



FOUNDATION INVESTIGATION AND DESIGN REPORT

*Dorchester Road (Middlesex Road 32) Underpass, Site No. 19X-0303/B0
Highway 401/Dorchester Road Underpass Replacement and Interchange
Improvements*

MTO DB 2022-3009, GWP 3053-11-03

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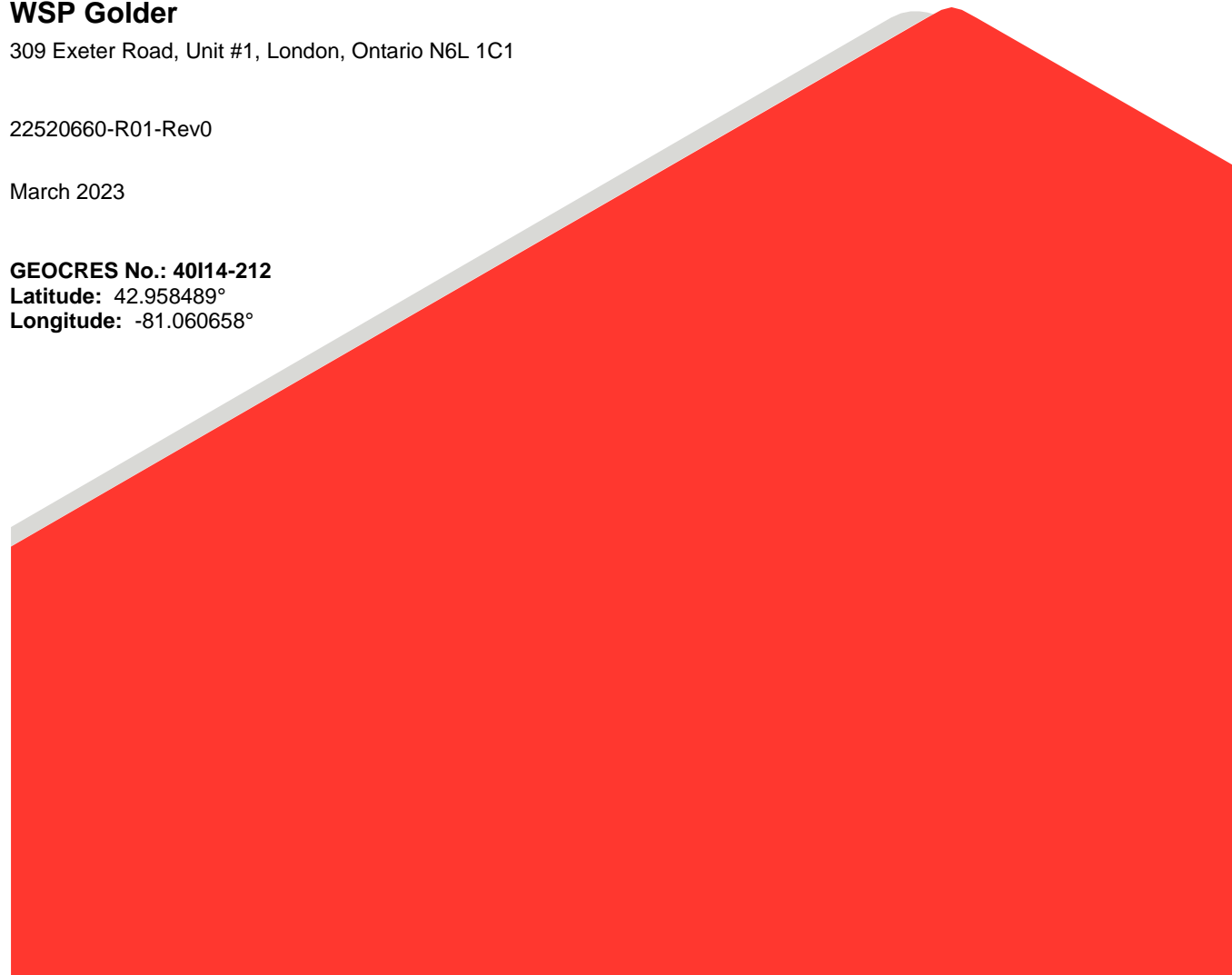
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March 2023

GEOCRES No.: 40114-212

Latitude: 42.958489°

Longitude: -81.060658°

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APPENDIX A

2013 Investigation - GEOCRES No. 40I14-155

APPENDIX B

2019 Investigation - GEOCRES No. 40I14-193

APPENDIX C

Current Investigation

APPENDIX D

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PART A

FOUNDATION INVESTIGATION REPORT

DORCHESTER ROAD (MIDDLESEX ROAD 32) UNDERPASS
STRUCTURE SITE NO. 19X-0303/B0
HIGHWAY 401/DORCHESTER ROAD UNDERPASS REPLACEMENT
AND INTERCHANGE IMPROVEMENTS
MT0 DB 2022-3009, GWP 3053-11-00

1.0 INTRODUCTION

WSP Golder (formerly Golder Associates Ltd., now a member of WSP Canada Inc.) has been retained by Dillon Consulting Limited (Dillon) on behalf of Green Infrastructure Partners (GIP) to provide detailed foundation engineering services as part of Ministry of Transportation Ontario (MTO) Contract DB 2022-3009 for the Highway 401/Dorchester Road underpass replacement and associated interchange improvements. This report addresses the replacement of the Dorchester Road (Middlesex Road 32) underpass structure (MTO Structure Site No. 19X-0303/B0).

2.0 SITE DESCRIPTION

2.1 General

The Highway 401/Dorchester Road underpass is located south of the Village of Dorchester in the Municipality of Thames Centre, Ontario. The structure is located about 3.7 kilometres east and west of Westchester Bourne and Elgin Road, respectively. The location of the project is shown on the Key Plan shown on Drawing 1.

For the purposes of this report, Highway 401 and Dorchester Road are taken to be oriented in an east-west and a north-south direction, respectively. This section of Highway 401 is currently a six-lane divided highway. Highway 401 was constructed in a partial cut at this location. The highway surface is at approximately Elevation 279 metres (m) at the underpass location, while the surrounding natural ground surface varies from approximately Elevation 282 m to 286 m. The pavement surface on the existing Dorchester Road structure is at about Elevation 285 m. The existing underpass structure was constructed in 1955 and consists of a single span, concrete, rigid frame structure. The area adjacent to the site consists of relatively flat-lying agricultural and commercial lands.

The Dorchester swamp, a Class 1 Provincially Significant Wetland (PSW) regulated by the Upper Thames River Conservation Authority, is located to the east of the Highway 401/Dorchester Road interchange. A drainage feature crosses Highway 401 on the east side of the interchange. The available topographic mapping indicates a small area of swamp south of the southeast quadrant.

2.2 Site Geology

This project lies within the physiographic region known as the Westminster Moraine. The physiographic mapping indicates that the Dorchester Road underpass is located on a till moraine.¹ Geological mapping indicates that the surficial material consists of Port Stanley silty clay and clayey silt till which is, in places, covered by thin patches of lacustrine silt.²

The rock formation in the area of the site is described as medium brown, microcrystalline limestone of the Dundee Formation which belongs to the Hamilton Group of Middle Devonian Age.³ The bedrock surface is estimated to be at about Elevation 229 m, some 49 m below Highway 401 at the structure location.

¹ Chapman, L.J. and Putnam, D.F., 1984: The Physiography of Southern Ontario, Third Edition. Ontario Geological Survey, Special Volume 2.

² Dreimanis, A., 1963: Pleistocene Geology of the St. Thomas Area (East Half), Southern Ontario. Ontario Department of Mines, Preliminary Geological Map P.606, scale 1:50,000.

³ Sanford, B.V., 1969: Geology, Toronto-Windsor Area, Ontario. Geological Survey of Canada, Map 1263A, scale 1:250,000.

3.0 INVESTIGATION PROCEDURES

3.1 2013 Investigation (Golder Associates Ltd., MTO GEOCREs No. 40114-155)

Golder Associates Ltd. (Golder) previously carried out a preliminary foundation investigation at this site in March 2013, the results of which were provided in the following report:

- Golder Report 12-1132-0076-2001 “Preliminary Foundation Investigation and Design Report, Middlesex Road 32 (Dorchester Road) Underpass, Site Number 19-303, Highway 401 Interchange Improvements/Structural Replacements, GWP 3053-11-00, Assignment No. 2 (3011-E-0047), Ministry of Transportation, Ontario – West Region” dated March 2017.

The borehole records and the related laboratory testing data from that report are attached in Appendix A. The approximate locations of the boreholes are shown on the Borehole Location Plan, Drawing 1. The table below summarizes the borehole location coordinates (MTM NAD83, Zone 11 northing and easting coordinates as well as latitude and longitude coordinates), ground surface elevations at the borehole locations and borehole depths.

Borehole No.	Location				Ground Surface Elevation (m)	Borehole Depth (m)
	Northing (m)	Easting (m)	Latitude (°)	Longitude (°)		
401	4,758,600.3	422,220.7	42.958153	-81.060822	285.5	30.6
402	4,758,589.5	422,223.2	42.958056	-81.060794	285.3	9.6
403	4,758,678.9	422,216.9	42.958861	-81.060852	286.1	9.6
404	4,758,666.6	422,219.5	42.958750	-81.060823	286.2	28.7

The investigation was carried out using truck-mounted drilling equipment supplied and operated by a specialist drilling contractor using a combination of hollow stem augering and wash boring techniques. In the boreholes, samples of the overburden were obtained at generally 0.75 m or 1.5 m intervals of depth using 50 millimetre (mm) outside diameter split spoon sampling equipment in accordance with ASTM D 1586. The samples used in the investigation limit the maximum particle size that can be sampled and tested to about 40 millimetres. Therefore, particles that may exist within the soils that are larger than this dimension will not be sampled or represented in the grain size distributions. Larger particle sizes including cobbles and boulders are known to be present in the glacial till materials. The results of the SPT testing as presented on the borehole records in Appendix A and discussed subsequently in this report have not been factored to account for the use of an automatic hammer.

Groundwater conditions in the boreholes were observed throughout the drilling operations and piezometers were installed in Boreholes 401 and 402 as indicated on the corresponding borehole records in Appendix A. The boreholes were backfilled in accordance with MTO procedures and Ontario Regulation 903 (as amended).

3.2 2019 Investigation (Peto MacCallum Ltd., GEOCREs No. 40I14-193)

Peto MacCallum Ltd. (PML) carried out a foundation investigation at this site in September 2019, the results of which were provided in the following report:

- PML Report 19KF028A titled “Foundation Investigation Report for Improvements to Highway 401 and Dorchester Road Interchange, Site No. 19-303, Highway 401, London, Ontario, GWP 3053-11-00, Assignment No. 3016-E-0009; Work Order No. 19” dated April 2020.

The records of four boreholes advanced at the underpass site and the related laboratory testing data from that report are attached in Appendix B and the approximate locations of the boreholes are shown on the Borehole Location Plan, Drawing 1. The table below summarizes the borehole location coordinates (referenced to MTM NAD83, Zone 11 coordinates as well as latitude and longitude), ground surface elevations at the borehole locations and borehole depths.

Borehole No.	Location				Ground Surface Elevation (m)	Borehole Depth (m)
	Northing (m)	Easting (m)	Latitude (°)	Longitude (°)		
N	4,758,664.6	422,208.9	42.958734	-81.060954	286.2	30.6
C1	4,758,632.7	422,231.3	42.958442	-81.060686	279.3	30.0
C2	4,758,626.0	422,200.1	42.958388	-81.061071	279.3	30.0
S	4,758,596.0	422,245.3	42.958110	-81.060523	284.5	30.6

According to the above-referenced report, the boreholes were advanced using continuous flight hollow stem augers.

3.3 Current Investigation

Two additional boreholes, identified as Boreholes 101 and 102, were advanced at the site on November 15, 2022 and January 9 to 11, 2023. The borehole records and the associated laboratory testing data are attached in Appendix C, and the locations of the boreholes are shown on the Borehole Location Plan on Drawing 1. The table below summarizes the borehole location coordinates (referenced to MTM NAD83, Zone 11 coordinates, as well as latitude and longitude), ground surface elevations at the borehole locations and borehole depths.

Borehole No.	Location				Ground Surface Elevation (m)	Borehole Depth (m)
	Northing (m)	Easting (m)	Latitude (°)	Longitude (°)		
101	4,758,711.4	422,218.6	42.959153	-81.060825	284.4	30.9
102	4,758,588.5	422,278.7	42.958038	-81.060114	279.6	30.9

The boreholes were advanced using casing with mud rotary drilling techniques. Samples of the overburden were obtained at 0.75 m, 1.5 m and 3.0 m intervals of depth using 50 mm outside diameter split spoon sampling

equipment. The results of the SPT testing as presented on the borehole records in Appendix C and discussed subsequently in this report have not been factored to account for the use of an automatic hammer.

On completion of drilling, 63.5 mm casing was installed in each of these boreholes to facilitate vertical seismic profiling (VSP) to determine the shear wave velocity of the soil within the upper 30 m of the soil profile. The field work was conducted on December 14, 2022 and January 18, 2023 by WSP Golder personnel. The seismic shear wave source used was a 2.4 m long, 150 mm by 150 mm beam. Each end of the beam was struck with a 5.5 kg sledgehammer to induce oppositely polarized shear waves.

4.0 SUBSURFACE CONDITIONS

4.1 Site Stratigraphy

The detailed subsurface soil and groundwater conditions encountered in the boreholes, together with the results of the in situ testing and laboratory testing carried out on selected samples, are provided on the borehole records in Appendices A, B and C and on Drawings 1 and 2. The stratigraphic boundaries shown on the borehole records are inferred from non-continuous samples and observations of drilling resistance and, therefore, may represent transitions between soil types rather than exact planes of geological change. Further, the subsurface conditions will vary between and beyond the borehole locations. Descriptions of the subsurface conditions encountered in the boreholes are provided in the following subsections of this report.

Groundwater levels/conditions encountered in the boreholes during and shortly after drilling may not be representative of static groundwater levels since the groundwater levels in the boreholes may not have stabilized. Groundwater levels and seepage conditions in the area will fluctuate seasonally and in response to precipitation.

The boreholes drilled at the site generally encountered the existing pavement structures and fill materials overlying an extensive deposit of clayey silt to silty clay which has been interpreted to be glacial till, which is in turn underlain by layers of sand, silty sand and/or sand and gravel.

4.1.1 Pavement Structure and Fill Dorchester Road

Boreholes 401 to 404 were advanced through the paved shoulder on Dorchester Road and encountered about 210 to 240 mms of asphaltic concrete at the pavement surface. Beneath the asphalt, about 60 to 400 mm of granular base was encountered. In Borehole 401, the granular base had an SPT N value of 65 blows per 0.3 m.

Inferred granular subbase was encountered beneath the granular base in Boreholes 402 and 403. The subbase was about 490 to 940 mm thick at the borehole locations. The subbase in Borehole 403 had an SPT N value of 20 blows per 0.3 m. Cobbles were noted in the granular subbase.

In Boreholes 401, 404 and N, sand and gravel embankment fill materials were encountered. The embankment fill ranged from about 4.6 to 5.2 m thick. Cobbles were noted in the fill during drilling. The granular embankment fill had SPT N values ranging from 6 to 50 blows per 0.3 m and water contents of about 2% to 20%. Grain size distribution test results for the fill are shown on Figure A-1 in Appendix A.

Remnants of temporary works may be buried in the fill. Further, Department of Highways (DHO) Drawing No. D-3492-2 "Abut. And Handrail Details – Fin. Road Elevs." Dated Dec. 8, 1954 indicates that mass concrete with a

compressive strength of 10 megapascals (MPa) was placed below the western half of the north abutment. The mass concrete extends about 0.6 to 1.8 m below the underside of the abutment footing. DHO Drawing No. D-3492-2 has been included in Appendix D for reference.

Highway 401

Boreholes C-1 and C-2 were drilled through the Highway 401 median shoulder. Borehole C-1 encountered about 400 mm of asphalt at the pavement surface underlain by about 1.4 m of sand and gravel fill, while Borehole C-2 encountered about 600 mm of asphalt overlying about 0.9 m of sand and gravel fill. The fill had N values of about 8 to 31 blows per 0.3 m.

4.1.2 Topsoil

Approximately 180 mm of topsoil was encountered beneath the fill material in Borehole 404 at about Elevation 281.2 m, and approximately 500 mm of topsoil was encountered at ground surface in Borehole 102. Materials designated as topsoil in this report were classified solely based on visual and textural evidence. Testing of organic content or for other nutrients was not carried out. Therefore, the use of materials classified as topsoil cannot be relied upon for support and growth of landscaping vegetation.

4.1.3 Surficial Silt to Silty Sand

Layers of compact to very dense silt were encountered beneath the fill materials in Boreholes 402 and 403, and a layer of loose to compact silty sand was encountered immediately below the ground surface in Borehole 101. These layers were about 0.8 m to 3.0 m thick, and the surface of these layers was encountered between about Elevations 284.8 m and 281.9 m. Measured SPT N values ranged from 7 to 57 blows per 0.3 m. Samples of the material had a water content of about 10% to 20%. A grain size distribution curve for a sample of the silt is shown on Figure A-2 in Appendix A.

4.1.4 Clayey Silt to Clayey Silt Glacial Till

Firm to hard clayey silt to clayey silt glacial till was encountered in all boreholes at the site; the surface of this deposit was generally encountered between about Elevation 284.5 m and 279.1 m, except within the Highway 401 cut where it was encountered at approximately Elevation 277.5 m to 277.8 m in Boreholes C-1 and C-2. The clayey silt to clayey silt glacial till was about 16.1 to 25.5 m thick where fully penetrated. Boreholes 402 and 403 were terminated in the clayey silt to clayey silt glacial till after exploring it for about 6.0 to 7.5 m.

These cohesive layers generally had SPT N values ranging from 5 to 36 blows per 0.3 m; however, an SPT N value greater than 100 blows per 0.3 m was recorded in Borehole 401 and an SPT N value of 71 blows per 0.3 m was recorded in Borehole 404. Samples of the clayey silt to clayey silt glacial till had water contents of between about 11% and 24%, plastic limits ranging from about 14% to 21%, liquid limits from about 21% to 35%, and plasticity indices of approximately 7 to 18. These data are shown on the Plasticity Charts shown on Figure A-7 in Appendix A and Figures PC-DR1-1A and -1B in Appendix B. The results of grain size determinations carried out on samples of the clayey silt are shown on Figures A-3 and A-4 in Appendix A, and Figures GS-DR1-1A and -1B in Appendix B. Cobbles and boulders should be expected in the clayey silt to clayey silt glacial till strata.

4.1.5 Sand and Gravel

Compact to very dense sand and gravel was encountered in Borehole 401 beneath the clayey silt to clayey silt till at Elevation 260.7 m; this sand and gravel layer was about 2.9 m thick. The sand and gravel had SPT N values of 19 to 53 blows per 0.3 m with a water content of about 14%. A grain size distribution curve for a sample of the

sand and gravel is shown on Figure A-5 in Appendix A. Cobbles were noted in the sand and gravel during drilling and boulders should be expected.

4.1.6 Lower Sand to Silty Sand

A lower deposit of sand to silty sand was encountered in all of the deeper boreholes at this site (i.e., Boreholes 401, 404, N, S, 101, 102 and C-1) with its surface between about Elevation 257.8 and 264.6 m. All these boreholes, except Borehole C-1, were terminated in the sand to silty sand after exploring it for about 2.9 to 7.2 m. Where fully penetrated in Borehole C-1, the sand to silty sand was about 10.6 m thick. The SPT N values measured in this deposit in boreholes advanced by Golder and WSP Golder were generally greater than 30 blows per 0.3 m, while lower SPT N values of 8 to 30 blows per 0.3 m were measured in Boreholes N, S, C-1 and C-2 advanced by PML; the PML boreholes were advanced using hollow stem augers and may not have included measures to mitigate sample disturbance in these water-bearing soils. Hence, it is considered that the SPT N values in Boreholes 101, 401 and 404 (advancing via casing and wash boring) are more representative, corresponding to a dense to very dense relative density.

The measured water contents range from about 8% to 21%. Grain size distribution curves for samples of the lower sand to silty sand are shown on Figure A6 in Appendix A, and Figure GS-DR1-2 in Appendix B. Cobbles were encountered in the lower sand to silty sand during drilling and boulders should be expected.

4.1.7 Lower Clayey Silt Glacial Till

A layer of hard clayey silt glacial was encountered at Elevation 250.8 m in Borehole C-1. Borehole C-1 was terminated in this lower glacial till after exploring it for about 1.5 m. The glacial till had SPT N values greater than 100 blows per 0.3 m and water contents of about 7% to 9%.

A grain size distribution test result is shown on Figure GS-DR1-3 in Appendix B. Cobbles and boulders should be expected in the clayey silt glacial till strata. An Atterberg Limits test was completed on a sample of this deposit and measured a plastic limit of about 12%, liquid limit of 20%, and plasticity index of approximately 8%; the result is plotted on Figure PC-DR1-3 in Appendix B.

4.2 Groundwater Conditions

Groundwater was encountered during drilling at depths of approximately 15 m to 21.7 m, corresponding to Elevation 262.5 m to 269.5 m (as observed in boreholes for which fluids were not introduced associated with wash boring or mud rotary drilling). However, these measurements do not represent the stabilized groundwater level at the site. A monitoring well was installed in the lower sand in Borehole 401, and shallow monitoring wells were installed in the upper portion of the clayey silt till deposit in Boreholes N, S and C-1. The water levels measured in the monitoring wells are summarized in the following table:

Borehole No.	Ground Elevation (m)	Deposit in Which Monitoring Well Screened	Measured Groundwater Level Depth / Elevation (m)		
			05 Jun 2013	03 Dec 2019	05 Mar 2020
401	285.5	Lower Sand	18.0 / 267.5	-	-
N	286.2	Till	-	Dry to 7.7 / 278.5	Dry to 7.7 / 278.5
S	284.5	Till	-	Dry to 4.6 / 279.9	Dry to 4.6 / 279.9
C-1	279.3	Till	-	Dry to 7.6 / 271.7	-

Based on the groundwater levels measured in the piezometers and the soil colour change from brown to grey, the groundwater level associated with the clayey silt to clayey silt glacial till is interpreted to be at approximately Elevation 279 m. Groundwater in the underlying lower sand to silty sand and sand and gravel deposits is interpreted at approximately Elevation 267.5 m based on one reading for a monitoring well screened in this deposit. Groundwater levels are expected to fluctuate seasonally and are expected to be higher during periods of sustained precipitation.

5.0 CLOSURE

This Foundation Investigation Report was prepared by Michael Beadle, P. Eng., a Senior Principal with WSP Golder. Lisa Coyne, P. Eng., a Fellow and MTO Designated Principal Foundations Contact with WSP Golder, conducted an independent review of the report.

WSP Golder



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[https://golderassociates.sharepoint.com/sites/160773/project files/6 deliverables/r01-fidr dorchester up/db 2022-3009 22520660-r01-rev0 final fidr dorchester up_2023-03-16.docx](https://golderassociates.sharepoint.com/sites/160773/project%20files/6%20deliverables/r01-fidr%20dorchester%20up/db%202022-3009%2022520660-r01-rev0%20final%20fidr%20dorchester%20up_2023-03-16.docx)

FOUNDATION DESIGN REPORT

**DORCHESTER ROAD (MIDDLESEX ROAD 32) UNDERPASS
STRUCTURE SITE NO. 19X-0303/B0
HIGHWAY 401/DORCHESTER ROAD UNDERPASS REPLACEMENT
AND INTERCHANGE IMPROVEMENTS
MTO DB 2022-3009, GWP 3053-11-00**

6.0 ENGINEERING RECOMMENDATIONS

6.1 General

This section of the report provides geotechnical recommendations and discussion on the foundation aspects for design of the Dorchester Road underpass replacement. The recommendations are based on our interpretation of the factual information obtained during the previous investigations completed by Golder in 2013, by PML in 2019 (as provided by MTO as part of the RFP for DB 2022-3009), and per our validation investigation in December 2022 and January 2023. This Foundation Design Report, with the interpretation and recommendations, is intended for the use of the design engineer. Where comments are made on construction, they are provided only to highlight those aspects which could affect the design of the project. Contractors must make their own interpretation of factual information provided as it may affect equipment section, proposed construction methods and scheduling.

