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REPORT ON

Foundation Investigation and Design Cataraqui River Bridge Replacement Structure Site No. 7-70 Highway 401, Kingston, Ontario G.W.P. 79-99-00

Submitted to:
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REPORT



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PART A

**FOUNDATION INVESTIGATION REPORT
CATARAQUI RIVER BRIDGE REPLACEMENT
HIGHWAY 401, KINGSTON, ONTARIO
G.W.P. 79-99-00**



1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by MMM Group Ltd. (MMM) on behalf of the Ministry of Transportation, Ontario (MTO) to carry out foundation investigations associated with the detailed design of the Cataraqui River bridge replacement as well as a culvert extension and several overhead sign installations as part of the widening of Highway 401 between Gardiners Road and Highway 15. This report presents the results of a foundation assessment conducted for the replacement of the Cataraqui River bridge (Site 7-70) on Highway 401, located about 2.2 km east of Montreal Road in Kingston, Ontario.

A Preliminary Foundation Investigation and Design Report prepared by Golder at the Functional Design stage of the bridge replacement project titled, “*Preliminary Foundation Investigation and Design, Cataraqui River Bridge Widening, Structure Site No. 7-70, Highway 401, Kingston, Ontario, W.P. 80-99-01*”, dated November 2013 (GEOCRES No. 31C-215), included previously collected subsurface information pertinent to the site from the following sources:

- Report prepared by Foundation of Canada Engineering Corporation Ltd. (FENCO) for the MTO (then the Ontario Department of Highways) titled “*Report to Ontario Department of Highways on Soil Conditions, Cataraqui River Bridge, Kingston, Ontario*”, dated April 21, 1954 (GEOCRES No. 54-F201C); and,
- Report prepared by Golder for the MTO titled “*Foundation Investigation and Design, Highway 401 Embankment Widening, Cataraqui Wetlands, Kingston, Ontario, G.W.P. 78-99-00*”, dated October 2012 (GEOCRES No. 31C-203).

The previously collected subsurface information relevant to the current design is also included in the Appendices that follow this report.

The terms of reference and scope of work for the foundation engineering services are outlined in MTO’s Request for Proposal (RFP) for Assignment No. 4013-E-0027 dated March 2014, and in Golder’s proposal for services related to the Cataraqui River bridge submitted to MMM on May 6, 2014.



2.0 SITE DESCRIPTION

The site is located on Highway 401, about 2.2 km east of Montreal Road in Kingston, Ontario. The bridge was originally constructed in 1957 and consists of a three-span structure, approximately 93 m long and 28 m wide. It currently accommodates four travelled lanes (two eastbound and two westbound). Based on the available General Arrangement drawings from 1954, prepared by FENCO, the piers and abutments are supported on concrete caissons founded on bedrock between about Elevation 68 m and 73 m.

The natural ground surface within the lowland area surrounding the Cataraqui River valley is relatively flat at about Elevation 76 m. The embankments that approach the existing bridge consist of embankment fill that is up to about 9 m high at the existing bridge structure. The Highway 401 pavement grade is at approximately Elevation 84.7 m at the bridge abutments. The highway embankment side slopes are oriented at about 2 horizontal to 1 vertical (2H:1V) in the immediate vicinity of the existing bridge. Based on visual observation during site reconnaissance visits and the drilling investigation, the existing embankment slopes are relatively well vegetated and appear to be stable.

The Cataraqui River flows south through the site and into Lake Ontario. The Cataraqui River water level was measured to range between about Elevation 74.7 m and 74.9 m in March and April 2015.



3.0 INVESTIGATION PROCEDURES

3.1 Current (2015) Investigation

The subsurface investigation for the proposed bridge replacement was carried out between March 2 and 23, 2015. During that time, fifteen boreholes (numbered 15-301 to 15-306 and 14-401 to 14-409, inclusive) were advanced at the locations shown on Drawings 1 to 4; the records for these boreholes are contained in Appendix A. The boreholes were advanced as follows:

- Boreholes 15-301, 15-302, 15-305, and 15-306 were advanced near the abutment locations with 108 mm inside diameter continuous-flight hollow-stem augers with a truck-mounted drill rig, supplied and operated by Marathon Drilling Ltd. of Ottawa, Ontario. The boreholes were advanced to depths between about 8.2 m and 17.1 m, below the existing pavement/ground surface in the overburden. Boreholes 15-301 and 15-302 were then cored between about 3.1 and 2.9 m, respectively, into the bedrock using NQ-size coring equipment.
- Boreholes 14-303 and 14-304 (advanced near the west pier), and 15-401 to 15-409, inclusive (advanced at the approach embankment toes), were advanced using portable drilling equipment supplied and operated by Marathon Drilling Ltd. of Ottawa, Ontario. The boreholes were advanced to depths between about 1.8 and 6.1 m below the existing ground surface in the overburden. Boreholes 15-303 and 15-304 were then cored about 2.7 and 2.3 m, respectively, into the bedrock using BQ-size coring equipment to facilitate a sustainable drilling production rate in the strong granitic bedrock.

Soil samples in the boreholes were obtained at vertical intervals of about 0.60 to 1.52 m, using a 50 mm outer diameter split-spoon sampler in accordance with Standard Penetration Test (SPT) procedures.

A monitoring well was installed in Boreholes 15-305 and 15-306 to monitor the groundwater level at the site. The monitoring wells consist of a 32 mm diameter rigid PVC pipe with a 1.5 m long slotted screen section, installed within silica sand backfill and sealed by a section of bentonite pellet backfill.

The boreholes were backfilled with bentonite pellets mixed with native soils in the overburden, and bentonite pellets in the bedrock. The site conditions were restored following completion of work.

The field work was supervised by a member of Golder's technical staff, who located the boreholes, supervised the drilling, sampling and in situ testing operations, logged the boreholes, and examined the soil and bedrock samples. The samples were identified in the field, placed in appropriate containers, labelled, and transported to Golder's laboratory facility in Ottawa for further examination. Index and classification tests consisting of grain size distributions, Atterberg limits, organic contents and water contents were carried out on selected soil samples at the Ottawa laboratory. Unconfined compressive strength tests were carried out on selected rock core samples in Golder's Mississauga laboratory. All of the laboratory tests were carried out to MTO and/or ASTM standards as appropriate.

The borehole locations were determined by Golder in relation to existing site features. The ground surface elevations were also surveyed by Golder using a precision GPS survey unit. The boreholes and locations, including MTM NAD83 northing and easting coordinates and ground surface elevations referenced to Geodetic datum, are summarized in the following table and are shown on Drawings 1 to 4.



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Borehole Number	Borehole Location	Northing (m)	Easting (m)	Ground Surface Elevation (m)
15-301	West Abutment (Median)	4904853.7	308916.6	84.7
15-302	East Abutment (Median)	4904888.6	309022.5	84.7
15-303	West Pier (Central)	4904864.8	308939.3	75.9
15-304	West Pier (North)	4904875.1	308937.6	75.9
15-305	West Embankment (WB Shoulder)	4904865.0	308905.0	84.5
15-306	East Embankment (EB Shoulder)	4904882.0	309025.0	84.5
15-401	West Embankment Toe (North)	4904868.2	308834.2	77.3
15-402	West Embankment Toe (North)	4904882.4	308878.2	76.8
15-403	West Embankment Toe (North)	4904892.1	308918.0	75.7
14-404	East Embankment Toe (North)	4904917.4	309002.1	75.9
14-405	East Embankment Toe (North)	4904929.1	309039.9	78.1
14-406	East Embankment Toe (North)	4904937.7	309073.8	78.3
14-407	East Embankment Toe (South)	4904856.5	309015.3	76.0
14-408	East Embankment Toe (South)	4904858.4	309057.0	76.0
14-409	East Embankment Toe (South)	4904873.8	309094.3	75.6

Notes: 1) Northing and Easting coordinates shown are relative to the MTM NAD83 (Zone 9) coordinate system.
2) Ground surface elevations shown are relative to Geodetic Datum.



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3.2 Previous Investigations

The field work for the original investigation was carried out by FENCO between January 25 and February 11, 1954. At that time, a total of eight boreholes with dynamic penetration tests and three additional dynamic penetration tests were carried out at the site.

The borehole locations were measured relative to site features shown on the original site plan included in the FENCO report and their approximate locations relative to the current bridge deck and highway alignment are shown on Drawings 1 to 4. Boreholes BH 2, BH 3, and BH 5 were put down on the west side of the river. Boreholes BH 6, BH 7, BH 8, BH 9, and BH 12, and penetration tests PT 13, PT 14, and PT 15 were put down on the east side of the river. The borehole records from the FENCO investigation are provided in Appendix B, along with FENCO's Drawing No. 102-C-2 showing the borehole locations.

Golder carried out a subsurface investigation west of the site in 2009 as part of the detail design for the proposed widening of Highway 401 to the west of the Cataraqui River Bridge. Boreholes E22, E23, and E24, put down as part of that investigation, are relevant to the current assignment. The boreholes were located near the toe of the existing embankment and advanced using portable/manual drilling equipment, to depths ranging from 2.1 to 3.2 m below the existing ground surface. The borehole logs and laboratory test results from the 2009 Golder investigation are provided in Appendix B. Grain size distribution tests carried out on samples of the overburden are shown on Figures B1 and B2 in Appendix B. The results of Atterberg limit tests carried out on samples in boreholes west of the bridge site are shown on A-line plots on Figures B3 and B4 in Appendix B.

The FENCO report provided approximate borehole locations, relative to site features, and surveyed borehole elevations (relative to Geodetic datum). The locations and ground surface elevations of the boreholes advanced in 2009 were determined following drilling by Golder personnel at the site using a Trimble R8 GPS unit. The ground surface elevations at the time of drilling and approximate site locations are summarized in the table below, and are shown on Drawings 1 to 4.

Borehole Number	Investigation	Adjacent Site Feature	Ground Surface Elevation (m)	Borehole Depth (m)
West Side of Cataraqui River				
BH 2	FENCO, 1954	West Abutment (North)	76.1	8.2
BH 3	FENCO, 1954	West Abutment (South)	75.1	9.9
BH 5	FENCO, 1954	West Shoreline (South)	74.6	9.6
E22	Golder, 2009	West Embankment Toe (South)	77.1	6.7
E23	Golder, 2009	West Embankment Toe (South)	76.6	3.2
E24	Golder, 2009	West Embankment Toe (South)	75.1	2.1



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Borehole Number	Investigation	Adjacent Site Feature	Ground Surface Elevation (m)	Borehole Depth (m)
East Side of Cataraqui River				
BH 6	FENCO, 1954	East Shoreline (North)	75.1	11.5
BH 7	FENCO, 1954	East Shoreline (South)	75.0	6.3
BH 8	FENCO, 1954	East Abutment (North)	75.6	11.0
BH 9	FENCO, 1954	East Abutment (South)	75.9	11.1
BH 12	FENCO, 1954	East Shoreline (Central)	75.1	6.3
PT 13	FENCO, 1954	East Shoreline (South)	75.1	2.1
PT 14	FENCO, 1954	East Shoreline (South)	75.6	2.1
PT 15	FENCO, 1954	East Shoreline (North)	75.0	3.4

Note: Ground surface elevations were determined at the time of each geotechnical investigation.



4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

This area of Highway 401 lies within the southern portion of the physiographic region known as the Napanee Plain, as delineated in *The Physiography of Southern Ontario*¹.

The Napanee Plain is flat to undulating, and is characterized by relatively shallow soil deposits overlying bedrock. Geologic mapping indicates that the bedrock within the southern portion of the area consists of both granitic rock and crystalline limestone. In many areas bedrock outcrops exist at ground surface, while deeper soil deposits (on the order of 10 m) are present in the northern and southern portion of the Plain, and within and adjacent to river valleys throughout the Plain.

The overburden soils within the Napanee Plain generally consist of glacial till, although alluvium is present in river and stream valleys and, in the southern portion of the Plain, low-lying areas are typically covered with deposits of stratified clay.

In particular, the study area lies within the lowland area surrounding the Cataraqui River. The Cataraqui River is characterized by a number of lakes joined by the river. This river flows southerly towards Kingston and is one of two major rivers in the area.

4.2 Subsurface Conditions

The detailed subsurface soil and groundwater conditions encountered in the boreholes put down as part of the current investigation and the results of related in situ and laboratory testing are given on the Record of Borehole and Drillhole sheets contained in Appendix A. The borehole logs and geotechnical test results from the previous investigations, carried out in 1954 (prior to construction of the bridge) and 2009 are included in Appendix B. The results of geotechnical laboratory testing carried out as part of the current investigation are included in Appendix C.

The interpreted stratigraphic conditions along the centreline of the existing bridge and at the proposed abutment and pier and locations are shown on Drawings 1 to 4. The stratigraphic boundaries shown on the Record of Borehole sheets and on the interpreted stratigraphic sections included on Drawings 1 to 4 are inferred from non-continuous sampling and, therefore, represent transitions between soil types rather than exact planes of geological change. The subsoil conditions will vary between and beyond the borehole locations.

In general, the subsurface conditions at the site consist of embankment or grade fill overlying very stiff to hard silty clay to clay, which is underlain by a thin deposit of sand and gravel at some borehole locations. The soil deposits are underlain by granite bedrock.

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

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



4.2.1 Pavement Structure and Embankment Fill

The Highway 401 pavement structure was penetrated in the eastbound median at Boreholes 15-301 and 15-302, the westbound shoulder at Borehole 15-305, and the eastbound shoulder at Borehole 15-306. At the borehole locations, the pavement structure consists of about 100 mm to 300 mm of asphalt overlying about 100 mm to 600 mm of gravelly sand base course. The granular base is underlain by a total of up to about 9.0 m of subbase/embankment fill.

The upper portion of the subbase/embankment fill generally consists of sand, with varying amounts of gravel and silt. This upper, granular embankment fill was penetrated at depths of up to about 2 m at the west approach embankment, and 1.0 m at the east approach embankment. The results of grain size distribution testing carried out on three samples of the upper, sand embankment fill are provided on Figure C1 in Appendix C. The measured water contents of the samples of upper, granular embankment fill vary from 2 to 8 percent.

The upper sandy embankment fill is underlain by sandy gravel to gravel with some sand. At the east abutment, the gravel embankment fill was fully penetrated at depths of about 3 m. At the west abutment, the gravel embankment fill extended to depths of about 8 m to 9 m. Samples of the gravel fill obtained from the lower portions of the west approach embankment, below about 6 m depth, contained fractured limestone pieces and cobbles and provided more difficult drilling, with grinding of the augers; however, it was penetrable by augering. The results of grain size distribution testing carried out on seven samples of the gravelly embankment fill are provided on Figure C2 in Appendix C. The results of grain size distribution testing carried out on a sample with a higher silt content are shown on Figure C3. The measured water contents of the samples of upper, granular embankment fill vary from 2 to 15 percent.

Silty clay fill was encountered beneath the gravelly embankment fill at Borehole 15-301, put down through the west approach embankment, and at all boreholes put down through the east approach embankment. The results of grain size distribution testing carried out on five samples of the silty clay fill are included on Figure C4 in Appendix C. Atterberg limits tests carried out on samples of the silty clay fill measured plasticity index values ranging from 20 to 47 percent and liquid limit values ranging from about 42 to 74 percent. The results of the Atterberg limits tests are shown on a plasticity chart on Figure C5 in Appendix C; these results confirm that the tested cohesive fill samples vary from silty clay to clay of intermediate to high plasticity. The measured water content of the silty clay fill ranged from approximately 32 to 40 percent.

The Standard Penetration Test (SPT) “N” values measured in the embankment fill range from 4 to greater than 50 blows per 0.3 m of penetration but were typically on the order of 8 to 12 blows per 0.3 m of penetration, indicating a generally compact relative density in the granular fill and a generally very stiff consistency in the silty clay to clay fill.

4.2.2 Organic Deposits and Grade Fill (Embankment Toes)

Organic deposits (topsoil or peat) were encountered at the ground surface at Boreholes 14-403 and 14-409 with a thickness of about 200 and 900 mm, respectively. A buried layer of organics and sand was encountered below the fill in Borehole 14-406 with a thickness of about 600 mm.

Fill was encountered at the ground surface at Boreholes 15-303, 15-304, and 14-401 to 14-408, except Borehole 14-403 where it was encountered below the topsoil. The fill generally consists of sand and silt with varying amounts of gravel and clay. The fill is present to depths ranging from about 0.6 m to 3.1 m below the existing ground surface (Elevation 73.7 to 76.7 m).



The SPT “N” values measured in the fill range from 4 to greater than 50 blows per 0.3 m of penetration indicating a loose to very dense relative density. The higher blow counts were typically recorded in the near surface portions of the fill deposits and may reflect penetration resistance due to the frozen ground encountered at the time of the investigation.

4.2.3 Silty Clay to Clay

A deposit of silty clay to clay was encountered beneath the fill materials at all borehole locations, with the exception of Borehole 15-304 where no native overburden was encountered. The silty clay to clay deposit was proven to depths ranging from about 11.6 m to 17.1 m below the existing Highway 401 grade (Elevations 67.4 to 73.9 m). The silty clay to clay contains varying amounts of sand and trace gravel. A 0.6 m thick layer of sandy silt was encountered in the cohesive deposit in Borehole 14-409 at a depth of about 2.4 m (Elevation 73.2 m).

The measured SPT ‘N’ values range from about 5 to 40 blows per 0.3 m of penetration, but more typically between 8 and 20 blows per 0.3 m of penetration. In situ vane shear strength testing was conducted at selected locations following lower SPT “N” values, and measured undrained shear strengths greater than 100 kPa. The results of the in-situ SPT and vane shear strength testing indicate that this deposit has a generally stiff to very stiff consistency.

The results of grain size distribution testing carried out on nine samples of the silty clay are included on Figure C6 in Appendix C. Atterberg limits tests carried out on samples of the deposit gave plasticity index values ranging from 17 to 35 percent and liquid limit values ranging from about 35 to 59 percent; these results are presented on a plasticity chart on Figure C7 in Appendix C, indicating that the cohesive deposit consists of silty clay to clay of intermediate to high plasticity. The measured water content of the silty clay ranged from approximately 25 to 41 percent.

4.2.4 Sandy Silt

A deposit consisting of sandy silt up to about 0.5 m thick was encountered below the silty clay to clay deposit and immediately above the bedrock at Borehole 15-302. Similar thin, non-cohesive layers were encountered below the silty clay and immediately above the bedrock in Boreholes 5 and 6 from the 1954 FENCO investigation; at these locations, the layer is described as sand and gravel.

One SPT “N” value of 76 blows per 0.3 m of penetration was measured in this deposit, indicating a very dense relative density.

The result of a grain size distribution test carried out on one sample of the sandy silt is included on Figure C8 in Appendix C.

4.2.5 Bedrock

Refusal to auger advancement was encountered in Boreholes 15-305 and 15-306, and refusal to split-spoon sampler advancement was encountered in Boreholes 14-401 to 14-403; this refusal has been inferred to represent the bedrock surface. Bedrock was encountered beneath the fill and/or silty clay at Boreholes 15-301 to 15-304 where it was cored for depths of between about 2.3 and 3.1 m.



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The following table summarizes the bedrock surface depths and elevations encountered at the borehole locations during the current and previous investigations. The bedrock surface elevation is variable within each proposed foundation area.

Foundation Area	Borehole Number	Existing Ground Surface Elevation (m)	Depth to Bedrock (m)	Bedrock Surface Elevation (m)
West Approach Embankment	15-401	77.3	3.3	74.0*
	15-402	76.8	3.7	73.1*
	E22 (2009)	77.1	-	-
	E23/E23A (2009)	76.6	3.2	73.4*
West Abutment	15-301	84.7	11.5	73.2
	15-305	84.5	11.9	72.6*
	15-403	75.7	3.5	72.2*
	BH 2 (1954)	76.1	2.9	73.2
	BH 3 (1954)	75.1	5.3	69.8
	E24 (2009)	75.1	2.1	73.0*
West Pier	15-303	75.9	2.4	73.5
	15-304	75.9	1.8	74.1
	BH 5 (1954)	74.6	4.9	69.7
East Pier	BH 6 (1954)	75.1	7.0	68.1
	BH 7 (1954)	75.0	1.6	73.4
	BH 12 (1954)	75.1	1.8	73.3
	PH 13 (1954)	75.1	2.1	73.0*
	PH 15 (1954)	75.0	3.4	71.6*
East Abutment	15-302	84.7	15.0	69.7
	15-306	84.5	17.1	67.4*
	BH 8 (1954)	75.6	6.4	69.2
	BH 9 (1954)	75.9	6.5	69.4
	PH 14 (1954)	75.6	2.1	73.5*

Note: * Depth and elevation to bedrock inferred from refusal to augering and split-spoon penetration.



The bedrock encountered in the cored boreholes of the current investigation typically consists of pink, green, and brown granite bedrock. The bedrock is generally fresh, fine to medium grained, non-porous, and strong to very strong.

The Rock Quality Designation (RQD) values measured on the recovered bedrock core samples typically ranged from about 63 to 100 percent, indicating fair to excellent quality rock. The discontinuities observed in the rock core were associated with the joints, veins, faults and fractures of the bedrock.

Laboratory unconfined compressive strength testing was carried out on three selected specimens of the bedrock core. The results of the testing are summarized on Figure C7 in Appendix C. The measured unconfined compressive strengths range from 45 to 112 MPa indicating a medium strong to very strong bedrock.

4.3 Groundwater Conditions

The groundwater conditions were observed in the open boreholes following completion of drilling, and these observations are noted on the borehole records in Appendix A; however, these measurements in cohesive deposits are not considered to represent the long-term, stabilized groundwater level at the site.

The groundwater levels measured in the piezometers in Boreholes 15-305 and 15-306 are summarized in the table below:

Borehole	Ground Surface Elevation (m)	Water Level Depth (m)	Water Level Elevation (m)	Date
15-305	84.5	7.4	77.1	March 18, 2015
15-306	84.5	9.5	75.0	March 24, 2015

The water level in the Cataraqui River varies from about Elevation 74.6 to 74.9 m as shown below:

Date	River Surface Elevation (m)
March 1954	74.6
March 10, 2015	74.7
April 17, 2015	74.9

It should be noted that groundwater levels in the area are subject to fluctuations both seasonally and with precipitation events.



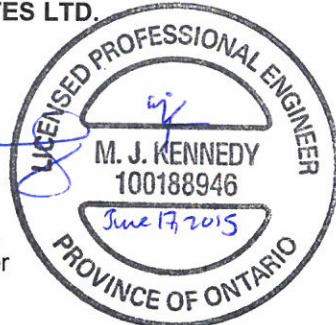
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5.0 CLOSURE

This Foundation Investigation Report was prepared by Mr. Matt Kennedy, P.Eng., and reviewed by Ms. Lisa Coyne, P.Eng., a geotechnical engineer and Principal with Golder. Mr. Fin Heffernan, P.Eng., the Designated MTO Foundations Contact for this assignment, conducted an independent quality review of this report.

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PART B

**FOUNDATION DESIGN REPORT
CATARAQUI RIVER BRIDGE REPLACEMENT
HIGHWAY 401, KINGSTON, ONTARIO
W.P. 79-99-00**



6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

6.1 General

This section of the report provides foundation design recommendations for the proposed replacement of the existing Cataraqui River bridge on Highway 401. The recommendations are based on interpretation of the factual data obtained from the boreholes advanced during the current subsurface investigation as well as those included in the original 1954 foundation design report (FENCO) and Golder's 2012 report addressing widening of the Highway 401 embankments to the west of the site. The discussion and recommendations presented are intended to provide the designers with sufficient information to assess the feasible foundation alternatives to carry out the detail design of the foundations for the replacement structure.

Where comments are made on construction, they are provided to highlight those aspects that could affect the detail design of the project, and for which special provisions 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, and scheduling.

6.2 Existing Foundations

The existing bridge consists of a four-lane, three-span structure that was originally constructed in 1957. The middle span is about 31.4 m long, and the two outer spans are about 29.3 m long. The existing bridge deck is about 27.5 m wide. Based on the original General Arrangement drawings, dated August 16, 1954 (Drawings TWP #20-70-1-A, TWP #20-70-2-A, and TWP #20-70-3-A), the abutments and piers are founded on concrete caissons bearing on the bedrock surface, which was encountered at the following elevations according to the 1954 FENCO investigation and the available drawings:

Foundation Element	Founding Elevation (m)	Depth Below Floodplain Grade (m)
West Abutment	70.1	5.9
West Pier	71.0 – 72.8	5.0 – 3.2
East Pier	70.4 – 73.2	5.6 – 2.9
East Abutment	68.6	7.4

Note: Floodplain elevation assumed to be about Elevation 76.0 m.

The 1954 General Arrangement drawings indicate that each abutment foundation consists of four caissons with rectangular footprints approximately 3.7 m by 5.5 m, in plan. The drawings also indicate that each pier consists of four columns, each supported on a circular concrete caisson with a diameter of about 2.6 m. The concrete caissons are encased in corrugated steel sheet piling.

According to the 1954 structural drawings and the base and contour plans provided therein, the average floodplain grade and base of the river channel are at about Elevation 76.0 and 71.9 m, respectively.



6.3 Foundation Options for Replacement Structure

It is understood that the preferred alternative for the proposed replacement consists of a three-span structure on the same highway alignment, with an overall bridge deck width increase of about 10 m to accommodate a total of six lanes. The new bridge structure is to have longer span lengths of about 31 m, 43 m, and 31 m and will be supported on foundation elements constructed behind the footprints of the existing foundation elements. The proposed Highway 401 pavement grades at the new structure will approximately match the existing pavement grades.

Based on the subsurface conditions, several foundation options have been considered for the replacement of the Cataraqui River Bridge. A summary of the advantages and disadvantages associated with each option is provided below, and a comparison of the alternative foundation options based on advantages, disadvantages, risks and approximate costs is provided in Table 1 following the text of this report.

- **Driven steel piles:** Based on the composition of the existing embankment fill, steel H-piles or pipe piles driven to refusal on the granitic bedrock are feasible and suitable for support of the bridge abutments and would minimize the need for protection system requirements and groundwater control requirements. This option is considered to be the most cost effective at the abutments. However, if rock sockets are required for pile toe fixity the steel piles would be less practical (as the piles will not be able to penetrate by driving into the medium strong to very strong granitic bedrock). At the pier locations, the depth to bedrock is relatively shallow and this would necessitate a socket into the bedrock to provide sufficient length and achieve pile toe fixity for lateral/seismic resistance. This option would also require construction of a pier pile cap in the water, inside a cofferdam or above the water level; the installation of a cofferdam to the surface of the sloping, strong to very strong bedrock poses construction, dewatering and concreting challenges. Given these considerations, driven piles are not considered suitable for support of the piers for the replacement structure.
- **Drilled steel casings:** Drilled steel casings, which are typically on the order of 450 mm to 750 mm in diameter, could also be considered as a deep foundation option for support of the replacement bridge structure. This foundation option would have similar advantages to steel H-piles at the abutments in terms of minimizing excavation depth, protection system requirements and groundwater control requirements. Drilled steel casings also handle obstructions better than driven steel piles or larger diameter caissons, although this is anticipated to be a low risk at the proposed abutment areas. If required for pile toe fixity, bedrock sockets for drilled steel casings are typically more cost effective to construct than for larger diameter caissons, and would not require a separate operation to form the rock socket as would be the case for driven steel H-piles. At the abutments, if rock sockets are required to resist lateral/seismic forces, drilled steel casings are preferred from a geotechnical perspective over driven H-piles. At the piers, drilled steel casings would require the construction of a pile cap inside a cofferdam (with the associated challenges noted above) or above the water level, the drilled steel casings are considered to be appropriate for construction at the new pier locations, as the smaller diameter socket holes are more readily constructible and cost effective than larger diameter caissons.



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- **Concrete caissons:** Caissons deriving their support from bearing within the granitic bedrock are also feasible for this site. Their key advantage lies in the fact that the structure columns could be designed at each caisson to eliminate the need for caisson cap construction, thereby minimizing the need for cofferdams and minimizing environmental impacts at the pier locations. However, this environmental benefit comes with significant costs and risks for the large, approximately 1.35 m diameter caisson that would be required to support a 1.2 m diameter structural column. The caissons must be socketed into the bedrock a sufficient length to provide the required bearing and lateral/seismic resistance; the large socket that would be required to support the piers at this site would have to be formed in the rock by drop hammer holes or cluster drilling, which will be time consuming and expensive. As an alternative, consideration may be given to socketting larger diameter caissons nominally (0.2 m to 0.5 m) into the bedrock, creating a smooth base/seal with tremie concrete, and then developing the required lateral resistance through the installation of a group of dowels/micropiles in the base of each caisson. This approach is considered to have improved constructability and reduced risk compared with a large diameter caisson socketted deeper into the bedrock; however, this approach may not satisfy lateral/seismic loading requirements, subject to further analysis by the structural engineer during detail design. In either approach, the rig required for installation of large diameter caissons is relatively large and heavy, and appropriate access, working platform, and overhead clearance would be required at the various construction stages.
- **Micropiles:** Deep foundations consisting of drilled and grouted micropiles are feasible for support of the replacement bridge structure, particularly at the pier locations where the variable bedrock surface is relatively shallow. These smaller diameter installations can handle obstructions within the embankment fill, and penetrate the strong to very strong bedrock. In addition, a micropile rig is smaller and lighter than the equipment required for the installation of driven piles, drilled steel casings or concrete caissons, and it may be able to fit underneath the existing structure for construction of the replacement piers during staging. However, a larger number of micropiles would be required as compared with a drilled steel casing or concrete caisson option. Construction of a pile cap for a micropiled foundation is anticipated to be challenging at the pier locations due to dewatering and temporary protection requirements, as described above. As noted above for the drilled caisson option, it is also geotechnically feasible to use micropiles (or dowels) in conjunction with a large diameter caisson socketted nominally onto/into the bedrock surface.
- **Spread footings founded on bedrock:** Spread footings could be considered for support of the piers where the bedrock is relatively shallow. Supporting the piers on a combined foundation consisting of spread footings, bearing directly on the shallow portions of the bedrock, and a deep foundation method (as described above) to support the pier where the bedrock is deeper, may be considered. However, construction of spread footings on the bedrock several metres below the river level is anticipated to be challenging at the pier locations due to dewatering and temporary protection requirements, as described above. Spread footings are not considered to be feasible for support of the abutment foundations due to the depth to bedrock at these locations.

Based on the above considerations, the preferred options from a geotechnical/foundations perspective are as follows:

- Support the new abutments on steel H-piles driven to found on the bedrock or, if pile toe fixity is required to resist lateral/seismic loads, on drilled steel casings extended into the bedrock. The subsurface conditions are suitable for integral abutments, if these are feasible from a structural perspective.



- Support the piers on drilled steel casings socketed into the bedrock, with a pile cap constructed within a cofferdam below the existing floodplain/river bed or, potentially depending on environmental considerations and permits, with a pile cap constructed at the floodplain/river bed or a “floating” pile cap. It is also recommended that a combined foundation, supported directly on bedrock where it is shallow and supported on drilled steel casings where the bedrock is deeper, be assessed from a structural perspective; this approach may be particularly valid if the westbound and eastbound bridges will be structurally separate (see the interpreted bedrock profile at the piers as shown on Drawings 2 and 3).

For the pier construction, it is recommended that the use of prefabricated cofferdams be assessed by the structural engineer, with consideration from environmental and heritage/aesthetic inputs, as an alternative to a conventional pile cap constructed below grade using a conventional cofferdam. Prefabricated cofferdams are pre-constructed with pre-drilled holes and steel tube sleeves through the base large enough to accommodate the foundation pile elements. These types of cofferdams could be floated and then anchored into place, act as a template during foundation element installation and, upon completion of the pile/drilled steel casing installation, could be backfilled with concrete to form the pile cap(s).

6.4 Driven Steel H-Pile or Steel Pipe Pile Foundations

6.4.1 Founding Elevations

The abutments for the replacement structure may be supported on steel H-piles or closed-ended steel pipe piles driven to found on the granitic bedrock. Based on the borehole results from the investigation, and assuming about 0.1 m of penetration into the bedrock to allow for some weathering in the upper portion of the rock, the following pile tip elevations are recommended for design of steel H-piles or pipe piles. Reference should also be made to the interpreted stratigraphic cross-sections provided for each of the foundation elements on Drawings 3 and 4, to assess the variability in pile length along the foundation unit.

Foundation Element	Borehole Numbers	Bedrock Surface Elevation (m)	Design Pile Tip Elevation (m)	Reference
West Abutment	BH 2, BH 3, 15-301, 15-305	69.8 – 73.2	69.7 – 73.1	See Dwg. 3
East Abutment	BH 9, BH 8, 15-302, 15-306	67.4 – 69.2	67.3 – 69.1	See Dwg. 4

If a rock socket is required at the abutments for pile toe fixity, coring or use of a down-hole hammer would be necessary to form a socket in the strong granitic bedrock. Temporary liners would be required to maintain an open hole through the embankment fill and overburden soils during bedrock socket formation.

The abutment pile caps should be constructed at a minimum depth of 1.5 m below the ground surface for frost protection purposes, per OPSD 3090.101 (*Foundation Frost Penetration Depths for Southern Ontario*).

For H-piles driven to refusal on the medium strong to very strong bedrock, the tips should be fitted with Titus injector rock points (or equivalent) to protect the piles from damage during driving and promote seating on the sloping bedrock, in accordance with OPSS 903 (Deep Foundations). If steel pipe piles are used, driving shoes should be in accordance with OPSD 3001.100 Type II (Steel Tube Pile Driving Shoe).



6.4.2 Axial Geotechnical Resistance/Reaction

For design of HP 310x110 piles driven to found on the strong granitic bedrock at the estimated tip elevations provided in Section 6.4.1, the factored axial geotechnical resistance at Ultimate Limit States (ULS) may be taken as 2,300 kN. The structural designers must also consider the structural limitations of the H-pile. Serviceability Limit States (SLS) resistances do not apply to piles founded on the granitic bedrock, since the SLS resistance for 25 mm of settlement is greater than the factored axial geotechnical resistance at ULS. The same axial resistances may be used in the design of closed-end, concrete-filled, 324 mm diameter steel pipe piles having a minimum wall thickness of 9.5 mm.

As the highway profile is to remain essentially unchanged, little to no embankment grade raise will occur. The embankment will be widened by approximately 5 m on each side, requiring a maximum vertical thickness of approximately 2.5 m of new fill on top of the existing embankment side slopes. As discussed under Section 6.11.3, the settlement of the underlying stiff to hard clayey soils will be elastic (essentially immediate) and will be less than 25 mm; it is anticipated that some downdrag force would be generated on the piles at the north and south ends of the new abutments, within approximately 6 m of the end of the foundation element. It is recommended that the fill for the embankment widening immediately behind the new abutments be placed in advance of the foundation installation for the new abutments, in order to preload the abutment widening area for a two-month period in advance of foundation construction. This is discussed further in Section 6.12.3.

Pile installation should be in accordance with OPSS 903 (*Deep Foundations*). The drawings should incorporate the appropriate note stating that the piles should be equipped with bearing points and should be driven to bedrock. For piles driven to refusal on bedrock, it is a generally accepted practice to reduce the hammer energy after abrupt peaking is met on the bedrock surface, and to then gradually increase the energy over a series of blows to seat the pile.

Vibration monitoring should be carried out during pile installation to ensure that the vibration levels at the existing structure are maintained below tolerable levels. Further discussion of the recommended vibration monitoring requirements is included in Section 6.12.4.

6.4.3 Downdrag Load (Negative Skin Friction)

If the fill for embankment widening is placed after construction of the foundations (i.e., no preloading), the additional embankment fill will raise the effective stress level in the clayey soil below the widening area, producing settlement that could generate downdrag loads on the new abutment piles within about 6 m of the ends of the new abutments (i.e., within the zone of influence of the embankment widening area). These downdrag loads (i.e., negative skin friction) should be considered in design.

Assuming the underside of the pile cap is at Elevation 79.0 m, the unfactored downdrag load acting on a single HP 310x110 pile over the maximum length of pile is estimated to be about 250 kN at the east abutment and about 350 kN at the west abutment. This downdrag load should be applied to the piles located within 6 m of the north and south ends of the new abutments (i.e., within the potential zone of influence of the embankment widening); downdrag loads need not be applied to the piles in the central portion of the abutment. The structural capacity of the piles must be checked for the factored dead and downdrag loads in accordance with Section 6.8.4 of the *Canadian Highway Bridge Design Code (CHBDC)*.



6.4.4 Resistance to Lateral Loads

The resistance to lateral loading can be derived from the soil in front of the piles, and it may be assumed that this resistance will be nearly the same for vertical and inclined foundation elements as indicated in Section C6.8.7.2 of the *Commentary to the CHBDC*.

The Serviceability Limit States (SLS) geotechnical response of the soil in front of the piles under lateral loading may be calculated using subgrade reaction theory where the coefficient of horizontal subgrade reaction, k_h , is based on the equation given below, as described by Terzaghi (1955) and the *Canadian Foundation Engineering Manual* (3rd Edition).

