



April 28, 2016

FOUNDATION INVESTIGATION AND DESIGN REPORT

AVENUE ROAD UNDERPASS HIGHWAY 401 EASTBOUND COLLECTOR REHABILITATION FROM JANE STREET TO AVENUE ROAD TORONTO, ONTARIO

G.W.P. 2087-13-00, AGREEMENT NO. 2009-E-0011

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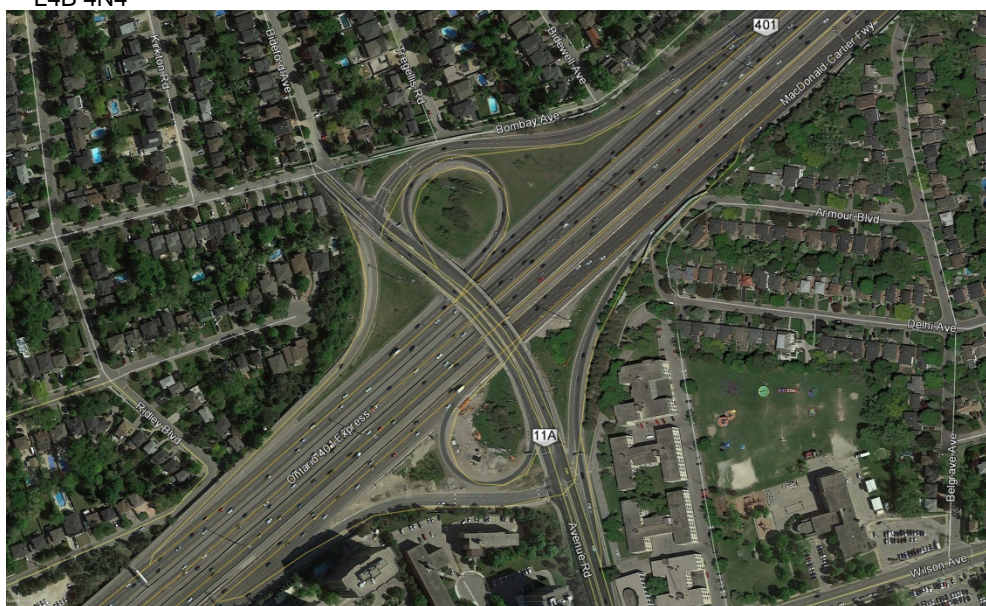


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REPORT





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PART A

**FOUNDATION INVESTIGATION REPORT
AVENUE ROAD UNDERPASS
HIGHWAY 401 EBC REHABILITATION FROM
JANE STREET TO AVENUE ROAD
TORONTO, ONTARIO
G.W.P. 2087-13-00, AGREEMENT NO. 2009-E-0011**



1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by AECOM on behalf of Ministry of Transportation, Ontario (MTO) to provide foundation engineering services in support of the rehabilitation of the Highway 401 eastbound collector lanes (EBC) between Jane Street and Avenue Road in Toronto, Ontario. Foundation engineering services are required under two phases:

- Phase 1: Foundation Engineering Assessment, in the form of a desktop study, of existing foundations of various structures including the Avenue Road Underpass.
- Phase 2: Detail Foundation Investigation at various bridge structures, including the Avenue Road Underpass.

This report addresses the Phase 2 Detail Foundation Investigation for the proposed Avenue Road Underpass.

The terms of reference and scope of work for the foundation investigation are outlined in MTO's Request for Proposal (RFP) for Agreement No. 2009-E-0011, issued on December 16, 2009 and MTO's revised Terms of Reference in Addenda dated May 2013. The scope of work for the foundation engineering services is presented in Golder's scope change letter, dated December 22, 2014.

Subsurface information from previous investigations associated with the Avenue Road Underpass structure was obtained from the MTO Geocres library and AECOM as follows:

- MTO GEOCREs No. 30M11-073: Report titled "Avenue Road Underpass, Hwy. #401, Twp. Of North York, Cty. Of York. District No. 6, W.J. 61-F-91 - - W.P. 193-58," prepared by the Department of Highways – Ontario, Materials and Research Section, dated November 7, 1961.
- Design Drawings by the Department of Highways Ontario –Bridge Division, titled "Avenue Road Underpass – Hwy No. 401", T.W.P. 72-202-1-E, dated June 1962; provided by AECOM.
- Design Drawings by CS Cole Sherman, titled "Hwy 401 Avenue Road Underpass Rehabilitation," dated February 1990, provided by AECOM.

2.0 SITE DESCRIPTION

The existing Highway 401-Avenue Road Underpass is a three span structure supported on shallow foundations. The bridge structure is approximately 104 m long and 26 m wide. Based on the 1962 design drawings, the natural ground surface at this site varies from approximately Elevation 179 m to 181 m, with Highway 401 constructed at a grade of approximately Elevation 179 m at the Avenue Road Underpass structure location. The Avenue Road grade is approximately Elevation 181 m to 182 m.

3.0 INVESTIGATION PROCEDURES

The field work for this subsurface investigation was carried out between September 23 and October 8, 2015, at which time six boreholes (designated AR15-1 to AR15-3/3A, AR15-4 to AR15-6) were advanced using a CME 75 Truck mount drill rig, supplied and operated by Geo-Environmental Drilling Inc. of Halton Hills, Ontario. As auger refusal was encountered at a depth of 1.9 m below ground surface in Borehole AR15-3, a second borehole



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(Borehole AR15-3A) was advanced 1.2 m north of the original location in order to achieve desired drill depth. Four boreholes (designated 73-1 to 73-4) advanced as part of the previous investigation at the Avenue Road site and used in this report have been renumbered to show the MTO GEOCRES reference number followed by the original borehole designation (i.e., 73-1). Boreholes AR15-1 to AR15-6 and 73-1 to 73-4 pertain to the existing underpass bridge structure and proposed new bridge structure, were advanced at the locations shown on Drawing 1.

Boreholes AR15-1 and AR15-6 were advanced using 215 mm outside diameter hollow stem augers, Boreholes AR15-2 to AR15-4 were advanced using 165 mm outside diameter hollow stem augers and Borehole AR15-5 was advanced using 110 mm outside diameter hollow stem augers. Soil samples were obtained at 0.75 m and 1.5 m intervals of depth in the boreholes, using a 50 mm outside diameter split-spoon sampler in accordance with the Standard Penetration Test (SPT) procedure (ASTM D1586) driven by an automatic hammer. The in situ test results presented in the borehole records are uncorrected.

The groundwater conditions were observed in the open boreholes during and immediately following the drilling operations. Standpipe piezometers were installed in Boreholes AR15-1 and AR15-6 to permit monitoring the groundwater level at the site. The piezometer consists of a 50 mm diameter, 3.0 m long PVC slotted screen installed within a filter sand pack, above which the borehole annulus was backfilled to ground surface with bentonite pellets. The details of the piezometer installation are shown on the Record of Boreholes AR15-1 and AR15-6. The remaining boreholes were backfilled to immediately below ground surface with bentonite pellets upon completion, in accordance with Ontario Regulation 903 (as amended).

The field work was observed on a full-time basis by a member of Golder's technical staff who located the boreholes in the field, arranged for the clearance of underground utilities, directed the drilling, sampling and in situ testing operations, and logged the boreholes. The soil samples were identified in the field, placed in labelled containers and transported to Golder's laboratory in Mississauga for further examination and laboratory testing. Index and classification tests (water contents, Atterberg limits and grain size distributions) were carried out on selected soil samples. All geotechnical laboratory testing was completed to ASTM and MTO LS standards, as applicable.

The borehole locations were measured on-site relative to the existing bridge and site features and the ground surface elevations were obtained from the Digital Terrain Model for the site, provided by AECOM. The borehole locations, including MTM NAD83 northing and easting coordinates, the ground surface elevations referenced to Geodetic datum and the drilled depths are summarized below and are shown on Drawing 1. Also presented below and shown on Drawing 1 is the location of the boreholes advanced as part of the previous investigation at the site (GEOCRES No 30M11-073).



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Borehole No.	MTM NAD83 Northing	MTM NAD83 Easting	Ground Surface Elevation	Borehole Depth
AR15-1	4,844,765.0	310,949.1	184.5 m	15.9 m
AR15-2	4,844,774.7	310,984.8	178.1 m	23.4 m
AR15-3	4,844,735.6	310,972.2	178.5 m	1.9 m
AR15-3A	4,844,736.3	310,971.5	178.5 m	24.6 m
AR15-4	4,844,723.2	311,027.2	178.5 m	29.2 m
AR15-5	4,844,684.2	311,007.7	178.5 m	28.0 m
AR15-6	4,844,689.4	311,040.5	185.2 m	15.7 m
73-1*	4,844,750.0	310,957.7	184.1 m	8.9 m
73-2*	4,844,754.0	311,000.2	177.6 m	4.3 m
73-3*	4,844,708.0	310,997.8	184.5 m	8.8 m
73-4*	4,844,710.0	311,036.2	180.0 m	4.4 m

* Approximate borehole locations obtained from the Digital Terrain Model as plotted relative to centerline of Highway 401 and the existing Avenue Road Underpass.

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

This section of Highway 401 is located within the physiographic region known as the Peel Plain, according to *The Physiography of Southern Ontario* (Chapman and Putnam, 1984)¹.

A surficial till sheet, which generally follows the surface topography, is generally present throughout much of this area. The till is typically comprised of clayey silt to silty clay, with occasional sand to silt zones and is mapped in this area as the Halton Till. Shallow, localized deposits of loose sand and silt and/or soft clay can overlie this uppermost till sheet, and these represent relatively recent deposits, formed in small glacial melt water ponds scattered throughout the Peel Plain and concentrated near river valleys, such as the West Don River valley. 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

As part of the current subsurface investigation, six boreholes (Boreholes AR15-1 to AR15-6) were advanced at the existing Avenue Road structure site. The borehole locations, ground surface elevations and interpreted stratigraphic conditions are shown on Drawings 1 and 2.

The detailed subsurface soil and groundwater conditions encountered in the boreholes advanced as part of the current investigation and the results of in situ and laboratory testing are given on the borehole records contained

¹ Chapman, L.J. and Putman, D.F., 1984. *The Physiography of Southern Ontario*, Ontario Geological Society, Special Volume 2, Third Edition. Accompanied by Map p. 2715, Scale 1:600,000.



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in Appendix A; the results of geotechnical laboratory testing are also presented on Figures B1 to B7 contained in Appendix B. The borehole information from the previous (MTO) investigation is presented in Appendix C.

The stratigraphic boundaries shown on the borehole records and on the interpreted stratigraphic profile and cross-sections on Drawings 1 and 2 are inferred observations of drilling progress and from 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 surficial layers of asphalt and varying thickness of fill (an upper layer of granular materials and a lower layer of cohesive material), underlain by a succession of very stiff to hard/compact to very dense glacial till deposits, which vary in composition from clayey silt to silty sand and contain interstitial sand deposits varying in thickness.

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

4.2.1 Asphalt and Topsoil

A layer of asphalt was encountered immediately below the ground surface in all boreholes from the current investigation, with the exception of Borehole AR15-2, which was advanced away from the paved roadway. The asphalt layer is approximately 150 mm thick in Boreholes AR15-1 and AR15-6, and approximately 200 mm thick in Boreholes AR15-3 to AR15-5.

4.2.2 Fill

A layer of fill was encountered below the asphalt or immediately below the ground surface in all boreholes from the current investigation (Boreholes AR15-1 to AR15-6). The extent and composition of the fill varies depending on the location of the boreholes, with thicker granular and cohesive layers encountered at the approach embankments (Boreholes AR15-1 and AR15-6), thinner granular and cohesive layers encountered in Boreholes AR15-2 and AR15-3, and thinner granular layers encountered in Boreholes AR15-4 and AR15-5. The surface of the fill layer was encountered between Elevation 185.0 m and 171.8 m.

The granular fill layer encountered in all boreholes ranges in thickness from 0.5 m to 1.0 m thick, and generally consists of sand, some gravel, trace to some silt to sand and gravel, trace to some silt. The cohesive fill was encountered underlying the granular fill in Boreholes AR15-1 to AR15-3 and AR15-6. In Boreholes AR15-1 and AR15-6, this layer is 3.6 m and 6.0 m thick, respectively, while the cohesive fill layer encountered in Boreholes AR15-2 and AR15-3 is 1.3 m and 0.8 m thick, respectively. A 250 mm thick layer of asphalt was encountered within the cohesive fill in Borehole AR15-6 at a depth of 3.3 m. Cohesive fill was encountered in Boreholes 74-1 and 74-3 immediately below ground surface to depths of 3.2 m and 4.7 m, respectively. The cohesive fill consists of clayey silt, trace sand to with sand, trace to some gravel. The cohesive fill also contains trace organics and an organic odour was present within the samples in Boreholes AR15-1, AR15-2, and AR15-6. An approximately 0.8 m thick layer of fill also was encountered in Borehole 74-2, but its composition is not classified in the borehole records.

The measured Standard Penetration Test (SPT) "N"-values within the granular fill range from 12 to 43 blows per 0.3 m of penetration, indicating a compact to dense relative density. The measured SPT "N"-values within the cohesive fill range from 5 to 32 blows per 0.3 m of penetration, suggesting a firm to hard consistency.



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The results of grain size distribution tests completed on two selected samples of the cohesive fill deposit from the current investigation and are shown on Figure B1 in Appendix B.

Atterberg limits tests were carried out on three selected samples of the cohesive fill and measured plastic limits ranging between 11 per cent and 15 per cent, liquid limits ranging between 17 per cent and 29 per cent, and corresponding plasticity indices ranging between 5 per cent and 14 per cent. These test results, which are plotted on Figure B2 in Appendix B, confirm that the fill consist of a clayey silt of low plasticity.

The laboratory water content measured on selected samples of the cohesive fill ranges from 9 per cent to 18 per cent, and the laboratory water content measured one selected sample of the granular fill is 9 per cent.

4.2.3 Silt and Sand to Silty Sand Till

A till deposit comprised of silt and sand to silty sand was encountered underlying the fill in Boreholes AR15-5 and AR15-6, underlying a clayey silt deposit (described below) in Borehole AR15-4 and interbedded with a sand deposit (described below) in Borehole AR15-2. The elevations of the surface and base of the till deposit and the thickness of this stratum as encountered in the boreholes are summarized below.

Borehole No.	Deposit Surface Depth (m)	Deposit Surface Elevation (m)	Deposit Thickness (m)	Deposit Base Elevation (m)	Deposit Description
AR15-2	5.6	172.5	3.8	168.7	Silt and Sand
	11.0	167.1	1.2	165.9	Silt and Sand
AR15-4	2.2	176.3	3.4	172.9	Silt and Sand
AR15-5	0.8	177.7	6.4	171.3	Silt and Sand
AR15-6	7.2	178.0	6.1	171.9	Silt and Sand to Silty Sand

The measured SPT “N” values within the silt and sand to silty sand till deposit range from 25 blows per 0.3 m of penetration to 188 per 0.28 m of penetration, indicating a compact to very dense relative density.

Grain size distribution testing was completed on eight selected samples of the silt and sand to silty sand till deposit and the results are shown on Figures B3A and B3B in Appendix B.

The natural water content measured on selected samples of the till deposit ranges from 5 per cent to 11 per cent.

4.2.4 Clayey Silt to Clayey Silt with Sand Till

A cohesive till deposit comprised of clayey silt to clayey silt with sand was encountered in all boreholes with the exception of Boreholes AR15-6 and 73-1 to 73-4. This till deposit was encountered underlying or interbedded with deposits of sand, silty clay, clay, and silt and sand till. The elevations of the surface and base of the native till deposit and the thickness of this stratum as encountered in the boreholes are summarized below.



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Borehole No.	Deposit Surface Depth (m)	Deposit Surface Elevation (m)	Deposit Thickness (m)	Deposit Base Elevation (m)	Deposit Description
AR15-1	4.7	179.8	4.0	175.8	Clayey Silt with Sand
	11.0	173.5	1.2	172.3	Clayey Silt
AR15-2	14.8	163.3	6.8	156.5	Clayey Silt
AR15-3	18.6	159.9	4.3	155.6	Clayey Silt
AR15-4	0.7	177.8	1.5	176.3	Clayey Silt with Sand
	15.3	163.2	4.8	158.4	Clayey Silt
AR15-5	17.1	161.4	10.8	150.6	Clayey Silt

The measured SPT “N”-values within the cohesive fill deposit range from 21 blows to 140 blows per 0.3 m of penetration, suggesting a very stiff to hard consistency.

The results of grain size distribution tests completed on two selected samples of the clayey silt till deposit from the current investigation are shown on Figure B4 in Appendix B.

Atterberg limits testing was carried out on seven selected samples of the cohesive till and measured plastic limits ranging between 10 per cent and 16 per cent, liquid limits ranging between 16 per cent and 34 per cent, and plasticity indices ranging between 5 per cent and 18 per cent. These test results, which are plotted on Figure B5 in Appendix B, indicate that the till consists of clayey silt of low plasticity.

The natural water content measured on selected samples of the till range from 7 per cent to 16 per cent.

4.2.5 Clayey Silt to Silty Clay

A clayey silt to silty clay deposit was encountered in all boreholes with the exception of Borehole AR15-2, AR15-6, 73-2 and 73-4. Borehole AR15-1 was terminated with the clayey silt deposit after penetrating into it for a thickness of 3.2 m. The elevations of the surface and base of the clayey silt to silty clay deposit and the thickness of this stratum as encountered in the boreholes are summarized below.

Borehole No.	Deposit Surface Depth (m)	Deposit Surface Elevation (m)	Deposit Thickness (m)	Deposit Base Elevation (m)	Deposit Description
AR15-1	12.7	171.8	>3.2	Below 168.6	Clayey Silt
AR15-3A	15.1	163.4	3.5	159.9	Silty Clay
AR15-4	5.6	172.9	3.1	169.8	Sandy Clayey Silt
	20.8	157.7	6.6	151.1	Clayey Silt
AR15-5	7.2	171.3	4.5	166.8	Silty Clay
	15.4	163.1	1.7	161.4	Clayey Silt
73-1	3.4	180.8	1.5	179.3	Clayey Silt with Sand



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Borehole No.	Deposit Surface Depth (m)	Deposit Surface Elevation (m)	Deposit Thickness (m)	Deposit Base Elevation (m)	Deposit Description
73-3	4.9	179.6	2.0	177.6	Clayey Silt with Sand

The measured SPT “N” values within the clayey silt to silty clay deposits range from 20 blows per 0.3 m of penetration to 102 blows per 0.2 m of penetration, suggesting a very stiff to hard consistency.

Grain size distribution testing was completed on one selected sample of the silty clay deposit and the result is shown on Figure B6 in Appendix B.

