

**FOUNDATION INVESTIGATION AND DESIGN REPORT
LEES AVENUE UNDERPASS REHABILITATION
HIGHWAY 417 EXPANSION FROM NICHOLAS STREET TO VANIER PARKWAY
OTTAWA, ONTARIO**

G.W.P. 4091-07-00, SITE No. 3-225

Geocres Number: 31G5-246

Report to

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PART 1: FACTUAL INFORMATION

1 INTRODUCTION

This report presents the factual findings obtained from a foundation investigation conducted for the proposed rehabilitation of the existing Lees Avenue underpass structure over Highway 417 in the City of Ottawa, Ontario. This investigation included the proposed construction staging area for the superstructure. The structure rehabilitation is part of the Highway 417 Expansion project, from Nicholas Street to Vanier Parkway.

The purpose of this investigation was to explore the subsurface conditions at the site and, based on the data obtained, to provide a borehole location plan, records of boreholes, stratigraphic profile and cross-sections, laboratory test results and a written description of the subsurface conditions. A model of the subsurface conditions was developed from the data obtained in the course of the investigation.

Thurber carried out the investigation as a sub-consultant to McCormick Rankin Corporation, under the Ministry of Transportation Ontario (MTO) Agreement Number 4009-E-0007.

2 SITE DESCRIPTION

The Lees Avenue Underpass is located at Exit 118 of Highway 417 approximately 2.5km east of Ottawa city centre. The underpass is approximately 800m east of the Rideau Canal and 400m west of the Rideau River. The site is located within an area of mixed land use, with low-rise apartment buildings, residential townhouses, sports fields and open space to the north of Highway 417, and high-rise apartment buildings, institutional centres, a bus station, and open fields to the south.

The existing Lees Avenue underpass is a three span structure with two piers and abutments. The piers are founded on spread footings and the abutments are founded on steel H-piles. The underpass spans a total length of 87 m across Highway 417. The north approach is approximately

7.0 m high and the south approach is approximately 4.5 m high. Photographs in Appendix C show the general layout of the site.

No stability issues were noted on the existing slopes adjacent to the abutments. A shallow erosion channel is present down the slope at the northeast end of the bridge. The existing pavement on the approaches exhibits fatigue cracking. Minor differential settlement appears to have occurred between the approach slabs and adjacent approaches as evidenced by asphalt patching on the concrete sidewalks.

The staging area that would be used for the prefabrication of the superstructure is located north-west of the structure in a grassed area between Robinson Avenue and Lees Avenue.

The site lies within the Ottawa Valley Clay Plains physiographic region, which comprises a clay plain interrupted by ridges of sand or rock. The bedrock consists of the Carlsbad Formation, comprising dark grey shale interbedded with calcareous siltstone and limestone.

3 SITE INVESTIGATION AND FIELD TESTING

The site investigation and field testing for this project were carried out in several stages. Initially between June 27 and June 30, 2011, three boreholes (Boreholes EBH-02, LA-02 and LA-09) were drilled along a potential alternate structure alignment located to the east of the current underpass. Subsequently between August 28 and September 8, 2011, seven boreholes (LE-01 to LE-07) were drilled at the underpass site. Additional rock core was recovered adjacent to Borehole LE-07 (designated Borehole LE-07A) on November 15, 2011. The investigation at the staging area consisted of three boreholes (Boreholes STG-7 to STG-9) and was undertaken between August 10 and August 12, 2011.

Details of the borehole depths and ground elevations are summarised in Table 3.1.

The approximate locations of the boreholes are shown on the attached Borehole Locations and Soil Strata Drawing in Appendix G.

The borehole locations were marked in the field and utility clearances were obtained prior to commencement of drilling operations. A road cut permit was obtained for boreholes drilled on Lees Avenue and City of Ottawa consent was obtained for the boreholes drilled in the proposed staging area off Lees Avenue/ Robinson Avenue.

The drilling was carried out using CME 55 and CME 75 truck-mounted drill rigs. A combination of hollow-stem auger drilling techniques and NQ coring methods was used to advance the boreholes. Overburden samples were obtained at selected intervals using a split spoon sampler in conjunction with Standard Penetration Testing (SPT).

A minimum 3.0 m length of bedrock core was recovered from Boreholes LE-02, LE-04, LE-07/7A, LA-02, LA-09, and STG-7 to STG-9. All rock cores were logged, and the Total Core Recovery (TCR), Solid Core Recovery (SCR), Rock Quality Designation (RQD) and the Fracture Indices (FI) were determined.

Table 3.1 – Borehole Termination Depths and Ground Elevation

Structure Elements	Borehole Number	Ground Elevation	Borehole Termination Elevation	Borehole Termination Depth (m)
South abutment & approach	LE-01	63.0	53.6	9.4
	LE-02	63.7	44.3	19.4
	LE-03	64.2	51.8	12.4
	LA-02	61.1	45.3	15.8
North abutment & approach	LE-04	68.6	46.2	22.4
	LE-05	68.3	58.9	9.4
	LE-06	69.3	53.8	15.5
	LA-09	60.7	46.4	14.3
New proposed pier	LE-07	60.1	47.0	13.1
	LE-07A	60.1	45.8	14.3
	EBH-02	60.1	49.2	10.9
Staging area	STG-7	61.2	48.1	13.1
	STG-8	60.8	45.9	14.9
	STG-9	61.4	47.1	14.3

The drilling and geotechnical sampling operations were supervised on a full time basis by a member of Thurber's technical staff. The supervisor logged the boreholes and processed the recovered soil and bedrock samples for transport to Thurber's laboratory for further examination and testing.

Several of the boreholes were advanced in conjunction with an environmental sampling program being carried out by Ecoplans Limited. The operations were supervised concurrently by a member of Ecoplan's technical staff, and selected soil samples recovered from the boreholes were transported to Ecoplan's laboratory for further examination and environmental testing. Several samples from Boreholes LE-01 and LE-02 and all soil samples recovered from Borehole LE-03 were used for environmental testing and accordingly these samples were not available for geotechnical testing.

Standpipe piezometers consisting of 19 to 25mm diameter PVC pipe with a slotted screen were installed in Boreholes LE-02, LE-05, LE-07 and STG-8. The completion details of the piezometer installations are summarised in Table 3.2. Following the final water level reading, the piezometers will be decommissioned in general accordance with MOE Regulation 903. Upon completion of drilling, boreholes without a piezometer installation were backfilled with a mixture of bentonite holeplug and cuttings then asphalt cold patch to surface.

Table 3.2 – Piezometer Details

Borehole	Tip Position (m)		Completion Details
	Depth	Elev.	
LE-02	17.7	46.0	Sand filter from 17.7 to 14.0 m, bentonite from 14.0 to 13.4m, bentonite/cutting mixture from 13.4 to 0.3m, then asphalt cold patch to surface.
LE-05	8.8	59.5	Sand filter from 8.8 to 5.2m, bentonite from 5.2 to 4.6m, bentonite/cutting mixture from 4.6 to 0.3m, then asphalt cold patch to surface.
LE-07	11.6	48.5	Sand filter from 11.6 to 5.5 m, bentonite from 5.5 to 0.3m, sand backfill from 0.3 to 0.1m, then asphalt cold patch to surface.
STG-8	12.2	48.6	Sand filter from 12.2 to 10.4 m, bentonite/cutting mixture from 10.4 to 0.6m, then bentonite to surface. PVC pipe protrudes 0.9 m above ground surface with 0.5 m diameter protective casing.

4 LABORATORY TESTING

All recovered soil samples were subjected to Visual Identification (VI) and moisture content determinations. Selected samples were also subjected to grain size distribution analyses (sieve and hydrometer) and Atterberg Limits testing, where appropriate. The results of this testing program are summarized on the Record of Borehole sheets included in Appendix A and on the figures presented in Appendix B. Point load tests were conducted on selected portions of the rock cores. The unconfined compressive strength (UCS) values of the rock were assessed from the point load data and these values are reported on the borehole logs.

5 DESCRIPTION OF SUBSURFACE CONDITIONS

Reference is made to the Record of Borehole sheets included in Appendix A and the Borehole Locations and Soil Strata Drawings in Appendix G. An overall description of the stratigraphy is given in the following paragraphs. However, the factual data presented in the Record of Borehole Sheets governs any interpretation of the site conditions.

In general terms, the stratigraphy encountered at this site consists of a pavement structure overlying granular fill, underlain by typically dense to very dense sand, sand and gravel and silty sand till. Shale bedrock was encountered beneath the sand/gravel and till.

More detailed descriptions of the individual strata encountered at the existing underpass site and the staging area are presented below. Information pertaining to the subsurface environmental conditions at the structure site are presented in a separate report by Ecoplans Limited.

5.1 Underpass Site (Boreholes LE-01 to LE-07, EBH-02, LA-02 and LA-09)

5.1.1 Asphalt and Concrete

In boreholes LE-01 to LE-06 drilled on Lees Avenue, a 75 to 130mm of layer of asphalt underlain by 225 to 650 mm of concrete was encountered over sand to sand and gravel fill.

Boreholes LE-07 and EBH-02 were drilled on the median of Highway 417. A 175 to 300mm thick layer of asphalt was encountered over sand fill.

In Borehole LA-09 drilled on Robinson Avenue which runs parallel to Highway 417, the pavement structure consisted of 100mm of asphalt over 200mm of sand and gravel fill.

5.1.2 Sand to Silty Sand Fill

The asphalt and concrete in the structure approaches (Boreholes LE-01 to LE-06) were underlain by sand to silty sand fill. The fill was generally described as brown, fine to coarse grained, with trace silt to silty, and trace to some gravel. Locally in Boreholes LE-03 and LE-05, sand and gravel layers were also encountered.

In Boreholes LE-07 and EBH-02 drilled at the median of Highway 417, the asphalt was underlain by fill consisting of grey to brown sand with some gravel and trace of silt.

Sand fill and silty sand fill were also encountered below the asphalt in Borehole LA-09 drilled on Robinson Street. A concrete obstruction was encountered in the fill at a depth of 1.5 m below ground surface in this borehole.

Borehole LA-02 was drilled in a grass area east of the south abutment and encountered brown to black sand fill. The upper 100mm was identified as topsoil, and possible hydrocarbon impact (coal tar) was noted in a sample from 0.8 to 1.2 m depth (Elev. 60.3 to 59.9).

Details of the fill thickness and elevations are provided in Table 5.1.

SPT 'N' values recorded in the fill typically ranged from 30 blows/ 0.3 m penetration to 50 blows with no penetration, indicating a dense to very dense relative density. Locally SPT 'N' values of 11 to 28 blows/ 0.3m penetration were recorded in Boreholes LE-02, LE-04, EBH-02, LA-02 and LA-09, indicating a compact condition.

The moisture content of the fill ranged from 3% to 17%.

Grain size distribution analyses were carried out on ten samples of the fill. The results of these tests are plotted on Figures B1 and B2 in Appendix B and summarised below.

Gravel %	3 to 35
Sand %	49 to 84
Silt & Clay %	7 to 46

Occasional cobbles and shale fragments were noted in the fill.

Table 5.1 – Fill Thickness Encountered in Boreholes

Underpass Elements	Borehole Number	Top Boundary of Fill (Elev.)	Base Boundary of Fill (Elev.)	Thickness of Fill (m)
South approach	LE-01	62.7	58.5	4.2
	LE-02	63.0	59.9	3.1
	LE-03	63.4	59.6	3.8
	LA-02	61.1	59.6	1.5
North approach	LE-04	68.2	60.7	7.5
	LE-05	67.8	61.6	6.2
	LE-06	69.0	63.2	5.8
	LA-09	60.6	56.1	4.5
New Pier	LE-07	59.9	58.1	1.8
	EBH-02	59.8	57.8	2.0

5.1.3 Topsoil

A topsoil layer consisting of black silt, some sand was encountered below the fill locally in Borehole LE-04 on the eastern side of the north approach. The organic layer was 0.6 m thick and the depth to base of the layer was 8.5 m (Elev. 60.1). The moisture content of the organic layer was 34%.

5.1.4 Silty Sand to Sand and Gravel

Various interbedded deposits of sand, sand and gravel, and silty sand were encountered in all boreholes except Borehole LE-05. These cohesionless deposits were variously described as brown, light brown, dark brown or grey. In borehole LE-02, a black zone (possible coal tar) was encountered between 4.0 to 4.5 m depth (Elev. 59.7 to 59.2) and a hydrocarbon odour was noted near 7.0 m depth (Elev. 56.7).

The thicknesses of the silty sand to sand and gravel deposits and the elevations at which these materials were encountered are summarised in Table 5.2.

SPT ‘N’ values within the sand/gravel deposits are typically between 35 blows/ 0.3 m penetration and 50 blows/ 0.025 m penetration, indicating a dense to very dense relative density. Locally in borehole LE-02, one SPT ‘N’ value of 23 blows/ 0.3 m penetration was recorded, indicating a compact condition.

Rock coring equipment was required to advance Borehole LE-04 through very dense sand/gravel with possible cobbles below 13.4 m depth.

Moisture contents of the silty sand to sand and gravel varied between 3% and 20%.

Table 5.2 – Sand and Gravel Deposit Thickness Encountered in Boreholes

Underpass Elements	Borehole Number	Top Boundary of Sand/Gravel Deposit (Elev.)	Base Boundary of Sand/Gravel Deposit (Elev.)	Thickness of Sand/Gravel Deposit (m)
South approach	LE-01	58.5	55.3	3.2
	LE-02	59.9	54.6	5.3
	LE-03	59.6	55.0	4.6
	LA-02	56.5	52.0	4.5
North approach	LE-04	56.4	51.9	4.5
	LE-06	61.7	57.1	4.6
	LA-09	56.1	53.8	2.3
		53.1	52.0	1.1
New Pier	LE-07	58.1	49.7	8.4
	EBH-02	57.8	49.2 *	> 8.6

* Borehole was terminated within the sand deposit.

Grain size distribution analyses were carried out on 13 samples of the sand/gravel. The results of these tests are plotted on Figures B3 to B5 in Appendix B and summarised below.

Gravel %	0 to 55
Sand %	29 to 97
Silt & Clay %	2 to 51

5.1.5 Silty Sand Till

Discontinuous layers of silty sand till were encountered below the fill in Boreholes LE-04, LE-05, LE-06 and LA-02, and below the sand deposits in all boreholes except Boreholes LE-05 and EBH-02. The silty sand till was described as brown, grey and dark grey, and locally grades to sandy silt and silt.

The till layer thickness and the elevations at which the till was encountered are summarised in Table 5.3.

SPT 'N' values recorded in the silty sand till typically ranged from 30 blows/ 0.3 m penetration to 50 blows for no penetration, indicating a dense to very dense relative density. The high recorded 'N' values may reflect the presence of cobbles and boulders in the till. Rock coring equipment was required to advance through very dense till with possible cobbles and boulders from 12.5 to 16.2 m in Borehole LE-02 and from 13.7 to 15.2 m in Borehole LE-06.

Table 5.3 – Silty Sand Till Thickness Encountered in Boreholes

Underpass Elements	Borehole Number	Top Boundary of Lower Till (Elev.)	Base Boundary of Lower Till (Elev.)	Thickness of Lower Till (m)
South approach	LE-01	55.3	53.6 *	> 1.7
	LE-02	54.6	47.4	7.2
	LE-03	55.0	51.8 *	> 3.2
	LA-02	59.6 52.0	56.5 48.9	3.1 3.1
North approach	LE-04	60.1	56.4	3.7
		51.9	49.3	2.6
	LE-05	61.6	58.9 *	> 2.7
	LE-06	63.2	61.7	1.5
	LE-06	57.1	53.8 *	> 3.3
New Pier	LA-09	53.8	53.1	0.7
		52.0	49.4	2.6
New Pier	LE-07	49.7	47.9	1.8

* Borehole was terminated within the till layer

Moisture contents of the till ranged from 2% to 25%, typically 8% to 18%.

Grain size distribution analyses were carried out on six samples of the till. The results of these tests are plotted on Figure B6 in Appendix B and summarised below.

Gravel %	3 to 11
Sand %	54 to 70
Silt %	22 to 31
Clay %	3 to 9

Glacial till inherently contains cobbles, boulders and shale slabs.

5.1.6 Shale Bedrock

Bedrock was encountered below the silty sand till and proven by coring in Boreholes LE-02, LE-04, LE-07, LE-07A, LA-02 and LA-09. The depths and elevations at which bedrock was encountered are summarised in Table 5.4. Based on visual identification and geologic mapping, the bedrock belongs to the Carlsbad Formation.

Table 5.4 – Depths and Elevations of Bedrock Surface

Underpass Elements	Borehole	Bedrock Surface	
		Depth (m)	Elevation (m)
South Approach	LE-02	16.3	47.4
	LA-02	12.2	48.9
North Approach	LE-04	19.4	49.3
	LA-09	11.3	49.4
New Pier	LE-07	12.2	47.9
	LE-07A	11.3	48.8

The bedrock was described as grey to dark grey shale. The shale is slightly weathered to fresh with hard thin limestone interbeds throughout. Fractured zones were noted at depths of 13.6 to 14.0 m in Borehole LA-02 and at 11.2 m depth in Borehole LA-09. A clay seam was noted at 13.3 m depth (Elev. 47.8) in Borehole LA-02 and at 12.0 m depth (Elev. 48.1) in Borehole LE-07A. Bituminous seams believed to be naturally occurring were encountered below 14.3 m (Elev. 46.8) in Borehole LA-02 and throughout the rock core in Borehole LE-07A.

Total Core Recovery (TCR) in the bedrock was between 90% and 100%. The RQD values are between 55% and 100%, indicating fair to excellent quality rock. The Fracture Index (FI) of rock, expressed as fractures per 0.3 m of core, ranged from 0 to 7.

The estimated unconfined compressive strength of the shale, interpreted from point load tests conducted on intact rock cores, was between 15 and 28 MPa, indicating a weak to medium strong rock. Limestone interbeds are likely to have higher strengths.

5.1.7 Water Levels

Water was recorded in open Boreholes LE-07A, EBH-02, LA-02 and LA-09 at depths of 2.4 to 5.4 m upon completion of drilling and coring. These water levels represent unstabilized measurements and may reflect the addition of water to the boreholes during coring.

Standpipe piezometers were installed in boreholes LE-02, LE-05 and LE-07. The groundwater depths and elevations measured in the piezometers are summarised in Table 5.5.

The groundwater level is susceptible to seasonal fluctuations. In particular, the groundwater level may be at a higher elevation after the spring snowmelt or after periods of heavy rainfall.

Table 5.5 – Ground Water Monitoring Data

Underpass Elements	Borehole	Date	Water Level (m)	
			Depth	Elevation
South Approach	LE-02	01-Sep-11	4.7	59.0
		12-Oct-11	4.7	59.0
North Approach	LE-05	01-Sep-11	8.3	60.0
		12-Oct-11	7.0	61.3
New Pier	LE-07	20-Sep-11	5.0	55.1
		12-Oct-11	5.1	55.0

5.2 Staging Area (Boreholes STG-7, STG-8, STG-9)

5.2.1 Silty Sand Fill

Brown fill was encountered surficially in the boreholes drilled at the staging area. The fill typically consisted of silty sand, locally sandy silt in the upper 0.8 m in Borehole STG-7. A black zone (possible coal tar impact) was encountered between 1.5 and 1.8 m depth (Elev. 59.7 and 59.4) in Borehole STG-7, and a hydrocarbon odour was noted within the sand fill below this depth. Concrete powder and possible concrete debris were encountered near 1.6 m depth in Borehole STG-8.

Details of the fill thickness and upper and lower boundaries are provided in Table 5.6.

Table 5.6 – Fill Thickness Encountered in Boreholes

Location	Borehole Number	Top Boundary of Fill (Elev.)	Base Boundary of Fill (Elev.)	Thickness of Fill (m)
Staging Area	STG-7	61.2	56.6	4.6
	STG-8	60.8	59.0	1.8
	STG-9	61.4	60.6	0.8

SPT ‘N’ values recorded in the fill typically ranged from 17 to 29 blows/ 0.3 m penetration, indicating a compact relative density. An SPT ‘N’ value of 50 blows/ 0.15 m penetration was recorded in Borehole STG-8, indicative of cobbles or concrete rubble within the fill.

Moisture contents of the fill ranged between 3% and 21%.

Grain size distribution analysis was carried out on one sample of the silty sand fill. The results of the test are plotted in Figure B7, Appendix B, and summarised below:

Gravel %	10
Sand %	61
Silt %	26
Clay %	3

5.2.2 Sand

A sand deposit was encountered below the fill in all boreholes. The sand layer was brown to grey, with some silt and trace to some gravel.

The thickness and boundary elevations of the sand layer are summarised in Table 5.7

Table 5.7 – Sand Layer Thickness Encountered in Boreholes

Location	Borehole Number	Top Boundary of Sand Layer (Elev.)	Base Boundary of Sand Layer (Elev.)	Thickness of Sand Layer (m)
Staging Area	STG-7	56.6	53.5	3.1
	STG-8	59.0	54.7	4.3
	STG-9	60.6	54.1	6.5

SPT ‘N’ values recorded in the sand layer ranged from 16 blows/ 0.3 m penetration to 54 blows /0.15 m penetration, indicating a compact to very dense condition. Cobbles and boulders may be present. Moisture contents varied between 3% and 18%.

Grain size distribution analyses were carried out on four samples taken from the sand layer. The results of these tests are presented in Figure B8 in Appendix B and summarised below:

Gravel %	0 to 14
Sand %	74 to 88
Silt & Clay %	11 to 15

5.2.3 Sand and Gravel

A 1.5 m thick layer of sand and gravel was encountered below the sand in Boreholes STG-8 and STG-9. The sand layer was dark grey. The thickness and boundary elevations of this layer are summarised in Table 5.8

Table 5.8 – Sand and Gravel Layer Thickness Encountered in Boreholes

Location	Borehole Number	Top Boundary of Layer (Elev.)	Base Boundary of Layer (Elev.)	Thickness of Layer (m)
Staging Area	STG-8	54.7	53.2	1.5
	STG-9	54.1	52.6	1.5

SPT ‘N’ values of 75 blows/ 0.3 m penetration and 50 blows /0.15 m penetration were recorded in the sand and gravel, indicating a very dense condition. Cobbles and boulders may be present. Moisture contents of 7% and 10% were measured.

The results of a grain size distribution analysis carried out on a sample of the sand and gravel are presented in Figure B9 in Appendix B and summarised below:

Gravel %	47
Sand %	46
Silt & Clay %	7

5.2.4 Silty Sand to Sandy Silt Till

Grey to dark grey silty sand to sandy silt till with some clay, trace of gravel and occasional shale fragments was encountered below the sand/gravel in the boreholes. The till boundary elevations and thicknesses are summarised in Table 5.9.

