



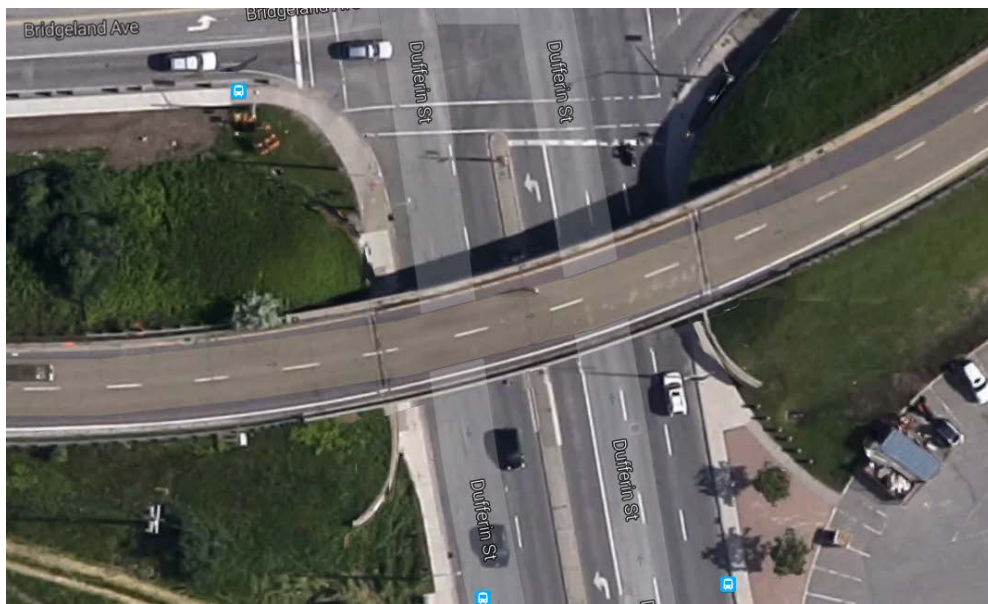
April 28, 2016

FOUNDATION INVESTIGATION AND DESIGN REPORT

**HIGHWAY 401 W - YORKDALE ROAD RAMP OVER
DUFFERIN STREET (SITE NO. 37-284)
HIGHWAY 401 EASTBOUND COLLECTOR
REHABILITATION FROM JANE STREET
TO AVENUE ROAD
GWP 2131-01-00, AGREEMENT NO. 2009-E-0011**

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REPORT





FOUNDATION REPORT HIGHWAY 401 W - YORKDALE ROAD RAMP (SITE NO. 37-284) OVER DUFFERIN STREET

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PART A

**FOUNDATION INVESTIGATION REPORT
HIGHWAY 401 W - YORKDALE ROAD RAMP OVER DUFFERIN STREET
(SITE NO. 37-284)
HIGHWAY 401 EBC REHABILITATION FROM
JANE STREET TO AVENUE ROAD
TORONTO, ONTARIO
G.W.P. 2131-01-00, AGREEMENT NO. 2009-E-0011**



FOUNDATION REPORT HIGHWAY 401 W - YORKDALE ROAD RAMP (SITE NO. 37-284) OVER DUFFERIN STREET

1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by AECOM on behalf of the 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 Highway 401 W - Yorkdale Road ramp over Dufferin Street.
- Phase 2: Detail Foundation Investigation at various bridge structures, including the Highway 401 W - Yorkdale Road ramp over Dufferin Street.

This report addresses the Phase 2 Detail Foundation Investigation for the proposed replacement of Highway 401 W - Yorkdale Road Ramp over Dufferin Street (Site No. 37-284).

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 July 30, 2014.

Subsurface information from previous investigations associated with the Highway 401 W - Yorkdale Road ramp over Dufferin Street structure was obtained from the MTO Geocres library and AECOM as follows:

- MTO GEOCREs No. 30M11-081: Report titled "Hwy #401, Spadina Expressway Bridge #2, Twp. Of North York, Cty. Of York. District No. 6, W.J. 63-F-24 A - W.P. 233-61-2-2," prepared by the Department of Highways – Ontario, Materials and Research Section, dated May 24, 1963.
- Design Drawings by the Department of Highways Ontario – Bridge Division, titled "Spadina Bridge # 2 – Hwy No. 401", T.W.P. 72-284-1-A, dated March 1963; provided by AECOM.
- General Arrangement Drawings for Dufferin #37-284, titled "Ramp 401W – Yorkdale E/W over Dufferin Bridge R" received November 3rd, 2015, provided by AECOM.

2.0 SITE DESCRIPTION

The existing Highway 401 W – Yorkdale Road ramp over Dufferin Street is a two-span structure supported on spread/strip footings founded at shallow depth. The bridge structure is approximately 32 m long and 10.5 m wide. Based on the 1963 drawings, the natural ground surface at this site is approximately Elevation 188 m with Dufferin Street constructed at a grade of approximately Elevation 189 m. The Highway 401 W – Yorkdale Road ramp slopes down from west to east from approximately Elevations 195 m to 194 m.

3.0 INVESTIGATION PROCEDURES

The field work for the current foundation investigation was carried out between September 28 and November 2, 2015, at which time a total of six boreholes (Boreholes DS15-1 to DS15-6 and DS15-6A) were advanced using a CME 75 truck-mounted drill rig, supplied and operated by Geo-Environmental Drilling Inc. of



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Halton Hills, Ontario and Davis Drilling Inc. of Milton, Ontario. Borehole DS15-6 was terminated at a depth of 2.9 m below the Dufferin street roadway surface due to proximity to an existing storm sewer as inferred by the presence of sand fill in the SPT samples. Borehole DS15-6A was advanced on the east side of the pier to the desired drill depth. Two boreholes (designated 3A and 3B in GEOCREs 30M11-081) advanced as part of a previous investigation at the Highway 401 W – Yorkdale Road ramp site are also used in this report to supplement the current investigation, and have been renumbered to show the MTO GEOCREs reference number followed by the original borehole designation (i.e., 81-3A and 81-3B). Boreholes DS15-1 to DS15-6, DS15-6A, 81-3A and 81-3B pertaining to the existing bridge structure and proposed new bridge structure, were advanced at the locations shown on Drawing 1.

Boreholes DS15-1 to DS15-4 and DS15-6 were advanced using 215 mm outside diameter hollow stem augers, while Boreholes BS15-5 and BS15-6A were advanced using 215 mm outside diameter hollow stem augers and, 125 mm outside NW casing using Tricone and wash boring techniques. Soil samples were obtained in the boreholes at 0.75 m and 1.5 m intervals of depth using a 50 mm outer diameter split-spoon sampler 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 in the open boreholes were observed during and immediately following the drilling operations. A standpipe piezometer was installed in each of the Boreholes DS15-1 and DS15-2 to permit monitoring of the groundwater level at the site. The piezometers 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 the ground surface with bentonite pellets and/or cement grout. The details of the piezometer installation are shown on the Records of Boreholes DS15-1 and DS15-2. The remaining boreholes were backfilled to immediately below ground surface with bentonite upon completion, in accordance with Ontario Regulation 903 (as amended). The boreholes advanced through the road surface were sealed at the surface with cold patch asphalt, approximately 0.2 m thick.

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. 30M11-081).

Borehole No.	MTM NAD83 Northing (m)	MTM NAD83 Easting (m)	Ground Surface Elevation (m)	Borehole Depth (m)
DS15-1	4,842,970.0	308,197.3	195.1	11.3
DS15-2	4,842,984.8	308,242.3	194.5	14.3
DS15-3	4,842,975.9	308,205.3	189.5	17.1
DS15-4	4,842,977.3	308,212.1	189.5	18.8



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Borehole No.	MTM NAD83 Northing (m)	MTM NAD83 Easting (m)	Ground Surface Elevation (m)	Borehole Depth (m)
DS15-5	4,843,006.0	308,237.8	189.0	17.2
DS15-6	4,842,965.0	308,214.2	189.2	2.9
DS15-6A	4,842,967.4	308,219.9	189.3	15.7
81-3A*	4,842,961.1	308,202.8	189.0	9.6
81-3B*	4,842,969.8	308,237.0	189.0	6.6

* Approximate borehole locations obtained from the Digital Terrain Model as plotted relative to centerline of Highway 401 and the existing Highway 401 W – Yorkdale Road ramp structure.

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 DS15-1 to DS15-6 and DS15-6A) were advanced at the existing Highway 401 W – Yorkdale Road ramp 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 the laboratory testing are provided on the borehole records contained in Appendix A; the results of the geotechnical laboratory testing are presented on Figures B1 to B6 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.

¹ Chapman, L.J. and Putnam, 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 general, the subsurface conditions at the site consist of surficial layers of asphalt and fill of varying thickness (an upper thin layer of granular material and a lower layer of clayey silt), underlain by a stiff to hard clayey silt to silty clay glacial till deposit.

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

4.2.1 Asphalt, Concrete and Topsoil

An approximately 150 mm thick layer of asphalt was encountered immediately below the ground surface in all the boreholes drilled during the current investigation. An approximately 250 mm to 300 mm thick layer of concrete was encountered below the asphalt in all the boreholes drilled on the Dufferin Street road surface (DS15-3, DS15-4, DS15-6 and DS15-6A). Approximately 200 mm of topsoil was encountered immediately below the ground surface in Boreholes 81-3A and 81-3B drilled as part of the previous investigation.

4.2.2 Fill

Fill materials were encountered underlying the asphalt or concrete in all the boreholes drilled as part of the current investigation (Boreholes DS15-1 to DS15-6 and DS15-6A). The extent and composition of the fill varies depending on the location of the boreholes, with thicker cohesive layers encountered at the approach embankments (Boreholes DS15-1 and DS15-2) and thinner cohesive layers encountered in Boreholes DS15-3, DS15-5 and DS15-6A. Borehole DS15-6 encountered a 2.1 m thick layer of sand fill inferred to be backfill to the local sewer pipe. The surface of the fill layer was encountered between Elevations 194.9 m and 188.8 m. The thickness of the fill layer varies between about 1.5 m and 3.0 m in the boreholes drilled along the Dufferin Street road surface (Boreholes DS15-3, DS15-4, DS15-5, DS15-6 and DS15-6A), and is 7.0 m and 6.8 m thick in Boreholes DS15-2 and DS15-1, respectively, drilled along the approach embankments. The base of the fill encountered in all boreholes is between Elevations 188.0 m and 186.3 m. Borehole DS15-6 terminated within the fill layer, penetrating it for a thickness of 2.5 m.

The fill material is variable in composition and comprised of an upper layer of sand and gravel to gravelly sand to sand below the asphalt and/or concrete, and a lower layer of cohesive material consisting of clayey silt to sandy clayey silt. Trace organics were encountered in Boreholes DS15-2 and DS15-4 within the fill layer, extending to about Elevation 190 m at the east abutment (Borehole DS15-2) and to about Elevation 188 m at the pier location (Borehole DS15-4).

The Standard Penetration Test (SPT) "N"-values measured within the non-cohesive fill generally range from 11 blows to 37 blows per 0.3 m of penetration, indicating a compact to dense relative density. The SPT "N"-value measured within the cohesive fill range from 2 blows to 20 blows per 0.3 m of penetration, suggesting that the clayey silt to sandy silty clay fill has a very soft to very stiff consistency.

Grain size distribution tests were carried out on four (4) samples of the clayey silt to sandy clayey silt fill and the results are provided on Figure B1 in Appendix B.

Atterberg limits testing was carried out on four (4) samples of the cohesive fill and measured plastic limits ranging between 13 per cent and 15 per cent, liquid limits ranging between 26 per cent and 33 per cent, and corresponding plasticity indices ranging between 13 per cent and 18 per cent. These test results, which are plotted on Figure B2 in Appendix B, indicate that the cohesive fill material consists of clayey silt of low plasticity.



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The laboratory water content measured on selected samples of the non-cohesive fill ranges from 7 per cent to 20 per cent, and the laboratory water content measured on selected samples of the cohesive fill ranges from 12 per cent to 28 per cent.

4.2.3 Clayey Silt

An approximately 1.5 m thick stratum of clayey silt, trace sand to sandy, was encountered underlying the cohesive fill in borehole DS15-1. The surface of this deposit was encountered at a depth of 7.2 m below ground surface, corresponding to Elevation 187.9.

The SPT "N"-value measured within the clayey silt deposit is 20 blows per 0.3 m of penetration, suggesting a very stiff consistency

4.2.4 Sandy Clayey Silt to Clayey Silt with Sand Till

A till deposit comprised of sandy clayey silt to clayey silt with sand was encountered underlying the fill Boreholes DS15-1 to DS15-5 and DS15-6A from the current investigation and underlying the topsoil in the boreholes from the previous investigation (Boreholes 81-3A and 81-3B). All boreholes from the current and previous investigation, with the exception of Borehole DS15-5 and DS15-6, were terminated within this deposit. 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.	Depth to Surface of Deposit (m)	Surface Elevation of Deposit (m)	Thickness of Deposit (m)	Base Elevation of Deposit (m)	Deposit Description
DS15-1	8.7	186.4	>2.6	Below 183.8	Sandy Clayey Silt (Till)
DS15-2	7.2	187.3	>7.1	Below 180.2	Sandy Clayey Silt to Clayey Silt with Sand (Till)
DS15-3	1.5	188.0	>15.6	Below 172.4	Sandy Clayey Silt (Till)
DS15-4	1.5	188.0	>17.3	Below 170.7	Sandy Clayey Silt to Clayey Silt with Sand (Till)
DS15-5	2.2	186.8	11.1	175.7	Sandy Clayey Silt (Till)
DS15-6A	3.0	186.3	>12.7	Below 173.6	Sandy Clayey Silt to Clayey Silt with Sand (Till)
81-3A	0.2	188.8	>9.4	Below 179.4	Silty Clay and Clayey Silt (Till)
81-3B	0.2	188.8	>6.4	Below 182.4	Clayey Silt and Silty Clay (Till)



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The cohesive till deposit generally consists of sandy clayey silt to clayey silt with sand, trace to some gravel; sand and silt seams. Cobbles and boulder are inferred present within the till deposit as noted by augur grinding and very small sample recovery coupled with greater than 100 blows SPT “N”-values, such as encountered in boreholes DS15-3 between depths of about 12.8 m and 16.8 m (corresponding to about Elevation 176.7 m and 172.7 m). A silt pocket was encountered within the till deposit at about Elevation 183 m.

The SPT “N”-values measured within cohesive till generally range from 10 blows per 0.3 m to greater than 100 blows per 0.01 m of penetration, suggesting a stiff to hard consistency.

Grain size distribution tests were carried out on twelve (12) selected samples of the sandy clayey silt to clayey silt with sand till deposit and the results are shown on Figures B3A and B3B in Appendix B.

Atterberg limits tests were carried out on thirteen (13) selected samples of the cohesive till deposit and measured liquid limits ranging between 18 per cent and 28 per cent, plastic limits ranging between 11 per cent and 14 per cent, and corresponding plasticity indices ranging between 7 per cent and 13 per cent. These results, which are plotted on a plasticity chart on Figures B4A and B4B in Appendix B, indicate that the till deposit generally consists of clayey silt of low plasticity. The result of an Atterberg Limits test of a sample of the silt pocket (from borehole DS15-6A) indicates a liquid limit of about 18 per cent, a plastic limit of about 16 per cent and a plasticity index of about 2 percent, as shown on Figure B4B, indicating that the material a silt of slight plasticity.

The natural water content measured on selected samples of the sandy clayey silt to clayey silt with sand till ranges from 7 per cent to 14 per cent.

4.2.5 Silt

An approximately 1.6 m thick silt deposit was encountered underlying the sandy clayey silt till in Borehole DS15-5. The surface of the deposit was encountered at a depth of 13.3 m below ground surface, corresponding to Elevation 175.7 m.

The SPT “N”-value measured within the silt deposit is 100 blows per 0.18 m of penetration, indicating a very dense relative density.

The natural water content measured on one sample of the silt deposit is 18 per cent.

4.2.6 Silt and Sand Till

A till deposit comprised of silt and sand was encountered below the silt deposit in Borehole DS15-5, at a depth of 14.9 m below ground surface, corresponding to Elevation 174.1 m. Borehole DS15-5 was terminated within this deposit, penetrating it for a thickness of 2.3m.