Atterberg limits testing was carried out on six selected samples of the clayey silt deposit and measured plastic limits ranging between 12 per cent and 18 per cent, liquid limits ranging between 18 per cent and 48 per cent, and plasticity indices ranging between 5 per cent and 29 per cent. These test results, which are plotted on Figure B7 in Appendix B, confirm that the deposit consists of clayey silt of low plasticity to silty clay of intermediate plasticity.

The natural water content measured on selected samples of the clayey silt to silty clay deposit ranges from 11 per cent to 23 per cent, although the natural water content measured on the clayey silt portion of the deposit is generally below 15 per cent.

4.2.6 Clay

A deposit of clay trace sand and some sand pockets was encountered in Boreholes AR15-3A and AR15-4. The clay deposit was encountered at depths of 8.6 m and 9.9 m below ground surface in Boreholes AR15-3A and AR15-4, respectively, corresponding to Elevations 169.9 m and 168.6 m. The deposit has a thickness of 1.6 m and 1.8 m in Boreholes AR15-3A and AR15-4, respectively.

The measured SPT “N” values within the clay deposit are 32 blows and 31 blows per 0.3 m of penetration, suggesting a hard consistency.

Atterberg limits testing was carried out on two selected samples of the clay deposit and measured plastic limits of 20 per cent and 23 per cent, liquid limits of 53 per cent and 63 per cent, and corresponding plasticity indices of 33 per cent and 40 per cent. These test results, which are plotted on Figure B8 in Appendix B, confirm that the deposit consists of clay of high plasticity.

The natural water content measured on two selected samples of the clay was 24 per cent and 27 per cent.

4.2.7 Silt to Sand

A deposit of interlayered silt to sandy silt to silt and sand to silty sand to sand was encountered in all boreholes. All boreholes terminated within this deposit, with the exception of Boreholes AR15-1. The elevations of the surface and base of the silt to sand deposit and the thickness of this stratum as encountered in the boreholes are summarized below.



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Borehole No.	Deposit Surface Depth (m)	Deposit Surface Elevation (m)	Deposit Thickness (m)	Deposit Base Elevation (m)	Deposit Description
AR15-1	8.7	175.8	2.3	173.5	Sand
	12.2	172.3	0.5	171.8	Sand
AR15-2	2.0	176.1	3.6	172.5	Sand
	9.4	168.7	1.6	167.1	Sand
	12.2	165.9	2.6	163.3	Sand
	21.6	156.5	>1.8	Below 154.7	Silt
AR15-3A	1.8	176.7	6.8	169.9	Silt and Sand to Silty Sand
	10.2	168.3	4.9	163.4	Sand
	22.9	155.6	>1.7	Below 153.9	Silt
AR15-4	8.7	169.8	1.2	168.6	Silty Sand
	11.7	166.8	3.6	163.2	Sand
	20.1	158.4	0.7	157.7	Silty Sand
	27.4	151.1	>1.8	Below 149.3	Sandy Silt
AR15-5	11.7	166.8	3.7	163.1	Sand
	27.9	150.6	>0.1	Below 150.5	Silt
AR15-6	13.3	171.9	>2.4	Below 169.5	Silt
73-1	4.9	179.3	3.9	175.4	Sandy Silt
	8.7	175.4	>0.3	Below 175.1	Sand
73-2	0.9	176.7	>3.4	Below 173.3	Sand
73-3	6.9	177.6	1.6	176.0	Sandy Silt
	8.5	176.0	>0.3	Below 175.7	Sand
73-4	0	180.0	1.5	178.5	Sand
	1.5	178.5	>2.9	Below 175.6	Sand

The measured SPT “N” values within the sand interlayers range from 13 blows per 0.3 m of penetration to 182 blows per 0.25 m of penetration, indicating a compact to very dense relative density.

Grain size distribution testing was completed on twelve selected samples of the silt to sand deposit and the results are shown on Figure B9A and B9B in Appendix B.

An Atterberg limits test was carried out on one selected sample of the silt deposit and measured a plastic limit of 16 per cent, a liquid limit of 19 per cent, and a corresponding plasticity index of 4 per cent. These result, which is plotted on Figure B10 in Appendix B, confirm that the deposit consists of silt of slight plasticity.

The natural water content measured on selected samples of the sand interlayers/deposit ranges from 2 per cent to 16 per cent.



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4.2.8 Groundwater Conditions

The observed/recorded water levels in the open boreholes following completion of drilling and in the piezometer (P) in Borehole 2 are shown on the Record of Borehole sheets and are summarized below.

Borehole Number	Ground Surface Elevation	Depth to Water Level below Ground Surface	Groundwater Elevation (P: Piezometer)	Date
AR15-1	184.5 m	12.2 m	172.3 m (P)	Nov. 1, 2015
		12.2 m	172.3 m (P)	Dec. 14, 2015
AR15-2	178.1 m	14.7 m	163.4 m	Sep. 24, 2015
AR15-3A	178.5 m	*	-	-
AR15-4	178.5 m	*	-	-
AR15-5	178.5 m	dry	dry	Oct. 1, 2015
AR15-6	185.2 m	dry	dry	Oct. 8, 2015
		12.6 m	172.6 m (P)	Nov. 1, 2015
		12.7 m	172.5 m (P)	Dec. 14, 2015
73-1	184.1 m	*	-	-
73-2	177.6 m	*	-	-
73-3	184.5 m	*	-	-
73-4	180.0 m	*	-	-

* Water level not recorded.

The water levels presented above and on the Record of Borehole sheets may not represent stabilized groundwater conditions at the time of the investigation. 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

This Foundation Investigation Report was prepared by Ms. Nikol Kochmanová, P.Eng., a geotechnical engineer with Golder. Mr. Jorge Costa, P.Eng., a Designated MTO Contact and Principal of Golder, conducted an independent review and quality control audit of this report.

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**FOUNDATION REPORT
AVENUE ROAD UNDERPASS**

PART B

**FOUNDATION DESIGN REPORT
AVENUE ROAD UNDERPASS
HIGHWAY 401 EBC REHABILITATION FROM
JANE STREET TO AVENUE ROAD
TORONTO, ONTARIO
G.W.P. 2087-13-00, AGREEMENT NO. 2009-E-0011**



6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

6.1 General

This section of the report provides foundation engineering recommendations for the detail design of the proposed replacement of the existing Avenue Road Underpass as part of the Highway 401 EBC rehabilitation from Jane Street to Avenue Road in the City of Toronto. The recommendations are based on interpretation of the factual data obtained from the boreholes advanced during the current and previous subsurface investigations at this site. The interpretation and recommendations contained in this report are intended to provide the designers with sufficient information to assess the feasible foundation alternatives and to carry out the detail design of the foundations for the proposed new structure.

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

It is understood that, as part of the Highway 401 EBC rehabilitation, the existing Avenue Road Underpass structure will be replaced. Based on the design drawings provided by AECOM, it is understood that a three-span replacement structure is proposed to be constructed along the existing Avenue Road alignment with the new foundations located at or near the existing foundation locations and elevations. It is also understood that the structure is proposed to be replaced in two stages, with the west half of the existing structure demolished and reconstructed first while two lanes of traffic are maintained on the east half; traffic would then be transferred to the new western portion of the structure, and the existing east half of the structure would be demolished and reconstructed.

6.2 Foundation Options

The existing structure is a three-span Underpass constructed in 1962, with span lengths of 33.5 m between the abutments and piers, and 37.2 m between the piers. Based on the available design drawings, the abutments and piers are supported on spread footings, with the north and south abutment footings founded at approximately Elevations 178.6 m and 177.7 m, respectively, and the north and south pier footings founded at approximately Elevation 175.3 m. Based on observations of the supporting structure during the current site investigation, the foundations appear to have performed satisfactorily to date.

Based on the General Arrangement and grade raise profile drawings provided by AECOM on February 24 and 25, 2016, respectively, the proposed replacement is to consist of a three-span open girder structure with a total span length of approximately 104.2 m. The replacement structure will be constructed on the existing Avenue Road alignment, with the new centreline matching the existing centreline. The Highway 401 grade at the site is to be maintained at approximately Elevation 178.0 m. The proposed Avenue Road grade will be raised by approximately 1 m at the mid-point of the new structure. At the approach embankments a maximum grade raise of approximately 0.75 m is anticipated and the existing 6 m to 7 m high Avenue Road approach embankments will be widened to accommodate the proposed grade raise.



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Based on the proposed Underpass geometry and the subsurface conditions at the site, both shallow foundations and deep foundations have been considered for support of the abutments and piers for the new Avenue Road Underpass. 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 is provided in Table 1 following the text of this report.

- **Strip or spread footings founded on the hard/dense to very dense till or sand deposits:** Spread footings are considered feasible and suitable to support the new abutments and piers given the competency of the native soils at this site and the relative cost of construction; this option would also allow for the use of semi-integral abutments. It is anticipated that the new abutments would be located at approximately the same location and elevation as the existing abutments, and approximately 6 m to 8 m of excavation would be required below the Avenue Road grade to remove the existing abutment walls and footings. This proposed founding level for the abutment foundations would be near the groundwater level, and some limited groundwater control is expected to be required. It is also anticipated that the new piers would be located at approximately the same location and elevation as the existing piers, and approximately 3 m to 4 m of excavation would be required below the Highway 401 road grade to remove the existing footings. Temporary protection systems will be required along the Highway 401 westbound and eastbound lanes, to facilitate the removal of the existing abutment and pier foundations and the construction of the new footings.

- **Footings on engineered fill:** Spread footings founded on Granular 'A' engineered fill (pad) within the existing fill materials are not considered a feasible alternative for this site, as removal of the existing foundations is required. Additionally the presence of organics within the fill will result in lower resistances and will increase the risk for future settlements. Alternatively, if the existing fill materials are fully excavated during removal of the existing abutment and pier foundations, spread footings for the abutments and piers may be founded on a Granular 'A' engineered fill placed on the native hard/dense to very dense till or sand deposits to raise the subexcavated subgrade level.

Driven steel H-piles or pipe (tube) piles founded within the hard clayey silt till deposit: 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 tube piles) or integral abutments (for H-piles). Pile foundations are not considered a practical option at the pier locations where the depth to "100-blow" material is shallow, at approximately 3 m to 6 m below the Highway 401 grade. The abutment pile caps could be "perched" within the reconstructed Avenue Road embankment, or constructed to match the existing abutment footings at approximately Elevations 179 m and 178 m for the north and south abutments, respectively. At the abutments the surface of the "100-blow" material is at approximately Elevation 174 m in the vicinity of the north abutment, and 173 m in the vicinity of the south abutment. The minimum required pile length of 5 m for integral abutments may be achievable without pre-augering; however, there is risk associated with penetrating through the till deposits or the piles "hanging up" within the very dense portions of till deposit due to the potential presence of cobbles, boulders encountered at higher elevation. Pile driving shoes are recommended to protect the pile tips from damage during driving into the very dense/hard soils.

- **Caissons founded within the hard clayey silt till deposit:** Caissons are feasible for the support of the abutments; however, caissons are not considered a practical option for the piers where the depth to "100-blow" material is shallow. 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



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steel piles that would be required. The abutment pile caps would be “perched” within the reconstructed Avenue Road embankment or at the underside of the bridge structure; or the pile cap could be constructed to match the existing abutment footings levels at approximately Elevations 179 m and 178 m for the north and south abutments, respectively. Caissons would extend into the water-bearing granular deposits, and temporary or permanent liners would be required during caisson construction to control potential ground losses and/or disturbance of the caisson base.

Based on the above considerations, steel H-Pile deep foundations are considered feasible but caisson foundations are considered not practical for the support of the new abutments; and deep foundations are not considered practical for the support of the new piers. Shallow foundations are considered feasible and most appropriate for the support of the new abutments and piers and are the preferred option from a geotechnical/foundations perspective due to the presence of a suitable bearing stratum at shallow depth.

6.3 Shallow Foundations

6.3.1 Frost Protection

All footings should be founded at a minimum depth of 1.2 m below lowest surrounding grade, or provided with an equivalent thickness of insulation for frost protection, in accordance with Ontario Provincial Standard Drawing (OPSD) 3090.101 (*Foundation, Frost Penetration Depths for Southern Ontario*). As a guide, the MTO has adopted 25 mm (1 inch) of rigid polystyrene foam insulation as equivalent to 0.3 m reduction in soil cover.

6.3.2 Founding Elevations

It is understood that the new abutments and piers will be founded at or near the same location and elevation as the existing foundations. Therefore, full removal of the existing abutment walls and foundations would be required, with removal excavations extending to the existing footing founding levels at approximately Elevations 178.6 m and 177.7 m at the north and south abutment, respectively, and Elevation 175.3 m at the pier locations (based on the available design drawings, dated April 1962).

For support of the new abutments and piers, strip or spread footings should be founded below any fill or softened/loosened surficial soil: on the hard clayey silt till and dense to very dense sand to silty sand deposits at the north abutment; the very dense silt and sand to silty sand deposit at the north pier; the compact to very dense silt and sand till and hard sandy clayey silt deposits at the south pier; and the very dense silt and sand to silty sand till deposits at the south abutment.

Alternatively, the footings could be founded on OPSS.PROV 1010 (Aggregates) Granular ‘A’ engineered fill placed/compacted to raise the founding subgrade to a suitable level, including to a higher elevation within the abutment backfill to lessen the height of the abutment stem wall.

The following summarizes the recommended maximum founding elevations for strip or spread footing for support of the new abutments and piers.



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Foundation Element	Founding Elevation (m)	Founding Soil
North abutment	178.6	Hard clayey silt with sand till/ Dense to very dense sand to silty sand
	180.6	Granular 'A' pad/engineered fill
North pier	175.3	Very dense silt and sand to silty sand
South pier	175.3	Compact to very dense silt and sand till / Hard sandy clayey silt
South abutment	177.7	Very dense silt and sand to silty sand till
	179.7	Granular 'A' pad/engineered fill

6.3.3 Geotechnical Resistance/Reaction

For 4 m or 5 m wide concrete footings founded at the elevations given in Section 6.3.2, the factored axial geotechnical resistance at Ultimate Limit States (ULS) and the geotechnical resistance at Serviceability Limit States (SLS), for 25 mm of settlement, may be taken as follows:

Foundation Element	Footing Foundation Subgrade	Factored Geotechnical Resistance at ULS (kPa)	Geotechnical Reaction at SLS (kPa)
Abutments and Piers	Native Stratum as noted above	700	450
	Minimum 2 m thick Granular 'A' Pad / Engineered Fill constructed on native stratum	750	350

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

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

For the option full sub-excavation of the existing fill and replacement with OPSS.PROV 1010 (Aggregates) Granular 'A' Engineered Fill, any existing topsoil, organics and loosened/softened or deleterious material that may be present within the foundation and engineered fill footprint must also be removed to minimize settlement. The area to be sub-excavated and backfilled to the footing level should be defined by a line extending from the top of the engineered fill at the footing level outward and downward at 1 horizontal to 1 vertical (1H:1V). The sub-excavation should be backfilled / constructed in accordance with OPSS.PROV 501 (Compacting) to at least a thickness of 2 m for support of the footings. The required thickness (i.e., 1.2 m) of conventional soil cover for frost protection of the footing (1.2 m) constructed on the granular pad/engineered fill is measured perpendicular from



the face of the abutment slope to the edge of the underside of the footing (it is not simply a vertical dimension when the footing is adjacent to a slope). If adequate soil cover cannot be provided for the footing, rigid styrofoam insulation shall be installed to compensate for the lack of cover and provide protection from frost action. As a guide, the MTO has adopted 25 mm (1 inch) of rigid polystyrene foam insulation as equivalent for every 0.3 m reduction in soil cover.

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

6.3.4 Resistance to Lateral Loads

Resistance to lateral forces / sliding resistance between the concrete footings and the subgrade should be calculated in accordance with Section 6.7.5 of the *CHBDC*. For cast-in-place concrete footings constructed on a concrete working slab that is cast on top of the generally hard/dense to very dense till and granular soils, the coefficient of friction, $\tan \delta$ or ϕ' , can be taken as follows:

- Cast-in-place footing to till deposit on native granular (sand) deposit: $\tan \phi' = 0.60$
- Cast-in-place concrete working slab to till deposit or native granular (sand) deposits: $\tan \phi' = 0.60$
- Cast-in-place footing to concrete working slab: $\tan \delta = 0.7$
- Cast-in-place footing to Granular 'A' Pad/Engineered Fill: $\tan \phi' = 0.62$

These values are unfactored.

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

Steel H-Piles or steel tube (pipe) piles driven to found within the hard clayey silt till may be used to support the new north and south abutments, especially if integral abutments are being considered. For the installation of the piles, consideration must be given to the potential presence of cobbles and boulders within the glacially-derived soils at this site. In addition, it is noted that zones of "100-blow" material was occasionally encountered higher than the proposed founding tip elevations. In this regard, steel H-piles are preferred over steel tube piles given that H-piles are more conventional for integral abutment design; and the fact that steel tube piles are considered to pose a higher risk of "hanging up" or being deflected from their vertical or battered orientation during installation, due to their larger and blunt end area. The piles should be reinforced at the tip for protection during driving to reduce the potential for damage to the piles in the event that cobbles/boulders and/or very dense layers are encountered within the till deposits. The steel H-piles should be reinforced with flange plates as per OPSD 3000.100 (Foundation Piles Steel H-Pile Driving Shoe) or driving shoes such as Titus Standard "H" Bearing Pile



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Point design for protection during driving as per OPSS 903 (Deep Foundations). Similarly, if steel tube piles are being considered, driving shoes should be in accordance with OPSD 3001.100 Type II (Steel Tube Pile Driving Shoe). The requirement for driving shoes should be included in the Contract Drawings.

If corrugated steel pipes (CSPs) are installed as part of the integral abutment design, the CSPs should be backfilled with loose, fine to medium sand. A Non Standard Special Provision (NSSP) detailing the installation method and gradation of this sand should be included in the Contract Documents; an example is included in Appendix D.

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

6.4.1 Friction Piles

HP 310 x 110 piles driven to found within the hard clayey silt till to a design tip Elevation 157 m (i.e., piles about 22 m long at the north abutment and 21 m long at the south abutment), the factored axial geotechnical resistance at ULS may be taken as 1,200 kN. The geotechnical reaction at SLS (for 25 mm of settlement) may be taken as 1,000 kN. The following note, (Note 2 from the Structural Manual Section 3.3.3 (MTO, 2008)) or similar, should be shown on the Contract Drawing assuming that a resistance factor of 0.5 is applied to the use of the Hiley calculation based on MTO experience in the Southern Ontario region:

"Piles to be driven in accordance with Standard SS-103-11 using an ultimate geotechnical resistance of 2,400 kN per pile, but must be driven below tip Elevation 157 m

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

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

Assessment of ultimate geotechnical resistance by the Hiley formula should commence once the pile reaches a depth of not more than 1.5 m above the design pile tip elevation shown above and at 0.5 m intervals of depth until the ultimate axial resistance is achieved. If the ultimate capacity as determined by the Hiley formula is not achieved within the 1.5 m interval down to the design pile tip elevation, the Contractor should stop pile driving and notify the Contract Administrator. At this depth, the pile should be allowed to rest for 48 hours and the Hiley formula should then be applied immediately upon re-striking the pile. If the ultimate capacity is still not achieved after the 48 hour wait period, the Contract Administrator should be notified and authorization given prior to driving the pile below the design pile tip elevation.

