



January 16, 2014

FOUNDATION INVESTIGATION AND DESIGN REPORT

**HIGHWAY 5 OVER HIGHWAY 6 INTERCHANGE STRUCTURE
FUTURE HIGHWAY 5/HIGHWAY 6 INTERCHANGE (IC) AND
ASSOCIATED MUNICIPAL ROADS, CITY OF HAMILTON
MINISTRY OF TRANSPORTATION, ONTARIO
GWP 2112-05-00**

Submitted to:

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Toronto, Ontario
M9W 5P3



REPORT



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PART A

**FOUNDATION INVESTIGATION REPORT
HIGHWAY 5 OVER HIGHWAY 6 INTERCHANGE STRUCTURE
FUTURE HIGHWAY 5/HIGHWAY 6 INTERCHANGE (IC) AND ASSOCIATED
MUNICIPAL ROADS, CITY OF HAMILTON
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1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by IBI Group (IBI) on behalf of the Ministry of Transportation, Ontario (MTO) to provide detail foundation engineering services for the proposed construction of a new interchange structure at the Highway 5 and Highway 6 intersection to replace the existing Highway 5 and Highway 6 at-grade crossings. The proposed work is part of the future Highway 5 and Highway 6 Interchange (IC) and associated Municipal Roads in the City of Hamilton, Ontario, which includes high fill embankments for the Highway 5 and Highway 6 re-alignments and interchange ramps, rock cut slope assessment, culvert extensions and replacements, high mast lighting and overhead signs.

The Terms of Reference (TOR) and the scope of work for the foundation investigation are outlined in MTO's Request for Proposal, dated January 2010, which forms part of the Consultant's Assignment Number (Number 2008-E-0038) for this project. Golder's proposal for foundation engineering services associated with the Highway 5/Highway 6 Interchange structure is contained in Section 6.8 of IBI's Technical Proposal for this assignment. The work has been carried out in accordance with Golder's Supplementary Specialty Quality Control Plan for foundation engineering services for this project, dated September 10, 2012.

This report addresses the investigation carried out for the Highway 5 structure over Highway 6 and the associated approach embankments and wing walls only. The purpose of this investigation is to establish the subsurface conditions at the proposed bridge structure location, including the associated approach embankments, by borehole drilling, rock coring, in situ testing and laboratory testing on selected soil and rock core samples. The investigation area is shown in plan on Drawing 1.

2.0 SITE DESCRIPTION

The existing Highway 5 and Highway 6 intersection is located west of Waterdown and approximately 3 km north of the Highway 403/Highway 6 Interchange at Clappison's Corners in the City of Hamilton, Ontario. The Highway 5 alignment in the project area is oriented generally in a west-east direction extending through the City of Brantford to the west and City of Mississauga to the east. The Highway 6 alignment is oriented generally in a north-south direction connecting with Highway 403 to the south and Highway 401 to the north, and it was last widened in 2005. At the location of the at-grade crossing of Highway 5 and Highway 6, Highway 5 consists of two lanes in both the eastbound and westbound directions with an additional two turning lanes; and Highway 6 consists of three lanes in both the northbound and southbound directions with an additional two turning lanes. The at grade crossing is to be modified to an interchange with the Highway 5 structure crossing over Highway 6 to accommodate future traffic forecast which involves re-alignment of Highway 5 slightly to the north and Highway 6 slightly to the east, in the vicinity of present crossing.

The topography at the site consists of relatively flat terrain which slopes downward further south of the intersection along Highway 6 down the Niagara Escarpment. Highway 6 generally traverses the escarpment in a cut up to 15 m deep through excavated bedrock as it approaches the intersection with Highway 5. Vegetation within the right of way at the intersection is sparse, consisting of grass, small shrubs and isolated treed areas. Surplus material from the Queen Elizabeth Way (QEW) construction sites are stockpiled on the northwest quadrant of the intersection. Commercial facilities are present along the Highway 5 and Highway 6 corridors and residential properties are located along Highway 5 further west of the intersection.

The existing Highway 5 and Highway 6 grades in the general area of the intersection vary between about Elevations 222.5 m and 222.1 m within a slight cut relative to the original ground surface; and the original ground



surface within the un-paved zone at the northeast corner of the intersection varies between about Elevations 221.7 m and 221.6 m.

3.0 INVESTIGATION PROCEDURES

3.1 Foundation Investigation

The field work for the detail foundation investigation of the Highway 5 structure over Highway 6 was carried out between November 13 and 21, 2012 during which time a total of eight (8) sampled boreholes were advanced at the locations of the structure foundation footprints and approach embankments as follows: two (2) boreholes were drilled in the vicinity of the proposed west abutment; two (2) boreholes were drilled in the vicinity of the proposed central pier; two (2) boreholes were drilled in the vicinity of the proposed east abutment; and one (1) borehole each was advanced in the vicinity of the west and east approach embankments. The boreholes, designated as Boreholes BH 1 to BH 6, H5-5 and H5-6, were advanced at the locations shown on Drawing 1 and the Record of Borehole/Drillhole sheets are presented in Appendix A.

The field borehole investigation was carried out using a track-mounted CME 55 drill rig and a truck-mounted CME 75 drill rig, supplied and operated by DBW Drilling Ltd. of Ajax, Ontario. The boreholes were advanced through the overburden using 102 mm outside diameter (O.D.) solid stem augers or 152 mm outside diameter (O.D.) hollow stem augers and NW casing. Where possible, soil samples were obtained at ground surface and at intervals of depth of 0.75 m, using a 50 mm outer diameter (O.D.) split-spoon sampler driven by an automatic hammer in accordance with Standard Penetration Test (SPT) procedures (ASTM D1586-08)¹. Samples of the bedrock were obtained using an 'NQ' size rock core barrel and coring techniques.

The boreholes at the locations of the foundation elements were typically advanced to auger and/or sampler refusal (i.e. inferred bedrock) and bedrock was confirmed by coring in three selected boreholes. The boreholes were advanced to depths ranging from about 5.2 m to 8.8 m below existing ground surface, including coring of bedrock for core lengths between about 3.1 m and 3.5 m in Boreholes BH 2, BH 3 and BH 6. Photographs of the recovered rock samples are provided in Appendix B.

The groundwater conditions and water levels in the open boreholes were observed during the drilling operations. A piezometer was installed in each of Boreholes BH 4 and BH 6 to permit monitoring of the ground water levels at these locations. The installed piezometers consist of 19 mm diameter PVC pipe, with a 1.5 m slotted screen sealed within a filter sand pack at a select depth within the boreholes. The boreholes and annulus surrounding the piezometer pipe above the screen and filter sand pack were backfilled to the ground surface with bentonite pellets. Piezometer installation details and water level readings are described on the Record of Borehole sheets presented in Appendix A. All boreholes in which standpipe piezometers are not installed were backfilled with bentonite upon completion in accordance with Ontario Regulation 903, Wells (as amended).

The field work was observed by members of Golder's engineering and technical staff, who located the boreholes, arranged for the clearance of underground services, observed the drilling, sampling and in situ testing operations, logged the boreholes, and examined and cared for the soil and rock samples. The samples were identified in the field, placed in appropriate containers, labelled and transported to our Mississauga geotechnical laboratory where the samples underwent further visual examination and laboratory testing. All of the laboratory

¹ ASTM D1586-08a – Standard Test Method for Standard Penetration Tests and Split Barrel Sampling of the soil.



tests were carried out to MTO and/or ASTM Standards, as appropriate. Classification testing (water content, Atterberg limits and grain size distribution) was carried out on selected samples. Unconfined compression (uniaxial) strength testing was carried out on selected specimens of the rock core. The results of the laboratory testing are included in Appendix B.

The borehole locations and the ground surface elevations for this current investigation were staked and surveyed by Callon Dietz, a licensed surveying company retained by Golder Associates. Where the location of a borehole was moved due to site constraints, the as-drilled borehole location and ground surface elevation was surveyed by a member of our technical staff, referenced to the original borehole (survey) stake put down by Callon Dietz. The locations given in the Record of Borehole/Drillhole sheets and shown on Drawing 1 are positioned relative to MTM NAD 83 northing and easting coordinates and the ground surface elevations are referenced to Geodetic datum. The borehole locations and ground surface elevations are summarized below.

Borehole No.	Location (MTM NAD 83)		Ground Surface Elevation (m)	Borehole Depth (m)
	Northing	Easting		
BH 1	4797035.5	270882.4	222.1	6.3
BH 2	4797021.9	270895.0	222.1	8.8
BH 3	4797061.7	270892.5	221.6	8.7
BH 4	4797046.4	270915.8	221.7	5.2
BH 5	4797082.1	270915.5	221.7	5.2
BH 6	4797064.2	270934.9	221.6	8.3
H5-5	4797014.8	270871.0	221.6	5.7
H5-6	4797086.5	270942.5	222.1	5.2

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

The study area is located on the Niagara Escarpment², a topographic break that separates the two levels of the Niagara Peninsula. The Niagara Escarpment extends from the Niagara River to the northern tip of the Bruce Peninsula and is generally flanked by landscapes of glacial origin. Capping the Niagara Escarpment is the Lockport Formation, consisting of white, grey and brown dolostone of Silurian Age (Karrow, 1987)³.

Overburden within the study area is comprised primarily of glacial till mapped as the Halton Till which extends as a sheet in the Hamilton area terminating in the Waterdown Moraines west of the Niagara Escarpment. The Halton Till is generally considered a fine-grained diamicton with minor fine-grained lacustrine sediments incorporated within the body of the unit, likely from glacial reworking of underlying lacustrine sediments. The Halton Till also contains cobbles and boulders and in some areas, “boulder pavements” (Watt, 1955)⁴ can be encountered where boulders are nested or concentrated within the till unit.

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

³ Karrow, P.F. 1987. Quaternary Geology of the Hamilton-Cambridge Area, Southern Ontario, Ontario Geological Survey, Report 255. Ministry of Northern Development and Mines, Ontario.

⁴ Watt, A.K. Pleistocene Geology and Groundwater resources of the Township of North York, York County, Ontario Department of Mines, Sixty Fourth Annual report, Volume LXIV, Part 7, 1955.



During the retreat of the last ice sheet, lakes were formed in depressions on the land surface in which were deposited sand, gravel, silt and clay materials. The last major meltwater system along the Escarpment occurred when the Waterdown Moraines were formed. Several channels among the Waterdown Moraines functioned at various times, feeding melt waters southwest toward glacial lakes to create lacustrine and outwash sand deposits.

4.2 Subsurface Conditions

The detailed subsurface soil and groundwater conditions as encountered in the boreholes advanced during this investigation and the results of the laboratory tests carried out on selected soil and bedrock core samples are presented on the Record of Borehole and Drillhole sheets provided in Appendix A. The results of the in situ field tests (i.e. SPT 'N'-values) as presented on the Record of Borehole sheets and in Section 4.0 are uncorrected. The results of the geotechnical laboratory testing are contained in Appendix B.

The stratigraphic boundaries shown on the Record of Borehole sheets and on the stratigraphic profile and cross-sections on Drawings 1 and 2 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. Furthermore, subsurface conditions will vary between and beyond the borehole locations, however, the factual data presented in the record of Borehole and Drillhole sheets governs any interpretation of the site conditions. It should be noted that the interpreted stratigraphy shown on Drawings 1 and 2 is a simplification of the subsurface conditions.

In general, the subsurface conditions in the area of the proposed interchange structure consist of a surficial layer of asphalt or topsoil over a deposit of fill associated with the construction of Highway 5 and Highway 6 and associated ditches. The fill is underlain by a till deposit comprised of clayey silt to clayey silt with sand, underlain by bedrock. The overburden thickness encountered at the boreholes ranges from about 5.0 m at the southeast quadrant of the proposed east abutment to about 6.3 m at the northwest quadrant of the proposed west abutment.

A detailed description of the subsurface conditions encountered in the boreholes at the proposed interchange structure and its approach embankments is provided in the following sections.

4.2.1 Asphalt

An approximately 200 mm thick layer of asphalt was encountered in Boreholes BH 1 and BH 2 drilled through the pavement on existing Highway 6 and an approximately 100 mm thick layer of prime surface treated asphalt was encountered in Borehole H5-6 advanced through a paved driveway near the northeast corner of the Highway 5/Highway 6 intersection.

4.2.2 Topsoil

A layer of sand and silt topsoil between about 0.1 m and 0.7 m thick was encountered at the existing ground surface in Boreholes BH 3 to BH 6 advanced on the eastern area of the existing Highway 5 and Highway 6 intersection, and in Borehole H5-5 drilled on the western approach to the existing intersection.



The Standard Penetration Test (SPT) “N”-values measured within the topsoil are 4 blows and 11 blows per 0.3 m of penetration, indicating a loose to compact relative density. A grain size distribution test on one (1) sample of the topsoil is shown on Figure B1 in Appendix B and the natural water content measured on this sample is about 22 per cent.

4.2.3 Fill

Fill was encountered underlying the asphalt and topsoil in all the boreholes advanced during this current investigation. The surface of the fill deposit was encountered between about Elevations 222.0 m and 220.9 m and the thickness of the fill deposit is between about 0.7 m and 1.9 m.

The fill material is variable in composition. In general, a layer of non-cohesive fill comprised of brown sand and gravel and trace silt was encountered as a granular base below the asphalt layer in Boreholes BH 1, BH 2 and H5-6 which were advanced through the Highway and paved driveway, and a layer of brown silty sand and gravel to sand and silt trace to some clay was encountered below the topsoil in Borehole BH 5 and below clayey silt fill in Borehole BH 6. The non-cohesive fill contains cobbles in places and clayey silt seams in Borehole BH 1 and BH 6. Underlying the non-cohesive fill and topsoil in all boreholes, a deposit of cohesive fill comprised of brown (to black in Boreholes BH 1 and BH 2) clayey silt trace sand to clayey silt with sand and trace to some gravel, trace organics and topsoil was encountered. The cohesive fill encountered in Borehole BH 4 contains sand seams.

The SPT “N”-values measured within the non-cohesive fill in Boreholes BH 5 and BH 6 are 13 blows and 22 blows per 0.3 m of penetration, indicating a compact relative density. The SPT “N”-values recorded within the cohesive fill generally range from 6 blows to 24 blows per 0.3 m of penetration but typically less than 10 blows per 0.3 m of penetration, suggesting that the clayey silt to clayey silt with sand fill has a firm to very stiff (but typically firm to stiff) consistency.

Grain size distribution tests were carried out on one (1) sample of the sand and silt fill and on two (2) samples of the clayey silt with sand portion of the cohesive fill and the results are presented on Figure B2 and Figure B3, respectively, in Appendix B.

Atterberg limits tests were carried out on two (2) samples of the cohesive fill and measured liquid limits of about 29 per cent and 34 per cent, plastic limits of about 14 per cent and 16 per cent, and corresponding plasticity indices of about 15 per cent and 18 per cent. The results of the Atterberg limits tests are shown on a plasticity chart on Figure B4 in Appendix B and indicate that the cohesive fill material consists of clayey silt of low plasticity.

The natural water content measured on two (2) samples of the non-cohesive fill is about 2 per cent and 9 per cent and the natural water content recorded on five (5) samples of the cohesive fill ranges from about 8 per cent to 15 per cent.

4.2.4 Clayey Silt to Clayey Silt with Sand Till

A cohesive till deposit was encountered underlying the fill in all the boreholes advanced at this structure site. The top of the till was encountered at depths between about 1.4 m and 2.1 m below ground surface (between about Elevations 220.7 m and 219.5 m), and the cohesive till deposit is between about 2.9 m and 4.2 m thick,



therefore extending to depths between about 5.0 m and 6.3 m below ground surface (between about Elevations 216.9 m and 215.8 m). Boreholes BH 1, BH 4, BH 5, H5-5 and H5-6 were terminated within the cohesive till upon refusal to further split-spoon penetration and/or auger advancement.

The cohesive till deposit is generally comprised of clayey silt some sand to clayey silt with sand, trace to some gravel containing sand and silty sand seams, sand interlayers, shale fragments and cobbles in places at varying depths/elevations within the deposit.

The SPT “N”-values measured within the cohesive till generally range from 11 blows to 39 blows per 0.3 m of penetration, suggesting that the clayey silt to clayey silt with sand till has a stiff to hard consistency. SPT “N”-values of 12 blows per 0.05 m and 0.23 m of penetration, 18 blows per 0.2 m of penetration and 20 blows per 0.1 m of penetration were recorded prior to termination of Boreholes BH 1 and H5-5 and immediately above the interface of the cohesive till with the underlying bedrock in Borehole BH 2, BH 3 and BH 6.

Grain size distribution tests were carried out on twelve (12) selected samples of the clayey silt to clayey silt with sand till deposit and the results are shown on Figures B5A and B5B in Appendix B.

Atterberg limits tests were carried out on thirteen (13) selected samples of this cohesive till deposit and measured liquid limits ranging from about 23 per cent to 33 per cent, plastic limits ranging from about 13 per cent to 19 per cent, and plasticity indices ranging from about 10 per cent to 17 per cent. These results, which are plotted on a plasticity chart on Figures B6A and B6B in Appendix B, indicate that the till deposit consists of clayey silt of low plasticity.

The natural water content measured on twenty three (23) selected samples of the clayey silt to clayey silt with sand till ranges from about 11 per cent to 43 per cent.

4.2.5 Bedrock/Refusal

Bedrock was encountered and core samples were recovered in Boreholes BH 2, BH 3 and BH 6. The bedrock surface was inferred from split-spoon and/or auger refusal in all other boreholes advanced at this site. The depths to bedrock or refusal below ground surface and the corresponding bedrock surface or refusal elevation are summarized below.

Foundation Element / Approach Embankment	Borehole	Depth to Bedrock Surface / Refusal (m)	Bedrock Surface / Refusal Elevation (m)	Comments
West Approach Embankment	H5-5	5.7	215.9	Auger Refusal
West Abutment	BH 1	6.3	215.8	Split-Spoon and Auger Refusal
	BH 2	5.7	216.4	Bedrock Cored
Central Pier	BH 3	5.2	216.4	Bedrock Cored
	BH 4	5.2	216.5	Auger Refusal
East Abutment	BH 5	5.2	216.5	Auger Refusal
	BH 6	5.0	216.6	Bedrock Cored
East Approach Embankment	H5-6	5.2	216.9	Auger Refusal



In general, the bedrock surface as encountered or inferred in the area of the proposed interchange structure is fairly level to gently sloping upwards from west to east.

Based on a review of the bedrock core samples, the bedrock consists of dolostone of the Lockport formation. In general, the bedrock samples are described as fresh, thin to medium bedded with some laminate, fine grained, non to faintly porous, strong to very strong, grey, with clastic vugs at varying intervals, as presented in the Record of Drillhole sheets in Appendix A, and shown on the photographs of the recovered core samples on Figures B7 to B9 in Appendix B. The degree of weathering of the bedrock samples (i.e. fresh to slightly weathered – W1 to W2), and the strength classification of the intact rock mass based on field identification (i.e. strong to very strong – R4 to R5) are described in accordance with the International Society for Rock Mechanics (ISRM⁵) standard classification system.

The Rock Quality Designation (RQD) measured on the core samples ranges from about 81 per cent to 96 per cent, indicating a rock mass of good to excellent quality as per Table 3.10 of CFEM (2006). The Total Core Recovery (TCR) and Solid Core Recovery (SCR) of samples recovered are between 91 per cent and 100 per cent and between 83 per cent and 94 per cent, respectively.

Three (3) Unconfined Compression (UC) tests (ASTM D7012)⁶ were carried out on selected core samples of the dolostone bedrock obtained in Boreholes BH 2, BH 3 and BH 6 and measured compressive strengths between about 77 MPa and 135 MPa, as summarized in Table B1 and detailed on Figures B10 to B12.

Based on the laboratory UC test, in accordance with Table 3.5 in CFEM (2006)⁷, the dolostone bedrock is classified as strong to very strong (R4, 50 MPa < UCS < 100 MPa to R5, 100 MPa < UCS < 250 MPa).