The SPT “N”-values measured within the silt and sand till deposit are 128 blows per 0.25 m of penetration and 127 blows per 0.2 m of penetration, indicating that the silt and sand till deposit has a very dense relative density.

A grain size distribution test was carried out on one selected sample of the silt and sand till deposit and the result is provided on Figure B5 in Appendix B.

An Atterberg limits test carried out on one (1) sample of the silt and sand till deposit measured a liquid limit of 15 per cent, a plastic limit of 11 per cent and a corresponding plasticity index of 4 per cent. The result of the Atterberg limits test is shown on a plasticity chart on Figure B6 in Appendix B and indicates that the deposit consists of silt of slight plasticity.



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The natural water content measured on two (2) selected samples of the silt and sand till deposit are 9 per cent and 10 per cent.

4.2.7 Groundwater Conditions

The observed/recorded water levels in the open boreholes following completion of drilling and in the piezometer (P) in Boreholes DS15-1 and DS15-2 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
DS15-1	195.1 m	dry	dry	Sept. 28, 2015
		7.1 m	188.0 m (P)	Nov. 19, 2015
		7.2 m	187.9 m (P)	Dec. 14, 2015
DS15-2	194.5 m	dry	dry	Sept. 29, 2015
		6.6 m	187.9 m (P)	Nov. 19, 2015
		6.7 m	187.8 m (P)	Dec. 14, 2015
DS15-3	189.5 m	dry	-	Oct. 12, 2015
DS15-4	189.5 m	dry	-	Oct. 13, 2015
DS15-5	189.0 m	*	-	Nov. 1, 2015
DS15-6	189.2 m	dry	-	Oct. 4, 2015
DS15-6A	189.3 m	*	-	Nov. 2, 2015
81-3A	189.0 m	6.1 m	182.9 m	Mar. 15, 1963
81-3B	189.0 m	4.5 m	184.5 m	Mar. 21, 1963

* Water level not recorded as water was introduced into the borehole as part of the drilling operations.

Based on the groundwater levels recorded during this current investigation, the water level at this site is at approximately Elevation of 187.9 m.

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 Qasim Cheema, P.Eng., a geotechnical engineer, and reviewed 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.



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QC/NK/JMAC/sm/rb

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**FOUNDATION REPORT
HIGHWAY 401 W - YORKDALE ROAD RAMP (SITE NO. 37-284)
OVER DUFFERIN STREET**

PART B

**FOUNDATION DESIGN REPORT
HIGHWAY 401 W - YORKDALE ROAD RAMP OVER DUFFERIN STREET
(SITE NO. 37-284)
HIGHWAY 401 EBC REHABILITATION FROM
JANE STREET TO AVENUE ROAD
TORONTO, ONTARIO
G.W.P. 2131-01-00, 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 Highway 401 W – Yorkdale Road ramp over Dufferin Street 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 Highway 401 W - Yorkdale Road ramp over Dufferin street structure will be replaced. Based on the design drawings provided by AECOM on 3rd November 2015, it is understood that a two-span replacement structure is proposed to be constructed approximately 7 m north of the existing Highway 401 W – Yorkdale Road ramp structure, with the new foundations located at or near the existing foundation and elevations. It is also understood that the structure is proposed to be replaced in three stages, with the north half of the existing structure demolished and the new structure constructed while one lane of traffic is maintained on the south half; traffic would then be transferred to the new structure, and the existing south half of the structure would be demolished.

6.2 Foundation Options

The existing two-span Highway 401 W – Yorkdale Road ramp structure over Dufferin Street was constructed in 1963. The structure is comprised of 14.8 m long span between the west abutment and central pier and 17.2 m long span between central pier and the east abutment. Based on the available design drawings, the abutments and pier are supported on spread footings, founded at approximate Elevation 187.4 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 drawing provided by AECOM on March 21, 2016, the proposed replacement structure is to consist of a two-span box girder with a total span length of approximately 35.4m m. The replacement structure will be constructed on a parallel alignment to the existing structure, with the new centreline shifted approximately 10 m to the north of the existing centreline. The Dufferin Street grade at the site is to be maintained at approximately Elevation 189.0 m. The grade of the proposed ramp will be raised nominally (less than approximately 0.6 m), while the existing 7 m to 8 m high approach embankments may have to be replaced/moved/widened to accommodate the centreline shift.

Based on the proposed ramp geometry and the subsurface conditions at the site, both shallow foundations and deep foundations have been considered for support of the abutments and pier for the new Highway 401 W – Yorkdale Road ramp structure over Dufferin Street. A summary of the advantages and disadvantages associated



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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 very stiff to hard sandy clayey silt to clayey silt with sand till deposits:** Spread footings are considered feasible and suitable to support the new abutments and pier given the competency of the native soils at this site and the relative cost of construction. This option however would not allow for the use of integral abutments. As the new abutments would be located to the north of the existing foundations, an approximately 7 m to 9 m deep excavation would be required below the ramp road grade to remove the existing abutment walls and footings. This proposed founding level for the abutment foundations would be near or below the groundwater level, and some limited groundwater control is expected to be required. Given the proposed location of the new pier at approximately 10 m to the north of the existing location, an approximately 2 m to 3 m of excavation would be required below the Dufferin Street road grade to remove the existing footings and construction of new footings. Footing founding levels on native sandy clayey silt till are approximately 0.9 m below the existing abutment footings and approximately 1.4 m below the existing pier footing, which would require excavations to below the existing footings. Based on the staging plan, excavations to below the existing footing level along the south side of the new structure could undermine the existing foundations, and is not recommended. Temporary protection systems will be required along the Dufferin Street 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 a 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 and very soft to soft layers within the fill will result in lower resistances and will increase the risk for future settlements. Therefore the existing fill materials should be fully excavated during removal of the existing abutment and pier foundations and the new spread footings for the abutments and piers could be founded on a Granular 'A' engineered fill pad placed on the native hard sandy clayey silt till deposit to raise the sub-excavated subgrade level.
- **Driven steel H-piles or pipe (tube) piles founded within the "100-blow" material:** Driven steel H-piles or steel pipe (tube) piles are feasible for support of the abutments, and would permit design of integral abutments (for H-piles) or conventional or semi-integral abutments (for tube piles). The abutment pile caps could be "perched" within the reconstructed approach embankments, or constructed to match the existing abutment and pier footings. The surface of the "100-blow" stratum is at approximately Elevation 175 m at the abutment and pier locations. There is risk associated with penetrating through the till deposits or the piles "hanging up" within portions of till deposit due to the potential presence of cobbles and potential boulders encountered at higher elevation. Pile driving shoes are recommended to protect the pile tips from damage during driving into the hard soils.
- **Caissons founded within the "100-blow" material:** Caissons are feasible for the support of the abutments and piers. This option would be more expensive than either shallow foundations or driven pile foundations, although fewer caisson elements would be required in comparison to the number of driven steel piles that would be required. The abutment pile caps would be "perched" within the reconstructed approach embankment or at the underside of the bridge structure; or the pile cap could be constructed to match the existing footing levels at approximately Elevations 187.4 m. Caissons would extend into the water-bearing till deposits, and temporary or permanent liners would be required during caisson construction to control potential ground losses and/or disturbance of the caisson base.



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Based on the above considerations, steel H-pile deep foundations are considered feasible for the support of the new abutments and pier foundations. 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 the 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 further to the north of the existing foundations but at the same elevation as the existing foundations if possible. 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 Elevation 187.4 m at the abutment and pier locations (based on the available design drawings, dated March 1963).

For support of the new abutments and pier, strip or spread footings should be founded below any fill or softened/loosened surficial soils, on the very stiff to hard sandy clayey silt to clayey silt with sand till deposit. Based on the current investigation, the existing fill material extends to below the founding elevation of the existing footings; as such, the recommended founding levels for the new footings are at lower elevations and on the native stratum.

Alternatively, the footings could be founded on OPSS.PROV 1010 (Aggregates) Granular 'A' engineered fill pad 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 alternative founding elevations for strip or spread footing for support of the new abutments and piers.

Foundation Element	Founding Elevation (m)	Founding Soil
East abutment	186.5	Very stiff to hard sandy clayey silt to clayey silt with sand till
	187.4	Very stiff to hard sandy clayey silt to clayey silt with sand till
	187.4	Granular 'A' pad/engineered fill on Very stiff to hard sandy clayey silt to clayey silt with sand till



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Central pier	186.0	Very stiff to hard sandy clayey silt to clayey silt with sand till
	187.4	Very stiff sandy clayey silt to clayey silt with sand till
	187.4	Granular 'A' pad/engineered fill on Very stiff to hard sandy clayey silt to clayey silt with sand till
West abutment	186.5	Hard sandy clayey silt till
	187.4	Very stiff clayey silt
	187.4	Granular 'A' pad/engineered fill on Very stiff to hard sandy clayey silt to clayey silt with sand till

6.3.3 Geotechnical Resistance/Reaction

For concrete footings founded at the elevations given in Section 6.3.2, the factored geotechnical axial 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 Width (m)	Founding Elevation (m)	Footing Foundation Subgrade	Factored Geotechnical Resistance at ULS (kPa)	Geotechnical Reaction at SLS (kPa)
East Abutment	5	186.5	Native Stratum as noted above	450	250
		187.4	Native Stratum as noted above	325	200
		187.4	Minimum 1 m thick Granular 'A' Pad / Engineered Fill constructed on native stratum as noted above	450	300
Pier	3	186.0	Native Stratum as noted above	400	275
		187.4	Native Stratum as noted above	275	200
		187.4	Minimum 1 m thick Granular 'A' Pad / Engineered Fill constructed on native stratum as noted above	400	275



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Foundation Element	Footing Width (m)	Founding Elevation (m)	Footing Foundation Subgrade	Factored Geotechnical Resistance at ULS (kPa)	Geotechnical Reaction at SLS (kPa)
West Abutment	5	186.5	Native Stratum as noted above	450	250
		187.4	Native Stratum as noted above	325	150
		187.4	Minimum 1 m thick Granular 'A' Pad / Engineered Fill constructed on native stratum as noted above	450	300

Based on conversations with AECOM, it is understood that the imposed loadings from the new bridge imposed on the foundation require founding conditions capable of providing a factored geotechnical axial resistance at ULS of 450 kPa and a geotechnical reaction at SLS (for 25 mm of settlement) of 300 kPa at the abutments; and a factored geotechnical axial resistance at ULS of 375 kPa and a geotechnical reaction at SLS (for 25 mm of settlement) of 275 kPa at the pier. The required factored geotechnical axial resistance at ULS and geotechnical reaction at SLS can be obtained for shallow foundations founded on a Granular 'A' Pad / Engineered Fill constructed on the native stratum at the founding levels as noted in Section 6.3.2 and given above. It is understood that due to space restrictions it may not be feasible to sub-excavate below the existing footing elevation along the south side of the replacement structure, in which case the foundations would have to be designed on the basis of the available lower resistances, larger footings, ensuring that adequate temporary protection support is provided to the existing foundations, or utilizing deep foundations. Additionally, with the resistances provided above, there is a potential for up to 25 mm of differential settlement between the northern portion and the southern portion of the bridge abutments and pier as the footings would be founded on virgin soil conditions on the north side of the proposed structure and on historically preloaded soils on the south side of the proposed structure.

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 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 from 1 m beyond the foundation of the footing boundary 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 1 m for support of the footings. The required thickness of conventional soil cover for frost protection of the footing (i.e. 1.2 m) constructed on the granular pad/engineered fill is measured perpendicular from the face of the abutment slope to the edge of



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the underside of the footing (it is not simply a vertical dimension when the footing is adjacent to a slope) and relative to the lowest adjacent final ground surface.

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 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 very stiff to hard till soils, the coefficient of friction, $\tan \delta$ or $\tan \phi'$, can be taken as follows:

- | | |
|-------------------------------------------------------------|---------------------|
| ■ Cast-in-place footing to concrete working slab: | $\tan \delta = 0.7$ |
| ■ Cast-in-place concrete working slab to till deposit: | $\tan \phi' = 0.5$ |
| ■ Cast-in-place footing to Granular 'A' Pad/Engineered Fill | $\tan \phi' = 0.6$ |

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 ("100-blow") sandy clayey silt to clayey silt with sand till to very dense ("100-blow") silt/silt and sand till may be used to support the pier and 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 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 hard or very dense layers are encountered within the till deposits. The steel H-piles should be reinforced with flange plates as per OPSD 3000.100 (Foundation Piles Steel H-Pile Driving Shoe) or driving shoes such as Titus Standard "H" Bearing Pile Point design for protection during driving as per OPSS 903 (Deep Foundations). Similarly, if steel tube piles are being considered, driving shoes should be in accordance with OPSD 3001.100 Type II (Steel Tube Pile Driving Shoe). The requirement for driving shoes should be included in the Contract Drawings.

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.



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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 and End-Bearing Piles

HP 310 x 110 piles driven at least 1.5 m into the “100-blow” material at or below a design tip Elevation 173.5 m (i.e., piles about 16 m long from the Dufferin street level), the factored axial geotechnical resistance at ULS may be taken as 1,400 kN. The geotechnical reaction at SLS (for 25 mm of settlement) may be taken as 1,200 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 2,800 kN per pile, but must be driven below tip Elevation 173.5 m.”

Similar axial resistances and drawing note 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 Note 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.

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 consistency/relative density 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.



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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 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 sandy clayey silt to clayey silt with sand till	-	150
Very dense silt	20,000	-
Very dense silt and sand till	20,000	-

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*.



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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 sandy clayey silt to clayey silt with sand till and the very dense silt and sand till for support of the foundation elements for the new abutments and pier.

If caisson foundations are adopted for support of any of the foundation elements, a temporary or permanent liner will 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, such as the silt deposit near the east abutment, 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 Non-Standard Special Provision (NSSP) be included in the Contract Documents to address the need for control of the ground and groundwater during caisson construction as discussed further under Construction Considerations in Section 6.9. 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 (hard sandy clayey silt till to clayey silt with sand till, or silt and sand till) at the base of the caissons. Each caisson excavation should be carefully cleaned to remove all loosened debris to ensure that the concrete is in intimate contact with the competent bearing stratum. The Ontario Occupational Health and Safety Act (2012) outlines appropriate safety procedures and requirements that must be implemented prior to the caissons for inspection. The inspection should 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.



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6.5.1 Founding Elevations

Caissons should be founded within the hard sandy clayey silt till to clay silt with sand till deposit, or silt and sand till deposit, socketed for at least 1.5 m into the “100-blow” material. The estimated caisson tip elevations for new abutment foundations are summarized below.

Foundation Element	Maximum Founding Elevation (m) ¹	Founding Soil
East abutment	173.5	Very dense silt and sand till
West abutment	173.5	Hard sandy clayey silt to clayey silt with sand till
Central Pier	173.5	Hard sandy clayey silt to clayey silt with sand

6.5.2 Geotechnical Resistances

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

Caisson Diameter	Factored Geotechnical Axial Resistance at ULS	Geotechnical Reaction at SLS (for 25 mm settlement)
0.6 m	2,250 kN	1,800 kN
1.2 m	7,000 kN	5,000 kN
1.5 m	9,500 kN	7,000 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 Bridge Retaining Walls

Based on the GA drawings provided by AECOM, a radial retaining wall is to be constructed at each quadrant of the bridge abutments, having a quarter circle configuration with a radius of 10 m to 15 m and a wall stem of approximately 0.4 m wide. The proposed retaining wall structures will consist of cast-in-place concrete cantilever walls founded on 2.5 m to 3 m wide spread footings constructed at similar founding elevations as the existing retaining wall structures, at Elevation 187.6 m.

The feasible retaining wall options in the northeast, southeast, northwest and southwest quadrants of the Highway 401 W – Yorkdale Road ramp over Dufferin Street bridge structures may include the following:

- Concrete retaining walls supported on spread footings founded on the very stiff to hard sandy clayey silt to clayey silt with sand till deposits. Based on the current investigation, the existing fill material extends below



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the founding elevation of the existing footings and consideration may be given to supporting the retaining wall on a Granular 'A' engineered fill placed on the native hard/dense to very dense till deposits to raise the sub-excavated subgrade level, or on the native strata below the fill, at between Elevations 187.6 m and 186.4 m.

- Concrete retaining walls supported on deep foundations (steel H-Piles, tube piles or caissons), are considered suitable for the support of the retaining walls. The foundation recommendations provided in Sections 6.4 and 6.5 for pile or caissons foundations can be used for design of the retaining walls .
- Retained Soil System (RSS) walls are not considered practical due to the space restrictions, construction staging and the existing embankment and retaining structure configuration. As such, RSS wall is not discussed further in this report.

