February 2013

REPORT

FOUNDATION INVESTIGATION AND DESIGN REPORT

MAVIS ROAD UNDERPASS
HIGHWAY 401 WIDENING FROM HIGHWAY 403/410 INTERCHANGE TO THE CREDIT RIVER
CITY OF MISSISSAUGA, REGION OF PEEL
GWP 2150-01-00

Submitted to:
AECOM Canada Ltd.
300 Water Street
Whitby, Ontario
L1N 9J2

Reference: ©2012 Google – Cnes/Spot Image Digital Globe, First base Solution

GEOCRES No.: 30M12-355

Report Number: 10-1111-0211-01

Distribution:

3 Copies – MTO Central Region
1 Copy – MTO Foundations Section
2 Copies – AECOM Canada Ltd.
2 Copies – Golder Associates Ltd.

Golder Associates

Table of Contents

PART A – FOUNDATION INVESTIGATION REPORT

PART B – FOUNDATION DESIGN REPORT

TABLES

DRAWINGS

FIGURES

APPENDICES

PART A

FOUNDATION INVESTIGATION REPORT
MAVIS ROAD UNDERPASS
HIGHWAY 401 WIDENING FROM HIGHWAY 403/410 INTERCHANGE TO THE CREDIT RIVER
CITY OF MISSISSAUGA, REGION OF PEEL
GWP 2150-01-00

1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by AECOM Canada Ltd. (AECOM) on behalf of the Ministry of Transportation, Ontario (MTO) to provide detail foundation engineering services for the proposed widening of Highway 401 from Highway 403/410 Interchange to the Credit River in the City of Mississauga, Region of Peel, Ontario.

This report addresses the foundation investigation carried out for the proposed rehabilitation and lengthening of the Mavis Road underpass structure and the modifications to the existing approach embankments to accommodate the proposed Highway 401 widening. The purpose of this investigation is to establish the subsurface conditions at the location of the proposed structure lengthening, by borehole drilling and laboratory testing on selected samples.

The Terms of Reference for the foundation engineering services are outlined in MTO’s Request for Proposal dated October 2010 and the associated MTO Clarification Packages No.1 to 3 issued between October and November 2010, which forms part of the Consultant’s Agreement Number 2010-E-0003 for this project. The work has been carried out in accordance with Golder’s Supplemental Specialty Quality Control Plan for this project, dated April 2011.

2.0 SITE DESCRIPTION

The Mavis Road underpass structure is located at the intersection of Highway 401 and Mavis Road, approximately 4.5 km west of the Highway 403/410 Interchange and approximately 2 km east of the Credit River, in the City of Mississauga, Ontario. The existing structure is a two-span bridge with span lengths of about 26 m and 27 m, with the existing north and south abutments supported on spread footings and the central pier supported on caissons. The existing closed abutments consist of Retained Soil System (RSS) walls with associated facing panels that are designed to be temporary walls to accommodate the future Highway 401 widening. Highway 401 in this area is a three lane freeway in both eastbound and westbound directions.

The topography across the site adjacent to Highway 401 consists of gently undulating terrain which slopes downward to the west towards the Credit River. Vegetation within the right of way and the associated interchange loops is sparse consisting of grass, small shrubs and occasional treed areas further east of the bridge. Residential properties are present along the Highway 401 corridor west of Mavis Road and commercial facilities are located along Highway 401 east of Mavis Road.

The existing Highway 401 grade in the general area of the underpass varies between about Elevation 187.5 m and 187.2 m; and the existing Mavis Road grade varies between about Elevation 195.5 m and 195.4 m. The existing approach embankments, which are about 8 m high, are sloped at approximately 2 horizontal to 1 vertical (2H:1V) with 2 m wide mid-height berms present on the west side-slopes where embankments are slightly higher than 8 m.

3.0 INVESTIGATION PROCEDURES

3.1 Previous Investigation by Others

A previous foundation investigation for the existing Mavis Road underpass structure was carried out by Terraprobe Ltd. (Terraprobe) in 1997, when a total of six (6) boreholes designated as Boreholes 97-1 to 97-6 were advanced near the existing foundation elements. Boreholes 97-1 and 97-2 are located at the existing north abutment (proposed north pier); Boreholes 97-3 and 97-4 are located at the central pier; and Boreholes 97-5 and 97-6 are located at the existing south abutment (proposed south pier). The results of the Terraprobe investigation are contained in their report titled “Foundation Investigation Report for Highway 401 – Mavis Road Underpass, City of Mississauga, MTO WP 311-89-00; Site No. 24-736”, dated February 16, 1998. The locations of these boreholes were converted to MTM NAD 83 coordinates and are shown on Drawing 1 and a copy of the borehole records are presented in Appendix C. Reference to the subsurface conditions at these borehole locations is made where appropriate to supplement the subsurface information collected during the current investigation.

The subsurface conditions encountered during the previous investigation consist of surficial fill and topsoil underlain by deposits of sandy clayey silt and/or sandy clayey silt till to silty clay till.

3.2 Current Investigation

The field work for the current foundation investigation at the Mavis Road underpass site was carried out between May 22 and June 7, 2012, during which time a total of five sampled boreholes were advanced at the bridge site: two boreholes were drilled in the vicinity of the proposed new north abutment and three boreholes were drilled in the vicinity of the proposed new south abutment. The boreholes (designated as Boreholes MR-1 to MR-4 and MR-3A) were advanced to depths up to about 39 m below ground surface and their locations are shown on Drawing 1. Refusal to advance the casing was encountered at a depth of about 18.3 m below ground surface in Borehole MR-3, in order to continue sampling the overburden beyond this depth, Borehole MR3-A was advanced about 1.5 m north of Borehole MR-3.

The field investigation was carried out using a CME 75 truck-mounted drill rig, supplied and operated by Geo-Environmental Drilling Inc. of Halton Hills, Ontario. The boreholes were advanced through the overburden using 108 mm inside diameter (I.D.) hollow stem augers and/or NW casing using Tricone and wash boring techniques. Soil samples were obtained in the boreholes at 0.75 m, 1.5 m and 3.0 m intervals of depth using a 50 mm outer diameter split-spoon sampler driven by an automatic hammer in accordance with Standard Penetration Test (SPT) procedures (ASTM D1586)[1].

Dynamic Cone Penetration Tests (DCPTs) were conducted in Boreholes MR-1, MR-2 and MR-3A to depths up to 39 m below ground surface. This test consists of continuously driving into undisturbed ground a 50 mm diameter cone (60°vertex angle) attached to a drill rod, with a driving energy of 475 J per blow (63.5 kg automatic hammer dropping freely a vertical distance of 0.76 m). The number of blows for each 300 mm of penetration is recorded and this provides an indication of the relative changes in the soil density/consistency with depth.

---

[1] ASTM D1586 – Standard Test Method for Standard Penetration Tests and Split Barrel Sampling of Soils.

The groundwater conditions in the open boreholes were observed during and immediately following the drilling operations. A standpipe piezometer was installed in Boreholes MR-1 and MR-4 to permit monitoring of the water level at the site. The installed piezometers consist of a 50 mm diameter PVC pipe, with a 3 m slotted screen sealed within a filter sand pack. The borehole and annulus surrounding the piezometer pipe above the filter sand pack was backfilled to the ground surface with bentonite pellets and/or cement grout. Piezometer installation details and water level readings are described on the borehole records presented in Appendix A. All boreholes in which standpipe piezometers were not installed were backfilled to ground surface with bentonite upon completion, in accordance with Ontario Regulation 903 (as amended). The boreholes advanced through the Mavis Road asphalt were sealed at the surface with cold patch asphalt, approximately 0.2 m thick.

The field work was monitored on a full-time basis by a member of Golder's technical staff who located the boreholes in the field, directed the sampling and in situ testing operations, logged the boreholes and examined and cared for the soil samples. Due to the volume of traffic and road occupancy time restrictions on Mavis Road, coupled with the relatively steep embankment slopes at this site, the borehole locations were carefully selected to be as close as possible to the desired foundation element footprints while allowing for safe operation of the drill rig and minimal traffic disruptions.

The recovered soil samples were identified in the field, placed in labelled containers and transported to Golder's laboratory in Mississauga for visual identification and selected samples were subjected to a laboratory testing program consisting of natural moisture content, Atterberg limits and grain size distribution analyses in accordance with MTO and/or ASTM Standards as applicable. The results of this testing program are shown on the Record of Borehole sheets in Appendix A and the laboratory figures contained in Appendix B.

The borehole locations were staked/marked in the field by Golder personnel relative to the existing bridge and on-site features shown on the digital terrain model for the site, provided by AECOM. The ground surface elevations at the borehole locations were surveyed by J.D. Barnes Ltd., a licensed surveying company retained by AECOM. The borehole locations (referenced to MTM NAD83 northing and easting coordinates), ground surface elevations (referenced to geodetic datum) and the borehole depths are shown on Drawing 1 and are summarized below.

Borehole No.	Location (MTM NAD 83)		Ground Surface Elevation (m)	Borehole Depth (m)
	Northing (m)	Easting (m)		
MR-1	4,831,249.0	287,997.7	194.7	26.5
MR-2	4,831,239.6	287,971.8	195.1	39.0
MR-3	4,831,172.8	288,079.3	194.6	18.3
MR-3A	4,831,173.9	288,078.3	194.6	32.6
MR-4	4,831,158.4	288,051.0	195.4	31.1

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

This section of Highway 401 is located in the Peel Plain close to the border of the South Slope physiographic region, as delineated in *The Physiography of Southern Ontario* (Chapman and Putnam, 1984)[2].

The Peel Plain physiographic region covers the central portions of the Regional Municipalities of York, Peel and Halton. The general topography of this region consists of level to gently rolling terrain, sloping gradually southward toward Lake Ontario. A surficial till sheet, which generally follows the surface topography, is present throughout much of this area. The till, which is mapped in this area as the Halton Till, typically consists of clayey silt to silty clay, with occasional sand to silt zones. Shallow, localized deposits of loose sand and silt and/or soft clay can overlie this uppermost till sheet, and these represent relatively recent deposits, formed in small glacial melt water ponds scattered throughout the Peel Plain and concentrated near river valleys. The recent sand, silt and clay and uppermost till deposits in this area overlie and are interbedded with stratified deposits of sand, silt and clay. The overburden within the majority of the Peel Plain area is underlain by shale bedrock of the Georgian Bay Formation which contains limestone interlayers.

4.2 Subsurface Conditions

The detailed subsurface soil and groundwater conditions encountered in the boreholes advanced as part of the current investigation and the results of the laboratory testing are provided on the borehole records contained in Appendix A; the results of the geotechnical laboratory testing are presented on Figures B1 to B9 contained in Appendix B. Copies of the borehole records 97-1 to 97-6 from the previous investigation by Terraprobe are presented in Appendix C.

The stratigraphic boundaries shown on the borehole records are inferred from non continuous sampling, observations of drilling progress and the results of Standard Penetration Tests. These boundaries, therefore, represent transitions between soil types rather than exact planes of geological change and the stratigraphy shown on the profile and in cross sections on Drawings 1 and 2 are interpretations of the subsurface conditions. Variation in the stratigraphic boundaries between and beyond boreholes will exist and is to be expected, however, the factual data presented in the Record of Borehole sheets governs any interpretation of the site conditions.

In general, the subsurface conditions at the site consist of a surficial layer of asphalt or topsoil over a deposit of fill associated with the existing approach embankments. The fill is underlain by a deposit of clayey silt to silty clay in places, which in turn is underlain by a deposit of clayey silt with sand till. In Borehole MR-2, the cohesive till deposit is underlain by a cohesionless deposit of sand and silt till which transitions to sand and gravel at depth. Silty clay and silty sand and gravel interlayers are present between the cohesive and cohesionless till deposits.

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

---

[2] Chapman, L.J. and Putnam, D,F. 1984. The Physiography of Southern Ontario, Ontario Geological Survey, Special Volume 2, Third Edition. Accompanied by Map P. 2715, Scale 1:600,000.

4.2.1 Asphalt

Boreholes MR-1, MR-2 and MR-4 were drilled through the surficial pavement on Mavis Road. An approximately 200 mm thick layer of asphalt was encountered at these locations.

4.2.2 Topsoil

A 200 mm thick surficial layer of topsoil was encountered in Borehole MR-3 advanced at the crest of the existing south approach embankment.

4.2.3 Fill

Fill materials were encountered underlying the topsoil and asphalt in all the boreholes drilled as part of the current investigation. The thickness of the fill deposit is between about 10.2 m and 11.4 m, with the base of the fill encountered between Elevations 184.4 m and 183.8 m.

The fill material is variable in composition. In general, a layer of cohesionless fill comprised of sand and gravel was encountered below the asphalt layer in Boreholes MR-1, MR-2 and MR-4. Underlying the sand and gravel fill and the topsoil in Borehole MR-3, a deposit of cohesive fill consisting of clayey silt to silty clay, trace to some sand and trace to some gravel was encountered. The upper portion of the cohesive fill contains rootlets, organic materials, wood and asphalt fragments and weathered shale fragments. The presence of cobbles is also inferred from difficulties advancing augers in Borehole MR-4 between depths of about 6.1 m and 6.7 m (corresponding to about Elevation 189.3 m and 188.7 m) during the drilling operations.

The Standard Penetration Test (SPT) “N”-values measured within the cohesive fill generally range from 5 blows to 45 blows per 0.3 m of penetration, suggesting that the clayey silt to silty clay fill has a firm to hard consistency. A SPT “N”-value of 80 blows per 0.3 m of penetration was recorded within the fill in MR-4; and this high value is attributed to the presence of cobbles at that depth.

Grain size distribution tests were carried out on two (2) samples of the clayey silt fill and the results are provided on Figure B1 in Appendix B.

Atterberg limits tests were carried out on five (5) samples of the clayey silt to silty clay fill. The liquid limits range from about 25 per cent to 35 per cent, the plastic limits range from about 15 per cent to 20 per cent, and the plasticity indices range from about 10 per cent to 19 per cent. The results of the Atterberg limits tests are shown on a plasticity chart on Figure B2 in Appendix B, and indicate that the fill material consists of clayey silt of low plasticity to silty clay of intermediate plasticity.

The natural water content measured on sixteen (16) samples of the fill material ranges from about 7 per cent to 14 per cent

4.2.4 Clayey Silt to Silty Clay

A deposit of brown clayey silt to silty clay was encountered underlying the fill material in Boreholes MR-1 and MR-2, advanced near the proposed new north abutment. The thickness of this deposit is about 0.9 m, and extends to depths of about 11.6 m below ground surface which corresponds to Elevations 183.1 m and 183.5 m in Boreholes MR-1 and MR-2, respectively.

The clayey silt to silty clay deposit contains trace to some sand and trace to some gravel. The SPT “N”-values measured within this deposit are 14 blows and 22 blows per 0.3 m of penetration, suggesting that the clayey silt to silty clay has a stiff to very stiff consistency.

A grain size distribution test was carried out on one (1) sample of the clayey silt portion of this deposit and the result is shown on Figure B3 in Appendix B.

Atterberg limits test was carried out on one (1) sample of the silty clay portion of this deposit and measured a liquid limit of about 43 per cent, a plastic limit of about 19 per cent and a corresponding plastic index of about 24 per cent. The result of the Atterberg limits test is shown on a plasticity chart on Figure B4 in Appendix B, and indicates that this material is silty clay of intermediate plasticity.

The natural water content measured on samples of this deposit is about 13 per cent and 18 per cent.

4.2.5 Clayey Silt with Sand Till

A predominantly cohesive till deposit was encountered underlying the clayey silt to silty clay deposit in Boreholes MR-1 and MR-2, and directly below the fill materials in Boreholes MR-3 and MR-4. The top of the deposit ranges from about 10.4 m to 11.6 m below the Mavis Road ground surface (Elevation 184.2 m to 183.1 m, and the boreholes penetrated 14.9 m to 22.2 m into the deposit. Borehole MR-2 was the only borehole that fully penetrated the till layer which was measured to be 15.5 m thick. In Borehole MR-3, refusal to advance the casing was encountered within the cohesive till deposit at a depth of about 18.3 m below ground surface. Borehole MR3-A was advanced about 1.5 m north of Borehole MR-3 and soil sampling continued to a depth of about 30.9 m below ground surface. All boreholes except Borehole MR-2 were terminated within the cohesive till between about Elevation 168.2 m and 162.0 m.

The cohesive till deposit generally consists of clayey silt with sand containing trace to some gravel, sand seams cobbles, boulder, and shale fragments were observed/inferred throughout the drilling and sampling of the cohesive till deposit. A boulder was encountered and cored within the cohesive till in Borehole MR-3 between depths of about 15.3 m and 15.9 m (corresponding to about Elevation 179.3 m and 178.7 m) during the drilling operations.

The SPT “N”-values measured within cohesive till generally range from 12 blows to 116 blows per 0.3 m of penetration, suggesting that the clayey silt with sand till has a stiff to hard consistency. The higher SPT “N”-values were generally recorded within the lower portion of the cohesive till deposit, suggesting the clayey silt with sand till is hard below about Elevation 173 m. SPT “N”-values of 89 blows per 0.23 m of penetration and 50 blows per 0.06 m of penetration were measured at depths where cobbles and boulders were inferred and/or encountered.

Dynamic cone penetration tests (DCPTs) were advanced from the bottom of the sampled Boreholes MR-1 and MR-3A at depths of about 25.2 m and 30.9 m below ground surface; the DCPTs were terminated on effective refusal (greater than 200 blows per 0.3 m of penetration) at depths of about 26.5 m (Elevation 168.2 m) and 32.6 m (Elevation 162.0 m), respectively.

Grain size distribution tests were carried out on twelve (12) selected samples of the clayey silt with sand till deposit and the results are shown on Figures B5A and B5B in Appendix B.

Atterberg limits tests were carried out on fourteen (14) selected samples of this cohesive till deposit and measured liquid limits varying from about 18 per cent to 30 per cent, plastic limits varying from about 11 per cent to 17 per cent, and plasticity indices varying from about 6 per cent to 14 per cent. These results, which are plotted on a plasticity chart on Figure B6 in Appendix B, indicate that the till deposit generally consists of clayey silt of low plasticity.

The natural water content measured on nineteen (19) selected samples of the clayey silt with sand till ranges from about 8 per cent to 24 per cent.

4.2.6 Silty Clay and Silty Sand and Gravel Interlayers

In Borehole MR-2, interlayers of silty clay of approximately 0.6 m thick and silty sand and gravel of approximately 1.0 m thick were encountered below the cohesive till deposit at about Elevation 168.0 m and 167.4 m. The silty clay interlayer contains trace sand and trace gravel, and the silty sand and gravel interlayer contains trace clay.

One SPT “N”-value recorded within these interlayers is 39 blows per 0.3 m of penetration, suggesting a hard consistency for the silty clay material and a dense relative density for the silty sand and gravel material.

The natural water content measured on a sample of the silty clay interlayer is about 27 per cent.

4.2.7 Sand and Silt Till

A cohesionless till deposit of sand and silt was encountered below the silty sand and gravel interlayer at about Elevation 166.4 m in Borehole MR-2. The thickness of the sand and silt till deposit is about 7.3 m, with the base of the deposit at about Elevation 159.1 m.

The till deposit consists of sand and silt containing trace to some gravel, trace to some clay and frequent shale fragments. The SPT “N”-values recorded within the cohesionless till deposit range from 29 blows to 64 blows per 0.3 m of penetration, indicating that the sand and silt till deposit has a compact to very dense relative density.

A grain size distribution test was carried out on one (1) selected sample of the sand and silt till deposit and the result is provided on Figure B7 in Appendix B. An Atterberg limits test was carried out on one (1) sample of the sand and silt till deposit and measured a liquid limit of about 17 per cent, a plastic limit of about 13 per cent and a corresponding plastic index of about 4 per cent. The result of the Atterberg limits test is shown on a plasticity chart on Figure B8 in Appendix B, and indicates that this material contains silt of low plasticity.

The natural water content measured on a sample of the sand and silt till deposit is about 9 per cent.

4.2.8 Sand and Gravel

A deposit of sand and gravel containing some silt and trace clay was encountered underlying the sand and silt till deposit at about Elevation 159.1 m in Borehole MR-2. The borehole penetrated 1.2 m into the sand and gravel deposit and sampling was terminated within this deposit at about Elevation 157.9 m.

One SPT "N"-value recorded within this deposit is 11 blows per 0.3 m of penetration, indicating that the sand and gravel deposit has a compact relative density.

A Dynamic cone penetration test (DCPT) was advanced from the bottom of the sampled Borehole MR-2 at a depth of about 37.2 m below ground surface; the DCPT was terminated on effective refusal (200 blows per 0.15 m of penetration) at about 39.0 m below ground surface (Elevation 156.1 m).

A grain size distribution test was carried out on a sample of the sand and gravel deposit and the result is provided on Figure B9 in Appendix B. The natural water content measured on one sample of the sand and gravel is about 6 per cent.

4.2.9 Groundwater Conditions

The groundwater levels in the current boreholes were measured during and upon completion of drilling operations. Borehole MR-3 was dry upon completion of drilling. A standpipe piezometer was installed in Boreholes MR-1 and MR-4 to permit monitoring of the groundwater level at this site. Details of the piezometer installation are shown on the Record of Borehole sheets in Appendix A, and the groundwater levels measured in the open boreholes and piezometers are summarised below.

Borehole No.	Ground Surface Elevation (m)	Depth to Water Level (m)	Depth to Water Elevation (m)	Date	Comments
MR-1	194.7	Dry 10.4 10.2 10.1	- 184.3 184.5 184.6	June 07,2012 August 10, 2012 October 09, 2012 November 05, 2012	Piezometer
MR-2	195.1	19.8	175.3	May 22, 2012	Open Borehole
MR-3/3A	194.6	Dry	-	May 30, 2012	Open Borehole
MR-4	195.4	18.3 18.6 18.4 18.4 18.3	177.1 176.8 177.0 177.0 177.1	May 28, 2012 May 30, 2012 August 10, 2012 October 09, 2012 November 05, 2012	Piezometer

Based on the groundwater levels recorded during this current investigation, the water level at this site is at approximately Elevation 177± m, which is about 10.5 m below the paved surface of Highway 401 at the time of this investigation, and is consistent with the groundwater levels recorded during the previous investigation. It is noted that a higher groundwater level (Elevation 184.6 m) was measured in the piezometer installed in MR-1; however, due to the fact that the screen/seal is located in close proximity to the fill/till interface, the water level may represent "perched" water conditions within the fill and above clayey silt/cohesive till deposit, and is likely not representative of the stabilized groundwater level.

The groundwater level observations at this site are short term and will be subject to seasonal fluctuations and precipitation events, therefore the water levels should be expected to be higher during the spring season or during any period of heavy precipitation.

5.0 CLOSURE

The field drilling program was supervised by Mr. Suresh Bainey, a senior technician with Golder. This Foundation Investigation Report was prepared by Ms. T. Veronica Ayetan, P.Eng., a geotechnical engineer, and reviewed by Mr. Kevin Bentley, P.Eng., a senior geotechnical engineer and Associate with Golder. Mr. Ty Garde, P.Eng., a Designated MTO Contact and Principal with Golder, conducted an independent review and quality control audit of this report.

GOLDER ASSOCIATES LTD.

LICENSED PROFESSIONAL ENGINEER
Feb 7, 2013
T. V. AYETAN
100130071
PROVINCE OF ONTARIO

VAyetan

T. Veronica Ayetan, P.Eng.
Geotechnical Engineer

Kevin Bentley, P.Eng.
Geotechnical Engineer, Associate

REGISTERED PROFESSIONAL ENGINEER
Feb 7, 2013
T. J. GARDE
PROVINCE OF ONTARIO

TJGarde

Ty Garde, P.Eng
Designated MTO Contact, Principal

TVA/KJB/TG/tva/jl

n:\active\2010\1111\10-1111-0211 aecom-hwy 401 widening-mississauga\9 - reports\1 - mavis road underpass\final\10-1111-0211 rpt01 13feb 07 mavis road underpass.docx

PART B

FOUNDATION DESIGN REPORT
MAVIS ROAD UNDERPASS
HIGHWAY 401 WIDENING FROM HIGHWAY 403/410 INTERCHANGE TO THE CREDIT RIVER
CITY OF MISSISSAUGA, REGION OF PEEL
GWP 2150-01-00

February 2013
Report No. 10-1111-0211-01

6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

6.1 General

This section of the report provides foundation engineering recommendations for the detail design of the proposed rehabilitation and lengthening of the existing Mavis Road underpass as part of the Highway 401 widening from Highway 403/410 interchange to the Credit River, in the City of Mississauga. The recommendations are based on interpretation of the factual data obtained from the boreholes advanced during the current subsurface investigation at this site, and the previous investigation conducted by Terraprobe in 1998. The interpretation and recommendations contained in this report are intended to provide the designers with sufficient information to assess the feasible foundation alternatives and to carry out the detail design of the foundations for the proposed extension of the structure. Where comments are made on construction, they are provided in order to highlight those aspects that could affect the design of the project, and for which special provisions or operational constraints may be required in the Contract Documents. Those requiring information on aspects of construction should make their own interpretation of the factual information provided as such interpretation may affect equipment selection, proposed construction methods, scheduling and the like.

The existing Mavis Road underpass consists of a two-span bridge with span lengths of about 26 m and 27 m. Based on the existing bridge General Arrangement (GA) drawing prepared by Giffels (dated February 1998), the existing north and south abutments are supported on spread footings (shallow foundations) and the centre pier is supported on caissons (deep foundations). Retained Soil System (RSS) walls are located directly behind the north and south abutments and from the closed face of the approach embankments which are up to about 8 m high. The approach embankment side-slopes are approximately 2 horizontal to 1 vertical (2H:1V) with a 2 m wide mid-height berm located on the west side of both the north and south approach embankments. It is understood that the existing bridge approach embankments and abutment footings were designed to accommodate the currently proposed Highway 401 widening/Mavis Road structure extension without major re-construction.

Based on the General Arrangement (GA) drawing provided by AECOM on July 25, 2012 titled “Highway 401, Mavis Road Bridge Rehabilitation”, the existing bridge structure will be extended to consist of four spans, with span lengths of 28 m between the north abutment and north pier, 26 m between the north and centre piers, 27 m between the centre and south piers, and 28 m between the south pier and south abutment. As a result, the existing north and south abutment footings will become the north and south piers and new north and south abutments are now being proposed for the Mavis Road structure. The proposed rehabilitation to the existing structure will result in increased loading on the existing north and south abutment footings (i.e. the proposed north and south piers); and negligible additional loading on the centre pier.

The proposed Mavis Road bridge deck varies between about Elevation 195.5 m and 195.2 m. The existing Mavis Road bridge approach embankments (including the RSS walls) immediately behind the proposed new north pier and south pier (which are currently the existing north and south abutments) will need to be removed/sub-excavated to about Elevation 187 m to allow for construction of the widened Highway 401 lanes and the new Mavis Road structure north and south abutments. The proposed Highway 401 pavement grade under the new Mavis Road structure extension varies between about Elevation 187.5 m and 187.2; and the proposed new approach embankments are approximately 8 m high, and oriented at approximately 2H:1V. New RSS walls are proposed for the wing walls at the new north and south abutment locations. According to the GA drawing provided by AECOM, there is no significant grade change between the existing and proposed Mavis Road and Highway 401 road profiles.

6.2 Assessment of Existing Structure Foundations

The existing Highway 401 – Mavis Road underpass structure was constructed in 1998 and was designed to accommodate the currently proposed extension. Based on the available design drawings (Giffels, 1998), the bridge abutments are supported on spread footings and the centre pier is supported on caissons. At the north and south abutments, the spread footings are about 4 m wide, 42 m long and 1.2 m thick. The footings are founded at about Elevation 183.8 m and 184.0 m at the north and south abutments, respectively. The subsurface information provided on the Record of Boreholes advanced at the north abutment (Boreholes 97-1 and 97-2) and south abutment (Boreholes 97-5 and 97-6) suggest that the footings are founded on hard clayey silt with sand/silty clay till at the north abutment and very stiff to hard clayey silt with sand till at the south abutment. Boreholes 97-3 and 97-4 advanced at the pier suggest that the caissons are terminated within the 100 blow clayey silt till. At the centre pier, seven columns are supported on individual caissons that are 1524 mm in diameter, and are shown to be terminated about 14 m to 15 m below ground surface (Elevation 172.0 m).

The design geotechnical axial resistance values for the existing Mavis Road Bridge footings are provided in the Terraprobe report titled “Foundation Investigation and Design Report, Highway 401 – Mavis Road Underpass (Geocres No. 30M12-237)”, dated February 1998, and are shown on the design drawings (Giffels, 1998) as follows:

North and South Abutment (Proposed “New” North and South Piers) Spread Footings

Factored Geotechnical Resistance at Ultimate Limit State (ULS) = 450 kPa

Geotechnical Reaction at Serviceability Limit State (SLS - for 25 mm settlement) = 300 kPa.

