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REPORT ON

**FOUNDATION INVESTIGATION AND DESIGN REPORT
RETAINING WALLS
MAITLAND AVE TO ISLAND PARK DRIVE
HIGHWAY 417
W.P. 4058-01-00**

Submitted to:

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PART A

**FOUNDATION INVESTIGATION REPORT
RETAINING WALLS
MAITLAND AVE TO ISLAND PARK DRIVE
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 five bridges on Highway 417 in the City of Ottawa. The section of Highway 417 included in this assignment (W.P. 4058-01-00) extends from Maitland Avenue to Island Park Drive.

Foundation investigation services are required for the following components under W.P. 4058-01-00:

- Bridge widenings at Clyde Avenue, Carling Avenue Eastbound (EB), Kirkwood Avenue, Carling Avenue Westbound (WB), and Merivale Road.
- Eighteen retaining walls were identified in the RFP based on the outcome of the environmental assessment. As the design of the widening has progressed, retaining walls have been combined at some locations, eliminated at other locations and added at other locations. The discussions in this report reflect the design requirements identified at the time of the report.

This report addresses the proposed addition and/or replacement of 16 retaining walls, which may also include reinforced earth slopes, along the widened alignment.

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 Quality Control Plan for this project dated December 7, 2005.

2.0 SITE DESCRIPTION

Highway 417, from Maitland Avenue to Island Park Drive, is typically six lanes wide (i.e., three lanes in each of the westbound and eastbound directions). The surrounding land use is primarily commercial, although some residential housing borders the highway on the north side near Maitland Avenue.

From Maitland Avenue to about 500 m east, the highway has been constructed in a shallow cut (1 to 2 m deep). The remainder of Highway 417, from about 500 m east of Maitland Avenue to Island Park Drive, is elevated 3 to 6 m above the surrounding ground. The cut and embankments are typically sloped at 2H:1V or flatter.

An existing concrete retaining wall, west of Merivale Avenue and behind the Westgate Mall on the south side of the highway, is some 2 to 3 m in height.

A major overhead hydro electric transmission corridor extends parallel to Highway 417 on the south side. The hydro towers for these overhead wires may be within the zone of influence of the widened embankments and/or retaining walls. It is understood that the towers are founded on steel raft foundations, some 6 to 7 m in plan dimension, at depths of about 3 m below ground surface.

There are also a number of subsurface utilities, such as storm sewers, sanitary sewers and watermain, potentially within the zone of influence of the widening. It is understood that the 1220 mm diameter watermain on the south side of highway will be moved prior to construction of the widening and that the impacts on this utility need not be considered during the geotechnical design.

3.0 INVESTIGATION PROCEDURES

The field work exclusive to the retaining wall subsurface investigation was carried out between July 5 and August 9, 2006. At that time, forty-five boreholes (Boreholes 06-101 to 06-145, inclusive) were put down at the locations shown on Drawings 1 to 16 in Appendix A. Proposed borehole 06-131 was not drilled as the underground services could not be located due to the extensive bushes in the vicinity which obscured the line of sight between manholes. An additional borehole (Borehole 06-143A) was put down on February 2, 2007, at the location of retaining wall 11S (see Drawing 15). The boreholes were drilled along the originally proposed retaining wall alignments.

Field work for the bridge foundation investigation was also carried out between May 8 and 26, 2006. At that time, eighteen boreholes (Boreholes 06-1 to 06-18, inclusive) were put down at the approximate location of the ends of the proposed bridge abutment widenings. Twelve of these boreholes (Boreholes 06-1 to 06-7, 06-9, 06-12, 06-14, 06-17 and 06-18) are also located at the ends of some of the retaining walls, as shown on Drawings 1 to 16 in Appendix A, and are correspondingly also discussed within this report.

The majority of the boreholes were advanced using track and truck mounted drill rigs supplied and operated by Marathon Drilling Company Ltd. of Ottawa, Ontario. Some of the boreholes were advanced using a portable drilling rig supplied and operated by OGS Drilling Inc. of Ottawa, Ontario due to limited access or height restrictions.

The boreholes were advanced to depths which vary from 0.9 to 15.1 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 unit.

In addition, six 75-millimetre diameter thin-walled relatively undisturbed Shelby tube samples of silty clay soils were obtained from Boreholes 06-104, 06-139, 06-140, 06-141, 06-142 and 06-143 using a fixed piston sampler. Three of these samples (from Boreholes 06-140, 06-141 and 06-143) were submitted for laboratory oedometer consolidation testing to assess the consolidation characteristics of the silty clay. Two additional 75-millimetre diameter thin-walled relatively undisturbed Shelby tube samples of silty clay were obtained at the location of Borehole 06-143A, drilled specifically for this purpose. One of these additional samples was submitted for laboratory oedometer consolidation testing to assess the consolidation characteristics of the silty clay.

The bedrock was cored at Boreholes 06-106, 06-109, 06-110, 06-113, and 06-114 for depths ranging from 1.0 to 1.4 m, after practical refusal to augering had been reached. The bridge

foundation boreholes were cored, for depths ranging from 3.0 to 4.6 m, after practical refusal to augering had been reached.

The water levels in the open boreholes were observed throughout the drilling operations. Monitoring wells were installed in retaining wall Boreholes 06-104, 06-132, 06-139 and 06-143. The monitoring wells consisted of 50-millimetre diameter PVC pipe, with 1.5 metre lengths of #10 slot screen.

Standpipes were installed in bridge foundation Boreholes 06-2, 06-5, 06-12, 06-14 and 06-18. The standpipes consisted of 20 mm outside diameter HDPE tubing with a 0.6 m long slotted tip.

The boreholes were backfilled with bentonite mixed with soil cuttings.

The site conditions were restored following completion of the field work. The monitoring wells and standpipes were completed at ground surface by means of a flush-mounted protective casing.

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' laboratory in Ottawa for further examination, and to Golder Associates' laboratory in Mississauga for testing. Index and classification tests consisting of water content determinations, Atterberg Limit testing, organic content and grain size distribution analyses were carried out on selected soil samples.

The groundwater level in the monitoring wells was measured on August 22, 2006. The groundwater level in the standpipes was measured on June 12, 2006.

The borehole locations were assessed by Golder relative to existing site features. The borehole elevations were established by MRC from a digital terrain model based on the relative locations provided by Golder. The borehole locations, including MTM NAD83 northing and easting coordinates and ground surface elevations referenced to geodetic datum, are summarized in the following table and Drawings 1 to 16 in Appendix A.

Retaining Wall 1S (Drawing 1)

Borehole Number	MTM NAD83 Northing (m)	MTM NAD83 Easting (m)	Ground Surface Elevation (m)
06-101	5026067.6	363365.3	81.4
06-102	5026120.1	363405.1	81.5
06-103	5026171.3	363444.9	81.5
06-104	5026216.5	363479.2	81.3
06-105	5026287.1	363534.4	80.1

Retaining Wall 2S & 3S (Drawing 2)

Borehole Number	MTM NAD83 Northing (m)	MTM NAD83 Easting (m)	Ground Surface Elevation (m)
06-106	5026391.4	363609.8	80.2
06-107	5026425.6	363637.2	80.0
06-108	5026464.8	363664.9	79.5
06-4	5026487.2	363672.2	78.4

Retaining Wall 4S (Drawing 3)

Borehole Number	MTM NAD83 Northing (m)	MTM NAD83 Easting (m)	Ground Surface Elevation (m)
06-3	5026503.3	363683.4	77.8
06-111	5026553.2	363730.9	78.5
06-112	5026609.8	363772.9	78.4
06-113	5026715.8	363852.9	78.6
06-114	5026721.5	363859.6	78.4

Retaining Wall 5S (Drawing 4)

Borehole Number	MTM NAD83 Northing (m)	MTM NAD83 Easting (m)	Ground Surface Elevation (m)
06-118	5026822.1	363932.7	78.3
06-119	5026871.9	363967.1	77.5
06-120	5026910.7	363997.9	77.5
06-121	5026930.8	364030.6	76.8
06-122	5026953.6	364053.4	77.3

Retaining Wall 6S (Drawings 5A & 5B)

Borehole Number	MTM NAD83 Northing (m)	MTM NAD83 Easting (m)	Ground Surface Elevation (m)
06-6	5027111.6	364143.8	75.1
06-127	5027168.4	364215.6	75.5
06-128	5027230.8	364247.1	75.2
06-129	5027295.3	364293.1	75.5
06-10	5027364.2	364331.9	75.0

Retaining Wall 7S (Drawing 6)

Borehole Number	MTM NAD83 Northing (m)	MTM NAD83 Easting (m)	Ground Surface Elevation (m)
06-9	5027378.2	364347.3	75.2
06-133	5027391.6	364366.9	76.6

Retaining Wall 8S (Drawing 7)

Borehole Number	MTM NAD83 Northing (m)	MTM NAD83 Easting (m)	Ground Surface Elevation (m)
06-134	5027468.4	364419.8	78.3
06-135	5027495.7	364431.5	80.3

Retaining Wall 9S (Drawing 8)

Borehole Number	MTM NAD83 Northing (m)	MTM NAD83 Easting (m)	Ground Surface Elevation (m)
06-136	5027479.3	364439.8	78.8
06-137	5027507.3	364454.5	79.3
06-14	5027540.4	364478.1	75.2

Retaining Wall 11S (Drawings 9A & 9B)

Borehole Number	MTM NAD83 Northing (m)	MTM NAD83 Easting (m)	Ground Surface Elevation (m)
06-138	5027626.2	364561.4	76.2
06-139	5027686.0	364595.5	75.1
06-140	5027733.3	364607.9	74.3
06-141	5027807.7	364671.7	74.7
06-142	5027865.9	364708.2	74.7
06-17	5027932.3	364740.8	74.1

Retaining Wall 12S (Drawing 10)

Borehole Number	MTM NAD83 Northing (m)	MTM NAD83 Easting (m)	Ground Surface Elevation (m)
06-18	5027945.9	364749.6	73.7
06-143	5027983.9	364775.9	73.4
06-143A	5027980.8	364776.0	73.4
06-144	5028028.8	364786.5	73.5

Retaining Wall 1N (Drawing 11)

Borehole Number	MTM NAD83 Northing (m)	MTM NAD83 Easting (m)	Ground Surface Elevation (m)
06-109	5026424.4	363566.5	79.2
06-110	5026487.7	363611.9	78.5
06-1	5026530.9	363653.6	77.7

Retaining Wall 2N (Drawing 12)

Borehole Number	MTM NAD83 Northing (m)	MTM NAD83 Easting (m)	Ground Surface Elevation (m)
06-2	5026548.7	363666.6	78.1
06-115	5026603.2	363701.7	78.6
06-116	5026659.1	363742.4	77.6
06-117	5026711.2	363783.3	77.5

Retaining Wall 3N (Drawing 13)

Borehole Number	MTM NAD83 Northing (m)	MTM NAD83 Easting (m)	Ground Surface Elevation (m)
06-123	5026903.5	363931.2	77.3
06-124	5026959.0	363969.2	76.8
06-125	5027012.6	364010.7	76.9
06-126	5027053.6	364048.1	80.6
06-7	5027104.4	364078.1	75.8

Retaining Wall 4N (Drawing 14)

Borehole Number	MTM NAD83 Northing (m)	MTM NAD83 Easting (m)	Ground Surface Elevation (m)
06-5	5027120.8	364090.8	75.6
06-130	5027152.5	364110.2	79.0

Retaining Wall 5N (Drawing 15)

Borehole Number	MTM NAD83 Northing (m)	MTM NAD83 Easting (m)	Ground Surface Elevation (m)
06-132	5027341.4	364260.1	75.4
06-12	5027402.2	364309.7	75.3

Retaining Wall 6N (Drawing 16)

Borehole Number	MTM NAD83 Northing (m)	MTM NAD83 Easting (m)	Ground Surface Elevation (m)
06-145	5027219.6	364171.3	78.0

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 that lies 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 subsurface investigation at this site, forty-six boreholes were advanced along the proposed retaining wall alignments for the proposed highway 417 widening. Twelve of the eighteen boreholes drilled for the bridge foundation investigation are also located at the ends of some of the retaining wall alignments and are considered herein. The borehole locations and ground surface elevations as well as the soil stratigraphy sections projected along the retaining wall alignments are shown on Drawings 1 to 16 in Appendix A.

The detailed subsurface soil, bedrock, 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 B and on Figures 1 to 33 in Appendix C. 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 consist of topsoil and fill materials overlying native soils ranging from silty clay to clay at the western project limit (Maitland Avenue) and the eastern project limit (Island Park Drive) to sands and glacial till over the middle portion of the alignment, from about Clyde Avenue to Kirkwood Avenue.

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.

These overburden materials are underlain by limestone to dolomitic limestone bedrock.

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

4.2.1 Retaining Wall 1S (EBL 22+150 to 22+475)

Boreholes 06-101 to 06-105, inclusive were advanced along the alignment of retaining wall 1S. The borehole locations and ground surface elevations as well as the soil stratigraphy section projected along retaining wall 1S are shown on Drawing 1 in Appendix A.

4.2.1.1 Topsoil, Pavement Structure and Fill Material

A topsoil layer was encountered at ground surface at boreholes 06-101 and 06-102 with thicknesses of about 0.2 and 0.5 m, respectively.

Boreholes 06-103 and 06-104 were drilled through the pavement structure of the existing parking lot adjacent to the proposed retaining wall location. The pavement structure consists of about 40 to 100 mm of asphaltic concrete underlain by about 0.3 m of sand and gravel/sandy gravel granular base materials.

The pavement structure at boreholes 06-103 and 06-104 is underlain by fill material, about 0.5 and 1.7 m in thickness, respectively. The fill material consists primarily of silty clay, with traces of sand and gravel at borehole 06-104. A thin layer of fine sand fill was also encountered within the silty clay fill at borehole 06-104. Two standard penetration test (SPT) "N" values in the silty clay fill of 3 and 6 blows per 0.3 m of penetration indicate this material to be stiff in consistency.

Crushed stone fill with cobbles was encountered extending to a depth of about 0.9 m from the ground surface at borehole 06-105.

4.2.1.2 Sand

A layer of fine sand with varying amounts of silt and gravel underlies the topsoil layer at boreholes 06-101 and 06-102 and the fill material at borehole 06-103. This layer is between 0.2 and 0.7 m thick. A measured SPT "N" value of 11 blows per 0.3 m of penetration indicates the deposit to have a compact relative density.

4.2.1.3 Silty Clay

The sand and fill materials are underlain by a deposit of silty clay. The upper portion of this deposit has been weathered to a grey brown crust and extends to depths between about 2.3 and 3.1 m below the existing ground surface. The measured SPT "N" values in this deposit ranged

from 2 to 14 blows per 0.3 m of penetration. These test results indicate that the weathered silty clay has a stiff to very stiff consistency.

The silty clay below the depth of weathering is grey in colour. The unweathered silty clay was not fully penetrated in Boreholes 06-101 to 06-104, inclusive, but was proven for depths which vary from 4.3 to 5.8 m below the existing ground surface. At Borehole 06-105 the deposit was fully penetrated to a depth of 3.3 m (i.e., elevation 76.8 m.). The results of in situ vane testing in this material gave undrained shear strengths ranging from 31 to 61 kilopascals, indicating a firm to stiff consistency. The measured natural water content of several samples of the unweathered silty clay varies from 55 to 61 percent.

4.2.1.4 Glacial Till

The unweathered silty clay at Borehole 06-105 is underlain by a deposit of glacial till which was proven to a depth of about 3.5 m. Based on local experience and observations of the drilling resistance, the glacial till likely consists of a heterogeneous mixture of gravel, cobbles, and boulders in a matrix of sandy silt, with a trace of clay.

4.2.1.5 Sampler Refusal

Practical refusal to advancement of the sampler was encountered at Borehole 06-105 at a depth of 3.5 m (i.e., elevation 76.6 m) below the existing ground surface. Sampler refusal may indicate the bedrock surface; or, it could also represent cobbles and/or boulders within the glacial till.

4.2.1.6 Groundwater

A monitoring well was installed in borehole 06-104 and sealed within the silty clay. The water level measured in that monitoring well is summarized in the following table:

Borehole No.	Retaining Wall	Date	Depth (m)	Elevation (m)
06-104	1S	August 22, 2006	2.2	79.1

The groundwater levels in boreholes 06-101, 06-102, 06-103 and 06-105 ranged in depth from about 2.4 to 3.4 m below the existing ground surface during the short time they remained open after completion of drilling operations.

Groundwater levels are expected to fluctuate seasonally. Higher groundwater levels are expected during wet periods of the year.

4.2.2 Retaining Walls 2S & 3S (EBL 22+600 to 22+715)

Boreholes 06-106 to 06-108, inclusive, and Borehole 06-4 from the bridge investigation were advanced along the alignment of retaining walls 2S and 3S. The borehole locations and ground surface elevations as well as the soil stratigraphy sections projected along retaining walls 2S & 3S are shown on Drawing 2 in Appendix A.

4.2.2.1 Topsoil and Fill Material

Topsoil fill was encountered at ground surface at Boreholes 06-106 and 06-4 with thicknesses of about 0.6 and 0.2 m, respectively.

The topsoil at Boreholes 06-106 and 06-4 is underlain by fill material which is about 0.3 and 0.7 m thick, respectively. The fill material consists of either silty sand or sand. Cobbles and boulders were encountered in the fill material at borehole 06-106. A measured SPT "N" value of greater than 36 blows per 0.3 m of penetration indicates the deposit to have a dense consistency.

Crushed stone fill material was encountered at ground surface at boreholes 06-107 and 06-108 with thicknesses of 0.2 and 0.1 m, respectively. The fill material at Borehole 06-108 is underlain by a layer of topsoil about 0.5 m in thickness.

4.2.2.2 Silty Clay

The topsoil at Borehole 06-108 and the fill material at Borehole 06-4 are underlain by a deposit of silty clay extending to depths of about 2.3 and 1.5 m below the existing ground surface, respectively. The deposit has been weathered to a grey brown crust. The measured SPT "N" values in this deposit ranged between 5 and 12 blows per 0.3 m of penetration. These test results indicate that the weathered silty clay has a very stiff consistency.

The results of Atterberg limit testing on one selected sample of the weathered silty clay from Borehole 06-4 indicate a plasticity index of 26 percent and a liquid limit of 48 percent. These results, summarized on the plasticity chart on Figure 1 in Appendix C, confirm that this material is of medium plasticity. The measured natural water contents of two samples of the weathered silty clay were 18 and 33 percent.

4.2.2.3 Glacial Till

The fill materials at Boreholes 06-106 and 06-107 and the silty clay at Boreholes 06-108 and 06-4 are underlain by a deposit of glacial till which extends to depths ranging from 1.8 to 3.0 m below the existing ground surface. The glacial till consists of a heterogeneous mixture of gravel, cobbles, and boulders in a matrix of sandy silt or sand, with a trace to some clay. The results of

grain size distribution testing of two samples of the glacial till are provided on Figure 2 in Appendix C. It should be noted however that this sample was retrieved using a 50 mm diameter sampler and therefore the results do not reflect the cobble and boulder content of the deposit.

The measured SPT “N” values range from 19 to greater than 100 blows per 0.3 m of penetration and indicate compact to very dense states of packing. The higher “N” values may also reflect the cobbles and boulders in the deposit.

4.2.2.4 Auger Refusal

Practical refusal to augering was encountered at boreholes 06-107 and 06-108 at depths of about 2.3 and 3.0 m, respectively, below the existing ground surface. Auger refusal may indicate the bedrock surface; however, it could also represent cobbles and/or boulders within the glacial till.

4.2.2.5 Limestone Bedrock

Limestone bedrock was encountered at Boreholes 06-106 and 06-4.

The following table summarizes the bedrock surface depths and elevations as encountered at the locations of boreholes 06-106, which was drilled with a portable drill rig, and 06-4. The bedrock consists of limestone.

Borehole Number	Ground Surface Elevation (m)	Depth to Bedrock (m)	Bedrock Surface Elevation (m)
06-106	80.2	2.1	78.1
06-4	78.4	1.8	76.6

The limestone bedrock at the site is a member of the Gull River Formation; it is medium-strong and thinly- to medium-bedded and is generally unweathered. Rock Quality Designation (RQD) values measured on recovered bedrock core samples typically ranged from about 19 to 100 percent, indicating very poor to excellent quality rock, generally increasing with depth. The lowest RQD values were recorded for the upper metre of rock in Borehole 06-4. The discontinuities observed in the rock core are typically horizontal to sub-horizontal, associated with the bedding planes, although some vertical fracturing was noted in the upper bedrock at Borehole 06-4. A description of some of the terms used in the description of the bedrock samples from this site is provided on the *Lithological and Geotechnical Rock Description Terminology* sheet which precedes the Record of Borehole sheets included with this report in Appendix B.

4.2.2.6 Groundwater

The groundwater level in borehole 06-108 was 2.4 m below the existing ground surface during the short time it remained open after completion of drilling operations. The water level in Borehole 06-2 in the north-west corner of the Clyde Avenue Bridge was at 3.1 m depth (i.e., Elevation 75.0) on June 12, 2006.

Groundwater levels are expected to fluctuate seasonally. Higher groundwater levels are expected during wet periods of the year.

4.2.3 Retaining Wall 4S (EBL 22+745 to 23+020)

Bridge investigation Borehole 06-3 and Boreholes 06-111 to 06-114, inclusive, were advanced along the alignment of retaining wall 4S. The borehole locations and ground surface elevations as well as the soil stratigraphy section projected along retaining wall 4S are shown on Drawing 3 in Appendix A.

4.2.3.1 Topsoil and Fill Material

Topsoil was encountered at ground surface at Borehole 06-3 extending to a depth of about 0.2 m below existing ground surface. Silty sand and sand with gravel, silt and clay fill materials underlie the topsoil and extend to a depth of about 1.1 m. A measured SPT "N" value of 10 blows per 0.3 m of penetration indicates a compact state of packing. The results of grain size distribution testing on one selected sample of the fill materials are provided on Figure 3.

At boreholes 06-111 to 06-114, inclusive, crushed stone fill extends from the parking lot surface to depths ranging from about 0.1 to 0.6 m. The crushed stone surface is underlain by fill materials composed of silty sand with gravel at Borehole 06-113 which is about 0.3 m in thickness.

4.2.3.2 Sand

A deposit of sand was encountered beneath the fill material at borehole 06-111 with a thickness of about 0.5 m.

4.2.3.3 Clayey Silt

The fill material at borehole 06-112 is underlain by a deposit of clayey silt with a thickness of approximately 1.5 m. A measured SPT "N" value of 9 blows per 0.3 m of penetration indicates a stiff consistency. The measured natural water content of one sample of the clayey silt was 22 percent.

4.2.3.4 Glacial Till

The fill material, sand or clayey silt where encountered is underlain by a deposit of glacial till which extends to depths between 1.3 to 2.5 m below the existing ground surface. The glacial till consists of a heterogeneous mixture of gravel, cobbles, and boulders in a matrix of sandy silt or silt, with a trace to some clay. The results of grain size distribution testing on one selected sample of the glacial till are provided on Figure 4. It should be noted however that this sample was retrieved using a 50 mm diameter sampler and therefore the results do not reflect the cobble and boulder content of the deposit.

The measured SPT "N" values ranged from 3 to greater than 100 blows per 0.3 m of penetration indicating a very loose to very dense state of packing. The measured natural water contents of three samples of the glacial till ranged from 6 to 18 percent.

4.2.3.5 Auger Refusal

Practical refusal to augering was encountered at boreholes 06-111 and 06-112 at depths of about 1.3 and 2.5 m, respectively, below the existing ground surface. Auger refusal may indicate the bedrock surface; however, it could also represent cobbles and/or boulders within the glacial till.

4.2.3.6 Limestone Bedrock

Limestone bedrock was encountered at Boreholes 06-3, 06-113 and 06-114.

The following table summarizes the bedrock surface depths and elevations as encountered at the locations of Boreholes 06-113 and 06-114, which were drilled with a portable drill rig, and at Borehole 06-3. The bedrock consists of limestone.

Borehole Number	Ground Surface Elevation (m)	Depth to Bedrock (m)	Bedrock Surface Elevation (m)
06-3	77.8	1.8	76.0
06-113	78.6	1.7	76.9
06-114	78.4	1.7	76.7

The limestone bedrock at the site is a member of the Gull River Formation; it is medium-strong and thinly- to medium-bedded and is generally unweathered, with the exception of the upper 0.4 m of bedrock at Borehole 06-114. Rock Quality Designation (RQD) values measured on recovered bedrock core samples typically ranged from about 0 to 100 percent, indicating very poor to excellent quality rock, generally increasing with depth. The lowest RQD values were recorded for the upper 1 metre of rock in Borehole 06-3 and the upper 0.4 m of bedrock in Borehole 06-114. The discontinuities observed in the rock core are typically horizontal to sub-

horizontal, associated with the bedding planes, although some vertical fracturing was noted in the upper bedrock at Boreholes 06-3 and 06-114. A description of some of the terms used in the description of the bedrock samples from this site is provided on the *Lithological and Geotechnical Rock Description Terminology* sheet which precedes the Record of Borehole sheets included with this report in Appendix B.

4.2.3.7 Groundwater

Boreholes 06-3, 06-111 and 06-112 remained dry during the short time they remained open after completion of drilling operations.

Groundwater levels are expected to fluctuate seasonally. Higher groundwater levels are expected during wet periods of the year.

4.2.4 Retaining Wall 5S (EBL 23+145 to 23+325)

Boreholes 06-118 to 06-122, inclusive, were advanced along the alignment of retaining wall 5S. The borehole locations and ground surface elevations as well as the soil stratigraphy section projected along retaining wall 5S are shown on Drawing 4 in Appendix A.

4.2.4.1 Topsoil, Pavement Structure and Fill Material

A topsoil layer was encountered at ground surface at Boreholes 06-119 and 06-120 with thicknesses of about 0.1 m.

Boreholes 06-121 and 06-122 were drilled through the pavement structure. The pavement structure consists of about 50 mm of asphaltic concrete underlain by about 0.3 m of crushed stone granular base. Crushed stone fill material was encountered at ground surface at Borehole 06-118 with a thickness of 0.1 m.

Buried topsoil layers were encountered at boreholes 06-118 and 06-122, below the crushed stone fill/base, with thicknesses of about 0.6 and 0.3 m, respectively.

The topsoil layer at borehole 06-120 is underlain by a sandy silt fill material with gravel and organic matter. This deposit is about 1.0 m in thickness.

4.2.4.2 Glacial Till

The topsoil, fill material, and/or pavement structure are underlain by a deposit of glacial till extending to depths ranging between 1.5 and 3.7 m below the existing ground surface. The glacial till consists of a heterogeneous mixture of gravel, cobbles, and boulders in a matrix of

sandy silt, with a trace clay. The results of grain size distribution testing of two samples of the glacial till are provided on Figure 5 in Appendix C. It should be noted however that these samples were retrieved using a 50 mm diameter sampler and therefore the results do not reflect the cobble and boulder content of the deposit.

The measured SPT "N" values range from 6 to 73 blows per 0.3 m of penetration indicating a loose to very dense consistency.

4.2.4.3 Auger Refusal

Practical refusal to augering was encountered at boreholes 06-118 to 06-121 at depths ranging from about 1.8 and 3.7 m, respectively, below the existing ground surface. Auger refusal may indicate the bedrock surface; however, it could also represent cobbles and/or boulders within the glacial till.

4.2.4.4 Groundwater

Boreholes 06-118 to 06-122, inclusive, remained dry during the short time they remained open after completion of drilling operations.

Groundwater levels are expected to fluctuate seasonally. Higher groundwater levels are expected during wet periods of the year, such as spring.

4.2.5 Retaining Wall 6S (EBL 23+510 to 23+818)

Bridge investigation Boreholes 06-6 and 06-10 and Boreholes 06-127 to 06-129, inclusive, were advanced along the alignment of retaining wall 6S. The borehole locations and ground surface elevations as well as the soil stratigraphy section projected along retaining wall 6S are shown on Drawings 5A and 5B in Appendix A.

4.2.5.1 Pavement/Sidewalk Structure and Fill Material

Borehole 06-6 was advanced through the pavement structure of Carling Avenue Eastbound, at the west end of the proposed retaining wall. The pavement structure at that location consists of about 50 mm of asphaltic concrete underlain by about 280 mm of Portland cement (concrete). The asphaltic concrete and concrete are in turn underlain by a thin layer of crushed stone base course material about 80 mm in thickness.

Borehole 06-10 was advanced through the sidewalk along Kirkwood Avenue at the east end of the proposed retaining wall. The pavement structure for the sidewalk at that location consists of about 180 mm of concrete underlain by about 70 mm of crushed stone base material.

Fill material was encountered at ground surface at Boreholes 06-127, 06-128 and 06-129 and beneath the pavement and sidewalk structure at Boreholes 06-6 and 06-10. The fill material varies in thickness from about 0.3 to 1.5 m. The composition of the fill material ranges from fine sand to sandy silt with varying quantities of gravel and organic matter. The sandy fill materials at Borehole 06-10 are underlain by silty clay fill about 1.3 m in thickness. Grain size distribution test results obtained from one sample of the sand fill at Borehole 06-129 are shown on Figure 6.

One SPT "N" value in the silty clay fill of 2 blows per 0.3 m of penetration indicates this material to be stiff. SPT "N" values for the sandy silt fill range from 23 to 27 blows per 0.3 m of penetration indicating a compact state of packing.

4.2.5.2 Peat

The fill materials at Boreholes 06-6, 06-127 and 06-128 are underlain by a deposit of peat ranging between 0.1 to 0.8 m in thickness. The natural water content of one sample of the peat was 123 percent as measured on a dry weight basis.

4.2.5.3 Silty Sand to Sand

The peat deposit at Borehole 06-128 is underlain by a layer of silty sand with a thickness of about 1.2 m. One standard penetration test N value in the silty sand of 8 blows per 0.3 m of penetration indicates a loose state of packing.

The fill material at Borehole 06-129 is underlain by a thin layer of fine sand with a thickness of approximately 0.3 m.

4.2.5.4 Clayey Silt and Silty Clay

The peat deposit at Borehole 06-127 is underlain by a layer of clayey silt overlying a silty clay layer, with thicknesses of 0.5 and 0.2 m, respectively. The measured natural water content of one sample of the silty clay was 50 percent.

A 0.6 m thick layer of silty clay, which has been weathered to a very stiff consistency, underlies the fine sand deposit at Borehole 06-129.

4.2.5.5 Glacial Till

The peat, silty clay, silty sand and fill material, where encountered, are underlain by a glacial till deposit. The glacial till at Boreholes 06-127 to 06-129, was not fully penetrated but was proven to depths ranging from about 3.5 to 3.7 m. The deposit was fully penetrated at boreholes 06-6 and 06-10 at depths of 2.4 and 3.4 m, respectively, below the existing ground surface. The glacial till consists of a heterogeneous mixture of gravel, cobbles, and boulders in a matrix of sandy silt or

sandy gravel, with a trace of clay. The results of grain size distribution testing of one sample of the glacial till from Borehole 06-6 are provided on Figure 8 in Appendix C. It should be noted however that this sample was retrieved using a 50 mm diameter sampler and therefore the results do not reflect the cobble and boulder content of the deposit. The measured natural water content of one sample of the glacial till was 5 percent.

The measured SPT "N" values range from 1 to greater than 70 blows per 0.3 m of penetration indicating a very loose to very dense state of packing. The higher "N" values may also be representative of the cobbles and boulders in the deposit.

4.2.5.6 Sand to Sandy Gravel

The glacial till deposit at Borehole 06-10 is underlain by sand to sandy gravel deposit which was proven to a depth of 7.9 m a thickness of about 4.5 m. The results of grain size distribution testing of two samples of the sand to sandy gravel deposit are provided on Figure 7 in Appendix C. The measured natural water content of two samples of the sand and sandy gravel deposit was 11 percent.

The measured SPT "N" values vary from 20 to greater than 100 blows per 0.3 m of penetration indicating a compact to very dense state of packing.

4.2.5.7 Auger Refusal

Practical refusal to augering was encountered at Borehole 06-127 at a depth of about 3.5 m below the existing ground surface. Auger refusal may indicate the bedrock surface however it could also represent cobbles and/or boulders within the glacial till.

4.2.5.8 Limestone and Dolomitic Limestone Bedrock

Limestone bedrock was encountered at Boreholes 06-6 and 06-10. At Borehole 06-10 a layer of dolomitic limestone bedrock, about 1.1 m in thickness, exists within the limestone bedrock.

The following table summarizes the bedrock surface depths and elevations as encountered at the locations of Boreholes 06-6 and 06-10.

Borehole Number	Ground Surface Elevation (m)	Depth to Bedrock (m)	Bedrock Surface Elevation (m)
06-6	75.1	2.4	72.7
06-10	75.0	7.9	67.1

The limestone and dolomitic limestone bedrock at the site is a member of the Gull River Formation; it is medium-strong and thinly- to medium-bedded. The upper 1.5 m of bedrock at Borehole 06-6 is weathered. Rock Quality Designation (RQD) values measured on recovered bedrock core samples typically ranged from about 0 to 100 percent, indicating very poor to excellent quality rock, generally increasing with depth. The lowest RQD values were recorded for the upper 1.5 m of rock in Borehole 06-6 and the upper 0.8 m of bedrock in Borehole 06-10. The discontinuities observed in the rock core are typically horizontal to sub-horizontal, associated with the bedding planes, although some vertical fracturing was noted in the upper bedrock at these boreholes. A description of some of the terms used in the description of the bedrock samples from this site is provided on the *Lithological and Geotechnical Rock Description Terminology* sheet which precedes the Record of Borehole sheets included with this report in Appendix B.

4.2.5.9 Groundwater

Boreholes 06-6 and 06-10 remained dry during the short period between completion of augering and prior to commencing coring operations. The groundwater level in Boreholes 06-127 to 06-129, inclusive, ranged from 1.8 to 2.4 m below existing ground surface during the short time they remained open after completion of drilling operations.

Groundwater levels are expected to fluctuate seasonally. Higher groundwater levels are expected during wet periods of the year.

4.2.6 Retaining Wall 7S (EBL 23+842 to 23+862)

Bridge investigation Borehole 06-9 and Borehole 06-133 were advanced along the alignment of retaining wall 7S. The borehole locations and ground surface elevations as well as the soil stratigraphy section projected along retaining wall 7S are shown on Drawing 6.

4.2.6.1 Topsoil and Fill Material

A topsoil fill layer was encountered at ground surface at Boreholes 06-9 and 06-133 with thicknesses of about 0.2 and 0.1 m, respectively.

The topsoil fill layer at Boreholes 06-9 and 06-133 is underlain by fill materials 1.4 and 3.4 m in thickness, respectively. The fill material ranges in composition from sand to silty sand with varying amounts of silt, gravel and silty clay. The measured SPT "N" values vary from 6 to 42 blows per 0.3 m of penetration indicating a loose to dense consistency. The measured natural water content of one sample of the silty sand fill material was 24 percent.

A thin buried topsoil layer was encountered within the fill material at borehole 06-133. A buried topsoil layer was also encountered beneath the fill material at Borehole 06-9 with a thickness of about 0.2 m.

4.2.6.2 Organic Silt

A layer of organic silt about 0.3 m in thickness was encountered underlying the fill materials at Borehole 06-133. The measured organic content of one sample of the silt was 5 percent.

4.2.6.3 Silty Clay

The buried topsoil layer at Borehole 06-9 is underlain by a deposit of silty clay. The upper portion of this deposit has been weathered to a grey brown crust and extends to a depth of about 2.4 m below the existing ground surface. The measured natural water content of one sample of the weathered silty clay was 51 percent.

The silty clay below the depth of weathering is grey in colour and extends to a depth of about 3.3 m. The results of in situ vane testing in this material gave undrained shear strengths ranging from 63 to 84 kilopascals, indicating a stiff consistency.

The results of Atterberg limit testing on one selected sample of the unweathered silty clay indicate a plasticity index of 19 percent and a liquid limit of 36 percent. These results confirm that this material is of medium plasticity. The measured natural water content of one sample of the unweathered silty clay was 44 percent.

4.2.6.4 Glacial Till

The organic silt and silty clay are underlain by a glacial till deposit. The deposit was not fully penetrated in Borehole 06-133 but was proven to a depth of 4.4 m below the existing ground surface. The deposit was fully penetrated at Borehole 06-9 to a depth of 4.3 m below the existing ground surface. The glacial till consists of a heterogeneous mixture of gravel, cobbles, and boulders in a matrix of sandy silt or sandy gravel, with a trace of clay.

The measured SPT "N" values range from 6 to 44 blows per 0.3 m of penetration indicating a loose to dense consistency.

4.2.6.5 Gravelly Sand and Sand

The glacial till deposit at Borehole 06-9 is underlain by a gravelly sand to sand deposit which extends to a depth of 8.6 m below the existing ground surface. Cobbles and boulders were encountered in the lower portion of this deposit at about 7.0 m below the existing ground surface.

The results of grain size distribution testing of two samples of the gravely sand and sand deposit are provided on Figure 9. The measured natural water content of two samples of the gravely sand and sand deposit were 8 and 17 percent, respectively.

The measured SPT "N" values range from 43 to greater than 100 blows per 0.3 m of penetration indicating a dense to very dense consistency. The higher "N" values may also be representative of the cobbles and boulders in the deposit or the bedrock surface.

4.2.6.6 Dolomitic Limestone and Limestone Bedrock

Dolomitic limestone bedrock underlain by limestone bedrock was encountered at Borehole 06-9.

The following table summarizes the bedrock surface depth and elevation as encountered at the location of borehole 06-9. The bedrock consists of dolomitic limestone and limestone.

Borehole Number	Ground Surface Elevation (m)	Depth to Bedrock (m)	Bedrock Surface Elevation (m)
06-9	75.2	8.6	66.6

The dolomitic limestone and limestone bedrock at the site is a member of the Gull River Formation; it is medium-strong and thinly- to medium-bedded and generally unweathered. Rock Quality Designation (RQD) values measured on recovered bedrock core samples typically ranged from about 0 to 72 percent, indicating very poor to fair quality rock, generally increasing with depth. The lowest RQD value was recorded for the upper 0.1 m of rock and the discontinuities observed in the rock core are typically horizontal to sub-horizontal, associated with the bedding planes, although some vertical fracturing was noted in the upper bedrock and in a more fractured layer about 1.3 m below the bedrock surface. A description of some of the terms used in the description of the bedrock samples from this site is provided on the *Lithological and Geotechnical Rock Description Terminology* sheet which precedes the Record of Borehole sheets included with this report in Appendix B.

4.2.6.7 Groundwater

The groundwater level in Borehole 06-9 was 4.6 m below the existing ground surface during the short period between completion of augering and commencing bedrock coring operations. Groundwater levels are expected to fluctuate seasonally. Higher groundwater levels are expected during wet periods of the year.

4.2.7 Retaining Wall 8S (EBL 23+960 to 23+990)

Boreholes 06-134 and 06-135 were advanced along the alignment of retaining wall 8S. The borehole locations and ground surface elevations as well as the soil stratigraphy section projected along retaining wall 8S are shown on Drawing 7 in Appendix A.

4.2.7.1 Topsoil and Fill Material

A topsoil fill layer was encountered at ground surface at Boreholes 06-134 and 06-135 with thicknesses of about 0.2 and 0.3 m, respectively.

The topsoil fill layer at both locations is underlain by a deposit of fill material between 3.3 and 5.3 m in thickness. The fill material consists of fine sand with some silt. The results of grain size distribution testing of one sample of the sand fill material are provided on Figure 10.

The measured SPT "N" values range from 2 to 32 blows per 0.3 m of penetration indicating a very loose to dense state of packing.

4.2.7.2 Silty Clay

The fill material is underlain by a deposit of silty clay which was not fully penetrated but was proven to depths between 4.3 and 5.9 m below the existing ground surface. A measured SPT "N" value of 14 blows per 0.3 m of penetration indicates a very stiff consistency.

4.2.7.3 Groundwater

Boreholes 06-134 and 06-135 remained dry during the short time they remained open after completion of drilling operations.

Groundwater levels are expected to fluctuate seasonally. Higher groundwater levels are expected during wet periods of the year.

4.2.8 Retaining Wall 9S (EBL 23+975 to 24+050)

Boreholes 06-136 and 06-137 and bridge investigation Borehole 06-14 were advanced along the alignment of retaining wall 9S. The borehole locations and ground surface elevations as well as the soil stratigraphy sections projected along the retaining wall 9S are shown on Drawing 8 in Appendix A.

4.2.8.1 Topsoil and Fill Material

Topsoil fill was encountered at ground surface at all of the boreholes with thicknesses ranging between about 0.1 to 0.6 m.

The topsoil fill layer at all of these boreholes is underlain by fill materials ranging between 1.5 to 4.0 m in thickness. The fill material ranges in composition from sand to sandy silt with varying amounts of gravel and organic matter. The sandy fill materials at Borehole 06-136 are underlain by silty clay fill about 0.5 m in thickness. The results of grain size distribution testing of one sample of the sand fill material from Borehole 06-136 is provided on Figure 11.

The measured SPT "N" values range from 2 to 16 blows per 0.3 m of penetration indicating a very loose to compact consistency.

A buried topsoil layer was encountered beneath the fill material at Borehole 06-136 with a thickness of about 0.1 m.

4.2.8.2 Organic Silty Clay, Clayey Silt and Silty Sand

An organic silty clay layer was encountered beneath the fill material at Borehole 06-137 with a thickness of about 0.2 m. The organic content of one sample of the organic silty clay was 5 percent.

A clayey silt layer containing organic matter, about 0.6 m in thickness, underlies the organic silty clay at Borehole 06-137. A measured SPT "N" value of 13 blows per 0.3 m of penetration indicates a very stiff consistency. The measured natural water content of one sample of the clayey silt was 24 percent.

A thin silty sand layer underlies the clayey silt deposit at Borehole 06-137 with a thickness of about 0.1 m.

4.2.8.3 Clay

The silty sand, buried topsoil and fill materials, where encountered, are underlain by a deposit of clay. The deposit at Boreholes 06-136 and 06-137 was not fully penetrated but was proven to depths of about 5.2 and 6.1 m, respectively. The clay at Borehole 06-14 was fully penetrated and extends to a depth of about 6.4 m.

The upper portion of the clay deposit at Borehole 06-14 and the deposit at Borehole 06-136 has been weathered to a grey brown crust. A measured SPT "N" value of 4 blows per 0.3 m of penetration indicates that the weathered clay has a very stiff consistency.

The results of Atterberg limit testing on one selected sample of the weathered silty clay from borehole 06-14 indicates a plasticity index of 54 percent and a liquid limit of 84 percent. These results, summarized on Figure 12 in Appendix C, confirm that this material is clay of high plasticity. The measured natural water content of one sample of the weathered silty clay was 55 percent.

The clay below the depth of weathering at borehole 06-14 and the deposit below the silty sand at borehole 06-137 are grey in colour. The unweathered clay extends to a depth of about 6.4 m below the existing ground surface at borehole 06-14 and was proven to a depth of 6.1 m below the existing ground surface at borehole 06-137. Measured SPT "N" values ranged from the weight of the hammer to 11 blows per 0.3 m of penetration. The results of in situ vane testing in this material gave undrained shear strengths ranging from 42 to 77 kilopascals. These results indicate a firm to stiff consistency.

The results of Atterberg limit testing on two selected samples of the unweathered silty clay indicate plasticity indices of 32 and 43 percent and liquid limits of 54 and 68 percent. These results, confirm that this material is a clay of high plasticity as shown on Figure 13 in Appendix C. The measured natural water contents of two samples of the unweathered silty clay were 54 and 59 percent.

4.2.8.4 Glacial Till

The silty clay deposit at borehole 06-14 is underlain by a glacial till deposit. The deposit was fully penetrated to a depth of 10.7 m below the existing ground surface. The glacial till consists of a heterogeneous mixture of gravel, cobbles, and boulders in a matrix of sandy silt with a trace clay.

The measured SPT "N" values range from 11 to 76 blows per 0.3 m of penetration indicating a compact to very dense state of packing.

4.2.8.5 Sand

A deposit of sand with some gravel and cobbles and boulders, about 0.7 m in thickness, underlies the glacial till at Borehole 06-14.

4.2.8.6 Limestone Bedrock

Limestone bedrock was encountered at Borehole 06-14.

The following table summarizes the bedrock surface depth and elevation as encountered at the location of Borehole 06-14.

Borehole Number	Ground Surface Elevation (m)	Depth to Bedrock (m)	Bedrock Surface Elevation (m)
06-14	75.2	11.4	63.8

The limestone bedrock at the site is a member of the Gull River Formation; it is medium-strong and thinly- to medium-bedded and unweathered. Rock Quality Designation (RQD) values measured on recovered bedrock core samples typically ranged from about 42 to 100 percent, indicating poor to excellent quality rock, generally constant with depth. A description of some of the terms used in the description of the bedrock samples from this site is provided on the *Lithological and Geotechnical Rock Description Terminology* sheet which precedes the Record of Borehole sheets included with this report in Appendix B.

4.2.8.7 Groundwater

A standpipe was installed in Borehole 06-14 and sealed within the bedrock. The water level measured in that standpipe is summarized in the following table:

Borehole No.	Retaining Wall	Date	Depth (m)	Elevation (m)
06-14	9S	June 12, 2006	3.3	71.9

The groundwater level in Borehole 06-136 was 3.4 m below the existing ground surface during the short time it remained open after completion of drilling operations.

Groundwater levels are expected to fluctuate seasonally. Higher groundwater levels are expected during wet periods of the year, such as spring.

4.2.9 Retaining Wall 11S (EBL 24+160 to 24+515)

Boreholes 06-138 to 06-142, inclusive, and Borehole 06-17 were advanced along the alignment of retaining wall 11S. The borehole locations and ground surface elevations as well as the soil stratigraphy section projected along retaining wall 11S are shown on Drawing 9 in Appendix A.

4.2.9.1 Topsoil, Pavement Structure and Fill Material

A topsoil fill layer was encountered at ground surface at Boreholes 06-139 and 06-17 with thicknesses of about 50 mm and 0.1 m, respectively.

Boreholes 06-140, 06-141 and 06-142 were drilled through an existing pavement structure. The pavement structure consists of about 80 mm of asphaltic concrete underlain by about 0.1 to 0.9 m of sandy gravel, gravel or sand and gravel granular base.

Fill material was encountered at ground surface at Borehole 06-138, beneath the pavement structure at Boreholes 06-140 and 06-142 and beneath the topsoil fill at boreholes 06-139 and 06-17. The fill material ranges in composition from sand to sandy silt to silty clay. The fill materials range in thickness from approximately 0.5 to 2.0 m in thickness. Measured SPT "N" values ranging between 20 to 37 blows per 0.3 m of penetration indicating a compact to dense state of packing.

A buried asphaltic concrete layer was encountered at a depth of about 1.3 m within the fill material at Borehole 06-138 with a thickness of about 0.1.

Buried topsoil layers were encountered at Boreholes 06-138, 06-142 and 06-17 below the fill materials with thicknesses ranging between about 0.3 and 0.4 m.

4.2.9.2 Silty Clay and Clayey Silt

A layer of silty clay to clayey silt was encountered at Borehole 06-138 beneath the buried topsoil layer with a thickness of about 0.3 m.

4.2.9.3 Silty Clay to Clay

The clayey silt, fill material, pavement structure, and buried topsoil layers, where encountered, are underlain by a deposit of silty clay. The upper portion of the deposit extends to depths between 2.3 and 4.7 m below the existing ground surface and has been weathered to a grey brown crust. Measured SPT "N" values range between 2 and 12 blows per 0.3 m of penetration. The results of in situ vane testing in this material at Borehole 06-141 gave undrained shear strengths ranging from 65 to 68 kilopascals. The results indicate that the weathered silty clay has a stiff to very stiff consistency.

The results of Atterberg limit testing on selected samples of the weathered silty clay indicate a plasticity index ranging from 36 to 54 percent and liquid limits ranging from 62 to 80 percent. These results confirm that this material is of high plasticity as shown on Figure 14 in Appendix C. The measured natural water contents of several samples of the weathered silty clay ranged from 49 to 75 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 6.3 to 7.8 m below the existing ground surface. The results of in situ vane testing in this material gave undrained shear strengths ranging from 33

to 67 kilopascals. These results indicate a firm to stiff consistency. The results of grain size distribution testing of one sample of the unweathered silty clay are provided on Figure 15 in Appendix C.

The results of Atterberg limit testing on selected samples of the unweathered silty clay indicate a plasticity index which ranges from 20 to 29 percent and a liquid limits ranging between 48 and 82 percent. These results, confirm that this material is of medium to high plasticity as shown on Figure 16. The measured natural water contents of selected samples of the unweathered silty clay range from 55 to 80 percent which is generally in excess of the measured liquid limit.

Laboratory oedometer consolidation testing was carried out on two thin-walled Shelby tube samples of the unweathered silty clay. The results of that testing are provided on Figures 17 and 18 in Appendix C and are summarized in the table below.

Borehole/ Sample No.	Sample Depth (m)	Unit Wt. (kN/m ³)	σ_p' (kPa)	σ_{vo}' (kPa)	$\sigma_p' - \sigma_{vo}'$ (kPa)	Cc	Cr	e_0	OCR
06-140	3.3	15.7	125	47	78	2.32	0.022	2.51	2.7
06-141	4.9	15.8	195	70	125	1.95	0.012	2.24	2.8

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

4.2.9.4 Glacial Till

The silty clay deposit is underlain by a glacial till which was proven to depths between about 8.2 and 15.1 below the existing ground surface at Boreholes 06-138 to 06-142, inclusive. The glacial till in Borehole 06-17 was fully penetrated to a depth of about 8.1 m below the existing ground surface. The glacial till consists of a heterogeneous mixture of gravel, cobbles, and boulders in a matrix of sandy silt with a trace clay. The results of grain size distribution testing of one sample of the glacial till is provided on Figure 19 in Appendix C. It should be noted however that this sample was retrieved using a 50 mm diameter sampler and therefore the results do not reflect the cobble and boulder content of the deposit.

The measured SPT "N" values range from 1 to 112 blows per 0.3 m of penetration indicating a very loose to very dense state of packing. The higher "N" values may also be representative of the cobbles and boulders in the deposit. Rotary diamond drilling techniques were required in Borehole 06-17 to penetrate boulders in the till.

Sand and gravel or sand layers were encountered within the glacial till deposit at Boreholes 06-138, 06-139 and 06-140. These layers vary in thickness from about 0.4 to 2.3 m.

4.2.9.5 Auger Refusal

Practical refusal to augering was encountered at Boreholes 06-138, 06-139 and 06-140 at depths between about 12.5 and 15.1 m, below the existing ground surface. Auger refusal may indicate the bedrock surface; however, it could also represent cobbles and/or boulders within the glacial till.

4.2.9.6 Limestone Bedrock

The bedrock encountered at Borehole 06-17, at the east end of retaining wall 10S, consists of limestone with thin shale interbeds.

The following table summarizes the bedrock surface depth and elevation as encountered at the location of borehole 06-17. The bedrock consists of limestone.

Borehole Number	Ground Surface Elevation (m)	Depth to Bedrock (m)	Bedrock Surface Elevation (m)
06-17	74.1	8.1	66.0

The limestone bedrock at the site is a member of the Gull River Formation; it is medium-strong and thinly- to medium-bedded and is slightly weathered to unweathered. Thin shale interbeds were also present in the rock core. Rock Quality Designation (RQD) values measured on recovered bedrock core samples typically ranged from about 25 to 94 percent, generally increasing with depth and indicating poor to excellent quality bedrock. The lowest RQD values were recorded for the upper 0.5 m of bedrock. The discontinuities observed in the rock core are typically horizontal to sub-horizontal, associated with the bedding planes, although some vertical fracturing was noted. A description of some of the terms used in the description of the bedrock samples from this site is provided on the *Lithological and Geotechnical Rock Description Terminology* sheet which precedes the Record of Borehole sheets included with this report.

4.2.9.7 Groundwater

A monitoring well was installed in borehole 06-139, sealed within the glacial till. The water level measured in that monitoring well is summarized in the following table:

Borehole No.	Retaining Wall	Date	Depth (m)	Elevation (m)
06-139	11S	August 22, 2006	3.6	71.5

The groundwater levels in boreholes 06-138, 06-140, 06-141 and 06-142 were between 3.1 and 6.1 m below the existing ground surface during the short time they remained open after completion of drilling operations.

Groundwater levels are expected to fluctuate seasonally. Higher groundwater levels are expected during wet periods of the year.

4.2.10 Retaining Wall 12S (EBL 24+540 to 24+595)

Bridge investigation Borehole 06-18 and Boreholes 06-143, 06-143A and 06-144 were advanced along the alignment of retaining wall 12S. The borehole locations and ground surface elevations as well as the soil stratigraphy section projected along retaining wall 11S are shown on Drawing 10.

4.2.10.1 Topsoil, Pavement Structure and Fill Material

A topsoil fill layer, with a thickness of about 0.3 m, was encountered at ground surface at Borehole 06-18.

Borehole 06-144 was drilled through the pavement structure. The pavement structure consists of about 0.1m of asphaltic concrete underlain by about 0.3 m of crushed stone base material.

Fill materials were encountered at ground surface at Borehole 06-143 as well as beneath the topsoil fill layer at Borehole 06-18 and the pavement structure at Borehole 06-144. The fill material ranges in composition from sand and gravel to silty clay and range in thickness from approximately 0.3 to 1.8 m. Measured SPT "N" values ranging from 7 to 21 blows per 0.3 m of penetration indicate a loose to compact state of packing.

A buried topsoil layer, about 0.2 m in thickness, was encountered at Borehole 06-143 below the fill materials.

4.2.10.2 Sandy Silt

A thin layer of sandy silt was encountered beneath the topsoil layer at Borehole 06-143 with a thickness of about 0.2 m.

4.2.10.3 Silty Clay

The fill materials and sandy silt, where encountered, are underlain by a deposit of silty clay. The silty clay was fully penetrated in all the boreholes, with the exception of Borehole 06-143A, and extends to depths below existing ground surface ranging from 4.7 to 7.2 m.

The upper portion of the deposit extends to depths between 3.5 and 3.7 m below the existing ground surface and has been weathered to a grey brown crust. Measured SPT "N" values ranging from the weight of the hammer to 11 blows per 0.3 m of penetration indicates a firm to stiff

consistency. The measured natural water content of one sample of the weathered silty clay was 61 percent.

In Borehole 06-18, the weathered silty clay is underlain by a 0.3 m thick sand layer.

The silty clay below the depth of weathering is grey in colour. The unweathered silty clay extends to depths between about 4.7 and 7.2 m below the existing ground surface. Measured SPT "N" values range from the weight of the hammer to 1 blow per 0.3 m of penetration. The results of in situ vane testing in this material gave undrained shear strengths ranging from 31 to 56 kilopascals. These results indicate a firm to stiff consistency.

The results of Atterberg limit testing on selected samples of the unweathered silty clay indicate a plasticity index ranging from 23 to 37 percent and liquid limits ranging from 45 to 58 percent. These results, summarized on Figure 20 in Appendix C, confirm that this material is of medium to high plasticity. The measured natural water contents of three samples of the unweathered silty clay were between 57 and 61 percent.

Laboratory oedometer consolidation testing was carried out on one thin-walled Shelby tube sample of the unweathered silty clay from Borehole 06-143 and the results are provided on Figure 21 in Appendix C and in the table below. The oedometer test results seem to indicate that the sample had been significantly disturbed as the results were inconsistent with those normally obtained for Champlain Sea clays in the Ottawa area. A second oedometer consolidation test was therefore carried out on one thin-walled Shelby tube sample of the unweathered silty clay from Borehole 06-143A. The results of that testing are provided on Figure 22 in Appendix C and are also summarized in the table below.

Borehole/ Sample No.	Sample Depth (m)	Unit Wt. (kN/m ³)	σ_p' (kPa)	σ_{vo}' (kPa)	$\sigma_p' - \sigma_{vo}'$ (kPa)	Cc	Cr	e_0	OCR
06-143	4.8	16.0	70	66	4	0.66	0.020	1.74	1.1
06-143A	3.9	16.5	135	60	75	1.38	0.013	1.65	2.3

Notes:

σ_p' - Apparent preconsolidation pressure

Cc - Compression index

e_0 - Initial void ratio

σ_{vo}' - Computed existing vertical effective stress

Cr - Recompression index

OCR - Overconsolidation ratio

4.2.10.4 Glacial Till

The silty clay deposit at Boreholes 06-143 and 06-144 is underlain by glacial till. The deposit was proven to depths between about 6.1 and 6.6 m below the existing ground surface. The glacial

till consists of a heterogeneous mixture of gravel, cobbles, and boulders in a matrix of sandy silt with some clay.

Measured SPT "N" values range ranging between 19 to greater than 100 blows per 0.3 m of penetration indicating a compact to very dense state of packing. The higher "N" values may also be representative of the cobbles and boulders in the deposit or the bedrock surface.

4.2.10.5 Auger Refusal

Practical refusal to augering was encountered at Boreholes 06-143 and 06-144 at depths of about 6.1 and 6.6 m, respectively, below the existing ground surface. Auger refusal may indicate the bedrock surface; however, it could also represent cobbles and/or boulders within the glacial till.

4.2.10.6 Limestone Bedrock

The bedrock encountered at Borehole 06-18, at the west end of retaining wall 11S, consists of limestone with thin shale interbeds.

The following table summarizes the bedrock surface depth and elevation as encountered at the location of borehole 06-18.

Borehole Number	Ground Surface Elevation (m)	Depth to Bedrock (m)	Bedrock Surface Elevation (m)
06-18	73.7	7.2	66.5

The limestone bedrock at the site is a member of the Gull River Formation; it is medium-strong and thinly- to medium-bedded and is unweathered. Thin shale interbeds were also present in the rock core. Rock Quality Designation (RQD) values measured on recovered bedrock core samples typically ranged from about 27 to 73 percent, generally increasing with depth and indicating poor to fair quality bedrock. The lowest RQD values were recorded for the upper 1.4 m of bedrock. The discontinuities observed in the rock core are typically horizontal to sub-horizontal, associated with the bedding planes, although some vertical fracturing was noted. A description of some of the terms used in the description of the bedrock samples from this site is provided on the *Lithological and Geotechnical Rock Description Terminology* sheet which precedes the Record of Borehole sheets included with this report.

4.2.10.7 Groundwater

A monitoring well was installed in Borehole 06-143 and sealed within the silty clay and glacial till. A standpipe was installed at Borehole 06-18 and was sealed within the bedrock. The water levels measured in that monitoring well and standpipe are summarized in the following table:

Borehole No.	Retaining Wall	Date	Depth (m)	Elevation (m)
06-18	12S	June 12, 2006	3.0	70.7
06-143	12S	August 22, 2006	3.0	70.4

The groundwater level in Borehole 06-144 was 4.2 m below the existing ground surface during the short time it remained open after completion of drilling operations.

Groundwater levels are expected to fluctuate seasonally. Higher groundwater levels are expected during wet periods of the year.

4.2.11 Retaining Wall 1N (WBL 22+610 to 22+747)

Boreholes 06-109, 06-110 and bridge investigation Borehole 06-1 were advanced along the alignment of retaining wall 1N. The borehole locations and ground surface elevations as well as the soil stratigraphy section projected along retaining wall 1N are shown on Drawing 11.

4.2.11.1 Topsoil and Fill Material

A topsoil fill layer was encountered at ground surface at all of the boreholes with thicknesses which range from about 0.1 to 0.3 m.

Fill material was encountered beneath the topsoil fill layer at all of the boreholes. The fill material ranges in composition from sand and silty sand to silty clay with gravel, cobbles and organic matter at some locations. This fill materials range in thickness from approximately 1.5 to 3.2 m in thickness. The measured SPT "N" values range from 8 to 37 blows per 0.3 m of penetration indicating a loose to dense state of packing. The higher "N" values may be representative of cobbles within the fill at Borehole 06-109. An "N" value of greater than 100 blows at the bottom of the fill in Borehole 06-109 likely represents refusal on the bedrock surface.

4.2.11.2 Clayey Silt

A layer of clayey silt underlies the fill material at Borehole 06-110 with a thickness of about 0.6 m. A measured SPT "N" value of 19 blows per 0.3 m of penetration indicates a compact state of packing.

4.2.11.3 Glacial Till

The clayey silt and fill material at Boreholes 06-110 and 06-1, respectively, are underlain by a glacial till deposit. The deposit extends to depths between about 2.2 to 2.5 m below the existing

ground surface. The glacial till consists of a heterogeneous mixture of gravel, cobbles, and boulders in a matrix of silty sand with a trace of clay. The results of grain size distribution testing of one sample of the glacial till is provided on Figure 23. It should be noted however that this sample was retrieved using a 50 mm diameter sampler and therefore the results do not reflect the cobble and boulder content of the deposit. The measured natural water content of one sample of the glacial till was 11 percent.

A measured SPT "N" value of 8 blows per 0.3 m of penetration indicates a loose state of packing. The higher "N" values may also be representative of the bedrock surface. An "N" value of greater than 100 blows at Borehole 06-110 likely represents refusal on the bedrock surface.

4.2.11.4 Dolomitic Limestone and Limestone Bedrock

The bedrock encountered along retaining wall 1N consists of dolomitic limestone and limestone bedrock. A thin layer of shaley limestone bedrock overlies the dolomitic limestone at Borehole 06-110.

The following table summarizes the bedrock surface depth and elevation as encountered at the locations of boreholes 06-109 and 06-110, which were drilled with a portable drill rig, and 06-1.

Borehole Number	Ground Surface Elevation (m)	Depth to Bedrock (m)	Bedrock Surface Elevation (m)
06-109	79.2	3.3	75.9
06-110	78.5	2.4	76.1
06-1	77.7	2.2	75.5

The dolomitic limestone and limestone bedrock at the site is a member of the Gull River Formation; it is medium-strong and thinly- to medium-bedded and is unweathered with the exception of a thin weathered layer at the bedrock surface at Boreholes 06-109 and 06-110. Rock Quality Designation (RQD) values measured on recovered bedrock core samples typically ranged from about 0 to 100 percent, generally increasing with depth and indicating very poor to excellent quality bedrock. The lowest RQD values were recorded for the upper 0.6 m or less of bedrock at Boreholes 06-109 and 06-110. The discontinuities observed in the rock core are typically horizontal to sub-horizontal, associated with the bedding planes, although some vertical fracturing was noted. A description of some of the terms used in the description of the bedrock samples from this site is provided on the *Lithological and Geotechnical Rock Description Terminology* sheet which precedes the Record of Borehole sheets included with this report.

4.2.11.5 Groundwater

Borehole 06-1 was dry during the short time it remained open after completion of drilling operations.

Groundwater levels are expected to fluctuate seasonally. Higher groundwater levels are expected during wet periods of the year.

4.2.12 Retaining Wall 2N (WBL 22+770 to 22+975)

Bridge investigation Borehole 06-2 and Boreholes 06-115 to 06-117, inclusive, were advanced along the alignment of retaining wall 2N. The borehole locations and ground surface elevations as well as the soil stratigraphy section projected along retaining wall 2N are shown on Drawing 12 in Appendix A.

4.2.12.1 Topsoil and Fill Material

Topsoil fill extends from the ground surface at Borehole 06-2 to a depth of about 0.2 m below existing ground surface. Fill material was encountered beneath the topsoil fill at Borehole 06-2 and at ground surface at the remaining borehole locations. The fill material ranges in composition from crushed stone and sand and gravel, to sandy silt, sand and silty clay. Gravel, cobbles and organic matter were encountered within the fill material at some locations. The fill materials range in thickness from approximately 0.8 to 1.3 m. A measured SPT "N" value of 7 blows per 0.3 m of penetration indicates a loose state of packing.

A buried topsoil layer was encountered beneath the fill material at all of the boreholes with the exception of Borehole 06-2. The buried topsoil layer ranges in thickness from approximately 0.2 to 0.4 m.

4.2.12.2 Sand and Sandy Silt

The fill material at Borehole 06-2 is underlain by a deposit of fine to coarse sand with a thickness of about 0.8 m. A measured SPT "N" value of 45 blows per 0.3 m of penetration indicates a dense state of packing.

The buried topsoil layer at Borehole 06-115 is underlain by a thin deposit of sandy silt with a thickness of about 0.2 m.

4.2.12.3 Silty Clay and Clayey Silt

A silty clay deposit with a thickness of approximately 0.5 m was encountered at Borehole 06-2 underlying the sand deposit. A measured SPT "N" value of 5 blows per 0.3 m of penetration indicates very stiff consistency.

The sandy silt deposit at Borehole 06-115 is underlain by a deposit of clayey silt. This deposit is about 0.2 m thick.

4.2.12.4 Glacial Till

The clayey silt layer at Borehole 06-115 and the buried topsoil layer at Boreholes 06-116 and 06-117, are underlain by a glacial till deposit. The deposit was proven to depths of 2.8 and 1.4 m below the existing ground surface at Borehole 06-115 and 06-117, respectively. The deposit was fully penetrated at Borehole 06-116 to a depth of about 1.5 m below the existing ground surface. The glacial till consists of a heterogeneous mixture of gravel, cobbles, and boulders in a matrix of sandy silt with some clay. The measured natural water content of one sample of the glacial till was 9 percent.

The measured SPT "N" values range from 13 to greater than 50 blows per 0.3 m of penetration indicating a compact to very dense state of packing. The higher "N" values may also be representative of the cobbles and boulders in the deposit or the bedrock surface.

4.2.12.5 Auger Refusal

Practical refusal to augering was encountered at Boreholes 06-115, 06-116 and 06-117 at depths between about 1.4 and 2.8 m, below the existing ground surface. Auger refusal may indicate the bedrock surface; however, it could also represent cobbles and/or boulders within the glacial till.

4.2.12.6 Dolomitic Limestone and Limestone Bedrock

The bedrock encountered along retaining wall 2N at Borehole 06-2 consists of dolomitic limestone underlain by limestone bedrock.

The following table summarizes the bedrock surface depth and elevation as encountered at the location of Borehole 6-2.

Borehole Number	Ground Surface Elevation (m)	Depth to Bedrock (m)	Bedrock Surface Elevation (m)
06-2	78.1	2.8	75.3

The dolomitic limestone and limestone bedrock at the site is a member of the Gull River Formation; it is medium-strong and thinly- to medium-bedded and is unweathered. Rock Quality Designation (RQD) values measured on recovered bedrock core samples typically ranged from about 92 to 100 percent, indicating excellent quality bedrock. The discontinuities observed in the rock core are typically horizontal to sub-horizontal, associated with the bedding planes. A description of some of the terms used in the description of the bedrock samples from this site is provided on the *Lithological and Geotechnical Rock Description Terminology* sheet which precedes the Record of Borehole sheets included with this report.

4.2.12.7 Groundwater

A standpipe was installed in Borehole 06-2 and sealed within the bedrock. The water level measured in that standpipe is summarized in the following table:

Borehole No.	Retaining Wall	Date	Depth (m)	Elevation (m)
06-2	2N	June 12, 2006	3.1	75.0

Groundwater levels are expected to fluctuate seasonally. Higher groundwater levels are expected during wet periods of the year.

4.2.13 Retaining Wall 3N (WBL 23+215 to 23+460)

Boreholes 06-123 to 06-126, inclusive, and Borehole 06-7 were advanced along the alignment of retaining wall 3N. The borehole locations and ground surface elevations as well as the soil stratigraphy section projected along retaining wall 3N are shown on Drawing 13.

4.2.13.1 Topsoil, Fill Material and Peat

A topsoil fill layer was encountered at ground surface at all of the borehole locations with the exception of Borehole 06-124. The topsoil layer varies in thickness from about 0.1 to 0.9 m with an average thickness of 0.4 m.

Fill materials were encountered at ground surface at Borehole 06-124 and beneath the topsoil fill layer at the remaining boreholes with the exception of Borehole 06-123. The fill material is between about 1.0 to 3.6 m in thickness and ranges in composition from sandy silt to clayey silt and silty clay. Gravel and organic matter were encountered within the fill material at some locations. The bottom 1.0 metre of the fill at Borehole 06-125 is composed of crushed stone. The results of grain size distribution testing of one sample of the sandy fill material from Borehole 06-126 are provided on Figure 24. The measured natural water contents of two samples of the fill material were 17 and 37 percent for the silty sand and silty clay fill, respectively.

The measured SPT “N” values for the cohesionless fill material range from 1 to 13 blows per 0.3 m of penetration indicating a very loose to compact state of packing. The measured SPT “N” values for the cohesive fill materials range from 29 to 34 indicating a very stiff consistency.

A layer of peat was encountered at Borehole 06-7 underlying the fill material with a thickness of 0.2 m.

4.2.13.2 Silty Sand

A layer of silty sand was encountered beneath the fill material at Borehole 06-124 with a thickness of about 0.4 m. A measured SPT “N” value of greater than 36 blows per 0.3 m of penetration indicates a dense state of packing, although this value may also represent cobbles within the deposit or the bedrock surface.

4.2.13.3 Silty Clay and Clayey Silt

A thin layer of silty clay and clayey silt underlies the peat deposit at Borehole 06-7 with a thickness of about 0.2 m.

4.2.13.4 Glacial Till

The fill material at Borehole 06-126 and the silty clay and clayey silt layer at Borehole 06-7 are underlain by a glacial till deposit. The deposit was proven to a depth of 4.0 m below the existing ground surface at Borehole 06-126 and fully penetrated to a depth of 2.7 m below the existing ground surface at Borehole 06-7. The glacial till consists of a heterogeneous mixture of gravel, cobbles, and boulders in a matrix of silty sand or silt with a trace to some clay. The results of grain size distribution testing of one sample of the glacial till material from Borehole 06-7 are provided on Figure 25. It should be noted however that this sample was retrieved using a 50 mm diameter sampler and therefore the results do not reflect the cobble and/or boulder content of the deposit.

The measured SPT “N” values range from greater than 42 to 79 blows per 0.3 m of penetration indicating a dense to very dense state of packing. The higher “N” values may also be representative of the cobbles and boulders in the deposit.

4.2.13.5 Auger Refusal

Practical refusal to augering was encountered at Boreholes 06-123, 06-124 and 06-125 at depths ranging between about 0.9 to 3.7 m below the existing ground surface. Auger refusal may indicate the bedrock surface or cobbles and/or boulders in the glacial till deposit where encountered.

4.2.13.6 Limestone Bedrock

The bedrock encountered at Borehole 06-7, at the east end of retaining wall 3N, consists of limestone bedrock.

The following table summarizes the bedrock surface depth and elevation as encountered at the location of borehole 06-7.

Borehole Number	Ground Surface Elevation (m)	Depth to Bedrock (m)	Bedrock Surface Elevation (m)
06-7	75.8	2.7	73.1

The limestone bedrock at the site is a member of the Gull River Formation; it is medium-strong and thinly- to medium-bedded. The upper 1.9 m of the bedrock is slightly weathered to weathered and overlies unweathered bedrock extending below about elevation 71.2 m. Rock Quality Designation (RQD) values measured on recovered bedrock core samples typically ranged from about 47 to 100 percent, generally increasing with depth and indicating poor to excellent quality bedrock. The discontinuities observed in the rock core are typically horizontal to sub-horizontal, associated with the bedding planes, although some vertical fracturing was noted. A description of some of the terms used in the description of the bedrock samples from this site is provided on the *Lithological and Geotechnical Rock Description Terminology* sheet which precedes the Record of Borehole sheets included with this report.

4.2.13.7 Groundwater

The groundwater level in Borehole 06-7 was 2.4 m below the existing ground surface during the short time it remained open after completion of drilling operations.

Groundwater levels are expected to fluctuate seasonally. Higher groundwater levels are expected during wet periods of the year.

4.2.14 Retaining Wall 4N (WBL 23+480 to 23+565)

Bridge investigation Borehole 06-5 and Borehole 06-130 were advanced at each end of the proposed alignment of retaining wall 4N. The borehole locations and ground surface elevations as well as the soil Stratigraphy section projected along retaining wall 4N are shown on Drawing 14 in Appendix A.

4.2.14.1 Topsoil, Fill Material and Peat

A topsoil fill layer was encountered at ground surface at Borehole 06-130 with a thickness of about 0.1 m.

The topsoil fill at Borehole 06-130 is underlain by existing embankment fill materials extending to a depth of about 4.9 m below existing ground surface. The fill material is composed of fine sand with some silt and clay and trace gravel. The results of grain size distribution testing of one sample of the fill material is provided on Figure 26. Measured SPT "N" values ranging between 1 to 4 blows per 0.3 m of penetration indicate a very loose state of packing.

Crushed stone fill material, about 0.2 m in thickness, was encountered at ground surface at Borehole 06-5. The crushed stone fill material is underlain by sand and sandy silt fill, containing varying amounts of gravel, silt and peat, which extends to a depth of about 1.3 m below existing ground surface. A measured SPT "N" value of 50 blows per 0.3 m of penetration indicates a dense state of packing.

The fill material at Borehole 06-5 is underlain by a layer of peat with a thickness of about 0.4 m. The measured natural water content of one sample of the peat was 163 percent.

4.2.14.2 Silty Clay and Silty Sand

A thin deposit of weathered silty clay crust was encountered beneath the peat at Borehole 06-5 with a thickness of 0.2 m. The silty clay at Borehole 06-5 is in turn underlain by a thin layer of silty sand with a thickness of about 0.2 m.

4.2.14.3 Glacial Till

The silty sand at Borehole 06-5 and the fill materials at Borehole 06-130 are underlain by a deposit of glacial till. The deposit was fully penetrated to a depth of 2.8 m below the existing ground surface at Borehole 06-5 and proven to a depth of 7.1 m below the existing ground surface at Borehole 06-130. The glacial till consists of a heterogeneous mixture of gravel, cobbles, and boulders in a matrix of sandy silt with a trace of clay. The results of grain size distribution testing of one sample of the glacial till material is provided on Figure 27. It should be noted however that this sample was retrieved using a 50 mm diameter sampler and therefore the results do not reflect the cobble and boulder content of the deposit. The measured natural water contents of two samples of the glacial till were 2 and 8 percent.

The measured SPT "N" values range from 9 to greater than 59 blows per 0.3 m of penetration indicating a loose to very dense state of packing. The higher "N" values may also be

representative of the cobbles and boulders in the deposit or the bedrock surface. Rotary diamond drilling techniques were required in Borehole 06-5 to penetrate boulders in the till.

4.2.14.4 Auger Refusal

Practical refusal to augering was encountered at Borehole 06-130 at a depth of 7.1 m, below the existing ground surface. Auger refusal may indicate the bedrock surface or cobbles and/or boulders in the glacial till.

4.2.14.5 Limestone Bedrock

The bedrock encountered at Borehole 06-5, at the west end of retaining wall 4N, consists of limestone bedrock.

The following table summarizes the bedrock surface depth and elevation as encountered at the location of borehole 06-5.

Borehole Number	Ground Surface Elevation (m)	Depth to Bedrock (m)	Bedrock Surface Elevation (m)
06-5	75.6	2.8	72.8

The limestone bedrock at the site is a member of the Gull River Formation; it is medium-strong and thinly- to medium-bedded and is generally unweathered. Rock Quality Designation (RQD) values measured on recovered bedrock core samples typically ranged from about 50 to 100 percent, generally increasing with depth, and indicating fair to excellent quality bedrock. The lowest RQD values were recorded for the fractured zone extending from about 1.2 to 1.6 m below the bedrock surface. The discontinuities observed in the rock core are typically horizontal to sub-horizontal, associated with the bedding planes, although some vertical fracturing was noted. A description of some of the terms used in the description of the bedrock samples from this site is provided on the *Lithological and Geotechnical Rock Description Terminology* sheet which precedes the Record of Borehole sheets included with this report.

4.2.14.6 Groundwater

A standpipe was installed in Borehole 06-5, sealed within the bedrock. The water level measured in that standpipe is summarized in the following table:

Borehole No.	Retaining Wall	Date	Depth (m)	Elevation (m)
06-5	4N	June 12, 2006	2.4	73.2

Groundwater levels are expected to fluctuate seasonally. Higher groundwater levels are expected during wet periods of the year.

4.2.15 Retaining Wall 5N (WBL 23+686 to 23+837)

Borehole 06-132 and bridge investigation Borehole 06-12 were advanced along the proposed alignment of retaining wall 5N. The borehole locations and ground surface elevations as well as the soil Stratigraphy section projected along retaining wall 5N are shown on Drawing 15 in Appendix A.

4.2.15.1 Topsoil and Fill Material

A topsoil fill layer was encountered at ground surface at both locations. The thickness of the topsoil layer is about 0.1 m.

Fill material was encountered beneath the topsoil fill layer at Boreholes 06-132 and 06-12 with thicknesses of 1.2 and 1.5 m, respectively. The fill material generally consists of sand and silty sand with a trace of clay in the lower part of the fill at Borehole 06-12. The results of grain size distribution testing of one sample of the sandy fill material from Borehole 06-12 are provided on Figure 28.

Measured SPT "N" values for the fill materials at Boreholes 06-132 and 06-12 of 21 and 4 blows per 0.3 m of penetration indicate compact and loose states of packing, respectively.

4.2.15.2 Silty Clay

The fill material at Borehole 06-12 is underlain by a deposit of silty clay. The deposit is about 1.5 m in thickness and has been weathered to a stiff grey-brown crust. An SPT "N" value of 1 blow per 0.3 m of penetration was measured in this deposit. The results of in situ vane testing in the weathered silty clay gave undrained shear strengths ranging from about 84 to 96 kilopascals. These results indicate a stiff consistency.

The results of Atterberg limit testing on one selected sample of the weathered silty clay indicate a plasticity index of 38 percent and a liquid limit of 66 percent. These results, confirm that this material is of high plasticity as shown on Figure 29 in Appendix C. The measured natural water content of one sample of the weathered silty clay was 74 percent which is in excess of the measured liquid limit.

4.2.15.3 Glacial Till

The silty clay at Borehole 06-12 and the fill materials at Borehole 06-132 are underlain by a deposit of glacial till. The deposit was fully penetrated to a depth of 4.6 m below the existing ground surface at Borehole 06-12 and proven to a depth of 3.7 m below the existing ground surface at Borehole 06-132. The glacial till consists of a heterogeneous mixture of gravel, cobbles, and boulders in a matrix of sandy silt and silty sand with a trace of clay. The results of grain size distribution testing of one sample of the glacial till are provided on Figure 30. It should be noted however that this sample was retrieved using a 50 mm diameter sampler and therefore the results do not reflect the cobble and boulder content of the deposit. The measured natural water contents of two samples of the glacial till were 8 and 12 percent.

The measured SPT "N" values range from 6 to 22 blows per 0.3 m of penetration indicating a loose to compact state of packing.

4.2.15.4 Sand

The glacial till deposit at Borehole 06-12 is underlain by a deposit of sand with a thickness of about 3.2 m. The results of grain size distribution testing of one sample of the sand is provided on Figure 31 in Appendix C. The measured natural water content of one sample of the sand was 16 percent.

The measured SPT "N" values range from 2 to greater than 100 blows per 0.3 m of penetration indicating a very loose to very dense state of packing.

4.2.15.5 Limestone Bedrock

The bedrock encountered at Borehole 06-12, at the east end of retaining wall 5N, consists of limestone bedrock.

The following table summarizes the bedrock surface depth and elevation as encountered at the location of Borehole 06-12.

Borehole Number	Ground Surface Elevation (m)	Depth to Bedrock (m)	Bedrock Surface Elevation (m)
06-12	75.3	7.8	67.5

The limestone bedrock at the site is a member of the Gull River Formation; it is medium-strong and thinly- to medium-bedded and is generally unweathered. Rock Quality Designation (RQD) values measured on recovered bedrock core samples typically ranged from about 39 to 91 percent, increasing with depth, and indicating fair to excellent quality bedrock. The lowest

RQD values were recorded for the upper 0.9 m of bedrock. The discontinuities observed in the rock core are typically horizontal to sub-horizontal, associated with the bedding planes, although some vertical fracturing was noted. A description of some of the terms used in the description of the bedrock samples from this site is provided on the *Lithological and Geotechnical Rock Description Terminology* sheet which precedes the Record of Borehole sheets included with this report.

4.2.15.6 Groundwater

A monitoring well was installed in Borehole 06-132, and sealed within the glacial till. A standpipe was installed at Borehole 06-12 and was sealed within the bedrock. The water levels measured in that monitoring well and standpipe are summarized in the following table:

Borehole No.	Retaining Wall	Date	Depth (m)	Elevation (m)
06-132	5N	June 12, 2006	3.5	71.9
06-12	5N	August 22, 2006	3.5	71.8

Groundwater levels are expected to fluctuate seasonally. Higher groundwater levels are expected during wet periods of the year.

4.2.16 Retaining Wall 6N (WBL 23+590 to 23+610)

Borehole 06-145 was advanced at the proposed location of retaining wall 6N. The borehole location and ground surface elevation as well as the soil stratigraphy section projected at retaining wall 6N are shown on Drawing 16 in Appendix A

4.2.16.1 Fill Material and Topsoil

Fill material was encountered from the ground surface at Borehole 06-145 to a depth of about 4.6 m. The fill materials range in composition from silty sand and sand to silty clay and crushed stone. The results of grain size distribution testing of one sample of the sand fill is provided on Figure 32 in Appendix C.

The measured SPT "N" values range from 2 to 12 blows per 0.3 m of penetration indicating a very loose to compact state of packing.

A buried topsoil layer, about 0.1 m in thickness, was encountered beneath the fill material.

4.2.16.2 Silty Clay

The topsoil layer is underlain by a deposit of unweathered grey silty clay about 1.1 m in thickness. Two measured SPT "N" values of 1 and 10 blows per 0.3 m of penetration indicate a stiff to very stiff consistency.

The results of Atterberg limit testing on one selected sample of the unweathered silty clay indicate a plasticity index of 18 percent and a liquid limit of 35 percent. These results, confirm that this material is of low to medium plasticity as shown on Figure 33 in Appendix C. The measured natural water content of one sample of the unweathered silty clay was 39 percent which was in excess of the measured liquid limit.

4.2.16.3 Glacial Till

The silty clay is underlain by a glacial till deposit which was proven to a depth of 7.3 m below the existing ground surface. The glacial till consists of a heterogeneous mixture of gravel, cobbles, and boulders in a matrix of sandy silt with a trace of clay.

Measured SPT "N" values range from 5 to 86 blows per 0.3 m of penetration indicating a loose to very dense state of packing.

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 Mr. James Samotowka, Mr. Robert Ireland and Mr. Douglas Grylls under the direction of Mr. William Cavers, P.Eng. The testing was carried out in the Mississauga laboratory of Golder Associates. The report was prepared by Mr. William Cavers, P.Eng., under the direction of Mr. Michael Snow, P.Eng. This report was reviewed by Mr. Fintan J. Heffernan P.Eng, the designated MTO contact for this project.

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PART B

**FOUNDATION DESIGN REPORT
CLYDE AVENUE OVERPASS BRIDGE WIDENING
STRUCTURE SITE 3-43
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 walls along the widening of Highway 417 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 options and to design the proposed retaining walls. 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.

A general description of the analysis methods and general guidelines for design are provided in Section 6.2 of this report.

The retaining wall options and geotechnical recommendations at each retaining wall/embankment location are presented in the sections following Section 6.2. A summary comparison of the advantages, disadvantages, relative costs, and risks associated with the retaining wall options at each location is presented in Table 1 following the text of this report.

6.2 Retaining Wall / Embankment Options

Retaining wall and embankment options that have been considered for the proposed retaining wall/embankment locations within the project limits include:

- Reinforced soil system (RSS) walls;
- “Perched” RSS walls founded on the existing embankment fill materials;
- 1H:1V mechanically stabilized earth (MSE) slopes;
- Cantilevered reinforced concrete retaining walls supported on shallow or deep foundations;
- Cantilevered soldier pile and concrete lagging walls;
- 2H:1V slopes with low height cantilevered reinforced concrete walls at the toe of the slope;
- 2H:1V slopes;
- Toe walls constructed in accordance with OPSD 3120-100; and,
- Lightweight fill materials (i.e., slag or expanded polystyrene – EPS).