6.6.1 Spread Footings

6.6.1.1 Founding Elevations

It is understood that the new retaining wall structure foundations will be founded at or near the same location and elevation as the existing foundations. Therefore, full removal of the existing retaining walls and foundations would be required, with removal excavations extending to the existing footing founding levels at approximately Elevation 187.4 m (based on the available design drawings, dated March 1963).

For support of the retaining walls, strip or spread footings should be founded below any fill or softened/loosened surficial soils, on the very stiff to hard sandy clayey silt to clayey silt with sand till deposit. Based on the current investigation, the existing fill material extends below the founding elevation of the existing footings; as such the recommended founding elevations for the new footings are at lower elevations on the native stratum.

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.

The following summarizes the recommended founding elevations for strip or spread footing for support of the new retaining wall structures.

Retaining Structure Location	Maximum Founding Elevation (m)	Founding Soil
Northwest and Southwest Quadrant	187.0	Very Stiff Clayey Silt/ Very stiff to hard sandy clayey silt to clayey silt with sand till
	187.4	Granular 'A' pad/engineered fill on Very stiff to hard sandy clayey silt to clayey silt with sand till
Northeast and Southeast Quadrant	186.8	Hard sandy clayey silt till
	187.4	Granular 'A' pad/engineered fill on Very stiff to hard sandy clayey silt to clayey silt with sand till



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6.6.1.2 Geotechnical Resistance/Reaction

For concrete footings founded at the elevations given in Section 6.6.1.1, the factored geotechnical axial resistance at Ultimate Limit States (ULS) and the geotechnical reaction at Serviceability Limit States (SLS), for 25 mm of settlement, may be taken as follows:

Retaining Structure Location	Footing Width (m)	Founding Elevation (m)	Footing Foundation Subgrade	Factored Geotechnical Resistance at ULS (kPa)	Geotechnical Reaction at SLS (kPa)
Northwest and Southwest Quadrant	2.5 to 3	187.0	Native Stratum as noted above	250	200
		187.4	Minimum 0.5 m thick Granular 'A' Pad / Engineered Fill constructed on native stratum as noted above	275	200
Northeast and Southeast Quadrant	2.5 to 3	186.8	Native Stratum as noted above	250	200
		187.4	Minimum 0.5 m thick Granular 'A' Pad / Engineered Fill constructed on native stratum as noted above	275	200

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 of full sub-excavation of the existing fill and replacement with OPSS.PROV 1010 (Aggregates) Granular 'A' Engineered Fill, 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 from 1m beyond the footprint of footing boundary 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 0.5 m for support of the footings. The required thickness of conventional soil cover for frost protection of the footing (i.e. 1.2 m) constructed on the granular pad/engineered fill is measured perpendicular from the face of the abutment slope to the edge of



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the underside of the footing (it is not simply a vertical dimension when the footing is adjacent to a slope) and relative to the lowest surrounding/adjacent final ground surface.

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 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.

6.6.2 Global Stability

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

Drained and undrained analyses were carried out for the retaining walls stability assessment. The critical soil parameters used in the analysis, as given below, were estimated from empirical correlations using the results of in-situ Standard Penetration Tests (Bowles, 1984) and geotechnical classification testing. The groundwater table was modelled at Elevation 187.9 m in the analyses.

Soil Type	Unit Weight (kN/m ³)	Undrained Shear Strength (kPa)	Angle of Internal Friction, ϕ' (degrees)
Existing Embankment Fill (Firm to stiff clayey silt to sandy clayey silt fill)	20	40	-
New Embankment Fill (Assumed Granular B)	21	-	30
Very stiff clayey silt	19	100	-
Stiff to hard sandy clayey silt to clayey silt with sand till	21	150	-
Granular 'A' pad (proposed)	22	0	34
Concrete Retaining Wall	24	-	-

A retaining wall section was analyzed for the maximum wall height anticipated as shown on the GA drawing provided by AECOM on March 21, 2016. In this analysis, the height of the retaining wall was considered to be 4.9 m, extending from the top of the new retaining wall stem (Elevation 192.3 m) and the underside of the lowest



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retaining panel footing at Elevation 187.4 m. The embankment side slopes between the concrete retaining wall the Yorkdale ramp grade are assumed to be sloped at a gradient of approximately 2 horizontal to 1 vertical (2H:1V)

Assuming a 3 m wide retaining wall foundation, and a 0.4 m thick wall stem retaining an earth embankment as noted above, a factor of safety greater than 1.5 was calculated against deep-seated global instability. The results of the analysis are shown on Figures 1 and 2 for the static condition for founding the retaining wall footings on the native soil stratum or on the Granular 'A' pad, respectively.

Under seismic loading conditions, using a design seismic coefficient equal to the 50 per cent of the site-specific design peak horizontal ground acceleration (PGA) which is about 0.03 g for this site, the Factor of Safety is greater than 1.1. The result of the seismic slope stability analysis for the retaining wall is shown on Figures 3 and 4 for founding the retaining wall on the native soil stratum and founding on the Granular 'A' pad, respectively.

6.7 Lateral Earth Pressures for Design

The lateral earth pressures acting on the abutment stem walls, and wing walls/retaining walls will depend on the type and method of placement of the backfill materials, the nature of the soils behind the backfill, the magnitude of surcharge including construction loadings, the freedom of lateral movement of the structure, and the drainage conditions behind the walls. 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:



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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

- 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*.

6.7.1 Seismic Considerations

6.7.1.1 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.

6.7.1.2 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 ratio A , 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.

6.8 Bridge Approaches

The proposed replacement of the Highway 401 W - Yorkdale Road ramp structure includes the raising of the bridge grade by a nominal amount (i.e., less than 0.6 m). The proposed bridge replacement also includes a shift in the alignment by approximately 7 m to the north. Based on the design drawings provided, removal of the existing



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structure and reconstruction of the new structure at this site will require the widening of the existing approach embankments to the north and a cut in the existing approach embankments to the south.

The cutting of the existing approach embankments to the south is considered to result in unloading the soil at the new abutments and, considering that there is no significant grade raise at this structure, the stability of the approach embankment will not be affected nor will there be any significant settlements. The existing embankment side-slopes are sloped at about 2H:1V and it is assumed the new embankment side-slopes will be maintained at 2H:1V or shallower. In addition, considering the site will essentially be “unloaded”, the total settlement of the foundation soil due to approach embankment loadings, is expected to be less than 25 mm. Granular fill should be used to backfill any excavated portions behind the new abutments and approach embankment to minimize the potential for additional settlement.

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

6.8.1 Cut Slope Construction

In accordance with MTO's standard practice, a minimum 2 m wide bench should be provided where cut slopes are equal to or greater than 6 m high. To reduce erosion of the cut slopes due to surface water runoff, it is recommended that topsoil and seeding as per OPSS 802 (topsoil) and OPSS.PROV 804 (seed and cover); or pegged sod as per OPSS803 (sodding), be placed as soon as practicable after completing cutting/shaping of the slopes. Consideration may also be given to the use armoured drainage channels on the cut slope face to direct surface water flow from the ramp grade.

6.8.2 Subgrade Preparation and Embankment Construction

The existing native soils are considered to be an appropriate subgrade for the new construction of the ramp 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 base of the existing fill extends to about Elevation 186.8 m in the widened approach embankment area as delineated by Borehole DS15-5, that is to a depth of about 2.2 m below existing ground surface. Any softened/loosened native soils that may be present below the fill should also be removed.

The use of granular fill, such as OPSS.PROV 1010 Granular 'B' Type I, 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 fill 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 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 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



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surficial sloughing of the topsoil material along the side-slopes and to establish vegetation prior to the Fall / Winter months.

6.8.3 Settlement under Widened Embankment Loading

It is understood that a limited grade raise on the Highway 401 W – Yorkdale Road ramp is proposed, together with an approximately 7 m northward widening of the existing 7 m to 8 m high approach embankments.

Settlement assessments have been completed for the northward widening using the commercially available computer program *Settle-3D* from Rocscience, using the consolidation parameters and estimated elastic deformation moduli given below, based on correlations with the SPT “N”-values and engineering judgement from experience with similar soils in this region of Ontario (Bowles, 1984; Kulhawy and Mayne, 1990; Peck et al., 1974).

Soil Deposit	Bulk Unit Weight (kN/m ³)	Elastic Modulus (MPa)
New embankment fill (Assumed Granular 'B')	21	-
Existing embankment fill	20	20 MPa
Very stiff clayey silt (West Abutment area)	19	20 MPa
Stiff sandy clayey silt to clayey silt with sand till	21	25 MPa
Very stiff to hard sandy clayey silt to clayey silt with sand till	21	45 MPa

Based on this assessment, the settlement of the foundation soils under the 7 m northward widening is estimated to be approximately 20 mm under the 7 m to 8 m high widened zone of the approach embankments. The majority of this settlement is associated with the clayey silt deposit and the upper zone of the sandy clayey silt to clayey silt with sand till deposit, and is estimated to be completed within approximately two to three months following completion of the widening. The above estimates do not include compression of the fill itself, which would occur during and after the construction of the embankment depending on the type of materials used. The magnitude of compression of the embankment fill may range from 0.5 to 1 per cent of the height of the embankment, assuming approximately 98 per cent compaction of the embankment fill is achieved, relative to the material's standard Proctor maximum dry density. In the case where granular fill is used for embankment construction, settlement of the fill itself is expected to occur essentially during embankment construction, whereas non-granular earth fill materials are expected to exhibit some additional settlement over time.

It is recommended that construction of the approach slab and the final grading and paving of the ramp should be carried out as late in the construction schedule as possible to minimize the effects of settlement of the widening fill on the final pavement structure. Construction of the widened embankment to final subgrade level and pre-loading for a minimum period of 30 days would allow the majority of the compression/consolidation settlement to occur prior to construction of the approach slab and final grading and paving of the ramp.



6.9 Construction Considerations

6.9.1 Open Cut Excavation

The foundation excavations at the abutments for spread footings or pile cap construction will extend to depths of about 7 m to 9 m below the present ramp road surface, through the existing fill and into the very stiff to hard till deposit. The foundation excavations at the piers for spread footing or pile cap construction will generally extend to depths of about 2 m to 3 m into the very stiff to hard till.

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.

6.9.2 Temporary Excavation Support

It is anticipated that temporary protection systems will be required along Dufferin Street to facilitate the removal of the existing abutment and pier foundations and to allow for the construction of the new abutment footings and pier foundations. Temporary protection systems will also be required approximately along the ramp embankment to facilitate the planned construction while maintaining one lane 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 up to approximately 9 m at the abutments and 2 m to 3 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 hard strata may affect the installation of the interlocking sheet pile system. An interlocking sheetpile system would contribute to both ground and, where applicable, control of groundwater seepage from cohesionless zones or interlayers/lenses within the cohesive deposits if such zones are present/intercepted. 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. The groundwater level in the piezometers was measured at about Elevation 187.9 m (depth of 7.2 m below ground surface from the ramp road surface) and Elevation 187.8 m (depth of 6.7 m below ground surface from the ramp road surface) in Boreholes DS15-1 and DS15-2, respectively, and may contribute to seepage into the excavations.

The sheetpiles or soldier piles would have to be driven to sufficient depth to provide the necessary passive resistance for the retained soil height of up to about 9 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.



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6.9.3 Groundwater Control

Excavations for removal of the existing abutment and pier foundations as well as for construction of the new abutment and pier footings will extend below the groundwater level at the site, which has been measured to be about between Elevation 188 m. Some groundwater seepage is expected from the 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.

6.9.4 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.

6.9.5 Vibration Monitoring During Pile Installation

Depending on the construction sequence, vibration monitoring may be warranted at the existing structure during pile driving 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 temporarily in place); however, this requires further assessment by the structural engineer. The pile or caissons further from the existing bridge 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 or caisson installation for the remaining piles/caissons. As there are some commercial structures in the vicinity of the site, monitoring of vibrations during construction should also 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.

6.9.6 Obstructions During Pile Driving / Caisson Installation

Cobbles and/or boulders were encountered and inferred present during the borehole 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.



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7.0 CLOSURE

This Foundation Design Report was prepared by Qasim Cheema, P.Eng., a geotechnical engineer with Golder, and reviewed 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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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



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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
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**TABLE 1: COMPARISON OF FOUNDATION ALTERNATIVES
FOR HIGHWAY 401 W – YORKDALE ROAD RAMP OVER DUFFERIN STREET
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 very stiff to hard clayey silt till.	<ul style="list-style-type: none">Feasible for support of new abutments and new piers at relatively shallow depth.	<ul style="list-style-type: none">Existing abutments and pier foundation which are supported on shallow foundations, and have performed well.Excavations to remove existing structure and foundations would extend to 1.0 m above the suitable founding stratum for the replacement structure. Thus not disturbing the new foundation subgrade.Lower costs than deep foundations.Standard construction methods; no specialized construction equipment required.	<ul style="list-style-type: none">Temporary excavation support required along Dufferin Street to facilitate removal and construction of the abutment and pier footingsPrecludes use of integral abutments; potentially greater maintenance required at abutments.Lower geotechnical resistances as compared with deep foundations or if founding on engineered fill.	<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.
Spread/strip footings founded on engineered fill placed on very stiff to hard clayey silt till.	<ul style="list-style-type: none">Feasible for support of new abutments and new pier at relatively shallow depth.	<ul style="list-style-type: none">Existing abutments and pier foundation are supported on shallow foundations, and have performed well, and so should the new foundations given the subsurface conditions in the immediate vicinity.Excavations to remove existing structure and foundations would extend to 1.0 m above the suitable founding stratum for the replacement structure, thus not disturbing the new foundation subgrade.New founding elevations can be at the same level as the existing founding levels, thus not undermining the existing foundations during construction.Lower costs than deep foundations.Standard construction methods; no specialized construction equipment required.	<ul style="list-style-type: none">Temporary excavation support required along Dufferin Street 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 \$1,100/m3 for construction of each spread footings on a granular pad, excluding temporary protection system.	<ul style="list-style-type: none">Potential traffic disruption during construction.



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OVER DUFFERIN STREET

FOUNDATION OPTION	FEASIBILITY	ADVANTAGES	DISADVANTAGES	RELATIVE COSTS	RISKS / CONSEQUENCES
Steel H-piles driven to found in “100-blow” material (hard sandy clayey silt to clayey silt with sand till/very dense silt/very dense silt and sand till).	<ul style="list-style-type: none">Feasible for support of new abutments and new piers.	<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 constructionHigher geotechnical resistance than for shallow foundations.Abutment pile caps could be constructed within approach embankment fill to allow for locating CSPs around the upper section of piles within the fill	<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.	<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;	<ul style="list-style-type: none">Traffic disruption during constructionPotential vibrations may be induced on existing structure.Risk of encountering obstructions that could impact pile installation.Potentially less costly maintenance over life of the structure than semi-integral abutment structures.
Steel pipe (tube) piles, driven to found in “100-blow” material (hard sandy clayey silt to clayey silt with sand till/very dense silt/very dense silt and sand till).	<ul style="list-style-type: none">Feasible for support of new abutments and new piers.	<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”.Higher long term maintenance costs than for integral abutments.



