ADDENDUM
PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT
HIGHWAY 407 EAST EXTENSION – WESTERN SECTION
REGION OF DURHAM, ONTARIO
W.O. 07-20015

Submitted to:

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EXECUTIVE SUMMARY

The proposed Highway 407 East Extension extends from the current terminus of Highway 407 at Brock Road in the City of Pickering to Highway 35/115 in the Municipality of Clarington. For the purposes of preliminary design, the project route has been divided into three (3) sections (the Western Section, the Central Section and the Eastern Section) as shown on Drawing 1.

The planning component and preliminary design of foundations component for the proposed Highway 407 East Extension project were carried out in two (2) separate phases. A Phase I desktop study for this project was completed in 2008 for each section of the proposed highway extension for planning and feasibility study purposes by Thurber Engineering Ltd. (Thurber) and is presented in three (3) separate reports for each section titled “Foundation Desktop Study, Highway 407 East Extension - Western Section; Central Section; Eastern Section, W.O. 07-20015”, dated November 2008. The Phase I Desktop Study was based on assessment of site geology using air-photo interpretation and hydrogeologic information, as well as borehole data obtained from previous investigations including the preliminary investigation conducted by MTO in 1994 for planning purposes.

In 2010, Golder Associates Ltd. (Golder) prepared the Preliminary Foundation Investigation and Design Report (FIDR) with the results of the Phase II foundation investigation and recommendations for the planning (including environmental assessment) and preliminary design of the proposed Highway 407 East Extension - Western Section, which extends from Brock Road in the City of Pickering to Ashburn Road in the Town of Whitby, including the proposed West Durham Link to Highway 401 (as shown on Drawing 2). The purpose of Golder’s Phase II study was to provide “as near as possible” preliminary design level foundation investigation and design information for environmental assessment purposes given the constraints at the time of the investigation. The Golder preliminary FIDR superseded all previous reports including the Desktop Study for the purpose of preliminary foundation design and EA submission.

To supplement Golder’s report, Peto MacCallum Ltd. (PML) prepared a Preliminary Foundation Investigation and Design Report (FIDR) on the Western Section (reference No. 10TF023-W) that was issued in February 2011, Geocres No. 30M15-110.

This report is prepared by PML as an addendum to the above report and consists of two (2) parts:

Part A – Preliminary Foundation Investigation Report (FIR): presents an overall description of the project, description of the regional geology/geomorphology and general groundwater conditions within the project limits, as well as site-specific subsurface and groundwater conditions at each of the proposed highway bridge crossings and interchanges, based on the results of limited borehole investigation and laboratory testing carried out at bridge and culvert sites. Individual Preliminary Foundation Investigation Report sheets summarizing the results of the field investigation and geotechnical laboratory testing for each structure site are presented following the text of the report.

Part B – Preliminary Foundation Design Report (FDR): provides project-wide engineering recommendations for preliminary design for each proposed structure, culvert, deep cut and high fill site. Individual site-specific recommendations are provided on the Preliminary Foundation Design Report sheets presented following the text of this report and are appended to their respective Preliminary Foundation Investigation Report sheets (refer to Part A above).

Each highway crossing site (i.e. bridge, culvert, etc.) was characterized in the Request for Proposal (RFP) as requiring low, medium or high level investigative effort. The definitions of the target effort levels are defined in the RFP and summarized in Section 3.0 of this report.

This addendum report includes the results of the foundation investigations completed for bridges M-9, M-10, M-17, M-18, W-8 and culverts M-6, W-18.

For deep cut and high fill sections (depth/height greater than 4.5 m), summary tables have been included that summarize the deep cut and high fill locations, depths/heights, the anticipated subsurface conditions, and preliminary geotechnical recommendations. This report includes the results of the foundation investigations completed for deep cuts DC-W1, DC-W11 and high fills HF-W6, HF-W7.

While the information presented in this report may be used for planning and preliminary design purposes, it is not sufficient nor intended for detail design purposes. The preliminary subsurface investigation was limited to borehole drilling within accessible parts of sites where permission to enter was granted. Where drilling was carried out, the boreholes were not necessarily advanced at or within the footprint of the foundation elements. Accordingly, further investigation at the final locations of the foundation elements, approaches, deep cut and high fill sections will be required during detail design to establish or confirm/reassess the preliminary recommendations provided herein.

PART A

ADDENDUM
PRELIMINARY FOUNDATION INVESTIGATION REPORT
HIGHWAY 407 EAST EXTENSION – WESTERN SECTION
REGION OF DURHAM, ONTARIO
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1.0 INTRODUCTION

This report presents the factual findings obtained from a preliminary foundation investigation carried out by Peto MacCallum Ltd. (PML) on December 2 and 22, 2010, and in the period of February 7 to May 10, 2011, to supplement the preliminary investigations carried out by PML and by Golder Associates Ltd. (Golder) for the preliminary design of the proposed Highway 407 East Extension – Western Section (refer to Drawing 1). The project limits extend from the present terminus of the existing Highway 407 at Brock Road in the City of Pickering to Ashburn Road in the Town of Whitby (approximately 15 km), including the West Durham Link (WDL) extending southerly from the proposed Highway 407, just east of Halls Road North, to Highway 401 in the Region of Durham, Ontario (approximately 10 km) as shown on Drawing 2.

This addendum report provides sufficient information for planning and preliminary foundation investigation and design for a total of seven (7) structure sites of which five (5) sites are bridges and two (2) sites are culverts. In addition, two (2) deep cut areas and two (2) high fill areas were included in the study for the Western Section.

PML conducted the investigation as a sub-consultant to Delcan Corporation (Delcan) under the Ministry of Transportation, Ontario (MTO) Purchase Order No. 2009-E-0048. The terms of reference and scope of work for the preliminary foundation investigation and design are outlined in MTO's Request for Proposal (RFP) for Work Order No. 07-20015.

2.0 PROJECT DESCRIPTION

The technically recommended route for the proposed Highway 407 East Extension starts at the current terminus at Brock Road in the City of Pickering and ends at Highway 35/115 in Clarington. The route includes two north-south links connecting the proposed Highway 407 extension to Highway 401 – the West Durham Link (WDL) in Whitby and the East Durham Link (EDL) in Clarington. The proposed highway extension is divided into three main sections: a Western Section which extends from Brock Road to Ashburn Road and includes the WDL, a Central Section which extends from Ashburn Road to Courtice Road, and an Eastern Section which extends from Courtice Road to Highway 35/115 in Clarington and includes the EDL. Drawing 1 shows the proposed alignment for the above described overall route.

For a detailed description of the Western and Central Sections, including the total number of structures, deep cuts and high fills, refer to PML's Preliminary Foundation Investigation Report dated February 2011.

Structures were originally designated as 'WM' (West Mainline) or 'WL' (West Durham Link) with sequential numbers. However, for structures that were added to the project or modified after the destkop study was completed, an alternate designation (such as WM-EDC-9 for West Mainline East Duffins Creek) was used. New structure designations for the West Mainline (designated 'M-') and West Durham Link (designated 'W-') with a sequential numbering system were provided by URS in October 2009. The cross-referenced structure designations, categories, locations and site ranking complexities for the structure sites covered by this report are summarized in Section 4.2.

The configuration for the proposed WDL – Highway 401 interchange includes the re-alignment to the north of an approximately 5 km long section of the existing Highway 401, and the re-construction of the Lake Ridge Road bridge over Highway 401 and CN/GO Rail. It is understood that future plans involve the extension of the WDL north of the proposed Highway 407.

The proposed Highway 407 West Mainline and WDL routes run mainly through farmland, crossing a number of creek valleys, tributaries, as well as municipal and regional roads. Several wide low-lying valleys are present where the mainline crosses East Duffins Creek (east of Paddock Road) and where the mainline crosses several tributaries to Lynde Creek (between Coronation Road and Winchester Road). The WDL also crosses the CP rail line north of Rossland Road. The overall surface topography along the proposed routes is gently sloping downward to the east and to the south towards Lake Ontario, and is incised by various creeks and associated tributaries, such as Urfe Creek, Brougham Creek, Spring Creek, East Duffins Creek, Carruthers Creek and Lynde Creek. There are no identified wetland areas crossed by the West Mainline nor by the WDL, but wetlands are present at various distances from the proposed highway, such as the South of Claremont Wetland Complex and Brock Road Wetland Complex to the north of the West Mainline and the Heber Down Wetland Complex and Lynde Creek Coastal Wetland Complex to the east and south of the WDL, respectively.

3.0 INVESTIGATION PROCEDURES

The subsurface investigation for this addendum Preliminary FIDR was conducted by PML at or adjacent to the locations of the proposed sites in the period of December 2010 to May 2011 and involved a total of 18 boreholes (8 for bridge sites, 4 for culvert sites, 4 for deep cut sections and 2 for high fill sections) drilled to depths of 5.6 to 26.2 m. Selected borehole data from Golder's investigation has also been used for preparation of this report. The borehole locations are shown on Drawings 3 to 9 relative to the proposed preliminary bridge structure locations provided by URS Canada Inc.

The complexity of each site (i.e. target investigative effort level) was defined by Golder based on existing geological information, available borehole information from previous investigations and seventy-five (75) site photographs provided by URS. The corresponding number of boreholes required to be advanced at each bridge/interchange site were determined by the site complexity designation as specified in the RFP and as summarized below:

- Low complexity sites: no borehole investigation required;
- Medium complexity sites: two (2) boreholes required; one (1) at or as close as possible to each of the proposed abutment locations; and
- High complexity sites: four (4) boreholes required; two boreholes at or near the proposed bridge abutment locations and two (2) boreholes at the locations of the approaches.

The field investigations were carried out using truck-mounted and track-mounted drill rigs supplied and operated by DBW Drilling Ltd. The boreholes were advanced using solid and hollow stem augers or wash boring methods to competent strata and generally penetrated 3 m into '100-blow' materials or shale bedrock.

Soil samples were obtained at selected intervals using a split-spoon sampler in accordance with the Standard Penetration Test (SPT) procedures (ASTM D1586 Standard Test Method for Standard Penetration Test). In-situ vane tests using an MTO 'N'-size vane (ASTM D2573 Standard Test Method for Field Vane Shear Test) were carried out at selected depths where soft to stiff cohesive soils were encountered, and relatively undisturbed, 76 mm outer diameter thin-walled Shelby tube (ASTM D1587 Standard Practice for Thin-Walled Tube Sampling) samples of these materials were obtained at selected locations.

The groundwater conditions in the open boreholes were observed throughout the drilling operations, and whenever possible, one piezometer was installed in a selected borehole at each bridge site. A total of forty-seven (47) piezometers were installed by Golder and PML as part of the subsurface investigation for this project. The piezometers consist of 19 mm or 50 mm outside diameter rigid PVC pipe with a 1.5 m long screen that is surrounded by a sand pack and sealed at a selected depth within the boreholes. The annulus between the borehole wall and the piezometer pipe above the filter pack was backfilled to ground surface using bentonite pellets. All other boreholes were backfilled to ground surface using bentonite pellets on completion of drilling in accordance with Ontario Regulation 903 Wells (as amended by Ontario Regulation 372).

Where artesian groundwater conditions were encountered in the boreholes, the artesian condition was sealed at the source. Details of the artesian condition and the sealing operations are included on the Record of Borehole sheets, where applicable.

The field work for the current study was supervised on a full-time basis by members of PML's technical staff who located the boreholes in the field, arranged for the clearance of underground service locations, directed the drilling, sampling, and in situ testing operations, and logged the boreholes. The soil samples were identified in the field, placed in labelled containers and transported to PML's laboratory in Toronto for further examination and testing. Various combinations of index and classification tests consisting of water content determinations, Atterberg limits and grain size distribution analyses were carried out on selected soil samples.

PML established borehole locations in the field and J.D. Barnes Land Surveyors provided their co-ordinates and ground surface elevations at the boreholes. Golder measured the borehole locations on-site using a Trimble Pathfinder ProXH GPS unit with an accuracy of +/- 1 m. Because the GPS unit does not provide a suitable accuracy for ground surface elevation, the elevation of the ground surface at the borehole locations was subsequently determined based on the Digital Terrain Model and topographical mapping provided by URS. The borehole locations (MTM NAD83 northing and easting coordinates) and the ground surface elevations (in m, referenced to Geodetic datum) at the borehole locations are presented on the Record of Borehole sheets, provided in Appendix A.

4.0 SITE GEOLOGY AND STRATIGRAPHY

4.1 Regional Geology

The alignment of the proposed Highway 407 East Extension – Western Section, including the West Durham Link, is situated within the Regional Municipality of Durham which encompasses three major physiographic regions – the Oak Ridges Moraine, the South Slope and the Iroquois Plain, as delineated in *The Physiography of Southern Ontario*[1] and described below.

The Oak Ridges Moraine region: forms the northern boundary of the western section alignment, and is comprised predominantly of sand and gravel deposits. The Oak Ridges Moraine is a major regional aquifer and groundwater recharge area.

The South Slope region: the majority of the Highway 407 mainline section lies within the South Slope region and is comprised of calcareous clay till with lacustrine clay and silt reworked by glaciers, with numerous scattered drumlins and deep valley cuts caused by flowing streams towards Lake Ontario.

The Iroquois Plain region: encompasses the area of the proposed West Durham Link and extends south to Lake Ontario. The area across the Regional Municipality of Durham is a complex mix of till plains,

[1] Chapman, L.J. and Putnam, D.F. *The Physiography of Southern Ontario*, Ontario Geological Survey Special Volume 2, Third Edition, 1984. Accompanied by Map P.2715, Scale 1:600,00

drumlins and areas of glaciolacustrine sediments deposited in Lake Iroquois – primarily sands, silts and gravels.

The bedrock within the project area is described as being comprised of blue-grey shales of the Blue Mountain Formation and limestones of the Lindsay Formation. The bedrock in the area is described as providing a deep aquifer unit, where groundwater flow occurs through the bedding plane fractures.

4.2 Site-Specific Descriptions and Subsurface Conditions

The structure designation, structure category (i.e. overpass, underpass, culvert), location, designated site complexity/ranking (desired level of investigative effort), boreholes advanced at or adjacent to the site as part of the current and/or previous investigations, and current status of investigation for each structure are summarized below.

The structure locations and designations as provided by URS on February 20, 2009 are shown on Drawings 3 to 9. The bridge structures located along the proposed Highway 407 West Mainline and WDL are designated as 'WM' structures and 'WL' structures, respectively. The water crossing structures are designated with a revised structure number (i.e. Watershed Number). A new numbering system (designated 'M-' for the West Mainline and 'W-' for the West Durham Link) was provided by URS in October 2009 and is cross-referenced with the original and revised structure numbering systems as shown below.

It should be noted that all culvert sites were originally designated as low complexity sites in the Phase I study report. Thus, no borehole investigation was carried out at the culvert sites as part of the Phase II foundation investigation by Golder. Subsequent to completion of the field investigation at sites for which PTE had been granted, the structural designer indicated that the designation of some culverts in the Phase I study had been changed to medium complexity bridge structures (span length greater than 6 m with open footing foundations as agreed upon by the Foundations Team for the project). In addition, new culvert and bridge structures have been identified for the project that were not included in the Phase I study.

All of the preliminary foundation investigations have now been completed by Golder and PML. A summary of the soil and groundwater conditions encountered at the sites investigated during the current study, together with site-specific drawings showing the borehole locations and stratigraphic profile, are presented on individual preliminary FIR sheets following the text of this report. For the remaining sites, refer to the two reports titled *Preliminary Foundation Investigation and Design Report – Highway 407 East Extension – Western Section, W.O. 07-20015* prepared by Golder, dated June 2010, Geocres No. 30M14-316 and by PML dated February 2011, Geocres No. 30M15-110.

New Structure No.	Original Structure No. (Original Site Ranking)	Watercourse No.	Revised Category (Original Category)	Revised Location (Original Location)	Revised Site Ranking	Golder Borehole Nos.	PML Borehole Nos.	Remarks[6]
WEST MAINLINE STRUCTURES								
M-6	n/a	WM-TABC-101	Culvert	Realigned Hwy 7 over Brougham Creek tributary at Site #101	Medium	-	M6-1, M6-2	New Structure – Refer to FIDR sheet
M-9	WM-7 (Medium)	n/a	Overpass	Realigned Hwy 7	Medium	WM7-1, WM7-2, WM7-1A	M9-1	Refer to FIDR sheet
M-10	WM-5/6 (Medium)	WM-TBBC-7 (WM-SC-7)	Bridge	WBL and EBL over Spring Creek	Medium	WMSC7-1	M10-1	Refer to FIDR sheet
M-17	WM-18/19 (Medium)	WM-TACC-11	Bridge	EBL&WBL over Carruthers Creek West tributary	Medium	-	M17-1, M17-2	Refer to FIDR sheet
M-18	WM-20/21 (Medium)	WM-TBCC-12	Bridge	EBL&WBL over Carruthers Creek West tributary	Medium	-	M18-1, M18-2	Refer to FIDR sheet
WEST DURHAM LINK STRUCTURES								
W-8	n/a	WL-TALC-51D	Bridge	Lake Ridge Road/401 IC – NS-E Ramp over West Lynde Creek at Site#51	Medium	-	W8-1, W8-2	New Structure – Refer to FIDR sheet
W-18	n/a	CPR Culvert (Site #46A)	Culvert	CPR over existing watercourse east of WDL	Medium	-	W18-1, W18-2	New Structure – Refer to FIDR sheet

The subsurface soil and groundwater conditions as encountered in the boreholes advanced during the current and previous investigations, and the results of geotechnical laboratory tests carried out on selected soil and rock samples, are given on the Record of Borehole sheets included in Appendix A and on the laboratory test result figures included in Appendix B. A copy of the referenced borehole logs from previous MTO investigations located along the Highway 407 alignment in this section are provided in Appendix C and approximate locations (converted to MTM NAD83 co-ordinates) are shown on Drawings 3 to 9.

It should be noted that the stratigraphic boundaries shown on the Record of Borehole sheets are inferred from non-continuous sampling, observations of drilling progress and the results of Standard Penetration Tests (SPTs). These boundaries, therefore, represent transitions between soil types rather than exact planes of geological change. Subsurface conditions will vary between and beyond the borehole locations. It should also be noted that the water levels which were observed in the open boreholes or measured in the piezometers are expected to fluctuate seasonally and should be expected to rise during the spring and other wet periods of the year.

The sections included in this addendum report where the proposed highway is to be constructed in a deep cut or as a high fill are summarized below. The summary shows the deep cut area (designated 'DC-') or high fill area (designated 'HF-') number, location (station to station), maximum depth and height of the proposed cut or fill, and existing boreholes in the area. The subsurface conditions at the deep cut and high fill sections are summarized in the Preliminary Foundation Investigation Report sheets for Deep Cuts and High Fills following the FIDR sheets for the structures.

Deep Cut or High Fill Section[1]	Approximate Station Limits[2]		Length (m)	Approximate Maximum Depth of Cut[2]	Approximate Maximum Height of Fill[2]	Existing Borehole(s) in Area	PML Boreholes
West Mainline							
DC-W1	18+917	19+272	355	18.0	-	WM8-1, P6	DCW1-1, DCW1-2
HF-W6	21+842	21+967	125	-	6.0	-	HFW6-1
HF-W7	22+217	22+427	210	-	6.0	-	HFW7-1
West Durham Link							
DC-W11	12+400	12+890	490	7.0	-	WL-19A-1A, WL-19A-2A, WL19-2A, WL-19-3A	DCW11-1, DCW11-2

Notes:
1. Deep cuts / high fills are defined as areas which are deeper/higher than 4.5 m.
2. The extent and depth/height of deep cuts and high fills were estimated from base plans and profiles provided in digital format by URS, drawing file titled "407E Western Section Plan & Profile (Ver4.2).dwg", received November 6, 2008.

It should be noted that the subsurface conditions presented in these addendum Preliminary Foundation Investigation Report sheets for High Fills and Deep Cuts are inferred from limited borehole information. The subsurface conditions described are therefore approximate and may differ from the actual subsurface conditions that exist along the proposed deep cut and high fill sections.

4.3 General Groundwater Conditions

The water level was observed in open boreholes at the time of drilling, and standpipe piezometers were installed at a total of forty-seven (47) borehole locations as part of the current and previous investigations for the project. The remaining boreholes were backfilled immediately after the completion of drilling and before the local water level had stabilized.

Details of the four (4) piezometer installations and history of water levels measured in the boreholes drilled for the sites covered by this report are shown on the Record of Borehole sheets in Appendix A. We refer to the Preliminary FIDR prepared by PML dated February 2011, Geocres No. 30M15-110 for a list of the forty-three (43) previously installed piezometers and reference water level readings.

The most recent water levels measured in the piezometers are summarized below and represent the stabilized groundwater levels (except where noted). The water level(s) in open boreholes at completion of drilling are presented on the Record of Borehole sheets but are not considered stabilized and are in fact affected by water introduced during drilling operations, or depressed due to advancement of the boreholes.

PML PIEZOMETERS

Borehole Number / Piezometer	Ground Surface Elevation (m)	Depth to Water Level Below Ground Surface (m)	Water Level Elevation (m)	Date
W8-2	77.6	0.9	76.7	April 1, 2011
W18-1	100.2	1.7	98.5	April 1, 2011
DCW1-1	177.9	0.9	177.0	May 24, 2011
DCW11-1	102.7	2.2	100.5	April 1, 2011

The measured groundwater levels in the four new piezometers range from 0.9 to 2.2 m below ground surface. It should be noted that artesian water conditions were observed at three (3) borehole locations (M17-1, M17-2, M18-2). The boreholes which encountered artesian conditions are located within low-lying creek or valley areas, specifically near Carruthers Creek. The artesian water pressures were estimated to be 0.5 and 1.0 m above existing ground surface and were encountered within granular layers present about 8.2 to 13.7 m below ground surface. Details of the site-specific groundwater conditions at each bridge site are provided on the Preliminary Foundation Investigation (FIR) sheets, following the text of this report.

It should be noted that the groundwater levels at the site are anticipated to fluctuate as a result of seasonal variations in precipitation and runoff at the site.

5.0 CLOSURE

The Addendum Preliminary Foundation Investigation Report was prepared by Mr. Grigory Degil, P.Eng., Senior Foundation Engineer, and reviewed by Mr. Brian R. Gray, MEng, P.Eng., MTO Designated Principal Contact. Mr. Carlos M.P. Nascimento, P.Eng., Manager, MTO Foundation Services, conducted an independent review of the report.

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PART B

ADDENDUM
PRELIMINARY FOUNDATION DESIGN REPORT
HIGHWAY 407 EAST EXTENSION – WESTERN SECTION
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6.0 ENGINEERING RECOMMENDATIONS FOR PRELIMINARY DESIGN

6.1 General

This section of the report provides foundation design recommendations for the preliminary design of the proposed bridge structures along the Highway 407 East Extension-Western Section Mainline and West Durham Link (WDL) routes. The preliminary foundation design recommendations provided herein are based on interpretation of the factual data obtained from limited current borehole investigations and previous borehole data obtained by MTO, at or near the site of the proposed structures, but not necessarily at or within the footprint of the foundation elements. The interpretation and recommendations are intended to provide the designers with adequate information to assess the feasible foundation alternatives for the preliminary design of the proposed structure foundations. Where comments are made on construction they are provided in order to highlight those aspects which could affect the current preliminary design of the project, and for which special provisions or operational constraints could potentially be required.

6.2 Structure Foundation Recommendations

For the current investigation, thirty-two (32) structures consisting of twenty-two (22) bridges and ten (10) culverts were proposed for the crossing of the Highway 407 West Mainline and WDL at the locations of creeks, municipal or regional roads, railways and associated new ramps/bridges. This addendum report contains preliminary foundation recommendations for seven (7) structures consisting of five (5) bridges and two (2) culverts, including a description of the proposed bridge structure(s) configuration assumed at the time of preparation of this report, in the individual Preliminary Foundation Investigation and Design Report (FIDR) sheets following the text of this report. For the twenty-five (25) previously investigated structures, we refer to PML's Preliminary Foundation Design Report dated February 2011, Geocres 30M15-110.

It is noted that the current subsurface investigation is generally limited to drilling boreholes near the locations of the bridge abutments to obtain subsurface information representative of the general site. No boreholes were advanced specifically within the foundation footprint of the bridge abutments, potential pier locations, nor at the approach embankment locations for any medium or high complexity sites. The boreholes were advanced to obtain subsurface information representative of the general site. Therefore, further investigations at the final locations of the bridge abutments and piers are required during detail design to obtain subsurface information specific to the foundation locations and to confirm that the subsurface conditions and the geotechnical parameters and resistance values provided in this preliminary design phase are appropriate for the detail design of the foundations.

The foundation design for all highway structures must be carried out in accordance with the latest Canadian Highway Bridge Design Code (CHBDC) requirements. Design of railway grade separations must also be carried out in conformance with the local railway authority requirements and American Railway Engineering and Maintenance-of-Way Association (AREMA) manual.

The following subsections provide project-wide recommendations generally applicable to all bridge sites, including design assumptions and limitations associated with the recommendations provided in the Preliminary Foundation Design Report sheets.

6.2.1 Spread Footings

Preliminary foundation recommendations for spread footings on native undisturbed soil or on a compacted Granular 'A' pad 'perched' within the bridge approaches are provided where subsoil conditions are considered to be suitable for shallow foundations, as indicated on the individual Preliminary FIDR sheets for each bridge site.

For spread footings placed (or perched) within the approach embankments on a compacted Granular 'A' core, the geotechnical resistance values provided in the FDR sheets assume a minimum 2 m thick Granular 'A' pad placed below the base of the footing. The Granular 'A' pad should extend at least 1 m beyond the plan limits of the footing and be sloped no steeper than 1 Horizontal : 1 Vertical (1H:1V) in general accordance with MTO guidelines (see Figure 1). The Granular 'A' pad should be constructed in accordance with MTO Special Provision 105S10, Compaction.

Preliminary geotechnical resistance values for spread footings are provided for factored Ultimate Limit States (ULS) and at Serviceability Limit States (SLS) for 25 mm of settlement assuming a 3 m wide footing. These preliminary values are given under the assumption that the loads are applied perpendicular to the surface of the footings. Where the load is not applied perpendicular to the surface of the footing, inclination of the load should be taken into account in accordance with Section 6.7.4 of the *CHBDC* (2006) and its *Commentary*. The geotechnical resistance values will have to be re-evaluated and modified if necessary during detail design based on future additional subsurface investigation at the locations of the foundation elements.

Resistance to lateral forces/sliding resistance between the concrete footings and the subgrade should be calculated in accordance with Section 6.7.5 of the *CHBDC* (2006).

All footings should be provided with a minimum of 1.2 m of soil cover or equivalent thickness of insulation for frost protection (OPSD 3090.101, Foundation Frost Depths for Southern Ontario).

6.2.2 Steel H-Piles

Preliminary recommendations for steel H-piles, assuming an HP 310 x 110 pile section, are provided where considered practical for foundation design of abutments and piers as indicated on the individual Preliminary FIDR sheets for each bridge site. The factored geotechnical axial resistance at Ultimate Limit States (ULS) and the geotechnical axial reaction at Serviceability Limit States (SLS) for 25 mm of displacement for the steel H-pile foundations founded at the anticipated pile depth/pile tip elevation are provided, based on the subsurface conditions encountered in the boreholes, respective to each bridge site.

The factored ULS resistance and SLS reaction values provided will have to be re-evaluated and modified, if necessary, during detail design in consideration of the additional subsurface investigations at the locations of each bridge foundation element. The factored geotechnical axial resistance at ULS should then be verified in the field by the use of the Hiley formula (MTO Structural Standard Drawing SS103-11 Pile Driving Control) during the final stages of driving. The ultimate geotechnical axial resistance predicted from the Hiley formula should then be multiplied by a geotechnical resistance factor equal to 0.4 in accordance with Table 6.6.2.1 in the *CHBDC* (2006) to verify the factored ULS design value. Based on MTO experience with the Hiley formula in the Southern Ontario region, a resistance factor equal to 0.5 may be used for this project. For complex bridge sites, if warranted during the detail design stage, the ultimate load capacity and/or load-settlement behavior (serviceability) should be verified by full-scale pile load tests.

Pile installation should be in accordance with OPSS 903, Deep Foundations. The pile termination or set criteria will be dependent on the pile driving hammer type, helmet, selected pile size and length of pile.

The structural design of the piles should be based on full downdrag load where applicable and as indicated on the FDR sheets, unless measures to significantly reduce anticipated post-construction settlements are undertaken. In this case the downdrag loads can be eliminated. For preliminary design, downdrag is not considered to be a concern if the differential movement between the settlement of the soil and the compression of the pile at the pile-soil interface is less than 10 mm (NCHRP, 1997).

Resistance to lateral loading can be derived using vertical piles, with enhanced support offered by battered piles, if required. For vertical piles, the resistance to lateral loading will be derived solely from the soil in front of the piles, whereas battered piles derive lateral resistance from the soil in front of the piles as well as the horizontal component of the axial load present in the inclined pile. The resistance to lateral loading in front of the pile and pile group action for lateral loading if the pile spacing in the direction of loading is less than six to eight pile diameters, should be accounted for and assessed during the detail design phase of the project. For preliminary design, lateral resistance values at factored ULS and at SLS for a lateral displacement of 10 mm at the pile head for a single vertical steel H-pile embedded in typical soil profiles are provided in Table C6.4 of the *Commentary of the CHBDC* (2006).

All pile caps should be provided with a minimum of 1.2 m of soil cover or equivalent thickness of insulation for frost protection (OPSD 3090.101, Foundation Frost Depths for Southern Ontario).

The soils at many structure locations are very dense or hard glacial tills (SPT ‘N’-values exceeding 100 blows) at depths of less than 5 m from the ground surface. To provide an adequate length of pile at these locations, pre-augering may be required to penetrate the very dense or hard glacial till soils.

For the installation of steel H-piles, consideration will have to be given to the possible presence of cobbles and/or boulders within the till deposits and bedrock encountered at the locations of a number of bridge sites as indicated on the FIDR sheets. Where applicable, the piles should be reinforced with driving shoes such as Titus Standard “H” Bearing Pile Point design or flange plates as per OPSD 3000.100, Foundation Piles – Steel H-Pile Driving Shoe, for protection during driving. For piles to be driven into bedrock, the following note should also be included in the Contract Drawings: “Piles to be driven to bedrock”. Pile installation and driving shoes should be in accordance with OPSS 903, Deep Foundations.

Where artesian groundwater conditions are present, specialized construction techniques will be required to mitigate the possible upward flow of water along the pile shaft. Such measures may include driving the piles within a large diameter liner filled with water to counteract artesian head, and provision for an impermeable plug and granular drainage layer.

6.2.3 Caissons

Preliminary foundation recommendations for caissons founded within “100-blow” deposits or within shale bedrock as applicable, were provided where caissons were considered to be practical for foundation design as indicated on the individual Preliminary FDR sheets for each bridge site. The factored geotechnical axial resistance at Ultimate Limit States (ULS) and the geotechnical axial resistance at Serviceability Limit States (SLS) for 25 mm of displacement are provided for caisson diameters equal to 1.2 m and 1.5 m. The geotechnical resistance values are associated with a recommended caisson base elevation and/or embedment depth into the “100-blow” materials or into shale bedrock as the caisson will typically derive the majority of its capacity from base resistance, although shaft resistance has also been taken into account assuming permanent steel liners are required.

The factored ULS and SLS resistance values provided will have to be re-evaluated and modified, if necessary, during detail design in consideration of the additional subsurface investigations at the locations of each bridge foundation element. For complex bridge sites, if warranted during the detail design stage, the ultimate load capacity and/or load-settlement behavior (serviceability) should be verified by full-scale caisson load tests.

The structural design of the caissons should be based on full downdrag load where applicable and as indicated on the FDR sheets, unless measures to significantly reduce anticipated post-construction settlements are undertaken in which case the downdrag loads can be eliminated. For preliminary design, downdrag is not a concern if the differential movement between the settlement of the soil and the compression of the caisson at the caisson-soil interface is less than 10 mm (NCHRP, 1997).

The resistance to lateral loading developed by the soils in front of the caissons (assuming vertical caissons) and the reductions due to group effects should be accounted for and assessed during the detail design phase of the project.

It should be noted that "running" or "flowing" of water-bearing cohesionless strata, where encountered, could occur during or after drilling of caisson foundations. Therefore, where caisson foundations are considered, temporary or permanent caisson liners may be required to support these type of soils during construction and permit cleaning and inspection of the caisson base (possibly with a downhole camera). At some locations (as indicated on the FDR sheets), it is recommended caissons be drilled while maintaining a constant head of water inside the caisson liners to counterbalance high groundwater or artesian conditions followed by tremied concrete placement (see Section 6.7.3). Where the caissons are relatively long, temporary liners may be difficult to withdraw due to the length of the liners and the typically hard/very dense nature of the "100-blow" material in which the caissons are installed can result in "necking" of the caissons. In such cases, permanent liners would be preferred for the construction of the caissons and the reduced shaft resistance (i.e. due to the smooth liner/soil interface) has been considered in the preliminary geotechnical resistance values provided in the FDR sheets for the full length of the caissons. The use of permanent liners should be re-assessed and geotechnical resistance values revised, if necessary, when the caisson installation method has been determined during detail design.

Consideration will have to be given to the possible presence of cobbles and/or boulders within the till deposits encountered at the locations of a number of bridge sites as indicated in the FDR sheets. Caisson drilling equipment must be capable of penetrating such obstacles, where applicable (refer to Section 6.7.4).

Pile caps for caissons, as applicable, should be provided with a minimum of 1.2 m of soil cover or equivalent thickness of insulation for frost protection (OPSD 3090.101, Foundation Frost Depths for Southern Ontario), unless the caissons are extended above ground surface to the underside of the deck with a pile cap.

6.3 Bridge Retaining/Wing Walls

Most of the proposed bridge structures may require the construction of retaining walls and/or wing walls depending on the proposed bridge crossing configuration, available space and surrounding ground elevations. Feasible bridge retaining wall/wing wall options may include:

- Concrete retaining walls supported on spread footings or on deep foundations (often cantilevered beyond the abutment foundation) depending on the site-specific subsoil conditions as discussed on the respective Foundation Design Report sheets following the text of this report. The preliminary foundation recommendations for this type of retaining wall can be considered to be similar to the recommendations provided for the preliminary design of the bridge foundations elements.

- Retained Soil System (RSS) walls: RSS walls are considered to be the most feasible wall option for most of the bridge abutment / approach locations provided differential settlements are within tolerable limits and an adequate Factor of Safety against global instability is achieved. The performance of an RSS wall during foundation settlement depends primarily on the characteristics of its front facing system. Total settlements up to about 75 mm can be tolerated and a typical precast concrete panel facing can tolerate up to about 1 % differential settlement (RECo, 2000). Specialized slip joints can be incorporated into the design if differential settlements exceed 1 %. Sub-excavation of surficial soft/loose materials, where encountered, and replacing with compacted granular material, will be required to construct the reinforced soil mass. The front facing is typically supported on a strip footing placed at shallow depth below the ground surface. The footing must be founded on competent native soils or approved engineered fill, after sub-excavation and backfilling the areas where topsoil, loose/soft fill or unsuitable native soils exist. The factored geotechnical axial resistance at Ultimate Limit States (ULS) and the geotechnical axial resistance at Serviceability Limit States (SLS) for up to 75 mm of displacement should be provided for the footings of the wall facing and reinforced earth mass during detail design. It should be noted that the limiting displacement value for SLS design should be re-assessed and confirmed during detail design and will be dependent on the actual facing type or possibly the serviceability limit of the supporting roadway or foundation (typically less than 25 mm), if applicable. The internal stability of a reinforced earth wall should be assessed by the proprietary product supplier/designer. The external stability of the RSS wall has been provided in the FDR sheets, where indicated, and should be confirmed by the geotechnical consultant at the detail design stage taking into account the final geometry and configuration.

For settlement sensitive sites (i.e. where soft cohesive deposits were encountered), retaining walls will be affected by the post-construction settlement of the wall backfill materials, depending on the height/thickness of the backfill. The selection of the wall option for such sites will thus be dependent on the predicted settlement and should be assessed during detail design. Measures to reduce settlement could be achieved by incorporating site improvement techniques such as using light weight fill materials (i.e. slag or expanded polystyrene (EPS)), installing wick drains, preloading or surcharging, and staged construction as discussed in the individual FDR sheets, where applicable. The preferred settlement mitigation option is site specific and should be confirmed when additional soil information and project scheduling is known during detail design.

6.4 Lateral Earth Pressures for Design

The lateral earth pressures acting on the bridge abutment stems and any associated retaining walls/wing 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 magnitude of surcharge including construction loadings, on the freedom of lateral movement of the structure, as well as on the drainage conditions behind the walls.

The following general recommendations are made concerning the design of the stems/walls. It should be noted that these recommendations and parameters assume level backfill and ground surface behind the walls. Where there is sloping ground behind the walls, the coefficient of lateral earth pressure must be adjusted to account for the slope in accordance with Section C6.9.2.2 of the *CHBDC* (2006).

- Select free-draining granular material meeting the specifications of MTO's Special Provision 110S13 Material Specifications for Aggregates, Granular 'A' or Granular 'B' Type II but with less than 5 per cent passing the 200 sieve should be used as backfill behind the walls. This material should be compacted in accordance with MTO's Special Provision 105S10, Compaction. Transverse drains and weep holes should be installed to provide positive drainage of the granular backfill. Other aspects of the granular backfill requirements with respect to sub-drains and frost taper should be in accordance with OPSD 3101.150, Walls Abutment, Backfill Minimum Granular Requirement and OPSD 3121.150, Walls Retaining, Backfill Minimum Granular Requirement.

- A minimum compaction surcharge of 12 kPa should be included in the lateral earth pressures for the structural design of the wall stem, in accordance with Section 6.9.3 and Figure 6.6 of the *CHBDC* (2006). Compaction equipment should be used in accordance with MTO's Special Provision 105S10, Compaction. Other surcharge loadings should be accounted for in the design, as required.

- The granular fill may be placed either in a zone with width equal to at least 1.2 m behind the back of the wall stem (Case I in Figure C6.20(a) of the *Commentary to the CHBDC* (2006)) or within the wedge-shaped zone defined by a line drawn at 1.5 horizontal to 1 vertical (1.5H:1V) extending up and back from the rear face of the footing (Case II in Figure C6.20(b) of the *Commentary to the CHBDC* (2006)).

- For the case where the pressures are based on granular fill behind the wall, the following parameters may be assumed.

	GRANULAR 'A'	**GRANULAR 'B' TYPE II**
Soil Unit Weight:	22 kN/m^3	21 kN/m^3
Coefficients of Static Lateral Earth Pressure:		
Active, K_a	0.27	0.27
At Rest, K_o	0.43	0.43

- For the case where the pressures are based on existing materials behind the wall, the required parameters for design should be assessed on a site-by site basis during detail design.

- If the wall support and superstructure allow lateral yielding of the abutment stem and retaining walls, active earth pressures may be used in the geotechnical design of the structure. If the abutment support does not allow lateral yielding, at-rest earth pressures should be assumed for geotechnical design. The movement to allow active pressures to develop within the backfill, and thereby assume an unrestrained structure, may be taken as presented in Section C6.9 and Table C6.6 of the *Commentary to the CHBDC* (2006).

6.5 Structure Approaches

The configuration of the structure approaches varies from site to site and includes approach embankment construction with fills and/or cuts depending on the design grades and ground elevations for each bridge crossing. Based on the available information provided at each bridge site, recommendations associated with the approaches stability and settlement are provided on the individual Preliminary FDR sheets following the text of this report. The following subsections provide additional project-wide recommendations associated with the preliminary design and construction of the bridge approaches.

6.5.1 Subgrade Preparation and Embankment Construction

For all proposed bridge sites, it is recommended that all topsoil and organic material be stripped from the proposed embankment footprint. The depth and extent of stripped material should be determined during detail design when additional subsurface information is available. Particular attention will be required in low valley areas where thicker layers of organic/alluvial soils may be present.

After stripping of organics, the exposed subgrade should be proof rolled to identify any loose/softened areas requiring sub-excavation or additional compaction prior to fill placement.

Embankment fill should be placed and compacted in accordance with MTO's Special Provision 206S03, Earth Excavation, Grading and Special Provision 105S10, Construction Specification for Compaction. In the case of approach cuts with a shallow water table condition, it is expected that measures will need to be undertaken to stabilize the embankment slope face due to possible groundwater seepage (refer to Section 8.0 on Deep Cuts and High Fills).

In the case of bridge/embankment widening, in order to minimize differential settlement between the widened portions of the approach embankments due to settlement of the fill itself, the use of granular fill is preferred over the use of cohesive fill, since the majority of settlement of granular fills will occur during construction whereas some settlement of cohesive fills, if used, would occur post-construction. The new embankment fill should be benched into the existing embankment in accordance with OPSD 208.010, Benching of Earth Slopes.

To reduce erosion of the embankment side slopes due to surface water runoff, placement of topsoil and seeding or pegged sod is recommended as soon as practicable after construction of the embankments. The erosion protection must be in accordance with OPSS 572, Seed and Cover.

6.5.2 Approach Embankment Stability

The preliminary assessment for the stability of the approaches at each bridge site was calculated based on limit equilibrium analyses using the commercially available slope stability program SLOPE/W developed by Geo-Slope International Ltd. (Bishop's modified method of slices was employed) and is provided on the respective Preliminary Foundation Design Report sheets for each proposed bridge/interchange site. The analyses assume approach cut/embankment side slopes no steeper than 2H:1V associated with a maximum approach height as indicated on the Preliminary General Arrangement drawings provided at the time of this report (including a minimum 2 m wide bench at mid-height for embankment heights greater than 8 m). Where designated as safe or adequate against deep-seated slope instability, a target Factor of Safety of 1.3 under static conditions is implied, assuming appropriate subgrade preparation and proper placement and compaction of embankment fill materials. Assessment of the overall stability of the embankment side slopes under seismic conditions is discussed in Section 6.6.

Approaches higher than 8 m, where deemed feasible, should be constructed with a 2 m wide mid-height bench in order to control surficial erosion and to improve stability.

The preliminary assessment of stability of the approach slopes should be reviewed and confirmed based on the actual subsoil conditions encountered within the proposed approach/embankment footprint during detail design. Mitigation measures to improve slope stability for greater embankment heights can be achieved by utilizing light weight fill materials, wick drains, and staged construction, or a combination of these options, which will also help to reduce settlements.

6.5.3 Approach Embankment Settlement

Settlement of the approach embankments will occur at bridge sites due to compression of the embankment fill itself, as well as compression and consolidation of the foundation soils. The total settlement within the founding soils has been estimated based on the existing site-specific subsoil conditions for preliminary design using elastic analysis and Terzaghi one-dimensional consolidation theory, with the results reported on the individual Preliminary Foundation Design Report sheets for each bridge/interchange site. These preliminary estimates do not include compression of the fill itself, which would occur during and after the construction of embankment depending on the type of materials used. The magnitude of fill compression usually ranges from 1% to 2% of the height of embankment. In the case where granular fill is used for embankment construction, settlement of the fill itself is expected to occur during or shortly after completion of embankment construction whereas non-granular earth fill or rock fill materials will exhibit additional consolidation settlement over time.

Embankment and platform width design should allow for the anticipated settlements and future padding of the pavement structure.

Where estimated post-construction (i.e. consolidation) settlement within the foundation soils exceeds acceptable limits, measures to reduce such settlement to acceptable values have been proposed. For preliminary design, acceptable settlement values are assumed to be less than 25 mm at or near structure locations. However, the highway design criteria will be site specific for each site and will likely be based on maintenance considerations at the detail design stage. Comprehensive analyses should be carried out during detail design to further estimate the anticipated amount and time rate of post-construction settlements and to develop the final design and construction requirements of the approach embankments in such site conditions, as well as develop mitigation measures to reduce anticipated settlements to acceptable levels.

6.6 Seismic Considerations

The peak zonal acceleration ratio for the project site is 0.05 g for The Town of Whitby/Ajax, Ontario, (*CHBDC* Table A3.1.1). The Site Coefficient, S, will be based on the type of soils encountered at the founding level at each site (to be determined during detailed design) in accordance with Section 4.4.6 and Table 4.4 of the *CHBDC* (2006).

Abutment Stem and Retaining Wall/Wing Wall design: seismic (earthquake) loading must be considered in the design of the foundations in accordance with Sections 4 and 6 of *CHBDC* (2006) as significant seismic loading will result, for example, in increased lateral earth pressures acting on the abutment stem and retaining walls. The walls should be designed to withstand the combined lateral loading for the appropriate static pressure conditions plus the applicable earthquake-induced dynamic earth pressure conditions (see Section 24.9 of CFEM (2006)). The static and seismic active earth pressure coefficients can be determined in accordance with Sections 6.9 and 4.6.4 of the *CHBDC* (2006) and its Commentary.

Approach Embankment design: liquefaction susceptibility of the soil deposits underlying the proposed embankments (and foundations) and the consequent stability of the embankments under seismic loading conditions should be assessed during the detail design stage in accordance with Section C.4.6.2 and C.4.6.3, respectively, of the *Commentary of the CHBDC* (2006).

6.7 Construction Considerations

6.7.1 Excavation and Backfill

Preliminary recommendations for open-cut excavations are provided on a site-specific basis on the Preliminary Foundation Design sheets for each bridge site and include the type of soils anticipated to be within the foundation excavations according to the Occupational Health and Safety Act (OHSA), as well as the recommended maximum side slope inclination for temporary excavations. All backfill is to be placed and compacted in accordance with MTO's Special Provision 105S10, Compaction.

6.7.2 Protection Systems

Excavation support systems may be required at the proposed bridge sites for temporary roadway protection. Where required, the temporary excavation support system should be designed and constructed in accordance with OPSS 539, Temporary Protection Systems. In general, the lateral movement of the temporary shoring system should meet Performance Level 2 as specified in OPSS 539. Performance Level 1 may be required adjacent to railways.

6.7.3 Groundwater and Surface Water Control

Anticipated groundwater levels within the foundation excavations at each proposed bridge site and anticipated groundwater and surface water control measures are reported on the individual Preliminary Foundation Design Report sheets. Groundwater levels were typically measured at ground surface down to a depth of about 5 m below ground surface. However, artesian conditions were recorded at some sites.

At locations where near surface granular (cohesionless) soils are present with a high water table, groundwater infiltration should be anticipated during excavation in such deposits, particularly during wet periods of the year. Dewatering at these sites will be required to allow for construction of foundation elements in a dry condition. For footing or pile cap construction in floodplains with a high groundwater table, no excavation should be undertaken without prior dewatering. Alternatively, the excavation should be carried out within the confines of a properly designed sheet pile cofferdam. For these sites, a Non-Standard Special Provision (NSSP) will be required for inclusion in the Contract Documents during detail design.

Caissons constructed with temporary or permanent liners in granular subsoils subjected to unbalanced hydrostatic head will require special measures to prevent 'boiling' or basal heave of the base materials. If caisson foundations are adopted for a site, it is recommended that a constant head of water be maintained inside the caisson liners to counterbalance the natural groundwater or artesian conditions. Concrete placement by tremie methods may be considered. For deep foundations at locations where artesian conditions are expected within the lower granular deposits, it is recommended that a sand filter, possibly in combination with a geotextile, be placed beneath the pile caps to prevent the migration of fines that may be transported along the piles or caisson liner during and after construction. Preliminary recommendations for such conditions (where considered practical) are given on the site-specific Preliminary Foundation Design report sheets and these aspects should be re-assessed during detail design.

General site drainage should be by gravity towards an outlet at a lower elevation and/or pumping.

The need for a Permit to Take Water (PTTW) should be assessed at each specific site during detail design.

6.7.4 Obstructions During Pile Driving / Caisson Installation

Till deposits have been encountered at a number of bridge sites along the proposed Highway 407 East Extension-Western Section route. It is anticipated that cobbles and/or boulders will be encountered within the till deposits, as noted in several boreholes, and may affect the installation of steel H-piles or drilled caissons. As such, an NSSP will need to be included in the Contract Documents during the detail design to identify to the contractor the possible presence of cobbles and/or boulders within the overburden soils on a site-by-site basis. Preliminary recommendations regarding potential obstructions during pile driving and caisson installation have been provided on the site-specific Preliminary Foundation Design Report sheets. An estimate of the range in size and quantity of cobbles / boulders for applicable sites should be incorporated into the detail design when additional borehole information is obtained.

6.7.5 Construction Access

Several creek valley crossings (i.e. environmentally sensitive areas) have been identified during the environmental assessment of the project. Potential environmental impacts will need to be minimized during construction access in the sensitive valleys such as near Lynde Creek and East Duffins Creek. Specific access preparation procedures such as the use of temporary work bridges, winter construction and/or gravel roadways underlain by geosynthetics should be considered to accommodate foundation construction at these locations.

7.0 CULVERTS

As noted in the previous sections of this report, culvert sites along the proposed route for the Highway 407 East Extension - Western Section Mainline and WDL were ranked as “low complexity” sites during the Phase I desktop study for this project. As such, no site specific borehole investigations were carried out by Golder at the proposed culvert sites during the Phase II foundation investigation for the planning and preliminary design. Anticipated Foundation Conditions (AFC) sheets were prepared by Thurber Engineering for each culvert site and were included in the Phase I desktop study. Copies of the Anticipated Foundation Conditions sheets for culvert sites were included in Golder’s report.

During the Phase II study, the project team identified new water crossing locations and many of the water crossings (i.e. culverts) identified during the Phase I study required larger span lengths to satisfy hydrology / geomorphology requirements. Based on the recent General Arrangement drawings provided by the structural designer, many of the culverts now require single span structures (longer than 6 m) with open footings and have been re-classified as ‘medium complexity’ investigative effort sites.

A list of all culvert structures is provided in Section 4.2 and the locations are shown on Drawings 3 to 9. Appropriate level site investigations to establish and/or confirm subsoil and groundwater conditions and design assumptions will be required during detail design for all culvert sites.

8.0 DEEP CUTS AND HIGH FILLS

Deep cut and high fill areas have been identified along the Highway 407 East Extension – Western Section Mainline alignment.

8.1 General

This section of the report provides geotechnical recommendations for preliminary design of deep cuts and high fill sections where the depth/height exceeds 4.5 m. Based on the roadway profiles available at the time of the assessment (January 2009), deep cuts have been identified at eleven (11) locations and high fills have been identified at fifteen (15) locations. The location, extent and depth/height of the four (4) deep cut/high fill areas included in this addendum report are summarized in Section 4.2. The maximum depth of cut is in the order of 18 m (DC-W1) and the maximum fill height is about 6 m (both HF-W6 and HF-W7).

The preliminary design recommendations provided herein are based on interpretation of the factual data obtained during limited borehole investigations conducted in the cut/fill sections as well as existing information obtained from previous investigations near the sites.

The anticipated subsurface conditions at the deep cut / high fill locations and preliminary recommendations for design are summarized on the “Preliminary Foundation Investigation Report - Deep Cuts” sheets and “Preliminary Foundation Investigation Report – High Fills” sheets presented following the FIDR sheets for the structures at the end of the text of this report.

The interpretation and recommendations are intended to provide the designers with preliminary information to assess design slope inclination, drainage requirements, and mitigation options for addressing potential stability or settlement issues. Where provided, comments regarding construction are presented to highlight aspects which could affect the preliminary design, and for which special provisions or operational constraints could potentially be required.

Geotechnical investigations will be required during detail design to confirm the subsurface conditions that were assumed throughout the cut/fill sections and confirm/re-assess the preliminary design recommendations.

8.2 Deep Cuts

8.2.1 Stability and Drainage

Preliminary assessment of the stability of the cut slopes was carried out at a typical cut section based on limit equilibrium analysis using the commercially available slope stability program SLOPE/W developed by Geo-Slope International Ltd. Bishop’s modified method of slices was employed. Cut slopes no steeper than 2H:1V, with a minimum 2 m wide mid-slope bench for cut depths greater than 8 m, were assumed.

For preliminary design, the target factors of safety were assumed to be 1.3 for short term stability, and 1.3 and 1.5 for long term stability in cohesionless and cohesive soils, respectively.

For cut slopes deeper than 8 m, the minimum requirement is to provide a 2 m wide mid-height bench in order to control surficial erosion and improve stability. Earth cut slopes must be provided with erosion protection in accordance with OPSS 572, Seed and Cover.

Permanent drainage of the cut slope is required. Roadside ditches are expected to provide an adequate level of permanent drainage in most areas. An interceptor ditch should be provided at the top of the cut as per OPSD 200.020 Earth/Shale Grading – Rural Divided.

Where cut excavation extends below the measured groundwater levels in cohesionless soils, more positive measures to provide permanent slope drainage and mitigate surficial instability may be required. Measures may include provision of subdrains positioned along the toe of slope and/or along the rear of the mid-slope bench, as well as gravel sheeting or rip-rap lined channels down the slope.

Seepage and surficial instability may also be experienced from localized permeable zones/sand layers within the less permeable soils. Determination of the frequency, extent and locations of the seepage zones from the limited borehole data is not possible. Therefore, consideration should be given to the observational approach involving examination of the cut slopes during and following construction to identify any areas of surficial instability, and provide mitigative measures such as a gravel sheeting or subdrains where required. All subdrains should be sloped on a positive grade to an outlet or pumping chamber.

The preliminary assessment of stability and drainage of the cut slopes should be reviewed and confirmed during the detail design investigation based on the subsoil conditions encountered in additional boreholes drilled within the cut sections.