4.2.6 Groundwater Conditions

The overburden samples taken in the boreholes were generally moist with wet condition in places. The water level observed in the boreholes upon completion of drilling varied between about Elevations 220.5 m and 216.6 m, measured between about 1.1 m and 5.0 m below ground surface except in Boreholes BH 1 and H5-5 that were observed to be dry upon completion of drilling operations. It is noted that water was introduced into the boreholes where bedrock was cored, therefore the water level measured upon completion of drilling may not be representative of the actual groundwater conditions at these locations.

A standpipe piezometer was installed in each of Boreholes BH 4, BH 5, H5-5 and H5-6 to allow monitoring of the groundwater level at the site. Details of the piezometer installation are shown on the Record of Borehole sheet in Appendix A. The groundwater level measured in the piezometer installation on February 13, 2013 is summarized below.

⁵ International Society for Rock Mechanics Commission on Test Methods, 1985. Int. J. Rock Mech. Min. Sci. & Geomech. Abstr. Vol 22, No. 2, pp. 51-60.

⁶ ASTM D7012 – Standard Test Method for Compressive Strength and Elastic Moduli of Intact Rock Core Specimens

⁷ Canadian Geotechnical Society, 1992. Canadian Foundation Engineering Manual (CFEM), 3rd Edition. The Canadian Geotechnical Society, BiTech Published Ltd., British Columbia.



**FOUNDATION REPORT – HIGHWAY 5 OVER HIGHWAY 6 INTERCHANGE
STRUCTURE, GWP 2112-05-00**

Foundation Element	Borehole No.	Ground Surface Elevation (m)	Groundwater Elevation (m)
West Approach Embankment	H5-5	221.6	219.9
Central Pier	BH 4	221.7	220.1
East Abutment	BH 6	221.6	220.1
East Approach Embankment	H5-6	222.1	220.3

It should be noted that the groundwater level in the area is subject to seasonal fluctuations and precipitation events, and should be expected to be higher during wet periods of the year.



5.0 CLOSURE

Mr. John Hagan, P.Eng., a pavement/geotechnical engineer with Golder directed the field drilling program. This report was prepared by Ms. T. Veronica Ayetan, P.Eng., a geotechnical engineer with Golder. Mr. Jorge M. A. Costa, P.Eng., Golder's Designated MTO Contact for this project and Principal with Golder, conducted a technical and quality control review of the report.

GOLDER ASSOCIATES LTD.



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PART B

**FOUNDATION DESIGN REPORT
HIGHWAY 5 OVER HIGHWAY 6 INTERCHANGE STRUCTURE
FUTURE HIGHWAY 5/HIGHWAY 6 INTERCHANGE (IC) AND ASSOCIATED
MUNICIPAL ROADS, CITY OF HAMILTON
MINISTRY OF TRANSPORTATION, ONTARIO
GWP 2112-05-00**



6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

This section of the report provides foundations engineering recommendations for the detail design of a new interchange structure at the Highway 5 and Highway 6 intersection, as part of the future construction of the Highway 5 and Highway 6 Interchange (IC) and associated Municipal Roads in the City of Hamilton, Ontario. The recommendations are based on interpretation of the factual data obtained from the boreholes advanced during the subsurface investigation. The discussion and recommendations presented are intended to provide the designers with sufficient information to assess the feasible foundation alternatives and to carry out the detail design of the structure foundations and approach embankments. Where comments are made on construction, they are provided to highlight those aspects which could affect the design of the project, and for which special provisions may be required in the Contract Documents. Those requiring information on the aspects of construction should make their own interpretation of the factual information provided as such interpretation may affect equipment selection, proposed construction methods, scheduling and the like.

6.1 General

Golder Associates Ltd. (Golder) has been retained by IBI Group (IBI) on behalf of the Ministry of Transportation, Ontario (MTO) to provide recommendations on foundation aspects for the detail design of the Highway 5/Highway 6 Interchange structure. The existing Highway 5 alignment is oriented in a west-east direction and the Highway 6 alignment is oriented in a north-south direction. The existing Highway 5 and Highway 6 intersection is an at-grade crossing which will be modified to an interchange with Highway 5 crossing over Highway 6. It is understood that the new interchange structure is to consist of a two-span pre-cast girder bridge with each span about 28.5 m long to accommodate the proposed re-alignment of the existing Highway 5 slightly to the north and Highway 6 slightly to the east. The proposed abutments and associated wing walls will be located on the west and east sides of the Highway 5 alignment while the central pier will be located in the median of the Highway 6 alignment.

Based on the General Arrangement (GA) Drawing provided by IBI on March 12, 2013, the grade of the proposed Highway 5 bridge deck varies between about Elevations 229.8 m and 229.6 m, requiring west and east approach embankments of about 8 m high relative to the original ground surface at about Elevations 222.1 m and 221.9 m, respectively. The existing Highway 6 pavement surface near the proposed west abutment is at about Elevation 222.1 m whereas the new Highway 6 grade is proposed to be between about Elevations 222.2 m and 221.7 m, therefore requiring slight regrading of the Highway 6 pavement surface beneath the new structure.

6.2 Foundation Options

The new interchange structure will consist of two spans with approximate lengths of 28.5 m each. The proposed bridge is about 38 m wide at the west abutment and about 40 m wide at the west abutment. Within the vicinity of the foundation elements, the subsurface conditions consist of a surficial layer of asphalt or topsoil, underlain by a deposits of loose to compact non-cohesive fill and firm to very stiff cohesive fill, underlain by a cohesive till deposit comprised of clayey silt to clayey silt with sand, which predominantly has a stiff to hard consistency. The till deposit is underlain by dolostone bedrock of generally good to excellent quality and is generally strong to very strong, encountered or inferred between about Elevations 216.9 m and 215.9 m. The groundwater level measured in the piezometers at this site is estimated to be at about Elevation 220.1 m.



Based on the subsurface conditions at this site, both shallow and deep foundations are considered suitable for support of the abutments and central pier for the new interchange structure. A summary of the advantages and the disadvantages associated with each option is provided below, and a comparison of the alternative foundation options based on advantages, disadvantages, risks/consequences and approximate costs is provided in Table 1 following the text of this report.

- **Strip or spread footings founded within the stiff to hard clayey silt to clayey silt with sand till:** Spread footings are considered feasible and suitable to support the new abutments and its associated wing walls and central pier given the competency of the native soils at this site and the relative lower cost of construction. Shallow foundations allow for the use of semi-integral abutments. Spread footing on native soil would require excavation to about 2.4 m deep below ground surface to remove existing pavement and fill materials. Depending on construction staging, temporary roadway protection would likely be required at the abutments and at the central pier to facilitate sub-excavation of the fill/native materials to the founding level.
- **Strip or spread footing “perched” on a Granular ‘A’ pad within the proposed approach embankment fill:** This option could be adopted to support the abutment footings and for an open structure with 2 horizontal to 1 vertical (2H:1V) abutment front slopes. This option may require a longer span to accommodate the granular pad at a higher elevation and may not be economical. In addition, the existing cohesive fill will remain in place and due to its firm to stiff consistency and potential presence of organics and rootlets (at depth) will result in lower geotechnical resistances and could result in greater future differential settlements.
- **Steel H-piles driven to found on the dolostone bedrock:** Driven steel H-piles are suitable and feasible for the support of the proposed abutments and central pier, and would allow for integral abutment construction. At the abutments, it is assumed that the new pile caps would be “perched” within the approach embankments above the original ground surface, thus minimizing the depth of excavation and need for temporary protection systems that would otherwise be required for pile cap construction. There is a relatively minor risk associated with penetrating through the till deposit or the piles “hanging up” within the till deposit as a result of the presence of cobbles and/or shale fragments and possible boulders within the till deposit. The varying bedrock surface within the footprint of the west abutment will result in the potential for variable pile lengths, which will need to be accommodated in the contract documents.
- **Steel tube (pipe) piles driven to found on the dolostone bedrock:** Driven steel tube (pipe) piles could also be considered as a deep foundation option for support of the abutments and central pier, but would only allow for semi-integral abutment construction, as MTO does not readily accept the use of pipe piles for integral abutment construction. At the abutments, it is assumed that the new pile caps would be “perched” within the approach embankments above the original ground surface, thus minimizing the depth of excavation and associated requirements for temporary protection systems that will be required for pile cap construction. Pipe piles are considered to have a greater potential than H-piles for “hanging up” or being deflected away from their vertical or battered orientation due to the presence of cobbles, shale fragments and possible boulders within the glacially-derived soils at this site. As for H-piles, the varying bedrock surface within the footprint of the west abutment will result in the need for variable pile lengths.
- **Caissons founded within the dolostone bedrock:** Caissons socketed into the dolostone bedrock are feasible for support of the new abutments and central pier although they would preclude integral abutments. Caissons would not require caps at/below grade as the caissons could be extended to the



underside of the superstructure, essentially eliminating the need for excavation and the requirements for temporary protection systems. Seams or interlayers of water-bearing non-cohesive soils (such as sand, silty sand or silt) were encountered within the clayey silt to clayey silt with sand till in the boreholes advanced at this site, therefore temporary or permanent liners would be required during caisson installation to control the ground loss and groundwater inflow into the caisson holes from these water-bearing non-cohesive zones/seams. In addition, coring and/or churn drilling techniques could be required to penetrate/socket into the bedrock to the target founding level.

At the abutments and central pier, spread footings founded on the stiff to hard clayey silt to clayey silt with sand till is considered to be the preferred option for the support of the bridge, from a foundation perspective. If greater geotechnical axial resistances are required by the structural designer and/or integral abutments are considered desirable for the proposed structure, the abutments should be supported on deep foundation comprised of steel H-piles.

Recommendations for the various foundation options for the abutments and central pier discussed above for the new interchange structure are provided in the following sections.

6.3 Strip or Spread Footings

6.3.1 Founding Elevations

Strip or spread footings founded on the generally stiff to hard clayey silt to clayey silt with sand till are considered feasible for support of the new abutments (and associated retaining/wing walls) and the central pier. The proposed finished grade of Highway 5 in this area is between about Elevations 229.8 m and 229.6 m and the proposed finished grade for Highway 6 is between about Elevations 222.2 m and 221.7 m, as shown on the GA drawing provided by IBI.

All footings should be founded at a minimum depth of 1.2 m below the adjacent final Highway 6 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 recommended maximum founding elevations for strip or spread footing for support of the new abutments and centre pier and the approximate maximum excavation depth below ground surface are summarized below.

Foundation Element	Reference Borehole No.	Founding Stratum	Founding Elevation (m)	Approximate Maximum Excavation Depth (m)
West Abutment	BH 1 and BH 2	Stiff to hard clayey silt to clayey silt with sand (Till)	219.8	2.3
Centre Pier	BH 3 and BH 4	Stiff to hard clayey silt to clayey silt with sand (Till)	220.0	1.7
East Abutment	BH 5 and BH 6	Very stiff to hard clayey silt to clayey silt with sand (Till)	219.3	2.4

In order to ensure the footings are founded on the stiff to hard clayey silt to clayey silt with sand till deposit, approximately 1.7 m to 2.4 m of the existing fill materials and disturbed surface zone of the till deposit should be sub-excavated and the spread footings founded at the founding elevations presented above. Alternatively, the



founding elevation could be raised to a level 1.2 m below final grade by backfilling the excavation with properly placed and compacted SP 110S13 Granular 'B' Type II (*Aggregates*) at each of the foundation elements.

6.3.2 Geotechnical Resistance/Reaction

Strip or spread footings for the new bridge abutments and centre pier placed on the properly prepared, undisturbed native clayey silt to clayey silt with sand till subgrade or on a pad of compacted granular backfill (i.e. Granular 'B' Type II up to about 1 m thick to backfill the sub-excavated area) at the founding elevations provided in Section 6.3.1, should be designed based on a factored geotechnical axial resistance at ULS of 400 kPa, and a geotechnical reaction at SLS (for 25 mm of settlement) of 300 kPa for a 3 m wide footing and 275 kPa for a 4.5 m wide footing.

The ULS resistance and settlement are dependent on the various factors, such as the embedment depth, footing size, 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*, 2006), using the curves for cohesive soils.

The base of each footing excavation should be cleaned of softened material. 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, as the founding soils 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 placed on the subgrade within three hours 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.

6.3.3 Resistance to Lateral Loads

Resistance to lateral forces / sliding resistance 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 stiff to hard clayey silt to clayey silt with sand till, the coefficient of friction, $\tan \phi'$, can be taken as 0.45 and if constructed on the Granular 'B' Type II material, the coefficient of friction $\tan \phi'$, can be taken as 0.5. These values are unfactored.

6.4 “Perched” Strip or Spread Footing

6.4.1 Founding Elevation

A “perched” strip or spread footing founded on a SP 110S13 Granular 'A' or Granular 'B' Type II (*Aggregates*) pad overlying the existing firm to very stiff clayey silt fill (free of organics and deleterious material) or cohesive till deposit is considered to be feasible option at the new abutments and associated wing walls. However,



considering the presence of firm cohesive fill near the west abutment in assessing the geotechnical resistances for this shallow foundation option, footings perched within the west approach embankment could experience longer-term settlement under embankment loading, resulting in differential settlement between the foundation elements.

For this option, sub-excavation of any existing pavement structures, topsoil, organics and loosened/softened or deleterious material that may be present above or within the existing fill is required prior to construction of the embankment and the granular pad to minimize settlement of the overburden soil due to loading from the embankment. The area to be sub-excavated is defined by a line extending from the top of the pad outward and downward at 1 horizontal to 1 vertical (1H:1V). The sub-excavation area should be backfilled with compacted Granular 'B' Type II or Granular 'A' material. The Granular 'A' or Granular 'B' Type II pad should be a minimum 2 m thick and should extend at least 1 m beyond the plan limits of the footing. A typical perched footing on a Granular 'A' pad is shown on Figure 1. The Granular 'A' or 'B' pad should be constructed in accordance with OPSS 501 (*Compacting*) as revised by SP 105S21 (*Amendment to OPSS 501*).

Spread footings perched on a pad should be provided with a minimum of 1.2 m of soil cover for protection from frost penetration in accordance with OPSD 3090.101 (*Foundation Frost Penetration Depths for Southern Ontario*). It should be noted that the required thickness of conventional soil cover for frost protection of the footing (1.2 m) is measured perpendicular from the face of the abutment slope to the edge of the underside of the footing (it is not simply a vertical dimension when the footing is adjacent to a slope). If adequate soil cover cannot be provided for the footing, rigid styrofoam insulation shall be installed to compensate for the lack of cover and provide protection from frost action. As a guide, the MTO has adopted 25 mm (1 inch) of rigid polystyrene foam insulation as equivalent for every 300 mm reduction in soil cover.

6.4.2 Geotechnical Resistance/Reaction

Considering the firm consistency of the cohesive fill as encountered near the proposed abutments and the above noted granular pad design and construction procedures, a factored geotechnical resistance at ULS of 500 kPa and a geotechnical reaction at SLS of 300 kPa (for 25 mm of settlement) may be used for the foundation design, assuming 3 m wide footings.

The geotechnical resistances provided above are for the loadings that will be applied perpendicular to the surface of the footings. Where the load is not applied perpendicular to the surface of the footing, inclination of the load should be taken into account in accordance with Section 6.7.4 of the CHBDC and its *Commentary* (2006), using the curves for cohesive soils and non-cohesive soil.

6.4.3 Resistance to Lateral Loads

Resistance to lateral force/sliding resistance between the concrete footing and the compacted Granular 'A' or Granular B Type II pad should be calculated in accordance with Section 6.7.5 of the CHBDC. For cast-in-place concrete footings constructed on the compacted Granular 'A' or 'B' Type II pad, the coefficient of friction $\tan \phi$, can be taken as 0.5. This value is unfactored.



6.5 Steel H-Pile or Steel Pipe (Tube) Foundations

6.5.1 Founding Elevations

Driven steel H-pile or steel pipe (tube) pile foundations are feasible for support of the west and east abutments and the centre pier. At the abutments, the pile caps would be perched within the approach embankments above the original ground surface thereby minimizing temporary excavation support requirements for pile cap construction. Steel H-Pile or Pipe (Tube) foundations provide higher geotechnical axial resistances compared to shallow foundations.

The new abutments and centre pier for the proposed structure may be supported on steel H-piles or steel pipe (tube) piles driven to found on the dolostone bedrock. The bedrock surface across the proposed foundation elements is relatively flat except near the west abutment footprint where the bedrock slopes down slightly from south to north. The estimated elevation of the underside of the pile caps, the bedrock surface elevation, the proposed pile tip elevation and associated length of the piles are presented below. At the abutments, the estimated pile length are referenced to the underside of the pile cap and at the centre pier, the pile length is referenced to the proposed finished grade of Highway 6.

Foundation Element	Reference Borehole No.	Estimated Elevation of Underside of Pile Cap (m)	Elevation of Bedrock Surface/Pile Tip (m)	Approximate Pile Length below Pile Cap (m)
West Abutment	BH 1 and BH 2	225.8	216.4 to 215.8	10.0*
Centre Pier	BH 3 and BH 4	222.0	216.4	5.6
East Abutment	BH 5 and BH 6	226.0	216.5	9.5

Note: * Referenced to the lower elevation

Based on the estimated pile length given above, a minimum pile length of 5 m for integral abutment design is achievable at this site. In addition, considering the sloping condition of the bedrock surface along the west abutment and the variability in the top of the bedrock surface inferred at the different foundation elements, it is recommended that provision be made in the Contract Documents to deal with piles of varying lengths as presented above.

For driven piles, consideration must be given to the potential presence of cobbles and boulders within the glacially-derived soils at this site, as well as the potential for damage to the pile tips during seating on the bedrock. The piles should be reinforced at the tip 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, or rock points to reduce the potential for damage to the piles during driving. All piles construction should be carried out in accordance with OPSS 903 (*Deep Foundations*). If steel pipe piles are used, 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.

If corrugated steel pipes (CSPs) are 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. As discussed further in Section 6.11 (Design and Construction



Considerations), vibration monitoring is anticipated to be required during deep foundation construction activities due to the commercial and residential properties present near the site.

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*). Alternatively, rigid polystyrene insulation could be used to reduce the required thickness of soil cover. As a guideline for design, the MTO has adopted a thickness of 25 mm of rigid polystyrene insulation assumed to be equivalent to about 300 mm of soil cover.

6.5.2 Geotechnical Axial Resistance/Reaction

For steel HP 310x110 piles driven to the bedrock surface/estimated pile tip elevations provided in Section 6.5.1, the factored geotechnical axial resistance at Ultimate Limit States (ULS) of 2,000 kN may be used for design. This value represents a structural limitation for the pile rather than a geotechnical limitation. In this case, bedrock is an unyielding material and the condition for 25 mm of settlement at Serviceability Limit States (SLS) is typically much higher than the ULS value and therefore the ULS case governs.

Similar axial resistances may be used in the design of closed-end, concrete-filled, 324 mm (12 ¾ in.) diameter steel pipe piles having a minimum wall thickness of 9.5 mm (3/8 in.).

At both abutments, the loading from the construction of the new approach embankments will cause consolidation settlement of the underlying firm to very stiff cohesive (clayey silt to clayey silt with sand) fill and stiff to very stiff portion of the cohesive till deposit since these deposits will remain in place. Settlement of the cohesive deposits relative to the stiff piles will result in the development of downdrag loads (negative skin friction) on the piles if the piles are installed prior to completion of this settlement and if the piles are end-bearing on bedrock. The structural design of the abutment should be based on the full downdrag load acting on the piles and it is recommended that a downdrag load of 100 kN be included in the design of the piles. The structural capacity of the piles must be checked for the factored dead and downdrag loads in accordance with Section C6.8.4 of the *Commentary* to the CHBDC (2006) for ULS conditions.