The recommendations in this report are in accordance with the *Canadian Highway Bridge Design Code* (CHBDC 2019, CAN/CSA-S6-19) and its *Commentary*, the *Canadian Foundation Engineering Manual* (CFEM, 2006), Ontario Provincial Standard Specifications (OPSS), Design-Build Special Provisions (DBSP) and Ontario Provincial Standard Drawings (OPSD).

6.2 Existing and Proposed Structure and Selected Foundation Type

As part of DB 2022-3009, the existing rigid frame structure will be demolished and a new two-span bridge will be constructed over Highway 401. The centreline of the new underpass will be offset approximately 12 m east of the centreline of the existing structure. The existing structure will not be in service during construction of the new bridge.

The existing underpass structure was constructed in 1955, and is a rigid frame structure with a single span of 32.9 m and a curb-to-curb width of 13.4 m. The deck is comprised of eight reinforced concrete “tee” girders at 2.15 m spacings on centre. Detail design information and as-built drawings related to the foundations for the existing underpass are not available; however, based on GEOCRE Report No. 40I14-9⁴ and the original contract drawings, it appears that the abutments and wingwalls are founded on shallow footings. Based on DHO Drawing Nos. D-3498-1 and D-3498-2, dated December 8, 1954, the underside of footing elevation for the existing abutments and retaining walls are summarized as follows:

Existing Bridge Element	Founding Elevation (m)
North Abutment	276.49
Northwest Retaining Wall	277.68
South Abutment	276.10
Northeast Retaining Wall	276.49
Southwest Retaining Wall	277.70
Southeast Retaining Wall	276.91

⁴ GEOCRE Report No. 40I14-9 “Soil Investigation for Department of Highways of Ontario (M.M. Dillon, Consulting Engineers) at Dorchester #10” dated September 22, 1954.

GEOCRE Report No. 40114-9 noted that a “soft” area was present in the west half of the north abutment excavation. The north abutment elevation drawing indicates that mass concrete with a compressive strength of 10 megapascals (MPa) was placed in subexcavations (below the underside of footing level) from the west end of the north abutment to a point about 7.3 m east of the west end of the abutment, with depths varying from about 1.8 m to 0.6 m respectively, stepped up twice over this length. DHO Drawing Nos. D-3498-1 and 2 are included in Appendix D for reference.

The new two-span underpass will have integral abutments supported on steel H-piles driven into the lower silty sand to sand deposit, with the centre pier supported on shallow foundations. In conjunction with the easterly alignment shift, a grade raise of up to approximately 1 m and 1.5 m is required immediately adjacent to the abutments at the north and south approaches, respectively.

6.3 General Design Considerations

6.3.1 Consequence and Site Understanding Classification

The proposed bridge crosses over Highway 401, which carries large volumes of traffic with the potential to impact alternative transportation corridors. In accordance with Section 6.5 of the 2019 *Canadian Highway Bridge Design Code CAN/CSA S6-19* and its *Commentary* (CHBDC 2019), the proposed bridge and its foundation system is classified as having a “typical consequence level” associated with exceeding limits states design. In addition, given the level of foundation investigation completed to date at this location in comparison to the degree of site understanding in Section 6.5 of the CHBDC (2019), the level of confidence for design is considered to be a “typical degree of site and prediction model understanding.” Accordingly, the appropriate corresponding ULS and SLS consequence factor, ψ , from Table 6.1 and geotechnical resistance factors, ϕ_{gu} and ϕ_{gs} , from Table 6.2 of the CHBDC have been used for design.

6.3.2 Seismic Design

The vertical seismic profiling (VSP) results report an average shear wave velocity, calculated from the time taken for the shear wave to travel from surface to a depth of 30 m, of 328 m/s in Borehole 101 near the north abutment and 306 m/s in Borehole 102 near the south abutment. According to the CHBDC (2019) classification for seismic site response, this corresponds to Site Class D (Stiff Soil) based solely on the shear wave velocity.

In accordance with the DB RFP, the 2020 National Building Code of Canada (NBCC) Seismic Hazard Tool is to be used for design. For a shear wave velocity of 328 m/s, the NBCC 2020 spectral acceleration for a 2% in 50 year probability of exceedance includes $S(0.2s, X_{328}) = 0.194$ and $S(1.0, X_{328}) = 0.0772$; for the governing (lower) shear wave velocity of 306 m/s, the NBCC 2020 spectral acceleration for a 2% in 50 year probability of exceedance includes $S(0.2s, X_{306}) = 0.192$ and $S(1.0, X_{306}) = 0.081$. Based on Site Class categorization criteria included in Table 4.10 of CHBDC (2019), the structure is categorized as Seismic Performance Category 1 and a seismic analysis is not required per Clause 4.4.5.1 of CHBDC (2019). As such, further geotechnical recommendations for seismic design are not included in this report.

6.3.3 Seismic Liquefaction Assessment

A preliminary screening of the soil stratigraphy was conducted using the procedure outlined by the FHWA.⁵ Although saturated granular materials are present at depth, they generally have normalized SPT N values greater

⁵ FHWA, 1997: “Design Guidance: Geotechnical Earthquake Engineering For Highways. Volume I – Design Principles.” *Geotechnical Engineering Circular No. 3: FHWA-SA-97-076*, Washington, D.C.

than 22 blows per 0.3 metres. The liquefaction potential is low based on the soil profile type, age of the deposits, relative density and the historically low seismicity of this area. Therefore, a detailed evaluation of the liquefaction potential of the foundation soils, impact of liquefaction on the bridge foundations, and the effect of seismic forces on embankment stability is not considered warranted unless the structure is considered a lifeline bridge.

6.4 Driven Steel H-Piles at Abutments

6.4.1 Geotechnical Resistance

For design, the factored ultimate and serviceability axial geotechnical resistances for HP 310 x 110 and HP 310 x 132 piles driven into the very dense lower silty sand to sand deposit are provided in the following table. The factored serviceability geotechnical resistance values correspond to an estimated total pile settlement of less than 25 mm.

Foundation Element	Founding Stratum	Maximum (Highest) Pile Tip Elevation (m)	Factored Ultimate Geotechnical Resistance (kN)	Factored Serviceability Geotechnical Resistance (kN)
South Abutment	Very dense sand	256.0	1,600	1,300
North Abutment	Very dense sand	260.0	1,600	1,300

The clayey silt to clayey silt glacial till and the lower sand to sand and gravel layers are known to contain cobbles and boulders which may interfere with driving of the piles or cause damage to pile tips, as discussed further in Section 6.8.3. Prefabricated driving shoes should be provided.

Pile installation should be in accordance with DBSP 903 (Deep Foundations). The pile termination or set criteria will be dependent on the pile driving hammer type, helmet, selected pile size and length of pile. The set criteria must therefore be established at the time of construction once the piling equipment is confirmed. The pile capacity should be verified in the field by the use of both the Hiley formula (MTO Standard Drawing SS103-11) during the final stages on all piles, and high strain dynamic testing (more commonly known as pile dynamic analyzer (PDA) testing) on a minimum of two piles at each foundation element. The following note from MTO's Structural Manual should be shown on the Contract Drawing, assuming the use of HP 310x110 or HP 310x132 piles and that a resistance factor of 0.5 is applied to the use of the Hiley formula and PDA test results:

- Piles to be driven in accordance with Standard SS103-11 and/or Pile Dynamic Analyzer (PDA) testing using an ultimate geotechnical resistance of 3,200 kN per pile at the abutments. *[Note to structural designers: Adjust the stated ultimate geotechnical resistance to reflect two times the design load, if less than the geotechnical resistances values recommended herein.]*

Assessment of the ultimate geotechnical resistance by the Hiley formula and high-strain dynamic testing should commence once the pile reaches a depth of not less 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.

6.4.2 Frost Protection

Where frost-susceptible soils are present, the pile caps should be provided with a minimum of 1.2 m of soil cover or thermal equivalent above the underside of pile cap elevation for frost protection in accordance with OPSD 3090.101. For the integral abutments at this site, frost protection can be achieved for lesser embedment of the abutment stem walls below the top of slope paving, provided that free-draining granular material and a subdrain are provided below the abutments to a minimum depth of 1.2 m below the lowest surrounding grade.

6.4.3 Downdrag Load (Negative Skin Friction)

Grade raises of approximately 1 m and 1.5 m are required immediately adjacent to the abutments at the north and south approaches, respectively; slightly thicker fills will be placed toward the east end of the new abutments associated with the eastward shift and embankment widening. Considering this relatively modest grade raise and the stiff to hard nature of the cohesive soil deposit at this site, settlements will be less than 25 mm and negligible negative skin friction is expected to develop for the new piles at both abutments.

6.4.4 Resistance to Lateral Loads

For vertical piles, the resistance to lateral loading will be derived solely from the soil surrounding the piles, whereas inclined 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. Where ground conditions are generally competent and the lateral loads on piles are relatively small such that the maximum lateral pile deflections will be relatively small, the resistance to lateral loading in front of a single pile can be estimated using subgrade reaction theory (as outlined below). However, the response of a pile to lateral loads is highly nonlinear and methods that assume linear behavior (such as subgrade reaction theory) are most appropriate where the maximum pile deflections are less than 1% of the pile width or diameter, where the loading is static (no cycling) and where the pile material is linear as per *Canadian Foundation Engineering Manual* (CFEM, 2006). Where these conditions are not met, and/or where required for the structural engineering model, the non-linear lateral behavior of the soil should be considered using P-y curves.

The factored serviceability geotechnical response of the soil in front of the piles under lateral loading at this site may be calculated using subgrade reaction theory, where the coefficient of horizontal subgrade reaction, k_h (kPa/m), is based on the equations given below, as described by Terzaghi (1955) and the *Canadian Foundation Engineering Manual* (CFEM, 1992).

$$\begin{aligned}
 k_h &= \text{coefficient of horizontal subgrade reaction (MPa/m)} &= n_h (z/d) &\text{for cohesionless soils} \\
 & &= \frac{67 S_u}{d} &\text{for cohesive soils}
 \end{aligned}$$

where:

- d = pile width or diameter (m)
- n_h = constant of horizontal subgrade reaction (MPa/m)
- S_u = undrained shear strength of the soil (MPa)
- z = depth below pile cap (m)

The following values of n_h (American Petroleum Institute (API), 2002) and S_u may be incorporated into the calculations of horizontal subgrade reaction (k_h) for structural analyses for a single vertical pile. The ranges in values reflect the variability in the subsurface conditions, the soil properties, the approximate nature of the analysis and the non-linear nature of the soil behaviour (such that k_h is a function of deflection).

Location	Soil Type	Elevation (m)	nh (MPa/m)	Su (kPa)
CSPs for integral abutments	Granular backfill	Where applicable	5 - 10	-
South Abutment	Existing granular fill – loose to compact sand and gravel	280 to 285	10 - 15	-
	Stiff to hard clayey silt to clayey silt glacial till	260 to 280	-	250
	Compact to very dense sand	Below 260	20 - 25	-
North Abutment	Existing granular fill – loose to compact sand and gravel	281 to 285	10 - 15	-
	Stiff to hard clayey silt to clayey silt glacial till	261 to 281	-	250
	Compact to very dense sand	Below 261	20 - 25	-

Both the structural and geotechnical resistances of the piles should be evaluated to establish the governing case at ULS. At SLS, the horizontal reaction of the piles will be controlled by deflections and the horizontal resistance of the piles should be calculated based on the coefficient of horizontal subgrade reaction (k_h) of the soil as discussed above. The SLS reaction should be taken as that corresponding to a horizontal deflection of 10 mm at the underside of the abutment wall for units supporting the abutments (Section C6.11.2.2 of the Commentary to CHBDC (2019)).

Group action for lateral loading should also be evaluated by reducing the coefficient of horizontal subgrade reaction either in the direction of loading or perpendicular to the direction of loading by relevant group pile efficiency factors as outlined in Section C6.11.3.4 including Figures C6.22 to C6.24 of the *Commentary to the CHBDC* (2019).

If the conditions are not met for use of horizontal subgrade reaction moduli, it is recommended that the lateral behaviour and resistance of the piles be assessed using non-linear P-y curves. P-y curves have been generated using RSPile for the north and south abutments, and these are included in Appendix E.

6.4.5 Corrugated Steel Pipe (CSP) Liners for Integral Abutments

For the proposed integral abutments at this structure site, based on the composition and stiffness of the site soils, corrugated steel pipe (CSP) liners filled with loose uniform sand will be required around the upper 3 m of the pile to allow appropriate pile flexibility to accommodate thermal effects and other forces at the bridge deck. Given the site grades, it is anticipated that pre-augering will be required to install these CSPs. A Non-Standard Special Provision (NSSP) for the CSPs detailing the sand gradation should be included in the Contract Documents where applicable.

6.5 Shallow Foundations at Centre Pier

6.5.1 Founding Elevation and Geotechnical Resistance

The median pier will be supported on spread footings. For a multi-span structure, the width of this footing is expected to be about 3.5 to 5 m. A factored ultimate geotechnical resistance of 400 kilopascals (kPa) and a factored serviceability geotechnical resistance of 300 kPa may be used for 3.5 m to 5 m wide footings founded on the stiff to very stiff clayey silt to clayey silt till at or below Elevation 277 m. The factored serviceability geotechnical resistance value corresponds to an estimated total settlement of 25 mm.

The factored ultimate and serviceability geotechnical resistances are dependent on the footing width and founding elevation and as such, the geotechnical resistances should be reviewed if the footing width or founding elevation vary from that given above. The factored ultimate geotechnical resistance provided is based on loading applied perpendicular to the surface of the footings. Where the load is not applied perpendicular to the surface of the footings, eccentricity and inclination of the load should be taken into account in accordance with CHBDC.

6.5.2 Resistance to Lateral Forces

Resistance to sliding between the concrete spread/strip footings and the native, undisturbed subsoil should be calculated in accordance with Section 6.10.4 of the CHBDC. Assuming that the founding soils are not loosened/disturbed during excavation and footing construction, an angle of friction between the cast-in-place concrete and the founding soils of 28° and corresponding unfactored coefficient of friction, $\tan \delta$, of 0.53 may be used.

6.5.3 Frost Protection

All footings should be provided with a minimum of 1.2 m of earth cover or thermal equivalent for frost protection purposes in accordance with OPSD 3090.101.

6.5.4 Subgrade Protection

The founding soils for the pier footing are sensitive to disturbance and softening or loosening due to water seepage and/or ponding and construction equipment or foot traffic when damp to wet. Following inspection and approval of the footing subgrade by geotechnical personnel, placement of a concrete working slab (100 mm thick of 10 MPa concrete) is recommended to protect the foundation subgrade level.

6.6 Lateral Earth Pressures for Design

The lateral pressures acting on the bridge abutments and associated wing/retaining walls, will depend on the type and method of placement of the backfill materials, on the nature of the soils behind the backfill, on the freedom of lateral movement of the structure, and on the drainage conditions behind the walls.

Select, free-draining granular fill meeting the specifications of OPSS.PROV 1010 Granular A or Granular B Type III should be used as backfill behind the abutments and walls. Compaction and placement should be carried out in accordance with OPSS.PROV 501 (Compacting). Longitudinal drains and weep holes should be installed to provide positive drainage of the granular backfill. Other aspects of the abutment granular backfill requirements with respect to subdrains and frost taper should be in accordance with OPSD 3101.150 (Walls, Abutment, Backfill, Minimum Granular Requirements) and 3190.100 (Walls, Retaining and Abutment, Wall Drain).

A minimum compaction surcharge equal to 12 kPa should be included in the lateral earth pressures for the structural design of the abutment walls in accordance with CHBDC Section 6.12.3 and Figure 6.6. Compaction

equipment should be used in accordance with OPSS.PROV 501. Other surcharge loads should be accounted for in the design, as required.

For restrained walls, granular fill should be placed in a zone with the width equal to at least 1.2 m behind the back of the wall in accordance with Figure C6.31(a) of the Commentary to the CHBDC (2019). For unrestrained walls, fill should be placed within the wedge-shaped zone defined by a line drawn at flatter than 1 horizontal to 1 vertical (1H:<1V) extending up and back from the rear face of the footing or pile cap in accordance with Figure C6.31(b) of the Commentary to the CHBDC (2019).

- For restrained walls, the pressures are based on the existing/proposed embankment fill materials and the following parameters (unfactored) may be used assuming the use of Select Subgrade Material (SSM):

Soil unit weight: 20 kN/m³

Coefficients of lateral earth pressure:

At rest, K_o 0.50

- For unrestrained walls, the pressures are based on the granular fill as placed and the following parameters (unfactored) may be assumed:

	<u>GRANULAR A</u>	<u>GRANULAR B</u> <u>Type III</u>
Soil unit weight:	22 kN/m ³	21 kN/m ³
Coefficients of lateral earth pressure:		
Active, K_a	0.27	0.31
Passive, K_p	3.7	3.3

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. 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 C6.12.1 and Table C6.12 of the *Commentary to the CHBDC* (2019).

If the wall support does not allow lateral yielding (i.e., restrained structure where the rotational or horizontal movement is not sufficient to mobilize an active earth pressure condition), at-rest earth pressures should be assumed for geotechnical design.

For integral abutments, passive earth pressures may be used in the geotechnical design of the structure. The movements required to fully mobilize passive pressure or resistance are much larger than those required to mobilize active pressure. In practice, movements may not be sufficient to mobilize the full passive resistance. A resistance factor equal to 0.5 should be applied to the calculated total passive resistance in accordance with Table 6.2 of the CHBDC.

It should be noted that the above design parameters assume level backfill and ground surface behind the wall. The lateral earth pressure coefficients should be adjusted if there is sloping ground at the back of the wall.

6.7 Embankments

The replacement underpass will be constructed east of the existing structure, requiring widening of the approach embankments and a grade raise of up to approximately 1 m at the north approach/abutment, and 1.5 m at the south approach/abutment. Settlement resulting from the embankment modifications will be less than approximately 25 mm, within MTO's "Embankment Settlement Criteria for Design" (2010).

Prior to construction of the realigned approach embankments, it is recommended that all topsoil/organic soils and any loose/soft deleterious fill be stripped from within the footprint of the embankments and from the side slopes of the existing embankments where the new embankments will encroach on the existing. From a geotechnical perspective, it is recommended that fill for construction of the new/widened approach embankments consist of granular fill or OPSS.PROV.1010 Select Subgrade Material (SSM). If there is a surplus of earth fill on the construction contract, or if there is a significant local source of earth fill at the time of construction, use of earth fill may be considered for the embankment widening; however, its suitability would be dependent on the quality of the source, including its plasticity and water content, and in this regard the native clayey silt deposit that predominates in this local area may be prone to sloughing/slumping on the embankment side slopes if used as earth fill.

Fill materials should be placed and compacted in accordance with OPSS.PROV 501 (Compacting) and OPSS.PROV 206 (Grading) and should be benched into the existing embankment side slopes in accordance with OPSD 208.010 (Benching of Earth Slopes). Although not anticipated, embankments greater than 8 m should incorporate into the side slopes a minimum 2 m wide bench at mid-height for all fill heights greater than 8 m as suggested in OPSD 202.010 (Slope Flattening) to promote surficial stability and erosion protection on the embankment side slopes.

To reduce surface water erosion on the granular embankment side slopes, topsoil and seeding as per OPSS 802 (Topsoil) and OPSS.PROV 804 (Seed and Cover) should be carried out as soon as possible after construction of the embankments. If this slope protection is not in place before winter, then alternate protection measures, such as covering the slope with straw, or gravel sheeting as per OPSS 511 (Rip Rap, Rock Protection and Granular Sheeting), and OPSS.PROV 1004 (Aggregates – Miscellaneous) will be required to reduce the potential for erosion and to reduce the potential for the requirement of remedial works on the side slopes in the spring prior to topsoil dressing and seeding.

Embankments constructed as noted above will have a factor of safety of greater than 1.5 against a deep-seated rotational failure.

6.8 Construction Considerations

6.8.1 Excavations and Groundwater Control

Excavations for the abutment pile caps and the centre pier footing will penetrate the existing fill and surficial silty sand to silt deposit (where present) and extend into the clayey silt till. The groundwater level in the till is interpreted to be at approximately Elevation 279 m and will fluctuate seasonally and due to climatic variations.

- **Abutments:** The excavations for the abutment pile caps will extend to approximately Elevation 281 m and 281.4 m at the north and south abutments, respectively, with augered holes for the CSPs extending approximately 3 m deeper. Hence, the abutment excavations are expected to remain above the groundwater level associated with the clayey silt till deposit, with the CSP auger holes extending below the water table. However, groundwater seepage from the clayey silt till into the auger holes for the CSPs is expected to be

minor. Localized “perched” groundwater may be encountered at the base of the granular fill or surficial silty sand to sand deposit, perched above the relatively low-permeability clayey silt till deposit.

- **Centre Pier:** The pier footing will be founded at approximately Elevation 276.0 m and is expected to extend below the groundwater level associated with the clayey silt till deposit. However, only minor seepage is anticipated from this cohesive soil or from groundwater perched above the clayey silt at the base of the granular fills.

If necessary to address groundwater seepage from perched groundwater at the base of the fill or surficial silty sand to silt deposit, groundwater control may be achieved by pumping from properly filtered sumps. Sumps should be maintained outside of the abutment pile cap or pier footing limits. Surface water runoff should be directed away from the excavations at all times.

All excavations should be carried out in accordance with the guidelines outlined in the latest edition of the Ontario Occupational Health and Safety Act and Regulations for Construction Projects. The fill materials at this site would be classified as Type 3 soils as would any cohesionless soils below the groundwater level. The native clayey materials and properly dewatered cohesionless soils would be classified as Type 2. Excavations and backfilling should be carried out in accordance with OPSS 902.

6.8.2 Temporary Protection Systems

Temporary protection systems will be required where space is restricted and will not permit open cuts, to support the sides of the excavation and permit the use of vertical cuts. The design and limits of the systems are to be determined by the contractor. Temporary protection systems are to be designed to Performance Level 2 in accordance with OPSS.PROV 539.

Temporary support systems could consist of soldier piles and lagging where the H-piles would be driven to a suitable depth and horizontal lagging installed as the excavation proceeds, or of driven steel sheet piling. Support to the systems could be in the form of struts and walers or rakers and anchors. The raker/anchor support must be designed to accommodate the loads applied from pressures and surcharge pressures from area, line, or point loads as well as the impact of sloping ground behind the system.

6.8.3 Obstructions

Cobbles and boulders should be expected in the glacial till and granular soils which could impact temporary protection system installation and pile driving operations. A non-standard special provision (NSSP) should be added to the Contract Documents to identify the presence of cobbles, boulders and other obstructions during pile installation. Based on the configuration of the new underpass structure, no interference will occur with the existing abutment foundations including the mass concrete at the existing north abutment.