Table 5.9 – Till Layer Thickness Encountered in Boreholes

Location	Borehole Number	Top Boundary of Till (Elev.)	Base Boundary of Till (Elev.)	Thickness of Till (m)
Staging Area	STG-7	53.5	52.0	1.5
	STG-8	53.2	51.0	2.2
	STG-9	52.6	50.1	2.5

SPT ‘N’ values typically ranged from 47 blows/ 0.3 m penetration to 50 blows/ 0 m penetration, indicating a dense to very dense relative density. In borehole STG-7, an SPT ‘N’ value of 12 blows/ 0.3 m penetration was recorded, indicating a localised compact condition.

Measured moisture contents ranged from 8% to 15%.

Glacial tills inherently contain cobbles, boulders and shale slabs.

5.2.5 Shale Bedrock

Bedrock was encountered below the silty sand till and proven by coring in all boreholes drilled at the staging area. The depths and elevations at which bedrock was encountered are summarised in Table 5.10.

The bedrock comprises grey to black, fresh shale with hard thin limestone interbeds throughout. In boreholes STG-7 and STG-9, highly fractured zones were encountered at depths of 10.0 to 10.2 m and 12.0 to 12.1m, respectively.

Table 5.10 – Depths and Elevations of Bedrock Surface

Location	Borehole	Bedrock Surface	
		Depth (m)	Elevation (m)
Staging Area	STG-7	9.1	52.0
	STG-8	9.8	51.0
	STG-9	11.3	50.1

Total Core Recovery (TCR) in the bedrock was 100%. The RQD values ranged from 90 to 100%, indicating excellent rock quality. The Fracture Index (FI) of the rock, expressed as fractures per 0.3 m of core, generally ranged from 0 to 3.

The estimated unconfined compressive strength of the shale, interpreted from point load tests conducted on intact rock cores, ranged from 18 to 24 MPa, indicating a weak rock strength classification. Limestone interbeds are likely to have higher strengths.

5.2.6 Water Levels

Water was observed in the open boreholes at depths of 3.9 to 8.8 m upon completion of coring. These water levels represent unstabilized measurements and may reflect the addition of water to the boreholes during coring operations.

A standpipe piezometer was installed in Borehole STG-8. The groundwater depths and elevations measured in the piezometer are summarised in Table 5.11.

Table 5.11 – Ground Water Monitoring Data

Location	Borehole	Date	Water Level (m)	
			Depth	Elevation
Staging Area	STG-8	02-Sep-11	7.0	53.8
		20-Sep-11	5.9	54.9
		12-Oct-11	6.2	54.6

The groundwater level is susceptible to seasonal fluctuations. In particular, the groundwater level may be at a higher elevation after the spring snowmelt or after periods of heavy rainfall.

6 MISCELLANEOUS

Borehole locations were selected and established in the field by Thurber Engineering Ltd. Surveyors from MMM Group provided co-ordinates and the ground surface elevations for the boreholes.

Underground Service Locators Inc. obtained utility clearances on behalf of Thurber for the selected borehole locations prior to drilling.

Eastern Ontario Diamond Drilling Ltd. from Hawkesbury, Ontario supplied truck mounted CME 55 and CME 75 drill rigs and conducted the drilling, sampling and in-situ testing operations.

The field investigation was supervised by Mr. Luke Gilarski, E.I.T., Mr. David Ametrano, E.I.T. and Mr. Ryan Kromer, E.I.T. of Thurber.

Routine laboratory testing was carried out by Thurber Engineering Ltd.

Overall planning and supervision of the field program was conducted by Ms. Lindsey Blaine, E.I.T. Interpretation of the data and preparation of the report were carried out by Ms. Mei Cheong, M.Phil. and Mr. M.R. Anderson, P.Eng.

The report was reviewed by Dr. P.K. Chatterji, P.Eng., a Designated Principal Contact for MTO Foundations Projects.

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**FOUNDATION INVESTIGATION AND DESIGN REPORT
LEES AVENUE UNDERPASS REHABILITATION
HIGHWAY 417 EXPANSION FROM NICHOLAS STREET TO VANIER PARKWAY
OTTAWA, ONTARIO**

G.W.P. 4091-07-00, SITE No. 3-225

Geocres Number: 31G5-246

PART 2: ENGINEERING DISCUSSION AND RECOMMENDATIONS

7 GENERAL

This report presents interpretation of the geotechnical data in the factual report and presents geotechnical design recommendations to assist the design team to select and design a suitable foundation system for the rehabilitation works proposed at Lees Avenue Underpass.

The existing Lees Avenue Underpass is a three-span structure supported by two piers and two abutments. The underpass spans a distance of approximately 87 m over the eight-lane Highway 417 and the bridge deck is approximately 15.5 m wide. The piers are founded on spread footings and the abutments are founded on steel H-piles driven to refusal. The existing approach embankment is approximately 7.0 m high at the north end of the structure and approximately 4.5 m high at the south end.

The preliminary General Arrangement drawing provided by McCormick Rankin Corporation (May 2011) indicates that the existing pier footings are founded near Elev. 57.0 m and the underside of the pile caps at the north and south abutments are at Elev. 63.5 and 59.5 m, respectively.

Rehabilitation of the structure will involve rapid removal and replacement of the existing bridge deck. The existing three-span bridge will be replaced with a two-span bridge in the following sequence:

- All three spans of the existing superstructure will be removed concurrently using rapid removal methods;
- The north pier will be removed to the top of footing and the south pier and footing will be removed entirely;
- The existing abutments and wing/retaining walls will be modified and the new pier will be constructed in the Highway 417 median;

- The new superstructure will be installed using rapid replacement methods.

As part of the rehabilitation works, the existing abutment wing walls will be extended and road grade on Lees Avenue will be raised by approximately 1.0 to 1.5 m.

A temporary staging area for prefabrication of the new superstructure is proposed in the area between Lees Avenue and Robinson Avenue to the northwest of the underpass. Design of temporary foundations in the staging area will be the responsibility of the Contractor.

Geotechnical recommendations and design parameters are presented in subsequent sections to enable assessment and design of the following:

- Foundations for the new pier to carry loads from the new bridge deck and to provide seismic/uplift resistance;
- The existing abutment foundations;
- Increased approach embankment height;
- Removal of existing pier foundations;
- Temporary roadway protection works;
- Foundations for superstructure fabrication in the staging area.

The discussion and recommendations presented in this report are based on the information provided by MRC and on the factual data obtained in the course of the investigation.

8 NEW PIER FOUNDATIONS

The two existing piers will be removed and replaced with a single new pier at the median of Highway 417. The new pier and foundation will support the new bridge deck as well as provide resistance to seismic uplift/rocking.

Augered caissons are the preferred foundation type for the new pier due to the narrow median access, required uplift resistance and structural sensitivity to differential settlement (including immediate elastic deflection). Recommendations for caisson design are provided in the following sections. Recommendations for alternative foundation types (driven piles, spread footings) are also presented in the event that the design concept changes. A comparison of foundation alternatives, including micropiles, based on advantages and disadvantages of each is provided in Appendix D.

8.1 Caissons

8.1.1 Axial Capacity

The sand and till at the site are typically dense to very dense but essentially non-cohesive in nature. The sand and till contain cobbles and boulders. Groundwater was measured at a depth of approximately 5 m (Elev. 55.1) at the proposed pier location.

Considering the potential for base and sidewall instability in the non-cohesive soils below the groundwater level, construction of caissons end-bearing within the cohesionless sand/till is not recommended. It is therefore recommended that the caissons be socketed into bedrock. The caissons will provide resistance to both foundation loads and seismic uplift at the new pier.

Shale bedrock was encountered at a depth of 12.2 m (Elev. 47.9) in Borehole LE-07. The factored axial geotechnical resistances at ULS recommended for typical caisson designs socketed 3 and 5 m into bedrock are provided in Table 8.1. These values include the geotechnical resistance factors of 0.4 and 0.3 specified in the CHBDC for axial compression and uplift, respectively.

Table 8.1- Axial Geotechnical Resistance of Caissons

Socket Length in Shale (m)	Caisson Diameter (m)	Factored Axial Resistance at ULS (kN)	Factored Uplift Resistance at ULS (kN)
3	0.9	5,000	2,000
	1.2	8,000	2,500
	1.5	10,000	3,000
5	0.9	6,500	3,000
	1.2	10,000	4,000
	1.5	12,500	5,000

The factored axial resistances provided in the table were computed based on factored end-bearing and shaft resistance values of 4,000 and 300 kPa, respectively. The SLS condition will not govern for caissons founded in bedrock.

Downdrag on the caisson is not considered to be an issue at this site.

8.1.2 Caisson Installation

Caisson installation must be in accordance with OPSS 903.

The caissons will generally be advanced through dense to very dense sand, sand and gravel, and silty sand till potentially containing cobbles, boulders and shale slabs. The caisson drilling equipment supplied by Contractor must be capable of advancing through these materials and penetrating or pushing aside potential obstructions. Augering/coring equipment must also be able to penetrate shale bedrock with frequent hard limestone layers.

Construction of the caissons will require use of a steel liner advanced 0.5 to 1.0 m into the bedrock surface to support the sidewalls, minimize groundwater inflow, and enable machine-cleaning of the socket base. After the liner is installed, the bedrock socket must be advanced to found the caisson in sound bedrock based on the minimum embedment lengths recommended in Section 8.1.1.

Caisson concrete should be placed within 8 hours of excavation to minimize softening of the shale bedrock in the socket.

An NSSP notifying the Contractor of the specific subsurface conditions and installation requirements at this site should be included in the contract documents. Suggested wording is presented in Appendix E. Selection of the type of equipment and method of installation is the responsibility of the Contractor.

8.1.3 Lateral Resistance

The lateral resistance of the caisson may be calculated using a value for the coefficient of horizontal subgrade reaction (k_s) and ultimate lateral resistance (p_{ult}) as follows:

$$k_s = n_h \cdot z / D \quad (\text{kN/m}^3)$$

$$p_{ult} = 3 \cdot \gamma \cdot z \cdot K_p \quad (\text{kPa})$$

where

z = depth of embedment of caisson in metres

D = caisson width in metres

n_h = value from Table 8.2

γ = unit weight (Table 8.2)

K_p = passive earth pressure coefficient (Table 8.2)

The above equations and recommended parameters may be used to analyse the interaction between a caisson and the surrounding soil. The lateral pressures obtained from the analysis should not exceed the ultimate lateral resistance.

Table 8.2 – Parameters for Lateral Caisson Resistance

Structure Element	Top Elevation	Bottom Elevation	n_h (kN/m^3)	K_p	Unit Weight (kN/m^3)	Soil Conditions
New Pier	60.1	57.1	4000	3.0	21	Sand fill, compact to dense
	57.1	47.9/ Bedrock	10,000	3.3	11*	Sand layer and silty sand till, dense to very dense.

* Buoyant unit weight below the water table.

Caisson interaction should be considered with reference to CHBDC Clause 6.8.9.2. Where a caisson group is oriented *perpendicular* to the direction of loading, group action may be considered by reducing values for k_s by a reduction factor R as follows:

Caisson Spacing Perpendicular to Direction of Loading	Horizontal Subgrade Reaction Reduction Factor, R
4 D*	1.00
1 D*	0.50
* D is the width of the caisson, and spacing is measured centre to centre	

Where a caisson group is oriented *parallel* to the direction of loading, group action may be considered by reducing values for k_s by a reduction factor R as follows:

Caisson Spacing Parallel to Direction of Loading	Horizontal Subgrade Reaction Reduction Factor, R
8 D	1.00
6 D	0.70
4 D	0.40
3 D	0.25

Intermediate values may be obtained by interpolation.

In bedrock, the caisson lateral resistance may be calculated as follows:

$$\begin{aligned}
 k_s &= 67 \cdot s_u / D \quad (\text{kN/m}^3) \\
 p_{ult} &= 6 \cdot s_u \quad (\text{kPa}) \\
 \text{where } D &= \text{caisson width in metres} \\
 s_u &= 400 \text{ kPa (undrained shear strength of bedrock mass, kPa)}
 \end{aligned}$$

The above equations and recommended parameters may be used to analyse the interaction between a caisson and the surrounding rock. The lateral pressures obtained from the analysis should not exceed the ultimate lateral resistance. The lateral resistance of the caisson in bedrock will not be significantly affected by caisson spacing.

The spring constant, K, for analysis may be obtained by the expression, $K = k_s \cdot L \cdot D$ (kN/m), where k_s is the coefficient of horizontal subgrade reaction (kN/m³), D is the caisson width (m) and L is the length (m) of the caisson segment or element used in the analysis. The ultimate lateral resistance on any one segment of caisson, P_{ult} , may be obtained from the expression, $P_{ult} = p_{ult} \cdot L \cdot D$. This represents the ultimate load at which the caisson fails and will not support any additional load at greater displacements.

Battering of caissons to provide horizontal resistance is not recommended.

8.2 Spread Footings on Sand and Gravel

Spread footings founded on dense to very dense sand to sand and gravel are considered feasible to support the new pier from a geotechnical perspective. However, construction access restrictions within the narrow median may preclude construction of spread footings and the accompanying roadway protection requirements. Further, spread footings are not expected to provide sufficient resistance to seismic events (longitudinal rocking).

If employed, spread footings should be founded on dense to very dense sand/gravel below the level of all existing fill and disturbed soil, at or below Elev. 57.5 m. Footings founded at this level should be designed using a factored geotechnical resistance at ULS of 750kPa and an unfactored resistance at SLS of 500 kPa. The recommended ULS value includes a resistance factor of 0.5 as per Table 6.1 of the CHBDC.

The geotechnical resistances quoted above are for concentric, vertical loads only. In the case of eccentric or inclined loading, the geotechnical resistance must be calculated as illustrated in the CHBDC Clause 6.7.3 and Clause 6.7.4.

The sliding resistance of concrete poured on the sand/gravel may be computed on the basis of an unfactored coefficient of friction of 0.6. This value requires a degree of sliding movement to occur to fully mobilize the resistance.

Where structural analyses suggest seismic uplift is a concern, anchorage may be provided using rock anchors or by considering alternative foundations such as piles or caissons socketed in rock.

Excavation and backfilling of foundations must be carried out in accordance with OPSS 902. The bases of the foundation excavations must be inspected by the QVE to confirm that the exposed surface conforms to the design requirements and has been adequately prepared to receive concrete. Any loose or disturbed material must be removed and the footing founded on competent dense to very dense native soil. Where sub-excavation is required to remove unsuitable material from below the design founding level, the founding surface should be re-established using concrete fill of the same class of concrete as used in the footing.

Although not encountered in Boreholes EBH-2 and LE-07 drilled at the pier, suspected coal tar contamination was encountered between Elevation 59.2 and 60.3 in Boreholes LE-02 and LA-02 located at the south abutment. The potential exists that contamination may be encountered during excavation at the pier.

8.3 Steel H-Piles

The use of driven H-piles may be considered to support the pier. As the on-site soils consist of dense to very dense sand/gravel or till with possible cobbles and boulders, it is considered unlikely that all piles could be driven to bedrock and some piles may encounter refusal in the sand/till. HP 310 X 110 piles driven to practical refusal in the very dense sand/gravel, till or shale bedrock should be designed using the following axial geotechnical resistances:

$$ULS_f = 1,800 \text{ kN}$$

$$SLS = 1,600 \text{ kN}$$

Due to the variable relative density of the sand recorded in Boreholes LE-07 and EBH-02, the depth to refusal of the piles could vary significantly between adjacent piles. Considering the potential for highly variable pile lengths, as well as the low resistance to uplift/ seismic forces provided by driven piles, it is recommended that steel H-piles, if employed, be installed by predrilling and socketing into shale bedrock.

Piles socketed in shale should be installed by drilling or coring 0.6 m diameter sockets to the required depth, inserting the pile, then backfilling the annular space between the pile and augered hole with concrete. The socket should extend at least 3 m below the shale surface.

For HP 310x110 steel H-piles placed in rock sockets, a factored axial geotechnical resistance at ULS of 2,000 kN is recommended. The SLS condition will not govern for piles founded in bedrock.

The structural resistance of the pile must be checked by the structural designer.

Piles socketed into bedrock to develop uplift resistance should be extended at least 3 m below the bedrock surface. The uplift resistance provided per pile socket should be based on a factored sidewall resistance at ULS of 200 kPa between socket concrete and weathered shale sidewall. This value includes a resistance factor of 0.3 as per the CHBDC.

Downdrag on the piles is not considered to be an issue at this site.

Lateral resistance provided by the piles may be computed as outlined for augered caissons. Alternatively battered piles may be employed.

Pile installation must be in accordance with OPSS 903.

For piles set in rock sockets, an appropriate installation note for the foundation drawing is "Piles to be placed in bedrock. Suitability of bedrock to be confirmed by Geotechnical Engineer during construction of predrilled hole." An NSSP will be required for installation of piles in rock sockets. Suggested wording is provided in Appendix E.

Construction of the predrilled holes will require use of a steel liner advanced to the bedrock surface to support the sidewalls, minimize groundwater inflow, and enable machine-

cleaning of the socket base. Installation procedures that deal with potential instability due to the presence of a high groundwater table and cohesionless soil deposits must be employed.

The Contractor must be prepared to drive piles or drill through very dense till deposits containing cobbles, boulders and shale slabs. Further, drilling equipment that can penetrate shale bedrock with hard limestone layers must be employed to prepare rock sockets. The Contract Documents should contain a NSSP alerting the Bidders to these conditions. Suggested texts for NSSP's are included in Appendix E.

Sockets and auger holes containing piles should be backfilled with concrete within 8 hours of excavation to minimize softening of the till and shale bedrock by groundwater. Suggested wording for an NSSP in this regard is provided in Appendix E.

8.4 Recommended Foundation

From a geotechnical perspective and based on the subsurface conditions, caissons socketed into bedrock are the preferred foundation type for the new pier foundations. Spread footings founded on the native sand/gravel or steel H-piles socketed into the shale are also considered feasible.

8.5 Frost Cover

The design depth of frost penetration at this site is 1.8 m.

Frost protection should be provided for footings and pile caps, if used, and should consist of a minimum of 1.8 m of soil cover.

9 EXISTING ABUTMENTS

9.1 Axial Capacity of Existing H-Piles

Replacement of the existing bridge superstructure and raising of road grades on Lees Avenue will result in increased loads on the abutment foundations. The additional loading must be taken into consideration in the assessment of the existing abutment foundations.

Based on archive drawings, the existing abutments are founded on 12 BP 53 steel H-piles driven to bedrock. The anticipated length was 40 and 50 ft (12.2 and 15.2 m) at the south and north abutments, respectively. The allowable load was 47 tons. The equivalent pile is an HP 310 x 79 steel pile with an SLS geotechnical resistance of 420 kN.

Site records documenting pile driving operations and the as-driven pile length are not available to confirm whether the piles were driven to bedrock. Given the dense to very dense soil conditions at this site, driving the piles to bedrock would be difficult and it is considered likely that the piles encountered effective refusal in the very dense soil above

the bedrock. It is recommended that assessment of the geotechnical resistance of the existing piles be based on a 10 m long pile bearing in the very dense soil.

On this basis, the following resistance values are recommended for evaluation of the existing piles:

Factored Geotechnical Resistance at ULS	1,400 kN
Geotechnical Resistance at SLS	1,200 kN

The values above represent the available geotechnical resistance. Any reduction in pile resistance due to structural deterioration, for example from corrosion, must be assessed by the structural designers.

Uplift resistance provided by the existing piles is limited to frictional resistance along the pile shaft within the sand and till layers. The factored uplift resistance at ULS, including a resistance factor of 0.3, is computed to be 175 kN per pile.

9.2 Existing Lateral Capacity

The lateral resistance of the existing piles may be calculated using the procedure previously outlined for caisson foundations (Section 8.1.3).

The recommended parameters to be used to analyse the interaction between the pile and surrounding soil at the abutments are presented in Table 9.1

Table 9.1 – Parameters for Lateral Pile Resistance

Structure Element	Top Elevation	Bottom Elevation	n_h (kN/m ³)	K_p	Unit Weight (kN/m ³)	Soil Conditions
South abutment	59.5	59.0	8,000	3.0	21	Silty sand, dense to very dense
	59.0	47.4/ Bedrock	10,000	3.3	11*	Sand, silty sand, silt till, silty sand till; dense to very dense.
North abutment	63.5	60.1	4,000	3.0	21	Sand fill, compact to dense
	60.1	49.3/ Bedrock	10,000	3.3	11*	Silty sand till, sand and gravel, sand; dense to very dense.

* Buoyant unit weight below the water table.

10 SOIL AND ROCK ANCHORS

If required for seismic upgrade, use of anchors or micropiles is considered feasible to provide additional uplift resistance to the foundations. The anchorage should be developed within very dense sand/gravel or till, or within the underlying shale bedrock.

The length of the unbonded zone below the underside of the footing/pile cap should be at least 3.0 m for a steel bar anchor and 4.5 m for a steel strand anchor. The minimum bond length should be 3.0 m for a rock anchor and 4.5 m for a soil anchor. Soil anchors should have a minimum 4.5 m of soil cover above the centre of the bonded zone.

The factored soil-grout bond strengths at ULS recommended for design of the anchors within soil and shale bedrock are as follows. These values include a geotechnical resistance value of 0.4:

Very dense silty sand till or sand/gravel	75 kPa
Shale bedrock	200 kPa

Each production anchor/ micropile should be proof tested as per Special Provision 999S26 to confirm that they can resist the required uplift load. Anchors providing resistance only during seismic events are considered to be passive anchors. Accordingly, it is recommended that each anchor be proof tested to 100% of the factored ULS design load, then unloaded to a lock-off value of 20% of the design value.

The rock anchors/ micropiles should be provided with double corrosion protection.

The drilling of holes for installation of anchors will encounter fill material and very dense sand, sand and gravel, and silty sand till. The sand/gravel and till may contain cobbles and boulders. The Contractor's drilling equipment must be able to penetrate very dense material and dislodge, remove or penetrate any cobbles or boulders encountered in the till. Temporary casing may be required to support the side walls of the holes drilled within the cohesionless deposits for installation of the anchors.

Rock anchors will be extended into the shale bedrock containing hard limestone interbeds. The Contractor's drilling equipment must be able to penetrate the sound bedrock and hard interbeds to achieve the design bond length.

