



December 2015

REPORT ON

**Foundation Investigation and Design Report
Retaining Wall 7N and Embankment 8N
Merivale Road to Kirkwood Avenue
Highway 417
W.P. 4058-01-00**

Submitted to:
MMM Group
300-1145 Hunt Club Road
Ottawa, Ontario
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REPORT



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Geocres Number: 31G5-269

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Highway 417 W.P. 4058-01-00.

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PART A

**FOUNDATION INVESTIGATION REPORT
RETAINING WALL 7N AND EMBANKMENT 8N
MERIVALE ROAD TO KIRKWOOD AVENUE
HIGHWAY 417
W.P. 4058-01-00**



1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by McCormick Rankin Corporation (MRC) on behalf of the Ministry of Transportation, Ontario (MTO) to carry out foundation investigations associated with the widening of Highway 417 in the City of Ottawa. The section of Highway 417 included in this assignment (W.P. 4058-01-00) extends from Merivale Road to Kirkwood Avenue on the north side of Highway 417.

This report addresses the proposed addition of a 130 metre long retaining wall north of the westbound Kirkwood/Carling off ramp (Retaining Wall 7N – RW7N), 2m to 3 m in height and an approximately 250 metre long high fill embankment (HF8N) north of Highway 417, between Merivale Road and the eastern end of RW 7N (as shown on Drawing 1). The embankments at RW7N and HF8N are being widened to accommodate an additional two lanes of traffic, increasing the number of lanes from 6 to 8, and the widened embankments will range in height, above the current ground level, from about 1.5 to 4.5 metres. The total length along this alignment is approximately 380 metres.

Retaining wall 7N is required at this location to provide access around the City of Ottawa playing fields at this location.

The terms of reference for the original scope of work are outlined in the MTO's Request for Proposal (RFP) dated January 2005. The work was carried out in accordance with Golder's revised foundation investigation proposal (P71-2135) for this project dated September 2, 2009.



2.0 SITE DESCRIPTION

Highway 417, from Merivale Road to Kirkwood Avenue, is typically six lanes wide (i.e., three lanes in each of the westbound and eastbound directions) with additional lanes for ramps. The property north of the highway is City of Ottawa recreational space (baseball diamonds) and National Capital Commission (NCC) parkland.

Between Merivale Road and Kirkwood Avenue, Highway 417 is elevated about 2 to 5 m above the surrounding ground. The embankments are typically sloped at 1.25H:1V or flatter (up to 3H:1V). The pavement structure on the Ramp to Carling Avenue is poor. The embankment slopes are brush covered and appear stable.

No major utilities run parallel to Highway 417 in the area of widening on the north side of the highway. A storm sewer crosses Highway 417, west of Merivale Road, and will be within the zone of influence of the widening. It is understood that this storm sewer, known as the Cave Creek collector, has been tunnelled through the bedrock below about elevation 53 m.



3.0 INVESTIGATION PROCEDURES

The field work for the retaining wall and embankment subsurface investigation was carried out between December 16, 2009 and January 5, 2010. At that time, eight boreholes (Boreholes 09-5 to 09-10, inclusive and including 09-9A and 09-10A) were put down at the locations shown on Drawing 1. Where possible, the boreholes were drilled directly along the retaining wall and embankment alignments at about 75 m spacing. Proposed Borehole 09-10 was not drilled at the planned location but was moved west to avoid conflicts with underground services. Borehole 09-9 was also moved west to provide better spacing between boreholes. Access constraints at the proposed location of retaining wall did not allow for placement of boreholes directly on the alignment. Instead, Borehole 09-6 was put down approximately 6 m north of retaining wall 7N and borehole 09-5 was put down approximately 4 m north of RW7N.

Seven of the boreholes were advanced using a track mounted drill rig supplied and operated by Marathon Drilling Company Ltd. of Ottawa, Ontario. A portable drilling rig supplied and operated by OGS Drilling Inc. of Almonte, Ontario was used to advance borehole 09-5 due to the limited access at that location.

The boreholes were advanced to depths which vary from 4.6 to 15.5 m below the existing ground surface.

Within the boreholes, standard penetration tests were carried out at regular intervals of depth and samples of the soil encountered were recovered using drive open sampling equipment. In situ vane testing using the MTO "N" vane was carried out where possible in the silty clay to measure the undrained shear strength of this soil.

Two relatively undisturbed 75-millimetre diameter thin-walled Shelby tube samples of the silty clay soils encountered were obtained from Boreholes 09-6 and 09-9 using a fixed piston sampler. An additional Shelby tube sample of silty clay was obtained at Borehole 09-10A which was drilled specifically for that purpose. Two of the samples were submitted for laboratory oedometer consolidation testing to assess the consolidation characteristics of the silty clay.

The water levels in the open boreholes were monitored during drilling operations. A standpipe piezometer, consisting of a 2 inch (50 mm) diameter PVC pipe with a 1.5 metre length of #10 slot screen was installed in borehole 09-9A (1 m north of borehole 09-9). Subsequent to the drilling operations, the groundwater level in the standpipe piezometer was measured on February 16, 2010.

Upon completion of drilling, the boreholes were backfilled with bentonite mixed with soil cuttings and the site conditions were restored.

The field work was supervised on a full-time basis by members of Golder's staff who located the boreholes in the field, directed the drilling, sampling, and in situ testing operations, and logged the boreholes. The soil and bedrock samples were identified in the field, placed in labelled containers and transported to Golder Associates' laboratories in Ottawa and Mississauga for further examination and testing. Index and classification tests consisting of water content determinations, Atterberg Limit testing, and grain size distribution analyses were carried out on selected soil samples.

The borehole locations and elevations were surveyed by Golder prior to obtaining utility clearances using a Trimble GPS unit. Where borehole locations were modified, the locations of the relocated boreholes were measured by Golder relative to existing site features and the original surveyed borehole locations. The approximate borehole locations, including MTM NAD83 northing and easting coordinates and ground surface elevations, referenced to Geodetic datum, are summarized in the following table and on Drawing 1.



FOUNDATION INVESTIGATION AND DESIGN REPORT

Retaining Wall 7N

Borehole Number	MTM NAD83 Northing (m)	MTM NAD83 Easting (m)	Ground Surface Elevation (m)
09-5	5027648.0	364476.0	77.7
09-6	5027702.0	364518.7	76.0
06-151	5027735.6	364548.5	75.4

High Fill Embankment 8N

Borehole Number	MTM NAD83 Northing (m)	MTM NAD83 Easting (m)	Ground Surface Elevation (m)
09-7	5027796.0	364593.9	74.9
09-8	5027846.0	364627.4	74.5
09-9	5027883.0	364649.5	74.4
09-9A	5027884.0	364649.5	74.4
09-10	5027922.0	364672.0	74.0
09-10A	5027923.0	364671.7	74.0



4.0 SITE GEOLOGY AND STRATIGRAPHY

4.1 Regional Geological Conditions

The study area for this assignment lies within the minor physiographic region known as the Ottawa Valley Clay Plain, as delineated in *The Physiography of Southern Ontario*¹ within the major physiographic region of the Ottawa-St. Lawrence Lowland.

The Ottawa Valley Clay Plain region is characterized by relatively thick deposits of sensitive marine clay, silt and silty clay that were deposited within the Champlain Sea basin. These deposits, known as the Champlain Sea clay or Leda clay, overlie relatively thin, commonly reworked glacial till and glaciofluvial deposits, that in turn overlie bedrock.² This region is underlain by a series of sedimentary rocks, consisting of sandstones, dolostones, limestones and shales that are, in turn, underlain by igneous and metamorphic bedrock of the Precambrian Shield.

4.2 Site Stratigraphy

As part of the current subsurface investigation at this site, eight boreholes were advanced along the proposed retaining wall and high fill embankment alignment for the proposed highway 417 widening. In addition, borehole 06-151, put down as part of a previous investigation for the Highway 417 widening project, was used to supplement the collected subsurface information. The borehole locations and ground surface elevations as well as the soil stratigraphy sections projected along the retaining wall and high fill embankment alignments are shown on Drawing 1.

The detailed subsurface soil and groundwater conditions encountered in the boreholes and the results of the in situ and laboratory testing are given on the Record of Borehole sheets in Appendix A and on Figures 1 to 4. The stratigraphic boundaries shown on the borehole records are inferred from non-continuous sampling and, therefore, represent transitions between soil types rather than exact planes of geological change. Subsoil conditions will vary between and beyond the borehole locations.

In summary, the soils encountered during the current investigation within the limits of the widening comprise topsoil and fill overlying native soils consisting of silty clay to clay overlying glacial till. Boreholes put down on the south side of this section of highway during previous investigations indicate that the overburden materials are likely underlain by limestone to dolomitic limestone bedrock at depth.

An interpreted summary of the subsurface conditions encountered in the boreholes is provided in the following sections.

4.2.1 Retaining Wall 7N (WBL 424+125 to 424+255 Ramp Stationing)

Boreholes 09-5 and 09-6 were advanced along the alignment of retaining wall 7N as part of this field investigation. Borehole 06-151, put down at the eastern end of the proposed retaining wall during a previous investigation, has also been included in the consideration of the subsurface conditions along the proposed retaining wall alignment. The borehole locations and ground surface elevations as well as the inferred soil stratigraphy section projected along the retaining wall (RW7N) are shown on Drawing 1.

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.

² Belanger, J.R. "Urban Geology of Canada's National Capital Area", in *Urban Geology of Canadian Cities*, Geological Association of Canada Special Paper 42, Ed. P.F. Karrow and O.L. White, 1998.



4.2.1.1 Topsoil and Fill Materials

Topsoil fill, about 100 mm in thickness, was encountered at the ground surface at Borehole 06-151.

Fill was encountered underlying the topsoil at Borehole 06-151 and extending from the ground surface at Boreholes 09-5 and 09-6. The fill ranges from about 1.5 to 3.1 m in thickness and generally consists of sand containing varying amounts of gravel, silt and clay. Measured SPT "N" values ranging between 4 and 25 blows per 0.3 m of penetration indicate a loose to compact state of packing.

4.2.1.2 Silty Clay to Clay

The fill is underlain by a deposit of silty clay to clay. The upper portion of the deposit has been weathered to a grey brown crust and is 1.6 to 2.0 m thick at Boreholes 09-5, 09-6 and 06-151. Measured SPT "N" values ranging between 2 and 13 blows per 0.3 m of penetration indicate that the weathered clay has a stiff to very stiff consistency. The measured natural water content of a sample of the weathered silty clay was 72 percent.

The silty clay to clay below the depth of weathering is grey in colour. The unweathered portion of this deposit extends to a depth between about 8.8 to 9.1 m below the existing ground surface (i.e., elevations 68.5 to 66.6 m). In situ vane testing results in this material indicate that the undrained shear strength ranges between 28 to 61 kilopascals, as indicated on Figure 2. These results indicate that the silty clay to clay has a firm to stiff consistency. At borehole 09-6, an undrained shear strength of 15 kPa was recorded near the silty clay/till interface and this result is not consistent with the results obtained during the current and previous investigations; likely due to disturbance.

The results of Atterberg limit testing on two samples of the silty clay to clay indicate a plasticity index which ranges from 26 to 50 percent and liquid limits ranging from approximately 44 to 82 percent. These results, as shown on Figure 1, indicate that this material is of intermediate to high plasticity. The measured natural water contents of selected samples of the grey silty clay ranged from 48 to 81 percent which is generally near or in excess of the measured liquid limit.

Laboratory oedometer consolidation testing was carried out on one thin-walled Shelby tube sample of the unweathered silty clay. The results of that testing are provided on Figure 3, indicating that the pre-consolidation pressure exceeds the existing overburden pressure by about 97 kPa, and are also summarized in the table below.

Borehole/ Sample No.	Sample Depth (m)	σ_P' (kPa)	σ_{vo}' (kPa)	$\sigma_P' - \sigma_{vo}'$ (kPa)	Cc	Cr	e_0	OCR
09-6/SA5	4.85	170	73	97	2.30	0.015	2.28	2.3

Notes:

σ_P' - Apparent preconsolidation pressure

σ_{vo}' - Computed existing vertical effective stress

Cc - Compression index

Cr - Recompression index

e_0 - Initial void ratio

OCR - Overconsolidation ratio

4.2.1.3 Glacial Till and Sand to Sand and Gravel

The silty clay to clay deposit is underlain by glacial till containing an interbedded deposit of sand and sand and gravel at boreholes 09-6 and 06-151. The glacial till and sandy deposits were proven to depths ranging from 10.4 to 14.5 m below ground surface (i.e., Elevations 67.3 to 61.2 m).

Based on local experience and observations of the drilling resistance, the glacial till consists of a heterogeneous mixture of gravel, cobbles, and boulders in a matrix of silty sand to sandy silt, with a trace of clay.

The measured SPT "N" values within the glacial till ranged from 2 to more than 99 blows per 0.3 m of penetration indicating a very loose to very dense state of packing. However, the higher N values may reflect the presence of cobbles and boulders in the deposit rather than the state of packing of the soil matrix.

The measured natural water content of one sample of the glacial till from Borehole 09-6 was 5 percent.

Sand to sand and gravel deposits, containing trace amounts of silt and clay, were encountered within the till layer at boreholes 09-6 and 06-151 and are about 1.9 and 2.9 m in thickness, respectively. The measured SPT "N" values within the sandy deposits ranged from 20 to 93 blows per 0.3 m of penetration indicating a compact to very dense state of packing.

The measured natural water content of one sample of the sand and gravel from borehole 09-6 was 9 percent.

4.2.1.4 Auger Refusal

Practical refusal to augering was encountered at boreholes 09-6 and 06-151 at elevations 61.5 and 61.2 m, respectively, corresponding to a depth 14.2 to 14.5 m below ground surface. Auger refusal may indicate the bedrock surface; however, it could also represent cobbles and/or boulders within the glacial till.

4.2.1.5 Groundwater Conditions

At the time of the current investigation, water levels were found to be near ground surface along the toe of the existing embankment/off-ramp slope. Measurements taken within open boreholes upon the completion of drilling may not reflect the stabilized water level elevations.

Borehole Number	Existing Ground Surface Elevation (m)	Water Level Depth (m)	Water Level Elevation (m)	Type of Reading	Date of Reading
09-6	76.0	0.2	75.8	open borehole	December 16, 2009
06-151	75.4	4.6	70.8	open borehole	August 3, 2006

It should be expected that the groundwater levels will fluctuate seasonally.

4.2.2 High Fill Embankment 8N (WBL 224+255 to 224+504)

As part of this investigation, boreholes 09-7, 09-8, 09-9, 09-9A, 09-10 and 09-10A were advanced just north of the existing MTO fenceline along the toe of the proposed high fill embankment widening (HF8N) between the Merivale Road overpass and the Kirkwood/Carling westbound off ramp. Borehole 06-151, put down at the western end of the proposed high fill embankment during a previous investigation for a light standard, has also been included in the consideration of the subsurface conditions along the proposed embankment widening. The borehole locations and ground surface elevations as well as the soil stratigraphy section projected along embankment widening 8N are shown on Drawing 1.



4.2.2.1 Topsoil, Fill Material and Organic Silt

A thin layer of topsoil fill, about 100 mm in thickness was encountered extending from the ground surface at Borehole 06-151.

Fill was encountered underlying the topsoil fill at Borehole 06-151 and extending from the ground surface at the remaining boreholes. The fill is of variable composition, consisting of silty sand or sandy silt and containing varying amounts of gravel and organic matter. The fill material ranges in thickness from 0.5 to 1.5 m. Measured SPT "N" values between 7 and 9 blows per 0.3 m of penetration indicate a loose state of packing. A much higher "N" value of 42 blows per 0.3 m was recorded at borehole 09-10, however, this value is likely due to a piece of gravel that jammed in the tip of the split spoon during driving.

4.2.2.2 Silty Clay to Clay

The fill is underlain by a deposit of silty clay to clay. The upper portion of this deposit has been weathered to a grey brown crust and is between 1.7 and 3.0 m in thickness. Measured SPT "N" values ranging between 1 and 13 blows per 0.3 m of penetration indicate that the weathered silty clay has a very stiff to stiff consistency. The measured natural water content of a sample of the weathered silty clay was 53 percent.

The silty clay to clay below the depth of weathering is grey in colour. This unweathered portion of the silty clay deposit extends to a depth between about 8.2 to 8.8 m below the existing ground surface. In situ vane testing carried out within this grey silty clay measured undrained shear strengths ranging from 42 to 81 kPa (as shown on Figure 4), indicating a firm to stiff consistency.

The results of Atterberg limit testing on selected samples of the grey silty clay indicate a plasticity index which ranges from 31 to 50 percent and a liquid limit ranging from approximately 53 to 80 percent. These results, as shown on Figure 1, indicate that this material is of high plasticity. The measured natural water contents of selected samples of the unweathered silty clay range from 49 to 81 percent, which is generally at or near the measured liquid limit.

Laboratory oedometer consolidation testing was carried out on a thin-walled Shelby tube sample of the grey silty clay. The results of that testing are provided on Figure 5, indicating that the pre-consolidation pressure exceeds the existing overburden pressure by about 107 kPa, and are summarized in the table below.

