Peto MacCallum Ltd.
CONSULTING ENGINEERS

ADDENDUM
PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT
HIGHWAY 407 EAST EXTENSION – CENTRAL SECTION (WEST PART)
ASHBURN ROAD TO HARMONY ROAD NORTH
REGION OF DURHAM, ONTARIO
W.O. 07-20016

Submitted to:

Delcan Corporation
625 Cochrane Drive
Suite 500
Markham, Ontario
L3R 9R9

GEOCRES NO: 30M15-113

DISTRIBUTION:

2 Copies - Delcan Corporation, Markham, Ontario + 2 digital copies

4 Copies - Ministry of Transportation Ontario, Downsview, Ontario + 2 digital copies

1 Copy - Peto MacCallum Ltd., Toronto, Ontario

1 Copy - Peto MacCallum Ltd., Kitchener, Ontario

July 2011

PML Ref.: 10TF023ADD-C
Index No.: 141FIDR

TABLE OF CONTENTS

LIST OF DRAWINGS

LIST OF APPENDICES

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 sections (refer to Drawing 1):

- the Western Section that extends from Brock Road in the City of Pickering to Ashburn Road in the Town of Whitby. This section includes a north-south link to Highway 401, designated the West Durham Link.
- the Central Section that extends from Ashburn Road to Courtice Road in the Municipality of Clarington (subsequently divided into west and east parts for the implementation stage).
- the Eastern Section that extends from Courtice Road to Highway 35/115 in the Municipality of Clarington. This section includes a north-south link to Highway 401, designated the East Durham Link.

In 2008, Thurber Engineering Limited (Thurber) carried out a Foundation Desktop Study for each section of the proposed highway extension to assess the potential geotechnical conditions affecting foundation design at the sites of individual structures in advance of site-specific field investigation. The 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 investigations conducted by MTO in 1994 for planning purposes. The results of the 2008 desktop study were presented in three separate reports (*"Foundation Desktop Study, Highway 407 East Extension-Western Section; Central Section; Eastern Section"*, Thurber Engineering Ltd., October 2008).

Subsequently, in 2010, Thurber prepared the Preliminary Foundation Investigation and Design Reports (FIDR) for the west part of the Central Section of the Highway 407 East Extension from Ashburn Road to Simcoe Street North. The preliminary investigation and design reports provided "as near as possible" preliminary design level foundation information for environmental assessment purposes and to assist planning, selection and preliminary design of foundations for bridge, culvert and grade separation structures, as well as for deep cuts and high fill embankments. The Thurber preliminary FIDR superseded all previous reports including the Desktop Study for the purpose of preliminary foundation design and EA submission.

To supplement Thurber's report, Peto MacCallum Ltd. (PML) prepared the Preliminary Foundation Investigation and Design Report (FIDR) on the west part of the Central Section, Reference No. 10TF023-C, dated February 2011, Geocres No. 30M15-111.

This report is prepared by PML as an addendum to the above report. This addendum report also includes the results of preliminary foundation investigations on two (2) bridges within a 3.3 km section between Simcoe Street North and Harmony Road North that has been added to the Phase 1 project and referred to as Phase 1A.

The report is presented in two 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 structures, based on the results of limited borehole investigation and laboratory testing or on the desktop study information.

Part B - Preliminary Foundation Design Report (FDR): provides project-wide engineering recommendations for preliminary design, as well as site-specific preliminary foundation recommendations for each proposed structure, culvert, deep cut and high fill site.

Each highway crossing (grade separation, bridge or culvert) was characterized by Thurber as requiring a low, medium or high level of investigative effort. The target levels are defined in the RFP and summarized in Section 3.0 of this report. The desired investigative effort was achieved at each of the four sites (three structures and one high fill) included in this report.

For each of the sites where borehole information was obtained at or near the site, an individual Preliminary Foundation Investigation and Design Report (FIDR) was prepared. Each FIDR consists of a Preliminary Foundation Investigation Report (FIR) sheet summarizing the results of the field investigation and geotechnical laboratory testing for the site, and a Preliminary Foundation Design Report (FDR) sheet presenting site-specific preliminary foundation design recommendations. The FIR and FDR sheets are presented following the text of the report.

For high fill section HF-C4 (height greater than 4.5 m), a summary table has been included that summarizes the high fill location, height, the anticipated subsurface conditions and preliminary geotechnical recommendations.

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 drilled 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 detail design level subsurface information and confirm/reassess the preliminary recommendations.

PART A

ADDENDUM
PRELIMINARY FOUNDATION INVESTIGATION REPORT
HIGHWAY 407 EAST EXTENSION – CENTRAL SECTION (WEST PART)
ASHBURN ROAD TO HARMONY ROAD NORTH
REGION OF DURHAM, ONTARIO
W.O. 07-20016

1.0 INTRODUCTION

This addendum report presents the factual findings obtained from a preliminary foundation investigation carried out by Peto MacCallum Ltd. (PML) on April 5 and June 6, 2011 to supplement PML's preliminary investigation conducted in December 2010 and the preliminary investigation carried out by Thurber Engineering Ltd. (Thurber) in the period of December 2007 to April 2009 for the preliminary design of the proposed Highway 407 East Extension - Central Section (West Part) from Ashburn Road in Whitby to Harmony Road North in Oshawa, Ontario (refer to Drawing 1).

This addendum report provides sufficient information for planning and preliminary foundation investigation and design for a total of three (3) structure sites of which two (2) sites are bridges and one (1) site is a culvert. In addition, one (1) high fill area was included in the study for the Phase 1/1A Central Section (West Part).

The purpose of the preliminary investigations was to explore the subsurface conditions in the vicinity of the proposed grade separation structures, bridges, culverts, deep cuts, and high fills along the alignment of the proposed highway extension and, based on the data obtained, to provide borehole location and soil strata drawings, records of boreholes, laboratory test results and written descriptions of the subsurface conditions for the investigated structures.

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. Thurber carried out the investigation as a sub-consultant to AECOM Canada Ltd. (Totten Sims Hubicki acting as AECOM), under MTO Purchase Order No. 2007-E-0041. The terms of reference and scope of work for the preliminary investigation and design are outlined in MTO's Request for Proposal (RFP) for Work Order No. 07-20016.

2.0 PROJECT DESCRIPTION

The technically recommended route for the Central Section of the proposed Highway 407 East Extension consists of an approximately 16 km long highway from Ashburn Road in Whitby to Courtice Road in Clarington. Phase 1 of the implementation stage is to include the west part of the Central Section, which is an approximately 6 km long section from Ashburn Road in Whitby to Simcoe Street North in Oshawa. MTO added Phase 1A extending 3.3 km to the east of the Phase 1 limits from Simcoe Street North to Harmony Road North in Oshawa.

The proposed Highway 407 Mainline route runs primarily through farmland, crossing a number of creek valleys, tributaries, and municipal and regional roads. The mainline section crosses the Lynde, West Oshawa and Oshawa Creek valleys. The overall surface topography is gently sloping downward to the east and south towards Lake Ontario.

Along the west part of the Central Section route to Harmony Road North there are a total of 14 structure sites, where the highway crosses roads or watercourses. These consist of 10 grade separation/bridge sites and 4 culvert sites. Each site includes one or more structure depending on the configuration of the crossing (e.g. twin bridge structures, interchange ramp grade separation, etc.). The location of each structure site is shown in Drawing 2 – Key Location Plan.

Each structure was initially designated with a prefix of 'CM' for Central Mainline and a sequential number. For multiple structures at a site, a letter was added for additional structures in the group (eg. CM-3 and CM-3b for twin overpasses at the same site). The initial structure numbering system was retained by Thurber for the preliminary foundation report, however a new structure numbering system was subsequently provided by AECOM for the Environmental Assessment submission. A cross-reference of site numbers is provided in Table 1, Section 4.2. It is noted that PML has used the new structure numbering system at the sites, with boreholes featuring a prefix 'M', site number and a sequential borehole number.

In addition to the bridge and culvert structures, this report also addresses one high fill (height of fill exceeds 4.5 m) along the proposed alignment. The high fill section is given in Table 2 in Section 4.2.

3.0 INVESTIGATION PROCEDURES

During the Desktop Study previously carried out by Thurber, each site was categorized as requiring either a low, medium or high level of investigative effort for the preliminary foundation investigation. The level of investigative effort was assigned by using existing geological information, available boreholes from previous investigations, and site photographs taken by Thurber, and was based on the anticipated soil conditions at the site as well as the type and span length of the structure.

Based on the level of investigative effort assigned to each structure site, the proposed number of boreholes for the preliminary foundation investigation was determined as specified in the RFP and summarized below:

- Low Level Investigative Effort: no borehole investigation required;
- Medium Level Investigative Effort: two representative boreholes at the site; and
- High Level Investigative Effort: four boreholes at strategic locations at the site.

During the course of the project, several structures were added, deleted or modified, which changed the structure category, configuration and target level of investigation. The structure designation, category, location and investigative effort applied during the preliminary investigation are summarized in Table 1 in Section 4.2.

The proposed number of boreholes for the deep cut and high fill sections was based on the length of the deep cut or high fill and the availability of existing information from boreholes drilled at adjacent structures.

The subsurface investigation by PML was conducted on April 5 and June 6, 2011, and involved a total of 6 boreholes (5 for structure sites M-51, M-57, M-58 and 1 for high fill section HF-C4) drilled to depths of 6.7 to 16.9 m. Selected borehole data from Thurber's investigation was also used for this report. The borehole locations are shown on Drawings 5 and 6 relative to the proposed highway alignment and structure locations provided by AECOM.

PML established borehole locations in the field and J.D. Barnes Land Surveyors provided their co-ordinates and ground surface elevations at the boreholes. Thurber measured the borehole locations and elevations in the field using a Trimble Pathfinder ProXRT GPS unit with an accuracy of +/- 0.5 m. The northing and easting coordinates were based on MTM NAD83, with the ground surface elevations referenced to the Geodetic datum. All borehole locations were checked for the presence of underground utilities prior to drilling.

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 stem augers, hollow stem augers or mud rotary drilling techniques. Soil samples were obtained at selected intervals using a split spoon sampler in accordance with the Standard Penetration Test (SPT) procedure.

The boreholes drilled for the structure sites were advanced to competent strata and generally penetrated 3 m into 'refusal' material, defined as material with a minimum SPT value of 100 blows per 0.3 m penetration. The borehole drilled for the high fill section was advanced to a depth at least equal to the height of the fill or to competent material.

The groundwater conditions in the open boreholes were observed throughout the drilling operations. At structure sites M-51 and M-57, at least one piezometer was installed in a selected borehole to permit longer term groundwater level monitoring. The piezometers consisted of 19 to 25 mm diameter PVC pipe with a 1.5 m long slotted screen installed and enclosed in filter sand. The annular space between the piezometer pipe and borehole wall above the filter sand was backfilled with bentonite.

A total of 23 piezometers were installed by Thurber and PML as part of the subsurface investigation for this section. The locations of the piezometers are listed in Table 3 in Section 4.3. All other boreholes were backfilled with bentonite to the ground surface on completion of drilling in accordance with Ontario Regulation 903 (as amended by Ontario Regulation 372/07). After the final water level readings, all piezometers were decommissioned in accordance with Ontario Regulation 903.

Where artesian groundwater conditions were encountered in the boreholes (e.g. M51-1), 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 current drilling and sampling operations were supervised on a full-time basis by members of PML's technical staff. The field supervisor logged the boreholes and processed the recovered soil samples for transport to PML's laboratory for further examination and testing.

The recovered soil samples were subjected to Visual Identification (VI) and to natural moisture content determination. Selected samples were also subjected to gradation analysis and Atterberg limits testing. Relevant laboratory test results prepared by Thurber were also used in this report. The results of the drilling and laboratory testing are shown on the Record of Borehole sheets in Appendix A and in the figures in Appendix B.

4.0 SITE GEOLOGY AND STRATIGRAPHY

4.1 Regional Geology

The alignment of the proposed Highway 407 East Extension – Central Section 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* and described below:

The South Slope region: the majority of the central 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 streams flowing towards Lake Ontario.

The Oak Ridges Moraine region: located north of the central 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 Iroquois Plain region: located south of the central section alignment and extending southward to Lake Ontario. The area across the Regional Municipality of Durham is a complex mix of till plains, drumlins and areas of glaciolacustrine sediments deposited in Lake Iroquois – primarily sands, silts and gravels.

The bedrock within the project area underlies thick overburden sediments throughout the analysis area and consists of blue-grey shale of the Blue Mountain Formation and limestone from the Lindsay Formation. The bedrock 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

Table 1 summarizes the structure sites, category (i.e. bridge or culvert), location, site ranking (level of investigative effort), and boreholes advanced at or adjacent to each site as part of the current and/or past investigations. Creek and floodplain crossings are also indicated, many of which are environmentally sensitive locations that will require special consideration in this regard during preliminary design. The table includes the new structure numbers (as of October 2009), cross-referenced with the structure numbers used for Thurber's foundation report, and the Watercourse IDs provided by AECOM.

For all medium or high ranking sites where boreholes were drilled during the investigations, a Preliminary Foundation Investigation Report (FIR) sheet was produced, which summarizes the results of the field investigation and geotechnical laboratory testing for each structure and includes a borehole location plan and soil strata drawing. The FIR sheets are presented following the text of the report. Following each FIR sheet is a Preliminary Foundation Design Report (FDR) sheet that includes site specific preliminary foundation recommendations for each site, referenced in Part B of this report.

For the sites investigated during the current study, a summary of the soil and groundwater conditions encountered at each site, together with site-specific drawings showing the borehole locations and stratigraphic profile, are presented on the individual Preliminary FIR sheets following the text of this report.

For the remaining sites, refer to the two *Preliminary Foundation Investigation and Design Report – Central Section, W.O. 07-20016* prepared by Thurber in April 2010, Ref. No. 19-2805-10, Geocres No. 30M15-103 and by PML Ref. No. 10TF023-C, Geocres No. 30M15-111.

Table 1 – Structure Summary

New Structure No.	Structure No. used for Thurber's Foundation Report	Watercourse ID	Category	Location	Site Ranking	Boreholes by Thurber	Boreholes by PML	Remarks
M-51	CM-13	CM-TAOCW-32	Culvert	Oshawa Creek West Branch East Tributary (Mainline)	Medium	–	M51-1, M51-2	Refer to FIDR sheet
M-57	CM-20/20b	–	Bridge	Oshawa Creek East Branch East Tributary (mainline)	Medium	CM20-2a, CM20b-4	M57-1, M57-2	Refer to FIDR sheet
M-58	CM-20c	–	Bridge	Oshawa Creek East Branch East Tributary (mainline)	Medium	CM20c-3	M58-1	Refer to FIDR sheet

Table 2 summarizes the section where the proposed highway is to be constructed over a high fill. The table shows the fill (HF) number, location (station to station), maximum fill height and the borehole advanced during the investigation.

The subsurface conditions at the high fill section are summarized in the Preliminary Foundation Investigation Report "High Fills" table following the FIDR sheets for the structures.

Table 2 – High Fill Summary

Deep Cut (DC) or High Fill (HF) Number	Station (From – To)	Maximum Depth Height (m)	Boreholes by Thurber	Boreholes by PML
HF-C4	11+366 to 11+616 (new chainage) 16+750 to 17+000 (old chainage)	5.5	–	HFC4-1

The detailed subsurface soil and groundwater conditions as encountered in the boreholes advanced during these investigations, and the results of geotechnical laboratory tests carried out on selected soil 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 the 1994 MTO investigations located along the Highway 407 alignment in this section are provided in Appendix C and approximate locations (converted to MTM NAD 83 coordinates) are shown on Drawings 3 to 6.

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.

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 23 borehole locations as part of the current and previous investigations for the project. Details of the four (4) piezometer installation 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. 30M15-111 for a list of the nineteen (19) previously installed piezometers and reference water level readings.

The groundwater levels measured in the four new piezometers range from 0.7 m above the ground surface to 4.2 m below the ground surface. The most recent water levels measured in the piezometers are summarized in Table 3.

It should be noted that artesian water conditions were observed at the location of borehole M51-1 advanced near an Oshawa Creek West tributary. Details of the site-specific groundwater conditions at each site are provided on the Preliminary FIR sheets, following the text of this report.

Groundwater levels are expected to fluctuate as a result of seasonal variations in precipitation and runoff.

Table 3 – Water Level Measurements

Borehole Number	Ground Surface Elevation (m)	Depth to Water Level below Ground Surface (m)	Water Level Elevation (m)	Date
M51-1	180.8	(0.7)*	181.5	May 4, 2011
M57-2	210.6	4.2	206.4	June 17, 2011
CM20-2a	203.9	0.4	203.5	February 12, 2009
CM20c-3	203.7	1.3	202.4	June 6, 2009

* Artesian conditions

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.

Peto MacCallum Ltd.

Grigory O. Degil, PhD, P.Eng.
Senior Foundation Engineer

Carlos M.P. Nascimento, P.Eng.
Manager, MTO Foundation Services

Brian R. Gray, MEng, P.Eng.
MTO Designated Principal Contact

GD/CN/BRG:mi

PART B

ADDENDUM
PRELIMINARY FOUNDATION DESIGN REPORT
HIGHWAY 407 EAST EXTENSION – CENTRAL SECTION (WEST PART)
ASHBURN ROAD TO HARMONY ROAD NORTH
REGION OF DURHAM, ONTARIO
W.O. 07-20016

6.0 ENGINEERING RECOMMENDATIONS FOR PRELIMINARY DESIGN

6.1 General

This section of the addendum report provides preliminary geotechnical recommendations to assist selection and preliminary design of foundation systems for the proposed bridge and grade separation structures along the Highway 407 East Extension-Central Section (West Part) mainline route to Harmony Road North . Preliminary geotechnical recommendations for the design of culverts are discussed in Section 7.0. Recommendations for the high fill section are discussed in Section 8.0.

The preliminary foundation design recommendations provided herein are based on interpretation of the factual data obtained during limited borehole investigations conducted for the current and previous studies as well as boreholes available from previous MTO investigations.

The subsurface investigation was generally limited to borehole drilling within accessible areas of the structure sites, but not necessarily within the footprint of the foundation elements. Further investigation at the final locations of the foundation elements and approaches will be required during detail design to establish detail design level subsurface information and confirm/reassess the preliminary design recommendations.

The interpretation and recommendations are intended to provide the designers with preliminary information to assess feasible foundation alternatives for the preliminary design of the proposed structure foundations. 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.

6.2 Structure Foundation Recommendations

As discussed in Section 2.0, 14 bridge and grade separation structures are currently proposed for the Highway 407 central section mainline (west part) extending to Harmony Road North. Of these, 10 structures were completely investigated and preliminary recommendations for design and construction were provided by Thurber. One of the two remaining structures (M-48) was investigated by PML in December 2010. This addendum report deals with three structures, namely culvert M-51 and bridges M-57 and M-58. Preliminary foundation recommendations for each individual site are provided following the text of this report, in the following form:

- Where boreholes were advanced, individual Preliminary Foundation Investigation and Design Report (FIDR) sheets were prepared, including a description of the proposed structure configuration at the time of report preparation. Part B of the FIDR sheets, referred to as the Preliminary Foundation Design Report (FDR), presents the preliminary foundation recommendations.

The FDR sheets provide a comparison of the advantages and disadvantages of the various foundation alternatives for each site, recommendations for preliminary design of the feasible foundation types, and a recommendation regarding the preferred foundation alternative from a geotechnical viewpoint. Site-specific comments concerning the abutment type, approaches, construction considerations, and recommendations for additional work are also presented.

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

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) code.

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 structure approaches are provided where subsoil conditions are considered to be suitable for shallow foundations, as indicated on the individual Preliminary FDR sheets for each 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 thickness of Granular ‘A’ is placed below the base of the footing. The Granular ‘A’ core should extend at least 1 m beyond the plan limits of the footing and be sloped no steeper than 1 Horizontal to 1 Vertical (1H:1V) in general accordance with MTO guidelines (See Figure 1). The Granular ‘A’ core should be compacted to 100% of its standard Proctor maximum dry density at ±2% of optimum moisture content.

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. The preliminary values are for vertical, concentric loads. In accordance with Sections 6.7.3 and 6.7.4 of the *Canadian Highway Bridge Design Code* (CHBDC 2006), the design must also account for the effects of any eccentric or inclined loads. The resistance values should be re-evaluated and modified if necessary during detail design based on 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).

6.2.2 Steel H-Piles

Preliminary recommendations for steel H-piles, assuming an HP 310 x 110 pile section, are provided on the individual Preliminary FDR sheets for sites where pile foundations are considered practical. 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 are provided, along with the anticipated pile depth/pile tip elevation based on the subsurface conditions encountered.

The factored ULS resistance, SLS reaction values and pile tip elevations should be re-evaluated during the detail design stage in consideration of additional subsurface data obtained during investigation at the locations of each foundation element.

The pile tip elevations are provided for preliminary estimating purposes only. The actual pile tip elevations will be controlled in the field by use of the Hiley formula. Pile installation should be in accordance with MTO's OPSS 903 and Standard Structural Drawing SS103-11 using an ultimate geotechnical resistance of two times the factored ULS design load. The pile termination or set criteria will be dependent on the pile driving hammer type, helmet, selected pile size and length of pile.

Where downdrag loads are indicated on the FDR sheets, the structural design of the piles should include a check to confirm that the factored permanent loads plus downdrag loads do not exceed the factored below-ground structural resistance of the pile at the neutral plane (CHBDC Section 6.8.4 and Commentary).

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 reaction values 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 CHBDC *Commentary* (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).

Where very dense or hard soils are present (SPT N-values exceeding 100 blows), pre-augering may be required to provide an adequate length of pile.

Till deposits often contain cobbles and boulders, and the potential exists that these will be encountered during pile installation. Where applicable, the piles should be reinforced with driving shoes as per OPSD 3000.100 for protection during driving. Pile installation and driving shoes should be in accordance with MTO's OPSS 903.

Where artesian groundwater conditions are present, specialized construction techniques will be required to mitigate the 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 of an impermeable plug and granular drainage layer. Specialized measures may also be required to minimize disturbance in sensitive wetland areas. Sites with artesian conditions should be extensively investigated and foundation installation procedures re-assessed during detail design.

6.2.3 Caissons

Preliminary foundation recommendations for caissons founded within "100-blow" deposits are provided on the individual Preliminary FDR sheets where caissons are considered to be a practical foundation alternative.