Centre Pier Caissons

Factored Geotechnical Resistance at Ultimate Limit State (ULS)

Shaft Resistance = 30 kPa

End Bearing = 1,800 kPa

Geotechnical Reaction at Serviceability Limit State (SLS - for 25 mm settlement)

Shaft Resistance = 25 kPa

End Bearing = 1,500 kPa

6.2.1 Existing Foundation Geotechnical Resistance for Rehabilitation

Based on the GA drawing provided for the proposed rehabilitation and extension to the Mavis Road structure, it is understood that the existing centre pier columns and foundations, and existing north and south abutment columns foundations (proposed new north and south pier foundations) will remain in place and will not be modified if the proposed new loading conditions does not exceed the design geotechnical resistances. The design geotechnical resistances for the new north and south pier (i.e. the existing north and south abutments) and the existing centre pier have been estimated based on the available subsurface information and assuming the foundations were constructed in accordance with the design drawings (Giffels, 1998). The factored geotechnical axial resistance at ULS and the geotechnical reaction at Serviceability Limit States (SLS) for the existing foundations are considered to be consistent with the previous recommendations provided by Terraprobe as summarized below.

Foundation Element	Foundation Type	Foundation Geometry	Founding Elevation	Factored Geotechnical Resistance at ULS (kPa)	Geotechnical Reaction at SLS (kPa) (for 25 mm settlement)
North Pier (Existing North Abutment)	Spread Footing	4 m x 42 m x1.2 m	183.8 m	450 kPa	300 kPa
South Pier (Existing South Abutment)	Spread Footing	4 m x 42 m x1.2 m	184.0 m	450 kPa	300 kPa
Centre Pier	Caisson	1524 mm Diameter	172.0 m	1,800 kPa	1,500 kPa

If the design geotechnical resistance of the existing foundations is not sufficient for the proposed new loading conditions, consideration can be given to supplementing the existing foundations (e.g. widening the existing spread footings or installing additional caissons at the pier location) as appropriate. If higher geotechnical resistances are required, additional geotechnical recommendations can be provided for the existing foundations upon request.

6.3 New Abutment Foundations

The proposed lengthening of the Highway 401 – Mavis Road Underpass bridge structure requires extending the north and south limits of the existing bridge by approximately 28 m, resulting in new north and south abutments. Within the vicinity of the proposed new foundation elements, the subsurface soil conditions encountered during the current investigation generally consist of fill soils (associated with the existing Mavis Road approach embankments), underlain by a layer of clayey silt to silty clay in places, underlain by a deposit of clayey silt with sand till which in turn is underlain by a sand and silt till that transitions to a sand and gravel deposit at depth.

Shallow and deep foundations options have been considered for support of the new abutments. A summary of the advantages and the disadvantages associated with each option is provided below, and a comparison of the alternative foundation options based on advantages, disadvantages, risks/consequences and approximate costs is provided in Table 1 following the text of this report.

- **Strip or spread footings founded within the stiff to hard clayey silt with sand till:** Spread footings are considered feasible and suitable to support the new abutments given the competency of the native soils at this site and the relative cost of construction; this option would also allow for the use of semi-integral abutments. Spread footings founded on the native soil would require excavation depth up to approximately 11 m to 12 m below the Mavis Road grade, which may not seem practical; however, these excavation depths are required to remove the existing fills to accommodate the Highway 401 widening and thus only excavation depths up to about 4 m below proposed Highway 401 grade would be required. Depending on construction staging, temporary roadway protection would likely be required at the abutments.

- **Strip or spread footing "perched" on the Granular 'A' pad over stiff to very stiff Mavis Road approach embankment Fill:** For a longer "open" structure configuration with 2H:1V abutment foreslopes, the abutment spread footings may be founded on a Granular 'A' pad within the generally stiff to very stiff clayey silt fill at higher elevations, to reduce the extent of excavation as compared to spread footings founded on the native cohesive till. However, the presence of organics, wood and construction debris within the fill will result in lower resistances and will increase the risk for future settlements.
- **Steel H-piles driven to found within the hard clayey silt with sand till deposit:** Steel 310 x 110 H-piles driven to within the hard clayey silt with sand till are feasible for the support of the proposed abutments, and would allow for integral abutment construction. However, there is risk associated with penetrating through the till deposits or the piles "hanging up" within the till deposit as a result of the occasional presence of boulders encountered at higher elevation across the site. Furthermore, the varying SPT "N"-values at variable depth within the footprint of each foundation element will result in the potential for variable pile lengths, which will need to be accommodated in the contract documents.
- **Steel tube (pipe) piles founded within the hard clayey silt with sand till deposit:** Steel tube (pipe) piles could also be considered as a deep foundation option for support of the abutments, and would allow for integral abutment construction. However, pipe piles are considered to have a higher risk than H-piles for "hanging up" or being deflected away from their vertical or battered orientation due to the presence of cobbles and/or boulders within the glacially-derived soils at this site. In addition, the highly variable SPT "N"-values at variable depths will result in the potential for variable pile lengths.
- **Caissons founded within the stiff to hard clayey silt with sand till deposit:** Consideration could be given to the use of caissons socketted into the hard clayey silt with sand till for support of the new abutments. There is potential for seams or interlayers of water-bearing cohesionless soils (such as sand, silty sand or silt) to be present within the clayey silt with sand till. If encountered, temporary or permanent liners would be required during caisson installation to control the ground and groundwater within these water-bearing cohesionless zones/seams, which would result in the caisson foundations being less cost-effective than the installation of driven steel piles.

At the new abutments, spread footings founded on the stiff to hard clayey silt with sand till is considered to be the preferred option from a geotechnical perspective.

Recommendations for the various foundation options discussed above for the new abutments at the Mavis Road underpass structure are provided in the following sections.

6.3.1 Spread Footings

6.3.1.1 Founding Elevations

Strip or spread footings founded on the generally stiff to hard clayey silt with sand till is considered feasible for support of the new abutments and associated retaining/wing walls. The proposed finished grade of Highway 401 in the area of the abutments is between about Elevation 187.2 m and Elevation 187.5 m, and the proposed finished grade for Mavis Road is at approximately Elevation 195.2 m near the abutments as shown on the GA drawing provided by AECOM.

All footings should be founded at a minimum depth of 1.2 m below the adjacent final grade to provide adequate protection against frost penetration, in accordance with Ontario Provincial Standard Drawing (OPSD) 3090.101

(*Foundation Frost Penetration Depths for Southern Ontario).* The following summarizes the recommended maximum founding elevations for strip or spread footing for support of the new abutments.

Foundation Element	Reference Borehole No.	Founding Stratum	Founding Elevation	Approximate Maximum Excavation Depth
New North Abutment	MR-1 and MR-2	Stiff to hard clayey silt with sand till	183.1 m	12.1 m
		Compacted Granular 'B' Type II	183.8 m	12.1 m
New South Abutment	MR-3/3A and MR-4	Stiff to hard clayey silt with sand till	183.8 m	11.4 m

At the north abutment, the subsurface data obtained indicates the presence of an approximate 0.9 m thick layer of stiff to very stiff clayey silt to silty clay overlying the cohesive till (in Boreholes MR-1 and MR-2). In order to ensure that the footings are founded on the stiff to hard clayey silt with sand till deposit, the clayey silt/silty clay layer should be sub-excavated and spread footings founded at about Elevation 183.1 m. Alternatively, the subexcavated depth (about 0.9 m of the cohesive soil) could be replaced with properly placed and compacted Granular 'B' Type II (SP 110S13 Aggregates) to found the spread footing at a higher elevation (Elevation 183.8 m) consistent with the founding elevation of the existing footing at the north pier, and the proposed new footing at the south abutment.

6.3.1.2 Geotechnical Resistance/Reaction

Strip or spread footings for the new bridge abutments placed on the properly prepared, undisturbed native clayey silt with sand till subgrade and/or a thin layer of compacted granular backfill (i.e. Granular 'B' Type II less than 1 m thick to backfill sub-excavated area) at the founding elevations provided in Section 6.3.1.1, should be designed based on the factored geotechnical resistance at ULS equal to 450 kPa and geotechnical reaction at SLS (for 25 mm of settlement) equal to 300 kPa.

The ULS resistance and settlement are dependent on the footing size (assumed to be 4 m wide), configuration and applied loads; the geotechnical resistances should, therefore, be reviewed if the selected footing width or founding elevation differs from those given above.

The geotechnical resistances provided above are given for loads applied perpendicular to the surface of the footing. Where the load is not applied perpendicular to the surface of the footing, inclination of the load should be taken into account in accordance with Sections 6.7.4 and C6.7.4 in the *Canadian Highway Bridge Design Code (CHBDC) and* its *Commentary,* using the curves for cohesive soils and non-cohesive soil.

The base of each footing excavation should be cleaned of softened material. It is recommended that the founding level for the footings be inspected by a Quality Verification Engineer (QVE) following excavation, in accordance with OPSS 902 *(Excavating and Backfilling Structures)* to verify that all existing fill and other unsuitable material have been removed, as the founding soils will be susceptible to disturbance. If the concrete for the footings cannot be poured immediately after excavation and inspection, it is recommended that a concrete working slab (100 mm thickness of 20 MPa compressive strength concrete) be placed on the subgrade within three hours to protect the integrity of the bearing stratum. This requirement can either be added as a note on the Contract Drawings or included as a Non-Standard Special Provision (NSSP) in the Contract Documents. A sample NSSP is included for this item in Appendix E.

6.3.1.3 Resistance to Lateral Loads

Resistance to lateral forces / sliding resistance between the concrete footings and the subgrade should be calculated in accordance with Section 6.7.5 of the *CHBDC*. For cast-in-place concrete footings constructed on the stiff to hard clayey silt with sand till, the coefficient of friction, tan ϕ', can be taken as 0.45; if constructed on the Granular ‘B’ Type II material, the coefficient of friction tan Φ', can be taken as 0.5. This value is unfactored.

6.3.2 “Perched” Strip or Spread Footing

6.3.2.1 Founding Elevation

A “perched” spread footing founded on a Granular ‘A’ or Granular ‘B’ Type II (SP 110S13 – Aggregates) pad overlying the existing stiff to very stiff clayey silt fill (free of organics and deleterious material) is considered to be feasible option at the new abutment and associated retaining/wing wall sites.

For this option, sub-excavation of any existing topsoil, organics and loosened/softened or deleterious material that may be present in the existing embankment fill within the foundation and granular pad footprint is required to minimize settlement due to loading on the embankment. The area to be sub-excavated should be defined by a line extending from the top of the pad outward and downward at 1 horizontal to 1 vertical (1H:1V). The sub-excavation should be backfilled with compacted Granular ‘B’ Type II or Granular ‘A’ material (SP 110S13). The Granular ‘A’ or Granular ‘B’ Type II pad should be a minimum of 2 m thick and should extend at least 1 m beyond the plan limits of the footing. The elevation of the top of the granular pad will need to be determined during the detail design, if this option is selected. A typical perched footing on a Granular ‘A’ pad is shown on Figure 1. The Granular ‘A’ or ‘B’ pad should be constructed in accordance with OPSS 501 (Compacting) and SP 105S21.

Spread footings perched on a Granular ‘A’ pad should be provided with a minimum of 1.2 m of soil cover for protection from frost penetration in accordance with OPSD 3090.101 (*Foundation Frost Penetration Depths for Southern Ontario).* It should be noted that the required thickness of conventional soil cover for frost protection of the footing (1.2 m) is measured perpendicular from the face of the abutment slope to the edge of the underside of the footing (it is not simply a vertical dimension when the footing is adjacent to a slope). If adequate soil cover cannot be provided for the footing, rigid styrofoam insulation shall be installed to compensate for the lack of cover and provide protection from frost action. As a guide, the MTO has adopted 25 mm (1 inch) of rigid polystyrene foam insulation as equivalent for every 0.3 m reduction in soil cover.

6.3.2.2 Geotechnical Resistance/Reaction

Based on the local fill quality and the above granular pad design and construction procedures, a factored geotechnical resistance at ULS of 450 kPa and a geotechnical reaction at SLS of 250 kPa (for 25 mm of settlement) may be used for the detail foundation design, assuming 4 m wide footings.

The geotechnical resistances provided above are for the loadings that will be applied perpendicular to the surface of the footings. Where the load is not applied perpendicular to the surface of the footing, inclination of the load should be taken into account in accordance with Section 6.7.4 of the *CHBDC and* its *Commentary,* using the curves for cohesive soils and non-cohesive soil.

6.3.2.3 Resistance to Lateral Loads

Resistance to lateral force/sliding resistance between the concrete footing and the compacted Granular 'A' pad should be calculated in accordance with Section 6.7.5 of the *CHBDC.* For cast-in-place concrete footings constructed on the compacted Granular 'A' or 'B' Type II pad, the coefficient of friction tan Φ', can be taken as 0.5. This value is unfactored.

6.3.3 Steel H-Pile / Steel Tube Pile Foundations

Steel H-Piles or steel tube (pipe) piles driven to found within the hard clayey silt with sand till may be used to support the new north and south abutments, especially if integral abutments are being considered. For the installation of either friction or end-bearing piles, consideration must be given to the potential presence of cobbles and boulders within the clayey silt with sand till deposits, as encountered in the boreholes advanced at the abutments. In addition, it is noted that "100-blow" material was occasionally encountered higher than the proposed founding tip elevations. In this regard, steel H-piles are preferred over steel tube piles given that H-piles are more conventional for integral abutment design and the fact that steel tubes are considered to pose a higher risk of "hanging up" or being deflected from their vertical or battered orientation during installation, due to their larger end area. The piles should be reinforced at the tip for protection during driving to reduce the potential for damage to the piles in the event that cobbles/boulders and/or very dense layers are encountered within the till deposits. The steel H-piles should be reinforced with flange plates as per OPSD 3000.100 (Foundation Piles Steel H-Pile Driving Shoe) or driving shoes such as Titus Standard "H" Bearing Pile Point design for protection during driving as per OPSS 903 (Deep Foundations). Similarly, if steel tube piles are being considered, driving shoes should be in accordance with OPSD 3001.100 Type II (Steel Tube Pile Driving Shoe). The requirement for driving shoes should be included in the Contract Drawings.

If corrugated steel pipes (CSPs) are installed as part of the integral abutment design, the CSPs should be backfilled with loose, fine to medium sand. A Non Standard Special Provision (NSSP) detailing the installation method and gradation of this sand should be included in the Contract Documents; an example is included in Appendix D.

The pile caps for the new abutments should be provided with a minimum of 1.2 m of soil cover to provide adequate protection against frost penetration (as per OPSD 3090.101 – *Foundation Frost Depths for Southern Ontario)*.

6.3.3.1 Friction and End-Bearing Piles

For HP 310 x 110 piles driven to found within the hard clayey silt with sand till below a design tip Elevation 170 m for the new north abutment and 167 m for the new south abutment (i.e. piles about 18.5 m and 21.5 m long assuming the pile cap is founded at about Elevation 188.5 m), the factored geotechnical axial resistance at ULS may be taken as 900 kN. The geotechnical reaction at SLS (for 25 mm of settlement) may be taken as 700 kN.

The following note, or similar, should be shown on the Contract Drawing assuming that a resistance factor of 0.5 is applied to the use of the Hiley calculation based on MTO experience in the Southern Ontario region (Note 2 from the Structural Manual Section 3.3.3 (MTO, 2008)):

> *"Piles to be driven in accordance with Standard SS-103-11 using an ultimate geotechnical resistance of 1,800 kN per pile, but must be driven below the following tip elevations:*

- *New North Abutment: Elevation 170.0 m*
- *New South Abutment: Elevation 167.0 m*

Similar axial resistances and drawing notes may be used in the design for closed-end, concrete filled 324 mm (12 ¾ in.) diameter steel tube piles having a minimum wall thickness of 6.4 mm (¼ in.).

Pile installation should be in accordance with OPSS 903 *(Deep Foundations).* The pile termination or set criteria will be dependent on the pile driving hammer type, helmet, selected pile and length of pile; the criteria must therefore be established at the time of construction after the piling equipment is known. The pile capacity should then be verified in the field by the use of the Hiley formula (MTO Standard Drawing SS103-11) during the final stages of driving to achieve an ultimate capacity, as indicated in the Contract Drawing Notes above.

Assessment of ultimate geotechnical resistance by the Hiley formula should commence once the pile reaches a depth of not more than 1.5 m above the design pile tip elevation shown above and at 0.5 m intervals of depth until the ultimate axial resistance is achieved. If the ultimate capacity as determined by the Hiley formula is not achieved within the 1.5 m interval down to the design pile tip elevation, the Contractor should stop pile driving and notify the Contract Administrator. At this depth, the pile should be allowed to rest for 48 hours and the Hiley formula should then be applied immediately upon re-striking the pile. If the ultimate capacity is still not achieved after the 48 hour wait period, the Contract Administrator should be notified and authorization given prior to driving the pile below the design pile tip elevation.

Given the variability in the SPT "N"-values at variable depths, it is recommended that an allowance for greater pile lengths be provided in the Contract Documents to ensure that adequate pile lengths are available on site and to reduce splicing needs. It is also recommended that the axial capacity be calculated by the Hiley formula on every pile installed.

Given the stiff to hard general nature of the overburden soils and the net unloading condition at the new approach embankment locations (due to removal of the existing approach embankment fill), downdrag loads are not anticipated.

6.3.3.2 Resistance to Lateral Loads

Resistance to lateral loading can be derived using vertical piles, with enhanced support offered by battered piles, if required. If vertical piles are used, the resistance to lateral loading will have to be derived solely from the soil in front of the piles, whereas battered piles derive lateral resistance from the soil in front of the piles as well as the horizontal component of the axial load present in the inclined pile.

The resistance to lateral loading in front of the pile may be calculated using subgrade reaction theory where the coefficient of horizontal subgrade reaction (k_h in kPa/m) is determined based on the equations given below (CFEM 1992 as noted in Section 6.8.7.1 of the *Commentary* to the *CHBDC, 2006*):

For cohesionless soils:

$$k_h = \frac{n_h z}{B}$$

Where

k_h is the coefficient of horizontal subgrade reaction (kPa/m);
n_h is the constant of subgrade reaction (kPa/m);
z is the depth (m); and
B is the pile diameter or width (m).

For cohesive soils:

$$k_h = \frac{67 s_u}{B}$$

Where

k_h is the coefficient of horizontal subgrade reaction (kPa/m);
s_u is the undrained shear strength of the soil (kPa); and
B is the pile diameter or width (m).

Although not anticipated, where integral abutment design includes the installation of 3 m long CSP liners (with the annular space between the pile and the liner filled with uniform grained, uncompacted sand), the upper portion of the H-pile will be generally free to flex and move laterally within the limits of the CSP. With this design, the passive lateral resistance over the length of the pile within the CSP liner should be based on the resistance provided by loose sand. The passive lateral resistance on the exterior of the CSP should be based on the resistance provided by the surrounding soil conditions.

The following values of n_h and s_u may be assumed in the structural analyses. The soil stratigraphy has been generalized and the values reflect the variability in the subsurface conditions within the foundation elements footprint, however, the deposit boundaries vary slightly at the abutments and reference can be made to the borehole records and to the interpreted stratigraphic sections for each foundation element on Drawing 2 to assess the variation.

Soil Unit	**n_h (kPa/m)**	**s_u (kPa)**
Loose sand within CSP (if applicable)	2,200	-
Firm to hard clayey silt to silty clay fill	-	75
Stiff to very stiff clayey silt to silty clay	-	100
Stiff to hard clayey silt with sand till	-	150
Hard silty clay	-	200
Dense silty sand and gravel	11,000	-
Compact to very dense sand and silt till	9,000	-
Compact sand and gravel	4,400	-

A factored geotechnical lateral resistance of 155 kN at ULS, and a geotechnical lateral reaction of 95 kN at SLS (for 10 mm of horizontal deflection at pile cap level) was calculated for a vertical free-headed HP 310x110 pile (driven predominantly within the stiff to hard clayey silt with sand till) based on an analysis using the commercially available program *LPILE Plus* (Version 5.0) produced by Ensoft Inc. The structural capacity of the pile should be checked and verified by the structural engineer.

Group action for lateral loading should be considered where the pile spacing in the direction of the loading is less than six to eight pile diameters. Group action can be evaluated by reducing the coefficient of horizontal subgrade reaction in the direction of loading by a reduction factor, R (NAVFAC DM-7.02, 1986) as follows:

Pile Spacing in direction of Loading (d = Pile Diameter)	**Subgrade Reaction Reduction Factor (R)**
8d	1.00
6d	0.70
4d	0.40

Pile Spacing in direction of Loading (d = Pile Diameter)	Subgrade Reaction Reduction Factor (R)
3d	0.25

The subgrade reaction reduction factor should be interpolated for pile spacings in between those provided above.

6.3.4 Caisson Foundations

Consideration could be given to the use of caissons socketted into the hard clayey silt with sand till for support of the foundation elements for the new abutments.

If caisson foundations are adopted for support of any of the foundation elements, a temporary or permanent liner may be required to support the soils during construction, to minimize disturbance and loss of ground if water-bearing cohesionless soil zones/seams are encountered within the overburden, although limited cohesionless seams/zones were encountered during this current investigation. If there is water infiltration such that there is standing water within the caisson excavation prior to concrete placement, the concrete must be placed using tremie techniques. After initial placement of concrete at the bottom of the caisson, the tremie discharge point should be maintained below the surface of the wet concrete during placement. It is recommended that a Non-Standard Special Provision (NSSP) be included in the Contract Documents to address the need for control of the ground and groundwater during caisson construction as discussed further under Construction Considerations in Section 6.8. It is expected that the liner would be installed (and removed, if a temporary liner is used) using vibratory methods. In this case, vibration monitoring is recommended during liner installation and removal. The liner must be maintained tight to the sides of the bore to minimize seepage of water.

The performance of caissons will depend upon the final cleaning and verification of the subgrade quality (hard clayey silt with sand till) at the base of the caissons. Each caisson excavation should be carefully cleaned to remove all loosened debris to ensure that the concrete is in intimate contact with the competent bearing stratum. The Ontario Occupational Health and Safety Act (2012) outlines appropriate safety procedures and requirements that must be implemented prior to entry of personnel into the caissons for inspection of the base or alternatively, the inspections may be carried out remotely using visual recording equipment.

The caisson caps for the new piers should be provided with a minimum of 1.2 m of soil cover to provide adequate protection against frost penetration (per OPSD 3090.101 – *Foundation Frost Depths for Southern Ontario)* unless the caps are positioned at the top of the columns.

6.3.4.1 Founding Elevations

Caissons may be founded within the stiff to hard clayey silt with sand till deposit and socketted at least 1.5 m into the hard till deposit at an estimated caisson tip Elevation 170 m for the north and south abutments.

6.3.4.2 Geotechnical Resistances

The recommended design values for factored geotechnical axial resistance at ULS and geotechnical reaction at SLS (for 25 mm of settlement) for caissons founded at the elevations given in Section 6.3.4.1 are provided below.

Foundation Element	Founding Stratum	Design Caisson Tip Elevation	Caisson Diameter	Factored Geotechnical Axial Resistance at ULS	Geotechnical Reaction at SLS (for 25 mm settlement)
New North and South Abutments	Hard clayey silt with sand till	170.0 m	0.9 m	2,000 kN	1,600 kN
			1.2 m	4,000 kN	3,250 kN
			1.5 m	7,000 kN	5,000 kN

6.3.4.3 Resistances to Lateral Loads

Resistance to lateral loading will be derived from the soil in front of the caissons. The resistance to lateral loading in front of the caisson may be calculated using subgrade reaction theory and the equations and soil parameters provided in Section 6.3.3.2 may be used for design.

6.4 Retained Soil System (RSS) Walls

It is understood that mechanically-reinforced soil retaining systems (retained soil system or RSS walls) are proposed as wing walls/retaining walls on both sides of the north and south abutments (refer to Drawing 1). Based on the GA drawing provided by AECOM, the RSS wall foundations are stepped to follow the approximate 2H:1V embankment front slope. As a result, the foundations are proposed to be founded at elevations ranging from about 191.0 m to 185.5 m.

6.4.1 Founding Elevations

A typical RSS wall has a front facing supported on a strip footing placed at shallow depth below the ground surface in front of the wall. The existing embankment fills generally extend from the Mavis Road ground surface to between about Elevation 184.4 m and 183.8 m near the north and south abutments, respectively. The existing clayey silt to silty clay fill contains variable amounts of organics and construction debris. It is recommended that the RSS soil mass and facing footing be supported on the native soils (below the existing fills) or compacted (minimum 98% SPMDD) suitable earth fills. As a result, RSS walls should be founded at or below Elevation 184.0 m at the north abutment and at or below Elevation 183.8 m at the south abutment, extending below any existing fill, organics or soft/loose materials in these areas.

The facing footing should be placed on a 300 mm thick layer of compacted SP 110S13 Granular ‘A’, as shown in Figure 5.2 in the MTO RSS Wall Design Guidelines (September 2008). The compacted granular pad should extend at least 1.0 m beyond the outside edge of the facing footing, then downward at 1H:1V. Where sub-excavation of fill and unsuitable soils has been carried out, the Granular ‘A’ pad and the reinforced soil mass can be constructed immediately on top of the native subgrade, such as the stiff to very stiff clayey silt to silty clay at the north abutment and stiff to hard clayey silt with sand till at the south abutment. Alternatively, the thickness of the granular pad can be increased to raise the grade after sub-excavation and the facing footing and reinforced soil mass founded at a higher elevation.

Since the height of the RSS walls foundations are proposed to be stepped into the existing approach embankment fills (i.e. RSS walls height decreases with increasing distance away from the bridge north and south abutments), consideration could be given to partial sub-excavation of the firm to hard clayey silt to silty clay fill soils to 1.5 m below the proposed underside of the facing footing for the RSS wall, and he RSS wall facing footing could be supported on a 1.5 m thick granular pad constructed of compacted Granular ‘A’ or Granular ‘B’ Type II material SP110S13 (Aggregates) on the existing clayey silt to silty clay fill. This compacted granular pad should extend at least 1.0 m beyond the outside edge of both sides of the facing footing, then outward/downward at 1H:1V.

The compacted Granular ‘A’ pad and the reinforced soil mass should be keyed into the existing embankment fills by benching into the embankment fill, similar to OPSD 208.010 (Benching of Earth Slopes).

6.4.2 Geotechnical Resistances

Assuming that the RSS wall acts as a unit and uses the full width of the reinforced soil mass (assumed to be about 70% of the retained height), the factored geotechnical resistance at ULS and the geotechnical reaction at SLS (for 25 mm of settlement) given below may be used for design of the reinforced mass and facing footing founded on the native subgrade or compacted granular fills on native subgrade) at the sub-excavation elevations given in Section 6.4.1.

Wall Height	Factored Geotechnical Resistance at ULS	Geotechnical Reaction at SLS (for 25 mm Settlement)	Founding Soil Condition
9.5 m	500 kPa	300 kPa	Granular pad on native subgrade
4m to 9.5 m	450 kPa	250 kPa	Granular pad on existing embankment fills

6.4.3 Global Stability

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

Drained and undrained analyses were carried out for the slope stability assessment. The critical soil parameters used in the analysis, as given below, were estimated from empirical correlations using the results of in-situ Standard Penetration Tests (SPTs) (Bowles, 1984) and geotechnical classification testing. The groundwater table was modelled at Elevation 177 m in the analyses.

Soil Type	Unit Weight (kN/m^3)	Undrained Shear Strength (kPa)	Angle of Internal Friction, φ' (degrees)
Existing embankment (clayey silt to silty clay) fill	20	75	-
Stiff to very stiff clayey silt to silty clay	21	100	-
Stiff to hard clayey silt with sand till	21	150	-
Granular 'A' pad (proposed)	21	0	34

A RSS wall section was analyzed for the maximum wall height anticipated as shown on the GA drawing provided by URS, dated July 25, 2012. In this analysis, the height of the RSS wall was considered to extend from the top of the Mavis Road pavement (Elevation 195.2 m) to the underside of the lowest RSS panel footing (top of the front facing footing) Elevation 185.7 m.

Given the proposed RSS wall height, the minimum reinforced width of RSS wall required to obtain a factor of safety equal to 1.5 or greater against deep-seated global instability has been calculated. The ratio of minimum reinforced mass width to reinforced wall height for this section is the "typical" ratio that is used by wall designers (i.e. approximately 0.7 times the wall height). The contract drawings will need to specify the width of the reinforced soil mass. The result of the analysis for the RSS wall adjacent to the abutment wall (about 9.5 m high wall) is shown on Figure 2 for the static condition, and indicates a factor of safety greater than 1.5 for global instability.

Under seismic loading conditions, using a design seismic coefficient equal to the 50 per cent of the site-specific design peak horizontal ground acceleration (PGA) which is about 0.03 g, the Factor of Safety is greater than 1.1. The result of the seismic slope stability analysis for the RSS wall (about 9.5 m high) is shown on Figure 3.

6.4.4 Resistance to Lateral Loads

The resistance to lateral forces / sliding resistance between the compacted granular fill of the RSS wall and the subgrade should be calculated in accordance with Section 6.7.5 of the *CHBDC*. The coefficient of friction, tan φ', between the compacted granular fill of the RSS wall and the properly prepared subgrade may be taken as 0.55.

6.5 Lateral Earth Pressures for Design

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

The following recommendations are made concerning the design of the abutment walls. These design recommendations and parameters assume level backfill and ground surface behind the abutment walls. Where there is sloping ground behind the stems/walls, the coefficient of lateral earth pressure must be adjusted to account for the slope.