11 WALL BACKFILL AND LATERAL EARTH PRESSURES

Backfill to existing abutment walls where required, should be in accordance with OPSS 902, and placed to the extents shown in OPSD 3101.150. Any backfill to the abutments should consist of Granular A or Granular B Type II material meeting the requirements of Special Provision 110S13. Compaction equipment to be used adjacent to retaining structures should be restricted in accordance with OPSS 501.

For the assessment of the existing abutments and wing walls, earth pressures acting on the structure may be assumed to be triangular and to be governed by the characteristics of the abutment backfill.

For a fully drained condition, the pressures should be computed in accordance with the CHBDC but generally are given by the expression:

$$P_h = K(\gamma h + q)$$

Where:

P_h = horizontal pressure on the wall at depth h (kPa)

K = earth pressure coefficient (see Table 11.1)

γ = unit weight of retained soil (see Table 11.1)

h = depth below top of fill where pressure is computed (m)

q = value of any surcharge (kPa)

Earth pressure coefficients for backfill to the abutment wall are dependent on the material used as backfill. Typical values are shown in Table 11.1.

In conventional design, the use of a material with a high friction angle and low active pressure coefficient (e.g. Granular A, Granular B Type II) might be preferred as it results in lower earth pressures acting on the wall.

Table 11.1 – Earth Pressure Coefficients (K)

Condition	Earth Pressure Coefficient (K)			
	OPSS Granular A or OPSS Granular B Type II		OPSS Granular B Type I or Existing Sand Fill	
	$\phi = 35^\circ, \gamma = 22.8 \text{ kN/m}^3$		$\phi = 32^\circ, \gamma = 21.2 \text{ kN/m}^3$	
	Horizontal Surface Behind Wall	Sloping * Backfill (2H:1V)	Horizontal Surface Behind Wall	Sloping * Backfill (2H:1V)
Active (Unrestrained Wall)	0.27	0.38	0.31	0.46
At rest (Restrained Wall)	0.43	-	0.47	-
Passive (Movement Towards Soil Mass)	3.7	2.1	3.3	1.7
* For wing walls				

The factors in Table 11.1 are “ultimate” values and require certain movements for the respective conditions to be mobilized. The values to use in design can be estimated from Figure C6.16 in the Commentary to the Canadian Highway Bridge Design Code.

In accordance with Clause 6.9.3 of the CHBDC, a compaction surcharge should be added. The magnitude should be 12 kPa at the top of fill and decreasing to 0 kPa at a depth of 2.0 m for Granular B Type I or 1.7 m for Granular A or Granular B Type II.

12 APPROACH EMBANKMENTS

12.1 Settlement and Slope Stability

The existing approach embankment is approximately 8.0 m high at the north end of the structure and approximately 4.5 m high at the south end. As part of the rehabilitation works for the Lees Avenue underpass, the road profile along Lees Avenue will be raised between 1.0 to 1.5 m.

The embankment foundation soils generally consist of dense to very dense, cohesionless sand, sand and gravel and silty sand till. Raising the approach embankments by 1.0 to 1.5 m is expected to result in immediate settlement of less than 15 mm in the foundation soils. These settlements are expected to be immediate and occur essentially as the fill is constructed.

A stability analysis was carried out for a maximum 9 m high embankment with a side slope of 2H:1V. The slope stability program GSLOPE developed by Mitre Software Inc. was used for the analysis with the option for Bishop's simplified method. The embankment was assessed under static and seismic loading, assuming an acceleration of 0.16g (conservative, as this is the peak acceleration at the site).

For this method, a factor of safety (FoS) of 1.3 is considered acceptable for static assessment of both short term and long term conditions. A FoS greater than 1.0 is suitable for seismic assessment.

The input parameters and soil model used in the analyses are shown in Figures F1 and F2, Appendix F, along with the results. The results indicate a FoS of approximately 1.5 for static conditions and marginally above 1.0 for seismic conditions. The stability of the embankment is therefore considered acceptable.

Raising the approach fill will increase the lateral earth pressures acting on the abutment. The effect of additional loading on the existing abutment and existing pile foundations must be considered in the design.

12.2 Embankment Construction

Embankment construction and widening must be in accordance with SP 206S03. The existing embankment must be benched in accordance with OPSD 208.010 prior to placement of the new fill.

The side slopes of the embankment widening should be inclined no steeper than 2H:1V. In general, mid-height berms comprising 2 m wide benches should be incorporated along the length of embankments exceeding 8 m in height. The bench should maintain a 2% slope to shed surface run-off. In areas where space is not available for inclined slopes and benching, retaining walls or RSS slopes may be required.

Disturbed or regraded earth slopes must be provided with erosion protection in accordance with OPSS 804.

12.3 Embankment Slope on East Side of North Approach

The embankment slope on the east side of the north approach presently has a maximum height of 8 m and is inclined at about 1.7H:1V adjacent to the abutment. At this location, widening of the embankment base, providing a 2H:1V slope inclination and incorporating mid-height berms to accommodate a grade raise of approximately 1.0 m is not possible within the available property. Maintaining the existing slope inclination and placement of additional fill at the top of the slope to raise the grade is planned.

The existing embankment slope is presently grass and brush covered and appears to be performing satisfactorily.

The stability analyses carried out for the continuous 9 m high slope were repeated for the existing slope inclination of 1.7H:1V. The results are shown in Figures F3 and F4, Appendix F. The computed FoS of 1.36 for static conditions is considered acceptable and the result of approximately 1.0 for seismic conditions is essentially unchanged from the previous analysis at 2H:1V.

Based in this analysis, the proposed grade raise is expected to have negligible impact on the stability of the embankment, and maintaining the existing slope inclination without the berm requirement is considered acceptable from a foundations perspective.

13 EXCAVATION AND GROUNDWATER CONTROL

Excavation associated with modification of the abutments, removal of the existing pier footings, and construction of the new pier footing/pile cap is expected to be limited to depths of approximately 3 m. The excavation will extend through the existing pavement materials (including concrete) and sand fill.

All excavations must be carried out in accordance with the Occupational Health and Safety Act (OHSA). For the purposes of the OHSA, the existing fill within the probable depth of excavation at this site may be classed as Type 3 soils above the water table. It is anticipated that foundation construction on this project will require a properly designed shored excavation.

The excavation and backfilling for foundations must be carried out in accordance with OPSS 902. Removal of the existing north pier to top of footing and the existing south pier and footing must be carried out in accordance with OPSS 510.

Groundwater was measured at depths of 4.7 to 8.3 m (Elev. 59.0 to 61.3 m) at the abutments and 5.0 to 5.1 m (Elev. 55.0 to 55.1 m) at the pier. Based on these water level measurements, excavation below the water level is not anticipated. Perched water may be encountered locally within the fill however. The Contractor should make provision to pump from sumps to remove any seepage, perched water or surface water collecting in an excavation.

Use of a steel liner is recommended to advance caissons or pile sockets to support the sidewalls, minimize groundwater inflow, and enable machine-cleaning of the socket base. Installation procedures that deal with potential instability due to the presence of a high groundwater table and cohesionless soil deposits must be employed.

The site is located in the vicinity of a former coal gasification plant where coal tar impacts have been documented. Potential hydrocarbon contaminants were encountered in Boreholes LE-02 and LA-02 drilled during the current investigation. Site specific handling and disposal procedures may be required for any material excavated from the site. These procedures should be established in the environmental investigation report prepared by others (Ecoplans Limited.) and brought to the attention of the Contractor in the Contract Documents.

14 ROADWAY PROTECTION

Roadway protection should be supplied in accordance with OPSS 539 and designed for Performance Level 2. The protection systems should be designed by a licensed Professional Engineer experienced in design of shoring with consideration of adjacent traffic loads and any sloping retained surfaces.

It is the Contractor's responsibility to select a suitable roadway protection system based on his evaluation of the data presented in the Foundation Investigation report. However, it is noted that installation of driven H-piles or sheet piles in the very dense fill, sand/gravel and till at the site may be difficult. Installation of piles for a soldier pile and lagging system may require pre-augered holes. A braced excavation or system of rakers may be considered.

Parameters for design of the roadway protection systems are provided in Sections 11 and 13. It is the responsibility of the Contractor to design the roadway protection system and any dewatering system required.

15 SEISMIC CONSIDERATIONS

The following seismic parameters should be used for design/ assessment of the existing underpass:

- | | |
|-------------------------------------|-------|
| • Velocity Related Seismic Zone | 2 |
| • Zonal Velocity Ratio | 0.1 |
| • Acceleration Related Seismic Zone | 4 |
| • Zonal Acceleration Ratio | 0.2 |
| • Peak Horizontal Acceleration | 0.16g |

The soil profile type at this site has been classified as Type I. Therefore, according to Table 4.4.6.1 of the CHBDC, a Site Coefficient "S" (ground motion amplification factor) of 1.0 should be used in seismic design.

In accordance with Clause 4.6.4 of the CHBDC, retaining structures should be designed using earth pressure coefficients that incorporate the effects of earthquake loading. The seismic component of the earth pressure distribution is additional to the static earth pressure distribution and may be taken as an inverted triangle with the maximum pressure at the top of the wall and the minimum pressure at the toe. The total (static plus seismic) pressure distribution for earthquake loading is therefore as follows:

$$p_{he} = K (\gamma h + q) + \Delta K_E \gamma (H - h)$$

where:

p_{he}	=	horizontal pressure on the wall at depth h (kPa)
K	=	earth pressure coefficient (see Table 11.1)
ΔK_E	=	seismic earth pressure coefficient (see Table 15.1)
γ	=	unit weight of retained soil (see Table 15.1)
h	=	depth below top of fill where pressure is computed (m)
H	=	height of wall (m)
q	=	value of any surcharge (kPa)

The seismic earth pressure parameters (ΔK_E) recommended for determining the seismic component are presented in Table 15.1.

Table 15.1 – Earth Pressure Coefficients for Earthquake Loading

Condition	Seismic Earth Pressure Coefficient (ΔK_E)			
	OPSS Granular A or OPSS Granular B Type II $\phi = 35^\circ, \gamma = 22.8 \text{ kN/m}^3$		OPSS Granular B Type I or Existing Sand Fill $\phi = 32^\circ, \gamma = 21.2 \text{ kN/m}^3$	
	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)
Active (ΔK_{AE})*	0.07	0.22	0.07	0.23
At Rest (ΔK_{OE})**	0.21	-	0.21	-

* After Mononobe and Okabe, passive case assumes a horizontal surface in front of the wall.

** After Woods

The foundation soils at the pier and abutments are not in danger of liquefaction under earthquake loading.

16 STAGING AREA

We understand that the maximum allowable differential settlement permitted for construction of the new structure within the staging area will be 12 mm between substructures and 6 mm across an individual substructure. To achieve these stringent settlement requirements, it is recommended that the temporary supports be constructed on caissons or driven piles extended to shale bedrock.

The use of deep foundations is preferred considering the following:

- Excavation depths of up to about 6.0 m would be required to penetrate existing fill and compact native soils to found spread footings on very dense native soil capable of achieving the settlement requirements.
- Construction of an engineered fill pad would require similar excavation depths as for spread footings on native soil, as well as disposal of potential environmentally impacted soils and importation of select granular material.
- The use of deep foundations, and in particular driven piles, reduces the need for excavation and disposal of potentially impacted site soils, and achieves the differential settlement requirements.

Steel H-piles driven to refusal within the shale bedrock should be designed using a factored axial geotechnical resistance of 2,000 kN at ULS. The SLS resistance will not govern design of piles founded in bedrock. The depth to bedrock encountered in the boreholes ranged from 9.1 to 11.3 m (Elev. 50.1 to 52.0 m). Pre-augering to the bedrock surface is recommended to avoid termination of the piles within the very dense soils above the bedrock.

Pile installation should be in accordance with OPSS 903. The tips of all piles should be fitted with H-Section driving shoes as per OPSD 3000.100. The appropriate pile driving note is “Piles to be driven to bedrock”.

If employed, caissons should be socketed at least 3 m into the shale bedrock to found in sound shale below the upper weathered bedrock surface. The recommended factored axial geotechnical resistances at ULS for typical caisson diameters socketed 3 m into shale bedrock are summarised in Table 16.1. The SLS condition will not govern for caissons founded in bedrock.

Table 16.1- Axial Geotechnical Resistance of Caissons

Socket Length in Shale (m)	Caisson Diameter (m)	Factored Axial Resistance at ULS (kN)
3	0.9	5,000
	1.2	8,000
	1.5	10,000

Additional comments and recommendations regarding pile and caisson design and installation are presented in Section 8.

17 CONSTRUCTION CONCERNS

Potential construction concerns include, but are not necessarily limited to the issues discussed below.

- The very dense sand/gravel and till at the site may contain cobbles, boulders and shale slabs. In addition, the shale bedrock underlying the till contains hard limestone layers. Equipment selected to install caissons and pile sockets must be capable of penetrating these materials.
- Care must be taken to avoid disturbance to the founding subgrade. Exposed subgrade soils and caisson/pile sockets should be inspected, approved and protected/concreted within 8 hours to prevent softening of the shale;
- Use of a steel liner is recommended to advance caissons and pile sockets to the bedrock surface to support the sidewalls, minimize groundwater inflow, and enable machine-cleaning of the socket base.
- Excavated on-site soils may require special handling and disposal procedures to deal with potential environmentally impacted soils.

18 CLOSURE

Engineering analysis and preparation of the report were carried out by Ms. M. Cheong, M.Phil. and Mr. M.R. Anderson, P.Eng. The report was reviewed by Dr. P.K. Chatterji, P.Eng., a Designated Principal Contact for MTO Foundations Projects.

Thurber Engineering Ltd.

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Appendix A

Record of Borehole Sheets

SYMBOLS, ABBREVIATIONS AND TERMS USED ON RECORDS OF BOREHOLES

1. TEXTURAL CLASSIFICATION OF SOILS

CLASSIFICATION	PARTICLE SIZE	VISUAL IDENTIFICATION
Boulders	Greater than 200mm	same
Cobbles	75 to 200mm	same
Gravel	4.75 to 75mm	5 to 75mm
Sand	0.075 to 4.75mm	Not visible particles to 5mm
Silt	0.002 to 0.075mm	Non-plastic particles, not visible to the naked eye
Clay	Less than 0.002mm	Plastic particles, not visible to the naked eye

2. COARSE GRAIN SOIL DESCRIPTION (50% greater than 0.075mm)

TERMINOLOGY	PROPORTION
Trace or Occasional	Less than 10%
Some	10 to 20%
Adjective (e.g. silty or sandy)	20 to 35%
And (e.g. sand and gravel)	35 to 50%

3. TERMS DESCRIBING CONSISTENCY (COHESIVE SOILS ONLY)

DESCRIPTIVE TERM	UNDRAINED SHEAR STRENGTH (kPa)	APPROXIMATE SPT ⁽¹⁾ 'N' VALUE
Very Soft	12 or less	Less than 2
Soft	12 to 25	2 to 4
Firm	25 to 50	4 to 8
Stiff	50 to 100	8 to 15
Very Stiff	100 to 200	15 to 30
Hard	Greater than 200	Greater than 30

NOTE: Hierarchy of Soil Strength Prediction

- 1) Laboratory Triaxial Testing
- 2) Field Insitu Vane Testing
- 3) Laboratory Vane Testing
- 4) SPT value
- 5) Pocket Penetrometer

4. TERMS DESCRIBING DENSITY (COHESIONLESS SOILS ONLY)

DESCRIPTIVE TERM	SPT "N" VALUE
Very Loose	Less than 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very Dense	Greater than 50

5. LEGEND FOR RECORDS OF BOREHOLES

SYMBOLS AND ABBREVIATIONS FOR SAMPLE TYPE	SS Split Spoon Sample	WS Wash Sample	AS Auger (Grab) Sample
	TW Thin Wall Shelby Tube Sample		TP Thin Wall Piston Sample
	PH Sampler Advanced by Hydraulic Pressure		PM Sampler Advanced by Manual Pressure
	WH Sampler Advanced by Self Static Weight		RC Rock Core
			SC Soil Core

$$\text{Sensitivity} = \frac{\text{Undisturbed Shear Strength}}{\text{Remoulded Shear Strength}}$$



Water Level

C_{pen}






Shear Strength Determination by Pocket Penetrometer

- (1) SPT 'N' Value Standard Penetration Test 'N' Value – refers to the number of blows from a 63.5kg hammer free falling a height of 0.76m to advance a standard 50 mm outside diameter split spoon sampler for 0.3 m depth into undisturbed ground.
- (2) DCPT Dynamic Cone Penetration Test – Continuous penetration of a 50 mm outside diameter, 60° conical steel point attached to "A" size rods driven by a 63.5 kg hammer free falling a height of 0.76 m. The resistance to cone penetration is the number of hammer blows required for each 0.3 m advance of the conical point into undisturbed ground.

UNIFIED SOILS CLASSIFICATION

MAJOR DIVISIONS		GROUP SYMBOL	TYPICAL DESCRIPTION
COARSE GRAINED SOILS	GRAVEL AND GRAVELLY SOILS	GW	Well-graded gravels or gravel-sand mixtures, little or no fines.
		GP	Poorly-graded gravels or gravel-sand mixtures, little or no fines.
		GM	Silty gravels, gravel-sand-silt mixtures.
		GC	Clayey gravels, gravel-sand-clay mixtures.
	SAND AND SANDY SOILS	SW	Well-graded sands or gravelly sands, little or no fines.
		SP	Poorly-graded sands or gravelly sands, little or no fines.
		SM	Silty sands, sand-silt mixtures.
		SC	Clayey sands, sand-clay mixtures.
FINE GRAINED SOILS	SILTS AND CLAYS $W_L < 50\%$	ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity.
		CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays. ($W_L < 30\%$).
		CI	Inorganic clays of medium plasticity, silty clays. ($30\% < W_L < 50\%$).
		OL	Organic silts and organic silty-clays of low plasticity.
	SILTS AND CLAYS $W_L > 50\%$	MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts.
		CH	Inorganic clays of high plasticity, fat clays.
		OH	Organic clays of medium to high plasticity, organic silts.
HIGHLY ORGANIC SOILS		Pt	Peat and other highly organic soils.
CLAY SHALE			
SANDSTONE			
SILTSTONE			
CLAYSTONE			
COAL			

EXPLANATION OF ROCK LOGGING TERMS

ROCK WEATHERING CLASSIFICATION		SYMBOLS	
Fresh (FR)	No visible signs of weathering.		
Fresh Jointed (FJ)	Weathering limited to the surface of major discontinuities.		CLAYSTONE
Slightly Weathered (SW)	Penetrative weathering developed on open discontinuity surfaces, but only slight weathering of rock material.		SILTSTONE
Moderately Weathered (MW)	Weathering extends throughout the rock mass, but the rock material is not friable.		SANDSTONE
Highly Weathered (HW)	Weathering extends throughout the rock mass and the rock is partly friable.		COAL
Completely Weathered (CW)	Rock is wholly decomposed and in a friable condition, but the rock texture and structure are preserved.		Bedrock (general)

DISCONTINUITY SPACING		STRENGTH CLASSIFICATION			
Bedding	Bedding Plane Spacing	Rock Strength	Approximate Uniaxial Compressive Strength		Field Estimation of Hardness*
			(MPa)	(psi)	
Very thickly bedded	Greater than 2m	Extremely Strong	Greater than 250	Greater than 36,000	Specimen can only be chipped with a geological hammer
Thickly bedded	0.6 to 2m				
Medium bedded	0.2 to 0.6m	Very Strong	100-250	15,000 to 36,000	Requires many blows of geological hammer to break
Thinly bedded	60mm to 0.2m				
Very thinly bedded	20 to 60mm	Strong	50-100	7,500 to 15,000	Requires more than one blow of geological hammer to break
Laminated	6 to 20mm				
Thinly Laminated	Less than 6mm	Medium Strong	25.0 to 50.0	3,500 to 7,500	Breaks under single blow of geological hammer.
		Weak	5.0 to 25.0	750 to 3,500	Can be peeled by a pocket knife with difficulty
		Very Weak	1.0 to 5.0	150 to 750	Can be peeled by a pocket knife, crumbles under firm blows of geological pick.
		Extremely Weak (Rock)	0.25 to 1.0	35 to 150	Indented by thumbnail

TERMS	
Total Core Recovery: (TCR)	Core recovered as a percentage of total core run length.
Solid Core Recovery: (SCR)	Percent Ratio of solid core of full cylindrical shape recovered. Expressed with respect to the total length of core run.
Rock Quality Designation: (RQD)	Total length of sound core recovered in pieces 0.1m in length or larger as a percentage of total core run length.
Uniaxial Compressive Strength (UCS)	Axial stress required to break the specimen
Fracture Index: (FI)	Frequency of natural fractures per 0.3m of core run.