Borehole/ Sample No.	Sample Depth (m)	σ_p' (kPa)	σ_{vo}' (kPa)	$\sigma_p' - \sigma_{vo}'$ (kPa)	Cc	Cr	e_0	OCR
09-10A/SA5	4.25	145	38	107	1.29	0.003	2.02	4.3

Notes:

σ_p'	-	Apparent preconsolidation pressure	σ_{vo}'	-	Computed existing vertical effective stress
Cc	-	Compression index	Cr	-	Recompression index
e_0	-	Initial void ratio	OCR	-	Overconsolidation ratio



4.2.2.3 Glacial Till with Interbedded Sand or Sand and Gravel

The silty clay to clay deposit is underlain by glacial till which was proven to depths ranging from about 9.1 to 15.5 m below the existing ground surface at all of the boreholes (with the exception of 09-9A and 09-10A) put down within the embankment widening footprint.

Based on local experience and observations of the drilling resistance, the glacial till consists of a heterogeneous mixture of gravel, cobbles, and boulders in a matrix of silty sand to sandy silt, with a trace of clay.

The measured SPT "N" values within the glacial till ranged from 2 to more than 100 blows per 0.3 m of penetration indicating a very loose to very dense state of packing. However, the higher "N" values may reflect the presence of cobbles and boulders in the deposit rather than the state of packing of the soil matrix.

Sand to sand and gravel deposits, containing trace amounts of silt and clay, were encountered within the till layer at boreholes 09-7, 09-9 and 06-151 and underlying the till at borehole 09-8. These predominantly granular deposits range from about 1.5 to 2.9 m in thickness. The sand deposit underlying the glacial till at Borehole 09-8 was not penetrated but was proven to a depth of about 12.6 m. The measured SPT "N" values within the granular deposits ranged from 31 to 93 blows per 0.3 m of penetration indicating a dense to very dense state of packing.

The results of grain size distribution testing of one sample of the sand and gravel deposit from borehole 09-7 are provided on Figure 6.

4.2.2.4 Auger Refusal

Practical refusal to augering was encountered at boreholes 09-7, 09-08, 09-9 and 09-10 at depths ranging from 9.1 to 15.5 m below the existing ground surface (i.e., Elevations 64.9 to 59.5 m). Auger refusal may indicate the bedrock surface; however, it could also represent cobbles and/or boulders within the glacial till.

4.2.2.5 Groundwater Conditions

At the time of the current investigation, water levels were found to be near ground surface along the toe of the existing embankment slope. A standpipe piezometer was installed in Borehole 09-9, sealed within the silty clay to clay deposit. The water level measured in that monitoring well is summarized in the table below. In addition, measurements taken within open boreholes upon the completion of drilling are also shown, although these may not reflect the stabilized water level elevations.

Borehole Number	Existing Ground Surface Elevation (m)	Water Level Depth (m)	Water Level Elevation (m)	Type of Reading	Date of Reading
06-151	75.4	4.6	70.8	open borehole	August 3, 2006
09-7	74.9	0.6	74.3	open borehole	December 18, 2009
09-8	74.5	0.6	73.9	open borehole	December 21, 2009
09-9A	74.4	0.7	73.7	monitoring well	February 16, 2010
09-10	74.0	Not recorded			

Groundwater levels should be expected to fluctuate seasonally.



5.0 CLOSURE

The investigation was carried out using equipment supplied and operated by Marathon Drilling and OGS Drilling. The field portions were supervised by Ms. Jacqueline Cormier and Mr. Jason Derouin under the direction of Mr. William Cavers, P.Eng. The testing was carried out in the Ottawa and Mississauga laboratories of Golder Associates. The report was prepared by Mr. William Cavers, P. Eng. under the direction of Mr. Fintan J. Heffernan P.Eng, the designated MTO contact for this project.

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Designated MTO Foundations Contact



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FOUNDATION INVESTIGATION AND DESIGN REPORT

PART B

FOUNDATION DESIGN REPORT RETAINING WALL 7N AND EMBANKMENT 8N MERIVALE ROAD TO KIRKWOOD AVENUE HIGHWAY 417 W.P. 4058-01-00

6.0 ENGINEERING RECOMMENDATIONS

6.1 General

This section of the report provides foundation design recommendations for the proposed retaining wall and high fill embankment widening along Highway 417 between Merivale Road and the westbound lane off ramp to Carling Avenue in Ottawa, Ontario. The recommendations are based on an interpretation of the factual data obtained from the boreholes advanced during the subsurface investigation at this site. The interpretation and recommendations provided are intended only to provide the designers with sufficient information to assess the feasible retaining wall and embankment options and to design the proposed retaining walls and high fill slopes. As such, where comments are made on construction they are provided only in order to highlight those aspects which could affect the design of the project. Those requiring information on aspects of construction should make their own interpretation of the factual information provided as it may affect equipment selection, proposed construction methods, scheduling and the like.

6.2 Retaining Wall 7N Options

The existing embankments along the alignment of retaining wall 7N are between 2.5 and 4 m in height, with side slopes ranging from 1.25H:1V to 3H:1V. The proposed widening would extend the crest of the existing roadway some 3 to 7 m north of its current location. The footprint for the proposed widening in this area is limited by the properties to the north and sloped embankments have therefore not been considered. To accommodate the 3 to 7 m of widening, 2 to 3 m high retaining walls are required. The following technically feasible options for the proposed widening in the area of retaining wall 7N have been considered:

- "Perched" reinforced soil system (RSS) walls founded on the existing embankment fill;
- Cantilevered reinforced concrete retaining wall founded on shallow or deep foundations;
- Permanent soldier pile and lagging wall (possibly with tie-backs);
- Pre-cast panel wall supported on shallow caissons;
- Lightweight fills in combination with the above wall configurations; and,
- Preloading (with or without surcharge) in combination with the above wall configurations.

The preferred retaining wall option will depend on property constraints, ramp closure constraints, appearance, performance and cost considerations. The main performance consideration for retaining wall 7N (RW7N) is the magnitude of the total and differential post-construction settlement that will be induced by the placement of additional fill along the existing embankment slope and the ability for the foundation solution to cope with these settlements. The additional fill placed along this section of the widening may result in post construction total settlements in the order of 15 to 30 mm due to primary consolidation of the underlying clay. Primary settlement will be almost entirely differential with respect to the existing embankment which has been in place for more than 40 years. Approximately 30 mm of additional secondary creep can also be expected over the next 50 years.

The settlement magnitudes will depend on the type of wall selected. A soldier pile and lagging wall, Pre-cast panel wall, RSS wall, or concrete retaining wall founded on shallow foundations will result in the largest magnitude of settlement since the full weight of additional fill will be supported by the underlying soil.



Lightweight fills could be considered to reduce the settlements, however, if this option is used, it may not be feasible to install the reinforcing strips for a RSS wall within the expanded polystyrene fill. Alternatively, the area of the retaining wall could be preloaded for a period of 3 to 6 months to reduce the magnitude of post construction settlements caused by primary and secondary consolidation. If a pile supported reinforced concrete wall is constructed, a significant portion of the weight of the additional fill could be supported by the pile caps and this will in return reduce the loading imposed on the underlying soils. This reduced loading would result in total settlement magnitudes of less than about 20 millimetres along most of the wall.

The following outlines each of the technically feasible foundation options for the retaining wall:

- 1) “Perched” Reinforced Soil System walls: Mechanically reinforced or retained soil system (RSS) vertical walls located above the toe of the existing slope (i.e. RSS walls located within the existing slope and typically lower than the full embankment height) and founded on the existing embankment fill are geotechnically feasible and could be considered for RW7N. Vertical slip joints (typically spaced at 10 to 20 metres) can be used to accommodate differential settlement of RSS walls; however, the differential settlement along each section between slip joints should not exceed 35 millimeters. Due to the limited anticipated post-construction settlement, it is anticipated that the differential settlements could be accommodated between slip joints.
- 2) Concrete retaining walls on shallow foundations: This type of wall and foundation can be considered where the total post-construction settlement will be less than approximately 25 mm and the differential settlement will be less than about 10 to 15 mm. As indicated previously, the post construction settlement magnitude resulting from primary consolidation is expected to be in the order of 15 to 30 mm and long term secondary compression settlement will result in about 30 mm of settlement over the next 50 years. In addition to these time-dependent settlements, elastic settlement during construction is expected to be in the order of 10 to 20 millimetres. The total settlement of the concrete retaining walls would therefore be in the order of 25 to 50 mm resulting from primary consolidation and elastic settlement and an additional 30 mm secondary settlement over the next 50 years. This type of foundation is therefore not considered suitable unless it is constructed in combination with the use of light weight fill or preloading.
- 3) Concrete retaining walls on deep foundations: This type of wall and foundation could be considered where the total post-construction settlements are greater than approximately 25 mm and is considered technically feasible for this site where refusal soils exist at relatively shallow depth.
- 4) Cantilevered soldier pile and concrete lagging walls: The soldier pile and concrete lagging system can be installed as part of the embankment widening, without significant temporary protection for the highway (as might be required for the installation of shallow foundations, pile caps or reinforced soil masses). This type of wall is considered feasible for RW7N.
- 5) Walls supported on retaining panels on shallow caissons founded at about 2 to 3 few metres depth. At the location of retaining wall 7N, this would result in caissons founded at or very near to the surface of the compressible silty clay. The limited bearing resistance available from that soil unit may not be sufficient to support the wall loads and this option is not considered to be technically feasible.
- 6) Lightweight fill materials: The amount of post-construction settlement and the associated roadway maintenance may be reduced by using lightweight fill materials. Lightweight fill could be used in place of conventional earth fill to reduce the applied loading to below the pre-consolidation pressure range and



could be used to reduce the loading sufficiently to allow the use of shallow foundations. This is a relatively costly option, depending on the type of lightweight fill required. Cellular concrete is technically challenging and not economical for this short wall.

- 7) Preloading: The amount of post-construction settlement and associated roadway maintenance may be significantly reduced by preloading, and possibly surcharging, the compressible soils below the widening footprint. This may be carried out by constructing the retaining wall and placing the backfill well in advance of paving for retaining walls with foundations not susceptible to settlement damage (i.e., such as a soldier pile and lagging retaining wall). For other foundation systems, temporary slopes could be constructed to temporarily load the widening footprint, after which the preload would be removed and the final wall could be constructed on the existing embankment fills. However, it would be relatively costly to construct temporary embankments for preloading the embankment area prior to construction of the retaining wall.

A summary comparison of the advantages, disadvantages, relative costs, and risks associated with the retaining wall options is presented in Table 1 following the text of this report.

Based on the site constraints, relative costs and estimated settlement at the site due to the widened embankment loading, the preferred foundation option for RW7N is a perched reinforced soil system wall. The next preferred option may be a soldier pile and lagging wall.

Geotechnical recommendations for the design of foundations for the retaining wall are presented in the following sections.

6.3 Retaining Wall Foundations – RW 7N

The following options have been considered for cantilevered reinforced concrete retaining wall foundations:

- Shallow foundations supported on the existing embankment fill;
- Foundations supported on steel H-piles founded on the bedrock; and,
- Foundations supported on caissons founded on the bedrock.

6.3.1 Shallow Foundations

Shallow foundations supported on the existing embankment fill are not considered practical or appropriate for this site since the bearing resistance of these embankment fills would be insufficient for support of the retaining wall using standard backfill and the total and differential settlement of the foundations during and after construction would be greater than the normally accepted values of 25 mm total and 15 mm differential settlement.

If lightweight fill were used to reduce loads and thereby reduce post construction settlements, the retaining wall could be founded on the surface of the stiff to very stiff grey brown native clayey soils or on a pad of engineered fill placed on the grey brown native clayey soils after removal of the existing fill materials within the foundation footprint.



6.3.1.1 Limit States Factored Geotechnical Resistance

The Ultimate Limit States ULS geotechnical resistances provided in the following sections are given under the assumption that the loads will be applied perpendicular to the surface of the footings. Where the load is not applied perpendicular to the surface of the footing, inclination of the load should be taken into account in accordance with the *Canadian Highway Bridge Design Code (CHBDC)*.

The existing fill and organic material should be removed from the full zone of influence of the foundations which is considered to extend down and out from the edge of the foundations at a slope of 1H:1V. The engineered fill should similarly be placed within the full zone of influence of the foundations. The engineered fill should consist of Granular A or Granular B Type II placed in maximum 300 mm thick lifts and compacted to at least 95% of its standard Proctor maximum dry density in accordance with MTO's Special Provision OPSS501.

Based on the existing grades at about Elevations 78 to 75 m (from west to east) and the requirement for a minimum 1.8 m of frost cover, the footings will need to be founded at about Elevations 75.9 to 73.6 m (from west to east).

A factored geotechnical resistance at ULS of 250 kPa and a geotechnical resistance at SLS of 150 kPa may be used for the design of concrete retaining walls founded on native clayey soil or engineered fill.

6.3.1.2 Resistance to Lateral Loads

Resistance to lateral forces for retaining wall footings founded on engineered fill should be calculated in accordance with Section 6.7.5 of the *CHBDC*. Where retaining walls will be supported on engineered fill, an unfactored $\tan \delta^*$ lateral sliding resistance value of 0.65 may be used at the base of footing – engineered fill interface based on two-thirds of the soil friction angle.

The resistances obtained using the provided parameters represent unfactored values; in accordance with the *CHBDC*, a resistance factor of 0.8 is to be applied in calculating the horizontal resistance.

6.3.1.3 Frost Protection

Shallow foundations should be provided with a minimum of 1.8 m of soil cover for frost protection.

6.3.1.4 Perched "RSS" walls

RSS walls should be founded on a minimum 0.3 metre thick pad of engineered fill consisting of OPSS.PROV Granular A or Granular B Type II placed in maximum 300 mm thick lifts and compacted to at least 95% of its standard Proctor maximum dry density in accordance with MTO's Special Provision SP105S10. The RSS wall supplier should confirm the adequacy of the recommended engineered fill pad prior to construction.

The RSS wall should meet high appearance and high performance.

RSS walls may require a pad of compacted engineered fill thicker than the minimum 0.3 metres specified above (typically to replace random fill or organic materials underlying the wall foundation). The fill should be placed to extend down and out from the front edge of the RSS wall at a slope of 1H:1V. The engineered fill should extend back behind the face of the wall a minimum distance of half the wall height. The new embankment fills should be benched into the existing embankment in accordance with OPSS 208.010.



Based on the above, the engineered fill pad should be placed on the surface of the existing fill at about Elevations 77 to 75 m, from west to east.

For slope stability, "perched RSS walls" will need to be embedded within the existing fill materials (i.e., the reinforcing strips extend below the grade outside the wall). Typically, the slopes should be sloped at 2H:1V or flatter. Toe slopes below retaining walls steeper than 2H:1V are susceptible to erosion and may require additional protective measures (i.e., such as a permanent turf reinforcement mat). The surface of the existing fill should be heavily proof-rolled prior to the placement of engineered fill.

A factored geotechnical resistance at ULS of 200 kPa and a geotechnical resistance at SLS of 125 kPa may be used for the design of perched RSS walls.

6.3.2 Pile Foundations

Steel H-piles driven to found on the bedrock may be used for support of RW7N.

For this site, the piles would essentially be driven to practical refusal on the bedrock. Along the retaining wall, bedrock is anticipated to be at an elevation of 61 to 62 m. Assuming that the underside of the pile cap will be at 1.8 m below the base of the retaining wall toe for frost protection purposes, the anticipated average pile length is in the order of 13 m.

All pile driving should be in accordance with OPSS.PROV 903. The drawings should incorporate the appropriate note stating that the piles should be equipped with suitable driving points (such as Titus standard rock bearing point or equivalent) and should be driven to bedrock. The pile termination or set criteria will be dependent on the pile driving hammer type, helmet, selected pile and length of pile. All of these factors must be taken into consideration in establishing the driving criteria to ensure that the piles are not overdriven and to avoid possible damage to the piles. In this regard, it is a generally accepted practice to reduce the hammer energy after abrupt peaking is met on the bedrock surface, and then to gradually increase the energy over a series of blows to seat the pile.

Consideration must also be given to the presence of cobbles and boulders within the glacial till and the sand and sand and gravel which exists at the location of RW7N. The piles should be designed to be founded on bedrock. However some of the piles could have difficulty penetrating to depth and could "hang up" at shallower depth in the glacial till. In that case pre-drilling of the overburden could be considered. Alternatively, a reduced capacity may apply to these piles, as discussed below. It is however expected that most of the piles will penetrate the glacial till and reach bedrock. A NSSP for driving of piles within till containing cobbles and boulders has been included in Appendix C.

To minimize the risk of damage to pile tips, vertical and battered driven piles should be equipped with suitable driving points (such as the Titus standard rock bearing point or equivalent) to ensure adequate seating of the piles on the bedrock.

6.3.2.1 Axial Geotechnical Resistance

A factored axial resistance at Ultimate Limit States (ULS) of 2,000 kN may be assumed for design of HP 310 x 110 piles driven to found on the bedrock which is believed to exist at or near the refusal level found in the boreholes. This value represents a structural limitation for the pile rather than a geotechnical limitation. The geotechnical resistance at Serviceability Limit States (SLS) for 25 mm of settlement will be greater than the



factored axial resistance at ULS, since the bedrock is considered to be an unyielding material; as such, ULS conditions will govern for this foundation type. For piles that meet refusal in the dense till or sands, a factored axial resistance at ultimate limit states (ULS) of 1,400 kN may be used in design.