6.8.4 Subgrade Protection at Centre Pier

As noted in Section 6.5.4, the founding soils for the pier footing are sensitive to disturbance and softening or loosening due to water seepage and/or ponding and construction equipment or foot traffic when damp to wet. Following inspection and approval of the footing subgrade by geotechnical personnel, placement of a concrete working slab (100 mm thick of 10 MPa concrete) is recommended to protect the foundation subgrade level.

6.8.5 Median Sewer Removal and New Construction Near Centre Pier

It is understood that the existing median sewer will be removed and reconstructed in the vicinity of the median pier footing. The existing sewer invert is at approximately Elevation 275.5 m to 275.0 m, declining from west to east,

about 0.5 m to 1 m below the proposed centre pier founding level at Elevation 276 m. It is recommended that the subexcavation for the existing sewer removal be backfilled with OPSS.PROV 1010 Granular A or B Type II, placed and compacted in accordance with OPSS.PROV 501.

It is further recommended that the new 450 mm diameter storm sewer pipe be installed such that the invert is located above the zone of influence of the centre pier footing (i.e., above a line project downward and outward at 1 horizontal to 1 vertical from the bottom edge of the footing). It is understood that the new sewer is planned to be offset 300 mm from the footing per OPSD 802.030 (Rigid Pipe Bedding, Cover and Backfill), with the invert elevation at the middle of the pier (at Station 13+769) at Elevation 276.35 m. Therefore, the new sewer will be located above the loading zone of influence for the new median pier foundation.

7.0 CLOSURE

This Foundation Design Report was prepared by Michael Beadle, P. Eng., a Senior Principal with WSP Golder. An independent review of this report was completed by Lisa Coyne, P. Eng., a Fellow and MTO Designated Principal Foundations Contact with WSP Golder.

WSP Golder



Michael E. Beadle, P.Eng.
Senior Principal



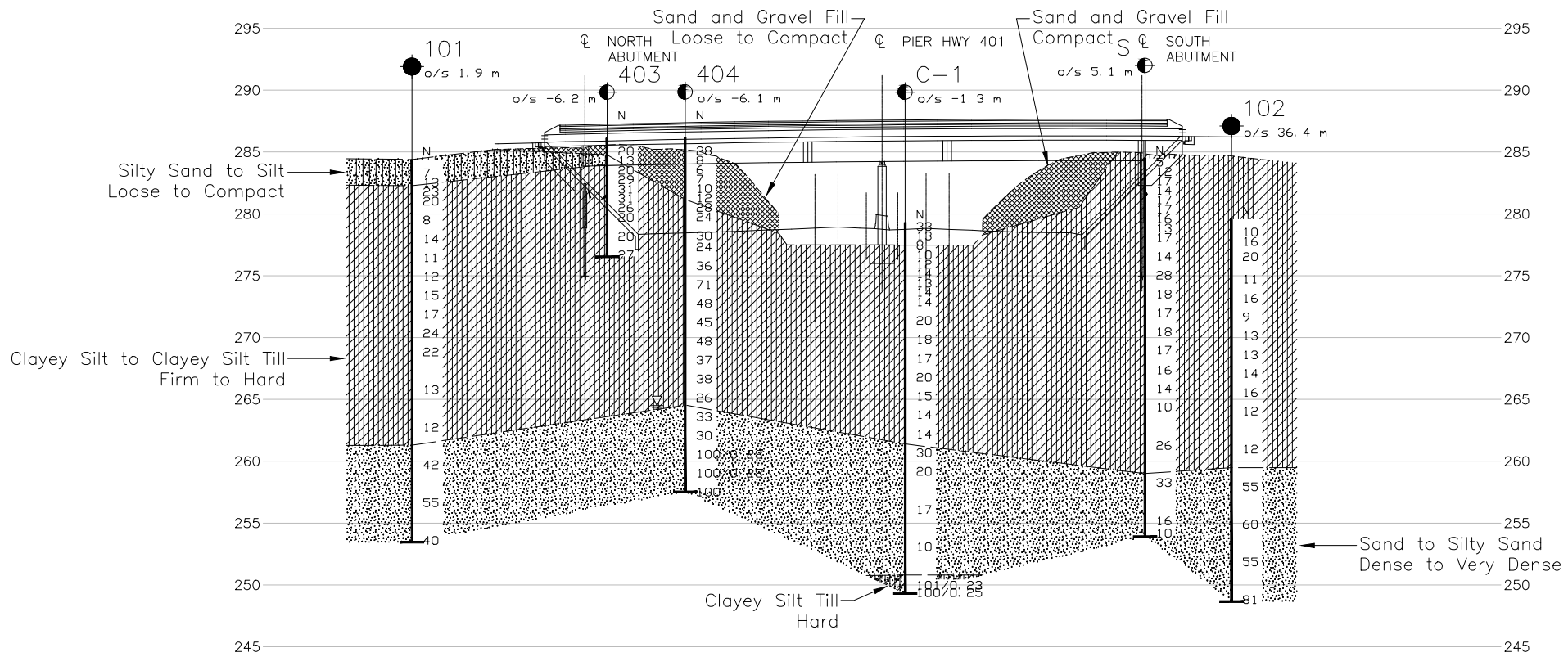
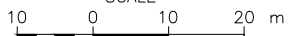
Lisa Coyne, P.Eng.
Fellow, MTO Designated Foundations Contact

MEB/LCC/cr

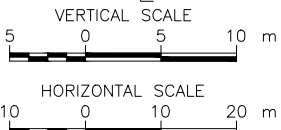
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PLAN SCALE



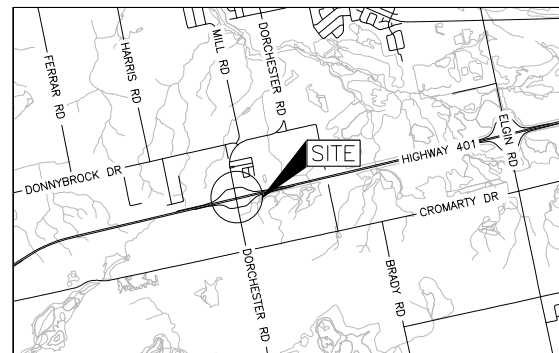
PROFILE A-A' ALONG CL DORCHESTER ROAD



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

CONT No. 2022-3009
WP No. 3053-11-03

HIGHWAY 401
DORCHESTER ROAD UNDERPASS REPLACEMENT
BOREHOLES LOCATIONS AND SOIL
STRATA



KEY PLAN SCALE



LEGEND

- Borehole - Current Investigation
- Borehole - Previous Investigation
- Seal
- Piezometer
- N Standard Penetration Test Value
- WL in piezometer, measured on MMM DD, YYYY
- WL upon completion of drilling

BOREHOLE CO-ORDINATES NAD83-ZONE 11

No.	ELEVATION	NORTHING	EASTING
C-1	279.3	4758632.7	422231.3
C-2	279.3	4758626.0	422200.1
N	286.2	4758664.6	422208.9
S	284.5	4758596.0	422245.3
101	284.4	4758711.4	422218.6
102	279.6	4758588.5	422278.7
401	285.5	4758600.3	422220.7
402	285.3	4758589.5	422223.2
403	286.1	4758678.9	422216.9
404	286.2	4758666.6	422219.5

NOTES

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

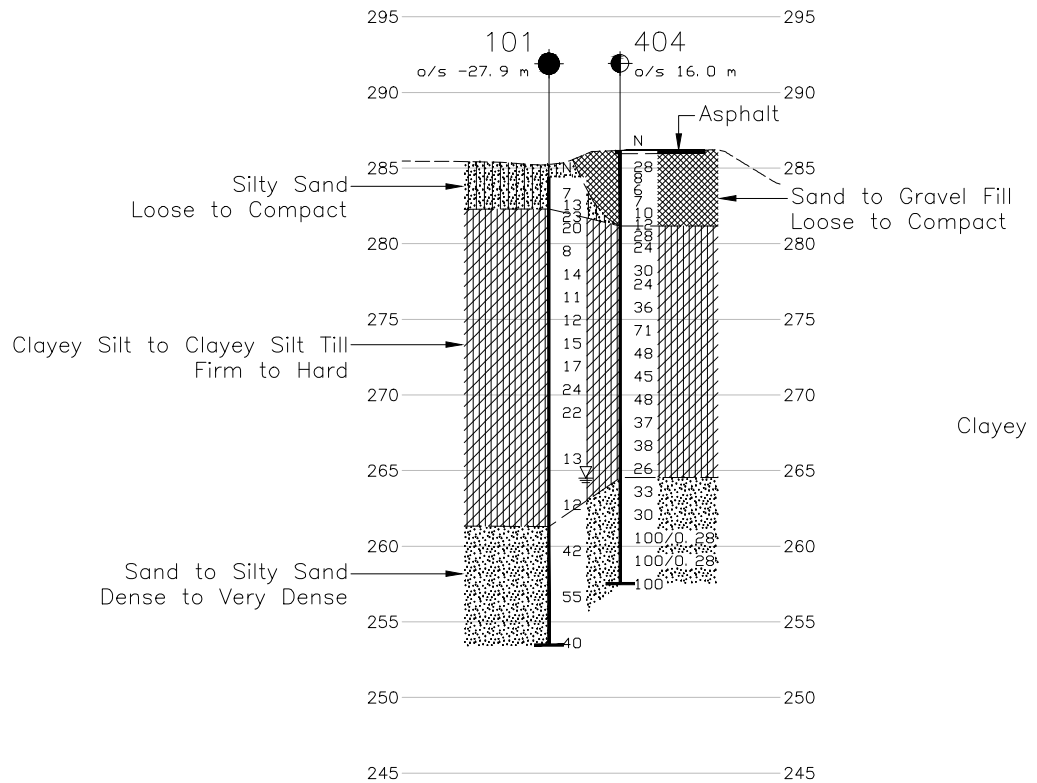
The boundaries between soil strata have been established only at borehole locations. Between boreholes the boundaries are assumed from geological evidence.

REFERENCE

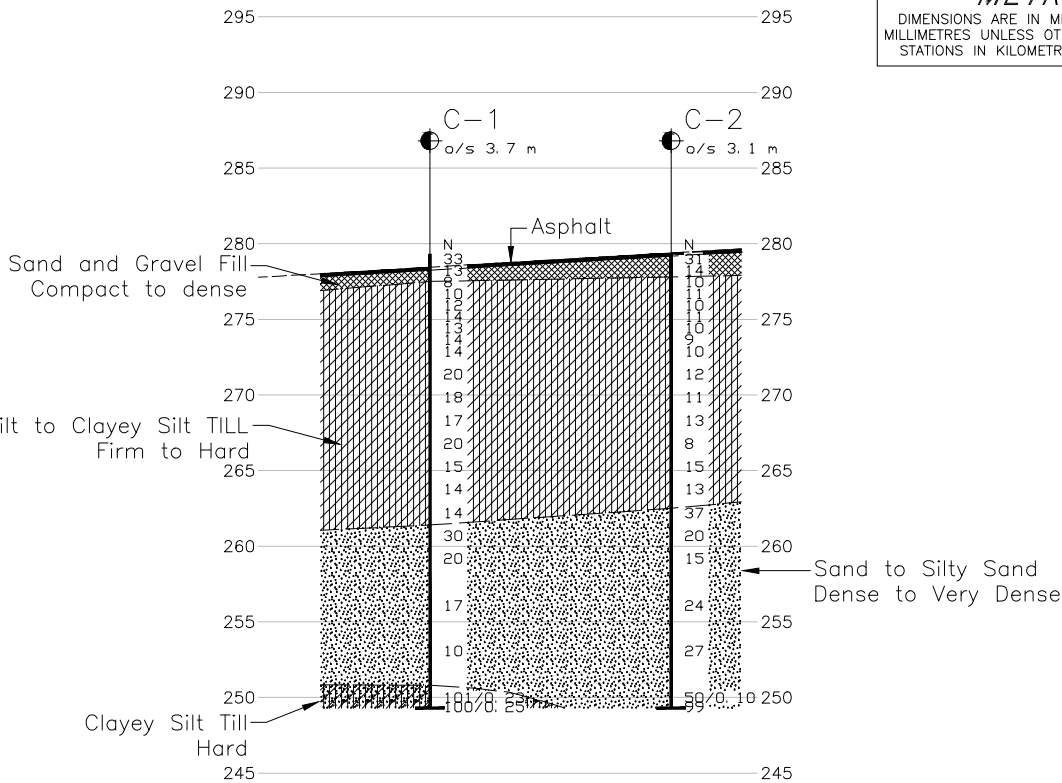
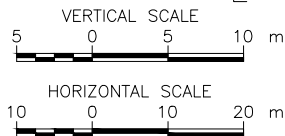
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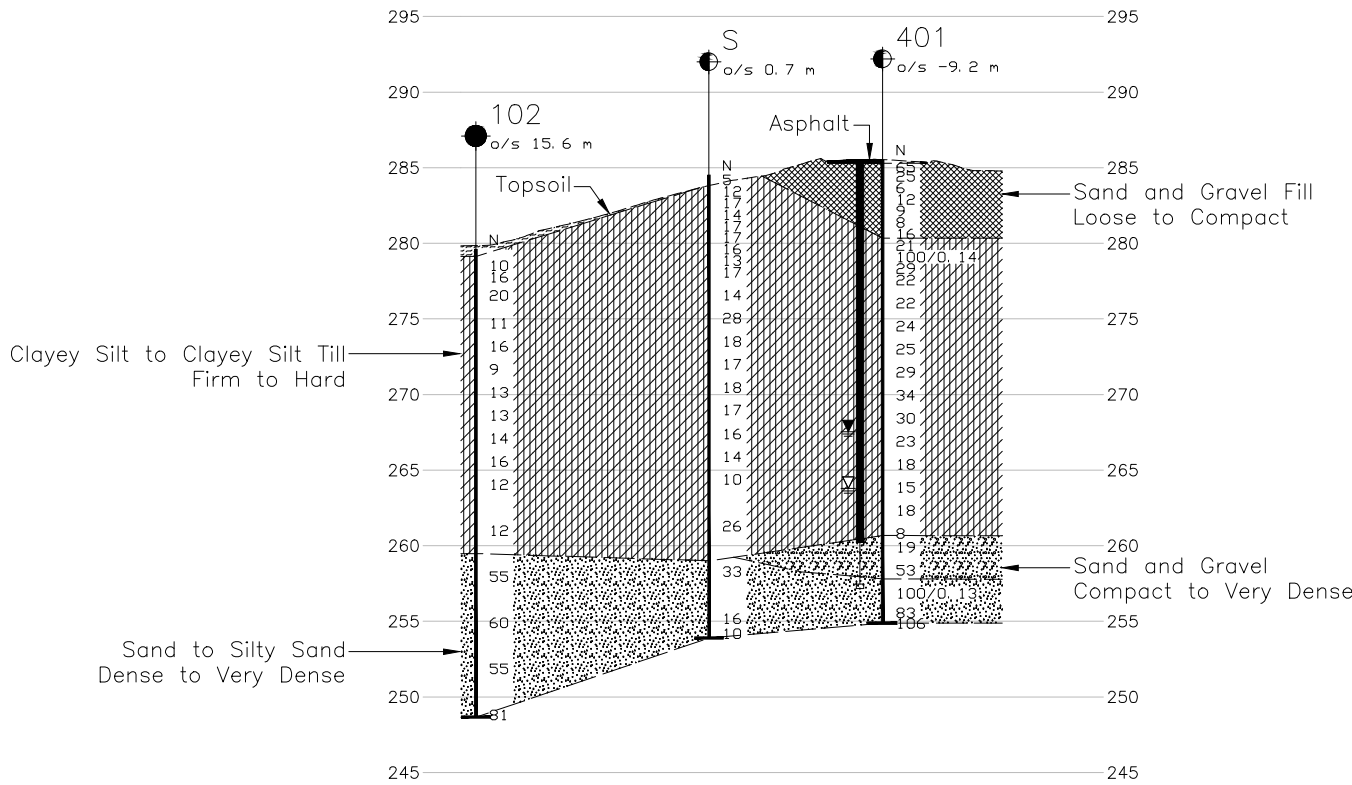
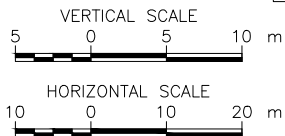
NO.	DATE	BY	REVISION
Geocres No. 40114-212			
HWY. 401	PROJECT NO. 22520660		DIST. .
SUBM'D. LCC	CHKD. LCC	DATE: 03/27/2023	SITE: .
DRAWN: SA/DD	CHKD. MEB	APPD. LCC	DWG. 1



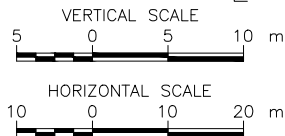
CROSS-SECTION B-B' ALONG ϕ NORTH ABUTMENT



CROSS-SECTION C-C' ALONG ϕ CENTRE PIER



CROSS-SECTION B-B' ALONG ϕ SOUTH ABUTMENT



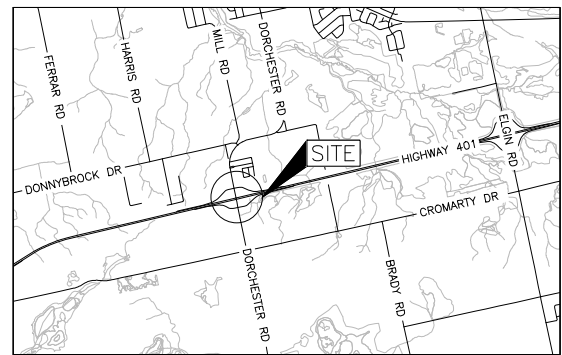
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DIMENSIONS ARE IN METRES AND/OR
MILLIMETRES UNLESS OTHERWISE SHOWN.
STATIONS IN KILOMETRES + METRES.

CONT No. 2022-3009
WP No. 3053-11-03

HIGHWAY 401
DORCHESTER ROAD UNDERPASS REPLACEMENT

SHEET

SOIL STRATA



LEGEND

- Borehole - Current Investigation
- Borehole - Previous Investigation
- Seal
- Piezometer
- N Standard Penetration Test Value
- WL in piezometer, measured on MMM DD, YYYY
- WL upon completion of drilling

BOREHOLE CO-ORDINATES NAD83-ZONE 11

No.	ELEVATION	NORTHING	EASTING
S	284.5	4758596.0	422245.3
C-1	279.3	4758632.7	422231.3
C-2	279.3	4758626.0	422200.1
101	284.4	4758711.4	422218.6
102	279.6	4758588.5	422278.7
401	285.5	4758600.3	422220.7
404	286.2	4758666.6	422219.5

NOTES

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

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

REFERENCE

Ground surface provided in digital format by Dillon, drawing file no. "2022-3009 - Alignments.dwg", received January 25, 2023.



NO.	DATE	BY	REVISION
Geocres No. 40114-212			
HWY. 401	PROJECT NO. 22520660		DIST. .
SUBM'D. LCC	CHKD. LCC	DATE: 03/27/2023	SITE: .
DRAWN: SA/DD	CHKD. MEB	APPD. LCC	DWG. 2

APPENDIX A

**2013 Investigation
- GEOCRES No. 40I14-155**

RECORD OF BOREHOLE No 401

1 OF 3

METRIC

PROJECT 12-1132-0076
W.P. 3053-11-00 LOCATION N 4758600.3 , E 422220.7 ORIGINATED BY BT
DIST HWY 401 BOREHOLE TYPE POWER AUGER, HOLLOW STEM / WASH BORING, CASED COMPILED BY WF/LK/AG
DATUM GEODETIC DATE March 5, 2013 - March 6, 2013 CHECKED BY SS

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100	W _P W W _L	WATER CONTENT (%)	GR SA SI CL		
285.51	PAVEMENT SURFACE													
0.00	ASPHALT													
0.21	FILL, granular base		1	SS	65		285							
284.90														
0.61	FILL, sand and gravel, trace to some silt, with cobbles Loose to compact Brown		2	SS	25		284							
			3	SS	6		283							
			4	SS	12		282							
			5	SS	9		281							
			6	SS	8		280							
			7	SS	16		279							
280.33														
5.18	CLAYEY SILT TILL, trace to some sand, trace gravel, with cobbles Very stiff to hard Brown becoming grey at about elev. 279.0m		8	SS	21		278							
			9	SS	100/ 140mm		277							
	Cobble/boulder at about elev. 279.3m													
			10	SS	29		276							
			11	SS	22		275							
			12	SS	22		274							
			13	SS	24		273							
			14	SS	25		272							
			15	SS	29		271							

Continued Next Page

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

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

RECORD OF BOREHOLE No 402

1 OF 1

METRIC

PROJECT 12-1132-0076
W.P. 3053-11-00 LOCATION N 4758589.5 , E 422223.2 ORIGINATED BY BT
DIST HWY 401 BOREHOLE TYPE POWER AUGER, HOLLOW STEM COMPILED BY WF/LK/AG
DATUM GEODETIC DATE March 7, 2013 CHECKED BY SSB

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _P	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa					
285.34 0.00	PAVEMENT SURFACE ASPHALT							20 40 60 80 100					GR SA SI CL
0.27	FILL, granular base Brown												
284.58 0.76	FILL, sand and gravel, trace silt, with cobbles Brown		1	SS	10								
	SILT, trace to some sand, trace gravel, with clayey silt layers Compact to very dense Brown		2	SS	19								
			3	SS	26								1 6 76 17
			4	SS	57								
281.68 3.66	CLAYEY SILT TILL, trace to some sand, trace gravel Stiff to very stiff Grey		5	SS	28								
			6	SS	23								2 12 51 35
			7	SS	14								
			8	SS	20								1 9 50 40
			9	SS	25								
275.74 9.60	END OF BOREHOLE		10	SS	23								
	Groundwater not established during drilling on March 7, 2013.												
	Standpipe dry on March 8, 2013.												
	Installation missing/destroyed on April 3, 2013.												