6.5.3 Set Criteria

Pile installation should be in accordance with OPSS 903 (*Construction Specification for Deep Foundations*). For piles driven to bedrock, the pile termination or set criteria will be highly dependent on the pile driving hammer type, helmet, selected pile and length of pile. The set criteria must therefore be established through a variety of methods, including empirical correlations and wave equation analyses, at the time of construction after the piling equipment is known to ensure that the piles are not overdriven and to avoid possible damage to the piles. Based on our experience, consideration should be given to the following preliminary criteria:

- The piles should be driven to an initial set equal to or greater than 10 blows per 12 mm of penetration (unless abrupt peaking occurs) using a hammer with rated energy of about 50 kilojoules, but not exceeding 60 kilojoules.
- On reaching the required set, the hammer energy should be reduced by about 75 per cent and the pile should be re-driven by increasing the hammer energy slowly up to a maximum rated energy over about 40 blows to improve the process of seating the pile on the bedrock surface.



- A final set of no less than 10 blows per 12 mm of penetration should be obtained at the maximum hammer energy. Provision should be made to re-tap all piles to confirm the set after adjacent piles have been driven.

6.5.4 Pile Driving Note

The pile driving note to be added to the Contract Drawing for this project is Note 5 in Clause 3.3.3 of MTO's Structural Manual (2008):

- "Piles to be driven to bedrock".

6.5.5 Resistance to Lateral Loads

The design of piles subjected to lateral loads should take into account such factors as the batter of the piles (if any), the relative rigidity of the pile to the surrounding soil, the fixity condition at the head of the pile (pile cap level), the structural capacity of the pile to withstand bending moments, the soil resistance that can be mobilised, the tolerable lateral deflections at the head of the pile and pile group effects. For a longer, more flexible pile, the maximum yield moment of the pile may be reached prior to mobilisation of the lateral geotechnical resistance. For design purposes, both the structural and geotechnical resistances should be evaluated to establish the governing case.

Lateral loading could be resisted fully or partially by the use of battered piles. For vertical piles, the resistance to lateral loading will have to be derived from the soil in front of the piles. Where integral abutments are under consideration, there will also be a requirement for the piles to move sufficiently to accommodate the bridge deck deflections. The resistance to lateral loading in front of a single pile may be calculated using subgrade reaction theory where the coefficient of horizontal subgrade reaction, k_h (kPa/m), is based on the following equations (CFEM, 1992 as referenced in Section 6.8.7.1 of the *Commentary* to CHBDC, 2006):

For non-cohesive soils:

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

where:

n_h	=	constant of subgrade reaction (kPa/m)
z	=	depth (m)
B	=	pile diameter or width (m)

For cohesive soils:

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

where:

s_u	=	undrained shear strength of the soil (kPa)
B	=	pile diameter or width (m)

We understand that an integral abutment foundation design is being considered for this bridge. Where the 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 will be generally



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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 (CFEM, 1992) and s_u (values estimated from insitu testing and empirical correlations) may be assumed in the structural analyses. The soil stratigraphy has been generalized (assuming that the topsoil and pavement structures will be sub-excavated and replaced with earth/granular fill) 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	Foundation Element	Elevation (m)	n_h (kPa/m)	s_u (kPa)
Loose sand within CSP	Where applicable	Upper 3 m of pile	2,200	--
Embankment fill (assumed to be compact earth/granular fill)	West Abutment Centre Pier East Abutment	Above 221.9 to 220.9	6,600	--
Compact silty sand and gravel to sand and gravel fill	West Abutment East Abutment	221.9 to 220.9	6,600	--
Firm to very stiff clayey silt to clayey silt with sand fill	West abutment Centre Pier East Abutment	221.7 to 220.9	--	65
Stiff to hard clayey silt to clayey silt with sand till	West abutment Centre Pier East Abutment	Below 220.3 to 219.5	--	150

For design of a single HP 310x110 pile surrounded by a 3 m long CSP liner below the pile cap and driven to the bedrock surface elevations in Section 6.5.1, the estimated maximum factored geotechnical lateral resistance at ULS of about 125 kN and a geotechnical lateral resistance at SLS of about 50 kN for 10 mm of deflection at the pile cap level may be used for design. These were calculated for a vertical free-headed HP 310x110 pile, based on analysis using the commercially available program LPILE Plus (Version 5.0) produced by Ensoft Inc. The structural capacity of the pile should be checked and verified by the structural engineer.

Group action for lateral loading should also be considered when the pile spacing in the direction of the loading is less than eight pile diameters. Group action can be evaluated by reducing the coefficient of lateral subgrade reaction in the direction of loading by a reduction factor, R (NAVFAC DM-7.2), as follows:

Pile Spacing in direction of Loading (d = Pile Diameter)	Subgrade Reaction Reduction Factor
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 listed above.



6.6 Caisson Foundations

As an alternative, consideration could be given to the use of caissons socketted into the bedrock for support of the bridge abutments and pier. If caisson foundations are adopted for support of any of the foundation elements, it is recommended that a temporary or permanent liner be used to support the soils during construction, to minimize disturbance and loss of ground if water-bearing non-cohesive soil zones/seams are encountered within the overburden below the groundwater level, although limited non-cohesive seams/zones were encountered during this current investigation. Consideration must also be given to the potential presence of cobbles and shale fragments, and potentially boulders, within the cohesive till deposits.

6.6.1 Founding Elevations

Caissons should be socketted a minimum of 2 m into the good quality dolostone bedrock encountered at this site. The estimated elevation of the base of the caisson socket to be used in design is presented below.

Foundation Element	Reference Borehole No.	Bedrock Surface Elevation	Caisson Base Elevation (m)
West Abutment	BH 1 and BH 2	216.4 to 215.8	213.8
Centre Pier	BH 3 and BH 4	216.4	214.0
East Abutment	BH 5 and BH 6	216.5	214.5

The dolostone bedrock is generally of good to excellent quality and is predominantly strong to very strong, with unconfined compressive strengths in the range of about 77 MPa to 135 MPa, therefore the sockets will have to be advanced into the rock by churn drilling or rock coring along the entire length of the caissons at the foundation elements. In addition, there could be some difficulty with achieving an adequate seal when socketing the large diameter caissons into the bedrock where there is sloping bedrock surface such as at the west abutment. If caissons are adopted, it is recommended that an NSSP be included in the Contract Documents to describe to the Contractor the strength and characteristics of the bedrock; an NSSP is included in Appendix C for this purpose.

The caisson caps for the new foundation elements 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 caps are positioned at the top of the columns.

6.6.2 Geotechnical Axial Resistance/Reaction

Caissons at this site will derive their axial resistance mainly from the shaft resistance of the bedrock socket. The contribution from end-bearing should be neglected due to the difficulties in cleaning and inspecting the base of the sockets. The factored geotechnical axial resistance at ULS for the following diameter caissons socketted a minimum of 2 m into the good quality dolostone bedrock at or below the caisson base elevations recommended in Section 6.6.1 are given below:



Foundation Element	Caisson Diameter (m)	Factored Geotechnical Axial Resistance at ULS (kN)	Geotechnical Reaction at SLS (for 25 mm of Settlement) ¹ (kN)
West abutment	1.2	6,000	N/A
Centre Pier	1.5	9,500	N/A
East Abutment			

Note: 1. The geotechnical reaction at SLS for 25 mm of settlement on dolostone bedrock will be greater than the factored axial resistance at ULS for caisson into bedrock and as a result the SLS condition does not apply.

As noted in Section 6.5.2, the loading from the new embankment will cause settlement of the underlying firm to very stiff cohesive fill and the stiff to very stiff portion of the cohesive till, and as a result downdrag loads will need to be taken into account in the design of the caissons at the abutments. The structural design of the abutment caissons should be based on the full downdrag load acting on the caissons; it is recommended that a downdrag load of 250 kN and 300 kN be included in the design of the 1.2 m and 1.5 m diameter caissons, respectively. The structural capacity of the caissons must be checked for the factored dead and downdrag loads in accordance with Section C6.8.4 of the *Commentary* to the CHBDC for ULS conditions.

6.6.3 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 6.5.5.

6.7 Retained Soil System (RSS) Walls

It is understood that mechanically-reinforced soil retaining systems (retained soil system or RSS walls) are proposed as wing walls/retaining walls on both sides and in front of the west and east abutments (refer to Drawing 1). Based on the GA drawing provided by IBI, there is a 2 horizontal to 1 vertical (2H:1V) concrete paved slope in front of the abutments. The RSS retaining walls are to be designed for high performance and appearance in accordance with MTO Special Provision SP 599S22 (*Retained Soil System*).

6.7.1 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. The footing for the front facing and the RSS mass should be founded below any softened/loosened materials. For this site, it is recommended that the existing pavement structures, topsoil, and the non-cohesive and cohesive fill within the proposed RSS wall footprint be sub-excavated (to between about Elevations 220.3 m and 219.5 m); and replaced with engineered fill consisting of SP 110S13 Granular 'A' or Granular 'B' Type II to about Elevation 221.0 m at the west and east abutments. The RSS soil mass and facing footing may be supported on the Granular 'A' or Granular 'B' Type II engineered fill (at Elevation 221.0 m) or on the native stiff to hard clayey silt to clayey silt with sand till at about Elevation 220.0 m at the west abutment and at about Elevation 219.5 m at the east abutment.



The facing footing should be placed on a minimum 300 mm thick layer of compacted SP 110S13 Granular 'A', as shown on Figure 5.2 in the MTO RSS 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.

6.7.2 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, assuming that the reinforced mass and facing footing are founded on engineered fill or native till subgrade at the founding elevations given in Section 6.7.1.

RSS Wall Location	Maximum Wall Height (m)	Factored Geotechnical Resistance at ULS (kPa)	Geotechnical Reaction at SLS (for 25 mm Settlement) (kPa)	Founding Soil Condition
West and East Abutments	8.5	700	500	Granular 'B' Type II engineered fill or stiff to hard clayey silt to clayey silt with sand till

6.7.3 Global Stability

The static and seismic global slope stability of RSS walls at the proposed Highway 5 structure over Highway 6 has been analyzed using the commercially-available program SLIDE (Version 6.0), produced by Rocscience Inc., employing the Morgenstern-Price method of analysis. For all analyses, the factor of safety of numerous potential failure surfaces was computed in order to establish the minimum Factor of Safety (FoS). A target FoS of 1.5 against deep-seated global instability of the RSS walls is normally accepted by MTO for wall design under static conditions; under seismic conditions, a target FoS of 1.1 has been used. These factors of safety are considered appropriate for the RSS 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 analysis, as given below, were estimated from empirical correlations using the results of in-situ Standard Penetration Tests (SPTs) (Bowles, 1984) and geotechnical classification testing. The groundwater table in the analyses was modelled at Elevation 220.1 m.



Soil Type	Unit Weight (kN/m³)	Undrained Shear Strength (kPa)	Angle of Internal Friction, ϕ' (degrees)
Existing (clayey silt to clayey silt with sand) fill (where present)	20	65	--
New earth/engineered fill	20	--	30
Granular 'A' pad (proposed)	21	--	35
Stiff to hard clayey silt to clayey silt with sand till	21	150	--

A RSS wall section was analyzed for the maximum wall height (8.5 m) anticipated as shown on the GA drawing provided by IBI, dated March 12, 2013. In this analysis, the height of the RSS wall was considered to extend from the underside of the RSS panel footing (bottom of the front facing footing) at about Elevation 220 m to the top of the proposed Highway 5 pavement (between Elevations 229.6 m and 229.7 m for the west and east abutments, respectively). The analysis was carried out considering a minimum of 0.8 m of soil cover over the front facing footing as the results are sensitive to the buried depth of the wall.

Given the proposed RSS wall height, the minimum reinforced width of RSS wall required to obtain a FoS of 1.5 or greater against deep-seated global instability was calculated. The ratio of minimum reinforced mass width to reinforced wall height for this section is the “typical” ratio that is used by wall designers (i.e. approximately 0.7 times the wall height). The contract drawings will need to specify the width of the reinforced soil mass. The result of the analysis for the RSS wall adjacent to the abutment wall (about 8.5 m high wall) is shown on Figure 2 for the static condition, and indicates a factor of safety greater than 1.5 for global instability.

6.7.4 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, $\tan \phi'$, between the compacted granular fill of the RSS wall and the compacted Granular 'A' or Granular 'B' Type II material over native stiff to hard cohesive till may be taken as 0.55. For the precast concrete footing supporting the front facing placed on compacted granular material, the coefficient of friction, $\tan \delta$, can be taken as 0.5. These values are unfactored.

6.8 Lateral Earth Pressures for Design

The lateral earth pressures acting on the new abutment stems and any associated wing/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 walls. These design recommendations and parameters assume level backfill and ground surface behind the abutment 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 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*, as applicable.
- 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 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 Figure C6.20(b) of the *Commentary* to the CHBDC).
- For restrained structures, the pressures are based on the proposed embankment fill materials and the following parameters (unfactored) may be used assuming the use of earth fill:

	Earth Fill
Soil Unit Weight	20 kN/m ³
Coefficient of static lateral earth pressure	
Active, K_a	0.33
At rest, K_o	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	0.27	0.27
At rest, K_o	0.43	0.43

If the wall support and superstructure allow lateral yielding of the stem, active earth pressures may be used in the geotechnical design of the structure. If the abutment support does not allow lateral yielding (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.

6.9 Seismic Site Coefficient

For seismic design purposes, the Site Coefficient (S) for this site may be taken as 1.0, consistent with Soil Profile Type I. The soil profile is based on the guidelines in Section 4.4.6 and Table 4.4 of the CHBDC (2006).

6.9.1 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 Hamilton is 0.05. Based on the subsurface conditions at this site, and the Site Coefficient, ($S = 1.0$ for Soil Profile I from Table 4.4 of CHBDC), the peak horizontal ground acceleration (PHA) is equal to $0.05g$ at the ground surface and a design seismic coefficient value of $0.025g$ (50% of the PGA). 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.

6.10 Approach Embankment Design

Based on the GA Drawing provided by IBI, the proposed road grade for the Highway 5 approaches to the interchange structure will be about Elevation 229.6 m at the west embankment and about Elevation 229.7 m at the east embankment, requiring placement of up to about 8 m of new fill above existing grade within the limits of the approach embankments. The current Highway 6 grade is at about Elevation 222.1 m (as recorded in Boreholes BH 1 and BH 2 advanced through existing Highway 6 pavement) and the proposed grade for the Highway 6 in the vicinity of the interchange structure is between about Elevations 222.2 m and 221.7 m. As a result, there is essentially no grade change planned for the Highway 6 grade, but up to about 0.5 m of fill may be required near the proposed centre pier for re-grading of Highway 6.

The results of stability and settlement analysis for the new approach embankments are presented in the following sections. Recommendations are also provided for subgrade preparation and embankment construction.

6.10.1 Approach Embankment Stability

Static and seismic slope stability analyses of the proposed Highway 5 approach embankments were carried out for the critical sections, which corresponds to the greatest embankment height at 8 m, using the commercially



available program Slide (produced by Rocscience Inc.) to check that the target minimum Factor of Safety (FoS) 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 considering the design requirements and the field data available.

The soils encountered below the proposed approach embankments consist primarily of pavement structures (asphalt and granular (sand and gravel) base) or topsoil underlain by compact sand and silt fill and firm to very stiff clayey silt to clayey silt with sand fill, underlain by stiff to hard clayey silt to clayey silt with sand till, which in turn is underlain by bedrock. It is recommended that the existing pavement structures, topsoil and all organics/loosened materials up to about 0.8 m thick (where encountered) should be removed and replaced with compacted earth/engineered fill.

The soil parameters used in the analysis, as given below, were estimated from empirical correlations suggested by Kulhawy and Mayne (1990) using the results of in-situ Standard Penetration Tests (SPT) and geotechnical classification testing, and the results were tempered by engineering judgement based on precedent experience in similar soils. For the purpose of analysis, earth and / or engineered fill materials has been considered for the construction of the approach embankments. It has been assumed that the embankments will be constructed with side slopes at 2 horizontal to 1 vertical (2H:1V). The groundwater table in the analyses was taken to be at Elevation 220.1 m.

Soil Type	Unit Weight (kN/m ³)	Undrained Shear Strength (kPa)	Angle of Internal Friction, ϕ' (degrees)
New earth/engineered fill	20	--	30
Existing (firm to very stiff clayey silt to clayey silt with sand) fill	20	65	--
Stiff to hard clayey silt to clayey silt with sand till	21	150	--

Assuming that appropriate subgrade preparation and proper placement and compaction of the new embankment fill materials have been carried out, including sub-excavation of topsoil, pavement structure materials and loosened/softened fill materials, the proposed 8 m high embankments with side slopes at 2H:1V will have a Factor of Safety greater than 1.3 against deep-seated slope instability. A simplified representation of the Highway 5 west approach embankment and the results of the stability analysis are shown on Figure 3.

6.10.2 Approach Embankment Settlement

Settlement of the proposed Highway 5 approach embankments will occur due to compression of the new embankment fill (up to about 8 m high at the west and east embankments), as well as due to compression of the existing fill and underlying native soils due to the new 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 (cm²/s),



required in the time-rate analysis was established using the results of the laboratory tests and/or estimated from the Unified Facilities Criteria U.S. Navy (1986) correlation with liquid limits assuming over consolidated soils.

The values of the parameters used in the analyses of settlement for both the west and east approach embankments as given below are based on the soil conditions encountered in the boreholes advanced for the approach embankments and in the boreholes closest to the abutments. The thickness of the compressible foundation soils is up to about 5.5 m, and as such, the settlements along the length of the approach embankment will similarly vary. Given that the analyses were carried out at the critical sections, the settlements generally represent the maximum estimated value along the approach embankments. The settlement assessment assumes that the existing pavement structures (consisting of asphalt and granular base (silty sand and gravel to sand and gravel fill)), topsoil and all organics/loosened materials up to about 0.8 m thick (where encountered) have been removed and replaced with compacted earth/engineered fill prior to embankment fill placement. The groundwater table in the analyses was taken to be at Elevation 220.1 m.

Soil Deposit	Bulk Unit Weight (kN/m ³)	Estimated Deformation Properties (kPa ⁻¹)
Existing (firm to very stiff clayey silt to clayey silt with sand) fill	20	$m_v = 1.2 \times 10^{-4}$
Stiff to hard clayey silt to clayey silt with sand till	21	$m_v = 7.2 \times 10^{-5}$

Below the approach embankments, where an up to about 5.5 m thick deposit of firm to very stiff clayey silt to clayey silt with sand fill and the stiff to hard clayey silt to clayey silt with sand till would remain in place, the results of the analyses indicate that up to about 75 mm of time-dependent settlement will occur due to primary consolidation of the cohesive deposits.

Based on an estimated coefficient of consolidation (c_v) equal to 3.9×10^{-2} cm²/s for the 5.5 m thick cohesive deposits and the imposed loading conditions, and assuming one-way drainage of the cohesive deposits, it is estimated that about 90 per cent of the primary consolidation settlement will be completed within about 75 days. The magnitude of secondary consolidation (creep) settlement for the cohesive deposits are considered to be negligible based on the overconsolidated condition of the glacial derived (till) overburden soils over a twenty-year (20-year) period following completion of construction.

6.10.2.1 Settlement of Embankment Fill

A maximum thickness of about 8 m of fill will be required for the construction of the approach embankments for the proposed interchange structure at the Highway 5/Highway 6 interchange. Provided that the new fill is comprised of suitable earth or granular fill meeting the requirements of and placed and compacted in accordance with SP 206S03 (*Earth Excavation and Grading*), 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.

6.10.2.2 Mitigation of Settlement

As discussed in Sections 6.10.2, up to about 75 mm of time-dependent settlement of the cohesive (fill and till) deposits is anticipated below the west and east approach embankments. 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) and to reduce some of the effects of potential differential settlement, it is recommended that the west and east approach embankments be constructed early in the construction schedule such that the approach embankments are allowed to preload the subsurface deposits 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 is estimated to be about 25 mm over a twenty (20) year period following completion of construction.

