

**FOUNDATION INVESTIGATION AND DESIGN REPORT  
BELFAST ROAD UNDERPASS REHABILITATION  
HIGHWAY 417 EXPANSION FROM VANIER PARKWAY TO OR 174  
OTTAWA, ONTARIO**

**G.W.P. 4320-06-00, SITE No. 3-071**

**Geocres Number: 31G5-242**

**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 Belfast Road underpass structure over Highway 417 in Ottawa, Ontario. The structure rehabilitation is part of the Highway 417 Expansion project, from Vanier Parkway to OR 174.

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, record of borehole sheets, 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 Belfast Road Underpass is located on Highway 417 approximately 5km east of Ottawa city centre. The underpass spans approximately 49 m across Highway 417, between Coventry Road (Regional Road 50) at the north and Tremblay Road at the south. The existing underpass is a two-span structure supported by a pier and two abutments. The substructures are founded on steel H-piles driven to bedrock.

Land use surrounding the site is commercial/industrial in the northeast, northwest and southwest quadrants, and residential to the southeast. A rigid frame bridge structure crosses the Central Transitway immediately south of the underpass. The Rideau River is located approximately 1km to the west.

The site lies within the Ottawa Valley Clay Plains physiographic region, which comprises a clay plain interrupted by ridges of sand or rock. At the specific underpass site however, the general stratigraphy comprises glacial silt/sand till overlying bedrock at relatively shallow depth. The bedrock consists of the Carlsbad Formation, comprising dark grey shale interbedded with calcareous siltstone and limestone.

Photographs in Appendix E show the general nature of the site. No stability or performance issues were noted on the roadways and existing slopes adjacent to the abutments.

### **3 SITE INVESTIGATION AND FIELD TESTING**

The site investigation and field testing for this project were carried out at various stages during the period July 16 to November 25, 2011 and consisted of the following:

- On July 16, 2011, Borehole BFR-01 was drilled to 8.2 m depth in the approach fill behind the north abutment. Borehole BFR-04 was attempted in the south approach but could not penetrate a concrete layer. Subsequently on November 15 and 16, 2011, both boreholes were extended into shale bedrock to evaluate anchor design behind the abutments. These boreholes were terminated at 16.2 and 16.5 m depth.
- Boreholes BFR-02 and BFR-03 were drilled adjacent to the existing pier on July 24, 2011. These boreholes were terminated in shale at 7.3 and 8.5 m depth.
- Boreholes 15N-05 and 23S-02 were drilled adjacent to the abutments on July 22 and 23, 2011 as part of the concurrent investigation for retaining walls. These boreholes were terminated in shale at 5.4 and 7.3 m depth.
- On August 10, 2011, Boreholes BFS-1 to BFS-3 were drilled within the initially proposed staging area located in the northeast quadrant of the Vanier Parkway interchange. The staging area has since been relocated and this borehole data is provided for information purposes only.
- Boreholes BRS-1 to BRS-3 were drilled on November 24 and 25, 2011 in the revised staging area located in the southeast quadrant of the Highway 417 / St. Laurent Boulevard interchange. These boreholes were terminated in shale at depths of 5.9 to 7.3 m.

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

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 Belfast Road, and City of Ottawa consent was obtained for the boreholes drilled in the proposed staging areas.

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

The drilling and 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.

Groundwater conditions in the open boreholes were observed during the drilling operations. Standpipe piezometers consisting of 19 mm PVC pipe with a slotted screen were installed in selected boreholes. The completion details of the piezometers are summarized in Table 3.1. 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 at the surface.

**Table 3.1 – Piezometer Details**

Borehole	Tip Position (m)		Completion Details
	Depth	Elev.	
BFR-01	16.2	52.9	Sand filter from 16.2 to 14.3m, bentonite from 14.3 to 0.3m, then concrete to surface.
BFR-03	6.1	56.8	Sand filter from 8.5 to 2.7m, bentonite from 2.7 to 0.6m, cuttings from 0.6 to 0.15m, then asphalt cold patch to surface.
BFR-04	16.5	52.2	Sand filter from 16.5 to 14.6m, bentonite from 14.6 to 0.9m, concrete from 0.9 to 0.15m, then asphalt cold patch to surface.
15N-05	5.4	57.4	Sand filter from 5.4 to 3.0m, bentonite from 3.0 to 0.15m, then asphalt cold patch to surface.
23S-02	5.8	56.7	Sand filter from 7.3 to 2.1m, bentonite from 2.1 to 0.15m, then asphalt cold patch to surface.
BRS-2	7.3	61.6	Sand filter from 7.3 to 3.0m, bentonite from 3.0m to ground surface.
BFS-2	7.6	54.0	Sand filter from 7.6 to 4.3m, bentonite from 4.3 to 2.4m, cuttings from 2.4 to 0.05m, then asphalt cold patch to surface.

#### 4 LABORATORY TESTING

All recovered soil samples were subjected to Visual Identification 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.

## **5 DESCRIPTION OF SUBSURFACE CONDITIONS**

Reference is made to the Record of Borehole sheets in Appendix A and to the Borehole Locations and Soil Strata Drawings in Appendix F. An overall description of the stratigraphy based on the conditions encountered in the boreholes is given in the following paragraphs. However, the factual data presented in the borehole logs takes precedence over this general description and interpretation of the site conditions.

In general terms, the stratigraphy encountered in the boreholes consists of a pavement structure overlying silty sand fill, underlain by silt or silt fill, and then silty sand till. Shale bedrock was encountered below the till.

More detailed descriptions of the individual strata encountered at the existing bridge site and the staging area are presented below.

### **5.1 Underpass Site (Boreholes BFR-01 to BFR-04, 15N-05 and 23S-02)**

#### **5.1.1 Pavement Structure**

A pavement structure consisting of 100 mm of asphalt over 700 mm of sand, some gravel, was encountered in Borehole BFR-01 drilled on Belfast Road at the north approach to the underpass. In Borehole BFR-04 drilled at the south approach, 75 mm of asphalt was encountered over approximately 1.0 m of concrete.

In Boreholes BFR-02, BFR-03, 15N-05 and 23S-02 drilled on Highway 417, the pavement structure consists of 200 mm of asphalt overlying 600 mm of gravelly sand.

Moisture contents of 1 to 3% were measured in the granular material.

#### **5.1.2 Sand Fill**

Fill was encountered below the pavement structure in all boreholes. In Boreholes BFR-01 and BFR-04 drilled in the approaches, the fill consists of brown to grey sand containing trace to some silt, trace gravel, and trace clay. In Boreholes BFR-02, BFR-03, 15N-05 and 23S-02, the fill consists of brown silty sand containing trace to some gravel and trace clay.

In Boreholes BFR-01 and BFR-04, the sand fill layer was 3.8 and 3.3 m thick with a lower boundary at 4.6 and 4.4 m depth (Elev. 64.4 and 64.3 m). In the other boreholes, the fill was 0.6 to 1.6 m thick with a lower boundary at depths of 1.4 to 2.3 m (Elev. 61.2 to 60.6 m).