8.2.2 Construction Considerations

Excavation for cut slope construction should be carried out in accordance with OPSS 206 as amended by MTO's most recent Special Provision 206S03, Earth Excavation, Grading.

The soil deposits in many of the cut sections, and notably till deposits, will typically be very dense/hard and often contain cobbles and boulders. Excavation in these deposits may be arduous and will require use of heavy duty excavators or dozers. The contract documents should include a NSSP to emphasize these conditions to the contractor. Selection of the method of excavation must remain the responsibility of the contractor, however, and be based on their equipment, experience and interpretation of the site conditions.

Temporary drainage of the cuts should be provided to maintain a relatively dry, stable excavation. Measures may include temporary drainage ditches or gravel sheeting to maintain surficial stability before permanent drainage measures are in effect and should be implemented in accordance with OPSS 577, Temporary Erosion and Sediment Control Measures.

8.3 High Fills

8.3.1 Slope Stability

Preliminary assessment of the stability of the fill embankment slopes was carried out for a typical high fill embankment based on limit equilibrium analysis using the commercially available slope stability program SLOPE/W developed by Geo-Slope International Ltd. Bishop's modified method of slices was employed. Embankment slopes no steeper than 2H:1V, with a minimum 2 m wide mid-slope bench for embankment heights greater than 8 m, were assumed.

For preliminary design, the target factors of safety were assumed to be 1.3 for short term stability, and 1.3 and 1.5 for long term stability of embankments founded on cohesionless and cohesive soils, respectively.

For embankment slopes higher than 8 m, the minimum requirement is to provide a 2 m wide mid-height bench in order to control surficial erosion and improve stability. Earth fill slopes must be provided with erosion protection in accordance with OPSS 572, Seed and Cover.

Assessment of the stability of the embankment side slopes under seismic conditions should be carried out during detail design.

The preliminary assessment of stability of the embankment slopes should be reviewed and confirmed based on the actual subsoil conditions encountered within the proposed embankment footprint during the detail design investigation. Mitigation measures to improve slope stability if required may include slope flattening, utilizing light weight fill materials, staged construction, or a combination of these options.

8.3.2 Settlement

Settlement of the fill embankments will occur due to compression and consolidation of the foundation soils under the weight of the overlying fill material as well as from compression of the embankment fill itself. The total settlement within the founding soils has been estimated using elastic analysis and Terzaghi one-dimensional consolidation theory, based on the subsoil conditions deduced from the existing borehole data and the maximum embankment heights indicated by profile and general arrangement drawings available at the time of the analysis.

Where the estimated embankment settlement exceeds 25 mm, the computed value is indicated on the Preliminary Foundation Investigation Report sheet for the particular section. The settlement tolerance for embankments may range from 25 mm to 100 mm depending on the distance from a structure. The highway design criteria will be site specific and based on maintenance considerations at the detail design stage.

The preliminary estimates do not include compression of the embankment fill itself, which would occur during and after the construction of embankment depending on the type of materials used. The magnitude of fill compression usually ranges from 1% to 2% of the height of embankment. Where granular fill is used for embankment construction, settlement of the fill itself is expected to occur during or shortly after completion of embankment construction. Non-granular earth fill or rock fill materials may exhibit additional consolidation settlement over time.

Embankment and platform width design should allow for the anticipated settlements and future padding of the pavement structure.

Further analyses should be carried out during detail design to confirm the anticipated magnitude of settlement, assess the time rate of post-construction settlement, and where required develop mitigation measures such as preloading, surcharging, wick drains or light weight fill to reduce anticipated settlements to acceptable levels.

8.3.3 Construction Considerations

It is recommended that all topsoil and organic material be stripped from the proposed embankment footprint. The depth and extent of stripped material shall be determined during detail design when additional subsurface information is available. Particular attention will be required in low valley areas where thicker layers of organic/alluvial soils may be present.

After stripping of organics, the exposed subgrade should be proof rolled to identify any loose/softened areas requiring sub-excavation or additional compaction prior to fill placement.

Embankment fill should be placed and compacted in accordance with MTO's Special Provision 206S03, Earth Excavation, Grading and Special Provision 105S10, Compaction. New embankment fill placed against existing embankment slopes or on a sloping ground surface should be benched into the existing slope in accordance with OPSD 208.010, Benching of Earth Slopes.

Trafficability of construction equipment may be problematic in low floodplain areas where soft/loose and organic alluvial material may be encountered and where environmental constraints may be imposed on site access. Further, drainage in these areas is likely to be poor, with groundwater levels varying subject to seasonal fluctuations. The contractor must be prepared to supply equipment capable of working on this terrain and/or provide alternative measures to improve trafficability such as placement of granular pads in working areas.

Potential environmental impacts will need to be minimized during construction access into sensitive floodplain or valley areas. Specific access preparation procedures such as the use of temporary work bridges, winter construction and/or gravel roadways underlain by geosynthetics should be considered. Further, sediment control measures such as silt fences, straw bales and/or granular check-dams will need to be installed downgradient of the works to reduce sediments impacts to surface water bodies, consistent with OPSS 577, Temporary Erosion and Sediment Control Measures.

9.0 CLOSURE

The Addendum Preliminary Foundation Design Report was prepared by Mr. Grigory Degil, P.Eng., Senior Foundation Engineer, and reviewed by Mr. Brian R. Gray, MEng, P.Eng., MTO Designated Principal Contact. Mr. Carlos M.P. Nascimento, P.Eng., Manager, MTO Foundation Services, conducted an independent review of the report.

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STANDARDS:

ASTM International:

ASTM D1586	Standard Test Method for Standard Penetration Test (SPT) and Split-Barrel Sampling of Soils
ASTM D1587	Standard Practice for Thin-Walled Tube Sampling of Soils for Geotechnical Purposes
ASTM D2573	Standard Test Method for Field Vane Shear Test in Cohesive Soil

Contract Design Estimating and Documentation (CDED):

Special Provision 105S10	Amendment to OPSS 501 – Construction Specification for Compaction. November 2004.
Special Provision 206S03	Amendment to OPSS 206 – Earth Excavation, Grading. July 2007.
Special Provision 110S13	Amendment to OPSS 1010 – Material Specification for Aggregates. August 2007.

Ontario Occupational Health and Safety Act:

Ontario Regulation 213/91	Construction Projects
Ontario Regulation 443/09	Amendment to Ontario Regulation 213

Ontario Provincial Standard Drawing:

OPSD 200.020	Earth/Shale Grading – Divided Rural. November 2009.
OPSD 208.010	Benching of Earth Slopes. November 2008.
OPSD 3000.100	Foundation Piles – Steel H-Pile Driving Shoe. November 2005.
OPSD 3090.101	Foundation Frost Depths for Southern Ontario. November 2005.
OPSD 3101.150	Walls – Abutment, Backfill Minimum Granular Requirement. November 2005.
OPSD 3121.150	Walls – Retaining, Backfill Minimum Granular Requirement. November 2005.

Ontario Provincial Standard Specification:

OPSS 539	Construction Specification for Temporary Protection Systems. November 2009.
OPSS 572	Construction Specification for Seed and Cover. November 2003.
OPSS 577	Construction Specification for Temporary Erosion And Sediment Control Measures. November 2006.
OPSS 903	Construction Specification for Deep Foundations. November 2009.

Ontario Water Resources Act:

Ontario Regulation 372/07	Amendment to Ontario Regulation 903
Ontario Regulation 903/90	Wells

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.

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
r_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_t	1	SENSITIVITY $= \frac{c_u}{\tau_r}$

PHYSICAL PROPERTIES OF SOIL

ρ_s	kg/m^3	DENSITY OF SOLID PARTICLES
γ_s	kN/m^3	UNIT WEIGHT OF SOLID PARTICLES
ρ_w	kg/m^3	DENSITY OF WATER
γ_w	kN/m^3	UNIT WEIGHT OF WATER
ρ	kg/m^3	DENSITY OF SOIL
γ	kN/m^3	UNIT WEIGHT OF SOIL
ρ_d	kg/m^3	DENSITY OF DRY SOIL
γ_d	kN/m^3	UNIT WEIGHT OF DRY SOIL
ρ_{sat}	kg/m^3	DENSITY OF SATURATED SOIL
γ_{sat}	kN/m^3	UNIT WEIGHT OF SATURATED SOIL
ρ'	kg/m^3	DENSITY OF SUBMERGED SOIL
γ'	kN/m^3	UNIT WEIGHT OF SUBMERGED SOIL
e	1, %	VOID RATIO

n	1, %	POROSITY
w	1, %	WATER CONTENT
S_r	%	DEGREE OF SATURATION
w_L	%	LIQUID LIMIT
w_P	%	PLASTIC LIMIT
w_S	%	SHRINKAGE LIMIT
I_P	%	PLASTICITY INDEX $= w_L - w_P$
I_L	1	LIQUIDITY INDEX $= \frac{w - w_P}{I_P}$
I_C	1	CONSISTENCY INDEX $= \frac{w_L - w}{I_P}$
DTPL		DRIER THAN PLASTIC LIMIT
APL		ABOUT PLASTIC LIMIT
WTPL		WETTER THAN PLASTIC LIMIT

e_{max}	1, %	VOID RATIO IN LOOSEST STATE
e_{min}	1, %	VOID RATIO IN DENSEST STATE
I_D	1	DENSITY INDEX $= \frac{e_{max} - e}{e_{max} - e_{min}}$
D	mm	GRAIN DIAMETER
D_n	mm	n PERCENT - DIAMETER
C_u	1	UNIFORMITY COEFFICIENT
h	m	HYDRAULIC HEAD OR POTENTIAL
q	m^3/s	RATE OF DISCHARGE
v	m/s	DISCHARGE VELOCITY
i	1	HYDRAULIC GRADIENT
k	m/s	HYDRAULIC CONDUCTIVITY
j	kN/m^3	SEEPAGE FORCE

PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT (FIDR) SHEETS – STRUCTURES

PART A - PRELIMINARY FOUNDATION INVESTIGATION REPORT
HWY 407 EAST EXTENSION – WESTERN SECTION
W.O. 07 – 20015

Structure Description: Culvert for the realigned Highway 7 over a Brougham Creek tributary

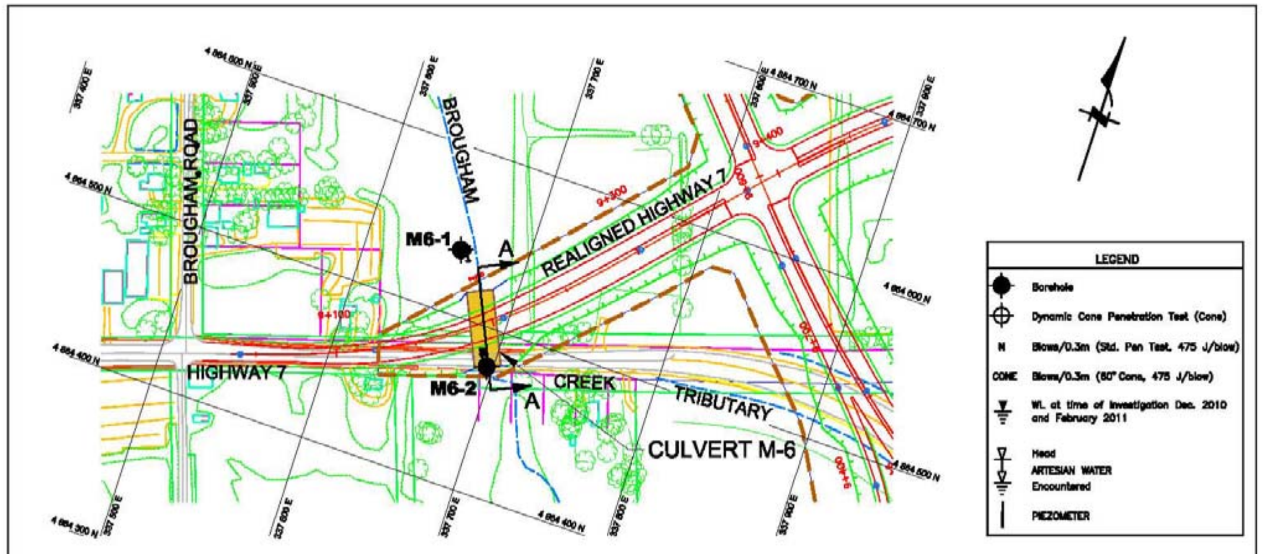
Location No: M-6 (WM-TABC-101)

Realigned Highway 7 Proposed Grade: 195.6 m

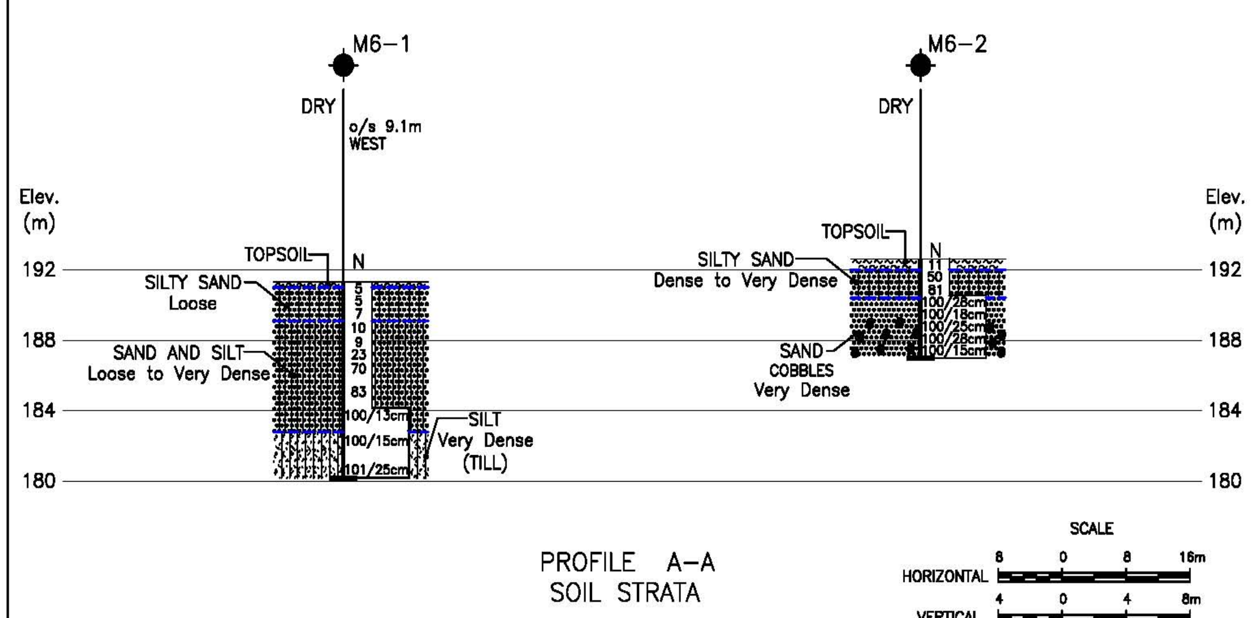
Existing Ground Elevation: 191.3 m – 192.6 m

Site Ranking: Medium

Station: 9+194



STRUCTURE AND BOREHOLE LOCATION PLAN



PROFILE A–A
SOIL STRATA

FOUNDATION INVESTIGATIONS

Site Description:

The site of the proposed culvert M-6 for the realigned Highway 7 over a Brougham Creek tributary is located some 450 m west of the proposed Highway 407 / realigned Brock Road interchange in the City of Pickering, Ontario. There is a culvert under existing Highway 7 at the proposed culvert location. The site is surrounded by cultivated farmland and densely treed areas. The overall topography of the terrain is sloping down towards the southeast.

Borehole Information:

Borehole No	Borehole Location	MTM NAD 83 – Northing	MTM NAD 83 – Easting	Borehole Elevation (m)	Borehole Depth (m)
M6-1	North End (Inlet)	4 864 539.0	336 658.1	191.3	11.1
M6-2	South End (Outlet)	4 864 477.2	336 696.0	192.6	5.6

Subsurface Conditions:

- **Topsoil:** surficial topsoil was present in both boreholes. The silty topsoil had a thickness of 300 mm in borehole M6-1, 600 mm in borehole M6-2 and was penetrated at respective Elevations 191.0 and 192.0 m.
- **Silty Sand:** directly beneath the topsoil at 0.3 m depth (Elev. 191.0 m) in borehole M6-1 and a depth of 0.6 m (Elev. 192.0 m) in borehole M6-2 was silty sand. Containing organic inclusions, this unit was 1.9 m in thickness and loose in relative density (SPT-'N' values of 5 to 7) in the former borehole. In the latter, it was 1.6 m thick and dense to very dense (SPT-'N' values of 50 and 81). The moisture content of the silty sand ranged from 8 to 12 percent, locally reaching 23 percent. The unit was penetrated at 2.2 m depth (Elev. 189.1 and 190.4 m) in both boreholes. The results of grain size distribution analysis performed on a sample of the unit are presented in Figure M6-GS-1 (Appendix B).
- **Sand:** overlain by the silty sand at a depth of 2.2 m (Elev. 190.4 m) in borehole M6-2 was cohesionless sand (with cobbles identified at Elevation 189.3 m). The sand was very dense (SPT-'N' values in excess of 100), had a moisture content of 3 to 5 percent and extended to the borehole termination depth of 5.6 m (Elev. 187.0 m).
- **Sand and Silt:** underlying the silty sand at 2.2 m depth (Elev. 189.1 m) in borehole M6-1 was cohesionless sand and silt. This stratum was 6.3 m thick and loose to very dense (SPT-'N' values of 9 to over 100), its moisture content varying between 8 and 14 percent. The sand and silt was penetrated at a depth of 8.5 m (Elev. 182.8 m). The results of grain size distribution analysis conducted on a sample of the stratum are presented in Figure M6-GS-2 (Appendix B).
- **Till:** a deposit of silt till was encountered below the sand and silt at 8.5 m depth (Elev. 182.8 m) in borehole M6-1. This deposit was very dense (SPT-'N' values in excess of 100) and had a moisture content of 12 to 15 percent. The borehole was terminated in the silt till at a depth of 11.1 m (Elev. 180.2 m). The results of grain size distribution analysis performed on a sample of the deposit are presented in Figure M6-GS-3 (Appendix B).

Groundwater Conditions:

- No groundwater was observed in either of the boreholes during or upon completion of drilling. It is noted, however, that groundwater levels may fluctuate subject to seasonal variations and precipitation patterns and should be expected in the floodplain.

Record of Borehole Sheets – Appendix A | **Laboratory Test Results – Appendix B** | **Key Location Plan – Drawing 3**

PART B - PRELIMINARY FOUNDATION DESIGN REPORT
HWY 407 EAST EXTENSION – WESTERN SECTION
W.O. 07 – 20015

LOCATION No: M-6 (WM-TABC-101)

FOUNDATION RECOMMENDATIONS

Note: The site-specific foundation recommendations are for planning purposes only. Refer to Section 6.0 of the Foundation Design Report for the project-wide foundation recommendations, design assumptions and limitations.

General: Based on the General Arrangement drawing of Culvert M-6 provided by URS in March 2010, the culvert will carry the realigned Highway 7 over a Brougham Creek tributary. The proposed open footing arch culvert will have a width of nearly 16 m and a length of 46 m. The invert levels of the culvert are specified to be at Elevation 191.4 m at the north end (inlet) and Elevation 190.6 m at the south end (outlet). Based on the existing subsurface information, the feasible foundation options for the proposed arch culvert foundations are listed below with advantages and disadvantages associated with each option.

Foundation Option	*Advantages*	*Disadvantages*
Spread footings founded on dense to very dense sandy soils	• Lower costs than deep foundations • Conventional construction	• Requires excavation of up to 4 m of surficial material to construct footings • Variability of surficial soils in floodplain • Scour protection required for footings
Steel H-Piles driven into "100-blow" sandy/silty soils	• Higher bearing resistance than for footings • Not affected by surficial soil variability	• Requires flange plate reinforcement to facilitate driving into very dense sand containing cobbles • Sub-excavation required for pile cap construction • Dewatering may be required for pile cap construction
Caissons bored to found within "100-blow" sandy/silty soils	• Higher bearing resistance than for footings • Not affected by surficial soil variability	• Drilling equipment must be capable of drilling through very dense sand with cobbles • Sub-excavation may be required for pile cap construction • Dewatering may be required for pile cap construction

A – Spread Footings: Spread footings founded on the dense to very dense sand and silt at or below Elevation 187 m at the north end (inlet) and on the very dense sand at or below Elevation 190 m at the south end (outlet). All footings should be placed at a minimum depth of 1.2 m below the lowest surrounding grade for frost protection.

Founding Stratum	*Geotechnical Resistance*	
	Factored ULS	**SLS**
Dense to Very Dense Sandy Soils	500 kPa	350 kPa

B – Steel H-Piles: Steel HP 310 x 110 piles driven into the "100-blow" sandy/silty soils at or below Elevation 182.5 m at the north end (inlet) and Elevation 185.0 m at the south end (outlet) are feasible for support of the foundation loads. Pile lengths would be approximately 8.0 and 4.5 m at the north and south ends, respectively.

Pile	*Geotechnical Axial Resistance*	
	Factored ULS	**SLS**
HP 310 x 110	1,600 kN	1,400 kN

C – Caissons: Caissons drilled to found within the "100-blow" sandy/silty soils at or below Elevation 181.5 m at the north end (inlet) and Elevation 185.0 m at the south end (outlet). Caissons should be socketed a minimum 2 m into the "100-blow" material. Caissons would be about 9.0 m at the north end and 4.5 m at the south.

Caisson Diameter	*Geotechnical Axial Resistance*	
	Factored ULS	**SLS**
1.2 m	4,500 kN	3,500 kN
1.5 m	6,500 kN	5,500 kN

Recommended Foundation Alternative: Spread footings founded on dense to very dense sandy soils.

• APPROACHES

Height: Based on the GA drawing, an embankment height of up to 5.5 m is anticipated. It is noted that sub-excavation of up to 2.2 m of surficial topsoil and silty sand containing organics would be required.

Stability: An embankment 5.5 m in height, constructed with select subgrade materials or granular fill, with side slopes no steeper than 2 horizontal to 1 vertical (2H:1V) will have an adequate factor of safety against deep-seated instability.

Settlement: Assuming the use of conventional earth or granular embankment fill materials, where applicable, the maximum settlement under the footprint of the new embankment is estimated to be less than 50 mm. The majority of this settlement is expected to occur during and immediately after construction (i.e. elastic settlement).

• CONSTRUCTION CONSIDERATIONS

Excavation: The surficial silty sand is classified as Type 3 soil, according to OHSA. Temporary excavations (i.e. open for a relatively short time period) should be stable with side slopes no steeper than 1H:1V assuming dewatering is provided. For saturated granular soils below the groundwater table in the floodplain area, temporary shoring may be required.

Groundwater/Surface Water Control: No groundwater was observed in the course of the field work. It is anticipated that groundwater within the foundation excavations can be adequately controlled by pumping from filtered sumps. Diversion of surface water from the excavation should be implemented as well.

Protection Systems: Refer to Section 6.7.2 of the Report.

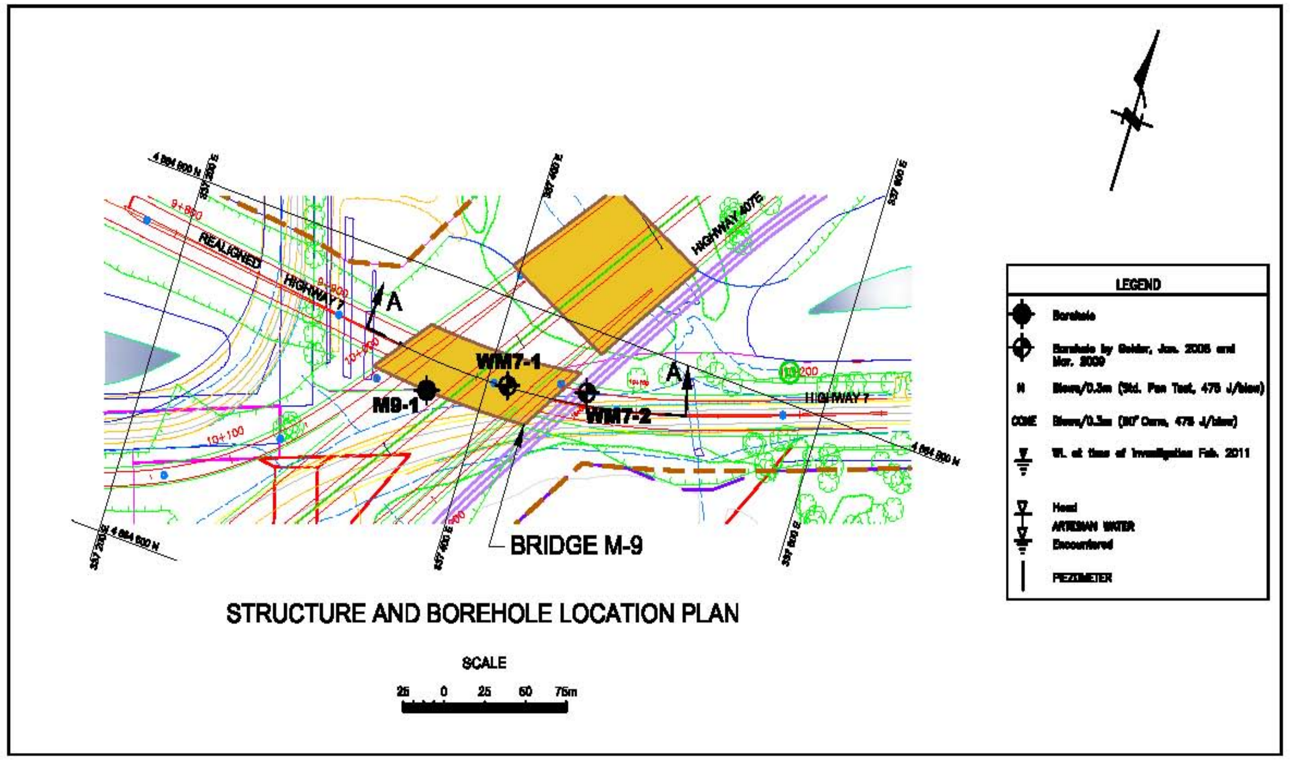
Obstructions During Pile Driving: Flange plate reinforcement for steel H-Piles if employed should be used to facilitate driving into the very dense sandy/silty soils containing cobbles and possible boulders. Caisson drilling equipment must be capable of penetrating obstructions such as cobbles and boulders.

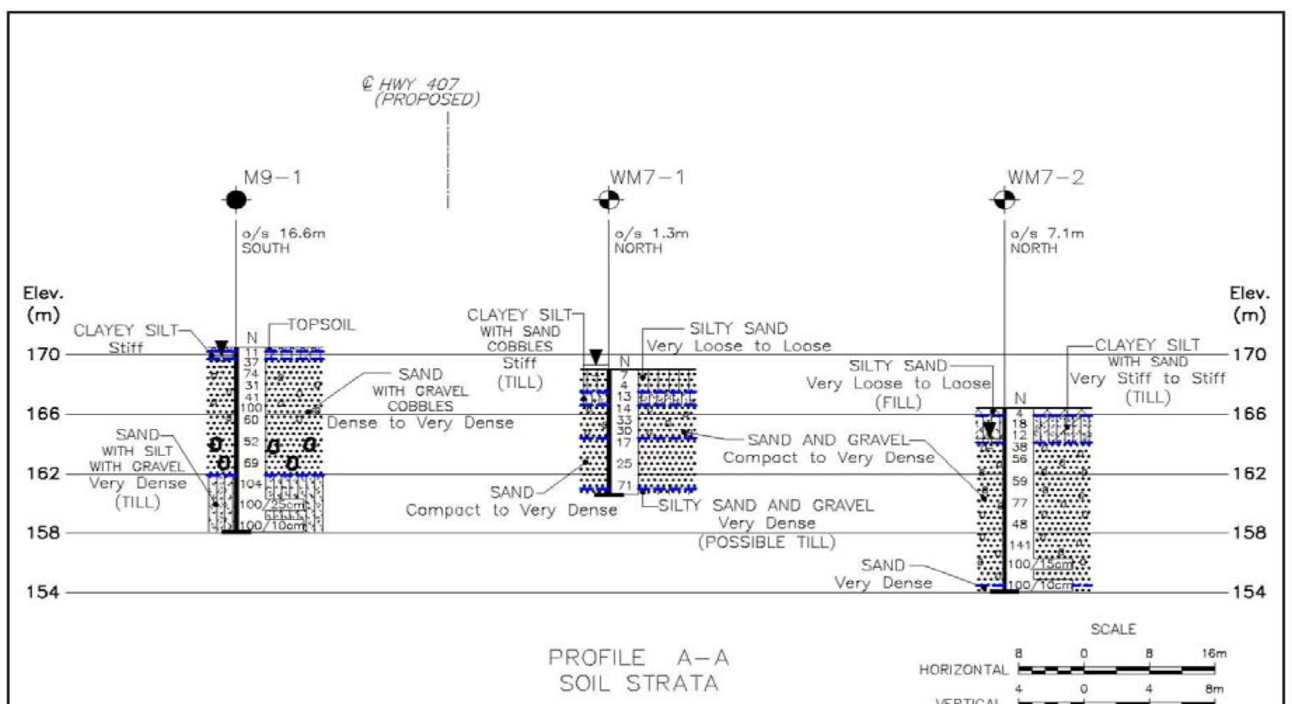
• RECOMMENDATIONS FOR ADDITIONAL WORK

Further subsurface investigation should be carried out during detail design to confirm the subsoil conditions at the locations of the arch culvert foundations.

PART A - PRELIMINARY FOUNDATION INVESTIGATION REPORT
HWY 407 EAST EXTENSION – WESTERN SECTION
W.O. 07 – 20015

Structure Description: Highway 407 / Realigned Highway 7 Overpass
Location No: M-9 (WM-7)
Hwy 407 Proposed Grade: 179.9 m – 180.4 m
Existing Ground Elevation: 165 m – 172 m
Site Ranking: Medium
Station: 18+367





FOUNDATION INVESTIGATIONS

Site Description:

The site of the proposed bridge structure M-9 is located on Highway 407 over the realigned Highway 7, just north of the existing Highway 7 and east of Sideline 16, in the City of Pickering, Ontario. The site is surrounded by a tree nursery to the west and a densely treed area to the southeast. A Brougham Creek tributary flows southerly across Highway 7, some 150 m to the east. The overall topography of the terrain is sloping down in the southeast direction.

Borehole Information:

Borehole No	Borehole Location	MTM NAD 83 – Northing	MTM NAD 83 – Easting	Borehole Elevation (m)	Borehole Depth (m)
M9-1	West Abutment	4 864 736.0	337 366.1	170.5	12.4
WM7-1	West / East Abutment	4 864 754.2	337 412.5	169.0	8.4
WM7-2	East Abutment	4 864 765.1	337 459.7	166.4	12.3

Note: Borehole WM7-1 was drilled as part of the hydrogeological investigation and monitored by Golder to supplement the existing borehole information.

Subsurface Conditions:

- **Topsoil:** surficial topsoil was present in borehole M9-1. The topsoil had a thickness of 300 mm and was penetrated at Elevation 170.2 m.
- **Fill:** silty sand fill was present surficially in borehole WM7-2. The fill was very loose to loose in relative density (SPT-'N' value of 4) and had a moisture content of 17 percent. The fill was 500 mm thick and penetrated at Elevation 165.9 m.
- **Silty Sand:** identified at the ground surface in borehole WM7-1 was silty sand containing organics. This unit was 1.5 m in thickness and very loose to loose in relative density (SPT-'N' values of 4 and 7), its moisture content varying between 19 and 24 percent. The silty sand extended to Elevation 167.5 m.
- **Clayey Silt:** directly beneath the topsoil at 0.3 m depth (Elev. 170.2 m) in borehole M9-1 was a cohesive deposit of clayey silt. This deposit was stiff in consistency and had a moisture content of about 21 percent. The clayey silt was 500 mm thick and penetrated at a depth of 0.8 m (Elev. 169.7 m).
- **Till:** overlain by the silty sand at 1.5 m depth (Elev. 167.5 m) in borehole WM7-1 or by the fill at a depth of 0.5 m (Elev. 165.9 m) in borehole WM7-2 was a cohesive deposit of clayey silt till. Containing cobbles in the former borehole, this deposit was stiff to very stiff in consistency and 11 to 12 percent in moisture content. The clayey silt till had a thickness of 0.9 m in borehole WM7-1 and 1.8 m in borehole WM7-2 and was penetrated at respective depths of 2.4 and 2.3 m (Elev. 166.6 and 164.1 m). The results of grain size distribution analyses and Atterberg limits testing conducted on 2 samples of the deposit are presented in Figures WM7-A and WM7-B (Appendix B), respectively.
- **Sandy/Gravelly Soils:** underlying the clayey silt deposits at depths of 0.8 to 2.4 m (Elev. 164.1 to 169.7 m) in all the boreholes were sandy/gravelly soils (sand and gravel, sand, sand till, possible silty sand and gravel till). This cohesionless stratum was compact to very dense (SPT-'N' values of 14 to over 100) and had a moisture content of 6 to 19 percent. The sandy/gravelly soils extended to the termination of drilling in boreholes M9-1 and WM7-2 at respective depths of 12.4 and 12.3 m (Elev. 158.1 and 154.1 m). Borehole WM7-1 was terminated in the possible silty sand and gravel till on encountering an inferred boulder at 8.4 m depth (Elev. 160.6 m). It is worth noting that the sand in borehole M9-1 contained cobbles. The results of grain size distribution analyses performed on 8 samples of the sandy/gravelly soils are presented in Figures M9-GS-1, M9-GS-2, WM7-C, WM7-D, WM7-E and WM7-F (Appendix B).

Groundwater Conditions:

- **Borehole M9-1:** Water was detected at a depth of 1.5 m (Elev. 169.0 m) in the process of augering. Groundwater was at 0.6 m depth (Elev. 169.9 m) upon completion of drilling.
- **Borehole WM7-1:** Groundwater was measured in piezometer to be 0.3 and 0.4 m above ground surface (Elev. 169.3 and 169.4 m) on March 23, 2009 and April 29, 2009, respectively.
- **Borehole WM7-2:** Groundwater was at a depth of 0.9 m (Elev. 165.5 m) upon completion of drilling. The piezometric water level was at depths of 2.2 and 2.1 m (Elev. 164.2 and 164.3 m) on February 28, 2008 and April 4, 2008, respectively.

Record of Borehole Sheets – Appendix A | **Laboratory Test Results – Appendix B** | **Key Location Plan – Drawing 3**

PART B - PRELIMINARY FOUNDATION DESIGN REPORT
HWY 407 EAST EXTENSION – WESTERN SECTION
W.O. 07 – 20015

LOCATION No: M-9 (WM-7)

FOUNDATION RECOMMENDATIONS

Note: The site-specific foundation recommendations are for planning purposes only. Refer to Section 6.0 of the Foundation Design Report for the project-wide foundation recommendations, design assumptions and limitations.

General: Based on the General Arrangement drawing of Structure M-9 provided by URS in March 2010, the bridge structure M-9 will carry Highway 407 and associated S-E and E-N/S ramps connecting to the realigned Brock Road over the realigned Highway 7. The proposed overpass consists of twin single span structures with a varying span length of 42.0 to 44.5 m and with approach embankments about 12 and 18 m high at the west and east abutments, respectively. Feasible foundation options for the proposed bridge abutments are listed below with advantages and disadvantages associated with each option.

Foundation Option	*Advantages*	*Disadvantages*
Spread footings founded on compact to very dense sandy/gravelly soils for abutments Spread footings founded on a compacted Granular 'A' pad or a clayey silt till deposit for wing walls	• Lower costs than deep foundations • Conventional construction	• Requires excavation of up to 2.5 m of surficial material when constructing for abutments • Dewatering measures are required for abutments • Requires excavation of up to 1.5 m of surficial material prior to placing a Granular 'A' pad for wing walls
Steel H-Piles driven into "100-blow" sandy/gravelly soils for abutment foundations	• Allows for integral abutment design	• Requires flange plate reinforcement to facilitate driving through the very dense sandy/gravelly soils and possible presence of cobbles / boulders within the till deposit • Dewatering may be required during construction (i.e. pile caps), special techniques may be required if artesian conditions are encountered
Caissons bored to found within "100-blow" sandy/gravelly soils for abutment foundations	• Higher bearing resistances than steel H-Piles	• Drilling must be advanced through the very dense sandy/gravelly soils and the till deposit containing cobbles / boulders • May require temporary or permanent liner extending above the prevailing groundwater level to prevent seepage inflow and softening of the caisson base • Dewatering may be required during construction (i.e. pile caps), special techniques may be required if artesian conditions are encountered

A - Spread Footings: Spread footings founded on the compact to very dense sandy/gravelly soils at or below Elevation 164 to 169 m for abutments. All footings should be placed at a minimum depth of 1.2 m below the lowest surrounding grade for frost protection. Spread footings founded on the stiff to very stiff clayey silt till at or below Elevation 167.5 m or on the dense to very dense sand at or below Elevation 169.0 m for wing walls. Alternatively, spread footings for the wing walls can be founded within the approach embankment on a compacted Granular 'A' pad.

Founding Stratum	*Geotechnical Resistance*	
	Factored ULS	**SLS**
Compact to Very Dense Sandy/Gravelly Soils	450 kPa	300 kPa
Stiff to Very Stiff Clayey Silt Till	400 kPa	250 kPa
Compacted Granular 'A' Pad	900 kPa	350 kPa

B – Steel H-Piles: Steel HP 310 x 110 piles driven to found within the "100-blow" sandy/gravelly soils at or below Elevation 159.5 m at the west abutment and Elevation 155.5 m at the east abutment are feasible for support of the foundation loads. Pile lengths would be approximately 9.5 and 12.5 m for the west and east abutments, respectively.

Location	*Pile*	*Geotechnical Axial Resistance*	
		Factored ULS	**SLS**
Abutments	HP 310 x 110	1,600 kN	1,400 kN

C – Caissons: Abutments on caissons founded within the "100-blow" sandy/gravelly soils at or below Elevation 159.0 m (west abutment) or Elevation 155.0 m (east abutment). Caissons should be socketed a minimum of 2 m into the "100-blow" material. Caissons would be about 10 m for the west abutment and 13 m for the east abutment.

Location	*Caisson Diameter*	*Geotechnical Axial Resistance*	
		Factored ULS	**SLS**
Abutments	1.2 m	4,500 kN	3,500 kN
	1.5 m	6,500 kN	5,500 kN

Recommended Foundation Alternative: Steel H-Piles for abutments; spread footings on a Granular 'A' pad for wing walls.

• ABUTMENT TYPE

The site soils are suitable for construction of conventional, integral or semi-integral abutments.

• APPROACHES

Embankment Height: Based on the GA drawing, embankment heights up to 12 m along the west approach and up to 18 m along the east approach are anticipated. It is noted that sub-excavation of up to 1.5 m of surficial topsoil, fill and silty sand containing organics would be required.

Stability: Approach embankments 12 to 18 m in height, constructed with select subgrade materials or granular fill, with side slopes no steeper than 2 horizontal to 1 vertical (2H:1V) and benches (minimum 2 m wide) located at vertical spacings not greater than 8 m will have an adequate factor of safety against deep-seated instability. Measures to stabilize the embankment slope surface due to potential surface water flow along the slope should be implemented.

Settlement: Assuming the use of conventional earth or granular embankment fill materials, where applicable, the maximum settlement under the footprint of the new embankments is estimated to be less than 50 mm. The majority of this settlement is expected to occur during and immediately after construction (i.e. elastic settlement).

• CONSTRUCTION CONSIDERATIONS

Excavation: The surficial fill, clayey and granular soils (i.e. sandy/gravelly soils above the groundwater table) are classified as Type 3 soils, according to OHSA. Temporary excavations (i.e. open for a relatively short time period) should be stable with side slopes no steeper than 1H:1V assuming dewatering is provided. For saturated granular soils below the groundwater table, temporary shoring may be required.

Groundwater / Surface Water Control: It is anticipated that groundwater within the foundation excavations for pile / caisson cap construction can be adequately controlled by pumping from filtered sumps; however, if artesian conditions are present, basal heave will need to be assessed and more elaborate dewatering measures may be required. Artesian groundwater conditions may be encountered when advancing deep foundations such as piles through the sandy/gravelly soils. Refer to Section 6.7.3 for options to control groundwater and migration of fines when driving piles at sites with possible artesian groundwater conditions.

Protection Systems: Refer to Section 6.7.2 of the Report.

PART B - PRELIMINARY FOUNDATION DESIGN REPORT
HWY 407 EAST EXTENSION – WESTERN SECTION
W.O. 07 – 20015

LOCATION No: M-9 (WM-7)

Obstructions During Pile Driving: Flange plate reinforcement for steel H-Piles should be used to facilitate driving through the clayey silt till and into the very dense sandy/gravelly soils possibly containing cobbles and boulders. Caisson drilling equipment must be capable of penetrating obstructions in event cobbles / boulders are present within the sandy/gravelly soils and till deposits.

Other: Due to the likely artesian water conditions within the sandy/gravelly soils, it is recommended that a sand filter possibly in combination with a geotextile be placed beneath the pile caps to prevent the migration of fines that may be transported along the steel H-Pile or along the caisson liner.

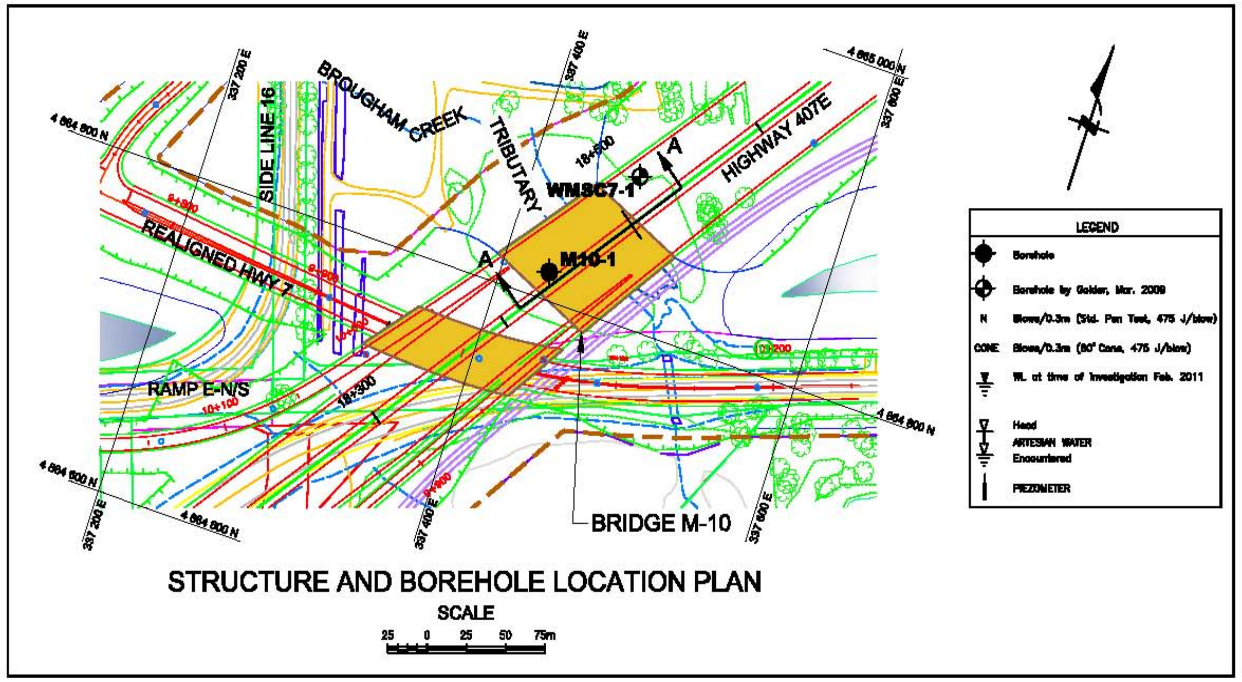
• RECOMMENDATIONS FOR ADDITIONAL WORK

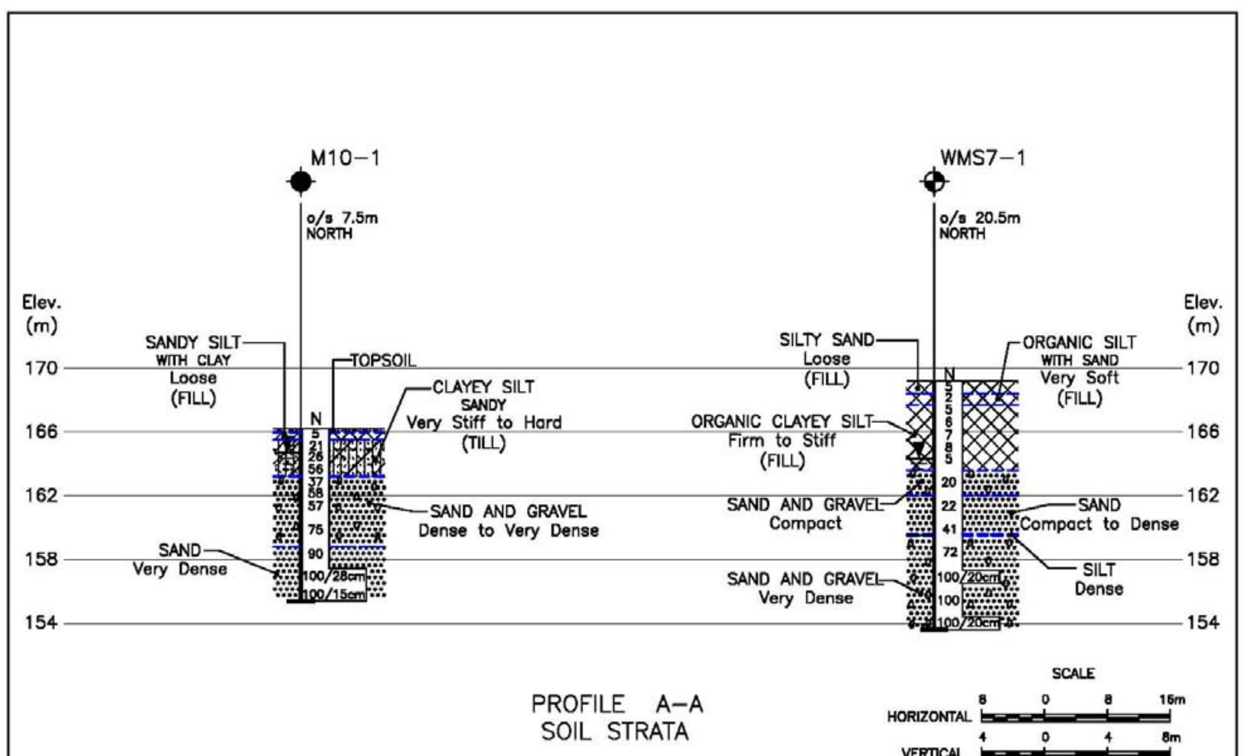
Further subsurface investigation should be carried out during detail design to confirm the subsoil conditions at the location of the bridge foundation elements.
In addition, footings will be founded at or below the prevailing groundwater level or artesian groundwater level (within granular subsoils), and these conditions will have to be assessed during detail design.

PART A - PRELIMINARY FOUNDATION INVESTIGATION REPORT
HWY 407 EAST EXTENSION – WESTERN SECTION
W.O. 07 – 20015

Structure Description: Highway 407 Bridge over Brougham Creek Tributary **Hwy 407 Proposed Grade:** 179.2 m – 179.7 m **Site Ranking:** Medium

Location No: M-10 (WM-5/6; WM-SC-7) **Existing Ground Elevation:** 161 m – 170 m **Station:** 18+466





FOUNDATION INVESTIGATIONS

Site Description:

The site of the proposed bridge structure M-10 is located on Highway 407 over a Brougham Creek tributary, some 70 m north of the existing Highway 7 and 190 m east of Sideline 16, in the City of Pickering, Ontario. The site is surrounded by cultivated farmland to the east and densely treed areas to the west, south and along the creek valley slopes. A pond locally described as the Dutchmaster Pond is situated northwest of the site. The overall topography of the terrain is sloping down towards the southeast.

Borehole Information:

Borehole No	Borehole Location	MTM NAD 83 – Northing	MTM NAD 83 – Easting	Borehole Elevation (m)	Borehole Depth (m)
M10-1	West Abutment	4 864 816.7	337 429.7	166.2	10.8
WMSC7-1	East Abutment	4 864 889.7	337 466.2	169.2	15.6

Subsurface Conditions:

Topsoil: surficial topsoil was present in borehole M10-1. The topsoil was 200 mm thick and penetrated at Elevation 166.0 m.

Fill: present surficially in borehole WMSC7-1 and below the topsoil at a depth of 0.2 m (Elev. 166.0 m) in borehole M10-1 was fill composed of silty sand / sandy silt. Loose in relative density (SPT-‘N’ value of 5) and about 15 percent in moisture content, this unit had a thickness of 800 mm in the former borehole and 500 mm in the latter and was penetrated at respective depths of 0.8 and 0.7 m (Elev. 168.4 and 165.5 m). Beneath the silty sand fill at 0.8 m depth (Elev. 168.4 m) in borehole WMSC7-1 were organic silt fill extended to a depth of 1.5 m (Elev. 167.7 m) and underlying organic clayey silt fill which was penetrated at 5.6 m depth (Elev. 163.6 m). The organic silt fill was 700 mm thick and very loose in relative density (SPT-‘N’ value of 2), with an organic content of 21.1 percent and moisture content of about 70 percent. Containing pockets of silty clay, the organic clayey silt fill was 4.1 m in thickness and firm to stiff in consistency, its organic content ranging from 5.6 to 6.2 percent and moisture content from 28 to 31 percent. The in situ vane testing yielded an undrained shear strength of 65 and 85 kPa (sensitivity of about 2), indicating a stiff consistency. The results of grain size distribution analysis and Atterberg limits testing are presented in Figures WMSC7-A and WMSC7-B (Appendix B), respectively.

Clayey Silt Till: directly beneath the sandy silt fill at a depth of 0.7 m (Elev. 165.5 m) in borehole M10-1 was a cohesive deposit of clayey silt till. This deposit was very stiff to hard in consistency and had a moisture content of 13 to 21 percent. The clayey silt till was 2.3 m thick and penetrated at 3.0 m depth (Elev. 163.2 m). The results of grain size distribution analysis and Atterberg limits testing conducted on a sample of the deposit are presented in Figures M10-GS-1 and M10-PC-1 (Appendix B), respectively.

Sandy/Gravelly Soils: underlying the fill in borehole WMSC7-1 and the clayey silt till in borehole M10-1 at respective depths of 5.6 and 3.0 m (Elev. 163.6 and 163.2 m) were cohesionless sandy/gravelly soils (sand and gravel, sand). These strata were compact to very dense (SPT-‘N’ values of 20 to over 100) with a moisture content varying between 5 and 19 percent. The sandy/gravelly soils extended to the termination of drilling at depths of 15.6 and 10.8 m (Elev. 153.6 and 155.4 m) in boreholes WMSC7-1 and M10-1 respectively. The results of grain size distribution analyses performed on 5 samples of the sand / sand and gravel are presented in Figures M10-GS-2, M10-GS-3, WMSC7-C and WMSC7-D (Appendix B).

Groundwater Conditions:

Borehole M10-1: Water was detected at a depth of 3.1 m (Elev. 163.1 m) in the process of augering. Groundwater was at a depth of 1.5 m (Elev. 164.7 m) upon completion of drilling.

Borehole WMSC7-1: Groundwater was at depths of 4.7 and 4.9 m (Elev. 164.5 and 164.3 m) in piezometer on March 27 and April 29, 2009, respectively.

PART B - PRELIMINARY FOUNDATION DESIGN REPORT
HWY 407 EAST EXTENSION – WESTERN SECTION
W.O. 07 – 20015

LOCATION No: M-10 (WM-5/6; WM-SC-7)

FOUNDATION RECOMMENDATIONS

Note: The site-specific foundation recommendations are for planning purposes only. Refer to Section 6.0 of the Foundation Design Report for the project-wide foundation recommendations, design assumptions and limitations.

General: Based on the General Arrangement drawing of Structure M-10 provided by URS in March 2010, the bridge structure M-10 will carry Highway 407 and associated S-E and E-N/S ramps connecting to the realigned Brock Road over a Brougham Creek tributary. The proposed bridge consists of twin three-span structures with a total length of 75 m each and with approach embankments approximately 18 m high at both abutments. Feasible foundation options for the proposed bridge abutments and piers are listed below with advantages and disadvantages associated with each option.

Foundation Option	*Advantages*	*Disadvantages*
Spread footings founded on dense to very dense sandy/gravelly soils for piers; spread footings founded on a compacted Granular 'A' pad for perched abutments	• Lower costs than deep foundations • Conventional construction	• Requires excavation of up to 5.6 m of organic fill materials when constructing • Requires construction of a Granular 'A' pad for perched abutments
Steel H-Piles driven into "100-blow" sandy/gravelly soils with "perched" pile caps within the bridge approaches	• Allows for integral abutment design	• Requires flange plate reinforcement to facilitate driving through the very dense stratum of sand / sand and gravel
Caissons bored to found within "100-blow" sandy/gravelly soils	• Higher bearing resistances than steel H-Piles	• Drilling must be advanced through the very dense stratum of sand / sand and gravel • May require temporary or permanent liner to prevent seepage inflow and loosening of the caisson base

A - Spread Footings: Spread footings "perched" on a compacted Granular 'A' pad may be considered for the bridge abutments, provided that the loose silty/sandy fill and organic fills extending to a depth of up to 5.6 m are subexcavated. Spread footings for piers founded within dense to very dense sand / sand and gravel at or below Elevation 160 m at the pier locations. All footings should be placed at a minimum depth of 1.2 m below the lowest surrounding grade for frost protection.