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 are provided for caisson diameters equal to 1.2 and 1.5 m. The geotechnical resistance values are associated with a recommended caisson base elevation and/or embedment depth into the "100-blow" material, as the caisson will typically derive the majority of its capacity from base resistance. Shaft resistance has also been taken into account assuming permanent steel liners are required.

The factored ULS resistance and SLS reaction values should be re-evaluated during the detail design stage in consideration of additional subsurface data obtained during detailed investigation at the locations of each foundation element.

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.

In general, the use of caisson foundations has not been recommended at locations where water-bearing cohesionless strata are anticipated, due to the potential for caving of the caisson sidewalls or instability or boiling at the caisson base. Where caisson foundations are considered, temporary or permanent caisson liners may be required to support cohesionless soils below the groundwater level and permit cleaning and inspection of the caisson base. Installation procedures, such as maintaining a constant head of water/drilling mud inside the caisson followed by tremied concrete placement, may also be required. Caissons should not be founded in cohesionless soils with artesian water conditions.

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. 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. 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.

Cobbles and/or boulders may be encountered within the till deposits as indicated in the FDR sheets. Caisson drilling equipment must be capable of penetrating such obstacles, where applicable.

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).

6.3 Abutment and Retaining Walls

Comments regarding the suitability of conventional, semi-integral or integral abutment types at each site are presented on the Preliminary FDR sheets. Abutment walls and associated retaining/wing walls may consist of either of the following:

- Concrete retaining walls supported on spread footings or on deep foundations depending on the site-specific subsoil conditions as discussed on the FDR sheets. The preliminary foundation recommendations for this type of retaining wall can be considered similar to those provided for the structure foundation elements.

- Retained Soil System (RSS) walls founded on soils that will limit settlements to tolerable levels and provide an adequate factor of safety against global instability. In general, RSS walls should be specified to be "High Performance" and "High Appearance".

The performance of a RSS is dependent on, among other factors, the characteristics of its foundation. To provide an acceptable foundation performance, the RSS mass must be founded on competent native soils or on engineered fill consisting of OPSS Granular "A" material. Topsoil, alluvium, loose fill, and any soft/wet native material should be stripped from the footprint of the RSS. The entire block of reinforced earth must be designed against various modes of failure including sliding and overturning, and the global stability must be analyzed after the location of the wall is known.

For sites where settlement of the approach fill has been identified as a potential issue (i.e. where soft cohesive deposits were encountered), the selected wall type and impact of approach fill settlement on the retaining wall must be assessed. 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 abutment walls and any associated retaining walls/wing walls will depend on the type and method of placement of the backfill materials, the nature of the soils behind the backfill, the magnitude of surcharge including construction loadings, the freedom of lateral movement of the structure, as well as the drainage conditions behind the walls.

The following general recommendations are made concerning the design of the walls. It should be noted that these recommendations and parameters assume a 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 C6.9.1 of the CHBDC (2006).

- Select free-draining granular fill meeting the specifications of Ontario Provincial Standard Specifications (OPSS 1010) 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 fill should be compacted in accordance with Special Provision SP 105S10. Backfill, subdrain and frost taper requirements must be in accordance with OPSD 3101.150 and 3121.150.

- 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:
  - Rotation of approximately 0.002 about the base of a vertical wall;
  - Horizontal translation of 0.001 times the height of the wall; or
  - A combination of both.

- 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 SP 105S10. Other surcharge loadings should be accounted for in the design, as required.

6.5 Structure Approaches

Based on the available information provided at each site, recommendations associated with the approach stability and settlement are provided on the individual Preliminary Foundation Design Report sheets following the text of this report. The following subsections provide additional generic recommendations associated with the preliminary design and construction of the approaches.

6.5.1 Subgrade Preparation and Embankment Construction

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 floodplain 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 subexcavation or additional compaction prior to fill placement.

Embankment fill should be placed and compacted in accordance with MTO's SP 206S03 and SP 105S10. 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.

Where approach cuts extend below the groundwater table, the design must include measures to stabilize the cut slope face if instability is experienced. Further comments in this regard are presented in Section 8.0.

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.

6.5.2 Approach Embankment Stability

Preliminary assessment of the stability of the approach embankments at selected sites was carried out 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.

The analyses were based on soil profiles deduced from the current limited borehole data and existing information, and the maximum embankment heights indicated by profile and general arrangement drawings available at the time of the analysis. Approach embankment side slopes no steeper than 2H:1V, with a minimum 2 m wide mid-slope bench for embankment heights greater than 8 m, were assumed. Where designated as safe 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.

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.

The preliminary assessment of stability of the approach 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.

6.5.3 Approach Embankment Settlement

Settlement of the approach 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 site-specific subsoil conditions deduced from the 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 Design Report sheet for the particular site. 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 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.

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 develop mitigation measures such as preloading, surcharging or use of light weight fill to reduce anticipated settlements to acceptable levels where necessary.

6.6 Seismic Considerations

The peak zonal acceleration ratio for the project site is 0.05 g as per The Town of Oshawa, 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).

Seismic (earthquake) loading on the abutment stem and retaining/wing wall must be considered in the design of the foundations in accordance with Sections 4 and 6 of CHBDC (2006). 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). The static and seismic earth pressure coefficients can be determined in accordance with Sections 6.9 and 4.6.4 of the CHBDC (2006) and its Commentary.

The susceptibility to liquefaction 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 Sections C.4.6.2 and C.4.6.3, respectively, of the CHBDC Commentary (2006).

6.7 Construction Considerations

6.7.1 Obstructions During Pile Driving / Caisson Installation

Glacial till often contains cobbles and/or boulders that may be encountered during installation of steel piles or drilled caissons. Accordingly, pile driving shoes as per OPSD 3000.100 have been recommended for tip protection during driving in till. In addition, caisson drilling rigs must be capable of dislodging and removing cobbles and boulders. An NSSP will be required in the Contract Documents during detail design to inform the contractor of the possible presence of cobbles and boulders.

6.7.2 Excavation and Backfill

Preliminary comments regarding open-cut excavations for foundation construction are provided on a site-specific basis on the Preliminary Foundation Design Report sheets. The soil type classification as per the Occupational Health and Safety Act (OHSA), as well as the recommended maximum side slope inclination for temporary excavations, are provided for the conditions anticipated within the foundation excavations. All backfill is to be placed and compacted in accordance with SP 105S10.

6.7.3 Groundwater and Surface Water Control

The anticipated groundwater conditions and requirements for groundwater and surface water control measures at each site are presented on the Preliminary Foundation Design Report sheets. The comments regarding groundwater control are based on the groundwater levels observed in the boreholes and the anticipated excavation depth required to construct the recommended foundation type.

At locations where near surface cohesionless soils and a high water table are present, prior dewatering will be required to accommodate foundation construction 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.

Caissons constructed with temporary or permanent liners and founded in cohesionless 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 such a site, it is recommended that a constant head of water be maintained inside the caisson liners to counterbalance the natural groundwater pressures. Concrete placement by tremie may be considered. Caissons should not be founded in cohesionless soils with artesian water conditions.

For other deep foundations installed where artesian conditions are expected, 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. Sites with artesian conditions should be extensively investigated and foundation installation procedures 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 Protection Systems

Excavation support systems may be required for temporary roadway protection during foundation construction. The temporary excavation support system should be designed and constructed in accordance with OPSS 539. 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.

6.7.5 Construction Access

Environmentally sensitive creek valley crossings have been identified during the environmental assessment of the project. Potential environmental impacts will need to be minimized during construction access in the sensitive floodplains. 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

All culvert sites with spans exceeding 6 m were classified as medium level effort sites. Where PTE was obtained, field investigations were conducted and FIDR sheets have been prepared.

Where PTE was not obtained, no site specific borehole investigations have been carried out. Copies of the Anticipated Foundation Conditions (AFC) sheets prepared during the Desktop Study were included in Thurber's report.

The FIDR sheet for culvert M-51 is included at the end of this report. The preliminary project-wide recommendations presented in Section 6.0 are generally applicable to the culvert sites.

8.0 HIGH FILLS

8.1 General

This section of the report provides geotechnical recommendations for preliminary design of high fill sections where the height exceeds 4.5 m. Based on the roadway profiles available at the time of analysis (February 2009), deep cuts have been identified at three locations and high fills were identified at four locations. The location and maximum height of high fill HF-C4 included in this addendum report are summarized in Table 2, Section 4.2. The maximum fill height is about 5.5 m. No deep cuts were identified for this addendum report.

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

The anticipated subsurface conditions at the high fill location and preliminary recommendations for design are summarized on the "Preliminary Foundation Investigation Report – High Fills" sheet 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.

Further investigation will be required during detail design to confirm the subsurface conditions that were assumed throughout the cut/fill sections and confirm/reassess the preliminary design recommendations.

8.2 High Fills

8.2.1 Embankment Slope Stability

Preliminary assessment of the stability of the fill embankment slopes was carried out 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.

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.

8.2.2 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 site-specific subsoil conditions deduced from the 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 Design Report sheet for the particular section. The settlement tolerance for embankments may range from 25 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.

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.

8.2.3 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 should be determined during detail design when additional subsurface information is available. Particular attention will be required in low floodplain 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 subexcavation or additional compaction prior to fill placement.

Embankment fill should be placed and compacted in accordance with SP 206S03 and SP 105S10. 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.

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 are 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 underlain by geosynthetics in working areas.

Potential environmental impacts will need to be minimized during construction access into sensitive floodplain or wetland 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.

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.

Peto MacCallum Ltd.

Grigory O. Degil, PhD, P.Eng.
Senior Foundation Engineer

Carlos M.P. Nascimento, P.Eng.
Manager, MTO Foundation Services

Brian R. Gray, MEng, P.Eng.
MTO Designated Principal Contact

GD/CN/BRG:mi

REFERENCES

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.

2. Gartner Lee Limited operating as AECOM, *Foundation Investigation Report for Environmental Assessment (Hydrogeology Specialty), Highway 407 East Extension – Central Section*, prepared for Ministry of Transportation Ontario, October 2008.

3. Ministry of Transportation Ontario, *Foundation Investigation and Design Report, Preliminary Design Study for Proposed Hwy 407 from Hwy 48 to Whitby/Oshawa Boundary*, Geocres No. 30M14-227, August 1994.

4. Ministry of Transportation Ontario, *Foundation Investigation and Design Report, Feasibility Study for Highway 407 from Whitby/Oshawa Boundary to Hwy 35/115*, Geocres No. 30M15-85, October 1994.

5. Thurber Engineering Ltd., *Foundation Desktop Study, Highway 407 East Extension – Central Section*, W.O. 07-20016, prepared for Ministry of Transportation Ontario, October 2008.

6. Thurber Engineering Ltd. *Preliminary Foundation Investigation and Design Report – Central Section*, W.O. 07-20016, Geocres No. 30M15-103, prepared for the Ministry of Transportation, April 2010.

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:

RQD (%)	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

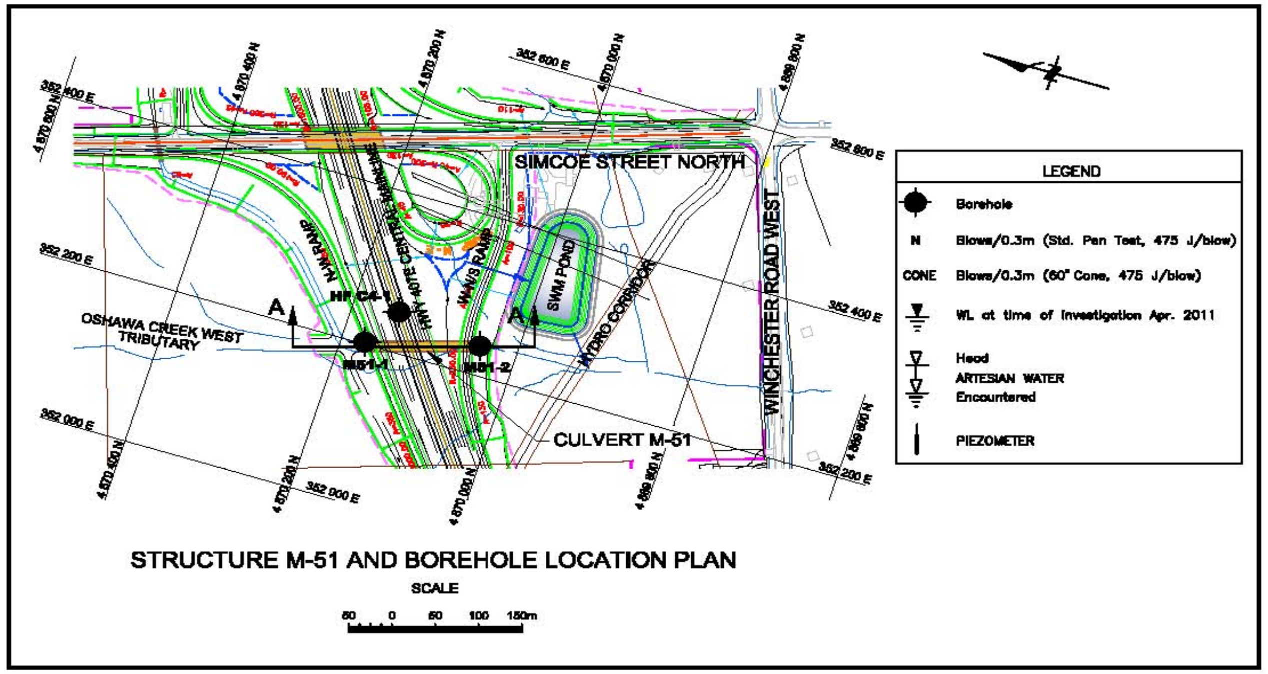
n	1, %	POROSITY
w	1, %	WATER CONTENT
S_r	%	DEGREE OF SATURATION
w_L	%	LIQUID LIMIT
w_P	%	PLASTIC LIMIT
w_S	%	SHRINKAGE LIMIT
I_P	%	PLASTICITY INDEX $= w_L - w_P$
I_L	1	LIQUIDITY INDEX $= \frac{w - w_P}{I_P}$
I_C	1	CONSISTENCY INDEX $= \frac{w_L - w}{I_P}$
DTPL		DRIER THAN PLASTIC LIMIT
APL		ABOUT PLASTIC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

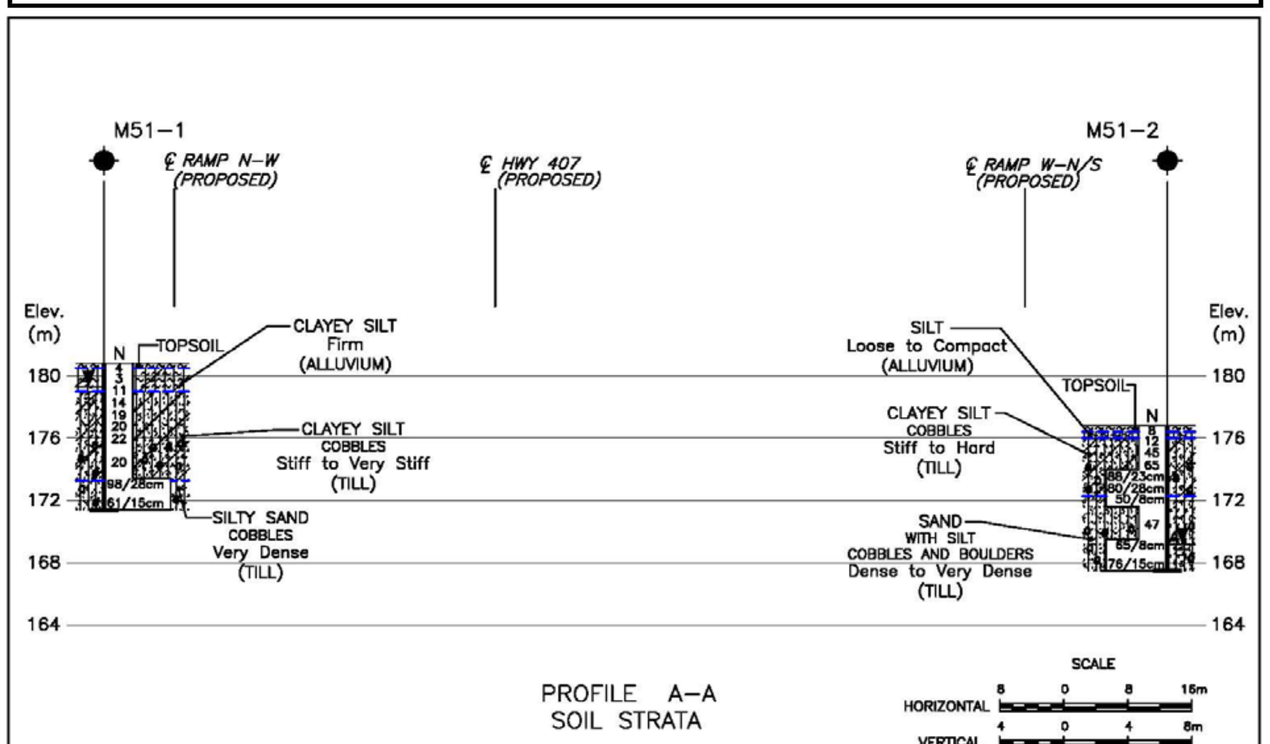
e_{max}	1, %	VOID RATIO IN LOOSEST STATE
e_{min}	1, %	VOID RATIO IN DENSEST STATE
I_D	1	DENSITY INDEX $= \frac{e_{max} - e}{e_{max} - e_{min}}$
D	mm	GRAIN DIAMETER
D_n	mm	n PERCENT - DIAMETER
C_u	1	UNIFORMITY COEFFICIENT
h	m	HYDRAULIC HEAD OR POTENTIAL
q	m^3/s	RATE OF DISCHARGE
v	m/s	DISCHARGE VELOCITY
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 – CENTRAL SECTION (WEST PART)
W.O. 07 – 20016

Structure Description: Culvert at Highway 407 over an Oshawa Creek West Tributary
Location No: M-51 (CM-13)
Highway 407 Proposed Grade: ~186 m
Existing Ground Elevation: 176.8 m – 180.8 m
Site Ranking: Medium
Station: 11+508





FOUNDATION INVESTIGATIONS

Site Description:

The site of the proposed culvert M-51 at Highway 407 and associated N-W and W-N/S ramps over an Oshawa Creek West tributary is located less than 300 m west of Simcoe Street North and some 400 m north of Winchester Road West in the City of Oshawa, Ontario. There is a Hydro One transmission corridor to the south of the culvert location. The relief in the general area is low, rolling, imperfectly drained.

Borehole Information:

Borehole No	Borehole Location	MTM NAD 83 – Northing	MTM NAD 83 – Easting	Borehole Elevation (m)	Borehole Depth (m)
M51-1	North End (Inlet)	4 870 172.9	352 202.7	180.8	9.4
M51-2	South End (Outlet)	4 870 044.3	352 236.1	176.8	9.3

Subsurface Conditions:

- **Topsoil:** surficial topsoil was present in both boreholes. The topsoil had a thickness of 300 mm in borehole M51-1, 400 mm in borehole M51-2 and was penetrated at Elevations 180.5 and 176.4 m respectively.
- **Alluvium:** directly beneath the topsoil in boreholes M51-1 and M51-2 at respective depths of 0.3 and 0.4 m (Elev. 180.5 and 176.4 m) was alluvium. The clayey silt alluvium in the former borehole was 1.5 m thick, firm in consistency and extended to a depth of 1.8 m (Elev. 179.0 m). The silt alluvium in the latter borehole was 400 mm in thickness, loose to compact in relative density and penetrated at 0.8 m depth (Elev. 176.0 m). The moisture content of the alluvium decreased with depth from about 28 to 13 percent.
- **Clayey Silt Till:** overlain by the clayey silt alluvium at a depth of 1.8 m (Elev. 179.0 m) in borehole M51-1 and by the silt alluvium at 0.8 m depth (Elev. 176.0 m) in borehole M51-2 was a cohesive deposit of clayey silt till. This deposit contained cobbles and was stiff to hard in consistency, its moisture content ranging from 8 to 19 percent. The clayey silt till was 5.7 m thick in borehole M51-1, 3.7 m thick in borehole M51-2 and penetrated at respective depths of 7.5 and 4.5 m (Elev. 173.3 and 172.3 m). The results of Atterberg limits testing and grain size distribution analyses conducted on two samples of the deposit are presented in Figures M51-PC-1 and M51-GS-1 (Appendix B) respectively.
- **Silty Sand Till / Sand Till:** underlying the clayey silt till at depths of 7.5 and 4.5 m (Elev. 173.3 and 172.3 m) in boreholes M51-1 and M51-2 was silty sand till / sand till. Containing cobbles and boulders, this stratum was dense to very dense and had a moisture content of 7 to 10 percent, locally 21 percent. The silty sand / sand till extended to the termination of boreholes M51-1 and M51-2 at respective depths of 9.4 and 9.3 m (Elev. 171.4 and 167.5 m). The results of grain size distribution analysis performed on a sample of the sand till are presented in Figure M51-GS-2 (Appendix B).

Groundwater Conditions:

- **Borehole M51-1:** Groundwater was at depths of 1.5 and 6.7 m (Elev. 179.3 and 174.1 m) during and upon completion of drilling, respectively. The piezometric water level was 0.8 and 0.7 m above ground surface (Elev. 181.6 and 181.5 m) on April 28 and May 4, 2011, respectively.
- **Borehole M51-2:** Groundwater was at a depth of 7.6 m (Elev. 169.2 m) upon completion of drilling.