An unfactored ULS uplift resistance of 240 kN may be assumed for HP 310 x 110 piles driven to found on the bedrock at this location; in accordance with the CHBDC, a resistance factor of 0.3 is to be applied.

6.3.2.2 Downdrag Load (Negative Skin Friction)

The construction of the widened embankment will raise the effective stress level in the unweathered grey silty clay to clay, leading to some consolidation of the deposit. As discussed subsequently in Section 6.5 of this report, the retaining wall subgrade settlements are estimated to be up to 60 mm (from both primary consolidation and secondary compression). The elastic shortening of the piles will be significantly less, at about 5 mm under service loads, and therefore the differential settlements would be sufficient to generate downdrag forces.

In calculating the magnitude of the downdrag force, both the methods described in the Canadian Foundation Engineering Manual (i.e., the methods attributed to Terzaghi and Peck, and Poulos and Davis in that manual) were considered. Considering the larger predicted settlement of the silty clay deposit versus the elastic shortening of the pile, the neutral plane used in those analyses was assumed to be at the underside of the compressible silty clay deposit.

Based on the above, the unfactored downdrag load acting on a single HP 310 x 110 pile over the length of pile within the native soils is estimated to be 110 kN. 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 CHBDC.

6.3.2.3 Resistance to Lateral Loads

Lateral loading can be resisted fully or partially by the use of battered piles. Alternatively, the resistance to lateral loading could be derived by socketing the piles into bedrock or from the soils in front of the piles.

For piles socketed at least 1 m into bedrock, the unfactored ULS lateral bearing resistance of the limestone may be taken as the lesser of 30 MPa or the compressive strength of the Portland cement grout or concrete placed in the bedrock socket (since the strength of the concrete or grout governs the lateral capacity of a pile socketed into a higher strength rock). It should be noted that the limestone / dolomitic limestone bedrock is generally medium strong and this would require socket formation using coring or churn drilling to advance the hole.

In accordance with Section C6.8.7.2 of the *Commentary to the CHBDC*, it may be assumed that the resistance to lateral loading derived from the soil will be nearly the same for vertical and inclined piles.

The SLS resistance to lateral loading in front of the piles may be calculated using subgrade reaction theory where the coefficient of horizontal subgrade reaction, k_h , is based on the equations given below, as described by Terzaghi (1955) and the Canadian Foundation Engineering Manual (3rd Edition).



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For cohesionless soils:

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

where: k_h is the coefficient of horizontal subgrade reaction (kN/m³);
 n_h is the constant of horizontal subgrade reaction;
 z is the depth (m) below ground surface; and,
 B is the pile diameter/width (m).

For cohesive soils:

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

where: k_h is the coefficient of horizontal subgrade reaction (kN/m³);
 s_u is the undrained shear strength of the soil (kPa); and
 B is the pile diameter/width (m)

The following ranges for the values of n_h and s_u may be assumed in the structural analysis for lateral resistance. The range in values reflects the variability in the subsurface conditions.

Soil Unit	n_h (MN/m ³)	s_u (kPa)
Compacted engineered fill	18	–
Existing variable fills	1.8	–
Weathered silty clay crust above about Elev. 72 m – approximately 1.5 to 2 m thick (see Record of Borehole sheets)	–	60
Unweathered silty clay to clay extending to about Elev. 66.5 to 68.5 m (see Record of Borehole sheets)	–	35
Glacial till and sand and gravel below about Elev. 66.5 to 68.5 m. (see Record of Borehole sheets)	8	–

Group action for lateral loading should be considered when the pile spacing in the direction of the loading is less than six to eight pile diameters. Group action can be evaluated by reducing the coefficient of lateral subgrade reaction in the direction of loading by a reduction factor as follows (Davisson, 1970):

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

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



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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*.

For individual piles in non-cohesive soils (i.e., new and existing fills), the unfactored ULS lateral passive resistance may be estimated by calculating the passive earth pressure over an equivalent wall area having a depth from the ground surface equal to six times the pile diameter, and with a width equal to three times the pile diameter.

In cohesive soils (i.e., silty clay and clay) the passive ULS lateral resistance is assumed to vary linearly from a value of $2S_u$ (i.e., where S_u is the undrained shear strength) at the surface of the deposit to a value of $9S_u$ at a depth equal to three pile diameters below the underside of the pile cap. Below a depth equal to 3 pile diameters, the passive ULS lateral resistance is assumed to be constant at a value of $9S_u$.

The ULS lateral passive resistance from the glacial till should be neglected since, in these non-cohesive soils, the *CHBDC Commentary* (Section C6.8.7.1) suggests that resistances only be considered within a depth equal to six diameters below the underside of the pilecap; these soils are below that depth.

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 following values for K_p and S_u may be assumed for estimating the ULS geotechnical lateral resistances:

Soil Unit	γ (kN/m ³)	K_p	S_u (kPa)
Compacted engineered fill	21	3.7	–
Existing variable fills	19	3.0	
Weathered silty clay crust above about Elev. 72 m – approximately 1.5 to 2 m thick (see Record of Borehole sheets)	–	–	60
Unweathered silty clay to clay extending to about Elev. 66.5 to 68.5 m (see Record of Borehole sheets)	–	–	35
Glacial till and sand and gravel below about Elev. 66.5 to 68.5 m. (see Record of Borehole sheets)	20.0	3.4	–

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.

6.3.2.4 Frost Protection

Pile caps should be provided with a minimum of 1.8 m of soil cover for frost protection.

6.3.3 Caisson Foundations

Caissons founded on or socketed into the bedrock may be used for support of the retaining wall; however the depth to bedrock may render this option cost-prohibitive for a lightly loaded structure. The elevation of the bedrock surface beneath the retaining wall ranges from 61 to 62 m. Assuming that the underside of the pile cap will be at 1.8 m below the base of the retaining wall toe for frost protection purposes, the anticipated average caisson length is about 13 metres.

There is a risk that the native silty clay, sands, and sandy till within the project limits will “flow” into the auger hole during caisson installation if left unsupported. As such, the use of a temporary liner or casing will be required to advance the caissons with minimal loss of ground. Additionally, these soils will be difficult to clean from the



bedrock surface, even with the use of liners, unless the liner is socketed into the bedrock. It is therefore recommended to socket the caissons into the rock, rather than found them on the bedrock surface.

The limestone and dolomitic limestone bedrock within the project limits is moderately strong. If socketing of the caissons into the bedrock is required, the sockets will have to be advanced by rock coring or churn drilling.

6.3.3.1 Axial Geotechnical Resistance

Caissons founded on the surface of the bedrock, or socketed nominally (less than 1 m) into the bedrock, should be designed based on end-bearing resistance and a factored geotechnical resistance at ULS of 4 MPa. Serviceability Limit States resistances do not apply to caissons founded on or socketed in the bedrock, since the SLS resistance for 25 mm of settlement is greater than the factored axial geotechnical resistance at ULS.

The bedrock was not proven and, if caissons are to be considered for this low height wall, additional investigations to confirm the above geotechnical resistance would be required.

6.3.3.2 Downdrag Load (Negative Skin Friction)

The construction of the widened embankment will raise the effective stress level in the unweathered silty clay to clay, leading to some consolidation of the deposit and will result in downdrag forces on caissons supporting the retaining walls. The unfactored downdrag load acting on a single 0.9 m diameter caisson over its length is estimated to be 240 kN. The structural capacity of the caissons must be checked for the factored dead and downdrag loads in accordance with Section 6.8.4 of the *CHBDC*. The assumptions and methods used in assessing that downdrag force are the same as those described previously in this report with respect to steel H-piles.

6.3.3.3 Resistance to Lateral Loads

The resistance to lateral loading developed by the soils in front of the caissons, and the reductions due to group effects, may be determined as per Section 6.3.2.3.

6.3.3.4 Frost Protection

Footings or pile caps should be provided with a minimum of 1.8 m of soil cover for frost protection.

6.4 Soldier Pile and Lagging Wall

A soldier pile and lagging retaining wall may be a preferred alternative for retaining wall 7N. The design of the soldier pile and lagging wall may be carried out according to the guidance provided in Section 6.3.2. Additional guidance for design of the anchors for a soldier pile and lagging wall is provided in the following sections.

6.4.1 Soil and Rock Anchors

A soldier pile and lagging wall will likely require tie-backs to resist the lateral loads from the backfill. This lateral restraint may be provided by tie-backs installed into the bedrock or tie-rods attached to a passive soil anchor (deadman) installed within the embankment fill.

Anchors should be installed in accordance with MTO's Special Provision No. OPSS.PROV 942 and should be provided with double corrosion protection which typically consists of a corrugated PVC sleeve sheathing placed over the full length of the anchor which is in turn encased within a smooth PVC sheathing over the length of the



free stressing zone. The space between the anchor and sheathing is grouted. Corrosion inhibiting wax or grease and a PVC cap should be used to protect the anchor head.

Prestressing of the tie-backs prior to loading will minimize anchor movement due to service loads.

6.4.1.1 Soil Anchors

If needed, resistance to lateral loads could be provided by tie-rods attached to a passive soil anchor or 'deadman' consisting of a continuous concrete anchor wall embedded within the backfill behind the retaining wall. A passive anchor should be located outside the active earth pressure zone behind the retaining wall defined by a line extending from the underside of the wall footing to the ground surface at an angle of 30 degrees from the back of the wall. At this location, the deadmen should therefore be located about 2 to 3 metres from the back of the retaining wall. Between chainage 424+170 and 424+180, where the widening is only about 3 m, the deadman will be located at or very near to the existing edge of pavement. In these locations, installation may be difficult and temporary protection would likely be required.

The capacity that can be developed by a passive soil anchor depends on the depth of embedment, height of the anchor and the distance from the back of the retaining wall and should be reviewed after the design is complete. As preliminary guidance, a deadman anchor along the retaining wall 1 m in height with the top of the anchor wall 1.8 m below the finished pavement (i.e., below frost depth) would develop an unfactored resistance of 400 kPa. This ULS resistance represents an unfactored value; in accordance with the CHBDC, a resistance factor of 0.4 is to be applied in calculating the anchor capacity.

6.4.1.2 Rock Anchors

Rock anchors could also be a technically feasible option for additional lateral restraint if needed, however, with bedrock at depths of 12 m or more and an assumed 45 degree angle from the attachment point at the back of the wall, rock anchors of more than 20 m in length will be necessary and may be costly to install.

The capacity of the rock anchor will be the lesser of the bond strength or the resistance calculated based on the buoyant weight of the potential mass of rock which could be mobilized by the anchor. This is typically considered as the mass of rock included within a cone having an apex at the tip of the anchor and having an apex angle of 60 degrees. For a group of anchors or for a line of closely spaced anchors the resistance must consider the potential overlap between the rock masses mobilized by individual anchors. The calculated resistance for a rock anchor group should not be greater than the sum of the individual rock anchor capacities calculated without group effects.

The factored bond stress at the concrete/rock interface may be taken as 1 MPa for ultimate limit state (ULS) design purposes.

It is recommended that proof load tests be carried out on anchors to confirm their performance at the time of construction. The proof load tests should be carried out to 1.5 times the anchor service loads, and at least 10 percent of the anchors should be tested in this manner.

6.5 Retaining Wall Stability

The global stability of the retaining wall was assessed using the commercially available program SLOPE/W (Version 5.13), produced by Geo-Slope International Ltd., employing the Morgenstern-Price method of analysis. For all analyses, the factor of safety of numerous potential failure surfaces was computed in order to establish the minimum factor of safety. A target factor of safety of 1.3 is normally used for design of retaining walls and embankments under static conditions. This target factor of safety is considered appropriate for the retaining walls at this site, considering the design and performance requirements and the available subsurface data.

Pseudo-static seismic slope stability analyses for the above configuration were also carried out assuming a horizontal acceleration of 0.1g ($k_h = 0.5A$). The minimum target factor of safety used for seismic loading conditions was 1.1.

6.5.1 Global Stability

With appropriate subgrade preparation and proper placement and compaction of fills, the 2 to 3 m high retaining walls will have a factor of safety greater than 1.3 against deep seated slope instability (as indicated in Figure 7). Pseudo-static seismic slope stability analyses for the above configuration also indicate that the retaining wall will have factors of safety of greater than 1.1 (as shown in Figure 8).

It was assumed for the global stability assessment of RSS walls that the reinforcement would prevent the global failure surface from approaching closer than 1/2 the slope height to the face of the reinforced wall (i.e., the retaining strips would extend behind the wall at least half the wall height). The supplier should provide sufficient development length for the reinforcement beyond these distances to satisfy the above assumptions.

Static stability analyses were carried out using the following parameters:

Material	Bulk Unit Weight (kN/m ³)	Drained Conditions		Undrained Conditions
		C (kPa)	Effective Friction Angle ϕ'	Undrained Shear Strength S_u (kPa)
Earth or Granular Embankment Fill	21	0	30°	—
Existing Fill	21	0	30°	—
Weathered Clay to Silty Clay	16.2	5	35°	75-100
Unweathered Clay to Silty Clay	15.8	7	29°	35-50
Glacial Till	20	0	33°	—



6.5.2 Overturning

Retaining walls should be designed to adequately resist overturning. In general, sufficient deadweight is provided behind the wall to resist the lateral earth pressures acting on the wall stem. The deadweight can be provided by the wall mass or by the weight of soil above the footing behind the wall. Overturning resistance can also be provided by extending the wall footing in front of the wall. However, it is more effective to provide additional deadweight behind the wall and extending the foundation in front of the wall is generally only considered when there is insufficient space behind the wall to extend the footing.

6.5.3 Internal Stability

The internal stability of mechanically-reinforced walls should be determined by the supplier.

6.6 Retaining Wall Settlement

Settlement of the retaining walls will occur as a result of compression of the new retaining wall fill and of the underlying subgrade as well as consolidation of the clayey soils on which the new retaining wall will be founded.

Provided that the new retaining wall fill material consists of granular fill, the magnitude of *post-construction settlement* of granular fills is expected to be less than 10 mm, as most will occur during construction.

Retaining wall 7N will be founded on or above significant thicknesses of sensitive silty clay. Construction of the widening will involve the placement of variable depths of additional fill behind the retaining wall. The results of oedometer consolidation and in situ vane shear strength testing indicate that the resulting additional loads on the subgrade will exceed the preconsolidation pressure of portions of the deposit and some settlement is expected to occur as the result of primary consolidation of the clayey deposits. Additionally, the sensitive silty clay soils in this area are expected to undergo secondary compression settlements of up to 100% of the primary consolidation settlement amount where loaded in excess of the deposit's preconsolidation pressure. These primary consolidation and secondary compression settlements will take place over months and years after the completion of construction.

The loading imposed on the underlying soils and the resulting magnitude of settlement will depend on the type of wall constructed. If an RSS wall or soldier pile and lagging wall is constructed, the loading due to the full weight of the widened embankment fill will be imposed on the underlying soils. The total estimated magnitude of the primary consolidation settlement may range between about 15 to 30 mm, based on the values measured in the oedometer test and the vane shear strength results. This settlement range reflects the varied filling conditions and natural variability in material properties along the retaining wall alignment. It is estimated that 90 per cent of the primary consolidation settlement should be completed within about 1 to 2 months. The consolidation testing and anticipated loads also indicate a potential for additional secondary compression settlements and the magnitude of those additional settlements is estimated to be up to about 100% of the primary consolidation settlement (i.e. another 30 mm) after 50 years. These settlements would be almost entirely differential with respect to the existing embankment and the existing flexible pavement.

It is considered that an RSS wall can tolerate the above ranges of settlement, although it may be necessary to provide slip joints to reduce the potential for the settlements to result in mis-alignment of the RSS panels.

If a pile-supported reinforced concrete wall is constructed, a portion of the weight of additional fill will be supported by the pile caps, and in turn by the piles, and this will reduce the loading imposed on the underlying soils. If the loads are assumed to be reduced by 50 percent, the computed settlement magnitudes due to primary



consolidation of the underlying clayey soils, along the wall will be less than about 15 millimetres. This portion of the wall alongside the E-N/S Kirkwood/Carling ramp has flexible pavement and is more settlement tolerant than the rigid pavement of the highway.

It is estimated that 90 per cent of the primary consolidation settlement should be completed within about 1 to 2 months. Based on the anticipated loading the magnitude of additional secondary compression for a pile supported reinforced concrete wall, total settlement at the edge of pavement is not considered to be an issue.

6.7 Adjacent Structures

There is a high mast light standard located towards the eastern end of the proposed retaining wall and offset by about 3.7 m from the face which we understand is founded on a deep foundation into till/bedrock. It is located within the zone of influence of the retaining wall loading. The total primary consolidation settlement of the soils adjacent to the piled foundation of the high mast light standard is anticipated to be in the order of 5 mm. This amount of settlement is not considered sufficient to induce downdrag forces on the existing light standard foundation.

6.8 Site Coefficient and Seismic Liquefaction

6.8.1 Site Coefficient

For seismic design purposes, the Site Coefficient, S , for this site in accordance with Section 4.4.6 of the CHBDC may be taken as 1.0, consistent with Soil Profile Type I.

