



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

Submitted to:

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

GEOCRES NO: 30M15-111

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

February 2011

PML Ref.: 10TF023-C
Index No.: 066FIDR

TABLE OF CONTENTS

SECTION

EXECUTIVE SUMMARY

PART A - PRELIMINARY FOUNDATION INVESTIGATION REPORT

1.0 INTRODUCTION.....

2.0 PROJECT DESCRIPTION.....

3.0 INVESTIGATION PROCEDURES

4.0 SITE GEOLOGY AND STRATIGRAPHY

4.1 Regional Geology

4.2 Site-Specific Descriptions and Subsurface Conditions.....

4.3 General Groundwater Conditions

5.0 CLOSURE.....

PART B - PRELIMINARY FOUNDATION DESIGN REPORT

6.0 ENGINEERING RECOMMENDATIONS FOR PRELIMINARY DESIGN

6.1 General.....

6.2 Structure Foundation Recommendations.....

6.2.1 Spread Footings

6.2.2 Steel H-Piles.....

6.2.3 Caissons

6.3 Abutment and Retaining Walls.....

6.4 Lateral Earth Pressures for Design

6.5 Structure Approaches.....

6.5.1 Subgrade Preparation and Embankment Construction

6.5.2 Approach Embankment Stability.....

6.5.3 Approach Embankment Settlement

6.6 Seismic Considerations.....

6.7 Construction Considerations.....

6.7.1 Obstructions During Pile Driving / Caisson Installation

6.7.2 Excavation and Backfill.....

6.7.3 Groundwater and Surface Water Control

6.7.4 Protection Systems.....

6.7.5 Construction Access.....

PAGE

i

1

1

1

2

2

3

4

5

6

6

6

7

7

8

8

9

9

9

10

10

10

10

11

11

11

11

11

7.0 CULVERTS

8.0 DEEP CUTS AND HIGH FILLS

8.1 General.....

8.2 Deep Cuts.....

8.2.1 Stability and Drainage.....

8.2.2 Construction Considerations

8.3 High Fills

8.3.1 Embankment Slope Stability.....

8.3.2 Settlement

8.3.3 Construction Considerations

9.0 CLOSURE.....

11

12

12

12

12

12

13

13

13

13

14

References

Explanation of Terms Used in Report

Preliminary Foundation Investigation and Design Report (FIDR) Sheets

Preliminary Foundation Investigation Report for Deep Cut DC-C8

Preliminary Foundation Investigation Report for High Fill HF-C4 (to be drilled)

Figure 1 – Abutment on Compacted Fill Showing Granular A Core

Drawings

Appendices

LIST OF DRAWINGS

Drawing 1 – Project Location Plan

Drawing 2 – Key Location Plan

Drawing 3 – Borehole Location – Central Mainline (West of Ashburn Road to Thickson Road)

Drawing 4 – Borehole Location – Central Mainline (Thickson Road to West of Simcoe Street)

Drawing 5 – Borehole Location – Central Mainline (Simcoe Street to Wilson Road)

LIST OF APPENDICES

Appendix A Record of Borehole Sheets

Appendix B Laboratory Test Results

Appendix C Record of Borehole Sheets from GEOCRES Reports



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.

Peto MacCallum Ltd. (PML) prepared this report to supplement Thurber’s preliminary report on the west part of the Central Section referenced above.

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 attempted at each of the four sites (two structures, one high fill and one deep cut); however, the target level could not be achieved at one structure site (culvert M-51) due to restricted access to private properties (no permission to enter).

For each of the accessed sites where borehole information was obtained at or near the site, an individual Preliminary Foundation Investigation and Design Report (FIDR) was prepared. Each FIDR consisted 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 deep cut and high fill sections (depth/height greater than 4.5 m), summary tables have been included that summarize the deep cut and high fill locations, depths/heights, the anticipated subsurface conditions, and preliminary geotechnical recommendations. Foundation investigations were completed at the deep cut section (DC-C8). The remaining high fill section (HF-C4) was not investigated due to lack of permission to enter.

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, or to desktop study level information. Where drilling was carried out, the boreholes were not necessarily drilled at or within the footprint of the foundation elements. As well, investigation was not possible at one of the structure sites (culvert M-51) due to lack of permission to enter. 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

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

1.0 INTRODUCTION

This report presents the factual findings obtained from a preliminary foundation investigation carried out by Peto MacCallum Ltd. (PML) in December 2010 to supplement 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 Simcoe Street North in Oshawa, Ontario (refer to Drawing 1).

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 One 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.

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 there are a total of 12 structure sites, where the highway crosses roads or watercourses. These consist of 8 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 site of bridge M-48 (former CM-10/10b), with boreholes featuring a prefix 'M48' and a sequential number.

In addition to the grade separation, bridge and culvert structures, this report also addresses deep cuts or high fills along the proposed alignment. These are defined as sections where the depth of cut or height of fill exceeds 4.5 m. The deep cut and high fill sections are summarized 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.

It was not possible to drill all of the proposed boreholes due to lack of permission to enter (PTE) private properties to access the borehole locations.

The subsurface investigation by PML was conducted in December 2010 and involved a total of 7 boreholes (4 for structure site M-48 and 3 for deep cut section DC-C8) drilled to depths of 6.6 to 14.2 m. Selected borehole data from Thurber's investigation was also used for this report. The borehole locations are shown on Drawings 3 and 4 relative to the proposed highway alignment and structure locations provided by AECOM. The locations of culvert M-51 and high fill HF-C4 yet to be investigated are indicated on Drawing 5.

PML established borehole locations in the field and J.D. Barnes 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. and Eastern Soil Drilling. 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 boreholes drilled for the deep cut sections were advanced to depths of 1.5 times the depth of the cut, and the boreholes for high fill sections were advanced to depths equal to the height of the fill or to competent material. The total depth of the boreholes ranged from 7.7 m to 34.1 m below the existing ground surface.

The groundwater conditions in the open boreholes were observed throughout the drilling operations. At each structure site and deep cut section where boreholes were drilled (except DC-C8), 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 19 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, 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. underpass, overpass 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 (for example, Lynde Creek crossing Site CM-3/3b). 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. In the case of any structure sites that were deleted after boreholes had been drilled (for example, CM-9), the FIR and FDR sheets have been included for information purposes.

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 *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.

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-42	CM-2	–	Underpass	Baldwin Street	Medium	CM2-1, P13 ¹	–	Refer to FIDR sheet ³ in Thurber report
M-43	CM-3 / 3b	CM-LC-24	Overpass	Lynde Creek	Medium	CM3-1, CM3-2, CM3b-1, CM3b-2	–	Refer to FIDR sheet in Thurber report
M-44	CM-4	CM-TBLC-25	Culvert	Lynde Creek Tributary	Medium	CM4-1, CM4-2	–	Refer to FIDR sheet ³ in Thurber report
M-45	CM-5	–	Underpass	Anderson Street	Medium	CM5-1, CM5-2	–	Refer to FIDR sheet ³ in Thurber report
M-46	CM-6 / 6b	–	Underpass	Thickson Road	Medium	CM6-1, CM6-2, CM6b-1, CM6b-2	–	Refer to FIDR sheet in Thurber report
M-47	CM-7	CM-TBPC-27	Culvert	Pringle Creek Tributary	Medium	CM7-1, CM7-2	–	Refer to FIDR sheet ³ in Thurber report
Deleted	CM-9	–	Underpass	Garrard Road	Medium	CM9-1/1a, CM9-2/2a, P15 ¹	–	Refer to FIDR sheet – Structure deleted
M-48	CM-10 / 10b	CM-OCE-28	Overpass	Oshawa Creek West Branch	High	CM10-1, CM10-2, CM10b-1, CM10b-2	M48-1, M48-2, M48-3, M48-4	Refer to FIDR sheet – Supersedes previous FIDR by Thurber
M-49	CM-11	–	Overpass	Thornton Road	Medium	CM11-1, CM11-2, P21 ²	–	Refer to FIDR sheet ³ in Thurber report
M-50	CM-12 / 12b	–	Overpass	Winchester Road West	Medium	CM12-1, CM12-2, CM12b-1, CM12b-2	–	Refer to FIDR sheet ³ in Thurber report
M-51	CM-13	CM-TAOCW-32	Culvert	Oshawa Creek West Branch East Tributary (Mainline)	Medium	–	To be drilled	No PTE – Refer to copy of AFC sheet from Desktop Study ³ in Thurber report
M-52	CM-15E	CM-TAOCW-33	Culvert	Oshawa Creek West Branch East Tributary at Simcoe Street	Medium	CM15E-1, CM15E-2	–	Refer to FIDR sheet in Thurber report
M-53	CM-14	–	Underpass	Simcoe Street	Medium	CM14-1, CM14-2, P22 ²	–	Refer to FIDR sheet ³ in Thurber report

¹ MTO Geocres No. 30M14-227
² MTO Geocres No. 30M15-85
³ Structure category, configuration or level of investigative effort changed since Desktop Study

Table 2 summarizes the sections where the proposed highway is to be constructed in a deep cut or as a high fill. The table shows the cut (DC) or fill (HF) number, locations (station to station), maximum cut depth or fill height, and the boreholes advanced during the investigations. At 4 deep cut and high fill sections, it was not possible to drill any boreholes due to lack of PTE private properties. Wherever possible, borehole information from the adjacent structures has been used to provide recommendations.



The subsurface conditions at the deep cut and high fill sections are summarized in the Preliminary Foundation Investigation Report “Deep Cuts” and “High Fills” tables following the FIDR sheets for the structures. Where relevant borehole information was not available within reasonable distance from the cut/fill section, the Terrain/Drainage Maps (prepared by AECOM based on air-photo interpretation) provided in the Foundation Desktop Study and the Geologic Cross-Sections provided in the *Foundation Investigation Report For Environmental Assessment (Hydrogeology Specialty)* prepared by AECOM were used to interpret anticipated subsurface conditions.

Table 2 – Deep Cut and High Fill Summary

Deep Cut (DC) or High Fill (HF) Number	Station (From – To)	Maximum Cut Depth or Fill Height	Boreholes by Thurber	Boreholes by PML
DC-C8	14+150 to 14+930 (new chainage) 11+200 to 11+980 (old chainage)	8.5	–	DCC8-1, DCC8-2, DCC8-3
HF-C1	12+680 to 12+750 (old chainage)	7	–	–
DC-C1	13+250 to 13+430 (old chainage)	8	CCM-1	–
DC-C2	14+490 to 14+660 (old chainage)	5.5	–	–
HF-C2	15+180 to 15+300 (old chainage)	15.5	–	–
HF-C3	15+500 to 16+000 (old chainage)	7	FCM-1, FCM-2	–
HF-C4	11+366 to 11+616 (new chainage) 16+750 to 17+000 (old chainage)	5.5	–	To be drilled

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 5.

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 19 borehole locations as part of the current and previous investigations for the project. Details of the piezometer installation and history of water levels measured in the boreholes are shown on the Record of Borehole sheets in Appendix A. Details of the site-specific groundwater conditions at each site are provided on the Preliminary FIR sheets, following the text of this report.

The groundwater levels measured in the piezometers generally range from 0.7 m to 10 m below ground surface, typically about 0.7 to 5.0 m below the ground surface. The most recent water levels measured in the piezometers are summarized in Table 3.