SPT ‘N’ values recorded in the cohesionless fill ranged from 11 to 53 blows/0.3 m, indicating a compact to very dense relative density. The moisture contents ranged from 2 to 8%.

Grain size distribution analyses were carried out on three samples of the sand fill. The results of these tests are plotted on Figure B1, Appendix B, and are summarized below.

Gravel %	7 to 16
Sand %	53 to 74
Silt %	15 to 33
Clay %	3 to 5

#### 5.1.3 Silt Fill

Brown silt fill containing trace to some sand and clay was encountered below the sand fill in Borehole BFR-04. The silt fill layer is 2.7 m thick with a lower boundary at 7.1 m depth (Elev. 61.6 m).

SPT ‘N’ values of 9 and 23 blows/0.3 m penetration were recorded in the silt fill, indicating a loose to compact condition. Moisture contents of 20 and 22% were measured.

#### 5.1.4 Silt

Native greenish-grey silt containing some sand and trace clay was encountered below the sand fill in Borehole BFR-01. The silt layer is 1.5 m thick with a lower boundary at 6.1 m depth (Elev. 62.9 m).

An SPT ‘N’ value of 7 blows/0.3 m penetration was recorded in the native silt layer, indicating a loose condition. A moisture content of 38% was measured.

#### 5.1.5 Silty Sand Till

Brown to dark grey silty sand till containing trace to some gravel and trace clay was encountered below the silt in Borehole BFR-01 and below the fill in the remaining boreholes.

The thickness of the silty sand till varied from 2.9 to 3.2 m in Boreholes BFR-02, BFR-03, 15N-05 and 23S-02, and the depth to the base of the till was 4.4 to 5.5 m (Elev. 58.3 to 57.4 m) in these boreholes. The till thickness was 7.0 and 4.8 m in Boreholes BFR-01 and BFR-04, with a lower boundary at 13.1 and 11.9 m depth (Elev. 55.9 and 56.8 m).

SPT ‘N’ values recorded in the silty sand till typically ranged from 32 blows/0.3 m penetration to 50 blows for no penetration, indicating a dense to very dense condition. Lower ‘N’ values of 4 and 26 blows/0.3 m were recorded in the upper part of the till in



Borehole BFR-01, indicating a loose to compact condition. Difficult augering was experienced in the till in five boreholes, and coring was required to advance Boreholes BFR-02 and BFR-03 through the till to the bedrock surface, indicating the possible presence of cobbles, boulders and shale slabs.

The moisture content of the silty sand till ranged from 3 to 12%.

Grain size distribution analyses were carried out on five samples of the silty sand till. The results of these tests are plotted on Figure B2, Appendix B, and are summarized below.

Gravel %	7 to 21
Sand %	44 to 56
Silt %	26 to 37
Clay %	3 to 12

Glacial tills are known to contain cobbles and boulders.

#### 5.1.6 Shale Bedrock

Bedrock was encountered below the silty sand till and proven by coring in all boreholes except Borehole 15N-05. Borehole 15N-05 was terminated at the bedrock surface. The depths and elevations at which bedrock was encountered are summarized in Table 5.1.

**Table 5.1 – Depths and Elevations of Bedrock Surface**

Borehole	Bedrock Surface	
	Depth (m)	Elevation (m)
BFR-01	13.1	55.9
BFR-02	4.4	58.3
BFR-03	5.5	57.4
BFR-04	11.9	56.8
15N-05	5.4	57.4
23S-02	4.6	57.9

The bedrock was described as grey shale with hard limestone interbeds up to 50 mm in thickness. It was generally described as moderately weathered in Borehole BFR-02 and slightly weathered to fresh in the remaining boreholes. Total Core Recovery (TCR) in the bedrock was 100% in all runs except Run 2 in Borehole BFR-04 where the core barrel jammed resulting in no recovery.

The RQD values recorded in the shale generally ranged from 27 to 100%, indicating a widely variable rock quality ranging from poor to excellent. An RQD value of 18% reported in the initial run in Borehole BFR-03 resulted from commencement of coring in very dense till above the bedrock surface. An RQD value of 0% (very poor) was recorded

in the upper 1.2 m run in Borehole 23S-02. The Fracture Index (FI) of the rock, expressed as fractures per 0.3 m of core, ranged from 0 to greater than 10.

The estimated unconfined compressive strength of the rock, interpreted from point load tests conducted on intact rock cores, ranged from 12 to 33 MPa, indicating a weak to medium strong rock strength classification.

#### 5.1.7 Water Levels

Groundwater was not observed in the boreholes during drilling. Water was added into the boreholes as part of the rock coring operations and therefore natural groundwater levels were not measured in the bedrock.

Standpipe piezometers were installed in selected borehole upon completion of drilling. The groundwater depths and elevations measured in the piezometer are shown in Table 5.2.

**Table 5.2 – Groundwater Depths and Elevations**

Borehole	Date	Water Level (m)	
		Depth	Elevation
BFR-01	29-Dec-11	5.4	63.6
BFR-03	26-Jul-11	3.5	59.4
	18-Aug-11	3.5	59.4
	20-Sep-11	3.8	59.1
	12-Oct-11	3.8	59.1
BFR-04	29-Dec-11	5.8	62.9
15N-05	26-Jul-11	2.9	59.9
	18-Aug-11	3.0	59.8
	12-Oct-11	3.2	59.6
23S-02	26-Jul-11	2.8	59.7
	18-Aug-11	2.8	59.7
	12-Oct-11	3.9	58.6

Seasonal fluctuations of the groundwater level are to be expected. In particular, the groundwater level may be at a higher elevation after the spring snowmelt or after periods of heavy rainfall.

## 5.2 Staging Area (Boreholes BRS-1 to BRS-3)

### 5.2.1 Topsoil

A 75 to 100 mm thick layer of topsoil was encountered in the boreholes drilled in the staging area in the southeast quadrant of the St. Laurent Boulevard interchange. The thickness and extent of the topsoil are expected to vary between and beyond the borehole locations, and the information in this report should not be used for quantity estimating.

### 5.2.2 Sand to Silty Sand Fill

Sand to silty sand fill containing trace to some gravel and clay was encountered below the topsoil layer in all boreholes. The fill was described variously as brown, dark brown, grey and orange-brown. The thickness of the fill layer was 0.5 to 1.6 m with a lower boundary at depths of 0.6 to 1.7 m (Elev. 66.4 to 67.8 m).

SPT 'N' values of 7 to 16 blows/0.3 m were recorded in the fill, indicating a loose to compact condition. Moisture contents of 4 to 18% were measured.

The results of a grain size distribution analysis carried out on a sample of the fill are plotted on Figure B3, Appendix B, and summarized below.