6.10.3 Subgrade Preparation and Embankment Construction

The existing overburden encountered at all the boreholes advanced for this current investigation consists of pavement structures, topsoil and deposits of non-cohesive and cohesive fill materials. In order to achieve adequate performance of the new approach embankments (i.e. reduce the potential for post-construction differential settlement and to achieve adequate stability of the new embankments), it is recommended that prior to the placement of the new embankment fill, all topsoil, organic matter, asphalt, granular base (silty sand and gravel to sand and gravel) and soft/loose fill should be stripped from the approach embankment areas. Embankment fill should be placed and compacted in accordance with SP 206S03 (*Earth Excavation and Grading*) and SP 105S21 (*Amendment to OPSS 501*), with inspection and field density testing carried out by qualified personnel during placement operations to confirm that appropriate materials are used and that adequate levels of compaction are achieved.

As the new embankment fill sections are about 8 m high, a 2 m wide bench is not required, but could be incorporated into the side slopes, as per OPSD 202.010 (*Slope Flattening*) to reduce the potential for erosion of the slope surfaces. 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 the 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.

We understood that surplus material from the Queen Elizabeth Way (QEW) construction presently stockpiled at the northwest quadrant of the existing Highway 5 and Highway 6 intersection is available for use for construction of the high fill embankments in the vicinity of the proposed interchange structure. Comments on the suitability of the stockpiled materials and recommendations pertaining to the construction of the high fill embankments using the stockpiled soil are presented in Golder (2013).

6.11 Design and Construction Considerations

6.11.1 Overburden Excavation

The foundation excavations at the abutments and centre pier for spread footings (or pile cap construction at the centre-pier) will extend through the existing fill into the stiff to hard clayey silt to clayey silt with sand till deposit up to a depth of about 2.4 m below the ground surface. 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) Ontario Regulation 213 (*Construction Projects*). The existing fill materials are classified as Type 3 soil and the 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 1 horizontal to 1 vertical (1H:1V) through the fill materials and through the till deposit to within 1.2 m of the bottom of the excavation.

6.11.2 Control of Groundwater and Surface Water

The groundwater level measured at this site is generally between about 1.5 m and 1.6 m below existing ground surface, corresponding to about Elevation 220.1 m. Construction of shallow foundations or pile caps for the bridge structure will be slightly above or at the groundwater level and groundwater inflow into the excavations is expected to be relatively minor, especially during drier periods of the year. Water inflow should be expected into the foundation excavations, particularly during wet months, however, it is anticipated that water inflow can be handled by pumping from filtered sump pumps placed at the base of the excavations. All shallow foundations should be constructed in dry conditions whereby the groundwater level has been depressed to at least 0.3 m below the base of the excavations prior to construction of the foundations. All surface water should be directed away from the excavations.

6.11.3 Temporary Excavation Support

Temporary excavation support may be required to facilitate the construction of the new abutments and RSS walls at the existing Highway 5 and Highway 6 intersection and to maintain traffic lanes in operation during the construction period. 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 any existing structure, as well as any adjacent utilities, can tolerate this magnitude of deformation.

A protection system along the proposed abutments and centre pier may be required to allow for excavations to depths of up to approximately 2.4 m below present grade through the Highway 5 and/or Highway 6 pavement structures and underlying fill materials and till deposit. The selection and design of the protection systems will be the responsibility of the Contractor. For conceptual planning and costing purposes, 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. However the presence of cobbles and shale fragments, and possibly boulders within the overburden may affect the installation of an interlocking sheet pile system. An interlocking sheetpile system would contribute to both ground and, where applicable, groundwater control of seepage from non-cohesive zones or interlayers/lenses within the cohesive deposits. 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 if non-cohesive soils/lenses are encountered.

The sheetpiles or soldier piles would have to be socketted to sufficient depth to provide the necessary passive resistance for the retained soil height of up to about 2.2 m. Lateral support to the sheetpiles or soldier piles could be provided in the form of struts, rakers or temporary anchors.



6.11.4 Subgrade Protection

The clayey silt to clayey silt with sand till 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.

6.11.5 Obstructions During Pile Driving / Caisson Installation

Cobbles and shale fragments were encountered at varying depths in the boreholes drilled during this current investigation, which may affect the installation of steel H-piles/tube piles or caissons if deep foundation option is adopted. In addition, while not specifically encountered in the boreholes advanced at this site, the presence of boulders should be expected in the glacial till deposit. It is recommended that driving shoes be used on all steel H-piles or tube piles to facilitate driving through 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.

6.11.6 Bedrock Excavation and/or Socket Formation

It should be noted that the bedrock at this site is classified as strong (R4) to very strong (R5) (i.e. estimated unconfined compressive strengths in the range of about 77 MPa to 135 MPa). If caissons are adopted and rock sockets are required, or if rock sockets are required for toe support for soldier pile and lagging protection systems, it is recommended that an NSSP be included in the Contract Documents to warn the Contractor that the dolostone bedrock is strong to very strong, which will require socket formation using coring or churn drilling to advance the hole. An NSSP is provided in Appendix C.

6.11.7 Vibration Monitoring During Pile Installation

Commercial and residential structures are present in the vicinity of the Highway 5 / Highway 6 Interchange. The closest structure is the Petro Canada Gas Stations located approximately 50 m to the southwest of the proposed interchange structure. Depending on the vibration tolerance of the buried utilities and structures in the vicinity of the Interchange and the construction sequence for the proposed structure, vibration monitoring may be warranted during driving of H-piles for new foundations or interlocking sheet piles or H-piles for protection systems, to ensure that the vibration levels at the adjacent utilities/structures are maintained below tolerable levels. A maximum peak particle velocity (PPV) of 100 mm/s is generally considered applicable at this site but this requires further assessment by the structural engineer. It is considered that vibrations induced by conventional construction activities such as pile driving, coring/churn drilling, or hoe-ramming will not reach this threshold level, however, the monitoring of vibrations during construction should be considered by the general contractor to defend against potential damage claims by the owners of the nearby utilities/structures.

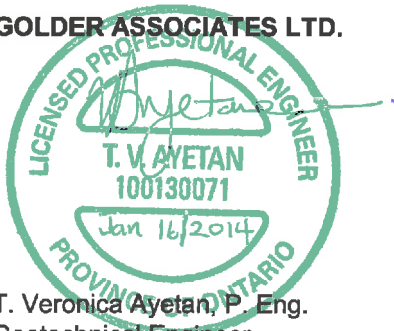
In the event that vibration monitoring is determined to be necessary, an example NSSP for such monitoring is provided in Appendix C for inclusion in the Contract Documents.



7.0 CLOSURE

This report was prepared by Ms. T. Veronica Ayetan, P. Eng., a geotechnical engineer with Golder, with assistance provided by Mr. Geoff Lay. Mr. Jorge M. A. Costa, P. Eng., the Designated MTO Contact for this project and Principal with Golder, conducted a technical and quality control review of the report.

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ASTM International:

ASTM D1586	Standard Test Method for Standard Penetration Test (SPT) and Split-Barrel Sampling of Soils
ASTM D7012	Standard Test Method for Compressive Strength and Elastic Moduli of Intact Rock Core Specimens under Varying States of Stress and Temperatures

Contract Design Estimating and Documentation (CDED) Special Provision:

SP 105S21	Amendment to OPSS 501 – Compacting
SP 110S13	Amendment to OPSS 1010 – Material Specification for Aggregates – Base, Subbase, Select Subgrade and Backfill Material
SP 206S03	Earth Excavation and Grading



SP 599S22 Retained Soil System, Wall/Slope, High Performance

Ontario Provisional Standard Drawing:

OPSD 202.010 Slope Flattening using Surplus Excavated Material on Earth or Rock Embankment.
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 Penetration Depths for Southern Ontario
OPSD 3101.150 Walls, Abutments, Backfill, Minimum Granular Requirements
OPSD 3121.150 Walls, Retaining, Backfill, Minimum Granular Requirements

Ontario Provincial Standard Specification:

OPSS 501 Construction Specifications for Compacting
OPSS 539 Construction Specification for Temporary Protection Systems
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

Ontario Water Resources Act:

Ontario Regulation 903 Wells (as amended)

Ontario Occupational Health and Safety Act:

Ontario Regulation 213 Construction Projects (as amended)

Ministry of Transportation Ontario:

MTO Foundations. Embankment Settlement Criteria for Design. March 2010.

Ministry of Transportation, Ontario, 2008. Structural Manual. Provincial Highway Management Division, Highways Standards Branch, Bridge Office.

Ministry of Transportation Ontario. Structural Office Report SO-96-01. Integral Abutment Bridges.

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



TABLES



**TABLE 1 - COMPARISON OF FOUNDATION ALTERNATIVES
HIGHWAY 5 OVER HIGHWAY 6 INTERCHANGE STRUCTURE
G.W.P. 2112-05-00**

Foundation Option	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Strip or Spread Footing (on stiff to hard clayey silt to clayey silt with sand till).	<ul style="list-style-type: none"> Lower costs than deep foundations; and Standard construction methods; no specialized construction equipment required. 	<ul style="list-style-type: none"> Up to about 2.2 m of excavation of existing fill (below Highway 5/6 grades) would be required to found the footing on competent native soil; Traffic protection system required during construction; Groundwater control may be required; Lower bearing geotechnical resistances compared to deep foundation options; Precludes use of integral abutments; potentially greater maintenance required at abutments; and Potential for differential settlement between foundation elements. 	<ul style="list-style-type: none"> Lower relative costs than deep foundations; and Additional cost for sub-excavation of existing fill. (3 m wide x 40 m long x 1.2 m thick) at \$ 600/m³ + excavation cost (2.2 m deep x 7.5 m wide x 40 m long) at \$ 10/m³ ≈ \$ 93,000/foundation element, excluding temporary protection cost. 	<ul style="list-style-type: none"> Potential traffic disruption during construction; Potential for additional sub-excavation to reach a suitable foundation base; and Potential requirement for groundwater control.
Strip or Spread Footing “perched” on Granular ‘A’ pad (in approach embankment fill).	<ul style="list-style-type: none"> Feasible for support of abutments only; Reduce depth of excavation compared to footings on native material; and Groundwater control would not be required. 	<ul style="list-style-type: none"> Up to about 0.8 m of excavation of existing Highway 5/Highway 6 pavement structures; Traffic protection system required during construction; Lower bearing capacities compared to deep foundation options; Longer bridge span may be required; Does not allow for integral abutment construction; and Potential for differential settlement between foundation elements. 	<ul style="list-style-type: none"> Lowest cost option; and Additional cost for sub-excavation of existing Highway 5/Highway 6 pavement structures. (3 m wide x 40 m long x 1.2 m thick) at \$ 600/m³ + excavation cost (0.8 m deep x 4.6 m wide x 40 m long) at \$ 10/m³ + Granular ‘A’ pad (5 m wide x 42 m long x 2 m thick) at \$ 60/m³ ≈ \$113,000/foundation abutment, excluding central pier footing and temporary protection costs. 	<ul style="list-style-type: none"> Potential traffic disruption during construction; and Depending on presence of organics and rootlets in existing fill, a greater volume of sub-excavation may be required.
Steel H-piles or steel pipe (tube) piles (driven to dolostone bedrock).	<ul style="list-style-type: none"> Negligible post-construction settlement; Allow for semi-integral or integral abutments design; and Sub-excavation depth for pile cap and groundwater control is not required as the pile cap is above ground surface (i.e. less temporary shoring efforts and cost) and protected by front slope cover soil. 	<ul style="list-style-type: none"> Traffic protection system required during construction; Possibility of piles “hanging up” on cobbles and/or boulders in stiff to hard cohesive till deposit above the bedrock, especially for steel pipe foundation; and Conventional construction methods for H-pile or steel pipe (tube) pile foundations. 	<ul style="list-style-type: none"> Higher cost than shallow foundations. Estimated cost for pile installation is approximately (18 piles x 11 m x 2 + 18 piles x 7 m) at \$250/m length ≈ \$130,000 plus \$600/m³ for pile cap construction; the cost may be higher to account for use of temporary liners if required. 	<ul style="list-style-type: none"> Potential traffic disruption during construction; Potential for encountering obstructions that could affect pile installation or to be deflected away from vertical alignment during driving; and Potentially less costly maintenance over life of the structure than semi-integral abutment structures.



FOUNDATION REPORT – HIGHWAY 5 OVER HIGHWAY 6 INTERCHANGE STRUCTURE, GWP 2112-05-00

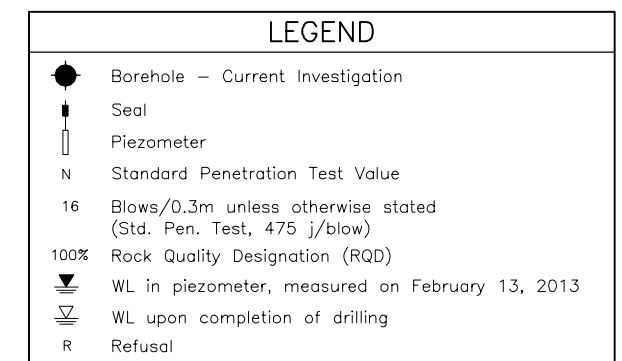
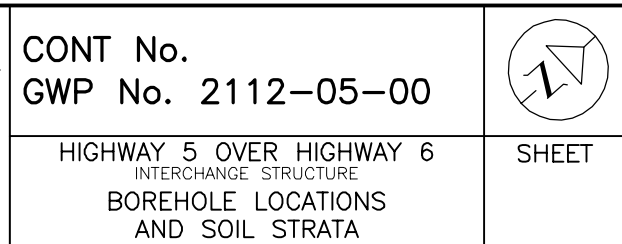
Foundation Option	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Caissons (socketted into dolostone bedrock).	<ul style="list-style-type: none">Negligible post-construction settlement;Higher geotechnical resistances per unit compared to spread footings and piles; so reduced number of deep foundation elements compared to steel H- or tube piles; andPossible elimination of pile cap.	<ul style="list-style-type: none">Temporary liners would be required for groundwater control and support through overburden;Concrete for caissons would have to be placed by tremie methods below the water level;May be difficult socketting caissons into strong to very strong dolostone bedrock to the design base elevation; and coring/chun drilling required to penetrate bedrock;Traffic protection system required during construction;Not suitable for integral abutment design; andGreater risk of encountering obstructions due to larger size of drill hole.	<ul style="list-style-type: none">Higher cost than steel H-piles and tube piles.Estimated cost for caisson installation is approximately (9 caisson x 12 m x 2 + 9 caissons x 8 m) at \$1000/m length ≈ \$288,000.	<ul style="list-style-type: none">Special construction procedures including use of temporary or permanent liners ;Difficulties achieving seal and drilling large diameter socket into strong to very strong bedrock to the design base elevation;Possible traffic disruption during construction due to space required for caisson drilling equipment;Risk of encountering obstructions that could impact caisson installation/costs; andConstruction cost could be higher if chun drilling/coring and temporary liners are required.

Prepared By: TVA

Reviewed By: JMAC



DRAWINGS



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.

Base plans provided in digital format by IBI Group, drawing file nos. br_hwy5_6_ga.dwg and 2010-03-19 A1-04 Site Plan-mod.dwg, received March 12, 2013.



NO.	DATE	BY	REVISION		
Geocres No. 30M5-289					
HWY. 5 & 6		PROJECT NO. 10-1184-0016		DIST.	
SUBM'D. TVA		CHKD. TVA		DATE: 4/17/2013	
DRAWN: DD		CHKD.		APPD: JMAC DWG. 1	

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

CONT No.
GWP No. 2112-05-00

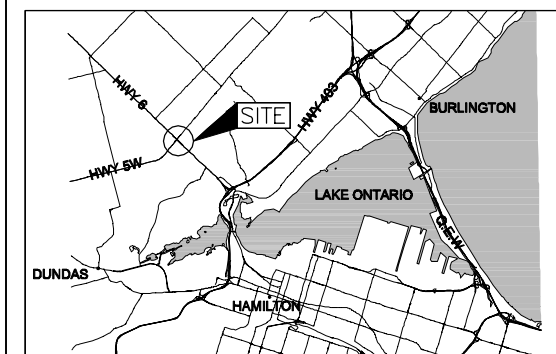
HIGHWAY 5 OVER HIGHWAY 6
INTERCHANGE STRUCTURE

SHEET

SOIL STRATA



Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



KEY PLAN
SCALE
3 0 3 6 km

LEGEND

- Borehole - Current Investigation
- Seal
- Piezometer
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- 100% Rock Quality Designation (RQD)
- WL in piezometer, measured on February 13, 2013
- WL upon completion of drilling
- Refusal

No.	ELEVATION	NORTHING	EASTING
BH 1	222.1	4797035.5	270882.4
BH 2	222.1	4797021.9	270895.0
BH 3	221.6	4797061.7	270892.5
BH 4	221.7	4797046.4	270915.8
BH 5	221.7	4797082.1	270915.5
BH 6	221.6	4797064.2	270934.9

NOTES

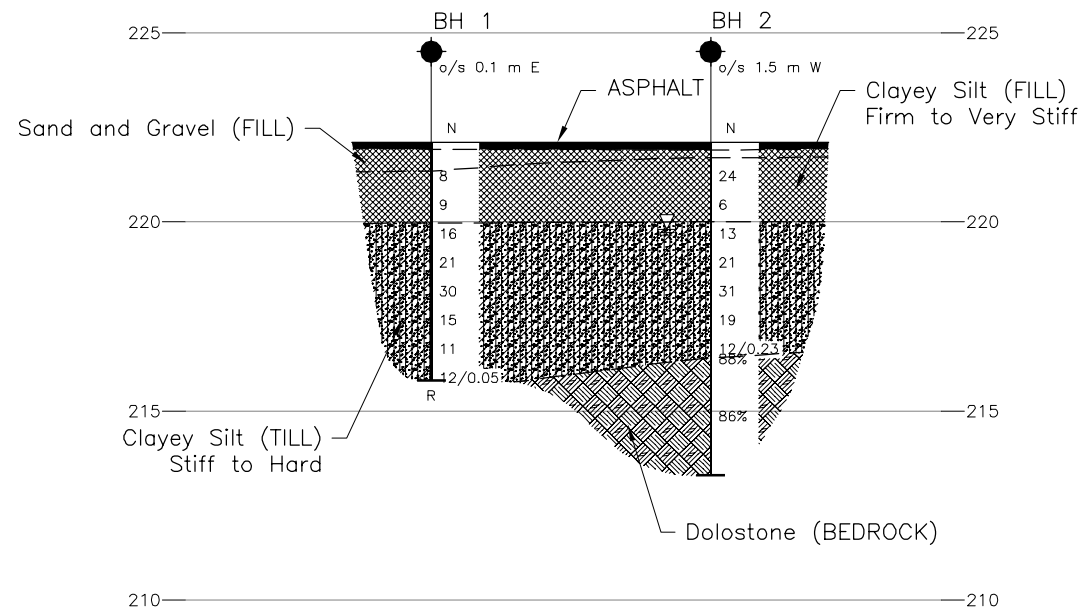
This drawing is for subsurface information only. The proposed structure details/works are shown for illustration purposes only and may not be consistent with the final design configuration as shown elsewhere in the Contracts Documents.

