



December 22, 2015

FOUNDATION INVESTIGATION AND DESIGN REPORT

**E-N RAMP RE-ALIGNMENT, BRIDGE / APPROACH EMBANKMENTS AND
ASSOCIATED STRUCTURES
NORMAN BETHUNE AVENUE AND HIGHWAY 404 S-E/W RAMP EXTENSION
FROM HIGHWAY 7 TO NORMAN BETHUNE AVENUE
YORK REGION TRANSPORTATION SERVICES, CONTRACT NO. T-14-53
CITY OF MARKHAM, ONTARIO**

Submitted to:

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GEOCRES NO: 30M14-425

Report Number: 1533525-1

Distribution:

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REPORT





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PART A

**FOUNDATION INVESTIGATION REPORT
E-N RAMP RE-ALIGNMENT, BRIDGE / APPROACH EMBANKMENTS AND
ASSOCIATED STRUCTURES
NORMAN BETHUNE AVENUE AND HIGHWAY 404 S-E/W RAMP
EXTENSION FROM HIGHWAY 7 TO NORMAN BETHUNE AVENUE
YORK REGION TRANSPORTATION SERVICES, CONTRACT NO. T-14-53
CITY OF MARKHAM, ONTARIO**



1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by AECOM on behalf of York Region to provide foundation engineering services in support of the design of the E-N Ramp, S-E/W Ramp extension and their associated structures, as well as a watermain replacement, at the northeast quadrant of the intersection of Highway 404 and Highway 7. This work is associated with the Midblock Crossing project in Markham, Ontario which was originally initiated by Coffey Geotechnics Inc. (Coffey Geotechnics).

This report addresses the Highway 404/Highway 7 E-N Ramp Re-alignment, bridge, approach embankments and retaining walls. Separate reports address the foundation investigations for the S-E/W Ramp Extension and watermain components of the project. The scope of work for foundation engineering services associated with the Midblock Crossing project is contained in Golder's proposal dated June 26, 2015.

The factual data, interpretations and recommendations contained in this report pertain to a specific project as described in the report and are not applicable to any other project or site location. If the project is modified in concept, location or elevation, or if the project is not initiated within eighteen months of the date of the report, Golder should be given an opportunity to confirm that the recommendations are still valid.

2.0 SITE DESCRIPTION

The E-N Ramp at the Highway 404 and Highway 7 is located east of Highway 404 and north of Highway 7. The site is located partially on Seneca College Markham Campus, partially on the City of Markham's property and partially within the Ministry of Transportation, Ontario (MTO) right-of-way. The existing E-N Ramp is located within a cut area, and does not have any associated structures.

The overall surface topography in the vicinity of the site is relatively flat with cut areas for the E-N Ramp and Highway 7. The area consists predominantly of commercial developments, an educational institution and a sports field on the Seneca College Markham Campus. The ground surface at the site ranges between approximately Elevations 195 m and 187 m.

3.0 INVESTIGATION PROCEDURES

3.1 Previous Borehole Investigation

Six boreholes were advanced at this site as part of previous geotechnical investigations carried out by Coffey Geotechnics for the Highway 7 and Highway 404 ramps associated with the Midblock Crossing. The investigation was carried out by Coffey Geotechnics in December 2007, January 2008 and May 2011 during which time Boreholes B101 to B104, BH 3 and BH 4 were advanced in the vicinity of the Ramp Bridge, to depths between about 13 m and 26 m. The approximate locations of the boreholes advanced in the Highway 7 area associated with the various facilities are shown on Drawing 1 and the locations of the boreholes specific to the N-E/W Ramp Re-Alignment, bridge, approach embankments and retaining structures are shown on Drawings 2 and 3.

The boreholes were advanced using solid stem augers, hollow stem augers and washboring techniques, and soil samples were obtained at intervals of depth of about 0.75 m and 1.5 m, using a 50 mm outer diameter



split-spoon sampler driven by an automatic hammer in accordance with the Standard Penetration Test (SPT) procedure (ASTM D1586).

The water level in the open boreholes was observed during and following the drilling operations, and piezometers were installed in Boreholes B102 and B103 to allow monitoring of the groundwater level at the site.

3.2 Current Borehole Investigation

A geotechnical field investigation was also carried out by Golder at the site of the E-N Ramp between September 9 and 14, 2015, during which time a total of twelve boreholes (Boreholes GA-BR-01 to GA-BR-03, GA-WM-01 to GA-WM-06 and GA-HF-01 to GA-HF-03) were advanced to supplement the existing (Coffey Geotechnics) subsurface information. The locations of these specific boreholes are shown in plan on Drawings 1, 2 and 3.

The borehole investigation was carried out using a track-mounted CME-75 drill rig and a track-mounted Acker Soil-Max drill rig supplied and operated by Lantech Drilling Service Inc. of Sharon, Ontario. The boreholes were advanced through the overburden using 210 mm outside diameter hollow stem augers and mud rotary drilling techniques. Soil samples were generally obtained at intervals of depth of about 0.75 m and 1.5 m, using a 50 mm outside diameter split-spoon sampler driven by manual hammers in accordance with the Standard Penetration Test (SPT) procedures (ASTM D1586). The boreholes were advanced to depths ranging from about 3.5 m to 18.7 m below existing ground surface.

The groundwater conditions and water level in the open boreholes were observed during and immediately following the completion of drilling operations. A piezometer was installed in each of Boreholes GA-BR-01, GA-BR-02, GA-WM-02, and GA-WM-04 to GA-WM-06 to allow monitoring of the groundwater level at this site. The piezometers consist of a 50 mm diameter PVC pipe, with a slotted screen sealed within the silt and sand to sand deposits in Boreholes GA-BR-01, GA-BR-02, GA-WM-05 and GA-WM-06, and within the sandy silt to silty sand till deposit in Boreholes GA-WM-02 and GA-WM-04. The borehole and annulus surrounding the piezometer pipe above the screen and sand pack were backfilled with bentonite pellets to the ground surface. The piezometer installation details and water level readings are noted on the Record of Boreholes in Appendix A. All other boreholes were backfilled upon completion of drilling in accordance with Ontario Regulation 903 (as amended).

Samples of groundwater were collected from select wells after completion of the field investigation, using appropriate sampling protocols and submitted to a specialist analytical laboratory under chain of custody procedures for testing for a suite of parameters. The results of the analytical testing are provided in Table B1 in Appendix B.

The field work was observed by members of Golder's engineering 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 samples. The samples were identified in the field, placed in appropriate containers, labelled and transported to Golder's Mississauga and Whitby geotechnical laboratories where the samples underwent further visual examination and laboratory testing. All of the laboratory tests were carried out to MTO and/or ASTM Standards, as appropriate. Classification testing (water content, grain size distribution and Atterberg limits) was carried out on selected soil samples. The results of the laboratory testing



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are presented on the Record of Borehole sheets in Appendix A and on the laboratory test results included in Appendix B.

The proposed borehole locations and the ground surface elevations were staked and surveyed by J.D. Barnes Limited (J.D. Barnes), a professional surveying company, and the as-drilled borehole locations were surveyed by Golder relative to the staked locations. The borehole locations provided on the Records of Boreholes and shown on Drawings 1, 2 and 3 are given using UTM NAD 83 northing and easting coordinates, and the ground surface elevations are referenced to Geodetic datum. The borehole locations, including the MTM NAD 83 coordinates, ground surface elevations and drilled depths are summarized below.

Borehole Number	Location in UTM NAD83 (MTM NAD 83)		Ground Surface Elevation	Borehole Depth
	Northing	Easting		
GA-BR-01	4856282.4 m (4856454.6 m)	631119.8 m (315354.5 m)	189.8 m	17.2 m
GA-BR-02	4856302.6 m (4856475.4 m)	631090.9 m (315325.9 m)	190.4 m	18.7 m
GA-BR-03	4856328.0 m (4856501.2 m)	631066.7 m (315302.2 m)	190.7 m	14.2 m
GA-WM-01	4856357.2 m (4856524.3 m)	631401.5 m (315637.5 m)	187.4 m	6.6 m
GA-WM-02	4856330.0 m (4856498.5 m)	631328.2 m (315563.7 m)	186.7 m	8.1 m
GA-WM-03	4856319.8 m (4856489.6 m)	631251.3 m (315486.6 m)	188.8 m	8.1 m
GA-WM-04	4856304.4 m (4856474.8 m)	631219.3 m (315454.3 m)	188.5 m	11.1 m
GA-WM-05	4856299.3 m (4856470.9 m)	631156.0 m (315390.9 m)	190.1 m	12.7 m
GA-WM-06	4856308.3 m (4856480.8 m)	631109.3 m (315344.4 m)	190.5 m	12.7 m
GA-HF-01	4856286.9 m (4856458.3 m)	631163.2 m (315399.5 m)	188.9 m	3.5 m
GA-HF-02	4856325.8 m (4856499.8 m)	631019.6 m (315255.0 m)	192.6 m	6.5 m
GA-HF-03	4856373.9 m (4856548.3 m)	631001.2 m (315237.5 m)	193.4 m	3.5 m



4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

The site at the intersection of Highway 404 and Highway 7 is located within the Peel Plain physiographic region, as delineated in *The Physiography of Southern Ontario* (Chapman and Putnam, 1984). The Peel Plain physiographic region covers the central portions of the Regional Municipalities of York, Peel and Halton. The general topography of this region consists of level to gently rolling terrain, sloping down gradually southward toward Lake Ontario. A surficial till sheet, which generally follows the surface topography, is present throughout much of this area. The till typically consists of clayey silt to silty clay, with occasional sand to silt zones. Shallow, localized deposits of loose sand and silt and/or soft clay can overlie this uppermost till sheet, and these represent relatively recent deposits, formed in small glacial meltwater ponds scattered throughout the Peel Plain and concentrated near river valleys. The recent sand, silt and clay and uppermost till deposits in this area overlie and are interbedded with stratified deposits of sand, silt and clay.

4.2 Subsurface Conditions

The detailed subsurface soil and groundwater conditions encountered in the boreholes advanced as part of the current investigation, together with the results of in situ and laboratory testing, are presented on the Record of Borehole sheets and laboratory test summary figures provided in Appendices A and B, respectively. The Record of Borehole sheets and laboratory testing results from the previous (Coffey Geotechnics) investigation are presented in Appendix C. The interpreted stratigraphic profile and cross-sections are shown on Drawings 2 to 4.

The results of the in situ field tests (i.e. SPT 'N'-values) carried out during the current investigation as presented on the Record of Borehole sheets and in Section 4.2 are uncorrected. According to the Canadian Foundation Engineering Manual (*CFEM*, 2006), the energy delivered to the drill rod varies with the hammer release system, hammer type, anvil and operator characteristics. It should be noted that different hammer release systems were used during the previous and current investigations (i.e. automatic versus manual) and as such SPT 'N'-values measured during the previous investigation, and in the boreholes of the current investigation may vary within the same deposit.

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

In general, the subsurface conditions at the site consist of a layer of topsoil or fill, underlain by a clayey silt to silty clay deposit, which is in turn underlain by a sandy silt to sand deposit. The sandy silt to sand deposit is underlain by a sandy silt to sand till along the southeastern extend of the site and underlain by a lower clayey silt deposit encountered at the southwest of the site.

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



It should be noted that the previous (Coffey Geotechnics) investigation describes portions of deposits as “till” units (i.e. sandy silt to sand till and clayey silt till), however, the current investigation did not encounter such similar units and the deposits are classified as silty sand to sand and clayey silt.

4.3 E-N Ramp Alignment

The plan and profile along the centerline of the E-N Ramp, showing the borehole locations and interpreted stratigraphy is shown on Drawing 2. A total of eighteen boreholes (GA-BR-01 to GA-BR-03, GA-WM-01 to GA-WM-06, GA-HF-01 to GA-HF-03, B101 to B104, BH 3 and BH 4) were completed to investigate the subsurface conditions along the E-N Ramp alignment.

4.3.1 Topsoil

An approximately 80 mm to 200 mm thick layer of topsoil was encountered in Boreholes GA-WM-03 to GA-WM-06, GA-BR-01 to GA-BR-03, GA-HF-01 to GA-HF-03, B101 to B104, BH 3 and BH 4.

4.3.2 Fill

A 0.8 m to 1.4 m thick layer of cohesive fill consisting of sandy clayey silt was encountered at ground surface in Boreholes GA-WM-01 to GA-WM-03. Wood fragments and rootlets were encountered with the clayey silt fill in Borehole GA-WM-03. A 0.3 m to 1.9 m thick layer of non-cohesive fill consisting of sandy silt to silty sand was encountered below the topsoil in Boreholes B101 to B104, BH 3 and BH 4. Clayey silt inclusions were found within the fill in Boreholes B102 to B104. The top of the fill layer ranges from Elevation 190.6 m to 186.7 m.

A SPT ‘N’-value measured within the cohesive fill layer is 9 blows per 0.3 m of penetration, suggesting a firm to stiff consistency. The SPT ‘N’-values measured within the non-cohesive fill range from 3 blows to 20 blows per 0.3 m of penetration, indicating a very loose to compact relative density.

The natural water content measured on a sample of the cohesive fill from the current investigation is about 16 per cent.

The grain size distributions of one sample from the clayey silt component of the sandy silt to silty sand fill from previous investigations is shown on B1-1 in Appendix C.

4.3.3 Clayey Silt to Silty Clay (Upper)

A 1.1 m to 3.8 m thick upper deposit of clayey silt to silty clay was encountered between Elevations 190.5 m and 185.9 m, below the fill in Boreholes GA-WM-01 to GA-WM-03, B101 to B103, and BH 4 and below the topsoil in Boreholes GA-BR-02, GA-BR-03, GA-WM-04 to GA-WM-06, and GA-HF-01. Sandy silt to sand layers, 80 mm and 150 mm thick, were encountered in Boreholes GA-WM-06 and GA-WM-04, respectively.

The SPT ‘N’-values measured within the clayey silt to silty clay deposit generally range from 15 blows to 47 blows per 0.3 m of penetration, suggesting a very stiff to hard consistency. The SPT ‘N’-values, in



Boreholes GA-WM-01 and GA-WM-03 at depths where auger grinding was noted on inferred cobbles, measured 50 blows per 0.15 m of penetration.

The natural water content measured on ten samples of the clayey silt to silty clay deposit from the current investigation range from about 9 per cent to 25 per cent. The natural water contents measured on the sand layer and sandy silt seam is about 23 per cent and 13 per cent respectively.

The results of two grain size distribution tests completed on samples of the clayey silt from the current investigation are shown on Figure B1 in Appendix B. The grain size distributions of two samples of the silty clay from previous investigations are shown on Figure B2-1 in Appendix C.

Atterberg limits tests carried out on four samples of the clayey silt deposit obtained during the current investigation measured liquid limits between about 20 per cent and 27 percent, plastic limits of between about 13 per cent and 18 per cent, corresponding to plasticity indices between about 7 per cent and 12 per cent. An Atterberg limits test carried out on a sample of the silty clay obtained during the current investigation measured a liquid limit of about 41 per cent and a plastic limit of about 19 per cent, corresponding to a plasticity index of about 22 per cent. The results of the Atterberg limits tests are shown on the plasticity chart on Figure B2 in Appendix B and indicate that the material is classified as a clayey silt of low plasticity and a silty clay of intermediate plasticity, respectively. An Atterberg limits test carried out on one sample of the sandy silt seam in Borehole GA-WM-06 indicates that the material is non-plastic. An Atterberg limits test carried out on one sample of the silty clay obtained during the previous investigation measured a liquid limit of about 42 per cent and a plastic limit of about 23 per cent, corresponding to a plasticity index of about 19 per cent. The test result from the previous investigation is shown on the plasticity chart on Figure B2-2 in Appendix C, and indicates that the deposit also consists of silty clay of intermediate plasticity.

4.3.4 Sandy Silt to Sand

A deposit of sandy silt to silt and sand to silty sand to sand, with typical thickness between 3.3 m and 20.1 m was generally encountered between Elevations 193.4 m and 186.7 m. This deposit was encountered below the clayey silt to silty clay in Boreholes GA-BR-02, GA-BR-03, GA-WM-01 to GA-WM-06, GA-HF-01, B101 to B103 and BH 4, below the topsoil in Boreholes GA-BR-01, GA-HF-02 and GA-HF-03, and below the fill in Boreholes BH 3 and B104.

In Boreholes GA-WM-01 to GA-WM-05 and GA-HF-01, along the southwest side of the site, the deposit was encountered between Elevations 188.0 m and 183.8 m and is between 0.4 m and 2.0 m thick. In this area the deposit is underlain by a sandy silt to sand till deposit, described below, with the exception of GA-WM-05 where the till deposit occurs as an interlayer. Additionally, this deposit contains a 0.9 m gravel and cobbles interlayer in Borehole GA-BR-03 and a 3.4 m thick pocket of clayey silt in Borehole B103. A 0.6 m clayey silt pocket was encountered in Borehole GA-HF-01 located directly below the silt and sand deposit. This deposit generally contains silt and clayey silt interbeds, and cobbles, as inferred by auger grinding. Boreholes GA-BR-02, GA-BR-03, GA-WM-06, GA-HF-02, GA-HF-03 B101 and BH 4 were terminated within the sandy silt to sand deposit.

The SPT 'N'-values measured within the sandy silt to sand deposit range from 8 blows per 0.3 m of penetration to 100 blows per 0.1 m of penetration indicating a loose to very dense relative density. In general, the relative



density of the deposit increases with depth. A SPT 'N'-value measured within the clayey silt pocket in Borehole GA-HF-01 is 40 blows per 0.3 m of penetration, suggesting a hard consistency.

The natural water content measured on 37 samples of the sandy silt to sand deposit from the current investigation range from about 5 per cent to 26 per cent.

The results of grain size distribution tests completed on fifteen samples of the sandy silt to sand deposit from the current investigation are shown on Figures B3A to B3C in Appendix B. The grain size distributions of fourteen samples of the silty sand to sand deposit from the previous investigation are shown on Figures B3-1 and B4-1 in Appendix C. The grain size distributions of one sample of a sand and gravel interlayer and one sample of a clayey silt interbed within the deposit from the previous investigation are shown on Figures B3-2 and B3-3 in Appendix C, respectively.

The Atterberg limits tests carried out on nine samples of the sandy silt to sand deposit indicated that the material is non-plastic.

4.3.5 Sandy Silt to Sand Till

A 0.6 m to 8.0 m till deposit, comprised of sandy silt to sand, was encountered at between Elevations 186.0 m and 183.0 m, below the upper clayey silt deposit in Borehole GA-WM-03, below the silt and sand deposit in Boreholes GA-WM-01, GA-WM-02, and GA-WM-04, below a clayey silt pocket in GA-HF-01 and within the sandy silt to sand deposit in Borehole GA-WM-05. The presence of cobbles was inferred by auger grinding within this deposit in Boreholes GA-WM-04 to GA-WM-06. Boreholes GA-WM-01 to GA-WM-04, and GA-HF-01 were terminated within this deposit.

The SPT 'N'-values measured within the sandy silt to sand till deposit range from 8 blows per 0.3 m of penetration to 151 blows per 0.18 m of penetration, indicating a loose to very dense relative density.

The natural water content measured on twelve samples of the sandy silt to sand till from the current investigation range from about 5 per cent to 23 per cent.

The results of grain size distribution tests completed on five samples of the sandy silt to sand till deposit from the current investigation are shown on Figure B4 in Appendix B.

The Atterberg limits tests carried out on four samples of the sandy silt to sand till deposit indicate that the material is non-plastic.

4.3.6 Clayey Silt (Lower)

A 0.2 m to 7.8 m thick lower deposit of clayey silt was encountered between Elevations 180.7 m and 167.3 m, below the sandy silt to sand deposit in Boreholes GA-BR-01, GA-WM-05, BH 3, B102, B103, and B104. These boreholes were terminated within the lower clayey silt deposit.

The SPT 'N'-values measured within the lower clayey silt deposit range from 26 blows per 0.3 m of penetration to 100 blows per 0.13 m of penetration, suggesting a very stiff to hard consistency.

The natural water content measured on three samples of the lower clayey silt deposit from the current investigation range from about 14 per cent to 27 per cent.



The grain size distributions of two samples of the lower clayey silt deposit from previous investigations are shown on Figure B5-1 in Appendix C.

Atterberg limits tests carried out on two samples of the clayey silt deposit obtained during the current investigation measured liquid limits of about 28 per cent and 29 percent and plastic limits of about 15 per cent and 17 per cent, corresponding to plasticity indices of about 13 per cent and 12 per cent. The results of the Atterberg limits tests are shown on the plasticity chart on Figure B5 in Appendix B and indicate that the material is classified as a clayey silt of low plasticity. Atterberg limits tests carried out on two samples of the clayey silt obtained during the previous investigation measured liquid limits of about 27 per cent and 30 per cent and plastic limits of about 14 per cent and 18 percent, corresponding to plasticity indices of about 13 per cent and 12 per cent. The results from the previous investigation are shown on the plasticity chart on Figure B5-2 in Appendix C, and also indicate that the material is classified as clayey silt of low plasticity.

4.4 E-N Ramp Bridge – East Abutment and Approach

The plan and profile along the centerline of the east abutment/approach and a cross-section at the east abutment of the E-N Ramp Bridge, showing the borehole locations and interpreted stratigraphy are shown on Drawings 3 and 4. A total of six boreholes (GA-BR-01, GA-WM-06, GA-HF-01, B103, B104 and BH 3) were completed to investigate the subsurface conditions at the east abutment and approach of the E-N Ramp Bridge.

4.4.1 Topsoil

An approximately 80 mm to 200 mm thick layer of topsoil was encountered in Boreholes GA-BR-01, GA-WM-06, GA-HF-01, B103, B104 and BH 3.

4.4.2 Fill

A 0.3 m to 0.6 m thick layer of non-cohesive fill consisting of sandy silt to silty sand was encountered below the topsoil in Boreholes B103, B104 and BH 3. Clayey silt inclusions were found within the fill in Borehole B103. The top of the fill layer ranges from Elevations 190.4 m to 189.1 m.

The SPT 'N'-values measured within the non-cohesive fill ranges from 5 blows to 16 blows per 0.3 m of penetration, indicating a loose to compact relative density.

4.4.3 Clayey Silt to Silty Clay (Upper)

A 1.4 m to 3.0 m thick upper deposit of clayey silt to silty clay, in places interlayered with sandy silt seams was encountered between Elevations 190.4 m and 188.8 m, below the fill in Borehole B103 and below the topsoil in Boreholes GA-WM-06, and GA-HF-01.

The SPT 'N'-values measured within the clayey silt to silty clay deposit range from 17 blows to 34 blows per 0.3 m of penetration, suggesting a very stiff to hard consistency.



The natural water content measured on a sample of the clayey silt from the current investigation is about 15 per cent. The natural water contents measured on a sample from the sandy silt seam is about 13 per cent.

The grain size distributions of one sample of the silty clay from previous investigations are shown on Figure B2-1 in Appendix C.

An Atterberg limits test carried out on one sample of the sandy silt seam in Borehole GA-WM-06 indicated that the seam is non-plastic.

4.4.4 Sandy Silt to Sand

A deposit of sandy silt to silt and sand to silty sand to sand, with thickness between 0.6 m and 16.6 m was encountered between Elevations 189.7 m and 187.5 m. This deposit was encountered below the upper deposit of clayey silt to silty clay in Boreholes GA-WM-06, GA-HF-01 and B103, below the topsoil in Borehole GA-BR-01 and below the fill in Boreholes BH 3 and B104. Additionally, a 0.6 m pocket of clayey silt is present below the silt and sand deposit in Borehole GA-HF-01, a 3.4 m thick pocket of clayey silt was encountered in Borehole B103 and cobbles as inferred from auger grinding was encountered in Borehole GA-WM-06. Borehole GA-WM-06 was terminated within the sandy silt to sand deposit.

The SPT 'N'-values measured within the sandy silt to sand deposit range from 9 blows per 0.3 m of penetration to 100 blows per 0.1 m of penetration indicating a loose to very dense relative density. In general, the relative density of the deposit increases with depth. A SPT 'N'-value measured within the clayey silt pocket is 40 blows per 0.3 m of penetration to suggesting a hard consistency.

The natural water content measured on 14 samples of the sandy silt to sand deposit from the current investigation range from about 5 per cent to 25 per cent.