6.8.2 Seismic Liquefaction

Seismic liquefaction occurs when earthquake vibrations cause an increase in the pore water pressure within the soil, which reduces the effective stress between the soil particles and the soil's frictional resistance to shearing. This phenomenon, which leads to a temporary reduction in the shear strength of the soil, may cause lateral spreading, reduced shear resistance (i.e., bearing capacity) of soils which support foundations and reduced shaft resistance for deep foundations as well as reduced resistance to lateral loading. In addition, seismic settlements may occur once the vibrations and shear stresses have ceased.

The following conditions are more prone to experiencing seismic liquefaction:

- Coarse grained soils (i.e., more probable for sands than for silts);
- Soils having a loose state of packing; and,
- Soils located below the groundwater level.

There is not considered to be a potential liquefaction hazard at this site, and therefore liquefaction need not be considered in the design of foundations for RW7N.



6.9 Lateral Earth Pressures for Design

The lateral earth pressures acting on the retaining walls will depend on the type and method of placement of the backfill materials, on the nature of the soils behind the backfill, on the magnitude of surcharge including construction loadings, on the freedom of lateral movement of the structure, and on 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 lateral earth pressures:

- Select free-draining granular fill meeting the specifications of Ontario Provincial Standard Specifications (OPSS.PROV) Granular 'A' or Granular 'B' Type II but with less than 5 per cent passing the 200 sieve should be used as backfill behind the walls. This fill should be compacted in accordance with MTO's Special Provision 105S10.
- Longitudinal drains and weep holes should be installed to provide positive drainage of the granular backfill. Other aspects of the granular backfill requirements with respect to sub-drains and frost taper should be in accordance with OPSD 3101.150 and 3121.150.
- A minimum compaction surcharge of 12 kPa should be included in the lateral earth pressures for the structural design of the retaining walls, in accordance with CHBDC Section 6.9.3 and Figure 6.6. Compaction equipment should be used in accordance with MTO's Special Provision 105S10. 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 1.8 m behind the back of the retaining wall (case (a) in Figure C6.20 of the Commentary to the CHBDC) or within the wedge-shaped zone defined by a line drawn at 1.5 horizontal to 1 vertical (1.5H:1V) extending up and back from the rear face of the footing (case (b) in Figure C6.20 of the Commentary to the CHBDC).

6.9.1 Static Lateral Earth Pressures for Design

The following parameters (unfactored) may be used for Select Subgrade Material (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

The following parameters (unfactored) may be used for granular backfill:

	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



If the wall support allows lateral yielding of the stem, active earth pressures may be used in the geotechnical design of the structure. If the wall support does not allow lateral yielding, at-rest earth pressures should be assumed for geotechnical design. The movement to allow active pressures to develop within the backfill, and thereby assume an unrestrained structure, may be taken as:

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

If the wall support 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.

6.9.2 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.

Seismic loading will result in increased lateral earth pressures acting on the retaining walls. The wall should be designed to withstand the combined lateral loading for the appropriate static pressure conditions given above, plus the earthquake-induced dynamic earth pressure. According to Table A3.1.1 of the CHBDC, this site is located in Seismic Performance Zone 4. The site-specific zonal acceleration ratio for Ottawa is 0.2. Based on experience, for the subsurface conditions at this site, up to 50% amplification could be expected for the ground conditions at this site, resulting in an increase in the design ground surface acceleration to 0.3. The seismic lateral earth pressure coefficients given below have therefore been derived based on a design zonal acceleration ratio of $A = 0.3$.

In accordance with Sections 4.6.4 and C.4.6.4 of the 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. $k_h = 0.45$). For structures which allow lateral yielding (i.e., the toe walls) k_h is taken as 0.5 times the zonal acceleration ratio (i.e. $k_h = 0.15$).

The following seismic active pressure coefficients (K_{AE}) for the two backfill cases (Case I and Case II) may be used in design. These coefficients reflect the K_{AE} obtained using the k_h value as described above.



Seismic Active Pressure Coefficients, K_{AE}

	SSM	Granular A	Granular B Type II
Yielding wall	0.42	0.34	0.34
Non-yielding wall	0.86	0.68	0.68

The above K_{AE} values for yielding walls are applicable provided that the calculated wall displacement is more than 250A (mm), where A is the design zonal acceleration ratio of 0.30. This corresponds to a displacement value of approximately 75 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 either the static active earth pressure coefficient, K_a , or the static at-rest earth pressure coefficient, K_0 ;

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).

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 present above the top of the wall, 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.

6.10 Embankment Design and Construction (High Fill Embankment 8N)

The existing embankments along the alignment of high fill embankment 8N are between 1.5 and 4.5 m high, with side slopes ranging from 1.25H:1V to 3H:1V. The proposed widening would extend the crest of the existing roadway 2 to 12 m north and would extend the toe 6 to 12 m north. The footprint for the proposed widening in this area is such that slopes of between 3H:1V to 4.5H:1V may be accommodated and, as such, retaining walls have not been considered along this section of the widening.

The following technically feasible options for the proposed widening in the area of high fill embankment 8N have been considered:

- Proposed 3H:1V to 4.5H:1V slopes;
- Steepened 2H:1V to 3H:1V slopes; and,
- 1H:1V mechanically stabilized earth (MSE) slopes.

The preferred embankment geometry and construction methodology will depend on property constraints, appearance, performance and cost considerations. As noted above, property constraints along the proposed high fill embankment 8N are such that slopes of between 3H:1V and 4.5H:1V may be accommodated. The main performance consideration for high fill embankment slopes is the overall slope stability (static and seismic) and the amount settlement induced by the placement of additional fills within the paved section of the roadway. The impacts of the embankment loading as the impacts of settlement below the embankment must be considered and, where negatively impacted, existing buried services should be relocated.

Based on the sections provided by MRC, the 3H:1V to 4.5H:1V slopes as shown meet or exceed MTO slope stability guidelines. Given the stiffness of the underlying clay deposit, consideration was also given to increasing the slopes to as steep as 2H:1V to limit the footprint of the widening. These steepened slopes also met the static and seismic stability guidelines and therefore, based on geotechnical considerations, the sideslopes may be steepened to slope angles of 2H:1V along the length of high fill embankment 8N.

Where slopes of steeper than 2H:1V are required, consideration could be given to using mechanically stabilized earth slopes. Reinforced, steepened embankment side slopes of up to 1H:1V are geotechnically feasible and could be considered in locations where there is insufficient space to construct slopes at flatter inclinations (e.g. near Merivale Road) or where transitioning from a retaining wall to a slope. MSE slopes at inclinations of up to 1H:0.5V may be considered where space is extremely limited, however, MSE slopes steeper than 1H:1V are susceptible to erosion, would require specialized facing for permanent applications, and may or may not meet aesthetic requirements. Temporary MSE walls with steep slopes could be considered for construction of temporary surcharges at the retaining wall 7N site.

With respect to settlement, the stiffness of the clays in this area is such that the placement of additional fills along this section of the widening are not expected to induce significant post-construction settlements of the existing or new portions of the roadway (i.e., less than 15 mm due to primary consolidation of the underlying unweathered clay). The settlement should be monitored prior to paving to assess the potential impact and to assess when paving may take place. Up to an additional 15 mm of secondary creep can also be expected over the next 20 years. The total settlement magnitude of 30 mm (due to primary and secondary consolidation) will be almost entirely differential with respect to the existing embankment which has been in place for more than 40 years.

The Cave Creek collector sewer crosses beneath high fill embankment 8N just west of Merivale Road. This sewer has been tunnelled through the bedrock and it is not anticipated that the loading from the embankment widening will result in any impacts to this utility.



6.10.1 Subgrade Preparation

Any surficial topsoil, organic matter and softened / loosened soils should be stripped from within the limits of the widening, including the existing embankment side slope and the new footprint.

Where 2H:1V or shallower side slopes are provided for the embankment widening, the existing fill materials within the footprint of the widening can generally be left in place provided that subgrade settlements on the order of 25 to 50 mm can be tolerated over 20 years. However the subgrade surface should be proof rolled and compacted to 95 percent of the standard Proctor maximum dry density in accordance with MTO's Special Provision OPSS501. This guideline is appropriate where earth filling will be used to construct the embankment widening.

If constructed, MSE embankments should generally be placed on the surface of the native soils after removal of any existing fill within the footprint of the widening. MSE embankments may also be placed on a pad of compacted engineered fill, placed on the surface of the native soils. Where the existing fill is greater than 1 m in depth, MSE embankments may be founded on a minimum 0.75 m thick pad of compacted engineered fill of Granular B Type II after removal of an equal depth of the existing fill. The surface of the existing fill should be heavily proof-rolled prior to the placement of engineered fill.

6.10.2 Embankment Construction

Construction of the widened embankment above the prepared subgrade may be carried out using either clean earth or rock fill (in accordance with OPSS.PROV 212) or Select Subgrade Material (in accordance with OPSS.PROV 1010), depending on material availability. The new embankment fills should be benched into the existing embankment in accordance with OPSD 208.010.

Embankment fill should be placed in regular lifts with a loose thickness not exceeding 300 mm, and be compacted to at least 95 percent of the material's standard Proctor maximum dry density in accordance with MTO's Special Provision 105S10. Inspection and field density testing should be carried out by qualified personnel during placement operations to ensure that appropriate materials are used and that adequate levels of compaction have been achieved.

To reduce surface water erosion on the final embankment side slopes, placement of topsoil and seeding or pegged sod is recommended.

6.10.3 Embankment Stability

The global stability of the embankment slopes was assessed using the commercially available program SLOPE/W (Version 5.13), produced by Geo-Slope International Ltd., employing the Morgenstern-Price method of analysis. For all analyses, the factor of safety of numerous potential failure surfaces was computed in order to establish the minimum factor of safety. A minimum factor of safety of 1.3 is normally used for design of embankments under static conditions. This factor of safety is considered appropriate for the embankments at this site, considering the design and performance requirements and the available subsurface data.

Pseudo-static seismic slope stability analyses for the above configuration were also carried out assuming a horizontal acceleration of 0.1g ($k_h = 0.5 A$). The minimum factor of safety used for seismic loading conditions was 1.1.

With appropriate subgrade preparation and proper placement of earth or lightweight fills, the 1.5 to 4.5 m high embankment will have a factor of safety greater than 1.3 against deep seated slope instability for slopes 2H:1V and shallower (as indicated in Figure 9). Pseudo-static seismic slope stability analyses for the above configuration also indicate that the embankment side slopes will have factors of safety of greater than 1.1 (as indicated in Figure 10). Static stability analyses were carried out using the following parameters:

Material	Bulk Unit Weight (kN/m ³)	Drained Conditions		Undrained Conditions
		C' (kpa)	Effective Friction Angle	Undrained Shear Strength (kPa)
Earth or Granular Embankment Fill	21	0	30°	—
Existing Fill	21	0	30°	—
Weathered Clay to Silty Clay	16.2	5	35°	75
Unweathered Clay to Silty Clay	15.8	7	29°	50
Glacial Till	21	0	32°	—

6.10.4 Embankment Settlement

Embankment 8N will be founded on or above significant thicknesses of stiff sensitive silty clay. Construction of the embankment widenings will generally involve the placement of variable depths of additional fill along the embankment alignment. Some settlement of the embankment subgrade can be expected due to compression of the clay soils (i.e., the weathered clay crust and, in particular, the underlying grey silty clay to clay). The results of oedometer consolidation and in-situ vane testing indicate that the effective stress level in the clayey deposits will not exceed the deposit's preconsolidation pressure. Provided that the new embankment fill material consists of granular fill, Select Subgrade Material or clean earth fill, the immediate settlement of the embankment fill itself during construction is expected to be less than about 25 mm. Consolidation settlements corresponding to recompression of the clayey deposits are expected to be less than 30 mm.

6.11 Design and Construction Considerations

Excavations to subgrade depth for pile cap or foundation construction for RW7N will require excavations of at least 1.8 m depth below the existing ground surface for frost protection. Because the excavations are part-way up the existing embankment slope, the excavations will typically extend through the ramp fill materials overlying weathered silty clay. The groundwater level at the site is expected to be below the anticipated founding level.

Excavations for tie-back anchors or geogrid placement behind the retaining wall may be in close proximity to the existing pavement structure and may require roadway protection as described below.

6.11.1 Excavations

Excavations should be carried out in accordance with the guidelines outlined in the latest edition of the Occupational Health and Safety Act (OHSA) for Construction Activities. The native weathered crust materials are classified as Type 2 soils and the existing fill materials are classified as Type 3 soil, according to the OHSA.



In general, temporary (i.e., during the construction period) excavations through the existing fill materials above the water table may be made at 1H:1V. These temporary slopes will be susceptible to sloughing as a result of drying during sunny periods or from erosion during periods of rainfall. Fully covering the cut slopes with tarps, securely pinned, during the duration of construction will reduce the potential for sloughing of the slope face.

6.11.2 Groundwater and Surface Water Control

Excavations through the surficial sand fill to subgrade depth for placement of compacted engineered fill or footing construction will likely involve minimal groundwater and surface water control. It should be possible to handle ground and surface water inflows by pumping from well filtered sumps established in the floor of the excavations.

The standpipe installed for this investigation in Borehole 09-9 will be decommissioned.

6.11.3 Roadway Protection

It is anticipated that temporary roadway protection may be required along Highway 417 to permit construction of the retaining wall or for installation of a deadman soil anchor. It may be feasible to embed soldier piles or steel sheet piling sufficiently into the overburden without additional lateral support for excavations up to 3 m in depth (i.e., it may be feasible to cantilever the shoring). For deeper excavations it will be necessary to provide lateral support using either rakers supported on footings within the excavation or using tie-backs grouted into the soils or bedrock behind the shoring. Deadman soil anchors may also be considered.

The design of the shoring will be the responsibility of the contractor. The shoring will have to be designed to resist lateral earth pressures that are controlled by the flexibility of the shoring and its method of support. However, conceptually, the temporary protection could consist of either soldier piles and lagging or steel sheet piling.

Temporary excavation support systems should be designed and constructed in accordance with OPSS.PROV 539. The lateral movement of temporary shoring systems should meet Performance Level 2 as specified in OPSS.PROV 539, provided that any buried utilities that may be present adjacent to the excavations can tolerate this magnitude of deformation.

To the anticipated depths of excavation, it is not expected that basal heaving or basal instability will be a concern.



7.0 CLOSURE

This report was prepared by Mr. William Cavers, P. Eng. This report was reviewed by Mr. Fintan J. Heffernan, P. Eng, the designated MTO contact for this project.

GOLDER ASSOCIATES LTD.



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Fintan J. Heffernan, P.Eng.
Designated MTO Foundations Contact

ESO/WC/MSS/FJH/sg

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FOUNDATION INVESTIGATION AND DESIGN REPORT

Table 1: Comparison of Retaining Wall 7N Alternatives
Merivale Road to Kirkwood Avenue
Highway 417
W.P. 4058-01-00

Retaining Wall	Retaining Wall Option	Feasibility	Advantages	Disadvantages	Relative Costs	Risks/ Consequences
RW7N (424+125 to 424+255)	"Perched" RSS wall Preferred option	<ul style="list-style-type: none"> Recommended foundation treatment 	<ul style="list-style-type: none"> Limited excavation of existing embankment required Standard backfill 	<ul style="list-style-type: none"> Post-construction settlement may cause some visual distortion of the wall 	<ul style="list-style-type: none"> Least expensive option 	<ul style="list-style-type: none"> N/A
	Reinforced concrete wall supported on shallow foundations	<ul style="list-style-type: none"> Not feasible without use of lightweight fill 	<ul style="list-style-type: none"> N/A 	<ul style="list-style-type: none"> Existing fill materials beneath footings would need to be replaced with engineered fill 	<ul style="list-style-type: none"> Less costly than concrete wall on deep foundations 	<ul style="list-style-type: none"> Predicted settlements exceed tolerance for structure
	Reinforced concrete wall supported on deep foundations	<ul style="list-style-type: none"> Feasible foundation treatment 	<ul style="list-style-type: none"> Relatively less maintenance required than for pile and lagging wall Reduced embankment settlement 	<ul style="list-style-type: none"> More involved construction Downdrag on piles will reduce capacity slightly 	<ul style="list-style-type: none"> More expensive than pile and lagging wall or shallow foundations 	<ul style="list-style-type: none"> N/A
	Soldier pile and concrete lagging (SPL) wall	<ul style="list-style-type: none"> Feasible 	<ul style="list-style-type: none"> Simple to construct 	<ul style="list-style-type: none"> Appearance may not be acceptable Increased maintenance 	<ul style="list-style-type: none"> Less expensive than reinforced concrete wall 	<ul style="list-style-type: none"> Increased maintenance costs
	Wall panels supported on shallow caissons	<ul style="list-style-type: none"> Likely not feasible 	<ul style="list-style-type: none"> N/A 	<ul style="list-style-type: none"> N/A 	<ul style="list-style-type: none"> N/A 	<ul style="list-style-type: none"> N/A
	Lightweight Fill in combination with other foundation options	<ul style="list-style-type: none"> Feasible 	<ul style="list-style-type: none"> Reduced settlement to within acceptable limits for concrete retaining walls on shallow foundations 	<ul style="list-style-type: none"> May not be compatible with RSS walls reinforcing strips 	<ul style="list-style-type: none"> Additional costs associated with purchase of lightweight fill 	<ul style="list-style-type: none"> Possible delays to construction schedule if time to end of primary consolidation exceeds predicted values
	Preloading combination with wall options	<ul style="list-style-type: none"> Feasible 	<ul style="list-style-type: none"> Reduced settlement of pavement structure to within acceptable limits 	<ul style="list-style-type: none"> Requires 1 to 2 months' time to reach primary consolidation 	<ul style="list-style-type: none"> Additional costs for instrumentation and monitoring of surcharge 	