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

RECORD OF BOREHOLE No 403

1 OF 1

METRIC

PROJECT 12-1132-0076
W.P. 3053-11-00 LOCATION N 4758678.9 , E 422216.9 ORIGINATED BY BT
DIST HWY 401 BOREHOLE TYPE POWER AUGER, HOLLOW STEM COMPILED BY WF/LK/AG
DATUM GEODETIC DATE March 7, 2013 CHECKED BY SSB

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa										WATER CONTENT (%)		
								○ UNCONFINED ● QUICK TRIAXIAL	+ FIELD VANE × LAB VANE											
286.14	PAVEMENT SURFACE																			
0.00	ASPHALT																			
0.24	FILL, granular base																			
0.43	Brown																			
	FILL, sand and gravel, some silt, with cobbles		1	SS	20															
284.77	Compact Brown																			
1.37	SILT, trace clay, trace sand		2	SS	13															
	Compact Brown																			
284.01																				
2.13	CLAYEY SILT, trace sand		3	SS	20															
	Very stiff Brown																			
283.24																				
2.90	CLAYEY SILT TILL, trace to some sand, trace gravel		4	SS	29															
	Very stiff to hard																			
	Brown becoming grey at about elev. 281.0m																			
			5	SS	31															
			6	SS	31															
			7	SS	26															
			8	SS	20															
			9	SS	20															

RECORD OF BOREHOLE No 404

1 OF 2

METRIC

PROJECT 12-1132-0076
W.P. 3053-11-00 LOCATION N 4758666.6 , E 422219.5 ORIGINATED BY DH
DIST HWY 401 BOREHOLE TYPE POWER AUGER, HOLLOW STEM / WASH BORING, CASED COMPILED BY WF/LK/AG
DATUM GEODETIC DATE March 12, 2013 - March 13, 2013 CHECKED BY SSB

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)					
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa		W _P	W	W _L		GR	SA	SI	CL		
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE	WATER CONTENT (%)										
286.17	PAVEMENT SURFACE							20	40	60	80	100							
0.00	ASPHALT																		
0.21	FILL, granular base						286												
0.37	FILL, sand and gravel, trace to some silt, with cobbles Loose to compact Brown		1	SS	28		285												
			2	SS	8		284												
			3	SS	6		283												
			4	SS	7		282												
			5	SS	10		281												
			6	SS	12		280												
281.17	TOPSOIL, clayey Black		7	SS	28		279												
5.00	CLAYEY SILT TILL, trace to some sand, trace gravel Very stiff to hard Brown becoming grey at about elev. 279.2m		8	SS	24		278												
5.18			9	SS	30		277												
			10	SS	24		276												
			11	SS	36		275												
			12	SS	71		274												
			13	SS	48		273												
			14	SS	45		272												

Continued Next Page

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

PROJECT 12-1132-0076

W.P. 3053-11-00

LOCATION N 4758666.6 , E 422219.5

ORIGINATED BY DH

DIST _____ HWY 401

BOREHOLE TYPE POWER AUGER, HOLLOW STEM / WASH BORING, CASED

COMPILED BY WF/LK/AG

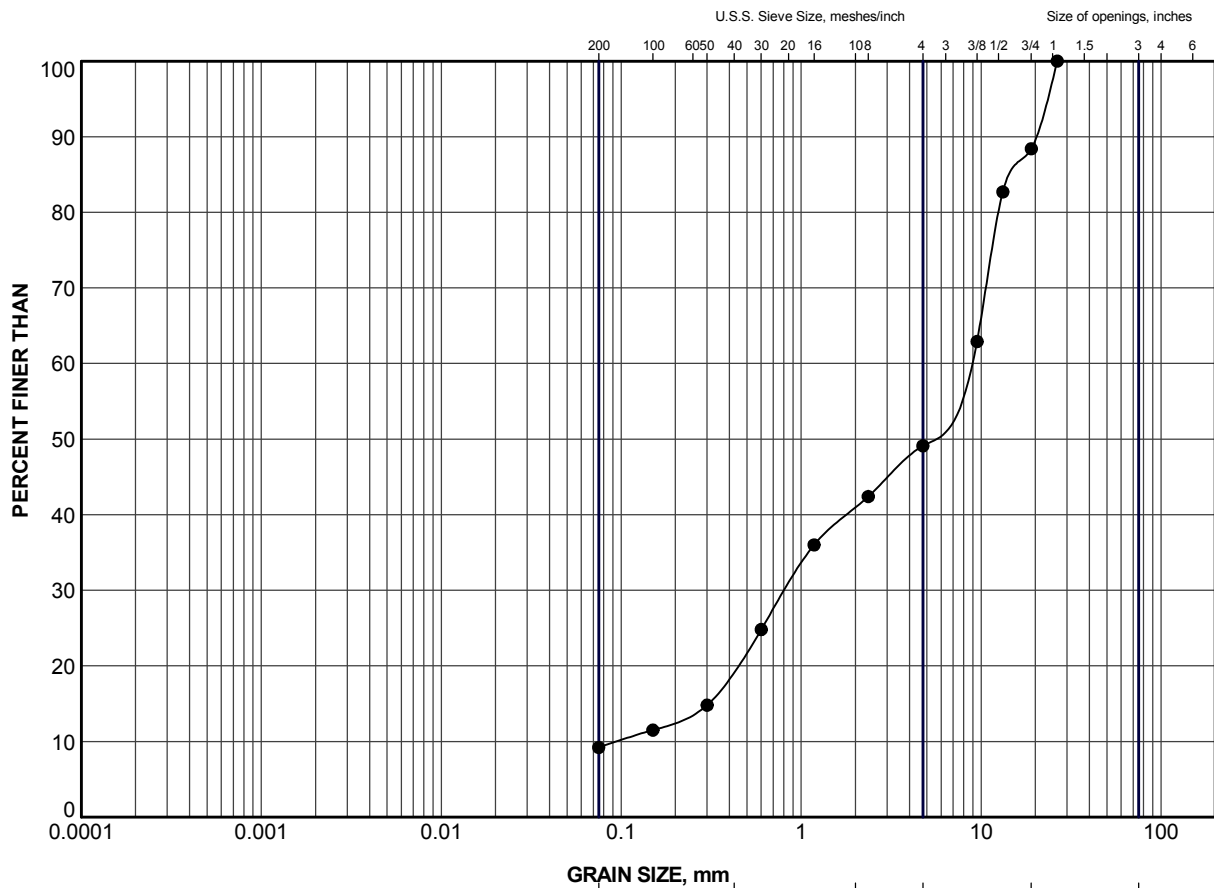
DATUM GEODETIC

DATE March 12, 2013 - March 13, 2013

CHECKED BY SSB

[illegible]


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

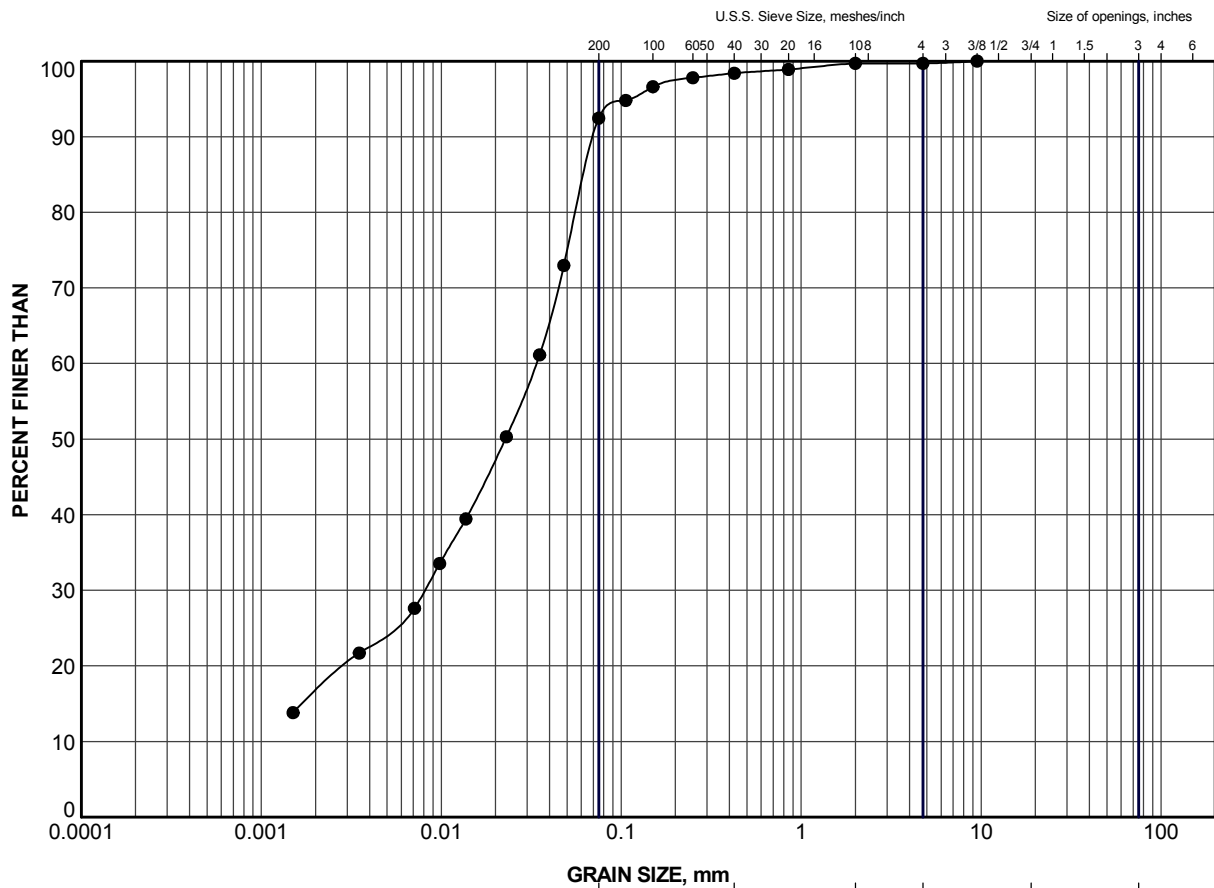


CLAY AND SILT	SAND SIZE, mm					Cobble Size
	fine	medium	coarse	fine	coarse	
	SAND SIZE			GRAVEL SIZE		

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	401	4	283.0


PROJECT	MIDDLESEX ROAD 32 UNDERPASS, SITE 19-303 HIGHWAY 401 INTERCHANGE IMPROVEMENTS GWP 3053-11-00		
TITLE	GRAIN SIZE DISTRIBUTION FILL		
 Golder Associates LONDON, ONTARIO	PROJECT No.12-1132-0076-2001	FILE No. 1211320076-2001-F020A1	
	DRAWN WDF	Apr 04/13	SCALE N/A REV.
	CHECK SSIS		
FIGURE A-1			



GRAIN SIZE, mm						
CLAY AND SILT	fine	medium	coarse	fine	coarse	Cobble Size
	SAND SIZE			GRAVEL SIZE		

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	402	3	282.8

PROJECT				MIDDLESEX ROAD 32 UNDERPASS, SITE 19-303 HIGHWAY 401 INTERCHANGE IMPROVEMENTS GWP 3053-11-00			
TITLE				GRAIN SIZE DISTRIBUTION SILT			
PROJECT No.12-1132-0076-2001		FILE No. 1211320076-2001-F020A2		SCALE		N/A	
DRAWN		WDF		Apr 04/13		REV.	
CHECK		SSS				FIGURE A-2	
 Golder Associates LONDON, ONTARIO							

CLAY AND SILT

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
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●

▲

401403404PROJECT

MIDDLESEX ROAD 32 UNDERPASS, SITE 19-303
HIGHWAY 401 INTERCHANGE IMPROVEMENTS
GWP 3053-11-00

TITLE	
-------	--

GRAIN SIZE DISTRIBUTION

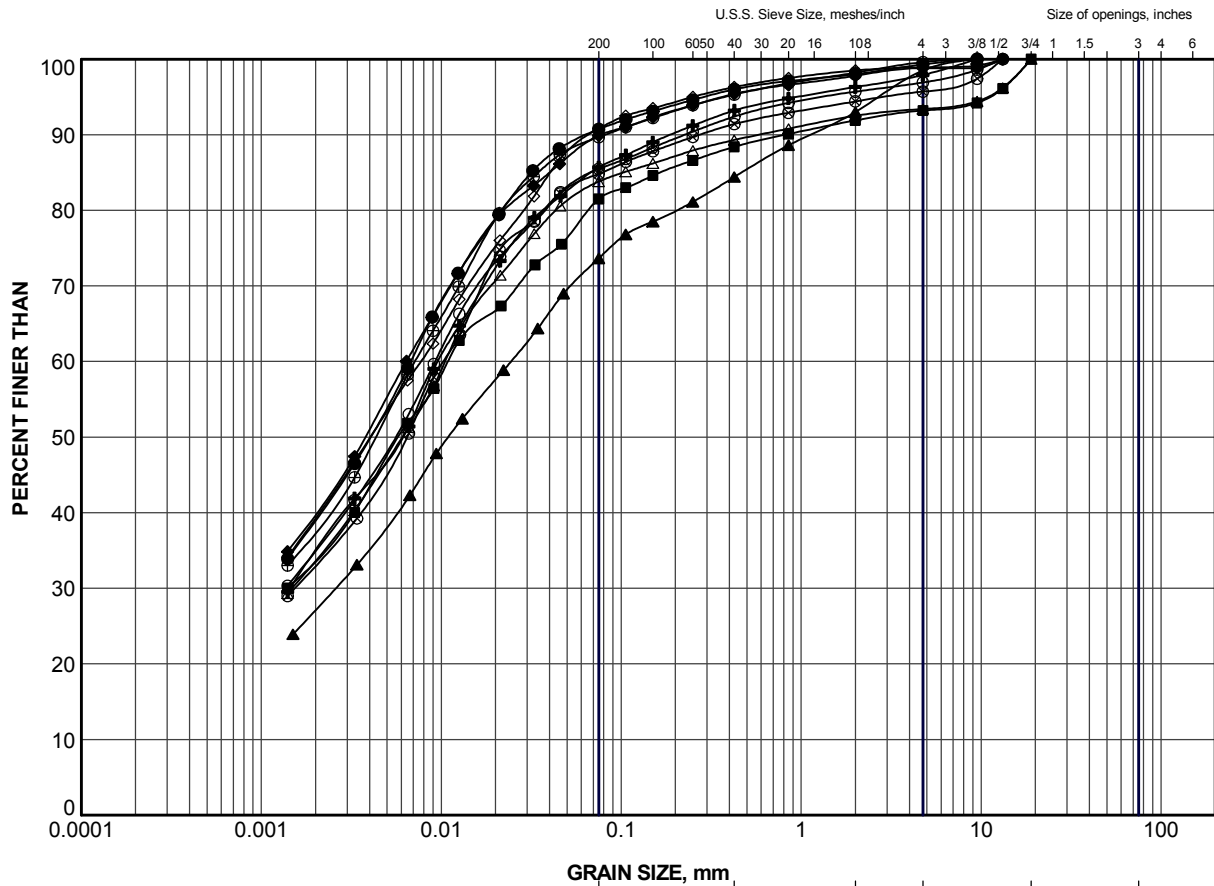
CLAYEY SILT



Golder Associates
LONDON, ONTARIO

PROJECT No.12-1132-0076-2001			FILE No. 1211320076-2001-F020A3		
			SCALE N/A		REV.
DRAWN	WDF	Apr 04/13	FIGURE A-3		
CHECK	SSB				

FIGURE A-3

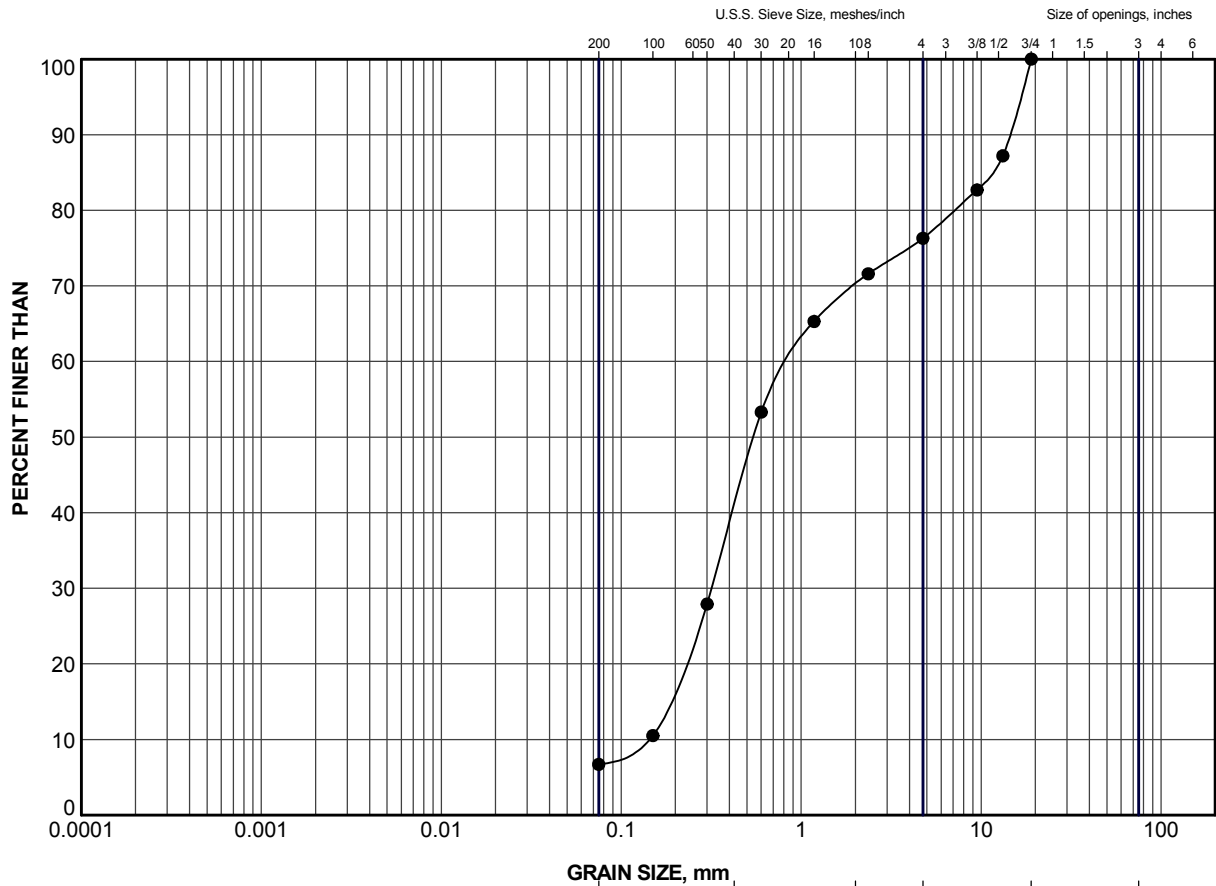


CLAY AND SILT	SAND SIZE, mm					Cobble Size
	fine	medium	coarse	fine	coarse	
	SAND SIZE			GRAVEL SIZE		

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	401	8	279.9
■	401	14	273.1
▲	401	17	268.5
+	402	6	280.5
◆	402	8	279.0
◇	403	7	280.5
○	403	9	278.3
△	404	8	279.9
⊗	404	11	275.9
⊕	404	14	271.3

PROJECT			
MIDDLESEX ROAD 32 UNDERPASS, SITE 19-303 HIGHWAY 401 INTERCHANGE IMPROVEMENTS GWP 3053-11-00			
TITLE			
GRAIN SIZE DISTRIBUTION CLAYEY SILT TILL			
PROJECT No.12-1132-0076-2001		FILE No. 1211320076-2001-F020A4	
DRAWN	WDF	Apr 04/13	SCALE N/A REV.
CHECK	SSS		
Golder Associates LONDON, ONTARIO			FIGURE A-4

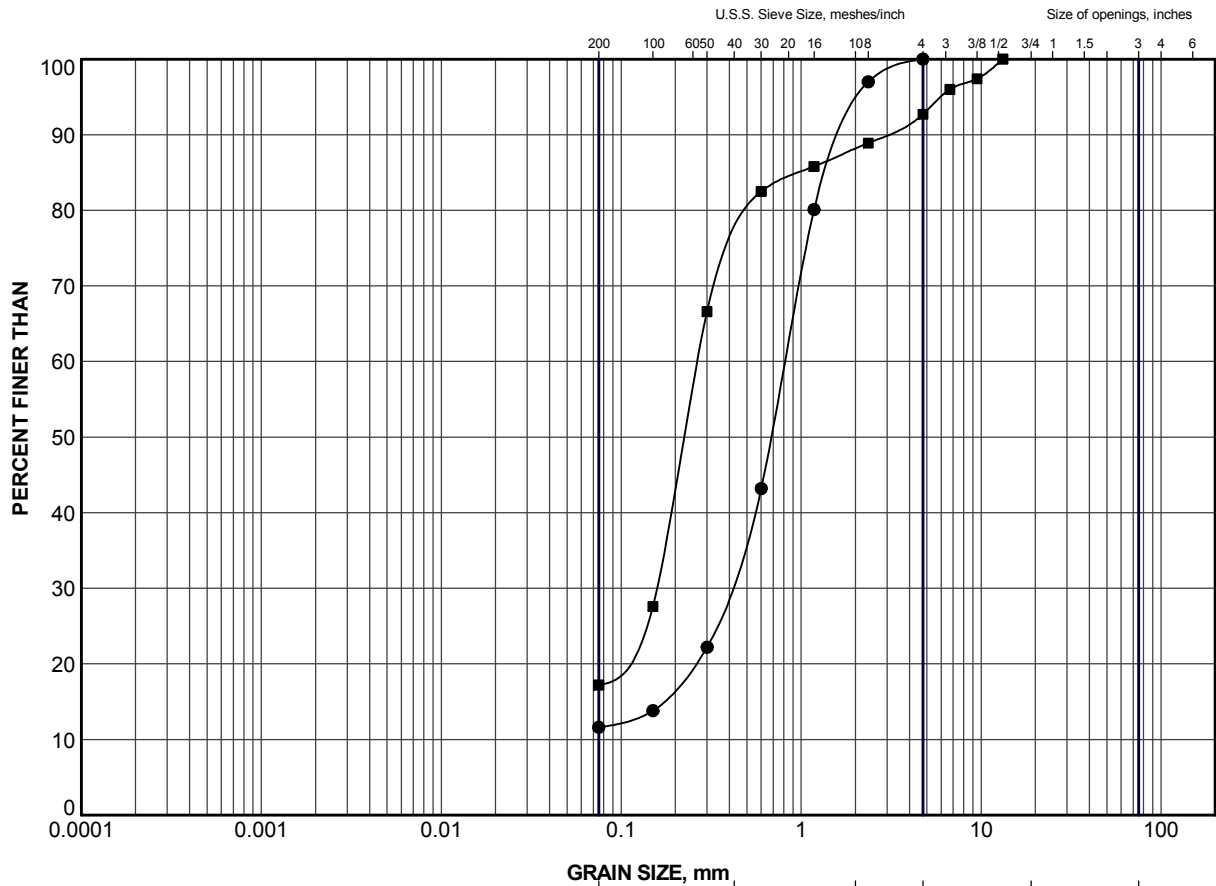


CLAY AND SILT	fine	medium	coarse	fine	coarse	Cobble Size
	SAND SIZE			GRAVEL SIZE		

LEGEND			
SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	401	23	260.0

PROJECT				MIDDLESEX ROAD 32 UNDERPASS, SITE 19-303 HIGHWAY 401 INTERCHANGE IMPROVEMENTS GWP 3053-11-00			
TITLE				GRAIN SIZE DISTRIBUTION SAND AND GRAVEL			
		PROJECT No.12-1132-0076-2001		FILE No. 1211320076-2001-F020A5			
		DRAWN	WDF	Apr 04/13	SCALE	N/A	REV.
		CHECK	SSS		FIGURE A-5		