Gravel %	18
Sand %	48
Silt %	24
Clay %	10

### 5.2.3 Sand and Gravel

A 0.6 m thick layer of sand and gravel was encountered below the fill in Borehole BRS-2. The lower boundary of this layer was at 2.3 m depth (Elev. 66.6 m).

An SPT 'N' value of 16 blows/0.3 m was recorded in the sand and gravel, indicating a compact condition. A moisture content of 3% was measured.

### 5.2.4 Silty Clay

Grey to dark brown silty clay was encountered below the fill and sand and gravel at depths of 0.6 to 2.3 m. The silty clay was sandy in Boreholes BRS-1 and BRS-2, and contained trace to some sand in Borehole BRS-3.

The thickness of the silty clay unit ranged from 1.1 to 2.1 m. The depth to the base of the silty clay was 1.9 to 3.4 m (Elev. 65.1 to 65.7 m).

SPT 'N' values recorded in the silty clay ranged from 6 to 18 blows/0.3 m penetration, indicating a firm to very stiff consistency. An 'N' value of 62 blows/0.225 m was recorded at the bedrock surface in Borehole BRS-2.

The moisture content of the silty clay varied from 13 to 42%.

Grain size distribution analyses were carried out on three samples of the silty clay. The results of these tests are plotted on Figure B4, Appendix B, and are summarized below. The results of Atterberg Limits tests conducted on the samples are plotted on Figure B5, Appendix B, and are also shown below.

Gravel %	0 to 5
Sand %	9 to 27
Silt %	36 to 38
Clay %	32 to 52
Liquid Limit	37 to 44
Plastic Limit	17 to 23

The results of the Atterberg Limits tests indicate that the silty clay has a medium plasticity with a group symbol of CI.

#### 5.2.5 Shale Bedrock

Bedrock was encountered below the silty clay and proven by coring in all boreholes. The depths and elevations at which bedrock was encountered are summarized in Table 5.3.

**Table 5.3 – Depths and Elevations of Bedrock Surface**

Borehole	Bedrock Surface	
	Depth (m)	Elevation (m)
BRS-1	1.9	65.1
BRS-2	3.4	65.5
BRS-3	2.9	65.7

The bedrock was described as dark grey shale with hard limestone interbeds. It was generally described as fresh. Total Core Recovery (TCR) in the bedrock was 68 to 100%. The RQD values varied significantly, from 0% in the initial run in Boreholes BRS-1 and BRS-2, to 100 % in the final run in Borehole BRS-3. These values indicate a very poor to excellent quality rock. The Fracture Index (FI) of the rock, expressed as fractures per 0.3 m of core, ranged from 1 to 15.

The estimated unconfined compressive strength of the rock, interpreted from point load tests conducted on intact rock cores, ranged from 27 to 67 MPa, indicating a medium strong to strong rock strength classification.

#### 5.2.6 Water Levels

Groundwater was not observed in the boreholes during drilling. Water was added into the boreholes as part of the rock coring operations and therefore natural groundwater levels were not measured in the bedrock.

A standpipe piezometer was installed in Borehole BRS-3 upon completion of drilling. The groundwater depths and elevations measured in the piezometer are shown in Table 5.4.

**Table 5.4 – Groundwater Depths and Elevations**

Borehole	Date	Water Level (m)	
		Depth	Elevation
BRS-2	29-Dec-11	3.0	65.9

Seasonal fluctuations of the groundwater level are to be expected. In particular, the groundwater level may be at a higher elevation after the spring snowmelt or after periods of heavy rainfall.

## 6 MISCELLANEOUS

The borehole locations were selected and established in the field by Thurber Engineering Ltd. Surveyors from MMM Group determined the co-ordinates and ground surface elevations at the boreholes after completion of the site investigation.

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

The field investigation was supervised by Mr. Luke Gilarski, E.I.T. and Mr. Ryan Kromer, E.I.T. of Thurber. Overall planning and supervision of the field program was conducted by Ms. Lindsey Blaine, E.I.T.

Interpretation of the field data and preparation of the report were carried out by Ms. Lindsey Blaine, E.I.T and Mr. Murray 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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**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 in developing suitable methodology and foundation systems for the rehabilitation works proposed at the Belfast Road underpass.

The existing Belfast Road Underpass is a two-span structure supported by a pier and two abutments. The underpass spans a distance of approximately 49 m over the eight-lane Highway 417 and has a deck approximately 12.5 m wide. The pier and abutments are founded on driven steel H-piles driven to bedrock. The pier consists of three columns, each supported on an individual pile cap.

Rehabilitation of the structure will involve the removal and replacement of the existing bridge superstructure (ie., bridge deck and girders) and pier. The new pier will utilize the existing pile foundations, augmented by new foundations to resist seismic uplift/rocking, all incorporated into a single new pile cap. Temporary pier bents will be required to support the existing bridge superstructure during replacement of the existing pier.

The preliminary strategy calls for rapid replacement of the bridge superstructure. A temporary staging area is proposed at St. Laurent Boulevard. Design of temporary foundations in the staging area will be the responsibility of the Contractor.

To accommodate highway widening, existing retaining walls located approximately 3.2 m in front of both abutments will be replaced with new walls constructed approximately 1.5 m closer to the abutments (ie., front face of new wall to be 1.7 m in front of abutment face). The existing retaining walls are concrete cantilever structures, having an approximate overall height of 3.45 m (not including the shear key) and an approximate retained soil height of 1.8 m with a backslope of

about 2H:1V to the abutment face. The new retaining wall will support a retained soil height of approximately 0.75 m higher than the existing walls.

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

- Upgrading of the pier to provide additional seismic/uplift resistance;
- Foundations for the temporary pier bents;
- Replacement retaining walls in front of the abutments;
- Temporary roadway protection works; and
- Temporary foundations in the staging area.

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

## **8 REPLACEMENT/ UPGRADING OF PIER FOUNDATIONS**

The existing pier consists of three columns, each supported on individual pile caps encompassing nine 14BP73 (HP 360x108) steel H-piles. Archive contract drawings indicate that the piles were to be driven “into good shale” to develop a capacity of 60 tons (534 kN) each.

We understand that the existing pier columns and pile caps will be removed and replaced, and the existing piles will be incorporated into the new foundation (extraction and replacement of the piles under the existing bridge deck is impractical). The new pier and foundation will be designed to support the new superstructure geometry and to meet the seismic requirements of the CHBDC including resistance to any potential uplift and/or rocking effects. Where structural analyses suggest seismic uplift to be a concern, anchorage may be provided with permanent anchors or by considering foundation alternatives such as caissons or H-piles socketed into rock.

To facilitate replacement of the pier, the existing bridge superstructure will be carried on temporary support bents provided on either side of the pier.

### **8.1 Axial Capacity of Existing Pile Foundations**

The existing HP 360x108 piles supporting the pier are assumed to be driven to refusal in shale bedrock as specified in the original contract drawings. For existing piles driven to bedrock, it is recommended that the factored axial geotechnical resistance at Ultimate Limit States not exceed 1,950 kN per pile in the design of the new pier. This value is based on the structural capacity of the pile, not the geotechnical resistance in the bedrock.