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

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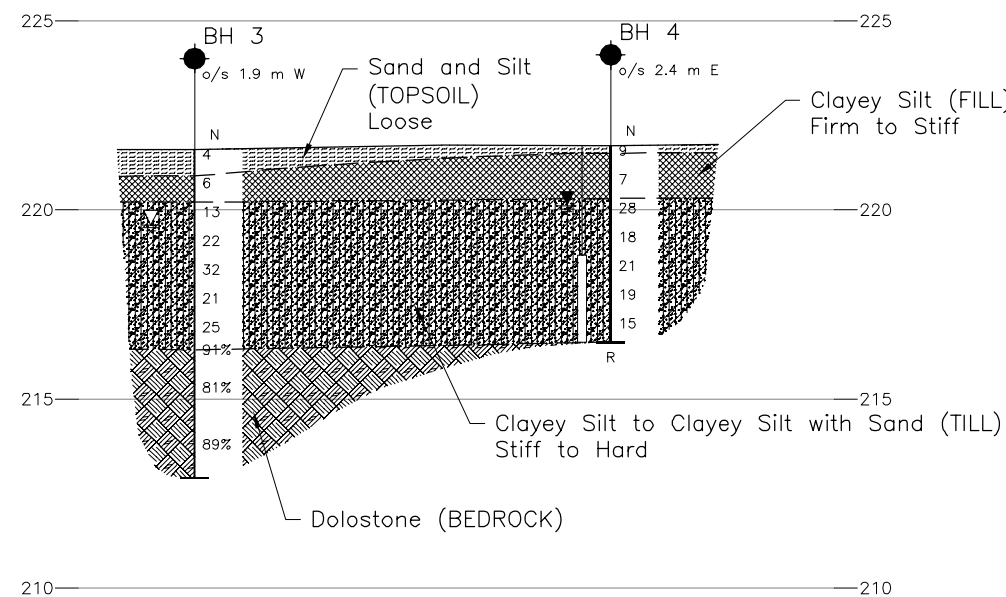


NO.	DATE	BY	REVISION
Geocres No. 30M5-289			
HWY. 5 & 6		PROJECT NO. 10-1184-0016	DIST.
SUBM'D. TVA	CHKD. TVA	DATE: 4/17/2013	SITE:
DRAWN: DD	CHKD.	APPD. JMAC	DWG. 2



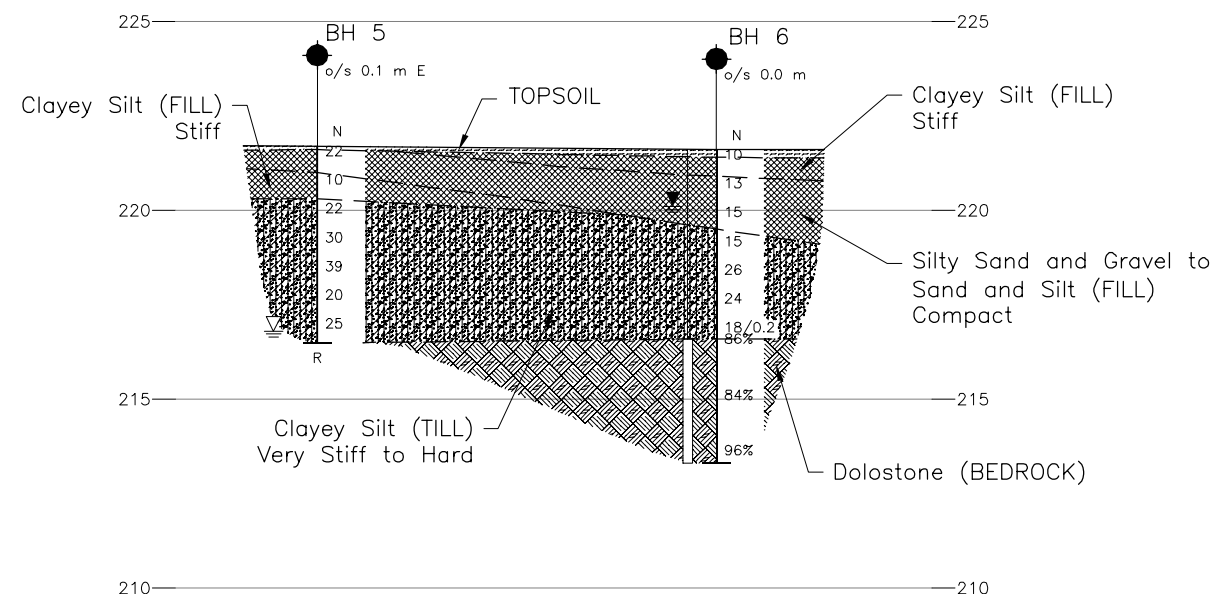
B-B' 1 WEST ABUTMENT

SCALE
HOR. 5 0 5 10 m
VER. 2 0 2 4 m



C-C' 1 CENTRE PIER

SCALE
HOR. 5 0 5 10 m
VER. 2 0 2 4 m



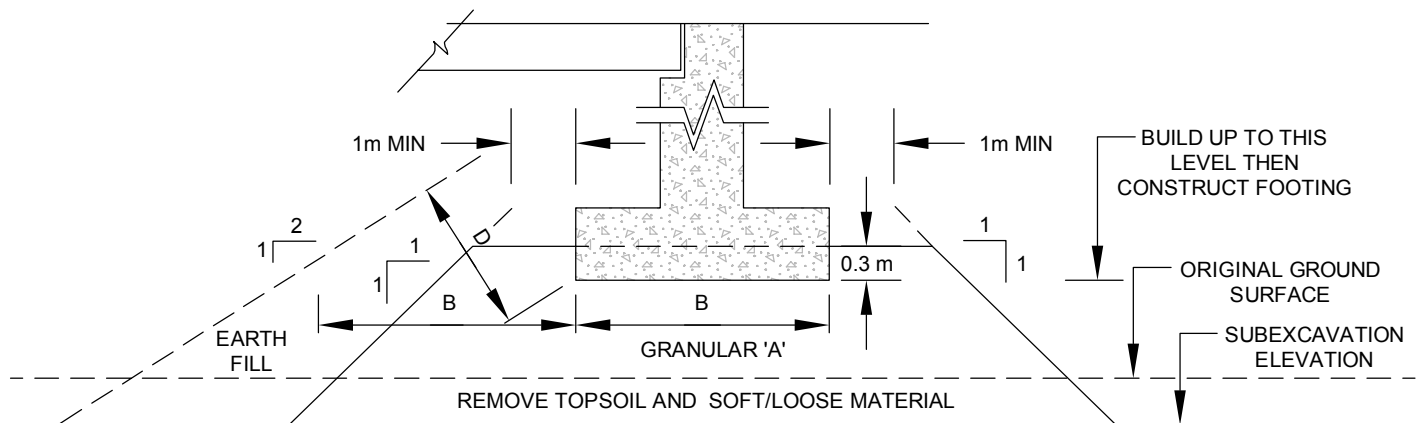
D-D' 1 EAST ABUTMENT

SCALE
HOR. 5 0 5 10 m
VER. 2 0 2 4 m



FIGURES

PLOT DATE: January 15, 2014
 FILENAME: T:\Projects\2010\10-1184-0016 (IG, Hamilton)\-FA- Hwy 5 Over Hwy 6 - Int\1011840016FA001.dwg




CROSS-SECTION

NOTES:

1. REMOVE TOPSOIL AND SOFT/LOOSE SUBSOIL UNDER FOOTPRINT OF COMPACTED GRANULAR 'A'.
2. PLACE GRANULAR 'A' TO BOTTOM OF FOOTING LEVEL, COMPACTED ACCORDING TO SP 105S21.
3. CONSTRUCT CONCRETE FOOTING.
4. PLACE REMAINDER OF GRANULAR 'A' AND EARTH FILL AS REQUIRED.
5. SOURCE M.T.C. 1982.

NOT TO SCALE

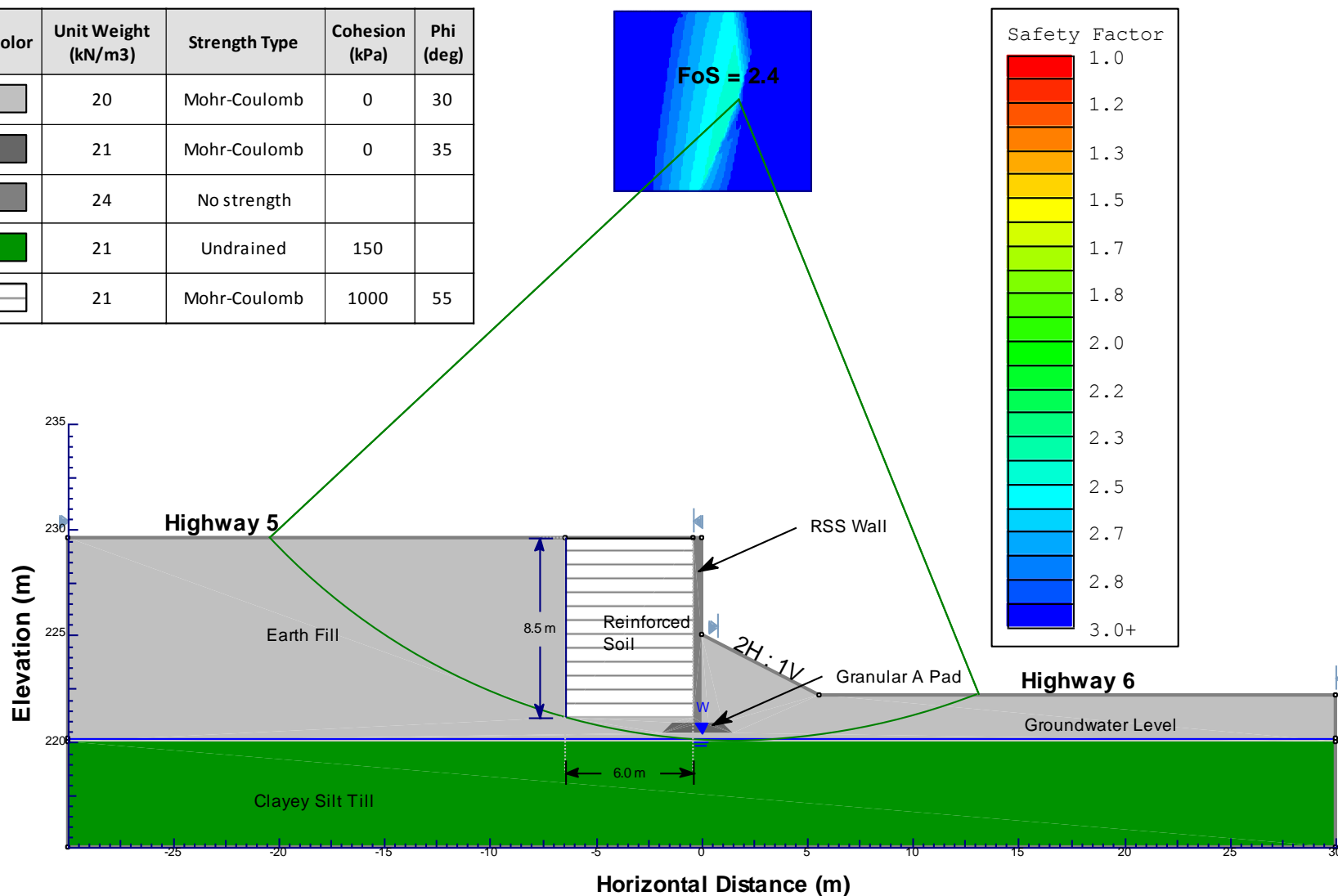
PROJECT				HIGHWAY 5 OVER HIGHWAY 6 INTERCHANGE STRUCTURE CITY OF HAMILTON, MINISTRY OF TRANSPORTATION, ONTARIO G.W.P 2112-05-00			
TITLE				TYPICAL ABUTMENT ON COMPACTED FILL SHOWING GRANULAR 'A' CORE			
		PROJECT No.		10-1184-0016		FILE No.	
		DESIGN				SCALE	
		CAD		DD		AS SHOWN	
		CHECK		TVA		REV.	
		REVIEW		JMAC		A	
				Apr. 18, 2013		FIGURE No.	
				Apr. 18, 2013		1	



Highway 5 over Highway 6 Interchange Structure RSS Wall Static Global Stability Analysis – 8.5 m High Wall

Figure 2

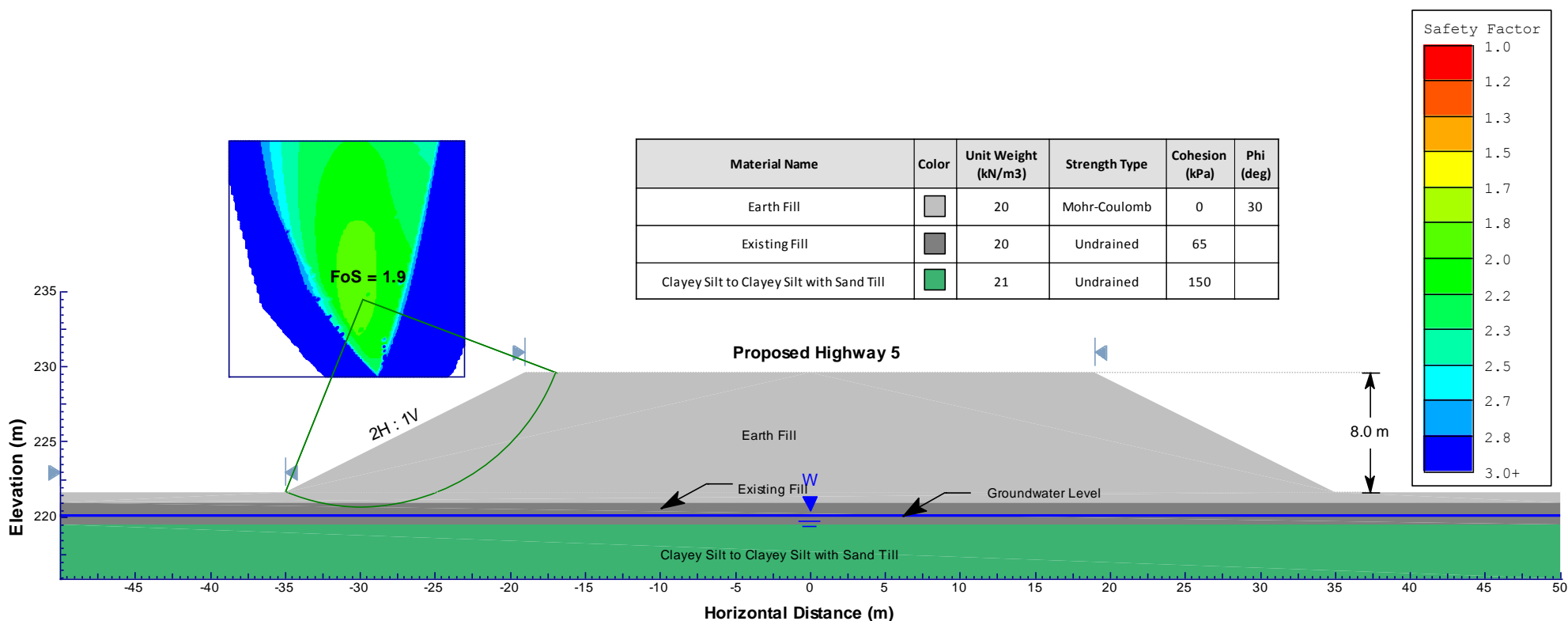
Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)
Earth Fill		20	Mohr-Coulomb	0	30
Granular A		21	Mohr-Coulomb	0	35
RSS Wall		24	No strength		
Clayey Silt Till		21	Undrained	150	
Reinforced Soil		21	Mohr-Coulomb	1000	55





Highway 5 over Highway 6 Interchange Structure West Approach Embankment – Static Global Stability Analysis

Figure 3





APPENDIX A

Record of Boreholes and Drillholes



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
FoS	factor of safety

II. STRESS AND STRAIN

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

III. SOIL PROPERTIES

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

(a) Index Properties (continued)

w	water content
w_l 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 = $\tan \delta$
c'	effective cohesion
c_u, s_u	undrained shear strength ($\phi = 0$ analysis)
p	mean total stress $(\sigma_1 + \sigma_3)/2$
p'	mean effective stress $(\sigma'_1 + \sigma'_3)/2$
q	$(\sigma_1 - \sigma_3)/2$ or $(\sigma'_1 - \sigma'_3)/2$
q_u	compressive strength $(\sigma_1 - \sigma_3)$
S_t	sensitivity

* 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
2

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



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

III. SOIL DESCRIPTION

(a) Non-Cohesive Soils

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

II. PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:

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

(b) Cohesive Soils Consistency

	<u>kPa</u>	<u>C_u, S_u</u>	<u>psf</u>
Very soft	0 to 12		0 to 250
Soft	12 to 25		250 to 500
Firm	25 to 50		500 to 1,000
Stiff	50 to 100		1,000 to 2,000
Very stiff	100 to 200		2,000 to 4,000
Hard	over 200		over 4,000

Dynamic Cone Penetration Resistance; N_d:

The number of blows by a 63.5 kg (140 lb.) hammer dropped 760 mm (30 in.) to drive uncased a 50 mm (2 in.) diameter, 60° cone attached to "A" size drill rods for a distance of 300 mm (12 in.).

PH: Sampler advanced by hydraulic pressure

PM: Sampler advanced by manual pressure

WH: Sampler advanced by static weight of hammer

WR: Sampler advanced by weight of sampler and rod

Piezo-Cone Penetration Test (CPT)

A electronic cone penetrometer with a 60° conical tip and a project end area of 10 cm² pushed through ground at a penetration rate of 2 cm/s. Measurements of tip resistance (Q_t), porewater pressure (PWP) and friction along a sleeve are recorded electronically at 25 mm penetration intervals.

IV. SOIL TESTS

w	water content
w _p	plastic limit
w _l	liquid limit
C	consolidation (oedometer) test
CHEM	chemical analysis (refer to text)
CID	consolidated isotropically drained triaxial test ¹
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement ¹
D _R	relative density (specific gravity, G _s)
DS	direct shear test
M	sieve analysis for particle size
MH	combined sieve and hydrometer (H) analysis
MPC	Modified Proctor compaction test
SPC	Standard Proctor compaction test
OC	organic content test
SO ₄	concentration of water-soluble sulphates
UC	unconfined compression test
UU	unconsolidated undrained triaxial test
V	field vane (LV-laboratory vane test)
γ	unit weight

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

V. MINOR SOIL CONSTITUENTS

Per cent 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 (non-cohesive) or With (cohesive)	Sand and Gravel Silty Clay with sand / Clayey Silt with sand



LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY

WEATHERINGS STATE

Fresh: no visible sign of weathering

Faintly weathered: weathering limited to the surface of major discontinuities.

Slightly weathered: penetrative weathering developed on open discontinuity surfaces but only slight weathering of rock material.

Moderately weathered: weathering extends throughout the rock mass but the rock material is not friable.

Highly weathered: weathering extends throughout rock mass and the rock material is partly friable.

Completely weathered: rock is wholly decomposed and in a friable condition but the rock and structure are preserved.

BEDDING THICKNESS

Description	Bedding Plane Spacing
Very thickly bedded	Greater than 2 m
Thickly bedded	0.6 m to 2 m
Medium bedded	0.2 m to 0.6 m
Thinly bedded	60 mm to 0.2 m
Very thinly bedded	20 mm to 60 mm
Laminated	6 mm to 20 mm
Thinly laminated	Less than 6 mm

JOINT OR FOLIATION SPACING

Description	Spacing
Very wide	Greater than 3 m
Wide	1 m to 3 m
Moderately close	0.3 m to 1 m
Close	50 mm to 300 mm
Very close	Less than 50 mm

GRAIN SIZE

Term	Size*
Very Coarse Grained	Greater than 60 mm
Coarse Grained	2 mm to 60 mm
Medium Grained	60 microns to 2 mm
Fine Grained	2 microns to 60 microns
Very Fine Grained	Less than 2 microns

Note: * Grains greater than 60 microns diameter are visible to the naked eye.

CORE CONDITION

Total Core Recovery (TCR)

The percentage of solid drill core recovered regardless of quality or length, measured relative to the length of the total core run.

Solid Core Recovery (SCR)

The percentage of solid drill core, regardless of length, recovered at full diameter, measured relative to the length of the total core run.