For cohesionless soils:

$$k_h = \frac{n_h z}{B} \quad \text{where:} \quad \begin{array}{ll} n_h & \text{is the constant of horizontal subgrade reaction, as given below;} \\ z & \text{is the depth (m); and,} \\ B & \text{is the pile diameter/width (m).} \end{array}$$

For cohesive soils:

$$k_h = \frac{67 s_u}{B} \quad \text{where:} \quad \begin{array}{ll} s_u & \text{is the undrained shear strength of the soil (kPa); and,} \\ B & \text{is the pile diameter/width (m).} \end{array}$$

The table below provides ranges for the values of n_h and s_u to be used in the structural analysis for the Cataraqui River bridge replacement. Although driven piles are not considered feasible at the pier locations, parameters are provided here for the pier locations in addition to the abutments, for reference for other deep foundation options as addressed in subsequent sections of this report. The ranges in values reflect the following:

- The variability in the subsurface conditions and the soil properties;
- The approximate nature of the analysis;
- The non-linear nature of the soil behaviour (such that n_h is a function of deflection); and,
- The two extremes of the design – i.e., the requirement for flexibility in the case of integral abutments and the requirement for lateral resistance of horizontal loads.

Location		Elevation (m)	Soil Type	n_h (MN/m ³)	s_u (kPa)
West Abutment (Assumed PCL > Elev. 79.0 m)	North Portion (15-305)	PCL ¹ – 76.1	Sand and Gravel (Embankment Fill)	4 to 8	-
		76.1 – 72.5	Stiff Very Stiff Silty Clay	-	100 to 120
		Below 72.5	Bedrock	-	-
	South Portion (BH3)	PCL ¹ – 74.4	Sand and Gravel (Embankment Fill)	4 to 8	-
		74.4 – 72.0	Very Stiff Silty Clay	-	100 to 120
		72.0 – 70.2	Firm to Stiff Silty Clay	-	50 to 80
		70.2 – 69.8	Compact Sand and Gravel	3 to 5	-
		Below 69.8	Bedrock	-	-



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Location		Elevation (m)	Soil Type	n_h (MN/m ³)	s_u (kPa)
West Pier (Assumed PCL ~ Elev. 74.0 m)	North Portion (15-304)	Below 74.1	Bedrock	-	-
	South Portion (BH5)	PCL ¹ – 72.5	Very Stiff Silty Clay	-	100 to 120
		72.5 – 70.0	Stiff Silty Clay	-	75
		70.0 – 69.6	Compact Sand and Gravel	3 to 5	-
East Pier (Assumed PCL ~ Elev. 74.0 m)	North Portion (BH6)	Below 69.6	Bedrock	-	-
		PCL ¹ – 72.4	Very Stiff Silty Clay	-	100 to 150
		72.4 – 68.5	Stiff Silty Clay	-	80
		68.5 – 68.1	Compact Sand, Gravel, and Boulders	3 to 5	-
		Below 68.1	Bedrock	-	-
East Abutment (Assumed PCL > Elev. 79.0 m)	(15-302, 15-306)	PCL ¹ – 73.4	Sand and Gravel (Assumed Fill)	3 to 5	-
		Below 73.4	Bedrock	-	-
		PCL ¹ – 74.8	Silty Clay (Embankment Fill)	-	75 to 100
		74.8 – 72.3	Very Stiff Silty Clay	-	100 to 150
		72.3 – 68.5	Firm to Stiff Silty Clay	-	50 to 75
		Below 68.5	Bedrock	-	-

Note: ¹ PCL = Pile Cap Level.

If sufficient lateral resistance cannot be developed within the embankment fill and overburden soils based on the parameters provided above, rock sockets will be required. Recommendations regarding lateral resistance developed within the bedrock are provided in Section 6.5.4.

Group action for lateral loading should be considered when the pile spacing in the direction of the loading is less than six to eight caisson diameters. Group action can be evaluated by reducing the coefficient of lateral subgrade reaction in the direction of loading by a reduction factor as follows:

Caisson Spacing in Direction of Loading (d = Caisson Diameter)	Reduction Factor
8d	1.0
6d	0.7
4d	0.4
3d	0.25

For establishing the factored *structural* resistance at ULS, the shear force and bending moment distribution in the piles under factored loading can be assessed using the procedures and parameters given above for evaluating the SLS response of the pile.

The ULS *geotechnical* resistance to lateral loading may be calculated using passive earth pressure theory as outlined in Section C6.8.7 of the *Commentary to the CHBDC*.



The ULS lateral resistance of a pile group may be estimated as the sum of the individual pile resistances across the face of the pile group, perpendicular to the direction of the applied lateral force.

The ULS resistances obtained using the above parameters represent unfactored values; in accordance with the CHBDC, a resistance factor of 0.5 is to be applied in calculating the horizontal resistance.

6.4.5 Feasibility of Integral Abutments

As outlined in MTO's report SO-96-01, integral abutment bridges are single span or multiple span continuous deck type bridges with a movement system composed primarily of abutments on flexible integral foundations and approach slabs, in lieu of movable deck expansion joints and bearings at abutments. The feasibility of integral abutments is influenced by a number of factors including geometry and subsurface conditions. The primary criterion is the need to support the abutments on relatively flexible piles. Where the load bearing stratum is near the surface or where the use of short piles or caissons (less than 5 m in length) is planned, the site is not considered suitable for integral abutment bridges. Geometric constraints on the use of integral abutments are also applicable and include: overall bridge length less than 150 m; skew angle less than 35°; and abutment wall heights less than 6 m without a retained soil system.

The flexible pile-supported abutment foundations described above for the Cataraqui River bridge replacement are considered to meet MTO's foundation criteria for integral abutments.

6.5 Drilled Steel Casings

6.5.1 Founding Elevations

Ultimately, the founding elevation for drilled steel casings will depend on the required depth of rock socket to resist the lateral/seismic loads at the abutments and at the piers. The following table summarizes the range in bedrock surface elevation as encountered in the boreholes at/near each of the foundation elements, from which the total length for each drilled steel casing can be assessed based on the required rock socket length. It is noted that reference should also be made to the interpreted stratigraphic cross-sections provided for each of the foundation elements on Drawings 2 to 4, to assess the variability in the length of each drilled steel casing along the foundation unit.

Foundation Element	Borehole Numbers	Bedrock Surface Elevation (m)	Reference
West Abutment	BH 2, BH 3, 15-301, 15-305	69.8 – 73.2	See Drawing 3
West Pier	BH 5, 15-303, 15-304	69.7 – 74.1	See Drawing 2
East Pier	BH 6, BH 7, BH 12, PT 13, PT 15	68.1 – 73.4	See Drawing 2
East Abutment	BH 9, BH 8, 15-302, 15-306	67.4 – 69.2	See Drawing 4



6.5.2 Axial Geotechnical Resistance/Reaction

This foundation type develops its axial capacity based on the shear resistance in the rock socket wall, not end-bearing. For design purposes, the factored axial resistance at ULS for drilled steel casing rock sockets may be taken as follows at the abutments and piers:

Drilled Steel Casing Diameter (mm)	Factored Axial Geotechnical Resistance at ULS	Geotechnical Reaction at SLS
450	1,850 kN/m length of rock socket	N/A
600	2,450 kN/m length of rock socket	N/A
750	3,050 kN/m length of rock socket	N/A

As noted in the above table, the geotechnical reaction at SLS (for 25 mm of settlement) for this foundation type in medium strong to very strong granitic bedrock will be greater than the ULS resistance. It is estimated that less than 5 mm of deformation will occur under loads approaching the ULS resistance as provided above.

As the highway profile is to remain essentially unchanged, little to no embankment grade raise will occur. The embankment will be widened by approximately 5 m on each side, requiring a maximum vertical thickness of approximately 2.5 m of new fill on top of the existing embankment side slopes. As discussed under Section 6.11.3, the settlement of the underlying stiff to hard clayey soils will be elastic (essentially immediate) and will be less than 25 mm; it is anticipated that some downdrag force would be generated on the piles at the north and south ends of the new abutments, within approximately 6 m of the end of the foundation element.. It is recommended that the fill for the embankment widening immediately behind the new abutments be placed in advance of the foundation installation for the new abutments, in order to preload the abutment widening area for a two-month period in advance of foundation construction. This is discussed further in Section 6.12.3.

6.5.3 Downdrag Load (Negative Skin Friction)

As described for driven H-piles, if the additional embankment widening fill is placed after construction of the foundations (i.e. no preloading), it will raise the effective stress level in the clayey soil below the widening area, producing settlement that could generate downdrag loads on the new drilled steel casings within 6 m of the north and south ends of the new abutments. These downdrag loads (i.e., negative skin friction) should be considered in design.

Assuming the underside of the pile cap is at Elevation 79.0 m, the unfactored downdrag load acting on a single 450 mm diameter drilled steel casing is estimated to be about 400 kN at the east abutment and about 700 kN at the west abutment. This downdrag load should be applied to the drilled steel casings located within 6 m of the north and south ends of the new abutments (i.e., within the zone of influence of the embankment widening); downdrag loads need not be applied to the drilled steel casings in the central portion of the abutment. The structural capacity of the steel casing must be checked for the factored dead and downdrag loads in accordance with Section 6.8.4 of the *Canadian Highway Bridge Design Code (CHBDC)*.



6.5.4 Resistance to Lateral Loads

The resistance to lateral loading developed by the soil in front of the drilled steel casings may be determined as outlined in Section 6.4.4. Where higher lateral resistances are required than can be achieved from the overburden soils alone, lateral resistance can be provided by the bedrock sockets.

The passive resistance of the portion of the foundation element socketted into bedrock has been analyzed using a Rock Mass Rating (RMR) profile for the bedrock, based on the RQD values and UCS values measured on the rock core recovered from the boreholes. The factored lateral geotechnical resistance at ULS per metre length of bedrock socket may be taken as follows:

Drilled Steel Casing Diameter (mm)	Factored Lateral Geotechnical Resistance at ULS (per metre length of bedrock socket)	
	Upper 0.5 m of Rock	Below 0.5 m Depth in Rock
450	11 MN	22 MN
600	15 MN	30 MN
750	18 MN	37 MN

The lateral load response of a single pile/drilled steel casing may be calculated using subgrade reaction theory and the coefficient of horizontal subgrade reaction, k_h (kPa/m) for the granitic gneiss bedrock. The rock mass will remain within the elastic range for the design loading, and therefore closed-form solutions are applicable for the estimation of the coefficient of horizontal subgrade reaction. Based on the rock mass modulus of the medium strong to very strong granitic bedrock (estimated to be approximately 15 GPa within the upper 0.5 m of the bedrock, and 30 GPa below this depth), and a Poisson's ratio of 0.2, the lateral rock mass spring constant may be taken as follows:

Rock Depth	Coefficient of Horizontal Subgrade Reaction, k_h
Upper 0.5 m of bedrock	25 MN/m per metre length of socket
Below 0.5 m depth in bedrock	50 MN/m per metre length of socket

6.6 Concrete Caissons

Support of the replacement bridge may be provided by drilled concrete caisson foundations extending to or into the medium strong to very strong granitic bedrock. At the piers in particular, the structure columns could be designed at each caisson to eliminate the need for caisson cap construction; this option would therefore minimize the need for cofferdams at the pier locations and the associated construction challenges and environmental impacts.

Temporary or permanent liners would be required for caisson construction through the overburden soils, to minimize ground loss and to provide groundwater control, particularly at the soil-bedrock interface (and particularly where non-cohesive soil layers are present above the bedrock).

While a large diameter caisson option is considered to be advantageous for environmental and structural reasons as noted above, there are challenges from a geotechnical perspective. It is understood that based on preliminary design calculations, the caissons will have to be socketted into the bedrock for approximately 1 m to



1.5 m in order to provide the required lateral resistance during seismic events. It is further understood that the pier columns will need to be approximately 1.2 m in diameter; the caisson diameter will need to be approximately 150 mm larger than the structural columns, i.e., approximately 1,350 mm in diameter. For this large diameter, the socket would be formed in the rock by drop hammer holes or cluster drilling.

The rig required for installation of large diameter caissons is relatively large and heavy, and appropriate access, working platform and overhead clearance will be required at the various construction stages.

6.6.1 Founding Elevations

It is recommended that the caissons be socketed a minimum of 0.2 m into the bedrock to allow for some weathering/fracturing of the upper portion of the bedrock, and to minimize the potential for loss of soils at the soil-bedrock interface during caisson construction. However, deeper sockets may be required to satisfy lateral/seismic loading requirements (as discussed further below). The table below summarizes the range in bedrock surface elevation as encountered in the boreholes at/near each of the foundation elements, from which the founding elevation and total length for each caisson can be assessed. It is noted that reference should also be made to the interpreted stratigraphic cross-sections provided for each of the foundation elements on Drawings 2 to 4, to assess the variability in the founding elevation and length of each caisson along the foundation unit.

Foundation Element	Borehole Numbers	Bedrock Surface Elevation (m)	Reference
West Abutment	BH 2, BH 3, 15-301, 15-305	69.8 – 73.2	See Drawing 3
West Pier	BH 5, 15-303, 15-304	69.7 – 74.1	See Drawing 2
East Pier	BH 6, BH 7, BH 12, PT 13, PT 15	68.1 – 73.4	See Drawing 2
East Abutment	BH 9, BH 8, 15-302, 15-306	67.4 – 69.2	See Drawing 4

For caissons that are socketed nominally into the bedrock (i.e., approximately 0.2 m to 0.5 m), tremie concrete should be placed to further seal the caisson/rock interface on the variable bedrock surface.

If caisson caps are to be included as part of the design, they should be constructed at a minimum depth of 1.5 m for frost protection purposes, per OPSD 3090.101 (*Foundation Frost Penetration Depths for Southern Ontario*).

6.6.2 Axial Geotechnical Resistance/Reaction

Caissons socketed approximately 0.2 m to 0.5 m into the bedrock should be designed based on end-bearing resistance, using a factored axial geotechnical resistance at ULS of 25 MPa; for a 1.35 m diameter caisson, this would equate to a factored axial geotechnical resistance at ULS of about 35,000 kN.

For caissons socketed 0.5 m or greater into the bedrock, the factored axial geotechnical resistance at ULS may be taken as 50 MPa; for a 1.35 m diameter caisson, this would equate to a factored axial geotechnical resistance at ULS of about 70,000 kN.



Serviceability Limit States (SLS) resistances for 25 mm of settlement will not apply to caissons founded within the medium strong to very strong granitic bedrock at this site, as the required loading would exceed the ULS resistance. It is estimated that less than 5 mm of deformation will occur under loads approaching the ULS resistance as provided above.

6.6.3 Resistance to Lateral Loads

The resistance to lateral loading developed by the soil in front of the caissons may be determined as outlined in Section 6.4.4.

Where higher lateral resistances are required than can be achieved from the overburden soils, it will be necessary to extend the caisson sockets into the bedrock. The passive resistance of the portion of the foundation element socketted into bedrock has been analyzed using a Rock Mass Rating (RMR) profile for the bedrock, based on the RQD values and UCS values measured on the rock core recovered from the boreholes. The factored lateral geotechnical resistance at ULS per metre length of bedrock socket may be taken as follows:

Drilled Steel Casing Diameter (mm)	Factored Lateral Geotechnical Resistance at ULS (per metre length of bedrock socket)	
	Upper 0.5 m of Rock	Below 0.5 m Depth in Rock
1,350	30 MN	60 MN

The lateral load response of a single caisson may be calculated using subgrade reaction theory and the coefficient of horizontal subgrade reaction, k_h (kPa/m) for the granitic gneiss bedrock. The rock mass will remain within the elastic range for the design loading, and therefore closed-form solutions are applicable for the estimation of the coefficient of horizontal subgrade reaction. Based on the rock mass modulus of the medium strong to very strong granitic bedrock (estimated to be approximately 15 GPa within the upper 0.5 m of the bedrock, and 30 GPa below this depth), and a Poisson's ratio of 0.2, the lateral rock mass spring constant may be taken as follows:

Rock Depth	Coefficient of Horizontal Subgrade Reaction, k_h
Upper 0.5 m of bedrock	25 MN/m per metre length of socket
Below 0.5 m depth in bedrock	50 MN/m per metre length of socket

6.7 Micropiles

Drilled and grouted micropiles may be considered for support of the new piers, in conjunction with a pile cap installed to support the above-grade structural pier columns. They could also be considered at the abutments, although access for construction equipment and headroom is not an issue at the abutments, and it is considered that driven steel piles or drilled steel casings will be more efficient and cost effective at the abutment locations.

Micropiles are small diameter (typically less than 300 mm), drilled and grouted non-displacement piles that are typically reinforced with steel casings and/or one or more steel thread bars. They can be designed to resist relatively high axial loads (in tension and compression) as well as moderate lateral loads. The type of micropile selected for a particular project will depend on the ground conditions and design loads, but installation generally includes advancing a casing (by drilling) through the overburden deposits and into the bearing stratum. Where



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high capacities are required, the drill casing is typically advanced into the upper portion of the bedrock, and then an uncased socket is drilled below the casing into the bedrock. Following placement of the central reinforcement, the micropile is filled with a neat cement grout by tremie methods. The casing may be partially or completely removed and additional grout may be injected into the micropile under pressure, depending on the structural stiffness and axial/lateral capacities required. If micropiles are adopted on this project, based on discussions to date with MMM and MTO regarding lateral/seismic loading, it is anticipated that the micropile casing would be advanced into the bedrock and left permanently in place at this site. The special drilling and grouting methods used in micropile installation allow for high grout-to-ground bond values along the grout/ground interface in the bearing stratum. The grout transfers the load through friction from the reinforcement to the ground in the micropile bond zone, in a manner similar to that of ground anchors. Due to the small pile diameter, any end-bearing contribution in micropiles is generally neglected.

The micropile design should be completed in accordance with the FHWA/NHI Micropile Design and Construction Reference Manual, Publication No. FHWA NHI-05-039 (FHWA/NHI 2005). The geotechnical aspects of micropile foundation design at this site have been preliminarily assessed based on FHWA/NHI (2005), the Canadian Foundation Engineering Manual (CFEM 2006), as well as the Recommendations for Prestressed Rock and Soil Anchors (PTI 2014).

If micropiles are selected for support of the new piers over the other foundation alternatives based on the preliminary recommendations provided in the following subsections, and consideration of the advantages, disadvantages, costs and risks, it is noted that additional detailed analysis and design will be required, including soil-structure interaction analysis to develop an efficient micropile group arrangement and to further assess the lateral loading response. A project-specific Non-Standard Special Provision (NSSP) for the micropile installation would also need to be developed; this will be prepared if this option is selected for construction.

6.7.1 Founding Elevations

Ultimately, the founding elevation for micropiles will depend on the required depth of rock socket to resist the axial loads and lateral/seismic loads. The following table summarizes the range in bedrock surface elevation as encountered in the boreholes at/near each of the foundation elements, from which the tip elevation and total length for each micropile can be assessed based on the required rock socket length. It is noted that reference should also be made to the interpreted stratigraphic cross-sections provided for each of the foundation elements on Drawings 2 to 4, to assess the variability in the tip elevation and length of each micropile along the foundation unit.

Foundation Element	Borehole Numbers	Bedrock Surface Elevation (m)	Reference
West Abutment	BH 2, BH 3, 15-301, 15-305	69.8 – 73.2	See Drawing 3
West Pier	BH 5, 15-303, 15-304	69.7 – 74.1	See Drawing 2
East Pier	BH 6, BH 7, BH 12, PT 13, PT 15	68.1 – 73.4	See Drawing 2
East Abutment	BH 9, BH 8, 15-302, 15-306	67.4 – 69.2	See Drawing 4



6.7.2 Axial Geotechnical Resistance/Reaction

The design capacity of a micropile is based on the bonded surface area of the cylinder of grout in contact with the ground in the bond zone. The total micropile length (i.e., embedment into the underlying bedrock) is determined by detailed pile design calculations. At this stage of design, it is anticipated that larger diameter micropiles, on the order of a 273 mm (10-3/4") diameter casing, will be required in order to meet the lateral/seismic loading requirements, and this casing diameter has been assumed for the initial recommendations provided herein. Such larger diameter micropiles have the advantage of requiring a shorter bond length to accommodate the axial design loads, and therefore would require less drilling into the bedrock. Larger diameter micropiles are also stiffer in section and provide higher lateral resistances.

The axial geotechnical resistance of the micropiles at this site will be developed within the bond zone of the socket in the granitic bedrock. The grout-to-ground bond strength to be used for design is estimated based on the results of uniaxial compressive strength (UCS) testing of the bedrock and these values are compared with typical values recommended in state-of-practice design manuals (such as FHWA/NHI (2005) and PTI (2014)). It is noted that where the strength of the bedrock is high (i.e. R4 or greater) such as is the case at this site, the strength of the grout can be the limiting factor in design and needs to be taken into consideration in the selection of the grout-to-ground bond value.

For preliminary design for a 273 mm casing diameter micropile socketted into the granitic bedrock, with a rock socket diameter on the order of 250 mm, a factored axial geotechnical resistance at ULS of 1,000 kN per metre length of bond zone could be achieved (assuming the bond zone is in good quality bedrock (RQD>75%) and a minimum grout strength of 30 MPa). For drilled micropiles founded in bedrock, the resistance required to achieve 25 mm of settlement is greater than that given for ULS and therefore SLS conditions do not apply.

6.7.3 Resistance to Lateral Loads

The passive resistance of the portion of the micropile socketed into bedrock has been analyzed using a Rock Mass Rating (RMR) profile for the bedrock, based on the RQD values and UCS values measured on the rock core recovered from the boreholes. The factored lateral geotechnical resistance at ULS per metre length of bedrock socket may be taken as follows:

Micropile Diameter (mm)	Factored Lateral Geotechnical Resistance at ULS (per metre length of bedrock socket)	
	Upper 0.5 m of Rock	Below 0.5 m Depth in Rock
273 (Casing) 250 (Socket)	6 MN	12 MN

The lateral load response of a single micropile may be calculated using subgrade reaction theory and the coefficient of horizontal subgrade reaction, k_h (kPa/m) for the granitic bedrock. The rock mass will remain within the elastic range for the design loading, and therefore closed-form solutions are applicable for the estimation of the



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coefficient of horizontal subgrade reaction. Based on the rock mass modulus of the medium strong to very strong granitic bedrock (estimated to be approximately 15 GPa within the upper 0.5 m of the bedrock, and 30 GPa below this depth), and a Poisson's ratio of 0.2, the lateral rock mass spring constant(s) may be taken as follows:

Rock Depth	Coefficient of Horizontal Subgrade Reaction, k_h
Upper 0.5 m of bedrock	25 MN/m per metre length of socket
Below 0.5 m depth in bedrock	50 MN/m per metre length of socket

It is recommended that the permanent casings for the micropiles be embedded a minimum of 1 m into the granitic bedrock in order to achieve a proper seal before drilling the uncased socket. In addition, and as noted above, if micropiles are selected for support of the new piers over other foundation types, then further detailed analysis and design will be required, including soil-structure interaction analysis of the proposed micropile group arrangement to further assess the minimum casing embedment requirements and the axial and lateral loading response of the micropile group.

6.8 Shallow Foundations

Spread footings may be considered for support of a portion of the west and east piers, where the depth to bedrock is relatively shallow. Based on the borehole results and the interpreted stratigraphic cross-sections at the piers as shown on Drawing 2, spread footings may be considered as follows:

- The northern portion of the west pier (i.e., the west pier for the westbound lane structure), and
- The southern three-quarters of the east pier (i.e., the east pier for the eastbound lane structure, and the southern portion of the east pier for the westbound lane structure).

Spread footings are not considered to be practicable for support of the remaining portions of the piers, where the depth to bedrock is on the order of 5 m to 7 m based on the borehole results. If spread footings are considered or adopted, the remaining portions of the piers would have to be supported using deep foundations extended to the bedrock, in conjunction with a pile cap.

Spread footings are also not considered practicable for support of the abutments, where significant excavations on the order of 11 m to 14 m below the existing Highway 401 grade would be required to reach the bedrock surface. The native silty clay soils are not considered suitable for support of spread footings for this long-span structure, due to the loading conditions and potential for differential settlement compared to foundation elements that are supported on bedrock.

6.8.1 Founding Elevations

The following table summarizes the recommended founding elevations for the applicable portions of the west and east piers. This would require excavation to a depth of approximately 2 m to 3 m below the floodplain/ground level, and below the water level in Cataraqui River. A cofferdam would be required for construction of spread footings, in order to facilitate excavation to the bedrock surface, and special consideration will be required to control the ground and water at the interface of the sheetpiling and the bedrock, which has a variable/sloping surface.



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Foundation Element	Borehole Number	Footing Founding Elevation (m)
West Pier – North Portion	15-303	73.5
	15-304	74.1
East Pier – South Portion	BH7	73.3
	BH12	73.2
	PT13	73.2
	PT15	71.6

Following excavation, the prepared bedrock surface should be inspected by a Quality Verification Engineer (QVE) in accordance with the requirements of OPSS 902 (Excavating and Backfilling Structures). The footings may either be cast directly against the variable bedrock surface, or a layer of mass concrete may be placed on the prepared bedrock surface following approval to create a level surface for forming and pouring of spread footings.

6.8.2 Geotechnical Resistance

Spread footings placed on the properly prepared granitic bedrock, or on mass concrete having a minimum 28-day compressive strength of 30 MPa, should be designed based on a factored geotechnical resistance at ULS of 10 MPa. The geotechnical reaction at SLS will not apply for footings founded on the medium strong to very strong granite at this site.

These geotechnical resistances are provided for loads applied perpendicular to the surface of the footings; where applicable, inclination of the load should be taken into account in accordance with Section 6.7.4 of the *Canadian Highway Bridge Design Code (CHBDC 2006)* and its *Commentary*.

6.8.3 Resistance to Lateral Loads

Resistance to lateral forces / sliding resistance between the concrete footings and granitic bedrock, or between the footings, mass concrete and granitic bedrock, should be calculated in accordance with Section 6.7.5 of the CHBDC. The coefficient of friction, $\tan \delta$, may be taken as follows:

- Cast-in-place footing to concrete working slab: $\tan \delta = 0.6$
- Cast-in-place footing or concrete working slab to bedrock: $\tan \delta = 0.6$

To supplement the sliding resistance and provide additional resistance to lateral/seismic forces, mechanical attachments such as dowels may be used to secure the footings to the bedrock; the dowels should be designed by the structural engineer. The resistance to lateral loads could also be increased by constructing a shear key at the bottom of the footing. The design of shear keys would require a specific analysis taking into consideration the magnitude of the horizontal loading, the magnitude of the vertical loading, and any variations in the bearing pressure due to overturning moments. This can be developed further if this option is selected for support of a portion of the new piers.

6.9 Seismic Considerations

The site is located in Kingston, Ontario and according to Table A.3.1.1 of the CHBDC, the zonal acceleration ratio, A , applicable to this site is 0.1. The corresponding acceleration-related seismic zone, Z_a , is 2.



6.10 Lateral Earth Pressures for Design

The lateral earth pressures acting on the abutment walls and any associated wing walls (if required) 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. Seismic (earthquake) loading must also be taken into account in the design.

The following recommendations are made concerning the design of the walls:

- Select free-draining granular fill meeting the specifications of OPSS Granular A or Granular B Type II but with less than 5 percent passing the 200 sieve should be used as backfill behind the abutment walls and wingwalls. This fill should be compacted in accordance with OPSS.PROV 501 (*Compacting*).
- Longitudinal drains and weep holes should be installed to provide positive drainage of the granular backfill. Other aspects of the granular backfill requirements with respect to sub-drains and frost tapers should be in accordance with OPSD 3101.150, 3190.100, and 3121.150.
- A minimum compaction surcharge of 12 kPa should be included in the lateral earth pressures for the structural design of the abutment walls and wingwalls, in accordance with CHBDC Section 6.9.3 and Figure 6.6. Care must be taken during the compaction operation not to overstress the wall. Heavy construction equipment should be maintained at a distance of at least 1 m away from the walls while the backfill soils are being placed. Hand-operated compaction equipment should be used to compact the backfill soils within a 1 m wide zone adjacent to the walls. Other surcharge loadings should be accounted for in the design, as required.
- The granular fill may be placed either in a zone with width equal to at least 1.5 m behind the back of the abutment stem (Case (a) on Figure C6.20 of the *Commentary to CHBDC*) or within the wedge-shaped zone defined by a line drawn at 1.5H:1V extending up and back from the rear face of the footing or pile cap (Case (b) on Figure C6.20 of the *Commentary to CHBDC*).

6.10.1 Static Lateral Earth Pressures for Design

The following guidelines and recommendations are provided regarding the lateral earth pressures for static (non-earthquake) loading conditions. These lateral earth pressures assume that the ground above the wall (i.e., the Highway 401 pavement and shoulder) will be flat, not sloping. If retaining walls are adopted at this site with a sloped portion above the top of the walls, then appropriate lateral earth pressures will need to be calculated to take account of the sloping ground.

- For Case (a), the pressures are based on the proposed embankment fill and the following parameters (unfactored) may be used assuming the use of earth fill or Select Subgrade Material (SSM):

Material	Earth Fill or SSM
Soil Unit Weight:	20 kN/m ³
Coefficients of static lateral earth pressure:	
Active, K_a	0.33
At rest, K_o	0.50
Passive, K_p	3.0



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- For Case (b), the pressures are based on using engineered granular fill and the following parameters (unfactored) may be used:

Material	Granular A	Granular B Type II
Soil Unit Weight:	22 kN/m ³	21 kN/m ³
Coefficients of static lateral earth pressure:		
Active, K_a	0.27	0.27
At rest, K_o	0.43	0.43
Passive, K_p	3.7	3.7

- If the wall support and superstructure allow lateral yielding, active earth pressures may be used in the geotechnical design of the structure. The movement to allow active pressures to develop within the backfill, and thereby assume an unrestrained structure, may be taken as follows:
 - Rotation of approximately 0.002 about the base of a vertical wall (where the rotation is calculated as the horizontal displacement divided by the height of the wall);
 - Horizontal translation of 0.001 times the height of the wall; or,
 - A combination of both.
- If the wall does not allow lateral yielding (i.e., restrained structure where the rotational or horizontal movement is not sufficient to mobilize an active earth pressure condition), at-rest earth pressures (plus any compaction surcharge) should be assumed for geotechnical design.
- Where movements are not sufficient to mobilize the full passive resistance, K_p may be determined in accordance with Figure C6.16 of the *Commentary* to CHBDC based on the amount of displacement.

6.10.2 Seismic Lateral Earth Pressures for Design

Seismic (earthquake) loading must be taken into account in the design in accordance with Section 4.6 of the CHBDC. In this regard, the following should be included in the assessment of lateral earth pressures:

- Seismic loading will result in increased lateral earth pressures acting on the wall. The wall should be designed to withstand the combined lateral loading for the appropriate static pressure conditions given in the previous Section, above, plus the earthquake-induced dynamic earth pressure. The site-specific zonal acceleration ratio (A) for the site is 0.1. The seismic lateral earth pressure coefficients given below have been derived based on a design zonal acceleration ratio of $A = 0.1$.
- In accordance with Sections 4.6.4 and C.4.6.4 of CHBDC and its *Commentary*, for structures which do not allow lateral yielding, the horizontal seismic coefficient (k_h) used in the calculation of the seismic active pressure coefficient is taken as 1.5 times the zonal acceleration ratio (i.e., $k_h = 0.15$). For structures which allow lateral yielding, (k_h) is taken as 0.5 times the zonal acceleration ratio (i.e., $k_h = 0.05$).
- The following seismic active pressure coefficients (K_{AE}) for the two backfill cases (Case (a) and Case (b)) may be used in the preliminary design. It should be noted that these seismic earth pressure coefficients assume that the back of the wall is vertical and the ground surface behind the wall is flat. Where sloping backfill is planned above the top of any associated retaining walls, the lateral earth pressures under seismic loading conditions should be calculated by treating the weight of the backfill located above the top of the wall as a surcharge.



Seismic Active Pressure Coefficients, K_{AE}

Material	Case (a)	Case (b)	
	SSM	Granular A	Granular B Type II
Yielding wall	0.37	0.30	0.30
Non-yielding wall	0.46	0.38	0.38

- The above K_{AE} values for yielding walls are applicable provided that the wall can move up to 250A mm, where A is the design zonal acceleration ratio of 0.1. This corresponds to displacements of up to approximately 25 mm at this site.
- The earthquake-induced dynamic pressure distribution, which is to be added to the static earth pressure distribution, is a linear distribution with maximum pressure at the top of the wall and minimum pressure at its toe (i.e., an inverted triangular pressure distribution). The total pressure distribution (static plus seismic) may be determined as follows:

$$\sigma_h(d) = K \gamma d + (K_{AE} - K) \gamma (H-d)$$

where:

- $\sigma_h(d)$ is the (static plus seismic) lateral earth pressure at depth, d, (kPa);
- K is the static active earth pressure coefficient, K_a (**to be used for yielding walls**);
- K is the static at-rest earth pressure coefficient, K_o (**to be used for non-yielding walls**);
- K_{AE} is the seismic active earth pressure coefficient;
- γ is the unit weight of the backfill soil (kN/m^3), as given previously;
- d is the depth below the top of the wall (m); and,
- H is the total height of the wall (m).

6.11 Approach Embankments

6.11.1 Subgrade Preparation and Embankment Construction

It is recommended that all topsoil/organic material or existing surficial fill present within the footprint of the widened Highway 401 approach embankments be stripped prior to placement of embankment fill. The widened approach embankment fill should be placed and compacted in accordance with OPSS.PROV 206 (*Earth Excavation and Grading*) and OPSS.PROV 501 (*Compacting*). Benching of the existing embankment side slopes should be carried out to “key in” the new fill materials to the existing fill materials, in accordance with OPSS 208.010 (*Benching of Earth Slopes*).

To reduce erosion of the widened embankment side slopes due to surface water runoff, placement of topsoil and seeding or pegged sod is recommended as soon as practicable after construction of the embankment widening, in accordance with OPSS 802 (*Topsoil*), 803 (*Sodding*) and/or OPSS 804 (*Seed and Cover*). Alternatively, the surface water run-off could be channelled from the curb via an armoured channel to control the discharge and minimize the potential for erosion.



6.11.2 Global Stability

Slope stability analyses have been performed for the proposed embankment widening using the commercially available slope stability analysis software package SlopeWTM Version 7.17 by GeoSlope International Ltd., to verify that a minimum factor of safety of 1.3 is achieved under static conditions and 1.1 under design seismic conditions. These minimum factors of safety are considered appropriate for the proposed bridge approach embankments, considering the design requirements and the available field and laboratory testing data.

The stability analyses were carried out considering that the widened portions of the embankment side slopes (widened up to about 5 m on each side) are constructed of either sand and gravel embankment fill, with side slopes oriented no steeper than 2H:1V, or rock fill, with side slopes oriented no steeper than 1.25H:1V. The soil stratigraphy used in the analyses was selected to represent soil conditions with the greatest thickness of overburden soil that may be expected at the site and was based on the information available from the many boreholes at the toes of the existing embankment from all phases of borehole investigation at the site. The analyses were carried out considering both short-term (undrained) and long-term (effective stress) conditions. The following parameters have been used in the analyses, based on field and laboratory test data as well as accepted correlations.

Soil Conditions	Bulk Unit Weight (kN/m ³)	Effective Friction Angle	Undrained Shear Strength (kPa)
Sand and Gravel (New Embankment Fill)	20	36°	-
Rock Fill (New Embankment Fill)	22	42°	-
Sand and Gravel (Existing Embankment Fill)	20	32°	-
Gravel (Existing West Embankment Fill)	21	32°	-
Silty Clay (Existing East Embankment Fill)	18	28°	75
Very Stiff Silty Clay	16	28°	100
Granitic Bedrock	23	-	-

Provided that the approach embankment side slopes are maintained no steeper than 2H:1V if constructed with sand and gravel embankment fill, and no steeper than 1.25H:1V if constructed with rock fill, and the existing embankment side slopes are benched in accordance with OPSD 208.010 (*Benching of Earth Slopes*), to “key in” any new fill materials placed on the slopes to accommodate the overall grade, the embankments should have an adequate minimum factor of safety of at least 1.3 under static conditions and 1.1 under design seismic conditions.

If steeper side slopes are to be constructed with rock fill to reduce the footprint at the toe of the embankments, the final embankment toe location may be near the existing embankment toe because the existing approach embankments have approximately 2H:1V side slopes. To maintain stability of the widened fill if rock fill is adopted, some subexcavation of the existing embankment side slopes will be required in order to maintain a minimum fill rock fill width equal to the width of widening at the top of the slope (i.e. minimum 5 m thickness of fill where the embankments are to be widened by 5 m).



6.11.3 Settlement

Some settlement of the existing embankments has likely occurred over time since the original bridge construction. Additional settlement under the widened portions of the approach embankments will occur following placement of the new embankment fill. Settlement analyses of the widened approach embankments were carried out using the material properties as given in the table below. The material properties were determined based on the available information, engineering judgement, and from experience with similar soils in this region of Ontario.

Soil Deposit	Bulk Unit Weight (kN/m ³)	Elastic Modulus (MPa)
Existing and New Embankment Fill	22	10 (for existing fill)
Stiff to Very Stiff Silty Clay	18	20
Compact to Very Dense Lower Sandy Silt to Sand and Gravel	21	100
Granitic Bedrock	Unyielding	

The settlement of the foundation soils under the proposed 5 m wide embankment widening is estimated to be a maximum of about 25 mm, decreasing to less than 10 mm near the new embankment toe. This settlement is expected to occur relatively quickly during and immediately following construction of the widened approach embankments based on the nature of the soils at the site.

The above estimates do not include compression of the new fill itself, which would occur during and after the construction of the embankment depending on the type of materials used. The magnitude of fill compression may range from 0.5 to 1 percent of the height of the embankment, assuming approximately 95 percent compaction of the embankment fill is achieved, relative to the material's standard Proctor maximum dry density. In the case where granular fill is used for embankment construction, settlement of the fill itself is expected to occur essentially during embankment construction, whereas non-granular earth fill materials are expected to exhibit some additional settlement over time.