The results of grain size distribution tests completed on five samples of the sandy silt to sand deposit from the current investigation are shown on Figures B3B and B3C in Appendix B. The grain size distributions of nine samples of the silty sand to sand deposit from the previous investigations are shown on Figures B3-1 and B4-1 in Appendix C. The grain size distributions of one sample of a sand and gravel interlayer and a clayey silt interbeds within the deposit from the previous investigation are shown on Figure B3-2 and B3-3, respectively, in Appendix C.

Atterberg limits tests carried out on two samples of the sandy silt to sand deposit indicate that the material is non-plastic.

4.4.5 Sand Till

A 0.6 m thick deposit of sand till was encountered at Elevation 186.0 m below a clayey silt pocket in Borehole GA-HF-01. The borehole was terminated within this deposit.

The SPT 'N'-value measured within the sand till is 25 blows per 0.3 m of penetration suggesting a compact relative density.

The natural water content measured on one sample of the sand till from the current investigation is about 9 per cent.



Atterberg limits tests carried out on four samples of the sand till deposit indicate that the material is non-plastic.

4.4.6 Clayey Silt (Lower)

A 0.2 m to 7.8 m thick lower deposit of clayey silt was encountered between Elevations 179.0 m and 172.1 m below the sandy silt to sand deposit in Boreholes GA-BR-01, BH 3, B103, and B104, all of which were terminated within this deposit.

The SPT 'N'-values measured within the lower clayey silt deposit range from 26 blows per 0.3 m of penetration to 100 blows per 0.13 m of penetration, suggesting a very stiff to hard consistency.

The natural water content measured on two samples of the lower clayey silt from the current investigation are about 18 per cent and 27 per cent.

The grain size distributions of two sample of the lower clayey silt from the previous investigation are shown on Figure B5-1 in Appendix C.

An Atterberg limits tests carried out on one sample of the lower clayey silt deposit obtained during the current investigation measured a liquid limit of about 29 per cent and a plastic limit of about 17 per cent, corresponding to a plasticity index of about 12 per cent. The result of the Atterberg limits test is shown on the plasticity chart on Figure B5 in Appendix B and indicates that the material is classified as a clayey silt of low plasticity. Atterberg limits tests carried out on two samples of the clayey silt obtained during the previous investigation measured liquid limits of about 27 per cent and 30 per cent and plastic limits of about 14 per cent and 18 percent, corresponding to plasticity indices of about 13 per cent and 12 per cent. The results of the tests from the previous investigation are shown on the plasticity chart on Figure B5-2 in Appendix C, and indicate that the material is clayey silt of low plasticity.

4.5 E-N Ramp Bridge – West Abutment and Approach

The plan and profile along the centerline of the west abutment/approach and a cross-section of the west abutment of the E-N Ramp Bridge, showing the borehole locations and interpreted stratigraphy are shown on Drawings 3 and 4. A total of five boreholes (GA-BR-02, GA-BR-03, B101, B102, and BH 4) were completed to investigate the subsurface conditions at the west abutment and approach of the E-N Ramp Bridge.

4.5.1 Topsoil

An approximately 100 mm to 200 mm thick layer of topsoil was encountered in Boreholes GA-BR-02, GA-BR-03, B101, B102, and BH 4.

4.5.2 Fill

A 0.7 m to 1.9 m thick layer of non-cohesive fill consisting of sandy silt to silty sand was encountered below the topsoil in Boreholes B101, B102 and BH 4. Clayey silt inclusions were found within the fill in Borehole B102. The top of the fill layer ranges from Elevations 190.6 m to 190.3 m.



The SPT 'N'-values measured within the layer of non-cohesive fill range from 3 blows to 20 blows per 0.3 m of penetration, indicating a very loose to compact relative density.

The grain size distribution of one sample of the clayey silt fill component of the sandy silt to silty sand fill layer from the previous investigation is shown on B1-1 in Appendix C.

4.5.3 Clayey Silt to Silty Clay (Upper)

A 1.1 m to 3.8 m thick upper deposit of clayey silt to silty clay was encountered between Elevations 190.5 m and 188.5 m, below the fill in Boreholes B101, B102 and BH 4 and below the topsoil in Boreholes GA-BR-02 and GA-BR-03.

The SPT 'N'-values measured within the clayey silt to silty clay deposit range from 15 blows to 47 blows per 0.3 m of penetration, suggesting a very stiff to hard consistency.

The natural water content measured on five samples of the upper clayey silt deposit from the current investigation range from about 9 per cent to 25 per cent.

The result of a grain size distribution test completed on one sample of the clayey silt deposit from the current investigation is shown on Figure B1 in Appendix B. The grain size distribution of one sample of the silty clay portion of the deposit from the previous investigation is shown on Figure B2-1 in Appendix C.

Atterberg limits tests carried out on two samples of the clayey silt deposit obtained during the current investigation measured liquid limits of about 21 per cent and 26 per cent and plastic limits of about 13 per cent and 14 per cent, corresponding to plasticity indices of about 8 per cent and 12 per cent. The results of the Atterberg limits test are shown on the plasticity chart on Figure B2 in Appendix B and indicate that the material is classified as a clayey silt of low plasticity. An Atterberg limits test carried out on one sample of the silty clay portion of the deposit obtained during the previous investigation measured a liquid limit of about 42 per cent and a plastic limit of about 23 per cent, corresponding to a plasticity index of about 19 per cent. The result of the test from the previous investigation is shown on the plasticity chart on Figure B2-2 in Appendix C and indicates that the deposit also consists of silty clay of intermediate plasticity.

4.5.4 Sandy Silt to Sand

A deposit of sandy silt to silt and sand to silty sand to sand, ranging in thickness between 9.5 m and 20.1 m was encountered between Elevations 188.3 m and 186.7 m. This deposit was encountered below the clayey silt to silty clay in Boreholes GA-BR-02, GA-BR-03, B101, B102, and BH 4. Additionally, a 0.9 m thick interlayer of gravel and cobbles was encountered within the silt and sand to silty sand deposit in Borehole GA-BR-03 and silt and clayey silt interbeds, and cobbles, as inferred from auger grinding in places, were also encountered in Boreholes GA-BR-02, GA-BR-03, B102 and BH 4. Boreholes GA-BR-02, GA-BR-03, B101 and BH 4 were terminated within the sandy silt to sand deposit.

The SPT 'N'-values measured within the sandy silt to sand deposit range from 12 blows per 0.3 m of penetration to 100 blows per 0.1 m of penetration indicating a compact to very dense relative density. In general, the relative density of the deposit increases with depth.



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The natural water content measured on ten samples of the sandy silt to sand deposit from the current investigation range from about 8 per cent to 26 per cent.

The results of grain size distribution tests completed on five samples of the sandy silt to sand deposit from the current investigation are shown on Figures B3B and B3C in Appendix B. The grain size distributions of seven samples of the silty sand to sand deposit from the previous investigation are shown on Figures B3-1 and B4-1 in Appendix C.

Atterberg limits tests carried out on four samples of the sandy silt to sand deposit indicated that the material is non-plastic.

4.5.5 Clayey Silt (Lower)

A 1.4 m thick lower deposit of clayey silt was encountered at Elevation 167.3 m below the silty sand to sand deposit in Borehole B102, which was terminated within this deposit.

An SPT 'N'-value measured within the lower clayey silt deposit is 100 blows per 0.15 m of penetration, suggesting a hard consistency.

4.6 Groundwater Conditions

In general, the overburden samples taken in the boreholes were moist to wet. The water level encountered during drilling and upon completion of drilling is between approximately Elevations 187.5 m and 181.6 m. However, the water level observed in the open boreholes during and/or on completion of drilling may not represent the longer-term, stabilized groundwater level at the site. Boreholes GA-WM-01 and GA-HF-01 to GA-HF-03 were dry upon completion of drilling.

A standpipe piezometer was installed in each of Boreholes GA-BR-01, GA-BR-02, GA-WM-02 and GA-WM-04 to GA-WM-06, as part of the current investigation, and a standpipe piezometer was installed in Borehole B102 and B103 during the previous investigation. The observed groundwater level in the standpipe piezometers is shown on the Record of Borehole sheets in Appendices A and C and summarized below.

Borehole	Depth to Water Level	Groundwater Elevation	Date of Measurement
GA-BR-01	4.1 m	185.7 m	September 14, 2015
	3.9 m	185.9 m	September 15, 2015
	4.0 m	185.8 m	October 13, 2015
GA-BR-02	4.4 m	186.0 m	September 11, 2015
	4.8 m	185.6 m	September 11, 2015
	4.6 m	185.8 m	September 15, 2015
	4.8 m	185.6 m	October 13, 2015
GA-WM-02	2.7 m	184.0 m	September 11, 2015
	2.9 m	183.8 m	September 15, 2015
	2.9 m	183.8 m	October 13, 2015
GA-WM-04	Dry	--	September 11, 2015
	Dry	--	September 15, 2015
	Dry	--	October 13, 2015



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Borehole	Depth to Water Level	Groundwater Elevation	Date of Measurement
GA-WM-05	4.6 m	185.5 m	September 11, 2015
	4.7 m	185.4 m	September 15, 2015
	4.6 m	185.5 m	October 13, 2015
GA-WM-06	5.0 m	185.5 m	September 10, 2015
	4.9 m	185.6 m	September 11, 2015
	4.9 m	185.6 m	September 15, 2015
	4.9 m	185.6 m	October 13, 2015
B102	4.6 m	186.0 m	May 31, 2011
	4.3 m	186.3 m	June 1, 2011
	4.3 m	186.3 m	June 8, 2011
	4.3 m	186.3 m	June 21, 2011
	4.3 m	186.3 m	June 22, 2011
	4.3 m	186.3 m	June 30, 2011
B103	4.3 m	186.3 m	May 27, 2011
	4.3 m	186.3 m	June 8, 2011
	4.4 m	186.2 m	June 21, 2011
	4.4 m	186.2 m	June 22, 2011
	4.3 m	186.3 m	June 30, 2011

The water level at the site is expected to fluctuate seasonally in response to changes in precipitation and snow melt, and is expected to be higher during the spring and periods of precipitation.



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5.0 CLOSURE

Ms. Madison Kennedy, B.A.Sc., Messrs. Alex Szot, E.I.T. and Oleg Skorik, E.I.T., supervised the borehole investigation program. This report was prepared by Ms. Madison Kennedy, B.A.Sc., and was reviewed by Mr. Christopher Ng, P.Eng., a senior geotechnical engineer and Associate of Golder. Mr. Jorge M. A. Costa, P.Eng., a Principal of Golder and Designated MTO Foundations Contact, conducted an independent review of this report.

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**FOUNDATION REPORT - E-N RAMP RE-ALIGNMENT BRIDGE /
APPROACH EMBANKMENTS AND ASSOCIATED
STRUCTURES**

PART B

**FOUNDATION DESIGN REPORT
E-N RAMP RE-ALIGNMENT, BRIDGE / APPROACH EMBANKMENTS AND
ASSOCIATED STRUCTURES
NORMAN BETHUNE AVENUE AND HIGHWAY 404 S-E/W RAMP
EXTENSION FROM HIGHWAY 7 TO NORMAN BETHUNE AVENUE
YORK REGION TRANSPORTATION SERVICES, CONTRACT NO. T-14-53
CITY OF MARKHAM, ONTARIO**



6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

This section of the report provides engineering design recommendations for the proposed E-N Ramp re-alignment associated with the Highway 404/Highway 7 interchange rehabilitation from Highway 7 to Bethune Avenue in Markham, 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 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 during construction. Those requiring information on aspects of construction should make their own interpretation of the factual information provided as such interpretation may affect equipment selection, proposed construction methods, scheduling and the like.

6.1 General

Golder Associates Ltd. (Golder) has been retained by AECOM (formerly URS Canada Inc.) on behalf of York Region (York) to provide recommendations on foundation aspects for the detail design of the proposed E-N Ramp re-alignment associated with the Highway 404/Highway 7 interchange in Markham, Ontario. The proposed work includes a bridge over the future S-E/W Ramp Extension, retaining walls along the bridge abutments, approach embankment and concrete toe walls.

Based on the General Arrangement (GA) drawing provided by AECOM on October 6, 2015, the proposed E-N Ramp Bridge will consist of a single-span, pre-cast girder structure with a span length of 38 m. The grade of the proposed bridge deck will be at about Elevation 194.7 m and 196.0 m at the east and west abutments, respectively, corresponding to a grade raise of up to about 4.9 m and 5.6 m above the original ground surface at the east and west approach embankment, respectively. In comparison, the proposed grade for the future S-E/W Ramp Extension is at about Elevation 186.6 m. The underside of the caps for a proposed pile foundation would be at about Elevation 190.4 m and 191.5 m at the east and west abutment, respectively. Further, the proposed bridge structure will require retained soil system (RSS) walls up to about 4.2 m and 6.3 m high, at the east and west abutments, respectively, generally oriented parallel to the future S-E/W Ramp Extension.

If the project is modified in concept, location or elevation, or if the project is not initiated within eighteen months of the date of the report, Golder should be given an opportunity to confirm that the recommendations are still valid.

6.2 Foundation Options

Based on the subsurface conditions at this site, both shallow and deep foundation options are feasible for the support of the proposed bridge structure. A summary of the advantages and disadvantages associated with each option is provided below, and a comparison of the alternative foundation options based on advantages, disadvantages, risks and relative costs are provided in Table 1 following the text of this report.

- **Shallow foundations – spread/strip footings:** Shallow foundations comprised of spread or strip footings, founded on the compact to very dense sandy silt to silt and sand to sand deposit or “perched” within the



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embankment fill, are feasible for support of the new abutments, although this foundation type will preclude the use of integral abutments. If the footings are founded on native soils, the proposed founding level will be below the groundwater level at the site, and groundwater control would be required to enable shallow foundations to be constructed in “dry” conditions.

- **Deep foundations – driven steel H-piles or pipe piles:** Driven steel H-piles or steel pipe (tube) piles are feasible for support of the abutments, and would permit design of conventional abutments, semi-integral abutments (for pipe piles) or integral abutments (for H-piles). At the locations of the abutments, the surface of the “100-blow” soil is at about Elevations 176 m and 180 m at the east and west abutment, respectively, and as such the minimum required pile length of 5 m for integral abutments will be achievable. A “perched” pile cap would also minimize excavation and groundwater control requirements at the abutments. Pile driving shoes are recommended to protect the pile tips from damage during driving into the very dense non-cohesive deposits.
- **Deep foundations – caisson:** Caissons are considered feasible for the support of the abutments; however this option would preclude integral abutment design. This option would be more expensive than either shallow foundations or driven pile foundations, although fewer caisson elements would be required in comparison to the number of driven steel piles that would be required. If caissons are adopted for support of the abutments, they would extend into water-bearing non-cohesive soil deposits and as such, temporary liners would be required during construction to control potential ground losses and/or disturbance of the caisson base.

Based on the above considerations, both shallow and deep foundation options are considered feasible for the support of the new abutments, although H-pile foundations are preferred from a foundations perspective as they would permit integral abutments design.

6.3 Shallow Foundations

6.3.1 Founding Elevation

For support of the new abutments on shallow foundations, spread/strip footings should be founded on the compact to very dense sandy silt to silt and sand to sand, or on compacted granular pads. Where spread/strip footings are to be founded on the native soil, the highest founding elevations recommended for design of footings are presented below.

Foundation Element	Highest Founding Elevation	Founding Soil
East Abutment	185.4 m	Compact to Very Dense Sandy Silt to Silt and Sand to Silty Sand to Sand
West Abutment	185.4 m	Compact to Dense Sandy Silt to Silty Sand to Sand

6.3.2 Geotechnical Axial Resistance

The following factored geotechnical axial resistances at Ultimate Limit States (ULS) and geotechnical reaction at Serviceability Limit States (SLS, for 25 mm of settlement) may be used for design of spread/strip footing founded



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on the properly prepared sandy silt to sand deposit, or on a compacted Granular 'A' pad having a minimum thickness of 1 m.

Foundation Alternative	Factored Geotechnical Axial Resistance at ULS ¹	Geotechnical Resistance at SLS for 25 mm of Settlement ¹
Footing on properly prepared compact to very dense sandy silt to sand	375 kPa	125 kPa
Footing on minimum 1 m thick compacted Granular 'A' pad	750 kPa	350 kPa

Note: 1. The geotechnical resistance values given above are estimated for a 3 m wide spread/strip footing.

The geotechnical resistances provided herein are given for loads will that be applied perpendicular to the surface of the footings. Where the load is not applied perpendicular to the footing, inclination of the load should be taken into account in accordance with Section 6.7.2 of the *CHBDC (2006)*.

6.3.3 Resistance to Lateral Loads

Resistance to lateral forces/sliding resistance between cast-in-place concrete footings and the subsoils should be calculated in accordance with Section 6.7.5 of the *CHBDC (2006)*. The following presents the coefficient of friction ($\tan \delta$) for the interface between the concrete footing and sand deposit or Granular 'A' pad.

Founding Material	Coefficient of Friction ($\tan \delta$)
Cast-in-place concrete footing on native compact to very dense sandy silt to sand	0.55
Cast-in-place concrete footing on compacted Granular 'A' pad	0.60

The values presented above are unfactored values.

6.3.4 Frost Protection

The footings should be provided with a minimum 1.2 m of soil cover for frost protection as per OPSD 3090.101 (Frost Penetration Depths for Southern Ontario), as measured vertically and perpendicular from the face of the abutment slope to the edge of the underside of the footing.

If adequate soil cover cannot be provided for the footing, rigid styrofoam insulation could be installed to compensate for the lack of soil cover and provide protection from frost penetration.

6.4 Driven Steel H-Pile or Steel Pipe (Tube) Pile Foundations

6.4.1 Founding Elevation

The east and west abutments for the proposed bridge structure may be supported on steel H-piles, or pipe piles, driven to found within the hard ("100-blow") clayey silt deposit and the very dense ("100-blow") sand deposit.



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Based on the GA Drawing, integral abutments have been adopted for the design of the replacement structure, and the abutments will be “perched” within the approach embankments, with the underside of the new pile caps at about Elevation 190.4 m and 191.5 m at the east and west abutment, respectively. The following pile tip elevations are recommended for design, based on an approximately 2 m to 3 m penetration into the “100-blow” deposits.

Foundation Element	Approximate Surface Elevation of “100-Blow” Soil	Estimated Design Tip Elevation	Founding Soil at Tip Elevation
East Abutment	176.0 m	174.0 m	Hard Clayey Silt
West Abutment	179.5 m	176.5 m	Very Dense Sand

Based on the above elevations, the proposed piles are estimated to be approximately 17.5 m and 15 m long at the east and west abutment, respectively.

6.4.2 Geotechnical Axial Resistance

The factored geotechnical axial resistance at ULS and the geotechnical resistance at SLS (for 25 mm of settlement) for driven steel H-piles and closed-end, concrete-filled 324 mm (12- $\frac{3}{4}$ in.) diameter steel pipe piles having a minimum wall thickness of 9.5 mm (3/8 in.) are presented below.

Pile Type	Approximate Pile Length	Factored Geotechnical Axial Resistance at ULS	Geotechnical Resistance at SLS for 25 mm of Settlement
HP 310x110 (Integral Abutment)	15 m to 17.5 m	1,600 kN	N/A ¹
HP 310x110 (Conventional or Semi-Integral Abutment)	13.5 m to 16.5 m	1,600 kN	N/A ¹
324 mm OD Pipe Pile (Conventional or Semi-Integral Abutment)	13.5 m to 16.5 m	1,400 kN	N/A ¹

Note: 1. The geotechnical resistance at SLS for 25 mm of settlement will be greater than the factored axial resistance at ULS and as such, the SLS condition does not apply.

As discussed in Section 4.0, cobbles are inferred to be present in the sandy silt to sand deposit. In this regard, steel H-piles are preferred over steel pipe piles as pipe piles are considered to pose a higher risk of experiencing refusal on cobbles and/or boulders or being deflected away from the vertical/battered orientation during installation due to their larger end area. Piles should be reinforced at the tip with driving shoes and/or flange plates in accordance with OPSD 3000.100 (Steel H-Pile Driving Shoe) Type I or OPSD 3001.100 (Steel Tube Pile Driving Shoe) Type II, as applicable, to reduce the potential for damage to the piles during driving.



6.4.3 Frost Protection

The pile caps should be provided with a minimum 1.2 m of soil cover for frost protection as per OPSD 3090.101 (Frost Penetration Depths for Southern Ontario), as measured vertically and perpendicular from the face of the abutment slope to the edge of the underside of the pile cap.

If adequate soil cover cannot be provided for the footing, rigid styrofoam insulation could be installed to compensate for the lack of soil cover and provide protection from frost penetration.

6.4.4 Set Criteria

Pile installation should be in accordance with OPSS 903 (Deep Foundations). The pile termination or set criteria will be dependent on the pile driving hammer type, helmet, selected pile and length of pile; the criteria must therefore be established at the time of construction after the piling equipment is known. The pile capacity should then be verified in the field by the use of Pile Driving Analyzer (PDA) or Hiley method (MTO's Standard Drawing SS103-11, Pile Driving Control) during the final stages of driving to verify that the required ultimate capacity has been achieved. Relaxation of soil surrounding the pile tips and/or heaving of the pile tips as a result of driving of adjacent piles could lead to reduced pile capacities. In this regard, it is recommended that a minimum of 10 per cent of piles be re-tapped at each foundation element to confirm that relaxation/heave is not occurring. If a significant reduction in the pile driving resistance is noted during re-tapping, all of the piles may need to be re-tapped and/or re-driven. The choice of set criteria is dependent on the experience of the engineer and traditional use where a substantial database has been developed over the years. The criteria need to be set to also avoid overdriving and possibly damaging the pile.

Depending on the pile type selected, the applicable pile driving notes should be added to the Contract Drawings (i.e. relevant notes in Clause 3.3.3 of the Structural Manual (MTO, 2014)):

For steel H-piles for integral and semi-integral abutment design:

- At the east abutment, piles to be driven in accordance with Standard Drawing SS 103-11 using an ultimate geotechnical resistance of 3,200 kN per pile, but must be driven below El. 173.0 m.
- At the west abutment, piles to be driven in accordance with Standard Drawing SS 103-11 using an ultimate geotechnical resistance of 3,200 kN per pile, but must be driven below El. 176.5 m.

For steel pipe piles for conventional or semi-integral abutment design:

- At the east abutment, piles to be driven in accordance with Standard Drawing SS 103-11 using an ultimate geotechnical resistance of 2,800 kN per pile, but must be driven below El. 173.0 m.
- At the west abutment, piles to be driven in accordance with Standard Drawing SS 103-11 using an ultimate geotechnical resistance of 2,800 kN per pile, but must be driven below El. 176.5 m.



6.5 Caisson Foundations

6.5.1 Founding Elevations

The abutment for the proposed bridge structure may also be supported on caissons founded within the hard ("100-blow") clayey silt and the very dense ("100-blow") sand deposits. The following caisson founding elevations may be used for preliminary design purposes, assuming about 2 m penetration into "100-blow" soil:

Foundation Element	Approximate Surface Elevation of "100-Blow" Soil	Estimated Design Tip Elevation	Founding Soil at Tip Elevation
East Abutment	176.0 m	174.0 m	Hard Clayey Silt
West Abutment	179.5 m	177.5 m	Very Dense Sand

If caisson foundations are adopted, a temporary liner and/or drilling slurry may be required to support the overburden soils during construction and balance groundwater pressures to minimize disturbance to the side walls and to control base disturbance/basal heave. In addition, placement of concrete by tremie methods would be required.

6.5.2 Geotechnical Axial Resistance

The following factored geotechnical axial resistance at ULS and the geotechnical resistance at SLS (for 25 mm of settlement) may be used for design of caisson foundations:

Caisson Diameter	Factored Geotechnical Axial Resistance at ULS	Geotechnical Resistance at SLS for 25 mm of Settlement
900 mm OD	3,000 kN	1,750 kN

6.5.3 Frost Protection

The pile caps should be provided with a minimum 1.2 m of soil cover for frost protection as per OPSD 3090.101 (Frost Penetration Depths for Southern Ontario), as measured vertically and perpendicular from the face of the abutment slope to the edge of the underside of the pile cap.

If adequate soil cover cannot be provided for the footing, rigid styrofoam insulation could be installed to compensate for the lack of soil cover and provide protection from frost penetration.