RECORD OF BOREHOLE No LE-01

1 OF 2

METRIC

W.P. 4091-07-00 LOCATION N 5 031 086.6 E 369 814.6 Lees Avenue - South Approach ORIGINATED BY DA
HWY 417 BOREHOLE TYPE Casing - CME 55 COMPILED BY AN
DATUM Geodetic DATE 2011.08.28 - 2011.08.30 CHECKED BY LRB

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			20 40 60 80 100	PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	
63.0												
0.0	ASPHALT: (75mm)		1	CORE			63					
0.1												
0.2	CONCRETE: (225mm)											
0.3	SAND, coarse grained, trace to some gravel Very Dense Light Brown Moist (FILL)		2	SS	60		62					
			3	SS	65							
60.8							61					
2.2	Silty SAND, trace gravel Dense Brown Moist to Wet (FILL)		4	SS	38		60					
	No recovery Occasional cobble		5	SS	65/ 0.150							
			6	SS	58		59					
58.5												
4.5	Gravelly SAND, some silt Very Dense Light Brown		7	SS	75		58					
			8	SS	90/ 0.280							
56.9			9	SS	50/ 0.075		57					
6.1	GRAVEL, some sand Very Dense											
	No recovery		10	SS	50/ 0.050		56					
55.3			11	SS	50/ 0.127		55					
7.8	Silty SAND, fine grained Very Dense Brown Wet (TILL)						54					
53.6			12	SS	70/ 0.150							
9.4	END OF BOREHOLE AT 9.4m. BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG, THEN											

Continued Next Page

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

RECORD OF BOREHOLE No LE-01

2 OF 2

METRIC

W.P. 4091-07-00 LOCATION N 5 031 086.6 E 369 814.6 Lees Avenue - South Approach ORIGINATED BY DA
HWY 417 BOREHOLE TYPE Casing - CME 55 COMPILED BY AN
DATUM Geodetic DATE 2011.08.28 - 2011.08.30 CHECKED BY LRB

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		
	Continued From Previous Page ASPHALT TO SURFACE.							20 40 60 80 100				20 40 60		

RECORD OF BOREHOLE No LE-02

1 OF 3

METRIC

W.P. 4091-07-00 LOCATION N 5 031 100.5 E 369 819.9 Lees Avenue - South Approach ORIGINATED BY DA
HWY 417 BOREHOLE TYPE Casing - CME 55 COMPILED BY AN
DATUM Geodetic DATE 2011.08.28 - 2011.08.30 CHECKED BY LRB

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 (%)		
63.7							20	40	60	80	100	20	40	60			
0.0	ASPHALT: (100mm)		1	AS													
0.1	CONCRETE: (650mm)		2	CORE													
63.0																	
0.8	SAND, coarse grained, some silt, trace to some gravel Very Dense to Compact Brown Moist to Wet (FILL)		3	SS	39												
	Fine to medium grained, trace silt, trace gravel		4	SS	60							○					
			5	SS	55							○					
			6	SS	27							○					
59.9																	
3.8	Silty SAND Very Dense to Dense Moist Black (possible coal tar) between 4.0m to 4.5m		7	SS	50/ 0.100												
			8	SS	37							○					
58.4																	
5.3	SAND, some gravel, trace silt, trace clay Compact to Very Dense Dark Brown to Grey Wet Faint hydrocarbon odour No odour		9	SS	23							○					
			10	SS	41							○					
			11	SS	83												
			12	SS	80							○					
54.8																	
9.1	SILT Very Dense Dark Brown Wet (TILL) Faint hydrocarbon odour		13	SS	110/ 0.275												

Continued Next Page

+³, ×³: Numbers refer to
Sensitivity 15-20 (5) STRAIN AT FAILURE

RECORD OF BOREHOLE No LE-02

2 OF 3

METRIC

W.P. 4091-07-00 LOCATION N 5 031 100.5 E 369 819.9 Lees Avenue - South Approach ORIGINATED BY DA
HWY 417 BOREHOLE TYPE Casing - CME 55 COMPILED BY AN
DATUM Geodetic DATE 2011.08.28 - 2011.08.30 CHECKED BY LRB

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			20	40	60	80	100		
	Continued From Previous Page													
53.1			14	SS	55/									
					0.125									
10.7	Silty SAND (probable), fine grained, trace silt, occasional cobbles and boulders Dark Brown Very Dense (Probable TILL)		15	SS	52/		53							
					0.100									
	Cored from 12.5m to 16.2m		16	SS	50/		52							
					0.080									
			1	RUN			51							
			2	RUN			50							
			3	RUN			49							
			4	RUN			48							
47.4	clayey silt pocket (150mm) at 16.0m		20	SS	50/								FI	
16.3	SHALE, slightly weathered, thinly bedded, dark grey, limestone interbeds through out		4	RUN	0.025		47						1	
			5	RUN			46						2	RUN #4
													7	TCR=100%
													0	SCR=82%
													4	RQD=82%
													4	UCS=14.7MPa
													3	
44.3							45						7	RUN #5
19.4	END OF BOREHOLE AT 19.4m. Piezometer installation consists of 25mm diameter Schedule 40 PVC pipe with a 3.0m slotted screen.												0	TCR=90%
														SCR=63%
														RQD=55%
														UCS=16.6MPa

Continued Next Page

+³, x³: Numbers refer to
Sensitivity

20
15 5
10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No LE-03

1 OF 2

METRIC

W.P. 4091-07-00 LOCATION N 5 031 097.0 E 369 827.6 Lees Avenue - 5m South of Expansion Joint ORIGINATED BY DA
 HWY 417 BOREHOLE TYPE Casing - CME 55 COMPILED BY AN
 DATUM Geodetic DATE 2011.08.30 - 2011.08.31 CHECKED BY LRB

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		× LAB VANE		
64.2							20	40	60	80	100	20	40	60						
0.0	ASPHALT: (100mm)						64													
0.1	CONCRETE: (650mm)																			
63.4																				
0.8	SAND and GRAVEL, trace silt		1	SS	50/															
63.1	Very Dense				0.100															
1.1	Brown						63													
	Wet (FILL)																			
	SAND, fine grained, trace gravel, trace silt		2	SS	103															
	Very Dense						62													
	Brown																			
	(FILL)																			
			3	SS	98															
							61													
			4	SS	61															
			5	SS	78		60													
59.6																				
4.6	Silty SAND, trace to some gravel		6	SS	56		59													
	Very Dense to Dense																			
	Brown																			
			7	SS	81															
							58													
			8	SS	40															
			9	SS	112		57													
			10	SS	86		56													
55.1							55													
9.1	Silty SAND		11	SS	70															
	Very Dense																			
	Dark Grey																			
	(TILL)																			

Continued Next Page

+³, x³: Numbers refer to
Sensitivity

20
15
10
(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No LE-03

2 OF 2

METRIC

W.P. 4091-07-00 LOCATION N 5 031 097.0 E 369 827.6 Lees Avenue - 5m South of Expansion Joint ORIGINATED BY DA
 HWY 417 BOREHOLE TYPE Casing - CME 55 COMPILED BY AN
 DATUM Geodetic DATE 2011.08.30 - 2011.08.31 CHECKED BY LRB

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	×	LAB VANE												
	Continued From Previous Page						20	40	60	80	100	20	40	60								
51.8			12	SS	50/ 0.100																	
52			13	SS	50/ 0.100																	
12.4	END OF BOREHOLE AT 12.4m. BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG, THEN ASPHALT TO SURFACE.				0.100																	

RECORD OF BOREHOLE No LE-04

1 OF 3

METRIC

W.P. 4091-07-00 LOCATION N 5 031 190.3 E 369 832.7 Lees Avenue - North Abutment ORIGINATED BY RK
HWY 417 BOREHOLE TYPE NW/INQ Casing - CME 72 COMPILED BY AN
DATUM Geodetic DATE 2011.09.08 - 2011.09.08 CHECKED BY LRB

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											
								WATER CONTENT (%)											
68.6							20	40	60	80	100	PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	GR	SA	SI	CL	
0.0	ASPHALT: (115mm)																		
0.1	CONCRETE: (315mm)																		
68.2																			
0.4	SAND, trace to some gravel, some silt Compact to Dense Brown Moist to Wet (FILL)																		
			1	SS	28		68												
			2	SS	35		67												
			3	SS	46		66												
			4	SS	33		65												
							64												
			5	SS	11		63												
							62												
	No recovery		6	SS	100/ 0.228		61												
							60												
	Silty Orange stains						59												
60.7			7	SS	27														
7.9	TOPSOIL, silt, some sand Black Moist																		
60.1																			
8.5	Silty SAND, some gravel Dense Brown Wet (TILL)																		
			8	SS	77/ 0.29														

ONTMT4S 1201A.GPJ 2/15/12

Continued Next Page

+³, X³: Numbers refer to
Sensitivity

20
15 5
10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No LE-04

2 OF 3

METRIC

W.P. 4091-07-00 LOCATION N 5 031 180.3 E 369 832.7 Lees Avenue - North Abutment ORIGINATED BY RK
 HWY 417 BOREHOLE TYPE NW/NQ Casing - CME 72 COMPILED BY AN
 DATUM Geodetic DATE 2011.09.08 - 2011.09.08 CHECKED BY LRB

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				
	Continued From Previous Page											
56.4			9	SS	30							10 58 28 4
12.2	SAND and GRAVEL Very Dense Brown Wet		10	SS	50/ 0.08							
	Cored from 13.7m to 19.4m Cobble (100mm)		11	SS	50/ 0.125							
53.4			12	SS	50/ 0.080							
15.2	SAND, some gravel to gravelly Very Dense Brown Moist											
51.9												
16.8	Probable Sandy SILT, with gravel Brown Wet (Probable TILL)											
49.3												
19.4	SHALE, fresh, thinly laminated, horizontally jointed, dark grey											

Continued Next Page

+³, X³: Numbers refer to
Sensitivity

20
15 5
10 (%) STRAIN AT FAILURE

ONTMT4S 1201A.GPJ 10/18/11

RECORD OF BOREHOLE No LE-04

3 OF 3

METRIC

W.P. 4091-07-00 LOCATION N 5 031 190.3 E 369 832.7 Lees Avenue - North Abutment ORIGINATED BY RK
HWY 417 BOREHOLE TYPE NWN/Q Casing - CME 72 COMPILED BY AN
DATUM Geodetic DATE 2011.09.08 - 2011.09.08 CHECKED BY LRB

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 (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			20	40	60	80	100	W _p	W	W _L		
	Continued From Previous Page		1	RUN		48									3	RUN #1 TCR=100% SCR=98% RQD=72% UCS=14.5MPa (Average)
			2	RUN		47									1	
46.2															1	
22.4	END OF BOREHOLE AT 22.4m. BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG AND CUTTINGS TO 1.5m, SAND TO 0.1m, THEN ASPHALT COLD PATCH TO SURFACE.															RUN #2 TCR=100% SCR=100% RQD=100% UCS=16.5MPa (Average)

+³, X³: Numbers refer to
Sensitivity

20
15 10 5
(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No LE-05

1 OF 2

METRIC

W.P. 4091-07-00 LOCATION N 5 031 192.8 E 369 823.7 Lees Avenue - 3m North of Expansion Joint ORIGINATED BY DA
HWY 417 BOREHOLE TYPE CME-55 COMPILED BY AN
DATUM Geodetic DATE 2011.08.31 - 2011.08.31 CHECKED BY LRB

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			20 40 60 80 100	20 40 60 80 100	20 40 60 80 100	20 40 60 80 100		
68.3	ASPHALT: (130mm)												
0.0	CONCRETE: (325mm)												
0.1													
67.8													
0.5	SAND, fine to coarse grained, some gravel, trace silt Very Dense Moist (FILL)		1	SS	68								
			2	SS	56								
66.0													
2.3	SAND and GRAVEL Brown Wet (FILL)		3	SS	53/ 0.125								
65.2													
3.0	Silty SAND, trace gravel Very Dense Brown Moist (FILL)		4	SS	54								
			5	SS	69								
62.2													
6.1	Silty SAND Dense Brown Moist (FILL)		6	SS	41								
61.6													
6.7	Silty SAND, trace gravel, trace clay Dense Brown to Dark Grey Wet (TILL)		7	SS	42								
			8	SS	53/ 0.100								
58.9													
9.4	END OF BOREHOLE AT 9.4m. Piezometer installation consists of 25mm diameter Schedule 40 PVC pipe with a 3.0m slotted screen.												

Continued Next Page

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

METRIC

ELEV DEPTH	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 (%)
	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	20 40 60 80 100	W _P W W _L	20 40 60		
	Continued From Previous Page											
							SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100		WATER CONTENT (%) 20 40 60			

WATER LEVEL READINGS:		
DATE	DEPTH (m)	ELEV. (m)
Sep.01/11	8.3	60.0
Oct.12/11	7.0	61.3

+³, ×³: Numbers refer to Sensitivity

RECORD OF BOREHOLE No LE-06

1 OF 2

METRIC

W.P. 4091-07-00 LOCATION N 5 031 210.6 E 369 826.5 Lees Avenue - North Approach ORIGINATED BY DA
 HWY 417 BOREHOLE TYPE CME-55 COMPILED BY AN
 DATUM Geodetic DATE 2011.08.31 - 2011.09.01 CHECKED BY LRB

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																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																
69.3							20	40	60	80	100																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																													

Continued Next Page

+³ . X³ : Numbers refer to
Sensitivity

20
15
10

(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No LE-06

2 OF 2

METRIC

W.P. 4091-07-00 LOCATION N 5 031 210.6 E 369 826.5 Lees Avenue - North Approach ORIGINATED BY DA
HWY 417 BOREHOLE TYPE CME-55 COMPILED BY AN
DATUM Geodetic DATE 2011.08.31 - 2011.09.01 CHECKED BY LRB

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			20	40	60	80	100		
	Continued From Previous Page							SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE						
								WATER CONTENT (%) W P W W L						
57.1	No recovery		9	SS	52/ 0.080		59							
12.2	Silty SAND, trace gravel Very Dense Dark Grey Wet (TILL)		10	SS	53		57							10 54 29 7
	Gravelly, with cobble						56							
	No recovery		1	RUN			55							
53.8			11	SS	97		54							
15.5	END OF BOREHOLE AT 15.5m UPON AUGER REFUSAL. BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG, THEN ASPHALT TO SURFACE.													

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

METRIC

[illegible]

+³, ×³: Numbers refer to Sensitivity

RECORD OF BOREHOLE No LE-07

2 OF 2

METRIC

W.P. 4091-07-00 LOCATION N 5 031 150.0 E 369 819.6 Lees Avenue ORIGINATED BY RK
HWY 417 BOREHOLE TYPE Hollow Stem Augers COMPILED BY AN
DATUM Geodetic DATE 2011.08.26 - 2011.08.26 CHECKED BY LRB

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			
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE											
	Continued From Previous Page																		
49.7							50												
10.4	Silty SAND, some gravel Very Dense Grey Wet (TILL)		9	SS	100/ 0.125		49					oo			8 64 25 3				
47.9																			
12.2	SHALE, slightly weathered to fresh, thinly laminated, horizontally jointed, grey		1	RUN			48							FI					
47.0														4	RUN #1 TCR=94% SCR=86% ROD=58%				
13.1	END OF BOREHOLE AT 13.1m. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen. WATER LEVEL READINGS: DATE DEPTH (m) ELEV. (m) Sep 20/11 5.0 55.1 Oct 12/11 5.1 55.0													2					

RECORD OF BOREHOLE No LE-07A

1 OF 2

METRIC

W.P. 4091-07-00 LOCATION N 5 031 150.0 E 369 818.6 Lees Avenue ORIGINATED BY GA
 HWY 417 BOREHOLE TYPE NQ Coring COMPILED BY AN
 DATUM Geodetic DATE 2011.11.15 - 2011.11.15 CHECKED BY LRB

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 (%)					
60.1														
0.0	Overburden sampled and described in Borehole LE-07.						60							
							59							
							58							
							57							
							56							
							55							
							54							
							53							
							52							
							51							

Continued Next Page

+ 3 . X 3 : Numbers refer to
Sensitivity

20
15 10 5
(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No LE-07A

2 OF 2

METRIC

W.P. 4091-07-00 LOCATION N 5 031 150.0 E 369 818.6 Lees Avenue ORIGINATED BY GA
 HWY 417 BOREHOLE TYPE NQ Coring COMPILED BY AN
 DATUM Geodetic DATE 2011.11.15 - 2011.11.15 CHECKED BY LRB

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 (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa			WATER CONTENT (%)				
	Continued From Previous Page							20 40 60 80 100							
								○ UNCONFINED + FIELD VANE							
								● QUICK TRIAXIAL × LAB VANE							
								20 40 60 80 100							

RECORD OF BOREHOLE No EBH-02

1 OF 2

METRIC

W.P. 4091-07-00 LOCATION N 5 031 148.1 E 369 844.2 Lees Avenue ORIGINATED BY GA
 HWY 417 BOREHOLE TYPE Hollow Stem Augers - CME-75 COMPILED BY AN
 DATUM Geodetic DATE 2011.06.29 - 2011.06.30 CHECKED BY LRB

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				UNIT WEIGHT kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
								20 40 60 80 100									
						○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB VANE				PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT w _p w w _L				WATER CONTENT (%) 20 40 60			
60.1																	
0.0	ASPHALT: (300mm)						60										
59.8																	
0.3	SAND, some gravel, trace silt and clay Dense to Compact Grey to Brown Dry (FILL)		1	SS	47		59									21 67 12 (SI+CL)	
			2	SS	42												
			3	SS	14		58										
57.8																	
2.3	Gravelly SAND, some silt and clay Dense to Very Dense Brown and Grey Dry		4	SS	35		57										
			5	SS	86/ 0.275												
			6	SS	86		56									25 52 23 (SI+CL)	
55.5																	
4.6	SAND, fine grained, trace silt to silty Dense to Very Dense Brown to Grey Moist to Wet		7	SS	92/ 0.275		55										
			8	SS	90/ 0.250		54									1 97 2 (SI+CL)	
			9	SS	84/ 0.250												
			10	SS	50/ 0.125		53										
			11	SS	38		52									0 68 32 (SI+CL)	
			12	SS	50/ 0.100		51										
	Cobbles at 9.8m																

Continued Next Page

+³, X³: Numbers refer to
Sensitivity

20
15
10
(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No EBH-02

2 OF 2

METRIC

W.P. 4091-07-00 LOCATION N 5 031 148.1 E 369 844.2 Lees Avenue ORIGINATED BY GA
 HWY 417 BOREHOLE TYPE Hollow Stem Augers - CME-75 COMPILED BY AN
 DATUM Geodetic DATE 2011.06.29 - 2011.06.30 CHECKED BY LRB

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 × LAB VANE					WATER CONTENT (%)						
						20 40 60 80 100					20 40 60						
	Continued From Previous Page						50										
49.2			13	SS	60/												
10.9	END OF BOREHOLE AT 10.9m UPON AUGER REFUSAL ON PROBABLE BEDROCK. BOREHOLE OPEN AND WATER LEVEL AT 5.4m UPON COMPLETION. BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG FROM 10.9m TO 0.4m, CONCRETE FROM 0.4m TO 0.3m, THEN ASPHALT PATCH TO SURFACE.				0.025												

+ ³ × ³ : Numbers refer to
Sensitivity

20
15
10
(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No LA-02

1 OF 2

METRIC

W.P. 4091-07-00 LOCATION N 5 031 106.9 E 369 842.3 Lees Avenue ORIGINATED BY GA
 HWY 417 BOREHOLE TYPE Hollow Stem Augers/NQ Coring - CME 75 COMPILED BY AN
 DATUM Geodetic DATE 2011.06.27 - 2011.06.28 CHECKED BY LRB

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			20 40 60 80 100	20 40 60 80 100	20 40 60 80 100	20 40 60 80 100		
61.1													
0.0	TOPSOIL: (100mm)												
0.1	SAND , trace silt, trace gravel, occasional rootlets Compact to Very Dense Brown to Black Dry (FILL) Black with hydrocarbon odour (possible coal tar) at 0.8m		1	SS	11		61						
			2	SS	61		60						
59.6													
1.5	Silty SAND , some gravel, trace clay Dense Brown to Grey Dry (TILL)		3	SS	38		59						
			4	SS	31		58						11 54 29 6
			5	SS	44		57						
			6	SS	48		56						13 76 11 (SI+CL)
56.5													
4.6	SAND , trace silt, trace to some gravel Very Dense Grey Dry		7	SS	86		55						
			8	SS	50/ 0.100		54						1 92 7 (SI+CL)
			9	SS	50/ 0.100		53						
			10	SS	50/ 0.125		52						
	Becomes wet		11	SS	98/ 0.250								
			12	SS	50/ 0.100								
			13	SS	50/ 0.100								
52.0													
9.1	Probable Silty SAND Very Dense Grey Wet (Probable TILL)		14	SS	50/ 0.100								

Continued Next Page

+³ × 3³ Numbers refer to
Sensitivity 20
15 10 5 (%) STRAIN AT FAILURE

ONTMT4S 1201A GPJ 2/15/12

METRIC

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	20	40	60	80	100		
Continued From Previous Page														
48.9	Probable Silty SAND Very Dense Grey Wet (Probable TILL)		16	SS	50/	0.025								
12.2	No recovery Probable cobbles and boulders from 10.7m		16	SS	50/	0.00								
48.9	SHALE, fresh, thinly bedded, occasional limestone interbeds, grey		1	RUN										
12.2	Horizontal joints at 12.8m, 13.7m, 13.8m Clay seam at 13.3m Limestone interbeds (between 25mm to 100mm thick) at 12.8m, 12.9m, 13.2m, 13.4m and 13.5m 50mm thick highly broken zone at 13.6m and 14.0m Bituminous seams		2	RUN										
45.3	END OF BOREHOLE AT 15.8m. BOREHOLE OPEN TO 15.8m AND WATER LEVEL AT 3.9m UPON COMPLETION.													
15.8														

ONTMT4S 1201A.GPJ 2/15/12

+³, ×³: Numbers refer to Sensitivity

RECORD OF BOREHOLE No LA-09

1 OF 2

METRIC

W.P. 4091-07-00 LOCATION N 5 031 175.7 E 369 856.0 Lees Avenue ORIGINATED BY GA
 HWY 417 BOREHOLE TYPE Hollow Stem Augers/NQ Coring - CME 75 COMPILED BY AN
 DATUM Geodetic DATE 2011.06.28 - 2011.06.29 CHECKED BY LRB

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			20 40 60 80 100	PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	
60.7												
0.0	ASPHALT: (100mm)											
60.4												
0.3	SAND and GRAVEL (FILL)		1	SS	58		60					
	SAND, some silt, occasional shale fragments Very Dense to Compact Brown Dry (FILL)		2	SS	23							
	Concrete obstruction at 1.5m		3	SS	50/ 0.00		59					
58.4												
2.3	Silty SAND, some gravel, trace clay Dense to Very Dense Brown Damp (FILL)		4	SS	40		58					16 60 20 4
			5	SS	90							
			6	SS	50/ 0.075		57					
56.1												
4.6	SAND, some silt, some gravel Very Dense Brown to Grey Dry to Wet		7	SS	82/ 0.275		56					20 64 13 3
			8	SS	86/ 0.250		55					
			9	SS	72		54					10 74 16 (SI+CL)
53.8			10	SS	50/ 0.100							
6.9	Silty SAND, trace gravel Very Dense Brown Wet (TILL)						53					0 49 48 3
53.1			11	SS	82							
7.6	SAND and SILT, trace clay Very Dense Grey Wet						52					
52.0			12	SS	50/ 0.125							
8.7	Silty SAND, trace gravel Very Dense Grey Wet (TILL)						51					

Continued Next Page

+³ ×³: Numbers refer to
Sensitivity

20
15
10



(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No LA-09

2 OF 2

METRIC

W.P. 4091-07-00 LOCATION N 5 031 175.7 E 369 856.0 Lees Avenue ORIGINATED BY GA
 HWY 417 BOREHOLE TYPE Hollow Stem Augers/NQ Coring - CME 75 COMPILED BY AN
 DATUM Geodetic DATE 2011.06.28 - 2011.06.29 CHECKED BY LRB






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 (%)				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE				WATER CONTENT (%) w _P w w _L				GR	SA	SI	CL	
	Continued From Previous Page																			
49.4	Silty SAND , trace gravel Very Dense Grey Wet (TILL)		13	SS	50/ 0.100		50													
11.3	SHALE , fresh, thinly bedded, grey, occasional limestone interbeds Limestone interbeds (between 25mm to 100mm thick) at 11.3m, 11.6m, 11.8m, 12.0m, 12.2m and 12.3m 50mm highly broken zone at 11.2m Horizontal joints at 11.3m and 11.6m Limestone interbeds (between 25mm to 100mm thick) at 12.8m, 12.9m, 13.3m, 13.4m and 13.5m		1	RUN			49													
							48													
			2	RUN			47													
46.4																				
14.3	END OF BOREHOLE AT 14.3m. BOREHOLE OPEN TO 14.3m AND WATER LEVEL AT 5.2m UPON COMPLETION. BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG FROM 14.3m TO 0.2m, THEN ASPHALT PATCH TO SURFACE.																			

