacent to the borehole were observed to exhibit water seepage. Although the saturated surficial sand layers (previously referred to as “quick sand” at the MTO start-up meeting) were not encountered in the boreholes advanced near the creek, visual examination and probing of the areas adjacent to the creek using a steel rod confirmed the presence of these surficial saturated layers/zones.

Based on the groundwater levels recorded during this investigation and our observation of the presence of surficial saturated sand layers, high hydrostatic pressures are present at this site. The artesian hydrostatic head present in the cohesionless layers at this site is estimated to be at approximately Elevation 172± m. In areas where there is sufficient thickness of cohesive soil above the cohesionless soils (i.e. confined aquifer), perched water conditions will also be present and the estimated perched groundwater level is assumed to be equivalent to the Fletcher’s Creek water level. Based on the 1957 drawing, the creek high water level is at approximately Elevation 164.5 m and the normal water level is at Elevation 162.9 m.

The groundwater level observations at this site are short term and will be subject to seasonal fluctuations and precipitation events, therefore the groundwater level should be expected to be higher during the spring season or during any period of heavy precipitation.

5.0 CLOSURE

Mr. Suresh Bainey, a senior technician with Golder and Mr. Chris Sternik, E.I.T, supervised the drilling program. This Foundation Investigation Report was prepared by Ms. T. Veronica Ayetan, P.Eng., a geotechnical engineer, and reviewed by Mr. Kevin Bentley, P.Eng., a senior geotechnical engineer and Associate with Golder. Mr. Ty Garde, P.Eng., a Designated MTO Contact and Principal with Golder, conducted an independent review and quality control audit of this report.

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PART B

DRAFT FOUNDATION DESIGN REPORT
FLETCHER'S CREEK BRIDGES
HIGHWAY 401 WIDENING FROM HIGHWAY 403/410
INTERCHANGE TO THE CREDIT RIVER
CITY OF MISSISSAUGA, REGION OF PEEL
GWP 2150-01-00

6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

6.1 General

This section of the report provides foundation engineering recommendations for the detail design of the proposed replacement of the existing Fletcher's Creek culvert as part of the Highway 401 widening from the Highway 403/410 interchange to the Credit River, in the City of Mississauga. The recommendations are based on interpretation of the factual data obtained from the boreholes advanced during the current subsurface investigation at this site. The interpretation and recommendations contained in this report are intended to provide the designers with sufficient information to assess the feasible foundation alternatives and to carry out the detail design of the foundations for the proposed new structure(s). It is noted that the Terms of Reference for this project were revised and structural design information to the 30% level (as opposed to detail design level) was provided to us. The assumptions and recommendations provided herein should be revised as necessary during the actual detail design. Where comments are made on construction, they are provided in order to highlight those aspects that could affect the design of the project, and for which special provisions or operational constraints may be required in the Contract Documents. Those requiring information on aspects of construction should make their own interpretation of the factual information provided as such interpretation may affect equipment selection, proposed construction methods, scheduling and the like.

The existing Fletcher's Creek culvert consists of a concrete double culvert; each box is 6.0 m wide x 3.6 m high x 48 m long with associated concrete cantilever wing walls at each end. Based on the original design drawing titled "*General Plan"* (Drawing No. D-9003-1, WP No. 160-57, prepared by the Departments of Highways Ontario, Bridge office, dated November 19th, 1957), the existing box culvert is closed and the base slab (about 13 m wide) is supported on native subsoils at about Elevation 161.8 m. The northeast and northwest wing walls are skewed at about a 30^{o} angle and are about 5.9 m long. The width of the north retaining wall foundations ranges from about 1.2 m to 1.8 m and the walls are founded at about Elevation 160.5 m. The southeast and southwest wing walls are attached parallel to the long axis of the culvert and are about 5.1 m long. The width of the south retaining wall foundations range from about 1.2 m to 1.8 m and the walls are founded at about Elevation 160.5 m. The approach embankments over the Fletcher's Creek culvert are up to about 5 m high and the side-slopes are angled between 2.3H:1V and 2.8H:1V (approximately 2.5H:1V). The existing Highway 401 in this area is a three lane freeway in both the eastbound and westbound directions, with the Highway 401 pavement grade varying from approximately Elevation 168.7 m to 168.5 m.

It is understood that Fletcher's Creek was previously designated as Meadowvale Creek which flowed in a northwest to southeast direction. Fletcher's Creek was re-aligned in this area to flow from north to south to accommodate the construction of Highway 401 and the existing culvert. The creek bed is approximately 8 m to 15 m wide at the structure site and is at approximately Elevation 161.5 m at the lowest point on the south limit of the structure site. The creek high water level of approximately Elevation 164.5 m and the normal water level of approximately Elevation 162.9 m were indicated on the 1957 design drawing.

Based on the General Arrangement (GA) drawing provided by AECOM on September 25, 2012 titled "Highway 401, Fletcher's Creek Culvert Replacement" (Drawing No. R2-1, dated October 2012), the existing Fletcher's Creek culvert will be replaced by two (north and south) bridge structures in order to accommodate the widening of the existing Highway 401, as follows:

- The North Bridge structure consists of a single-span bridge that is approximately 24 m long and 69 m wide, proposed to carry the Highway 401 eastbound and westbound core (express) lanes (EBL and WBL) and the Highway 401 westbound collector lanes over Fletcher's Creek. The proposed Highway 401 eastbound core will consist of three lanes with an ultimate configuration of five lanes, the Highway 401 westbound core will consist of three lanes and the Highway 401 westbound collector will consist of four lanes. The surface of the proposed North Bridge deck varies between about Elevation 168.7 m and 168.5 m near the proposed east and west abutments, respectively, with the proposed Highway 401 grade declining from east to west across the structure. Based on the GA drawing, the existing Fletcher's Creek culvert (including the wing walls) and the existing approach embankments leading up to the existing culvert (i.e. about 10 m east and 8 m west) will be removed/sub-excavated as part of the construction for the new North Bridge structure. The existing Highway 401 pavement grade at this location ranges from about Elevation 168.7 m to 168.5 m and the existing ground surface at the proposed widened area at the north limit (in the vicinity of the proposed structure) ranges from about Elevation 164.4 m to 165.9 m. There is essentially no grade change between the existing and proposed Highway 401 road profiles at the north structure; however, the proposed new approach embankments required for the embankment widening at the north limit of the structure are approximately 5 m high, with side-slopes at approximately 2H:1V.

- The South Bridge structure consists of a single span bridge that is approximately 26 m long and 19 m wide, proposed to carry the Highway 401 eastbound collector lanes over Fletcher's Creek. The proposed Highway 401 eastbound collector will consist of three lanes. The surface of the proposed Fletcher's Creek South Bridge deck varies between about Elevation 168.4 m and 168.2 m near the proposed east and west abutments, respectively, with the proposed Highway 401 grade declining from east to west across the structure. The existing ground surface in the vicinity of the proposed structure varies between about Elevation 164.4 m and 163.9 m, therefore the new approach embankments will be up to about 4.1 m high at the east approach and up to about 4.3 m high at the west approach.

Wing walls (about 8 m to 9 m long) are proposed at each quadrant of the proposed bridges and retaining walls may be required to support the front slope between the North and South Bridge structures. Based on the GA drawing, both bridges are shown to be closed-end structures with integral abutments.

6.2 New Abutment Foundations

Within the vicinity of the proposed new foundation elements, the subsurface soil conditions encountered during the current investigation generally consist of fill soils (associated with the existing Highway 401 approach embankments), underlain by a deposit of loose to compact sand and silt in places, which in turn is underlain by either a deposit of very soft to very stiff clayey silt to clayey silt with sand or firm to hard clayey silt with sand to clayey silt till over a dense to very dense cohesionless till deposit. The clayey silt deposit and cohesionless till deposit at the east abutment of the proposed South Bridge are separated by a compact gravelly sand to silty sand interlayer.

Shallow and deep foundation options have been considered for support of the abutments for the new Fletcher's Creek North and South Bridge structures. A summary of the advantages and disadvantages associated with each option is provided below, and a comparison of the alternative foundation options based on advantages, disadvantages, risks/consequences and relative costs is provided in Table 1 following the text of this report.

- **Strip or spread footings founded within the dense to very dense silty sand to sand and gravel till deposit:** Spread footings could be considered to support the new abutments for the new North and South bridges given the competency of this native soil; however, the need for deep sub-excavations and temporary shoring (for staging requirements) to allow Highway 401 to remain in operation throughout construction make this option less desirable from a foundation construction perspective. This option would allow for the use of semi-integral abutments. The subsurface data obtained at the site indicate the presence of a generally very soft to firm clayey silt to clayey silt with sand deposit containing variable amount of organics (which is not considered suitable to support the new abutments) overlying the competent dense to very dense silty sand to sand and gravel till. Spread footings founded on the dense to very dense cohesionless till would require excavation (through the existing highway embankment fill, loose sand and silt layers, and the very soft to soft cohesive soil) to depths up to approximately 11 m below the exiting Highway 401 grade and up to approximately 6 m below the existing ground surface (adjacent to the highway embankment) to reach competent soil. Temporary protection systems would be required to facilitate excavation through the existing Highway 401 embankments and the native soils; extensive dewatering systems will be required in the loose sand and silt, gravelly sand to silty sand and potentially in the founding cohesionless till deposits. In addition, given the potential for significant traffic disruption and environmental impact adjacent to the creek; this option is not considered to be a practical alternative at the abutments for the new bridges.

- **Steel H-piles driven to found within the dense to very dense cohesionless till deposit:** Steel 310 x 110 H-piles driven to found within the dense to very dense cohesionless till are feasible for the support of the proposed abutments for the new Fletcher's Creek Bridges, and would allow for integral abutment construction. High hydrostatic pressures (artesian conditions) with hydrostatic head measured at about 5 m above the ground surface (Elevation 172 m) were encountered within the cohesionless till below about Elevation 158. The piles should be designed to penetrate a minimum socket length into the dense to very dense cohesionless till deposit, to reduce the potential for fines to migrate between the piles and subsoils due to the artesian condition. Specialized construction techniques should be considered to reduce environmental impact and reduce the potential for migration of fines (i.e. use of a sand blanket). Furthermore, the varying SPT "N"-values and presence of cobbles/boulders within the cohesionless till will result in the potential for variable pile lengths, which will need to be accommodated in the contract documents.

- **Steel tube (pipe) piles founded within the dense to very dense cohesionless till deposit:** Steel tube (pipe) piles could also be considered as a deep foundation option for support of the abutments for the Fletcher's Creek Bridges, however, MTO does not allow the use of pipe piles for integral abutment construction. Due to the presence of artesian conditions (discussed above), the pile tip should be terminated as high as possible in the cohesionless till deposit. Pipe piles will offer increased tip resistance at shallower depths compared to H-piles. Pipe piles are considered to have a higher risk than H-piles for "hanging up" or being deflected away from their vertical or battered orientation due to the presence of cobbles and/or boulders within the glacially-derived soils at this site. The presence of these obstructions in combination with the variable SPT "N"-values at depth will result in the potential for variable pile lengths.

- **Caissons founded within the dense to very dense cohesionless till deposit:** Consideration could be given to the use of caissons socketted into the very dense cohesionless till for support of the new abutments for the Fletcher's Creek Bridges. However, temporary or permanent liners would be required during caisson installation to control the ground and groundwater given the high hydrostatic head present at the site; and reduce the potential for running/flowing soil when excavating through the water-bearing sand and silt, gravelly sand to silty sand and cohesionless till deposits, which would result in the caisson foundations being less cost-effective as well as a higher risk of difficulties occurring during construction than the installation of driven steel piles.

At the new abutments, steel H-piles founded on the very dense cohesionless till is considered to be the preferred option from a geotechnical perspective. If conventional abutment design is preferred, caissons founded on the very dense cohesionless till could also be considered.

Recommendations for the various foundation options discussed above for the new abutments at the Fletcher's Creek North and South Bridges are provided in the following sections.

6.2.1 Spread Footings

6.2.1.1 Founding Elevations

Strip or spread footings founded on the dense to very dense cohesionless till (comprised of silty sand, sand and silt, gravelly sand, silty sand and gravel, sandy silt and gravel, and sand and gravel) is considered possible for support of the new abutment foundations and associated retaining walls for the North and South Bridges. The proposed finished grade of Highway 401 in the area of the east and west abutments is between about Elevation 168.7 m and Elevation 168.5 m for the north structure, and between about Elevation 168.4 m and 168.2 m for the south structure, as shown on the GA drawing provided by AECOM.

All footings should be founded at a minimum depth of 1.2 m below the adjacent final grade to provide adequate protection against frost penetration, in accordance with Ontario Provincial Standard Drawing (OPSD) 3090.101 (*Foundation Frost Penetration Depths for Southern Ontario).*

In general, across the abutment foundation footprints, the subsoils consist of fill over very soft to stiff (but typically very soft to firm) clayey silt to clayey silt with sand soil containing organics overlying the dense to very dense till present between about Elevations 159.2 m and 158.1 m. A localized layer of loose to compact sand and silt was also encountered between the fill and clayey silt at the east abutment location. However, near the southwest quadrant of the proposed west abutment for the South Bridge, a localized deposit of very stiff to hard clayey silt till was encountered underlying the existing fill. The northwest quadrant of the proposed west abutment consists of the firm to stiff clayey silt soil. Given that the stiff to hard clayey silt till was only encountered in a localized area, the overburden soils above the cohesionless till deposit are not considered capable of supporting the foundations for the abutments. The existing fill, loose sand and silt and the clayey silt to clayey silt with sand/clayey silt till soils should be sub-excavated (between about 5.6 m and 11.0 m thick) and shallow/spread footings founded on the cohesionless till deposit.

Alternatively, the subexcavated material depth (between about 5.6 m and 11.0 m deep) could be replaced with properly placed and compacted Granular 'A' or Granular 'B Type II (SP 110S13 Aggregates) to found spread footings at a higher elevation (a minimum of 1.2 m depth below the lowest surrounding grade to provide adequate protection against frost penetration) at the east and west abutments of the North and South Bridges. If replacement with engineered fill is being considered, the area to be subexcavated should be defined by a line extending from the top of the pad outward and downward at 1H:1V. The top of the granular engineered fill should extend at least 1 m beyond the plan limits of the footing, and constructed in accordance with OPSS 501 (Compacting) and SP 105S21.

The following summarizes the recommended founding elevations for strip or spread footings founded on the relatively undisturbed native dense to very dense cohesionless till or the compacted Granular 'A' or Granular 'B' Type II, for the support of the east and west abutments of the Fletcher's Creek North and South Bridges. As discussed below, significant dewatering effort would be required to allow sub-excavation, placement of engineered fill, and to avoid disturbance to the founding soils.

<table>
<tr><th></th><th colspan="3">North Bridge</th><th colspan="3">South Bridge</th></tr>
<tr><th>Foundation Element</th><th>Reference Borehole No.</th><th>Founding Stratum</th><th>Maximum Founding Elevation</th><th>Reference Borehole No.</th><th>Founding Stratum</th><th>Maximum Founding Elevation</th></tr>
<tr><td rowspan="2">East Abutment</td><td rowspan="2">FC-1, FC-5 and FC-7</td><td>Very dense sand and silt to silty sand and gravel till</td><td>157.7 m</td><td rowspan="2">FC-7 and FC-11</td><td>Very dense silty sand and gravel till</td><td>158.8 m</td></tr>
<tr><td>Compacted Granular ‘A’ or Granular ‘B’ Type II</td><td>162.5 m</td><td>Compacted Granular ‘A’ or Granular ‘B’ Type II</td><td>162.5 m</td></tr>
<tr><td rowspan="2">West Abutment</td><td rowspan="2">FC-2, FC-6 and FC-8</td><td>Dense to very dense silty sand to sand and gravel till</td><td>157.7 m</td><td rowspan="2">FC-8 and FC-12</td><td>Very dense silty sand and gravel to sand and silt till</td><td>158.3 m</td></tr>
<tr><td>Compacted Granular ‘A’ or Granular ‘B’ Type II</td><td>161.5 m</td><td>Compacted Granular ‘A’ or Granular ‘B’ Type II</td><td>161.5 m</td></tr>
</table>

Based on the depth of excavation required to found the footings on the competent cohesionless till deposit, temporary roadway protection systems (up to 11 m deep) would be required at the proposed east and west abutments of the North and South Bridges. Given the high water pressures (measured artesian hydrostatic head up to Elevation 172 m) encountered in the cohesionless till deposit and the potential running/flowing “quick” sand present near the existing ground surface at this site, extensive temporary dewatering would be required in advance of the excavation and during the construction of shallow foundations.

In addition, it is noted that the soils at the site would potentially be susceptible to erosion and scour at the abutments; therefore adequate protection against scour under the design hydraulic conditions should be included in the design. Construction considerations addressing the geotechnical aspects of the temporary protection systems, dewatering requirements and the scour protection are discussed further in Section 6.7.

6.2.1.2 Geotechnical Resistance/Reaction

Strip or spread footings (for the new North and South Bridge abutments) placed on the properly prepared, undisturbed native dense to very dense cohesionless till subgrade and/or compacted granular backfill (i.e. Granular ‘A’ or Granular ‘B’ Type II) at the founding elevations provided in Section 6.2.1.1, should be designed based on the following factored geotechnical resistance at Ultimate Limit State (ULS) and geotechnical reaction at Serviceability Limit States (SLS) for 25 mm of settlement:

		North Bridge		South Bridge	
Foundation Element	**Founding Stratum**	**Factored Geotechnical Resistance at ULS**	**Geotechnical Reaction at SLS** (for 25 mm of Settlement)	**Factored Geotechnical Resistance at ULS**	**Geotechnical Reaction at SLS** (for 25 mm of Settlement)
East and West Abutments	Dense to very dense cohesionless till	900 kPa	600 kPa	900 kPa	600 kPa
	Compacted Granular 'A' or Granular 'B' Type II	750 kPa	350 kPa	750 kPa	350 kPa

The ULS resistance and SLS values are dependent on the footing size (assumed to be 4 m wide), founding depth, and configuration and applied loads; the geotechnical resistances should, therefore, be reviewed if the selected footing width or founding elevation differs from those given above.

The geotechnical resistances provided above are given for loads applied perpendicular to the surface of the footing. 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 6.7.4 and C6.7.4 in the *Canadian Highway Bridge Design Code (CHBDC) and* its *Commentary,* using the curves for cohesive soils and non-cohesive soil, as applicable.

The base of each footing excavation should be cleaned of softened/loosened material and should be dry prior to placement of concrete. It is recommended that the founding level for the footings be inspected by a Quality Verification Engineer (QVE) following excavation, in accordance with OPSS 902 *(Excavating and Backfilling Structures)* to verify that all existing fill, clayey silt, and other unsuitable soils have been removed and to confirm the condition of the founding soils which will be susceptible to disturbance. If the concrete for the footings cannot be poured immediately after excavation and inspection, it is recommended that a concrete working slab (100 mm thickness of 20 MPa compressive strength concrete) be immediately placed on the subgrade to protect the integrity of the bearing stratum. This requirement can either be added as a note on the Contract Drawings or included as a Non-Standard Special Provision (NSSP) in the Contract Documents. A sample NSSP is included for this item in Appendix C.

6.2.1.3 Resistance to Lateral Loads

Resistance to lateral forces / sliding between the concrete footings and the subgrade should be calculated in accordance with Section 6.7.5 of the *CHBDC*. For cast-in-place concrete footings constructed on the dense to very dense cohesionless till or on the Granular 'A' or Granular 'B' Type II material, the coefficient of friction, tan ϕ', can be taken as 0.5. This value is unfactored.

6.2.2 Steel H-Pile / Steel Tube Pile Foundations

Steel H-Piles or steel tube (pipe) piles driven to found within the dense to very dense cohesionless till (comprised of silty sand, sand and silt, gravelly sand, silty sand and gravel, sandy silt and gravel, and sand and gravel) may be used to support the proposed east and west abutments of the Fletcher's Creek North and South Bridges, especially if integral abutments are being considered. For the installation of piles, consideration must be given to the presence of cobbles and boulders within the cohesionless till deposits, as encountered in the boreholes (below about Elevation 157 m) advanced at the abutments. Auger grinding observed at variable depths during the borehole investigation suggests obstructions may be encountered above the proposed founding tip elevations. In this regard, steel H-piles are preferred over steel tube piles given that H-piles are more commonly used for integral abutment design and that steel tubes are considered to pose a higher risk of “hanging up” or being deflected from their vertical or battered orientation during installation, due to their larger end area. The piles should be reinforced at the tip for protection during driving to reduce the potential for damage to the piles in the event that cobbles/boulders and/or very dense layers are encountered within the till deposits. The steel H-piles should be reinforced with flange plates as per OPSD 3000.100 (Foundation Piles Steel H-Pile Driving Shoe) or driving shoes such as Titus Standard “H” Bearing Pile Point design for protection during driving as per OPSS 903 (Deep Foundations). Similarly, if steel tube piles are being considered, driving shoes should be in accordance with OPSD 3001.100 Type II (Steel Tube Pile Driving Shoe). The requirement for driving shoes should be included in the Contract Drawings.

Referring to the GA drawing, the base of the pile caps would be at Elevation 161.5 m and 162.5 m at the west and east abutments, respectively, for both bridges. The soils within the upper portion of the piles (directly below the pile cap) consist of very soft to very stiff (predominantly soft to firm) clayey silt soils.

If corrugated steel pipes (CSPs) are required to be installed as part of the integral abutment design, the CSPs should be backfilled with loose, fine to medium sand as specified in Appendix-1 of the Ministry of Transportation Ontario, Structural Office Report SO-96-01 titled “Integral Abutment Bridges”. A Non Standard Special Provision (NSSP) detailing the installation method and gradation of this sand should be included in the Contract Documents; an example is included in Appendix C.

The pile caps for the new abutments should be provided with a minimum of 1.2 m of soil cover to provide adequate protection against frost penetration (as per OPSD 3090.101 – *Foundation Frost Depths for Southern Ontario)*.

Given the presence of high hydrostatic pressures in the very dense cohesionless till deposit and the groundwater seepage issues observed during the field investigation at the site, suitable construction techniques will be required to mitigate and/or control the possible upward flow of water along the pile shaft, especially at the proposed west abutment of the South Bridge where the hydrostatic pressure was encountered at a shallow depth (about Elevation 158 m). It is recommended that a sand drainage/filter, possibly in combination with a geotextile and adjacent drainage ditches, be placed beneath the pile caps to minimize the migration of fines that may be transported along the piles during and after construction and to control any water seepage. The drainage/filter layer should consist of a minimum 0.5 m thick layer of concrete fine aggregate, meeting the gradation requirements of OPSS 1002 (Aggregates Concrete) and should be included in the Design Drawings. Further details on the use of sand drainage/filter blankets are provided in Section 6.7 (Construction Considerations).

6.2.2.1 Steel H-Pile/Steel Tube Pile Founding Elevations

For steel HP 310 x 110 piles or steel tube piles (324 mm diameter x 6.4 mm thickness) driven to found within the “100-blow” very dense cohesionless till, the following design pile tip elevations may be used for the proposed east and west abutments of the Fletcher’s Creek North and South Bridges. The corresponding pile lengths were estimated assuming the pile cap is founded at the elevations shown on the GA drawing provided by AECOM.

	Option A – Lower Founding Elevation					
	North Bridge			South Bridge		
Foundation Element	**Reference Borehole No.**	**Founding Stratum**	**Estimated Design Pile Tip Elevation / Pile Length**	**Reference Borehole No.**	**Founding Stratum**	**Estimated Design Pile Tip Elevation / Pile Length**
East Abutment	FC-1, FC-5 and FC-7	Very dense silty sand and gravel to gravelly sand till	153.5 m / 9 m	FC-7 and FC-11	Very dense silty sand and gravel till	152.5 m / 10 m
West Abutment	FC-2, FC-6 and FC-8	Very dense silty sand and gravel to sand and gravel till	153.5 m / 8 m	FC-8 and FC-12	Very dense silty sand and gravel to sand and silt till	152.5 m / 9 m

These recommended founding elevations are located at depths where artesian groundwater conditions were measured at the boreholes located at the north and south bridge structures. Therefore, appropriate measures are required to mitigate risks associated with penetrating into the artesian groundwater zone.

Alternatively, shorter pile foundations could be considered for the support of the new abutments by terminating the steel HP 310 x 110 piles or steel tube piles (324 mm diameter x 6.4 mm thickness) at relatively higher elevation within the dense to very dense cohesionless till as shown below. The higher pile tip elevations provided below will reduce the risks associated with artesian control (i.e. migration of fine soil particles along the pile shafts) as well as, reduce the risk associated with encountering obstructions (i.e. cobbles and boulders) within the overburden soils that were typically encountered at or below these higher founding elevations. However, higher founding elevations will result in relatively low geotechnical resistance values which may not be adequate to resist the structural loads or for integral abutment design which typically requires a minimum 5 m long pile.

	Option B – Higher Founding Elevation					
	North Bridge			South Bridge		
Foundation Element	**Reference Borehole No.**	**Founding Stratum**	**Estimated Design Pile Tip Elevation / Pile Length**	**Reference Borehole No.**	**Founding Stratum**	**Estimated Design Pile Tip Elevation / Pile Length**
East Abutment	FC-1, FC-5 and FC-7	Dense to very dense sand and silt to silty sand and gravel till	157.0 m / 5.5 m	FC-7 and FC-11	Very dense silty sand and gravel till	157.0 m / 5.5 m
West Abutment	FC-2, FC-6 and FC-8	Dense to very dense silty sand to silty sand and gravel till	157.0 m / 4.5 m	FC-8 and FC-12	Very dense silty sand and gravel to sand and silt till	157.0 m / 4.5 m

6.2.2.2 Geotechnical Axial Resistances

For steel HP 310 x 110 piles or steel tube piles (324 mm diameter x 6.4 mm thickness) driven to the estimated design pile tip elevations provided in Section 6.2.2.1 Option A (approximately 8 m to 10 m long), the factored geotechnical axial resistance at ULS and the geotechnical reaction at SLS for 25 mm of settlement are given below. Given the high artesian pressures within the cohesionless till soils near the estimated pile tip elevations, the resistances are lower than would typically be given for driven piles in similar soil conditions with similar pile lengths.

		Option A – Lower Founding Elevation			
		North Bridge		South Bridge	
Foundation Element	**Founding Stratum**	**Factored Geotechnical Resistance at ULS**	**Geotechnical Reaction at SLS (for 25 mm of Settlement)**	**Factored Geotechnical Resistance at ULS**	**Geotechnical Reaction at SLS (for 25 mm of Settlement)**
East Abutment	Very dense cohesionless till	1,000 kN	800 kN	900 kN	700 kN
West Abutment	Very dense cohesionless till	1,000 kN	800 kN	900 kN	700 kN

The following note, or similar notation, should be shown on the Contract Drawing assuming that a resistance factor of 0.5 is applied to the use of the Hiley calculation based on MTO experience in the Southern Ontario region (refer to the Structural Manual Section 3.3.3 (MTO, 2008)) for the North and South bridges:

For the North Bridge (East and West Abutments):

> *"Piles to be driven in accordance with Standard SS-103-11 using an ultimate geotechnical resistance of 2,000 kN per pile but must be driven below Elevation 155.*5m and not below Elevation 153.5 m without approval of the Engineer."

Similarly, for the South Bridge (East and West Abutments):

> *"Piles to be driven in accordance with Standard SS-103-11 using an ultimate geotechnical resistance of 1,800 kN per pile but must be driven below Elevation 154.5 m and not below Elevation 152.5 m without approval of the Engineer."*

Alternatively, for steel HP 310 x 110 piles or steel tube piles (324 mm diameter x 6.4 mm thickness) driven to the estimated design pile tip elevations provided in Section 6.2.2.1 Option B (approximately 4.5 m to 5.5 m long), the factored geotechnical axial resistance at ULS and the geotechnical reaction at SLS for 25 mm of settlement are given below. It is recommended that a pile load test be conducted due to the relatively short pile lengths for this option to verify these recommendations.

		Option B – Higher Founding Elevation			
		North Bridge		South Bridge	
Foundation Element	**Founding Stratum**	**Factored Geotechnical Resistance at ULS**	**Geotechnical Reaction at SLS (for 25 mm of Settlement)**	**Factored Geotechnical Resistance at ULS**	**Geotechnical Reaction at SLS (for 25 mm of Settlement)**
East Abutment	Dense to very dense cohesionless till	500 kN	425 kN	500 kN	425 kN
West Abutment	Dense to very dense cohesionless till	500 kN	425 kN	500 kN	425 kN

The following note, or similar notation, should be shown on the Contract Drawing assuming that a resistance factor of 0.5 is applied to the use of the Hiley calculation based on MTO experience in the Southern Ontario region (refer to from the Structural Manual Section 3.3.3 (MTO, 2008)) for the North and South bridges:

For the North Bridge (East and West Abutments):

> *"Piles to be driven in accordance with Standard SS-103-11 using an ultimate geotechnical resistance of 1,000 kN per pile but must be driven below Elevation 158 m and not below Elevation 157 m without approval of Engineer."*

Similarly, for the South Bridge (East and West Abutments):

> *"Piles to be driven in accordance with Standard SS-103-11 using an ultimate geotechnical resistance of 1,000 kN per pile but must be driven below Elevation 158 m and not below Elevation 157 m without approval of Engineer."*

For both options, similar axial resistances and drawing notes may be used in the design for closed-end, concrete filled 324 mm (12 ¾ in.) diameter steel tube piles having a minimum wall thickness of 6.4 mm (¼ in.).