6.6 Resistance to Lateral Loads

The design of piles foundations subjected to lateral loads should take into account such factors as the batter of the pile, 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 mobilized, 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 mobilization of the lateral geotechnical resistance.



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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.

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 the CHBDC Commentary, 2006):

For non-cohesive soils:

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

where: n_h = coefficient related to soil density (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)

The values of n_h (Terzaghi, 1955 and Reese, 1975) and s_u to be incorporated into the calculations of the coefficient of horizontal subgrade reaction (k_h) within the fill materials and the native subsoils to be utilized for the structural analysis of the piles and casings at this site are summarized below.

Foundation Element (Relevant Boreholes)	Soil Unit	Elevation (m)	n_h (kPa/m)	s_u (kPa)
West Abutment (GA-BR-02)	New Embankment Fill (assumed to be Compacted Granular Fill)	191.5 – 188.0	10,000 ¹	-
	Compact Silt	188.0 – 187.5	7,000	-
	Dense Sand (above groundwater table)	187.5 – 186.5	15,000	
	Dense Sand (below groundwater table)	186.5 – 186.0	8,000	-
	Compact Sand (below groundwater table)	186.0 – 179.6	6,000	-
	Very Dense Sand (below groundwater table)	179.6 – 171.7	12,000	-



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Foundation Element (Relevant Boreholes)	Soil Unit	Elevation (m)	n_h (kPa/m)	s_u (kPa)
East Abutment (GA-BR-01)	New Embankment Fill (assumed to be Compacted Granular Fill)	190.4 – 188.7	10,000 ¹	-
	Compact Sand (above groundwater table)	188.7 – 186.5	10,000	-
	Compact Sand (below groundwater table)	186.5 – 185.8	6,000	-
	Very Dense Silt and Sand (below groundwater table)	185.8 – 184.2	10,000	-
	Dense to Very Dense Sand (below groundwater table)	184.2 – 176.6	12,000	-
	Hard Clayey Silt (below groundwater table)	176.6 – 172.6	-	200

Note: 1. The new granular fill must extend a minimum distance of five pile diameters away from the outer edges of the piles in all directions.

Where integral abutment design includes the installation of Corrugated Steel Pipe (CSP) liners (with the annular space between the pile and liner filled with uniform-grained, uncompacted sand), the upper portion of the H-piles installed inside the CSP will be free to flex and move laterally. With this design, the passive lateral resistance over the length of the CSP liner should be neglected.

For a single HP 310x110 vertical pile, 324 mm OD pipe pile or a single 900 mm OD caisson, the estimated factored lateral resistances at ULS as well as the estimated lateral resistances at SLS (for 10 mm of horizontal deflection at the pile caps) are presented below. These values are based on analysis carried out using the commercially available program LPILE Plus (version 5.0), developed by Ensoft Inc.

Foundation Location	Pile Type	Axial Load Applied at the Top of Pile/Casing	Factored Geotechnical Lateral Resistance at ULS ¹	Geotechnical Lateral Resistance at SLS for 10 mm of Deflection ¹
East and West Abutment	HP 310 x 110 (Integral Abutment)	1,600 kN	65 kN	20 kN
	HP 310 x 110 (Semi-Integral Abutment)	1,600 kN	110 kN	40 kN
	324 mm OD Pipe Pile	1,400 kN	70 kN	55 kN
	900 mm OD caisson	3,000 kN	650 kN	340 kN

Notes: 1. Analyses assumes a pinned-head condition.

Based on the above, it is considered that both the structural and geotechnical resistances of the piles/caissons should be evaluated to establish the governing case at ULS. At SLS, the horizontal resistance of the piles will be controlled by deflections and the horizontal resistance of the pile should be calculated based on the coefficient of horizontal subgrade reaction (k_h) of the soil as discussed above. The SLS resistance should be taken as that corresponding to a horizontal deflection of 10 mm at the underside of the pile cap for units supporting the abutments (*CHBDC Commentary C6.8.7.1*).



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The upper zone of the soil (down to a depth below the pile cap equal to about $1.5 \cdot B$ (after Broms, 1964, where B is the pile diameter) should be neglected in the calculation of lateral resistance of the pile to account for disturbance effects during installation.

Group action for lateral loading should also be considered when the pile spacing in the direction of loading is less than eight pile diameters between rows of driven steel H-pile, steel pipe pile or caisson. Group action can be evaluated by reducing the coefficient of lateral subgrade reaction in the direction of loading by a reduction factor, R (U.S. Navy, 1986), as follows:

Pile Spacing in Direction of Loading (d = pile diameter)	Subgrade Reaction Reduction Factor, R
8d	1.00
6d	0.70
4d	0.40
3d	0.25

The subgrade reaction reduction factor should be interpolated for H-pile/casing spacing in between those listed above.

6.7 Seismic Considerations

6.7.1 Site Coefficient

For seismic design purposes, the Site Coefficient, S , for this site, based on experience and considering the guidelines in Section 4.4.6 of the *CHBDC* may be taken as 1.2, consistent with Soil Profile Type II.

6.7.2 Seismic Analysis Coefficient

According to the National Building Code of Canada (2010) seismic hazard values (as referenced in the *CHBDC* and its *Commentary*), the site specific peak horizontal ground acceleration for the Markham area is 0.028g (for a probability of exceedance of 10 per cent in 50 years). For the thicknesses and type of overburden soils at the site, an amplification factor of 1.2 of the ground motion is recommended for design. As such, the ground surface acceleration is about 0.034g and this site is classified as Seismic Performance Zone 1.

Given that the proposed bridge structure is a single-span bridge and in accordance with Sections 4.4.5.2 of the *CHBDC*, seismic analysis is not required for this structure.

6.8 Lateral Earth Pressures

The lateral earth pressures acting on the abutment stem walls and any associated wingwalls/retaining walls will depend on the type and method of placement of the backfill material, 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.



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

- Select, free draining granular fill meeting the specifications of OPSS.PROV 1010 (Aggregates) Granular 'A' or Granular 'B' Type II, but with less than 5 per cent passing the No. 200 sieve, should be used as backfill behind the walls. Longitudinal drains and weep holes should be installed to provide positive drainage of the granular backfill. Compaction (including type of equipment, target densities, etc.) should be carried out in accordance with OPSS.PROV 501 (Compacting). Other aspects of the granular backfill requirements with respect to sub-drains and frost taper should be in accordance with OPSS 3121.150 (Retaining Walls, Backfill, Minimum Granular Requirement).
- 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. Other surcharge loadings should be accounted for in the design as required.
- For restrained walls, granular fill should be placed in a zone with the width equal to at least 1.2 m behind the back of the wall (in accordance with Figure C6.20(a) of the *Commentary* to the *CHBDC*). For unrestrained walls, 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 (in accordance with Figure C6.20(b) of the *Commentary* to the *CHBDC*). The pressures are based on the proposed embankment fill material and the following parameters (unfactored) may be used:

Fill Type	Soil Unit Weight	Coefficients of Static Lateral Earth Pressure	
		At-Rest, K_o	Active, K_a
Granular 'A'	22 kN/m ³	0.43	0.27
Granular 'B' Type II	21 kN/m ³	0.43	0.27

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

6.9 Approach Embankments

Based on the profile drawing provided by AECOM on October 6, 2015, the proposed grade for the E-N Ramp will require placement of up to about 4.5 m and 5.5 m for fill at the east and west approach embankments, respectively. In addition, given that the future S-E/W Ramp will be in an earth cut approximately 3 m deep at the proposed bridge structure, the total height of the east and west approach embankment immediately at the abutments will be 7.5 m and 8.5 m, respectively.

In general, the approach embankments will be founded on deposits of compact to very dense sandy silt to sand and hard clayey silt fill in places.



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The results of stability and settlement analysis for the approach embankments are presented in the following sections.

6.9.1 Stability

6.9.1.1 Methodology

Limit equilibrium slope stability analyses for the approach embankments were carried out using the commercially available program Slide (version 6.0), developed by Rocscience Inc., employing the Morgenstern-Price method of analysis. For all analyses, the Factors of Safety (FoS) of numerous potential failure surfaces were computed for the highest embankment cross-section in order to establish the minimum FoS. The FoS is defined as the ratio of the forces tending to resist failure to the driving forces tending to cause failure. A target minimum FoS of 1.3 is considered appropriate for the design of embankment slopes under static conditions. For stability analysis involving retaining walls, a target minimum FoS of 1.5 is considered appropriate for design under static conditions. The stability analyses assume that all organics and other deleterious materials (i.e. rootlets and wood fragments) are removed prior to constructing the approach embankments.

6.9.1.2 Parameter Selection

For the non-cohesive soils present at this site, the effective stress parameters employed in the analysis were estimated from empirical correlations based on the results of the in situ Standard Penetration Tests (SPT). The correlations proposed by Peck et al (1974) and U.S. Navy (1986) were employed and the results were adjusted by engineering judgment based on precedent experience in similar soils.

For the purpose of the stability analysis, the groundwater level was assumed to be at Elevation 186.5 m, which is based on the average piezometric groundwater level measured on site.

The following presents the simplified stratigraphy and the associated strengths and unit weights employed for the existing and new fill material as well as the native overburden deposits encountered at the approach embankment areas:

Embankment	Soil Type	Unit Weight, γ	Cohesion, c'	Effective Friction Angle, ϕ'
East and West Approach Embankment / E-N Ramp Embankment	New Granular Embankment Fill	21 kN/m ³	-	34°
	Existing Very Loose to Compact Sandy Silt Fill	19 kN/m ³	-	28°
	Stiff to Hard Clayey Silt to Silty Clay	18 kN/m ³	100 kPa	-
	Compact to Dense Sandy Silt to Sand	19 kN/m ³	-	30°
	Very Dense Silt and Sand	19 kN/m ³	-	32°
	Very Dense Sand	19 kN/m ³	-	34°
	Very Dense Gravelly Silt and Sand to Gravelly Sand	21 kN/m ³	-	36°
	Hard Clayey Silt	20 kN/m ³	200 kPa	-



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6.9.1.3 Results of Analysis

The results of the stability analyses for the approach embankments are summarized below. The minimum FoS is based on a deep-seated, global trial failure surface that would impact the operation of the highway.

Embankment	Analysis	Slope Profile	Embankment Height at Critical Section	Minimum Factor of Safety
East Approach / E-N Ramp Embankment (STA 10+100 to 10+205)	Side Slope	2H:1V	4.5 m	≥ 1.3
	Front Slope	2H:1V with an up to 4.2 m high RSS Wall	7.5 m	≥ 1.5
West Approach / E-N Ramp Embankment (STA 10+240 to 10+350)	Side Slope	2H:1V	5.5 m	≥ 1.3
	Front Slope	2H:1V with an up to 6.3 m high RSS Wall	8.5 m	≥ 1.5

Notes: 1. The width of the reinforced soil mass is assumed to be 0.6 times the wall height.

6.9.2 Settlement

6.9.2.1 Methodology

To estimate the magnitude of expected settlement of the embankments, analyses were carried out at the critical section of the east and west approach embankments, corresponding to the highest grade raise. Settlement analyses were carried out using the commercially available program *Settle*^{3D} (version 3.0), developed by Rocscience Inc. The settlement analyses assume that all organics and other deleterious materials (i.e. rootlets and wood fragments) are removed prior to constructing the approach embankments.

6.9.2.2 Parameter Selection

The simplified stratigraphy and the associated unit weights and strengths employed for the estimation of settlement of the foundation soils at the approach embankment areas are presented below. The immediate compression of the non-cohesive overburden soils were modelled by estimating an elastic modulus of deformation based on the SPT 'N'-values and using correlations proposed by Bowles (1984) and Kulhawy and Mayne (1990). These estimated values were compared with the typical range of expected values for similar soil types, as outlined in *CHBDC* and adjusted, as appropriate. The consolidation settlement parameters of the cohesive deposits was assessed using empirical correlations proposed in literature by Koppula (1986), Terzaghi and Peck (1967), Kulhawy and Mayne (1990) and Azzouz et al. (1976). The coefficient of consolidation, c_v , required in the time-rate analysis was estimated from the U.S. Navy (1986) correlation with liquid limits assuming over-consolidated soils.



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Embankment	Soil Type	Thickness at Critical Section	Unit Weight, γ	Deformation Parameter(s)
East Approach / E-N Ramp Embankment (STA 10+100 to 10+205)	Existing Loose Sandy Silt Fill	~ 0.8 m	19 kN/m ³	$E' = 5 \text{ MPa}$
	Very Stiff Clayey Silt to Silty Clay	~ 1.5 m	18 kN/m ³	$e_o = 0.7$ $C_c = 0.28$ $C_r = 0.06$ $c_v = 1.34 \times 10^{-2} \text{ cm}^2/\text{s}$
	Compact Sandy Silt to Sand	~ 2.8 m	19 kN/m ³	$E' = 15 \text{ MPa}$
	Very Dense Sandy Silt to Sand	~ 4.0 m	19 kN/m ³	$E' = 50 \text{ MPa}$
	Very Dense Gravelly Silty Sand to Gravelly Sand	~ 17.2 m	21 kN/m ³	$E' = 75 \text{ MPa}$
West Approach / E-N Ramp Embankment (STA 10+240 to 10+350)	Existing Very Loose to Loose Sandy Silt Fill	~ 0.9 m	19 kN/m ³	$E' = 5 \text{ MPa}$
	Very Stiff to Hard Clayey Silt to Silty Clay	~ 2.4 m	18 kN/m ³	$e_o = 0.7$ $C_c = 0.28$ $C_r = 0.06$ $c_v = 1.34 \times 10^{-2} \text{ cm}^2/\text{s}$
	Compact Sandy Silt to Sand	~ 6.6 m	19 kN/m ³	$E' = 20 \text{ MPa}$
	Sandy Silt to Sand Till	~ 2.9 m	21 kN/m ³	$E' = 50 \text{ MPa}$

For the purpose of the settlement analysis, the groundwater level was assumed to be at Elevation 186.5 m, which is based on the average piezometric groundwater level measured on site.

6.9.2.3 Settlement of Foundation Soils

The results of the analyses of the estimated settlement of the foundation soils at the approach embankments are presented below.

Embankment	Estimated Immediate Settlement of Foundation Soils	Estimated Consolidation Settlement of Foundation Soils	Estimated Total Settlement of Foundation Soils
East Approach / E-N Ramp Embankment (STA 10+100 to 10+205)	55 mm	35 mm	90 mm
West Approach / E-N Ramp Embankment (STA 10+240 to 10+350)	80 mm	60 mm	140 mm

The immediate settlements are expected to occur relatively quickly (i.e. during construction) in response to the embankment construction. Based on the estimated coefficient of consolidation (c_v about $1.34 \times 10^{-2} \text{ cm}^2/\text{s}$) for the cohesive deposit, it is estimated that 90 per cent consolidation settlement will be completed in about



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15 days. As such, it is recommended that a preload period of 30 days be included in the construction schedule to reduce the post-construction settlement to less than 25 mm. Taking into consideration that the native cohesive deposit is highly over-consolidated and the recommended duration of the preload period of 30 days (as compared to the estimated time for 90 per cent consolidation of 15 days), instrumentation and settlement monitoring is not considered necessary.

6.10 Retained Soil System (RSS) Walls

6.10.1 Founding Elevations

Based on the GA drawing, the E-N Ramp Bridge will require retaining walls up to about 4.2 m and 6.3 m high at the east and west abutments, respectively, which are generally oriented parallel to the future S-E/W Ramp Extension. Retained soil system (RSS) walls are considered to be a suitable option for the required retaining walls at this site.

Based on the retaining wall drawing provided by AECOM on October 6, 2015, the RSS walls are located along the cut section of the future S-E/W Ramp Extension. The highest founding elevation for the front facing panels and the reinforced soil mass of the RSS walls recommended for design are presented below.

Retaining Wall	Elevation of the Finished Ground Surface in Front of Retaining Wall	Highest Wall Founding Elevation ¹	Founding Soil
East Abutment RSS Walls STA 10+033 to 10+067 ²	187.0 m to 188.5 m	185.8 m to 187.3 m	Compact to Dense Sandy Silt to Sand
West Abutment RSS Walls STA 10+058 to 10+114 ²	187.3 m to 190.0 m	186.1 m to 188.8 m	Compact to Dense Sandy Silt to Sand / Very Stiff to Hard Clayey Silt

Note: 1. The highest founding elevation is 1.2 m below the base of retaining wall due to depth of frost penetration at this site.

2. Stations transposed along the future S-E/W Ramp Extension.

The front facing panels should be supported on a footing constructed of a granular pad. The granular pad should consist of a minimum thickness of 0.3 m of compacted Granular 'A' material, which should extend at least 0.5 m beyond the outside edge of both sides of the facing footing, then outward/downward at a slope of 1H:1V.

6.10.2 Geotechnical Resistance

For the RSS facing panels supported on a 0.3 m wide footing constructed on a compacted granular pad as described in Section 6.10.1, the following factored geotechnical axial resistances at Ultimate Limit States (ULS) and geotechnical resistance at Serviceability Limit States (SLS, for 25 mm of settlement) may be used for design:



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Foundation	Factored Geotechnical Axial Resistance at ULS	Geotechnical Axial Resistance at SLS for 25 mm of Settlement
Footing on a 0.5 m thick compacted Granular 'A' pad	250 kPa	N/A ¹

Note: 1. The geotechnical resistance at SLS for 25 mm of settlement will be greater than the factored axial resistance at ULS and as such, the SLS condition does not apply.

Assuming that the RSS walls acts as a unit and uses the full width of the reinforced soil mass (which is assumed to be 0.8 times the height of the wall) and is founded on the native compact to dense sandy silt to sand, the following factored geotechnical axial resistances at Ultimate Limit States (ULS) and geotechnical resistance at Serviceability Limit States (SLS, for 25 mm of settlement) may be used for design:

Retaining Wall	Factored Geotechnical Axial Resistance at ULS	Geotechnical Resistance at SLS for 25 mm of Settlement
East RSS Wall	400 kPa	200 kPa
West RSS Wall	500 kPa	100 kPa

6.10.3 Frost Protection

The footing for the RSS facing panels should be provided with a minimum 1.2 m of soil cover for frost protection as per OPSD 3090.101 (Frost Penetration Depths for Southern Ontario), as measured vertically and perpendicular from the face of the abutment slope to the edge of the underside of the footing.

If adequate soil cover cannot be provided for the footing, rigid styrofoam insulation could be installed to compensate for the lack of soil cover and provide protection from frost penetration.

6.10.4 Resistance to Lateral Loads/Sliding Resistance

Resistance to lateral forces/sliding resistance between the compacted fill of the RSS wall mass and the subgrade should be calculated in accordance with Section 6.7.5 of the CHBDC (2006). The coefficient of friction ($\tan \delta$), between the compacted granular fill of the RSS wall and the properly prepared native subgrade may be taken as 0.45.

6.10.5 Global Stability

The static global stability analyses for the RSS walls were completed using the parameters outlined in Section 6.9.1.2, and assume that all organics and other deleterious materials (i.e. soil with rootlets and wood fragments) are removed prior to constructing the RSS walls. As discussed in Section 6.9.1.1, a target minimum FoS of 1.5 is considered appropriate for design of the RSS walls for global stability. The results of the stability analyses indicate that a FoS for the east and west RSS walls equal to or greater than 1.5 is achieved (refer to Section 6.9.1.3). It should be noted, however, that the internal stability of a reinforced earth structure is to be designed and assessed by the proprietary product designer/manufacturer to ensure that the internal stability of the walls are adequate.



6.10.6 Settlement

The estimated settlement along the RSS walls is presented below.

Retaining Wall	Estimated Settlement of Foundation Soils ¹
East and West Abutment RSS Walls	Less than 25 mm

Note: 1. Assumes that that retaining wall footing and front facing is constructed after the preload period of 30 days (refer to Section 6.9.2.3).

6.10.7 Performance and Appearance

Given that the RSS walls at the bridge abutments are along the S-E/W Ramp Extension which is associated with a 400-series highway, a high site performance rating and a high appearance rating is required in accordance with the MTO RSS Design Guidelines (2008).

6.11 Concrete Toe Wall No. 5

Based on the drawings provided by AECOM on October 6, 2015, an up to 1.8 m high concrete toe wall (Concrete Toe Wall No. 5) will be required between STA 10+000 and 10+107 along the north side of the E-N Ramp. In addition, the base of the toe wall is to be founded at a depth of 0.45 m below the proposed finished grade and as such, the toe wall will be founded on the native very stiff to hard clayey silt deposit, which is underlain by a layer of loose to compact silt and sand. Given the founding conditions encountered in the boreholes drilled along the toe wall (i.e. Boreholes GA-WM-02 and GA-WM-03), a factored geotechnical axial resistance at ULS of 300 kPa and a coefficient of friction ($\tan \delta$) of 0.30 may be used for design for horizontal sliding resistance along the base of the toe wall.

The concrete toe wall should be constructed in accordance with OPSD 3120.100 (Concrete Toe Wall) Type III.

6.12 Construction Considerations

6.12.1 Overburden Excavation

The existing fill and native overburden soils are considered Type 3 soils according to the Occupational Health and Safety Act and Regulation for Construction Projects (OHSA). Temporary open-cut excavations above the groundwater level, or above the lowered groundwater level with proper groundwater control in place, should be carried out with side slopes no steeper than 1 horizontal to 1 vertical (1H:1V).

All excavations must be carried out in accordance with Ontario Regulation 213 Ontario Occupational Health and Safety Act for Construction Projects (as amended).

The native sandy silt to sand deposits are highly erodible and as such consideration should be given to either placing of erosion protection on the exposed slope, or cutting into the slope in the areas where the sandy silt to sand deposit is exposed and backfilling to the final slope configuration using Granular 'B' Type II.



6.12.2 Subgrade Preparation

The existing native very stiff to hard clayey silt to silty clay and loose to very dense sandy silt to sand deposits are considered to be appropriate subgrade for the support of the RSS walls and embankments; however, prior to any construction or the placement of any fill, any organic materials and softened soil should be stripped from the plan limits of the proposed works and the subgrade soils should be proof-rolled, especially in areas of loose/loosened soil.

6.12.3 Placement of Embankment Fill

Upon completion of stripping of all organics and other deleterious materials from the plan limits of the proposed works, placement of granular fill for the construction of the approach embankments should be carried out in accordance with OPSS.PROV 501 (Compacting). Materials use for the construction of the embankment should meet the specification of OPSS.PROV 1010 Granular 'B' Type I. The granular fill should be compacted in lifts not exceeding 300 mm thick to a minimum 95 per cent of Standard Proctor Maximum Dry Density (SPMDD).

Where granular fill is used to widen or tie-in to existing earth slopes, benching of the existing earth slopes should be carried out in accordance with OPSD 208.010 (Benching of Earth Slopes).

6.12.4 Erosion Protection

To reduce erosion of the embankment side slopes due to surface water runoff, placement of topsoil and seeding or pegged sod should be carried out on the embankment slopes as soon as practicable after construction of the embankments. In the short term, if placement of cover material cannot be carried out soon after the construction of the embankments, erosion control blankets should be installed to minimize erosion of the embankment slopes. The erosion protection should be in accordance with OPSS.PROV 804 (Seed and Cover).

6.12.5 Temporary Protection Systems

Based on the construction staging drawings provided by AECOM on October 6, 2015, temporary protection systems will be required along the existing E-N Ramp to facilitate the construction of the proposed E-N Ramp. Where required, temporary protection systems should be designed and constructed in accordance with OPSS.PROV 539 (Temporary Protection System), and the lateral movement should meet Performance Level 2.

The selection and design of the protection system is the responsibility of the Contractor.

The design of a braced soldier pile and lagging wall should be based on a rectangular earth pressure distribution as presented in NAVFAC DM 7.02 (1986) using the design parameters given below. Where the support to the wall is provided by anchors or rakers, the wall design should be based on a triangular earth pressure distribution using the design parameters given below. The raker/anchor support must be designed to accommodate the loads applied from pressures and surcharge pressures from area, line or point loads as well as the impact of sloping ground behind the system. Passive toe restraint to the soldier piles may be determined using a triangular pressure distribution acting over an equivalent width equal to three times the diameter of the pile socket.