RECORD OF BOREHOLE No STG-7

1 OF 2

METRIC

W.P. 4091-07-00 LOCATION N 5 031 237.0 E 369 654.0 Lees Avenue Staging Area ORIGINATED BY GA
HWY 417 BOREHOLE TYPE Hollow Stem Augers/NQ Coring COMPILED BY AN
DATUM Geodetic DATE 2011.08.11 - 2011.08.12 CHECKED BY LRB

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						
								WATER CONTENT (%)						
61.2							20 40 60 80 100	20 40 60						
0.0	Sandy SILT, trace gravel, occasional rootlets		1	SS	21		61							
60.4	Compact Brown Dry (FILL)													
0.8	Silty SAND, trace gravel		2	SS	25		60							
	Compact Brown Dry (FILL)													
	Black (possible coal tar) from 1.5m to 1.8m		3	SS	19		59							
	Grey, with hydrocarbon odour	4	SS	23	58									
		5	SS	17	57									
56.6			6	SS	32		56							
4.6	SAND, some silt, some gravel													
	Dense to Very Dense Grey Wet		7	SS	111	55								
53.5			8	SS	12	53								
7.6	Sandy SILT													
	Compact Grey Wet (TILL)													
52.0			9	SS	50/ 0.0	52								
9.1	SHALE, fresh, thinly bedded, grey/black, limestone interbeds through out													
	Augered to 10.0m													

Continued Next Page


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

RECORD OF BOREHOLE No STG-7

2 OF 2

METRIC

W.P. 4091-07-00 LOCATION N 5 031 237.0 E 369 654.0 Lees Avenue Staging Area ORIGINATED BY GA
HWY 417 BOREHOLE TYPE Hollow Stem Augers/NQ Coring COMPILED BY AN
DATUM Geodetic DATE 2011.08.11 - 2011.08.12 CHECKED BY LRB

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		
Continued From Previous Page																		
48.1	Highly broken zone (50mm thick) at 10.0m and 10.2m		1	RUN		51								3	RUN #1 TCR=100% SCR=93% RQD=93% UCS=20MPa (Average)			
						50								0				
			2	RUN		49								1		RUN #2 TCR=100% SCR=100% RQD=100% UCS=18MPa (Average)		
														0				
13.1	END OF BOREHOLE AT 13.1m. BOREHOLE OPEN TO 13.1m AND WATER LEVEL AT 3.9m UPON COMPLETION. BOREHOLE BACKFILLED WITH CUTTING AND BENTONITE HOLEPLUG TO SURFACE.																	

+³, x³: Numbers refer to
Sensitivity

20
15 10 5
(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No STG-8

1 OF 2

METRIC

W.P. 4091-07-00 LOCATION N 5 031 195.6 E 369 707.7 Lees Avenue Staging Area ORIGINATED BY GA
 HWY 417 BOREHOLE TYPE Hollow Stem Augers/NQ Coring COMPILED BY AN
 DATUM Geodetic DATE 2011.08.10 - 2011.08.11 CHECKED BY LRB

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 (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa		WATER CONTENT (%)					
								20 40 60 80 100		20 40 60					
								○ UNCONFINED + FIELD VANE							
								● QUICK TRIAXIAL × LAB VANE							
60.8								20 40 60 80 100		20 40 60					
0.0	Silty SAND, trace gravel, occasional rootlets Compact to Very Dense Brown Dry (FILL)		1	SS	25										
			2	SS	50/ 0.150										
	Concrete powder, possible rubble zone														
59.0			3	SS	23										
1.8	SAND, some silt, trace gravel Compact to Very Dense Brown Dry														
			4	SS	16										
			5	SS	19										
			6	SS	54/ 0.150										
	Grey														
54.7															
6.1	SAND and GRAVEL, trace silt and clay Very Dense Dark Grey Damp		7	SS	75										
53.2															
7.6	Silty SAND Dense to Very Dense Grey Wet (TILL)		8	SS	47										
			9	SS	64										
51.0															
9.8	SHALE, thinly bedded, grey														

Continued Next Page

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

METRIC

[illegible]

+³, ×³: Numbers refer to Sensitivity

RECORD OF BOREHOLE No STG-9

1 OF 2

METRIC

W.P. 4091-07-00 LOCATION N 5 031 233.7 E 369 750.1 Lees Avenue Staging Area ORIGINATED BY GA
 HWY 417 BOREHOLE TYPE Hollow Stem Augers/NQ Coring COMPILED BY AN
 DATUM Geodetic DATE 2011.08.11 - 2011.08.11 CHECKED BY LRB

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			20	40	60	80	100		
61.4														
0.0	Silty SAND, some gravel, trace rootlets Compact Brown Dry (FILL)		1	SS	29		61						o	
60.6														
0.8	SAND, some silt, trace gravel Compact to Very Dense Brown to Dark Brown Dry		2	SS	26		60						o	
			3	SS	32		59						o	3 82 15 (SI+CL)
			4	SS	27		58						o	
			5	SS	76		57						o	
			6	SS	112		56						o	
	Moist													
			7	SS	65		55						o	4 85 11 (SI+CL)
54.1														
7.3	SAND and GRAVEL Very Dense Dark Grey Wet		8	SS	50/ 0.150		54						o	
							53							
52.6														
8.8	Silty SAND, some clay, trace gravel, occasional shale fragments Very Dense Dark Grey Wet (TILL)		9	SS	50/ 0.150		52						o	

Continued Next Page

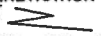
+³, x³: Numbers refer to Sensitivity 20 15 10 (% STRAIN AT FAILURE

RECORD OF BOREHOLE No STG-9

2 OF 2

METRIC

W.P. 4091-07-00 LOCATION N 5 031 233.7 E 369 750.1 Lees Avenue Staging Area ORIGINATED BY GA
 HWY 417 BOREHOLE TYPE Hollow Stem Augers/NQ Coring COMPILED BY AN
 DATUM Geodetic DATE 2011.08.11 - 2011.08.11 CHECKED BY LRB

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
	Continued From Previous Page						20 40 60 80 100							
							20 40 60 80 100	○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE						
							20 40 60 80 100	WATER CONTENT (%) PLASTIC LIMIT (w _p) NATURAL MOISTURE CONTENT (w) LIQUID LIMIT (w _L)						
50.1			10	SS	50/ 0.0		51						FI	
11.3	SHAILE, thinly bedded, fresh, grey, very thin limestone interbeds through out Highly broken zone from 12.0m to 12.1m		1	RUN			50						1	
													0	
													>3	
							49						0	RUN #1 TCR=100% SCR=95% RQD=90% UCS=24MPa (Average)
													0	
													0	
			2	RUN			48						0	RUN #2 TCR=100% SCR=100% RQD=100% UCS=23MPa (Average)
													0	
47.1														
14.3	END OF BOREHOLE AT 14.3m. BOREHOLE OPEN TO 14.3m AND WATER LEVEL AT 5.1m UPON COMPLETION. BOREHOLE BACKFILLED WITH CUTTINGS AND BENTONITE HOLEPLUG TO SURFACE.													

+³, X³: Numbers refer to
Sensitivity

20
15 10 5
(%) STRAIN AT FAILURE

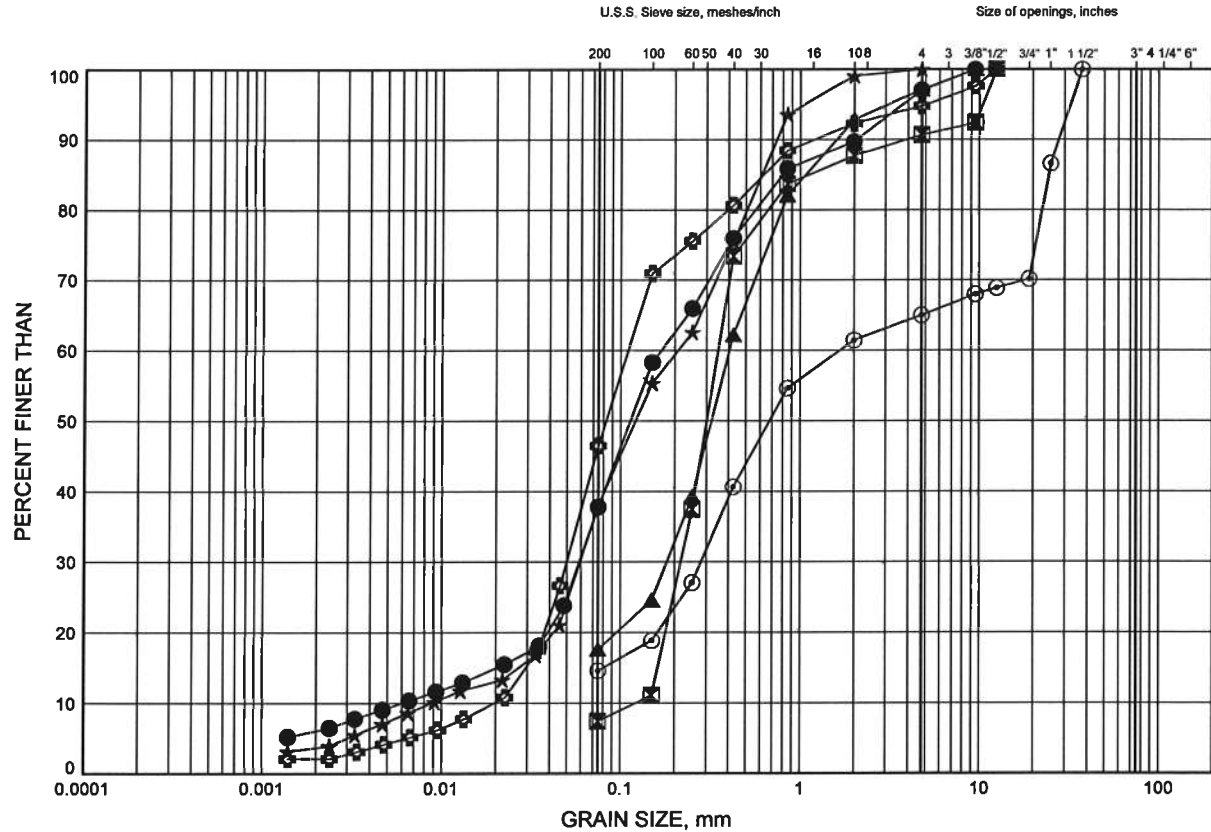
Appendix B

Laboratory Test Results

Highway 417 Ottawa: Nicholas to Vanier GRAIN SIZE DISTRIBUTION

FIGURE B1

SILTY SAND/SAND FILL



SILT and CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND			GRAVEL		

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	LE-01	2.59	60.45
⊠	LE-02	3.35	60.37
▲	LE-04	3.35	65.27
★	LE-04	7.77	60.85
⊙	LE-05	1.83	66.44
⊕	LE-05	4.85	63.42

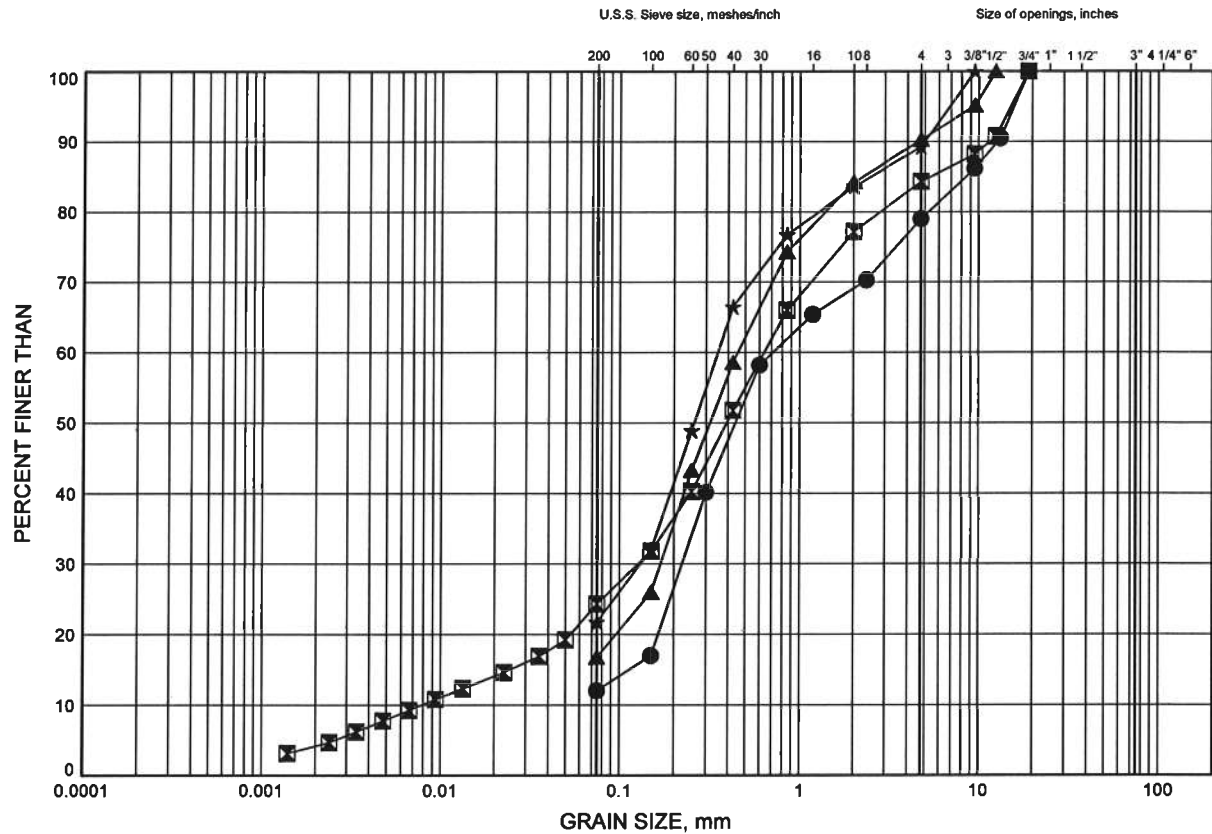


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Prepared By .AN.....
Checked By .MC.....

Highway 417 Ottawa: Nicholas to Vanier GRAIN SIZE DISTRIBUTION

FIGURE B2

SILTY SAND/SAND FILL



SILT and CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND			GRAVEL		

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	EBH-02	1.07	59.03
⊠	LA-09	2.59	58.11
▲	LE-06	2.59	66.70
★	LE-06	4.78	64.51

GRAIN SIZE DISTRIBUTION - THURBER 1201A.GPJ 11/10/11

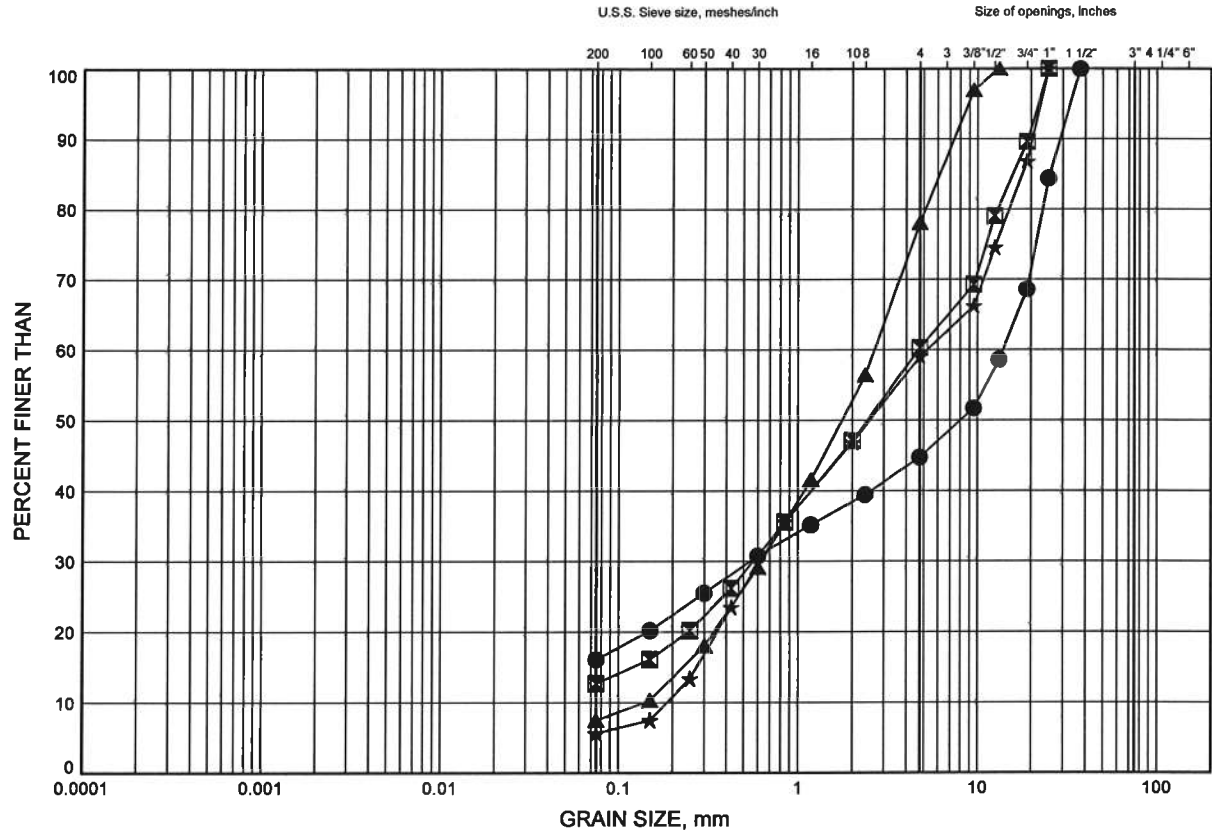
W.P.# .4091-07-00.....
Prepared By .AN.....
Checked By .MC.....



Highway 417 Ottawa: Nicholas to Vanier GRAIN SIZE DISTRIBUTION

FIGURE B3

SAND and GRAVEL



SILT and CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND			GRAVEL		

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	LE-06	9.45	59.84
⊠	LE-07	2.59	57.51
▲	LE-07	4.88	55.22
★	LE-07	6.40	53.70

FIGURE B4

U.S.S. Sieve size, meshes/inch

Size of openings, inches

PERCENT FINER THAN

GRAIN SIZE, mm

Grain Size (mm)	Percent Finer Than (%) - Triangles	Percent Finer Than (%) - Squares	Percent Finer Than (%) - Circles
0.075	32	21	23
0.15	65	31	30
0.3	94	86	41
0.6	99	96	52
1.2	100	99	59
2.5	100	100	65
5.0	100	100	75
10.0	100	100	81
20.0	100	100	89
40.0	100	100	100

SILT and CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND			GRAVEL		

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	EBH-02	4.11	55.99
⊠	EBH-02	6.40	53.70
▲	EBH-02	7.92	52.18
★	LE-07	7.85	52.25

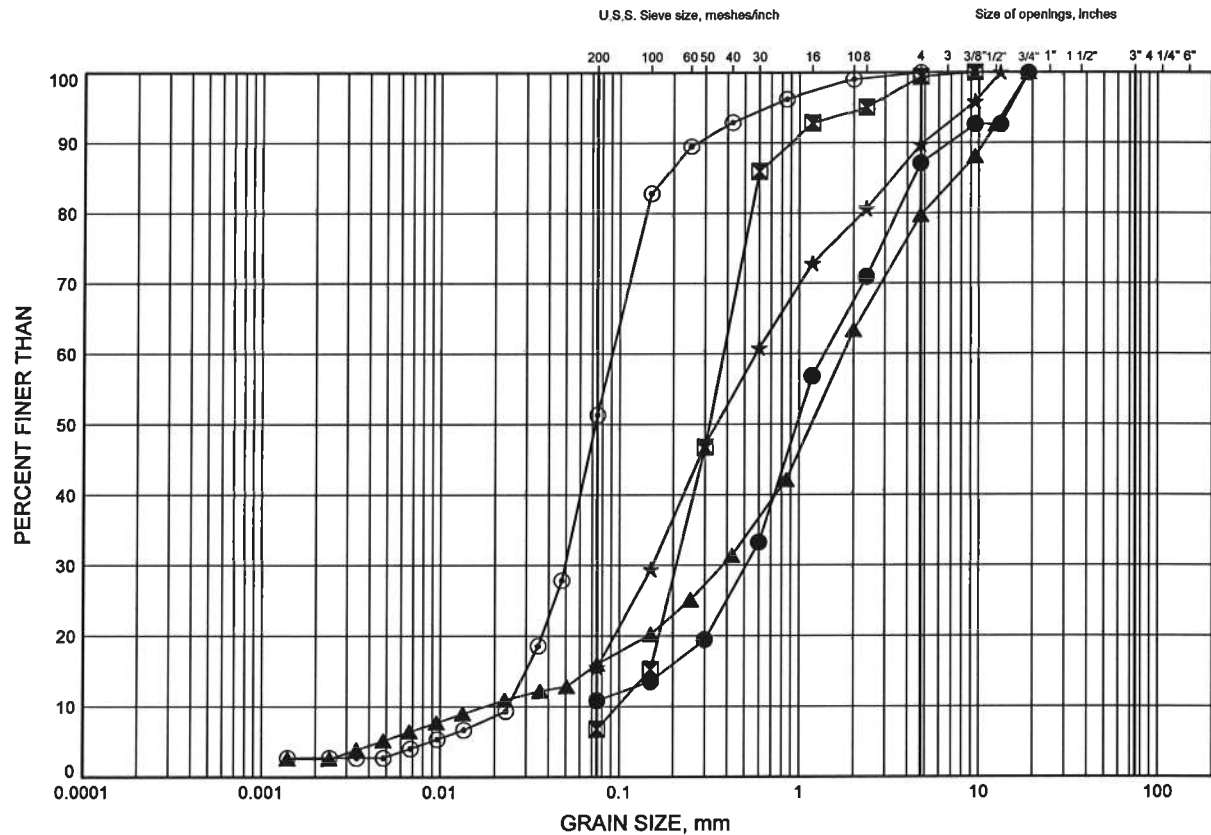


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Prepared By AN
Checked By MC