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
CM2-1	158.9	Piezometer Damaged	-	July 21, 2009
CM3-2	152.1	4.0	148.1	February 12, 2009
CM3b-1	152.5	4.7	147.8	February 12, 2009
CM4-1	156.0	1.3	154.7	June 6, 2009
CM5-1	161.1	6.0	155.1	July 21, 2009
CM6-2	167.0	5.8	161.2	February 12, 2009
CM6b-1	166.3	3.3	163.0	February 12, 2009
CM7-2	166.9	5.4	161.5	May 9, 2009
CM9-1	171.7	9.6	162.1	March 12, 2008
M48-2	155.0	0.7	154.3	January 14, 2011
CM10-1	161.9	4.8	157.1	March 12, 2009
CM10b-2	156.0	2.2	153.8	July 28, 2008
CM11-2	171.1	9.9	161.2	January 16, 2008
CM12b-1	172.2	2.2	170.0	June 6, 2009
CM12-2	173.3	1.2	172.1	May 4, 2009
CM14-1	186.8	2.5	184.3	May 4, 2009
CM15E-2	189.4	0.7	188.7	May 4, 2009
CCM1	166.4	1.6	164.8	February 12, 2009
FCM2	173.5	2.7	170.8	February 10, 2009



5.0 CLOSURE

The Preliminary Foundation Investigation Report was prepared by Mrs. Nesam Balakumaran, BSc, Engineer-in-Training, and 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.

Ns. Balala
Feb 28, 2011

Nesam Balakumaran, BSc
Engineer-in-Training



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

NB/GD/CN/BRG:mi



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



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

PART B

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

6.0 ENGINEERING RECOMMENDATIONS FOR PRELIMINARY DESIGN

6.1 General

This section of the 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. Preliminary geotechnical recommendations for the design of culverts are discussed in Section 7.0. Recommendations for deep cut and high fill sections 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, 12 bridge and grade separation structures are currently proposed for the Highway 407 central section mainline (west part). 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 and the remaining structure M-51 will be investigated when the permission to enter the property is granted. 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 $\pm 2\%$ 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 yet to be developed is included with the FIDR sheet for bridge M-48 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 DEEP CUTS AND HIGH FILLS

8.1 General

This section of the report provides geotechnical recommendations for preliminary design of deep cuts and high fill sections where the depth/height exceeds 4.5 m. Based on the roadway profiles available at the time of analysis (February 2009), deep cuts have been identified at three locations and high fills were identified at four locations. The locations and maximum depth/height are summarized in Table 2, Section 4.2. The maximum depth of cut is in the order of 8.5 m and the maximum fill height is about 15.5 m.

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. Where relevant borehole information was not available within reasonable distance from the cut/fill section, the Terrain/Drainage Maps (prepared by AECOM based on air photo interpretation) provided in the Foundation Desktop Study and the Geologic Cross-Sections provided in the *Foundation Investigation Report For Environmental Assessment (Hydrogeology Specialty)* prepared by AECOM were used to interpret anticipated subsurface conditions.

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

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

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 Deep Cuts

8.2.1 Stability and Drainage

Preliminary assessment of the stability of the cut 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. Cut slopes no steeper than 2H:1V, with a minimum 2 m wide mid-slope bench for cut depths greater than 8 m, were assumed.

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

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

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

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

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

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

8.2.2 Construction Considerations

Excavation for cut slope construction should be carried out in accordance with OPSS 206 as amended by the most recent Special Provision (SP 206S03).

Excavation in very dense/hard till deposits may be arduous and will require use of heavy duty excavators or dozers. In addition, tills often contain cobbles and boulders. The contract documents should include a NSSP to emphasize these conditions to the contractor. Selection of the method of

excavation must remain the responsibility of the contractor however and be based on his equipment, experience and interpretation of the site conditions.

Temporary drainage of the cuts should be provided to maintain a relatively dry, stable excavation. Measures may include temporary drainage ditches or gravel sheeting to maintain surficial stability before permanent drainage measures are in effect.

8.3 High Fills

8.3.1 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.

8.3.2 Settlement

Settlement of the fill embankments will occur due to compression and consolidation of the foundation soils under the weight of the overlying fill material as well as from compression of the embankment fill itself. The total settlement within the founding soils has been estimated using elastic analysis and Terzaghi one-dimensional consolidation theory, based on the 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.

8.3.3 Construction Considerations

It is recommended that all topsoil and organic material be stripped from the proposed embankment footprint. The depth and extent of stripped material 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 Preliminary Foundation Design Report was prepared by Mrs. Nesam Balakumaran, BSc, Engineer-in-Training, and 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.

Ns-Balakumaran
Feb 28, 2011



Nesam Balakumaran, BSc
Engineer-in-Training

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



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



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

NB/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 (RQD), 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

SS	SPLIT SPOON	TP	THINWALL PISTON
WS	WASH SAMPLE	OS	OSTERBERG SAMPLE
ST	SLOTTED TUBE SAMPLE	RC	ROCK CORE
BS	BLOCK SAMPLE	PH	TW ADVANCED HYDRAULICALLY
CS	CHUNK SAMPLE	PM	TW ADVANCED MANUALLY
TW	THINWALL OPEN	FS	FOIL SAMPLE
FV	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_r	1	SENSITIVITY = $\frac{c_u}{\tau_r}$

PHYSICAL PROPERTIES OF SOIL

ρ_s	kg/m ³	DENSITY OF SOLID PARTICLES	n	1, %	POROSITY	e_{max}	1, %	VOID RATIO IN LOOSEST STATE
γ_s	kN/m ³	UNIT WEIGHT OF SOLID PARTICLES	w	1, %	WATER CONTENT	e_{min}	1, %	VOID RATIO IN DENSEST STATE
ρ_w	kg/m ³	DENSITY OF WATER	S_r	%	DEGREE OF SATURATION	I_D	1	DENSITY INDEX = $\frac{e_{max} - e}{e_{max} - e_{min}}$
γ_w	kN/m ³	UNIT WEIGHT OF WATER	w_L	%	LIQUID LIMIT	D	mm	GRAIN DIAMETER
ρ	kg/m ³	DENSITY OF SOIL	w_p	%	PLASTIC LIMIT	D_n	mm	n PERCENT - DIAMETER
γ	kN/m ³	UNIT WEIGHT OF SOIL	w_s	%	SHRINKAGE LIMIT	C_u	1	UNIFORMITY COEFFICIENT
ρ_d	kg/m ³	DENSITY OF DRY SOIL	I_p	%	PLASTICITY INDEX = $w_L - w_p$	h	m	HYDRAULIC HEAD OR POTENTIAL
γ_d	kN/m ³	UNIT WEIGHT OF DRY SOIL	I_L	1	LIQUIDITY INDEX = $\frac{w - w_p}{I_p}$	q	m ³ /s	RATE OF DISCHARGE
ρ_{sat}	kg/m ³	DENSITY OF SATURATED SOIL	I_C	1	CONSISTENCY INDEX = $\frac{w_L - w}{I_p}$	v	m/s	DISCHARGE VELOCITY
γ_{sat}	kN/m ³	UNIT WEIGHT OF SATURATED SOIL				i	1	HYDRAULIC GRADIENT
ρ'	kg/m ³	DENSITY OF SUBMERGED SOIL	DTPL		DRIER THAN PLASTIC LIMIT	k	m/s	HYDRAULIC CONDUCTIVITY
γ'	kN/m ³	UNIT WEIGHT OF SUBMERGED SOIL	APL		ABOUT PLASTIC LIMIT	j	kN/m ²	SEEPAGE FORCE
e	1, %	VOID RATIO	WTPL		WETTER THAN PLASTIC LIMIT			

PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT (FIDR) SHEETS

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

Structure Description: Overpass Highway 407 Central Mainline / Oshawa Creek West Branch

Highway 407 Proposed Grade: ~El. 165.5 to 168.7 m

Site Ranking: High

Location No.: M-48 (CM-10/10b)

Existing Ground Elevation: ~El. 154.3 – 161.9 m

Station: ~18+030

FOUNDATION INVESTIGATION

Site Description:

The site of the proposed bridge M-48 is located approximately 300 m west of Thornton Road North over Oshawa Creek West Branch in the City of Oshawa. At this site, the creek flows in a 250 m wide valley consisting of a silty, gravelly sand alluvial plain. The relief is low plain and poorly to very poorly drained. Peat is mapped at the east side of the alluvial plain.

Borehole Information:

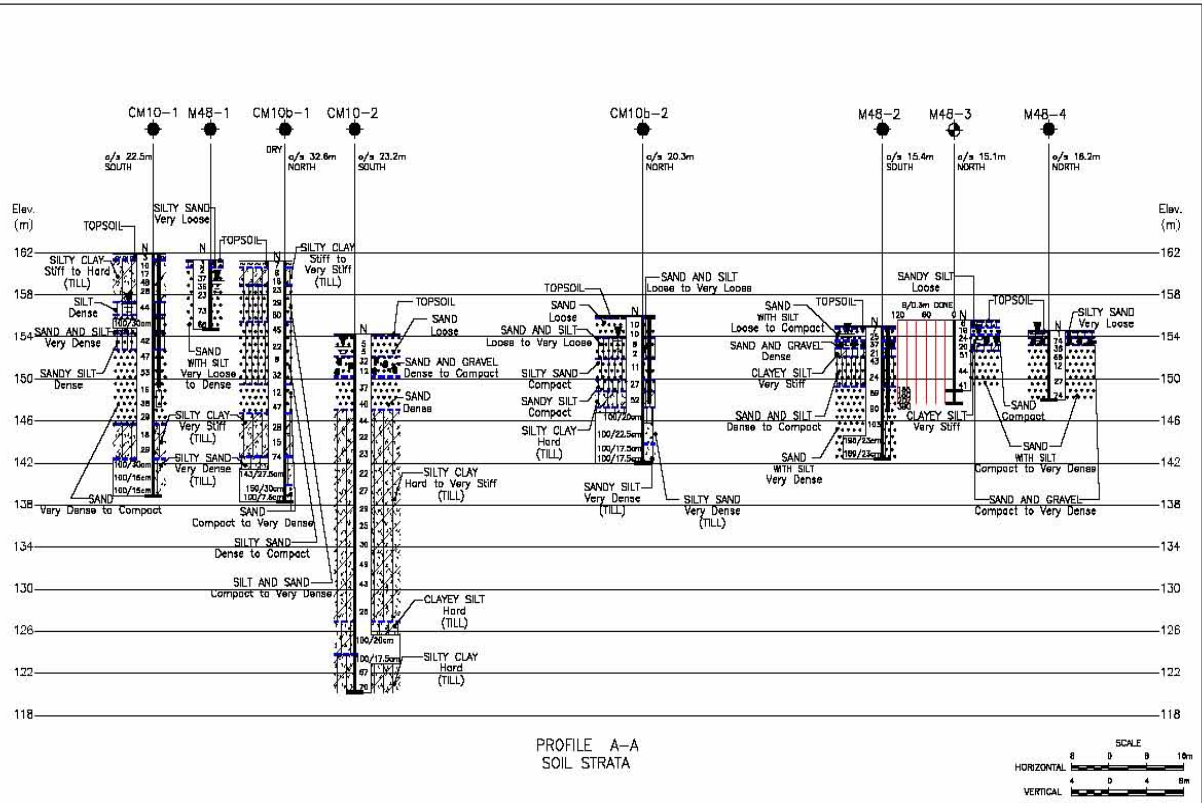
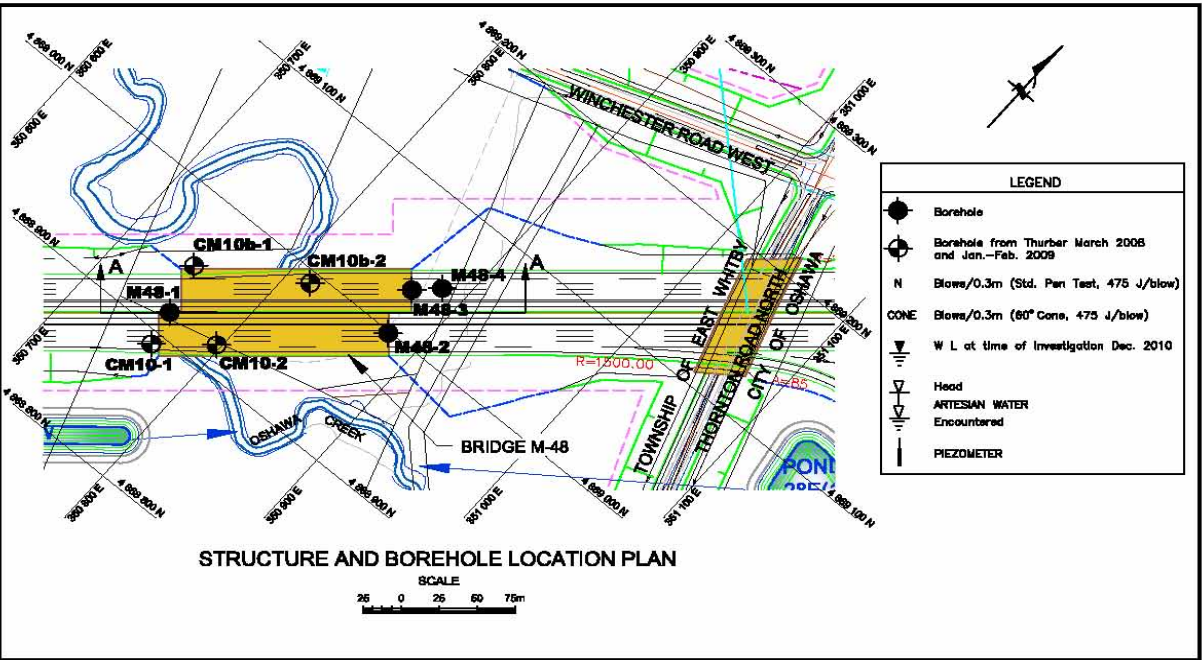
Borehole No.	Borehole Location	MTM NAD 83 - Northing	MTM NAD 83 - Easting	Borehole Elevation (m)	Borehole Depth (m)
M48-1	West Approach (eastbound)	4 868 916.0	350 758.4	161.3	6.6
M48-2	East Abutment (eastbound)	4 868 998.1	350 877.5	155.0	12.6
M48-3	East Abutment (westbound)	4 869 031.3	350 869.3	155.6	7.9
M48-4	East Approach (westbound)	4 869 045.2	350 884.0	154.6	6.6
CM10-1	West Abutment (eastbound)	4 868 891.2	350 763.6	161.9	23.0
CM10-2	Piers (eastbound)	4 868 918.7	350 796.7	154.3	34.1
CM10b-1	West Abutment (westbound)	4 868 951.4	350 749.1	161.2	22.9
CM10b-2	Piers (westbound)	4 868 991.9	350 815.3	156.0	14.0