The preferred retaining wall/embankment option will depend on property constraints, appearance, performance and cost considerations.

- 1- Reinforced soil system walls:** Mechanically reinforced or retained soil system (RSS) vertical walls are geotechnically feasible and can be considered for some of the retaining walls required as part of the Highway 417 widening project. 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 variability in the thickness of additional fill to be placed and natural variability in the thickness and depth of the zone of firm to stiff silty clay to clay between Carling Avenue WB and Island Park Drive, it is considered that RSS walls would not be practical for retaining walls between these limits since the differential settlements would likely exceed the limiting settlement magnitude noted above. At Carling Avenue WB, where settlements are slightly above 35 mm, consideration could be given to their use.
- 2- 'Perched' RSS 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 that are typically lower than the full embankment height) and founded on the existing embankment fill materials may be feasible at some locations west of Carling EB. However, the performance of 'perched' walls may not be satisfactory since the behavior of the existing fill materials is difficult to predict and the potential exists for excessive differential settlement.
- 3- Mechanically stabilized earth (MSE) slopes:** Reinforced, steepened embankment side slopes are geotechnically feasible and could be considered in some locations, where there is sufficient space to construct reinforced earth slopes at an inclination of 1H:1V. MSE slopes at inclinations of up to 1H:5V may be considered behind Westgate Mall due to the limited space at that location. MSE slopes steeper than 1H:1V are susceptible to erosion and would require specialized facing.
- 4- Concrete retaining walls on shallow foundations:** This type of wall and foundation can be considered where the total post-construction settlements are less than approximately 25 mm – i.e. for the retaining walls between Clyde Avenue and Carling Avenue EB where the overburden soils are relatively thin.
- 5- Concrete retaining walls on deep pile foundations:** This type of wall and foundation can be considered where the total post-construction settlements are greater than approximately 25 mm. It would be appropriate for the retaining walls behind Westgate Mall and east of Merivale Avenue, where the predicted post-

construction settlements may exceed 50 mm (assuming the use of earth or granular fill for the embankment widening and retaining wall backfill).

- 6- **Cantilevered soldier pile and concrete lagging walls:** This type of wall is considered feasible for the retaining walls behind Westgate Mall. The soldier pile and concrete lagging system can be installed as part of the cut widening, without significant temporary protection for the highway (as might be required for the installation of shallow foundations, pile caps or reinforced soil masses).
- 7- **2H:1V slopes (with or without toe walls):** Embankments with 2H:1V side slopes are considered feasible at some locations where sufficient property within the right of way is available. At some of these locations, low height (<2.5 m) retaining walls at the toe of the slope may be constructed to provide clearance around local structures, such as hydro towers.
- 8- **OPSD (concrete gravity) toe wall:** Relatively low height (< 1.8 m) toe walls may be constructed in accordance with OPSD 3120-100. These walls may be feasible east of Maitland Avenue, where the highway is in cut, between Carling EB and Carling WB south of the highway, between Kirkwood Avenue and the Carling EB to Highway 417 EB ramp, and between the Carling EB to Highway 417 EB ramp and Highway 417.
- 9- **Lightweight fill materials:** The amount of time-dependent settlement and the associated roadway maintenance may be reduced by employing lightweight fill materials below the pavement structure in areas where compressible clayey soils exist (i.e., at the retaining walls behind Westgate Mall and east of Merivale Avenue). Lightweight fill could be used in place of conventional earth fill to reduce the applied loading to below the pre-consolidation range.

Three types of lightweight fill are available for use:

- Expanded polystyrene (EPS) fill, with a bulk unit weight of less than 1 kN/m³;
- Ultra-lightweight slag fill from Hamilton (Litex-143), with a bulk unit weight of about 11.5 kN/m³; and,
- Lightweight slag fill (Superior Slag) from Sault Ste. Marie or from Hamilton (Litex-149), with a bulk unit weight of about 14 kN/m³.

6.3 General Recommendations

6.3.1 Foundation Design

6.3.1.1 Shallow Foundations

Shallow foundations on the native soils may be considered for the support of cantilevered reinforced concrete walls. Existing fill materials and surficial or buried topsoil and peat are not suitable for support of shallow footings and need to be removed from the foundation areas. Footings may also be supported on compacted engineered fill placed on native soils at locations where shallow foundations are considered feasible.

Where the retaining walls are to be founded on a pad of compacted engineered fill, the existing fill and organic materials 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 to fill 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 SP105S10.

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)*.

Resistance to lateral forces for retaining wall footings founded on the native soils should be calculated in accordance with Section 6.7.5 of the *CHBDC*. The parameter values that may be used to calculate the lateral resistance to sliding across the base of the footing-soil interface are provided in the sections herein specific to each retaining wall location.

For footings on silty clay, the lateral sliding resistance to both long term and short term loading needs to be evaluated. Resistance values for both conditions are also provided in the section herein specific to each retaining wall location.

At locations where the footings are founded on sands or weathered silty clay *underlain* by unweathered silty clay, the short term shear resistance within the silty clay shall also be checked using the unfactored (S_u) value provided. As a preliminary guideline, the loaded area can be assumed to be distributed down from the footing level to the unweathered silty clay at a slope of 1H:1V. A more detailed assessment of the resistance due to sliding along this weaker layer where appropriate can be provided once the footing geometry is known at each location.

Where retaining walls will be supported on engineered fill, an unfactored $\tan \delta^*$ lateral sliding resistance value of 0.50 may be used at the base of footing – engineered fill interface.

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.

RSS walls or MSE slopes should be founded on a minimum 0.3 metre thick pad of engineered fill consisting of OPSS 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 suggested engineered fill pad prior to construction.

RSS walls or MSE slopes 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 or the toe of the MSE slope at a slope of 1H:1V. The engineered fill should extend a minimum distance of $\frac{2}{3}$ of the slope height back (i.e., towards the highway) from the toe of a MSE slope and a minimum distance of $\frac{1}{2}$ the wall height back from the face of the RSS wall.

At some locations 'perched' RSS walls have been considered. 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) to a minimum depth identified in the specific section for that wall and a minimum slope extending from the exposed face of the wall is also identified in each specific section where appropriate. Typically these toe 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 100 kPa may be used for the design of perched RSS walls. However, the behavior of the existing fill materials is difficult to predict and the differential settlement of 'perched' RSS walls may exceed the 35 mm limit considered to be tolerable along each section between slip joints even when loaded to less than the SLS geotechnical resistance specified above.

6.3.1.2 Deep Foundations

Steel H-Pile Foundations

Steel H-piles driven to found on the bedrock may be used for support of the retaining walls at some locations.

A factored axial resistance at Ultimate Limit States (ULS) of 2,000 kN may be assumed at all the retaining wall locations for design of HP 310 x 110 piles driven to found on the bedrock, or socketed at least 2 m into the bedrock. 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.

Uplift resistance and the additional load due to downdrag forces are provided in the appropriate section for each location where required.

Consideration must be given to the presence of cobbles and boulders within the glacial till and sand which exists at most of the retaining wall locations within the project limits. In this regard, vertically driven piles should be equipped with Type I flange reinforcement as per OPSD 3000.100. Any battered 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.

Pile installation should be in accordance with SP903S01. For this site, the piles will essentially be driven to practical refusal on the bedrock. The drawings should incorporate the appropriate note stating that the piles should be equipped with flange reinforcement and/or rock points 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.

It may be necessary to socket the piles into the bedrock to resist lateral or seismic forces. The limestone / dolomitic limestone bedrock is generally medium strong, however, and this would require socket formation using coring or churn drilling to advance the hole. Lateral loading could also be resisted fully or partially by the use of battered steel H-piles.

If vertical piles are used, the resistance to lateral loading will have to be derived from the soil in front of the 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).

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)

Ranges for the values of n_h and s_u that may be assumed for use in the structural analysis at each location are provided in the section for each retaining wall location. The range in values provided at each location reflects the variability that may exist in the subsurface conditions.

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):

<i>Pile Spacing in Direction of Loading (d = Pile Diameter)</i>	<i>Reduction Factor</i>
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.

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., sands and till), the ULS lateral passive resistance may be assumed to act over the pile shaft to a depth equal to six pile diameters below the underside of the pile cap and may be calculated as:

Above the water table: $P_p(z) = 3 K_p \gamma z$

Below the water table: $P_p(z) = 3 K_p \gamma D_w + 3 K_p (z - D_w) (\gamma - \gamma_w)$

where:

$P_p(z)$ is the ULS lateral resistance at depth 'z' below ground surface (kN);

γ average unit weight of overlying soil (kN/m³);

K_p is the coefficient of passive earth pressure;

D_w is the depth to groundwater table below ground surface (m); and

γ_w is the unit weight of water, use 9.8 kN/m³

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 and 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 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.

Values for K_p and S_u that may be assumed for estimating the ULS geotechnical lateral resistances at each wall location are provided in the relevant section.

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.

Caisson Foundations

Caissons founded on or socketed into the bedrock may be used for support of retaining walls.

It is noted 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. 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.

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 should be used. Serviceability Limit States resistances do not apply to caissons founded on or socketed in the limestone bedrock, since the SLS resistance for 25 mm of settlement is greater than the factored axial geotechnical resistance at ULS.

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 6.3.2.1

6.3.1.3 Frost Protection

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

6.3.1.4 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) 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. To prevent the migration of fines from the existing embankment fill into the retaining wall backfill and to allow for adequate drainage the backfill material should be graded such that the D_{15} (i.e., the sieve diameter through which 15% of the material will pass) is greater than 0.3 mm and less than 0.85 mm. The crushed limestone typically used in the Ottawa area and conforming to OPSS Granular 'A' or Granular 'B' Type II will generally have a D_{15} within these limits. 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 6.9.3. 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 I in Figure C6.9.1(I) 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 II in Figure C6.9.1(I) of the *Commentary to the CHBDC*). For Case I, the pressures are based on the proposed embankment fill materials and the following parameters (unfactored) may be used assuming the use of Select Subgrade material:

Soil unit weight:	20 kN/m³
Coefficients of static lateral earth pressure:	
Active, K_a	0.35
At rest, K_o	0.50

For Case II, the pressures are based on the granular fill as placed and the following parameters (unfactored) may be assumed:

	Granular 'A'	Granular 'B' Type II
Soil unit weight:	22 kN/m ³	21 kN/m ³
Coefficients of static lateral earth pressure:		
Active, K_a	0.27	0.27
At rest, K_o	0.43	0.43

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.

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 the *CHBDC*, this site is located in Seismic Performance Zone 3. The site-specific zonal acceleration ratio for Ottawa is 0.2. Based on experience, for the subsurface conditions at this site, no significant

amplification of the ground motion will occur. The seismic lateral earth pressure coefficients given in each section have been derived based on a design zonal acceleration ratio of $A = 0.2$.

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. $k_h = 0.3$). 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. $k_h = 0.1$).

Seismic active pressure coefficients (K_{AE}) for the two backfill cases (Case I and Case II) are provided in the section for each wall location to be used in design; these coefficients reflect the K_{AE} obtained using the k_h value as described above. It should be noted that these seismic earth pressure coefficients assume that the back of the wall is vertical and the ground surface behind the wall is flat. Where sloping backfill is 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.

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:

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

where:

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

6.3.1.5 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:

- Large lateral movements of even gently sloping ground, referred to as “lateral spreading”, which could impact embankment stability;
- Reduced shear resistance (i.e., bearing capacity) of soils which support foundations, as well as reduced resistance to sliding; 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. Seismic settlement is the process where the soils stabilize into a denser arrangement after an earthquake, causing potentially large surface settlements.

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.

At locations where these conditions exist an assessment of the potential seismic liquefaction hazard was carried out and involves comparing the cyclic shear stresses applied to the soil by the design earthquake (represented by the cyclic stress ratio, CSR) to the cyclic shear strength offered by the soil (represented as the cyclic resistance ratio, CRR). The CSR is primarily a function of the effective overburden pressure, the design ground acceleration, and the earthquake magnitude and ground acceleration specific to the site. The CRR is primarily related to the relative density of the soil and its gradation.

The seismic liquefaction assessment was carried out consistent with the state of practice outlined by the National Center for Earthquake Engineering Research (NCEER). The NCEER methodology compares the CRR to the CSR required to liquefy the soil.

Two important parameters required for that assessment are the design peak horizontal ground acceleration (PHGA) from the seismic event as well as its magnitude. The reference PHGA provided for this area in the CHBDC is 0.2g (g = acceleration due to gravity). The seismic magnitude, M, has been selected as being 6.2, based on the past seismic history for this area and the deaggregated seismic hazard data.

Guidelines relating to seismic considerations and liquefaction are provided for each wall location in the site specific sections.

6.3.2 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.

6.3.3 Global Stability

The global stability of the retaining walls and 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 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 these sites, considering the design and performance requirements and the available subsurface data.

Pseudo-static seismic slope stability analyses were also carried out using a seismic coefficient of 0.1.

The internal stability of mechanically-reinforced walls and slopes should be checked by the supplier.

It was assumed for the global stability assessment of MSE slopes that the reinforcement would prevent the global failure surface from approaching closer than $2/3$ the slope height to the toe of the reinforced slope. Similarly for RSS walls it was assumed 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. 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	Undrained Shear Strength (kPa)
Earth or Granular Embankment Fill	21	0	32°	—
Existing Sand Fill	19	0	29°	—
Existing Silty Clay Fill	18	0	28°	50
Topsoil	18	0	25°	20
Peat	15	0	25°	15 – 20
Organic Silty Clay	17.5	0	25°	25
Weathered Clay to Silty Clay	17.5	5	35°	80
Unweathered Clay to Silty Clay	16 – 16.5	7.5	28° - 31°	30 – 60
Clayey Silt	19	3	30°	60
Sand	19	0	30° - 32°	—
Glacial Till	21	0	32°	—

6.3.4 Embankment Settlement

Settlement of the embankment widenings will occur as a result of compression of the new embankment fill and of the underlying subgrade as well as consolidation of the clayey soils on which some of the widened embankments will be founded.

Provided that the new embankment fill material consists of granular fill, Select Subgrade Material or clean earth fill, the settlement of the embankment fill itself is expected to be less than about 25 mm. The use of granular fill for the new embankment construction would reduce the magnitude of *post-construction settlement* (likely to less than 10 mm), since the majority of settlement of granular fills will occur during construction.

In general, the subgrade soils along the retaining walls located along the eastern portion of the project alignment are non-cohesive and the resulting settlements will be due to compression of the underlying soils. The settlement magnitudes resulting from this compression of the underlying subgrade soils in these areas should be modest, up to 25 millimetres, and will largely occur during construction.

Over the western portion of the project some of the retaining walls will be founded on or above deposits of sensitive silty clay, particularly at retaining walls 11S and 12S where significant

thicknesses of these cohesive soils exist. Construction of the embankment widenings will generally involve the placement of variable depths of additional fill along each retaining wall alignment, depending on the geometry of the widening, retaining existing embankment. The resulting additional loads on the widening subgrade will in some cases exceed the deposits' preconsolidation pressures and significant settlement magnitudes may occur as the result of primary consolidation of the clayey deposits. Additionally, the sensitive silty clay soils in this area can undergo secondary compression settlements, of up to 100% of the primary consolidation settlement, when 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.

Further discussion of the potential settlements and possible mitigation measures are provided for each wall location in the site specific sections

6.3.5 Embankment Design and Construction

Any surficial topsoil, organic matter and softened / loosened soils should be stripped from within the limits of the widening, including the existing embankment sideslope and the new footprint. All subgrade soils should be proof-rolled prior to fill placement.

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 mm can be tolerated. 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 105S10. This guideline is appropriate where earth filling will be used to construct the embankment widening.

MSE embankments should generally be placed on the surface of the native soils after removal of any existing fill materials 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 materials are 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.

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.

The final lift prior to placement of the granular subbase and base courses should be compacted to 100 percent of the 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.

The new embankment fills should be benched into the existing embankment in accordance with OPSD 208.010.

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

6.3.6 Roadway Protection

It is anticipated that temporary roadway protection may be required along Highway 417 at some retaining wall locations to permit construction of the retaining wall or MSE slope, depending on the construction staging

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.

It may be feasible to embed soldier piles or sheet steel piling sufficiently into the overburden at some retaining wall locations 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.

Temporary excavation support systems should be designed and constructed in accordance with MTO's Special Provision 105S19. The lateral movement of temporary shoring systems should meet Performance Level 2 as specified in SP105S19, provided that any buried utilities that may be present adjacent to the excavations can tolerate this magnitude of deformation.

6.3.7 Design and Construction Considerations

The design and construction considerations at each retaining wall/embankment location are discussed in the following sections.

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.

The monitoring wells installed for this investigation in Boreholes 06-104, 06-132, 06-139, 06-143 and the standpipes installed in Boreholes 06-2, 06-5, 06-12, 06-14 and 06-18 will be decommissioned.

6.4 Retaining Wall 1S (EBL 22+150 to 22+475)

6.4.1 General – RW 1S

Highway 417 has been constructed in cut along this section and the existing side slopes along the south side of the highway range in height from about 1.2 to 1.5 m.

A toe wall about 0.8 to 1 m. in height has been proposed along this alignment and this wall may be constructed in accordance with OPSD 3120.100.

6.4.2 Limits States Factored Geotechnical Resistance – RW 1S

The toe wall may be founded on a minimum 0.6 m thick compacted levelling pad of Granular A or Granular B Type II engineered fill, compacted to 95 percent of the Standard Proctor maximum dry density in accordance with MTO's Special Provision 105S10, constructed on the surface of the native sand or silty clay soils. A factored geotechnical resistance at Ultimate Limit States (ULS) of 300 kPa at this location may be assumed as per OPSD 3120.100. The Servicability Limit States (SLS) resistance for this toe wall is considered to be 150 kPa.

6.4.3 Design and Construction Considerations – RW 1S

Excavations to subgrade depth for placement of the engineered fill would extend to about 1 m depth below the existing ground surface. The excavations will typically extend through limited thicknesses of fill materials overlying native sand and very stiff weathered silty clay. The groundwater level at the site is typically about 2 to 3 m below the proposed profile grade.

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 overburden materials are classified as Type 1 soils and the existing fill materials are classified as Type 3 soil, according to the OHSA. Temporary excavations (i.e. those which are only open for a

relatively short period) through these overburden soils above the water table should be made with side slopes no steeper than 1 horizontal to 1 vertical (1H:1V).

The groundwater level at the site is typically below the anticipated founding depth. Excavations to subgrade depth for placement of compacted engineered fill 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.

A 406 mm diameter watermain extends parallel to this retaining wall and the obvert of the watermain is at about elevation 78.7 m. at the western end of the wall and at about 78.2 m. at the eastern end of the wall. The watermain is about 2.5 m from the toe wall over the western portion of the alignment and directly under the toe wall over the eastern portion of the alignment. From the above it is likely that the existing fill materials surrounding the watermain will be encountered during construction of the toe wall over the eastern portion of the alignment. Where these fill materials are encountered they should be removed and replaced with engineered fill compacted to 95 percent of the standard Proctor maximum dry density in accordance with MTO's Special Provision 105S10.

In addition, the existing depth of earth cover for frost protection of the watermain ranges from about 1.5 to 1.8 m. and the widening will reduce that depth of earth cover to about 1 m along the eastern portion of the alignment. The 406 mm watermain should be insulated with high density extruded polystyrene foam where the earth cover over the watermain will be reduced to less than existing. The insulation should extend horizontally a distance of 1.8 metres, less the depth of overburden above the insulation, on either side of the watermain along the length of the pipe. In preparation for the insulation, a levelling pad consisting of 25 millimetres of mortar sand meeting the gradation limits of Table 1 in Canadian Standards Association (CSA) standard A17-04 (clause 5.3.2.2) should be placed on the approved subgrade. Care must be taken to ensure that the insulation is not damaged during construction. All joints should be carefully lap jointed and glued where and if possible.

The watermain will likely experience a significant increase in vertical stress and the watermain may be vulnerable to damage during construction of the widening. If it is considered fragile, moving the watermain should be considered.

6.5 Retaining Wall 2S & 3S (EBL 22+600 to 22+715)

6.5.1 Retaining Wall / Embankment Options – RW 2S & 3S

The existing embankments along the alignment of retaining wall 2S & 3S are about 3 to 4 m in height and sloped at approximately 2H:1V. Feasible retaining wall alternatives for the proposed widening at this location include:

- Full height reinforced soil system (RSS) wall;
- 1H:1V mechanically stabilized earth (MSE) slope;
- 2H:1V slope with a reinforced concrete toe wall up to 1.5 m in height; and
- 2H:1V slope with an OPSD toe wall up to 1.5 m in height.

These four alternatives are considered to be feasible at this location provided that the retaining wall is supported on the native soils underlying the fill materials and topsoil. 2H:1V, or flatter, slopes may be placed on the existing fill materials overlying the buried topsoil layer.

It is understood that a 'perched' retaining wall is under consideration at this location. A buried layer of topsoil is indicated to exist at the western end of this wall and it is not considered feasible to support a 'perched' RSS wall on the fill materials overlying this low strength soil layer. Additionally, the required embedment depth for a 'perched' RSS wall would result in founding elevations at or very near the surface of the native soils over most of the alignment; the much higher resistance available from the native soils make those soils much preferred for founding the retaining wall.

6.5.2 Retaining Wall Foundations – RW 2S & 3S

Shallow foundations on the native weathered silty clay or glacial till are recommended for the support of reinforced concrete toe walls, OPSD toe walls or RSS walls. The existing fill materials and buried topsoil are not suitable for support of walls and need to be removed from the foundation areas. For footings on native soil, the borehole information indicates maximum founding levels for the retaining walls as follows:

Borehole No.	Station	Maximum Founding Elevation (m)	Founding Material
06-106	22+610	79.3	Till
06-107	22+655	79.8	Till
06-108	22+700	78.9	Weathered Silty Clay
06-4	22+725	77.5	Weathered Silty Clay

The above design founding elevations are provided based on the borehole data available. It must be confirmed during construction that the native soil elevations are consistent with those anticipated. Provision should be made in the Contract documents for additional excavation and placement of engineered fill should the existing fill materials extend deeper than anticipated. MTO's Special Provision 902S01 should be included in the Contract Documents requiring inspection and approval of the foundation area by the Quality Verification Engineer prior to placement of engineered fill or footing construction.

Higher founding levels can also be used provided the grade is raised from the native subgrade up to the founding level using compacted engineered fill.

A factored geotechnical resistance at ULS of 250 kPa and a geotechnical resistance at SLS (for 25 mm of settlement) of 150 kPa may be used for the design of footings for a cantilevered reinforced concrete wall placed on the surface of the weathered silty clay or the glacial till. Both the weathered silty clay and the glacial till are not considered to be compressible soils and any differential settlement should be less than 15 mm.

An OPSD toe wall may be founded on a compacted leveling pad of Granular A or Granular B Type II engineered fill, compacted to 95 percent of the Standard Proctor maximum dry density in accordance with MTO's Special Provision 105S10, constructed on the surface of the native silty clay or glacial till soils. A factored geotechnical resistance at Ultimate Limit States (ULS) of 300 kPa at this location may be assumed as per OPSD 3120.100. The Servicability Limit States (SLS) resistance for this toe wall is considered to be 150 kPa.

A RSS wall, designed with the geotechnical resistances given above for cantilever retaining walls, may be founded on a minimum 0.3 m thick compacted levelling pad constructed, as indicated above, on the surface of the native soils.

The unfactored parameter values in the following table may be used to calculate the lateral resistance to sliding across the footing-soil interface as noted in Section 6.3.1:

<i>Borehole Number</i>	<i>Station</i>	<i>Founding Elevation (m)</i>	<i>Founding Material</i>	<i>Drained Conditions (Long term Loading)</i>		<i>Undrained Conditions (Short Term Loading)</i>
				<i>tan δ^*</i>	<i>c' (kPa)</i>	<i>Su (kPa)</i>
06-106	22+610	79.3	Weathered Silty Clay	0.43	0	50
06-107	22+655	79.8	Till	0.50	0	
06-108	22+700	78.9	Till	0.50	0	
06-4	22+725	77.5	Weathered Silty Clay	0.43	0	50

Note: The $\tan \delta^*$ values are based on 2/3 of the soil friction angle.

At Boreholes 06-106 and 06-4 where the footings may be placed on the weathered silty clay, the lateral resistance to both long term and short term loading needs to be evaluated. Resistance values for both conditions are provided in the above table.

6.5.3 Site Coefficient & Seismic Liquefaction – RW 2S & 3S

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.

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 or embankments.

6.5.4 Lateral Earth Pressures for Design – RW 2S & 3S

The following seismic active pressure coefficients (K_{AE}) for the two backfill cases (Case I and Case II) may be used in design:

SEISMIC ACTIVE PRESSURE COEFFICIENTS, K_{AE}

	Case I	Case II	
		Granular A	Granular B Type II
Yielding wall	0.39	0.30	0.30
Non-yielding wall	0.62	0.50	0.50

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 0.20. This corresponds to a displacement value of approximately 50 mm at this site.

6.5.5 Retaining Wall and Embankment Stability – RW 2S & 3S

RSS walls up to 3.6 m in height, low height (i.e., less than 1.5 m) cantilevered reinforced concrete walls at the toe of a 2H:1V slope or OPSD toe walls below a 2H:1V slope founded on the native weathered silty clay or glacial till after removal of the surficial random fill materials and topsoil will have a factor of safety of greater than 1.3 against deep-seated global instability.

Embankment widening at this location may be accomplished with a 1H:1V MSE slope in place of RSS or toe walls. Based on the borehole results, the embankment widening subgrade soils will consist of fill materials underlain by glacial till or weathered silty clay. The existing fill materials are generally composed of crushed stone or sand with varying amounts of silt, cobbles and boulders and these materials may be left in place after removal of any overlying topsoil.

With appropriate subgrade preparation and proper placement of earth or granular soils, the 3 to 4 m high MSE embankment with 1H:1V side slopes founded on the existing fill materials and native soils, will have a factor of safety greater than 1.3 against deep seated slope instability.

Pseudo-static seismic slope stability analyses for the above configurations also indicate that the RSS walls and embankment side slopes will have factors of safety of greater than 1.1.

6.5.6 Embankment Settlement – RW 2S & 3S

Some settlement of the embankment subgrade can be expected due to compression of the clayey soils (i.e., the weathered clay crust). However, the resulting consolidation settlements are estimated to correspond solely to recompression of the clayey deposits (i.e. no consolidation into the virgin compression range). The total estimated magnitude of the settlements resulting from this recompression will be less than 25 mm. It is expected that most of the settlement would occur quite rapidly during construction.

6.5.7 Design and Construction Considerations – RW 2S & 3S

Excavations to subgrade depth for placement of engineered fill or footing construction would extend to less than about 2 m depth below the existing ground surface. The excavations will typically extend through limited thicknesses of fill materials and topsoil overlying sandy fill materials, very stiff weathered silty clay or glacial till. The groundwater level along this alignment is typically about 2 m below the present ground surface.

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 overburden materials are classified as Type 1 soils and the existing fill materials are classified as Type 3 soil, according to the OHSA. Temporary excavations (i.e., those which are only open for a relatively short period) through these overburden soils above the water table should be made with side slopes no steeper than 1 horizontal to 1 vertical (1H:1V).

The groundwater level at the site is typically at or below the anticipated founding depth. Excavations 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.

A 406 mm diameter watermain extends parallel to this retaining wall and the obvert of the watermain is at about elevation 78 m. The watermain is about 1.5 m from the toe of the proposed 1H:1V MSE slope and it is unlikely it will experience a significant increase in vertical stress. However, the watermain may be vulnerable to damage during construction of the widening and if it is fragile moving the watermain should be considered. It is also understood that if 2:1 side slopes with toe walls are considered for this location, the watermain will be relocated.

6.6 Retaining Wall 4S (EBL 22+745 to 23+020)

6.6.1 Retaining Wall / Embankment Options – RW 4S

The existing embankments along the alignment of retaining wall 4S are about 4.0 to 5.5 m. in height and sloped at approximately 2.5H:1V or shallower. Feasible retaining wall alternatives for the proposed widening at this location include:

- 'Perched' RSS wall;
- Full height reinforced soil system (RSS) wall; and
- 1H:1V mechanically stabilized earth (MSE) slope.

6.6.2 Retaining Wall Foundations – RW 4S

Shallow foundations on the native sand at Borehole 06-111, on the clayey silt at Borehole 06-112 or glacial till are recommended for the support of full height RSS walls founded on a minimum 0.3 m thick compacted levelling pad.

The existing fill materials and topsoil are not suitable for support of full height retaining wall footings and need to be removed from the foundation areas. For footings on native soil, the borehole information indicates maximum founding levels for the RSS wall as follows:

Borehole No.	Station	Maximum Founding Elevation (m)	Founding Material
06-3	22+746	76.7	Till
06-111	22+812	78.4	Sand
06-112	22+882	78.2	Clayey Silt
06-113	22+948	77.8	Till
06-114	23+024	77.8	Till

The above design founding elevations are provided based on the borehole data available. It must be confirmed during construction that the native soil elevations are consistent with those anticipated. Provision should be made in the Contract documents for additional excavation and placement of engineered fill should the existing fill materials extend deeper than anticipated. MTO's Special Provision 902S01 should be included in the Contract Documents requiring inspection and approval of the foundation area by the Quality Verification Engineer prior to placement of engineered fill or footing construction.

Higher founding levels can also be used provided the grade is raised from the native subgrade up to the founding level using compacted engineered fill.

A factored geotechnical resistance at ULS of 250 kPa and a geotechnical resistance at SLS (for 25 mm of settlement) of 150 kPa may be used for the design of a RSS wall supported on the surface of the native sand, clayey silt or the glacial till.

A 'perched' RSS wall at the new edge of pavement (EP), up to about 2.6 m in height, may be constructed on the existing sandy embankment fill materials provided that the reinforcing extends to at least 1 metre below the surface at the face of the wall. The slope in front of the wall, extending from at least 1 metre above the founding elevation laterally away from the wall should be no steeper than 2H:1V.

The unfactored parameter values in the following table may be used to calculate the lateral resistance to sliding across the footing-soil interface as noted in Section 6.3.1:

<i>Borehole Number</i>	<i>Station</i>	<i>Founding Elevation (m)</i>	<i>Founding Material</i>	<i>Drained Conditions (Long term Loading)</i>		<i>Undrained Conditions (Short Term Loading)</i>
				<i>tan δ^*</i>	<i>c' (kPa)</i>	<i>Su (kPa)</i>
06-3	22+746	76.7	Till	0.50	0	
06-111	22+812	78.4	Sand	0.50	0	
06-112	22+882	78.2	Clayey Silt	0.43	0	50
06-113	22+948	77.8	Till	0.50	0	
06-114	23+024	77.8	Till	0.50	0	

At Borehole 06-112 where the footings may be placed on the clayey silt, the resistance to both long term and short term loading needs to be evaluated. Resistance values for both conditions are provided in the above table.

6.6.3 Site Coefficient & Seismic Liquefaction – RW 4S

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.

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 or embankments.

6.6.4 Lateral Earth Pressures for Design – RW 4S

The following seismic active pressure coefficients (K_{AE}) for the two backfill cases (Case I and Case II) may be used in design:

SEISMIC ACTIVE PRESSURE COEFFICIENTS, K_{AE}

	Case I	Case II	
		Granular A	Granular B Type II
Yielding wall	0.39	0.30	0.30
Non-yielding wall	0.62	0.50	0.50

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 0.20. This corresponds to a displacement value of approximately 50 mm at this site.

6.6.5 Retaining Wall and Embankment Stability – RW 4S

A RSS wall, up to 5.5 m in height, founded on the native soils after removal of the surficial random fill materials and topsoil will have a factor of safety of greater than 1.3 against deep-seated global instability.

A 'perched' RSS wall at the new edge of pavement (EP), up to about 2.6 m in exposed height, with an embedment depth of at least 1 m and with a toe slope no steeper than 2H:1V will also have a factor of safety greater than 1.3 against deep-seated global instability.

Embankment widening at this location may be accomplished with a 1H:1V MSE slope. Based on the borehole results, the embankment widening subgrade soils will consist of clayey silt, sand or glacial till. The existing fill materials are not considered to be suitable for support of MSE embankments and should be removed within the widening footprint.

With appropriate subgrade preparation and proper placement of earth or granular soils, the up to 5.5 m high MSE embankment with 1H:1V side slopes founded on the native soils, will have a factor of safety greater than 1.3 against deep seated slope instability.

Pseudo-static seismic slope stability analyses for the above configurations also indicate that the retaining walls and embankment side slopes will have factors of safety of greater than 1.1.

6.6.6 Embankment Settlement – RW 4S

Some settlement of the embankment subgrade can be expected due to compression of the clayey soils (i.e., the clayey silt). The total estimated magnitude of the settlements resulting from this recompression will be less than 25 mm and it is expected that most of the settlement would occur quite rapidly during construction.

6.6.7 Design and Construction Considerations – RW 4S

Excavations to subgrade depth for placement of engineered fill or footing construction would extend to less than 1 m depth below the existing ground surface at the borehole locations. The excavations will typically extend through fill materials and topsoil overlying clayey silt, sand or glacial till. The groundwater level along this alignment appears to be 1 to 2 m. below the present ground surface.

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 overburden materials are classified as Type 1 soils and the existing fill materials are classified as Type 3 soil, according to the OHSA. Temporary excavations (i.e. those which are only open for a relatively short period) through these overburden soils above the water table should be made with side slopes no steeper than 1 horizontal to 1 vertical (1H:1V).

Excavations 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.

A 406 mm diameter watermain extends parallel to this retaining wall and the obvert of the watermain is at about elevation 76.5 m indicating that the watermain extends through the bedrock or glacial till at this site. The watermain is indicated to be directly below the toe of the proposed MSE embankment and will likely experience an increase in vertical stress. Additionally, the watermain may be vulnerable to damage during construction of the widening and, if it is fragile, moving the watermain should be considered.

6.7 Retaining Wall 5S (EBL 23+145 to 23+325)

6.7.1 Retaining Wall / Embankment Options – RW 5S

The existing embankments along the alignment of retaining wall 5S are about 1.0 to 2.5 m in height and sloped at approximately 3H:1V or shallower. Feasible retaining wall alternatives for the proposed widening at this location include:

- Reinforced soil system (RSS) wall;
- Soldier pile and lagging wall; and,
- Cantilevered reinforced concrete retaining wall.

Construction of the RSS wall may require full closure of the highway off-ramp during construction to install the reinforcement. The use of a MSE slope is not considered feasible due to space restrictions.

6.7.2 Retaining Wall Foundations – RW 5S

6.7.2.1 Shallow Foundations

Shallow foundations on the native glacial till are recommended for the support of reinforced concrete or RSS retaining walls.

The existing fill materials and topsoil are also not suitable for support of retaining wall footings and need to be removed from the foundation areas. For footings on native soil, the borehole information indicates maximum founding levels for the retaining walls as follows:

Borehole No.	Station	Maximum Founding Elevation (m)	Founding Material
06-118	23+148	77.6	Till
06-119	23+208	77.4	Till
06-120	23+259	76.4	Till
06-121	23+294	76.5	Till
06-122	23+326	76.7	Till

The above design founding elevations are provided based on the borehole data available. It must be confirmed during construction that the native soil elevations are consistent with those anticipated. Provision should be made in the Contract documents for additional excavation and placement of engineered fill should the existing fill materials extend deeper than anticipated. MTO's Special Provision 902S01 should be included in the Contract Documents requiring inspection and approval of the foundation area by the Quality Verification Engineer prior to placement of engineered fill or footing construction.

Higher founding levels can also be used provided the grade is raised from the native subgrade up to the founding level using compacted engineered fill.

A factored geotechnical resistance at ULS of 250 kPa and a geotechnical resistance at SLS (for 25 mm of settlement) of 150 kPa may be used for the design of footings placed on the surface of the glacial till.

A RSS wall, designed with the geotechnical resistances given above for cantilever retaining walls, may be founded on a minimum 0.3 m thick compacted levelling pad constructed on the surface of the native soils.

Resistance to lateral forces for toe wall footings founded on the native soils should be calculated in accordance with Section 6.7.5 of the *CHBDCI* as noted in Section 6.3.1 of this report. An unfactored $\tan \delta^*$ value of 0.50 may be used to calculate the lateral resistance to sliding across the base of footing-soil interface for all footings on native soil at this location and also where retaining walls will be supported on engineered fill.

6.7.2.2 Steel H-Pile Foundations

Steel H-piles socketed and full grouted into the shallow limestone bedrock may be used for construction of a soldier pile and concrete lagging retaining wall. The bedrock and grout have compatible strength properties and the bearing resistance of the bedrock may be taken as equal to the compressive strength of the grout.

6.7.3 Site Coefficient & Seismic Liquefaction – RW 5S

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.

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 or embankments.

6.7.4 Lateral Earth Pressures for Design – RW 5S

The following seismic active pressure coefficients (K_{AE}) for the two backfill cases (Case I and Case II) may be used in design:

SEISMIC ACTIVE PRESSURE COEFFICIENTS, K_{AE}

	Case I	Case II	
		Granular A	Granular B Type II
Yielding wall	0.39	0.30	0.30
Non-yielding wall	0.62	0.50	0.50

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 0.20. This corresponds to a displacement value of approximately 50 mm at this site.

6.7.5 Retaining Wall and Embankment Stability – RW 5S

Retaining walls, up to 2.5 m in height, founded on the native glacial till after removal of the surficial random fill materials and topsoil will have a factor of safety of greater than 1.3 against deepseated global instability.

Pseudo-static seismic slope stability analyses for the above configurations also indicate that the retaining wall will have factors of safety of greater than 1.1.

6.7.6 Embankment Settlement – RW 5S

Some settlement of the embankment subgrade can be expected due to compression of the glacial till soils. The total estimated magnitude of the settlements resulting from this compression will be less than 25 mm and it is expected that most of the settlement would occur quite rapidly during construction.

6.7.7 Design and Construction Considerations – RW 5S

Excavations to subgrade depth for placement of engineered fill or footing construction would extend to less than about 1.8 m depth below the existing ground surface. The excavations will typically extend through limited thicknesses of fill materials and topsoil overlying glacial till. The groundwater level at the site appears to be lower than the anticipated founding level.

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 overburden materials are classified as Type 1 soils and the existing fill materials are classified as Type 3 soil, according to the OHSA. Temporary excavations (i.e. those which are only open for a relatively short period) through these overburden soils above the water table should be made with side slopes no steeper than 1 horizontal to 1 vertical (1H:1V).

Excavations 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.

A 406 mm diameter watermain extends parallel to this alignment over the western portion and crosses the alignment about midway along the alignment. Depending on the final configuration of the slopes and retaining walls the watermain may experience an increase in vertical stress due to the weight of additional fill. Additionally, the watermain may be vulnerable to damage during construction of the widening and, if it is considered fragile, moving the watermain should be considered.

6.8 Retaining Wall 6S (EBL 23+510 to 23+818)

6.8.1 Retaining Wall / Embankment Options – RW 6S

The existing embankments along the alignment of retaining wall 6S are about 4.5 to 5.0 m in height and sloped at approximately 2H:1V to 3H:1V. It is understood that embankments are under consideration for the proposed widening at this location. The toe of the embankment will extend past the bases of the two existing hydro towers along this alignment and the configurations in the following table are considered feasible:

Embankment Slope	Wall Height at Hydro Tower (m)	
	Station 23+585	Station 23+730
2H:1V	1.5	N/A
2.5H:1V	2	0
3H:1V	N/A	0.5

Note: N/A indicates that the slope geometry is not under consideration at that location.

Cantilevered reinforced concrete walls, OPSD toe walls or RSS walls have therefore been proposed at the hydro tower locations. A toe/cut-off wall in conjunction with the bridge wingwall, is also under consideration at Carling EB.

A buried layer of peat exists along this alignment and the presence of this low strength deposit limits the side slopes of embankments placed above this layer to 2.5H:1V or greater. Embankments with sides sloped at 2H:1V or steeper will require removal of the buried peat.

6.8.2 Retaining Wall Foundations – RW 6S

Shallow foundations on the native sand, clayey silt or glacial till are recommended for the support of reinforced concrete, RSS toe walls or OPSD toe walls.

The existing fill materials and buried peat are not suitable for support of retaining wall footings and need to be removed from the foundation areas. For footings on native soil, the borehole information indicates maximum founding levels for the retaining walls as follows:

Borehole No.	Station	Maximum Founding Elevation (m)	Founding Material
06-6	23+507	73.9	Till
06-127	23+595	73.7	Clayey Silt
06-128	23+664	74.1	Sand
06-129	23+743	74.1	Sand

The above design founding elevations are provided based on the borehole data available. It must be confirmed during construction that the native soil elevations are consistent with those anticipated. Provision should be made in the Contract documents for additional excavation and placement of engineered fill should the existing fill materials extend deeper than anticipated. MTO's Special Provision 902S01 should be included in the Contract Documents requiring inspection and approval of the foundation area by the Quality Verification Engineer prior to placement of engineered fill or footing construction.

Higher founding levels can also be used provided the grade is raised from the native subgrade up to the founding level using compacted engineered fill.

A factored geotechnical resistance at ULS of 250 kPa and a geotechnical resistance at SLS (for 25 mm of settlement) of 150 kPa may be used for the design of footings placed on the surface of the sand or the glacial till. For footings placed on the surface of the clayey silt a factored geotechnical resistance at ULS of 150 kPa and a geotechnical resistance at SLS (for 25 mm of settlement) of 100 kPa may be used for design. Footings supported on different materials (i.e., clayey silt and till) should be designed using the geotechnical resistance of the least competent material (i.e., the clayey silt).

A RSS wall, designed with the geotechnical resistances given above for cantilever retaining walls, may be founded on a minimum 0.3 m thick compacted levelling pad constructed on the surface of the native soils.

An OPSD toe wall may be founded on a compacted levelling pad of Granular A or Granular B Type II engineered fill, compacted to 95 percent of the Standard Proctor maximum dry density in accordance with MTO's Special Provision 105S10, constructed on the surface of the native silty clay or glacial till soils. A factored geotechnical resistance at Ultimate Limit States (ULS) of 300 kPa at this location may be assumed as per OPSD 3120.100. The Servicability Limit States (SLS) resistance for this toe wall is considered to be 150 kPa.

The unfactored parameter values in the following table may be used to calculate the lateral resistance to sliding across the footing-soil interface as noted in Section 6.3.1:

<i>Borehole Number</i>	<i>Station</i>	<i>Founding Elevation (m)</i>	<i>Founding Material</i>	<i>Drained Conditions (Long term Loading)</i>		<i>Undrained Conditions (Short Term Loading)</i>
				<i>tan δ^*</i>	<i>c' (kPa)</i>	<i>Su (kPa)</i>
06-6	23+507	73.9	Till	0.50	0	
06-127	23+595	73.7	Clayey Silt	0.43	0	50
06-128	23+664	74.1	Sand	0.50	0	
06-129	23+743	74.1	Sand	0.50	0	

At Borehole 06-127 where the footings may be placed on the clayey silt, the resistance to both long term and short term loading needs to be evaluated. Resistance values for both conditions are provided in the above table.

6.8.3 Site Coefficient & Seismic Liquefaction – RW 6S

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.

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 or embankments.

6.8.4 Lateral Earth Pressures for Design – RW 6S

The following seismic active pressure coefficients (K_{AE}) for the two backfill cases (Case I and Case II) may be used in design:

SEISMIC ACTIVE PRESSURE COEFFICIENTS, K_{AE}

	Case I	Case II	
		Granular A	Granular B Type II
Yielding wall	0.39	0.30	0.30
Non-yielding wall	0.62	0.50	0.50

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 0.20. This corresponds to a displacement value of approximately 50 mm at this site.

6.8.5 Retaining Wall and Embankment Stability – RW 6S

Retaining walls at the toe of the slope, up to 2 m in height, founded on the native sand, weathered silty clay or engineered fill at least 2 m in width at the underside of the footing, will have a factor of safety greater than 1.3 against deep seated slope instability.

Based on the borehole results, the embankment widening subgrade soils will consist of fill materials underlain by peat, clayey silt to silty clay, sand or glacial till. The existing fill materials are generally composed of sand to sandy silt with varying amounts of clay, silt and gravel and these materials may be left in place.

Slope stability analyses indicate that embankments with 2H:1V, or steeper, side slopes will have a factor of safety of less than 1.3. The existing random fill materials and peat within the limits of the embankment widening may be removed and replaced with compacted engineered fill if slopes steeper than 2.5H:1V are considered.

Pseudo-static seismic slope stability analyses for the above configurations also indicate that the embankment side slopes will have factors of safety of greater than 1.1.

6.8.6 Embankment Settlement – RW 6S

Some settlement of the embankment subgrade can be expected due to compression of the clayey soils (i.e., weathered silty clay and, in particular, the clayey silt and unweathered silty clay at Borehole 06-127). However, the resulting consolidation settlements are estimated to correspond solely to recompression of the weathered deposits (i.e., no consolidation into the virgin compression range). Primary consolidation of the unweathered deposits may occur, however the magnitude of resulting settlement will be relatively small due to the limited thicknesses (i.e., less than 0.7 m) of these materials. The total estimated magnitude of the settlements resulting from the recompression and primary consolidation will be less than 25 mm. It is expected that most of the settlement would occur quite rapidly and would likely be completed prior to final paving of the new lanes.

The foundations for the hydro towers along this retaining wall alignment will be within the zone of influence of the embankment loading but it is anticipated that the differential settlement of the tower foundations, founded in the glacial till near the surface of auger refusal, will be less than 25 mm in magnitude.

6.8.7 Design and Construction Considerations – RW 6S

Excavations to subgrade depth for placement of engineered fill or footing construction may extend to about 2.5 m depth below the existing ground surface. The excavations will typically

extend through limited thicknesses of fill materials and topsoil and sandy fill materials, loose clayey silt, silty clay, sand or glacial till. The groundwater level at the site ranges from about Elevation 73 to 74 m.

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 overburden materials above the water table are classified as Type 1 soils and the clayey silt, unweathered silty clay and existing fill materials are classified as Type 3 soil, according to the OHSA. Temporary excavations (i.e. those which are only open for a relatively short period) through these overburden soils should be made with side slopes no steeper than 1 horizontal to 1 vertical (1H:1V).

Excavations 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.

A 406 mm diameter watermain extends parallel to this retaining wall, in addition to the 1,220 mm watermain which is being moved, and the obvert of the watermain ranges from about elevation 71 to 74 m. indicating that the watermain extends through the bedrock or glacial till at this site. The watermain is indicated to be about 1.5 m from the proposed toe walls at the hydro towers and may experience a significant increase in vertical stress due to the weight of additional embankment fill. Additionally, the watermain may be vulnerable to damage during construction of the toe walls and, if it is fragile, moving the watermain should be considered. However if the watermain is left in place, the existing fill materials surrounding the watermain may be encountered during construction of the toe walls at the hydro towers. Where these fill materials are encountered below the toe wall foundations they should be removed and replaced with engineered fill compacted to 95 percent of the standard proctor maximum dry density in accordance with MTO's special Provision 105S10.

A 1,525 mm diameter storm sewer and a 381 mm diameter sanitary sewer cross under this alignment at about Station 23+800 and the invert elevations are about 73 and 71.5 m, respectively, which is near or within the glacial till. These services will experience some increase in vertical stress due to the weight of additional embankment fill and will undergo some differential settlement, over the width of the embankment side slope, which will likely be less than about 15 mm.

A 1,800 mm storm sewer crosses highway 417 at about 23+600 at an angle. Existing information indicates that this sewer was constructed by tunnelling through the bedrock and the construction of this retaining wall should not have any significant impacts on this sewer.

6.9 Retaining Wall (7S EBL 23+842 to 23+862)

6.9.1 Retaining Wall / Embankment Options – RW 7S

The existing embankments along the alignment of retaining wall 7S are about 5 to 6 m in height and sloped at approximately 2H:1V. It is understood that embankments with 2H:1V, or flatter, side slopes are under consideration for the proposed widening at this location and embankments with sides sloped at this inclination are considered feasible at this location.

An OPSD toe-wall, about 0.5 m in height, around the hydro tower is also under consideration at this location.

6.9.2 Limits States Factored Geotechnical Resistance – RW 7S

The toe wall may be founded on a minimum 0.5 m thick compacted levelling pad of Granular A or Granular B Type II engineered fill, compacted to 95 percent of the Standard Proctor maximum dry density in accordance with MTO's Special Provision 105S10, constructed on the surface of the existing fill materials at a suitable founding elevation to be specified by the designer. A factored geotechnical resistance at Ultimate Limit States (ULS) of 300 kPa at this location may be assumed as per OPSD 3120.100.

6.9.3 Embankment Stability – RW 7S

Slope stability analyses indicate that embankments with side slopes maintained at 2H:1V, or flatter, founded on the existing fill materials and native soils after stripping of any surficial topsoil will have factors of safety greater than 1.3 against deep-seated slope instability. A 2H:1V, or flatter slope, with a toe-wall up to 0.5 m in height will also have a factor of safety of greater than 1.3.

Pseudo-static seismic slope stability analyses for the above configurations also indicate that the embankment side slopes will have factors of safety of greater than 1.1.

6.9.4 Embankment Settlement – RW 7S

Some settlement of the embankment subgrade can be expected due to compression of the clayey soils (i.e., the weathered silty clay and, in particular, the unweathered silty clay at Borehole 06-9). However, the resulting consolidation settlements are estimated to correspond solely to recompression of the weathered deposits (i.e. no consolidation into the virgin compression range). Primary consolidation of the unweathered deposits may occur, however the magnitude of resulting settlement will be relatively small due to the limited thickness (i.e., about 0.9 m) of these materials. The total estimated magnitude of the settlements resulting from the

recompression and primary consolidation will be less than 25 mm. It is expected that most of the settlement would occur quite rapidly and would likely be completed prior to final paving of the new lanes.

The hydro tower at this location is in close proximity to the proposed toe wall and the foundations for this tower, which are understood to be founded in the glacial till or sand at a depth of about 3 m, will be within the zone of influence of the embankment loading. These towers will therefore experience differential settlement; although likely less than 25 mm.

6.10 Retaining Wall 8S (EBL 23+960 to 23+990)

6.10.1 General – RW 8S

This retaining wall will be located between Highway 417 and the Carling EB to EBL ramp. The difference in elevation between the highway and the on-ramp is up to about 2.5 m at this location.

A toe wall up to about 1 m in height will be required at this location and this wall may be constructed in accordance with OPSD 3120.100.

6.10.2 Limits States Factored Geotechnical Resistance – RW 8S

The toe wall may be founded on a minimum 1.0 m thick compacted levelling pad of Granular A or Granular B Type II engineered fill, compacted to 95 percent of the Standard Proctor maximum dry density in accordance with MTO's Special Provision 105S10, constructed on the surface of the existing fill materials at a suitable founding elevation to be specified by the designer. A factored geotechnical resistance at Ultimate Limit States (ULS) of 300 kPa at this location may be assumed as per OPSD 3120.100.

6.11 Retaining Wall (9S EBL 23+975 to 24+050)

6.11.1 Retaining Wall / Embankment Options – RW 9S

The existing embankments along the alignment of retaining wall 9S are about 5 to 6 m in height and sloped at approximately 4H:1V to 2H:1V. It is understood that 2H:1V side slopes are being considered at this location with a retaining wall at the hydro tower at Station 23+995.

The existing sandy fill materials extend to depths of about 4 m at the location of the hydro tower. These very loose to loose fill materials are not suitable for founding a reinforced concrete retaining wall. Removing the fill materials and founding the retaining wall on the native soils at depths greater than 4 m is likely not practical or economic.

Deep foundations could also be considered for support of the wall at this location but would be relatively costly.

A RSS wall is a feasible option for the retaining wall required at the hydro tower. The estimated total settlements due to compression of the fill materials and consolidation (primary and secondary) of the underlying silty clay are in excess of the 35 mm limit generally considered acceptable for this type of wall from an appearance standpoint. Considering that it is located behind the hydro tower and that the differential settlements will likely be less than this value and also since the wall is relatively short and low in height the effects of this settlement should not be significantly noticeable. A layer of bi-axial geo-grid should also be placed underneath the pad of engineered fill to reduce the magnitude of differential settlements.

6.11.2 Site Coefficient & Seismic Liquefaction – RW 9S

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.

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 or embankments.

6.11.3 Retaining Wall and Embankment Stability – RW 9S

The RSS wall at the hydro tower should be founded on a minimum 1.0 m thick compacted levelling pad of Granular B Type II engineered fill, compacted to 95 percent of the standard Proctor maximum dry density in accordance with MTO's Special Provision 105S10, constructed on the surface of the existing fill materials. With appropriate subgrade preparation and proper placement of earth or granular soils, the approximately 2 m high RSS wall will have a factor of safety greater than 1.3 against deep seated slope instability.

Slope stability analyses indicate that the 5 to 6 m high embankments with side slopes maintained at 2H:1V founded on the existing fill materials and native soils after stripping of any surficial topsoil will have factors of safety greater than 1.3.

Pseudo-static seismic slope stability analyses for the above configurations also indicate that the retaining walls/embankments will have factors of safety of greater than 1.1.

6.11.4 Embankment Settlement – 9S

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 of the underlying grey silty clay to clay).

Analysis indicates that the effective stress level in the clayey deposits will approach the deposit's preconsolidation pressure. The resulting consolidation settlements therefore correspond to recompression of the clayey deposits and, to a much lesser extent, some consolidation in the virgin compression range.

The total magnitude of the primary consolidation settlements is estimated to be about 35 mm. It is expected that most of the settlement should be completed within about 6 months. Based on the anticipated loading approximately 35 mm of additional secondary compression is anticipated over a twenty-year time span, by which time resurfacing of the highway might be expected.

Based on the estimated settlements, it is recommended that the widened embankments be constructed as early as possible in the contract to allow the maximum amount of time available for settlement prior to paving of the Carling EB to 417 EB ramp.

The hydro tower along at this location is in close proximity to the proposed retaining wall and the foundations for this tower, which are understood to be founded in the weathered crust at a depth of about 3 m, will be within the zone of influence of the embankment loading. These towers will therefore experience differential settlement, although the magnitude of this differential settlement will likely be less than 25 mm.

6.12 Retaining Wall (11S EBL 24+160 to 24+515)

6.12.1 Retaining Wall / Embankment Options – RW 11S

The existing embankments along the alignment of retaining wall 11S are up to about 4.3 m in height. An existing retaining wall about 2 to 3 m in height is located along this alignment from about Station 24+250 extending to the eastern end of the alignment. Six options may be considered for the proposed widening at this location:

- Soldier pile and lagging wall (possibly with tie-backs);
- Cantilevered reinforced concrete wall;
- Reinforced soil system (RSS) wall;
- MSE slope;
- Combinations of the above (e.g., MSE slope with soldier pile and lagging at hydro towers); and,
- Lightweight fills in combination with the above wall and slope configurations.

The additional fill placed along this section of the widening may result in settlement magnitudes of up to 250 mm due to primary consolidation of the underlying unweathered

clay at this location. This settlement will be almost entirely differential with respect to the existing embankment which has been in place for more than 40 years.

The settlement magnitudes will depend on the type of wall selected. A soldier pile and lagging wall or RSS wall will result in the largest magnitude of settlement since the full weight of additional fill will be supported by the underlying soil. For these types of wall lightweight fills may be considered to reduce the settlements. However, it may not be feasible to install the reinforcing strips for a RSS wall within the expanded polystyrene fill. If a pile supported reinforced concrete wall is constructed, a significant portion of the weight of additional fill will be supported by the pile caps and this will in return reduce the loading imposed on the underlying soils. This reduced loading may reduce the total settlement magnitudes to less than 50 millimetres along most of the wall. Along the western end of the wall the primary consolidation settlement may be up to 120 mm due to the laterally larger extent of filling and the use of lightweight fills may be considered along this section to reduce the settlements.

In addition, the hydro towers along the alignment are in close proximity to the proposed retaining wall and the foundations for these towers, which are understood to be founded in the weathered crust at a depth of about 3 m, will be within the zone of influence of the embankment loading if a soldier pile and lagging wall is constructed. These towers will therefore experience differential settlement in excess of 25 mm. If a pile supported reinforced concrete wall is constructed the differential settlement of the hydro towers will likely be less than 25 mm.

It is therefore considered that the preferred option along this alignment is a reinforced concrete retaining wall supported on deep foundations.

6.12.2 Retaining Wall Foundations – RW 11S

6.12.2.1 Foundation Options

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

- Shallow foundations supported on the native silty clay soil;
- Foundations supported on steel H-piles founded on, or socketed into, the bedrock; and,
- Foundations supported on caissons founded on, or socketed into, the bedrock.

Shallow foundations supported on the native soils, is not considered practical or appropriate for this site since the bearing resistance of these intermediate to highly plastic soils of relatively low

shear strength would be insufficient for support of the retaining wall and the settlement of the foundations would be excessive as noted above.

6.12.2.2 Steel H-Pile Foundations

It is considered that the most feasible and cost-effective foundation options for this retaining wall would be piles or caissons, founded on or socketed into the bedrock.

Steel H-piles driven to found on the limestone / dolomitic limestone bedrock may be used for support of the retaining walls.

An unfactored ULS uplift resistance of 190 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.

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. As discussed subsequently in Section 6.12.6 of this report, the embankment subgrade settlements are estimated to exceed about 50 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, the methods described in both the Canadian Foundation Engineering Manual as well as the US Transportation Research Board's report, "Design and Construction Manual For Downdrag on Uncoated and Bitumen-Coated Piles" [Briaud and Tucker (1994)] 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 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 300 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.

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.

<i>Soil Unit</i>	<i>n_h</i>	<i>s_u (kPa)</i>
Compacted engineered fill	18 MPa/m	—
Weathered silty clay crust above about Elev. 70 m – approximately 1 to 2 m thick (see Record of Borehole sheets)	—	60
Unweathered silty clay to clay extending to about 1.0 to 3.5 m. below about Elev. 70 m (see Record of Borehole sheets)	—	35
Glacial till below about Elev. 68 to 69 m. (see Record of Borehole sheets)	2 - 8 MPa/m	—

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

<i>Soil Unit</i>	<i>γ (kN/m³)</i>	<i>K_p</i>	<i>S_u (kPa)</i>
Compacted engineered fill	21	3.7	—
Weathered silty clay crust above about Elev. 70 m approximately 1 to 2 m thick (see Record of Borehole sheets)	—	—	60
Unweathered silty clay to clay extending to about 1.0 to 3.5 m. below about Elev. 70 m (see Record of Borehole sheets)	—	—	35
Glacial till below about Elev. 68 to 69 m. (see Record of Borehole sheets)	19.0	3.3	—

Additional lateral resistance can be provided by socketing the piles into the bedrock. 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.

6.12.2.3 Caisson Foundations

Caissons founded on or socketed into the limestone / dolomitic limestone bedrock may be used for support of the retaining walls.

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 1.5 m diameter caisson over its length is estimated to be 1150 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.

6.12.3 Site Coefficient & Seismic Liquefaction – RW 11S

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.

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 or embankments.

6.12.4 Lateral Earth Pressures for Design – RW 11S

The following seismic active pressure coefficients (K_{AE}) for the two backfill cases (Case I and Case II) may be used in design:

SEISMIC ACTIVE PRESSURE COEFFICIENTS, K_{AE}

	Case I	Case II	
		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.

6.12.5 Soil and Rock Anchors – RW11S

A soldier pile and lagging wall will likely require tie-backs to resist the lateral loads from the backfill. It is also understood that the crash barrier along this wall will be integral with the retaining wall and that as a result the lateral forces acting on this wall may be potentially high enough to require additional lateral restraint. This lateral restraint may be provided by tie-backs installed into the bedrock or attached to a passive soil anchor (deadman) installed within the embankment fill.

Rock anchors may be a practical option along the eastern portion of the wall where the bedrock may be at relatively shallow depths of about 8 m. Assuming a 45 degree angle from the attachment point at the back of the wall, rock anchors about 11 to 12 m in length would be

required. However, at the western end of the wall where rock may be at depths greater than 14 m, rock anchors more than 20 m in length may be necessary and may be costly to install.

Rock anchors could consist of grouted anchors and the factored bond stress at the concrete/rock interface may be taken as 1,000 kilopascals for ultimate limit state (ULS) design purposes.

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.

It is recommended that pull-out tests be carried out on anchors to confirm their pull-out capacity at the time of construction. The pull-out 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.

Resistance to lateral loads may also be provided by tie-backs 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 deadman would therefore be located about 3 to 4 metres from the back of the retaining wall or very near to the existing edge of pavement where 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.

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

Anchors should be installed in accordance with MTO's Special Provision No. 999S26 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.

A Non Standard Special Provision for the installation and testing of rock anchors has been included in Appendix D of the report.

6.12.6 Retaining Wall and Embankment Stability – RW 11S

With appropriate subgrade preparation and proper placement of earth or lightweight fills, the 4.3 m high reinforced concrete retaining walls founded on deep foundations, will have a factor of safety greater than 1.3 against deep seated slope instability.

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.

6.12.7 Embankment Settlement – RW 11S

If earth or granular fills are used settlement of the approach embankments will occur as a result of consolidation of the clayey soils on which the approaches will be founded.

The 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 the oedometer consolidation testing indicate that the effective stress level in the clayey deposits will exceed the deposit's preconsolidation pressure if earth fills are used for the embankment. The resulting consolidation settlements therefore correspond to recompression of the clayey deposits and, to a greater magnitude, consolidation in the virgin compression range.

The loading imposed on the underlying soils and the resulting magnitude of settlement will also depend on the wall construction. If a 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 50 to 250 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. 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 after 20 years. These settlements would be almost entirely differential with respect to the existing embankments and the existing rigid (i.e., concrete) pavement.

If a reinforced concrete wall, supported on piles, is constructed a portion of the weight of additional fill will be supported by the pile caps and this will in return reduce the loading imposed

on the underlying soils. The settlement magnitudes, due to primary consolidation of the underlying clayey soils, along most of the wall will be less than 50 millimetres. Between the western end of the wall and about Station 24+250 the primary consolidation settlement may be up to 120 mm due to the laterally larger extent of filling. However, this portion of the wall is alongside the Carling WB to 417 EB ramp which has flexible pavement, 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 3 years.

Based on the anticipated loading the magnitude of additional secondary compression, for a pile supported reinforced concrete wall, is anticipated to be less than 50 mm over a 20 year time span, by which time resurfacing of the highway might be expected.

Four options may be considered to reduce the primary and secondary consolidation settlements along the western end of a pile supported reinforced concrete retaining wall:

1. Ultra-light weight fill (i.e., slag with a unit weight of about 11.5 kN/m³) may be used to reduce the loading to below the deposit's preconsolidation pressure.
2. EPS fill may be used to reduce the net increase in loading along this section of wall to almost zero. This would require up to 1.5 to 2 metres of EPS fill.
3. Surcharge the approach embankment areas to increase the magnitude of settlement during the preload period, prior to paving.
4. Install wick drains, in combination with surcharging, to accelerate the consolidation settlement within the silty clay to clay.

The time required for most of the primary consolidation to take place is anticipated to be greater than 2 years and preloading/surcharging for this time period may not be practical. Wick drains could be used to accelerate the consolidation of the clayey soils but it may not be practical to install the wick drains through the existing embankment fills. The area where wick drains might be required is on the highway side slope immediately adjacent to, and potentially under overhead electric lines. One of the large towers for these lines is also in the area where wick drains might be required. Filling of the area to create a level pad might be necessary before installation of the wick drains and this would exacerbate any clearance issues between the wick drain installation rig and the overhead electrical lines. Additionally, the existing subsurface conditions might be very challenging. A 1200 mm watermain extends through the area where wick drains might be required. Although this utility is scheduled to be relocated prior to construction, the abandoned main and its associated fill and bedding materials would remain as an obstruction for driving wick drains. The costs and potential difficulties to install a relatively small number of wick drains in an area with the above challenges appear to make this option impractical.

Based on the estimated settlements, it is recommended that ultra-lightweight fill be used in the construction of the widening at the western end of the retaining wall. It is anticipated that this approach would reduce the primary consolidation settlements to less than 50 mm. EPS fill could also be considered, and could potentially reduce the magnitude of the settlements to near zero, but is significantly more expensive.

The estimates of primary and secondary settlement for the pile supported reinforced concrete wall given above are based upon preliminary information about the wall and backfill configuration. These estimates should be refined as more information becomes available during the course of the design.

6.12.8 Design and Construction Considerations – RW 11S

Excavations to subgrade depth for placement of EPS fill or pile cap construction would extend to less than about 1.8 m depth below the existing ground surface. The excavations will typically extend through limited thicknesses of fill materials overlying weathered silty clay. The groundwater level at the site is lower than the anticipated founding level.

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 overburden materials are classified as Type 1 soils and the existing fill materials are classified as Type 3 soil, according to the OHSA. Temporary excavations (i.e. those which are only open for a relatively short period) through these overburden soils above the water table should be made with side slopes no steeper than 1 horizontal to 1 vertical (1H:1V).

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.

6.12.9 Temporary Roadway Protection – RW 11S

It is anticipated that temporary roadway protection will be required along Highway 417 to permit construction of the retaining wall, MSE slope 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.

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

6.13 Retaining Wall 12S (EBL 24+540 to 24+595)

6.13.1 Retaining Wall / Embankment Options – RW 12S

The existing embankments along the alignment of retaining wall 12S are about 5.0 to 5.5 m in height and sloped at approximately 1.5H:1V or shallower. An existing retaining wall extends from about 24+595 to Island Park Drive. Three alternatives may be considered for the proposed widening at this location:

- Reinforced concrete retaining wall;
- 1H:1V mechanically stabilized earth (MSE) slope; and
- Reinforced soil system (RSS) wall.

The additional fill placed along this section of the widening may result in settlement magnitudes of up to 100 mm due to primary consolidation of the underlying unweathered clay at this location. This settlement will be almost entirely differential with respect to the existing embankment which has been in place for more than 40 years.

The settlement magnitudes will depend on the type of wall selected. A soldier pile and lagging wall will result in the largest magnitude of settlement since the full weight of additional fill will be supported by the underlying soil. However, if a pile supported reinforced concrete wall is constructed, a significant portion of the weight of additional fill will be supported by the pile caps and this will in return reduce the loading imposed on the underlying soils. This reduced loading may result in settlement magnitudes of less than 25 millimetres along most of the wall.

In addition, the hydro towers along the alignment are in close proximity to the proposed retaining wall and the foundations for these towers, which are understood to be founded in the weathered crust at a depth of about 3 m, will be within the zone of influence of the embankment loading if a soldier pile and lagging wall is constructed. These towers will therefore experience differential settlement in excess of 25 mm. If a pile supported reinforced concrete wall is constructed the differential settlement of the hydro towers will likely be much less than 25 mm.

It is therefore considered that the preferred option along this alignment is a reinforced concrete retaining wall supported on deep foundations.

6.13.2 Retaining Wall Foundations – RW 12S

6.13.2.1 Foundation Options

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

- Shallow foundations supported on the native silty clay soil;
- Foundations supported on steel H-piles founded on, or socketed into, the bedrock; and
- Foundations supported on caissons founded on, or socketed into, the bedrock.

The first option, using shallow foundations supported on the native soils, is not considered practical or appropriate for this site since the bearing resistance of these intermediate to highly plastic soils of relatively low shear strength would be insufficient for support of the retaining wall and the settlement of the foundations would be excessive.

It is considered that the most feasible and cost-effective foundation options for this retaining wall would be piles or caissons, founded on or socketed into the bedrock.

6.13.2.2 Steel H-Pile Foundations

Steel H-piles driven to found on the limestone / dolomitic limestone bedrock may be used for support of the retaining walls.

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. As discussed subsequently in Section 6.13.6 of this report, the embankment subgrade settlements are estimated to exceed about 50 mm (from both primary consolidation and secondary compression) for a soldier pile and lagging retaining wall. 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, the methods described in both the Canadian Foundation Engineering Manual as well as the US Transportation Research Board's report, "Design and Construction Manual For Downdrag on Uncoated and Bitumen-Coated Piles" [Briaud and Tucker (1994)] 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 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 230 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.

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.

<i>Soil Unit</i>	<i>n_h</i>	<i>s_u (kPa)</i>
Compacted engineered fill	18 MPa/m	–
Weathered silty clay crust above about Elev. 70 m – approximately 1 to 2 m thick (see Record of Borehole sheets)	–	60
Unweathered silty clay to clay extending to about 1.0 to 3.5 m. below about Elev. 70 m (see Record of Borehole sheets)	–	35
Glacial till below about Elev. 68 to 69 m. (see Record of Borehole sheets)	2 - 8 MPa/m	–

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

<i>Soil Unit</i>	<i>γ (kN/m³)</i>	<i>K_p</i>	<i>S_u (kPa)</i>
Compacted engineered fill	21	3.7	–
Weathered silty clay crust above about Elev. 70 m approximately 1 to 2 m thick (see Record of Borehole sheets)	–	–	60
Unweathered silty clay to clay extending to about 1.0 to 3.5 m. below about Elev. 70 m (see Record of Borehole sheets)	–	–	35
Glacial till below about Elev. 68 to 69 m. (see Record of Borehole sheets)	19.0	3.3	–

Additional lateral resistance can be provided by socketing the piles into the bedrock.

6.13.2.3 Caisson Foundations

Caissons founded on or socketed into the limestone / dolomitic limestone bedrock may be used for support of the retaining walls.

6.13.3 Site Coefficient & Seismic Liquefaction – RW 12S

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.

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 or embankments.

6.13.4 Lateral Earth Pressures for Design – RW 12S

The following seismic active pressure coefficients (K_{AE}) for the two backfill cases (Case I and Case II) may be used in design:

SEISMIC ACTIVE PRESSURE COEFFICIENTS, K_{AE}

	Case I	Case II	
		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.

6.13.5 Rock Anchors – RW 12S

A soldier pile and lagging wall will likely require tie-backs to resist the lateral loads from the backfill and, based on the depth to bedrock, rock anchors may be a practical option.

Rock anchors could consist of grouted anchors and the factored bond stress at the concrete/rock interface may be taken as 1,000 kilopascals for ultimate limit state (ULS) design purposes.

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.

It is recommended that pull-out tests be carried out on anchors to confirm their pull-out capacity at the time of construction. The pull-out 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.

Anchors should be installed in accordance with MTO's Special Provision No. 999S26 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 and the free stressing zone.

A Non Standard Special Provision for the installation and testing of rock anchors has been included in Appendix D of the report.

6.13.6 Retaining Wall and Embankment Stability – RW 12S

Embankment widening at this location may be accomplished with a 1H:1V MSE slope. Based on the borehole results, the embankment widening subgrade soils will consist of fill materials underlain by silty clay and clay.

With appropriate subgrade preparation and proper placement of earth or granular soils, the 5.0 to 5.5 m high MSE embankment with 1H:1V side slopes founded on a minimum 0.75 m thick pad of engineered fill placed on the existing fill materials and native soils, will have a factor of safety greater than 1.3 against deep seated slope instability.

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.

6.13.7 Embankment Settlement – RW 12S

If earth or granular fills are used settlement of the approach embankments will occur as a result of consolidation of the clayey soils on which the approaches will be founded.

The 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 the oedometer consolidation testing indicate that the effective stress level in the clayey deposits will exceed the deposit's preconsolidation pressure if earth fills are used for the embankment. The resulting consolidation settlements therefore correspond to recompression of the clayey deposits and, to a greater magnitude, consolidation in the virgin compression range.

The loading imposed on the underlying soils and the resulting magnitude of settlement will also depend on the wall construction. If a 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 30 to 100 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. 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 after 20 years. These settlements would be almost entirely differential with respect to the existing embankments and the existing rigid (i.e., concrete) pavement.

If a reinforced concrete wall supported on piles is constructed a significant portion of the weight of additional fill will be supported by the pile caps and this will in return reduce the loading imposed on the underlying soils. The settlement magnitudes, due to primary consolidation of the underlying clayey soils, along most of the wall will be less than 25 millimetres.

Based on the anticipated loading the magnitude of additional secondary compression, for a pile supported reinforced concrete wall, is anticipated to be much less than 25 mm over a 20 year time span, by which time resurfacing of the highway might be expected.

The estimates of primary and secondary settlement for the pile supported reinforced concrete wall given above are based upon preliminary information about the wall and backfill configuration. These estimates should be refined as more information becomes available during the course of the design.

6.13.8 Design and Construction Considerations – RW 12S

Excavations to subgrade depth for placement of engineered fill or footing construction would extend to about Elevation 73.0 m. The excavations will typically extend through the existing fill materials overlying the native weathered silty clay. The groundwater level at the site appears to be between about elevations 70 and 71 m.

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 overburden materials are classified as Type 1 soils and the existing fill materials are classified as Type 3 soil, according to the OHSA. Temporary excavations (i.e. those which are only open for a relatively short period) through these overburden soils above the water table should be made with side slopes no steeper than 1 horizontal to 1 vertical (1H:1V).

Excavations 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 base of the excavations.

A 1,825 mm diameter storm sewer crosses under this embankment at about Station 24+560. The invert of this sewer is indicated to be at about Elevation 70.5 m or just above the unweathered silty clay. The embankment settlement due to primary consolidation of the unweathered silty clay at this location will likely be less than about 25 mm and the storm sewer will also undergo the same settlement.

A 1,220 mm diameter watermain extends parallel to this retaining wall and the invert of the watermain is at about elevation 70 m or just below the surface of the unweathered silty clay. It is understood that this watermain is to be moved prior to the widening construction, although the extent of the relocation has not been confirmed at the time of this report. However, based on the current location of the watermain at this location it will be outside the zone of influence of the additional embankment fill placed for the widening at this location.

6.13.9 Temporary Roadway Protection – RW 12S

It is anticipated that temporary roadway protection will be required along Highway 417 to permit construction of the retaining wall, MSE slope 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.

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

6.14 Retaining Wall 1N (WBL 22+610 to 22+747)

6.14.1 Retaining Wall / Embankment Options – RW 1N

The existing embankments along the alignment of retaining wall 1N are about 5 m in height and sloped at approximately 2H:1V. Feasible retaining wall alternatives at this location include:

1. Reinforced concrete retaining wall at the edge of the highway pavement;
2. Soldier pile and lagging wall at the edge of the highway pavement;

3. Full height reinforced soil system (RSS) wall;
4. A 'perched' RSS wall founded within the existing embankment fills approximately half way up the existing embankment side slope; and
5. A 1H:1V MSE slope.

A 900 mm diameter storm sewer extends along the alignment at this location and this sewer is about 3 m inside the toe of the existing side slope. It is understood that replacement of this sewer is under consideration but at the time of this report the timing of the replacement was not known.

A retaining wall, of either reinforced concrete or soldier pile and lagging construction, could be constructed between the edge of the widened highway and the storm sewer allowing for future access to this service without shoring. A RSS wall or MSE slope would be constructed almost directly over the existing storm sewer and access to this service for future maintenance or replacement would be severely limited since the reinforced soil structures would be significantly undermined by any excavations to expose the sewer; shoring of the highway and reconstruction of the wall or slope would likely be required.

It is understood that the storm sewer may be moved and the trench abandoned. If the storm sewer trench is abandoned by backfilling with compacted engineered fill all four options may be acceptably founded on the trench backfill.

The preferred option for this location will therefore depend on if the storm sewer will be moved or the desired level of future access if it is left in place.

6.14.2 Retaining Wall Foundations – RW 1N

6.14.2.1 Shallow Foundations

The existing fill materials are not considered to be suitable for support of a reinforced concrete wall. It is also not considered practical to support a retaining wall on spread footings on or within the very thin glacial till layer. At the western end of the alignment, the glacial till is absent at Borehole 06-109 where the existing fill materials overlie the bedrock. Over the remainder of the alignment, the till deposit is less than about 0.5 m thick and the much higher resistance available from the underlying bedrock make that strata much preferred for supporting the foundation loads.

It is therefore considered that the most feasible foundations for a reinforced concrete retaining wall is spread footings founded on the shallow bedrock.

The borehole information indicates bedrock surface levels along the wall alignment as follows:

Station	Borehole Number	Bedrock Surface Elevation (m)
22+610	06-109	75.9
22+688	06-110	76.0
22+747	06-1	75.5

The above bedrock surface elevations are provided based on the borehole data available. It must be confirmed during construction by the Quality Verification Engineer that the bedrock surface elevations are consistent with those anticipated. Provision should be made in the Contract documents for additional excavation and placement of mass concrete wherever the competent bedrock surface is below the design founding level. A Non-Standard Special Provision for the placement of mass concrete has been included in Appendix D of this report.

The upper portion of the bedrock is, in local areas, of poor quality (e.g., RQD values of 0 percent, as encountered in Boreholes 06-109 and 06-110). MTO's Special Provision SP902S01 should be included in the Contract Documents requiring inspection and approval of the foundation area by the Quality Verification Engineer prior to footing construction.

Spread footings placed on the surface of limestone bedrock or on mass concrete may be designed using a factored geotechnical resistance at Ultimate Limit States (ULS) of 2,000 kPa taking into account the poor quality of the upper portion of the bedrock. Serviceability Limit States (SLS) resistances do not apply to the design of footings on the limestone bedrock since the SLS resistance for 25 mm of settlement is greater than the factored axial geotechnical resistance at ULS.

Resistance to lateral forces (i.e., sliding resistance between the concrete footings and bedrock) should be calculated in accordance with Section 6.7.5 of the *CHBDC*. The coefficient of friction, $\tan \delta^*$, may be taken as 0.70 for cast-in-place concrete footings constructed on bedrock. This represents an unfactored value; in accordance with the *CHBDC*, a resistance factor of 0.8 is to be applied in calculating the horizontal resistance. If necessary, the sliding resistance of spread footings founded on the bedrock can be supplemented by dowelling into the bedrock. The horizontal resistance of the dowels is dependent on the strength of the bedrock, grout and steel. For this site, where the rock mass is essentially as strong as or is stronger than concrete, the design of the dowels in the rock may be handled in the same way as the dowel embedment into the concrete. This assumes that the unconfined compressive strength of the grout will be similar to that of the concrete. The dowels should have a minimum embedded length within the bedrock of 1 m and the structural strength of the dowel and compressive strength of the grout should not be exceeded. 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. A non-standard Special Provision for the installation, materials and testing of the dowels has been included in Appendix D of this report.

In order to limit the excavation width and the requirements for temporary protection of the highway the retaining wall footing may be significantly reduced in width and it may be necessary to install rock anchors to provide overturning resistance.

Rock anchors could consist of grouted anchors and the factored bond stress at the concrete/rock interface may be taken as 1,000 kilopascals for ultimate limit state (ULS) design purposes.

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.

It is recommended that pull-out tests be carried out on anchors to confirm their pull-out capacity at the time of construction. The pull-out 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.

Anchors should be installed in accordance with MTO's Special Provision No. 999S26 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 and the free stressing zone.

A factored geotechnical resistance at ULS of 250 kPa and a geotechnical resistance at SLS (for 25 mm of settlement) of 150 kPa may be used for the design of a RSS wall supported on a minimum 0.3 m. thick pad of compacted engineered fill placed on the surface of the native soils extending below about Elevation 78m.

A 'perched' RSS wall at the new edge of pavement, up to about 2 m in exposed height, may be constructed on the existing sandy embankment fill materials provided that the reinforcing extends to at least 1 metre below the surface at the face of the wall. The slope in front of the wall, extending from at least 1 metre above the founding elevation laterally away from the wall should be no steeper than 2H:1V.

An unfactored $\tan \delta^*$ value of 0.50 may be used to calculate the lateral resistance to sliding across the base of footing-soil interface for all footings on native soil at this location and also where retaining walls will be supported on engineered fill.

6.14.2.2 Steel H-Pile Foundations

Steel H-piles socketed into the shallow limestone bedrock may be used for construction of a soldier pile and concrete lagging retaining wall. The bedrock and grout have compatible strength properties and the bearing resistance of the bedrock may be taken as equal to the compressive strength of the grout.

6.14.3 Site Coefficient & Seismic Liquefaction – RW 1N

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.

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 or embankments.

6.14.4 Lateral Earth Pressures for Design – RW 1N

The following seismic active pressure coefficients (K_{AE}) for the two backfill cases (Case I and Case II) may be used in design:

SEISMIC ACTIVE PRESSURE COEFFICIENTS, K_{AE}

	Case I	Case II	
		Granular A	Granular B Type II
Yielding wall	0.39	0.30	0.30
Non-yielding wall	0.62	0.50	0.50

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 0.20. This corresponds to a displacement value of approximately 50 mm at this site.

6.14.5 Rock Anchors – RW 1N

A soldier pile and lagging wall will likely require tie-backs to resist the lateral loads from the backfill and, based on the depth to bedrock, rock anchors may be a practical option.

Rock anchors could consist of grouted anchors and the factored bond stress at the concrete/rock interface may be taken as 1,000 kilopascals for ultimate limit state (ULS) design purposes.

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.

It is recommended that pull-out tests be carried out on anchors to confirm their pull-out capacity at the time of construction. The pull-out 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.

Anchors should be installed in accordance with MTO's Special Provision No. 999S26 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 and the free stressing zone.

A Non Standard Special Provision for the installation and testing of rock anchors has been included in Appendix D of the report.

6.14.6 Retaining Wall and Embankment Stability – RW 1N

RSS walls, up to 5 m in height, founded on a minimum 0.3 m. thick pad of engineered fill placed on the existing fill materials below Elevation 78 m will have a factor of safety greater than 1.3 against deep seated slope instability.

A 'perched' RSS wall at the new edge of pavement up to about 2 m in height and with an embedment depth of at least 1 m and with a toe slope no steeper than 2H:1V will also have a factor of safety greater than 1.3 against deep-seated global instability.

Based on the borehole results, the embankment widening subgrade soils will consist of fill materials underlain by clayey silt or glacial till. The existing fill materials are generally composed of sand to sandy silt with varying amounts of clay, silt, gravel, cobbles and organic matter. The sandy fill materials at Borehole 06-110 are underlain by a layer of silty clay fill material containing sand, gravel and organic matter which is less than about 1 m in thickness. The fill materials may be left in place after stripping the surficial topsoil.

Slope stability analyses indicate that 1H:1V MSE slopes would also have factors of safety of greater than 1.3 if founded on a minimum 0.3 m thick pad of engineered fill extending down from about Elevation 78 m.

Pseudo-static seismic slope stability analyses for the above configurations also indicate that the embankment side slopes will have factors of safety of greater than 1.1.

6.14.7 Embankment Settlement – RW 1N

Some minor settlement of the embankment subgrade can be expected due to compression of the thin layer of clayey soils (i.e., the clayey silt at Borehole 06-110). However, the resulting consolidation settlements are estimated to correspond solely to recompression of the deposits (i.e., no consolidation into the virgin compression range), and the magnitude of resulting settlement will be relatively small due to the limited thickness (i.e., less than 0.6 m.) of this material. The total estimated magnitude of the settlements resulting from the recompression will be much less than 25 mm. It is expected that most of the settlement would occur quite rapidly and would likely be completed prior to final paving of the new lanes.

6.14.8 Design and Construction Considerations – RW 1N

Excavations to subgrade depth for footing construction would extend to about Elevation 75 m (i.e. the bedrock surface) or to about Elevations 77 to 78 m for the placement of engineered fill. The excavations will extend through the existing fill materials and the native clayey silt and glacial till. The groundwater level at the site appears to be below the bedrock surface (as evidenced by the standpipe in Borehole 06-2).

For a soldier pile and lagging wall, any surficial topsoil should be stripped prior to placing the embankment fill. The existing fill materials may be left in place.

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 overburden materials are classified as Type 1 soils and the existing fill materials are classified as Type 3 soil, according to the OHSA. Temporary excavations (i.e. those which are only open for a relatively short period) through these overburden soils above the water table should be made with side slopes no steeper than 1 horizontal to 1 vertical (1H:1V).