It is also recommended that the ultimate resistances of the driven piles also be assessed but the Wave Equation Analysis (PDA) for further comparison of the data to those obtained by the Hiley Formula, as presented in Section 6.8.7, Monitoring of Driven Piles.



Given the variability in the relative density of the various subsurface strata, it is recommended that an allowance for varying pile lengths be provided in the Contract Documents to ensure that adequate pile lengths are available on site and to reduce splicing needs. It is also recommended that the axial capacity be calculated by the Hiley formula on every pile installed.

Given the hard/very dense general nature of the overburden soils and the net unloading condition at the new approach embankment locations (due to removal of the existing approach embankment fill), downdrag loads are not anticipated.

6.4.2 Resistance to Lateral Loads

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

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

For cohesionless soils:

$$k_h = \frac{n_h z}{B} \quad \text{Where}$$

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

For cohesive soils:

$$k_h = \frac{67 s_u}{B} \quad \text{Where}$$

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

For an integral abutment design using steel H-Piles as the foundation element, the design should include the installation of 3 m long CSP liners (with the annular space between the pile and the liner filled with uniform grained, uncompacted sand), where the upper portion of the H-pile will be generally free to flex and move laterally within the limits of the CSP. With this design, the passive lateral resistance over the length of the pile within the CSP liner should be based on the resistance provided by loose sand. The passive lateral resistance on the exterior of the CSP should be based on the resistance provided by the surrounding soil conditions.

The following values of n_h and s_u may be assumed in the structural analyses. The soil stratigraphy has been generalized and the values reflect the variability in the subsurface conditions within the foundation elements footprint, however, the deposit boundaries vary slightly at the abutments and reference can be made to the



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borehole records and to the interpreted stratigraphic sections for each foundation element on Drawing 2 to assess the variation.

Soil Unit	n_h (kPa/m)	S_u (kPa)
Loose sand within CSP (if applicable)	2,200	-
Existing firm to stiff clayey silt fill	-	75
Very stiff to hard clayey silt with sand till	-	150
Dense to very dense silt and sand to silty sand till	20,000	-
Dense to very dense sand	15,000	-

For design of HP 310x110 piles, the maximum factored lateral resistances at ULS may be taken as 160 kN, and the maximum lateral resistance at SLS (for 10 mm of horizontal deflection at pile cap level) may be taken as 65 kN. These values are based on the "Assessed Horizontal Passive Resistance Values for Various Pile Types" provided in Table C6.8.7.1(a) of the *Commentary* to the CHBDC.

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

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

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

6.5 Caisson Foundations

Consideration could be given to the use of caissons socketted into the hard clayey silt till deposit for support of the foundation elements for the new abutments.

If caisson foundations are adopted for support of any of the foundation elements, a temporary or permanent liner may be required to support the soils during construction, to minimize disturbance and loss of ground if water-bearing cohesionless soil zones/seams are encountered within the overburden. If there is water infiltration such that there is standing water within the caisson excavation prior to concrete placement, the concrete must be placed using tremie techniques. After initial placement of concrete at the bottom of the caisson, the tremie discharge point should be maintained below the surface of the wet concrete during placement. It is recommended that a NSSP be included in the Contract Documents to address the need for control of the ground and groundwater



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during caisson construction as discussed further under Construction Considerations in Section 6.8. It is expected that the liner would be installed (and removed, if a temporary liner is used) using vibratory methods. In this case, vibration monitoring is recommended during liner installation and removal. The liner must be maintained tight to the sides of the bore to minimize seepage of water.

The performance of caissons will depend upon the final cleaning and verification of the subgrade quality at the base of the caissons. Each caisson excavation should be carefully cleaned to remove all loosened debris to ensure that the concrete is in intimate contact with the competent bearing stratum. The Ontario Occupational Health and Safety Act (2016) outlines appropriate safety procedures and requirements that must be implemented prior to entry of personnel into the caissons for inspection of the base or alternatively, the inspections may be carried out remotely using visual recording equipment.

The caisson caps for the new piers should be provided with a minimum of 1.2 m of soil cover to provide adequate protection against frost penetration (per OPSD 3090.101 – *Foundation, Frost Penetration Depths for Southern Ontario*) unless the caps are positioned at the top of the columns.

6.5.1 Founding Elevations

Caissons may be founded within the hard clayey silt till deposit, socketed for at least 1.5 m into the hard deposit. The estimated caisson tip elevations for new abutment foundations are summarized below.

Foundation Element	Maximum Founding Elevation (m) ¹	Founding Soil
North abutment	157.0	Hard clayey silt till
South abutment	157.0	Hard clayey silt till

6.5.2 Geotechnical Resistances

The recommended values for factored geotechnical axial resistance at ULS and geotechnical resistance at SLS (for 25 mm of settlement) for caissons founded at the elevations provided in Section 6.5.1 are provided below.

Caisson Diameter	Factored Geotechnical Axial Resistance at ULS	Geotechnical Resistance at SLS (for 25 mm settlement)
0.6 m	1,500 kN	1,200 kN
1.2 m	3,750 kN	3,000 kN
1.5 m	5,250 kN	4,000 kN

6.5.3 Resistances to Lateral Loads

Resistance to lateral loading will be derived from the soil in front of the caissons. The resistance to lateral loading in front of the caisson may be calculated using subgrade reaction theory and the equations and soil parameters provided in Section 6.4.2 may be used for design.



6.6 Lateral Earth Pressures for Design

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

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

- Select, free-draining granular fill in accordance with OPSS.PROV 1010 Granular 'A' or Granular 'B' Type II should be used as backfill behind the walls. Longitudinal drains and weep holes should be installed to provide positive drainage of the granular backfill. Other aspects of the granular backfill requirements with respect to sub drains and frost taper should be in accordance with OPSD 3101.150 (Wall, Abutments, Backfill) and OPSD 3121.150 (Walls, Retaining, Backfill).
- A minimum compaction surcharge of 12 kPa should be included in the lateral earth pressures for the structural design of the wall stem, in accordance with CHBDC Section 6.9.3 and Figure 6.6. Compaction equipment should be used in accordance with OPSS.PROV 501 (Compacting). Other surcharge loadings should be accounted for in the design as required.
- For restrained structures, the granular fill may be placed in a zone with the width equal to at least 1.5 m behind the back of the walls (Figure C6.20 (a) of the *Commentary* to the CHBDC). For unrestrained structures, the granular fill should be placed within the wedge-shaped zone defined by a line drawn at 1.5 horizontal to 1 vertical (1.5H:1V) extending up and back from the rear face of the footing (Figure C6.20 (b) of the *Commentary* to the CHBDC).
- For restrained structures, the pressures are based on the existing embankment fill materials and the following parameters (unfactored) may be used assuming the use of native clayey silt fill:

Unfactored Parameters		Earth Fill
Soil unit weight:		20 kN/m ³
Coefficients of static lateral earth pressure:	At rest, K_o	0.50
	Active, K_a	0.33



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- For unrestrained structures, where the pressures are based on OPSS.PROV 1010 granular fill behind the wall, the following parameters (unfactored) may be assumed:

Unfactored Parameters		Granular A	Granular B Type II
Soil unit weight:		22 kN/m ³	22 kN/m ³
Coefficients of static lateral earth pressure:	At rest, K_o	0.43	0.43
	Active, K_a	0.27	0.27

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

6.6.1 Seismic Considerations

6.6.1.1 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* (2006) may be taken as 1.2, consistent with Soil Profile Type II.

6.6.1.2 Seismic Site Coefficient

According to the National Building Code (1995) seismic hazard values (as reference in the *Commentary* of the *CHBDC* (2006), the site-specific peak horizontal ground acceleration for the Toronto area is 0.05. According to Table 4.1 of the *CHBDC*, this site is located in Seismic Performance Zone 1 and the corresponding site specific zonal acceleration A_s is 0.05.

Given this assessment and the fact that the proposed structure is not designated as a lifeline or truss bridge, and in accordance with Section 4.4.5.1, Table 4.2 of the *CHBDC*, no seismic analysis is required for structures located in Seismic Performance Zone 1.

6.7 Bridge Approaches

The proposed replacement of the Avenue Road Underpass structure includes the raising of the bridge grade by approximately 1 m at the mid-point of the new structure. At the approach embankments a maximum grade raise of approximately 0.75 m is anticipated and the existing 6 m to 7 m high Avenue Road approach embankments will be widened to accommodate the proposed grade raise. The proposed bridge replacement option also involves an approximately 1 m eastward widening to accommodate the staged removal of the existing structure and reconstruction of the new structure at this site. The existing embankment side-slopes are inclined at about 2H:1V and it is assumed the new widened side-slopes will be maintained at 2H:1V or shallower. It is considered that this embankment widening and maintaining the 2H:1V side slope inclination will not detrimentally impact the stability of the approach embankment nor result in settlements greater than 25 mm.



Recommendations pertinent to the widening of the approach embankments, subgrade preparation and embankment construction are provided below.

6.7.1 Subgrade Preparation and Embankment Construction

The existing native soils are considered to be an appropriate subgrade for the new construction of the Avenue Road approach embankments. However, to improve performance and minimize the potential differential settlement between the widening / new construction and the existing approach embankments, it is recommended that prior to the placement of any fill, all topsoil, organic matter, and any softened/loosened native soils be stripped from below the approach embankment areas, including the side slopes at the transitions immediately adjacent to the abutments.

The use of granular fill for embankment widening is recommended rather than the use of cohesive fill, since the majority of settlement of granular fills will occur during construction whereas some settlement of cohesive fills, if used, would occur post-construction. The new embankment fills should be benched into the existing embankments in accordance with OPSD 208.010 (Benching of Earth Slopes). The fill for the widened embankment should be placed and compacted in accordance with OPSS.PROV 501 (Compacting), with inspection and field density testing by qualified personnel during placement operations to confirm that appropriate materials are used and that adequate levels of compaction are achieved.

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

6.8 Construction Considerations

6.8.1 Open Cut Excavation

The foundation excavations at the abutments for spread footings or pile cap construction will extend to depths of about 6 m to 8 m below the present Avenue Road grade, through the existing fill into the very stiff to hard/very dense till and dense granular deposits. The foundation excavations at the piers for spread footing construction will generally extend to depths of about 2 m to 3 m into the very stiff to hard/compact to very dense soils.

Where space permits, open-cut excavations into these materials should be carried out in accordance with the guidelines outlined in the Occupational Health and Safety Act (OHSA) for Construction Activities. The existing fill is classified as Type 3 soil and the native soil is classified as Type 2 soil, according to the OHSA. Temporary excavations (i.e., those that are open for a relatively short time period) should be made with side slopes no steeper than 1 horizontal to 1 vertical (1H:1V) through the fill materials and through the till to within 1.2 m of the bottom of the excavation.



6.8.2 Temporary Excavation Support

It is anticipated that temporary protection systems will be required along Highway 401 to facilitate the removal of the existing abutment and pier foundations and the construction of the new abutment footings and pier foundations. Temporary protection systems will also be required approximately along the mid-line of the Avenue Road embankment to facilitate the planned staged construction while maintaining two lanes of traffic. These temporary protection systems should be designed and constructed in accordance with OPSS.PROV 539 (Temporary Protection Systems). The lateral movement of the temporary shoring system should meet Performance Level 2 as specified in OPSS.PROV 539, provided any adjacent utilities can tolerate this magnitude of deformation.

The selection and design of the protection system will be the responsibility of the Contractor. However, the following comments are provided to aid in the preliminary costing and assessment of temporary protection system options for this site.

The protection system is required for an estimated excavation depth of approximately 8 m at the abutments and 3 m to 4 m at the pier locations (through fill and into the native materials). It is considered that either a driven, interlocking sheetpile system or a soldier pile and timber lagging system would be suitable for the temporary excavation support at this site, based on the inferred subsurface soil conditions and groundwater conditions; however the potential presence of cobbles and boulders and the presence of dense to very dense/hard material may affect the installation of the interlocking sheet pile system. An interlocking sheetpile system would contribute to both ground and, where applicable, groundwater control of seepage from cohesionless zones or interlayers/lenses within the cohesive deposits that may be required. For a soldier pile and lagging system, it would be necessary to control seepage or include measures to mitigate loss of soil particles through the lagging boards if cohesionless soils/lenses are encountered, but the groundwater level in the piezometers was measured at about Elevation 172.3 m (depth of 12.2 m below ground surface) and Elevation 172.5 m (depth of 12.7 m below ground surface) in Boreholes AR15-1 and AR15-6, respectively and should not contribute to seepage into the excavations.

The sheetpiles or soldier piles would have to be socketted to sufficient depth to provide the necessary passive resistance for the retained soil height of up to about 8 m. Lateral support to the sheetpiles or soldier piles could be provided in the form of rakers or temporary anchors. The selection and design of the protection system will be the responsibility of the Contractor.

6.8.3 Groundwater Control

Excavations for removal of the existing abutment and pier foundations, construction of the new abutment and pier footings will extend near the groundwater level at the site, which has been measured to be between Elevation 172.3 m and 172.5 m. Groundwater seepage is expected from the non-cohesive deposits below this elevation; some "perched" groundwater may also be present at the base of the fill materials, on top of the cohesive deposit. Some water inflow should be expected into the foundation excavations, particularly during wet months; however, it is anticipated that water inflow can be handled by pumping from filtered sump pumps placed at the base of the excavations.

6.8.4 Subgrade Protection

The soils that will be exposed at the foundation subgrade level will be susceptible to disturbance from construction traffic and/or ponded water. To limit this degradation, it is recommended that, if the concrete foundations or pile



caps are not constructed within four hours after preparation, inspection and approval of the founding subgrade, a concrete working slab should be placed on the prepared subgrade. This requirement can be addressed with a note on the drawings and/or with an NSSP. An example NSSP for the concrete working slab is included in Appendix D.

6.8.5 Vibration Monitoring During Pile Installation

Depending on the construction sequence, vibration monitoring may be necessary at the existing structure during driven pile or caisson installation, if pile or caissons foundations are selected for support of the structure, to ensure that the vibration levels at the existing structure are maintained below tolerable levels. A maximum peak particle velocity (PPV) of 100 mm/s is generally considered applicable for bridge structures in good condition (i.e. considering portions of the existing structure and foundations are to remain permanently in place); however, this requires further assessment by the structural engineer. The piles further from the existing Underpass structure should be driven/installed first, in order to monitor the vibration level at the existing structure and, if necessary, alter the pile driving criteria for caisson installation for the remaining piles/caissons. As there are several residential and commercial structures in the vicinity of the site, monitoring of vibrations during construction should be considered by the general contractor to defend against potential damage claims by the owners of the nearby structures.

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

6.8.6 Obstructions During Pile Driving / Caisson Installation

Cobbles and/or boulders were encountered and inferred during drilling operations, based on the observed difficulty to augering at varying depths in the boreholes drilled during the current subsurface investigation, which may affect the installation of steel H-piles/tube piles or caissons. It is recommended that driving shoes be used on all steel H-piles or tube piles to facilitate driving into the overburden soils. In addition it is recommended that an NSSP be included in the Contract Documents to warn the Contractor of the possible presence of cobbles and/or boulders within the overburden soils, an example NSSP is presented in Appendix D.

6.8.7 Monitoring of Driven Piles

Pile installation and monitoring should be in accordance with OPSS 903 (Deep Foundations). As recommended in Section 6.4.1, driven pile resistance should be verified in the field by the use of the Hiley formula (MTO Standard Drawing SS103-11) during the final stages of driving to verify that the ultimate capacity required as noted on the Contract Drawings has been achieved. It is recommended that consideration be given to also applying dynamic testing to monitor the pile resistance during driving operations, using a PDA (Pile Driving Analyzer) on selected piles for comparison of the ultimate resistance data and assessment of differences between the two methods to provide the MTO with a basis/rationale for adopting the use of the PDA method in the future.

The PDA testing program is proposed to confirm mobilized axial resistance, set criteria and to monitor compressive and tensile stresses along the pile during installation. PDA testing consists of attaching gauges (strain gauges and accelerometers) to a pile to measure strains and movements within the pile during driving. The pile is then struck using the pile driving hammer and data are recorded by a data acquisition system connected to the gauges by a cable. The result of the PDA tests (with appropriate interpretation) is then used to (among other applications):



FOUNDATION REPORT AVENUE ROAD UNDERPASS

- assess the geotechnical axial resistance of the tested pile;
- confirm hammer driving energies;
- check pile stresses and evaluate potential pile damage; and,
- confirm/determine relationships between energy, set and mobilized geotechnical axial compression resistance of the pile for a given hammer.

Where PDA testing is performed on a representative number of piles, the geotechnical resistance factor in compression should be 0.5 to calculate the factored geotechnical resistance, as per the CHBDC (2006) Section 6.6.2.3. Where the PDA testing indicates resistances significantly different than design, the PDA testing could be used to calibrate the design resistances. It is recommended that the results of the PDA testing be reviewed by a qualified professional to verify that the analysis has been carried out correctly.

Should both Hiley and PDA be specified for this project, it is recommended that the Contract Administrator determine which test results govern in field. An example NSSP providing an amendment to OPSS 903 is presented in Appendix D for the addition of PDA testing on a minimum of 20 per cent of piles at each foundation element for both stages of the replacement structure construction.



FOUNDATION REPORT AVENUE ROAD UNDERPASS

7.0 CLOSURE

This Foundation Design Report was prepared by Ms. Nikol Kochmanová, P.Eng., a geotechnical engineer with Golder. Mr. Jorge Costa, P.Eng., a Designated MTO Contact and Principal of Golder, conducted an independent review and quality control audit of this report.

GOLDER ASSOCIATES LTD.



Nikol Kochmanová, Ph.D., P.Eng., PMP
Geotechnical Engineer



Jorge M.A. Costa, P.Eng.
Designated MTO Foundations Contact, Principal

NK/JMAC/sm/rb

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FOUNDATION REPORT AVENUE ROAD UNDERPASS

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Occupational Health and Safety Act and Regulations for Construction Projects, January 2012.