Record of Borehole Sheets – Appendix A	Laboratory Test Results – Appendix B	Key Location Plan – Drawing 5

PART B - PRELIMINARY FOUNDATION DESIGN REPORT HWY 407 EAST EXTENSION – CENTRAL SECTION (WEST PART) W.O. 07 – 20016

LOCATION No: M-51 (CM-13)

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-51 prepared by AECOM in March 2009, the culvert will carry Highway 407 and the associated N-W and W-N/S ramps over an Oshawa Creek West tributary. The proposed open footing arch culvert will be 9.0 m wide and 142.5 m long. The stream bed levels of the culvert are specified to be at Elevation 179.8 m at the north end (inlet) and Elevation 176.9 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 stiff to hard clayey silt till	• Lower costs than deep foundations • Conventional construction	• Requires excavation of some 2 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 hard clayey silt till and very dense silty sand till / sand till containing cobbles and boulders • Sub-excavation and dewatering required for pile cap construction; special techniques necessary due to artesian conditions
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 penetrating hard clayey silt till and very dense sandy till with cobbles and boulders • Sub-excavation and dewatering required for caisson cap construction; special techniques necessary due to artesian conditions

A – Spread Footings: Spread footings founded on the stiff to hard clayey silt till at or below Elevation 178.5 m at the north (inlet) end of the culvert and Elevation 175.5 m at the south (outlet) end. 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**
Stiff to Very Stiff Clayey Silt Till	400 kPa	250 kPa
Hard Clayey Silt Till	600 kPa	400 kPa

B – Steel H-Piles: Steel HP 310 x 110 piles driven into the “100-blow” till deposit at or below Elevation 172.0 m are feasible for support of the foundation loads. Pile lengths would be approximately 6 and 3 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 171.0 m. Caissons should be socketed a minimum 2 m into the “100-blow” material. Caissons would be about 7 m at the north end and 4 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 stiff to very stiff clayey silt till.

• APPROACHES

Height: Based on the GA drawing, an embankment height of 6 to 9 m is anticipated. It is noted that sub-excavation of up to 1.8 m of surficial topsoil and silt / clayey silt alluvium would be required.

Stability: An embankment up to 9 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. In addition, construction of a mid-height bench (minimum 2 m wide) will be required for embankments exceeding a height of 8 m to control surficial erosion and improve stability.

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 200 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. Therefore, measures to reduce post-construction settlement should be undertaken. Detailed geotechnical analyses need to be carried out during the detail design.

• CONSTRUCTION CONSIDERATIONS

Excavation: The silt / clayey silt alluvium is classified as a Type 4 soil and the underlying stiff clayey silt till as a 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 in Type 3 soils and at 3H:1V in Type 4 soils.

Groundwater/Surface Water Control: It is anticipated that groundwater within the foundation excavations for footing construction can be adequately controlled by pumping from filtered sumps. Depending on construction season, diversion of surface water from the excavation may need to be implemented as well. In case deep foundations such as piles are employed, basal heave will need to be assessed and more elaborate dewatering measures will 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 if employed should be used to facilitate driving into the hard clayey silt till and very dense silty sand till / 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 location of the arch culvert foundations.

PART A - PRELIMINARY FOUNDATION INVESTIGATION REPORT
HWY 407 EAST EXTENSION – WESTERN SECTION
W.O. 07 – 20016

Structure Description: Highway 407 Bridge over Oshawa Creek East Branch Tributary

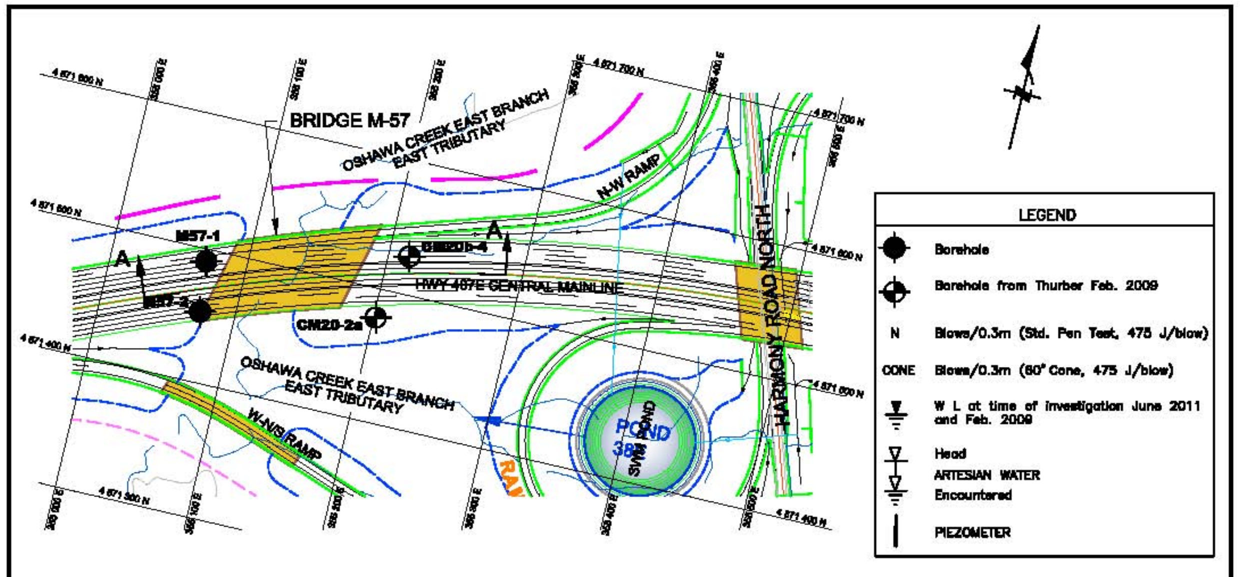
Location No: M-57 (CM-20/20b)

Hwy 407 Proposed Grade: 215.6 m – 218.6 m

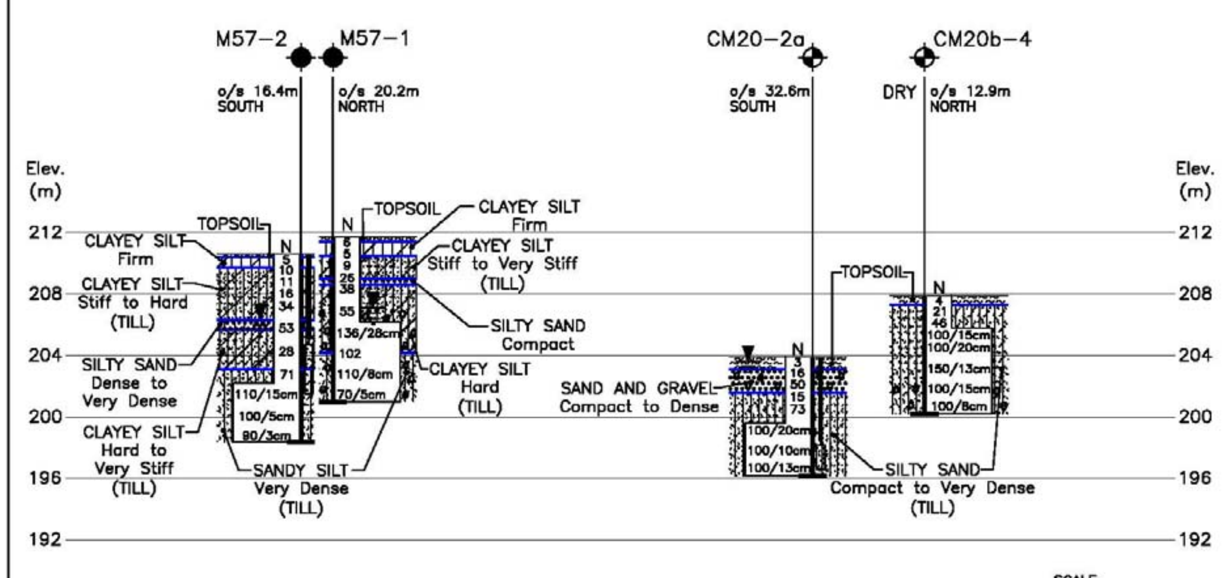
Existing Ground Elevation: 203.9 m – 211.7 m (~200 m at creek level)

Site Ranking: Medium

Station: 20+121



STRUCTURE M-57 AND BOREHOLE LOCATION PLAN



PROFILE A–A
SOIL STRATA

FOUNDATION INVESTIGATIONS

Site Description:

The site of the proposed bridge structure M-57 is located on Highway 407 some 300 m west of Harmony Road North and 750 m north of Winchester Road East in the City of Oshawa, Ontario. The bridge crosses an Oshawa Creek East Branch east tributary flowing southerly. The overall topography of the terrain is sloping down in the southern direction.

Borehole Information:

Borehole No	Borehole Location	MTM NAD 83 – Northing	MTM NAD 83 – Easting	Borehole Elevation (m)	Borehole Depth (m)
M57-1	West Abutment (westbound)	4 871 490.8	355 069.7	211.7	10.7
M57-2	West Abutment (eastbound)	4 871 452.9	355 074.0	210.6	12.2
CM20-2a	East Abutment	4 871 478.2	355 201.5	203.9	7.7
CM20b-4	East Abutment	4 871 528.7	355 215.2	207.9	7.7

Subsurface Conditions:

- **Topsoil:** surficial topsoil was present in all the boreholes. The topsoil was 200 to 800 mm thick and penetrated at Elevation 203.1 to 211.4 m.
- **Clayey Silt:** directly beneath the topsoil at depths of 0.3 and 0.2 m (Elev. 211.4 and 210.4 m) in boreholes M57-1 and M57-2 respectively was a cohesive deposit of clayey silt with organic inclusions. This deposit was firm in consistency and 12 to 17 percent in moisture content. The clayey silt had a thickness of 900 mm in borehole M57-1 and 700 mm in borehole M57-2 and was penetrated at respective depths of 1.2 and 0.9 m (Elev. 210.5 and 209.7 m).
- **Sand and Gravel:** a cohesionless layer of sand and gravel was identified below the topsoil at 0.8 m depth (Elev. 203.1 m) in borehole CM20-2a. Containing cobbles, this layer was compact to dense (SPT-‘N’ values of 16 and 50) with a moisture content of 7 to 10 percent. The sand and gravel was 1.5 m thick and penetrated at a depth of 2.3 m (Elev. 201.6 m).
- **Clayey Silt Till:** overlain by the clayey silt at 1.2 m depth (Elev. 210.5 m) in borehole M57-1 and a depth of 0.9 m (Elev. 209.7 m) in borehole M57-2 was a cohesive deposit of clayey silt till. This deposit was interlayered with 400 to 600 mm of compact to very dense silty sand encountered at 2.7 m depth (Elev. 209.0 m) in the former borehole and a depth of 4.3 m (Elev. 206.3 m) in the latter. The clayey silt till was stiff to hard in consistency and contained cobbles, its moisture content ranging from 9 to 15 percent. The deposit had a thickness of 6.3 m in borehole M57-1 and 6.6 m in borehole M57-2 and was penetrated at 7.5 m depth (Elev. 204.2 and 203.1 m). The results of grain size distribution analyses and Atterberg limits testing conducted on two samples of the clayey silt till are presented in respective Figures M57-PC-1 and M57-GS-1 (Appendix B).
- **Cohesionless Till:** underlying the topsoil, sand and gravel or clayey silt till at depths of 0.6 to 7.5 m (Elev. 203.1 to 207.3 m) in all the boreholes was sandy silt till / silty sand till. This stratum was compact to very dense (SPT-‘N’ values of 15 to over 100) and had a moisture content of 5 to 17 percent. The sandy silt till / silty sand till extended to the termination of drilling at depths of 7.7 to 12.2 m (Elev. 196.2 to 201.0 m). It is worth noting that the stratum contained cobbles and boulders in boreholes M57-1 and CM20b-4. The results of grain size distribution analyses performed on one sample of the sandy silt till and four samples of the silty sand till are presented in Figures M57-GS-2 and CM20/20b-B3 (Appendix B) respectively.

Groundwater Conditions:

- **Borehole M57-1:** Water was detected at a depth of 4.6 m (Elev. 207.1 m) in the process of augering. Groundwater was at 7.3 m depth (Elev. 204.4 m) upon completion of drilling.
- **Borehole M57-2:** Water was detected at a depth of 4.3 m (Elev. 206.3 m) in the process of augering. Groundwater was at 6.7 m depth (Elev. 203.9 m) upon completion of drilling. The water level in piezometer was at depths of 4.0 and 4.2 m (Elev. 206.6 and 206.4 m) on June 10 and 17, 2011, respectively.
- **Borehole CM20-2a:** The water level was measured in piezometer at a depth of 0.4 m (Elev. 203.5 m) on February 12, 2009.

Record of Borehole Sheets – Appendix A | **Laboratory Test Results – Appendix B** | **Key Location Plan – Drawing 6**

PART B - PRELIMINARY FOUNDATION DESIGN REPORT
HWY 407 EAST EXTENSION – WESTERN SECTION
W.O. 07 – 20016

LOCATION No: M-57 (CM-20/20b)

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-57 provided by AECOM in March 2009, the bridge structure M-57 will carry Highway 407 over an Oshawa Creek East Branch east tributary. The proposed bridge consists of twin three (3) span structures with a total length of 107.5 m and with approach embankments of 5.0 m high at the west abutment and up to 15.0 m high at the east abutment. The foundation options considered are listed below with advantages and disadvantages associated with each option.

Foundation Option	*Advantages*	*Disadvantages*
Spread footings founded on hard clayey silt till or dense silty sand till for abutments and on very dense cohesionless till for piers with high groundwater table within floodplain Spread footings perched on compacted Granular 'A' pads at both abutments	• Conventional construction • Lower cost than deep foundations • Minimize excavation requirements in case of using Granular 'A' pads	• Variability of surficial soils; requires sub-excavation some 3 m depth to reach competent founding soils • Unwatering and protection (temporary shoring) systems will likely be required for footing construction • High east abutment wall (>10 m) • Scour protection is required for footings in floodplain
Steel H-Piles driven into "100-blow" sandy silt till / silty sand till	• Higher bearing resistance • Permits use of integral abutments • Not affected by surficial soil variability	• Higher cost than spread footings • Sub-excavation of topsoil, organics and clayey silt at shallow depths to construct pile caps • Unwatering and protection (temporary shoring) systems may be required for pile cap construction; special techniques may be required if artesian conditions are encountered
Caissons founded within "100-blow sandy silt till / silty sand till	• Higher bearing resistances than steel H-Piles • Not so affected by surficial soil variability	• Higher cost than spread footings • Does not allow integral abutment design • Potential installation problems including side sloughing, liner sealing and base boiling associated with cohesionless soils below the grounwater table • Need to dislodge and handle cobbles and boulders • Sub-excavation of surficial soils for cap construction • Unwatering and protection (temporary shoring) systems may be required

A - Spread Footings: Spread footings founded on the hard clayey silt till at or below Elevations 207.5 m (EBL) and 208.5 m (WBL) at the west abutment or on the dense silty sand till at or below Elevations 201.0 m (EBL) and 206.5 m (WBL) at the east abutment may be used to support the foundation loads. Alternatively, footings for perched abutments may be founded on compacted Granular 'A' cores in accordance with current MTO practices. Since the creek bed is at approximate Elevation 200, spread footings for piers would be founded on the very dense sandy silt till / silty sand till at or below Elevation 198.5 m.

Founding Stratum	*Geotechnical Resistance*	
	Factored ULS	**SLS**
Hard Clayey Silt Till or Dense Silty Sand Till	600 kPa	400 kPa
Very Dense Sandy Silt Till / Silty Sand Till	750 kPa	500 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 silt till / silty sand till at or below Elevations 200.0 m (EBL) and 203.5 m (WBL) at the west abutment or Elevations 197.5 m (EBL) and 204.0 m (WBL) at the east abutment may be used to provide foundation support. Pile lengths would be approximately 7.0 to 10.5 m for the west abutment and 9.0 to 15.5 m for 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 may be founded on caissons installed within the "100-blow" cohesionless till at or below Elevations 197.5 m (EBL) and 201.5 m (WBL) at the west abutment or Elevations 195.5 m (EBL) and 201.5 m (WBL) at the east abutment. Caissons would be about 9.0 to 13.0 m long for the west abutment and 11.5 to 17.5 m long for the east abutment. The preliminary design geotechnical resistances for caissons extending 4 m into the "100-blow" till are as follows:

Location	*Caisson Diameter*	*Geotechnical Axial Resistance*	
		Factored ULS	**SLS**
Abutments	1.2 m	4,500 kN	3,600 kN
	1.5 m	6,500 kN	5,200 kN

Recommended Foundation Alternative: Steel H-Piles. Additional drilling is required in the floodplain to confirm suitable foundation design, especially at the pier locations.

• ABUTMENT TYPE

The soil conditions at this site are suitable for conventional, integral or semi-integral abutment design.

• APPROACHES

Embankment Height: Based on the GA drawing, 5 m of fill will be required to construct the highway west approach, while up to 15 m are anticipated at the east approach. It is noted that sub-excavation of up to 1.2 m of surficial topsoil and clayey silt with organics would be required.

Stability: Approach embankments 5 to 15 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 a bench (minimum 2 m wide) for slopes exceeding 8 m in height will have an adequate factor of safety against deep-seated instability. Global stability for the east approach fill should be confirmed during detail deign. 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, foundation settlement will occur as fill is placed and should be completed by the end of construction. It is estimated that post construction foundation settlement and fill compression combined will be in the order of 100 mm and be virtually complete over a period of six to nine months.

• CONSTRUCTION CONSIDERATIONS

Excavation: Excavations will be required for footing or pile cap construction. No excavation should be carried out in the floodplain without prior unwatering. Temporarily unsupported side slopes should not be steeper than 1H:1V where groundwater control measures are implemented as outlined below. In accordance with the OHSA, sands and silts below the grounwater level are classified as Type 4 soils. For saturated granular soils below the groundwater table, temporary shoring may be required.

Groundwater / Surface Water Control: The piezometric water level is 0.4 to 4.2 m below the existing ground surface. Prior to excavation in the floodplain, groundwater control systems such as well points and/or interlocking sheetpiled cofferdams would be required. Diversion of surface runoff from the excavation and pumping from carefully constructed, filtered sumps may be used to supplement the above systems. The required groundwater control systems should be further assessed during detail design. Artesian groundwater conditions may be encountered when advancing deep foundations such as piles through the cohesionless 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: Protection systems would be required at excavation locations where stable slopes cannot be constructed due to space limitations and where vertically sided excavations are used for footing or pile cap construction. Protection systems will be required for foundation construction in the floodplain. One possible system is an interlocking sheetpiled cofferdam which can also be used for groundwater cutoff as outlined above. The feasibility of installing protection systems should be assessed once further subsurface investigation is carried out during detail design.

Obstructions During Pile Driving: During pile installation through glacially derived soils at this site, there is a medium probability of encountering cobbles and boulders. Driving shoes should be fitted to the pile tips for reinforcement and enhancing seating of the piles. Caisson drilling equipment if used must be capable of penetrating obstructions such as cobbles and boulders that are expected within the till deposits.

Floodplain Access: Potential environmental impacts will need to be minimized during construction access into the sensitive floodplain. Specific access preparation procedures including the use of gravel roadways underlain by the geosynthetics should be considered.

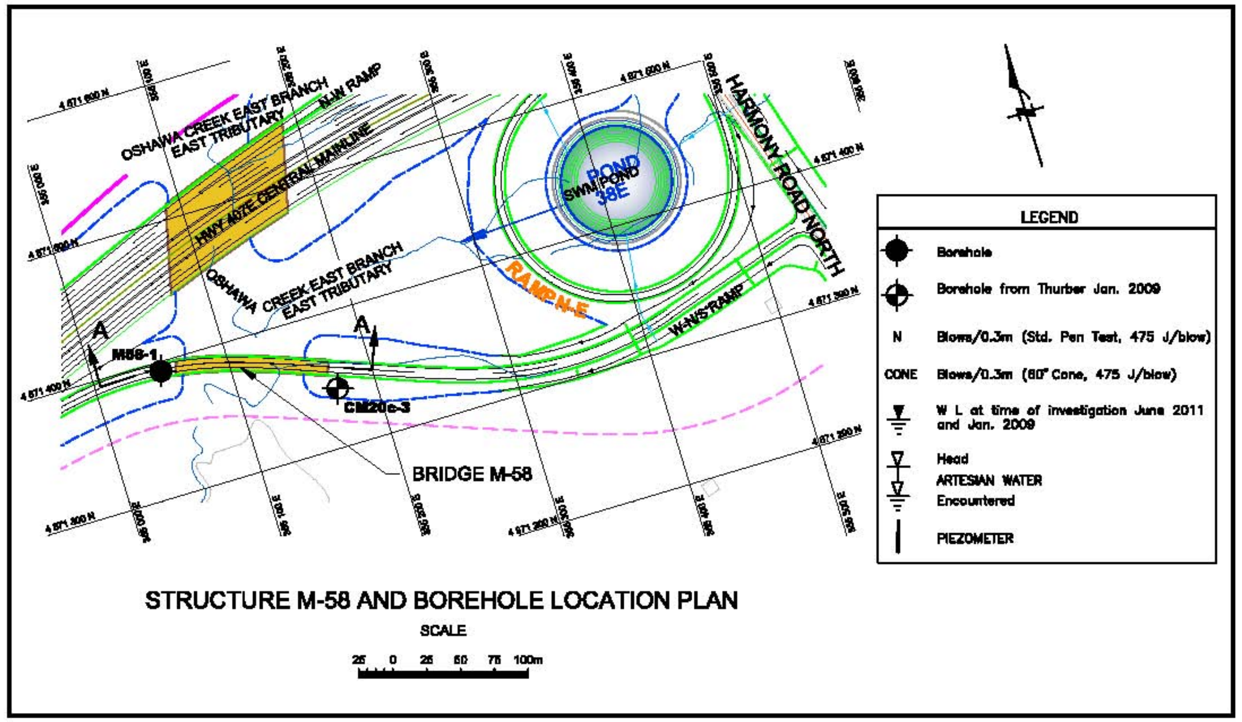
• RECOMMENDATIONS FOR ADDITIONAL WORK

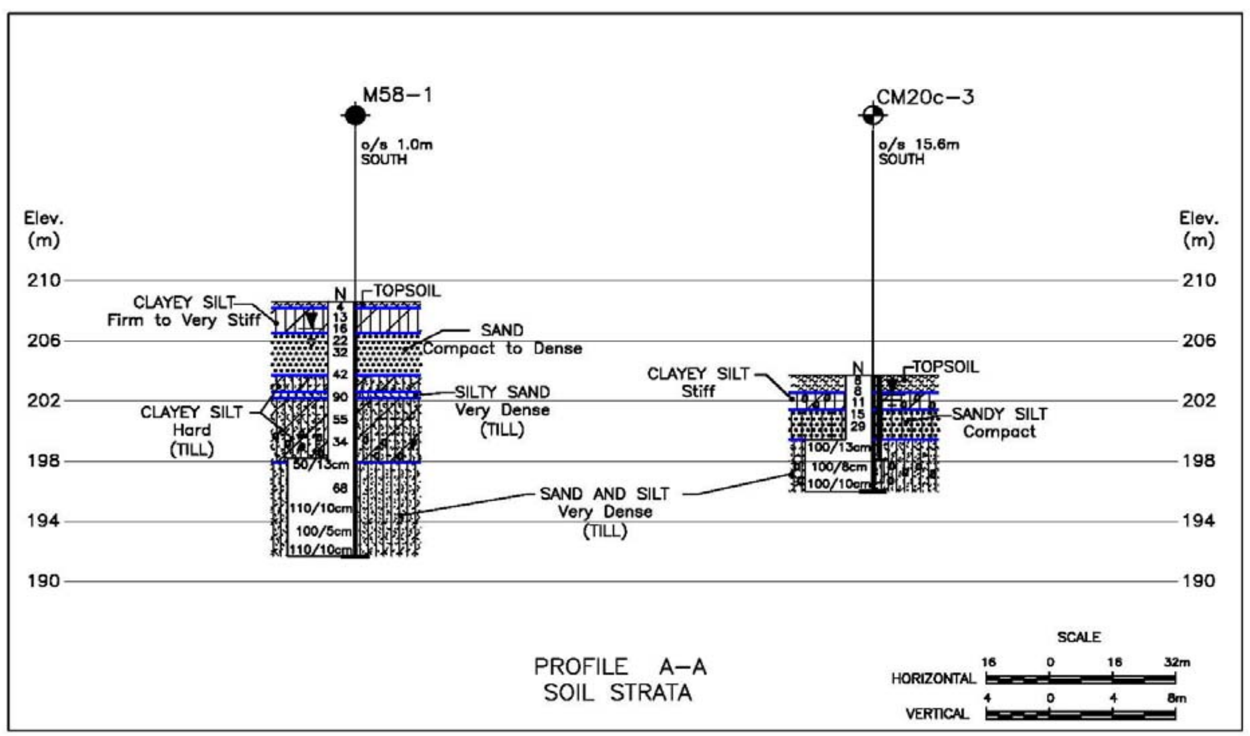
Further subsurface investigation should be carried out during detail design to confirm the subsoil and groundwater conditions at the location of the bridge foundation elements. As a minimum, this is likely to require additional boreholes at the actual abutment and pier locations and along the approaches. The feasibility and cost effectiveness of alternative unwatering systems would need to be investigated.