FOUNDATION INVESTIGATION AND DESIGN REPORT

Table 2: Comparison of High Fill Embankment 8N Alternatives
Merivale Road to Kirkwood Avenue
Highway 417
W.P. 4058-01-00

Embankment	Retaining Wall Option	Feasibility	Advantages	Disadvantages	Relative Costs	Risks/Consequences
8N (WBL 224+255 to 224+504)	3H:1V or shallower slopes	<ul style="list-style-type: none"> Recommended 	<ul style="list-style-type: none"> Simple to construct 	<ul style="list-style-type: none"> Larger land footprint used Greater fill volumes required 	<ul style="list-style-type: none"> Higher cost than 2H:1V slopes 	<ul style="list-style-type: none">
	2H:1V slopes	<ul style="list-style-type: none"> Feasible 	<ul style="list-style-type: none"> Simple to construct Smaller footprint than shallower slopes 	<ul style="list-style-type: none"> Slightly increased amounts of settlement, still below preconsolidation pressure 	<ul style="list-style-type: none"> Less than 3H:1V slopes 	<ul style="list-style-type: none"> Steeper side slopes may require special vegetation and maintenance techniques
	1H:1V MSE slopes	<ul style="list-style-type: none"> Feasible where footprint is limited 	<ul style="list-style-type: none"> Allows for placement of stable slopes in confined spaces Reduced fill volumes 	<ul style="list-style-type: none"> More time-consuming to construct Additional costs for geosynthetics 	<ul style="list-style-type: none"> More expensive than unreinforced slopes 	<ul style="list-style-type: none"> N/A

Oct 75, FF-S-21

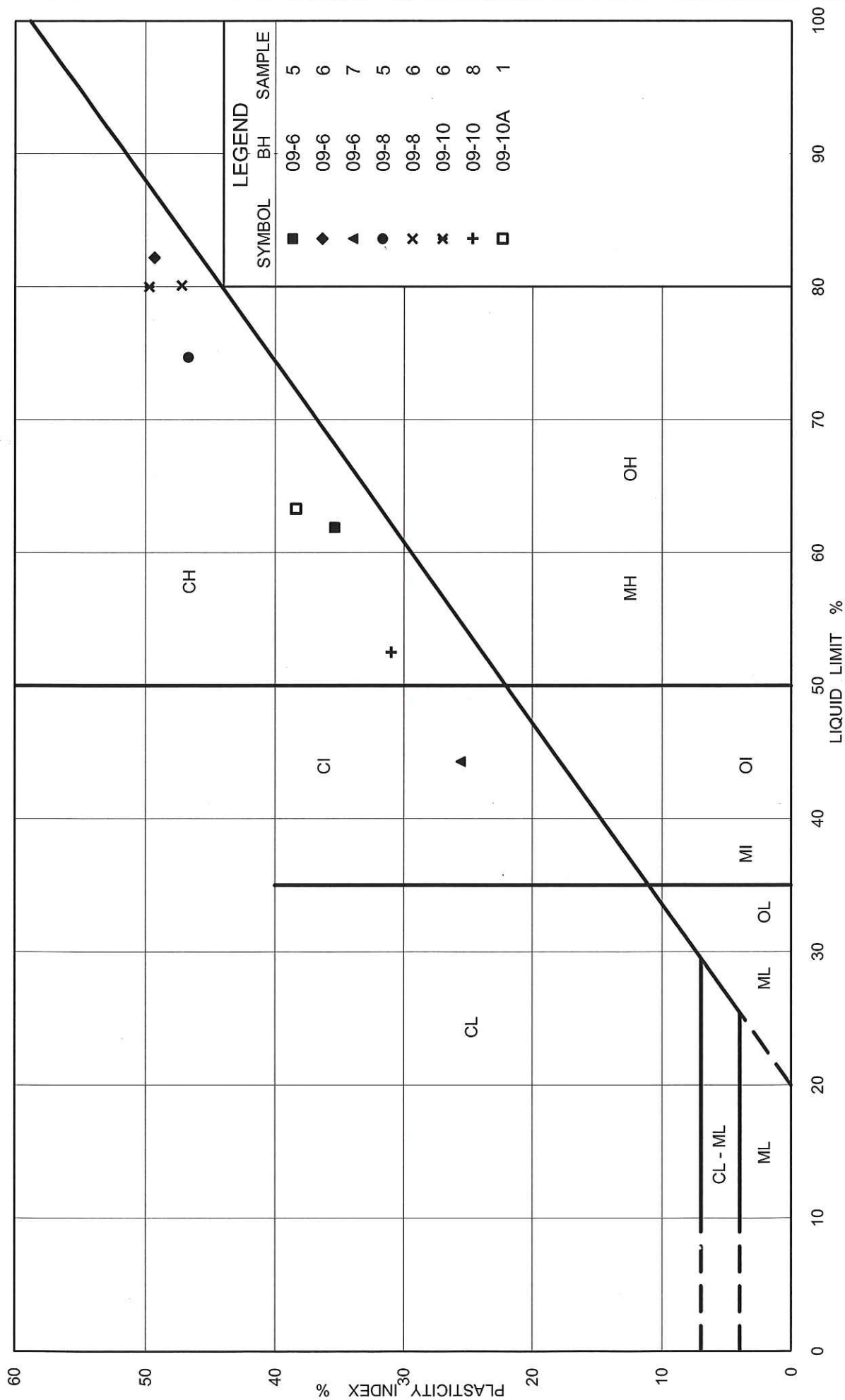


Figure No. 1

PLASTICITY CHART Silty Clay to Clay

Ministry of Transportation



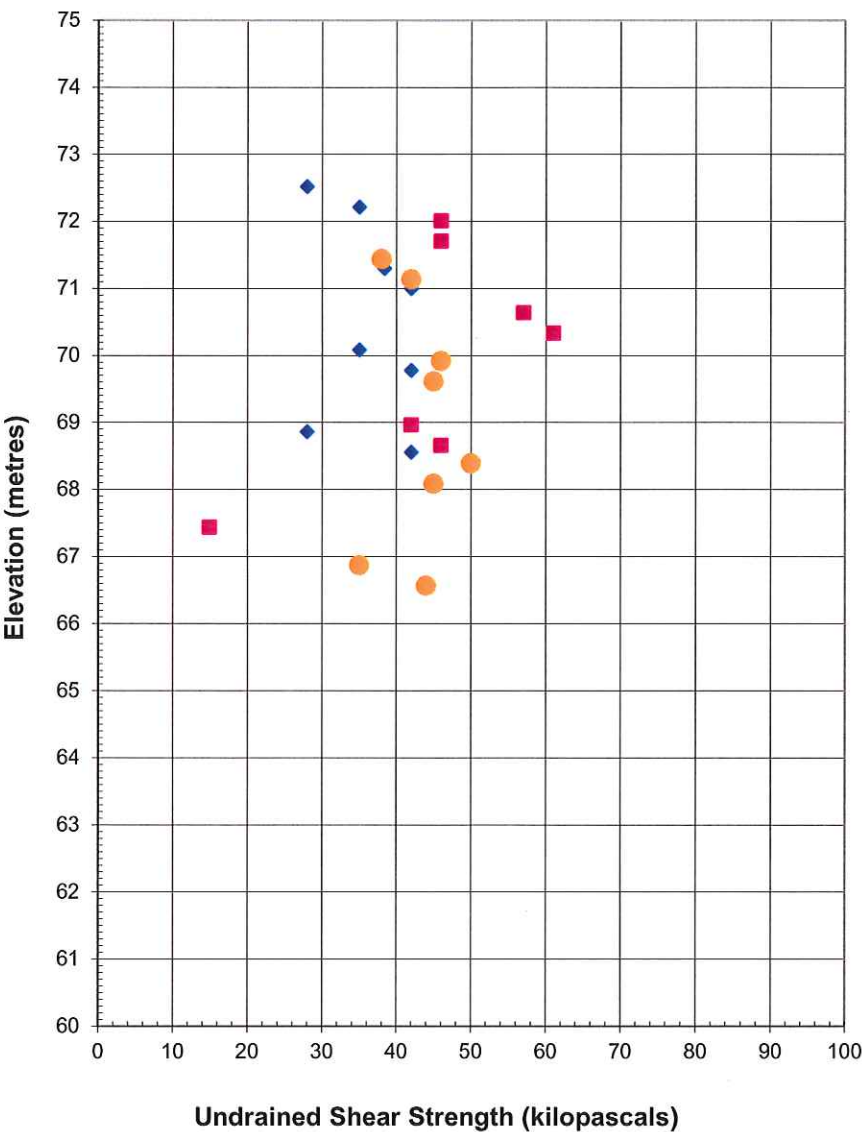
Project No. 05-1120-210 / 6000

Checked By : CNM

Ontario

SUMMARY OF UNDRAINED SHEAR STRENGTHS
VERSUS ELEVATION - RW 7N

FIGURE 2

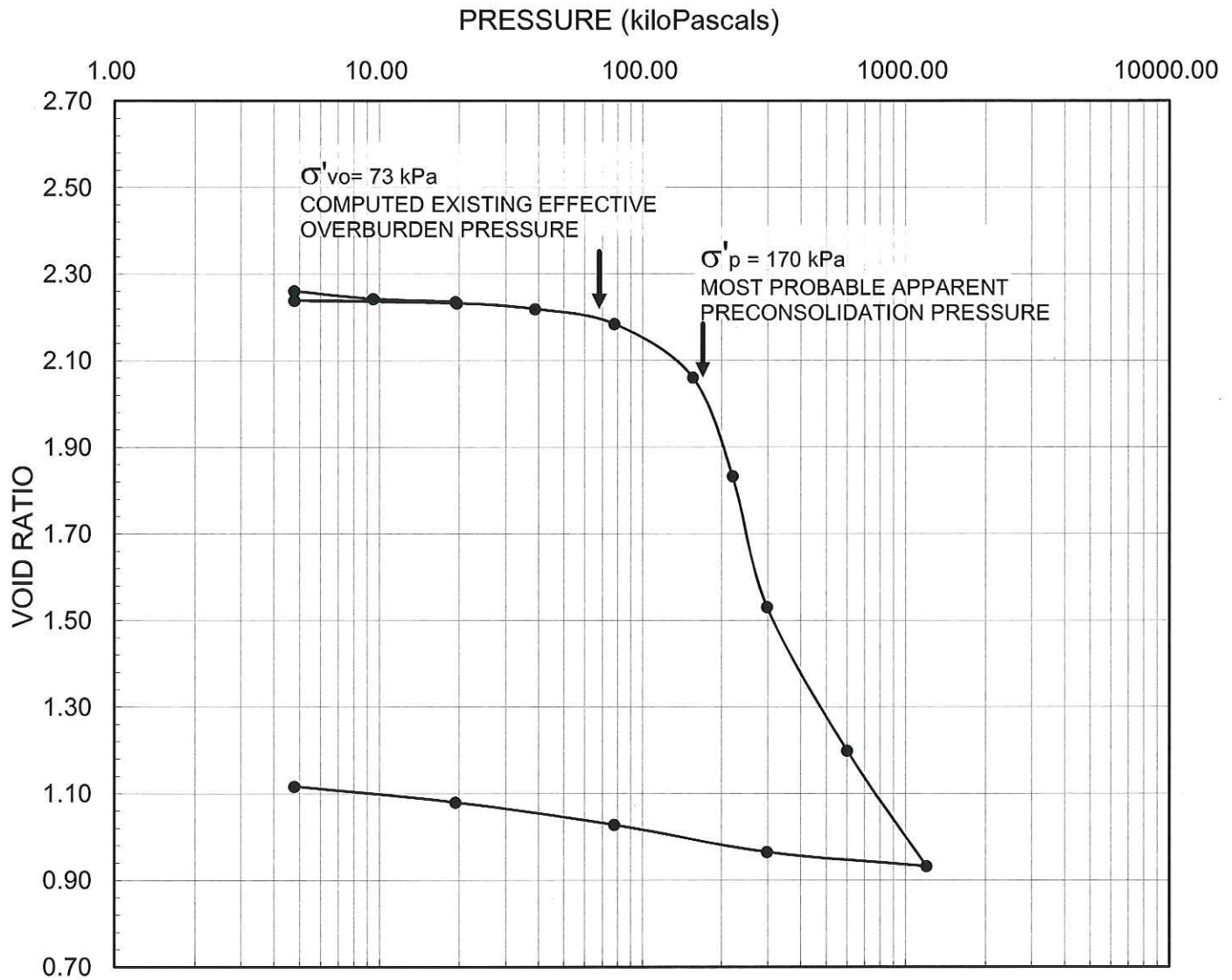


◆ BH 09-5 ■ BH09-6 ● BH 06-151

Date December 07, 2015
Project 05-1120-210

Golder Associates

Drawn ESO/JC
Chkd WC



LEGEND

Borehole: 09-6	$w_i = 80.2\%$	$S_o = 99\%$
Sample: 5	$w_f = 44.6\%$	$C_c = 2.30$
Depth (m): 4.6-5.1	$w_l = 61.9\%$	$C_r = 0.015$
	$w_p = 26.5\%$	



SCALE	AS SHOWN
DATE	11/23/15
DESIGN	NA
CADD	NA

TITLE

CONSOLIDATION TEST RESULTS

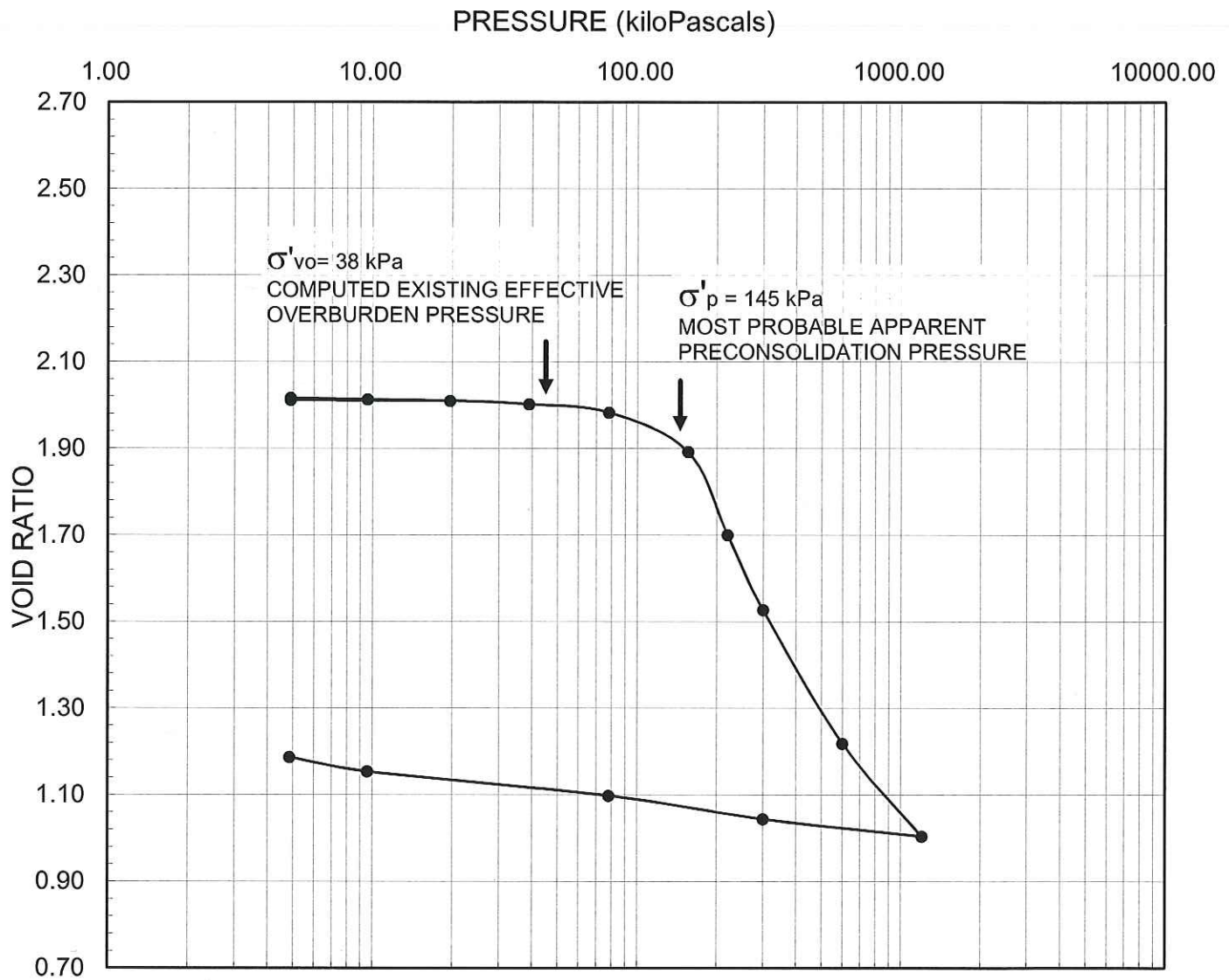
FILE No. Consolidation summary

PROJECT No. 05-1120-210/6000 REV. 1

CHECK	CNM
REVIEW	WC

FIGURE

3



LEGEND

Borehole: 09-10A	$w_i = 71.9\%$	$S_o = 99\%$
Sample: 1	$w_f = 43.4\%$	$C_c = 1.29$
Depth (m): 4.0-4.5	$w_l = 63.3\%$	$C_r = 0.003$
	$w_p = 24.9\%$	