Rock Quality Designation (RQD)

The percentage of solid drill core, greater than 100 mm length, recovered at full diameter, measured relative to the length of the total core run. RQD varied from 0% for completely broken core to 100% for core in solid sticks.

DISCONTINUITY DATA

Fracture Index

A count of the number of discontinuities (physical separations) in the rock core, including both naturally occurring fractures and mechanically induced breaks caused by drilling.

Dip with Respect to Core Axis

The angle of the discontinuity relative to the axis (length) of the core. In a vertical borehole a discontinuity with a 90° angle is horizontal.

Description and Notes

An abbreviation description of the discontinuities, whether naturally occurring separations such as fractures, bedding planes and foliation planes or mechanically induced features caused by drilling such as ground or shattered core and mechanically separated bedding or foliation surfaces. Additional information concerning the nature of fracture surfaces and infillings are also noted.

Abbreviations

JN Joint	PL Planar
FLT Fault	CU Curved
SH Shear	UN Undulating
VN Vein	IR Irregular
FR Fracture	K Slickensided
SY Stylolite	PO Polished
BD Bedding	SM Smooth
FO Foliation	SR Slightly Rough
CO Contact	RO Rough
AXJ Axial Joint	VR Very Rough
KV Karstic Void	
MB Mechanical Break	

PROJECT		10-1184-0016		RECORD OF BOREHOLE No BH 1		SHEET 1 OF 1		METRIC							
G.W.P.		2112-05-00		LOCATION		N 4797035.5 ; E 270882.4		ORIGINATED BY							
DIST		HWY 5 & 6		BOREHOLE TYPE		102 mm O.D. Continuous Flight Solid Stem Augers		COMPILED BY							
DATUM		Geodetic		DATE		November 21, 2012		CHECKED BY							
								TVA							
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							
222.1	GROUND SURFACE														
0.0	ASPHALT														
0.2	Sand and gravel, trace silt, containing cobbles at 0.5 m depth (FILL)														
221.3	Brown Moist		1	SS	8										21 29 34 16
0.8	Clayey silt, sandy, some gravel, trace organics and topsoil (FILL)														
	Stiff		2	SS	9										
	Brown to black Moist														
220.0															
2.1	CLAYEY SILT, sandy, trace gravel, containing sand seams to a depth of 2.7 m (TILL)		3	SS	16										1 25 49 25
	Stiff to very stiff		4	SS	21										
	Brown to grey Moist		5	SS	30										
			6	SS	15										
			7	SS	11										
			8	SS	12/0.05										
215.8	END OF BOREHOLE SPOON BOUNCING AND AUGER REFUSAL INFERRED BEDROCK														
6.3	NOTE: 1. Borehole dry upon completion of drilling														

PROJECT		10-1184-0016		RECORD OF BOREHOLE No BH 2		SHEET 1 OF 2		METRIC												
G.W.P.		2112-05-00		LOCATION		N 4797021.9 ; E 270895.0		ORIGINATED BY												
DIST		HWY 5 & 6		BOREHOLE TYPE		152 mm O.D. Continuous Flight Hollow Stem Augers, NW Casing		COMPILED BY												
DATUM		Geodetic		DATE		November 21, 2012		CHECKED BY												
								TVA												
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV	DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	SHEAR STRENGTH kPa					WATER CONTENT (%)			γ	GR	SA	SI	CL
							20 40 60 80 100	20 40 60 80 100	W _p	W	W _L	10 20 30	kN/m ³							
222.1	0.0	GROUND SURFACE					222													
	0.4	ASPHALT																		
		Sand and gravel, trace silt (FILL)																		
		Brown Moist																		
		Clayey silt, trace to some gravel, trace sand, trace organics and topsoil (FILL)		1	SS	24														
		Firm to very stiff																		
		Brown to black Moist		2	SS	6														
220.0	2.1	CLAYEY SILT, some sand, trace gravel (TILL)					220													
		Stiff to hard		3	SS	13														
		Brown to grey Moist		4	SS	21														
				5	SS	31														
		Sandy		6	SS	19														
				7	SS	12/0.23														
216.4	5.7	DOLOSTONE (BEDROCK)					216													
		Bedrock cored from depths of 5.7 m to 8.8 m		1	RC	REC 96%														
		For bedrock coring details, refer to Record of Drillhole BH 2		2	RC	REC 91%														
							215													
							214													
213.3	8.8	END OF BOREHOLE																		
		NOTES:																		
		1. Spoon bouncing and auger refusal at a depth of 5.7 m (Elev. 215.4 m).																		
		2. Water added for bedrock coring, unstabilized water level in open borehole, measured at a depth of 2.3 m below ground surface (Elev. 219.8 m) upon completion of drilling.																		

PROJECT		10-1184-0016		RECORD OF BOREHOLE No BH 3				SHEET 1 OF 2		METRIC							
G.W.P.		2112-05-00		LOCATION		N 4797061.7 ; E 270892.5		ORIGINATED BY		JBH							
DIST		HWY 5 & 6		BOREHOLE TYPE		152 mm O.D. Continuous Flight Hollow Stem Augers, NW Casing		COMPILED BY		BM							
DATUM		Geodetic		DATE		November 15, 2012		CHECKED BY		TVA							
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
221.6	GROUND SURFACE																
0.0	Sand and silt (TOPSOIL) Loose Dark brown to black Moist		1	SS	4												10 41 40 9
220.9																	
0.7	Clayey silt, trace sand, trace gravel, trace organics and rootlets (FILL)		2	SS	6												
220.2	Firm																
1.4	Brown Moist		3	SS	13												
	CLAYEY SILT, some sand, trace gravel (TILL) Stiff to hard Brown to grey Moist		4	SS	22												
			5	SS	32												2 19 51 28
			6	SS	21												
	Shale fragments inclusions, sand interlayers		7	SS	25												
216.4																	
5.2	DOLOSTONE (BEDROCK)																
	Bedrock cored from depths of 5.2 m to 8.7 m		1	RC	REC 98%												RQD = 91%
	For bedrock coring details, refer to Record of Drillhole BH 3		2	RC	REC 94%												RQD = 81%
			3	RC	REC 97%												RQD = 89%
212.9																	
8.7	END OF BOREHOLE																
	NOTES: 1. Auger refusal at a depth of 5.2 m below ground surface (Elev. 216.4 m). 2. Unstabilized water level measured in open borehole at a depth of 5.0 m below ground surface (Elev. 216.6 m) upon completion of augering/soil sampling. 3. Water added for bedrock coring, unstabilized water level in open borehole, measured at a depth of 2.0 m below ground surface (Elev. 219.6 m) upon completion of drilling.																

PROJECT		10-1184-0016		RECORD OF BOREHOLE No BH 4		SHEET 1 OF 1		METRIC															
G.W.P.		2112-05-00		LOCATION		N 4797046.4 ; E 270915.8		ORIGINATED BY															
DIST		HWY 5 & 6		BOREHOLE TYPE		102 mm O.D. Continuous Flight Solid Stem Augers		COMPILED BY															
DATUM		Geodetic		DATE		November 19, 2012		CHECKED BY															
								TVA															
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)								
ELEV	DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa					WATER CONTENT (%)			γ						
221.7	0.0	GROUND SURFACE							20 40 60 80 100 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED					W _p W W _L 10 20 30			kN/m ³			GR SA SI CL			
0.2		TOPSOIL		1	SS	9		221															
		Clayey silt, trace gravel, trace SAND, containing sand seams (FILL) Firm to stiff Brown Moist		2	SS	7																	
220.3	1.4	CLAYEY SILT, some to with SAND, trace to some gravel (TILL) Very stiff Brown to grey Moist to wet		3	SS	28		220															
				4	SS	18		219															
				5	SS	21		218															
				6	SS	19																	
				7	SS	15		217															
216.5	5.2	END OF BOREHOLE AUGER REFUSAL INFERRED BEDROCK																					
		NOTES:																					
		1. Split Spoon Sampler wet below a depth of 3.0 m below ground surface (Elev. 218.7 m).																					
		2. Unstabilized water level in open borehole measured at a depth of 4.1 m below ground surface (Elev. 217.6 m) upon completion of drilling.																					
		3. WATER LEVEL READINGS:																					
		Date Depth (mm) Elev. (m)																					
		11/23/12 3.2 218.5																					
		01/22/13 1.5 220.2																					
		02/07/13 1.6 220.1																					
		02/13/13 1.6 220.1																					

PROJECT		10-1184-0016		RECORD OF BOREHOLE No BH 5		SHEET 1 OF 1		METRIC							
G.W.P.		2112-05-00		LOCATION		N 4797082.1 ; E 270915.5		ORIGINATED BY							
DIST		HWY 5 & 6		BOREHOLE TYPE		102 mm O.D. Continuous Flight Solid Stem Augers		COMPILED BY							
DATUM		Geodetic		DATE		November 15, 2012		CHECKED BY							
								TVA							
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							
221.7	GROUND SURFACE														
0.0	TOPSOIL														
0.1	Silty sand and gravel (FILL)		1	SS	22										
221.0	Compact Brown Dry														
0.7	Clayey silt, some sand, trace gravel, trace organics (FILL)		2	SS	10										
220.3	Stiff Brown Moist														
1.4	CLAYEY SILT, some sand, trace gravel (TILL)		3	SS	22										
	Very stiff to hard Brown to grey Moist														
			4	SS	30										
			5	SS	39										
	Sandy, cobbles at 4.0 m depth		6	SS	20										
			7	SS	25										
216.5	END OF BOREHOLE AUGER REFUSAL INFERRED BEDROCK														
5.2	NOTE: 1. Unstabilized water level in open borehole measured at a depth of 4.9 m below ground surface (Elev. 216.8 m) upon completion of drilling.														

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

GTA-MTO 001 10-1184-0016.GPJ GAL-GTA.GDT 01/15/14 DD

SHEET 2 OF 2

DATUM: Geodetic

DRILLING CONTRACTOR: DBW Drilling

CHECKED: TVA

GTA-RCK 004 10-1184-0016.GPJ GAL-MISS.GDT 01/15/14 DD

PROJECT <u>10-1184-0016</u>	RECORD OF BOREHOLE No H5-5	SHEET 1 OF 1	METRIC
G.W.P. <u>2112-05-00</u>	LOCATION <u>N 4797014.8 ; E 270871.0</u>	ORIGINATED BY <u>JBH</u>	
DIST <u> </u> HWY <u>5 & 6</u>	BOREHOLE TYPE <u>102 mm O.D. Continuous Flight Solid Stem Augers</u>	COMPILED BY <u>BM</u>	
DATUM <u>Geodetic</u>	DATE <u>November 13, 2012</u>	CHECKED BY <u>TVA</u>	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE LIQUID CONTENT LIMIT			UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					w _p	w	w _L		GR	SA	SI	CL
								20	40	60	80	100								
221.6	GROUND SURFACE																			
0.0	Sand and silt (TOPSOIL) Compact Dark brown Moist		1	SS	11															
220.9																				
0.7	Clayey silt with sand, trace gravel, trace organics (FILL) Firm Brown Moist		2	SS	6															
			3	SS	7															
219.5																				
2.1	CLAYEY SILT, sandy, trace gravel (TILL) Very stiff to hard Brown Moist		4	SS	16								○							
			5	SS	31								○	—			2	24	44	30
			6	SS	31															
			7	SS	18								○	—			5	27	45	23
			8	SS	20/0.1								○	—						
215.9																				
5.7	END OF BOREHOLE AUGER REFUSAL INFERRED BEDROCK NOTES: 1. Borehole dry upon completion of drilling. 2. WATER LEVEL READINGS: Date Depth (mm) Elev. (m) 11/23/12 4.6 217.0 01/22/13 2.2 219.4 02/07/13 1.8 219.8 02/13/13 1.7 219.9																			

PROJECT		10-1184-0016		RECORD OF BOREHOLE No H5-6		SHEET 1 OF 1		METRIC											
G.W.P.		2112-05-00		LOCATION		N 4797086.5 ; E 270942.5		ORIGINATED BY											
DIST		HWY 5 & 6		BOREHOLE TYPE		102 mm O.D. Continuous Flight Solid Stem Augers		COMPILED BY											
DATUM		Geodetic		DATE		November 15, 2012		CHECKED BY											
								TVA											
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)				
ELEV	DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa		W _p W W _L		WATER CONTENT (%)		γ		GR SA SI CL		
222.1		GROUND SURFACE							20 40 60 80 100	20 40 60 80 100	10 20 30								
0.0		ASPHALT						222											
0.2		Sand and gravel, trace silt (FILL) Brown Moist		1	SS	18													
		Clayey silt with sand, some gravel (FILL) Stiff to very stiff Brown Moist		2	SS	8		221										19 34 36 11	
220.7		CLAYEY SILT, some sand, trace gravel (TILL) Very stiff Brown to grey Moist		3	SS	24		220											
1.4				4	SS	28												1 16 56 27	
		Silty sand seams		5A 5B	SS	22		219											
				6	SS	25		218											
				7	SS	23		217											
216.9		END OF BOREHOLE AUGER REFUSAL INFERRED BEDROCK																	
5.2		NOTES: 1. Unstabilized water level in open borehole measured at a depth of 3.8 m below ground surface (Elev. 218.3 m) upon completion of drilling. 2. WATER LEVEL READINGS: Date Depth (mm) Elev. (m) 11/16/12 5.1 217.0 11/23/12 4.3 217.8 01/22/13 1.7 220.4 02/07/13 1.8 220.3 02/13/13 1.8 220.3																	



APPENDIX B

Laboratory Test Results (Soil and Rock) and Bedrock Core Photographs

TABLE B1
SUMMARY OF UNIAXIAL COMPRESSIVE STRENGTH TEST RESULTS
HIGHWAY 5 OVER HIGHWAY 6 INTERCHANGE STRUCTURE

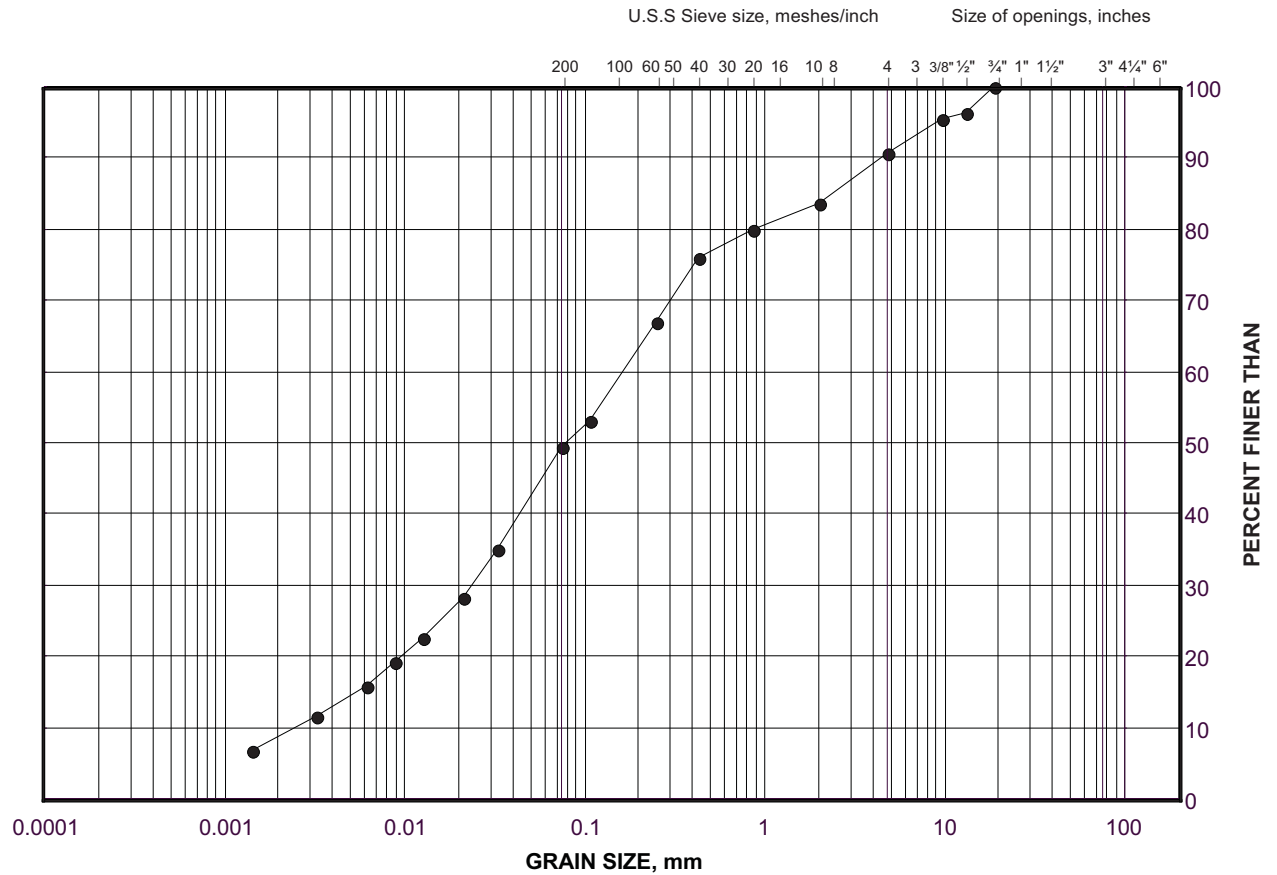
Borehole Number (Core Run)	Sample Depth (m)	Sample Elevation (m)	Rock Type	Core Diameter (mm)	Uniaxial Compressive Strength (MPa)
BH 2 (2)	8.1	214.0	Dolostone	4.74	85.6
BH 3 (2)	6.9	214.7	Dolostone	4.72	134.7
BH 6 (3)	8.1	213.5	Dolostone	4.71	76.6

Compiled By: TVAReviewed By: JMAC

GRAIN SIZE DISTRIBUTION

Sand and Silt (TOPSOIL)

FIGURE B1



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
•	BH 3	1	221.3

Project Number: 10-1184-0016

Checked By: TVA

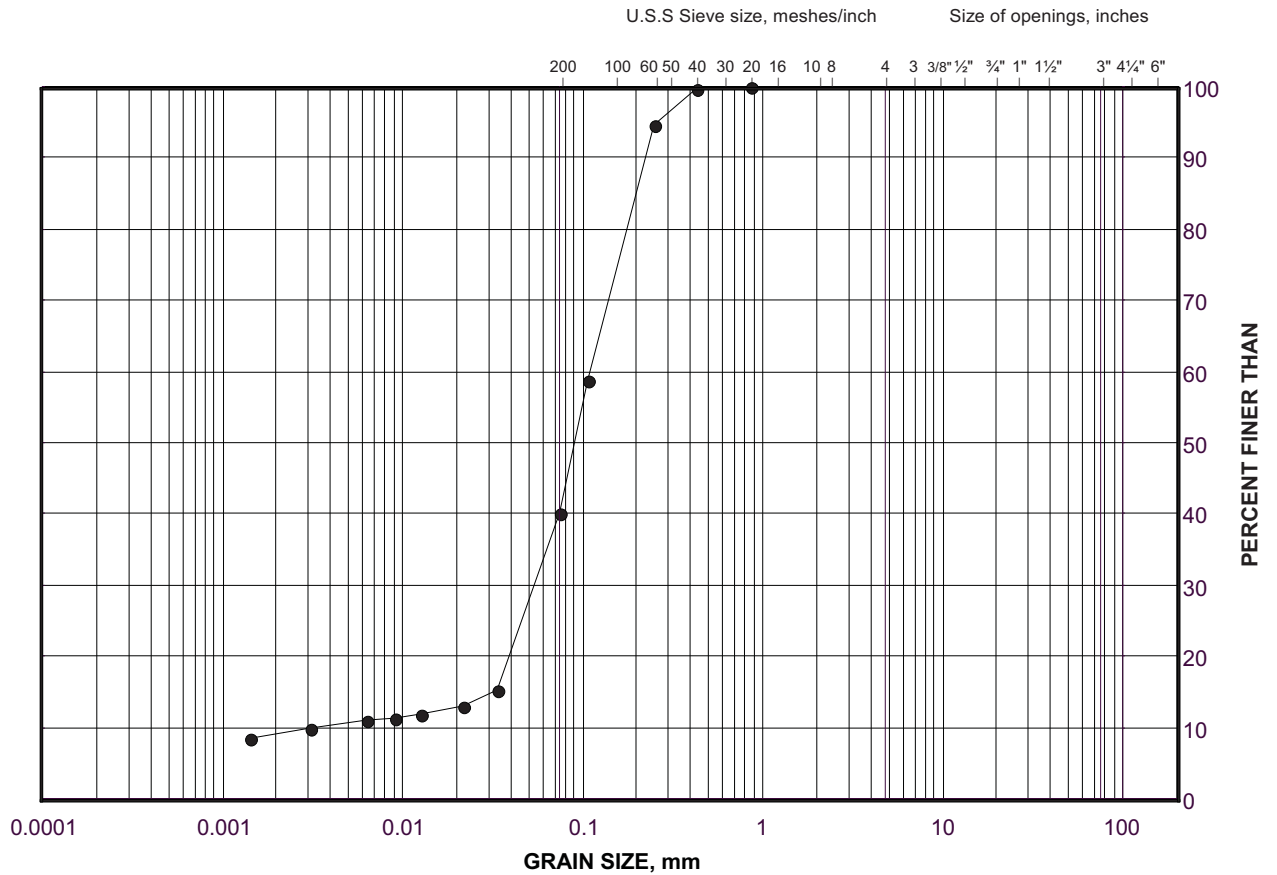
Golder Associates

Date: 12-Apr-13

GRAIN SIZE DISTRIBUTION

Sand and Silt (FILL)