As the existing piles were not socketed into bedrock, uplift resistance provided by the existing foundations is limited to frictional resistance along the pile shaft within the

approximate 3m thickness of very dense silty sand till. The factored uplift resistance at ULS is computed to be 40 kN per pile based on a factored bond stress of 10 kPa.

## **8.2 New Permanent Pier Foundations**

A comparison of foundation alternatives for the new pier is presented in Appendix C. Recommendations for feasible foundation types are presented below. The preferred option is steel H-piles socketed into rock.

### **8.2.1 Steel H-Piles**

Steel H-piles driven to refusal in bedrock are considered suitable to obtain additional geotechnical resistance to pier loads. However, it is recommended that the piles be socketed into the bedrock to develop uplift resistance. To obtain additional uplift resistance in conjunction with the pile sockets or as an alternative to socketing piles into bedrock, rock anchors could be considered.

Shale bedrock was encountered at depths of 4.4 and 5.5 m below the existing pavement surface (Elev. 58.3 and 57.4 m) in Boreholes BFR-02 and BFR-03, respectively.

For HP 310x110 steel H-piles driven to refusal in bedrock or placed in rock sockets, a factored axial geotechnical resistance at ULS of 2,000 kN is recommended. This value is based on the structural capacity of the pile and no further geotechnical reduction factors need be applied. The SLS condition will not govern for piles founded on/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 in view of the variable quality of the shale. Piles socketed in shale should be installed by drilling/coring to the required depth, inserting the pile, then backfilling around the pile with concrete.

The uplift resistance provided per pile socket should be based on a factored sidewall resistance at ULS of 200 kPa between the socket concrete and weathered shale sidewall. For a 610 mm diameter socket required to install an H-pile, the factored axial resistance (in uplift) at ULS would be 1,150 kN for a 3m long socket and 1,900 kN for a 5 m long socket (all values include a geotechnical resistance factor of 0.3 as per the CHBDC).

The locations of the piles must be selected to avoid interference with the existing pier foundation piles.

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

### **8.2.2 Pile Installation**

Pile installation must be in accordance with OPSS 903.



For piles driven to bedrock, the appropriate pile driving note is “Piles to be driven to bedrock”. The tips of all driven piles must be fitted with cast steel, H-section rock points from an approved manufacturer such as Titus Steel (Standard H-point) or APF Hard Bite or approved equivalent.

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 D.

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 D.

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 D.

### 8.2.3 Caissons

Caissons (drilled shafts) could be considered as an alternative to steel piles at the pier. The caissons should be socketed into bedrock to develop resistance to both foundation loads and uplift.

Caisson socketed into bedrock to develop uplift resistance should be extended at least 3 m below the bedrock surface in view of the variable quality of the shale. Shale bedrock was encountered at depths of 4.4 and 5.5 m below the existing pavement surface (Elev. 58.3 and 57.4 m) in Boreholes BFR-02 and BFR-03, respectively.

The factored axial geotechnical resistances at ULS recommended for typical caisson designs socketed 3 and 5 m into shale 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.

The SLS condition will not govern for caissons founded in bedrock.

**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	3,000	1,500
	1.2	4,500	2,000
	1.5	6,300	2,500
5	0.9	4,000	2,500
	1.2	6,000	3,300
	1.5	8,000	4,200

The caissons must be advanced through the overburden and into the bedrock and must be advanced using a steel liner to support the walls and reduce the risk of material falling in from the sides of the hole. It will be necessary to advance the liner to the top of the bedrock.

The method of installation can be selected by the contractor, provided it meets the requirements of the foundation design.

### **8.3 Permanent Anchors**

Rock anchors may be used to provide uplift resistance as an alternative to socketing piles into bedrock or to obtain additional uplift resistance in conjunction with pile sockets.

The length of the unbonded zone below the underside of the footing 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.

The factored rock-grout bond strength at ULS recommended for design of the anchors within shale bedrock is 200 kPa. This value includes a geotechnical resistance factor of 0.4 as per Table 6.1 of the CHBDC.

Each production anchor must be proof tested as per Special Provision 999S26 to confirm that the required resistance against uplift load is achieved.

The rock anchors should be provided with double corrosion protection.

The drilling of holes for installation of anchors will encounter pavement materials, silty sand fill, and very dense silty sand till with cobbles and shale slabs. The Contractor's drilling equipment must be able to dislodge, remove or penetrate any cobbles, boulders or shale encountered in the till.

Hard limestone interbeds will be encountered while advancing the holes within the shale bedrock. The Contractor's drilling equipment must be able to penetrate the sound bedrock and hard interbeds to achieve the design bond length.

## 8.4 Lateral Resistance

Piles supporting the existing pier were driven with a batter of 1H:6V to resist lateral loads. If additional resistance to lateral loading is required, the new piles should be installed as batter piles. In view of the short length of the batter piles founded on unyielding bedrock as well as the number of batter piles relative to the total number of piles within the pier foundation, it is considered unlikely that the lateral movement of the pier will be sufficient to permit development of passive resistance on the face of piles within till or bedrock. Therefore passive resistance on the face of vertical and batter piles should be neglected in design, and the horizontal resistance derived exclusively from the batter.

## 8.5 Temporary Pier Foundations

Temporary support bents will be provided on either side of the existing pier to support the existing bridge superstructure during replacement of the pier in advance of superstructure replacement. It is proposed to support the temporary bents on spread footings founded at the same level as the underside of the existing pile cap (Elev. 60.96 m).

Supporting the temporary pier bents on spread footings is considered feasible at this site. The footings should be founded on very dense native silty sandy till below the level of all fill and disturbed material.

The upper boundary of the native till was encountered at depths of 1.5 and 2.3 m (Elev. 61.2 and 60.6 m) in Boreholes BFR-02 and BFR-03, respectively. Excavation to approximately 0.5 m below the underside of the existing pile cap will therefore be required at the location of BFR-03.

Spread footings bearing on the native very dense silty sand till at Elev. 60.96 m (Borehole BFR-02) to Elev. 60.6 m (Borehole BFR-03) may be designed on the basis of the following geotechnical resistances. These values include a resistance factor of 0.5 as per Table 6.1 of the CHBDC.

**Table 8.2 – Recommended Geotechnical Resistances for Spread Footings**

	1.5m Wide Footing	2.5m Wide Footing
Factored Geotechnical Resistance at ULS	450 kPa	600 kPa
Geotechnical Resistance at SLS	350 kPa	250 kPa

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 geotechnical SLS resistance values given above are based on an estimated total settlement not exceeding 10 mm.

The sliding resistance of concrete poured on the native silty sand till may be computed on the basis of a coefficient of friction of 0.55. This is an ultimate value and requires a degree of sliding movement to occur to fully mobilize the resistance. As per the CHBDC, a resistance factor of 0.8 must be applied to the sliding resistance computed using this coefficient; this factor is not included in the noted value.