6.12 Construction Considerations

The following sections identify future construction issues that should be considered during the design stage, and for which appropriate provisions should be made in the Contract Documents.

6.12.1 Excavation and Groundwater/Surface Water Control

Depending on the foundation type adopted, excavation for the pile caps or drilled steel casing caps at the abutments are expected to extend through the existing embankment fill (consisting of sand and gravel to gravel fill at the west abutment, and sand and gravel overlying silty clay to clay fill at the east abutment). The excavations are expected to extend up to about 3 m below the existing Highway 401 grade at the abutments. Depending on the foundation option adopted at the piers, excavations are expected to extend approximately 2 m to 3 m below the existing ground surface, and may require in-ground or floating cofferdams.



Where space permits, open-cut excavations at the abutments 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 embankment fill above the water table would be classified as Type 3 soil, based on the OHSA. According to OHSA excavations that extend to, or into, Type 3 soils should be made with side slopes no steeper than 1 horizontal to 1 vertical (1H:1V). Excavations in the embankment fill or native silty clay below the water table would be classified as Type 4 soil, based on OSHA, and excavations in these materials should be sloped no steeper than 3H:1V. However, with appropriate groundwater control, it is anticipated that temporary excavation slopes through these could also be maintained at 1H:1V.

While excavations for new abutment pile caps will be maintained above the groundwater level, any excavations for spread footings or for below-grade pile caps at the piers will extend below the groundwater level and the Cataraqui River level to reach bedrock. A cofferdam is recommended to minimize unwatering requirements and potential environmental impacts. A cut-off/cofferdam could consist of interlocking steel sheetpiles driven to the bedrock surface. The bedrock surface elevation is variable (refer to the interpreted stratigraphic cross-sections on Drawing 2), and the bedrock is medium strong to very strong, such that sheetpiles will not penetrate into the rock. If excavation to bedrock is required for a spread footing option, it is likely that “gaps” will exist at the base of some of the sheetpile sections at the bedrock interface. Measures would be required to control water seepage and prevent loss of soil through these gaps during excavation to the bedrock surface.

Alternatively, an above-grade or floating cofferdam approach could be adopted to create the pile caps for deep foundation elements at the west and east pier.

A Non-Standard Special Provision (NSSP) has been provided in Appendix D for inclusion in the Contract Documents, to address water control/cofferdams for the pier foundations at this site.

6.12.2 Temporary Protection Systems

If the above open-cut excavation side slopes at the abutments cannot be accommodated, then a temporary protection system will be required. At this stage, it is anticipated that temporary protection systems will be required parallel to the Highway 401 lanes in order to permit construction staging, as well as in the vicinity of the existing abutment to permit their removal or partial removal. Where required, the protection system should be designed and constructed in accordance with OPSS.PROV 539 (*Temporary Protection Systems*). The lateral movement of the temporary shoring system should meet Performance Level 2 as specified in OPSS.PROV 539.

The selection and design of the protection systems will be the responsibility of the Contractor. For conceptual planning and costing purposes, 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 the abutments. At the piers, an interlocking sheetpile system would contribute to both ground and groundwater/surface water control.

The sheet piles or soldier piles would have to be socketted to sufficient depth to provide the necessary passive resistance for a retained soil height of up to about 6 m at the abutments. Lateral support to the sheetpiles or soldier piles could be provided in the form of rakers, struts or temporary anchors.

6.12.3 Preloading Behind New Abutments

The Highway 401 approach embankments will be widened by approximately 5 m, requiring placement of a vertical thickness of up to approximately 2.5 m on the existing embankment side slopes adjacent to the new abutments. This widening will induce some elastic settlement in the stiff clayey soils underlying the site, which may induce some downdrag loading on the piles near the north and south ends of the new abutments (within



about 6 m of each end). Assuming that the construction schedule can accommodate it, this can be mitigated by preloading the embankment widening area for a period of approximately two months, prior to construction of the abutment foundations. It is recommended that an Operational Constraint (OC) be included in the Contract Documents to address this requirement; an OC is provided in Appendix D.

6.12.4 Vibration Monitoring During Deep Foundation Installation

The bridge replacement construction will be carried out in a staged manner to maintain traffic flow on Highway 401. It is recommended that vibration monitoring be carried out during any deep foundation installation (driving piles, drilled steel casings, drilled micropiles or drilled caissons) to assist in maintaining vibration levels within tolerable ranges for the existing and new portions of the bridge.

A maximum peak particle velocity of 100 mm per second is recommended at the existing piers and abutments.

An NSSP has been provided in Appendix D to address vibration monitoring, for inclusion in the Contract Documents.

6.12.5 Bedrock Excavation (Socket Formation)

The granitic bedrock at this site is medium strong to very strong. Excavation/socketting into the bedrock will be required to achieve the required lateral/seismic resistances at the piers, and bedrock sockets may also be required at the abutments to resist lateral/seismic loads (subject to detail design assessment by the structural engineers). It is recommended that an NSSP be included in the Contract Documents to warn the contractor of the bedrock strength, that excavation into the bedrock will require appropriate equipment and construction procedures, and that the bedrock excavation/socket formation shall not disturb the existing bridge footings. An NSSP is provided in Appendix D for inclusion in the Contract Documents.

6.12.6 Obstructions

The existing Highway 401 embankment fill as well as the native soil deposits may contain cobbles and boulders, which could affect the installation of deep foundations or protection systems. If driven pile foundations are adopted at the abutments, it is recommended that rock points be used to protect the piles during driving and seating on the medium strong to very strong granitic bedrock.

6.12.7 Erosion and Scour Protection

The near-surface soils in the vicinity of the piers and at the river's edge are expected to be susceptible to erosion and scour under the design flood/flow velocities. The requirements for design of erosion/scour protection should be assessed by the hydraulic design engineer. As a minimum, it is recommended that erosion protection (e.g., rip-rap or granular sheeting) be provided on the river banks and adjacent to the piers to protect the foundations/pile caps from being exposed. The rip-rap should be consistent with the standard R-10 classification or granular sheeting classification in accordance with OPSS 1004 (Aggregates) but should be approved by the hydraulic design engineer.



FOUNDATION INVESTIGATION AND DESIGN REPORT CATARAQUI RIVER BRIDGE, HIGHWAY 401, KINGSTON, ONTARIO

7.0 CLOSURE

This Foundation Design Report was prepared by Mr. Matt Kennedy, P.Eng., and reviewed by Ms. Lisa Coyne, P.Eng., a geotechnical engineer and Principal with Golder, with technical input from Mr. Murty Devata, P.Eng. Mr. Fin Heffernan, P.Eng., the Designated MTO Foundations Contact for this assignment, conducted an independent quality review of this report.

GOLDER ASSOCIATES LTD



Matt Kennedy, P.Eng.
Geotechnical Engineer



Lisa Coyne, P.Eng.
Senior Geotechnical Engineer, Principal



Fin Heffernan, P.Eng.
Designated MTO Foundations Contact



WAM/MJK/LCC/FJH/ob

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Table 1 – Comparison of Foundation Alternatives

Foundation Option	Feasibility		Advantages	Disadvantages	Constructability/Risks	Approximate Costs
	Abutments	Piers				
Driven steel piles (H-piles or pipe piles) driven to refusal on the bedrock, or installed in pre-cored socket holes in bedrock	<ul style="list-style-type: none">Feasible, although less practical than other options if a rock socket is required for pile toe fixity at the abutments	<ul style="list-style-type: none">Not practical for portions of piers where depth to bedrock is shallowFeasible where bedrock is deeper; however, would require coring to achieve rock sockets	<ul style="list-style-type: none">Conventional construction technique for abutment locations, with reasonable access for pile driving rig during construction staging; it is understood that access roadway will be constructed to pier locations and at this stage it is anticipated that the rig could work from the shore for construction of pier foundation elementsAllows pile cap to be maintained relatively high, minimizing excavation; negligible dewatering at abutmentsWould permit use of integral abutments, if other structural requirements are also satisfied, improving long-term maintenance costsExisting embankment fill penetrable by augers at proposed abutment locations, suggesting low risk of encountering rock fill obstructions during pile drivingPile tips can be reinforced with Titus injector rock points or equivalent to minimize damage during pile driving and aid in seating pile onto the sloping granitic bedrock	<ul style="list-style-type: none">Driven piles cannot penetrate medium strong to very strong granitic bedrock; if a rock socket is required for pile toe fixity, a separate rig would be required with a temporary liner through the overburden during coring/churn drilling into the bedrock to form the socket; such sockets would typically be on the order of 600 mm in diameterDepth to bedrock is shallow (less than 3 m) over more than half of each pier, making this option impractical within those areasAt piers, this option may require construction of a pier pile cap in the water, with excavation inside a cofferdam with moderate risk of encountering rock fill/obstructions near the ground surface, and challenges associated with driving to sloping bedrock; however, may also consider a pre-fabricated, “floating” cofferdam if acceptable from an environmental and structural perspective or by forming pile cap above the water level, as this would mitigate some of the disadvantages associated with use of pile caps	<ul style="list-style-type: none">Low risk of encountering obstructions during pile driving at abutmentsSome risk of difficulty in seating driven piles on sloping bedrock; Titus injector rock points and careful driving onto bedrock would aid in controlling this risk at the abutments, based on interpreted bedrock surface from the borehole resultsIf pile toe fixity is required at the abutments, and for all piles if adopted at the piers, constructability of rock sockets into granitic bedrock must be considered, using separate equipment/ operation – however, for anticipated rock socket diameter of less than 600 mm, such constructability is reasonableEnvironmental protection considerations for construction of in-water pile cap; moderate risk of encountering rock fill/obstructions in driving sheetpiling at piers, although this may be mitigated by pre-excavating; shallow depth to bedrock over portion of the piers, and variable bedrock surface, may pose some difficulty in sheetpile installation and achieving cut-off; these issues may be partially mitigated if a pre-fabricated floating cofferdam is constructable within the water depth and permitted from an environmental and aesthetic perspective at the piers or by forming pile cap above the water level	<ul style="list-style-type: none">H-Piles driven to bedrock at \$250 per metre length.This cost excludes formation of a bedrock socket via separate operation prior to placement of the H-pile in the socketThis cost excludes mobilization, cofferdam and pile cap, construction access road, and working platform at river’s edge



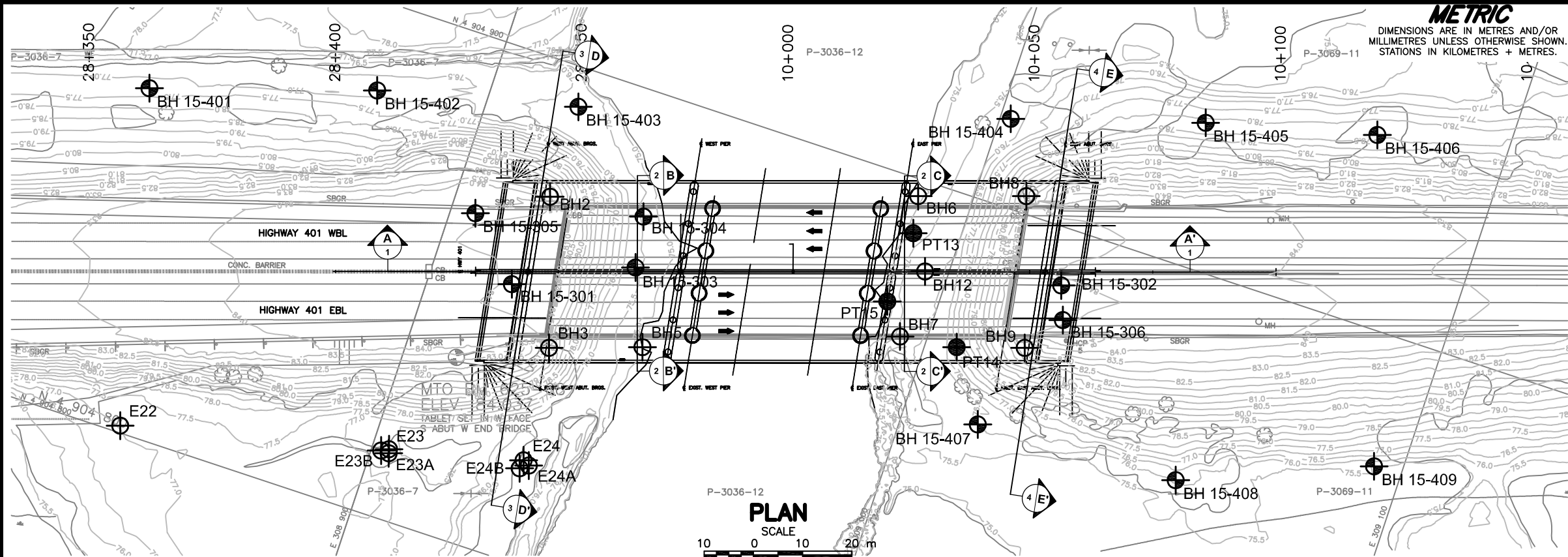
Foundation Option	Feasibility		Advantages	Disadvantages	Constructability/Risks	Approximate Costs
	Abutments	Piers				
Drilled steel casings socketted into bedrock	<ul style="list-style-type: none">Feasible in diameters ranging from about 450 mm to 750 mm; preferred at the abutments if rock sockets are required for pile toe fixity	<ul style="list-style-type: none">Feasible in diameters ranging from about 450 mm to 750 mm; a cost effective solution where rock sockets are required	<ul style="list-style-type: none">Reasonably conventional construction technique for abutment and pier locations; access at abutments is good throughout construction staging; access to pier locations for heavy rig reasonable via rock fill access roadway; special working platform/pad expected to be requiredAllows pile cap to be maintained relatively high, minimizing excavation and dewatering at abutmentsProven effective where rock sockets are required, whether at the abutments or piers; this relatively smaller option will be more constructable and cost effective than larger diameter caissons; better able to handle abrasive rock conditions and sloping bedrock conditions than driven piles or larger diameter caissonsCleaning of base of rock socket not as critical because axial capacity developed based on shaft resistance in socketAlthough significant obstructions were not encountered in boreholes advanced through the existing embankment fill and the risk of such encounter is considered low, drilled steel casings will readily handle obstructions if present within the proposed abutment areas	<ul style="list-style-type: none">Precludes use of integral abutmentsAt piers, would require construction of a pier pile cap in the water, with excavation inside a cofferdam with moderate risk of encountering rock fill/obstructions near the ground surface, and challenges associated with driving sheetpiles to sloping bedrock; however, may also consider a pre-fabricated, “floating” cofferdam if acceptable from an environmental and structural perspective or forming pile cap above the water level, as this would mitigate some of the disadvantages associated with use of pile caps	<ul style="list-style-type: none">Drilled steel casings with diameters of 450 mm to 600 mm are considered to be the most readily constructable of rock socket options (similar to micropiles)Environmental protection considerations for construction of in-water pile cap; moderate risk of encountering rock fill/obstructions in driving sheetpiling at piers, although this may be mitigated by pre-excavating; shallow depth to bedrock over a portion of the piers, and variable bedrock surface, may pose some difficulty in sheetpile installation and achieving a cut-off; these issues may be partially mitigated if a pre-fabricated, “floating” cofferdam is constructable within the water depth and permitted from an environmental and aesthetic perspective at the piers or by forming pile cap above the water level	<ul style="list-style-type: none">750 mm diameter drilled steel casings on the order of \$8,000 per element for average pier depth, socketted 1.5 m into bedrockThe above cost excludes mobilization, cofferdam and pile cap, construction access road, and working platform at river’s edge
Spread footings founded on the bedrock surface	<ul style="list-style-type: none">Not feasible at abutments due to depth of excavation to reach rock; abutment footings founded at higher level in native soils or perched in embankment fill are not recommended due to potential for differential settlement relative to bedrock-supported piers	<ul style="list-style-type: none">Feasible for support of portions of the piers where bedrock is relatively shallow; would require combined shallow/ deep foundation approach	<ul style="list-style-type: none">Reasonably conventional in-water construction for northern half of West Pier, and southern three-quarters of East PierSpread footing construction at piers could potentially be completed underneath existing bridge deckLimits significant excavation into bedrock (as would be the case for a large diameter caisson option)	<ul style="list-style-type: none">Requires design of combined shallow/deep foundation element at both west and east piers, and spread footings supported on bedrock are not considered feasible where bedrock is deeper (on the order of 5 m to 7 m depth) at south end of West Pier, and north end of East PierConstruction of spread footings directly on the bedrock surface will require a cofferdam in the river, with associated environmental concerns and constructability challenges: moderate risk of encountering rock fill/obstructions near surface, challenges associated with driving sheetpiles to sloping bedrock, and challenges in “sealing” the soil-bedrock interfaceWould likely require dowels in order to resist lateral/seismic loading	<ul style="list-style-type: none">Environmental protection considerations for in-water work associated with excavation for spread footingCofferdam required; moderate risk of encountering rock fill/obstructions in driving sheetpiling at piers although this can likely be mitigated by pre-excavation of near-surface zone; shallow depth to bedrock over applicable portion of the piers, and variable bedrock surface, may pose some difficulty in sheetpile installation and achieving a cut-off at the soil-bedrock interfaceAdditional bedrock excavation or placement of mass concrete may be required to create more level bearing surface for spread footings	<ul style="list-style-type: none">Concrete for footings at \$600 per cubic metre.This cost excludes mobilization, cofferdam and pile cap, construction access road, working platform at river’s edge, and installation and testing associated with dowels (if required to supplement lateral resistance)



Foundation Option	Feasibility		Advantages	Disadvantages	Constructability/Risks	Approximate Costs
	Abutments	Piers				
Large diameter caissons socketted into bedrock	<ul style="list-style-type: none">This foundation type is not required for support at the abutments given the depth to bedrock and other feasible options available	<ul style="list-style-type: none">Technically feasible in diameters ranging from about 0.9 m to 1.35 m or larger; however, significant risks and cost premiums; not recommended from a foundations perspective	<ul style="list-style-type: none">Key advantage at the piers in that a single caisson could support each pier column, eliminating the need for a pile cap and cofferdam at the piers, and thereby minimizing environmental impacts in the river and associated constructability challenges	<ul style="list-style-type: none">For anticipated 1.2 m diameter structural columns, 1.35 m diameter caissons would be required, and under this option the structural engineers have estimated that a rock socket on the order of 1.5 m deep would be required; such a large diameter rock socket will have significant construction challenges in the medium strong to very strong granitic bedrock, which is very abrasive; contractor feedback regarding the constructability of this option has been negative	<ul style="list-style-type: none">Lower environmental and constructability risk for to in-water work because pile cap/cofferdam can be eliminated; liners would be required through water and overburden soils to support caisson sides during excavation and constructionSignificant constructability issues associated with forming deep socket in bedrock at this large caisson diameter; negative feedback from three independent contractor sourcesDifficulties associated with seating large diameter caissons into sloping bedrock surface at the piers; more challenging than smaller diameter drilled steel casings or micropiles	<ul style="list-style-type: none">Construction of approximately 1.5 m deep, 1.35 m diameter rock socket will be time-consuming and expensiveApproximately \$85,000 per caisson, assuming 1.35 m diameter caisson socketted approximately 1 m into bedrockThe above cost excludes mobilization, cofferdam and pile cap, construction access road, and working platform at river's edge
Large diameter caissons socketted nominally into bedrock, with dowels/micropiles at base of caisson	<ul style="list-style-type: none">This foundation type is not required for support at the abutments given the depth to bedrock and other feasible options available	<ul style="list-style-type: none">Technically feasible in diameters ranging from about 0.9 m to 1.35 m or larger; constructability challenges are slightly improved over large diameter caissons with deeper rock sockets; however, further structural evaluation required for seismic loading cases	<ul style="list-style-type: none">As for caissons socketted deeper into the bedrock, the key advantage at the piers is that a single caisson could support each pier column, eliminating the need for a pile cap and cofferdam at the piers, and thereby minimizing environmental impacts in the river and associated constructability challengesUnder this option, caissons would be socketted nominally (between about 0.2 m and 0.5 m based on variability in bedrock surface) into the bedrock; drop hammer holes or cluster drilling would still be required to form the nominal socket, and this would still be time-consuming and expensive, but would be less onerous than for deeper socketSmaller diameter micropiles/dowels at the base of the caisson are more constructable than large diameter rock sockets in the medium strong to very strong bedrock; however, this approach must be assessed from a structural perspective	<ul style="list-style-type: none">Even with a shallower (0.2 m to 0.5 m nominal depth) bedrock socket, the use of drop hammer and cluster drilling techniques is anticipated in the granitic bedrock, and this work will be time-consuming and expensiveWill require careful drilling operations as it may be difficult to seat and seal liner/casing before extending large diameter sockets nominally into sloping granitic bedrockRequires separate mobilization of rigs for selected foundation type at abutments, rig capable of installing large diameter caissons at piers, and specialist contractor with micropiling capabilitiesThis approach may not satisfy lateral/seismic loading requirements, subject to further analysis by the structural engineer during detail design	<ul style="list-style-type: none">Lower environmental and constructability risk related to in-water work because pile cap/cofferdam can be eliminatedLesser constructability issues for nominal socket in granitic bedrock, compared to deep socket in bedrock; however, this will still be a time-consuming and expensive operationDifficulties associated with seating large diameter caissons into sloping bedrock surface at the piers; more challenging than smaller diameter drilled steel casings or micropiles	<ul style="list-style-type: none">Approximately \$50,000 per caisson, assuming 1.35 metre diameter caisson socketed approximately 0.3 metre into bedrock.This cost excludes mobilization, cofferdam and pile cap, construction access road, and working platform at river's edge and installation and testing associated with dowels (if required to supplement lateral resistance)



Foundation Option	Feasibility		Advantages	Disadvantages	Constructability/Risks	Approximate Costs
	Abutments	Piers				
Micropiles	<ul style="list-style-type: none">Feasible at abutments, but would be more expensive than driven steel piles or drilled steel casings (if a rock socket is required); therefore not preferred from foundations perspective	<ul style="list-style-type: none">Feasible at piers where rock sockets are required; similar to drilled steel casings but smaller in diameter, so more elements will be required	<ul style="list-style-type: none">Smaller equipment footprint when compared to the rigs required for any other deep foundation element, representing an advantage for access to and temporary working platform construction at the river's edge; some micropile rigs can work in as little as 7 m of headroom (using shorter micropile sections), which may permit working under the existing bridge deck during some stages of constructionRequires smaller diameter socketting (on the order of 275 mm); smaller sockets are more constructable in the granitic bedrock at this site, as compared with larger diameter elements; smaller sockets also better able to handle sloping bedrock conditions than driven piles or larger diameter caissons	<ul style="list-style-type: none">Specialized detail foundation design required to achieve lateral resistance with smaller diameter foundation elementsMay require specialist contractorAlthough smaller diameter is more constructable than larger diameter rock sockets (eg., drilled steel casings), more micropile elements would be required in designWould require construction of a pier pile cap in the water, with excavation inside a cofferdam with moderate risk of encountering rock fill/obstructions, and challenges associated with driving sheetpiles to sloping bedrock; however, may also consider a pre-fabricated, "floating" cofferdam if acceptable from an environmental and structural perspective or by forming pile cap above the water levelSufficient steel would be required in structural design of micropile to meet lateral/seismic loading requirements	<ul style="list-style-type: none">Micropiles (approximately 275 mm diameter) are considered the most readily constructable of rock socket options (similar to smaller diameter drilled steel casings)Environmental protection considerations for construction of in-water pile cap; moderate risk of encountering rock fill/obstructions in driving sheetpiling at piers; shallow depth to bedrock over a portion of the piers, and variable bedrock surface, may pose some difficulty in sheetpile installation and achieving a cut-off; these issues may be partially mitigated if a pre-fabricated, "floating" cofferdam is constructable within the water depth and permitted from an environmental and aesthetic perspective or by forming pile cap above the water level	<ul style="list-style-type: none">Micropiles drilled through overburden and socketed into granitic bedrock at approximately \$1000 per metre length.Additional costs required for detail design and load testing during constructionThis cost excludes mobilization, cofferdam and pile cap, construction access road, and working platform at river's edge



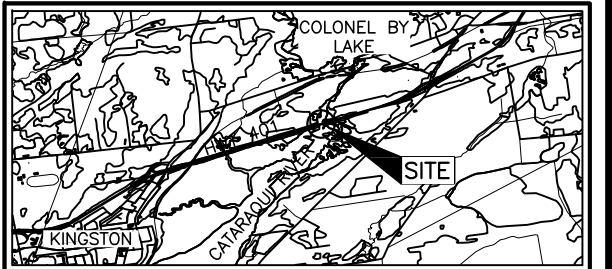
CONT No.
WP No. 79-99-00

SHEET

CATARAQUI RIVER BRIDGE
HIGHWAY 401
BOREHOLE LOCATIONS AND SOIL STRATA



Golder Associates Ltd.
 OTTAWA ONTARIO, CANADA



LEGEND

Borehole - Current Investigation

Borehole - Previous Investigation

Penetration Test - Previous Investigation

N
 16
 100%

 Standard Penetration Test Value
 Blows/0.3m unless otherwise stated
 (Std. Pen. Test, 475 j/blow)

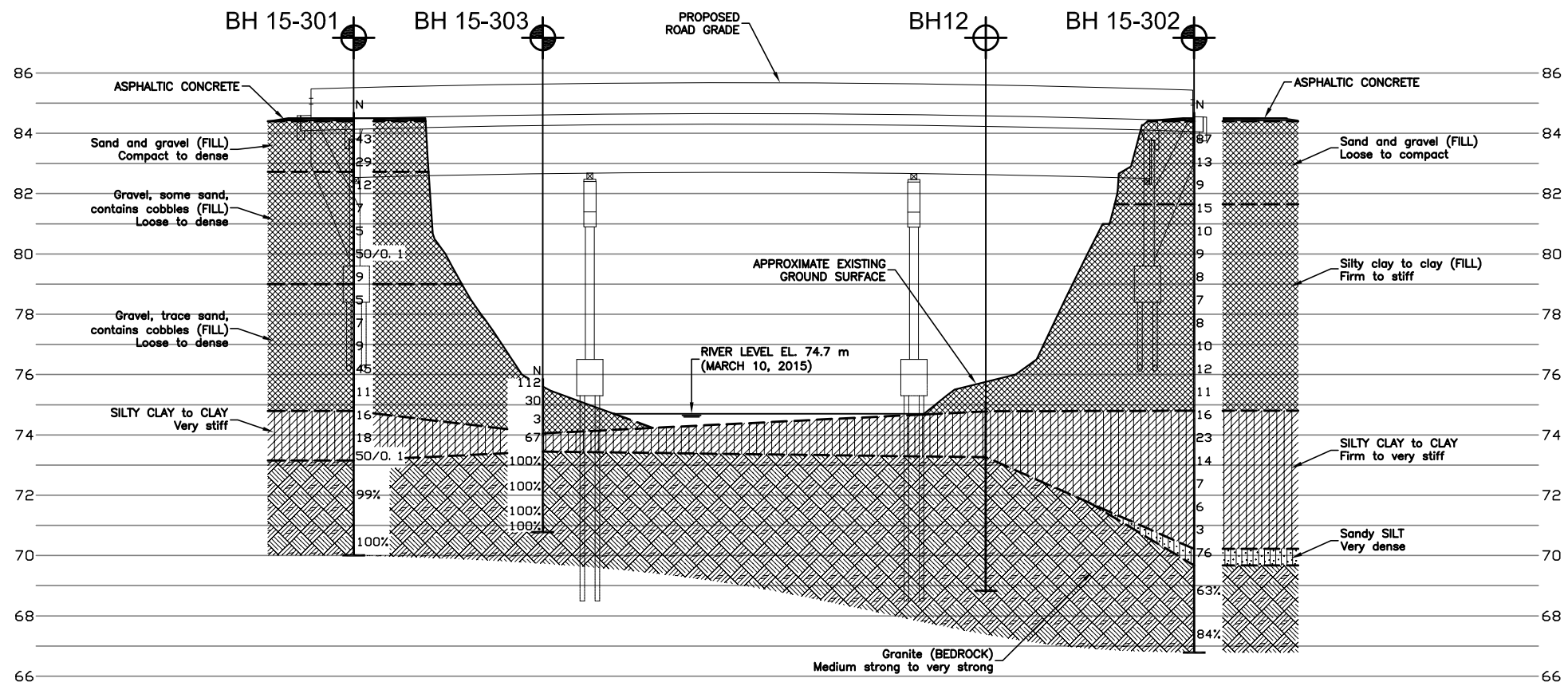
WL in open borehole

WL in piezometer

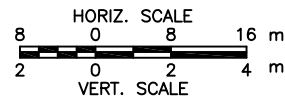
Total Core Recovery (REC)

Seal

Piezometer



PROFILE A-A' - ALONG HIGHWAY 401



REFERENCE
 Base plans provided in digital format by MMM Group Limited, drawing file no. 3414034-XB1.dwg, received April 7, 2015.

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 Contract Documents.

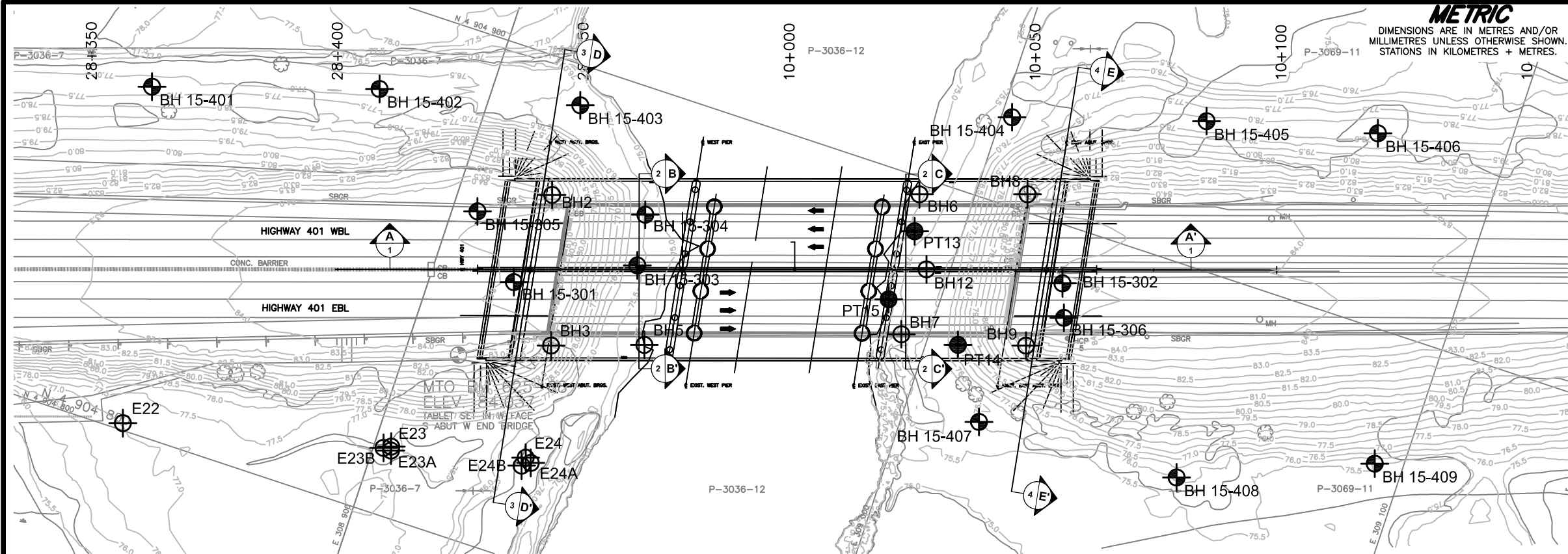
The boundaries between soil strata have been established only at borehole locations. Between boreholes the boundaries are assumed from geological evidence.

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.

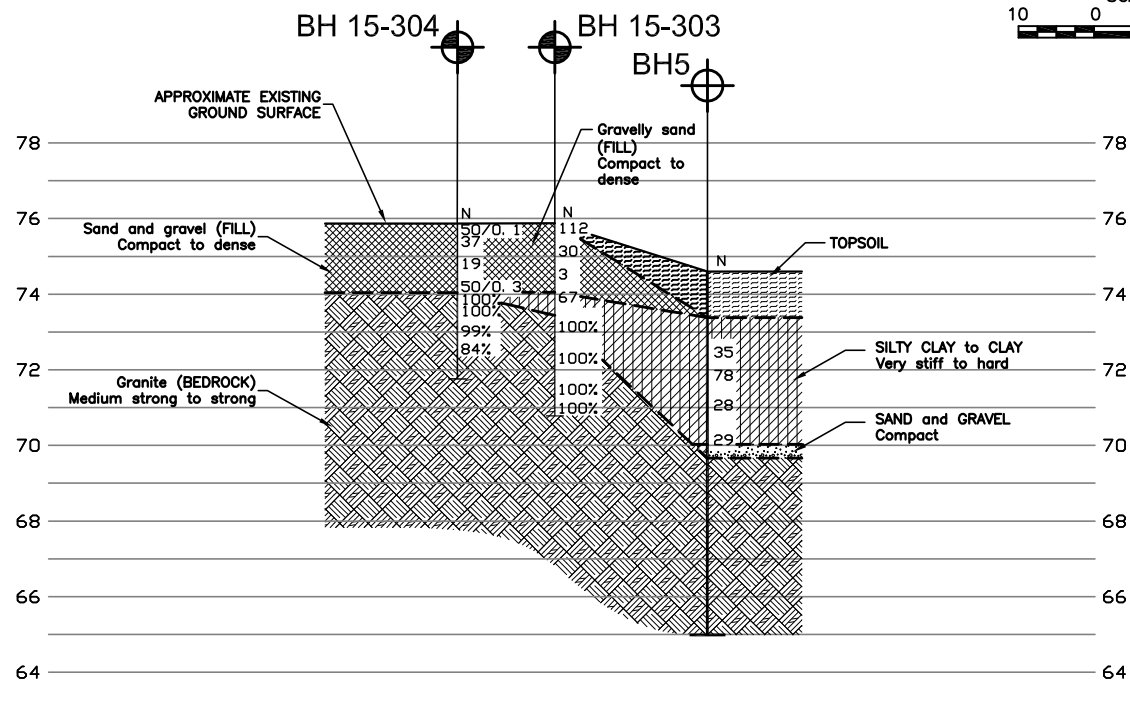
BOREHOLE CO-ORDINATES			
No.	ELEVATION	NORTHING	EASTING
15-301	84.7	4904853.7	308916.6
15-302	84.7	4904888.6	309022.5
15-303	75.9	4904864.8	308939.3
15-304	75.9	4904875.1	308937.6
15-305	84.5	4904865.0	308905.0
15-306	84.5	4904882.0	309025.0
15-401	77.3	4904868.2	308834.2
15-402	76.8	4904882.4	308878.2
15-403	75.7	4904892.1	308918.0
15-404	75.9	4904917.4	309002.1
15-405	78.1	4904929.1	309039.9
15-406	78.3	4904937.7	309073.8
15-407	76.0	4904856.5	309015.3
15-408	76.0	4904858.4	309057.0
15-409	75.6	4904873.8	309094.3
BH2	76.1	4904873.0	308918.3
BH3	75.1	4904843.8	308927.8
BH5	74.6	4904849.9	308945.7
BH6	75.1	4904896.6	308989.2
BH7	75.0	4904868.4	308994.7
BH8	75.6	4904903.5	309010.0
BH9	75.9	4904874.2	309019.4
BH12	75.1	4904882.6	308995.3
PT13	75.1	4904882.6	308995.3
PT15	75.0	4904874.4	308990.0
E-22	77.1	4904801.4	308850.1
E-23	76.6	4904814.1	308903.4
E24	75.1	4904820.5	308930.1



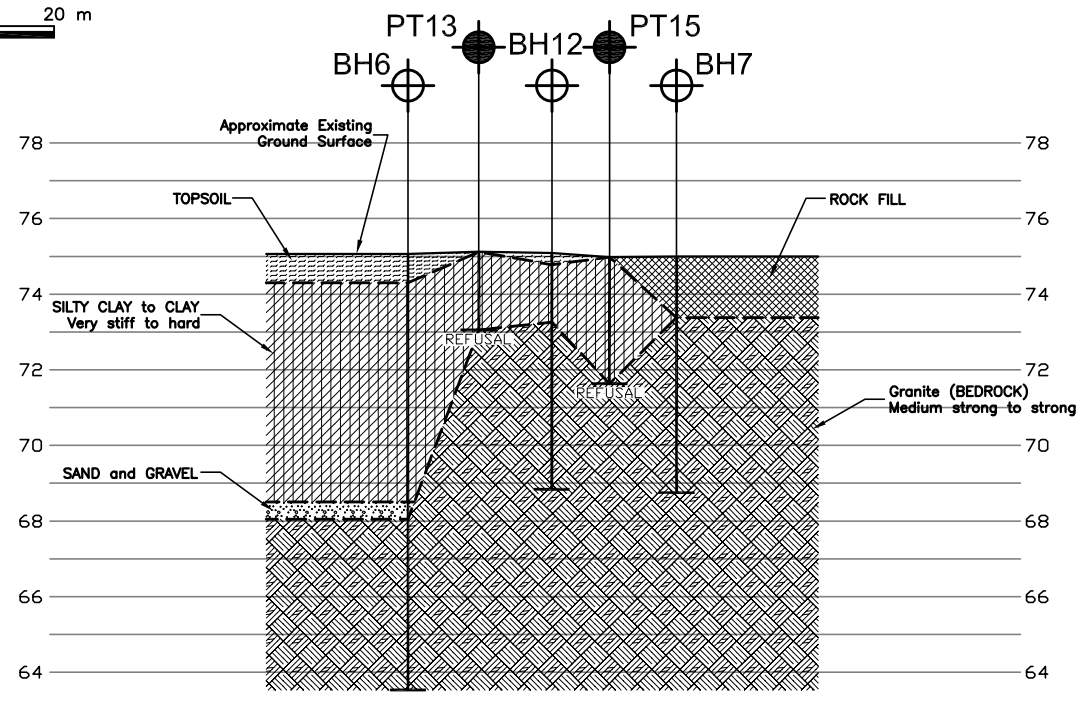
NO.	DATE	BY	REVISION
Geocres No. 31C-233			
HWY. 401		PROJECT NO. 1403255-001	DIST. Eastern
SUBM'D. WAM	CHKD. WAM	DATE: 5/21/2015	SITE: 7-70
DRAWN: JM	CHKD. MJK	APPD. FJH	DWG. 1



PLAN
SCALE
10 0 10 20 m



SECTION B-B' - WEST PIER
HORIZ. SCALE
8 0 8 16 m
VERT. SCALE
2 0 2 4 m



SECTION C-C' - EAST PIER
HORIZ. SCALE
8 0 8 16 m
VERT. SCALE
2 0 2 4 m

NOTES

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REFERENCE

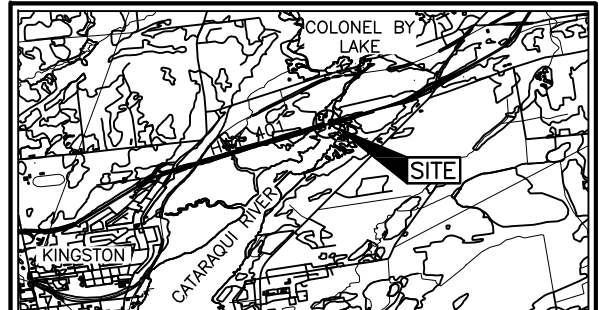
Base plans provided in digital format by MMM Group Limited, drawing file no. 3414034-XB1.dwg, received April 7, 2015.