Highway 417 Ottawa: Nicholas to Vanier GRAIN SIZE DISTRIBUTION

FIGURE B5

SILTY SAND to GRAVELLY SAND



SILT and CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND			GRAVEL		

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	LA-02	4.88	56.22
⊠	LA-02	7.16	53.94
▲	LA-09	4.88	55.82
★	LA-09	6.40	54.30
⊙	LA-09	7.92	52.77

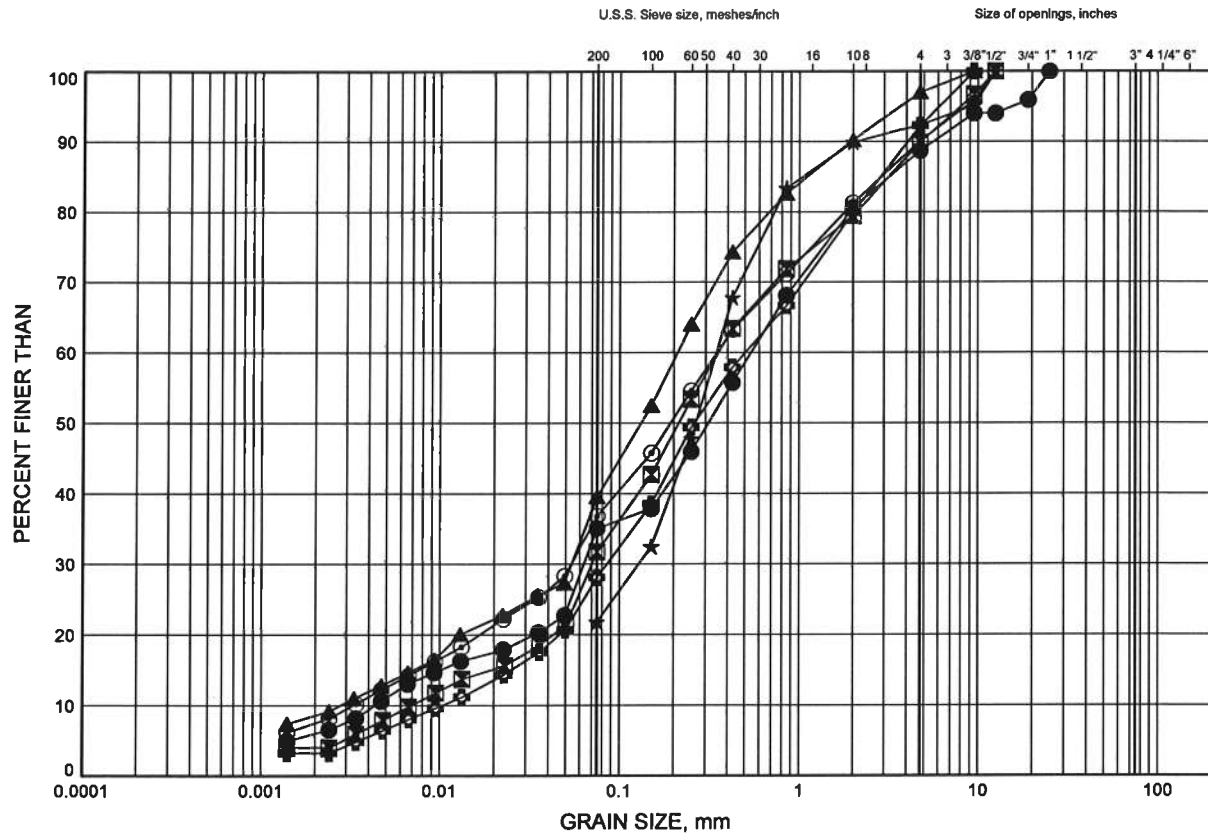


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Highway 417 Ottawa: Nicholas to Vanier GRAIN SIZE DISTRIBUTION

FIGURE B6

SILTY SAND TILL



SILT and CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND			GRAVEL		

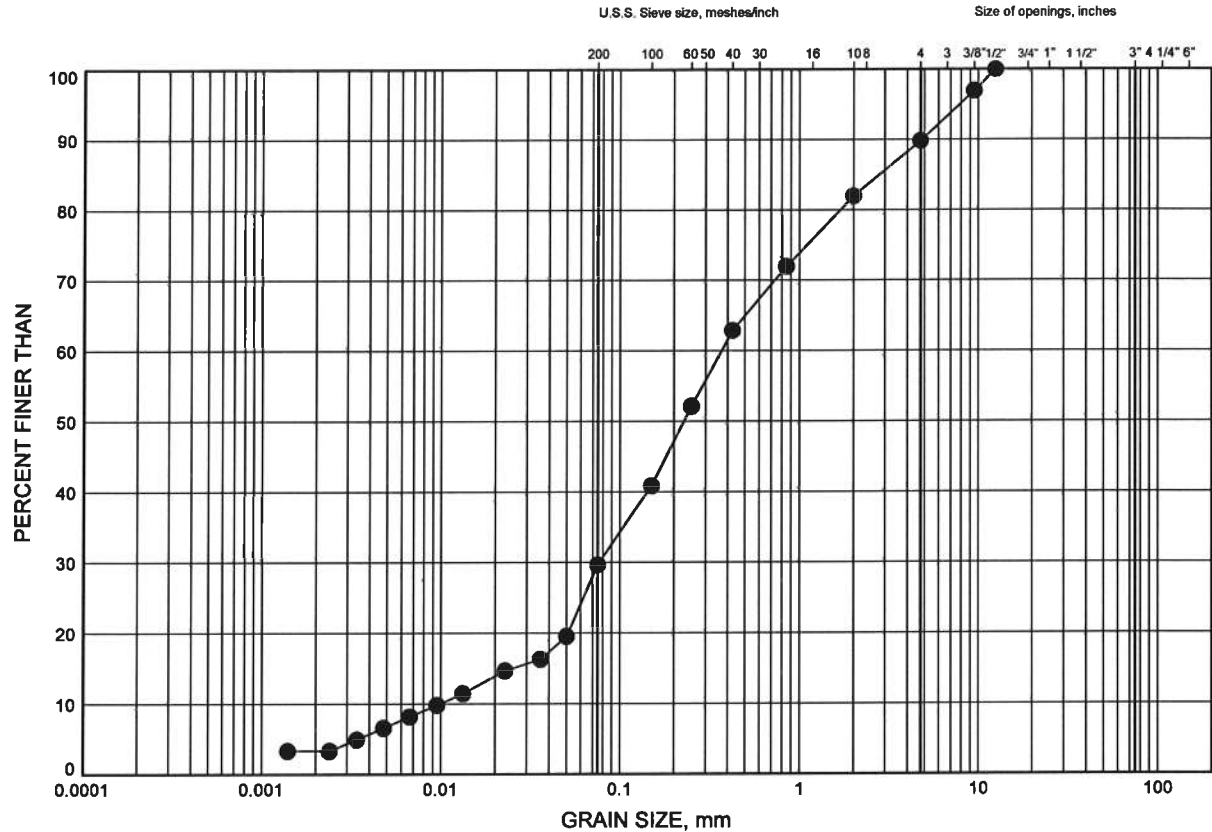
LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	LA-02	2.59	58.51
⊠	LE-04	10.97	57.65
▲	LE-05	7.92	60.34
★	LE-06	6.38	62.91
⊙	LE-06	12.50	56.79
⊕	LE-07	10.74	49.35

Highway 417 Ottawa: Nicholas to Vanier GRAIN SIZE DISTRIBUTION

FIGURE B7

SILTY SAND FILL STAGING AREA



SILT and CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND			GRAVEL		

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	STG-7	1.07	60.09

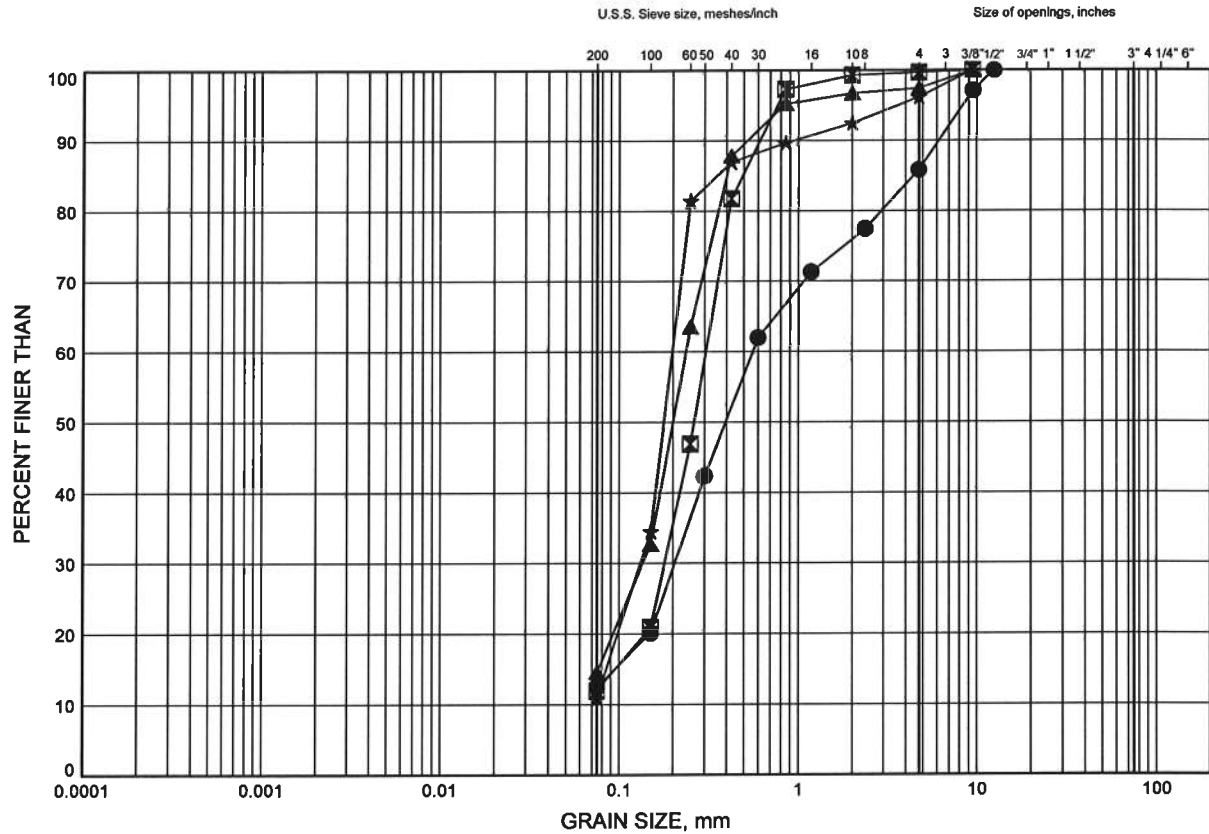


W.P.# 4091-07-00.....
Prepared By AN.....
Checked By MC.....

Highway 417 Ottawa: Nicholas to Vanier GRAIN SIZE DISTRIBUTION

FIGURE B8

SAND LAYER with Some GRAVEL STAGING AREA



SILT and CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND			GRAVEL		

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	STG-7	6.40	54.76
⊠	STG-8	2.59	58.20
▲	STG-9	1.83	59.58
★	STG-9	6.40	55.01

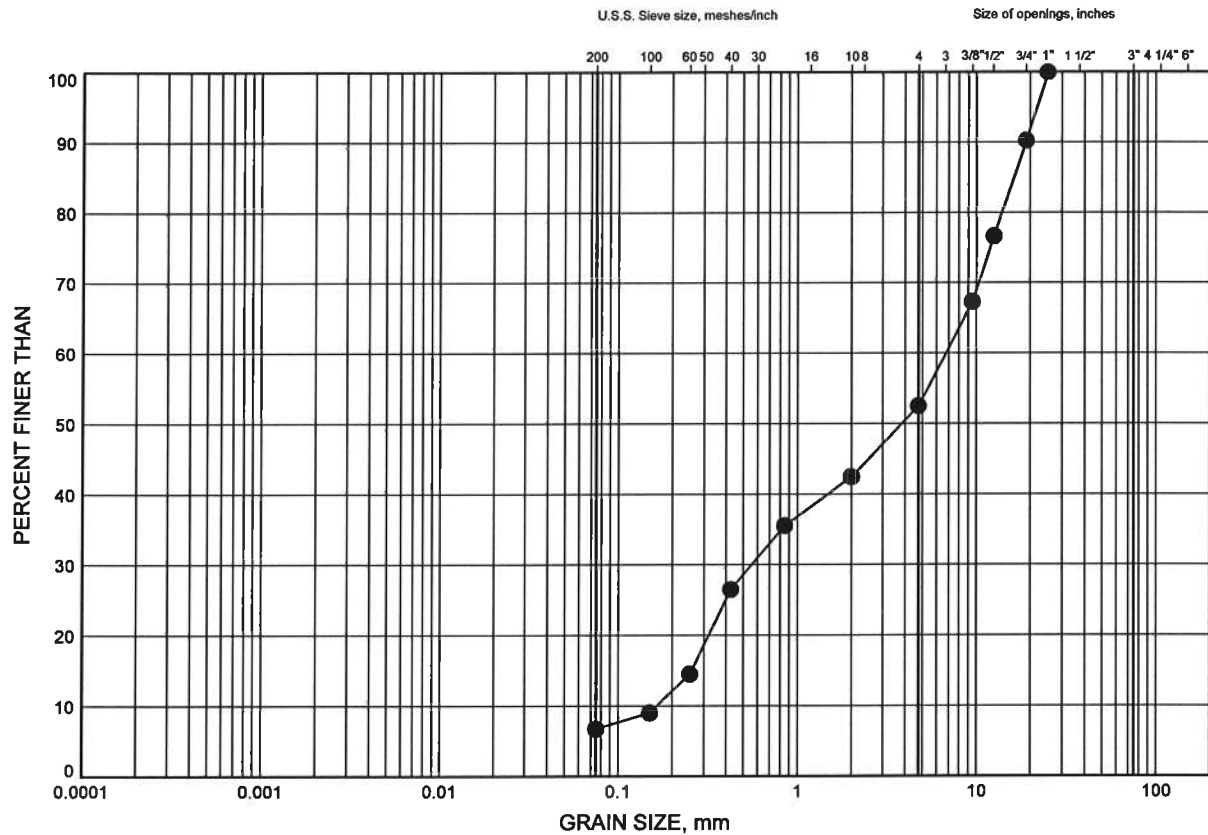


W.P.# 4091-07-00.....
Prepared By .AN.....
Checked By .MC.....

Highway 417 Ottawa: Nicholas to Vanier GRAIN SIZE DISTRIBUTION

FIGURE B9

SAND and GRAVEL



SILT and CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND			GRAVEL		

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	STG-8	6.40	54.39



W.P.# 4091-07-00
Prepared By AN
Checked By MC

Appendix C

Site Photographs



Photograph 1: South Abutment of Lees Avenue Underpass



Photograph 2: North Pier of Lees Abutment Underpass



Photograph 3: Lees Avenue Underpass from Robinson Avenue



Photograph 4: East Side of North Approach to Lees Avenue Underpass

Appendix D

Foundation Comparison

COMPARISON OF FOUNDATION ALTERNATIVES FOR NEW PIER

Footings on Native Soil	Footings on Engineered Fill	Caissons Socketed into Bedrock	Socketed Steel H-Piles	Micropiles
<p>Advantages:</p> <ul style="list-style-type: none"> i. Straightforward and economical to construct. ii. High geotechnical resistance available on native soils at this site. 	<p>Advantages:</p> <ul style="list-style-type: none"> i. Relatively economical to install. ii. High geotechnical resistance available. 	<p>Advantages:</p> <ul style="list-style-type: none"> i. High geotechnical resistance available by socketing caissons into bedrock. ii. Provides uplift and overturning resistance. iii. Facilitates construction within narrow median work zone. iv. Reduced differential settlement. v. Construction of caissons could continue in freezing weather. vi. Subexcavation of fill and variable material not required. 	<p>Advantages:</p> <ul style="list-style-type: none"> i. High geotechnical resistance available by socketing piles into bedrock. ii. Provide uplift and overturning resistance. iii. Reduced differential settlement. iv. Readily installed. v. Installation less influenced by weather and groundwater than spread footings. vi. Subexcavation of fill and variable material not required. 	<p>Advantages:</p> <ul style="list-style-type: none"> i. Provide uplift and overturning resistance. ii. Reduced differential settlement. iii. Readily installed. iv. Installation less influenced by weather and groundwater than spread footings. v. Subexcavation of fill and variable material not required.
<p>Disadvantages:</p> <ul style="list-style-type: none"> i. Does not provide sufficient resistance to seismic events. ii. Roadway protection required. iii. Narrow median work zone may limit footing width. 	<p>Disadvantages:</p> <ul style="list-style-type: none"> i. Cost of engineered fill placement is not warranted at this site. ii. Does not provide sufficient resistance to seismic events. iii. Roadway protection required. iv. Narrow median work zone may limit footing width. 	<p>Disadvantages:</p> <ul style="list-style-type: none"> i. Difficulty in unwatering, cleaning and inspecting bases. ii. Possibility of cobbles and boulders impeding drilling. iii. Higher unit cost compared to other foundation options such as footings or piles. 	<p>Disadvantages:</p> <ul style="list-style-type: none"> i. Pre-drilling required for installation of socket piles. ii. Concrete placement may require tremie pipe iii. Higher unit cost compared to footings. 	<p>Disadvantages:</p> <ul style="list-style-type: none"> i. Inadequate lateral resistance. ii. Does not provide sufficient resistance to seismic events. iii. Higher cost compared to other foundation options.
FEASIBLE	NOT RECOMMENDED	RECOMMENDED	FEASIBLE	NOT RECOMMENDED

Appendix E

List of SPs and OPSS, and Suggested Text for Selected NSSP

1. List of Special Provisions and OPSS Documents Referenced in this Report

- OPSS 501
- OPSS 510
- OPSS 539
- OPSS 804
- OPSS 902
- OPSS 903
- OPSS 1010
- OPSD 208.010
- OPSD 3000.100
- OPSD 3101.150
- Special Provision 110S13
- Special Provision 206S03
- Special Provision 999S26

2. Suggested text for NSSP on Caisson Installation [Socketed Piles]

The soils on site consist of dense to very dense sand, sand and gravel, and silty sand till. The sand/gravel and till may contain cobbles, boulders and shale slabs. Further, the till is underlain by shale bedrock containing hard limestone layers. These materials will potentially have an impact on the installation of caissons [socketed piles] at the site. Some possible impacts that must be taken into consideration include, but are not necessarily limited to:

- The cobbles, boulders and shale slabs may impede the drilling of the caissons [pile sockets] resulting in lower production and faster wear of drilling bits.
- Rock coring equipment may be required in cases where augering cannot push aside or penetrate obstructions in the till or where thick layers of hard limestone are encountered in the bedrock.
- The cobbles, boulders and shale slabs may impact the alignment of the caissons [piles] during drilling and accordingly drill advancement must be carefully controlled.

The Contractor is further advised that non-cohesive soils and high groundwater levels are present on site. Non-cohesive soil is susceptible to disturbance under conditions of unbalanced hydrostatic head. The Contractor is responsible for constructing the caisson excavation and rock socket without disturbing the sides or base of the excavation, and for cleaning of the socket base. The construction method is the responsibility of the Contractor, but consideration could be given to temporary liners, mud drilling and tremie concrete techniques where conditions warrant.

3. Suggested Text for NSSP on Rock Sockets

Pile installation shall be in accordance with OPSS 903 and the following:

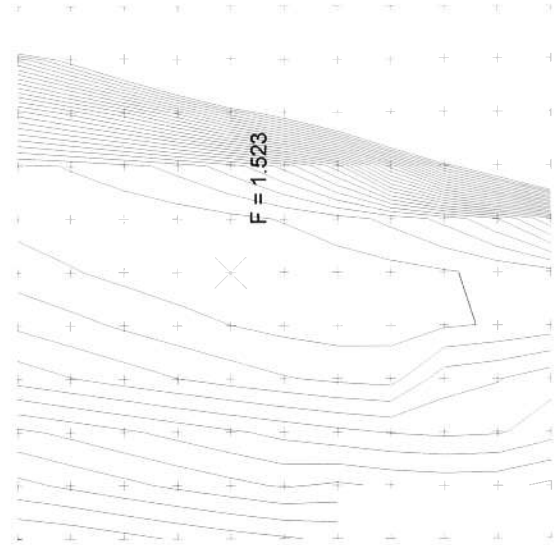
- Foundation piles are to be installed to the specified socket depth in shale bedrock by drilling or coring to the required depth, cleaning of the socket base, inserting the pile complete with bottom pin or plate, and backfilling the socket around the pile with concrete, all as shown on the contract drawings.
- The Contractor is advised that non-cohesive soils and high groundwater levels are present on site. Non-cohesive soil is susceptible to disturbance under conditions of unbalanced hydrostatic head.
- The Contractor is responsible for constructing the excavation and rock socket for socketed pile installation without disturbing the sides or base of the excavation. The construction method is the responsibility of the Contractor, but consideration could be given to temporary liners and tremie concrete techniques where conditions warrant.

4. Suggested Text for NSSP on Caisson [Socket] Concrete

The shale in the caisson [pile] socket must be protected from deterioration by placement of concrete as soon as practical after completion of the excavation and in no case later than 8 hours after excavation.