Pile installation should be in accordance with OPSS 903 *(Deep Foundations).* The pile termination or set criteria will be dependent on the pile driving hammer type, helmet, selected pile and length of pile; the criteria must therefore be established at the time of construction after the piling equipment is known. The pile capacity should then be verified in the field by the use of the Hiley formula (MTO Standard Drawing SS103-11) during the final stages of driving to achieve an ultimate capacity, as indicated in the Contract Drawing Notes above.

For steel H-piles driven into "100-blow" soil (Option A), assessment of ultimate geotechnical resistance by the Hiley formula should commence once the pile reaches a depth of not more than 2 m above the design pile tip elevation shown in Section 6.2.2.1 and at 0.5 m intervals of depth until the ultimate axial resistance is achieved. For piles driven into the till deposit at higher elevation (Option B), assessment of ultimate geotechnical resistance by the Hiley formula should commence once the pile reaches a depth of not more than 1.0 m above the design pile tip elevation shown in Section 6.2.2.1 and at 0.5 m intervals of depth until the ultimate axial resistance is achieved.

If the ultimate capacity as determined by the Hiley formula is not achieved within the 2 m and 1.0 m interval down to the design pile tip elevation for the Options A and B, respectively, the Contractor should stop pile driving and notify the Contract Administrator. At this depth, the pile should be allowed to rest for 48 hours and the Hiley formula should then be applied immediately upon re-striking the pile. If the ultimate capacity is still not achieved after the 48 hour wait period, the Contract Administrator should be notified and authorization given prior to driving the pile below the design pile tip elevation.

Given the variability in the SPT “N”-values at variable depths, it is recommended that an allowance for greater pile lengths be provided in the Contract Documents to ensure that adequate pile lengths are available on site and to reduce splicing needs. It is also recommended that the axial capacity be calculated by the Hiley formula on every pile installed.

Given the dense to very dense / very stiff to hard general nature of the till soils and the net unloading condition (due to removal of the existing approach embankment fill) at the North Bridge, to allow for greater span length, downdrag loads are not anticipated. In the widening areas where up to 5 m of new embankment fill will be placed (i.e. the South Bridge and north limit of the North Bridge), it is recommended that a downdrag load of 100 kN be included in the structural design for piles designed at the east and west abutments for both the North and South Bridges to account for the consolidation of the very soft to stiff clayey silt to clayey silt with sand layer present in the overburden.

6.2.2.3 Resistance to Lateral Loads

Resistance to lateral loading can be derived using vertical piles, with enhanced support offered by battered piles, if required. If vertical piles are used, the resistance to lateral loading will have to be derived solely from the soil in front of the piles, whereas battered piles derive lateral resistance from the soil in front of the piles as well as the horizontal component of the axial load present in the inclined pile.

The resistance to lateral loading in front of the pile may be calculated using subgrade reaction theory where the coefficient of horizontal subgrade reaction (k_h in kPa/m) is determined based on the equations given below (CFEM 1992 as noted in Section 6.8.7.1 of the *Commentary* to the *CHBDC, 2006*):

For cohesionless soils:

$$k_h = \frac{n_h z}{B}$$

Where
k_h is the coefficient of horizontal subgrade reaction (kPa/m);
n_h is the constant of subgrade reaction (kPa/m);
z is the depth (m); and
B is the pile diameter or width (m).

For cohesive soils:

$$k_h = \frac{67 s_u}{B}$$

Where
k_h is the coefficient of horizontal subgrade reaction (kPa/m);
s_u is the undrained shear strength of the soil (kPa); and
B is the pile diameter or width (m).

Although not anticipated, where integral abutment design includes the installation of 3 m long CSP liners (with the annular space between the pile and the liner filled with uniformly graded, uncompacted sand), the upper portion of the H-pile or tube pile will be generally free to flex and move laterally within the limits of the CSP. With this design, the passive lateral resistance over the length of the pile within the CSP liner should be based on the resistance provided by loose sand. The passive lateral resistance on the exterior of the CSP should be based on the resistance provided by the surrounding soil conditions.

The following values of n_h and s_u may be assumed in the structural analyses. The soil stratigraphy has been generalized and the values reflect the variability in the subsurface conditions within the foundation elements footprint, however, the deposit boundaries vary slightly at the abutments and reference can be made to the borehole records and to the interpreted stratigraphic sections for each foundation element on Drawing 2 to assess the variation.

Soil Unit	n_h (kPa/m)	s_u (kPa)
Loose sand within CSP (if applicable)	2,200	-
Soft to hard clayey silt to clayey silt with sand fill	-	40
Loose to compact sandy silt fill	3,000	-
Compact sand and silt to silty sand	4,400	-
Firm to stiff clayey silt to clayey silt with sand	-	40
Very soft to firm clayey silt to clayey silt with sand	-	20
Very Stiff to hard clayey silt with sand to clayey silt till	-	150
Dense to very dense silty sand to sand and gravel till	11,000	-

A factored geotechnical lateral resistance of 120 kN at ULS, and a geotechnical lateral reaction of 40 kN at SLS (for 10 mm of horizontal deflection at pile cap level) was calculated for a vertical free-headed HP 310x110 pile (driven predominantly within the very dense cohesionless till to Elevation 153.5 m) for the North and South Bridges, based on an analysis using the commercially available program *LPILE Plus* (Version 5.0) produced by Ensoft Inc., and checked under Clause C6.8.7.1, Table C6.4 of the *Commentary on CHBDC.* The structural capacity of the pile should be checked and verified by the structural engineer.

Group action for lateral loading should be considered where the pile spacing in the direction of the loading is less than six to eight pile diameters. Group action can be evaluated by reducing the coefficient of horizontal subgrade reaction in the direction of loading by a reduction factor, R (NAVFAC DM-7.02, 1986) as follows:

Pile Spacing in direction of Loading (d = Pile Diameter)	Subgrade Reaction Reduction Factor (R)
8d	1.00
6d	0.70
4d	0.40
3d	0.25

The subgrade reaction reduction factor should be interpolated for pile spacings in between those provided above.

6.2.3 Caisson Foundations

Consideration could be given to the use of caissons socketted into the dense to very dense cohesionless till (comprised of silty sand, sand and silt, to gravelly sand, silty sand and gravel, sandy silt and gravel, and sand and gravel) for support of the foundation elements for the proposed east and west abutments of the North and South Bridges.

If caisson foundations are adopted for support of any of the foundation elements, a temporary or permanent liner would be required to support the soils during construction, to minimize disturbance and loss of ground in the water-bearing cohesionless soils present at the site. Specialized construction techniques would be required during advancement of the caisson in order to maintain a sufficient head of water and or drilling fluid within the liner to prevent basal heave and disturbance of the water-bearing cohesionless till.

Given the artesian conditions and requirement to balance the hydrostatic head, concrete must be placed using tremie techniques. After initial placement of concrete at the bottom of the caisson, the tremie discharge point should be maintained below the surface of the wet concrete during placement. It is recommended that a Non-Standard Special Provision (NSSP) be included in the Contract Documents to address the need for control of the ground and groundwater during caisson construction as discussed further under Construction Considerations in Section 6.7.

The performance of caissons will depend upon the final cleaning and verification of the subgrade quality (dense to very dense cohesionless till) at the base of the caissons. Each caisson excavation should be carefully cleaned to remove all loosened debris to ensure that the concrete is in intimate contact with the competent bearing stratum. Difficulties verifying the base of the caisson are anticipated and base verification may not be feasible given the high artesian groundwater conditions.

The caisson caps for the new abutments at the North and South Bridges should be provided with a minimum of 1.2 m of soil cover to provide adequate protection against frost penetration (per OPSD 3090.101 – *Foundation Frost Depths for Southern Ontario)* unless the caissons are continuous to form the abutment columns, in which case caisson caps are not required.

6.2.3.1 Founding Elevations

Caissons may be founded within the cohesionless till deposit and socketted at least 1.5 m into the very dense silty sand and gravel till to sand and gravel till at the following design base elevations.

	Option A – Lower Founding Elevation					
	North Bridge			**South Bridge**		
Foundation Element	**Reference Borehole No.**	**Founding Stratum**	**Estimated Founding Elevation / Caisson Length**	**Reference Borehole No.**	**Founding Stratum**	**Estimated Founding Elevation / Caisson Length**
East Abutment	FC-1, FC-5 and FC-7	Very dense silty sand and gravel to gravelly sand till	154.5 m / 8 m	FC-7 and FC-11	Very dense silty sand and gravel till	154.5 m / 8 m
West Abutment	FC-2, FC-6 and FC-8	Very dense silty sand and gravel to sand and gravel till	154.5 m / 7 m	FC-8 and FC-12	Very dense silty sand and gravel to sand and silt till	154.5 m / 7 m

Similar to the driven pile foundation options, shorter caissons founded at a higher founding elevation within the dense to very dense till deposit may be considered to reduce the risks associated with the artesian groundwater conditions and potential difficulties augering through obstructions (i.e. cobbles and boulders) as shown below.

	Option B – Higher Founding Elevation					
	North Bridge			South Bridge		
Foundation Element	**Reference Borehole No.**	**Founding Stratum**	**Estimated Design Pile Tip Elevation / Pile Length**	**Reference Borehole No.**	**Founding Stratum**	**Estimated Design Pile Tip Elevation / Pile Length**
East Abutment	FC-1, FC-5 and FC-7	Dense to very dense sand and silt to silty sand and gravel till	157.0 m / 5.5 m	FC-7 and FC-11	Very dense silty sand and gravel till	157.0 m / 5.5 m
West Abutment	FC-2, FC-6 and FC-8	Dense to very dense silty sand to silty sand and gravel till	157.0 m / 4.5 m	FC-8 and FC-12	Very dense silty sand and gravel to sand and silt till*	157.0 m / 4.5 m

6.2.3.2 Geotechnical Resistance/Reaction

The recommended design values for factored geotechnical axial resistance at ULS and geotechnical reaction at SLS (for 25 mm of settlement) for caissons founded within the very dense till deposit at the elevations given in Section 6.2.3.1 Option A for the North and South Bridges are provided below.

			Option A – Lower Founding Elevation			
			North Bridge		South Bridge	
Foundation Element	**Founding Stratum**	**Caisson Diameter**	**Factored Geotechnical Resistance at ULS**	**Geotechnical Reaction at SLS (for 25 mm of Settlement)**	**Factored Geotechnical Resistance at ULS**	**Geotechnical Reaction at SLS (for 25 mm of Settlement)**
East Abutment	Very dense cohesionless till	0.9 m	1,500 kN	1,300 kN	1,300 kN	1,000 kN
		1.2 m	2,700 kN	2,250 kN	2,300 kN	1,900 kN
		1.5 m	4,500 kN	3,500 kN	3,700 kN	3,100 kN
West Abutment	Very dense cohesionless till	0.9 m	1,500 kN	1,300 kN	1,300 kN	1,000 kN
		1.2 m	2,700 kN	2,250 kN	2,300 kN	1,900 kN
		1.5 m	4,500 kN	3,500 kN	3,700 kN	3,100 kN

Alternatively, caissons founded within the dense to very dense cohesionless till deposit at the estimated founding elevations given in Section 6.2.3.1 Option B for the North and South Bridges, could be designed using the following recommended values for factored geotechnical axial resistance at ULS and geotechnical reaction at SLS (for 25 mm of settlement).

			Option B – Higher Founding Elevation			
			North Bridge		South Bridge	
Foundation Element	**Founding Stratum**	**Caisson Diameter**	**Factored Geotechnical Resistance at ULS**	**Geotechnical Reaction at SLS (for 25 mm of Settlement)**	**Factored Geotechnical Resistance at ULS**	**Geotechnical Reaction at SLS (for 25 mm of Settlement)**
East Abutment	Dense to very dense cohesionless till	0.9 m	750 kN	600 kN	750 kN	600 kN
		1.2 m	1,300 kN	1,100 kN	1,300 kN	1,100 kN
		1.5 m	2,100 kN	1,700 kN	2,100 kN	1,700 kN
West Abutment	Dense to very dense cohesionless till	0.9 m	750 kN	600 kN	750 kN	600 kN
		1.2 m	1,300 kN	1,100 kN	1,300 kN	1,100 kN
		1.5 m	2,100 kN	1,700 kN	2,100 kN	1,700 kN

6.2.3.3 Resistance to Lateral Loads

Resistance to lateral loading will be derived from the soil in front of the caissons. The resistance to lateral loading in front of the caisson may be calculated using subgrade reaction theory and the equations and soil parameters provided in Section 6.2.2.3 may be used for design.

A factored geotechnical lateral resistance of 500 kN at ULS, and a geotechnical lateral reaction of 215 kN at SLS (for 10 mm horizontal deflection at the caisson cap level) was calculated assuming a 1.5 m diameter free-headed caisson founded at Elevation 154.5 m based on an analysis using the commercially available program *LPILE Plus* (Version 5.0) produced by Ensoft Inc. The structural capacity of the caisson should be checked and verified by the structural engineer.

6.3 Bridge Retaining Walls

A total of eight (8) retaining wall structures are associated with the proposed Fletcher's Creek North and South Bridges: four (4) walls at the North Bridge (consisting of two (2) walls each at the east and west abutments) and four walls at the South Bridge (consisting of two (2) walls each at the east and west abutments). The retaining walls are oriented parallel to Highway 401. According to the GA drawing provided by AECOM, the retaining walls for both bridges range from 8 m to 9 m long and the bottom of the retaining walls generally follows the slope of the embankment front slope (oriented at 2H:1V). The location and approximate length of the retaining walls for the North and South Bridges are summarized below.

	North Bridge	South Bridge
Foundation Element	Retaining Wall Length	Retaining Wall Length
Northeast Retaining Wall	8.7 m	8.0 m
Southeast Retaining Wall	8.0 m	8.0 m
Northwest Retaining Wall	9.3 m	8.6 m
Southwest Retaining Wall	8.6 m	8.6 m

The feasible retaining wall options in the northeast, southeast, northwest and southwest quadrants of the Fletcher's Creek bridge structures may include the following:

- Concrete retaining walls supported on spread footings founded on the dense to very dense cohesionless till for the North and South Bridges, and/or founded on compact silty sand or very stiff to hard clayey silt till at the South Bridge are considered feasible for the support of the retaining walls. Consideration could also be given to supporting the retaining wall on engineered fill consisting of Granular 'B' Type II founded on these soils or on the clayey silt to clayey silt with sand deposit provided settlement mitigation options are provided as outlined in Section 6.6.3.2. The foundation recommendations for this option are detailed in Section 6.3.1.
- Concrete retaining walls supported on deep foundations (steel H-Piles, tube piles or caissons), often cantilevered from the abutment foundation for integral abutments, are considered suitable for the support of the retaining walls. The foundation recommendations provided in Sections 6.2.2 and 6.2.3 for piles and caissons can be used to design this type of retaining wall foundations.
- Retained Soil System (RSS) walls supported on engineered fill (i.e Granular 'B' Type II) founded on the dense to very dense cohesionless tills, very stiff to hard clayey silt till or compact silty sand are considered to be a feasible option. Consideration could also be given to founding the RSS walls on the clayey silt to clayey silt with sand deposit provided settlement mitigation measures are performed such that differential settlements are within tolerable limits. The foundation recommendations for this option are detailed in Section 6.3.3.

6.3.1 Spread Footings

6.3.1.1 Founding Elevations

For support of the new retaining walls, strip or spread footings founded below the topsoil, fill, loose surficial soils and clayey silt to clayey silt with sand deposit, on the dense to very dense cohesionless till or compact silty sand or very stiff to hard clayey silt till deposits can be considered. As a result, sub-excavation depths between about 1.5 m and 6 m through the existing fill and cohesive soil will be required.

All footings should be founded at a minimum depth of 1.2 m below the adjacent final grade to provide adequate protection against frost penetration, in accordance with Ontario Provincial Standard Drawing (OPSD) 3090.101 (*Foundation Frost Penetration Depths for Southern Ontario).*

The following founding elevations for new retaining walls are recommended for strip or spread footings founded on competent native soil or for engineered fill placed to support shallow footings at a higher founding elevation at the proposed North and South Bridges.

	North Bridge			South Bridge		
Foundation Element	**Reference Borehole No.**	**Founding Stratum**	**Maximum Founding Elevation**	**Reference Borehole No.**	**Founding Stratum**	**Maximum Founding Elevation**
Northeast Retaining Wall	FC-1	Very dense sand and silt to sand and gravel till	159.2 m	FC-7 and FC-11	Very dense silty sand and gravel till	158.1 m
Southeast Retaining Wall	FC-7	Very dense silty sand and gravel till	158.1 m	FC-11	Compact silty Sand	159.9 m
Northwest Retaining Wall	FC-2	Dense to very dense sandy silt and gravel to sand and gravel till	158.6 m	FC-8	Very dense silty sand and gravel till	158.4 m
Southwest Retaining Wall	FC-8	Very dense silty sand and gravel till	158.0 m	FC-12	Very stiff to hard clayey silt till	162.4 m

Considering the recommended founding depth for most of the retaining walls, the subexcavated area (between about 1.5 m and 6 m deep) could be replaced with properly placed and compacted Granular ‘A’ or Granular ‘B’ Type II (SP 110S13 Aggregates) to found the shallow footings at a higher elevation, as described in Section 6.2.1.1. Temporary shoring would be required and is discussed in Section 6.7.

In addition, given the high water pressures encountered in the cohesionless till deposit and the potential running/flowing of the sand and silt to silty sand layers within the excavation depths, temporary dewatering would be required in advance of excavation and during backfilling and construction of the retaining wall foundations at the majority of the retaining wall locations.

6.3.1.2 Geotechnical Resistance/Reaction

Strip or spread footings placed on the properly prepared subgrade, at or below the design elevations given in the Section 6.3.1.1, should be designed based on the factored geotechnical resistances at ULS and geotechnical reaction at SLS (for 25 mm of settlement) given below. These design values take into account the depth of embedment (based on the design founding elevations) and proximity to the Fletcher’s Creek valley slope, where applicable.

	North Bridge		South Bridge	
Foundation Element	**Factored Geotechnical Resistance at ULS**	**Geotechnical Reaction at SLS (for 25 mm of Settlement)**	**Factored Geotechnical Resistance at ULS**	**Geotechnical Reaction at SLS (for 25 mm of Settlement)**
Northeast Retaining Wall	900 kPa	600 kPa	900 kPa	600 kPa
Southeast Retaining Wall	900 kPa	600 kPa	600 kPa	350 kPa
Northwest Retaining Wall	900 kPa	600 kPa	900 kPa	600 kPa
Southwest Retaining Wall	900 kPa	600 kPa	500 kPa	300 kPa

If the strip or spread footings are founded on compacted Granular 'A' or Granular 'B' Type II at higher elevations, a factored geotechnical resistance at ULS of 750 kPa and a geotechnical reaction at SLS (for 25 mm of settlement) of 350 kPa could be employed for the design of the retaining wall foundations, assuming the granular pad is at least 2 m thick.

The ULS resistance and settlement are dependent on the footing size (assumed to be at least 2 m wide), configuration and applied loads; the geotechnical resistances should, therefore, be reviewed if the selected footing width or founding elevation differs from those given above.

The geotechnical resistances provided above are given for loads applied perpendicular to the surface of the footing. 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 6.7.4 and C6.7.4 in the *Canadian Highway Bridge Design Code (CHBDC) and* its *Commentary,* using the curves for cohesive soils and non-cohesive soil.

The base of each footing excavation should be cleaned of softened / loosened soil and it is recommended that the founding level for the footings be inspected by a Quality Verification Engineer (QVE) following excavation, in accordance with OPSS 902 *(Excavating and Backfilling Structures)* to verify that all existing fill and other unsuitable material have been removed. If the concrete for the footings cannot be poured immediately after excavation and inspection, it is recommended that a concrete working slab (100 mm thickness of 20 MPa compressive strength concrete) be placed on the subgrade to protect the integrity of the bearing stratum. This requirement can either be added as a note on the Contract Drawings or included as a Non-Standard Special Provision (NSSP) in the Contract Documents. A sample NSSP is included for this item in Appendix C.

6.3.1.3 Resistance to Lateral Loads

Resistance to lateral forces / sliding between the concrete footings and the subgrade should be calculated in accordance with Section 6.7.5 of the *CHBDC*. For cast-in-place concrete footings constructed on the dense to very dense cohesionless till or on the Granular 'A' or Granular 'B' Type II engineered fill, the coefficient of friction, tan ϕ', can be taken as 0.5; if constructed on the to very stiff to hard clayey silt till or the compact silty sand, the coefficient of friction, tan ϕ', can be taken as 0.45. These values are unfactored.

6.3.2 Steel H-Pile / Steel Tube Foundations

Concrete retaining walls supported on steel 310 x 110 piles or steel tube piles (324 mm x 6.4 mm thickness) or cantilevered from the abutments which are founded on deep foundations is the most practical design option at all retaining wall locations.

The geotechnical founding elevations, resistances and recommendations for the design of pile and caisson foundations can be taken from Sections 6.2.2 and 6.2.3, respectively.

6.3.3 Retained Soil System (RSS) Walls

6.3.3.1 Founding Elevations

A typical RSS wall has a front facing supported on a strip footing placed at shallow depth below the ground surface in front of the wall; this footing, and the RSS mass, should be founded below any topsoil, existing fill and the clayey silt to clayey silt with sand layer. For this site, it is recommended that the existing fill material, topsoil and very soft to stiff clayey silt soils within the proposed RSS wall footprint be subexcavated and replaced with engineered fill consisting of Granular ‘A’ or Granular ‘B’ Type II as described in Section 6.3.1.1. The RSS soil mass and facing footing may be supported on the Granular ‘A’ or Granular ‘B’ Type II engineered fill founded on competent native soils of dense to very dense cohesionless till, compact silty sand or very stiff to hard clayey silt till. The RSS retaining walls are to be designed for high performance and appearance in accordance with MTO Special Provision (SP) 599S22 (Retained Soil System).

The facing footing should be placed on a minimum 300 mm thick layer of compacted SP 110S13 Granular ‘A’, as shown in Figure 5.2 in the MTO RSS Wall Design Guidelines (September 2008). The compacted granular pad should extend at least 1.0 m beyond the outside edge of the facing footing, then downward at 1H:1V.

The reinforced soil mass should be keyed into the existing embankment fill side-slope, if applicable, by benching into the embankment fill, similar to OPSD 208.010 (Benching of Earth Slopes).

6.3.3.2 Geotechnical Resistances

Assuming that the RSS wall acts as a unit and uses the full width of the reinforced soil mass (assumed to be about 70% of the retained height), the factored geotechnical resistance at ULS and the geotechnical reaction at SLS (for 25 mm of settlement) given below may be used for design. It is assumed that the reinforced mass and facing footing are founded on Granular ‘A’ or Granular ’B’ Type II engineered fill supported on native subgrade at the founding elevations given in Section 6.3.3.1 for the North and South Bridge structures.

RSS Wall Location	Approximate Maximum Exposed Wall Height	Factored Geotechnical Resistance at ULS	Geotechnical Reaction at SLS (for 25 mm Settlement)	Founding Soil Condition
South Bridge - Southeast RSS Wall	5 m	600 kPa	350 kPa	Granular ‘A’ or Granular ‘B’ Type II engineered fill over compact silty sand
South Bridge - Southwest RSS Wall	5 m	500 kPa	300 kPa	Granular ‘A’ or Granular ‘B’ Type II engineered fill over compact very stiff to hard clayey silt till
All other RSS Walls	5 m	900 kPa	600 kPa	Granular ‘A’ or Granular ‘B’ Type II engineered fill over compact dense to very dense cohesionless till

6.3.3.3 *Resistance to Lateral Loads*

The resistance to lateral forces / sliding between the compacted granular fill (Granular ‘A’ or Granular ‘B’ Type II) and the subgrade should be calculated in accordance with Section 6.7.5 of the *CHBDC*. The coefficient of friction, tan ϕ’, between the compacted granular fill of the RSS wall and the compacted Granular ‘A’ or Granular ‘B’ Type II material or native dense to very dense cohesionless till may be taken as 0.55; if constructed on the very stiff to hard clayey silt till or the compact silty sand, the coefficient of friction, tan ϕ’, can be taken as 0.45. For the precast footing on compacted granular material, the coefficient of friction, tan ϕ’, can be taken as 0.5. These values are unfactored.

6.3.4 Global Stability

The static and seismic global stability of the retaining walls founded on strip or spread footings at the proposed Fletcher’s Creek bridge structures has been analyzed using the commercially-available program SLIDE (Version 6.0), produced by Rocscience Inc., employing the Janbu Corrected, GLE/Morgenstern-Price, and Spencer method of analyses. For all analyses, the factor of safety of numerous potential failure surfaces was computed in order to establish the global minimum factor of safety. A target factor of safety of 1.5 against deep-seated global instability of the retaining walls is normally accepted by MTO for wall design under static conditions; under seismic conditions, a target Factor of Safety of 1.1 has been used. These factors of safety are considered appropriate for the retaining walls at this site, considering the design requirements and the field data available.

Drained and undrained analyses were carried out for the slope stability assessment. The critical soil parameters used in the analyses, as given below, were estimated from empirical correlations using the results of in-situ Standard Penetration Tests (Bowles, 1984) and geotechnical classification testing. The groundwater table in the upper soil layers was modelled to be at Elevation 164.4 m (i.e. Fletcher’s Creek High Water Level) in the analyses. The artesian hydrostatic head within the cohesionless and silt to gravelly sand till was modelled to be at Elevation 172 m (as measured in September 2012).

Soil Type	Unit Weight (kN/m^3)	Undrained Shear Strength (kPa)	Angle of Internal Friction, φ' (degrees)
Existing embankment fill	20	40	-
Compact silt to sand and silt	20	-	30
Firm to stiff clayey silt	20	40	-
Very soft to firm clayey silt to clayey silt with sand	19	20	-
Dense to very dense sand and silt to gravelly sand till	21	-	35

Based on the analysis results, the factor of safety against global instability of the retaining walls adjacent to the abutment walls at the North and South Bridge structures is greater than 1.5 for the analysed retaining wall areas. The result of a static slope stability analysis for the retaining walls adjacent to the east abutment is provided on Figure 1.

Under seismic loading conditions, using a design seismic coefficient equal to the 50 per cent of the site-specific design peak horizontal ground acceleration (PGA) which is about 0.03 g, the Factor of Safety is greater than 1.1. The result of the seismic slope stability analysis for the retaining walls along the east abutment is shown on Figure 2.

6.4 Lateral Earth Pressures for Design

The lateral earth pressures acting on the proposed east and west abutment stems and any associated retaining walls will depend on the type and method of placement of the backfill materials, the nature of the soils behind the backfill, the magnitude of surcharge including construction loadings, the freedom of lateral movement of the structure, and the drainage conditions behind the walls.

The following recommendations are made concerning the design of the abutment and retaining walls. These design recommendations and parameters assume level backfill and ground surface behind the walls. Where there is sloping ground behind the stems/walls, the coefficient of lateral earth pressure must be adjusted to account for the slope.

- Select, free draining granular fill in general accordance with SP 110S13 (Aggregates) Granular 'A' or Granular 'B' Type II but with less than 5 per cent passing the 200 sieve should be used as backfill behind the walls. Compaction (including type of equipment, target densities, etc.) should be carried out in accordance with OPSS 501 (*Compacting*). Longitudinal drains and weep holes should be installed to provide positive drainage of the granular backfill. Other aspects of the granular backfill requirements with respect to sub-drains and frost taper should be in accordance with OPSD 3101.150 – *Wall, Abutments Backfill* and OPSD 3121.150 – *Walls Retaining, Backfill.*

- 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. Other surcharge loadings should be accounted for in the design as required.

- For restrained structures, the granular fill may be placed either in a zone with the width equal to at least 1.2 m behind the back of the walls (see Case A in Figure C6.20 (a) of the *Commentary* to the *CHBDC*). For unrestrained structures, the granular fill should be placed within the wedge shaped zone defined by a line drawn at 1.5 horizontal to 1 vertical (1.5H:1V) extending up and back from the rear face of the footing (see Case B in Figure C6.20(b) of the *Commentary* to the *CHBDC*).

- For restrained structures, the pressures are based on the existing embankment fill materials and the following parameters (unfactored) may be used assuming the use of the native clayey silt to clayey silt with sand fill or the sandy silt fill:

	Earth Fill
Soil Unit Weight	20 kN/m^3
Coefficient of static lateral earth pressure Active, K_a At rest, K_o	 0.33 0.50

- For unrestrained structures, where the pressures are based on SP 110S13 granular fill behind the wall, the following parameters (unfactored) may be assumed:

	Granular 'A'	**Granular 'B' Type II**
Soil Unit Weight	22 kN/m^3	21 kN/m^3
Coefficient of static lateral earth pressure Active, K_a At rest, K_o	 0.27 0.43	 0.27 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 (such as for a rigid frame structure), at-rest earth pressures should be assumed for geotechnical design. The movement required to allow active pressures to develop within the backfill, and thereby assume an unrestrained structure for design, should be calculated in accordance with Section C6.9.1 and Table C6.6 of the *Commentary* to the *CHBDC*.

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

6.5 Seismic Site Coefficient

For seismic design purposes, the Site Coefficient (S) for this site may be taken as 1.2, consistent with Soil Profile Type II. The soil profile is based on the guidelines in Section 4.4.6 and Table 4.4 of the *CHBDC (2006)* and local experience.