CONT No.
WP No. 79-99-00

CATARAQUI RIVER BRIDGE
HIGHWAY 401
BOREHOLE LOCATIONS AND SOIL STRATA



Golder Associates Ltd.
OTTAWA ONTARIO, CANADA



KEY PLAN
SCALE
1 0 2 km

LEGEND

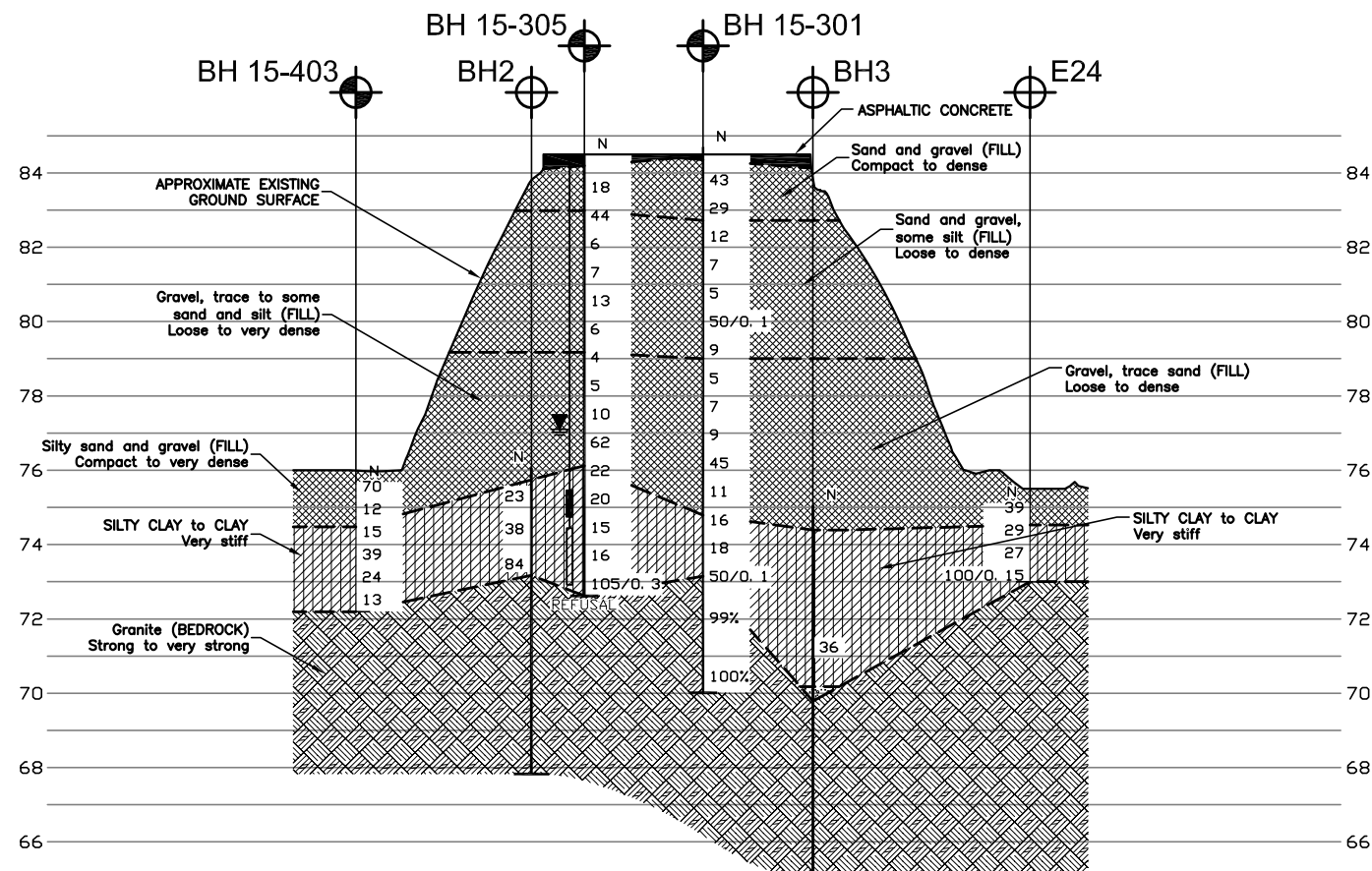
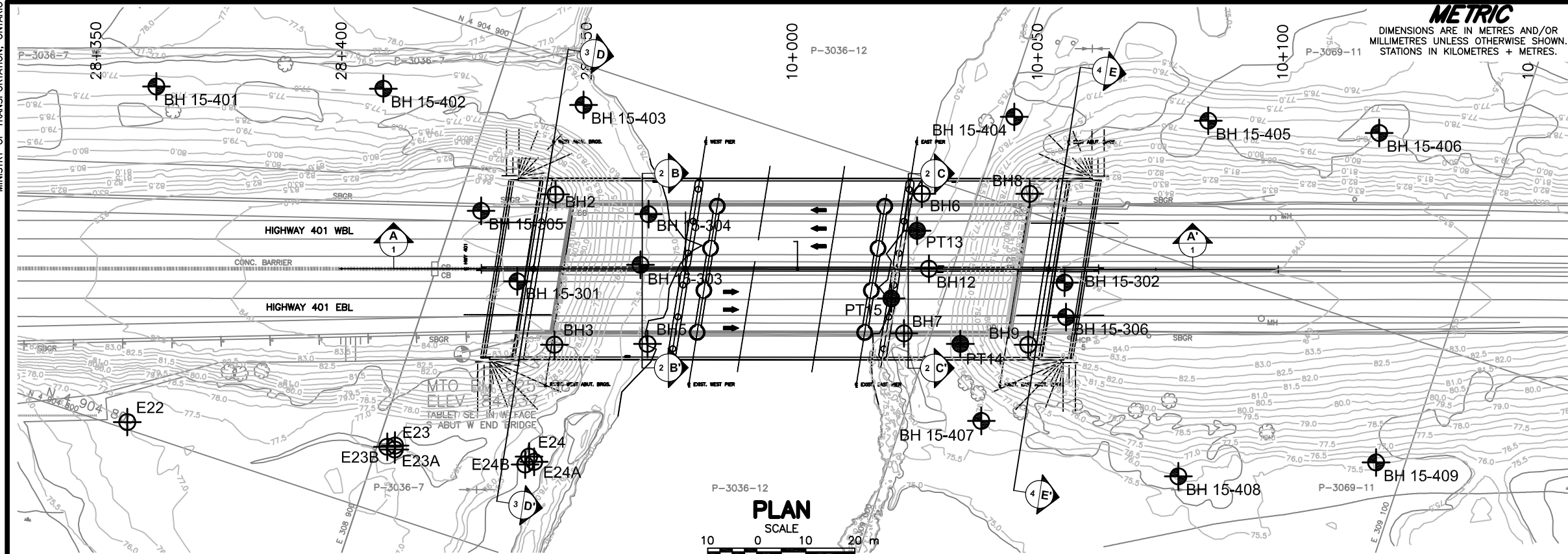
- Borehole - Current Investigation
- Borehole - Previous Investigation
- Penetration Test - Previous Investigation
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- 100% Total Core Recovery (REC)

BOREHOLE CO-ORDINATES

No.	ELEVATION	NORTHING	EASTING
15-301	84.7	4904853.7	308916.6
15-302	84.7	4904888.6	309022.5
15-303	75.9	4904864.8	308939.3
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15-401	77.3	4904868.2	308834.2
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15-403	75.7	4904892.1	308918.0
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15-408	76.0	4904858.4	309057.0
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BH3	75.1	4904843.8	308927.8
BH5	74.6	4904849.9	308945.7
BH6	75.1	4904896.6	308989.2
BH7	75.0	4904868.4	308994.7
BH8	75.6	4904903.5	309010.0
BH9	75.9	4904874.2	309019.4
BH12	75.1	4904882.6	308995.3
PT13	75.1	4904882.6	308995.3
PT15	75.0	4904874.4	308990.0
E-22	77.1	4904801.4	308850.1
E-23	76.6	4904814.1	308903.4
E24	75.1	4904820.5	308930.1



NO.	DATE	BY	REVISION
Geocres No. 31C-233			
HWY. 401		PROJECT NO. 1403255-001	DIST. Eastern
SUBM'D. WAM	CHKD. WAM	DATE: 5/21/2015	SITE: 7-70
DRAWN: JM	CHKD. MJK	APPD. FJH	DWG. 2



D-D' SECTION D-D' - WEST ABUTMENT

HORIZ. SCALE
0 8 16 m
2 0 2 4 m
VERT. SCALE

REFERENCE

Base plans provided in digital format by MMM Group Limited, drawing file no. 3414034-XB1.dwg, received April 7, 2015.

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 Contract Documents.

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

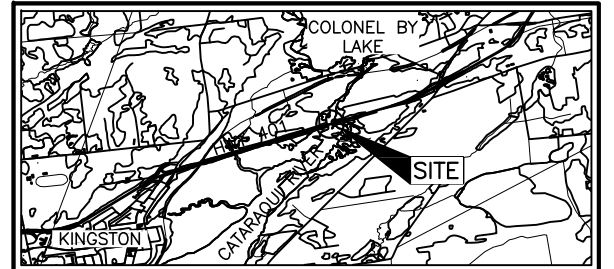
The complete Foundation Investigation and Design Report for this project and other related documents may be examined at the Materials Engineering and Research Office, Downsview. Information contained in this report and related documents is specifically excluded in accordance with Section GC 2.01 of OPS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

CONT No.
WP No. 79-99-00

CATARAQUI RIVER BRIDGE
HIGHWAY 401
BOREHOLE LOCATIONS AND SOIL STRATA



Golder Associates Ltd.
OTTAWA, ONTARIO, CANADA



LEGEND

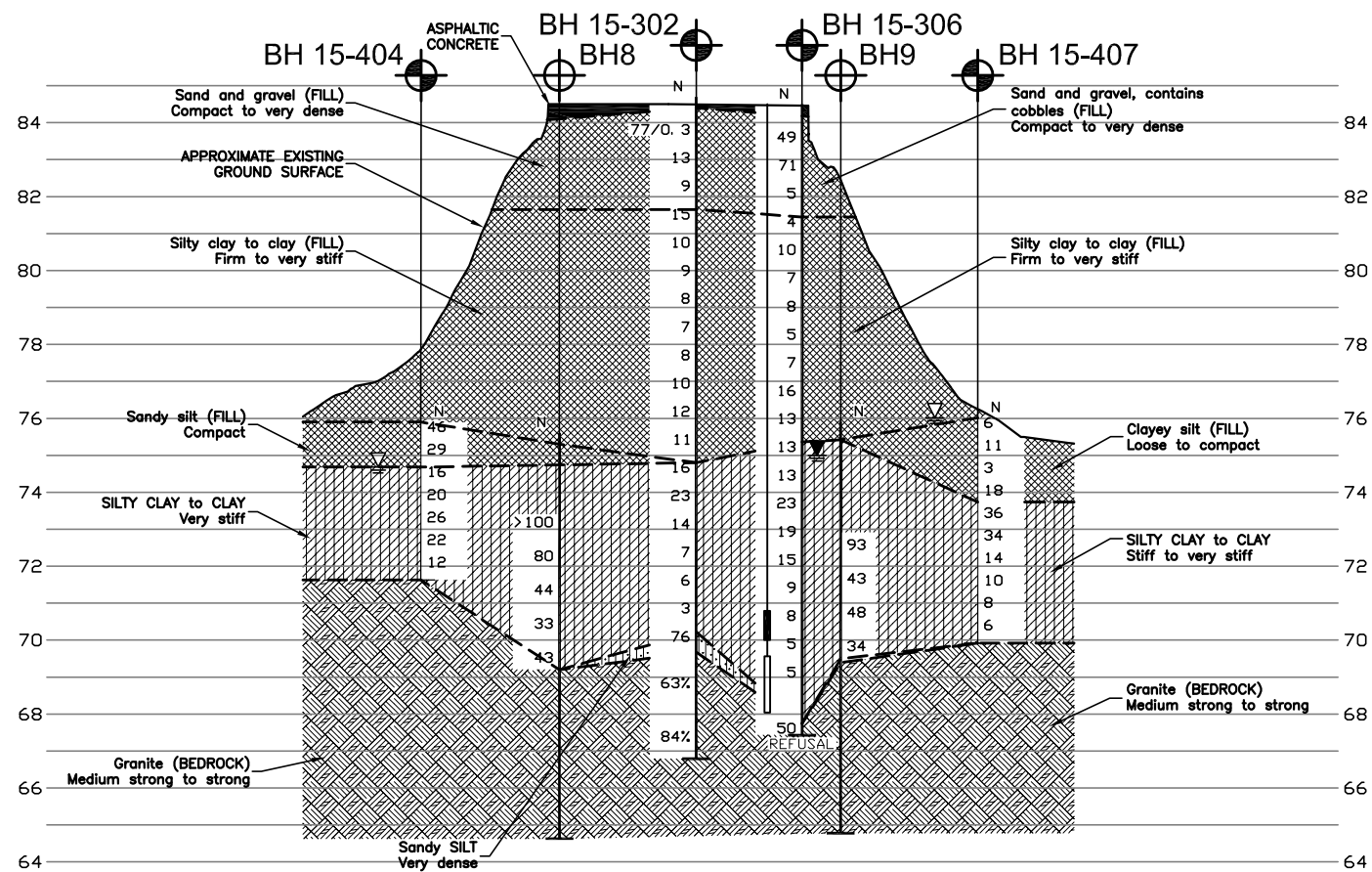
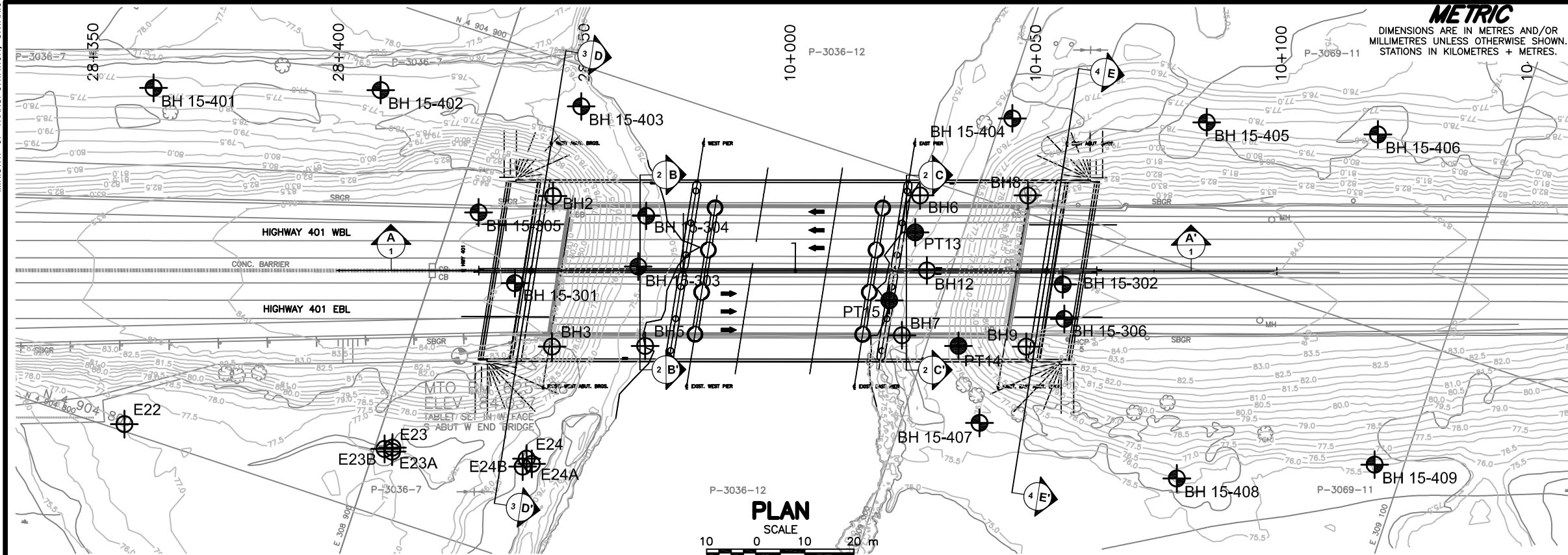
- Borehole - Current Investigation
- Borehole - Previous Investigation
- Penetration Test - Previous Investigation
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- WL in open borehole
- WL in piezometer
- 100% Total Core Recovery (REC)
- Seal
- Piezometer

BOREHOLE CO-ORDINATES

No.	ELEVATION	NORTHING	EASTING
15-301	84.7	4904853.7	308916.6
15-302	84.7	4904888.6	309022.5
15-303	75.9	4904864.8	308939.3
15-304	75.9	4904875.1	308937.6
15-305	84.5	4904865.0	308905.0
15-306	84.5	4904882.0	309025.0
15-401	77.3	4904868.2	308834.2
15-402	76.8	4904882.4	308878.2
15-403	75.7	4904892.1	308918.0
15-404	75.9	4904917.4	309002.1
15-405	78.1	4904929.1	309039.9
15-406	78.3	4904937.7	309073.8
15-407	76.0	4904856.5	309015.3
15-408	76.0	4904858.4	309057.0
15-409	75.6	4904873.8	309094.3
BH2	76.1	4904873.0	308918.3
BH3	75.1	4904843.8	308927.8
BH5	74.6	4904849.9	308945.7
BH6	75.1	4904896.6	308989.2
BH7	75.0	4904868.4	308994.7
BH8	75.6	4904903.5	309010.0
BH9	75.9	4904874.2	309019.4
BH12	75.1	4904882.6	308995.3
PT13	75.1	4904882.6	308995.3
PT15	75.0	4904874.4	308990.0
E-22	77.1	4904801.4	308850.1
E-23	76.6	4904814.1	308903.4
E24	75.1	4904820.5	308930.1



NO.	DATE	BY	REVISION
Geocres No. 31C-233			
HWY. 401	PROJECT NO. 1403255-001		DIST. Eastern
SUBM'D. WAM	CHKD. WAM	DATE: 5/21/2015	SITE: 7-70
DRAWN: JM	CHKD. MJK	APPD. FJH	DWG. 3



REFERENCE

Base plans provided in digital format by MMM Group Limited, drawing file no. 3414034-XB1.dwg, received April 7, 2015.

NOTES

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

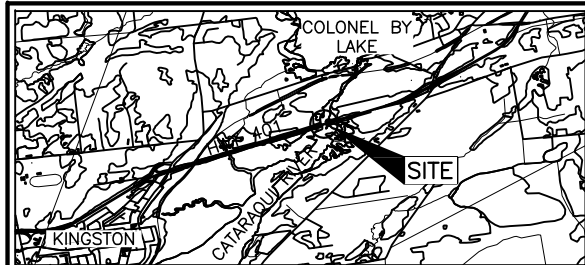
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CONT No.
WP No. 79-99-00

CATARAQUI RIVER BRIDGE
HIGHWAY 401
BOREHOLE LOCATIONS AND SOIL STRATA



Golder Associates Ltd.
OTTAWA ONTARIO, CANADA



KEY PLAN
SCALE
1 0 2 2
KM

LEGEND

- Borehole - Current Investigation
- Borehole - Previous Investigation
- Penetration Test - Previous Investigation
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- WL in open borehole
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15-401	77.3	4904868.2	308834.2
15-402	76.8	4904882.4	308878.2
15-403	75.7	4904892.1	308918.0
15-404	75.9	4904917.4	309002.1
15-405	78.1	4904929.1	309039.9
15-406	78.3	4904937.7	309073.8
15-407	76.0	4904856.5	309015.3
15-408	76.0	4904858.4	309057.0
15-409	75.6	4904873.8	309094.3
BH2	76.1	4904873.0	308918.3
BH3	75.1	4904843.8	308927.8
BH5	74.6	4904849.9	308945.7
BH6	75.1	4904896.6	308989.2
BH7	75.0	4904868.4	308994.7
BH8	75.6	4904903.5	309010.0
BH9	75.9	4904874.2	309019.4
BH12	75.1	4904882.6	308995.3
PT13	75.1	4904882.6	308995.3
PT15	75.0	4904874.4	308990.0
E-22	77.1	4904801.4	308850.1
E-23	76.6	4904814.1	308903.4
E24	75.1	4904820.5	308930.1



NO.	DATE	BY	REVISION
Geocres No. 31C-233			
HWY. 401	PROJECT NO. 1403255-001		DIST. Eastern
SUBM'D. WAM	CHKD. WAM	DATE: 5/21/2015	SITE: 7-70
DRAWN: JM	CHKD. MJK	APPD. FJH	DWG. 4



APPENDIX A

Borehole and Drillhole Records, 2015 Investigation

Lists of Abbreviations and Symbols

Lithological and Geotechnical Rock Description Terminology

Records of Boreholes 15-301 to 15-306 and 15-401 to 15-409



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
DS	Denison type sample
FS	Foil sample
RC	Rock core
SC	Soil core
SS	Split-spoon
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, 60° cone attached to "A" size drill rods for a distance of 300 mm (12 in.).

PH: Sampler advanced by hydraulic pressure

PM: Sampler advanced by manual pressure

WH: Sampler advanced by static weight of hammer

WR: Sampler advanced by weight of sampler and rod

Piezo-Cone Penetration Test (CPT)

A electronic cone penetrometer with a 60° conical tip and a project end area of 10 cm² pushed through ground at a penetration rate of 2 cm/s. Measurements of tip resistance (Q_t), porewater pressure (PWP) and friction along a sleeve are recorded electronically at 25 mm penetration intervals.

III. SOIL DESCRIPTION

(a) Non-Cohesive (Cohesionless) Soils

Density Index	N
Relative Density	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

Per cent 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 (non-cohesive (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
FoS	factor of safety

II. STRESS AND STRAIN

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

III. SOIL PROPERTIES

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

(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 = σ'_p / σ'_{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 = $\tan \delta$
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
2

$$\tau = c' + \sigma' \tan \phi'$$

$$\text{shear strength} = (\text{compressive strength})/2$$



LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY

WEATHERINGS STATE

Fresh: no visible sign of weathering

Faintly weathered: weathering limited to the surface of major discontinuities.

Slightly weathered: penetrative weathering developed on open discontinuity surfaces but only slight weathering of rock material.

Moderately weathered: weathering extends throughout the rock mass but the rock material is not friable.

Highly weathered: weathering extends throughout rock mass and the rock material is partly friable.

Completely weathered: rock is wholly decomposed and in a friable condition but the rock and structure are preserved.

BEDDING THICKNESS

Description	Bedding Plane Spacing
Very thickly bedded	Greater than 2 m
Thickly bedded	0.6 m to 2 m
Medium bedded	0.2 m to 0.6 m
Thinly bedded	60 mm to 0.2 m
Very thinly bedded	20 mm to 60 mm
Laminated	6 mm to 20 mm
Thinly laminated	Less than 6 mm

JOINT OR FOLIATION SPACING

Description	Spacing
Very wide	Greater than 3 m
Wide	1 m to 3 m
Moderately close	0.3 m to 1 m
Close	50 mm to 300 mm
Very close	Less than 50 mm

GRAIN SIZE

Term	Size*
Very Coarse Grained	Greater than 60 mm
Coarse Grained	2 mm to 60 mm
Medium Grained	60 microns to 2 mm
Fine Grained	2 microns to 60 microns
Very Fine Grained	Less than 2 microns

Note: * Grains greater than 60 microns diameter are visible to the naked eye.

CORE CONDITION

Total Core Recovery (TCR)

The percentage of solid drill core recovered regardless of quality or length, measured relative to the length of the total core run.

Solid Core Recovery (SCR)

The percentage of solid drill core, regardless of length, recovered at full diameter, measured relative to the length of the total core run.

Rock Quality Designation (RQD)

The percentage of solid drill core, greater than 100 mm length, recovered at full diameter, measured relative to the length of the total core run. RQD varied from 0% for completely broken core to 100% for core in solid sticks.

DISCONTINUITY DATA

Fracture Index

A count of the number of discontinuities (physical separations) in the rock core, including both naturally occurring fractures and mechanically induced breaks caused by drilling.

Dip with Respect to Core Axis

The angle of the discontinuity relative to the axis (length) of the core. In a vertical borehole a discontinuity with a 90° angle is horizontal.

Description and Notes

An abbreviation description of the discontinuities, whether naturally occurring separations such as fractures, bedding planes and foliation planes or mechanically induced features caused by drilling such as ground or shattered core and mechanically separated bedding or foliation surfaces. Additional information concerning the nature of fracture surfaces and infillings are also noted.

Abbreviations

JN Joint	PL Planar
FLT Fault	CU Curved
SH Shear	UN Undulating
VN Vein	IR Irregular
FR Fracture	K Slickensided
SY Stylolite	PO Polished
BD Bedding	SM Smooth
FO Foliation	SR Slightly Rough
CO Contact	RO Rough
AXJ Axial Joint	VR Very Rough
KV Karstic Void	
MB Mechanical Break	



PROJECT <u>1403255</u>		RECORD OF BOREHOLE No 15-301		SHEET 1 OF 3		METRIC	
G.W.P. <u>79-99-00</u>		LOCATION <u>N 4904853.7 ; E 308916.6</u>		ORIGINATED BY <u>DWM</u>			
DIST <u>Eastern</u> HWY <u>401</u>		BOREHOLE TYPE <u>Portable Drill, NW Casing/BQ Core</u>		COMPILED BY <u>JJL</u>			
DATUM <u>Geodetic</u>		DATE <u>March 20, 2015</u>		CHECKED BY <u>WAM</u>			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					W _P	W	W _L		GR	SA	SI	CL
								20	40	60	80	100	WATER CONTENT (%)							
84.7	GROUND SURFACE																			
0.0	ASPHALTIC CONCRETE																			
84.4																				
0.3	Gravelly sand (FILL) Grey-brown Dry																			
84.0																				
0.7	Sand, trace silt (FILL) Dense Brown Dry		1	SS	43												0	90	6	4
			2	SS	29															
82.7																				
2.0	Gravel, some sand, contains cobbles (FILL) Compact to loose Grey-brown Dry		3	SS	12												74	20	(6)	
			4	SS	7															
			5	SS	5															
			6	SS	50/0.1															
			7	SS	9															
78.6																				
6.1	Gravel, trace sand, contains cobbles (limestone) (FILL) Loose to dense Dry		8	SS	5															
			9	SS	7															
			10	SS	9															
			11	SS	45															
75.9																				
8.8	Silty clay, some sand, trace organic matter (FILL) Stiff Grey-brown to black Moist																			
			12	SS	11															
74.8																				

Continued Next Page

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

GTA-MTO 001 N:\ACTIVE\SPATIAL_IMM_MM_GROUP\CATARAQUIRIVERBRIDGE02_DATA\GINTV1403255.GPJ GAL-GTA.GDT 04/29/15 JM

PROJECT 1403255			RECORD OF BOREHOLE No 15-301			SHEET 2 OF 3			METRIC								
G.W.P. 79-99-00			LOCATION N 4904853.7 ; E 308916.6			ORIGINATED BY DWM											
DIST Eastern HWY 401			BOREHOLE TYPE Portable Drill, NW Casing/BQ Core			COMPILED BY JJJ											
DATUM Geodetic			DATE March 20, 2015			CHECKED BY WAM											
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
	--- CONTINUED FROM PREVIOUS PAGE ---							20	40	60	80	100					
9.9	SILTY CLAY, trace sand Very stiff Brown Moist		13	SS	16												
73.2			14	SS	18												
73.2			15	SS	50/0.1												
11.5	Granite (BEDROCK) Bedrock cored from depths of 11.6 m to 14.7 m For bedrock coring details refer to Record of Drillhole 15-301		1	RC	REC 99%												
70.0			2	RC	REC 100%												
14.7	END OF BOREHOLE																

PROJECT: 1403255

RECORD OF DRILLHOLE: 15-301

SHEET 3 OF 3

LOCATION: N 4904853.7 ; E 308916.6

DRILLING DATE: March 20, 2015

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: Portable Drill

DRILLING CONTRACTOR: Marathon Drilling

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	NOTE: For abbreviations, symbols and descriptions refer to LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY														FEATURES																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																												
						RECOVERY		R.Q.D. %	FRACT. INDEX PER	DIP w.r.t. CORE AXIS °	DISCONTINUITY DATA		HYDRAULIC CONDUCTIVITY K, cm/sec		WEATH- ERING INDEX																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																	
						TOTAL CORE %	SOLID CORE %				TYPE AND SURFACE DESCRIPTION	Jr	Ja	10 ⁻⁹	10 ⁻⁸	10 ⁻⁷	10 ⁻⁶	W1	W2		W3	W4	W5	W6																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																								
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		Continued from Record of Borehole 15-301		73.20																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																												</

DEPTH SCALE

1 : 50



LOGGED: DWM

CHECKED: WAM

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PROJECT <u>1403255</u>		RECORD OF BOREHOLE No 15-302		SHEET 1 OF 3		METRIC	
G.W.P. <u>79-99-00</u>		LOCATION <u>N 4904888.6 ; E 309022.5</u>		ORIGINATED BY <u>DWM</u>			
DIST <u>Eastern</u> HWY <u>401</u>		BOREHOLE TYPE <u>Portable Drill, NW Casing/BQ Core</u>		COMPILED BY <u>JJL</u>			
DATUM <u>Geodetic</u>		DATE <u>March 23, 2015</u>		CHECKED BY <u>WAM</u>			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)				GR	SA	SI	CL	
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED	20	40	60	80	100	w _p	w		w _L				
84.7	GROUND SURFACE																				
0.0	ASPHALTIC CONCRETE																				
84.4																					
0.3	Gravelly sand (FILL) Grey-brown																				
84.0																					
0.7	Sand, trace silt (FILL) Very dense Brown Dry		1	SS	87								○					6	84	6	4
83.6																					
1.1	Sand and gravel, some silt, contains clayey silt layers (FILL) Loose to compact Grey-brown Dry to moist		2	SS	13								○								
			3	SS	9													53	30	(17)	
81.7																					
3.1	Silty sandy gravel (FILL) Compact Grey-brown Moist		4	SS	15								○					35	26	23	16
80.9																					
3.8	Silty clay to clay, some sand, trace gravel (FILL) Firm to stiff Grey-brown Moist		5	SS	10																
			6	SS	9																
			7	SS	8									○				0	2	36	62
			8	SS	7																
			9	SS	8																
			10	SS	10									○							
			11	SS	12																
			12	SS	11																
74.8																					

Continued Next Page

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

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PROJECT 1403255		RECORD OF BOREHOLE No 15-302		SHEET 2 OF 3		METRIC						
G.W.P. 79-99-00		LOCATION N 4904888.6 ; E 309022.5		ORIGINATED BY DWM								
DIST Eastern HWY 401		BOREHOLE TYPE Portable Drill, NW Casing/BQ Core		COMPILED BY JJL								
DATUM Geodetic		DATE March 23, 2015		CHECKED BY WAM								
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC NATURAL LIQUID UNIT REMARKS			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	20 40 60 80 100	W _p W W _L	WATER CONTENT (%)	γ	GR SA SI CL
--- CONTINUED FROM PREVIOUS PAGE ---												
9.9	SILTY CLAY, trace sand, trace gravel Very stiff Brown Moist		13	SS	16		74					0 12 45 43
			14	SS	23							
			15	SS	14		73					
72.3 12.5	SILTY CLAY, trace to some sand, with wet sand seams Firm to stiff Grey Moist		16	SS	7		72					
			17	SS	6		71					
			18	SS	3							
70.2 14.5	Sandy SILT, trace gravel Very dense Wet		19	SS	76		70					7 25 64 4
69.7 15.0	Granite (BEDROCK) Bedrock cored from depths of 15.0 m to 17.9 m For bedrock coring details refer to Record of Drillhole 15-302		1	RC	REC 99%		69					RQD = 63%
			2	RC	REC 99%		68					RQD = 84%
							67					
66.8 17.9	END OF BOREHOLE											

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PROJECT: 1403255

RECORD OF DRILLHOLE: 15-302

SHEET 3 OF 3

LOCATION: N 4904888.6 ;E 309022.5

DRILLING DATE: March 23, 2015

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: Portable Drill

DRILLING CONTRACTOR: Marathon Drilling

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	NOTE: For abbreviations, symbols and descriptions refer to LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY												FEATURES																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																													
						FLUSH RETURN	RECOVERY		R.Q.D. %	FRACT. INDEX PER 100	DIP w.r.t. CORE AXIS °	DISCONTINUITY DATA		HYDRAULIC CONDUCTIVITY K, cm/sec			WEATH- ERING INDEX																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																														
							TOTAL CORE % 000000 000000	SOLID CORE % 000000 000000				TYPE AND SURFACE DESCRIPTION	Jr	Ja	10 ⁻⁶ 10 ⁻⁵ 10 ⁻⁴ 10 ⁻³	10 ⁻⁶ 10 ⁻⁵ 10 ⁻⁴ 10 ⁻³	W1 W2 W3 W4 W5 W6																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																														
		Continued from Record of Borehole 15-302		69.67 15.03																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																											</

DEPTH SCALE



1 : 50



LOGGED: DWM

CHECKED: WAM

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PROJECT 1403255		RECORD OF BOREHOLE No 15-303		SHEET 1 OF 2		METRIC												
G.W.P. 79-99-00		LOCATION N 4904864.8 ; E 308939.3		ORIGINATED BY DWM														
DIST Eastern HWY 401		BOREHOLE TYPE Portable Drill, BW Casing/BQ Core		COMPILED BY JM														
DATUM Geodetic		DATE March 5, 2015		CHECKED BY WAM														
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa					WATER CONTENT (%)			γ kN/m ³	GR SA SI CL	
							20 40 60 80 100	20 40 60 80 100	W _p	W	W _L	25 50 75						
75.9 0.0	GROUND SURFACE Sandy gravel (FILL) Compact to very dense Grey-brown Wet		1	SS	112												63 29 6 2	
			2	SS	30			75										
74.7 1.2	Clayey silt, some sand (FILL) Very loose Grey-brown Wet to moist		3	SS	3												0 12 34 54	
74.1 1.8	Sandy CLAYEY SILT Grey		4	SS	67			74										
73.8 2.1	SILTY CLAY, trace sand Very stiff Brown																1 18 32 49	
73.5 2.4	Dry Granite (BEDROCK) Bedrock cored from depths of 2.4 m to 5.1 m For bedrock coring details refer to Record of Drillhole 15-303		1	RC	REC 100%			73									RQD = 100%	
			2	RC	REC 100%													RQD = 100%
			3	RC	REC 100%													RQD = 100%
70.8 5.1	END OF BOREHOLE		4	RC	REC 100%												RQD = 100%	

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SHEET 2 OF 2

DATUM: Geodetic

DRILLING CONTRACTOR: Marathon Drilling

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CHECKED: WAM

PROJECT 1403255		RECORD OF BOREHOLE No 15-304		SHEET 1 OF 2		METRIC												
G.W.P. 79-99-00		LOCATION N 4904875.1 ; E 308937.6		ORIGINATED BY DWM														
DIST Eastern HWY 401		BOREHOLE TYPE Portable Drill, NW Casing/BQ Core		COMPILED BY JM														
DATUM Geodetic		DATE March 2, 2015		CHECKED BY WAM														
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED					WATER CONTENT (%) W _p W W _L			γ	GR SA SI CL	
75.9 0.0	GROUND SURFACE Sand and gravel, some silt, containing cobbles (FILL) Compact to dense Grey Wet		1	SS	50/0.1													
			2	SS	37													
			3	SS	19													
74.4 1.5	Silty clay, some sand (FILL) Grey-brown Wet		4	SS	50/0.3													
74.1 1.8	Granite (BEDROCK) Bedrock cored from depths of 1.8 m to 4.1 m For bedrock coring details refer to Record of Drillhole 15-304		1	RC	REC 100%												RQD = 100%	
			2	RC	REC 100%												RQD = 100%	
			3	RC	REC 100%												RQD = 99%	
			4	RC	REC 100%												RQD = 84%	
71.8 4.1	END OF BOREHOLE																	

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PROJECT: 1403255

RECORD OF DRILLHOLE: 15-304

SHEET 2 OF 2

LOCATION: N 4904875.1 ;E 308937.6

DRILLING DATE: March 2, 2015

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: Portable Drill

DRILLING CONTRACTOR: Marathon Drilling

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	NOTE: For abbreviations, symbols and descriptions refer to LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY																		FEATURES
				ELEV. DEPTH (m)	RUN No.	FLUSH RETURN	RECOVERY		R.Q.D. %	FRACT INDEX PER	DIP W/L CORE AXIS	DISCONTINUITY DATA		HYDRAULIC CONDUCTIVITY K, cm/sec	WEATH- ERING INDEX							
							TOTAL CORE %	SOLID CORE %				TYPE AND SURFACE DESCRIPTION	Jr		Ja	W1	W2	W3	W4	W5	W6	
							000															

DEPTH SCALE

1 : 50



LOGGED: DWM

CHECKED: WAM

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PROJECT <u>1403255</u>		RECORD OF BOREHOLE No 15-305		SHEET 1 OF 2		METRIC	
G.W.P. <u>79-99-00</u>		LOCATION <u>N 4904865.0 ; E 308905.0</u>		ORIGINATED BY <u>DWM</u>			
DIST <u>Eastern</u> HWY <u>401</u>		BOREHOLE TYPE <u>Portable Drill, NW Casing/BQ Core</u>		COMPILED BY <u>JJL</u>			
DATUM <u>Geodetic</u>		DATE <u>March 17, 2015</u>		CHECKED BY <u>WAM</u>			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)				GR	SA	SI	CL
								20 40 60 80 100					W _P W W _L							
84.5	GROUND SURFACE																			
0.0	ASPHALTIC CONCRETE																			
84.2																				
0.3	Gravelly sand (FILL) Grey-brown Dry						84													
83.7																				
0.8	Sand, trace silt and gravel (FILL) Brown Dry		1	SS	18												2	86 9 3		
83.0																				
1.5	Sand and gravel, contains cobbles (FILL) Loose to dense Grey-brown Dry to moist		2	SS	44		83										60	27 10 3		
			3	SS	6		82													
			4	SS	7		81													
			5	SS	13															
79.9							80													
4.6	Sandy gravel, some silt (FILL) Loose Brown Moist to wet		6	SS	6												53	24 12 11		
79.2																				
5.3	Gravel, trace to some sand and silt (FILL) Loose to very dense Grey-brown Dry		7	SS	4		79													
			8	SS	5		78													
			9	SS	10															
			10	SS	62		77													
76.1																				
8.4	SILTY CLAY, trace sand Very stiff to stiff Brown Moist		11	SS	22		76										0	3 38 59		
			12	SS	20		75													

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

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PROJECT 1403255		RECORD OF BOREHOLE No 15-305				SHEET 2 OF 2		METRIC									
G.W.P. 79-99-00		LOCATION N 4904865.0 ; E 308905.0				ORIGINATED BY DWM											
DIST Eastern HWY 401		BOREHOLE TYPE Portable Drill, NW Casing/BQ Core				COMPILED BY JJL											
DATUM Geodetic		DATE March 17, 2015				CHECKED BY WAM											
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa									
	--- CONTINUED FROM PREVIOUS PAGE ---						20 40 60 80 100 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED					WATER CONTENT (%) 25 50 75					
72.6	SILTY CLAY, trace sand Very stiff to stiff Brown Moist		13	SS	15												
				14	SS	16											
				15	SS	105/0.3											
11.9	END OF BOREHOLE AUGER REFUSAL NOTES: 1. Water level in well screen at a depth of 7.4 m below ground surface (Elev. 77.1 m), measured on March 18, 2015.																