- Select, free draining granular fill in accordance with SP 110S13 (Aggregates) Granular ‘A’ or Granular ‘B’ Type II but with less than 5 per cent passing the 200 sieve should be used as backfill behind the walls. Compaction (including type of equipment, target densities, etc.) should be carried out in accordance with OPSS 501 (*Compacting*). Longitudinal drains and weep holes should be installed to provide positive drainage of the granular backfill. Other aspects of the granular backfill requirements with respect to sub-drains and frost taper should be in accordance with OPSD 3101.150 – *Wall, Abutments Backfill* and OPSD 3121.150 – *Walls Retaining, Backfill.*

- A minimum compaction surcharge of 12 kPa should be included in the lateral earth pressures for the structural design of the wall stem, in accordance with *CHBDC* Section 6.9.3 and Figure 6.6. Compaction equipment should be used in accordance with OPSS 501. Other surcharge loadings should be accounted for in the design as required.

- For restrained structures, the granular fill may be placed either in a zone with the width equal to at least 1.2 m behind the back of the walls (see Case A in Figure C6.20 (a) of the *Commentary* to the *CHBDC*). For unrestrained structures, the granular fill should be placed within the wedge shaped zone defined by a line drawn at 1.5 horizontal to 1 vertical (1.5H:1V) extending up and back from the rear face of the footing (see Case B in Figure C6.20(b) of the *Commentary* to the *CHBDC*).

- For restrained structures, the pressures are based on the existing embankment fill materials and the following parameters (unfactored) may be used assuming the use of the native clayey silt to silty clay fill :

	Earth Fill
Soil Unit Weight	20 kN/m^3
Coefficient of static lateral earth pressure Active, K_a At rest, K_o	 0.33 0.50

- For unrestrained structures, where the pressures are based on SP 110S13 granular fill behind the wall, the following parameters (unfactored) may be assumed:

	Granular ‘A’	**Granular ‘B’ Type II**
Soil Unit Weight	22 kN/m^3	21 kN/m^3
Coefficient of static lateral earth pressure Active, K_a At rest, K_o	 0.27 0.43	 0.27 0.43

If the wall support and superstructure allow lateral yielding of the stem, active earth pressures may be used in the geotechnical design of the structure. If the abutment support does not allow lateral yielding (such as for a rigid frame structure), at-rest earth pressures should be assumed for geotechnical design. The movement required to allow active pressures to develop within the backfill, and thereby assume an unrestrained structure for design, should be calculated in accordance with Section C6.9.1 and Table C6.6 of the *Commentary* to the *CHBDC*.

A restrained structure is typically a concrete box culvert or a rigid frame bridge structure where the rotational and/or horizontal movement is not sufficient to mobilize the active pressure condition. For this condition, an at-rest pressure plus any compaction surcharge should be included in the design of the structure.

6.6 Seismic Site Coefficient

For seismic design purposes, the Site Coefficient (S) for this site may be taken as 1.2, consistent with Soil Profile Type II. The soil profile is based on the guidelines in Section 4.4.6 and Table 4.4 of the *CHBDC (2006)* and local experience.

6.6.1 Seismic Analysis Coefficient

The potential for seismic (earthquake) loading may also need to be considered for the design of new abutment stems/reinforced soil mass and for the assessment of liquefaction potential of foundation soils in accordance with Section 4.6 of the *CHBDC,* as significant seismic loading will result in increased lateral earth pressures acting on the abutment stem and reinforced soil mass systems. At this site, the requirements for seismic analysis are outlined as follows:

According to Table A3.1.1 of the *CHBDC*, this site is located in Seismic Zone 1. The site-specific zonal acceleration ratio for the City of Mississauga is 0.05. Based on experience, for the subsurface conditions at this site, a 20 per cent amplification of the ground motion may occur (i.e. Site Coefficient, S=1.2 for Soil Profile II from Table 4.4 of *CHBDC*), resulting in an increase in the peak horizontal ground acceleration (PHA) from 0.05 g to 0.06 g at the ground surface and a design seismic coefficient value of 0.03 g (50% of the PGA). Based on Section 4.4.4 of the *CHBDC*, this bridge structure is assigned Seismic Performance Zone 1. Given this, and in accordance with Section 4.4.5.1 of the *CHBDC*, no seismic analysis is required for structures located in Seismic Performance Zone 1.

6.7 Bridge Approaches

The existing Mavis Road structure approach embankments are in a fill over the proposed Highway 401 widening footprint. The existing Mavis Road grade in the general area of the underpass varies from approximately Elevation 195.4 m to 195.2 m. The existing Highway 401 pavement surface at the underpass varies from about Elevation 187.5 m to 187.2 m. For the proposed rehabilitation of the underpass, which involves constructing two new abutments further north and south of the existing bridge to accommodate the Highway 401 widening, the Mavis Road and Highway 401 grades are proposed to be at about Elevation 195.2 m and Elevation 187.2 m, respectively. As a result, there is essentially no grade change planned at the existing highway embankment at the underpass structure.

The proposed rehabilitation (lengthening) of the bridge will require removal (i.e. deep cuts) of the existing approach embankments to about 28 m north and south of the existing abutments. It is considered that removal of the existing embankment fill will result in unloading the soil at the new abutments and, considering that there is no significant grade raise at this structure, the stability of the approach embankment will not be affected nor will there be any significant settlements. The existing embankment side-slopes are sloped at about 2H:1V and it is assumed the new embankment side-slopes will be maintained at 2H:1V or shallower.

Further verification of the global stability of the new approach embankment at the maximum height, using the commercially available program Slide (produced by Rocscience Inc.) and soil parameters estimated in Section 6.4.3, confirms a target minimum factor of safety of 1.3 is achieved for the proposed embankment and geometries.

In addition, considering the site will essentially be “unloaded”, the total settlement of the foundation soil due to approach embankment loadings, is expected to be less than 25 mm. Granular fill should be used to backfill any excavated portions behind the new abutments and approach embankment to minimize the potential for additional settlement as described in Section 6.7.1.

Recommendations pertinent to embankment construction and subgrade preparation are provided below.

6.7.1 Subgrade Preparation and Embankment Construction

The existing embankments have been constructed using cohesive fill which was encountered at all the boreholes during this current investigation. To reduce the potential for post-construction differential settlement and to achieve stability of the overall embankment, it is recommended that prior to the placement of any additional fill, all topsoil, organic matter and soft/loose fill should be stripped from below the approach embankment areas. Any new embankment fill should be placed and compacted in accordance with SP 206S03 (Earth Excavation and Grading) and OPSS 501 (Compacting), with inspection and field density testing by qualified personnel during placement operations to confirm that appropriate materials are used and that adequate levels of compaction are achieved.

The use of granular fill behind the abutment walls is recommended rather than the reuse of the existing cohesive fill, since the majority of settlement of granular fills would occur during construction whereas some settlement of cohesive fills, if used, would occur post-construction.

Upon completion of the embankment construction, topsoil and seeding or pegged sod is recommended to be placed as soon as practicable after construction of the embankments to reduce the potential for erosion of the embankment side slopes due to surface water runoff and to establish vegetation within the affected portion of the slopes. Topsoil should be placed on granular fill side-slopes in accordance with OPSS 802 (*Topsoil*) and covered with erosion protection in accordance with OPSS 804 *(Seed and Cover)* or pegged sod in accordance with OPSS 803 *(Sodding)*. Topsoil and erosion protection should be placed in early summer to avoid wet periods of the year which may cause surficial sloughing of the topsoil material along the side-slopes and to establish vegetation prior to the Fall / Winter months.

6.8 Construction Considerations

6.8.1 Excavation and Groundwater Control

The foundation excavations at the abutments for spread footings or pile cap construction will extend to depths of about 11 m to 12 m below the present Mavis Road grade, through the existing fill into the stiff to hard clayey silt with sand till. Where space permits, open-cut excavations into these materials should be carried out in accordance with the guidelines outlined in the Occupational Health and Safety Act (OHSA) for Construction Activities. The existing fill is classified as Type 3 soil and the till is classified as Type 2 soil, according to the OHSA. Temporary excavations (i.e. those that are open for a relatively short time period) should be made with side slopes no steeper than 1 horizontal to 1 vertical (1H:1V) through the fill materials and through the till to within 1.2 m of the bottom of the excavation.

Excavations for the new north and south abutment foundations will be maintained above the groundwater level, and groundwater inflow is expected to be relatively minor, especially during drier periods of the year. Some water inflow should be expected into the foundation excavations, particularly during wet months; however, it is anticipated that water inflow can be handled by pumping from filtered sump pumps placed at the base of the excavations.

6.8.2 Temporary Excavation Support

Temporary excavation support may be required to facilitate the construction of the new abutments and RSS walls at the Mavis Road Underpass and to maintain traffic lanes in operation during the construction period. The temporary excavation support systems should be designed and constructed in accordance with OPSS 539 *(Temporary Protection Systems).* The lateral movement of the temporary shoring system should meet Performance Level 2 as specified in OPSS 539, provided that the existing structure, as well as any adjacent utilities, can tolerate this magnitude of deformation.

The protection system is required for an estimated maximum excavation depth of approximately 12 m (i.e., the difference in elevation between the Mavis Road and Highway 401 grades plus excavation below the Mavis Road pavement thickness). It is considered that either a driven, interlocking sheetpile system or a soldier pile and timber lagging system would be suitable for the temporary excavation support at this site, based on the inferred subsurface soil conditions and groundwater conditions; however the presence of cobbles and boulders may affect the installation of the interlocking sheet pile system. An interlocking sheetpile system would contribute to both ground and, where applicable, groundwater control of seepage from cohesionless zones or interlayers/lenses within the cohesive deposits that may be required. For a soldier pile and lagging system, it would be necessary to control seepage or include measures to mitigate loss of soil particles through the lagging boards if cohesionless soils/lenses are encountered.

The sheetpiles or soldier piles would have to be socketted to sufficient depth to provide the necessary passive resistance for the retained soil height of up to about 12 m. Lateral support to the sheetpiles or soldier piles could be provided in the form of rakers or temporary anchors. The selection and design of the protection system will be the responsibility of the Contractor.

6.8.3 Subgrade Protection

The clayey silt with sand till that will be exposed at the foundation subgrade level will be susceptible to disturbance from construction traffic and/or ponded water. To limit this degradation, it is recommended that a concrete working slab be placed on the subgrade within four hours after preparation, inspection and approval of the footing subgrade. This requirement can be addressed with a note on the drawings and/or with an NSSP. An example NSSP for the concrete working slab is included in Appendix D.

6.8.4 Vibration Monitoring During Pile Installation

Depending on the construction sequence, vibration monitoring may be necessary at the existing structure during driven pile or caisson installation, if piles or caissons are selected, to ensure that the vibration levels at the existing structure are maintained below tolerable levels. A maximum peak particle velocity (PPV) of 100 mm/s is generally considered applicable for bridge structures in good condition (i.e. considering portions of the existing structure and foundations are to remain permanently in place); however, this requires further assessment by the structural engineer. The piles further from the existing underpass structure should be driven/installed first, in order to monitor the vibration level at the existing structure and, if necessary, alter the pile driving criteria for caisson installation for the remaining piles/caissons. As there are several residential and commercial structures in the vicinity of the site, monitoring of vibrations during construction should be considered by the general contractor to defend against potential damage claims by the owners of the nearby structures.

In the event that vibration monitoring is determined to be necessary, an example NSSP for such monitoring is provided in Appendix D for inclusion in the Contract Documents.

6.8.5 Obstructions During Pile Driving / Caisson Installation

Cobbles and/or boulders were encountered and inferred due to difficulty to augering at varying depths in the boreholes drilled during the current subsurface investigation, which may affect the installation of steel H-piles/tube piles or caissons. It is recommended that driving shoes be used on all steel H-piles or tube piles to facilitate driving into the overburden soils. In addition it is recommended that an NSSP be included in the Contract Documents to warn the Contractor of the possible presence of cobbles and/or boulders within the overburden soils and an example NSSP is presented in Appendix D.

7.0 CLOSURE

This Foundation Design Report was prepared by Ms. T. Veronica Ayetan, P.Eng., a geotechnical engineer with Golder , and reviewed by Mr. Kevin Bentley, P.Eng., a senior geotechnical engineer and Associate with Golder. Mr. Ty Garde, P.Eng., a Designated MTO Contact and Principal with Golder, conducted an independent review and quality control audit of this report.

GOLDER ASSOCIATES LTD.

LICENSED PROFESSIONAL ENGINEER
Feb 7, 2013
T. V. AYETAN
100130071
PROVINCE OF ONTARIO

T. Veronica Ayetan, P.Eng
Geotechnical Engineer

Kevin Bentley, P.Eng.
Geotechnical Engineer, Associate

REGISTERED PROFESSIONAL ENGINEER
Feb 7, 2013
T. J. GARDE
PROVINCE OF ONTARIO

Ty Garde, P.Eng
Designated MTO Contact, Principal

TVA/KJB/TG/tva/jl

n:\active\2010\1111\10-1111-0211 aecom-hwy 401 widening-mississauga\9 - reports\1 - mavis road underpass\final\10-1111-0211 rpt01 13feb 07 mavis road underpass.docx

REFERENCES

Bowles, J.E., 1984. *Physical and Geotechnical Properties of Soils*, Second Edition. McGraw Hill Book Company, New York.

Canadian Geotechnical Society, 2006. *Canadian Foundation Engineering Manual*, 4th Edition. The Canadian Geotechnical Society, BiTech Publisher Ltd., British Columbia.

Canadian Geotechnical Society, 1992. *Canadian Foundation Engineering Manual*, 3rd Edition. The Canadian Geotechnical Society, BiTech Published Ltd., British Columbia.

Canadian Standards Association (CSA), 2006. *Canadian Highway Bridge Design Code* and *Commentary on CAN/CSA-S6-06*. CSA Special Publication, S6.1 06.

Chapman, L.J., and Putnam, D.F., 1984. *The Physiography of Southern Ontario*. Ontario Geological Survey, Special Volume 2, 3rd Edition. Ontario Ministry of Natural Resources.

Ministry of Transportation, Ontario, 2008. Structural Manual. Provincial Highway Management Division, Highways Standards Branch, Bridge Office.

NAVFAC, 1986. *Design Manual DM 7.02: Soil Mechanics, Foundation and Earth Structures*. U.S. Navy. Alexandria, Virginia.

Occupational Health and Safety Act and Regulations for Construction Projects, January 2012.

Ministry of Transportation Engineering Standards Branch. RSS Design Guidelines. September 2008

Ontario Provincial Standard Specifications (OPSS)

OPSS 501	Construction Specifications for Compacting
OPSS 539	Construction Specification for Temporary Protection Systems
OPSS 802	Construction Specification for Topsoil
OPSS 803	Construction Specification for Sodding
OPSS 804	Construction Specification for Seed and Cover
OPSS 902	Construction Specification for Excavating and Backfilling Structures
OPSS 903	Construction Specification for Deep Foundations

Ontario Provincial Standard Drawings (OPSD)

OPSD 208.010	Benching of Earth Slopes
OPSD 3000.100	Foundation, Piles, Steel H-Pile Driving Shoe
OPSD 3001.100	Foundation, Piles, Steel Tube Pile Driving Shoe
OPSD 3090.101	Foundation Frost Depths for Southern Ontario
OPSD 3101.150	Walls, Abutments, Backfill, Minimum Granular Requirements
OPSD 3121.150	Walls, Retaining, Backfill, Minimum Granular Requirements

Construction Design Estimating and Documentation (CDED) Special Provisions (SP)

SP110S13	Material Specification for Aggregates – Base, Subbase, Select Subgrade and Backfill Material
SP 206S03	Earth Excavation and Grading; Excavation for Pavement Widening
SP 105S21	Amendment to OPSS 501

ASTM International

ASTM D1586	Standard Test Method for Standard Penetration Test (SPT) and Split Barrel Sampling of Soils

TABLE 1 - COMPARISON OF FOUNDATION ALTERNATIVES
MAVIS ROAD UNDERPASS, HIGHWAY 401 WIDENING
G.W.P. 2150-01-00

Foundation Option	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Strip or Spread Footing on stiff to hard clayey silt with sand till.	• Lower costs than deep foundations; • Standard construction methods; no specialized construction equipment required; and • Allows for semi-integral abutments.	• Although required to reach the proposed Hihway 401 widening grade, between about 11 m and 12 m depth excavation through the existing approach embankment fill and native soil would be required (i.e. abut 4 m below proposed Highway 401 grade); • Traffic protection system required during construction; • Lower bearing geotechnical resistances compared to deep foundation options; and • Does not allow for integral abutment construction.	• Lower relative costs than deep foundations; and • Additional cost for sub-excavation of existing embankment fill and native soil. • (4 m wide x 42 m long x 1.2 m thick) at \$ 600/m^3 + (4 m deep x 14 m wide x 42 m long) at \$ 10/m^3 = \$ 144,480/foundation element.	• Potential traffic disruption during construction.
Strip or Spread Footing "perched" on Granular 'A' pad in approach embankment fill.	• Reduce depth of existing embankment excavation compared to footings on native material.	• Traffic protection system required during construction; • Lower bearing capacities compared to other foundation options; • Longer bridge span may be required; and • Does not allow for integral abutment construction.	• Lowest cost option; and, • Relatively lower cost for excavation of existing embankment fill. • (4 m wide x 42 m long x 1.2 m thick) at \$ 600/m^3 = 121,000/foundation element plus Granular 'A' Pad.	• Potential traffic disruption during construction; and • Presence of organics, wood and construction debris in existing fill may result in post-construction settlement/creep.
Steel H-Piles driven to found within hard clayey silt with sand till.	• Higher geotechnical resistances, compared to shallow foundations; • Negligible post-construction settlement; and • Allow for support of semi-integral or integral abutments.	• Potential vibrations may be induced on existing abutment footings (new north and south piers) from pile driving operations; • Requires 42 m long excavation for pile cap; • Traffic protection system required during construction with complex construction staging; • Long piles may be required to reach hard materials; and • Requirement for temporary supports.	• Higher cost than shallow foundations; and • Installation costs could be impacted by presence of obstructions. • Assume (40 piles x 21.5 m long) at \$ 250/m = \$ 215,000/abutment plus excavation and pile cap costs of \$ 140,000/abutment.	• Potential traffic disruption during construction; • Potential vibrations may be induced on existing abutment footings (new north and south piers); • Risk of encountering obstructions that could impact pile installation; and • Potentially less costly maintenance over life of the structure than semi-integral abutment structures.
Steel Tube Piles (closed-end, concrete filled) driven to found in hard clayey silt with sand till.	• Higher geotechnical axial resistance, compared to spread footings; • Negligible post-construction settlement; and • Can be used for support of semi-integral or integral abutments but likely not suitable for integral abutments due to greater rigidity.	• Requires sub-excavation for cap construction; • Traffic protection system required during construction; • Long piles may be required to reach hard materials; • Greater disturbances to immediately adjacent ground due to larger base area if end is closed; • Requires staged construction for driving, cleaning and concrete filling of tube; and • Greater potential for crumpling if obstructions encountered.	• Higher cost than shallow foundations; • Cost for steel tube (pipe) piles slightly higher than for steel H-piles; and • Installation costs could be impacted by presence of obstructions. • Assume same cost as steel H-piles = \$ 355,000.	• Potential traffic disruption during construction; and • Slightly greater risk than for steel H-pile foundations if obstructions (cobbles and/or boulders) are encountered during driving; resulting in piles "hanging up".

Foundation Option	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Caissons founded within hard clayey silt with sand till	• Higher geotechnical resistances per unit compared to spread footings and piles; so reduced number of deep foundation elements compared to steel H- or tube piles. • Negligible post-construction settlement; and • No excavation required for pile cap.	• There may be need for temporary or permanent liners; • Cleaning of the base below the water table could be difficult; • Potential requirement for placement of concrete by tremie method; • Traffic protection system required during construction; • Not suitable for integral abutment design; and • Greater risk of encountering obstructions due to larger size of drill hole required.	• Higher cost than steel H-piles and tube piles; and • Installation cost could be impacted by need for liner to minimize disturbance and loss of ground and for tremie concrete placement. • Assume (15 caissons x 21.5 m long) at $ 2,000/m^3 = $ 645,000 per abutment.	• Risk of disturbance of water-bearing soils, if encountered, requiring special construction procedures including use of temporary or permanent liners; • Significant traffic disruption during construction due to space required for caisson drilling equipment; and • Risk of encountering obstructions that could impact caisson installation/costs.

Prepared By: TVA

Reviewed By: KJB/TG

MINISTRY OF TRANSPORTATION, ONTARIO

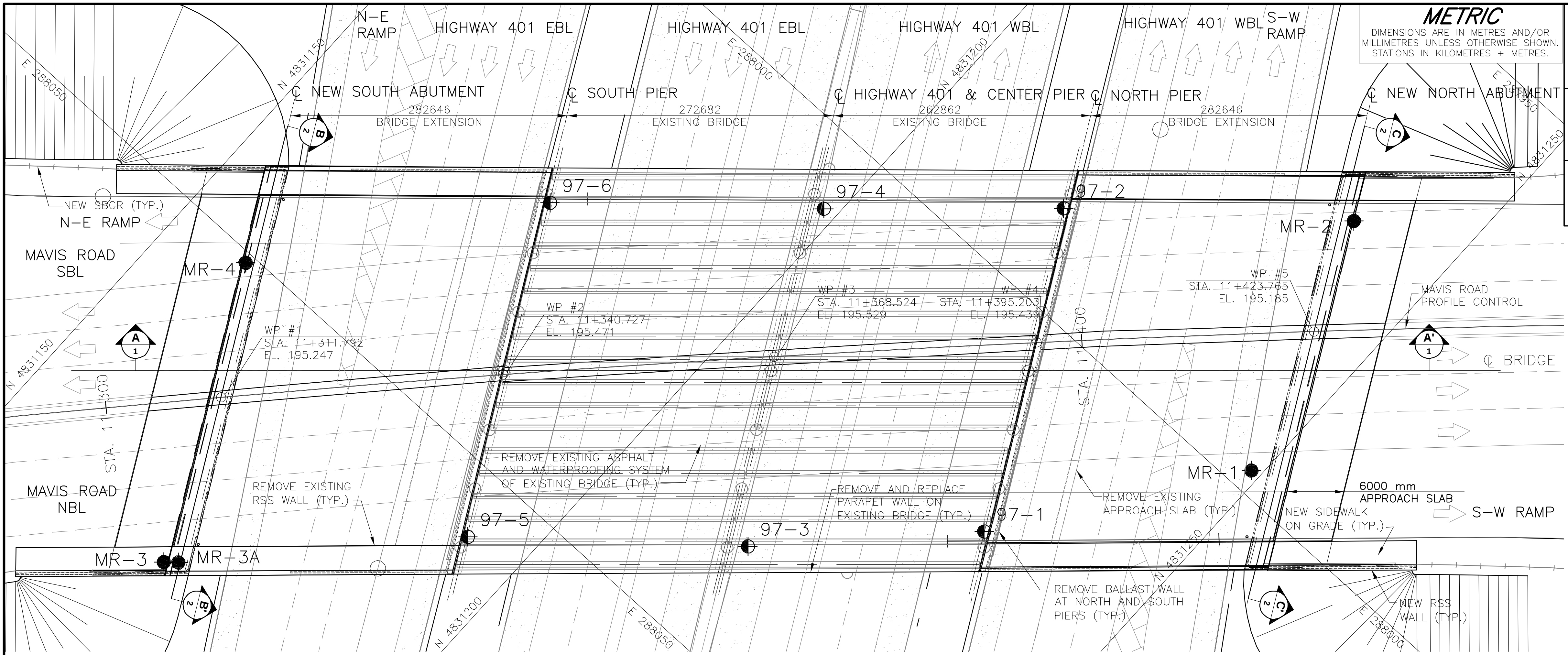
CONT No.
GWP No. 2150-01-00

HIGHWAY 401
MAVIS ROAD UNDERPASS
BOREHOLE LOCATION AND SOIL STRATA

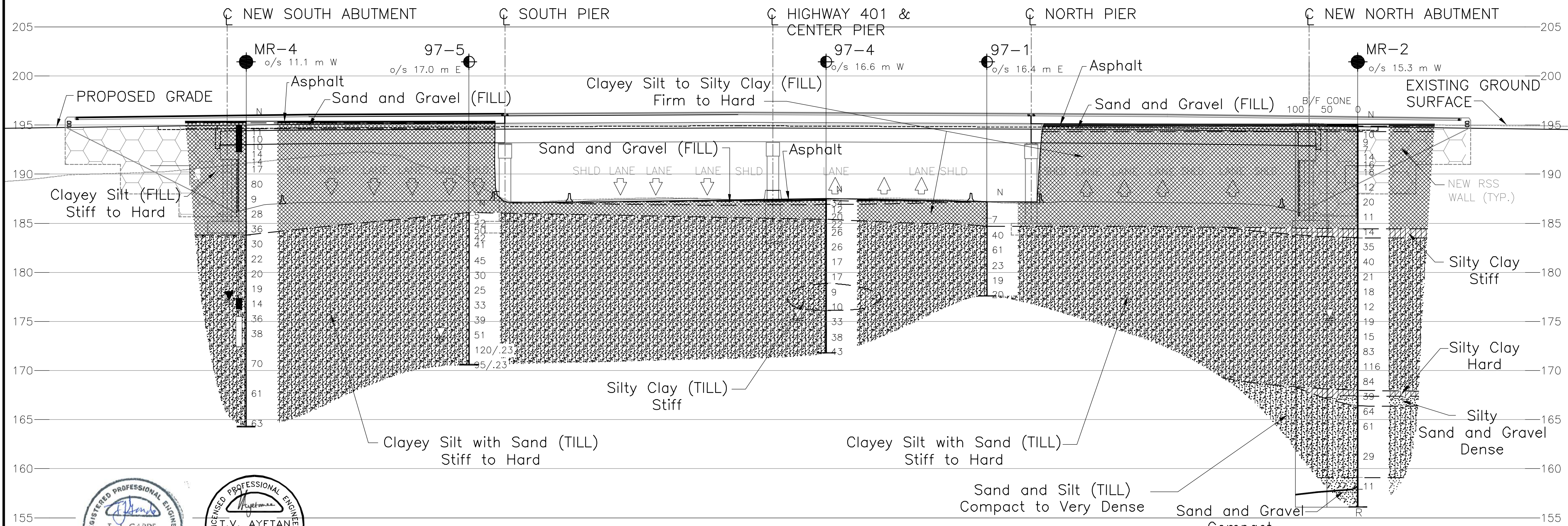
SHEET

Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA

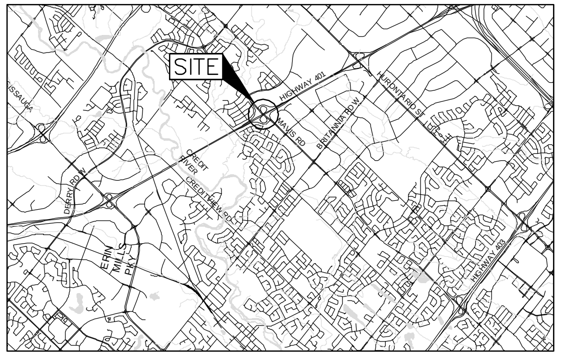
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.



PLAN
SCALE
5 0 5 10 m



A-A' CENTRELINE PROFILE
SCALE
5 0 5 10 m



KEY PLAN
SCALE
1.5 0 1.5 3 km

LEGEND

●	Borehole – Current Investigation
⊕	Borehole – Previous Investigation (Terraprobe, 1998)
▮	Seal
▯	Piezometer
N	Standard Penetration Test Value
16	Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
▼	WL in piezometer, measured on August 10, 2012.
▽	WL upon completion of drilling
R	Refusal

BOREHOLE CO-ORDINATES

No.	ELEVATION	NORTHING	EASTING
97-1	187.2	4831232.9	288020.7
97-2	186.6	4831216.8	287990.7
97-3	186.8	4831216.0	288038.0
97-4	187.5	4831198.6	288007.2
97-5	186.1	4831194.1	288056.5
97-6	185.2	4831177.4	288025.5
MR-1	194.7	4831249.0	287997.7
MR-2	195.1	4831239.6	287971.8
MR-3	194.6	4831172.8	288079.3
MR-3A	194.6	4831173.9	288078.3
MR-4	195.4	4831158.4	288051.0

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

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

REFERENCE
Base plans provided in digital format by AECOM, drawing file no. 60213979_ST-24-736-1 GA_30%.dwg, received July 25, 2012.