PART A - PRELIMINARY FOUNDATION INVESTIGATION REPORT
HWY 407 EAST EXTENSION – WESTERN SECTION
W.O. 07 – 20016

Structure Description: Highway 407 W-N/S Ramp over Oshawa Creek East Branch Tributary **W-N/S Ramp Proposed Grade:** 212.5 m – 212.8 m **Site Ranking:** Medium

Location No: M-58 (CM-20c) **Existing Ground Elevation:** 203.7 m – 208.6 m (~198 m at creek level) **Station:** 10+201





FOUNDATION INVESTIGATIONS

Site Description:

The site of the proposed bridge structure M-58 is located on the Highway 407 W-N/S Ramp connecting to Harmony Road North some 600 m north of Winchester Road East in the City of Oshawa, Ontario. The bridge crosses an Oshawa Creek East Branch east tributary flowing southerly. The overall topography of the terrain is sloping down in the southern direction.

Borehole Information:

Borehole No	Borehole Location	MTM NAD 83 – Northing	MTM NAD 83 – Easting	Borehole Elevation (m)	Borehole Depth (m)
M58-1	West Abutment	4 871 393.3	355 050.3	208.6	16.9
CM20c-3	East Abutment	4 871 343.4	355 171.6	203.7	7.7

Subsurface Conditions:

- **Topsoil:** surficial topsoil was present in both boreholes. The topsoil had a thickness of 0.4 m in borehole M58-1 and 1.2 m in borehole CM20c-3 and was penetrated at respective Elevations 208.2 and 202.5 m.
- **Clayey Silt:** directly beneath the topsoil at depths of 0.4 and 1.2 m (Elev. 208.2 and 202.5 m) in both boreholes was a cohesive deposit of clayey silt. This deposit was firm to very stiff in consistency and 11 to 17 percent in moisture content. The clayey silt was 1.7 m thick and penetrated at 2.1 m depth (Elev. 206.5 m) in borehole M58-1. Containing cobbles, the deposit was 1.1 m in thickness and penetrated at a depth of 2.3 m (Elev. 201.4 m) in borehole CM20c-3.
- **Sand / Sandy Silt:** overlain by the clayey silt in boreholes M58-1 and CM20c-3 at respective depths of 2.1 and 2.3 m (Elev. 206.5 and 201.4 m) was sand / sandy silt. This unit was compact to dense (SPT-'N' values of 15 to 32) and had a moisture content of 14 to 19 percent. The sand / sandy silt had a thickness of 2.8 m in borehole M58-1 and 2.0 m in borehole CM20c-3 and was penetrated at respective depths of 4.9 and 4.3 m (Elev. 203.7 and 199.4 m). The results of grain size distribution analysis performed on a sample of the sand are presented in Figure M58-GS-1 (Appendix B).
- **Clayey Silt Till:** a cohesive deposit of clayey silt till was revealed below the sand at 4.9 m depth (Elev. 203.7 m) in borehole M58-1. This deposit was hard in consistency and 12 to 20 percent in moisture content. With a 400 mm thick layer of very dense silty sand till encountered at a depth of 6.0 m (Elev. 202.6 m) and cobbles underneath, the clayey silt till had a thickness of 5.8 m and was penetrated at 10.7 m depth (Elev. 197.9 m). The results of grain size distribution analysis and Atterberg limits testing conducted on a sample of the deposit are presented in Figures M58-PC-1 and M58-GS-2 (Appendix B) respectively.
- **Sand and Silt Till:** underlying the sandy silt at a depth of 4.3 m (Elev. 199.4 m) in borehole CM20c-3 or the clayey silt till at 10.7 m depth (Elev. 197.9 m) in borehole M58-1 was sand and silt till. This stratum was very dense (SPT-'N' values of 68 to over 100) and had a moisture content of 6 to 14 percent. The sand and silt till extended to the termination of drilling in boreholes CM20c-3 and M58-1 at respective depths of 7.7 and 16.9 m (Elev. 196.0 and 191.7 m). It is worth noting that the stratum contained cobbles in borehole CM20c-3. The results of grain size distribution analyses performed on 2 samples of the sand and silt till are presented in Figures M58-GS-3 and CM20c-B1 (Appendix B).

Groundwater Conditions:

- **Borehole M58-1:** Water was detected at 2.1 m depth (Elev. 206.5 m) in the process of augering. Groundwater was at a depth of 1.8 m (Elev. 206.8 m) upon completion of drilling.
- **Borehole CM20c-3:** Groundwater was measured in piezometer at 0.4 m depth (Elev. 203.3 m) on February 12, 2009, and at 1.3 m depth (Elev. 202.4 m) on May 4 and June 6, 2009.

LOCATION No: M-58 (CM-20c)

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-58 provided by AECOM in September 2009, the bridge structure M-58 will carry the Highway 407 W-N/S ramp connecting to Harmony Road North over an Oshawa Creek East Branch east tributary. The proposed bridge is a three (3) span structure with a total length of 112.0 m and with approach embankments of 4.5 m high at the west abutment and up to 11.0 m high at the east abutment. The foundation options considered are listed below with advantages and disadvantages associated with each option.

Foundation Option	*Advantages*	*Disadvantages*
Spread footings founded on compact to dense sand / sandy silt for abutments and on very dense sand and silt till for piers with high groundwater table within floodplain Spread footings perched on compacted Granular 'A' pads at both abutments	• Conventional construction • Lower cost than deep foundations • Minimize excavation requirements in case of using Granular 'A' pads	• Variability of surficial soils; requires sub-excavation up to 3.2 m depth to reach competent founding soils • Unwatering and protection (temporary shoring) systems will likely be required for footing construction • Does not allow integral abutment design • Scour protection is required for footings in floodplain
Steel H-Piles driven into "100-blow" sand and silt till	• Higher bearing resistance • Permits use of integral abutments • Not affected by surficial soil variability	• Higher cost than spread footings • Sub-excavation of topsoil, organics and clayey silt at shallow depths to construct pile caps • Unwatering and protection (temporary shoring) systems may be required for pile cap construction; special techniques may be required if artesian conditions are encountered
Caissons founded within "100-blow" sand and silt till	• Higher bearing resistances than steel H-Piles • Not so affected by surficial soil variability	• Higher cost than spread footings • Does not allow integral abutment design • Potential installation problems including side sloughing, liner sealing and base boiling associated with cohesionless soils below the grounwater table • Need to dislodge and handle cobbles and possible boulders • Sub-excavation of surficial soils for cap construction • Unwatering and protection (temporary shoring) systems may be required

A - Spread Footings: Spread footings founded on the compact to dense sand / sandy silt at or below Elevation 205.5 m at the west abutment or Elevation 200.5 m at the east abutment may be used to support the foundation loads. Alternatively, footings for perched abutments may be founded on compacted Granular 'A' cores in accordance with current MTO practices. Since the creek level is at approximate Elevation 198, spread footings for piers would be founded on the very dense sand and silt till at or below Elevation 196.5 m. The preliminary design geotechnical resistances are as follows:

Founding Stratum	*Geotechnical Resistance*	
	Factored ULS	**SLS**
Compact to Dense Sand / Sandy Silt	450 kPa	300 kPa
Very Dense Sand and Silt Till	600 kPa	400 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" sand and silt till at or below Elevation 193.5 m at the west abutment and Elevation 197.5 m at the east abutment may be used to provide foundation support. Pile lengths would be approximately 15.5 and 8.5 m for the west and east abutments, respectively. The preliminary design geotechnical resistances are as follows:

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

C – Caissons: Abutments may be founded on caissons installed within the "100-blow" sand and silt till at or below Elevation 191.0 m (west abutment) or Elevation 195.0 m (east abutment). Caissons would be about 18 m long for the west abutment and 11 m long for the east abutment. The preliminary design geotechnical resistances for caissons extending 4 m into the "100-blow" till are as follows:

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	5,200 kN

Recommended Foundation Alternative: Steel H-Piles. Additional drilling is required in the floodplain to confirm suitable foundation design, especially at the pier locations.

• ABUTMENT TYPE

The soil conditions at this site are suitable for conventional, integral or semi-integral abutment design.

• APPROACHES

Embankment Height: Based on the GA drawing, 4.5 m of fill will be required to construct the highway ramp west approach, while up to 11 m are anticipated at the east approach. It is noted that sub-excavation of up to 1.2 m of surficial topsoil would be required.

Stability: Approach embankments 4.5 to 11.0 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 a bench (minimum 2 m wide) for slopes exceeding 8 m in height will have an adequate factor of safety against deep-seated instability. Global stability for the east approach fill should be confirmed during detail deign. 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, foundation settlement will occur as fill is placed and should be completed by the end of construction. It is estimated that post construction foundation settlement and fill compression combined should not exceed 50 mm at the west approach and 100 mm at the east approach and be virtually complete over a period of six to nine months.

• CONSTRUCTION CONSIDERATIONS

Excavation: Excavations will be required for footing or pile cap construction. No excavation should be carried out in the floodplain without prior unwatering. Temporarily unsupported side slopes should not be steeper than 1H:1V where groundwater control measures are implemented as outlined below. In accordance with the OHSA, sands and silts below the groundwater level are classified as Type 4 soils. For saturated granular soils below the groundwater table, temporary shoring may be required.

Groundwater / Surface Water Control: The piezometric water level is 0.4 to 1.8 m below the existing ground surface. Prior to excavation in the floodplain, groundwater control systems such as well points and/or interlocking sheetpiled cofferdams would be required. Diversion of surface runoff from the excavation and pumping from carefully constructed, filtered sumps may be used to supplement the above systems. The required groundwater control systems should be further assessed during detail design. Artesian groundwater conditions may be encountered when advancing deep foundations such as piles through the cohesionless 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.

LOCATION No: M-58 (CM-20c)

Protection Systems: Protection systems would be required at excavation locations where stable slopes cannot be constructed due to space limitations and where vertically sided excavations are used for footing or pile cap construction. Protection systems will be required for foundation construction in the floodplain. One possible system is an interlocking sheetpiled cofferdam which can also be used for groundwater cutoff as outlined above. The feasibility of installing protection systems should be assessed once further subsurface investigation is carried out during detail design.

Obstructions During Pile Driving: During pile installation through glacially derived soils at this site, there is a medium probability of encountering cobbles and boulders. Driving shoes should be fitted to the pile tips for reinforcement and enhancing seating of the piles. Caisson drilling equipment if used must be capable of penetrating obstructions such as cobbles and boulders that are expected within the till deposits.

Floodplain Access: Potential environmental impacts will need to be minimized during construction access into the sensitive floodplain. Specific access preparation procedures including the use of gravel roadways underlain by the geosynthetics should be considered.

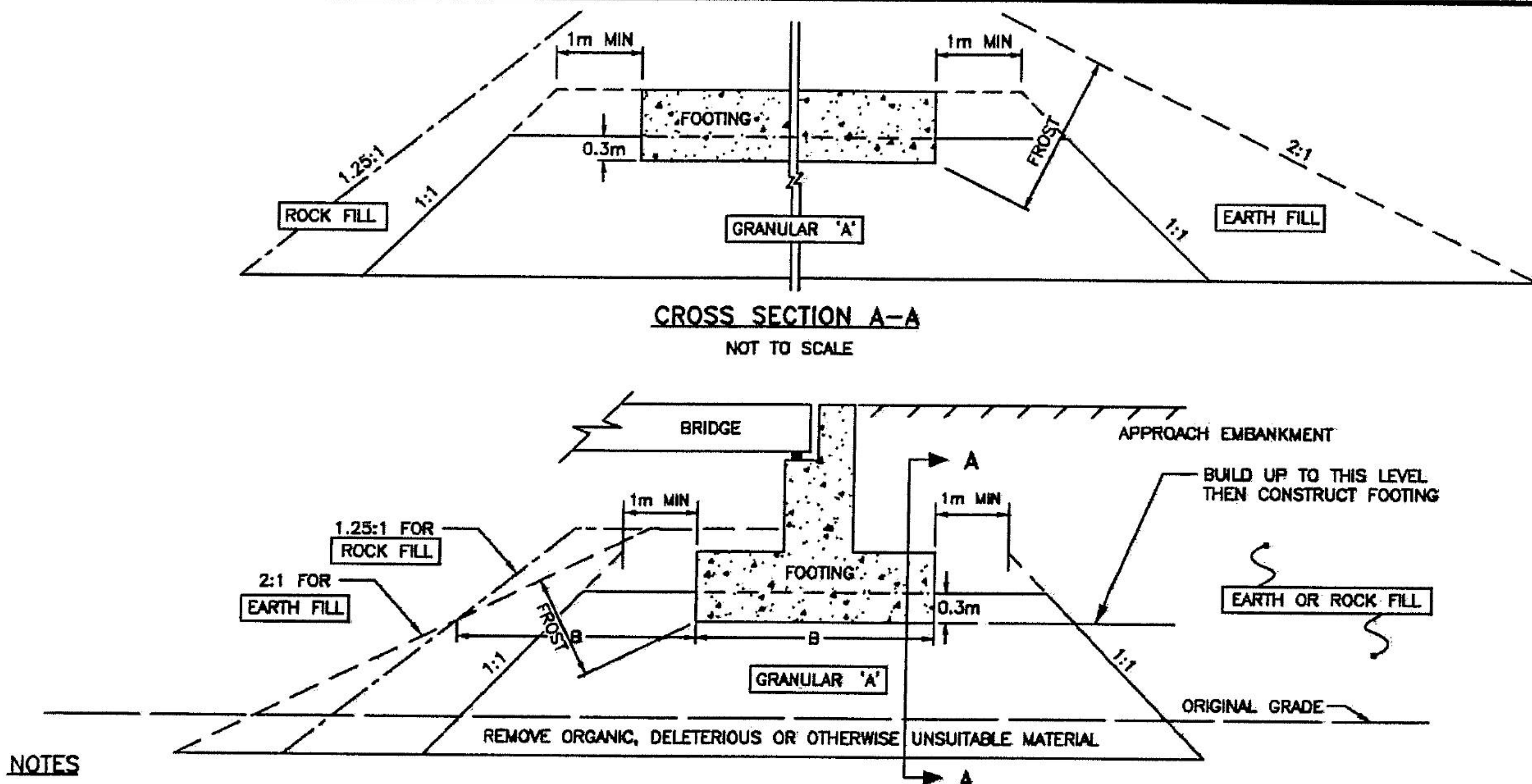
• RECOMMENDATIONS FOR ADDITIONAL WORK

Further subsurface investigation should be carried out during detail design to confirm the subsoil and groundwater conditions at the location of the bridge foundation elements. As a minimum, this is likely to require additional boreholes at the actual abutment and pier locations and along the approaches. The feasibility and cost effectiveness of alternative unwatering systems would need to be investigated.

PRELIMINARY FOUNDATION INVESTIGATION REPORT – HIGH FILLS

PRELIMINARY FOUNDATION INVESTIGATION REPORT
HIGH FILLS
HWY 407 EAST EXTENSION - CENTRAL SECTION (WEST PART)
W.O.07 - 20016

High Fill No.	Station (From - To)	Proposed Highway Grade (m)	Maximum Fill Height (m)	Reference Data	Subsurface Conditions	Preliminary Recommendations
Hwy 407	Central Mainline					
HF-C4	11+366 to 11+616	185.0 to 186.3	5.5	HFC4-1, M51-1, M51-2, Hydrogeology Report	**Stratigraphy:** Topsoil and/or silt / clayey silt alluvium (up to 1.8 m) overlying firm to hard clayey silt till underlain at depths of 4.5 to 7.5 m (Elev. 172.3 to 174.5 m) by sand / silty sand till which extended to the borehole termination depths of 6.7 to 9.4 m (Elev. 167.5 to 174.0 m). **Groundwater:** Estimated near 2 to 3 m depth (Elev. 177 m). Borehole HFC4-1 – depths of 0.0 and 4.7 m (Elev. 180.7 and 176.0 m) during and upon completion of drilling, respectively. Borehole M51-1 – depths of 1.5 and 6.7 m (Elev. 179.3 and 174.1 m) during and upon completion of drilling, respectively. The piezometric water level was 0.8 and 0.7 m above ground surface (Elev. 181.6 and 181.5 m) on April 28, 2011 and May 4, 2011, respectively. Borehole M51-2 – depth of 7.6 m (Elev. 169.2 m) upon completion of drilling.	**Design Slope Inclination:** Fill embankments up to 5.5 m high may be constructed with slopes no steeper than 2H : 1V. **Stability:** No stability issues are anticipated along this fill section.. **Settlement:** No settlement issues are anticipated. Topsoil and/or alluvium 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.



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

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

W.O. No. 07-20016

SHEET

HIGHWAY 407 EAST EXTENSION
CENTRAL SECTION
PROJECT LOCATION PLAN

Peto MacCallum Ltd.
CONSULTING ENGINEERS

PLAN

SCALE APPROX.
0 1.6 3.2 4.8 Km

NOTES

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

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

REFERENCE

Base provided by AECOM (AUGUST 2009).