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Soil Type	Coefficient of Earth Pressure			Angle of Friction (ϕ)	Unit Weight (γ)
	Active, K_a	At Rest, K_o	Passive, K_p		
New Granular Embankment Fill	0.3	0.4	3.5	34°	21 kN/m ³
Existing Loose to Compact Sandy Silt Fill / Stiff Sandy Clayey Silt Fill	0.4	0.5	2.8	28°	19 kN/m ³
Very Stiff to Hard Clayey Silt to Silty Clay	0.3	0.5	3.3	32°	18 kN/m ³
Compact to Dense Sandy Silt to Sand	0.3	0.5	3.0	30°	19 kN/m ³
Compact to Dense Sandy Silt to Sand Till	0.3	0.5	3.0	30°	19 kN/m ³
Very Dense Silt and Sand to Silty Sand	0.3	0.5	3.3	32°	19 kN/m ³
Very Dense Sand	0.3	0.4	3.5	34°	19 kN/m ³
Hard Clayey Silt	0.3	0.4	3.5	34°	20 kN/m ³

The earth pressure coefficients noted above are based on a horizontal surface adjacent to the excavation. If sloped surfaces are present, the coefficients should be adjusted accordingly.

6.12.6 Control of Groundwater and Surface Water

The soils at the base of the excavation along the abutments and RSS walls consist of water-bearing, relatively permeable sandy silt, silt and sand, silty sand, and sand. The groundwater level measured in the standpipe piezometers installed in the previous and current investigations range from Elevations 186.3 m to 183.8 m and as such, control of groundwater may be required during wet periods of the year. Where required, it is considered that pumping from within trenches/ditching with adequately size and properly filtered sumps will be sufficient to control the groundwater inflow.

Surface water should be directed away from the excavation at all times.

6.12.7 Obstructions

It should be noted that obstructions (inferred as cobbles, from auger grinding and split-spoon sampling) were encountered within the upper clayey silt to silty clay, sandy silt to sand and sandy silt to sand till deposits. Although boulders were not encountered in the boreholes along the alignment, the presence of boulders is inferred elsewhere on site. The presence of such obstructions could affect excavation works, installation of temporary protection systems as well as construction of deep foundation. The Contractor should be made aware of the potential presence of cobbles and/or boulders within the overburden soils as noted in the NSSP for



Obstructions included in Appendix D. The borehole drilling method does not permit accurate measurement of the cobbles or boulders; nor can an estimate to be made of the quantity (overall volume) of these materials.

6.12.8 Control of Ground and Groundwater for Caisson Construction

As discussed in Section 6.5, disturbance (i.e. running or flowing) of the water-bearing non-cohesive soil deposits could occur during or after drilling of caissons (if adopted), and basal heave could occur at the caisson base. If caisson foundations are adopted, temporary caisson liners with a balancing head of water and/or drilling slurry will be required to support the overburden soils and balance groundwater pressures during construction. In addition, placement of concrete by tremie methods would be required.

6.12.9 Protection of Subgrade

The non-cohesive soils that will be exposed within the excavations at the abutments and RSS walls will be susceptible to disturbance from construction traffic and/or precipitation and ponded water. To limit the effects of this disturbance, a concrete working slab should be placed on the prepared subgrade if concrete or compacted Granular 'A' is not placed within four hours after preparation, inspection and approval of the subgrade. The minimum thickness of the concrete working slab should be 100 mm and the concrete should have a minimum 28-day compressive strength of 20 MPa.

6.12.10 Analytical Testing for Construction Materials

The results of analytical tests carried out on two samples of groundwater taken from Boreholes GA-WM-02 and GA-WM-06 are presented in Table B1 in Appendix B. The suite of parameters tested is intended to allow the design engineer to assess the requirements for the appropriate type of cement to be used in construction and the need for corrosion protection of steel elements.



FOUNDATION REPORT - E-N RAMP RE-ALIGNMENT BRIDGE / APPROACH EMBANKMENTS AND ASSOCIATED STRUCTURES

7.0 CLOSURE

This Foundation Design Report was prepared by Ms. Madison C. Kennedy, B.A.Sc., a member of the geotechnical engineering group, and reviewed by Mr. Christopher Ng, P.Eng., a senior geotechnical engineer and Associate with Golder. Mr. Jorge M. A. Costa, P.Eng., a Principal with Golder and Designated MTO Foundations Contact, conducted an independent review of this report.

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FOUNDATION REPORT - E-N RAMP RE-ALIGNMENT BRIDGE / APPROACH EMBANKMENTS AND ASSOCIATED STRUCTURES

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FOUNDATION REPORT - E-N RAMP RE-ALIGNMENT BRIDGE / APPROACH EMBANKMENTS AND ASSOCIATED STRUCTURES

ASTM International:

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

Commercial Software:

Settle3D (Version 3.0) by Rocscience Inc.

Slide (Version 6.0) by Rocscience Inc.

LPILE Plus (Version 5.0) by Ensoft Inc.

Ministry of Transportation Ontario:

Drawing SS103-11 Pile Driving Control

Ontario Occupational Health and Safety Act:

Ontario Regulation 213 Construction Projects (as amended)

Ontario Provincial Standard Specifications (OPSS)

OPSS.PROV 501	Construction Specification for Compacting
OPSS.PROV 539	Construction Specification for Temporary Protection Systems
OPSS.PROV 804	Construction Specification for Seed and Cover
OPSS 903	Construction Specification for Deep Foundations
OPSS.PROV 1010	Material Specification for Aggregates – Base, Subbase, Select Subgrade, and Backfill Material

Ontario Provincial Standard Drawings (OPSD)

OPSD 208.010	Benching of Earth Slopes
OPSD 3000.100	Foundation, Piles, Steel H-Pile, Driving Shoe
OPSD 3001.100	Foundation, Piles, Steel Tube Piles, Driving Shoe
OPSD 3090.101	Foundation, Frost Penetration Depths for Southern Ontario
OPSD 3120.100	Walls, Retaining, Concrete Toe Wall
OPSD 3121.150	Walls, Retaining, Backfill, Minimum Granular Requirement

Ontario Water Resources Act:

Ontario Regulation 903 Wells (as amended)



TABLES



FOUNDATION REPORT - E-N RAMP RE-ALIGNMENT BRIDGE / APPROACH EMBANKMENTS AND ASSOCIATED STRUCTURES

Table 1 – Evaluation of Foundation Alternatives – N-E Ramp Bridge

Foundation Option	Feasibility	Advantages	Disadvantages	Estimated Costs	Risk / Consequences
Spread/strip footings	<ul style="list-style-type: none"> Feasible for support of new abutments founded on the compact to very dense sandy silt to sand deposit or “perched” on a granular pad within the approach embankments. 	<ul style="list-style-type: none"> Conventional excavation and construction techniques. Lower cost compared to deep foundations. Requires limited excavation for footings “perched” within the approach embankments. 	<ul style="list-style-type: none"> Excavation at the abutments will extend up to about 1 m below the groundwater level, and groundwater control will be required. Requires larger footing excavation and disposal of a larger volume of soil compared to the excavation for a pile cap. Does not allow for integral abutment construction. 	<ul style="list-style-type: none"> Estimated cost is approximately \$600/m³ for construction of shallow foundations. 	<ul style="list-style-type: none"> Softening / loosening of subgrade due to groundwater would require a concrete working slab to be placed immediately after excavation to design depth, inspection and approval of subgrade. Subgrade should be protected from freezing.
Steel H-piles or pipe piles	<ul style="list-style-type: none"> Feasible for support of abutments with the pile caps constructed either “perched” within the approach embankments or below grade. 	<ul style="list-style-type: none"> Conventional construction methods for H-pile or steel pipe pile foundations. Abutment pile caps could be maintained higher than spread footings, potentially reducing depth of excavation and protection system requirements. Steel H-piles allow for integral abutment configuration; and pipe piles allow for semi-integral abutment configuration. Higher geotechnical resistance than for shallow foundations. 	<ul style="list-style-type: none"> Piles may refuse above design tip elevation due to very dense overburden soils, especially pipe piles which have a larger displacement base. Pipe piles not readily accepted for integral abutment construction. 	<ul style="list-style-type: none"> Estimated cost is approximately \$250/m length for pile installation and \$600/m³ for pile cap construction. 	<ul style="list-style-type: none"> Minor potential for pile damage / deflection if cobbles and boulders are encountered during pile driving. Slightly greater risk in this regard for pipe piles as compared with H-piles if boulders are encountered during pile driving.



FOUNDATION REPORT - E-N RAMP RE-ALIGNMENT BRIDGE / APPROACH EMBANKMENTS AND ASSOCIATED STRUCTURES

Table 1 – Evaluation of Foundation Alternatives – N-E Ramp Bridge

Foundation Option	Feasibility	Advantages	Disadvantages	Estimated Costs	Risk / Consequences
Caissons	<ul style="list-style-type: none"> Feasible but not recommended for support of abutments. 	<ul style="list-style-type: none"> Higher capacity than for driven piles, so reduced number of deep foundation elements compared to piles. Caps may be constructed at the level of the underside of the bridge thus eliminating the need for excavation. 	<ul style="list-style-type: none"> Caissons would extend below the groundwater level at the site into water-bearing non-cohesive soils, with potential for loss of ground or base disturbance. Temporary liners would be required, plus special measures such as use of drilling mud and tremie placement of concrete; likely not possible to inspect caisson base. Precludes use of integral abutments. More expensive compared to shallow foundations. 	<ul style="list-style-type: none"> Estimated cost is approximately \$1,000/m length for caisson installation and \$600/m³ for pile cap construction; the cost may be higher to account for temporary/permanent liners. 	<ul style="list-style-type: none"> Risk of loosening and leaving in place disturbed founding soils at base of caissons.



DRAWINGS



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

CONT No.
WP No.

HIGHWAY 7 TO HIGHWAY 404 E-N RAMP S-E/W
RAMP AND WATERMAIN
E-N RAMP
BOREHOLE LOCATIONS INDEX PLAN

SHEET



KEY PLAN
NOT TO SCALE

LEGEND

Borehole - Current Investigation

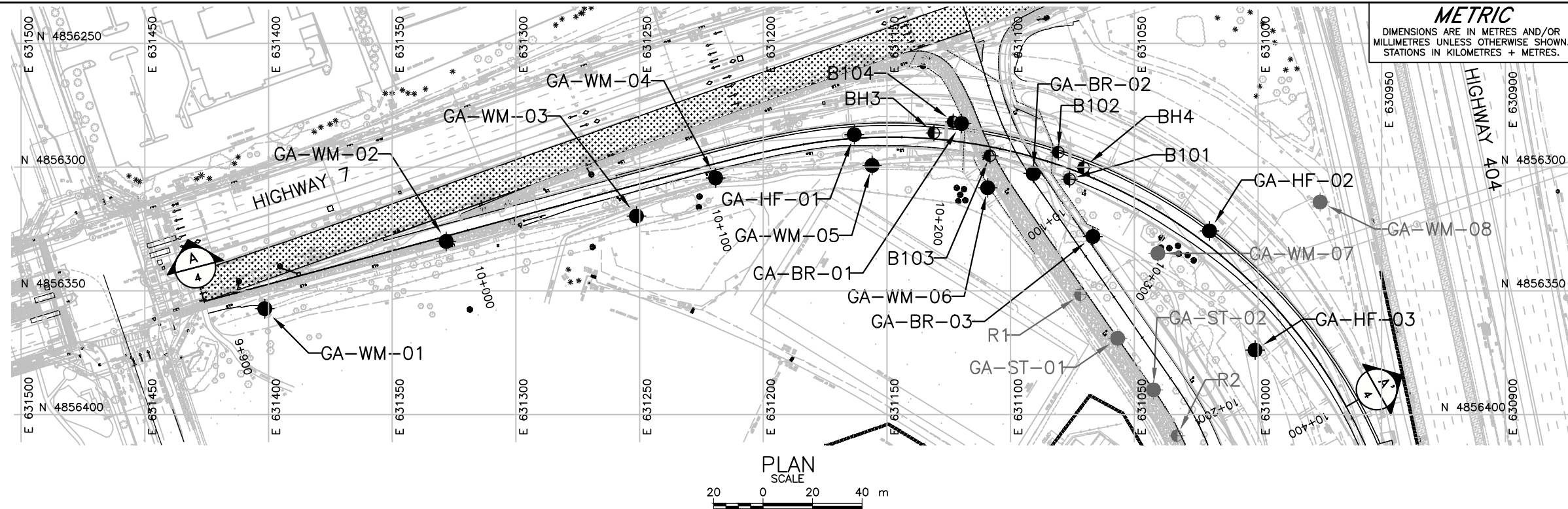
Borehole - Previous Investigation

REFERENCES

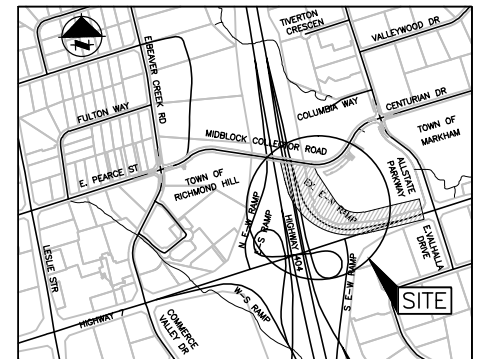
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Bgd-2015.dwg, Mdbk_Ramp_Pln.dwg, Ramp_Aln.dwg, Ramp_Pln.dwg and
received October 13, 2015.

NO.	DATE	BY	REVISION
Geocres No. 30M14-425			
HWY. 404		PROJECT NO. 1533525	
SUBM'D. MCK		CHKD. MCK	DATE: Oct. 2015
DRAWN: JFC		CHKD. CN	APPD. JMAC
		DIST. .	
		SITE: .	
		DWG. 1	



CONT No.
WP No.HIGHWAY 7 TO HIGHWAY 404
E-N RAMP REALIGNMENT
BOREHOLE LOCATIONS AND
SOIL STRATA

SHEET

KEY PLAN
NOT TO SCALE

LEGEND

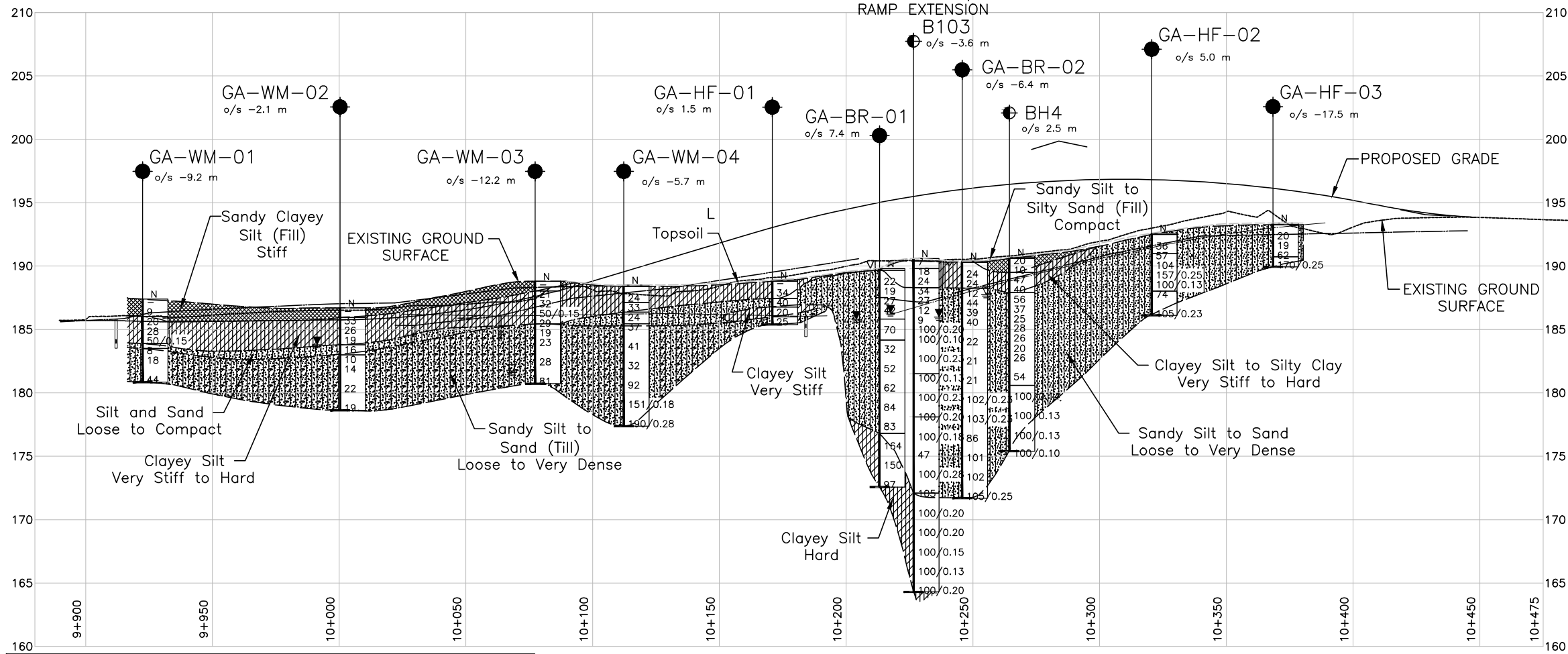
- Borehole - Current Investigation
- Borehole - Previous Investigation
- ⊥ Seal
- ⊥ Piezometer
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- ≡ WL in piezometer

BOREHOLE CO-ORDINATES

No.	ELEVATION	NORTHING	EASTING
GA-WM-06	190.5	4856308.3	631109.3
GA-WM-05	190.1	4856299.3	631156.0
GA-WM-04	188.5	4856304.4	631219.3
GA-WM-03	188.8	4856319.8	631251.3
GA-WM-02	186.7	4856330.0	631328.2
GA-WM-01	187.4	4856357.2	631401.5
GA-HF-03	193.4	4856373.9	631001.2
GA-HF-02	192.6	4856325.8	631019.6
GA-HF-01	188.9	4856286.9	631163.2
GA-BR-03	190.7	4856328.0	631066.7
GA-BR-02	190.4	4856302.6	631090.9
GA-BR-01	189.8	4856282.4	631119.8
BH4	190.7	4856300.2	631070.5
BH3	189.2	4856286.2	631131.0
B104	189.3	4856281.7	631123.2
B103	190.6	4856295.4	631108.3
B102	190.6	4856294.0	631080.7
B101	190.5	4856304.9	631076.2

REFERENCES

Base plans provided in digital format by AECOM, drawing file nos. Bgd-2015.dwg, Mdbk_Ramp_Pln.dwg, Ramp_Aln.dwg, Ramp_Pln.dwg and Ramp_Prf.dwg, received October 13, 2015.



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.

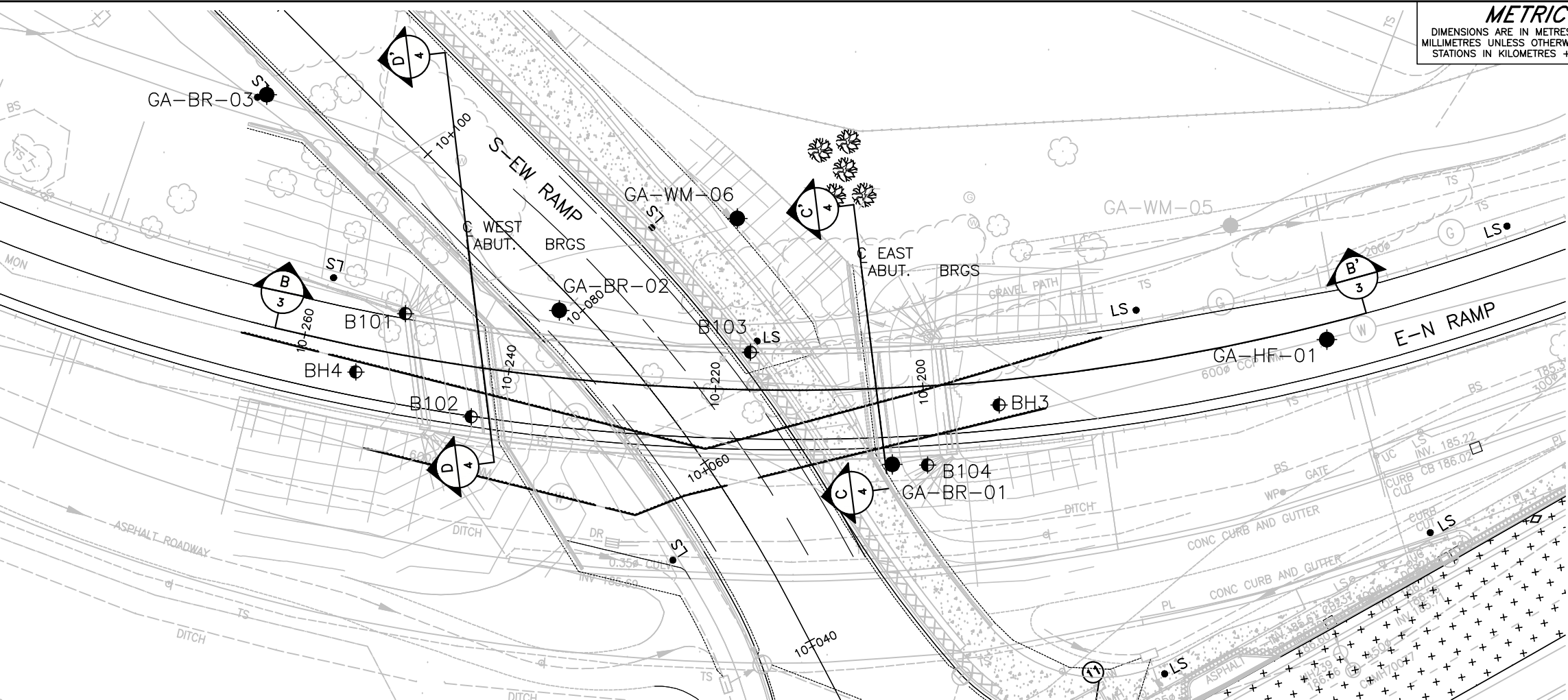
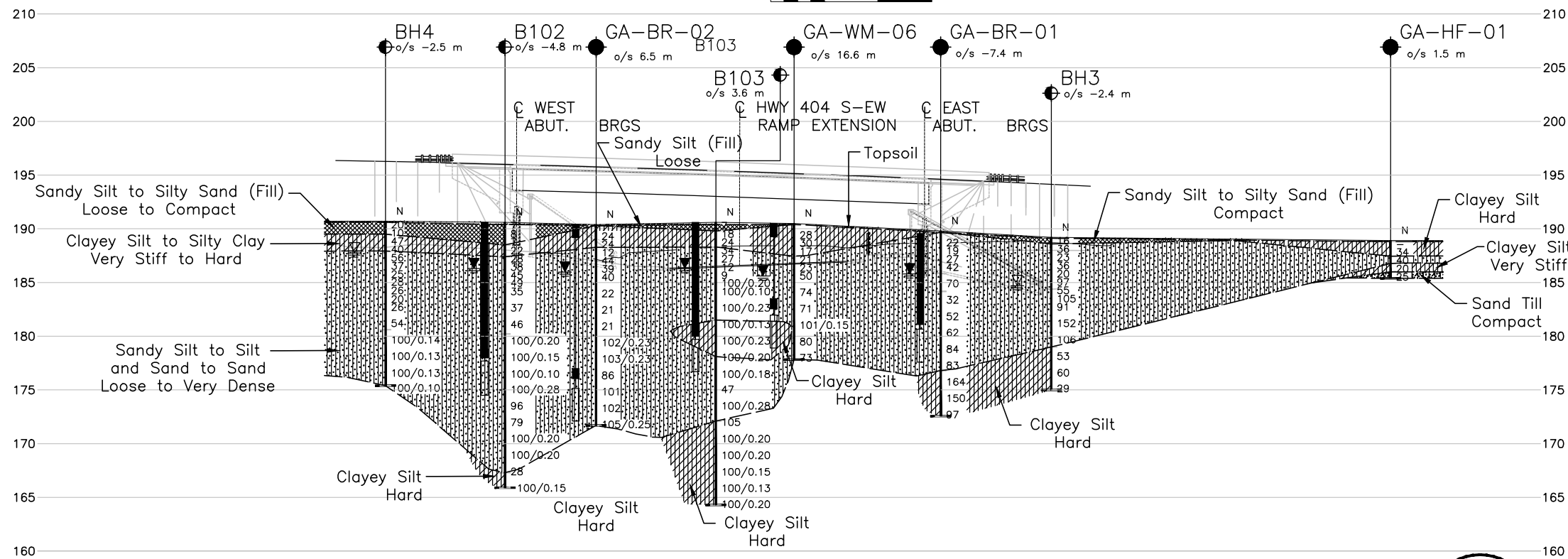
The complete Foundation Investigation and Design Report for this project and other related documents may be examined at the Materials Engineering and Research Office, Downsview. Information contained in this report and related documents is specifically excluded in accordance with Section GC 2.01 of OPS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



NO.	DATE	BY	REVISION

Geocres No. 30M14-425

HWY. 404	PROJECT NO. 1533525	DIST. .
SUBM'D. MCK	CHKD. MCK	DATE: Nov. 2015
DRAWN: JFC	CHKD. CN	APPD. JMAC
		SITE: .
		DWG. 2

PLAN
SCALE
5 0 5 10 m

REFERENCES

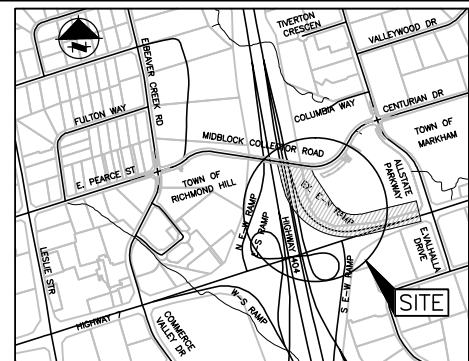
Base plans provided in digital format by AECOM, drawing file nos.
Bgd-2015.dwg, Mdbk_Ramp_Pln.dwg, Ramp_Aln.dwg, Ramp_Pln.dwg and
Ramp_Prf.dwg, received October 13, 2015.