T. J. GARDE
Feb. 7, 2013

T.V. AYETAN
100130071
Feb. 7, 2013

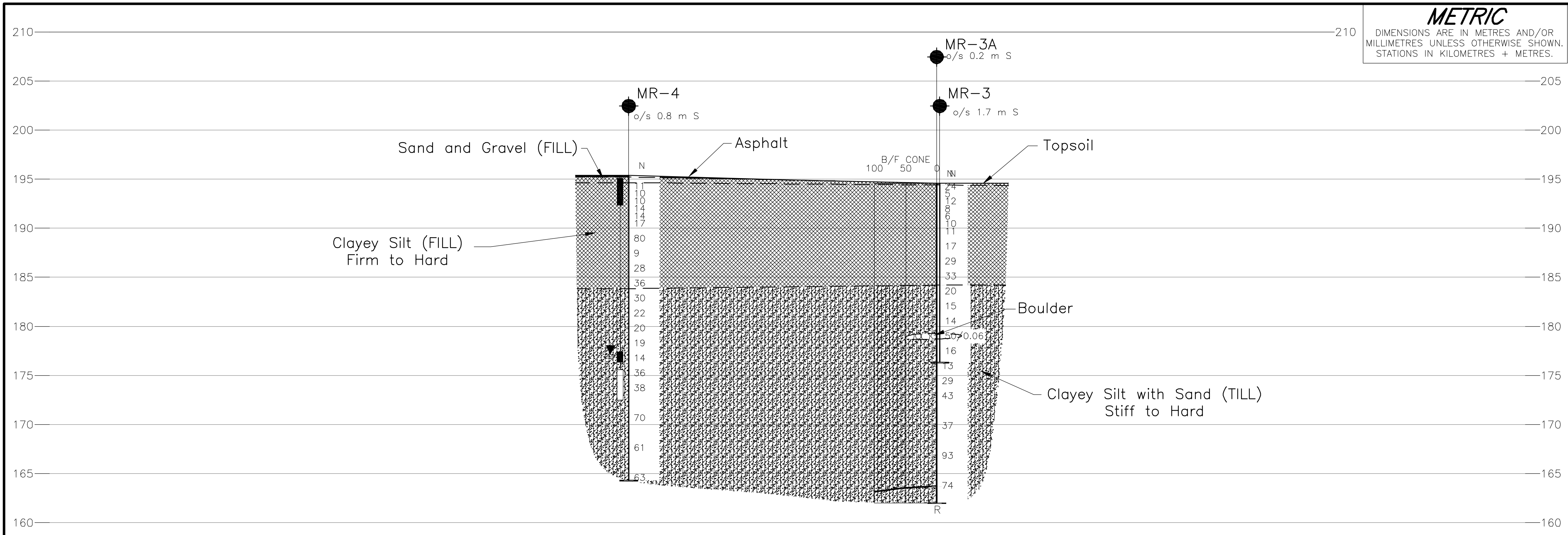
NO.	DATE	BY	REVISION

Geocres No. 30M12-355

HWY. 401	PROJECT NO. 10-1111-0211		DIST.
SUBM'D. TVA	CHKD. TVA	DATE: Feb, 2013	SITE:
DRAWN: JFC	CHKD. KJB	APPD. TG	DWG. 1

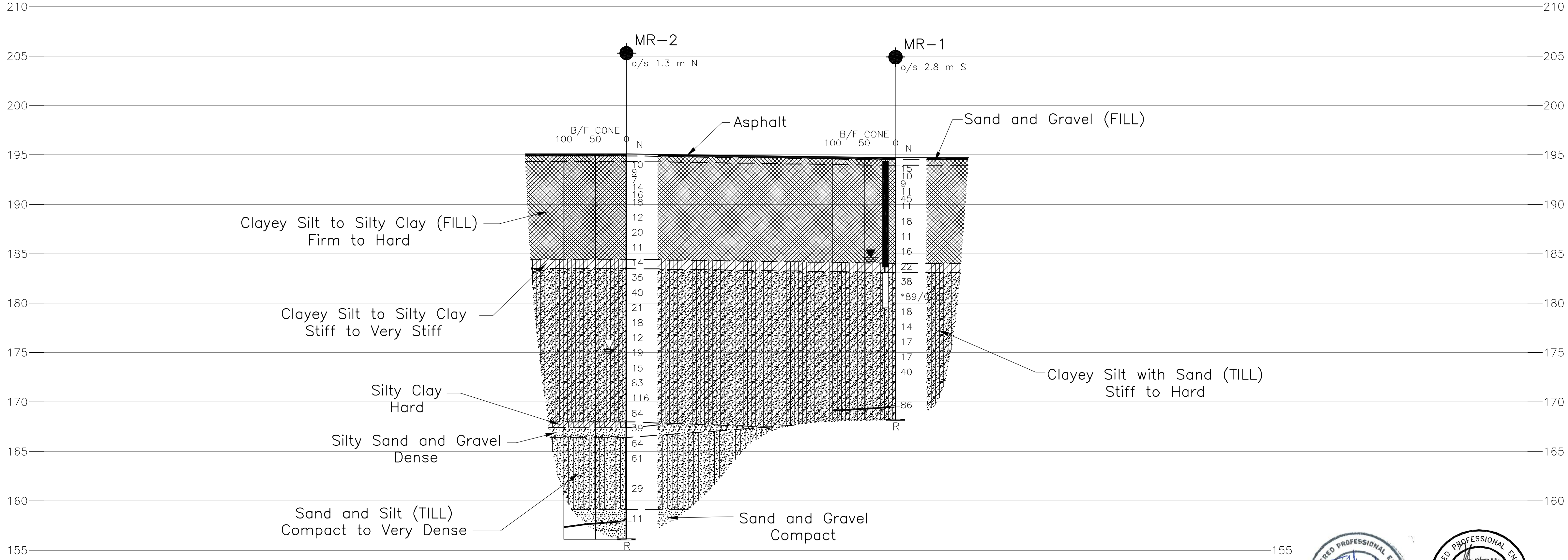
PLOT DATE: February 7, 2013
FILENAME: T:\Projects\2010\10-1111-0211 (AECOM, Mississauga)\-AB- Mavis Road\1011110211AB01.dwg

MINISTRY OF TRANSPORTATION, ONTARIO



B-B' 1 NEW SOUTH ABUTMENT

SCALE
5 0 5 10 m



C-C' 1 NEW NORTH ABUTMENT

SCALE
5 0 5 10 m

T. J. GARDE
Feb. 7, 2013

T.V. AYETAN
100130071
Feb. 7, 2013

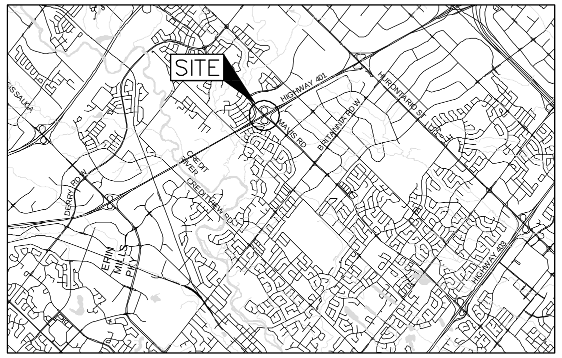
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

CONT No.
GWP No. 2150-01-00

HIGHWAY 401
MAVIS ROAD UNDERPASS
SOIL STRATA

SHEET

Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



KEY PLAN
SCALE
1.5 0 1.5 3 km

LEGEND

●	Borehole – Current Investigation
⊕	Borehole – Previous Investigation (Terraprobe, 1998)
▮	Seal
▯	Piezometer
N	Standard Penetration Test Value
16	Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
▼	WL in piezometer, measured on August 10, 2012.
▽	WL upon completion of drilling
R	Refusal

BOREHOLE CO-ORDINATES

No.	ELEVATION	NORTHING	EASTING
MR-1	194.7	4831249.0	287997.7
MR-2	195.1	4831239.6	287971.8
MR-3	194.6	4831172.8	288079.3
MR-3A	194.6	4831173.9	288078.3
MR-4	195.4	4831158.4	288051.0

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

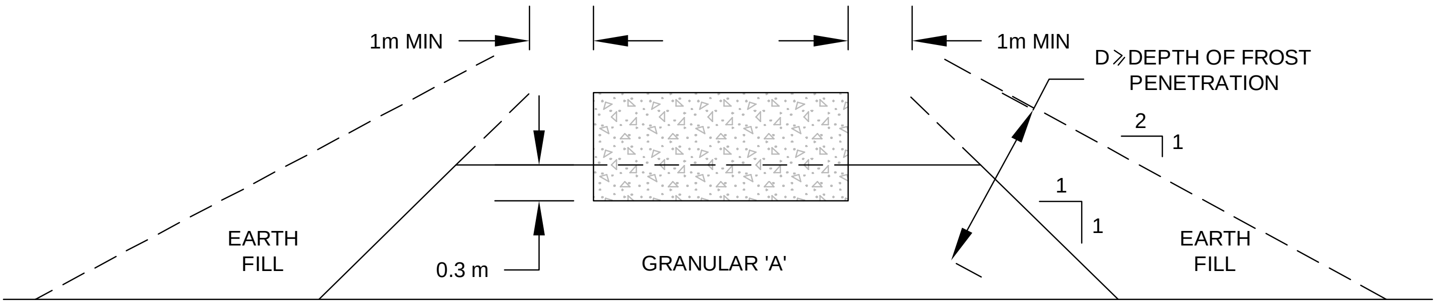
The complete Foundation Investigation and Design Report for this project and other related documents may be examined at the Materials Engineering and Research Office, Downsview. Information contained in this report and related documents is specifically excluded in accordance with Section GC 2.01 of OPS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

NO.	DATE	BY	REVISION

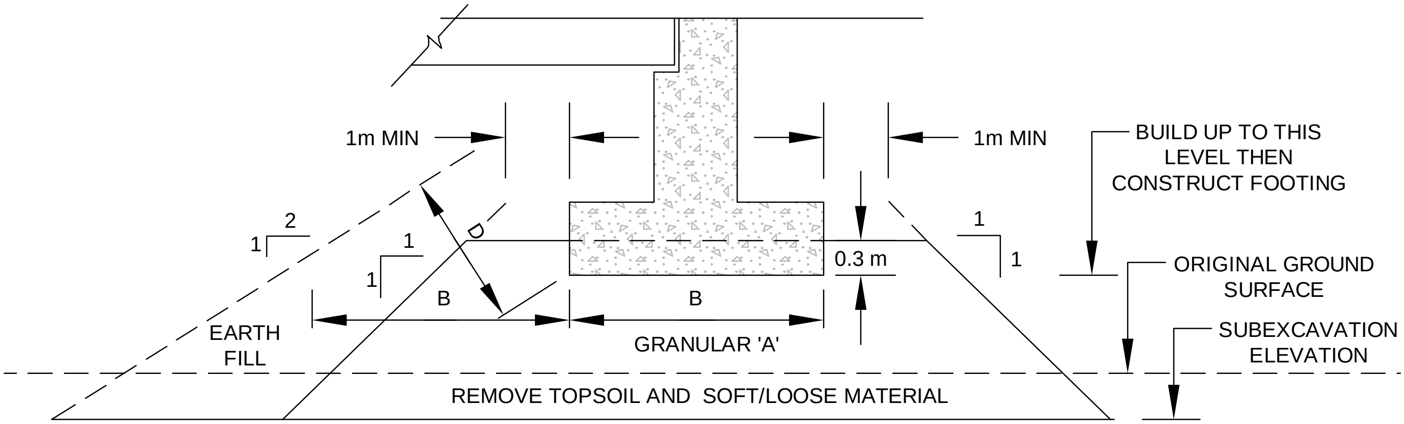
Geocres No. 30M12-355

HWY. 401		PROJECT NO. 10-1111-0211	DIST.
SUBM'D. TVA	CHKD. TVA	DATE: Feb, 2013	SITE:
DRAWN: JFC	CHKD. KJB	APPD. TG	DWG. 2

PLOT DATE: February 7, 2013
FILENAME: T:\Projects\2010\10-1111-0211 (AECOM, Mississauga)\-AB- Mavis Road\10111102111AB02.dwg



CROSS-SECTION



LONGITUDINAL SECTION

NOTES:

1. REMOVE TOPSOIL AND SOFT/LOOSE SUBSOIL UNDER FOOTPRINT OF COMPACTED GRANULAR 'A'.
2. PLACE GRANULAR 'A' AND EARTH FILL TO BOTTOM OF FOOTING LEVEL, COMPACTED ACCORDING TO SP 105 S10.
3. CONSTRUCT CONCRETE FOOTING.
4. PLACE REMAINDER OF GRANULAR 'A' AND EARTH FILL AS REQUIRED.
5. SOURCE M.T.C. 1982.

NOT TO SCALE

PROJECT
HWY 401 - MAVIS ROAD UNDERPASS
CITY OF MISSISSAUGA, REGION OF PEEL
MTO, G.W.P 2150-01-00

TITLE

TYPICAL ABUTMENT ON COMPACTED FILL SHOWING GRANULAR 'A' CORE

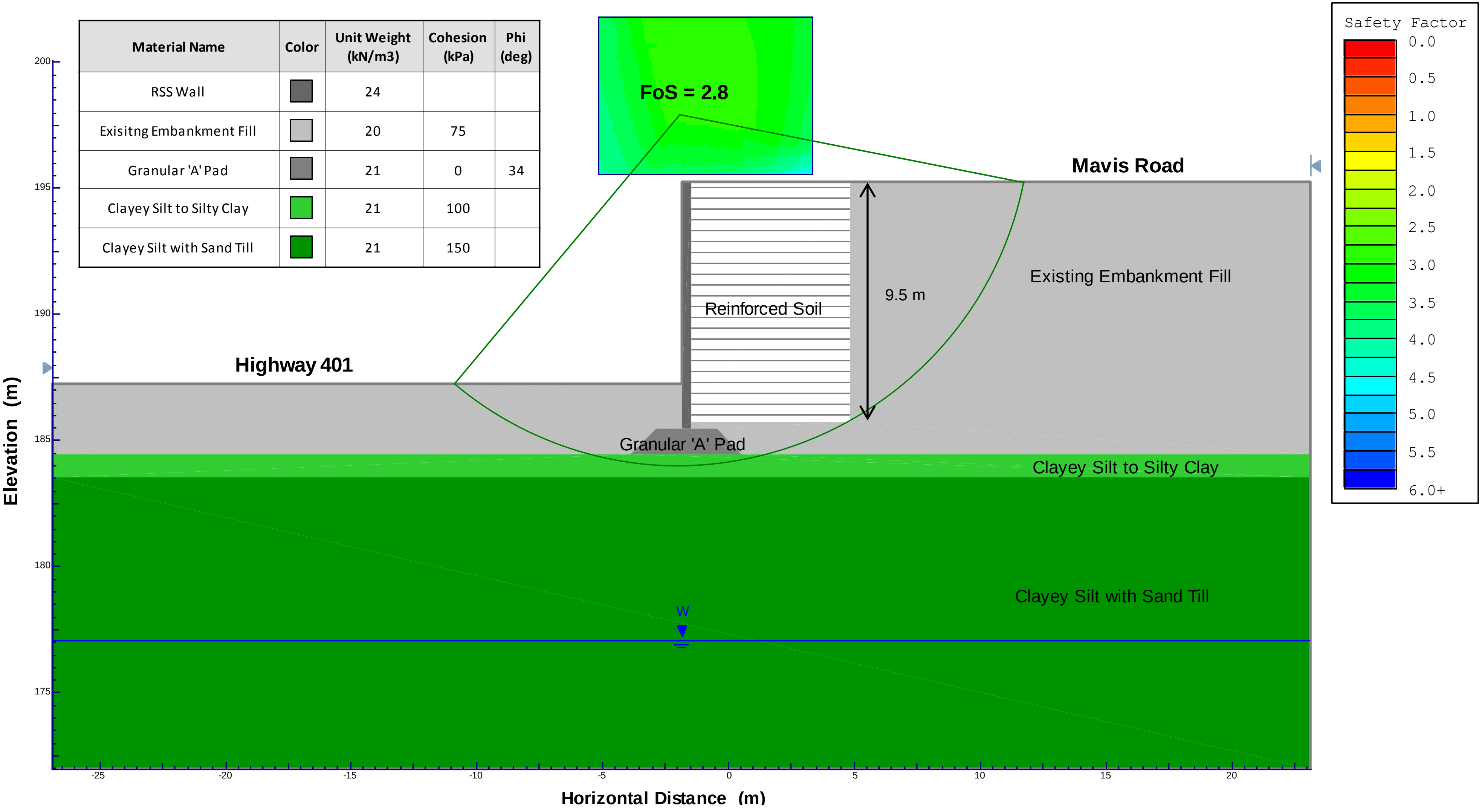
Golder Associates
Mississauga, Ontario, Canada

PROJECT No.		10-1111-0211	FILE No. 1011110211AA001.dwg
DESIGN			SCALE AS SHOWN REV. A
CAD	DD/JFC	Sep. 12, 2011	FIGURE No.
CHECK	TVA	Sep. 12, 2011	1
REVIEW	KJB/TG	Dec. 11, 2012	

PLOT DATE: December 11, 2012
FILENAME: T:\Projects\2010\10-1111-0211 (AECOM, Mississauga)\-AA- Mavis Road\1011110211AA001.dwg

RSS Wall Static Global Stability Analysis – 9.5 m High Wall
Hwy 401 Widening – Mavis Road Underpass

Figure 2



Material Name	Color	Unit Weight (kN/m3)	Cohesion (kPa)	Phi (deg)
RSS Wall	■	24		
Exisitng Embankment Fill	□	20	75	
Granular 'A' Pad	▨	21	0	34
Clayey Silt to Silty Clay	🟩	21	100	
Clayey Silt with Sand Till	🟩	21	150	

Date: September 2012

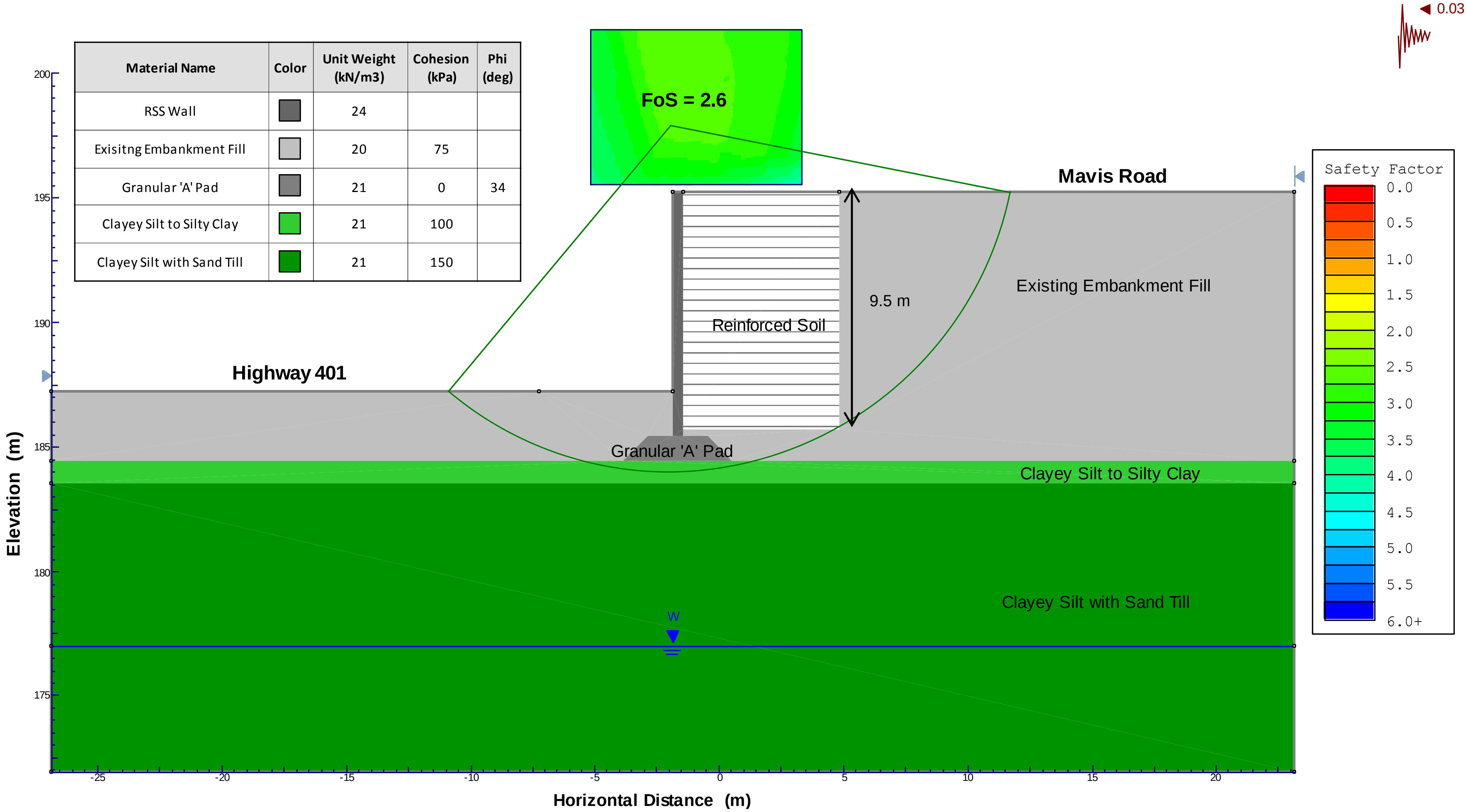
Project No: 10-1111-0211-01

Analysis By: TVA **Reviewed By:** KJB

Golder Associates

RSS Wall Seismic Global Stability Analysis – 9.5 m High Wall
Hwy 401 Widening – Mavis Road Underpass

Figure 3



Material Name	Color	Unit Weight (kN/m3)	Cohesion (kPa)	Phi (deg)
RSS Wall	⬛	24		
Exisitng Embankment Fill	⬜	20	75	
Granular 'A' Pad	⬛	21	0	34
Clayey Silt to Silty Clay	🟩	21	100	
Clayey Silt with Sand Till	🟩	21	150	

Date: September 2012

Project No: 10-1111-0211-01

Analysis By: TVA **Reviewed By:** KJB

Golder Associates

APPENDIX A

Record of Borehole Sheets

LIST OF ABBREVIATIONS

The abbreviations commonly employed on Records of Boreholes, on figures and in the text of the report are as follows:

I. SAMPLE TYPE

AS	Auger sample
BS	Block sample
CS	Chunk sample
SS	Split-spoon
DS	Denison type sample
FS	Foil sample
RC	Rock core
SC	Soil core
ST	Slotted tube
TO	Thin-walled, open
TP	Thin-walled, piston
WS	Wash sample

II. PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:

The number of blows by a 63.5 kg. (140 lb.) hammer dropped 760 mm (30 in.) required to drive a 50 mm (2 in.) drive open sampler for a distance of 300 mm (12 in.)

Dynamic Cone Penetration Resistance; N_d:

The number of blows by a 63.5 kg (140 lb.) hammer dropped 760 mm (30 in.) to drive uncased a 50 mm (2 in.) diameter, 60º cone attached to “A” size drill rods for a distance of 300 mm (12 in.).

PH: Sampler advanced by hydraulic pressure
PM: Sampler advanced by manual pressure
WH: Sampler advanced by static weight of hammer
WR: Sampler advanced by weight of sampler and rod

Piezo-Cone Penetration Test (CPT)

A electronic cone penetrometer with a 60° conical tip and a project end area of 10 cm^2 pushed through ground at a penetration rate of 2 cm/s. Measurements of tip resistance (Q_t), porewater pressure (PWP) and friction along a sleeve are recorded electronically at 25 mm penetration intervals.

III. SOIL DESCRIPTION

(a) Cohesionless Soils

Density Index Relative Density	N Blows/300 mm or Blows/ft
Very loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very dense	over 50

(b) Cohesive Soils

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

IV. SOIL TESTS

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

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

V. MINOR SOIL CONSTITUENTS

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

LIST OF SYMBOLS

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

I. GENERAL

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

II. STRESS AND STRAIN

γ	shear strain
Δ	change in, e.g. in stress: $\Delta\,\sigma$
ε	linear strain
ε_v	volumetric strain
η	coefficient of viscosity
υ	Poisson's ratio
σ	total stress
σ'	effective stress ($\sigma' = \sigma - u$)
σ'_{vo}	initial effective overburden stress
σ_1, σ_2, σ_3	principal stress (major, intermediate, minor)
σ_{oct}	mean stress or octahedral stress $= (\sigma_1 + \sigma_2 + \sigma_3)/3$
τ	shear stress
u	porewater pressure
E	modulus of deformation
G	shear modulus of deformation
K	bulk modulus of compressibility

III. SOIL PROPERTIES

(a) Index Properties

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

(a) Index Properties (continued)

w	water content
w_l or LL	liquid limit
w_p or PL	plastic limit
I_p or PI	plasticity index $= (w_l - w_p)$
w_s	shrinkage limit
I_L	liquidity index $= (w - w_p) / I_p$
I_C	consistency index $= (w_l - w) / I_p$
e_{max}	void ratio in loosest state
e_{min}	void ratio in densest state
I_D	density index $= (e_{max} - e) / (e_{max} - e_{min})$ (formerly relative density)

(b) Hydraulic Properties

h	hydraulic head or potential
q	rate of flow
v	velocity of flow
i	hydraulic gradient
k	hydraulic conductivity (coefficient of permeability)
j	seepage force per unit volume

(c) Consolidation (one-dimensional)

C_c	compression index (normally consolidated range)
C_r	recompression index (over-consolidated range)
C_s	swelling index
C_α	secondary compression index
m_v	coefficient of volume change
c_v	coefficient of consolidation (vertical direction)
c_h	coefficient of consolidation (horizontal direction)
T_v	time factor (vertical direction)
U	degree of consolidation
σ'_p	pre-consolidation stress
OCR	over-consolidation ratio $= \sigma'_p / \sigma'_{vo}$

(d) Shear Strength

τ_p, τ_r	peak and residual shear strength
ϕ'	effective angle of internal friction
δ	angle of interface friction
μ	coefficient of friction = tan δ
c'	effective cohesion
c_u, s_u	undrained shear strength ($\phi = 0$ analysis)
p	mean total stress $(\sigma_1 + \sigma_3)/2$
p′	mean effective stress $(\sigma'_1 + \sigma'_3)/2$
q	$(\sigma_1 - \sigma_3)/2$ or $(\sigma'_1 - \sigma'_3)/2$
q_u	compressive strength $(\sigma_1 - \sigma_3)$
S_t	sensitivity

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

Notes: 1 $\tau = c' + \sigma' \tan \phi'$

2 shear strength = (compressive strength)/2

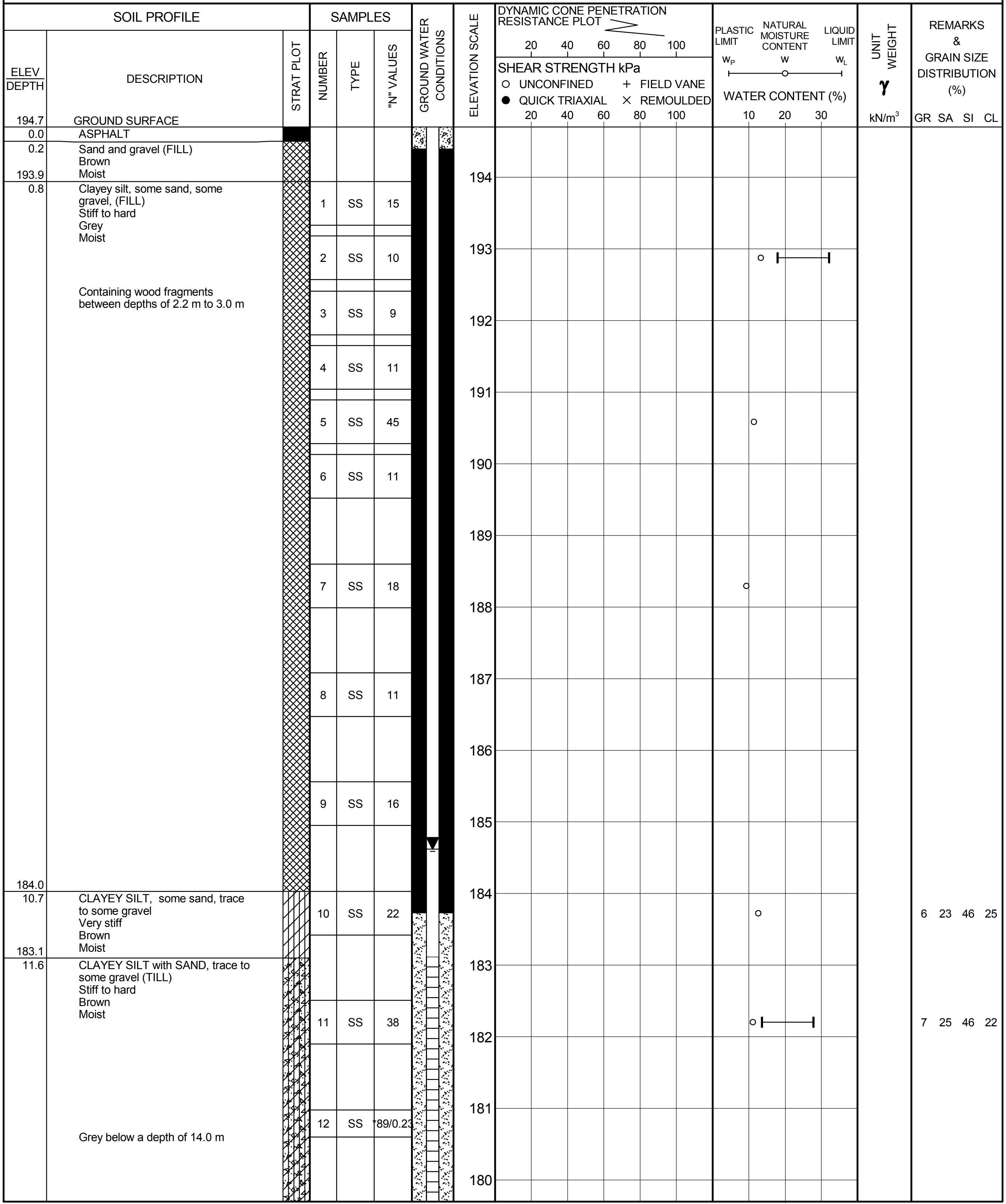
Golder Associates

RECORD OF BOREHOLE No MR-1

SHEET 1 OF 3

METRIC

PROJECT 10-1111-0211
G.W.P. 2150-01-00
LOCATION N 4831249.0 ;E 287997.7
ORIGINATED BY SB
DIST HWY 401
BOREHOLE TYPE 108 mm I.D. Hollow Stem Augers
COMPILED BY CC/TVA
DATUM Geodetic
DATE June 6 and 7, 2012
CHECKED BY KJB



ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL
194.7	GROUND SURFACE							
0.0	ASPHALT							
193.9 0.2	Sand and gravel (FILL) Brown Moist							
0.8	Clayey silt, some sand, some gravel, (FILL) Stiff to hard Grey Moist	1	SS	15				
		2	SS	10				
	Containing wood fragments between depths of 2.2 m to 3.0 m	3	SS	9				
		4	SS	11				
		5	SS	45				
		6	SS	11				
		7	SS	18				
		8	SS	11				
		9	SS	16				
184.0 10.7	CLAYEY SILT, some sand, trace to some gravel Very stiff Brown Moist	10	SS	22	6	23	46	25
183.1 11.6	CLAYEY SILT with SAND, trace to some gravel (TILL) Stiff to hard Brown Moist							
		11	SS	38	7	25	46	22
		12	SS	*89/0.23				
	Grey below a depth of 14.0 m							

Continued Next Page

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

RECORD OF BOREHOLE No MR-1

SHEET 2 OF 3

METRIC

PROJECT 10-1111-0211

G.W.P. 2150-01-00

DIST

HWY 401

DATUM Geodetic

LOCATION N 4831249.0 ;E 287997.7

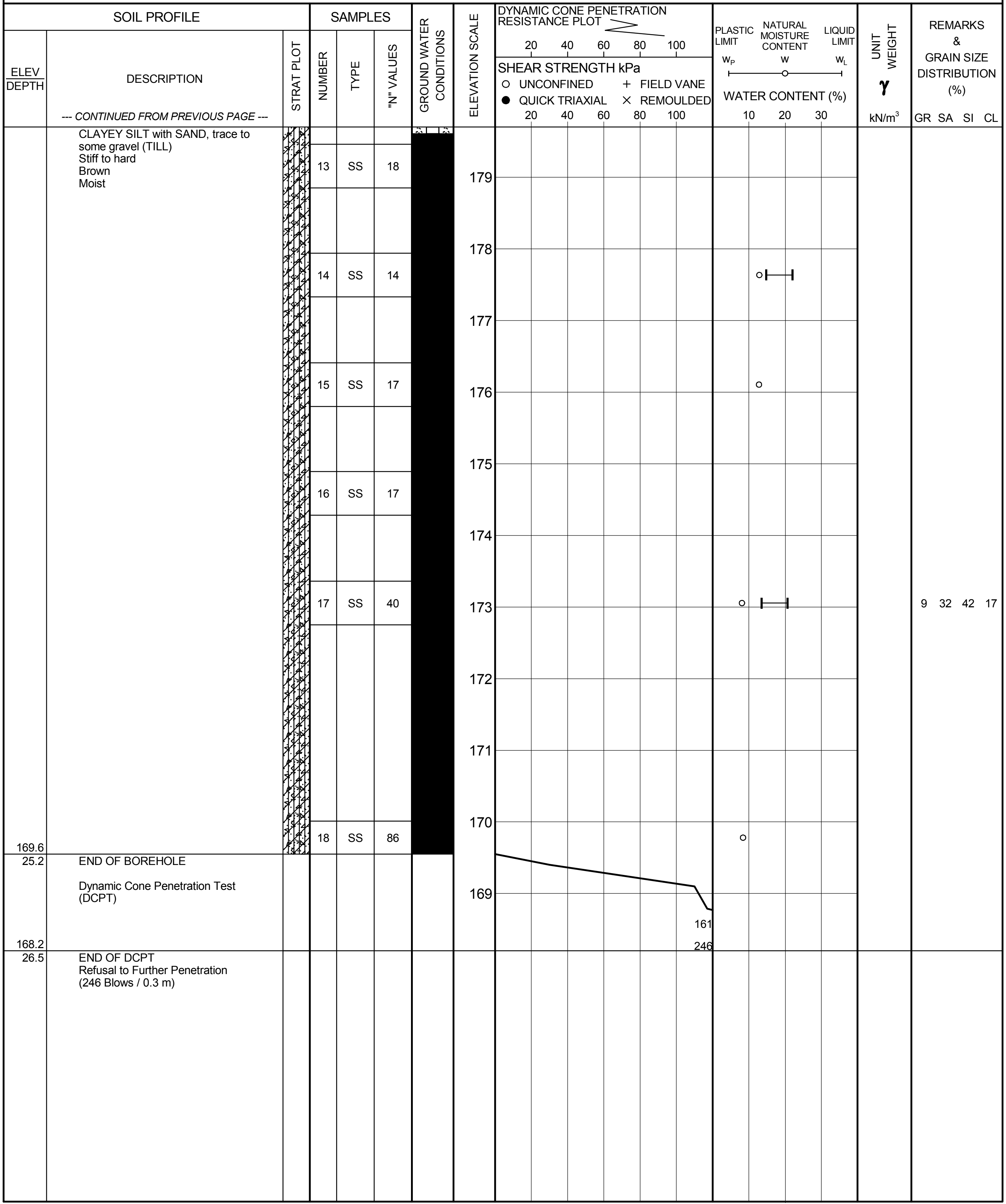
BOREHOLE TYPE 108 mm I.D. Hollow Stem Augers

DATE June 6 and 7, 2012

ORIGINATED BY SB

COMPILED BY CC/TVA

CHECKED BY KJB



--- CONTINUED FROM PREVIOUS PAGE ---

ELEV DEPTH	DESCRIPTION
	CLAYEY SILT with SAND, trace to some gravel (TILL) Stiff to hard Brown Moist
169.6 25.2	END OF BOREHOLE Dynamic Cone Penetration Test (DCPT)
168.2 26.5	END OF DCPT Refusal to Further Penetration (246 Blows / 0.3 m)

NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL
13	SS	18				
14	SS	14				
15	SS	17				
16	SS	17				
17	SS	40	9	32	42	17
18	SS	86				

Continued Next Page

$+^3, \times^3$: Numbers refer to Sensitivity

$\circ^{3\%}$ STRAIN AT FAILURE

RECORD OF BOREHOLE No MR-1

SHEET 3 OF 3

METRIC

PROJECT 10-1111-0211
G.W.P. 2150-01-00
LOCATION N 4831249.0 ;E 287997.7
ORIGINATED BY SB
DIST HWY 401
BOREHOLE TYPE 108 mm I.D. Hollow Stem Augers
COMPILED BY CC/TVA
DATUM Geodetic
DATE June 6 and 7, 2012
CHECKED BY KJB

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

NOTES:

* Split-Spoon bouncing on possible cobbles/boulders.

1. A piece of cobble was observed at top of split-spoon upon completion of sampling No. 18.

2. Water level inside augers at a depth of 18.0 m below ground surface (Elev. 176.7 m), measured at start of work day on June 7, 2012.

3. Piezometer installation consists of 50 mm diameter PVC pipe with a 3.0 m slotted screen.

Water Level Readings

Date	Depth (m)	Elev. (m)
06/07/12	Dry	-
08/10/12	10.4	184.3
10/09/12	10.2	184.5
11/05/12	10.1	184.6

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

RECORD OF BOREHOLE No MR-2

SHEET 1 OF 3

METRIC

PROJECT	10-1111-0211				
G.W.P.	2150-01-00	LOCATION	N 4831239.6 ;E 287971.8	ORIGINATED BY	SB
DIST	HWY 401	BOREHOLE TYPE	108 mm I.D. Hollow Stem Augers	COMPILED BY	CC/TVA
DATUM	Geodetic	DATE	May 22 to 23, 2012	CHECKED BY	KJB

SOIL PROFILE

ELEV DEPTH	DESCRIPTION
195.1	GROUND SURFACE
0.0	ASPHALT
0.2	Sand and gravel (FILL) Brown Moist
194.3 0.8	Clayey silt to silty clay, trace to some sand, trace to some gravel, containing organic and wood fragments to a depth of 5.2 m (FILL) Firm to very stiff Grey Moist - - - - - - - - - - - with gravel - with gravel - - - - - - - - - - -
184.4 10.7	SILTY CLAY, trace sand, trace gravel Stiff Brown Moist
183.5 11.6	CLAYEY SILT with SAND, trace to some gravel (TILL) Stiff to hard Brown Moist - - - - - - - - - - - containing sand seams and cobbles - - - - - - - - - - -

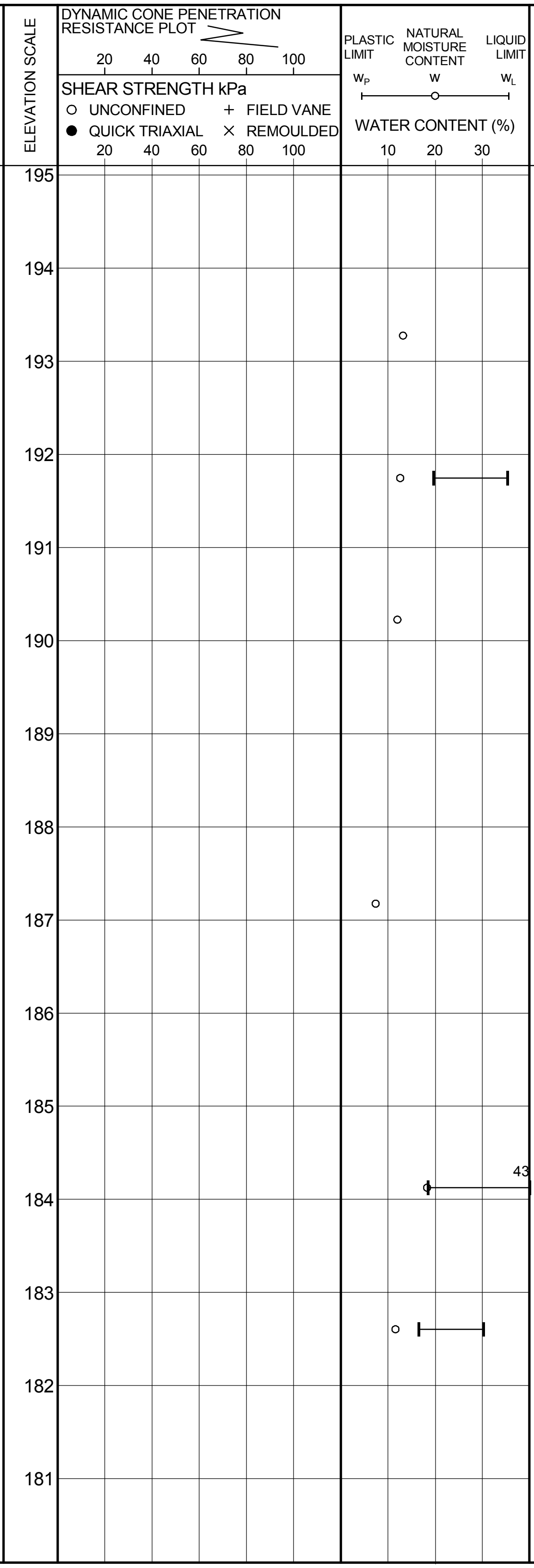
SAMPLES

NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL
1	SS	10				
2	SS	9				
3	SS	7				
4	SS	14	40	16	29	15
5	SS	16				
6	SS	18				
7	SS	12				
8	SS	20				
9	SS	11				
10	SS	14				
11	SS	35	11	27	42	20
12	SS	40				

GROUND WATER CONDITIONS

UNIT WEIGHT γ kN/m³

REMARKS & GRAIN SIZE DISTRIBUTION (%)



Continued Next Page

+³, ×³: Numbers refer to Sensitivity

○³% STRAIN AT FAILURE

RECORD OF BOREHOLE No MR-2

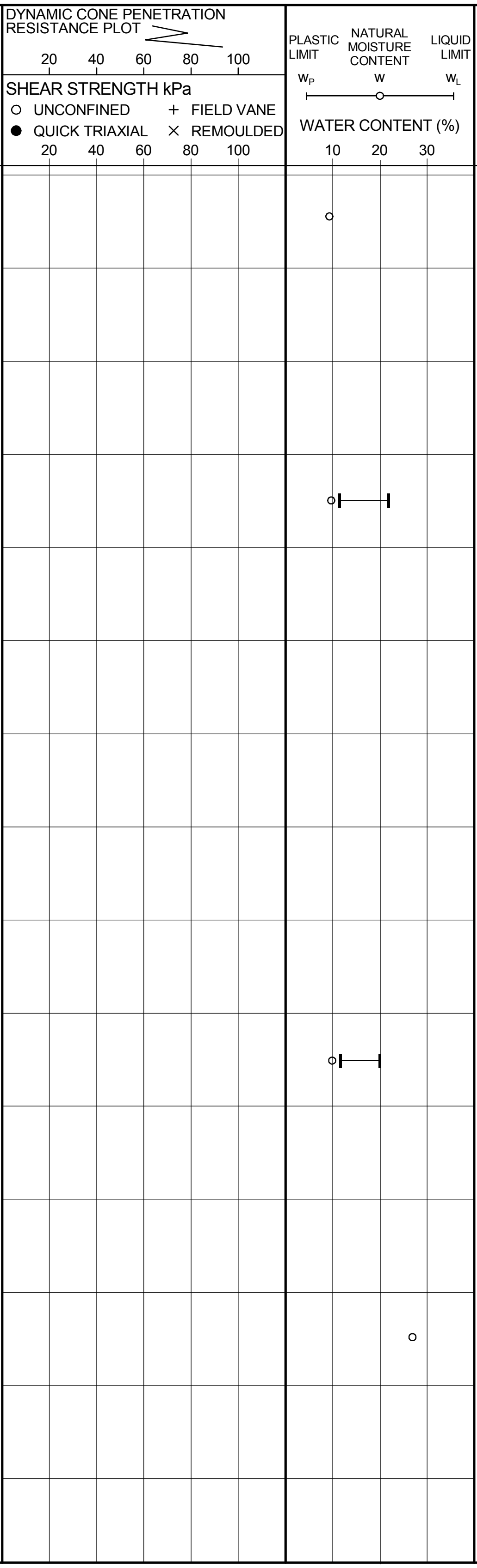
SHEET 2 OF 3

METRIC

PROJECT 10-1111-0211
G.W.P. 2150-01-00
LOCATION N 4831239.6 ;E 287971.8
ORIGINATED BY SB
DIST HWY 401
BOREHOLE TYPE 108 mm I.D. Hollow Stem Augers
COMPILED BY CC/TVA
DATUM Geodetic
DATE May 22 to 23, 2012
CHECKED BY KJB

--- CONTINUED FROM PREVIOUS PAGE ---

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	CLAYEY SILT with SAND, trace to some gravel (TILL) Stiff to hard Brown Moist					180	
		13	SS	21			
						179	
		14	SS	18		178	
						177	
		15	SS	12			10 30 43 17
						176	
					∇		
		16	SS	19		175	
						174	
		17	SS	15			
						173	
		18	SS	83		172	
						171	
		19	SS	116			13 46 29 12
						170	
		20	SS	84		169	
168.0 27.1	SILTY CLAY, trace sand, trace gravel Hard Grey Wet					168	
167.4 27.7	Silty SAND and GRAVEL, trace clay Dense Grey Wet	21A 21B	SS	39			
						167	
166.4 28.7	SAND and SILT, trace to some gravel, trace to some clay (TILL) Compact to very dense Grey Wet						
		22	SS	64		166	



UNIT WEIGHT γ kN/m³

Continued Next Page

+³, ×³: Numbers refer to Sensitivity

○³% STRAIN AT FAILURE

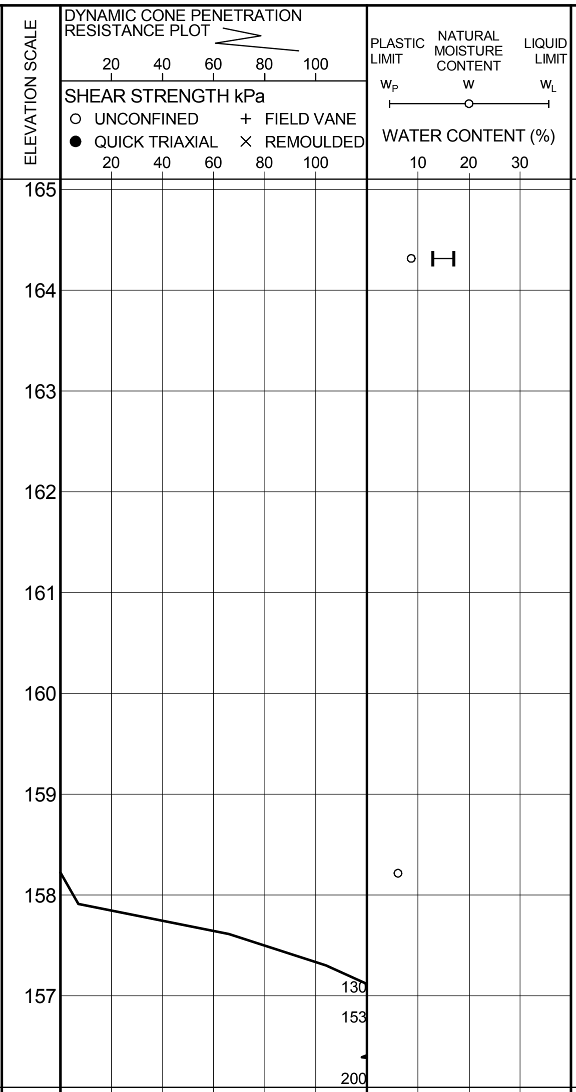
RECORD OF BOREHOLE No MR-2

SHEET 3 OF 3

METRIC

PROJECT 10-1111-0211
G.W.P. 2150-01-00
LOCATION N 4831239.6 ;E 287971.8
ORIGINATED BY SB
DIST
HWY 401
BOREHOLE TYPE 108 mm I.D. Hollow Stem Augers
COMPILED BY CC/TVA
DATUM Geodetic
DATE May 22 to 23, 2012
CHECKED BY KJB

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL
	--- CONTINUED FROM PREVIOUS PAGE ---							
	SAND and SILT, trace to some gravel, trace to some clay (TILL) Compact to very dense Grey Wet Frequent shale fragment inclusions below a depth of 30.5 m	23	SS	61	9	38	43	10
		24	SS	29				
159.1 36.0	SAND and GRAVEL, some silt, trace clay Compact Grey Wet	25	SS	11	32	51	15	2
157.9 37.2	END OF BOREHOLE Dynamic Cone Penetration Test (DCPT)							
156.1 39.0	END OF DCPT Refusal to Further Penetration (200 Blows / 0.15 m) NOTE: 1. Water level inside augers at a depth of 19.8 m below ground surface (Elev. 175.3 m), measured at the start of work day on May 22, 2012.							



UNIT WEIGHT γ kN/m³

REMARKS & GRAIN SIZE DISTRIBUTION (%)

+ ^{3}, × ^{3}: Numbers refer to Sensitivity

○ $^{3\%}$ STRAIN AT FAILURE

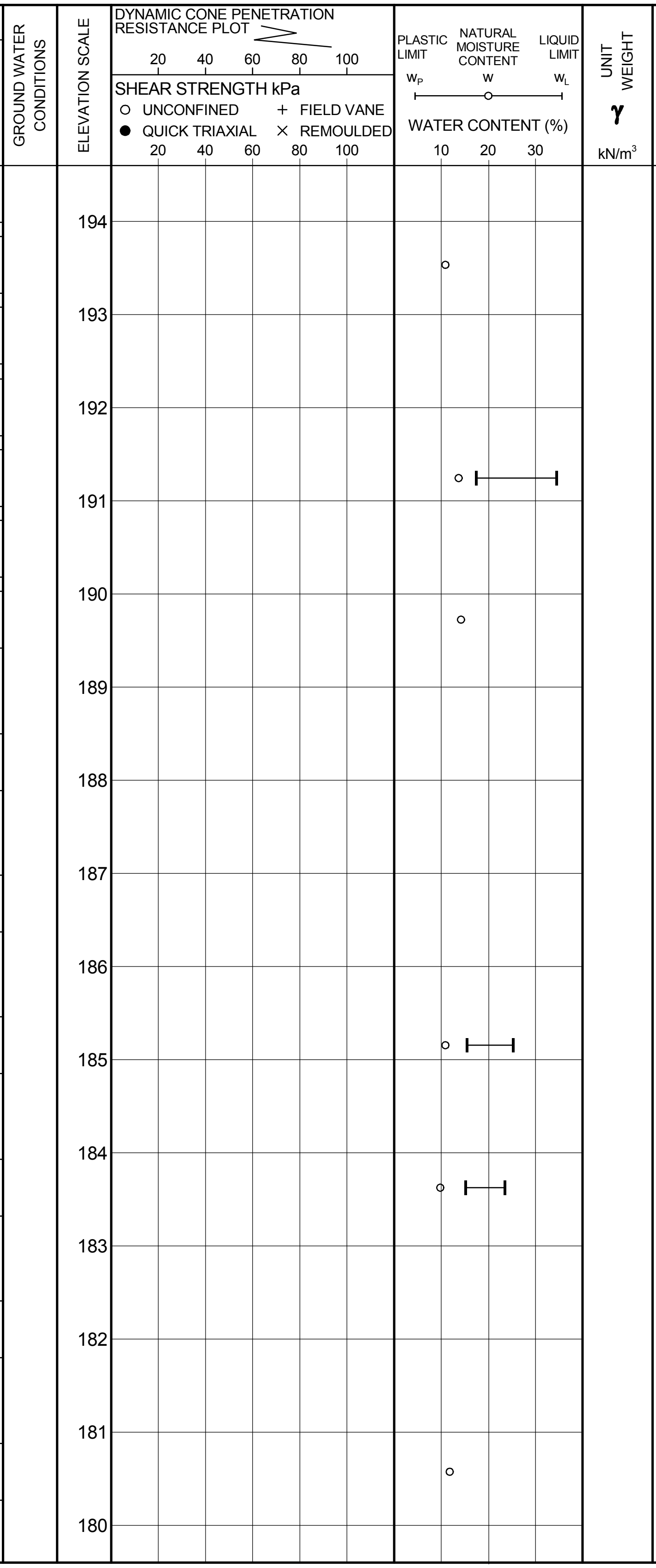
RECORD OF BOREHOLE No MR-3

SHEET 1 OF 2

METRIC

PROJECT 10-1111-0211
G.W.P. 2150-01-00
LOCATION N 4831172.8 ;E 288079.3
ORIGINATED BY SB
DIST
HWY 401
BOREHOLE TYPE 108 mm I.D. Hollow Stem Augers, NW Casing with Tricone
COMPILED BY CC/TVA
DATUM Geodetic
DATE May 29 to 30, 2012
CHECKED BY KJB

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLES NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL
194.6	GROUND SURFACE							
0.0	TOPSOIL							
0.2	Clayey silt, some sand, trace to some gravel, containing rootlets, organics, asphalt and shale fragments to a depth of 6.7 m (FILL) Firm to hard Brown to grey Moist	1	SS	24				
		2	SS	5				
		3	SS	12				
		4	SS	8				
		5	SS	6				
	with gravel	6	SS	10				
		7	SS	11				
		8	SS	17				
		9	SS	29				
		10	SS	33				
184.2 10.4	CLAYEY SILT with SAND, trace to some gravel (TILL) Stiff to very stiff Grey Moist							
		11	SS	20	7	32	42	19
		12	SS	15				
		13	SS	14				



REMARKS & GRAIN SIZE DISTRIBUTION (%)

Continued Next Page

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

RECORD OF BOREHOLE No MR-3

SHEET 2 OF 2

METRIC

PROJECT 10-1111-0211
G.W.P. 2150-01-00
LOCATION N 4831172.8 ;E 288079.3
ORIGINATED BY SB
DIST HWY 401
BOREHOLE TYPE 108 mm I.D. Hollow Stem Augers, NW Casing with Tricone
COMPILED BY CC/TVA
DATUM Geodetic
DATE May 29 to 30, 2012
CHECKED BY KJB

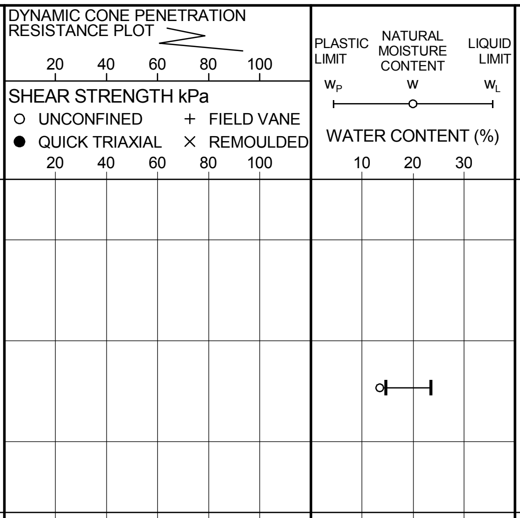
ELEV DEPTH	SOIL PROFILE DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	SAMPLES TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	--- CONTINUED FROM PREVIOUS PAGE ---								
179.3			14	SS	50/0.06				
15.3	BOULDER Grey and black						179		
178.7 15.9	CLAYEY SILT with SAND, trace to some gravel (TILL) Very stiff Grey Wet						178		
			15	SS	16		177		
176.3 18.3	END OF BOREHOLE CASING REFUSAL								

NOTES:

1. Casing refusal on boulder at 15.3 m depth, cored through boulder using NQ size core barrel and continued sampling using NW Casing & Tricone.

2. Unable to advance borehole beyond a depth of 18.3 m due to casing refusal. Backfilled borehole, moved drilling 1.5 m north, and advanced Borehole MR-3A and continued sampling below 18.3 m depth.