Founding Stratum	*Geotechnical Resistance*	
	Factored ULS	**SLS**
Dense to Very Dense Sand / Sand and Gravel (piers)	600 kPa	400 kPa
Compacted Granular 'A' (abutments)	900 kPa	350 kPa

B – Steel H-Piles: Piers and abutments with pile caps "perched" within the approaches for the bridge abutments, driven to found within the "100-blow" sandy/gravelly soils at or below Elevation 156 m. Piles lengths would be approximately 19.5 m at the west abutment and 20.5 m at the east abutment. The piles at the pier locations would be 4 to 5 m long.

Location	*Pile*	*Geotechnical Axial Resistance*	
		Factored ULS	**SLS**
Abutments and Piers	HP 310 x 110	1,600 kN	1,400 kN

C – Caissons: Piers and abutments on caissons, possibly extending to the underside of the bridge deck, founded a minimum 2 m within the "100-blow" sand / sand and gravel stratum below Elevation 155 m. Caissons would be approximately 20.5 m and 21.5 m long at the west and east abutment, respectively, and 5 to 6 m at the pier locations.

Location	*Caisson Diameter*	*Geotechnical Axial Resistance*	
		Factored ULS	**SLS**
Abutments	1.2 m	4,500 kN	3,500 kN
	1.5 m	6,500 kN	5,500 kN
Piers	1.2 m	3,800 kN	3,100 kN
	1.5 m	5,900 kN	5,000 kN

Recommended Foundation Alternative: Shallow foundations for piers and steel H-Piles for "perched" abutments.

• ABUTMENT TYPE

The site soils are not suitable for construction of conventional, integral or semi-integral abutments; "perched" abutments are considered suitable.

• APPROACHES

Embankment Height: Up to 18 m total height front slope (from crest to toe). Excavation of up to 5.6 m thick surficial silty/sandy fill, organic silt and organic clayey silt fills would be required.

Stability: West and east approach embankments up to 18 m high, constructed with select subgrade materials or granular fill, with side slopes no steeper than 2 horizontal to 1 vertical (2H:1V) and benches (minimum width of 2 m) located at maximum 8 m high intervals, will have an adequate factor of safety against deep-seated instability, assuming the fills (up to about 5.6 m deep) are completely removed prior to embankment construction. Measures to stabilize the embankment slope face due to potential surface water flow / seepage at the slope surface will have to be implemented.

Settlement: Assuming the use of conventional earth or granular embankment fills and assuming all existing fill materials are removed, it is expected that settlements in the order of 200 mm will occur under the footprint of the new embankment over a period of six months.
Alternatively, consideration could be given to removing the silty sand fill and very soft organic silt fill to 1.5 m depth at the location of borehole WMSC7-1 and preloading the existing organic clayey silt fill. Based on consolidation parameters and elastic deformation moduli of the foundation soils (estimated based on correlations with the undrained shear strength, Atterberg limits and SPT 'N' values), the maximum predicted total settlement within the embankment foundation soils is some 400 mm. About 5 percent of the total settlement is expected to take place during and immediately after completion of construction (i.e. elastic settlement). The majority (about 90 percent) of the remaining consolidation settlement is anticipated to occur over a period of twelve months. Additional settlements (long-term creep due to the presence of organics) are anticipated if the organic clayey silt fill is left in place. Measures to reduce post-construction settlement can be undertaken (such as surcharging); however, cognisant of the variable nature of the fill and high percentage of organics combined with the high approach embankment loading, it is recommended that the fill material be fully excavated.

• CONSTRUCTION CONSIDERATIONS

Excavation: The fill materials are classified as Type 3 soils, while the sand / sand and gravel strata are classified as Type 2 soils according to OSHA. Temporary excavations (i.e. open for a relatively short time period) should be made with side slopes no steeper than 1H:1V in Type 3 soils. Temporary excavations in Type 2 soils should be sloped to within 1.2 m of the bottom of the excavation assuming dewatering is provided where necessary.

Groundwater / Surface Water Control: It is considered that conventional sump pumping may not be sufficient to control groundwater within the foundation excavations. For shallow foundations and pile cap excavations in granular soils below the groundwater table, special construction techniques or more elaborate dewatering measures may be required in order to prevent loosening of the foundation soils.

Protection Systems: Refer to Section 6.7.2 of the Report.

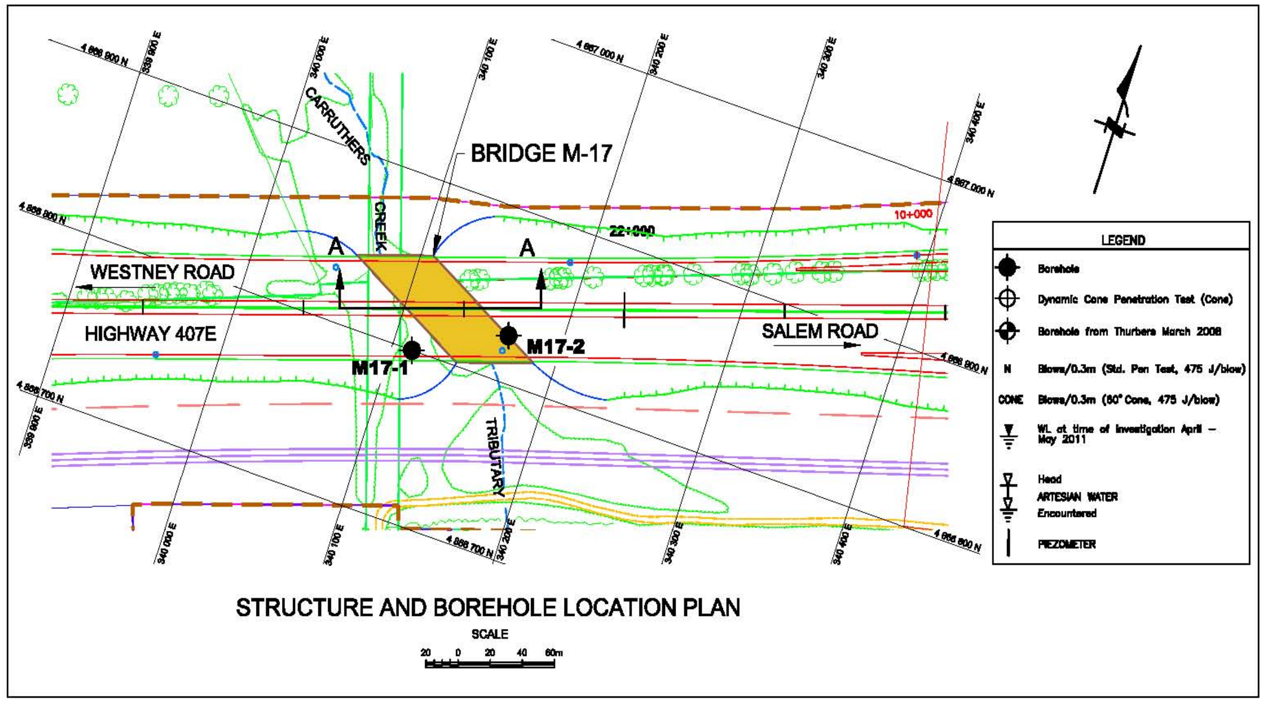
Obstructions During Pile Driving: Flange plate reinforcement for steel H-Piles should be used to facilitate driving into or through the very dense sand / sand and gravel stratum. Caisson drilling equipment must be capable of penetrating the very dense sandy/gravelly soils.

• RECOMMENDATIONS FOR ADDITIONAL WORK

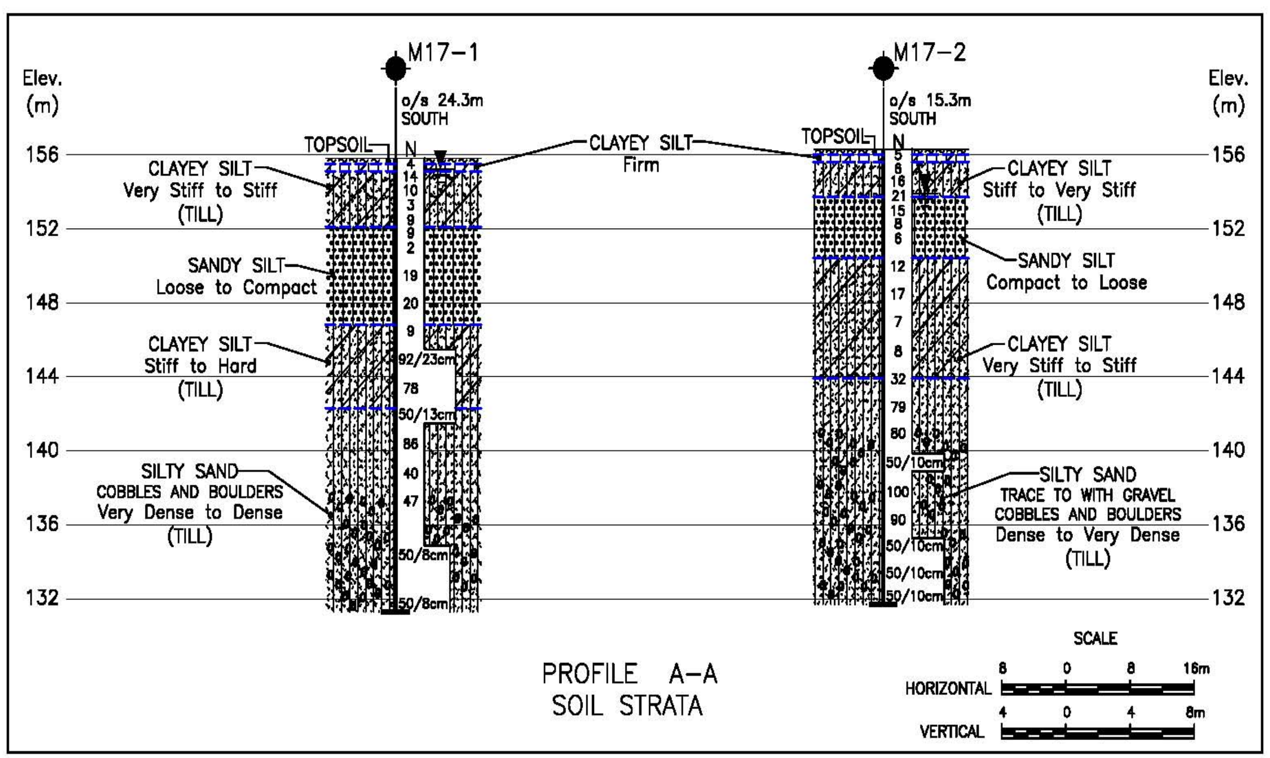
Further subsurface investigation should be carried out during detail design to confirm the subsoil conditions at the location of the bridge foundation elements as well as the extent of the organic fill and its consolidation characteristics to assess whether the fill may be left in place. In addition, since spread footings will be founded on granular soils at or below the prevailing groundwater level, groundwater conditions will have to be further assessed.

PART A - PRELIMINARY FOUNDATION INVESTIGATION REPORT
HWY 407 EAST EXTENSION – WESTERN SECTION
W.O. 07 – 20015

Structure Description: Highway 407 Bridge over Carruthers Creek Tributary
Location No: M-17 (WM-TACC-11)
Hwy 407 Proposed Grade: 162.3 m – 162.6 m
Existing Ground Elevation: 155 8 – 156.3 m
Site Ranking: Medium
Station: 21+888



STRUCTURE AND BOREHOLE LOCATION PLAN



PROFILE A–A
SOIL STRATA

FOUNDATION INVESTIGATIONS

Site Description:

The site of the proposed bridge structure M-17 is located on Highway 407 over a Carruthers Creek tributary between Westney Road and Salem Road in the City of Pickering, Ontario. The site is surrounded by cultivated farmland to the west and densely treed areas to the east and along the creek valley slopes. The overall topography of the terrain is sloping down towards the southeast.

Borehole Information:

Borehole No	Borehole Location	MTM NAD 83 – Northing	MTM NAD 83 – Easting	Borehole Elevation (m)	Borehole Depth (m)
M17-1	West Abutment (eastbound)	4 866 799.9	340 112.5	155.8	24.5
M17-2	East Abutment (eastbound)	4 866 827.2	340 166.8	156.3	24.6

Subsurface Conditions:

Topsoil: surficial topsoil was present in both boreholes. The topsoil was about 300 mm thick and penetrated at Elevation 155.5 m in borehole M17-1 and Elevation 156.0 m in borehole M17-2.

Clayey Silt: directly beneath the topsoil at a depth of 0.3 m (Elev. 155.5 to 156.0 m) in both boreholes was clayey silt containing rootlets. This unit was firm in consistency and up to 33 percent in moisture content. The clayey silt was about 400 mm in thickness and penetrated at 0.7 m depth (Elev. 155.1 to 155.6 m).

Clayey Silt Till: overlain by the clayey silt at a depth of 0.7 m (Elev. 155.1 to 155.6 m) in both boreholes was a cohesive deposit of clayey silt till. This deposit was typically stiff to very stiff in consistency and had a moisture content of 16 to 28 percent. The penetrometer tests performed on samples of the clayey silt till indicated undrained shear strength values in a range of 75 to 150 kPa. The deposit was interlayered with sandy silt / silt and extended to 13.5 m depth (Elev. 142.3 m) in borehole M17-1 and a depth of 12.4 m (Elev. 143.9 m) in borehole M17-2. The results of Atterberg limits testing and grain size distribution analyses conducted on two samples of the clayey silt till are presented in Figures M17-PC-1 and M17-GS-1 (Appendix B), respectively.

Sandy Silt: a cohesionless layer of sandy silt was revealed within the clayey silt till at 3.7 m depth (Elev. 152.1 m) in borehole M17-1 and a depth of 2.6 m (Elev. 153.7 m) in borehole M17-2. This layer was loose to compact in relative density (SPT-'N' values of 6 to 21, locally 2) and 11 to 20 percent in moisture content. The layer had a thickness of 5.3 m in borehole M17-1 and 3.3 m in borehole M17-2 and was penetrated at respective depths of 9.0 and 5.9 m (Elev. 146.8 and 150.4 m). The results of grain size distribution analysis performed on a sample of the sandy silt are presented in Figure M17-GS-2 (Appendix B).

Silty Sand Till: underlying the clayey silt till at 13.5 m depth (Elev. 142.3 m) in borehole M17-1 and a depth of 12.4 m (Elev. 143.9 m) in borehole M17-2 was cohesionless silty sand till. This stratum contained cobbles and boulders and was dense to very dense (SPT-'N' values in excess of 40), its moisture content varying between 9 and 29 percent. The silty sand till was at least 11.0 and 12.2 m thick and extended to the termination of drilling at depths of 24.5 and 24.6 m (Elev. 131.3 and 131.7 m) in boreholes M17-1 and M17-2 respectively.

Groundwater Conditions:

Borehole M17-1: Water was detected at a depth of 0.6 m (Elev. 155.2 m) in the process of augering. Artesian groundwater conditions were encountered at 13.7 m depth (Elev. 142.1 m), with an estimated head of 1.0 m above the ground surface.

Borehole M17-2: Water was detected at a depth of 2.4 m (Elev. 153.9 m) in the process of augering. Artesian groundwater conditions were encountered at 12.4 m depth (Elev. 143.9 m), with an estimated head of 1.0 m above the ground surface.

PART B - PRELIMINARY FOUNDATION DESIGN REPORT
HWY 407 EAST EXTENSION – WESTERN SECTION
W.O. 07 – 20015

LOCATION No: M-17 (WM-TACC-11)

FOUNDATION RECOMMENDATIONS

Note: The site-specific foundation recommendations are for planning purposes only. Refer to Section 6.0 of the Foundation Design Report for the project-wide foundation recommendations, design assumptions and limitations.

General: Based on the General Arrangement drawing of Structure M-17 provided by URS in March 2010, the bridge structure M-17 will carry Highway 407 over a Carruthers Creek tributary. The proposed bridge consists of twin single span structures with a span of 46 m each and with approach embankments approximately 6.5 m high at both abutments. Feasible foundation options for the proposed bridge abutments are listed below with advantages and disadvantages associated with each option.

Foundation Option	*Advantages*	*Disadvantages*
Spread footings founded on a compacted Granular 'A' pad for perched abutments	• Lower costs than deep foundations • Conventional construction	• Requires construction of a Granular 'A' pad for perched abutments
Steel H-Piles driven into "100-blow" silty sand till with "perched" pile caps within the bridge approaches	• Allows for integral abutment design	• Requires flange plate reinforcement to facilitate driving through the very dense stratum of silty sand till
Caissons bored to found within "100-blow" silty sand till	• Higher bearing resistances than steel H-Piles	• Drilling must be advanced through the very dense stratum of silty sand till • May require temporary or permanent liner to prevent seepage inflow and loosening of the caisson base

A - Spread Footings: Spread footings "perched" on a compacted Granular 'A' pad may be considered for the bridge abutments. All footings should be placed at a minimum depth of 1.2 m below the lowest surrounding grade for frost protection.

Founding Stratum	*Geotechnical Resistance*	
	Factored ULS	**SLS**
Compacted Granular 'A'	900 kPa	350 kPa

B – Steel H-Piles: Abutments with pile caps "perched" within the approaches for the bridge abutments should be driven to found within the "100-blow" silty sand till at or below Elevation 134 m. Piles lengths would be approximately 22 m at the west abutment and 21 m at the east abutment.

Location	*Pile*	*Geotechnical Axial Resistance*	
		Factored ULS	**SLS**
Abutments	HP 310 x 110	1,600 kN	1,400 kN

C – Caissons: Abutments on caissons, possibly extending to the underside of the bridge deck, founded a minimum 2 m within the "100-blow" silty sand till below Elevation 133 m. Caissons would be approximately 23 m and 22 m long at the west and east abutment, respectively.

Location	*Caisson Diameter*	*Geotechnical Axial Resistance*	
		Factored ULS	**SLS**
Abutments	1.2 m	4,500 kN	3,500 kN
	1.5 m	6,500 kN	5,500 kN

Recommended Foundation Alternative: Steel H-Piles founded within the "100-blow" silty sand till.

• ABUTMENT TYPE

The site soils are suitable for construction of conventional, integral or semi-integral abutments.

• APPROACHES

Embankment Height: Based on the GA drawing, embankment heights up to 6.5 m along both approaches to the bridge are anticipated. It is noted that sub-excavation of up to about 0.7 m of surficial topsoil and clayey silt containing organic material would be required.

Stability: Approach embankments up to 6.5 m in height, constructed with select subgrade materials or granular fill, with side slopes no steeper than 2 horizontal to 1 vertical (2H:1V) will have an adequate factor of safety against deep-seated instability.

Settlement: Assuming the use of conventional earth or granular embankment fill materials and based on consolidation parameters and elastic deformation moduli of the foundation soils, the maximum predicted total settlement within the embankment foundation soils is in the order of 150 mm. About 10 percent of the total settlement is expected to take place during and immediately after completion of construction (i.e. elastic settlement). The remaining settlement is anticipated to occur over a period of nine to twelve months. Measures to reduce post-construction settlement can be undertaken (such as surcharging). Detailed geotechnical analyses need to be carried out during the detail design.

• CONSTRUCTION CONSIDERATIONS

Excavation: The firm to stiff clayey soils and loose to compact sandy silt are classified as Type 3 soils according to OSHA. Temporary excavations (i.e. open for a relatively short time period) should be stable with side slopes no steeper than 1H:1V assuming dewatering is provided.

Groundwater / Surface Water Control: It is considered that conventional sump pumping will not be sufficient and interlocking sheetpiled cofferdams would be required to control groundwater within the foundation excavations for pile cap construction in the floodplain. Depending on construction season, diversion of surface runoff from the excavation may need to be implemented as well. Basal heave will need to be assessed and more elaborate dewatering measures may be required due to the artesian conditions present at the site. Refer to Section 6.7.3 for options to control groundwater and migration of fines when driving piles at sites with possible artesian groundwater conditions.

Protection Systems: Refer to Section 6.7.2 of the Report.

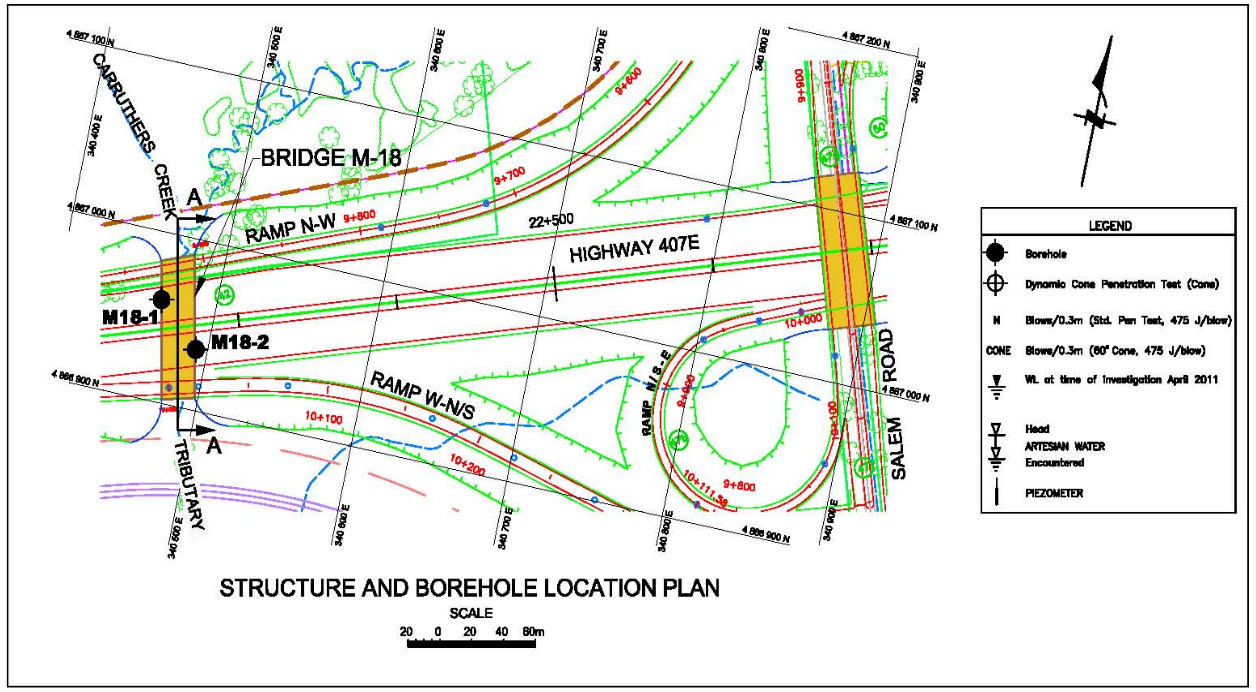
Obstructions During Pile Driving: Flange plate reinforcement for steel H-Piles should be used to facilitate driving into the hard clayey silt till and very dense silty sand till containing cobbles and boulders. Caisson drilling equipment must be capable of penetrating obstructions such as cobbles and boulders.

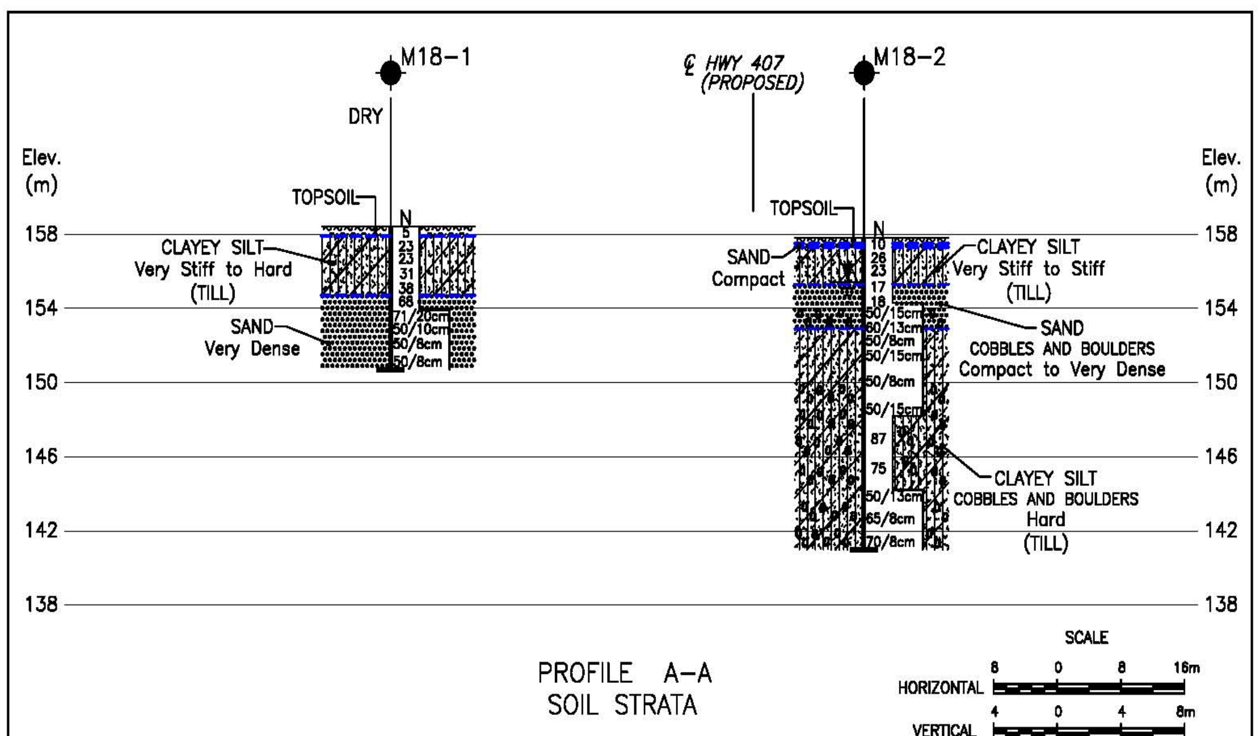
• RECOMMENDATIONS FOR ADDITIONAL WORK

Further subsurface investigation should be carried out during detail design to confirm the subsoil and groundwater conditions at the locations of the bridge abutments.

PART A - PRELIMINARY FOUNDATION INVESTIGATION REPORT
HWY 407 EAST EXTENSION – WESTERN SECTION
W.O. 07 – 20015

Structure Description: Highway 407 Bridge over Carruthers Creek Tributary
Location No: M-18 (WM-TBCC-12)
Hwy 407 Proposed Grade: 164.2 m – 164.4 m
Existing Ground Elevation: 157.8 m – 158.4 m
Site Ranking: Medium
Station: 22+262





FOUNDATION INVESTIGATIONS

Site Description:

The site of the proposed bridge structure M-18 is located on Highway 407 over a Carruthers Creek tributary some 400 m west of the interchange with Salem Road in the City of Pickering, Ontario. The site is surrounded by cultivated farmland and densely treed areas. The overall topography of the terrain is sloping down towards the southeast.

Borehole Information:

Borehole No	Borehole Location	MTM NAD 83 – Northing	MTM NAD 83 – Easting	Borehole Elevation (m)	Borehole Depth (m)
M18-1	West Abutment (westbound)	4 866 960.1	340 465.4	158.4	7.7
M18-2	East Abutment (eastbound)	4 866 935.7	340 492.6	157.8	16.8

Subsurface Conditions:

Topsoil: surficial topsoil was present in both boreholes. The topsoil had a thickness of 500 mm in borehole M18-1 and 300 mm in borehole M18-2 and was penetrated at Elevation 157.9 and 157.5 m respectively.

Sand (with organics): directly beneath the topsoil at 0.3 m depth (Elev. 157.5 m) in borehole M18-2 was sand with organic inclusions. This unit was compact in relative density (SPT-'N' value of 10) and about 15 percent in moisture content. The sand had a thickness of 200 mm and was penetrated at a depth of 0.5 m (Elev. 157.3 m).

Clayey Silt Till: overlain by the topsoil or sand at 0.5 m depth (Elev. 157.9 and 157.3 m) in boreholes M18-1 and M18-2 was a cohesive deposit of clayey silt till. This deposit was stiff to hard in consistency and had a moisture content of 7 to 23 percent. The penetrometer tests performed on samples of the clayey silt till indicated undrained shear strength values in a range of 62 to 163 kPa. The deposit was interlayered with a layer of sand (not penetrated upon termination of drilling in borehole M18-1) and extended to a depth of 16.8 m depth (Elev. 141.0 m) in borehole M18-2. The results of Atterberg limits testing and grain size distribution analyses conducted on two samples of the clayey silt till are presented in Figures M18-PC-1 and M18-GS-1 (Appendix B), respectively. It is noteworthy that cobbles and boulders were encountered in the deposit at 7.0 m depth (Elev. 150.8 m) in borehole M18-2.

Sand: cohesionless sand was revealed below the 3.2 and 2.0 m thick upper portions of the clayey silt till at depths of 3.7 and 2.5 m (Elev. 154.7 and 155.3 m) in boreholes M18-1 and M18-2 respectively. This stratum was compact to very dense (SPT-'N' values of 17 to over 100) with a moisture content varying between 5 and 20 percent. The sand was at least 4.0 m in thickness and extended to the termination of drilling at a depth of 7.7 m (Elev. 150.7 m) in boreholes M18-1. Containing cobbles and boulders, the stratum was 2.4 m thick and penetrated at 4.9 m depth (Elev. 152.9 m) in borehole M18-2. The results of grain size distribution analysis performed on a sample of the sand are presented in Figure M18-GS-2 (Appendix B).

Groundwater Conditions:

Borehole M18-1: Groundwater was not observed during or upon completion of drilling.

Borehole M18-2: Water was detected at a depth of 2.4 m (Elev. 155.4 m) in the process of augering. Artesian groundwater conditions were encountered at 8.2 m depth (Elev. 149.6 m), with an estimated head of 0.5 m above the ground surface.

PART B - PRELIMINARY FOUNDATION DESIGN REPORT
HWY 407 EAST EXTENSION – WESTERN SECTION
W.O. 07 – 20015

LOCATION No: M-18 (WM-TBCC-12)

FOUNDATION RECOMMENDATIONS

Note: The site-specific foundation recommendations are for planning purposes only. Refer to Section 6.0 of the Foundation Design Report for the project-wide foundation recommendations, design assumptions and limitations.

General: Based on the General Arrangement drawing of Structure M-18 provided by URS in March 2010, the bridge structure M-18 will carry Highway 407 and associated N-W and W-N/S ramps connecting to Salem Road over a Carruthers Creek tributary. The proposed bridge consists of four single span structures with a span of 20 m each and with approach embankments up to 6.5 m high at both abutments. Feasible foundation options for the proposed bridge abutments are listed below with advantages and disadvantages associated with each option.

Foundation Option	*Advantages*	*Disadvantages*
Spread footings founded on very stiff to hard clayey silt till or on a compacted Granular 'A' pad	• Lower costs than deep foundations • Conventional construction	• Requires construction of a Granular 'A' pad for perched abutments, if employed
Steel H-Piles driven into "100-blow" clayey silt till or sandy soils	• Allows for integral abutment design	• Requires flange plate reinforcement to facilitate driving through the very dense sand / hard clayey silt till with cobbles and boulders
Caissons bored to found within "100-blow" clayey silt till or sandy soils	• Higher bearing resistances than steel H-Piles	• Drilling must be advanced through the very dense sand / hard clayey silt till with cobbles / boulders • May require temporary or permanent liner to prevent seepage inflow and loosening of the caisson base

A - Spread Footings: Spread footings should be founded on the very stiff to hard clayey silt till at or below Elevation 156 m. Spread footings "perched" on a compacted Granular 'A' pad placed on the stiff to very stiff clayey silt till may also be considered. All footings should be placed at a minimum depth of 1.2 m below the lowest surrounding grade for frost protection.

Founding Stratum	*Geotechnical Resistance*	
	Factored ULS	**SLS**
Very Stiff Clayey Silt Till	450 kPa	300 kPa
Hard Clayey Silt Till	600 kPa	400 kPa
Compacted Granular 'A'	900 kPa	350 kPa

B – Steel H-Piles: Steel HP 310 x 110 piles driven to found within the "100-blow" clayey silt till or sandy soils at or below Elevation 152 are feasible for support of the west and east abutments. Piles lengths would be approximately 5 m at both abutments.

Location	*Pile*	*Geotechnical Axial Resistance*	
		Factored ULS	**SLS**
Abutments	HP 310 x 110	1,600 kN	1,400 kN

C – Caissons: Caissons should be founded a minimum 2 m within the "100-blow" clayey silt till or sandy soils at or below Elevation 151 m. Caissons would be approximately 6 m.

Location	*Caisson Diameter*	*Geotechnical Axial Resistance*	
		Factored ULS	**SLS**
Abutments	1.2 m	4,500 kN	3,500 kN
	1.5 m	6,500 kN	5,500 kN

Recommended Foundation Alternative: Shallow foundations founded on the clayey silt till deposit.

• ABUTMENT TYPE

The site soils are suitable for construction of conventional, integral or semi-integral abutments.

• APPROACHES

Embankment Height: Based on the GA drawing, embankment heights up to 6.5 m along both approaches to the bridge are anticipated. It is noted that sub-excavation of up to about 0.5 m of surficial topsoil and sand containing organic inclusions would be required.

Stability: Approach embankments up to 6.5 m in height, constructed with select subgrade materials or granular fill, with side slopes no steeper than 2 horizontal to 1 vertical (2H:1V) will have an adequate factor of safety against deep-seated instability.

Settlement: Assuming the use of conventional earth or granular embankment fill materials and based on consolidation parameters and elastic deformation moduli of the foundation soils, the maximum predicted total settlement within the embankment foundation soils is in the order of 100 mm. About 10 percent of the total settlement is expected to take place during and immediately after completion of construction (i.e. elastic settlement). The remaining settlement is anticipated to occur over a period of six to nine months. Measures to reduce post-construction settlement may be undertaken (such as surcharging). Detailed geotechnical analyses need to be carried out during the detail design.

• CONSTRUCTION CONSIDERATIONS

Excavation: The stiff clayey silt till and compact sand are classified as Type 3 soils according to OSHA. Temporary excavations (i.e. open for a relatively short time period) should be made with side slopes no steeper than 1H:1V assuming dewatering is provided where necessary.

Groundwater / Surface Water Control: It is considered that conventional sump pumping will not be sufficient and interlocking sheetpiled cofferdams would be required to control groundwater within the foundation excavations in the floodplain. Depending on construction season, diversion of surface runoff from the excavation may need to be implemented as well. Basal heave will need to be assessed and more elaborate dewatering measures may be required due to the artesian conditions present at the site. Refer to Section 6.7.3 for options to control groundwater and migration of fines when driving piles at sites with possible artesian groundwater conditions.

Protection Systems: Refer to Section 6.7.2 of the Report.

Obstructions During Pile Driving: Flange plate reinforcement for steel H-Piles should be used to facilitate driving into the hard clayey silt till containing cobbles and boulders. Caisson drilling equipment must be capable of penetrating obstructions such as cobbles and boulders.

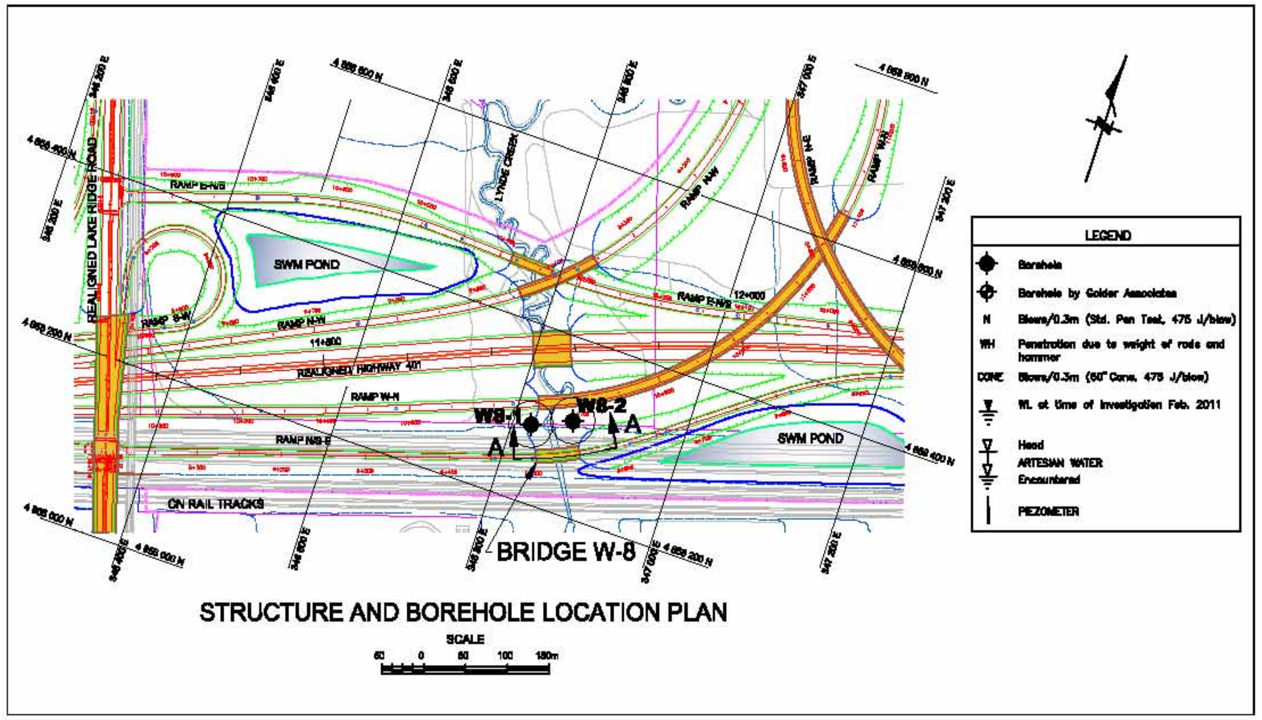
• RECOMMENDATIONS FOR ADDITIONAL WORK

Further subsurface investigation should be carried out during detail design to confirm the subsoil and groundwater conditions at the locations of the bridge abutments.

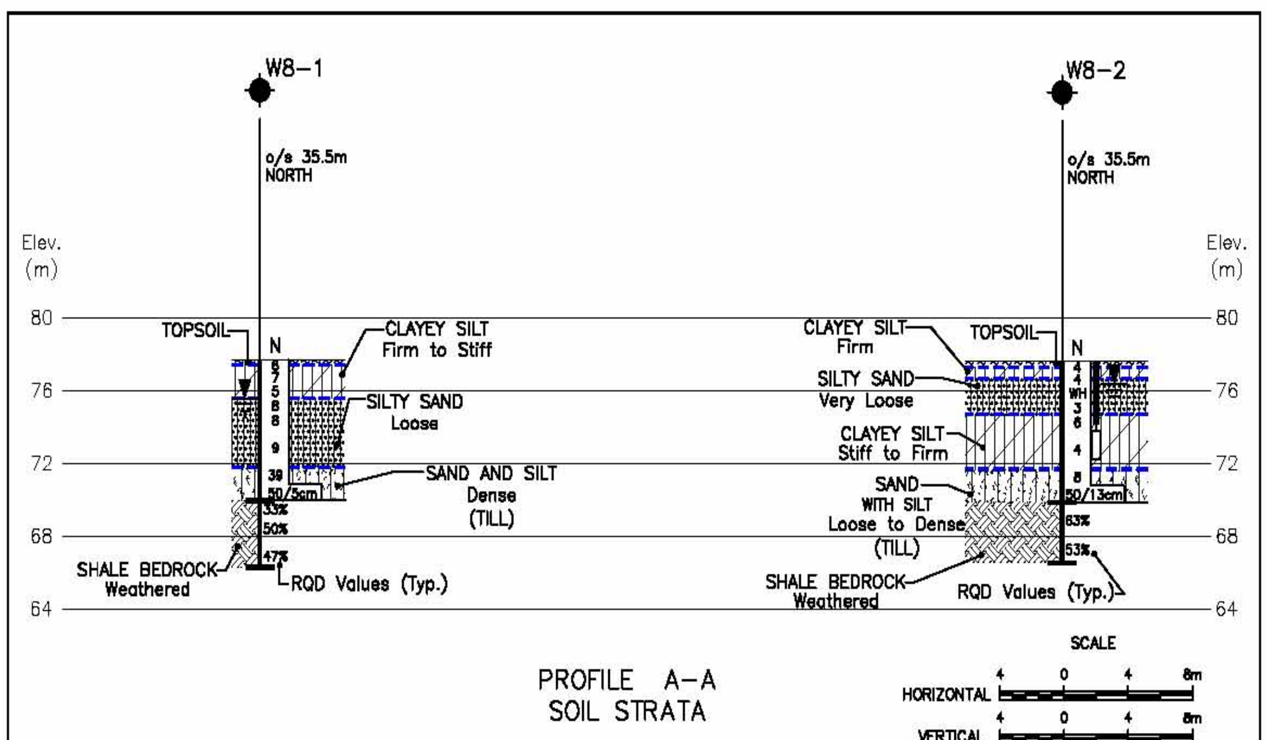
PART A - PRELIMINARY FOUNDATION INVESTIGATION REPORT
HWY 407 EAST EXTENSION – WESTERN SECTION
W.O. 07 – 20015

Structure Description: Lake Ridge Road / Hwy 401, N/S-E Ramp over West Lynde Creek
N/S-E Ramp Proposed Grade: 86.8 m – 88.0 m
Site Ranking: Medium

Location No: W-8 (WL-TALC-51D)
Existing Ground Elevation: 77.6 m – 82.5 m
Station: 9+530



STRUCTURE AND BOREHOLE LOCATION PLAN



PROFILE A–A
SOIL STRATA

FOUNDATION INVESTIGATIONS

Site Description:

The site of the proposed bridge structure W-8 for the Lake Ridge Road / Highway 401 N/S-E Ramp is located just south of the realigned Highway 401, approximately 500 m east of Lake Ridge Road and over West Lynde Creek, in the Town of Whitby, Ontario. The site topography is generally flat, determined by the existing Highway 401.

Borehole Information:

Borehole No	Borehole Location	MTM NAD 83 – Northing	MTM NAD 83 – Easting	Borehole Elevation (m)	Borehole Depth (m)
W8-1	West Abutment	4 858 287.7	346 821.3	77.7	11.4
W8-2	East Abutment	4 858 307.5	346 867.3	77.6	11.0

Subsurface Conditions:

- **Topsoil:** 300 mm thick topsoil was present surficially in both boreholes.
- **Clayey Silt:** directly beneath the topsoil at 0.3 m depth (Elev. 77.3 and 77.4 m) was a cohesive deposit of clayey silt. This deposit was 1.8 m thick and penetrated at a depth of 2.1 m (Elev. 75.6 m) in borehole W8-1. The clayey silt was interlayered with silty sand in borehole W8-2 and extended to 5.9 m depth (Elev. 71.7 m). In situ vane tests yielded an undrained shear strength of 30 to 35 kPa, indicating a firm consistency. Penetrometer test values of 25 to 75 kPa indicated a firm to stiff consistency. The results of Atterberg limits testing and grain size distribution analysis conducted on a sample of the deposit are presented in respective Figures W8-PC-1 and W8-GS-1 (Appendix B). The moisture content of the clayey silt varied between 24 and 90 percent.
- **Silty Sand:** 3.8 and 2.0 m thick layers of silty sand were revealed below or within the clayey silt at depths of 2.1 and 0.9 m (Elev. 75.6 and 76.7 m) in boreholes W8-1 and W8-2 respectively. The silty sand was very loose to loose in relative density (SPT ‘N’ values of 0 to 9) and had a moisture content of 9 to 34 percent. The silty sand was penetrated at depths of 5.9 and 2.9 m (Elev. 71.8 and 74.7 m) in boreholes W8-1 and W8-2 respectively. The results of one grain size distribution analysis are presented in Figure W8-GS-2 (Appendix B).
- **Till:** underlying the silty sand or clayey silt at a depth of 5.9 m (Elev. 71.8 and 71.7 m) in boreholes W8-1 and W8-2 was sand and silt till or sand till with silt. The cohesionless till was 1.8 m thick and loose to dense (SPT ‘N’ values of 8 and 39), its moisture content ranging from 10 to 13 percent. The till extended to bedrock encountered at 7.7 m depth (Elev. 70.0 and 69.9 m). The results of two grain size distribution analyses performed on sand and silt till and sand till are presented in respective Figures W8-GS-3 and W8-GS-4 (Appendix B).
- **Bedrock:** shale bedrock was contacted in both boreholes at a depth of 7.7 m (Elev. 70.0 and 69.9 m). Boreholes W8-1 and W8-2 were terminated within the bedrock at respective depths of 11.4 and 11.0 m (Elev. 66.3 and 66.6 m). Recovery of rock core samples was 95 to 100 percent. RQD values ranged from 33 to 63 percent, indicating a poor to fair quality rock.

Groundwater Conditions:

- **Borehole W8-1:** Water was detected at 2.1 m depth (Elev. 75.6 m) in the process of augering. No groundwater was present upon completion of drilling.
- **Borehole W8-2:** Water was detected at a depth of 1.2 m (Elev. 76.4 m) in the process of augering. Groundwater was at 1.0 m depth (Elev. 76.6 m) in piezometer on February 11, 2011 and a depth of 0.9 m (Elev. 76.7 m) on April 1, 2011.

PART B - PRELIMINARY FOUNDATION DESIGN REPORT
HWY 407 EAST EXTENSION – WESTERN SECTION
W.O. 07 – 20015

LOCATION No: W-8 (WL-TALC-51D)

FOUNDATION RECOMMENDATIONS

Note: The site-specific foundation recommendations are for planning purposes only. Refer to Section 6.0 of the Foundation Design Report for the project-wide foundation recommendations, design assumptions and limitations.

General: Based on the General Arrangement drawing of Structure W-8 provided by URS in March 2010, the bridge structure will carry the Lake Ridge Road / Realigned Hwy 401 N/S-E Ramp over West Lynde Creek. The proposed N/S-E Ramp bridge is a three (3) span structure with a total length of 60 m and with approach embankments up to about 6 m high. Feasible foundation options for the proposed bridge abutments are listed below with advantages and disadvantages associated with each option. Shallow foundations are not considered to be a practical option given the weak near surface subsoils at the site.

Foundation Option	*Advantages*	*Disadvantages*
Steel H-Piles driven to found on shale bedrock for abutments with “perched” / closed-end type pile caps.	• Allows for integral abutment design	• Requires flange plate reinforcement to facilitate driving through possible presence of cobbles within the till deposit
Caissons bored to found within shale bedrock.	• Larger caissons have higher bearing resistances than steel H-Piles	• Drilling must be advanced through possible presence of cobbles • May require temporary or permanent liner

A - Steel H-Piles: Steel HP 310 x 110 piles driven to refusal into the shale bedrock at or below Elev. 69.9 to 70.0 m are feasible for support of abutments with “perched” pile caps and piers. Pile lengths would be about 8 to 9 m. The structural design of the abutment and pier piles should be based on the full downdrag load acting on the piles as provided below, unless preloading and surcharging are undertaken to minimize post-construction settlements under the new embankment loading, in which case downdrag loads can be eliminated.

Pile	*Geotechnical Axial Resistance*		*Downdrag Load (Unfactored) abutments only*
	Factored ULS	**SLS**	
HP 310 x 110 (abutments, ‘perched’ / closed-end type pile caps)	2,000 kN	Does not govern	250 kN

B - Caissons: Abutments and piers on caissons founded within shale bedrock at or below Elev. 67.9 to 68.0 m. Caissons should be socketed a minimum of 2 m into the shale bedrock. Caissons would be about 10 to 11 m long. Full downdrag loads as provided below should be accounted for unless long-term settlement mitigation measures as discussed above for pile foundations are undertaken.

Caisson Diameter	*Geotechnical Axial Resistance*		*Downdrag Load (Unfactored) abutments only*
	Factored ULS	**SLS**	
1.2 m	6,500 kN	Does not govern	550 kN
1.5 m	9,500 kN	Does not govern	700 kN

Recommended Foundation Alternative: Steel H-Piles.

- ## ABUTMENT TYPE

The site soils are suitable for construction of conventional, integral or semi-integral abutments.

- ## APPROACHES

Embankment Height: Based on the GA drawings, embankment heights up to 6 m are anticipated. Based on the subsoil conditions encountered at the site, it is recommended that approach embankment fills be constructed with a maximum height of 6 m, provided that preloading with surcharge and construction staging be carried out prior to construction (refer to Settlement section below). It is further noted that sub-excavation of up to about 1.4 m of very wet soils (topsoil, clayey silt) would be required.

Stability: Approach embankments up to 6 m high, constructed with select subgrade materials or granular fill, with side slopes no steeper than 2 horizontal to 1 vertical (2H: 1V), will be safe against deep seated failure provided that the embankments are constructed in stages as discussed above.

Settlement: Assuming the use of conventional earth or granular embankment fill materials and based on consolidation parameters and elastic deformation moduli of the foundation soils (estimated based on the results of oedometer testing on samples from adjacent boreholes with similar soil characteristics and correlations with the undrained shear strength, Atterberg limits, and SPT ‘N’ values), the maximum predicted total settlement within the embankment foundation soils (based on a 6 m high embankment constructed of conventional granular fill) is in the order of 200 mm. Less than 5 percent of the total settlement is expected to take place during and immediately after completion of construction (i.e. elastic settlement); the majority (about 95 percent) of the remaining consolidation settlement is anticipated to occur over a period of three to six months. Measures to reduce post-construction settlement to acceptable values may include preloading with a surcharge and construction staging, use of lightweight fills or a combination of both lightweight fill and conventional earth fills. Detailed geotechnical analyses need to be carried out during the detail design to assess the construction requirements of the new embankment fills, including appropriate settlement monitoring instrumentation, and use of lightweight fill materials.

- ## CONSTRUCTION CONSIDERATIONS

Excavation: The firm to stiff clayey silt and loose silty sand are classified as Type 3 soils and very loose silty sand as Type 4 soil, according to OHSA. Temporary excavations (i.e. open for a relatively short time period) should be stable with side slopes no steeper than 1H:1V for Type 3 soils, assuming dewatering is provided, and no steeper than 3H:1V for Type 4 soils and sands / silts below the groundwater level.

Groundwater/Surface Water Control: It is considered that conventional sump pumping will not be sufficient to control groundwater within excavations for pile / caisson cap construction and more elaborate dewatering measures such as pile sheeted excavations will be required.

Protection Systems: Refer to Section 6.7.2 of the Report.

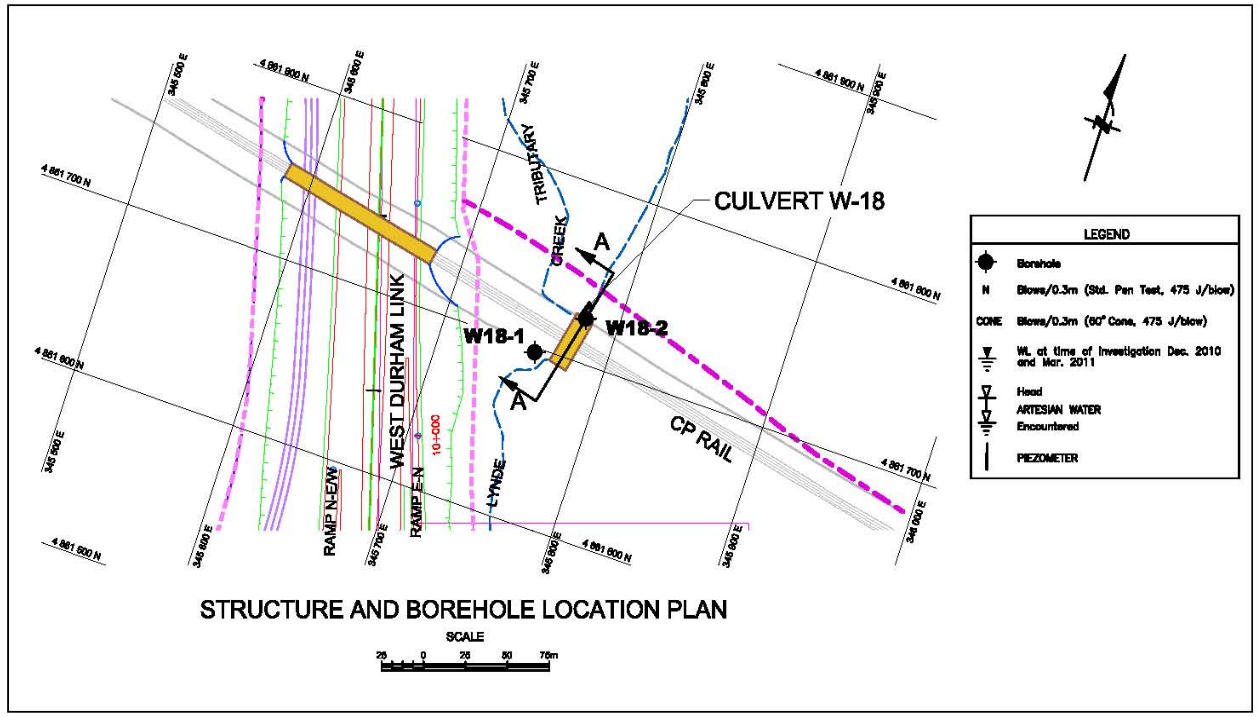
Obstructions During Pile Driving: Flange plate reinforcement for steel H-Piles should be used. No major obstructions (e.g. boulders) are anticipated at the site based on the borehole data at this site, although cobbles should be expected to be present within the till soils.