Appendix F

Slope Stability Analyses



	Gamma C kN/m3	Phi deg	Piezo Surf.
Sand Fill	21	0	32
Sand/Till	21	0	34
Bedrock	21	0	45

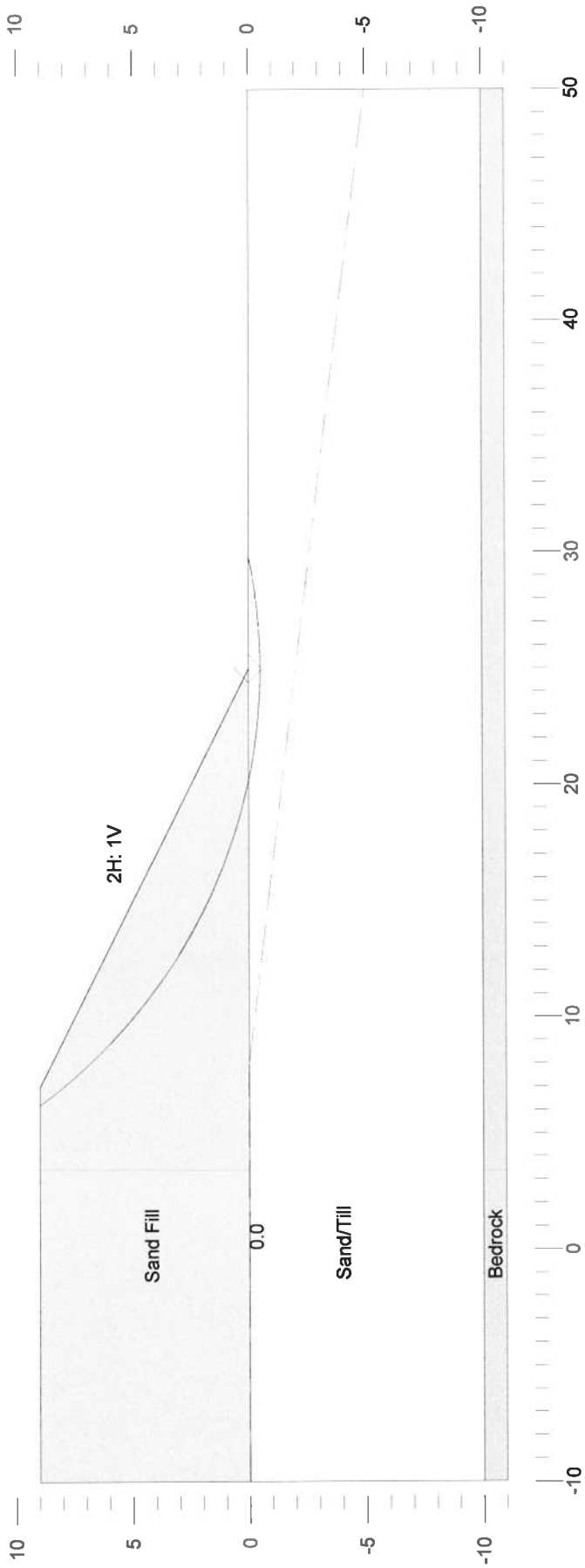
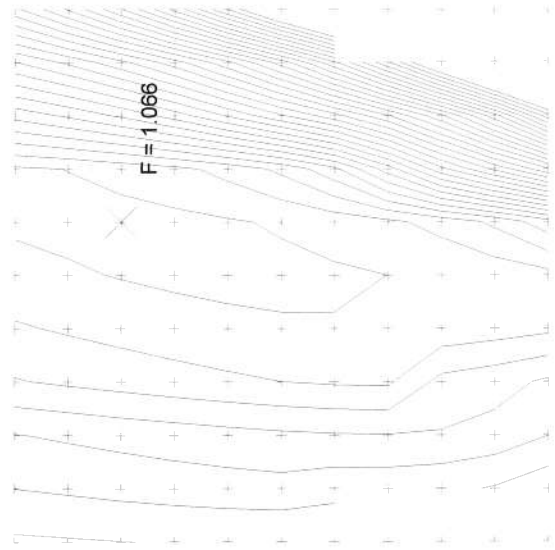


Figure F1

Thurber Engineering Ltd. - Toronto
 19-1351-201
 Lees Avenue
 Feb 2012
 9m high 2H:1V



	Gamma	C	Phi	Piezo
	kN/m3	kPa	deg	Surf.
Sand Fill	21	0	32	1
Sand/Till	21	0	34	1
Bedrock	21	0	45	1

Seismic coefficient = 0.16

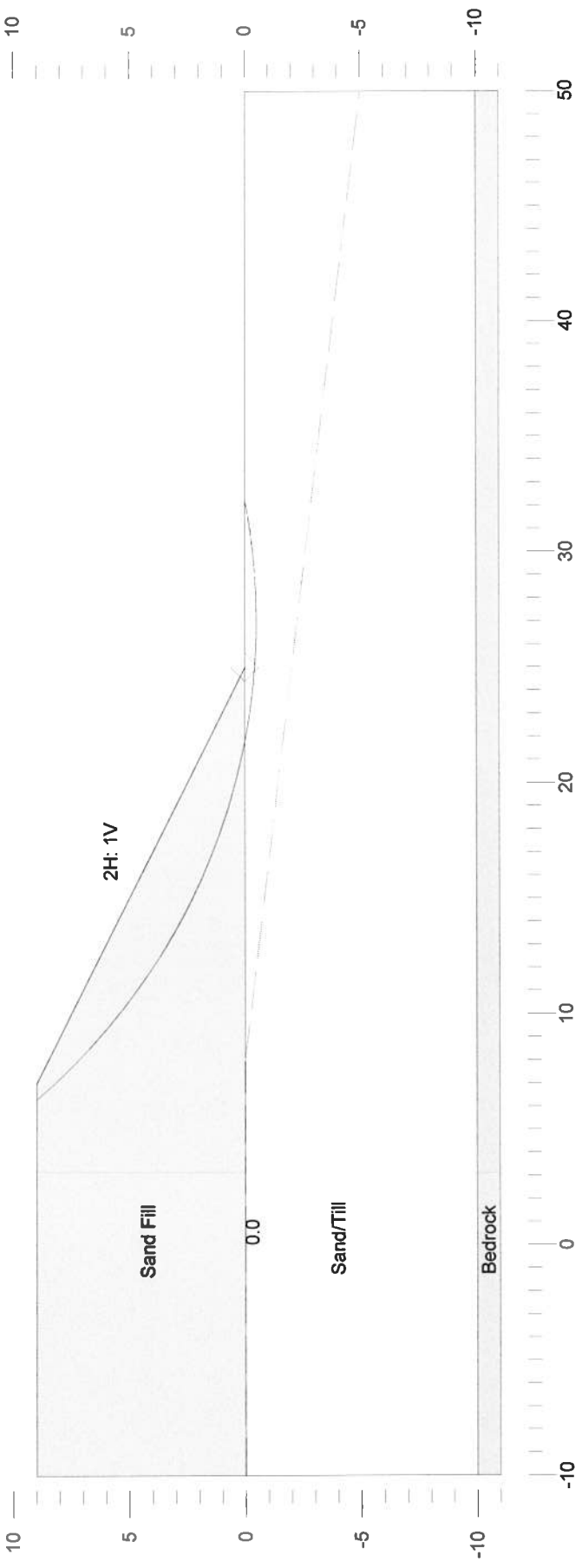
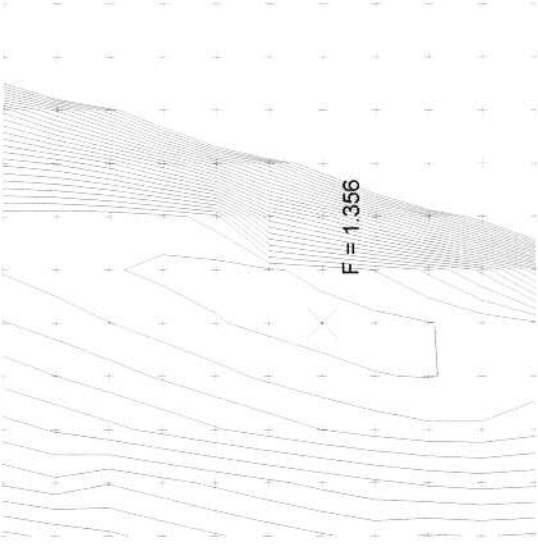


Figure F2

	Gamma C kN/m3	Phi kPa	Piezo deg	Surf.
Sand Fill	21	0	32	1
Sand/Till	21	0	34	1
Bedrock	21	0	45	1



Thurber Engineering Ltd. - Toronto
19-1351-201
Lees Avenue
Feb 2012
9m high 1.7H:1V

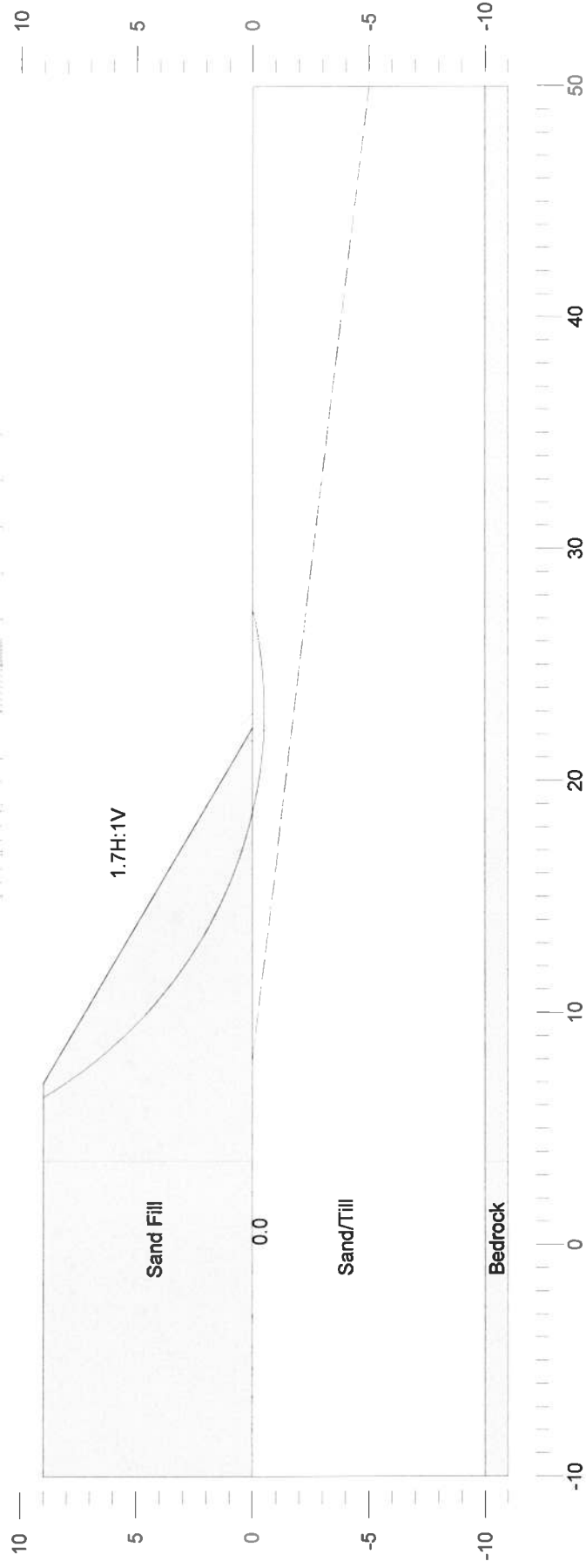
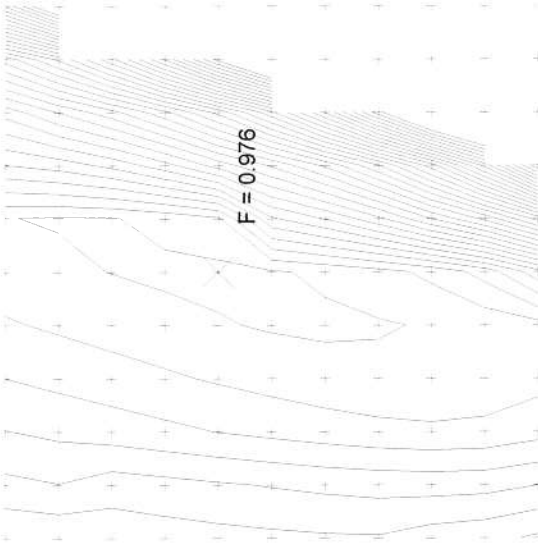


Figure F3



	Gamma	C	Phi	Piezo
	kN/m3	kPa	deg	Surf.
Sand Fill	21	0	32	1
Sand/Till	21	0	34	1
Bedrock	21	0	45	1

Seismic coefficient = 0.16

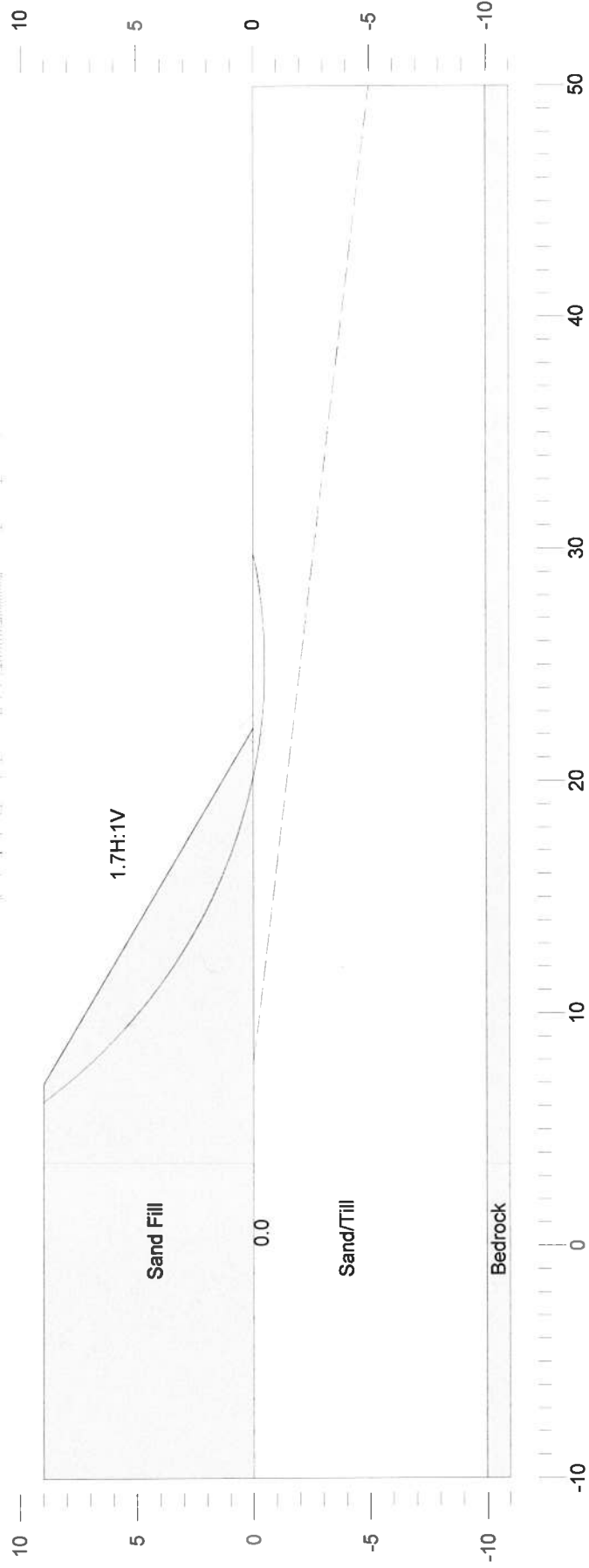


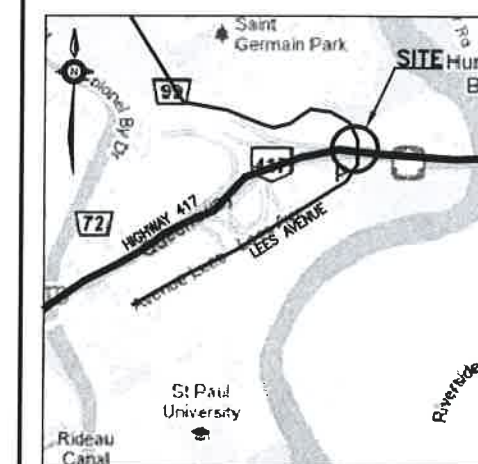
Figure F4

Appendix G

Drawing Borehole Locations and Soil Strata








SHEET
1



KEYPLAN

LEGEND

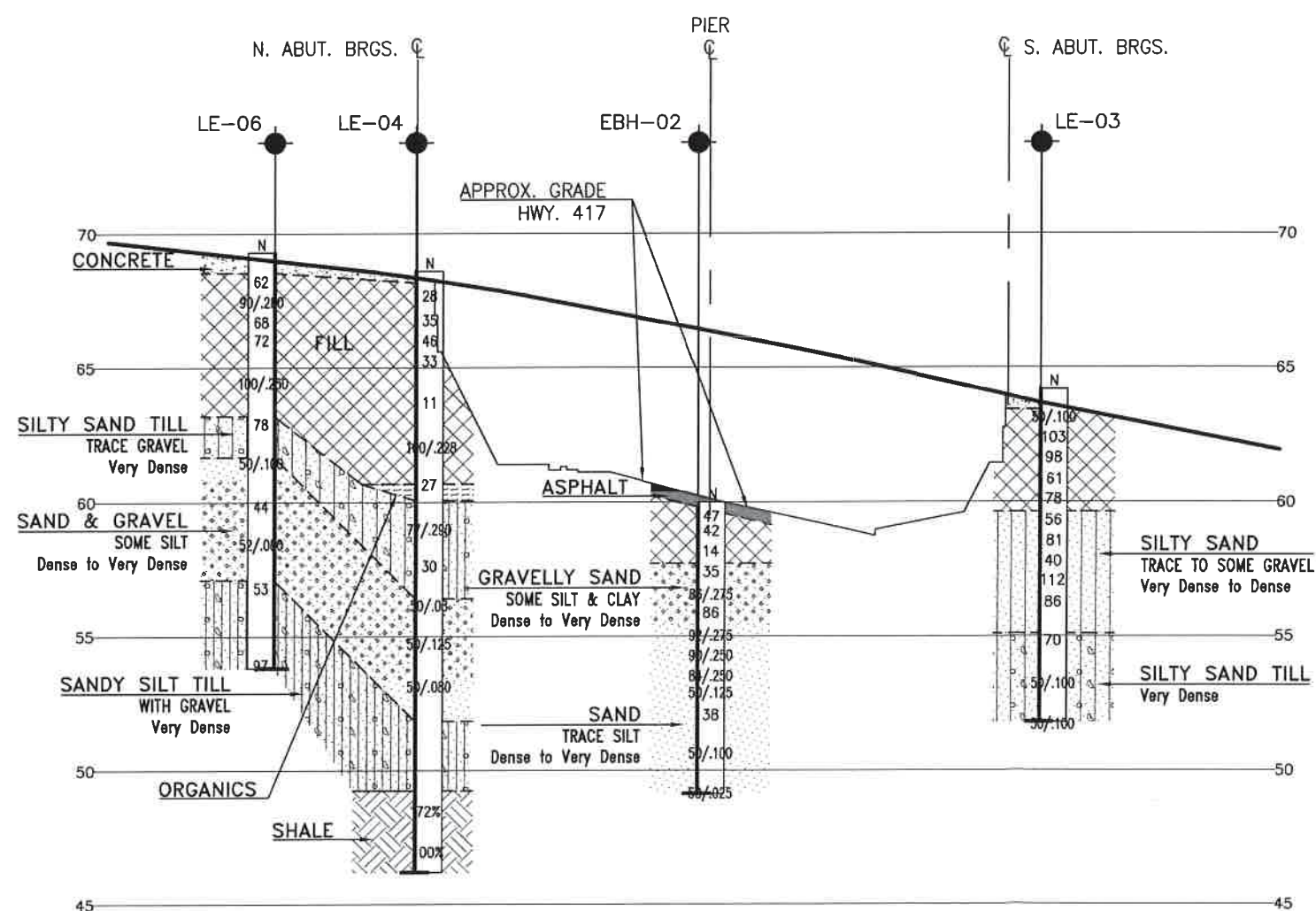
- | | |
|---|---------------------------------------|
|  | Borehole |
|  | Borehole and Cone |
| N | Blows /0.3m (Std Pen Test, 475J/blow) |
| CONE | Blows /0.3m (60° Cone, 475J/blow) |
| PH | Pressure, Hydraulic |
|  | Water Level |
|  | Head Artesian Water |
|  | Piezometer |
| 90% | Rock Quality Designation (RQD) |
| A/R | Auger Refusal |

NO	ELEVATION	NORTHING	EASTING
LE-01	63.0	5 031 087.0	369 814.
LE-02	63.7	5 031 101.0	369 819.
LE-03	64.2	5 031 097.0	369 827.
LE-04	68.6	5 031 190.0	369 832.
LE-05	68.3	5 031 193.0	369 823.
LE-06	69.3	5 031 211.0	369 826.
LE-07	60.1	5 031 150.0	369 819.
EBH-02	60.1	5 031 148.0	369 844.
LA-02	61.1	5 031 107.0	369 842.
LA-09	60.7	5 031 176.0	369 856.