LDN_MTO_GSD_GLDR_LDN.GDT



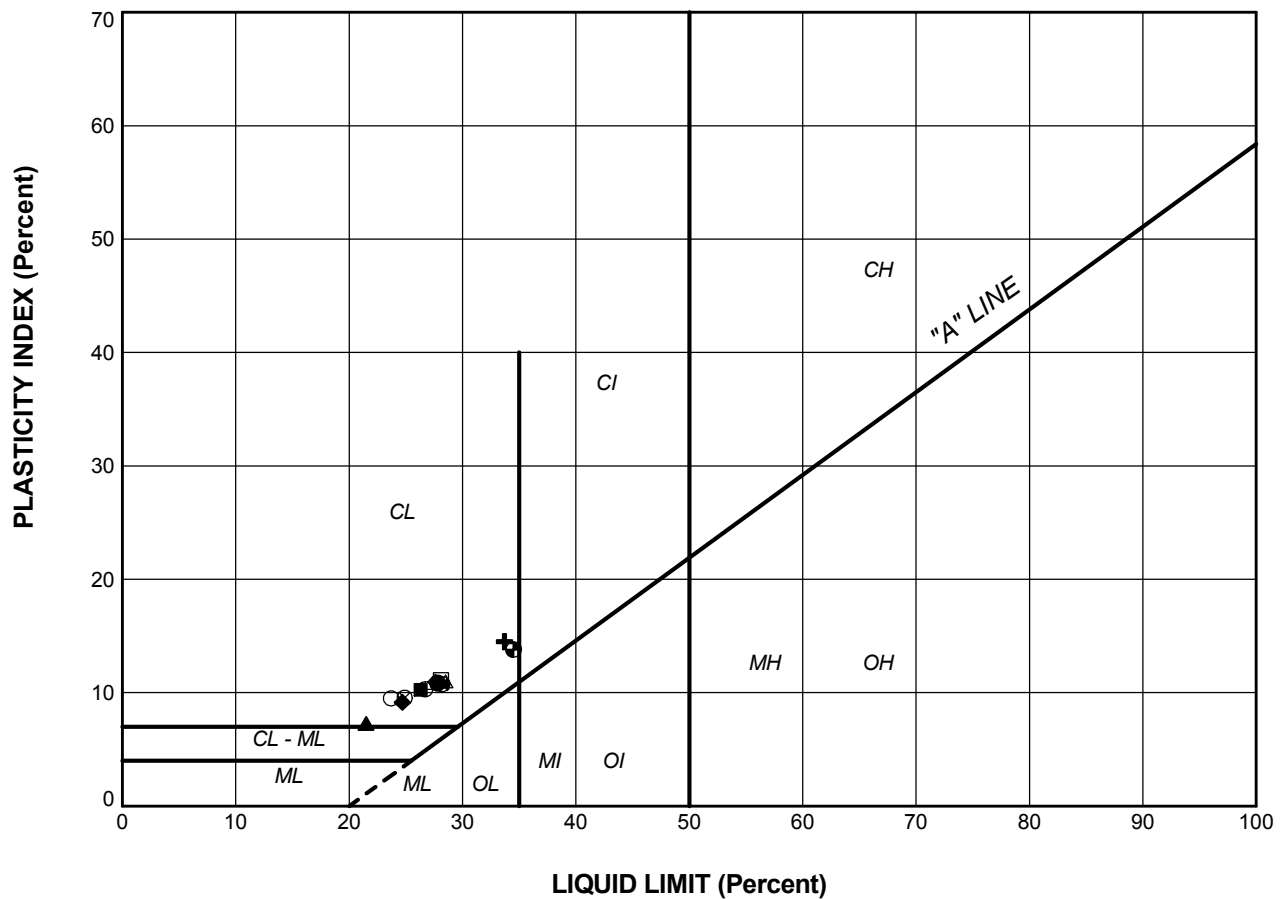
CLAY AND SILT	SAND SIZE, mm					Cobble Size
	fine	medium	coarse	fine	coarse	
	SAND SIZE			GRAVEL SIZE		

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	401	25	257.1
■	404	21	260.7


PROJECT				MIDDLESEX ROAD 32 UNDERPASS, SITE 19-303 HIGHWAY 401 INTERCHANGE IMPROVEMENTS GWP 3053-11-00			
TITLE				GRAIN SIZE DISTRIBUTION SAND			
PROJECT No.12-1132-0076-2001				FILE No. 1211320076-2001-F020A6			
DRAWN		LMK		Jun 14/13		SCALE N/A REV.	
CHECK		SSS				FIGURE A-6	





LEGEND

SYMBOL	BOREHOLE	SAMPLE	LL(%)	PL(%)	PI
CLAYEY SILT TILL					
●	401	8	27.8	17.0	10.9
■	401	14	26.3	16.1	10.3
▲	401	17	21.5	14.2	7.3
◆	402	6	24.7	15.6	9.2
◇	402	8	27.6	16.7	11.0
△	403	7	28.5	17.5	11.0
⊗	403	9	24.9	15.4	9.6
⊕	404	8	26.7	16.4	10.3
□	404	11	28.1	17.0	11.1
⊙	404	14	28.2	17.5	10.8
CLAYEY SILT					
+	401	21	33.7	19.2	14.5
○	403	3	23.7	14.2	9.5
⊙	404	18	34.5	20.7	13.8

PROJECT				MIDDLESEX ROAD 32 UNDERPASS, SITE 19-303 HIGHWAY 401 INTERCHANGE IMPROVEMENTS GWP 3053-11-00			
TITLE				PLASTICITY CHART			
PROJECT No.12-1132-0076-2001				FILE No. 1211320076-2001-F020A7			
DRAWN		WDF/AMG		AUG. 14/13		SCALE N/A	
CHECK		SSS				REV.	
 Golder Associates LONDON, ONTARIO				FIGURE A-7			

APPENDIX B

**2019 Investigation
- GEOCRES No. 40I14-193**

EXPLANATION OF TERMS USED IN REPORT

N VALUE: THE STANDARD PENETRATION TEST (SPT) N VALUE IS THE NUMBER OF BLOWS REQUIRED TO CAUSE A STANDARD 51mm O.D. SPLIT BARREL SAMPLER TO PENETRATE 0.3m INTO UNDISTURBED GROUND IN A BOREHOLE WHEN DRIVEN BY A HAMMER WITH A MASS OF 63.5kg, FALLING FREELY A DISTANCE OF 0.76m. FOR PENETRATIONS OF LESS THAN 0.3m N VALUES ARE INDICATED AS THE NUMBER OF BLOWS FOR THE PENETRATION ACHIEVED. AVERAGE N VALUE IS DENOTED THUS \bar{N} .

DYNAMIC CONE PENETRATION TEST: CONTINUOUS PENETRATION OF A CONICAL STEEL POINT (51mm O.D. 60° CONE ANGLE) DRIVEN BY 475 J IMPACT ENERGY ON 'A' SIZE DRILL RODS. THE RESISTANCE TO CONE PENETRATION IS MEASURED AS THE NUMBER OF BLOWS FOR EACH 0.3m ADVANCE OF THE CONICAL POINT INTO THE UNDISTURBED GROUND.

SOILS ARE DESCRIBED BY THEIR COMPOSITION AND CONSISTENCY OR DENSENESS.

COMPOSITION: SECONDARY SOIL COMPONENTS ARE DESCRIBED ON THE BASIS OF PERCENTAGE BY MASS OF THE WHOLE SAMPLE AS FOLLOWS:

PERCENT BY MASS	0 - 10	10 - 20	20 - 30	30 - 40	> 40
	TRACE	SOME	WITH	ADJECTIVE (SILTY)	AND (AND SILT)

CONSISTENCY: COHESIVE SOILS ARE DESCRIBED ON THE BASIS OF THEIR UNDRAINED SHEAR STRENGTH (c_u) AS FOLLOWS:

c_u (kPa)	0 - 12	12 - 25	25 - 50	50 - 100	100 - 200	> 200
	VERY SOFT	SOFT	FIRM	STIFF	VERY STIFF	HARD

DENSENESS: COHESIONLESS SOILS ARE DESCRIBED ON THE BASIS OF DENSENESS AS INDICATED BY SPT N VALUES AS FOLLOWS:

N (BLOWS/0.3m)	0 - 5	5 - 10	10 - 30	30 - 50	> 50
	VERY LOOSE	LOOSE	COMPACT	DENSE	VERY DENSE

ROCKS ARE DESCRIBED BY THEIR COMPOSITION AND STRUCTURAL FEATURES AND / OR STRENGTH.

RECOVERY: SUM OF ALL RECOVERED ROCK CORE PIECES FROM A CORING RUN EXPRESSED AS A PERCENT OF THE TOTAL LENGTH OF THE CORING RUN.

MODIFIED RECOVERY: SUM OF THOSE INTACT CORE PIECES, 100mm* IN LENGTH EXPRESSED AS A PERCENT OF THE LENGTH OF THE CORING RUN. THE ROCK QUALITY DESIGNATION (R Q D), FOR MODIFIED RECOVERY, IS:

R Q D (%)	0 - 25	25 - 50	50 - 75	75 - 90	90 - 100
	VERY POOR	POOR	FAIR	GOOD	EXCELLENT

JOINTING AND BEDDING:

SPACING	50mm	50 - 300mm	0.3m - 1m	1m - 3m	> 3m
JOINTING	VERY CLOSE	CLOSE	MOD. CLOSE	WIDE	VERY WIDE
BEDDING	VERY THIN	THIN	MEDIUM	THICK	VERY THICK

ABBREVIATIONS AND SYMBOLS

FIELD SAMPLING

S S SPLIT SPOON	T P THINWALL PISTON
W S WASH SAMPLE	O S OSTERBERG SAMPLE
S T SLOTTED TUBE SAMPLE	R C ROCK CORE
B S BLOCK SAMPLE	P H T W ADVANCED HYDRAULICALLY
C S CHUNK SAMPLE	P M T W ADVANCED MANUALLY
T W THINWALL OPEN	F S FOIL SAMPLE
F V FIELD VANE	

STRESS AND STRAIN

u_w	kPa	PORE WATER PRESSURE
u	1	PORE PRESSURE RATIO
σ	kPa	TOTAL NORMAL STRESS
σ'	kPa	EFFECTIVE NORMAL STRESS
τ	kPa	SHEAR STRESS
$\sigma_1, \sigma_2, \sigma_3$	kPa	PRINCIPAL STRESSES
ϵ	%	LINEAR STRAIN
$\epsilon_1, \epsilon_2, \epsilon_3$	%	PRINCIPAL STRAINS
E	kPa	MODULUS OF LINEAR DEFORMATION
G	kPa	MODULUS OF SHEAR DEFORMATION
μ	1	COEFFICIENT OF FRICTION

MECHANICAL PROPERTIES OF SOIL

m_v	kPa ⁻¹	COEFFICIENT OF VOLUME CHANGE
C_c	1	COMPRESSION INDEX
C_s	1	SWELLING INDEX
C_α	1	RATE OF SECONDARY CONSOLIDATION
c_v	m ² /s	COEFFICIENT OF CONSOLIDATION
H	m	DRAINAGE PATH
T_v	1	TIME FACTOR
U	%	DEGREE OF CONSOLIDATION
σ'_{vo}	kPa	EFFECTIVE OVERBURDEN PRESSURE
σ'_p	kPa	PRECONSOLIDATION PRESSURE
τ_f	kPa	SHEAR STRENGTH
c'	kPa	EFFECTIVE COHESION INTERCEPT
ϕ'	-°	EFFECTIVE ANGLE OF INTERNAL FRICTION
c_u	kPa	APPARENT COHESION INTERCEPT
ϕ_u	-°	APPARENT ANGLE OF INTERNAL FRICTION
τ_R	kPa	RESIDUAL SHEAR STRENGTH
τ_r	kPa	REMOULDED SHEAR STRENGTH
S_i	1	SENSITIVITY = $\frac{c_u}{\tau_r}$

PHYSICAL PROPERTIES OF SOIL

ρ_s	kg/m ³	DENSITY OF SOLID PARTICLES	n	1, %	POROSITY	e_{max}	1, %	VOID RATIO IN LOOSEST STATE
γ_s	kN/m ³	UNIT WEIGHT OF SOLID PARTICLES	w	1, %	WATER CONTENT	e_{min}	1, %	VOID RATIO IN DENSEST STATE
ρ_w	kg/m ³	DENSITY OF WATER	S_r	%	DEGREE OF SATURATION	I_D	1	DENSITY INDEX = $\frac{e_{max} - e}{e_{max} - e_{min}}$
γ_w	kN/m ³	UNIT WEIGHT OF WATER	w_L	%	LIQUID LIMIT	D	mm	GRAIN DIAMETER
ρ	kg/m ³	DENSITY OF SOIL	w_p	%	PLASTIC LIMIT	D_n	mm	n PERCENT - DIAMETER
γ	kN/m ³	UNIT WEIGHT OF SOIL	w_s	%	SHRINKAGE LIMIT	C_u	1	UNIFORMITY COEFFICIENT
ρ_d	kg/m ³	DENSITY OF DRY SOIL	I_p	%	PLASTICITY INDEX = $w_L - w_p$	h	m	HYDRAULIC HEAD OR POTENTIAL
γ_d	kN/m ³	UNIT WEIGHT OF DRY SOIL	I_L	1	LIQUIDITY INDEX = $\frac{w - w_p}{I_p}$	q	m ³ /s	RATE OF DISCHARGE
ρ_{sat}	kg/m ³	DENSITY OF SATURATED SOIL	I_C	1	CONSISTENCY INDEX = $\frac{w_L - w}{I_p}$	v	m/s	DISCHARGE VELOCITY
γ_{sat}	kN/m ³	UNIT WEIGHT OF SATURATED SOIL	DTPL		DRIER THAN PLASTIC LIMIT	i	1	HYDRAULIC GRADIENT
ρ'	kg/m ³	DENSITY OF SUBMERGED SOIL	APL		ABOUT PLASTIC LIMIT	k	m/s	HYDRAULIC CONDUCTIVITY
γ'	kN/m ³	UNIT WEIGHT OF SUBMERGED SOIL	WTP		WETTER THAN PLASTIC LIMIT	j	kN/m ³	SEEPAGE FORCE
e	1, %	VOID RATIO						

RECORD OF BOREHOLE No C-1

1 OF 3

METRIC

G.W.P. 3016-E-0009 LOCATION Coords: 4 758 632.7 N; 422 231.3 E ORIGINATED BY M.M.
DIST West Region HWY 401 BOREHOLE TYPE Continuous Flight Hollow Stem Augers COMPILED BY N.L.
DATUM Geodetic DATE 2019.10.10 LATITUDE 42.958442 LONGITUDE -81.060686 CHECKED BY N.R.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								WATER CONTENT (%)		
								○ UNCONFINED	+	● QUICK TRIAXIAL						×		
279.3	Ground						20	40	60	80	100							
0.0	400 mm ASPHALT over SAND and GRAVEL		1	SS	33													
	Dense to compact, Brown, Moist (PAVEMENT STRUCTURE)		2	SS	13													
277.5	CLAYEY SILT, trace sand, trace gravel		3	SS	8													
1.8	Stiff to very stiff, Grey, Moist		4	SS	10													
			5	SS	12													
			6	SS	14													
			7	SS	13													
			8	SS	14													
			9	SS	14													
			10	SS	20													
			11	SS	18													
			12	SS	17													
			13	SS	20													
			14	SS	15													
264.3																		

Continued Next Page

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

RECORD OF BOREHOLE No C-1

2 OF 3

METRIC

G.W.P. 3016-E-0009 LOCATION Coords: 4 758 632.7 N; 422 231.3 E ORIGINATED BY M.M.
DIST West Region HWY 401 BOREHOLE TYPE Continuous Flight Hollow Stem Augers COMPILED BY N.L.
DATUM Geodetic DATE 2019.10.10 LATITUDE 42.958442 LONGITUDE -81.060686 CHECKED BY N.R.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)					
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa		W _p W W _L				GR SA SI CL					
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE		WATER CONTENT (%)									
264.3 15.0	CLAYEY SILT, trace sand Stiff, Grey, Moist	(Cont'd)					264												
			15	SS	14		263												
	_____ silty clay		16	SS	14		262												0 5 42 53
261.4 17.9	SILTY SAND, trace silt, trace clay, trace gravel Compact, Grey, Wet						261												
			17	SS	30		260												
	_____ gravelly sand		18	SS	20		259												30 64 5 1
							258												
							257												
			19	SS	17		256												
							255												
							254												
			20	SS	10		253												
							252												
250.8 28.5	sandy CLAYEY SILT, trace gravel Hard, Grey, Moist						251												
			21	SS	101/23cm		250												7 34 41 18
			22	SS	100/25cm														

Continued Next Page

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

RECORD OF BOREHOLE No C-1

3 OF 3

METRIC

G.W.P. 3016-E-0009 LOCATION Coords: 4 758 632.7 N; 422 231.3 E ORIGINATED BY M.M.
 DIST West Region HWY 401 BOREHOLE TYPE Continuous Flight Hollow Stem Augers COMPILED BY N.L.
 DATUM Geodetic DATE 2019.10.10 LATITUDE 42.958442 LONGITUDE -81.060686 CHECKED BY N.R.

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			20	40	60	80	100	W _p	W	W _L							
249.3 30.0	End of borehole																					
<p>NOTES:</p> <p>1. Borehole was charged with drilling water at a depth of 3.8 m (El. 278.8) below the existing ground surface, thus groundwater level could not be established during or upon completion of drilling.</p> <p>2. Borehole caved-in at a depth of 18.9 m (El. 263.7) below the existing ground surface, upon extraction of hollow stem augers.</p> <p>Monitoring Well Readings:</p> <table border="1"> <thead> <tr> <th>Date</th> <th>Depth (m)</th> <th>Elev.</th> </tr> </thead> <tbody> <tr> <td>Dec.03/19</td> <td>Dry</td> <td>---</td> </tr> </tbody> </table> <p>Monitoring Well Legend:</p> <ul style="list-style-type: none"> Flush Mount Bentonite Filter Sand 19 mm PVC Screen Filter Bottom Cave-In 																	Date	Depth (m)	Elev.	Dec.03/19	Dry	---
Date	Depth (m)	Elev.																				
Dec.03/19	Dry	---																				

RECORD OF BOREHOLE No C-2

1 OF 3

METRIC

G.W.P. 3016-E-0009 LOCATION Coords: 4 758 626.0 N; 422 200.1 E ORIGINATED BY M.M.
DIST West Region HWY 401 BOREHOLE TYPE Continuous Flight Hollow Stem Augers COMPILED BY N.L.
DATUM Geodetic DATE 2019.10.09 LATITUDE 42.958388 LONGITUDE -81.061071 CHECKED BY N.R.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100		
279.3	Ground													
0.0	600 mm ASPHALT over SAND and GRAVEL		1	SS	31		279							
	Dense to compact, Brown, Moist (PAVEMENT STRUCTURE)		2	SS	14		278							
277.8	CLAYEY SILT, trace/some sand, trace gravel		3	SS	10		277							3 9 48 40
1.5	Stiff, Grey, Moist		4	SS	11		276							
			5	SS	10		275							1 8 48 43
			6	SS	11		274							
			7	SS	10		273							6 12 47 35
			8	SS	9		272							
			9	SS	10		271							
			10	SS	12		270							
			11	SS	11		269							
			12	SS	13		268							
			13	SS	8		267							
			14	SS	15		266							
264.3							265							

Continued Next Page

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

RECORD OF BOREHOLE No C-2

2 OF 3

METRIC

G.W.P. 3016-E-0009 LOCATION Coords: 4 758 626.0 N; 422 200.1 E ORIGINATED BY M.M.
DIST West Region HWY 401 BOREHOLE TYPE Continuous Flight Hollow Stem Augers COMPILED BY N.L.
DATUM Geodetic DATE 2019.10.09 LATITUDE 42.958388 LONGITUDE -81.061071 CHECKED BY N.R.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE									WATER CONTENT (%)
264.3 15.0	CLAYEY SILT, trace sand Stiff, Grey, Moist	(Cont'd)	15	SS	13		264									0 1 68 31	
							263										
262.5 16.8	SILTY SAND, trace clay, trace gravel Compact, Grey, Wet		16	SS	37		262										
							261										
			17	SS	20		260										
							259										0 85 13 2
			18	SS	15		258										
							257										
			19	SS	24		256										
							255										
							254										
			20	SS	27	253											
						252											
						251											
						250											
														</			

Continued Next Page



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

RECORD OF BOREHOLE No C-2

3 OF 3

METRIC

G.W.P. 3016-E-0009 LOCATION Coords: 4 758 626.0 N; 422 200.1 E ORIGINATED BY M.M.
 DIST West Region HWY 401 BOREHOLE TYPE Continuous Flight Hollow Stem Augers COMPILED BY N.L.
 DATUM Geodetic DATE 2019.10.09 LATITUDE 42.958388 LONGITUDE -81.061071 CHECKED BY N.R.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100	W _p	W	W _L		
249.3 30.0	End of Borehole  Groundwater level observed during drilling  Groundwater level measured upon completion of drilling NOTE: Borehole caved-in at a depth of 18.3 m (El. 261.0) below the existing ground surface, upon extraction of hollow stem augers.																

ONTARIO MTO 19KF028A-DORCHESTER ROAD.GPJ ONTARIO MTO.GDT 3/6/20

RECORD OF BOREHOLE No N

1 OF 3

METRIC

G.W.P. 3016-E-0009 LOCATION Coords: 4 758 664.6 N; 422 208.9 E ORIGINATED BY M.M.
 DIST West Region HWY 401 BOREHOLE TYPE Continuous Flight Hollow Stem Augers COMPILED BY N.L.
 DATUM Geodetic DATE 2019.10.08 LATITUDE 42.958734 LONGITUDE -81.060954 CHECKED BY N.R.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)					
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							W _p	W	W _L		
								○ UNCONFINED + FIELD VANE	● QUICK TRIAXIAL × LAB VANE						WATER CONTENT (%)				
286.2 0.0	Ground gravelly SAND		1	SS	50		286												
	Compact to loose, Brown, Moist (FILL)		2	SS	14		285												
			3	SS	8		284												
			4	SS	8		283												
			5	SS	6		282												
			6	SS	9		281												
			7	SS	11		280												
281.0 5.2	CLAYEY SILT, trace/some sand, trace gravel		8	SS	17		280												5 10 50 35
	Stiff to very stiff, Grey, Moist		9	SS	16		279												
			10	SS	10		278												
			11	SS	14		277												
			12	SS	9		276												
			13	SS	10		275												
			14	SS	16		274												3 13 48 36
						273													
271.2					272														

Continued Next Page





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

RECORD OF BOREHOLE No N

2 OF 3

METRIC

G.W.P. 3016-E-0009 LOCATION Coords: 4 758 664.6 N; 422 208.9 E ORIGINATED BY M.M.
 DIST West Region HWY 401 BOREHOLE TYPE Continuous Flight Hollow Stem Augers COMPILED BY N.L.
 DATUM Geodetic DATE 2019.10.08 LATITUDE 42.958734 LONGITUDE -81.060954 CHECKED BY N.R.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)												
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							WATER CONTENT (%)			GR	SA	SI	CL						
								○ UNCONFINED + FIELD VANE														20	40	60			
								● QUICK TRIAXIAL × LAB VANE																			
271.2 15.0								20	40	60	80	100		20	40	60											
CLAYEY SILT, Trace/some sand, trace gravel Stiff to very stiff, Grey, Moist	(Cont'd)		15	SS	15		271								○												
							270																				
			16	SS	14		269									▬						1	11	54	34		
							268																				
			17	SS	19		267									○											
							266										○										
							265																				
							264																				
							19	SS	9	263								▬	○	▬				0	0	46	54
										262																	
						261																					
260.3 25.9	SILTY SAND, trace clay Loose to compact, Grey, Moist		20	SS	8		260							○													
							259																				
							258																				
				21	SS		29	257								○						0	74	20	6		
256.2																											

Continued Next Page

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

RECORD OF BOREHOLE No N

3 OF 3

METRIC

G.W.P. 3016-E-0009 LOCATION Coords: 4 758 664.6 N; 422 208.9 E ORIGINATED BY M.M.
 DIST West Region HWY 401 BOREHOLE TYPE Continuous Flight Hollow Stem Augers COMPILED BY N.L.
 DATUM Geodetic DATE 2019.10.08 LATITUDE 42.958734 LONGITUDE -81.060954 CHECKED BY N.R.