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

Geocres No. 30M15-113

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

MINISTRY OF TRANSPORTATION, ONTARIO

PLOT DATE: January 31, 2011
FILENAME: Z:\10TF023\Hwy 407E Drawings\10TF023C Final\10TF023C-001 JULY 13 2011.dwg

MINISTRY OF TRANSPORTATION, ONTARIO

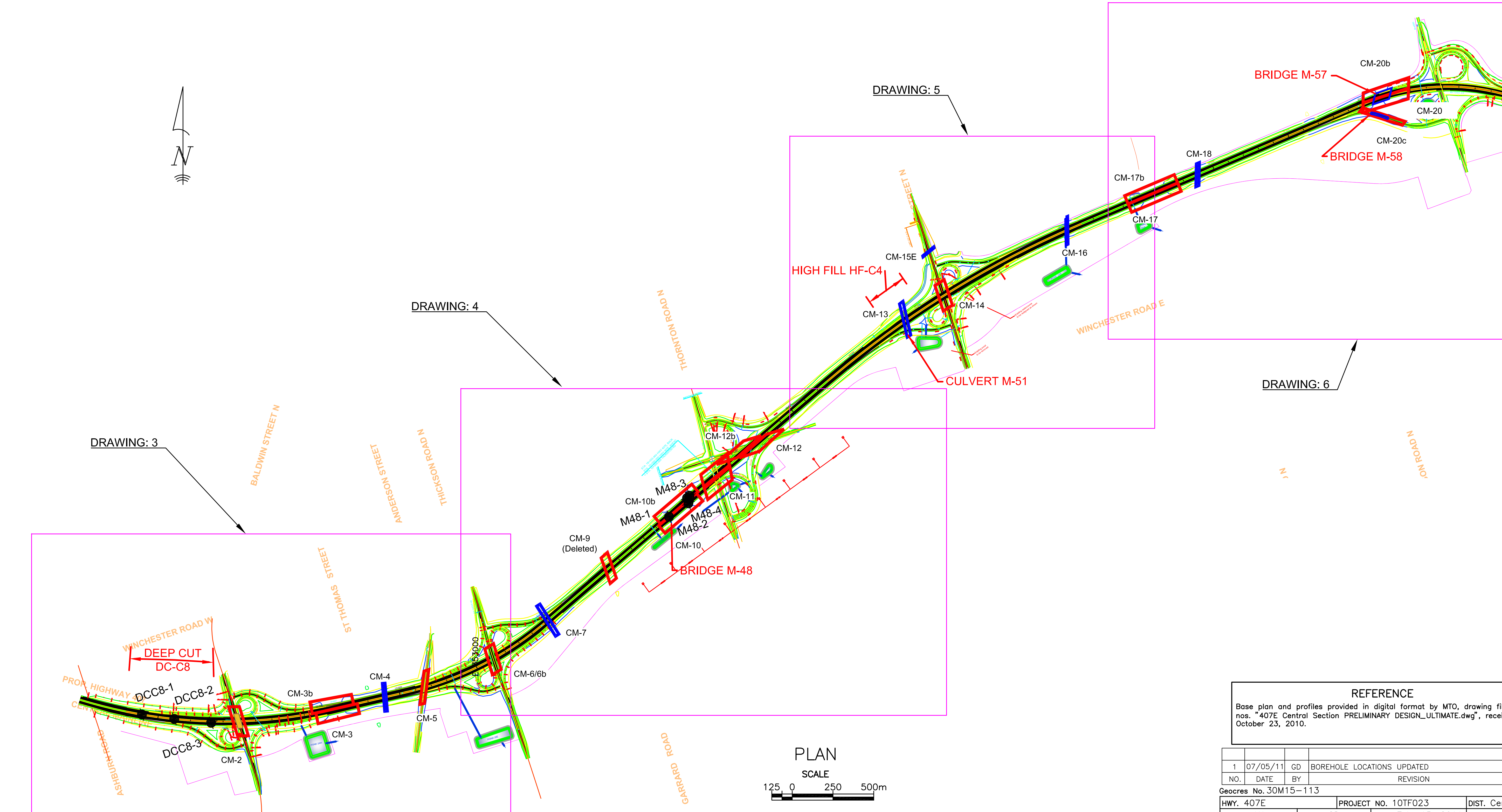
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

W.O. No. 07–20016

SHEET

HIGHWAY 407 EAST EXTENSION
CENTRAL SECTION
KEY LOCATION PLAN

Peto MacCallum Ltd.
CONSULTING ENGINEERS



REFERENCE

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

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

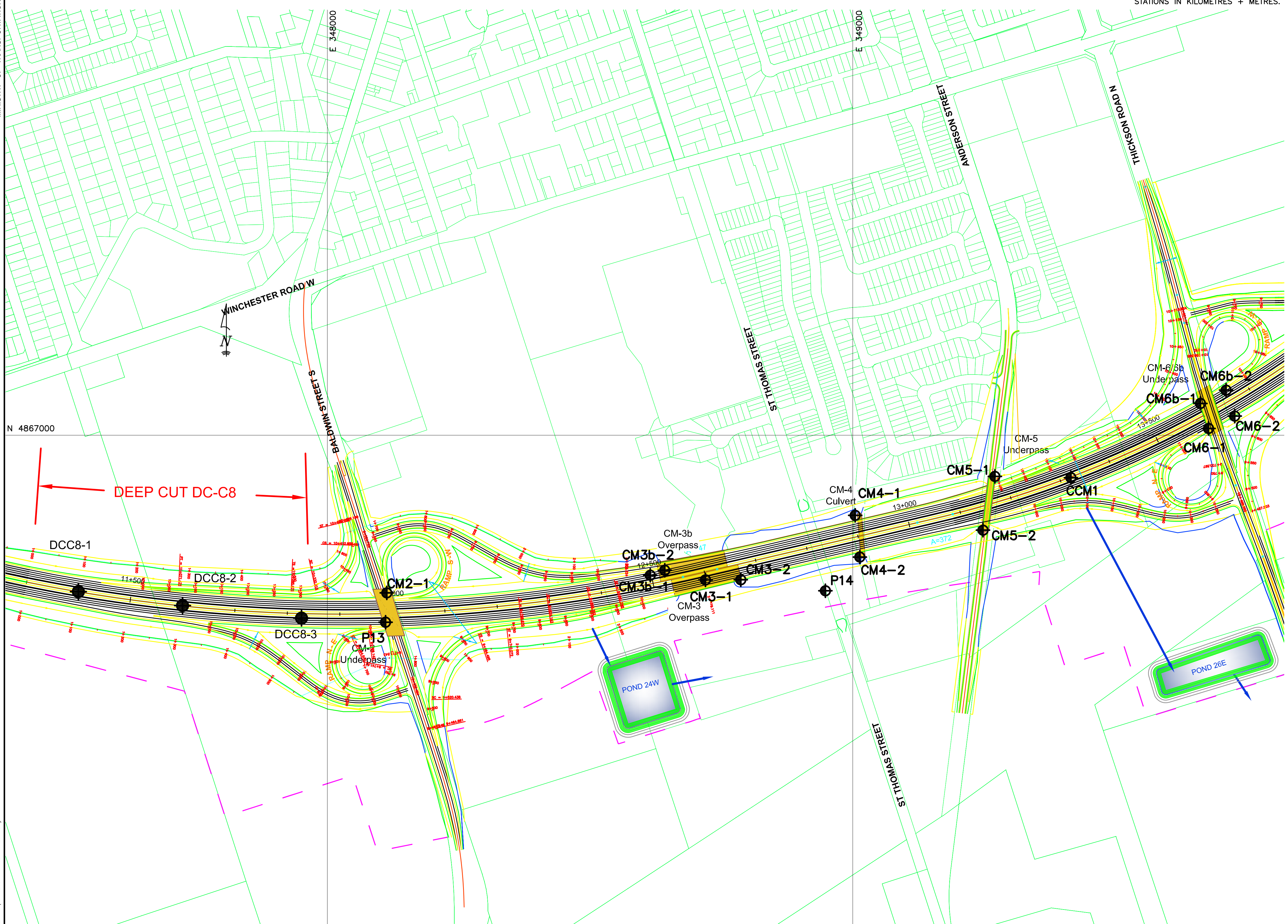
Geocres No. 30M15–113

HWY. 407E		PROJECT NO. 10TF023	DIST. Central
SUBM'D. NA	CHKD. GD	DATE: Feb. 15, 2011	SITE:
DRAWN: AL	CHKD. CN	APPD. BRG	DWG. 2

PLOT DATE: January 31, 2011
FILENAME: Z:\10TF023\Hwy 407E Drawings\10TF023C Final\10TF023C–002 JULY 13 2011.dwg

MINISTRY OF TRANSPORTATION, ONTARIO

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



PLAN

SCALE

80 0 100 200

W.O. No. 07-20016

SHEET

HIGHWAY 407 EAST EXTENSION
CENTRAL SECTION
BOREHOLE LOCATION – CENTRAL MAINLINE
East of Ashburn Road to Thickson Road N

Peto MacCallum Ltd.
CONSULTING ENGINEERS

LEGEND

- Borehole – Current Investigation
- Borehole – MTO Geocres

No.	ELEVATION	CO-ORDINATES	
		NORTHING	EASTING
DCC8-1	165.9	4867702.4	347526.3
DCC8-2	164.6	4867675.8	347724.5
DCC8-3	162.8	4867651.9	347950.9

NOTES

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

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

REFERENCE

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

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

Geocres No. 30M15-113

HWY. 407E		PROJECT NO. 10TF023	DIST. Central
SUBM'D. NA	CHKD. GD	DATE: Feb, 15, 2011	SITE:
DRAWN: AL	CHKD. CN	APPD. BRG	DWG. 3

MINISTRY OF TRANSPORTATION, ONTARIO

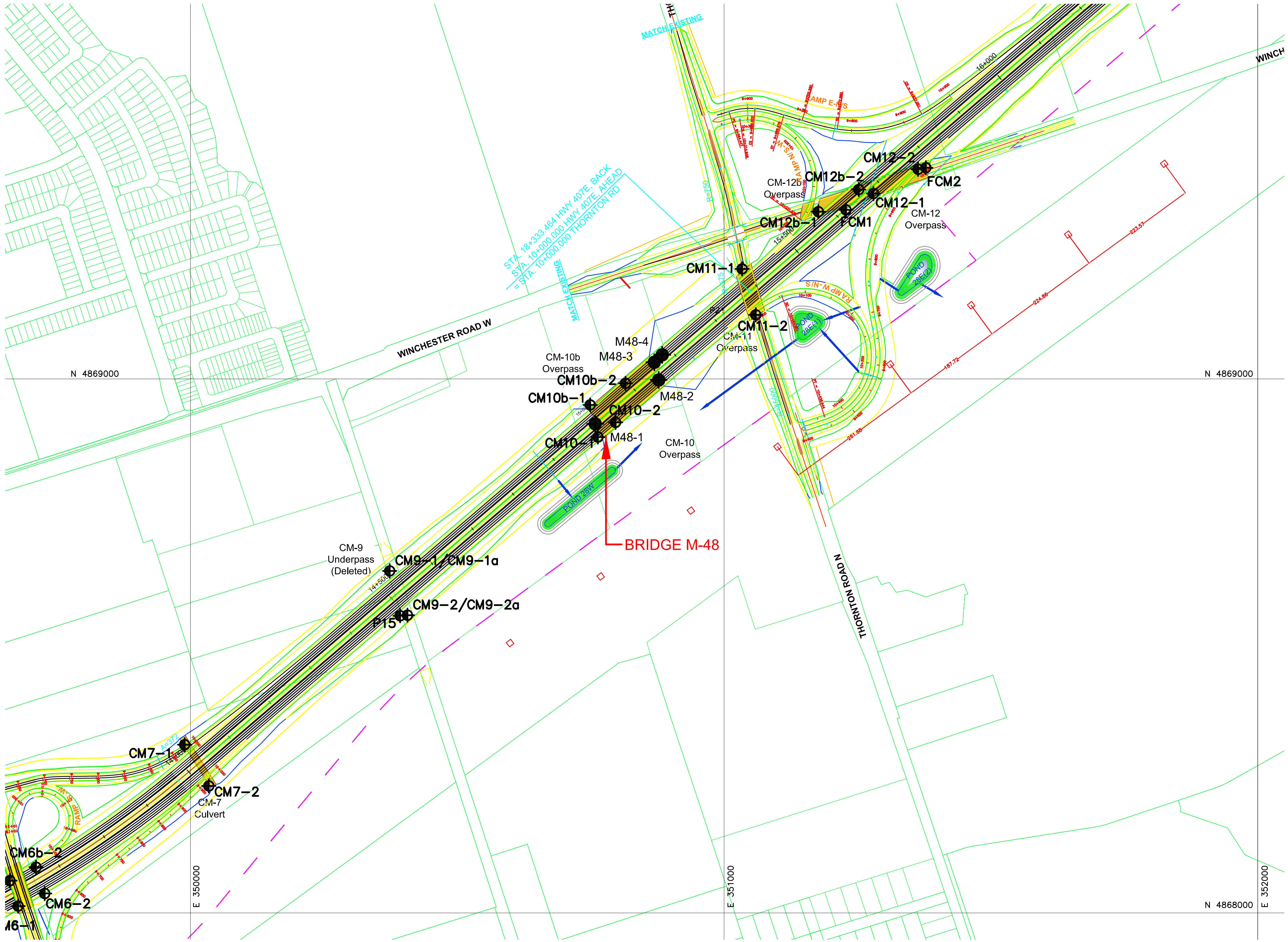
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

W.O. No. 07–20016

SHEET

HIGHWAY 407 EAST EXTENSION
CENTRAL SECTION
BOREHOLE LOCATION – CENTRAL MAINLINE
Thickson Road N to East of Thornton Road N

Peto MacCallum Ltd.
CONSULTING ENGINEERS



PLAN
SCALE
80 0 100 200

LEGEND

- Borehole – Current Investigation
- Borehole – MTO Geocres

No.	ELEVATION	CO–ORDINATES	
		NORTHING	EASTING
M48–1	161.3	4868916.0	350758.4
M48–2	155.0	4868998.1	350877.5
M48–3	155.6	4869031.3	350869.3
M48–4	154.6	4869045.2	350884.0

NOTES

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

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

REFERENCE

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

NO.	DATE	BY	REVISION

Geocres No. 30M15–113

HWY. 407E		PROJECT NO. 10TF023	DIST. Central
SUBM'D. NA	CHKD. GD	DATE: Feb, 15, 2011	SITE:
DRAWN: AL	CHKD. CN	APPD. BRG	DWG. 4

MINISTRY OF TRANSPORTATION, ONTARIO

METRIC

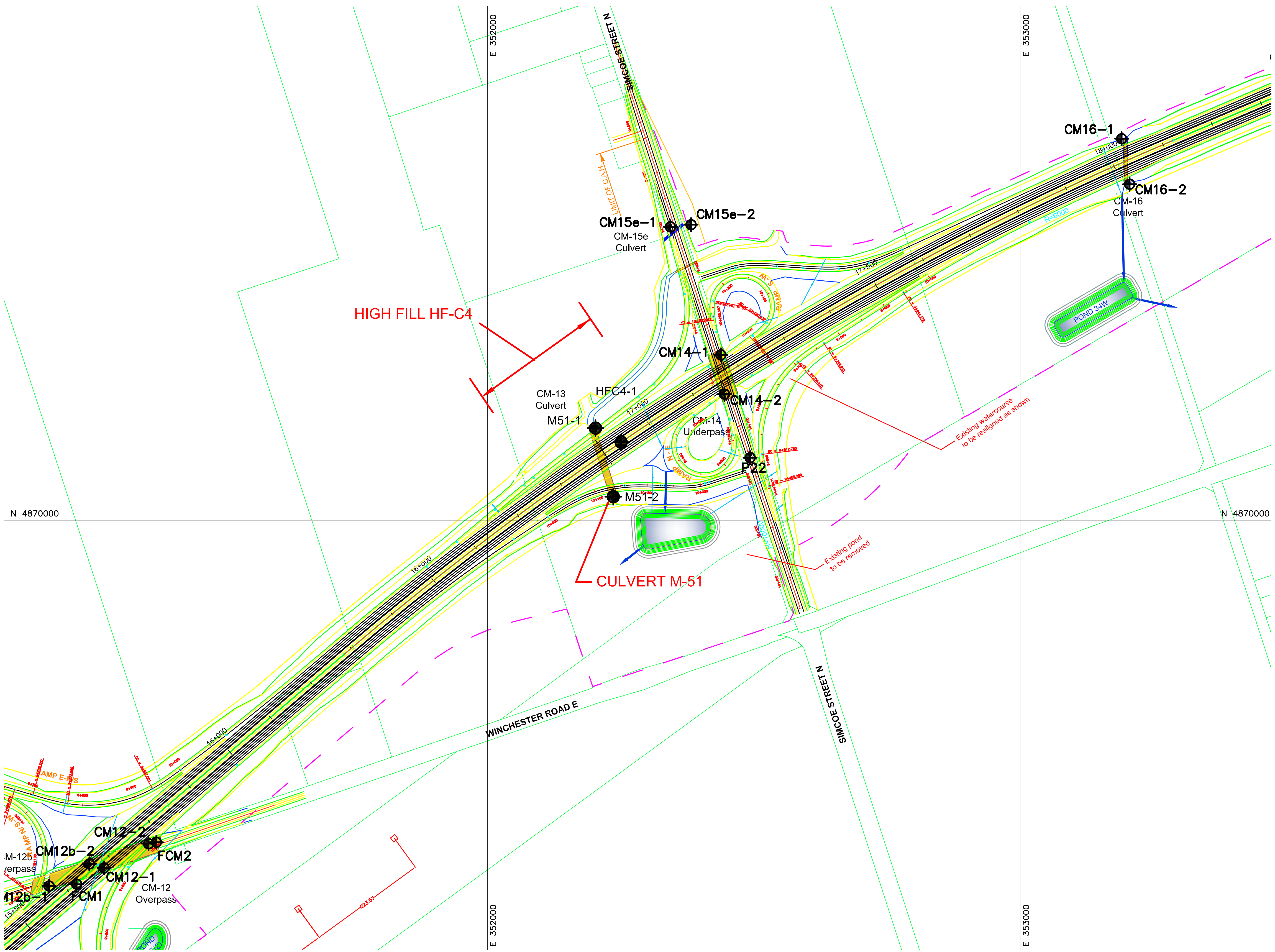
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

W.O. No. 07–20016

SHEET

HIGHWAY 407 EAST EXTENSION
CENTRAL SECTION
BOREHOLE LOCATION – CENTRAL MAINLINE
East of Thornton Road N to
East of Simcoe Street N

Peto MacCallum Ltd.
CONSULTING ENGINEERS



PLAN

SCALE

80 0 100 200

LEGEND

Borehole – Current Investigation

Borehole – MTO Geocres

No.	ELEVATION	CO–ORDINATES	
		NORTHING	EASTING
M51–1	180.8	4870172.9	352202.7
M51–2	176.8	4870044.3	352236.1
HFC4–1	180.7	4870146.5	352250.9

NOTES

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

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

REFERENCE

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

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

Geocres No. 30M15–113

HWY. 407E		PROJECT NO. 10TF023	DIST. Central
SUBM'D. NA	CHKD. GD	DATE: Feb. 15, 2011	SITE:
DRAWN: AL	CHKD. CN	APPD. BRG	DWG. 5

PLOT DATE: January 31, 2011

MINISTRY OF TRANSPORTATION, ONTARIO

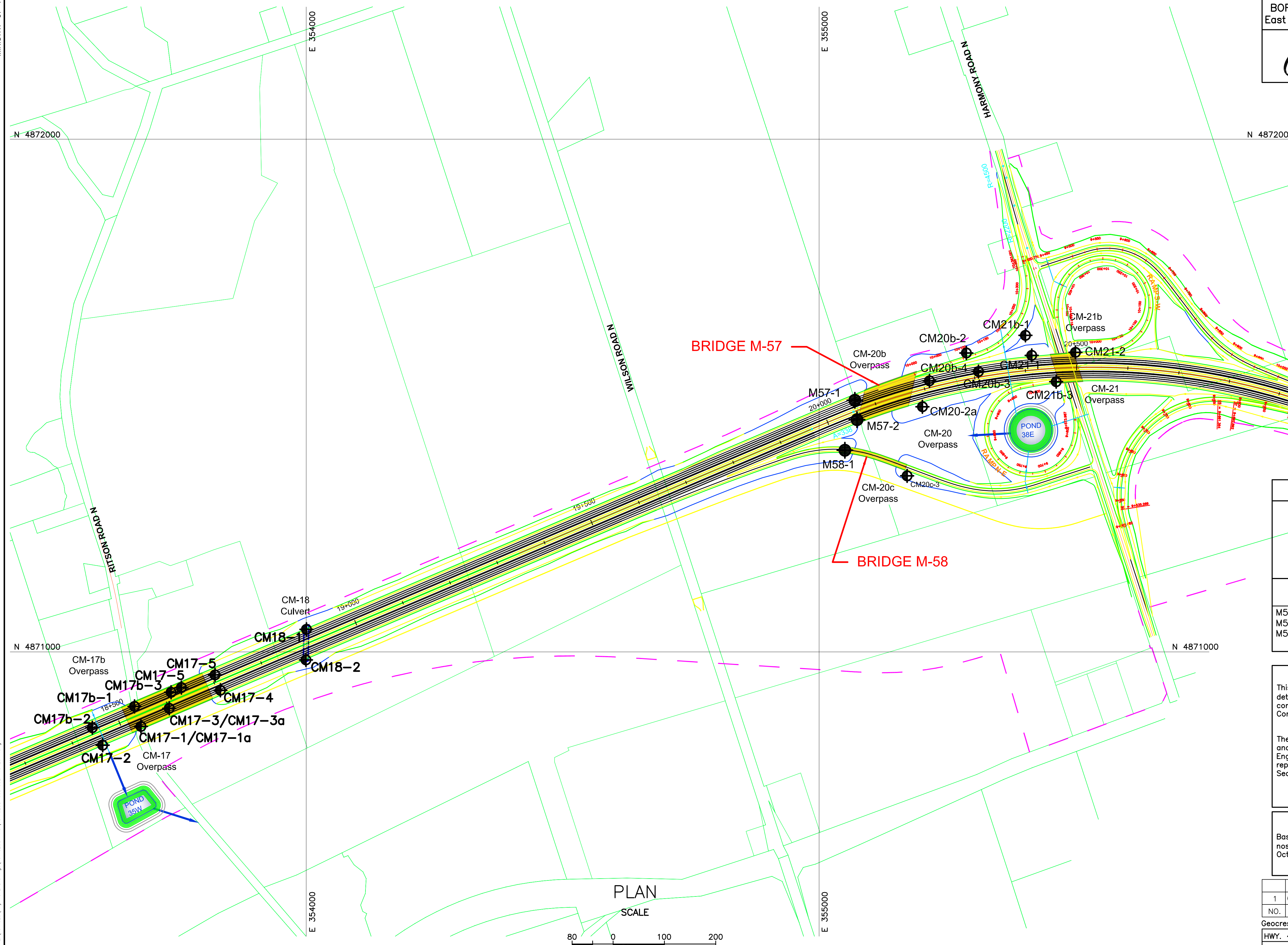
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

W.O. No. 07–20016

SHEET

HIGHWAY 407 EAST EXTENSION
CENTRAL SECTION
BOREHOLE LOCATION – CENTRAL MAINLINE
East of Simcoe Street N to Harmony Road N

Peto MacCallum Ltd.
CONSULTING ENGINEERS



LEGEND

Borehole – Current Investigation

Borehole – MTO Geocres

No.	ELEVATION	CO–ORDINATES	
		NORTHING	EASTING
M57–1	211.7	4871490.8	355069.7
M57–2	210.6	4871452.9	355074.0
M58–1	208.6	4871393.3	355050.3

NOTES

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

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

REFERENCE

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

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

Geocres No. 30M15–113

HWY. 407E		PROJECT NO. 10TF023	DIST. Central
SUBM'D. NA	CHKD. GD	DATE: Feb, 15, 2011	SITE:
DRAWN: AL	CHKD. CN	APPD. BRG	DWG. 6

PLOT DATE: January 31, 2011
FILENAME: Z:\10TF023\Hwy 407E Drawings\10TF023C Final\10TF023C-003 JULY 13 2011.dwg

APPENDIX A

RECORD OF BOREHOLE SHEETS

Ontario

Peto MacCallum Ltd. CONSULTING ENGINEERS

Foundation Design

RECORD OF BOREHOLE No M51-1

1 of 1 **METRIC**

G.W.P. 07-20016 LOCATION Coords: 4 870 172.9 N; 352 202.7 E ORIGINATED BY S.A.

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

DATUM Geodetic DATE April 05, 2011 CHECKED BY G.D.