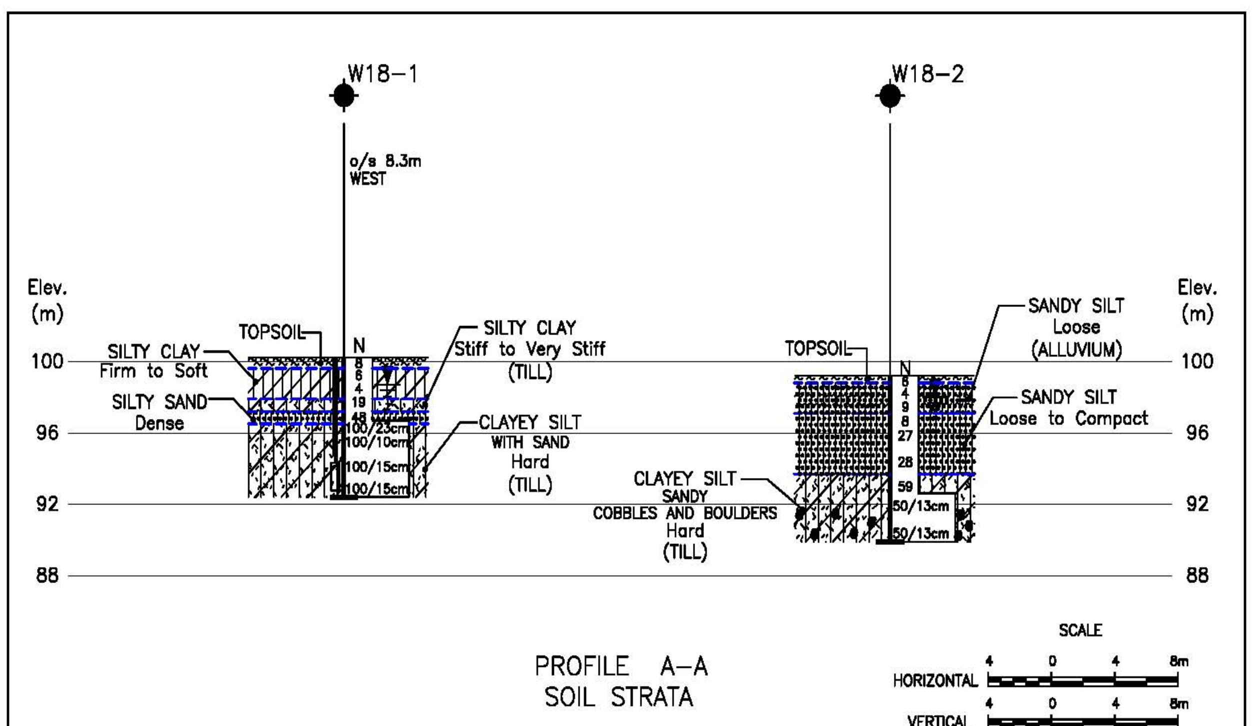
- ## RECOMMENDATIONS FOR ADDITIONAL WORK

Further subsurface investigation should be carried out during detail design to confirm the subsoil conditions at the location of the bridge foundation elements.

PART A - PRELIMINARY FOUNDATION INVESTIGATION REPORT
HWY 407 EAST EXTENSION – WESTERN SECTION
W.O. 07 – 20015

Structure Description: Culvert at the CP Rail over a Lynde Creek Tributary
Location No: W-18
CP Rail Proposed Grade: 105.3 m
Existing Ground Elevation: 99.2 m – 100.2 m
Site Ranking: Medium
Station: 10+133





FOUNDATION INVESTIGATIONS

Site Description:

The site of the proposed culvert W-18 at the CP Rail over a Lynde Creek tributary is located some 130 m east of the proposed West Durham Link and 350 m west of the realigned Coronation Road in the City of Whitby, Ontario. There is a culvert under existing CP Rail at the proposed culvert location. The site is surrounded by densely treed areas. The overall topography of the terrain is sloping down towards the south.

Borehole Information:

Borehole No	Borehole Location	MTM NAD 83 – Northing	MTM NAD 83 – Easting	Borehole Elevation (m)	Borehole Depth (m)
W18-1	South End (Outlet)	4 861 697.3	345 757.3	100.2	7.8
W18-2	North End (Inlet)	4 861 724.9	345 779.6	99.2	9.3

Subsurface Conditions:

- **Topsoil:** surficial topsoil was present in both boreholes. The topsoil had a thickness of 600 mm in borehole W18-1, 400 mm in borehole W18-2 and was penetrated at respective Elevations 99.6 and 98.8 m.
- **Alluvium:** directly beneath the topsoil at a depth of 0.4 m (Elev. 98.8 m) in borehole W18-2 was sandy silt alluvium. This unit was loose in relative density (SPT-'N' values of 4 and 9) and had a moisture content of 37 to 50 percent. The sandy silt alluvium was 1.7 m in thickness and penetrated at 2.1 m depth (Elev. 97.1 m).
- **Sandy Silt:** overlain by the alluvium at a depth of 2.1 m (Elev. 97.1 m) in borehole W18-2 was cohesionless sandy silt. This stratum was loose to compact in relative density (SPT-'N' values of 8 to 28) and 15 to 20 percent in moisture content. The sandy silt was 3.4 m thick and penetrated at 5.5 m depth (Elev. 93.7 m). The results of grain size distribution analysis performed on a sample of the stratum are presented in Figure W18-GS-1 (Appendix B).
- **Silty Clay:** a cohesive deposit of silty clay was identified below the topsoil at a depth of 0.6 m (Elev. 99.6 m) in borehole W18-1. This deposit was firm to soft in consistency and had a moisture content of 28 to 32 percent. In situ vane testing yielded an undrained shear strength of 20 and 32 kPa. Penetrometer tests indicated values of 25 and 50 kPa. The silty clay was 1.7 m in thickness and penetrated at 2.3 m depth (Elev. 97.9 m).
- **Till:** underlying the silty clay in borehole W18-1 or the sandy silt in borehole W18-2 at respective depths of 2.3 and 5.5 m (Elev. 97.9 and 93.7 m) was a cohesive deposit of silty clay till and/or clayey silt till. A 700 mm thick layer of dense silty sand was encountered within the till deposit at a depth of 3.0 m (Elev. 97.2 m) in borehole W18-1. The till was stiff to hard in consistency and extended to the termination depths of 7.8 and 9.3 m (Elev. 92.4 and 89.9 m) in boreholes W18-1 and W18-2 respectively. The results of grain size distribution analysis conducted on a sample of the clayey silt till are presented in Figure W18-GS-2 (Appendix B).

Groundwater Conditions:

- **Borehole W18-1:** Groundwater was at depths of 3.1 and 1.5 m (Elev. 97.1 and 98.7 m) during and upon completion of drilling, respectively. The water level measured in piezometer on March 11 and April 1, 2011 was at respective depths of 2.0 and 1.7 m (Elev. 98.2 and 98.5 m).
- **Borehole W18-2:** Groundwater was at depths of 1.5 and 1.2 m (Elev. 97.7 and 98.0 m) during and upon completion of drilling, respectively.

Record of Borehole Sheets – Appendix A **Laboratory Test Results – Appendix B** **Key Location Plan – Drawing 8**

PART B - PRELIMINARY FOUNDATION DESIGN REPORT
HWY 407 EAST EXTENSION – WESTERN SECTION
W.O. 07 – 20015

LOCATION No: W-18

FOUNDATION RECOMMENDATIONS

Note: The site-specific foundation recommendations are for planning purposes only. Refer to Section 6.0 of the Foundation Design Report for the project-wide foundation recommendations, design assumptions and limitations.

General: Based on the General Arrangement drawing of Culvert W-18 provided by URS in March 2010, the culvert will carry the CP Rail over a Lynde Creek tributary. The proposed open footing arch culvert will be 10.6 m in width and 31.6 m in length. The invert levels of the culvert are specified to be at Elevation 98.7 m at the north end (inlet) and Elevation 98.3 m at the south end (outlet). Based on the existing subsurface information, the feasible foundation options for the proposed arch culvert foundations are listed below with advantages and disadvantages associated with each option.

Foundation Option	*Advantages*	*Disadvantages*
Spread footings founded on compact to dense sandy/silty soils	• Lower costs than deep foundations • Conventional construction	• Requires excavation of up to 3 m of surficial material to construct footings • Dewatering required for footing construction • Variability of surficial soils in floodplain • Scour protection required for footings
Steel H-Piles driven into "100-blow" till deposit	• Higher bearing resistance than for footings • Not affected by surficial soil variability	• Requires flange plate reinforcement to facilitate driving into very dense sand containing cobbles and boulders • Sub-excavation required for pile cap construction • Dewatering required for pile cap construction
Caissons bored to found within "100-blow" till deposit	• Higher bearing resistance than for footings • Not affected by surficial soil variability	• May require temporary or permanent liner • Drilling equipment must be capable of drilling through hard clayey silt till with cobbles and boulders • Sub-excavation required for caisson cap construction • Dewatering required for caisson cap construction

A – Spread Footings: Spread footings founded on the compact to dense sandy silt at or below Elevation 96 m at the north end (inlet) and on the dense silty sand at or below Elevation 97 m at the south end (outlet). All footings should be placed at a minimum depth of 1.2 m below the lowest surrounding grade for frost protection.

Founding Stratum	*Geotechnical Resistance*	
	Factored ULS	**SLS**
Compact Sandy Silt (Inlet)	450 kPa	300 kPa
Dense Silty Sand (Outlet)	750 kPa	500 kPa

B – Steel H-Piles: Steel HP 310 x 110 piles driven into the "100-blow" till deposit at or below Elevation 90.5 m at the north end (inlet) and Elevation 94.5 m at the south end (outlet) are feasible for support of the foundation loads. Pile lengths would be approximately 7.0 and 3.0 m at the north and south ends, respectively.

Pile	*Geotechnical Axial Resistance*		*Downdrag Load (Unfactored)*
	Factored ULS	**SLS**	
HP 310 x 110	1,600 kN	1,400 kN	100 kN

C – Caissons: Caissons drilled to found within the "100-blow" till deposit at or below Elevation 89.5 m at the north end (inlet) and Elevation 94.0 m at the south end (outlet). Caissons should be socketed a minimum 2 m into the "100-blow" material. Caissons would be about 8.0 m at the north end and 3.5 m at the south.

Caisson Diameter	*Geotechnical Axial Resistance*		*Downdrag Load (Unfactored)*
	Factored ULS	**SLS**	
1.2 m	4,500 kN	3,500 kN	200 kN
1.5 m	6,500 kN	5,500 kN	250 kN

Recommended Foundation Alternative: Spread footings founded on compact to dense sandy/silty soils.

• APPROACHES

Height: Based on the GA drawing, an embankment height of 5 to 6 m is anticipated. It is noted that sub-excavation of up to 2.3 m of surficial topsoil, loose sandy silt alluvium and soft silty clay would be required.

Stability: An embankment up to 6 m in height, constructed with select subgrade materials or granular fill, with side slopes no steeper than 2 horizontal to 1 vertical (2H:1V) will have an adequate factor of safety against deep-seated instability.

Settlement: Assuming the use of conventional earth or granular embankment fill materials and based on consolidation parameters and elastic deformation moduli of the foundation soils, the total settlement within the embankment foundation soils is assessed to be 100 to 150 mm. About 10 percent of the total settlement is expected to take place during and immediately after completion of construction (i.e. elastic settlement). The remaining settlement is anticipated to occur over a period of six to nine months. Measures to reduce post-construction settlement may need to be undertaken. Detailed geotechnical analyses should be carried out during the detail design.

• CONSTRUCTION CONSIDERATIONS

Excavation: The silty clay and sandy silt are classified as Type 3 soils and the sandy silt alluvium as a Type 4 soil, according to OHSA. Temporary excavations (i.e. open for a relatively short time period) should be made with side slopes no steeper than 1H:1V in Type 3 soils and at 3H:1V in Type 4 soils. For saturated granular soils below the groundwater table in the floodplain area, temporary shoring may be required.

Groundwater/Surface Water Control: The groundwater is above the proposed footing level. Prior to excavations in the floodplain, groundwater control systems such as interlocking sheetpiled cofferdams would be required. Depending on construction season, diversion of surface water from the excavation and pumping from filtered sumps should be implemented as well.

Protection Systems: Refer to Section 6.7.2 of the Report.

Obstructions During Pile Driving: Flange plate reinforcement for steel H-Piles if employed should be used to facilitate driving into the hard clayey silt till containing cobbles and boulders. Caisson drilling equipment must be capable of penetrating obstructions such as cobbles / boulders.

• RECOMMENDATIONS FOR ADDITIONAL WORK

Further subsurface investigation should be carried out during detail design to confirm the subsoil conditions at the location of the arch culvert foundations.

PRELIMINARY FOUNDATION INVESTIGATION REPORT – DEEP CUTS AND HIGH FILLS

PRELIMINARY FOUNDATION INVESTIGATION REPORT – DEEP CUTS
HWY 407 EAST EXTENSION – WESTERN SECTION
W.O. 07 – 20015

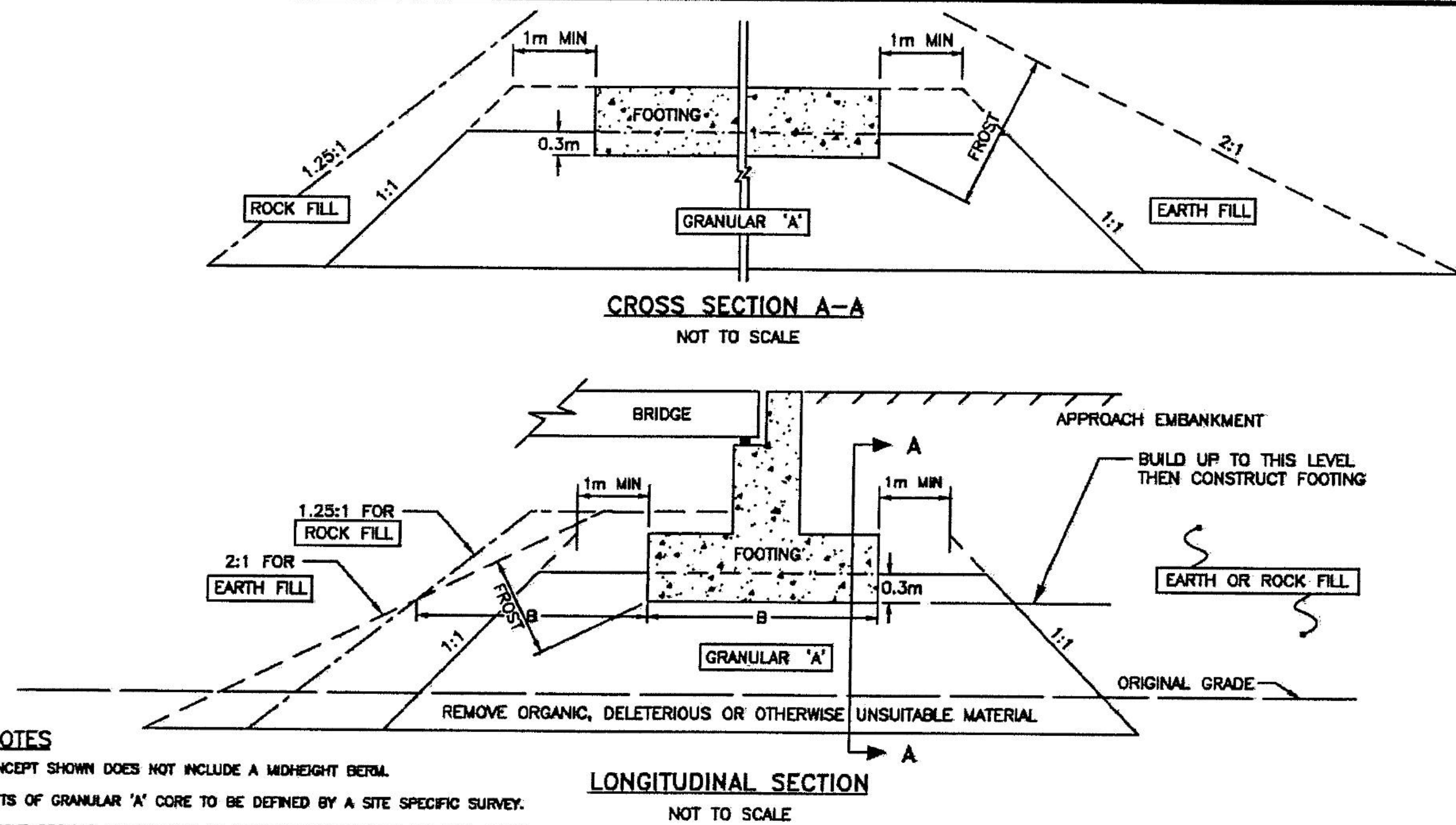
Deep Cut No.	Station (From – To)	Proposed Hwy 407/WDL Grade (m)	Maximum Cut Depth (m)	Reference Information/ Borehole Nos.	General Subsurface Conditions	Preliminary Recommendations
DC-W1	18+917 to 19+272 (Highway 407)	168 to 174	18.0	DCW1-1, DCW1-2, Hydrogeology Report	**Stratigraphy:** Surficial topsoil and loose silty sand with rootlets overlying at 1.4 m depth (Elev. 176.5 and 188.3 m) compact to very dense sand till with cobbles underlain in boreholes DCW1-1 and DCW1-2 at respective depths of 6.9 and 5.6 m (Elev. 171.0 and 184.1 m) by very dense sand penetrated at 9.8 m depth (Elev. 168.1 and 179.9 m) and overlying very dense sandy silt to silty sand till with cobbles and boulders that extends to the termination of drilling at depths of 13.8 and 26.2 m (Elev. 164.1 and 163.5 m). **Groundwater:** Estimated to range from near the ground surface to approximately 5 m below the ground surface. Borehole DCW1-1 – depths of 0.8 and 0.9 m (Elev. 177.1 and 177.0 m) in piezometer on May 16 and 24, 2011, respectively.	**Design Slope Inclination:** Drained cut slopes up to 18 m deep may be constructed at an inclination no steeper than 2H:1V and with a minimum 2 m wide mid-height bench for slopes from 8 to 16 m deep, and two 2 m wide benches equally spaced on slopes exceeding 16 m in depth. **Drainage:** Groundwater seepage should be anticipated in the granular soils along the cut slopes below the groundwater table. Depending on actual subsoil conditions and groundwater conditions, dewatering measures such as gravity drained 'pilot trenches are likely to be required prior to subexcavation to control groundwater and improve stability. Permanent groundwater control measures such as subdrains outletting to drainage ditches or a storm water collection system will be required.. **Surficial Instability:** Gravel sheeting or alternative methods may be required to control surficial erosion and instability at areas of localized seepage. **Recommendations for Further Investigation:** Subsurface investigation should be carried out to confirm the subsoil conditions and groundwater levels at the location of the cut section.
DC-W11	12+400 to 12+890 (West Durham Link)	96.0 to 98.4	7.0	DCW11-1, DCW11-2, W19-1, WL19A-1A, WL19A-2A, WL19-2A, WL19-3A	**Stratigraphy:** Surficial topsoil (up to 400 mm thick) and 0.9 to 2.7 m thick loose to compact sand / sandy silt (interlayered with clayey silt in borehole WL19A-1A) overlying a 1.0 to 4.2 m thick layer of soft to very stiff clayey silt / silty clay typically underlain by compact to very dense sand and silt till / silty sand till and/or stiff to hard clayey silt till with shale fragments, cobbles and boulders. Thin layers (0.6 to 0.7 m thick) of sand / sand and gravel were encountered within the till at depths of 3.5 and 6.1 m (Elev. 102.3 and 98.7 m) in boreholes WL19-2A and WL19-3A respectively. The till deposits extended to weathered shale bedrock contacted at 13.6 m depth (Elev. 89.8 m) in borehole W19-1 and inferred at a depth of 9.5 m (Elev. 96.3 m) in borehole WL19-2A. The remaining boreholes were terminated within the till deposits at depths of 7.0 to 8.5 m (Elev. 94.0 to 96.4 m). **Groundwater:** Borehole DCW11-1 – depths of 2.3 and 2.2 m below ground surface (Elev. 100.4 and 100.5 m) in piezometer on March 11, 2011 and April 1, 2011, respectively. Borehole DCW11-2 – depths of 1.5 and 6.1 m (Elev. 101.5 and 96.9 m) during and upon completion of drilling, respectively. Borehole WL19-2A – depth of 2.5 m below ground surface (Elev. 103.3 m) in piezometer on March 23, 2009.	**Design Slope Inclination:** Cut slopes up to 7.0 m deep may be constructed at an inclination no steeper than 2H:1V. **Drainage:** Permanent groundwater control measures will be required due to seepage from the sandy layers along the cut slopes. A passive gravity drain system could be considered to convey groundwater into ditches/a storm water collection system. **Surficial Instability:** Gravel sheeting or alternative methods may be required to control surficial erosion and instability at areas of localized seepage. **Recommendations for Further Investigation:** Further subsurface investigation should be carried out to confirm the subsoil conditions and groundwater levels at the location of the cut section.

Note: Deep Cut Sections have been identified based on the profile drawings provided by URS on November 6, 2008. It is noted that profiles were provided for the Highway 407 Mainline and West Durham Link (WDL) for the Western Section except for the sections along the WDL north of Highway 7 (i.e. WDL/Hwy 407 ramps) and south of Dundas Street. Deep Cut Sections may be present along the WDL, north of Highway 7 and south of Dundas Street, respectively, and these areas should be identified and assessed during detail design.

PRELIMINARY FOUNDATION INVESTIGATION REPORT – HIGH FILLS
HWY 407 EAST EXTENSION – WESTERN SECTION
W.O. 07 – 20015

High Fill No.	Station (From – To)	Proposed Hwy 407/WDL Grade (m)	Maximum Fill Height (m)	Reference Information/ Borehole Nos.	General Subsurface Conditions	Preliminary Recommendations
HF-W6	21+842 to 21+967 (Highway 407)	162 to 163	6.0	HFW6-1, M17-1, M17-2, Hydrogeology Report	**Stratigraphy:** Surficial topsoil and/or firm clayey silt overlying firm to hard clayey silt till interlayered with loose to compact sandy silt and underlain at depths of 12.4 to 13.5 m (Elev. 142.3 to 143.9 m) by dense to very dense silty sand till containing cobbles and boulders and extending to the borehole termination depths of 24.5 and 24.6 m (Elev. 131.3 and 131.7 m). **Groundwater:** Estimated to be at or near ground surface. Borehole HFW6-1 – depths of 2.4 and 3.0 m (Elev. 155.5 and 154.9 m) during and upon completion of drilling, respectively. Borehole M17-1 – depth of 0.6 m (Elev. 155.2 m) during drilling; artesian conditions encountered at 9.1 m depth (Elev. 146.7 m) on May 3, 2011. Borehole M17-2 – depth of 2.4 m (Elev. 153.9 m) during drilling; artesian conditions encountered at 12.2 m depth (Elev. 144.1 m) on April 27, 2011.	**Design Slope Inclination:** Fill embankments up to 6.0 m high may be constructed with slopes no steeper than 2H:1V. **Stability:** No stability issues are anticipated along this fill section. **Settlement:** Settlements in the order of 200 mm are anticipated due to consolidation of the clayey foundation soils under a maximum 6 m high granular embankment. The settlements are expected to occur over a period of about 12 months. The settlements are assessed to be virtually complete if the embankment is preloaded with a 2 m surcharge for 6 months. The surficial topsoil is to be removed prior to embankment construction. **Recommendations for Further Investigation:** Additional subsurface investigation with laboratory testing should be carried out to confirm the subsoil and groundwater conditions along the fill section.
HF-W7	22+217 to 22+427 (Highway 407)	164 to 167	6.0	HFW7-1, M18-1, M18-2, Hydrogeology Report	**Stratigraphy:** Surficial topsoil (and sand with organic inclusions in one borehole) overlying stiff to hard clayey silt till interlayered with compact to very dense silt / sand at depths of 2.5 to 3.7 m (Elev. 154.7 to 158.2 m) and containing cobbles and boulders. The boreholes were terminated within the silt / sand layers at depths of 6.6 and 7.7 m (Elev. 154.2 and 150.7 m) or in the clayey silt till deposit at 16.8 m depth (Elev. 141.0 m). **Groundwater:** Estimated to be at or near ground surface. Borehole HFW7-1 – depths of 2.6 and 2.1 m (Elev. 158.2 and 158.7 m) during and upon completion of drilling, respectively. Borehole M18-2 – depth of 2.4 m (Elev. 155.4 m) during drilling; artesian conditions encountered at 8.2 m depth (Elev. 149.6 m) on April 19, 2011.	**Design Slope Inclination:** Fill embankments up to 6.0 m high may be constructed with slopes no steeper than 2H:1V. **Stability:** No stability issues are anticipated along this fill section. **Settlement:** Settlements in the order of 150 mm are anticipated due to consolidation of the clayey foundation soils under a maximum 6 m high granular embankment. The settlements are expected to occur over a period of 9 to 12 months. The settlements are assessed to be complete if the embankment is preloaded with a 2 m surcharge for 6 months. The surficial topsoil and sand with organic inclusions are to be removed prior to embankment construction. **Recommendations for Further Investigation:** Additional subsurface investigation with laboratory testing should be carried out to confirm the subsoil and groundwater conditions along the fill section.

Note: High Fill Sections have been identified based on the profile drawings provided by URS on November 6, 2008. It is noted that profiles were provided for the Highway 407 Mainline and West Durham Link (WDL) for the Western Section except for the sections along the WDL north of Highway 7 (i.e. WDL/Hwy 407 ramps) and south of Dundas Street. High Fill Sections may be present along the WDL, north of Highway 7 and south of Dundas Street, respectively, and these areas should be identified and assessed during detail design.



NOTES

1. CONCEPT SHOWN DOES NOT INCLUDE A MIDHEIGHT BERM.
2. LIMITS OF GRANULAR 'A' CORE TO BE DEFINED BY A SITE SPECIFIC SURVEY.
3. REMOVE ORGANIC, DELETERIOUS OR OTHERWISE UNSUITABLE MATERIAL UNDER AREA OF COMPACTED GRANULAR 'A' AND EARTH OR ROCK FILL AS NOTED IN TEXT OF REPORT.
4. PLACE GRANULAR 'A' AND EARTH OR ROCK FILL ON APPROVED SUBGRADE TO BOTTOM OF FOOTING LEVEL, COMPACTED ACCORDING TO CURRENT M.T.O. STANDARDS.
5. CONSTRUCT CONCRETE FOOTING.
6. PLACE REMAINDER OF GRANULAR 'A' AND EARTH OR ROCK FILL INCLUDING MIDHEIGHT BENCHES, AS REQUIRED.
7. REFER TO TEXT OF REPORT FOR FROST DEPTH.

FIGURE 1: ABUTMENT ON COMPACTED FILL SHOWING GRANULAR A CORE

DRAWINGS

MINISTRY OF TRANSPORTATION, ONTARIO

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

W.O. No.07–20015

SHEET

HIGHWAY 407 EAST EXTENSION
WESTERN SECTION
PROJECT LOCATION PLAN

Peto MacCallum Ltd.
CONSULTING ENGINEERS

PLAN

SCALE APPROX.

0 1.6 3.2 4.8 Km

NOTES

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REFERENCE

Base provided by AECOM (AUGUST 2009).

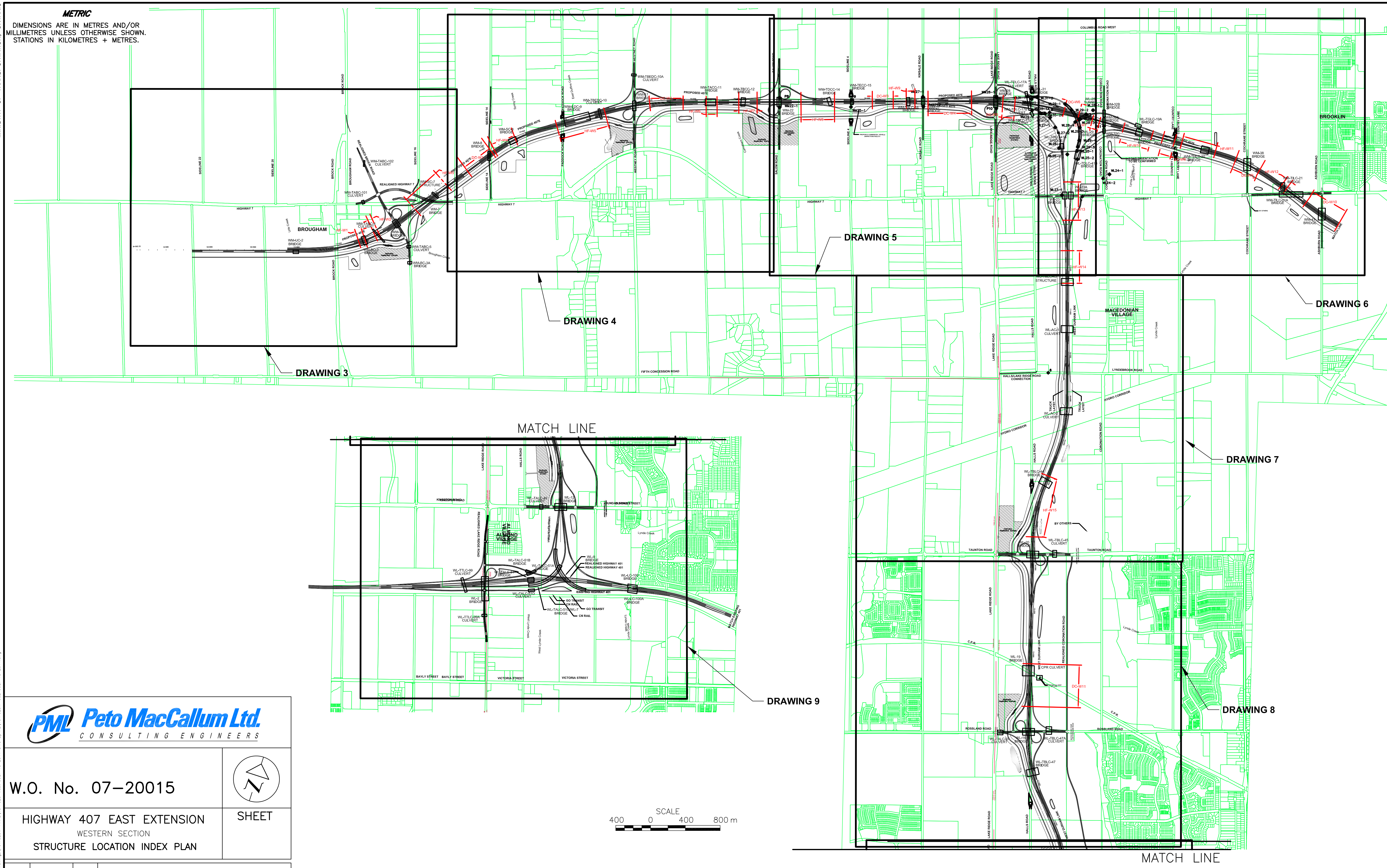
NO.	DATE	BY	REVISION
1	07/05/11	GD	BOREHOLE LOCATIONS UPDATED

Geocres No. 30M15–112

HWY. 407E		PROJECT NO. 10TF023	DIST. Central
SUBM'D. NA	CHKD. GD	DATE: Feb. 23, 2011	SITE:
DRAWN: AL	CHKD. CN	APPD. BRG	DWG. 1

METRIC

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



Peto MacCallum Ltd.
CONSULTING ENGINEERS

W.O. No. 07–20015

SHEET

HIGHWAY 407 EAST EXTENSION
WESTERN SECTION
STRUCTURE LOCATION INDEX PLAN

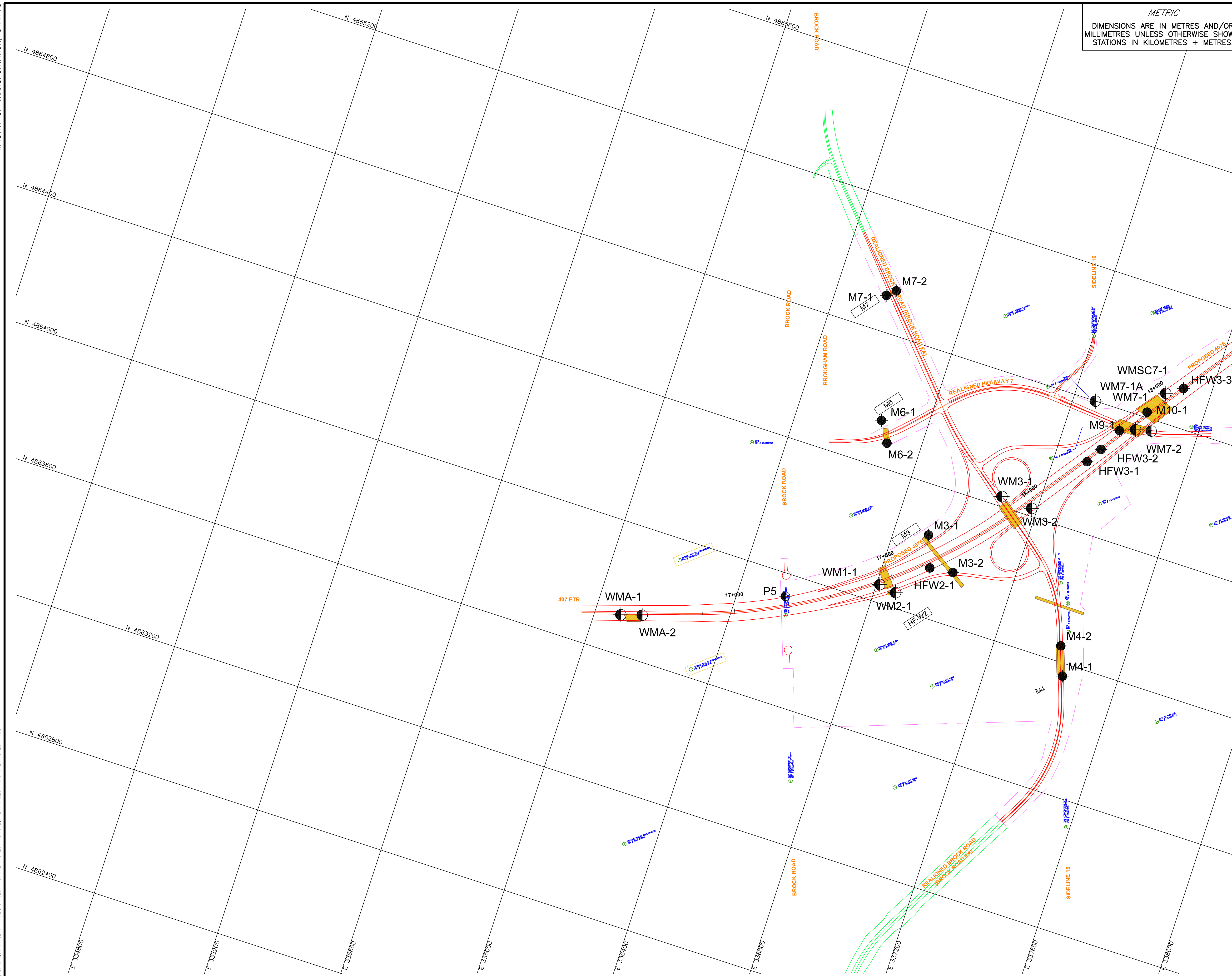
NO.	DATE	BY	REVISION
1	07/05/11	GD	BOREHOLE LOCATIONS UPDATED

GEOCRES No. 30M15–112

HWY. 407E	PROJECT NO. 10TF023		DIST. Central
SUBM'D. NA	CHKD. GD	DATE: Feb. 23, 2011	SITE:
DRAWN: AL	CHKD. CN	APPD. BRG	DWG. 2

REFERENCE

Base plan and profiles provided in digital format by MTO, drawing file nos. "407E Western Section PRELIMINARY DESIGN_ULTIMATE.dwg", received October 23, 2010.



PLAN

SCALE

120 0 120 240 m

METRIC

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

W.O. No. 07–20015

SHEET

HIGHWAY 407 EAST EXTENSION

WESTERN SECTION

BOREHOLE LOCATION – MAINLINE

West of Brock Rd To East of Sideline 16

Peto MacCallum Ltd.
CONSULTING ENGINEERS

KEY PLAN

SCALE APPROX.

0 1,600 3,200 4,800 m

LEGEND

Borehole – Current Investigation

Borehole – MTO Geocres

HF-W3 High Fill Section

No.	ELEVATION	CO–ORDINATES	
		NORTHING	EASTING
M3–1	184.2	4864247.4	336905.8
M3–2	182.5	4864160.7	337013.1
M4–1	165.1	4863961.3	337433.5
M4–2	166.2	4864048.7	337399.8
M6–1	191.3	4864539.0	336658.1
M6–2	192.6	4864477.2	336696.0
M7–1	202.4	4864910.2	336552.9
M7–2	205.1	4864933.1	336578.1
M9–1	170.5	4864736.0	336366.1
M10–1	166.2	4864816.7	337429.7
HFW2–1	185.3	4864152.1	336940.9
HFW3–1	174.5	4864614.1	337300.8
HFW3–2	173.2	4864663.3	337330.0
HFW3–3	170.8	4864921.0	337513.8

NOTES

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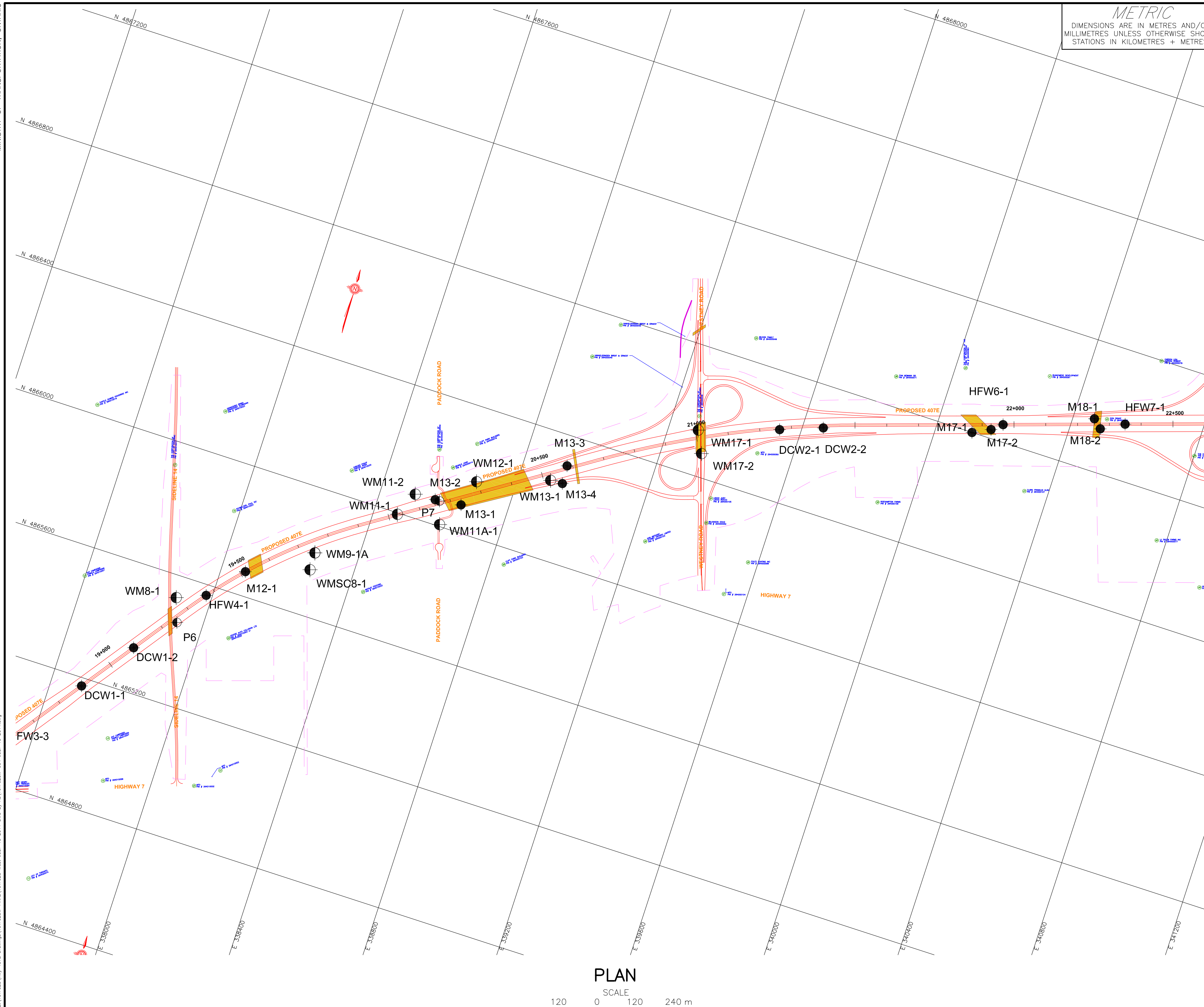
NO.	DATE	BY	REVISION
1	07/04/11	GD	BOREHOLE LOCATIONS UPDATED

Geocres No. 30M15–112

HWY. 407E		PROJECT NO. 10TF023	DIST. Central
SUBM'D. NA	CHKD. GD	DATE: Feb. 23, 2011	SITE:
DRAWN: AL	CHKD. CN	APPD. BRG	DWG. 3



PLAN

SCALE

120 0 120 240 m

METRIC

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

W.O. No. 07–20015

SHEET

HIGHWAY 407 EAST EXTENSION

WESTERN SECTION

BOREHOLE LOCATION – MAINLINE

East of Sideline 16 to Salem Road

Peto MacCallum Ltd.

CONSULTING ENGINEERS

KEY PLAN

SCALE APPROX.

0 1,600 3,200 4,800 m

LEGEND

Borehole – Current Investigation

Borehole – MTO Geocres

DC-W2 Deep Cut Section

HF-W7 High Fill Section

No.	ELEVATION	CO–ORDINATES	
		NORTHING	EASTING
M12–1	152.2	4865676.0	338075.2
M13–1	141.2	4866085.8	338655.0
M13–2	144.2	4866076.2	338572.6
M13–3	141.1	4866305.8	338934.1
M13–4	142.7	4866248.3	338937.2
M17–1	155.8	4866799.9	340112.5
M17–2	156.3	4866827.2	340166.8
M18–1	158.4	4866960.1	340465.4
M18–2	157.8	4866935.7	340492.6
DCW1–1	177.9	4865174.6	337696.5
DCW1–2	189.7	4865339.4	337814.9
DCW2–1	165.2	4866621.1	339534.4
DCW2–2	165.2	4866668.9	339663.3
HFW4–1	156.0	4865566.9	337980.9
HFW6–1	157.9	4866853.6	340197.9
HFW7–1	160.8	4866973.7	340562.4

NOTES

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NO.	DATE	BY	REVISION
1	07/05/11	GD	BOREHOLE LOCATIONS UPDATED

Geocres No. 30M15–112

HWY. 407E		PROJECT NO. 10TF023	DIST. Central
SUBM'D. NA	CHKD. GD	DATE: Feb. 23, 2011	SITE:
DRAWN: AL	CHKD. CN	APPD. BRG	DWG. 4

MINISTRY OF TRANSPORTATION, ONTARIO

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

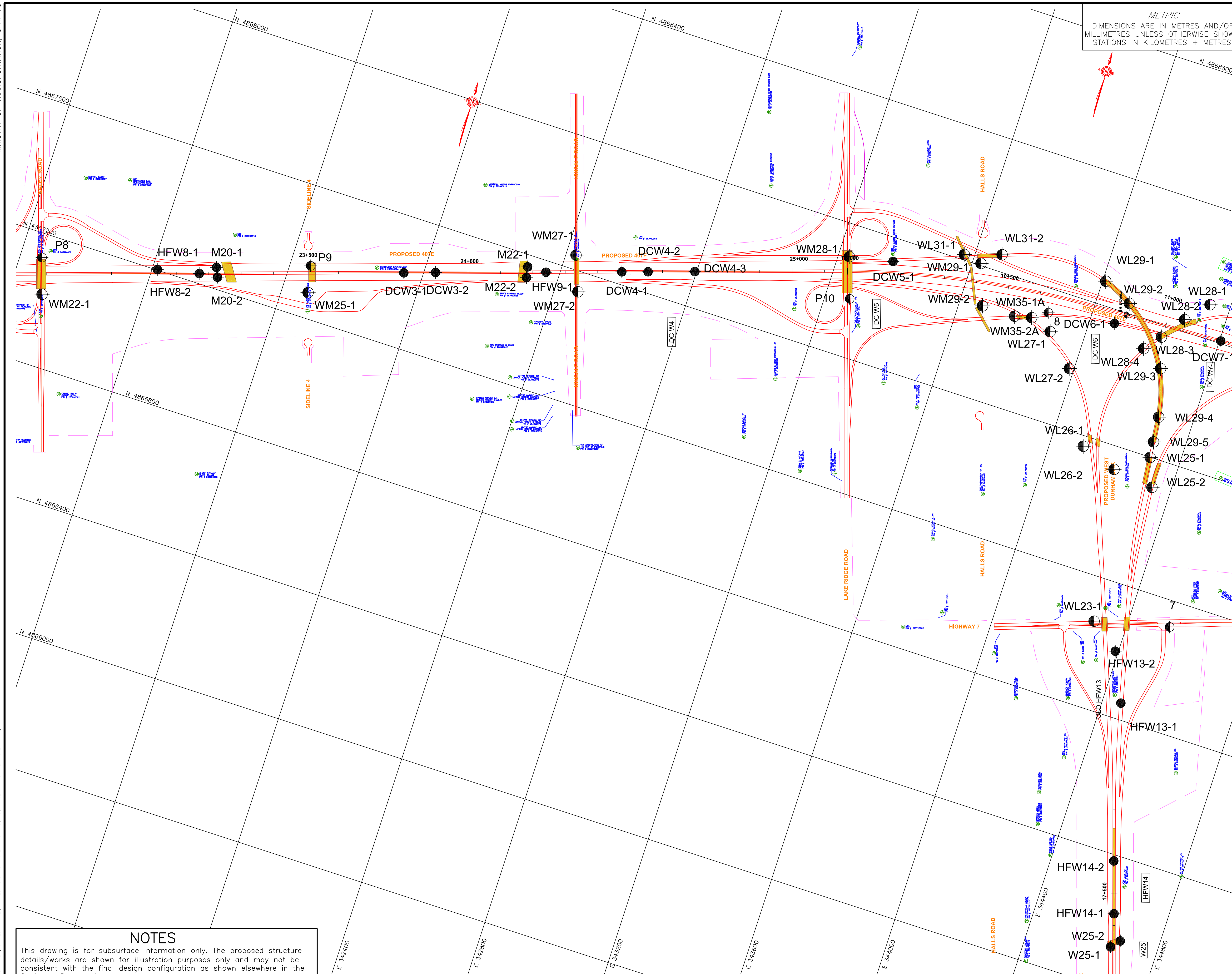
W.O. No. 07–20015

SHEET

HIGHWAY 407 EAST EXTENSION

WESTERN SECTION
BOREHOLE LOCATION – MAINLINE
Salem Road to East of Coronation Rd

Peto MacCallum Ltd.
CONSULTING ENGINEERS



NOTES

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PLAN

SCALE
120 0 120 240 m

KEY PLAN
SCALE APPROX.
0 1,600 3,200 4,800 m

LEGEND

Borehole – Current Investigation
Borehole – MTO Geocres
DC-W7 Deep Cut Section
HF-W14 High Fill Section

No.	ELEVATION	CO-ORDINATES NORTHING	CO-ORDINATES EASTING
M20–1	177.9	4867264.6	341387.1
M20–2	176.0	4867236.9	341399.5
M22–1	186.2	4867563.5	342299.7
M22–2	185.3	4867530.8	342306.5
W25–1	143.5	4866125.7	344659.0
W25–2	143.9	4866152.5	344682.1
DCW3–1	198.8	4867427.7	341942.0
DCW3–2	198.7	4867459.1	342034.9
DCW4–1	200.7	4867638.9	342580.6
DCW4–2	201.7	4867664.1	342657.5
DCW4–3	201.3	4867709.4	342794.8
DCW5–1	197.7	4867918.3	343366.4
DCW6–1	184.7	4867957.8	344075.0
DCW7–1	174.8	4868008.0	344404.0
HFW8–1	174.0	4867201.7	341215.9
HFW8–2	176.8	4867230.2	341342.5
HFW9–1	187.1	4867565.1	342358.4
HFW13–1	162.7	4866850.5	344456.5
HFW13–2	163.3	4866997.5	344390.6
HFW14–1	145.7	4866226.3	344637.5
HFW14–2	146.9	4866380.9	344586.5

REFERENCE

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NO.	DATE	BY	REVISION
1	07/05/11	GD	BOREHOLE LOCATIONS UPDATED

Geocres No. 30M15–112

HWY. 407E	PROJECT NO. 10TF023		DIST. Central
SUBM'D. NA	CHKD. GD	DATE: Feb. 23, 2011	SITE:
DRAWN: AL	CHKD. CN	APPD. BRG	DWG. 5

PLOT DATE: January 28, 2011
FILENAME: Z:\10TF023\Hwy 407E Drawings\10TF023W–FINAL\10TF023 West JULY 13 2011 DWG By AL\10TF023W–005 JULY 13 2011.dwg

MINISTRY OF TRANSPORTATION, ONTARIO

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

W.O. No. 07–20015

SHEET

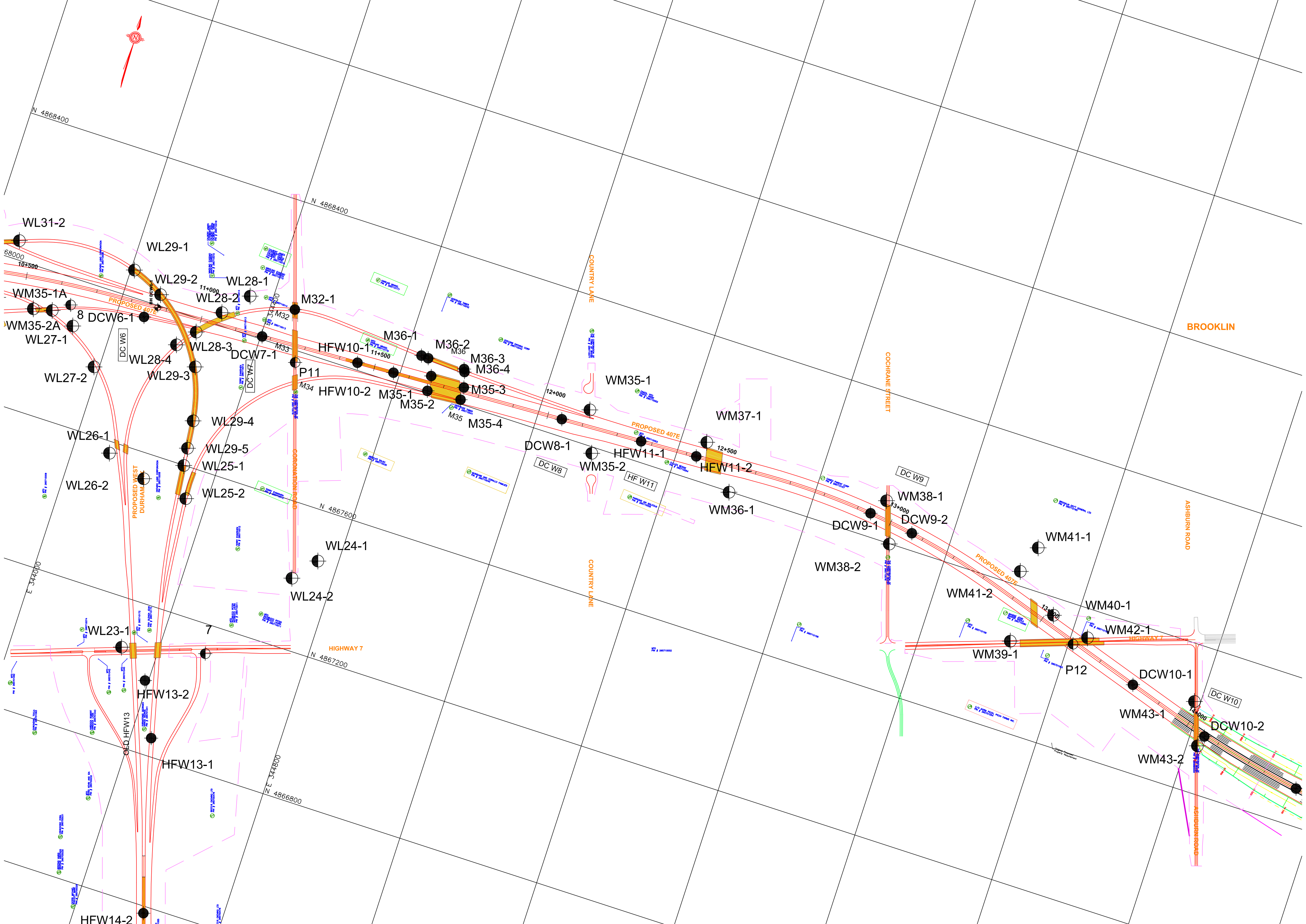
HIGHWAY 407 EAST EXTENSION

WESTERN SECTION

BOREHOLE LOCATION – MAINLINE

East of Coronation Rd to Ashburn Rd

PML Peto MacCallum Ltd.
CONSULTING ENGINEERS



NOTES

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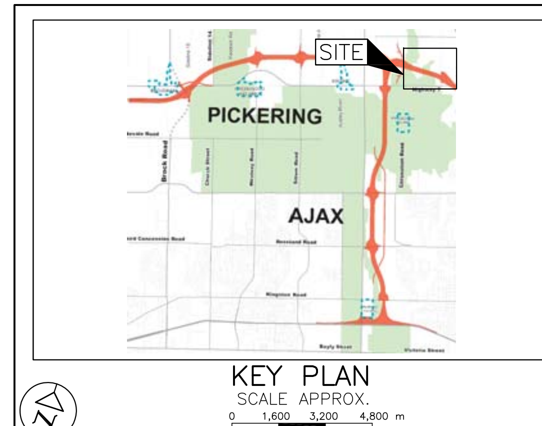
PLAN

SCALE

120 0 120 240 m



KEY PLAN

SCALE APPROX.