Subsurface Conditions:

- Topsoil:** Between 100 and 600 mm of topsoil containing roots was encountered in all the boreholes. The thickness and extent of topsoil are expected to vary between and beyond the borehole locations, and the information in this report should not be used for quantity estimating purposes.
- Sands and Silts:** Layers of brown to grey sands and silts were encountered below the topsoil (silty clay till in boreholes CM10-1 and CM10b-1). The combined thickness of these strata varies between at least 6.5 and 12.4 m, with underside elevations ranging from 142.4 to 154.7 m where boreholes M48-2 and M48-1 were terminated. These cohesionless soils are typically loose becoming compact to very dense with depth ('N' values ranging between 1 and 195 blows/0.23 m penetration). Figures M48-GS-2, M48-GS-3, CM10/10b-B1 to -B3 present grain size distribution curves for samples of these soils. Measured moisture contents of these soils varied between 8 and 32 percent.
- Sand and Gravel:** A layer of sand and gravel was found interlayered within the sands and silts in boreholes M48-2 to M48-4 and CM10-2. This layer is 0.7 to 1.9 m thick and its underside is at Elev. 150.2 to 153.2 m. This cohesionless soil is compact to very dense ('N' values of 12 to 74 blows/0.3m penetration). Figures M48-GS-1 and CM10/10b-B4 present grain size distribution curves for two samples of this soil. Measured moisture contents of this soil were at 8 to 17 percent.
- Clayey Silt:** A cohesive deposit of clayey silt was identified within the stratum of sands and silts at depths of 2.1 and 2.4 m (Elev. 152.9 and 153.2 m) in boreholes M48-2 and M48-3 respectively. This deposit was very stiff in consistency (based on 'N' values of 20 and 21) and had a moisture content of 18 to 21 percent. The clayey silt had a thickness of 800 mm in the former borehole and 500 mm in the latter and was penetrated at 2.9 m depth (Elev. 152.1 and 152.7 m).
- Silty Clay Till:** A deposit of grey silty clay till, trace to some sand and trace gravel was encountered below the topsoil or surficial sands and silts in boreholes CM10-1, CM10-2, CM10b-1 and CM10b-2. This till ranges between 1.5 and 23.8 m in thickness, with undersides ranging between Elev. 147.3 m in borehole CM10b-2 to Elev. 120.2 m where borehole CM10-2 was terminated. This till has a stiff to hard consistency ('N' values ranging from 8 to 70 blows/0.3m penetration with occasional 'N' value of >100 for <0.3m penetration). Glacial tills typically contain cobbles and boulders. Figures CM10/10b-B5 to -B8 present laboratory test results for samples of this till. The Atterberg limits results indicate that it has a low to medium plasticity (LL = 24 to 37 and PI = 11 to 20). Measured moisture contents of this soil ranged between 10 and 30 percent.
- Clayey Silt Till:** A deposit of clayey silt till, some sand was found interlayered with the silty clay till in borehole CM10-2. This layer is 3.1 m thick with an underside at Elev. 123.8 m. This till has a hard consistency ('N' value >100 for <0.3m penetration). Glacial tills typically contain cobbles and boulders. Figure CM10/10b-B9 presents the grain size distribution of a sample of the clayey silt till. A moisture content of about 12 percent was measured.
- Silty Sand/Sandy Silt Till:** Silty sand and sandy silt till containing some clay and trace gravel was encountered below the silty clay till in boreholes CM10-1, CM10b-1 and CM10b-2. The combined thickness of these till deposits is at least 2.7 m to greater than 5.3 m. The three boreholes were terminated within the till. The till is very dense throughout ('N' values >100 blows for <0.3m penetration). Glacial tills typically contain cobbles and boulders. Figure CM10/10b-B10 presents the grain size distribution curves of samples of silty sand till. Measured moisture contents of these soils ranged between 8 and 15 percent.

Groundwater Conditions:

- Boreholes M48-1 to M48-4:** In the process of augering, water was detected at depths of 0.7 to 2.1 m (Elev. 153.7 to 159.2 m). Groundwater was at depths of 0.3 to 2.0 m (Elev. 153.2 to 159.3 m) upon completion of drilling. The water level in a piezometer installed in borehole M48-2 was at 0.7 m depth (Elev. 154.3 m) on January 4 and 14, 2011.
- Borehole CM10-2:** 1.2 m depth (Elev. 153.1 m) in open borehole on March 12, 2008.
- Borehole CM10-1:** 4.8 m depth (Elev. 157.1 m) in open borehole on March 12, 2009.
- Borehole CM10b-2:** 2.2 m depth (Elev. 153.8 m) in piezometer on July 28, 2008.



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

LOCATION No: M-48 CM-10/10b

FOUNDATION RECOMMENDATIONS

Note: The site specific foundation recommendations are for planning purposes only. Refer to Section 6 of the Foundation Design Report for the project-wide foundation recommendations, design assumptions and limitations.

General : Based on the General Arrangement drawing prepared by AECOM in March 2009, the proposed twin bridges will carry Highway 407 over Oshawa Creek West Branch. The proposed twin bridges are four (4) span structures with a total length of 152 m and with approach embankments of 4 m high at the west abutment and up to 15 m high at the east abutment. The foundation options considered are listed below with advantages and disadvantages associated with each option.

<i>Foundation Option</i>	<i>Advantages</i>	<i>Disadvantages</i>
<i>Spread Footings founded on sands and silts with high groundwater table within floodplain</i>	- Conventional construction	- Variable density and very low bearing resistance for soils in floodplain; requires sub-excavation up to 5 to 6m depth to reach competent founding soils in floodplain - Extensive unwatering and protection (temporary shoring) systems will be required for footing construction - Scour protection is required for footings in floodplain
<i>Spread Footings perched on Granular A pads at both abutments</i>	- Lower cost than deep foundations - Minimize excavation requirements	- Variability of surficial soils in floodplain; sub-excavation of approximately 2 to 4 m depth may be required - Protection systems (temporary shoring) may be required for footing construction
<i>Steel H-Piles driven to very stiff to hard silty clay till or very dense sand and silt till</i>	- 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 native sands and silts at shallow depths to construct pile caps - Unwatering and protection (temporary shoring) systems may be required for pile cap construction
<i>Caissons founded within very stiff to hard silty clay till or very dense sand and silt till</i>	- Higher bearing resistance - Not so affected by surficial soil variability	- Higher cost than spread footings - Does not permit integral abutment design - Potential installation problems including side sloughing and base boiling associated with sands and silts and cohesionless till (westbound) - Need to dislodge and handle cobbles and boulders

A – Spread Footings Spread footings founded on native very loose to compact sands and silts below high groundwater table are not recommended. As such, footings are not a preferred foundation option at this site. Footings for perched abutments may be founded on a minimum 2 m thick compacted Granular A cores in accordance with current MTO practices. The preliminary design geotechnical resistances and founding levels are as follows:

Founding Stratum	Geotechnical Resistance		Foundation Level
	Factored ULS	SLS	
Compacted Granular A	900 kPa	350 kPa	Fill base at or below Elev. 159.5 m (west abutments) and 154.0 (east abutments)

B – Steel H-Piles

Steel H-piles driven to refusal within the very dense sand and silt till, or very stiff to hard silty clay till, may be used to provide foundation support. The preliminary design geotechnical resistances and tip elevations are as follows:

Pile	Axial Geotechnical Resistance		Downdrag Load	Anticipated Pile Tip Elevation
	Factored ULS	SLS		
HP 310 x 110	1600 kN	1400 kN	Not applicable	At or below Elev.140 m (west abutment, eastbound) At or below Elev.140 m (west abutment, westbound) At or below Elev.124 m (piers, eastbound) At or below Elev.145 m (piers, westbound) At or below Elev.144 m (east abutment, eastbound) At or below Elev.148 m (east abutment, westbound)

C – Caissons

In the creek valley, consideration may be given to using augered caissons socketted within the very stiff to hard silty clay till or very dense sand and silt till. The preliminary design geotechnical resistances and base elevations for

caissons extending 4 m into the hard ('N' >100 blows) silty clay till, or 4 m into the very dense sand and silt till, and developing resistance through shaft friction and a portion of end-bearing within the till are as follows:

Caisson Diameter	Axial Geotechnical Resistance		Downdrag Load (Factored ULS)	Highest Founding Level
	Factored ULS	SLS		
1.2 m	6,400 kN	5,000 kN	Not Applicable	Elev. 138 m (west abutment, eastbound) Elev. 138 m (west abutment, westbound) Elev. 122 m (piers, eastbound) Elev. 143 m (piers, westbound) Elev. 142 m (east abutment, eastbound) Elev. 145 m (east abutment, westbound)
1.5 m	9,000 kN	6,000 kN	Not Applicable	

Given the uncertainties associated with cleaning and inspection of the base, the extent and apparent lack of continuity of the cohesive silty clay till across the site and the high artesian pressures encountered, the above recommended values and founding levels must be reassessed during detail design.

Recommended Foundation Alternative

From a foundation engineering perspective, the recommended foundation alternative at this site is steel H-piles driven to the very stiff to hard clayey silt to silty clay till, or the very dense sand and silt till.