B-B
3

CENTRELINE PROFILE OF E-N RAMP BRIDGE

SCALE
5 0 5 10 m

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

CONT No.
WP No.HIGHWAY 7 TO HIGHWAY 404
E-N RAMP BRIDGE
BOREHOLE LOCATIONS AND
SOIL STRATAKEY PLAN
NOT TO SCALE

LEGEND

- Borehole - Current Investigation
- Borehole - Previous Investigation
- ⊥ Seal
- ⊥ Piezometer
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- ≡ WL in piezometer (Refer to report text)
- ≡ WL upon completion of drilling

BOREHOLE CO-ORDINATES

No.	ELEVATION	NORTHING	EASTING
B101	190.5	4856304.9	631076.2
B102	190.6	4856294.0	631080.7
B103	190.6	4856295.4	631108.3
B104	189.3	4856281.7	631123.2
BH3	189.2	4856286.2	631131.0
BH4	190.7	4856300.2	631070.5
GA-BR-01	189.8	4856282.4	631119.8
GA-BR-02	190.4	4856302.6	631090.9
GA-BR-03	190.7	4856328.0	631066.7
GA-HF-01	188.9	4856286.9	631163.2
GA-WM-06	190.5	4856308.3	631109.3

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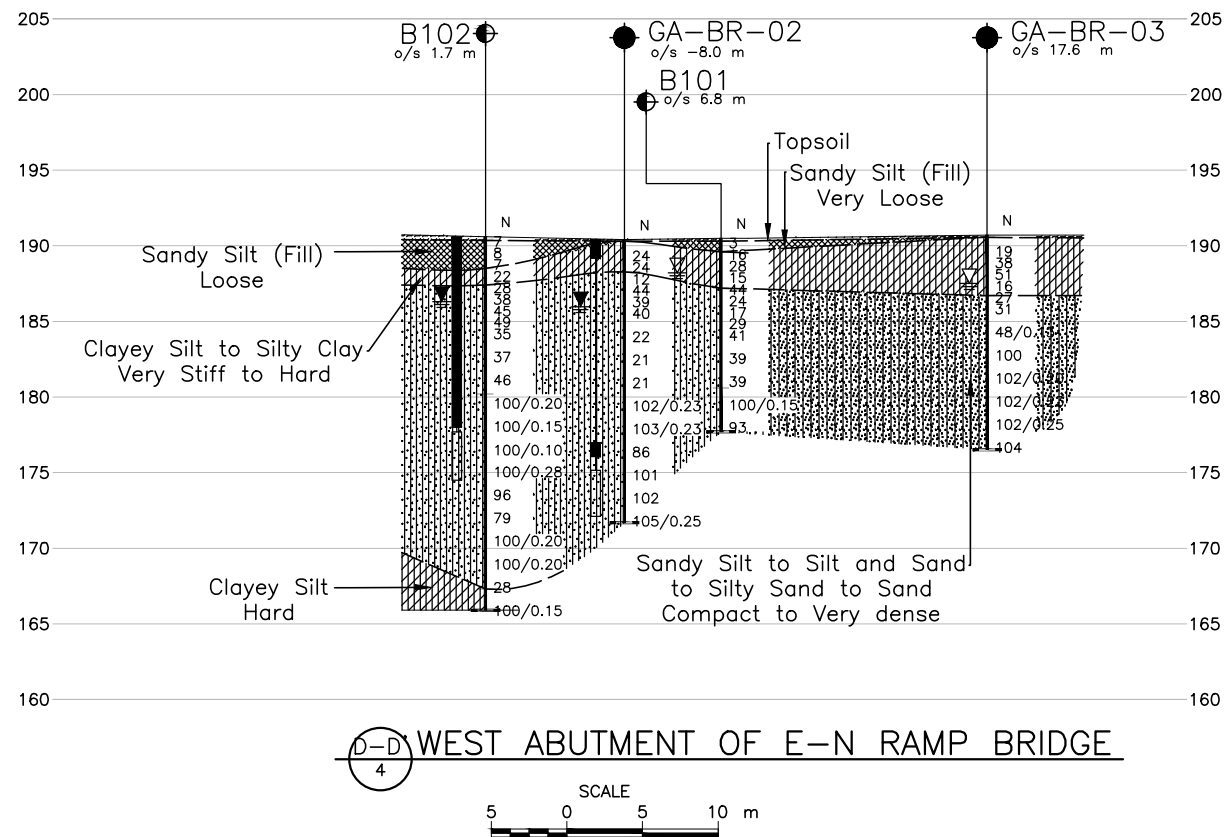
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NO.	DATE	BY	REVISION

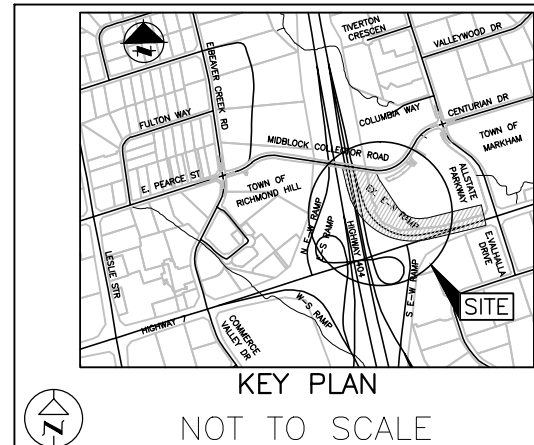
Geocres No. 30M14-425

HWY. 404	PROJECT NO. 1533525	DIST. .
SUBM'D. MCK	CHKD. MCK	DATE: Nov. 2015
DRAWN: JFC	CHKD. CN	APPD. JMAC
		DWG. 3











CONT No., WP No.,	
HIGHWAY 7 TO HIGHWAY 404 E-N RAMP BRIDGE SOIL STRATA	SHEET



LEGEND

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

BOREHOLE CO-ORDINATES			
No.	ELEVATION	NORTHING	EASTING
B101	190.5	4856304.9	631076.2
B102	190.6	4856294.0	631080.7
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GA-BR-03	190.7	4856328.0	631066.7
GA-WM-06	190.5	4856308.3	631109.3

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-	-	-	-
NO.	DATE	BY	REVISION
Geocres No. 30M14-425			
HWY. 404		PROJECT NO. 1533525	DIST. .
SUBM'D. MCK		CHKD. MCK	DATE: Nov. 2015
DRAWN: JFC		CHKD. CN	APPD. JMAC DWG. 4





APPENDIX A

Record of Boreholes – Golder 2015 Investigation



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

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

(a) Index Properties (continued)

w	water content
w_l 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

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

II. PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:

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

Dynamic Cone Penetration Resistance; N_d :

The number of blows by a 63.5 kg (140 lb.) hammer dropped 760 mm (30 in.) to drive uncased a 50 mm (2 in.) diameter, 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.

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

(b) Cohesive Soils Consistency

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

IV. SOIL TESTS

w	water content
w_p	plastic 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 (cohesionless)) or With (cohesive)	Sand and Gravel Silty Clay with sand / Clayey Silt with sand



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

PROJECT 1533525		RECORD OF BOREHOLE No GA-BR-01				SHEET 2 OF 2		METRIC																			
W.P. _____		LOCATION N 4856282.4 ; E 631119.8				ORIGINATED BY OS																					
DIST _____ HWY _____		BOREHOLE TYPE 108 mm I.D. Hollow Stem Power Augers				COMPILED BY JM																					
DATUM Geodetic		DATE September 14, 2015				CHECKED BY CN																					
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 (%) GR SA SI CL											
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa																			
	--- CONTINUED FROM PREVIOUS PAGE ---						20 40 60 80 100 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED					WATER CONTENT (%) 10 20 30															
172.6	CLAYEY SILT, trace sand, sandy silt pockets Hard Grey Moist to wet		12	SS	150		174																				
17.2	END OF BOREHOLE		13	SS	97		173																				
	NOTE: 1. Water level measurements in piezometer: <table border="1" style="display: inline-table; border-collapse: collapse;"> <thead> <tr> <th>Date</th> <th>Depth (m)</th> <th>Elev. (m)</th> </tr> </thead> <tbody> <tr> <td>09/14/15</td> <td>4.1</td> <td>185.7</td> </tr> <tr> <td>09/15/15</td> <td>3.9</td> <td>185.9</td> </tr> <tr> <td>10/13/15</td> <td>4.0</td> <td>185.8</td> </tr> </tbody> </table>	Date	Depth (m)	Elev. (m)	09/14/15	4.1	185.7	09/15/15	3.9	185.9	10/13/15	4.0	185.8														
Date	Depth (m)	Elev. (m)																									
09/14/15	4.1	185.7																									
09/15/15	3.9	185.9																									
10/13/15	4.0	185.8																									

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

PROJECT 1533525		RECORD OF BOREHOLE No GA-BR-02				SHEET 2 OF 2		METRIC																								
W.P. _____		LOCATION N 4856302.6 ; E 631090.9				ORIGINATED BY MCK																										
DIST _____ HWY _____		BOREHOLE TYPE 108 mm I.D. Hollow Stem Power Augers / 98 mm Open Hole Mud Rotary/Tricone				COMPILED BY JM																										
DATUM Geodectic		DATE September 10 and 11, 2015				CHECKED BY CN																										
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC NATURAL LIQUID LIMIT MOISTURE LIMIT CONTENT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL																
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa					WATER CONTENT (%)																			
--- CONTINUED FROM PREVIOUS PAGE ---							20 40 60 80 100 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100					W _p W W _L 10 20 30																				
171.7	SAND, trace to some silt, trace clay, trace to some gravel Compact to very dense Brown to grey Wet	[Pattern]	13	SS	101	[Pattern]	175																									
							174																									
			14	SS	102		173																									
18.7	END OF BOREHOLE		15	SS	105/0.25	[Pattern]	172																									
NOTE: 1. Water level measurements in piezometer: <table style="margin-left: 40px;"> <tr> <th>Date</th> <th>Depth (m)</th> <th>Elev. (m)</th> </tr> <tr> <td>09/11/15</td> <td>4.4</td> <td>186.0</td> </tr> <tr> <td>09/11/15</td> <td>4.8</td> <td>185.6</td> </tr> <tr> <td>09/15/15</td> <td>4.6</td> <td>185.8</td> </tr> <tr> <td>10/13/15</td> <td>4.8</td> <td>185.6</td> </tr> </table>		Date	Depth (m)	Elev. (m)	09/11/15	4.4	186.0	09/11/15	4.8	185.6	09/15/15	4.6	185.8	10/13/15	4.8	185.6																
Date	Depth (m)	Elev. (m)																														
09/11/15	4.4	186.0																														
09/11/15	4.8	185.6																														
09/15/15	4.6	185.8																														
10/13/15	4.8	185.6																														

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



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

PROJECT		RECORD OF BOREHOLE				No GA-WM-01		SHEET 1 OF 1		METRIC							
W.P. _____		LOCATION				N 4856357.2 ; E 631401.5		ORIGINATED BY		AJS							
DIST _____ HWY _____		BOREHOLE TYPE				108 mm I.D. Hollow Stem Power Augers		COMPILED BY		JM							
DATUM Geodectic		DATE				September 9, 2015		CHECKED BY		CN							
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 (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
187.4	GROUND SURFACE																
0.0	Sandy clayey silt, some gravel to gravelly (FILL) Stiff Brown Dry		1	AS	-												
186.0			2	SS	9												
1.4	CLAYEY SILT, trace to some sand to sandy, trace rootlets to 2.0 m Very stiff to hard Brown Moist		3	SS	20												
			4	SS	28												
			5	SS	50/0.15												
183.9	- Auger grinding at a depth of 3.2 m																
183.5	SILT and SAND Brown to grey Wet		6A	SS	8												
3.9	SILT and SAND, some clay, trace gravel (TILL) Loose to dense Grey Wet		6B														
			7	SS	18												
			8	SS	44												
180.8	END OF BOREHOLE																
6.6	NOTES: 1. Open borehole dry, upon completion of drilling.																

PROJECT 1533525		RECORD OF BOREHOLE No GA-WM-02				SHEET 1 OF 1		METRIC																					
W.P. _____		LOCATION N 4856330.0 ; E 631328.2				ORIGINATED BY AJS																							
DIST _____ HWY _____		BOREHOLE TYPE 108 mm I.D. Hollow Stem Power Augers				COMPILED BY JM																							
DATUM Geodetic		DATE September 9, 2015				CHECKED BY CN																							
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										WATER CONTENT (%)											
186.7	GROUND SURFACE																												
0.0	Sandy clayey silt, rootlets (FILL) Brown Dry		1	AS	-																								
185.9			2	SS	33																								
0.8	CLAYEY SILT, some sand to sandy Very stiff to hard Brown Moist		3	SS	26																								
			4	SS	19																								
183.8																													
2.9	SILT and SAND, some gravel, trace clay Compact Brown Wet		5	SS	16																								
183.0																													
3.7	SILT and SAND, trace to some gravel, trace to some clay (TILL) Compact Grey Moist		6	SS	10																								
			7	SS	14																								
			8	SS	22																								
			9	SS	19																								
178.6																													
8.1	END OF BOREHOLE																												
NOTE: 1. Water level measurements in piezometer: <table style="margin-left: 40px;"> <tr> <td>Date</td> <td>Depth (m)</td> <td>Elev. (m)</td> </tr> <tr> <td>09/11/15</td> <td>2.7</td> <td>184.0</td> </tr> <tr> <td>09/15/15</td> <td>2.9</td> <td>183.8</td> </tr> <tr> <td>10/13/15</td> <td>2.9</td> <td>183.8</td> </tr> </table>																		Date	Depth (m)	Elev. (m)	09/11/15	2.7	184.0	09/15/15	2.9	183.8	10/13/15	2.9	183.8
Date	Depth (m)	Elev. (m)																											
09/11/15	2.7	184.0																											
09/15/15	2.9	183.8																											
10/13/15	2.9	183.8																											

PROJECT <u>1533525</u>	RECORD OF BOREHOLE No GA-WM-03	SHEET 1 OF 1	METRIC
W.P. _____	LOCATION <u>N 4856319.8 ; E 631251.3</u>	ORIGINATED BY <u>AJS</u>	
DIST _____ HWY _____	BOREHOLE TYPE <u>108 mm I.D. Hollow Stem Power Augers</u>	COMPILED BY <u>JM</u>	
DATUM <u>Geodectic</u>	DATE <u>September 9, 2015</u>	CHECKED BY <u>CN</u>	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC NATURAL LIQUID LIMIT MOISTURE CONTENT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)				GR	SA	SI	CL
								20	40	60	80	100	W _p	W	W _L					
188.8	GROUND SURFACE																			
0.0	Sandy clayey silt, trace to some gravel (FILL) Brown Dry		1	AS	-															
188.0																				
0.8	CLAYEY SILT, some sand, some gravel Very stiff to hard Brown Moist - Auger grinding at a depth of 2.6 m		2	SS	21															
			3	SS	32															
			4	SS	50/0.15															
			5A																	
185.4			5B	SS	29															
3.4	SILT and SAND, trace to some clay, trace to some gravel (TILL) Compact to very dense Grey Moist to wet - 15 cm silt layer encountered at a depth of 3.2 m		5C																	
			6	SS	19															
			7	SS	23															
			8	SS	28															
			9	SS	81															
180.7	- 5 cm silty clay pocket encountered at a depth of 7.9 m																			
8.1	END OF BOREHOLE																			
NOTES: 1. Water level in open borehole at a depth of 7.2 m below ground surface (Elev. 181.6 m), measured upon completion of drilling.																				

PROJECT 1533525		RECORD OF BOREHOLE No GA-WM-04 SHEET 1 OF 1				METRIC						
W.P. _____		LOCATION N 4856304.4 ; E 631219.3				ORIGINATED BY OS						
DIST _____ HWY _____		BOREHOLE TYPE 108 mm I.D. Hollow Stem Power Augers				COMPILED BY JM						
DATUM Geodetic		DATE September 10, 2015				CHECKED BY CN						
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT		UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	20 40 60 80 100	W _p W W _L			WATER CONTENT (%)
188.5	GROUND SURFACE											
0.0	TOPSOIL											
	CLAYEY SILT, some sand, trace wood fragments, rootlets Very stiff Brown Moist		1	SS	24							
187.1												
1.4	CLAYEY SILT, trace sand Hard Grey Moist		2A 2B	SS	33							
186.4												
2.1	- 15 cm sand layer at a depth of 1.7 m		3	SS	24							
	SILT and SAND, trace gravel, trace clay Compact Brown Wet		4	SS	37							
185.4												
3.1	Sandy SILT to Silty SAND, some gravel to gravelly, trace to some clay, inferred cobbles at a depth of 9.5 m and 11.1 m (TILL) Dense to very dense Grey Moist		5	SS	41							
			6	SS	32							
			7	SS	92							
			8	SS	151/0.18							
			9	SS	190/0.28							
177.4	END OF BOREHOLE											
11.1	NOTE: 1. Water level measurements in piezometer: Date Depth (m) Elev. (m) 09/11/15 Dry - 09/15/15 Dry - 10/13/15 Dry - 2. Well installed 1.0 m west of GA-WM-04.											

[illegible]

PROJECT 1533525		RECORD OF BOREHOLE No GA-WM-05					SHEET 2 OF 2		METRIC																		
W.P. _____		LOCATION N 4856299.3 ; E 631156.0					ORIGINATED BY OS																				
DIST _____ HWY _____		BOREHOLE TYPE 108 mm I.D. Hollow Stem Power Augers					COMPILED BY JM																				
DATUM Geodectic		DATE September 10 and 11, 2015					CHECKED BY CN																				
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 (%) GR SA SI CL											
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa																			
--- CONTINUED FROM PREVIOUS PAGE ---							<div style="display: flex; justify-content: space-between;"> 20 40 60 80 100 20 40 60 80 100 </div> <div style="display: flex; justify-content: space-between;"> ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED </div>																				
NOTES: 1. Water level measurements in piezometer <table style="width: 100%; border-collapse: collapse;"> <thead> <tr> <th style="text-align: left;">Date</th> <th style="text-align: left;">Depth (m)</th> <th style="text-align: left;">Elev. (m)</th> </tr> </thead> <tbody> <tr> <td>09/11/15</td> <td>4.6</td> <td>185.5</td> </tr> <tr> <td>09/15/15</td> <td>4.7</td> <td>185.4</td> </tr> <tr> <td>10/13/15</td> <td>4.6</td> <td>185.5</td> </tr> </tbody> </table> 2. Well installed 1.0 m west of GA-WM-05.		Date	Depth (m)	Elev. (m)	09/11/15	4.6	185.5	09/15/15	4.7	185.4	10/13/15	4.6	185.5														
Date	Depth (m)	Elev. (m)																									
09/11/15	4.6	185.5																									
09/15/15	4.7	185.4																									
10/13/15	4.6	185.5																									

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PROJECT 1533525		RECORD OF BOREHOLE No GA-WM-06				SHEET 1 OF 1		METRIC											
W.P. _____		LOCATION N 4856308.3 ; E 631109.3				ORIGINATED BY MCK													
DIST _____ HWY _____		BOREHOLE TYPE 108 mm I.D. Hollow Stem Power Augers				COMPILED BY JM													
DATUM Geodetic		DATE September 10, 2015				CHECKED BY CN													
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS		ELEVATION SCALE		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							W _p	W	W _L	γ	GR	SA	SI	CL
190.5	GROUND SURFACE																		
0.9	TOPSOIL																		
	CLAYEY SILT, trace to some sand, trace to some gravel Very stiff Brown Moist		1	SS	28								o						
			2	SS	30														
	- 8 cm sandy silt layer encountered at a depth of 4.0 m		3A	SS	17								o						
			3B	SS															
187.4			4	SS	21								o						
3.1	SILT and SAND to SAND, trace clay, trace to some silt, trace gravel to gravelly, inferred cobbles at depths of 3.8 m, 5.3 m and 8.5 m to Compact to very dense Brown to grey Dry to wet		5A	SS	23														
			5B	SS															
	- 6 cm clayey silt layer encountered at a depth of 4.0 m		6	SS	50														
			7	SS	74														
			8	SS	71														
			9	SS	101/0.15														
			10A	SS	80														
			10B	SS															
			11A	SS	73														
			11B	SS															
177.8	END OF BOREHOLE																		
12.7	NOTE: 1. Water level measurements in piezometer: Date Depth (m) Elev. (m) 09/10/15 5.0 185.5 09/11/15 4.9 185.6 09/15/15 4.9 185.6 10/13/15 4.9 185.6																		

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PROJECT <u>1533525</u>		RECORD OF BOREHOLE No GA-HF-01		SHEET 1 OF 1		METRIC	
W.P. _____		LOCATION <u>N 4856286.9 ; E 631163.2</u>		ORIGINATED BY <u>AJS</u>			
DIST _____ HWY _____		BOREHOLE TYPE <u>108 mm I.D. Hollow Stem Power Augers / 98 mm Open Hole Mud Rotary/Tricone</u>		COMPILED BY <u>JM</u>			
DATUM <u>Geodetic</u>		DATE <u>September 9, 2015</u>		CHECKED BY <u>CN</u>			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)							
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED					w _p w w _L							
188.9	GROUND SURFACE							20	40	60	80	100								
0.0	TOPSOIL						188													
0.1	CLAYEY SILT, some sand to sandy, trace to some gravel Hard Brown Dry		1	AS	-															
			2	SS	34															
187.5							187													
1.5	SILT and SAND Dense Brown Moist		3	SS	40															
186.8																				
2.1	CLAYEY SILT Very stiff Grey Moist		4	SS	20		186													
186.0																				
2.9	SAND, some silt, some gravel (TILL) Compact Grey Moist		5	SS	25															
185.4																				
3.5	END OF BOREHOLE																			
NOTES: 1. Open borehole dry, upon completion of drilling.																				

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PROJECT		1533525		RECORD OF BOREHOLE No GA-HF-02				SHEET 1 OF 1		METRIC							
W.P.				LOCATION		N 4856325.8 ; E 631019.6		ORIGINATED BY		MCK							
DIST		HWY		BOREHOLE TYPE		108 mm I.D. Hollow Stem Power Augers		COMPILED BY		JM							
DATUM		Geodectic		DATE		September 9, 2015		CHECKED BY		CN							
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									
192.6	GROUND SURFACE																
0.0	TOPSOIL																
	Silty SAND, trace organics Dense Dark brown Moist		1A 1B	SS	36												
191.0																	
1.6	Sandy SILT, trace to some clay, trace gravel Very dense Brown Moist		2A 2B	SS	57												
			3	SS	104												
			4	SS	157/0.25												
			5	SS	100/0.13												
188.0																	
4.6	SAND, trace silt to silty, trace gravel Very dense Brown Moist to wet		6	SS	74												
			7	SS	105/0.23												
186.1																	
6.5	END OF BOREHOLE																
NOTES:																	
1. Open borehole dry, upon completion of drilling.																	