0 1,600 3,200 4,800 m

LEGEND

Borehole – Current Investigation

Borehole – MTO Geocres

DC-W10 Deep Cut Section

HF-W13 High Fill Section

No.	ELEVATION	CO-ORDINATES	
		NORTHING	EASTING
M32–1	170.4	4868107.1	344467.5
M35–1	155.4	4868049.0	344885.1
M35–2	155.3	4868006.3	344887.8
M35–3	162.8	4868046.5	344981.3
M35–4	162.1	4868011.4	344983.7
M36–1	156.0	4868094.4	344841.8
M36–2	155.7	4868093.2	344861.8
M36–3	162.5	4868095.6	344966.0
M36–4	163.5	4868088.0	344969.7
DCW8–1	172.2	4868047.1	345267.5
DCW9–1	167.1	4868064.1	346164.5
DCW9–2	167.5	4868046.8	346290.9
DCW10–1	165.7	4867836.5	347005.7
DCW10–2	163.1	4867761.3	347239.1
HFW10–1	156.8	4868020.5	344678.7
HFW10–2	158.2	4868025.1	344782.4
HFW11–1	153.6	4868056.8	345496.2
HFW11–2	153.1	4868064.5	345654.8

REFERENCE

Base plan and profiles provided in digital format by MTO, drawing file nos. "407E Western Section PRELIMINARY DESIGN_ULTIMATE.dwg", received October 23, 2010.

NO.	DATE	BY	REVISION
1	07/05/11	GD	BOREHOLE LOCATIONS UPDATED

Geocres No. 30M15–112

HWY. 407E	PROJECT NO. 10TF023		DIST. Central
SUBM'D. NA	CHKD. GD	DATE: Feb. 23, 2011	SITE:
DRAWN: AL	CHKD. CN	APPD. BRG	DWG. 6

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

W.O. No. 07–20015

SHEET

HIGHWAY 407 EAST EXTENSION

WESTERN SECTION

BOREHOLE LOCATION – WDL

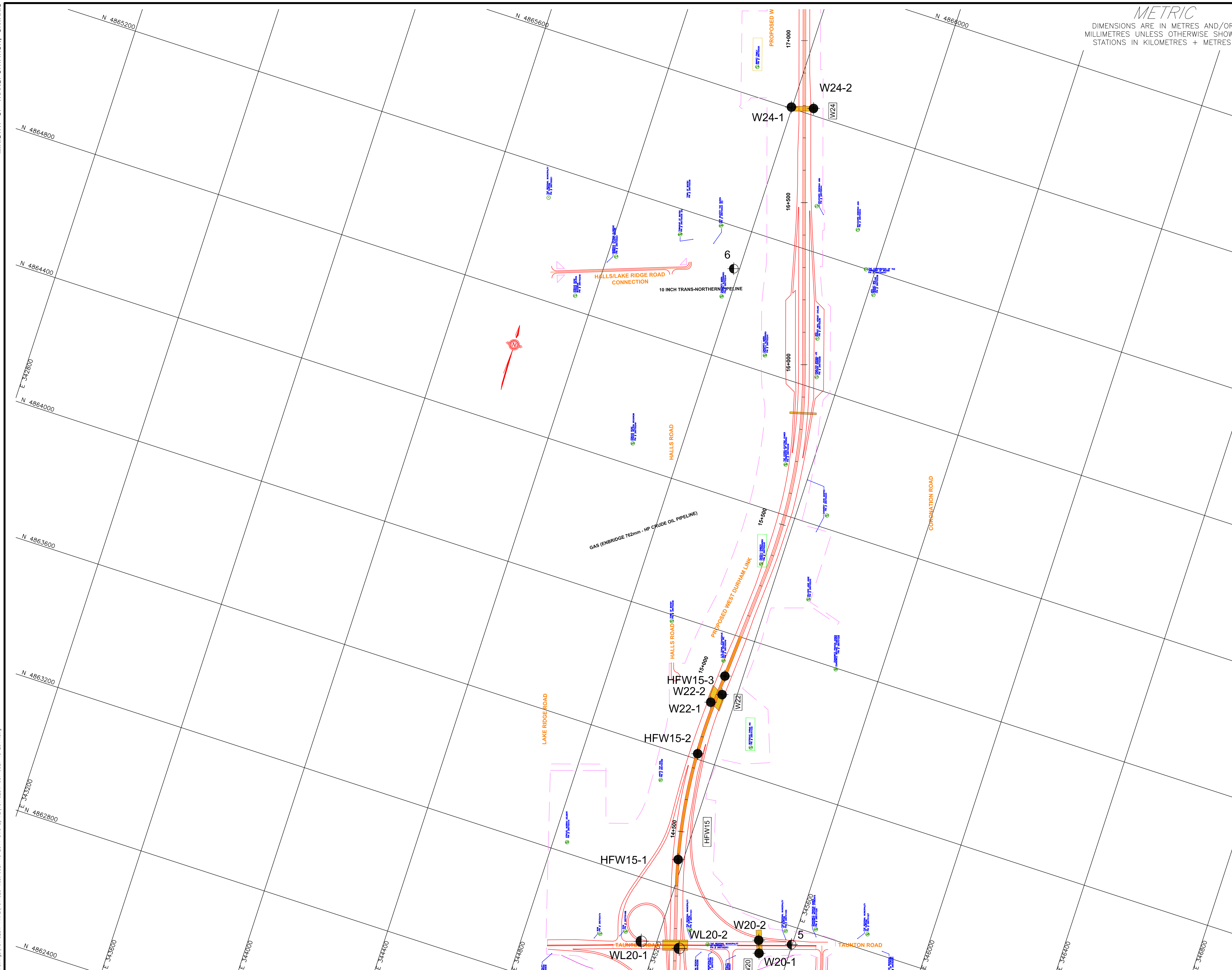
Winchester Rd West to Taunton Rd

PML Peto MacCallum Ltd.
CONSULTING ENGINEERS

KEY PLAN

SCALE APPROX.

0 1,600 3,200 4,800 m



PLAN

SCALE

120 0 120 240 m

LEGEND

Borehole – Current Investigation

Borehole – MTO Geocres

HF-W15 High Fill Section

No.	ELEVATION	CO-ORDINATES	
		NORTHING	EASTING
W20–1	109.2	4863089.6	345511.1
W20–2	110.2	4863127.7	345497.7
W22–1	119.2	4863780.3	345129.4
W22–2	114.8	4863813.5	345155.5
W24–1	138.1	4865603.9	344797.5
W24–2	138.3	4865621.0	344863.7
HFW15–1	113.3	4863287.5	345184.1
HFW15–2	116.9	4863616.6	345140.3
HFW15–3	116.4	4863870.5	345146.0

NOTES

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

REFERENCE

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NO.	DATE	BY	REVISION
1	07/05/11	GD	BOREHOLE LOCATIONS UPDATED

Geocres No. 30M15–112

HWY. 407		PROJECT NO. 10TF023	DIST. Central
SUBM'D. NA	CHKD. GD	DATE: Feb. 23, 2011	SITE:
DRAWN: AL	CHKD. CN	APPD. BRG	DWG. 7

PLOT DATE: January 28, 2011
FILENAME: Z:\10TF023\Hwy 407E Drawings\10TF023W-FINAL\10TF023 West JULY 13 2011.DWG By AL\10TF023W-007 JULY 13 2011.dwg

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

W.O. No. 07–20015

SHEET

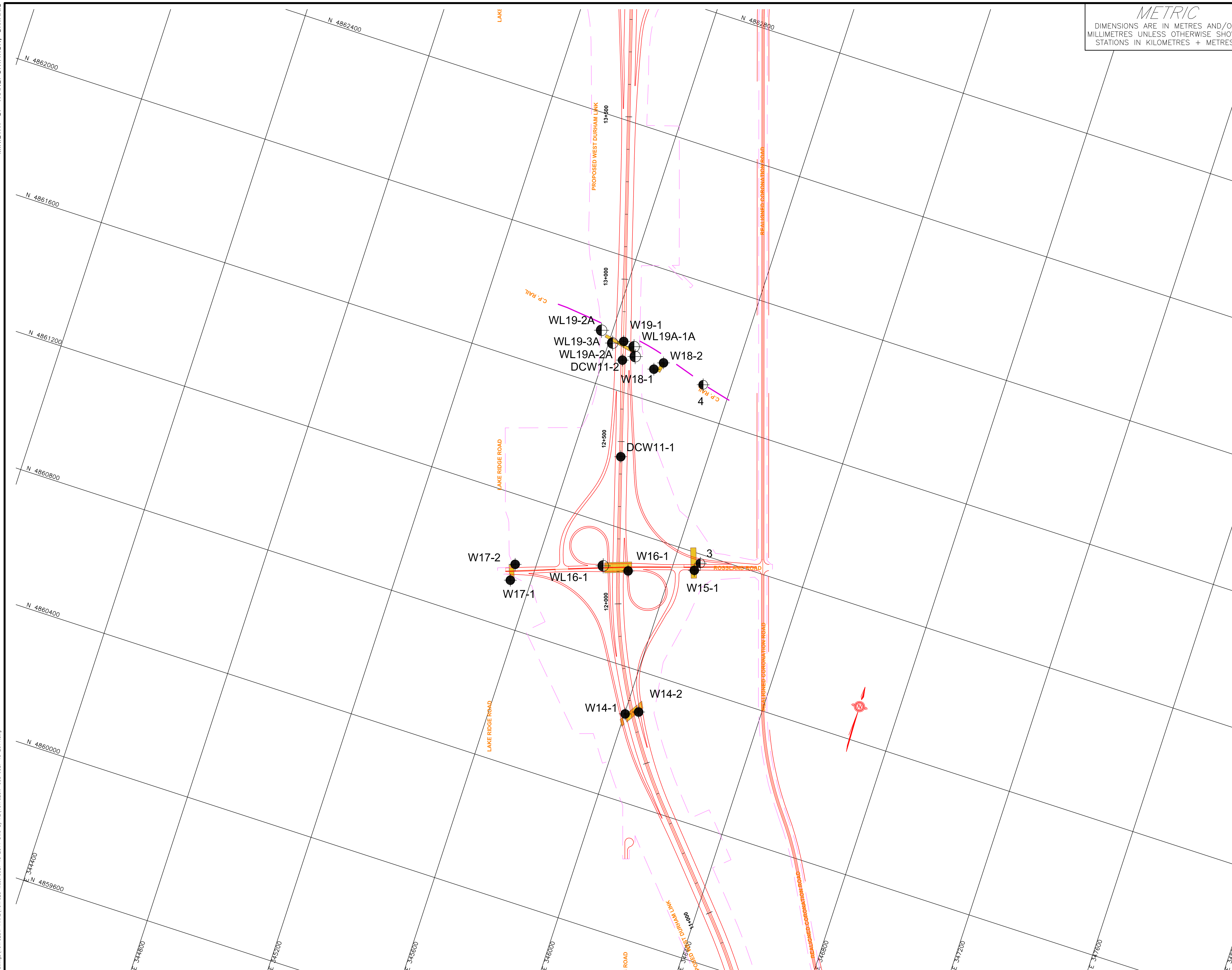
HIGHWAY 407 EAST EXTENSION

WESTERN SECTION

BOREHOLE LOCATION – WDL

Taunton Rd to South of Rossland Rd

Peto MacCallum Ltd.
CONSULTING ENGINEERS



PLAN

SCALE

120 0 120 240 m

KEY PLAN

SCALE APPROX.

0 1,600 3,200 4,800 m

LEGEND

- Borehole – Current Investigation
- Borehole – MTO Geocres
- DC-W11 Deep Cut Section

No.	ELEVATION	CO–ORDINATES	
		NORTHING	EASTING
W14–1	91.1	4860660.6	346001.8
W14–2	92.5	4860679.0	346039.5
W15–1	100.8	4861146.9	346067.5
W16–1	98.8	4861082.1	345874.2
W17–1	92.8	4860942.7	345538.4
W17–2	92.9	4860993.7	345537.4
W18–1	100.2	4861697.3	345757.3
W18–2	99.2	4861724.9	345779.6
W19–1	103.4	4861749.6	345642.6
DCW11–1	102.7	4861409.3	345744.0
DCW11–2	103.0	4861693.7	345658.8

NOTES

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REFERENCE

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NO.	DATE	BY	REVISION
1	07/05/11	GD	BOREHOLE LOCATIONS UPDATED

Geocres No. 30M15–112

HWY. 407E		PROJECT NO. 10TF023	DIST. Central
SUBM'D. NA	CHKD. GD	DATE: Feb. 23, 2011	SITE:
DRAWN: AL	CHKD. CN	APPD. BRG	DWG. 8

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

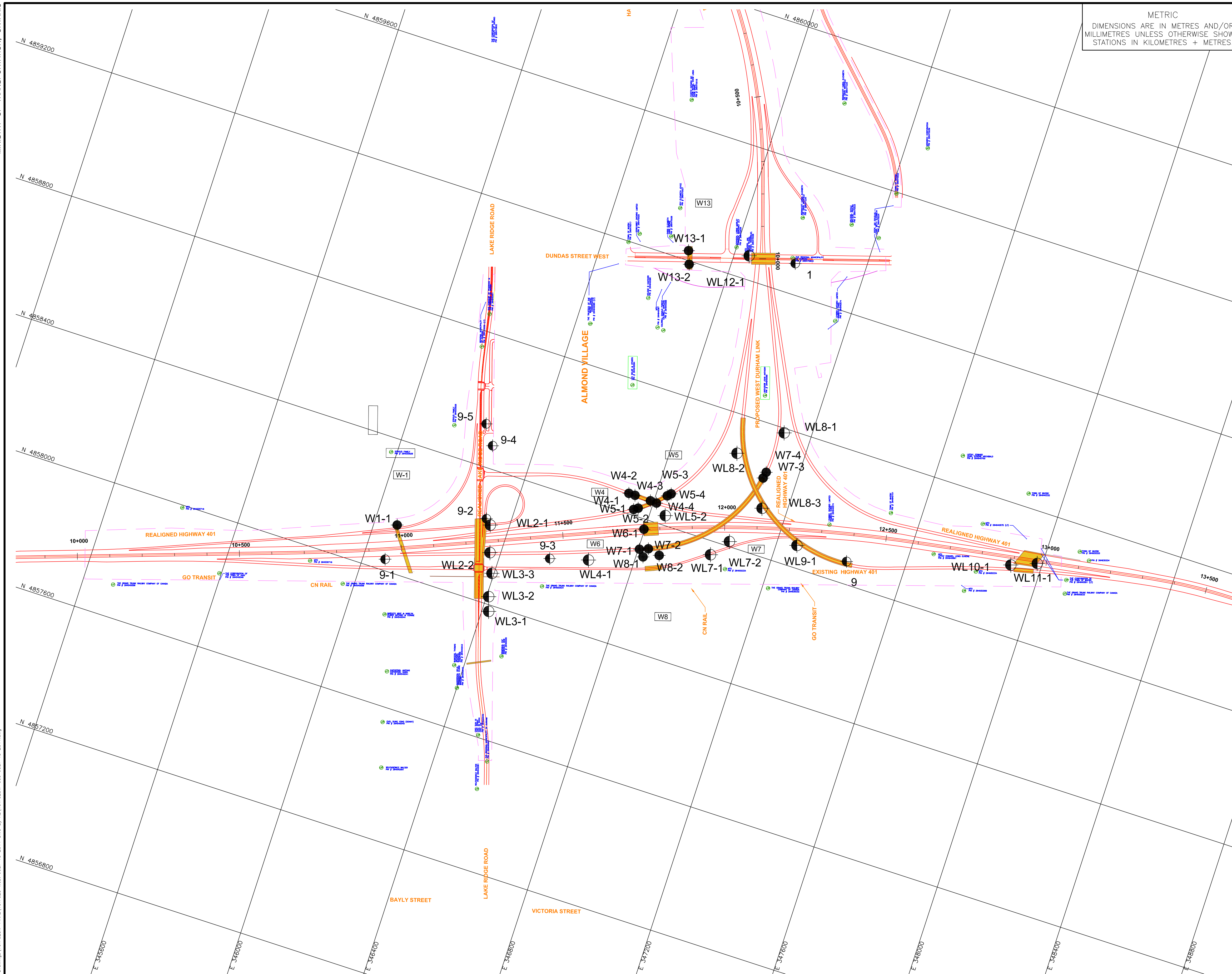
W.O. No. 07–20015

SHEET

HIGHWAY 407 EAST EXTENSION

WESTERN SECTION
BOREHOLE LOCATION – WDL
South of Rossland Rd to Highway 401

Peto MacCallum Ltd.
CONSULTING ENGINEERS



PLAN
SCALE
120 0 120 240 m

KEY PLAN
SCALE APPROX.
0 1,600 3,200 4,800 m

LEGEND

Borehole – Current Investigation

Borehole – MTO Geocres

No.	ELEVATION	CO–ORDINATES	
		NORTHING	EASTING
W1–1	83.9	4858146.2	346069.5
W4–1	79.3	4858460.6	346719.1
W4–2	79.3	4858460.7	346739.0
W4–3	79.2	4858459.3	346789.4
W4–4	79.1	4858458.5	346809.4
W5–1	79.1	4858417.8	346748.5
W5–2	79.3	4858426.0	346760.6
W5–3	79.4	4858490.0	346834.0
W5–4	79.9	4858499.6	346843.7
W6–1	78.7	4858369.8	346797.6
W7–1	78.7	4858306.7	346803.5
W7–2	78.8	4858316.6	346829.3
W7–3	82.3	4858633.8	347098.3
W7–4	82.8	4858653.5	347102.0
W8–1	77.7	4858287.7	346821.3
W8–2	77.6	4858307.5	346867.3
W13–1	83.9	4859229.4	346662.5
W13–2	82.0	4859190.1	346677.2

NOTES

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REFERENCE

Base plan and profiles provided in digital format by MTO, drawing file nos. "407E Western Section PRELIMINARY DESIGN_ULTIMATE.dwg", received October 23, 2010.

NO.	DATE	BY	REVISION
1	07/05/11	GD	BOREHOLE LOCATIONS UPDATED

Geocres No. 30M15–112

HWY. 407E		PROJECT NO. 10TF023	DIST. Central
SUBM'D. NA	CHKD. GD	DATE: Feb. 23, 2011	SITE:
DRAWN: AL	CHKD. CN	APPD. BRG	DWG. 9

APPENDIX A

RECORD OF BOREHOLE SHEETS

Ontario

Peto MacCallum Ltd.
CONSULTING ENGINEERS
Foundation Design

RECORD OF BOREHOLE No M6-1

METRIC

G.W.P. 07-20015 | LOCATION Coords: 4 864 539.0 N; 336 658.1 E | ORIGINATED BY Z.I

DIST Central HWY 407E | BOREHOLE TYPE Continuous Flight Solid Stem Augers | COMPILED BY N.S.B.

DATUM Geodetic | DATE December 02, 2010 | CHECKED BY G.D.

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	GR	SA	SI	CL
191.3	Ground Surface								
0.0	Topsoil	1	SS	5	191				
191.0 / 0.3	Silty sand, some clay, trace gravel, organics. Loose Brown Moist to wet	2	SS	5	190				
		3	SS	7		2	47	38	13
189.1	sand seams								
189.1 / 2.2	Sand and silt, trace clay. Loose to compact Brown Moist to wet	4	SS	10	189				
		5	SS	9	188				
	silt seams	6	SS	23	187				
	Very dense Grey	7	SS	70	186	0	43	48	9
		8	SS	83	185				
		9	SS	100/13cm	184, 183				
182.8 / 8.5	Silt, some clay, trace sand. Very dense Grey Moist (TILL)	10	SS	100/15cm	182	0	9	72	19
		11	SS	101/25cm	181				
180.2 / 11.1	End of borehole								

Samples 9, 10 & 11: Sampler bouncing

* Borehole dry

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

Ontario

Peto MacCallum Ltd.
CONSULTING ENGINEERS
Foundation Design

RECORD OF BOREHOLE No M6-2

METRIC

G.W.P. 07-20015 | LOCATION Coords: 4 864 477.2 N; 336 696.0 E | ORIGINATED BY Z.I

DIST Central HWY 407E | BOREHOLE TYPE Continuous Flight Solid Stem Augers | COMPILED BY N.S.B.

DATUM Geodetic | DATE February 28, 2011 | CHECKED BY G.D.

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE
192.6	Ground Surface				
0.0	Topsoil	1	SS	11	192
192.0 / 0.6	Silty sand, trace gravel. Dense to very dense Brown Moist	2	SS	50	
		3	SS	81	191
190.4 / 2.2	Sand, trace to with gravel. Very dense Brown Damp	4	SS	100/28cm	190
		5	SS	100/18cm	
	cobbles	6	SS	100/25cm	189
		7	SS	100/28cm	188
		8	SS	100/15cm	187
187.0 / 5.6	End of borehole				

Samples 4, 5, 6, 7 and 8: Sampler bouncing

* Borehole dry

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

Ontario

Peto MacCallum Ltd. Consulting Engineers

Foundation Design

RECORD OF BOREHOLE No M9-1

METRIC

G.W.P. 07-20015 | LOCATION Coords: 4 864 736.0 N; 337 366.1 E | ORIGINATED BY Z.I

DIST Central HWY 407E | BOREHOLE TYPE Continuous Flight Solid Stem Augers | COMPILED BY N.S.B.

DATUM Geodetic | DATE February 28, 2011 | CHECKED BY G.D.

ELEV / DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
170.5	Ground Surface				
0.0	Topsoil	1	SS	11	
170.2 / 0.3	Clayey silt, some sand, trace gravel. Stiff Brown Moist				
169.7 / 0.8	Sand, with gravel. Dense to very dense Brown Wet	2	SS	37	
		3	SS	74	
	some silt, trace clay, trace gravel	4	SS	31	8 74 14 4
		5	SS	41	
	with gravel, trace silt, cobbles	6	SS	100	
		7	SS	60	26 64 8 2
		8	SS	52	
		9	SS	69	
161.9 / 8.6	Sand, with silt, with gravel, trace clay. Very dense Grey Wet (TILL)	10	SS	104	22 43 28 7
		11	SS	100/25cm	
		12	SS	100/10cm	
158.1 / 12.4	End of borehole				

Elevation scale: 170, 169, 168, 167, 166, 165, 164, 163, 162, 161, 160, 159

Ground water conditions: ▼* (water level measured after drilling) at approx. 170.0; ▽* (water level observed during drilling) at approx. 169.0

Dynamic cone penetration resistance plot 20 40 60 80 100; Shear strength kPa: ○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE; 20 40 60 80 100

Plastic limit w_P, natural moisture content w, liquid limit w_L; WATER CONTENT (%) 20 40 60

UNIT WEIGHT γ kN/m³

Samples 11 and 12: Sampler bouncing

* 2011 02 28

▽ Water level observed during drilling

▼ Water level measured after drilling

$+^3, \times^5$: Numbers refer to Sensitivity; 20 / 15 –○– 5 / 10 (%) STRAIN AT FAILURE

Ontario

Peto MacCallum Ltd. Consulting Engineers

Foundation Design

RECORD OF BOREHOLE No M10-1

METRIC

G.W.P. 07-20015 | LOCATION Coords: 4 864 816.7 N; 337 429.7 E | ORIGINATED BY Z.I

DIST Central HWY 407E | BOREHOLE TYPE Continuous Flight Solid Stem Augers | COMPILED BY N.S.B.

DATUM Geodetic | DATE February 28, 2011 | CHECKED BY G.D.

ELEV / DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
166.2	Ground Surface				
0.0	Topsoil	1	SS	5	
166.0 / 0.2	Sandy silt with clay, trace gravel. Loose Brown Wet (FILL)				
165.5 / 0.7	Clayey silt, sandy, trace gravel. Very stiff Brown Wet to moist	2	SS	21	
		3	SS	26	8 35 39 18
	sand seams. Hard (TILL)	4	SS	56	
163.2 / 3.0	Sand and gravel, trace silt. Dense to very dense Brown Wet	5	SS	37	44 48 (8)
		6	SS	58	
	shale fragments	7	SS	57	
		8	SS	75	
158.8 / 7.4	Sand, some silt, some gravel, trace clay. Very dense Brown Wet	9	SS	90	18 67 11 4
		10	SS	100/28cm	
		11	SS	100/15cm	
155.4 / 10.8	End of borehole				

Elevation scale: 166, 165, 164, 163, 162, 161, 160, 159, 158, 157, 156

Ground water conditions: ▼* (water level measured after drilling) at approx. 164.8; ▽* (water level observed during drilling) at approx. 163.0

Dynamic cone penetration resistance plot 20 40 60 80 100; Shear strength kPa: ○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE; 20 40 60 80 100

Plastic limit w_P, natural moisture content w, liquid limit w_L; WATER CONTENT (%) 20 40 60

UNIT WEIGHT γ kN/m³

Samples 10 and 11: Sampler bouncing

* 2011 02 28

▽ Water level observed during drilling

▼ Water level measured after drilling

$+^3, \times^5$: Numbers refer to Sensitivity; 20 / 15 –○– 5 / 10 (%) STRAIN AT FAILURE

Ontario | Peto MacCallum Ltd. CONSULTING ENGINEERS | Foundation Design

RECORD OF BOREHOLE No M17-1

METRIC

G.W.P. 07-20015 LOCATION Coords: 4 866 799.9 N; 340 112.5 E ORIGINATED BY A.L.
DIST Central HWY 407E BOREHOLE TYPE C.F.S.S.A. and Wash Boring COMPILED BY N.S.B.
DATUM Geodetic DATE May 02 to 04, 2011 CHECKED BY G.D.

ELEV / DEPTH	DESCRIPTION (SOIL PROFILE)	SAMPLE NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL
155.8	Ground Surface							
0.0	Topsoil	1	SS	4				
155.5 / 0.3	Clayey silt, trace sand rootlets; Firm Brown Wet							
155.1 / 0.7	Clayey silt trace sand, trace gravel; Very stiff to stiff Grey Moist to wet (TILL)	2	SS	14				
		3	SS	10				
		4	SS	3	2	4	54	40
		5	SS	9				
152.1 / 3.7	Sandy silt trace clay, trace gravel; Loose to compact Grey Wet	6	SS	9				
		7	SS	2				
		8	SS	19	1	31	61	7
		9	SS	20				
146.8 / 9.0	Clayey silt, trace sand layers of silt and sandy silt; Stiff to hard Grey Moist to wet (TILL)	10	SS	9				
		11	SS	92/23cm				
		12	SS	78	0	1	75	24
142.3 / 13.5	Silty sand trace clay, trace gravel sand layers; Very dense to dense Grey Wet (TILL) Cont'd	13	SS	50/13cm				
140.8								

Groundwater conditions: water level symbol ∇* at about El. 155.3 (between samples 1 and 2).

Shear strength (kPa) values printed beside the plot: 150 (sample 2), 125 (sample 3).

Elevation scale: 155, 154, 153, 152, 151, 150, 149, 148, 147, 146, 145, 144, 143, 142, 141

Column headings: DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100; PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

$+^3, \times^3$: Numbers refer to Sensitivity; 15 –○– 5 (20 above, 10 below): (%) STRAIN AT FAILURE

Ontario | Peto MacCallum Ltd. CONSULTING ENGINEERS | Foundation Design

RECORD OF BOREHOLE No M17-1

METRIC

G.W.P. 07-20015 LOCATION Coords: 4 866 799.9 N; 340 112.5 E ORIGINATED BY A.L.
DIST Central HWY 407E BOREHOLE TYPE C.F.S.S.A. and Wash Boring COMPILED BY N.S.B.
DATUM Geodetic DATE May 02 to 04, 2011 CHECKED BY G.D.

ELEV / DEPTH	DESCRIPTION (SOIL PROFILE)	SAMPLE NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL
140.8 / 15.0	Silty sand trace clay, trace gravel sand layers; Very dense to dense Grey Wet (TILL) (Cont'd)	14	SS	86				
		15	SS	40				
	cobbles and boulders	16	SS	47				
		17	SS	50/8cm				
		18	SS	50/8cm				
131.3 / 24.5	End of borehole							

Elevation scale: 140, 139, 138, 137, 136, 135, 134, 133, 132

Samples 11, 13, 17 and 18: Sampler bouncing

2011 05 02

∇ Water level observed during drilling

■ Penetrometer test

C.F.S.S.A. denotes Continuous Flight Solid Stem Augers

Note: Artesian groundwater conditions encountered at a depth of 13.7m (El. 142.1m). Slow water flow, estimated head of 1.0m above ground surface. Sealed with 1 bag of cement in addition to bentonite.

$+^3, \times^3$: Numbers refer to Sensitivity; 15 –○– 5 (20 above, 10 below): (%) STRAIN AT FAILURE

Ontario — Peto MacCallum Ltd. Consulting Engineers — Foundation Design

RECORD OF BOREHOLE No M17-2

METRIC

G.W.P. 07-20015 LOCATION Coords: 4 866 827.2 N; 340 166.8 E ORIGINATED BY A.L.

DIST Central HWY 407E BOREHOLE TYPE C.F.S.S.A. and Wash Boring COMPILED BY N.S.B.

DATUM Geodetic DATE April 27 and 29, 2011 CHECKED BY G.D.

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS
156.3	Ground Surface				
156.0 / 0.0	Topsoil	1	SS	5	
155.6 / 0.3	Clayey silt, trace sand; Firm, Brown, Wet				
0.7	Clayey silt, trace sand, trace gravel; Stiff, Brown, Wet to moist (TILL)	2	SS	6	
		3	SS	16	
	sandy silt layers; Very stiff, Grey	4	SS	21	∇*
153.7 / 2.6	Sandy silt, trace clay, trace gravel; Compact to loose, Grey, Wet	5	SS	15	
		6	SS	8	
		7	SS	6	
150.4 / 5.9	Clayey silt, trace sand, trace gravel, layers of silt and sandy silt; Very stiff to stiff, Grey, Moist to wet (TILL)	8	SS	12	
		9	SS	17	
		10	SS	7	
		11	SS	8	
143.9 / 12.4	Silty sand, trace clay, trace gravel, sand layers; Dense to very dense, Grey, Wet (TILL)	12	SS	32	
		13	SS	79	
141.3	Cont'd				

Elevation scale: 156, 155, 154, 153, 152, 151, 150, 149, 148, 147, 146, 145, 144, 143, 142

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE; 20 40 60 80 100 (penetrometer values: 150 at sample 3; 150 at sample 8)

PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60

UNIT WEIGHT γ kN/m³

REMARKS & GRAIN SIZE DISTRIBUTION (%): GR SA SI CL

$+^{7}, \times^{5}$: Numbers refer to Sensitivity; 20 / 15 –○– 5 / 10 (%) STRAIN AT FAILURE

Ontario — Peto MacCallum Ltd. Consulting Engineers — Foundation Design

RECORD OF BOREHOLE No M17-2

METRIC

G.W.P. 07-20015 LOCATION Coords: 4 866 827.2 N; 340 166.8 E ORIGINATED BY A.L.

DIST Central HWY 407E BOREHOLE TYPE C.F.S.S.A. and Wash Boring COMPILED BY N.S.B.

DATUM Geodetic DATE April 27 and 29, 2011 CHECKED BY G.D.

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES
141.3 / 15.0	Silty sand, trace clay, trace gravel, sand layers, cobbles; Very dense, Grey, Wet (TILL) (Cont'd)	14	SS	80
		15	SS	50/10cm
		16	SS	100
		17	SS	90
		18	SS	50/10cm
		19	SS	50/10cm
	with gravel, cobbles and boulders	20	SS	50/10cm
131.7 / 24.6	End of borehole			

Elevation scale: 141, 140, 139, 138, 137, 136, 135, 134, 133, 132

Samples 15, 18, 19 and 20: Sampler bouncing

2011 04 27

∇ Water level observed during drilling

■ Penetrometer test

C.F.S.S.A. denotes Continuous Flight Solid Stem Augers

Note: Artesian groundwater conditions encountered at a depth of 12.4m (El. 143.9m). Slow water flow, estimated head of 1.0m above ground surface. Sealed with 1 bag of cement in addition to bentonite.

$+^{7}, \times^{5}$: Numbers refer to Sensitivity; 20 / 15 –○– 5 / 10 (%) STRAIN AT FAILURE

Ontario | Peto MacCallum Ltd. Consulting Engineers | Foundation Design

RECORD OF BOREHOLE No M18-1 — 1 of 1 — METRIC

G.W.P.	07-20015	LOCATION	Coords: 4 866 960.1 N; 340 465.4 E	ORIGINATED BY	F.P.
DIST	Central HWY 407E	BOREHOLE TYPE	C.F.S.S.A. and Wash Boring	COMPILED BY	N.S.B.
DATUM	Geodetic	DATE	April 25, 2011	CHECKED BY	G.D.

SOIL PROFILE

ELEV / DEPTH	DESCRIPTION
158.4	Ground Surface
0.0	Topsoil
157.9 / 0.5	Clayey silt, trace sand, trace gravel. Very stiff Brown/grey Moist (TILL) — — — silty sand layers. Hard Grey
154.7 / 3.7	Sand, trace silt, trace gravel. Very dense Grey Moist
150.7 / 7.7	End of borehole

Samples 7 to 10: Sampler bouncing

* Borehole dry

■ Penetrometer test

C.F.S.S.A. denotes Continuous Flight Solid Stem Augers

SAMPLES

NUMBER	TYPE	"N" VALUES	REMARKS
1	SS	5	
2	SS	23	Penetrometer 163
3	SS	23	
4	SS	31	
5	SS	38	
6	SS	68	
7	SS	71/20cm	
8	SS	50/10cm	
9	SS	50/8cm	
10	SS	50/8cm	

Elevation scale: 158, 157, 156, 155, 154, 153, 152, 151

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100; PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

+3, ×3: Numbers refer to Sensitivity; 20 / 15 –○– 5 / 10 (%) STRAIN AT FAILURE

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RECORD OF BOREHOLE No M18-2 — 1 of 2 — METRIC

G.W.P.	07-20015	LOCATION	Coords: 4 866 935.7 N; 340 492.6 E	ORIGINATED BY	F.P.
DIST	Central HWY 407E	BOREHOLE TYPE	C.F.S.S.A. and Wash Boring	COMPILED BY	N.S.B.
DATUM	Geodetic	DATE	April 18 to 21, 2011	CHECKED BY	G.D.

SOIL PROFILE

ELEV / DEPTH	DESCRIPTION
157.8	Ground Surface
0.0	Topsoil
157.5 / 0.3	Sand, trace silt, organic inclusions. Compact Brown Moist
157.3 / 0.5	Clayey silt, trace sand, trace gravel. Very stiff to stiff Grey Moist (TILL)
155.3 / 2.5	Sand, trace silt, trace gravel. Compact to very dense Grey Wet — — — cobbles and boulders
152.9 / 4.9	Clayey silt, trace sand, trace gravel. Hard Grey Moist (TILL) — — — silty sand layers, cobbles and boulders — — — gravelly sand layers

Cont'd

SAMPLES

NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	SS	10	
2	SS	26	Penetrometer 138
3	SS	23	2 7 50 41
4	SS	17	
5	SS	18	1 89 (10)
6	SS	50/15cm	
7	SS	60/13cm	
8	SS	50/8cm	
9	SS	50/15cm	
10	SS	50/8cm	
11	SS	50/15cm	
12	SS	87	13 26 32 29
13	SS	75	
14	SS	50/13cm	

Ground water: ∇* at approx. elev. 155.4

Elevation scale: 157, 156, 155, 154, 153, 152, 151, 150, 149, 148, 147, 146, 145, 144, 143

+3, ×3: Numbers refer to Sensitivity; 20 / 15 –○– 5 / 10 (%) STRAIN AT FAILURE

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RECORD OF BOREHOLE No M18-2

METRIC

G.W.P. 07-20015 LOCATION Coords: 4 866 935.7 N; 340 492.6 E ORIGINATED BY F.P.

DIST Central HWY 407E BOREHOLE TYPE C.F.S.S.A. and Wash Boring COMPILED BY N.S.B.

DATUM Geodetic DATE April 18 to 21, 2011 CHECKED BY G.D.

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%)
142.8						
	Clayey silt, trace sand gravelly sand layers Hard Grey Moist (TILL)	15	SS	65/8cm	142	
141.0 / 16.8		16	SS	70/8cm	141	
	End of borehole					

Samples 6 to 11 and 14 to 16: Sampler bouncing

Borehole backfilled and grouted April 25, 2011

2011 04 18 to 21

∇ Water level observed during drilling

■ Penetrometer test

C.F.S.S.A. denotes Continuous Flight Solid Stem Augers

Note: Artesian groundwater conditions encountered at a depth of 8.2m (Elev. 149.6m) on April 25, 2011. Slow water flow, estimated head of 0.5m above ground surface. Sealed with 1 bag of cement in addition to bentonite.

+ 7, × 5 : Numbers refer to Sensitivity; 15 –○– 5 (20 / 10) (%) STRAIN AT FAILURE

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RECORD OF BOREHOLE No W8-1

METRIC

G.W.P. 07-20015 LOCATION Coords: 4 858 287.7 N; 346 821.3 E ORIGINATED BY A.L.

DIST Central HWY 407E BOREHOLE TYPE C.F.H.S.A. and NQ Diamond Coring COMPILED BY N.S.B

DATUM Geodetic DATE February 8 & 9, 2011 CHECKED BY G.D.

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
77.7	Ground Surface						
0.0	Topsoil	1	SS	6			
77.4 / 0.3	Clayey silt, trace sand rootlets, topsoil inclusions Firm to stiff Brown Wet to moist	2	SS	7		77	
		3	SS	5	∇*	76	
75.6 / 2.1	Silty sand some clay, trace gravel Loose Grey Wet	4	SS	8		75	
		5	SS	8		74	
	some gravel	6	SS	9		73	
71.8 / 5.9	Sand and silt some clay, some gravel Dense Grey Wet (TILL)	7	SS	39		72, 71	14 38 36 12
70.0 / 7.7	Shale bedrock Weathered Low strength Poor quality (Whitby Formation)	8	SS	50/5cm		70	
		9	RC NQ	REC 100%		69	RQD 33%
		10	RC NQ	REC 100%		68	RQD 50%
		11	RC NQ	REC 100%		67	RQD 47%
66.3 / 11.4	End of borehole						

Sample 8: Sampler bouncing

* 2011 02 08 & 09

∇ Water level observed during drilling

■ Penetrometer test

C.F.H.S.A. denotes Continuous Flight Hollow Stem Augers

+ 7, × 5 : Numbers refer to Sensitivity; 15 –○– 5 (20 / 10) (%) STRAIN AT FAILURE

Ontario

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RECORD OF BOREHOLE No W8-2

METRIC

G.W.P. 07-20015 LOCATION Coords: 4 858 307.5 N; 346 867.3 E ORIGINATED BY A.L.

DIST Central HWY 407E BOREHOLE TYPE C.F.H.S.A. and NQ Diamond Coring COMPILED BY N.S.B

DATUM Geodetic DATE February 7 & 8, 2011 CHECKED BY G.D.

ELEV / DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
77.6	Ground Surface				
77.3 / 0.0	Topsoil	1	SS	4	
76.7 / 0.3	Clayey silt, trace sand rootlets Firm Brown Wet				
0.9	Silty sand, some clay Very loose Grey Wet	2	SS	4	
		3	SS	WH**	
		4	SS	3	0 45 37 18
74.7 / 2.9	Clayey silt, some sand Stiff to firm Grey Wet	5	SS	6	0 15 36 49
			FV		
	silty clay layers	6	SS	4	
			FV		
71.7 / 5.9	Sand, with silt some clay, some gravel Loose to dense Grey Wet (TILL)	7	SS	8	15 46 28 11
		8	SS	50/13cm	
69.9 / 7.7	Shale bedrock Weathered Low strength Fair quality (Whitby Formation)	9	RC NQ	REC 100%	RQD 63%
		10	RC NQ	REC 95%	RQD 53%
66.6 / 11.0	End of borehole				

Column headings: STRAT PLOT; GROUND WATER CONDITIONS; ELEVATION SCALE (77 – 67); DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100; PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L, WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m³

Plotted values: water content 115 (at ground surface); field vane sensitivity 4 and 3.

Sample 8: Sampler bouncing

* 2011 02 07 & 08

∇ Water level observed during drilling

■ Penetrometer test

WH** denotes penetration due to weight of rods and hammer

C.F.H.S.A. denotes Continuous Flight Hollow Stem Augers

Cont'd

$+^{7}, \times^{5}$: Numbers refer to Sensitivity; 15–○–5 (20 / 10) (%) STRAIN AT FAILURE

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RECORD OF BOREHOLE No W8-2

METRIC

G.W.P. 07-20015 LOCATION Coords: 4 858 307.5 N; 346 867.3 E ORIGINATED BY A.L.

DIST Central HWY 407E BOREHOLE TYPE C.F.H.S.A. and NQ Diamond Coring COMPILED BY N.S.B

DATUM Geodetic DATE February 7 & 8, 2011 CHECKED BY G.D.

Piezometer Readings:

Date	Depth (m)	Elev.
Feb. 11,'11	1.0	76.6
Apr. 01,'11	0.9	76.7

Piezometer Legend:

- Bentonite seal
- Filter sand
- 25mm dia. PVC screen
- Bentonite bed

$+^{7}, \times^{5}$: Numbers refer to Sensitivity; 15–○–5 (20 / 10) (%) STRAIN AT FAILURE

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RECORD OF BOREHOLE No W18-1

 METRIC

G.W.P. 07-20015 | LOCATION Coords: 4 861 697.3 N; 345 757.3 E | ORIGINATED BY Z.I.

DIST Central HWY 407E | BOREHOLE TYPE Continuous Flight Solid Stem Augers | COMPILED BY N.S.B.

DATUM Geodetic | DATE March 01, 2011 | CHECKED BY G.D.

ELEV / DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
100.2	Ground Surface				
0.0	Topsoil	1	SS	8	
99.6 / 0.6	Silty clay, trace sand; Firm, Grey, Wet	2	SS	6	
			FV		
		3	SS	4	
	Soft		FV		
97.9 / 2.3	Silty clay, trace to with sand, trace to some gravel; Stiff to very stiff, Grey, Wet (TILL)	4	SS	19	
97.2 / 3.0	Silty sand, trace clay, trace gravel; Dense, Grey, Wet	5	SS	48	
96.5 / 3.7	Clayey silt, with sand, some gravel; Hard, Grey, Moist	6	SS	100/23cm	
	shale fragments (TILL)	7	SS	100/10cm	
		8	SS	100/15cm	
		9	SS	100/15cm	
92.4 / 7.8	End of borehole				

Samples 6, 7, 8 and 9: Sampler bouncing

2011 03 01

▽ Water level observed during drilling

▼ Water level measured after drilling

■ Penetrometer test

Water Level Readings:

Date	Depth (m)	Elev.
Mar. 11, 2011	2.0	98.2
Apr. 01, 2011	1.7	98.5

Piezometer Legend:

- Flush Mount Casing
- Cement concrete
- Bentonite Seal
- 25mm dia. PVC Screen

Chart headings: GROUND WATER CONDITIONS; ELEVATION SCALE; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: ○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE; PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m³

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

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RECORD OF BOREHOLE No W18-2

 METRIC

G.W.P. 07-20015 | LOCATION Coords: 4 861 724.9 N; 345 779.6 E | ORIGINATED BY A.L.

DIST Central HWY 407E | BOREHOLE TYPE Continuous Flight Solid Stem Augers | COMPILED BY N.S.B.

DATUM Geodetic | DATE December 22, 2010 | CHECKED BY G.D.

ELEV / DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
99.2	Ground Surface				
0.0	Topsoil	1	SS	6	
98.8 / 0.4	Sandy silt, trace clay, trace gravel, topsoil and organic inclusions; Loose, Brown/black, Wet (ALLUVIUM)	2	SS	4	
		3	SS	9	
97.1 / 2.1	Sandy silt, some clay, trace gravel; Loose to compact, Grey, Wet	4	SS	8	
		5	SS	27	3 37 49 11
		6	SS	28	
93.7 / 5.5	Clayey silt, sandy, some gravel, cobbles and boulders; Hard, Grey, Moist (TILL)	7	SS	59	15 40 29 16
		8	SS	50/13cm	
	weathered shale	9	SS	50/13cm	
89.9 / 9.3	End of borehole				

Samples 8 and 9: Sampler bouncing

* 2010 12 22

▽ Water level observed during drilling

▼ Water level measured after drilling

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

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RECORD OF BOREHOLE No DCW1-1

METRIC

G.W.P. 07-20015 | LOCATION Coords: 4 865 174.6 N; 337 696.5 E | ORIGINATED BY A.K.

DIST Central HWY 407E | BOREHOLE TYPE Continuous Flight Hollow Stem Augers | COMPILED BY N.S.B.

DATUM Geodetic | DATE May 9 & 10, 2011 | CHECKED BY G.D.

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE
177.9	Ground Surface				
177.6 / 0.0	Topsoil				
0.3	Silty sand, trace clay rootlets Loose Brown Moist	1	SS	6	177
176.5 / 1.4	Sand, with silt to silty trace clay, trace gravel cobbles Compact to very dense Brown Moist (TILL)	2	SS	25	176
					175
		3	SS	81	174
		4	SS	50/10cm	173
		5	SS	50/5cm	172
171.0 / 6.9	Silty sand, trace clay Very dense Grey Moist				171
		6	SS	50/8cm	170
		7	SS	50/10cm	169
168.1 / 9.8	Sandy silt some clay, trace gravel cobbles Very dense Grey Moist (TILL)				168
		8	SS	50/8cm	167
					166
		9	SS	50/8cm	165
164.1 / 13.8	End of borehole	10	SS	50/13cm	

Cont'd

Column headings: SOIL PROFILE; SAMPLES; GROUND WATER CONDITIONS *; ELEVATION SCALE; DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE (20 40 60 80 100); PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) (20 40 60); UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

+[7], ×[5]: Numbers refer to Sensitivity; 15–○–5 (20, 10) (%) STRAIN AT FAILURE

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RECORD OF BOREHOLE No DCW1-1

METRIC

G.W.P. 07-20015 | LOCATION Coords: 4 865 174.6 N; 337 696.5 E | ORIGINATED BY A.K.

DIST Central HWY 407E | BOREHOLE TYPE Continuous Flight Hollow Stem Augers | COMPILED BY N.S.B.

DATUM Geodetic | DATE May 9 & 10, 2011 | CHECKED BY G.D.

* Borehole dry

Water Level Readings:

Date	Depth (m)	Elev.
May 16, 2011	0.8	177.1
May 24, 2011	0.9	177.0

Piezometer Legend:

- Bentonite seal
- Filter sand
- 25mm dia. PVC screen
- Filter bed

+[7], ×[5]: Numbers refer to Sensitivity; 15–○–5 (20, 10) (%) STRAIN AT FAILURE

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RECORD OF BOREHOLE No DCW1-2

METRIC

G.W.P. 07-20015 | LOCATION Coords: 4 865 339.4 N; 337 814.9 E | ORIGINATED BY A.L.

DIST Central HWY 407E | BOREHOLE TYPE Continuous Flight Hollow Stem Augers | COMPILED BY N.S.B.

DATUM Geodetic | DATE May 6 & 9, 2011 | CHECKED BY G.D.

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
189.7	Ground Surface				
0.0 / 189.4 / 0.3	Topsoil	1	SS	5	
	Silty sand, trace clay rootlets Loose Brown Moist				
188.3 / 1.4	Sand, with silt to silty some clay, trace gravel cobbles Compact to very dense Brown Moist (TILL)	2	SS	56	
		3	SS	25	7 53 27 13
		4	SS	65	
184.1 / 5.6	Sand trace to some silt Very dense Greyish brown Moist	5	SS	59	
		6	SS	52	0 88 (12)
		7	SS	95	
179.9 / 9.8	Sandy silt to silty sand some to with clay trace gravel cobbles and boulders Very dense Grey Moist (TILL)	8	SS	50/10cm	
		9	SS	50/13cm	
		10	SS	50/13cm	
174.7	Cont'd				

ON MTO_VER3 NEW LOGO 10TF023. HF N DCGPJ.GPJ ON_MOT.GDT 07/07/2011 10:43:34 AM

+ [3], × [3]: Numbers refer to Sensitivity; 15–○–5 (20/10) (%) STRAIN AT FAILURE

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RECORD OF BOREHOLE No DCW1-2

METRIC

G.W.P. 07-20015 | LOCATION Coords: 4 865 339.4 N; 337 814.9 E | ORIGINATED BY A.L.

DIST Central HWY 407E | BOREHOLE TYPE Continuous Flight Hollow Stem Augers | COMPILED BY N.S.B.

DATUM Geodetic | DATE May 6 & 9, 2011 | CHECKED BY G.D.

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
174.7 / 15.0	Sandy silt to silty sand some to with clay trace gravel cobbles and boulders Very dense Grey Moist (TILL) (Cont'd)	11	SS	50/10cm	4 34 40 22
		12	SS	50/10cm	
		13	SS	50/13cm	
		14	SS	50/13cm	
		15	SS	50/13cm	7 40 33 20
		16	SS	50/13cm	
		17	SS	50/10cm	
		18	SS	50/13cm	
163.5 / 26.2	End of borehole * Borehole dry				

ON MTO_VER3 NEW LOGO 10TF023. HF N DCGPJ.GPJ ON_MOT.GDT 07/07/2011 10:43:35 AM

+ [3], × [3]: Numbers refer to Sensitivity; 15–○–5 (20/10) (%) STRAIN AT FAILURE

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RECORD OF BOREHOLE No DCW11-1 — 1 of 1 — METRIC

G.W.P. 07-20015 — LOCATION Coords: 4 861 409.3 N; 345 744.0 E — ORIGINATED BY Z.I

DIST Central HWY 407E — BOREHOLE TYPE Continuous Flight Solid Stem Augers — COMPILED BY N.S.B.

DATUM Geodetic — DATE March 01, 2011 — CHECKED BY G.D

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
102.7	Ground Surface				
0.0 / 102.3	Topsoil	1	SS	22	
0.4	Clayey silt with sand, trace gravel Stiff Brown Wet	2	SS	13	4 24 40 32
101.3 / 1.4	Clayey silt some to with sand trace gravel Stiff Brown Moist (TILL)	3	SS	13	
		4	SS	22	
	Very stiff Grey to hard				
	shale fragments	5	SS	40	
		6	SS	50	
		7	SS	50/25cm	
	cobbles and boulders	8	SS	50/23cm	
		9	SS	50/10cm	4 29 54 13
94.2 / 8.5		10	SS	50/10cm	
	End of borehole				

Samples 7, 8, 9 and 10: Sampler bouncing

* Borehole dry

Water Level Readings:

Date	Depth (m)	Elev.
Mar. 11, 2011	2.3	100.4
Apr. 01, 2011	2.2	100.5

Piezometer Legend:

- Bentonite seal
- Filter sand
- 25mm dia. PVC screen
- Filter bed

ON MTO_VER3 NEW LOGO 10TF023. HF N DCGPJ.GPJ ON_MOT.GDT 07/07/2011 10:43:30 AM

+3, ×3: Numbers refer to Sensitivity — (%) STRAIN AT FAILURE

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RECORD OF BOREHOLE No DCW11-2 — 1 of 1 — METRIC

G.W.P. 07-20015 — LOCATION Coords: 4 861 693.7 N; 345 658.8 E — ORIGINATED BY Z.I

DIST Central HWY 407E — BOREHOLE TYPE Continuous Flight Solid Stem Augers — COMPILED BY N.S.B.

DATUM Geodetic — DATE March 01, 2011 — CHECKED BY G.D

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
103.0	Ground Surface				
0.0	Topsoil				
102.8 / 0.2	Sandy silt, some clay Compact Brown Wet	1	SS	12	
		2	SS	10	0 39 45 16
101.4 / 1.6	Silty clay, some sand clayey silt seams Stiff to very stiff Brown Moist to wet	3	SS	10	
		4	SS	31	
		5	SS	16	
99.3 / 3.7	Clayey silt some sand to sandy trace gravel Very stiff to hard Grey Moist (TILL)	6	SS	21	
		7	SS	34	
	shale fragments	8	SS	32	6 31 36 27
95.2 / 7.8		9	SS	50/13cm	
	End of borehole				

Sample 9: Sampler bouncing

* 2011 03 01

∇ Water level observed during drilling

▼ Water level measured after drilling

■ Penetrometer test

ON MTO_VER3 NEW LOGO 10TF023. HF N DCGPJ.GPJ ON_MOT.GDT 07/07/2011 10:43:32 AM

+3, ×3: Numbers refer to Sensitivity — (%) STRAIN AT FAILURE

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RECORD OF BOREHOLE No HFW6-1

METRIC

G.W.P. 07-20015 LOCATION Coords: 4 866 853.6 N; 340 197.9 E ORIGINATED BY F.P.