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			20	40	60	80	100	W _p	W	W _L											
256.2 30.0	SILTY SAND, trace clay Dense, Grey, Moist		22	SS	38		256											0 71 19 10								
255.6 30.6	End of Borehole																									
<p> Groundwater level observed during drilling</p> <p> Groundwater measured upon completion of drilling</p> <p>NOTE: Borehole caved-in at a depth of 10.0 m (El. 276.2) below the existing ground surface, upon extraction of hollow stem augers.</p> <p>Monitoring Well Readings:</p> <table border="1"> <thead> <tr> <th>Date</th> <th>Depth (m)</th> <th>Elev.</th> </tr> </thead> <tbody> <tr> <td>Dec.03/19</td> <td>Dry</td> <td>---</td> </tr> <tr> <td>Mar.05/20</td> <td>Dry</td> <td>---</td> </tr> </tbody> </table> <p>Monitoring Well Legend:</p> <ul style="list-style-type: none"> Flush Mount Bentonite Filter Sand 19 mm PVC Screen Filter Bottom Cave-In 																		Date	Depth (m)	Elev.	Dec.03/19	Dry	---	Mar.05/20	Dry	---
Date	Depth (m)	Elev.																								
Dec.03/19	Dry	---																								
Mar.05/20	Dry	---																								

ONTARIO MTO 19KF028A-DORCHESTER ROAD.GPJ ONTARIO MTO.GDT 3/6/20

METRIC

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

RECORD OF BOREHOLE No S

2 OF 3

METRIC

G.W.P. 3016-E-0009 LOCATION Coords: 4 758 596.0 N; 422 245.3 E ORIGINATED BY M.M.
 DIST West Region HWY 401 BOREHOLE TYPE Continuous Flight Hollow Stem Augers COMPILED BY N.L.
 DATUM Geodetic DATE 2019.10.11 LATITUDE 42.95811 LONGITUDE -81.060523 CHECKED BY N.R.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100		
269.5 15.0	CLAYEY SILT, trace sand, trace gravel Stiff to very stiff, Grey, Moist		15	SS	17		269							1 10 53 36
			16	SS	16		268							
			17	SS	14		266							
			18	SS	10		264							0 1 75 24
			19	SS	26		261							
259.0 25.5	SILTY SAND, trace clay, trace gravel Dense to compact, Grey, Moist		20	SS	33		259							
			21	SS	16		255							1 85 10 4
254.5														

Continued Next Page

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

RECORD OF BOREHOLE No S

3 OF 3

METRIC

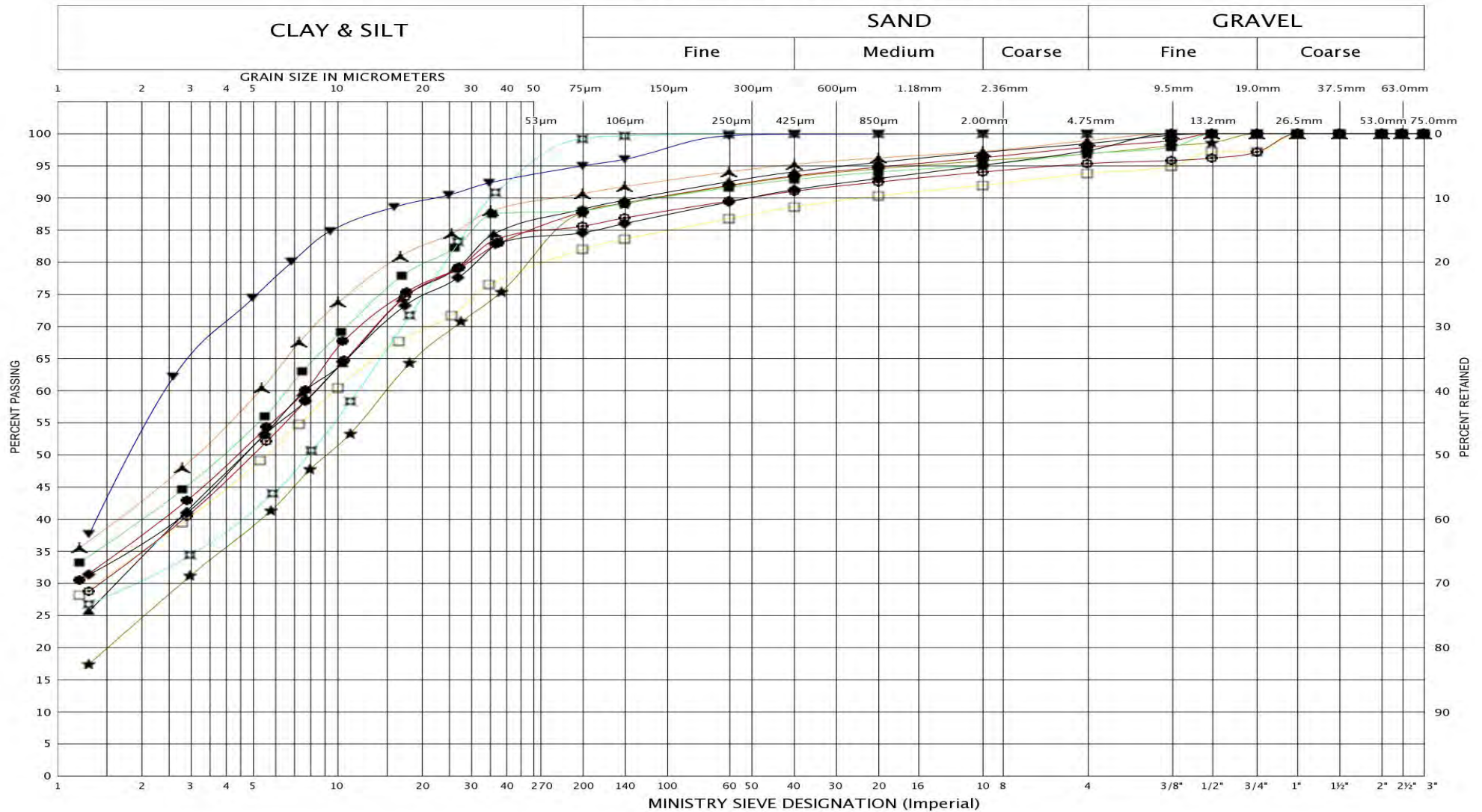
G.W.P. 3016-E-0009 LOCATION Coords: 4 758 596.0 N; 422 245.3 E ORIGINATED BY M.M.
DIST West Region HWY 401 BOREHOLE TYPE Continuous Flight Hollow Stem Augers COMPILED BY N.L.
DATUM Geodetic DATE 2019.10.11 LATITUDE 42.95811 LONGITUDE -81.060523 CHECKED BY N.R.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC NATURAL LIQUID LIMIT MOISTURE LIMIT CONTENT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100	W _p	W	W _L		
254.5																	
30.0	SILTY SAND, trace clay, trace gravel Compact, Grey, Moist		22	SS	10												
253.9																	
30.6	End of borehole																
	 Groundwater level observed during drilling Groundwater measured upon completion of drilling NOTE: Borehole caved-in at a depth of 19.8 m (El. xxx.x) below the existing ground surface, upon extraction of hollow stem augers. Monitoring Well Readings: Date Depth Elev. Dec.03/19 Dry --- Mar.05/20 Dry ---																

Monitoring Well Legend:

- Stick-up Monument
- Bentonite
- Filter Sand
- 19 mm PVC Screen
- Filter Bottom
- Cave-In

UNIFIED SOIL CLASSIFICATION SYSTEM



LEGEND	BH	C1	C1	C1	C1	C2	C2	C2	C2	N	N
	SAMPLE	4	8	12	16	3	6	9	15	8	13
	SYMBOL	●	▲	★	▼	■	▲	□	⊠	⊕	◆



GRAIN SIZE DISTRIBUTION

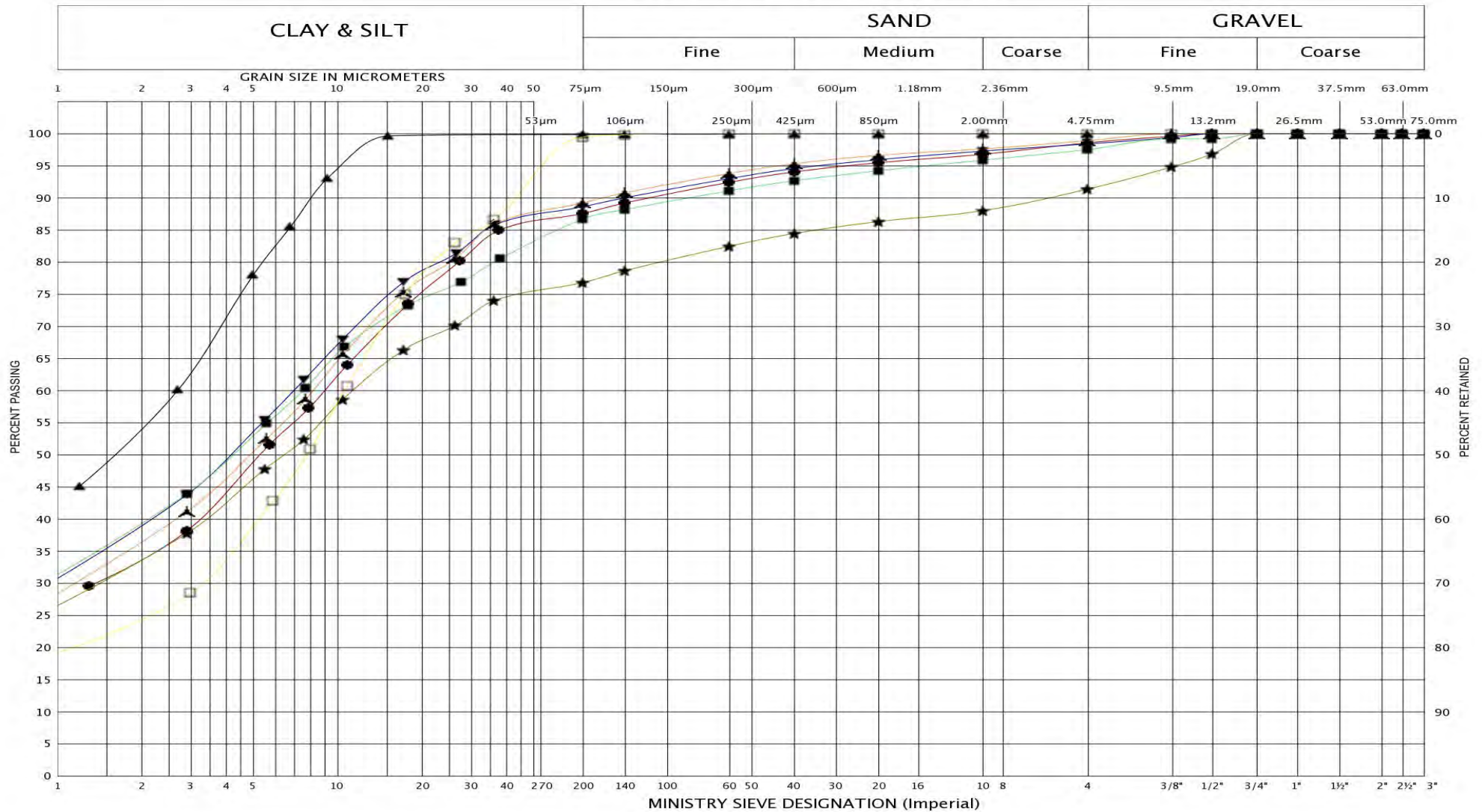
Clayey Silt, Trace/Some Sand, Trace Gravel

FIG No.: GS-DR1-1A

HWY : 401

GWP 3016-E-2009

UNIFIED SOIL CLASSIFICATION SYSTEM



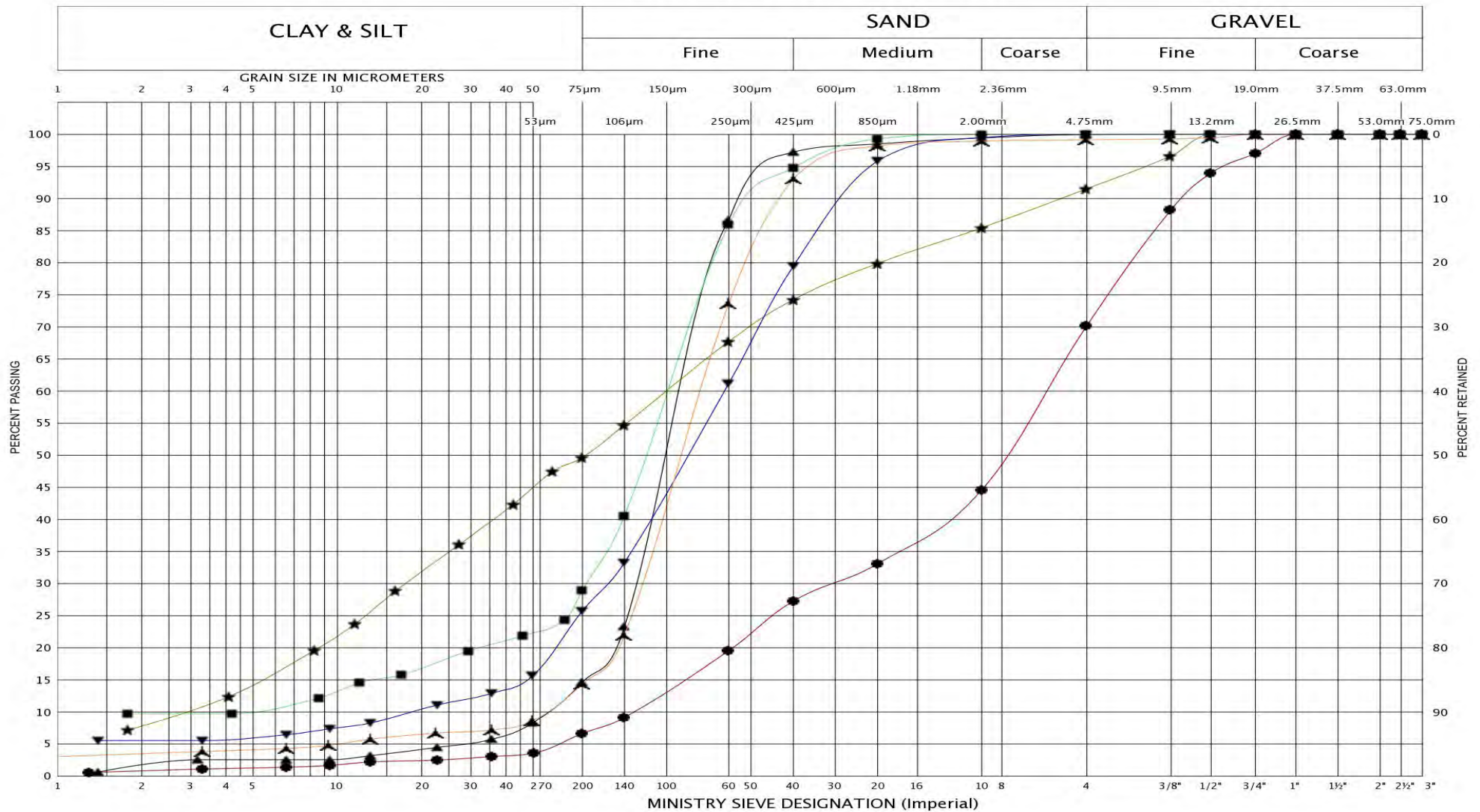
LEGEND	BH	N	N	S	S	S	S	S
	SAMPLE	16	19	4	9	12	15	18
	SYMBOL	●	▲	★	▼	■	▲	□