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

Ontario Provincial Standard Specifications (OPSS)

OPSS.PROV 501	Construction Specifications for Compacting
OPSS.PROV 539	Construction Specification for Temporary Protection Systems
OPSS 802	Construction Specification for Topsoil
OPSS 803	Construction Specification for Sodding
OPSS.PROV 804	Construction Specification for Seed and Cover
OPSS 902	Construction Specification for Excavating and Backfilling Structures
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 Pile Driving Shoe
OPSD 3090.101	Foundation Frost Penetration Depths for Southern Ontario
OPSD 3101.150	Walls, Abutments, Backfill, Minimum Granular Requirements
OPSD 3121.150	Walls, Retaining, Backfill, Minimum Granular Requirements

ASTM International

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



FOUNDATION REPORT AVENUE ROAD UNDERPASS

**TABLE 1: COMPARISON OF FOUNDATION ALTERNATIVES
FOR AVENUE ROAD UNDERPASS RECONSTRUCTION
HIGHWAY 401 EBC REHABILITATION – JANE STREET TO AVENUE ROAD
G.W.P. 2131-01-00, AGREEMENT NO. 2009-E-0011**

FOUNDATION OPTION	FEASIBILITY	ADVANTAGES	DISADVANTAGES	RELATIVE COSTS	RISKS / CONSEQUENCES
Spread/strip footings on hard clayey silt till (north abutment), very dense silt and sand to silty sand (north pier), compact to very dense silt and sand till (south pier) and very dense silt and sand to silty sand till (south abutment).	<ul style="list-style-type: none"> Feasible for support of new abutments and new piers. 	<ul style="list-style-type: none"> Existing abutments and piers supported on shallow foundations, and have performed well. Excavations to remove existing structure and foundations would extend to suitable founding stratum for the replacement structure. Lower costs than deep foundations. Standard construction methods; no specialized construction equipment required. 	<ul style="list-style-type: none"> Temporary excavation support required along Highway 401 to facilitate removal and construction of the abutment and pier footings. Precludes use of integral abutments; potentially greater maintenance required at abutments. Lower geotechnical resistances as compared with deep foundations. 	<ul style="list-style-type: none"> Less expensive than deep foundations although bridge construction /maintenance costs may be higher due to non-integral abutment configurations. Estimated cost is \$600/m3 for construction of spread footings, excluding temporary protection system. 	<ul style="list-style-type: none"> Potential traffic disruption during construction.



FOUNDATION REPORT AVENUE ROAD UNDERPASS

FOUNDATION OPTION	FEASIBILITY	ADVANTAGES	DISADVANTAGES	RELATIVE COSTS	RISKS / CONSEQUENCES
Steel H-piles driven to found in "100-blow" material (hard clayey silt till).	<ul style="list-style-type: none"> Feasible but not considered practical for support of abutments. Not practical for the support of the piers where the depth to "100-blow" soil is shallow (approximately 3 m to 6 m). 	<ul style="list-style-type: none"> Abutment pile caps could be maintained higher than spread footings, potentially reducing depth of excavation and temporary excavation support requirements; however due to removal of existing foundations excavation depths may remain constant. Allows for integral abutment construction Higher geotechnical resistance than for shallow foundations. 	<ul style="list-style-type: none"> Temporary excavation support still anticipated to be required to facilitate removal of the existing abutments. Potential for encountering cobbles and/or boulders during pile driving; this could result in piles "hanging up" and lower geotechnical resistances being achieved. Potential for encountering "100-blow" material at higher elevations; this could result in piles "hanging up" and lower geotechnical resistances. May have to pre-drill through very dense portions of the silt to sand strata to ensure piles can be installed to the required founding depths. 	<ul style="list-style-type: none"> Lower relative cost compared with caisson option. Estimated cost is approximately \$250/m length for pile installation and \$600/m³ for pile cap construction; the actual cost is expected to be higher to account for pre-augering if needed. 	<ul style="list-style-type: none"> Potential traffic disruption during construction Potential vibrations may be induced on existing abutment footings. Risk of encountering obstructions that could impact pile installation. Potentially less costly maintenance over life of the structure than semi-integral abutment structures.



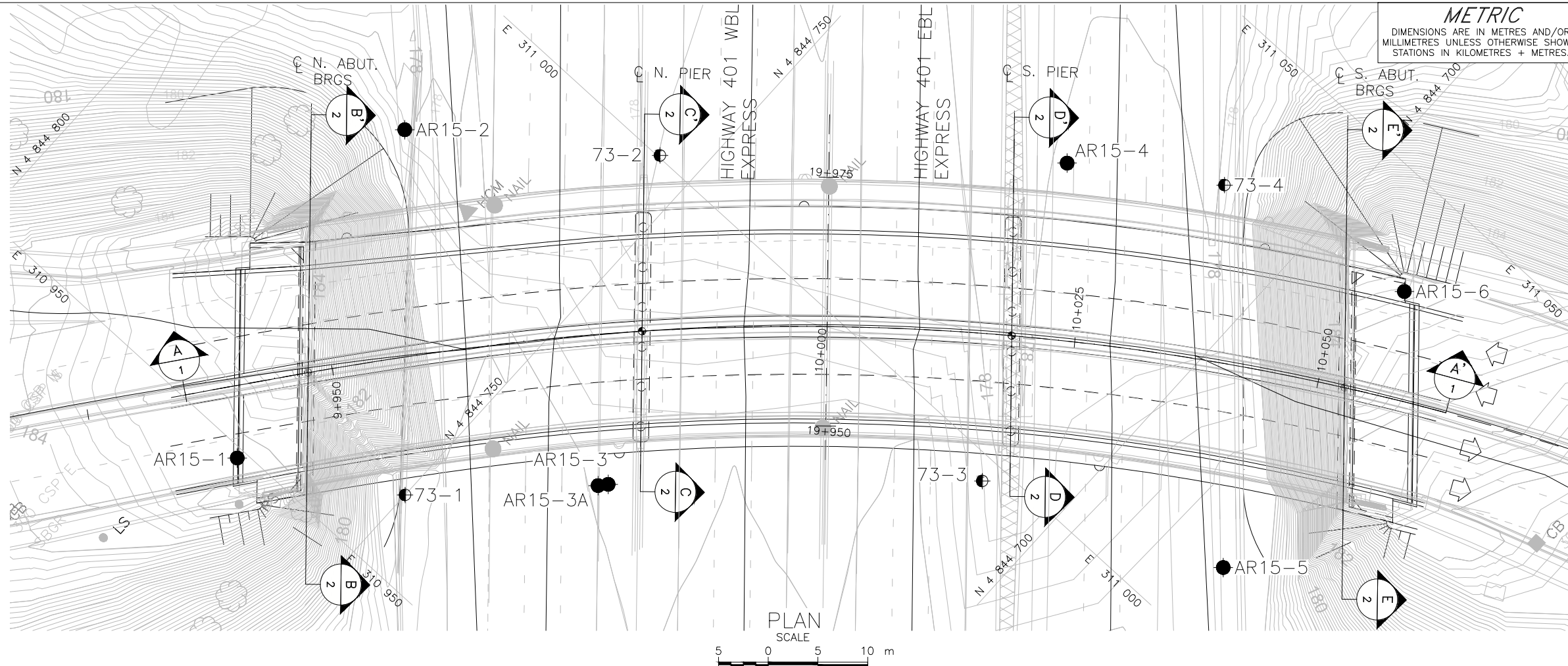
FOUNDATION REPORT AVENUE ROAD UNDERPASS

FOUNDATION OPTION	FEASIBILITY	ADVANTAGES	DISADVANTAGES	RELATIVE COSTS	RISKS / CONSEQUENCES
Steel pipe (tube) piles, driven to found in “100-blow” material (hard clayey silt till).	<ul style="list-style-type: none">• Feasible but not considered practical for support of abutments.• Not feasible for the support of the piers where the depth to “100-blow” soil is shallow (approximately 3 m to 6 m).	<ul style="list-style-type: none">• Abutment pile caps could be maintained higher than spread footings, potentially reducing depth of excavation and temporary excavation support requirements; however due to removal of existing foundations excavation depths may remain constant.• Higher geotechnical resistance than for shallow foundations.	<ul style="list-style-type: none">• Temporary excavation support still anticipated to be required to facilitate removal of the existing abutments.• Slightly greater risk than for steel H-pile foundations if cobbles and/or boulders are encountered during driving; this could result in piles “hanging up” and lower geotechnical resistances being achieved.• Greater potential for crumpling if obstructions or very dense stratum encountered.• MTO does not typically accept pipe pile foundation for integral abutment design.	<ul style="list-style-type: none">• Costs for steel pipe (tube) piles slightly higher than for steel H-piles.	<ul style="list-style-type: none">• Potential traffic disruption during construction.• Slightly greater risk than for steel H-pile foundations if obstructions (cobbles and/or boulders) are encountered during driving; resulting in piles “hanging up”.



FOUNDATION REPORT AVENUE ROAD UNDERPASS

FOUNDATION OPTION	FEASIBILITY	ADVANTAGES	DISADVANTAGES	RELATIVE COSTS	RISKS / CONSEQUENCES
Caissons founded in "100-blow" material (hard clayey silt till).	<ul style="list-style-type: none"> • Feasible but not considered practical for support of abutments. • Not practical for the support of the piers where the depth to "100-blow" soil is shallow (approximately 3 m to 6 m). 	<ul style="list-style-type: none"> • Abutment pile caps could be maintained higher than spread footings or even at underside of the bridge structure, potentially reducing depth of excavation and temporary excavation support requirements, or eliminating excavation needs altogether; however due to removal of existing foundations excavation depths may remain unchanged compared to other alternatives. • Higher axial resistances than for steel H-piles, so reduced number of deep foundation elements compared to steel H-piles. 	<ul style="list-style-type: none"> • Caissons would potentially extend below the groundwater level at the site, into water-bearing cohesionless deposits, with potential for loss of ground or base disturbance. • Temporary or permanent 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 • Greater risk of encountering obstructions due to larger size of drill hole required. 	<ul style="list-style-type: none"> • Estimated cost is approximately \$1000/m length for caisson installation and \$600/m³ for pile cap construction; the cost may be higher to account for pre-augering/coring and temporary liners. 	<ul style="list-style-type: none"> • Risk of disturbance of water-bearing soils, if encountered, requiring special construction procedures (tremie concrete) including use of temporary or permanent liners. • Significant traffic disruption during construction due to space required for caisson drilling equipment. • Risk of encountering obstructions that could impact caisson installation/costs.



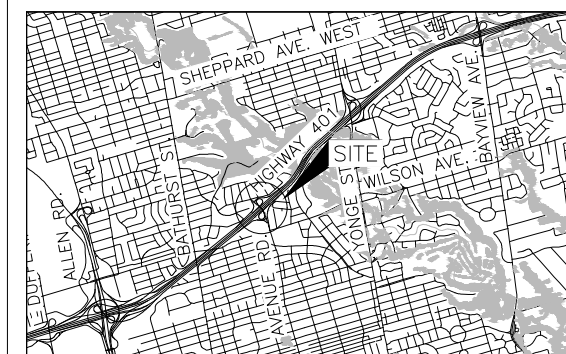
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CONT No.
GWP No. 2087-13-00

HIGHWAY 401 WIDENING
AVENUE ROAD UNDERPASS
BOREHOLE LOCATIONS AND
SOIL STRATA



SHEET

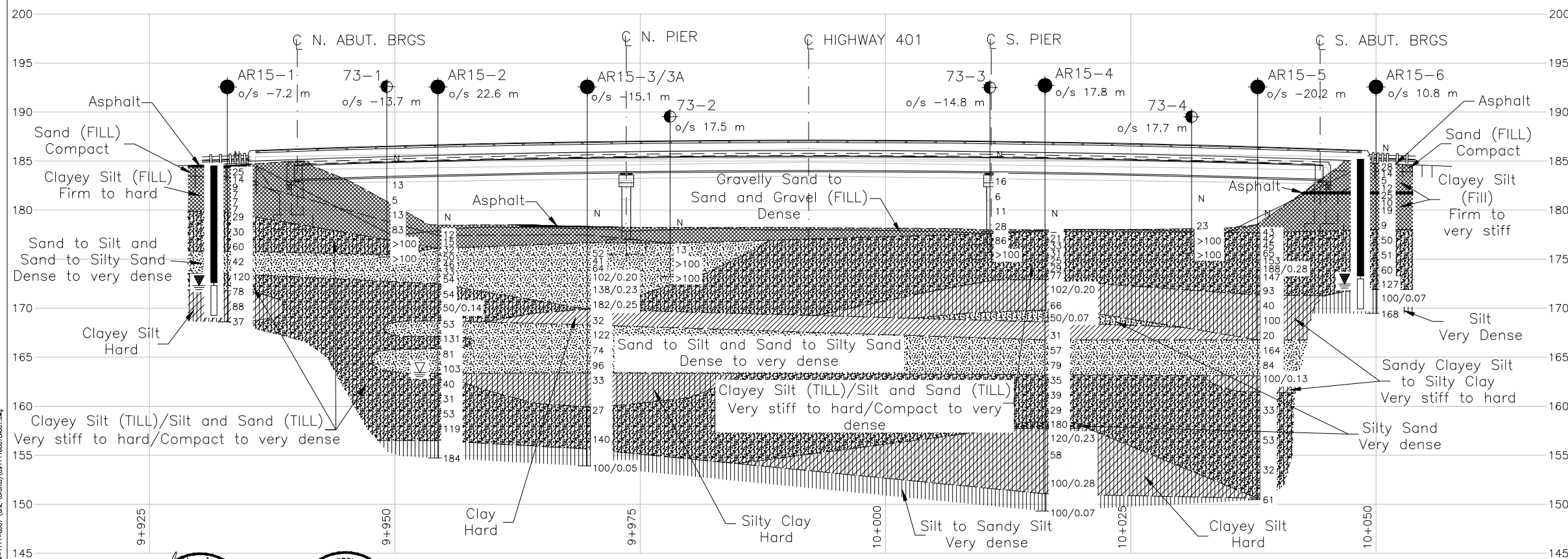


KEY PLAN
SCALE

1 0 1 2 km

LEGEND

- Borehole - Current Investigation
- Borehole - Previous Investigation 1 (Geocres No. 30M11-073)
- Seal
- Piezometer
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- WL in piezometer, measured on DEC 14, 2015
- WL upon completion of drilling



BOREHOLE CO-ORDINATES

No.	ELEVATION	NORTHING	EASTING
73-1	184.1	4844750.0	310957.7
73-2	177.6	4844754.0	311000.2
73-3	184.5	4844708.0	310997.8
73-4	180.0	4844710.0	311036.2
AR15-1	184.5	4844765.0	310949.1
AR15-2	178.1	4844774.7	310984.8
AR15-3	178.5	4844735.6	310972.2
AR15-3A	178.5	4844736.3	310971.5
AR15-4	178.5	4844723.2	311027.2
AR15-5	178.5	4844684.2	311007.7
AR15-6	185.2	4844689.4	311040.5

NOTES

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

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

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

NO. DATE BY REVISION

Geocres No. 30M11-261

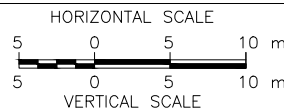
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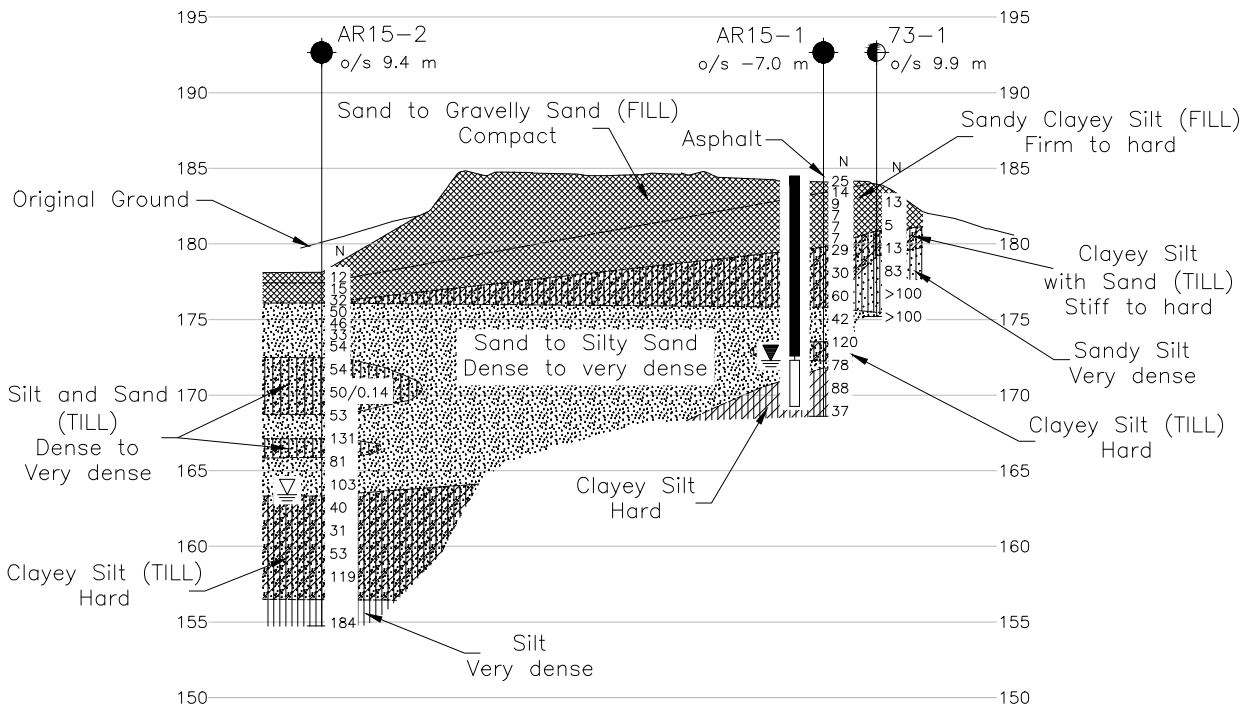
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HWY. 401	PROJECT NO. 09-1111-6007	DIST. CENTRAL
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DRAWN: JFC/TB	CHKD. JMAC	APPD. JMAC
		DWG. 1