Excavations to subgrade depth in the sandy fill 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.

6.14.9 Temporary Roadway Protection – RW 1N

Temporary roadway protection may be required along Highway 417 to permit construction of the retaining wall if rock anchors are not used to limit the footing widths or for construction of a

reinforced earth slope or wall. Soldier pile and lagging would likely be the preferred shoring type. Soldier piles may be cantilevered by socketing and grouting into the bedrock or lateral support could alternatively be provided by internal struts, rakers braced to footings within the excavation, or tie-backs grouted into the bedrock behind the shoring.

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

The invert of the storm sewer extending along the wall length is indicated to be at about elevation 76.5 m, or just above the bedrock surface, and is about 3 to 4 m from the retaining wall. Minimal temporary protection may be required to maintain adequate support for the sewer. Low height lagging (i.e., less than 1 m) and internal bracing would likely be sufficient.

6.15 Retaining Wall 2N (WBL 22+770 to 22+975)

6.15.1 Retaining Wall / Embankment Options – RW 2N

The existing embankments along the alignment of retaining wall 2N are about 4 m in height and sloped at approximately 2H:1V. Feasible retaining wall alternatives at this location include:

1. Reinforced concrete retaining wall at the edge of the highway pavement.
2. Soldier pile and lagging wall at the edge of the highway.
3. Reinforced soil system (RSS) wall;
4. A 1H:1V MSE slope.

It is understood that 'perched' retaining wall is under consideration at this location. A buried layer of topsoil is indicated to exist along this wall and it is not considered feasible to support a 'perched' RSS wall on the fill materials overlying this low strength soil layer.

A 1200 mm diameter storm sewer extends along the alignment at this location and this sewer is about 2 to 3 m. inside the toe of the existing side slope. It is understood that replacement of this sewer with a larger diameter line is under consideration but at the time of this report the timing of the replacement was not known.

A retaining wall, of either reinforced concrete or soldier pile and lagging construction, could be constructed between the edge of the widened highway and the storm sewer allowing for future access to this service without shoring. A RSS wall or MSE slope would be constructed almost directly over the existing storm sewer and access to this service for future maintenance or replacement would be severely limited since the reinforced soil structures would be significantly undermined by any excavations to expose the sewer; shoring of the highway and reconstruction of the wall or slope would likely be required.

It is understood that the storm sewer may be moved and the trench abandoned. If the storm sewer trench is abandoned by backfilling with compacted engineered fill all three suitable options may be acceptably founded on the trench backfill.

The preferred option for this location will therefore depend on if the storm sewer will be moved or the desired level of future access if it is left in place.

6.15.2 Retaining Wall Foundations – RW 2N

6.15.2.1 Shallow Foundations

The existing fill materials and buried topsoil layers are not considered to be suitable for support of shallow foundations for a reinforced concrete retaining wall.

It is also not considered practical to support a reinforced concrete retaining wall on spread footings on or within the discontinuous native sand, weathered silty clay or glacial till layers. These deposits may range in thickness from about 1.2 m to as little as 0.1 m and the much higher resistance available from the underlying bedrock make that strata much preferred for supporting the foundation loads.

It is therefore considered that the most feasible and cost-effective foundations for a reinforced concrete retaining wall is spread footings founded on the shallow bedrock.

The borehole information indicates bedrock surface or refusal levels along the wall alignment as follows:

Station	Borehole Number	Bedrock Surface Elevation (m)	Refusal Elevation (m)
22+769	06-2	75.3	-
22+834	06-115	-	75.8
22+904	06-116	-	75.9
22+970	06-117	-	76.1

The above bedrock surface and refusal elevations are provided based on the borehole data available. It must be confirmed during construction by the Quality Verification Engineer that the bedrock surface elevations are consistent with those anticipated. Provision should be made in the Contract documents for additional excavation and placement of mass concrete wherever the competent bedrock surface is below the design founding level.

A retaining wall can be supported on spread footings placed on the limestone bedrock. The upper portion of the bedrock may be, in local areas, of poor quality. MTO's Special Provision

SP902S01 should be included in the Contract Documents requiring inspection and approval of the foundation area by the Quality Verification Engineer prior to footing construction.

Spread footings placed on the surface of properly prepared limestone bedrock or on mass concrete may be designed using a factored geotechnical resistance at Ultimate Limit States (ULS) of 2,000 kPa. Serviceability Limit States (SLS) resistances do not apply to the design of footings on the limestone bedrock since the SLS resistance for 25 mm of settlement is greater than the factored axial geotechnical resistance at ULS.

Resistance to lateral forces (i.e., sliding resistance between the concrete footings and bedrock) should be calculated in accordance with Section 6.7.5 of the *CHBDC*. The coefficient of friction, $\tan \delta^*$, may be taken as 0.70 for cast-in-place concrete footings constructed on bedrock. This represents an unfactored value; in accordance with the *CHBDC*, a resistance factor of 0.8 is to be applied in calculating the horizontal resistance. If necessary, the sliding resistance of spread footings founded on the bedrock can be supplemented by dowelling into the bedrock. The horizontal resistance of the dowels is dependent on the strength of the bedrock, grout and steel. For this site, where the rock mass is essentially as strong as or is stronger than concrete, the design of the dowels in the rock may be handled in the same way as the dowel embedment into the concrete. This assumes that the unconfined compressive strength of the grout will be similar to that of the concrete. The dowels should have a minimum embedded length within the bedrock of 1 m and the structural strength of the dowel and compressive strength of the grout should not be exceeded. 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. A non-standard Special Provision for the installation, materials and testing of the dowels has been included in Appendix D of this report.

In order to limit the excavation width and the requirements for temporary protection of the highway the retaining wall footing may be significantly reduced in width and it may be necessary to install rock anchors to provide overturning resistance.

Rock anchors could consist of grouted anchors and the factored bond stress at the concrete/rock interface may be taken as 1,000 kilopascals for ultimate limit state (ULS) design purposes.

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.

It is suggested that pull-out tests be carried out on anchors to confirm their pull-out capacity at the time of construction. The pull-out 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.

Anchors should be installed in accordance with MTO's Special Provision No. 999S26 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 and the free stressing zone.

A Non Standard Special Provision for the installation and testing of rock anchors has been included in Appendix D of the report.

A factored geotechnical resistance at ULS of 250 kPa and a geotechnical resistance at SLS (for 25 mm of settlement) of 150 kPa may be used for the design of a RSS wall supported on a minimum 0.3 m. thick pad of compacted engineered fill placed on the surface of the native sand, sandy silt or till.

An unfactored $\tan \delta^*$ value of 0.50 may be used to calculate the lateral resistance to sliding across the base of footing-soil interface for all footings on native soil at this location and also where retaining walls will be supported on engineered fill.

6.15.2.2 Steel H-Pile Foundations

Steel H-piles socketed into the shallow limestone bedrock may be used for construction of a soldier pile and concrete lagging retaining wall. The bedrock and grout have compatible strength properties and the bearing resistance of the bedrock may be taken as equal to the compressive strength of the grout.

6.15.3 Site Coefficient & Seismic Liquefaction – RW 2N

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.

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 or embankments.

6.15.4 Lateral Earth Pressures for Design – RW 2N

The following seismic active pressure coefficients (K_{AE}) for the two backfill cases (Case I and Case II) may be used in design:

SEISMIC ACTIVE PRESSURE COEFFICIENTS, K_{AE}

	Case I	Case II	
		Granular A	Granular B Type II
Yielding wall	0.39	0.30	0.30
Non-yielding wall	0.62	0.50	0.50

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 0.20. This corresponds to a displacement value of approximately 50 mm at this site.

6.15.5 Rock Anchors – RW 2N

A soldier pile and lagging wall will likely require tie-backs to resist the lateral loads from the backfill and, based on the depth to bedrock, rock anchors may be a practical option.

Rock anchors could consist of grouted anchors and the factored bond stress at the concrete/rock interface may be taken as 1,000 kilopascals for ultimate limit state (ULS) design purposes.

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.

It is recommended that pull-out tests be carried out on anchors to confirm their pull-out capacity at the time of construction. The pull-out 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.

Anchors should be installed in accordance with MTO's Special Provision No. 999S26 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 and the free stressing zone.

A Non Standard Special Provision for the installation and testing of rock anchors has been included in Appendix D of the report.

6.15.6 Retaining Wall and Embankment Stability – RW 2N

RSS walls, up to 2.5 m in height, founded on a minimum 0.3 m thick pad of engineered fill placed on the native soils will have factors of safety greater than 1.3 against deep seated slope instability.

The existing fill materials and buried topsoil are not suitable for support of a MSE embankment and should be removed from within the widening footprint.

Slope stability analyses indicate that 1H:1V MSE slopes would also have factors of safety of greater than 1.3 if founded on a minimum 0.3 m thick pad of engineered fill placed on the surface of the native soils.

Pseudo-static seismic slope stability analyses for the above configurations also indicate that the embankment side slopes will have factors of safety of greater than 1.1.

6.15.7 Embankment Settlement – RW 2N

Some settlement of the embankment subgrade can be expected due to compression of the clayey soils (i.e., the weathered silty clay at Boreholes 06-2 and 06-115). However, the resulting consolidation settlements are estimated to correspond solely to recompression of the weathered deposits (i.e. no consolidation into the virgin compression range), and the magnitude of resulting settlement will be relatively small due to the limited thickness (i.e., less than 0.5 m.) of this material. The total estimated magnitude of the settlements resulting from the recompression will be less than 25 mm. It is expected that most of the settlement would occur quite rapidly and would likely be completed prior to final paving of the new lanes.

6.15.8 Design and Construction Considerations – RW 2N

Excavations to subgrade depth for footing construction would extend to about Elevations 74 to 76 m (i.e. the bedrock surface) or slightly higher for the placement of engineered fill. The excavations will typically extend through the existing fill materials and buried topsoil and the native sand, silty clay and glacial till. The groundwater level at the site appears to be at or below Elevation 75 m.

For a soldier pile and lagging wall, any surficial topsoil should be stripped prior to placing the embankment fill. The existing fill materials and buried topsoil may be left in place.

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 overburden materials are classified as Type 1 soils and the existing fill materials and topsoil are classified as Type 3 soil, according to the OHSA. Temporary excavations (i.e. those which are only open for a relatively short period) through these overburden soils above the water table should be made with side slopes no steeper than 1 horizontal to 1 vertical (1H:1V).

Excavations to subgrade depth in the sandy fill 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.

6.15.9 Temporary Roadway Protection – RW 2N

Temporary roadway protection may be required along Highway 417 to permit construction of the retaining wall if rock anchors are not used to limit the footing widths or for construction of a reinforced earth slope or wall. Soldier pile and lagging would likely be the preferred shoring type. Soldier piles may be cantilevered by socketing and grouting into the bedrock or lateral support could alternatively be provided by internal struts, rakers braced to footings within the excavation, or tie-backs grouted into the bedrock behind the shoring.

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

The invert of the storm sewer extending along the wall length is indicated to be at about Elevation 75.3 m, at or below the bedrock surface, and would be about 3 m from the proposed retaining wall. Temporary protection would likely not be required to maintain support for the sewer.

6.16 Retaining Wall 3N (WBL 23+215 to 23+460)

6.16.1 Retaining Wall / Embankment Options – RW 3N

The existing embankments along the alignment of retaining wall 3N range from about 2 to 5 m. in height and are sloped at approximately 2H:1V or shallower. Feasible retaining wall alternatives at this location include:

- Reinforced concrete retaining wall at the edge of the highway;
- Soldier pile and concrete lagging wall at the edge of the highway;

- Reinforced soil system (RSS) wall;
- A 'perched' RSS wall founded within the existing embankment fills approximately half way up the existing embankment side slope; and
- A 1H:1V MSE slope.

A 1,200 mm diameter storm sewer extends along the alignment at this location and this sewer is about 2 m inside the toe of the existing side slope. It is understood that replacement of this sewer with a larger diameter line is under consideration but at the time of this report the timing of the replacement was not known.

A retaining wall, of either reinforced concrete or soldier pile and lagging construction, could be constructed between the edge of the widened highway and the storm sewer allowing for future access to this service without shoring. A RSS wall or MSE slope would be constructed almost directly over the existing storm sewer and access to this service for future maintenance or replacement would be severely limited since the reinforced soil structures would be significantly undermined by any excavations to expose the sewer; shoring of the highway and reconstruction of the wall or slope would likely be required.

It is understood that the storm sewer may be moved and the trench abandoned. If the storm sewer trench is abandoned by backfilling with compacted engineered fill all four options may be acceptably founded on the trench backfill.

The preferred option for this location will therefore depend on if the storm sewer will be moved or the desired level of future access if it is left in place.

6.16.2 Retaining Wall Foundations – RW 3N

6.16.2.1 Shallow Foundations

The existing fill materials, and the buried peat at Borehole 06-7, are not considered to be suitable for support of shallow foundations. The retaining wall can be founded on spread footings supported on the very stiff weathered silty clay and/or the compact glacial till. These deposits may be very thin over the western portion of the alignment and the wall may also be supported on footings placed on the bedrock surface.

The borehole information indicates bedrock surface levels along the wall alignment as follows:

Station	Borehole Number	Bedrock Surface Elevation (m)	Refusal Elevation (m)
23+212	06-123	-	76.4
23+280	06-124	-	75.0
23+347	06-125	-	73.2
23+403	06-126	-	76.6
23+461	06-7	73.1	-

The above bedrock surface and refusal elevations are provided based on the borehole data available. It must be confirmed during construction by the Quality Verification Engineer that the bedrock surface elevations are consistent with those anticipated. Provision should be made in the Contract documents for additional excavation and placement of mass concrete wherever the competent bedrock surface is below the design founding level. A Non-Standard Special Provision for placement of mass concrete has been included in Appendix D of this report.

A retaining wall can be supported on spread footings placed on the limestone bedrock. The upper portion of the bedrock may be, in local areas, of poor quality. MTO's Special Provision SP902S01 should be included in the Contract Documents requiring inspection and approval of the foundation area by the Quality Verification Engineer prior to footing construction.

Spread footings placed on the surface of limestone bedrock or on mass concrete may be designed using a factored geotechnical resistance at Ultimate Limit States (ULS) of 2,000 kPa. Serviceability Limit States (SLS) resistances do not apply to the design of footings on the limestone bedrock since the SLS resistance for 25 mm of settlement is greater than the factored axial geotechnical resistance at ULS.

Resistance to lateral forces (i.e., sliding resistance between the concrete footings and bedrock) should be calculated in accordance with Section 6.7.5 of the *CHBDC*. The coefficient of friction, $\tan \delta^*$, may be taken as 0.70 for cast-in-place concrete footings constructed on bedrock. This represents an unfactored value; in accordance with the *CHBDC*, a resistance factor of 0.8 is to be applied in calculating the horizontal resistance. If necessary, the sliding resistance of spread footings founded on the bedrock can be supplemented by dowelling into the bedrock. The horizontal resistance of the dowels is dependent on the strength of the bedrock, grout and steel. For this site, where the rock mass is essentially as strong as or is stronger than concrete, the design of the dowels in the rock may be handled in the same way as the dowel embedment into the concrete. This assumes that the unconfined compressive strength of the grout will be similar to that of the concrete. The dowels should have a minimum embedded length within the bedrock of 1 m and the structural strength of the dowel and compressive strength of the grout should not be exceeded. 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. A non-standard Special Provision for the installation, materials and testing of the dowels has been included in Appendix D of this report.

Rock anchors could consist of grouted anchors and the factored bond stress at the concrete/rock interface may be taken as 1,000 kilopascals for ultimate limit state (ULS) design purposes.

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.

It is suggested that pull-out tests be carried out on anchors to confirm their pull-out capacity at the time of construction. The pull-out 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.

Anchors should be installed in accordance with MTO's Special Provision No. 999S26 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 and the free stressing zone.

A Non Standard Special Provision for the installation and testing of rock anchors has been included in Appendix D of the report.

A factored geotechnical resistance at ULS of 250 kPa and a geotechnical resistance at SLS (for 25 mm of settlement) of 150 kPa may be used for the design of a RSS wall supported on a minimum 0.3 m thick pad of compacted engineered fill placed on the surface of the native weathered silty clay to clayey silt or till.

A 'perched' RSS wall at the new edge of pavement, up to about 2.8 m in exposed height, may be constructed on the existing sandy embankment fill materials provided that the reinforcing extends to at least 1.3 metres below the surface at the face of the wall. The slope in front of the wall, extending from at least 1.3 metres above the founding elevation laterally away from the wall should be no steeper than 2H:1V.

An unfactored $\tan \delta^*$ value of 0.50 may be used to calculate the lateral resistance to sliding across the footing-soil interface for all footings on native soil at this location and also where retaining walls will be supported on engineered fill. Where footings are supported on the weathered silty clay to clayey silt the short term resistance should also be checked using a C_u value of 50 kPa.

6.16.2.2 Steel H-Pile Foundations

Steel H-piles socketed into the shallow limestone bedrock may be used for construction of a soldier pile and concrete lagging retaining wall.

6.16.3 Site Coefficient & Seismic Liquefaction – RW 3N

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.

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 or embankments.

6.16.4 Lateral Earth Pressures for Design – RW 3N

The following seismic active pressure coefficients (K_{AE}) for the two backfill cases (Case I and Case II) may be used in design:

SEISMIC ACTIVE PRESSURE COEFFICIENTS, K_{AE}

	Case I	Case II	
		Granular A	Granular B Type II
Yielding wall	0.39	0.30	0.30
Non-yielding wall	0.62	0.50	0.50

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 0.20. This corresponds to a displacement value of approximately 50 mm at this site.

6.16.5 Rock Anchors – RW 3N

A soldier pile and lagging wall will likely require tie-backs to resist the lateral loads from the backfill and, based on the depth to bedrock, rock anchors may be a practical option.

Rock anchors could consist of grouted anchors and the factored bond stress at the concrete/rock interface may be taken as 1,000 kilopascals for ultimate limit state (ULS) design purposes.

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.

It is recommended that pull-out tests be carried out on anchors to confirm their pull-out capacity at the time of construction. The pull-out 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.

Anchors should be installed in accordance with MTO's Special Provision No. 999S26 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 and the free stressing zone.

A Non Standard Special Provision for the installation and testing of rock anchors has been included in Appendix D of the report.

6.16.6 Retaining Wall and Embankment Stability – RW 3N

The buried peat and surficial topsoil should be removed from within the widening footprint.

RSS walls, up to 4.0 m in height, founded on a minimum 0.3 m thick pad of engineered fill placed on the existing fill materials below about elevation 76 m will have a factor of safety greater than 1.3 against deep seated slope instability.

A 'perched' RSS wall at the new edge of pavement up to about 2.8 m in height and with an embedment depth of at least 1.3 m and with a toe slope no steeper than 2H:1V will also have a factor of safety greater than 1.3 against deep-seated global instability.

Slope stability analyses indicate that 1H:1V MSE slopes would also have factors of safety of greater than 1.3 if founded on a minimum 0.3 m thick pad of engineered fill placed on the surface of the native soils.

Pseudo-static seismic slope stability analyses for the above configurations also indicate that the embankment side slopes will have factors of safety of greater than 1.1.

6.16.7 Embankment Settlement – RW 3N

Some settlement of the embankment subgrade can be expected due to compression of the native soils. The total estimated magnitude of the settlements resulting from the recompression and primary consolidation will likely be much less than 25 mm. It is expected that most of the settlement would occur quite rapidly and would likely be completed during construction.

6.16.8 Design and Construction Considerations – RW 3N

Excavations to subgrade depth for placement of engineered fill or footing construction would extend to about Elevations 73 m or higher. The excavations will typically extend through the existing fill materials and the native clayey silt and silty clay and glacial till. The groundwater level at the site appears to be near the bedrock surface (as evidenced by the standpipe in Borehole 06-5).

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 overburden materials are classified as Type 1 soils and the existing fill materials are classified as Type 3 soil, according to the OHSA. Temporary excavations (i.e. those which are only open for a relatively short period) through these overburden soils above the water table should be made with side slopes no steeper than 1 horizontal to 1 vertical (1H:1V).

Excavations to subgrade depth in the sandy fill 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 base of the excavations.

6.16.9 Temporary Roadway Protection – RW 3N

Temporary roadway protection may be required along Highway 417 to permit construction of the retaining wall if rock anchors are not used to limit the footing widths or for construction of a reinforced earth slope or wall. Soldier pile and lagging would likely be the preferred shoring type. Soldier piles may be cantilevered by socketing and grouting into the bedrock or lateral support could alternatively be provided by internal struts, rakers braced to footings within the excavation, or tie-backs grouted into the bedrock behind the shoring.

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

The invert of the storm sewer extending along the wall length is indicate to be at about elevation 74 m, or just above the bedrock surface, and is about 3 to 4 m from the retaining wall. Minimal

temporary protection may be required to maintain adequate support for the sewer. Low height lagging (i.e., less than 1 m) and internal bracing would likely be sufficient.

6.17 Retaining Wall 4N (WBL 23+480 to 23+565)

6.17.1 Retaining Wall / Embankment Options – RW 4N

The existing embankments along the alignment of retaining wall 4N are about 5 to 6 m in height and sloped at approximately 3H:1V. Feasible retaining wall alternatives for the proposed widening at this location include:

- Reinforced soil system (RSS) wall;
- A 'perched' RSS wall founded within the existing embankment fills approximately half way up the existing embankment side slope; and
- 1H:1V mechanically stabilized earth (MSE) slope; and
- 2H:1V slope with a reinforced concrete toe wall up to 2.5 m in height.

6.17.2 Retaining Wall Foundations – RW 4N

Shallow foundations on the native weathered silty clay or glacial till are recommended for the support of reinforced concrete or RSS toe walls. The existing fill materials and peat are not suitable for support of retaining wall footings and need to be removed from the foundation areas. For footings on native soil, the borehole information indicates maximum founding levels for the retaining walls as follows:

Borehole No.	Station	Maximum Founding Elevation (m)	Founding Material
06-5	23+482	73.9	Weathered Silty Clay
06-130	23+518	74.1	Till

The above design founding elevations are provided based on the borehole data available. It must be confirmed during construction that the native soil elevations are consistent with those anticipated. Provision should be made in the Contract documents for additional excavation and placement of engineered fill should the existing fill materials extend deeper than anticipated.

Higher founding levels can also be used provided the grade is raised from the native subgrade up to the founding level using compacted engineered fill.

A factored geotechnical resistance at ULS of 250 kPa and a geotechnical resistance at SLS (for 25 mm of settlement) of 150 kPa may be used for the design of footings placed on the surface of the weathered silty clay or the glacial till. Both the weathered silty clay and the glacial till are not

considered to be compressible soils and any differential settlement should be less than 15 mm. For foundations at a higher elevation on a pad of compacted engineered fill up to 1 m in thickness the factored geotechnical resistance can be increased assuming a 1H:2V load dispersal.

A RSS wall, designed with the geotechnical resistances given above for cantilever retaining walls, may be founded on a minimum 0.3 m thick compacted levelling pad constructed on the surface of the native soils.

A 'perched' RSS wall at the new edge of pavement, up to about 3 m in exposed height, may be constructed on the existing sandy embankment fill materials provided that the reinforcing extends to at least 1 metres below the surface at the face of the wall. The slope in front of the wall, extending from at least 1 metres above the founding elevation laterally away from the wall should be no steeper than 2H:1V.

The parameter values in the following table may be used to calculate the lateral resistance to sliding across the footing-soil interface as noted in Section 6.3.1:

<i>Borehole Number</i>	<i>Station</i>	<i>Founding Elevation (m)</i>	<i>Founding Material</i>	<i>Drained Conditions (Long term Loading)</i>		<i>Undrained Conditions (Short Term Loading)</i>
				<i>tan δ^*</i>	<i>c' (kPa)</i>	<i>Su (kPa)</i>
06-5	23+482	73.9	Weathered Silty Clay	0.43	0	50
06-130	23+518	74.1	Till	0.50	0	

Note: The $\tan \delta^*$ values are based on 2/3 of the soil friction angle.

At Boreholes 06-5 where the footings may be placed on the weathered silty clay, the resistance to both long term and short term loading needs to be evaluated. Resistance values for both conditions are provided in the above table.

6.17.3 Site Coefficient & Seismic Liquefaction – RW 4N

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.

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 or embankments.

6.17.4 Lateral Earth Pressures for Design – RW 4N

The following seismic active pressure coefficients (K_{AE}) for the two backfill cases (Case I and Case II) may be used in design:

SEISMIC ACTIVE PRESSURE COEFFICIENTS, K_{AE}

	Case I	Case II	
		Granular A	Granular B Type II
Yielding wall	0.39	0.30	0.30
Non-yielding wall	0.62	0.50	0.50

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 0.20. This corresponds to a displacement value of approximately 50 mm at this site.

6.17.5 Retaining Wall and Embankment Stability – RW 4N

Retaining walls, up to 3.6 m in height, founded on the native weathered silty clay or glacial till after removal of the surficial random fill materials and peat will have a factor of safety of greater than 1.3 against deepseated global instability.

A 'perched' RSS wall at the new edge of pavement up to about 3 m in exposed height and with an embedment depth of at least 1 m and with a toe slope no steeper than 2H:1V will also have a factor of safety greater than 1.3 against deep-seated global instability.

Embankment widening at this location may be accomplished with a 1H:1V MSE slope in place of retaining or toe walls. Based on the borehole results, the embankment widening subgrade soils will consist of weathered silty clay or glacial till. The existing fill materials are not considered to be suitable for support of the embankment widening and should be removed from within the MSE slope footprint within 2/3 of the wall height from the proposed toe of slope

With appropriate subgrade preparation and proper placement of earth or granular soils, the 5 to 6 m high MSE embankment with 1H:1V side slopes founded on the native soils, will have a factor of safety greater than 1.3 against deep seated slope instability.

Pseudo-static seismic slope stability analyses for the above configurations also indicate that the retaining walls and embankment side slopes will have factors of safety of greater than 1.1.

6.17.6 Embankment Settlement – RW 4N

Some settlement of the embankment subgrade can be expected due to compression of the clayey soils (i.e., the weathered clay crust). However, the resulting consolidation settlements are estimated to correspond solely to recompression of the clayey deposits (i.e., no consolidation into the virgin compression range). The total estimated magnitude of the primary consolidation settlements resulting from this recompression will be less than 25 mm. It is expected that most of the settlement would occur quite rapidly during construction.

6.17.7 Design and Construction Considerations – RW 4N

Excavations to subgrade depth for placement of engineered fill or footing construction would extend to less than about 1 m depth below the existing ground surface. The excavations will typically extend through topsoil and fill materials overlying very stiff weathered silty clay or glacial till. The groundwater level at the site is typically at about Elevation 73 m.

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 overburden materials are classified as Type 1 soils and the existing fill materials are classified as Type 3 soil, according to the OHSA. Temporary excavations (i.e. those which are only open for a relatively short period) through these overburden soils above the water table should be made with side slopes no steeper than 1 horizontal to 1 vertical (1H:1V).

The groundwater level at the site is typically below the anticipated founding depth. Excavations to subgrade depth in the sandy fill 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 base of the excavations.

Temporary roadway protection may be required along Highway 417 to permit construction of the retaining wall. It is understood that the design of the shoring will be entirely 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.

Soldier pile and lagging would likely be the preferred shoring type. Soldier piles may be cantilevered by socketing and grouting into the bedrock or lateral support could alternatively be provided by internal struts, rakers braced to footings within the excavation, or tie-backs grouted into the bedrock behind the shoring.

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

The temporary excavation support should be in accordance with MTO Special Provision 902S01 and should be designed to Performance Level 2 as defined in SP 902S01.

6.17.8 Temporary Roadway Protection – RW 4N

Temporary roadway protection may be required along Highway 417 to permit construction of the retaining wall if rock anchors are not used to limit the footing widths or for construction of a reinforced earth slope or wall. Soldier pile and lagging would likely be the preferred shoring type. Soldier piles may be cantilevered by socketing and grouting into the bedrock or lateral support could alternatively be provided by internal struts, rakers braced to footings within the excavation, or tie-backs grouted into the bedrock behind the shoring.

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

6.18 Retaining Wall 5N (WBL 23+686 to 23+837)

6.18.1 Retaining Wall / Embankment Options – RW 5N

The existing embankments along the alignment of retaining wall 5N are about 5 to 6 m in height and sloped at approximately 1H:1V to 3H:1V at the western limit and are on average sloped at 2H:1V. Feasible retaining wall alternatives for the proposed widening at this location include:

- Reinforced soil system (RSS) wall;
- 1H:1V mechanically stabilized earth (MSE) slope; and
- 2H:1V slope with a reinforced concrete toe wall up to 3.5 m. in height.

6.18.2 Retaining Wall Foundations – RW 5N

Shallow foundations on the native weathered silty clay or glacial till are recommended for the support of cantilevered reinforced concrete toe walls. The existing fill materials are not suitable for support of retaining wall footings and need to be removed from the foundation areas. For footings on native soil, the borehole information indicates maximum founding levels for the retaining walls as follows:

Borehole No.	Station	Maximum Founding Elevation (m)	Founding Material
06-132	23+760	74.1	Till
06-12	23+838	73.7	Weathered Silty Clay

The above design founding elevations are provided based on the borehole data available. It must be confirmed during construction that the native soil elevations are consistent with those anticipated. Provision should be made in the Contract documents for additional excavation and placement of engineered fill should the existing fill materials extend deeper than anticipated.

Higher founding levels can also be used provided the grade is raised from the native subgrade up to the founding level using compacted engineered fill.

A factored geotechnical resistance at ULS of 250 kPa and a geotechnical resistance at SLS (for 25 mm of settlement) of 150 kPa may be used for the design of footings placed on the surface of the weathered silty clay or the glacial till. Both the weathered silty clay and the glacial till are not considered to be compressible soils and any differential settlement should be less than 15 mm.

A full height RSS wall, designed with the geotechnical resistances given above for cantilever toe walls, may be founded on a minimum 0.3 m thick compacted levelling pad constructed on the surface of the native soils.

The unfactored parameter values in the following table may be used to calculate the lateral resistance to sliding across the footing-soil interface as noted in Section 6.3.1:

<i>Borehole Number</i>	<i>Station</i>	<i>Founding Elevation (m)</i>	<i>Founding Material</i>	<i>Drained Conditions (Long term Loading)</i>		<i>Undrained Conditions (Short Term Loading)</i>
				<i>tan δ^*</i>	<i>c' (kPa)</i>	<i>Su (kPa)</i>
06-132	23+760	74.1	Till	0.50	0	-
06-12	23+838	73.7	Weathered Silty Clay	0.43	0	50

Note: The $\tan \delta^*$ values are based on 2/3 of the soil friction angle.

At Borehole 06-12, near the eastern end of the alignment, where the footings may be placed on the weathered silty clay, the resistance to both long term and short term loading needs to be evaluated. Resistance values for both conditions are provided in the above table.

6.18.3 Site Coefficient & Seismic Liquefaction – RW 5N

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.

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 or embankments.

6.18.4 Lateral Earth Pressures for Design – RW 5N

The following seismic active pressure coefficients (K_{AE}) for the two backfill cases (Case I and Case II) may be used in design:

SEISMIC ACTIVE PRESSURE COEFFICIENTS, K_{AE}

	Case I	Case II	
		Granular A	Granular B Type II
Yielding wall	0.39	0.30	0.30
Non-yielding wall	0.62	0.50	0.50

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 0.20. This corresponds to a displacement value of approximately 50 mm at this site.

6.18.5 Retaining Wall and Embankment Stability – RW 5N

Reinforced concrete toe walls, up to 3.5 m in height, founded on the native weathered silty clay or glacial till after removal of the surficial random fill materials will have a factor of safety of greater than 1.3 against deep-seated global instability.

RSS walls, up to 5 to 6 m in height, founded on the native soils or engineered fill, after removal of the existing fill materials and peat, will also have a factor of safety greater than 1.3 against deep seated slope instability.

Embankment widening at this location may be accomplished with a 1H:1V MSE slope in place of retaining or toe walls.

Based on the borehole results, the embankment widening subgrade soils will consist of existing fill materials, weathered silty clay or glacial till. With appropriate subgrade preparation and proper placement of earth or granular soils, the 5 to 6 m high MSE embankment with 1H:1V side slopes founded on a minimum 0.75 m thick pad of compacted engineered fill placed within the existing fill materials (i.e. after removal of the same depth of existing fill materials), will have a factor of safety greater than 1.3 against deep seated slope instability.

Pseudo-static seismic slope stability analyses for the above configurations also indicate that the retaining walls and embankment side slopes will have factors of safety of greater than 1.1.

6.18.6 Embankment Settlement – RW 5N

Some settlement of the embankment subgrade can be expected due to compression of the clayey soils (i.e., the weathered clay crust). However, the resulting consolidation settlements are estimated to correspond solely to recompression of the clayey deposits (i.e. no consolidation into the virgin compression range). The total estimated magnitude of the primary consolidation settlements resulting from this recompression will be less than 25 mm. It is expected that most of the settlement would occur quite rapidly during construction.

6.18.7 Design and Construction Considerations – RW 5N

Excavations to subgrade depth for placement of engineered fill or footing construction would extend up to about 4 m depth (i.e., to about Elevation 74 m) below the existing ground surface. The excavations will typically extend through topsoil and existing fill materials overlying very stiff weathered silty clay or glacial till. The groundwater level at the site is typically at or below Elevation 72 m.

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 overburden materials are classified as Type 1 soils and the existing fill materials are classified as Type 3 soil, according to the OHSA. Temporary excavations (i.e. those which are only open for a relatively short period) through these overburden soils above the water table should be made with side slopes no steeper than 1 horizontal to 1 vertical (1H:1V).

The groundwater level at the site is typically below the anticipated founding depth. Excavations to subgrade depth through the sandy fill 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 base of the excavations.

6.17.8 Temporary Roadway Protection – RW 5N

Temporary roadway protection may be required along Highway 417 to permit construction of the retaining wall. Soldier pile and lagging would likely be the preferred shoring type. Soldier piles may be cantilevered by socketing and grouting into the bedrock or lateral support could alternatively be provided by internal struts, rakers braced to footings within the excavation, or tie-backs grouted into the bedrock behind the shoring.

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

6.19 Retaining Wall 6N (WBL 23+590 to 23+610)

6.19.1 Retaining Wall / Embankment Options – RW 6N

The existing embankments along the alignment of retaining wall 6N are about 2 to 3 m in height and sloped at approximately 4H:1V, or shallower.

The existing sandy fill materials extend to depths of about 3 to 4 m. at the location of the hydro tower. These loose to compact materials are not suitable for founding a reinforced concrete retaining wall. Removing the fill materials and founding the retaining wall on the native soils at these depths is likely not practical or economic.

A RSS wall would be more settlement tolerant and is the preferred option for the relatively low height retaining wall required at the hydro tower.

6.19.2 Retaining Wall Foundations – RW 6N

A factored geotechnical resistance at ULS of 200 kPa and a geotechnical resistance at SLS of 100 kPa may be used for the design of a RSS wall placed on the existing fill materials. The behavior of the existing fill materials is difficult to predict and the differential settlement of a RSS wall may exceed the 35 mm limit considered to be tolerable even when loaded to less than the SLS geotechnical resistance specified above.

A RSS wall should be founded on a minimum 0.3 metre thick pad of engineered fill consisting of OPSS 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 suggested engineered fill pad prior to construction.

For a RSS wall supported on engineered fill as noted above, an unfactored $\tan \delta^*$ lateral sliding resistance value of 0.50 may be used at the base of the footing – engineered fill interface.

6.19.3 Site Coefficient & Seismic Liquefaction – RW 6N

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.

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 or embankments.

6.19.4 Retaining Wall Stability – RW 6N

The RSS wall at the hydro tower should be founded on a minimum 1.0 m thick compacted levelling pad of Granular B Type II engineered fill, compacted to 95 percent of the Standard Proctor maximum dry density in accordance with MTO's Special Provision 105S10, constructed on the surface of the existing fill materials. With appropriate subgrade preparation and proper placement of earth or granular soils, the approximately 1.5 m high RSS wall will have a factor of safety greater than 1.3 against deep seated slope instability.

Pseudo-static seismic slope stability analyses for the above configuration also indicate that the retaining walls/embankments will have factors of safety of greater than 1.1.

6.19.5 Embankment Settlement – 6N

Some settlement of the embankment subgrade can be expected due to compression of the existing fill materials. It is difficult to predict the performance of the existing fill materials but the post-construction settlement magnitude should be less than 25 mm.


7.0 CLOSURE

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

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TABLE 1
COMPARISON OF RETAINING WALL / EMBANKMENT ALTERNATIVES
MAITLAND AVENUE TO ISLAND PARK DRIVE
HIGHWAY 417
W.P. 4058-01-00

Retaining Wall	Retaining Wall Option	Feasibility	Advantages	Disadvantages	Relative Costs	Risks/Consequences
1S (EBL 22+150 to 22+475)	Toe wall as per OPSD 3120.100	<ul style="list-style-type: none"> Feasible 	<ul style="list-style-type: none"> Simple to construct 	<ul style="list-style-type: none"> N/A 	<ul style="list-style-type: none"> Least expensive option 	<ul style="list-style-type: none"> N/A
2S & 3S (EBL 22+600 to 22+715)	2H:1V slope with toe wall	<ul style="list-style-type: none"> Feasible 	<ul style="list-style-type: none"> Landscaping can be more easily carried out and maintained. Widely used construction (Atypical methods or materials are not required) 	<ul style="list-style-type: none"> More property required 	<ul style="list-style-type: none"> Less expensive than RSS wall 	<ul style="list-style-type: none"> N/A
	1H:1V MSE slope	<ul style="list-style-type: none"> Feasible 	<ul style="list-style-type: none"> Simple to construct 	<ul style="list-style-type: none"> Can be difficult to carry out and maintain landscaping (particularly with southern exposure as at this location) 	<ul style="list-style-type: none"> Less expensive than RSS wall 	<ul style="list-style-type: none"> May be difficult to achieve and maintain vegetation
	RSS wall	<ul style="list-style-type: none"> Feasible 	<ul style="list-style-type: none"> Appearance may be considered preferable to MSE slopes or toe walls 	<ul style="list-style-type: none"> Excavation of existing embankment may be required for installation of reinforcing strips 	<ul style="list-style-type: none"> More expensive than MSE slope or toe walls 	<ul style="list-style-type: none"> Most costly
	'Perched' RSS wall	<ul style="list-style-type: none"> Not feasible due to low strength soils underlying existing fill materials 	<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
4S (EBL 22+745 to 23+020)	1H:1V MSE slope	<ul style="list-style-type: none"> Feasible 	<ul style="list-style-type: none"> Simple to construct 	<ul style="list-style-type: none"> Can be difficult to carry out and maintain landscaping (particularly with southern exposure as at this location) 	<ul style="list-style-type: none"> Less expensive than RSS wall 	<ul style="list-style-type: none"> May be difficult to achieve and maintain vegetation

Retaining Wall	Retaining Wall Option	Feasibility	Advantages	Disadvantages	Relative Costs	Risks/Consequences
5S (EBL 23+140 to 23+330)	RSS wall	<ul style="list-style-type: none"> Feasible 	<ul style="list-style-type: none"> Appearance may be considered preferable to MSE slope 	<ul style="list-style-type: none"> Excavation of existing embankment may be required for installation of reinforcing strips 	<ul style="list-style-type: none"> More expensive than MSE slope 	<ul style="list-style-type: none"> Most costly
	'Perched' RSS wall	<ul style="list-style-type: none"> Feasible 	<ul style="list-style-type: none"> Reduced excavation required compared to full height walls. 	<ul style="list-style-type: none"> Settlement of existing fill materials difficult to estimate. 	<ul style="list-style-type: none"> Less costly than full height RSS wall 	<ul style="list-style-type: none"> Magnitude of post-construction settlement may be higher than anticipated
	RSS wall	<ul style="list-style-type: none"> Feasible 	<ul style="list-style-type: none"> Appearance may be considered preferable to concrete wall 	<ul style="list-style-type: none"> Excavation of existing embankment may be required for installation of reinforcing strips Full closure of ramp may be required to excavate for installation of reinforcing strips 	<ul style="list-style-type: none"> Less expensive than concrete wall 	<ul style="list-style-type: none"> N/A
	Soldier pile and lagging wall	<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 	<ul style="list-style-type: none"> N/A
	Reinforced concrete wall	<ul style="list-style-type: none"> Feasible 	<ul style="list-style-type: none"> N/A 	<ul style="list-style-type: none"> More involved construction 	<ul style="list-style-type: none"> More expensive than RSS wall 	<ul style="list-style-type: none"> Most costly
6S (EBL 23+510 to 23+818)	2H:1V slopes with local walls around hydro towers	<ul style="list-style-type: none"> Feasible 	<ul style="list-style-type: none"> Landscaping can be readily carried out and maintained. Widely used construction (Unusual methods or materials are not required) 	<ul style="list-style-type: none"> Subexcavation of buried peat would be required over full width of widening 	<ul style="list-style-type: none"> More costly than 2.5H:1V slopes 	<ul style="list-style-type: none"> N/A
	2.5H:1V slopes with local walls around hydro towers	<ul style="list-style-type: none"> Feasible 	<ul style="list-style-type: none"> Landscaping can be readily carried out and maintained. Widely used construction (Unusual methods or materials are not required) 	<ul style="list-style-type: none"> More property required than for 2H:1V slopes Subexcavation of buried peat would not be required 	<ul style="list-style-type: none"> Less costly than 2H:1V slopes 	<ul style="list-style-type: none"> N/A

Retaining Wall	Retaining Wall Option	Feasibility	Advantages	Disadvantages	Relative Costs	Risks/Consequences
	3H:1V slopes with local walls around hydro towers	<ul style="list-style-type: none"> Feasible 	<ul style="list-style-type: none"> Landscaping can be readily carried out and maintained. Widely used construction (Unusual methods or materials are not required) 	<ul style="list-style-type: none"> More property required than for 2.5H:1V slopes Subexcavation of buried peat would not be required 	<ul style="list-style-type: none"> Less costly than 2H:1V slopes 	<ul style="list-style-type: none"> N/A
7S EBL (23+842 to 23+862)	2H:1V slopes	<ul style="list-style-type: none"> Feasible 	<ul style="list-style-type: none"> Landscaping can be readily carried out and maintained. Widely used construction (Unusual methods or materials are not required) 	<ul style="list-style-type: none"> Property requirements 	<ul style="list-style-type: none"> N/A 	<ul style="list-style-type: none"> N/A
	OPSD toe wall at hydro tower	<ul style="list-style-type: none"> Feasible 	<ul style="list-style-type: none"> Simple to construct 	<ul style="list-style-type: none"> N/A 	<ul style="list-style-type: none"> N/A 	<ul style="list-style-type: none"> N/A
8S (EBL 23+958 to 23+975)	Toe wall as per OPSD 3120.100	<ul style="list-style-type: none"> Feasible 	<ul style="list-style-type: none"> Simple to construct 	<ul style="list-style-type: none"> N/A 	<ul style="list-style-type: none"> Least expensive option 	<ul style="list-style-type: none"> N/A
9S (EBL 23+975 to 24+050)	RSS wall	<ul style="list-style-type: none"> Not feasible due to settlement 	<ul style="list-style-type: none"> Simple to construct 	<ul style="list-style-type: none"> Excavation of existing embankment may be required for installation of reinforcing strips 	<ul style="list-style-type: none"> Less expensive than concrete wall 	<ul style="list-style-type: none"> N/A
	Reinforced concrete wall	<ul style="list-style-type: none"> Not feasible due to depth of existing fills 	<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

Retaining Wall	Retaining Wall Option	Feasibility	Advantages	Disadvantages	Relative Costs	Risks/Consequences
11S (EBL 24+160 to 24+515)	Reinforced concrete wall supported on piles	<ul style="list-style-type: none"> Feasible 	<ul style="list-style-type: none"> May have a more acceptable appearance Relatively less maintenance required than for pile and lagging wall Reduced embankment settlement 	<ul style="list-style-type: none"> More involved construction 	<ul style="list-style-type: none"> More expensive than pile and lagging wall 	<ul style="list-style-type: none"> N/A
	Soldier pile and lagging 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
	Soldier pile and concrete lagging 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
	Reinforced concrete wall supported on deep foundations	<ul style="list-style-type: none"> Feasible 	<ul style="list-style-type: none"> May have a more acceptable appearance Relatively less maintenance required than for pile and lagging wall Reduced embankment settlement 	<ul style="list-style-type: none"> More involved construction 	<ul style="list-style-type: none"> More expensive than pile and lagging wall 	<ul style="list-style-type: none"> N/A
	RSS wall	<ul style="list-style-type: none"> Not feasible due to settlement 	<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
	MSE slope with local walls (RSS or soldier pile and lagging) at hydro towers	<ul style="list-style-type: none"> Not feasible due to settlement 	<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

Retaining Wall	Retaining Wall Option	Feasibility	Advantages	Disadvantages	Relative Costs	Risks/Consequences
12S (EBL 24+540 to 24+595)	Reinforced concrete wall supported on deep foundations	<ul style="list-style-type: none"> Feasible 	<ul style="list-style-type: none"> May have a more acceptable appearance Relatively less maintenance required than for pile and lagging wall Reduced embankment settlement 	<ul style="list-style-type: none"> More involved construction 	<ul style="list-style-type: none"> More expensive than pile and lagging wall N/A 	<ul style="list-style-type: none"> N/A
	1H:1V MSE slope	<ul style="list-style-type: none"> Feasible 	<ul style="list-style-type: none"> Simple to construct 	<ul style="list-style-type: none"> Differential settlement with respect to existing roadway Can be difficult to carry out and maintain landscaping (particularly with southern exposure as at this location) 	<ul style="list-style-type: none"> Less expensive than RSS wall 	<ul style="list-style-type: none"> Increased roadway maintenance due to differential settlement
	RSS wall	<ul style="list-style-type: none"> Not feasible due to settlement 	<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
1N (WBL 22+610 to 22+747)	Soldier pile and lagging wall with tie-backs to bedrock	<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
	Reinforced concrete wall supported on shallow foundations	<ul style="list-style-type: none"> Feasible 	<ul style="list-style-type: none"> May have a more acceptable appearance Relatively less maintenance required than for pile and lagging wall 	<ul style="list-style-type: none"> Rock anchors may be required to limit excavation width 	<ul style="list-style-type: none"> More expensive than pile and lagging wall 	<ul style="list-style-type: none"> N/A
	RSS wall	<ul style="list-style-type: none"> Feasible 	<ul style="list-style-type: none"> Appearance may be considered preferable to MSE slope 	<ul style="list-style-type: none"> Will restrict access to storm sewer Excavation of existing embankment may be required for installation of reinforcing strips 	<ul style="list-style-type: none"> More expensive than MSE slope 	<ul style="list-style-type: none"> Restricted access to storm sewer

Retaining Wall	Retaining Wall Option	Feasibility	Advantages	Disadvantages	Relative Costs	Risks/Consequences
2N (WBL 22+770 to 22+975)	'Perched' RSS wall	<ul style="list-style-type: none"> Feasible 	<ul style="list-style-type: none"> Reduced excavation required compared to full height walls. 	<ul style="list-style-type: none"> Settlement of existing fill materials difficult to estimate. 	<ul style="list-style-type: none"> Less costly than full height RSS wall but more expensive than MSE slope 	<ul style="list-style-type: none"> Magnitude of post-construction settlement may be higher than anticipated
	1H:1V MSE slope	<ul style="list-style-type: none"> Feasible 	<ul style="list-style-type: none"> Simple to construct 	<ul style="list-style-type: none"> Will restrict access to storm sewer Can be difficult to carry out and maintain landscaping 	<ul style="list-style-type: none"> Less expensive than RSS wall 	<ul style="list-style-type: none"> Restricted access to storm sewer
	Soldier pile and lagging wall with tie-backs to bedrock	<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
	Reinforced concrete wall supported on shallow foundations	<ul style="list-style-type: none"> Feasible 	<ul style="list-style-type: none"> May have a more acceptable appearance Relatively less maintenance required than for pile and lagging wall 	<ul style="list-style-type: none"> Rock anchors may be required to limit excavation width 	<ul style="list-style-type: none"> More expensive than pile and lagging wall 	<ul style="list-style-type: none"> N/A
	RSS wall	<ul style="list-style-type: none"> Feasible 	<ul style="list-style-type: none"> Appearance may be considered preferable to MSE slope 	<ul style="list-style-type: none"> Will restrict access to storm sewer Excavation of existing embankment may be required for installation of reinforcing strips 	<ul style="list-style-type: none"> More expensive than MSE slope 	<ul style="list-style-type: none"> Restricted access to storm sewer
	'Perched' RSS wall	<ul style="list-style-type: none"> Not feasible due to low strength soil underlying existing fill materials 	<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
	1H:1V MSE slope	<ul style="list-style-type: none"> Feasible 	<ul style="list-style-type: none"> Simple to construct 	<ul style="list-style-type: none"> Will restrict access to storm sewer Can be difficult to carry out and maintain landscaping 	<ul style="list-style-type: none"> Less expensive than RSS wall 	<ul style="list-style-type: none"> Restricted access to storm sewer

Retaining Wall	Retaining Wall Option	Feasibility	Advantages	Disadvantages	Relative Costs	Risks/Consequences
3N (WBL 23+215 to 23+460)	Soldier pile and concrete lagging 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
	Reinforced concrete wall supported on shallow foundations	<ul style="list-style-type: none"> Feasible 	<ul style="list-style-type: none"> May have a more acceptable appearance Relatively less maintenance required than for pile and lagging wall 	<ul style="list-style-type: none"> Rock anchors may be required to limit excavation width 	<ul style="list-style-type: none"> More expensive than pile and lagging wall 	<ul style="list-style-type: none"> N/A
	RSS wall	<ul style="list-style-type: none"> Feasible 	<ul style="list-style-type: none"> Appearance may be considered preferable to MSE slope 	<ul style="list-style-type: none"> Will restrict access to storm sewer Excavation of existing embankment may be required for installation of reinforcing strips 	<ul style="list-style-type: none"> More expensive than MSE slope 	<ul style="list-style-type: none"> Restricted access to storm sewer
	'Perched' RSS wall	<ul style="list-style-type: none"> Feasible 	<ul style="list-style-type: none"> Reduced excavation required compared to full height walls. 	<ul style="list-style-type: none"> Settlement of existing fill materials difficult to estimate. 	<ul style="list-style-type: none"> Less costly than full height RSS wall but more expensive than MSE slope 	<ul style="list-style-type: none"> Magnitude of post-construction settlement may be higher than anticipated
	1H:1V MSE slope	<ul style="list-style-type: none"> Feasible 	<ul style="list-style-type: none"> Simple to construct 	<ul style="list-style-type: none"> Will restrict access to storm sewer Can be difficult to carry out and maintain landscaping 	<ul style="list-style-type: none"> Less expensive than RSS wall 	<ul style="list-style-type: none"> Restricted access to storm sewer
	RSS wall	<ul style="list-style-type: none"> Feasible 	<ul style="list-style-type: none"> Appearance may be considered preferable to MSE slopes or toe walls 	<ul style="list-style-type: none"> Excavation of existing embankment may be required for installation of reinforcing strips 	<ul style="list-style-type: none"> More expensive than MSE slope or toe walls 	<ul style="list-style-type: none"> Most costly
4N (WBL 23+480 to 23+565)	'Perched' RSS wall	<ul style="list-style-type: none"> Feasible 	<ul style="list-style-type: none"> Reduced excavation required compared to full height walls 	<ul style="list-style-type: none"> Settlement of existing fill materials difficult to estimate. 	<ul style="list-style-type: none"> Less costly than full height RSS wall but more expensive than MSE slope 	<ul style="list-style-type: none"> Magnitude of post-construction settlement may be higher than anticipated

Retaining Wall	Retaining Wall Option	Feasibility	Advantages	Disadvantages	Relative Costs	Risks/Consequences
5N (WBL 23+686 to 23+837)	1H:1V MSE slope	<ul style="list-style-type: none"> Feasible 	<ul style="list-style-type: none"> Simple to construct 	<ul style="list-style-type: none"> Can be difficult to carry out and maintain landscaping 	<ul style="list-style-type: none"> Less expensive than RSS wall 	<ul style="list-style-type: none"> May be difficult to achieve and maintain vegetation
	2H:1V slope with toe wall	<ul style="list-style-type: none"> Feasible 	<ul style="list-style-type: none"> Landscaping can be more easily carried out and maintained. Widely used construction (Unusual methods or materials are not required) 	<ul style="list-style-type: none"> More property required 	<ul style="list-style-type: none"> Less expensive than RSS wall 	<ul style="list-style-type: none"> N/A
	RSS wall	<ul style="list-style-type: none"> Feasible 	<ul style="list-style-type: none"> Appearance may be considered preferable to MSE slopes or toe walls 	<ul style="list-style-type: none"> Excavation of existing embankment may be required for installation of reinforcing strips 	<ul style="list-style-type: none"> More expensive than MSE slope or toe walls 	<ul style="list-style-type: none"> Most costly
	1H:1V MSE slope	<ul style="list-style-type: none"> Feasible 	<ul style="list-style-type: none"> Simple to construct 	<ul style="list-style-type: none"> Can be difficult to carry out and maintain landscaping 	<ul style="list-style-type: none"> Less expensive than RSS wall 	<ul style="list-style-type: none"> May be difficult to achieve and maintain vegetation
	2H:1V slope with toe wall	<ul style="list-style-type: none"> Feasible 	<ul style="list-style-type: none"> Landscaping can be more easily carried out and maintained. Widely used construction (Unusual methods or materials are not required) 	<ul style="list-style-type: none"> More property required 	<ul style="list-style-type: none"> Less expensive than RSS wall 	<ul style="list-style-type: none"> N/A
	Reinforced concrete wall	<ul style="list-style-type: none"> Not feasible due to depth of existing fills 	<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
6N (WBL 23+590 to 23+610)	RSS wall	<ul style="list-style-type: none"> Feasible 	<ul style="list-style-type: none"> Simple to construct 	<ul style="list-style-type: none"> Excavation of existing embankment may be required for installation of reinforcing strips 	<ul style="list-style-type: none"> Less expensive than concrete wall 	<ul style="list-style-type: none"> N/A

APPENDIX A
DRAWINGS 1 TO 16

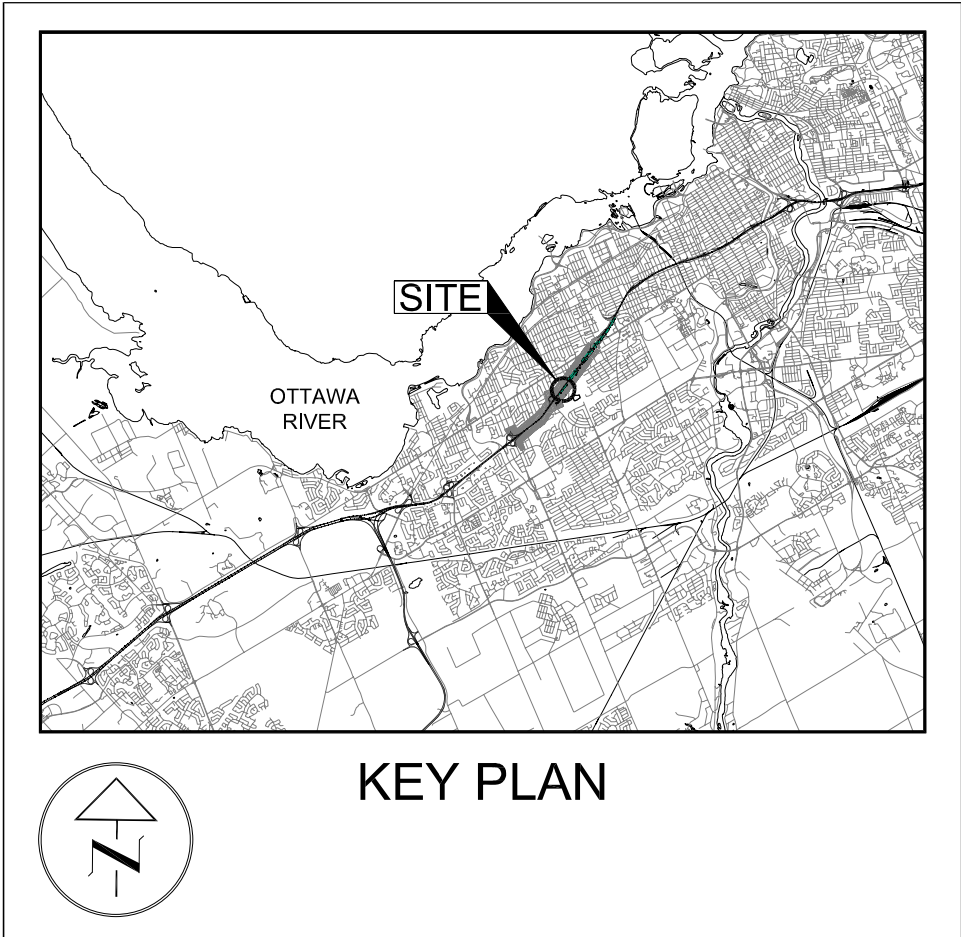
HWY. 417

WP No. WP 4058-01-00

HIGHWAY 417
RETAINING WALL 1S
PLAN AND PROFILE

SHEET

Golder Associates Ltd.
OTTAWA, ONTARIO, CANADA



LEGEND

Borehole — Current Golder Associates Ltd. Investigation

Seal

Piezometer

NStandard Penetration Test value

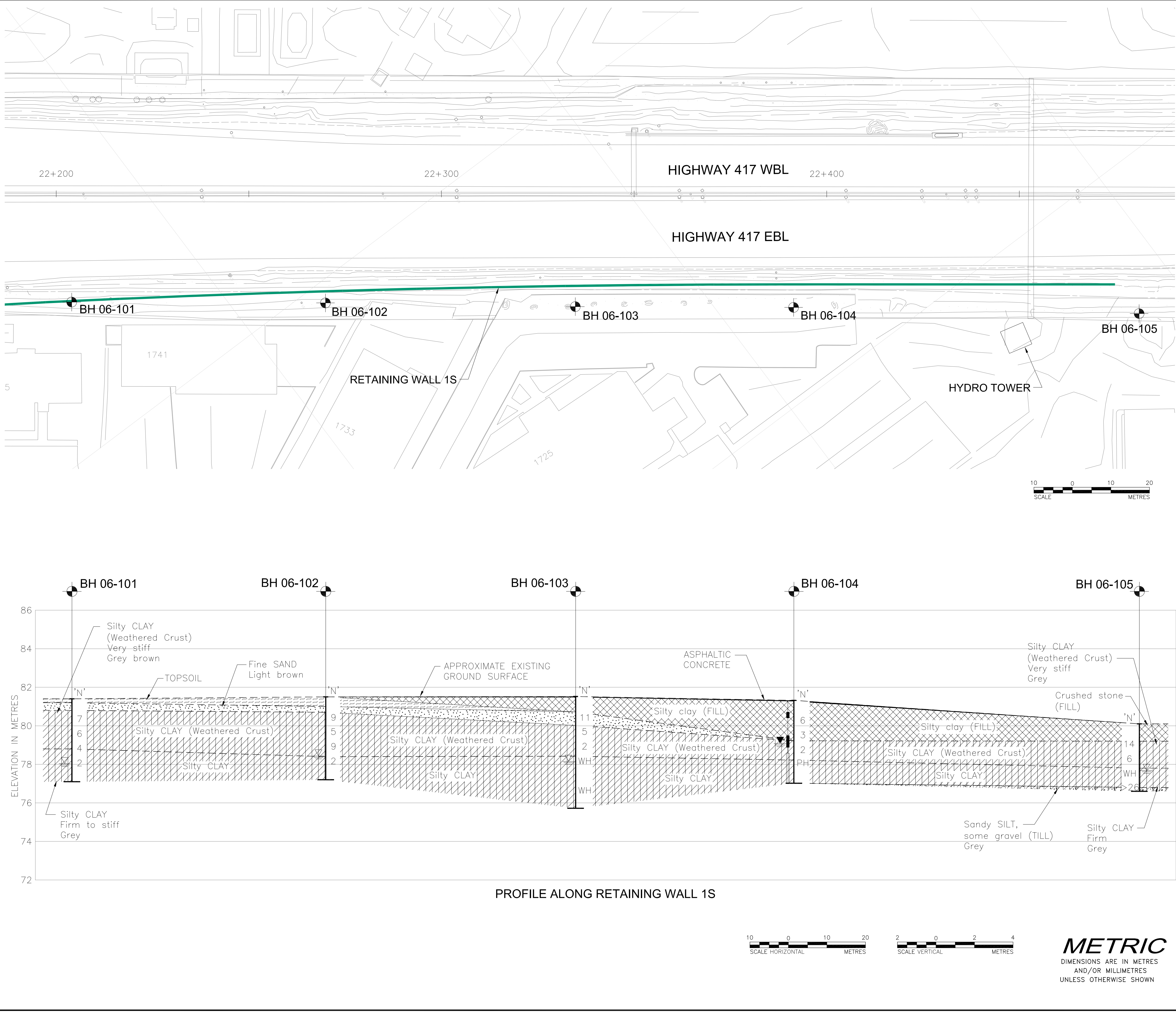
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100%Rock Quality Designation (RQD)

WL in piezometer

NOTES

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NO.	DATE	BY	REVISION
Geocres No. 31G5-218			
HWY. 417	PROJECT NO. 05-1120-210-2000		DIST.
SUBM'D. W.C.	CHKD. M.I.C.	DATE: OCTOBER 2006	SITE:
DRAWN: J.M.	CHKD. W.C.	APPD.	DWG. 1

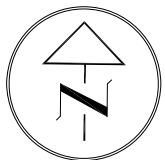
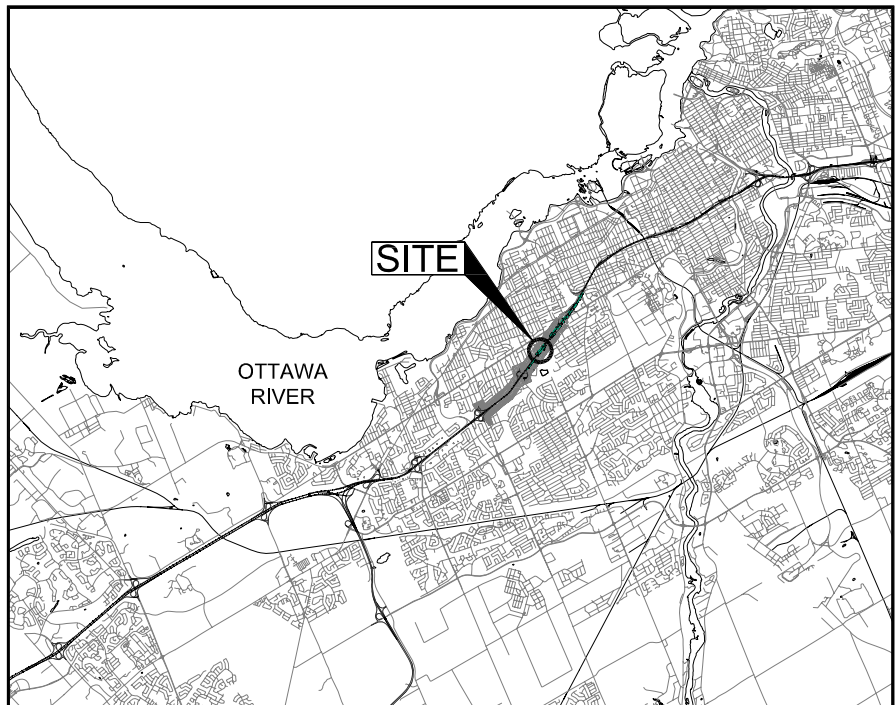
HWY. 417

WP No. WP 4058-01-00

HIGHWAY 417
RETAINING WALL 2S & 3S
PLAN AND PROFILE

SHEET

Golder Associates Ltd.
OTTAWA, ONTARIO, CANADA



KEY PLAN

LEGEND

- Borehole — Current Golder Associates Ltd. Investigation
- Seal
- Piezometer
- N Standard Penetration Test value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- 100% Rock Quality Designation (RQD)
- WL in piezometer
- WL upon completion of drilling

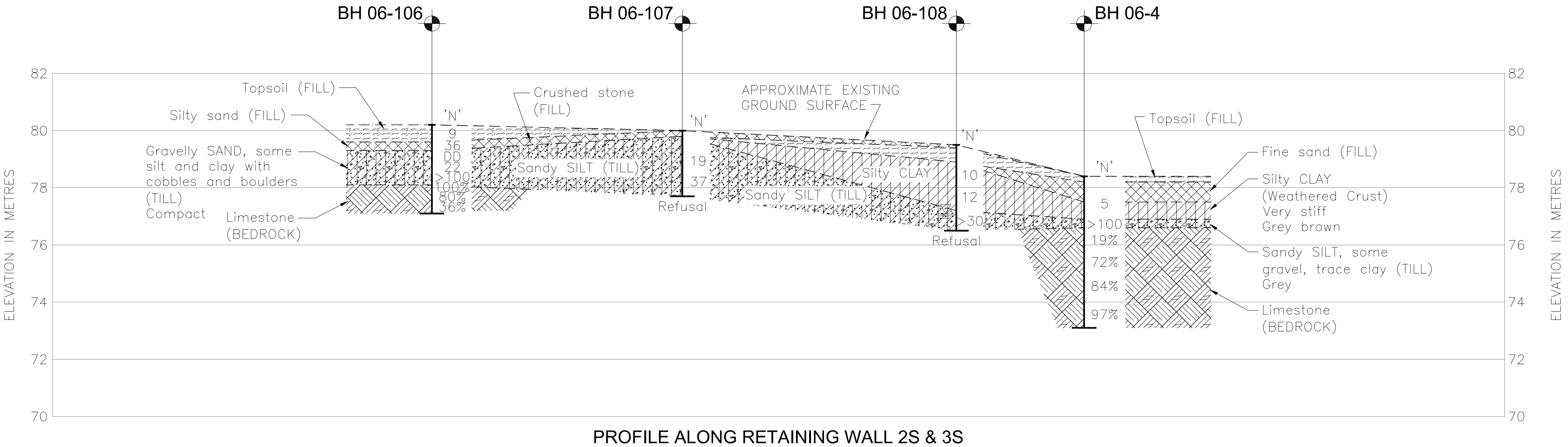
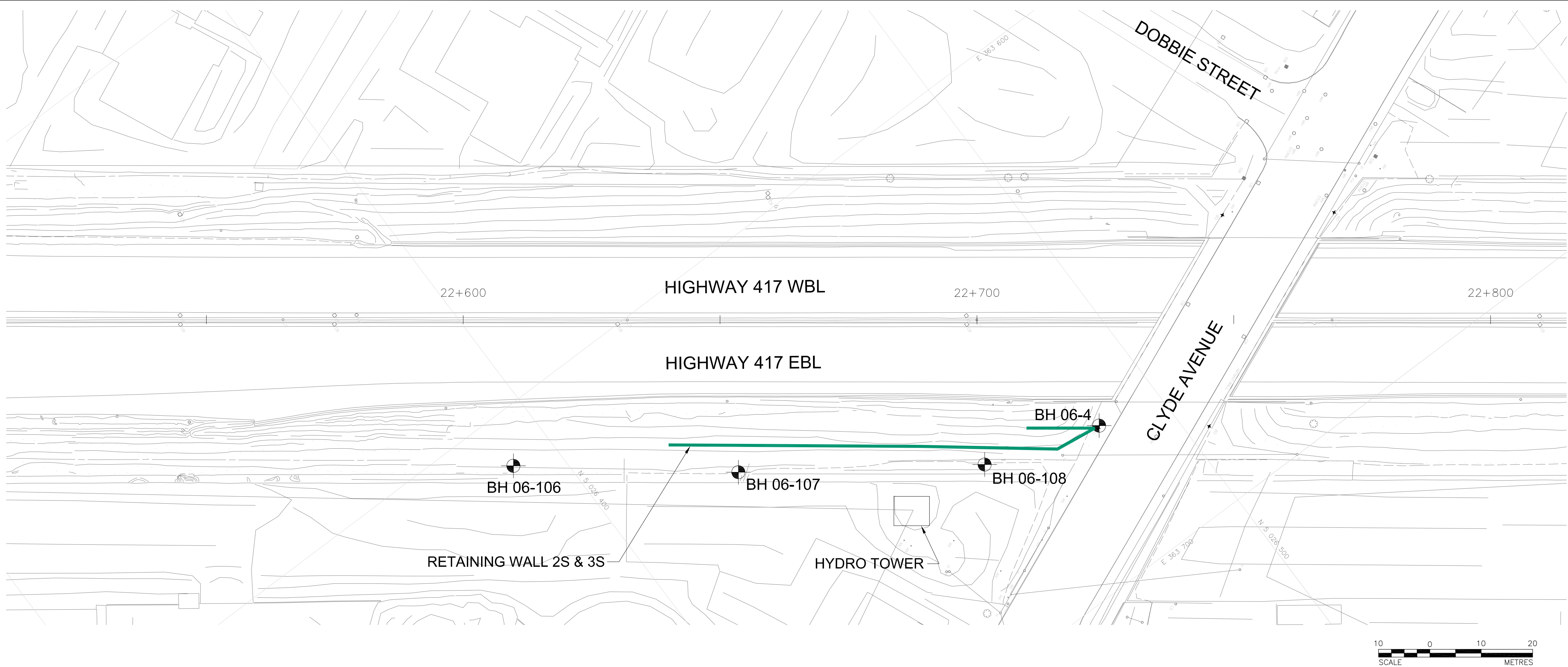
No.	ELEVATION	LOCATION	
		NORTHING	EASTING
06-106	80.2	5026391.4	363609.8
06-107	80.0	5026425.6	363637.2
06-108	79.5	5026464.8	363664.9
06-4	78.4	5026487.2	363672.2

NOTES

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DRAWN: J.M.	CHKD. W.C.	APPD.	DWG. 2



PROFILE ALONG RETAINING WALL 2S & 3S

METRIC
DIMENSIONS ARE IN METRES
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UNLESS OTHERWISE SHOWN

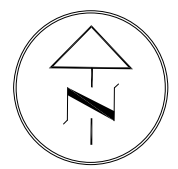
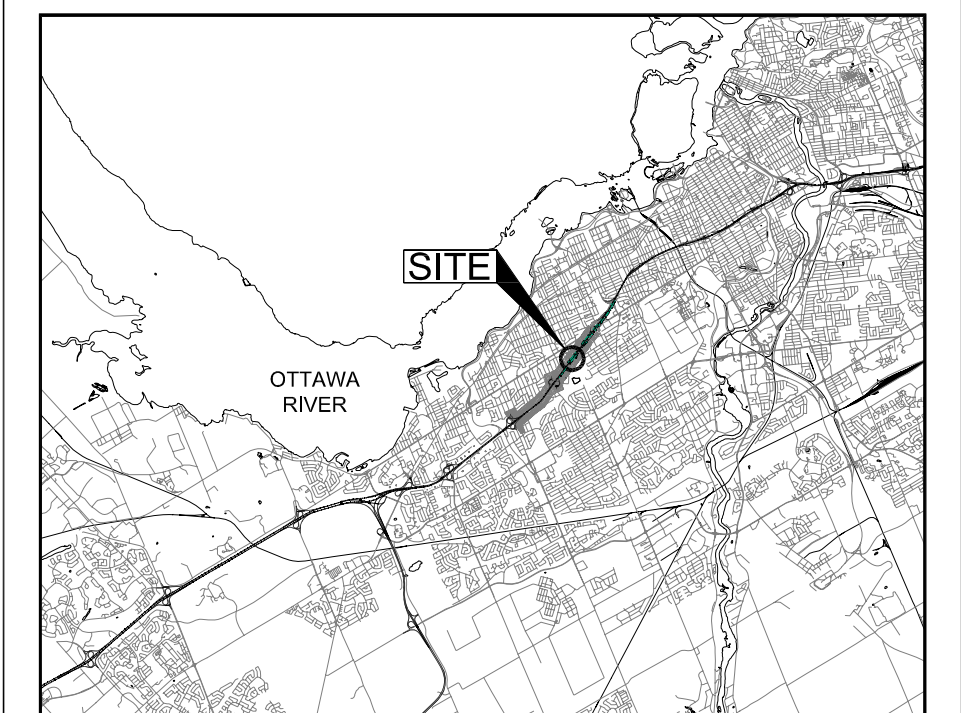
HWY. 417

WP No. WP 4058-01-00






HIGHWAY 417
RETAINING WALL 4S
PLAN AND PROFILE

SHEET

Golder Associates Ltd.
OTTAWA, ONTARIO, CANADA



KEY PLAN

LEGEND			
	Borehole — Current Golder Associates Ltd. Investigation		
	Seal		
	Piezometer		
N	Standard Penetration Test value		
16	Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)		
100%	Rock Quality Designation (RQD)		
	WL in piezometer		
	WL upon completion of drilling		

No.	ELEVATION	LOCATION	
		NORTHING	EASTING
06-111	78.5	5026553.2	363730.9
06-112	78.4	5026609.8	363772.9
06-113	78.6	5026715.8	363852.9
06-114	78.4	5026721.5	363859.6
06-3	77.8	5026503.3	363683.4

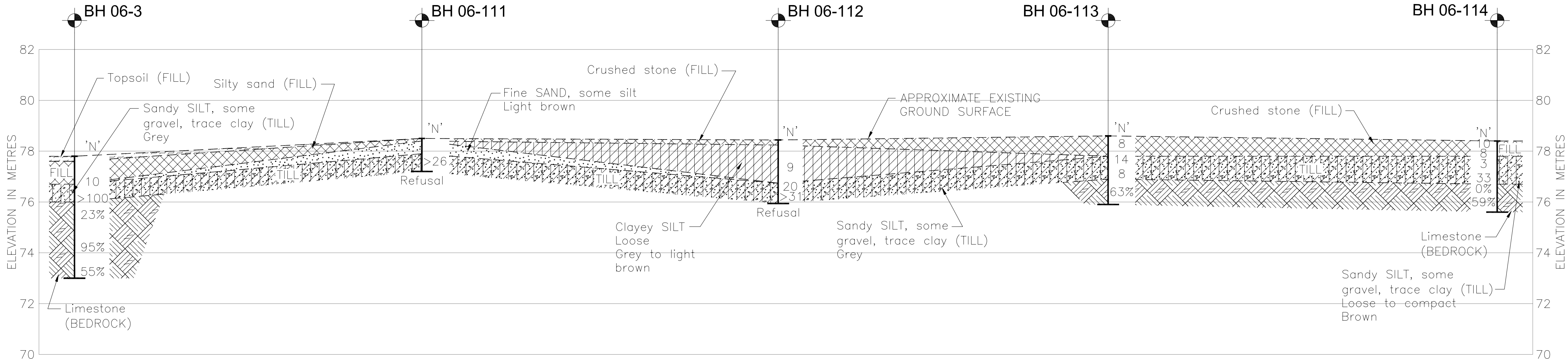
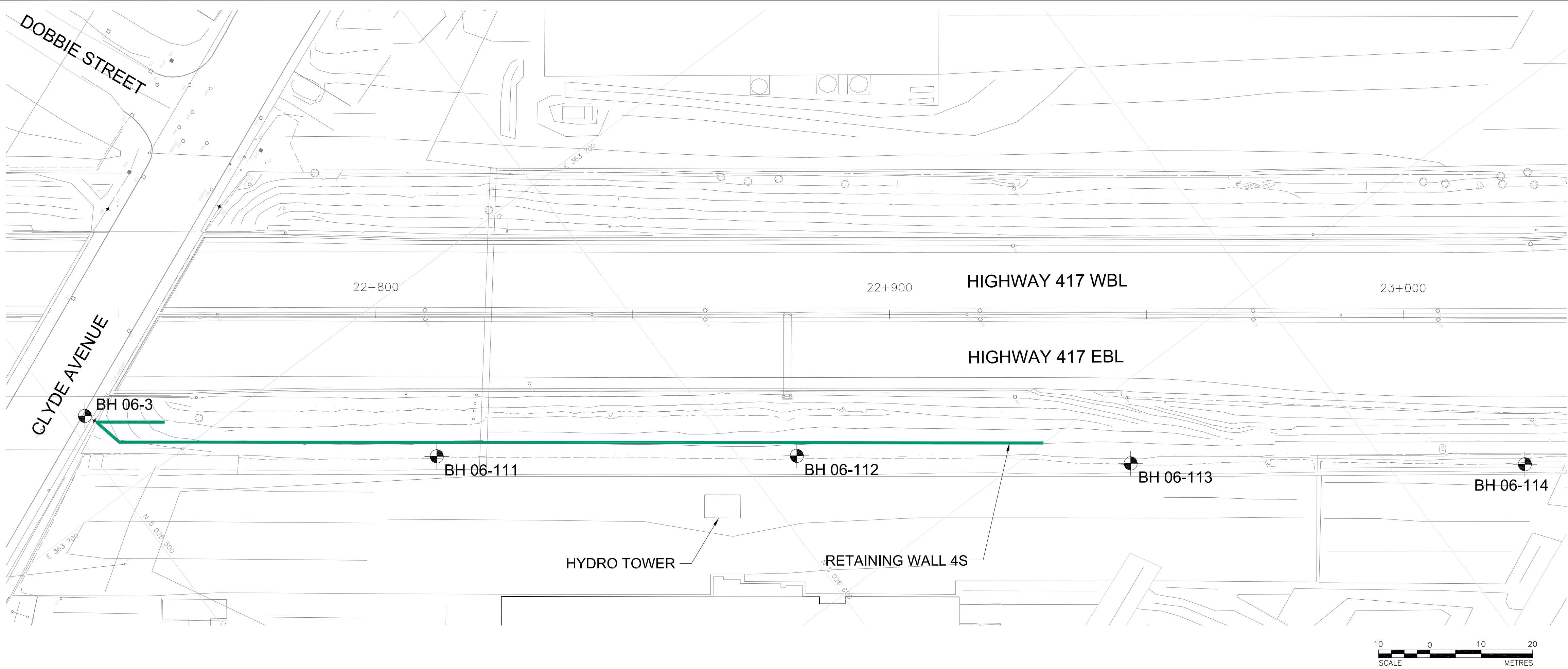
NOTES

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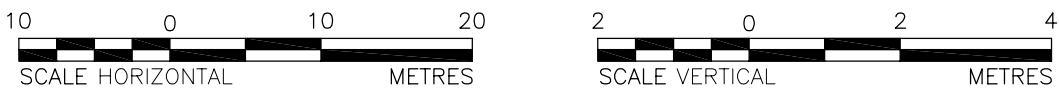
The boundaries between soil strata have been established only at borehole locations. Between Boreholes the boundaries are assumed from geological evidence.