• **ABUTMENT TYPE**

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

• **APPROACHES**

Up to 15 m of fill (approximately 150 m in length) will be required to construct the highway mainline east approach, while up to 4 m of fill will be required for the west approach.

Stability

For the west approach, fill embankments up to 4 m in height are anticipated to be stable at side slope inclinations of 2H : IV using SSM or better material. For the east abutment, approach embankments up to 15 m in height, in conjunction with two 2 m wide mid-height berms, are anticipated to be stable at side slope inclinations of 2H : I V using SSM or better material.

Settlement

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 will not exceed 25 and 75 mm at the west and east approaches, respectively.

• **CONSTRUCTION CONSIDERATIONS**

Pile Installation

During pile installation through glacially derived soils at this site, there is a medium probability of encountering cobbles or boulders. Driving shoes should be fitted to the pile tips for reinforcement and enhancing seating of the piles.

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 : IV 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.

Groundwater/Surface Water Control

The groundwater table is near the floodplain grade. Prior to excavations 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 should be used to supplement the above systems. The required groundwater control systems should be further assessed during detail design.

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. One possible system at the floodplain level is an interocking 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.

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 geosynthetics should be considered.

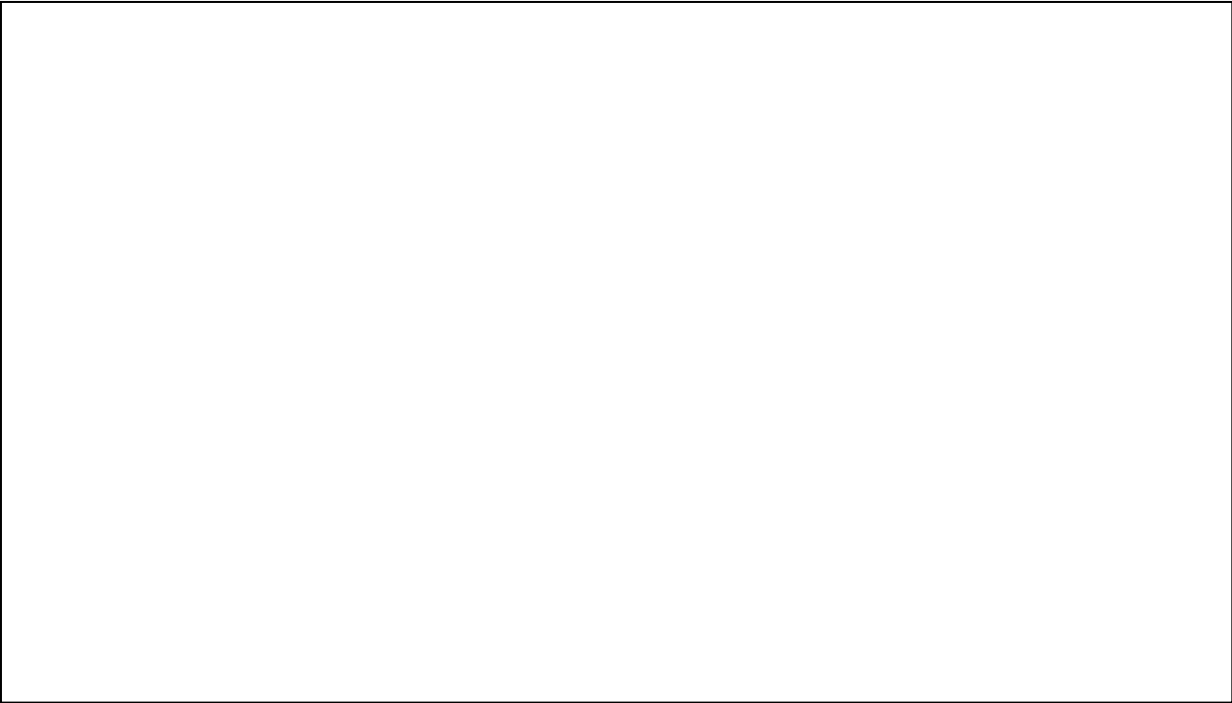
• **RECOMMENDATIONS FOR ADDITIONAL WORK**

Further subsurface investigation, analysis and design should be carried out during detail design to confirm the subsoil conditions at the locations of the bridge foundation elements. As a minimum, this is likely to require additional boreholes at the actual abutment and pier locations and at the approaches. The feasibility and cost effectiveness of alternate unwatering systems would need to be investigated. Should caissons be adopted for foundation support in the creek valley, additional deeper boreholes will need to be drilled to obtain additional subsurface and groundwater information to reassess the caisson design and installation techniques.

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

Structure Description: Culvert at Highway 407 and associated ramps over Oshawa Creek West
Location No.: M-51 (CM-13)

Highway 407 Proposed Grade: ~El. 186 m
Existing Ground Elevation: 179 to 180 m
Site Ranking: Medium
Station: 11+508



TO BE INVESTIGATED

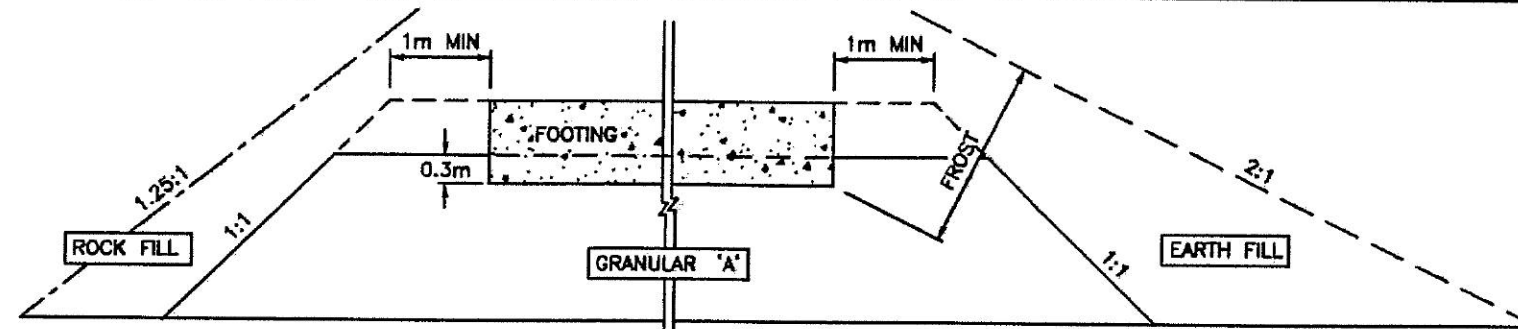
PRELIMINARY FOUNDATION INVESTIGATION REPORT – DEEP CUTS AND HIGH FILLS

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

Deep Cut No.	Station (From – To)	Proposed Highway Grade (m)	Maximum Cut Depth (m)	Reference Data	Subsurface Conditions	Preliminary Recommendations
Hwy 407	Central Mainline					
DC-C8	14+150 to 14+930	155 to 159	8.5	DCC8-1, DCC8-2, DCC8-3, P13, Hydrogeology Report	<p>Stratigraphy: Topsoil and sandy silt with organic inclusions (up to 1.4 m) over clayey soils to 2.2 m depth in two boreholes overlain by compact to very dense silty sand till to sandy silt till (and silt with sand in one borehole) underlain by sand at depths of 7.1 to 11.9 m (Elev. 154.0 to 156.0 m).</p> <p>Groundwater: Estimated near 2 to 3 m depth (Elev. 156.0 to 162.0 m). Borehole DCC8-1 – depth of 10.7 m (Elev. 155.2 m) upon completion of drilling. Borehole DCC8-2 – depths of 6.1 and 5.5 m (Elev. 158.5 and 159.1 m) during and upon completion of drilling, respectively. Borehole DCC8-3 – depths of 1.5 and 1.2 m (Elev. 161.0 and 161.3 m) during and upon completion of drilling, respectively.</p>	<p>Design Slope Inclination: Cut slopes up to 8.5 m high may be constructed at 2H : 1 V with a 2 m wide mid height bench on slopes exceeding 8 m in height.</p> <p>Drainage: Groundwater seepage should be anticipated in the granular soils below the groundwater table, especially from the sand encountered at approximate Elev. 154 to 156 m in boreholes DCC8-1 to DCC8-3. Depending on actual subsoil conditions and groundwater conditions, dewatering measures such as gravity drained ‘pilot trenches’ may be required prior to subexcavation to control groundwater and improve stability. Side ditches should be adequate for surface drainage.</p> <p>Surficial Instability: Gravel sheeting or alternative methods may be required to control surficial erosion and instability at areas of localized seepage.</p> <p>Recommendations for Further Investigation: Subsurface investigation should be carried out to confirm the subsoil conditions and groundwater levels at the location of the cut section.</p>

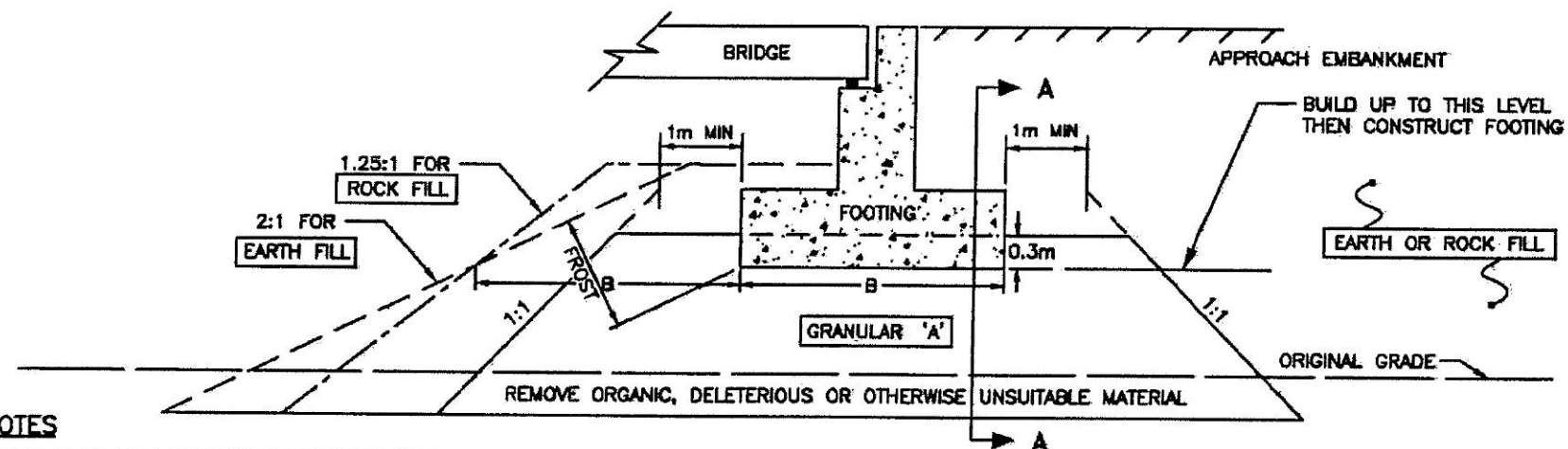
PRELIMINARY FOUNDATION INVESTIGATION REPORT
HIGH FILLS
HWY 407 EAST EXTENSION - CENTRAL SECTION (WEST PART)
W.O.07 - 20016

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	Desktop Study, Hydrogeology Report (1 borehole to be drilled)	Stratigraphy: Clayey silt till ground moraine overlaying sand and silt till. Groundwater: Estimated near 2 to 3 m depth (Elev. 177 m).	Design Slope Inclination: Fill slopes up to 5.5 m high may be constructed at 2H : 1 V. Stability: No stability issues are anticipated. Settlement: No settlement issues are anticipated. Recommendations for Further Investigation: Boreholes should be advanced to confirm the stratigraphy within the fill section.