PROJECT 1533525		RECORD OF BOREHOLE No GA-HF-03				SHEET 1 OF 1		METRIC									
W.P. _____		LOCATION N 4856373.9 ; E 631001.2				ORIGINATED BY MCK											
DIST _____ HWY _____		BOREHOLE TYPE 108 mm I.D. Hollow Stem Power Augers				COMPILED BY JM											
DATUM Geodectic		DATE September 9, 2015				CHECKED BY CN											
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									
193.4	GROUND SURFACE																
0.0	TOPSOIL																
0.2	SILT and SAND, some clay, trace gravel, trace organics Compact to very dense Brown Dry to wet		1	SS	20												
			2	SS	19												
			3A	SS	62												
190.7	Silty SAND, trace to some gravel Very dense Brown Moist		3B														
2.7			4	SS	170/0.25												
189.9	END OF BOREHOLE																
3.5	NOTE: 1. Open borehole dry from observation upon completion of drilling.																



APPENDIX B

Laboratory Test Results



FOUNDATION REPORT - E-N RAMP RE-ALIGNMENT BRIDGE / APPROACH EMBANKMENTS AND ASSOCIATED STRUCTURES

TABLE B1 – SUMMARY OF ANALYTICAL TESTING OF GROUNDWATER

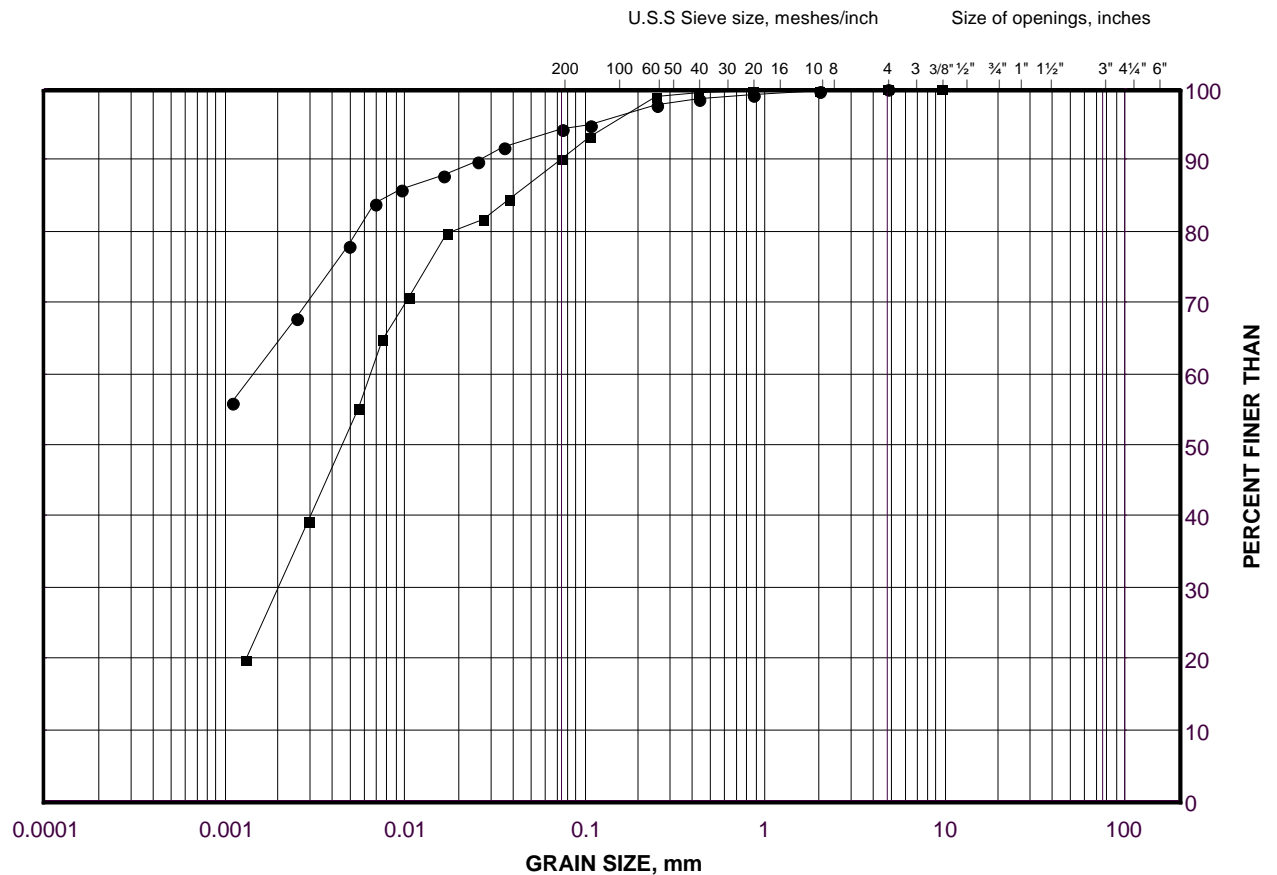
Borehole Number	Parameter (Units / Detection Limit)				
	Dissolved Chloride (mg/L / 0.5)	Dissolved Sulfate (mg/L / 0.5)	Conductivity (uS/cm / 2)	Resistivity (ohm-cm)	pH
GA-WM-02	230 / 0.5	72.6 / 0.5	1330 / 2	752	7.85
GA-WM-06	143 / 0.5	59.2 / 0.5	1190 / 2	840	7.88

Notes: 1. Samples obtained October 13, 2015.
2. Analytical testing carried out by Agat Laboratories.

Prepared by: MCK
Checked by: CN

Clayey Silt (Upper)
E-N Ramp

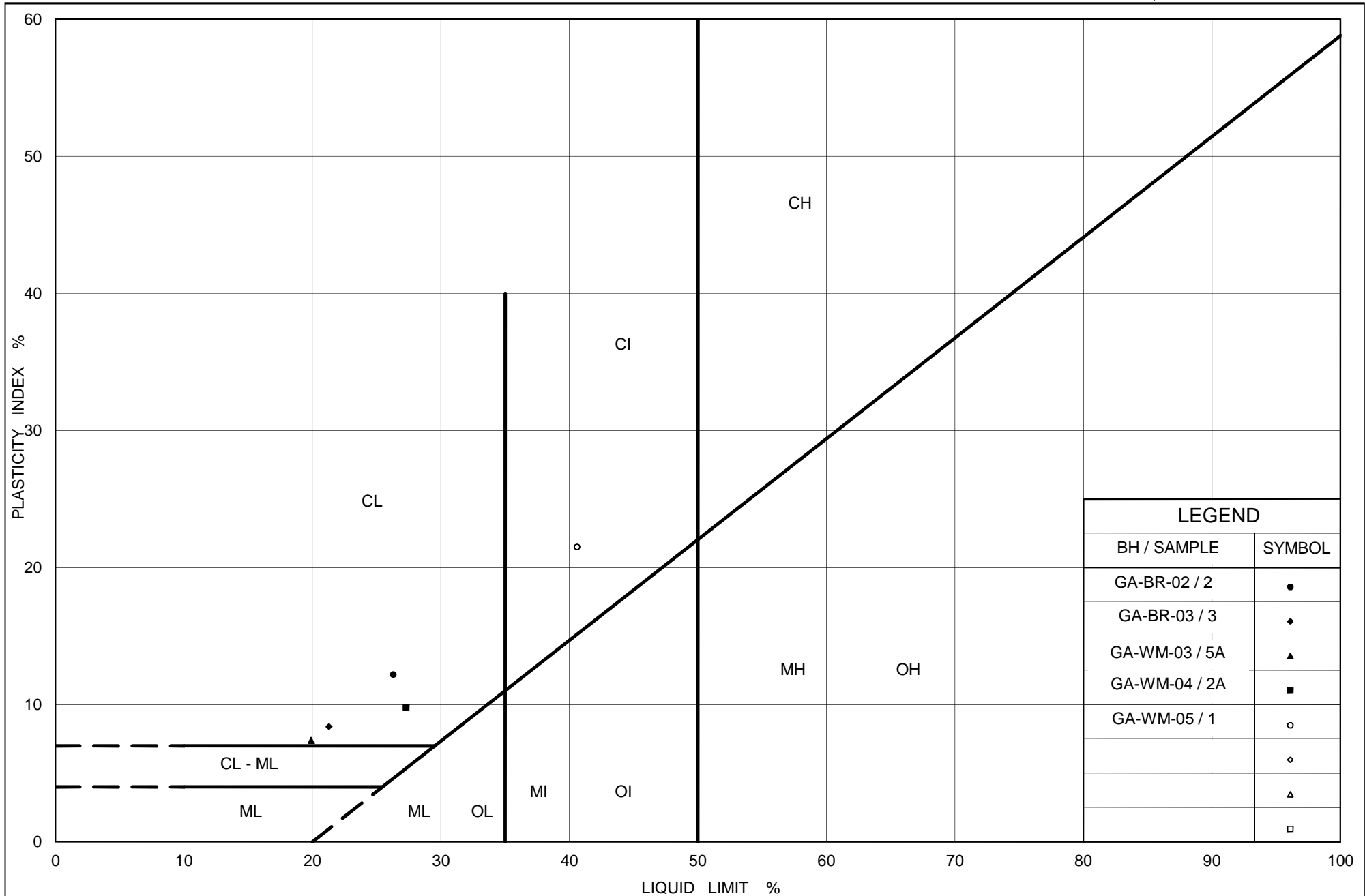
FIGURE B1



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	GA-WM-01	3	185.7
■	GA-BR-03	4B	187.3



Ministry of Transportation

Ontario

PLASTICITY CHART Clayey Silt to Silty Clay (Upper) E-N Ramp

Figure No. B2

Project No. 1533525

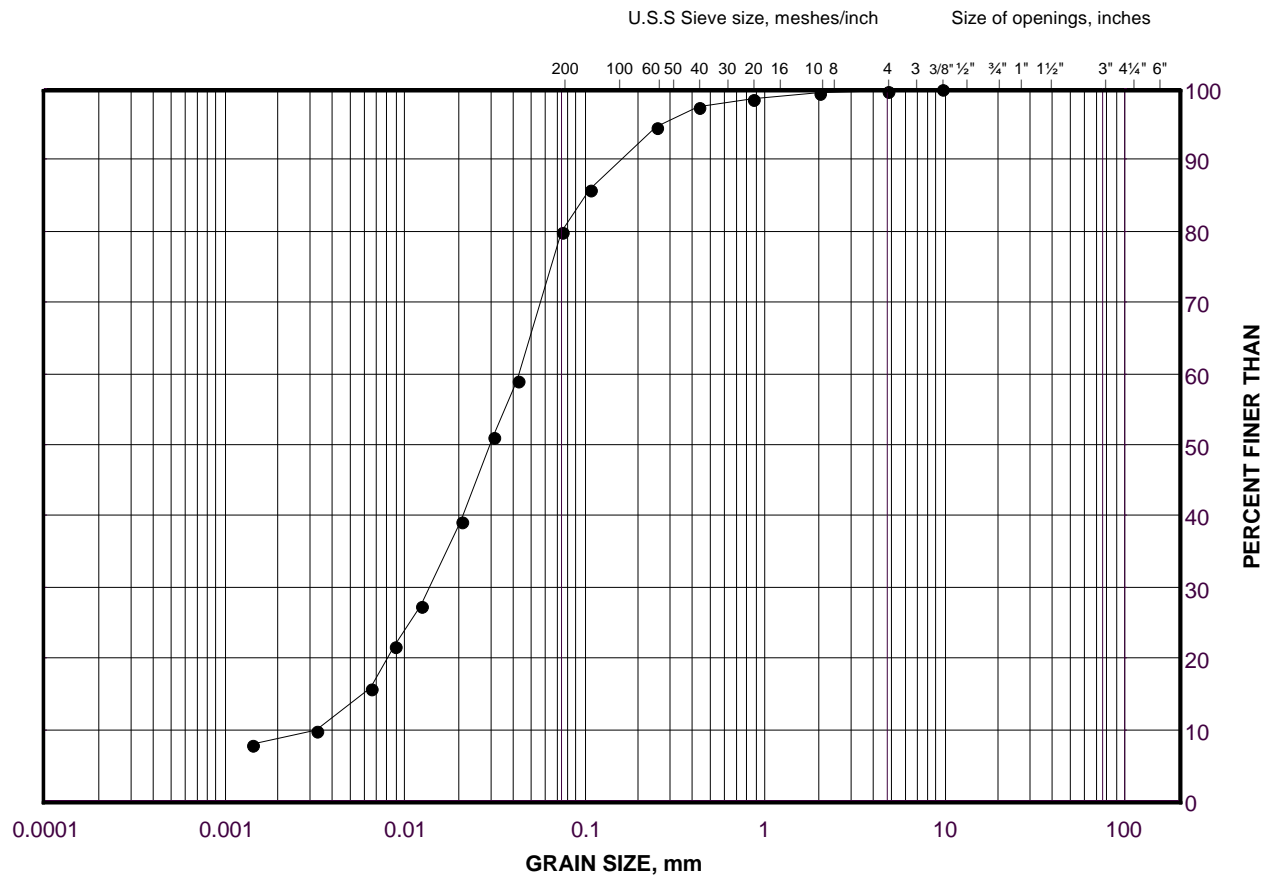
Checked By: CN

GRAIN SIZE DISTRIBUTION

Sandy Silt

E-N Ramp

FIGURE B3A



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
•	GA-HF-02	4	189.3

Project Number: 1533525

Checked By: CN

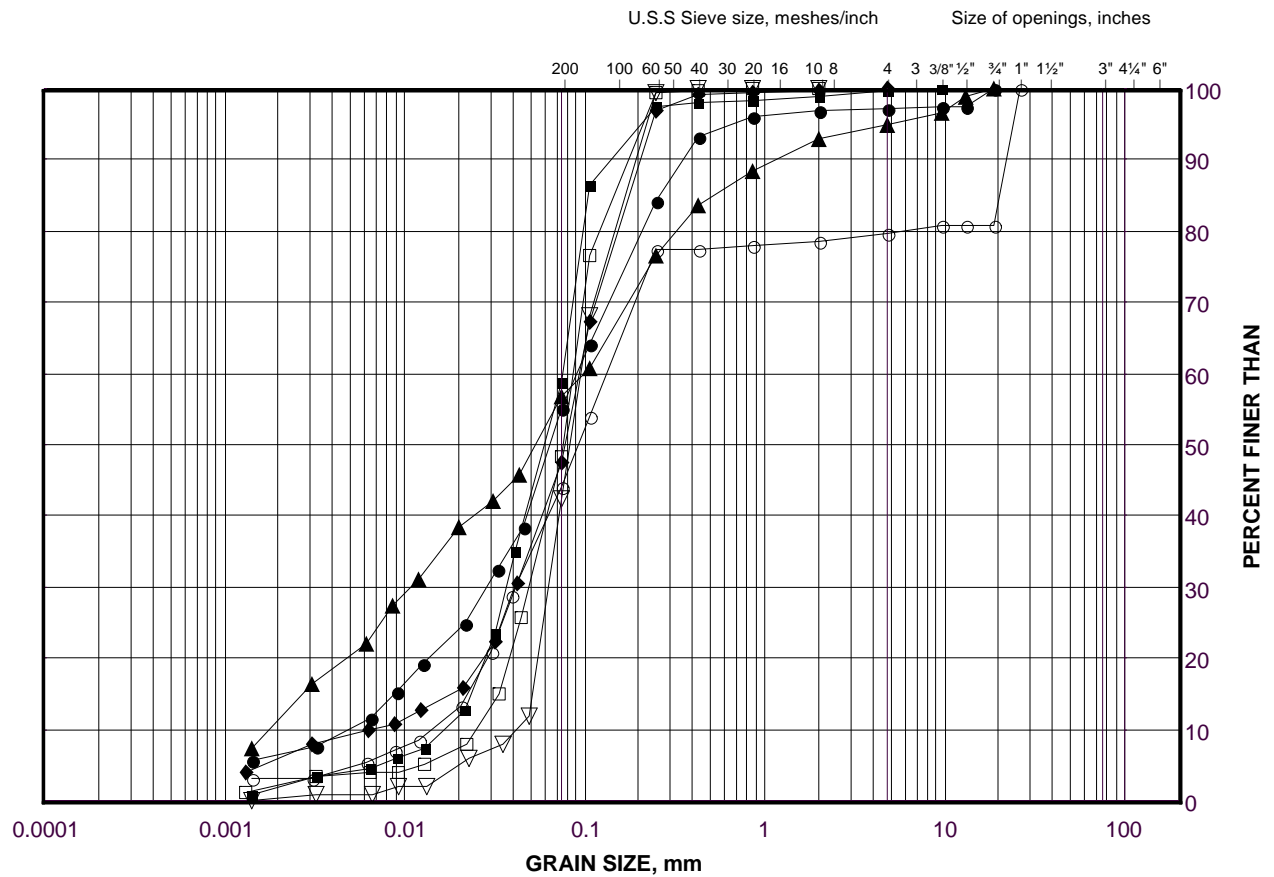
Golder Associates

Date: 18-Oct-15

GRAIN SIZE DISTRIBUTION

Silt and Sand
E-N Ramp

FIGURE B3B



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	GA-HF-03	2	191.7
■	GA-WM-04	3	186.0
◆	GA-WM-05	4	186.8
▲	GA-BR-01	5	185.0
▽	GA-BR-03	6	185.9
○	GA-WM-06	7	184.2
□	GA-WM-05	7	182.2

Project Number: 1533525

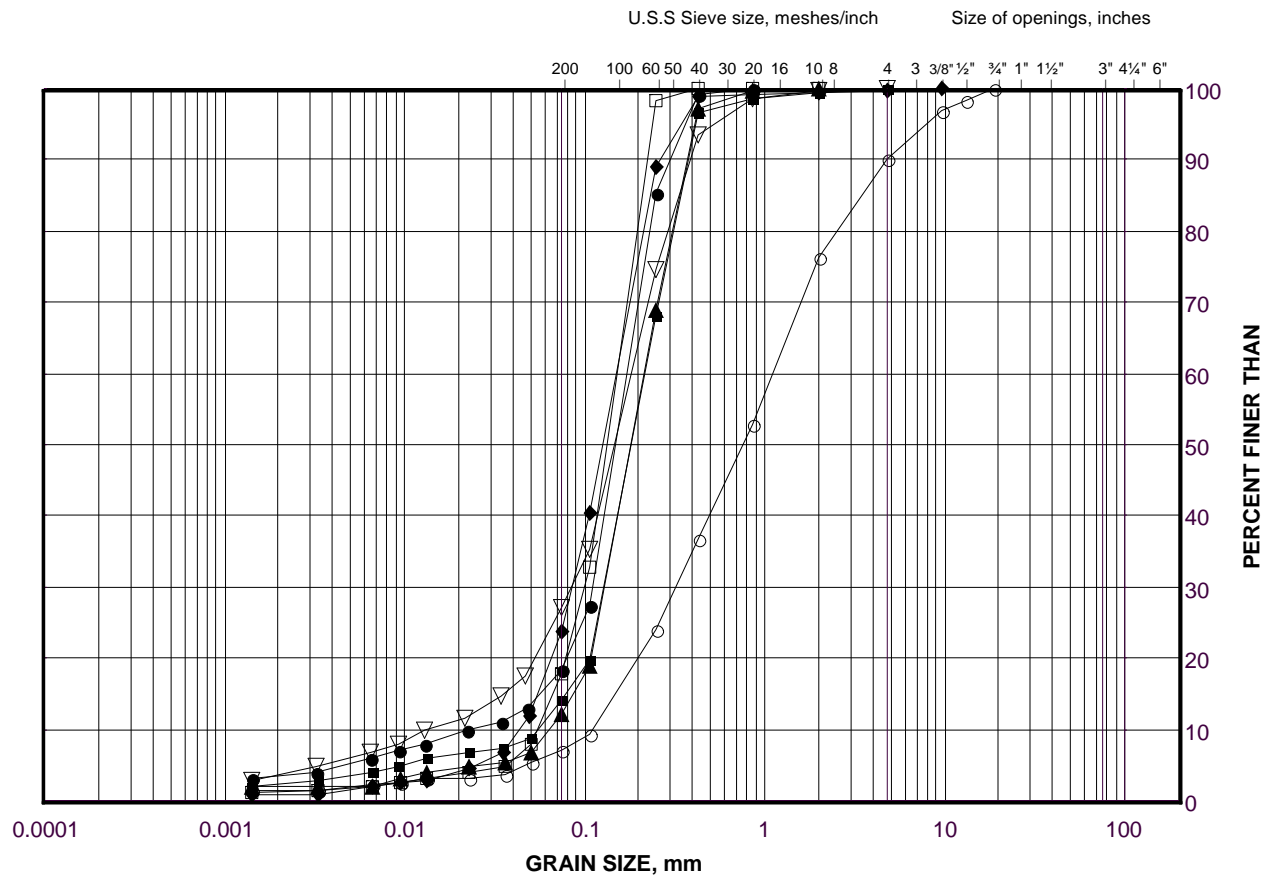
Checked By: CN

Golder Associates

Date: 18-Oct-15

Silty Sand to Sand
E-N Ramp

FIGURE B3C



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	GA-WM-06	11B	177.9
■	GA-BR-02	14	173.4
◆	GA-BR-01	3	187.3
▲	GA-BR-02	6	185.6
▽	GA-BR-03	8	182.8
○	GA-BR-02	9	181.0
□	GA-BR-01	9	178.9