ELEV / DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES
180.8	Ground Surface			
0.0	Topsoil	1	SS	4
180.5 / 0.3	Clayey silt, organic inclusions, sand seams. Firm, Brown/grey, Wet to moist (ALLUVIUM)	2	SS	3
	some gravel			
179.0 / 1.8	Clayey silt, trace gravel, sand seams, cobbles. Stiff to very stiff, Grey, Moist (TILL)	3	SS	11
		4	SS	14
		5	SS	19
		6	SS	20
		7	SS	22
		8	SS	20
173.3 / 7.5	Silty sand, some clay, trace gravel, cobbles. Very dense, Grey, Moist to wet (TILL)	9	SS	98/28cm
		10	SS	61/15cm
171.4 / 9.4	End of borehole			

Samples 9 and 10: Sampler bouncing

* 2011 04 05

▽ Water level observed during drilling

▼ Water level measured after drilling

Piezometer Readings:

Date	Depth (m)	Elev.
Apr. 28, '11	(0.8)	181.6
May 04, '11	(0.7)	181.5

Piezometer Legend:

- Backfill
- Bentonite seal
- Filter sand
- 19mm dia. PVC screen

Column headings: GROUND WATER CONDITIONS; ELEVATION SCALE (180, 179, 178, 177, 176, 175, 174, 173, 172); 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, ×⁵: Numbers refer to Sensitivity; 15–○–5 (20, 10) (%) STRAIN AT FAILURE

Ontario

Peto MacCallum Ltd. CONSULTING ENGINEERS

Foundation Design

RECORD OF BOREHOLE No M51-2

1 of 1 **METRIC**

G.W.P. 07-20016 LOCATION Coords: 4 870 044.3 N; 352 236.1 E ORIGINATED BY S.A.

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

DATUM Geodetic DATE April 05, 2011 CHECKED BY G.D.

ELEV / DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GR SA SI CL
176.8	Ground Surface				
0.0	Topsoil	1	SS	8	
176.4 / 0.4	Silt, sand seams, organics				
176.0 / 0.8	Loose to compact, Brown/grey, Moist (ALLUVIUM)	2	SS	12	
	Clayey silt, some sand. Stiff to hard, Brown/grey, Moist	3	SS	45	0 11 52 37
	cobbles. Grey (TILL)	4	SS	65	
	trace gravel, silt layers	5	SS	88/23cm	
		6	SS	80/28cm	1 10 49 40
172.3 / 4.5	Sand with silt, some clay, trace to some gravel, cobbles and boulders. Dense to very dense, Grey, Moist (TILL)	7	SS	50/8cm	
		8	SS	47	10 44 29 17
		9	SS	65/8cm	
		10	SS	76/15cm	
167.5 / 9.3	End of borehole				

Samples 5, 6, 7, 9 and 10: Sampler bouncing

* 2011 04 05

▼ Water level measured after drilling

Column headings: GROUND WATER CONDITIONS; ELEVATION SCALE (176, 175, 174, 173, 172, 171, 170, 169, 168); 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, ×⁵: Numbers refer to Sensitivity; 15–○–5 (20, 10) (%) STRAIN AT FAILURE

Ontario | Peto MacCallum Ltd. Consulting Engineers | Foundation Design

RECORD OF BOREHOLE No M57-1

 METRIC

G.W.P. 07-20016 LOCATION Coords: 4 871 490.8 N; 355 069.7 E ORIGINATED BY S.A.

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

DATUM Geodetic DATE June 06, 2011 CHECKED BY G.D.

ELEV / DEPTH	SOIL PROFILE – DESCRIPTION	NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
211.7	Ground Surface				
0.0 / 211.4	Topsoil				
0.3	Clayey silt organic inclusions sand seams; Firm, Brown/dark brown, Moist	1	SS	6	
		2	SS	5	
210.5 / 1.2	Clayey silt, trace gravel sand seams, cobbles; Stiff to very stiff, Brown/grey, Moist (TILL)	3	SS	9	7 37 40 16
		4	SS	26	
209.0 / 2.7	Silty sand, trace gravel; Compact, Brown/grey, Moist				
208.6 / 3.1	Clayey silt, trace gravel sand seams, cobbles; Hard, Brown/grey, Moist (TILL)	5	SS	38	
	sand and gravel layer	6	SS	55	
	Grey	7	SS	136/28cm	7 32 37 24
204.2 / 7.5	Sandy silt trace gravel, trace clay cobbles to 9.0m; Very dense, Grey, Wet to moist (TILL)	8	SS	102	9 38 48 5
		9	SS	110/8cm	
	clayey silt seams	10	SS	70/5cm	
201.0 / 10.7	End of borehole				

Samples 7, 9 and 10: Sampler bouncing

* 2011 06 06

▽ Water level observed during drilling

▼ Water level measured after drilling

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

Ontario | Peto MacCallum Ltd. Consulting Engineers | Foundation Design

RECORD OF BOREHOLE No M57-2

 METRIC

G.W.P. 07-20016 LOCATION Coords: 4 871 452.9 N; 355 074.0 E ORIGINATED BY S.A.

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

DATUM Geodetic DATE June 06, 2011 CHECKED BY G.D.

ELEV / DEPTH	SOIL PROFILE – DESCRIPTION	NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
210.6 / 0.0	Ground Surface; Topsoil				
210.4 / 0.2	Clayey silt, rootlets organic inclusions; Firm, Brown, Moist	1	SS	5	
209.7 / 0.9	Clayey silt, trace gravel sand seams, cobbles; Stiff to hard, Brown/grey, Moist (TILL)	2	SS	10	
		3	SS	11	
		4	SS	16	
		5	SS	34	
206.3 / 4.3	Silty sand, trace gravel; Dense to very dense, Grey, Wet	6	SS	53	
205.7 / 4.9	Clayey silt, trace gravel sand seams; Hard to very stiff, Brown/grey, Moist (TILL)	7	SS	28	
203.1 / 7.5	Sandy silt, trace gravel clayey silt seams; Very dense, Grey, Wet to moist (TILL)	8	SS	71	
		9	SS	110/15cm	
		10	SS	100/5cm	
		11	SS	90/3cm	
198.4 / 12.2	End of borehole				

Samples 9 to 11: Sampler bouncing

* 2011 06 06

▽ Water level observed during drilling

▼ Water level measured after drilling

Cont'd

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

Ontario — Peto MacCallum Ltd. Consulting Engineers — Foundation Design

RECORD OF BOREHOLE No M57-2

 METRIC

G.W.P. 07-20016 LOCATION Coords: 4 871 452.9 N; 355 074.0 E ORIGINATED BY S.A.

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

DATUM Geodetic DATE June 06, 2011 CHECKED BY G.D.

Water Level Readings:

Date	Depth (m)	Elev.
Jun. 10,'11	4.0	206.6
Jun. 17,'11	4.2	206.4

Piezometer Legend:

- Bentonite seal
- Filter sand
- 19mm dia. PVC screen
- Filter bed

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

Ontario — Peto MacCallum Ltd. Consulting Engineers — Foundation Design

RECORD OF BOREHOLE No M58-1

 METRIC

G.W.P. 07-20016 LOCATION Coords: 4 871 393.3 N; 355 050.3 E ORIGINATED BY S.A.

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

DATUM Geodetic DATE June 06, 2011 CHECKED BY G.D.

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
208.6	Ground Surface				
0.0	Topsoil	1	SS	4	
208.2 / 0.4	Clayey silt, trace gravel, rootlets to 0.9m; Firm, Brown, Moist; sand seams; Stiff to very stiff, Brown/grey	2	SS	13	
		3	SS	16	
206.5 / 2.1	Sand, trace silt, trace gravel; Compact to dense, Brown/grey, Wet	4	SS	22	2 91 (7)
		5	SS	32	
		6	SS	42	
203.7 / 4.9	Clayey silt, some sand, trace gravel; Hard, Brown/grey, Moist (TILL)				
202.6 / 6.0	Silty sand, trace gravel, trace clay; Very dense, Brown/grey, Wet (TILL)	7	SS	90	
202.2 / 6.4	Clayey silt, trace gravel, cobbles; Hard, Brown/grey, Moist (TILL); sand seams; Grey	8	SS	55	4 27 39 30
		9	SS	34	
197.9 / 10.7	Sand and silt, trace to some clay; Very dense, Grey, Moist (TILL)	10	SS	50/13cm	
		11	SS	68	0 48 42 10
		12	SS	110/10cm	
193.6	Cont'd				

Elevation scale: 208, 207, 206, 205, 204, 203, 202, 201, 200, 199, 198, 197, 196, 195, 194

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

Ontario

Peto MacCallum Ltd. CONSULTING ENGINEERS

Foundation Design

RECORD OF BOREHOLE No M58-1

METRIC

G.W.P. 07-20016 LOCATION Coords: 4 871 393.3 N; 355 050.3 E ORIGINATED BY S.A.

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

DATUM Geodetic DATE June 06, 2011 CHECKED BY G.D.

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
193.6								
15.0	Sand and silt trace to some clay Very dense Grey Moist (TILL) (Cont'd)	13	SS	100/5cm		193		
						192		
191.7		14	SS	110/10cm				
16.9	End of borehole Samples 10, 12, 13 and 14: Sampler bouncing							

SOIL PROFILE / SAMPLES / DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH kPa: O 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

* 2011 06 06

∇ Water level observed during drilling

▼ Water level measured after drilling

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

Ontario

Peto MacCallum Ltd. CONSULTING ENGINEERS

Foundation Design

RECORD OF BOREHOLE No HFC4-1

METRIC

G.W.P. 07-20016 LOCATION Coords: 4 870 146.5 N; 352 250.9 E ORIGINATED BY S.A.

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

DATUM Geodetic DATE April 05, 2011 CHECKED BY G.D

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
180.7	Ground Surface				∇*				
0.0	Organic clayey silt rootlets, topsoil inclusions	1	SS	2					
180.3 0.4	Soft Dark brown/black Moist to wet (ALLUIVIUM)	2	SS	9		180			
	Clayey silt some sand, trace gravel Firm to very stiff Grey/brown Moist	3	SS	10		179			
	sand seams cobbles	4	SS	12		178	175		1 12 55 32
	Grey (TILL)	5	SS	12		177			
		6	SS	14					
		7	SS	19	▼*	176			
						175			
174.5 6.2	Silty sand some clay, trace gravel	8	SS	7					4 47 37 12
174.0 6.7	Loose to compact Grey Wet (TILL) End of borehole					174			

SOIL PROFILE / SAMPLES / DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH kPa: O 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

* 2011 04 05

∇ Water level observed during drilling

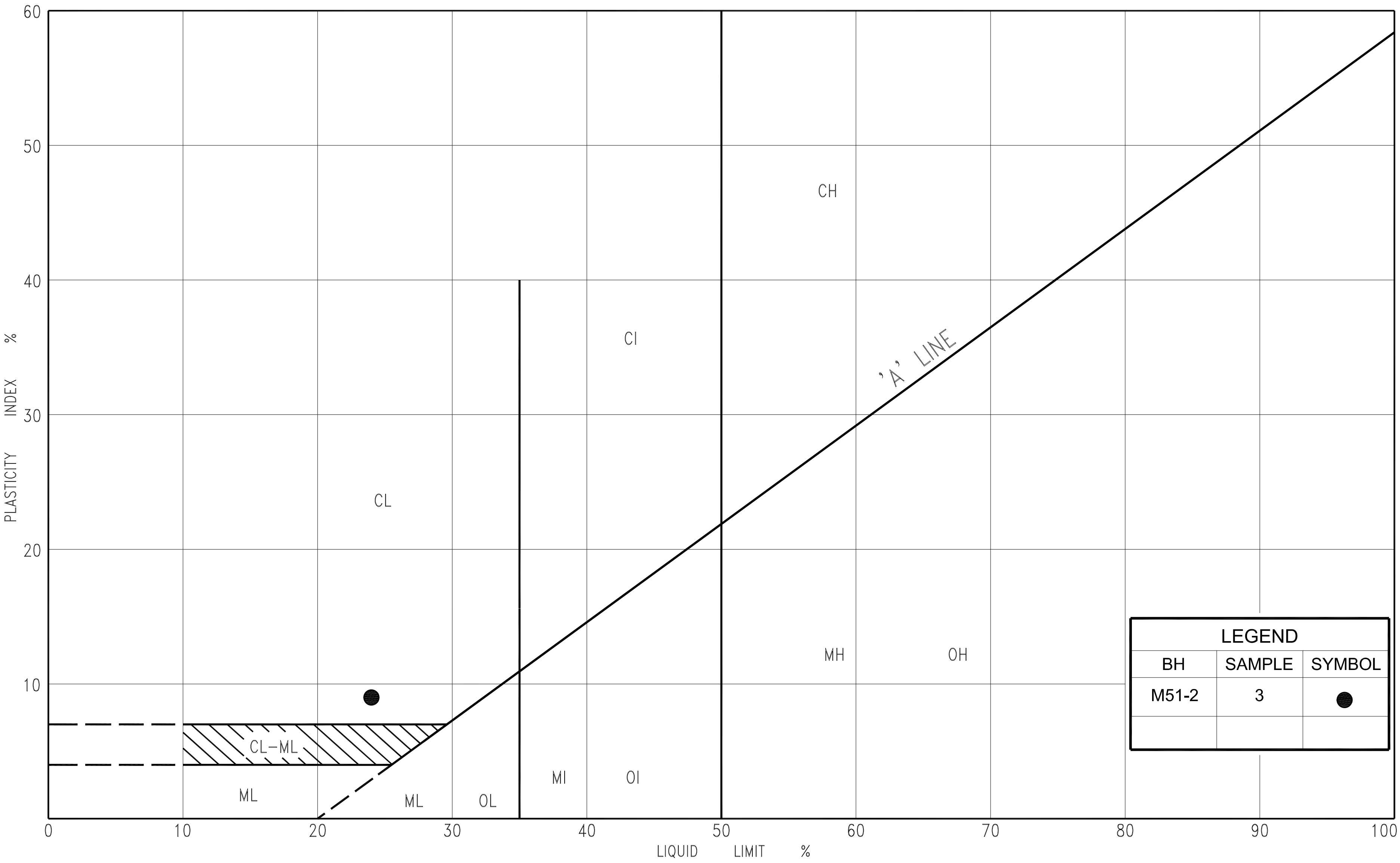
▼ Water level measured after drilling

■ Penetrometer test

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

APPENDIX B

LABORATORY TEST RESULTS

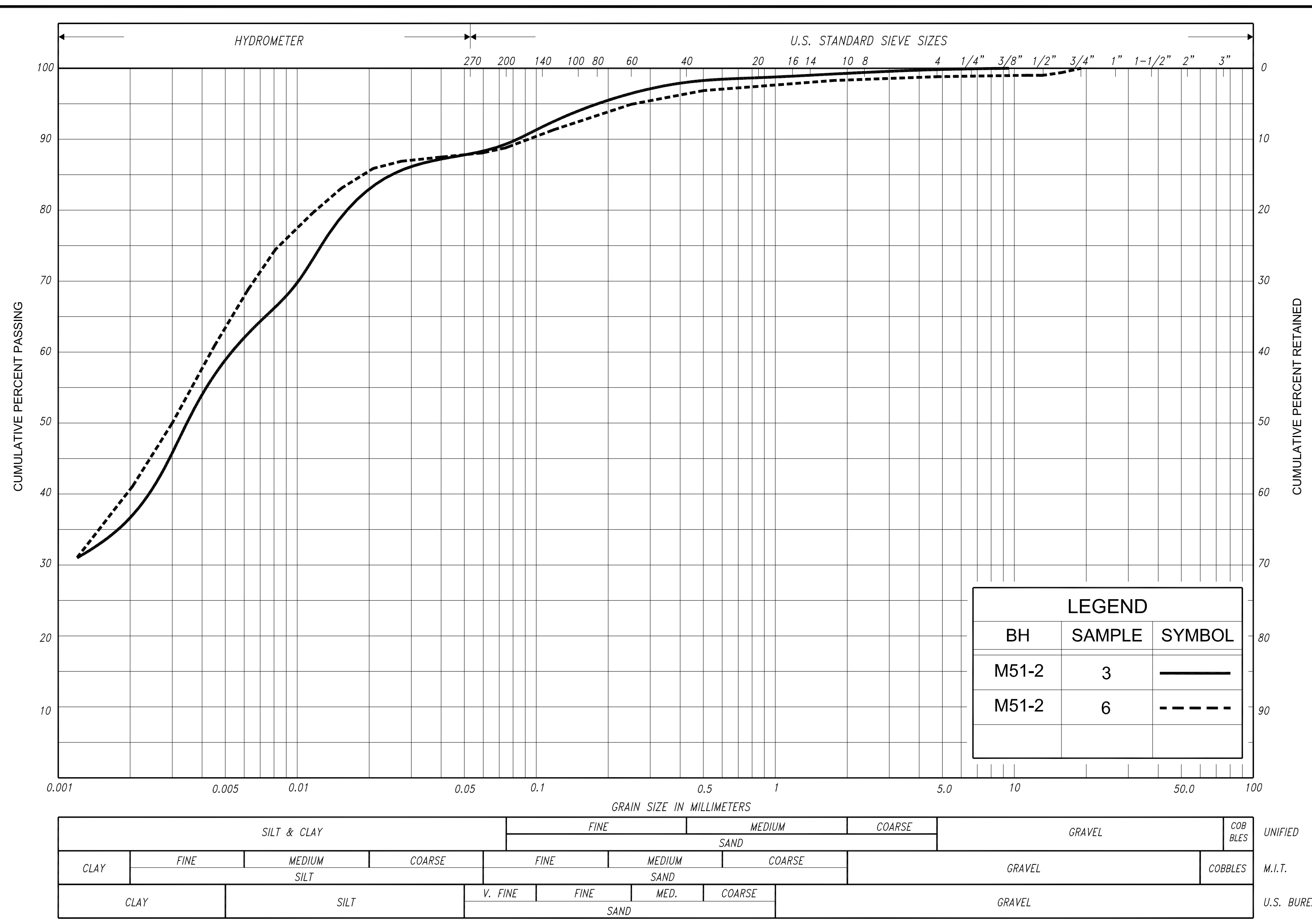


Ontario
Peto MacCallum Ltd. CONSULTING ENGINEERS

PLASTICITY CHART

CLAYEY SILT, some sand, trace gravel
(TILL)

FIG No.	M51-PC-1
HWY:	407E
W.P. No.	07-20016

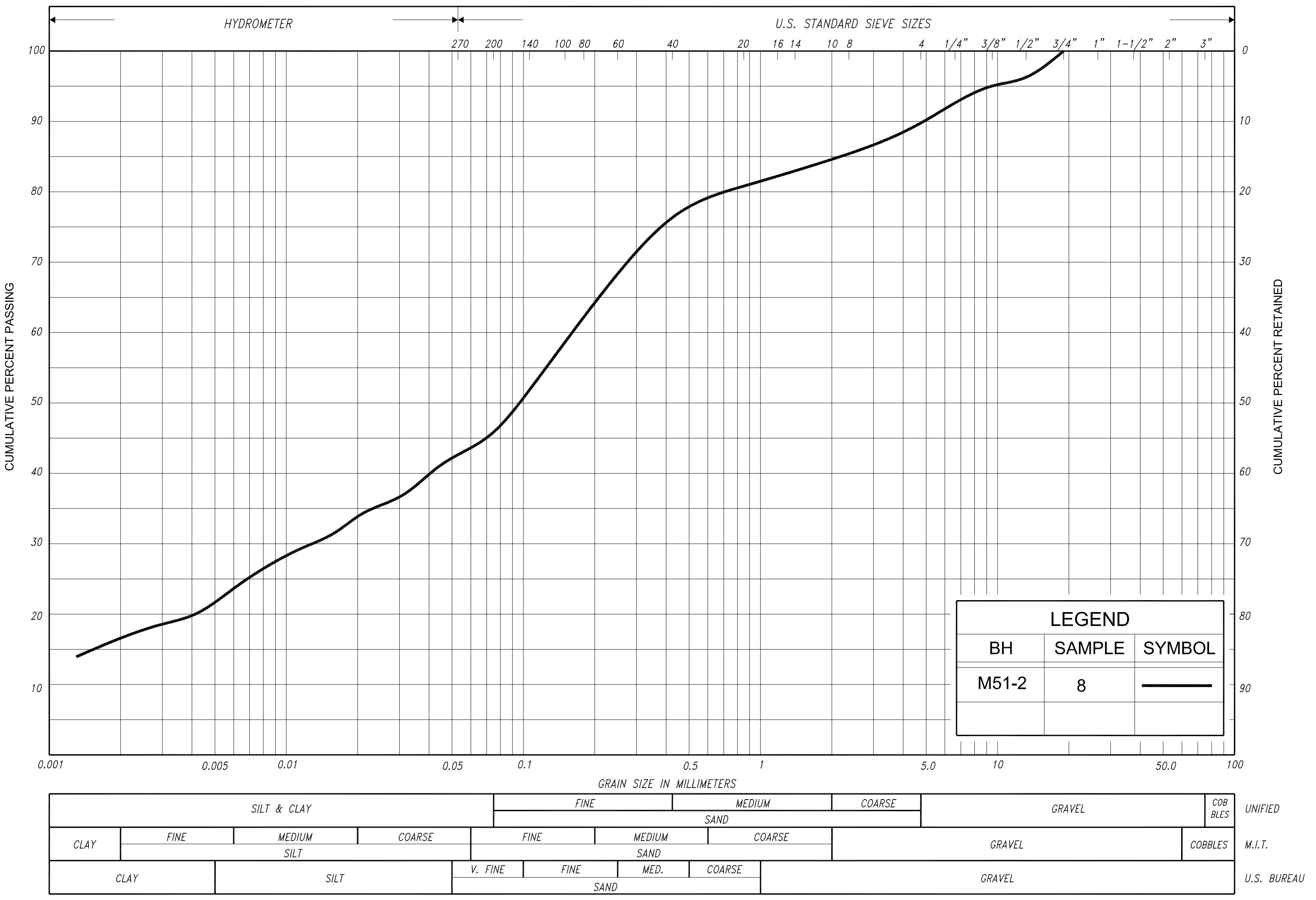


Ontario
Peto MacCallum Ltd. CONSULTING ENGINEERS

GRAIN SIZE DISTRIBUTION

CLAYEY SILT, some sand, trace gravel
(TILL)