GRAIN SIZE DISTRIBUTION
Clayey Silt, Trace/Some Sand, Trace Gravel

FIG No.: GS-DR1-1B
HWY : 401
GWP 3016-E-2009

UNIFIED SOIL CLASSIFICATION SYSTEM



GRAIN SIZE DISTRIBUTION

Silty Sand, Trace Clay, Trace Gravel/Gravelly

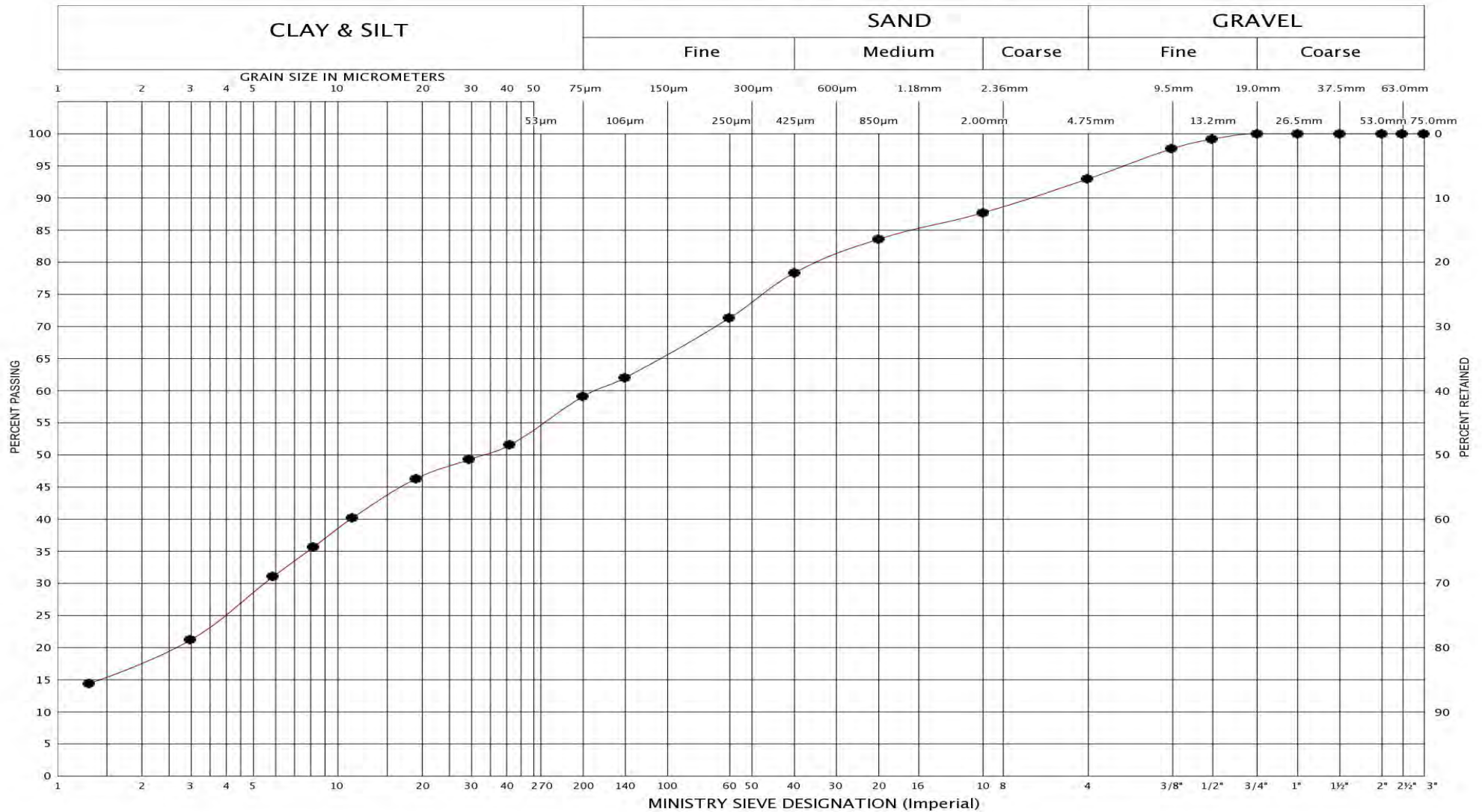
FIG No.: GS-DR1-2

HWY : 401

GWP 3016-E-2009



UNIFIED SOIL CLASSIFICATION SYSTEM



LEGEND	BH	C1
	SAMPLE	21
	SYMBOL	●



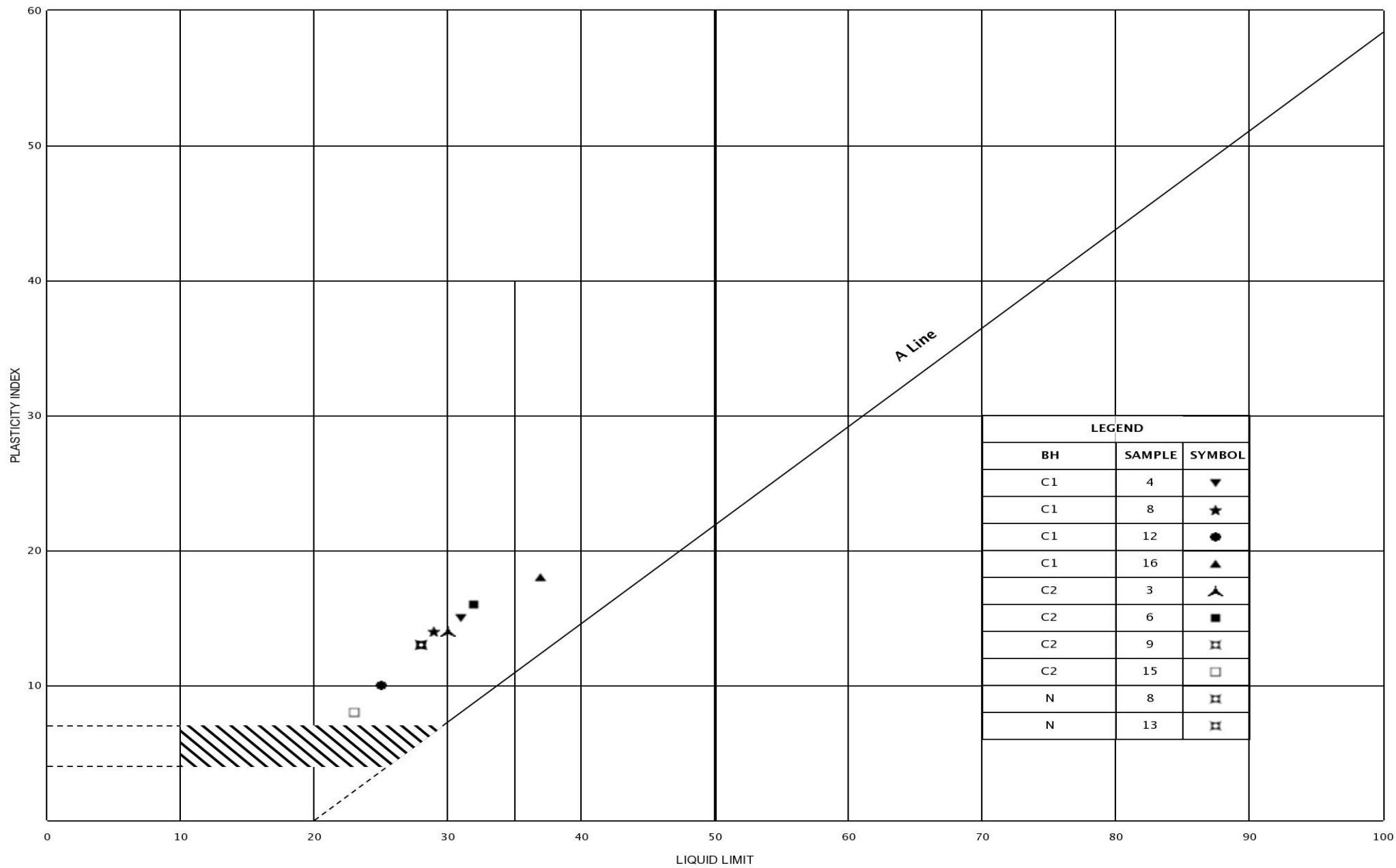
GRAIN SIZE DISTRIBUTION

Sandy Clayey Silt, Trace Gravel

FIG No.: GS-DR1-3

HWY : 401

GWP 3016-E-2009



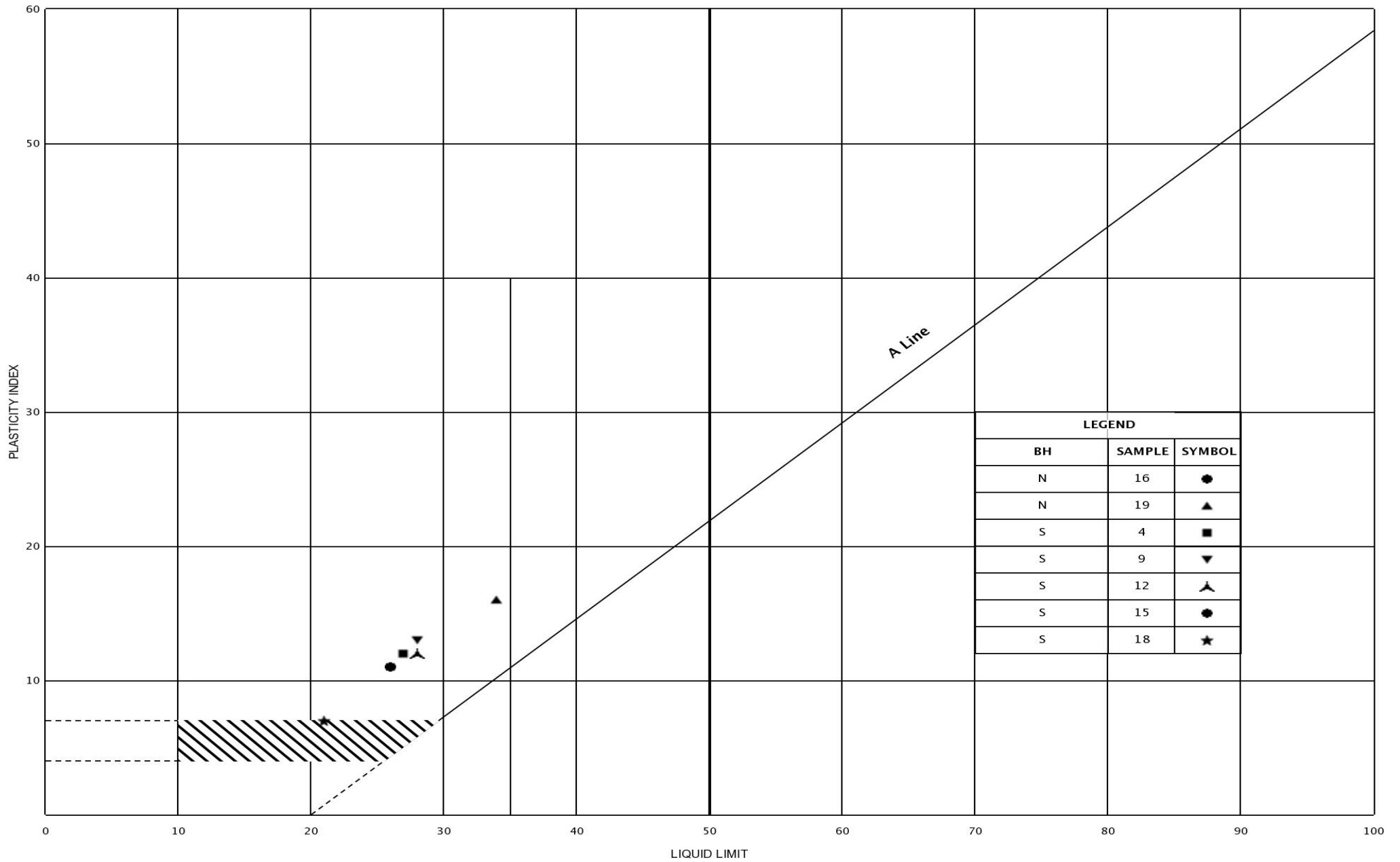
PLASTICITY CHART

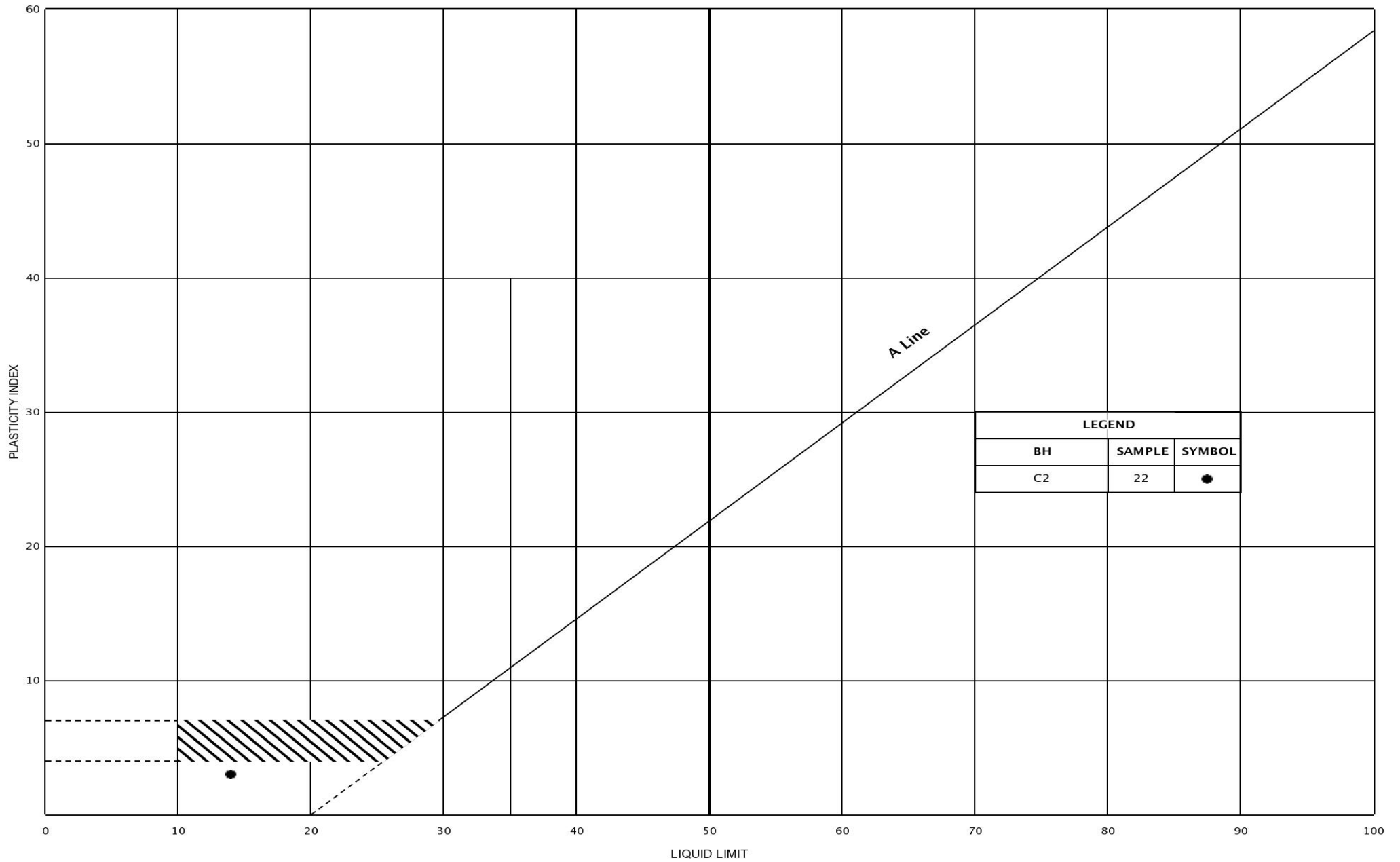
Clayey Silt/Silty Clay, Trace/Some Sand, Trace Gravel

FIG No.: PC-DR1-1A

HWY.: 401

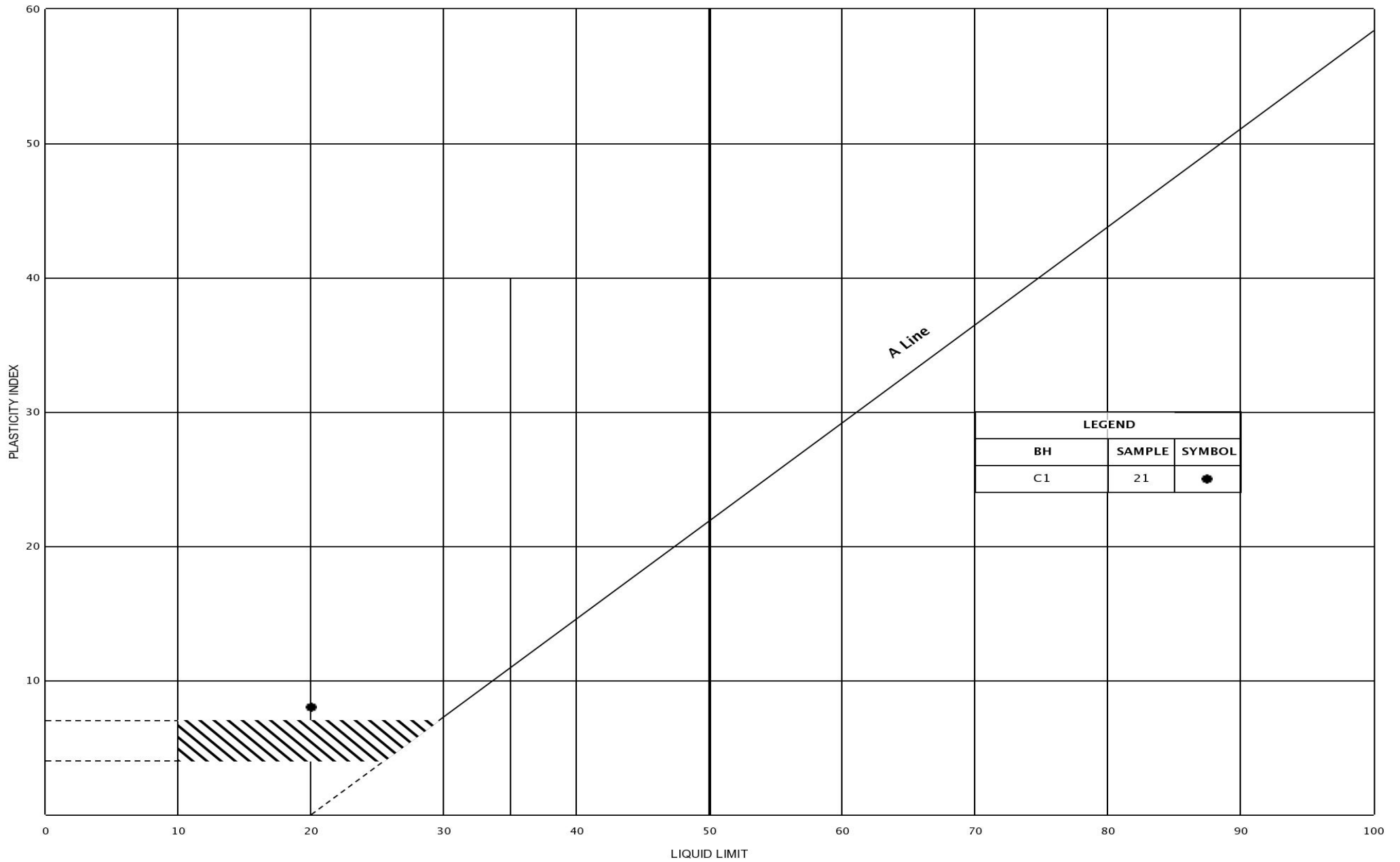
GWP 3016-E-2009





PLASTICITY CHART
Silty Sand, Trace Clay, Trace Gavel

FIG No.:	PC-DR1-2
HWY.:	401
GWP	3016-E-2009



PLASTICITY CHART
Sandy Clayey Silt, Trace Gravel

FIG No.:	PC-DR1-3
HWY.:	401
GWP	3016-E-2009

APPENDIX C

Current Investigation

ABBREVIATIONS AND TERMS USED ON RECORDS OF BOREHOLES AND TEST PITS

MINISTRY OF TRANSPORTATION, ONTARIO

PARTICLE SIZES OF CONSTITUENTS

Soil Constituent	Particle Size Description	Millimetres	Inches (US Std. Sieve Size)
BOULDERS	Not Applicable	>200	>8
COBBLES	Not Applicable	75 to 200	3 to 8
GRAVEL	Coarse	19 to 75	0.75 to 3
	Fine	4.75 to 19	(4) to 0.75
		2.00 to 4.75	(10) to (4)
SAND	Coarse	0.425 to 2.00	(40) to (10)
	Medium	0.075 to 0.425	(200) to (40)
	Fine		
FINES	Classified by plasticity	<0.075	< (200)

MODIFIERS FOR SECONDARY COMPONENTS^{1,2}

Percentage by Mass	Modifier
> 35	Use 'and' to combine primary and secondary component (<i>i.e.</i> , SAND and gravel)
> 20 to 35	Primary soil name prefixed with "gravelly, sandy" as applicable
> 10 to 20	some (<i>i.e.</i> , some sand)
≤ 10	trace (<i>i.e.</i> , trace fines)

1. Only applicable to components not described by Primary Group Name.

2. Classification of Primary Group Name based on Unified Soil Classification System (ASTM D2487) for coarse-grained soils; fine-grained soils described per current MTO Soil Classification System.

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.) split-spoon sampler for a distance of 300 mm (12 in.). Values reported are as recorded in the field and are uncorrected.

Cone Penetration Test (CPT)

An 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 (*u*) and sleeve friction (*f_s*) are recorded electronically at 25 mm penetration intervals.

Dynamic Cone Penetration Resistance (DCPT); 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

SAMPLES

AS	Auger sample
BS	Block sample
CS	Chunk sample
DD	Diamond Drilling
DO or DP	Seamless open ended, driven or pushed tube sampler – note size
DS	Denison type sample
GS	Grab Sample
MC	Modified California Samples
MS	Modified Shelby (for frozen soil)
RC / SC	Rock core / Soil core
SS	Split spoon sampler – note size
ST	Slotted tube
TO	Thin-walled, open – note size (Shelby tube)
TP	Thin-walled, piston – note size (Shelby tube)
WS	Wash sample
OD / ID	Outer Diameter / Inner Diameter
HSA / SSA	Hollow-Stem Augers / Solid-Stem Augers

SOIL TESTS

w	water content
PL, w _p	plastic limit
LL, 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
GS	specific gravity
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 (FV)	field vane (LV-laboratory vane test)
γ	unit weight

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

COARSE-GRAINED SOILS

Compactness¹

Term	SPT 'N' (blows/0.3m) ²
Very Loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very Dense	> 50

1. Definition of compactness terms are based on SPT 'N' ranges as provided in Terzaghi, Peck and Mesri (1996). Many factors affect the recorded SPT 'N' value, including hammer efficiency (which may be greater than 60% in automatic trip hammers), overburden pressure, groundwater conditions, and grain size. As such, the recorded SPT 'N' value(s) should be considered only an approximate guide to the soil compactness. These factors need to be considered when evaluating the results, and the stated compactness terms should not be relied upon for design or construction.

2. SPT 'N' in accordance with ASTM D1586, uncorrected for the effects of overburden pressure.

FINE-GRAINED SOILS

Consistency

Term	Undrained Shear Strength (kPa)	SPT 'N' ^{1,2} (blows/0.3m)
Very Soft	< 12	0 to 2
Soft	12 to 25	2 to 4
Firm	25 to 50	4 to 8
Stiff	50 to 100	8 to 15
Very Stiff	100 to 200	15 to 30
Hard	> 200	> 30

1. SPT 'N' in accordance with ASTM D1586, uncorrected for overburden pressure effects; approximate only.

2. SPT 'N' values should be considered ONLY an approximate guide to consistency; for sensitive clays (e.g., Champlain Sea clays), the N-value approximation for consistency terms does NOT apply. Rely on direct measurement of undrained shear strength or other manual observations.

Field Moisture Condition

Term	Description
Dry	Soil flows freely through fingers.
Moist	Soils are darker than in the dry condition and may feel cool.
Wet	As moist, but with free water forming on hands when handled.

LIST OF SYMBOLS

MINISTRY OF TRANSPORTATION, ONTARIO

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

I. GENERAL

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

II. STRESS AND STRAIN

γ	shear strain
Δ	change in, e.g. in stress: $\Delta\sigma$
ε	linear strain
ε_v	volumetric strain
η	coefficient of viscosity
ν	Poisson's ratio
σ	total stress
σ'	effective stress ($\sigma' = \sigma - u$)
σ'_{vo}	initial effective overburden stress
$\sigma_1, \sigma_2, \sigma_3$	principal stress (major, intermediate, minor)

σ_{oct}	mean stress or octahedral stress $= (\sigma_1 + \sigma_2 + \sigma_3)/3$
τ	shear stress
u	porewater pressure
E	modulus of deformation
G	shear modulus of deformation
K	bulk modulus of compressibility

III. SOIL PROPERTIES

(a) Index Properties

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

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

(a) Index Properties (continued)

w	water content
w_L or LL	liquid limit
w_P or PL	plastic limit
I_P or PI	plasticity index = $(w_L - w_P)$
NP	non-plastic
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
$C_{a(e)}$	secondary compression index
C_a	rate of secondary compression
$C_{a(e)}$	modified 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
c'	effective cohesion
ϕ'	effective angle of internal friction
δ	angle of interface friction
μ	coefficient of friction = $\tan \delta$
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 or q'	$(\sigma_1 - \sigma_3)/2$ or $(\sigma'_1 - \sigma'_3)/2$
q_u	compressive strength $(\sigma_1 - \sigma_3)$
S_t	sensitivity

Notes: 1
2

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



PROJECT		RECORD OF BOREHOLE No 101				SHEET 3 OF 3		METRIC								
G.W.P. 3053-11-03		LOCATION N 4758711.4; E 422218.6 MTM NAD 83 ZONE 11 (LAT. 42.959153; LONG. -81.060825)				ORIGINATED BY KB										
DIST West HWY 401		BOREHOLE TYPE CME5 5, 114 mm Dia Casing, Mud Rotary				COMPILED BY SA										
DATUM Geodetic		DATE November 17, 2022				CHECKED BY MEB/LCC										
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								
	— CONTINUED FROM PREVIOUS PAGE —						20	40	60	80	100					
253.5	SAND (SP), trace silt, containing silt pockets Dense to very dense Grey Wet		17A	SS	40											
30.9	END OF BOREHOLE		17B													
	NOTES: 1. 210 mm diameter hollow stem augers set to a depth of approximately 3 m before running 114 mm diameter casing using mud rotary techniques. 2. 64 mm diameter casing installed to a depth of 30.6 m and grouted into place to permit vertical seismic profile (VSP) testing.															

GTA-MTO 001 S:\CLIENTS\MTOWHY_401_&_DORCHESTER02_DATA\GINTHWY_401-&_DORCHESTER.GPJ GAL-GTA.GDT 3/27/23



PROJECT		22520660		RECORD OF BOREHOLE No 102		SHEET 1 OF 3		METRIC					
G.W.P.		3053-11-03		LOCATION		N 4758588.5; E 422278.7 MTM NAD 83 ZONE 11 (LAT. 42.958038; LONG. -81.060114)		ORIGINATED BY KB/JAK					
DIST		West HWY 401		BOREHOLE TYPE				COMPILED BY SA					
DATUM		Geodetic		DATE		January 9, 2023		CHECKED BY MEB/LCC					
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			REMARKS & GRAIN SIZE DISTRIBUTION (%)				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	20 40 60 80 100	PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	GR SA SI CL
279.6 0.0	GROUND SURFACE TOPSOIL (460 mm)												
279.1 0.5	CLAYEY SILT (CL), trace to some sand, trace gravel (TILL) Stiff to very stiff Brown becoming grey below approximately 3.8 m depth Moist		1	SS	10		279						
			2	SS	16		278						
			3	SS	20		277						
			4	SS	11		276						
			5	SS	16		275						
			6	SS	9		274						
			7	SS	13		273						
			8	SS	13		272						
			9	SS	14		271						
			10	SS	16		270						
							269						
							268						
							267						
							266						
							265						

Continued Next Page

+ 3, X 3: Numbers refer to Sensitivity O 3% STRAIN AT FAILURE

GTA-MTO 001 S:\CLIENTS\MTOWHY_401 & DORCHESTER02_DATA\GTA-MTO 001 GAL-GTA-GDT 3/27/23

S:\CLIENTS\MT01 HWY 401 & DORCHESTER\02 DATA\GINT\HWY 401-& DORCHESTER.GPJ GAL-GTA.GDT 3/27/23

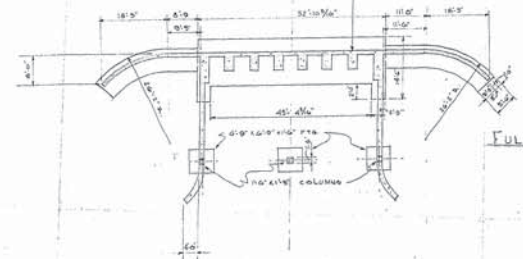
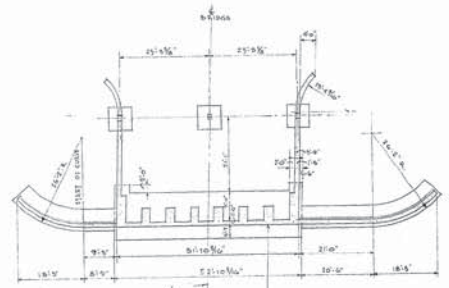
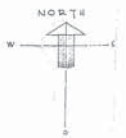


PROJECT		RECORD OF BOREHOLE No 102				SHEET 3 OF 3		METRIC								
G.W.P. 22520660		LOCATION N 4758588.5; E 422278.7 MTM NAD 83 ZONE 11 (LAT. 42.958038; LONG. -81.060114)				ORIGINATED BY KB/JAK										
DIST West HWY 401		BOREHOLE TYPE				COMPILED BY SA										
DATUM Geodetic		DATE January 9, 2023				CHECKED BY MEB/LCC										
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa					WATER CONTENT (%)			
							20	40	60	80	100	W _p	W	W _L		
							UNCONFINED + FIELD VANE									
							QUICK TRIAXIAL x REMOULDED									
							20	40	60	80	100	10	20	30		
248.7	Sandy SILT (ML), some gravel Very dense Grey Wet		16	SS	81		249									
30.9	END OF BOREHOLE															
	NOTES: 1. 210 mm diameter hollow stem augers set to a depth of approximately 3 m before running 114 mm diameter casing using mud rotary techniques. 2. 64 mm diameter casing installed to a depth of 30.6 m and grouted into place to permit vertical seismic profile (VSP) testing.															