6.5.1 Seismic Analysis Coefficient

The potential for seismic (earthquake) loading may also need to be considered for the design of new abutment stems/reinforced soil mass and for the assessment of liquefaction potential of foundation soils in accordance with Section 4.6 of the *CHBDC,* as significant seismic loading will result in increased lateral earth pressures acting on the abutment stem and reinforced soil mass systems. At this site, the requirements for seismic analysis are outlined as follows:

According to Table A3.1.1 of the *CHBDC*, this site is located in Seismic Zone 1. The site-specific zonal acceleration ratio for the City of Mississauga is 0.05. Based on experience, for the subsurface conditions at this site, a 20 per cent amplification of the ground motion may occur (i.e. Site Coefficient, S=1.2 for Soil Profile II from Table 4.4 of *CHBDC*), resulting in an increase in the peak horizontal ground acceleration (PGA) from 0.05 g to 0.06 g at the ground surface. Based on Section 4.4.4 of the *CHBDC*, this bridge structure is assigned Seismic Performance Zone 1. Given this, and in accordance with Section 4.4.5.1 of the *CHBDC*, no seismic analysis is required for structures located in Seismic Performance Zone 1.

6.6 Approach Embankments

The existing Highway 401 approach embankments in the vicinity of Fletcher's Creek are constructed from fill. The existing Highway 401 pavement surface in the general area of Fletcher's creek varies from about Elevation 168.7 m to 168.5 m. The proposed widening of Highway 401 involves constructing two new single-span North and South Bridges with east and west abutments to accommodate the Highway 401 widening. The proposed north widening of Highway 401 (proposed to carry Highway 401 westbound collector lanes over Fletcher's Creek) will require removal of the existing culvert and the approach embankments immediately behind the culvert up to about 10 m east and 8 m west; and the widening of the existing embankment up to about 30 m north. The proposed Highway 401 grade near the North Bridge is between about Elevation 168.7 m and 168.5 m near the abutments. As a result, there is essentially no grade change between the existing and proposed Highway 401 road profiles. The existing ground surface at the north limit of this site varies between Elevation 165.9 m and 163.7 m at the east approach, and between Elevation 164.4 m and 163.8 m at the west approach, with the ground surface generally sloping down towards Fletcher's Creek. Therefore, the maximum height of the widened approach embankments near the Fletcher's Creek valley will be about 5 m.

The proposed south widening of Highway 401 (proposed to carry Highway 401 eastbound collector lanes over Fletcher's Creek) will require construction of a new embankment and South Bridge in the vicinity of the creek and includes widening of the existing embankment up to about 40 m south. The existing ground surface at the south side of this site varies from between Elevation 164.4 m and 164.3 m at the new east approach and between about Elevation 164.1 m and 163.9 m at the new west approach, with the ground surface generally flat in this area. Therefore, the new approach embankments will be constructed by the placement of up to about 4.1 m of new fill at the east approach and up to about 4.3 m of new fill at the west approach.

The existing embankment side-slopes are sloped between about 2.3H:1V and 2.8H:1V (generally about 2.5 H:1V) and the new embankment side-slopes are proposed to be sloped at 2H:1V.

Based on the results of the boreholes drilled at this site, the existing east and west approach embankments for the North and South Bridges are generally founded on a very soft to very stiff clayey silt to clayey silt with sand deposit containing organics underlain by dense to very dense cohesionless till deposits of varying composition. At the east abutment, a loose to compact sand and silt layer was encountered below the existing embankment fill and above the clayey silt deposit.

6.6.1 Subgrade Preparation and Embankment Construction

The existing embankments have been constructed using cohesive fill which was encountered at all the boreholes advanced during the current investigation, with a layer of sand and gravel fill overlying the cohesive fill for pavement construction. It is understood that the existing embankment fill will be excavated and removed to 10 m east and to 8 m west of the existing culvert to allow construction of the new longer span North Bridge. In order to achieve adequate performance of the new approach embankments (i.e. reduce the potential for post-construction settlement and to achieve adequate stability of the embankment), it is recommended that the clayey silt embankment material (containing organics and wood fragments) not be re-used for the new approach embankments at the north and south limits of the site but could be re-used as fill in less settlement sensitive areas.

Prior to the placement of any embankment fill, all topsoil, organic matter and existing soft/loose fill should be stripped from below the approach embankment areas. Any new embankment fill should be placed and compacted in accordance with SP 206S03 (Earth Excavation and Grading), OPSS 501 (Compacting) and SP 105S21 (Amendment to OPSS 501), with inspection and field density testing by qualified personnel during placement operations to confirm that appropriate materials are used and that adequate levels of compaction are achieved.

The use of suitable granular fill for the approach embankments is recommended rather than the reuse of the existing cohesive fill, since the majority of settlement of granular fills would occur during construction whereas some settlement of cohesive fills, if used, would occur post-construction.

Topsoil and seeding or pegged sod should be placed as soon as practicable following the completion of the approach embankment construction to reduce the potential for erosion of the embankment side slopes due to surface water runoff and to establish vegetation within the affected portion of the slopes. Topsoil should be placed on granular fill side-slopes in accordance with OPSS 802 (*Topsoil*) and covered with erosion protection in accordance with OPSS 804 *(Seed and Cover)* or pegged sod in accordance with OPSS 803 *(Sodding)*. Topsoil and erosion protection should be placed in early summer to avoid wet periods of the year which may cause surficial sloughing of the topsoil material along the side-slopes and to establish vegetation prior to the Fall / Winter months.

6.6.2 Approach Embankment Stability

Static and seismic slope stability analyses for the proposed widened approach embankments at the north and south limits of the site were carried out using the commercially available program Slide (produced by Rocscience Inc.) to check that the target minimum factor of safety was achieved for the proposed embankment heights and geometries. 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 used in the design of embankment slopes under static conditions. This factor of safety is considered adequate for the embankments at this site.

The soil parameters used in the analysis, as given below, were estimated from empirical correlations proposed by Kulhawy and Mayne (1990) and the CHBDC (2006) using the results of in-situ Standard Penetration Tests (SPT) and geotechnical classification testing. For the purpose of analysis, granular fill has been considered for the construction of the widened approach embankment with side slopes at 2H:1V. The shallow groundwater table used in the analyses was taken to be Elevation 164.4 m. The deep artesian hydrostatic head was modelled to be at Elevation 172 m within the cohesionless till deposit.

Approach Embankment	Soil Type	Unit Weight (kN/m^3)	Undrained Shear Strength (kPa)	Angle of Internal Friction, φ' (degrees)
East Approach (Boreholes FC-3, FC-9 and FC-13/13A)	New embankment fill	21	-	30
	Existing embankment fill	20	40	-
	Firm to stiff clayey silt	20	40	-
	Compact silt to sand and silt	20	-	30
	Very soft to firm clayey silt to clayey silt with sand	19	20	-
	Dense to very dense sand and silt to gravelly sand till	21	-	35
West Approach (Boreholes FC-4, FC-10 and FC-12)	New embankment fill	21	-	30
	Existing embankment fill	20	30	-
	Firm to stiff clayey silt	20	40	-
	Very soft to soft clayey silt to clayey silt with sand	20	8	-
	Very Dense sand and silt till	21	-	35

At the east and west approach embankments, assuming appropriate subgrade preparation (i.e. removal of topsoil, organics or any loose/soft surficial soils) and proper placement and compaction of the new embankment fill materials, the proposed 5 m high embankments with side slopes maintained at 2H:1V will have a Factor of Safety (FoS) greater than 1.3 against deep-seated slope instability. The results of the analysis along the east approach embankment are shown on Figure 3. Under seismic loading conditions, the design seismic coefficient value of 0.03 g (50% of the PGA) was modelled and the Factor of Safety is greater than 1.1. The results of the seismic stability analysis at the east approach embankment are shown on Figure 4.

6.6.3 Approach Embankment Settlement

Settlement of the widened sections of the Highway 401 approach embankments will occur due to compression of the new embankment fill (up to about 5.0 m high at the east and west embankments at the north widening; and between about 4.1 m and 4.3 m high at the east and west embankments of the south widening, respectively), as well as due to compression of the existing embankment fill side-slope (where present) and underlying native soils due to the widened embankment load. The compression of the subsoils was modelled by estimating an elastic modulus of deformation based on the SPT "N"-values and correlations proposed by Bowles (1984) and Kulhawy and Mayne (1990). The coefficient of consolidation, c_v (cm^2/s), required in the time-rate analysis was established using the results of the laboratory tests and/or estimated from the U.S. Navy (1986) correlation with liquid limits assuming normally-consolidated soils.

The values of the parameters used in the analyses of settlement for both the east and west approach embankments at the north and south widening are given below and are based on the soil conditions encountered in the closest boreholes advanced near the approach embankments. The shallow groundwater table used in the analyses was taken to be at Elevation 164.4 m.

Soil Deposit	Bulk Unit Weight	Estimated Deformation Properties
Existing clayey silt fill	20 kN/m^3	E = 15 MPa - 40 MPa
Compact silty sand	20 kN/m^3	E = 27 MPa
Very soft to very stiff clayey silt to clayey silt with sand	19 kN/m^3 - 20 kN/m^3	Cc = 0.138 - 0.221 Cr = 0.024 - 0.040
Very Stiff to Hard clayey silt till	21 kN/m^3	E = 40 MPa
Dense to very dense silty sand to sand and gravel till	21 kN/m^3	E = 150 MPa

East Approach Embankment Settlement Results

Below the east approach embankment at the proposed north widening (north east quadrant of North Bridge) where up to 3.7 m of the very soft to very stiff clayey silt deposit would remain in place below the existing embankment footprint, the results of the analyses estimate that, about 45 mm to 115 mm of time-dependent settlement due to primary consolidation of the cohesive deposit; and about 15 mm to 45 mm of immediate settlement due to compression of the cohesionless till would occur.

At the proposed south widening of the east approach embankment (south approach embankment to South Bridge) where up to 2.3 m of the very soft to very stiff clayey silt deposit would remain in place below the existing embankment footprint, the results of the analyses estimate that about 50 mm to 80 mm of time-dependent settlement due to primary consolidation of the cohesive deposit; and about 5 mm to 15 mm of immediate settlement due to compression of the cohesionless till and compact silty sand (where present) would occur.

The "immediate" settlements are expected to occur during or shortly after construction in response to the placement of the new embankment fill.

Based on an estimated coefficient of consolidation (C_v) of 6 x 10^{-3} cm^2/s and 9 x 10^{-3} cm^2/s for the very soft/stiff to very stiff cohesive layers (and the imposed loading conditions, and assuming two-way drainage of the clayey silt deposit), it is estimated that about 90% of the primary consolidation settlement will be completed within about 60 days and 30 days at the North and South Bridge east approach embankments, respectively. The magnitude of secondary consolidation (creep) settlement for the cohesive deposit on both the north and south widening of the east approach embankment is expected to be about 6 mm and 4 mm per log-cycle of time, respectively, corresponding to about 15 mm (at the north widening) and 10 mm (at the south widening) over a twenty-year (20-year) period following completion of construction.

West Approach Embankment Settlement Results

Below the west approach embankment at the proposed north widening (north west quadrant of North Bridge), where a 3.8 m thick layer of very soft to stiff clayey silt deposit would remain in place below the existing embankment footprint, the results of the analyses estimate that (as a result of estimated maximum fill placement height of up to about 5.0 m) about 85 mm to 160 mm of time-dependent settlement due to primary consolidation of the cohesive deposit; and about 5 mm to 10 mm of immediate settlement due to compression of the cohesionless till would occur.

At the proposed south widening of the west approach embankment (South Bridge) where up to 4.1 m of a firm to hard clayey silt till deposit would remain in place below the existing embankment footprint, the results of the analyses estimate that (as a result of a maximum fill placement height of up to about 4.8 m) about 15 mm to 40 mm of time-dependent settlement due to primary consolidation of the cohesive deposit; and about 5 mm to 10 mm of immediate settlement due to compression of the cohesionless till would occur.

The "immediate" settlements are expected to occur during or shortly after construction in response to the placement of the new widened embankment fill.

Based on an estimated coefficient of consolidation (C_v) of 7 x 10^{-3} cm^2/s and 8 x 10^{-3} cm^2/s for the very soft to stiff/firm to hard cohesive layers (and the imposed loading conditions, and assuming two-way drainage of the clayey silt deposit), it is estimated that about 90% of the primary consolidation settlement will be completed within about 50 days at the proposed North and South Bridge west approach embankments. The magnitude of secondary consolidation (creep) settlement for the cohesive deposit on both the north and south widening of the west approach embankment is expected to be about 5 mm per log-cycle of time, corresponding to about 10 mm over a twenty-year (20-year) period following completion of construction.

6.6.3.1 Settlement of Embankment Fill

A maximum thickness of up to about 5.0 m and 4.3 m of additional fill will be required as part of the north and south widening of Highway 401. Provided that the new fill is comprised of suitable (granular) earth fill meeting the requirements of and placed and compacted in accordance with SP 206S03, the settlement of the fill itself is expected to be less than about 15 mm, and this settlement is expected to occur relatively quickly, during and immediately following construction.

6.6.3.2 Mitigation of Settlement at Approach Embankments

East Approach Embankments

As discussed in Sections 6.6.3 and 6.6.3.1, between 45 mm and 115 mm of time-dependent settlement of the cohesive deposits is estimated below the east approach embankment at the North Bridge and between 50 mm and 80 mm of time-dependent settlement of the cohesive deposits is estimated below the east approach embankment at the South Bridge. To reduce the magnitude of the post-construction settlements to within the target settlement performance of less than 25 mm (in accordance with Section 1.2 of the MTO Foundation Guideline, Embankment Settlement Criteria for Design, dated March 2010), it is recommended that the east approach embankment be constructed early in the construction schedule such that the North Bridge approach embankment is preloaded for a minimum period of 60 days and the South Bridge approach embankment is preloaded for a minimum period of 30 days to allow the majority of the compression/consolidation settlement to occur prior to construction of the approach slab and final grading and paving of the highway. If the construction schedule can accommodate this preload period, the magnitude of remaining primary consolidation settlement and secondary consolidation settlement is estimated to be about 25 mm and 10 mm at the North and South Bridge east approach embankments, respectively, within the first twenty (20) years following completion of construction.

Alternatively, if a surcharge fill (2 m high) is incorporated into the east approach embankment during construction, the recommended surcharge period could be reduced to 30 days for the North and South Bridge sites. If the construction schedule can accommodate these surcharge periods, the magnitude of remaining primary consolidation settlement and the secondary consolidation settlement is estimated to be less than 5 mm on both sides of the widened embankment, within the first twenty (20) years following completion of construction. Based on the results of the stability analysis for the east approach embankment incorporating a 2 m high surcharge, a Factor of Safety greater than 1.3 will be achieved against deep seated failure.

Alternatively, sub-excavation of the very soft to stiff clayey silt to clayey silt with sand soils (up to about 11 m below the Highway 401 and 6 m below the surrounding grade) within the footprint of the north and south widening of the east approach embankment to eliminate consolidation settlement is possible. However, this option will require significant temporary roadway protection and likely result in major traffic disruption on the Highway 401 and environmental impact to Fletcher's Creek. As a result, this option is not considered practical or economical at this site.

West Approach Embankment

As discussed in Sections 6.6.3 and 6.6.3.1, between 85 mm and 160 mm of time-dependent settlement of the cohesive deposits is anticipated below the west approach embankment at the North Bridge and between 15 mm and 40 mm of time-dependent settlement of the cohesive deposits is anticipated below the west approach embankment at the South Bridge. To reduce the magnitude of the post-construction settlements to within the target settlement performance of less than 25 mm (in accordance with Section 1.2 of the MTO Foundation Guideline, Embankment Settlement Criteria for Design, dated March 2010), it is recommended that the west approach embankment be constructed early in the construction schedule such that the north side of the widened embankment (i.e. North Bridge approach) is preloaded for a period of at least 60 days and the south side of the widened embankment (i.e. South Bridge approach) is preloaded for a period of at least 30 days to allow the majority of the compression/consolidation settlement to occur prior to construction of the approach slab, final grading and paving of the highway. If the construction schedule can accommodate this preload period, the magnitude of remaining primary consolidation settlement and the secondary consolidation settlement is estimated to be about 20 mm at the North and South approach embankments, respectively, within the first twenty (20) years following completion of construction.

Alternatively, if a surcharge fill (2 m high) is incorporated into the west approach embankment during construction, the recommended surcharge period could be reduced to 30 days for the North and South Bridge approaches. If the construction schedule can accommodate these surcharge periods, the magnitude of remaining primary consolidation settlement and the secondary consolidation settlement is estimated to be less than 10 mm on both sides of the widened embankment, within the first twenty (20) years following completion of construction. Based on the results of the stability analysis for the west approach embankment incorporating a 2 m high surcharge, a Factor of Safety greater than 1.3 will be achieved against deep seated failure.

Alternatively, sub-excavation of the very soft to stiff/firm to hard clayey silt to clayey silt with sand soils (up to about 11 m below Highway 401 grade and about 6 m below the surrounding grade) within the footprint of the north and south widening of the west approach embankment widening is possible. However, this option will require significant temporary roadway protection and likely result in major traffic disruption on the Highway 401. As a result, this option is not considered practical or economical at this site.

6.7 Construction Considerations

6.7.1 Open-Cut Excavation

The foundation excavations at the abutments for spread footings or pile cap construction at the North and South Bridge structures will extend through the existing firm to hard clayey silt fill, loose to compact sand and silt, and possibly into the very soft to firm clayey silt deposits and underlying very stiff to hard clayey silt till and dense to very dense cohesionless till deposits. Where space permits, open-cut excavations into these materials should be carried out in accordance with the guidelines outlined in the Occupational Health and Safety Act (OHSA) for Construction Activities. The existing fill and sand and silt materials are classified as Type 3 soil; the native very soft to firm clayey silt deposits and cohesionless till (containing silt, sand, and gravel zones under high artesian pressures) is classified as Type 4 soil; and the clayey silt till is classified as Type 2 soil, according to the OHSA. Temporary excavations (i.e. those which are open for a relatively short time period) should be made with side slopes no steeper than 1H:1V through the Type 2 and 3 soils and to within 1.2 m of the bottom of the excavation in Type 2 soils only. If Type 4 soils are expected to be encountered within a sub-excavation (near the ground surface or at depth), the sides of the excavation should be sloped at 3H:1V from ground surface down to the bottom of the Type 4 soils.

6.7.2 Temporary Excavation Support

Temporary excavation support is likely required to facilitate the construction of the new abutments, approach embankments and associated retaining walls at the North and South Bridge structures in order to maintain traffic on Highway 401, limit environmental impact on the floodplain and creek alignment, and to reduce the quantity of sub-excavation required for the project. The temporary excavation support systems should be designed and constructed in accordance with OPSS 539 *(Temporary Protection Systems).* The lateral movement of the temporary shoring system should meet Performance Level 2 as specified in OPSS 539, provided that the existing structure, as well as any adjacent utilities, can tolerate this magnitude of deformation.

It is considered that either a driven, interlocking sheetpile system or a soldier pile and timber lagging system would be suitable for the temporary excavation support at this site, based on the inferred subsurface soil conditions and groundwater conditions. An interlocking sheetpile system would contribute to both ground and, where applicable, groundwater control of seepage from cohesionless zones or interlayers/lenses within the cohesive deposits will be required. For a soldier pile and lagging system, it would be necessary to control seepage or include measures to mitigate loss of soil particles through the lagging boards in the water-bearing cohesionless soils encountered, and with the presence of high groundwater level at this site.

If deep excavation for the shallow foundation options is considered (up to 11 m below the Highway 401 road structure), a more elaborate excavation support system will be required. Lateral support to the sheetpiles or soldier piles could be provided in the form of rakers, temporary anchors or cross-bracing. The selection and design of the protection system will be the responsibility of the Contractor.

6.7.3 Groundwater and Surface Water Control

During the initial MTO start-up meeting for this project, Golder was informed that springs or “quick” sand was present in the vicinity of the site. These springs (i.e. groundwater seepage) were confirmed during our initial site visit in areas near the creek where surficial loosened sand was present.

Artesian groundwater conditions were encountered in most of the boreholes advanced at the proposed east and west abutments. The high artesian pressures (measured to be as high as Elevation 172 m in September 2012) were typically encountered within the cohesionless till deposit. The groundwater levels recorded in the augers and /or casing in the open boreholes where artesian conditions were not encountered typically range from about 0.3 m to 2.2 m below the existing ground surface/Highway 401 grade, corresponding to about Elevation 164 m and 164.4 m, respectively. In addition, it was noted during the course of the fieldwork that the ground surface in the area of the as-drilled abandoned Borehole FC-1 (observed a day after abandoning the borehole), advanced near the proposed east abutment foundation footprint exhibited signs of groundwater seepage. Although, groundwater was not observed emanating from the abandoned borehole, groundwater was observed to be seeping from localized areas adjacent to the borehole. The seepage rates were periodically monitored and appeared to dissipate over time. No erosion or visual signs of instability were observed throughout the duration of the fieldwork.

Based on the water level measurements and visual observations of soil colour/moisture changes, the estimated shallow groundwater level is at about Elevation 164.4 m, corresponding to the existing ground surface and approximate water level at Fletcher's Creek.

Due to the proximity of the abutments to the edge of the Fletcher's Creek, a groundwater cut-off (cofferdam or similar measure) is recommended to minimize dewatering requirements and potential environmental impacts for excavation of the pile caps or other foundation options such as spread footings. A cut-off/cofferdam could consist of interlocking steel sheetpiles driven below the proposed base of excavation. In addition, measures would be required to control groundwater seepage and prevent loss of soil through the "gaps" that may exist at the base of some of the sheetpile sections during excavation.

If spread footings or sub-excavation and replacement options (i.e. removal of the clayey silt deposit) down to the top of the cohesionless till (approximately Elevation 158 m, and about 6 m below the creek level) are considered, a well point dewatering system (would be required to allow placement of concrete or compaction of engineered fill in the dry. Considering high artesian pressures (up to Elevation 172 m) were encountered in the cohesionless till soils, dewatering may not be feasible for the deep sub-excavations.

Caissons constructed with temporary or permanent liners within the cohesionless till subjected to unbalanced hydrostatic head at this site will require special measures to prevent 'boiling' or basal heave of the base materials. If caisson foundations are adopted, it is recommended that a constant head of water or drilling mud be maintained inside the caisson liners to counterbalance the natural groundwater or artesian conditions. Concrete placement by tremie methods would be required.

Due to the artesian conditions, driven steel H-pile/tube pile or caisson installations within the cohesionless till for both Option A and Option B will require that a sand filter, in combination with a geotextile, be placed beneath the pile caps to prevent the migration of fines that may be transported along the piles or caisson liner during and after construction. The filter/drainage blanket should consist of a minimum 0.5 m thick layer of concrete fine aggregate meeting the gradation requirements of OPSS 1002 (*Aggregates – Concrete*). The concrete fine aggregate should extend a minimum of 0.5 m horizontally beyond each of the piles. Appropriate drainage from under the pile cap should be provided for the granular blanket by using a 100mm perforated subdrain as per OPSS 405 (*Pipe Subdrains*) wrapped in knitted sock geotextile and draining to a temporary ditch or sump during construction. The geotextile should consist of non-woven, Class 1 geotextile with filtration opening size (FOS) of 75 µm to 115 µm in accordance with OPSS 1860. Further, the excavation at the front of the abutment (towards the river) should be backfilled with free-draining material extending at least 0.5 m horizontally from the front face of the abutment.

Based on the GA drawing provided, the approximate creek level is above the proposed underside of the abutment wall/pile cap. In order to allow proper drainage of the groundwater that may flow upward from beneath the pile cap through the sand drainage layer after construction of the foundation elements, Granular 'A' or 'B' Type II soil should be placed over the sand drainage blanket up to the original ground surface (above the creek water level). Rip-Rap or river stone should be placed over the full extent of the granular material to prevent scouring and erosion of fine soil particles, as specified by the Hydraulic Engineer. A typical illustration of the drainage blanket is depicted on Figure 5.

In addition, as a result of groundwater noted to be seeping from localized areas to the ground surface and the environmental sen

sitive nature of this site, an environmental permit may be required prior to discharging drained water to the creek.

It is recommended that an NSSP be included in the Contract Documents to warn the contractor of the artesian groundwater levels during foundation construction; an example NSSP is presented in Appendix C.

6.7.3.1 Permit to Take Water

A drawdown/seepage analysis has been carried out to estimate the volume(s) of groundwater flow that will have to be pumped at the east and west abutment locations of both the north and south bridges in order to lower and maintain the groundwater level below the base of the excavations during spread footing or pile cap construction for the abutments. Based on an assumedhydraulic conductivity (k) of 5 x10^{-7} m/s for the silty sand to sand and gravel till at and below the base of the proposed excavated areas, , groundwater pumping volumes of about 80 m^3/day would be required to facilitate the construction of each pile cap. The actual pumping volumes could increase depending on the weather (i.e. precipitation) conditions, the time of construction (i.e. snow melt) and the construction methodology employed by the Contractor.

The Ontario Ministry of Environment (MOE) requires a Permit To Take Water (PTTW) for any groundwater pumping in excess of 50 m^3/day. Based on the result of the drawdown/seepage analysis, the proximity to Fletcher's Creek and the high groundwater pressure observed locally, it is expected that an Ontario Ministry of Environment Permit To Take Water will be required to support the construction of the bridges at this site.

6.7.4 Subgrade Protection

The native soils that will be exposed at the foundation subgrade level will be susceptible to disturbance from construction traffic, groundwater infiltration and/or ponded water. To limit this degradation, it is recommended that a concrete working slab be placed on the subgrade immediately after preparation, inspection and approval of the footing subgrade. This requirement can be addressed with a note on the drawings and/or with an NSSP. An example NSSP for the concrete working slab is included in Appendix C.

6.7.5 Obstructions During Pile Driving / Caisson Installation

Cobbles and/or boulders were inferred due to difficulty to advance the augers/auger grinding at varying depths (typically below Elevation 157 m) in the boreholes drilled during the current subsurface investigation, which may affect the installation of steel H-piles/tube piles or caissons. It is recommended that driving shoes be used on all steel H-piles or tube piles to facilitate driving into the overburden soils. In addition it is recommended that an

NSSP be included in the Contract Documents to warn the Contractor of the possible presence of cobbles and/or boulders within the overburden soils and an example NSSP is presented in Appendix C.

6.7.6 Vibration Monitoring During Construction

A maximum peak particle velocity (PPV) of 100 mm/s is generally considered applicable for bridge structures in good condition. Based on vibration monitoring experience, it is considered unlikely that vibrations induced by conventional construction activities such as pile driving, coring/churn drilling, or hoe-ramming will reach this threshold level and, therefore, vibration monitoring of the existing structure is not expected to be required during construction at this site. As there are several residential and commercial structures in the vicinity of the site, monitoring of vibrations during construction should be considered by the general contractor to defend against potential damage claims by the owners of the nearby structures.

6.7.7 Erosion / Scour Protection

The existing soils near the east and west abutments may be susceptible to erosion and scour under the design creek flow velocities and given the presence of springs/artesian groundwater conditions. The requirements for design of erosion/scour protection should be assessed by the hydraulic design engineer. As a minimum, it is recommended that erosion protection (e.g. rip-rap or granular sheeting) be provided on the creek banks and adjacent to the abutments (i.e. on top of the sand filter layer recommended in Section 6.2.2) to protect the foundations/pile caps from being exposed. The rip-rap should be consistent with the standard R-10 classification or granular sheeting classification in accordance with OPSS 1004 (Aggregates) but should be approved by the hydraulic design engineer.

6.7.8 Construction Access

Trafficability of construction equipment may be problematic near the creek as a result of soft/loose soils present near the surface and evidence of springs (i.e. groundwater seepage at the ground surface due to artesian pressures) in the general vicinity of the proposed abutments. Drainage in this area is likely to be poor and the groundwater levels/creek water level will vary and are subject to seasonal fluctuations. The contractor must be prepared to supply equipment capable of working on this terrain and/or provide alternative measures to improve trafficability such as placement of granular pads in working areas.

Fletcher’s Creek has been identified as an environmentally sensitive area by the Ministry of Natural Resources (MNR), as the creek contains Redside Dace and Jefferson Salamander, both of which are endangered species protected under the *Endangered Species Act, 2007 (ESA 2007)*, as detailed in the Letter of Advice (LOA), dated April 11, 2012, issued by the MNR for the fieldwork. Potential environmental impacts will need to be minimized during construction access in this sensitive area. Specific access preparation procedures such as the use of temporary work bridges, winter construction and/or gravel roadways underlain by geosythetics should be considered to accommodate foundation construction at the proposed North and South Bridge locations. Further, sediment control measures such as silt fences, straw bales and/or granular check-dams will need to be installed downgradient of the works to reduce sediment impacts to the creek, consistent with OPSS 577, Temporary Erosion and Sediment Control Measures.

7.0 CLOSURE

This Foundation Design Report was prepared by Ms. T. Veronica Ayetan, P.Eng., a geotechnical engineer with Golder , and reviewed by Mr. Kevin Bentley, P.Eng., a senior geotechnical engineer and Associate with Golder. Mr. Ty Garde, P.Eng., a Designated MTO Contact and Principal with Golder, conducted an independent review and quality control audit of this report.

GOLDER ASSOCIATES LTD.