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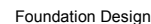
PROJECT <u>1403255</u>		RECORD OF BOREHOLE No 15-306		SHEET 1 OF 2		METRIC	
G.W.P. <u>79-99-00</u>		LOCATION <u>N 4904882.0 ; E 309025.0</u>		ORIGINATED BY <u>DWM</u>			
DIST <u>Eastern</u> HWY <u>401</u>		BOREHOLE TYPE <u>Portable Drill, NW Casing/BQ Core</u>		COMPILED BY <u>JJL</u>			
DATUM <u>Geodetic</u>		DATE <u>March 18 and 19, 2015</u>		CHECKED BY <u>WAM</u>			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							WATER CONTENT (%)
								○ UNCONFINED + FIELD VANE							
						● QUICK TRIAXIAL × REMOULDED									
84.5	GROUND SURFACE							20 40 60 80 100		25 50 75					
84.3	ASPHALTIC CONCRETE														
0.3	Gravelly sand (FILL) Grey-brown Dry Sand, trace to some silt (FILL) Brown						84								
83.3			1	SS	49										
1.2	Sand and gravel, some silt, containing cobbles and boulders (FILL) Loose to very dense Grey Dry						83							54 33 10 3	
			2	SS	71										
			3	SS	5		82								
81.5															
3.1	Silty clay to clay, trace sand and gravel, contains organic matter (FILL) Firm to stiff Grey-brown Moist						81								
			4	SS	4										
			5	SS	10								0 2 28 70		
			6	SS	7		80								
			7	SS	8		79								
			8	SS	5								0 2 32 66		
							78								
			9	SS	7										
			10	SS	16										
			11	SS	13		76								
75.4															
9.1	SILTY CLAY, trace to some sand, trace gravel Very stiff to stiff Brown to grey-brown Moist						75						3 12 41 44		
			12	SS	13										

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

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

PROJECT 1403255		RECORD OF BOREHOLE No 15-401				SHEET 1 OF 1		METRIC									
G.W.P. 79-99-00		LOCATION N 4904868.2 ; E 308834.2				ORIGINATED BY DWM											
DIST Eastern HWY 401		BOREHOLE TYPE Portable Drill, NW Casing/BQ Core				COMPILED BY SL											
DATUM Geodetic		DATE March 11, 2015				CHECKED BY WAM											
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
77.3	GROUND SURFACE							20	40	60	80	100					
0.0	Clayey, sandy silt (FILL) Loose Dark brown Frozen- moist		1	SS	6		77										
76.7																	
0.6	SILTY CLAY, some sand Very stiff Brown Moist		2	SS	16												
76.1																	
1.2	SILTY CLAY, contains sand seams Very stiff Brown Wet		3	SS	20		76										0 3 55 42
			4	SS	19												
			5	SS	27		75										0 2 43 55
			6	SS	50/0.3												
74.0	END OF BOREHOLE SAMPLER REFUSAL						74										
3.3	NOTES: 1. Water level in open borehole at a depth of 1.8 m below ground surface (Elev. m), measured during drilling.																

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PROJECT 1403255		RECORD OF BOREHOLE No 15-402		SHEET 1 OF 1		METRIC						
G.W.P. 79-99-00		LOCATION N 4904882.4 ;E 308878.2		ORIGINATED BY DWM								
DIST Eastern HWY 401		BOREHOLE TYPE Portable Drill, NW Casing/BQ Core		COMPILED BY SL								
DATUM Geodetic		DATE March 10, 2015		CHECKED BY WAM								
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC NATURAL LIQUID UNIT REMARKS			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	20 40 60 80 100	20 40 60 80 100	W _p W W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
76.8	GROUND SURFACE											
0.0	Clayey silt to silty clay, trace sand (FILL) Loose to compact Brown Moist		1	SS	10							
76.2	SILTY CLAY to CLAY, trace sand, fissured/layered Very stiff Brown Moist		2	SS	27		76					0 0 39 61
0.6			3	SS	36							
			4	SS	26		75					
			5	SS	18							
			6	SS	19		74					0 1 63 36
73.1	END OF BOREHOLE SAMPLER REFUSAL											
3.7	NOTES: 1. Open borehole dry upon completion of drilling.											

GTA-MTO 001 N:\ACTIVE\SPATIAL_IMMMM_GROUP\CATARAQUIRIVERBRIDGE02_DATA\GINTV1403255.GPJ GAL-GTA.GDT 05/14/15 JM



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

PROJECT 1403255		RECORD OF BOREHOLE No 15-404		SHEET 1 OF 1		METRIC												
G.W.P. 79-99-00		LOCATION N 4904917.4 ; E 309002.1		ORIGINATED BY DWM														
DIST Eastern HWY 401		BOREHOLE TYPE Portable Drill, NW Casing/BQ Core		COMPILED BY SL														
DATUM Geodetic		DATE March 11, 2015		CHECKED BY WAM														
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									WATER CONTENT (%)	
75.9	GROUND SURFACE							20	40	60	80	100						
0.0	Silty sand, some gravel, contains organics (FILL) Very dense Dark brown Moist		1	SS	46	▽												
75.3																		
0.6	Sandy silt, some gravel, contains organics (FILL) Compact Dark brown Wet		2	SS	29													
74.7																		
1.2	SILTY CLAY, some sand Very stiff Grey-brown Moist		3	SS	16													
			4	SS	20													
			5	SS	26													
72.9																		
3.1	SILTY CLAY to CLAY, trace sand and gravel Stiff Grey Wet		6	SS	22													
			7	SS	12													
71.6																		
4.3	END OF BOREHOLE																	
NOTES:																		
1. Water level in open borehole at a depth of 1.2 m below ground surface (Elev. m), measured during drilling.																		

PROJECT 1403255		RECORD OF BOREHOLE No 15-405		SHEET 1 OF 1		METRIC											
G.W.P. 79-99-00		LOCATION N 4904929.1 ; E 309039.9		ORIGINATED BY DWM													
DIST Eastern HWY 401		BOREHOLE TYPE Portable Drill, NW Casing/BQ Core		COMPILED BY JJL													
DATUM Geodetic		DATE March 16, 2015		CHECKED BY WAM													
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa					WATER CONTENT (%)			γ kN/m³	GR SA SI CL
							20 40 60 80 100	20 40 60 80 100	W _p	W	W _L	25 50 75					
78.1	GROUND SURFACE																
0.0	Silty sand, contains organic matter (FILL)																
0.2	Black to brown Wet		1	SS	87		78										
77.5	Gravel and sand, some clayey silt (FILL)																
0.6	Frozen to very dense Grey		2	SS	12		77										
	Clayey silt to silty clay, contains organic matter and sand layers (FILL)																
	Compact Brown Moist to wet		3	SS	10												
			4	SS	12		76									0 2 33 65	
			5	SS	13												
75.1	SILTY CLAY, contains organic matter Black to green Moist		6	SS	13		75										
74.4	SILTY CLAY, some sand Grey-brown Moist																
3.7	SILTY CLAY, some sand, trace gravel Very stiff Grey Moist		7	SS	17		74									1 20 35 44	
			8	SS	31												
73.2	SILTY CLAY to CLAY, some sand, trace gravel Very stiff Moist		9	SS	33		73									3 17 29 51	
4.9																	
72.6	END OF BOREHOLE																
5.5																	

PROJECT <u>1403255</u>		RECORD OF BOREHOLE No 15-406		SHEET 1 OF 1		METRIC	
G.W.P. <u>79-99-00</u>		LOCATION <u>N 4904937.7 ; E 309073.8</u>		ORIGINATED BY <u>DWM</u>			
DIST <u>Eastern</u> HWY <u>401</u>		BOREHOLE TYPE <u>Portable Drill, NW Casing/BQ Core</u>		COMPILED BY <u>JJL</u>			
DATUM <u>Geodetic</u>		DATE <u>March 16, 2015</u>		CHECKED BY <u>WAM</u>			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)				
								20	40	60	80	100	W _p	W	W _L		
78.3	GROUND SURFACE																
0.0	Clayey silt, trace sand and gravel, contains organic matter (FILL) Compact Brown Moist		1	SS	16											0 3 40 57	
			2	SS	10												
			3	SS	10												
76.5	Inferred ORGANIC SILT and SAND		4	SS	6												
75.9	SILTY CLAY to CLAY, trace sand Stiff Grey to black Moist		5	SS	8												
			6	SS	9												
			7	SS	10											0 3 24 73	
74.0	SILTY CLAY, trace sand Very stiff to hard Grey-brown Moist		8	SS	19												
4.3			9	SS	35												
			10	SS	21											0 4 28 68	
72.2	END OF BOREHOLE																
6.1																	

GTA-MTO 001 N:\ACTIVE\SPATIAL_IMM\MM\GROUP\CATARAQUIRIVERBRIDGE02_DATA\GINTV1403255.GPJ GAL-GTA.GDT 05/14/15 JM

PROJECT 1403255		RECORD OF BOREHOLE No 15-407				SHEET 1 OF 1		METRIC								
G.W.P. 79-99-00		LOCATION N 4904856.5 ; E 309015.3				ORIGINATED BY DWM										
DIST Eastern HWY 401		BOREHOLE TYPE Portable Drill, NW Casing/BQ Core				COMPILED BY JJJ										
DATUM Geodetic		DATE March 13, 2015				CHECKED BY WAM										
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								
76.0	GROUND SURFACE															
0.0	Clayey silt, trace sand, contains rootlets (FILL) Loose to compact Grey-brown Moist		1	SS	6											
			2	SS	11											
74.6	Clayey silt and peat, some sand, contains wood (FILL) Loose to compact Wet		3	SS	3											
1.4			4	SS	18											
73.7	SILTY CLAY to CLAY, trace sand Very stiff Brown Moist		5	SS	36											
2.3			6	SS	34											
			7	SS	14											
71.7	SILTY CLAY, trace sand Stiff Grey Moist		8	SS	10											
4.3			9	SS	8											
			10	SS	6											
69.9	END OF BOREHOLE															
6.1	NOTES: 1. Water level in open borehole at ground surface, measured during drilling.															

GTA-MTO 001 N:\ACTIVE\SPATIAL_IMMMM_GROUP\CATARAQUIRIVERBRIDGE02_DATA\GINTV1403255.GPJ GAL-GTA.GDT 05/14/15 JM

PROJECT 1403255		RECORD OF BOREHOLE No 15-408		SHEET 1 OF 1		METRIC															
G.W.P. 79-99-00		LOCATION N 4904858.4 ;E 309057.0		ORIGINATED BY DWM																	
DIST Eastern HWY 401		BOREHOLE TYPE Portable Drill, NW Casing/BQ Core		COMPILED BY SL																	
DATUM Geodetic		DATE March 13, 2015		CHECKED BY WAM																	
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)						
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa					WATER CONTENT (%)			γ			GR SA SI CL		
76.0	GROUND SURFACE							20 40 60 80 100	20 40 60 80 100	25 50 75											
0.0	Silt, some clay, some sand, contains rootlets (FILL) Compact Dark brown Frozen/moist		1	SS	13																
75.4																					
0.6	SILTY CLAY to CLAY, trace sand Very stiff to hard Brown Moist		2	SS	24		75														
			3	SS	28																
			4	SS	40		74														
			5	SS	39																
			6	SS	26		73														
			7	SS	36																
71.7							72														
4.3	SILTY CLAY to CLAY Very stiff Grey-brown Moist		8	SS	29																
71.1																					
4.9	END OF BOREHOLE																				
	NOTES: 1. Water level in open borehole at a depth of 4.0 m below ground surface (Elev. m), measured during drilling.																				

GTA-MTO 001 N:\ACTIVE\SPATIAL_IMMMM_GROUP\CATARAQUIRIVERBRIDGE02_DATA\GINTV1403255.GPJ GAL-GTA.GDT 05/14/15 JM

PROJECT 1403255		RECORD OF BOREHOLE No 15-409		SHEET 1 OF 1		METRIC											
G.W.P. 79-99-00		LOCATION N 4904873.8 ; E 309094.3		ORIGINATED BY DWM													
DIST Eastern HWY 401		BOREHOLE TYPE Portable Drill, NW Casing/BQ Core		COMPILED BY SL													
DATUM Geodetic		DATE March 12, 2015		CHECKED BY WAM													
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									WATER CONTENT (%)
75.6	GROUND SURFACE							20	40	60	80	100					
0.0	ORGANIC SILT Soft Black Wet		1	SS	3												
74.7			2	SS	3												
0.9	SILTY CLAY, trace sand Very stiff Grey-brown Moist		3	SS	14												
73.8			4	SS	71												
1.8	Sandy SILTY CLAY Hard Grey-brown Moist		5	SS	39												
73.2			6	SS	26												
2.4	SILTY CLAY, trace sand Very stiff to hard Grey-brown Moist		7	SS	13												
71.9			8	SS	16												
3.7	SILTY CLAY to CLAY, trace sand Stiff to very stiff Grey Moist		9	SS	5												
			10	SS	9												
69.5																	
6.1	END OF BOREHOLE																

GTA-MTO 001 N:\ACTIVE\SPATIAL_IMM_MM_GROUP\CATARAQUIRIVERBRIDGE02_DATA\GINTV1403255.GPJ GAL-GTA.GDT 05/14/15 JM



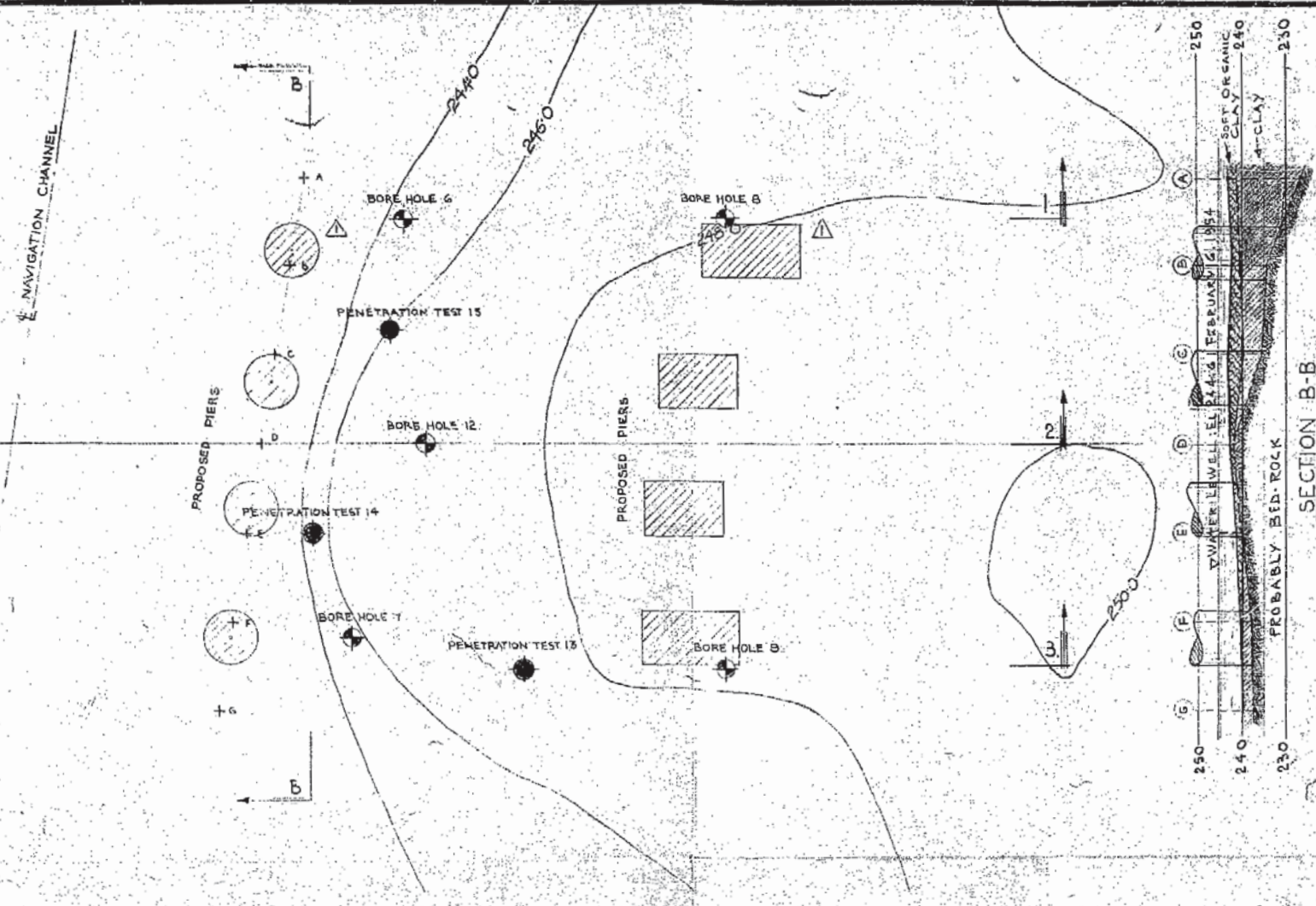
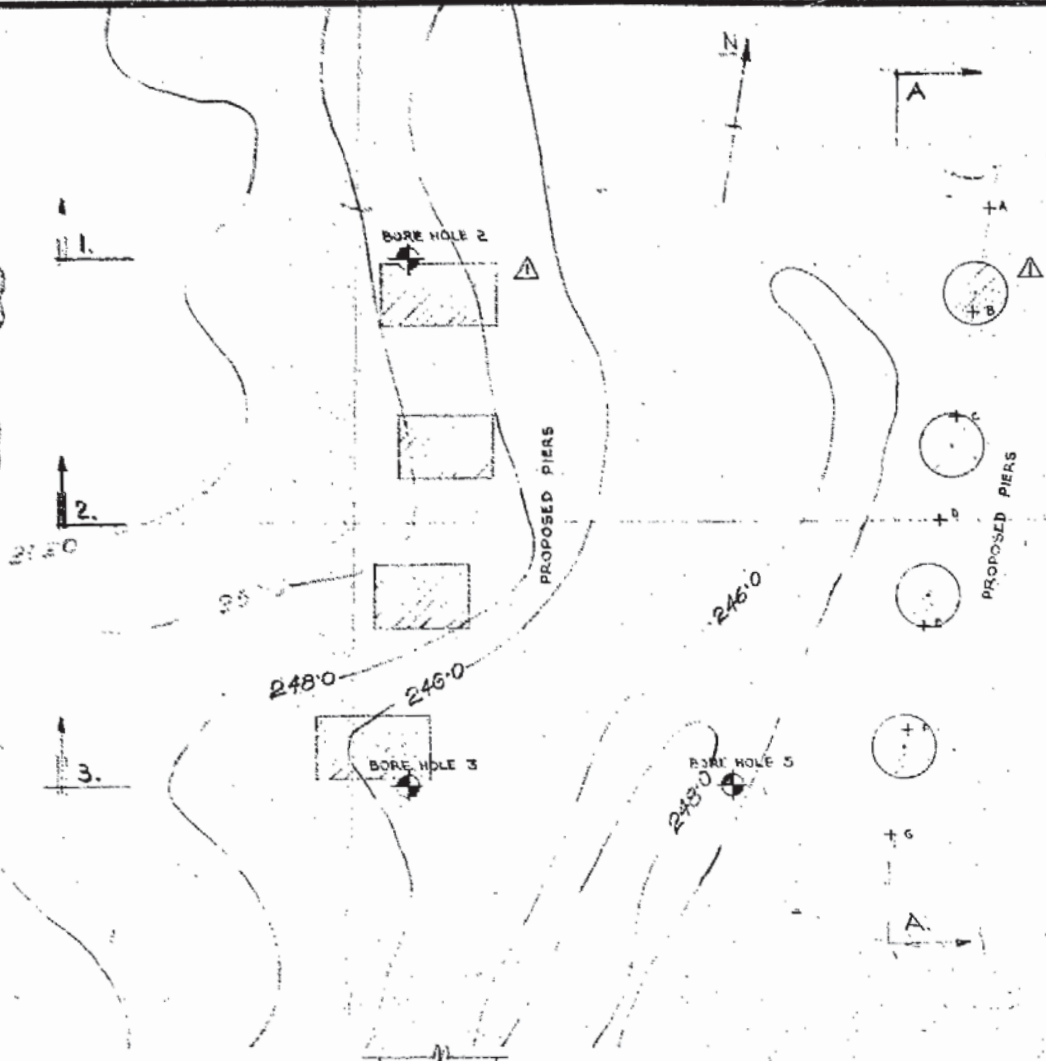
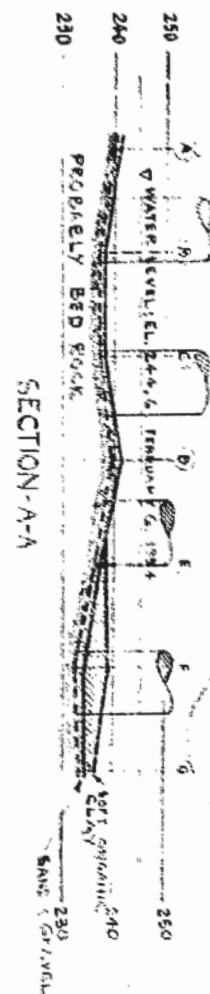
APPENDIX B

**Borehole Records and Geotechnical Laboratory Test Results,
Previous Investigations**

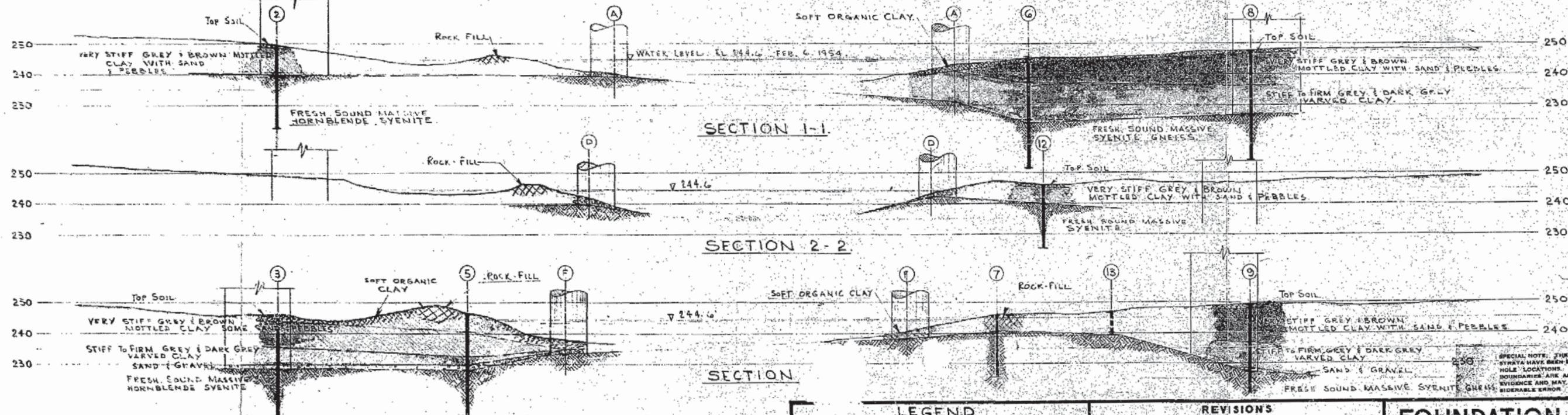
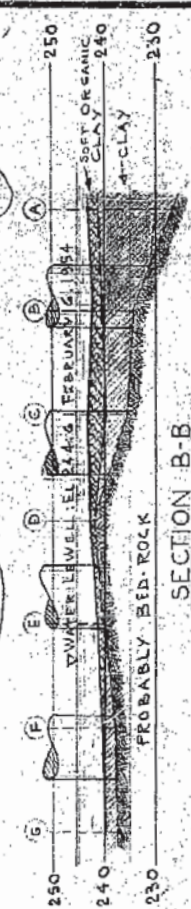
FENCO, 1954

Golder Associates Ltd., 2009

SECTION A-A



SECTION B-B



SPECIAL NOTE: THE BOUNDARIES BETWEEN SOIL STRATA HAVE BEEN ESTABLISHED ONLY AT BORE HOLE LOCATIONS. BETWEEN BOREHOLES THE BOUNDARIES ARE ASSUMED FROM GEOLOGICAL EVIDENCE AND MAY BE SUBJECT TO QUOTE "CONSIDERABLE ERROR"

LEGEND

	PLAN	SECTION	MARK	DATE	DESCRIPTION
PENETRATION TEST	⊙	⊞	△	APRIL 19/54	ADDED LINES OF PROPOSED PIERS
BORING & PENETRATION TEST	⊙	⊞			
JET PROBING	+	+			

REVISIONS

MARK	DATE	DESCRIPTION
△	APRIL 19/54	ADDED LINES OF PROPOSED PIERS

FOUNDATION COMPANIES CANADA

DEPARTMENT OF HIGHWAYS
ONTARIO

CATARAQUI RIVER BRIDGE

BORING PLAN & SOIL STRATIGRAPHY
DATE: March 4, 1954 SCALE 1" = 20'-0"

MADE BY P.F.A. CHKD. APPD. No. 1020-C-2

OFFICE REPORT ON SOIL EXPLORATION

APPENDIX I

DRILL RIG MACHINIC JOB 1020 BORING # 2
 CASING 4" (STANDARD SAMPLERS TO FIT UNLESS NOTED) DATUM GEODETIC DATE REPORT MAR. 4/54
 SAMPLER HAMMER, WT. 430 DROP 15 INCHES COMPILED BY JA CHECKED BY M.A. & M. BORING DATE FEB 9/54

SAMPLE CONDITION



DISTURBED
FAIR
GOOD
LOST

SAMPLE TYPES

C.S. - CHUNK
D.O. - DRIVE-OPEN
D.F. - DRIVE-FOOT VALVE
D.P. - DRIVE PISTON
T.O. - THIN WALLED OPEN
T.P. - THIN WALLED PISTON

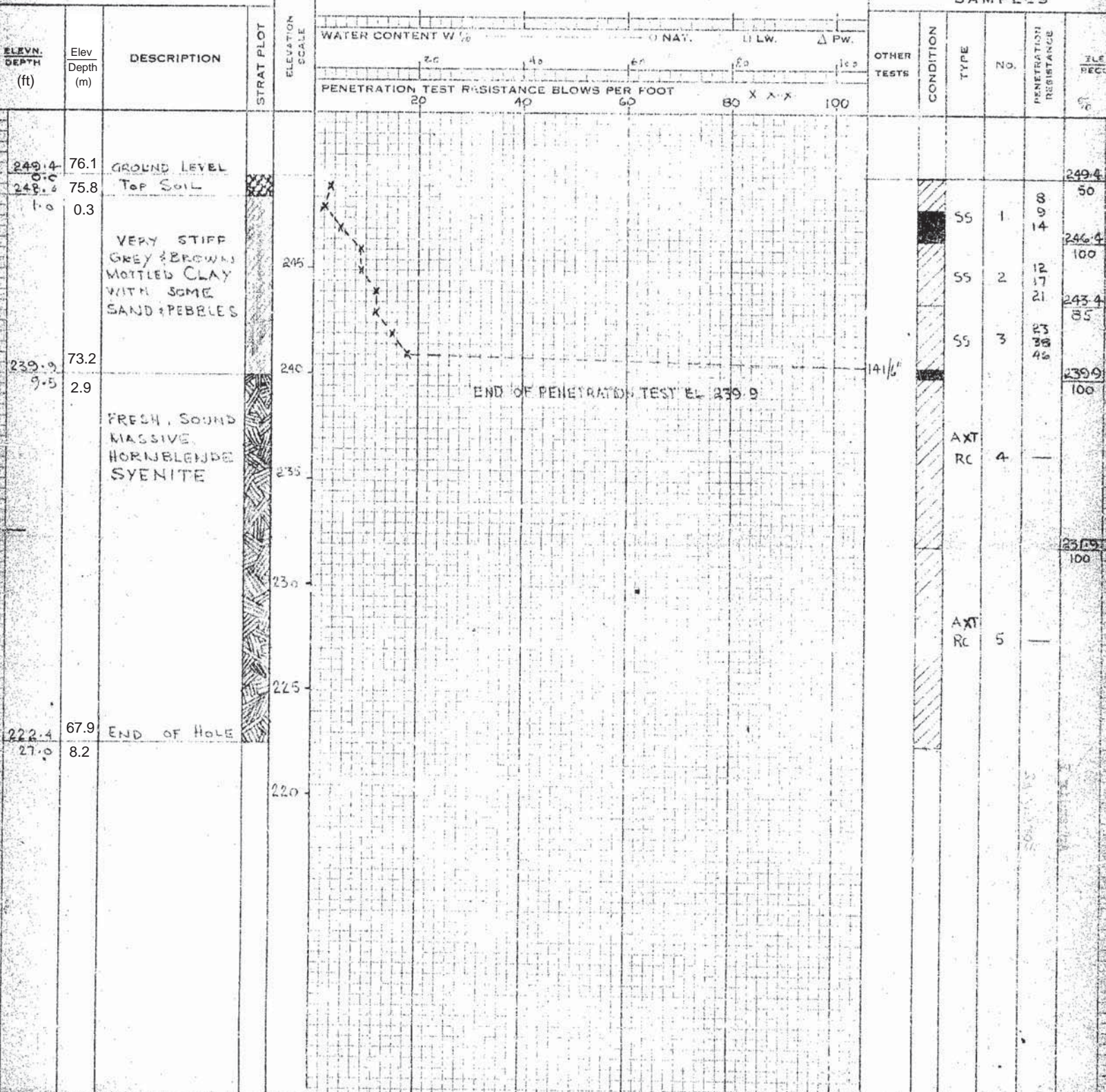
F.S. - FOIL SAMPLE
B.A. - BARREL AUGER
S.A. - SPIRAL AUGER
W.S. - WASHED SAMPLE
R.C. - ROCK CORE
S.S. - SLEEVE SAMPLE

ABBREVIATIONS

V. - IN-SITU VANE SHEAR TEST
M. - MECHANICAL ANALYSIS
U. - UNCONFINED COMPRESSION
QC. - TRIAXIAL CONSOLIDATED QUICK
Q. - TRIAXIAL QUICK
S. - TRIAXIAL SLOW
γ. - UNIT WEIGHT
K. - PERMEABILITY
C. - CONSOLIDATION
CA. - CASING
WL. - WATER LEVEL IN CASING
WT. - WATER TABLE IN SOIL

SOIL PROFILE

SAMPLES



OFFICE REPORT ON SOIL EXPLORATION

APPENDIX I

DRILL RIG. MACHINE JOB 1020 BORING 3
 CASING 4" (STANDARD SAMPLERS TO FIT UNLESS NOTED) DATUM GEODETIC DATE REPORT MAR 14/54
 SAMPLER HAMMER WT. 430 DROP 15 INCHES COMPILED BY JA CHECKED BY M.A.M. BORING DATE FEB 8/54

SAMPLE CONDITION



DISTURBED
FAIR
GOOD
LOST

SAMPLE TYPES

C.S. - CHUNK F.S. - FOIL SAMPLE
 D.O. - DRIVE-OPEN B.A. - BARREL AUGER
 D.F. - DRIVE-FOOT VALVE S.A. - SPIRAL AUGER
 D.P. - DRIVE PISTON W.S. - WASHED SAMPLE
 T.O. - THIN WALLED OPEN H.C. - ROCK CORE
 T.P. - THIN WALLED PISTON S.S. - SLEEVE SAMPLE