Project Number: 1533525

Checked By: CN

Golder Associates

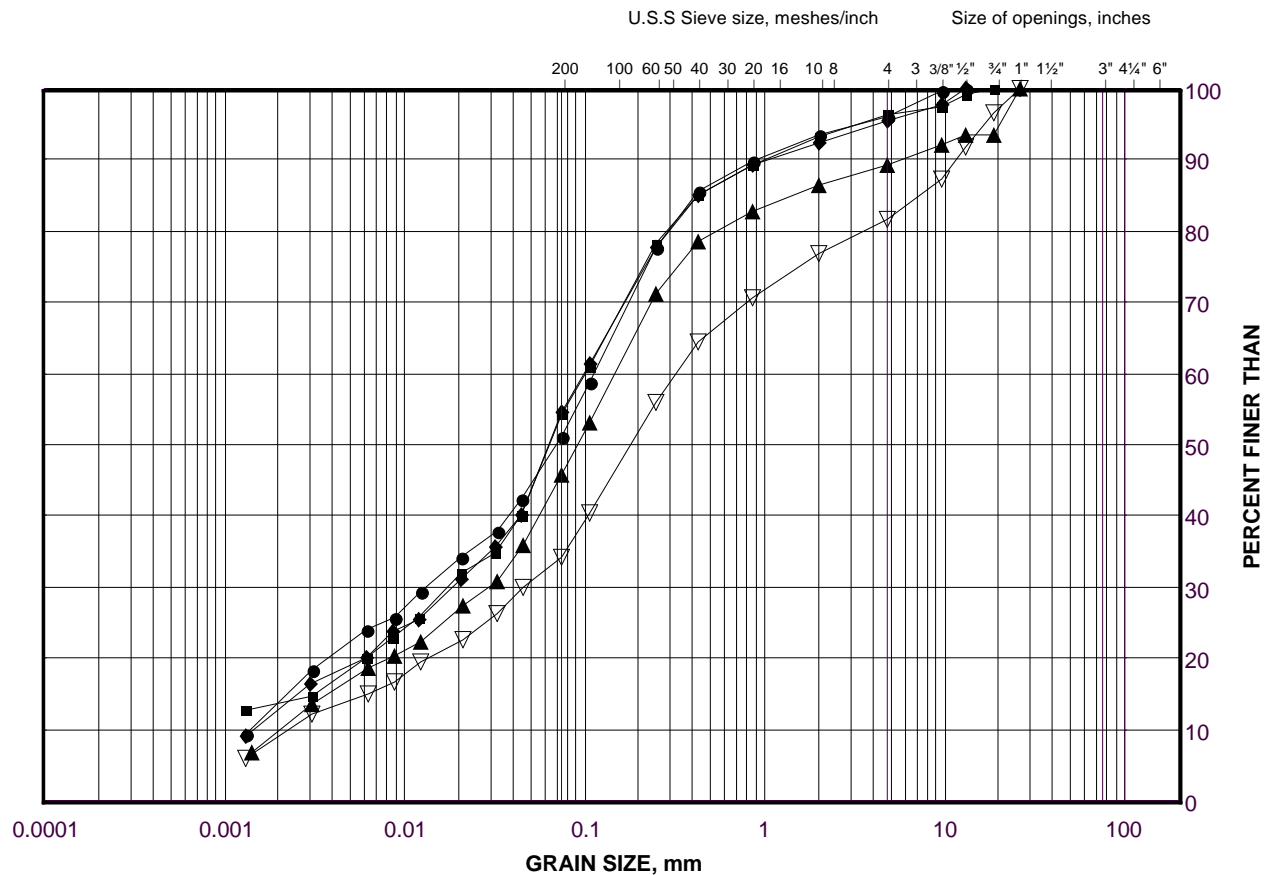
Date: 18-Oct-15

GRAIN SIZE DISTRIBUTION

Silt and Sand to Silty Sand Till

E-N Ramp

FIGURE B4



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

LEGEND

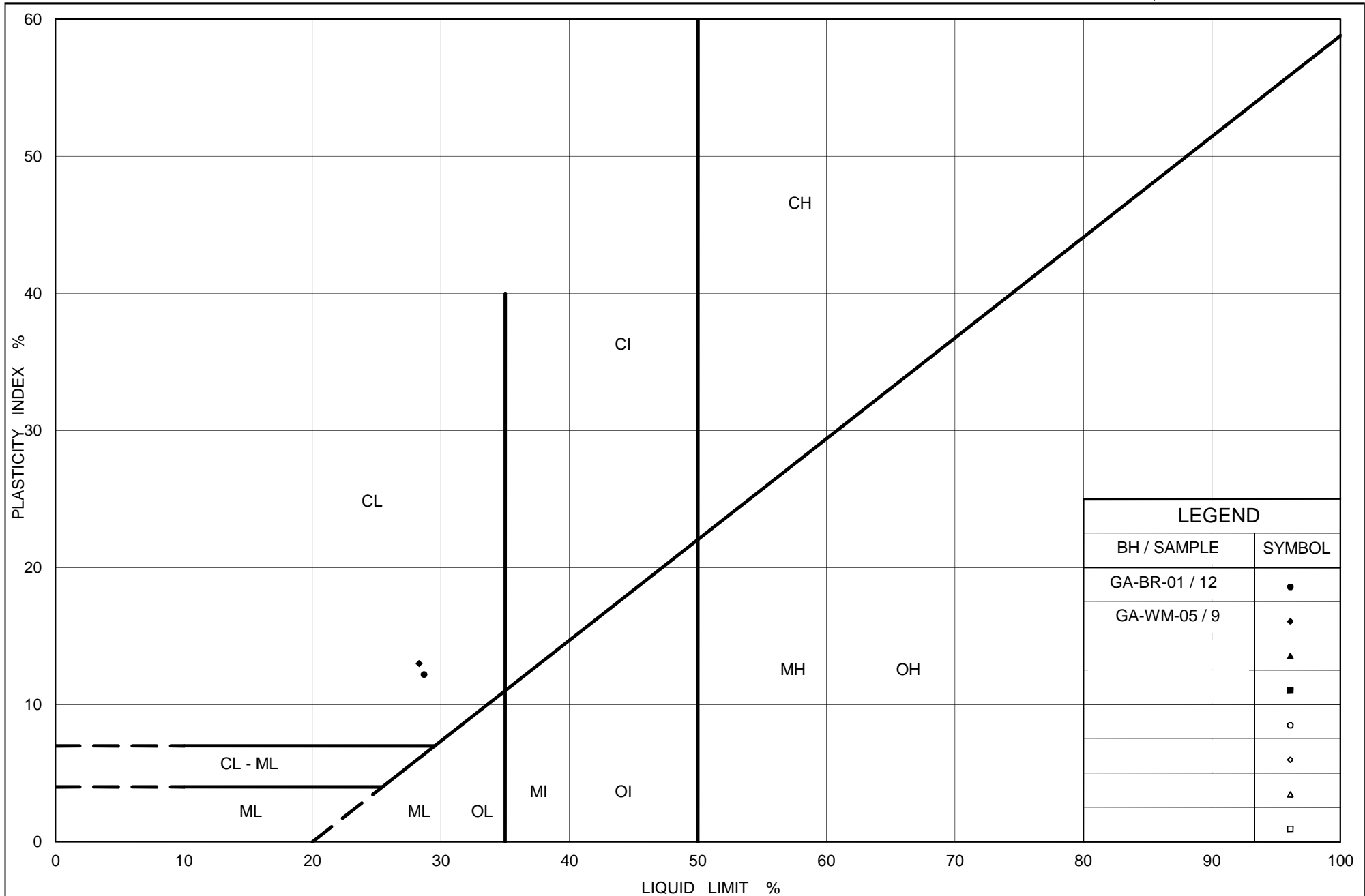
SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	GA-WM-02	6	182.7
■	GA-WM-01	7	182.6
◆	GA-WM-03	7	184.0
▲	GA-WM-02	8	180.4
▽	GA-WM-04	9	177.6

Project Number: 1533525

Checked By: CN

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Date: 18-Oct-15



Ministry of Transportation

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PLASTICITY CHART Clayey Silt (Lower) E-N Ramp

Figure No. B5

Project No. 1533525

Checked By: CN



APPENDIX C

Previous Investigation – Record of Boreholes and Laboratory Test Results

TRANETOB10757AA: Highway 404/7

RECORD OF BOREHOLE No B101

1 OF 1

METRIC

GWP G.W.P LOCATION 404 E-N On Ramp-Station 0+240, 6 m Rt of C/L (631076.2 E, 4856304.9 N) ORIGINATED BY LG
 DIST HWY 404 BOREHOLE TYPE Hollow Stem Auger, NW Casing, Wash Boring, DCPT COMPILED BY AS
 DATUM Geodetic DATE 5/20/2011 CHECKED BY ZO

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 (%) GR SA SI CL
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH (kPa) ○ UNCONFINED + FIELD VANE ● POCKET PENETR. × LAB VANE						
190.5 0.0	GROUND SURFACE						20 40 60 80 100							
189.6 0.9	0.2 m TOPSOIL FILL: Sandy Silt some organics, trace clay dk. brown to brown, loose to compact, moist		1	SS	3									
			2	SS	16									
	SILTY CLAY trace gravel, trace sand brown, v. stiff to hard, moist		3	SS	28									
			4	SS	15								42.4	
187.2 3.3			5	SS	44									
			6	SS	24									
			7	SS	17									
	SILTY SAND TO SANDY SILT trace gravel brown, compact to dense, wet		8	SS	29									
			9	SS	41									
			10	SS	39									
			11	SS	39									
180.6 9.9			12	SS	100 / 15 cm									
			13	SS	93									
177.7 12.8	End of borehole @ 12.8 m Water level @ 2.3 m (not stabilized)* upon completion Hole caved-in @ 4.0 m upon completion DCPT performed from 12.8 to 13.0 m													

+³, x³: Numbers refer to
Sensitivity

20
15
10

(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No B102

1 OF 2

METRIC

+³, ×³: Numbers refer to Sensitivity

TRANETOB10757AA: Highway 404/7

RECORD OF BOREHOLE No B102

2 OF 2

METRIC

GWP G.W.P LOCATION 404 E-N On Ramp-Station 0+234, 3 m Lt of C/L (631080.7 E, 4856294.0 N) ORIGINATED BY LG
 DIST HWY 404 BOREHOLE TYPE Hollow Stem Auger, NW Casing, Wash Boring, DCPT COMPILED BY AS
 DATUM Geodetic DATE 5/30/2011 5/31/2011 CHECKED BY ZO

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 (%) GR SA SI CL
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH (kPa)							
175.6			15	SS100 / 28			175								
							174								
			16	SS	96		173								
							172								
			17	SS	79		171								
							170								
			18	SS100 / 20			169								
							168								
			19	SS100 / 20			167								
							166								
167.3			20	SS	28										
23.3															
165.9			21	SS100 / 15											
24.7															
	End of borehole Water level in borehole @ 4.6 m (not stabilized)* and caved-in @ 5.2 m upon completion Monitoring well installed to 16.0 m Water Level Records : May 31, 2011 4.6 m June 1, 2011 4.3 m June 8, 2011 4.3 m June 21, 2011 4.3 m June 22, 2011 4.3 m June 30, 2011 4.3 m														

+ 3, x 3; Numbers refer to
Sensitivity

20
15
10

(%) STRAIN AT FAILURE

0.3 m backup
casing stuck,
pumped more
water
N value not
reliable

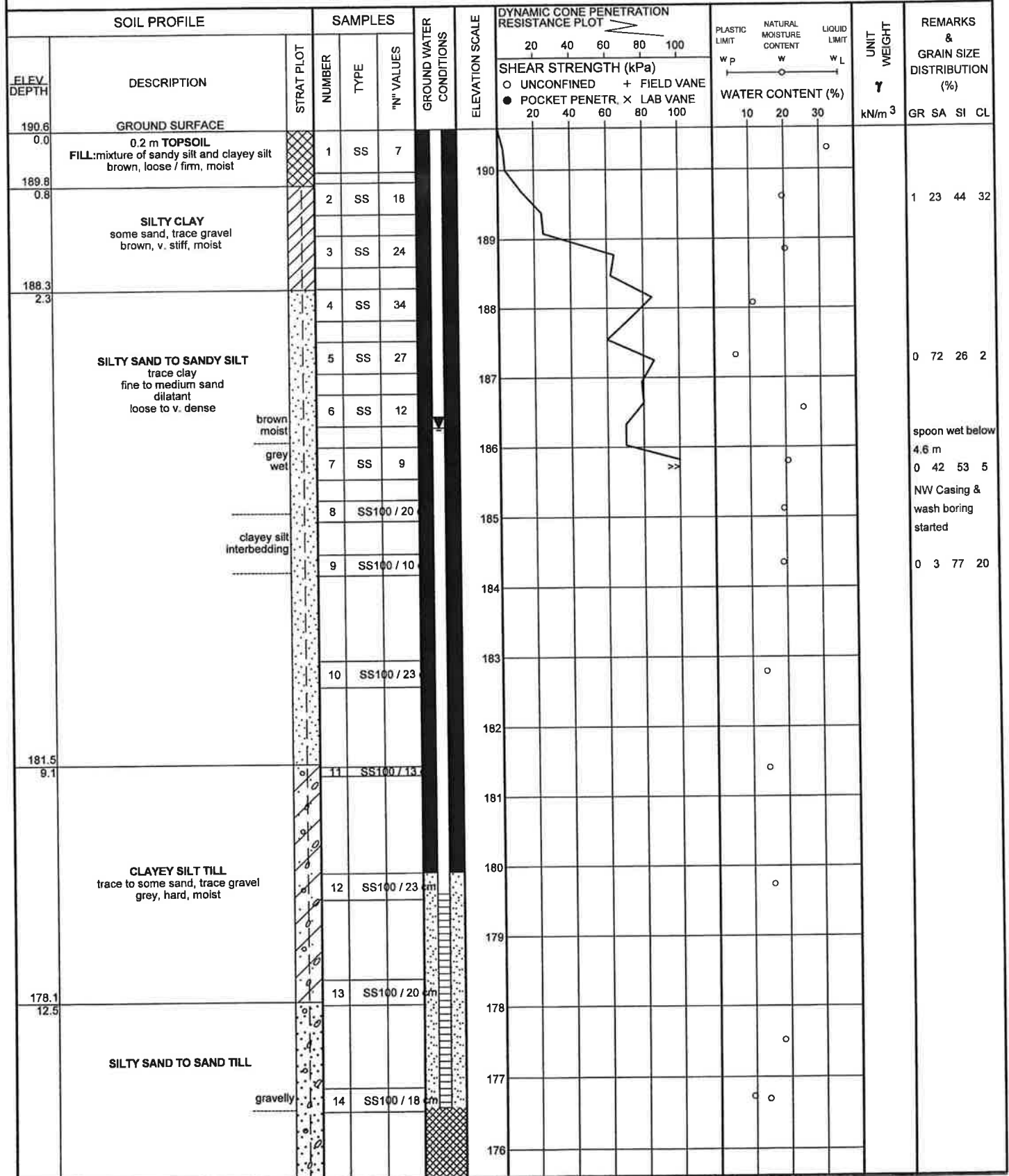
TRANETOB10757AA: Highway 404/7

RECORD OF BOREHOLE No B103

1 OF 2

METRIC

GWP G.W.P LOCATION 404 E-N On Ramp-Station 0+207, 5 m Rt of C/L (631108.3 E, 4856295.4 N) ORIGINATED BY LG
 DIST HWY 404 BOREHOLE TYPE Hollow Stem Auger, NW Casing, Wash Boring, DCPT COMPILED BY SH
 DATUM Geodetic DATE 5/24/2011 5/27/2011 CHECKED BY ZO



Continued Next Page

+ 3, × 3 Numbers refer to
Sensitivity 20
15 5
10 (%) STRAIN AT FAILURE

TRANETOB10757AA: Highway 404/7

RECORD OF BOREHOLE No B103

2 OF 2

METRIC

GWP G.W.P LOCATION 404 E-N On Ramp-Station 0+207, 5 m Rt of C/L (631108.3 E, 4856295.4 N) ORIGINATED BY LG
 DIST HWY 404 BOREHOLE TYPE Hollow Stem Auger, NW Casing, Wash Boring, DCPT COMPILED BY SH
 DATUM Geodetic DATE 5/24/2011 5/27/2011 CHECKED BY ZO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT		UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH (kPa)		WATER CONTENT (%)				
								○ UNCONFINED	+ FIELD VANE	W _P	W			W _L
								● POCKET PENETR.	× LAB VANE					
175.6							20 40 60 80 100							
172.1 18.5	SILTY SAND TO SAND TILL trace to some gravel, trace clay grey, dense to v. dense, wet		15	SS	47									
			16	SS100 / 28	60									
	CLAYEY SILT TILL trace to some sand, trace gravel grey, hard, moist		17	SS	105									
			18	SS100 / 20	60									
			19	SS100 / 20	60									
164.3 26.3			20	SS100 / 15	60									
			21	SS100 / 13	60									
			22	SS100 / 20	60									
End of borehole Hole caved-in @ 4.4 m upon completion DCPT performed 3.4 m away from borehole from ground to 4.9 m Monitoring well installed to 14.0 m Water Level Records : May 27, 2011 4.3 m June 08, 2011 4.3 m June 21, 2011 4.4 m June 22, 2011 4.4 m June 30, 2011 4.3 m														

TRANETOB10757AA: Highway 404/7

RECORD OF BOREHOLE No B104

1 of 1

METRIC

GWP G.W.P. LOCATION 404 E-N On Ramp-Station 0+192, 6.4 m Lt of C/L (631123.2 E, 4856281.7 N) ORIGINATED BY LG
DIST HWY 404 BOREHOLE TYPE Hollow Stem Auger, NW Casing, Wash boring, DCPT COMPILED BY AS
DATUM Geodetic DATE 5/24/2011 CHECKED BY ZO

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			20 40 60 80 100	20 40 60 80 100					
189.3	GROUND SURFACE													
0.0	0.2 m TOPSOIL		1	SS	5		189							
188.8	FILL: mixture of clayey silt, topsoil and sandy silt, firm		2	SS	15		188							
0.5			3	SS	23		187							
	SILTY SAND TO SANDY SILT		4	SS	33		186							36 52 (12)
	trace to some gravel		5	SS	25		185							spoon wet below 3.8 m
	brown		6	SS	55		184							NW Casing & wash boring started
	moist		7	SS	66		183							
	wet, compact		8	SS	53		182							
	gravelly		9	SS	35		181							
	dense to v. dense		10	SS	65		180							
182.7			11	SS	43		179							
6.6			12	SS	91		178							0 74 21 5
	SILTY SAND TO SAND TILL		13	SS	26		177							
	trace gravel, trace clay grey, dense to v. dense, wet													
	brown													
	grey													
176.8														
12.5														
176.7	CLAYEY SILT TILL													
12.7	trace to some sand, trace gravel grey, v. stiff, moist													
	End of borehole @ 12.7 m Water level and caved-in @ 3.5 m (not stabilized)* upon completion DCPT performed from bottom of borehole to 13.3 m													

+ 3, X 3 Numbers refer to 20
Sensitivity 15-5 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No BH 3 (2007)

1 OF 2

METRIC

GWP G.W.P. LOCATION Hwy 404 E-N On Ramp Extension ORIGINATED BY GH
DIST HWY 404 BOREHOLE TYPE Solid Stem & N Casing Wash Boring COMPILED BY SK
DATUM Geodetic DATE 12/19/2007 1/11/2008 CHECKED BY RM

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 (%)
FLEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100					
189.2	GROUND SURFACE													
0.0	80 mm TOPSOIL		1	SS	16		189							
188.6	FILL: sandy silt to silty sand trace organics, brown, compact, moist to wet													
0.6			2	SS	36		188							
	SILTY SAND TO SANDY SILT trace to some gravel brown, moist to wet		3	SS	23		187							
			4	SS	38		186							
	wet		5	SS	20		185							
	compact to dense		6	SS	97*		184							
	very dense		7	SS	55		183							
184.4	rock fragment @ spoon tip		8	SS	105		182							
4.8			9	SS	91		181							
	SILTY SAND TO SAND TILL trace to some gravel brown, v. dense, wet		10	SS	152		180							
			11	SS	106		179							
179.0			12	SS	53		178							
10.2			13	SS	60		177							
	CLAYEY SILT TILL heterogeneous mixture of silt and clay trace sand, trace gravel grey, v. stiff to hard, moist		14	SS	29		176							
175.0							175							
14.2	End of Borehole Borehole caved-in @ 4.6 m - Dec 19, 2007													

Continued Next Page

+ 3, x 3

Numbers refer to
Sensitivity

20
15
10

(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No BH 3 (2007)

2 OF 2

METRIC

GWP G.W.P LOCATION Hwy 404 E-N On Ramp Extension ORIGINATED BY GH
 DIST HWY 404 BOREHOLE TYPE Solid Stem & N Casing Wash Boring COMPILED BY SK
 DATUM Geodetic DATE 12/19/2007 1/11/2008 CHECKED BY RM

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 (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100					
174.2							174										
173.3																	
15.9	End of DCPT DCPT from 14.2 m to 16.0 m																

+ 3 . x 3

Numbers refer to
Sensitivity

20
15
10

(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No BH 4 (2007)

1 OF 2

METRIC

GWP G.W.P LOCATION Hwy 404 E-N On Ramp Extension ORIGINATED BY GH
 DIST HWY 404 BOREHOLE TYPE Solid Stem & N Casing Wash Boring COMPILED BY SK
 DATUM Geodetic DATE 12/19/2007 1/14/2008 CHECKED BY RM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100	20 40 60 80 100		
190.7 0.0	GROUND SURFACE											
	0.1 m TOPSOIL		1	SS	20		190					
189.5 1.2	FILL: sandy silt to silty sand trace of organics, trace gravel, trace clay dark brown, oxidized, compact, moist		2	SS	10							
	SILTY CLAY some gravel, trace sand mottled brown, hard, moist		3	SS	47		189					
	silt seam		4	SS	40		188					
187.9 2.8			5	SS	56		187					1 23 63 13
			6	SS	37		186					0 85 (15) N casing wash boring was performed from 4.6 m
	moist		7	SS	25		185					0 92 (8)
	wet		8	SS	28		184					0 97 (3)
	sand getting coarser		9	SS	26		183					
	SILTY SAND to SANDY SILT brown		10	SS	20		182					
			11	SS	26		181					
	compact to v. dense						180					
	v. dense						179					
	tr. gravel		12	SS	54		178					
180.6 10.1			13	SS	100/14 cm		177					No recovery
	SILTY SAND to SAND TILL trace to some gravel, occ. cobbles v. dense, wet		14	SS	100/13 cm		176					
	brown											
	grey, some silt		15	SS	100/13 cm							0 82 (18)

Continued Next Page

+ ³ , × ³ Numbers refer to
Sensitivity

20
15
10

(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No BH 4 (2007)

2 OF 2

METRIC

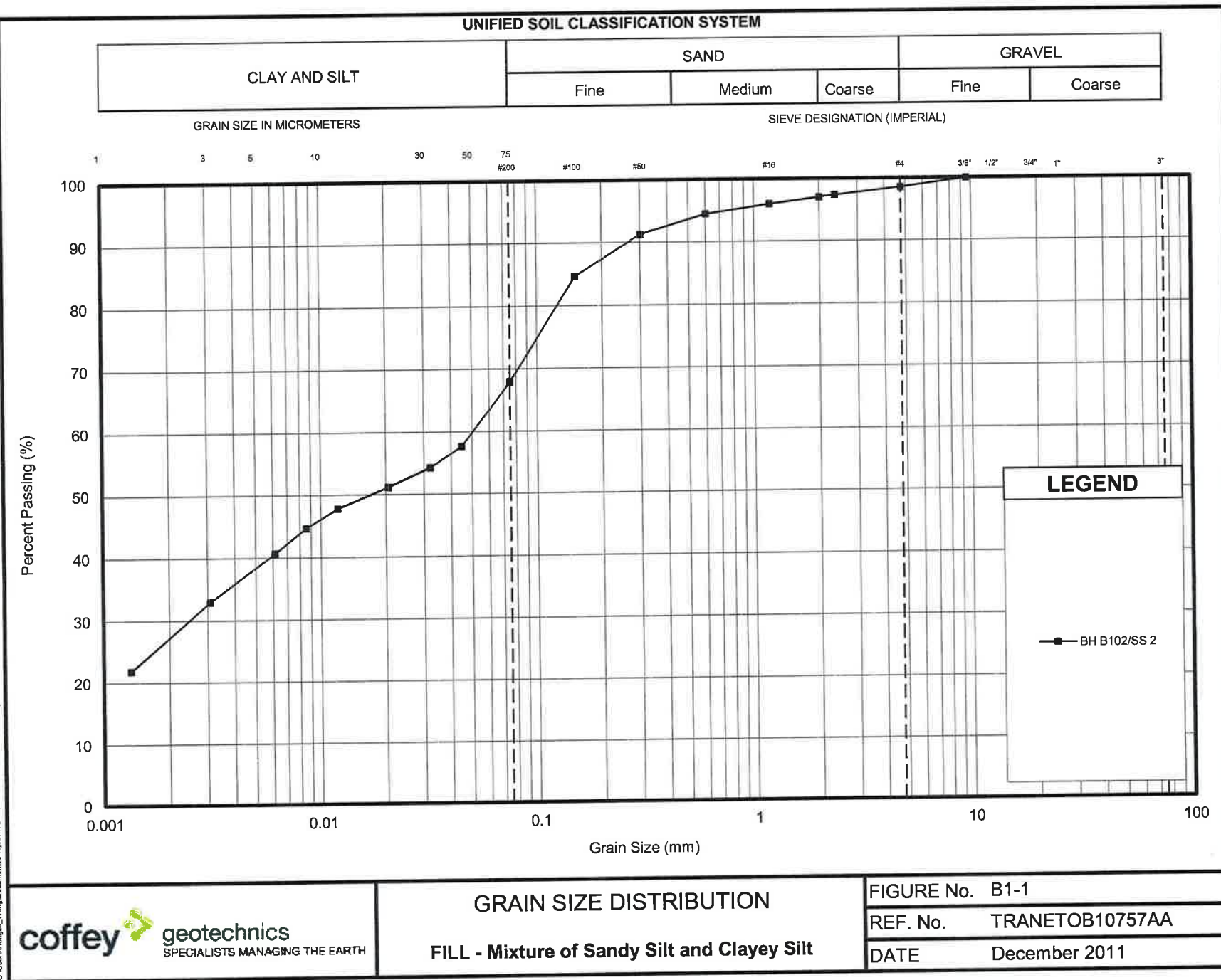
GWP G.W.P LOCATION Hwy 404 E-N On Ramp Extension ORIGINATED BY GH
 DIST HWY 404 BOREHOLE TYPE Solid Stem & N Casing Wash Boring COMPILED BY SK
 DATUM Geodetic DATE 12/19/2007 1/14/2008 CHECKED BY RM