SCALE	AS SHOWN
DATE	11/23/15
DESIGN	NA
CADD	NA

TITLE

CONSOLIDATION TEST RESULTS

FILE No. Consolidation summary

PROJECT No. 05-1120-210/6000 REV. 1

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REVIEW	WC

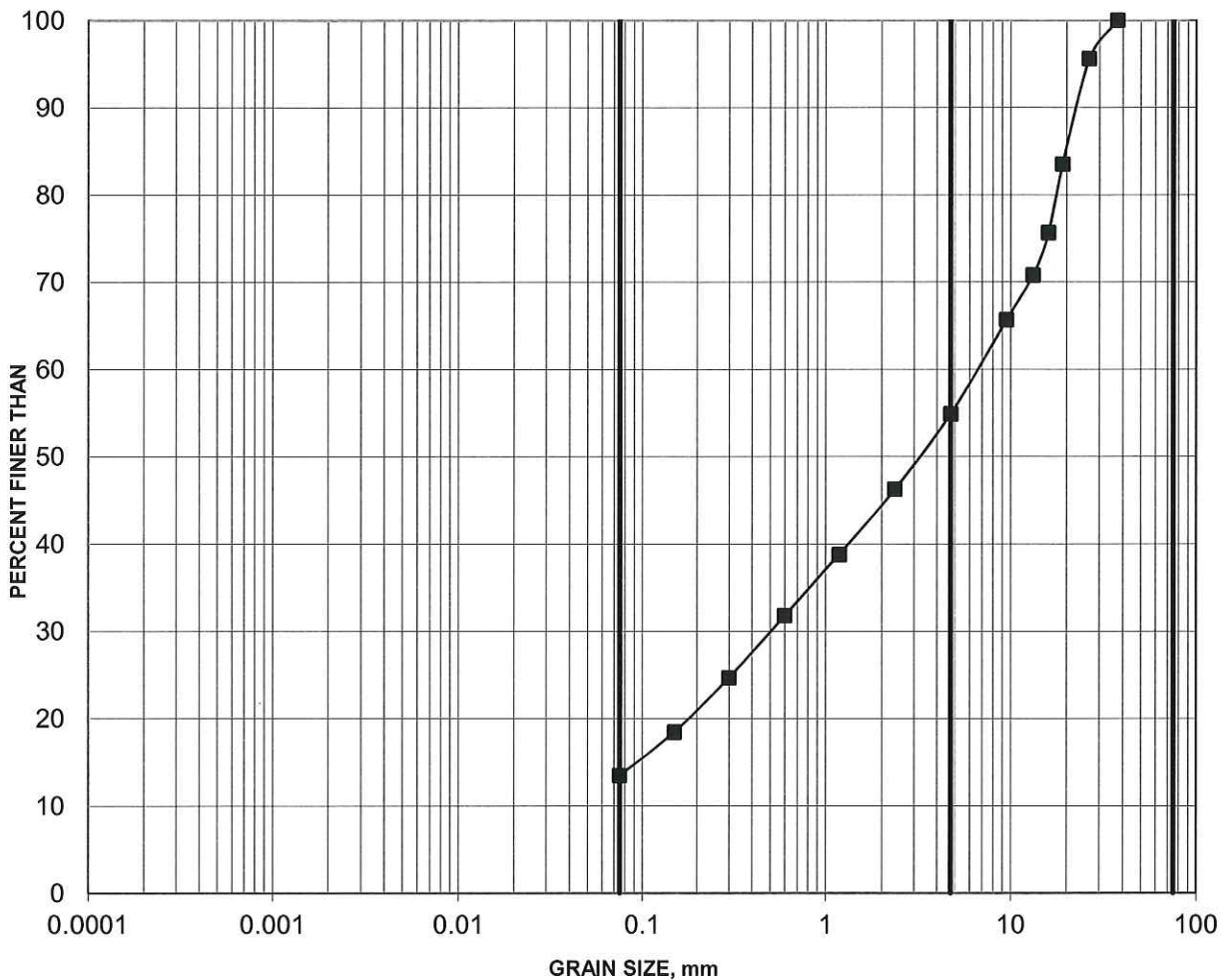
FIGURE

5

GRAIN SIZE DISTRIBUTION

FIGURE 6

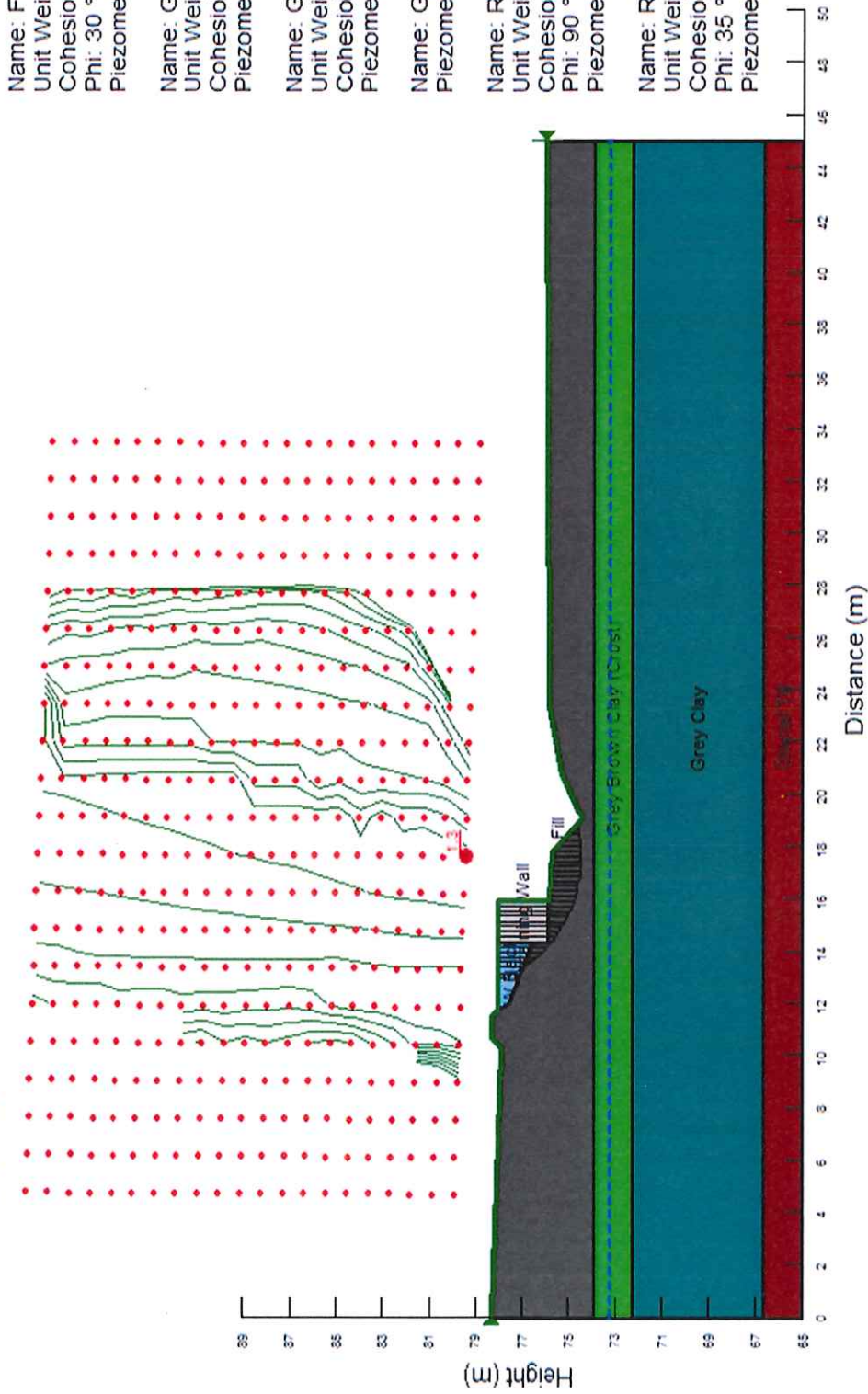
SAND AND GRAVEL, TRACE SILT AND CLAY



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

Borehole	Sample	Depth (m)
—■— 09-7	11	10.67-11.28

Title: Slope Stability (undrained)
 Comments: Retaining Wall - Highway 417
 Horiz Seismic Load: 0



Name: Fill
 Unit Weight: 21 kN/m³
 Cohesion: 0 kPa
 Phi: 30 °
 Piezometric Line: 1

Name: Grey Brown Clay (Crust)
 Unit Weight: 16.2 kN/m³
 Cohesion: 75 kPa
 Piezometric Line: 1

Name: Grey Clay
 Unit Weight: 15.8 kN/m³
 Cohesion: 35 kPa
 Piezometric Line: 1

Name: Glacial Till
 Piezometric Line: 1

Name: Retaining Wall
 Unit Weight: 24 kN/m³
 Cohesion: 100 kPa
 Phi: 90 °
 Piezometric Line: 1

Name: RW Backfill
 Unit Weight: 21 kN/m³
 Cohesion: 0 kPa
 Phi: 35 °
 Piezometric Line: 1

Foundation Investigation and Design Report

Retaining Wall 7N

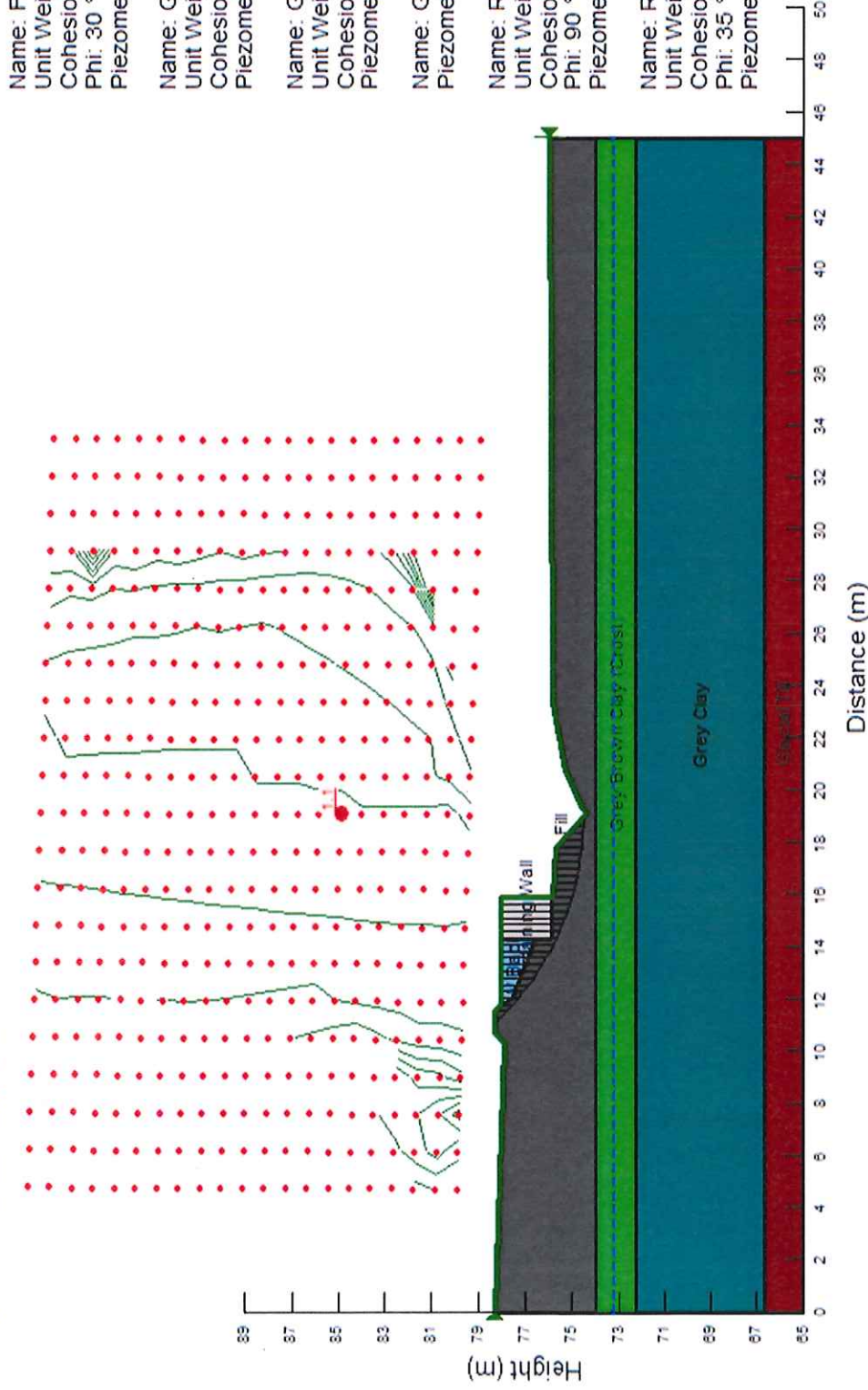
Static Slope Stability Analysis Output

Project No. 05-1120-210-6000
 Drawn: WC
 Date: 12/7/2015
 Checked: FJH
 Review: FJH

Figure 7



Title: Slope Stability (undrained)
 Comments: Retaining Wall - Highway 417
 Horiz Seismic Load: 0.1



Name: Fill
 Unit Weight: 21 kN/m³
 Cohesion: 0 kPa
 Phi: 30 °
 Piezometric Line: 1

Name: Grey Brown Clay (Crust)
 Unit Weight: 16.2 kN/m³
 Cohesion: 75 kPa
 Piezometric Line: 1

Name: Grey Clay
 Unit Weight: 15.8 kN/m³
 Cohesion: 35 kPa
 Piezometric Line: 1

Name: Glacial Till
 Piezometric Line: 1

Name: Retaining Wall
 Unit Weight: 24 kN/m³
 Cohesion: 100 kPa
 Phi: 90 °
 Piezometric Line: 1

Name: RW Backfill
 Unit Weight: 21 kN/m³
 Cohesion: 0 kPa
 Phi: 35 °
 Piezometric Line: 1

Foundation Investigation and Design Report

Retaining Wall 7N

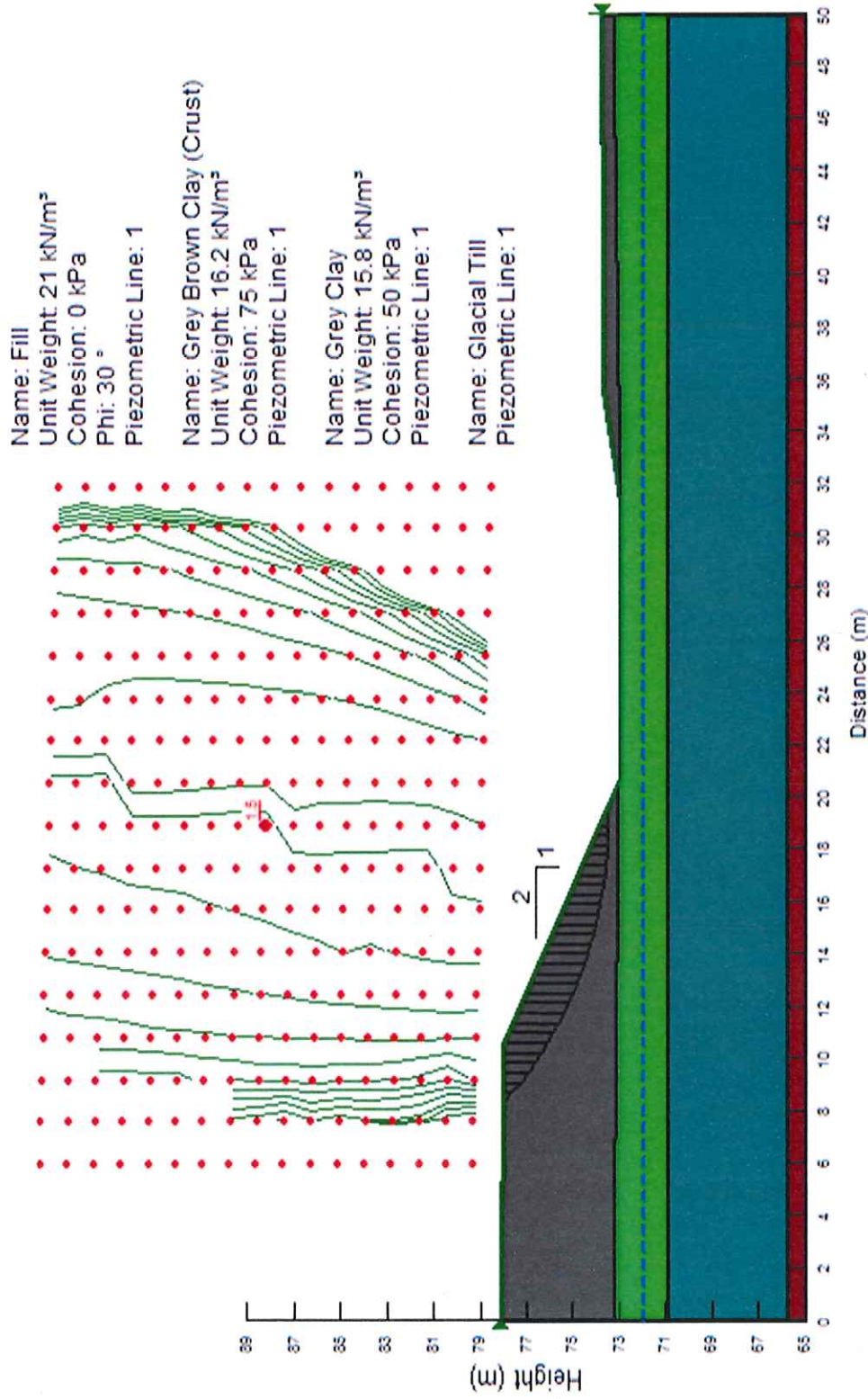
Static Slope Stability Analysis Output



Project No.	05-1120-210-6000
Drawn:	WC
Date:	12/7/2015
Checked:	FJH
Review:	FJH

Figure 8

Title: Slope Stability (undrained)
 Comments: High Fill Embankment - Highway 417
 Horiz Seismic Load: 0