Excavation and backfilling of foundations must be carried out in accordance with OPSS 902. The base of the foundation excavations must be inspected by qualified personnel to confirm that the exposed surface conforms to the design requirements and has been adequately prepared to receive concrete. Where subexcavation is required to remove unsuitable material from below the design founding level, the founding surface should be re-established using concrete of the same class as used in the footing.

### **8.6 Frost Cover**

The design depth of frost penetration at this site is 1.8 m. It is recommended that pile caps and footings be provided with a minimum of 1.8 m of earth cover above the underside of the footing or pile cap.

## **9 RETAINING WALL REPLACEMENT**

To accommodate highway widening, the existing retaining walls located approximately 3.2 m in front of both abutments will be replaced with new walls constructed approximately 1.5 m closer to the abutments (ie., front face of new wall to be 1.7 m in front of abutment face). The existing retaining walls are concrete cantilever structures, having an approximate overall height of 3.45 m (not including the shear key) and an approximate retained soil height of 1.8 m with a backslope of about 2H:1V to the abutment face. The new retaining wall will support a retained soil height of approximately 0.75 m higher than the existing walls.

The bridge abutments are founded on steel H-piles driven to bedrock. The underside of the pile cap is at Elev. 64.0 m, and the road grade at the wall is near Elev. 62.5 m, indicating that the pile cap is perched approximately 1.5 m above the grade along Highway 417.

Design and construction of the new retaining wall must take into consideration the constraints specific to this site, such as the close proximity to the existing abutment, the presence of the perched pile caps, and the limited overhead clearance to the bridge deck as traffic must be maintained on Belfast Road. The following design concepts were considered:

- Installation of a soldier pile and lagging system is considered feasible provided low clearance augering equipment is used to install the piles. Installation of driven piles is not practical due to the low overhead clearance available for pile installation equipment.
- Construction of a concrete retaining wall supported on new spread footings, potentially incorporating the existing retaining wall footings, may be considered provided a shoring

system is installed to enable excavation without undermining the existing pile cap. The shoring system could incorporate lagging using the existing abutment foundation piles along with possible soil or rock anchors.

- RSS walls are not feasible due to the limited space to the abutment.

With either wall concept, a lagging system will be required to enable excavation for partial or complete removal of the existing retaining wall without undermining the pile cap.

Design of the retaining wall must address overturning, sliding and bearing resistance. Global stability is not considered to be an issue at this site.

### 9.1 Augered Soldier Piles and Lagging

Installation of a soldier pile and lagging system is considered feasible provided low clearance drilling equipment is available that can penetrate very dense silty sand till with cobbles, boulders and shale slabs, as well as shale bedrock with limestone layers. Penetration of the existing retaining wall footing will also be required.

The horizontal loads imposed on the wall will be resisted by passive forces developed on the face of the soldier piles within the native till and shale below the level of existing foundation backfill. The design founding level of the existing wall footing was approximately Elev. 60.7 to 61.0 m. The pile tip elevations will be governed by the embedment depth required to resist the horizontal loads.

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

Silty Sand Till (Elev. 60.7 to 57.6 m):

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

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

where	$z$	=	depth of embedment of pile in metres
	$D$	=	pile augerhole diameter in metres
	$n_h$	=	coefficient related to soil density
		=	9,000 kN/m <sup>3</sup>
	$\gamma$	=	bulk unit weight
		=	21 kN/m <sup>3</sup> (above the water table)
		=	11 kN/m <sup>3</sup> (submerged unit weight below water table)
	$K_p$	=	passive earth pressure coefficient
		=	3.7

Weathered Shale (below Elev. 57.6m):

$$k_s = 25,000 \text{ kN/m}^3 \text{ at the bedrock surface, increasing linearly to } 50,000 \text{ kN/m}^3 \text{ at a depth of 3 caisson diameters and below}$$

$$p_{ult} = 400 \text{ kPa at the bedrock surface, increasing linearly to } 2,000 \text{ kPa at a depth of 3 caisson diameters and below}$$

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

The spring constant,  $K$ , for analysis may be obtained by the expression,  $K = k_s \times L \times D$  (kN/m), where  $k_s$  is the coefficient of horizontal subgrade reaction (kN/m<sup>3</sup>),  $D$  is the pile width (m) and  $L$  is the length (m) of the pile segment or element used in the analysis. The ultimate lateral resistance,  $P_{ult}$ , may be obtained from the expression,  $P_{ult} = p_{ult} \times L \times D$ . This represents the ultimate load at which the soil fails and will not support any additional load at greater displacements.

The modulus of subgrade reaction may have to be reduced, based on the pile spacing. The reduction factors to be used for a pile group oriented perpendicular or parallel to the direction of loading are provided in the following table. Intermediate values may be obtained by linear interpolation.

**Table 9.1 - Subgrade Reaction Reduction Factors for Pile/Caisson Spacing**

Condition	Pile Spacing, Centre to Centre*	Reduction Factor
Pile group oriented <i>perpendicular</i> to direction of loading	4D	1.0
	1D	0.5
Pile group oriented <i>parallel</i> to direction of loading	8D	1.0
	6D	0.7
	4D	0.4
	3D	0.25

\* where  $D$  is the width of pile augerhole

The locations of the soldier piles must be selected to avoid interference with the existing abutment foundation piles.

As noted above, low clearance drilling equipment will be required to install soldier piles under the existing bridge superstructure. The equipment selected must be capable of drilling through very dense till deposits containing cobbles, boulders and shale slabs, and advancing into shale bedrock with hard limestone layers. The Contract Documents

should contain a NSSP alerting the Bidders to these requirements. Suggested texts for NSSP's are included in Appendix D.

## 9.2 Spread Footings

Provided a shoring system is installed to prevent loss of soil from under the existing pile cap during excavation, supporting the new wall on spread footings (new and/or existing) is considered feasible at this site. The footings should be founded on very dense native silty sand till below the level of all fill and disturbed material.

The design founding level of the existing wall footing was approximately Elev. 60.7 to 61.0 m. New footings bearing on the native very dense silty sand till at this level may be designed on the basis of the following geotechnical resistances. These values include a resistance factor of 0.5 as per Table 6.1 of the CHBDC.

**Table 9.2 – Recommended Geotechnical Resistances for Spread Footings**

	2m Wide Footing	3m Wide Footing
Factored Geotechnical Resistance at ULS	400 kPa	500 kPa
Geotechnical Resistance at SLS	325 kPa	300 kPa

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 geotechnical SLS resistance values given above are based on an estimated total settlement not exceeding 25 mm.

The sliding resistance of concrete poured on the native silty sand till may be computed on the basis of a coefficient of friction of 0.55. This is an ultimate value and requires a degree of sliding movement to occur to fully mobilize the resistance. As per the CHBDC, a resistance factor of 0.8 must be applied to the sliding resistance computed using this coefficient; this factor is not included in the noted value.