CROSS SECTION A-A

NOT TO SCALE



LONGITUDINAL SECTION

NOT TO SCALE

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

HIGHWAY 407 EAST EXTENSION
CENTRAL SECTION
PROJECT LOCATION

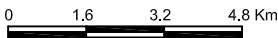


SHEET



PLAN

SCALE APPROX.



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).

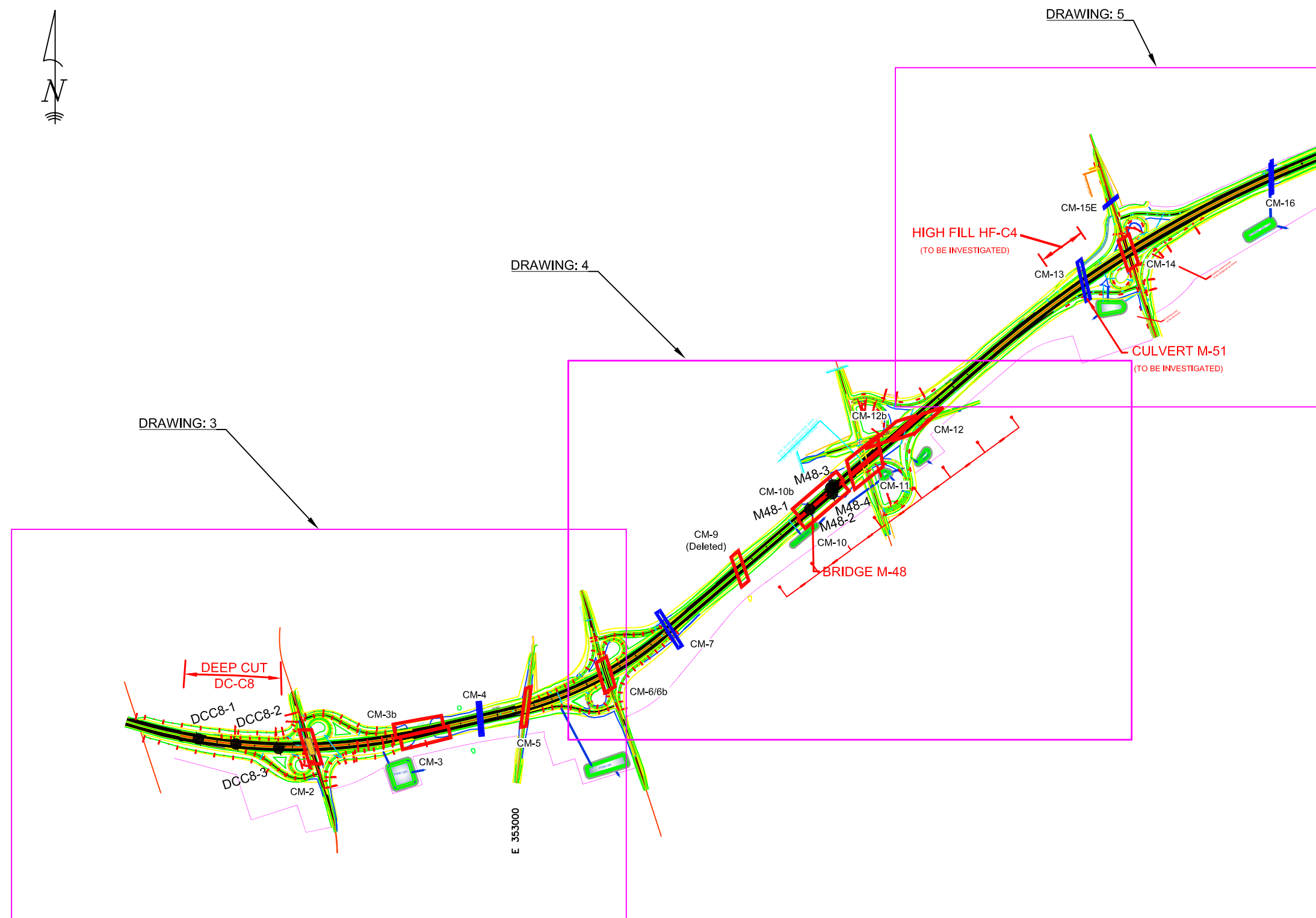
NO.	DATE	BY	REVISION
Geocres No. 30M15-111			
HWY. 407E		PROJECT NO. 10TF023	DIST. Central
SUBM'D. NA	CHKD. GD	DATE: Feb. 23, 201	SITE:
DRAWN: AL	CHKD. CN	APPD. BRG	DWG. 1

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

W.O. No. 07-20016

HIGHWAY 407 EAST EXTENSION
CENTRAL SECTION
KEY LOCATION PLAN

SHEET



PLAN

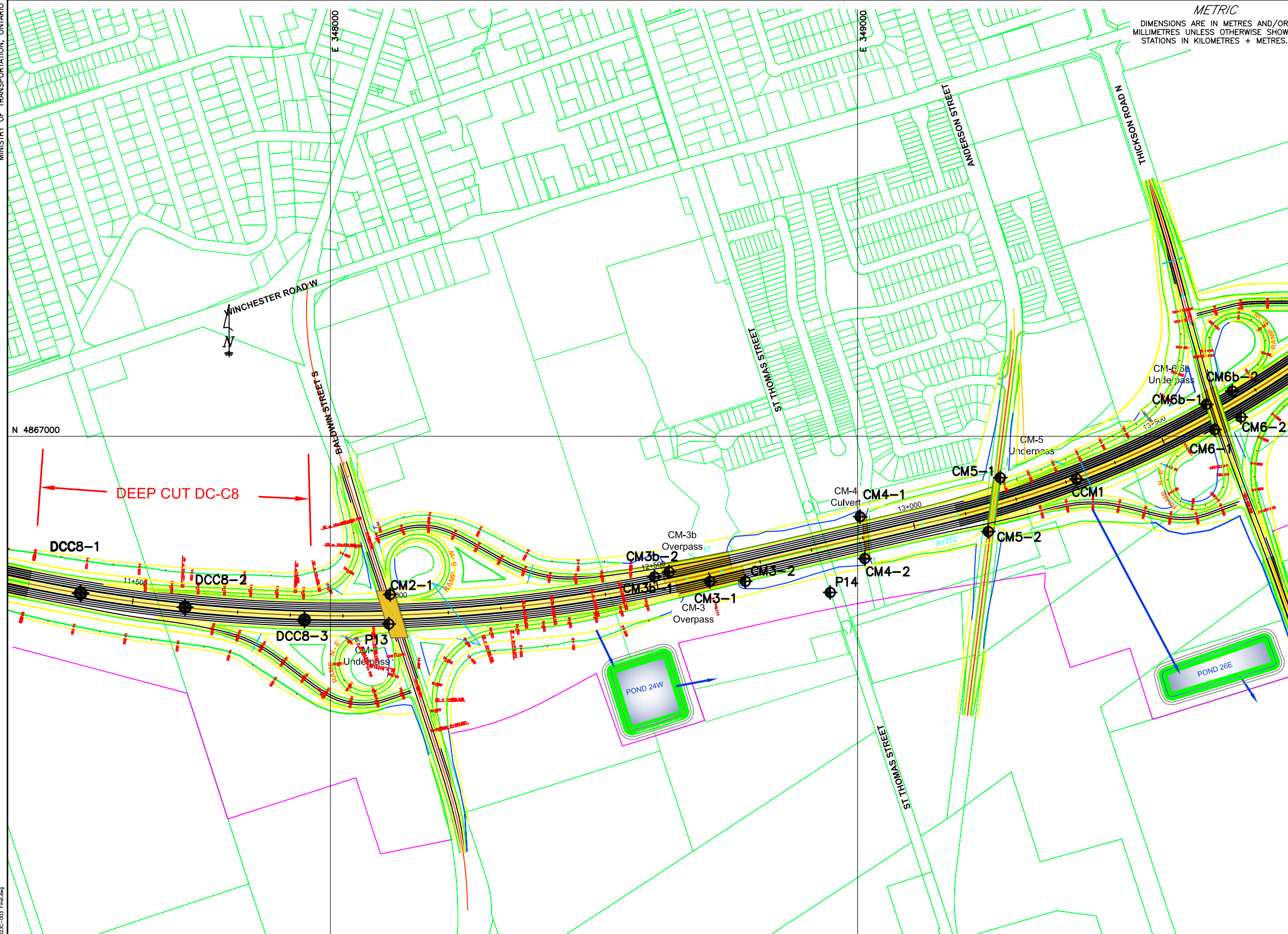
SCALE

125 0 250 500m

REFERENCE

Base plan and profiles provided in digital format by MT0, drawing file nos. "407E Central Section PRELIMINARY DESIGN_ULTIMATE.dwg", received October 23, 2010.

NO.	DATE	BY	REVISION
Geocres No. 30M15-111			
HWY. 407E		PROJECT NO. 10TF023	DIST. Central
SUBM'D. NA	CHKD. GD	DATE: Feb. 23, 201	SITE:
DRAWN: AL	CHKD. CN	APPD. BRG	DWG. 2

PLAN
SCALE

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

W.O. No. 07-20016

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



SHEET

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

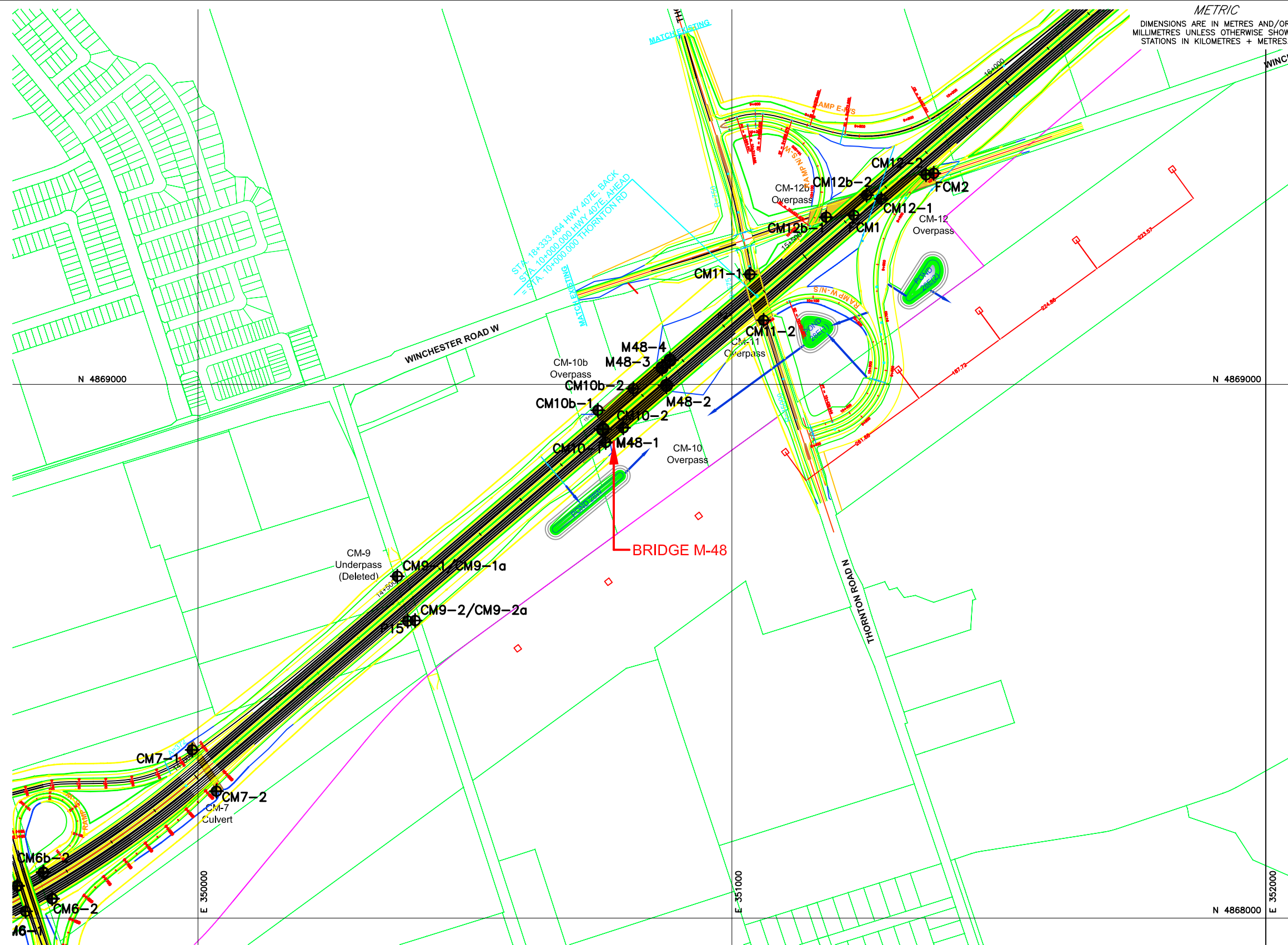
This drawing is for subsurface information only. The proposed structure details/works are shown for illustration purposes only and may not be consistent with the final design configuration as shown elsewhere in the Contracts Documents.