LICENSED PROFESSIONAL ENGINEER
Mar 13/13
T.V. AYETAN
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PROVINCE OF ONTARIO

T. Veronica Ayetan, P.Eng.
Geotechnical Engineer

Kevin Bentley, P.Eng.
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Mar 13/13
T. J. GARDE
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Ty Garde, P.Eng
Designated MTO Contact, Principal

TVA/KJB/TG/tva/jl

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Ministry of Transportation Engineering Standards Branch. RSS Design Guidelines. September 2008

Ontario Provincial Standard Specifications (OPSS)

OPSS 405	Construction Specifications for Pipe Subdrains
OPSS 501	Construction Specifications for Compacting
OPSS 539	Construction Specification for Temporary Protection Systems
OPSS 577	Temporary Erosion and Sediment Control Measures
OPSS 802	Construction Specification for Topsoil
OPSS 803	Construction Specification for Sodding
OPSS 804	Construction Specification for Seed and Cover
OPSS 902	Construction Specification for Excavating and Backfilling Structures
OPSS 903	Construction Specification for Deep Foundations
OPSS 1002	Material Specification for Aggregates – Concrete
OPSS 1004	Material Specification for Aggregates – Miscellaneous
OPSS 1860	Material Specification for Geotextiles

Ontario Provincial Standard Drawings (OPSD)

OPSD 208.010	Benching of Earth Slopes
OPSD 3000.100	Foundation, Piles, Steel H-Pile Driving Shoe
OPSD 3001.100	Foundation, Piles, Steel Tube Pile Driving Shoe
OPSD 3090.101	Foundation Frost Depths for Southern Ontario
OPSD 3101.150	Walls, Abutments, Backfill, Minimum Granular Requirements
OPSD 3121.150	Walls, Retaining, Backfill, Minimum Granular Requirements

Construction Design Estimating and Documentation (CDED) Special Provisions (SP)

SP 110S13	Material Specification for Aggregates – Base, Subbase, Select Subgrade and Backfill Material
SP 206S03	Earth Excavation and Grading; Excavation for Pavement Widening
SP 105S21	Amendment to OPSS 501
SP 599S22	Retained Soil System, Wall/Slope, High Performance

ASTM International

ASTM D1586	Standard Test Method for Standard Penetration Test (SPT) and Split Barrel Sampling of Soils

TABLE 1 - COMPARISON OF FOUNDATION ALTERNATIVES FLETCHER'S CREEK BRIDGES, HIGHWAY 401 WIDENING G.W.P. 2150-01-00

Foundation Option	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Strip or Spread Footing on very dense cohesionless till (or founded on engineered fill placed on dense to very dense cohesionless till).	• Allows for semi-integral abutments.	• Up to between about 6 m to 11 m depth of excavation required at the North Structure, and up to about 6 m depth of excavation required at the South Bridge structure; extensive dewatering and temporary shoring efforts required; • Artesian groundwater condition encountered with hydrostatic head at about 5 m high above the ground surface, extensive groundwater control/dewatering required to minimize disturbance to founding soils and facilitate excavation (and replacement with engineered fill, if applicable) and placement of concrete in the dry; • Traffic protection system required during construction; • Increased environmental impact on designated environmentally sensitive area; and • Does not allow for integral abutment construction.	• Excavation (of existing embankment fill and native soil) and groundwater control costs would be significantly higher than the cost for pile cap construction; • North Structure - (4 m wide x 69 m long x 1.2 m thick at $ 600/m^3) + excavation costs (6 m deep x 18 m wide x 69 m long at $ 10/m^3) = $ 273,240/foundation. • South Structure - (4 m wide x 20 m long x 1.2 m thick at $ 600/m^3) + (6m deep x 18 m wide x 20 m long at $ 10/m^3) = $79,200/foundation.	• Potential significant traffic disruption during construction; • Potential for difficulties in achieving adequate groundwater control/dewatering requirements; • Potential for loosening of the founding soils due to artesian pressures if groundwater control measures are inadequate; • Trafficability of construction equipment may be problematic near the creek due to presence of soft/loose soils and "quick" sand; • High environmental impact due to large construction/excavation footprint; and • Creates a direct pathway between cohesionless till (i.e. artesian conditions) and ground surface which may result in significant upward groundwater seepage at the site.
Steel H-Piles driven to found within dense to very dense cohesionless till.	• Negligible post-construction settlement; and • Allow for semi-integral or integral abutments; • Allows for cantilevered retaining walls; and • Sub-excavation depth for pile cap is much shallower compared to spread footing option (i.e. less dewatering and temporary shoring efforts).	• Traffic protection system required during construction; • Difficult driving likely to be encountered through the cohesionless till containing cobbles/boulders; and • If CSPs are required for integral abutment design, difficulties sub-excavating below groundwater level may be encountered.	• Higher cost than shallow foundations; however total construction costs may be comparable or less than shallow footings considering extra costs for dewatering and temporary shoring for shallow foundation option. • North Structure - Assume (70 piles x 10 m long at $ 250/m) plus excavation and pile cap costs of (1.5 m wide x 69 m long x 3 m thick at $ 600/m^3) + (3m deep x 4.5 m wide x 41 m long at $ 10/m^3) = $ 367,000/abutment. • South Structure - Assume (20 piles x 10 m long at $ 250/m) plus excavation and pile cap costs of (1.5 m wide x 20 m long x 3 m thick at $ 600/m^3) + (3m deep x 4.5 m wide x 20 m long at $ 10/m^3) = $ 107,000/abutment.	• Potential traffic disruption during construction; • Risk of encountering obstructions that could complicate pile installation or to be deflected away from vertical alignment during driving; • Due to artesian condition encountered on site, sand drainage/filter in combination with a geotextile will need to be placed beneath the pile caps to minimize the migration of fines along the pile during construction; • Risk of driving beyond the design pile tip elevations, or "hanging-up" above pile tip elevations on gravelly soils containing cobbles; • Trafficability of construction equipment may be problematic near the creek due to presence of soft/loose soils and "quick" sand; and • Potentially less costly maintenance over life of the structure than semi-integral abutment structures.

Foundation Option	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Steel Tube Piles (closed-end, concrete filled) driven to found within cohesionless till.	• Negligible post-construction settlement; and • Can be used for support of semi-integral or integral abutments; provided the pile size can accommodate the lateral flexibility required for such abutment design; • Allows for cantilevered retaining walls; and • Sub-excavation depth for pile cap is much shallower compared to spread footing option (i.e. less dewatering and temporary shoring efforts).	• Traffic protection system required during construction; • Difficult driving likely encountered through the cohesionless till containing cobbles/boulders; • Greater disturbance to immediately adjacent ground due to larger base area if end is closed; • Requires staged construction for driving and concrete filling of tube; and • MTO does not allow the use of pipe piles for integral abutment design.	• Higher total cost than shallow foundations; however total construction costs may be comparable or less than shallow footings considering extra costs for dewatering and temporary shoring for shallow foundation option. • Cost for steel tube (pipe) piles may be slightly higher than for steel H-piles; and • North Structure - Assume same cost as steel H-piles = $ 367,000/abutment. • South Structure - Assume same cost as steel H-piles = $ 107,000/abutment.	• Potential traffic disruption during construction; • If obstructions (cobbles and/or boulders) are encountered during driving; piles may “hang up” at a higher elevation; • Due to artesian condition encountered on site, sand drainage/filter in combination with a geotextile will need to be placed beneath the pile caps to minimize the migration of fines along the pile during construction; • Risk of driving beyond the design pile tip elevations; and • Trafficability of construction equipment may be problematic near the creek due to presence of soft/loose soils and “quick” sand.
Caissons founded within dense to very dense cohesionless till	• Higher geotechnical resistances per unit compared to spread footings and piles; so reduced number of deep foundation elements compared to steel H- or tube piles; and • Negligible post-construction settlement.	• Temporary or permanent liners required through the sand and silt, silty sand to gravelly sand till, and water bearing cohesionless seams/layers; • Caisson will be extended below the groundwater level, and into artesian conditions; therefore specialized techniques to balance high water pressure is required. • Cleaning and verification of the base will be difficult; • Requirement for placement of concrete by tremie method; • Traffic protection system required during construction; • Not suitable for integral abutment design; and • Greater risk of encountering obstructions due to larger size of drill hole.	• Higher cost than steel H-piles and tube piles; and • Installation cost will be impacted by need for liner to minimize disturbance and loss of ground and for tremie concrete placement. • North Structure - Assume (30 caissons x 10 m long at $ 2,000/m) = $ 600,000 per abutment. • South Structure - Assume (10 caissons x 10 m long at $ 2,000/m) = $ 200,000 per abutment.	• Risk of disturbance of caisson base in cohesionless till due to artesian groundwater conditions; • Special construction procedures including use of temporary or permanent liners and possibly effective dewatering; • Difficulty may be encountered in drilling and extending liner through the cobbles and boulders; • Significant traffic disruption during construction due to space required for caisson drilling equipment; • Risk of encountering obstructions that could impact caisson installation/costs; • Trafficability of construction equipment may be problematic near the creek due to presence of soft/loose soils and “quick” sand; and • Increased risk of environmental impact compared to piles.

Prepared By: TVA

Reviewed By: KJB/TG

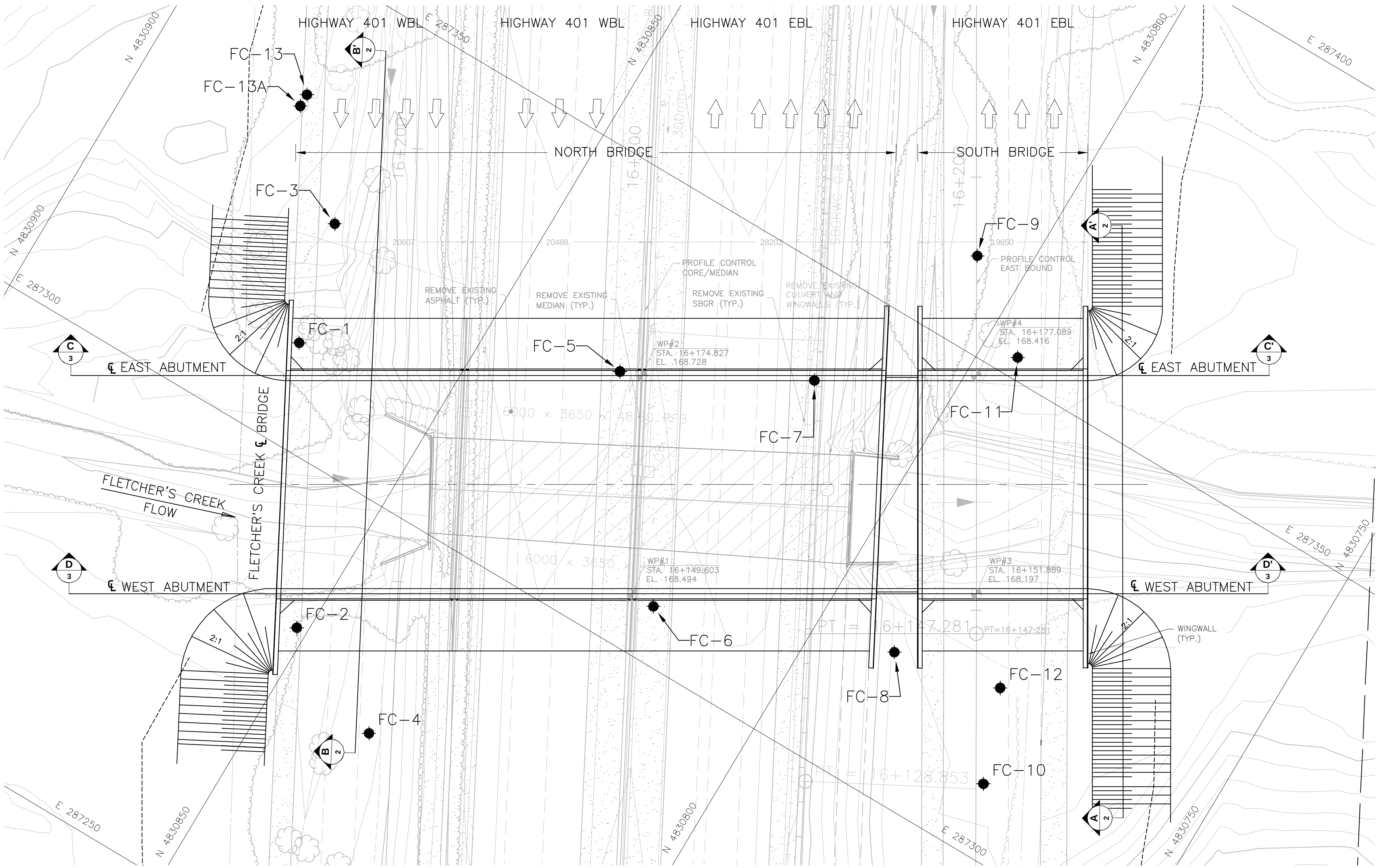
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

CONT No.
GWP No. 2150-01-00

SHEET

HIGHWAY 401
FLETCHER'S CREEK BRIDGES
BOREHOLE LOCATIONS

Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



PLAN

SCALE
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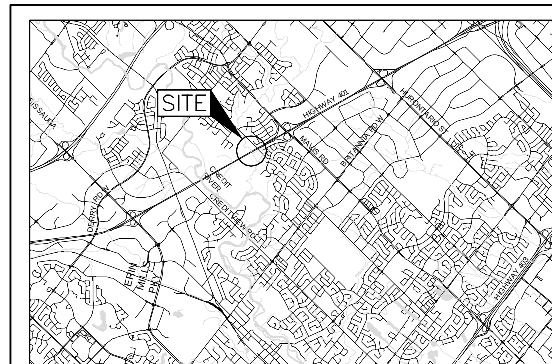


KEY PLAN
SCALE
1.5 0 1.5 3 km

LEGEND

● Borehole – Current Investigation

No.	ELEVATION	NORTHING	EASTING
FC-1	163.7	4830867.9	287311.4
FC-2	163.8	4830851.3	287283.0
FC-3	165.9	4830871.4	287325.3
FC-4	164.4	4830837.9	287276.8
FC-5	168.6	4830834.4	287327.5
FC-6	168.3	4830817.2	287306.2
FC-7	168.5	4830814.6	287338.1
FC-8	164.0	4830790.6	287315.9
FC-9	164.3	4830805.8	287360.1
FC-10	164.1	4830774.0	287308.1
FC-11	164.4	4830795.8	287352.4
FC-12	163.9	4830778.0	287318.6
FC-13	167.1	4830881.8	287336.5
FC-13A	167.1	4830881.8	287335.0

NOTES

This drawing is for subsurface information only. The proposed structure details/works are shown for illustration purposes only and may not be consistent with the final design configuration as shown elsewhere in the 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 plans provided in digital format by Aecom, drawing file nos. "60213979_ST-24-0129-C-1 GA_30%.dwg", received September 25, 2012 and "X-60213979-C-CTR-HWY401_HALFm.dwg" received September 27, 2012.

NO.	DATE	BY	REVISION

Geocres No. 30M12-356

HWY. 401		PROJECT NO. 10-1111-0211	DIST.
SUBM'D. TVA	CHKD. TVA	DATE: Feb-2013	SITE:
DRAWN: DD	CHKD. KJB	APPD. TG	DWG. 1

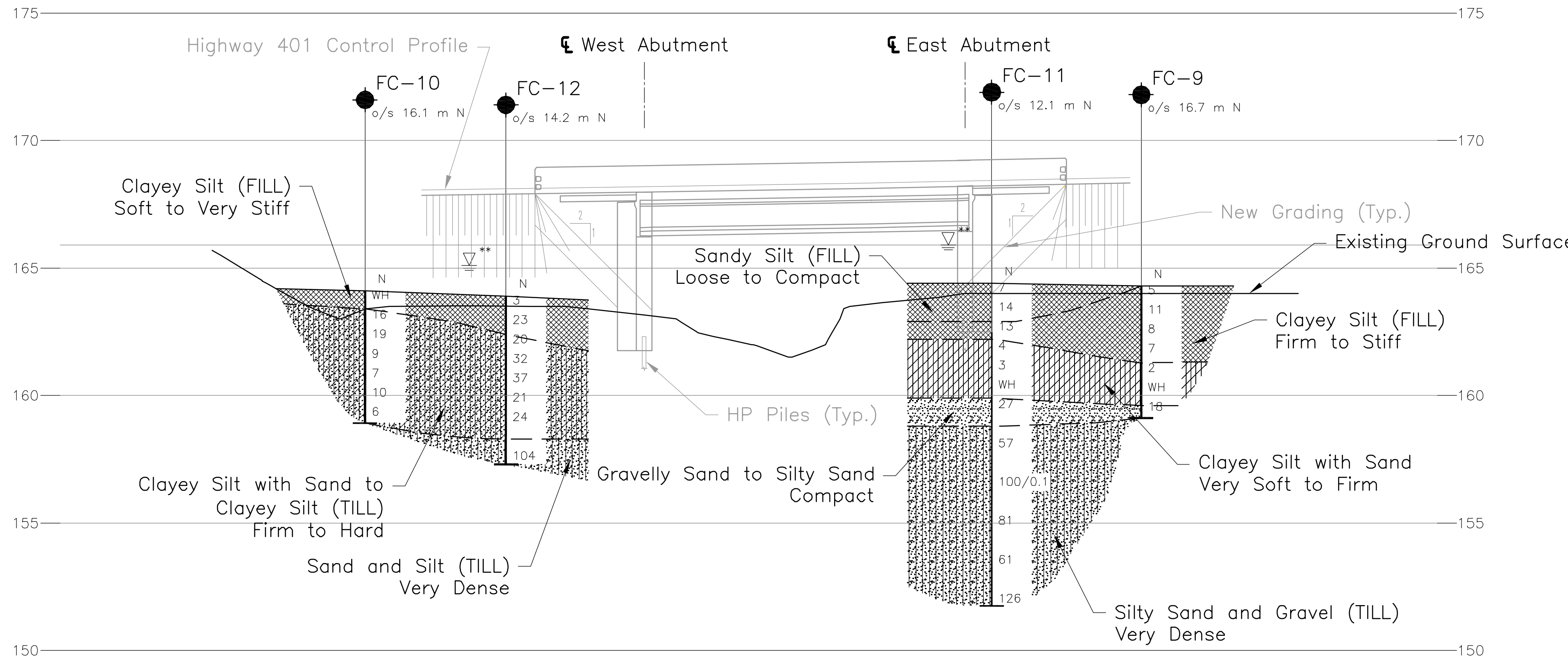
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

CONT No.
GWP No. 2150-01-00

HIGHWAY 401
FLETCHER'S CREEK BRIDGES
SOIL STRATA

SHEET

Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



A-A' 1
SOUTH PROFILE
SCALE
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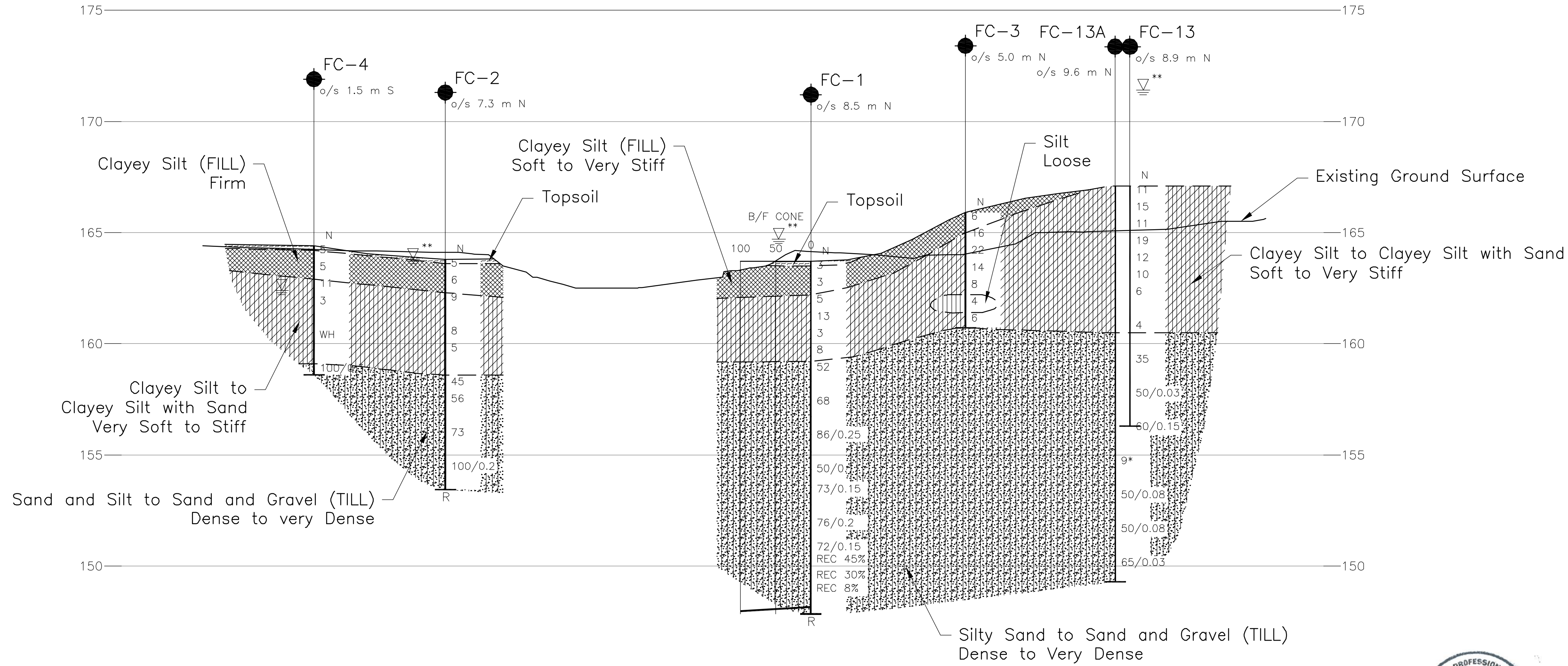
VER. 2.5 0 2.5 5 m



B-B' 1
NORTH PROFILE
SCALE
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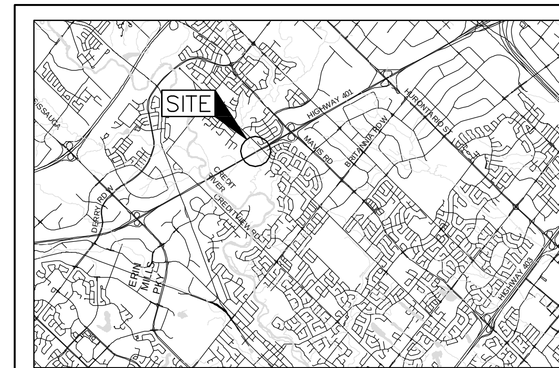
VER. 2.5 0 2.5 5 m



KEY PLAN
SCALE
1.5 0 1.5 3 km

LEGEND

- ● Borehole – Current Investigation
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- ∇** Artesian WL conditions encountered upon completion of drilling
- ∇ WL upon completion of drilling
- R Refusal

No.	ELEVATION	NORTHING	EASTING
FC-1	163.7	4830867.9	287311.4
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REGISTERED PROFESSIONAL ENGINEER T. GARDE Feb. 7, 2013 PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER T.V. AYETAN 100130071 Feb. 7, 2013 PROVINCE OF ONTARIO

NO.	DATE	BY	REVISION

Geocres No. 30M12-356

HWY. 401	PROJECT NO. 10-1111-0211		DIST.
SUBM'D. TVA	CHKD. TVA	DATE: Feb-2013	SITE:
DRAWN: DD	CHKD. KJB	APPD. TG	DWG. 2

MINISTRY OF TRANSPORTATION, ONTARIO

PLOT DATE: February 7, 2013

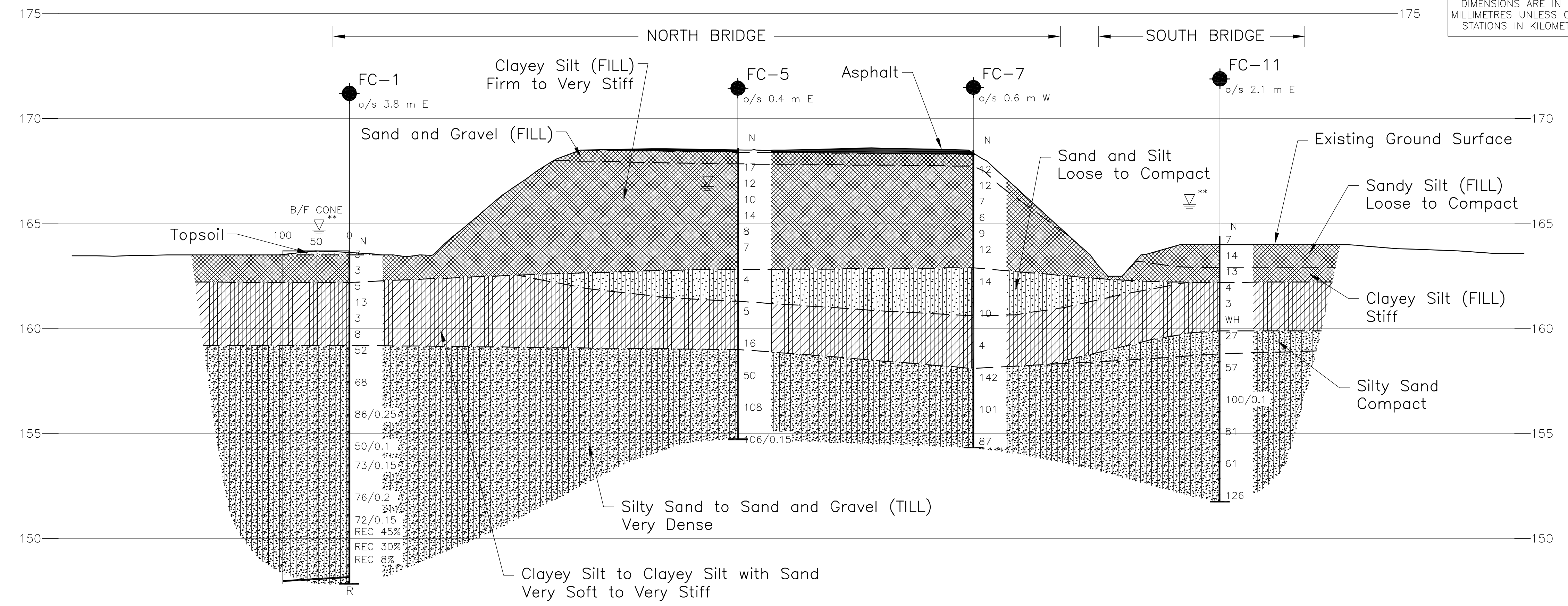
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

CONT No.
GWP No. 2150-01-00

SHEET

HIGHWAY 401
FLETCHER'S CREEK BRIDGES
SOIL STRATA

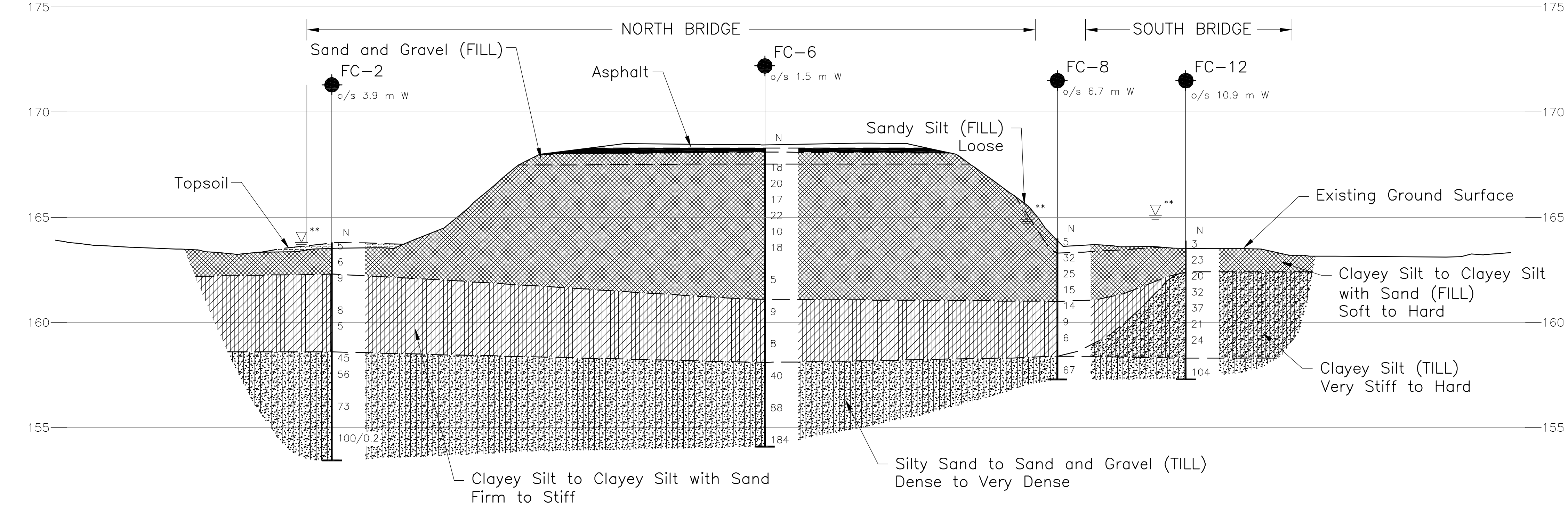
Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



C-C' 1

EAST ABUTMENT

SCALE
HOR. 5 0 5 10 m
VER. 2.5 0 2.5 5 m



D-D' 1

WEST ABUTMENT

SCALE
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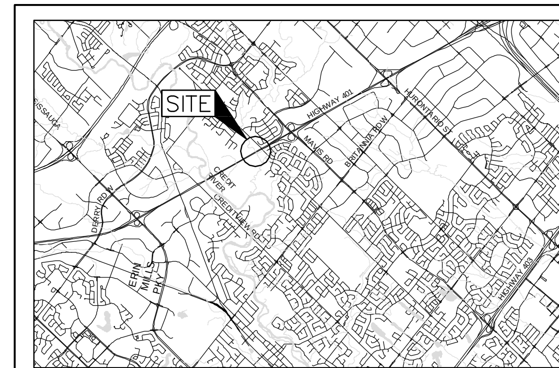
VER. 2.5 0 2.5 5 m



KEY PLAN
SCALE
1.5 0 1.5 3 km

LEGEND

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- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
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FC-7	168.5	4830814.6	287338.1
FC-8	164.0	4830790.6	287315.9
FC-11	164.4	4830795.8	287352.4
FC-12	163.9	4830778.0	287318.6

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

Existing Ground Surface Cut obtained from digital files provided by Aecom, (Drawing File "X-60213979-C-CTR-HWY401_HALFm.dwg") received September 27, 2012.