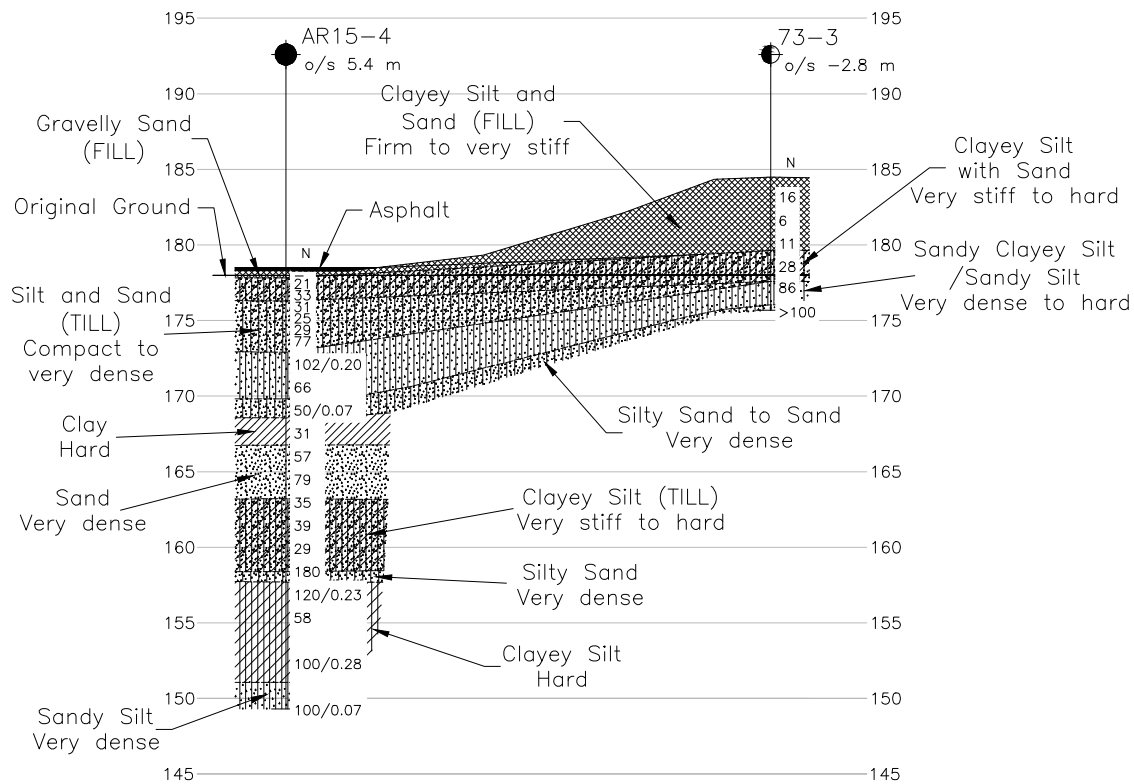


CENTRELINE PROFILE OF AVENUE ROAD

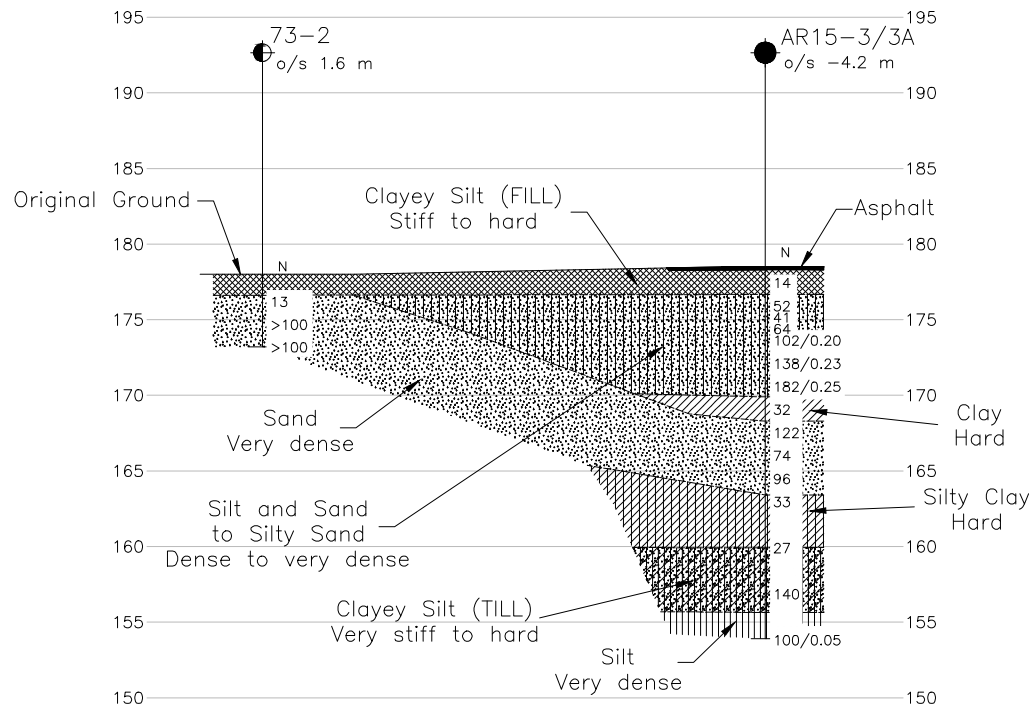




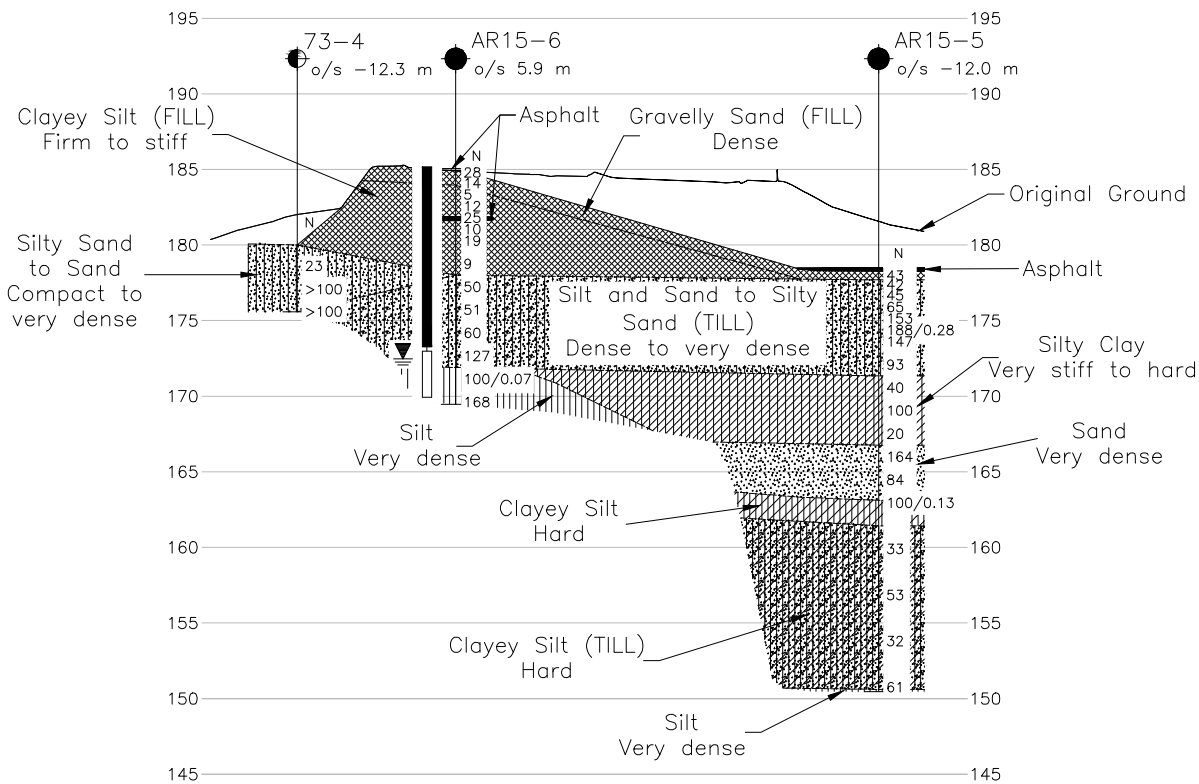
B-B'
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D-D'
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CROSS SECTION AT SOUTH PIER
HORIZONTAL SCALE
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VERTICAL SCALE
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C-C'
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CROSS SECTION AT NORTH PIER
HORIZONTAL SCALE
5 0 5 10 m
VERTICAL SCALE
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E-E'
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CROSS SECTION AT SOUTH ABUTMENT
HORIZONTAL SCALE
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METRIC
DIMENSIONS ARE IN METRES AND/OR
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STATIONS IN KILOMETRES + METRES.

CONT No.
GWP No. 2087-13-00

HIGHWAY 401 WIDENING
AVENUE ROAD UNDERPASS
SOIL STRATA

SHEET



LEGEND

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NO.	DATE	BY	REVISION
1	2/1/2016	TB	1
Geocres No. 30M11-261			
HWY. 401		PROJECT NO. 09-1111-6007	
SUBM'D. AJS		DATE: 2/1/2016	
DRAWN: TB		DWG. 2	



APPENDIX A

Record of Boreholes from Current 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

(a) Index Properties (continued)

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

(b) Hydraulic Properties

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

(c) Consolidation (one-dimensional)

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

(d) Shear Strength

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

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

Notes: 1
2

$$\tau = c' + \sigma' \tan \phi'$$

$$\text{shear strength} = (\text{compressive strength})/2$$



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 (Cohesionless) 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

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

IV. SOIL TESTS

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

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

V. MINOR SOIL CONSTITUENTS

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



LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY

WEATHERINGS STATE

Fresh: no visible sign of weathering

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

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

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

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

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

BEDDING THICKNESS

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

JOINT OR FOLIATION SPACING

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

GRAIN SIZE

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

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

CORE CONDITION

Total Core Recovery (TCR)

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

Solid Core Recovery (SCR)

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

Rock Quality Designation (RQD)

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

DISCONTINUITY DATA

Fracture Index

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

Dip with Respect to Core Axis

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

Description and Notes

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

Abbreviations

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

PROJECT		RECORD OF BOREHOLE				No AR15-1		SHEET 2 OF 2		METRIC															
G.W.P. 09-1111-6007		LOCATION				N 4844765.0 ; E 310949.1				ORIGINATED BY QC															
DIST Central HWY 401		BOREHOLE TYPE				CME 75 Truck-mount, 215 mm O.D. Hollow Stem Augers				COMPILED BY AJS															
DATUM Geodetic		DATE				September 23, 2015				CHECKED BY JMAC															
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 (%)													
							20	40	60	80	100	10	20	30											
168.6			14	SS	37		169																		
15.9	END OF BOREHOLE NOTE: 1. Water level in standpipe piezometer measured as follows: <table style="margin-left: 40px;"> <tr> <td>Date</td> <td>Depth (m)</td> <td>Elev. (m)</td> </tr> <tr> <td>11/01/15</td> <td>12.2</td> <td>172.3</td> </tr> <tr> <td>12/14/15</td> <td>12.2</td> <td>172.3</td> </tr> </table>	Date	Depth (m)	Elev. (m)	11/01/15	12.2	172.3	12/14/15	12.2	172.3															
Date	Depth (m)	Elev. (m)																							
11/01/15	12.2	172.3																							
12/14/15	12.2	172.3																							

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PROJECT <u>09-1111-6007</u>		RECORD OF BOREHOLE No AR15-2		SHEET 1 OF 2		METRIC	
G.W.P. <u>2087-13-00</u>		LOCATION <u>N 4844774.7 ; E 310984.8</u>		ORIGINATED BY <u>QC/AV</u>			
DIST <u>Central</u> HWY <u>401</u>		BOREHOLE TYPE <u>CME 75 Truck-mount, 165 mm O.D. Hollow Stem Augers</u>		COMPILED BY <u>AJS</u>			
DATUM <u>Geodetic</u>		DATE <u>September 23 - 24, 2015</u>		CHECKED BY <u>JMAC</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/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 ● QUICK TRIAXIAL × REMOULDED	W _p	W	W _L					
178.1	GROUND SURFACE						20	40	60	80	100					
0.0	Gravelly sand (FILL) Compact Brown Moist		1	SS	12											
177.4																
0.7	Sandy clayey silt, trace organics (FILL) Stiff to hard Brown Moist		2	SS	15											
			3A	SS	32											
176.1			3B													
2.0	SAND, trace to some silt to silty SAND Dense to very dense Brown Moist		4	SS	50											
			5	SS	46											
			6	SS	33											
			7	SS	54											
172.5																
5.6	SILT and SAND, trace to some clay, trace gravel (TILL) Very dense Brown Moist to wet		8	SS	54											
			9	SS	50/0.14											
168.7			10A													
9.4	SAND, some silt Very dense Brown Moist		10B	SS	53											
			11A													
167.1			11B	SS	131											
11.0	SILT and SAND, trace to some gravel, some clay (TILL) Very dense Brown Moist															
165.9																
12.2	SAND, trace to some silt, trace to some gravel Very dense Brown Wet		12	SS	81											
			13	SS	103											
163.3																
14.8																

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

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PROJECT 09-1111-6007		RECORD OF BOREHOLE No AR15-2		SHEET 2 OF 2		METRIC						
G.W.P. 2087-13-00		LOCATION N 4844774.7 ; E 310984.8		ORIGINATED BY QC/AV								
DIST Central HWY 401		BOREHOLE TYPE CME 75 Truck-mount, 165 mm O.D. Hollow Stem Augers		COMPILED BY AJS								
DATUM Geodetic		DATE September 23 - 24, 2015		CHECKED BY JMAC								
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					
--- CONTINUED FROM PREVIOUS PAGE ---												
	CLAYEY SILT, trace to some sand, trace gravel, some silt pockets (TILL) Hard Grey Moist to wet		14	SS	40							
			15	SS	31							
			16	SS	53							
			17	SS	119							
156.5 21.6	SILT Very dense Grey Dry to moist		18	SS	184							
154.7 23.4	END OF BOREHOLE NOTE: 1. Water level inside hollow stem augers at a depth of 14.7 m below ground surface (Elev. 163.4 m) upon completion of drilling.											

PROJECT		RECORD OF BOREHOLE		No AR15-3		SHEET 1 OF 1		METRIC									
G.W.P. 2087-13-00		LOCATION		N 4844735.6 ; E 310972.2		ORIGINATED BY		QC									
DIST Central HWY 401		BOREHOLE TYPE		CME 75 Truck-mount, 165 mm O.D. Hollow Stem Augers		COMPILED BY		AJS									
DATUM Geodetic		DATE		September 27, 2015		CHECKED BY		JMAC									
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									
178.5	GROUND SURFACE																
0.0	ASPHALT (200 mm)																
0.2	Sand and gravel, some silt (FILL) Brown Dry to moist		1A 1B	SS	50/0.1		178										
177.5																	
1.0	Clayey silt, some sand, trace to some gravel (FILL) Stiff to hard Brown Moist		2	SS	14		177										
176.7																	
1.9	Silty SAND, trace to some gravel Brown Dry to moist AUGER REFUSAL END OF BOREHOLE NOTE: 1. Water level not recorded in open borehole upon completion of drilling. 2. Auger refusal encountered at a depth of 1.9 m below ground surface on inferred cobbles/boulder. Borehole backfilled with bentonite and new borehole AR15-3A advanced 1.2 m north of original location (See Borehole AR15-3A).		3A 3B	SS	56/0.23												


PROJECT <u>09-1111-6007</u>		RECORD OF BOREHOLE No AR15-3A		SHEET 1 OF 2		METRIC	
G.W.P. <u>2087-13-00</u>		LOCATION <u>N 4844736.3 ; E 310971.5</u>		ORIGINATED BY <u>QC</u>			
DIST <u>Central</u> HWY <u>401</u>		BOREHOLE TYPE <u>CME 75 Truck-mount, 165 mm O.D. Hollow Stem Augers</u>		COMPILED BY <u>AJS</u>			
DATUM <u>Geodetic</u>		DATE <u>September 27, 2015</u>		CHECKED BY <u>JMAC</u>			

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							WATER CONTENT (%)
								○ UNCONFINED ● QUICK TRIAXIAL	+ FIELD VANE × REMOULDED						
178.5 0.0	GROUND SURFACE Refer to Record of Borehole AR15-3														
176.7 1.8	SILT and SAND to silty SAND, trace to some gravel, trace clay Dense to very dense Brown Dry to moist														
		4	SS	52											
		5	SS	41											
		6A	SS	64											
		6B													
		7	SS	102/0.20											
		8A	SS	138/0.23											
		8B													
			9	SS	182/0.25										
169.9 8.6	CLAY, trace sand Hard Brown to grey Moist		10	SS	32										
168.3 10.2	SAND, trace to some silt, trace to some gravel Very dense Brown Moist		11	SS	122										
			12	SS	74										
			13	SS	96										

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

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PROJECT		RECORD OF BOREHOLE		No AR15-3A		SHEET 2 OF 2		METRIC						
G.W.P. 2087-13-00		LOCATION		N 4844736.3; E 310971.5		ORIGINATED BY		QC						
DIST Central HWY 401		BOREHOLE TYPE		CME 75 Truck-mount, 165 mm O.D. Hollow Stem Augers		COMPILED BY		AJS						
DATUM Geodetic		DATE		September 27, 2015		CHECKED BY		JMAC						
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						
163.4 159.9	SILTY CLAY, some sand and silt seams Hard Grey Moist		14	SS	33		163							
							162							
							161							
							160							
18.6	CLAYEY SILT, trace to some sand, trace to some gravel (TILL) Very stiff to hard Grey Moist		15	SS	27		159							
							158							
							157							
155.6 22.9	SILT, trace to some clay, trace sand Very dense Grey Moist		16	SS	140		156							
							155							
153.9 24.6	END OF BOREHOLE NOTE: 1. Water level not recorded in open borehole upon completion of drilling.		17	SS	100/0.05		154							0 1 96 3

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PROJECT <u>09-1111-6007</u>		RECORD OF BOREHOLE No AR15-4		SHEET 1 OF 3		METRIC	
G.W.P. <u>2087-13-00</u>		LOCATION <u>N 4844723.2 ; E 311027.2</u>		ORIGINATED BY <u>QC</u>			
DIST <u>Central</u> HWY <u>401</u>		BOREHOLE TYPE <u>CME 75 Truck-mount, 165 mm O.D. Hollow Stem Augers</u>		COMPILED BY <u>AJS</u>			
DATUM <u>Geodetic</u>		DATE <u>September 26, 2015</u>		CHECKED BY <u>JMAC</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/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa		W _p	W	W _L		
178.5	GROUND SURFACE													
0.0	ASPHALT (200 mm)													
0.2	Gravelly sand, some silt (FILL)		1	AS	-									
177.8	Brown Moist													
0.7	CLAYEY SILT with SAND, trace gravel, some silt pockets (TILL)		2	SS	21									
	Very stiff to hard													
	Brown Moist		3	SS	33									
176.3														
2.2	SILT and SAND, trace to some gravel, trace to some clay (TILL)		4	SS	31									
	Compact to very dense													
	Brown Moist to wet		5	SS	25									
			6	SS	29									
			7	SS	77									
172.9														
5.6	Sandy CLAYEY SILT, some sand seams		8	SS	102/0.20									
	Hard													
	Brown to grey Moist		9	SS	66									
169.8														
8.7	Silty SAND		10	SS	50/0.07									
	Very dense													
	Brown Moist to wet													
	- Inferred cobble/boulder at 9.2 m													
168.6														
9.9	CLAY, some sand pockets		11	SS	31									
	Hard													
	Grey Moist													
166.8														
11.7	SAND, trace gravel to gravelly, trace to some silt, trace clay		12	SS	57									
	Very dense													
	Brown Wet		13	SS	79									
	Auger grinding from 14.3 m to 15.2 m on inferred cobble/boulder													