DIST Central HWY 407E BOREHOLE TYPE Continuous Flight Solid Stem Augers COMPILED BY N.S.B.

DATUM Geodetic DATE April 18, 2011 CHECKED BY G.D

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION / SHEAR STRENGTH kPa	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
157.9	Ground Surface									
0.0	Topsoil									
157.6 / 0.3	Clayey silt, trace sand Firm to hard, Brown, Moist (TILL)	1	SS	11						
		2	SS	7		157				
		3	SS	10		156	213			0 7 50 43
	sandy silt layers to 3.7m Grey	4	SS	12	∇*					
		5	SS	13	▼*	155				
		6	SS	32		154				
		7	SS	10		153				0 3 63 34
						152				
		8	SS	31						
151.3 / 6.6	End of borehole									

* 2011 04 18

∇ Water level observed during drilling

▼ Water level measured after drilling

■ Penetrometer test

+3, ×3: Numbers refer to Sensitivity; 20 / 15–○–5 / 10 (%) STRAIN AT FAILURE

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RECORD OF BOREHOLE No HFW7-1

METRIC

G.W.P. 07-20015 LOCATION Coords: 4 866 973.7 N; 340 562.4 E ORIGINATED BY F.P.

DIST Central HWY 407E BOREHOLE TYPE Continuous Flight Solid Stem Augers COMPILED BY N.S.B.

DATUM Geodetic DATE April 18, 2010 CHECKED BY G.D

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION / SHEAR STRENGTH kPa	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
160.8	Ground Surface									
0.0	Topsoil									
160.6 / 0.2	Clayey silt, trace sand, trace gravel Very stiff, Brown/grey, Moist (TILL)	1	SS	9						
		2	SS	14		160				2 9 63 26
		3	SS	31	▼*	159	125			
	Grey									
158.2 / 2.6	Silt with sand, trace clay Compact, Grey, Moist to wet	4	SS	27	∇*	158				
		5	SS	17		157				
		6	SS	9						0 28 66 6
		7	SS	19		156				
						155				
		8	SS	29						
154.2 / 6.6	End of borehole									

* 2011 04 18

∇ Water level observed during drilling

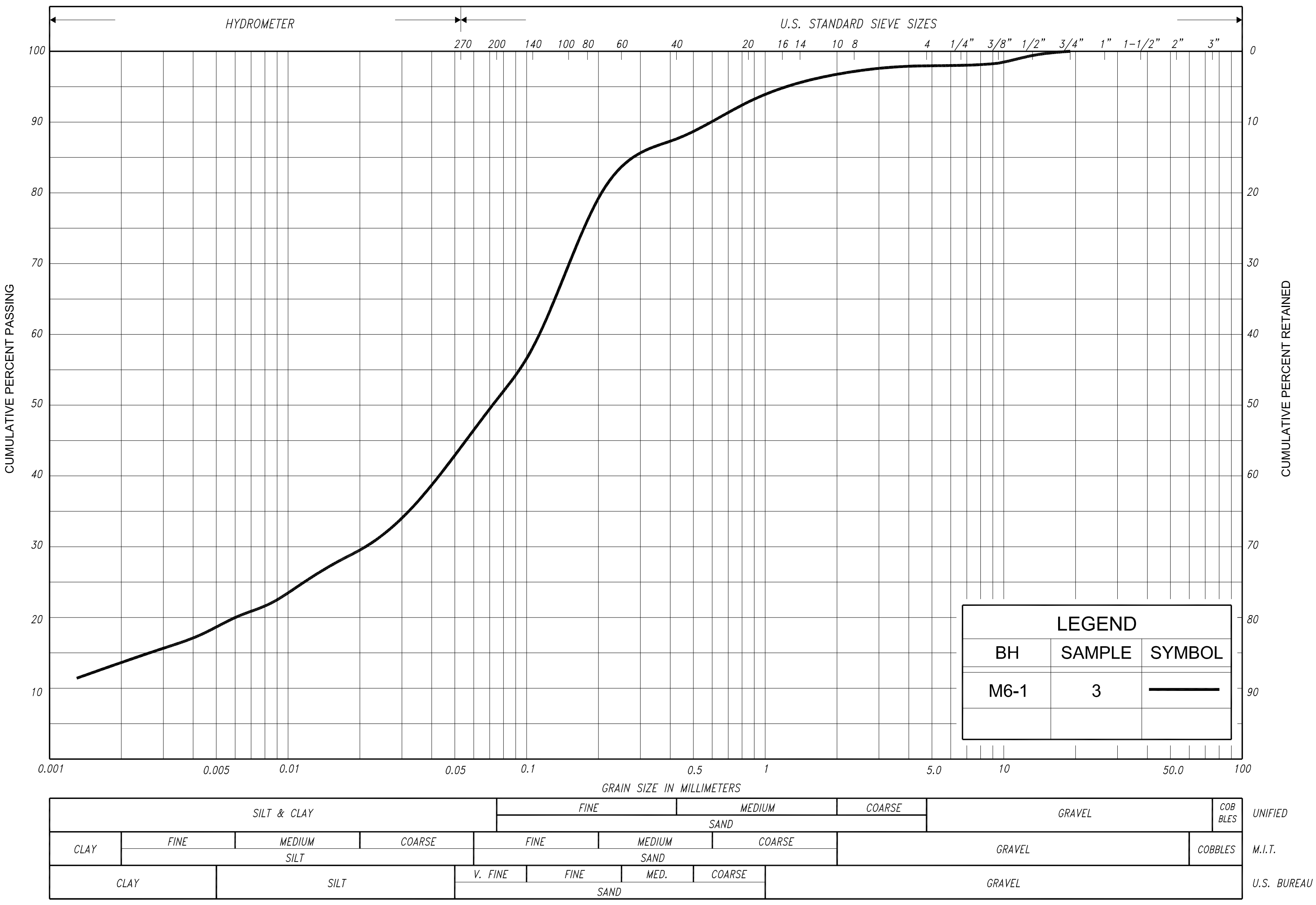
▼ Water level measured after drilling

■ Penetrometer test

+3, ×3: Numbers refer to Sensitivity; 20 / 15–○–5 / 10 (%) STRAIN AT FAILURE

APPENDIX B

LABORATORY TEST RESULTS



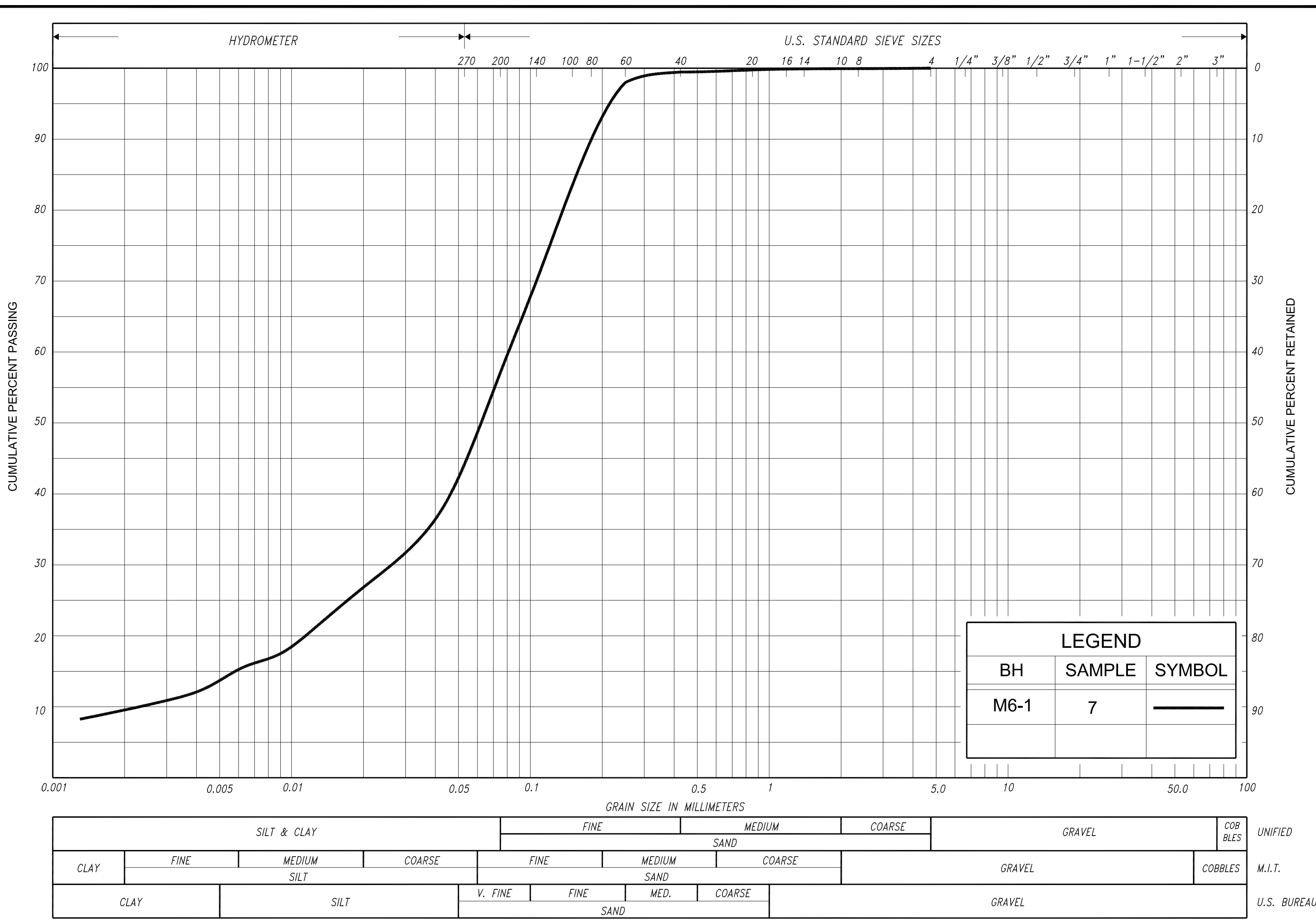
Ontario

Peto MacCallum Ltd. CONSULTING ENGINEERS

GRAIN SIZE DISTRIBUTION

SILTY SAND, some clay, trace gravel

FIG No.	M6-GS-1
HWY:	407E
W.P. No.	07-20015



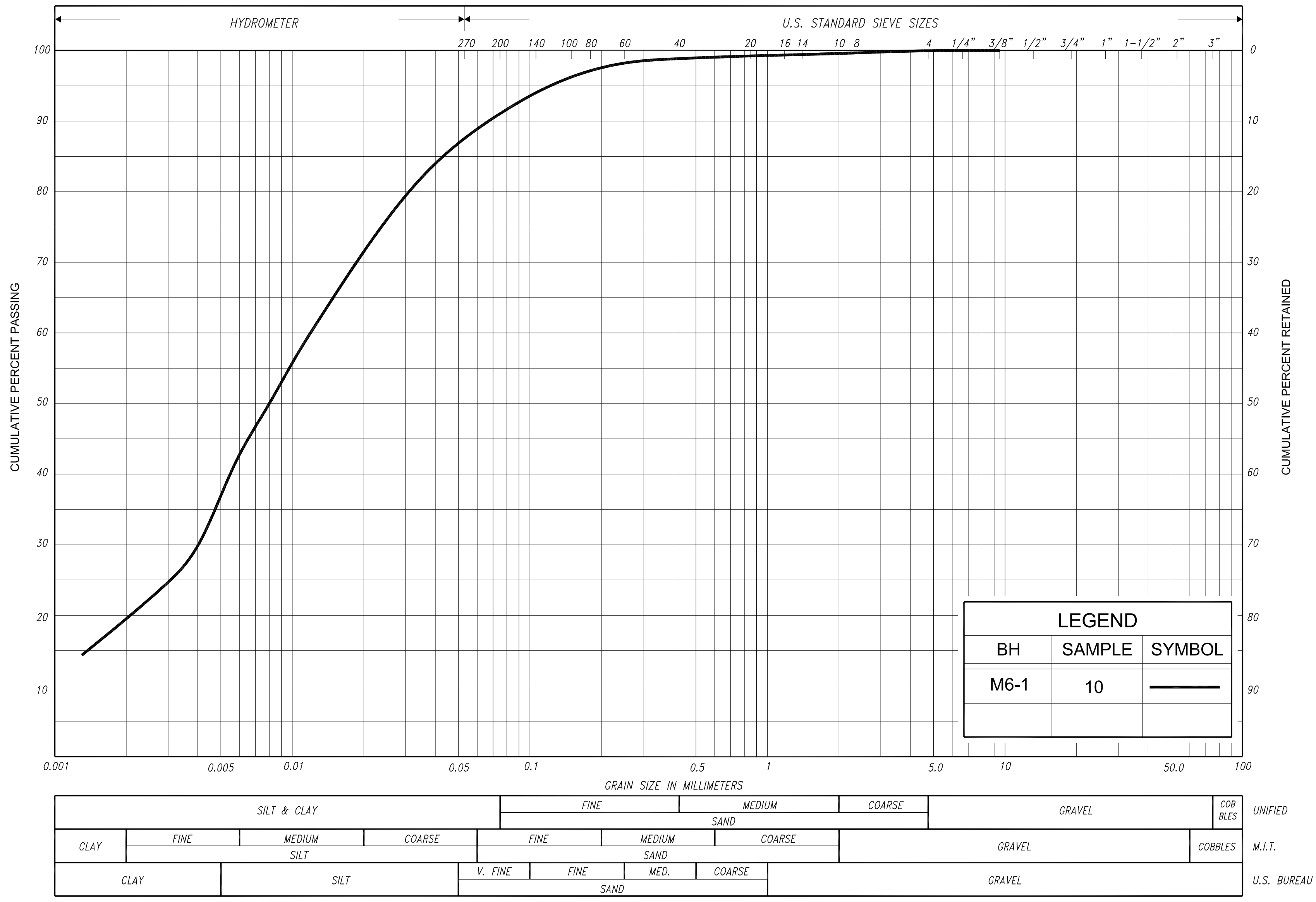
Ontario

Peto MacCallum Ltd. CONSULTING ENGINEERS

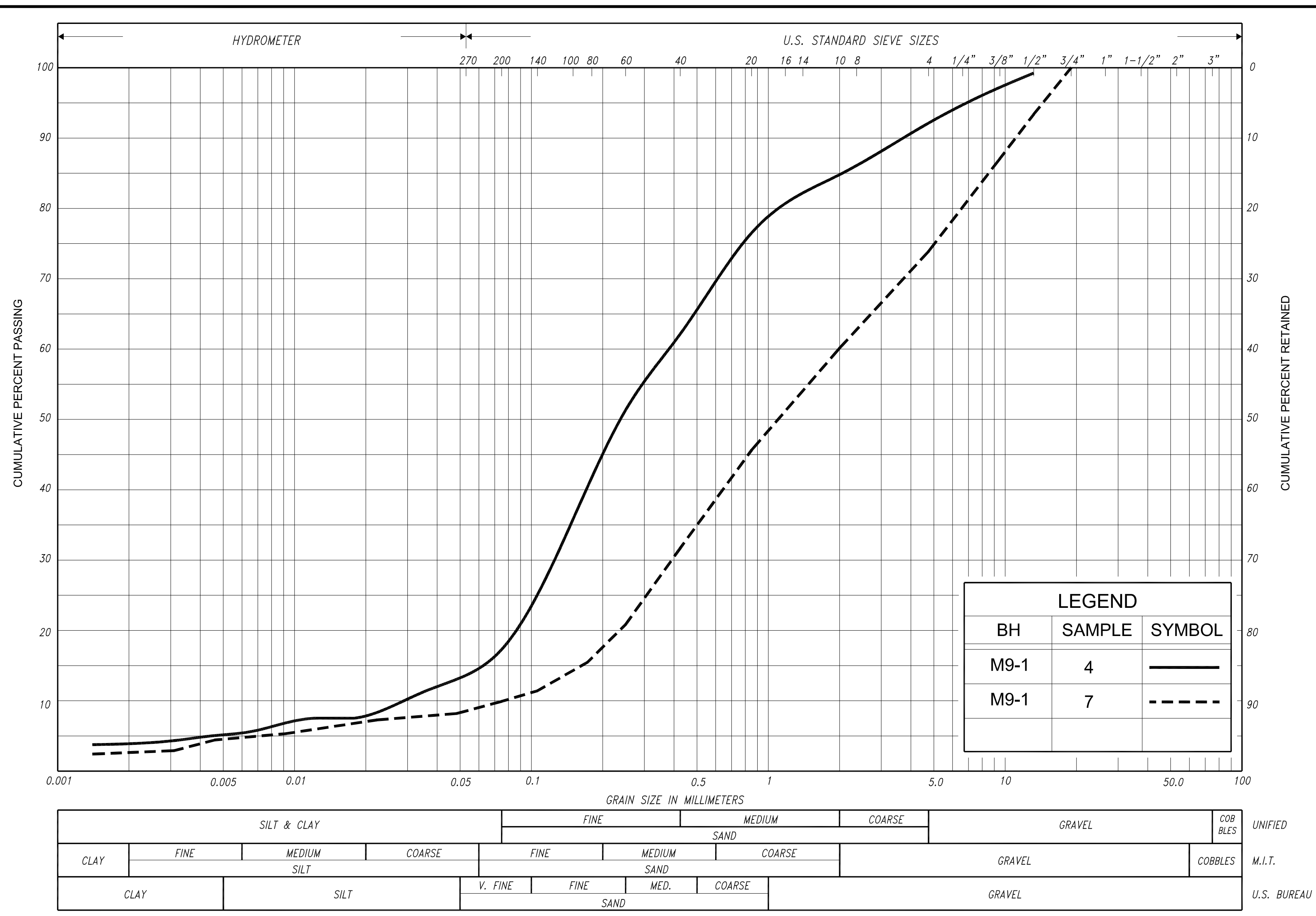
GRAIN SIZE DISTRIBUTION

SAND and SILT, trace clay

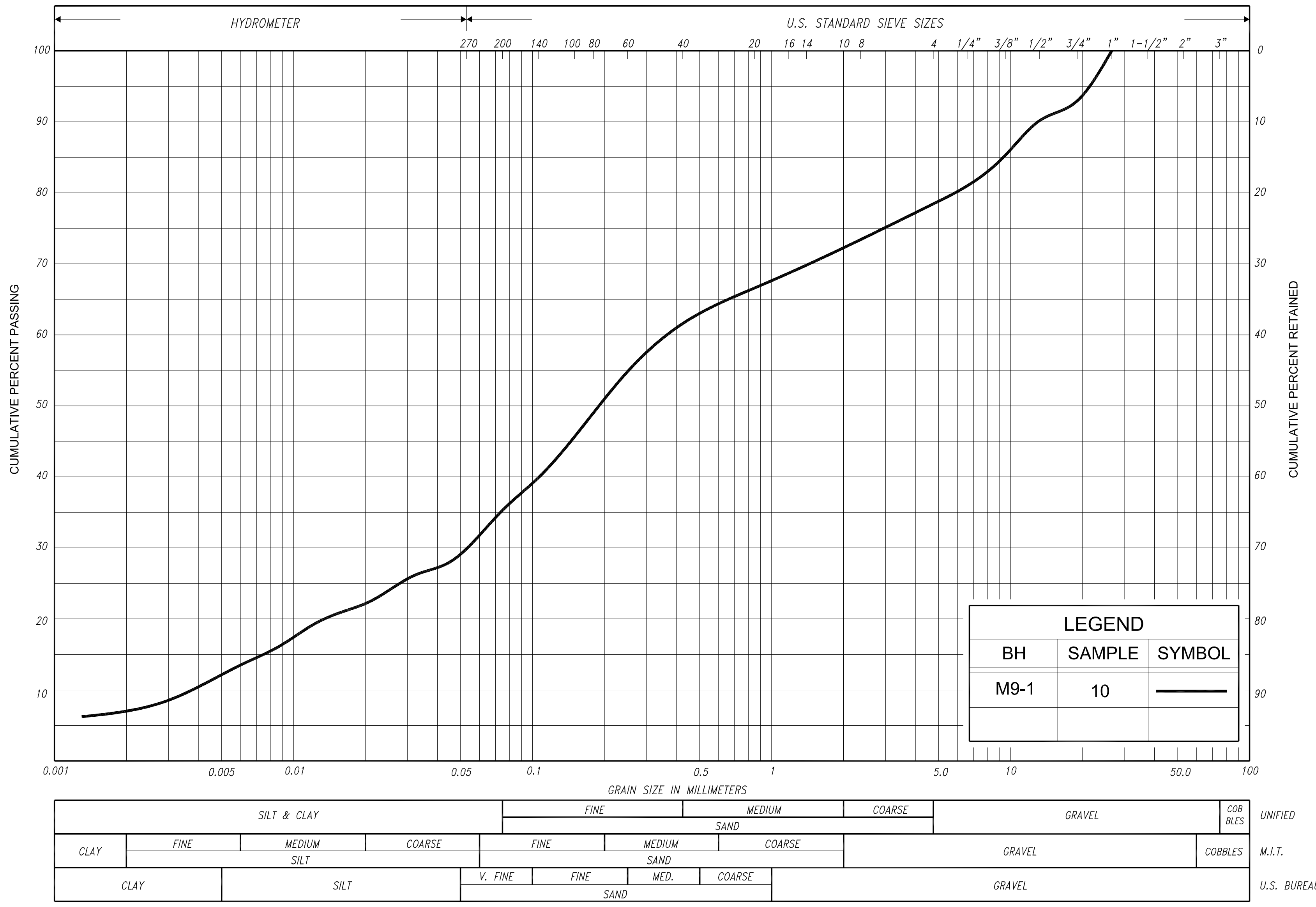
FIG No.	M6-GS-2
HWY:	407E
W.P. No.	07-20015



GRAIN SIZE DISTRIBUTION SILT, some clay, trace sand (TILL)	
FIG No.	M6-GS-3
HWY:	407E
W.P. No.	07-20016



GRAIN SIZE DISTRIBUTION SAND, trace to with gravel, trace to some silt, trace clay	
FIG No.	M9-GS-1
HWY:	407E
W.O. No.	07-20015



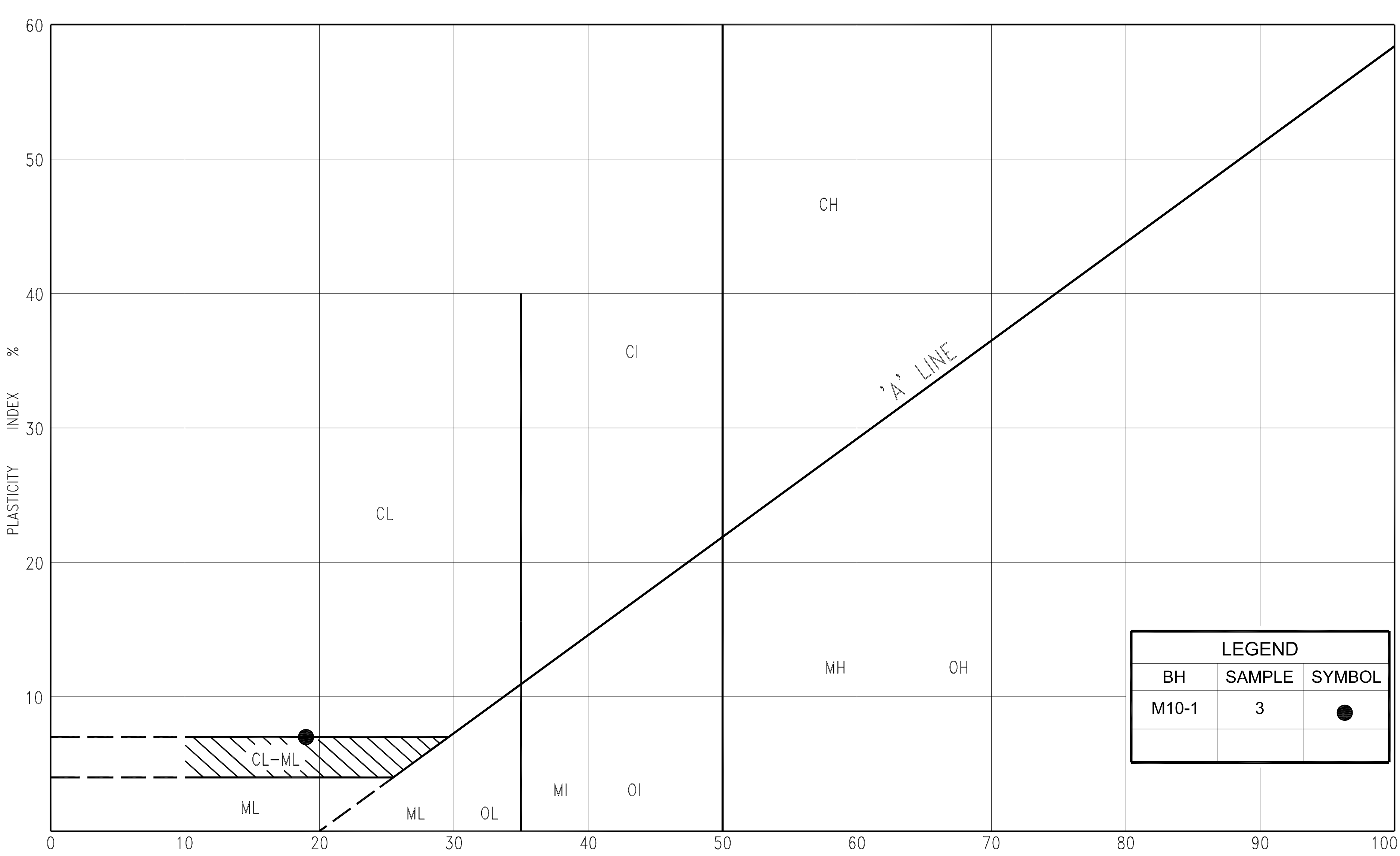
Ontario

Peto MacCallum Ltd. CONSULTING ENGINEERS

GRAIN SIZE DISTRIBUTION

SAND, with silt, with gravel, trace clay
(TILL)

FIG No.	M9-GS-2
HWY:	407E
W.O. No.	07-20015



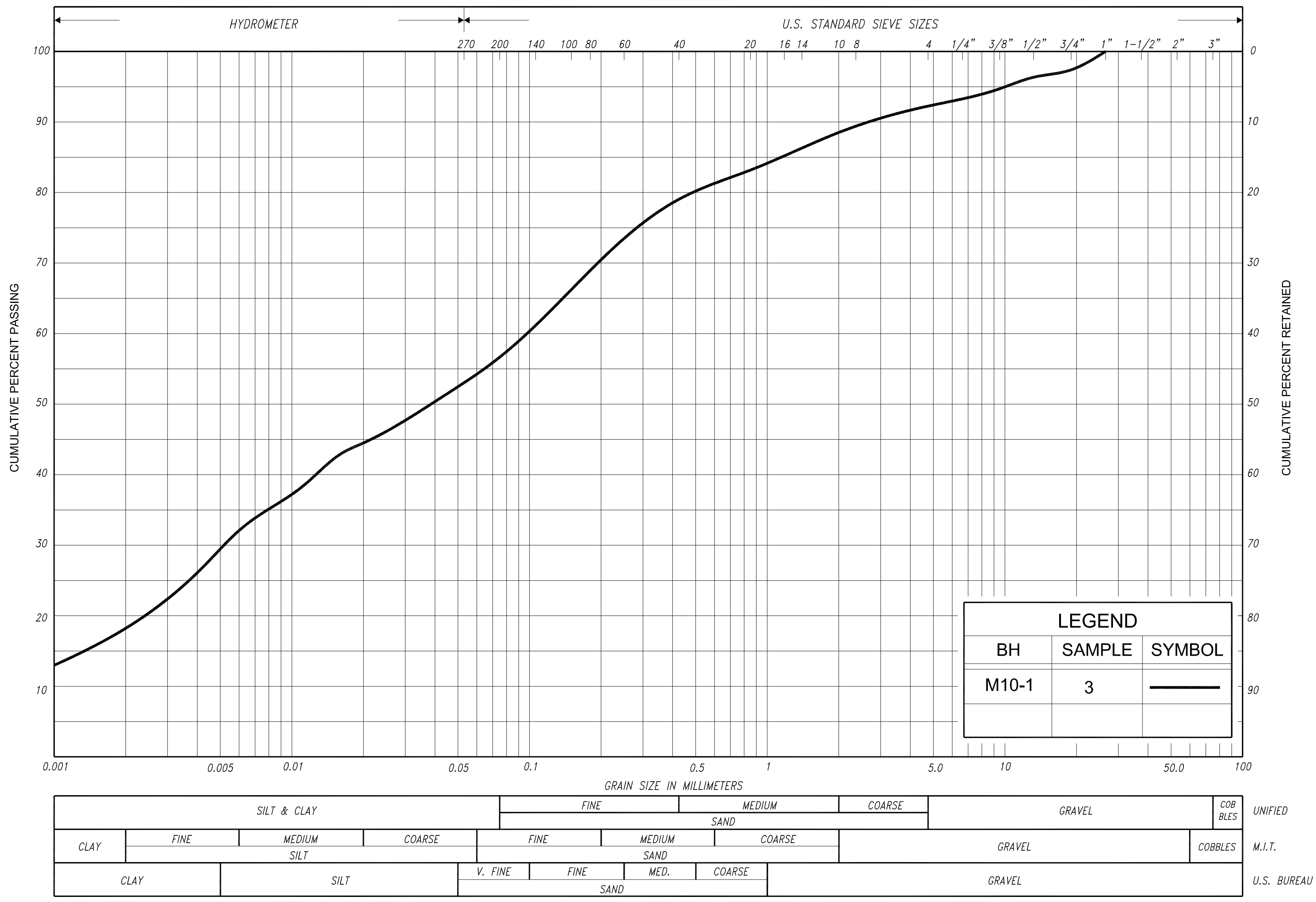
Ontario

Peto MacCallum Ltd. CONSULTING ENGINEERS

PLASTICITY CHART

CLAYEY SILT, sandy, trace gravel
(TILL)

FIG No.	M10-PC-1
HWY:	407E
W.O. No.	07-20015



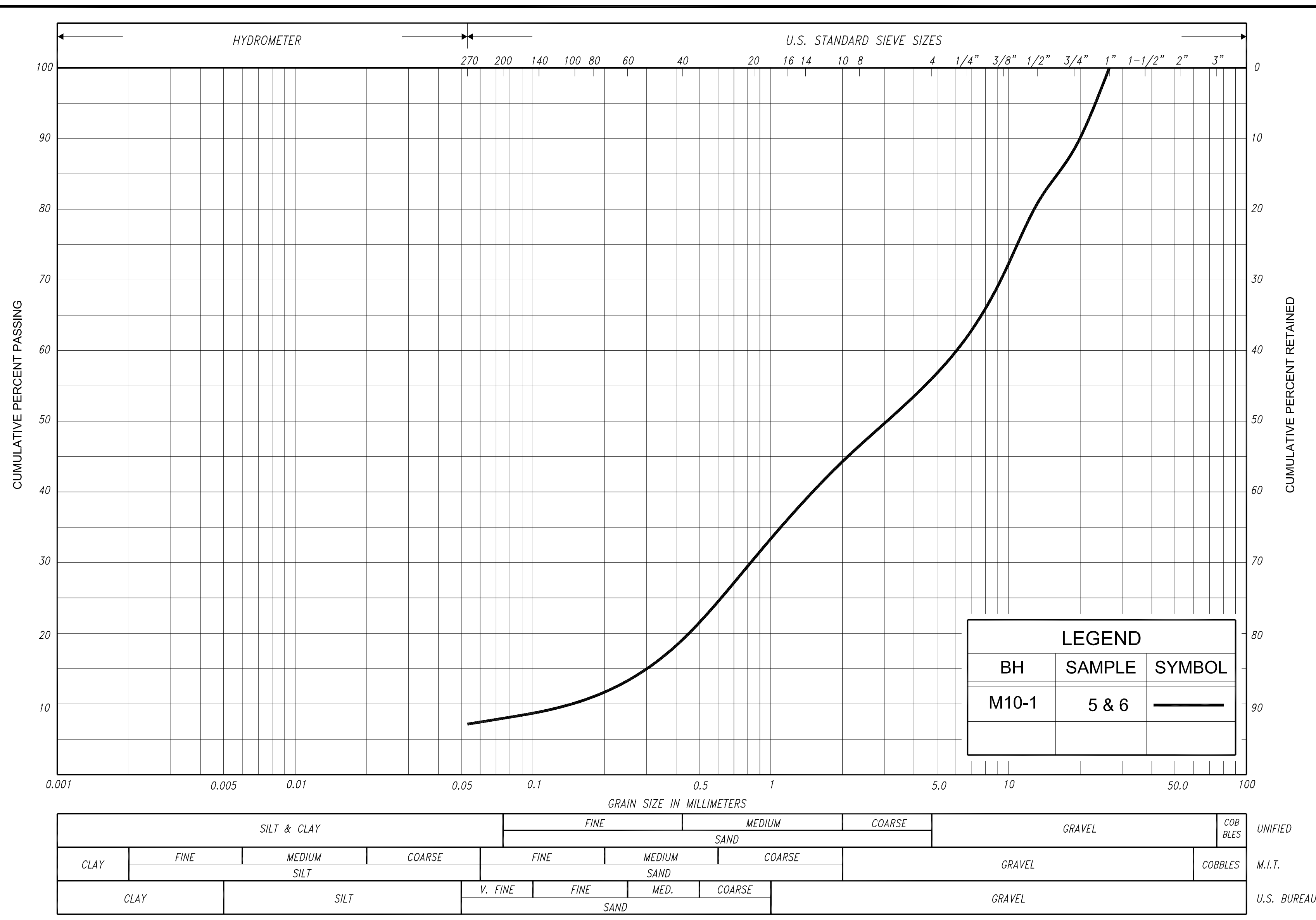
Ontario

Peto MacCallum Ltd. CONSULTING ENGINEERS

GRAIN SIZE DISTRIBUTION

CLAYEY SILT, sandy, trace gravel
(TILL)

FIG No.	M10-GS-1
HWY:	407E
W.O. No.	07-20015



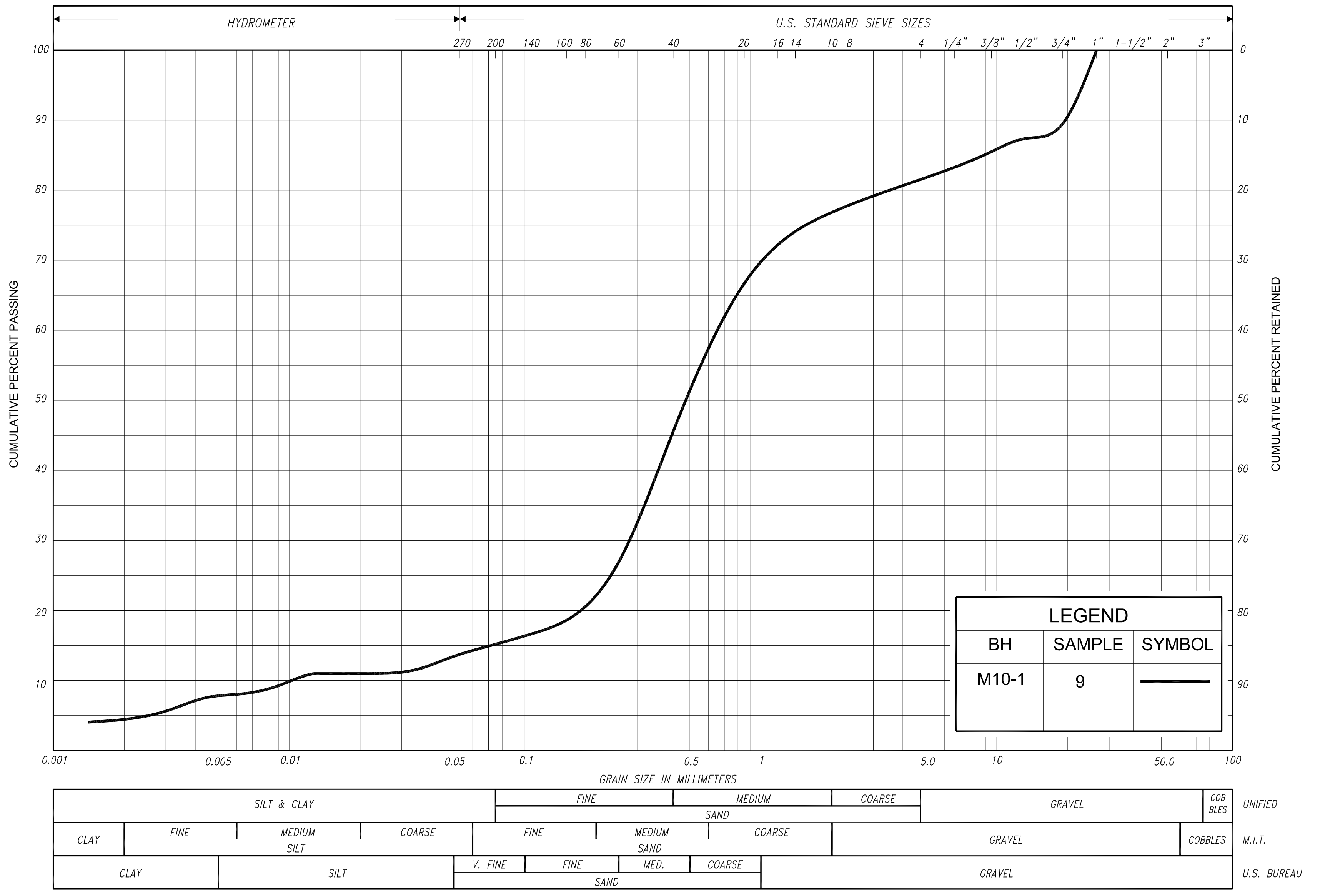
Ontario

Peto MacCallum Ltd. CONSULTING ENGINEERS

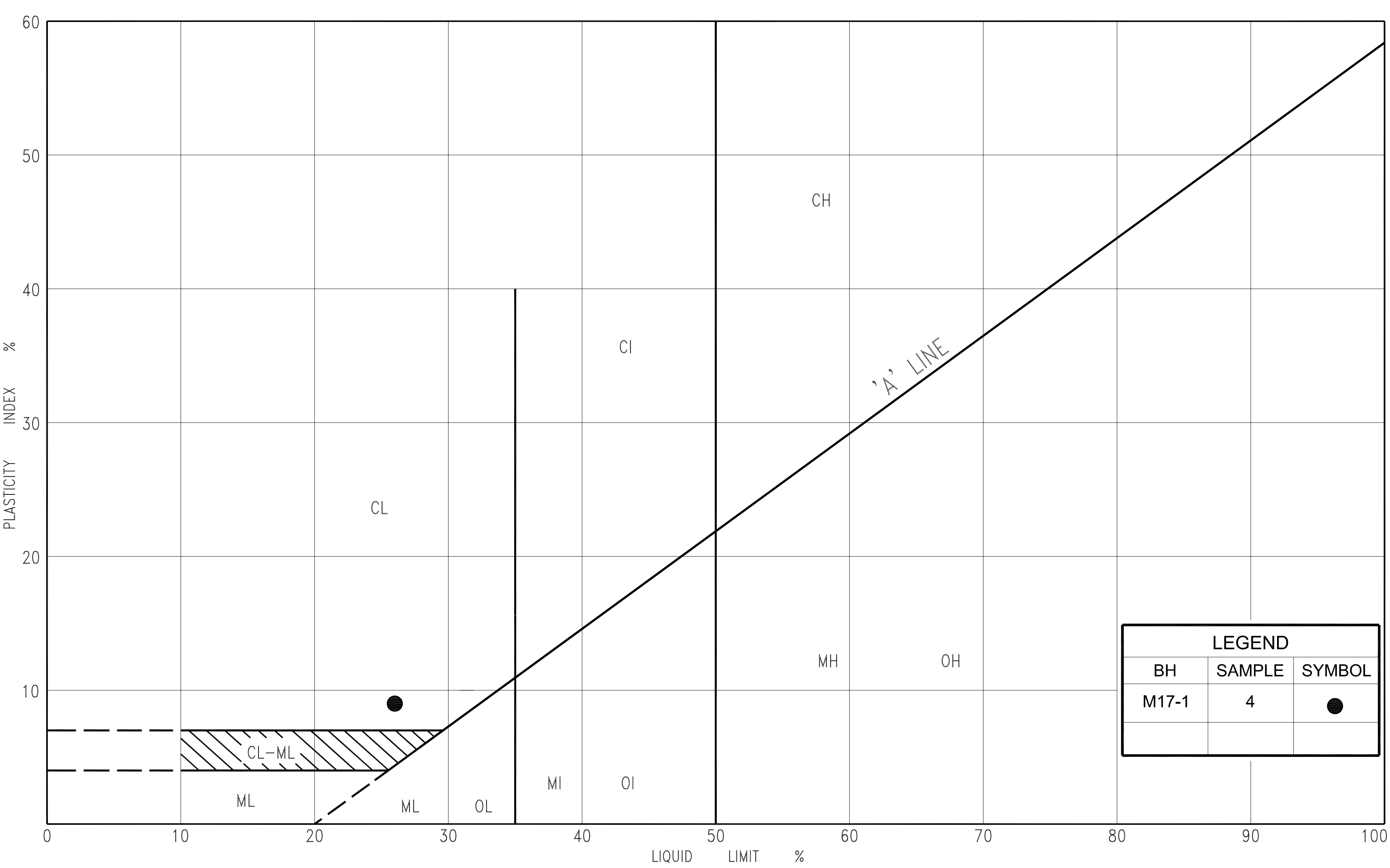
GRAIN SIZE DISTRIBUTION

SAND AND GRAVEL, trace silt

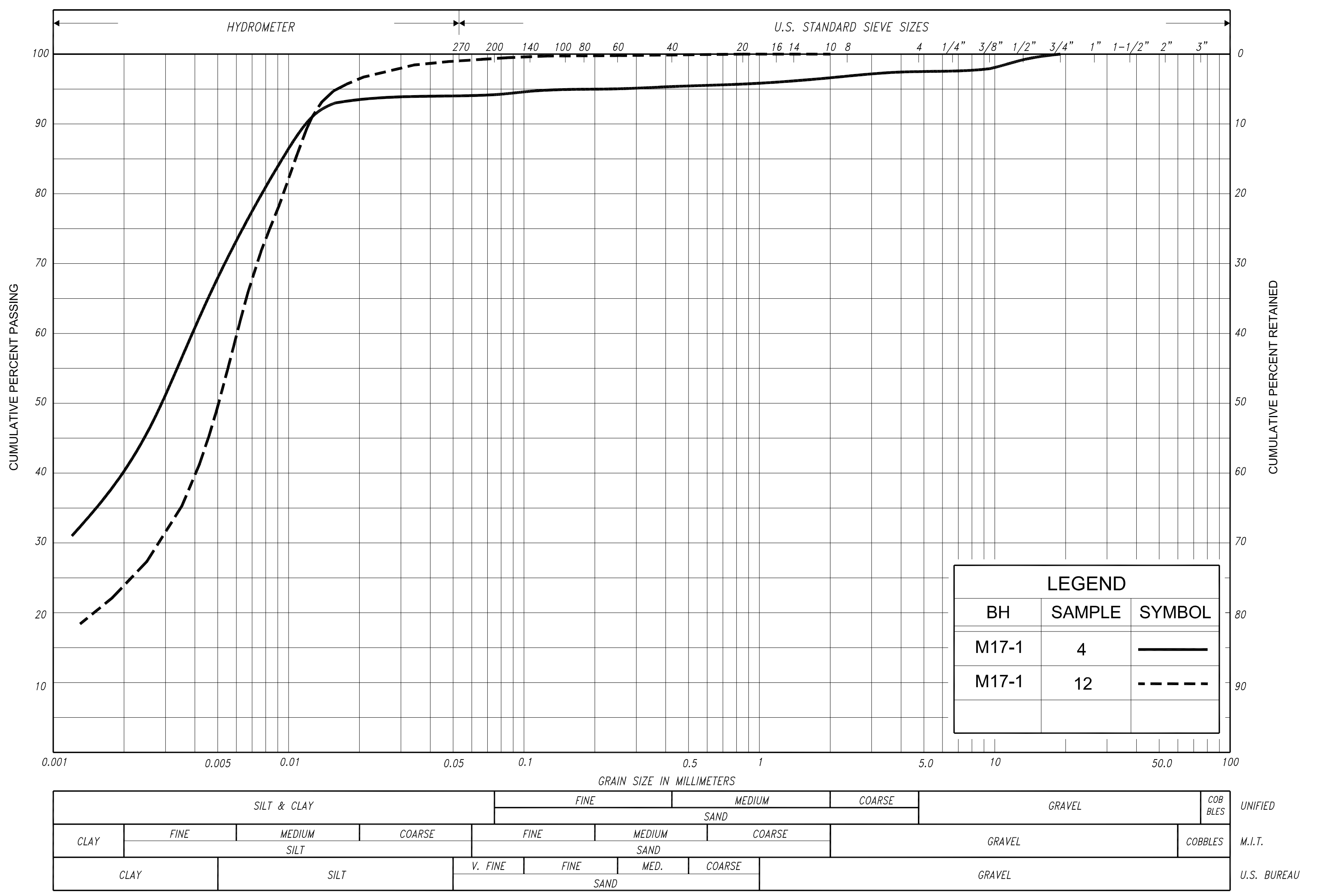
FIG No.	M10-GS-2
HWY:	407E
W.O. No.	07-20015



Ontario / Peto MacCallum Ltd. Consulting Engineers	**GRAIN SIZE DISTRIBUTION** SAND, some silt, some gravel, trace clay	FIG No.	M10-GS-3
		HWY:	407E
		W.O. No.	07-20015



Ontario / Peto MacCallum Ltd. Consulting Engineers	**PLASTICITY CHART** CLAYEY SILT, trace sand, trace gravel (TILL)	FIG No.	M17-PC-1
		HWY:	407E
		W.P. No.	07-20015

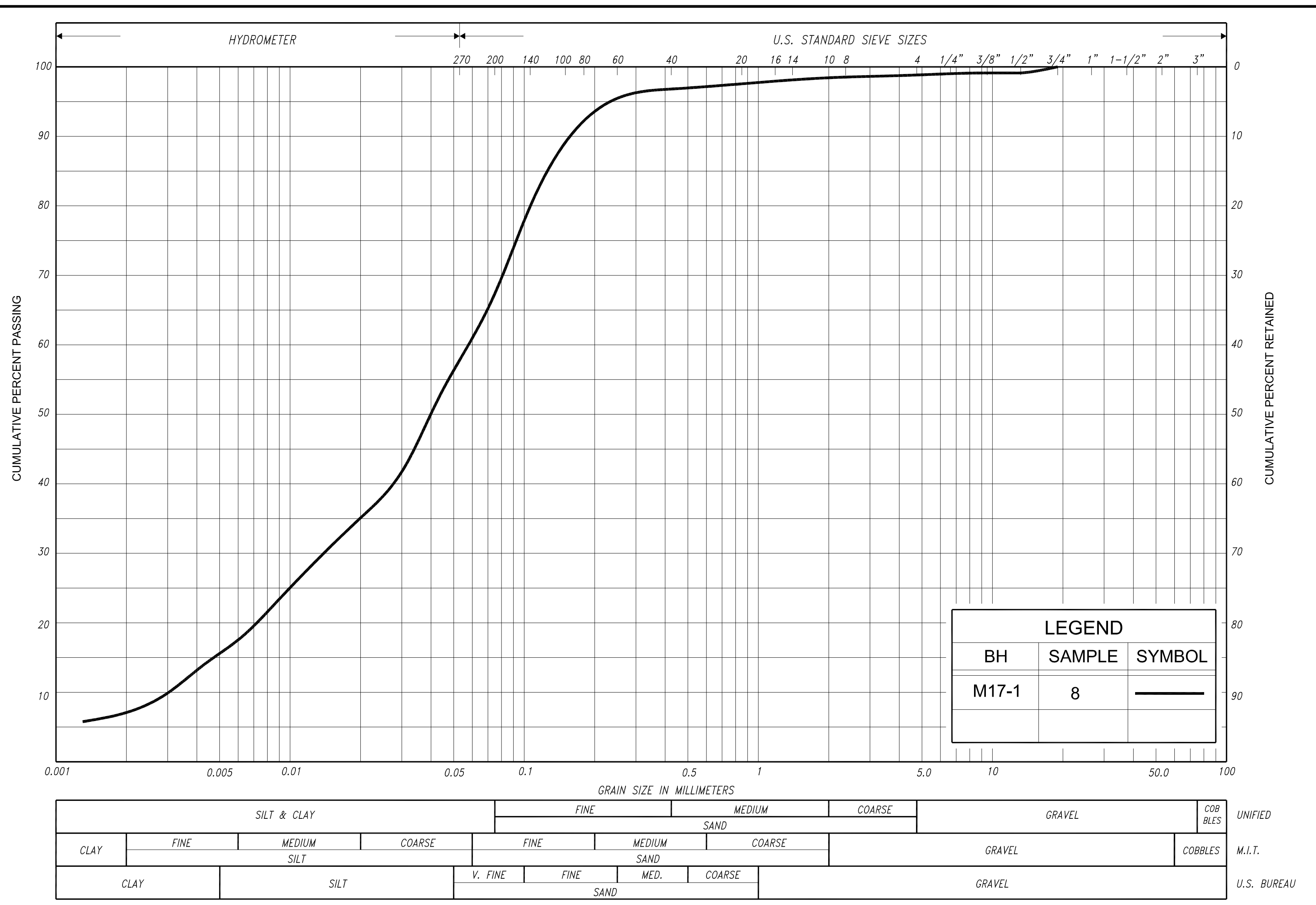


Ontario
Peto MacCallum Ltd. CONSULTING ENGINEERS

GRAIN SIZE DISTRIBUTION

CLAYEY SILT, trace sand, trace gravel
(TILL)