REGISTERED PROFESSIONAL ENGINEER T. GARDE Feb. 7, 2013 PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER T.V. AYETAN 100130071 Feb. 7, 2013 PROVINCE OF ONTARIO

NO.	DATE	BY	REVISION

Geocres No. 30M12-356

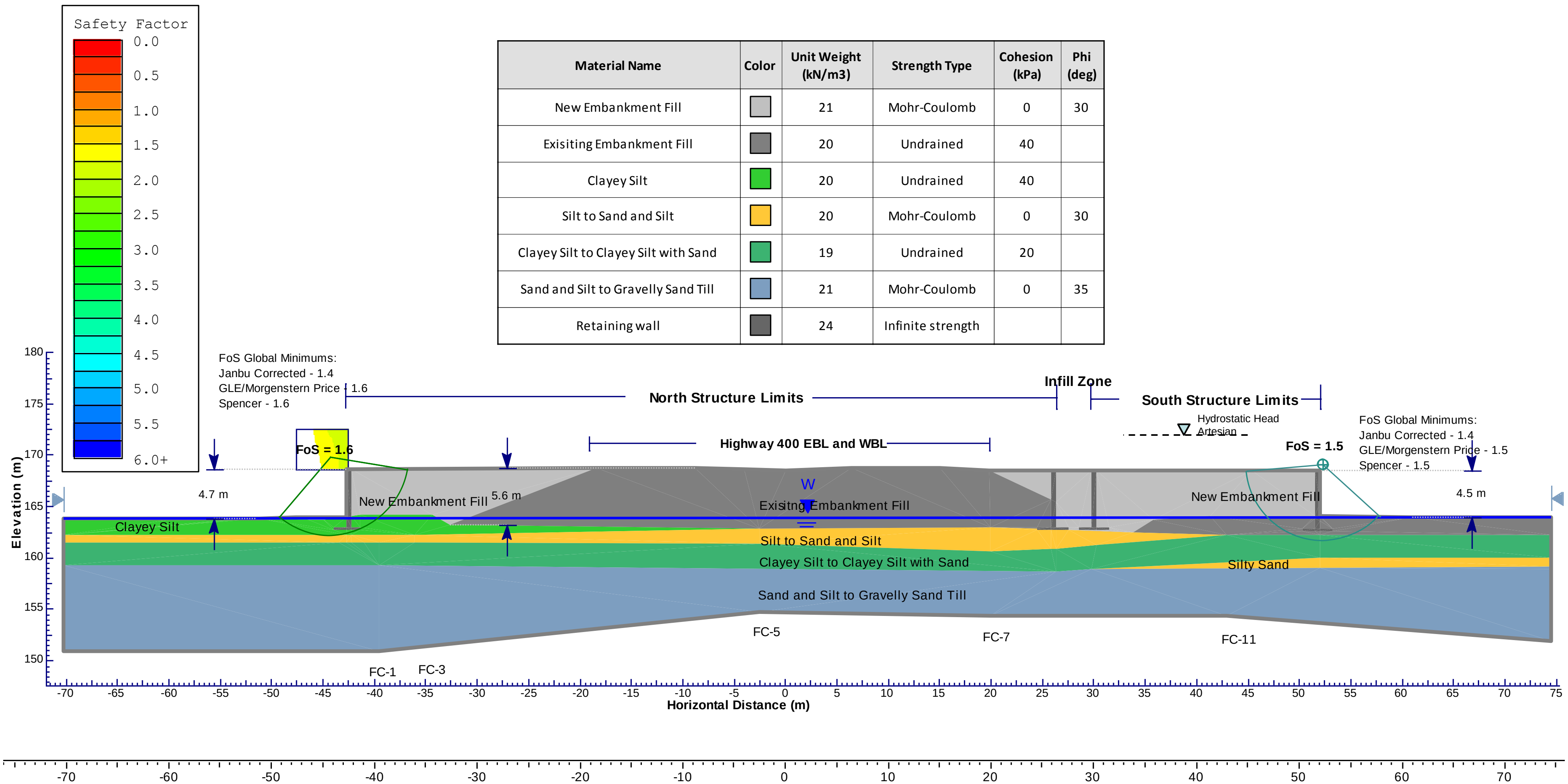
HWY. 401		PROJECT NO. 10-1111-0211	DIST.
SUBM'D. TVA	CHKD. TVA	DATE: Feb-2013	SITE:
DRAWN: DD	CHKD. KJB	APPD. TG	DWG. 3

PLOT DATE: February 7, 2013

Retaining Wall (East Side Profile) – Static Global Stability Analysis
Hwy 401 Widening – Fletcher's Creek Bridges

Figure 1

Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)
New Embankment Fill		21	Mohr-Coulomb	0	30
Exisiting Embankment Fill		20	Undrained	40	
Clayey Silt		20	Undrained	40	
Silt to Sand and Silt		20	Mohr-Coulomb	0	30
Clayey Silt to Clayey Silt with Sand		19	Undrained	20	
Sand and Silt to Gravelly Sand Till		21	Mohr-Coulomb	0	35
Retaining wall		24	Infinite strength		



Date: October 2012

Project No: 10-1111-0211-02

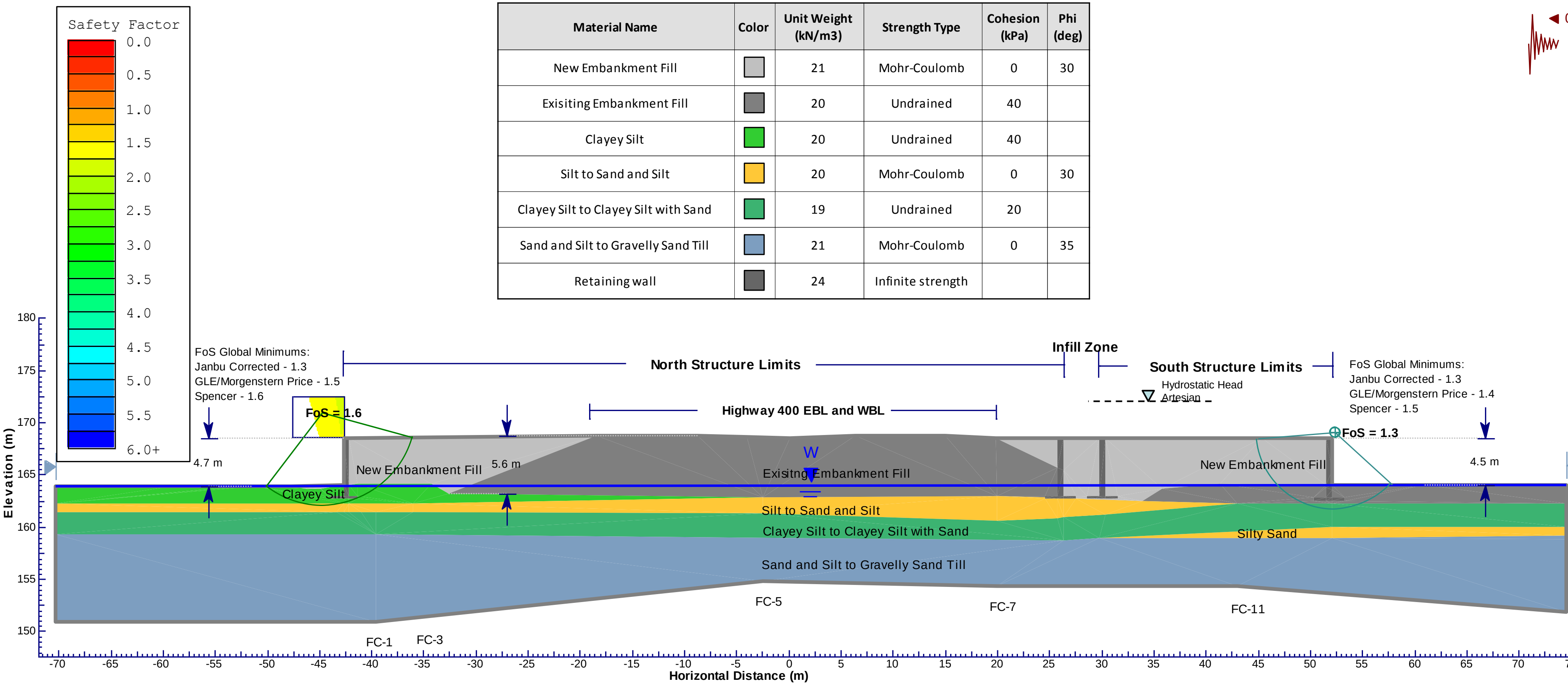
Analysis By: TVA **Reviewed By:** KJB

Golder Associates

Retaining Wall (East Side Profile) – Seismic Global Stability Analysis
Hwy 401 Widening – Fletcher's Creek Bridges

Figure 2

Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)
New Embankment Fill		21	Mohr-Coulomb	0	30
Exisiting Embankment Fill		20	Undrained	40	
Clayey Silt		20	Undrained	40	
Silt to Sand and Silt		20	Mohr-Coulomb	0	30
Clayey Silt to Clayey Silt with Sand		19	Undrained	20	
Sand and Silt to Gravelly Sand Till		21	Mohr-Coulomb	0	35
Retaining wall		24	Infinite strength		



Date: October 2012
Project No: 10-1111-0211-02

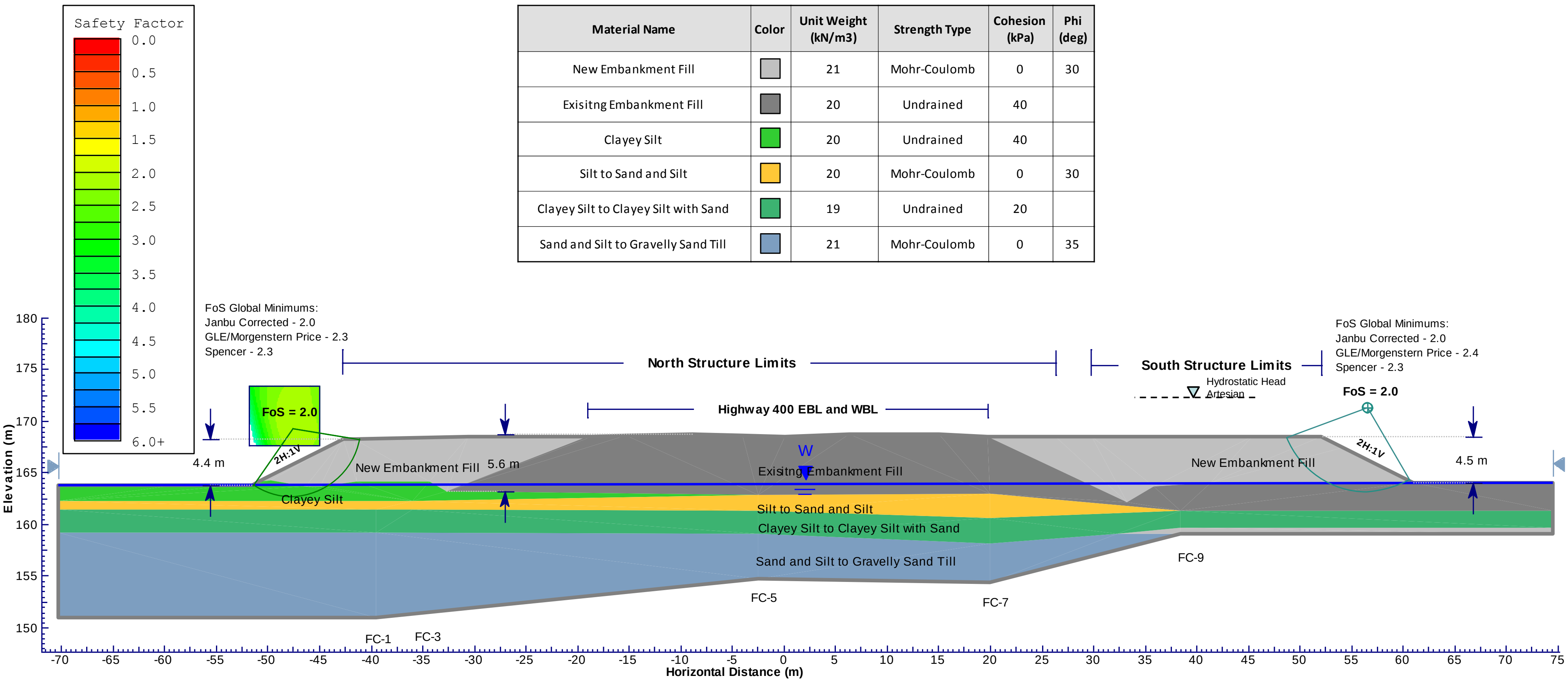
Analysis By: TVA Reviewed By: KJB

Golder Associates

East Approach Embankment – Static Global Stability Analysis
Hwy 401 Widening – Fletcher's Creek Bridges

Figure 3

Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)
New Embankment Fill		21	Mohr-Coulomb	0	30
Exisitng Embankment Fill		20	Undrained	40	
Clayey Silt		20	Undrained	40	
Silt to Sand and Silt		20	Mohr-Coulomb	0	30
Clayey Silt to Clayey Silt with Sand		19	Undrained	20	
Sand and Silt to Gravelly Sand Till		21	Mohr-Coulomb	0	35



Date: October 2012
Project No: 10-1111-0211-02

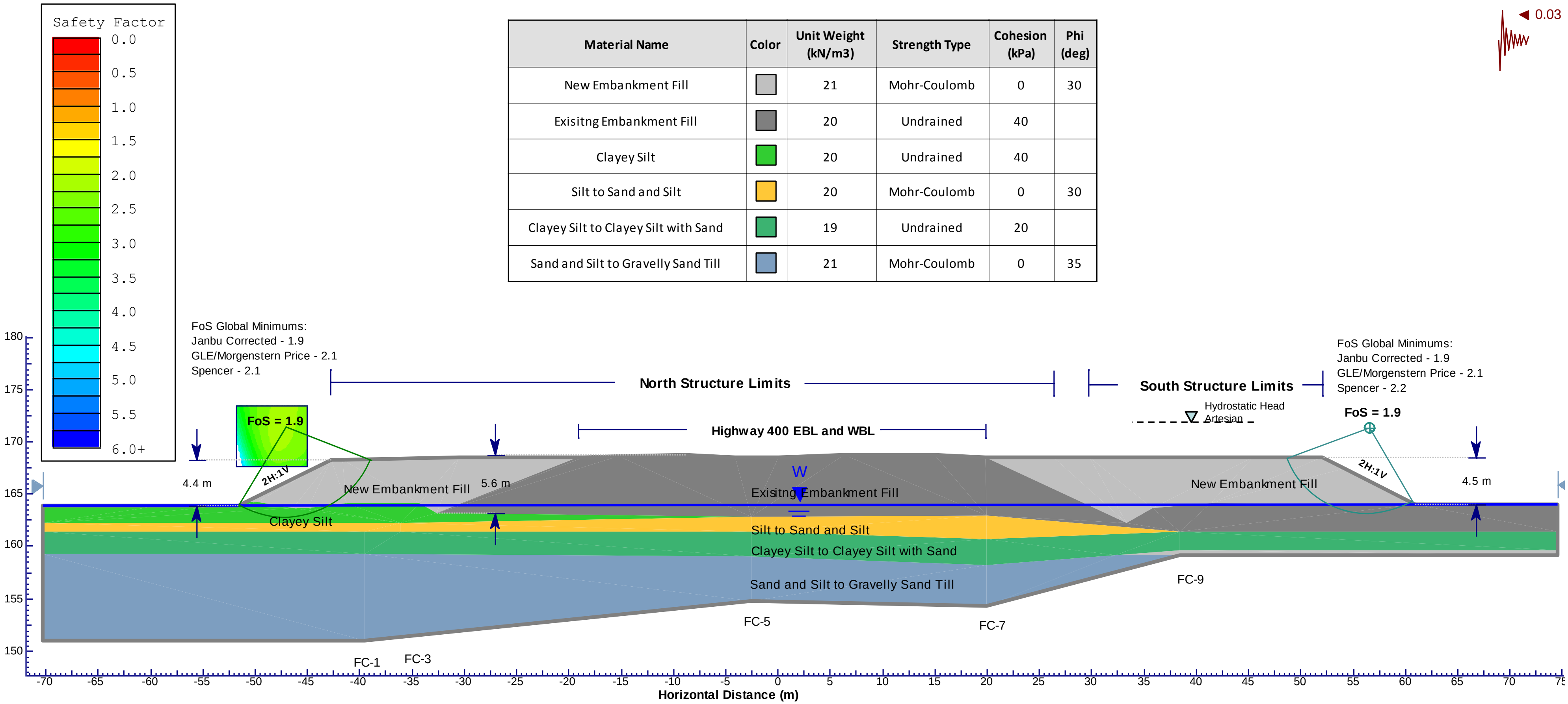
Analysis By: TVA **Reviewed By:** KJB

Golder Associates

East Approach Embankment – Seismic Global Stability Analysis
Hwy 401 Widening – Fletcher's Creek Bridges

Figure 4

Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)
New Embankment Fill		21	Mohr-Coulomb	0	30
Exisitng Embankment Fill		20	Undrained	40	
Clayey Silt		20	Undrained	40	
Silt to Sand and Silt		20	Mohr-Coulomb	0	30
Clayey Silt to Clayey Silt with Sand		19	Undrained	20	
Sand and Silt to Gravelly Sand Till		21	Mohr-Coulomb	0	35

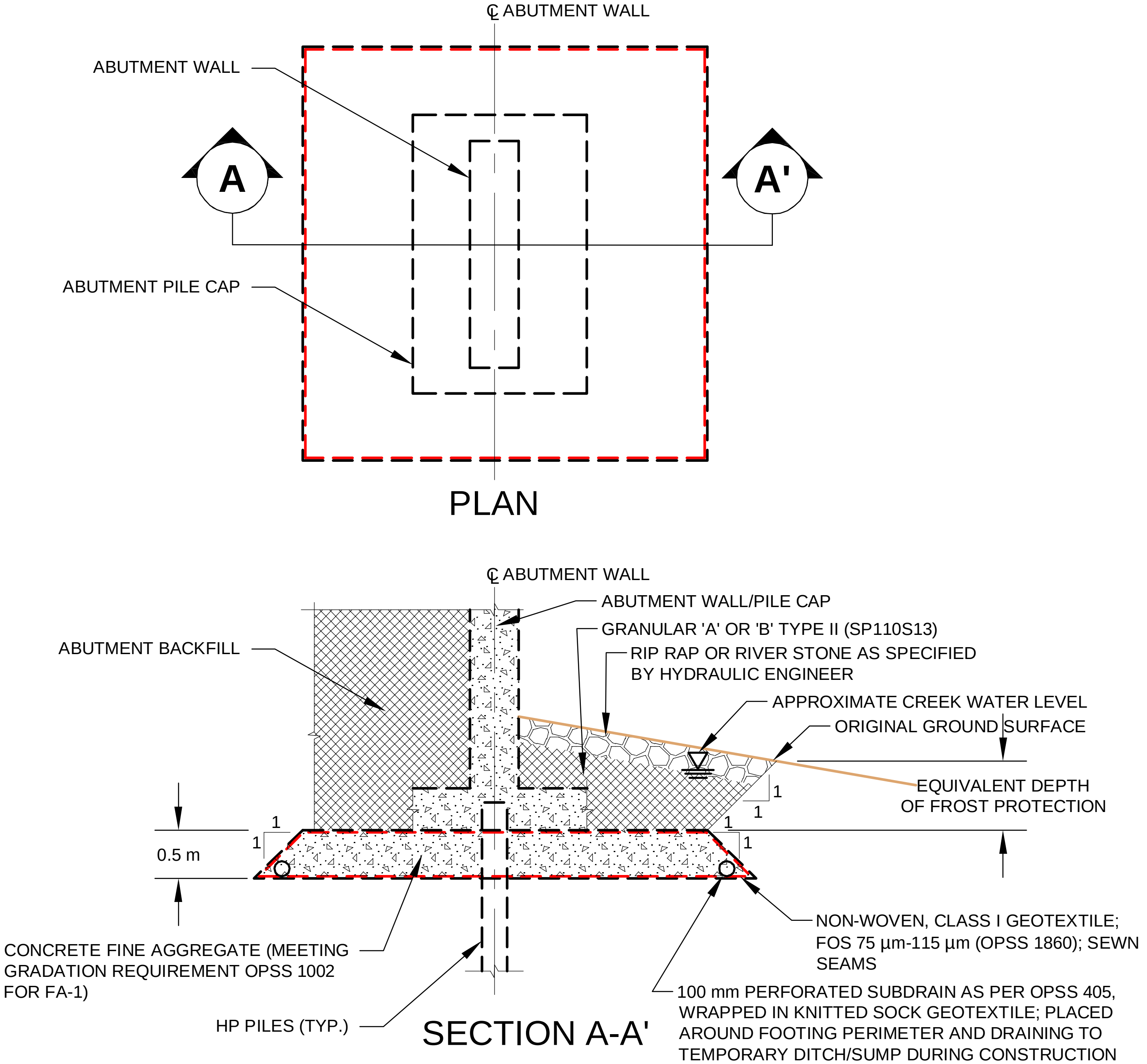


Date: October 2012

Project No: 10-1111-0211-02

Analysis By: TVA **Reviewed By:** KJB

Golder Associates



NOTES:

1. THE DRAINAGE BLANKETS SHOULD BE IN PLACE PRIOR TO PILE DRIVING.
2. THE GEOTEXTILE SHOULD BE CUT WITH A 300 mm X 300 mm "X" AT LOCATIONS WHERE PILE WILL PENETRATE IT.
3. IF BLANKET IS DISTURBED DURING PILE DRIVING, THE BLANKET SHOULD BE RESTORED TO THE DETAILS SHOWN ON THIS FIGURE AFTER THE COMPLETION OF THE PILE DRIVING.
4. DRAINAGE BLANKET SHOULD EXTEND A MIN. 0.5 m HORIZONTALLY BEYOND EACH OF THE PILES.

NOT TO SCALE

PROJECT
HWY 401 - FLETCHER'S CREEK BRIDGES
CITY OF MISSISSAUGA, REGION OF PEEL
MTO, G.W.P 2150-01-00

TITLE
DRAINAGE BLANKET DETAILS

Golder Associates
Mississauga, Ontario, Canada

PROJECT No.		10-1111-0211	FILE No. 1011110211BC005.dwg	
DESIGN			SCALE AS SHOWN	REV. A
CAD	JFC	Dec. 14, 2012	FIGURE No.	
CHECK	TVA	Dec. 14, 2012		
REVIEW	KJB/JMAC	Dec. 14, 2012	5	

PLOT DATE: February 7, 2013
FILENAME: T:\Projects\2010\10-1111-0211 (AECOM, Mississauga)\-BC- Fletcher's Creek GA\1011110211BC005.dwg

APPENDIX A

Record of Borehole Sheets

LIST OF ABBREVIATIONS

The abbreviations commonly employed on Records of Boreholes, on figures and in the text of the report are as follows:

I. SAMPLE TYPE

AS	Auger sample
BS	Block sample
CS	Chunk sample
SS	Split-spoon
DS	Denison type sample
FS	Foil sample
RC	Rock core
SC	Soil core
ST	Slotted tube
TO	Thin-walled, open
TP	Thin-walled, piston
WS	Wash sample

II. PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:
The number of blows by a 63.5 kg. (140 lb.) hammer dropped 760 mm (30 in.) required to drive a 50 mm (2 in.) drive open sampler for a distance of 300 mm (12 in.)

Dynamic Cone Penetration Resistance; N_d:
The number of blows by a 63.5 kg (140 lb.) hammer dropped 760 mm (30 in.) to drive uncased a 50 mm (2 in.) diameter, 60º cone attached to “A” size drill rods for a distance of 300 mm (12 in.).

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

Piezo-Cone Penetration Test (CPT)
A electronic cone penetrometer with a 60° conical tip and a project end area of 10 cm^2 pushed through ground at a penetration rate of 2 cm/s. Measurements of tip resistance (Q_t), porewater pressure (PWP) and friction along a sleeve are recorded electronically at 25 mm penetration intervals.

III. SOIL DESCRIPTION

(a) Cohesionless Soils

Density Index Relative Density	**N Blows/300 mm or Blows/ft**
Very loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very dense	over 50

(b) Cohesive Soils
Consistency

	c_u, s_u kPa	c_u, s_u psf
Very soft	0 to 12	0 to 250
Soft	12 to 25	250 to 500
Firm	25 to 50	500 to 1,000
Stiff	50 to 100	1,000 to 2,000
Very stiff	100 to 200	2,000 to 4,000
Hard	over 200	over 4,000

IV. SOIL TESTS

w	water content
w_p	plastic limit
w_l	liquid limit
C	consolidation (oedometer) test
CHEM	chemical analysis (refer to text)
CID	consolidated isotropically drained triaxial test[1]
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement[1]
D_R	relative density (specific gravity, G_s)
DS	direct shear test
M	sieve analysis for particle size
MH	combined sieve and hydrometer (H) analysis
MPC	Modified Proctor compaction test
SPC	Standard Proctor compaction test
OC	organic content test
SO_4	concentration of water-soluble sulphates
UC	unconfined compression test
UU	unconsolidated undrained triaxial test
V	field vane (LV-laboratory vane test)
γ	unit weight

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

V. MINOR SOIL CONSTITUENTS

Percent by Weight	Modifier	Example
0 to 5	Trace	Trace sand
5 to 12	Trace to Some (or Little)	Trace to some sand
12 to 20	Some	Some sand
20 to 30	(ey) or (y)	Sandy
over 30	And (cohesionless) or With (cohesive)	Sand and Gravel Silty Clay with sand / Clayey Silt with sand

LIST OF SYMBOLS

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

I. GENERAL

π	3.1416
ln x,	natural logarithm of x
$\log_{10}$	x or log x, logarithm of x to base 10
g	acceleration due to gravity
t	time

II. STRESS AND STRAIN

γ	shear strain
Δ	change in, e.g. in stress: $\Delta\,\sigma$
ε	linear strain
ε_v	volumetric strain
η	coefficient of viscosity
υ	Poisson's ratio
σ	total stress
σ'	effective stress ($\sigma' = \sigma$ - u)
σ'_{vo}	initial effective overburden stress
σ_1, σ_2, σ_3	principal stress (major, intermediate, minor)
σ_{oct}	mean stress or octahedral stress $= (\sigma_1 + \sigma_2 + \sigma_3)/3$
τ	shear stress
u	porewater pressure
E	modulus of deformation
G	shear modulus of deformation
K	bulk modulus of compressibility

III. SOIL PROPERTIES

(a) Index Properties

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

(a) Index Properties (continued)

w	water content
w_l or LL	liquid limit
w_p or PL	plastic limit
I_p or PI	plasticity index = ($w_l - w_p$)
w_s	shrinkage limit
I_L	liquidity index = ($w - w_p$) / I_p
I_C	consistency index = ($w_l - w$) / I_p
e_{max}	void ratio in loosest state
e_{min}	void ratio in densest state
I_D	density index = ($e_{max} - e$) / (e_{max} - e_{min}) (formerly relative density)

(b) Hydraulic Properties

h	hydraulic head or potential
q	rate of flow
v	velocity of flow
i	hydraulic gradient
k	hydraulic conductivity (coefficient of permeability)
j	seepage force per unit volume

(c) Consolidation (one-dimensional)

C_c	compression index (normally consolidated range)
C_r	recompression index (over-consolidated range)
C_s	swelling index
C_α	secondary compression index
m_v	coefficient of volume change
c_v	coefficient of consolidation (vertical direction)
c_h	coefficient of consolidation (horizontal direction)
T_v	time factor (vertical direction)
U	degree of consolidation
σ'_p	pre-consolidation stress
OCR	over-consolidation ratio = σ'_p / σ'_{vo}

(d) Shear Strength

τ_p, τ_r	peak and residual shear strength
ϕ'	effective angle of internal friction
δ	angle of interface friction
μ	coefficient of friction = tan δ
c′	effective cohesion
c_u, s_u	undrained shear strength ($\phi = 0$ analysis)
p	mean total stress $(\sigma_1 + \sigma_3)/2$
p′	mean effective stress $(\sigma'_1 + \sigma'_3)/2$
q	$(\sigma_1 - \sigma_3)/2$ or $(\sigma'_1 - \sigma'_3)/2$
q_u	compressive strength $(\sigma_1 - \sigma_3)$
S_t	sensitivity

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

Notes:
1. $\tau = c' + \sigma' \tan \phi'$
2. shear strength = (compressive strength)/2

Golder Associates

RECORD OF BOREHOLE No FC-1

SHEET 1 OF 2 **METRIC**

PROJECT 10-1111-0211

G.W.P. 2150-01-00 LOCATION N 4830867.9 ;E 287311.4 ORIGINATED BY CS

DIST HWY 401 BOREHOLE TYPE 108 mm I.D. Hollow Stem Augers, HW Casing and HQ Coring COMPILED BY CC/TVA

DATUM Geodetic DATE May 1 and 3, 2012 CHECKED BY KJB

ELEV / DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
163.7	GROUND SURFACE						
0.0	TOPSOIL						
0.2	Clayey silt, some gravel, some sand, containing rootlets (FILL) Soft Brown and grey Moist	1	SS	3	163		
		2	SS	3		o	
162.2 / 1.5	CLAYEY SILT with SAND, trace to some gravel Soft to stiff Brown Moist	3A / 3B	SS	5	162		
		4	SS	13	161	o ⊢⊣	0 30 53 17
	sandy grey wet	5	SS	3	160		
		6	SS	8		o ⊢⊣	16 24 40 20
159.2 / 4.5	SAND and SILT, some gravel, trace to some clay (TILL) Very dense Grey Wet	7	SS	52	159	o	16 33 45 6
					158		
		8	SS	68	157		
156.5 / 7.2	Silty SAND and GRAVEL, trace to some clay (TILL) Very dense Grey Wet	9	SS	86/0.25	156	o	
					155		
		10	SS	50/0.1	154		
		11	SS	73/0.15	153		
		12	SS	76/0.2	152	o	29 34 29 8
150.9 / 12.8	SAND and GRAVEL, trace silt, containing cobbles and shale fragments (TILL) Very dense Grey Wet	13	SS	72/0.15	151		
		1	RC	REC 45%	150		
		2	RC	REC 30%			
		3	RC	REC 8%	149		

Column headings also include: 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.