Excavation and backfilling of foundations must be carried out in accordance with OPSS 902. The base of the foundation excavations must be inspected by qualified personnel to confirm that the exposed surface conforms to the design requirements and has been adequately prepared to receive concrete. Where subexcavation is required to remove unsuitable material from below the design founding level, the founding surface should be re-established using concrete of the same class as used in the footing.

### 9.3 Impact on Abutment Piles

Construction of the new retaining wall will involve temporary removal of the soil in front of the abutment piles.

In general, the proposed excavation is not expected to impact the performance of the existing pile foundations. The front row of abutment piles were installed on a batter of 1H:6V, and resistance to horizontal loads on the abutment will primarily be developed by the horizontal component of the axial load in the pile driven to bedrock.

As noted previously for the pier foundations, the existing HP 360x108 piles supporting the abutment are assumed to be driven to refusal in shale bedrock. It is recommended that the factored axial geotechnical resistance at Ultimate Limit States not exceed 1,950 kN per pile.

Additional lateral resistance may be provided by the passive forces developed on the face of the piles within the native till and shale. The lateral resistance may be calculated using values for the coefficient of horizontal subgrade reaction ( $k_s$ ) and ultimate lateral resistance ( $p_{ult}$ ) computed as outlined in Section 9.1. For the leading row of piles, the lateral resistance above the base of the excavation should be taken as zero. For the second and succeeding rows of piles, the subgrade reaction values should be reduced as per Table 9.1, and the following parameters are recommended for the section of pile above the excavation base:

Silt/Silty Sand Till (Elev. 64.0 to 62.0 m):

$$\begin{aligned} n_h &= 2,500 \text{ kN/m}^3 \\ \gamma &= 20 \text{ kN/m}^3 \\ K_p &= 2.9 \end{aligned}$$

Silty Sand Till (Elev. 62.0 to 60.7 m):

$$\begin{aligned} n_h &= 5,000 \text{ kN/m}^3 \\ \gamma &= 21 \text{ kN/m}^3 \\ K_p &= 3.3 \end{aligned}$$

The length of pile exposed by excavation should also be assessed for buckling. If required, anchors may be considered to provide lateral support for the exposed section of the existing piles.

The anchors should be developed within the very dense silty sand till below Elevation 60m or in shale bedrock. Installation of anchors in the fill, loose silt, or loose to compact upper zone of the silty sand till is not recommended due to the low bond strength in these deposits and the potential for long term creep of the anchors.

The factored soil-grout bond strength at ULS recommended for design of anchors within the very dense silty sand till is 75 kPa. The factored rock-grout bond strength at ULS recommended for design of the anchors in shale bedrock is 200 kPa. These values include a geotechnical resistance factor of 0.4 as per Table 6.1 of the CHBDC.



The length of the unbonded (free-stressing) zone should be at least 3.0 m for a steel bar anchor and 4.5 m for a steel strand anchor. The unbonded length should extend at least 1.5 m beyond the back row of abutment stem piles. 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.

In view of the depth to competent anchorage material behind the abutments and the resulting potential that steeply inclined anchors would apply additional axial load on the existing piles, installation of soil or rock anchors through the abutment or lagging system may not be practical. Use of deadman anchors or other means such as temporary unloading of the walls to eliminate the need for abutment anchors should be considered.

The above recommendations are provided to estimate the anchor capacity for design purposes only. It is necessary that selected anchors be performance tested and all remaining production anchors on site be proof tested to confirm their carrying capacities. Anchor testing and other relevant anchor installation details should be in accordance with Special Provision 999S26. To reduce the secondary load effects on the piles, proof testing should be limited to 1.0 times the design load.

## 10 WALL BACKFILL AND LATERAL EARTH PRESSURES

Backfill to the retaining wall, and existing abutment walls where required, should be in accordance with OPSS 902. Granular backfill should be placed to the extents shown in OPSD 3121.150 and 3101.150. The design of the retaining wall must include a subdrain as shown in OPSD 3190.100.

All granular material should meet the specifications of OPSS 1010 as amended by Special Provision 110S13. Compaction equipment to be used adjacent to retaining structures should be restricted in accordance with OPSS 501.

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 10.1)

$\gamma$  = unit weight of retained soil (see Table 10.1)

$h$  = 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 given in Table 10.1.

**Table 10.1 – Earth Pressure Coefficients (K)**

Condition	Earth Pressure Coefficient (K)					
	OPSS Granular A or Granular B Type II $\phi = 35^\circ, \gamma = 22.8 \text{ kN/m}^3$		OPSS Granular B Type I $\phi = 32^\circ, \gamma = 21.2 \text{ kN/m}^3$		Silt Fill/Silt/Till (for roadway protection) $\phi = 30^\circ, \gamma = 21.0 \text{ kN/m}^3$	
	Horizontal Surface Behind Wall	Sloping Backfill (2H:1V)	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*	0.33	0.54*
At Rest (Restrained Wall)	0.43	-	0.47	-	0.50	-
Passive	3.7	-	3.3	-	3.0	-

\* For wing walls.

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. In the case of integral abutments, material with a lower passive pressure coefficient (e.g. Granular B, Type I) might be preferred as it results in lower forces acting on the ballast wall as the soil moves towards the soil mass.

Archive drawings for the structure indicate that sand (“sand cushion”) was used to backfill behind the abutments to the pile cap level (Elev. 64.0 m). The lower boundary of the sand fill encountered in the boreholes (Boreholes BFR-01 and BFR-04) is at Elev. 64.4 and 64.3 m, consistent with the archive drawing. For assessment of lateral earth pressures acting on the existing abutment stem and wing walls that will not receive new backfill, the parameters presented in Table 10.1 for Granular B Type I are recommended for the sand backfill to the level of the pile cap. Below this depth, the parameters shown in the table for silt fill, native silt and silty sand till are recommended.

The factors in Table 10.1 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 at a depth of 1.7 m for Granular A or Granular B Type II.

## **11 EXCAVATION AND GROUNDWATER CONTROL**

Excavation for structure rehabilitation is expected to be limited to the existing pavement materials, fill and native silt/sand above the underside of the existing pile caps and footings, except where replacement of the retaining walls in front of the abutments may require excavation to greater depth.

All excavations must be carried out in accordance with the Occupational Health and Safety Act (OHSA). For the purposes of the OHSA, the fill and native soils within the probable depth of excavation may be classed as Type 3 soils.

Excavation in front of the abutments for replacement of the retaining wall must be carried out in a manner that avoids undermining of the existing pile cap and abutment backfill. This may involve restricting excavation to above a line inclined downward at 1H:1V from the forward face of the abutment underside. Alternatively, installing lagging, using the existing pile foundations as soldier piles, may be considered. Grouting should be specified to backfill any void that may develop below the pile cap during excavation.

The excavation and backfilling for foundations must be carried out in accordance with OPSS 902.