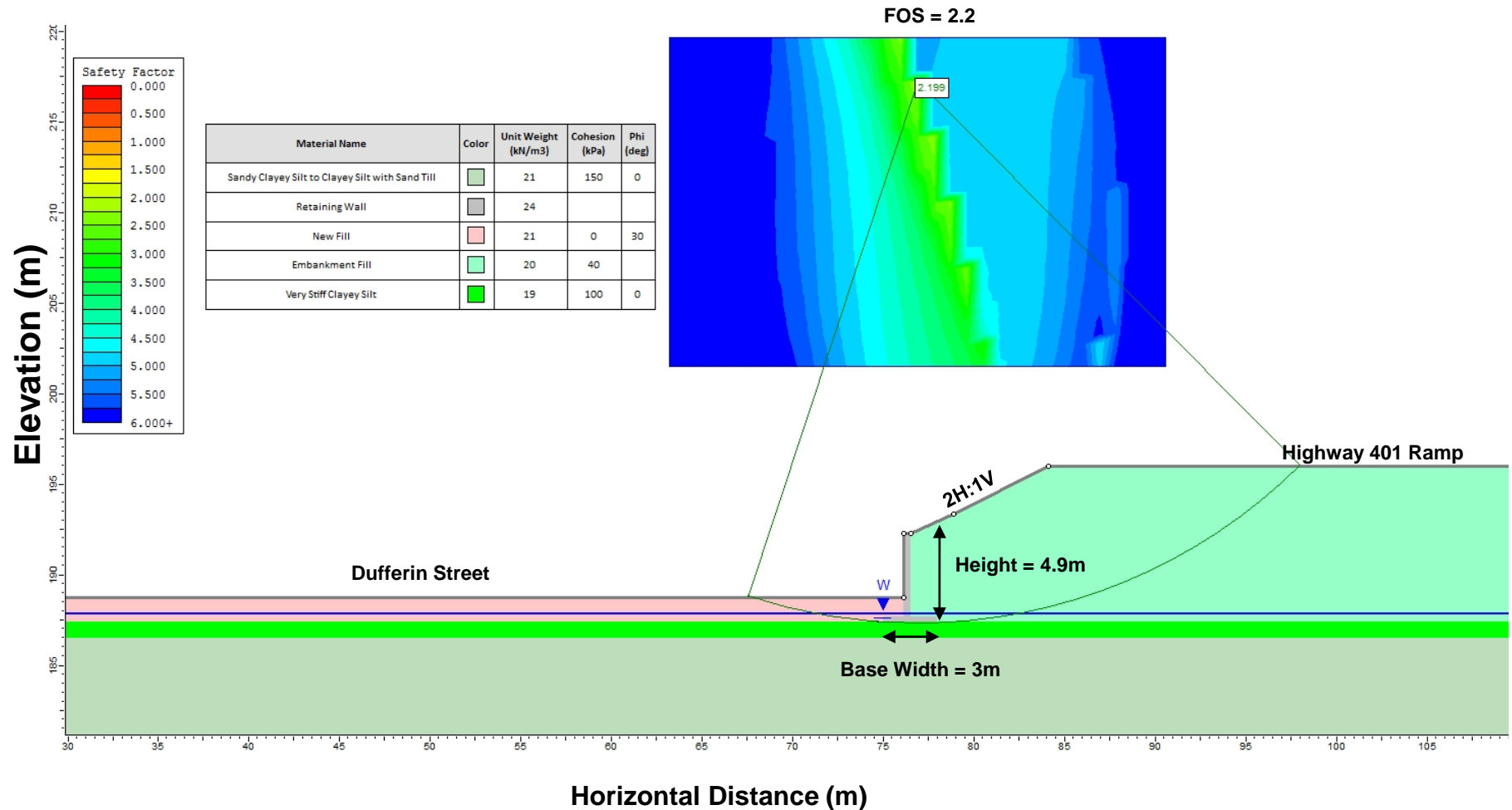
FOUNDATION REPORT
HIGHWAY 401 W - YORKDALE ROAD RAMP (SITE NO. 37-284)
OVER DUFFERIN STREET

FOUNDATION OPTION	FEASIBILITY	ADVANTAGES	DISADVANTAGES	RELATIVE COSTS	RISKS / CONSEQUENCES
Caissons founded in “100-blow” material (hard sandy clayey silt to clayey silt with sand till/very dense silt/very dense silt and sand till).	<ul style="list-style-type: none">Feasible for support of new abutments and new piers.	<ul style="list-style-type: none">Abutment pile caps could be constructed at underside of the bridge structure, Thus eliminating excavation needs altogether;(removal of existing foundations excavation only 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 abutmentsGreater 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.



Retaining Wall Static Global Stability Analysis – Native Stratum Highway 401 W– Yorkdale Road Ramp over Dufferin Street

Figure 1



Date: April 2016

Project: 09-1111-6007

Analysis By: KW

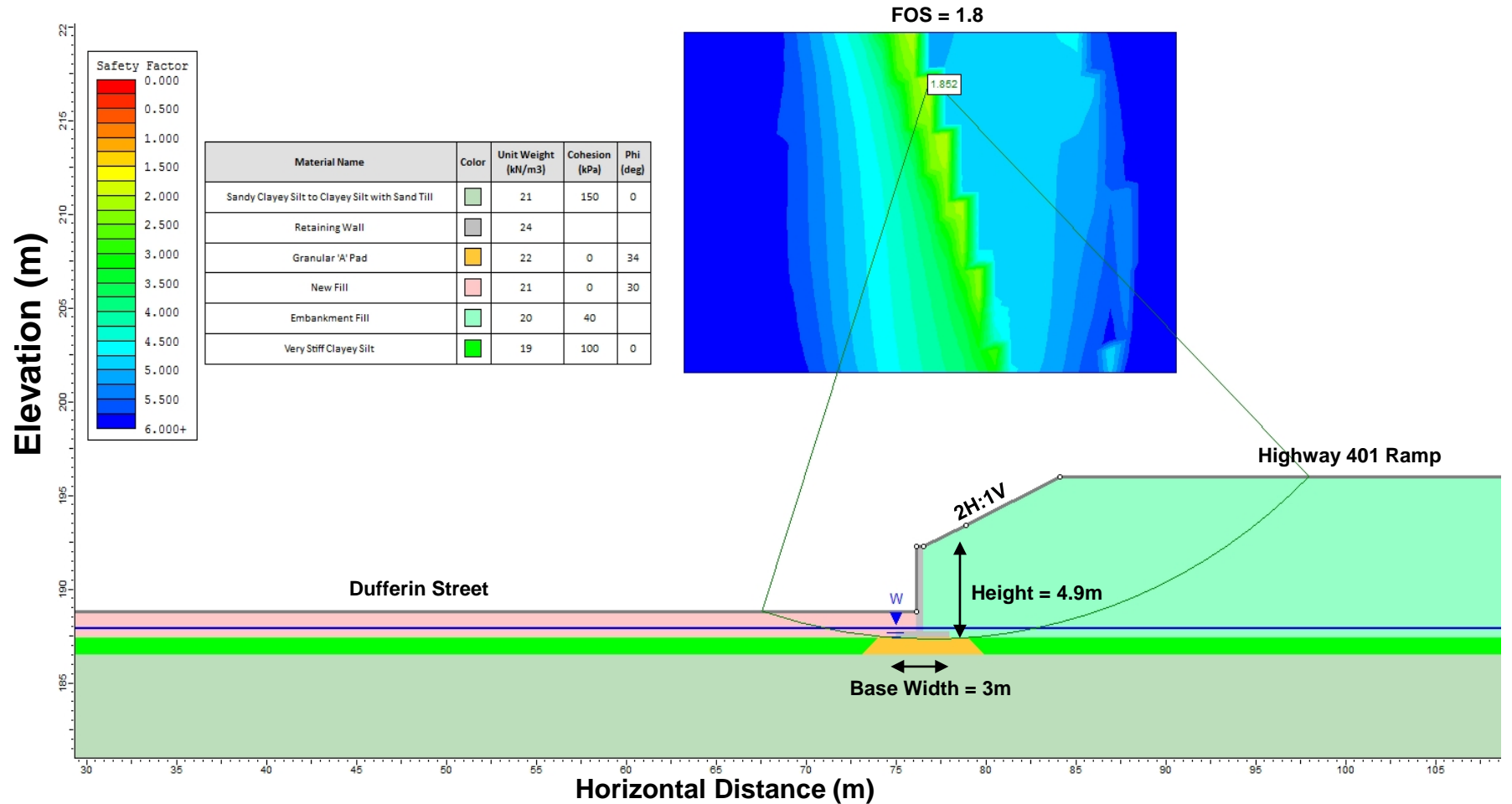
Reviewed By: NK





Retaining Wall Static Global Stability Analysis– Granular ‘A’ Pad Highway 401 W– Yorkdale Road Ramp over Dufferin Street

Figure 2



Date: April 2016

Project: 09-1111-6007

Analysis By: KW

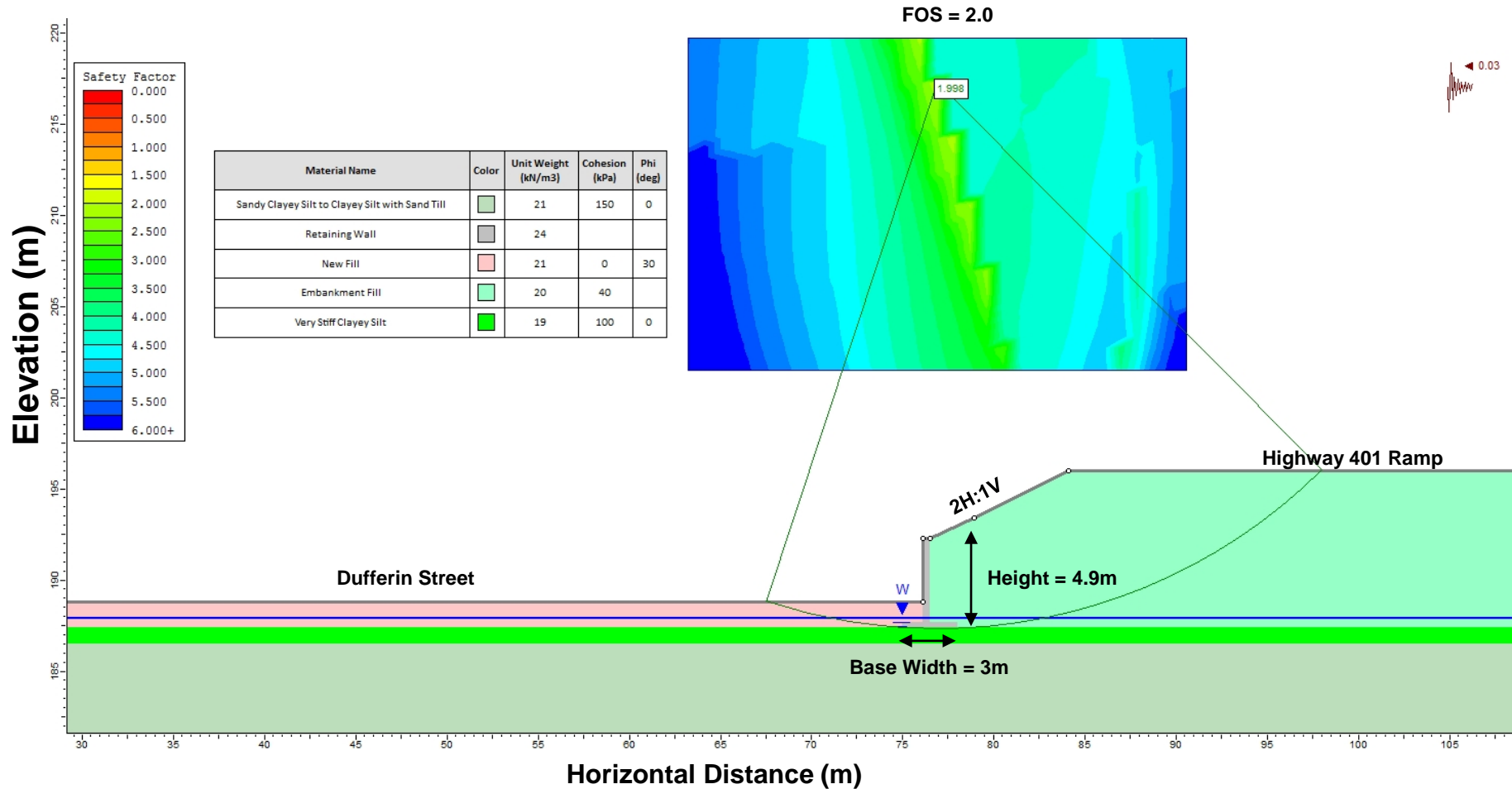
Reviewed By: NK





Retaining Wall Seismic Global Stability Analysis – Native Stratum Highway 401 W– Yorkdale Road Ramp over Dufferin Street

Figure 3



Date: April 2016

Project: 09-1111-6007

Analysis By: KW

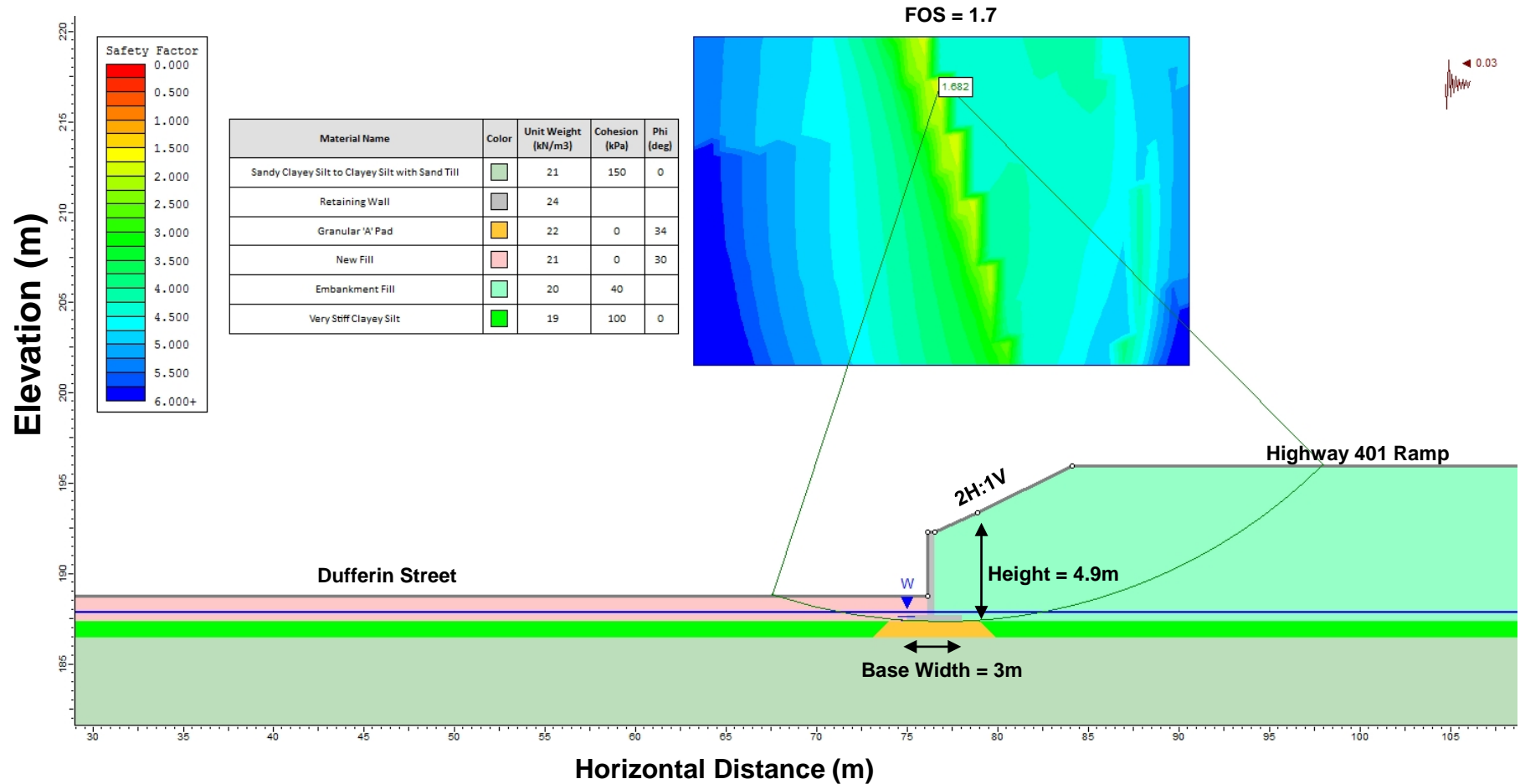
Reviewed By: NK





Retaining Wall Seismic Global Stability Analysis – Granular ‘A’ Pad Highway 401 W– Yorkdale Road Ramp over Dufferin Street