ABBREVIATIONS

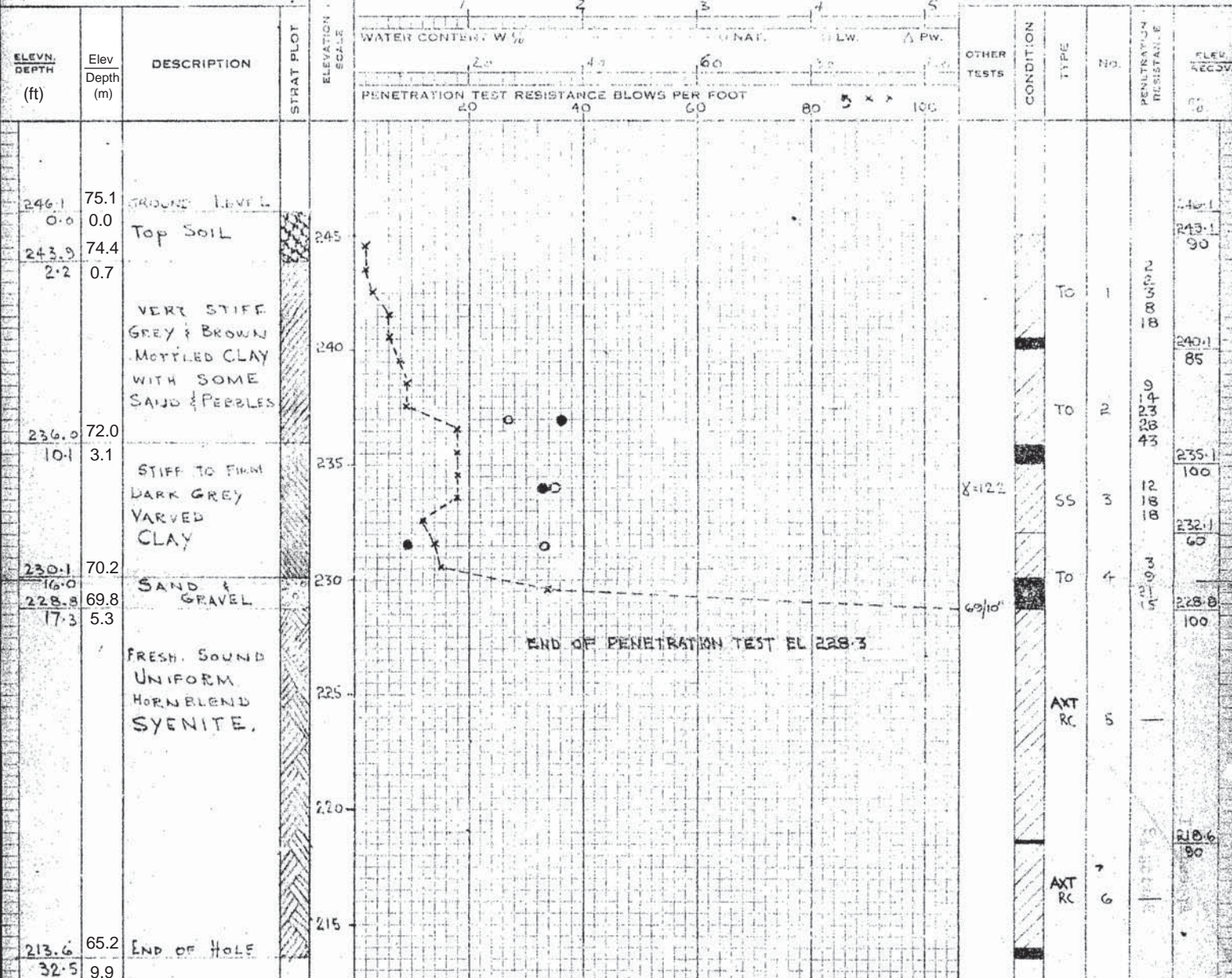
V. - IN-SITU VANE SHEAR TEST Y. - UNIT WEIGHT
 M. - MECHANICAL ANALYSIS K. - PERMEABILITY
 U. - UNCONFINED COMPRESSION C. - CONSOLIDATION
 QC. - TRIAXIAL CONSOLIDATED QUICK CA. - CASING
 Q. - TRIAXIAL QUICK WL. - WATER LEVEL IN CASING
 SL. - TRIAXIAL SLOW WT. - WATER TABLE IN SOIL

SOIL PROFILE

UNCONFINED COMPRESSIVE STRENGTH

Ton./Sq. Foot

SAMPLES



OFFICE REPORT ON SOIL EXPLORATION

APPENDIX I

DRILL RIG: MACHINE JOB: 1020 BORING # 5
 CASING: 4" (STANDARD SAMPLERS TO FIT UNLESS NOTED) DATUM: GEODETIC DATE REPORT: MAR 4/54
 SAMPLER HAMMER WT. 430 DROP: 15 INCHES COMPILED BY: JA CHECKED BY: MANM BORING DATE: FEB 5/54

SAMPLE CONDITION



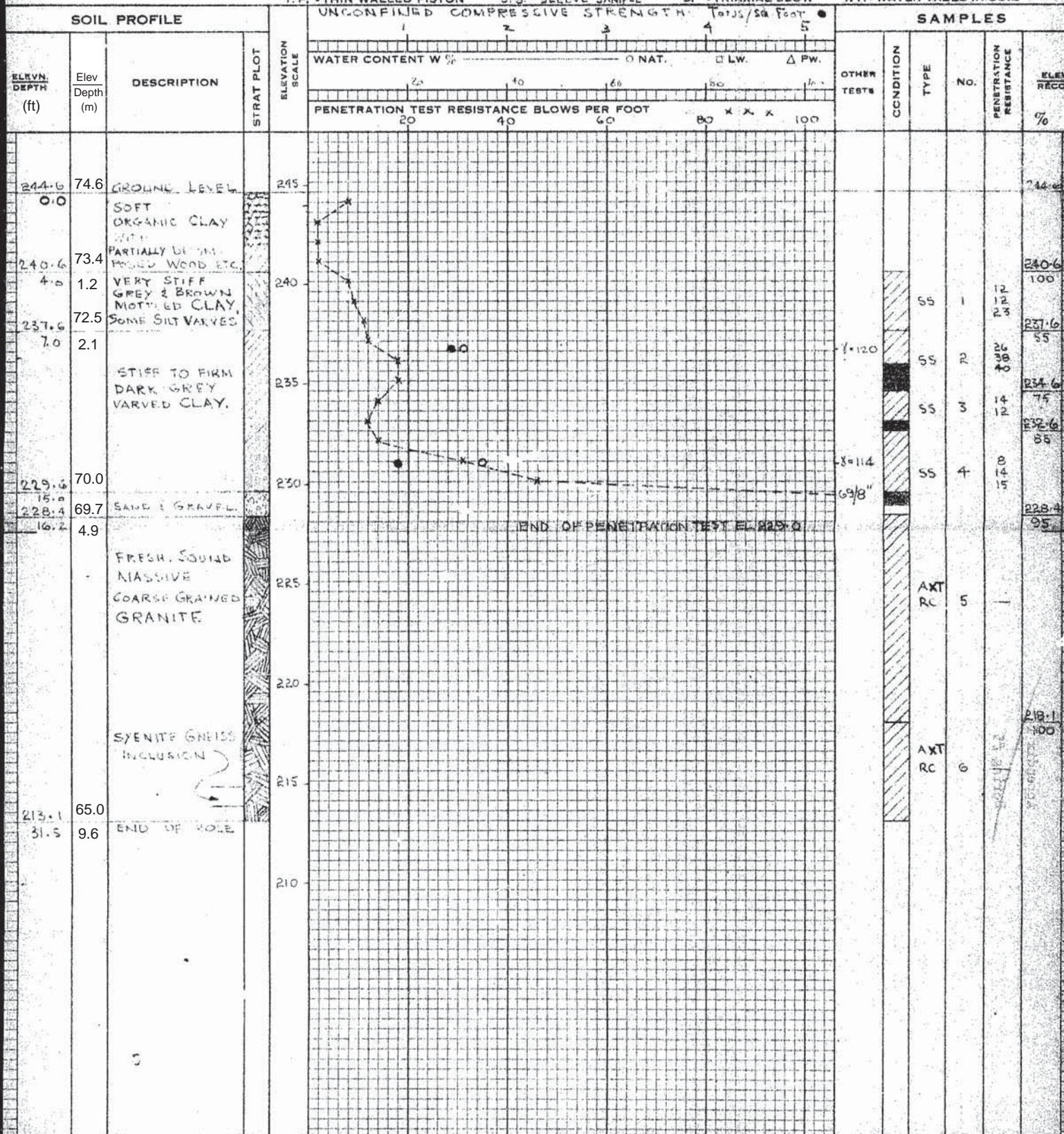
DISTURBED
FAIR
GOOD
LOST

SAMPLE TYPES

C.S. - CHUNK
D.O. - DRIVE-OPEN
D.F. - DRIVE-FOOT VALVE
D.P. - DRIVE PISTON
T.O. - THIN WALLED OPEN
T.P. - THIN WALLED PISTON
F.S. - FOIL SAMPLE
B.A. - BARREL AUGER
S.A. - SPIRAL AUGER
W.S. - WASHED SAMPLE
R.C. - ROCK CORE
S.S. - SLEEVE SAMPLE

ABBREVIATIONS

V. - IN-SITU VANE SHEAR TEST
M. - MECHANICAL ANALYSIS
U. - UNCONFINED COMPRESSION
QC. - TRIAXIAL CONSOLIDATED QUICK
Q. - TRIAXIAL QUICK
S. - TRIAXIAL SLOW
Y. - UNIT WEIGHT WET
K. - PERMEABILITY
C. - CONSOLIDATION
CA. - CASING
WL. - WATER LEVEL IN CASING
WT. - WATER TABLE IN SOIL



OFFICE REPORT ON SOIL EXPLORATION

DRILL RIG. MACHINE

JOB _____ 1020

APPEX I
BORING # 6

CASING (STANDARD SAMPLERS TO FIT UNLESS NOTED)

DATUM 56725710

DATE REPORT March 4, 54

SAMPLER HAMMER, WT. 430 DROP, 15 INCHES

COMPILED BY JA CHECKED BY MA/M

BORING DATE JAN 25-27-54

SAMPLE CONDITION



DISTURBED
FAIR
GOOD
LOST

SAMPLE TYPES

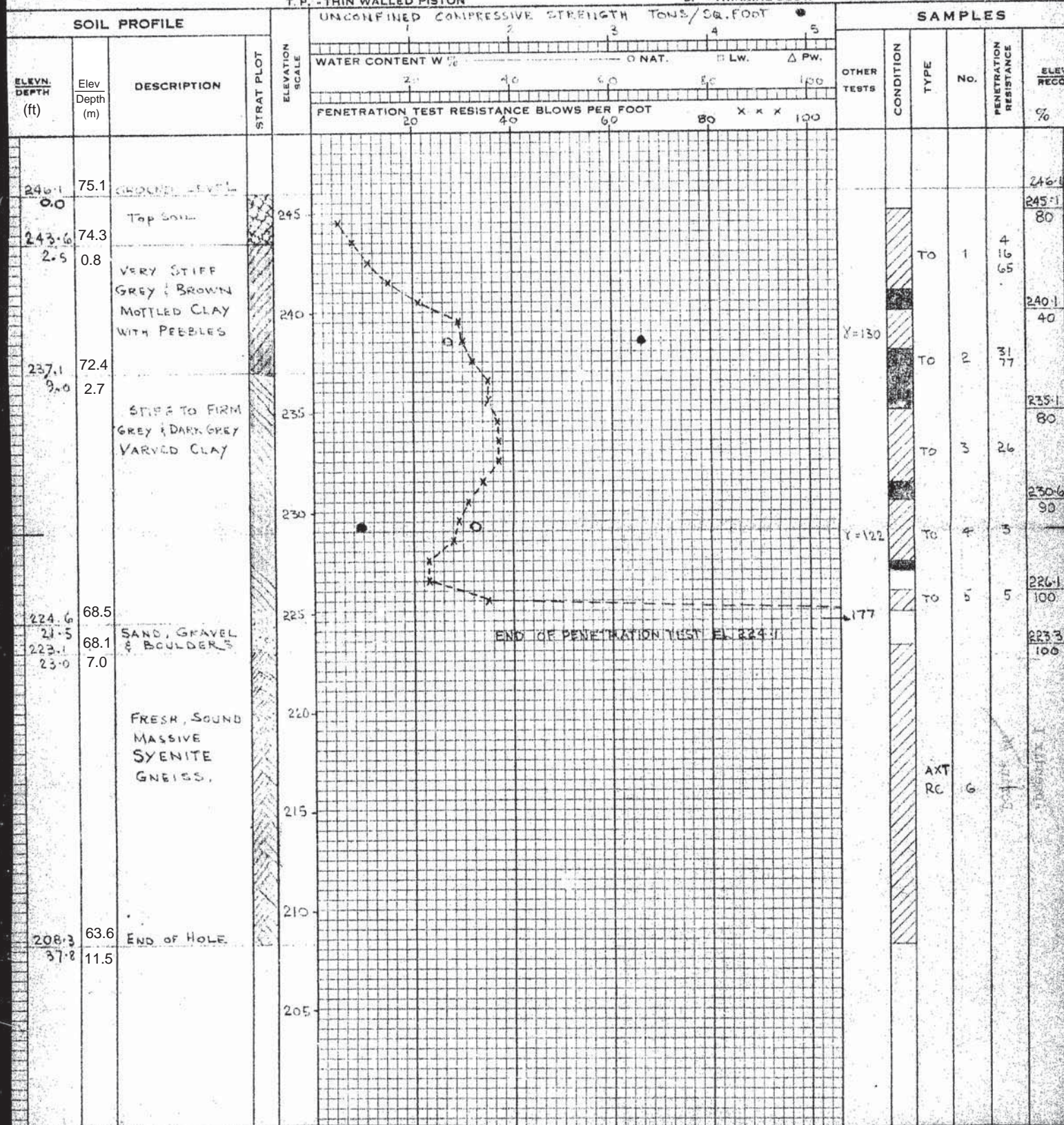
C. S. - CHUNK
D. O. - DRIVE-OPEN
D. F. - DRIVE-FOOT VALVE
D. P. - DRIVE PISTON
T. O. - THIN WALLED OPEN
T. P. - THIN WALLED PISTON

F. S. - FOIL SAMPLE
B. A. - BARREL AUGER
S. A. - SPIRAL AUGER
W. S. - WASHED SAMPLE
R. C. - ROCK CORE

ABBREVIATIONS

ABBREVIATIONS

V. -IN-SITU VANE SHEAR TEST	7. -UNIT WEIGHT WET
M. -MECHANICAL ANALYSIS	K. -PERMEABILITY
U. -UNCONFINED COMPRESSION	C. -CONSOLIDATION
QC. -TRIAxIAL CONSOLIDATED QUICK	CA. -CASING
Q. -TRIAxIAL QUICK	WL. -WATER LEVEL IN CASING
S. -TRIAxIAL SLOW	WT. -WATER TABLE IN SOIL



OFFICE REPORT ON SOIL EXPLORATION

APPENDIX I

DRILL RIG MACHINE JOB 1020 BORING # 7
CASING BX (STANDARD SAMPLERS TO FIT UNLESS NOTED) DATUM GEODATIC DATE REPORT MARCH 4/54
SAMPLER HAMMER. WT. 430 DROP 15 INCHES COMPILED BY JA CHECKED BY MAJ BORING DATE JAN 28-29/54

SAMPLE CONDITION

DISTURBED
FAIR
GOOD
LOST

SAMPLE TYPES

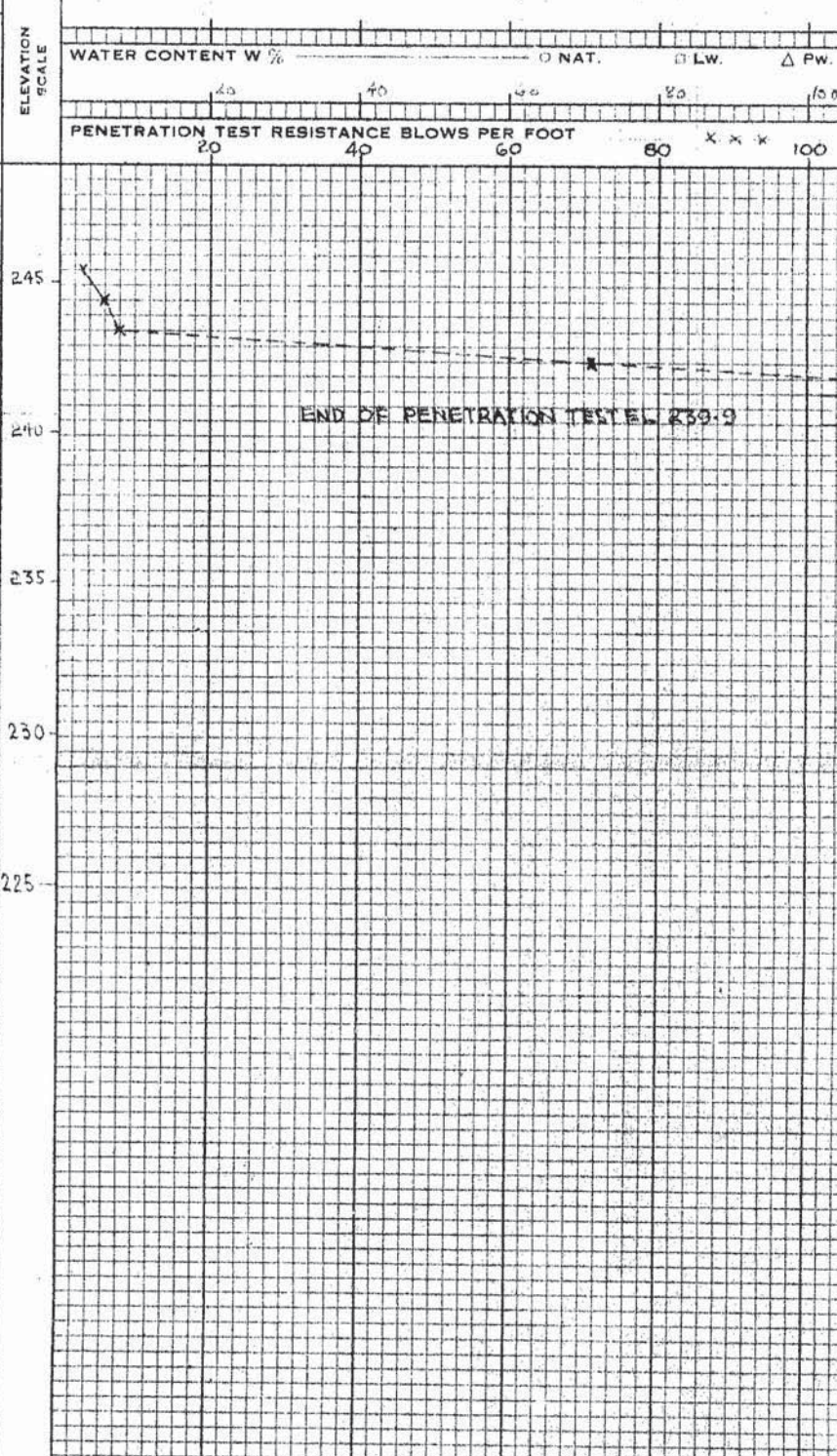
G. S. - CHUNK	F. S. - FOIL SAMPLE
D. O. - DRIVE-OPEN	B. A. - BARREL AUGER
D. F. - DRIVE-FOOT VALVE	S. A. - SPIRAL AUGER
D. P. - DRIVE PISTON	W. S. - WASHED SAMPLE
T. O. - THIN WALLED OPEN	R. C. - ROCK CORE
T. P. - THIN WALLED PISTON	

ABBREVIATIONS


V. - IN-SITU VANE SHEAR TEST γ. - UNIT WEIGHT
M. - MECHANICAL ANALYSIS K. - PERMEABILITY
U. - UNCONFINED COMPRESSION C. - CONSOLIDATION
Qc. - TRIAXIAL CONSOLIDATED QUICK CA. - CASING
Q. - TRIAXIAL QUICK WL. - WATER LEVEL IN CASING
S. - TRIAXIAL SLOW WT. - WATER TABLE IN SOIL

SOIL PROFILE

ELEV. DEPTH (ft)	Elev Depth (m)	DESCRIPTION	STRAT PLOT
245.9 0.0	75.0 0.0	GROUND LEVEL	
		ROCK FILL	
240.6 5.3	73.4 1.6		
		FRESH, SOUND MASSIVE FINE GRAINED GRANITE.	
225.4 20.5	68.8 6.3	END OF HOLE	



SAMPLES

OTHER TESTS	CONDITION	TYPE	No.	PENETRATION RESISTANCE	ELE. RECORDED %
224 345		AST RC	1	—	245.9 240.6 100

OFFICE REPORT ON SOIL EXPLORATION

APPEX I 8
BORING!

DRILL RIG. MACHINE
CASING 1" (STANDARD SAMPLERS TO FIT UNLESS NOTED)
SAMPLER HAMMER. WT. 430 DROP 15 INCHES

JOB 1020 BORING # 8
DATUM GEODETIC DATE REPORT MAR 4/54
COMPILED BY JA CHECKED BY MAJAL BORING DATE FEB 3/54

SAMPLE CONDITION



DISTURBED
 FAIR
 GOOD
 LOST

SAMPLE TYPES

C. S. - CHUCK
D. O. - DRIVE-OPEN
D. F. - DRIVE-FOOT VALVE
D. P. - DRIVE PISTON
T. O. - THIN WALLED OPEN
T. P. - THIN WALLED PISTON

F. S. - FOIL SAMPLE

B. A. - BARREL AUGER
S. A. - SPIRAL AUGER
W. S. - WASHED SAMPLE
R. C. - ROCK CORE
S. S. - SLEEVE SAMPLE

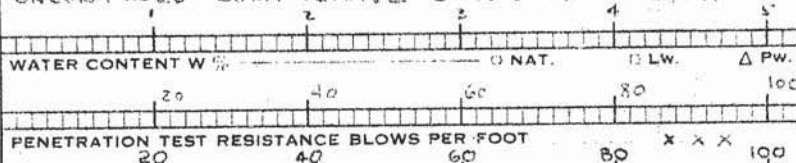
ABBREVIATIONS

V. -IN-SITU VANE SHEAR TEST 7. -UNIT WEIGHT
M. -MECHANICAL ANALYSIS K. -PERMEABILITY
U. -UNCONFINED COMPRESSION C. -CONSOLIDATION
QC. -TRIAxIAL CONSOLIDATED QUICK CA. -CASING
Q. -TRIAxIAL QUICK WL. -WATER LEVEL IN CASING
S. -TRIAxIAL SLOW WF. -WATER TABLE IN SOIL

SOIL PROFILE

ELEV. DEPTH (ft)	Elev Depth (m)	DESCRIPTION	STRAT. PLOT
247.9 8.8	75.6	GROUND LEVEL	
246.9 1.0	75.3	TOP SOIL	
	0.3		
		VERY STIFF GREY & BROWN MOTTLED CLAY WITH SAND & TEREBLES.	
236.4 11.5	72.1		
	3.5	STIFF GREY & DARK GREY VARVED CLAY	
226.9 21.0	69.2		
	6.4	FRESH, SOUND HORN BLENDER SYENITE	
211.9 31.0	64.6	END OF HOLE	
	11.0		

UNCONFINED COMPRESSIVE STRENGTH TONS/SQ.FT.



SAMPLES

OTHER TESTS	CONDITION	TYPE	No.	PENETRATION RESISTANCE	ELEV. RECORDED
					%
					247.9
					246.9
		TO	1	17	65
				18	
				29	
				35	
				58	
					241.9
					50
		SS	2	31	
				49	
				92	
					238.9
					65
		SS	3	26	
				34	
				46	
					235.9
					100
		SS	4	12	
				18	
				26	
					232.9
					65
		SS	5	17	
				15	
				18	
					229.9
					100
		SS	6	20	
				21	
				22	
					226.9
					100
		AXT RC	7	—	
					221.9
					90
		AXT RC	8	—	

OFFICE REPORT ON SOIL EXPLORATION

APPENDIX I

DRILL RIG. MACHINE

JOB

1020

BORING # 9

CASING 4' (STANDARD SAMPLERS TO FIT UNLESS NOTED)

DATUM GEODAYIC

DATE REPORT MARCH 4 1954

SAMPLER HAMMER WT. 430 DROP 15 INCHES

COMPILED BY JA CHECKED BY M.A. N BORING DATE JAN 30 - FEB 1 1954

SAMPLE CONDITION



**DISTURBED
FAIR
GOOD
LOST :**

SAMPLE TYPES

C.S. - CHUNK	F.S. - FOIL SAMPLE
D.O. - DRIVE-OPEN	B.A. - BARREL AUGER
D.F. - DRIVE-FOOT VALVE	S.A. - SPIRAL AUGER
D.P. - DRIVE PISTON	W.S. - WASHED SAMPLE
T.O. - THIN WALLED OPEN	R.C. - ROCK CORE
T.P. - THIN WALLED PISTON	S.S. - SLEEVE SAMPLE

ABBREVIATIONS

V. -IN-SITU VANE SHEAR TEST	7. -UNIT WEIGHT WET
M. -MECHANICAL ANALYSIS	K. -PERMEABILITY
U. -UNCONFINED COMPRESSION	C. -CONSOLIDATION
Qc. -TRIAxIAL CONSOLIDATED QUICK	CA. -CASING
Q. -TRIAxIAL QUICK	WL. -WATER LEVEL IN CASING
S. -TRIAxIAL SLOW	WT. -WATER TABLE IN SOIL

SOIL PROFILE

[illegible]

OFFICE REPORT ON SOIL EXPLORATION

APPENDIX I

DRILL RIG. MACHINE JOB 1020 BORING # 12
 CASING BX (STANDARD SAMPLERS TO FIT UNLESS NOTED) DATUM GEODETIC DATE REPORT MAR 4 / 1954
 SAMPLER HAMMER WT. 430 DROP 15 INCHES COMPILED BY JA CHECKED BY MAJ M BORING DATE FEB 2 / 1954

SAMPLE CONDITION

SAMPLE TYPES

ABBREVIATIONS



DISTURBED
FAIR
GOOD
LOST

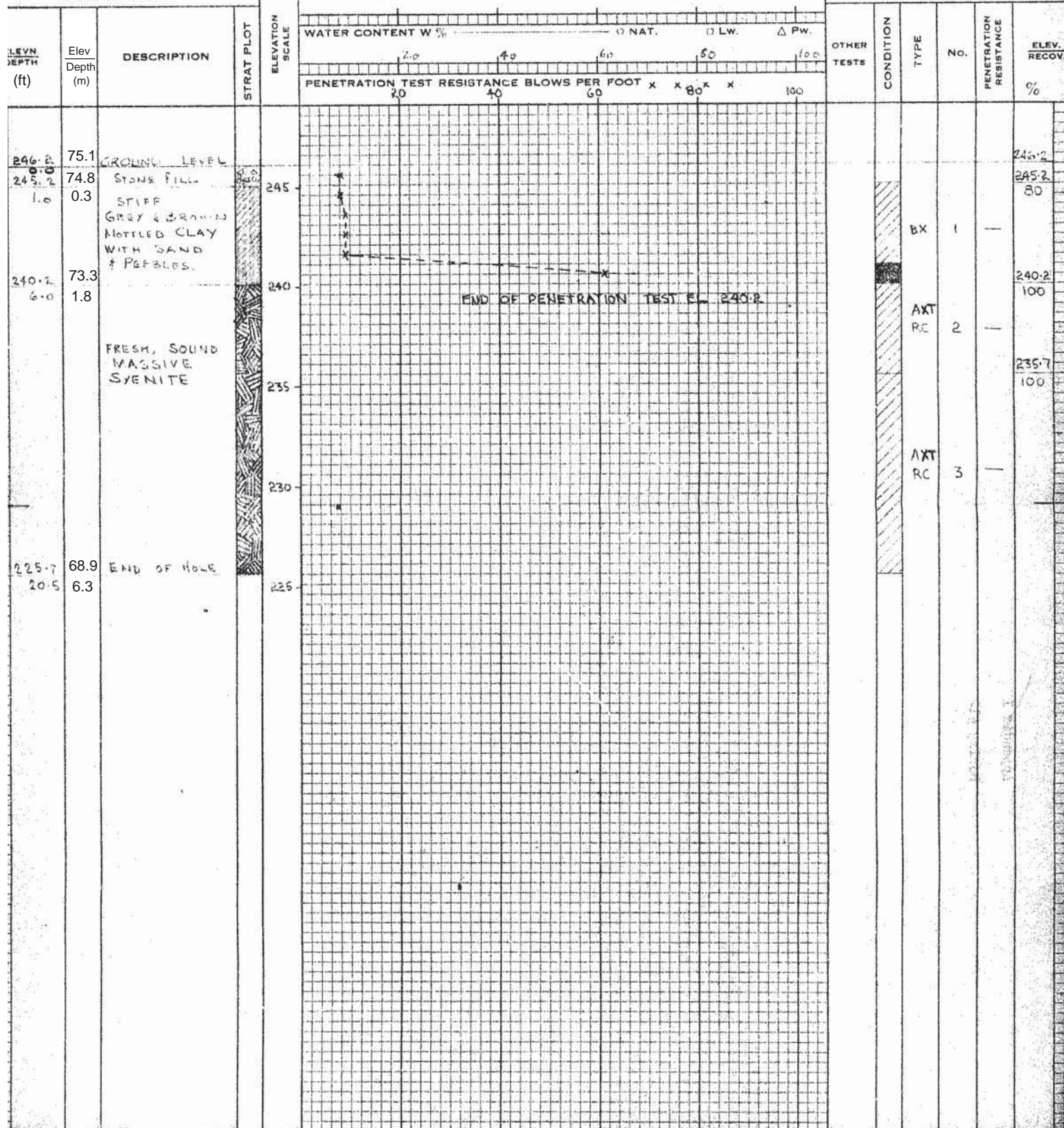
C.S. - CHUNK
D.O. - DRIVE-OPEN
D.F. - DRIVE-FOOT VALVE
D.P. - DRIVE PISTON
T.O. - THIN WALLED OPEN
T.P. - THIN WALLED PISTON

F.S. - FOIL SAMPLE
B.A. - BARREL AUGER
S.A. - SPIRAL AUGER
W.S. - WASHED SAMPLE
R.C. - ROCK CORE

V. - IN-SITU VANE SHEAR TEST
M. - MECHANICAL ANALYSIS
U. - UNCONFINED COMPRESSION
QC. - TRIAXIAL CONSOLIDATED QUICK
Q. - TRIAXIAL QUICK
S. - TRIAXIAL SLOW
Y. - UNIT WEIGHT
K. - PERMEABILITY
C. - CONSOLIDATION
CA. - CASING
WL. - WATER LEVEL IN CASING
WT. - WATER TABLE IN SOIL

SOIL PROFILE

SAMPLES



OFFICE REPORT ON SOIL EXPLORATION

APPENDIX I

DRILL RIG: 1020
 CASING: 1020
 STANDARD SAMPLERS TO FIT UNLESS NOTED
 SAMPLER HAMMER WT: 400 LBS
 DROP: 15 INCHES

JOB: 1020
 DATUM: 1020
 COMPILED BY: JA
 CHECKED BY: JA
 PENETRATION BORING: 1020
 DATE REPORT: 1020
 BORING DATE: 1020

SAMPLE CONDITION



DISTURBED
 FAIR
 GOOD
 LOST

SAMPLE TYPES

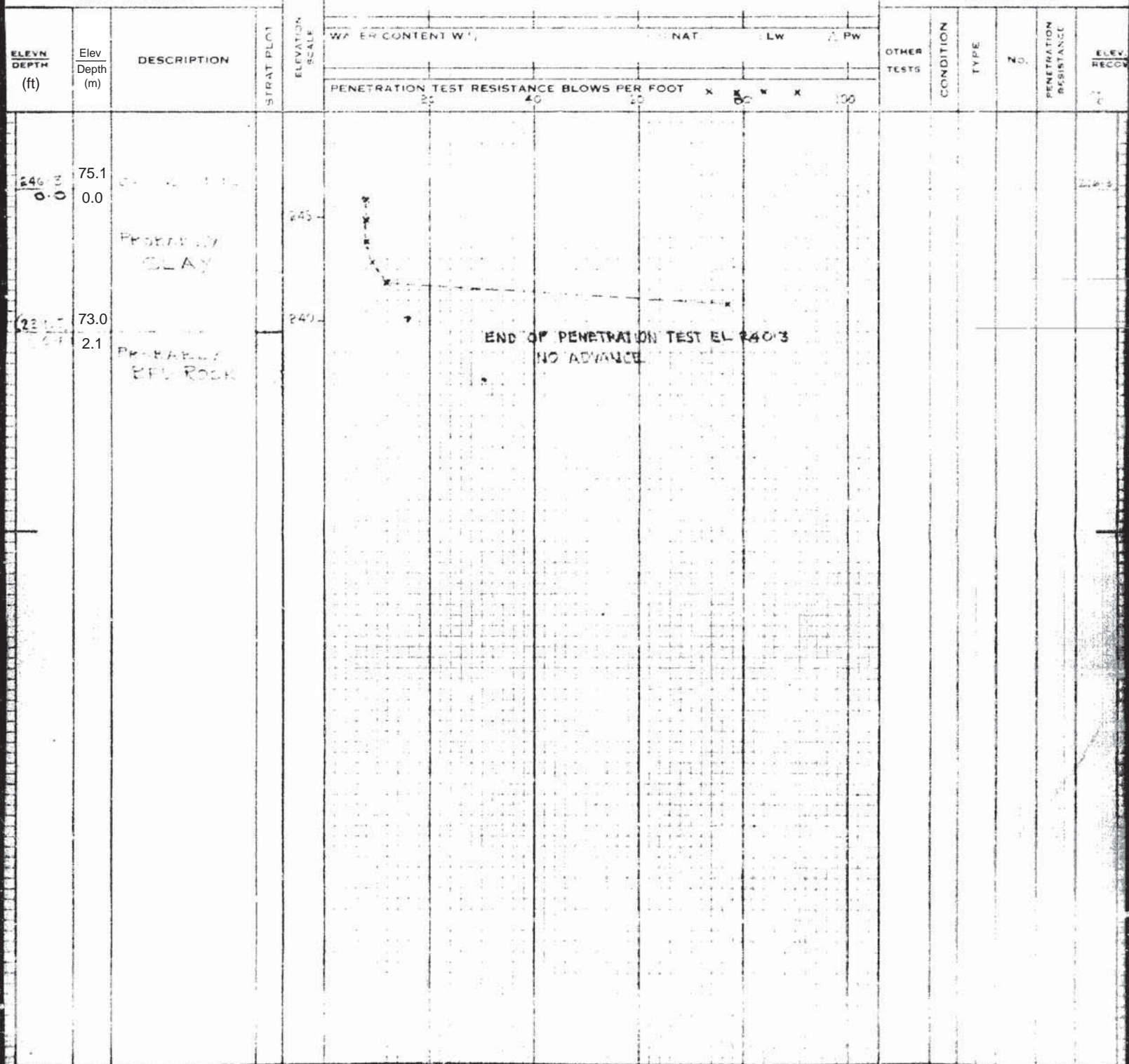
C.S. - CHUNK
 D.O. - DRIVE-OPEN
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ABBREVIATIONS

V. - IN-SITU VANE SHEAR TEST
 M. - MECHANICAL ANALYSIS
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 WL. - WATER LEVEL IN CASING
 WT. - WATER TABLE IN SOIL

SOIL PROFILE

SAMPLES



OFFICE REPORT ON SOIL EXPLORATION

APPENDIX I

DRILL RIG. M4. 11. 12. 13

158

REPERATION
DURING: 14

CASING NO. 116 STANDARD SAMPLERS TO FIT UNLESS NOTED

DATUM 220657Z

DATE REPORT MAY 4 1954


SAMPLER HAMMER, WT. 430 DROP 15 INCHES

COMPILED BY J.A.