FIG No.	M51-GS-1
HWY:	407E
W.P. No.	07-20016



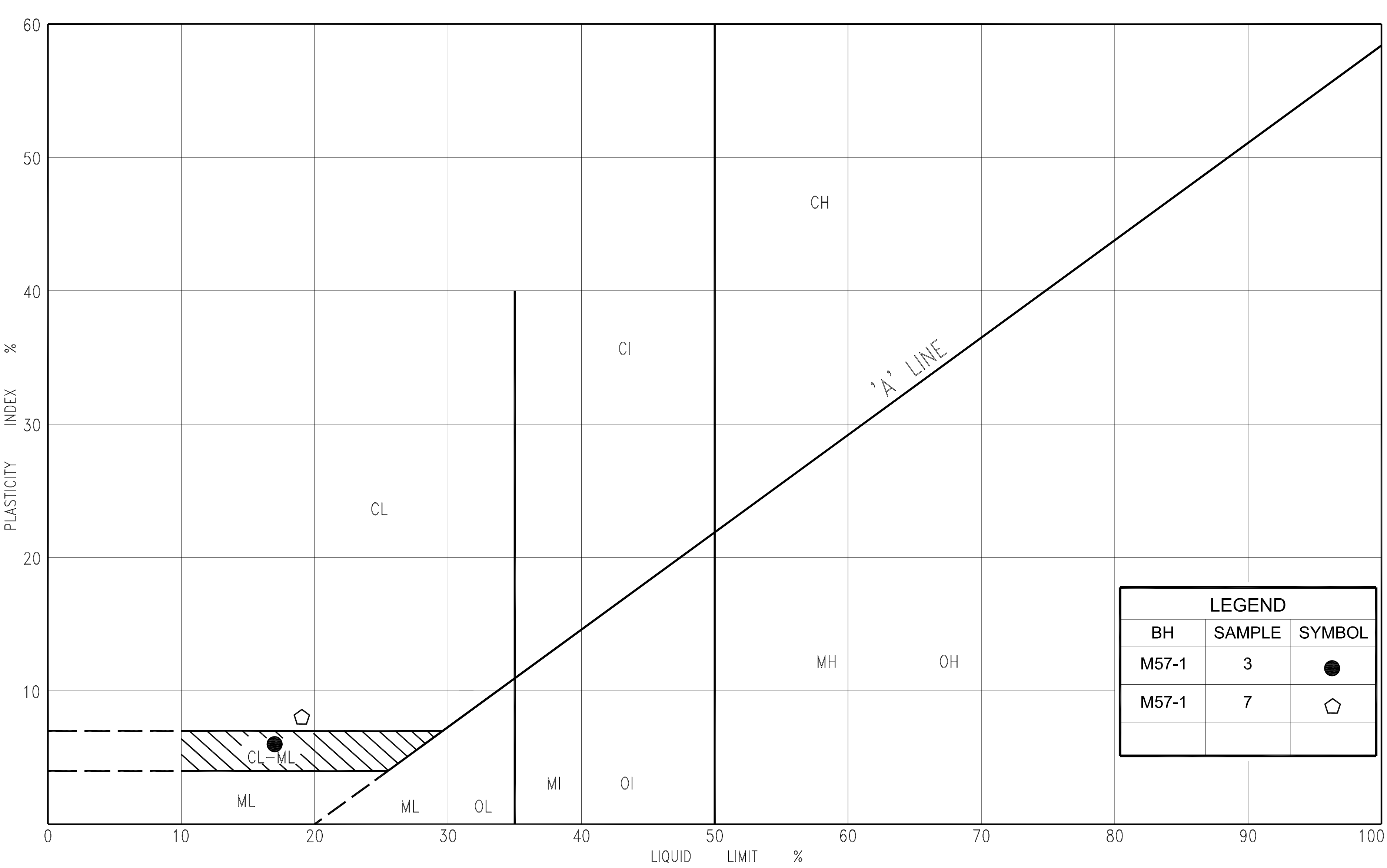
Ontario

Peto MacCallum Ltd. CONSULTING ENGINEERS

GRAIN SIZE DISTRIBUTION

SAND, with silt, some clay, trace to some gravel

FIG No.	M51-GS-2
HWY:	407E
W.P. No.	07-20016



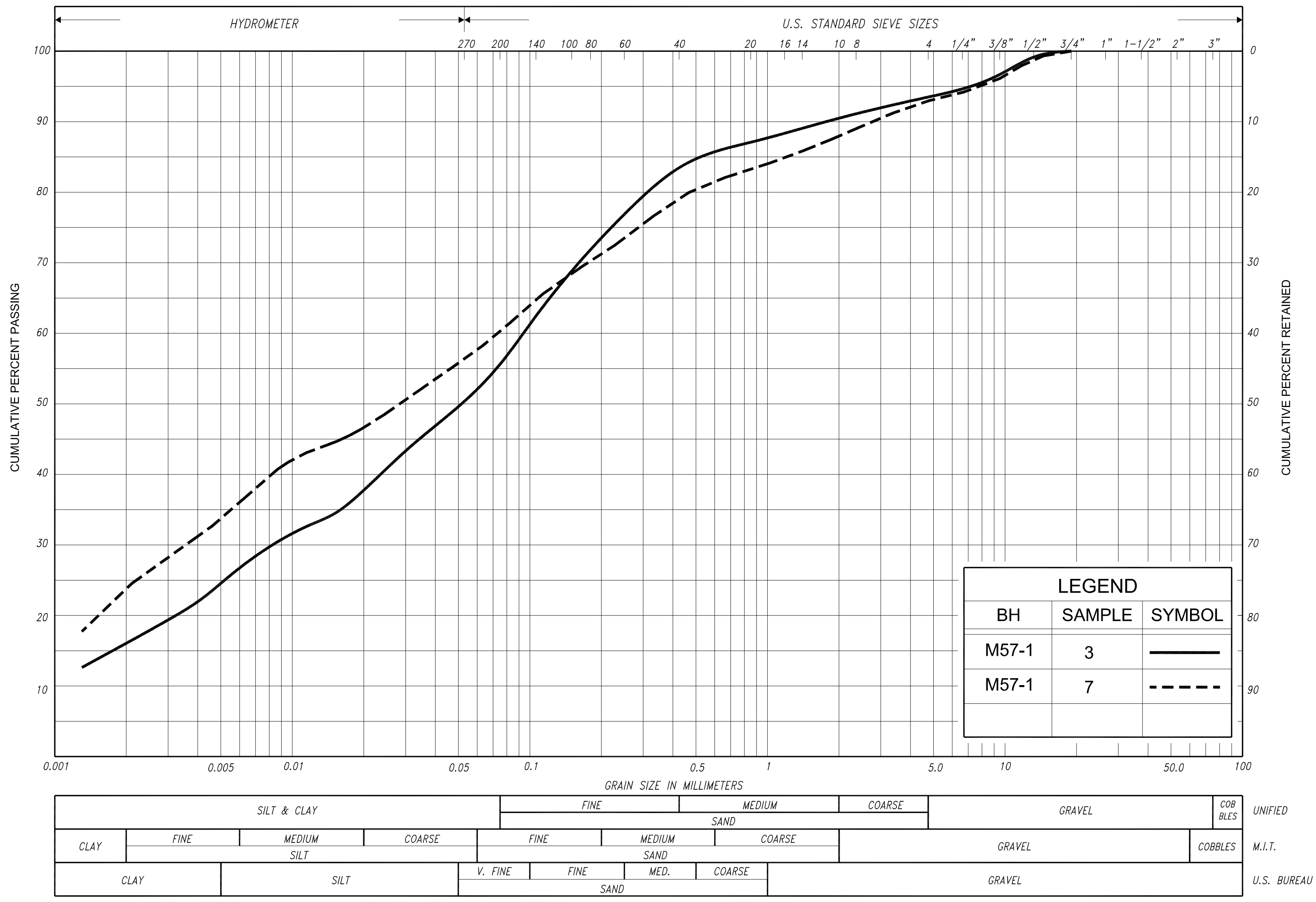
Ontario

Peto MacCallum Ltd. CONSULTING ENGINEERS

PLASTICITY CHART

CLAYEY SILT, sandy, trace gravel
(TILL)

FIG No.	M57-PC-1
HWY:	407E
W.P. No.	07-20016

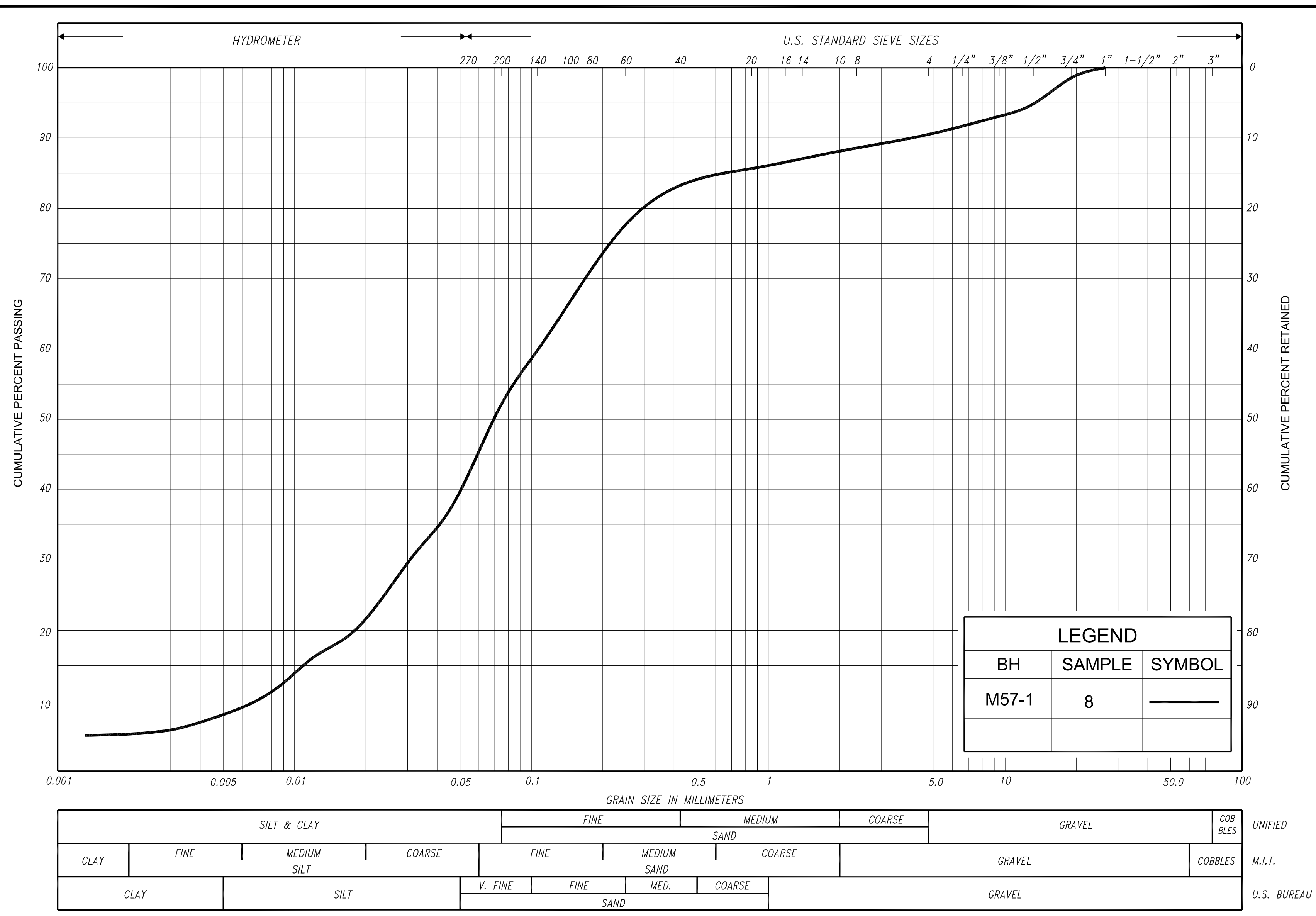


Ontario
Peto MacCallum Ltd. CONSULTING ENGINEERS

GRAIN SIZE DISTRIBUTION

CLAYEY SILT, sandy, trace gravel
(TILL)

FIG No.	M57-GS-1
HWY:	407E
W.O. No.	07-20016

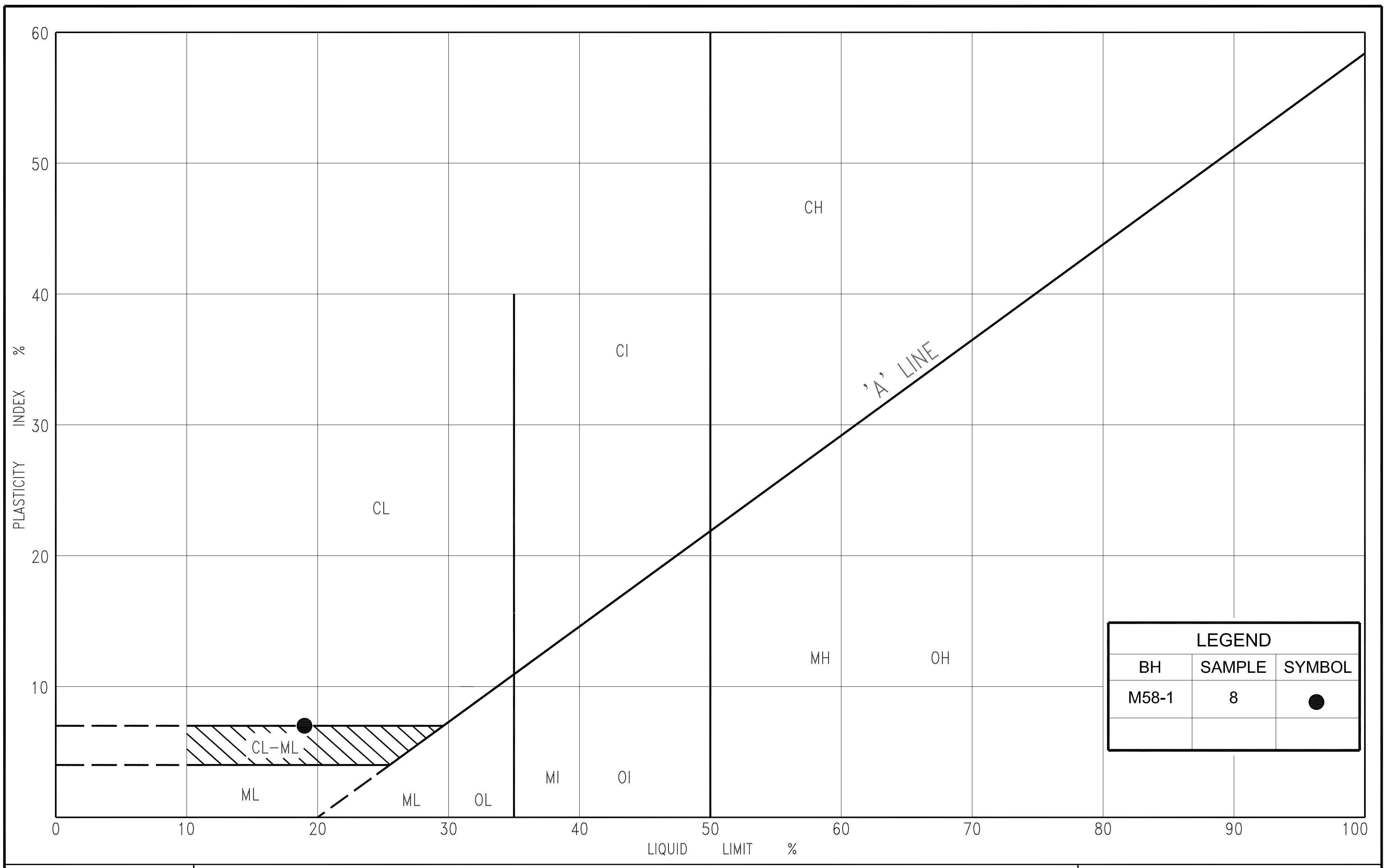


Ontario
Peto MacCallum Ltd. CONSULTING ENGINEERS

GRAIN SIZE DISTRIBUTION

SANDY SILT, trace gravel, trace clay
(TILL)

FIG No.	M57-GS-2
HWY:	407E
W.P. No.	07-20016

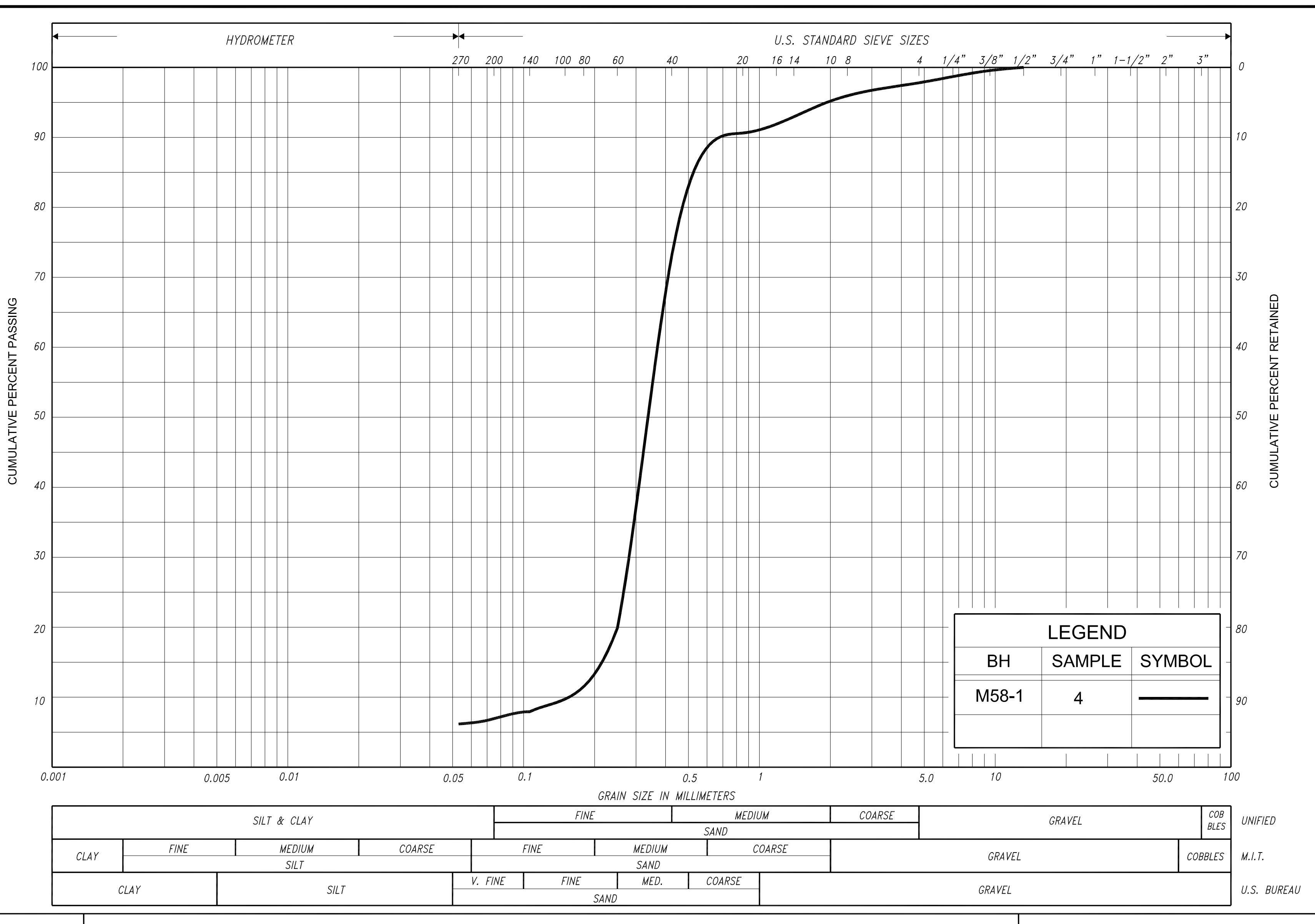


Ontario
Peto MacCallum Ltd. CONSULTING ENGINEERS

PLASTICITY CHART

CLAYEY SILT, with sand, trace gravel
(TILL)