FIGURE B2



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
•	BH 6	2	220.6

Project Number: 10-1184-0016

Checked By: TVA

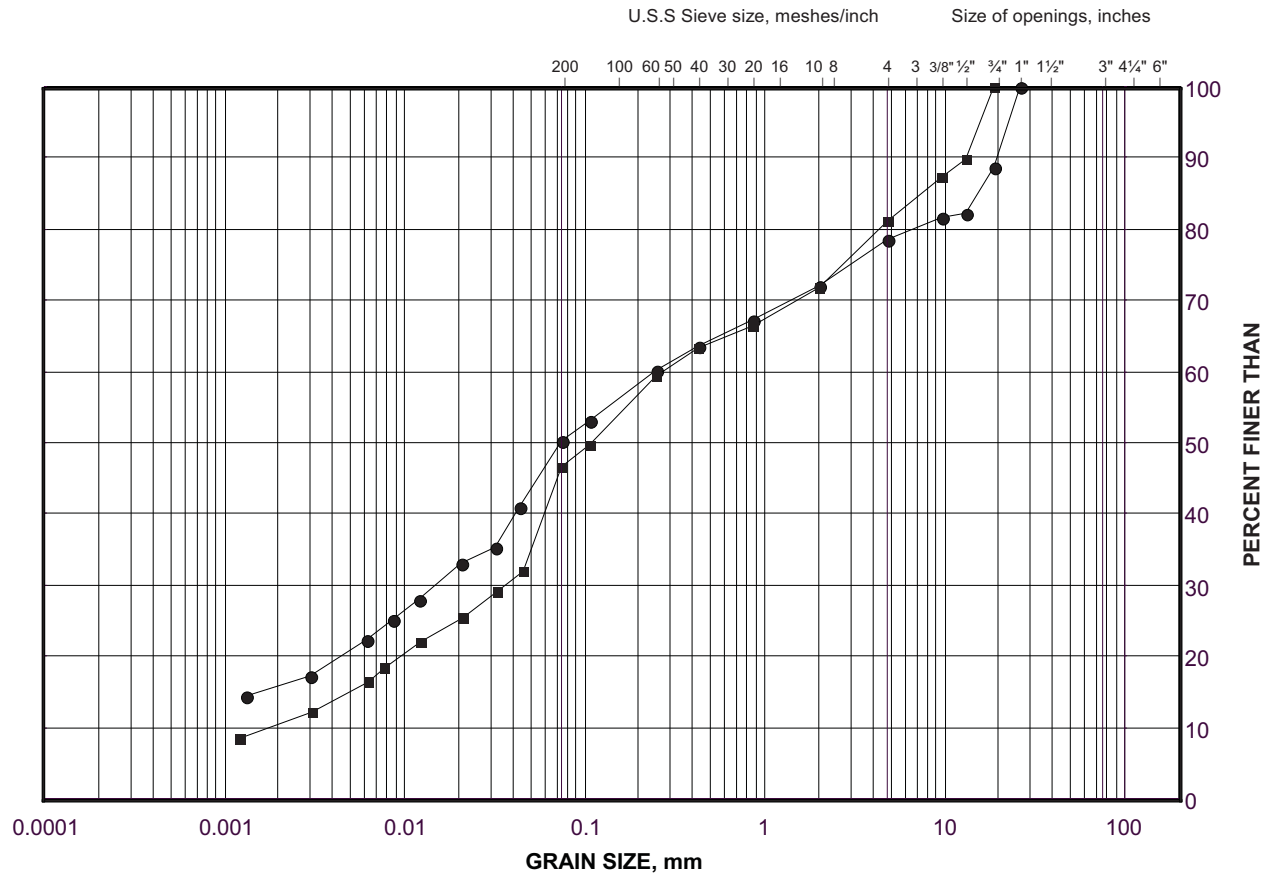
Golder Associates

Date: 12-Apr-13

GRAIN SIZE DISTRIBUTION

Clayey Silt with Sand (FILL)

FIGURE B3



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

LEGEND

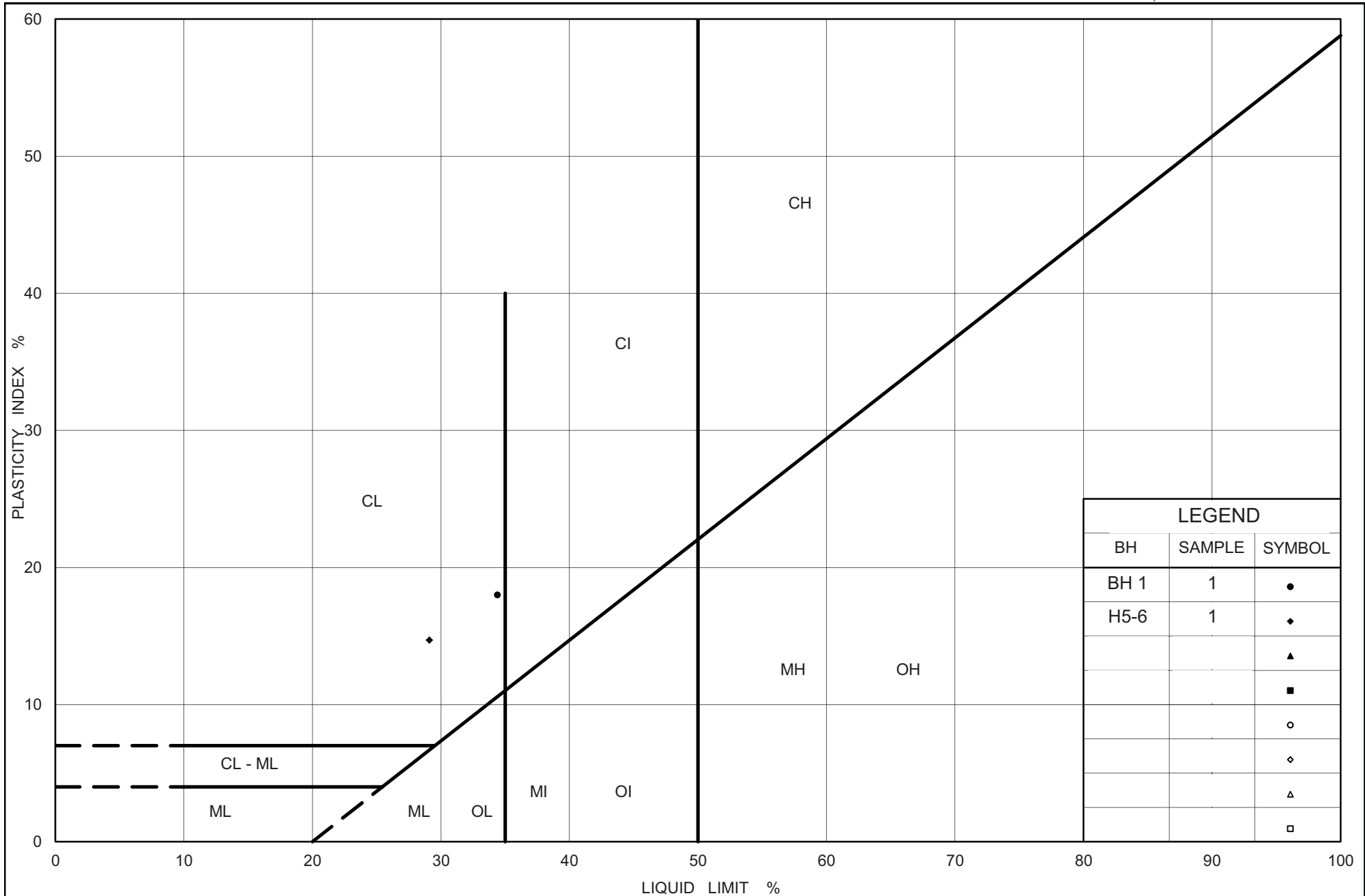
SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	BH 1	1	221.1
■	H5-6	2	221.1

Project Number: 10-1184-0016

Checked By: TVA

Golder Associates

Date: 12-Apr-13



Ministry of Transportation

Ontario

PLASTICITY CHART Clayey Silt with Sand (FILL)

Figure No. B4

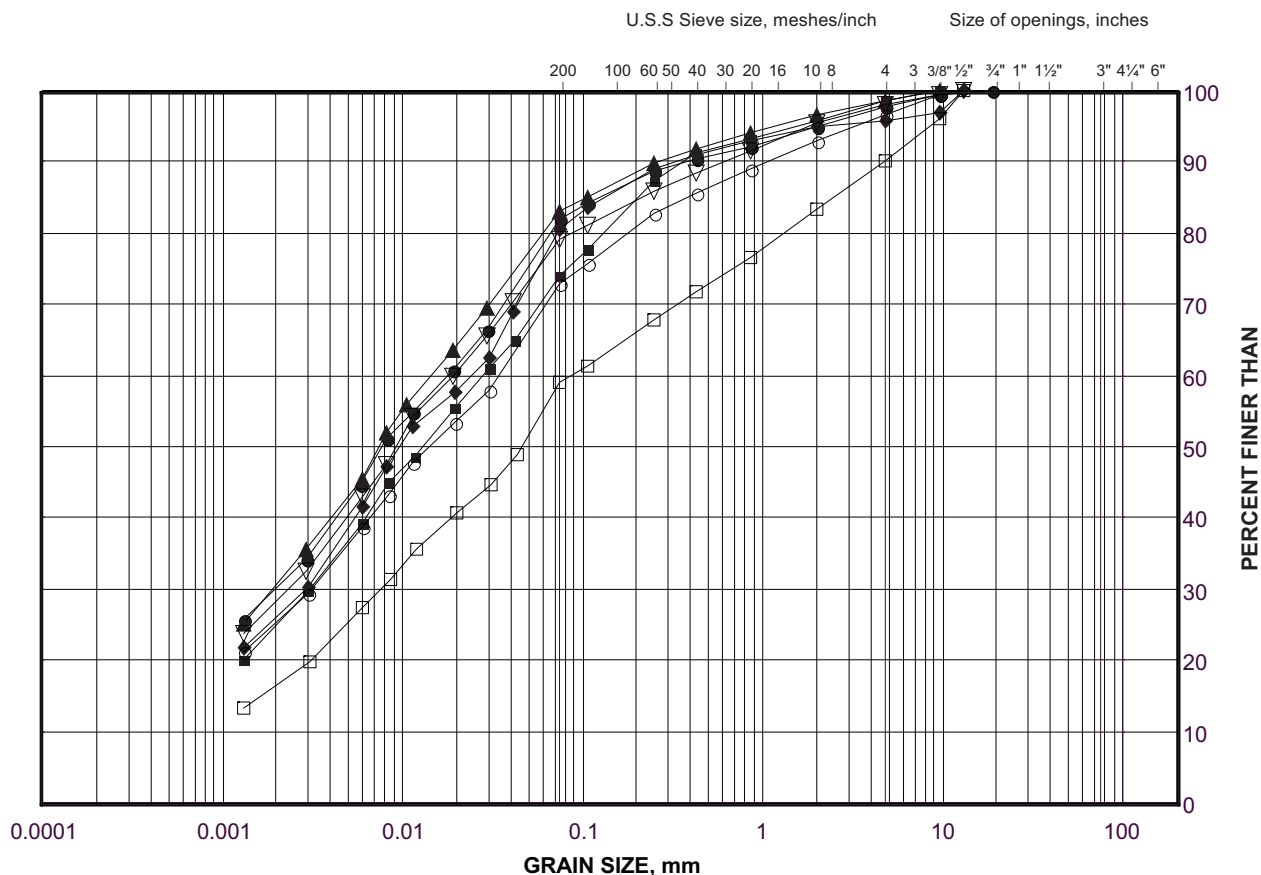
Project No. 10-1184-0016

Checked By: TVA

GRAIN SIZE DISTRIBUTION

Clayey Silt to Clayey Silt with Sand (TILL)

FIGURE B5A



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	BH 5	3	219.9
■	BH 1	3	219.6
◆	BH 4	4	219.2
▲	BH 2	4	218.8
▽	BH 3	5	218.3
○	BH 2	6	217.3
□	BH 4	7	216.9

Project Number: 10-1184-0016

Checked By: TVA

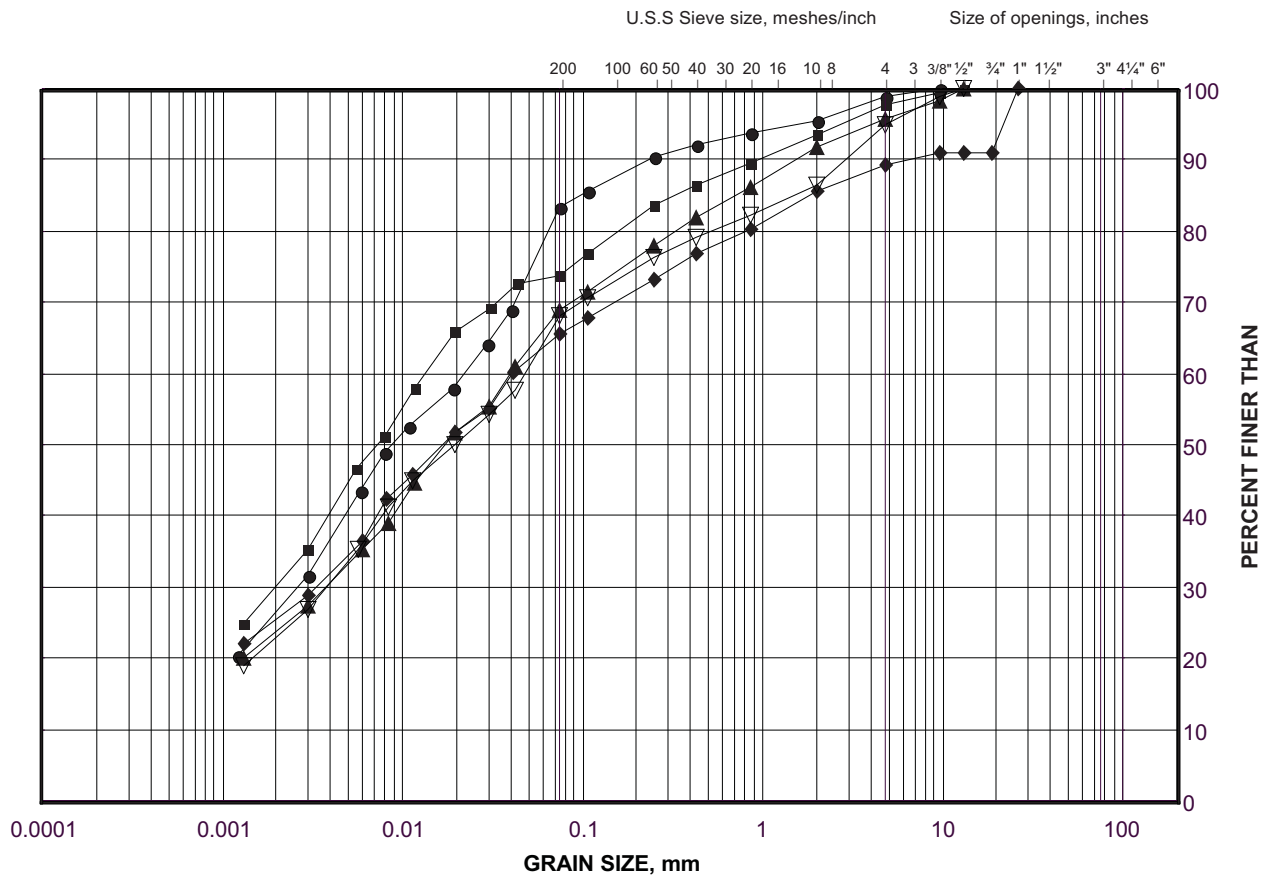
Golder Associates

Date: 12-Apr-13

GRAIN SIZE DISTRIBUTION

Clayey Silt to Clayey Silt with Sand (TILL)

FIGURE B5B



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

LEGEND

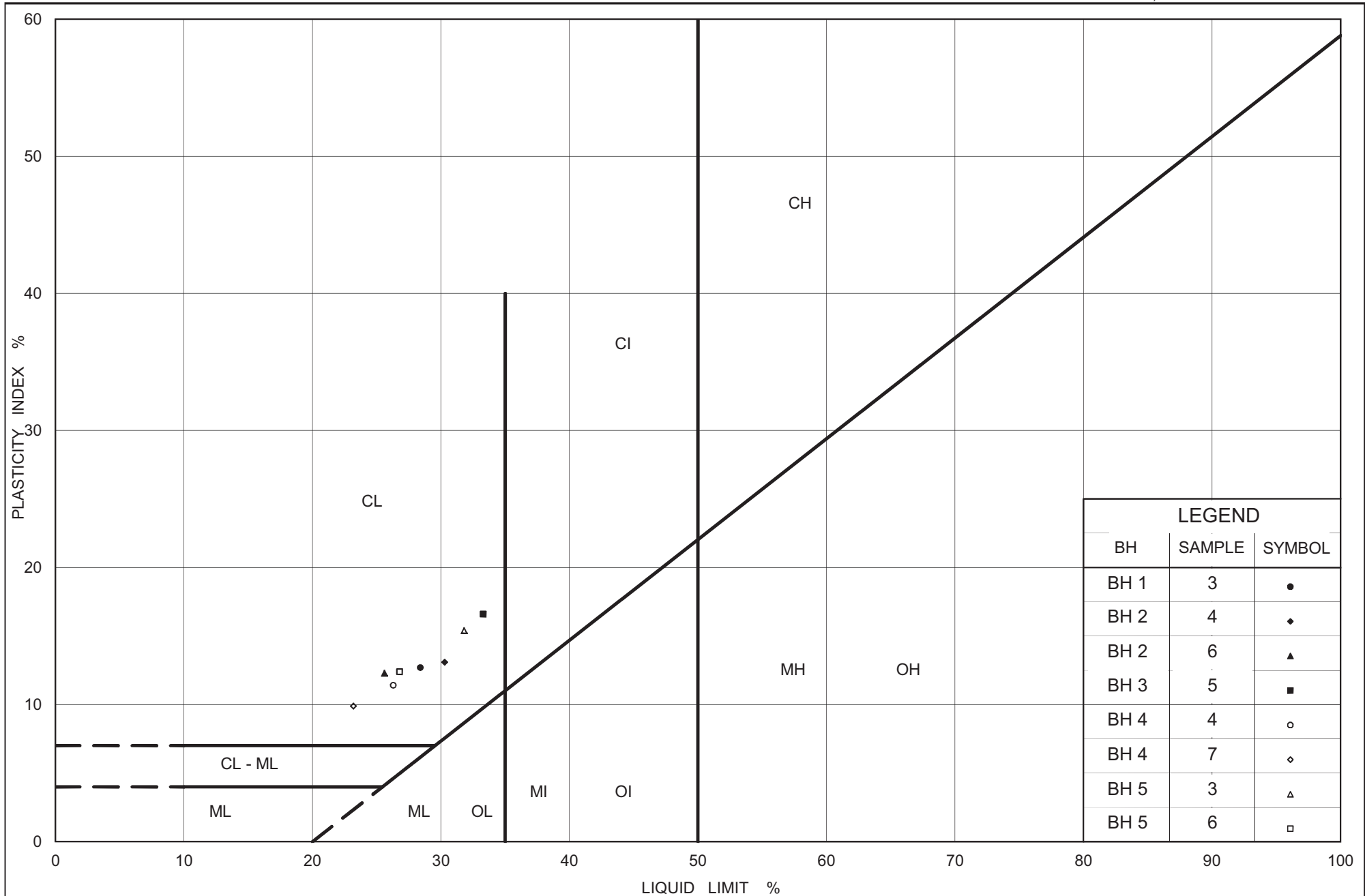
SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	H5-6	4	219.6
■	H5-5	5	218.3
◆	BH6	6	217.6
▲	BH5	6	217.6
▽	H5-5	7	216.8