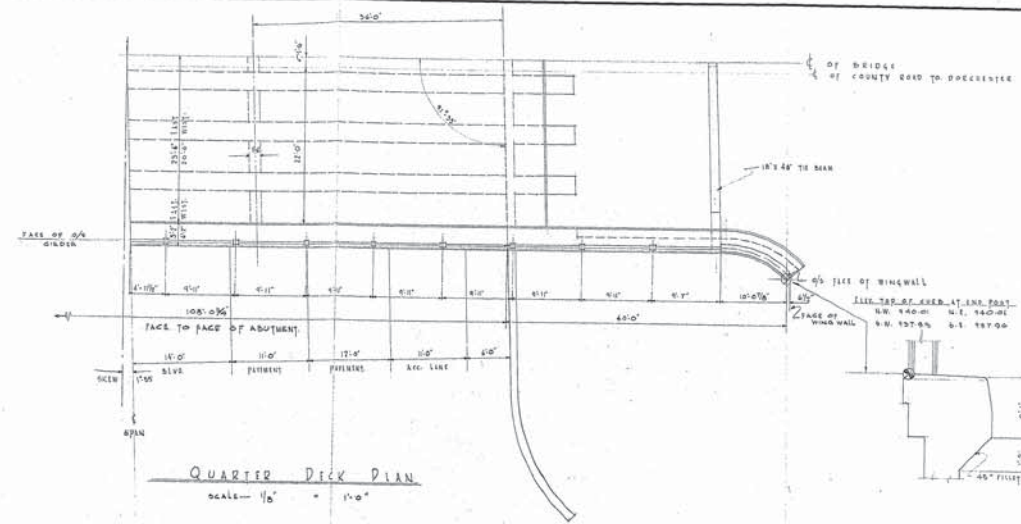
GTA-MTO 001 S:\CLIENTS\MTOWHY_401_&_DORCHESTER02_DATA\GINTHWY_401-&_DORCHESTER.GPJ GAL-GTA.GDT 3/27/23

APPENDIX D

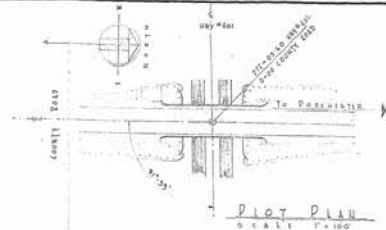
DHO Drawing Nos. D-3498-1 and 2



Full Footing Plan
SCALE - 1/4" = 1'-0"



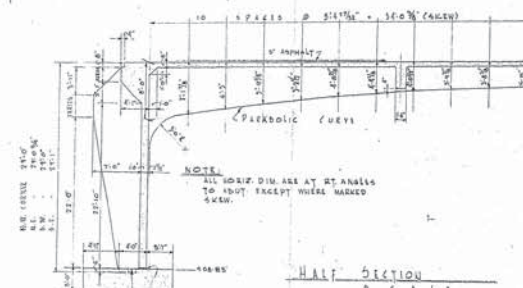
Quarter Deck Plan
SCALE - 1/8" = 1'-0"



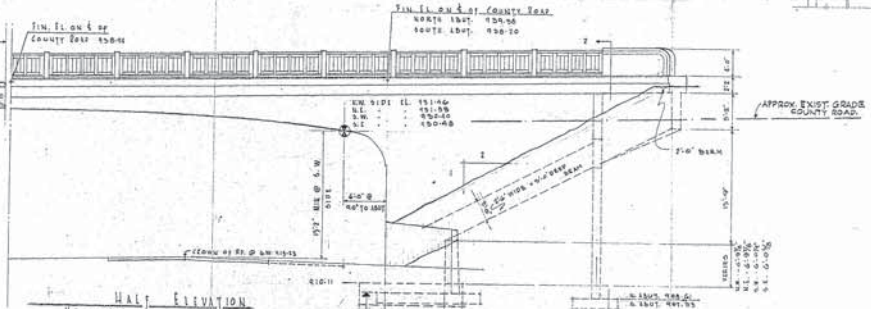
Plot Plan
SCALE - 1" = 100'



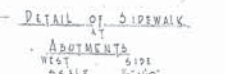
Key for Loc. of Girder



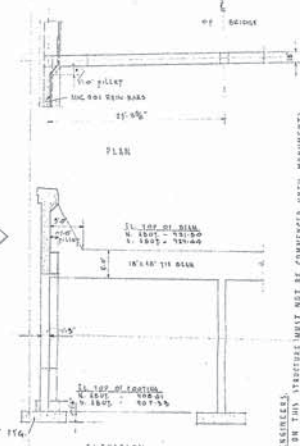
Half Section
SCALE - 1/4" = 1'-0"



Half Elevation
SCALE - 1/8" = 1'-0"



Detail of Sidewalk



Elevation

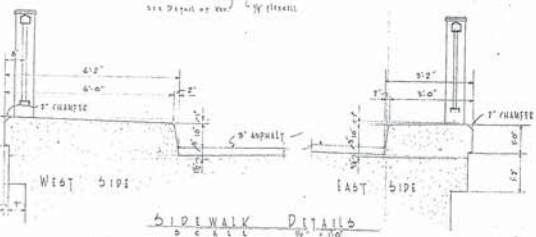
SECTION 2-2
Tie Beam Detail
SCALE - 1/4" = 1'-0"

REFERENCE PLANS (ISSUED JULY 23/54)
PLAN - F-3528-12 (DW-4559)
PROFILE - F-3528-11

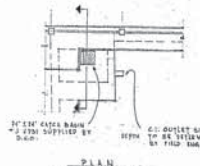


DORCHESTER NORTH TWP BRIDGE #10			
M. M. DILLON & CO. LTD.			
CONSULTING ENGINEERS			
DEPARTMENT OF HIGHWAYS-ONTARIO			
BRIDGE OFFICE-TORONTO			
DORCHESTER NORTH TWP UNDERPASS			
COUNTY ROAD			
THE KING'S HIGHWAY NO. 401		DIV. NO. 7	
CO. MIDDLERY		CON. 1	
GENERAL LAYOUT			
APPROVED			
R. M. DILLON			
CHIEF ENGINEER			
DESIGN	BY R. M. DILLON	CHECKED BY W. C. DILLON	DATE DEC. 8, 1954
TRACING	BY R. M. DILLON	CHECKED BY W. C. DILLON	DATE DEC. 8, 1954
55-72			

PRINT RECORD
NO.
REV.
DATE
BY
CHKD.
APP'D.



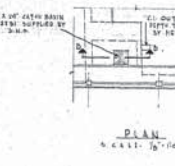
Sideview Details
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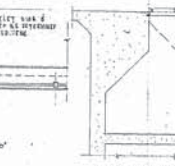
Catch Basin Detail



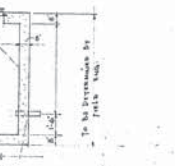
Catch Basin Detail



Catch Basin Detail



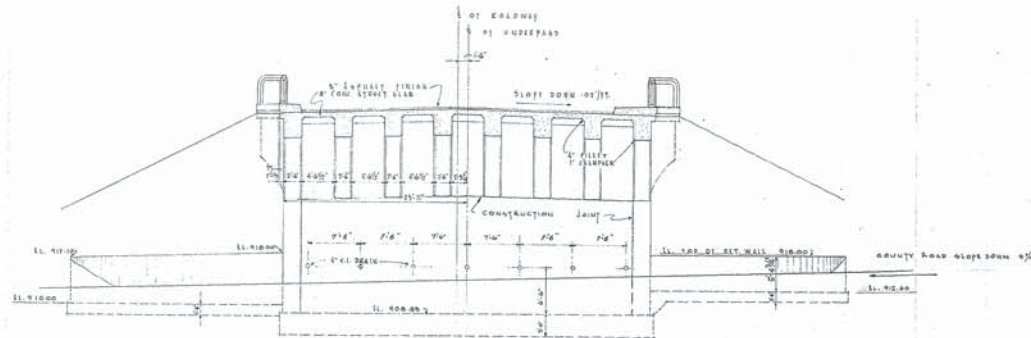
Catch Basin Detail



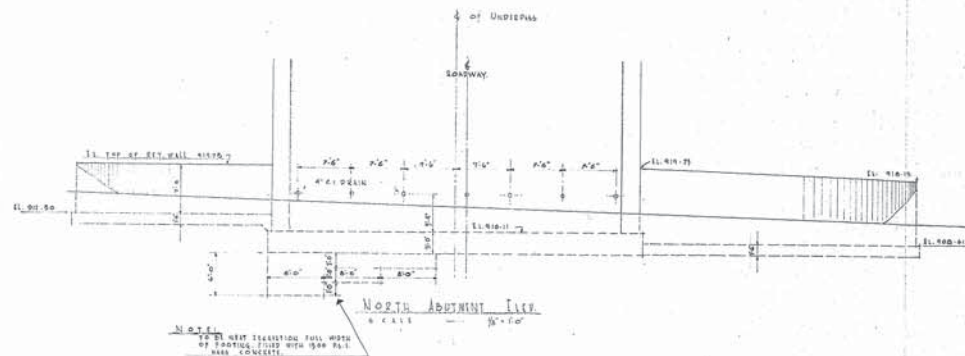
Catch Basin Detail

Catch Basin Details
SCALE - 1/4" = 1'-0"

NOTE TO BUILDERS (ENGINEERS): CONTRACTORS MUST NOT BE CONCERNED WITH THE MEASUREMENTS OF THE BRIDGE STRUCTURE UNLESS THEY ARE SPECIFICALLY MENTIONED IN THE CONTRACT DOCUMENTS. TO THE CONTRACT DOCUMENTS HAVE BEEN LATESTLY AND CORRECTED BY THE DIVISION ENGINEER. HERE TO CONTRACTOR. STRUCTURE TO BE BUILT IN ACCORDANCE WITH THE GENERAL SPECIFICATIONS FOR HIGHWAY BRIDGES, OUTSTANDING, 1953, FORM NO. 7, AND THE SPECIAL SPECIFICATIONS ATTACHED TO THE "INFORMATION TO BIDDERS" SHEET, SPECIAL COPIES OF WHICH MAY BE OBTAINED FROM THE DIVISION ENGINEER. CONTRACTOR SHALL BE RESPONSIBLE FOR THE CORRECTNESS OF THE CONTRACT DOCUMENTS. CONTRACTOR SHALL BE RESPONSIBLE FOR THE CORRECTNESS OF THE CONTRACT DOCUMENTS.

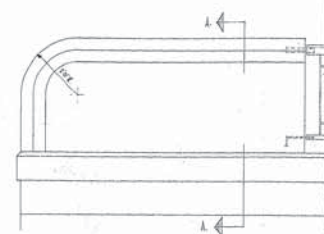


SOUTH ABUTMENT ELEV
SCALE - 1/4" = 1'-0"

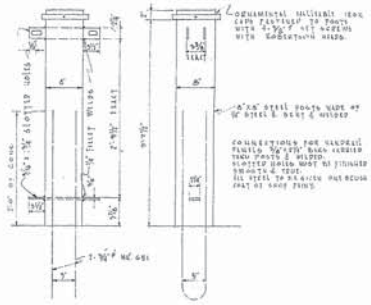
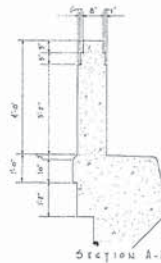


NORTH ABUTMENT ELEV
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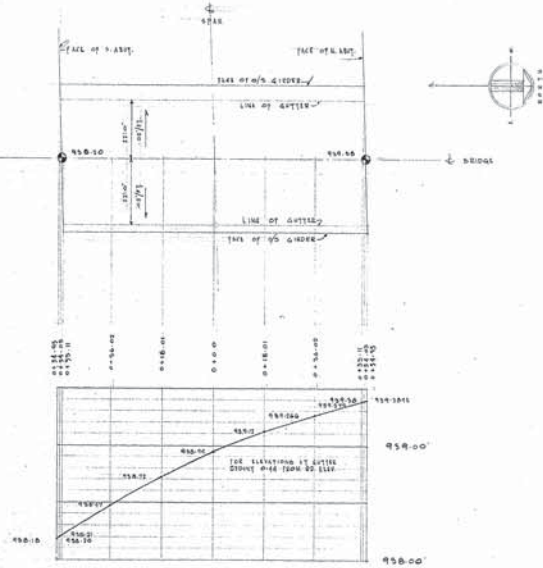
NOTE: SEE NEXT ELEVATION FOR NORTH ABUTMENT, FIRST WITH TYPICAL PILE CONNECTION.



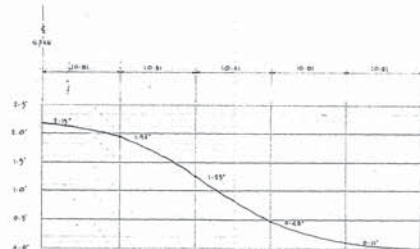
CONCRETE AND POST
SCALE - 1/4" = 1'-0"



Steel Post Detail
SCALE - 1/4" = 1'-0"



PLAN OF FIN. RD. ON & OF BRIDGE
PLAN - SCALE - 1/4" = 1'-0" PROFILE - SCALE - 1/4" = 1'-0"



CORRECTION FOR DEAD LOAD - 1/2 LIVE LOAD DEFLECTION
ADD TO FINISHED ROAD ELEVATIONS
SCALE - 1/4" = 1'-0"

PRINT RECORD	
9/21/40	1940



REQ'N #95587

DORCHESTER NORTH TWP BRIDGE #10

M. M. DILLON & CO. LTD.
ENGINEERS

DEPARTMENT OF HIGHWAYS-ONTARIO
BRIDGE OFFICE-TORONTO

DORCHESTER NORTH TWP UNDERPASS
COUNTY ROAD

THE KING'S HIGHWAY NO. 401 DIV. No. 2

CO. MIDDLETOWN

TWP. DORCHESTER NORTH LOT 16 & 17 CON. 1

ABUT. & HAUPDRAL DETAILS - FIN. ROAD ELEV.

APPROVED: *[Signature]* CHIEF ENGINEER

60840

REVISION	DATE	BY	DESCRIPTION

DESIGN	E.L.C.	CHKD	INCH	CONTRACT	NO.

DESIGN	E.L.C.	CHKD	INCH	CONTRACT	NO.

DATE DEC. 8, 1954

NO. 516

55-72

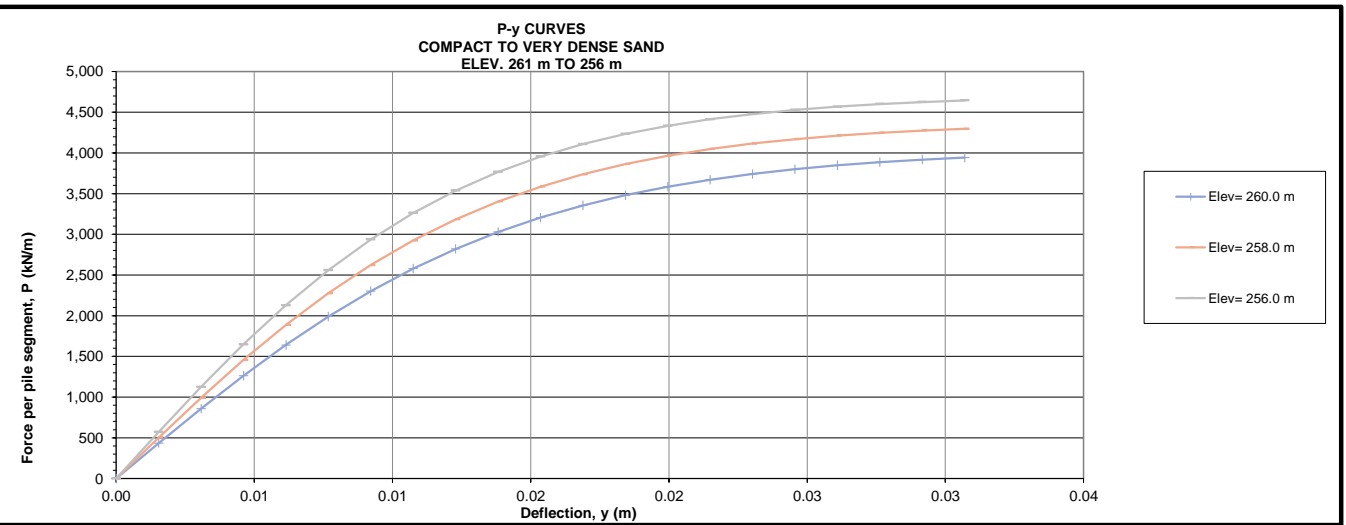
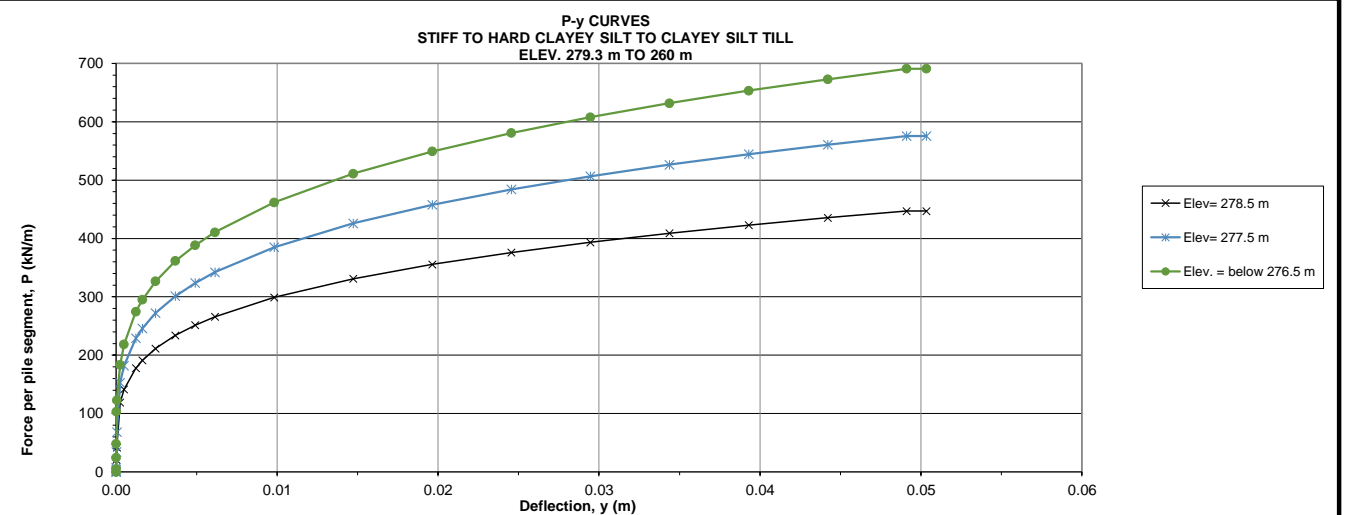
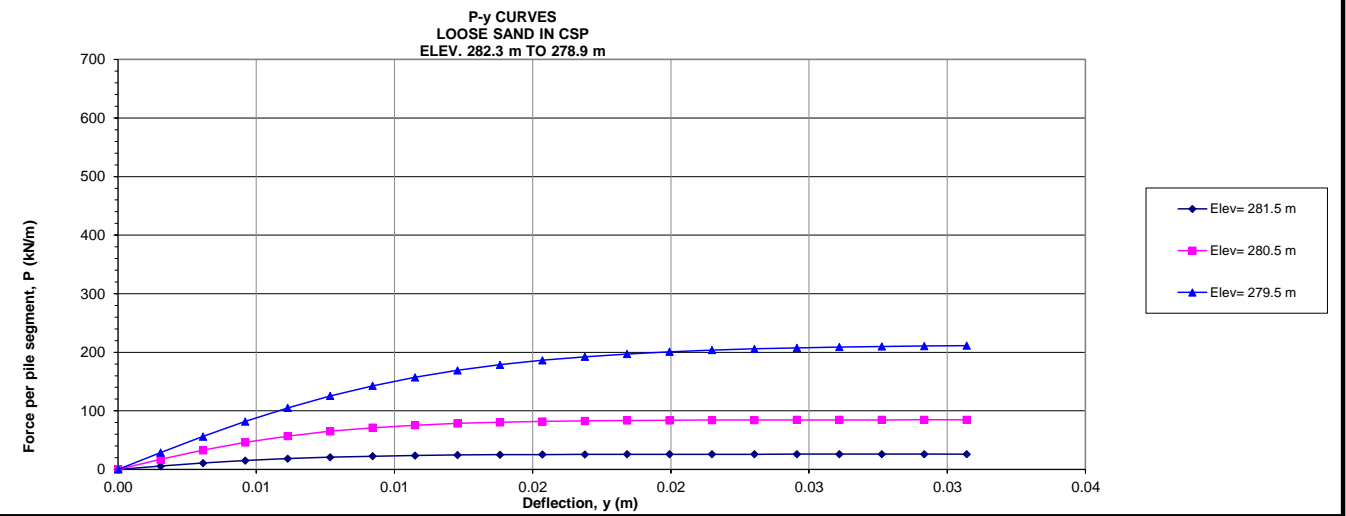
5-498

APPENDIX E

**P-y Curves for Abutment Pile
Design**

Description Depth (z) * Elevation P-y Curves	LOOSE SAND IN CSP						STIFF TO HARD CLAYEY SILT TO CLAYEY SILT TILL						COMPACT TO VERY DENSE SAND					
	z= .5 m		z= 1.5 m		z= 2.5 m		z= 3.5 m		z= 4.5 m		x = below 5.5 m		z= 22.0 m		z= 24.0 m		z= 26.0 m	
	Elev= 281.5 m		Elev= 280.5 m		Elev= 279.5 m		Elev= 278.5 m		Elev= 277.5 m		Elev = below 276.5 m		Elev= 260.0 m		Elev= 258.0 m		Elev= 256.0 m	
	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)
0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
0.002	5.664	0.002	17.033	0.002	28.607	0.000	0.417	0.000	0.683	0.000	4.794	0.002	433.543	0.002	502.054	0.002	570.540	
0.003	10.818	0.003	32.744	0.003	56.203	0.000	2.087	0.000	3.414	0.000	23.975	0.003	857.273	0.003	991.105	0.003	1124.600	
0.005	15.118	0.005	46.190	0.005	81.914	0.000	4.175	0.000	6.828	0.000	47.942	0.005	1262.244	0.005	1455.468	0.005	1647.700	
0.006	18.455	0.006	56.981	0.006	105.099	0.000	20.872	0.000	34.141	0.000	103.300	0.006	1641.097	0.006	1885.859	0.006	2128.300	
0.008	20.902	0.008	65.203	0.008	125.413	0.000	41.746	0.000	68.283	0.000	122.800	0.008	1988.482	0.008	2275.841	0.008	2559.200	
0.009	22.623	0.009	71.223	0.009	142.759	0.000	118.891	0.000	153.031	0.000	183.700	0.009	2301.198	0.009	2622.014	0.009	2937.000	
0.011	23.797	0.011	75.505	0.011	157.237	0.000	141.380	0.000	182.006	0.000	218.400	0.011	2578.103	0.011	2923.832	0.011	3261.900	
0.012	24.584	0.012	78.487	0.012	169.109	0.001	177.786	0.001	228.919	0.001	274.700	0.012	2819.639	0.012	3182.818	0.012	3536.600	
0.014	25.102	0.014	80.532	0.014	178.706	0.002	191.031	0.002	245.956	0.002	295.100	0.014	3027.764	0.014	3402.127	0.014	3765.700	
0.015	25.441	0.015	81.923	0.015	186.358	0.002	211.420	0.002	272.231	0.002	326.600	0.015	3205.032	0.015	3585.582	0.015	3954.400	
0.017	25.661	0.017	82.861	0.017	192.396	0.004	233.982	0.004	301.225	0.004	361.500	0.017	3354.632	0.017	3737.582	0.017	4108.400	
0.018	25.804	0.018	83.491	0.018	197.138	0.005	251.426	0.005	323.681	0.005	388.400	0.018	3479.932	0.018	3862.582	0.018	4233.000	
0.020	25.897	0.020	83.913	0.020	200.835	0.006	265.843	0.006	342.238	0.006	410.700	0.020	3584.132	0.020	3964.682	0.020	4333.200	
0.021	25.956	0.021	84.196	0.021	203.695	0.010	299.049	0.010	384.931	0.010	461.900	0.021	3670.276	0.021	4047.605	0.021	4413.400	
0.023	25.995	0.023	84.383	0.023	205.910	0.015	330.911	0.015	425.963	0.015	511.200	0.023	3741.264	0.023	4114.727	0.023	4477.300	
0.025	26.020	0.025	84.509	0.025	207.617	0.020	355.572	0.020	457.738	0.020	549.300	0.025	3799.408	0.025	4168.773	0.025	4528.000	

* Depth (z) is measured to be positive below the underside of abutment (Elevation 282 m).





golder.com