FIG No.	M58-PC-1
HWY:	407E
W.P. No.	07-20016

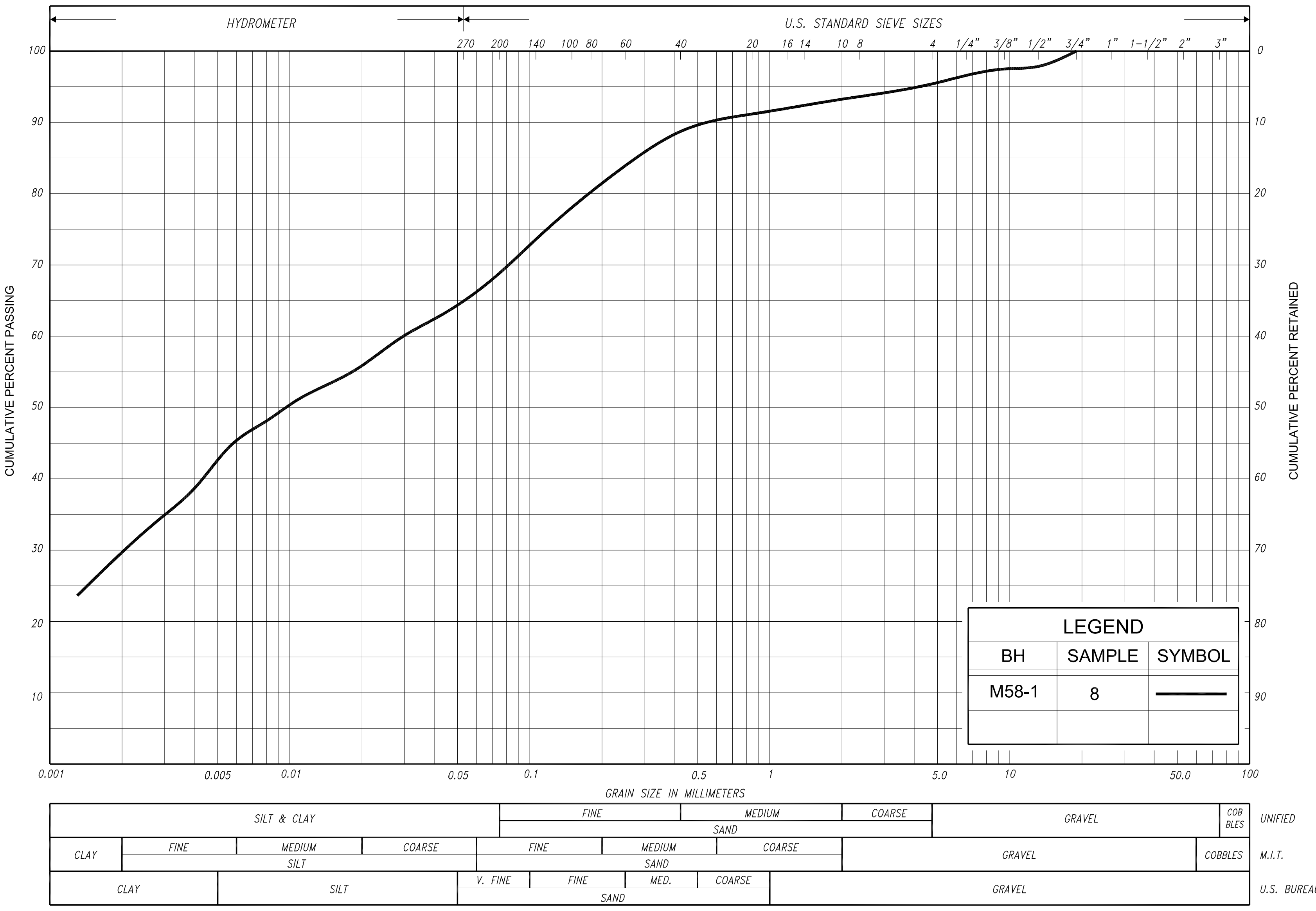


Ontario
Peto MacCallum Ltd. CONSULTING ENGINEERS

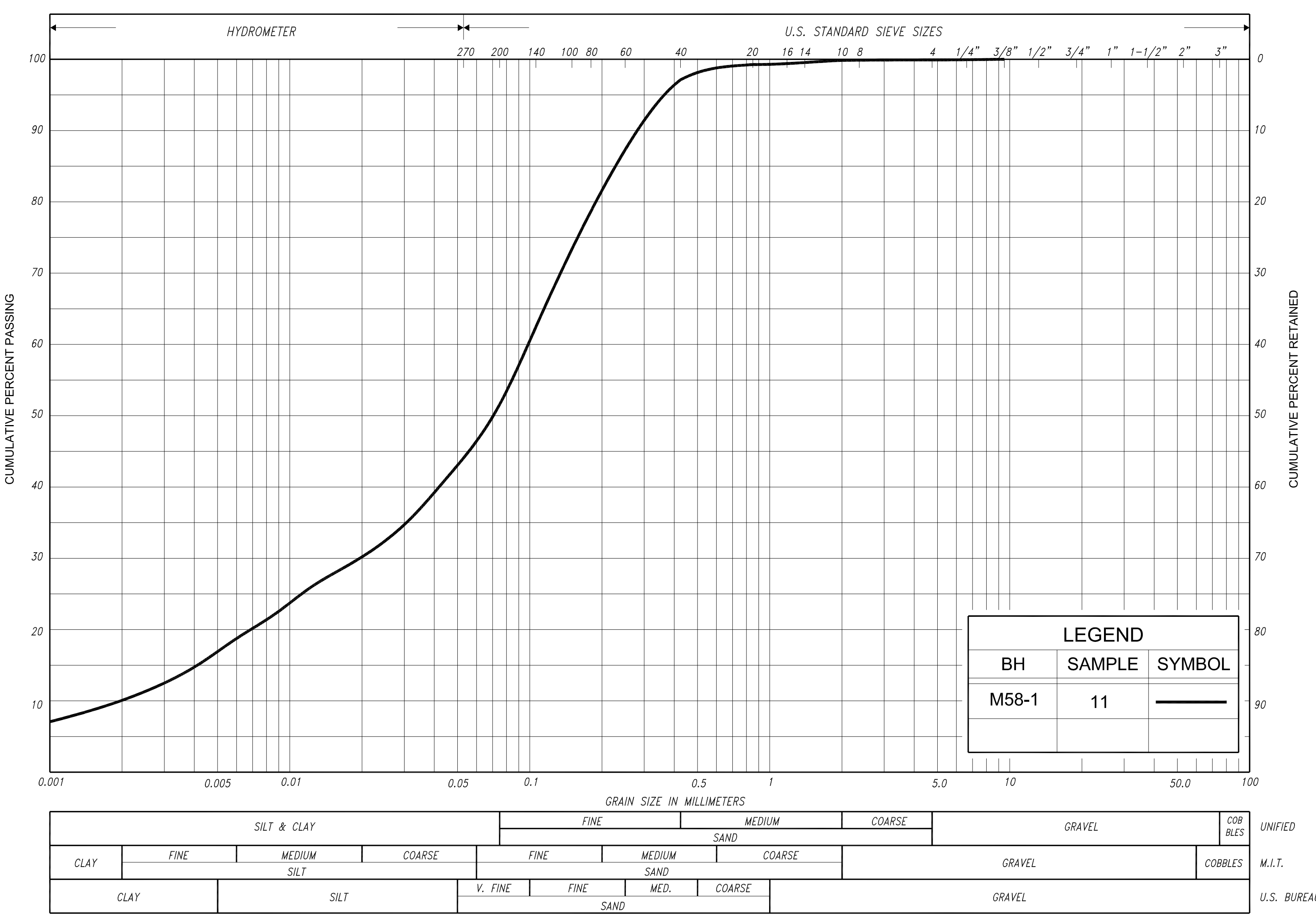
GRAIN SIZE DISTRIBUTION

SAND, trace silt, trace gravel

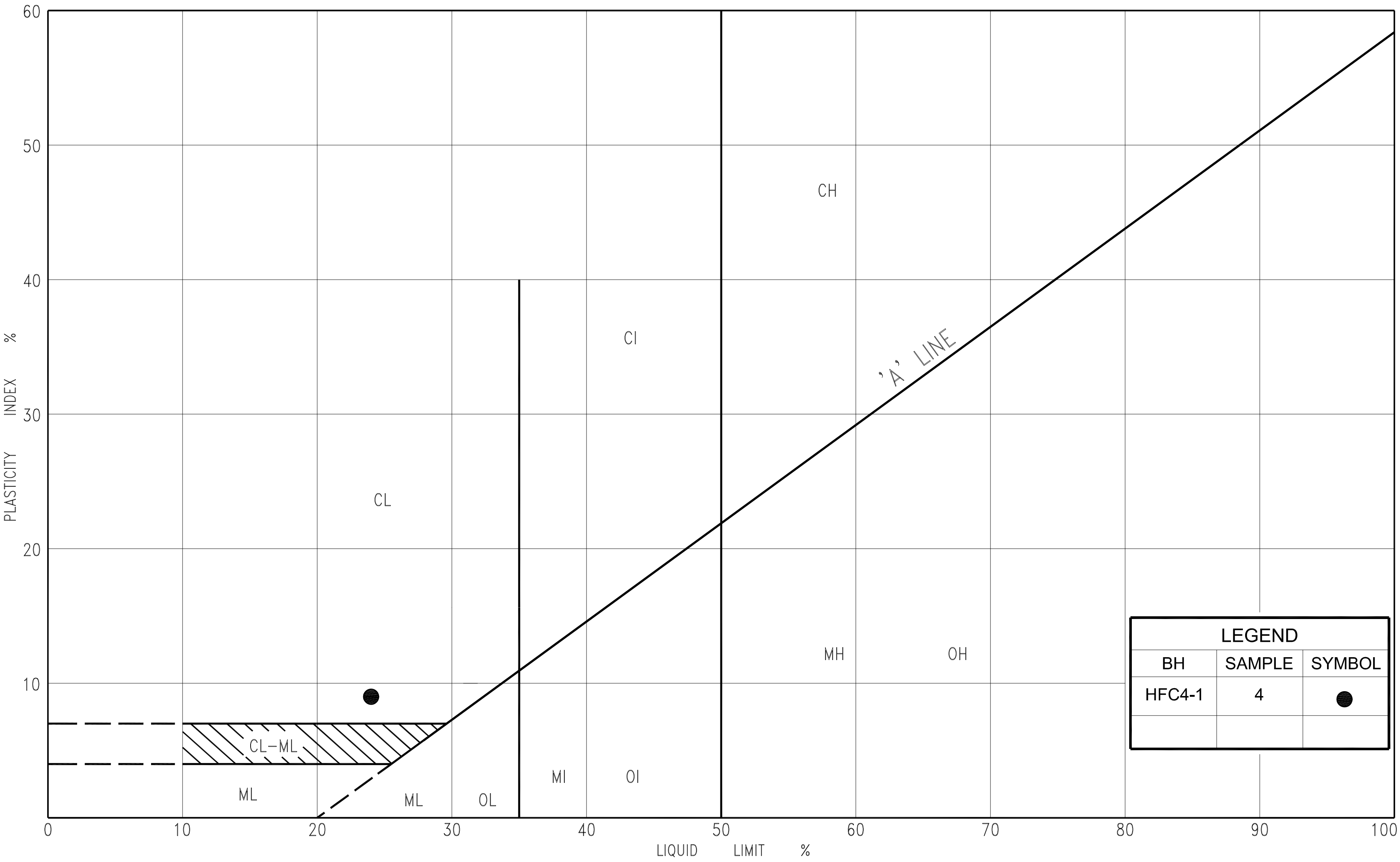
FIG No.	M58-GS-1
HWY:	407E
W.P. No.	07-20016



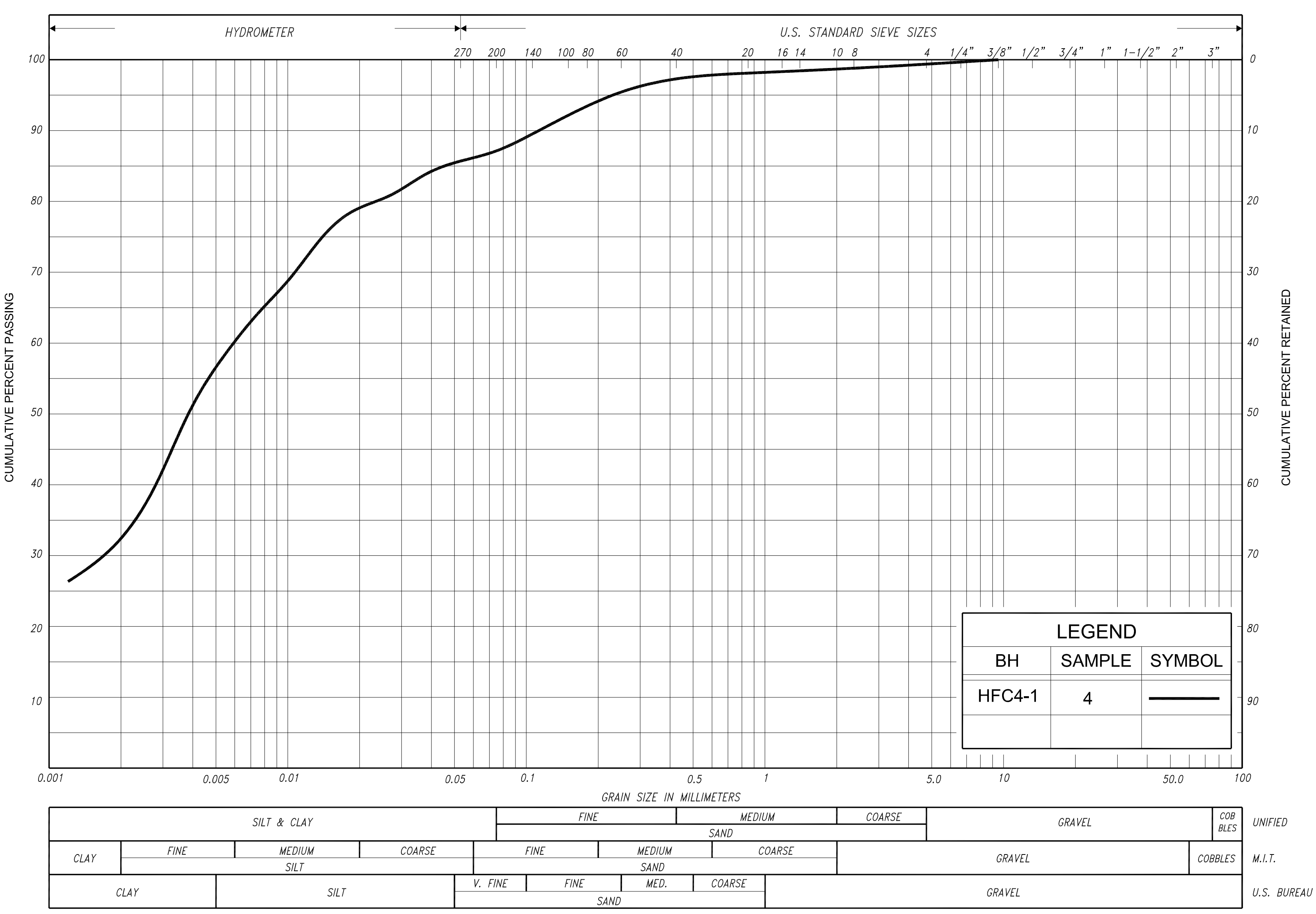
GRAIN SIZE DISTRIBUTION CLAYEY SILT, with sand, trace gravel (TILL)	FIG No.	M58-GS-2
	HWY:	407E
	W.P. No.	07-20016



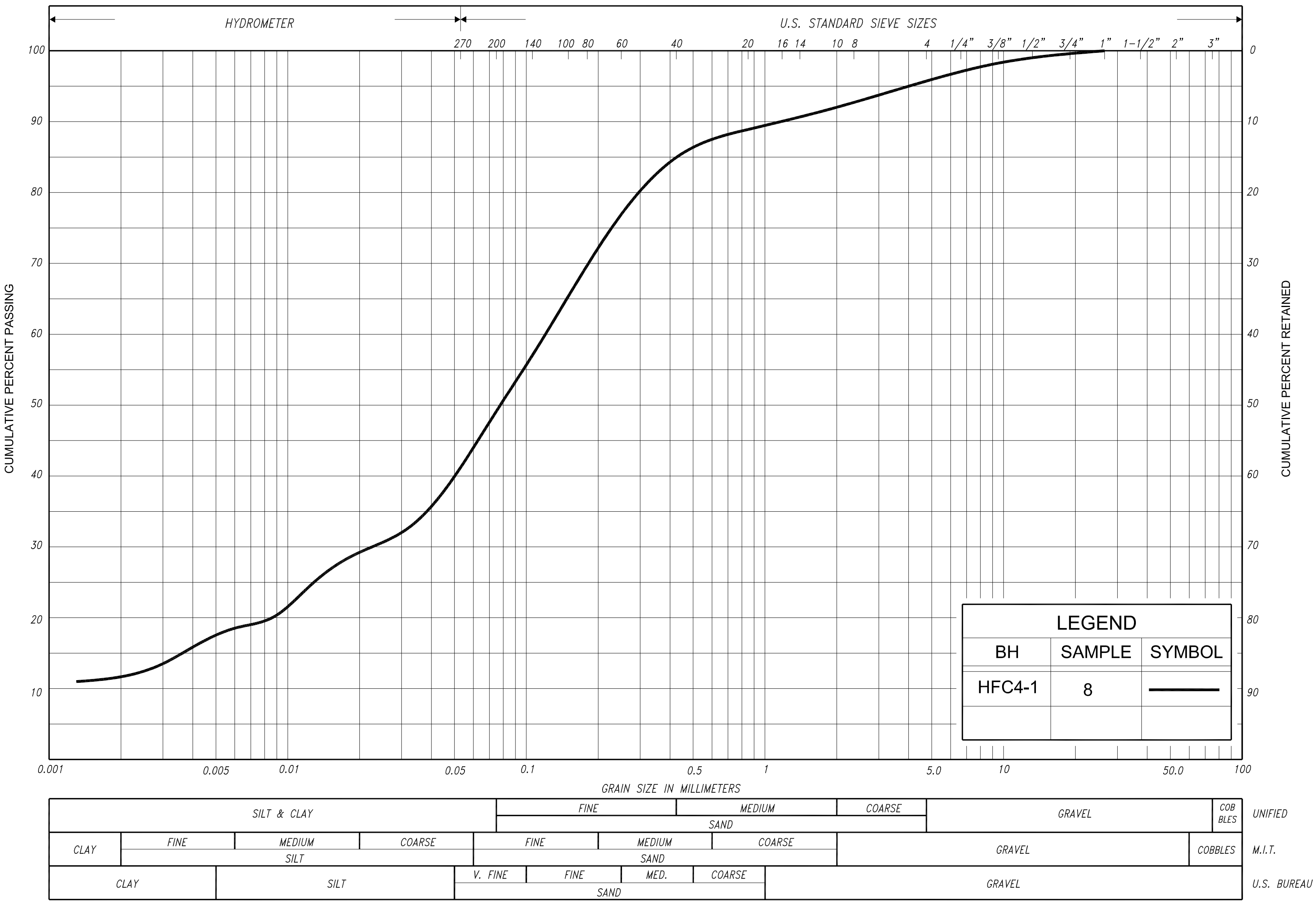
GRAIN SIZE DISTRIBUTION SAND and SILT, trace to some clay (TILL)	FIG No.	M58-GS-3
	HWY:	407E
	W.P. No.	07-20016



Ontario / Peto MacCallum Ltd. CONSULTING ENGINEERS	**PLASTICITY CHART** CLAYEY SILT, some sand, trace gravel (TILL)	FIG No. HFC4-PC-1 HWY: 407E W.P. No. 07-20016



Ontario / Peto MacCallum Ltd. CONSULTING ENGINEERS	**GRAIN SIZE DISTRIBUTION** CLAYEY SILT, some sand, trace gravel (TILL)	FIG No. HFC4-GS-1 HWY: 407E W.P. No. 07-20016



Ontario

Peto MacCallum Ltd. CONSULTING ENGINEERS

GRAIN SIZE DISTRIBUTION

SILTY SAND, some clay, trace gravel
(TILL)

FIG No.	HFC4-GS-2
HWY:	407E
W.P. No.	07-20016

APPENDIX C

RECORD OF BOREHOLE SHEETS FROM GEOCRES REPORTS

Ministry of Transportation Ontario

RECORD OF BOREHOLE No CM20-2a

METRIC

G.W.P. W.O. 07-20016	LOCATION N 4 871 478.2 E 355 201.5 Oshawa Creek East		ORIGINATED BY LH
HWY 407	BOREHOLE TYPE Solid Stem Augers		COMPILED BY AN
DATUM Geodetic	DATE 2009.02.02 - 2009.02.02		CHECKED BY MEF

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
203.9 / 0.0	**TOPSOIL** Soft to Firm Brown	1	SS	3	
203.1 / 0.8	**SAND** and **GRAVEL**, trace silt, occasional cobbles and organics Compact to Dense Brown Wet	2	SS	16	
		3	SS	50	
201.6 / 2.3	Silty **SAND**, trace gravel, some clay Compact to Very Dense Brown Wet to Moist (TILL)	4	SS	15	
		5	SS	73	2 50 31 17
		6	SS	100/ 0.200	
		7	SS	100/ 0.100	1 45 34 20
196.2 / 7.7		8	SS	100/ 0.125	

ELEVATION SCALE: 203, 202, 201, 200, 199, 198, 197

END OF BOREHOLE AT 7.7m.
Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen.

WATER LEVEL READINGS:

DATE	DEPTH (m)	ELEV. (m)
2009.02.12	0.4	203.5

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH kPa: O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL X 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

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

Ministry of Transportation Ontario

RECORD OF BOREHOLE No CM20b-4

METRIC

G.W.P. W.O. 07-20016	LOCATION N 4 871 528.7 E 355 215.2 Oshawa Creek East		ORIGINATED BY LH
HWY 407	BOREHOLE TYPE Solid Stem Augers		COMPILED BY AN
DATUM Geodetic	DATE 2009.02.02 - 2009.02.02		CHECKED BY MEF

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
207.9 / 0.0	**TOPSOIL** Soft Brown	1	SS	4	
207.3 / 0.6	Silty **SAND**, some clay, trace gravel, occasional organics Compact to Very Dense Brown Wet to Moist (TILL)	2	SS	21	
		3	SS	46	2 50 30 18
		4	SS	100/ 0.150	
		5	SS	100/ 0.200	
	becoming grey				
		6	SS	150/ 0.125	1 49 30 20
		7	SS	100/ 0.150	
	occasional cobbles and boulders				
200.2 / 7.7		8	SS	100/ 0.075	

ELEVATION SCALE: 207, 206, 205, 204, 203, 202, 201

END OF BOREHOLE AT 7.7m.
BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG TO SURFACE.

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH kPa: O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL X LAB VANE: 40 80 120 160 200
PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L; WATER CONTENT (%): 20 40 60
UNIT WEIGHT γ kN/m³

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

Ministry of Transportation Ontario

RECORD OF BOREHOLE No CM20c-3

METRIC

G.W.P. W.O. 07-20016	LOCATION N 4 871 343.4 E 355 171.6 Oshawa Creek East	ORIGINATED BY LH
HWY 407	BOREHOLE TYPE Solid Stem Augers	COMPILED BY ES
DATUM Geodetic	DATE 2009.01.30 - 2009.01.30	CHECKED BY MEF

SOIL PROFILE | SAMPLES | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) | SHEAR STRENGTH kPa (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, X LAB VANE; 40 80 120 160 200) | 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

ELEV DEPTH	DESCRIPTION
203.7	
0.0	TOPSOIL Brown
202.5 1.2	Clayey SILT, trace to some sand, trace gravel, trace oxide staining, occasional cobbles Stiff Brown
201.4 2.3	Sandy SILT, trace gravel, trace oxide staining Compact Brown Wet
199.4 4.3	SAND and SILT, some clay, trace gravel, occasional cobbles Very dense Brown Moist (TILL)
196.0 7.7	END OF BOREHOLE AT 7.7m. Piezometer installation consists of 25mm diameter schedule 40 PVC pipe with a 1.52m slotted screen.

NUMBER	TYPE	"N" VALUES	Remarks (GR SA SI CL)
1	SS	6	
2	SS	8	
3	SS	11	
4	SS	15	
5	SS	29	
6	SS	100/ .125	
7	SS	100/ .075	4 44 38 16
8	SS	100/ .100	

Elevation scale: 203, 202, 201, 200, 199, 198, 197

WATER LEVEL READINGS:

DATE	DEPTH (m)	ELEV. (m)
2009.02.12	0.4	203.3
2009.05.04	1.3	202.4
2009.06.06	1.3	202.4

+³, ×³: Numbers refer to Sensitivity

20, 15 ⊕ 5, 10 (%) STRAIN AT FAILURE