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SUBM'D. W.C.	CHKD. M.I.C.	DATE: OCTOBER 2006	SITE:
DRAWN: J.M.	CHKD. W.C.	APPD.	DWG. 3

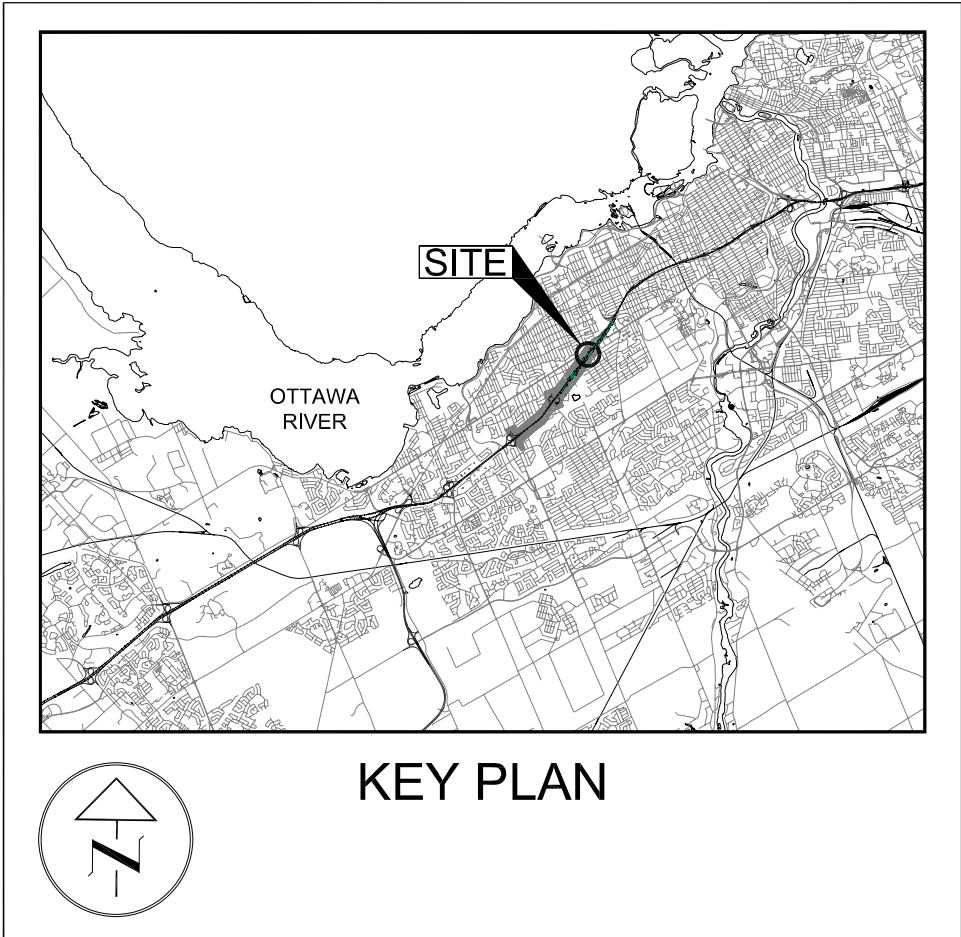







PROFILE ALONG RETAINING WALL 4S



METRIC

DIMENSIONS ARE IN METRES
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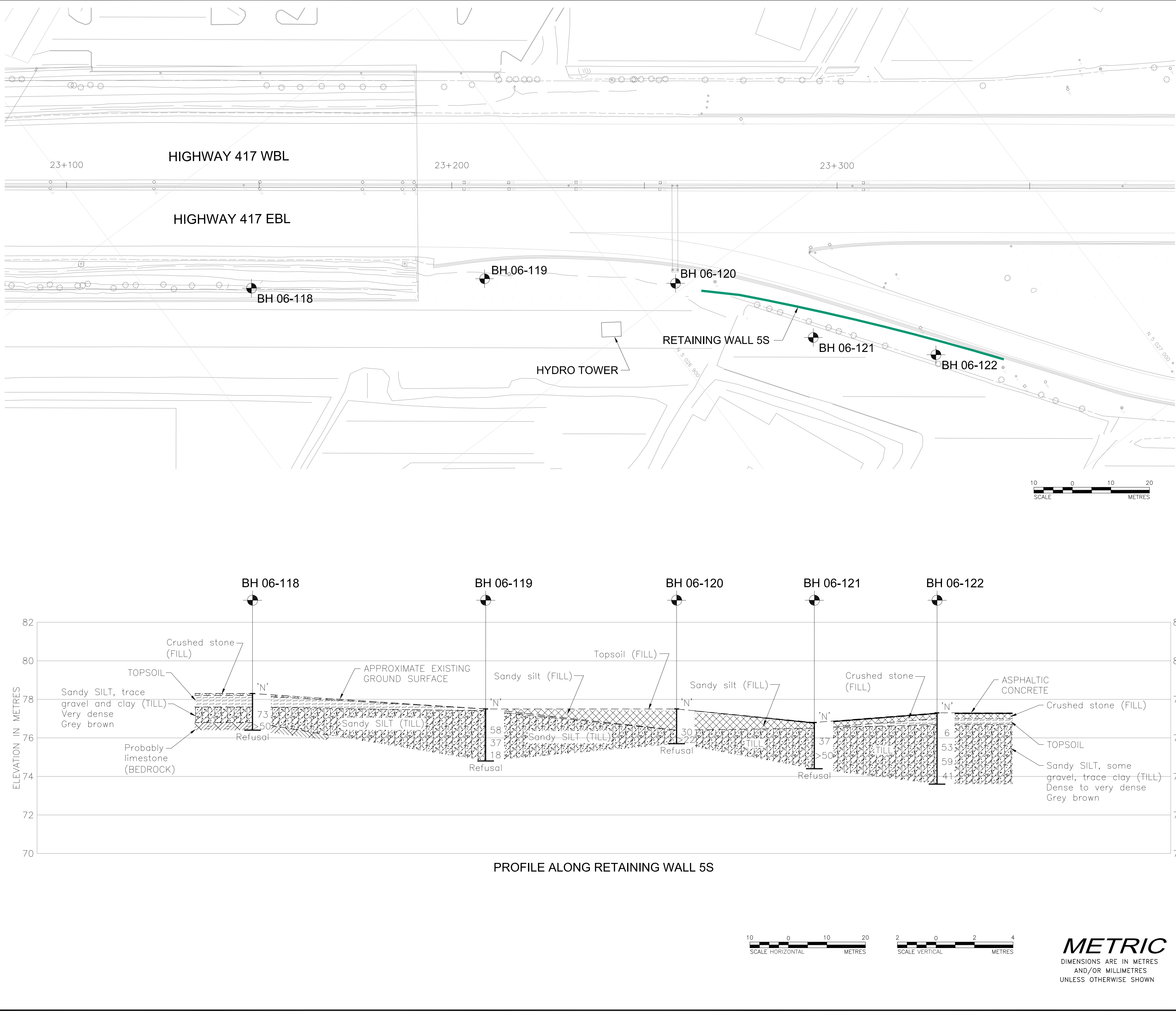
LEGEND			
	Borehole — Current Golder Associates Ltd. Investigation		
	Seal		
	Piezometer		
N	Standard Penetration Test value		
16	Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)		
100%	Rock Quality Designation (RQD)		
	WL in piezometer		
	WL upon completion of drilling		
LOCATION			
No.	ELEVATION	LOCATION	
		NORTHING	EASTING
06-118	78.3	5026822.1	363932.7
06-119	77.5	5026871.9	363967.1
06-120	77.5	5026910.7	363997.9
06-121	76.8	5026930.8	364030.6
06-122	77.3	5026953.6	364053.4

NOTES

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SUBM'D. W.C.	CHKD. M.I.C.	DATE: OCTOBER 2006	SITE:
DRAWN: J.M.	CHKD. W.C.	APPD.	DWG. 4

METRIC
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
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05-1120-210-6000-01.dwg

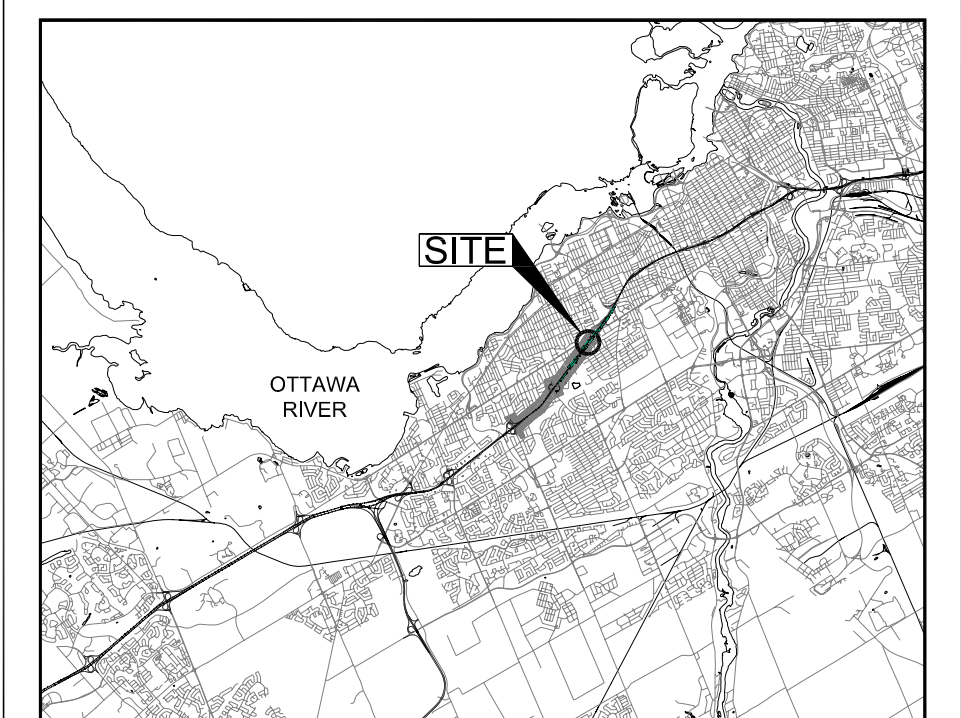
HWY. 417

WP No. WP 4058-01-00

HIGHWAY 417
RETAINING WALL 6S
PLAN AND PROFILE

SHEET
1 of 2

Golder Associates Ltd.
OTTAWA, ONTARIO, CANADA



KEY PLAN

- LEGEND
- Borehole — Current Golder Associates Ltd. Investigation
 - Seal
 - Piezometer
 - N Standard Penetration Test value
 - 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
 - 100% Rock Quality Designation (RQD)
 - WL in piezometer
 - WL upon completion of drilling

No.	ELEVATION	LOCATION	
		NORTHING	EASTING
06-127	75.5	5027168.4	364215.6
06-128	75.2	5027230.8	364247.1
06-129	75.5	5027295.3	364293.1
06-6	75.1	5027111.6	364143.8
06-10	75.0	5027364.2	364331.9

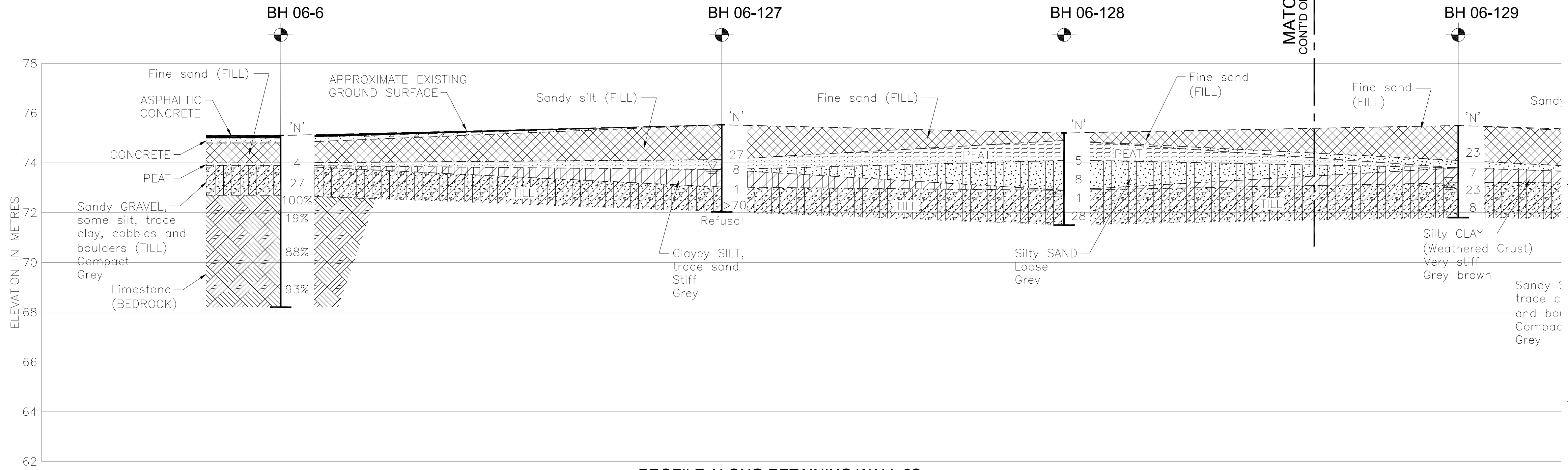
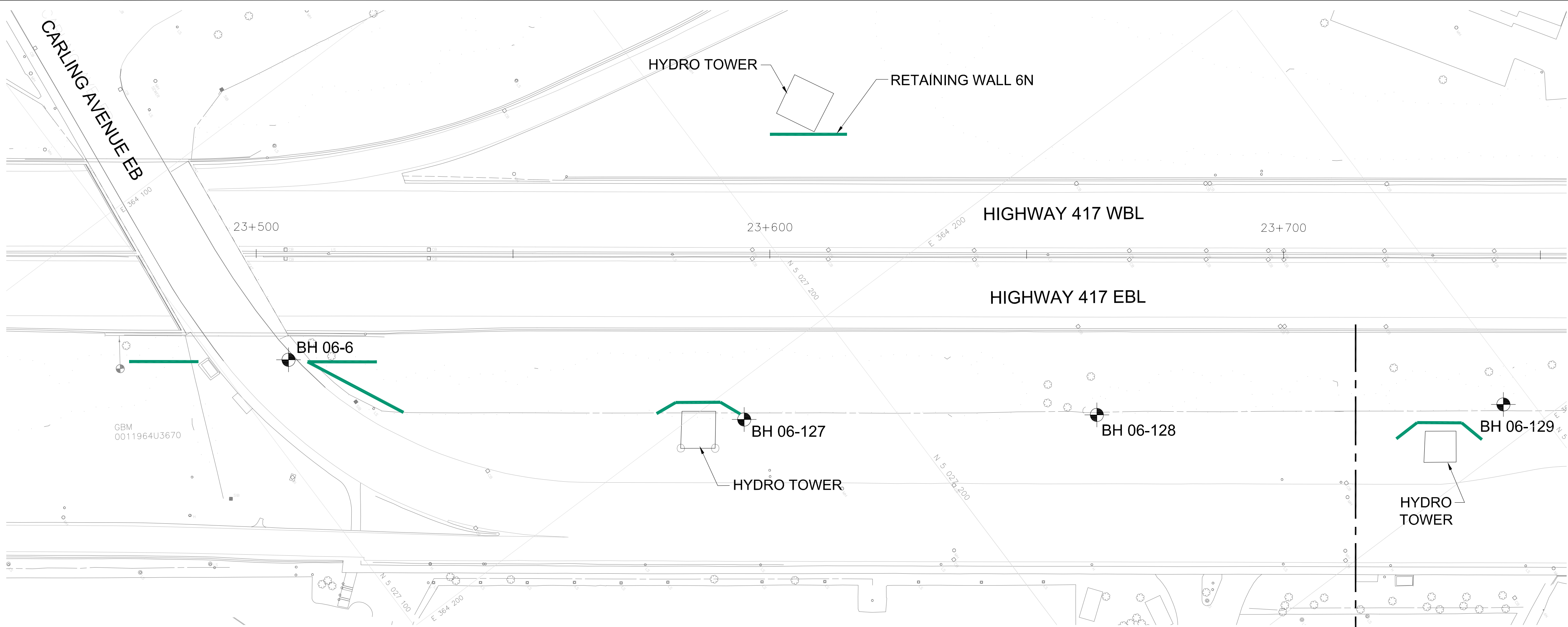
NOTES

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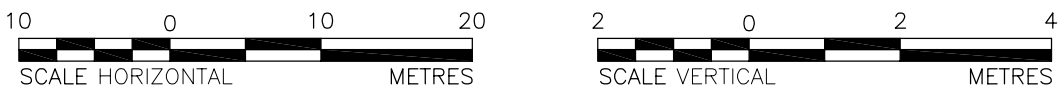
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Base plan provided in electronic format by McCormick Rankin Corporation

NO.	DATE	BY	REVISION
Geocres No. 31G5-218			
HWY. 417	PROJECT NO. 05-1120-210-2000		DIST.
SUBM'D. W.C.	CHKD. M.I.C.	DATE: OCTOBER 2006	SITE:
DRAWN: J.M.	CHKD. W.C.	APPD.	DWG. 5A



PROFILE ALONG RETAINING WALL 6S



METRIC

DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN

HWY. 417

WP No. WP 4058-01-00

HIGHWAY 417

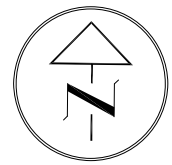
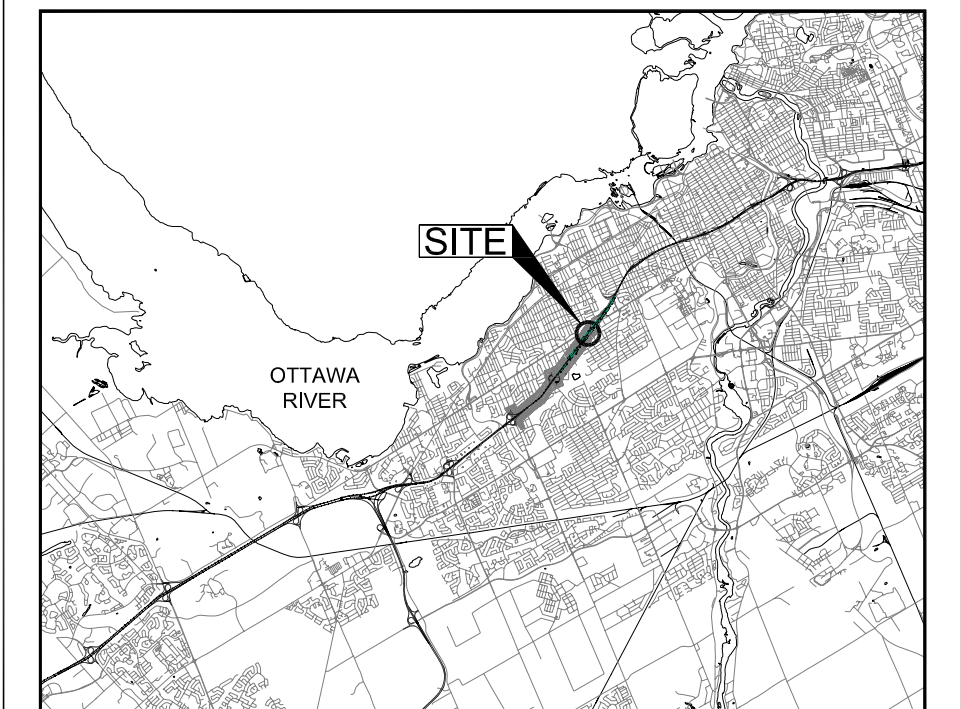
RETAINING WALL 6S

PLAN AND PROFILE

SHEET

2 of 2

Golder Associates Ltd.
OTTAWA, ONTARIO, CANADA



KEY PLAN

LEGEND

- Borehole — Current Golder Associates Ltd. Investigation
- Seal
- Piezometer
- N Standard Penetration Test value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- 100% Rock Quality Designation (RQD)
- WL in piezometer
- WL upon completion of drilling

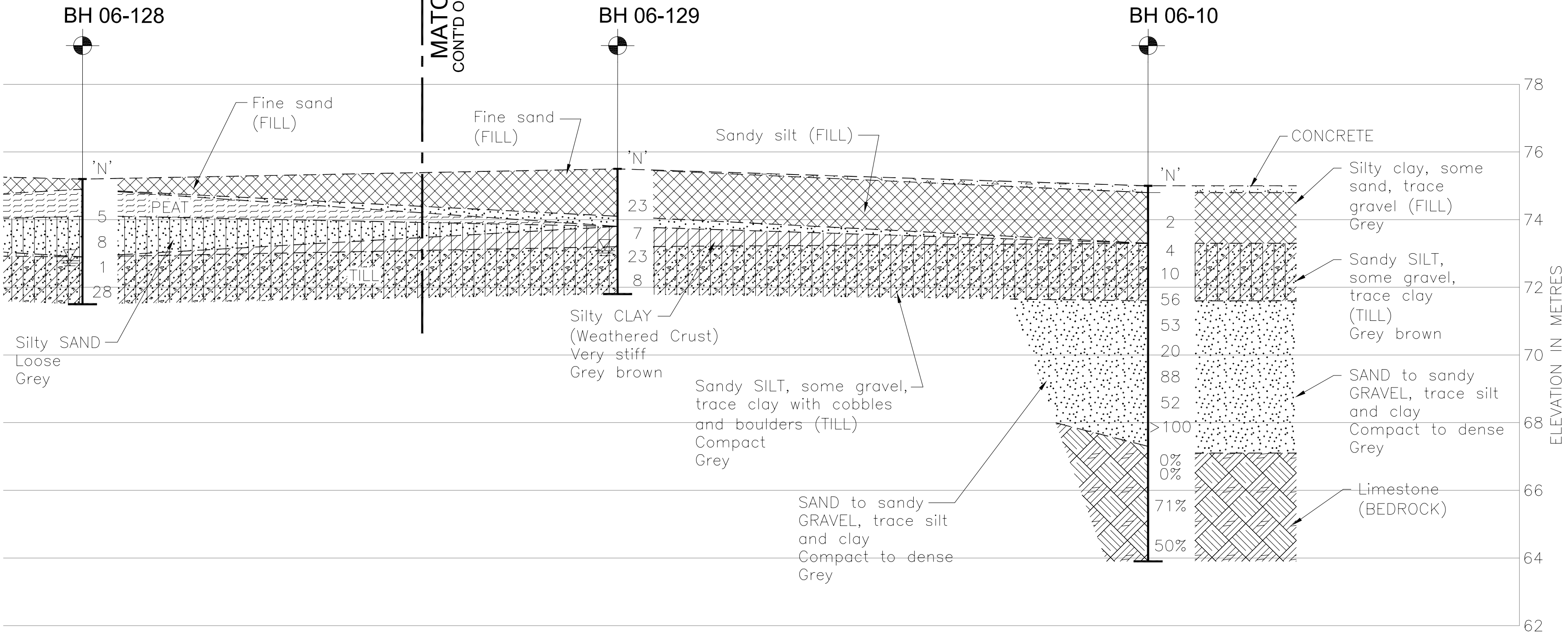
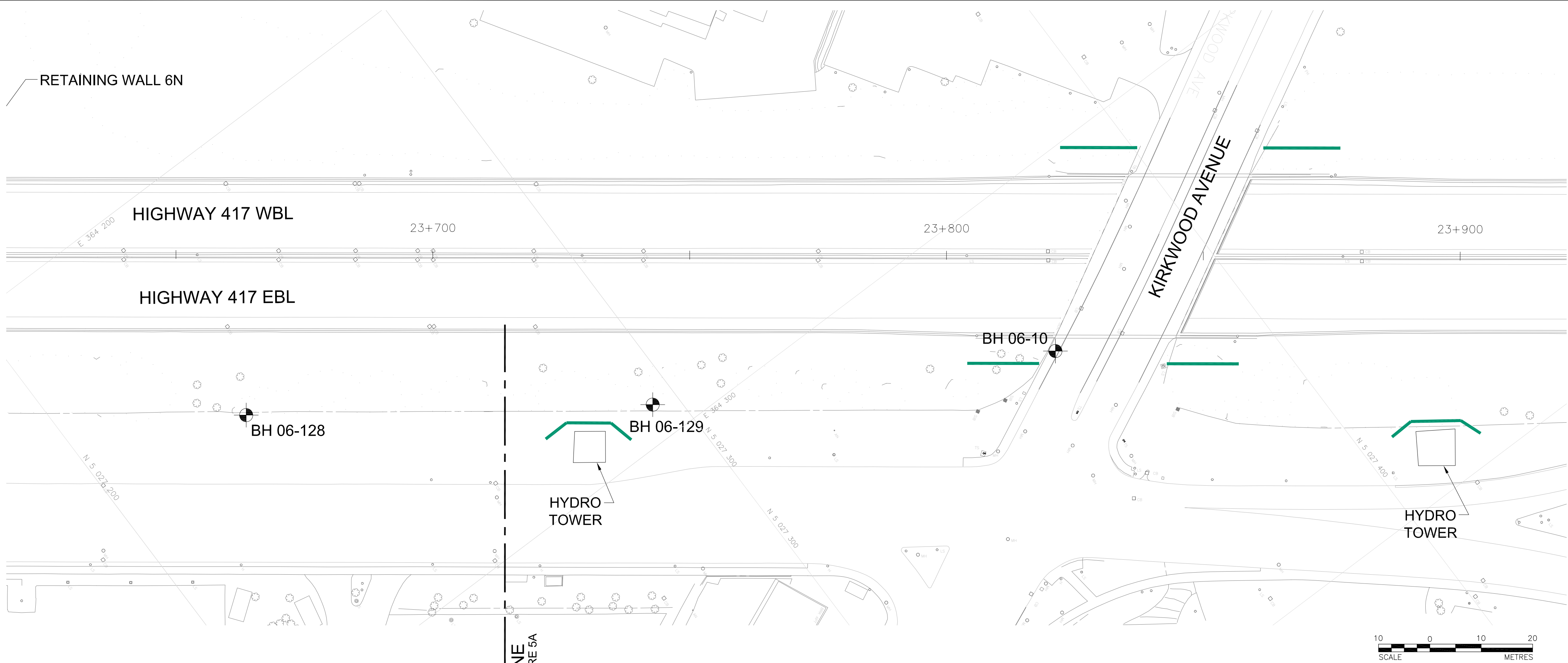
No.	ELEVATION	LOCATION	
		NORTHING	EASTING
06-127	75.5	5027168.4	364215.6
06-128	75.2	5027230.8	364247.1
06-129	75.5	5027295.3	364293.1
06-6	75.1	5027111.6	364143.8
06-10	75.0	5027364.2	364331.9

NOTES

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Base plan provided in electronic format by McCormick Rankin Corporation

NO.	DATE	BY	REVISION
Geocres No. 31G5-218			
HWY. 417	PROJECT NO. 05-1120-210-2000		DIST.
SUBM'D. W.C.	CHKD. M.I.C.	DATE: OCTOBER 2006	SITE:
DRAWN: J.M.	CHKD. W.C.	APPD.	DWG. 5B



PROFILE ALONG RETAINING WALL 6S



METRIC
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN

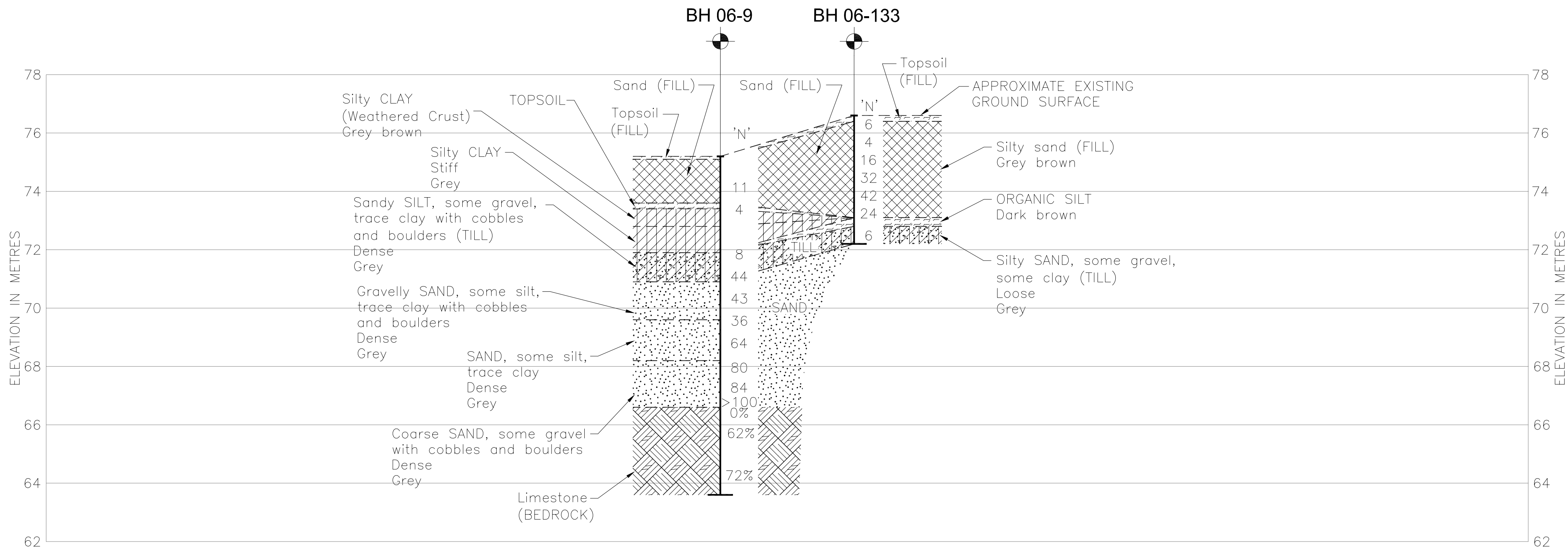
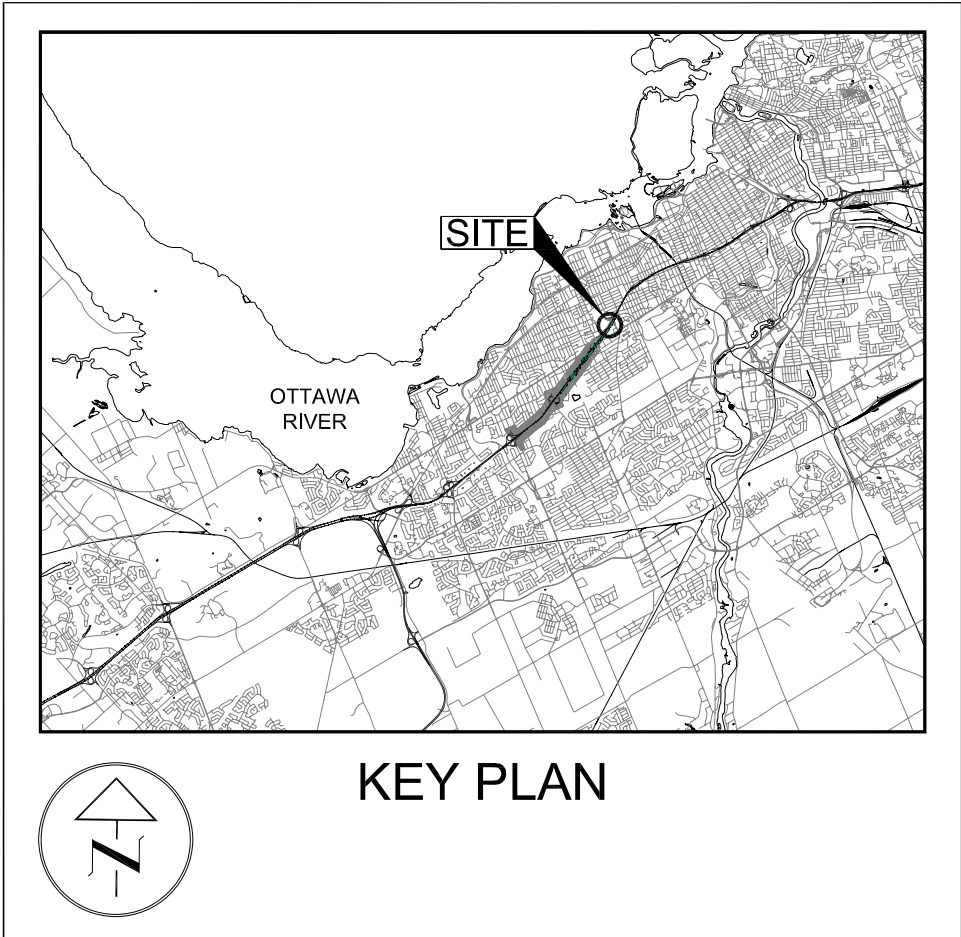
HWY. 417

WP No. WP 4058-01-00

HIGHWAY 417
RETAINING WALL 7S
PLAN AND PROFILE

SHEET

Golder Associates Ltd.
OTTAWA, ONTARIO, CANADA



LEGEND

Borehole — Current Golder Associates Ltd. Investigation

Seal

Piezometer

N

Standard Penetration Test value

16

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

100%

Rock Quality Designation (RQD)

WL in piezometer

WL upon completion of drilling

No.	ELEVATION	LOCATION	
		NORTHING	EASTING
06-133	76.6	5027391.6	364366.9
06-9	75.2	5027378.2	364347.3

NOTES

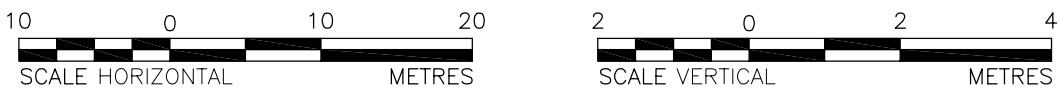
This drawing is for subsurface information only. Any surface details are for conceptual illustration.

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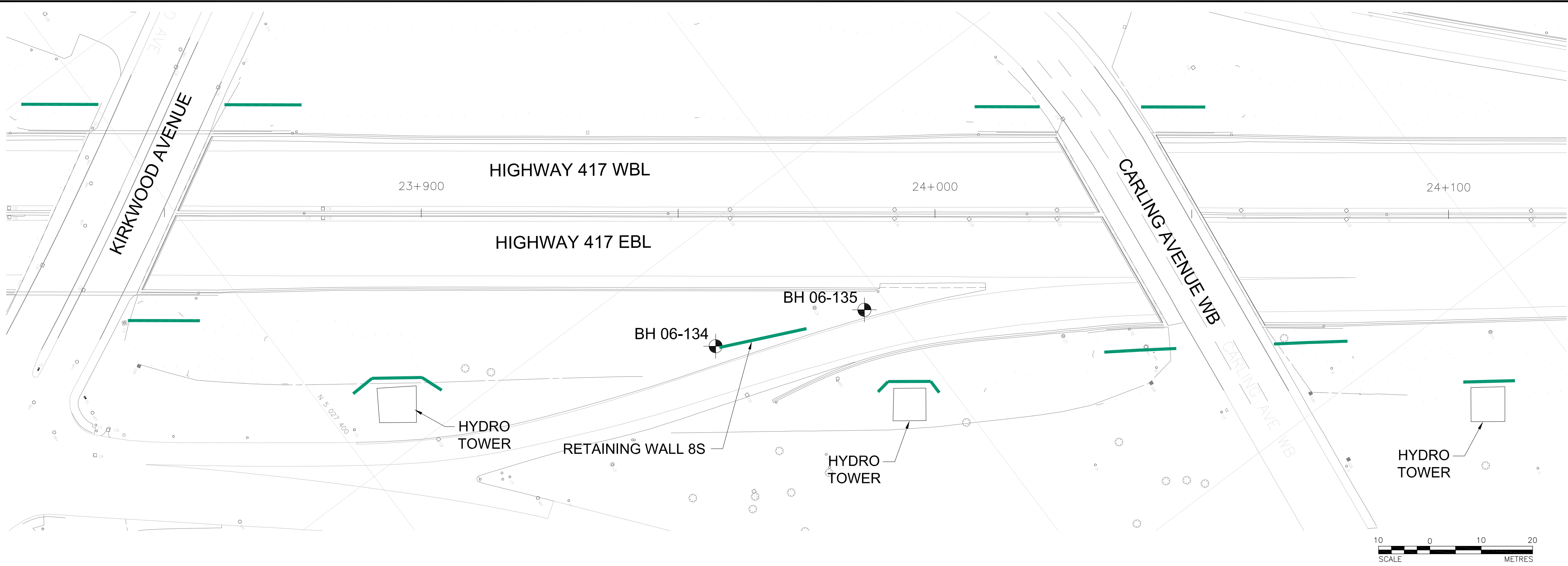
Base plan provided in electronic format by McCormick Rankin Corporation

NO.	DATE	BY	REVISION
Geocres No. 31G5-218			
HWY. 417	PROJECT NO. 05-1120-210-2000		DIST.
SUBM'D. W.C.	CHKD. M.I.C.	DATE: OCTOBER 2006	SITE:
DRAWN: J.M.	CHKD. W.C.	APPD.	DWG. 6

METRIC
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN



05-1120-210-6000-01.dwg



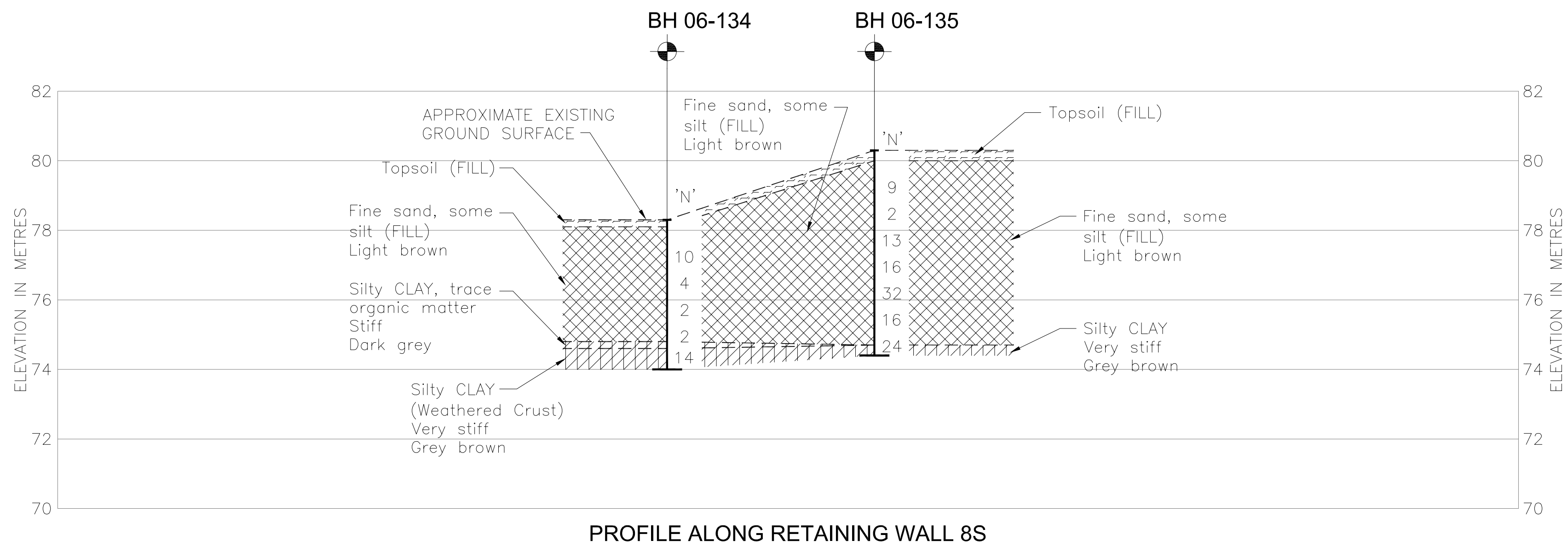
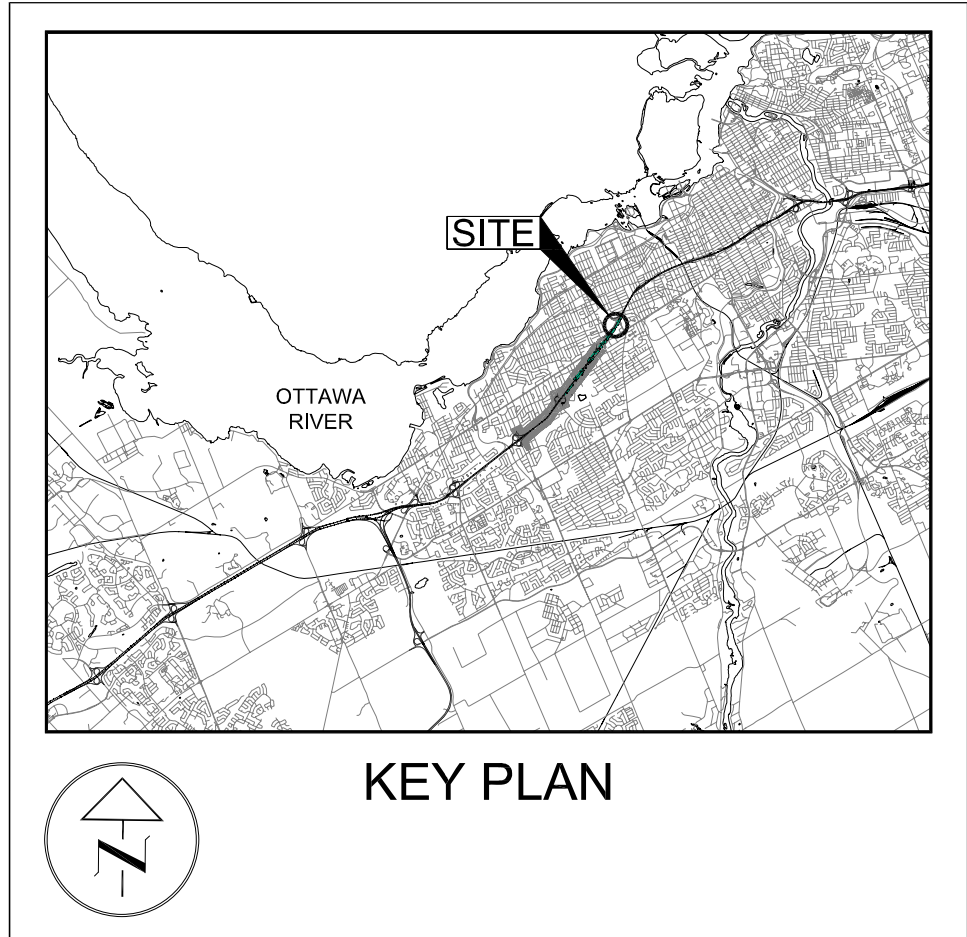
HWY. 417

WP No. WP 4058-01-00

HIGHWAY 417
RETAINING WALL 8S
PLAN AND PROFILE

SHEET

Golder Associates Ltd.
OTTAWA, ONTARIO, CANADA



LEGEND			
	Borehole — Current Golder Associates Ltd. Investigation		
	Seal		
	Piezometer		
N	Standard Penetration Test value		
16	Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)		
100%	Rock Quality Designation (RQD)		
	WL in piezometer		
	WL upon completion of drilling		
No.	ELEVATION	LOCATION	
		NORTHING	EASTING
06-134	78.3	5027468.4	364419.8
06-135	80.3	5027495.7	364431.5

NOTES

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Base plan provided in electronic format by McCormick Rankin Corporation

NO.	DATE	BY	REVISION
Geocres No. 31G5-218			
HWY. 417	PROJECT NO. 05-1120-210-2000		DIST.
SUBM'D. W.C.	CHKD. M.I.C.	DATE: OCTOBER 2006	SITE:
DRAWN: J.M.	CHKD. W.C.	APPD.	DWG. 7



METRIC

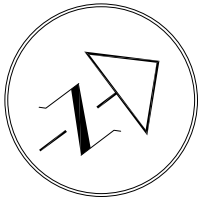
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN

05-1120-210-6000-01.dwg


HWY. 417

WP No. WP 4058-01-00

HIGHWAY 417
RETAINING WALL 9S
PLAN AND PROFILE

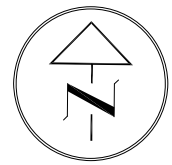
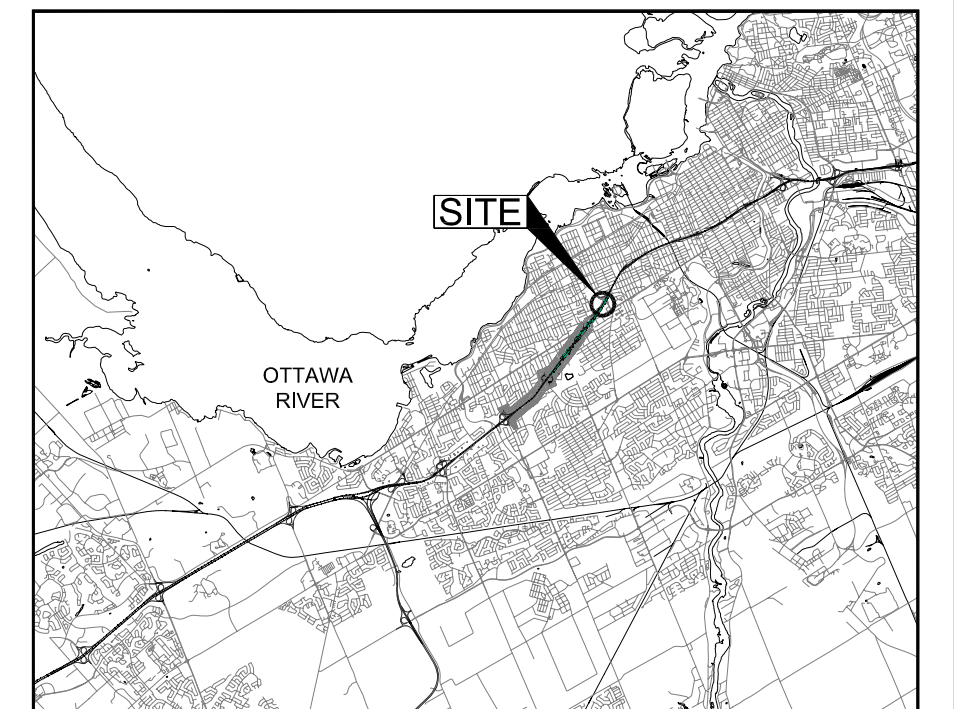


SHEET




Golder Associates Ltd.
OTTAWA, ONTARIO, CANADA


Golder Associates Ltd.
OTTAWA, ONTARIO, CANADA




KEY PLAN

LEGEND

 Borehole – Current Golder Associates Ltd. Investigation

 Seal

 Piezometer

N


Standard Penetration Test value


16

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

100%

Rock Quality Designation (RQD)

 WL in piezometer

 WL upon completion of drilling

No.	ELEVATION	LOCATION	
		NORTHING	EASTING
06-136	78.8	5027479.3	364439.8
06-137	79.3	5027507.3	364454.5
06-14	75.2	5027540.4	364478.1

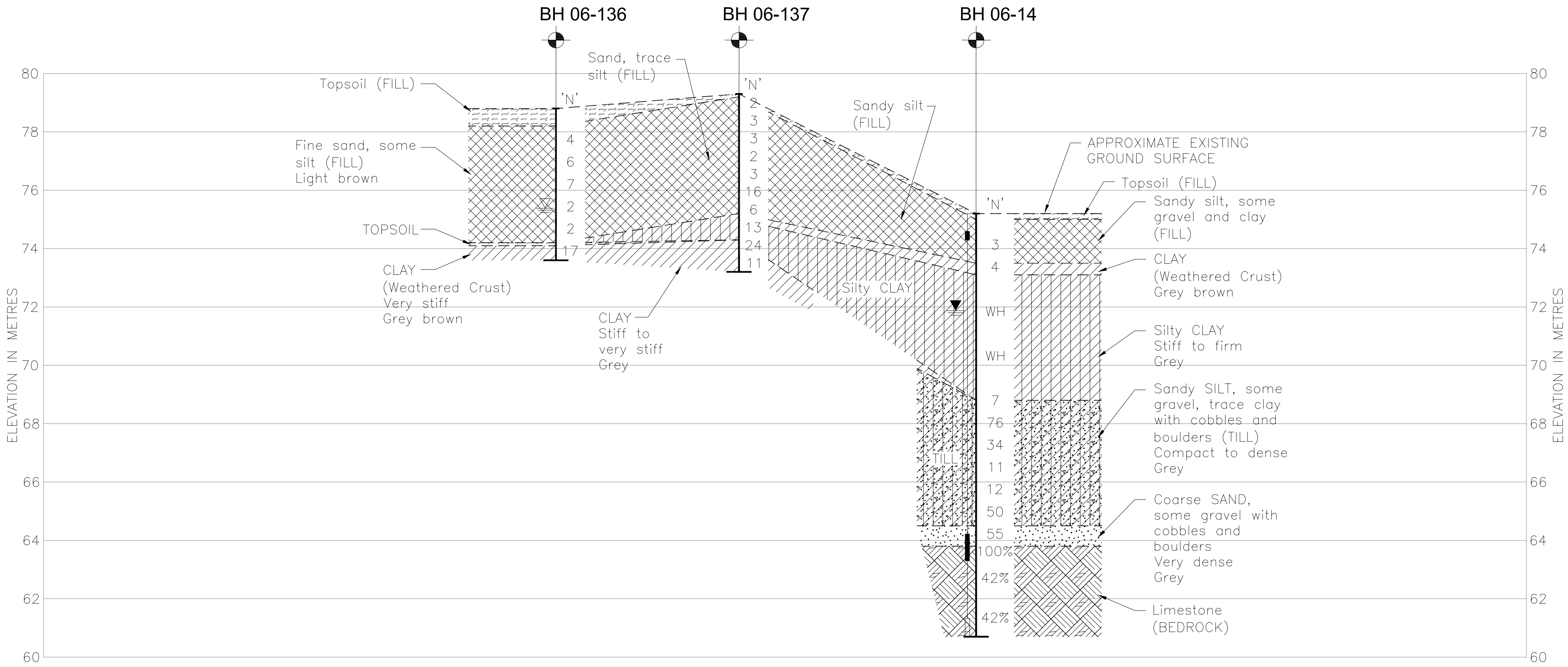
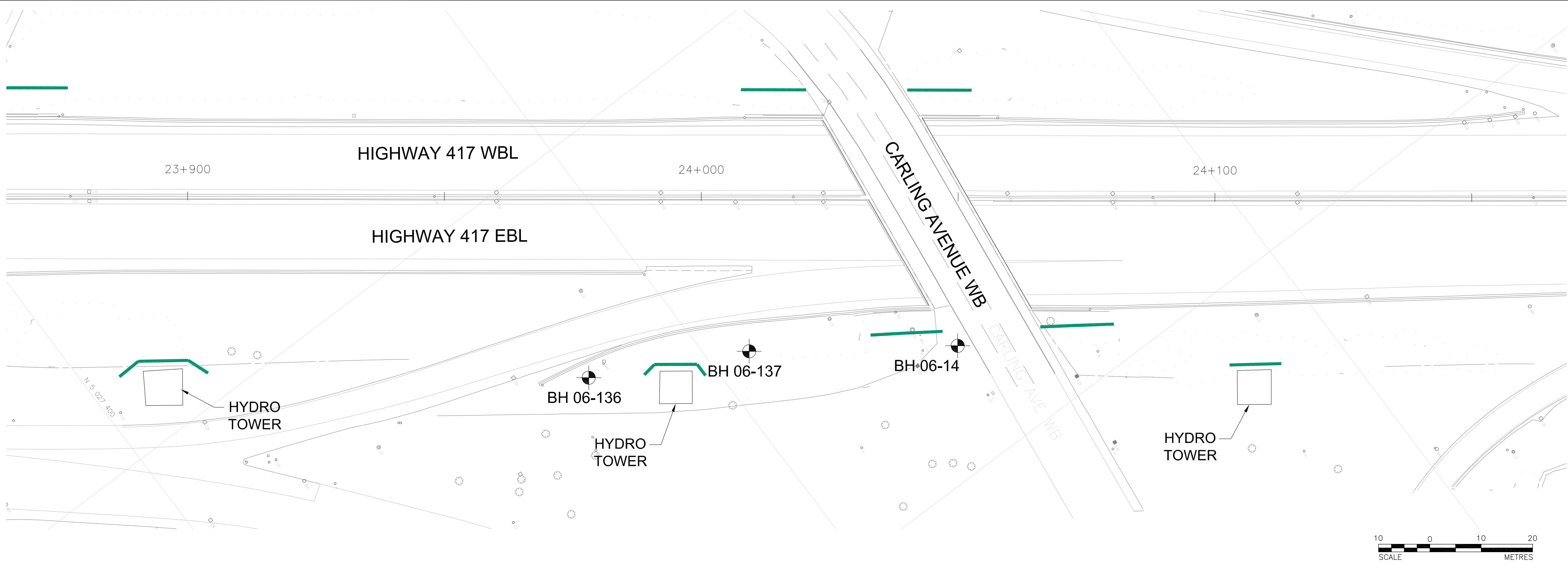
NOTES

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Base plan provided in electronic format by McCormick Rankin Corporation

NO.	DATE	BY	REVISION
Geocres No. 31G5-218			
HWY. 417	PROJECT NO. 05-1120-210-2000		DIST.
SUBM'D. W.C.	CHKD. M.I.C.	DATE: OCTOBER 2006	SITE:
DRAWN: J.M.	CHKD. W.C.	APPD.	DWG. 8

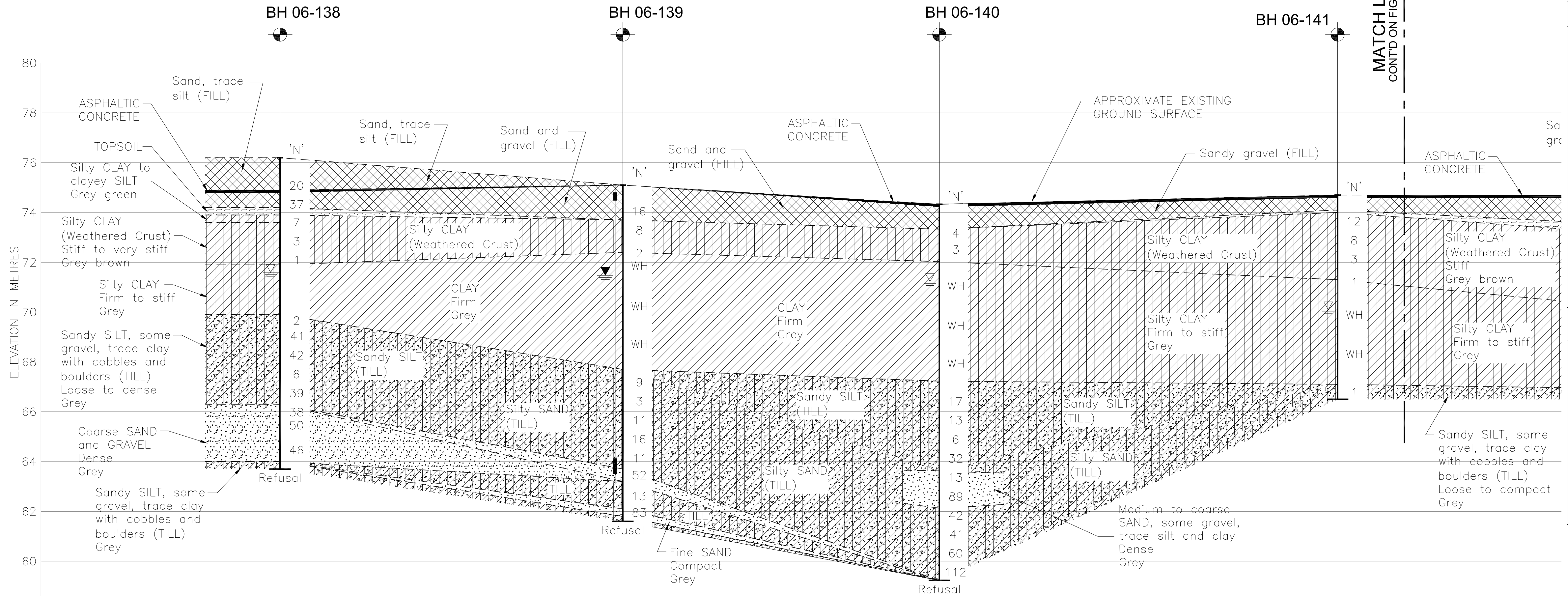
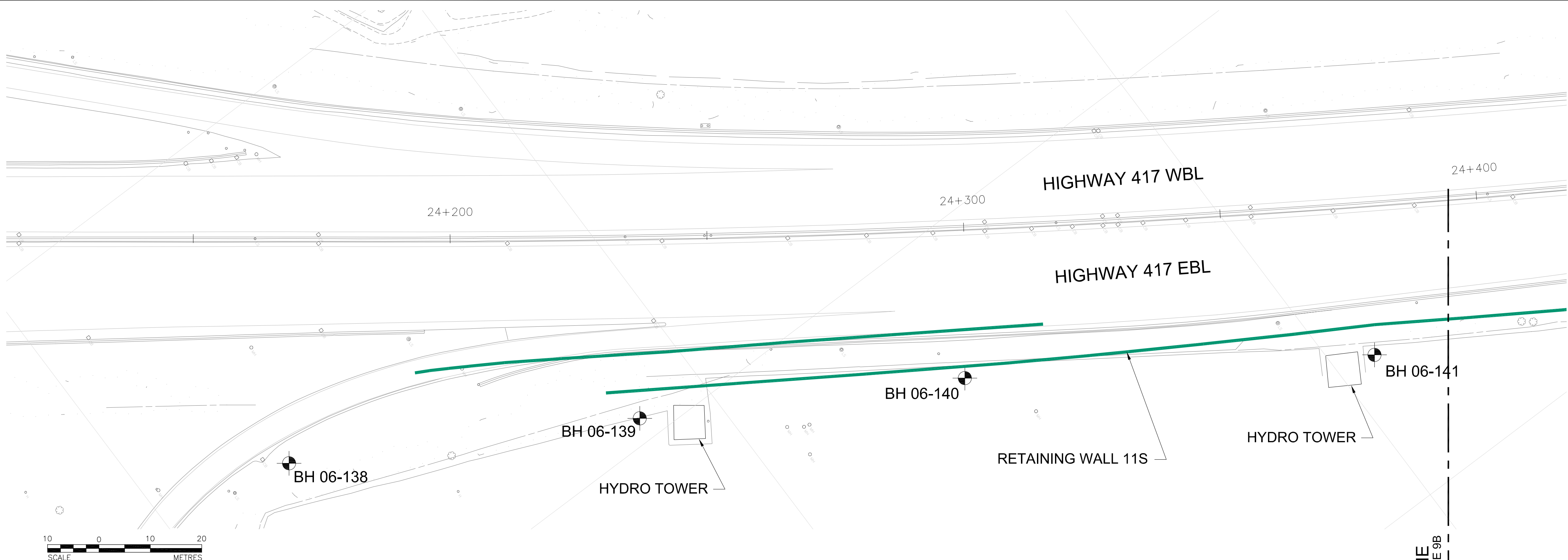
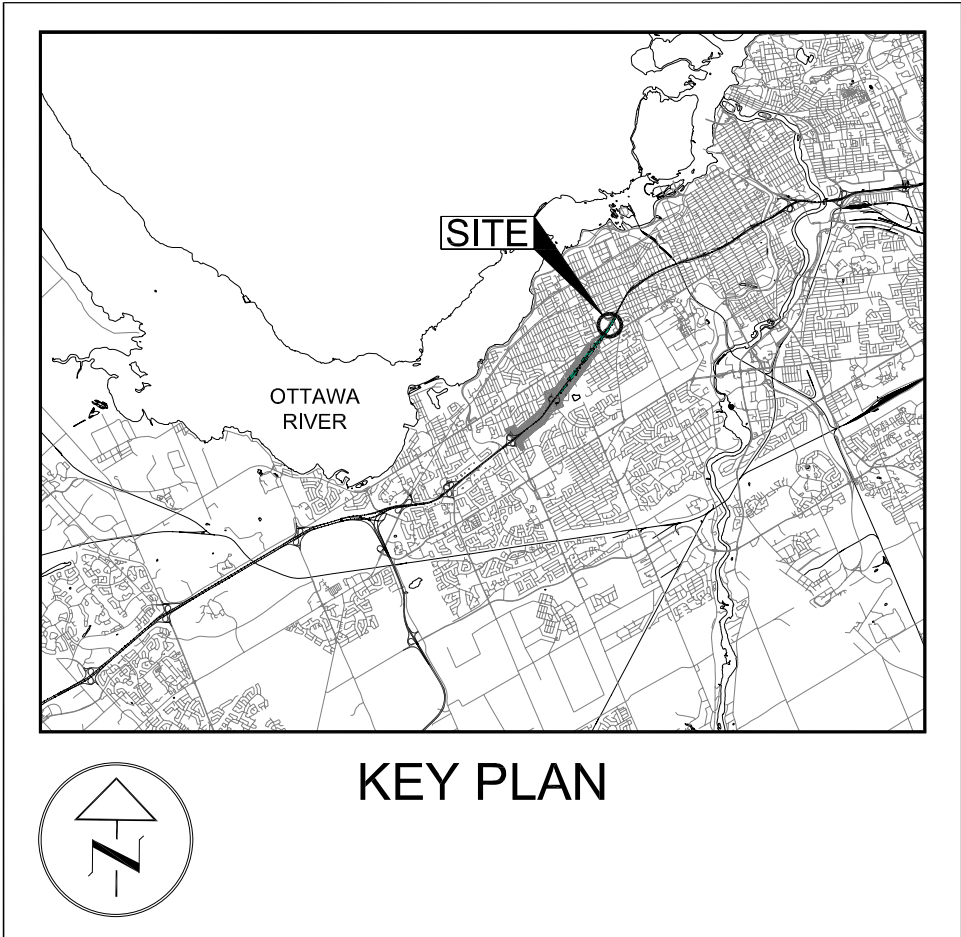


PROFILE ALONG RETAINING WALL 9S

METRIC
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN

05-1120-210-6000-01.dwg

1:1



PROFILE ALONG RETAINING WALL 11S

LEGEND

Borehole – Current Golder Associates Ltd. Investigation

Seal

Piezometer

N

 Standard Penetration Test value

16

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

100%

 Rock Quality Designation (RQD)

WL in piezometer

WL upon completion of drilling

NOTES

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Base plan provided in electronic format by McCormick Rankin Corporation

NO.	DATE	BY	REVISION
Geocres No. 31G5-218			
HWY. 417	PROJECT NO. 05-1120-210-2000		DIST.
SUBM'D. W.C.	CHKD. M.I.C.	DATE: OCTOBER 2006	SITE:
DRAWN: J.M.	CHKD. W.C.	APPD.	DWG. 9A

METRIC
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN

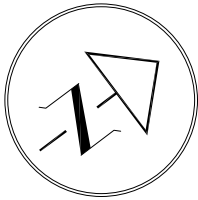


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HWY. 417

WP No. WP 4058-01-00

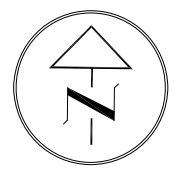
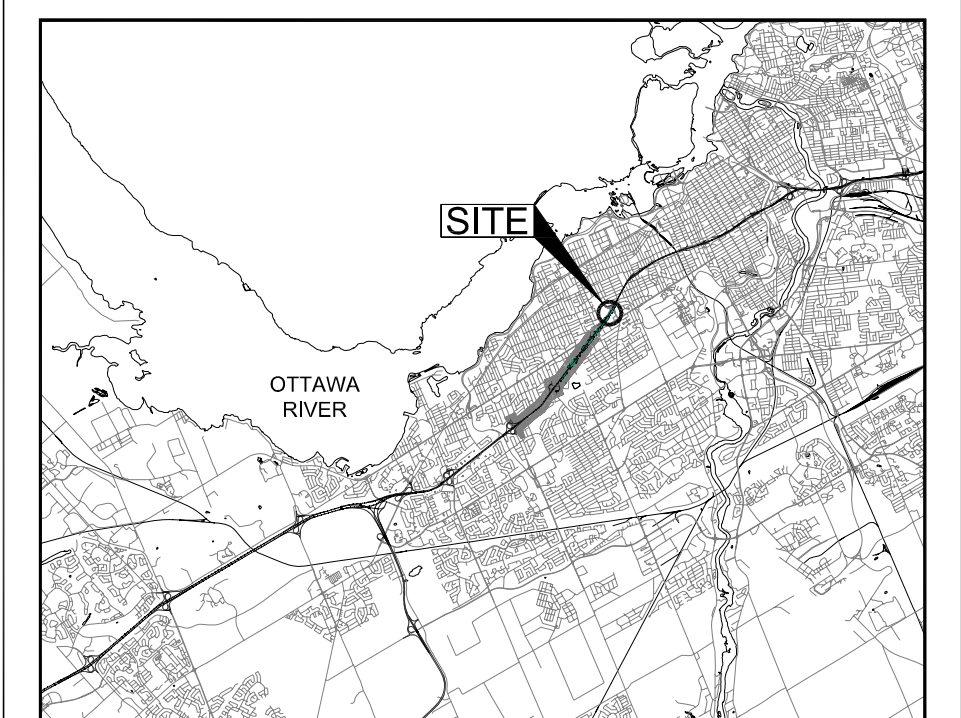
HIGHWAY 417
RETAINING WALL 11S
PLAN AND PROFILE



SHEET
2 of 2





Golder Associates Ltd.
OTTAWA, ONTARIO, CANADA




KEY PLAN

LEGEND

 Borehole – Current Golder Associates Ltd. Investigation


 Seal


 Piezometer

N Standard Penetration Test value

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

100% Rock Quality Designation (RQD)

 WL in piezometer

 WL upon completion of drilling

No.	ELEVATION	LOCATION	
		NORTHING	EASTING
06-138	76.2	5027626.2	364561.4
06-139	75.1	5027686.0	364595.5
06-140	74.3	5027733.3	364607.9
06-141	74.7	5027807.7	364671.7
06-142	74.7	5027865.9	364708.2
06-17	74.1	5027932.3	364740.8

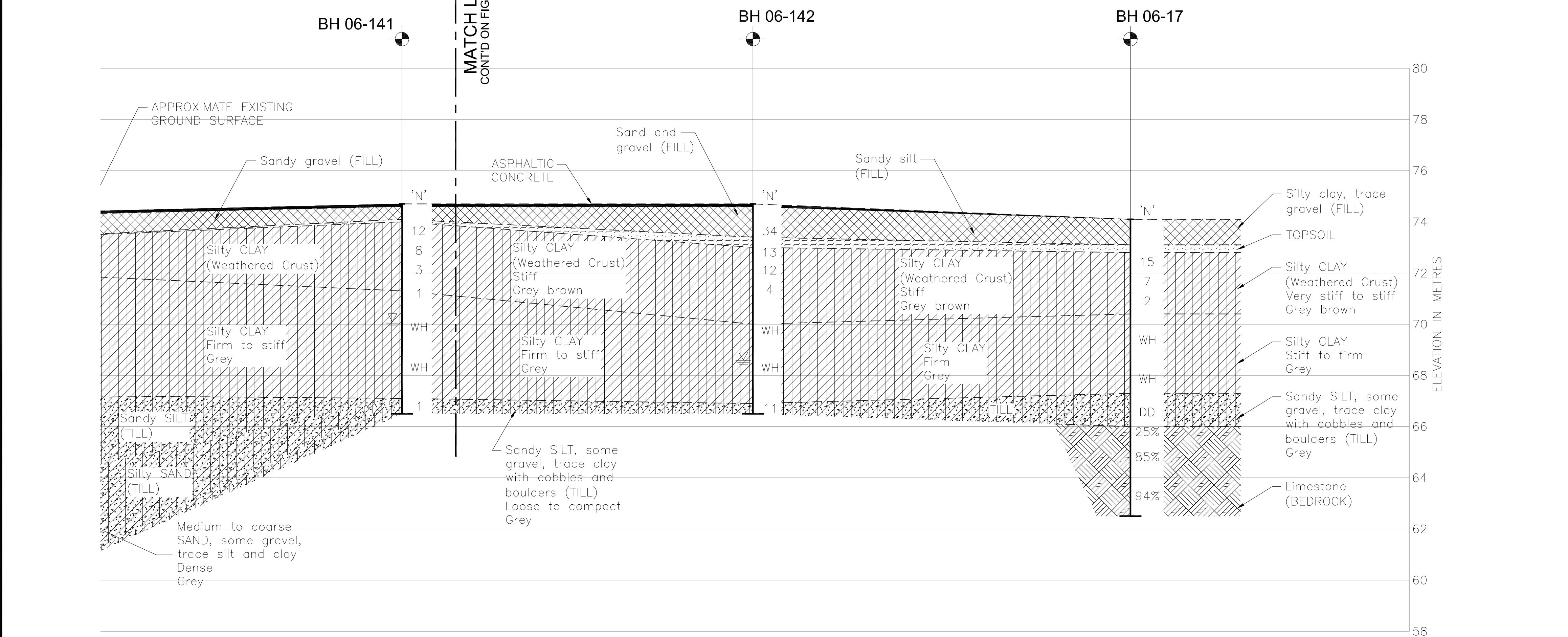
NOTES

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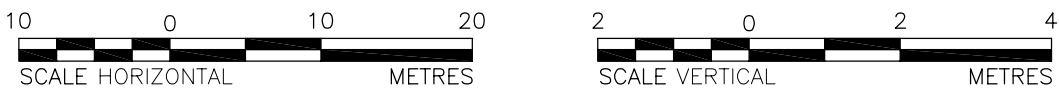
The boundaries between soil strata have been established only at borehole locations. Between Boreholes the boundaries are assumed from geological evidence.

Base plan provided in electronic format by McCormick Rankin Corporation

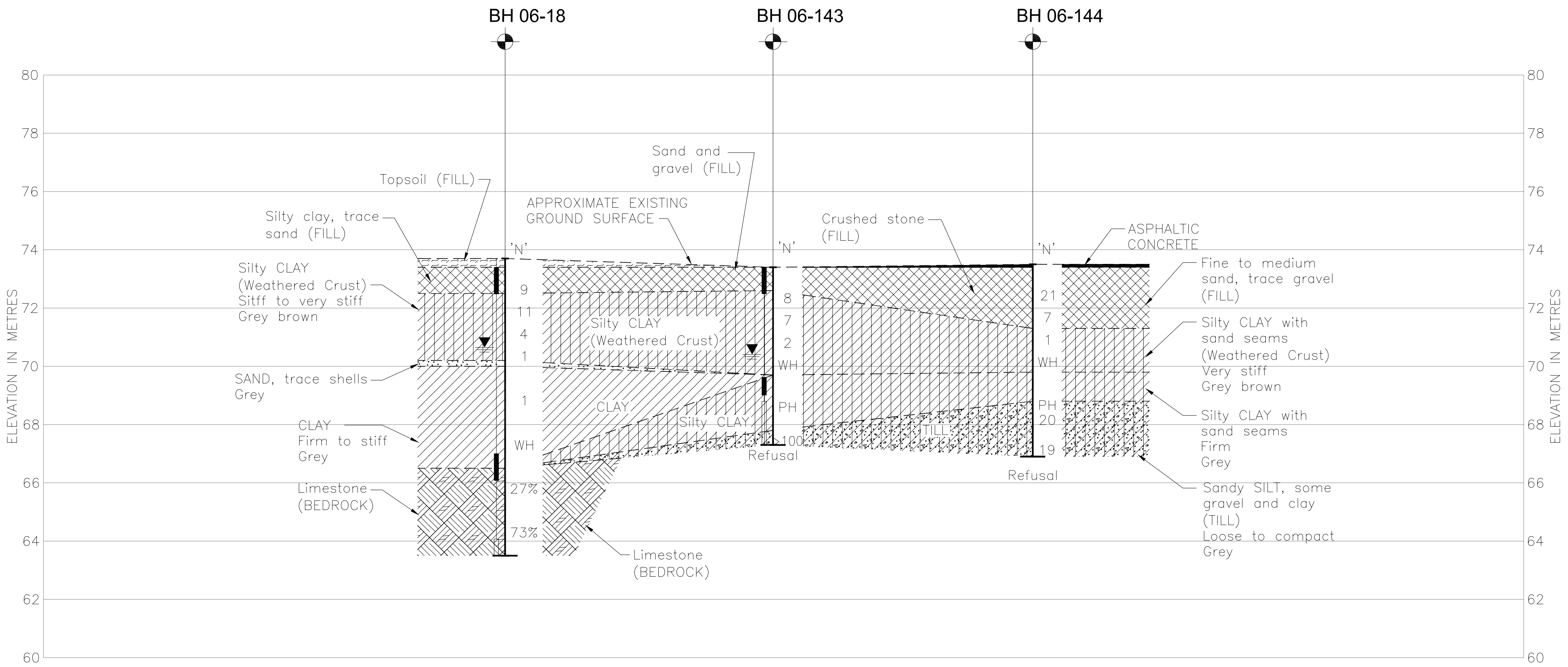
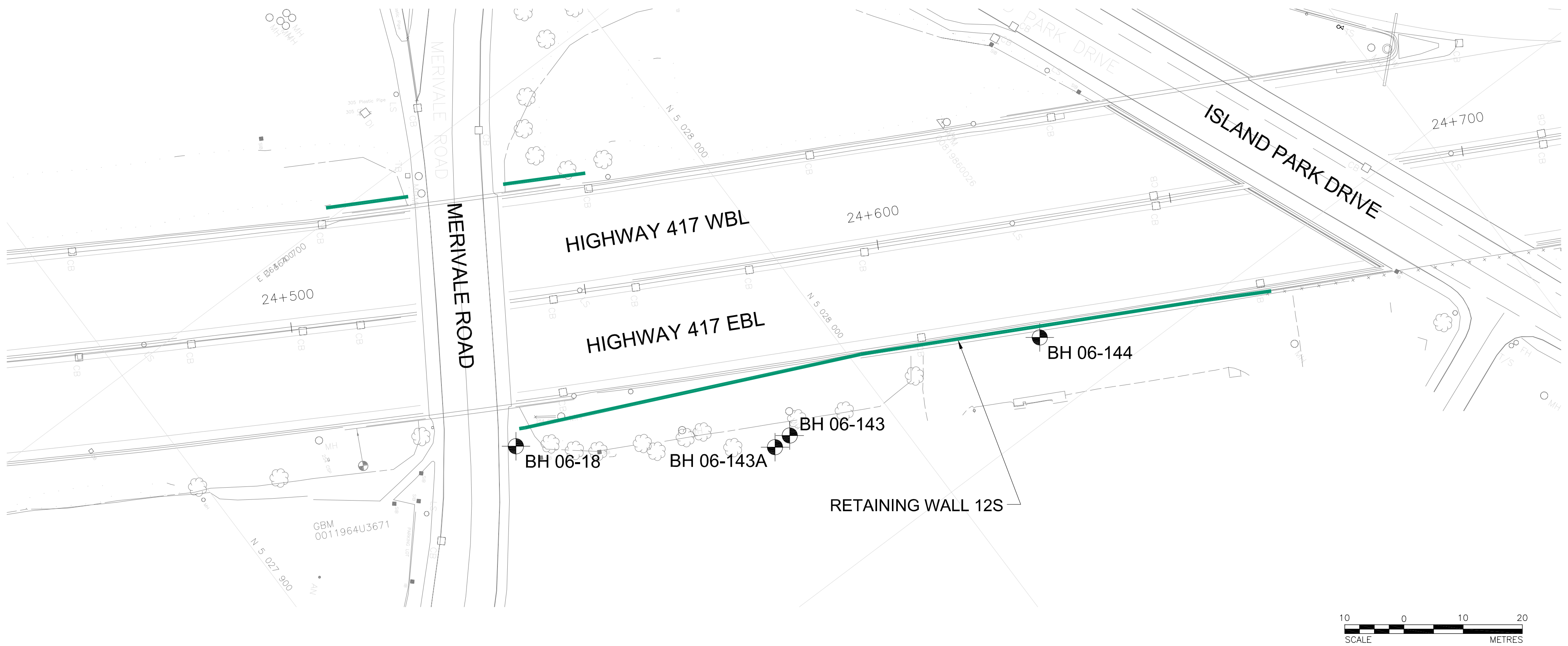
NO.	DATE	BY	REVISION	
Geocres No. 31G5—218				
HWY. 417		PROJECT NO. 05-1120-210-2000		DIST.
SUBM'D. W.C.	CHKD. M.I.C.	DATE: OCTOBER 2006	SITE:	
DRAWN: J.M.	CHKD. W.C.	APPD.	DWG. 9B	



PROFILE ALONG RETAINING WALL 11S



METRIC
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN



PROFILE ALONG RETAINING WALL 12S



METRIC
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN

HWY. 417

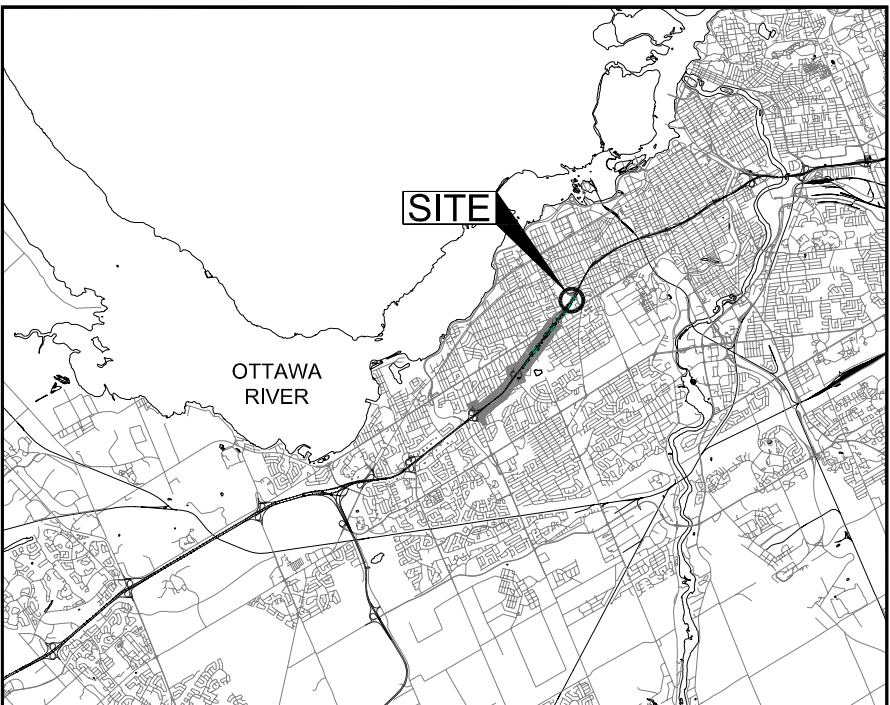
WP No. WP 4058-01-00

HIGHWAY 417
RETAINING WALL 12S
PLAN AND PROFILE

SHEET



Golder Associates Ltd.
OTTAWA, ONTARIO, CANADA



KEY PLAN



LEGEND

Borehole — Current Golder Associates Ltd. Investigation

Seal

Piezometer

N Standard Penetration Test value

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

Rock Quality Designation (RQD)

WL in piezometer

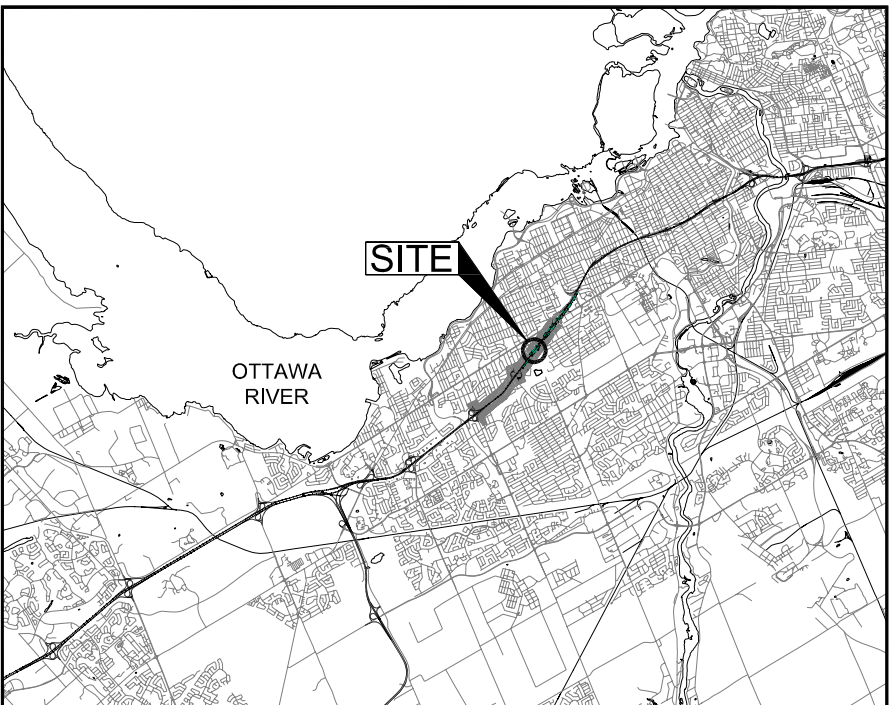
WL upon completion of drilling

No.	ELEVATION	LOCATION	
		NORTHING	EASTING
06-143	73.4	5027983.9	364775.9
06-143A	73.4	5027980.8	364776.0
06-144	73.5	5028028.8	364786.5
06-18	73.7	5027945.9	364749.6

NOTES

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Base plan provided in electronic format by McCormick Rankin Corporation

NO.	DATE	BY	REVISION
Geocres No. 31G5-218			
HWY. 417	PROJECT NO. 05-1120-210-2000		DIST.
SUBM'D. W.C.	CHKD. M.I.C.	DATE: OCTOBER 2006	SITE:
DRAWN: J.M.	CHKD. W.C.	APPD.	DWG. 10



KEY PLAN

- LEGEND
- Borehole — Current Golder Associates Ltd. Investigation
 - Seal
 - Piezometer
 - N Standard Penetration Test value
 - 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
 - 100% Rock Quality Designation (RQD)
 - WL in piezometer
 - WL upon completion of drilling

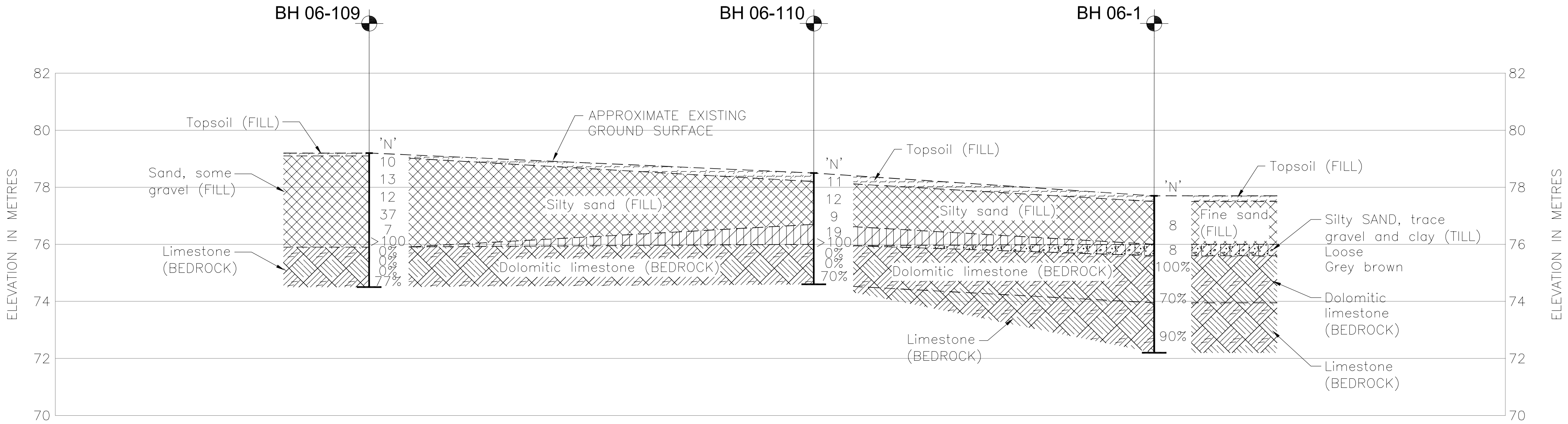
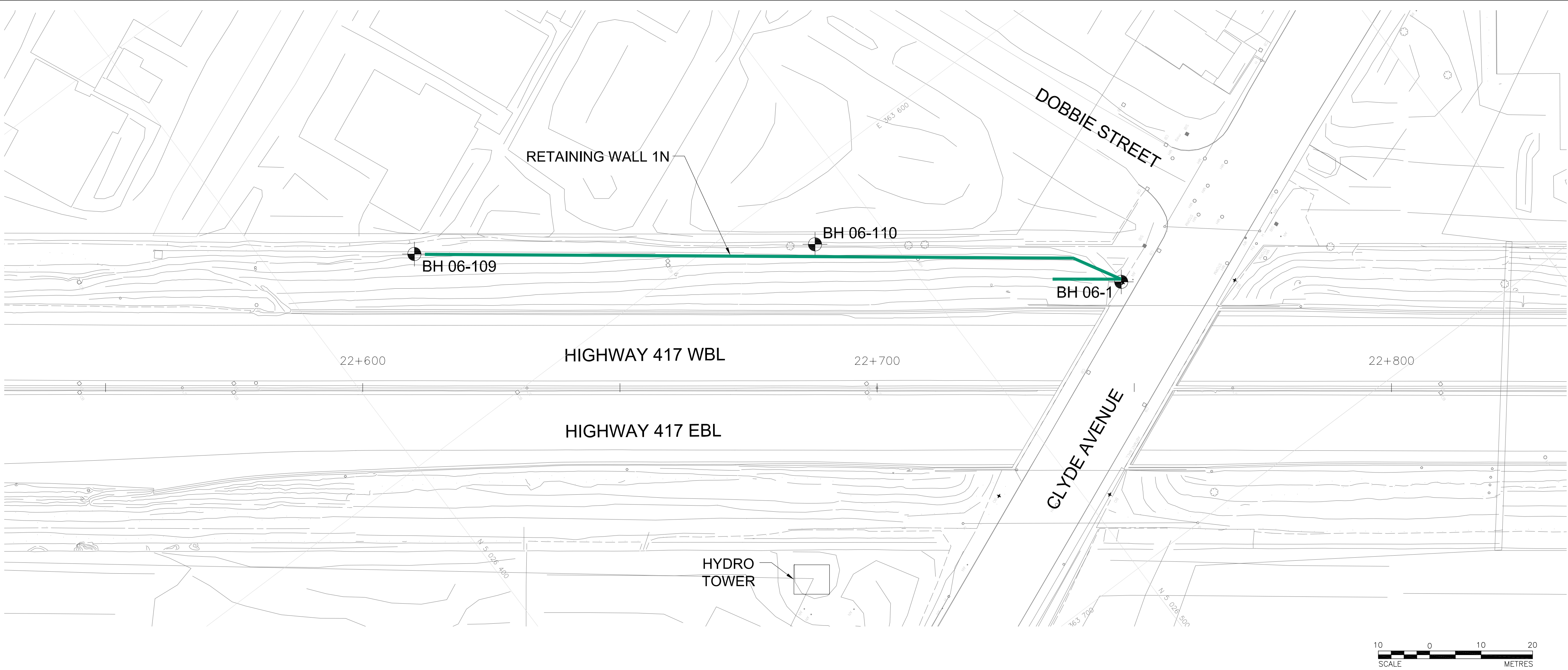
No.	ELEVATION	LOCATION	
		NORTHING	EASTING
06-109	79.2	5026424.4	363566.5
06-110	78.5	5026487.7	363611.9
06-1	77.7	5026530.9	363653.6

NOTES

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Base plan provided in electronic format by McCormick Rankin Corporation

NO.	DATE	BY	REVISION
Geocres No. 31G5-218			
HWY. 417	PROJECT NO. 05-1120-210-2000		DIST.
SUBM'D. W.C.	CHKD. M.I.C.	DATE: OCTOBER 2006	SITE:
DRAWN: J.M.	CHKD. W.C.	APPD.	DWG. 11