The Contractor should be prepared to pump from sumps to remove any seepage water or surface water collecting in an excavation.

Use of a steel liner is recommended to advance pile sockets 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.

## **12 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.

Based on the information obtained in Boreholes BFR-01 and BFR-04, roadway protection along Belfast Road will be installed within existing pavement materials, dense to compact sand fill, and loose to compact silt/silty sand till. Potential shoring systems may consist of steel sheet piles or H-piles with timber lagging.

Roadway protection along Highway 417 at the pier and at the abutments if required will encounter the existing pavement structure and silty sand fill, overlying very dense silty sand till and shale. Installation of driven H-piles or sheeting is not considered to be feasible in view of the low overhead clearance, the very dense conditions, and shallow depth to bedrock. Use of a soldier

pile and lagging system with H-piles installed in augered holes may be considered. A braced excavation or system of rakers may also be considered.

Parameters for design of the roadway protection systems are provided in Sections 9 and 10. Selection and design of the roadway protection system and any dewatering system that may be required are the responsibility of the Contractor.

### 13 SEISMIC CONSIDERATIONS

The following seismic parameters should be used for design:

- 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 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 10.1)
- $\Delta K_E$  = seismic earth pressure coefficient (see Table 13.1)
- $\gamma$  = unit weight of retained soil (see Table 13.1)
- $h$  = 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 ( $\Delta K_E$ ) recommended for determining the seismic component are presented in Table 13.1.

**Table 13.1 – Earth Pressure Coefficients for Earthquake Loading**

Condition	Seismic Earth Pressure Coefficient ( $\Delta K_E$ )				
	OPSS Granular A or OPSS Granular B Type II $\phi = 35^\circ, \gamma = 22.8 \text{ kN/m}^3$		OPSS Granular B Type I $\phi = 32^\circ, \gamma = 21.2 \text{ kN/m}^3$		Silt Fill/Silt/ Sand Till $\phi = 30^\circ$ $\gamma = 21 \text{ kN/m}^3$
	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)	Horizontal Surface Behind Wall
Active ( $\Delta K_{AE}$ )*	0.07	0.22	0.07	0.23	0.08
At Rest ( $\Delta K_{OE}$ )**	0.21	-	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 site are not in danger of liquefaction under earthquake loading.

## 14 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 spread footings or short augered caissons founded in bedrock encountered at depths of 1.9 to 3.4 m below the ground surface.

Spread footings or short augered caissons founded at the bedrock surface should be designed using a factored geotechnical resistance of 800 kPa at ULS. This value includes a resistance factor of 0.5 as per Table 6.1 of the CHBDC. The SLS condition noted above will not govern design of footings/ caissons founded on bedrock.

If higher resistance values are required, the caissons should be socketed at least 2 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 2 m into shale bedrock are summarised in Table 14.1. The SLS conditions noted above will not govern for caissons founded in bedrock.

The resistance values are for vertical, concentric loads only. In accordance with the CHBDC Clauses 6.7.3 and 6.7.4, the design must also account for the effects of any eccentric or inclined loads applied.

**Table 14.1- Axial Geotechnical Resistance of Caissons**

<b>Socket Length in Shale (m)</b>	<b>Caisson Diameter (m)</b>	<b>Factored Axial Resistance at ULS (kN)</b>
2	0.9	2,000
	1.2	3,000
	1.5	4,000

The sliding resistance of concrete poured on shale bedrock may be computed on the basis of a coefficient of friction of 0.55. This is an ultimate value and requires a degree of sliding movement to occur to fully mobilize the resistance. As per the CHBDC, a resistance factor of 0.8 must be applied to the sliding resistance computed using this coefficient; this factor is not included in the noted value.

Excavation and caisson installation will generally be advanced through the surficial fill and firm to very stiff silty clay potentially containing shale slabs. The equipment supplied by the 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 caissons may 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.

Concrete should be placed over the founding surfaces within 8 hours of excavation to minimize softening of the exposed shale bedrock.

## **15 CONSTRUCTION CONCERNS**

Potential construction concerns include, but are not necessarily limited to:

- The very dense 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 augered soldier piles must be capable of penetrating these materials.
- As much of the rehabilitation work will be carried out below the existing bridge deck, low clearance equipment will be required to install soldier piles and carry out excavations.
- 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.
- Excavation and pile installation may encounter existing foundations or other obstructions. Further, existing foundation conditions may vary from those anticipated. The design of the proposed structure modifications may need to be reviewed when the as-built foundation conditions are established during construction.

- Use of a steel liner is recommended to advance pile sockets to the bedrock surface to support the sidewalls, minimize groundwater inflow, and enable machine-cleaning of the socket base.

Implementation of a monitoring program to identify potential movement of the existing superstructure during construction of the temporary piers and the new permanent pier should be considered. The monitoring program and tolerance criteria must be established by the structural designer, but as a minimum should include monitoring of both vertical and lateral movements of the existing superstructure. The monitoring should include baseline readings, and readings during and after foundation construction works. The monitoring program should be conducted by a monitoring sub-consultant retained by the contract administrator.

## 16 CLOSURE

Engineering analysis and preparation of the foundation design report were carried out by Mr. Murray 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.

Murray R. Anderson, P.Eng., M.Eng.  
Senior Foundations Engineer



P.K. Chatterji, P.Eng., Ph.D.  
Review Principal



## **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 <sup>(1)</sup> '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

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

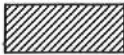
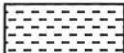



 Water Level  
 Shear Strength Determination by Pocket Penetrometer

- (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 (MPa) (psi)		Field Estimation of Hardness*
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				Breaks under single blow of geological hammer.
Thinly Laminated	Less than 6mm	Medium Strong	25.0 to 50.0	3,500 to 7,500	Can be peeled by a pocket knife with difficulty
TERMS		Weak	5.0 to 25.0	750 to 3,500	Can be peeled by a pocket knife, crumbles under firm blows of geological pick.
Total Core Recovery: (TCR)	Core recovered as a percentage of total core run length.	Very Weak	1.0 to 5.0	150 to 750	Indented by thumbnail
Solid Core Recovery: (SCR)	Percent Ratio of solid core of full cylindrical shape recovered. Expressed with respect to the total length of core run.	Extremely Weak (Rock)	0.25 to 1.0	35 to 150	
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 BFR-01

1 OF 2

METRIC



W.P. 4320-06-00 LOCATION N 5 031 442 8 E 371 542 6 Belfast Rd ORIGINATED BY LPG/GA  
HWY 417 BOREHOLE TYPE Hollow Stem Augers COMPILED BY AN  
DATUM Geodetic DATE 2011 07.16 - 2011 07.16 CHECKED BY LRB