-NOTES-

- 1) 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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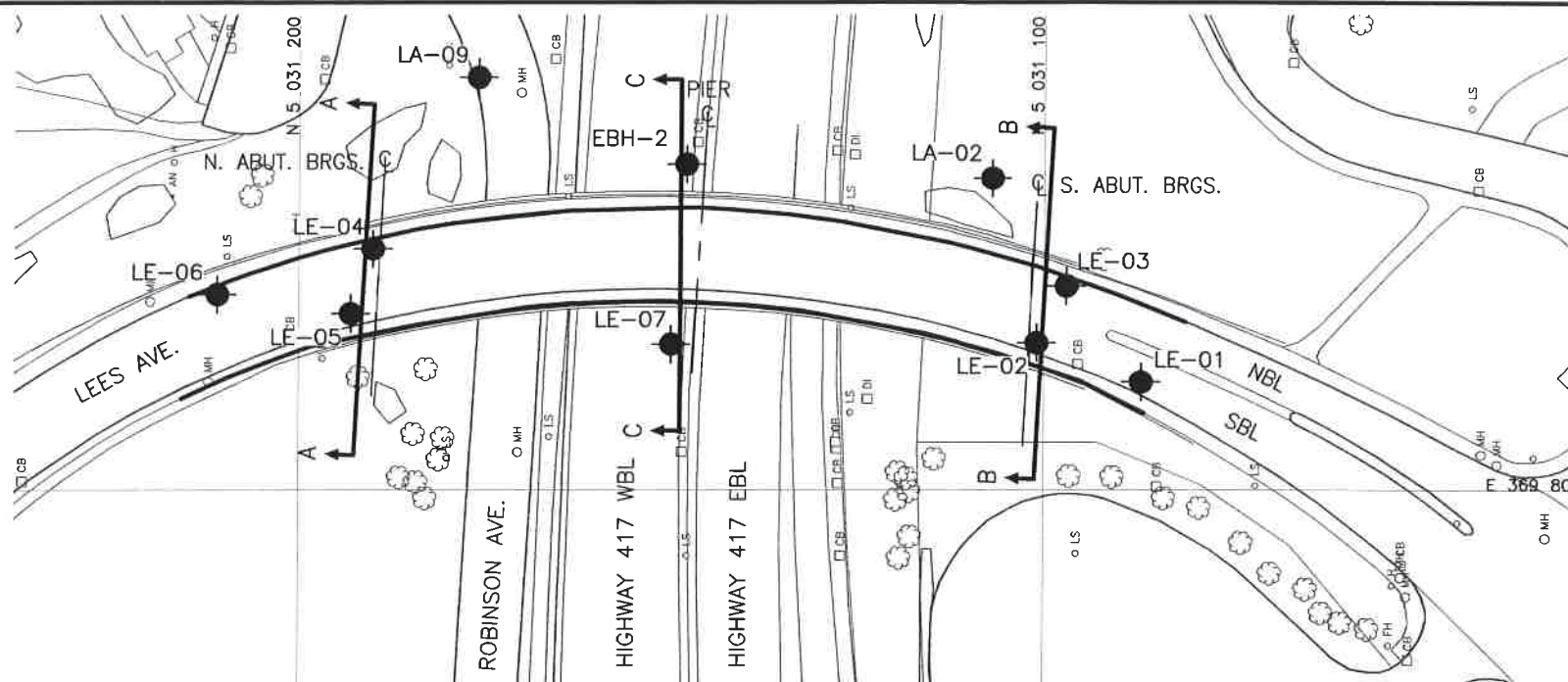
GEOCRES No. 31G5-246



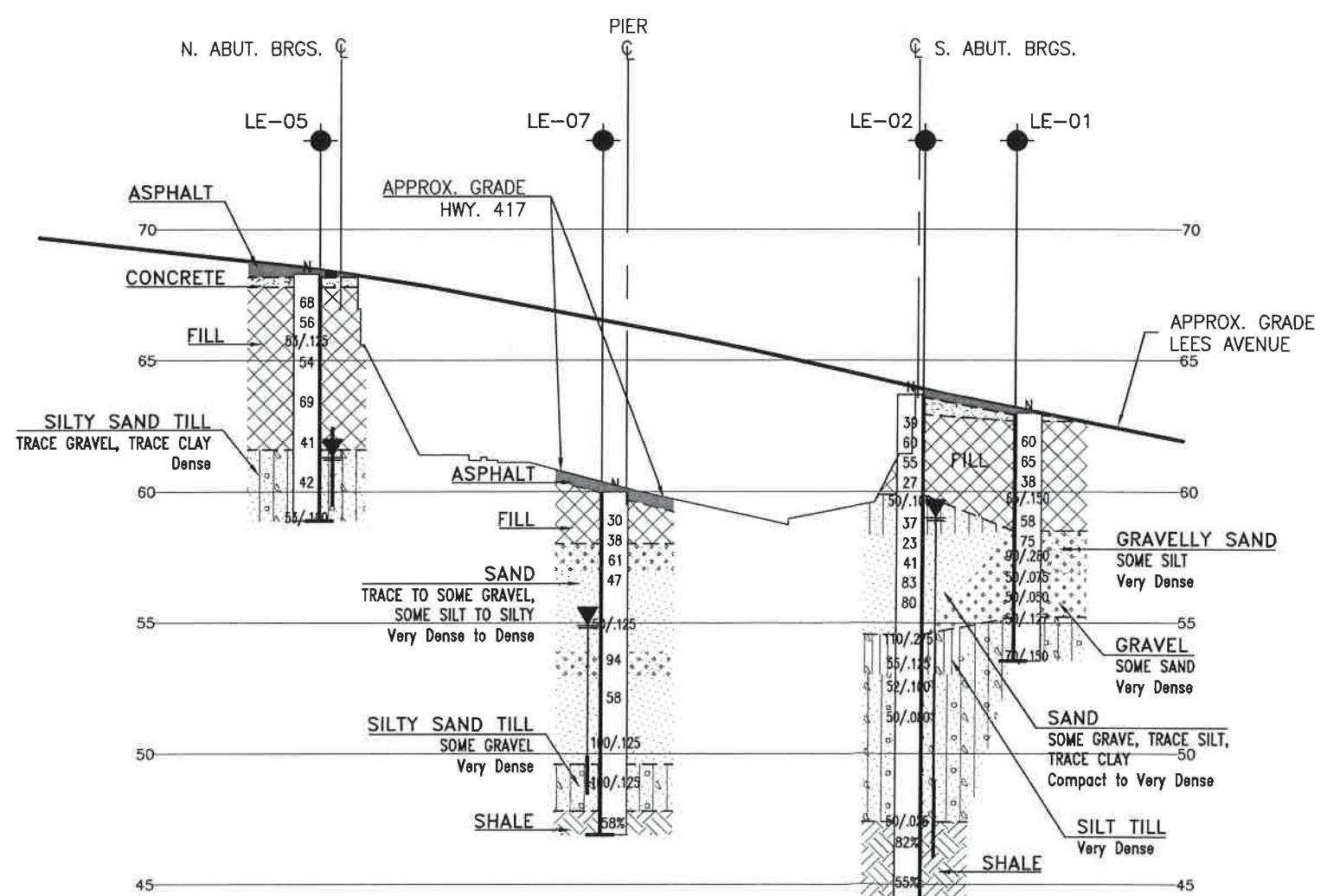
PROFILE ALONG LEES AVE. NBL



REVISIONS									
	DATE	BY			DESCRIPTION				
DESIGN	MC	CHK	MRA	CODE	LOAD		DATE	APR. 201	
DRAWN	AN	CHK		SITE	INSTRCT	WDWG	1		



PLAN
SCALE 1:1000



PROFILE ALONG LEES AVE. SBL
H 1:1000
V 1:250

METRIC
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN

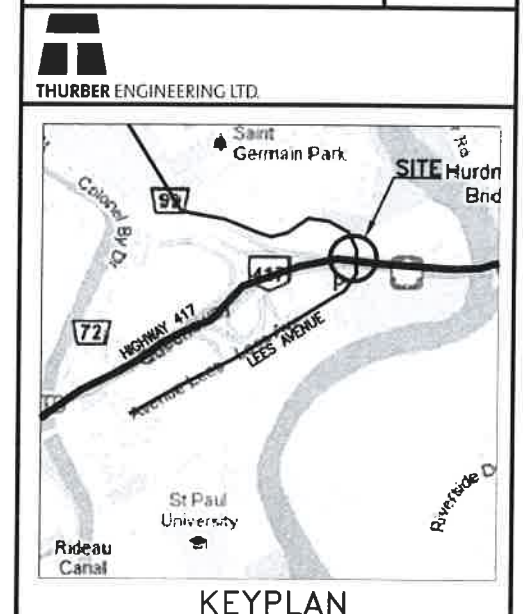


CONT No
WP No 4091-07-00

HIGHWAY 417
LEES AVENUE UNDERPASS
BOREHOLE LOCATIONS AND SOIL STRATA

McCORMICK RANKIN
CORPORATION

THURBER ENGINEERING LTD.



LEGEND			
	Borehole		
	Borehole and Cone		
N	Blows /0.3m (Std Pen Test, 475J/blow)		
CONE	Blows /0.3m (60° Cone, 475J/blow)		
PH	Pressure, Hydraulic		
	Water Level		
	Head Artesian Water		
	Piezometer		
90%	Rock Quality Designation (RQD)		
A/R	Auger Refusal		

NO	ELEVATION	NORTHING	EASTING
LE-01	63.0	5 031 087.0	369 814.6
LE-02	63.7	5 031 101.0	369 819.9
LE-03	64.2	5 031 097.0	369 827.6
LE-04	68.6	5 031 190.0	369 832.7
LE-05	68.3	5 031 193.0	369 823.9
LE-06	69.3	5 031 211.0	369 826.5
LE-07	60.1	5 031 150.0	369 819.7
EBH-02	60.1	5 031 148.0	369 844.2
LA-02	61.1	5 031 107.0	369 842.3
LA-09	60.7	5 031 176.0	369 856.0

- NOTES-**
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GEOCRES No. 31G5-246

REVISIONS			
DATE	BY	DESCRIPTION	
DESIGN MC	CHK MRA	CODE	LOAD
DRAWN AN	CHK	SITE	STRUCT
			DWG 2
			DATE APR. 2012

METRIC

DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN

CONT No
WP No 4091-07-00



HIGHWAY 417
LEES AVENUE UNDERPASS
BOREHOLE LOCATIONS AND SOIL STRATA

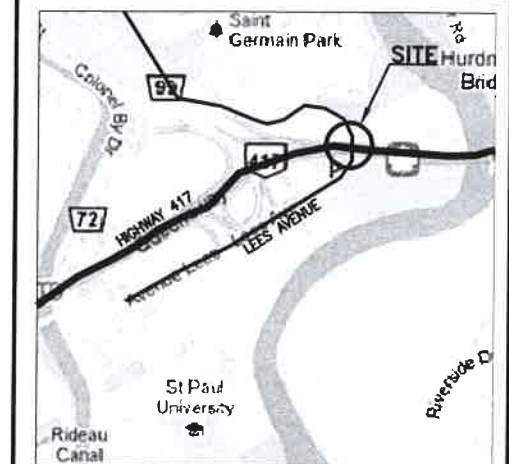
SHEET
3



McCORMICK RANKIN
CORPORATION



THURBER ENGINEERING LTD.



KEYPLAN

LEGEND

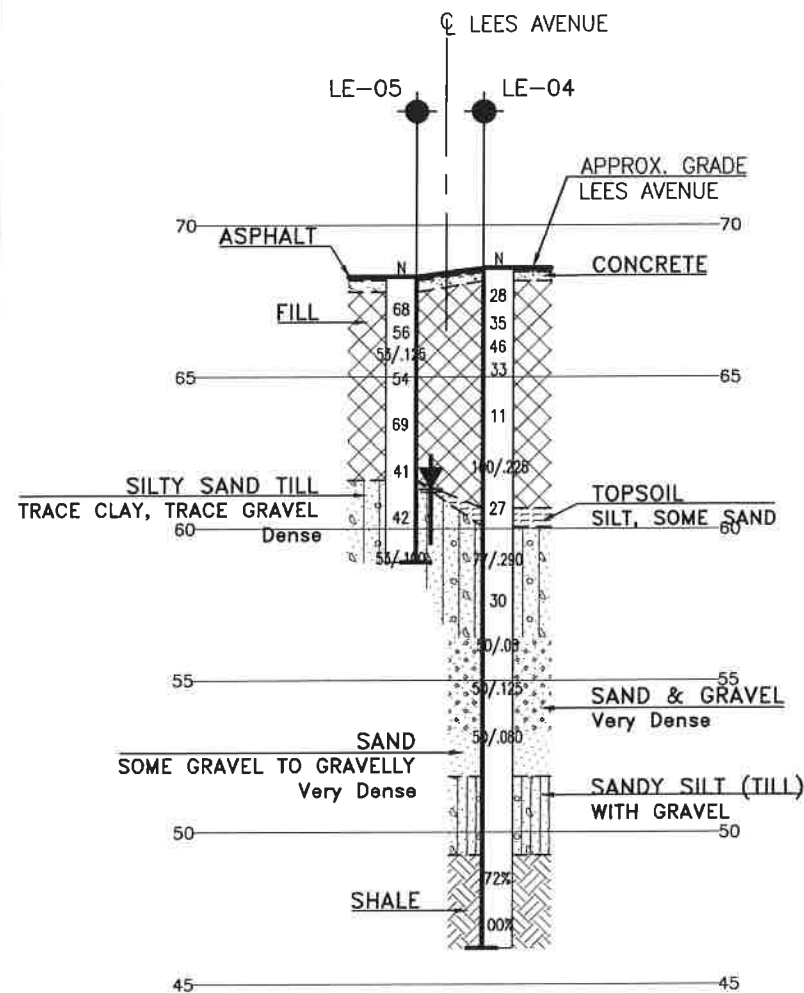
●	Borehole
⊕	Borehole and Cone
N	Blows /0.3m (Std Pen Test, 475J/blow)
CONE	Blows /0.3m (60' Cone, 475J/blow)
PH	Pressure, Hydraulic
W	Water Level
HA	Head Artesian Water
P	Piezometer
90%	Rock Quality Designation (RQD)
A/R	Auger Refusal

NO	ELEVATION	NORTHING	EASTING
LE-01	63.0	5 031 087.0	369 814.6
LE-02	63.7	5 031 101.0	369 819.9
LE-03	64.2	5 031 097.0	369 827.6
LE-04	68.6	5 031 190.0	369 832.7
LE-05	68.3	5 031 193.0	369 823.9
LE-06	69.3	5 031 211.0	369 826.5
LE-07	60.1	5 031 150.0	369 819.7
EBH-02	60.1	5 031 148.0	369 844.2
LA-02	61.1	5 031 107.0	369 842.3
LA-09	60.7	5 031 176.0	369 856.0

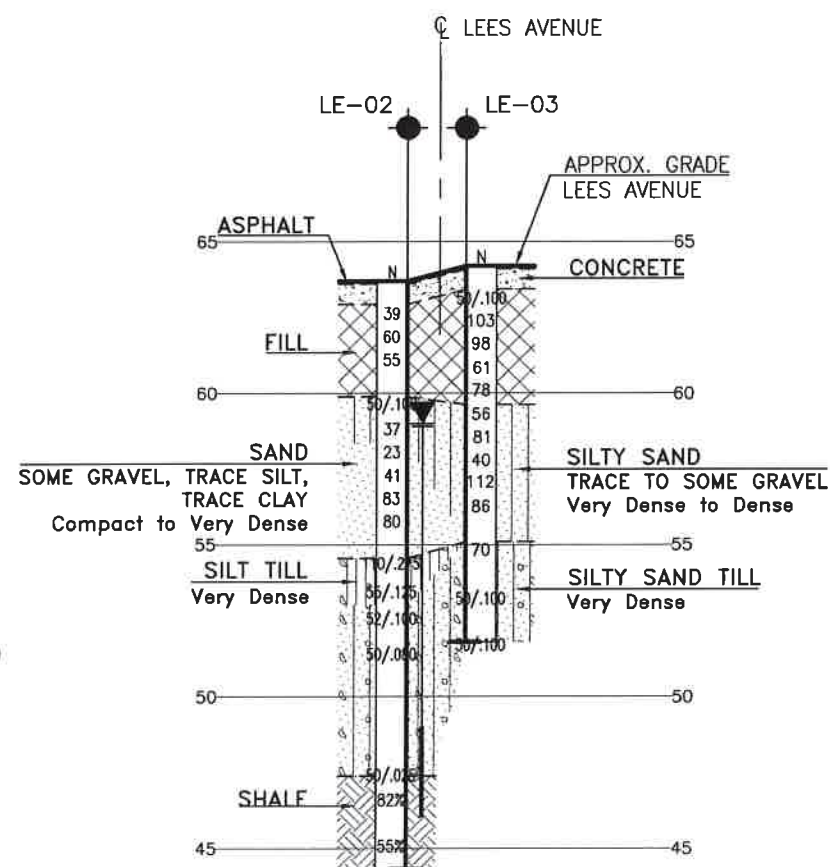
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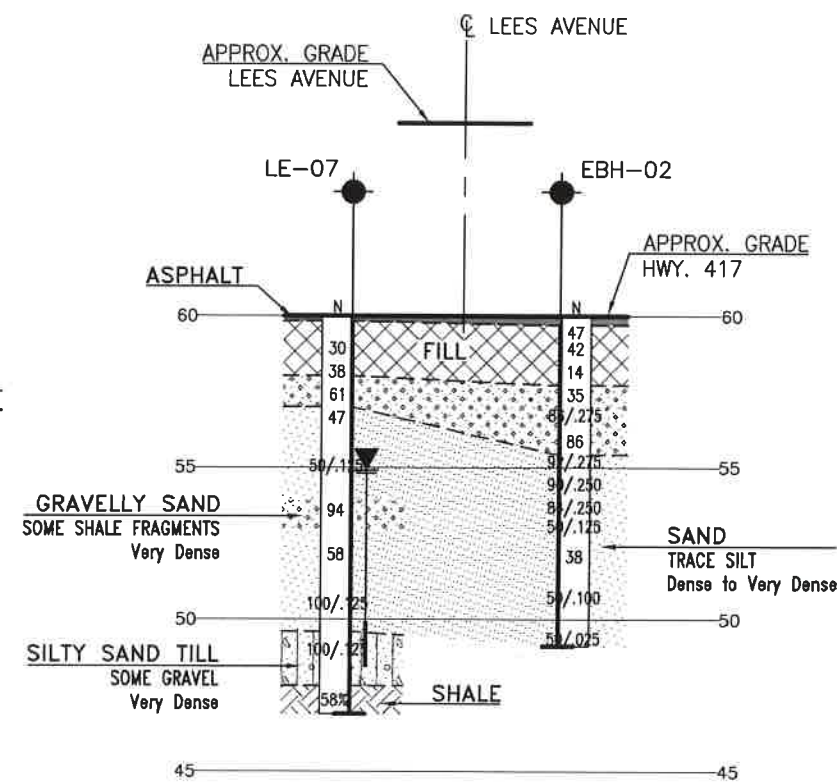
GEOCRES No. 31G5-246



SECTION A-A



SECTION B-B



SECTION C-C



H 1:1000

V 1:250



REVISIONS	DATE	BY	DESCRIPTION
DESIGN	MC	CHK	MRA
DRAWN	AN	CHK	SITE
			LOAD
			STRUCT
			DWG 3
			DATE APR. 2012

CONT No
WP No 4091-07-00



HIGHWAY 417
LEES AVENUE STAGING AREA
BOREHOLE LOCATION PLAN

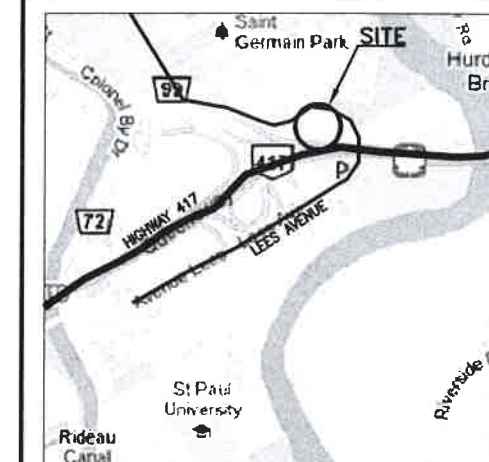
SHEET
1








McCORMICK RANKIN
CORPORATION



THURBER ENGINEERING LTD.



KEYPLAN
LEGEND

	Borehole
	Borehole and Cone
N	Blows /0.3m (Std Pen Test, 475J/blow)
CONE	Blows /0.3m (60° Cone, 475J/blow)
PH	Pressure, Hydraulic
	Water Level
	Head Artesian Water
	Piezometer
90%	Rock Quality Designation (RQD)
A/R	Auger Refusal

NO	ELEVATION	NORTHING	EASTING
STG-7	61.2	5 031 237.0	369 654.0
STG-8	60.8	5 031 195.6	369 707.7
STG-9	61.4	5 031 233.7	369 750.1

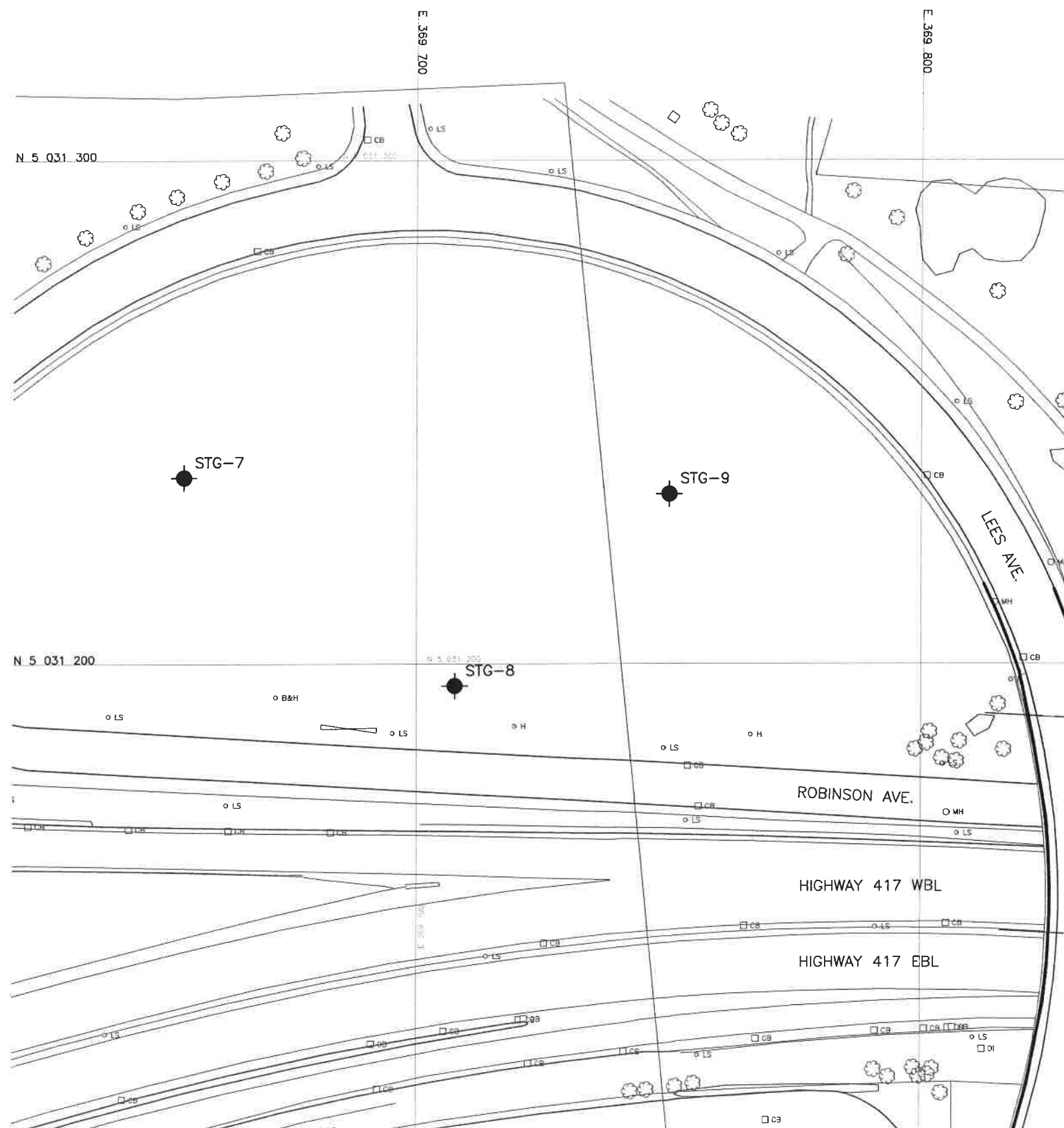
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GEOCRES No. 31G5-246

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FILENAME: H:\Drafting\19\135\201\201-BoresolePlan&Profile(LessAvenue).dwg



PLAN