PROFILE ALONG RETAINING WALL 1N



METRIC

DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN

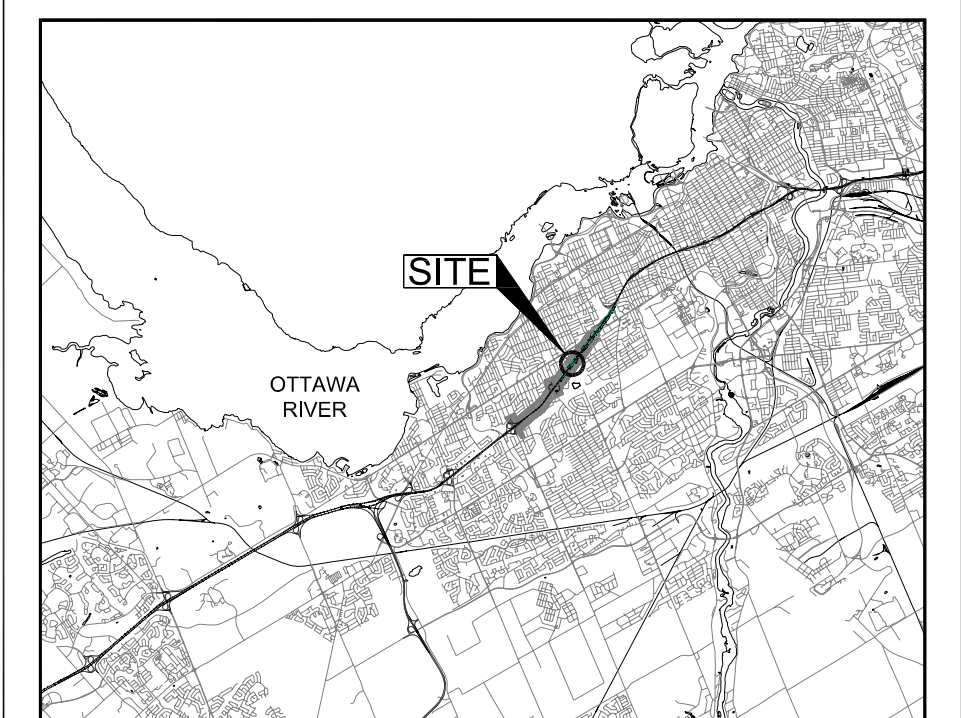
HWY. 417

WP No. WP 4058-01-00

HIGHWAY 417
RETAINING WALL 2N
PLAN AND PROFILE

SHEET

Golder Associates Ltd.
OTTAWA, ONTARIO, CANADA



KEY PLAN

LEGEND

Borehole — Current Golder Associates Ltd. Investigation

Seal

Piezometer

N

Standard Penetration Test value

16

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

100%

Rock Quality Designation (RQD)

WL in piezometer

WL upon completion of drilling

No.	ELEVATION	LOCATION	
		NORTHING	EASTING
06-115	78.6	5026603.3	363701.7
06-116	77.6	5026659.1	363742.4
06-117	77.5	5026711.2	363783.3
06-2	78.1	5026548.7	363666.6

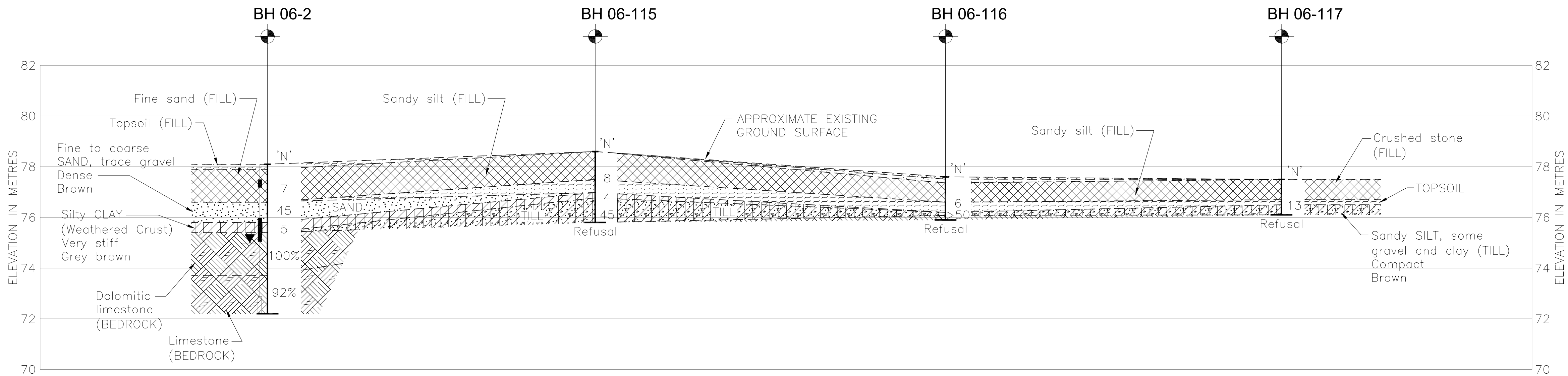
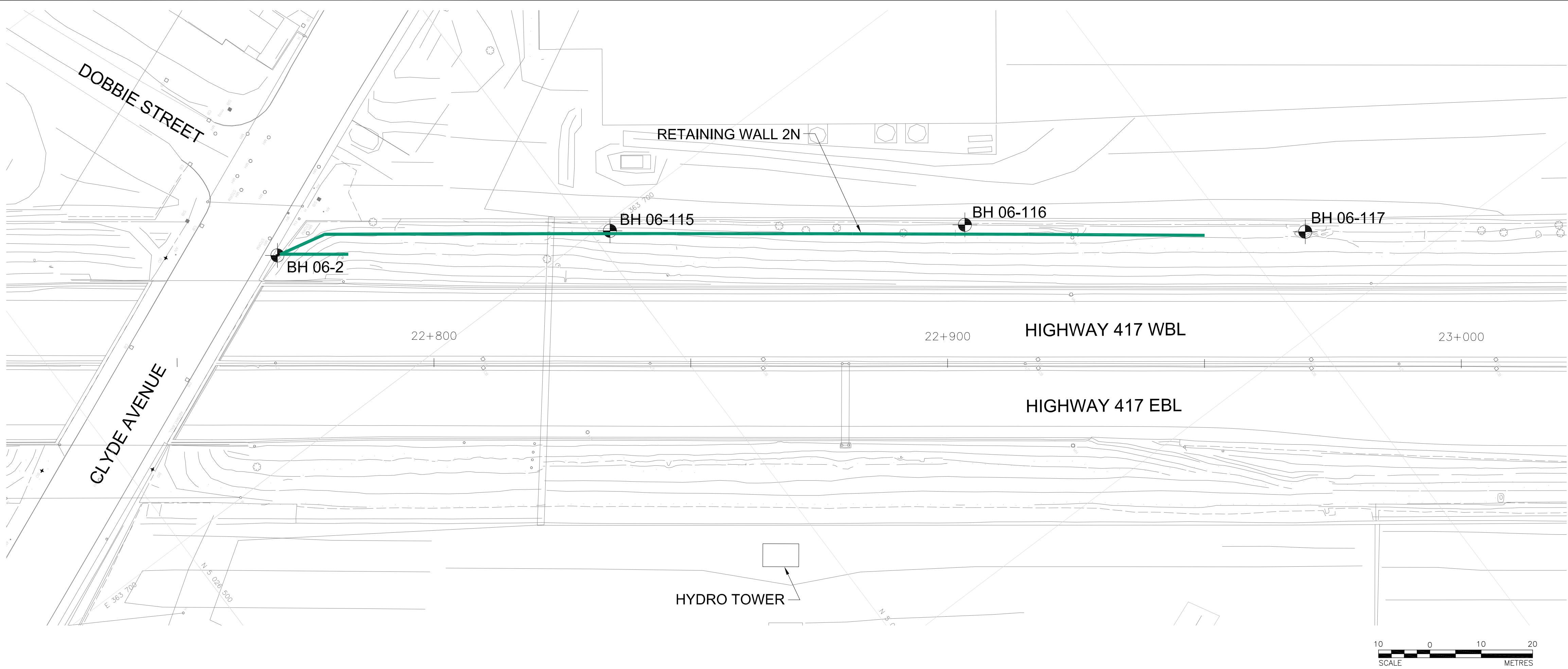
NOTES

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Base plan provided in electronic format by McCormick Rankin Corporation

NO.	DATE	BY	REVISION
Geocres No. 31G5-218			
HWY. 417	PROJECT NO. 05-1120-210-2000		DIST.
SUBM'D. W.C.	CHKD. M.I.C.	DATE: OCTOBER 2006	SITE:
DRAWN: J.M.	CHKD. W.C.	APPD.	DWG. 12



PROFILE ALONG RETAINING WALL 2N

METRIC

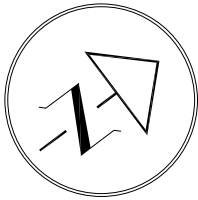
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN

05-1120-210-6000-01.dwg


HWY. 417

WP No. WP 4058-01-00

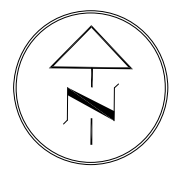
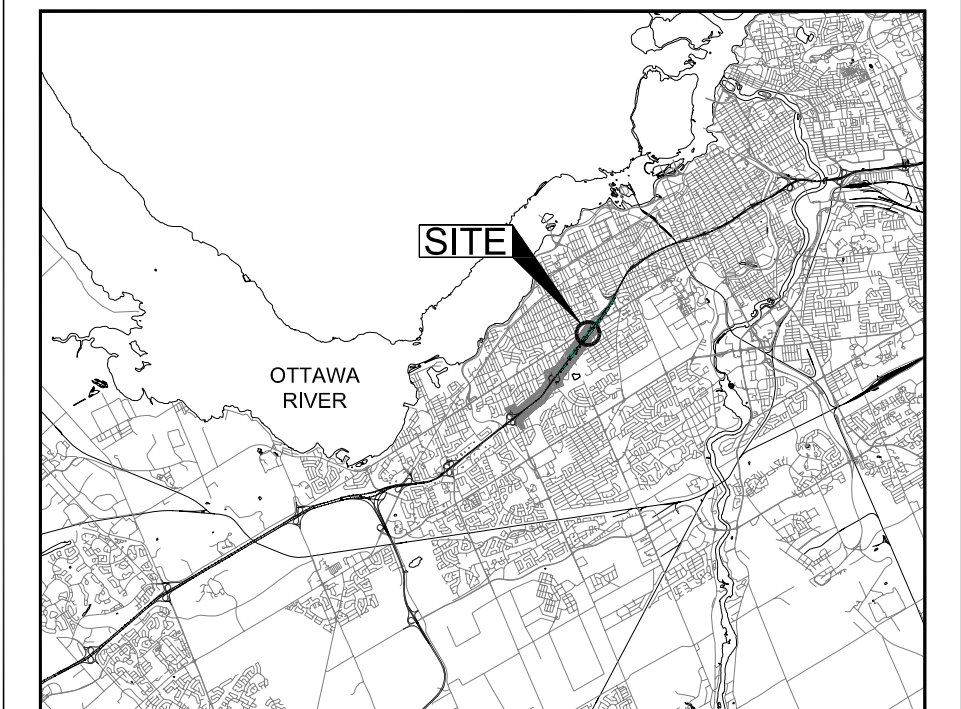
HIGHWAY 417
RETAINING WALL 3N
PLAN AND PROFILE








SHEET

**Golder Associates**

Golder Associates Ltd.
OTTAWA, ONTARIO, CANADA



KEY PLAN

LEGEND			
	Borehole — Current Golder Associates Ltd. Investigation		
	Seal		
	Piezometer		
N	Standard Penetration Test value		
16	Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)		
100%	Rock Quality Designation (RQD)		
	WL in piezometer		
	WL upon completion of drilling		
TABLE DATA			
No.	ELEVATION	LOCATION	
		NORTHING	EASTING
06-123	77.3	5026903.5	363931.2
06-124	76.8	5026959.0	363969.2
06-125	76.9	5027012.6	364010.7
06-126	80.6	5027053.6	364048.1
06-7	75.8	5027104.4	364078.1

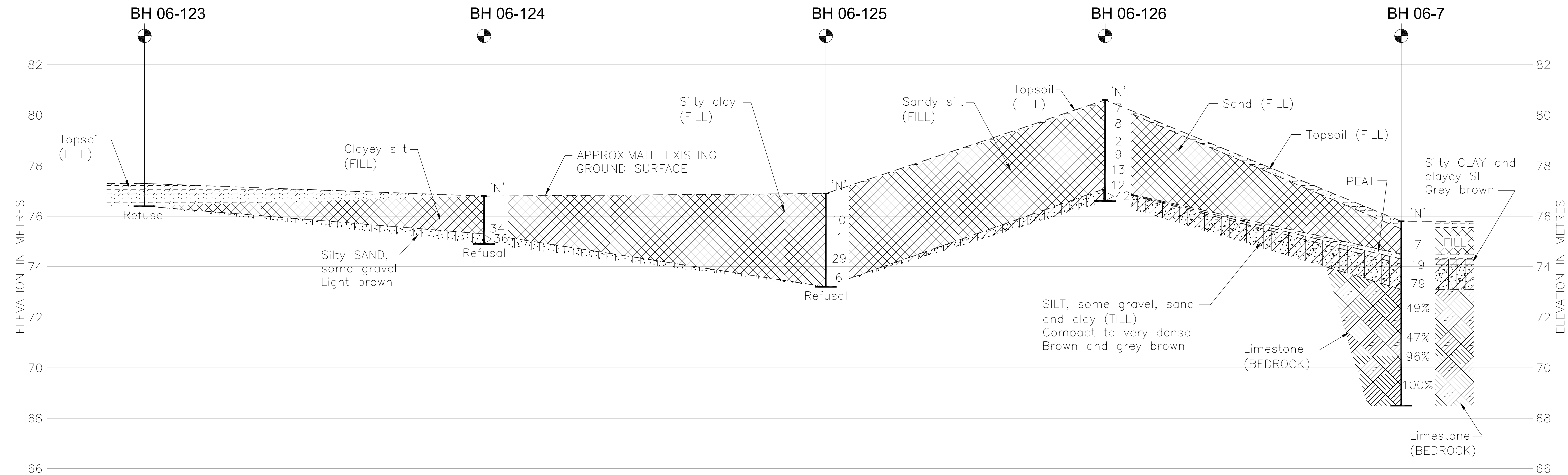
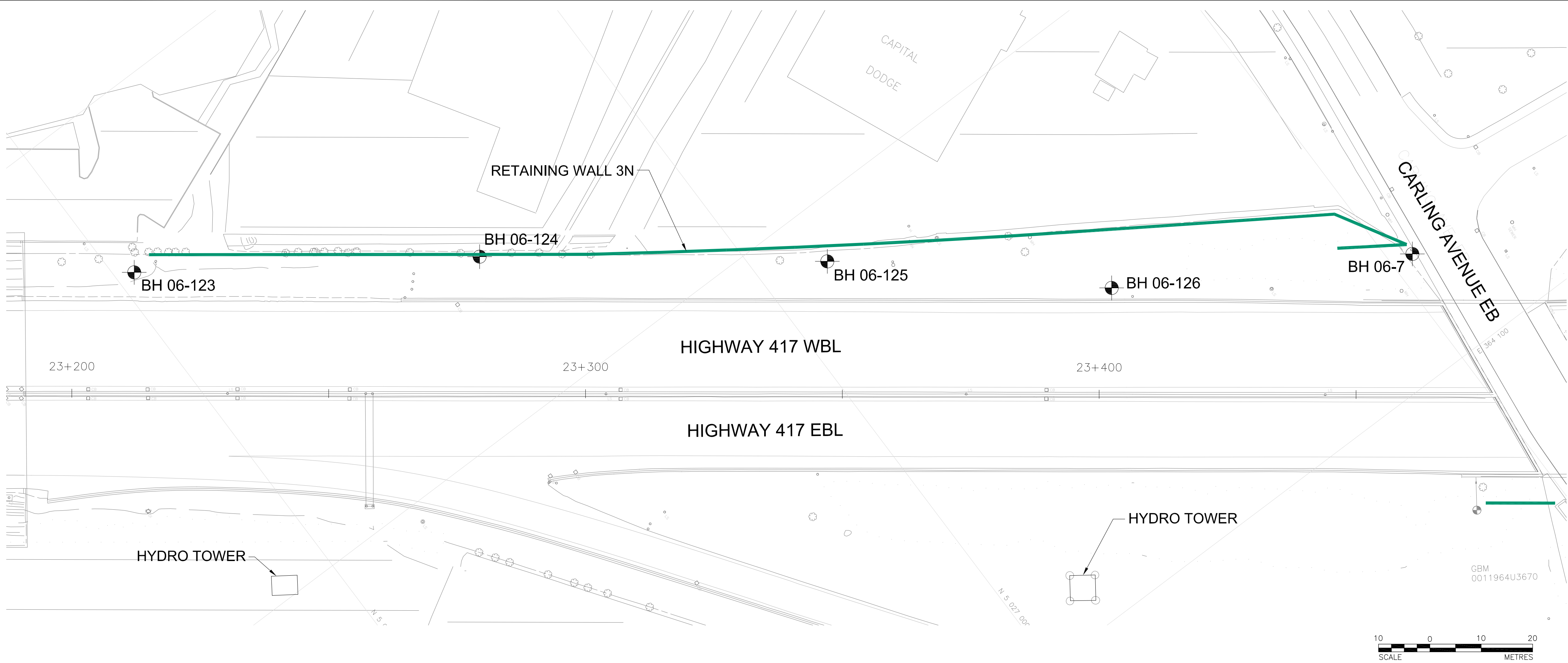
NOTES

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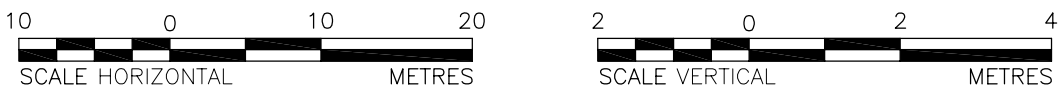
The boundaries between soil strata have been established only at borehole locations. Between Boreholes the boundaries are assumed from geological evidence.

Base plan provided in electronic format by McCormick Rankin Corporation

NO.	DATE	BY	REVISION
Geocres No. 31G5-218			
HWY. 417	PROJECT NO. 05-1120-210-2000		DIST.
SUBM'D. W.C.	CHKD. M.I.C.	DATE: OCTOBER 2006	SITE:
DRAWN: J.M.	CHKD. W.C.	APPD.	DWG. 13



PROFILE ALONG RETAINING WALL 3N



METRIC

DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN

HWY. 417

WP No. WP 4058-01-00

HIGHWAY 417

RETAINING WALL 4N

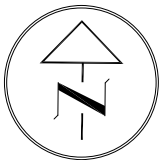
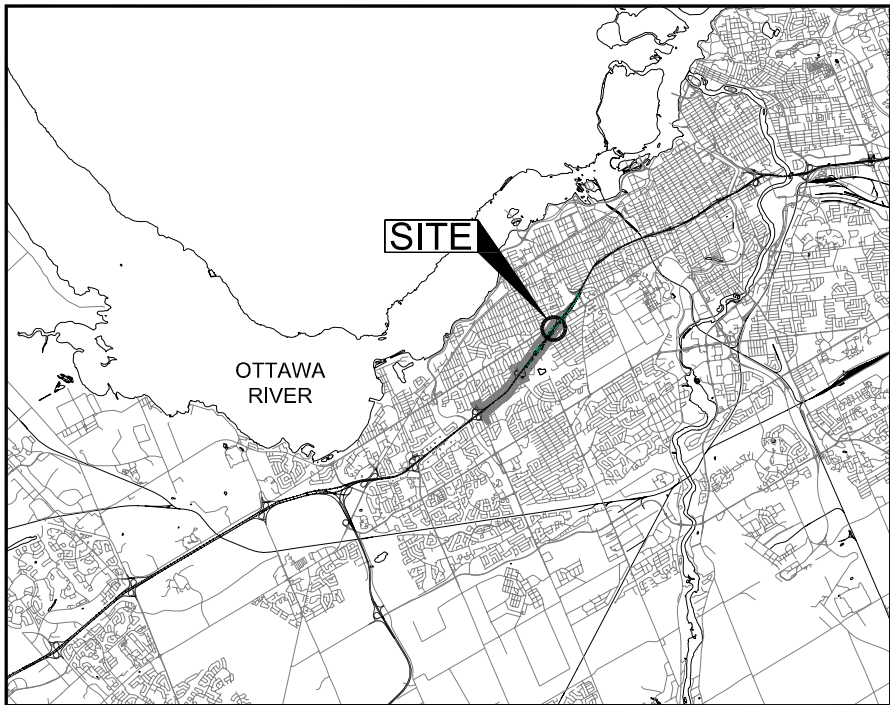
PLAN AND PROFILE




Golder Associates Ltd.
OTTAWA, ONTARIO, CANADA





SHEET



KEY PLAN

- LEGEND
-  Borehole – Current Golder Associates Ltd. Investigation

 Seal

 Piezometer

N


Standard Penetration Test value


16

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

100%

Rock Quality Designation (RQD)

 WL in piezometer

 WL upon completion of drilling

No.	ELEVATION	LOCATION	
		NORTHING	EASTING
06-130	79.0	5027152.5	364110.2
06-5	75.6	5027120.8	364090.8

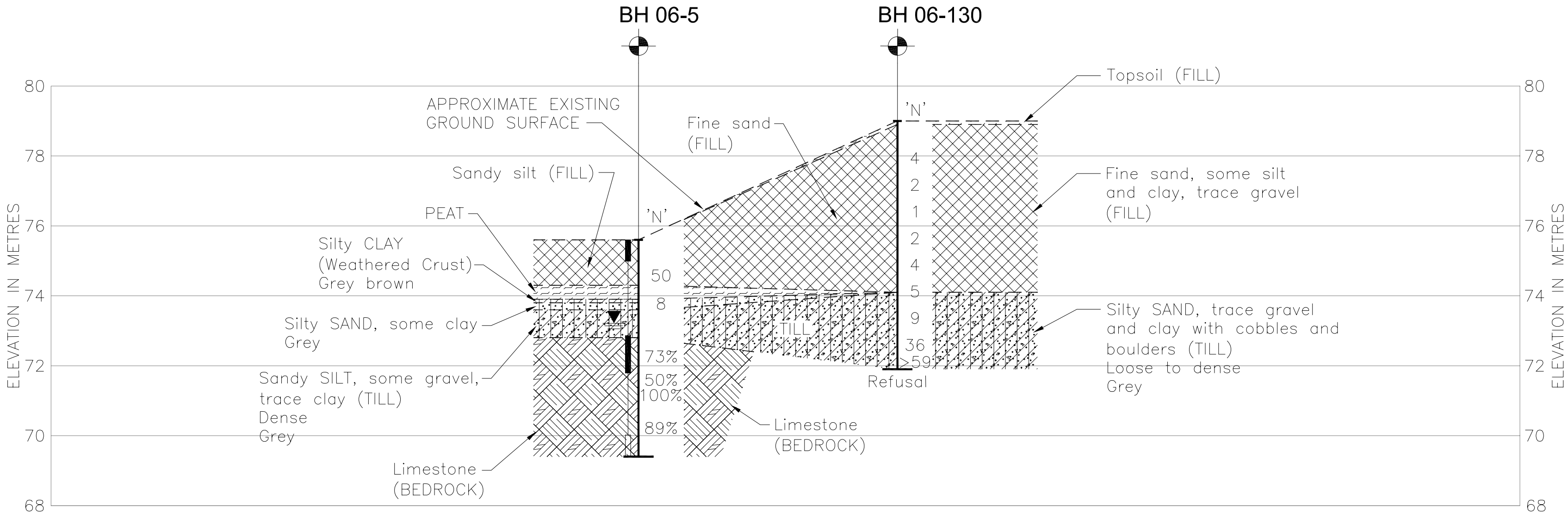
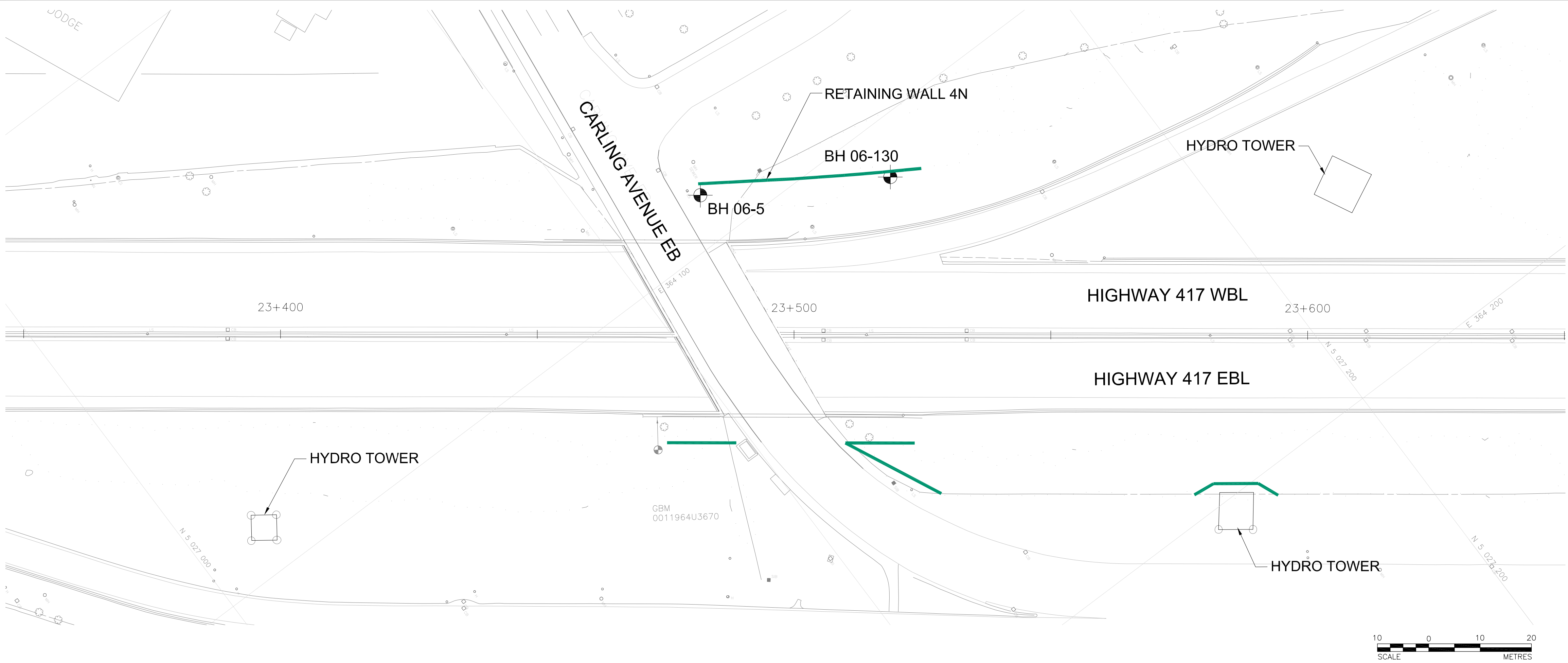
NOTES

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Base plan provided in electronic format by McCormick Rankin Corporation

NO.	DATE	BY	REVISION	
Geocres No. 31G5-218				
HWY. 417			PROJECT NO. 05-1120-210-2000	DIST.
SUBM'D. W.C.		CHKD. M.I.C.	DATE: OCTOBER 2006	SITE:
DRAWN: J.M.		CHKD. W.C.	APPD.	DWG. 14

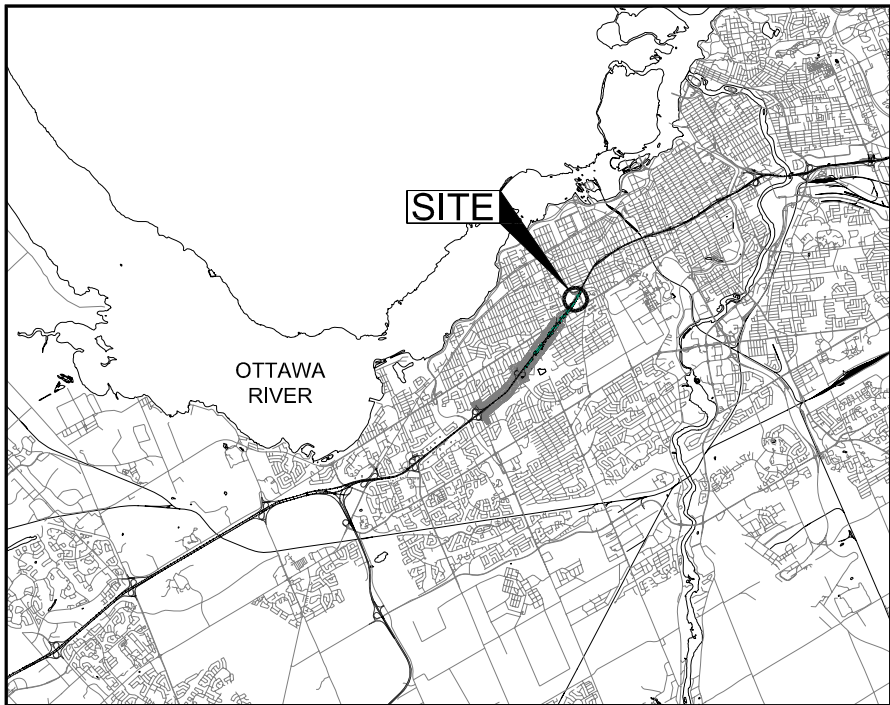


PROFILE ALONG RETAINING WALL 4N








METRIC

DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN



KEY PLAN

LEGEND

-  Borehole – Current Golder Associates Ltd. Investigation
-  Seal
-  Piezometer
- N Standard Penetration Test value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- 100% Rock Quality Designation (RQD)
-  WL in piezometer
-  WL upon completion of drilling

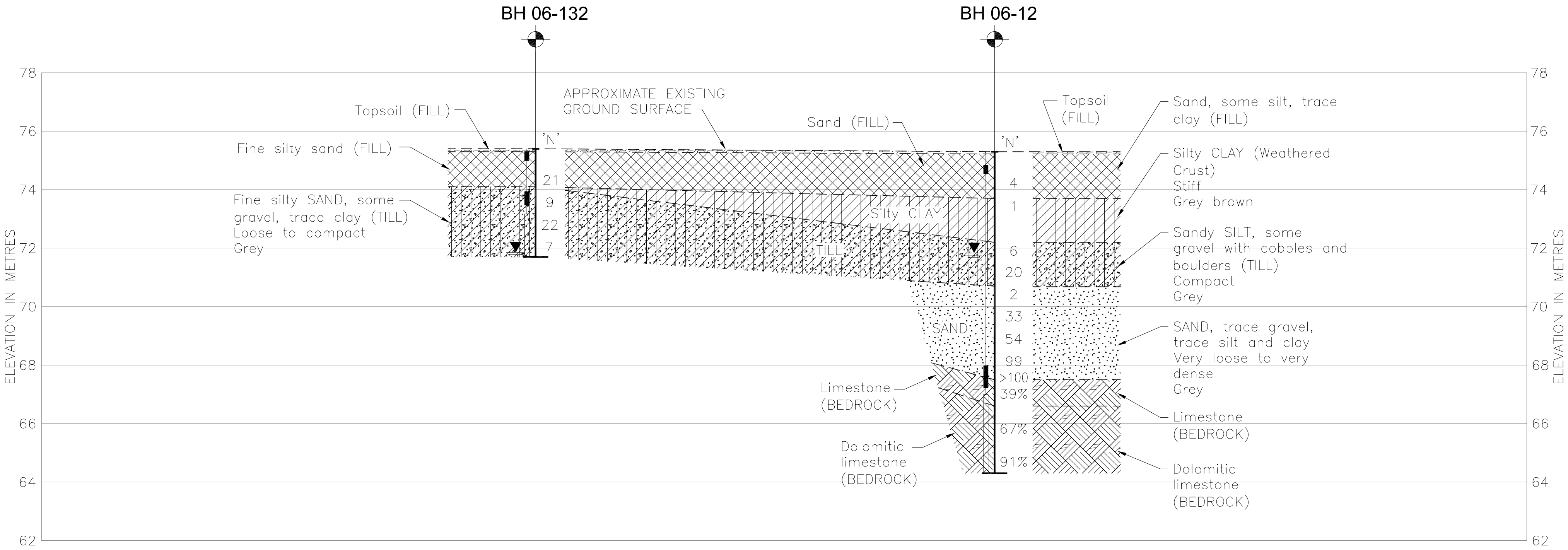
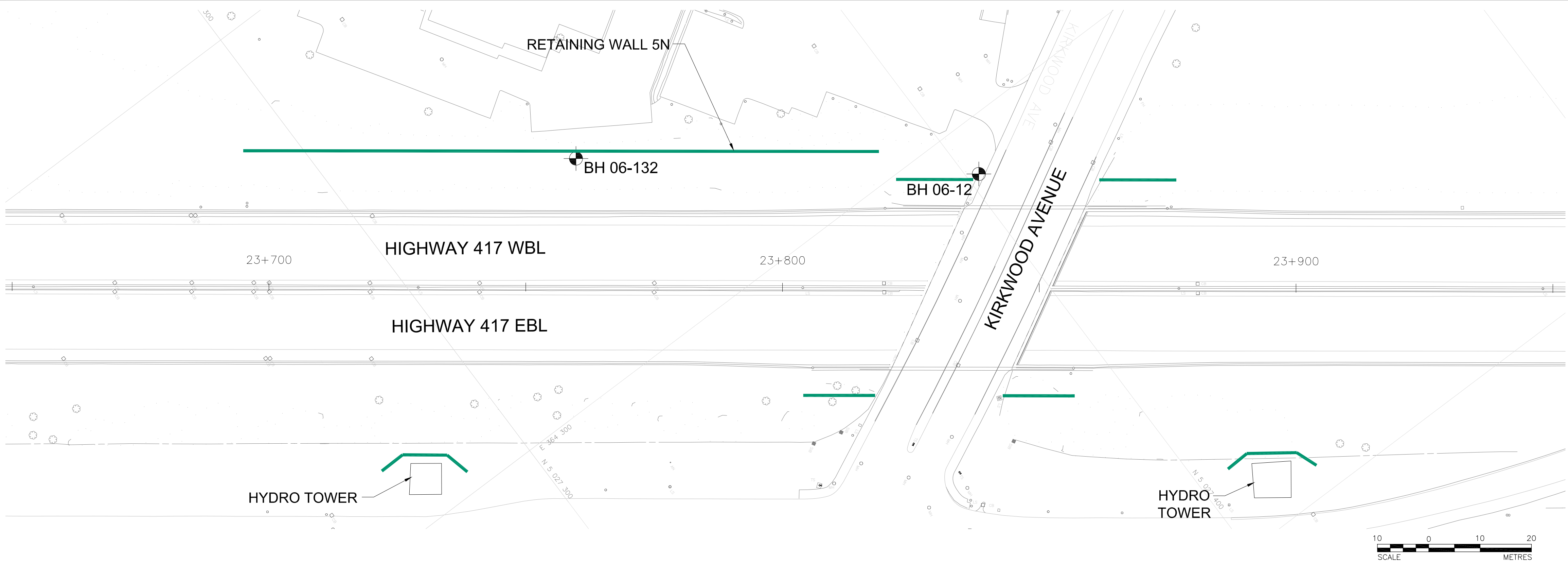
No.	ELEVATION	LOCATION	
		NORTHING	EASTING
06-132	75.4	5027341.4	364260.1
06-12	75.3	5027402.2	364309.7

NOTES

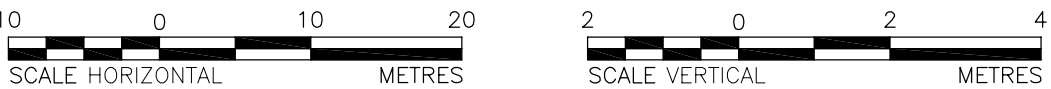
This drawing is for subsurface information only. Any surface details are for conceptual illustration. The boundaries between soil strata have been established only at borehole locations. Between Boreholes the boundaries are assumed from geological evidence.

Base plan provided in electronic format by McCormick Rankin Corporation

NO.	DATE	BY	REVISION
Geocres No. 31G5-218			
HWY. 417	PROJECT NO. 05-1120-210-2000		DIST.
SUBM'D. W.C.	CHKD. M.I.C.	DATE: OCTOBER 2006	SITE:
DRAWN: J.M.	CHKD. W.C.	APPD.	DWG. 15



PROFILE ALONG RETAINING WALL 5N



METRIC
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN

HWY. 417

WP No. WP 4058-01-00

HIGHWAY 417

RETAINING WALL 6N

PLAN AND PROFILE



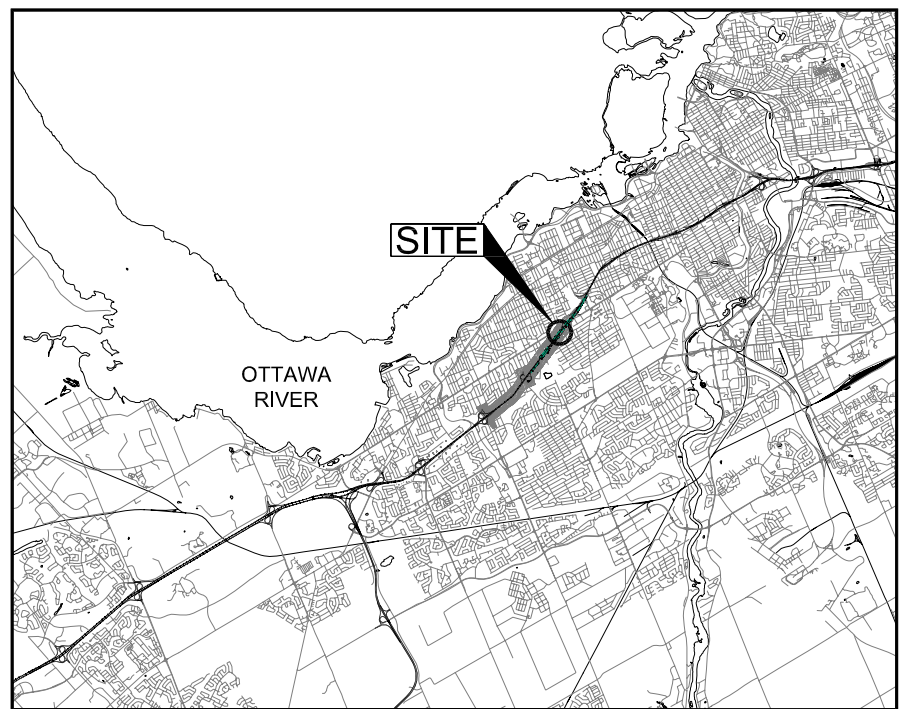
SHEET








Golder Associates

Golder Associates Ltd.

OTTAWA, ONTARIO, CANADA



KEY PLAN

- LEGEND
-  Borehole – Current Golder Associates Ltd. Investigation
 -  Seal
 -  Piezometer
 - N Standard Penetration Test value
 - 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
 - 100% Rock Quality Designation (RQD)
 -  WL in piezometer
 -  WL upon completion of drilling

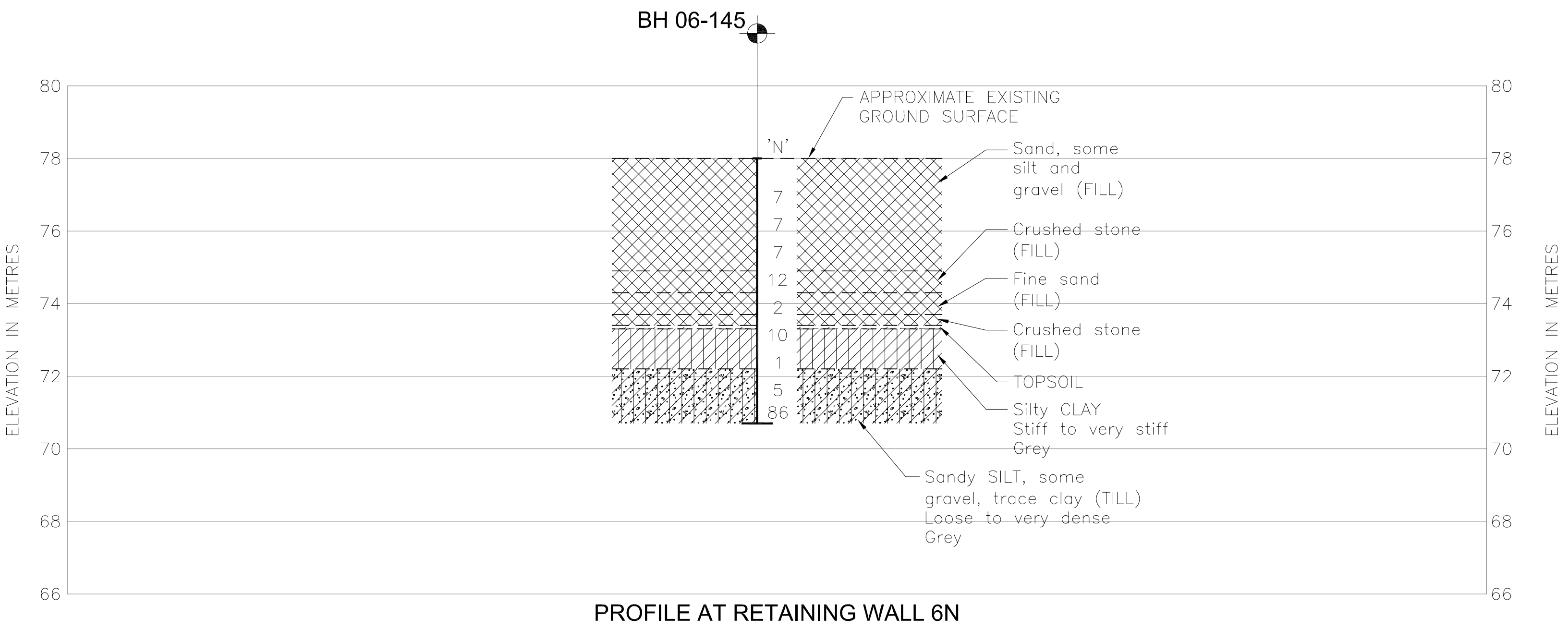
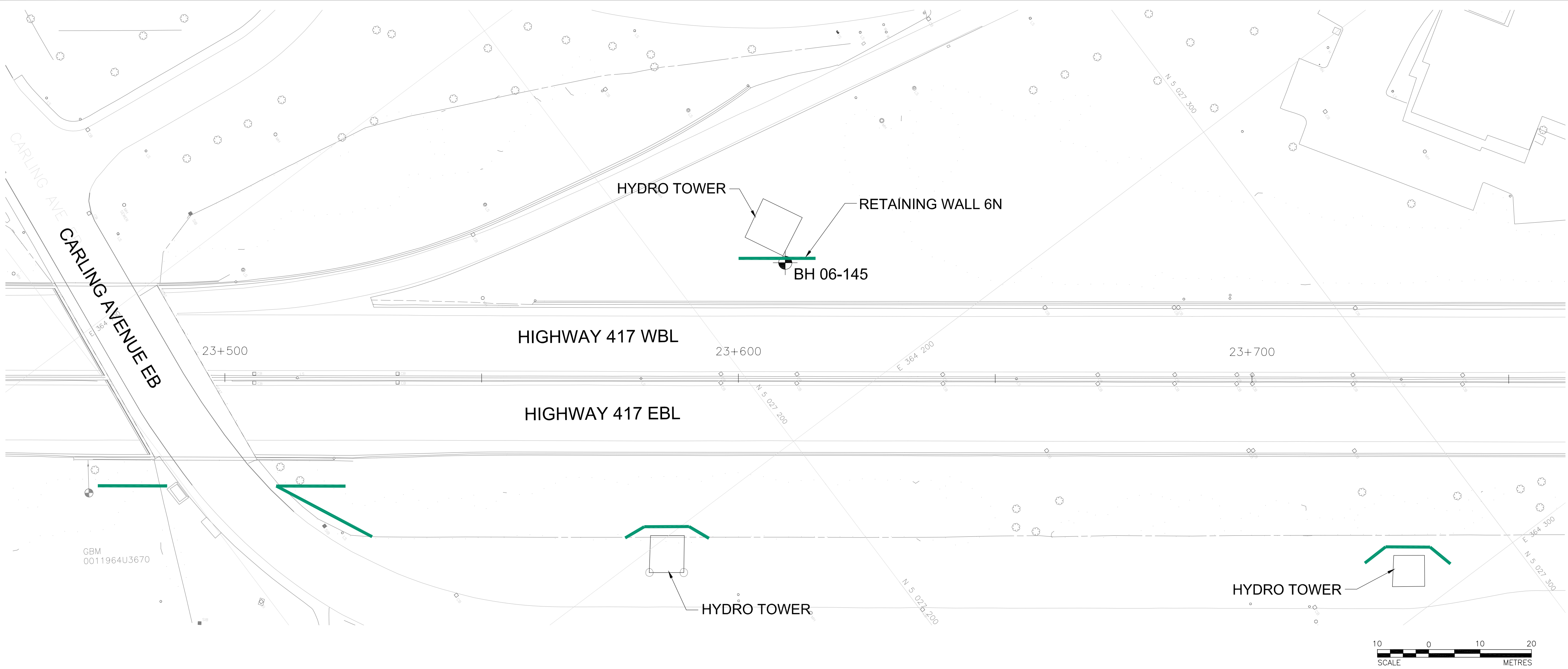
No.	ELEVATION	LOCATION	
		NORTHING	EASTING
06-145	78.0	5027219.6	364171.3

NOTES

This drawing is for subsurface information only. Any surface details are for conceptual illustration. The boundaries between soil strata have been established only at borehole locations. Between Boreholes the boundaries are assumed from geological evidence.

Base plan provided in electronic format by McCormick Rankin Corporation

NO.	DATE	BY	REVISION	
Geocres No. 31G5-218				
HWY. 417			PROJECT NO. 05-1120-210-2000	DIST.
SUBM'D. W.C.		CHKD. M.I.C.	DATE: OCTOBER 2006	SITE:
DRAWN: J.M.		CHKD. W.C.	APPD.	DWG. 16



METRIC

DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN

APPENDIX B
RECORD OF BOREHOLE SHEETS

LIST OF ABBREVIATIONS

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

I. SAMPLE TYPE

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

II. PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:

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

Dynamic Penetration Resistance; N_d :

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

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

Peizo-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 (PWP) and friction along a sleeve are recorded Electronically at 25 mm penetration intervals.

III. SOIL DESCRIPTION

(a)

Cohesionless Soils

Density Index (Relative Density)

Very loose
Loose
Compact
Dense
Very dense

N
Blows/300 mm
Or Blows/ft.

0 to 4
4 to 10
10 to 30
30 to 50
over 50

(b)

Consistency

Very soft
Soft
Firm
Stiff
Very stiff
Hard

Kpa

0 to 12
12 to 25
25 to 50
50 to 100
100 to 200
Over 200

Cohesive Soils

C_{u2S_u}

Psf

0 to 250
250 to 500
500 to 1,000
1,000 to 2,000
2,000 to 4,000
Over 4,000

IV. SOIL TESTS

w	water content
w_p	plastic limited
w_l	liquid limit
C	consolidaiton (oedometer) test
CHEM	chemical analysis (refer to text)
CID	consolidated isotropically drained triaxial test ¹
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement ¹
D_R	relative density (specific gravity, G_s)
DS	direct shear test
M	sieve analysis for particle size
MH	combined sieve and hydrometer (H) analysis
MPC	modified Proctor compaction test
SPC	standard Proctor compaction test
OC	organic content test
SO_4	concentration of water-soluble sulphates
UC	unconfined compression test
UU	unconsolidated undrained triaxial test
V	field vane 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
F	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 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 x acceleration due to gravity)

(a) Index Properties (cont'd.)

w	water content
w_L	liquid limit
w_p	plastic limit
I_p	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_a	coefficient of secondary consolidation
m_v	coefficient of volume change
c_v	coefficient of consolidation
T_v	time factor (vertical direction)
U	degree of consolidation
σ'_p	pre-consolidation pressure
OCR	Overconsolidation ratio = σ'_p/σ'_{vo}

(d) Shear Strength

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

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

2. Shear strength = (Compressive strength)/2

LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY

WEATHERING STATE

Fresh: no visible sign of weathering

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

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

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

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

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

BEDDING THICKNESS

<u>Description</u>	<u>Bedding Plane Spacing</u>
Very thickly bedded	>2 m
Thickly bedded	0.6 m to 2m
Medium bedded	0.2 m to 0.6 m
Thinly bedded	60 mm to 0.2 m
Very thinly bedded	20 mm to 60 mm
Laminated	6 mm to 20 mm
Thinly laminated	<6 mm

JOINT OR FOLIATION SPACING

<u>Description</u>	<u>Spacing</u>
Very wide	>3 m
Wide	1 – 3 m
Moderately close	0.3 – 1 m
Close	50 – 300 mm
Very close	<50 mm

GRAIN SIZE

<u>Term</u>	<u>Size*</u>
Very Coarse Grained	>60 mm
Coarse Grained	2 – 60 mm
Medium Grained	60 microns - 2mm
Fine Grained	2 – 60 microns
Very Fine Grained	<2 microns

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

O:\Templates\Rock Description Terminology

CORE CONDITION

Total Core Recovery

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

Solid Core Recovery (SCR)

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

Rock Quality Designation (RQD)

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

DISCONTINUITY DATA

Fracture Index

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

Dip with Respect to (W.R.T.) Core Axis

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

Description and Notes

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

Abbreviations

B –	Bedding	Ca-	Calcite
FO-	Foliation/Schistosity	P-	Polished
CL -	Cleavage	S-	Slickensided
SH -	Shear Plane/Zone	SM-	Smooth
VN-	Vein	R-	Ridged/Rough
F -	Fault	ST-	Stepped
CO-	Contact	PL-	Planar
J -	Joint	FL-	Flexured
FR-	Fracture	UE-	Uneven
MF -	Mechanical	W-	Wavy
A-	Angular	C-	Curved
BP-	Bedding Plane	H-	Hackly
BL-	Blast Induced	SL-	Sludge Coated
	Parallel To	TCA-	To Core Axis
	Perpendicular To	STR-	Stress Induced

PROJECT 05-1120-210-2000		RECORD OF BOREHOLE No 06-1		1 OF 1	METRIC
W.P. 4058-01-00		LOCATION N 5026530.9; E 363653.6		ORIGINATED BY D.G.	
DIST HWY 417		BOREHOLE TYPE Power Auger 108 mm I.D. Hollow Stem Auger		COMPILED BY J.M.	
DATUM Geodetic		DATE May 9, 2006		CHECKED BY M.I.C.	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC NATURAL LIQUID LIMIT MOISTURE 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					w _p	w	w _L		
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x REMOULDED									
77.7	GROUND SURFACE																
0.0 77.5	Topsoil (FILL)																
0.2	Fine sand, trace silt (FILL) Brown Moist		1	A.S.													
76.9																	
0.8	Silty clay, some sand (FILL) Grey brown Moist		2	SS	8												
76.0																	
1.7	Silty SAND, trace gravel and clay (TILL) Loose Grey brown Moist		3	SS	8												
75.5																	
2.2	Dolomitic limestone (BEDROCK) Fresh Thinly to medium bedded Grey Medium strong		4	NQ RC	DD												
74.0			5	NQ RC	DD												
3.8	Limestone (BEDROCK) Fresh Thinly to medium bedded Grey Medium strong Bedrock cored between 2.2m 5.5m depth. For bedrock coring details refer to Record of Drillhole 06-1.																
			6	NQ RC	DD												
72.2																	
5.5	End of Borehole Note: Borehole dry upon completion of drilling.																

PROJECT: 05-1120-210-2000

RECORD OF DRILLHOLE: 06-1

SHEET 1 OF 1

LOCATION: N 5026530.9; E 363653.5


DRILLING DATE: May 9, 2006

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: —

DRILL RIG: CME 75

DRILLING CONTRACTOR: Marathon Drilling

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	PENETRATION RATE (mm/min)	FLUSH	COLOR	% RETURN	FR/FX-FRACTURE F-FAULT CL-CLEAVAGE J-JOINT SH-SHEAR P-POLISHED VN-VEIN S-SLICKENSIDED PL-PLANAR										SM-SMOOTH R-ROUGH ST-STEPPED C-CURVED				FL-FLEXURED UE-UNEVEN W-WAVY				BC-BROKEN CORE MB-MECH. BREAK B-BEDDING				DIAMETRAL POINT LOAD INDEX (MPa)	NOTES WATER LEVELS INSTRUMENTATION
										RECOVERY		R.Q.D. %	FRACT. INDEX PER 0.3	DISCONTINUITY DATA		HYDRAULIC CONDUCTIVITY																	
										TOTAL CORE %	SOLID CORE %			DIP w.r.t. CORE AXIS	TYPE AND SURFACE DESCRIPTION	10 ⁻⁴	10 ⁻³																
										0 20 40 60 80 100	0 20 40 60 80 100			0 30 60 90	0 30 60 90	0 10 ⁻⁴ 10 ⁻³ 10 ⁻² 10 ⁻¹ 10 ⁰	0 10 ⁻⁴ 10 ⁻³ 10 ⁻² 10 ⁻¹ 10 ⁰																
3		ROCK SURFACE		75.25 2.20																													
	Rotary Drill No Core	Dolomitic limestone (BEDROCK) Fresh Thinly to medium bedded Grey Medium strong			1																												
4		Limestone (BEDROCK) Fresh Thinly to medium bedded Grey Medium strong																															
5																																	
6		End of Drillhole		71.95 5.50																													
7																																	
8																																	
9																																	
10																																	
11																																	
12																																	

PROJECT		05-1120-210-2000		RECORD OF BOREHOLE No 06-2		1 OF 1		METRIC					
W.P.		4058-01-00		LOCATION		N 5026548.7; E 363666.6		ORIGINATED BY					
DIST		HWY 417		BOREHOLE TYPE		Power Auger 108 mm I.D. Hollow Stem Auger		COMPILED BY					
DATUM		Geodetic		DATE		May 9, 2006		CHECKED BY					
								M.I.C.					
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa					
78.1	GROUND SURFACE												
0.0 77.9	Topsoil (FILL)		1	A.S.									
0.2	Fine sand, trace silt (FILL) Brown Moist												
77.2 0.9	Silty clay, some sand, trace gravel (FILL) Grey brown Moist		2/3	SS	7								
76.5 1.5	Fine to coarse SAND, trace gravel Dense Brown Moist		4	SS	45								
75.8 2.3	Silty CLAY (Weathered Crust) Very stiff Grey brown Moist		5/6	SS	5								
75.3 2.8	Dolomitic limestone (BEDROCK) Fresh Thinly to medium bedded Grey Medium strong		7	NQ RC	DD								
73.7 4.4	Limestone (BEDROCK) Fresh Thinly to medium bedded Grey Medium strong Bedrock cored between 2.8m 5.8m depth. For bedrock coring details refer to Record of Drilling 06-2.		8	NQ RC	DD								
72.2 5.8	End of Borehole Note: Water level in standpipe at 3.1m depth below ground surface on June 12, 2006												

MISS_MTO_05-1120-210-1000.GPJ ON MOT_GDT 4/10/07

PROJECT: 05-1120-210-2000

RECORD OF DRILLHOLE: 06-2

SHEET 1 OF 1

LOCATION: N 5026548.7; E 363666.6

DRILLING DATE: May 9, 2006

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: —

DRILL RIG: CME 75

DRILLING CONTRACTOR: Marathon Drilling

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	PENETRATION RATE (mm/min)	FLUSH	RECOVERY % RETURN	FR/FX-FRACTURE F-FAULT CL-CLEAVAGE J-JOINT SH-SHEAR P-POLISHED VN-VEIN S-SLICKENSIDED PL-PLANAR	SM-SMOOTH R-ROUGH ST-STEPPED PL-PLANAR	FL-FLEXURED UE-UNEVEN W-WAVY C-CURVED	BC-BROKEN CORE MB-MECH. BREAK B-BEDDING	HYDRAULIC CONDUCTIVITY K _v cm/sec	DIP w.r.t. CORE AXIS	TYPE AND SURFACE DESCRIPTION	NOTES WATER LEVELS INSTRUMENTATION
3	Relay Drill No Core	ROCK SURFACE		75.26												
		Dolomitic limestone (BEDROCK) Fresh Thinly to medium bedded Grey Medium strong		2.80												
4					1											
5		Limestone (BEDROCK) Fresh Thinly to medium bedded Grey Medium strong		73.66 4.40												
6		End of Drillhole		72.26 5.80												
7																
8																
9																
10																
11																
12																

DEPTH SCALE

1 : 50



LOGGED: D.G.

CHECKED: W.C.

MIS-RCK 001 05-1120-210-1000-ROCK.GPJ GLDR CAN.GDT 4/10/07

PROJECT 05-1120-210-2000			RECORD OF BOREHOLE No 06-3			1 OF 1			METRIC								
W.P. 4058-01-00			LOCATION N 5026503.3; E 363683.4			ORIGINATED BY D.G.											
DIST HWY 417			BOREHOLE TYPE Power Auger 108 mm I.D. Hollow Stem Auger			COMPILED BY J.M.											
DATUM Geodetic			DATE May 8, 2006			CHECKED BY M.I.C.											
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC NATURAL LIQUID LIMIT MOISTURE CONTENT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED			WATER CONTENT (%) w _p — w — w _L			γ	GR SA SI CL		
77.8	GROUND SURFACE							20 40 60 80 100									
0.0	Topsoil (FILL)																
0.2	Silty sand (FILL) Grey brown																
77.2																	
0.6	Fine sand, trace silt, gravel and clay (FILL) Brown																
76.7	Moist		1/2	SS	10		77									3	89 5 3
1.1	Sandy SILT, some gravel, trace clay (TILL) Grey																
	Moist																
76.0			3	SS	>100		76										
1.8	Limestone (BEDROCK) Fresh Thinly to medium bedded Grey Medium strong		4	NQ RC	DD		75										
	Bedrock cored between 1.8m 4.8m depth. For bedrock coring details refer to Record of Drillhole 06-3.																
			5	NQ RC	DD		74										
			6	NQ RC	DD		73										
73.0																	
4.8	End of Borehole Note: Borehole dry prior to commencing coring operations.																

PROJECT: 05-1120-210-2000

RECORD OF DRILLHOLE: 06-3

SHEET 1 OF 1

LOCATION: N 5026503.3; E 363683.4

DRILLING DATE: May 8, 2006

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: CME 75

DRILLING CONTRACTOR: Marathon Drilling

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	PENETRATION RATE (mm/min)	FLUSH	COLOUR % RETURN	FR/FX-FRACTURE F-FAULT CL-CLEAVAGE J-JOINT SM-SMOOTH SH-SHEAR P-POLISHED R-ROUGH FL-FLEXURED VN-VEIN S-SLICKENSIDED PL-PLANAR W-WAVY C-CURVED										8C-BROKEN CORE MB-MECH. BREAK B-BEDDING				DIAMETRAL POINT LOAD INDEX (MPa)	NOTES WATER LEVELS INSTRUMENTATION
									RECOVERY		R.Q.D. %	FRACT. INDEX PER 0.3	DISCONTINUITY DATA		HYDRAULIC CONDUCTIVITY K, cm/sec									
									TOTAL CORE %	SOLID CORE %			DIP w.r.t. CORE AXIS	TYPE AND SURFACE DESCRIPTION										
									10 ⁻⁶	10 ⁻⁵			10 ⁻⁴			10 ⁻³								
		ROCK SURFACE		75.99																				
2	Rotary Drill NQ Core	Limestone (BEDROCK) Fresh Thinly to medium bedded Grey Medium strong		1.80	1																			
3				2																				
4				3																				
5		End of Drillhole		72.99 4.80																				
6																								
7																								
8																								
9																								
10																								
11																								

DEPTH SCALE

1:50



LOGGED: D.G.

CHECKED: W.C.

MIS-RCK-001 05-1120-210-1000-ROCK.GPJ GLDR CAN GDT 4/10/07

PROJECT 05-1120-210-2000			RECORD OF BOREHOLE No 06-4			1 OF 1			METRIC								
W.P. 4058-01-00			LOCATION N 5026487.2; E 363672.2			ORIGINATED BY D.G.											
DIST HWY 417			BOREHOLE TYPE Power Auger 108 mm I.D. Hollow Stem Auger			COMPILED BY J.M.											
DATUM Geodetic			DATE May 8, 2006			CHECKED BY M.I.C.											
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC NATURAL LIQUID			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED			WATER CONTENT (%) w _p — w — w _L			γ	GR SA SI CL		
78.4	GROUND SURFACE							20 40 60 80 100									
0.0	Topsoil (FILL)		1	A.S.													
0.2	Moist Fine sand, some silt (FILL) Brown Moist						78										
77.5																	
0.9	Silty CLAY (Weathered Crust) Very stiff Grey brown Moist		2/3	SS	5		77										
76.9																	
1.5	Sandy SILT, some gravel, trace clay (TILL)		4	SS	>100												
76.6	Grey Moist																
1.8	Limestone (BEDROCK) Fresh Thinly to medium bedded Grey Medium strong Bedrock cored between 1.8m 5.3m depth. For bedrock coring details refer to Record of Drillhole 06-4.		5	NQ RC	DD		76										
			6	NQ RC	DD		75										
			7	NQ RC	DD		74										
			8	NQ RC	DD												
73.1																	
5.3	End of Borehole Note: Borehole dry prior to commencing coring operations.																

MISS_MTO 05-1120-210-1000.GPJ ON_MOT.GDT 4/10/07

PROJECT: 05-1120-210-2000

RECORD OF DRILLHOLE: 06-4

SHEET 1 OF 1

LOCATION: N 5026487.2; E 363672.2

DRILLING DATE: May 8, 2006

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: --

DRILL RIG: CME 75

DRILLING CONTRACTOR: Marathon Drilling

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	PENETRATION RATE (mm/min)	FLUSH	RECOVERY	R.Q.D. %	FRACT. INDEX PER 0.3	DISCONTINUITY DATA	HYDRAULIC CONDUCTIVITY K _{cm/sec}	DIA METRAL POINT LOAD INDEX (MPa)	NOTES WATER LEVELS INSTRUMENTATION
		ROCK SURFACE		78.58										
2		Limestone (BEDROCK) Fresh Thinly to medium bedded Grey Medium strong		1.80										
3														
4														
5														
6		End of Drillhole		73.08										
7				5.30										
8														
9														
10														
11														

DEPTH SCALE

1:50



LOGGED: D.G.

CHECKED: W.C.

MIS-ROCK 001 05-1120-210-1000-ROCK GPJ GLDR CAN GDT 4/10/07

PROJECT 05-1120-210-2000			RECORD OF BOREHOLE No 06-5			1 OF 1			METRIC				
W.P. 4058-01-00			LOCATION N 5027120.8; E 364090.8			ORIGINATED BY D.J.S.							
DIST HWY 417			BOREHOLE TYPE Power Auger 108 mm I.D. Hollow Stem Auger			COMPILED BY J.M.							
DATUM Geodetic			DATE May 16, 2006			CHECKED BY M.I.C.							
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT							
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	20 40 60 80 100	PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION (%)
75.6	GROUND SURFACE												
0.0	Crushed stone (FILL)												
75.4	Grey												
0.2	Fine to coarse sand, some silt, trace gravel (FILL)												
74.8	Brown Moist												
0.8	Sandy silt, some gravel, trace peat (FILL)		1	SS	50								
74.3	Dense Grey Moist												
1.3	PEAT												
73.9	Moist												
	Silty CLAY (Weathered Crust)		2	SS	8								
	Grey brown Moist to wet												
2.0	Silty SAND, some clay												
	Grey Moist to wet												
	Sandy SILT, some gravel, trace clay (TILL)		3	NQ RC	DD								
72.8	Dense Grey		4	SS									
2.8	Wet												
	Limestone (BEDROCK)												
	Fresh Thinly to medium bedded		5	NQ RC	DD								
72.0	Grey Medium strong												
3.6	Limestone (BEDROCK)												
	Fractured Thinly to medium bedded		6	NQ RC	DD								
71.6	Grey Medium strong												
4.0	Limestone (BEDROCK)												
	Fresh Thinly to medium bedded		7	NQ RC	DD								
	Grey Medium strong												
	Bedrock cored between 2.8m 6.2m depth. For bedrock coring details refer to Record of Drillhole 06-5.		8	NQ RC	DD								
69.4													
6.2	End of Borehole												
	Note: Water level in standpipe at 2.4m depth below ground surface on June 12, 2006												

MISS MTO 05-1120-210-2000.GPJ ON MOT.GDT 4/10/07

PROJECT: 05-1120-210-2000

RECORD OF DRILLHOLE: 06-5

SHEET 1 OF 1

LOCATION: N 5027120.8; E 364090.8


DRILLING DATE: May 16, 2006

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: --

DRILL RIG: CME 55

DRILLING CONTRACTOR: Marathon Drilling

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN NO.	PENETRATION RATE (mm/min)	FLUSH RETURN	COLOUR % RETURN	FRFX-FRACTURE F-FAULT				SM-SMOOTH				FL-FLEXURED				BC-BROKEN CORE				DIAMETRAL POINT LOAD INDEX (MPa)	NOTES WATER LEVELS INSTRUMENTATION				
									CL-CLEAVAGE				J-JOINT				R-ROUGH				UE-UNEVEN						MB-MECH. BREAK			
									SH-SHEAR				P-POLISHED				ST-STEPPED				W-WAVY						B-BEDDING			
									VN-VEIN				S-SLICKENSIDED				PL-PLANAR				C-CURVED									
									RECOVERY		R.Q.D. %	FRACT. INDEX PER 0.3	DISCONTINUITY DATA		DIP wrt L CORE AXIS	TYPE AND SURFACE DESCRIPTION	HYDRAULIC CONDUCTIVITY K, cm/sec													
									TOTAL CORE %	SOLID CORE %							10 ⁻⁶	10 ⁻⁵	10 ⁻⁴	10 ⁻³										
		ROCK SURFACE		72.80																										
3	Rotary Drill NQ Core	Limestone (BEDROCK) Fresh Thinly to medium bedded Grey Medium strong		2.80																										
4		Limestone (BEDROCK) Fractured Thinly to medium bedded Grey Medium strong		72.00 3.60																										
		Limestone (BEDROCK) Fresh Thinly to medium bedded Grey Medium strong		71.60 4.00	2																									
					3																									
5					4																									
6		End of Drillhole		69.40 6.20																										
7																														
8																														
9																														
10																														
11																														
12																														

DEPTH SCALE

1:50



LOGGED: D.J.S.

CHECKED: W.C.

MIS-ROK 001 05-1120-210-2000-ROCK GPJ GLDR CAN GDT 4/10/07 JM

PROJECT		05-1120-210-2000		RECORD OF BOREHOLE No 06-6		1 OF 1		METRIC									
W.P.		4058-01-00		LOCATION		N 5027111.6; E 364143.8		ORIGINATED BY									
DIST		HWY 417		BOREHOLE TYPE		Power Auger 108 mm I.D. Hollow Stem Auger		COMPILED BY									
DATUM		Geodetic		DATE		May 26, 2006		CHECKED BY									
								M.I.C.									
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	20 40 60 80 100	20 40 60 80 100	W _p	W	W _L	γ	GR	SA	SI	CL
75.1	GROUND SURFACE																
74.8	ASPHALTIC CONCRETE CONCRETE						75										
0.4	Crushed stone (BASE) Grey Fine sand, trace silt (FILL) Grey brown Moist																
74.0	PEAT Moist		1/2/3	SS	4		74										
1.2	Sandy GRAVEL, some silt, trace clay, cobbles and boulders (TILL) Compact Grey Moist		4	SS	27												
72.7							73										
2.4	Limestone (BEDROCK) Highly fractured and weathered Thinly to medium bedded Grey Medium strong		5	NQ RC	DD												
71.2							72										
3.9	Limestone (BEDROCK) Fresh Thinly to medium bedded Grey Medium strong		6	NQ RC	DD												
	Bedrock cored between 2.4m 6.9m depth. For bedrock coring details refer to Record of Drillhole 06-6.		7	NQ RC	DD		71										
68.2							70										
6.9	End of Borehole		8	NQ RC	DD		69										
	Note: Borehole dry upon completion of augering, and prior to commencing coring operations.																

PROJECT: 05-1120-210-2000

RECORD OF DRILLHOLE: 06-6

SHEET 1 OF 1

LOCATION: N 5027111.6; E 364143.8

DRILLING DATE: May 26, 2006

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: CME 75

DRILLING CONTRACTOR: Marathon Drilling

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	PENETRATION RATE (mm/min)	FLUSH	COLOUR & RETURN	FR/FX-FRACTURE F-FAULT				SM-SMOOTH				FL-FLEXURED				BC-BROKEN CORE				DIAMETRAL POINT LOAD INDEX (MPa)	NOTES WATER LEVELS INSTRUMENTATION				
									CL-CLEAVAGE				J-JOINT				R-ROUGH				UE-UNEVEN						MB-MECH. BREAK			
									SH-SHEAR				P-POLISHED				ST-STEPPED				W-WAVY						B-BEDDING			
									VN-VEIN				S-SLICKENSIDED				PL-PLANAR				C-CURVED									
RECOVERY		R.O.D. %		FRACT. INDEX PER 0.3		DISCONTINUITY DATA		HYDRAULIC CONDUCTIVITY		TYPE AND SURFACE DESCRIPTION																				
TOTAL CORE %	SOLID CORE %	%	%	PER 0.3	DIP w.r.t CORE AXIS																									
88	88	88	88	88	88																									
3		ROCK SURFACE		72.70																										
		Limestone (BEDROCK)		2.40	1																									
		Highly fractured and weathered																												
		Thinly to medium bedded																												
		Grey																												
		Medium strong																												
					2																									

DEPTH SCALE

1:50



LOGGED: D.G.

CHECKED: W.C.

MIS-ROCK-001 05-1120-210-2000-ROCK.GPJ GLDR CAN GDT 4/10/07 JM

PROJECT		05-1120-210-2000		RECORD OF BOREHOLE No 06-7		1 OF 1		METRIC									
W.P.		4058-01-00		LOCATION		N 5027104.4; E 364078.1		ORIGINATED BY									
DIST		HWY 417		BOREHOLE TYPE		Power Auger 108 mm I.D. Hollow Stem Auger		COMPILED BY									
DATUM		Geodetic		DATE		May 18, 2006		CHECKED BY									
								M.I.C.									
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV	DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	20 40 60 80 100	20 40 60 80 100	25 50 75	W _p W W _L	γ	GR SA SI CL			
75.8	0.0	GROUND SURFACE															
	0.0	Topsoil (FILL)															
	0.3	Fine sand (FILL) Loose Brown Moist		1	SS	7		75									
	74.5																
	74.3	PEAT Moist															
	74.1	Silty CLAY and clayey SILT Grey brown Moist		2	SS	19		74						21 22 38 19			
	1.7	SILT, some gravel, sand and clay (TILL) Compact to very dense Brown and grey brown Moist		3	SS	79											
	73.1																
	2.7	Limestone (BEDROCK) Slightly weathered Thinly to medium bedded Grey						73									
	72.6	Medium strong Limestone with voids (BEDROCK) Highly fractured and weathered, with mud seams		4	NQ RC	DD											
	72.4	Thinly to medium bedded Grey															
	72.1	Medium strong Limestone (BEDROCK) Weathered Thinly to medium bedded Grey						72									
	3.7	Medium strong Limestone with voids (BEDROCK) Highly fractured and weathered, with mud seams		5	NQ RC	DD											
	71.2	Thinly to medium bedded Grey															
	4.6	Medium strong Limestone with occasional thin black shale interbeds (BEDROCK) Fresh Thinly to medium bedded Grey Medium strong		6	NQ RC	DD											
		Bedrock cored between 2.7m 7.3m depth. For bedrock coring details refer to Record of Drillhole 06-7.		7	NQ RC	DD											
	69																
	68.5																
	7.3	End of Borehole															
		Note: Water level in casing at 2.4m depth below ground surface upon completion of coring.															

PROJECT: 05-1120-210-2000

RECORD OF DRILLHOLE: 06-7

SHEET 1 OF 1

LOCATION: N 5027104.4; E 364078.1

DRILLING DATE: May 18, 2006

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: —

DRILL RIG: CME 55

DRILLING CONTRACTOR: Marathon Drilling

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	PENETRATION RATE (mm/min)	FLUSH % RETURN	FR/FX-FRACTURE F-FAULT CL-CLEAVAGE J-JOINT SH-SHEAR P-POLISHED VN-VEIN S-SLICKENSIDED PL-PLANAR					SM-SMOOTH R-ROUGH ST-STEPPED C-CURVED			FL-FLEXURED UE-UNEVEN W-WAVY C-CURVED			8C-BROKEN CORE MB-MECH. BREAK 8-BEDDING			DIAMETRAL POINT LOAD INDEX (MPa)	NOTES WATER LEVELS INSTRUMENTATION							
								RECOVERY		R.Q.D. %	FRACT. INDEX PER 0.3	DISCONTINUITY DATA		DIP w.r.t. CORE AXIS	TYPE AND SURFACE DESCRIPTION	HYDRAULIC CONDUCTIVITY K _f cm/sec														
								TOTAL CORE %	SOLID CORE %			10 ⁻⁵	10 ⁻⁴			10 ⁻³														
								8 8 8 8 8	8 8 8 8 8			8 8 8 8 8	5 5 5 5 5			10 10 10	0 0 0	0 0 0	0 0 0											
3	Rotary Drill N.Q. Core	ROCK SURFACE		73.10																										
		Limestone (BEDROCK)		2.70																										
		Slightly weathered																												
		Thinly to medium bedded																												
		Grey		72.60																										
		Medium strong		3.20																										
		Limestone with voids (BEDROCK)		72.40																										
		Highly fractured and weathered, with		3.40	1																									
		mud seams		72.10																										
		Thinly to medium bedded		3.70																										
		Grey																												
		Medium strong		71.20																										
		Limestone with voids (BEDROCK)		4.60	2																									
		Highly fractured and weathered, with																												
		mud seams																												
		Thinly to medium bedded																												
		Grey																												
		Medium strong																												
		Limestone with occasional thin black																												
		shale interbeds (BEDROCK)																												
		Fresh																												
		Thinly to medium bedded																												
		Grey																												
		Medium strong																												
6																														
7																														
8		End of Drillhole		68.50																										
				7.30																										
8																														
9																														
10																														
11																														
12																														

DEPTH SCALE

1:50



LOGGED: D.J.S.

CHECKED: W.C.

MIS-RCK 001 05-1120-210-2000-ROCK GPJ GLDR CAN.GDT 4/10/07 JM

PROJECT 05-1120-210-2000		RECORD OF BOREHOLE No 06-9		1 OF 2	METRIC
W.P. 4058-01-00	LOCATION N 5027378.2; E 364347.3	ORIGINATED BY D.G.			
DIST HWY 417	BOREHOLE TYPE Power Auger 108 mm I.D. Hollow Stem Auger	COMPILED BY J.M.			
DATUM Geodetic	DATE May 18, 2006	CHECKED BY M.I.C.			


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			20	40						60	80	100	20	40	60	80	100	25	50	75
								SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED																	
75.2	GROUND SURFACE																								
0.0	Topsoil (FILL)																								
0.2	Fine sand, trace silt (FILL) Light brown Moist		1	A.S.																					
74.5																									
0.7	Medium sand, trace gravel (FILL) Compact Brown Moist		2	SS	11																				
73.6																									
73.4	TOPSOIL																								
1.8	Silty CLAY (Weathered Crust) Stiff Grey brown Moist		3/4	SS	4																				
72.8																									
2.4	Silty CLAY Stiff Grey Moist																								
71.9																									
3.3	Sandy SILT, some gravel, trace clay with cobbles and boulders (TILL) Dense Grey Moist		5/6	SS	8																				
70.9																									
4.3	Gravelly SAND, some silt, trace clay with cobbles and boulders Dense Grey Moist to Wet		7	SS	44																				
			8	SS	43														32 52 13 3						
69.6																									
5.6	SAND, some silt, trace clay Very dense Grey Wet		9	SS	36																				
			10	SS	64														0 85 13 2						
68.2																									
7.0	Coarse SAND, some gravel with cobbles and boulders Very dense Grey Wet		11	SS	80																				
			12	SS	84																				
66.6																									
8.6	Dolomitic limestone (BEDROCK) Fresh Thinly to medium bedded Grey Medium strong		13	SS	>100																				
			14	NQ RC	DD																				
			15	NQ RC	DD																				
65.3																									

Continued Next Page

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

MISS_MTO 05-1120-210-3000.GPJ ON_MOT.GDT 4/10/07

PROJECT 05-1120-210-2000		RECORD OF BOREHOLE No 06-9		2 OF 2	METRIC
W.P. 4058-01-00		LOCATION N 5027378.2; E 364347.3		ORIGINATED BY D.G.	
DIST HWY 417		BOREHOLE TYPE Power Auger, 108 mm I.D. Hollow Stem Auger		COMPILED BY J.M.	
DATUM Geodetic		DATE May 18, 2006		CHECKED BY M.I.C.	

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 (%)				
								20 40 60 80 100												
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED												
9.9	Limestone (BEDROCK)		16	NQ RC	DD		65													
64.9	Highly fractured																			
10.3	Thinly to medium bedded Grey Medium strong Limestone with thin shale interbeds (BEDROCK) Fresh Thinly to medium bedded Grey to black Medium strong																			
63.6	Bedrock cored between 8.6m 11.6m depth. For bedrock coring details refer to Record of Drillhole 06-9.																			
11.6	End of Borehole																			
	Note: Water level in open borehole at 4.6m depth below ground surface prior to commencing coring operations																			

PROJECT: 05-1120-210-2000

RECORD OF DRILLHOLE: 06-9

SHEET 1 OF 1

LOCATION: N 5027378.2 E 364347.3

DRILLING DATE: May 18, 2006

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: CME 75

DRILLING CONTRACTOR: Marathon Drilling

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV.		RUN No.	PENETRATION RATE (mm/min)	FLUSH	COLOUR % RETURN	FR/FX-FRACTURE F-FAULT CL-CLEAVAGE J-JOINT SM-SMOOTH SH-SHEAR P-POLISHED R-ROUGH VN-VEIN S-SLICKENSIDED PL-PLANAR UE-UNEVEN W-WAVY C-CURVED										BC-BROKEN CORE		DIAMETRAL POINT LOAD INDEX (MPa)		NOTES WATER LEVELS INSTRUMENTATION
				DEPTH (m)	RECOVERY					R.Q.D. %	FRACT. INDEX PER 0.3	DISCONTINUITY DATA		HYDRAULIC CONDUCTIVITY K _o cm/sec	3	4	5	6						
												TYPE AND SURFACE DESCRIPTION	DIP w.r.t. CORE AXIS											
																			10 ⁻⁸	10 ⁻⁵	10 ⁻¹	10 ⁻³		
		ROCK SURFACE		66.60																				
9	Rotary Drill NQ Core	Dolomitic limestone (BEDROCK) Fresh Thinly to medium bedded Grey black Medium strong		8.60	1																			
					2																			
10		Limestone (BEDROCK) Highly fractured Thinly to medium bedded Grey Medium strong		65.30 9.90																				
			Limestone with thin shale interbeds (BEDROCK) Fresh Thinly to medium bedded Grey to black Medium strong		64.90 10.30																			
11						3																		
		End of Drillhole		63.60 11.60																				
12																								
13																								
14																								
15																								
16																								
17																								
18																								

DEPTH SCALE

1:50



LOGGED: D.G.

CHECKED: W.C.

MIS-RCK 001 05-1120-210-3000-ROCK GPJ GLDR CAN GDT 4/10/07 JM

PROJECT 05-1120-210-2000

RECORD OF BOREHOLE No 06-10

1 OF 2

METRIC

W.P. 4058-01-00

LOCATION N 5027364.2; E 364331.9

ORIGINATED BY D.G.

DIST HWY 417

BOREHOLE TYPE Power Auger 108 mm I.D. Hollow Stem Auger

COMPILED BY J.M.

DATUM Geodetic

DATE May 24, 2006

CHECKED BY M.I.C.

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 x REMOULDED								WATER CONTENT (%)
75.0	GROUND SURFACE							20	40	60	80	100				
74.9	CONCRETE							20	40	60	80	100				
74.8	Crushed stone (BASE)															
74.6	Grey		1	A.S.												
0.4	Moist															
	Fine sand (FILL)															
	Brown															
	Moist															
	Silty clay, some sand, trace gravel (FILL)		2	SS	2		74									
	Very loose															
	Grey brown															
	Moist															
73.3																
1.7	Sandy SILT, some gravel, trace clay (TILL)		3	SS	4		73									
	Loose															
	Grey brown															
	Moist															
72.7																
2.3	Sandy SILT, some gravel, trace clay with cobbles and boulders (TILL)		4	SS	10		72									
	Compact to dense															
	Grey															
	Moist to wet															
71.7																
3.4	SAND to sandy GRAVEL, trace silt and clay		5	SS	56											
	Compact to very dense															
	Grey															
	Wet															
			6	SS	53		71						○			34 58 6 2
			7	SS	20		70									
			8	SS	88											
							69									
			9	SS	52								○			3 82 11 4
			10	SS	>100		68									
67.1																
			11	NQ RC	DD											
7.9	Limestone (BEDROCK)						67									
	Highly fractured															
	Thinly to medium bedded															
	Grey															
	Medium strong															
66.3			12	NQ RC	DD											
8.7	Dolomitic limestone (BEDROCK)															
	Fresh															
	Thinly to medium bedded															
	Grey															
	Medium strong															
65.2			13	NQ RC	DD		66									
9.8																

MISS MTO 05-1120-210-3000.GPJ ON MOT.GDT 12/1/06

Continued Next Page

+³, ×³

Numbers refer to Sensitivity

○ 3% STRAIN AT FAILURE

PROJECT		RECORD OF BOREHOLE				No 06-10		2 OF 2		METRIC			
W.P.		LOCATION		BOREHOLE TYPE		ORIGINATED BY		COMPILED BY		CHECKED BY			
DIST		DATE		BOREHOLE TYPE		ORIGINATED BY		COMPILED BY		CHECKED BY			
DATUM		DATE		BOREHOLE TYPE		ORIGINATED BY		COMPILED BY		CHECKED BY			
PROJECT 05-1120-210-2000		LOCATION N 5027364.2; E 364331.9				ORIGINATED BY D.G.		COMPILED BY J.M.		CHECKED BY M.I.C.			
W.P. 4058-01-00		DIST HWY 417		BOREHOLE TYPE Power Auger 108 mm I.D. Hollow Stem Auger		ORIGINATED BY D.G.		COMPILED BY J.M.		CHECKED BY M.I.C.			
DATUM Geodetic		DATE May 24, 2006		BOREHOLE TYPE Power Auger 108 mm I.D. Hollow Stem Auger		ORIGINATED BY D.G.		COMPILED BY J.M.		CHECKED BY M.I.C.			
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa					
	--- CONTINUED FROM PREVIOUS PAGE ---												
63.9	Limestone with thin shale interbeds (BEDROCK) Fresh Thinly to medium bedded Grey to black Medium strong		14	NQ RC	DD								
11.1	Bedrock cored between 7.9m 11.1m depth. For bedrock coring details refer to Record of Drillhole 06-10. End of Borehole												
	Note: Water level in open borehole at 4.60m depth below ground surface prior to commencing coring operations												

MISS. MTO 05-1120-210-3000.GPJ ON MOT.GDT 12/1/06

PROJECT: 05-1120-210-2000

RECORD OF DRILLHOLE: 06-10

SHEET 1 OF 1

LOCATION: N 5027364.2; E 364331.9

DRILLING DATE: May 24, 2006

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: —

DRILL RIG: CME 75

DRILLING CONTRACTOR: Marathon Drilling

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	PENETRATION RATE (mm/min)	FLUSH	COLOUR % RETURN	FR/FX-FRACTURE F-FAULT CL-CLEAVAGE J-JOINT SM-SMOOTH SH-SHEAR P-POLISHED R-ROUGH VN-VEIN S-SLICKENSIDED PL-PLANAR ST-STEPPED W-WAVY C-CURVED										BC-BROKEN CORE MB-MECH. BREAK B-BEDDING			DIAMETRAL POINT LOAD INDEX (MPa)	NOTES WATER LEVELS INSTRUMENTATION		
									RECOVERY		R.Q.D. %	FRACT. INDEX PER 0.3	DISCONTINUITY DATA		HYDRAULIC CONDUCTIVITY K, cm/sec										
									TOTAL CORE %	SOLID CORE %			TYPE AND SURFACE DESCRIPTION		10 ⁻⁴	10 ⁻³	10 ⁻²								
									10 ⁻⁴	10 ⁻³			10 ⁻²	10 ⁻¹	10 ⁻¹	10 ⁻¹	10 ⁻¹								
8	Rotary Drill NQ Core	ROCK SURFACE		87.10																					
		Limestone (BEDROCK) Highly Fractured Thinly to medium bedded Medium strong		7.90	1																				
						2																			
		Dolomitic limestone (BEDROCK) Fresh Thinly to medium bedded Grey Medium strong		66.30 8.70																					
9					3																				
10		Limestone with thin shale interbeds (BEDROCK) Fresh Thinly to medium bedded Gray to black Medium strong		65.20 9.80																					
11		End of Drillhole		63.90 11.10																					
12																									
13																									
14																									
15																									
16																									
17																									

DEPTH SCALE

1:50



LOGGED: D.G.