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
Geocres No. 30M15-111			
HWY. 407E		PROJECT NO. 10TF023	DIST. Central
SUBM'D. NA	CHKD. GD	DATE: Feb, 23, 201	SITE:
DRAWN: AL	CHKD. CN	APPD. BRG	DWG. 3

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

W.O. No. 07-20016

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

SHEET

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

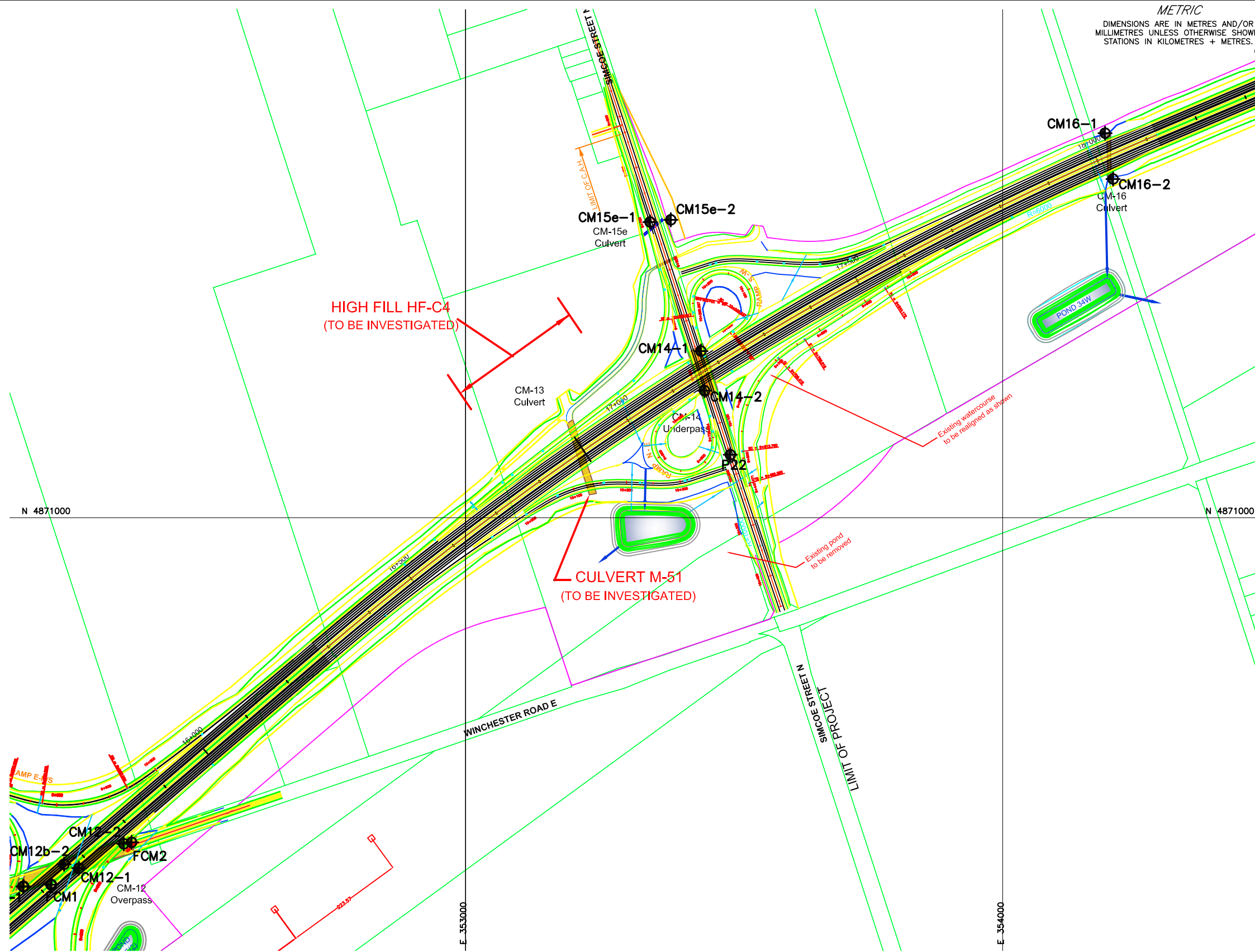
This drawing is for subsurface information only. The proposed structure details/works are shown for illustration purposes only and may not be consistent with the final design configuration as shown elsewhere in the Contracts Documents.

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. 30M15-111			
HWY. 407E		PROJECT NO. 10TF023	DIST. Central
SUBM'D. NA	CHKD. GD	DATE: Feb, 23, 201	SITE:
DRAWN: AL	CHKD. CN	APPD. BRG	DWG. 4



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

W.O. No. 07-20016

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

Peto MacCallum Ltd.
CONSULTING ENGINEERS

SHEET

LEGEND

Borehole - Current Investigation
 Borehole - MTO Geocres

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. 30M15-111			
HWY. 407E	PROJECT NO. 10TF023		DIST. Central
SUBM'D. NA	CHKD. GD	DATE: Feb, 23, 201	SITE:
DRAWN: AL	CHKD. CN	APPD. BRG	DWG. 5

APPENDIX A

RECORD OF BOREHOLE SHEETS

RECORD OF BOREHOLE No M48-1 1 of 1 METRIC

G.W.P. 07-20016 LOCATION Coords: 4 868 916.0 N; 350 758.4 E ORIGINATED BY D.S.
 DIST Central HWY 407E BOREHOLE TYPE Continuous Flight Solid Stem Augers COMPILED BY N.S.B.
 DATUM Geodetic DATE December 06, 2010 CHECKED BY G.D.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES		SHEAR STRENGTH kPa									
							20 40 60 80 100									
161.3	Ground Surface															
161.2	Topsoil		1	SS	3											
160.6	Very loose Brown Silty sand, with silt		2	SS	2											
0.7	Very loose Brown to dense		3	SS	37											

* 2010 12 06

▽ Water level observed during drilling

▽ Water level measured after drilling

RECORD OF BOREHOLE No M48-2 1 of 2 METRIC

G.W.P. 07-20016 LOCATION Coords: 4 868 998.1 N; 350 877.5 E ORIGINATED BY D.S.
 DIST Central HWY 407E BOREHOLE TYPE Continuous Flight Hollow / Solid Stem Augers + Casing COMPILED BY N.S.B.
 DATUM Geodetic DATE December 07 & 08, 2010 CHECKED BY G.D.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES		SHEAR STRENGTH kPa										WATER CONTENT (%)		
							○ UNCONFINED		+ FIELD VANE		● QUICK TRIAXIAL						× LAB VANE		
155.0	Ground Surface					20	40	60	80	100	20	40	60						
0.0	Topsoil																		
154.8	Sand, with silt		1	SS	7							○							
0.2	Loose to compact		2	SS	25							○							
153.6	Sand and gravel																		
1.4	trace silt, trace clay		3	SS	37							○			46 43 8 3				
152.9	Dense Brown Wet																		
2.1	Clayey silt, trace sand																		
152.1	Very stiff Brownish grey		4	SS	21							○							
2.9	Sand and silt, trace clay																		
	Dense to compact		5	SS	43							○			0 51 45 4				
	Brownish grey to wet																		
			6	SS	24							○							
149.3	Sand with silt, trace clay																		
5.7	Very dense Brownish grey		7	SS	89							○							
			8	SS	90							○							
	trace to some silt																		
	trace gravel		9	SS	103							○			3 86 (11)				
			10	SS	195/23cm														

* 2010 12 07 & 08

▽ Water level observed during drilling

▽ Water level measured after drilling

2 of 2

DATUM Geodetic DATE December 07 & 08, 2010 CHECKED BY G.D.

ON MTO_VER3 NEW LOGO 10TF023, M.GPJ ON_MOT.GDT 1/24/2011 3:52:50 PM

1 of 1

DATUM Geodetic DATE December 07, 2010 CHECKED BY G.D.

ON MTO_VER3 NEW LOGO 10TF023.MGPJ ON_MOT GDT 24/01/2011 11:09:30 AM
+ , X⁵ Numbers refer to 20
Sensitivity 15 5 (%) STRAIN AT FAILURE
10

G.W.P. 07-20016 LOCATION Coords: 4 869 045.2 N; 350 884.0 E ORIGINATED BY D.S.
DIST Central HWY 407E BOREHOLE TYPE Continuous Flight Solid Stem Augers COMPILED BY N.S.B.
DATUM Geodetic DATE December 08, 2010 CHECKED BY G.D.

ON MTO_VER3 NEW LOGO 10TF023. M.GPJ CN_MOT.GDT 03/02/2011 12:13:11 PM

G.W.P. 07-20015 LOCATION Coords: 4 867 702.4 N; 347 526.3 E ORIGINATED BY A.L.
DIST Central HWY 407E BOREHOLE TYPE Continuous Flight Solid Stem Augers COMPILED BY N.S.B.
DATUM Geodetic DATE December 10, 2010 CHECKED BY G.D.

ON MTO_VER3 NEW LOGO 10TF023. HF N DCGPJ.GPJ ON_MOT.GDT 31/01/2011 10:12:28 AM
 $\times 5$ Numbers refer to 20
 Sensitivity 15 5 (% STRAIN AT FAILURE)
 10

1 of 2

CHECKED BY G.D

ON MTO_VER3 NEW LOGO 10TF023 HF N DCGPJ.GPJ ON_MOT.GDT 31/01/2011 10:12:30 AM
 + , X⁵; Numbers refer to
 Sensitivity 15-0-5 (%) STRAIN AT FAILURE
 10

2 of 2

CHECKED BY G.D

ON MTO_VER3 NEW LOGO 10TF023, HF N DCGPJ.GPJ ON_MOT.GDT 31/01/2011 10:12:30 AM

1 of 2

ORIGINATED BY D.W.

COMPILED BY N.S.B.

CHECKED BY G.D

ON MTO, VER3 NEW LOGO 10TF023, HF NDCGPJ.GPJ ON MOT GDT 31/01/2011 10:12:32 AM

A crosshair with the number 20 at the top, 15 on the left, and 10 at the bottom.

2 of 2

ORIGINATED BY D.W.

COMPILED BY N.S.B.