3. Borehole dry (inside augers) at start of work day on May 30, 2012.



$+^3, \times^3$: Numbers refer to Sensitivity

○ 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No MR-3A

SHEET 1 OF 3

METRIC

PROJECT 10-1111-0211

G.W.P. 2150-01-00

LOCATION N 4831173.9 ;E 288078.3

ORIGINATED BY SB

DIST HWY 401

BOREHOLE TYPE 108 mm I.D. Hollow Stem Augers

COMPILED BY CC/TVA

DATUM Geodetic

DATE May 31 and June 4, 2012

CHECKED BY KJB

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100	PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m^3	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
194.6	GROUND SURFACE										
0.0	Refer to Record of Borehole MR-3 for subsurface conditions within these elevations.										
							194				
							193				
							192				
							191				
							190				
							189				
							188				
							187				
							186				
							185				
							184				
							183				
							182				
							181				
							180				

Continued Next Page

$+^3, \times^3$: Numbers refer to Sensitivity

○$^{3\%}$ STRAIN AT FAILURE

RECORD OF BOREHOLE No MR-3A

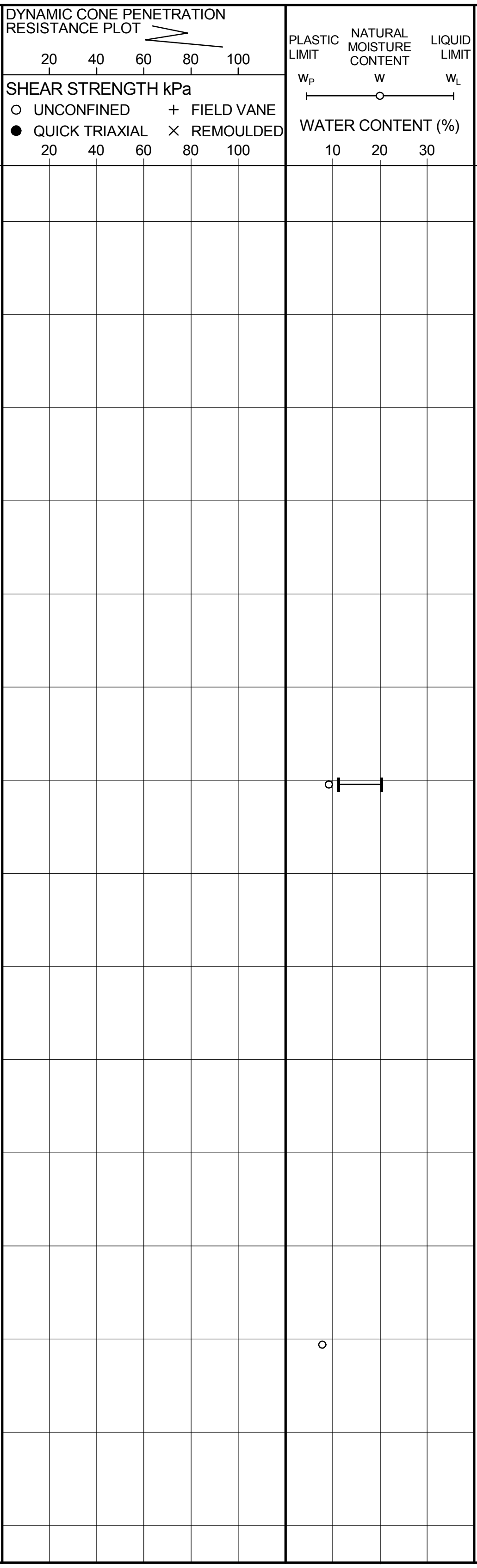
SHEET 2 OF 3

METRIC

PROJECT 10-1111-0211
G.W.P. 2150-01-00
DIST
HWY 401
DATUM Geodetic
LOCATION N 4831173.9 ;E 288078.3
BOREHOLE TYPE 108 mm I.D. Hollow Stem Augers
DATE May 31 and June 4, 2012
ORIGINATED BY SB
COMPILED BY CC/TVA
CHECKED BY KJB

--- CONTINUED FROM PREVIOUS PAGE ---

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	GR	SA	SI	CL
	Refer to Record of Borehole MR-3 for subsurface conditions within these elevations.				179				
					178				
					177				
176.3 18.3	CLAYEY SILT with SAND, trace to some gravel (TILL) Stiff to hard Grey Moist	16	SS	13	176				
					175				
		17	SS	29	174				
		18	SS	43	173	18	29	39	14
					172				
					171				
		19	SS	37	170				
					169				
					168				
		20	SS	93	167				
					166				
					165				



UNIT WEIGHT γ kN/m³

REMARKS & GRAIN SIZE DISTRIBUTION (%)

Continued Next Page

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

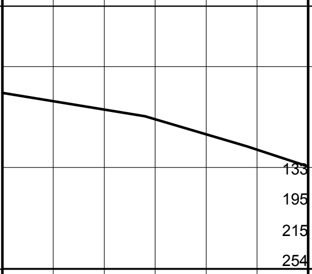
RECORD OF BOREHOLE No MR-3A

SHEET 3 OF 3

METRIC

PROJECT 10-1111-0211
G.W.P. 2150-01-00
DIST ____ HWY 401
DATUM Geodetic
LOCATION N 4831173.9 ;E 288078.3
BOREHOLE TYPE 108 mm I.D. Hollow Stem Augers
DATE May 31 and June 4, 2012
ORIGINATED BY SB
COMPILED BY CC/TVA
CHECKED BY KJB

--- CONTINUED FROM PREVIOUS PAGE ---



SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100	PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						GR	SA	SI	CL
	CLAYEY SILT with SAND, trace to some gravel (TILL) Stiff to hard Grey Moist		21	SS	74		164		○ ├─┤		7	38	40	15
163.7 30.9	END OF BOREHOLE Dynamic Cone Penetration Test (DCPT)						163	133 195 215 254						
162.0 32.6	END OF DCPT Refusal to Further Penetration (254 Blows / 0.3 m) NOTE: 1. Groundwater conditions were not recorded upon completion of drilling, refer to Borehole MR-3.						162							

$+^3$, $\times^3$: Numbers refer to Sensitivity

○ $^{3\%}$ STRAIN AT FAILURE

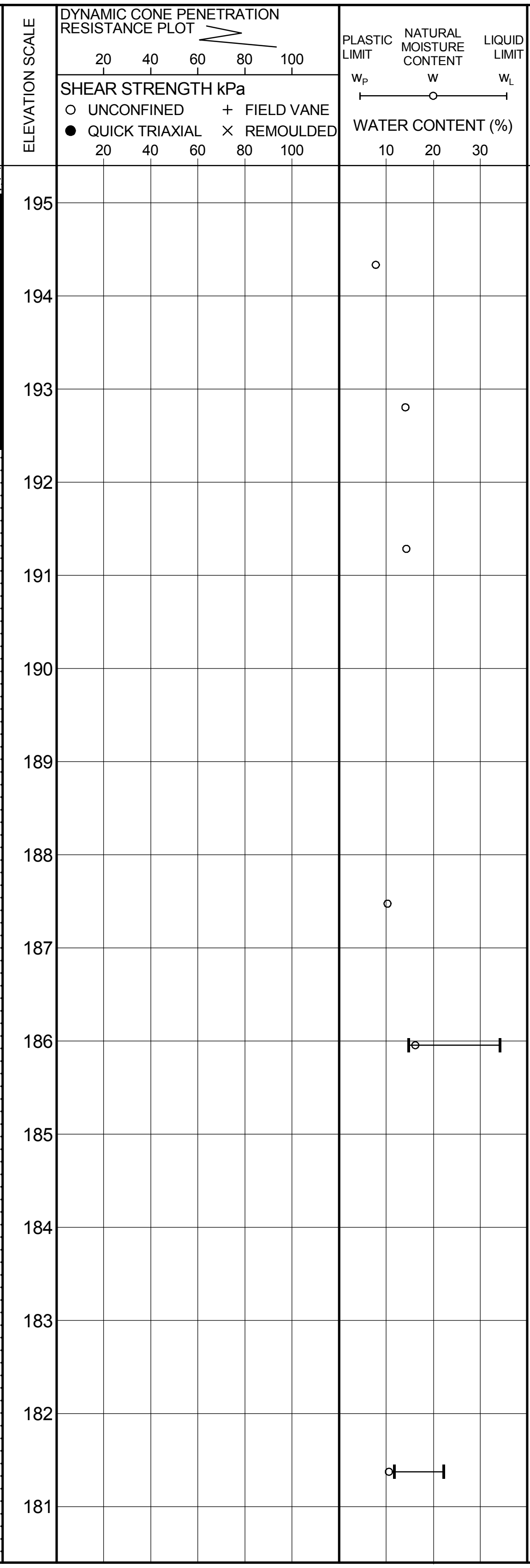
RECORD OF BOREHOLE No MR-4

SHEET 1 OF 3

METRIC

PROJECT 10-1111-0211
G.W.P. 2150-01-00
DIST HWY 401
DATUM Geodetic
LOCATION N 4831158.4 ;E 288051.0
BOREHOLE TYPE 108 mm I.D. Hollow Stem Augers
DATE May 24 to 28, 2012
ORIGINATED BY SB/CC
COMPILED BY CC/TVA
CHECKED BY KJB

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL
195.4	GROUND SURFACE							
0.0	ASPHALT							
0.2	Sand and gravel, (FILL) Brown Moist							
194.6 0.8	Clayey silt, trace to some sand, trace gravel, containing wood fragments, (FILL) Stiff to hard Brown Moist	1	SS	11				
		2	SS	10				
		3	SS	10				
		4	SS	14				
		5	SS	14				
		6	SS	17				
	inferred cobbles	7	SS	80				
	with sand	8	SS	9	3	36	40	21
		9	SS	28				
		10	SS	36				
183.8 11.6	CLAYEY SILT with SAND, trace to some gravel, (TILL) Very stiff to hard Brown to grey Moist							
		11	SS	30				
		12	SS	22	10	29	42	19



GROUND WATER CONDITIONS

UNIT WEIGHT γ kN/m^3

REMARKS & GRAIN SIZE DISTRIBUTION (%)

Continued Next Page

+^{3}, ×^{3}: Numbers refer to Sensitivity

○$^{3\%}$ STRAIN AT FAILURE

RECORD OF BOREHOLE No MR-4

SHEET 2 OF 3

METRIC

PROJECT 10-1111-0211

G.W.P. 2150-01-00

DIST HWY 401

DATUM Geodetic

LOCATION N 4831158.4 ;E 288051.0

BOREHOLE TYPE 108 mm I.D. Hollow Stem Augers

DATE May 24 to 28, 2012

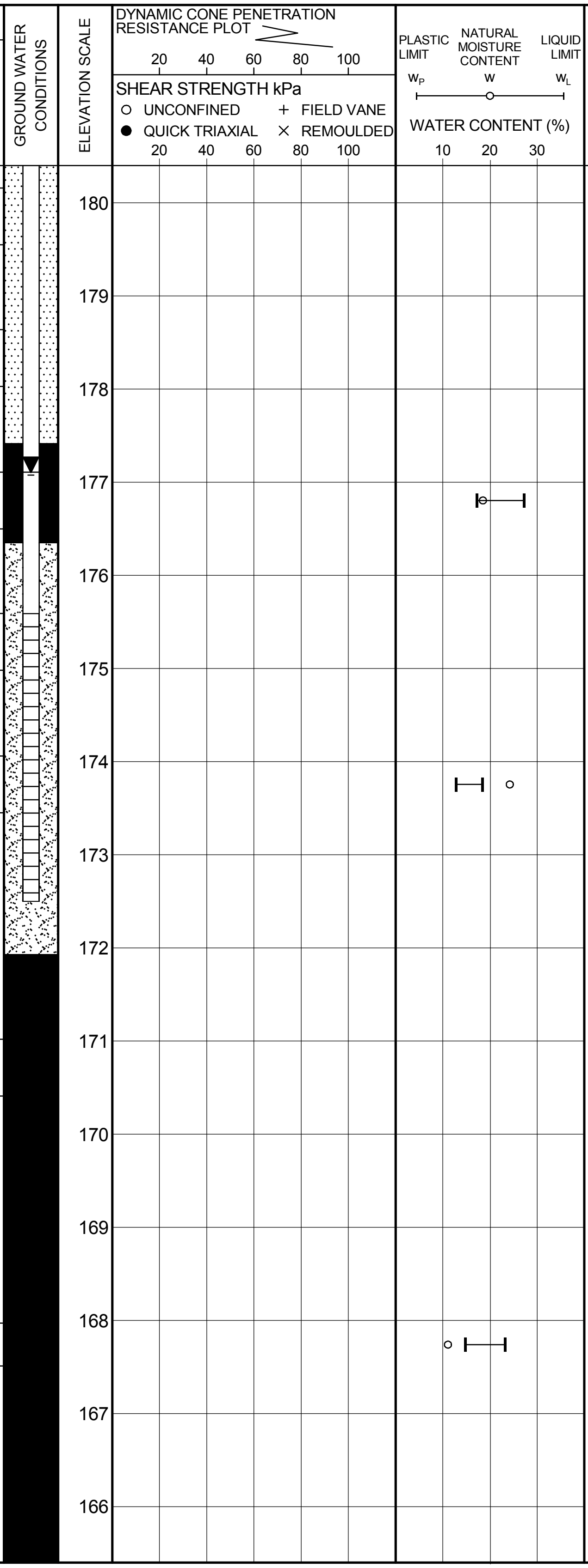
ORIGINATED BY SB/CC

COMPILED BY CC/TVA

CHECKED BY KJB

--- CONTINUED FROM PREVIOUS PAGE ---

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
	CLAYEY SILT with SAND, trace to some gravel, (TILL) Very stiff to hard Brown to grey Moist	13	SS	20					
	inferred cobbles								
		14	SS	19					
	wet								
		15	SS	14		1	21	49	29
		16	SS	36					
		17	SS	38		11	40	37	12
		18	SS	70					
		19	SS	61		7	27	46	20



Continued Next Page

+³, ×³: Numbers refer to Sensitivity

○³% STRAIN AT FAILURE

RECORD OF BOREHOLE No MR-4

SHEET 3 OF 3

METRIC

PROJECT 10-1111-0211

G.W.P. 2150-01-00

LOCATION N 4831158.4 ;E 288051.0

ORIGINATED BY SB/CC

DIST ____ HWY 401

BOREHOLE TYPE 108 mm I.D. Hollow Stem Augers

COMPILED BY CC/TVA

DATUM Geodetic

DATE May 24 to 28, 2012

CHECKED BY KJB

SOIL PROFILE			SAMPLES					DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100	PLASTIC LIMIT W_P / NATURAL MOISTURE CONTENT W / LIQUID LIMIT W_L / WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE				GR	SA	SI	CL
	--- CONTINUED FROM PREVIOUS PAGE ---													
	CLAYEY SILT with SAND, trace to some gravel, (TILL) Very stiff to hard Brown to grey Moist						165							
164.3 31.1	- - - - - - - - - - - containing shale fragments		20	SS	63									
	END OF BOREHOLE													

NOTES:

1. Difficulties advancing auger was observed between depths of 15.2 m and 16.8 m (Elev. 180.2 m and 178.6 m) below ground surface.

2. Water level inside augers at a depth of 0.9 m below ground surface (Elev. 194.5 m), measured at start of work day on May 25, 2012.

3. Water level inside augers at a depth of 21.3 m below ground surface (Elev. 174.1 m) upon completion of sampling on May 28, 2012.

4. Piezometer installation consists of 50 mm diameter PVC pipe with a 3.0 m slotted screen.

Water Level Readings

Date	Depth (m)	Elev. (m)
05/28/12	18.3	177.1
05/30/12	18.6	176.8
08/10/12	18.4	177.0
10/09/12	18.4	177.0
11/05/12	18.3	177.1

$+^3, \times^3$: Numbers refer to Sensitivity

$\circ^{3\%}$ STRAIN AT FAILURE

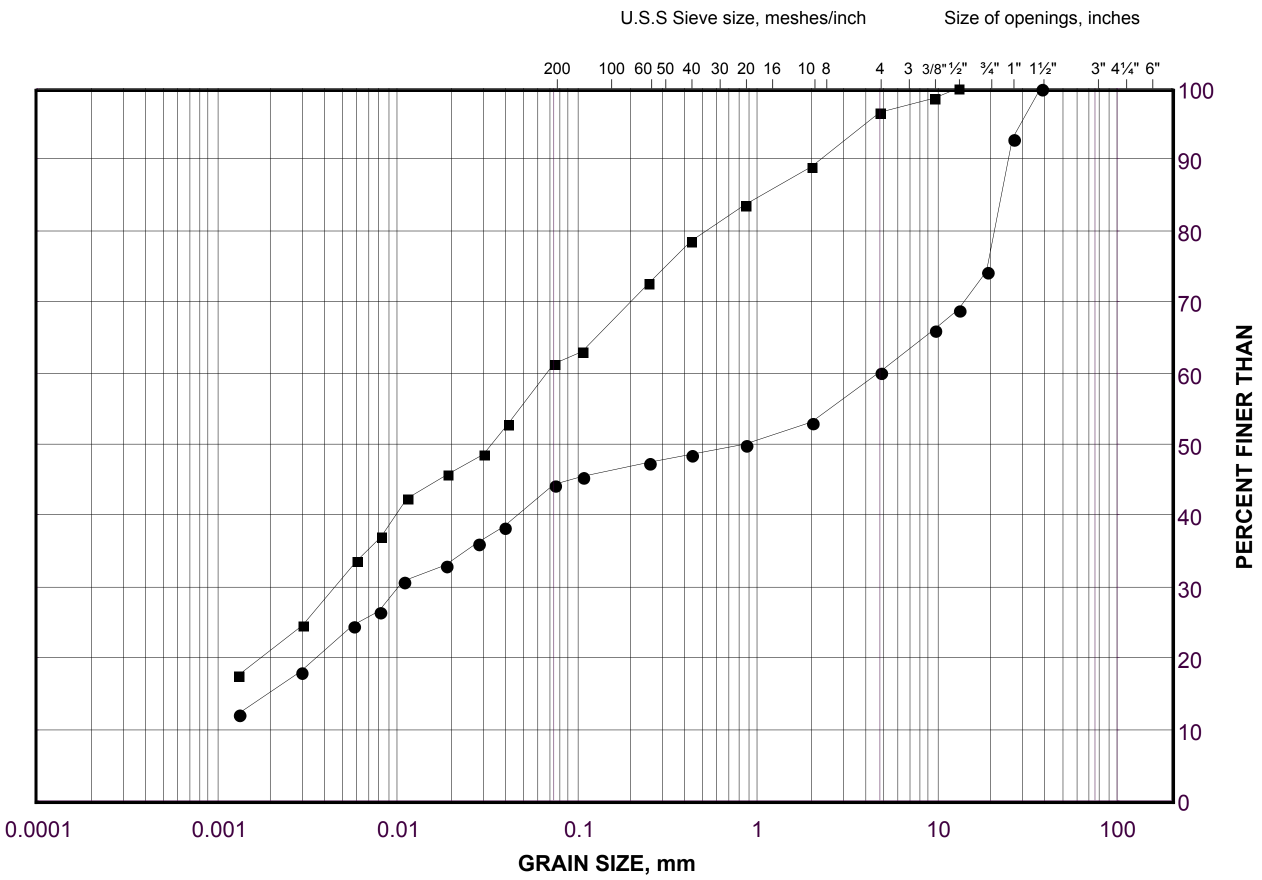
APPENDIX B

Laboratory Test Results

GRAIN SIZE DISTRIBUTION

Clayey Silt Fill

FIGURE B1



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	MR-2	4	191.7
■	MR-4	8	187.5