CHECKED: W.C.

MIS-RCK 001 05-1120-210-3000-ROCK.GPJ GLDR CAN GDT 12/1/06 JM

PROJECT 05-1120-210-2000		RECORD OF BOREHOLE No 06-12		1 OF 2		METRIC							
W.P. 4058-01-00		LOCATION N 5027402.2; E 364309.7		ORIGINATED BY D.G.									
DIST HWY 417		BOREHOLE TYPE Power Auger 108 mm I.D. Hollow Stem Auger		COMPILED BY J.M.									
DATUM Geodetic		DATE May 19, 2006		CHECKED BY M.I.C.									
SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa					
75.3	GROUND SURFACE						20 40 60 80 100						
0.0	Topsoil (FILL)												
75.0	Sand, some silt, trace clay (FILL)		1	A.S.									
0.3	Grey brown Moist												
	Sand, some silt, trace clay (FILL)												
	Loose Light brown Moist		2	SS	4								0 71 21 8
73.7													
1.6	Silty CLAY (Weathered Crust)												
	Stiff Grey brown Moist		3	SS	1								
72.2													
3.1	Sandy SILT, some gravel with cobbles and boulders (TILL)												
	Loose to compact Grey Moist		4	SS	6								
			5	SS	20								
70.7													
4.6	SAND, trace gravel, trace silt and clay												
	Very loose to very dense Grey Wet		6	SS	2								
			7	SS	33								
			8	SS	54								4 87 7 2
			9	SS	99								
			10	SS	>100								
67.5													
7.8	Limestone (BEDROCK)												
	Slightly fractured Thinly to medium bedded Fresh Grey Medium strong		11	NQ RC	DD								
66.6													
8.7	Dolomitic limestone (BEDROCK)												
	Fresh Thinly to medium bedded Grey Medium strong												
	Bedrock cored between 7.8m 11.0m depth. For bedrock coring details refer to Record of Drillhole 06-12.		12	NQ RC	DD								

Continued Next Page

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

MISS_MTO_05-1120-210-3000.GPJ ON MOT_GDT 4/10/07

PROJECT: 05-1120-210-2000		RECORD OF BOREHOLE No 06-12		2 OF 2	METRIC
W.P. 4058-01-00	LOCATION N 5027402.2; E 364309.7	ORIGINATED BY D.G.			
DIST _____ HWY 417	BOREHOLE TYPE Power Auger 108 mm I.D. Hollow Stem Auger	COMPILED BY J.M.			
DATUM Geodetic	DATE May 19, 2006	CHECKED BY M.I.C.			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC NATURAL LIQUID LIMIT MOISTURE CONTENT 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 (%)					
	— CONTINUED FROM PREVIOUS PAGE —							20	40	60	80	100		W _p	W	W _L		

PROJECT: 05-1120-210-2000

RECORD OF DRILLHOLE: 06-12

SHEET 1 OF 1

LOCATION: N 5027402.2 ; E 364309.7

DRILLING DATE: May 19, 2006

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: —

DRILL RIG: CME 75

DRILLING CONTRACTOR: Marathon Drilling

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	PENETRATION RATE (mm/min)	FLUSH	FRTX-FRACTURE F-FAULT CL-CLEAVAGE J-JOINT SH-SHEAR P-POLISHED VN-VEIN S-SLICKENSIDED PL-PLANAR	SM-SMOOTH R-ROUGH ST-STEPPED PL-PLANAR	PL-FLEXURED UE-UNEVEN W-WAVY C-CURVED	BC-BROKEN CORE MB-MECH. BREAK B-BEDDING	DIAMETRAL POINT LOAD INDEX (MPa)	NOTES WATER LEVELS INSTRUMENTATION
8	Rotary Drill NQ Core	ROCK SURFACE		67.50									
		Limestone (BEDROCK) Slightly fractured Thinly to medium bedded Fresh Grey Medium strong		7.80	1								
9		Dolomitic limestone (BEDROCK) Fresh Thinly to medium bedded Grey Medium strong		66.60	2								
				8.70	2								
10					3								
					3								
11		End of Drillhole		64.30									
				11.00									
12													
13													
14													
15													
16													
17													

DEPTH SCALE

1:50



LOGGED: D.G.

CHECKED: W.C.

MIS-RCK 001 05-1120-210-3000-ROCK.GPJ GLDR CAN.GDT 4/10/07 JM

PROJECT 05-1120-210-2000

RECORD OF BOREHOLE No 06-14

1 OF 2

METRIC

W.P. 4058-01-00

LOCATION N 5027540.4; E 364478.1

ORIGINATED BY D.G.

DIST HWY 417

BOREHOLE TYPE Power Auger 108 mm I.D. Hollow Stem Auger

COMPILED BY J.M.

DATUM Geodetic

DATE May 15, 2006

CHECKED BY M.I.C.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa				
75.2	GROUND SURFACE							20 40 60 80 100				
0.0	Topsoil (FILL)							20 40 60 80 100				
0.2	Fine sand, trace silt (FILL)		1	A.S.			75					
74.7	Brown Moist											
0.5	Sandy silt, some gravel and clay (FILL)		2	SS	3							
	Very loose Grey brown Moist											
73.5							74					
1.7	CLAY (Weathered Crust)		3	SS	4							
73.1	Very stiff Grey brown Moist											
2.1	CLAY						73					
	Stiff to firm Grey Moist to wet											
			4	SS	WH		72					
							71					
			5	SS	WH							
							70					
							69					
68.8			6	SS	7							
6.4	Sandy SILT, some gravel, trace clay with cobbles and boulders (TILL)											
	Compact to very dense Grey Wet											
			7	SS	76		68					
			8	SS	34		67					
			9	SS	11		66					
			10	SS	12							

Continued Next Page

+ 3, x 3: Numbers refer to Sensitivity

○ 3% STRAIN AT FAILURE

MISS_MTO_05-1120-210-4000 GPJ ON MOT GDT 4/10/07

PROJECT 05-1120-210-2000		RECORD OF BOREHOLE No 06-14		2 OF 2	METRIC
W.P. 4058-01-00		LOCATION N 5027540.4; E 364478.1		ORIGINATED BY D.G.	
DIST HWY 417		BOREHOLE TYPE Power Auger 108 mm I.D. Hollow Stem Auger		COMPILED BY J.M.	
DATUM Geodetic		DATE May 15, 2006		CHECKED BY M.L.C.	

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 (%)	
								20 40 60 80 100	20 40 60 80 100						25 50 75	
-- CONTINUED FROM PREVIOUS PAGE --								○ UNCONFINED + FIELD VANE								
64.5	Sandy SILT, some gravel, trace clay with cobbles and boulders (TILL) Compact to very dense Grey Wet		11	SS	50			● QUICK TRIAXIAL × REMOULDED								
10.7	Coarse SAND, some gravel with cobbles and boulders Very dense Grey Wet		12	SS	55											
63.8																
11.4	Limestone (BEDROCK) Fresh Thinly to medium bedded Grey Medium strong Bedrock cored between 11.4m 14.5m depth. For bedrock coring details refer to Record of Drillhole 06-14.		13	NQ RC	DD											
			14	NQ RC	DD											
			15	NQ RC	DD											
60.7																
14.5	End of Borehole															
	Note: Water level in standpipe at 3.3m depth below ground surface on June 12, 2006															

PROJECT: 05-1120-210-2000

RECORD OF DRILLHOLE: 06-14

SHEET 1 OF 1

LOCATION: N 5027540.4; E 384478.1

DRILLING DATE: May 15, 2006

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: CME 75

DRILLING CONTRACTOR: Marathon Drilling

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV.		RUN NO.	PENETRATION RATE (m/min)	FLUSH	COLOUR % RETURN	FR/FX-FRACTURE F-FAULT CL-CLEAVAGE J-JOINT SM-SMOOTH FL-FLEXURED BC-BROKEN CORE SH-SHEAR P-POLISHED R-ROUGH UE-UNEVEN MB-MECH. BREAK VN-VEIN S-SLICKENSIDED PL-PLANAR W-WAVY B-BEDDING C-CURVED										DISCONTINUITY DATA				HYDRAULIC CONDUCTIVITY			NOTES WATER LEVELS INSTRUMENTATION																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																					
				DEPTH (m)	R.Q.D. %					RECOVERY		FRACT. INDEX PER 0.3	DIP w.r.t CORE AXIS	TYPE AND SURFACE DESCRIPTION	K cm/sec																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																	
										TOTAL CORE %	SOLID CORE %				10 ⁻⁵	10 ⁻⁴	10 ⁻³																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																															
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		ROCK SURFACE		63.80																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																												

DEPTH SCALE

1:50



LOGGED: D.G.

CHECKED: W.C.

MIS-RCK 001 05-1120-210-4000-ROCK GPJ GLDR CAN GDT 4/10/07

PROJECT 05-1120-210-2000		RECORD OF BOREHOLE No 06-17		1 OF 1	METRIC
W.P. 4058-01-00	LOCATION N 5027932.3; E 364740.8	ORIGINATED BY D.J.S.			
DIST HWY 417	BOREHOLE TYPE Power Auger 108 mm I.D. Hollow Stem Auger	COMPILED BY J.M.			
DATUM Geodetic	DATE May 11, 2006	CHECKED BY M.I.C.			

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						
74.1	GROUND SURFACE													
0.0	Topsoil (FILL)													
0.1	Silty clay, trace gravel (FILL) Grey brown													
73.1	TOPSOIL													
1.3	CLAY (Weathered Crust) Very stiff to stiff Grey brown Moist to wet		1	SS	15									
			2	SS	7									
			3	SS	2									
70.4	Silty CLAY Stiff to firm Grey Wet													
3.7			4	SS	WH									
			5	SS	WH									
67.3	Sandy SILT, some gravel, trace clay with cobbles and boulders (TILL) Grey Wet													
6.8			6	NQ RC	DD									
66.0	Limestone with thin shale interbed (BEDROCK) Slightly weathered to fresh Grey Medium strong													
8.1	Bedrock cored between 8.1m 11.6m depth. For bedrock coring details refer to Record of Drillhole 06-17.		7	NQ RC	DD									
			8	NQ RC	DD									
			9	NQ RC	DD									
62.5	End of Borehole													
11.6														

MISS_MTO 05-1120-210-5000.GPJ ON_MOT.GDT 4/10/07

PROJECT		05-1120-210-2000		RECORD OF BOREHOLE No 06-18		1 OF 1		METRIC					
W.P.		4058-01-00		LOCATION		N 5027945.9; E 364749.6		ORIGINATED BY					
DIST		HWY 417		BOREHOLE TYPE		Power Auger 108 mm I.D. Hollow Stem Auger		COMPILED BY					
DATUM		Geodetic		DATE		May 10, 2006		CHECKED BY					
								M.I.C.					
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa					
73.7	GROUND SURFACE												
73.4	Topsoil (FILL)												
0.5	Sand, some gravel (FILL) Brown Moist		1	A.S.									
72.5	Silty clay, trace sand (FILL) Grey brown Moist		2/3	SS	9								
1.2	Silty CLAY (Weathered Crust) Stiff to very stiff Grey brown Moist to wet		4	SS	11								
			5	SS	4								
70.2			6/7	SS	1								
3.7	SAND, trace shells Grey Wet CLAY Firm to stiff Grey Wet		8	SS	1								
			9	SS	WH								
66.5			10	NQ RC	DD								
7.2	Limestone with thin shale interbeds (BEDROCK) Fresh Grey Medium strong Bedrock cored between 7.2m 10.2m depth. For bedrock coring details refer to Record of Drillhole 06-18.		11	NQ RC	DD								
63.5													
10.2	End of Borehole Note: Water level in standpipe at 3.0m depth below ground surface on June 12, 2006												

PROJECT: 05-1120-210-2000

RECORD OF DRILLHOLE: 06-18

SHEET 1 OF 1

LOCATION: N 5027945.9; E 364749.6

DRILLING DATE: May 10, 2006

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: CME 75

DRILLING CONTRACTOR: Marathon Drilling

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	PENETRATION RATE (mm/min)	FLUSH	COLOUR % RETURN	FR/FX-FRACTURE F-FAULT CL-CLEAVAGE J-JOINT SH-SHEAR P-POLISHED ST-STEPPED VN-VEIN S-SLICKENSIDED PL-PLANAR										SM-SMOOTH R-ROUGH FL-FLEXURED UE-UNEVEN MB-MECH. BREAK B-BEDDING C-CURVED										DIAMETRAL POINT LOAD INDEX (MPa)	NOTES WATER LEVELS INSTRUMENTATION			
									RECOVERY		R.Q.D. %	FRACT. INDEX PER 0.3	DISCONTINUITY DATA		DIP w.r.t. CORE AXIS	TYPE AND SURFACE DESCRIPTION	HYDRAULIC CONDUCTIVITY																
									TOTAL CORE %	SOLID CORE %			DIP w.r.t. CORE AXIS	TYPE AND SURFACE DESCRIPTION			K, cm/sec																
									80 60 40 20	80 60 40 20							0 20 40 60 80	10 ⁻⁸ 10 ⁻⁶ 10 ⁻⁴ 10 ⁻²	2 4 6														
8	Relay Drill NQ Core	ROCK SURFACE		66.50																													
9		Limestone with thin shale interbeds (BEDROCK) Fresh Grey Medium strong		7.20	1																												
10				63.50	2																												
11		End of Drillhole		10.20																													
12																																	
13																																	
14																																	
15																																	
16																																	
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21																																	
22																																	

DEPTH SCALE

1 : 75







LOGGED: D.G.

CHECKED: W.C.

MIS-RCK 001 05-1120-210-5000-ROCK GPJ GLDR CAN GDT 4/10/07 JM

PROJECT		RECORD OF BOREHOLE		No 06-101		1 OF 1		METRIC					
W.P.		LOCATION		N 5026067.6; E 363365.3		ORIGINATED BY		D.G.					
DIST		BOREHOLE TYPE		Power Auger 108 mm I.D. Hollow Stem Auger		COMPILED BY		J.M.					
DATUM		DATE		July 24, 2006		CHECKED BY		M.I.C.					
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			REMARKS & GRAIN SIZE DISTRIBUTION (%)				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	20 40 60 80 100	PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	GR SA SI CL
81.4	GROUND SURFACE												
0.0	TOPSOIL		1	A.S.									
81.2	Moist												
81.0	Fine SAND, some silt		2	A.S.									
0.4	Light brown												
	Moist												
	Silty CLAY (Weathered Crust)												
	Very stiff												
	Grey brown												
	Moist		3	SS	7								
			4	SS	6								
79.0													
2.4	Silty CLAY		5	SS	4								
	Firm to stiff												
	Grey												
	Moist to wet		6	SS	2								
77.1													
4.3	End of Borehole												
	Note: Water level in open borehole at 3.4m depth below ground surface upon completion of drilling.												

PROJECT 05-1120-210-2000		RECORD OF BOREHOLE No 06-102		1 OF 1	METRIC
W.P. 4058-01-00		LOCATION N 5026120.1; E 363405.1		ORIGINATED BY D.G.	
DIST HWY 417		BOREHOLE TYPE Power Auger 108 mm I.D. Hollow Stem Auger		COMPILED BY J.M.	
DATUM Geodetic		DATE July 24, 2006		CHECKED BY M.I.C.	

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 (%)				
81.5	GROUND SURFACE																
0.0	TOPSOIL Moist		1	A.S.													
81.0																	
0.5	Fine SAND, trace silt Light brown		2	A.S.													
80.7	Moist																
0.8	Silty CLAY (Weathered Crust) Very stiff Grey brown Moist		3	SS	9												
			4	SS	5												
			5	SS	9												
78.5																	
3.1	Silty CLAY Firm to stiff Grey Wet		6	SS	2												
77.2																	
4.3	End of Borehole																
	Note: Water level in open borehole at 3.1m depth below ground surface upon completion of drilling.																



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

MISS_MTO 05-1120-210-6000.GPJ ON MOT,GDT 4/10/07

PROJECT		RECORD OF BOREHOLE		No 06-104		1 OF 1		METRIC											
W.P.		LOCATION		ORIGINATED BY		D.G.													
DIST		BOREHOLE TYPE		COMPILED BY		J.M.													
DATUM		DATE		CHECKED BY		M.I.C.													
05-1120-210-2000		N 5026216.5; E 363479.2																	
4058-01-00		Power Auger 108 mm I.D. Hollow Stem Auger																	
HWY 417		July 24, 2008																	
Geodetic																			
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)				
ELEV	DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	20 40 60 80 100	20 40 60 80 100	25 50 75	W _p	W	W _L	γ	GR	SA	SI	CL
81.3		PARKING LOT SURFACE																	
80.9	0.1	ASPHALTIC CONCRETE		1	A.S.														
80.9	0.4	Sand and gravel (BASE) Brown Moist		2/3	SS	6													
80.1		Silty clay, trace gravel (FILL) Grey brown Moist		4	SS	3													
79.2	1.4	Fine sand, trace silt (FILL) Light brown Moist		5	SS	2													
78.3	2.1	Silty CLAY (Weathered Crust) Stiff Grey brown Moist		6	TP	PH													
77.0	3.1	Silty CLAY Firm Grey Wet																	
4.3		End of Borehole																	
Note: Water level in well screen at 2.2m depth below ground surface on Aug. 22, 2006.																			

PROJECT 05-1120-210-2000		RECORD OF BOREHOLE No 06-105		1 OF 1	METRIC
W.P. 4058-01-00		LOCATION N 5026287.1; E 363534.4		ORIGINATED BY D.G.	
DIST HWY 417		BOREHOLE TYPE Power Auger 108 mm I.D. Hollow Stem Auger		COMPILED BY J.M.	
DATUM Geodetic		DATE July 26, 2008		CHECKED BY M.I.C.	

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 (%)
								20	40	60	80	100						
80.1	GROUND SURFACE																	
0.0	Crushed stone with cobbles (FILL) Grey Moist		1	A.S.														
79.2																		
0.9	Silty CLAY (Weathered Crust) Very stiff Grey Moist		2	SS	14													
			3	SS	6													
77.8																		
2.3	Silty CLAY Firm Grey Wet		4	SS	WH													
76.8																		
76.6	Sandy SILT, some gravel, trace clay with cobbles and boulders (TILL) Grey Wet		5	SS	>26													
3.5	End of Borehole Spoon Refusal																	
	Note: Water level in open borehole at 2.4m depth below ground surface upon completion of drilling.																	

MISS_MTO_05-1120-210-6000.GPJ ON MOT GDT 4/10/07

PROJECT		RECORD OF BOREHOLE		No 06-106		1 OF 1		METRIC					
W.P.		LOCATION		N 5026391.4; E 363609.8		ORIGINATED BY		R.I.					
DIST		BOREHOLE TYPE		Portable Auger		COMPILED BY		J.M.					
DATUM		DATE		July 11, 2006		CHECKED BY		M.I.C.					
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC NATURAL LIQUID UNIT REMARKS				
ELEV	DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	20 40 60 80 100	W _p W W _L	WATER CONTENT (%)	γ	GR SA SI CL
80.2	0.0	GROUND SURFACE											
	0.6	Topsoil (FILL) Moist		1	SS	9		80					
	79.6	Silty sand with cobbles and boulders (FILL)		2	SS	>36							
	79.3	Compact Brown Moist		3	AW RC	DD		79					
	0.9	Gravelly SAND, some silt and clay, with cobbles and boulders (TILL)		4	SS	22							
		Compact Wet		5	SS	>100							
	78.1	Limestone (BEDROCK)		6	AW RC	DD		78					
	2.1	Fresh Medium strong		7	EW RC	DD							
		Bedrock cored between 2.1m 3.1m depth. For bedrock coring details refer to Record of Drillhole 06-106.		8	EW RC	DD							
	77.2												
	3.1	End of Borehole											

PROJECT: 05-1120-210-2000

RECORD OF DRILLHOLE: 06-106

SHEET 1 OF 1

LOCATION: N 5026391.4; E 363809.8

DRILLING DATE: July 11, 2006

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: Portable Auger

DRILLING CONTRACTOR:

DEPTH SCALE METRES	DRILLING RECORD		DESCRIPTION	SYMBOLIC LOG	ELEV.		RUN No.	PENETRATION RATE (mm/min)	FLUSH	COLOUR & RETURN	FR/FX-FRACTURE F-FAULT				SM-SMOOTH				FL-FLEXURED				BC-BROKEN CORE				DIAMETRAL POINT LOAD INDEX (MPa)	NOTES WATER LEVELS INSTRUMENTATION
					DEPTH (m)	ELEV. (m)					CL-CLEAVAGE		J-JOINT		R-ROUGH		UE-UNEVEN		MB-MECH. BREAK									
	SH-SHEAR										P-POLISHED		ST-STEPPED		W-WAVY		B-BEDDING											
	VN-VEIN										S-SLICKENSIDED		PL-PLANAR		C-CURVED													

DEPTH SCALE

1:50



LOGGED: R.I.

CHECKED: W.C.

MIS-RCK-001: 05-1120-210-6000-ROCK.GPJ GLDR. CAN.GDT 4/10/07

PROJECT 05-1120-210-2000			RECORD OF BOREHOLE No 06-107			1 OF 1			METRIC		
W.P. 4058-01-00			LOCATION N 5026425.6; E 363637.2			ORIGINATED BY D.G.					
DIST HWY 417			BOREHOLE TYPE Power Auger 108 mm I.D. Hollow Stem Auger			COMPILED BY J.M.					
DATUM Geodetic			DATE July 26, 2006			CHECKED BY M.I.C.					

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
						○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED											
						20 40 60 80 100 20 40 60 80 100											
						WATER CONTENT (%)											
						25 50 75											
80.0	GROUND SURFACE																
0.0	Crushed stone (FILL)																
0.2	Grey Fine silty SAND, trace gravel and sand (TILL) Compact Light brown Moist		1	SS	19											8 53 30 9	
78.5																	
1.5	Sandy SILT, some gravel, trace clay with cobbles and boulders (TILL) Dense Grey brown Moist		2	SS	37												
77.7																	
2.3	End of Borehole Auger Refusal																

MISS_MTO 05-1120-210-5000.GPJ ON MOT.GDT 4/10/07

PROJECT 05-1120-210-2000		RECORD OF BOREHOLE No 06-108		1 OF 1		METRIC										
W.P. 4058-01-00		LOCATION N 5026464.8; E 363864.9		ORIGINATED BY D.G.												
DIST HWY 417		BOREHOLE TYPE Power Auger 108 mm I.D. Hollow Stem Auger		COMPILED BY J.M.												
DATUM Geodetic		DATE July 26, 2006		CHECKED BY M.I.C.												
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa							WATER CONTENT (%)	
79.5	GROUND SURFACE		1	A.S.			20	40	60	80	100					
0.0	Crushed stone (FILL)															
0.1	Grey black Moist TOPSOIL		2	A.S.												
78.9	Moist															
0.6	Silty CLAY, trace sand (Weathered Crust)															
	Compact Grey brown Moist		3	SS	10											
			4	SS	12											
77.2																
2.3	Sandy SILT, some gravel, trace clay with cobbles and boulders (TILL)		5	SS	>30	▽										
	Compact Brown Wet															
76.5																
3.0	End of Borehole Auger Refusal															
	Note: Water level in open borehole at 2.4m depth below ground surface upon completion of drilling.															

PROJECT		RECORD OF BOREHOLE		No 06-109		1 OF 1		METRIC					
W.P.		LOCATION		N 5026424.4; E 363566.5		ORIGINATED BY		R.I.					
DIST		BOREHOLE TYPE		Portable Auger		COMPILED BY		J.M.					
DATUM		DATE		July 11, 2006		CHECKED BY		M.I.C.					
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			REMARKS & GRAIN SIZE DISTRIBUTION (%)				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	20 40 60 80 100	PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	GR SA SI CL
79.2	GROUND SURFACE												
0.0	Topsoil (FILL)												
0.1	Moist Sand, some gravel and silt with cobbles (FILL) Dark brown Moist		1	SS	10		79						
78.3			2	SS	13		78						
0.9	Sand, trace silt (FILL) Compact Brown Moist to wet		3	SS	12								
77.1			4	SS	37		77						
76.9	Sand and gravel with cobbles (FILL) Brown Wet		5	SS	7								
2.3	Sand, trace silt (FILL) Loose Brown Moist		6	SS	>100		76						
75.9			7	EW RC	DD								
3.3	Limestone (BEDROCK) Highly fractured Weathered Grey Medium strong		8	EW RC	DD								
75.3			9	EW RC	DD								
3.9	Limestone (BEDROCK) Fresh Grey Medium strong		10	EW RC	DD		75						
74.6	Bedrock cored between 3.3m 4.7m depth. For bedrock coring details refer to Record of Drillhole 06-109.												
4.7	End of Borehole												

MISS_MTO 05-1120-210-6000.GPJ ON_MOT_GDT 4/10/07

PROJECT: 05-1120-210-2000

RECORD OF DRILLHOLE: 06-109

SHEET 1 OF 1

LOCATION: N 5026424.4; E363566.5

DRILLING DATE: July 11, 2006

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: Portable Auger

DRILLING CONTRACTOR:

DEPTH SCALE METRES	DRILLING RECORD		DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	PENETRATION RATE (mm/min)	FLUSH COLOUR & RETURN	FR/FX-FRACTURE F-FAULT CL-CLEAVAGE J-JOINT SH-SHEAR P-POLISHED ST-STEPPED VN-VEIN S-SLICKENSIDED PL-PLANAR										SM-SMOOTH R-ROUGH UE-UNEVEN W-WAVY C-CURVED				FL-FLEXURED UE-UNEVEN MB-MECH. BREAK B-BEDDING				DIAMETRAL POINT LOAD INDEX (MPa)	NOTES WATER LEVELS INSTRUMENTATION
									RECOVERY		R.Q.D. %	FRACT. INDEX PER 0.3	DISCONTINUITY DATA				HYDRAULIC CONDUCTIVITY K _o cm/sec											
									TOTAL CORE %	SOLID CORE %			TYPE AND SURFACE DESCRIPTION															
									0 5 10 15 20	0 5 10 15 20			0 5 10 15 20	0 5 10 15 20	0 5 10 15 20	0 5 10 15 20	0 5 10 15 20	0 5 10 15 20	10 ⁻⁶	10 ⁻⁵	10 ⁻⁴	10 ⁻³						
4	Rotary Drill EW Core	ROCK SURFACE		75.00																								
		Limestone (BEDROCK)		3.30	1																							
		Highly fractured																										
		Weathered																										
	Grey																											
		Medium strong		75.30	2																							
		Limestone (BEDROCK)		3.90	3																							
	Fresh																											
	Grey																											
	Medium strong					4																						
		End of Drillhole		74.50																								
5				4.70																								
6																												
7																												
8																												
9																												
10																												
11																												
12																												
13																												

DEPTH SCALE

1:50



LOGGED: R.I.

CHECKED: W.C.

MIS-ROCK 001 05-1120-210-6000-ROCK GPJ GLDR CAN GDT 4/10/07

PROJECT		RECORD OF BOREHOLE		No 06-110		1 OF 1		METRIC					
W.P.		LOCATION		N 5026487.7; E 363611.9		ORIGINATED BY		R.I.					
DIST		BOREHOLE TYPE		Portable Auger		COMPILED BY		J.M.					
DATUM		DATE		July 12, 2008		CHECKED BY		M.I.C.					
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			REMARKS & GRAIN SIZE DISTRIBUTION (%)				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	20 40 60 80 100	PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ	GR SA SI CL
78.5	GROUND SURFACE												
0.0	Topsoil (FILL)												
78.3	Moist												
0.3	Silty sand, some gravel, trace clay with organic matter (FILL)		1	SS	11		78						
77.6	Compact Dark brown Moist												
0.9	Silty clay, trace to some sand, trace gravel with organic matter (FILL)		2	SS	12								
76.6	Stiff Grey brown Moist						77						
1.8	Clayey SILT, trace sand Compact Grey brown Moist		3	SS	9								
76.1													
2.6	Silty SAND, trace gravel (TILL) Grey Moist to wet Shale limestone (BEDROCK) Weathered Grey black Medium strong Dolomitic limestone (BEDROCK) Fresh Grey Medium strong		4	SS	19								
74.6													
3.9	Bedrock cored between 2.5m 3.9m depth. For bedrock coring details refer to Record of Drillhole 06-110. End of Borehole		5	SS	>100		76						
			6	EW RC	DD								
			7	EW RC	DD								
			8	EW RC	DD		75						

PROJECT 05-1120-210-2000		RECORD OF BOREHOLE No 06-111		1 OF 1	METRIC
W.P. 4058-01-00		LOCATION N 5026553.2; E 363730.9		ORIGINATED BY D.G.	
DIST HWY 417		BOREHOLE TYPE Power Auger 108 mm I.D. Hollow Stem Auger		COMPILED BY J.M.	
DATUM Geodetic		DATE July 26, 2006		CHECKED BY M.I.C.	

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												
78.5	GROUND SURFACE																			
0.0	Crushed stone (FILL)		1	A.S.																
0.1	Grey Moist		2	A.S.																
77.9	Fine SAND, some silt																			
0.6	Light brown Moist																			
	SILT, some sand and clay, trace gravel (TILL)		3	SS	>26											2 28 49 21				
77.2	Brown Moist																			
1.3	End of Borehole Auger Refusal																			

MISS_MTO_05-1120-210-6000.GPJ ON_MOT_GDT_4/10/07

PROJECT 05-1120-210-2000		RECORD OF BOREHOLE No 06-112		1 OF 1	METRIC
W.P. 4058-01-00	LOCATION N 5026609.8; E 363772.9	ORIGINATED BY D.G.			
DIST HWY 417	BOREHOLE TYPE Power Auger 108 mm I.D. Hollow Stem Auger	COMPILED BY J.M.			
DATUM Geodetic	DATE July 26, 2006	CHECKED BY M.I.C.			


SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							WATER CONTENT (%)			
								20 40 60 80 100										

78.4	GROUND SURFACE													
0.0	Crushed stone (FILL)		1	A.S.										
0.2	Coarse Grey Moist Clayey SILT Loose Grey to light brown Moist		2	SS	9									
76.7														
1.7	Sandy SILT, some gravel, trace clay with cobbles and boulders Compact Grey Moist		3	SS	20									
75.9			4	SS	>31									
2.5	End of Borehole Auger Refusal													

MISS_MTO 05-1120-210-6000.GPJ ON MOT.GDT 4/10/07

PROJECT		05-1120-210-2000		RECORD OF BOREHOLE No 06-113		1 OF 1		METRIC									
W.P.		4058-01-00		LOCATION		N 5026715.8; E 363852.9		ORIGINATED BY									
DIST		HWY 417		BOREHOLE TYPE		Portable Auger		COMPILED BY									
DATUM		Geodetic		DATE		July 13, 2006		CHECKED BY									
								M.I.C.									
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV	DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa			WATER CONTENT (%)			Y		
									20 40 60 80 100	20 40 60 80 100	25 50 75	25 50 75	25 50 75	25 50 75	25 50 75	25 50 75	25 50 75
78.6		GROUND SURFACE															
0.0		Crushed stone (FILL) Loose Brown Moist		1	SS	8											
78.1																	
0.5		Silty sand, some gravel (FILL) Loose to compact Brown Moist		2/3	SS	14											
77.8																	
0.8		Sandy SILT, trace gravel and clay (FILL) Loose Brown Wet		4	SS	8											
76.9																	
1.7		Dolomitic limestone (BEDROCK) Fresh Grey Medium strong		5	EW RC	DD											
76.2																	
2.4		Dolomitic limestone (BEDROCK) Fractured Grey Medium strong															
75.9																	
2.7		Bedrock cored between 1.7m 2.7m depth. For bedrock coring details refer to Record of Drillhole 06-113. End of Borehole															

[illegible]

PROJECT		RECORD OF BOREHOLE No 06-114				1 OF 1		METRIC									
W.P. 4058-01-00		LOCATION N 5026721.5; E 363859.6				ORIGINATED BY R.I.											
DIST HWY 417		BOREHOLE TYPE Portable Auger				COMPILED BY J.M.											
DATUM Geodetic		DATE July 14, 2006				CHECKED BY M.I.C.											
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								WATER CONTENT (%)	
78.4	GROUND SURFACE							20	40	60	80	100					
0.0	Crushed stone (FILL) Grey Moist		1	SS	10												
77.8			2	SS	8												
0.6	Sandy SILT, some gravel, trace clay (TILL) Loose to compact Brown Moist to wet		3	SS	3												
76.7			4	SS	33												
1.7	Limestone (BEDROCK) Weathered Grey Medium strong		5	EW RC	DD												
75.6	Bedrock cored between 1.7m 2.8m depth. For bedrock coring details refer to Record of Drillhole 06-114.		6	EW RC	DD												
2.8	End of Borehole																

MISS_MTO 05-1120-210-6000.GPJ ON_MOT_GDT 4/10/07

PROJECT: 05-1120-210-2000

RECORD OF DRILLHOLE: 06-114

SHEET 1 OF 1

LOCATION: N 5026721.5; E 363859.6

DRILLING DATE: July 14, 2006

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: —

DRILL RIG: Portable Auger

DRILLING CONTRACTOR:

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	PENETRATION RATE (mm/min)	FLUSH	RECOVERY	R.O.D. %	FRACT. INDEX PER 0.3	DISCONTINUITY DATA	HYDRAULIC CONDUCTIVITY K _h cm/sec	DIAMETRAL POINT LOAD INDEX (MPa)	NOTES WATER LEVELS INSTRUMENTATION
		ROCK SURFACE		78.70										
2	Rotary Drill EM Core	Limestone (BEDROCK) Weathered Grey Medium strong		1.70	1									
					2									
3		End of Drillhole		75.60 2.80										
4														
5														
6														
7														
8														
9														
10														
11														

DEPTH SCALE

1 : 50



LOGGED: R.I.

CHECKED: W.C.

MIS-RCK-001 05-1120-210-8000-ROCK.GPJ GLDR_CAN.GDT 4/10/07

PROJECT 05-1120-210-2000		RECORD OF BOREHOLE No 06-115				1 OF 1		METRIC	
W.P. 4058-01-00		LOCATION N 5026603.2; E 363701.7				ORIGINATED BY D.J.S.			
DIST HWY 417		BOREHOLE TYPE Power Auger 108 mm I.D. Hollow Stem Auger				COMPILED BY J.M.			
DATUM Geodetic		DATE July 5, 2006				CHECKED BY M.I.C.			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			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	W _p W W _L	25 50 75	KN/m ³		
78.6	GROUND SURFACE													
0.0	Crushed stone (FILL)													
78.2	Sand and gravel (FILL)													
0.4	Brown													
77.8	Sandy silt, some gravel and organic matter (FILL)													
0.8	Dark brown													
77.5	Fine sand (FILL)													
1.1	Loose		1	SS	8									
77.1	Grey													
	Moist													
77.1	TOPSOIL													
	Dark brown													
	Moist													
76.8	Sandy SILT													
	Grey brown		2	SS	4									
1.8	Moist to wet													
	Clayey SILT													
	Grey													
	Moist to wet													
	Sandy SILT, some gravel and clay (TILL)													
	Loose to dense		3	SS	45									
75.8	Grey													
2.8	Moist to wet													
	End of Borehole													
	Auger Refusal													

MISS MTO 05-1120-210-6000.GPJ ON MOT.GDT 4/10/07

PROJECT 05-1120-210-2000		RECORD OF BOREHOLE No 06-116				1 OF 1		METRIC	
W.P. 4058-01-00		LOCATION N 5026659.1; E 363742.4				ORIGINATED BY D.J.S.			
DIST HWY 417		BOREHOLE TYPE Power Auger 108 mm I.D. Hollow Stem Auger				COMPILED BY J.M.			
DATUM Geodetic		DATE July 6, 2006				CHECKED BY M.I.C.			

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 20 40 60 80 100 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED						
77.6	GROUND SURFACE													
0.0	Crushed stone (FILL) Grey													
77.1	Topsoil (FILL)													
0.5	Fine to medium sand, some gravel (FILL) Brown													
76.6	Sandy silt, some gravel and clay (FILL) Brown													
1.0	Moist TOPSOIL		1	SS	6									
76.2	Moist Sandy SILT (TILL) Brown		2	SS	50									
1.7	Dry Limestone (BEDROCK) Weathered End of Borehole Auger Refusal													

MISS_MTO 05-1120-210-6000.GPJ ON_MOT_GDT 4/10/07

PROJECT 05-1120-210-2000		RECORD OF BOREHOLE No 06-117		1 OF 1	METRIC
W.P. 4058-01-00	LOCATION N 5026711.2; E 363783.3	ORIGINATED BY D.J.S.			
DIST HWY 417	BOREHOLE TYPE Power Auger 108 mm I.D. Hollow Stem Auger	COMPILED BY J.M.			
DATUM Geodetic	DATE July 6, 2006	CHECKED BY M.I.C.			

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					w _p	w	w _L		
77.5	GROUND SURFACE																
0.0	Crushed stone (FILL) Grey																
0.3	Sand and gravel (FILL) Brown																
76.7	Sandy silt, some gravel and cobbles (FILL) Brown																
76.5	TOPSOIL																
1.0	Moist Sandy SILT, some gravel and clay (TILL)		1	SS	13												
76.1	Compact Brown																
1.4	Moist End of Borehole Auger Refusal																

PROJECT 05-1120-210-2000		RECORD OF BOREHOLE No 06-118		1 OF 1	METRIC
W.P. 4058-01-00	LOCATION N 5026822.1; E 363932.7	ORIGINATED BY D.G.			
DIST HWY 417	BOREHOLE TYPE Power Auger 108 mm I.D. Hollow Stem Auger	COMPILED BY J.M.			
DATUM Geodetic	DATE July 27, 2006	CHECKED BY M.I.C.			



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/m³	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										
78.3	GROUND SURFACE																	
8.0 8.1	Crushed stone (FILL) Grey Moist TOPSOIL Moist		1	A.S.														
77.6 0.7	Sandy SILT, trace gravel and clay (TILL) Very dense Grey brown Moist		2	SS	73													
76.8 1.5	Probably limestone (BEDROCK) Weathered Medium strong		3	SS	>50													
76.4 1.9	End of Borehole Auger Refusal																	

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PROJECT 05-1120-210-2000		RECORD OF BOREHOLE No 06-119		1 OF 1	METRIC
W.P. 4058-01-00	LOCATION N 5026871.9; E 363967.1	ORIGINATED BY D.G.			
DIST HWY 417	BOREHOLE TYPE Power Auger 108 mm I.D. Hollow Stem Auger	COMPILED BY J.M.			
DATUM Geodetic	DATE July 27, 2006	CHECKED BY M.I.C.			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
77.5	GROUND SURFACE							20	40	60	80	100					
8.9	TOPSOIL																
76.4	Sandy SILT, trace clay and gravel (TILL) Dense Brown Moist						77										
1.1	Sandy SILT, some gravel, trace clay with cobbles and boulders (TILL) Compact to dense Grey brown Moist		1/2	SS	58		76										7 31 52 10
			3	SS	37												
			4	SS	18		75										
74.8																	
2.7	End of Borehole Auger Refusal																

PROJECT 05-1120-210-2000		RECORD OF BOREHOLE No 06-120		1 OF 1	METRIC
W.P. 4058-01-00	LOCATION N 5026910.7; E 363997.9	ORIGINATED BY D.G.			
DIST HWY 417	BOREHOLE TYPE Power Auger 108 mm I.D. Hollow Stem Auger	COMPILED BY J.M.			
DATUM Geodetic	DATE July 27, 2006	CHECKED BY M.I.C.			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC NATURAL LIQUID LIMIT MOISTURE CONTENT			UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					W _p	W	W _L		
								20 40 60 80 100									
77.5	GROUND SURFACE																
8.0 0.1	Topsoil (FILL) Sandy silt, some gravel with organic matter (FILL) Dark brown Moist		1	A.S.													
76.4																	
1.1	Sandy SILT, some gravel, trace clay, cobbles and boulders (TILL) Compact Grey brown Moist		2	SS	30												
75.8																	
1.8	End of Borehole Auger Refusal		3	SS	>22												

MISS_MTO 05-1120-210-6000.GPJ ON_MOT_GDT 4/10/07

PROJECT <u>05-1120-210-2000</u>		RECORD OF BOREHOLE No 06-121		1 OF 1	METRIC
W.P. <u>4058-01-00</u>	LOCATION <u>N 5026930.8; E 364030.6</u>	ORIGINATED BY <u>D.G.</u>			
DIST <u>HWY 417</u>	BOREHOLE TYPE <u>Power Auger 108 mm I.D. Hollow Stem Auger</u>	COMPILED BY <u>J.M.</u>			
DATUM <u>Geodetic</u>	DATE <u>July 27, 2006</u>	CHECKED BY <u>M.I.C.</u>			


SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m³	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												
76.8	PARKING LOT SURFACE						20	40	60	80	100									
0.0	ASPHALTIC CONCRETE																			
76.5	Crushed stone (BASE)		1	A.S.																
0.3	Grey Moist																			
	Sandy SILT, some gravel, trace clay (TILL) Compact Brown Moist																			
75.7																				
1.1	Sandy SILT, some gravel, trace clay with cobbles and boulders (TILL) Compact Grey brown Moist		2/3	SS	37															
			4	SS	>50															
74.4																				
2.4	End of Borehole Auger Refusal		5	SS																

MISS_MTO 05-1120-210-6000.GPJ ON MOT.GDT 4/10/07

PROJECT 05-1120-210-2000		RECORD OF BOREHOLE No 06-122		1 OF 1	METRIC
W.P. 4058-01-00		LOCATION N 5026953.6; E 364053.4		ORIGINATED BY D.G.	
DIST HWY 417		BOREHOLE TYPE Power Auger 108 mm I.D. Hollow Stem Auger		COMPILED BY J.M.	
DATUM Geodetic		DATE July 27, 2006		CHECKED BY M.I.C.	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC NATURAL LIQUID LIMIT MOISTURE LIMIT CONTENT			UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							WATER CONTENT (%)		
77.3	GROUND SURFACE						20	40	60	80	100	W _p	W	W _L			
0.0	ASPHALTIC CONCRETE																
77.0	Crushed stone (BASE)		1	A.S.													
0.3	Grey Moist		2	A.S.													
76.7	TOPSOIL																
0.6	Moist Silty SAND, trace gravel and clay (TILL) Loose Brown Moist		3	SS	6												
75.8																	
1.5	Sandy SILT, some gravel, trace clay with cobbles and boulders (TILL) Dense to very dense Grey brown Moist		4	SS	53												
			5	SS	59												
			6	SS	41												
74																	
73.6																	
3.7	End of Borehole																

PROJECT <u>05-1120-210-2000</u>		RECORD OF BOREHOLE No 06-123		1 OF 1	METRIC
W.P. <u>4058-01-00</u>	LOCATION <u>N 5026903.5, E 363931.2</u>	ORIGINATED BY <u>K.L.</u>			
DIST <u>HWY 417</u>	BOREHOLE TYPE <u>Power Auger 108 mm I.D. Hollow Stem Auger</u>	COMPILED BY <u>J.M.</u>			
DATUM <u>Geodetic</u>	DATE <u>Aug. 9, 2006</u>	CHECKED BY <u>M.I.C.</u>			

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 (%)				
								20 40 60 80 100					w _p w w _L				
77.3 0.0	GROUND SURFACE Topsoil (FILL)		1	A.S.		77											
76.4 0.9	End of Borehole Auger Refusal		2	SS													

PROJECT		RECORD OF BOREHOLE		No 06-124		1 OF 1		METRIC					
W.P. 4058-01-00		LOCATION		N 5026959.0; E 363969.2		ORIGINATED BY		K.L.					
DIST HWY 417		BOREHOLE TYPE		Power Auger 108 mm I.D. Hollow Stem Auger		COMPILED BY		J.M.					
DATUM Geodetic		DATE		Aug. 9, 2006		CHECKED BY		M.I.C.					
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC NATURAL LIQUID LIMIT MOISTURE LIMIT 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 (%)			
76.8 0.0	GROUND SURFACE Silty sand, some gravel (FILL) Brown						20 40 60 80 100	20 40 60 80 100	25 50 75				
76.3 0.5	Clayey silt, some gravel, trace sand (FILL) Dense Brown Moist		1	SS		76							
75.3 1.5	Silty SAND, some gravel Light brown Moist		2	SS	34								
75.0 1.9	End of Borehole Auger Refusal		3	SS	>36	75							

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PROJECT		RECORD OF BOREHOLE		No 06-125		1 OF 1		METRIC																				
W.P.		LOCATION		ORIGINATED BY		K.L.																						
DIST		BOREHOLE TYPE		COMPILED BY		J.M.																						
DATUM		DATE		CHECKED BY		M.I.C.																						
05-1120-210-2000		N 5027012.6; E 364010.7																										
4058-01-00		Power Auger 108 mm I.D. Hollow Stem Auger																										
Geodetic		Aug. 9, 2006																										
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			SHEAR STRENGTH kPa			WATER CONTENT (%)			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)							
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	20	40	60	80	100	20	40	60	80	100	25	50	75	W _p	W	W _L	γ	GR	SA	SI	CL
76.9	GROUND SURFACE																											
76.0	Topsoil (FILL) Silty clay, trace sand and gravel with organic matter (FILL) Grey to dark grey Moist																											
75.1	Silty SAND, some gravel, trace clay (FILL) Loose to compact Grey brown Moist		1	SS	10		76																					
74.6	Sandy silt, some gravel (FILL) Very loose Brown Moist		2	SS	1		75																					
74.2	Silty clay, trace sand (FILL) Grey brown Wet		3	SS	29		74																					
73.2	Crushed stone, some clay (FILL) Loose Grey Wet		4	SS	6																							
73.2	End of Borehole Auger Refusal																											

PROJECT 05-1120-210-2000			RECORD OF BOREHOLE No 06-126			1 OF 1			METRIC													
W.P. 4058-01-00			LOCATION N 5027053.6; E 364048.1			ORIGINATED BY R.I.																
DIST HWY 417			BOREHOLE TYPE Portable Auger			COMPILED BY J.M.																
DATUM Geodetic			DATE July 17, 2006			CHECKED BY M.I.C.																
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)							
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED					WATER CONTENT (%)			γ						
80.6	GROUND SURFACE							20 40 60 80 100														
80.9	Topsoil (FILL) Moist Sandy silt, some gravel (FILL) Loose Dark brown Moist		1	SS	7		80															
79.7	Sand, some silt (FILL) Loose to compact Brown Moist		2	SS	8																	
0.9			3	SS	2		79															
			4	SS	9																	
			5	SS	13		78															
77.1			6	SS	12																	
3.5	Silty SAND, some gravel, trace clay (TILL) Compact Grey brown Wet		7	SS	>42		77															
76.6	End of Borehole																					
4.0																						

MISS_MTO 05-1120-210-6000.GPJ ON_MOT.GDT 4/10/07

PROJECT 05-1120-210-2000		RECORD OF BOREHOLE No 06-127		1 OF 1	METRIC
W.P. 4058-01-00		LOCATION N 5027168.4; E 364215.6		ORIGINATED BY D.G.	
DIST HWY 417		BOREHOLE TYPE Power Auger 108 mm I.D. Hollow Stem Auger		COMPILED BY J.M.	
DATUM Geodetic		DATE July 27, 2006		CHECKED BY M.I.C.	

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					25 50 75				
75.5	GROUND SURFACE																
0.0	Fine sand, some silt (FILL) Light brown Moist		1	A.S.													
74.6																	
0.9	Sandy silt, trace organic matter and gravel (FILL) Dark grey Moist		2	SS	27												
74.1																	
1.4	PEAT Moist																
73.7																	
1.8	Clayey SILT, trace sand Loose Grey Wet		3/4	SS	8												
73.2																	
73.0	Silty CLAY Firm Grey Wet																
2.5	Sandy SILT, some gravel, trace clay with cobbles and boulders (TILL) Loose to compact Grey Wet		5/6	SS	1												
72.1																	
3.5	End of Borehole Auger Refusal																
	Note: Water level in open borehole at 1.8m depth below ground surface upon completion of drilling.																

PROJECT		RECORD OF BOREHOLE		No 06-128		1 OF 1		METRIC									
W.P.		LOCATION		N 5027230.8; E 364247.1		ORIGINATED BY		D.G.									
DIST		BOREHOLE TYPE		Power Auger 108 mm I.D. Hollow Stem Auger		COMPILED BY		J.M.									
DATUM		DATE		July 28, 2006		CHECKED BY		M.I.C.									
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV	DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa			WATER CONTENT (%)			Y		
75.2		GROUND SURFACE							20 40 60 80 100 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED			25 50 75 W _p W W _L			GR SA SI CL		
0.0		Fine sand, some silt (FILL)		1	A.S.			75									
74.9		Light brown Moist															
0.3		PEAT Moist															
74.1				2	SS	5		74									
1.1		Silty SAND Loose Grey Moist															
				3	SS	8											
72.9								73									
2.3		Clayey SILT, trace sand and gravel with cobbles and boulders (TILL) Very loose Grey Wet		4	SS	1											
71.9				5	SS	28		72									
3.4		Sandy SILT, some gravel, trace clay (TILL) Compact Grey Wet															
71.5																	
3.7		End of Borehole															
		Note: Water level in open borehole at 2.4m depth below ground surface upon completion of drilling.															

MISS_MTO 05-1120-210-6000.GPJ ON MOT.GDT 4/10/07

PROJECT 05-1120-210-2000		RECORD OF BOREHOLE No 06-129				1 OF 1		METRIC								
W.P. 4058-01-00		LOCATION N 5027295.3; E 364293.1				ORIGINATED BY D.G.										
DIST HWY 417		BOREHOLE TYPE Power Auger 108 mm I.D. Hollow Stem Auger				COMPILED BY J.M.										
DATUM Geodetic		DATE July 28, 2006				CHECKED BY M.I.C.										
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa								
75.5	GROUND SURFACE						20	40	60	80	100					
0.0	Fine sand, some silt, trace clay (FILL) Light brown Moist		1	A.S.												1 71 21 9
74.8																
0.7	Sandy silt with clay and organic matter (FILL) Dark grey Moist		2	SS	23											
74.1																
1.4	Fine SAND, some silt Light brown Moist															
73.8																
1.7	Silty CLAY (Weathered Crust) Very stiff Grey brown Moist		3/4	SS	7											
73.2																
2.3	Sandy SILT, some gravel, trace clay with cobbles and boulders (TILL) Loose to compact Grey Wet		5	SS	23											
			6	SS	8											
71.8																
3.7	End of Borehole Note: Water level in open borehole at 2.4m depth below ground surface upon completion of drilling.															

PROJECT 05-1120-210-2000

RECORD OF BOREHOLE No 06-130

1 OF 1

METRIC

W.P. 4058-01-00

LOCATION N 5027152.5; E 364110.2

ORIGINATED BY H.E.C.

DIST HWY 417

BOREHOLE TYPE Power Auger 108 mm I.D. Hollow Stem Auger

COMPILED BY J.M.

DATUM Geodetic

DATE Aug. 8, 2006

CHECKED BY M.I.C.

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						
79.0	GROUND SURFACE							20 40 60 80 100						
0.0	Topsoil (FILL)							○ UNCONFINED + FIELD VANE						
0.1	Fine sand, some silt and clay, trace gravel (FILL) Very loose to loose Brown Moist to wet		1	SS	4			● QUICK TRIAXIAL × REMOULDED						
			2	SS	2									2 80 (18)
			3	SS	1									
			4	SS	2									
			5	SS	4									
74.1			6	SS	5									
4.9	Silty SAND, trace gravel and clay with cobbles and boulders (TILL) Loose to dense Grey Wet		7	SS	9									9 41 (50)
			8	SS	36									
			9	SS	>59									
71.9	End of Borehole													
7.1	Auger Refusal													

MISS_MTO 05-1120-210-6000.GPJ ON MOT.GDT 4/10/07

PROJECT 05-1120-210-2000		RECORD OF BOREHOLE No 06-132		1 OF 1	METRIC
W.P. 4058-01-00		LOCATION N 5027341.4; E 364260.1		ORIGINATED BY H.E.C.	
DIST HWY 417		BOREHOLE TYPE Power Auger 108 mm I.D. Hollow Stem Auger		COMPILED BY J.M.	
DATUM Geodetic		DATE Aug. 8, 2006		CHECKED BY M.I.C.	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC NATURAL LIQUID LIMIT MOISTURE LIMIT			UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100	W _p W W _L	WATER CONTENT (%)			
75.4 8.9 0.1	GROUND SURFACE Topsoil (FILL) Fine silty sand (FILL) Compact Brown Moist													
74.1 1.3	Fine silty SAND, some gravel, trace clay (TILL) Loose to compact Grey Moist to wet		1	SS	21									
			2	SS	9									
			3	SS	22					o			14 44 (42)	
			4	SS	7									
71.7 3.7	End of Borehole Note: Water level in well screen at 3.5m depth below ground surface on Aug. 22, 2006.													

PROJECT <u>05-1120-210-2000</u>		RECORD OF BOREHOLE No 06-133		1 OF 1	METRIC
W.P. <u>4058-01-00</u>	LOCATION <u>N 5027391.6; E 364366.9</u>	ORIGINATED BY <u>R.I.</u>			
DIST <u>HWY 417</u>	BOREHOLE TYPE <u>Portable Auger</u>	COMPILED BY <u>J.M.</u>			
DATUM <u>Geodetic</u>	DATE <u>July 10, 2006</u>	CHECKED BY <u>M.I.C.</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					w _p	w	w _L		
								20 40 60 80 100									
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED									
											WATER CONTENT (%) 25 50 75						
76.6	GROUND SURFACE																
0.0	Topsoil (FILL)																
0.1	Moist Sand, trace silt (FILL) Loose Brown Moist		1	SS	6												
			2	SS	4												
75.2																	
1.4	Silty sand (FILL) Compact to dense Grey brown Moist to wet		3	SS	16												
			4	SS	32												
74.0																	
2.6	Topsoil (FILL) Silty sand, occasional silty clay seam (FILL) Compact to dense Brown Moist to wet		5	SS	42												
			6	SS	24												
73.2																	
3.5	Organic SILT Dark brown Moist to wet																
72.8																	
3.8	Silty SAND, trace gravel, some clay (TILL) Loose Grey Moist to wet		7	SS	6												
72.2																	
4.4	End of Borehole																
								</									

MISS_MTO_05-1120-210-6000.GPJ ON_MOT.GDT 5/23/07

PROJECT 05-1120-210-2000		RECORD OF BOREHOLE No 06-134				1 OF 1		METRIC								
W.P. 4058-01-00		LOCATION N 5027468.4; E 364419.8				ORIGINATED BY D.G.										
DIST HWY 417		BOREHOLE TYPE Power Auger 108 mm I.D. Hollow Stem Auger				COMPILED BY J.M.										
DATUM Geodetic		DATE July 28, 2006				CHECKED BY M.I.C.										
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								
78.3	GROUND SURFACE															
0.0	Topsoil (FILL)		1	A.S.												
78.1	Moist															
0.2	Fine sand, some silt (FILL)															
	Very loose to loose															
	Light brown															
	Moist															
			2	SS	10											
			3	SS	4											
			4	SS	2											
			5	SS	2											
74.8																
	Silty CLAY, trace organic matter															
	Stiff															
	Dark grey															
	Moist															
	Silty CLAY (Weathered Crust)		6	SS	14											
	Very stiff															
74.0																
	Grey brown															
	Moist															
4.3	End of Borehole															

PROJECT 05-1120-210-2000		RECORD OF BOREHOLE No 06-135		1 OF 1	METRIC
W.P. 4058-01-00		LOCATION N 5027495.7; E 364431.5		ORIGINATED BY D.G.	
DIST HWY 417		BOREHOLE TYPE Power Auger 108 mm I.D. Hollow Stem Auger		COMPILED BY J.M.	
DATUM Geodetic		DATE July 28, 2006		CHECKED BY M.I.C.	

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 (%)					
								20 40 60 80 100							25 50 75					
80.3	GROUND SURFACE																			
0.0	Topsail (FILL)		1	A.S.																
80.0	Moist																			
0.3	Fine sand, some silt (FILL)																			
	Very loose to compact		2	SS	9															
	Light brown																			
	Moist		3	SS	2															
			4	SS	13															
			5	SS	16															
			6	SS	32															
			7	SS	16															
74.7			8	SS	24															
5.6	Silty CLAY																			
74.4	Very stiff																			
5.9	Grey brown																			
	Moist																			
	End of Borehole																			

PROJECT		RECORD OF BOREHOLE		No 06-136		1 OF 1		METRIC										
W.P. 4058-01-00		LOCATION		N 5027479.3; E 364439.8		ORIGINATED BY		D.G.										
DIST HWY 417		BOREHOLE TYPE		Power Auger 108 mm I.D. Hollow Stem Auger		COMPILED BY		J.M.										
DATUM Geodetic		DATE		July 31, 2006		CHECKED BY		M.I.C.										
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV	DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa					WATER CONTENT (%)			γ	GR SA SI CL
								20 40 60 80 100	20 40 60 80 100	25 50 75	25 50 75	25 50 75	25 50 75	25 50 75				
78.8	0.0	GROUND SURFACE		1	A.S.													
		Topsoil (FILL)																
		Moist																
78.2	0.6	Fine sand, some silt (FILL)		2	SS	4		78										0 79 (21)
		Light brown																
		Moist to wet																
				3	SS	6		77										
				4	SS	7		76										
				5	SS	2	∇	75										
75.0	3.8	Sandy silt, some gravel, trace clay (FILL)		6/7	SS	2		74										
		Grey																
		Moist																
		Silty clay, some sand, trace gravel (FILL)																
		Grey																
		Moist																
		TOPSOIL		8/9	SS	17												
		CLAY (Weathered Crust)																
		Very stiff																
		Grey brown																
		Moist																
		End of Borehole																
		Note: Water level in open borehole at 3.4m depth below ground surface upon completion of drilling.																

PROJECT 05-1120-210-2000			RECORD OF BOREHOLE No 06-137			1 OF 1			METRIC										
W.P. 4058-01-00			LOCATION N 5027507.3; E 364454.5			ORIGINATED BY R.I.													
DIST HWY 417			BOREHOLE TYPE Portable Auger			COMPILED BY J.M.													
DATUM Geodetic			DATE July 10, 2006			CHECKED BY M.I.C.													
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED			WATER CONTENT (%)			γ			GR SA SI CL		
79.3	GROUND SURFACE							20 40 60 80 100											
0.0	Topsoil (FILL)																		
0.1	Moist Sand, trace silt (FILL)		1	SS	2		79												
	Loose Brown Moist		2	SS	3														
			3	SS	3		78												
			4	SS	2														
			5	SS	3		77												
76.4	Silty sand, some gravel with organic matter (FILL)																		
2.9	Loose to compact Dark brown Moist		6	SS	16		76												
75.6	Silty sand (FILL)																		
3.7	Loose Brown Moist to wet		7	SS	6														
75.2	Organic silty CLAY																		
75.0	Grey green Moist to wet						75												
4.3	Clayey SILT, trace sand with organic matter		8	SS	13														
74.4	Compact Grey black Moist to wet																		
5.0	Silty SAND with organic matter		9	SS	24		74												
	Grey brown Moist to wet																		
	CLAY Stiff to very stiff		10	SS	11														
	Grey Wet																		
73.2	End of Borehole																		
6.1																			



MISS_MTO 05-1120-210-8000.GPJ ON MOT.GDT 4/10/07

PROJECT 05-1120-210-2000		RECORD OF BOREHOLE No 06-138		1 OF 2		METRIC												
W.P. 4058-01-00		LOCATION N 5027626.2; E 364561.4		ORIGINATED BY D.G.														
DIST HWY 417		BOREHOLE TYPE Power Auger 108 mm I.D. Hollow Stem Auger		COMPILED BY J.M.														
DATUM Geodetic		DATE July 31, 2006		CHECKED BY M.I.C.														
SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)						
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER			TYPE	"N" VALUES						20	40	60	80	100	25
76.2 0.0	GROUND SURFACE Sand, trace silt (FILL) Light brown Moist																	
75.3 0.9	Silty clay (FILL) Grey Moist		1/2/3	SS	20													
74.9 1.4	ASPHALTIC CONCRETE Medium sand, trace gravel (FILL) Dark grey Moist																	
74.2 2.0	TOPSOIL Moist		4/5	SS	37													
73.9 2.3	Silty CLAY to clayey SILT Grey green Moist																	
73.6 2.6	Silty CLAY (Weathered Crust) Stiff to very stiff Grey brown Moist		6/7	SS	7													
			8	SS	3													
			9	SS	1													
71.9 4.3	Silty CLAY Firm to stiff Grey Moist																	
69.9 6.3	Sandy SILT, some gravel, trace clay with cobbles and boulders (TILL) Loose to dense Grey Wet		10	SS	2													
			11	SS	41													
			12	SS	42													
			13	SS	6													
			14	SS	39													
66.3																		

Continued Next Page

+³, X³: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

MISS_MTO 05-1120-210-5000.GPJ ON MOT.GDT 5/17/07

PROJECT 05-1120-210-2000			RECORD OF BOREHOLE No 06-138			2 OF 2			METRIC													
W.P. 4058-01-00			LOCATION N 5027626.2; E 364561.4			ORIGINATED BY D.G.																
DIST HWY 417			BOREHOLE TYPE Power Auger 108 mm I.D. Hollow Stem Auger			COMPILED BY J.M.																
DATUM Geodetic			DATE July 31, 2006			CHECKED BY M.I.C.																
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)							
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa 20 40 60 80 100 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED					WATER CONTENT (%) 25 50 75			γ kN/m³			GR SA SI CL			
--- CONTINUED FROM PREVIOUS PAGE ---																						
9.9	Coarse SAND and GRAVEL Dense Grey Wet		15	SS	38		66															
			16	SS	50																	
			17	SS	46																	
64.0																						
12.2	Sandy SILT, some gravel, trace clay with cobbles and boulders (TILL)		18	SS			64															
63.7	Grey Wet																					
12.5	End of Borehole Auger Refusal Note: Water level in open borehole at 4.6m depth below ground surface upon completion of drilling.																					

MISS_MTO 05-1120-210-6000.GPJ ON_MOT.GDT 4/10/07

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

PROJECT		RECORD OF BOREHOLE No 06-139		2 OF 2		METRIC													
W.P. 05-1120-210-2000		LOCATION N 5027686.0; E 364595.5		ORIGINATED BY D.G.															
DIST HWY 417		BOREHOLE TYPE Power Auger 108 mm I.D. Hollow Stem Auger		COMPILED BY J.M.															
DATUM Geodetic		DATE Aug. 1, 2006		CHECKED BY M.I.C.															
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa			WATER CONTENT (%)			Y			GR SA SI CL		
	--- CONTINUED FROM PREVIOUS PAGE ---							20 40 60 80 100 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED			25 50 75 W _p W W _L			25 50 75 W _p W W _L					
63.7	Silty SAND, some gravel, trace clay with cobbles and boulders (GLACIAL TILL) Loose to compact Grey Wet		11	SS	16		65												
63.2	Medium SAND Very dense Grey Wet		12	SS	11		64												
61.8	Silty SAND, some gravel, trace clay with cobbles and boulders (TILL) Compact Grey Wet		13	SS	52		63												
61.6	Fine SAND Very dense Grey Wet		14	SS	13		62												
61.6	Sandy SILT, some gravel, trace clay with cobbles and boulders (TILL) Compact Grey Wet End of Borehole Auger Refusal		15	SS	83		62												
13.5	Note: Water level in well screen at 3.6m depth below ground surface on Aug. 22, 2006.																		

MISS_MTO 05-1120-210-6000.GPJ ON MOT.GDT 4/10/07

PROJECT 05-1120-210-2000		RECORD OF BOREHOLE No 06-140		1 OF 2	METRIC
W.P. 4058-01-00		LOCATION N 5027733.3; E 364607.9		ORIGINATED BY D.G.	
DIST HWY 417		BOREHOLE TYPE Power Auger 108 mm I.D. Hollow Stem Auger		COMPILED BY J.M.	
DATUM Geodetic		DATE Aug. 2, 2006		CHECKED BY M.I.C.	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa	WATER CONTENT (%)					
74.3	GROUND SURFACE													
0.0	ASPHALTIC CONCRETE													
0.1	Sandy gravel (BASE) Brown Wet		1	A.S.										
73.3														
1.0	Silty CLAY (Weathered Crust) Stiff Grey brown Moist		2	SS	4									
			3	SS	3									
72.0														
2.3	CLAY Firm Grey Wet													
			4	TP	WH									
			5	SS	WH									
			6	SS	WH									
			7	SS	17									
			8	SS	13									
			9	SS	6									
67.2														
7.1	Sandy SILT, some gravel, trace clay with cobbles and boulders (TILL) Loose to compact Grey Wet													

MISS_MTO 05-1120-210-6000.GPJ ON MOT.GDT 4/10/07

Continued Next Page

+³, X³: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

PROJECT <u>05-1120-210-2000</u>		RECORD OF BOREHOLE No 06-140		2 OF 2	METRIC
W.P. <u>4058-01-00</u>	LOCATION <u>N 5027733.3; E 364607.9</u>	ORIGINATED BY <u>D.G.</u>			
DIST <u>HWY 417</u>	BOREHOLE TYPE <u>Power Auger 108 mm I.D. Hollow Stem Auger</u>	COMPILED BY <u>J.M.</u>			
DATUM <u>Geodetic</u>	DATE <u>Aug. 2, 2006</u>	CHECKED BY <u>M.I.C.</u>			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa										WATER CONTENT (%)		
								20 40 60 80 100										25 50 75		
— CONTINUED FROM PREVIOUS PAGE —																				
63.6	Sandy SILT, some gravel, trace clay with cobbles and boulders (TILL) Loose to compact Grey Wet		10	SS	32		64													
10.7	Medium to coarse SAND, some gravel, trace silt and clay Dense Grey Wet		11	SS	13		63													
			12	SS	89															
62.1	Silty SAND, some gravel with cobbles and boulders (TILL) Dense to very dense Grey Wet		13	SS	42		62													
12.2			14	SS	41		61													
			15	SS	60															
			16	SS	112		60													
59.2	End of Borehole Auger Refusal																			
15.1	Note: Water level in open borehole at 3.1m depth below ground surface upon completion of drilling.																			

MISS_MTO_05-1120-210-6000.GPJ ON MOT_GDT 4/10/07

PROJECT		RECORD OF BOREHOLE		No 06-141		1 OF 1		METRIC	
W.P.		LOCATION		ORIGINATED BY		D.G.			
DIST		BOREHOLE TYPE		COMPILED BY		J.M.			
DATUM		DATE		CHECKED BY		M.I.C.			
PROJECT 05-1120-210-2000		N 5027807.7; E 364671.7							
4058-01-00		Power Auger 108 mm I.D. Hollow Stem Auger							
HWY 417		Aug. 3, 2006							
Geodetic									

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa					WATER CONTENT (%)			
						20 40 60 80 100 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100					25 50 75 W _p W W _L					
74.7	GROUND SURFACE															
0.0	ASPHALTIC CONCRETE															
0.2	Medium gravel (BASE)															
74.1	Brown Silty clay (FILL) Grey brown															
0.7	TOPSOIL Silty CLAY (Weathered Crust) Stiff Grey brown Moist		1	SS	12											
			2	SS	8											
			3	SS	3											
71.4	Silty CLAY Firm to stiff Grey Moist to wet		4	SS	1											
3.4																
			5	TP	WH											
			6	SS	WH											
67.1	Sandy SILT, some gravel, trace clay with cobbles and boulders (TILL) Very loose Grey Wet		7	SS	1											
7.6																
66.5	End of Borehole															
8.2	Note: Water level in open borehole at 4.6m depth below ground surface upon completion of drilling.															

MISS_MTO 05-1120-210-6000.GPJ ON_MOT_GDI 4/10/07



1 OF 1

METRIC

ORIGINATED BY D.G.

COMPILED BY J.M.

CHECKED BY M.I.C.

MISS_MTO 05-1120-210-6000.GPJ ON MOT.GDT 4/10/07

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

PROJECT		RECORD OF BOREHOLE		No 06-143		1 OF 1		METRIC											
W.P. 4058-01-00		LOCATION		N 5027983.9; E 364775.9		ORIGINATED BY		D.J.S.											
DIST HWY 417		BOREHOLE TYPE		Power Auger 108 mm I.D. Hollow Stem Auger		COMPILED BY		J.M.											
DATUM Geodetic		DATE		July 4, 2006		CHECKED BY		M.I.C.											
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED			WATER CONTENT (%) w _p — w — w _L			γ			GR SA SI CL		
73.4	GROUND SURFACE							20 40 60 80 100											
0.0	Sand and gravel (FILL)																		
73.2	Brown																		
72.9	Silty clay, trace gravel (FILL)																		
	Grey brown																		
	TOPSOIL																		
72.6	Sandy SILT																		
0.8	Light brown																		
	Moist																		
	Silty CLAY (Weathered Crust)		1	SS	8														
	Very stiff																		
	Grey brown																		
	Moist to wet																		
			2	SS	7														
			3	SS	2														
70.7	Silty CLAY, some sand (Weathered Crust)																		
2.7	Stiff																		
70.4	Grey brown																		
3.1	Wet																		
	Silty CLAY (Weathered Crust)		4	SS	WH														
	Firm																		
69.7	Grey brown																		
3.7	Wet																		
	Silty CLAY																		
	Firm																		
	Grey																		
	Wet																		
			5	TP	PH														
67.9	Sandy SILT, some gravel and clay																		
5.6	(TILL)																		
	Grey																		
	Wet																		
67.3																			
6.1	End of Borehole		6	SS	100														
	Auger Refusal																		
	Note: Water level in well screen at 3.0m depth below ground surface on Aug. 22, 2006.																		

PROJECT 05-1120-210-2000		RECORD OF BOREHOLE No 06-143A		1 OF 1		METRIC								
W.P. 4058-01-00		LOCATION N 5027980.8; E 364776.0		ORIGINATED BY J.A.S.										
DIST HWY 417		BOREHOLE TYPE Power Auger 108 mm I.D. Hollow Stem Auger		COMPILED BY J.M.										
DATUM Geodetic		DATE Feb. 2, 2007		CHECKED BY M.I.C.										
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 20 40 60 80 100						
73.4 0.0	GROUND SURFACE See Record of Borehole 06-143 for soil description.													
						73								
						72								
						71								
						70								
			1	73 TP	PH									
			2	73 TP	PH									
						69								
						68								
67.6 5.8	End of Borehole													

PROJECT		RECORD OF BOREHOLE		No 06-144		1 OF 1		METRIC					
W.P.		LOCATION		N 5028028.8; E 364786.5		ORIGINATED BY		D.J.S.					
DIST		BOREHOLE TYPE		Power Auger 108 mm I.D. Hollow Stem Auger		COMPILED BY		J.M.					
DATUM		DATE		July 5, 2006		CHECKED BY		M.I.C.					
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			REMARKS & GRAIN SIZE DISTRIBUTION (%)				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	20 40 60 80 100	PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	GR SA SI CL
73.5	GROUND SURFACE												
0.0	ASPHALTIC CONCRETE												
0.1	Crushed stone (BASE)												
73.1	Grey												
0.4	Fine to medium sand, trace gravel (FILL) Loose to compact Brown Moist		1	SS	21		73						
			2	SS	7		72						
71.3													
2.2	Silty CLAY with sand seams (Weathered Crust) Very stiff Grey brown Wet		3	SS	1		71						
			4	SS	WH		70						
69.8													
3.7	Silty CLAY with sand seams Firm Grey Wet						69	x	+				
								x	+				
68.8													
4.7	Sandy SILT, some gravel and clay (TILL) Loose to compact Grey Wet		5	TP	PH								
			6	SS	20		68						
			7	SS	19		67						
66.9													
6.6	End of Borehole Auger Refusal												

MISS_MTO 05-1120-210-6000.GPJ ON MOT.GDT 4/10/07

PROJECT		RECORD OF BOREHOLE No 06-145				1 OF 1		METRIC							
W.P. 4058-01-00		LOCATION N 5027219.6; E 364171.3		ORIGINATED BY H.E.C.											
DIST HWY 417		BOREHOLE TYPE Power Auger 108 mm I.D. Hollow Stem Auger		COMPILED BY J.M.											
DATUM Geodetic		DATE Aug. 8, 2006		CHECKED BY M.I.C.											
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x REMOULDED							
78.0	GROUND SURFACE														
0.0	Silty sand to fine sand, trace gravel and organic matter (FILL) Brown														
77.3															
0.7	Intermixed silty clay and silty sand (FILL) Grey brown Moist		1	SS	7										
76.7															
1.3	Sand, some silt and gravel (FILL) Loose Brown Moist to wet		2	SS	7										
			3	SS	7										
75.0															
3.1	Crushed stone (FILL) Compact Grey Wet		4	SS	12										
74.3															
3.7	Fine sand (FILL) Very loose Brown Wet		5	SS	2										
73.7															
4.3	Crushed stone (FILL) Compact Grey Wet														
73.4															
4.7	TOPSOIL Silty CLAY Stiff to very stiff Grey Wet		6	SS	10										
			7	SS	1										
72.2															
5.8	Sandy SILT, some gravel, trace clay (TILL) Loose to very dense Grey Wet		8	SS	5										
			9	SS	86										
70.7															
7.3	End of Borehole														

MISS MTO 05-1120-210-6000.GPJ ON MOT.GDT 4/10/07

APPENDIX C
FIGURES 1 TO 33

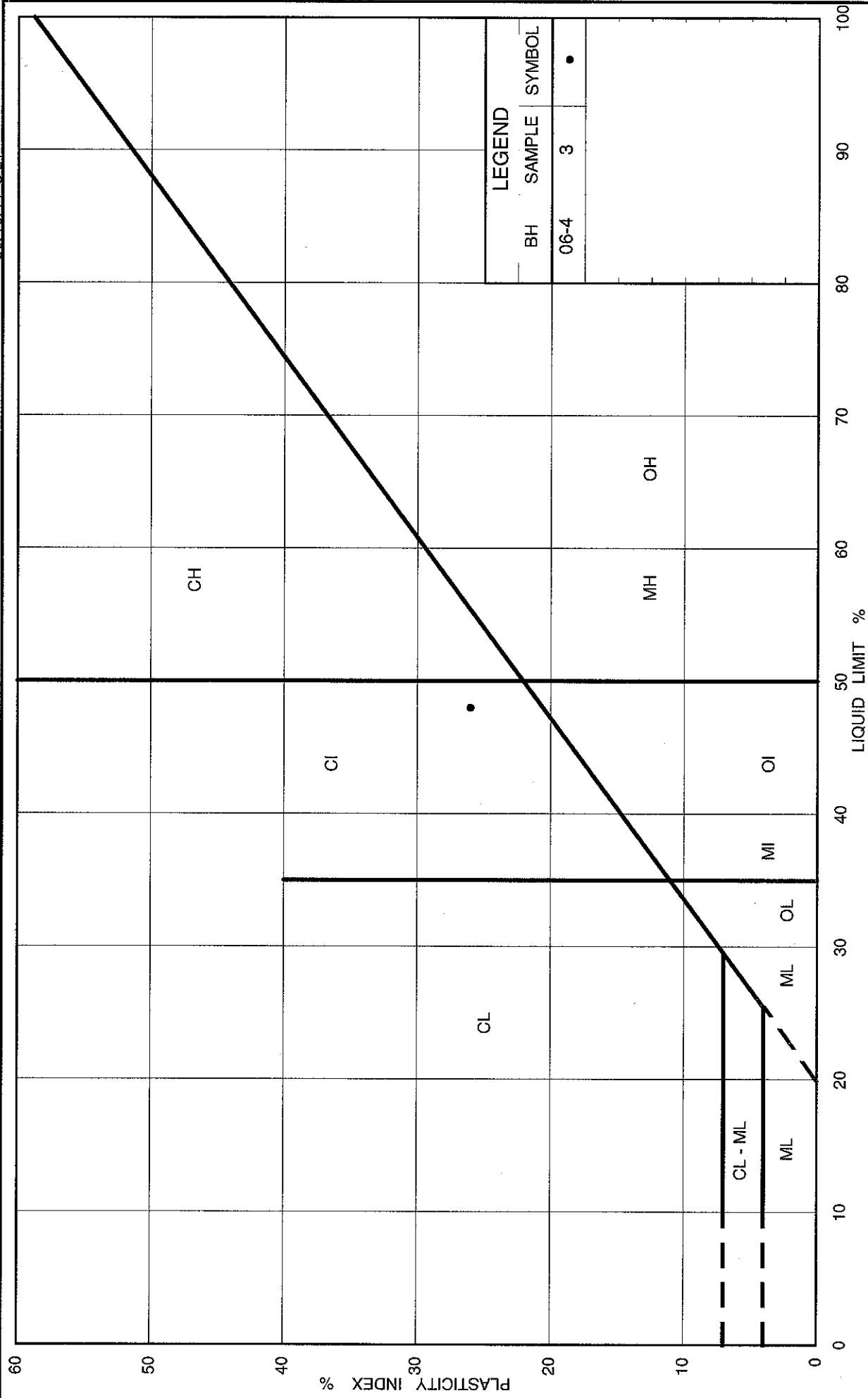
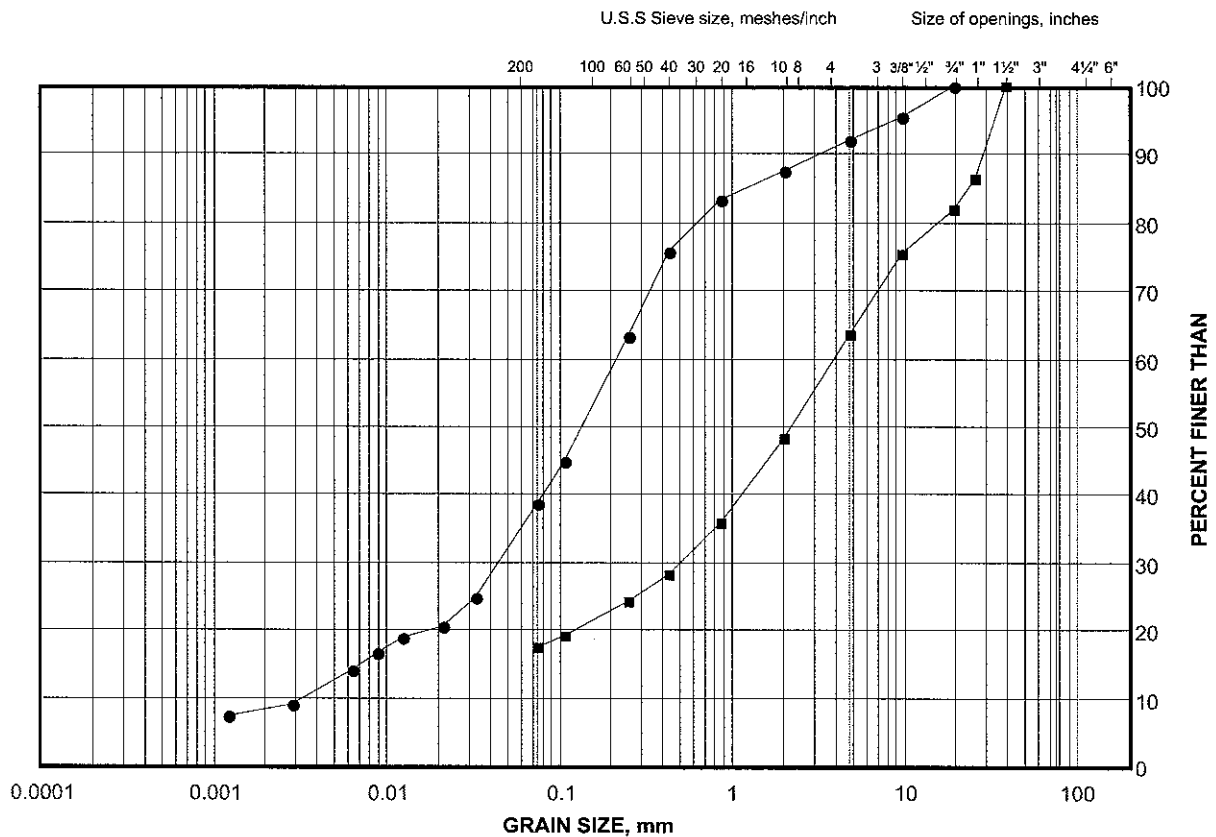


	FIG No. 1 Retaining Wall 1S	
	Project No. 05-1120-210 - 2000	

GRAIN SIZE DISTRIBUTION

Retaining Wall 2S and 3S - Till

FIGURE 2



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	DEPTH(m)
●	06-107	1	0.8-1.4
■	06-106	4	0.6-1.2