Continued Next Page

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

RECORD OF BOREHOLE No FC-1

SHEET 2 OF 2 **METRIC**

PROJECT 10-1111-0211			
G.W.P. 2150-01-00	LOCATION N 4830867.9 ;E 287311.4	ORIGINATED BY CS	
DIST	HWY 401	BOREHOLE TYPE 108 mm I.D. Hollow Stem Augers, HW Casing and HQ Coring	COMPILED BY CC/TVA
DATUM Geodetic	DATE May 1 and 3, 2012	CHECKED BY KJB	

ELEV DEPTH	DESCRIPTION (SOIL PROFILE)	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	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 (%) GR SA SI CL
	--- CONTINUED FROM PREVIOUS PAGE ---										
148.2											
147.9	END OF BOREHOLE						148	163			
15.9	END OF DCPT Refusal to Further Penetration (163 Blows/0.3 m) ** Artesian Conditions - see Note 3. NOTES: 1. Auger refusal at a depth of 12.9 m, cored through overburden soil and cobbles using HQ size core barrel to a depth of 15.5 m. Advanced Dynamic Cone Penetration Test (DCPT) from depths of 15.5 m to 15.9 m. 2. Water level inside augers at a depth of 1.8 m below ground surface (Elev. 161.9 m), measured at the end of work day on May 1, 2012, when augers advanced to a depth of 12.8 m below ground surface (Elev. 150.9 m). 3. Water level inside casing at 1.0 m above ground surface (Elev. 164.7 m), measured at start of work day on May 2, 2012, when bottom of casing at a depth of 13.0 m below ground surface (Elev. 150.7 m).										

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

Golder Associates

RECORD OF BOREHOLE No FC-2

SHEET 1 OF 1

METRIC

PROJECT 10-1111-0211			
G.W.P. 2150-01-00	LOCATION N 4830851.3 ;E 287283.0		ORIGINATED BY CS
DIST	HWY 401	BOREHOLE TYPE 108 mm I.D. Hollow Stem Augers, NW Casing with Tricone, Wash Boring	COMPILED BY CC/TVA
DATUM Geodetic	DATE May 8, 2012		CHECKED BY KJB

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa (+ FIELD VANE, sensitivity)	WATER CONTENT (%)	UNIT WEIGHT kN/m³	GR	SA	SI	CL
163.8	GROUND SURFACE				**								
0.0	TOPSOIL												
0.2	Clayey silt, some gravel, some sand, containing organics and rootlets to a depth of 0.8 m (FILL) Firm Brown Moist	1	SS	5									
		2	SS	6		163							
162.3 / 1.5	CLAYEY SILT with SAND, trace to some gravel Firm to stiff Grey Moist	3	SS	9		162		w ≈ 11, wP ≈ 15, wL ≈ 21		8	28	47	17
							+ ≈ 95 (4)						
		4	SS	8		161							
		5	SS	5		160		w ≈ 12					
						159	+ ≈ 40 (2); + ≈ 40 (4)						
158.6 / 5.2	Sandy SILT and GRAVEL, trace clay (TILL) Dense to very dense Grey Moist	6	SS	45		158		w ≈ 11		33	27	36	4
		7	SS	56		157							
156.6 / 7.2	SAND and GRAVEL, some silt, trace clay, containing cobbles (TILL) Very dense Grey Wet	8	SS	73		156		w ≈ 7		43	38	15	4
		9	SS	100/0.2		155							
						154							
153.4 / 10.4	END OF BOREHOLE SPOON AND CASING REFUSAL												

** Artesian Conditions - see Note 2.

NOTES:

1. Unable to advance casing below a depth of 10.4 m below ground surface (Elev. 153.4 m).

2. Water flowing from top of casing at the end of work day on May 8, 2012, when bottom of casing at a depth of 10.4 m below ground surface (Elev. 153.4 m).

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

Golder Associates

RECORD OF BOREHOLE No FC-3

SHEET 1 OF 1 | METRIC

PROJECT 10-1111-0211			
G.W.P. 2150-01-00	LOCATION N 4830871.4 ;E 287325.3	ORIGINATED BY CS
DIST	HWY 401	BOREHOLE TYPE 108 mm I.D. Hollow Stem Augers	COMPILED BY CC/TVA
DATUM Geodetic		DATE May 7, 2012	CHECKED BY KJB

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	WATER CONTENT (%)	GR	SA	SI	CL
165.9	GROUND SURFACE								
0.0	Clayey silt, some sand, trace gravel, containing rootlets (FILL) Firm Brown Moist	1	SS	6					
165.0 / 0.9	CLAYEY SILT, some sand, some gravel Stiff to very stiff Brown to grey Moist	2	SS	16					
		3	SS	22	plotted	12	20	42	26
		4	SS	14					
	firm grey wet	5	SS	8	plotted				
162.2 / 3.7	SILT, trace to some sand, trace to some clay, trace gravel Loose Grey Wet	6	SS	4	plotted	3	10	77	10
161.4 / 4.5	CLAYEY SILT, some gravel, some sand Firm Grey Moist	7	SS	6					
160.7 / 5.2	END OF BOREHOLE								

NOTE:

1. Open borehole dry upon completion of drilling.

Column headings: SOIL PROFILE; SAMPLES; STRAT PLOT; GROUND WATER CONDITIONS; ELEVATION SCALE (165, 164, 163, 162, 161); 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 FC-4

SHEET 1 OF 1

METRIC

PROJECT 10-1111-0211			
G.W.P. 2150-01-00	LOCATION N 4830837.9 ;E 287276.8	ORIGINATED BY CS	
DIST	HWY 401	BOREHOLE TYPE 108 mm I.D. Hollow Stem Augers	COMPILED BY CC/TVA
DATUM Geodetic	DATE May 9, 2012	CHECKED BY KJB	

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
164.4	GROUND SURFACE					
0.0	TOPSOIL					
0.2	Clayey silt, some sand, containing rootlets (FILL) Firm Brown Moist	1	SS	5		
		2	SS	5		
162.9 / 1.5	CLAYEY SILT, trace to some gravel, trace to some sand, containing rootlets to a depth of 2.2 m Very soft to stiff Brown Moist to wet	3	SS	11	∇	
		4	SS	3		
		5	SS	WH		
159.1 / 5.3	Auger grinding at a depth of 5.2 m SAND and SILT, some gravel, trace to some clay (TILL) Very dense Grey Wet	6	SS	100/0.3		20 43 30 7
158.6 / 5.8	END OF BOREHOLE					

Field vane shear strength values plotted (sensitivity): + 2, + 1, + 1, + 9.

Elevation scale: 164, 163, 162, 161, 160, 159

NOTES:

1. Water level inside augers at a depth of 0.3 m below ground surface (Elev. 164.1 m) when advanced to a depth of 4.6 m below ground surface (Elev. 159.8 m).

2. Water level inside augers at a depth of 2.2 m below ground surface (Elev. 162.2 m) upon completion of sampling.

3. Water level in open borehole at a depth of 2.0 m below ground surface (Elev. 162.4 m), measured one hour 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

UNIT WEIGHT γ kN/m³

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

Golder Associates

RECORD OF BOREHOLE No FC-5

SHEET 1 OF 2

METRIC

PROJECT 10-1111-0211
G.W.P. 2150-01-00
LOCATION N 4830834.4 ;E 287327.5
ORIGINATED BY SB
DIST HWY 401
BOREHOLE TYPE 108 mm I.D. Hollow Stem Augers, NW Casing with Tricone, Wash Boring
COMPILED BY CC/TVA
DATUM Geodetic
DATE May 14 and 15, 2012
CHECKED BY KJB

ELEV / DEPTH	DESCRIPTION (SOIL PROFILE)	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
168.6	GROUND SURFACE							
0.0	ASPHALT							
0.2 / 167.8	Sand and gravel (FILL) Brown Moist					168		
0.8	Clayey silt, some sand, trace to some gravel (FILL) Firm to very stiff Brown and grey Moist	1	SS	17				
		2	SS	12	∇	167		
		3	SS	10		166	○	
		4	SS	14		165		
		5	SS	8		164		
		6	SS	7			○ ├──┤	8 19 47 26
162.8						163		
5.8	SAND and SILT, some gravel, trace to some clay Loose Grey Wet	7	SS	4		162	○	13 43 35 9
161.3								
7.3	CLAYEY SILT, some sand, trace to some gravel Firm to very stiff Grey Wet to moist	8	SS	5		161	├○─┤	
						160		
159.0		9	SS	16				
9.6	Silty SAND and GRAVEL to Gravelly SAND, some silt, trace clay (TILL) Very dense Grey Wet					159		
						158		
		10	SS	50			○	24 55 16 5
						157		
		11	SS	108		156	○	
154.7		12	SS	106/0.15		155		
13.9	END OF BOREHOLE							

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

Continued Next Page

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

RECORD OF BOREHOLE No FC-5

SHEET 2 OF 2

METRIC

PROJECT 10-1111-0211
G.W.P. 2150-01-00
LOCATION N 4830834.4 ;E 287327.5
ORIGINATED BY SB
DIST HWY 401
BOREHOLE TYPE 108 mm I.D. Hollow Stem Augers, NW Casing with Tricone, Wash Boring
COMPILED BY CC/TVA
DATUM Geodetic
DATE May 14 and 15, 2012
CHECKED BY KJB

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	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 (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
	--- CONTINUED FROM PREVIOUS PAGE ---										
	NOTE: 1. Water level inside casing at a depth of 1.8 m below ground surface (Elev. 166.8 m) upon completion of drilling.										

+ 3, × 3: Numbers refer to Sensitivity

○ 3% STRAIN AT FAILURE

Foundation Design

RECORD OF BOREHOLE No FC-6

SHEET 1 OF 2

METRIC

PROJECT 10-1111-0211

G.W.P. 2150-01-00

LOCATION N 4830817.2 ;E 287306.2

ORIGINATED BY SB

DIST HWY 401

BOREHOLE TYPE 108 mm I.D. Hollow Stem Augers, NW Casing with Tricone, Wash Boring

COMPILED BY CC/TVA

DATUM Geodetic

DATE May 13 and 14, 2012

CHECKED BY KJB

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL
168.3	GROUND SURFACE							
0.0	ASPHALT							
0.2	Sand and gravel (FILL) Brown Moist							
167.5 0.8	Clayey silt, trace to some gravel, trace sand (FILL) Firm to very stiff Brown Moist	1	SS	18				
		2	SS	20				
		3	SS	17				
		4	SS	22				
		5	SS	10				
		6	SS	18				
		7	SS	5				
161.1 7.2	CLAYEY SILT, some sand, trace to some gravel, containing sandy silt interlayers Stiff Grey Moist	8	SS	9	7	25	55	13
		9	SS	8				
158.1 10.2	Silty SAND, trace to some gravel, trace to some clay (TILL) Dense Grey Wet	10	SS	40	10	59	22	9
156.6 11.7	SAND and GRAVEL, trace to some silt, trace clay, containing cobble (TILL) Very dense Grey Wet	11	SS	88	46	39	13	2
		12	SS	184				
154.1 14.2	END OF BOREHOLE							

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; GROUND WATER CONDITIONS; ELEVATION SCALE (168–155); 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 (%) GR SA SI CL

Continued Next Page

+3, ×3: Numbers refer to Sensitivity

○3% STRAIN AT FAILURE

Golder Associates

RECORD OF BOREHOLE No FC-6

SHEET 2 OF 2

METRIC

PROJECT 10-1111-0211			
G.W.P. 2150-01-00	LOCATION N 4830817.2 ;E 287306.2		ORIGINATED BY SB
DIST	HWY 401	BOREHOLE TYPE 108 mm I.D. Hollow Stem Augers, NW Casing with Tricone, Wash Boring	COMPILED BY CC/TVA
DATUM Geodetic	DATE May 13 and 14, 2012		CHECKED BY KJB

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED (20 40 60 80 100)	PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) (10 20 30)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION --- CONTINUED FROM PREVIOUS PAGE ---	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
	NOTE: 1. Given that Wash Boring techniques were used to advance the NW casing, the groundwater condition was not measured upon completion of drilling.										

GTA-MTO 001 1011110211.GPJ GAL-GTA.GDT 2/8/13

$+^3, \times^3$: Numbers refer to Sensitivity

$\circ^{3\%}$ STRAIN AT FAILURE

RECORD OF BOREHOLE No FC-7

SHEET 1 OF 2

METRIC

PROJECT 10-1111-0211			
G.W.P. 2150-01-00	LOCATION N 4830814.6 ;E 287338.1	ORIGINATED BY MB
DIST HWY 401	BOREHOLE TYPE 108 mm I.D. Hollow Stem Augers, NW Casing with Tricone, Wash Boring	COMPILED BY CC/TVA
DATUM Geodetic	DATE May 10 and 11, 2012	CHECKED BY KJB

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
168.5	GROUND SURFACE					
0.0	ASPHALT					
0.2	Sand and gravel (FILL) Brown Moist				168	
167.7 / 0.8	Clayey silt, some sand, trace to some gravel (FILL) Firm to stiff Brown Moist	1	SS	12		
		2	SS	12	167	
		3	SS	7	166	
		4	SS	6	165	6 22 45 27
		5	SS	9		
		6	SS	12	164	
162.9 / 5.6	SAND and SILT, some gravel, trace to some clay, containing clayey silt seams Compact Brown Moist				163	
		7	SS	14	162	15 41 33 11
					161	
160.6 / 7.9	CLAYEY SILT, some sand, trace to some gravel Firm to stiff Grey Wet	8	SS	10	160	
		9	SS	4	159	6 24 51 19
158.1 / 10.4	Silty SAND and GRAVEL, trace clay (TILL) Very Dense Grey Moist to wet				158	
		10	SS	142	157	
		11	SS	101	156	28 41 28 3
					155	
		12	SS	87		
154.3 / 14.2	END OF BOREHOLE					

Plot headings: 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; GROUND WATER CONDITIONS; STRAT PLOT

Continued Next Page

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

RECORD OF BOREHOLE No FC-7

SHEET 2 OF 2 **METRIC**

PROJECT 10-1111-0211
G.W.P. 2150-01-00 LOCATION N 4830814.6 ;E 287338.1 ORIGINATED BY MB
DIST HWY 401 BOREHOLE TYPE 108 mm I.D. Hollow Stem Augers, NW Casing with Tricone, Wash Boring COMPILED BY CC/TVA
DATUM Geodetic DATE May 10 and 11, 2012 CHECKED BY KJB

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	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 (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
	--- CONTINUED FROM PREVIOUS PAGE ---										
	NOTE: 1. Given that Wash Boring techniques were used to advance the NW casing, the groundwater condition was not measured upon completion of drilling.										

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

Foundation Design

Golder Associates

RECORD OF BOREHOLE No FC-8

SHEET 1 OF 1 **METRIC**

PROJECT 10-1111-0211

G.W.P. 2150-01-00 LOCATION N 4830790.6 ;E 287315.9 ORIGINATED BY SB

DIST HWY 401 BOREHOLE TYPE 108 mm I.D. Hollow Stem Augers, NW Casing with Tricone, Wash Boring COMPILED BY CC/TVA

DATUM Geodetic DATE May 9 and 10, 2012 CHECKED BY KJB

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
164.0	GROUND SURFACE				
0.0	Sandy silt, trace to some clay, trace gravel, containing organics and rootlets (FILL) Loose Brown Moist	1	SS	5	
163.3 / 0.7	Clayey silt with sand, trace to some gravel, containing organics, rootlets and cobbles to a depth of 1.5 m (FILL) Very stiff to hard Brown to grey Moist to wet	2	SS	32	
		3	SS	25	
		4	SS	15	16 28 39 17
161.0 / 3.0	CLAYEY SILT, trace gravel, trace sand Firm to stiff Brown and grey Wet	5	SS	14	
		6	SS	9	
		7	SS	6	
158.4 / 5.6	Silty SAND and GRAVEL, trace clay (TILL) Very dense Grey Moist	8	SS	67	30 34 30 6
157.3 / 6.7	END OF BOREHOLE				

GROUND WATER CONDITIONS: **

ELEVATION SCALE: 163, 162, 161, 160, 159, 158

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/m^3

** Artesian Conditions - see Note 2.

NOTES:

1. Water flowing from top of casing when advanced to a depth of 6.1 m below ground surface (Elev. 157.9 m).

2. Water level inside casing measured at 0.9 m above ground surface (Elev. 164.9 m) upon completion of drilling.

GTA-MTO 001 1011110211.GPJ GAL-GTA.GDT 2/8/13

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

Golder Associates

RECORD OF BOREHOLE No FC-9

SHEET 1 OF 1

METRIC

PROJECT 10-1111-0211

G.W.P. 2150-01-00 LOCATION N 4830805.8 ;E 287360.1 ORIGINATED BY SB

DIST HWY 401 BOREHOLE TYPE 108 mm I.D. Hollow Stem Augers COMPILED BY CC/TVA

DATUM Geodetic DATE May 3, 2012 CHECKED BY KJB

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
164.3	GROUND SURFACE					
0.0	Clayey silt, some sand, trace to some gravel, containing organics and rootlets (FILL) Firm to stiff Brown to grey Moist	1	SS	5	164	
		2	SS	11	163	
		3	SS	8		
		4	SS	7	162	
161.3 / 3.0	CLAYEY SILT with SAND, trace to some gravel Very soft to soft Grey Wet	5	SS	2	161	10 41 37 12
		6	SS	WH	160	
159.6 / 4.7	Gravelly SAND, some silt, trace clay Compact Grey Wet	7A / 7B	SS	18		20 61 16 3
159.1 / 5.2	END OF BOREHOLE					

NOTE:

1. Open borehole dry upon completion of drilling.

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; GROUND WATER CONDITIONS; ELEVATION SCALE; 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.

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

RECORD OF BOREHOLE No FC-10

SHEET 1 OF 1

METRIC

PROJECT 10-1111-0211
G.W.P. 2150-01-00
LOCATION N 4830774.0 ;E 287308.1
ORIGINATED BY SB
DIST HWY 401
BOREHOLE TYPE 108 mm I.D. Hollow Stem Augers
COMPILED BY CC/TVA
DATUM Geodetic
DATE May 8, 2012
CHECKED BY KJB

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
164.1	GROUND SURFACE					
0.0	Clayey silt, trace to some sand, trace to some gravel, containing organics and rootlets (FILL) Soft Brown Moist	1	SS	WH	164	
163.4 / 0.7	CLAYEY SILT with SAND, trace gravel (TILL) Firm to very stiff Brown and grey Moist	2	SS	16	163	
		3	SS	19	162	
		4	SS	9		5 30 48 17
		5	SS	7	161	
		6	SS	10	160	
		7	SS	6		3 33 44 20
158.9 / 5.2	END OF BOREHOLE				159	

NOTE:

1. Open borehole dry upon completion of drilling.

Chart headings: 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; GROUND WATER CONDITIONS; STRAT PLOT.

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

Golder Associates

RECORD OF BOREHOLE No FC-11

SHEET 1 OF 1

METRIC

PROJECT 10-1111-0211

G.W.P. 2150-01-00

LOCATION N 4830795.8 ;E 287352.4

ORIGINATED BY SB

DIST HWY 401

BOREHOLE TYPE 108 mm I.D. Hollow Stem Augers, NW Casing, Wash Boring

COMPILED BY CC/TVA

DATUM Geodetic

DATE May 3 and 4, 2012

CHECKED BY KJB

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
164.4	GROUND SURFACE					
0.0	Sandy silt, trace to some clay, trace gravel, containing organics, rootlets, and wood fragments (FILL) Loose to compact Brown and grey Moist	1	SS	7	164	
		2	SS	14	163	
162.9 / 1.5	Clayey silt, some sand, trace to some gravel (FILL) Stiff Brown Moist	3	SS	13		
162.2 / 2.2	CLAYEY SILT with SAND, trace to some gravel Very soft to firm Grey Moist	4	SS	4	162	6 32 48 14
		5	SS	3	161	
		6	SS	WH		7 31 44 18
159.9 / 4.5	Silty SAND, trace gravel Compact Grey Wet	7	SS	27	160	
158.8 / 5.6	Silty SAND and GRAVEL, some silt, trace to some clay, contains sand and gravel layers (TILL) Very dense Grey Wet	8	SS	57	159, 158	32 36 26 6
		9	SS	100/0.1	157, 156	
		10	SS	81	155	49 35 13 3
		11	SS	61	154, 153	
		12	SS	126	152	
151.8 / 12.7	END OF BOREHOLE					

Ground water conditions: ▽ **

** Artesian Conditions - see Note 1.

NOTE:

1. Water flowing from top of casing which was 1.5 m above ground surface (Elev. 165.9 m) when advanced to a depth of 12.2 m below ground surface (Elev. 152.2 m).

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

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

Foundation Design

RECORD OF BOREHOLE No FC-12

SHEET 1 OF 1 **METRIC**

PROJECT 10-1111-0211
G.W.P. 2150-01-00
LOCATION N 4830778.0 ;E 287318.6
ORIGINATED BY SB
DIST HWY 401
BOREHOLE TYPE 108 mm I.D. Hollow Stem Augers, NW Casing with Tricone, Wash Boring
COMPILED BY CC/TVA
DATUM Geodetic
DATE May 6 and 8, 2012
CHECKED BY KJB

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	GR	SA	SI	CL
163.9	GROUND SURFACE								
0.0	Clayey silt, trace to some sand, trace to some gravel, containing organics and rootlets (FILL) Soft to very stiff Brown and grey Moist	1	SS	3					
		2	SS	23	163				
162.4 1.5	CLAYEY SILT, some sand, trace to some gravel (TILL) Very stiff to hard Brown and grey Moist to wet	3	SS	20	162	4	22	58	16
		4	SS	32	161				
		5	SS	37	160				
		6	SS	21					
		7	SS	24	159				
158.3 5.6	SAND and SILT, some gravel, trace to some clay (TILL) Very dense Grey Wet				158				
157.3		8	SS	104		17	46	30	7
6.6	END OF BOREHOLE								

Groundwater 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/m^3
REMARKS & GRAIN SIZE DISTRIBUTION (%)

** Artesian Conditions - see Note 2.

NOTES:

1. Water level inside casing at a depth of 2.7 m below ground surface (Elev. 161.2 m) at start of work day on May 8, 2012, when bottom of casing at a depth of 3.2 m below ground surface (Elev. 160.7 m).

2. Water flowing from top of casing which was 1.2 m above ground surface (Elev. 165.1 m) when advanced to a depth of 6.0 m below ground surface (Elev. 157.9 m).

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

GTA-MTO 001 1011110211.GPJ GAL-GTA.GDT 2/8/13

Golder Associates

RECORD OF BOREHOLE No FC-13

SHEET 1 OF 1

METRIC

PROJECT 10-1111-0211
G.W.P. 2150-01-00
LOCATION N 4830881.8 ;E 287336.5
ORIGINATED BY SB
DIST HWY 401
BOREHOLE TYPE 108 mm I.D. Hollow Stem Augers, NW Casing with Tricone, Wash Boring
COMPILED BY BM/TVA
DATUM Geodetic
DATE September 4, 2012
CHECKED BY KJB

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
167.1	GROUND SURFACE				
0.0	CLAYEY SILT with SAND, trace gravel, containing rootlets to a depth of 0.6 m Stiff to very stiff Brown to grey Moist	1	SS	11	
		2	SS	15	
		3	SS	11	5 30 44 21
		4	SS	19	
		5	SS	12	
		6	SS	10	
		7	SS	6	13 29 35 23
		8A / 8B	SS	4	
160.5 6.6	SAND and SILT, trace to some clay, trace gravel (TILL) Dense Grey Moist to wet				
		9	SS	35	2 37 54 7
158.4 8.7	Silty SAND, some gravel, trace clay (TILL) Very dense Wet	10	SS	50/0.03	
156.3		11	SS	60/0.15	
10.8	END OF BOREHOLE BROKEN CASING				

Field vane readings at approx. elev. 161.5: >136 Kpa (sensitivity 1); >144 Kpa (sensitivity 1)

NOTE:

1. Unable to advance borehole beyond a depth of 10.8 m as part of the casing broke while penetrating through the very dense overburden. Backfilled borehole, moved drilling 1.5 m west and advanced Borehole FC-13A, and continued sampling below 10.8 m depth.

Column headings: STRAT PLOT; GROUND WATER CONDITIONS; ELEVATION SCALE (167–157); 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

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

Golder Associates

RECORD OF BOREHOLE No FC-13A

SHEET 1 OF 2 **METRIC**

PROJECT 10-1111-0211

G.W.P. 2150-01-00 LOCATION N 4830881.8 ;E 287335.0 ORIGINATED BY SB

DIST HWY 401 BOREHOLE TYPE 108 mm I.D. Hollow Stem Augers, NW Casing with Tricone, Wash Boring COMPILED BY BM/TVA

DATUM Geodetic DATE September 4 to 6, 2012 CHECKED BY KJB

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
167.1	GROUND SURFACE						
0.0	Refer to Record of Borehole FC-13 for subsurface conditions within these elevations.				167		
					166		
					165		
					164		
					163		
					162		
					161		
					160		
					159		
					158		
					157		
156.3 / 10.8	Silty SAND, some gravel, trace clay (TILL) Very dense Wet				156		
					155		
	Loose, containing cobbles	12	SS	9*			
					154		
153.4 / 13.7	Gravelly SAND, trace to some silt, trace clay (TILL) Very dense Wet	13	SS	50/0.08	153	~10	25 59 11 5

Continued Next Page

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

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

RECORD OF BOREHOLE No FC-13A

SHEET 2 OF 2 **METRIC**

PROJECT 10-1111-0211
G.W.P. 2150-01-00
LOCATION N 4830881.8 ;E 287335.0
ORIGINATED BY SB
DIST HWY 401
BOREHOLE TYPE 108 mm I.D. Hollow Stem Augers, NW Casing with Tricone, Wash Boring
COMPILED BY BM/TVA
DATUM Geodetic
DATE September 4 to 6, 2012
CHECKED BY KJB

ELEV DEPTH	SOIL PROFILE DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH kPa	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	--- CONTINUED FROM PREVIOUS PAGE ---										
	Gravelly SAND, trace to some silt, trace clay (TILL) Very dense Wet						152				
			14	SS	50/0.08						
							151				
			15	SS	65/0.03				o		
							150				
149.3 17.8	END OF BOREHOLE										

* SPT "N" value may have been influenced by difficulties advancing augers / wash boring at this depth.

** Artesian Conditions - see Note 4.

NOTES:

1. Borehole FC-13A augered from ground surface to a depth of 7.6 m; then switched to 'NW' casing method. Lost casing shoe at depth of 11.9 m while penetrating through the very dense overburden, then switched back to auger with difficulties advancing auger to 12.2 m depth, then completed borehole using NW Casing.

2. Water level inside augers at a depth of 2.7 m below ground surface (Elevation 164.4 m), measured at end of work day (at 4:16 pm) on Sept. 05, 2012, when augers advanced to a depth of 12.2 m below ground surface (Elev. 154.9 m).

3. Water level inside augers at a depth of 2.1 m below ground surface (Elevation 165.0 m), measured at start of work day (at 7:00 am) on Sept. 06, 2012, when augers advanced to a depth of 12.8 m below ground surface (Elev. 154.3 m).