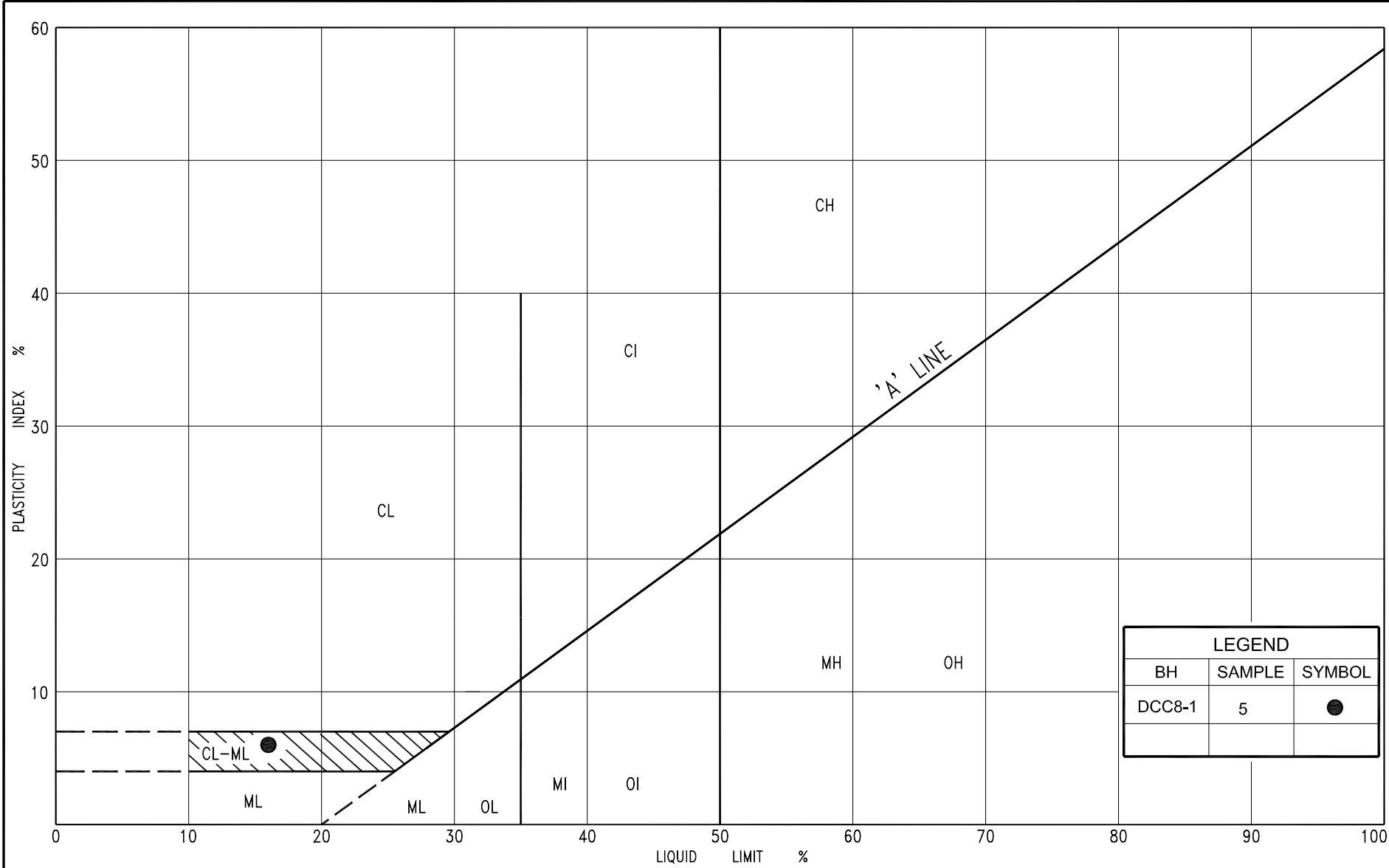
CHECKED BY G.D.



ON MTO_VER3 NEW LOGO 10TF023. HF NDCGPJ.GPJ ON MOT GDT 31/01/2011 10:12:32 AM

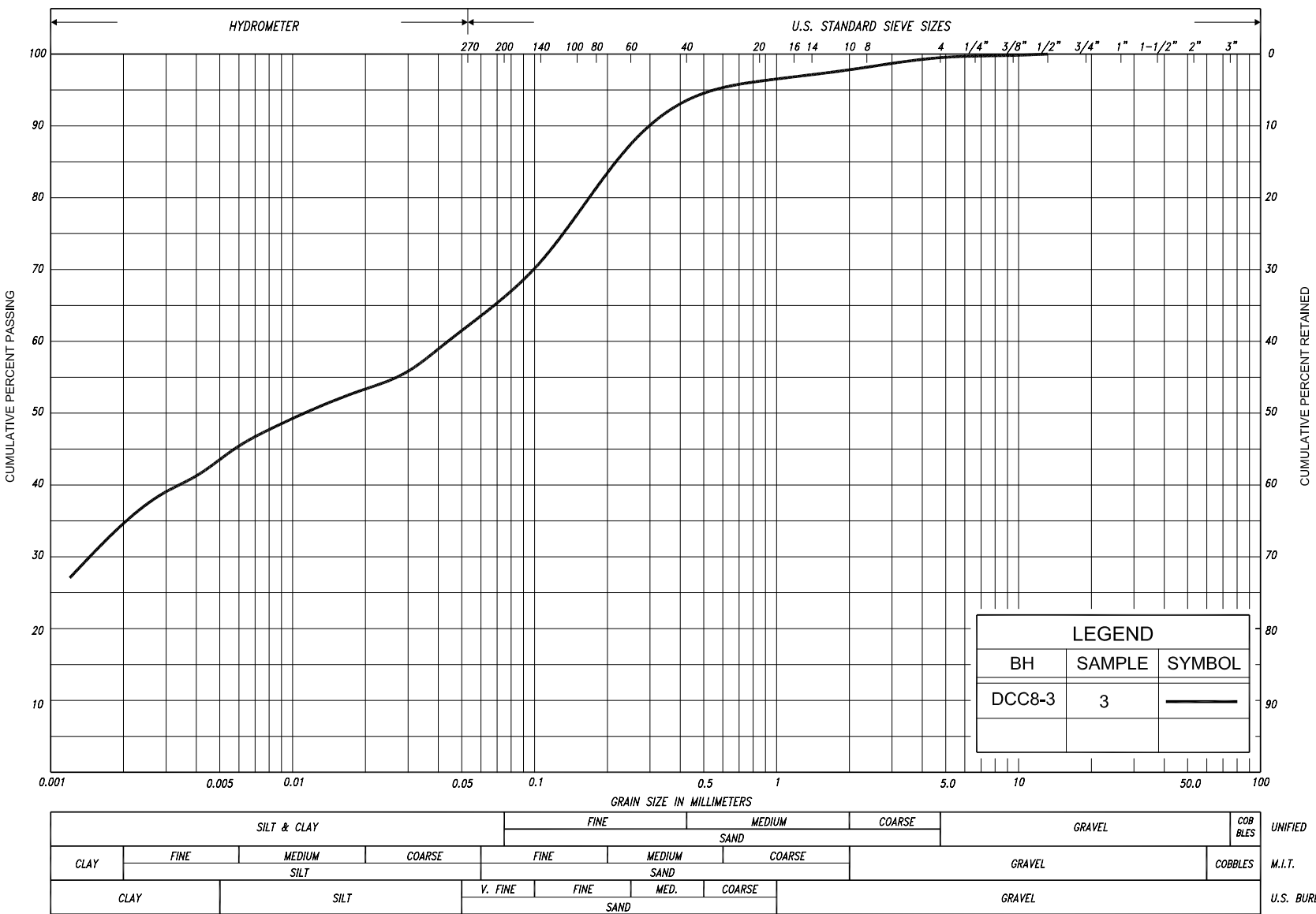
A vertical number line with tick marks at 10, 15, and 20. A circle is drawn around the number 15.

APPENDIX B



LABORATORY TEST RESULTS

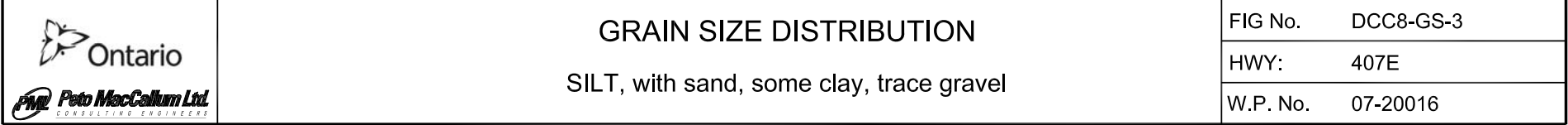
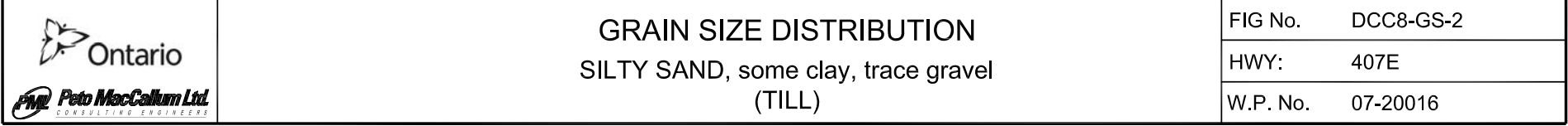


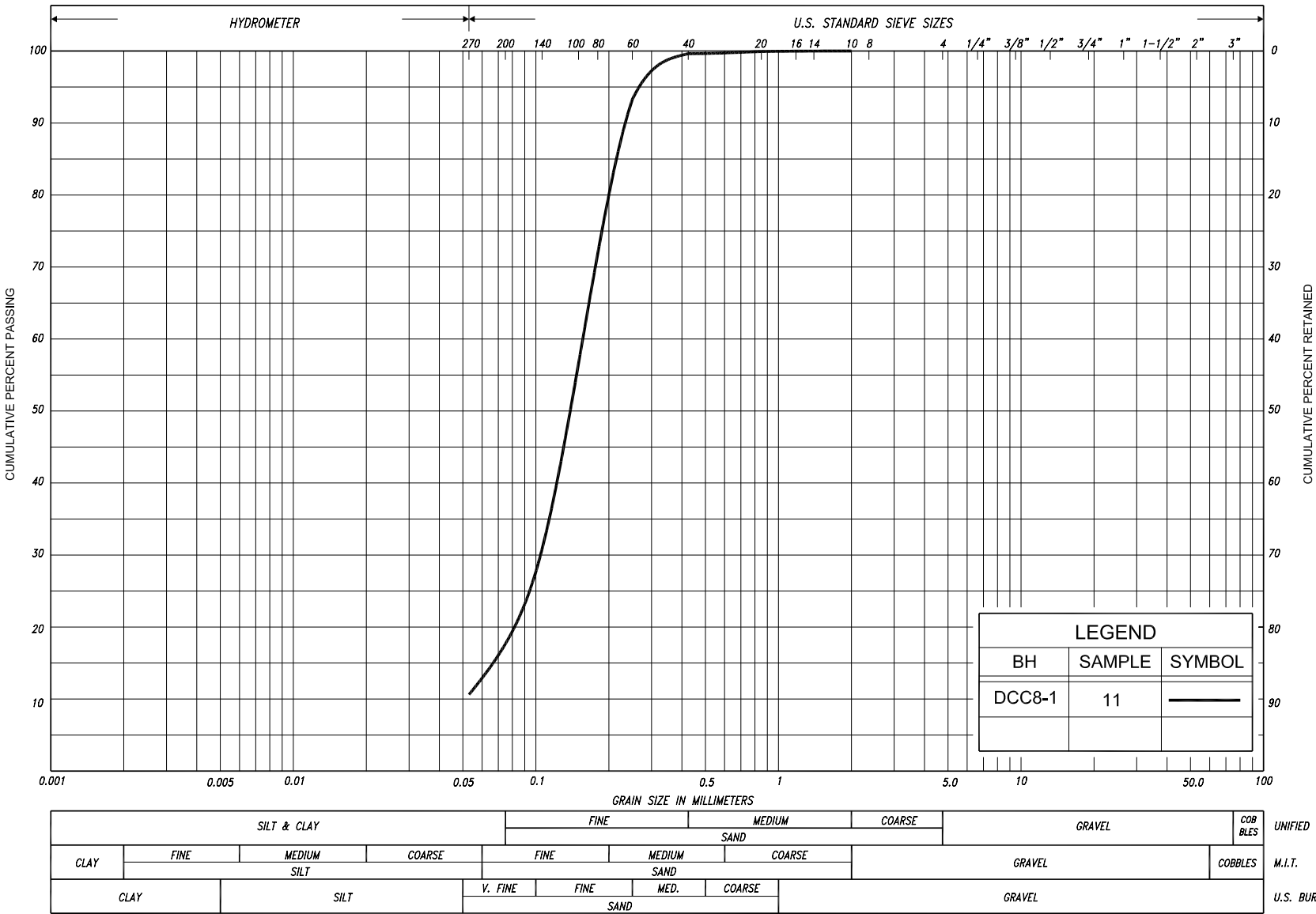
 	PLASTICITY CHART SILTY SAND, some clay, trace gravel (TILL)	FIG No. DCC8-PC-2
		HWY: 407E
		W.P. No. 07-20015