Project Number: 05-1120-210

Checked By: _____

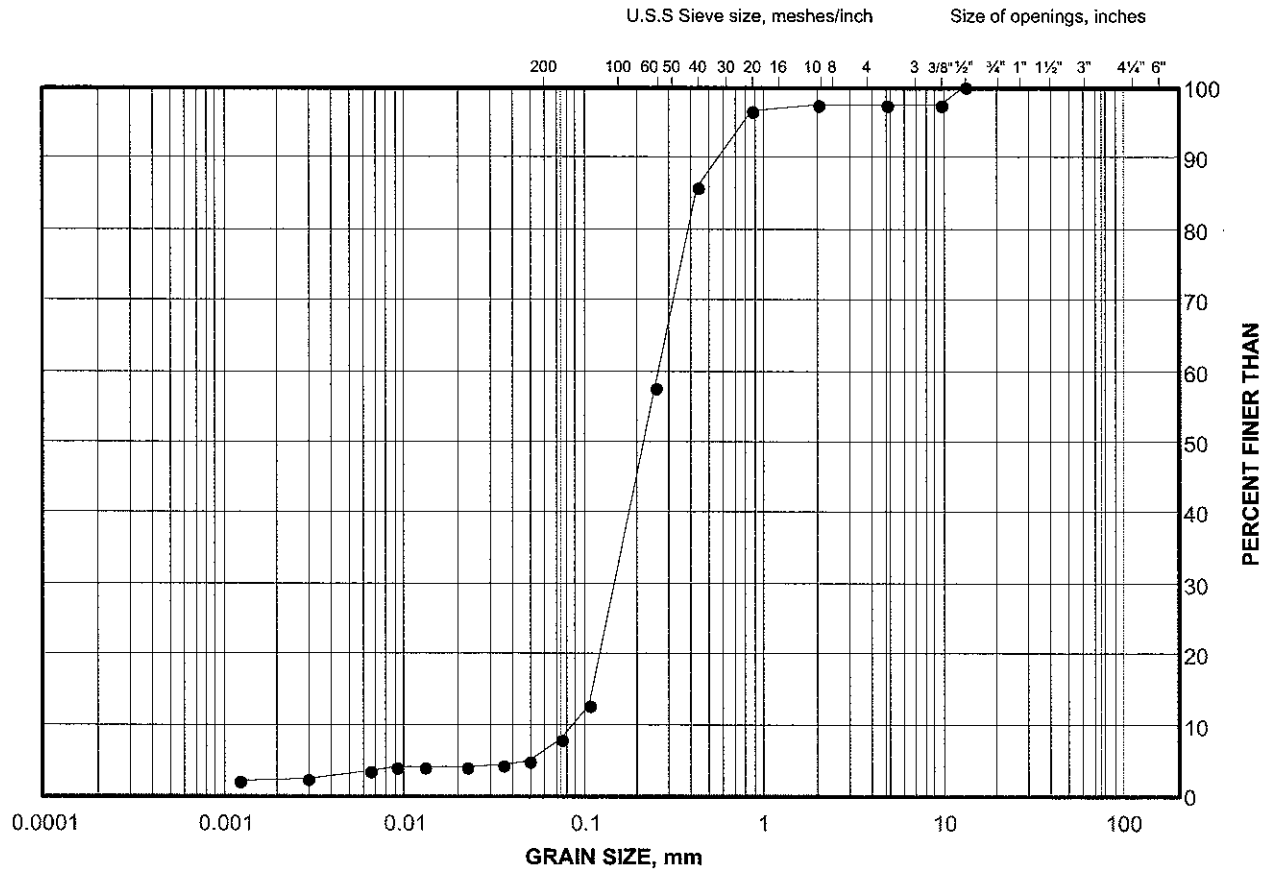
Golder Associates

Date: 27-Sep-06

GRAIN SIZE DISTRIBUTION

Retaining Wall 4S - Sand Fill

FIGURE 3



LEGEND

SYMBOL	BOREHOLE	SAMPLE	DEPTH(m)
•	06-3	1	0.8-1.1

Project Number: 05-1120-210

Checked By: _____

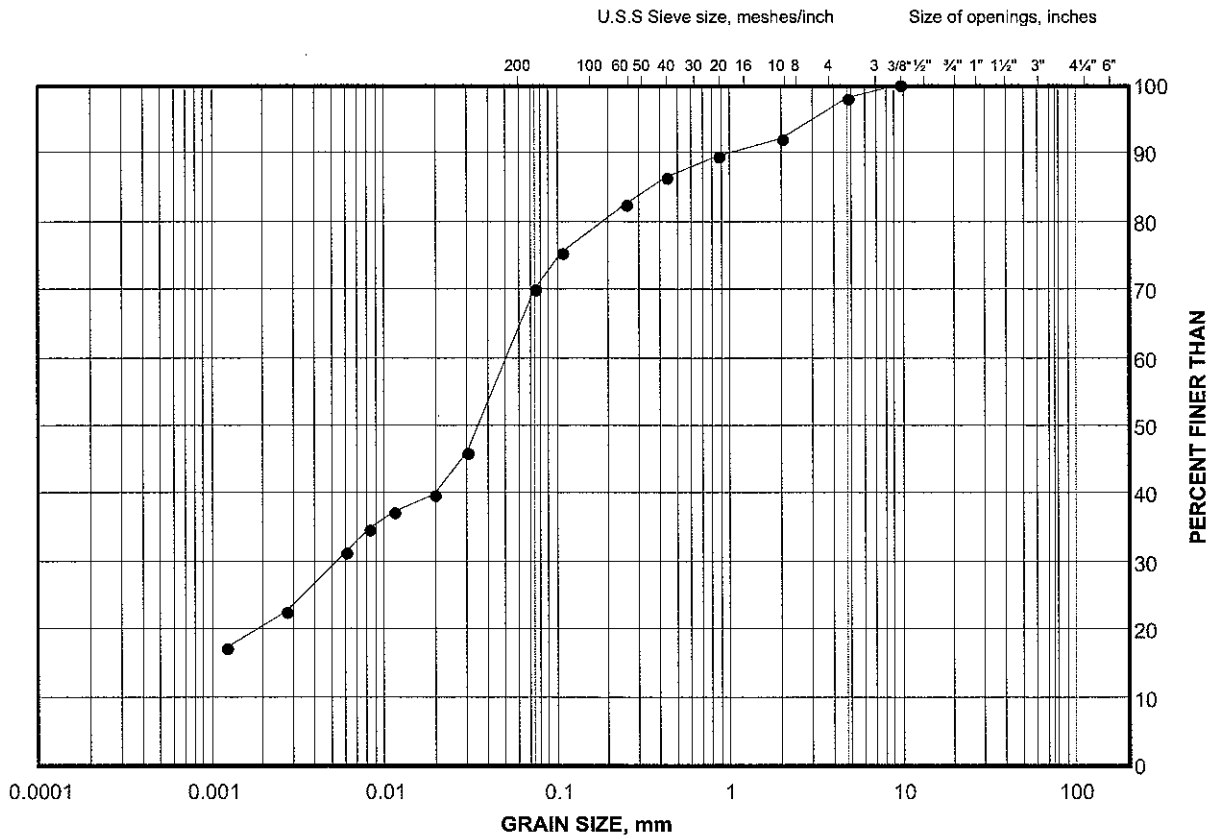
Golder Associates

Date: 08-Jun-06

GRAIN SIZE DISTRIBUTION

Retaining Wall 4S - Till

FIGURE 4



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	DEPTH(m)
•	06-111	3	0.8-1.1

Project Number: 05-1120-210

Checked By: _____

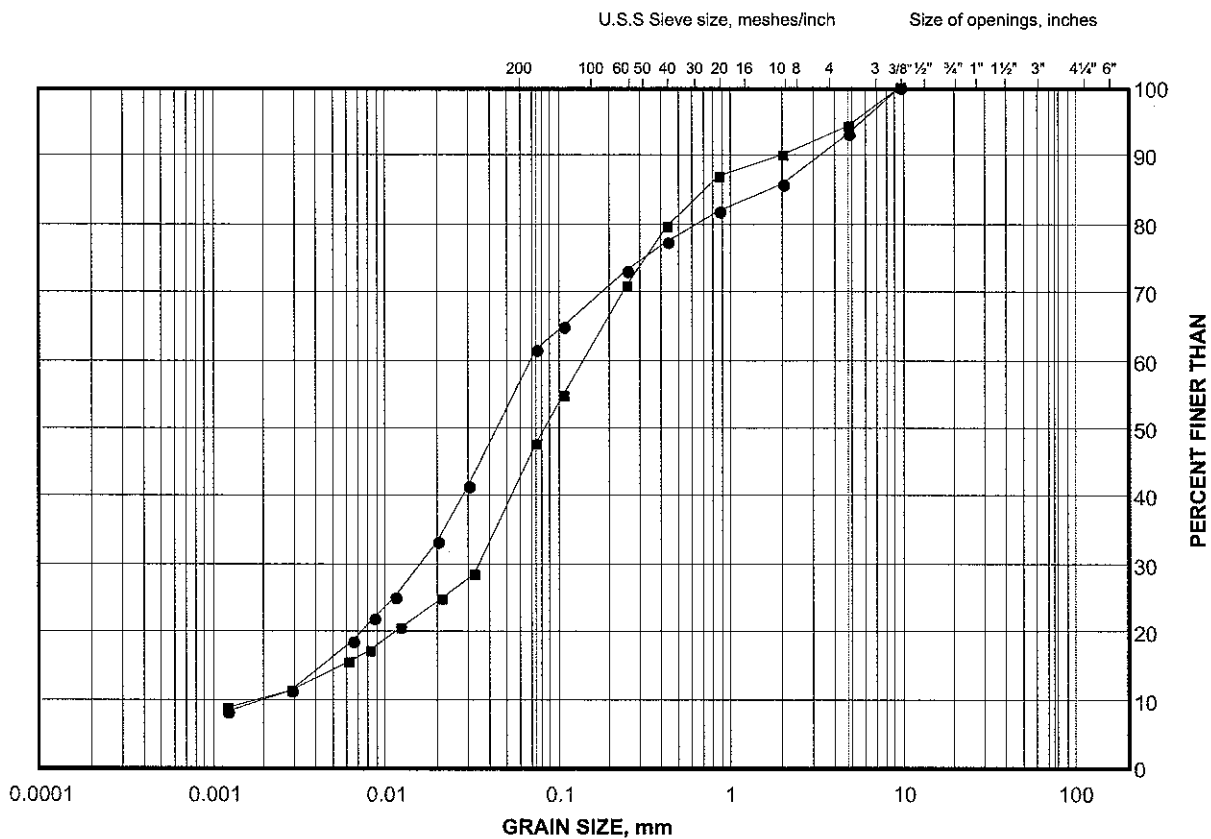
Golder Associates

Date: 27-Sep-06

GRAIN SIZE DISTRIBUTION

Retaining Wall 5S - Glacial Till

FIGURE 5



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	DEPTH(m)
•	06-119	1	0.8-1.1
■	06-122	3	0.8-1.4

Project Number: 05-1120-210

Checked By: _____

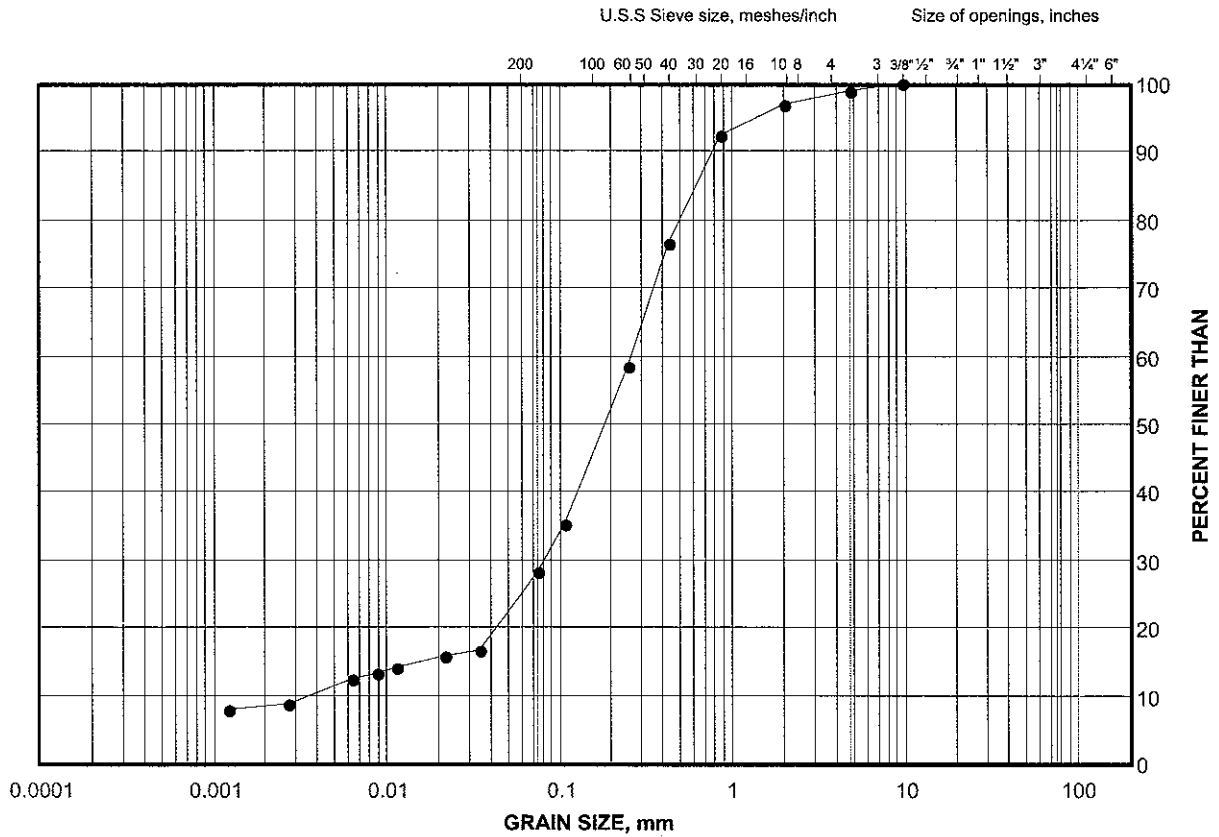
Golder Associates

Date: 27-Sep-06

GRAIN SIZE DISTRIBUTION

Retaining Wall 6S - Sand Fill

FIGURE 6



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	DEPTH(m)
•	06-129	1	0-0.6

Project Number: 05-1120-210

Checked By: _____

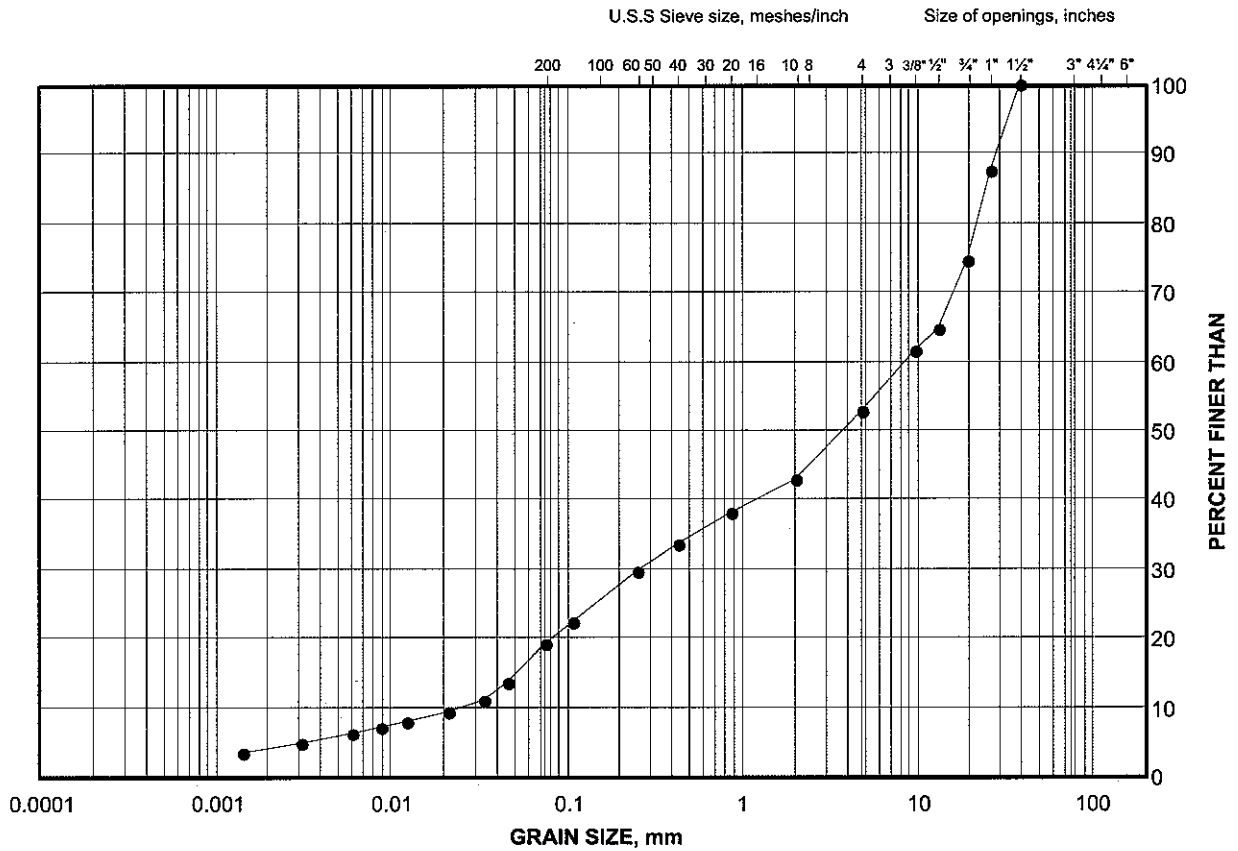
Golder Associates

Date: 27-Sep-06

GRAIN SIZE DISTRIBUTION

Retaining Wall 6S - Till

FIGURE 7



LEGEND

SYMBOL	BOREHOLE	SAMPLE	DEPTH(m)
•	06-6	4	1.70 - 2.10

Project Number: 05-1120-210

Checked By: _____

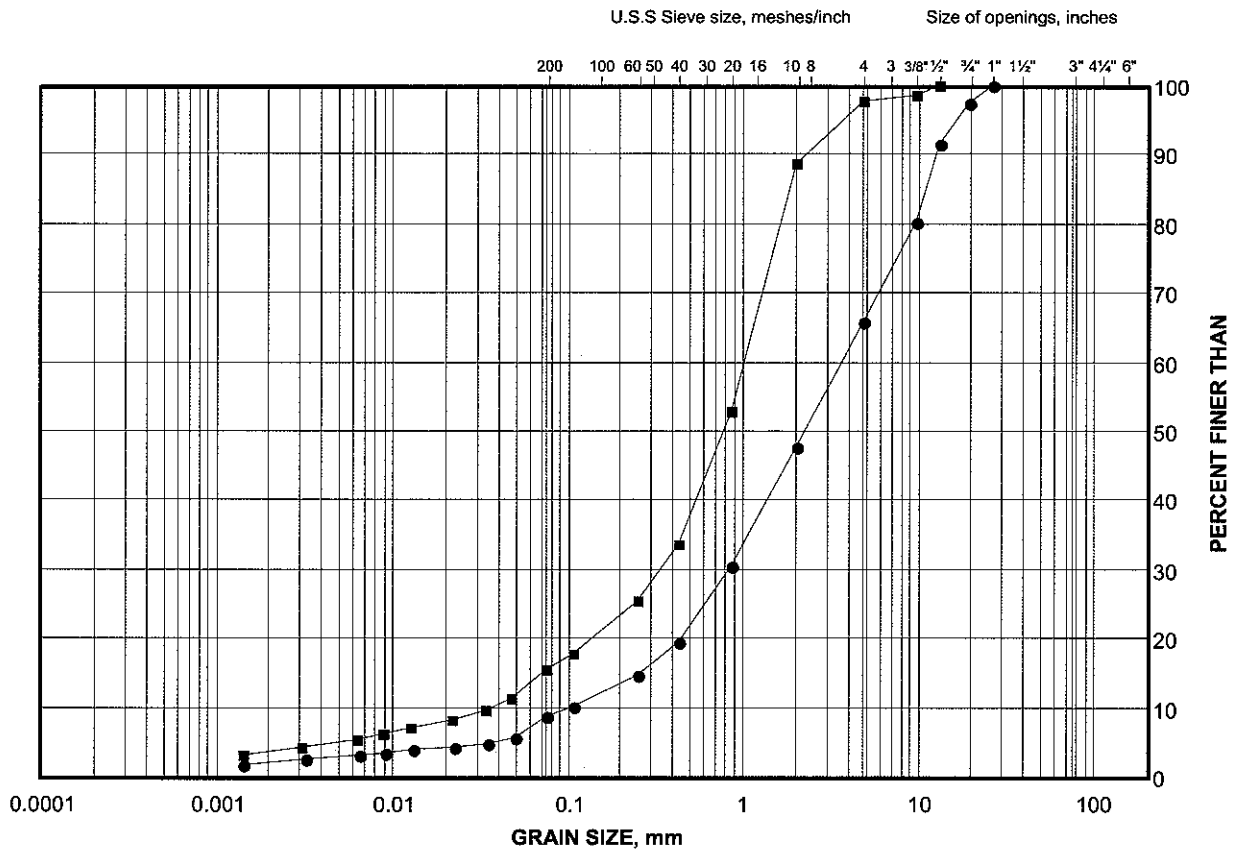
Golder Associates

Date: 16-Apr-07

GRAIN SIZE DISTRIBUTION

FIGURE 8

Retaining Wall 6S - Sand to Sandy Gravel



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	DEPTH(m)
•	06-10	6	3.80 - 4.40
■	06-10	9	6.10 - 6.70

Project Number: 05-1120-210

Checked By: _____

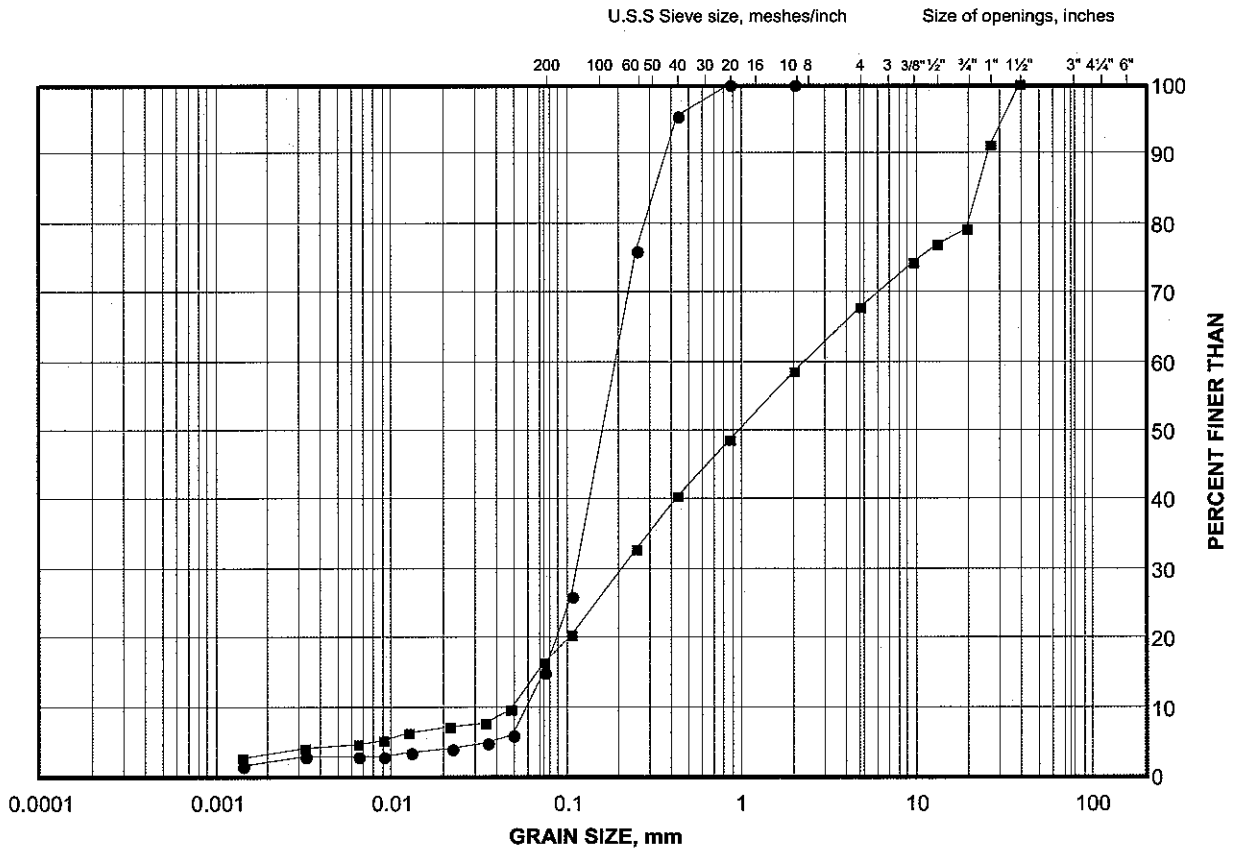
Golder Associates

Date: 16-Apr-07

GRAIN SIZE DISTRIBUTION

Retaining Wall 7S - Gravelly Sand and Sand

FIGURE 9



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	DEPTH(m)
●	06-9	10	6.10 - 6.70
■	06-9	8	4.60 - 5.20

Project Number: 05-1120-210

Checked By: _____

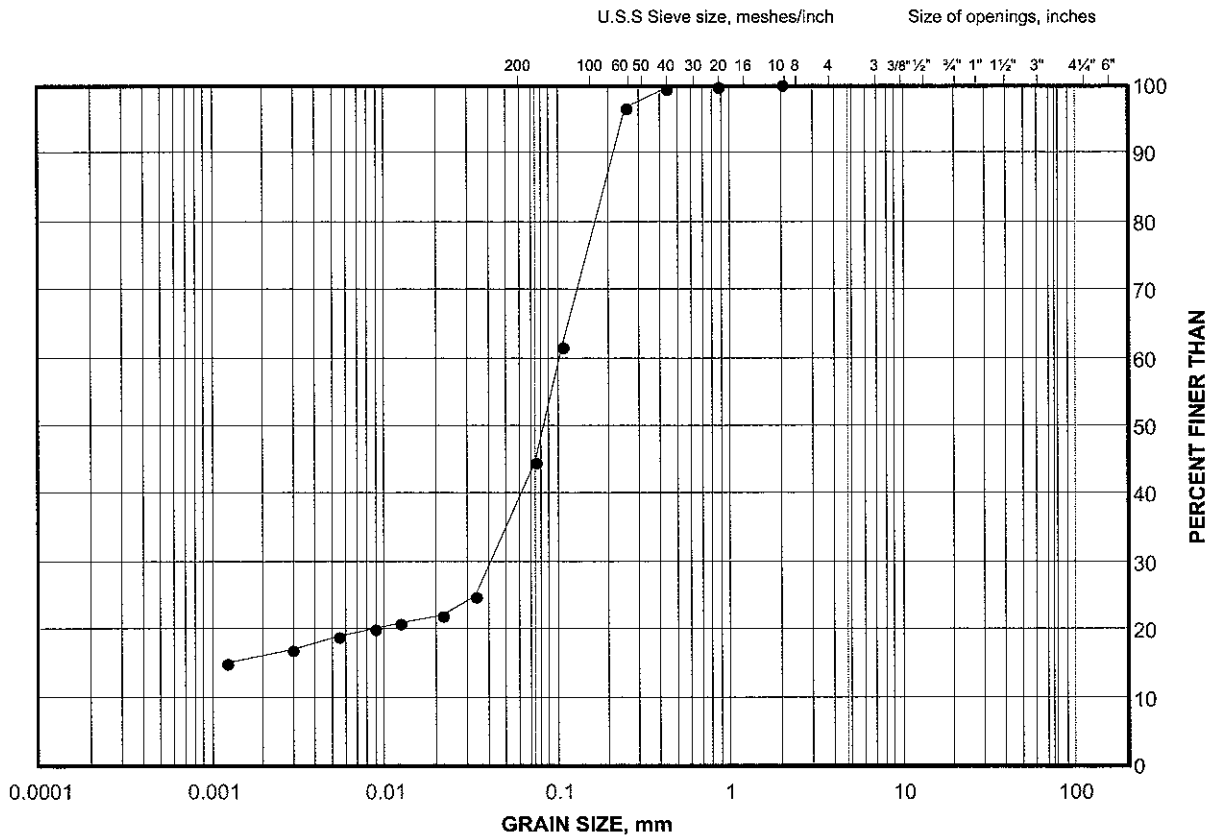
Golder Associates

Date: 16-Apr-07

GRAIN SIZE DISTRIBUTION

Retaining Wall 8S - Sand Fill

FIGURE 10



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	DEPTH(m)
•	06-134	3	1.5-2.1

Project Number: 05-1120-210

Checked By: _____

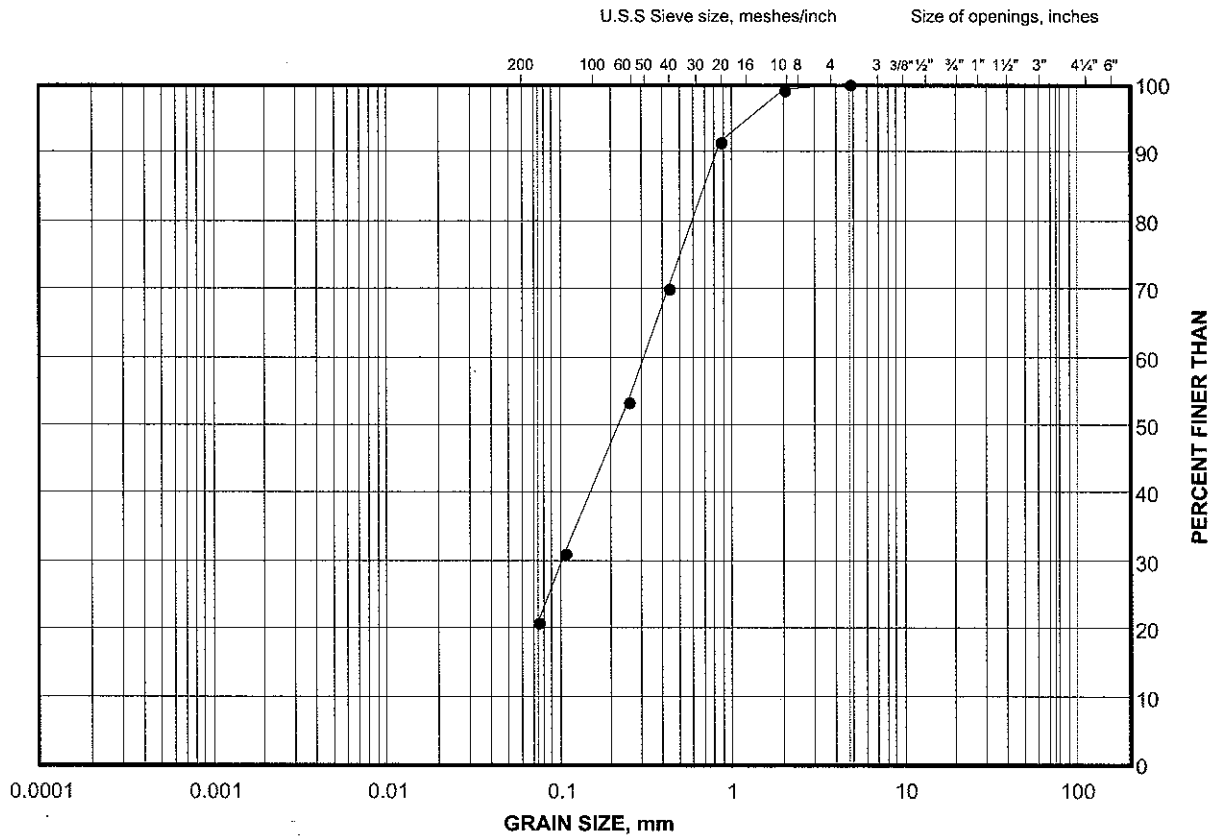
Golder Associates

Date: 27-Sep-06

GRAIN SIZE DISTRIBUTION

Retaining Wall 9S - Sand Fill

FIGURE 11



LEGEND

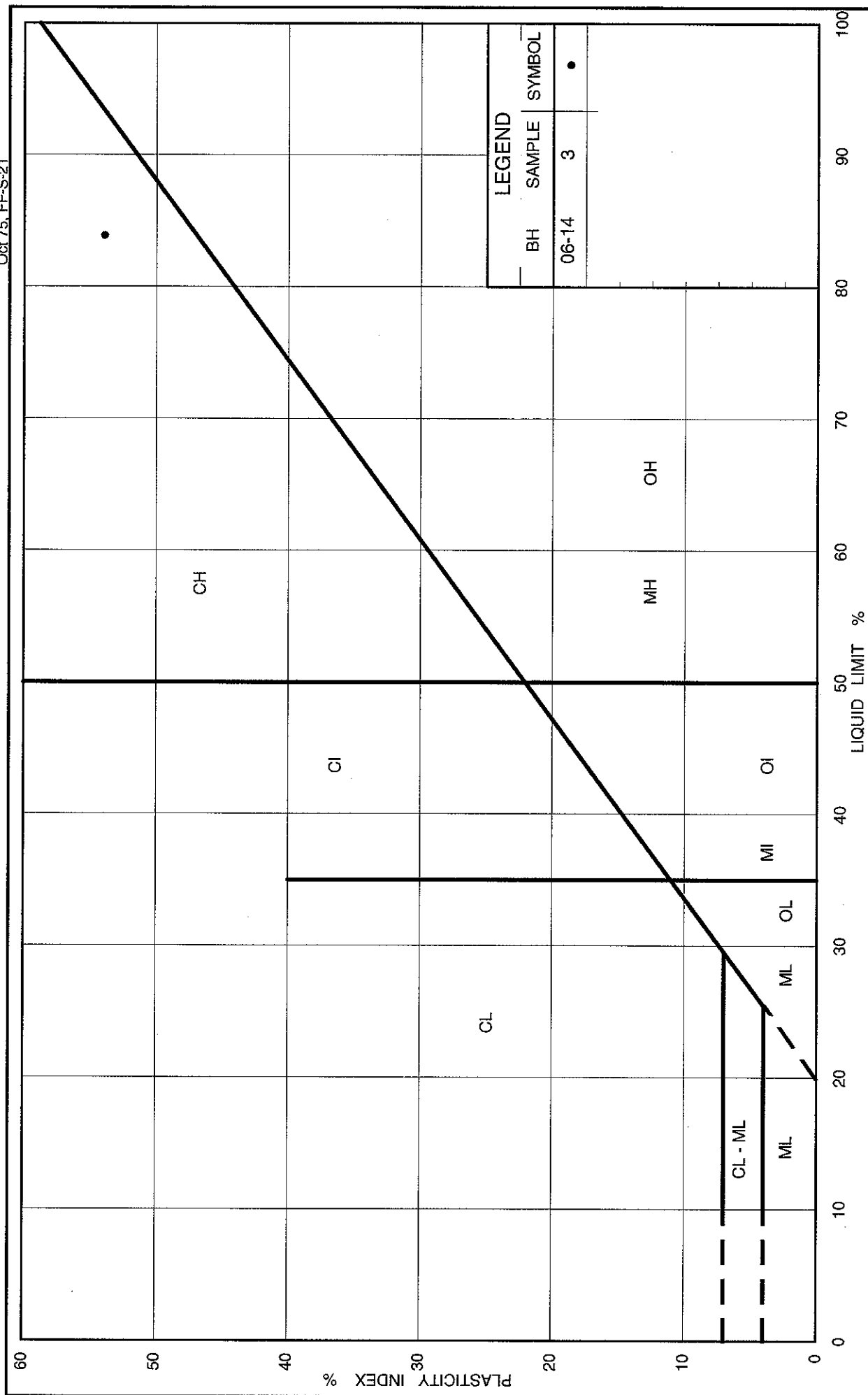
SYMBOL	BOREHOLE	SAMPLE	DEPTH(m)
•	06-136	2	0.8-1.4

Project Number: 05-1120-210

Checked By: _____

Golder Associates

Date: 27-Sep-06



LEGEND		
BH	SAMPLE	SYMBOL
06-14	3	•

PLASTICITY CHART Clay (Weathered Crust)

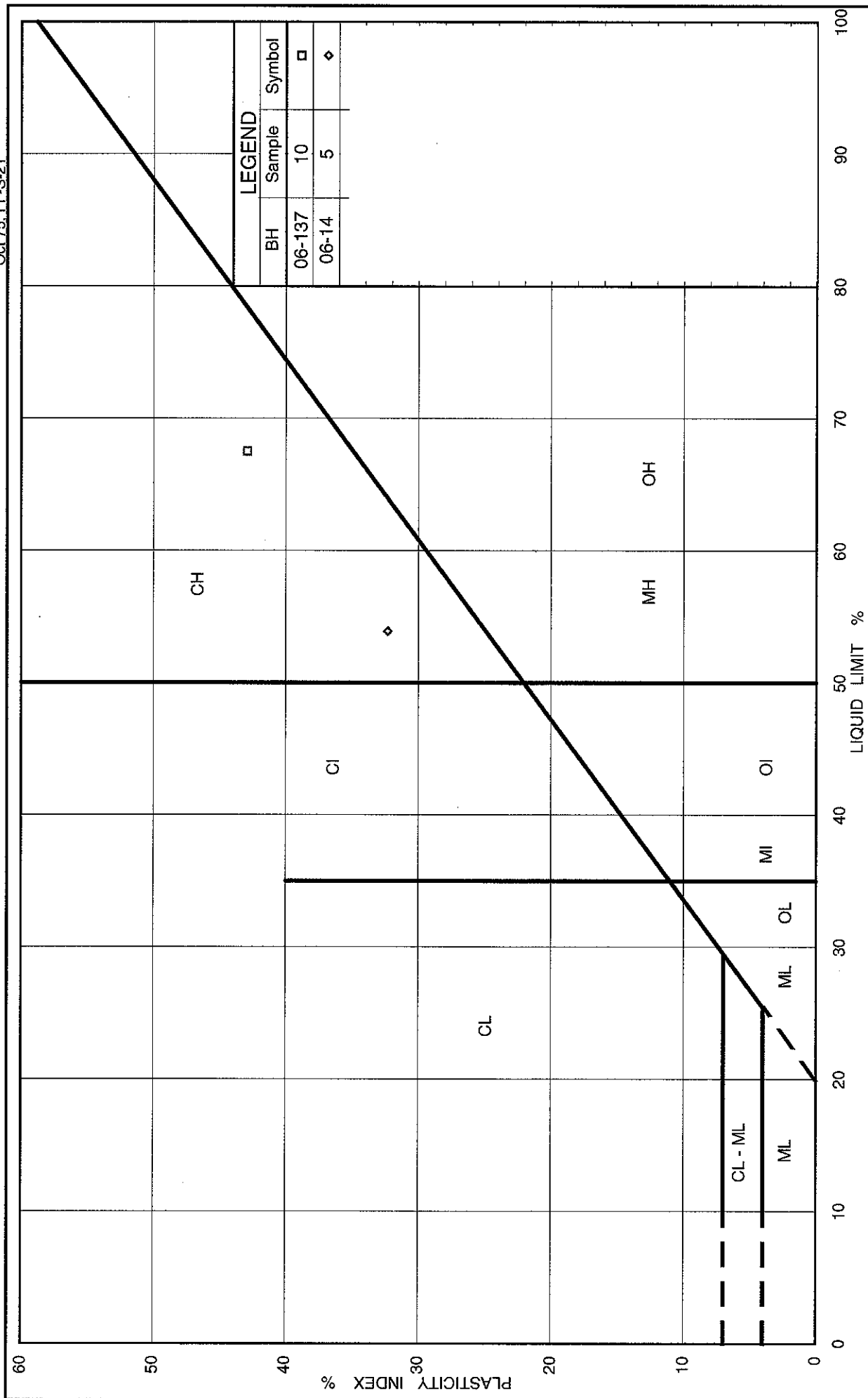
FIG No.12 - Retaining Wall 9S

Project No. 05-1120-210 - 2000

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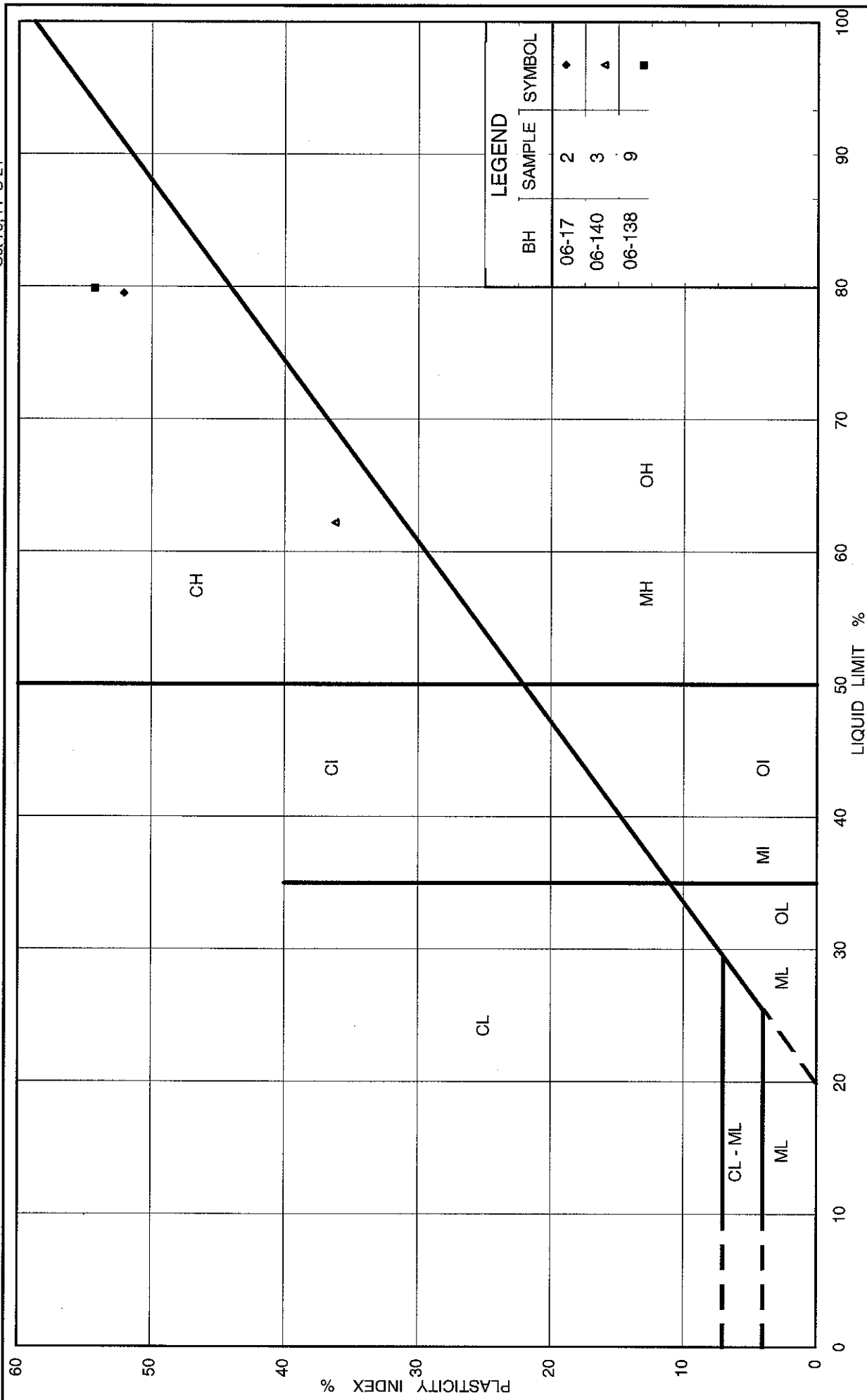


FIG No. 14 - Retaining Wall 11S

Project No. 05-1120-210 - 2000

PLASTICITY CHART **Silty Clay (Weathered Crust)**

Ministry of Transportation

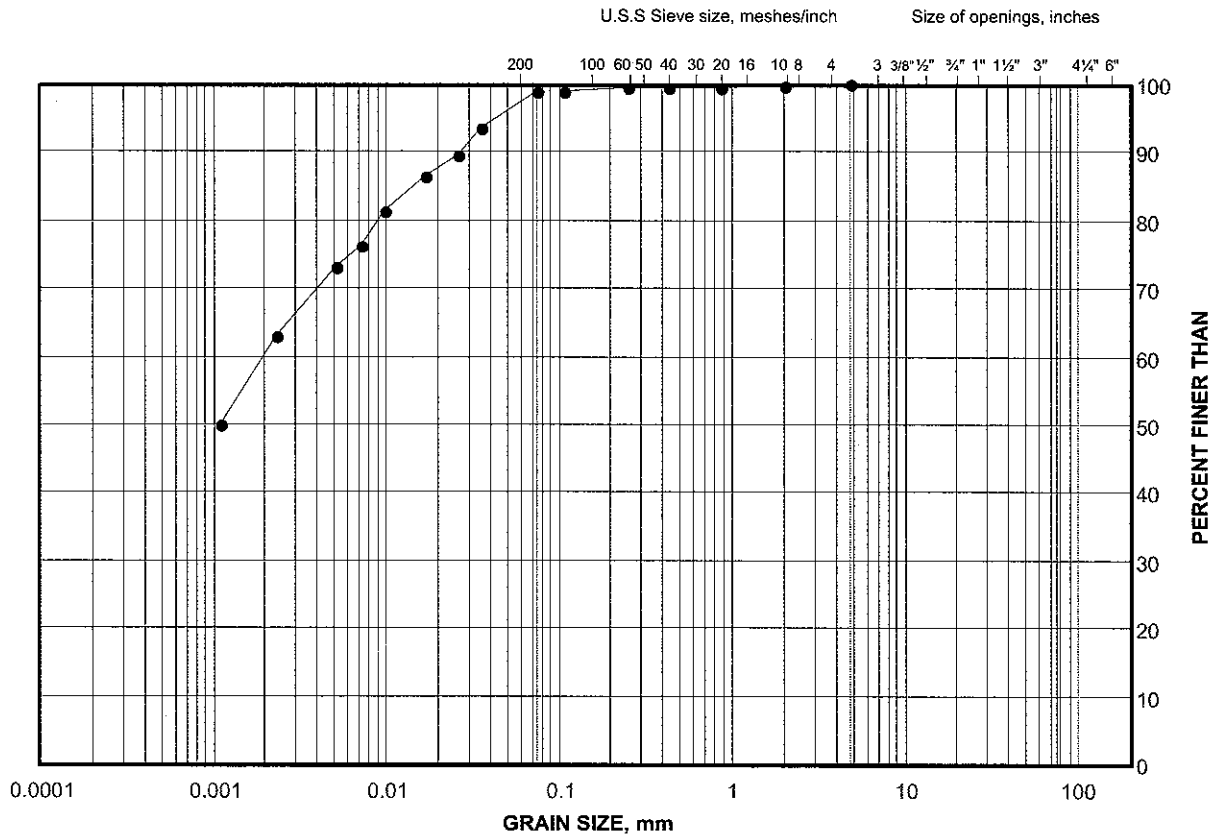


Ontario

GRAIN SIZE DISTRIBUTION

Retaining Wall 11S - Silty Clay

FIGURE 15



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

LEGEND

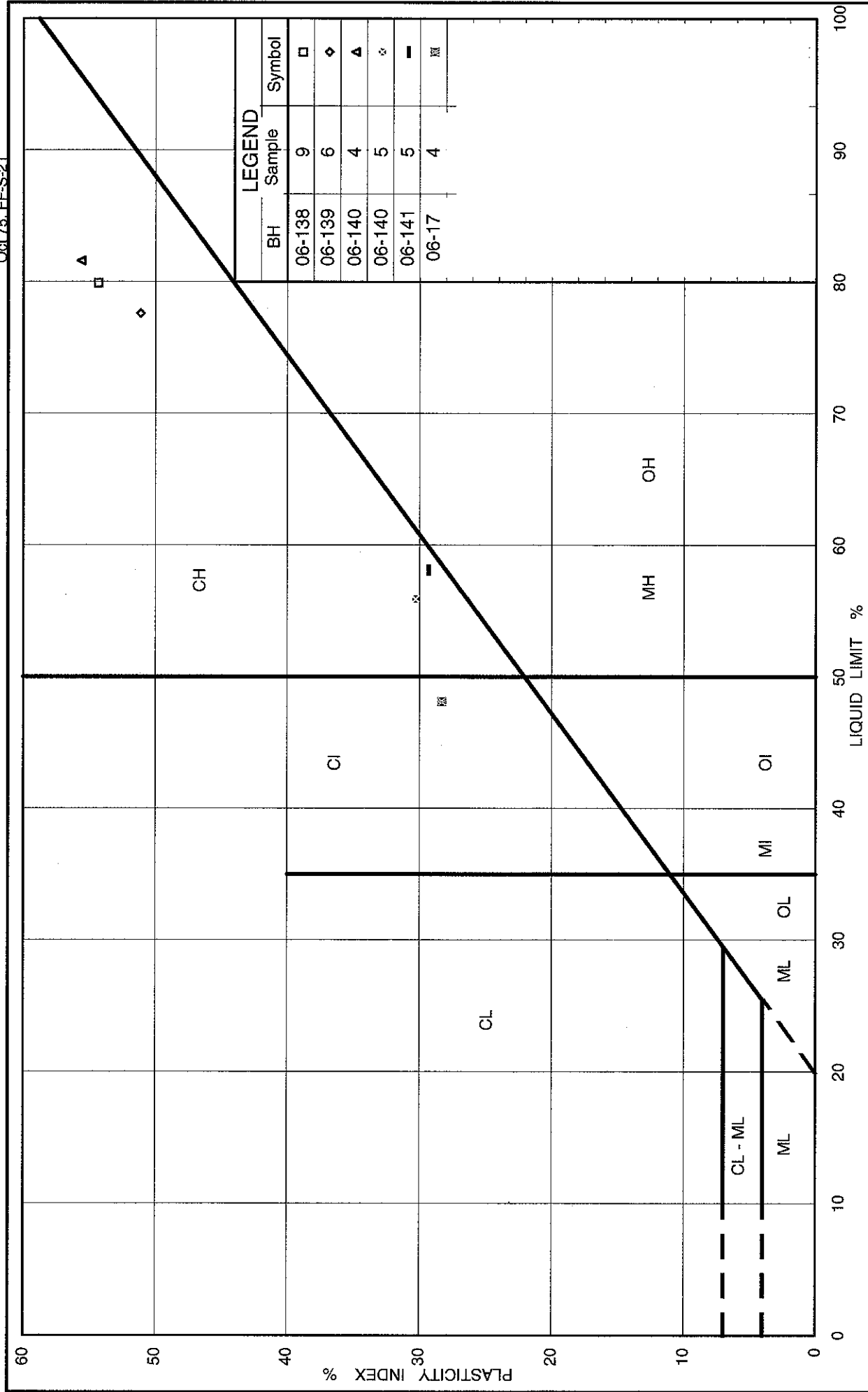
SYMBOL	BOREHOLE	SAMPLE	DEPTH(m)
•	06-17	4	4.4-5.0

Project Number: 05-1120-210

Checked By: _____

Golder Associates

Date: 08-Jun-06



PLASTICITY CHART Silty Clay to Clay

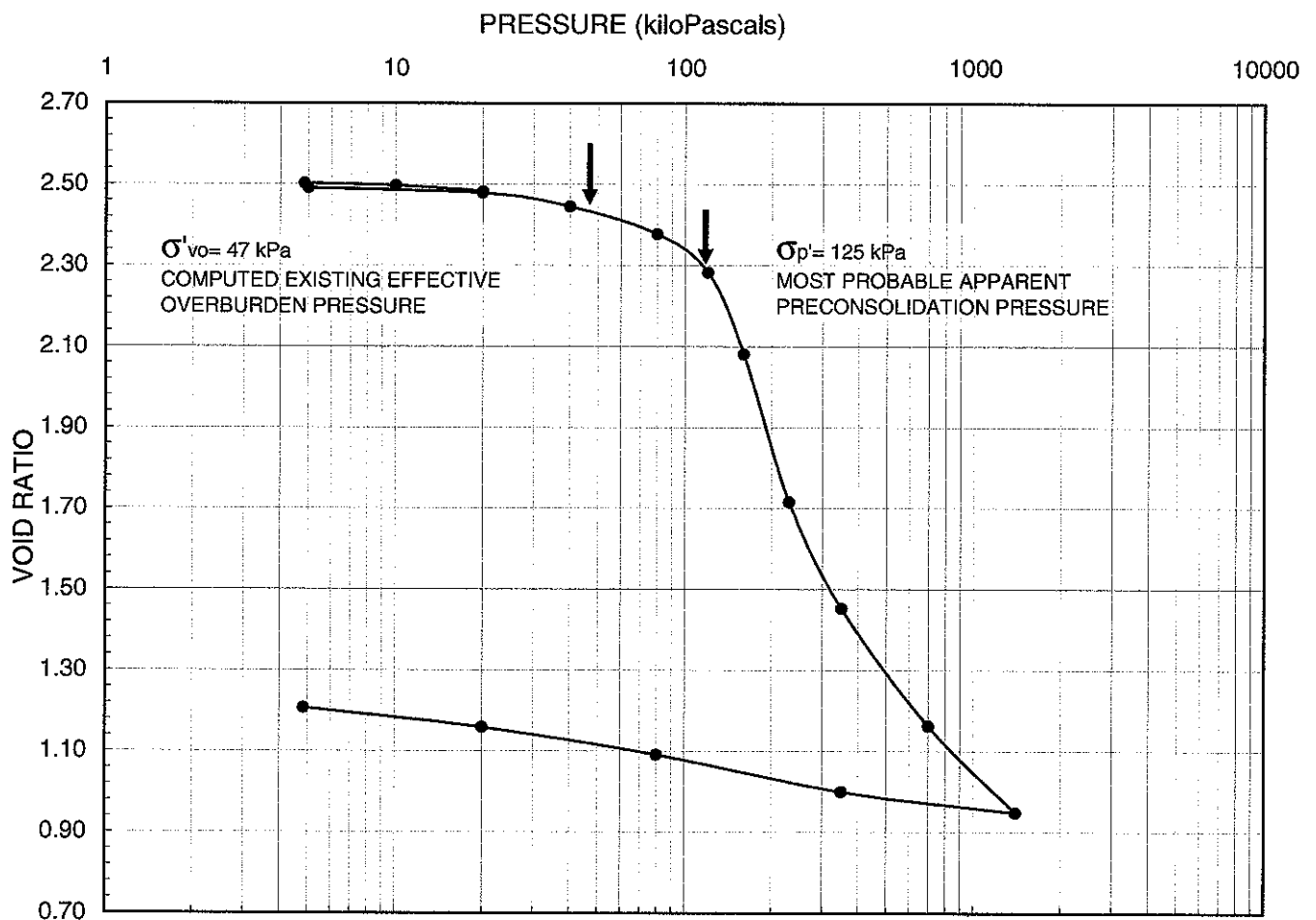
FIG No. 16 - Retaining Wall 11S

Project No. 05-1120-210

Ministry of Transportation



Ontario



LEGEND

Borehole: 06-140	$w_i = 86.6\%$	$S_o = 97\%$
Sample: 4	$w_f = 43.7\%$	$C_c = 2.32$
Depth (m): 3.30	$w_l = 81.6\%$	$C_r = 0.022$
	$w_p = 26.0\%$	



SCALE	AS SHOWN
DATE	05/15/07
DESIGN	NA
CADD	NA

TITLE

CONSOLIDATION TEST RESULTS

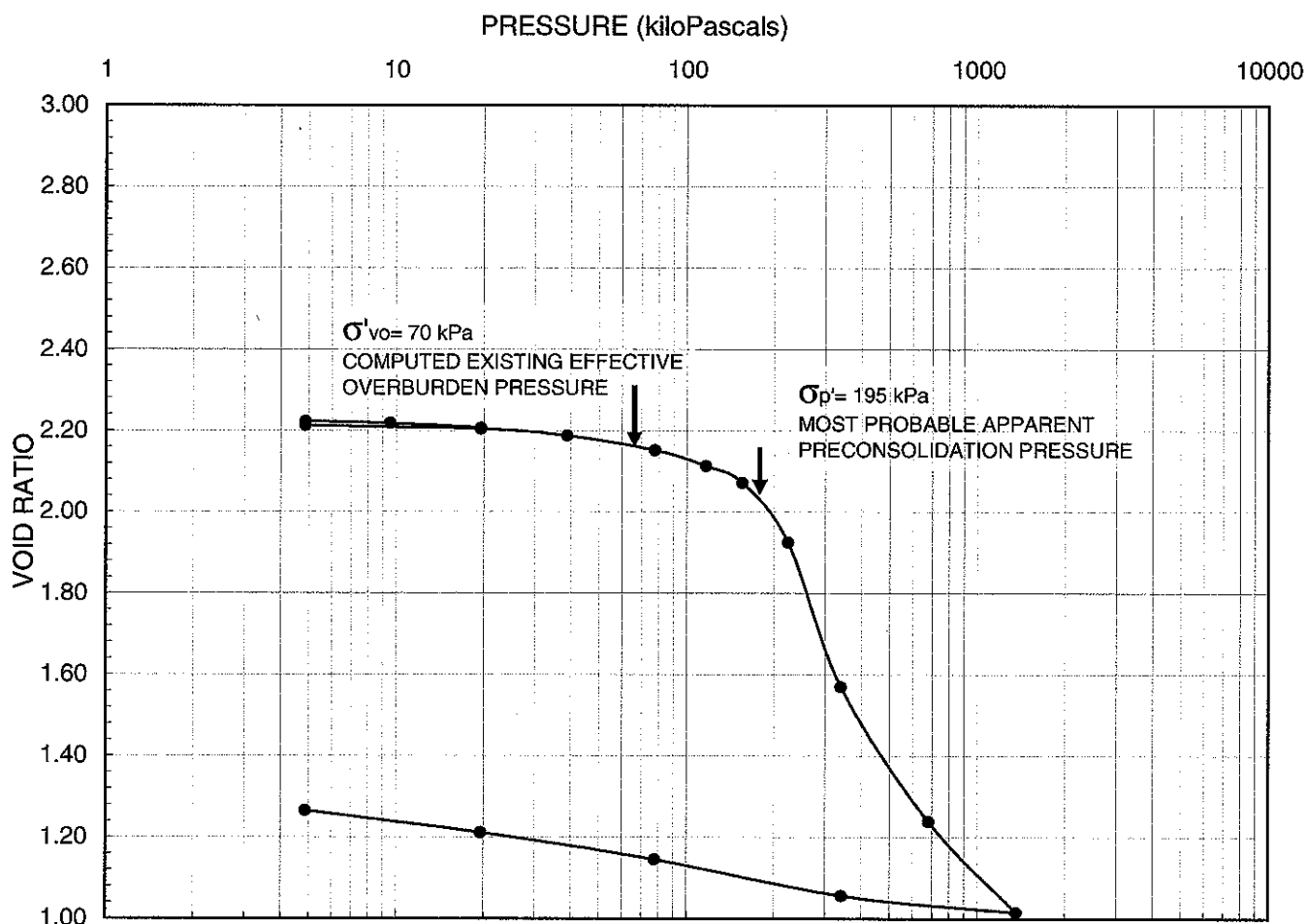
FILE No.	Consolidation summary
PROJECT No.	05-1120-210 REV. 0

CHECK
REVIEW

Retaining Wall 11S

FIGURE

17



LEGEND

Borehole: 06-141	$w_i = 73.4\%$	$S_o = 95\%$
Sample: 5	$w_f = 44.0\%$	$C_c = 1.95$
Depth (m): 4.90	$w_l = 58.1\%$	$C_r = 0.012$
	$w_p = 28.8\%$	



SCALE AS SHOWN

DATE 05/15/07

DESIGN NA

CADD NA

CHECK

REVIEW

TITLE

CONSOLIDATION TEST RESULTS

FILE No. Consolidation summary

PROJECT No. 0 REV. 0

Retaining Wall 11S

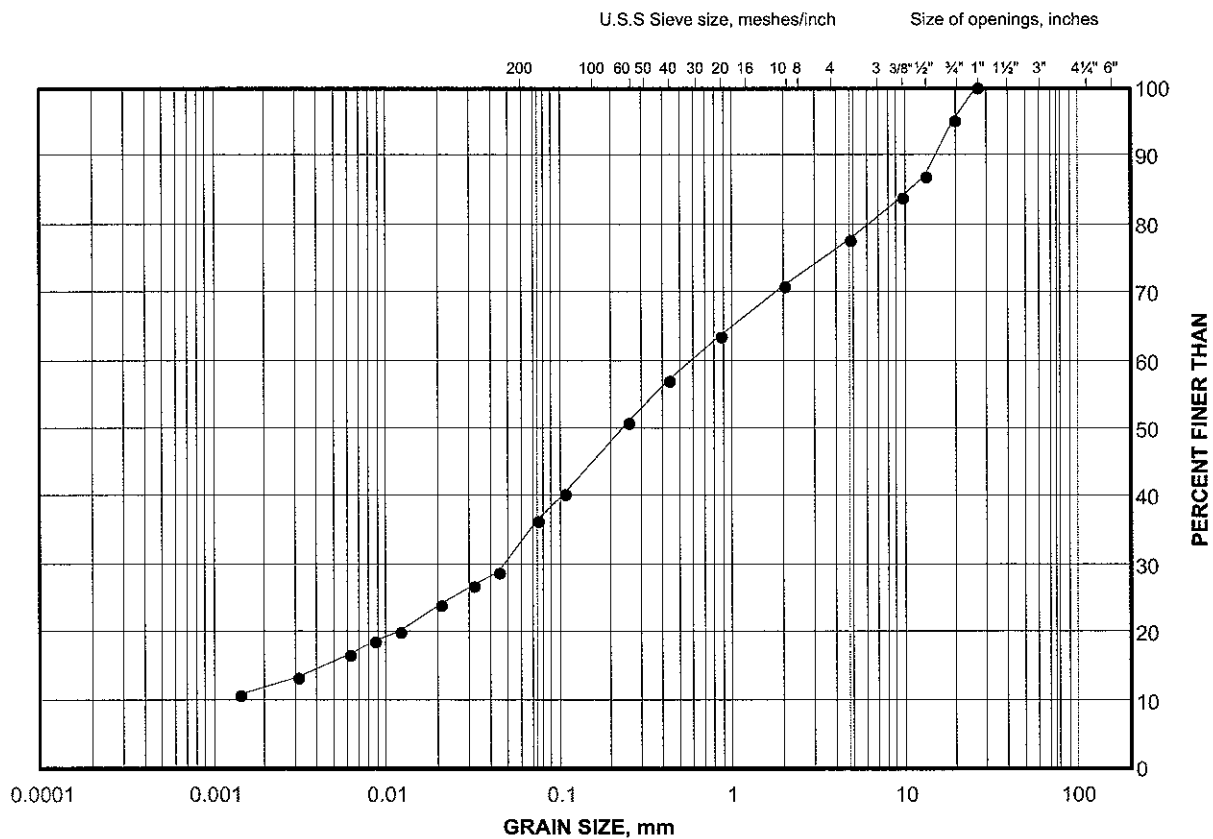
FIGURE

18

GRAIN SIZE DISTRIBUTION

Retaining Wall 11S - Till

FIGURE 19



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

LEGEND

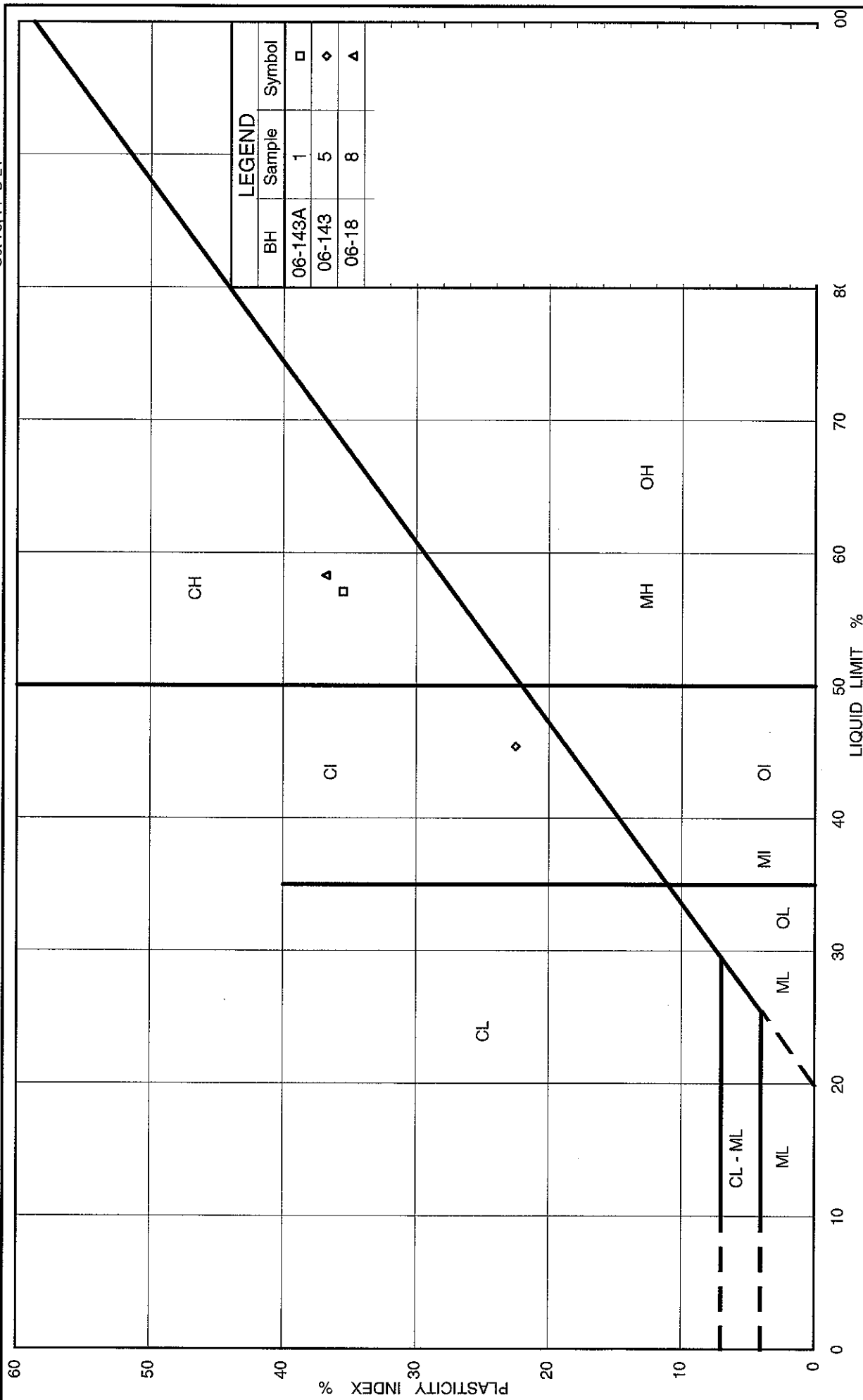
SYMBOL	BOREHOLE	SAMPLE	DEPTH(m)
•	06-139	9	8.4-9.0

Project Number: 05-1120-210

Checked By: _____

Golder Associates

Date: 27-Sep-06



PLASTICITY CHART Silty Clay to Clay

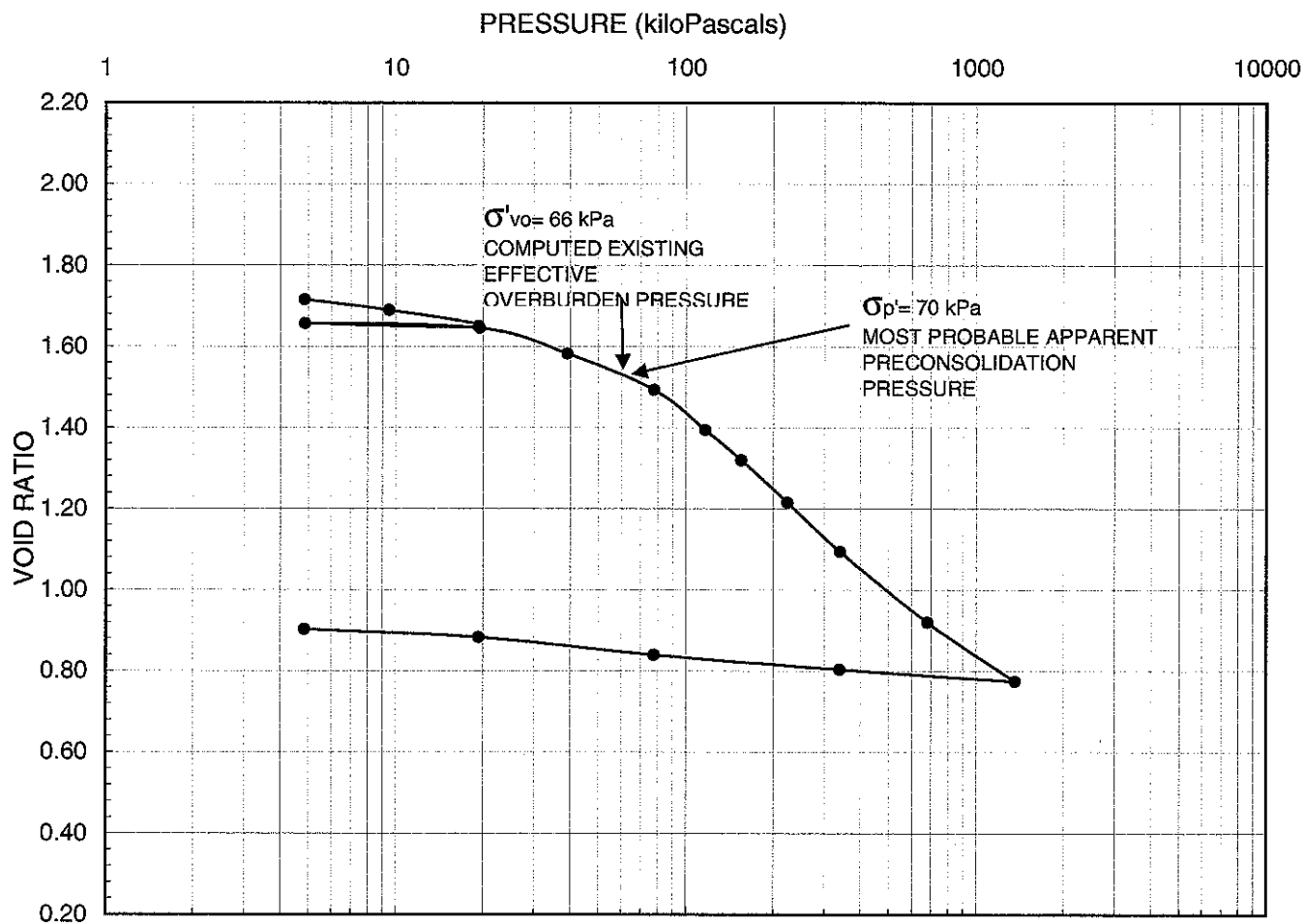
FIG No. 20 Retaining Wall 12S

Project No. 05-1120-210

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LEGEND

Borehole: 06-143	$w_i = 60.6\%$	$S_o = 97\%$
Sample: 5	$w_f = 32.5\%$	$C_e = 0.66$
Depth (m): 4.80	$w_l = 45.4\%$	$C_r = 0.020$
	$w_p = 22.9\%$	



SCALE	AS SHOWN
DATE	05/15/07
DESIGN	NA
CADD	NA

TITLE

CONSOLIDATION TEST RESULTS

FILE No. Consolidation summary

CHECK

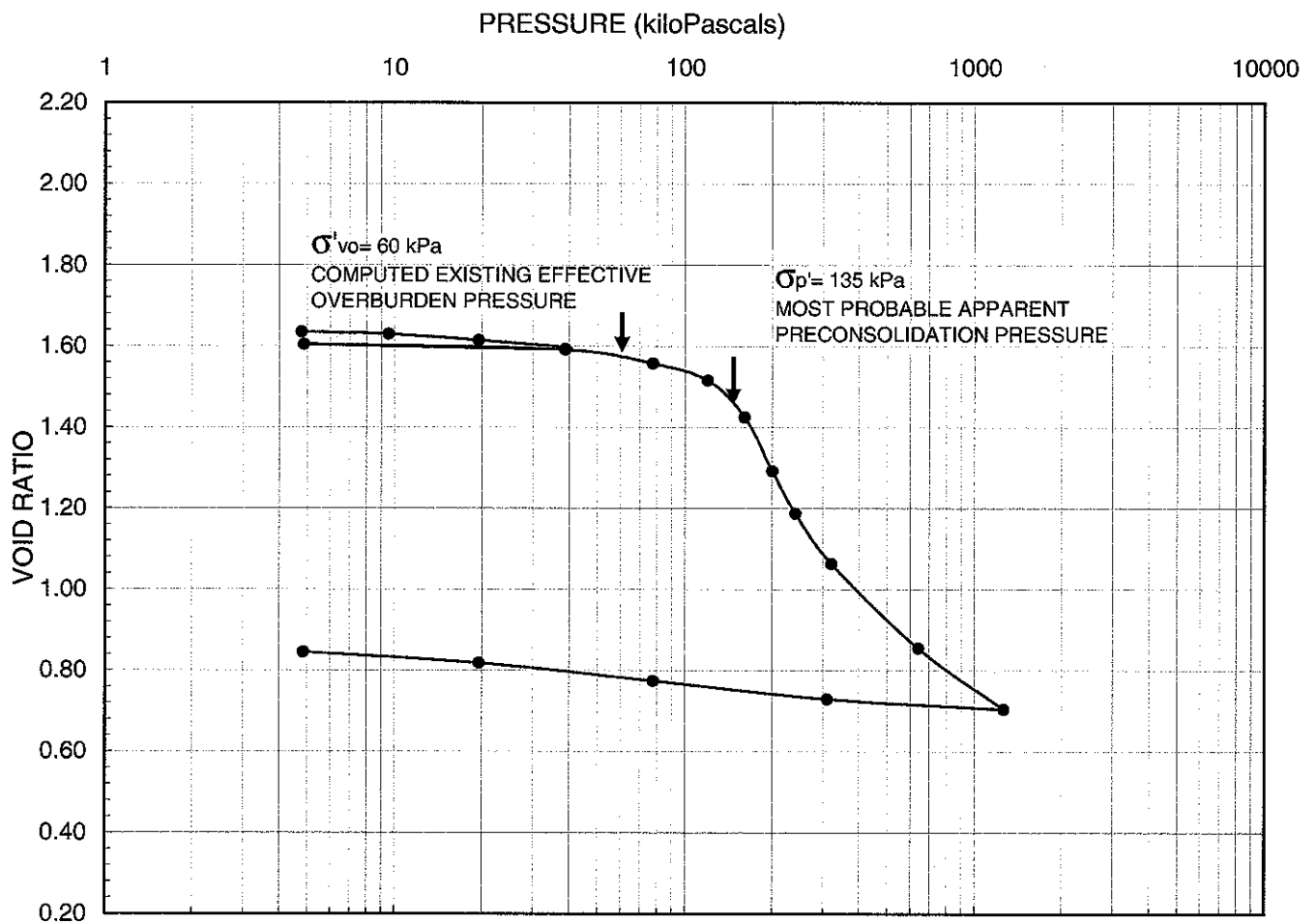
PROJECT No. 0 REV. 0

REVIEW

Retaining Wall 12S

FIGURE

21



LEGEND

Borehole: 06-143A	$w_i = 58.6\%$	$S_o = 100\%$
Sample: 1	$w_f = 32.1\%$	$C_c = 1.38$
Depth (m): 3.9m	$w_l = 57.1\%$	$C_r = 0.013$
	$w_p = 21.6\%$	



SCALE	AS SHOWN
DATE	05/15/07
DESIGN	NA
CADD	NA

TITLE

CONSOLIDATION TEST RESULTS

FILE No. Consolidation summary

CHECK

PROJECT No. 0 REV. 0

REVIEW

Retaining Wall 12S

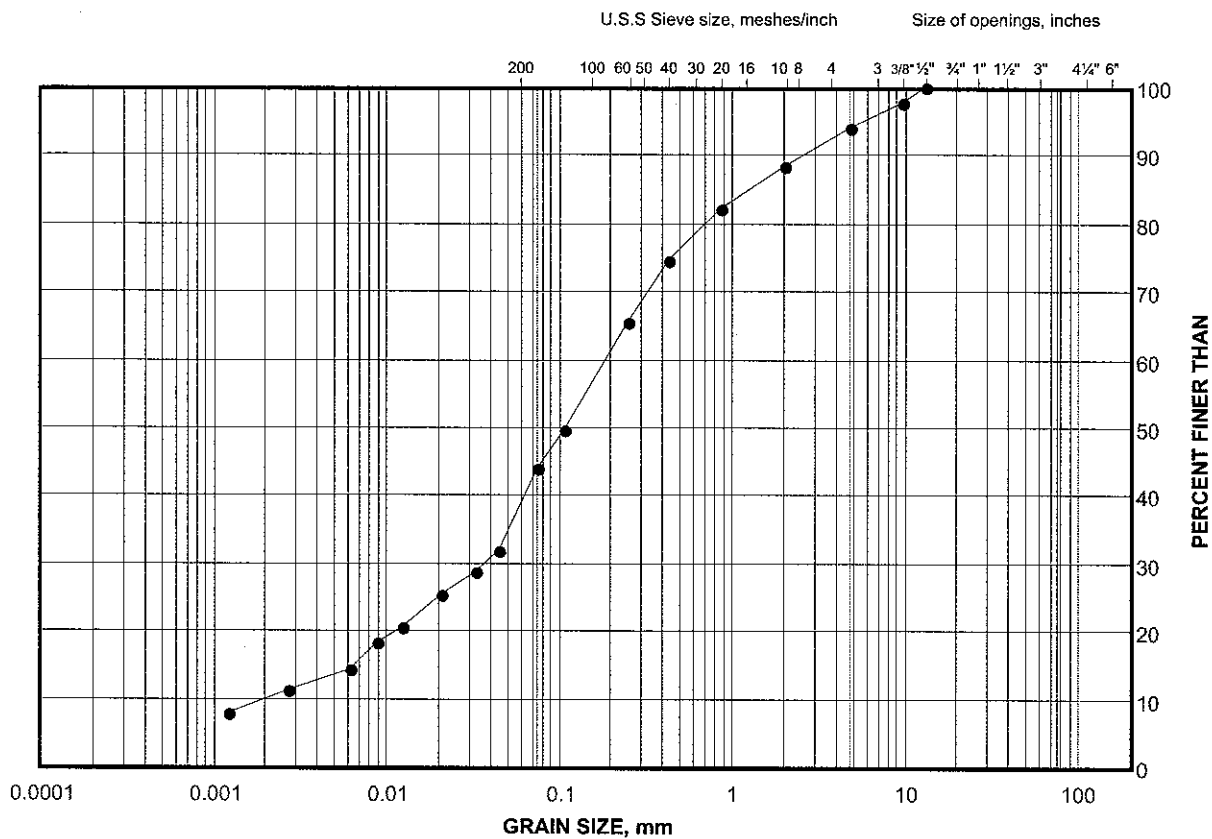
FIGURE

22

GRAIN SIZE DISTRIBUTION

Retaining Wall 1N -Till

FIGURE 23



LEGEND

SYMBOL	BOREHOLE	SAMPLE	DEPTH(m)
•	06-1	3	1.7-2.1

Project Number: 05-1120-210

Checked By: _____

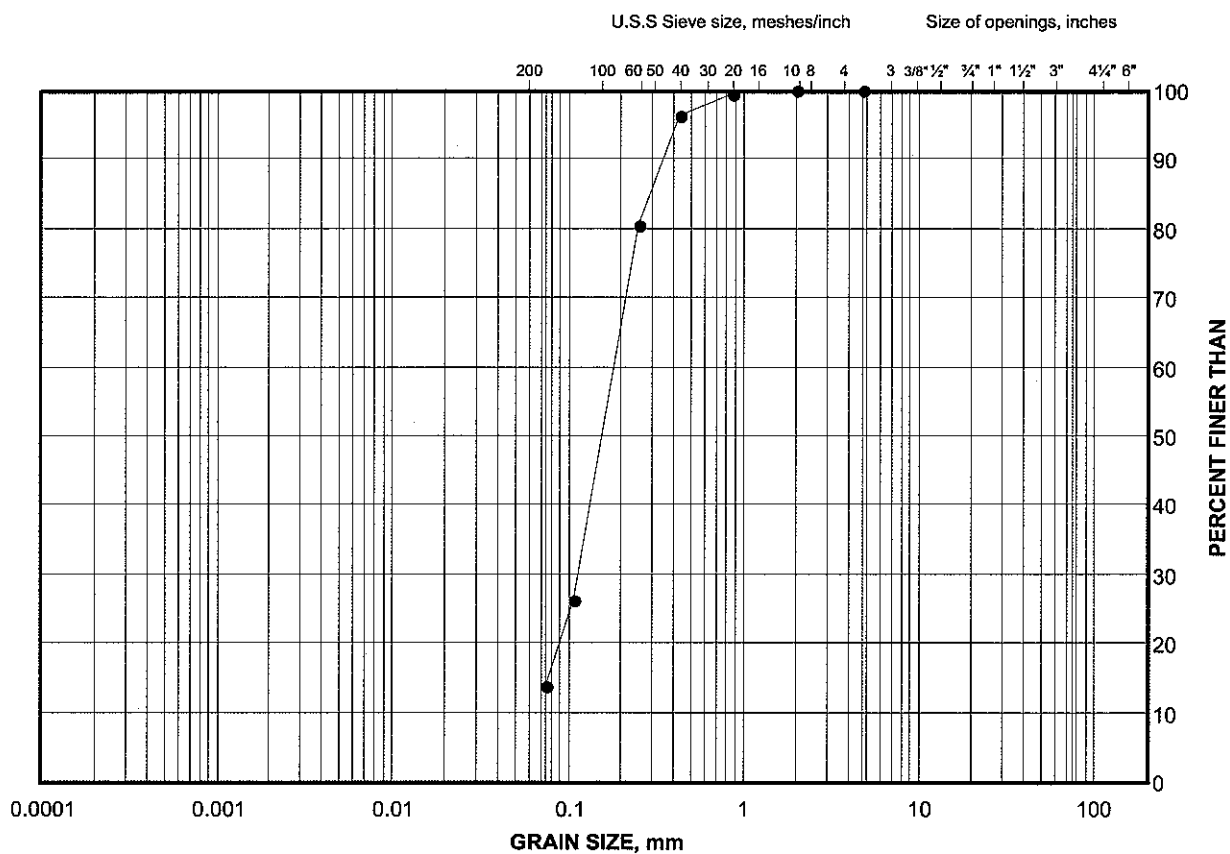
Golder Associates

Date: 07-Jun-06

GRAIN SIZE DISTRIBUTION

Retaining Wall 3N - Sand Fill

FIGURE 24



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	DEPTH(m)
•	06-126	5B	2.4-3.0