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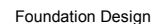
+³, ×³: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

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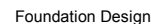
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+ 3, × 3: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE



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

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

PROJECT		RECORD OF BOREHOLE		No AR15-6		SHEET 1 OF 2		METRIC						
G.W.P. 2087-13-00		LOCATION		N 4844689.4 ; E 311040.5		ORIGINATED BY		QC						
DIST Central HWY 401		BOREHOLE TYPE		CME 75 Truck-mount, 215 mm O.D. Hollow Stem Augers		COMPILED BY		AJS						
DATUM Geodetic		DATE		October 8, 2015		CHECKED BY		JMAC						
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						
185.2	GROUND SURFACE							20 40 60 80 100	20 40 60 80 100	10 20 30				
0.0	ASPHALT (150 mm)													
0.2	Sand, some gravel to gravelly, trace silt (FILL) Compact Brown Dry to moist		1	SS	28									
184.0			2A	SS	14									
1.2	Clayey silt, trace to some sand, trace to some gravel, hydrocarbon odour (FILL) Firm to stiff Brown to black Dry to moist		2B											
			3	SS	5									
			4	SS	12									
181.9			5A											
	ASPHALT		5B	SS	25									
			5C											
3.6	Clayey silt, with sand, trace to some gravel, oxidation staining (FILL) Stiff to very stiff Brown Dry to moist		6	SS	10									1 47 38 14
			7	SS	19									
			8	SS	9									
178.0			9	SS	50									1 42 44 13
7.2	SILT and SAND to Silty SAND, trace to some gravel, trace to some clay, oxidation staining (TILL) Very dense Brown Dry to moist		10	SS	51									
			11	SS	60									
			12	SS	127									18 47 30 5
171.9			13	SS	100/0.07									
13.3	SILT, trace to some clay, trace to some sand Very dense Brown to grey Moist to wet													

Continued Next Page

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

GTA-MTO 001 H:\PROJECTS\2009\09-1111-6007 (URS, TORONTO)\GINT\09-1111-6007.GPJ GAL-GTA.GDT 4/27/16 CD

PROJECT <u>09-1111-6007</u>		RECORD OF BOREHOLE No AR15-6				SHEET 2 OF 2		METRIC	
G.W.P. <u>2087-13-00</u>		LOCATION <u>N 4844689.4 ;E 311040.5</u>				ORIGINATED BY <u>QC</u>			
DIST <u>Central</u> HWY <u>401</u>		BOREHOLE TYPE <u>CME 75 Truck-mount, 215 mm O.D. Hollow Stem Augers</u>				COMPILED BY <u>AJS</u>			
DATUM <u>Geodetic</u>		DATE <u>October 8, 2015</u>				CHECKED BY <u>JMAC</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/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)								
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	20	40	60	80	100	W _p	W			W _L							
169.5	--- CONTINUED FROM PREVIOUS PAGE ---	14	SS	168	170							o			0 3 86 11									
15.7	END OF BOREHOLE NOTE: 1. Borehole dry upon completion of drilling. 2. Water level in standpipe piezometer measured as follows: <table style="margin-left: 40px;"> <tr> <td>Date</td> <td>Depth (m)</td> <td>Elev. (m)</td> </tr> <tr> <td>11/01/15</td> <td>12.6</td> <td>172.6</td> </tr> <tr> <td>12/14/15</td> <td>12.7</td> <td>172.5</td> </tr> </table>	Date	Depth (m)	Elev. (m)	11/01/15	12.6	172.6	12/14/15	12.7	172.5														
Date	Depth (m)	Elev. (m)																						
11/01/15	12.6	172.6																						
12/14/15	12.7	172.5																						

GTA-MTO 001 H:\PROJECTS\2009\09-1111-6007 (URS, TORONTO)\GINT\09-1111-6007.GPJ GAL-GTA.GDT 4/27/16 CD



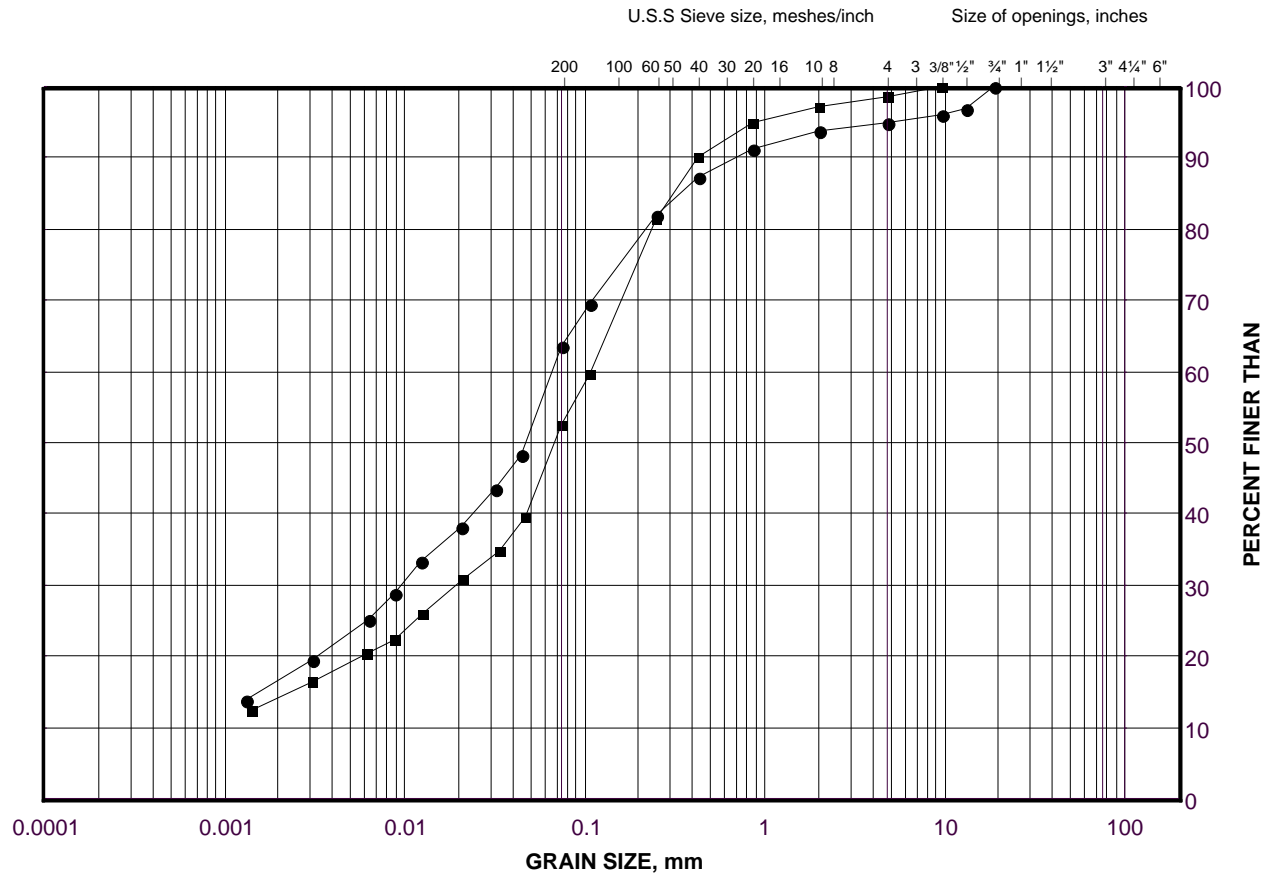
APPENDIX B

Laboratory Test Results

GRAIN SIZE DISTRIBUTION

Clayey Silt with Sand (Fill)

FIGURE B1



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

LEGEND

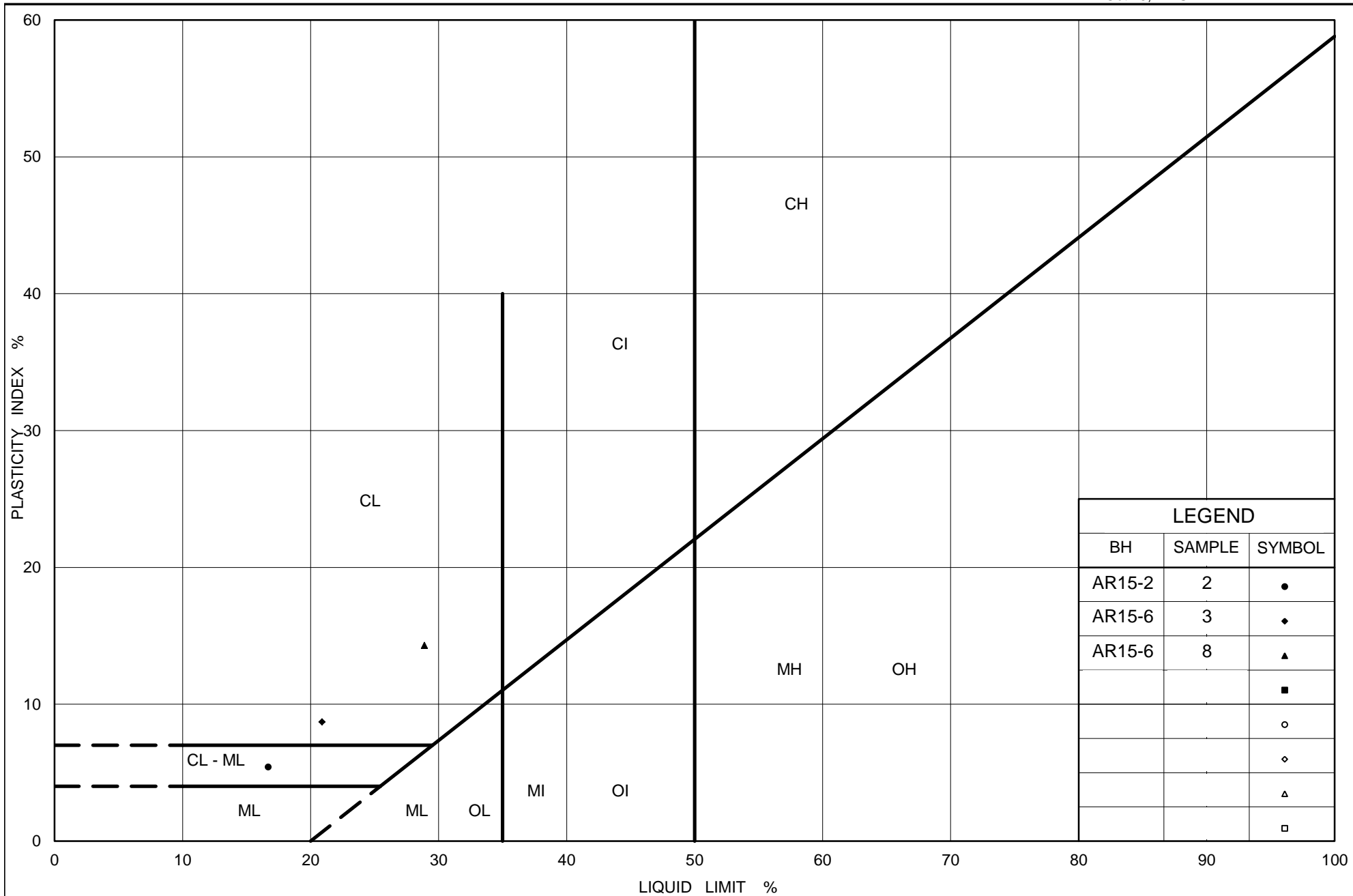
SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	AR15-1	4	181.9
■	AR15-6	6	181.08

Project Number: 09-1111-6007

Checked By: NK

Golder Associates

Date: 28-Jan-16



Ministry of Transportation

Ontario

PLASTICITY CHART Clayey Silt with Sand (Fill)

Figure No. B2

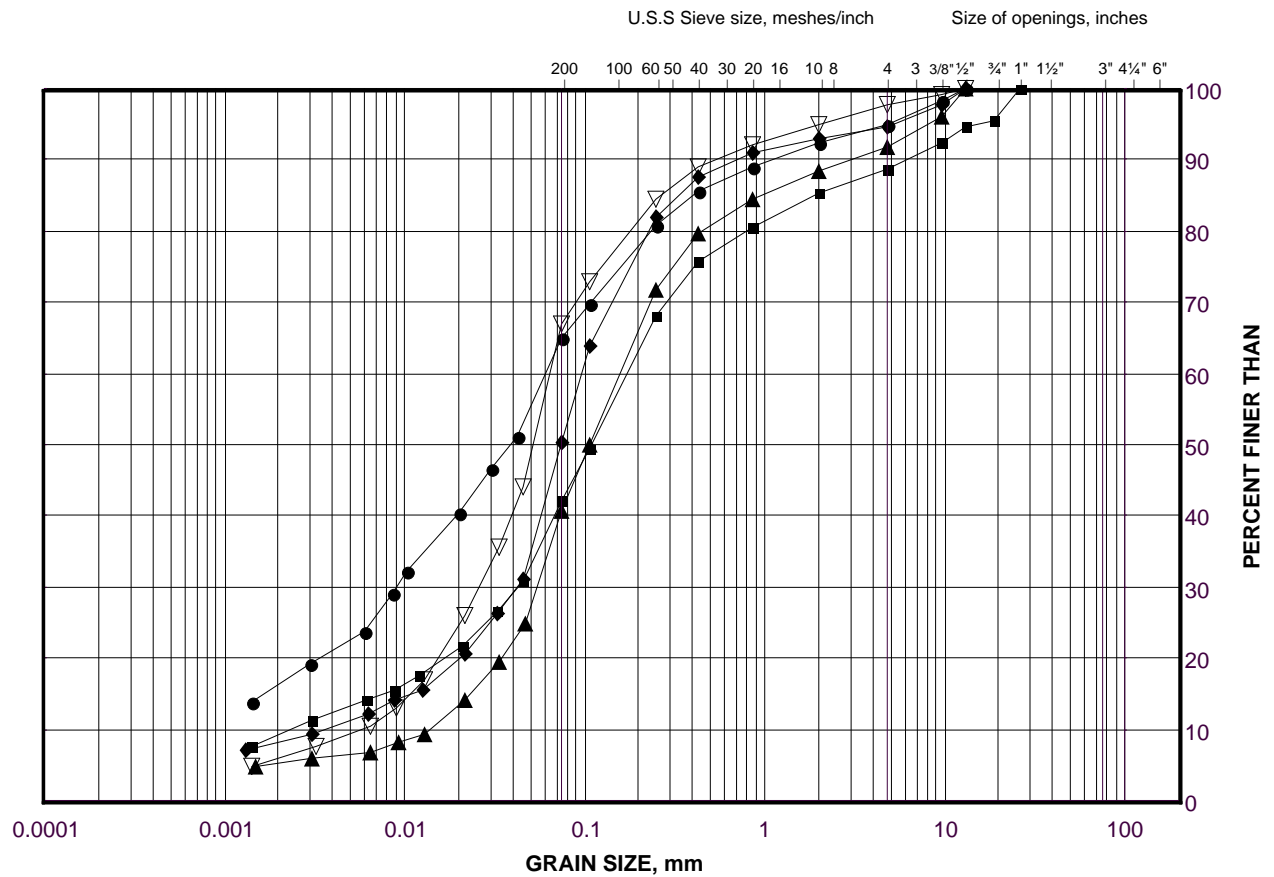
Project No. 09-1111-6007

Checked By: NK

GRAIN SIZE DISTRIBUTION

Silt and Sand (Till)

FIGURE B3A



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	AR15-2	11B	167.0
■	AR15-5	3	176.7
◆	AR15-5	5	175.1
▲	AR15-4	7	173.7
▽	AR15-2	9	170.2

Project Number: 09-1111-6007

Checked By: NK

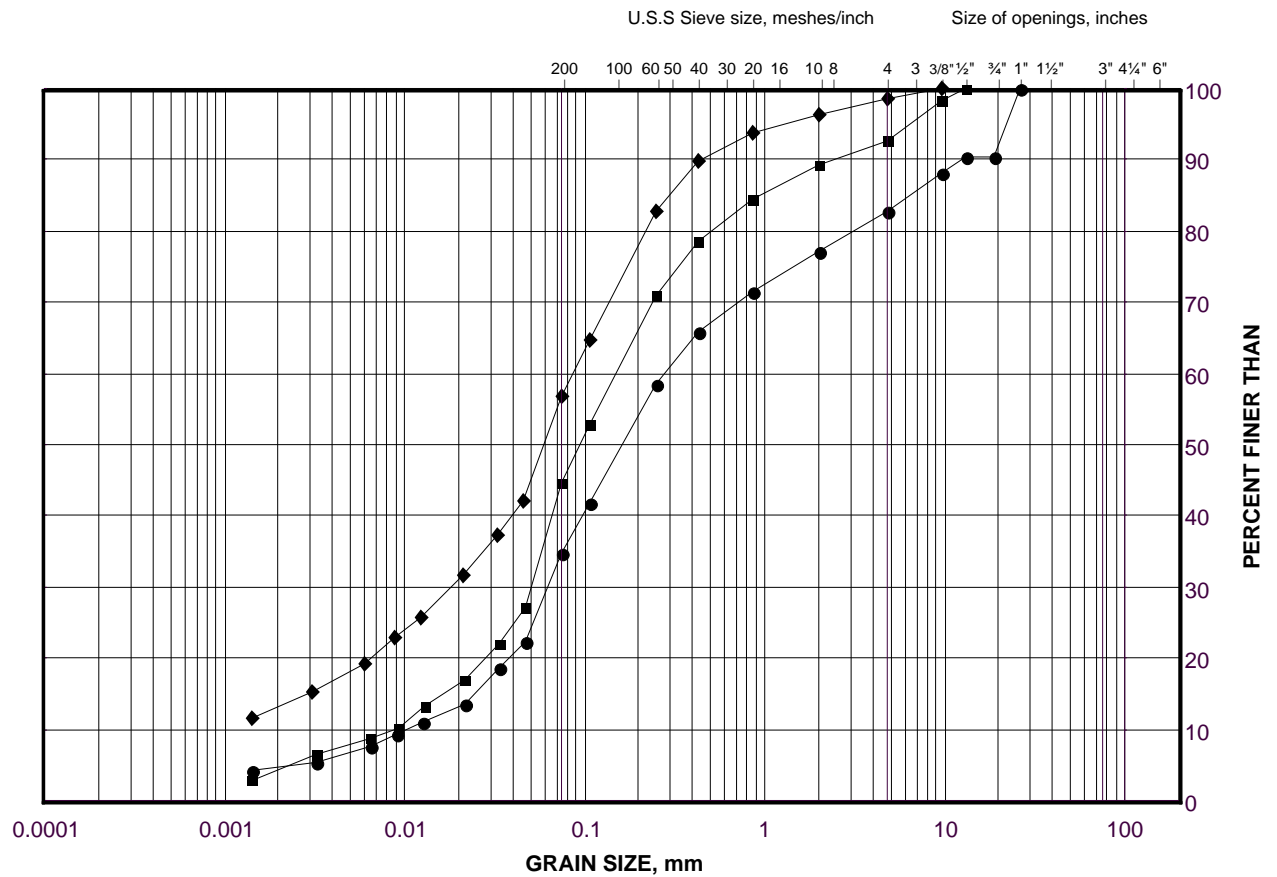
Golder Associates

Date: 28-Jan-16

GRAIN SIZE DISTRIBUTION

Silt and Sand to Silty Sand (Till)