Project Number: 10-1184-0016

Checked By: _____

Golder Associates

Date: 16-Apr-13



Ministry of Transportation

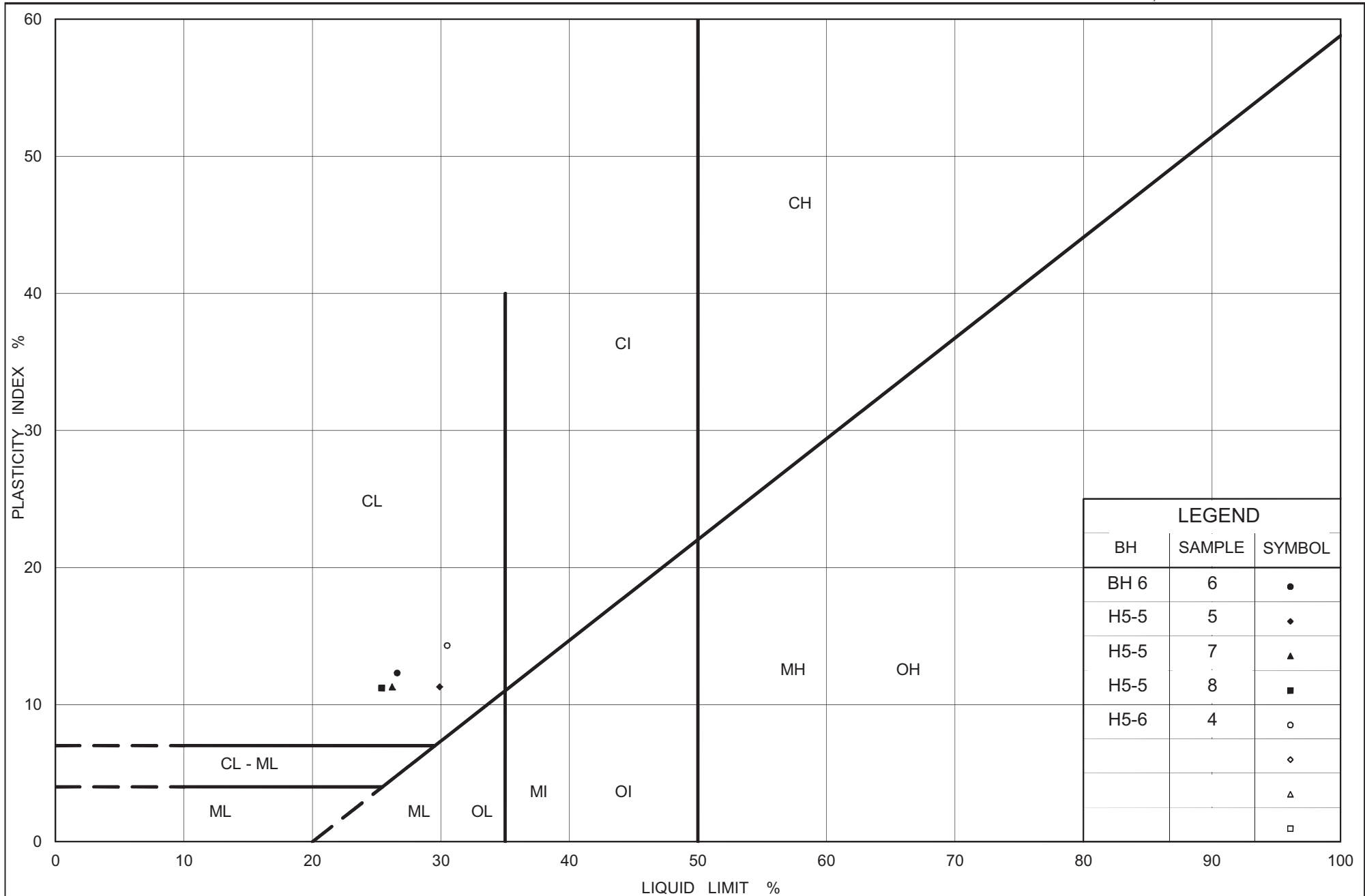
Ontario

PLASTICITY CHART Clayey Silt to Clayey Silt with Sand (TILL)

Figure No. B6A

Project No. 10-1184-0016

Checked By: TVA



Ministry of Transportation

Ontario

PLASTICITY CHART Clayey Silt to Clayey Silt with Sand (TILL)


Figure No. B6B

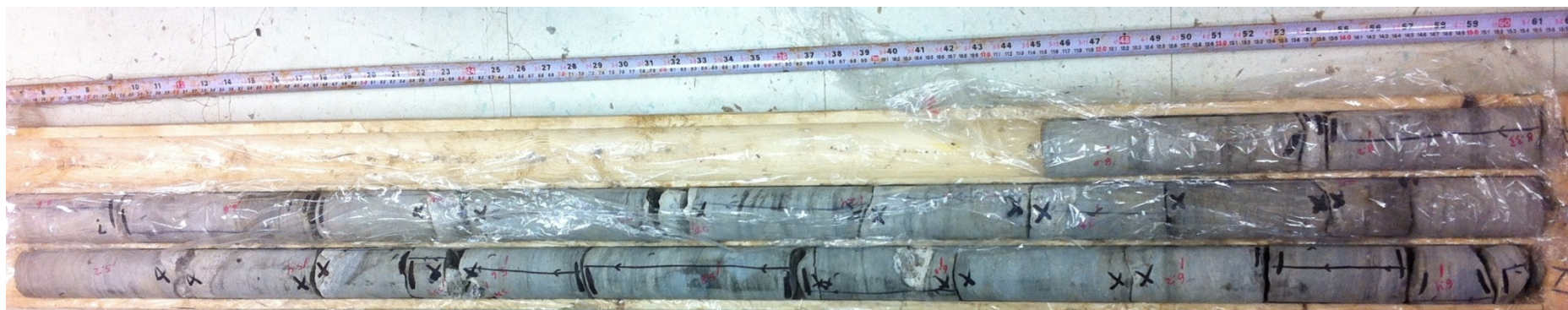
Project No. 10-1184-0016

Checked By: TVA




BH 2: Box 1 of 1: 5.72 m – 8.76 m

PROJECT HIGHWAY 5 OVER HIGHWAY 6 INTERCHANGE STRUCTURE CITY OF HAMILTON, MINISTRY OF TRANSPORTATION, ONTARIO GWP 2112-05-00				
TITLE Bedrock Core Photograph – Borehole BH 2				
	PROJECT No.: 10-1184-0016		FILE No. ----	
	DESIGN	--	APRIL 2013	SCALE AS SHOWN REV.
	CADD	--		
	CHECK	TVA	APRIL 2013	FIGURE B7
	REVIEW	JMAC	APRIL 2013	



BH 6: Box 1 of 1 : 5.00 m to 8.30 m

PROJECT HIGHWAY 5 OVER HIGHWAY 6 INTERCHANGE STRUCTURE CITY OF HAMILTON, MINISTRY OF TRANSPORTATION, ONTARIO GWP 2112-05-00				
TITLE Bedrock Core Photograph – Borehole BH 6				
	PROJECT No.: 10-1184-0016		FILE No. ----	
	DESIGN	--	APRIL 2013	SCALE AS SHOWN REV.
	CADD	--		FIGURE B9
	CHECK	TVA	APRIL 2013	
	REVIEW	JMAC	APRIL 2013	

UNCONFINED COMPRESSION TEST (UC)**ASTM D 7012-07****FIGURE B10****Sheet 1 of 2****SAMPLE IDENTIFICATION**

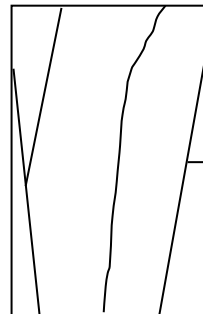
PROJECT NUMBER	10-1184-0016	SAMPLE NUMBER	Run 2
BOREHOLE NUMBER	2	SAMPLE DEPTH, m	8.1

TEST CONDITIONS

MACHINE SPEED, mm/min	-	TYPE OF SPECIMEN	Rock Core
DURATION OF TEST,min	>2 <15	L/D	2.22

SPECIMEN INFORMATION

SAMPLE HEIGHT, cm	10.54	WATER CONTENT, (specimen) %	0.08
SAMPLE DIAMETER, cm	4.74	UNIT WEIGHT, kN/m ³	26.20
SAMPLE AREA, cm ²	17.68	DRY UNIT WT., kN/m ³	26.18
SAMPLE VOLUME, cm ³	186.27	SPECIFIC GRAVITY	-
WET WEIGHT, g	497.83	VOID RATIO	-
DRY WEIGHT, g	497.43		

VISUAL INSPECTION**FAILURE SKETCH****TEST RESULTS**

STRAIN AT FAILURE, %	-	COMPRESSIVE STRESS, MPa	85.6
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REMARKS:

DATE:

1/24/2013

Checked By: TVA

Golder Associates

UNCONFINED COMPRESSION TEST

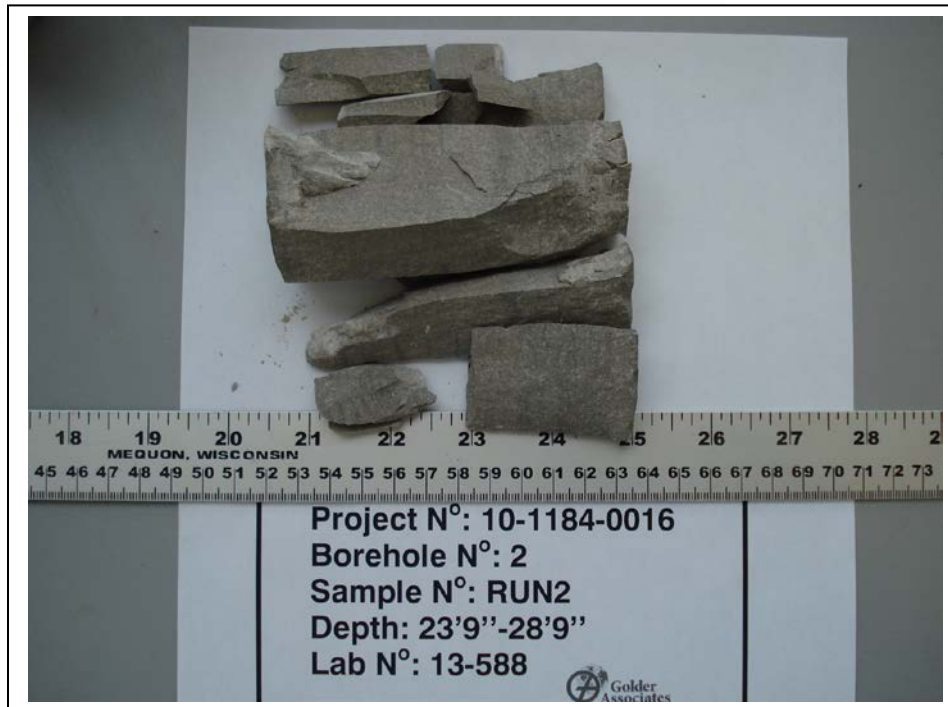
ASTM D7012-07

FIGURE B10

Sheet 2 of 2



BEFORE COMPRESSION



AFTER COMPRESSION

Date 1/25/2013
Project 10-1184-0016

Golder Associates

Drawn Frank
Chkd. TVA

UNCONFINED COMPRESSION TEST (UC)**ASTM D 7012-07****FIGURE B11****Sheet 1 of 2****SAMPLE IDENTIFICATION**

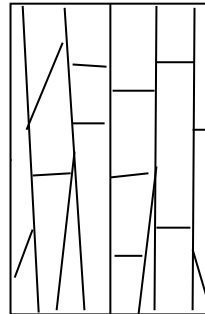
PROJECT NUMBER	10-1184-0016	SAMPLE NUMBER	Run 2
BOREHOLE NUMBER	3	SAMPLE DEPTH, m	6.9

TEST CONDITIONS

MACHINE SPEED, mm/min	-	TYPE OF SPECIMEN	Rock Core
DURATION OF TEST,min	>2 <15	L/D	2.25

SPECIMEN INFORMATION

SAMPLE HEIGHT, cm	10.61	WATER CONTENT, (specimen) %	0.21
SAMPLE DIAMETER, cm	4.72	UNIT WEIGHT, kN/m ³	26.38
SAMPLE AREA, cm ²	17.50	DRY UNIT WT., kN/m ³	26.32
SAMPLE VOLUME, cm ³	185.65	SPECIFIC GRAVITY	-
WET WEIGHT, g	499.56	VOID RATIO	-
DRY WEIGHT, g	498.51		

VISUAL INSPECTION**FAILURE SKETCH****TEST RESULTS**

STRAIN AT FAILURE, %	-	COMPRESSIVE STRESS, MPa	134.7
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REMARKS:

DATE:

1/24/2013

Checked By: TVA

Golder Associates

UNCONFINED COMPRESSION TEST

ASTM D7012-07

FIGURE B11

Sheet 2 of 2



BEFORE COMPRESSION



AFTER COMPRESSION

Date 1/25/2013
Project 10-1184-0016

Golder Associates

Drawn Frank
Chkd. TVA

UNCONFINED COMPRESSION TEST (UC)**ASTM D 7012-07****FIGURE B12****Sheet 1 of 2****SAMPLE IDENTIFICATION**

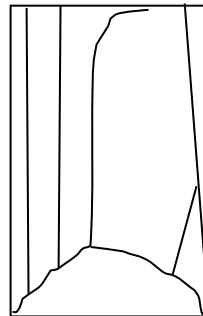
PROJECT NUMBER	10-1184-0016	SAMPLE NUMBER	Run 3
BOREHOLE NUMBER	6	SAMPLE DEPTH, m	8.10

TEST CONDITIONS

MACHINE SPEED, mm/min	-	TYPE OF SPECIMEN	Rock Core
DURATION OF TEST,min	>2 <15	L/D	2.22

SPECIMEN INFORMATION

SAMPLE HEIGHT, cm	10.48	WATER CONTENT, (specimen) %	0.11
SAMPLE DIAMETER, cm	4.71	UNIT WEIGHT, kN/m ³	25.84
SAMPLE AREA, cm ²	17.42	DRY UNIT WT., kN/m ³	25.81
SAMPLE VOLUME, cm ³	182.56	SPECIFIC GRAVITY	-
WET WEIGHT, g	481.25	VOID RATIO	-
DRY WEIGHT, g	480.72		

VISUAL INSPECTION**FAILURE SKETCH****TEST RESULTS**

STRAIN AT FAILURE, %	-	COMPRESSIVE STRESS, MPa	76.6
----------------------	---	-------------------------	------

REMARKS:

DATE:

1/24/2013

Checked By: TVA

Golder Associates

UNCONFINED COMPRESSION TEST

ASTM D7012-07

FIGURE B12

Sheet 2 of 2



BEFORE COMPRESSION



AFTER COMPRESSION

Date 1/25/2013
Project 10-1184-0016

Golder Associates

Drawn Frank
Chkd. TVA



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



DEEP FOUNDATIONS - Item No.

Non-Standard Special Provision

Where steel pile or caisson foundations are used for support of the bridge foundation elements, the foundations will extend to or into the dolostone bedrock, which is strong to very strong and which contains calcitic vugs at varying depths/elevations. Appropriate equipment and construction procedures will be required to penetrate into the bedrock to reach the founding level as required.

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



OBSTRUCTIONS - Item No.

Non-Standard Special Provision

The existing fill and the native cohesive till contain cobbles and/or shale fragments as indicated in the Record of Borehole sheets. Although not encountered in the boreholes advanced at this site, the cohesive (clayey silt to clayey silt with sand) till deposit should be expected to contain boulders. 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



VIBRATION MONITORING - Item No.

Non-Standard Special Provision

Scope

This special provision describes requirements for vibration monitoring during piling / caisson installation works for the construction of the Highway 5 over Highway 6 Interchange structure.

References

The subsurface conditions at the site are described in the following Foundation Investigation Report for GWP 2112-05-00:

Foundation Investigation Report, Highway 5 over Highway 6 Interchange Structure, Future Highway 5/6 Interchange (IC) and Associated Municipal Roads, City of Hamilton, Ministry of Transportation, Ontario, GWP 2112-05-00

Definitions

Quality Verification Engineer (QVE): An Engineer with a minimum of five (5) years experience in the field of installation of piling and vibration monitoring or, alternatively, with expertise demonstrated by providing satisfactory quality verification services for a minimum of two (2) projects of similar scope to the contract. The QVE shall be retained by the Contractor to ensure general conformance with the contract documents and issue certificates of conformance.

Submission Requirements

The Contractor/QVE shall submit details of the vibration monitoring plan to the Contract Administrator for review. The submittals shall satisfy the specifications and at a minimum contain the following specific information:

- Qualifications of vibration monitoring specialist.
- Details regarding proposed instrumentation.
- Proposed location of instruments.
- Proposed frequency of readings.
- Proposed methods for adjusting piling methods if readings show vibrations exceeding tolerable levels.

Monitoring

The vibration monitoring equipment shall be placed as close as possible to the piling works. The Contractor/QVE shall take readings on the existing structures during driving of each pile, starting with the pile furthest away for each span area.

The vibrations measured at the site shall not exceed 100 mm/s (peak particle velocity).



FOUNDATION REPORT – HIGHWAY 5 OVER HIGHWAY 6 INTERCHANGE STRUCTURE, GWP 2112-05-00

The results shall be submitted to the Contract Administrator after each pile has been driven, prior to continuing with the subsequent piles. As a minimum, the pile number, location, set criteria and driving log must be submitted with vibration monitoring results.

If the vibration monitoring results are acceptable, the Contractor may continue with the next pile(s) with readings taken during driving of each pile. The results of subsequent piles should be submitted to the Contract Administrator after each pile has been driven.

If the readings are not within the limits stated above, the Contractor must alter the driving procedures until the vibrations at the existing structures are within acceptable levels. The above process must be repeated for each pile.

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

At Golder Associates we strive to be the most respected global company providing consulting, design, and construction services in earth, environment, and related areas of energy. Employee owned since our formation in 1960, our focus, unique culture and operating environment offer opportunities and the freedom to excel, which attracts the leading specialists in our fields. 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 who operate from offices located throughout Africa, Asia, Australasia, Europe, North America, and South America.

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