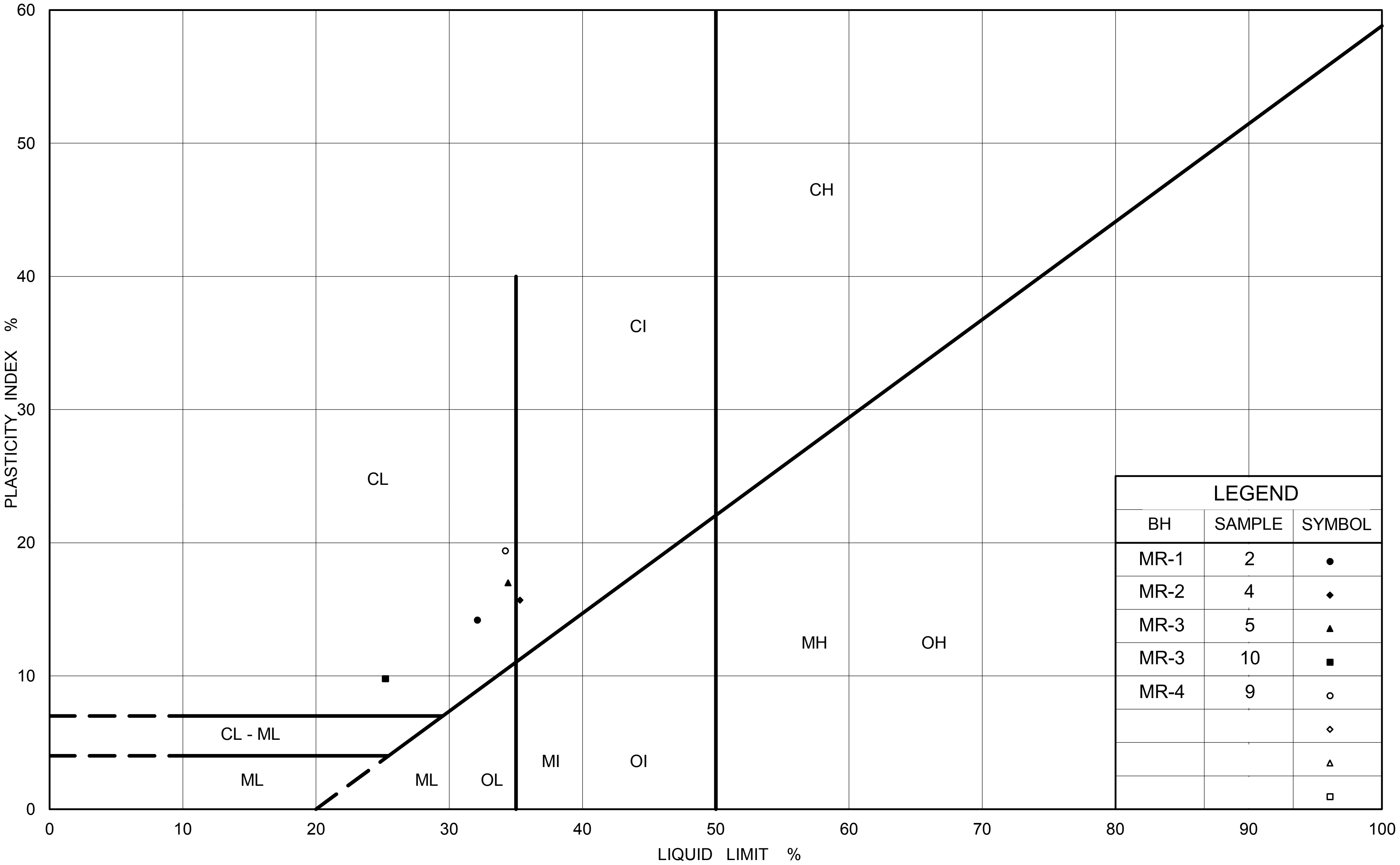
Project Number: 10-1111-0211

Checked By: TVA

Golder Associates

Date: 05-Sep-12

Oct 75, FF-S-21



LEGEND

BH	SAMPLE	SYMBOL
MR-1	2	●
MR-2	4	◆
MR-3	5	▲
MR-3	10	■
MR-4	9	○
		◇
		△
		□

Ministry of Transportation

Ontario

PLASTICITY CHART
Clayey Silt to Silty Clay Fill

Figure No. B2

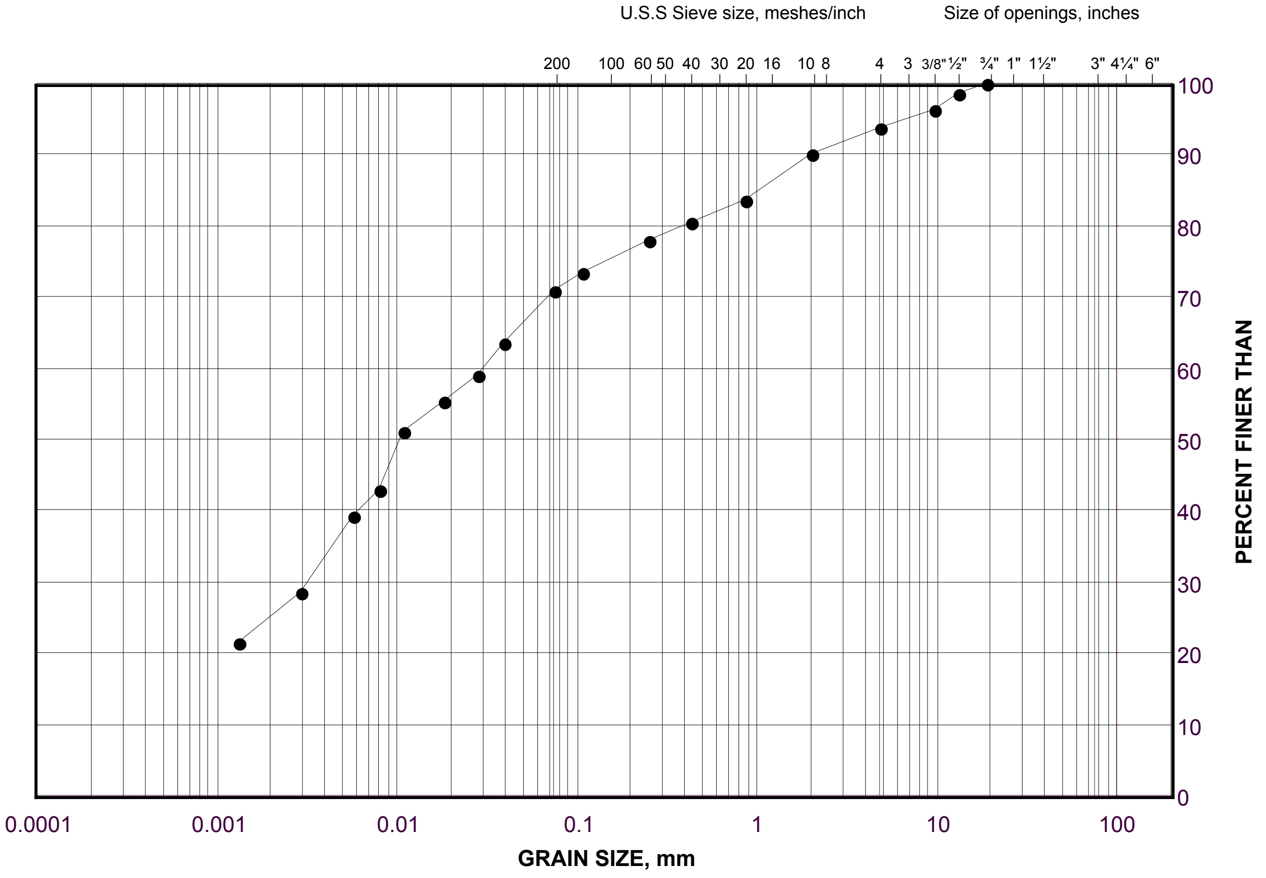
Project No. 10-1111-0211

Checked By: TVA

GRAIN SIZE DISTRIBUTION

Clayey Silt

FIGURE B3



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	MR-1	10	183.7

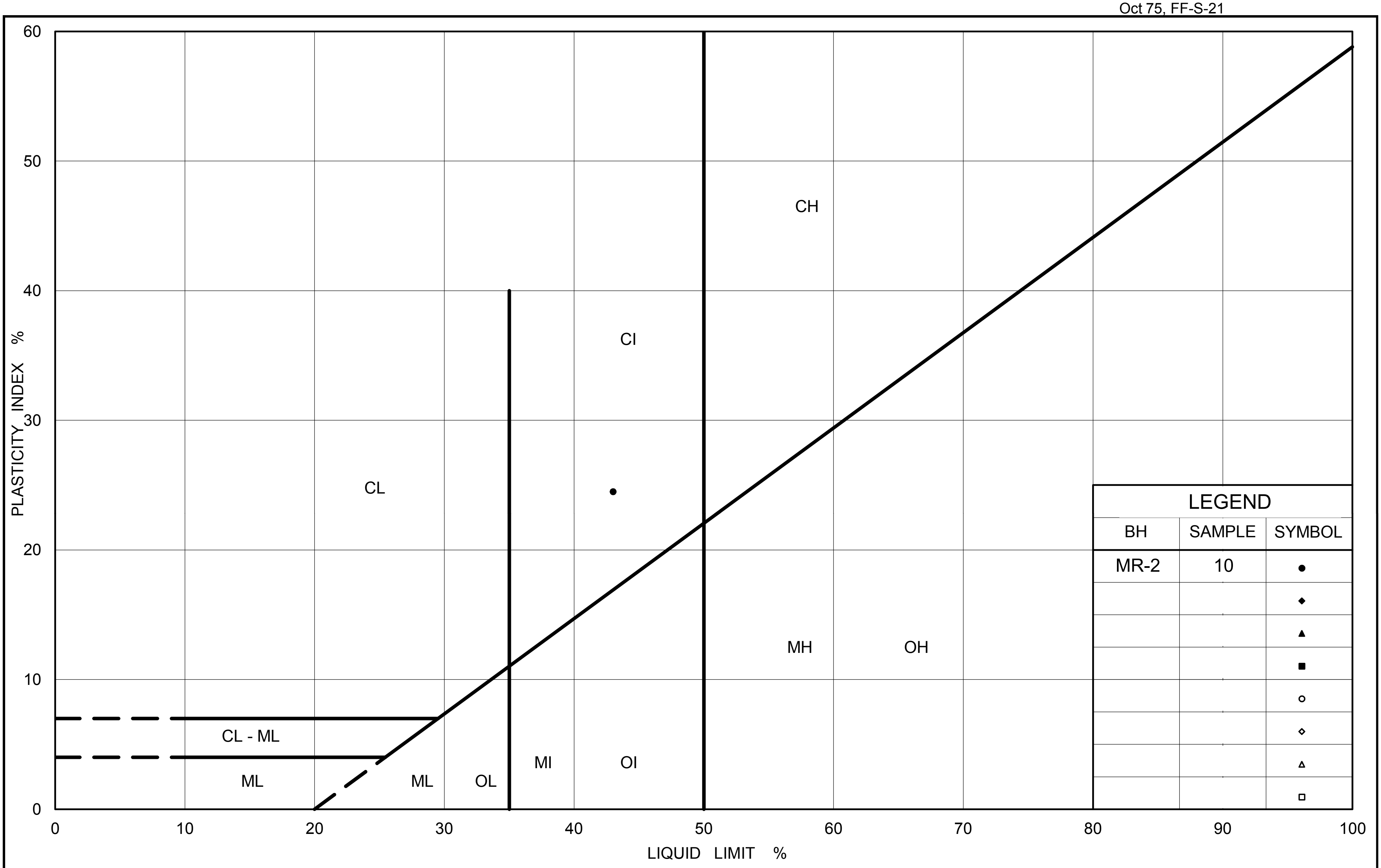
Project Number: 10-1111-0211

Checked By: TVA

Golder Associates

Date: 05-Sep-12

Oct 75, FF-S-21



Ministry of Transportation

Ontario

PLASTICITY CHART
Silty Clay

Figure No. B4

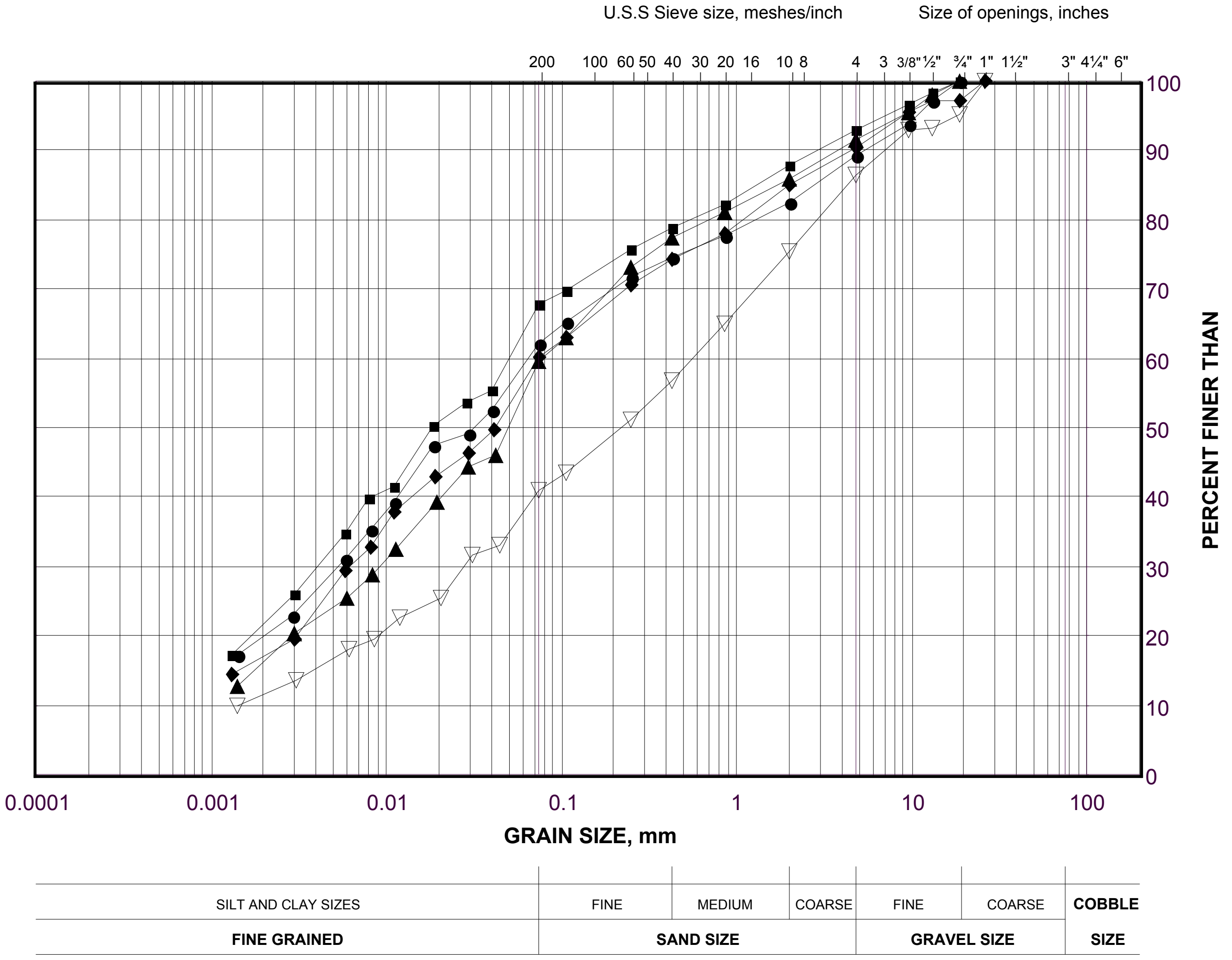
Project No. 10-1111-0211

Checked By: TVA

GRAIN SIZE DISTRIBUTION

Clayey Silt with Sand Till

FIGURE B5A



LEGEND

SYMBOL	BOREHOLE	SAMPLE	DEPTH(m)
●	MR-2	11	182.6
■	MR-1	11	182.2
◆	MR-2	15	176.5
▲	MR-1	17	173.1
▽	MR-2	19	170.5

Project Number: 10-1111-0211

Checked By: TVA

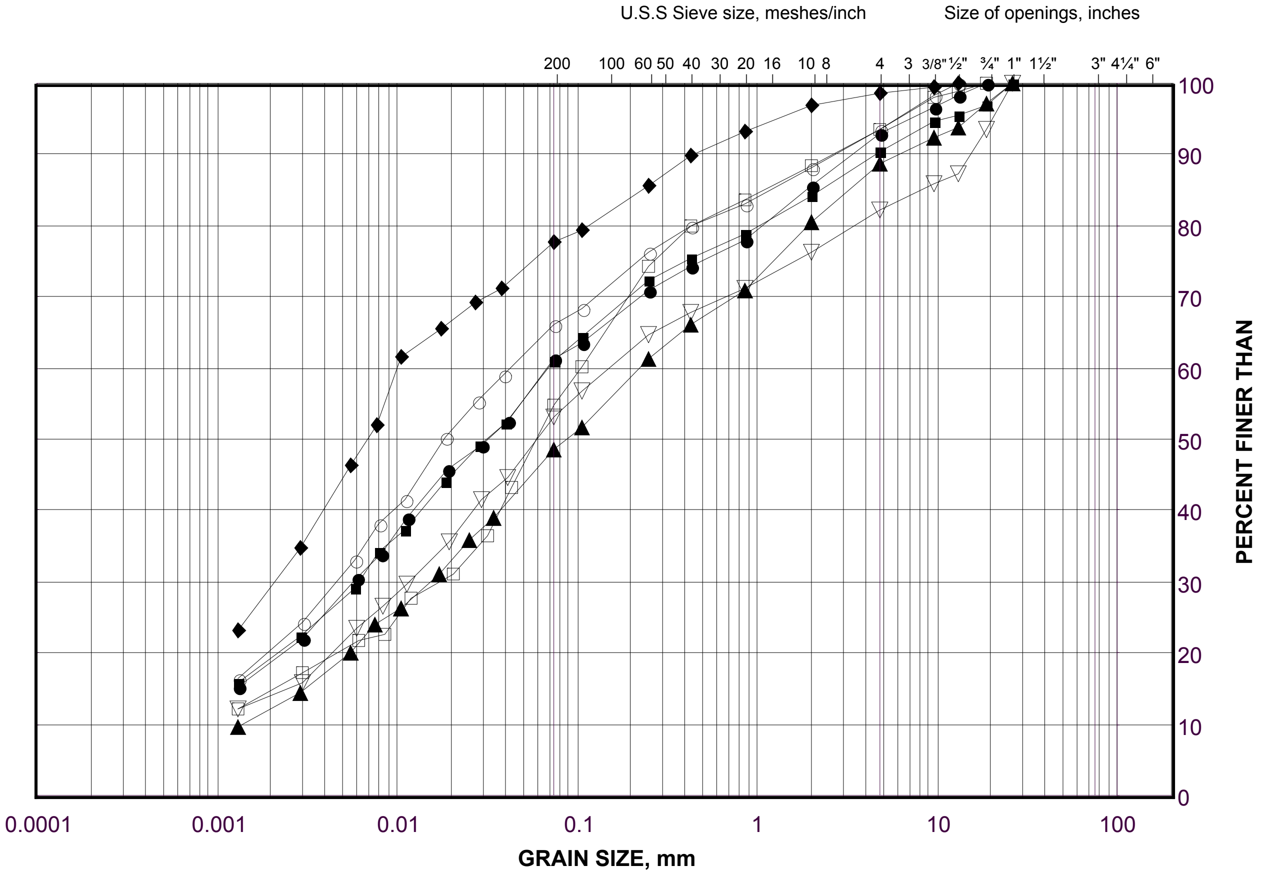
Golder Associates

Date: 05-Sep-12

GRAIN SIZE DISTRIBUTION

Clayey Silt with Sand Till

FIGURE B5B



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	MR-3	11	183.6
■	MR-4	12	181.4
◆	MR-4	15	176.8
▲	MR-4	17	173.7
▽	MR-3A	18	172.9
○	MR-4	19	167.7
□	MR-3A	21	163.8

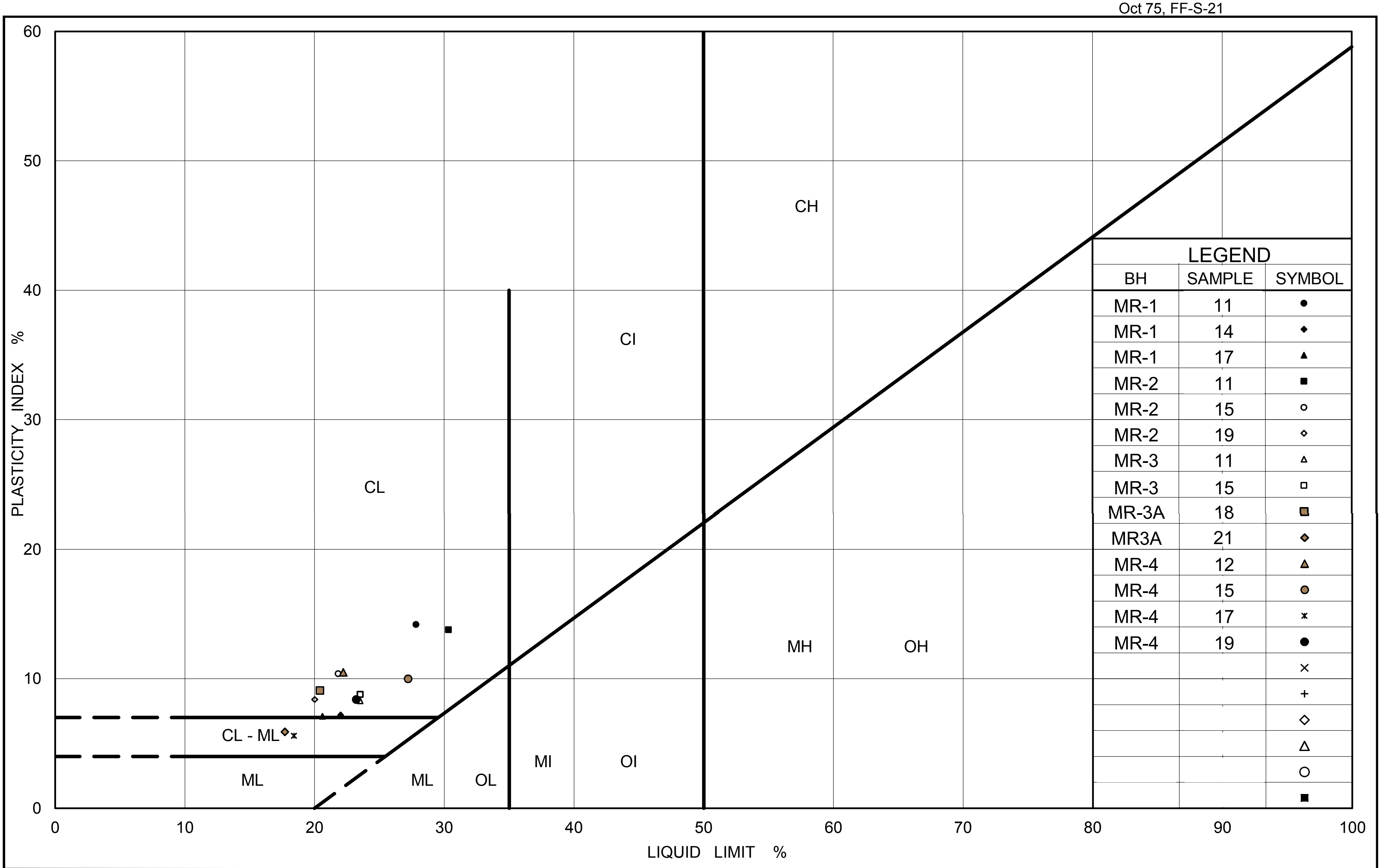
Project Number: 10-1111-0211

Checked By: TVA

Golder Associates

Date: 05-Sep-12

Oct 75, FF-S-21



LEGEND

BH	SAMPLE	SYMBOL
MR-1	11	●
MR-1	14	◆
MR-1	17	▲
MR-2	11	■
MR-2	15	○
MR-2	19	◇
MR-3	11	△
MR-3	15	□
MR-3A	18	■
MR3A	21	◆
MR-4	12	▲
MR-4	15	●
MR-4	17	✱
MR-4	19	●
		×
		+
		◇
		△
		○
		■

Ministry of
Transportation

Ontario

PLASTICITY CHART
Clayey Silt with Sand Till

Figure No. B6

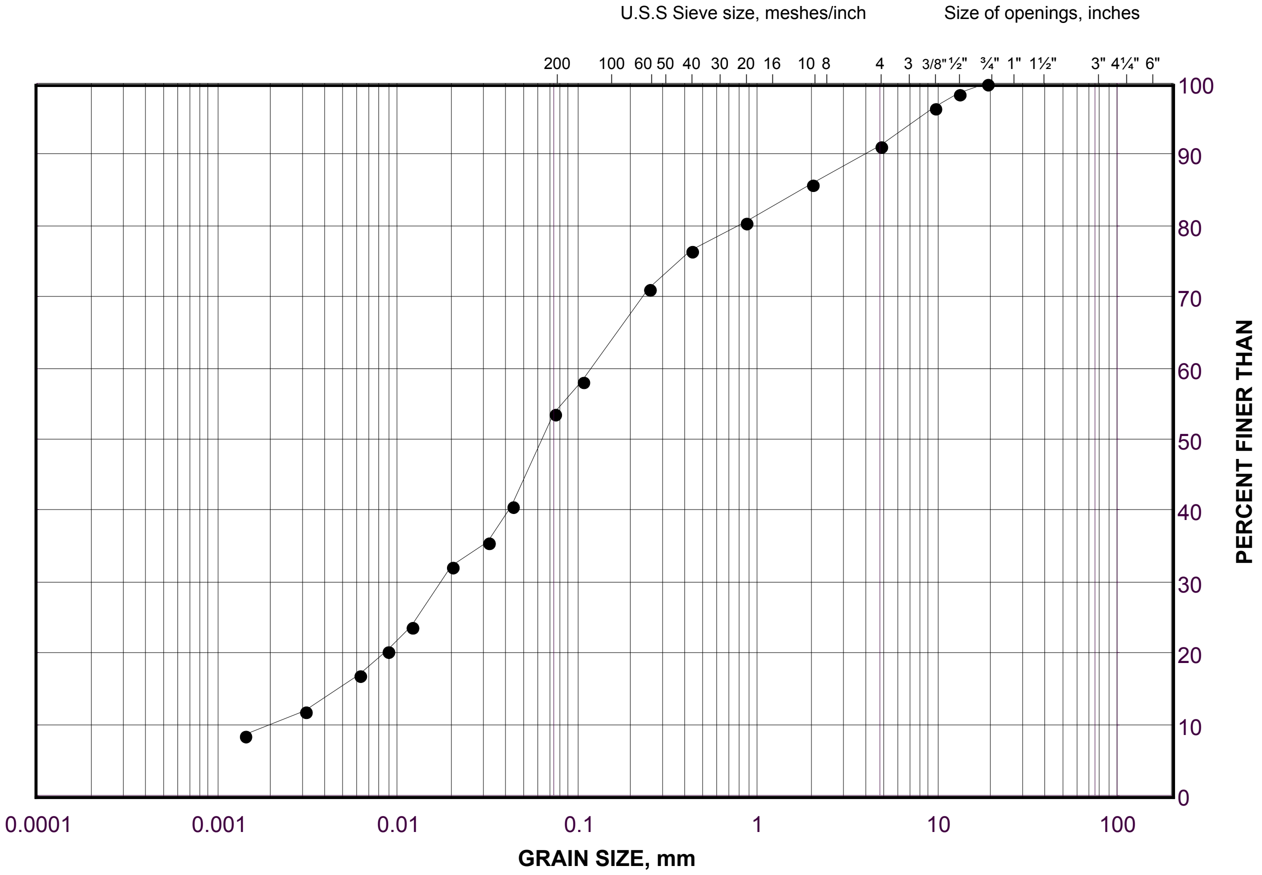
Project No . 10-1111-0211

Checked By: TVA

GRAIN SIZE DISTRIBUTION

Sand and Silt Till

FIGURE B7



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	MR-2	23	164.3

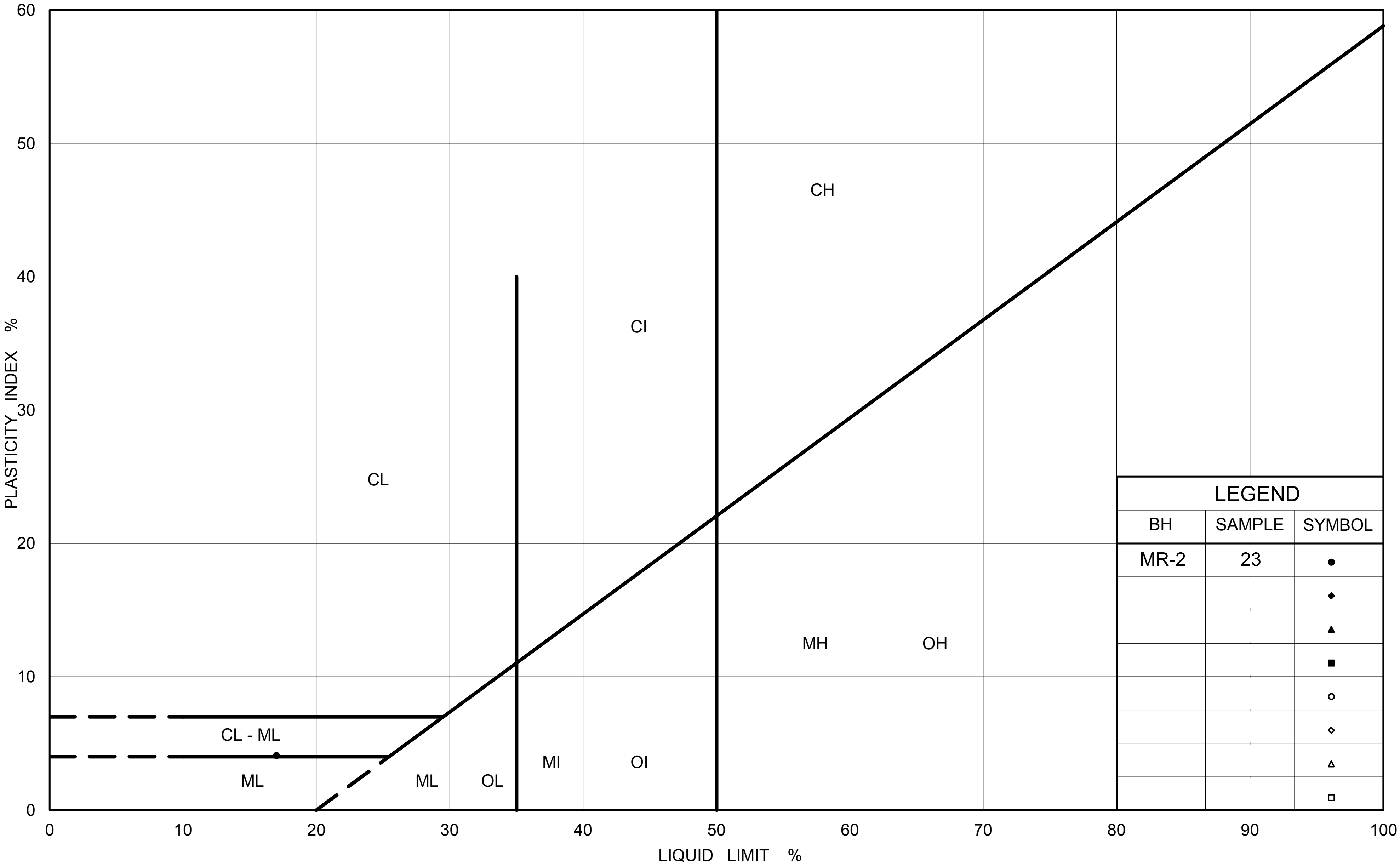
Project Number: 10-1111-0211

Checked By: TVA

Golder Associates

Date: 05-Sep-12

Oct 75, FF-S-21



Ministry of Transportation

Ontario

PLASTICITY CHART
Sand and Silt Till

Figure No. B8

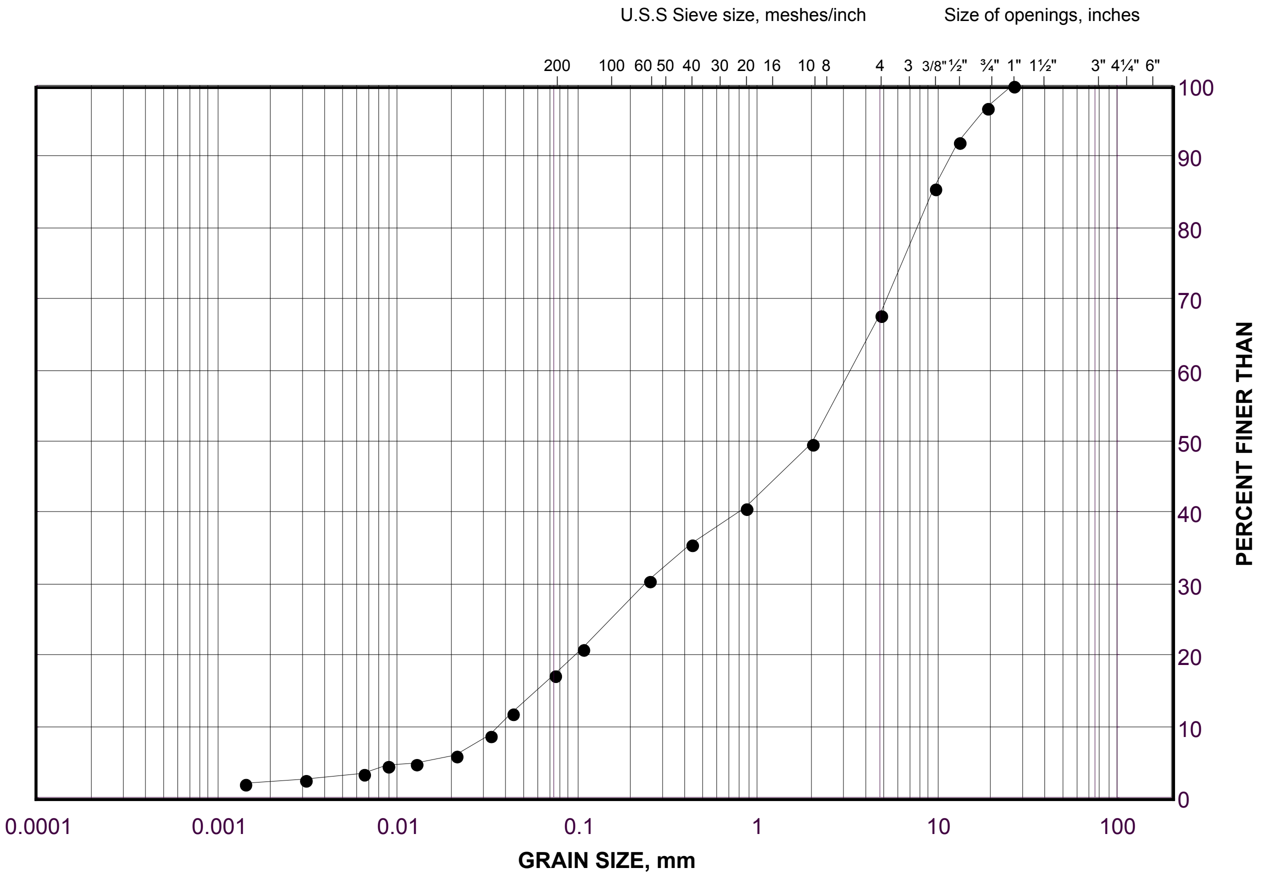
Project No. 10-1111-0211

Checked By: TVA

GRAIN SIZE DISTRIBUTION

Sand and Gravel

FIGURE B9



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	MR-2	25	158.2

Project Number: 10-1111-0211

Checked By: TVA

Golder Associates

Date: 05-Sep-12

APPENDIX C

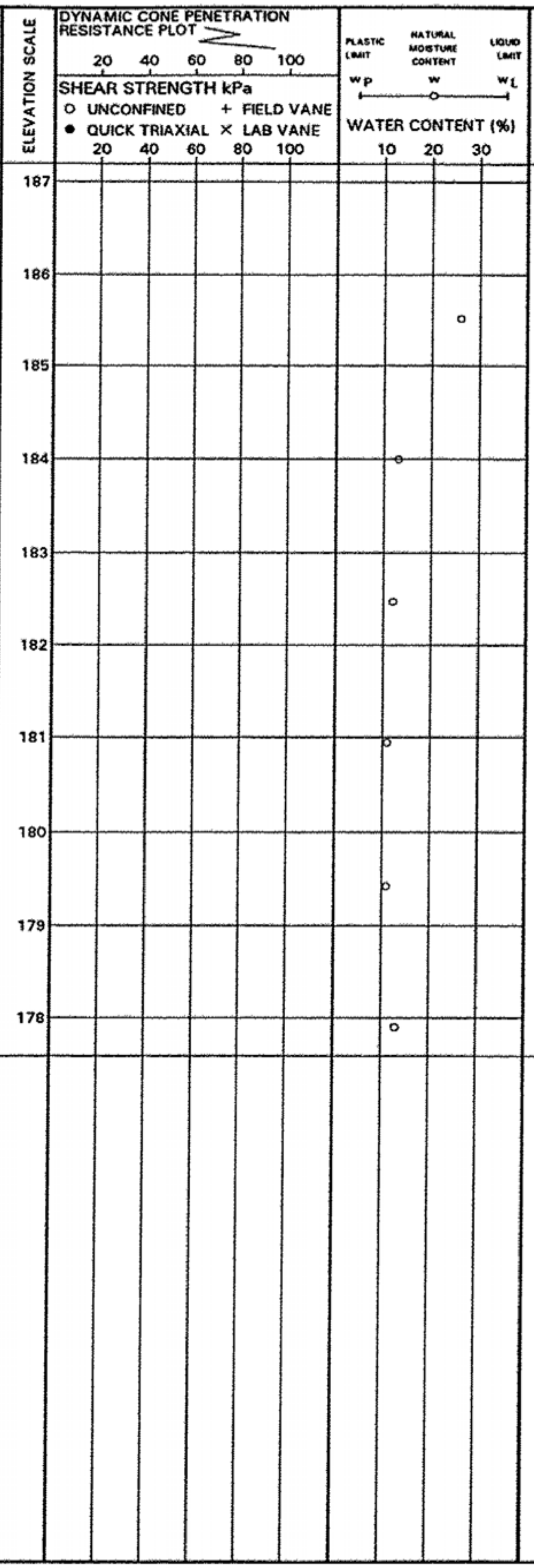
Records of Boreholes from Previous Investigation

RECORD OF BOREHOLE No 97-1

1 OF 1 **METRIC**

W.P. 311-89-00
LOCATION Co-ordinates 4 830 351.1 N; 604 238.4 E.
ORIGINATED BY TR
DIST 6 HWY 401
BOREHOLE TYPE 100mm diameter Solid Stem Auger
COMPILED BY JB
DATUM Geodetic
DATE 12.18.97 - 12.18.97
CHECKED BY IC

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
187.2	Ground Level						
0.0	Fill - mixture of shale fragments and silt Loose Grey						
185.5 1.7	Fill - Clayey Silt, some sand, traces of gravel and organics. Firm Dark Brown	1	SS	7			
184.7 2.5	Sandy Clayey Silt, traces of gravel Hard Brown	2	SS	40			
		3	SS	61			
181.6 5.6	Sandy Clayey Silt, traces of gravel Very Stiff Grey	4	SS	23			
		5	SS	19			
		6	SS	20			
177.6 9.6	End of Borehole Borehole dry upon completion of drilling.						