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 (%) GR SA SI CL
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH (kPa) ○ UNCONFINED + FIELD VANE ● POCKET PENETR. × LAB VANE							
175.7								20 40 60 80 100							
175.4								20 40 60 80 100							
15.3	End of Borehole Borehole caved-in @ 3.1 m - Dec19,2007		1B	SS 100/100mm			175								
174.1															
16.6	End of DCPT DCPT from 15.6 to 16.6 m														

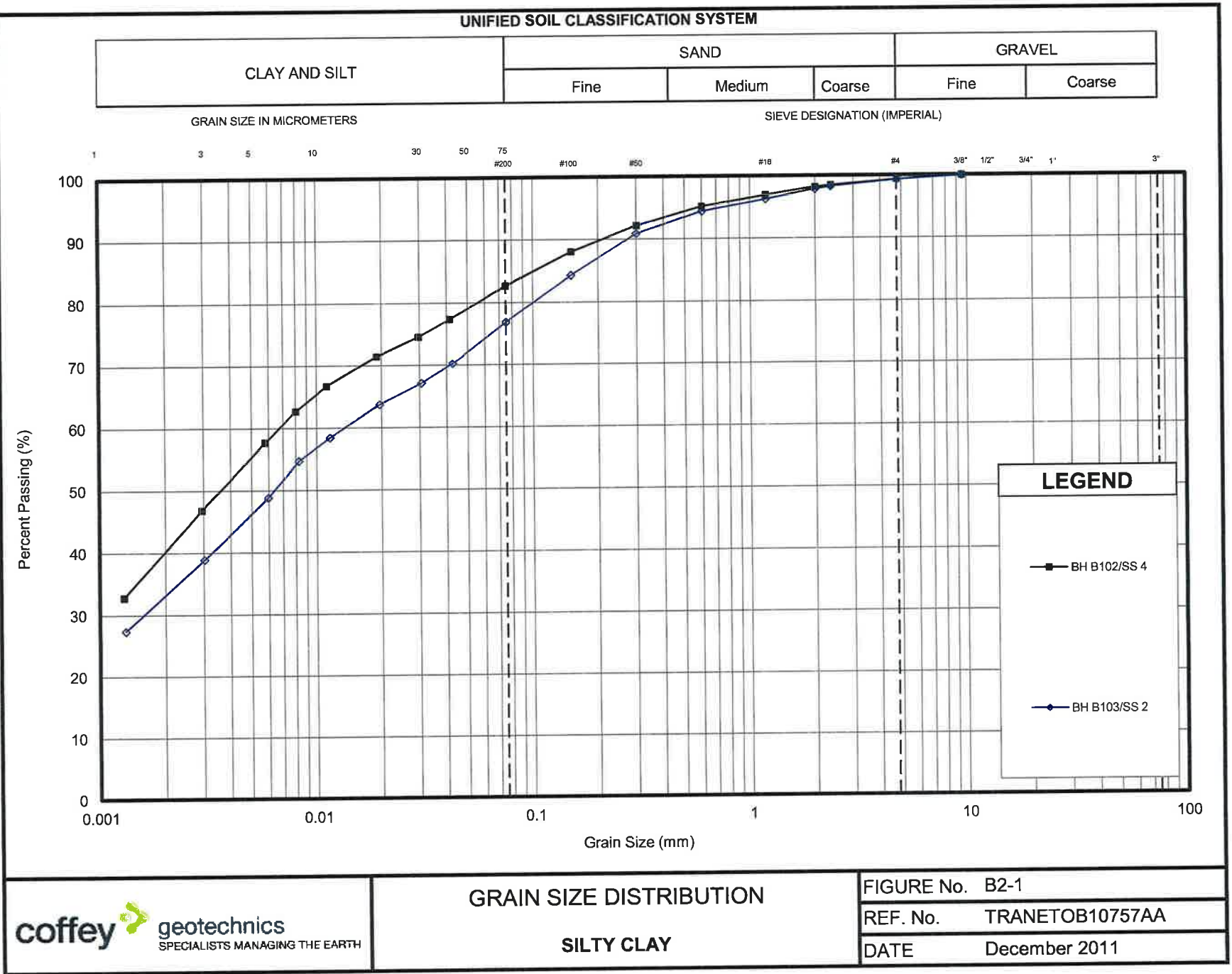
+³, ×³: Numbers refer to
Sensitivity

20
15
10
5
(%) STRAIN AT FAILURE

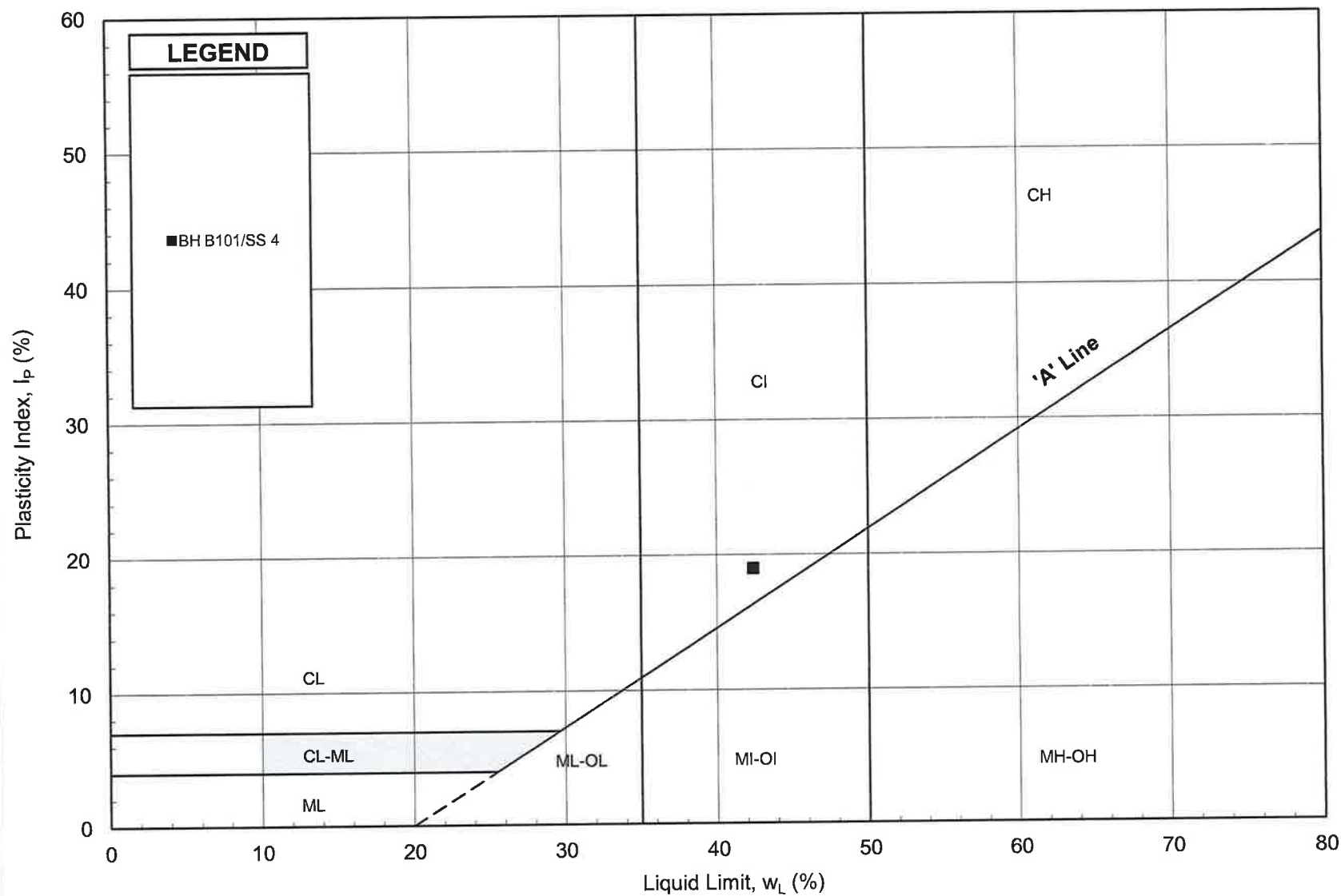
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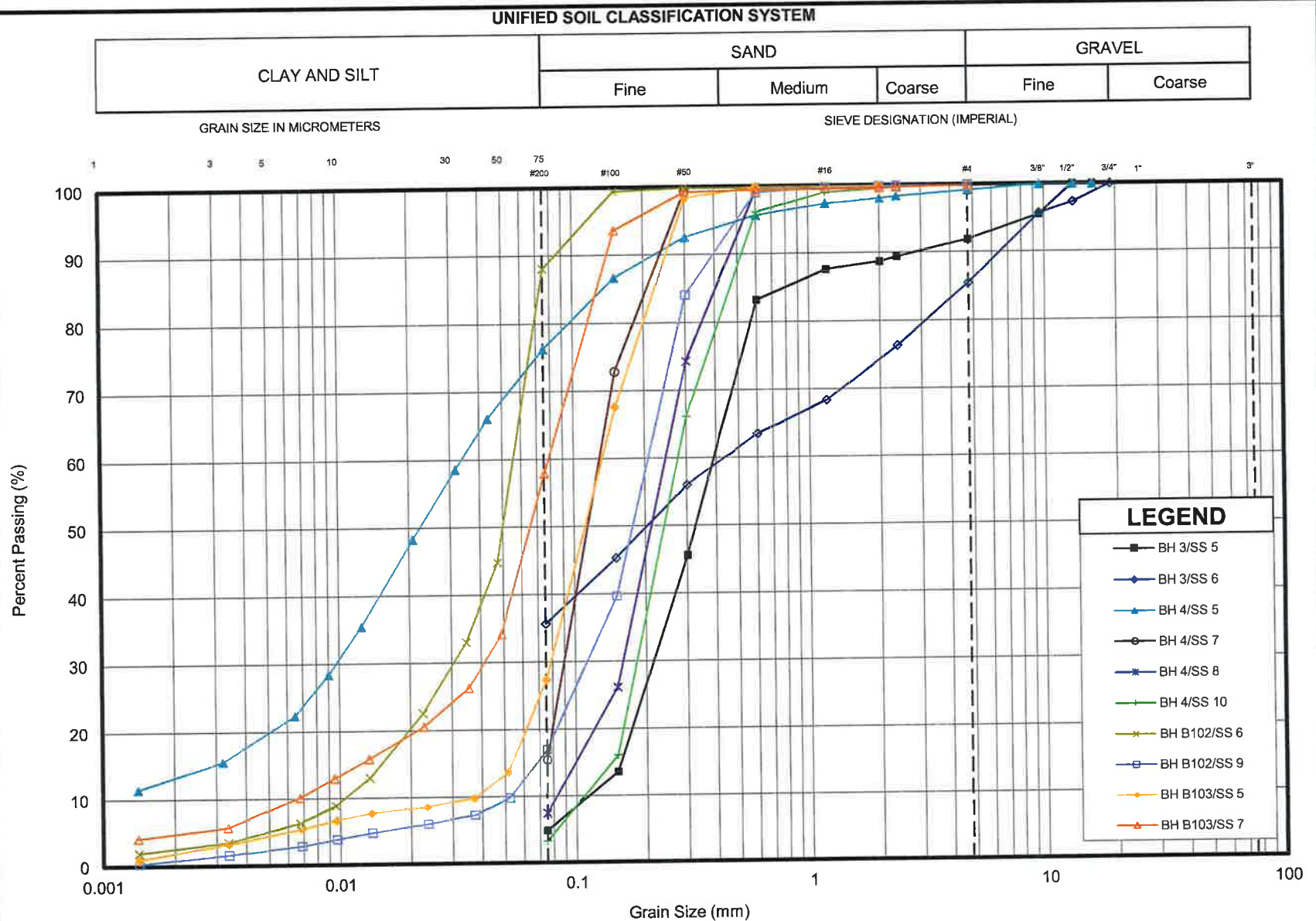
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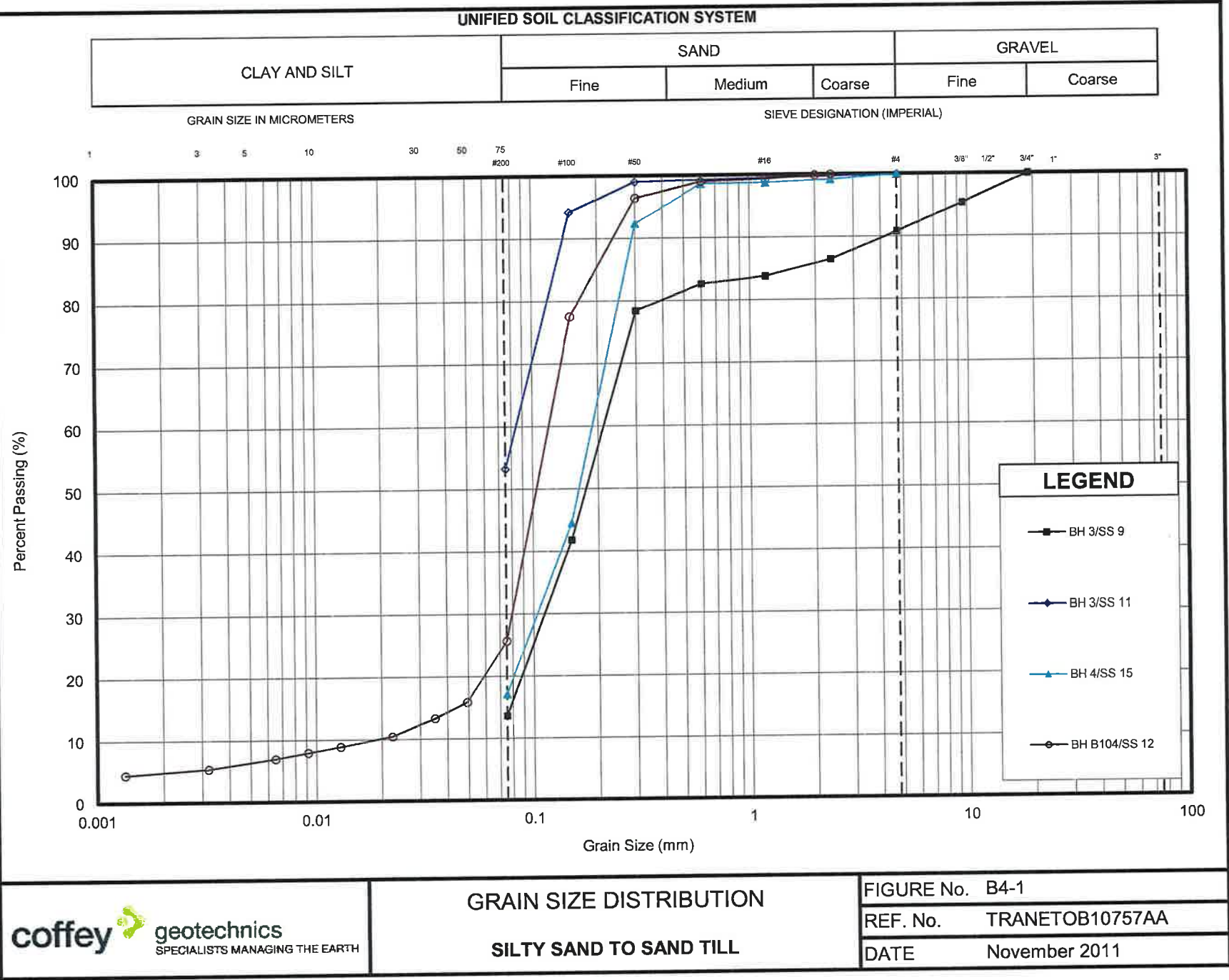
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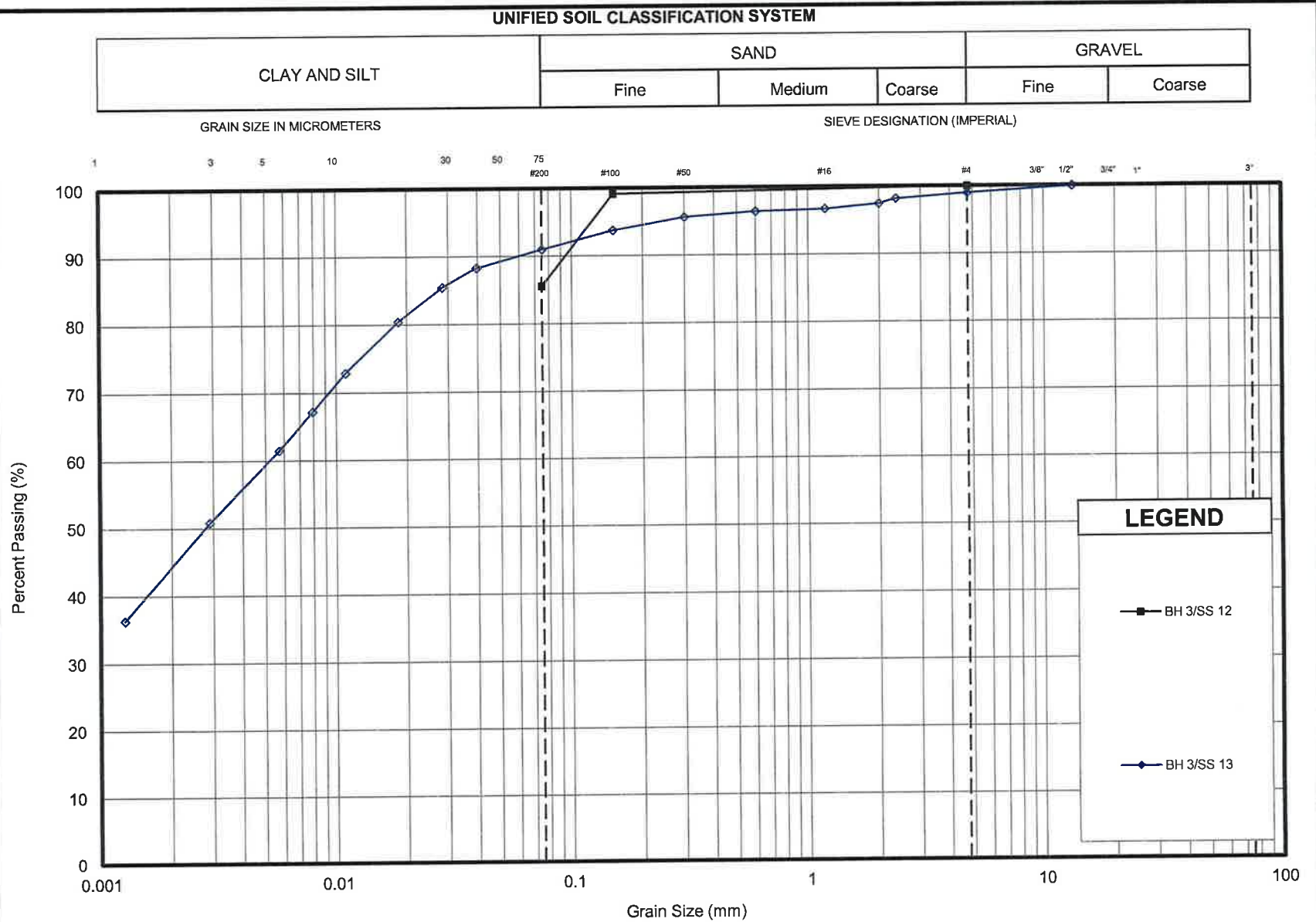


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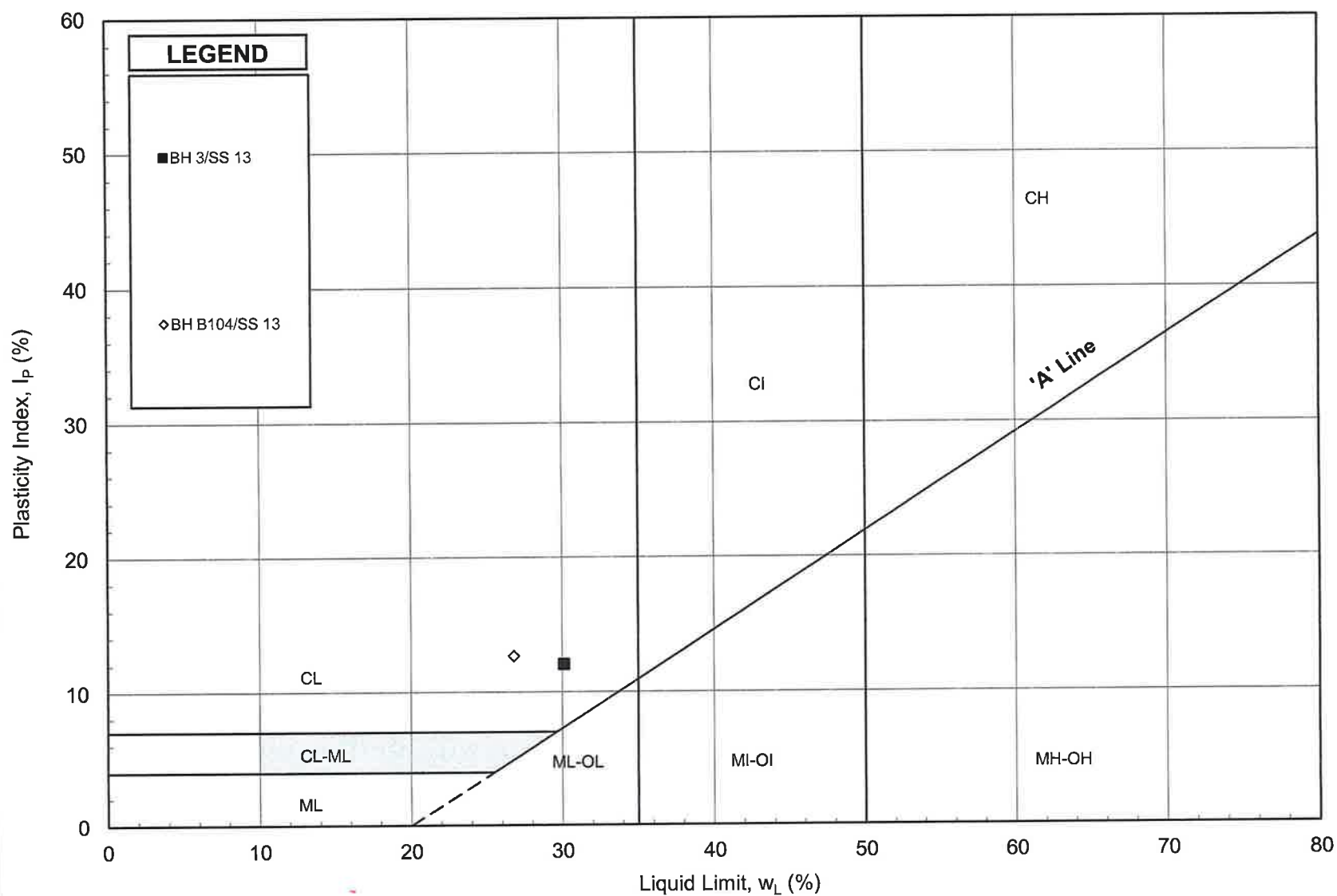


C:\Users\Hongliu.Wang\Documents\Project\TRANETOB10757AA\URS 44-7B\docs on E-N On Ramp\TRANETOB10757AA Grain Size Distribution.xls





C:\Users\hongkui_wang\Documents\Project\TRANETOB10757AA\URS 404-76\ridge on E-N On Ramp\TRANETOB10757AA Grain Size Distribution.xls





APPENDIX D

Non-Standard Special Provision (NSSP)

OBSTRUCTIONS - Item No.

Non-Standard Special Provision

The presence of cobbles and boulders within the clayey silt, sandy silt to sand and sandy silt to sand till deposits is inferred from auger grinding and resistance to split-spoon driving encountered during the subsurface investigation. Consideration of the presence of these obstructions must be made in the selection of appropriate equipment and procedures for sub-excavation through this material for foundation construction and/or pile driving.

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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Asia	+ 86 21 6258 5522
Australasia	+ 61 3 8862 3500
Europe	+ 356 21 42 30 20
North America	+ 1 800 275 3281
South America	+ 55 21 3095 9500

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