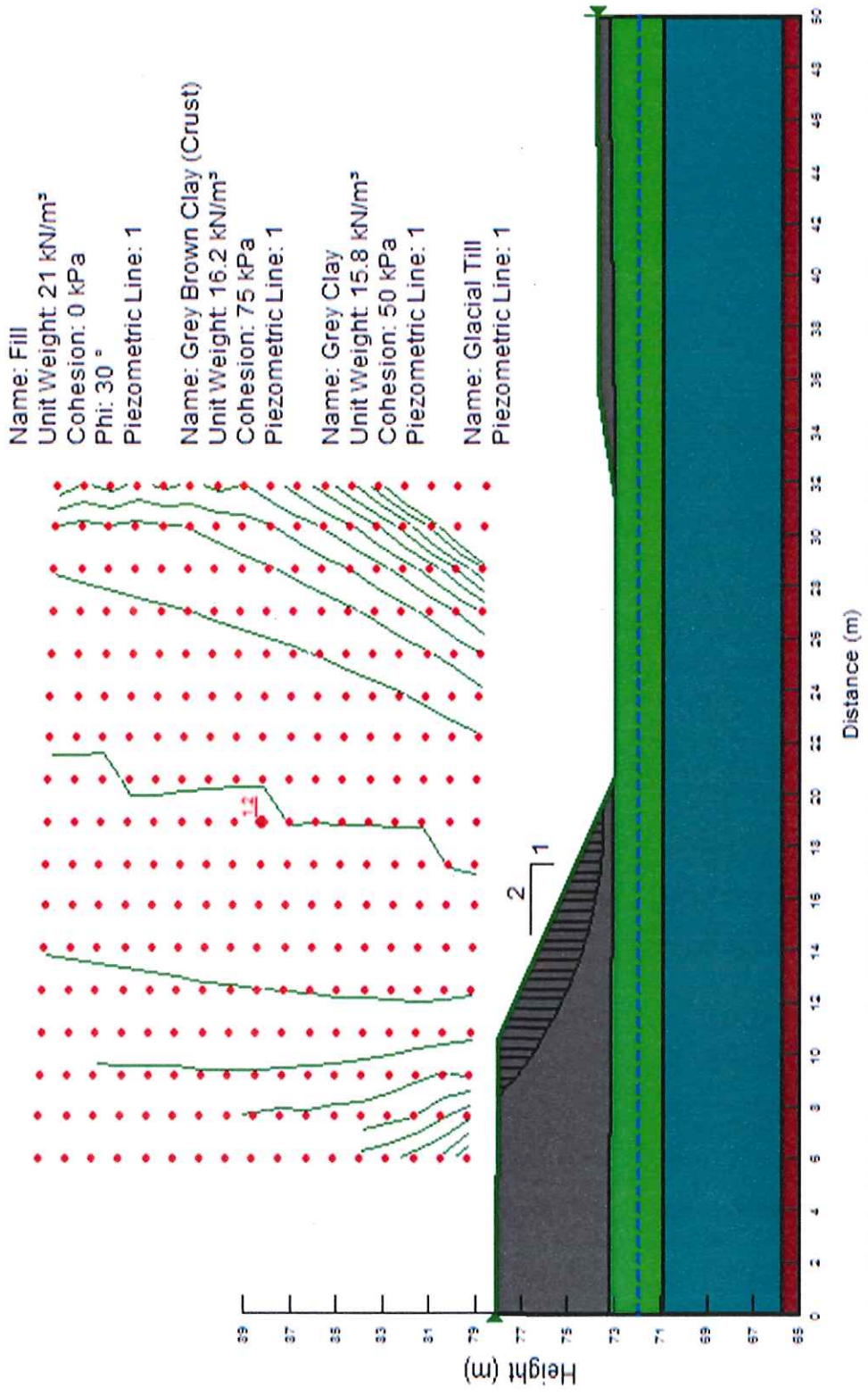


Foundation Investigation and Design Report
 High Fill Embankment 8N
 Static Slope Stability Analysis Output

Project No.	05-1120-210-6000
Drawn:	WC
Date:	12/7/2015
Checked:	FJH
Review:	FJH

Figure 9

Title: Slope Stability (undrained)
 Comments: High Fill Embankment - Highway 417
 Horiz Seismic Load: 0.1



Foundation Investigation and Design Report
 High Fill Embankment 8N
 Static Slope Stability Analysis Output

Project No.	05-1120-210-6000
Drawn:	WC
Date:	12/7/2015
Checked:	FJH
Review:	FJH

Figure 10



APPENDIX A

Record of Boreholes

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
DO or DP	Seamless open-ended, driven or pushed tube samplers
DS	Denison type sample
FS	Foil sample
RC	Rock core
SC	Soil core
SS	Split spoon sampler
ST	Slotted tube
TO	Thin-walled, open
TP	Thin-walled, piston
WS	Wash sample
DT	Dual tube sample
DD	Diamond drilling

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.) split spoon sampler for a distance of 300 mm (12 in.).

Dynamic Cone Penetration Resistance (DCPT); N_d :

The number of blows by a 63.5 kg (140 lb.) hammer dropped 760 mm (30 in.) to drive an uncased 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

Cone Penetration Test (CPT):

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

III. SOIL DESCRIPTION

(a) Cohesionless Soils

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

(b) Cohesive Soils C_u or S_u

Consistency	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 or PL	Plastic limited
w_l or LL	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
DS	Direct shear test
Gs	Specific gravity
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 test (LV-laboratory vane test)
γ	Unit weight

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

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
V	volume
W	weight

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 vertical effective overburden stress
$\sigma_1 \sigma_2 \sigma_3$	principal stresses (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
*	Density symbol is ρ . Unit weight symbol is γ where $\gamma = \rho g$ (i.e. mass density multiplied by acceleration due to gravity)

(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 (overconsolidated range)
C_s	swelling index
C_α	coefficient of secondary consolidation
m_v	coefficient of volume change
c_v	coefficient of consolidation (vertical direction)
T_v	time factor (vertical direction)
U	degree of consolidation
σ'_p	pre-consolidation stress
OCR	overconsolidation ratio = σ'_p / σ'_{vo}

(d) Shear Strength

τ_p or 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 or 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

Notes: ¹ $\tau = c' + \sigma' \tan \phi'$
² shear strength = (compressive strength) / 2



RECORD OF BOREHOLE No 09-5

SHEET 1 OF 1

METRIC

PROJECT 05-1120-210-6000

G.W.P. 4058-01-00

LOCATION N 5027648.0 ; E 364476.0

ORIGINATED BY J.D.

DIST HWY 417

BOREHOLE TYPE Portable Equipment, Continuous Sampling (64kg hammer; 760mm drop with cat head)

COMPILED BY J.M.

DATUM Geodetic

DATE Jan. 4, 2010

CHECKED BY E.S.O.

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 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED									
77.7	GROUND SURFACE							20	40	60	80	100					
0.0	Sand, some silt, trace gravel (FILL) Loose Light orange-brown Moist		1	SS	8		77										
			2	SS	9												
			3	SS	4		76										
			4	SS	4												
75.3																	
2.4	SAND, some silt and clay (FILL) Compact Orange-brown Wet		5	SS	25		75										
74.7																	
3.1	SILTY CLAY to CLAY (Weathered Crust) Stiff to very stiff Grey-brown Wet		6	SS	12		74										
			7	SS	13												
			8	SS	5		73										
73.0																	
4.7	SILTY CLAY to CLAY Firm Grey Wet						72	×	+								
			9	SS	2			×	+								
							71	×	+								
			10	SS	1			×	+								
							70	×	+								
		11	SS	1				×	+								
							69	×	+								
68.6																	
9.1	Silty SAND and GRAVEL, some clay (TILL) Loose to dense Grey Wet		12	SS	7		68										
			13	SS	31												
67.3																	
10.4	End of Borehole																

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

PROJECT 05-1120-210-6000

RECORD OF BOREHOLE No 09-6

SHEET 1 OF 2

METRIC

G.W.P. 4058-01-00

LOCATION N 5027702.0 : E 364518.7

ORIGINATED BY J.C.

DIST HWY 417

BOREHOLE TYPE Power Auger, 108mm Diam. Hollow Stem (64kg hammer, 760mm drop, automatic Compactor)

DATUM Geodetic

DATE Dec. 16, 2010

CHECKED BY E.S.O.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100	20 40 60 80 100		
76.0 0.0	GROUND SURFACE Sand, some silt (FILL) Loose Mottled grey-brown to orange Wet		1	SS	7		75					
74.5 1.5	SILTY CLAY to CLAY (Weathered Crust) Stiff Brown Wet		2	SS	5		74					
			3	SS	3		73					
72.5 3.5	SILTY CLAY to CLAY Firm to stiff Light brown to grey Wet		4	SS	2		72					
			5	TP	PH		71					
			6	SS	WH		70					
			7	SS	WH		69					
67.2 8.8	Silty SAND and GRAVEL, some clay (TILL) Compact to dense Grey Wet		8	SS	26		68					
			9	SS	15		67					
65.3 10.8	SAND, some silt Compact Grey Wet		10	SS	23		66					
	SAND and GRAVEL, trace silt and clay Compact to dense Grey Wet		11	SS	44		65					
63.4 12.6	SAND and GRAVEL, some silt, trace clay (TILL) Very dense Grey to black Wet		12	SS	20		64					
			13	SS	80/0.23		63					
			14	SS	60/0.10		62					
61.5 14.5	- Contains shaley limestone fragments from 13.7m to 14.0m depth											

Continued Next Page

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

GTA-MTO 001 \\GOLDER.GDS\GALVOTTAWA\ACTIVE\2005\1120-210 MRC HWY 417 BRIDGES MAINTLAND TO ISLAND PARK DRIVE\PHASE 6000 RETAINING WALLS 7N & 8N INVESTIGATION\GINT051120210-6000-01.GPJ GAL-GTA.G

GTA-MTO 001 \\GOLDER.GDS\GALLOTTAWA\ACTIVE\2005\1120\GEOTECHNICAL\05-1120-210 MRC HWY 417 BRIDGES MAITLAND TO ISLAND PARK DRIVE\PHASE 6000 RETAINING WALLS 7N & 8N INVESTIGATION\GINT\051120210-6000-01.GPJ GAL-GTA.G

RECORD OF BOREHOLE No 09-6

SHEET 2 OF 2

METRIC

PROJECT 05-1120-210-6000
 G.W.P. 4058-01-00 LOCATION N 5027702.0 ; E 364518.7 ORIGINATED BY J.C.
 DIST HWY 417 BOREHOLE TYPE Power Auger, 108mm Diam. Hollow Stem (64kg hammer; 760mm drop, automatic sampler) COMPILED BY J.M.
 DATUM Geodetic DATE Dec. 16, 2010 CHECKED BY E.S.O.

SOIL PROFILE		SAMPLES				GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					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		
	— CONTINUED FROM PREVIOUS PAGE — End of Borehole Auger Refusal Note: Water level in open hole at 0.2 m depth (Elev. 75.8) upon completion of drilling.													

+ 3, x 3: Numbers refer to
Sensitivity

○ 3% STRAIN AT FAILURE

GTA-MTO 001 \\GOLDER.GDS\GALLOTTAWA\ACTIVE\2005\1120\210\GEOTECHNICAL\05-1120-210 MRC HWY 417 BRIDGES MAITLAND TO ISLAND PARK DRIVE\PHASE 6000 RETAINING WALLS 7N & 8N INVESTIGATION\GINT051120210-6000-01.GPJ GAL-GTA.G

PROJECT 05-1120-210-6000

RECORD OF BOREHOLE No 09-7

SHEET 1 OF 2

METRIC

G.W.P. 4058-01-00

LOCATION N 5027796.0 :E 364593.9

ORIGINATED BY J.C.

DIST HWY 417

BOREHOLE TYPE Power Auger, 108mm Diam. Hollow Stem (64kg hammer; 760mm drop, automatic)

DATUM Geodetic

DATE Dec. 18, 2009

CHECKED BY E.S.O.


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 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED		WATER CONTENT (%) W _P W W _L				
74.9	GROUND SURFACE						20 40 60 80 100			25 50 75				
0.0	Silty sand to sandy silt, with organic matter (FILL) Very loose Brown to grey Moist		1	SS	7									
74.4														
0.8	Organic SILT, trace sand Very loose Black Moist		2	SS	3									
	SILTY CLAY to CLAY (Weathered Crust) Stiff Grey-brown Wet		3	SS	5									
			4	SS	2									
71.1														
3.8	SILTY CLAY to CLAY Stiff Grey Wet		5	SS	WH									
			6	SS	WH									
	- Silt seam at 6.6m depth		7	SS	WH									
			8	SS	PM									
66.2														
8.7	Silty SAND and GRAVEL, some clay (TILL) Compact to very dense Grey Wet		9	SS	10									
			10	SS	21									
64.2														
10.7	SAND and GRAVEL, trace silt and clay Very dense Grey Wet		11	SS	49									
			12	SS	50/0.02									
61.5														
13.4	Silty SAND, some gravel, trace clay (TILL) Very dense Grey Wet		13	SS	50/0.13									
									</					

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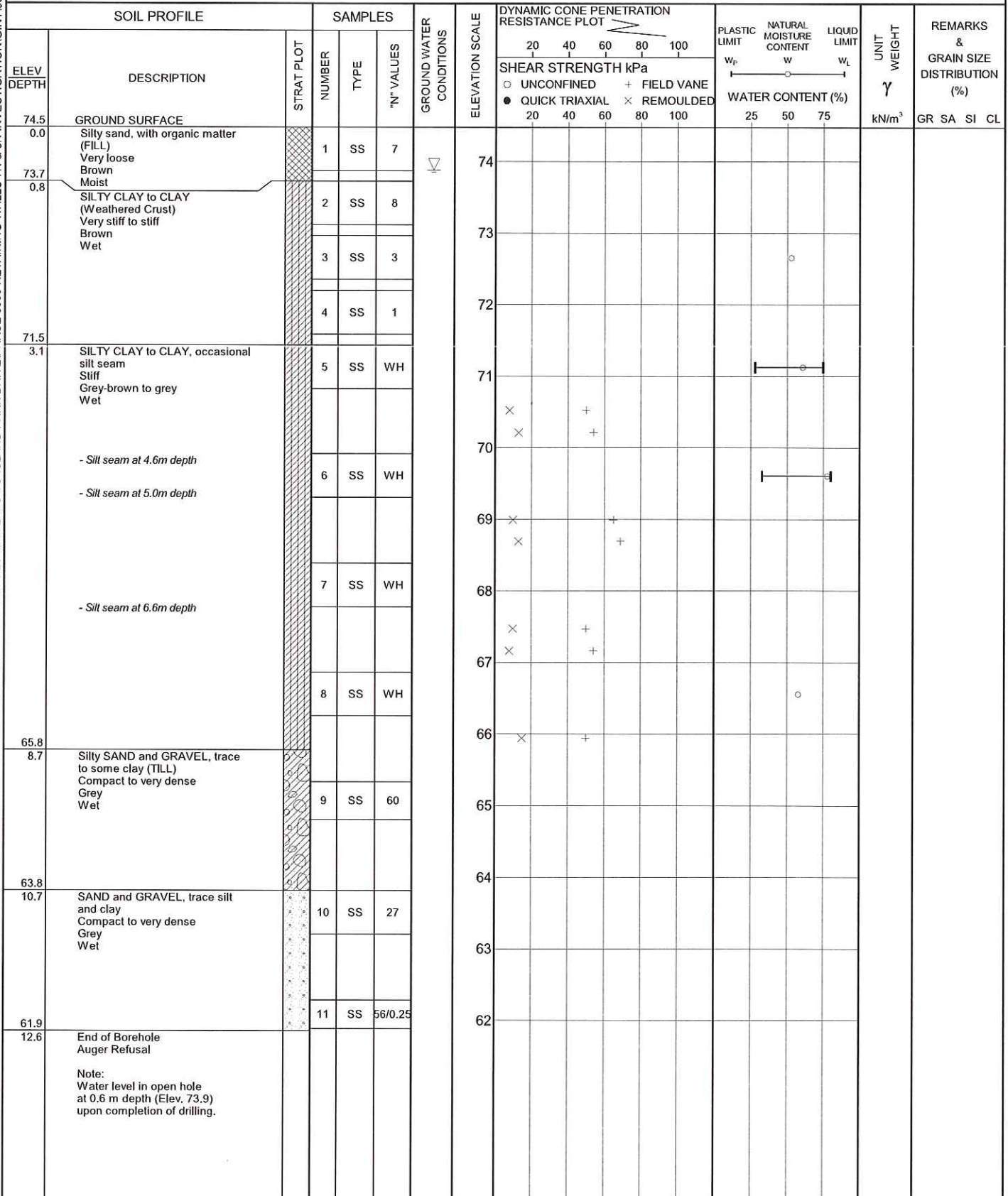
+ 3, x 3: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

GTA-MTO 001 \\GOLDER.GDS\GAL\IOTTAWA\ACTIVE\2005\1120\GEO\TECHNICAL\05-1120-210 MRC HWY 417 BRIDGES MATTLAND TO ISLAND PARK DRIVE\PHASE 6000 RETAINING WALLS 7N & 8N INVESTIGATION\GINT051120210-6000-01.GPJ GAL-GTA.G

PROJECT 05-1120-210-6000		RECORD OF BOREHOLE No 09-7		SHEET 2 OF 2		METRIC	
G.W.P. 4058-01-00		LOCATION N 5027796.0 ;E 364593.9		ORIGINATED BY J.C.			
DIST HWY 417		BOREHOLE TYPE Power Auger, 108mm Diam. Hollow Stem (64kg hammer; 760mm drop, automatic)		COMPILED BY J.M.			
DATUM Geodetic		DATE Dec. 18, 2009		CHECKED BY E.S.O.			