FIG No.	M17-GS-1
HWY:	407E
W.P. No.	07-20015

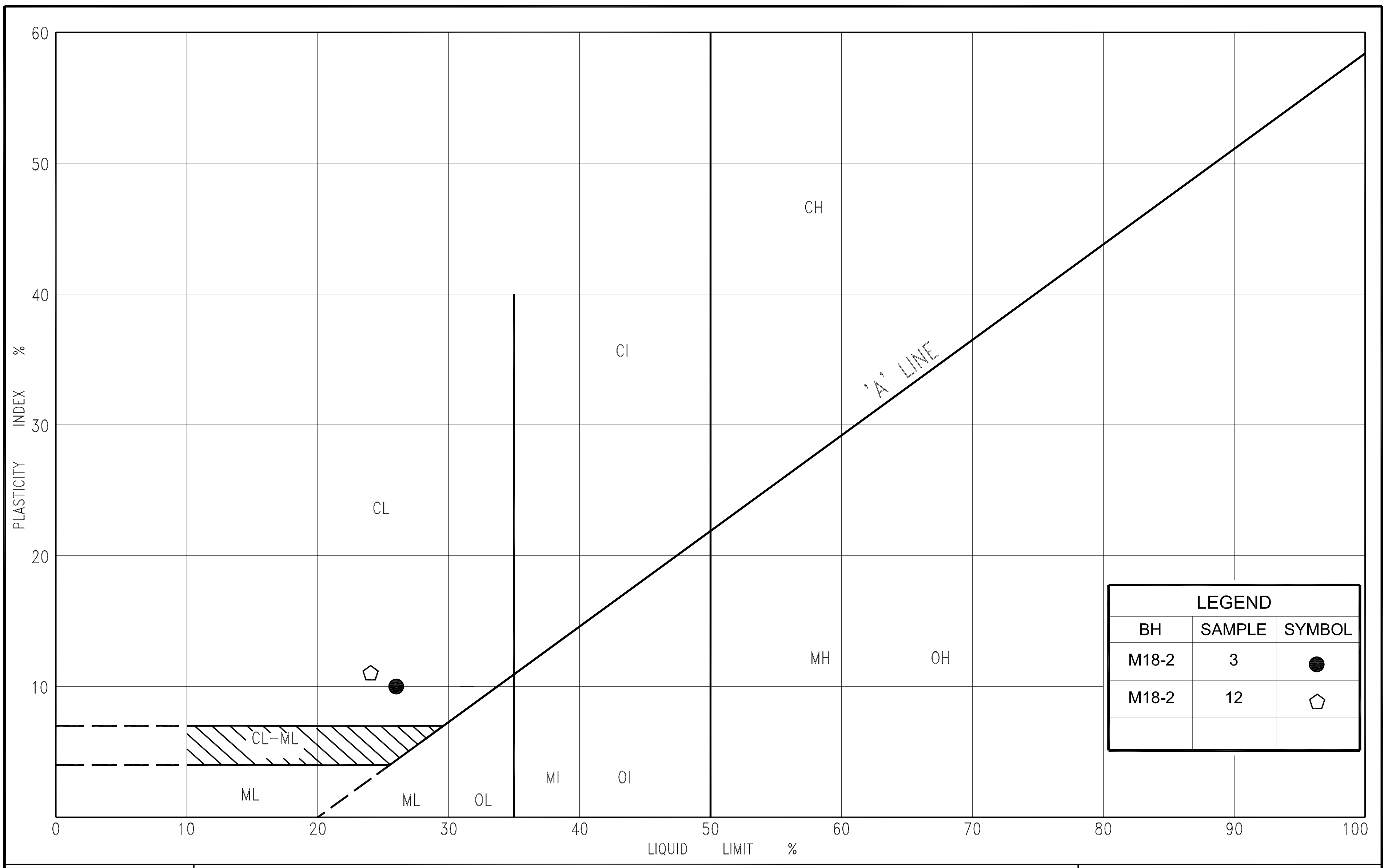


Ontario
Peto MacCallum Ltd. CONSULTING ENGINEERS

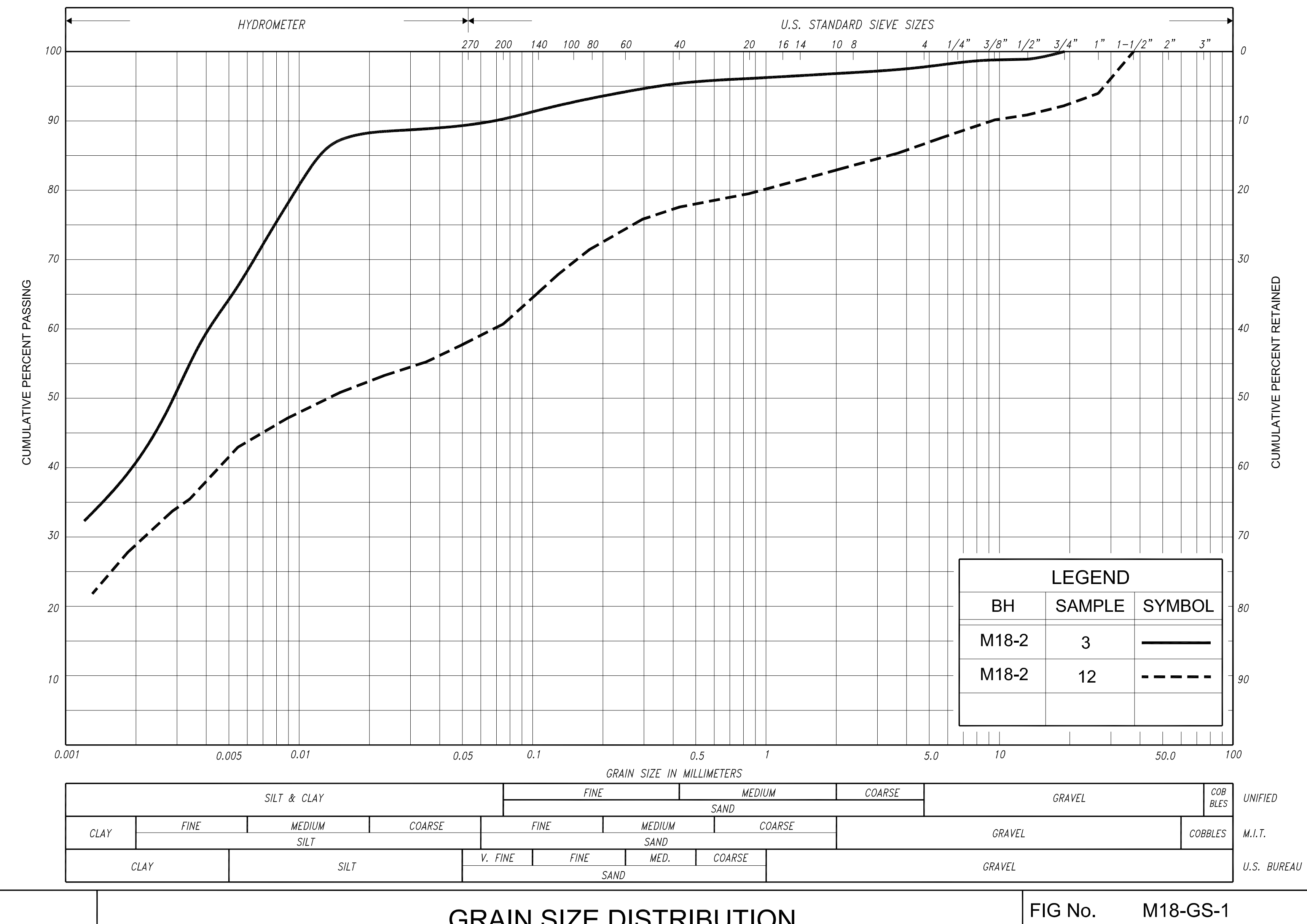
GRAIN SIZE DISTRIBUTION

SANDY SILT, trace clay, trace gravel

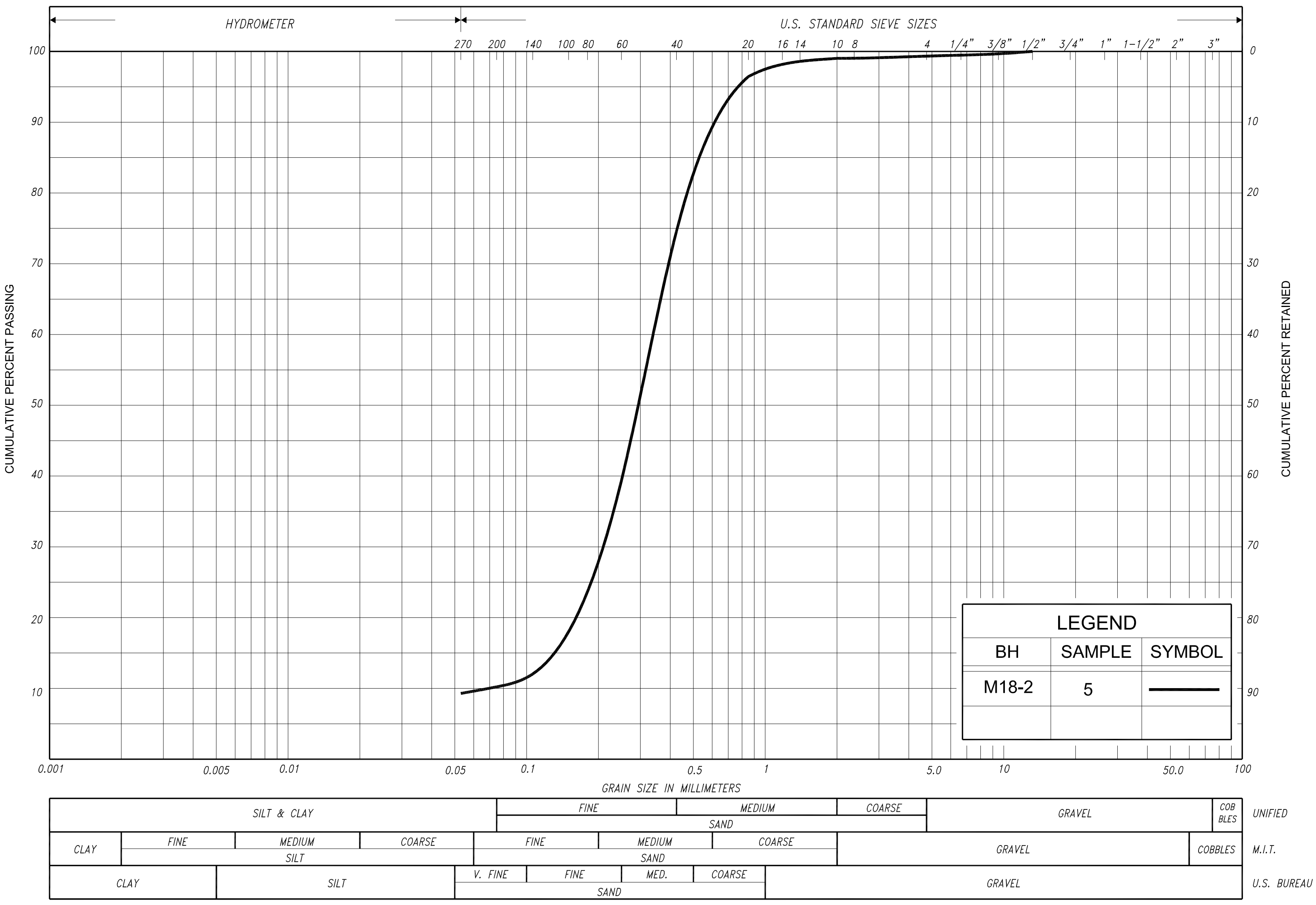
FIG No.	M17-GS-2
HWY:	407E
W.P. No.	07-20015



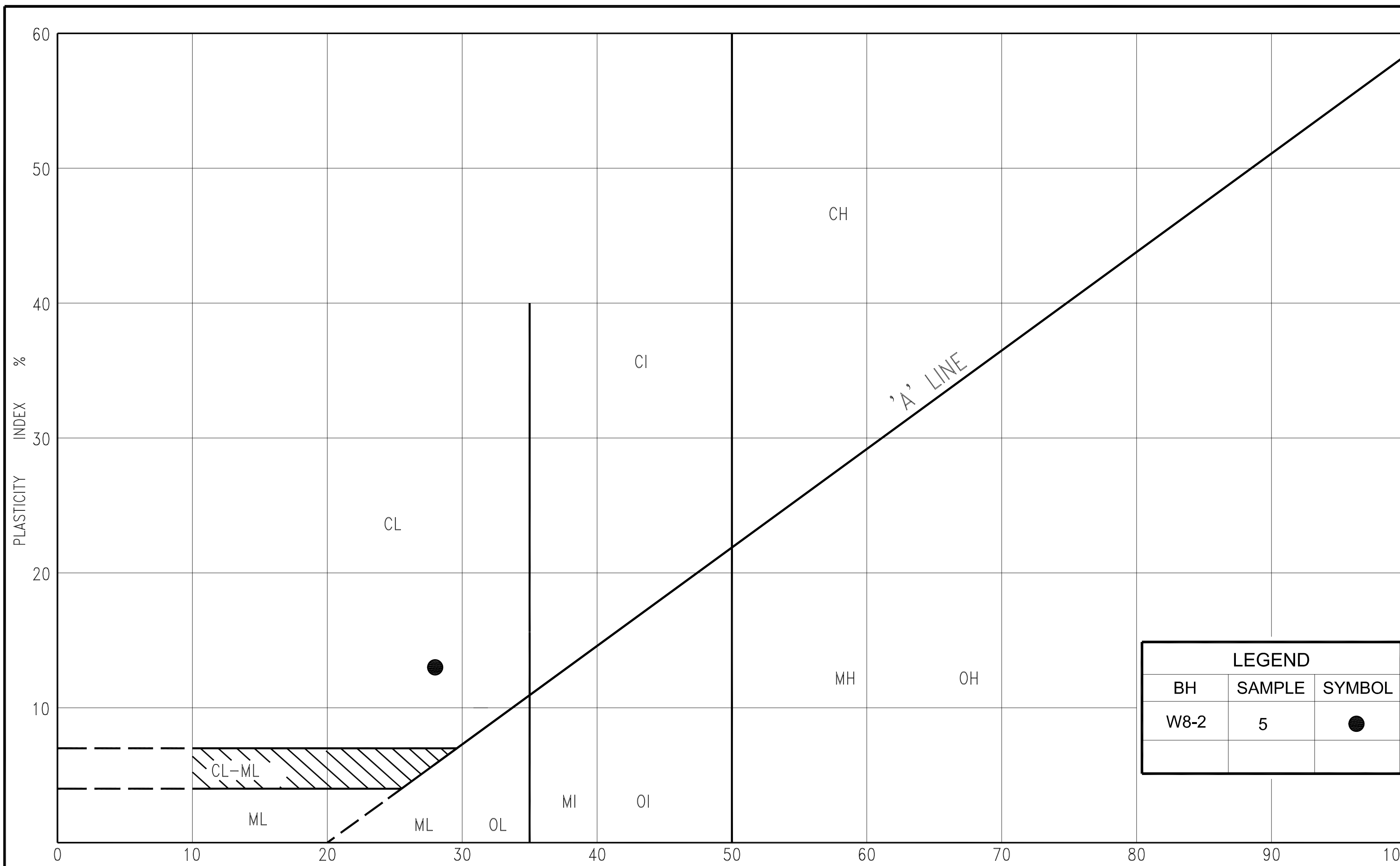
Ontario / Peto MacCallum Ltd. Consulting Engineers	PLASTICITY CHART CLAYEY SILT, trace to with sand, trace to some gravel (TILL)	FIG No.	M18-PC-1
		HWY:	407E
		W.P. No.	07-20015



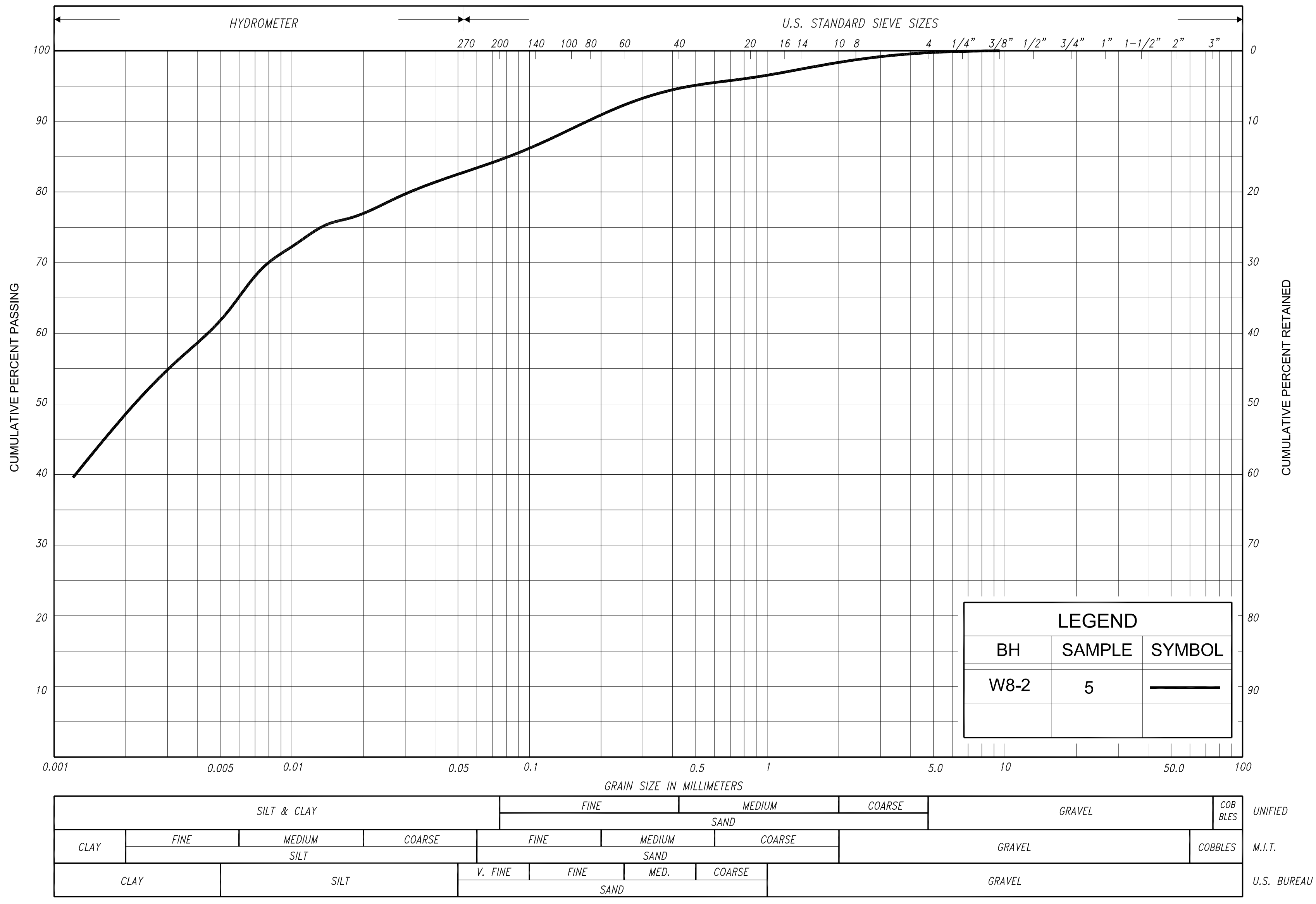
Ontario / Peto MacCallum Ltd. Consulting Engineers	GRAIN SIZE DISTRIBUTION CLAYEY SILT, trace to with sand, trace to some gravel (TILL)	FIG No.	M18-GS-1
		HWY:	407E
		W.P. No.	07-20015



Ontario / Peto MacCallum Ltd.	GRAIN SIZE DISTRIBUTION SAND, trace silt, trace gravel	FIG No.	M18-GS-2
		HWY:	407E
		W.P. No.	07-20015



Ontario / Peto MacCallum Ltd.	PLASTICITY CHART CLAYEY SILT, some sand	FIG No.	W8-PC-1
		HWY:	407E
		W.O. No.	07-20015

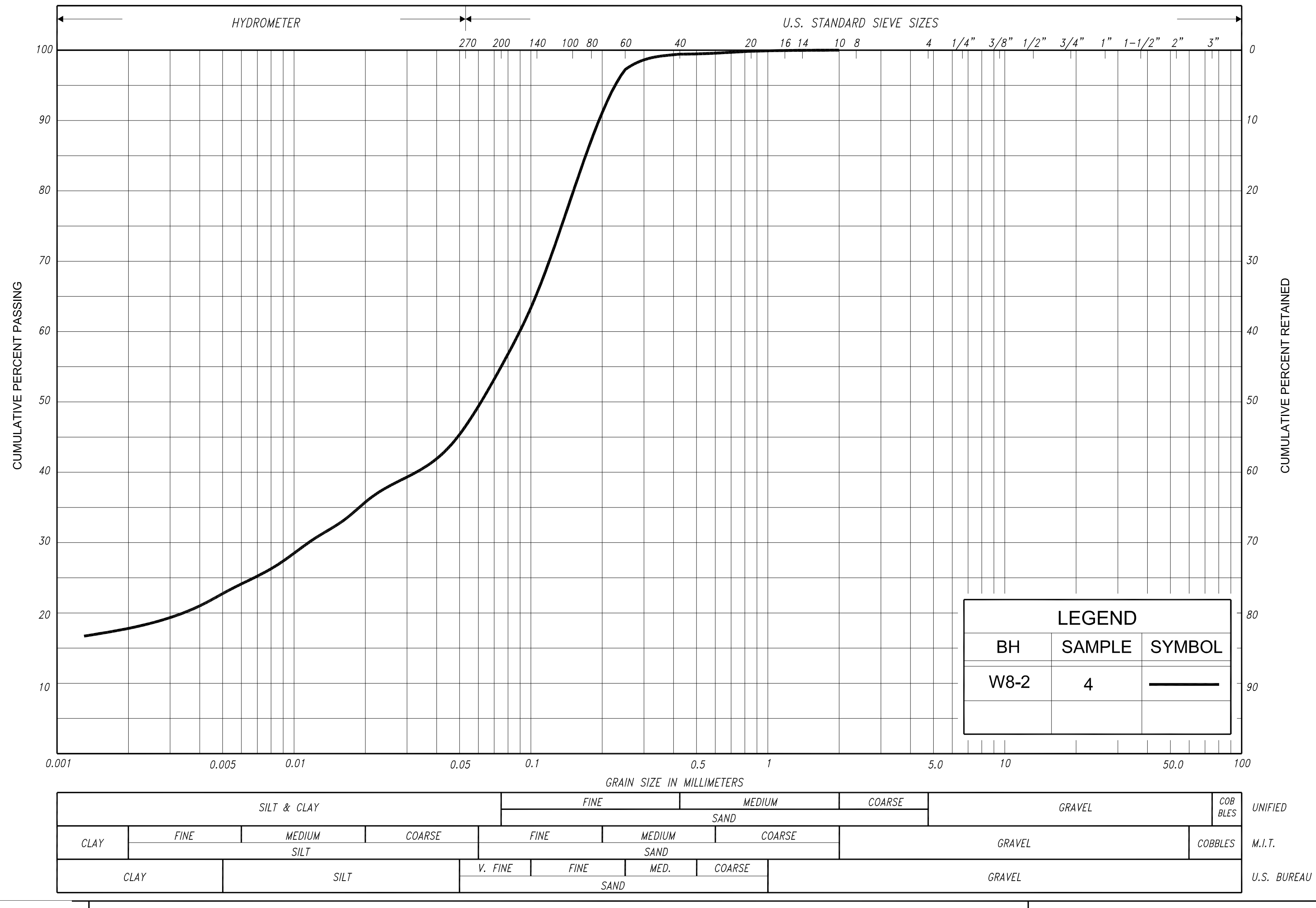


Ontario
Peto MacCallum Ltd.
CONSULTING ENGINEERS

GRAIN SIZE DISTRIBUTION

CLAYEY SILT, some sand

FIG No.	W8-GS-1
HWY:	407E
W.O. No.	07-20015

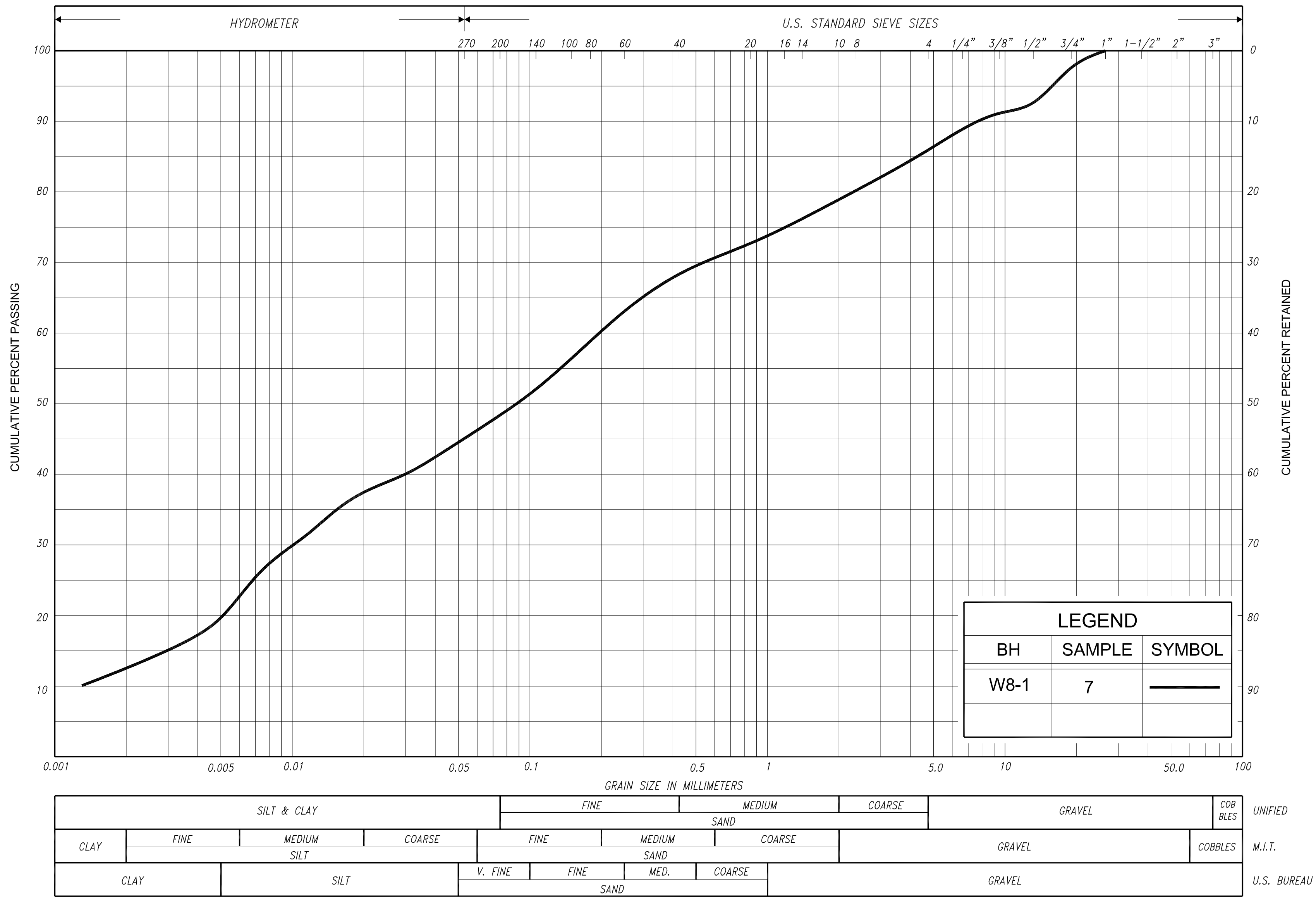


Ontario
Peto MacCallum Ltd.
CONSULTING ENGINEERS

GRAIN SIZE DISTRIBUTION

SILTY SAND, some clay

FIG No.	W8-GS-2
HWY:	407E
W.O. No.	07-20015



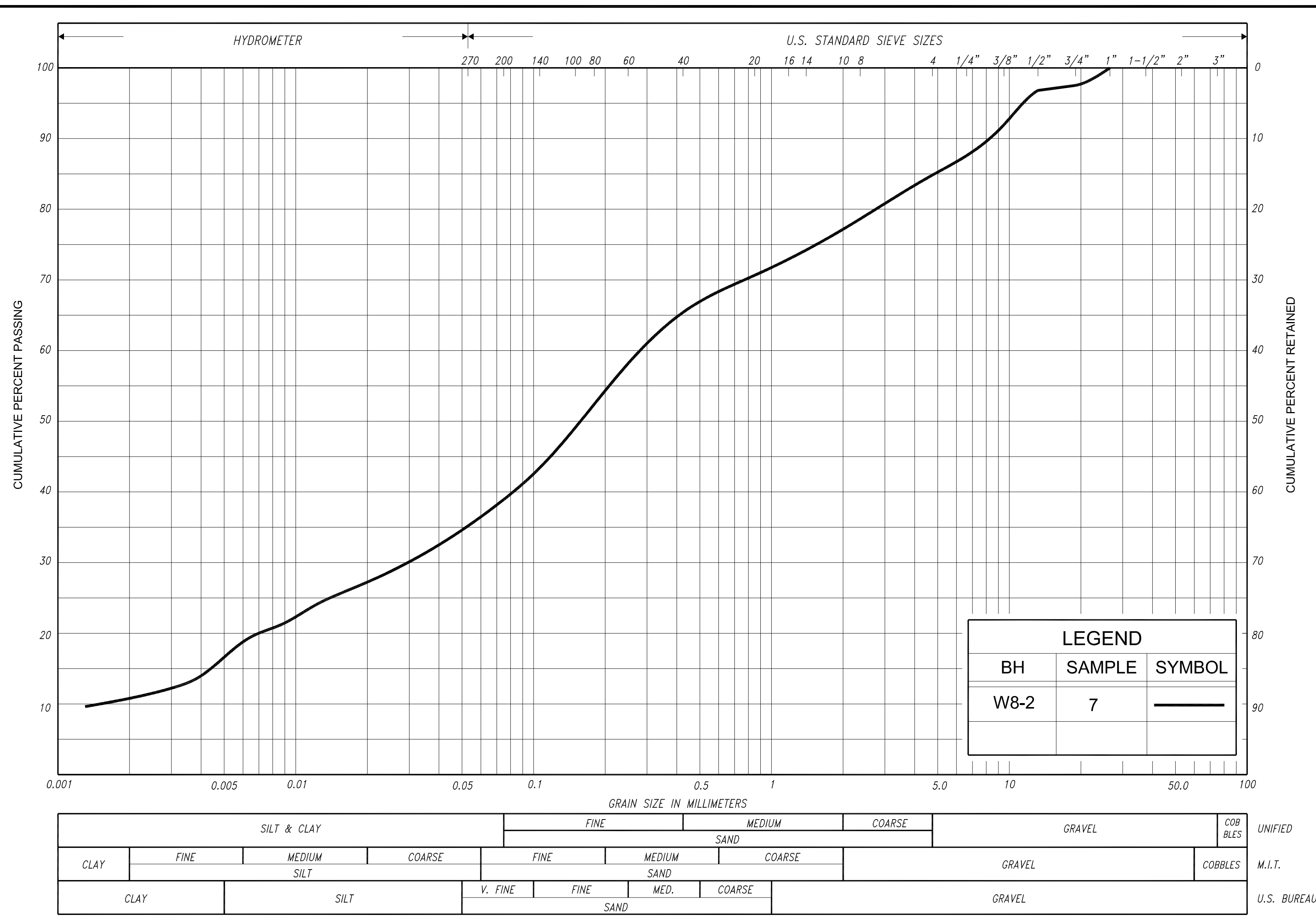
Ontario

Peto MacCallum Ltd. CONSULTING ENGINEERS

GRAIN SIZE DISTRIBUTION

SAND and SILT, some clay, some gravel
(TILL)

FIG No.	W8-GS-3
HWY:	407E
W.O. No.	07-20015



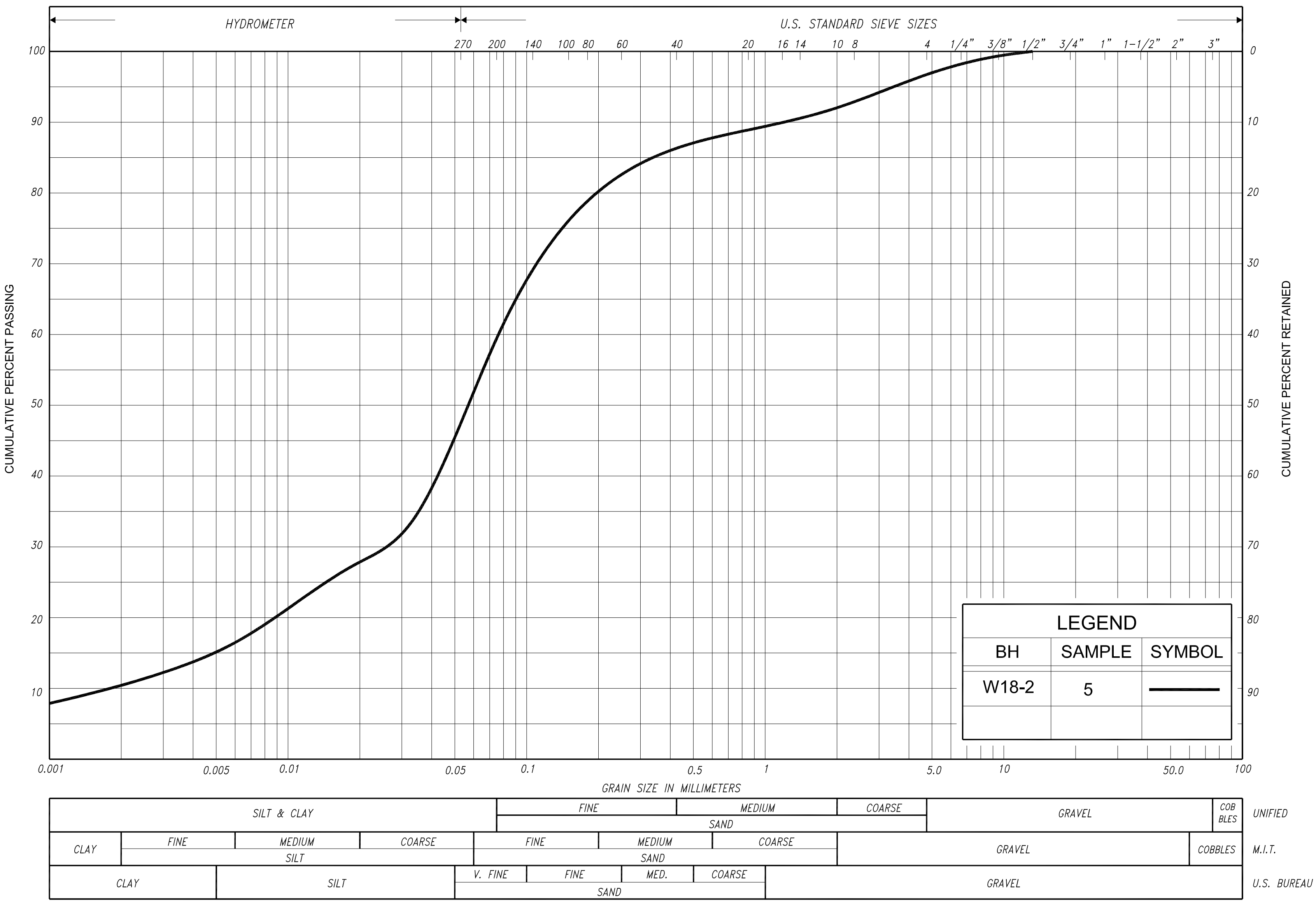
Ontario

Peto MacCallum Ltd. CONSULTING ENGINEERS

GRAIN SIZE DISTRIBUTION

SAND, with silt, some clay, some gravel
(TILL)

FIG No.	W8-GS-4
HWY:	407E
W.O. No.	07-20015



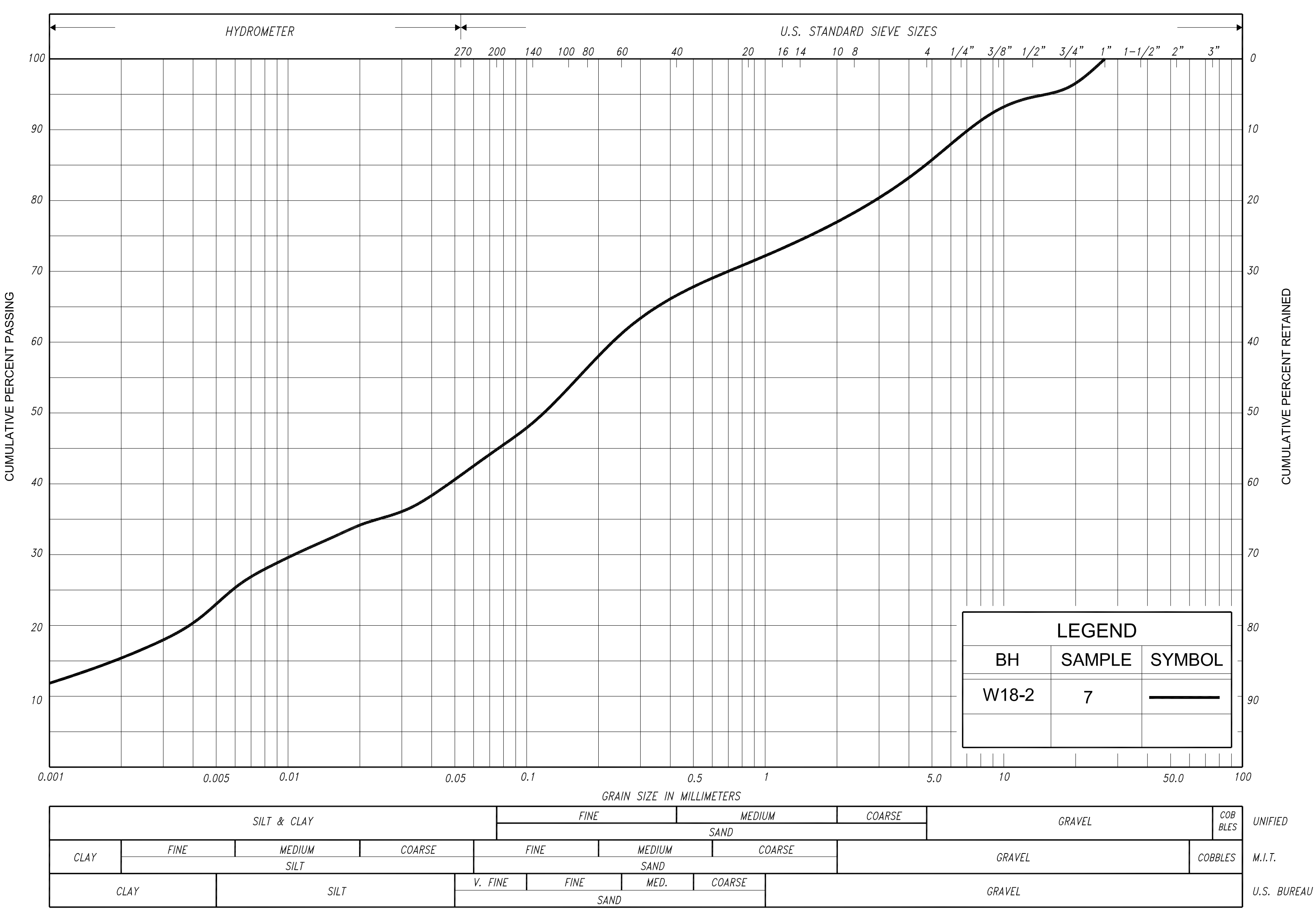
Ontario

Peto MacCallum Ltd. CONSULTING ENGINEERS

GRAIN SIZE DISTRIBUTION

SANDY SILT, some clay, trace gravel

FIG No.	W18-GS-1
HWY:	407E
W.O. No.	07-20015



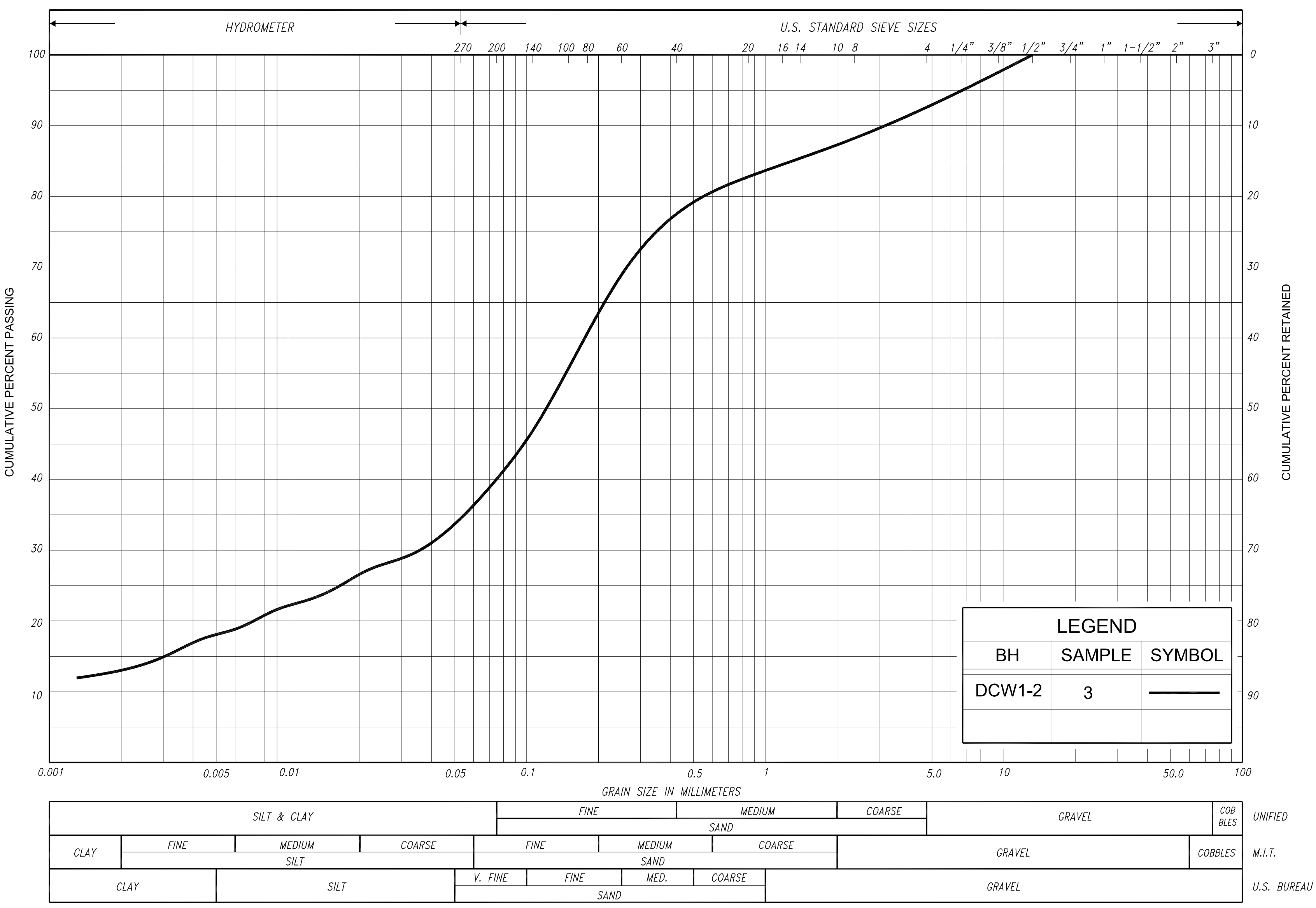
Ontario

Peto MacCallum Ltd. CONSULTING ENGINEERS

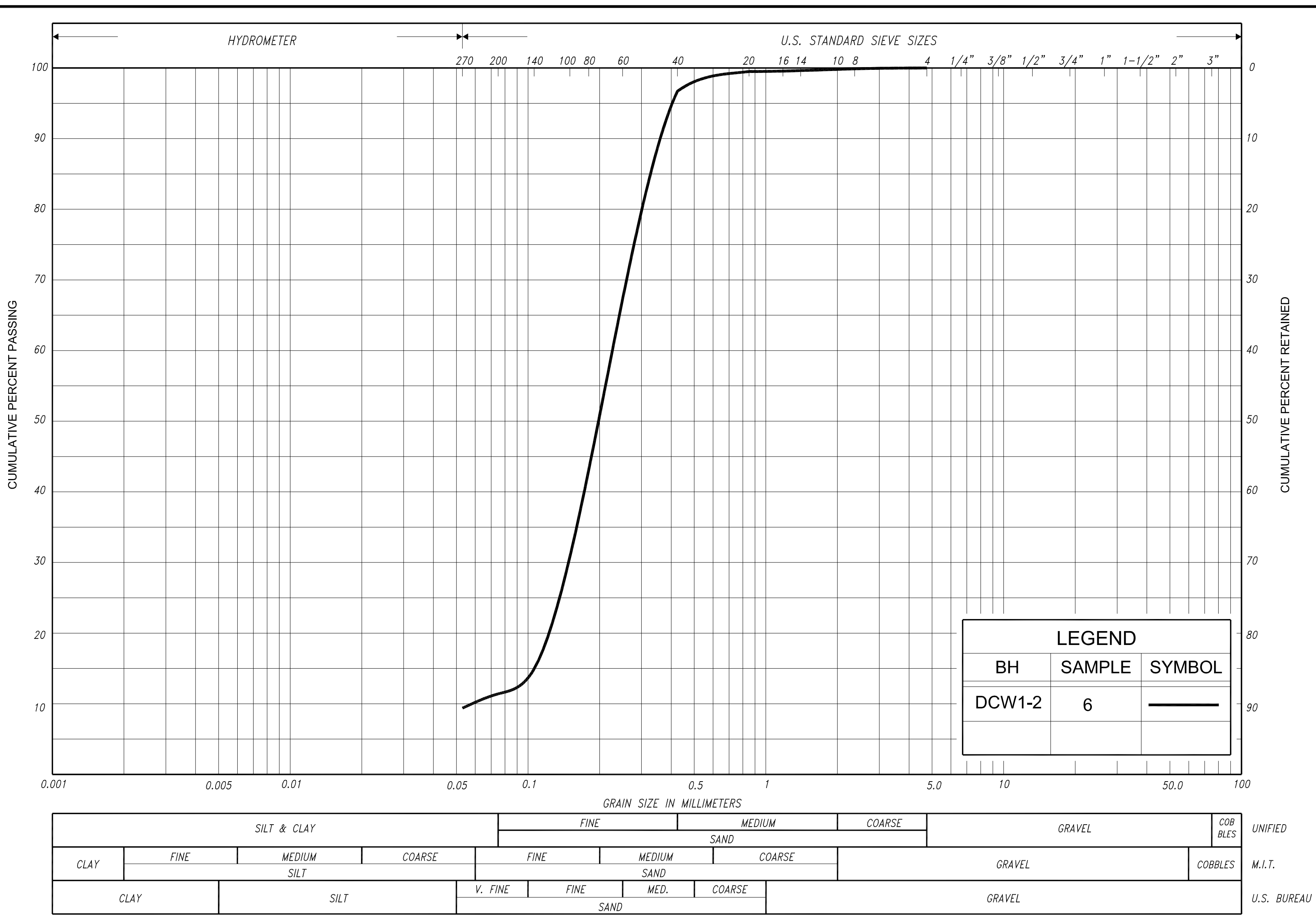
GRAIN SIZE DISTRIBUTION

CLAYEY SILT, sandy, some gravel
(TILL)

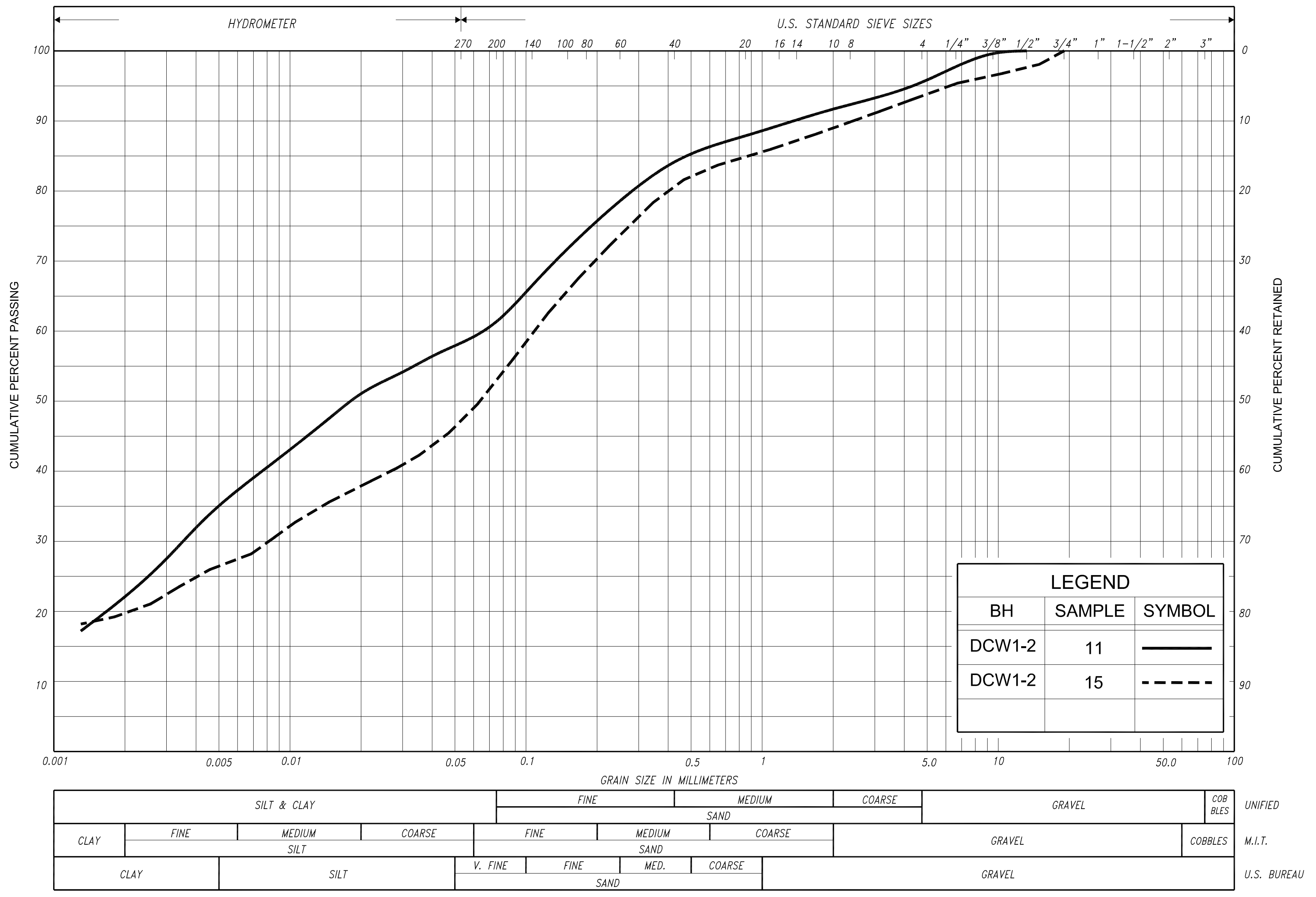
FIG No.	W18-GS-2
HWY:	407E
W.O. No.	07-20015



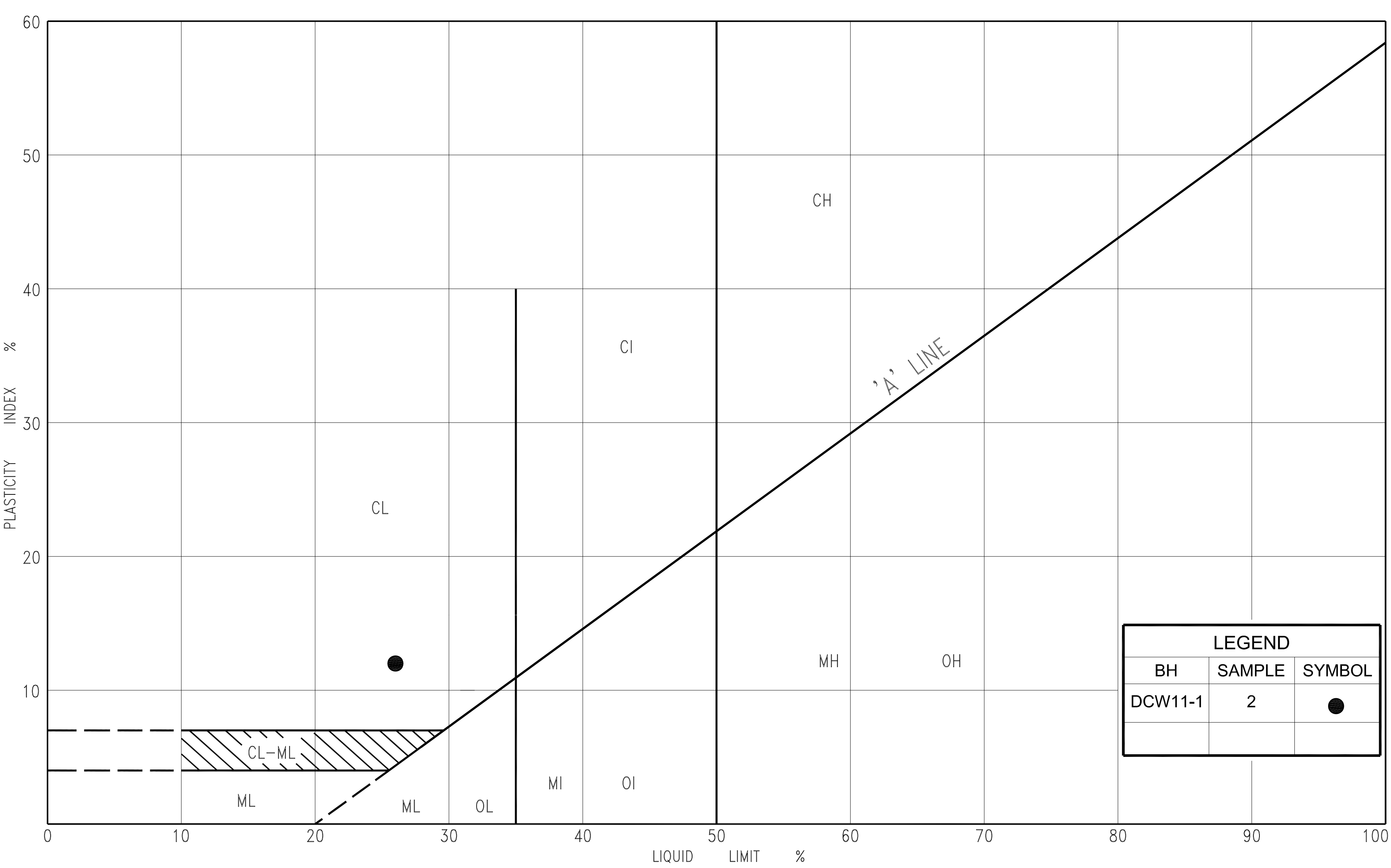
Ontario / Peto MacCallum Ltd. CONSULTING ENGINEERS	**GRAIN SIZE DISTRIBUTION** SAND, with silt, some clay, trace gravel (TILL)	FIG No.	DCW1-GS-1
		HWY:	407E
		W.P. No.	07-20015