SILT & CLAY					FINE		MEDIUM		COARSE		GRAVEL			COB BLES	UNIFIED		
					SAND												
CLAY	FINE		MEDIUM		COARSE		FINE		MEDIUM		COARSE		GRAVEL			COBBLES	M.I.T.
	SILT						SAND										
CLAY			SILT			V. FINE	FINE	MED.	COARSE		GRAVEL						U.S. BUREAU
						SAND											

 	GRAIN SIZE DISTRIBUTION SILTY CLAY, sandy, trace gravel	FIG No. DCC8-GS-1
		HWY: 407E
		W.P. No. 07-20016





GRAIN SIZE DISTRIBUTION

SAND, some silt, trace clay

FIG No. DCC8-GS-4

HWY: 407E

W.P. No. 07-20016

APPENDIX C

RECORD OF BOREHOLE SHEETS FROM GEOCRES REPORTS

ONTMT4S 0510.GPJ 3/12/09

+ 3, X 3: Numbers refer to Sensitivity

ONTMT4S 0510 GPJ 3/12/09

+³ ×³ Numbers refer to Sensitivity

+ 3, X 3. Numbers refer to Sensitivity

Continued Next Page

ONTMT4S 0510.GPJ 12/8/08

QNTMT4S 0510 GPI 12/8/08



RECORD OF BOREHOLE No CM10-2												4 OF 4		METRIC	
G.W.P. W.O. 07-20016		LOCATION N 4 868 918.7 E 350 796.7 Oshawa Creek West						ORIGINATED BY JM							
HWY 407		BOREHOLE TYPE Solid Stem Augers						COMPILED BY ES							
DATUM Geodetic		DATE 2008.03.06 - 2008.03.12						CHECKED BY MEF							
SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	20 40 60 80 100						20 40 60 80 100	
Continued From Previous Page															
123.8 30.5	Clayey SILT, some sand Hard Grey (TILL)		19	SS	100/ .175										
	Silty CLAY, some sand, trace gravel Hard Grey (TILL)(CL)														
			20	SS	67								0 20 38 43		
			21	SS	70										
120.2 34.1	END OF BOREHOLE AT 34.1m. BOREHOLE OPEN TO 12.2m AND WATER LEVEL AT 1.2m UPON COMPLETION. BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG TO SURFACE.														

+ 3 . x 3 : Numbers refer to
Sensitivity

20
15 10 5
(%) STRAIN AT FAILURE



RECORD OF BOREHOLE No CM10b-1												1 OF 3		METRIC	
G.W.P. W.O. 07-20016		LOCATION N 4 868 951.4 E 350 749.1 Oshawa Creek West						ORIGINATED BY LH							
HWY 407		BOREHOLE TYPE Solid Stem Augers						COMPILED BY ES							
DATUM Geodetic		DATE 2009.01.29 - 2009.02.11						CHECKED BY MEF							
SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	20 40 60 80 100						20 40 60 80 100	
161.2 0.0	TOPSOIL: (600mm) Firm Brown Moist		1	SS	7										
160.6 0.6	Silty CLAY, trace sand, oxidized staining Stiff to Very Stiff Brown (TILL)(CL)		2	SS	8										
			3	SS	16								0 8 57 35		
158.9 2.3	SILT and SAND, trace clay, trace gravel, oxidized staining Compact to Very Dense Brown Moist to Wet		4	SS	23										
			5	SS	29								2 36 55 7		
			6	SS	60										
155.4 5.8	Silty SAND Dense to Compact Brown Moist to Wet		7	SS	45								0 79 21 (SI+CL)		
			8	SS	22										
			9	SS	8										

Continued Next Page

+ 3 . x 3 : Numbers refer to
Sensitivity

20
15 10 5
(%) STRAIN AT FAILURE



RECORD OF BOREHOLE No CM10b-1															2 OF 3		METRIC	
G.W.P. W.O. 07-20016		LOCATION N 4 868 951.4 E 350 749.1 Oshawa Creek West										ORIGINATED BY LH						
HWY 407		BOREHOLE TYPE Solid Stem Augers										COMPILED BY ES						
DATUM Geodetic		DATE 2009.01.29 - 2009.02.11										CHECKED BY MEF						
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa										WATER CONTENT (%)
Continued From Previous Page							20 40 60 80 100					W P W W L					kN/m ³	GR SA SI CL
						O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL X LAB VANE					40 80 120 160 200							
149.5	Silty SAND Dense Brown Moist to Wet		10	SS	32											2 65 28 5		
11.7	SAND, trace silt, trace gravel Compact to Dense Brown Wet		11	SS	12													
146.7			12	SS	47													
14.5	Silty CLAY, trace sand Very Stiff Grey (TILL)(CL)		13	SS	28													
			14	SS	15											0 1 64 35		
142.6	Silty SAND, trace gravel, trace clay Very Dense Grey Moist (TILL)		15	SS	74											1 62 28 9		
			16	SS	143													

Continued Next Page

+ 3 . X 3 : Numbers refer to Sensitivity 20 15 10 5 (%) STRAIN AT FAILURE



RECORD OF BOREHOLE No CM10b-1															3 OF 3		METRIC	
G.W.P. W.O. 07-20016		LOCATION N 4 868 951.4 E 350 749.1 Oshawa Creek West										ORIGINATED BY LH						
HWY 407		BOREHOLE TYPE Solid Stem Augers										COMPILED BY ES						
DATUM Geodetic		DATE 2009.01.29 - 2009.02.11										CHECKED BY MEF						
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa										WATER CONTENT (%)
Continued From Previous Page							20 40 60 80 100					W P W W L					kN/m ³	GR SA SI CL
						O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL X LAB VANE					40 80 120 160 200							
139.9	Silty SAND, trace gravel, trace clay Very Dense Grey Moist (TILL)				0.275													
21.3	SAND, some gravel, trace silt, trace clay Very Dense Brown Wet		17	SS	150/											15 66 19 (SI+CL)		
					0.300													
138.3																		
22.9	END OF BOREHOLE AT 22.9m. BOREHOLE BACKFILLED WITH BENTONITE GROUT TO SURFACE.				0.075													

+ 3 . X 3 : Numbers refer to Sensitivity 20 15 10 5 (%) STRAIN AT FAILURE



RECORD OF BOREHOLE No CM10b-2															1 OF 2		METRIC	
G.W.P. W.O. 07-20016		LOCATION N 4 868 991.9 E 350 815.3 Oshawa Creek West										ORIGINATED BY JM						
HWY 407		BOREHOLE TYPE Solid Stem Augers										COMPILED BY ES						
DATUM Geodetic		DATE 2008.03.06 - 2008.03.07										CHECKED BY MEF						
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa										
						○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE				WATER CONTENT (%)								
156.0																		
0.0	TOPSOIL: (150mm), with roots																	
0.2	Brown Moist																	
	SAND, some silt, trace gravel, trace clay, trace to some rootlets																	
	Loose Brown Moist		1	SS	10													
			2	SS	10													
153.8																		
2.1	SAND and SILT, trace clay																	
	Loose to Very Loose Brown to Grey Wet		3	SS	8										0 46 46 8			
			4	SS	2													
151.9																		
4.1	Silty SAND, trace gravel																	
	Compact Grey Wet		5	SS	11										1 70 29 (SI+CL)			
149.9																		
6.1	Sandy SILT, some clay																	
	Compact Grey Moist		6	SS	27													
148.8																		
7.2	Silty CLAY, trace sand, trace gravel																	
	Hard Grey (TILL)(CL)		7	SS	52													
147.3																		
8.7	Silty SAND, trace gravel																	
	Very Dense Grey Moist (TILL)		8	SS	100/200													

Continued Next Page

+ 3, x 3, Numbers refer to Sensitivity
20
15 10
(%) STRAIN AT FAILURE



RECORD OF BOREHOLE No CM10b-2															2 OF 2		METRIC	
G.W.P. W.O. 07-20016		LOCATION N 4 868 991.9 E 350 815.3 Oshawa Creek West										ORIGINATED BY JM						
HWY 407		BOREHOLE TYPE Solid Stem Augers										COMPILED BY ES						
DATUM Geodetic		DATE 2008.03.06 - 2008.03.07										CHECKED BY MEF						
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa										
						○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE				WATER CONTENT (%)								
	Continued From Previous Page																	
	Silty SAND, trace gravel																	
	Very Dense Grey Moist (TILL)		9	SS	100/225										4 73 23 (SI+CL)			
143.8																		
12.2	Sandy SILT, some clay, trace gravel																	
	Very Dense Grey Moist (TILL)		10	SS	100/.175													
141.9																		
14.0	END OF BOREHOLE AT 14.1m.																	
	Piezometer installation consists of 19mm diameter schedule 40 PVC pipe with a 1.52m slotted screen.																	
	WATER LEVEL READINGS:																	
	DATE DEPTH (m) ELEV. (m)																	
	2008.04.28 2.3 153.7																	
	2008.07.28 2.2 153.8																	

+ 3, x 3, Numbers refer to Sensitivity
20
15 10
(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No P13													1 OF 1		METRIC	
W.P. 282-85-01		LOCATION N 4 867422.8 E 348100.7				ORIGINATED BY DK										
DIST 6		HWY 407		BOREHOLE TYPE H.S. Auger, Cone Test				COMPILED BY DT								
DATUM Geodetic		DATE 93 12 13				CHECKED BY BT										
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			'N' VALUES	20						40	60	80
158.8	Ground Surface															
0.0	Granular Fill															
0.5	Clayey SIL, Trace Sand, with Organic Inclusions (Fill)		1	SS	9											
157.4			2	SS	20											
1.4	Heterogeneous Mixture of Clayey SIL, Gravel, Brown, Very Stiff		3	SS	37											
156.7			4	SS	123											
2.1	Brown, Dense Grey, Very Dense Heterogeneous Mixture of SIL, Sand and Gravel, Occasional Cobbles and Boulders (Glacial Till)		5	SS	120											
152.7			6	SS	103											
6.1			7	SS	117											
	Heterogeneous Mixture of Clayey SIL, Trace Gravel Occasional Sand layers, Cobbles and Boulders, Grey, Hard (Glacial Till)		8	SS	120											
146.4			9	SS	170											
12.4	End of Borehole		10	SS	104											
	• Unstabilized water level measured upon completion of drilling on 93 12 13															