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)
●	AR15-6	12	172.7
■	AR15-5	8	172.1
◆	AR15-6	9	177.3

Project Number: 09-1111-6007

Checked By: NK

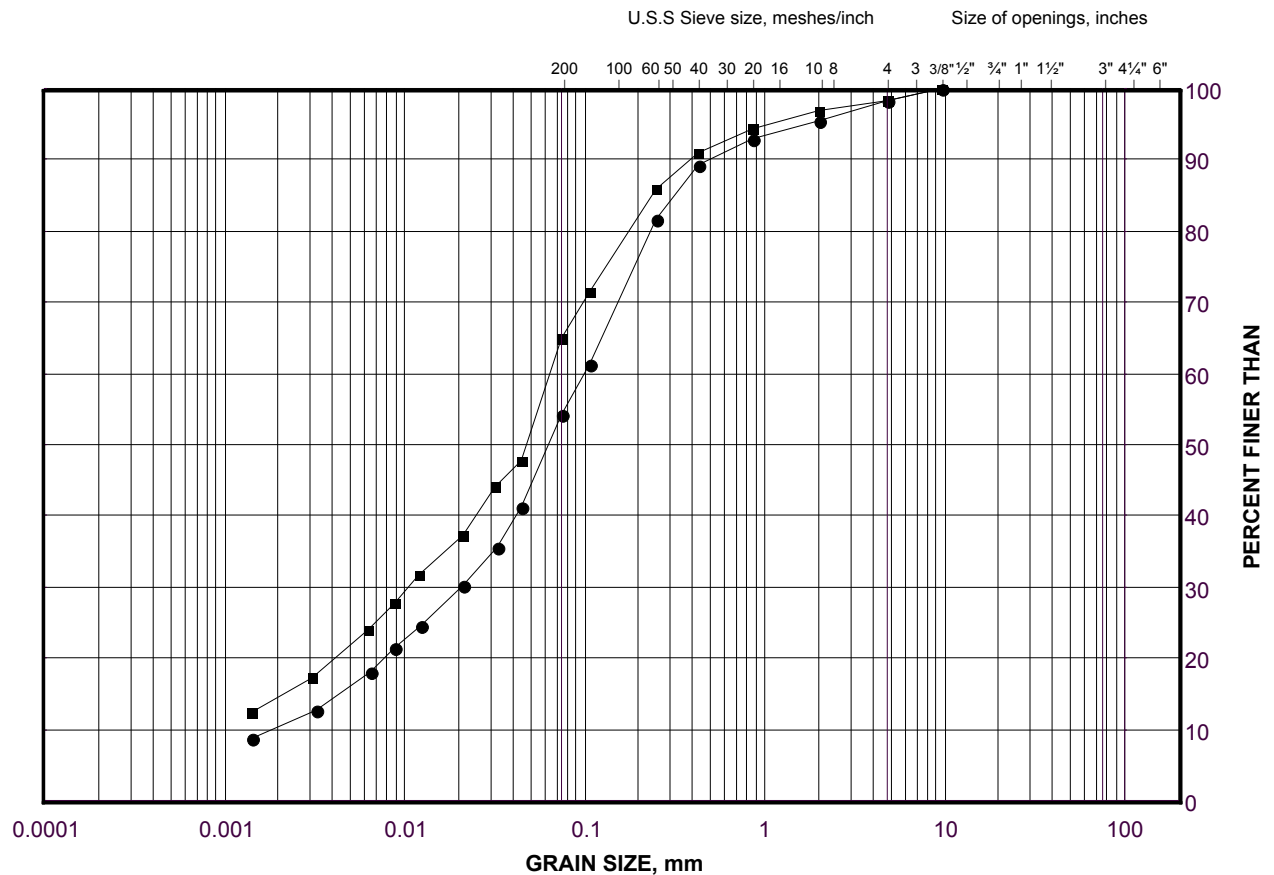
Golder Associates

Date: 28-Jan-16

GRAIN SIZE DISTRIBUTION

Clayey Silt with Sand (Till)

FIGURE B4



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

LEGEND

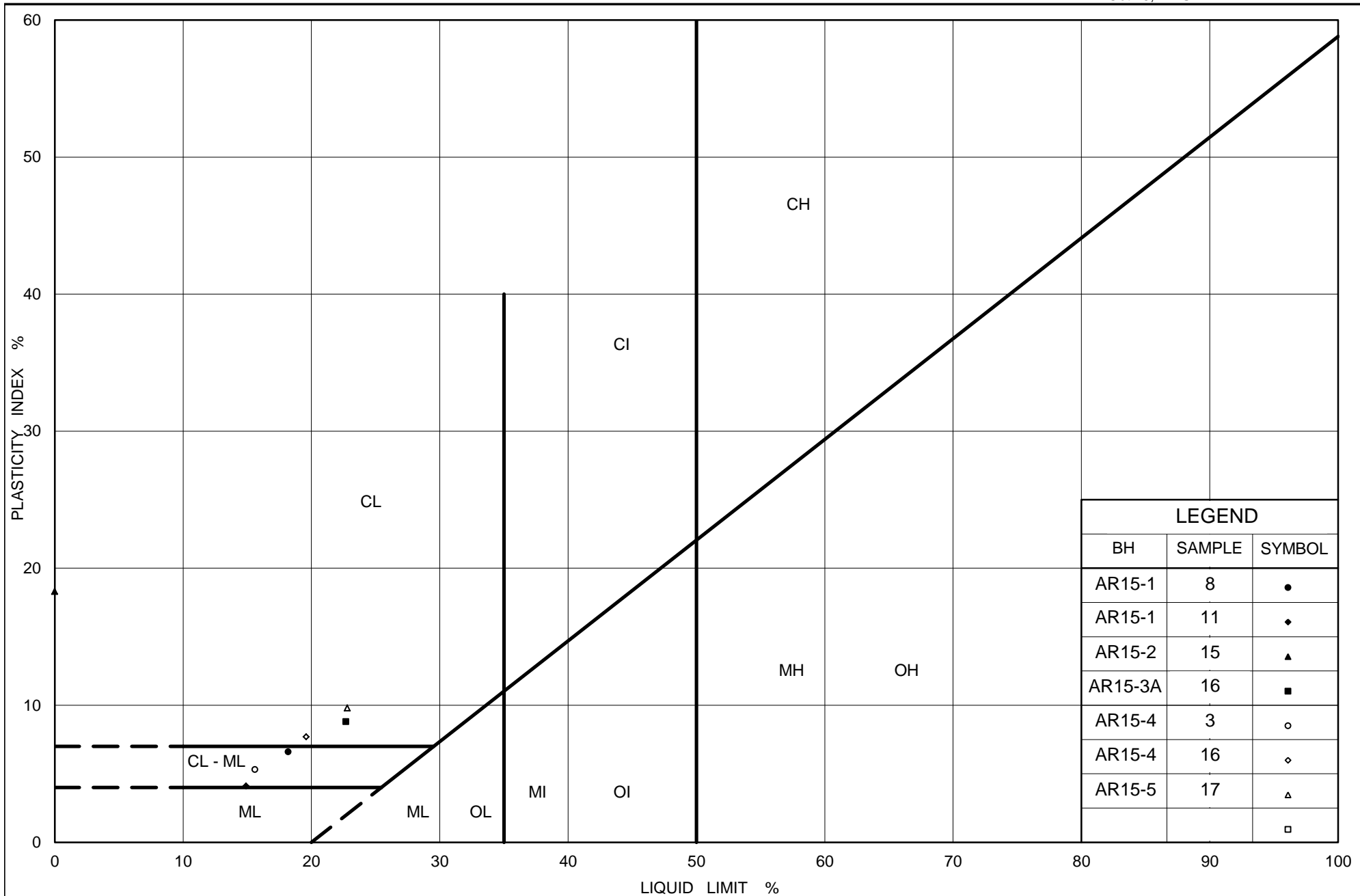
SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	AR15-4	3	182.8
■	AR15-1	8	169.1

Project Number: 09-1111-6007

Checked By: NK

Golder Associates

Date: 28-Jan-16



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

Figure No. B5

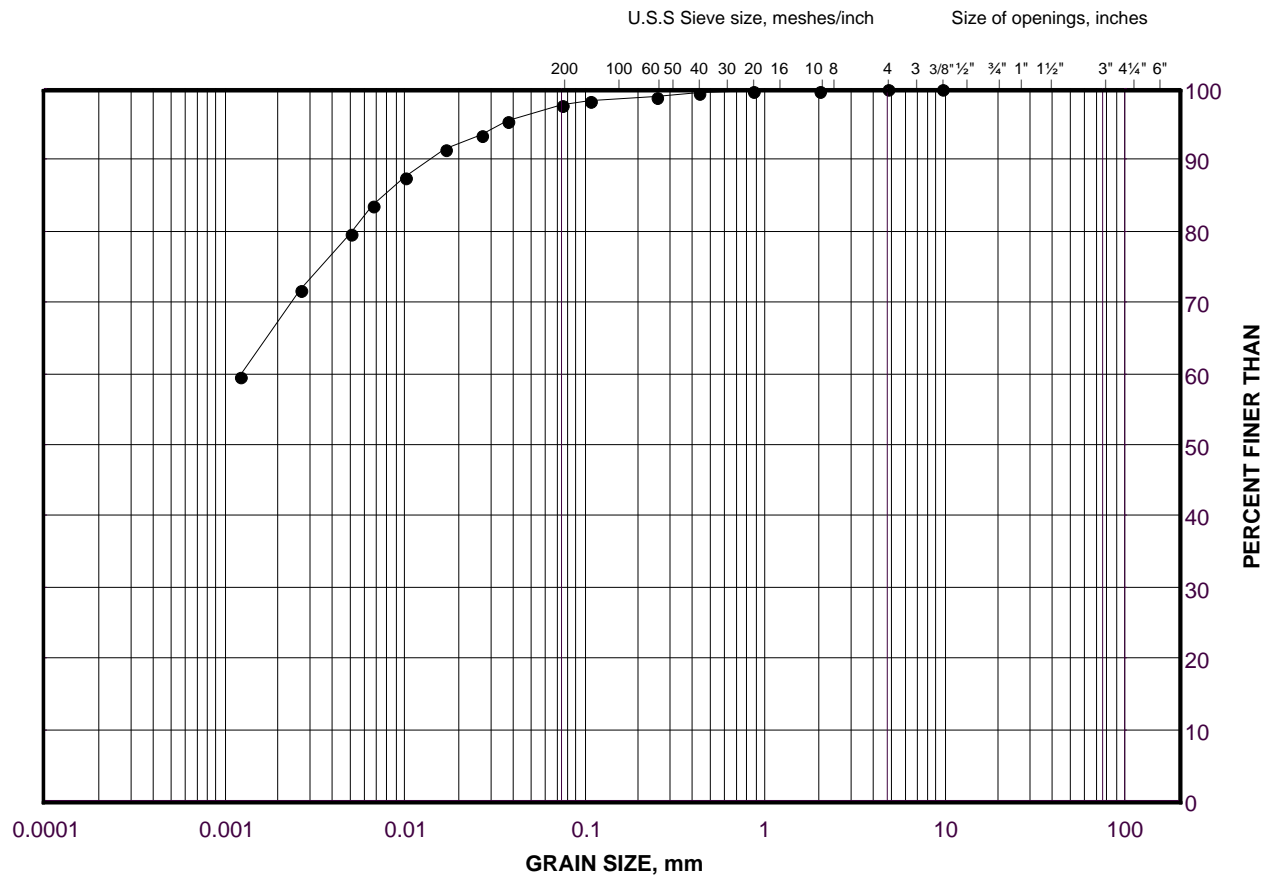
Project No. 09-1111-6007

Checked By: NK

GRAIN SIZE DISTRIBUTION

Silty Clay

FIGURE B6



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

LEGEND

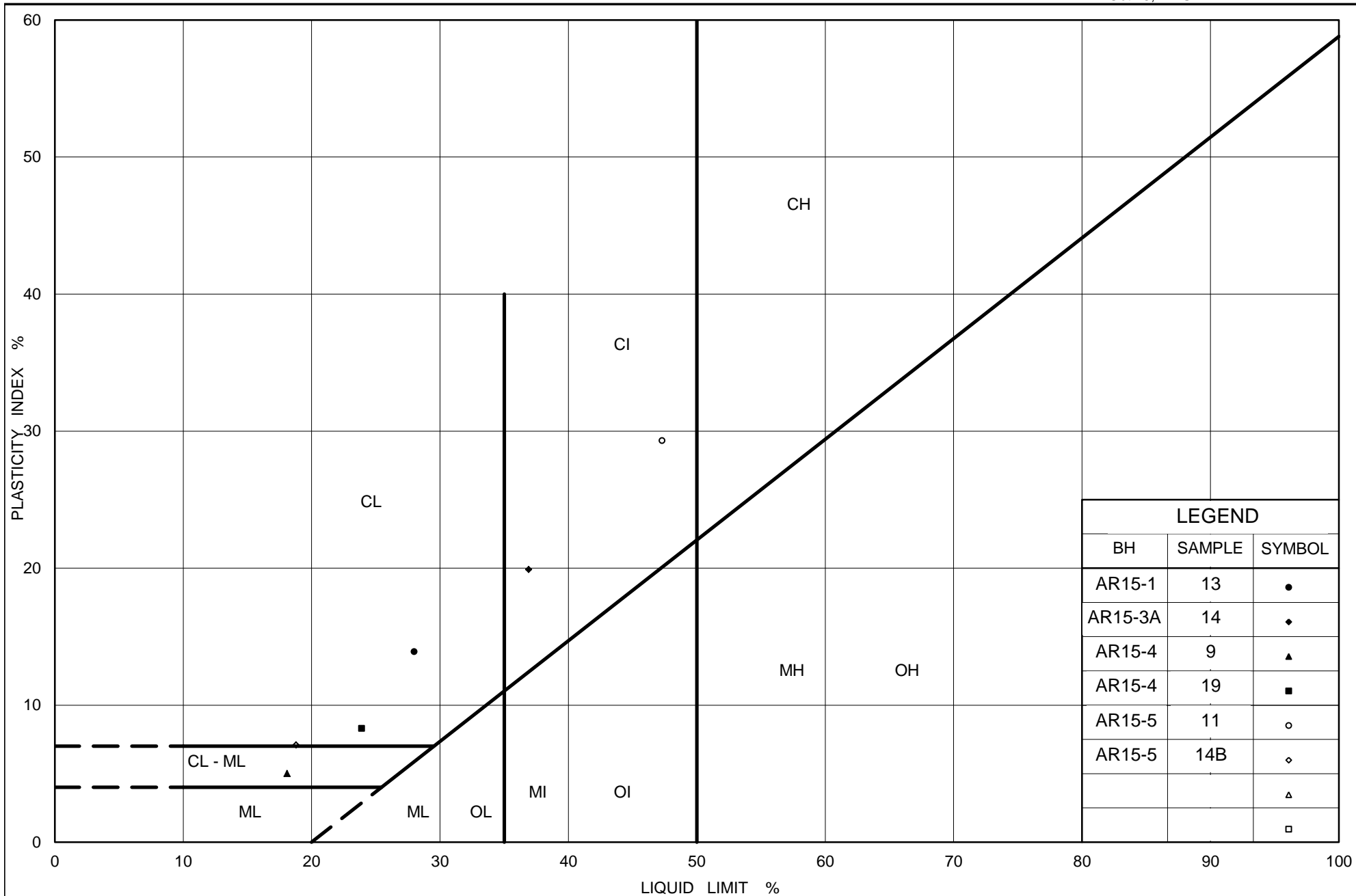
SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
•	AR15-5	11	167.54

Project Number: 09-1111-6007

Checked By: NK

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Date: 28-Jan-16



Ministry of Transportation

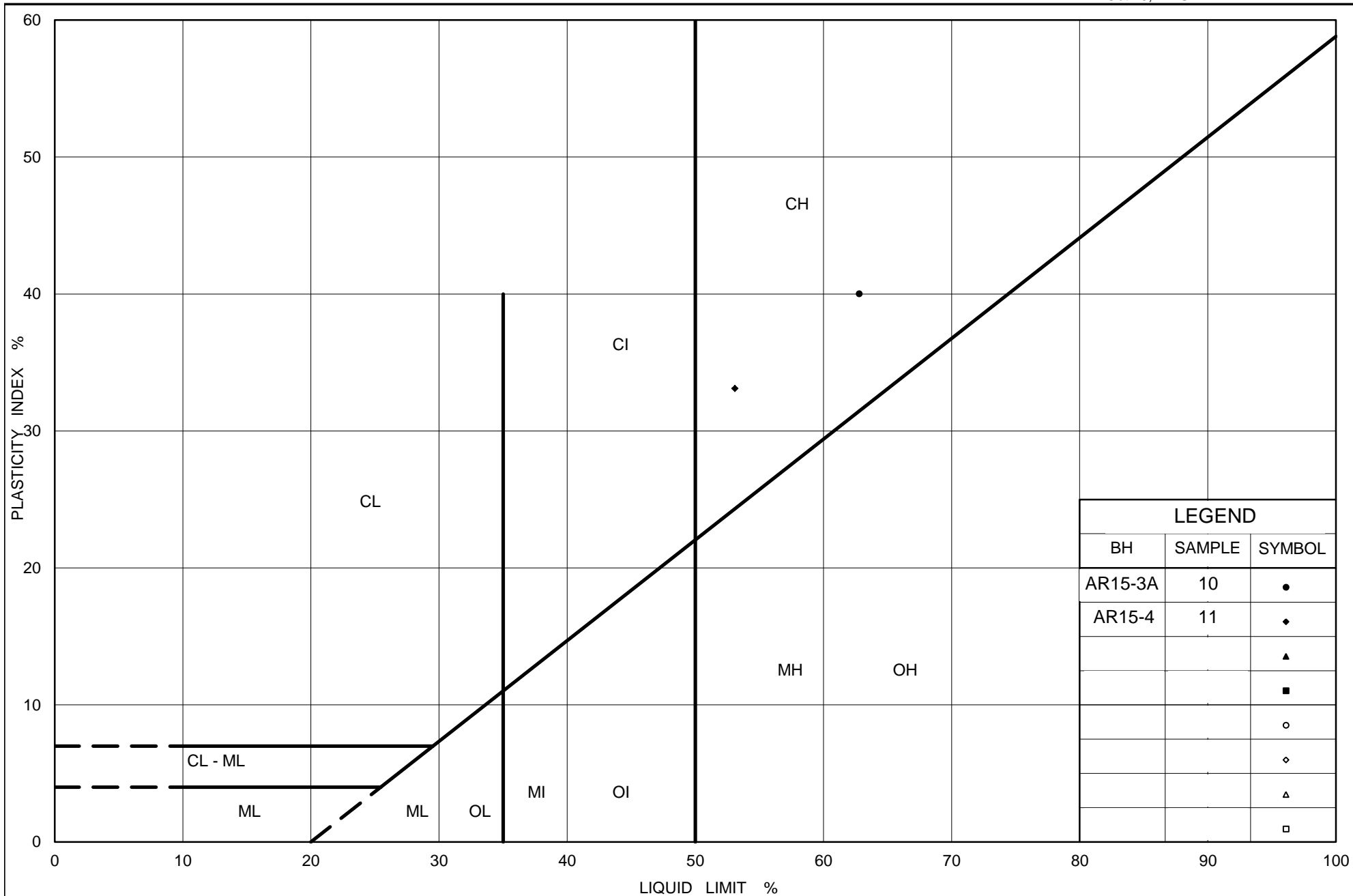
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PLASTICITY CHART Clayey Silt to Silty Clay