Figure 4



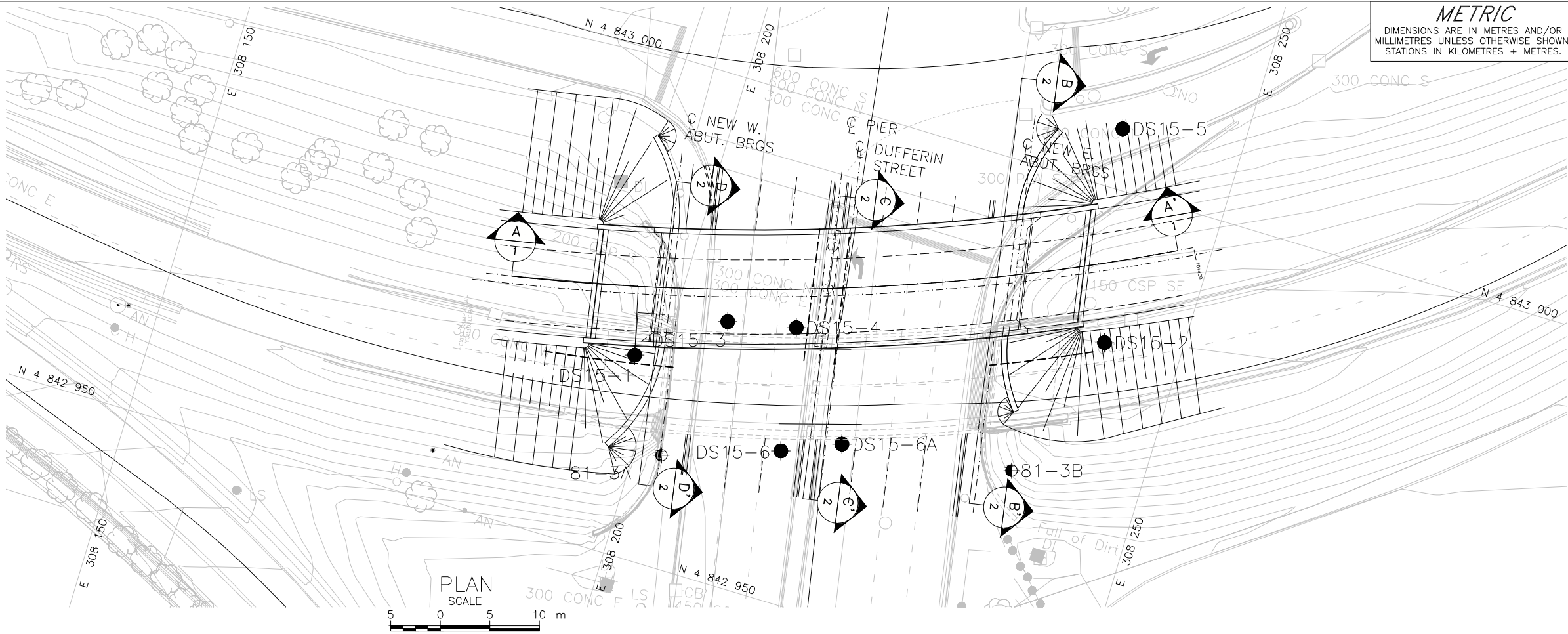
Date: April 2016

Project: 09-1111-6007

Analysis By: KW

Reviewed By: NK





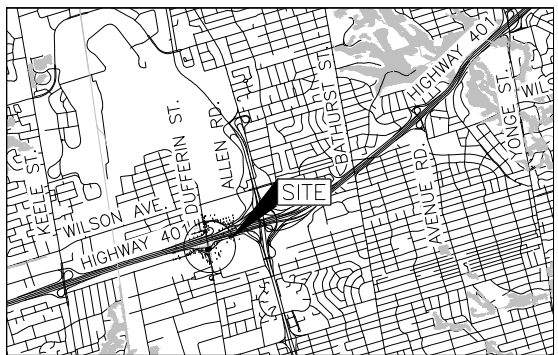
METRIC
DIMENSIONS ARE IN METRES AND/OR
MILLIMETRES UNLESS OTHERWISE SHOWN.
STATIONS IN KILOMETRES + METRES.

CONT No. _____
GWP No. 2131-01-00



HIGHWAY 401
HIGHWAY 401 W - YORKDALE RAMP OVER
DUFFERIN STREET SITE NO. 37-284
**BOREHOLE LOCATIONS AND
SOIL STRATA**

SHEET



KEY PLAN
SCALE

1 0 1 2 km

LEGEND

- Borehole - Current Investigation
- Borehole - Previous Investigation 1 (Geocres No. 30M11-081)
- ⊥ 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

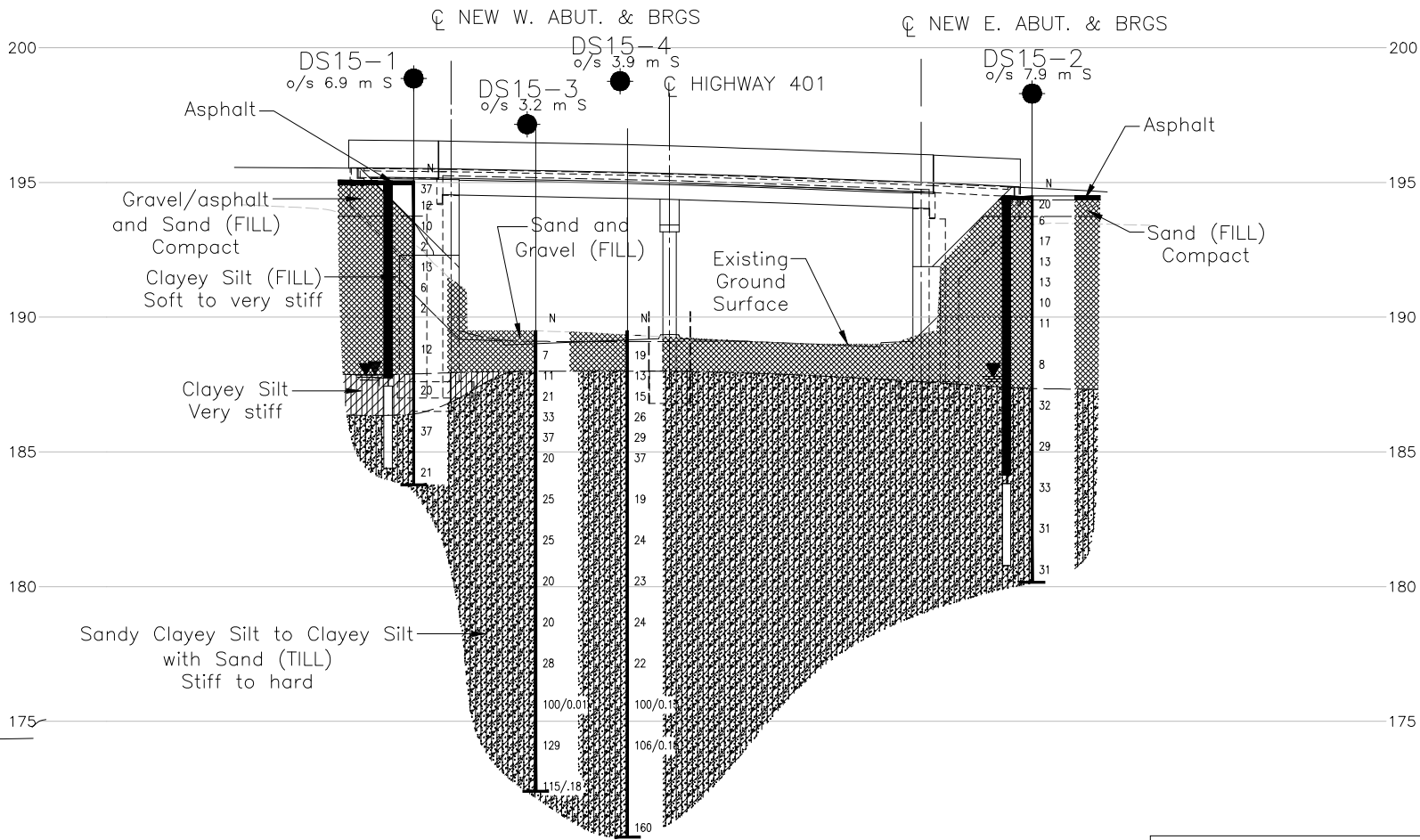
No.	ELEVATION	NORTHING	EASTING
81-3A	189.0	4842961.1	308202.8
81-3B	189.0	4842969.8	308237.0
DS15-1	195.1	4842970.0	308197.3
DS15-2	194.5	4842984.8	308242.3
DS15-3	189.5	4842975.9	308205.3
DS15-4	189.5	4842977.3	308212.1
DS15-5	189.0	4843006.0	308237.8
DS15-6	189.2	4842965.0	308214.2
DS15-6A	189.3	4842967.4	308219.9

NOTES

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

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

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.



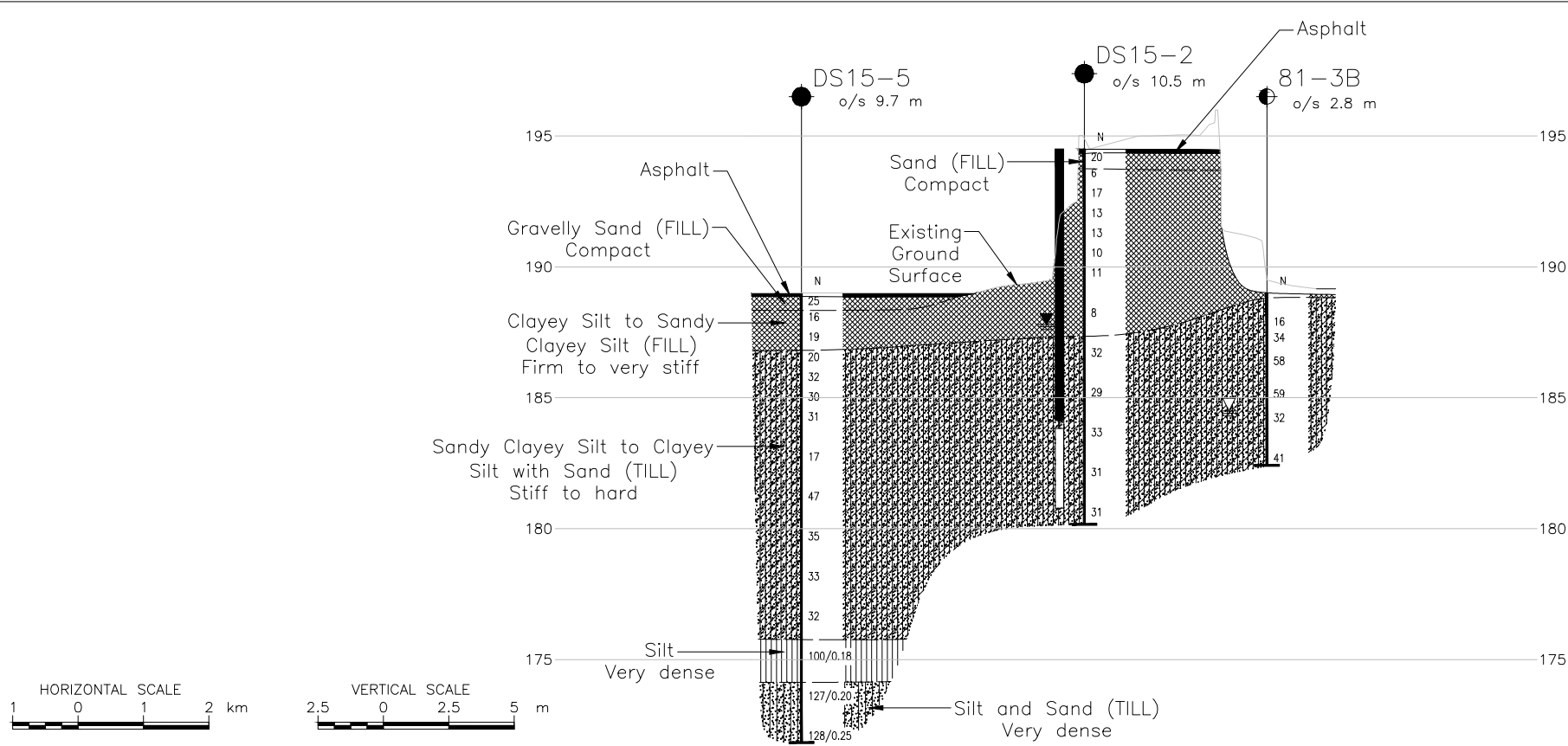
A-A CENTRELINE PROFILE OF HWY 401W-YORKDALE RAMP

HORIZONTAL SCALE
5 0 5 10 m
VERTICAL SCALE
2.5 0 2.5 5 m

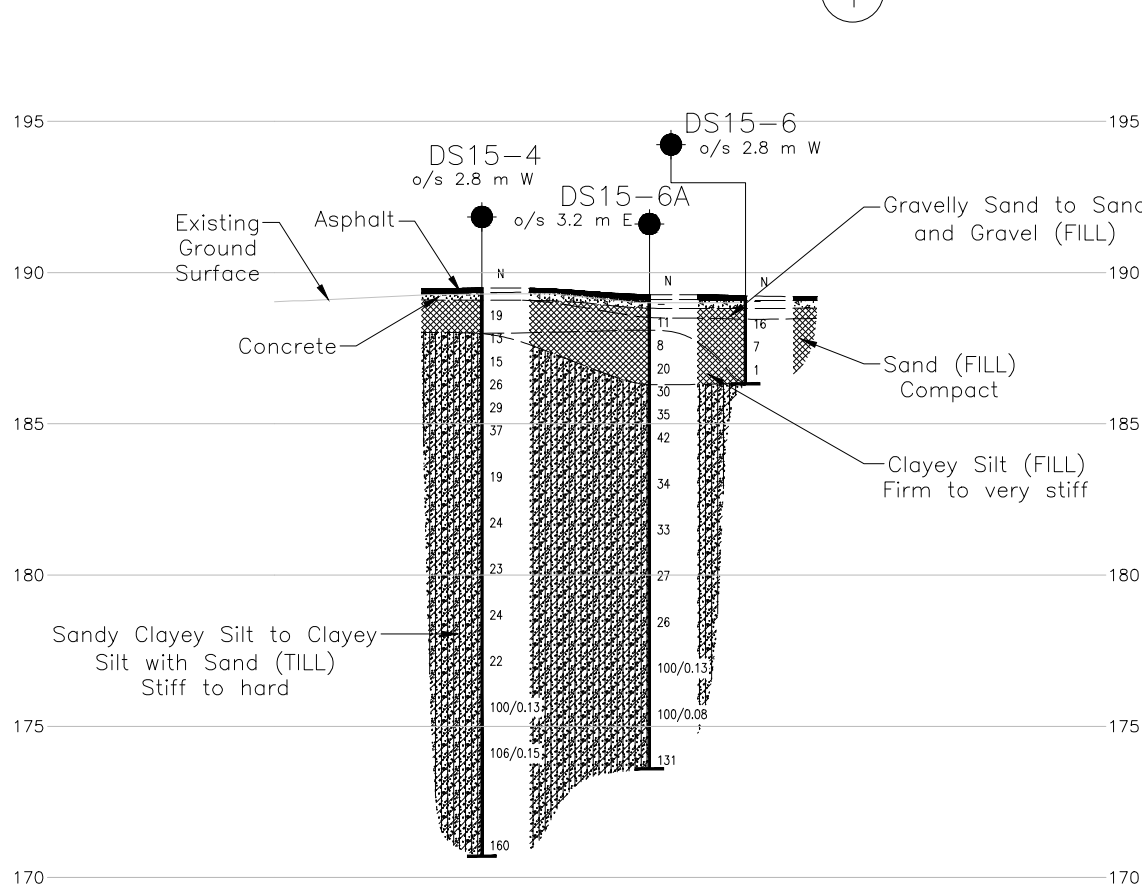
REFERENCE

Base plans and Ramp general arrangement provided in digital format by AECOM, drawing file nos. Hwy401_bgd-Duffern.dwg, Hwy401_contours-Duffern.dwg and Dufferin #37-284_GA.dwg, received November 3, 2015, and 01_Dufferin_Ramp_GA.dwg, received March 21 2016.