4. Water flowing from top of casing when advanced to a depth of 17.8 m (Elev. 149.3 m). Stacked up casing to about 5.0 m above ground surface to monitor the hydrostatic head upon completion of drilling operations on September 6, 2012. The recorded water level readings are:

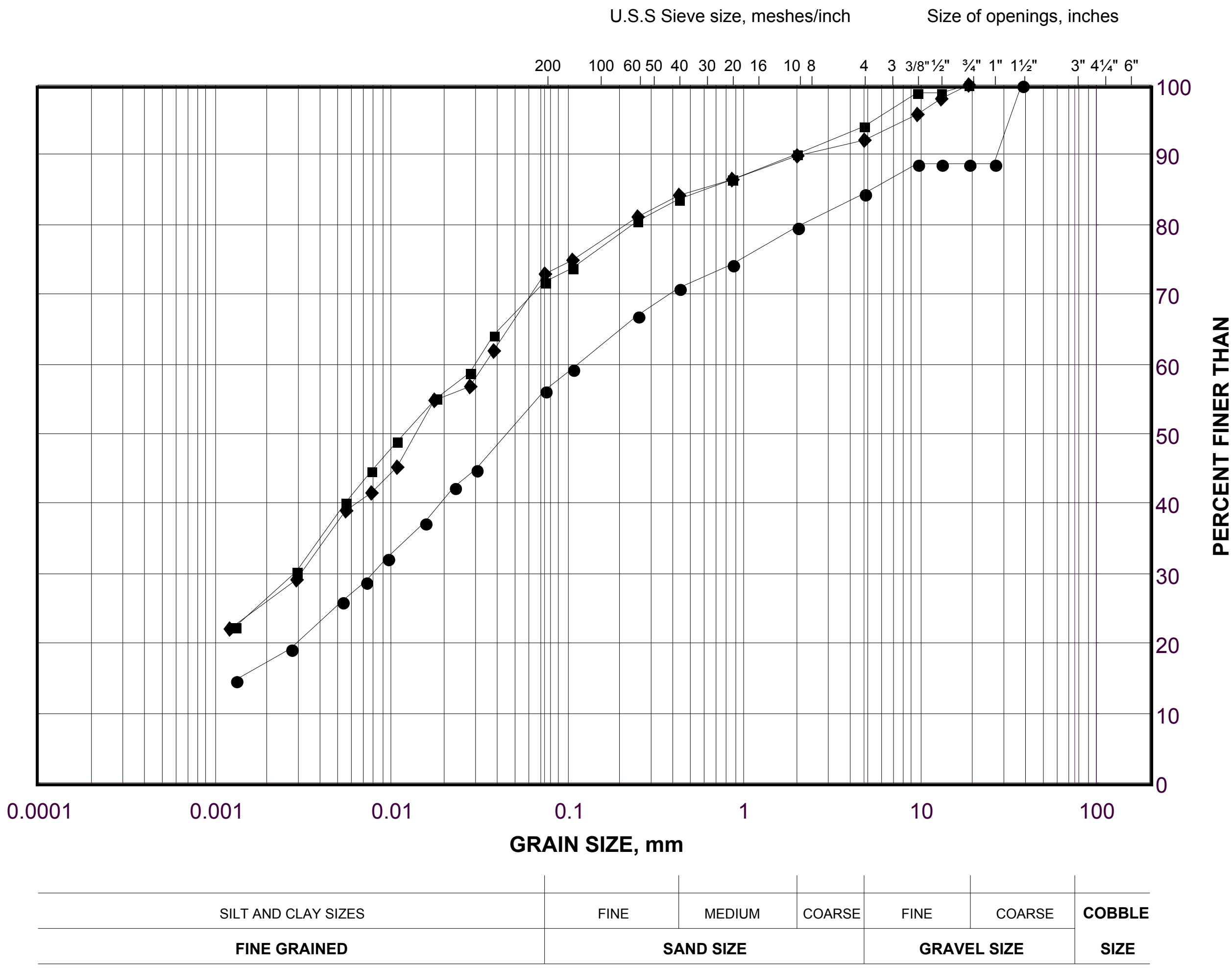
Time	Depth (m) to W.L.	W.L. Elev(m)
12:30 pm	-0.6	167.7
12:36 pm	-2.1	169.2
12:41 pm	-3.5	170.6
1:30 pm	-4.8	171.9

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

APPENDIX B

Laboratory Test Results

GRAIN SIZE DISTRIBUTION Clayey Silt to Clayey Silt with Sand Fill	FIGURE B1



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	FC-8	4	161.4
■	FC-7	4	165.2
◆	FC-5	6	163.7

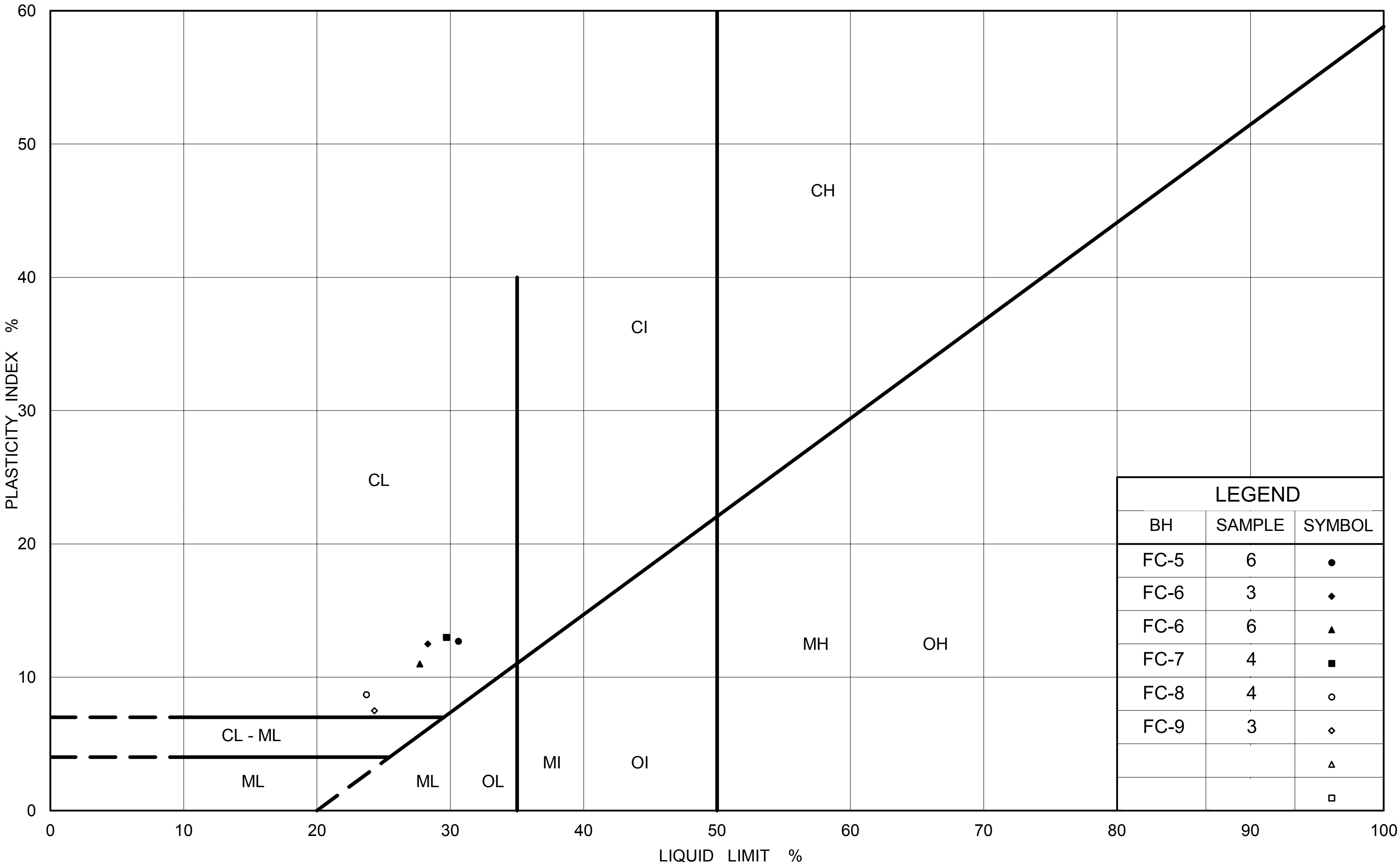
Project Number: 10-1111-0211

Checked By: TVA

Golder Associates

Date: 05-Oct-12

Oct 75, FF-S-21



LEGEND		
BH	SAMPLE	SYMBOL
FC-5	6	●
FC-6	3	◆
FC-6	6	▲
FC-7	4	■
FC-8	4	○
FC-9	3	◇
		△
		□

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PLASTIC ITY CHART
Clayey Silt to Clayey Silt with Sand Fill

Figure No. B2

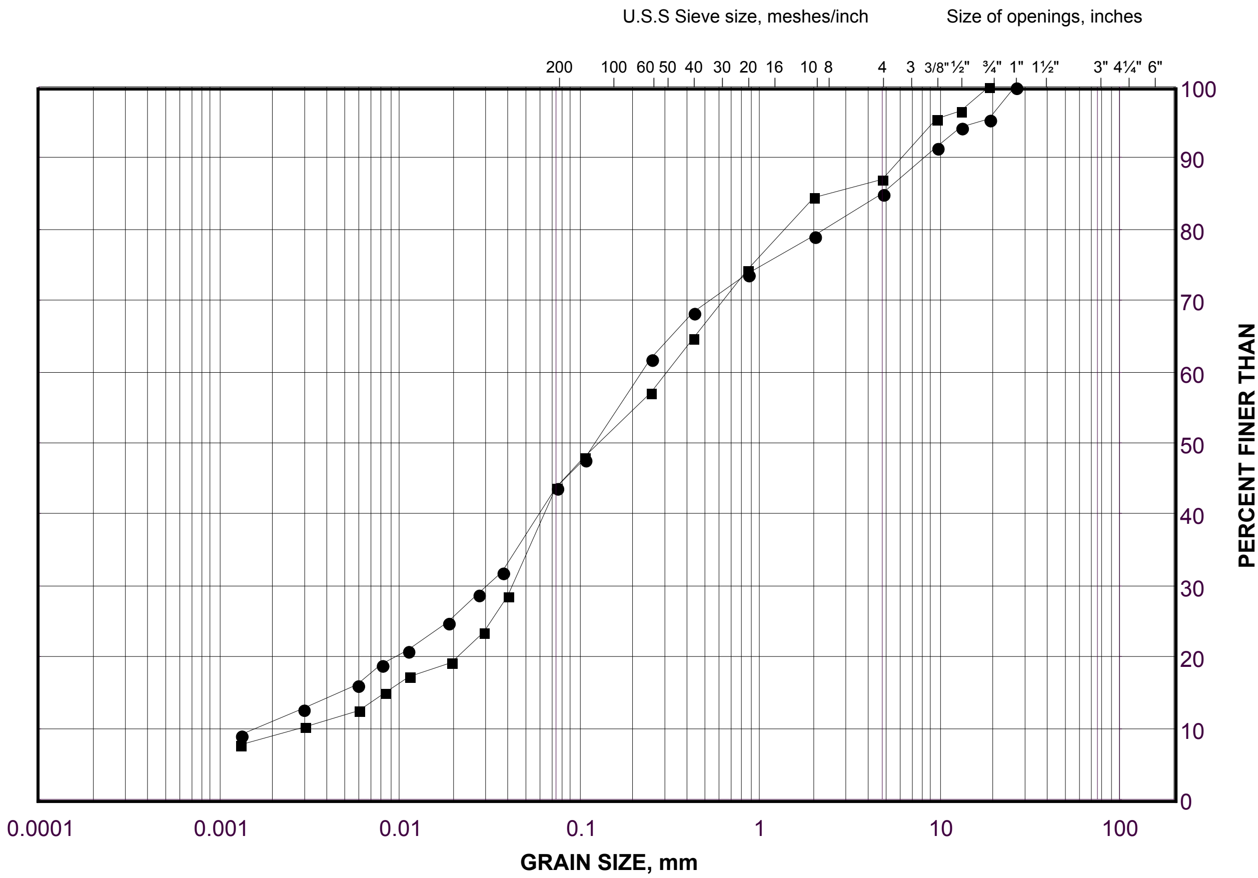
Project No. 10-1111-0211

Checked By: TVA

GRAIN SIZE DISTRIBUTION

Sand and Silt

FIGURE B3



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	FC-7	7	162.1
■	FC-5	7	162.2

Project Number: 10-1111-0211

Checked By: TVA

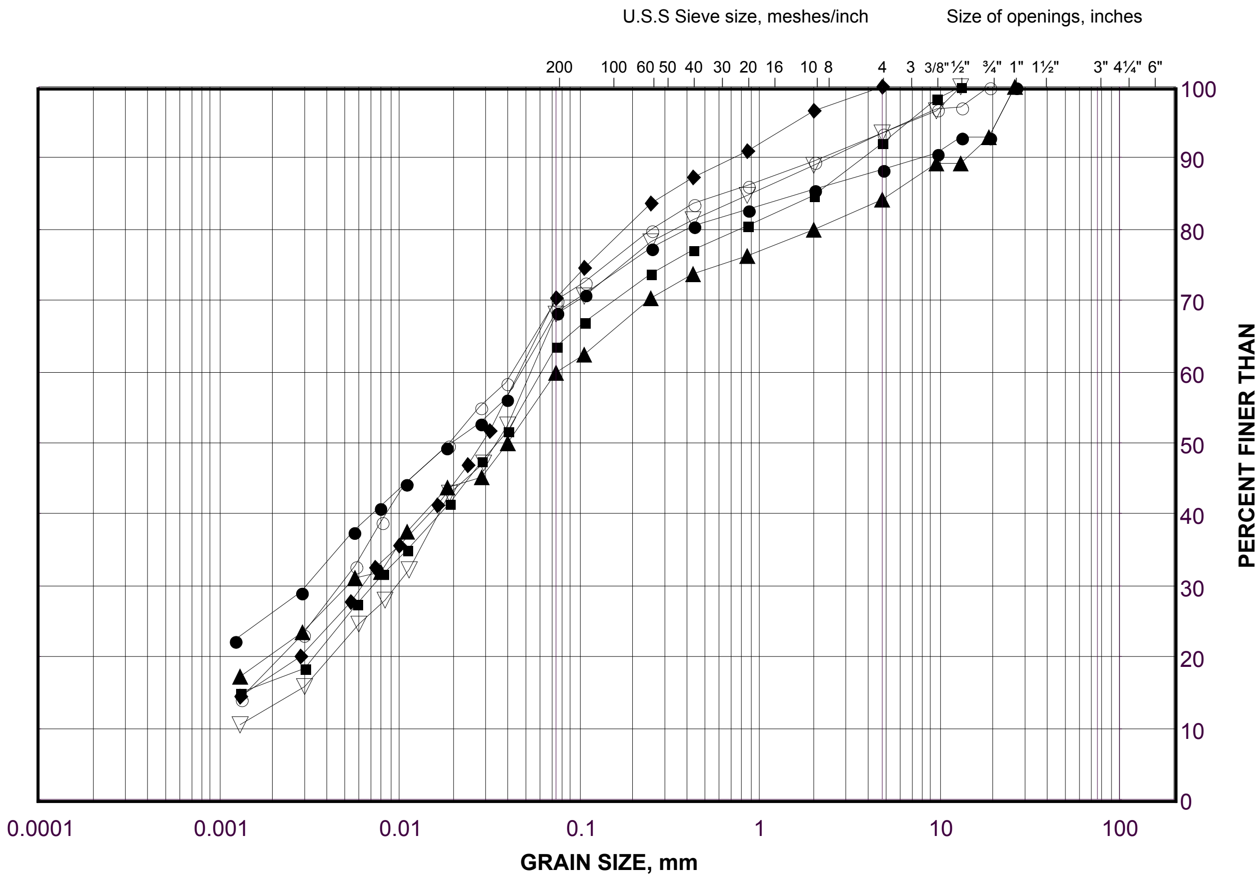
Golder Associates

Date: 05-Oct-12

GRAIN SIZE DISTRIBUTION

Clayey Silt to Clayey Silt with Sand

FIGURE B4A



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	FC-3	3	164.1
■	FC-2	3	161.9
◆	FC-1	4	161.1
▲	FC-1	6	159.6
▽	FC-6	8	160.4
○	FC-7	9	159.1

Project Number: 10-1111-0211

Checked By: TVA

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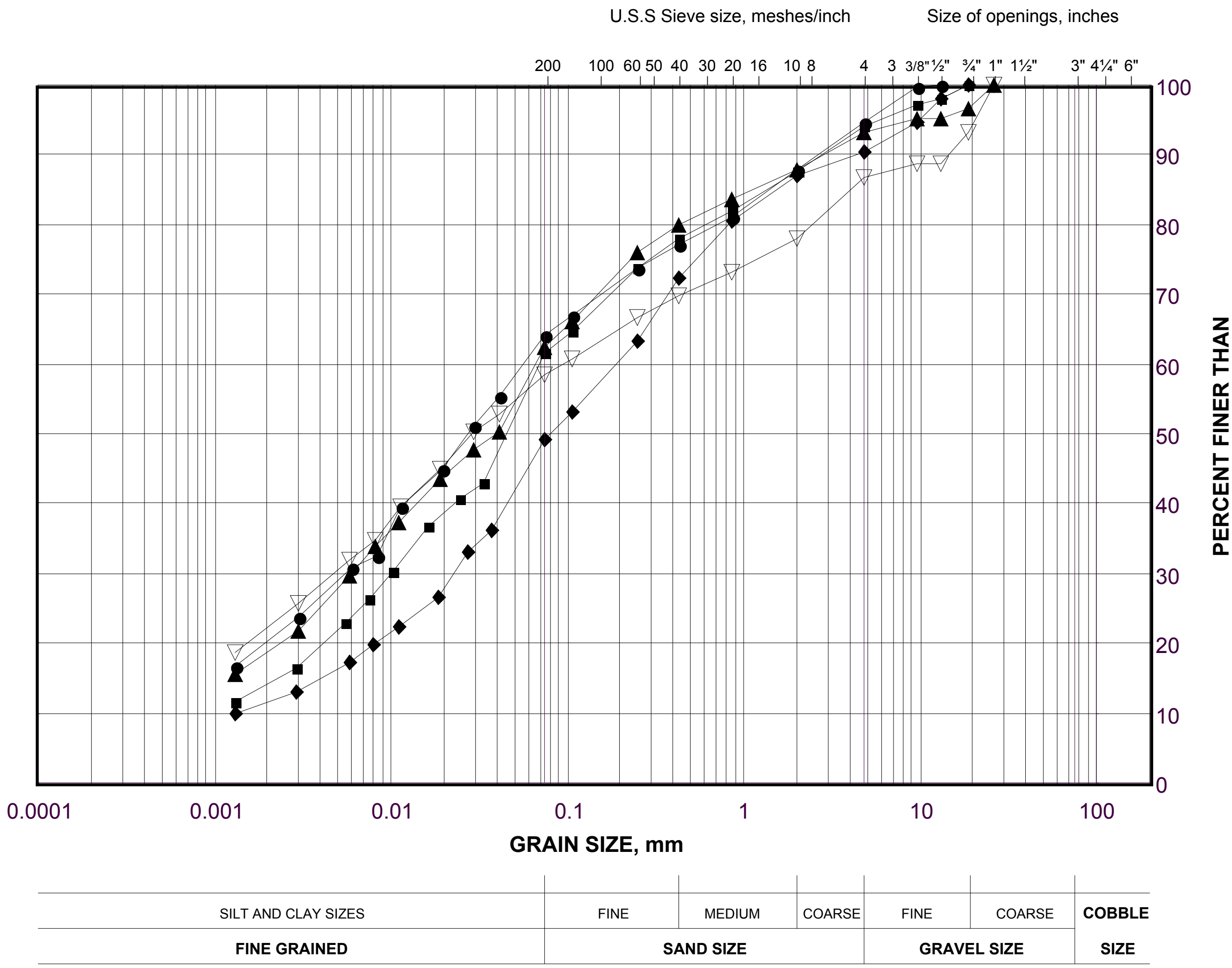
Date: 05-Oct-12

GRAIN SIZE DISTRIBUTION

Clayey Silt to Clayey Silt with Sand

FIGURE B4B



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	FC-13	3	165.3
■	FC-11	4	161.8
◆	FC-9	5	160.9
▲	FC-11	6	160.3
▽	FC-13	7	162.2

Project Number: 10-1111-0211

Checked By: TVA

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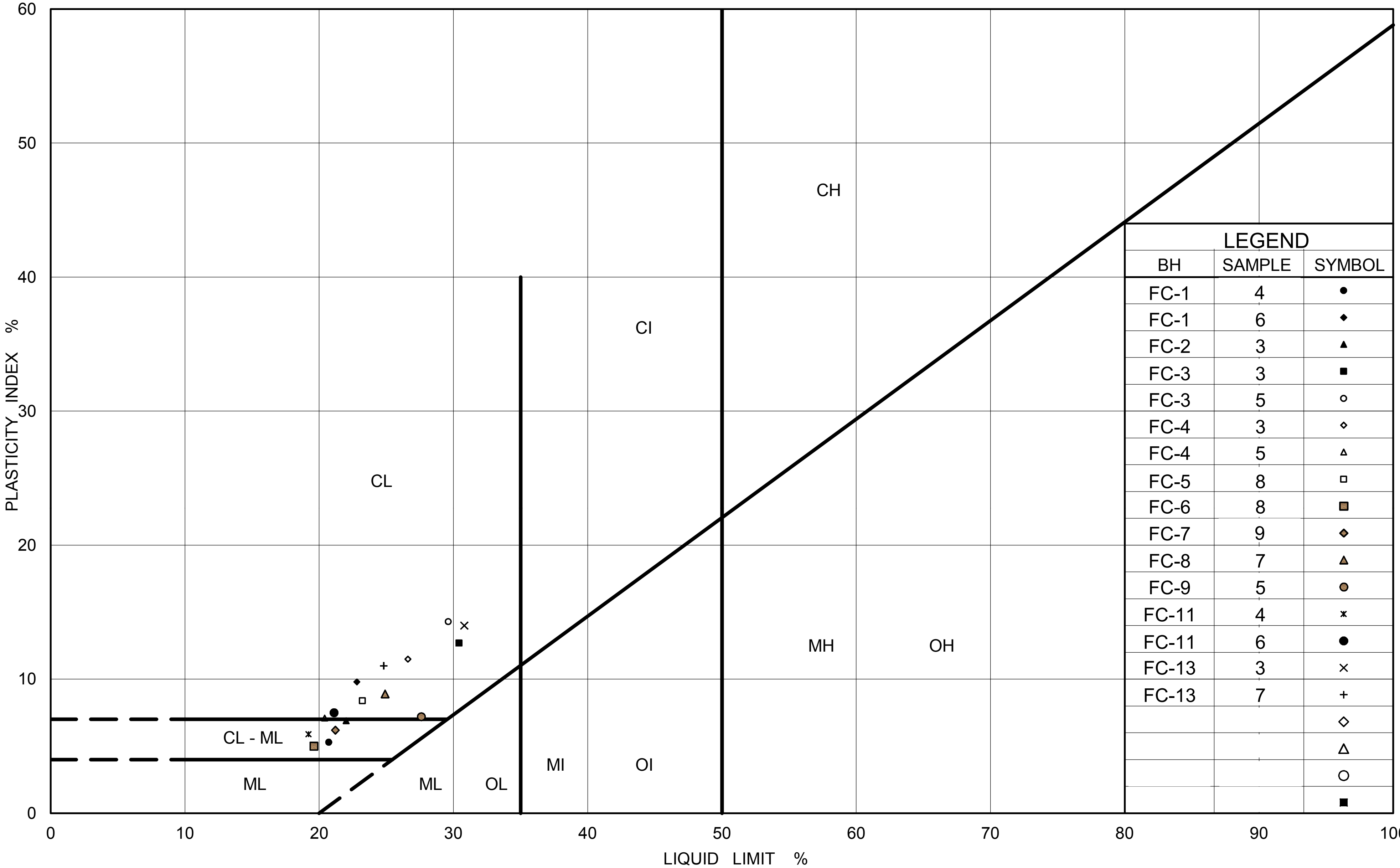
Date: 09-Oct-12

Oct 75, FF-S-21



LEGEND

BH	SAMPLE	SYMBOL
FC-1	4	●
FC-1	6	◆
FC-2	3	▲
FC-3	3	■
FC-3	5	○
FC-4	3	◇
FC-4	5	△
FC-5	8	□
FC-6	8	■
FC-7	9	◆
FC-8	7	▲
FC-9	5	●
FC-11	4	✱
FC-11	6	●
FC-13	3	×
FC-13	7	+
		◇
		△
		○
		■

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PLASTICITY CHART
Clayey Silt to Clayey Silt with Sand

Figure No. B5

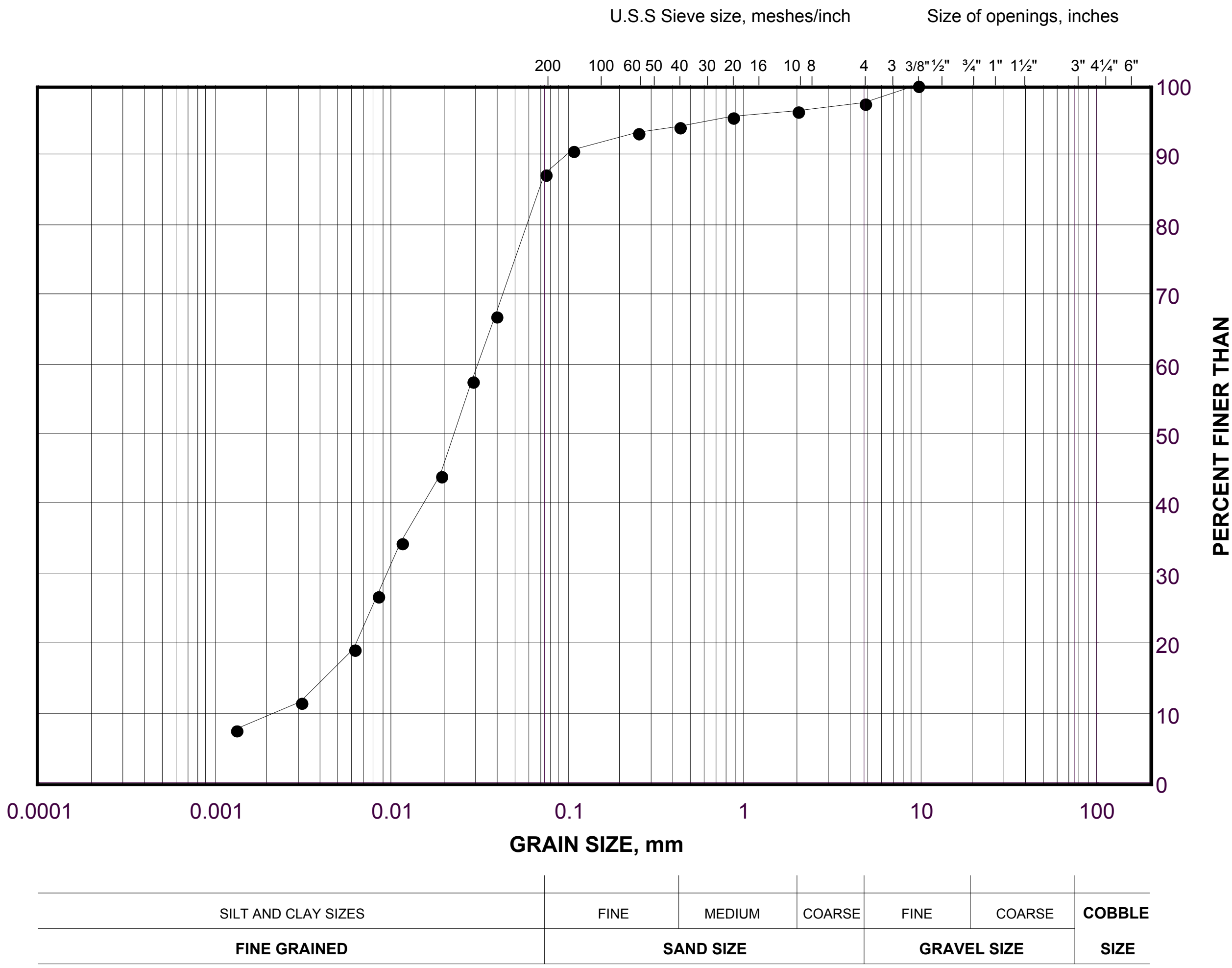
Project No. 10-1111-0211

Checked By:

GRAIN SIZE DISTRIBUTION

Silt

FIGURE B6



LEGEND

SYMBOL	BOREHOLE	SAMPLE	DEPTH(m)
●	FC-3	6	161.8

Project Number: 10-1111-0211

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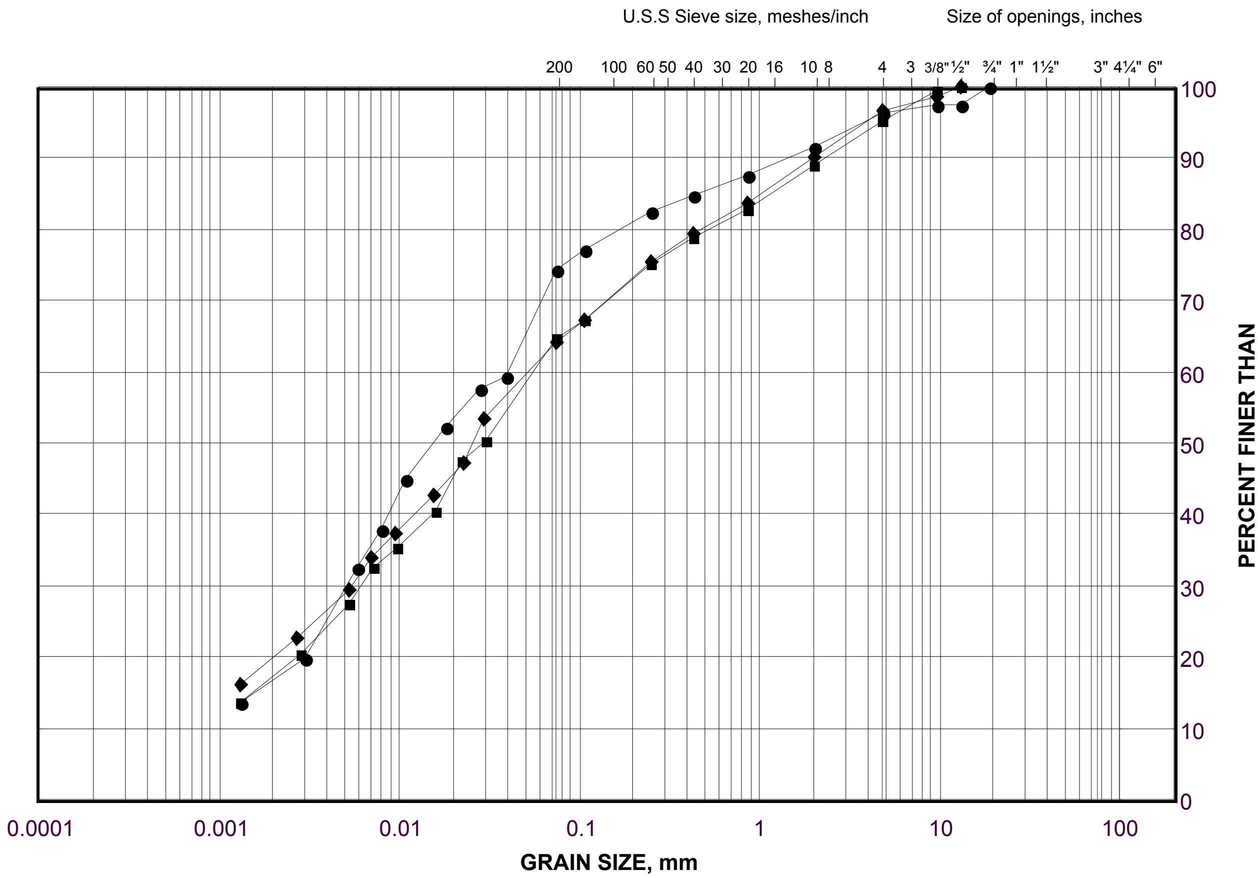
GRAIN SIZE DISTRIBUTION

Clayey Silt to Clayey Silt with Sand Till

FIGURE B7



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	FC-12	3	162.1
■	FC-10	4	161.5
◆	FC-10	7	159.3

Project Number: 10-1111-0211

Checked By: TVA

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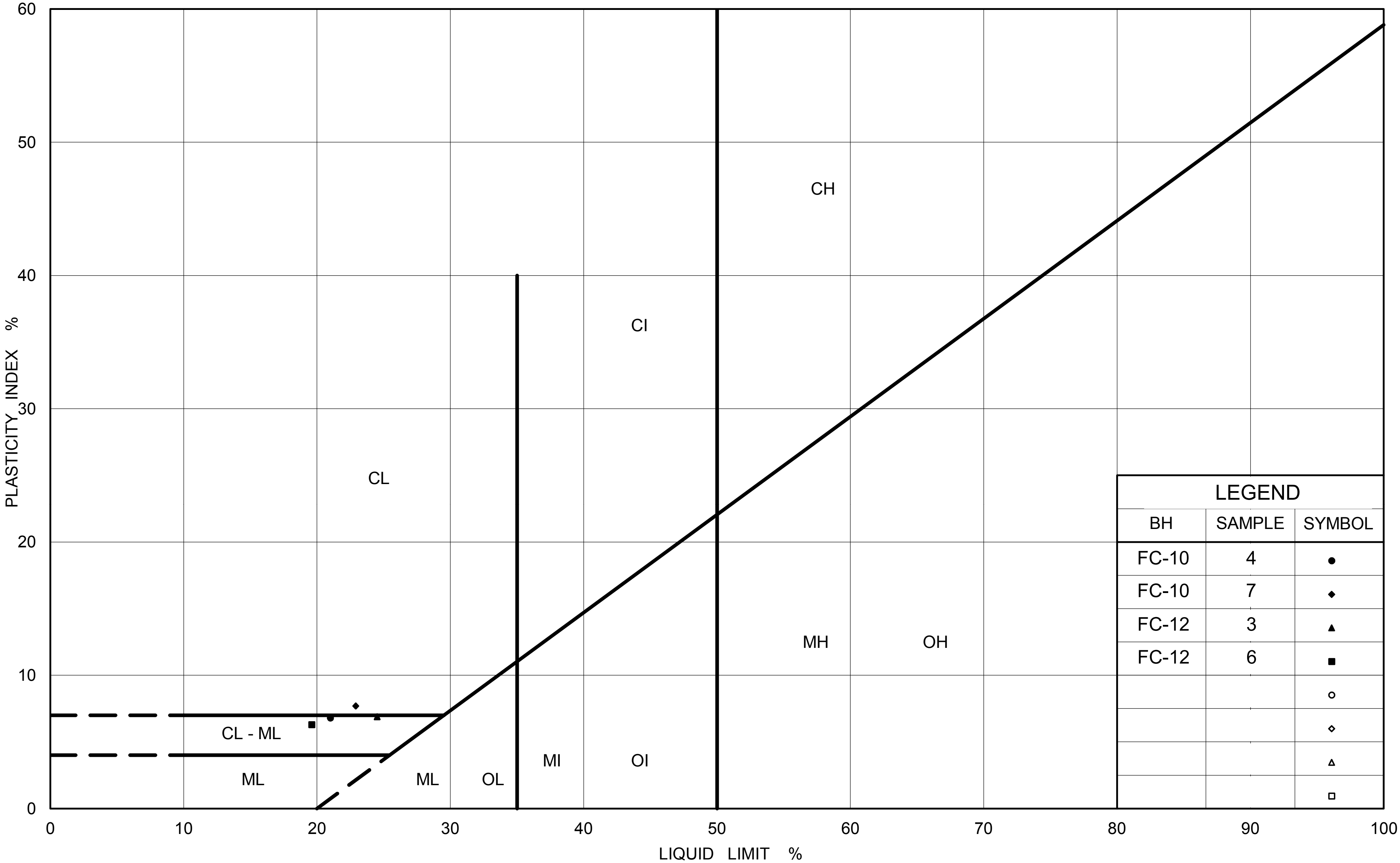
Date: 18-Oct-12

Oct 75, FF-S-21



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PLASTICITY CHART
Clayey Silt to Clayey Silt with Sand Till

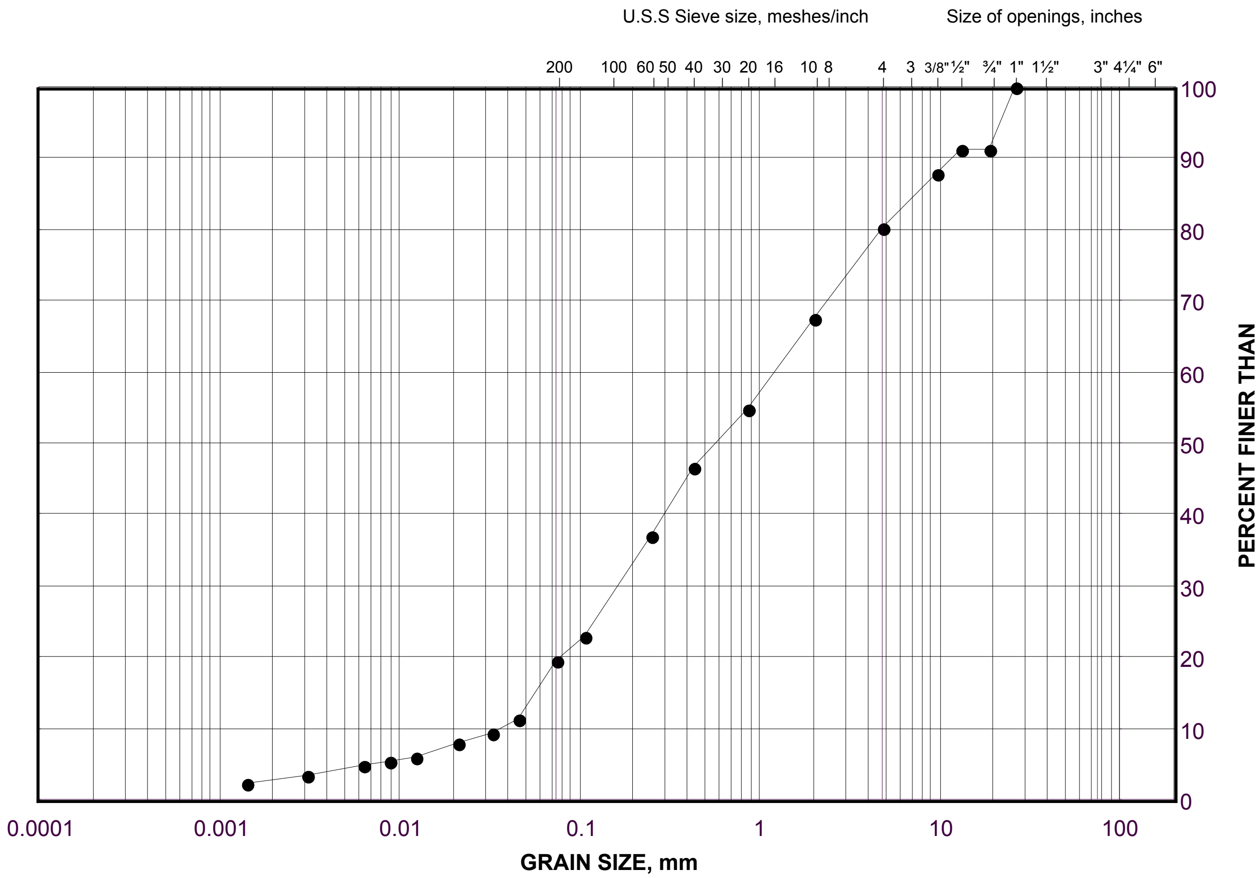
Figure No. B8

Project No. 10-1111-0211

Checked By: TVA

GRAIN SIZE DISTRIBUTION
Gravelly Sand

FIGURE B9



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	FC-9	7B	159.3

Project Number: 10-1111-0211

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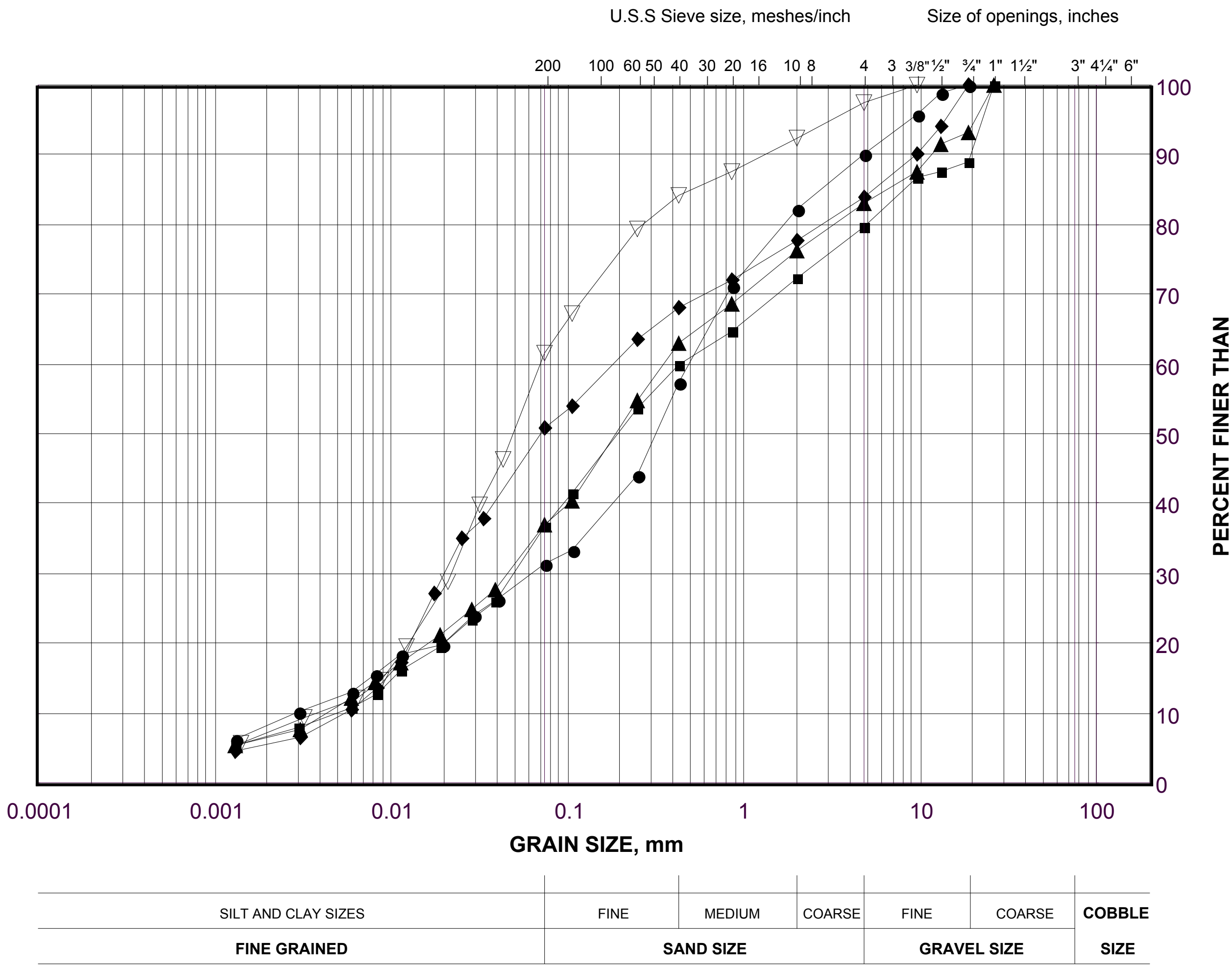
Date: 09-Oct-12

GRAIN SIZE DISTRIBUTION

Silty Sand to Sand and Silt Till

FIGURE B10A



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	FC-6	10	157.3
■	FC-4	6	158.8
◆	FC-1	7	158.8
▲	FC-12	8	157.6
▽	FC-13	9	159.2

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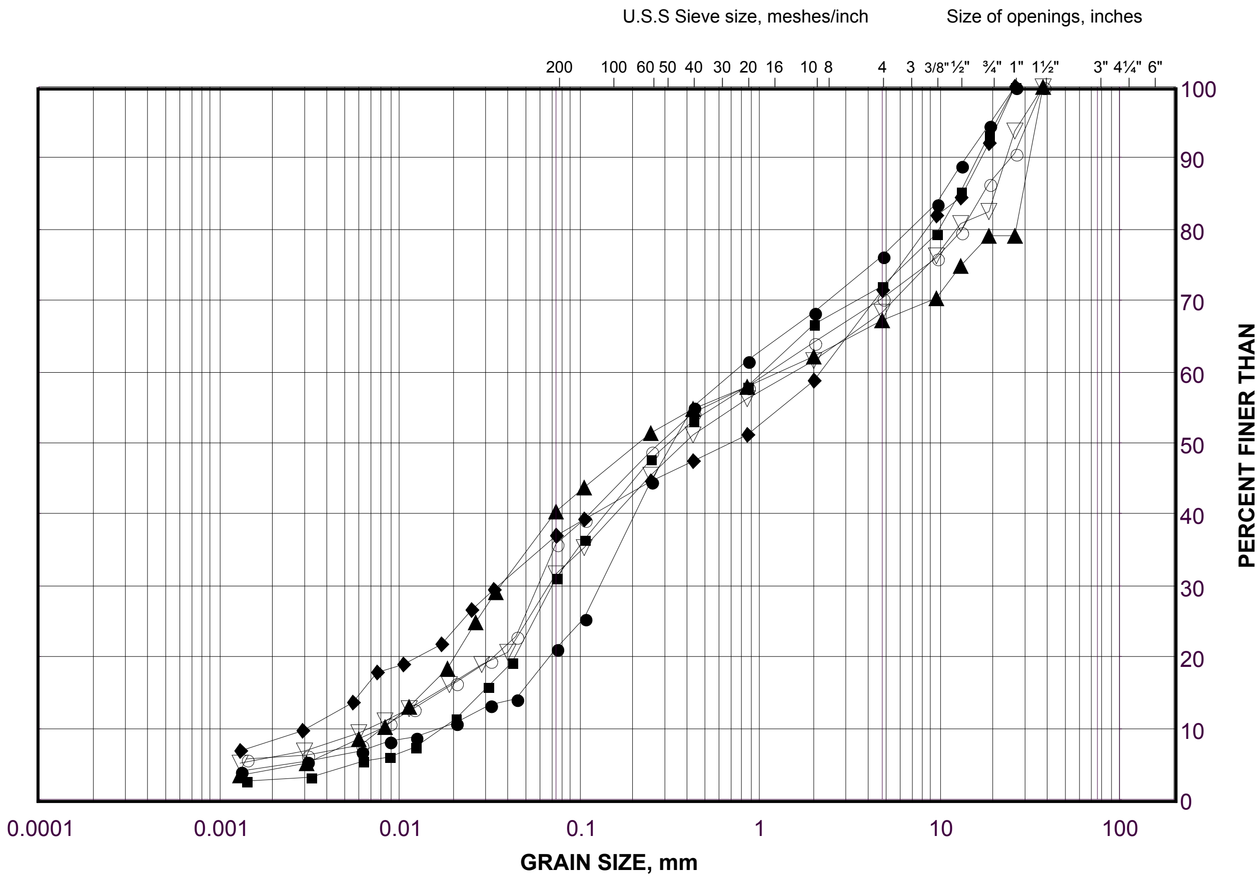
Date: 09-Oct-12

GRAIN SIZE DISTRIBUTION

Silty Sand and Gravel to Sandy Silt and Gravel Till

FIGURE B10B



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	FC-5	10	157.6
■	FC-7	11	156.1
◆	FC-1	12	151.9
▲	FC-2	6	158.2
▽	FC-11	8	158.1
○	FC-8	8	157.6

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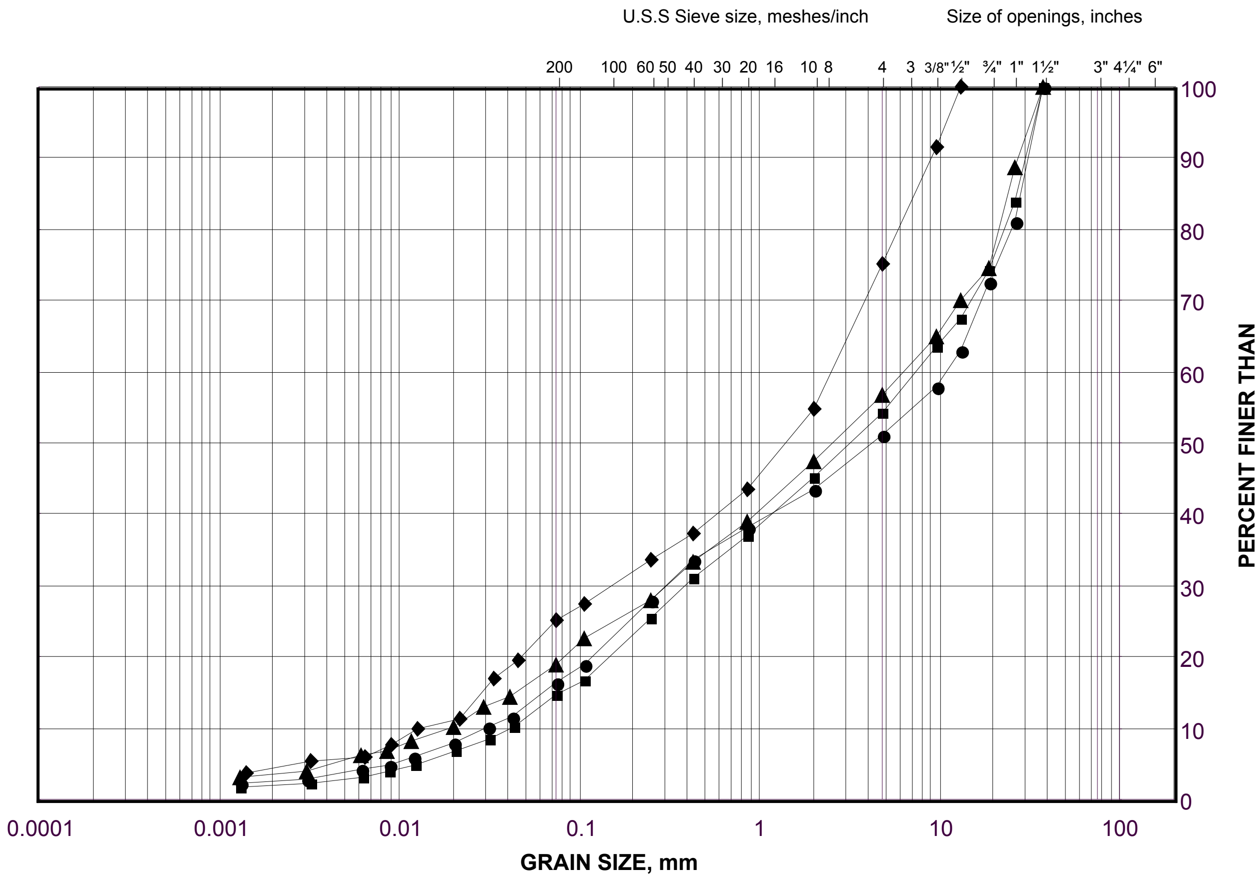
Date: 18-Oct-12

GRAIN SIZE DISTRIBUTION

Gravelly Sand to Sand and Gravel Till

FIGURE B10C



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	FC-11	10	155.6
■	FC-6	11	155.8
◆	FC-13	13	153.3
▲	FC-2	8	155.9

Project Number: 10-1111-0211

Checked By: TVA

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Date: 18-Oct-12

APPENDIX C

Non-Standard Special Provisions

WORKING SLAB - Item No.

Non-Standard Special Provision

1.0 Scope

This Special Provision covers the requirements for the supply and placement of a concrete working slab under structure foundations.

2.0 References

This Special Provision refers to the following standards, specifications or publications:

Ontario Provincial Standard Specifications, Construction

OPSS 902 Excavating and Backfilling - Structures

3.0 Definitions - Not Used

4.0 Design And Submission Requirements - Not Used

5.0 Materials

Concrete for working slabs shall have a minimum 28 day strength of 20 MPa.

6.0 EQUIPMENT - Not Used

7.0 CONSTRUCTION

7.01 Excavation

Excavation for the working slab shall be according to OPSS 902.

7.02 Protection of Founding Soil

Following inspection and approval of the prepared subgrade, a working slab with a minimum thickness of 100 mm shall be placed on the foundation subgrade as specified in the Contract Documents.

7.03 Protection of Founding Bedrock

The surface of the footing founding rock shall be exposed, cleaned and any loose or fractured parts removed so that sound rock is exposed. The working slab shall be placed on the exposed cleaned sound founding rock surface as specified in the Contract Documents.

Thickness of the mass concrete pad shall depend on the slope and irregularities in the exposed founding rock surface. A nominal thickness and a footprint plan view area has been specified on the Contract Documents

7.04 Dewatering

Dewatering shall be carried out according to OPSS 902.

8.0 Quality Assurance - Not Used

9.0 Measurement For Payment - Not Used

10.0 Basis of Payment

10.01 Working Slab - Item

Payment at the Contract price for the above tender item shall be full compensation for all labour, Equipment and Material to do the work.

END OF SECTION

CSP FOR INTEGRAL ABUTMENTS – Item No

Non-Standard Special Provision

Scope

This specification covers the requirements for the installation of the corrugated steel pipes (CSPs) at the integral abutments.

Submission and Design Requirements

All submissions shall bear the seal and signature of an Engineer.

At least two weeks prior to commencement of installation of the abutment piles, the Contractor shall submit to the Contract Administrator, for information purposes only, three (3) sets of the working drawings.

The Contractor shall have a copy of the submitted working drawings on site at all times. Working drawings shall include at least the following:

1. Layout and elevations of the CSPs;
2. Location of reference points, and location of the centroid of each pile with respect to the reference points;
3. Construction sequence and details;
4. Source of the sand fill, and description of placing methods and equipment;
5. Location and details of all temporary bracing and spacers for the piles and CSPs;
6. Method for preventing water and debris from entering the CSP prior to placing sand; and
7. Method for preventing concrete from abutment pours from entering the CSPs during placement.

The Contractor shall be responsible for the complete detailed design of all temporary bracing, including spacers required to maintain the piles, CSP spacing and abutment stems in their specified positions through all stages of construction until the CSPs have been backfilled. All temporary bracing shall be removed.

Material

Corrugated steel pipe

CSP shall be in accordance with OPSS 1801, and shall be from a supplier listed under DSM#4.60.80. The CSP shall be of the diameter and wall thickness specified on the Contract drawings, and shall be galvanized in accordance with CSA G164-M.

CSPs shall be supplied in the lengths and with the end treatments, either square or skew, as specified on the Contract drawings; field cutting and splicing of CSPs will not be permitted. Cut ends shall be neat and free of burrs. The planes defined by the end treatments of each CSP shall be parallel to each other.

Handling and storage of CSPs shall be in accordance with the manufacturer's recommendations. Damaged CSPs shall be rejected. Localized areas of damaged galvanizing on otherwise acceptable CSPs shall be repaired with two coats of zinc-rich paint.

Sand Fill

The sand fill for backfilling the CSP shall meet the gradation requirements of Table 1 below:

Table 1 – Sand Fill Gradation Requirements

MTO Sieve Designation		Percentage Passing by Mass
2 mm	#10	100%
600 μm	#30	80% to 100%
425 μm	#40	40% to 80%
250 μm	#60	5% to 25%
150 μm	#100	0% to 6%

Construction

The sequence of construction shall be in accordance with the working drawings and as follows, unless otherwise approved:

1. Construct levelling pad and place CSPs and spacers.
2. Install piles by driving to design criteria.
3. Place loose sand into 600 diameter CSP.
4. Remove temporary spacers.

The CSP shall be positioned such that the piles are centrally positioned within the CSP. Temporary blocking and bracing shall be used to hold the CSP in position.

The Contractor shall ensure the full perimeters of the tops of all CSPs at each abutment are at the elevation and orientation shown on the working drawings.
The CSP at each pile shall be constructed to the following tolerances:

Criteria	Tolerance
Maximum deviation of CSP from pile centroid	+/- 50 mm
Maximum deviation of any point on the top perimeter of the CSP from the specified elevation	+/- 10 mm

The sand fill shall be placed dry of optimum and free-flowing, completely filling the volume between the CSP and pile. No additional compaction effort other than the action of placing the sand itself shall be applied to the sand fill.

The placing of the sand fill shall be carried out in a manner such as to not damage and displace the CSP.

Basis of Payment

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

END OF SECTION

OBSTRUCTIONS - Item No.

Non-Standard Special Provision

The existing fill and the native cohesionless till contains cobbles and boulders as indicated in the Record of Borehole sheets and as inferred from difficulties in advancing augers/auger grinding. Consideration of the presence of these obstructions must be made in the selection of appropriate equipment and procedures for driving steel H-piles/ pipe piles or advancing caissons such that the design tip levels are achieved.

Basis of Payment

Payment at the lump sum contract price for this tender item shall be full compensation for all labour, equipment and materials for completion of the work.

END OF SECTION

DEWATERING FOR FOUNDATION EXCAVATION - Item No.

Non-Standard Special Provision

The contractor shall be alerted that high artesian groundwater levels (with hydrostatic head measured up to about 5 m above ground surface to Elevation 172 m) were encountered at the proposed Fletcher's Creek Bridges site. It is estimated that the base of temporary excavations for the foundations may be up to 7 m below the creek water level (and up to 14 m below the measured artesian hydrostatic head) estimated at the time of the geotechnical investigation in September 2012. The subsoil conditions generally consist of existing fill, loose to compact sand and silts underlain by clayey silt to clayey silt with sand, underlain by a cohesionless till (comprised of silty sand, sand and silt, gravelly sand, silty sand and gravel, sandy silt and gravel, and sand and gravel). Construction of shallow foundations, pile caps, or excavation and replacement with engineered fill must be carried out in the dry. Dewatering within (and possibly surrounding) the foundation excavations will be required and the excavation shall be kept stable during the work.

Due to the proximity of the proposed abutments to the edge of the Fletcher's Creek, a groundwater cut-off (cofferdam or similar measure) is likely required to minimize dewatering requirements and potential environmental impacts.

Basis of Payment

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

END OF SECTION

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

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