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

RECORD OF BOREHOLE No 97-2

1 OF 2

METRIC

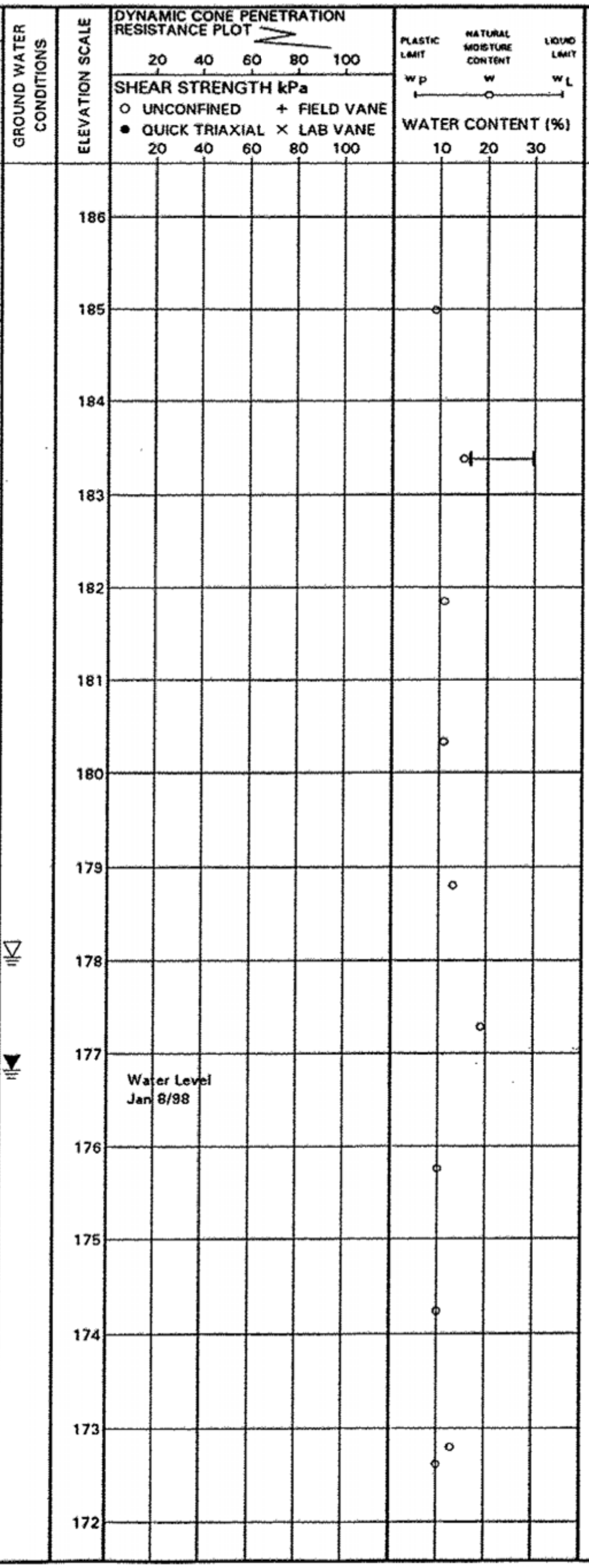
W.P. 311-89-00	LOCATION Co-ordinates 4 830 334.4 N; 604 208.7 E.	ORIGINATED BY TR
DIST 6 HWY 401	BOREHOLE TYPE 100mm diameter Solid Stem Auger	COMPILED BY JB
DATUM Geodetic	DATE 12.18.97 - 12.18.97	CHECKED BY IC

SOIL PROFILE

ELEV DEPTH	DESCRIPTION
186.6	Ground Level
0.0	Fill - mixture of shale fragments and silt Loose/Compact Grey
184.0 2.5	Silty Clay, some sand, trace of gravel (Till) Hard Brown/Grey
182.5 4.1	Sandy Clayey Silt, traces of gravel (Till) Brown Grey Very Stiff to Hard
178.0 8.6	Silty Clay, some sand, traces of gravel Very Stiff Grey
176.5 10.1	Sandy Clayey Silt, traces of gravel Very Stiff to Hard Grey

SAMPLES

NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR	SA	SI	CL
1	SS	50/ 5cm				
2	SS	41	3	18	37	42
3	SS	36	4	30	40	26
4	SS	24				
5	SS	16				
6	SS	19				
7	SS	48				
8	SS	27				
9	SS	85/ 25cm				



UNIT WEIGHT γ kN/m³

Continued Next Page

+³, ×³: Numbers refer to Sensitivity

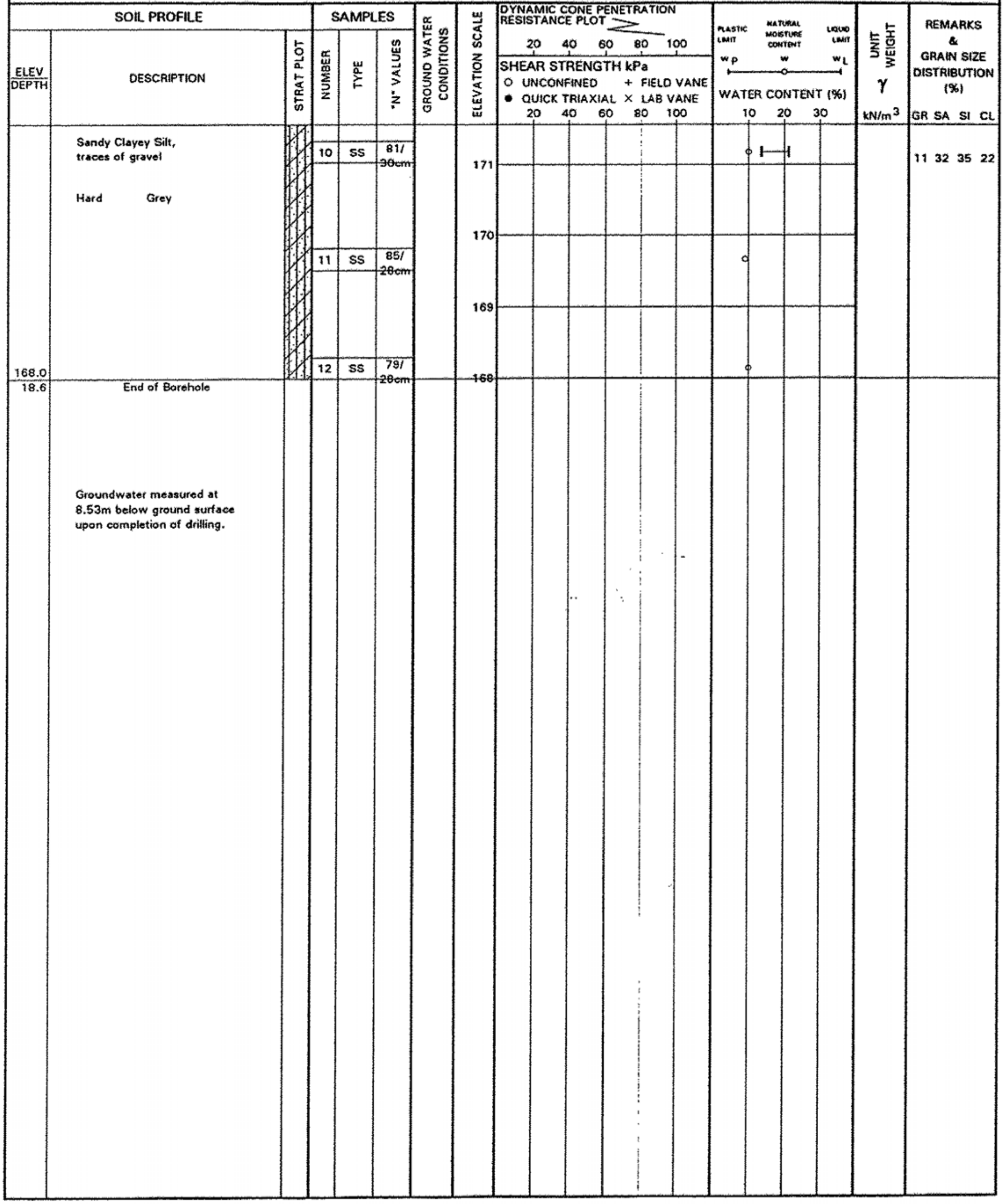
○³% STRAIN AT FAILURE

Ministry of Transportation
Ontario

RECORD OF BOREHOLE No 97-2

2 OF 2 **METRIC**

W.P. 311-89-00
LOCATION Co-ordinates 4 830 334.4 N; 604 208.7 E.
ORIGINATED BY TR
DIST 6 HWY 401
BOREHOLE TYPE 100mm diameter Solid Stem Auger
COMPILED BY JB
DATUM Geodetic
DATE 12.18.97 - 12.18.97
CHECKED BY IC



ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	Sandy Clayey Silt, traces of gravel Hard Grey	10	SS	81/30cm	11 32 35 22
		11	SS	85/28cm	
168.0 18.6		12	SS	79/28cm	
	End of Borehole				

Groundwater measured at 8.53m below ground surface upon completion of drilling.

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

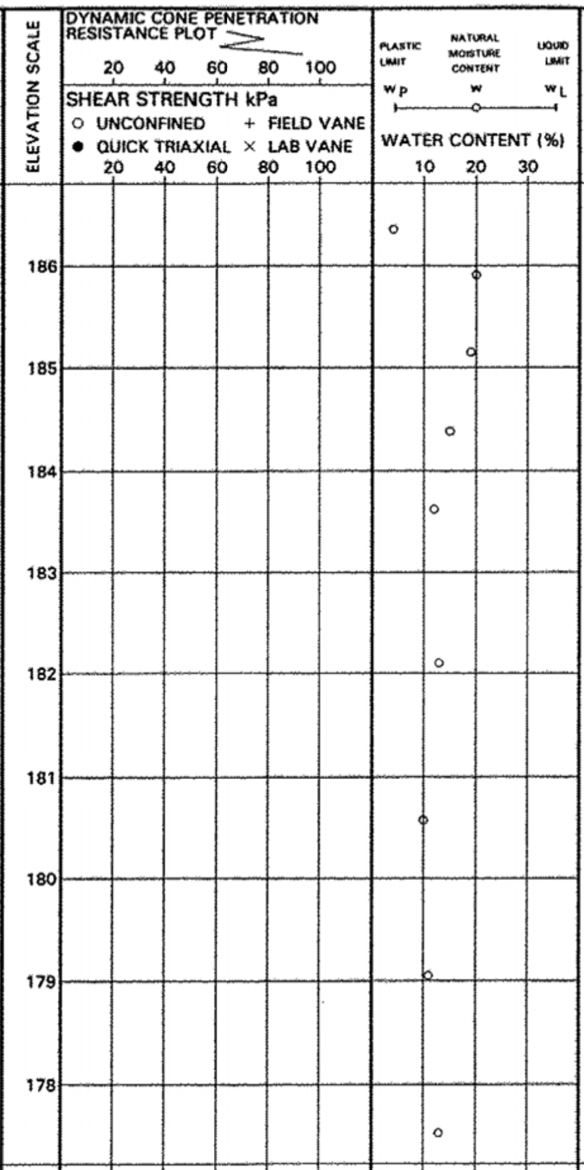
Ministry of Transportation
Ontario

RECORD OF BOREHOLE No 97-3

1 OF 1 **METRIC**

W.P. 311-89-00
LOCATION Co-ordinates 4 830 334.5 N; 604 256.0 E.
ORIGINATED BY TR
DIST 6 HWY 401
BOREHOLE TYPE 100mm diameter Solid Stem Auger
COMPILED BY JB
DATUM Geodetic
DATE 12.17.97 -
CHECKED BY IC

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
186.8	Ground Level									
0.0 186.4	Fill - Sandy Gravel	1	SS	79						
0.5	Fill - Silty Clay, some sand, traces of gravel Stiff to Very Stiff Greenish-Grey	2	SS	19						
		3	SS	13						
184.7 2.1	Sandy Clayey Silt, traces of gravel (Till) Very Stiff to Hard Brown	4	SS	18						
		5	SS	21						
		6	SS	36						
	Grey	7	SS	26						
179.7 7.1	Silty Clay, some sand trace of gravel (Till) Very Stiff Grey	8	SS	29						
		9	SS	19						
177.2 9.6	End of Borehole Borehole dry upon completion of drilling.									



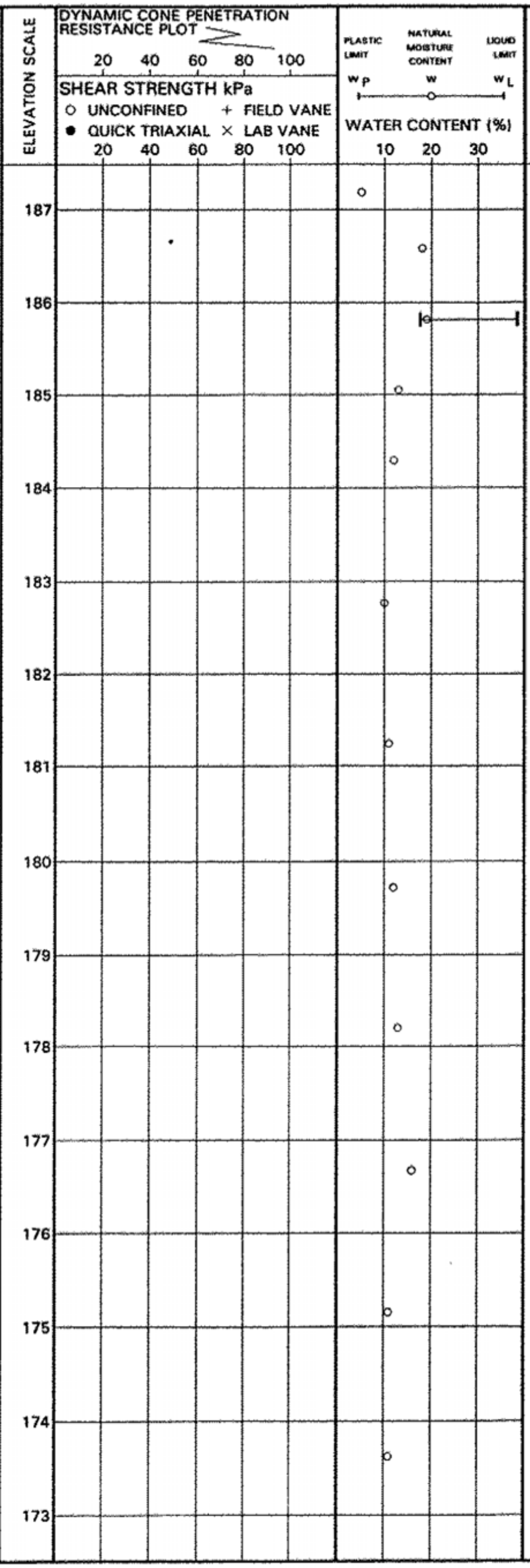
+³, ×³: Numbers refer to Sensitivity ○³% STRAIN AT FAILURE

RECORD OF BOREHOLE No 97-4

1 OF 2 **METRIC**

W.P. 311-89-00
LOCATION Co-ordinates 4 830 316.5 N; 604 225.5 E.
ORIGINATED BY TR
DIST 6 HWY 401
BOREHOLE TYPE 100mm diameter Solid Stem Auger
COMPILED BY JB
DATUM Geodetic
DATE 12.16.97 -
CHECKED BY IC

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES		GR	SA	SI	CL
187.5	Ground Level								
180.0	Asphalt								
0.2 186.9	Granular	1	SS	34					
0.6	Fill - Silty Clay, some sand traces of gravel	2	SS	12					
185.4	Stiff to Very Stiff Brown & Grey	3	SS	20		2	11	34	53
2.1	Sandy Clayey Silt, traces of gravel (Till)	4	SS	22					
	Very Stiff Brown	5	SS	26					
	Grey	6	SS	26					
		7	SS	17		8	39	31	22
178.9		8	SS	17					
8.6	Silty Clay, some sand, traces of gravel (Till) Stiff Grey	9	SS	9					
176.1		10	SS	10					
11.4	Sandy Clayey Silt, traces of gravel (Till) Hard Grey	11	SS	33	▽				
		12	SS	38					



UNIT WEIGHT γ kN/m³

Continued Next Page

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

RECORD OF BOREHOLE No 97-4

2 OF 2

METRIC

W.P. 311-89-00

LOCATION Co-ordinates 4 830 316.5 N; 604 225.5 E.

ORIGINATED BY TR

DIST 6 HWY 401

BOREHOLE TYPE 100mm diameter Solid Stem Auger

COMPILED BY JB

DATUM Geodetic

DATE 12.16.97 -

CHECKED BY IC

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100	PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m^3	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
	Sandy Clayey Silt, trace of gravel (Till)	▨	13	SS	43		172		○		
171.8 15.7	End of Borehole										
	Borehole dry upon completion of drilling.										

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

RECORD OF BOREHOLE No 97-5

1 OF 2

METRIC

W.P. 311-89-00
LOCATION Co-ordinates 4 830 312.9 N; 604 274.9 E.
ORIGINATED BY TR
DIST 6 HWY 401
BOREHOLE TYPE 100mm diameter Solid Stem Auger
COMPILED BY JB
DATUM Geodetic
DATE 12.15.97 - 12.15.97
CHECKED BY IC

SOIL PROFILE		SAMPLES			REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL
186.1 0.0	Ground Level Topsoil	1	SS	5				
185.8 0.3	Sandy Clayey Silt, trace gravel (Till) Hard Greyish Brown	2	SS	42				
		3	SS	50	3	32	40	25
		4	SS	42				
		5	SS	41				
	Grey	6	SS	45				
180.6 5.5	Sandy Clayey Silt, trace to some gravel (Till) Very Stiff Grey	7	SS	30				
		8	SS	25	11	28	42	19
177.4 8.6	Sandy Clayey Silt, traces of gravel (Till) Hard Grey	9	SS	33				
		10	SS	39				
		11	SS	51				
		12	SS	120/ 23cm	8	37	41	14

GROUND WATER CONDITIONS

ELEVATION SCALE

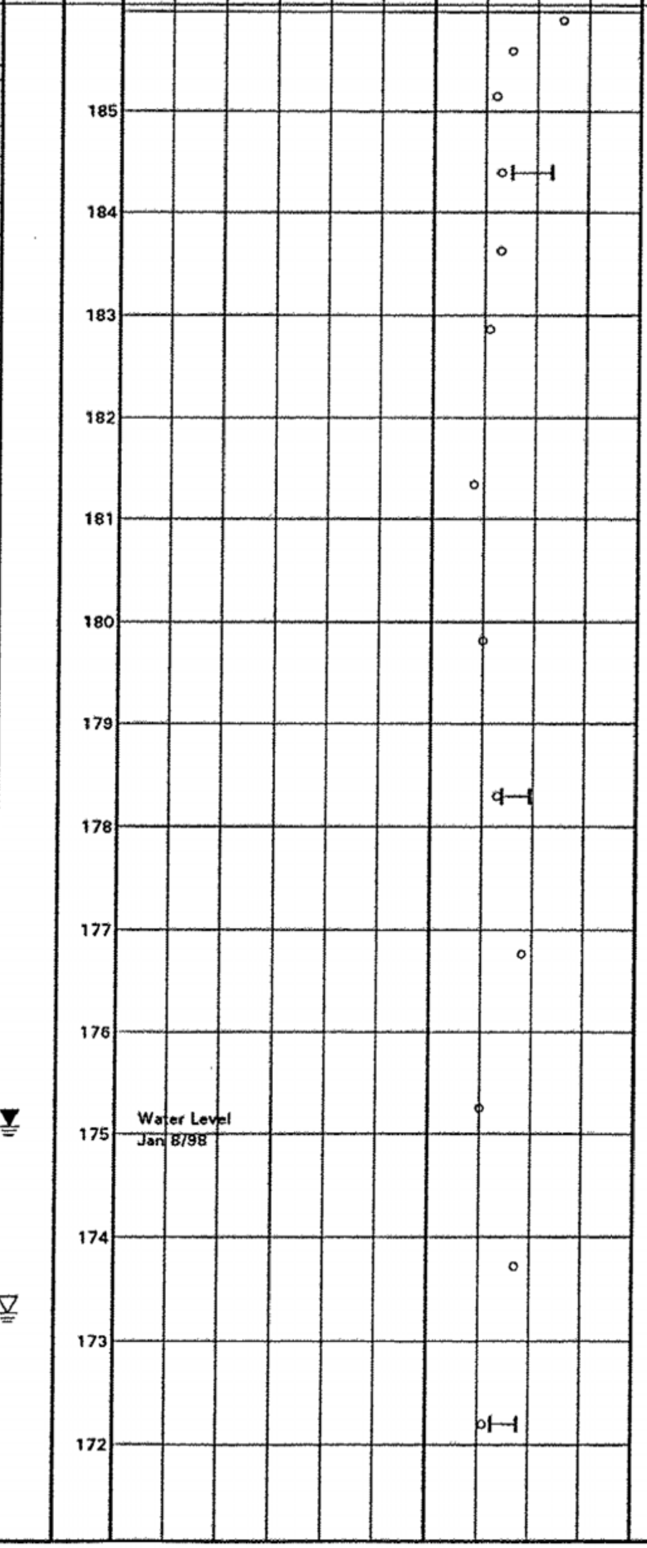
DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100

SHEAR STRENGTH kPa
○ UNCONFINED + FIELD VANE
● QUICK TRIAXIAL × LAB VANE
20 40 60 80 100

PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L

WATER CONTENT (%) 10 20 30

UNIT WEIGHT γ kN/m³



Continued Next Page

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

RECORD OF BOREHOLE No 97-5

2 OF 2

METRIC

W.P. 311-89-00

LOCATION Co-ordinates 4 830 312.9 N; 604 274.9 E.

ORIGINATED BY TR

DIST 6 HWY 401

BOREHOLE TYPE 100mm diameter Solid Stem Auger

COMPILED BY JB

DATUM Geodetic

DATE 12.15.97 - 12.15.97

CHECKED BY IC

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100	PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m^3	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
170.5	Sandy Clayey Silt, traces of gravel	▨	13	SS	95/ 23cm				o		
15.5	End of Borehole										

Groundwater measure at 12.8m below ground surface at end of drilling.

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

Ministry of Transportation
Ontario

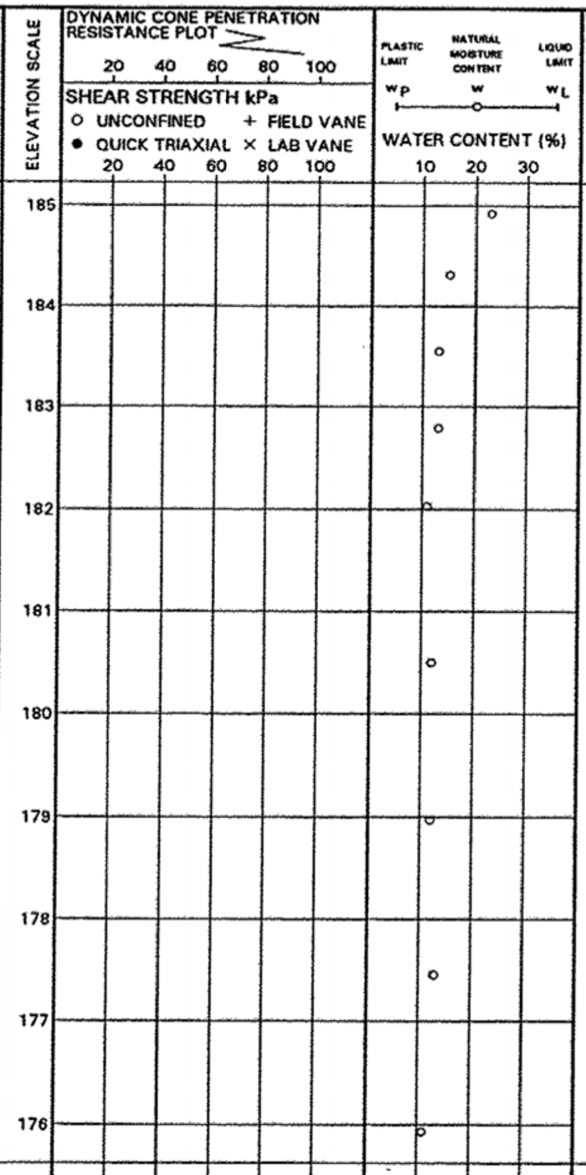
Foundation Design

RECORD OF BOREHOLE No 97-6

1 OF 1 **METRIC**

W.P. 311-89-00
LOCATION Co-ordinates 4 830 295.7 N; 604 244.2 E.
ORIGINATED BY TR
DIST 6 HWY 401
BOREHOLE TYPE 100mm diameter Solid Stem Auger
COMPILED BY JB
DATUM Geodetic
DATE 12.15.97 - 12.15.97
CHECKED BY IC

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
185.2 0.0	Ground Level						
184.9 0.3	Topsoil	1	SS	6			
	Sandy Clayey Silt, traces of gravel (Till) Very Stiff Brown to Hard	2	SS	28			
		3	SS	85			
		4	SS	26			
		5	SS	24			
	Grey	6	SS	21			
		7	SS	29			
		8	SS	24			
176.7 8.5					▽		
	Sandy Clayey Silt, traces of gravel (Till) Hard Grey	9	SS	42			
175.6 9.6	End of Borehole						



Groundwater measured at 8.57m below ground surface upon completion of drilling.

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

APPENDIX D

Non-Standard Special Provisions

WORKING SLAB - Item No.

Non-Standard Special Provision

1.0 Scope

This Special Provision covers the requirements for the supply and placement of a concrete working slab under structure foundations.

2.0 References

This Special Provision refers to the following standards, specifications or publications:

Ontario Provincial Standard Specifications, Construction

OPSS 902	Excavating and Backfilling - Structures

3.0 Definitions - Not Used

4.0 Design And Submission Requirements - Not Used

5.0 Materials

Concrete for working slabs shall have a minimum 28 day strength of 20 MPa.

6.0 EQUIPMENT - Not Used

7.0 CONSTRUCTION

7.01 Excavation

Excavation for the working slab shall be according to OPSS 902.

7.02 Protection of Founding Soil

Following inspection and approval of the prepared subgrade, a working slab with a minimum thickness of 100 mm shall be placed on the foundation subgrade as specified in the Contract Documents.

7.03 Protection of Founding Bedrock

The surface of the footing founding rock shall be exposed, cleaned and any loose or fractured parts removed so that sound rock is exposed. The working slab shall be placed on the exposed cleaned sound founding rock surface as specified in the Contract Documents.

Thickness of the mass concrete pad shall depend on the slope and irregularities in the exposed founding rock surface. A nominal thickness and a footprint plan view area has been specified on the Contract Documents

7.04 Dewatering

Dewatering shall be carried out according to OPSS 902.

8.0 Quality Assurance - Not Used

9.0 Measurement For Payment - Not Used

10.0 Basis of Payment

10.01 Working Slab - Item

Payment at the Contract price for the above tender item shall be full compensation for all labour, Equipment and Material to do the work.

END OF SECTION

VIBRATION MONITORING - Item No.

Non-Standard Special Provision

Scope

This special provision describes requirements for vibration monitoring during piling / caisson installation works for the Rehabilitation of the Highway 401 – Mavis Road Underpass structure.

References

The subsurface conditions at the site are described in the following Foundation Investigation Report for G.W.P 2150-01-00:

> Foundation Investigation Report, Mavis Road Underpass, Highway 401 Widening from Highway 403/410 Interchange to the Credit River, City of Mississauga, Region of Peel, GWP 2150-01-00

Definitions

Quality Verification Engineer (QVE): An Engineer with a minimum of five (5) years experience in the field of installation of piling and vibration monitoring or, alternatively, with expertise demonstrated by providing satisfactory quality verification services for a minimum of two (2) projects of similar scope to the contract. The QVE shall be retained by the Contractor to ensure general conformance with the contract documents and issue certificates of conformance.

Submission Requirements

The Contractor/QVE shall submit details of the vibration monitoring plan to the Contract Administrator for review. The submittals shall satisfy the specifications and at a minimum contain the following specific information:

- Qualifications of vibration monitoring specialist.
- Details regarding proposed instrumentation.
- Proposed location of instruments on existing Highway 401 structures.
- Proposed frequency of readings.
- Proposed methods for adjusting piling methods if readings show vibrations exceeding tolerable levels.

Monitoring

The vibration monitoring equipment shall be placed on the existing Mavis Road structure, as close as possible to the piling works. The Contractor/QVE shall take readings on the existing structures during driving of each pile, starting with the pile furthest away for each span area.

The vibrations measured on the existing structure shall not exceed 100 mm/s (peak particle velocity) for permanent components of the bridge that will remain as part of the bridge rehabilitation option.

The results shall be submitted to the Contract Administrator after each pile has been driven, prior to continuing with the subsequent piles. As a minimum, the pile number, location, set criteria and driving log must be submitted with vibration monitoring results.

If the vibration monitoring results are acceptable, the Contractor may continue with the next pile(s) with readings taken during driving of each pile. The results of subsequent piles should be submitted to the Contract Administrator after each pile has been driven.

If the readings are not within the limits stated above, the Contractor must alter the driving procedures until the vibrations at the existing structures are within acceptable levels. The above process must be repeated for each pile.

Basis of Payment

Payment at the lump sum contract price for this tender item shall be full compensation for all labour, equipment and materials for completion of the work.

END OF SECTION

CSP FOR INTEGRAL ABUTMENTS – Item No

Non-Standard Special Provision

Scope

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

Submission and Design Requirements

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

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

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

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

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

Material

Corrugated steel pipe

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

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

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

Sand Fill

The sand fill for backfilling the CSP shall meet the gradation requirements of Table 1 below:

Table 1 – Sand Fill Gradation Requirements

MTO Sieve Designation		Percentage Passing by Mass
2 mm	#10	100%
600 μm	#30	80% to 100%
425 μm	#40	40% to 80%
250 μm	#60	5% to 25%
150 μm	#100	0% to 6%

Construction

The sequence of construction shall be in accordance with the working drawings and as follows, unless otherwise approved:

1. Construct levelling pad and place CSPs and spacers.
2. Install piles by driving to design criteria.
3. Place loose sand into 600 diameter CSP.
4. Remove temporary spacers.

The CSP shall be positioned such that the piles are centrally positioned within the CSP. Temporary blocking and bracing shall be used to hold the CSP in position.

The Contractor shall ensure the full perimeters of the tops of all CSPs at each abutment are at the elevation and orientation shown on the working drawings.

The CSP at each pile shall be constructed to the following tolerances:

Criteria	Tolerance
Maximum deviation of CSP from pile centroid	+/- 50 mm
Maximum deviation of any point on the top perimeter of the CSP from the specified elevation	+/- 10 mm

The sand fill shall be placed dry of optimum and free-flowing, completely filling the volume between the CSP and pile. No additional compaction effort other than the action of placing the sand itself shall be applied to the sand fill.

The placing of the sand fill shall be carried out in a manner such as to not damage and displace the CSP.

Basis of Payment

Payment at the contract price for the above tender item shall include all labour, equipment and material required to do the work.

END OF SECTION

OBSTRUCTIONS - Item No.

Non-Standard Special Provision

The clayey silt with sand till deposit contains cobbles and boulders as indicated in the Record of Borehole sheets and as inferred from difficulties in advancing augers/auger grinding. Consideration of the presence of these obstructions must be made in the selection of appropriate equipment and procedures for driving steel H-piles/ pipe piles or caissons and possible pre-augering for deep foundations.

Basis of Payment

Payment at the lump sum contract price for this tender item shall be full compensation for all labour, equipment and materials for completion of the work.

END OF SECTION

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

Africa	+ 27 11 254 4800
Asia	+ 852 2562 3658
Australasia	+ 61 3 8862 3500
Europe	+ 356 21 42 30 20
North America	+ 1 800 275 3281
South America	+ 55 21 3095 9500

solutions@golder.com
www.golder.com

Golder Associates Ltd.
6925 Century Avenue, Suite #100
Mississauga, Ontario, L5N 7K2
Canada
T: +1 (905) 567 4444

Golder Associates