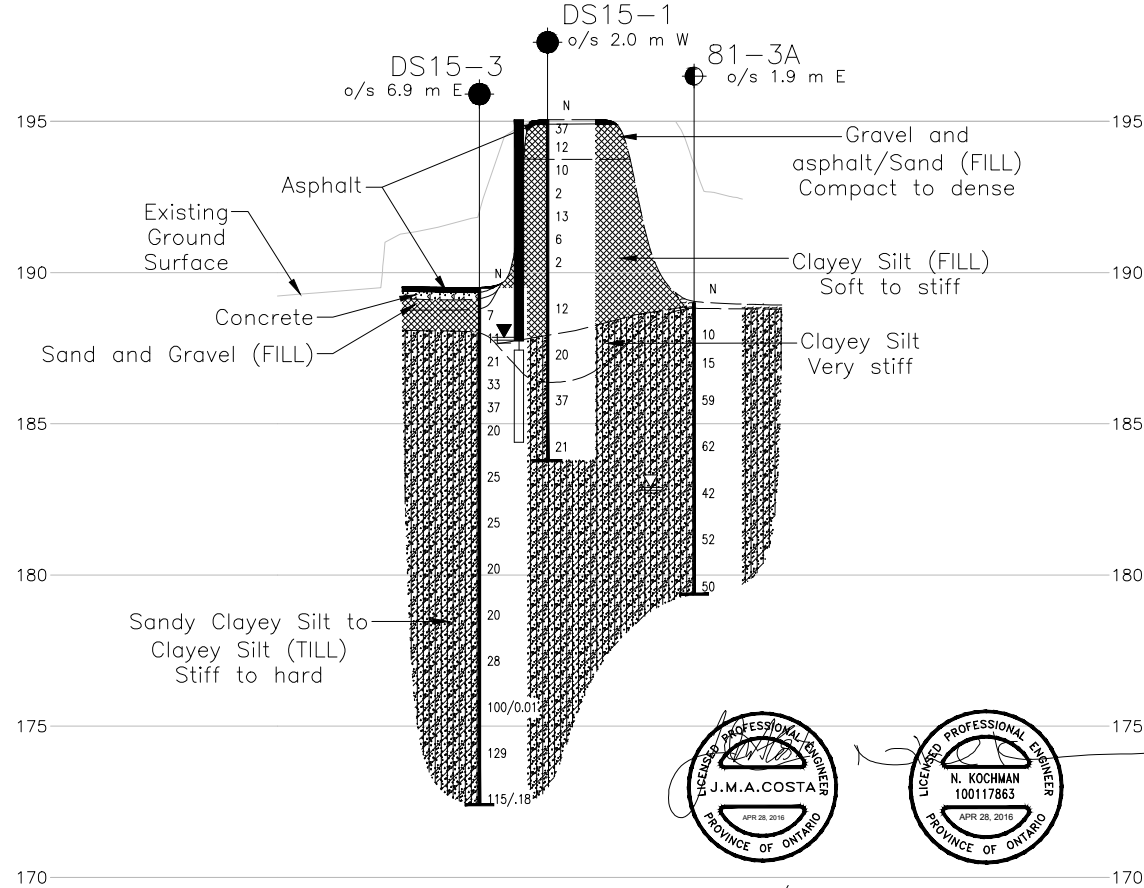
NO.	DATE	BY	REVISION
Geocres No. 30M11-262			
HWY. 401	PROJECT NO. 09-1111-6007		DIST. CENTRAL
SUBM'D. AJS	CHKD. QC	DATE: Nov. 2015	SITE: .
DRAWN: JFC/MR	CHKD. NK	APPD. JMAC	DWG. 1



B-B' EAST ABUTMENT OF HWY401 W - YORKDALE RAMP



C-C' DUFFERIN STREET OVERPASS PIER



D-D' WEST ABUTMENT OF HWY 401 W/YORKDALE RAMP

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

CONT No. GWP No. 2131-01-00

HIGHWAY 401
HIGHWAY 401 W - YORKDALE RAMP OVER
DUFFERIN STREET SITE NO. 37-284
SOIL STRATA



- LEGEND**
- Borehole - Current Investigation
 - Borehole - Previous Investigation 1 (Geocres No. 30M11-081)
 - 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
81-3A	189.0	4842961.1	308202.8
81-3B	189.0	4842969.8	308237.0
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DS15-2	194.5	4842984.8	308242.3
DS15-3	189.5	4842975.9	308205.3
DS15-4	189.5	4842977.3	308212.1
DS15-5	189.0	4843006.0	308237.8
DS15-6	189.2	4842965.0	308214.2
DS15-6A	189.3	4842967.4	308219.9

NOTES

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

REFERENCE

Base plans provided in digital format by AECOM, drawing file nos. Hwy401_bgd-Dufferin.dwg, Hwy401_contours-Dufferin.dwg and Dufferin #37-284_GA.dwg, received November 3, 2015.

NO.	DATE	BY	REVISION
Geocres No. 30M11-262			
HWY. 401	PROJECT NO. 09-1111-6007		DIST. CENTRAL
SUBM'D. QC	CHKD. QC	DATE: Nov. 2015	SITE:
DRAWN: MR	CHKD. NK	APPD. JMAC	DWG. 2



**FOUNDATION REPORT
HIGHWAY 401 W - YORKDALE ROAD RAMP (SITE NO. 37-284)
OVER DUFFERIN STREET**

APPENDIX A

Record of Borehole Sheets



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

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

IV. SOIL TESTS

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

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

V. MINOR SOIL CONSTITUENTS

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



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 09-1111-6007		RECORD OF BOREHOLE No DS15-2		SHEET 1 OF 2		METRIC	
G.W.P. 2131-01-00		LOCATION N 4842984.8 ; E 308242.3		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 29, 2015		CHECKED BY NK			

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			
194.5	GROUND SURFACE							20	40	60	80	100			
0.0	ASPHALT (150 mm)														
0.2	Sand, some silt, trace gravel (FILL)		1	SS	20								○		
193.7	Compact Brown Moist		2	SS	6										
0.8	Clayey silt, some sand to sandy clayey silt, trace gravel, trace organics (FILL)		3	SS	17										
	Firm to very stiff		4	SS	13								○		
	Brown to grey Moist		5	SS	13										
			6	SS	10										
			7	SS	11										
			8	SS	8								○		
187.3	Sandy CLAYEY SILT, to CLAYEY SILT with SAND, trace gravel, oxidation staining to a depth of 8.7 m (TILL)		9	SS	32										
7.2	Very stiff to hard														
	Brown, becoming grey below a depth of 9.3 m		10	SS	29										
	Dry to moist														
			11	SS	33								○		
			12	SS	31										
			13	SS	31								○		
180.2	END OF BOREHOLE														
14.3															

Continued Next Page


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

GTA-MTO 001 T:\PROJECTS\2009\09-1111-6007 (URS, TORONTO)\LOG\09-1111-6007.GPJ GAL-GTA.GDT 24/02/16 CD



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

GTA-MTO 001 T:\PROJECTS\2009\09-1111-6007 (URS, TORONTO)\LOG\09-1111-6007.GPJ GAL-GTA.GDT 24/02/16 CD

PROJECT		RECORD OF BOREHOLE No DS15-3		SHEET 2 OF 2		METRIC										
G.W.P. 09-1111-6007		LOCATION N 4842975.9 ; E 308205.3		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 12, 2015		CHECKED BY NK												
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 ---															
172.4	Sandy CLAYEY SILT, trace gravel, some silt and sand pockets, oxidation staining to a depth of 4.4 m (TILL) Stiff to hard Mottled brown, becoming grey below a depth of 3.8 m Dry to moist		13	SS	129	174										4 24 50 22
173																
17.1	END OF BOREHOLE		14	SS	115/18											
	NOTE: 1. Borehole dry upon completion of drilling.															

PROJECT 09-1111-6007		RECORD OF BOREHOLE No DS15-4		SHEET 1 OF 2		METRIC	
G.W.P. 2131-01-00		LOCATION N 4842977.3 ;E 308212.1		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 October 13, 2015		CHECKED BY		NK	

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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 DS15-4			SHEET 2 OF 2			METRIC															
G.W.P. 2131-01-00			LOCATION N 4842977.3 :E 308212.1			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 October 13, 2015			CHECKED BY NK																		
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS			ELEVATION SCALE			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES																			
	--- CONTINUED FROM PREVIOUS PAGE ---																							
	Sandy CLAYEY SILT to CLAYEY SILT with SAND, trace gravel, oxidation staining at a depth of 5.6 m (TILL) Stiff to hard Mottled brown, becoming grey below a depth of 4.6 m Moist		14	SS	106/0.15																			
170.7			15	SS	160																			
18.8	END OF BOREHOLE																							
	NOTE: 1. Borehole dry upon completion of drilling.																							

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PROJECT 09-1111-6007		RECORD OF BOREHOLE No DS15-5		SHEET 1 OF 2	METRIC
G.W.P. 2131-01-00		LOCATION N 4843006.0 ; E 308237.8		ORIGINATED BY QC	
DIST Central HWY 401		BOREHOLE TYPE CME 75, 215 mm O.D. Hollow Stem Augers/Tricone with 125 mm O.D. Casing		COMPILED BY AJS	
DATUM Geodetic		DATE November 1, 2015		CHECKED BY NK	


SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE LIQUID LIMIT			UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa		WATER CONTENT (%)				
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED	20 40 60 80 100	w _p w w _L				
189.0	GROUND SURFACE													
0.0	ASPHALT (150 mm)													
0.2	Gravelly sand, trace silt (FILL)		1	SS	25									
188.3	Compact Brown													
0.7	Dry to moist													
	Sandy clayey silt, trace gravel, some silt pockets (FILL)		2	SS	16									
	Very stiff													
	Mottled brown and grey		3	SS	19									
	Moist													
186.8														
2.2	Sandy CLAYEY SILT, trace to some gravel, oxidation staining to a depth of 4.6 m (TILL)		4	SS	20									
	Very stiff to hard													
	Brown, becoming grey below a depth of 4.6 m		5	SS	32									
	Moist													
			6	SS	30									
			7	SS	31									
			8	SS	17									
			9	SS	47									
			10	SS	35									
			11	SS	33									
			12	SS	32									
175.7														
13.3	SILT, some sand to sandy		13	SS	100/0.18									
	Very dense													
	Grey													
	Wet													
174.1														

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

GTA-MTO 001 T:\PROJECTS\2009\09-1111-6007 (URS, TORONTO)\LOG\09-1111-6007.GPJ GAL-GTA.GDT 24/02/16 CD

PROJECT <u>09-1111-6007</u>		RECORD OF BOREHOLE No DS15-5		SHEET 2 OF 2		METRIC	
G.W.P. <u>2131-01-00</u>		LOCATION <u>N 4843006.0 ; E 308237.8</u>		ORIGINATED BY <u>QC</u>			
DIST <u>Central</u> HWY <u>401</u>		BOREHOLE TYPE <u>CME 75, 215 mm O.D. Hollow Stem Augers/Tricone with 125 mm O.D. Casing</u>		COMPILED BY <u>AJS</u>			
DATUM <u>Geodetic</u>		DATE <u>November 1, 2015</u>		CHECKED BY <u>NK</u>			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC NATURAL LIQUID LIMIT MOISTURE LIMIT CONTENT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)				GR	SA	SI	CL	
								<div><div>20406080100</div><div>○ UNCONFINED + FIELD VANE</div><div>● QUICK TRIAXIAL × REMOULDED</div></div>						<div><div>102030</div><div>w_p w w_L</div></div>							
14.9	SILT and SAND, some clay, trace gravel (TILL) Very dense Grey Moist		14	SS	127/0.20																
171.8				15	SS	128/0.25															
17.2	END OF BOREHOLE																				
	NOTES: 1. Water level not recorded in open borehole upon completion of drilling as water was introduced into the borehole due to method of advancement.																				

PROJECT		RECORD OF BOREHOLE		No DS15-6		SHEET 1 OF 1		METRIC									
G.W.P. 09-1111-6007		LOCATION		N 4842965.0 ; E 308214.2		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 14, 2015		CHECKED BY		NK									
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									
189.2 0.0	GROUND SURFACE ASPHALT (150 mm) CONCRETE (250 mm)		1	AS	-												
188.4 0.8	Sand and gravel (FILL) Brown Moist Sand, trace silt, trace to some gravel (FILL) Very loose to compact Brown Moist to wet		2	SS	16												
			3	SS	7												
			4	SS	1												
186.3 2.9	END OF BOREHOLE NOTE: 1. Water level not recorded upon completion of drilling. 2. Drilling stopped due to proximity to storm sewer as inferred by presence of sand fill.																


PROJECT	09-1111-6007	RECORD OF BOREHOLE No DS15-6A		SHEET 1 OF 2	METRIC
G.W.P.	2131-01-00	LOCATION	N 4842967.4 ;E 308219.9		ORIGINATED BY QC
DIST	Central	HWY	401	BOREHOLE TYPE	CME 75, 215 mm O.D. Hollow Stem Augers/Tricone with 125 mm O.D. Casing
DATUM	Geodetic	DATE	November 2, 2015		CHECKED BY NK
COMPILED BY AJS					

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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 DS15-6A				SHEET 2 OF 2		METRIC										
G.W.P. 2131-01-00		LOCATION N 4842967.4 ; E 308219.9				ORIGINATED BY QC												
DIST Central HWY 401		BOREHOLE TYPE CME 75, 215 mm O.D. Hollow Stem Augers/Tricone with 125 mm O.D. Casing				COMPILED BY AJS												
DATUM Geodetic		DATE November 2, 2015				CHECKED BY NK												
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 20 40 60 80 100					WATER CONTENT (%) 10 20 30						
173.6			14	SS	131		174							4				0 24 53 23
15.7	END OF BOREHOLE NOTES: 1. Water level not recorded in open borehole upon completion of drilling as water was introduced into the borehole due to method of advancement.																	



**FOUNDATION REPORT
HIGHWAY 401 W - YORKDALE ROAD RAMP (SITE NO. 37-284)
OVER DUFFERIN STREET**

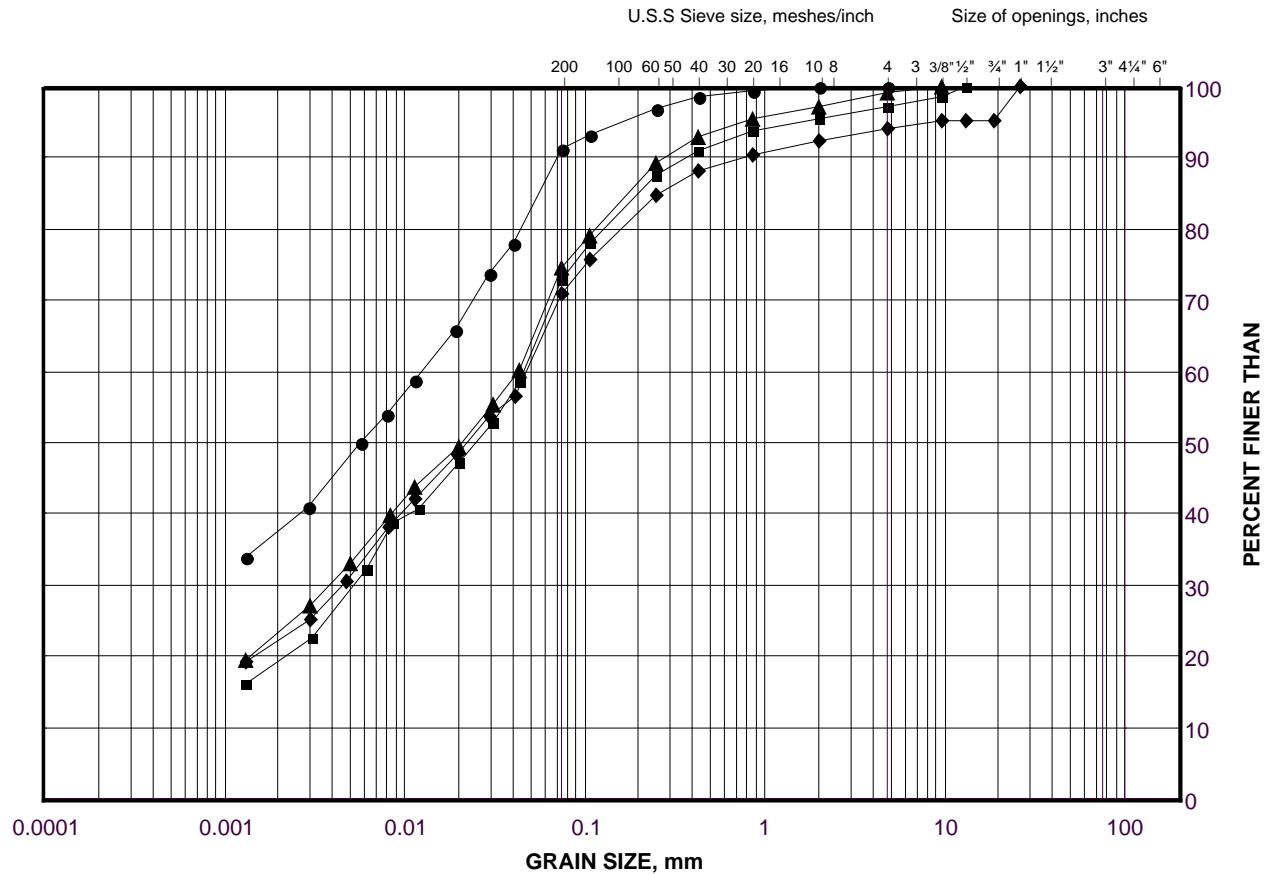
APPENDIX B

Laboratory Test Results

GRAIN SIZE DISTRIBUTION

Clayey Silt to Sandy Clayey Silt (Fill)