CHECKED BY:

BORING DATE Feb 1 1954

SAMPLE CONDITION



DISTURBED
FAIR
GOOD
LOST

SAMPLE TYPES



C.S. - CHUNK	F.S. - FOIL SAMPLE
D.O. - DRIVE OPEN	B.A. - BARREL AUGER
D.F. - DRIVE FOOT VALVE	S.A. - SPIRAL AUGER
D.P. - DRIVE PISTON	W.S. - WASHED SAMPLE
T.O. - THIN WALLED OPEN	R.C. - ROCK CORE
T.P. - THIN WALLED PISTON	

ABBREVIATIONS

V. - IN-SITU VANE SHEAR TEST	7. - UNIT WEIGHT
M. - MECHANICAL ANALYSIS	K. - PERMEABILITY
U. - UNCONFINED COMPRESSION	C. - CONSOLIDATION
Qc. - TRIAXIAL CONSOLIDATED QUICK	CA. - CASING
Q. - TRIAXIAL QUICK	WL. - WATER LEVEL IN CASING
S. - TRIAXIAL SLOW	WT. - WATER TABLE IN SOIL

SOIL PROFILE





SAMPLES

ELEV. DEPTH (ft)	Elev Depth (m)	DESCRIPTION	STRAT PLOT	ELEVATION SCALE	WATER CONTENT W _L			NAT.			LW			PW			OTHER TESTS	CONDITION	TYPE	NO.	PENETRATION RESISTANCE	ELEV. RECOVER
					PENETRATION TEST RESISTANCE BLOWS PER FOOT			X	X	X	X	X	X	X	X							
247.9 0.0	75.6 0.0	PROBABLY CLAY		247																	247.9	
241.1 6.9	73.5 2.1	PROBABLY BED ROCK		241	END OF PENETRATION TEST EL. 241.1																241.1	

OFFICE REPORT ON SOIL EXPLORATION

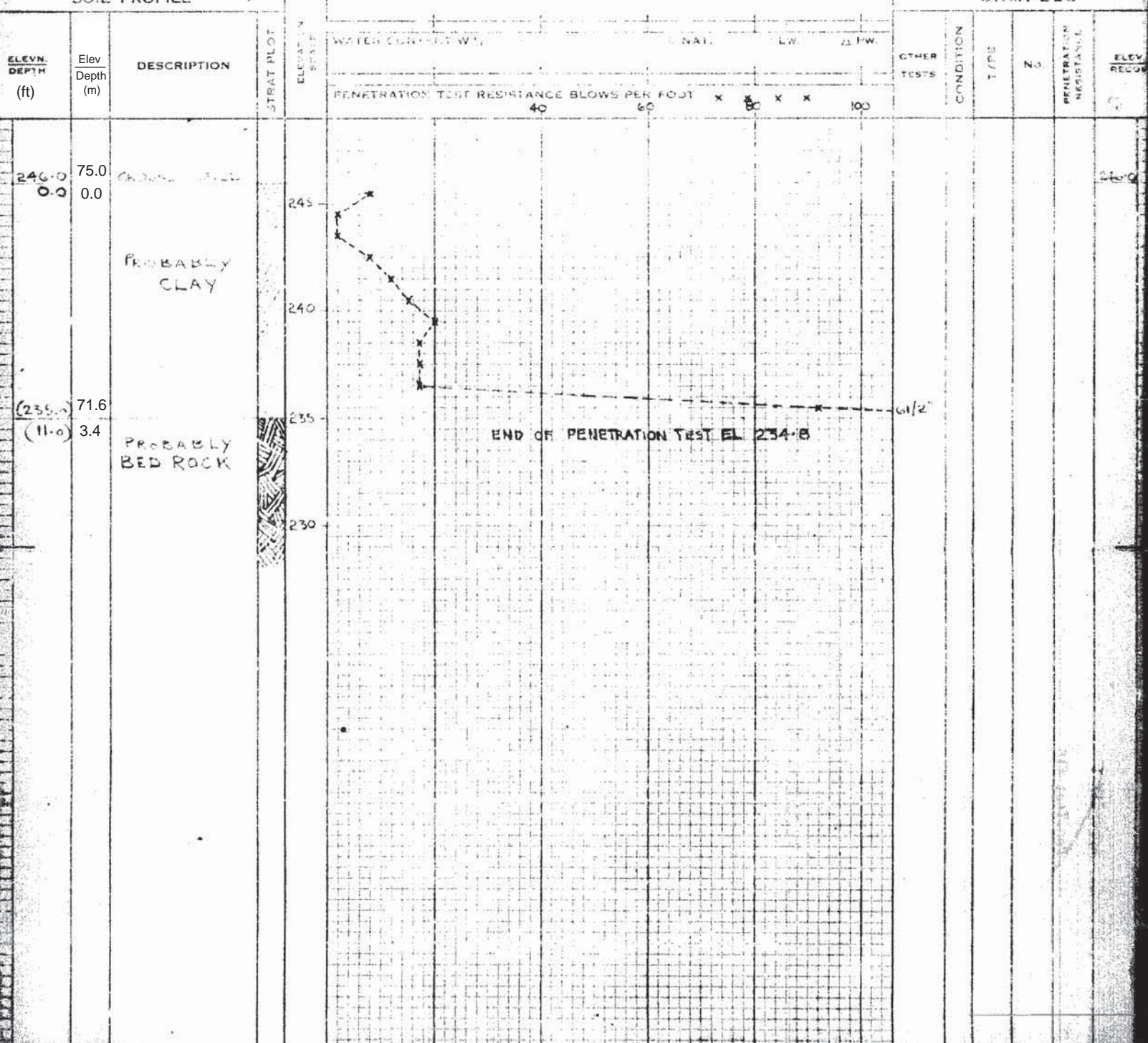
APPENDIX I

DRILL RIG: MACHINE JOB: 1020 PENETRATION: 15
 CASING: 1 1/2" (STANDARD SAMPLERS TO FIT UNLESS NOTED) DATUM: GEODETIC DATE REPORT: MAR 4/54
 SAMPLER HAMMER, WT.: 420 DROP: 15 INCHES COMPILED BY: JA CHECKED BY: MA BORING DATE: FEB 2/54


SAMPLE CONDITION		SAMPLE TYPES		ABBREVIATIONS	
	DISTURBED	C.S. - CHUNK	E.S. - FOUL SAMPLE	V. - IN-SITU VANE SHEAR TEST	U. - UNCONFINED COMPRESSION
	FAIR	C.O. - DRIVE-OPEN	B.A. - BARREL AUGER	M. - MECHANICAL ANALYSIS	K. - PERMEABILITY
	GOOD	D.F. - DRIVE-FOOT VALVE	B.A. - BARREL AUGER	U. - UNCONFINED COMPRESSION	C. - CONSOLIDATION
	LOST	D.F. - DRIVE-FOOT VALVE	W.C. - WASHED SAMPLE	Q. - TRIAXIAL QUICK	W.L. - WATER LEVEL IN CASING
		D.C. - THIN WALLED OPEN	R.C. - ROCK CORE	S. - TRIAXIAL SLOW	W.T. - WATER TABLE IN SOIL
		T.P. - THIN WALLED OPEN			

SOIL PROFILE

SAMPLES






PROJECT 08-1111-0044		RECORD OF BOREHOLE No E22				1 OF 1 METRIC								
G.W.P. 78-99-01		LOCATION N 4904801.4 ; E 308850.1				ORIGINATED BY JEB/DM								
DIST _____ HWY 401		BOREHOLE TYPE Portable Equipment, Continuous Sampling				COMPILED BY AT								
DATUM Geodetic		DATE March 5 and 6, 2009				CHECKED BY KSL								
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa		WATER CONTENT (%)				
77.1	GROUND SURFACE							20 40 60 80 100	20 40 60 80 100	25 50 75				
0.0	CLAY, trace gravel, trace sand Very stiff to hard Brown Moist		1	SS	26									
			2	SS	61									
			3	SS	88									
			4	SS	73									
74.7	SILTY CLAY, trace gravel, trace sand Very stiff to hard Brown and grey Moist		5	SS	75									
2.4			6	SS	73									
			7	SS	45									
			8	SS	19									
			9	SS	22									
			10	SS	16									
			11	SS	70									
70.4	END OF BOREHOLE													
6.7	NOTE: 1. Water level in open borehole at ground surface (Elev. 77.1 m) upon completion of drilling.													

PROJECT 08-1111-0044				RECORD OF BOREHOLE No E23				1 OF 1 METRIC										
G.W.P. 78-99-01				LOCATION N 4904814.1 ; E 308903.4				ORIGINATED BY DM										
DIST _____ HWY 401				BOREHOLE TYPE Portable Equipment, Continuous Sampling				COMPILED BY AT										
DATUM Geodetic				DATE March 9, 2009				CHECKED BY KSL										
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa										
76.6	GROUND SURFACE																	
0.0	Clayey silt, trace sand, containing rootlets (FILL)		1	SS	3													
76.0	Soft Brown Moist		2	SS	15													
0.6	SILTY CLAY, trace sand		3	SS	38													
	Very stiff to hard		4	SS	53													
	Brown to grey Moist		5	SS	35													
73.4	END OF BOREHOLE SPOON REFUSAL		6	SS	100/0.1													
3.2	NOTES: 1. Water level in open borehole at a depth of 2.1 m below ground surface (Elev. 74.5 m) upon completion of drilling. 2. A Dynamic Cone Penetration Test was advanced 0.8 m South of Borehole E23, refusal encountered at a depth of 3.2 m below ground surface (Elev. 73.4 m). 3. An additional borehole was advanced 2 m West of Borehole E23, refusal encountered at a depth of 3.1 m below ground surface (Elev. 73.5 m).																	

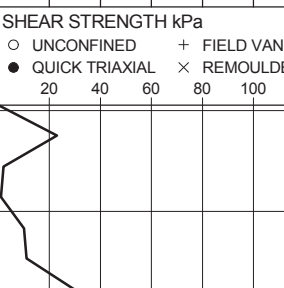


+ 3, × 3: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

PROJECT		RECORD OF BOREHOLE No E23B				1 OF 1 METRIC											
G.W.P. 08-1111-0044		LOCATION N 4904813.3 ; E 308902.0		ORIGINATED BY DM													
DIST _____ HWY 401		BOREHOLE TYPE Portable Equipment, Continuous Sampling		COMPILED BY AT													
DATUM Geodetic		DATE March 9, 2009		CHECKED BY KSL													
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa									WATER CONTENT (%)
76.7	GROUND SURFACE																
0.0	Clayey silt, some sand, containing rootlets (FILL)		1	SS	3												
76.1	Soft Brown Moist		2	SS	14		76										
0.6	SILTY CLAY/CLAYEY SILT, trace to some sand, trace gravel		3	SS	18		75										0 3 51 46
	Firm to hard		4	SS	40												
	Brown Moist		5	SS	39		74										0 6 48 46
73.6	END OF BOREHOLE SPOON REFUSAL		6	SS	750.05												
3.1	NOTE: 1. Water level in open borehole at a depth of 1.8 m below ground surface (Elev. 74.9 m) upon completion of drilling.																

PROJECT 08-1111-0044			RECORD OF BOREHOLE No E24			1 OF 1 METRIC								
G.W.P. 78-99-01			LOCATION N 4904820.5 ; E 308930.1			ORIGINATED BY DM								
DIST _____ HWY 401			BOREHOLE TYPE Portable Equipment, Continuous Sampling			COMPILED BY AT								
DATUM Geodetic			DATE March 10, 2009			CHECKED BY KSL								
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
75.1	GROUND SURFACE							20 40 60 80 100	20 40 60 80 100					
0.0	Peat (Fibrous), trace sand, containing rootlets and decomposed wood fragments (FILL)		1	SS	39									
74.5	Hard Black to brown Wet		2	SS	29									
0.6	SILTY CLAY, trace to some sand, trace gravel Very stiff Brown to grey Wet		3	SS	27									
73.2	SILT, some sand, trace gravel Very dense Brown Wet		4	SS	100/0.15									
2.1	END OF BOREHOLE SPOON REFUSAL													
NOTE: 1. Water level in open borehole at ground surface (Elev.75.1 m) upon completion of drilling. 2. Two Dynamic Cone Penetration Tests were advanced 1.8 m East and 2.0 m South of Borehole E24, refusal encountered at a depth of 1.8 m and 2.0 m below ground surface (Elev.73.3 m and 73.1 m).														

PROJECT <u>08-1111-0044</u>		RECORD OF PENETRATION TEST No E24A		1 OF 1 METRIC	
G.W.P. <u>78-99-01</u>		LOCATION <u>N 4904819.9 ; E 308931.4</u>		ORIGINATED BY <u>DM</u>	
DIST <u> </u> HWY <u>401</u>		BOREHOLE TYPE <u>Portable Equipment, Dynamic Cone Penetration Test</u>		COMPILED BY <u>AT</u>	
DATUM <u>Geodetic</u>		DATE <u>March 10, 2009</u>		CHECKED BY <u>KSL</u>	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT	PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED	WATER CONTENT (%) w _p w w _L				
75.1 0.0	GROUND SURFACE Start of Dynamic cone Penetration Test (DCPT)												
73.2 1.8	END OF DCPT Refusal to further Penetration (Hammer Bouncing)												

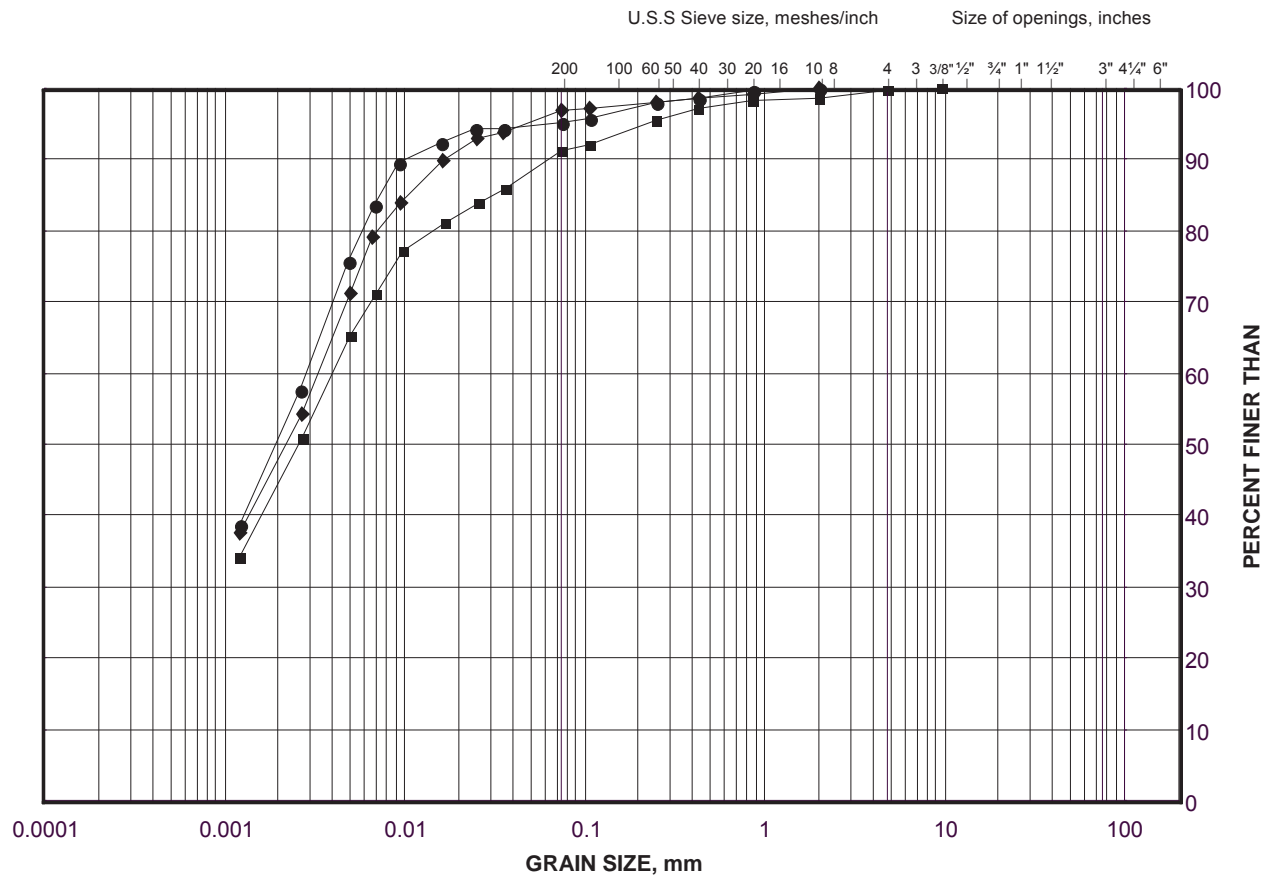
PROJECT <u>08-1111-0044</u>		RECORD OF PENETRATION TEST No E24B		1 OF 1 METRIC	
G.W.P. <u>78-99-01</u>		LOCATION <u>N 4904818.7 ; E 308929.8</u>		ORIGINATED BY <u>DM</u>	
DIST <u> </u> HWY <u>401</u>		BOREHOLE TYPE <u>Portable Equipment, Dynamic Cone Penetration Test</u>		COMPILED BY <u>AT</u>	
DATUM <u>Geodetic</u>		DATE <u>March 10, 2009</u>		CHECKED BY <u>KSL</u>	

SOIL PROFILE		SAMPLES				GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100	W _p	W	W _L		
75.1 0.0	GROUND SURFACE Start of Dynamic cone Penetration Test (DCPT)						75										
73.1 2.0	END OF DCPT Refusal to Further Penetration (100 blows/0.15 m)						74										

GRAIN SIZE DISTRIBUTION

Silty Clay

FIGURE B1



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	E22	10	71.3
■	E24	3	73.6
◆	E23	3	75.1

Project Number: 08-1111-0044

Checked By: KSL

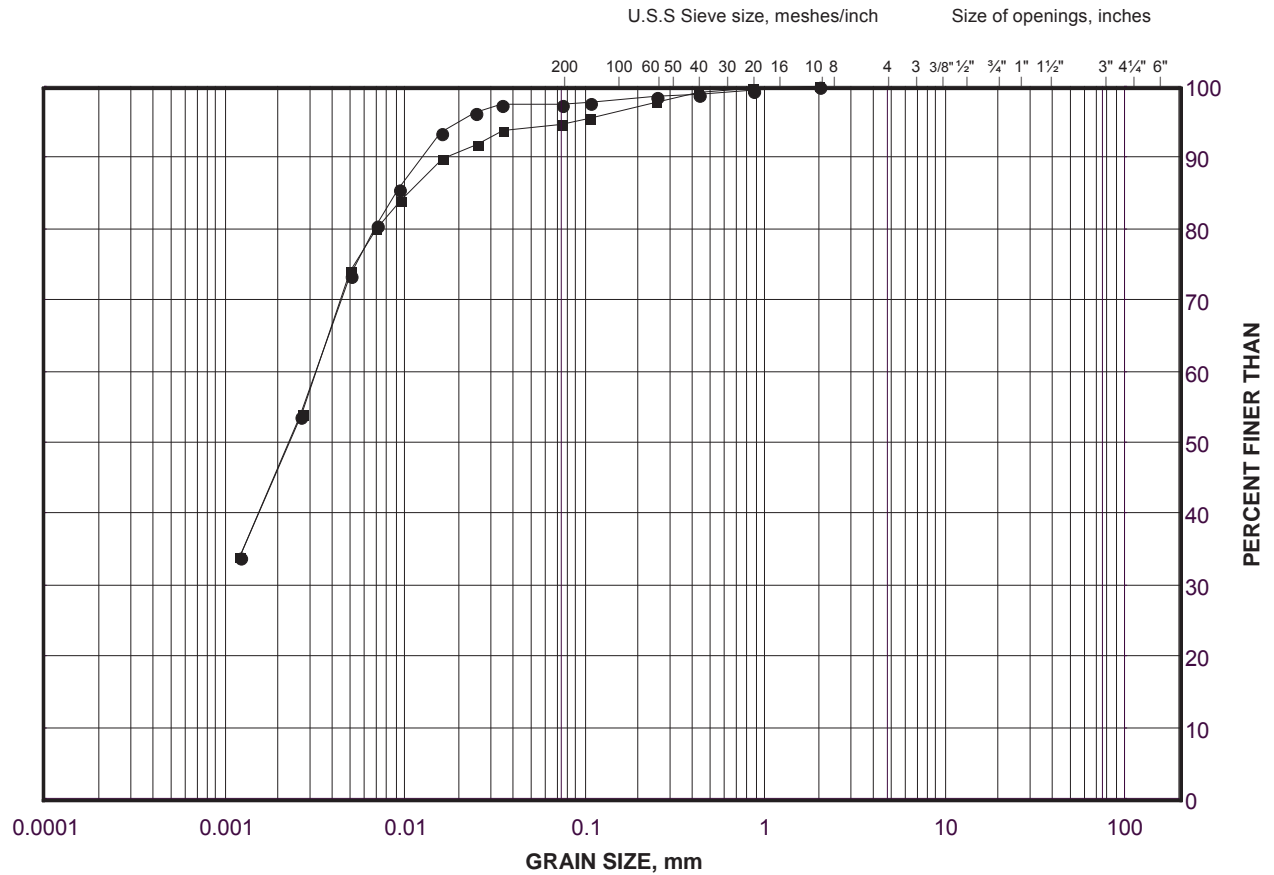
Golder Associates

Date: 30-Mar-10

GRAIN SIZE DISTRIBUTION

Clayey Silt

FIGURE B2



LEGEND

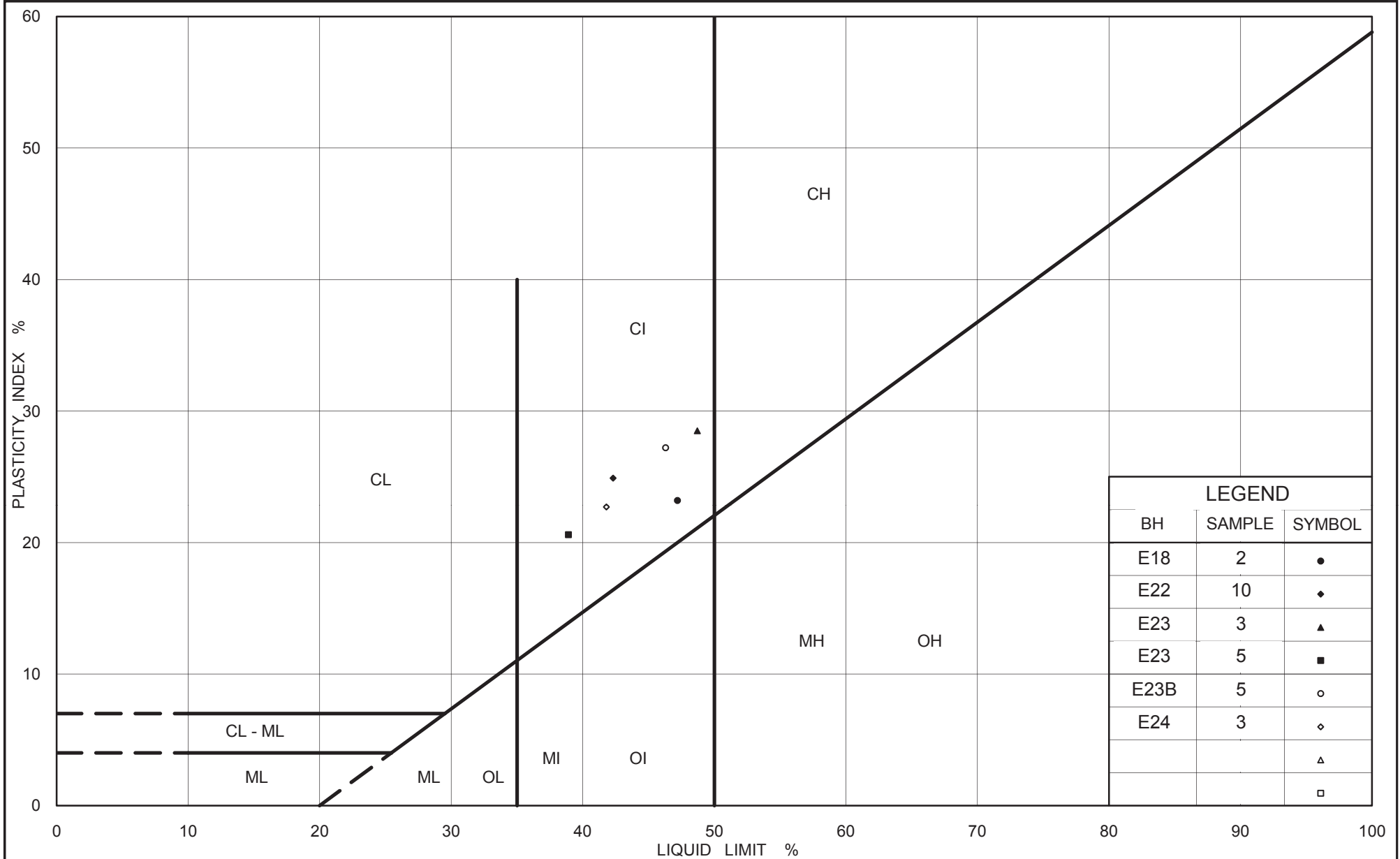
SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	E23B	3	75.2
■	E23B	5	74.0

Project Number: 08-1111-0044

Checked By: KSL

Golder Associates

Date: 30-Mar-10



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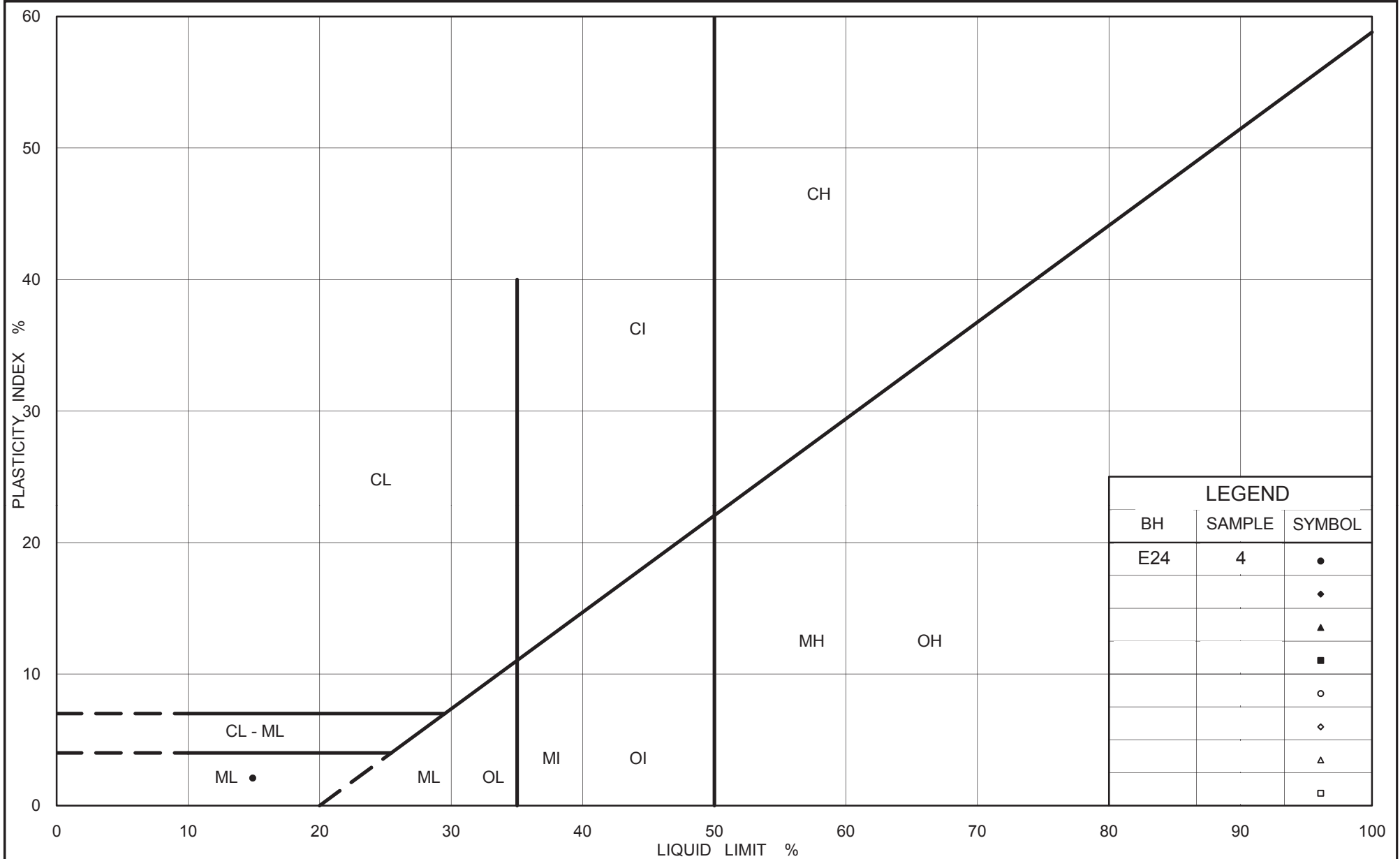
Ontario

PLASTICITY CHART Silty Clay

Figure No. B3

Project No. 08-1111-0044

Checked By: KSL



Ministry of Transportation

Ontario

PLASTICITY CHART

Silt

Figure No. B4

Project No. 08-1111-0044

Checked By: KSL



APPENDIX C

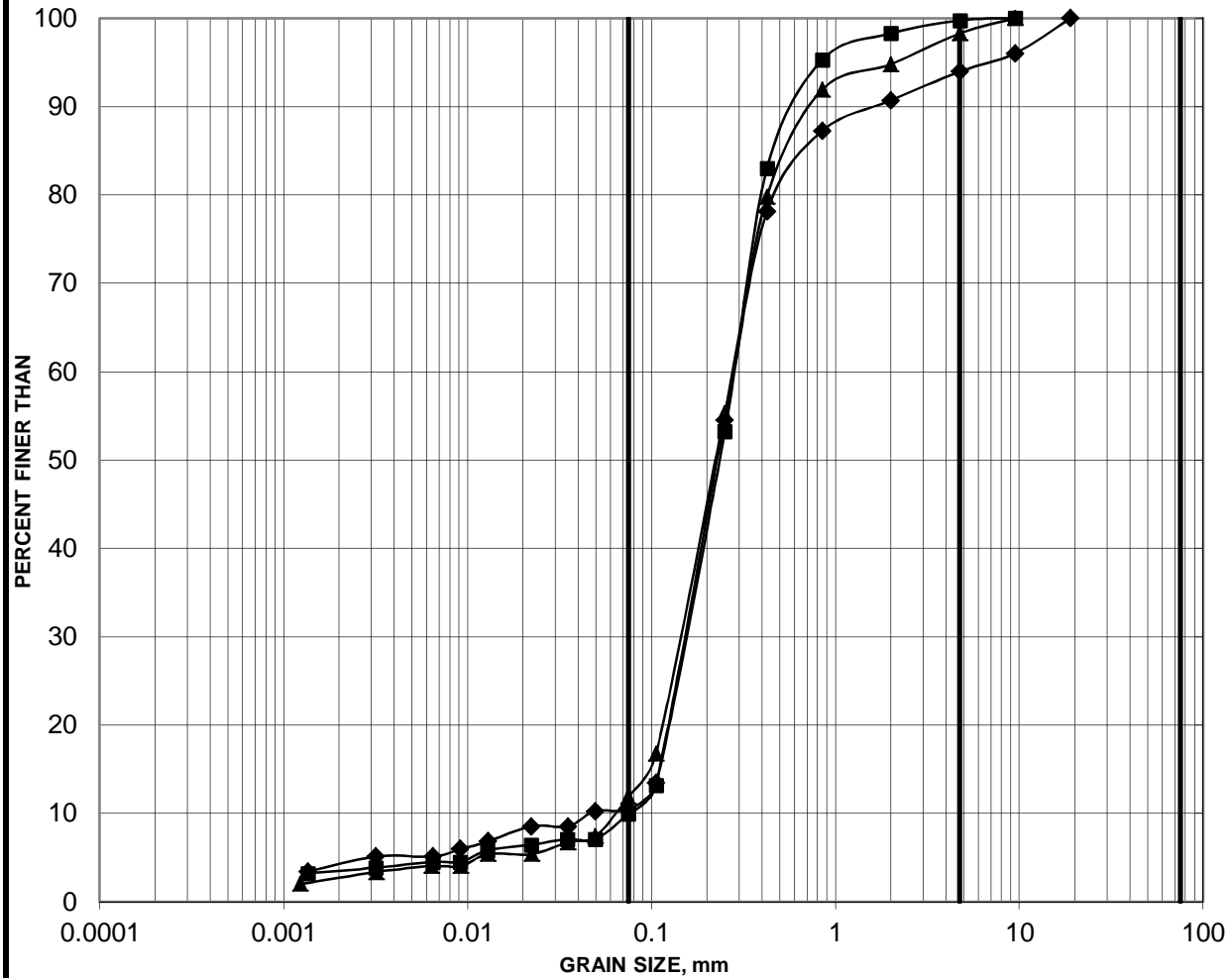
Geotechnical Laboratory Test Results, 2015 Investigation Figures:

Figure C1	Grain Size Distribution – Sand, Some Silt (Upper Embankment Fill)
Figure C2	Grain Size Distribution – Gravel, Some Sand to Sandy (Embankment Fill)
Figure C3	Grain Size Distribution – Silty Sandy Gravel (Embankment Fill)
Figure C4	Grain Size Distribution – Silty Clay to Clay (Embankment Fill)
Figure C5	Plasticity Chart – Silty Clay (Embankment Fill)
Figure C6A	Grain Size Distribution – Silty Clay to Clay – 300-Series Boreholes
Figure C6B	Grain Size Distribution – Silty Clay to Clay – 400-Series Boreholes
Figure C7	Plasticity Chart – Silty Clay to Clay
Figure C8	Grain Size Distribution – Sandy Silt
Figure C9	Summary of Laboratory Compressive Strength – Unconfined Compression Tests

GRAIN SIZE DISTRIBUTION

FIGURE C1

SAND, SOME SILT (UPPER EMBANKMENT FILL)



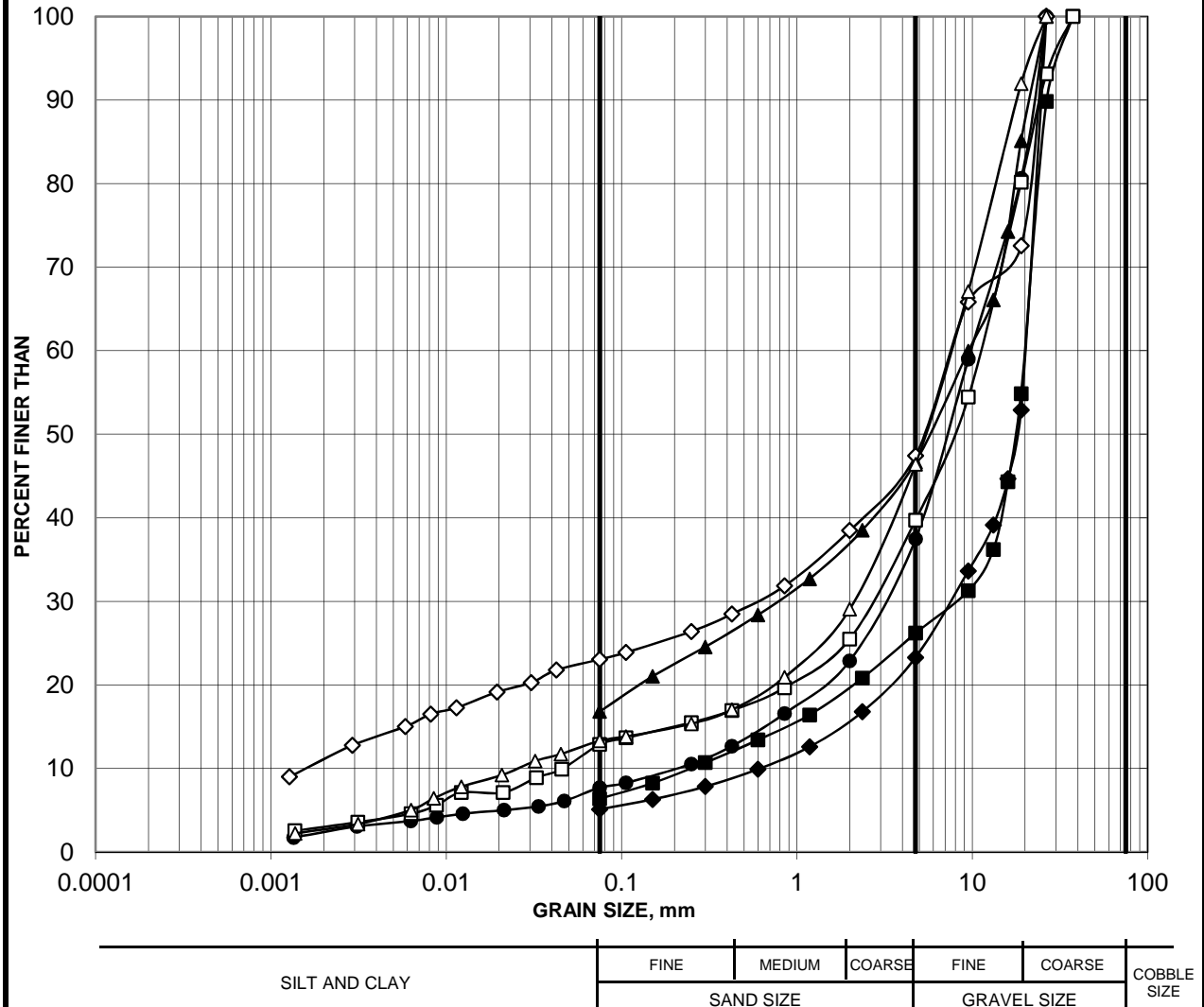
SILT AND CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
	SAND SIZE			GRAVEL SIZE		

Borehole	Sample	Depth (m)
15-301	1	0.76-1.37
15-302	1	0.76-1.07
15-305	1	0.76-1.37

GRAIN SIZE DISTRIBUTION

FIGURE C2

GRAVEL, SOME SAND TO SANDY (EMBANKMENT FILL)

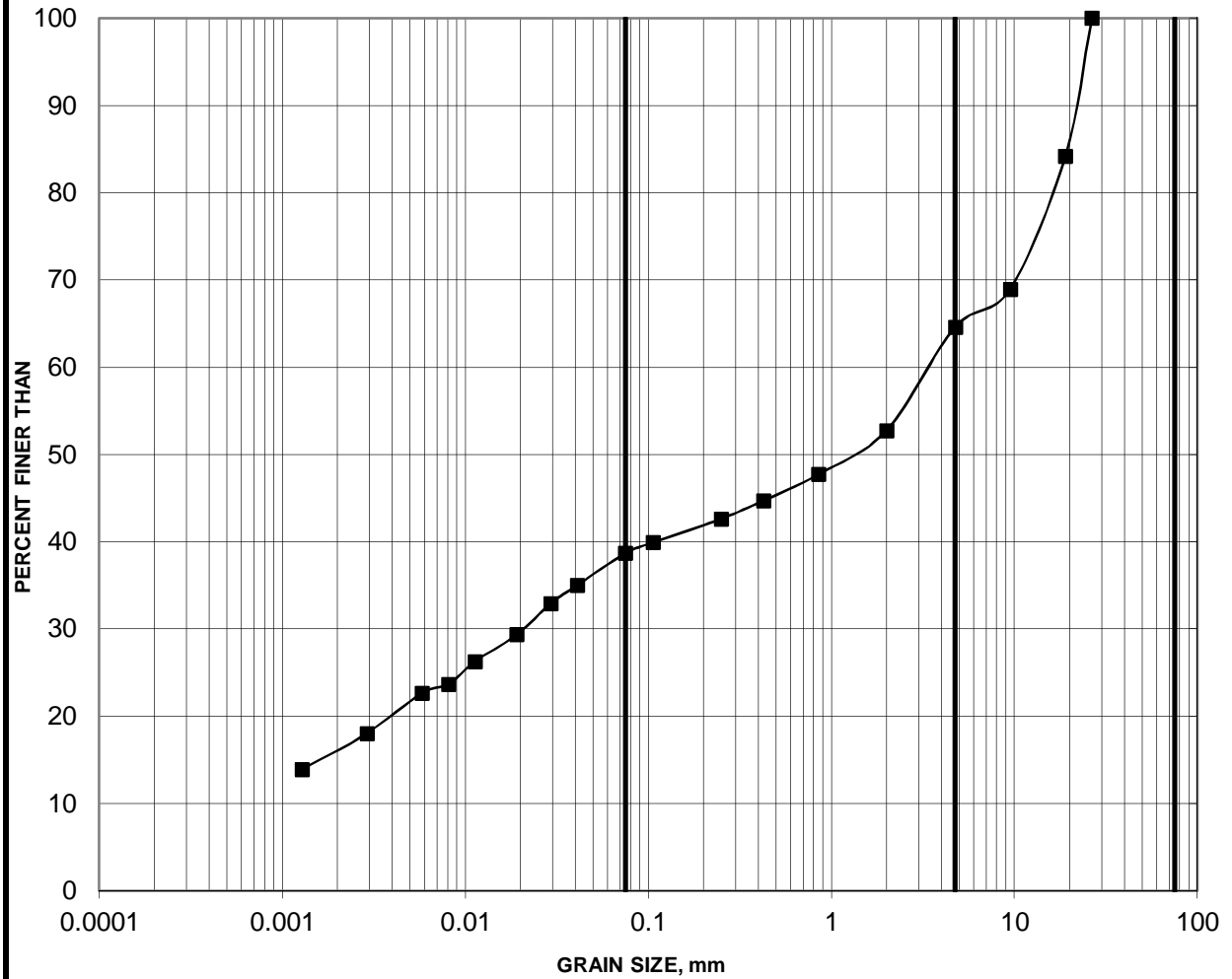


Borehole	Sample	Depth (m)
15-301	3	2.29-2.90
15-301	5	3.81-4.42
15-302	3	2.29-2.90
15-303	1	0.00-0.61
15-305	2	1.52-2.13
15-305	6	4.57-5.18
15-306	2	1.52-2.13