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
	— CONTINUED FROM PREVIOUS PAGE —															
59.4	- Dark grey-brown limestone fragments at 15.2m depth		14	SS	50/0.08											
15.5	End of Borehole Auger Refusal Note: Water level in open hole at 0.6 m depth (Elev. 74.3) upon completion of drilling.															

PROJECT 05-1120-210-6000		RECORD OF BOREHOLE No 09-8		SHEET 1 OF 1		METRIC	
G.W.P. 4058-01-00		LOCATION N 5027846.0 :E 364627.4		ORIGINATED BY J.C.			
DIST _____ HWY 417		BOREHOLE TYPE Power Auger, 108mm Diam. Hollow Stem (64kg hammer, 760mm drop, automatic sampler)		COMPILED BY J.M.			
DATUM Geodetic		DATE Dec. 21, 2009		CHECKED BY E.S.O.			



GTA-MTO 001 \\GOLDER.GDS\GALOTTAWA\ACTIVE\2005\1120\GEOTECHNICAL\05-1120-210 MRC HWY 417 BRIDGES MAINTLAND TO ISLAND PARK DRIVE\PHASE 6000 RETAINING WALLS 7N & 8N INVESTIGATION\GINT051120210-6000-01.GPJ GAL-GTA.G

PROJECT 05-1120-210-6000		RECORD OF BOREHOLE No 09-9		SHEET 1 OF 1		METRIC	
G.W.P. 4058-01-00		LOCATION N 5027883.0 ; E 364649.5		ORIGINATED BY J.C.			
DIST _____ HWY 417		BOREHOLE TYPE Power Auger, 108mm Diam. Hollow Stem (64kg hammer; 760mm drop, automatic Control)		COMPILED BY J.M.			
DATUM Geodetic		DATE Jan. 4, 2010		CHECKED BY E.S.O.			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC NATURAL LIQUID LIMIT MOISTURE CONTENT			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		W _P W W _L				
						● QUICK TRIAXIAL × REMOULDED										
						20 40 60 80 100				25 50 75						
74.4	GROUND SURFACE															
0.0	Silty sand, some gravel, with organic matter (FILL) Very loose Brown Moist		1	SS	9											
73.6																
0.8	SILTY CLAY to CLAY (Weathered Crust) Very stiff to stiff Brown Moist to wet		2	SS	11											
			3	SS	6											
			4	SS	3											
71.4																
3.1	SILTY CLAY to CLAY Firm to stiff Grey Wet		5	SS	WH											
			6	TP	WH											
			7	SS	WH											
			8	SS	WH											
65.6																
8.8	Silty SAND and GRAVEL, some clay (TILL) Dense to very dense Grey Wet		9	SS	33											
63.7																
10.7	SAND and GRAVEL, trace silt and clay Very dense Grey Wet		10	SS	62											
62.2																
12.4	Silty SAND and GRAVEL, trace to some clay (TILL) Dense to very dense Grey Wet End of Borehole Auger Refusal		11	SS	75/0.18											

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

GTA-MTO 001 \\GOLDER.GDS\GAL\IOTTAWA\ACTIVE\2005\1120\GEOTECHNICAL\05-1120-210 MRC HWY 417 BRIDGES MAITLAND TO ISLAND PARK DRIVE\PHASE 6000 RETAINING WALLS 7N & 8N INVESTIGATION\GINT051120210-6000-01.GPJ GAL-GTA.G

PROJECT <u>05-1120-210-6000</u>		RECORD OF BOREHOLE No 09-9A		SHEET 1 OF 1		METRIC	
G.W.P. <u>4058-01-00</u>		LOCATION <u>N 5027884.0 ; E 364649.5</u>		ORIGINATED BY <u>J.C.</u>			
DIST <u>HWY 417</u>		BOREHOLE TYPE <u>Power Auger, 108mm Diam. Hollow Stem (64kg hammer; 760mm drop, automatic)</u>		COMPILED BY <u>J.M.</u>			
DATUM <u>Geodetic</u>		DATE <u>Jan. 4, 2010</u>		CHECKED BY <u>E.S.O.</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						
						○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x REMOULDED									
74.4	GROUND SURFACE														
0.0	Silty sand, some gravel, with organic matter (FILL) Very loose Brown Moist						74								
73.6															
0.8	SILTY CLAY to CLAY (Weathered Crust) Very stiff to stiff Brown Moist to wet						73								
							72								
71.4															
3.1	SILTY CLAY to CLAY Firm to stiff Grey Wet						71								
							70								
69.7															
4.7	End of Borehole														
	Note: Water level in well screen at 0.7 m depth (Elev. 73.7) on Feb. 16, 2010.														

GTA-MTO 001 \\GOLDER.GDS\GAL\OTTAWA\ACTIVE\2005\1120\GEOTECHNICAL\05-1120-210 MRC HWY 417 BRIDGES MAINTLAND TO ISLAND PARK DRIVE\PHASE 6000 RETAINING WALLS 7N & 8N INVESTIGATION\GINT05\1120\210-6000-01.GPJ GAL-GTA.G

PROJECT 05-1120-210-6000		RECORD OF BOREHOLE No 09-10		SHEET 1 OF 1	METRIC
G.W.P. 4058-01-00		LOCATION N 5027922.0 ; E 364672.0		ORIGINATED BY J.C.	
DIST HWY 417		BOREHOLE TYPE Power Auger, 108mm Diam. Hollow Stem (64kg hammer, 760mm drop, automated)		COMPILED BY J.M.	
DATUM Geodetic		DATE Jan. 5, 2009		CHECKED BY E.S.O.	

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 (%)		
74.0	GROUND SURFACE																
0.0	Sandy silt, trace gravel, with organic matter (FILL) Dense Brown Moist		1	SS	42												
73.2	SILTY CLAY to CLAY (Weathered Crust) Very stiff to stiff Brown Moist to wet		2	SS	13												
0.8			3	SS	6												
			4	SS	3												
71.0	SILTY CLAY to CLAY Stiff Grey-brown Wet		5	SS	WH												
3.1																	
			6	SS	WH												
			7	SS	WH												
65.8	Silty SAND, trace to some clay and gravel (TILL) Compact to dense Grey Wet		8	SS	WH												
8.2			9	SS	24												
64.9	End of Borehole Auger Refusal																
9.1																	

[illegible]



APPENDIX B

Record of Previous Boreholes

[illegible]

Continued Next Page

+ 3, X 3. Numbers refer to Sensitivity

MS-100 001 05-1120-210-6000 GPJ GAL-MISS GDT 12/14/09

+ 3, X 3: Numbers refer to Sensitivity

MIS-MTO 001 05-1120-210-6000 GPJ GAL-MISS.GDT 12/14/09



APPENDIX C

NSSP - Cobbles and Boulders

BOULDERS/COBBLES DURING PILE INSTALLATION - Item No.

Special Provision

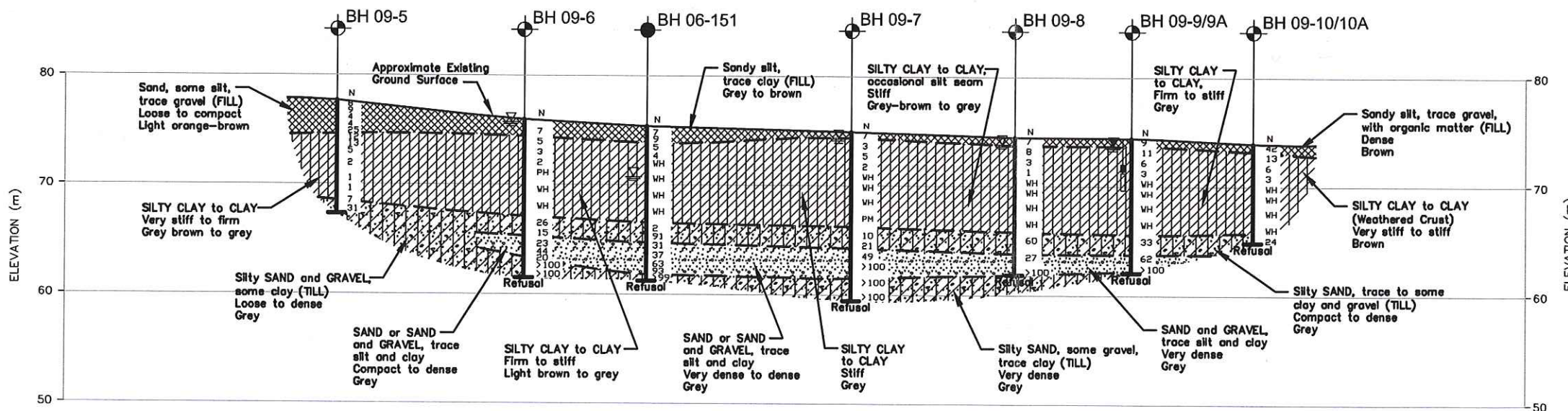
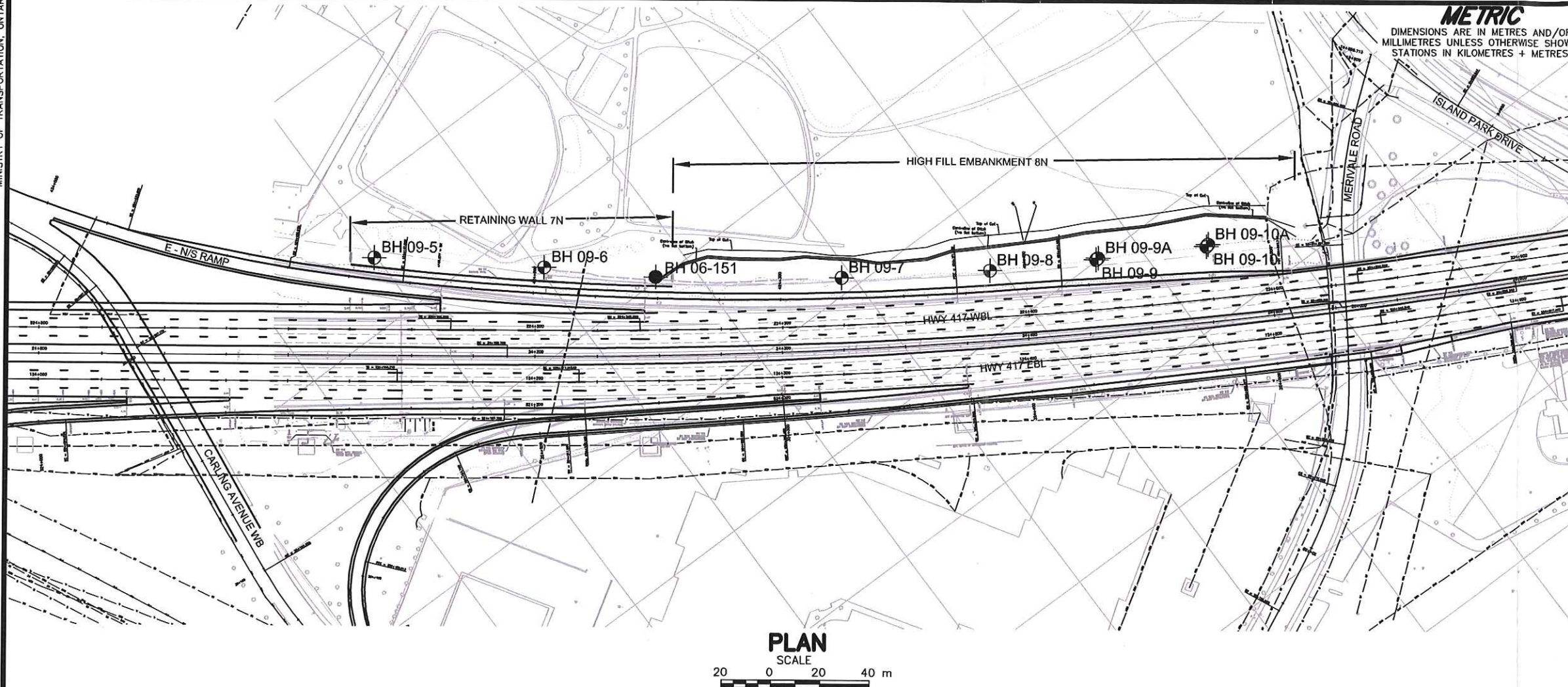
The overburden soils at the site include embankment fill and sandy silt to silty sand till containing cobbles and boulders.

Appropriate equipment and procedures will be required to penetrate/remove cobbles/boulders that are encountered during pile driving.

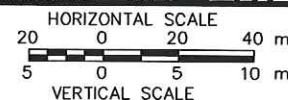
Basis of Payment

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

END OF SECTION



PROFILE ALONG RETAINING WALL 7N & EMBANKMENT 8N



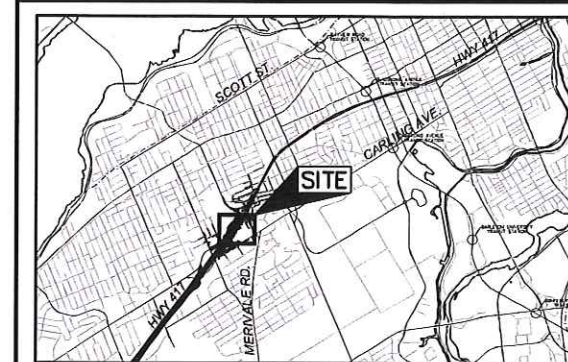
CONT No.
WP No. 4058-01-00

HIGHWAY 417
RETAINING WALL 7N AND
EMBANKMENT 8N
BOREHOLE LOCATIONS AND SOIL STRATA

SHEET



Golder Associates Ltd.
OTTAWA, ONTARIO, CANADA



KEY PLAN
SCALE 1 0 1 2 km

LEGEND

Borehole - Current Investigation

Borehole - Previous MTO Investigation
Geocres No. 31G5-218

N Standard Penetration Test Value

16 Blows/0.3m unless otherwise stated
(Std. Pen. Test, 475 J/blow)

WL in piezometer, Feb. 16, 2010SealPiezometer

No.	ELEVATION	CO-ORDINATES	
		NORTHING	EASTING
09-5	77.7	5027648.0	364476.0
09-6	76.0	5027702.0	364518.7
09-7	74.9	5027796.0	364593.9
09-8	74.5	5027846.0	364627.4
09-9	74.4	5027883.0	364649.5
09-9A	74.4	5027884.0	364649.5
09-10	74.0	5027922.0	364672.0
09-10A	74.0	5027923.0	364671.7
06-151	75.4	5027735.6	364548.5

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 Preliminary Design Report.

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

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

REFERENCE

Base plan provided in digital format by MRC (Drawing File No. "7N&8N for Golder-nov2009.dwg", received Nov. 17, 2009.

NO.	DATE	BY	REVISION
Geocres No. 31G5-269			
HWY. 417	PROJECT NO. 05-1120-210	DIST.	
SUBM'D. ESO	CHKD. WC	DATE: JAN. 20, 2011	SITE:
DRAWN: JM	CHKD. WC	APPD. FJH	DWG. 1