FIGURE B1



LEGEND

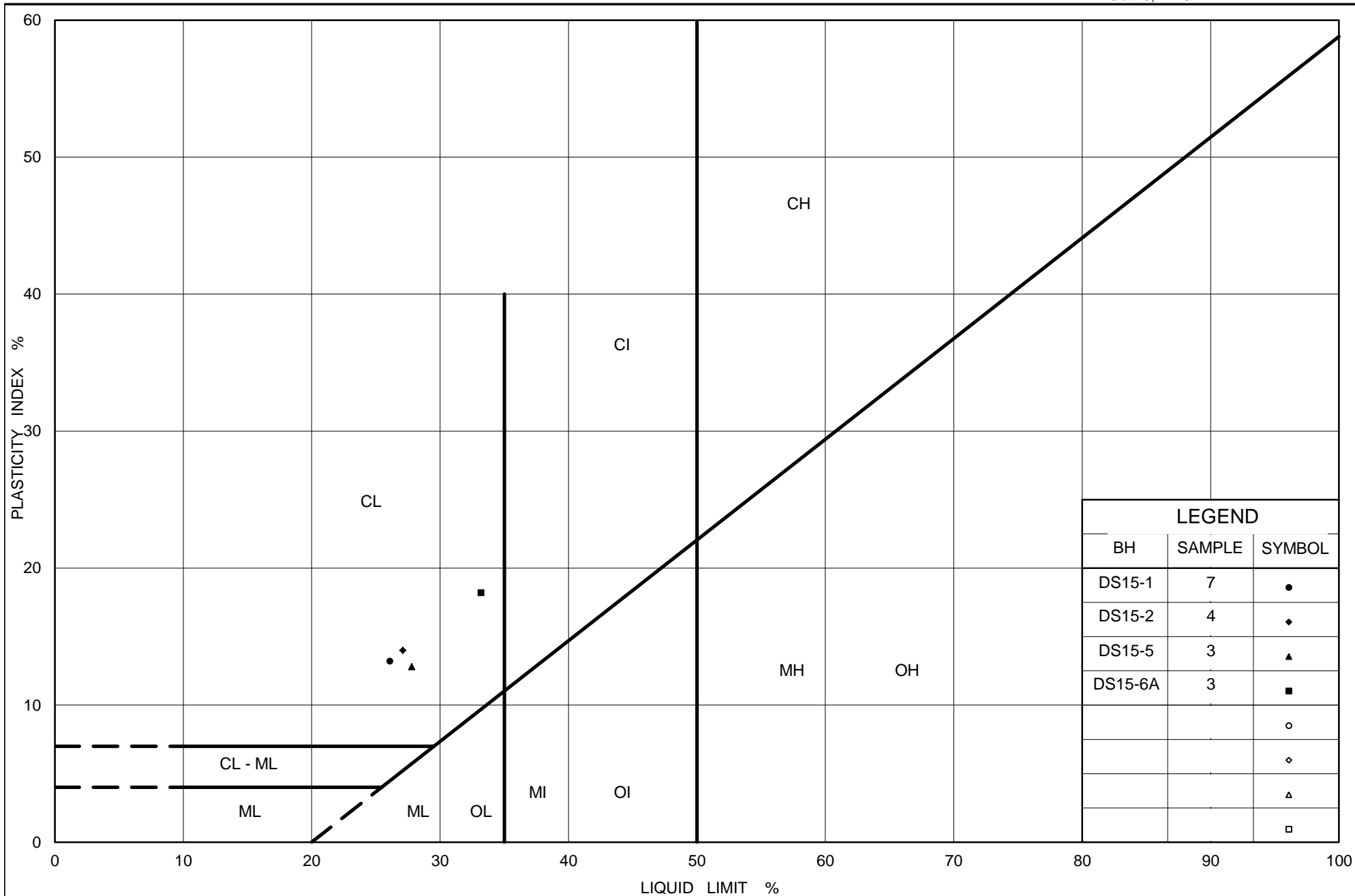
SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	DS15-3	1	188.4
■	DS15-5	2	187.9
◆	DS15-2	6	190.4
▲	DS15-1	7	190.2

Project Number: 09-1111-6007

Checked By: NK

Golder Associates

Date: 11-Feb-16



Ministry of Transportation

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PLASTICITY CHART Clayey Silt to Sandy Clayey Silt (Fill)

Figure No. B2

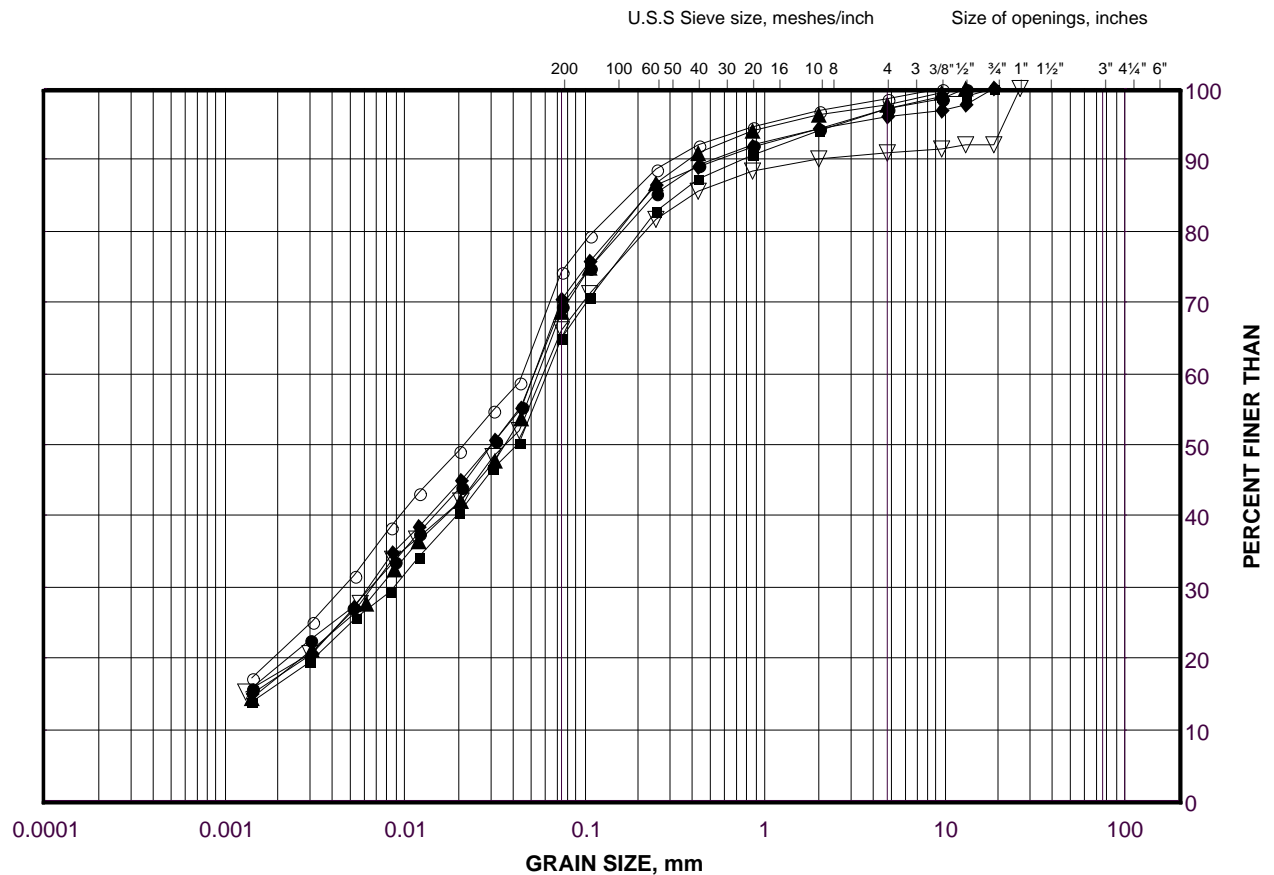
Project No. 09-1111-6007

Checked By: NK

GRAIN SIZE DISTRIBUTION

Sandy Clayey Silt to Clayey Silt with Sand (Till)

FIGURE B3A



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

LEGEND

SYMBOL	STATION	SAMPLE	ELEVATION(m)
●	DS15-1	10	185.6
■	DS15-2	13	180.5
◆	DS15-4	5	186.1
▲	DS15-3	5	185.3
▽	DS15-6A	6	185.2
○	DS15-2	9	186.5

Project Number: 09-1111-6007

Checked By: NK

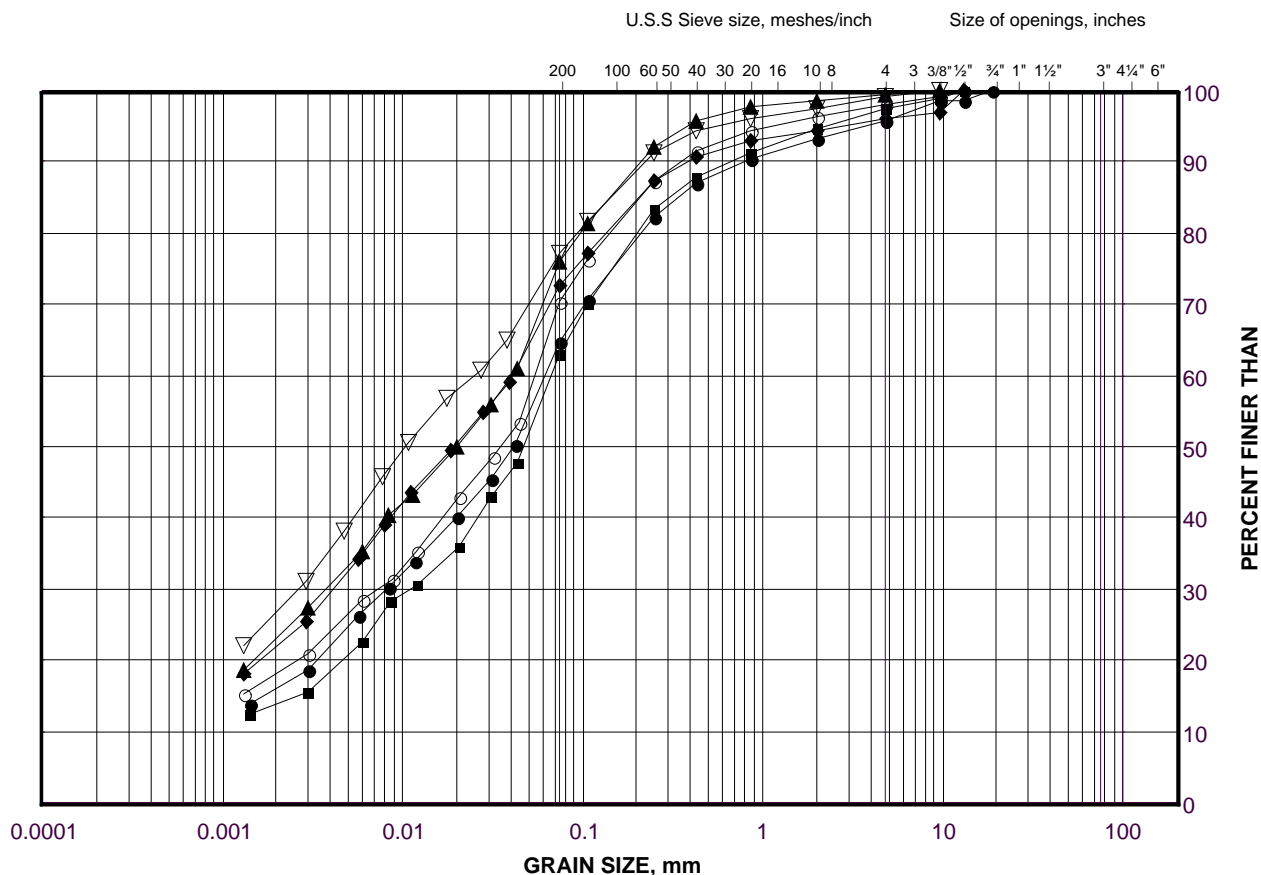
Golder Associates

Date: 11-Feb-16

GRAIN SIZE DISTRIBUTION

Sandy Clayey Silt to Clayey Silt with Sand (Till)

FIGURE B3B



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

LEGEND

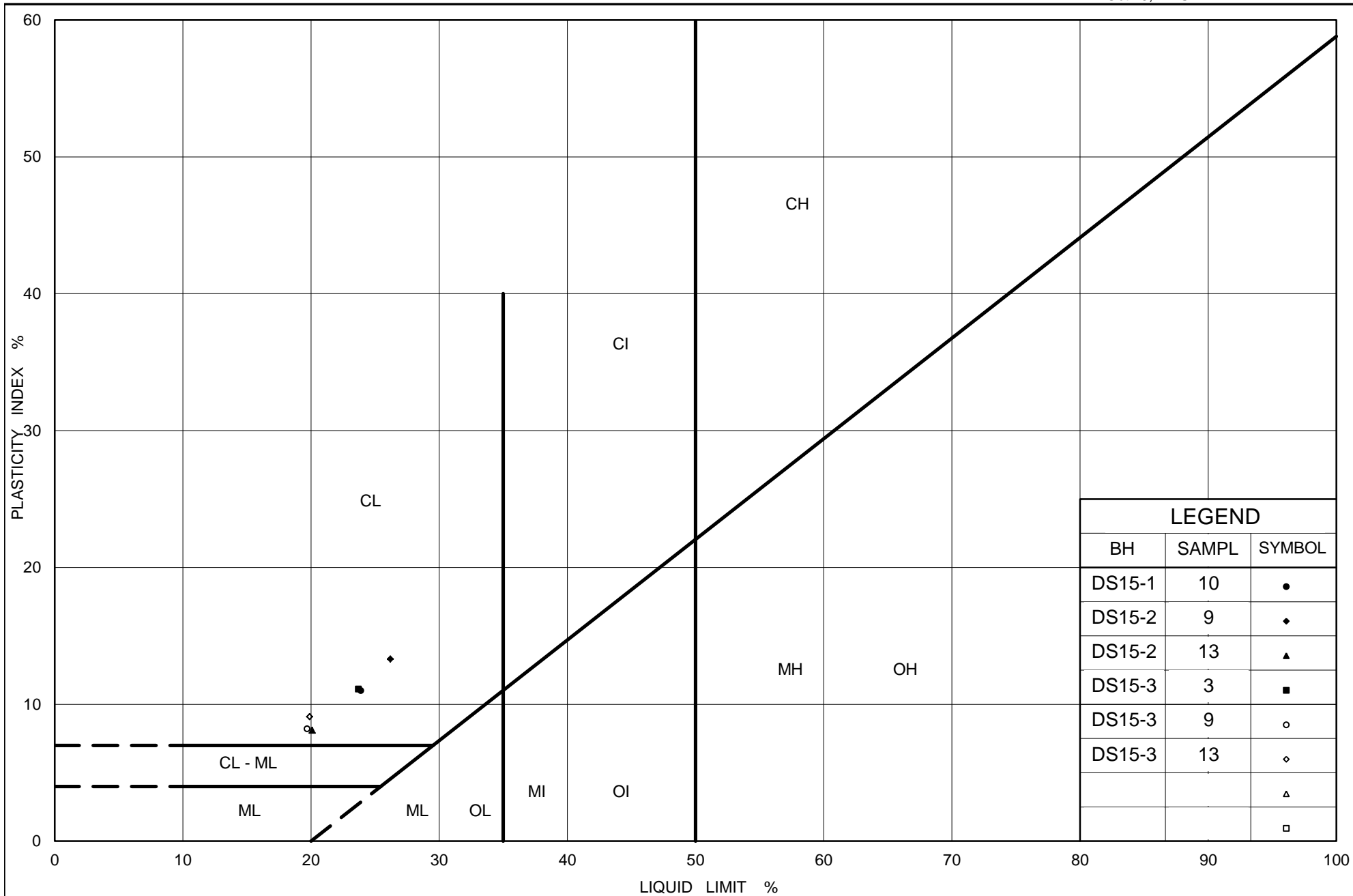
SYMBOL	BOREHOLE	SAMPLE	DEPTH(m)
●	DS15-6A	10	179.8
■	DS15-4	11	178.5
◆	DS15-3	13	174.0
▲	DS-6A	14	173.8
▽	DS15-4	15	171.0
○	DS15-5	7	184.1