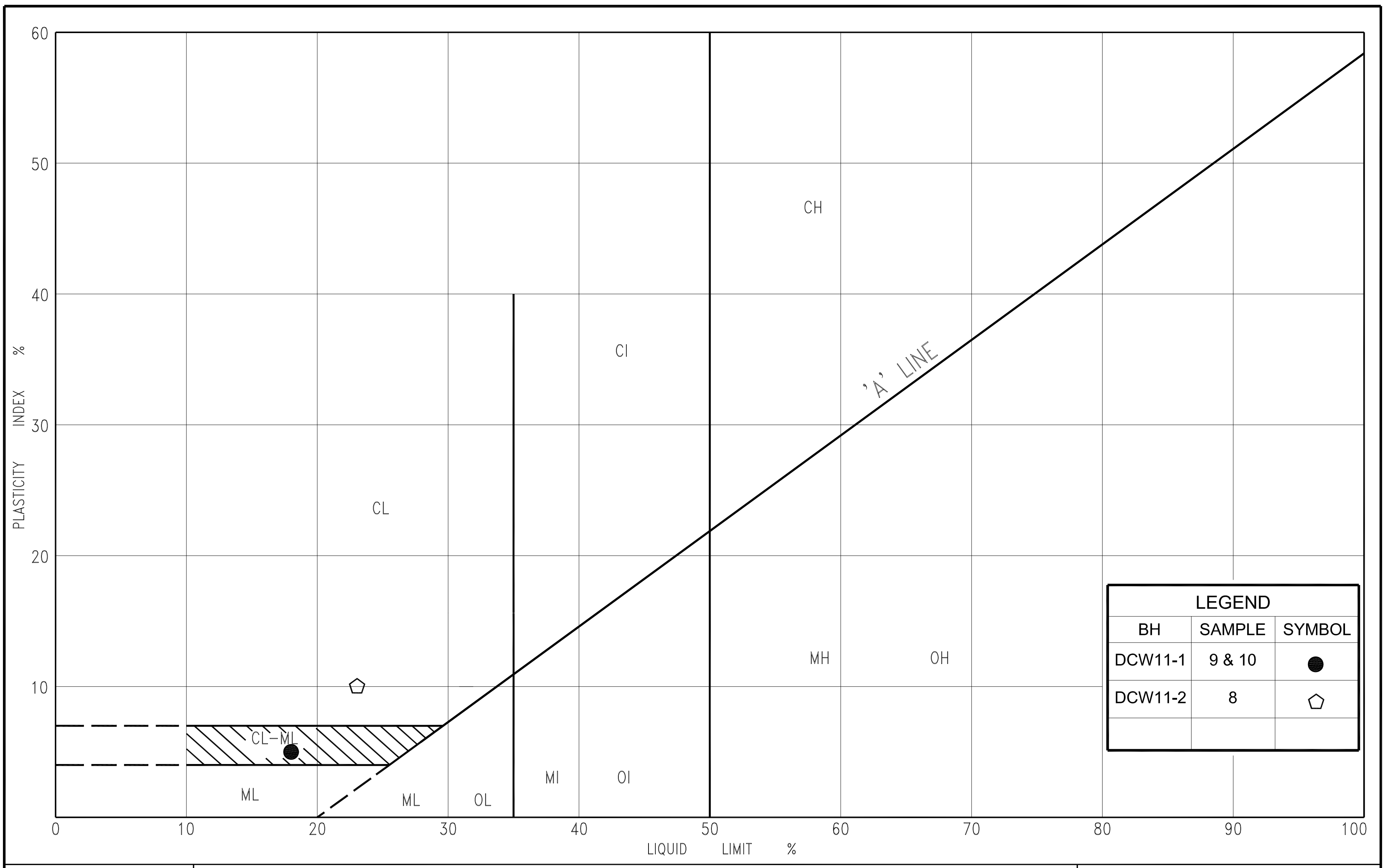
Ontario / Peto MacCallum Ltd. CONSULTING ENGINEERS	**GRAIN SIZE DISTRIBUTION** SAND, trace to some silt	FIG No.	DCW1-GS-2
		HWY:	407E
		W.P. No.	07-20015



Ontario / Peto MacCallum Ltd.	GRAIN SIZE DISTRIBUTION	FIG No.	DCW1-GS-3
	SANDY SILT to SILTY SAND, some to with clay, trace gravel (TILL)	HWY:	407E
		W.P. No.	07-20015



Ontario / Peto MacCallum Ltd.	PLASTICITY CHART	FIG No.	DCW11-PC-1
	CLAYEY SILT, with sand, trace gravel	HWY:	407E
		W.O. No.	07-20015

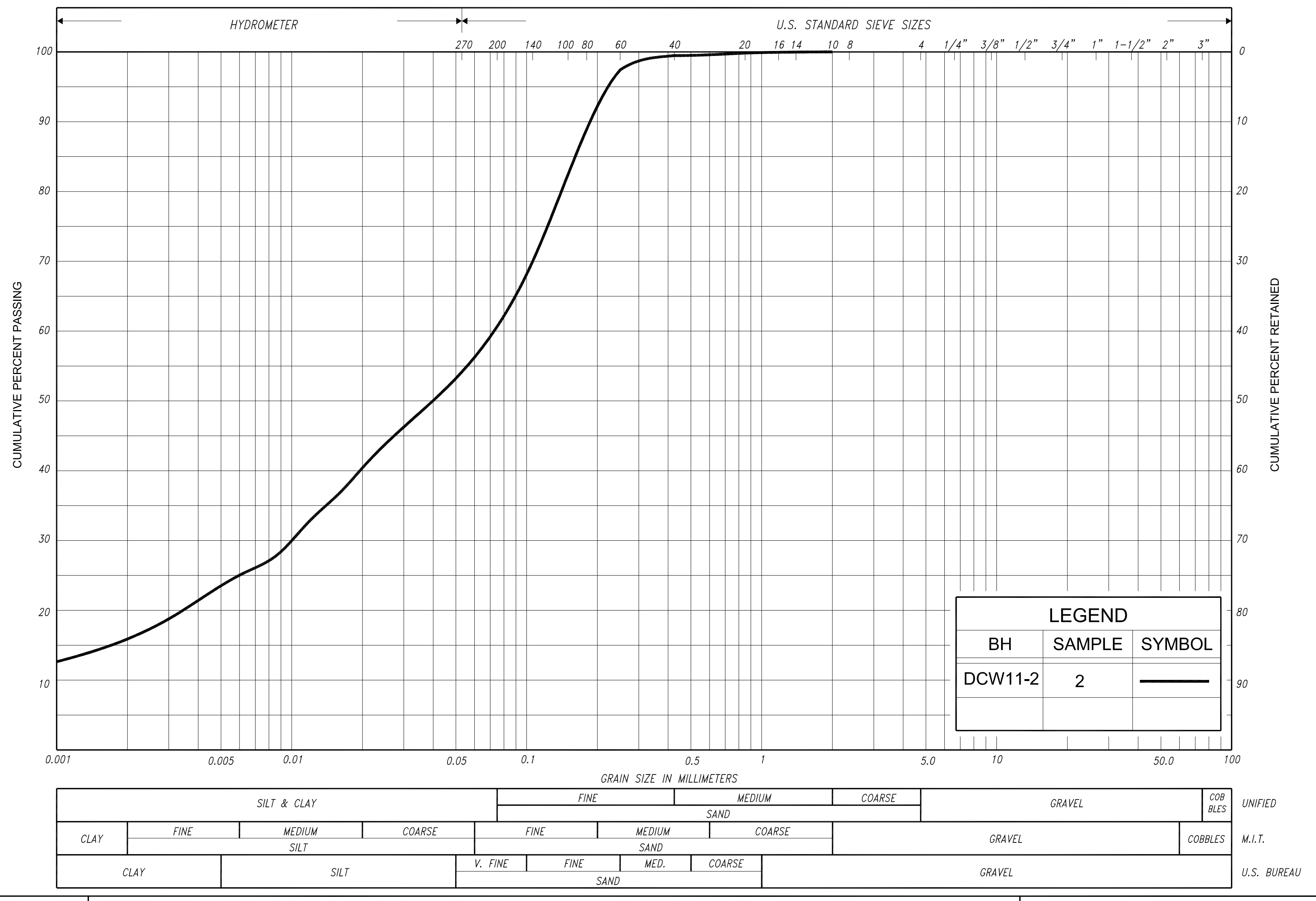


Ontario
Peto MacCallum Ltd. CONSULTING ENGINEERS

PLASTICITY CHART

CLAYEY SILT, with sand to sandy, trace gravel (TILL)

FIG No.	DCW11-PC-2
HWY:	407E
W.O. No.	07-20015

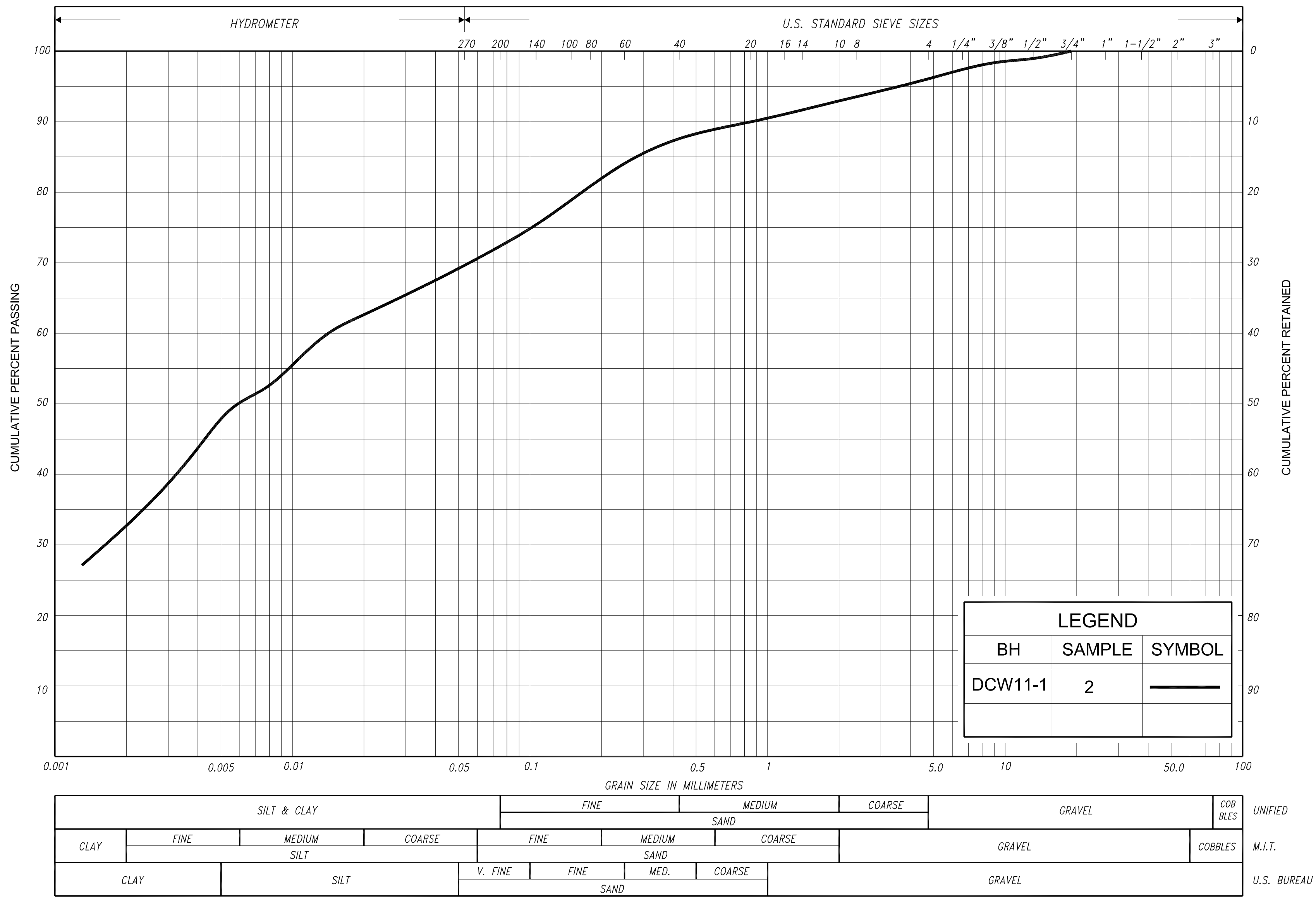


Ontario
Peto MacCallum Ltd. CONSULTING ENGINEERS

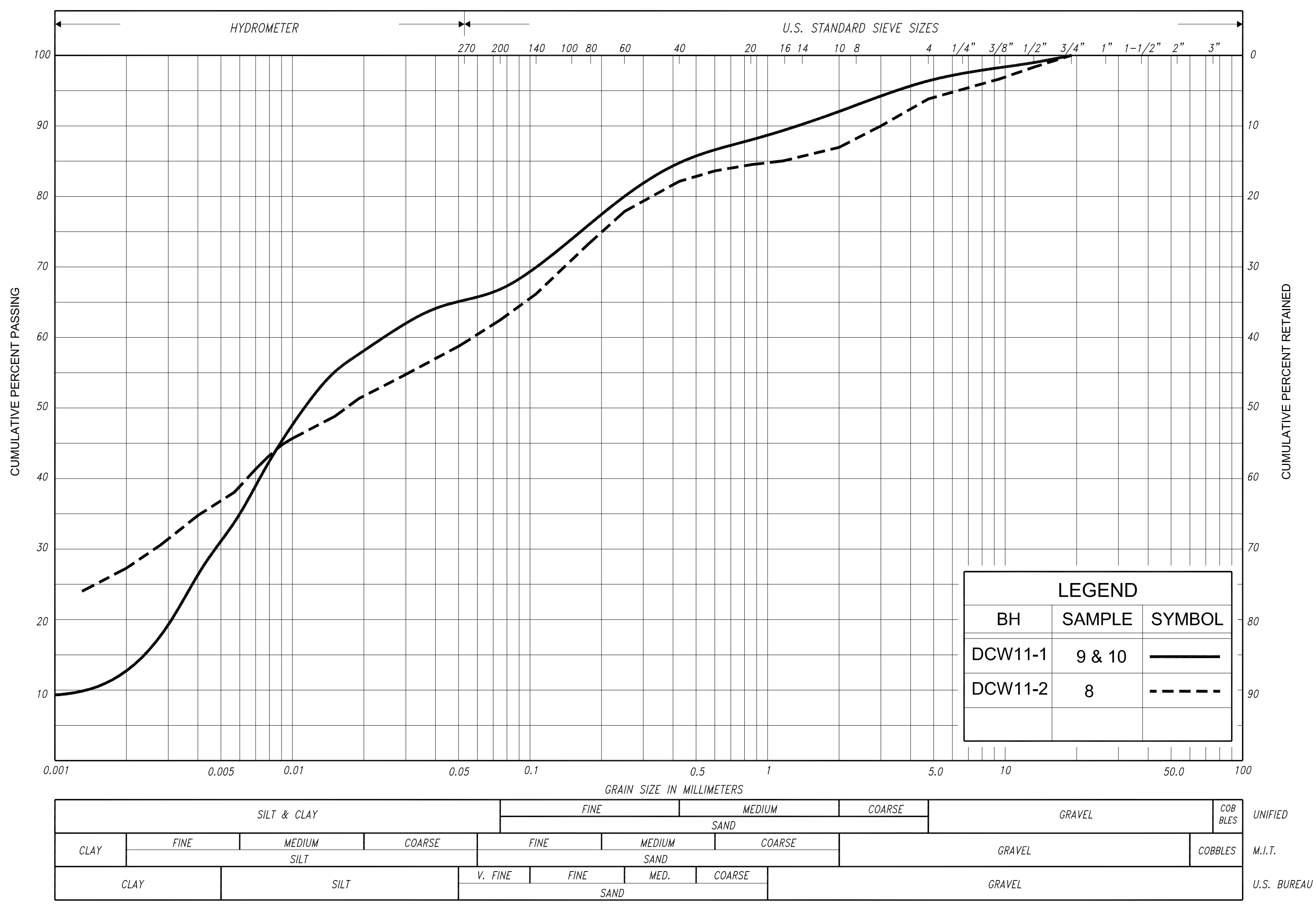
GRAIN SIZE DISTRIBUTION

SANDY SILT, some clay

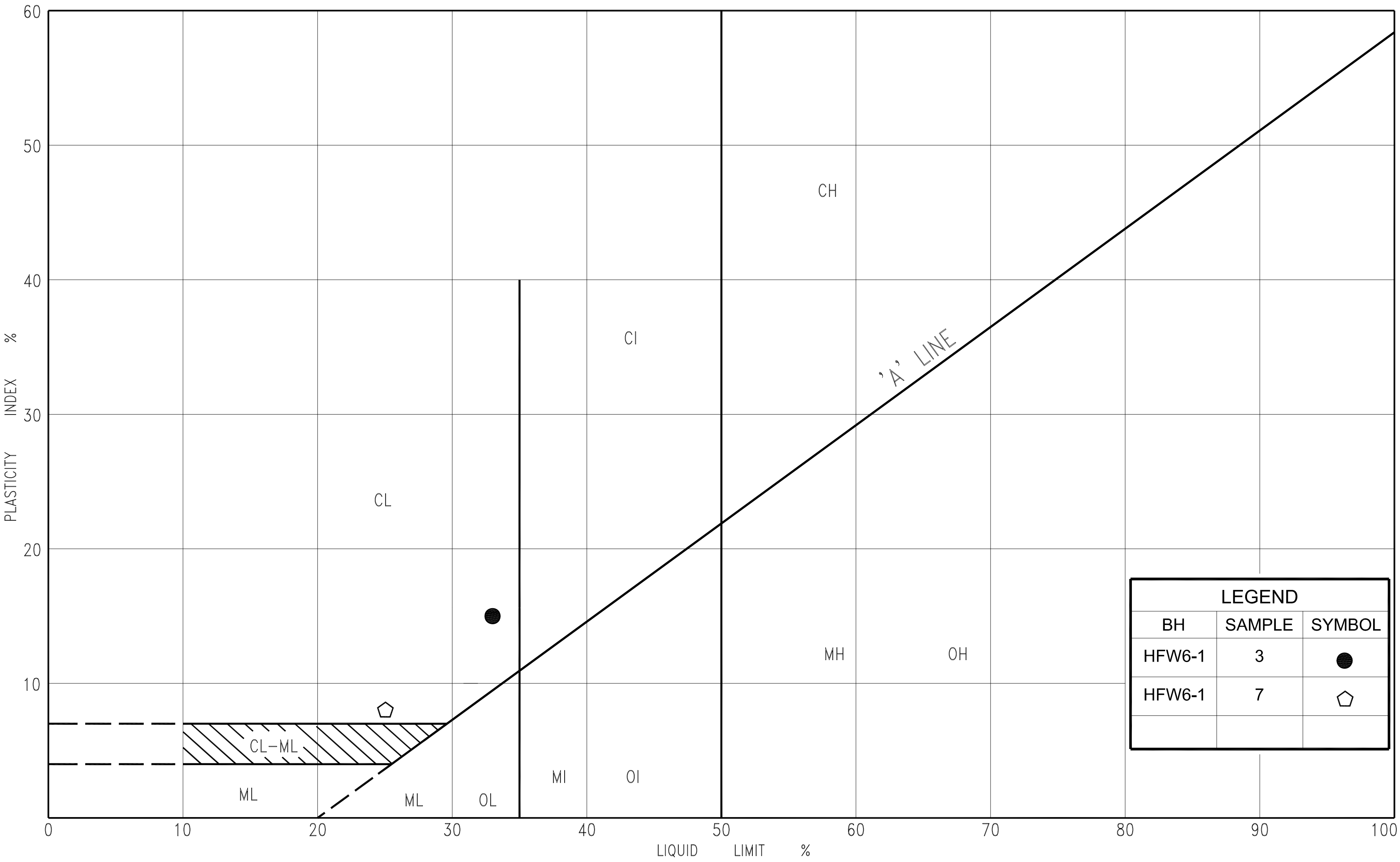
FIG No.	DCW11-GS-1
HWY:	407E
W.O. No.	07-20015



Ontario / Peto MacCallum Ltd.	GRAIN SIZE DISTRIBUTION CLAYEY SILT, with sand, trace gravel	FIG No.	DCW11-GS-2
		HWY:	407E
		W.O. No.	07-20015



Ontario / Peto MacCallum Ltd.	GRAIN SIZE DISTRIBUTION CLAYEY SILT, with sand to sandy, trace gravel (TILL)	FIG No.	DCW11-GS-3
		HWY:	407E
		W.O. No.	07-20015



Ontario

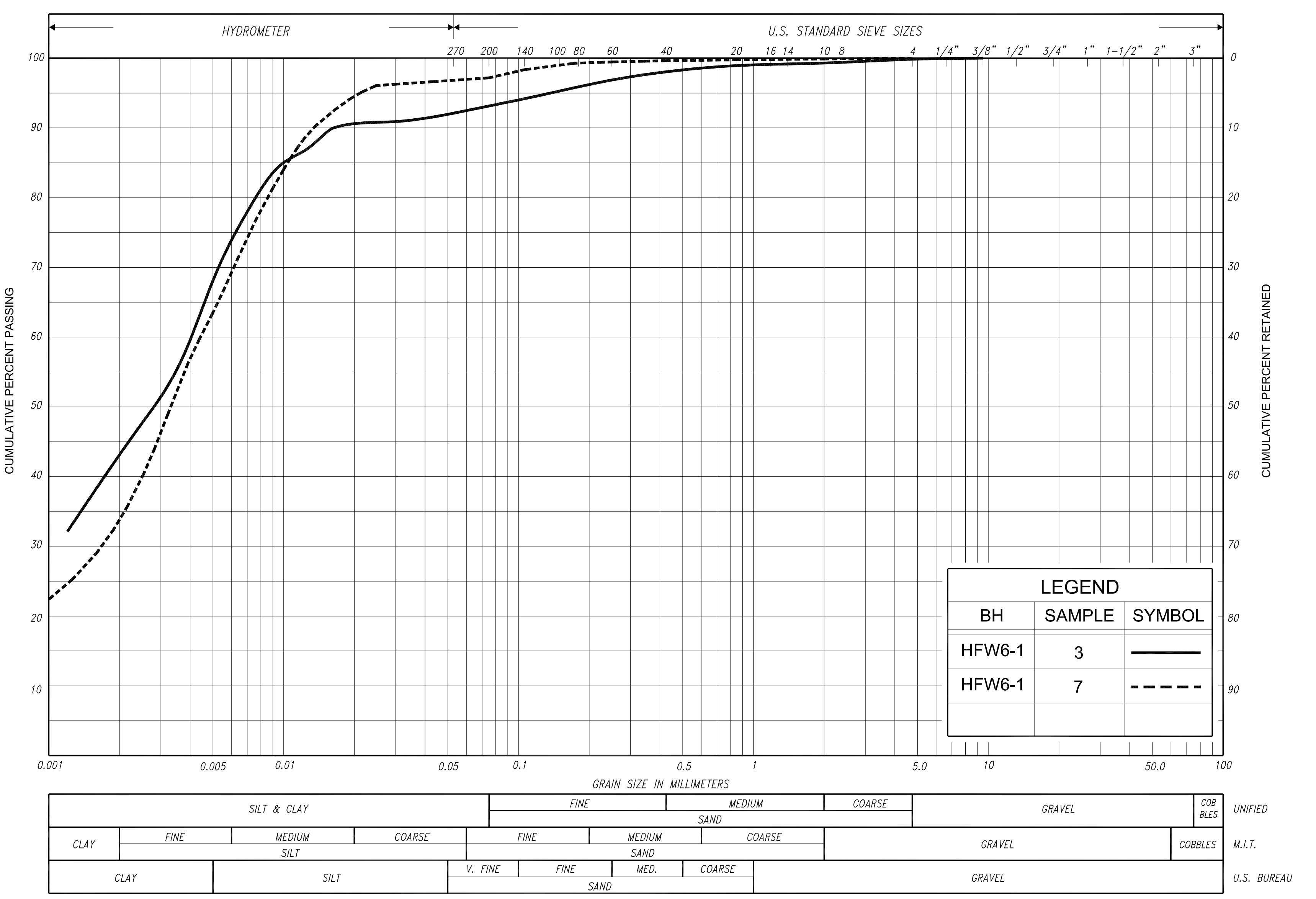
Peto MacCallum Ltd. CONSULTING ENGINEERS

PLASTICITY CHART

CLAYEY SILT, trace sand

(TILL)

FIG No.	HFW6-PC-1
HWY:	407E
W.P. No.	07-20015



Ontario

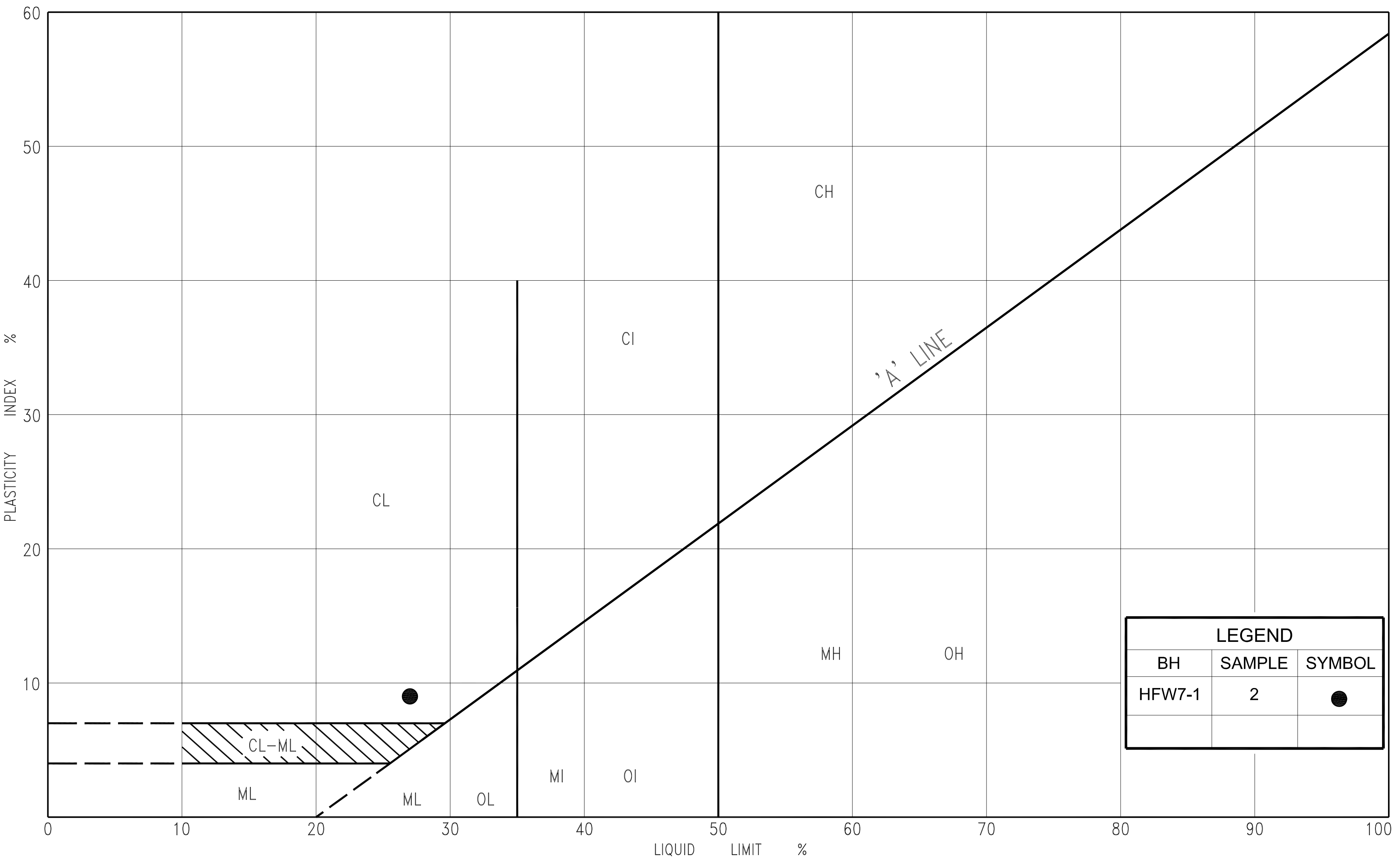
Peto MacCallum Ltd. CONSULTING ENGINEERS

GRAIN SIZE DISTRIBUTION

CLAYEY SILT, trace sand

(TILL)

FIG No.	HFW6-GS-1
HWY:	407E
W.P. No.	07-20015



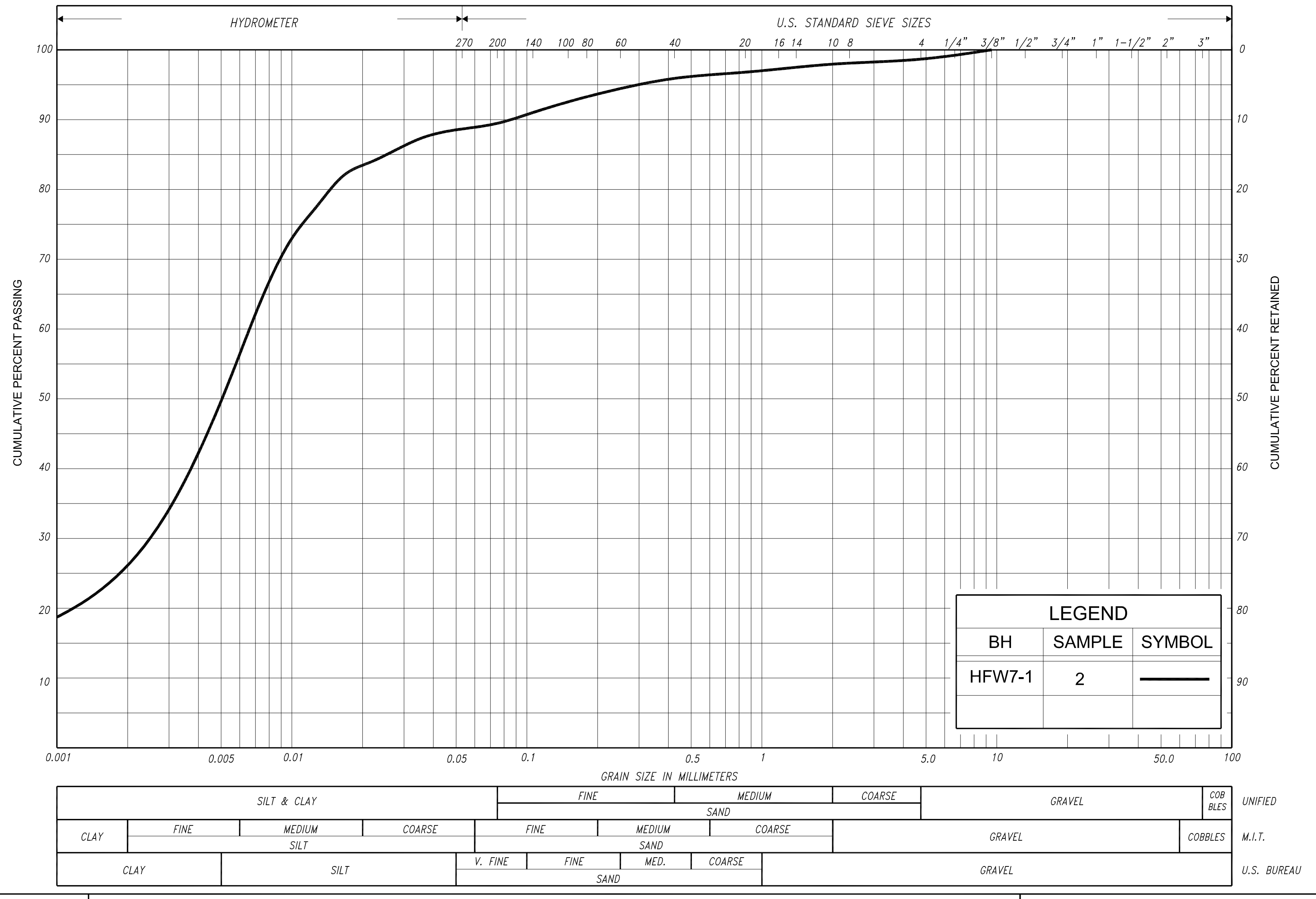
Ontario

Peto MacCallum Ltd. CONSULTING ENGINEERS

PLASTICITY CHART

CLAYEY SILT, trace sand, trace gravel
(TILL)

FIG No.	HFW7-PC-1
HWY:	407E
W.P. No.	07-20015



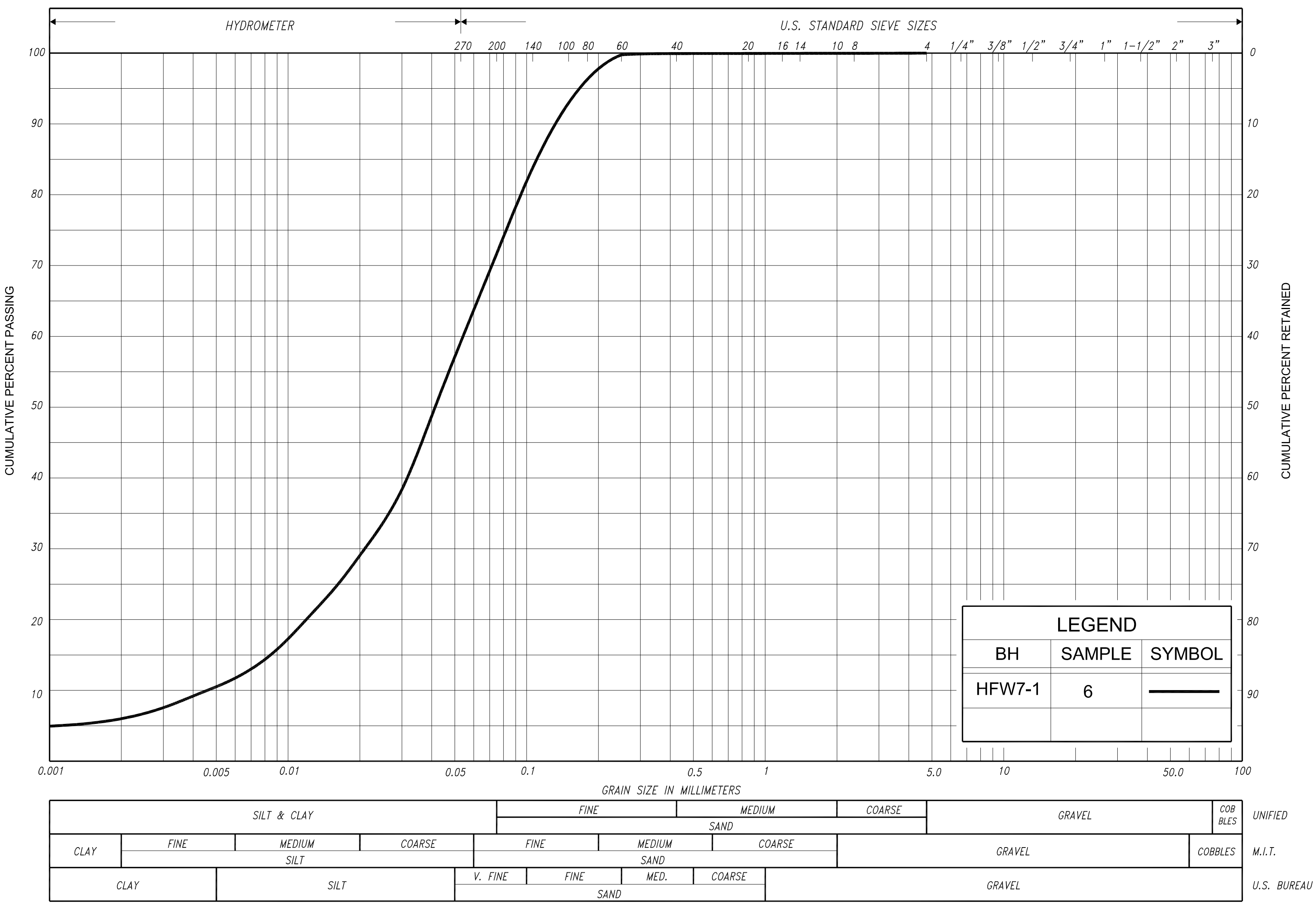
Ontario

Peto MacCallum Ltd. CONSULTING ENGINEERS

GRAIN SIZE DISTRIBUTION

CLAYEY SILT, trace sand, trace gravel
(TILL)

FIG No.	HFW7-GS-1
HWY:	407E
W.P. No.	07-20015



Ontario

Peto MacCallum Ltd. CONSULTING ENGINEERS

GRAIN SIZE DISTRIBUTION

SILT, with sand, trace clay

FIG No.	HFW7-GS-2
HWY:	407E
W.P. No.	07-20015

APPENDIX C

RECORD OF BOREHOLE SHEETS FROM PREVIOUS INVESTIGATIONS

Golder Associates

Foundation Design

RECORD OF BOREHOLE No WM7-1

 METRIC

PROJECT	07-1111-0053		
W.O.	07-20015	LOCATION	N 4864754.2 ;E 337412.5
DIST	Central HWY 407	BOREHOLE TYPE	210 mm O.D. Hollow Stem Augers
DATUM	Geodetic	DATE	March 20, 2009
ORIGINATED BY	TZ	COMPILED BY	DD
CHECKED BY	TZ/KJB		

SOIL PROFILE

ELEV / DEPTH	DESCRIPTION
169.0	GROUND SURFACE
0.0	Silty SAND, trace gravel, trace clay, containing organics; Very loose to loose; Brown; Moist
167.5 / 1.5	CLAYEY SILT, with sand, some gravel, containing cobbles (TILL); Stiff; Brown; Moist
166.6 / 2.4	SAND and GRAVEL, trace to some silt, trace clay; Compact to dense; Brown; Wet
164.4 / 4.6	SAND, some gravel, trace silt, trace clay; Compact to very dense; Brown; Wet
161.0	Silty SAND and GRAVEL, trace clay (Possible TILL); Very dense; Grey; Wet; Grinding of augers on inferred boulder at a depth of 8.4 m
160.6 / 8.4	END OF BOREHOLE; AUGER REFUSAL (POSSIBLE BOULDER)

SAMPLES

NUMBER	TYPE	"N" VALUES	GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	SS	7	
2	SS	4	
3	SS	13	21 28 33 18
4	SS	14	
5	SS	33	40 51 8 1
6	SS	30	
7	SS	17	18 76 5 1
8	SS	25	
9A / 9B	SS	71	29 41 27 3

Elevation scale: 168, 167, 166, 165, 164, 163, 162, 161

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100; SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED; PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 10 20 30; UNIT WEIGHT γ kN/m³

NOTES:

1. Water level measured in piezometer at 0.3 m above ground surface (Elevation 169.3 m) on March 23, 2009.

2. Water level measured in piezometer at 0.4 m above ground surface (Elevation 169.4 m) on April 29, 2009.

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

MIS-MTO 001 07-1111-0053.GPJ GAL-MISS.GDT 5/19/10 DD/SAC

Golder Associates

Foundation Design

RECORD OF BOREHOLE No WM7-2

 METRIC

PROJECT	07-1111-0053		
W.O.	07-20015	LOCATION	N 4864765.1 ;E 337459.7
DIST	Central HWY 407	BOREHOLE TYPE	210 mm O.D. Hollow Stem Augers
DATUM	Geodetic	DATE	January 28 and 29, 2008
ORIGINATED BY	GD	COMPILED BY	DD
CHECKED BY	TZ/HJ		

SOIL PROFILE

ELEV / DEPTH	DESCRIPTION
166.4	GROUND SURFACE
0.0	Silty sand, trace gravel, containing organics (FILL); Very loose to loose; Dark brown; Moist
165.9 / 0.5	CLAYEY SILT with sand, trace to some gravel (TILL); Very stiff to stiff; Brown; Moist
164.1 / 2.3	SAND and GRAVEL, trace to some silt, trace clay; Dense to very dense; Brown; Wet
154.5	SAND, some silt and gravel; Very dense; Grey; Moist
154.1 / 12.3	END OF BOREHOLE

SAMPLES

NUMBER	TYPE	"N" VALUES	GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	SS	4	
2	SS	18	
3	SS	12	8 38 40 14
4	SS	38	
5	SS	56	52 39 7 2
6	SS	59	
7	SS	77	
8	SS	48	50 39 9 2
9	SS	141	
10	SS	100/0.1	
11	SS	100/0.1	

Elevation scale: 166, 165, 164, 163, 162, 161, 160, 159, 158, 157, 156, 155

NOTES:

1. Water level measured in open borehole upon completion of drilling at a depth of 0.9 m below ground surface (Elevation 165.5 m).

Continued Next Page

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

MIS-MTO 001 07-1111-0053.GPJ GAL-MISS.GDT 5/19/10 DD/SAC

RECORD OF BOREHOLE No WM7-2

 METRIC

PROJECT 07-1111-0053			
W.O. 07-20015	LOCATION N 4864765.1 ;E 337459.7	ORIGINATED BY GD	
DIST Central HWY 407	BOREHOLE TYPE 210 mm O.D. Hollow Stem Augers	COMPILED BY DD	
DATUM Geodetic	DATE January 28 and 29, 2008	CHECKED BY TZ/HJ	

SOIL PROFILE: ELEV / DEPTH; DESCRIPTION; STRAT PLOT — SAMPLES: NUMBER; TYPE; "N" VALUES — GROUND WATER CONDITIONS — ELEVATION SCALE — DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100 — PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 10 20 30 — UNIT WEIGHT γ kN/m³ — REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

--- CONTINUED FROM PREVIOUS PAGE ---

2. Water level measured in piezometer at a depth of 2.2 m below ground surface (Elevation 164.2 m) on February 28, 2008.

3. Water level measured in piezometer at a depth of 2.1 m below ground surface (Elevation 164.3 m) on April 4, 2008.

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

RECORD OF BOREHOLE No WMSC7-1

 METRIC

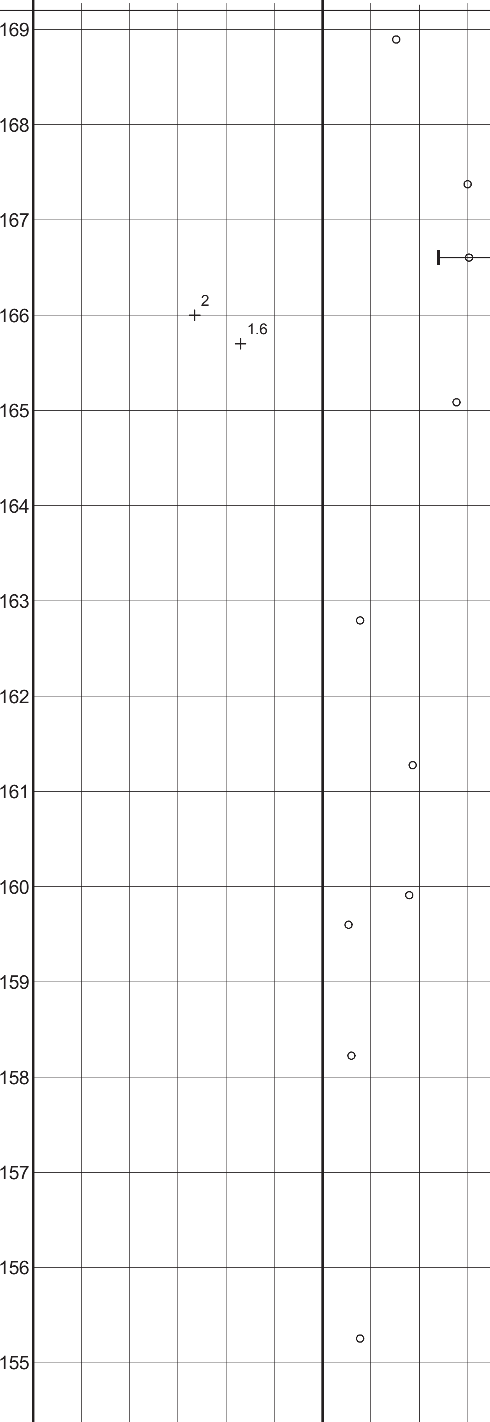
PROJECT			
W.O. 07-20015	LOCATION N 4864889.7 ;E 337466.2	ORIGINATED BY TZ	
DIST Central HWY 407	BOREHOLE TYPE 210 mm O.D. Hollow Stem Augers	COMPILED BY DD	
DATUM Geodetic	DATE March 20 & 23, 2009	CHECKED BY TZ/KJB	

ELEV / DEPTH	DESCRIPTION
169.2	GROUND SURFACE
0.0	Silty sand, trace gravel, trace clay (FILL) Loose Brown Moist
168.4 / 0.8	Organic silt with sand, containing wood fragments (FILL) Very soft Black Moist
167.7 / 1.5	Organic clayey silt, some sand, trace gravel, containing pockets of silty clay (FILL) Firm to stiff Grey Wet
163.6 / 5.6	SAND and GRAVEL Compact Grey Wet
162.0 / 7.2	SAND, some silt, trace to some gravel, trace clay Compact to dense Grey Wet
159.6 / 9.7	SILT Dense Grey Wet
	SAND and GRAVEL, trace to some silt, trace clay Very dense Grey Wet

NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	SS	5	
2	SS	2	OC=21.1%
3	SS	5	
4	SS	6	1 16 47 36
5	SS	7	OC=6.2%
6	SS	8	
7	SS	5	OC=5.6%
8	SS	20	
9	SS	22	8 78 13 1
10A / 10B	SS	41	
11	SS	72	39 45 13 3
12	SS	100/0.20	
13	SS	100	35 55 8 2

ELEVATION SCALE: 169, 168, 167, 166, 165, 164, 163, 162, 161, 160, 159, 158, 157, 156, 155

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 1000 2000 3000 4000 5000 — PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 10 20 30 — UNIT WEIGHT γ kN/m³



Continued Next Page

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

RECORD OF BOREHOLE No WMSC7-1

 METRIC

PROJECT			
W.O. 07-20015	LOCATION N 4864889.7 ;E 337468.2	ORIGINATED BY TZ	
DIST Central HWY 407	BOREHOLE TYPE 210 mm O.D. Hollow Stem Augers	COMPILED BY DD	
DATUM Geodetic	DATE March 20 & 23, 2009	CHECKED BY TZ/KJB	

SOIL PROFILE · SAMPLES · GROUND WATER CONDITIONS · ELEVATION SCALE · DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) · SHEAR STRENGTH kPa (○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED; 1000 2000 3000 4000 5000) · PLASTIC LIMIT W_P / NATURAL MOISTURE CONTENT W / LIQUID LIMIT W_L · WATER CONTENT (%) (10 20 30) · UNIT WEIGHT γ kN/m³ · REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE
	--- CONTINUED FROM PREVIOUS PAGE ---				154
153.6		14	SS	100/0.20	
15.6	END OF BOREHOLE				

NOTES:

1. Water level measured in piezometer at a depth of 4.7 m below ground surface (Elevation 164.5 m) on March 27, 2009.

2. Water level measured in piezometer at a depth of 4.9 m below ground surface (Elevation 164.3 m) on April 29, 2009.

3. An additional borehole was drilled 3.0 m east of Borehole WMSC7-1 to carry out in situ vane testing at depths of 3.2 m and 3.5 m below ground surface (Elevation 166.0 m and 165.7 m).

MIS-MTO 001 07-1111-0053.GPJ GAL-MISS.GDT 4/6/10 DD/SAC

$+^3, \times^3$: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

Golder Associates

RECORD OF BOREHOLE No WL19A-1A

 METRIC

PROJECT 07-1111-0053		
W.O. 07-20015	LOCATION N 4861744.1 ;E 345677.9	ORIGINATED BY TZ
DIST Central HWY 407	BOREHOLE TYPE Portable (Tripod); Wash boring from 3.8 m to 7.0 m depth	COMPILED BY DD
DATUM Geodetic	DATE September 4 and 5, 2008	CHECKED BY TZ/BLT

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
101.0	GROUND SURFACE										
0.0 / 0.1	TOPSOIL; Loose, Black, Moist	1	SS	9							
	SAND, some silt, trace clay; Loose to compact, Brown, Wet	2	SS	10		100		0	6	62	32
99.8 / 99.5 / 1.5	CLAYEY SILT, trace to some sand; Stiff, Grey, Wet	3	SS	9		99					
98.8 / 2.3	SAND, some silt, trace clay; Loose, Brown, Wet	4	SS	5		98		0	78	19	3
98.2 / 2.8	CLAYEY SILT, trace to some sand; Stiff, Grey, Wet	5	SS	4	∇			0	15	36	49
	SAND, trace clay; Loose, Brown, Wet	6	TO	PM		97	17.9				
96.4 / 4.6	SILTY CLAY, some sand; Soft to firm, Grey, Wet	7	SS	13		96					
	SAND and SILT, trace to some gravel and clay (TILL); Compact to very dense, Grey, Wet; Clayey silt with gravel seam from 5.8 m to 6.0 m depth	8	SS	95		95					
94.0		9	SS	100/0.13		94					
7.0	END OF BOREHOLE										

Water content plot annotation: 40.8

NOTE:

1. Water level measured in open borehole before wash boring at a depth of 3.8 m below ground surface (Elevation 97.2 m).

MIS-MTO 001 07-1111-0053.GPJ GAL-MISS.GDT 5/19/10 DD/SAC

$+^3, \times^3$: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No WL19A-2A

 METRIC

PROJECT 07-1111-0053
W.O. 07-20015 | LOCATION N 4861716.4 ;E 345690.7 | ORIGINATED BY TZ
DIST Central HWY 407 | BOREHOLE TYPE Portable (Tripod); Wash boring from 3.0 m to 7.5 m depth | COMPILED BY DD
DATUM Geodetic | DATE September 10, 2008 | CHECKED BY TZ/BLT

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
102.0	GROUND SURFACE				
0.0	TOPSOIL				
0.2	Loose, Black, Moist	1	SS	53	
	SAND, some silt, trace clay; Compact to very dense; Brown; Wet	2	SS	15	
100.1 / 1.9	CLAYEY SILT, some sand; Firm to very stiff; Brown; Wet	3	SS	7	
		4	SS	17	
99.0 / 3.1	SILTY CLAY, trace to some sand; Soft to firm; Grey; Wet	5	SS	6	
		6	SS	4	0 9 39 52
95.9 / 6.1	SAND and SILT, trace to some gravel and clay (TILL); Very dense; Grey; Wet	7	SS	100/0.15	9 41 39 11
		8	SS	100/0.15	
94.5 / 7.5	END OF BOREHOLE				

Shear strength (field vane) values plotted: 1.9, 2.1, 1.7, 2.6

NOTE:

1. Wet soil samples noted at a depth of 0.5 m below ground surface (Elev. 101.5 m) during drilling.

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

RECORD OF BOREHOLE No WL19-2A

 METRIC

PROJECT 07-1111-0053
W.O. 07-20015 | LOCATION N 4861761.3 ;E 345567.1 | ORIGINATED BY TZ
DIST Central HWY 407 | BOREHOLE TYPE Portable (Tripod); Wash boring from 3.5 m to 9.6 m depth | COMPILED BY DD
DATUM Geodetic | DATE September 5 and 8, 2008 | CHECKED BY TZ/BLT

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
105.8	GROUND SURFACE				
0.0	TOPSOIL; Loose; Black; Moist	1	SS	4	
104.9 / 0.9	SAND, some silt, trace clay; Loose; Brown; Moist	2	SS	14	
	CLAYEY SILT, trace to some gravel and clay; Stiff to hard; Brown; Wet	3	SS	33	
		4	SS	56	10 42 36 12
102.8 / 3.1	SAND and SILT, some gravel, trace to some clay (TILL); Very dense; Brown to grey; Wet	5	SS	236	
102.3 / 3.5	SAND, trace to some silt and gravel; Very dense; Grey; Wet	6	SS	62	
101.6 / 4.2	SAND and SILT, some gravel, trace to some clay (TILL); Dense to very dense; Brown to grey; Wet	7	SS	43	
		8	SS	52	30 32 29 9
		9	SS	95	
		10	SS	191	
96.3 / 9.6	SHALE (BEDROCK); END OF BOREHOLE				

NOTE:

1. Water level measured in open borehole before wash boring at a depth of 1.1 m below ground surface. (Elevation 104.7 m).

2. Water level measured in piezometer at a depth of 2.5 m below ground surface (Elevation 103.3 m) on March 23, 2009.

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

Golder Associates

Foundation Design

RECORD OF BOREHOLE No WL19-3A

 METRIC

PROJECT 07-1111-0053

W.O. 07-20015 | LOCATION N 4861734.3 ;E 345611.8 | ORIGINATED BY TZ

DIST Central HWY 407 | BOREHOLE TYPE Portable (Tripod); Wash boring from 3.8 m to 8.4 m depth | COMPILED BY DD

DATUM Geodetic | DATE September 9, 2008 | CHECKED BY TZ/BLT

ELEV / DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	SHEAR STRENGTH kPa	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
104.8	GROUND SURFACE						
0.0 / 0.1	TOPSOIL; Loose, Black, Moist	1	SS	7			
103.8 / 1.0	SAND, some silt, containing organics; Loose, Brown, Moist	2	SS	6	104		
	CLAYEY SILT to SILTY CLAY, trace sand; Firm to hard, Brown, Moist				103	>120; >120	
		3	SS	57	102		0 2 61 37
101.8 / 3.1	SAND and SILT, trace to some gravel, trace clay (TILL); Very dense, Brown, Wet	4	SS	75	101		
100.2 / 4.6	CLAYEY SILT with sand, trace to some gravel (TILL); Stiff, Grey, Wet	5	SS	12	100		11 25 33 31
98.7 / 6.1	SAND and GRAVEL, containing shale pieces; Compact, Grey/black, Wet	6	SS	29	99		
98.1 / 6.7	SAND and SILT, trace to some gravel, trace clay (TILL); Very dense, Brown to grey, Wet	7	SS	152	98 / 97		
		8	SS	100/0.10			10 52 33 5
96.4 / 8.4	END OF BOREHOLE						

Column headings: SOIL PROFILE; SAMPLES; GROUND WATER CONDITIONS; ELEVATION SCALE; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: ○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED, 20 40 60 80 100; PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L, WATER CONTENT (%) 10 20 30; UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

NOTE:

1. Wet soil samples noted at a depth of 3.1 m below ground surface (Elev. 101.8 m) during drilling.

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

MIS-MTO 001 07-1111-0053.GPJ GAL-MISS.GDT 5/19/10 DD/SAC