Figure No. B7

Project No. 09-1111-6007

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PLASTICITY CHART Clay

Figure No. B8

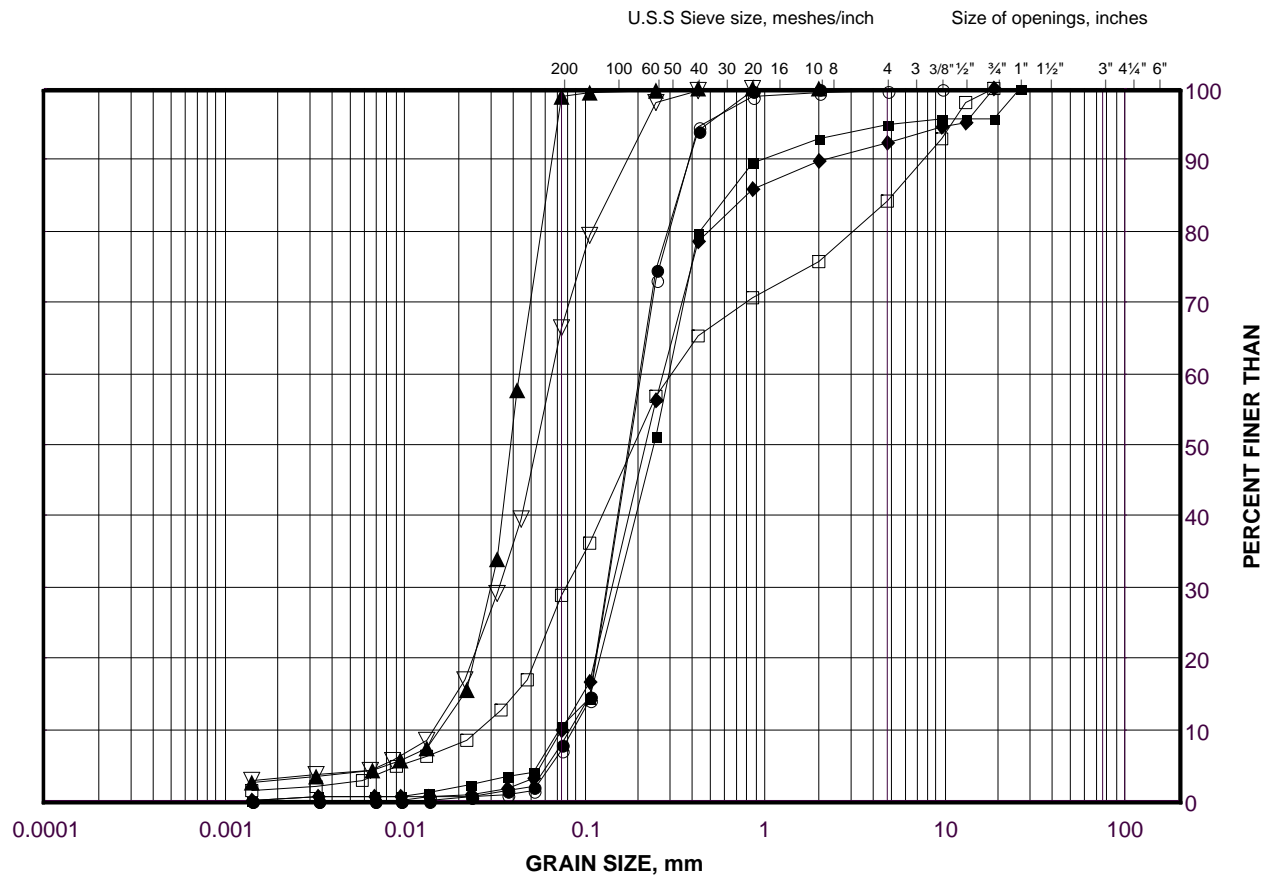
Project No. 09-1111-6007

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GRAIN SIZE DISTRIBUTION

Silt to Sand

FIGURE B9A



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	AR15-1	10	175.05
■	AR15-3A	12	166.00
◆	AR15-2	13	164.07
▲	AR15-3A	17	154.01
▽	AR15-3A	5	175.14
○	AR15-2	5	174.74
□	AR15-3A	8A	172.25

Project Number: 09-1111-6007

Checked By: NK

Golder Associates

Date: 28-Jan-16

Silt to Sand

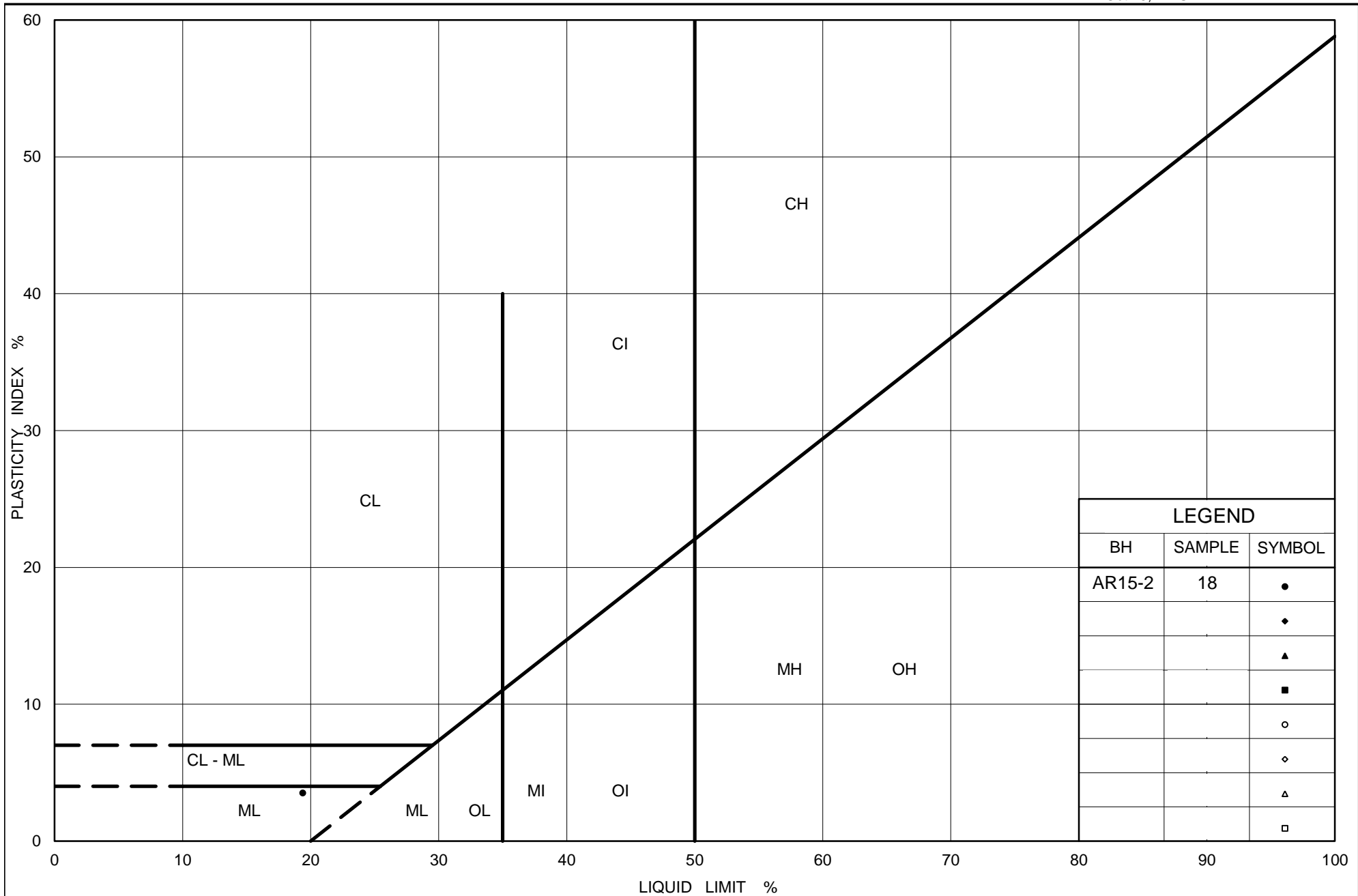
FIGURE B9B



SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	AR15-5	13	164.5
■	AR15-4	13	164.6
◆	AR15-6	14	169.7
▲	AR15-5	18B	150.5
▽	AR15-4	21	149.5

Checked By: NK

Date: 28-Jan-16



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Ontario

PLASTICITY CHART Silt

Figure No. B10

Project No. 09-1111-6007

Checked By: NK



APPENDIX C

**Record of Boreholes from Previous Investigation (GEOCRES No.:
30M11-073)**

Borehole No. 73-1

GEOTECHNICAL DATA SHEET FOR BOREHOLE ONE.

OUR REFERENCE NO. 2-2-26

CLIENT: DEPARTMENT OF HIGHWAYS, ONTARIO
 PROJECT: AVENUE ROAD, HWY 401 UNDERPASS
 LOCATION: TORONTO, DISTRICT #6
 DATUM ELEVATION: 604.10

METHOD OF BORING: AUGER
 DIAMETER OF BORING: 3 1/2"
 DATE: FEB. 27, 1962.

ENCLOSURE NO. TWO

ELEVATION ft.	DEPTH ft.	STRATIFICATION DESCRIPTION	STRATIFICATION SYMBOL	SAMPLES			PENETRATION RESISTANCE Blows per foot	SHEAR STRENGTH lbs. sq. ft.	CONSISTENCY Water content % PL W LI	REMARKS
				NUMBER	TYPE	N No. increments of Sample				
604.10	0									
600	5	BROWN MOIST TO WET CLAYEY SILT WITH SAND FILL [ML]		1	SS	13	0			
595	10			2	SS	5	0			
590	15	BROWN DAMP STIFF CLAYEY SILT WITH SAND [ML]		3	SS	13	0			
585	20	BROWN DAMP VERY DENSE SANDY SILT WITH EMBEDDED FINE ANGULAR GRAVEL [SM-ML]		4	SS	83				
580	25			5	SS	>100				
575	30	DAMP FINE SAND [SP]		6	SS	>100				

SS DENOTES
SPLIT SPOON
SAMPLE

Borehole No. 73-2

GEOTECHNICAL DATA SHEET FOR BOREHOLE T.W.O

OUR REFERENCE NO. 2-2-26

CLIENT: DEPARTMENT OF HIGHWAYS, ONTARIO
PROJECT: AVENUE ROAD, HWY 401 UNDERPASS
LOCATION: TORONTO, DISTRICT #6
DATUM ELEVATION: 582.55

METHOD OF BORING: AUGER
DIAMETER OF BOREHOLE: 3 1/2"
DATE: FEB. 28, 1962.

ENCLOSURE NO. THREE

ELEVATION ft.	DEPTH ft.	STRATIFICATION DESCRIPTION	STRATIFICATION SYMBOL	SAMPLES			PENETRATION RESISTANCE blows per foot		CONSISTENCY water content %		REMARKS
				NUMBER	TYPE	N- or Advancement of Sample	0	20	40	60	
582.55	0										
		fill									
580											
	5	MOIST PALE BROWN VERY DENSE FINE SAND TRACES OF FINE ANGULAR GRAVEL		1	SS	13					
575											
	10			2	SS	>100					
570											
	15			3	SS	>100					

GEOTECHNICAL DATA SHEET FOR BOREHOLE THREE

OUR REFERENCE NO 2-2-26

METHOD OF BORING: **AUGER**

DIAMETER OF BOREHOLE. **3 1/8"**

ENCLOSURE NO **FOUR**

LOCATION: **TORONTO, DISTRICT # 6**

DATE: FEB. 28, 1962.

DATUM ELEVATION. 605.28

ELEVATION ft.	DEPTH ft.	STRATIFICATION DESCRIPTION	STRATIFICATION SYMBOL	SAMPLES			PENETRATION RESISTANCE blows per foot	CONSISTENCY water content %	REMARKS
				NUMBER	TYPE	IN- or Advance- ment of Sample			
605.25	0						0 20 40 60 80 100	PL W LI	
600	5	BROWN MOIST CLAYEY SILT AND SAND FILL		1	SS	16	0		
595	10	ML		2	SS	6	0		
590	15	traces of roots probably original topsoil		3	SS	11	0		
585	20	BROWN, DAMP HARD CLAYEY SILT WITH SAND		4	SS	28	0		
580	25	BROWN SATURATED SANDY SILT VERY DENSE		5	SS	86			
		SM-ML							
575	30	BROWN FINE TO MED. SAND							



APPENDIX D

Non-Standard Special Provisions



FOUNDATION REPORT AVENUE ROAD UNDERPASS

WORKING SLAB - Item No.

Special Provision

1.0 SCOPE

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

2.0 REFERENCES

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

Ontario Provincial Standard Specifications, Construction

OPSS 902 Excavating and Backfilling - Structures

3.0 DEFINITIONS - Not Used

4.0 DESIGN AND SUBMISSION REQUIREMENTS - Not Used

5.0 MATERIALS

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

6.0 EQUIPMENT - Not Used

7.0 CONSTRUCTION

7.01 Excavation

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

7.02 Protection of Founding Soil

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

7.03 Dewatering

Dewatering shall be carried out according to OPSS 902.

8.0 QUALITY ASSURANCE - Not Used

9.0 MEASUREMENT FOR PAYMENT - Not Used



FOUNDATION REPORT AVENUE ROAD UNDERPASS

10.0 BASIS OF PAYMENT

10.01 Working Slab - Item

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

END OF SECTION



FOUNDATION REPORT AVENUE ROAD UNDERPASS

VIBRATION MONITORING - Item No.

Non-Standard Special Provision

Scope

This special provision describes requirements for vibration monitoring during piling / caisson installation works for the replacement of the Highway 401 – Avenue Road Underpass structure.

References

The subsurface conditions at the site are described in the following Foundation Investigation Report for G.W.P 2131-01-00:

Foundation Investigation Report, Avenue Road Underpass, Highway 401 Eastbound Collector Rehabilitation from Jane Street to Avenue Road, Toronto, Ontario, GWP 2131-01-00, Agreement No. 2009-E-0011, GEOCREC No. 30M11-261.

Definitions

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

Submission Requirements

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

- Qualifications of vibration monitoring specialist.
- Details regarding proposed instrumentation.
- Proposed location of instruments on existing Highway 401 and adjacent structure.
- Proposed frequency of readings.
- Proposed methods for adjusting piling methods if readings show vibrations exceeding tolerable levels.

Monitoring

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

The vibrations measured on the existing structure shall not exceed 100 mm/s (peak particle velocity) for permanent components of the bridge that will remain as part of the bridge rehabilitation option.



FOUNDATION REPORT AVENUE ROAD UNDERPASS

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

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

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

Basis of Payment

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

END OF SECTION



CSP FOR INTEGRAL ABUTMENTS – Item No

Non-Standard Special Provision

Scope

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

Submission and Design Requirements

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

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

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

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

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

Material

Corrugated steel pipe

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

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

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



FOUNDATION REPORT AVENUE ROAD UNDERPASS

Sand Fill

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

Table 1 – Sand Fill Gradation Requirements

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

Construction

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

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

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

The Contractor shall ensure the full perimeters of the tops of all CSPs at each abutment are at the elevation and orientation shown on the working drawings.

The CSP at each pile shall be constructed to the following tolerances:

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



FOUNDATION REPORT AVENUE ROAD UNDERPASS

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

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

Basis of Payment

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

END OF SECTION



FOUNDATION REPORT AVENUE ROAD UNDERPASS

OBSTRUCTIONS - Item No.

Non-Standard Special Provision

It should be anticipated that the till deposits contain cobbles and boulders and some soil deposits at this site contain cobbles and likely boulders as indicated in the Record of Borehole sheets as inferred from difficulties in advancing augers/auger grinding. Consideration of the presence of these obstructions must be made in the selection of appropriate equipment and procedures for driving steel H-piles/ pipe piles or caissons and possible pre-augering for deep foundations.

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



FOUNDATION REPORT AVENUE ROAD UNDERPASS

DEEP FOUNDATIONS - Item No.

Non-Standard Special Provision

Amendment to OPSS 903

903.07.02.07 Monitoring Driven Piles

903.07.02.07.04 Wave Equation Analysis

Section 903.07.02.07.04 is amended by the addition of the following:

The Contractor shall complete pile dynamic analyzer (PDA) testing on a minimum of 20 per cent of piles at each foundation element for each stage of construction, in conjunction with re-tapping of piles in accordance with Section 903.07.02.07.06. The piles to be assessed by PDA testing shall be distributed evenly across the foundation element. The piles subjected to PDA testing shall be selected by the Contractor based on their piling operation works and schedule, subject to agreement by the Contract Administrator. All piles subjected to PDA testing shall also be tested using the Hiley formula.

The results of the PDA testing should be reviewed by a qualified professional.

If the results of the PDA testing and the Hiley testing differ, the contract administrator will determine which test results govern in the field.

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