Project Number: 09-1111-6007

Checked By: NK

Golder Associates

Date: 11-Feb-16



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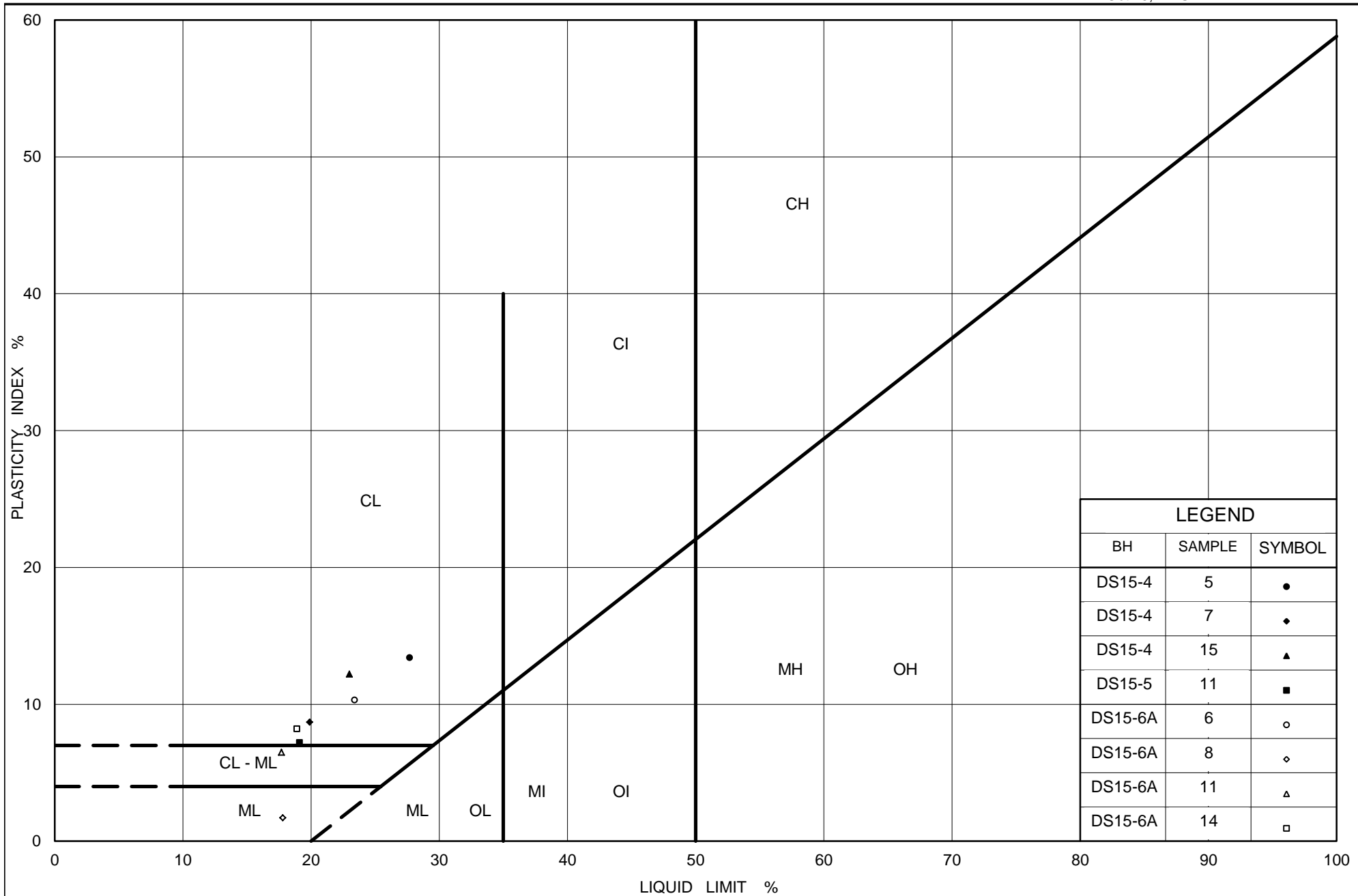
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PLASTICITY CHART **Sandy Clayey Silt to Clayey Silt with Sand (Till)**

Figure No. B4A

Project No. 09-1111-6007

Checked By: NK



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PLASTICITY CHART Sandy Clayey Silt to Clayey Silt with Sand (Till), and Silt (Pocket)

Figure No. B4B

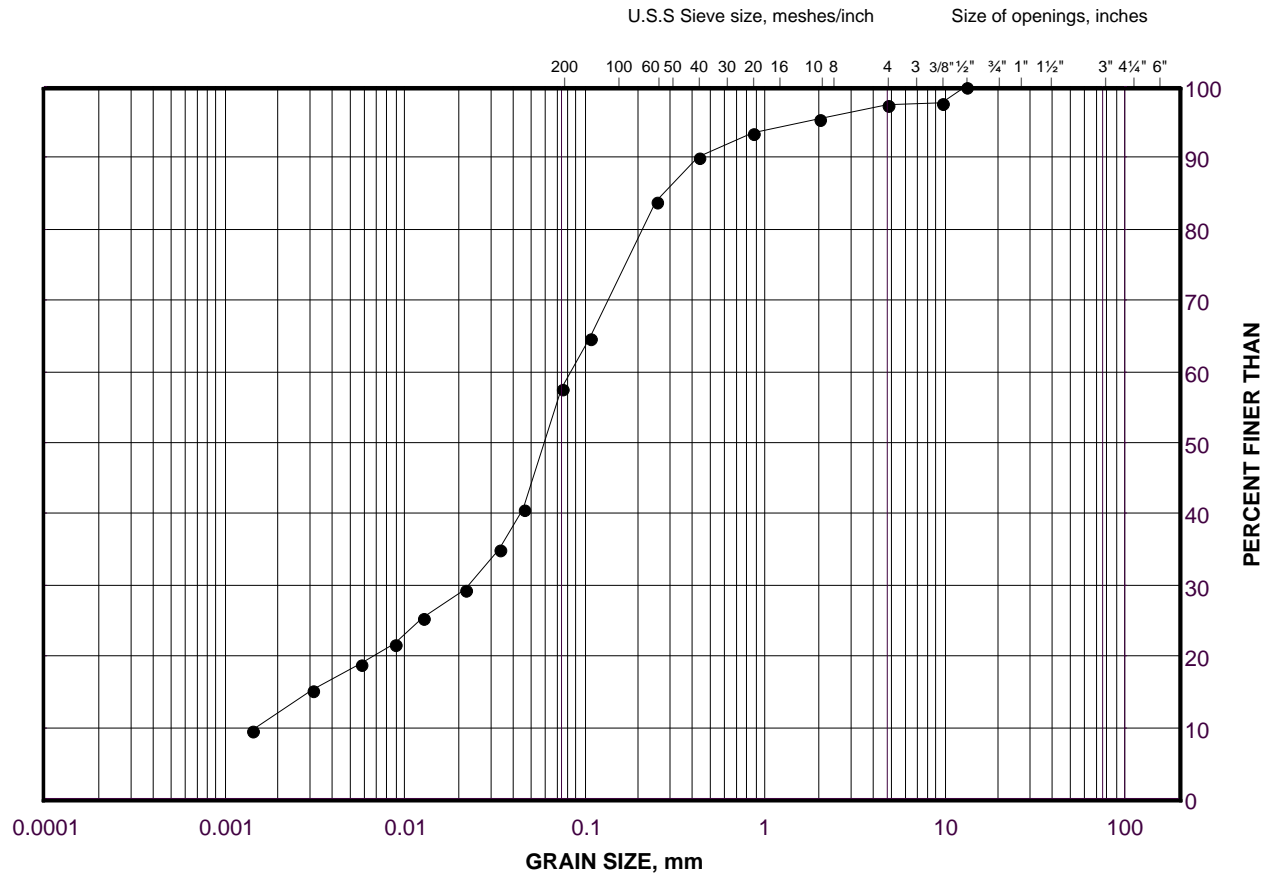
Project No. 09-1111-6007

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GRAIN SIZE DISTRIBUTION

Sand and Silt (Till)

FIGURE B5



LEGEND

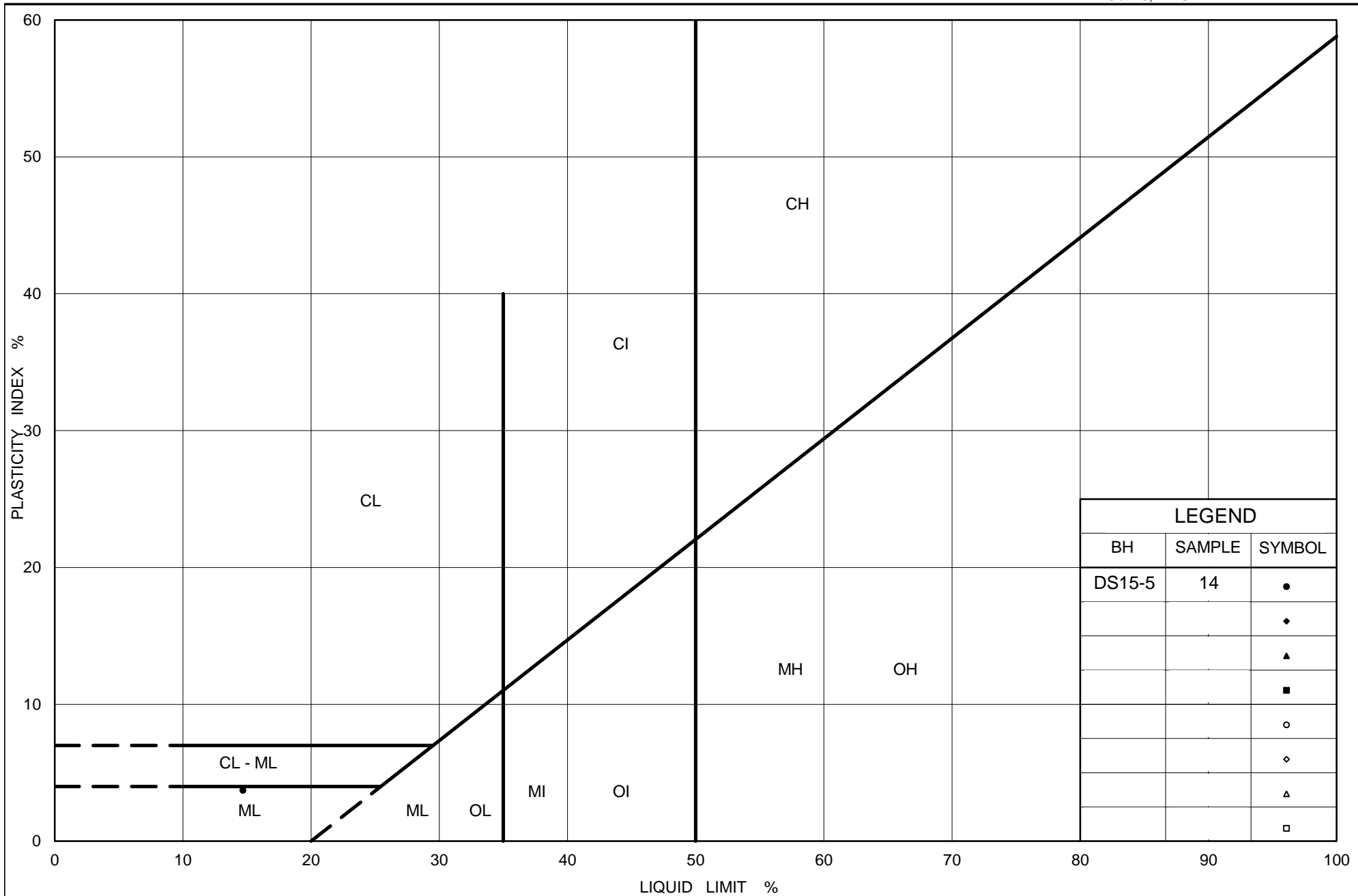
SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
•	DS15-5	15	172.0

Project Number: 09-1111-6007

Checked By: NK

Golder Associates

Date: 11-Feb-16



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PLASTICITY CHART Silt and Sand (Till)

Figure No. B6

Project No. 09-1111-6007

Checked By: NK



**FOUNDATION REPORT
HIGHWAY 401 W - YORKDALE ROAD RAMP (SITE NO. 37-284)
OVER DUFFERIN STREET**

APPENDIX C

Records of Boreholes from Previous Investigation

Borehole No. 81-3B

DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & RESEARCH DIVISION

RECORD OF BOREHOLE NO. 3B

FOUNDATION SECTION

JOB 63-F-24 LOCATION 218+95 470' Rt. ORIGINATED BY B.M.G.
W.P. 233-61-2-2 BORING DATE March 21, 1963. COMPILED BY B.M.G.
DATUM Geodetic BOREHOLE TYPE Pennsylvania Auger - 4 1/2" Ø CHECKED BY K.G.S.

SOIL PROFILE		SAMPLES			DYNAMIC PENETRATION RESISTANCE					LIQUID LIMIT w_L PLASTIC LIMIT w_P WATER CONTENT w		BULK DENSITY γ P.C.F.	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	BLOWS / FOOT	ELEV SCALE	20	40	60	80	100		
620	Topsoil	SS											
0.6	Clayey silt and silty clay with trace of sand and fine gravel. (Glacial Till) Trace of organics to El. 616 V. stiff to hard. Brown changing to grey at El. 609'		1	SS	16	615							
			2	SS	34								
			3	SS	58	610							
			4	SS	59								
			5	SS	32	605							El. 605.2
598.5			6	SS	41	600							
21.6	End of borehole.					595							



**FOUNDATION REPORT
HIGHWAY 401 W - YORKDALE ROAD RAMP (SITE NO. 37-284)
OVER DUFFERIN STREET**

APPENDIX D

Non-Standard Special Provisions



FOUNDATION REPORT HIGHWAY 401 W - YORKDALE ROAD RAMP (SITE NO. 37-284) OVER DUFFERIN STREET

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

10.0 BASIS OF PAYMENT



**FOUNDATION REPORT
HIGHWAY 401 W - YORKDALE ROAD RAMP (SITE NO. 37-284)
OVER DUFFERIN STREET**

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 HIGHWAY 401 W - YORKDALE ROAD RAMP (SITE NO. 37-284) OVER DUFFERIN STREET

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 W – Yorkdale Road ramp over Dufferin Street structure wherein “piling” refers to both driven piles and caissons (bored piles).

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, Highway 401 W – Yorkdale Road Ramp over Dufferin Street (Site No. 37-284), Highway 401 Eastbound Collector Rehabilitation from Jane Street to Avenue Road, Toronto, Ontario, GWP 2131-01-00, Agreement No. 2009-E-0011, GEOCREC No. 30M11-262.

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 Highway 401 W – Yorkdale Road Ramp over Dufferin Street 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
HIGHWAY 401 W - YORKDALE ROAD RAMP (SITE NO. 37-284)
OVER DUFFERIN STREET**

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



FOUNDATION REPORT HIGHWAY 401 W - YORKDALE ROAD RAMP (SITE NO. 37-284) OVER DUFFERIN STREET

CSP FOR INTEGRAL ABUTMENTS – Item No

Non-Standard Special Provision

Scope

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

Submission and Design Requirements

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

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

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

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

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

Material

Corrugated steel pipe

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

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

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

Sand Fill



FOUNDATION REPORT HIGHWAY 401 W - YORKDALE ROAD RAMP (SITE NO. 37-284) OVER DUFFERIN STREET

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
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OVER DUFFERIN STREET**

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
HIGHWAY 401 W - YORKDALE ROAD RAMP (SITE NO. 37-284)
OVER DUFFERIN STREET**

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

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

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