GRAIN SIZE DISTRIBUTION

FIGURE C3

SILTY SANDY GRAVEL (EMBANKMENT FILL)



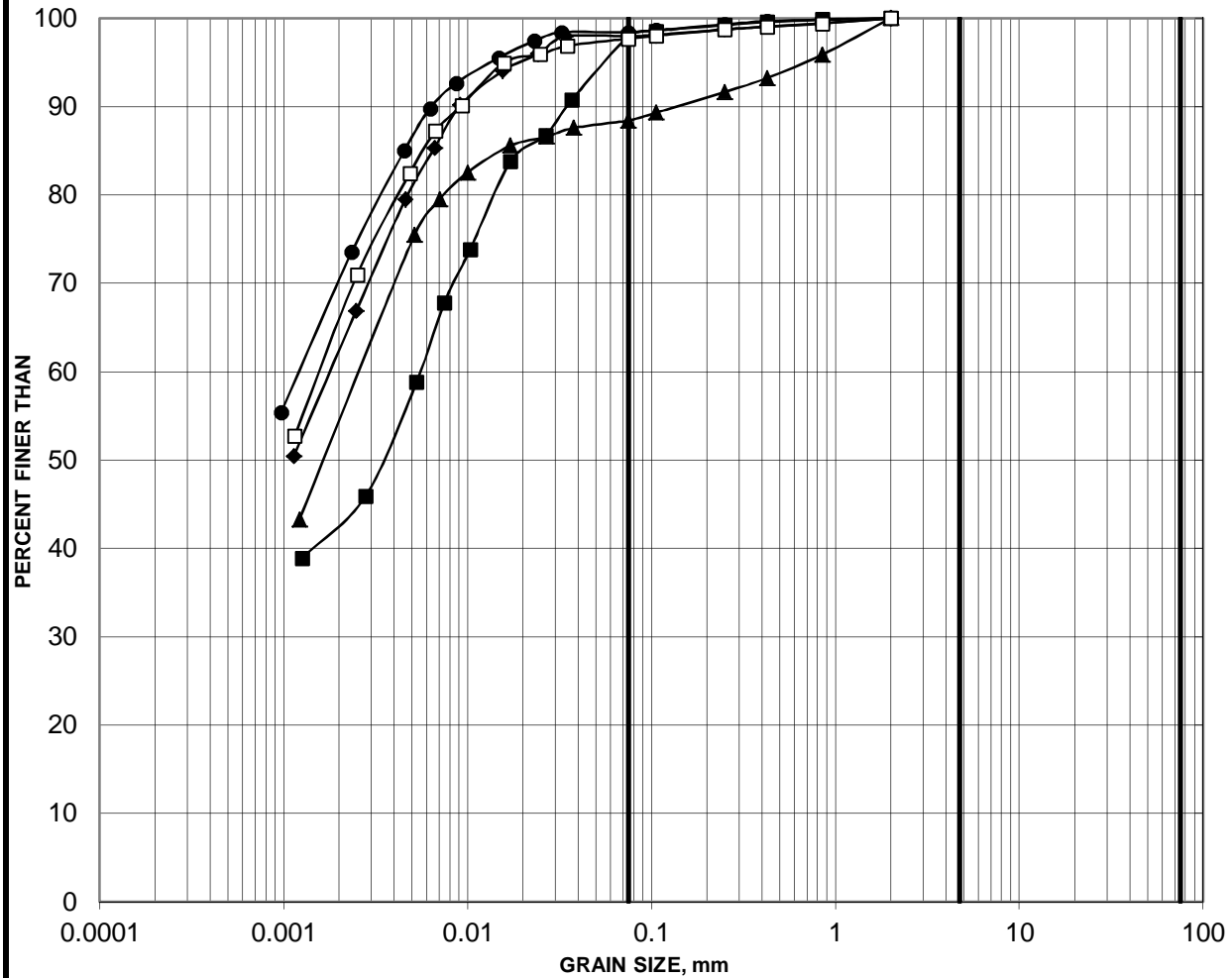
SILT AND CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
	SAND SIZE			GRAVEL SIZE		

Borehole	Sample	Depth (m)
15-302	4	3.05-3.66

GRAIN SIZE DISTRIBUTION

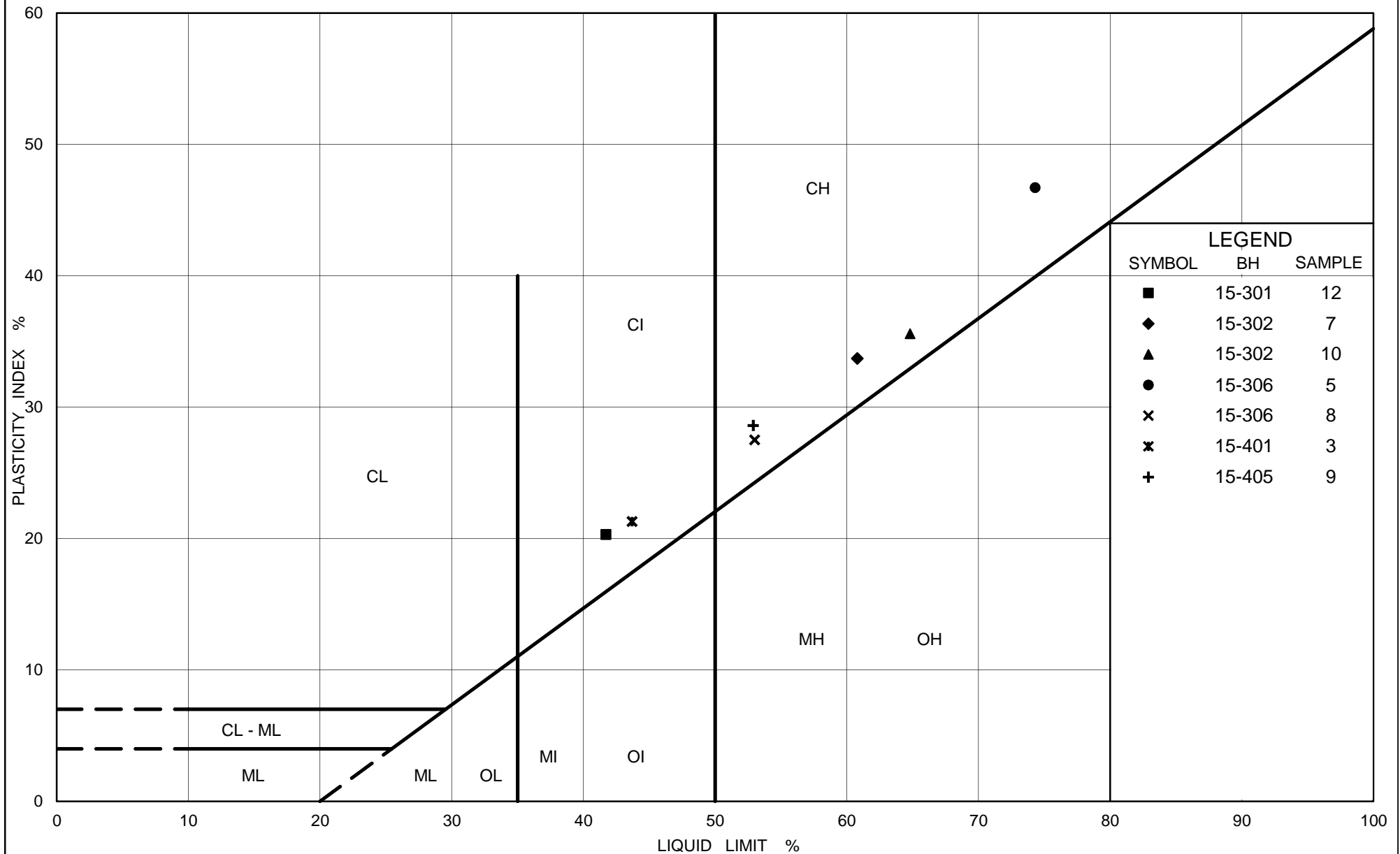
FIGURE C4

SILTY CLAY TO CLAY (EMBANKMENT FILL)



SILT AND CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
	SAND SIZE			GRAVEL SIZE		

Borehole	Sample	Depth (m)
15-301	12	9.15-9.76
15-302	7	5.34-5.95
15-303	3	1.22-1.83
15-306	5	3.81-4.42
15-306	8	6.10-6.71



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PLASTICITY CHART SILTY CLAY TO CLAY (EMBANKMENT FILL)

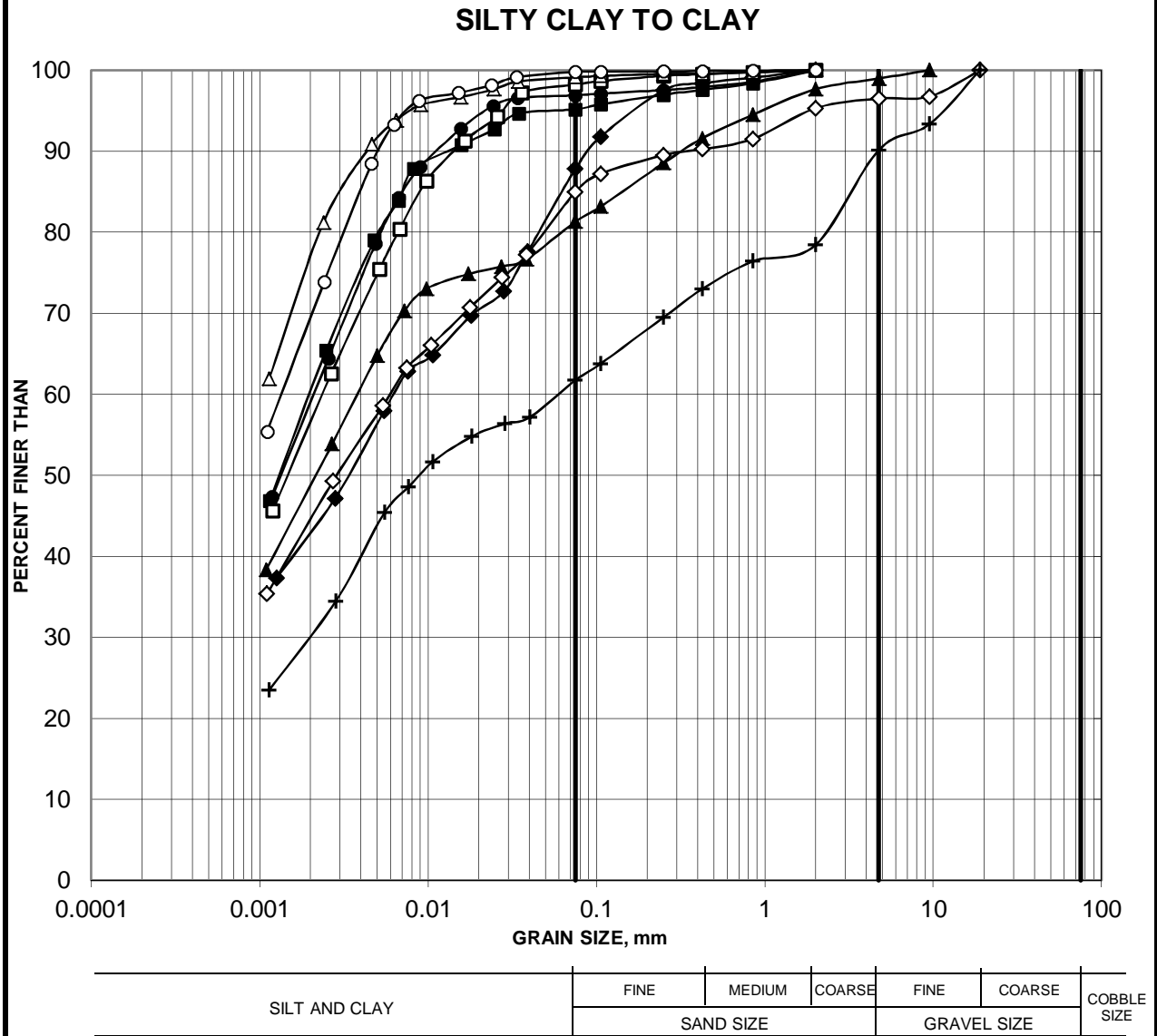
FIG No. C5

Project No. 1403255-001

Compiled By : MI Checked By : CNM

GRAIN SIZE DISTRIBUTION

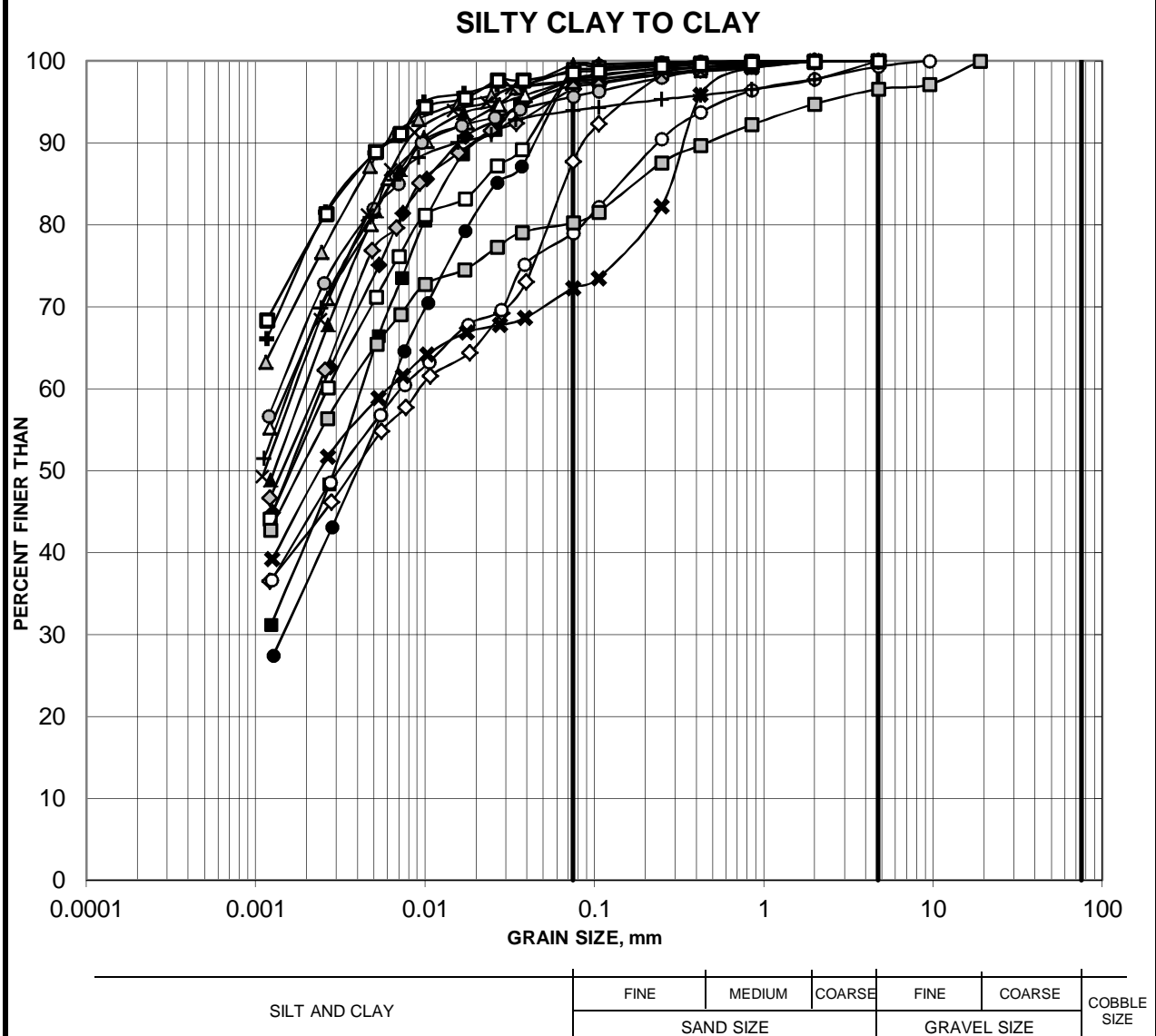
FIGURE C6A



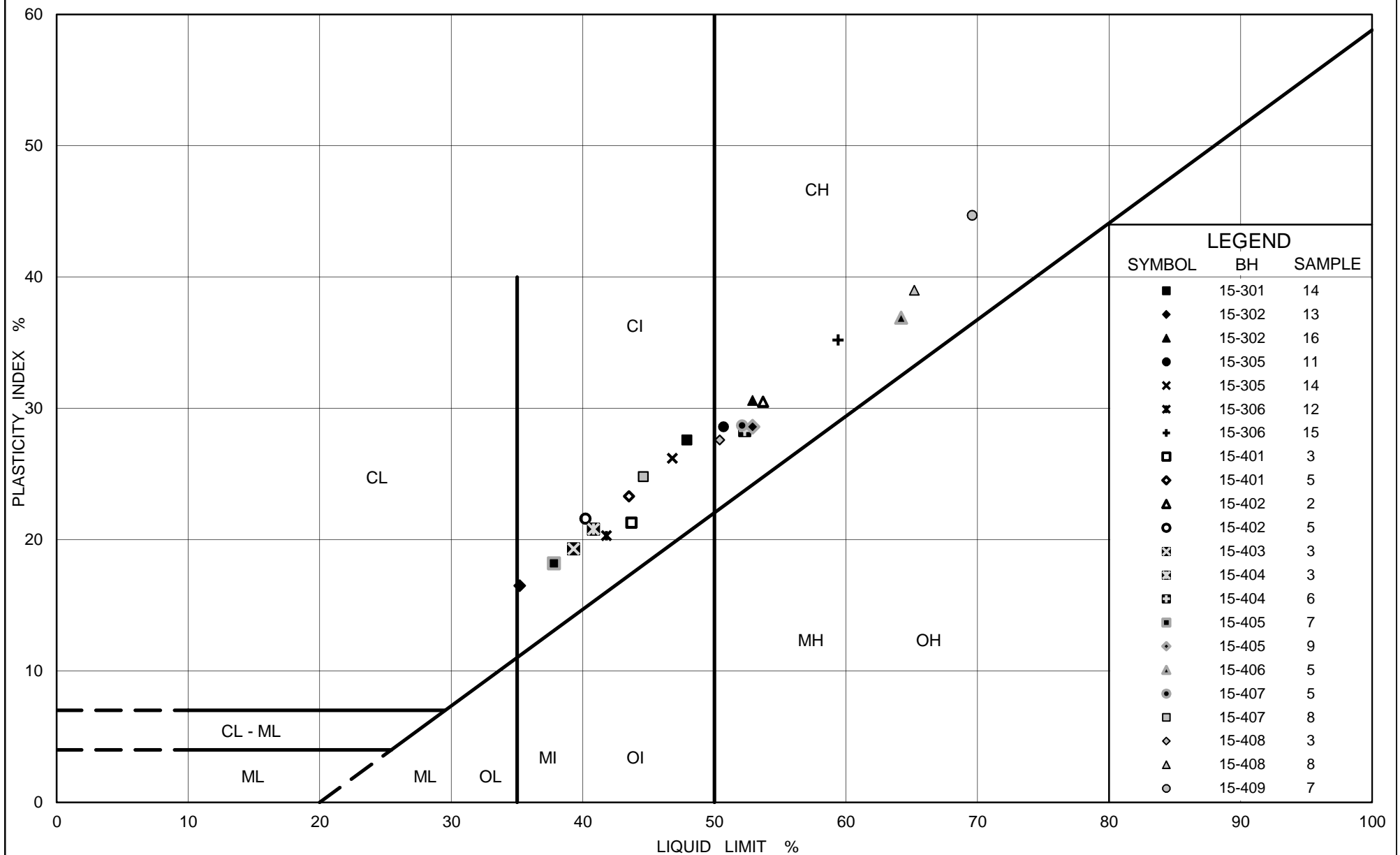
Borehole	Sample	Depth (m)
■ 15-301	14	10.67-11.28
◆ 15-302	13	9.91-10.52
▲ 15-303	4A	2.13-2.44
● 15-305	11	8.38-8.99
□ 15-305	14	10.67-11.28
◇ 15-306	12	9.15-9.76
△ 15-306	15	11.43-12.04
○ 15-306	18	13.72-14.33
+ 15-306	21	16.77-17.04

GRAIN SIZE DISTRIBUTION

FIGURE C6B



Borehole	Sample	Depth (m)
15-401	3	1.22-1.83
15-401	5	2.44-3.05
15-402	2	0.61-1.22
15-402	5	2.43-3.05
15-403	5	2.43-3.05
15-404	3	1.22-1.83
15-405	4	1.83-2.43
15-405	7	3.66-4.27
15-405	9	4.88-5.49
15-406	2	0.61-1.22
15-406	7	3.66-4.27
15-406	10	5.49-6.10
15-407	2	0.61-1.22
15-407	5	2.43-3.05
15-408	6	3.05-3.66
15-409	4	1.83-2.43
15-409	7	3.66-4.27



Ontario

Ministry of Transportation

PLASTICITY CHART SILTY CLAY TO CLAY

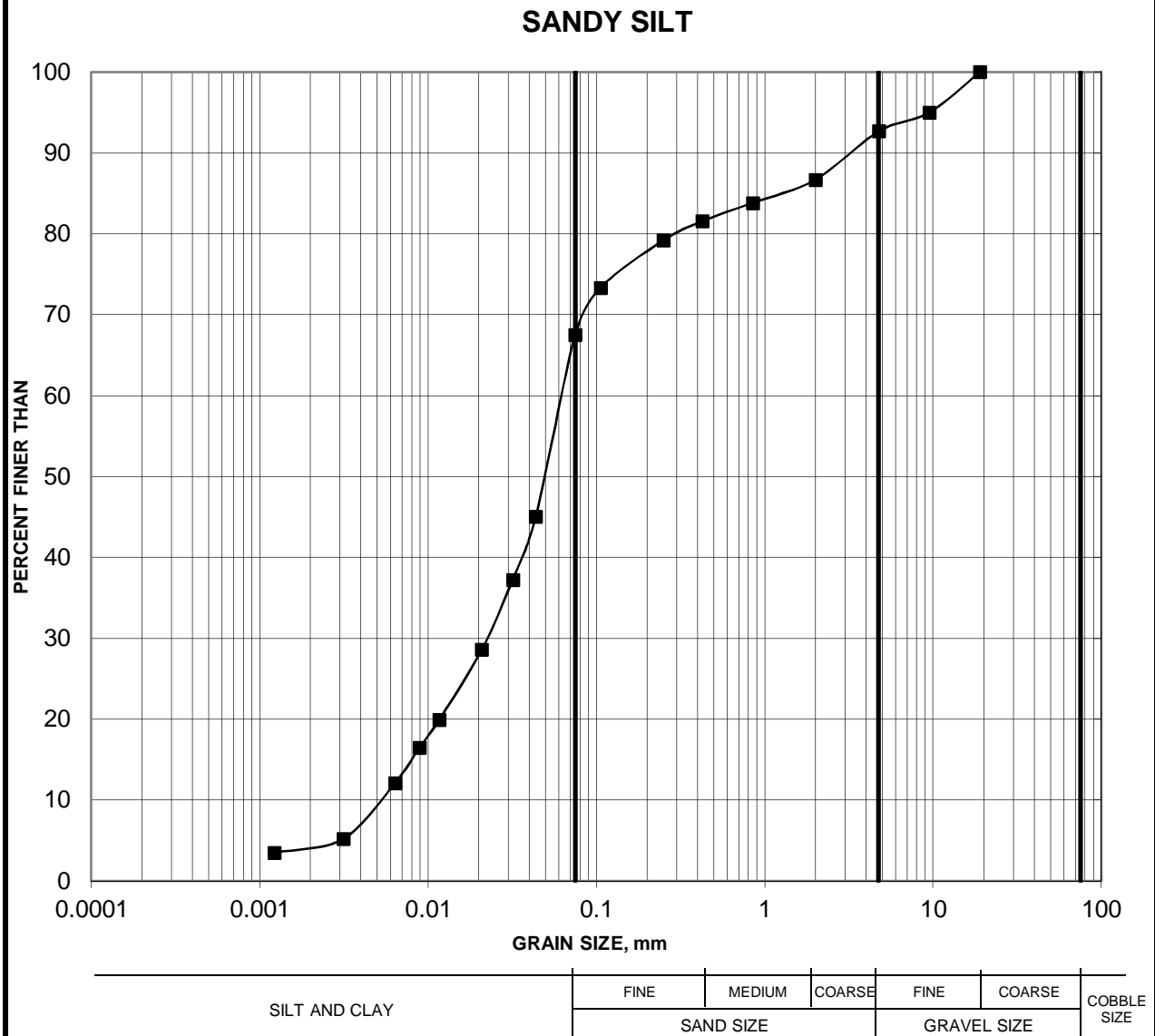
FIG No. C7

Project No. 1403255-001

Compiled By : MI Checked By : CNM

GRAIN SIZE DISTRIBUTION

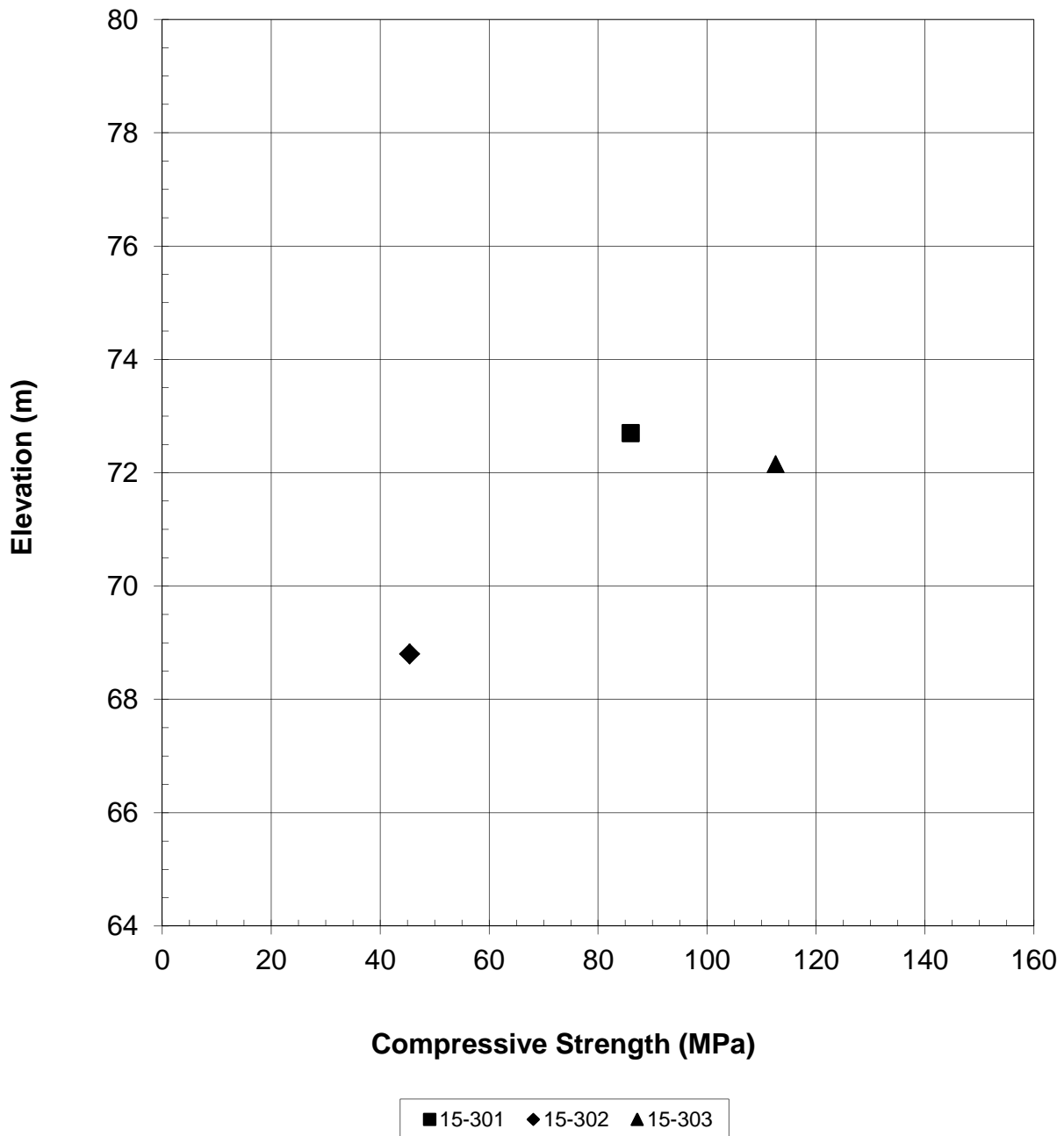
FIGURE C8



Borehole	Sample	Depth (m)
15-302	19	14.48-15.03

**SUMMARY OF LABORATORY COMPRESSIVE STRENGTH
UNCONFINED COMPRESSION TESTS**

FIGURE C9





APPENDIX D

Non-Standard Special Provisions

COFFER DAMS – Item No.

Special Provision

CONSTRUCTION SPECIFICATION FOR COFFER DAMS

1.0 SCOPE

As part of the work under this item, the Contractor shall:

- Design, supply, install, maintain and remove coffer dams as required in order to construct the piers for the Cataraqui River bridge, as shown on the Contract Drawings.
- Dewater the coffer dams as required during construction.

2.0 REFERENCES

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

Ontario Provincial Standard Specifications, Construction

OPSS 518 Control of Water from Dewatering Operations

3.0 DEFINITIONS

Stamped means drawings or details that have been reviewed and stamped “Conforms With Contract Documents”. The stamp shall include the date and signature of the Quality Verification Engineer (QVE).

Quality Verification Engineer (QVE) means an Engineer licensed to practice in the Province of Ontario who has a minimum of five (5) years of experience in the *demolition of structures*. The Quality Verification Engineer shall be retained by the Contractor to ensure conformance with the contract documents and issue certificate(s) of conformance.

Coffer Dam Design Engineer means An Engineer licensed to practice in the Province of Ontario who has a minimum of five (5) years of experience in the field of design and/or construction of bridges. In addition, the Coffer Dam Design Engineer shall have had responsible experience in the design of at least 5 other coffer dams. The Contractor shall retain the Coffer Dam Design Engineer to ensure conformance with the contract documents and issue certificate(s) of conformance for the design.

4.0 DESIGN AND SUBMISSION REQUIREMENTS

4.01 Design

Design of concrete coffer dams shall be in accordance with CAN/CSA-S6-00.

4.02 Submission Requirements

The Contractor shall, at least three (3) weeks prior to the commencement of the coffer dam installation, submit to the QVE for review and stamping, four (4) sets of drawings and calculations indicating the following:

- the coffer dam design;
- the location, type and dimensions of each coffer dam to be used based on the staging of the bridge replacement; and
- a schematic showing the configuration of all coffer dams.

The QVE shall review all calculations, construction details, shop drawings and procedures.

At least two weeks prior to the commencement of coffer dam construction, the Contractor shall submit to the Contract Administrator, for information purposes only, four (4) sets of stamped drawings/calculations of the coffer dam system. All submissions shall bear the seal and signature of the Cofferdam Design Engineer and QVE.

4.02.01 Dewatering

At least two weeks prior to the commencement of cofferdam construction, the Contractor shall submit a detailed unwatering scheme to the Contract Administrator, for information purposes only, showing the unwatering methods and measures to complete the work in dry conditions. The submission shall also provide details of the proposed methods of preventing unwatering or displaced water from directly entering Cataraqui River.

4.02.02 Certificates of Conformance

The Cofferdam Design Engineer shall inspect the installation of each coffer dam. After the installation of the coffer dam has been completed, the Contractor shall submit a Certificate of Conformance to the Contract Administrator, sealed and signed by the Cofferdam Design Engineer. The Certificates of Conformance shall state that the coffer dam is in place, and has been installed in conformance with the stamped shop drawings and the Contract Drawings.

The Contractor will note that several Certificates of Conformance may be required, to coincide with each coffer dam installation.

5.0 MATERIALS – Not Used

6.0 EQUIPMENT - Not Used

7.0 CONSTRUCTION

The Contractor shall ensure stability of the coffer dams during all stages of construction.

The Contractor is alerted that sheet piles or other elements that are to be driven at this site may encounter existing rip-rap or rock fill near the existing ground surface, and appropriate materials, equipment and procedures shall be adopted to facilitate construction and minimize damage to or deflection of the coffer dam elements.

The Contractor shall cut the coffer dam at the limits indicated on the Contract Drawings at the completion of the construction of the pier footings.

7.01 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 Coffers Dam - 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.

OPERATIONAL CONSTRAINT - Preload Period – Embankment Widening Construction

Special Provision

The Contractor shall schedule his operations to include the following preloading times for the northward and southward widening of the existing Highway 401 embankments at the location of the new abutments for the Cataragui River bridge. To allow time for settlement of the embankment widening, the following time constraint shall apply:

- For the east and west approach embankments, extending from the new abutment to 20 m outside of the new abutment, the embankment widening shall be constructed up to the top of the granular sub-base material, and the fills shall remain in place for a minimum period of two months prior to construction of the new abutments.

Prior to placement of the Granular A base material and paving, the Contractor shall conduct a survey to determine the elevations of the top of the Granular B sub-base material, and shall place additional Granular B Type II material as and where required to achieve the pavement design sub-base elevation. The Contractor shall not proceed with final granular placement and paving until approval has been given by the Contract Administrator.

VIBRATION MONITORING - Item No.

Special Provision

CONSTRUCTION SPECIFICATION FOR VIBRATION MONITORING

1.0 SCOPE

This special provision describes requirements for vibration monitoring during deep foundation installation and sheet-pile/protection system installation and/or removal for the Cataraqui River bridge.

2.0 REFERENCES

The subsurface conditions at the structure site are described in the Foundation Investigation Report for the Cataraqui River Bridge replacement.

3.0 DEFINITIONS

Quality Verification Engineer (QVE): An Engineer with a minimum of five (5) years of 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.

4.0 DESIGN AND SUBMISSION REQUIREMENTS

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

- Equipment and methods used by the Contractor to perform the work that may cause vibration.
- Qualifications of vibration monitoring specialist.
- Details regarding proposed vibration monitoring instrumentation.
- Proposed location of instruments on the existing Cataraqui River bridge, and on new portions of the bridge during subsequent stages of construction.
- Proposed frequency of readings.
- Action plan to be taken to adjust deep foundation or protection system installation methods if readings show vibrations exceeding tolerable levels.

5.0 MATERIALS - Not Used

6.0 EQUIPMENT - Not Used

7.0 CONSTRUCTION

7.01 Monitoring

The vibration monitoring equipment shall be placed on the existing Cataraqui River bridge, and subsequently on new portions of the Cataraqui River bridge, for all stages during removals and new construction. The Contractor shall take readings on the structures throughout pile driving or protection

system installation operations, as applicable at these sites, and shall immediately notify the Contract Administrator if the vibrations exceed the limits specified herein.

The vibrations measured on the bridge structures shall not exceed 100 mm/s (peak particle velocity).

If the readings are not within the limits stated above, the Contractor shall alter the deep foundation or protection system installation procedures until the vibrations at the existing bridge structures are within acceptable levels.

8.0 QUALITY ASSURANCE - Not Used

9.0 MEASUREMENT FOR PAYMENT - Not Used

10.0 BASIS OF PAYMENT

10.01 Vibration Monitoring - Item

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

BEDROCK EXCAVATION – Item No.

Special Provision

The granitic bedrock at this site is medium strong to very strong (unconfined compressive strengths in the range of 50 MPa to 200 MPa), and abrasive. Appropriate construction equipment and procedures will be required for foundation construction within the bedrock.

BASIS OF PAYMENT

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

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.

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