Project Number: 05-1120-210

Checked By: _____

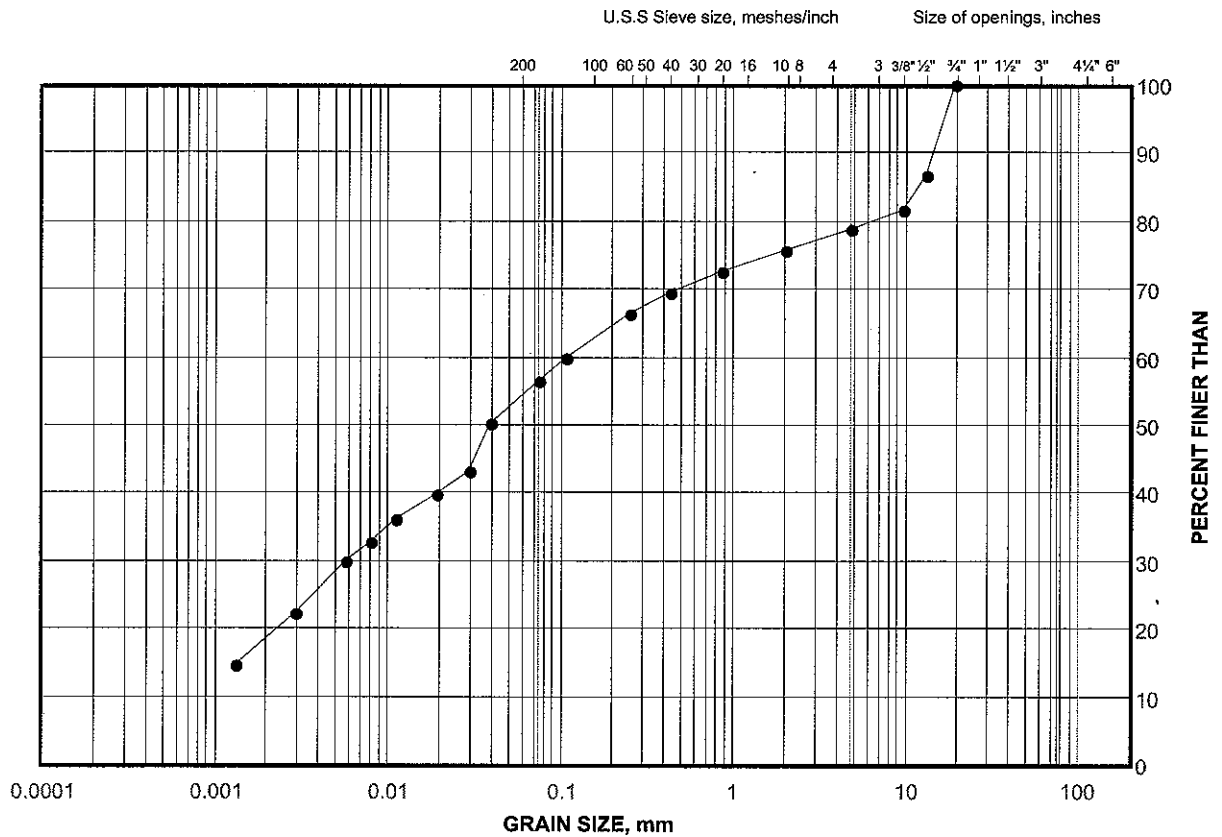
Golder Associates

Date: 31-Jul-06

GRAIN SIZE DISTRIBUTION

Retaining Wall 3N -Till

FIGURE 25



LEGEND

SYMBOL	BOREHOLE	SAMPLE	DEPTH(m)
•	06-7	2	1.4-2.0

Project Number: 05-1120-210

Checked By: _____

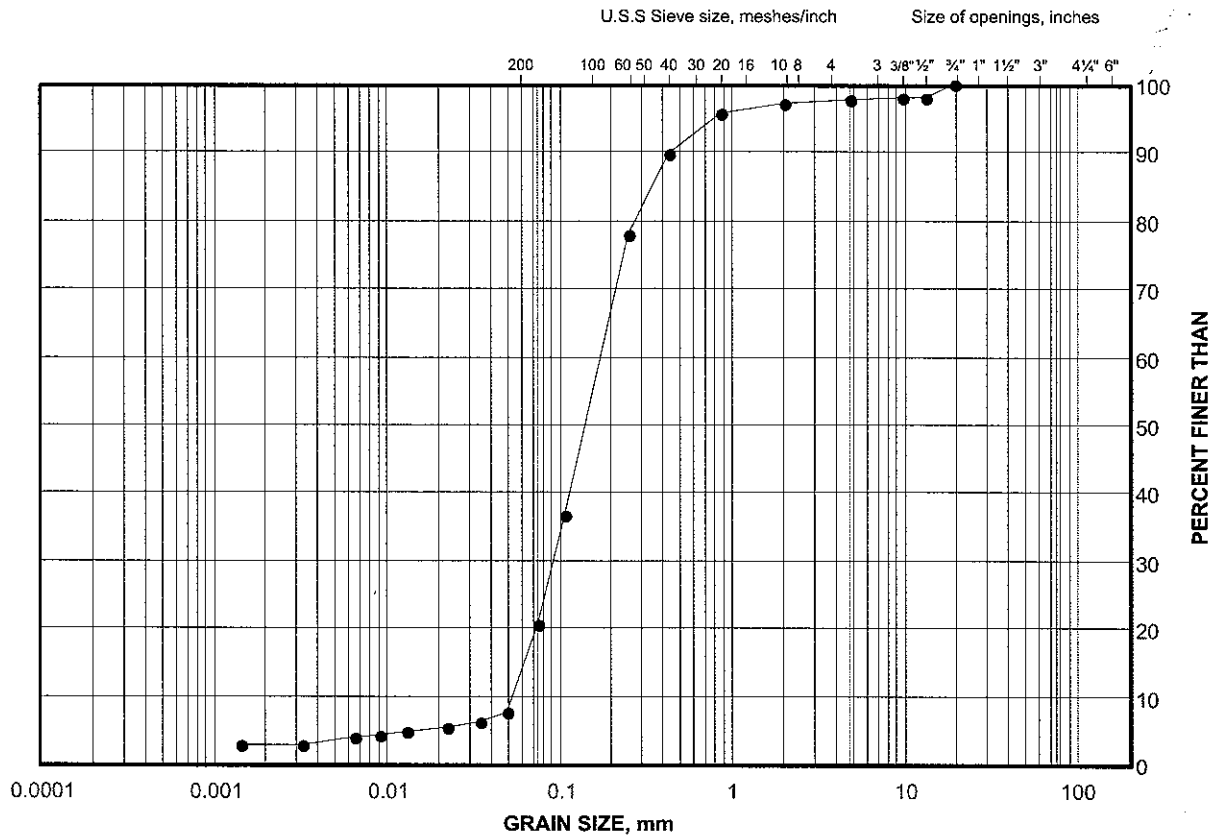
Golder Associates

Date: 22-Jun-06

GRAIN SIZE DISTRIBUTION

Retaining Wall 4N - Sand Fill

FIGURE 26



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	DEPTH(m)
•	06-130	2	1.5-2.1

Project Number: 05-1120-210

Checked By: _____

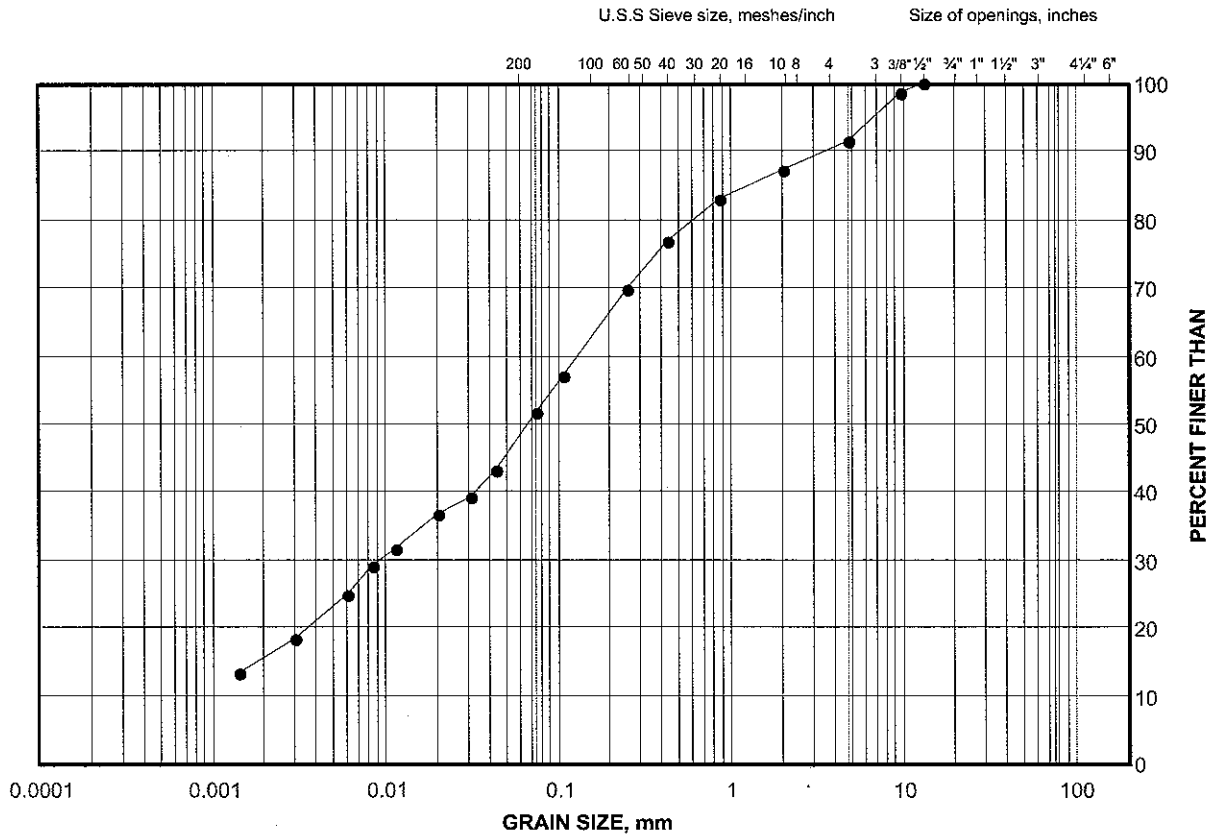
Golder Associates

Date: 27-Sep-06

GRAIN SIZE DISTRIBUTION

Retaining Wall 4N - Till

FIGURE 27



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	DEPTH(m)
•	06-130	7	5.3-5.9

Project Number: 05-1120-210

Checked By: _____

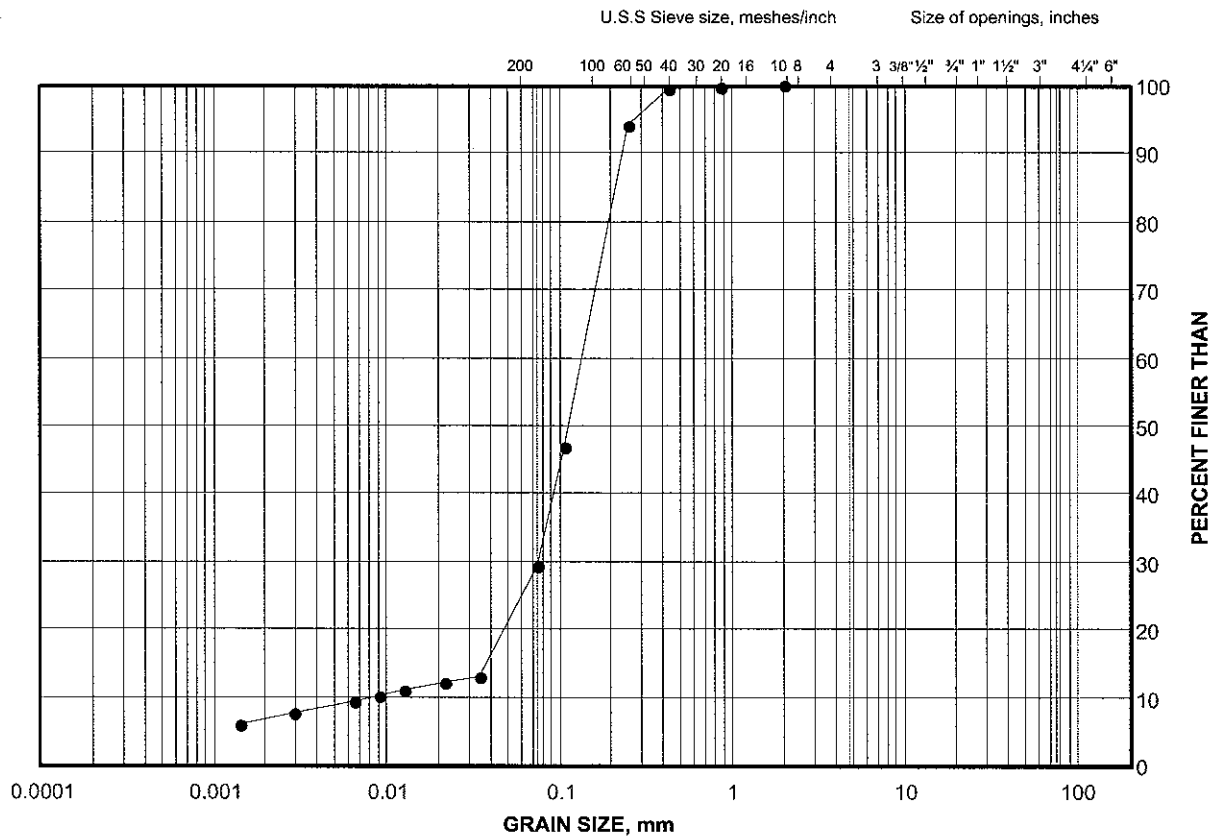
Golder Associates

Date: 27-Sep-06

GRAIN SIZE DISTRIBUTION

Retaining Wall 5N - Sand Fill

FIGURE 28



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

LEGEND

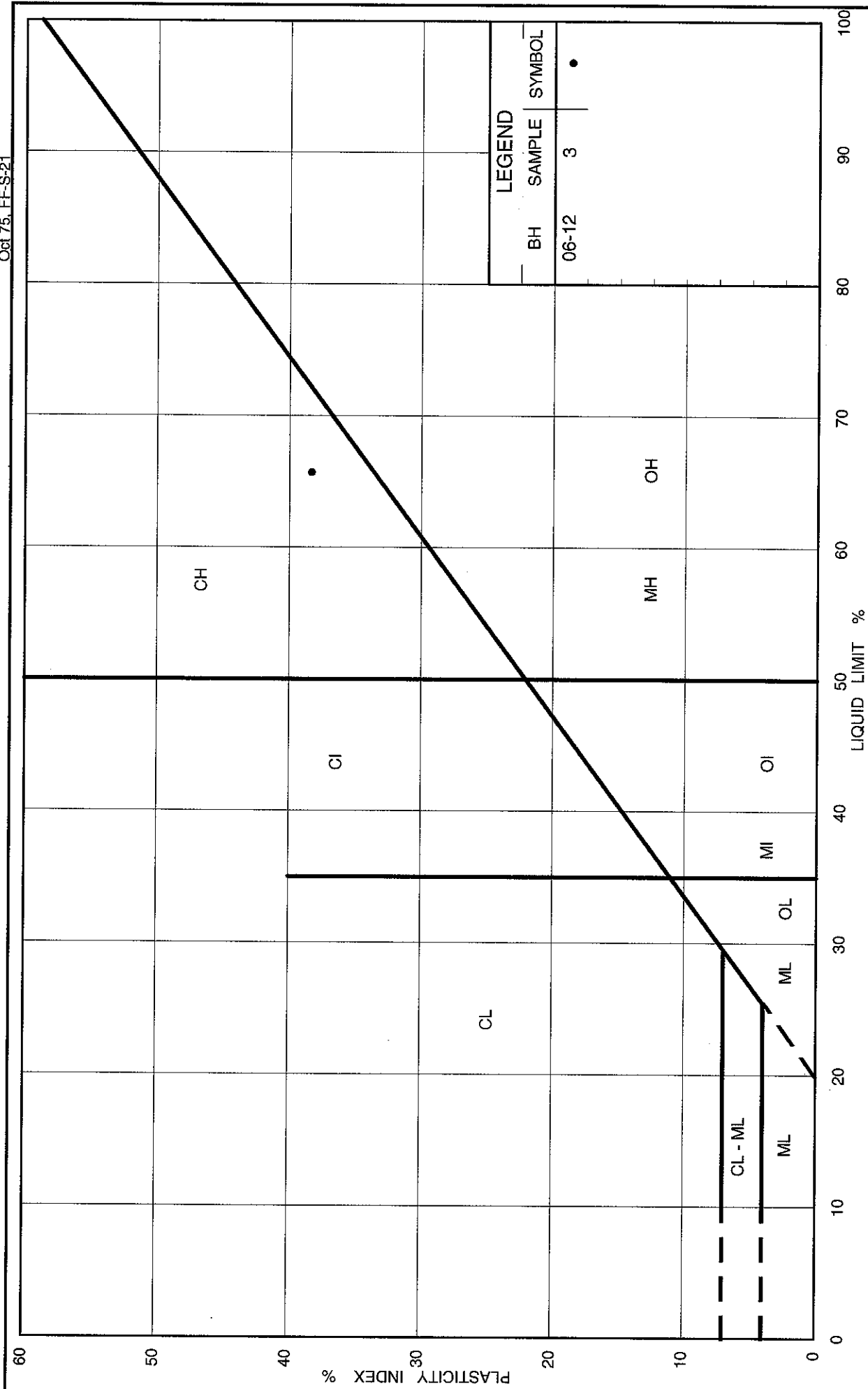
SYMBOL	BOREHOLE	SAMPLE	DEPTH(m)
•	06-12	2	0.9-1.4

Project Number: 05-1120-210

Checked By: _____

Golder Associates

Date: 28-Jun-06



PLASTICITY CHART Silty Clay to Clay

FIG No. 29 Retaining Wall 5N

Project No. 05-1120-210 - 2000

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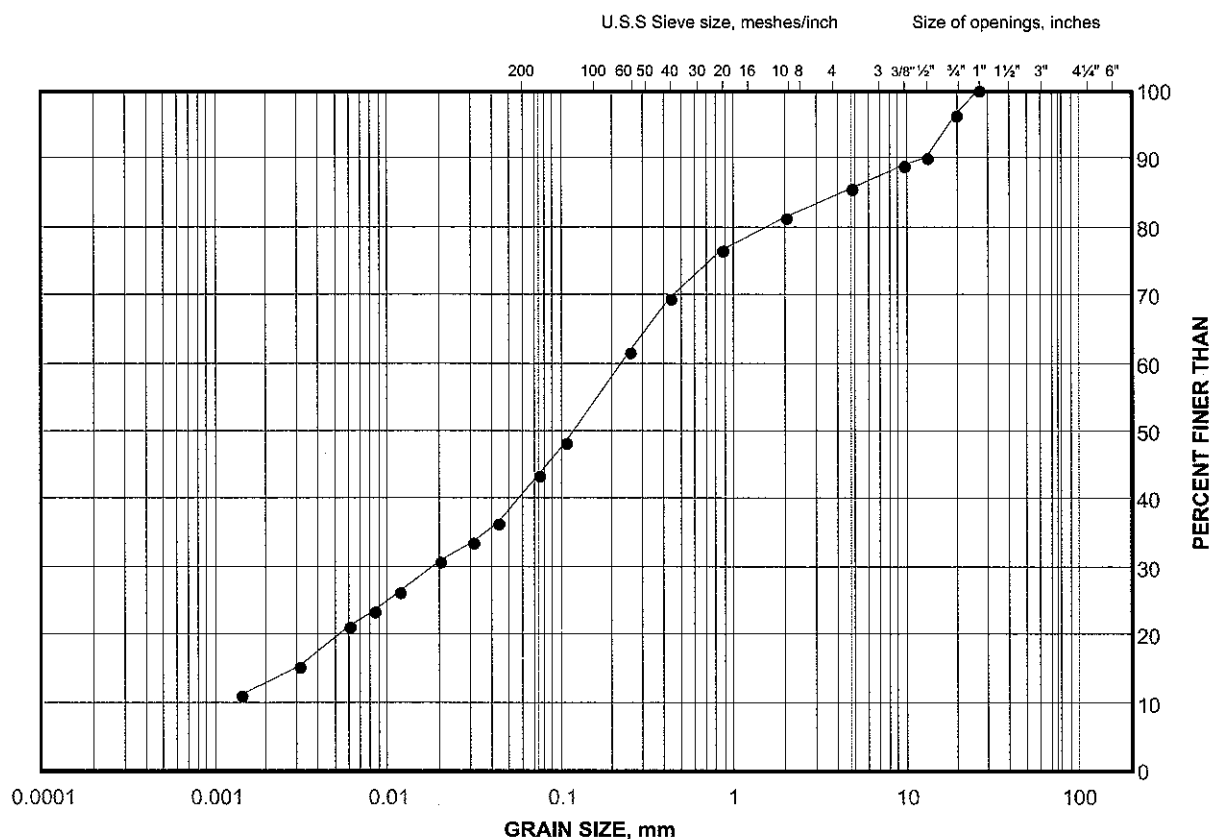


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GRAIN SIZE DISTRIBUTION

Retaining Wall 5N - Till

FIGURE 30



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	DEPTH(m)
•	06-132	3	2.3-2.9

Project Number: 05-1120-210

Checked By: _____

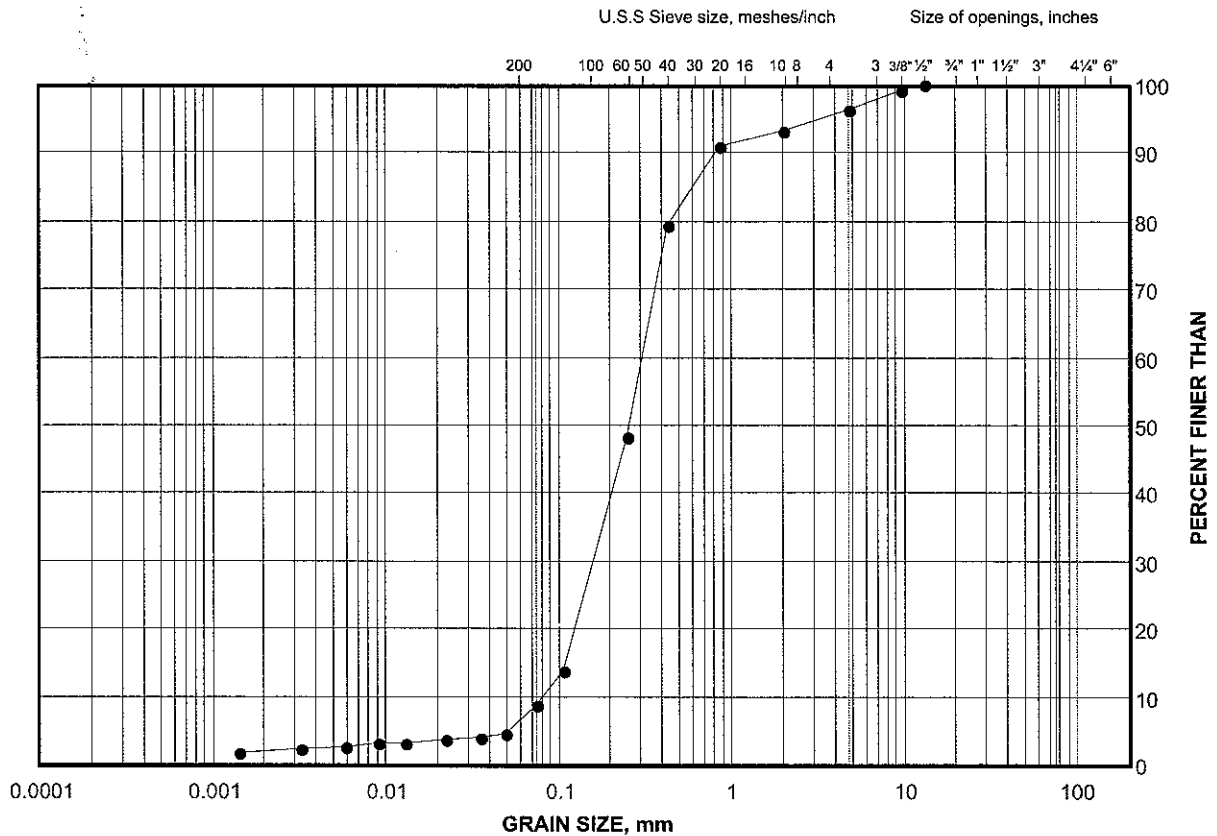
Golder Associates

Date: 27-Sep-06

GRAIN SIZE DISTRIBUTION

Retaining Wall 5N - Sand

FIGURE 31



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	DEPTH(m)
•	06-12	8	6.1-6.7

Project Number: 05-1120-210

Checked By: _____

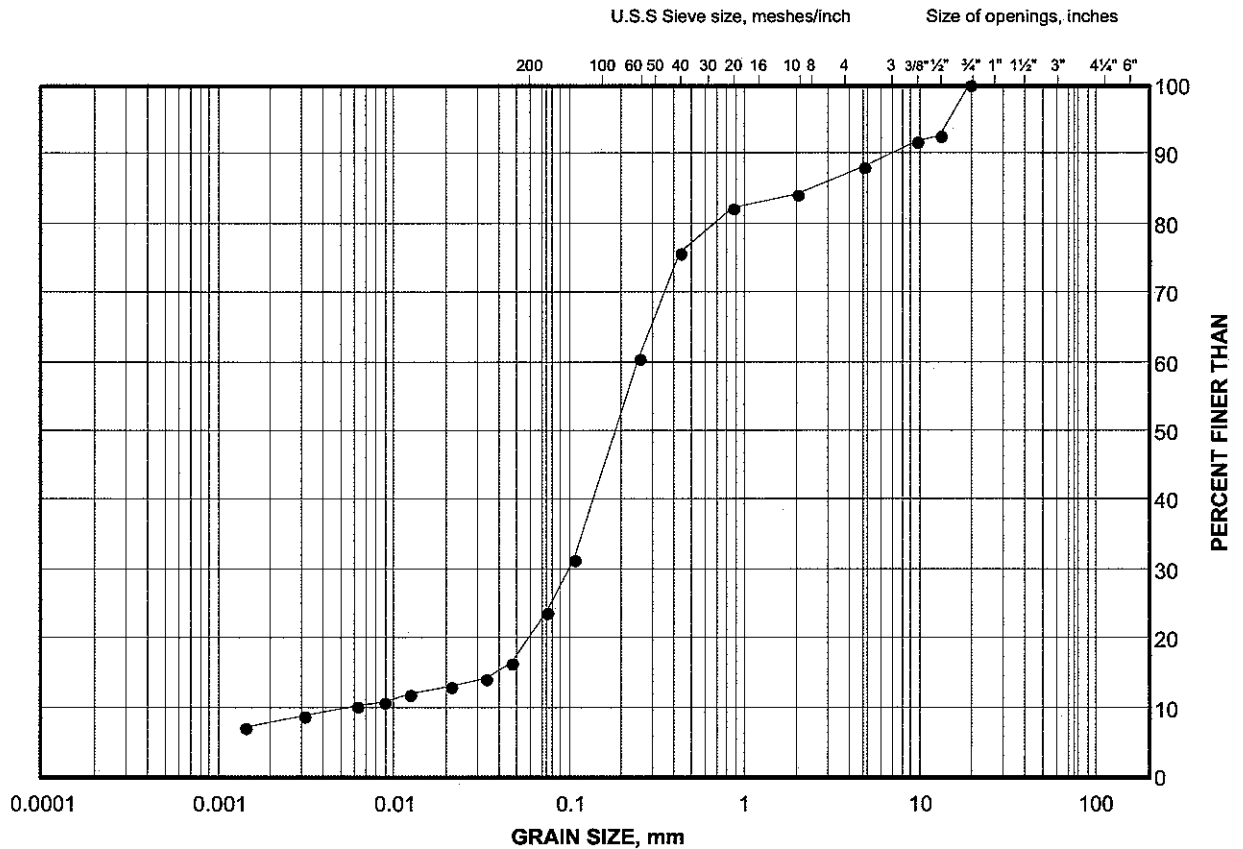
Golder Associates

Date: 22-Jun-06

GRAIN SIZE DISTRIBUTION

Retaining Wall 6N - Sand Fill

FIGURE 32



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

LEGEND

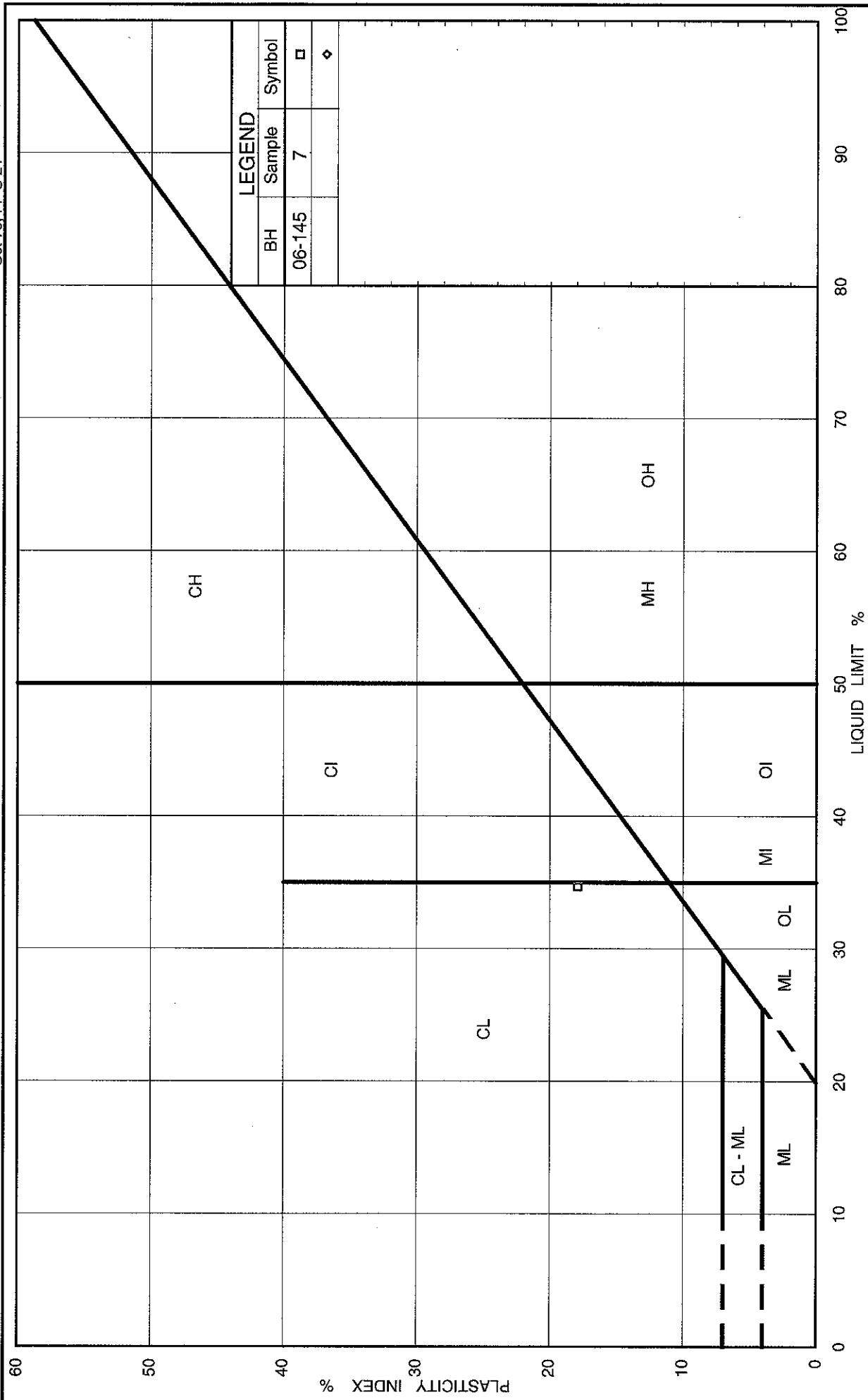
SYMBOL	BOREHOLE	SAMPLE	DEPTH(m)
•	06-145	3	2.3-2.9

Project Number: 05-1120-210

Checked By: _____

Golder Associates

Date: 27-Sep-06



PLASTICITY CHART Silty Clay

FIG No. 33 - Retaining Wall 6N

Project No. 05-1120-210

Ministry of Transportation



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APPENDIX D
NON-STANDARD SPECIAL PROVISIONS

Non-Standard Special Provision

Scope of Work

This Special Provision covers the requirements for mass concrete under the abutment and/or retaining wall footings. The purpose of the mass concrete pad is to provide a level working surface at the appropriate elevation for the construction of the abutment and/or retaining wall footings.

Construction

Work under this item shall adhere to the following requirements:

- The surface of the footing founding rock shall be exposed, cleaned and any loose rock removed.
- The mass concrete shall have a minimum 28 day strength of 30 MPa.
- The mass concrete shall be placed on the exposed cleaned founding rock surface as per the contract drawings and documents.
- Thickness of the mass concrete pad shall depend on the slope and irregularities in the exposed founding rock surface and on the underside of footing elevation. A nominal thickness and a footprint plan view area has been specified on the contract drawings and documents.
- Unwatering of the excavation for the footing construction, including the construction of the mass concrete pad, might be required and is covered under a separate Tender Item. The dewatering scheme shall be done in such a manner as to prevent any disturbance to the surrounding original soil.

Basis of Payment

Payment at the contract price for the above noted Tender Item includes full compensation for all labour, equipment and materials to do the required work.

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TEST ANCHORS – Item No.
PRODUCTION ANCHOR – Item No.
POST GROUTING OF BOND LENGTH – Item No.

Special Provision No. 999S26

November 2006

1.0 SCOPE

This specification covers the requirements for the design, installation and testing of temporary and permanent pre-stressed anchors in soil and rock.

2.0 REFERENCES

This specification refers to the following standards, specifications or publications:

Ontario Provincial Standard Specifications, General:

OPSS 180 Management of Excess Material

Ontario Provincial Standard Specifications, Construction:

OPSS 904 Concrete
OPSS 906 Structural Steel

Ontario Provincial Standard Specifications, Material:

OPSS 1301 Hydraulic Cementing Materials
OPSS 1302 Water
OPSS 1350 Concrete (Materials and Production)
OPSS 1440 Steel Reinforcement for Concrete

Canadian Standards Association Standards, CSA:

A23.1-00/A23.2-00 Concrete Materials and Methods of Concrete Construction/Method of Test of Concrete
A283-00 Qualification Code for Concrete Testing Laboratories
G40.20-98/G40.21-98 General Requirements for Rolled or Welded Structural Quality Steel / Structural Quality Steels

American Society for Testing and Materials Standards, ASTM:

A53/A53M-02 Standard Specification for Pipe, Steel, Black and Hot-Dipped, Zinc-Coated, Welded and Seamless
A416/A416M-99 Steel Strand, Uncoated Seven-Wire for Prestressed Concrete
A500-03a Standard Specification for Cold-Formed Welded and Seamless Carbon Steel Structural Tubing in Rounds and Shapes
A722/A722M-98 Uncoated High-Strength Bar for Prestressing Concrete
D1143-81 Standard Test for Piles Under Static Axial Compressive Load
D1248-84 (1998) Polyethylene Plastics Molding and Extrusion Materials
D1784-83 (1983) Rust Protection for Metal Preservatives in the Humidity Cabinet

International Organization for Standardization/International Electrotechnical Committee, ISO/IEC

DIS 17025:1999 General Requirements for the Competence of the Testing and Calibration Laboratories

American Petroleum Institute, API:

13A-93 Oil Well Drilling Fluid Materials Fifteenth Edition, May 1, 1993
RP 13B-1 Standard Procedures for Field Testing Water Based Drilling Fluids, Second Edition, September 1997

Others:

Post Tensioning Institute Publications - Recommendations for Prestressed Rock and Soil Anchors - 1996

3.0 DEFINITIONS

Alignment Load (AL): means a nominal minimum load applied to an anchor during testing to keep the testing equipment positioned correctly.

Anchor: means a system, used to transfer tensile loads to soil or rock, which includes the prestressing steel, anchorage, corrosion protection, sheathings, spacers, centralizers and grout.

Anchor Head: means the device by which the prestressing force is permanently transmitted from the prestressing steel to the bearing plate.

Anchorage: means the combined system of anchor head, bearing plate, trumpet and anchorage corrosion protection that is used to transmit the prestressing force from the prestressing steel to the surface of the ground or the supported structure.

Bond Length: means the length of the tendon that is bonded to the primary grout and capable of transmitting the applied tensile load to the surrounding soil or rock.

Centralizer: means a device to support and position the tendon and sleeves in the drill hole so that a minimum grout cover is provided.

Coupler: means the means by which the prestressing force can be transmitted from one partial-length of prestressing tendon to another.

Design Load (DL): means the anticipated final maximum effective load in the anchor after allowance for time-dependent losses or gains. The design load includes appropriate load factors to ensure that the overall structure has adequate capacity for its intended use.

Free Stressing (unbonded) Length: means the designed length of the tendon that is not bonded to the surrounding ground or grout during stressing.

Lift-Off: means checking the load (lift-off load) in the tendon at any specified time with the use of hydraulic jack, by lifting the anchor head off the bearing plate.

Lock-Off Load: means the prestressing force in an anchor immediately after transferring the load from the jack to the stressing anchorage.

Permanent Anchor: means any prestressed anchor intended for permanent use, generally with more than a 24- month service life.

Performance Test: means the incremental cyclic test loading and unloading of an anchor, while recording the total movement of the anchor at each increment, including the residual movement at alignment load.

Post-Grouting: means regrouting an anchor after the primary grout has set.

Prestressing Steel: means strands, a group of strands combined to form a tendon or a high strength steel bar.

Primary Grout: means Portland cement based grout that is injected into the anchor hole prior to or after the installation of the anchor tendon to provide for the force transfer to the surrounding ground along the bond length of the tendon.

Proof Test: means incremental loading of an anchor, and recording the total movement of the anchor at each increment.

Quality Verification Engineer: means an Engineer with a minimum of five (5) years experience related to the design, installation and stressing of anchors, or alternatively has demonstrated expertise by providing satisfactory quality verification services for the work at a minimum of two (2) projects of similar scope to the Contract. The Quality Verification Engineer shall be retained by the Contractor to certify that the work is in general conformance with the contract documents and to issue certificate(s) of conformance.

Temporary Anchor: means a prestressed anchor intended for temporary use, generally with less than a 24-month service life. Temporary anchors installed in corrosive environments may require corrosion protection.

Test Anchor: means an anchor installed and then loaded to verify the design parameters prior to the installation of the production anchors.

Test Load (TL): means the maximum load to which the anchor is subjected during testing.

Total Movement: means the total movement of the pulling head measured at maximum load in each cycle.

4.0 SUBMISSION AND DESIGN REQUIREMENTS

4.01 Submissions

4.01.01 General

At least two weeks prior to the commencement of the work, three copies of the working drawings shall be submitted to the Contract Administrator for information purposes only. These working drawings shall bear the seal and signature of the design and checking Engineers who have a minimum of five years experience on projects of a similar nature and scope to the required work.

4.01.02 Working Drawings

Information to be shown on the working drawings shall describe and illustrate the complete details of the anchor system, anchor testing equipment, and reaction system for the production and when specified test anchors. This information shall include but not be limited to the following:

1. Plans, Elevations and Sections
 - i anchor spacing
 - ii orientation
 - iii minimum total anchor length
 - iv free stressing length
 - v design load
 - vi a unique identification number for each anchor.
 - vii anchor components and details
2. Installation
 - i construction methods
 - ii work restrictions
 - iii schedule
 - iv sequence and coordination of work
 - v monitoring
 - vi type and number of tests
 - vii evaluation of test results
3. Materials
 - i Physical properties of monobar / multistrand anchors.
 - ii Bond length grout materials and mix proportions.
 - iii Post grouting materials and mix proportions.
 - iv Free stressing length materials and mix proportions.
 - v Corrosion protection material physical/mechanical properties.
4. Anchor Hole Construction
 - i Method of constructing the anchor holes and maintaining the stability of the holes during the anchor installation. The drilling equipment and materials including drill bit/auger diameter and lengths, casing diameter and lengths, slurry materials or other materials to facilitate the construction of the anchor hole. The method of verifying the lengths of anchor holes shall also be identified.
 - ii Details of the assembling of the anchor in the hole.
 - iii Method of placing and centring the anchor tendons including the method used to maintain them in the centre of the hole over the design bond length.
 - iv Bond zone primary grouting placement. Grout mixing procedure and the method of installation including grout pressures. The method to determine the surface of the hardened bond length grout shall be identified.
 - v Bond zone post grouting placement. Grout mixing procedure and the method of installation including grout pressures.
 - vi Free stressing zone grouting placement. Grout mixing procedure and the method of installation including grout pressures.

- vii Waterproofing of drilled holes in rock for permanent anchors. Details of water tightness tests including setup, water pressure, method of applying pressure, details of consolidation, grouting, redrilling and retesting.
5. Stressing Information
- i Anchor stressing schedule that includes the working loads and test loads.
 - ii Anchor stressing equipment, and the method for testing the stressing of test anchors and production anchors. Details of the reaction system used to support the applied loads.
 - iii Equipment, including the calibration records of the gauges and jacks and procedure to monitor the applied loads and movements during anchor testing. Details of the reference system and equipment to monitor the applied loads and movement.
6. All design assumptions, loads, parameters and bond stresses used for production and test anchors.
7. Testing records when testing has been done to determine bond stress.
8. Details of destressing and removal of temporary anchors.

4.01.03 Bentonite Slurry

At least two weeks prior to the commencement of the work, the following information for the slurry shall be submitted to the Contract Administrator by the Quality Verification Engineer:

- 1. The type, source, physical and chemical properties of the bentonite or polymer.
- 2. The source of water.
- 3. Method of mixing slurry.
- 4. The water solids ratio, the mass and volumes of the constituent parts including any chemical admixtures or physical treatment employed to produce a slurry with the required physical properties.
- 5. Details of procedure to be used for monitoring the quality of the slurry.
- 6. A test report describing the properties of the slurry (density, viscosity).
- 7. Method of disposal of the slurry.

4.01.04 Couplers

At least two weeks prior to commencement of the work, a copy of the manufacturers catalogue giving complete data on the coupler material and installation procedures as well as test reports from the manufacturer certifying strength and fatigue requirements shall be submitted to the Contract Administrator.

4.01.05 Prestressing Steel

4.01.05.01 Mill Certificates

One copy of the mill certificates, indicating that the steel meets the requirements of the Contract Documents shall be submitted to the Contract Administrator, at time of delivery to the job site.

Identification on the anchor tendon shall allow tracing of the prestressing steel to its heat or reel number.

Where mill test certificates originate from a mill outside Canada or the United States of America the Contractor shall have the information on the mill certificate verified by a Canadian testing laboratory. The laboratory shall be accredited by the Standards Council of Canada as complying with the requirements of ISO/IEC DIS 17025 for the specific tests or type of tests required by the material standard specified on the mill test certificate. The mill test certificates shall be stamped with the name of the testing laboratory and appropriate wording stating that the material conforms to the specified material requirements. The stamp shall include the appropriate material specification number, the date and the signature of an authorized officer of the testing laboratory.

One copy of the stress-strain curves representative of the lots to be used shall be submitted to the Contract Administrator together with the mill certificates detailed in OPSS 1440.

4.02 Design

Except for Owner designed anchors, the Contractor shall be responsible for the determination of the applied loads, design assumptions, installation procedures and the detailed design of the anchor.

The anchors shall be designed to safely withstand the applied loads specified in the Test Anchor clause and fulfill the acceptance criteria specified in the Production Anchor clause and perform satisfactorily at the design load through the required service life.

The design assumptions shall accurately represent the subsurface conditions prevalent at the site.

Temporary anchors in a corrosive environment shall be designed as permanent anchors unless otherwise approved.

Except as specified herein the anchors shall be designed in accordance with the design recommendation of the Post Tensioning Institute Recommendations for Prestressed Rock and Soil Anchors.

4.03 Certificate of Conformance

The Contractor shall submit, to the Contract Administrator, a Certificate of Conformance sealed and signed by the Quality Verification Engineer upon completion of the anchor installation and stressing.

The Certificate of Conformance shall state that the anchors have been supplied, installed and stressed in general conformance with the working drawings.

5.0 MATERIALS

5.01 Anchors

5.01.01 Permanent Anchors

5.01.01.01 General

Tendons shall be manufactured from steel bars or strand either in single or multiple element tendons.

Unless otherwise approved the permanent anchor shall be Dywidag Threadbar Anchors, BBR Cona Multi-Strand Anchors or VSL Multi-Strand Anchors.

5.01.01.02 Anchorages

The components of the anchorage shall be capable of developing at least 100% of the guaranteed minimum ultimate capacity of the tendon or bar.

The anchor head shall be wedges for prestressing strands and anchor nuts for prestressing bars. The wedges shall be designed to uniformly engage the strand with no notch or pinching effects that might lead to premature failure of the prestressing steel.

The bearing plate shall be fabricated from steel conforming to CAN/CSA G40.20/21.

The trumpet shall be fabricated from pipe conforming to ASTM A53 or tubing conforming to ASTM A500. The trumpet shall have a minimum wall thickness of 3 mm for diameters up to 100 mm and 5 mm for larger diameters. The joint between the trumpet and the bearing plate, and the joint between the trumpet and sheath shall be watertight. The trumpet shall overlap the unbonded length corrosion protection by at least 100 mm. The trumpet shall be long enough to accommodate movements of the structure and the tendon during stressing and testing.

The anchorage covers shall completely encapsulate the anchor head with a watertight seal between the cover and the bearing plate.

5.01.01.03 Prestressing Steel

Prestressing steel shall be according to ASTM A416 and ASTM A722.

Bars shall be high-tensile strength bars grade 1030 MPa, grade 1100 MPa, or 1230 MPa.

Strand shall be seven-wire, uncoated, stress relieved and low relaxation strand grade 1720 MPa, 1760 MPa or 1860 MPa

Prestressing steel shall be according to OPSS 1440.

5.01.01.04 Couplers

Couplers for bars shall be as specified by the supplier of the anchor and it shall develop at least 100% of the guaranteed minimum ultimate strength of the tendon. Strand tendons shall not be coupled.

5.01.01.05 Cement

Cement shall be according to OPSS 1301 and shall be certified free of false set.

5.01.01.06 Water

Water shall be according to OPSS 1302.

5.01.01.07 Sheath

Plastic sheathing shall be made from high density polyethylene according to ASTM D 1248, Type III, or from polyvinyl chloride in conformance with ASTM D 1784, Class 13464-B or equivalent. The plastic sheathing shall have a minimum compressive strength of 102 MPa and a minimum tensile strength of 48 MPa. The plastic sheathing shall be such that a bond of 5 MPa is developed when grout with a compressive strength of 30 MPa is used.

Plastic tubing made from polyethylene or polypropylene shall have an average minimum wall thickness of 1.5 mm. Polyvinyl chloride (PVC) tubing shall have an average minimum wall thickness of 1.0 mm and steel tubing or pipe shall have an average minimum wall thickness of 5.0 mm.

The materials for the sheathing accessories such as end caps, grouting caps, grout tubes and sealing caps shall have properties equivalent to the plastic sheathing.

5.01.01.08 Tendon Bond Length Encapsulation

The prestressing steel shall be protected with a grout filled corrugated plastic encapsulation. Centralizers shall be used to ensure a grout cover of at least 12 mm over the encapsulation.

5.01.01.09 Centralizers and Spacers

Centralizers and spacers shall be steel, plastic or a material that is non-detrimental to the prestressing steel. Wood spacers shall not be used.

Spacers shall be used in multiple element tendons to separate the strands or bars individually or into small groups.

5.01.01.10 Grout or Concrete for Bond Zone

The cube strength of the grout and the compressive strength of the concrete shall be at least 20 MPa at 7 days and 30 MPa at 28 days. The type of cement used shall be suitable for the required use of the grout. Accelerators shall not be used. The grout shall bleed less than 2 percent when allowed to stand for 1 hour.

Concrete shall be according to OPSS 1350 with a nominal 28-Day compressive strength of 30 Mpa.

The slump shall be 150 to 180 mm.

5.01.01.11 Corrosion Inhibiting Compound

The corrosion-inhibiting compound placed in either the free length or the anchorage area shall be an organic compound, grease or wax, with appropriate polar moisture displacing, corrosion inhibiting additives and self-healing properties. The compound shall permanently stay viscous and be chemically stable and nonreactive with the prestressing steel, the sheathing material and the anchor grout.

5.01.01.12 Corrosion Protection

The anchor shall be provided with Class I, Encapsulated Tendon, double corrosion protection.

The tendon shall be fully encased within a corrugated PVC sheathing that is, in turn, encased within a smooth PVC sheathing over the length of the free stressing zone and protected with grout having the 7d and 28d compressive strengths specified.

5.01.01.13 Bond Breaker

The bond breaker shall be fabricated from plastic tube or pipe made from medium to high density polyethylene according to ASTM D 1248 or from polyvinyl chloride according to ASTM D 1784, Class 13464-B or equivalent, with a minimum wall thickness of 1 mm.

5.01.01.14 Heat Shrinkable Sleeves

Heat shrinkable sleeves shall be fabricated from a radiation cross-linked polyolefin tube internally coated with an adhesive sealant.

5.01.02 Temporary Anchors

The material for temporary anchors shall be the same as specified for the permanent anchor except the double protection system is not required unless specified.

5.02 Test Anchors

The material for the test anchors shall be the same as specified for the anchor being evaluated.

5.03 Bentonite Slurry or Lean Mix in Free Stressing Zone

The purpose of the backfill in the free stressing zone is to prevent settlement and preventing transfer of the anchor load to the free stressing zone. The backfill shall completely fill the annular space between the tendon and the anchor hole and shall render a stable hole without any hole collapse or cave-in.

Bentonite slurry or lean mix concrete can be used. Bentonite shall meet the requirements of API 13A. For soil anchors, a lean mix concrete (0.4 MPA) can be used to backfill the free stressing zone.

The Contractor shall submit the proposed bentonite slurry or lean mix concrete, reviewed and approved by the Quality Verification Engineer, to the Contract Administrator a minimum of 10 working days prior to placement.

6.0 EQUIPMENT

6.01 General

All equipment for the installation of the anchor, anchor stressing, anchor testing and monitoring the test shall be suitable for the intended purposes and capable of working on the site under the prevailing access and clearance conditions.

The equipment used shall not cause damage to the anchor tendon or corrosion protection, or the soldier piles.

6.02 Anchor Testing Equipment

The equipment shall be capable of stressing the tendon to the maximum specified Test Load within the rated capacity.

The equipment shall permit the tendon to be stressed in increments so that the load in the tendon can be raised or lowered in accordance with the test specifications, and allow the anchor to be lift-off tested to confirm the lock-off load.

Dial gauges shall have at least a 75 mm travel and longer gauge stems or sufficient gauge blocks shall be provided to allow for greater travel where required. Gauges shall have precision of at least 0.02 mm.

Dial gauges shall permit the measurement of total tendon movement at every load increment to be read to the nearest 0.02 mm. The gauge shall have sufficient travel to record the total anchor movement at Test load without the need to reset at an interim point.

Jacks used for stressing tendons shall have a minimum ram extension of 150 mm.

Stressing equipment shall be calibrated within an accuracy of $\pm 2\%$ immediately prior to use.

Current calibration curves, bearing the seal and signature of an Engineer shall be provided for all gauges and jacks.

6.03 Grouting Equipment

Mixers and pumps shall be of an adequate capacity and hoses shall be sized to allow continuous grouting of an individual anchor within one hour. A colloidal mixer with a gauge to measure the water shall be used.

6.04 Temporary Anchor Concrete Placement Equipment

Continuous flight augers shall be used for the placement of concrete for temporary anchors up to a maximum ratio of hole diameter to length of 1:35. Open hole concrete placement shall be limited to a minimum hole diameter of 600 mm and a maximum ratio of hole diameter to length of 1:15.

7.0 CONSTRUCTION

7.01 General

The Contractor shall be responsible for the material, fabrication, installation, testing and monitoring of production and test anchors.

In addition for non-Owner designed anchors, the Contractor shall be responsible for design parameters and the design of the anchors.

The anchor system shall be according to this specification and the stamped shop drawings.

7.02 Structural Steel

Structural steel components shall be fabricated according to OPSS 906.

7.03 Prestressing Steel Bond Capacity

If not available from the prestressing steel manufacturer a prestressing strand bond capacity test shall be conducted on the strand in accordance with Appendix A of Recommendations for Prestressed Rock and Soil Anchors as published by the Post Tensioning Institute. The test information shall be submitted to the Contract Administrator prior to commencement of work.

7.04 Anchor Fabrication

Anchors shall be either shop or field fabricated in accordance with the approved drawings and schedules using personnel trained and qualified for this work.

The entire bond length shall be free of dirt, manufacturers lubricants, corrosion-inhibiting coatings or other deleterious substances that may significantly affect the grout-to-tendon bond or the service life of the anchor.

Joints in the protection system shall be made watertight.

7.05 Storage and Handling

Upon delivery, the fabricated anchors and the prestressing steel for fabrication of the tendons on site and all hardware shall be stored and handled in a manner that avoids mechanical damage, corrosion, and contamination with dirt or deleterious substances.

Handling of the tendons shall not cause mechanical damage or contamination to the prestressing steel nor the corrosion protection.

Cement and additives for grout shall be stored under cover and protected against moisture.

Lifting of any pregrouted tendons shall not cause excessive bending, which may debond the prestressing steel from the surrounding grout.

7.06 Corrosion Protection Details

7.06.01 Anchorage Protection

The corrosion protection of the tendon in the vicinity of the anchorage shall be carefully designed and built for a proper protection.

All stressing anchorages permanently exposed to the atmosphere or that have a concrete cover less than 50 mm shall be covered with a corrosion inhibiting compound-filled or grout-filled cover.

On strand tendons, the trumpet shall be long enough to enable the tendon to make a transition from the diameter of the tendon along the unbonded length to the diameter of the tendon at the wedge plate without damaging the encapsulation.

The trumpet shall be completely filled with a corrosion inhibiting compound or grout.

Corrosion inhibiting compound filled trumpets may be placed any time during construction. Grout shall be placed after the anchor has been tested and stressed to the lock-off load.

Corrosion inhibiting compound-filled trumpets shall have a permanent seal between the trumpet and the free stressing length corrosion protection.

Trumpets filled with grout shall have either a temporary seal between the trumpet and the unbonded length corrosion protection or the trumpet shall fit tightly over the unbonded length corrosion protection for a minimum of 0.3 m.

7.06.02 Free Stressing Length Protection

Corrosion protection of the free stressing length shall be provided by a sheath filled with a corrosion inhibiting compound or grout, or a heat shrinkable tube internally coated with a mastic compound. The corrosion inhibiting compound shall completely coat the tendon elements, fill the void between them and the sheath and fill the interstices between the wires of 7-wire strands. Provisions shall be made to retain the compound within the sheath.

The corrosion protective sheath surrounding the free stressing length of the tendon shall be long enough to extend into the trumpet, but shall not come into contact with the stressing anchorage during testing.

For pregouted encapsulations, a separate bond breaker shall be provided to prevent the tendon from bonding to the grout surrounding the free stressing length.

7.06.03 Free Stressing Length/Bond Length Transition

The transition between the corrosion protection for the bonded and free stressing lengths shall be designed and fabricated to ensure continuous protection from corrosion.

The corrosion protection surrounding the free stressing length of the tendon shall not contact the bearing plate or anchor head.

7.06.04 Coupler Protection

On encapsulated bar tendons the coupler and any exposed bar section next to it shall be covered with a corrosion proof compound or wax impregnated cloth tape. The coupler area shall be covered by a smooth plastic tube overlapping the adjacent sheathed tendon by at least 25 mm. The two joints shall be sealed each by a coated heat shrink sleeve of at least 150 mm length or approved equal. The corrosion proof compound shall completely fill the space inside the cover tube.

7.07 Construction of Anchor Holes

7.07.01 General

The holes shall be constructed to the diameter, orientation and length specified and detailed on the stamped working drawings. A drilling method that will establish a stable hole within the tolerances specified shall be used.

The sides and end of the completed hole shall be maintained in a stable condition.

The anchor hole entry shall be located within 300 mm of its plan location. The deviation of the holes entry angle from its specified inclination shall be no greater than ± 3 degrees.

Open holes and drilled casings shall be cleaned upon completion of drilling.

Holes open for longer than eight hours shall be re-cleaned prior to insertion of the tendon and grouting.

The following information shall be recorded for each hole and shall be submitted to the Contract Administrator:

1. Identification number.
2. Hole diameter.
3. Hole length.
4. Drilling procedure.
5. Soil, rock and ground water conditions encountered.
6. Time required to drill the hole.
7. Problems encountered.

The construction of the holes shall be inspected by the Quality Verification Engineer.

7.07.02 Waterproofing Holes

Waterproofing of holes shall be done where specified in accordance with the stamped drawings, procedures and equipment.

If during the water tightness test the leakage from a hole over a ten minute period exceeds 9.5 L the hole shall be consolidation grouted, redrilled and retested.

Redrilling shall be done when the grout strength is considerably less than the strength of the surrounding rock.

7.08 Anchor Installation

The anchors shall be installed as specified in the Contract Documents and detailed on the stamped working drawings.

Care shall be taken to ensure the sheathing, corrosion protection and grout tubes are not damaged during installation of the anchors.

Damaged anchors that cannot be repaired to the satisfaction of the Contract Administrator shall be replaced.

The devices used to centre the tendon and sleeves in the hole throughout the bond length of the tendon shall be maintained in position during installation. The centralizer shall support the tendon in the drill hole and position the tendon so a minimum grout cover of 12 mm is achieved. Centralizers used inside a sheath shall provide a nominal grout cover of 5 mm over the prestressing steel. All centralizers shall be designed to permit grout to flow freely around the tendon and up the drill hole.

The Contractor shall be responsible for determining the number of centring devices required, however, one unit shall be placed within 1 m of the bottom of the hole and another at the bond length - free stressing length interface. The centring devices shall not interfere with the required grouting.

The Quality Verification Engineer shall inspect anchor installation.

7.09 Grouting of Permanent Anchors

7.09.01 Quality of Grout Mixture

Any grout mixture showing evidence of dampness, lumps, harden pieces, or contamination shall not be incorporated in the work.

The Contractor shall be responsible for testing of bleeding, preparation and initial storage of grout cubes for determination of compressive strength, and delivery of the grout cubes to a testing laboratory designated by the Owner.

The Contractor shall employ staff from a testing company certified according to CSA A283 - Certification for Additional Tests 1B, by an organization accredited by the Standards Council of Canada, to carry out testing for bleeding, making and curing of grout cubes and early strength determination.

Making of grout cubes for compressive strength test and testing of bleeding, shall be done on a level, vibration free surface.

7.09.02 Bleeding Requirements

The testing for bleeding of the grout shall be according to CSA-A23.2-1B.

Prior to the grouting operation, in the presence of the Quality Verification Engineer and the Contract Administrator, a trial batch shall be mixed and the grout tested for bleeding, to ensure that the grout meets the requirements specified in the Contract Documents. The trial batch of grout shall not be used in the actual grouting operation unless approved by the Contract Administrator

During the grouting operation, bleeding measurements shall be performed on the grout sampled at the mixer. The measurements shall be performed at least once a day and as requested by the Contract Administrator.

The bleeding test results shall be submitted to the Contract Administrator in writing. The test results that indicate the grout is not meeting the requirements of the Contract Documents shall be reported immediately to the Contract Administrator and the grouting operation halted until the cause of the problem is identified and corrected.

7.09.03 Strength Requirements

7.09.03.01 Grout

Grout cubes shall be prepared as follows on site from the grout pumped into the anchor body:

- a) One set of grout cubes, consisting of three cubes, shall be made each day the grouting operations are carried out.
- b) The grout cubes shall be prepared according to CSA-A23.2-1B, and stored at a temperature between 15°C and 25°C and shall not be moved prior to demolding.
- c) The grout cubes shall be demolded and transported to the laboratory designated by the Owner within 24 hours \pm 4 hours.

- d) The grout cubes shall be transported in a sealed white opaque plastic bag containing at least 250 mL of water and maintained at a temperature between 15°C and 25°C.

7.09.03.02 Concrete

Concrete specimens shall be prepared and tested in conformance with OPSS 904.

7.09.03.03 Early Strength Determination

The Contractor shall prepare and test additional grout cubes to determine when the grout has attained a strength of 20 MPa.

The laboratory conducting the test shall be as specified herein.

7.10 Primary Grouting of Anchors

The grout shall be installed as specified in the Contract Documents and as detailed on the stamped working drawings.

The grout shall entirely fill the annular space between the anchor and the bore hole wall in the bond length.

Anchors shall be grouted as soon as practical after installation. The stressing tails of prestressing steel strands shall be aligned prior to initial set of the grout.

After grouting, the anchor shall remain undisturbed until the grout has reached the strength specified in the Contract Documents.

The following information shall be recorded for each anchor and submitted to the Contract Administrator:

1. Identification number.
2. Type of grout.
3. Grout pressure.
4. Volume of grout used.
5. Location of the top of the bond length grout.

The Quality Verification Engineer shall inspect primary grouting of anchors.

7.11 Post Grouting of Bond Length

When specified in the Contract Documents, post grouting of bond length shall be done in accordance with the submitted procedures and equipment.

The information required for recording primary grouting shall also be recorded for post grouting.

Ground movement shall be monitored and if excess movement is observed the grouting shall be terminated and the situation reported to the Contract Administrator.

The Contract Administrator shall be notified prior to the commencement of post grouting of both permanent and temporary anchors.

The Quality Verification Engineer shall inspect post grouting.

7.12 Placing of Bentonite Slurry or Lean Mix Concrete in the Free-Stressing Length

The method of placing the cement bentonite slurry or lean mix concrete shall be as specified in the Contract Documents and as detailed on the stamped working drawings.

The cement bentonite slurry for the free stressing length or lean mix concrete shall completely fill the annular space between the prestressing steel and the borehole wall and shall prevent any transfer of the anchor load to the free stressing zone.

The Quality Verification Engineer shall inspect placement of the cement bentonite slurry or lean mix concrete in the free stressing length.

7.13 Installation of Anchorage

The anchor bearing plate and the anchor head or nut shall be installed perpendicular to the tendon, within ± 3 degrees and centred on the bearing plate, without bending or kinking of the prestressing steel elements. Wedge holes and wedges shall be free of rust, grout and dirt. Special care shall be exercised to obtain the continuity of corrosion protection in the vicinity of the anchorage.

Anchorage permanently exposed to the atmosphere shall be covered with a corrosion inhibiting compound filled or grout filled cover.

7.14 Testing

7.14.01 General

Testing shall be carried out according to the stamped working drawings and as specified herein.

The maximum anchor load shall not exceed 80% of the guaranteed minimum ultimate strength of the tendon.

Stressing shall not commence until the grout has reached the specified 28 d strength.

Anchor tests shall be conducted at a time mutually acceptable to the Contractor and Contract Administrator.

The anchor tests shall be constantly monitored by the Quality Verification Engineer and the test results recorded and submitted to the Contract Administrator

7.14.02 Reaction System

The reaction system shall be designed by the Contractor and shall be installed as detailed on the stamped working drawings.

7.14.03 Reference System and Testing Equipment

The layout of the reference systems and testing equipment required for testing shall be as detailed on the stamped working drawings and as specified herein.

All reference beams shall be independently supported with the support firmly embedded in the ground at a distance of not less than 2.5 m from the reaction system. Reference beams shall be sufficiently rigid to support instrumentation such that variations in readings do not occur.

All gauges, scales and reference points attached to the test anchor shall be mounted so as to prevent movement relative to the test anchor during the test.

Dial gauges shall bear on the pulling head of the jack and their stems shall be coaxial with the tendon direction.

The jacks shall be secured with chains to provide adequate protection to personnel in the event of breakage of the anchor or stressing system.

7.14.04 Reference System Enclosures

The Contractor shall construct suitable enclosures to provide complete protection for personnel, equipment and instruments from variations in the weather conditions and disturbances during the test program.

These provisions shall include the following specific requirements:

- a) The test enclosures shall be weatherproof and provide adequate lighting and consistent and controllable heat in order to eliminate temperature variations;
- b) The test enclosure shall be provided with a level dry floor;
- c) A field office, equipped with tables, chairs, heating and lighting shall be provided adjacent to the test anchors.

7.14.05 Test Anchors

7.14.05.01 Installation of Test Anchors

Test anchors are required for temporary and permanent anchors. For contractor-designed anchors, at least one test anchor shall be installed and tested in each significantly different ground condition. For owner-designed anchors, the number of test anchors shall be as specified in the contract documents.

Test anchors shall be constructed using the materials, methods and procedures specified herein and as detailed on the stamped working drawings.

The test anchors shall not be incorporated in permanent or temporary works unless approved by the Contract Administrator.

7.14.05.02 Test Procedures and Measurements

The Anchor Test shall be carried out generally in accordance with the prevailing requirements of ASTM D1143-81 superseded where applicable by the procedure specified in this document.

With measurements recorded at intervals as directed by the Contract Administrator, the Anchor Test shall be conducted by incrementally loading and unloading according to the following schedule, or until the anchor fails.

AL, 0.25DL, 0.50DL, 0.75DL, 1.00DL,
0.75DL, 0.50DL, 0.25DL,
0.50DL, 0.75DL, 1.00DL, 1.25DL, 1.50DL,
1.25DL, 1.00DL, 0.75DL, 0.50DL, 0.25DL,
0.50DL, 0.75DL, 1.00DL, 1.25DL, 1.50DL, 1.75DL, 2.00DL*,
1.75DL, 1.50DL, 1.25DL, 1.00DL, 0.75DL, 0.50DL, 0.25DL
0.00DL**

Where AL = Alignment Load
DL = Design Load of Anchor

* At 2.00T, the load shall be maintained for 24 hours.

** At this point, an additional loading cycle, directly to a load up to the ultimate load as specified shall be conducted.

Each load shall be maintained for a minimum of 15 minutes or until the rate of displacement is not greater than 0.25 mm per hour.

Vertical and horizontal movement of the reaction system and tendon elongation shall be recorded with respect to an independent fixed reference point. A record of the test enclosure temperature is also required.

During the load hold periods, the anchor load shall not be allowed to deviate from the Test Pressure by more than 0.35 MPa.

When required repumping back to test load shall be done to compensate for small movements, hydraulic oil seepage and changes in temperature of the hydraulic oil.

The load shall always be returned to the specified test load prior to taking the movement reading at the specified interval. The test load shall not be exceeded during the period of observation.

7.14.05.03 Removal of Test Anchors

Unless otherwise specified the test anchors shall be removed flush with the surrounding ground and the test site shall be restored to its pretest conditions.

The test anchorages shall not be removed until the Contract Administrator has given permission to remove in writing.

7.14.06 Production Anchors

7.14.06.01 General

Every anchor shall be tested as specified in the Contract Documents according to the test and lift off subsections below.

7.14.06.02 Test Procedures and Measurements

During all tests, a record of load, time and movement shall be maintained at each loading interval. The Contractor shall provide to the Contract Administrator complete test records for all tests including plots of tendon movement versus tendon load, tendon load versus time, and tendon movement versus time.

7.14.06.03 Acceptance Tests

7.14.06.03.01 General

The anchorages shall meet the following proof test and lift-off test prior to acceptance.

7.14.06.03.02 Proof Test – Permanent Anchors

The following proof test shall be applied to the anchorages:

- a) After preloading the anchor to 1.5T for 30 minutes, the proof test shall be conducted by incrementally loading and unloading the anchor according to the following schedule, or until the anchorage fails, as determined by the Contract Administrator. The acceptance criteria apply to only the peak loading.

Loads shall be applied as follows:

0.25DL, 1.00DL, 1.50DL*, 1.00DL, 0.25DL

Where:

DL = Specified Working Load of Anchorage

- * At 1.50DL, the load shall be maintained for a minimum of 30 minutes. Measurement shall be recorded according to the following time increment schedule.

0 min, 1 min, 2 min, 3 min, 6 min, 9 min, 12 min, 15 min, 18 min, 30 min.

If the acceptance criteria as specified herein are met, the anchorage shall be prestressed to 1.50DL, then shall be locked-off at the transfer load of 1.10DL.

If the acceptance criteria as specified herein are not met in the 30 minute period the test shall be extended as required, with readings at 30 minute increments, up to 180 minutes.

If the acceptance criteria as specified herein are not met, the provisions of the clause "Unacceptable Results" as specified herein shall apply.

7.14.06.03.03 Proof Test – Temporary Anchors

Proof tests shall be performed on all the production anchors. The test shall be conducted by incrementally loading the anchor according to the following schedule:

AL, 0.25 DL, 0.50 DL, 0.75 DL, 1.00 DL, 1.20 DL, 1.33 DL, AL, and Adjust to Lock-off Load

Where:

AL = Alignment Load

DL = Specified Working Load of Anchorage

At the test load of 1.33 DL, the load shall be maintained constant for 10 minutes and the total movement shall be recorded at 1, 2, 3, 4, 5, 6, and 10 minutes. If the difference between the total movements at 1 minute and 10 minutes exceeds 1 mm, the test load shall be maintained for an additional 50 minutes and the movements

readings shall be recorded at 20, 30, 40, 50, and 60 minutes. The anchor shall be returned to AL after holding the load at test load for 50 minutes and the residual movement recorded.

The hydraulic pressure during the load hold period shall not deviate by more than 0.35 MPa and the load shall be returned to the test load prior to taking the movement reading. The total movements at each load increment shall be recorded.

7.14.06.04 Lock-Off Procedure

After testing has been completed, the load in the tendon shall be such that after seating losses (wedge seating), the specified Lock-Off Load has been applied to the anchor tendon.

The magnitude of the Lock-Off Load shall be 1.10 DL or as specified by the Contract Administrator and shall not exceed 70% of the ultimate load of the tendon or bar (F_{pu}).

The wedges shall be seated at a minimum load of 50% of F_{pu} . If the Lock-Off Load is less than 50%, shims shall be used under the wedge plate and the wedges seated at 50% of F_{pu} . The shims shall then be removed to reduce the load in the tendon to the desired Lock-Off Load. Bar tendons may be locked off at any load less than 70% of F_{pu} .

7.14.06.05 Lift-Off Tests

A minimum of three lift off tests shall be conducted at each site. Lift-off tests shall be conducted at times and locations determined by the Quality Verification Engineer. The lift-off test shall not be performed until 48 hours has elapsed after transferring the lock-off load. The method of testing shall be as detailed on the stamped working drawings, but will generally be as follows:

After transferring the load to the anchorage and prior to removing the jack, a lift-off test shall be conducted to confirm the magnitude of the load in the anchor tendon. This load is determined by re-applying load to the tendon to lift off the wedge plate (or anchor nut) without unseating the wedges (or turning the anchor nut). This moment represents zero time for any long term monitoring.

The stressing anchorages shall be suitable for conducting lift-off tests until the locked-in anchor load has been verified.

Acceptance criteria for lift-off tests shall be as specified herein.

If acceptance criteria for lift-off tests are not met, the provisions of the clause "Unacceptable Stressing Results" apply.

7.14.06.06 Acceptance Criteria

An anchorage is acceptable when:

1. The tendon movement for the last log time cycle is less than 1.5 mm where log time cycle is defined as 1/10 final time to final time*.

*3 for 30 min, 6 for 60 min, 9 for 90 min, 12 for 120 min, 15 for 150 min, 18 for 180 min.

and

2. The recorded elastic movement of the tendon exceeds 0.80 of the theoretical elongation of the free-stressing length.

The Quality Verification Engineer shall evaluate the anchorage lift-off test results and determine whether the anchor is acceptable.

An anchorage is acceptable when the load measured in the lift-off test is within 10% of the locked-in transfer load.

7.14.06.07 Unacceptable Stressing Results

Anchorage that do not meet the acceptance criteria for proof test shall be treated as follows:

1. Abandon the deficient anchor and install new anchors.
 2. Use the deficient anchor at a reduced working load and add another anchor to compensate for the load deficiency. In this case the acceptable working load for the anchorage shall be determined by conducting modified Proof Tests for the reduced loads until acceptance criteria are met. The acceptable working load shall be 50% of the load achieved in the acceptable modified Proof Test. The modified Proof Test will involve maintaining the test load for up to 12 hours for permanent anchors, and up to 6 hours for temporary anchors.
- or
3. Use post-grouting techniques to increase the anchor capacity to meet the acceptable criteria. Post-grout pressures shall not exceed 3600 kPa without approval of the Contract Administrator. The ground surface shall be observed and if pressure induced distress occurs the post-grouting operation should be immediately stopped.

Unless otherwise determined by the Contract Administrator anchorages that do not meet the acceptance criteria for lift-off tests shall be treated as follows:

1. The transfer load shall be adjusted to 1.10T. The lift-off test shall be repeated after a minimum time of 48 hours.
2. If the criteria for the lift-off test are not met after completing this procedure anchorage shall be treated as specified for anchorages that do not meet the proof test acceptance criteria.

7.15 Management of Excess Material

Management of excess material shall be according to OPSS 180.

7.16 As Built Drawings

As built drawings shall be prepared by the Contractor for Owner designed installations as follows:

1. For all work incorporated in the completed structure that required the submission of working drawings.
2. For all changes from the original Contract requirements.

The as built drawings shall be submitted to the Contract Administrator in a reproducible format prior to final acceptance of work.

The as built drawings shall bear the seal and signature of the Quality Verification Engineer.

8.0 MEASUREMENT FOR PAYMENT

8.01 Actual Measurement

8.01.01 Production Anchor

Measurement will be in metres of actual length of the anchor from anchor plate to tip.

8.01.02 Test Anchor

Measurement will be in metres of actual length of the anchor from anchor plate to tip.

8.01.03 Post-Grouting of Bond Length

Measurement will be in kg. of grout used.

9.0 BASIS OF PAYMENT

**9.01 Test Anchors - Item
Production Anchor - Item
Post-Grouting of Bond Length - Item**

Payment at the contract price for the above items shall be full compensation for all labour, equipment and material to do the work.

NOTES TO DESIGNER:

Anchor materials, installation and stressing may be modified on a project specific basis depending on the subsurface conditions and consequently depending on whether soil or rock anchors are being used and also depending on whether the anchors are temporary or permanent.

WARRANT: Always with these tender items.

DOWELS INTO ROCK – Item No.

Special Provision

May 22, 2007

1.0 GENERAL

1.1 Scope

The work for the above noted tender item shall be in accordance with OPSS 904, including all special provision, except as extended herein. This document specifies additional requirements for the supply, installation and testing of Dowels into Rock for the structure footings.

1.2 Instructions to Contractor

- 1.2.1 These instructions are to be read in conjunction with the Contract Drawings.
- 1.2.2 A total of 1 test Dowels into Rock are required for the Dowels into Rock at each structure footing.
- 1.2.3 Dowels shall extend through tremie concrete and into sound bedrock to the specified embedment depth.

1.3 Qualifications

- 1.3.1 **Qualifications of Staff from Contractor or Sub-Contractor Completing Work for the Dowels into Rock:** All work shall be performed under the direction of personnel experienced with all aspects associated with the installation of Dowels into Rock. Such experience shall have been obtained within the preceding five (5) years on projects of similar nature and scope to the work required for this project.
- 1.3.2 **Qualifications of the Quality Verification Engineer:** A resume of the work experience of the Quality Verification Engineer shall be submitted to the Contract Administrator for record purposes. The Quality Verification Engineer shall be a Professional Engineer licensed in the Province of Ontario having a minimum of five years of experience on projects of similar nature and scope to the work required for this project.
- 1.3.3 **Qualifications of the Design Engineer:** A resume of the work experience of the Design Engineer shall be submitted to the Contract Administrator for record purposes. The Design Engineer shall be a Professional Engineer licensed in the Province of Ontario having a minimum of five years of

experience of projects of similar nature and scope to the work required for this project.

1.4 Responsibilities of the Contractor

- 1.4.1 The Contractor shall prove the allowable bond stress by tests of the Dowels into Rock on non-production Dowels into Rock.
- 1.4.2 The Contractor shall supply equipment, materials and skilled personnel to install production Dowels into Rock and conduct the specified acceptance tests. It shall be the responsibility of the Contractor to constantly monitor the acceptance tests, maintain specified test loads and record test measurements as specified by the Contract Administrator.
- 1.4.3 The Contractor is responsible for materials and workmanship. Any remedial measures, required because of defects in materials or workmanship, shall be completed by the Contractor at no cost to the Owner.
- 1.4.4 The Contractor shall submit 4 copies of all Working Drawings to the Contract Administrator as outlined in Section 1.6.

1.5 Definitions

- 1.5.1 Dowels into Rock: reinforcing steel bar and non-shrink grout.
- 1.5.2 Design Engineer: An Engineer who has a minimum of five (5) years experience in all aspects associated with the installation of Dowels into Rock, including drilling, grouting and doweling work. The Design Engineer shall be retained by the Contractor to design various components for the installation and testing for the Dowels into Rock.
- 1.5.3 Quality Verification Engineer: An Engineer who has a minimum of five (5) years experience in all aspects associated with the installation of Dowels into Rock, including drilling, grouting and doweling work. The Quality Verification Engineer shall be retained by the Contractor to ensure conformance with the contract documents and issue certificate(s) of conformance.

1.6 Submissions and Working Drawings

- 1.6.1 Working Drawings shall consist of drawings, testing and installation records, procedures and reports, and work plans.
- 1.6.2 The Contractor shall submit Working Drawings to the Contract Administrator as follows:

- All Working Drawings that include drawing, testing and installation procedures and reports, and work plans shall be sealed and signed by the Design Engineer.
 - All Working Drawings that include testing and installation results and reports shall be signed and sealed by the Quality Verification Engineer.
- 1.6.3 Upon completion of testing or installation and testing for each component, the Contractor shall submit to the Contract Administrator a Certificate of Conformance sealed and signed by a Quality Verification Engineer. The Certificate shall state that the work has been carried out in conformance with the Working Drawings and in general conformance with the contract documents.
- 1.6.4 Working Drawings consisting of testing and installation records and reports shall be submitted four days after completion of testing and installation. All other Working Drawings shall be submitted two weeks prior to construction.
- 1.6.5 Working Drawings to be submitted include the following with further details outlined in the remainder of this specification:
- Design calculations, specifications and shop drawings covering all aspects of fabrication, installation and acceptance testing of Dowels into Rock.
 - Test results verifying the 28 day strength of non-shrink grout.
 - The method for constructing of the holes, maintaining the holes, and placing reinforcing steel bars, grout and other materials in the holes, including casing sizes, bit sizes and tremie grouting methods.
 - The procedures to verify hole length. Records of measurements that verify the hole length.
 - Records of all drilling procedures, rock conditions encountered, and installation times.
 - Test procedures for Dowels into Rock.
 - Drawings and design calculations for a suitable reaction system for the applied test loads.
 - Records of vertical and horizontal movements of the reaction system, and elongation of the reinforcing steel bar.

- Drawings and details for reference system arrangement.
- Current calibration curves shall be provided for all gauges.
- Complete test records for all tests including plots of dowel movement versus dowel load, dowel load versus time, and dowel movement versus time.
- Remedial measures for unacceptable stressing results.

1.7 Subsurface Conditions

1.7.1 Rock and groundwater conditions are described in the Foundation Investigation Report for this Contract.

2.0 MATERIALS

The non-shrink grout shall be an approved DSM 9.10.35 non-shrink grout.

The Contractor shall provide the following information from the manufacturer for non-shrink grout:

- Data sheets for the non-shrink grout,
- installation procedures

3.0 EQUIPMENT

3.1 General

3.1.1 All equipment for the installation of the Dowels into Rock shall be suitable for the intended purposes and capable of working on the site under the prevailing access and clearance conditions.

3.1.2 The equipment shall not cause damage to the reinforcing steel bars.

4.0 INSTALLATION

All work for the installation of Dowels into Rock shall be inspected by the Quality Verification Engineer.

4.1 Construction of Holes

4.1.1 The sides and end of the hole shall not be disturbed. The Contractor shall submit Working Drawings to the Contract Administrator that include the

method for constructing of the holes, maintaining the holes, and placing reinforcing steel bar, grout and other materials in the holes. All excavated material shall be removed from the site.

4.1.2 The hole diameters and hole length for this project are as specified on the Contract Drawings. Prior to commencing drilling operations, the Contractor shall submit Working Drawings to the Contract Administrator outlining devised procedures to verify hole length. The Contractor shall submit Working Drawings that include drilling operations records to the Contract Administrator that include the above noted records.

4.1.3 At all times, the Contractor shall keep a record of all drilling procedures, rock conditions encountered, and installation times. The Contractor shall submit Working Drawings to the Contract Administrator that include the above noted records.

4.2 Installation of Reinforcing Steel Bar

4.2.1 Reinforcing steel bar shall be installed in strict accordance with the Contract Drawings and installation procedures.

4.2.2 Centering devices shall be provided to ensure that the reinforcing steel bar is located centrally in the hole.

4.2.3 Dowels shall extend through the tremie concrete for the footing and into sound bedrock.

4.2.4 Reinforcing steel bar shall be installed after the dowel hole has been filled with non-shrink grout.

4.3 Grout

4.3.1 The non-shrink grout shall entirely fill the annular space between the reinforcing steel bar and side for the dowel hole.

4.3.2 The placement of grout for the test Dowels into Rock shall be identical to the production Dowels into Rock.

4.3.3 Non-shrink grout shall be placed into the dowel hole using tremie placement methods.

5.0 TESTING REQUIREMENTS

All work for the testing of Dowels into Rock shall be inspected by the Quality Verification Engineer.

5.1 General Testing Requirements

- 5.1.1 Refer to the attached Instructions to Contractor and the Contract Drawings for specific test details.
- 5.1.2 The Contractor shall install the number of Dowels into Rock specified in the contract documents for testing purposes. The purpose of the testing the Dowels into Rock is to prove the adequacy of the proposed anchor configuration and installation procedures under the site conditions, and to provide design parameters.
- 5.1.3 The equipment, labour and materials for test dowels shall be identical to Dowels into Rock at the each structure location.
- 5.1.4 The Contractor shall submit Working Drawings that include proposed procedures for testing of the dowels into Rock to the Contract Administrator. Such testing shall be executed in strict accordance with the proposed procedures of the Contractor.
- 5.1.5 The Quality Verification Engineer shall supervise the testing of the Dowels into Rock. The Contractor will notify the Contract Administrator of the testing schedule at least 10 days prior to commencement of the testing program. Testing for Dowels into Rock shall be conducted concurrently, as scheduled by the Contract Administrator. The tests shall normally be conducted between 8:00 hrs and 20:00 hrs from Monday to Friday, unless otherwise directed by the Contract Administrator.
- 5.1.6 The Contractor shall supply materials and skilled personnel to conduct the tests for the Dowels into Rock. The equipment and materials shall be capable of stressing the Dowels into Rock to the specified loads. It shall be the responsibility of the Contractor to constantly monitor the test, maintain specified test loads and to record test measurements as specified by the Quality Verification Engineer.
- 5.1.7 The test site shall be restored to its pre-test condition. Reinforcing steel bars used in tests shall be cut down 25 mm below the top of the sound bedrock.

5.2 Testing Location

- 5.2.1 The Contractor shall remove all loose rock down to sound bedrock at the test location.
- 5.2.2 The test Dowels into Rock shall be constructed at locations specified by the Contract Administrator.

- 5.2.3 If site conditions dictate, changes to the test locations will be considered. The Contractor shall provide the Contract Administrator at least 2 days notice in writing of this operation.

5.3 Testing Equipment

- 5.3.1 The dowels into rock will be carried out generally in accordance with the prevailing requirements of A.S.T.M. (Designation D1143-81) superseded where applicable by the procedures specified in this document.
- 5.3.2 The Contractor shall submit Working Drawings for a suitable reaction system for the applied test loads to the Contract Administrator. Jacks must be secured with chains to provide adequate protection for the personnel in the event of breakage of the reinforcing steel bar or stressing system.
- 5.3.3 The Contractor shall submit Working Drawings for the reference system arrangement to the Contract Administrator. All reference beams shall be as follows:
- The beams shall be independently supported with the support firmly embedded in the ground.
 - The testing device shall not apply compression to the bedrock surrounding the test for the Dowels into Rock, within a circle concentric with the dowel hole and a diameter equal to 4.0 m.
 - Reference beams shall be sufficiently rigid to support instrumentation such that variations in readings do not occur.
- 5.3.4 The Contractor shall construct suitable enclosures to provide complete protection for equipment and instruments from variations in the weather conditions and disturbances during the test program. These provisions must meet the approval of the Quality Verification Engineer and will include that the test enclosures must be weather-proof and provide a consistent temperature in order to eliminate temperature variations that could affect instrumentation.

5.4 Testing for Dowels Into Rock, and Report

- 5.4.1 At all times, the Contractor shall keep records of vertical and horizontal movements of the reaction system, elongation of reinforcing steel bar, and the record of test enclosure temperature. The movements shall be recorded with respect to an independent fixed reference point. The Contractor shall submit Working Drawings that include the above noted records to the Contract Administrator.

- 5.4.2 Dial gauges shall have at least a 76.2 mm (3.0 in.) travel. Longer gauge stems or sufficient gauge blocks shall be provided to allow for greater travel if required. Gauges shall have precision of at least 0.025 mm (0.0001 in.). The dial gauges shall be placed on smooth bearing surfaces mounted perpendicular to the direction of movement. All gauges, scales or reference points attached to the test anchor shall be mounted so as to prevent movement relative to the test anchor during the test. The Contractor shall submit Working Drawings that include details for current calibration and curves for all gauges to the Contract Administrator.
- 5.4.3 Jacks used for reinforcing steel bars shall have a minimum ram dimension of 153 mm (6.0 in.). The Contractor shall submit Working Drawings that include details for current calibration and curves for all gauges to the Contract Administrator.
- 5.4.4 Requirements for Clauses 5.4.1 to 5.4.4 shall be repeated as required at different testing locations.

5.5 Testing Loading

- 5.5.1 The testing procedures shall safely load test the Dowels into Rock in tension at a rate of approximately 100kN per minute to the specified test load. The load shall be increased by an additional 50 kN beyond this level as directed by the Quality Verification Engineer.
- 5.5.2 Each load shall be maintained for a minimum time of 15 minutes and until the rate of displacement is not greater than 0.25 mm (0.01 inches) per hour.

5.6 Acceptance Criteria

- 5.6.1 The following acceptance criteria apply:

The testing of dowels shall be carried out in advance of the instalment of Dowels into Rock at each structure location.

Tests for Dowels into Rock shall have a capacity of at least [insert value] kN. The Quality Verification Engineer shall report on the acceptance of the tests for Dowels into Rock. The Quality Verification Engineer shall report on the testing of the Dowels into Rock including recommendations for increasing embedment depth, if necessary.

6.0 BASIS OF PAYMENT

Payment at the contract unit price for the above tender item shall include full compensation for all labour, equipment, and materials to do the work. No additional payment will be made for

tests for Dowels into Rock which are deemed as included as part of the work for the above noted item.