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				UNIT WEIGHT γ kN/m <sup>3</sup>	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		
69.0													
0.0	ASPHALT: (100mm)												
0.1													
68.2	SAND, some gravel, some silt Brown Moist (FILL)		1	AS									
0.8													
	SAND, some silt, trace gravel, trace clay Dense to Compact Brown to Grey Moist (FILL)		2	SS	47								
			3	SS	21								7 74 15 4
			4	SS	17								
			5	SS	11								
64.4													
4.6	SILT, some sand, trace clay Loose Green-Grey Moist		6	SS	7								
62.9													
6.1	Silty SAND, some gravel, trace to some clay Loose to Very Dense Grey Damp to Moist (TILL)		7	SS	4								18 44 26 12
			8	SS	26								21 48 28 3
			9	SS	52								

Continued Next Page

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

## METRIC

SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 		UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE						
	Continued From Previous Page						SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100	WATER CONTENT (%) 20 40 60	$\gamma$ kN/m <sup>3</sup>	GR SA SI

[illegible]

+ 3 × 3: Numbers refer to Sensitivity

# RECORD OF BOREHOLE No BFR-02

1 OF 1

METRIC

W.P. 4320-06-00 LOCATION N 5 031 405.7 E 371 539.7 Belfast Rd. ORIGINATED BY LPG  
 HWY 417 BOREHOLE TYPE Hollow Stem Augers/NQ Coring COMPILED BY AN  
 DATUM Geodetic DATE 2011 07 24 - 2011 07 24 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 <sup>3</sup>	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 <sub>P</sub> w      w <sub>L</sub>									
62.7							20	40	60	80	100		20	40	60		GR	SA	SI	CL
0.0	ASPHALT: (200mm)																			
0.2	Gravelly SAND, some silt Brown Moist (FILL)		1	AS																
61.9																				
0.8	Silty SAND, trace gravel Very Dense Brown Moist (FILL)		2	SS	53															
61.2																				
1.5	Silty SAND, trace to some gravel, trace clay Very Dense Brown Moist (TILL)		3	SS	61															
			4	SS	50/ 0.075															
	Extremely difficult augering Occasional cobbles		5	SS	50/ 0.075															
			1	RUN																
58.3																				
4.4	SHALE, moderately weathered, sub-horizontal laminations, grey																			
	50mm thick limestone interbed at 5.2m 25mm thick limestone interbed at 5.3m		2	RUN																
	Clay infill in fractures		3	RUN																
55.4																				
7.3	END OF BOREHOLE AT 7.3m. BOREHOLE BACKFILLED WITH CUTTINGS AND BENTONITE HOLEPLUG TO 0.15m, THEN ASPHALT TO SURFACE.																			

ONTMT4S 1201B GPJ 12/15/11

+ 3, x 3: Numbers refer to  
Sensitivity

20  
15  
10

(%) STRAIN AT FAILURE

# RECORD OF BOREHOLE No BFR-03

1 OF 1

METRIC

W.P. 4320-06-00 LOCATION N 5 031 413.4 E 371 559.6 Belfast Rd. ORIGINATED BY LPG  
HWY 417 BOREHOLE TYPE Hollow Stem Augers/NQ Coring COMPILED BY AN  
DATUM Geodetic DATE 2011.07.24 - 2011.07.24 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 <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa ○ UNCONFINED      + FIELD VANE ● QUICK TRIAXIAL    x LAB VANE			WATER CONTENT (%) w <sub>p</sub> w      w <sub>L</sub>				
62.9								20	40	60	80	100			
0.0	ASPHALT: (200mm)														
0.2	Gravelly SAND, some silt Brown Moist (FILL)		1	AS											
62.1															
0.8	Silty SAND, some gravel, trace clay Compact Brown Moist (FILL)		2	SS	17										
			3	SS	22										16 56 23 5
60.6															
2.3	Silty SAND, trace to some gravel, trace clay Very Dense Dark Grey Moist to Wet (TILL)		4	SS	59/ 0.275										
			5	SS	50/ 0.075										
	Difficult augering from 3.6m														
			6	SS	50/ 0.125										
	Auger refusal at 5.1m														
57.4			1	RUN										FI	RUN #1 TCR=68% SCR=39% RQD=18%
5.5	SHALE, fresh, laminated, grey, limestone interbeds less than 10mm thick through out													2	
	Fraclured from 6.1m to 6.2m and 6.6m to 6.7m		2	RUN										>5	RUN #2 TCR=100% SCR=82% RQD=78%
														>5	
														1	
														0	
														0	
														0	
			3	RUN										0	RUN #3 TCR=100% SCR=100% RQD=100%
54.3														0	
8.5	END OF BOREHOLE AT 8.5m. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 3.0m slotted screen.  WATER LEVEL READINGS: DATE      DEPTH (m)      ELEV. (m) July26/ 11      3.5      59.4 Aug.18/ 11      3.5      59.4 Sep.20/ 11      3.8      59.1 Oct.12/ 11      3.8      59.1														

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

ONTMT4S 1201B.GPJ 12/15/11

## METRIC

[illegible]

+ 3, X 3: Numbers refer to Sensitivity



# RECORD OF BOREHOLE No BFR-04

2 OF 2

METRIC

W.P. 4320-06-00 LOCATION N 5 031 381.0 E 371 549.6 Belfast Rd. ORIGINATED BY GA  
 HWY 417 BOREHOLE TYPE Hollow Stem Augers/MQ Coring COMPILED BY AN  
 DATUM Geodetic DATE 2011.11.16 - 2011.11.16 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 <sup>3</sup>	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	20   40   60   80   100	20   40   60	w <sub>p</sub> w   w <sub>L</sub>				
	Continued From Previous Page														
56.8															
11.9	<b>SHALE</b> , slightly weathered to fresh, thinly bedded, grey, occasional limestone interbeds  Horizontal joint at 11.9m   <														

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

## METRIC

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 <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI C												
ELEV. (m)	DEPTH (m)	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE			"N" VALUES	20 40 60 80 100	20 40 60	w <sub>p</sub> w w <sub>L</sub>															
62.8	0.0	ASPHALT: (200mm)																								
62.0	0.2	Gravelly SAND, some silt Brown Moist (FILL)		1	AS																					
62.0	0.8	Silty SAND, some gravel, trace clay, occasional cobbles Dense Brown Moist (FILL)		2	SS	36																				
60.6	2.2	Silty SAND, trace gravel Very Dense Dark Grey Moist to Wet (TILL)		3	SS	47																				
				4	SS	84																				
				5	SS	50/ 0.075																				
				6	SS	50/ 0.025																				
57.4	5.4	SHALE, slightly weathered		7	SS	50/ 0.025																				
<p>END OF BOREHOLE AT 5.4m. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen.</p> <p>WATER LEVEL READINGS:</p> <table border="1"> <thead> <tr> <th>DATE</th> <th>DEPTH (m)</th> <th>ELEV. (m)</th> </tr> </thead> <tbody> <tr> <td>July 26/ 11</td> <td>2.9</td> <td>59.9</td> </tr> <tr> <td>Aug. 18/ 11</td> <td>3.0</td> <td>59.8</td> </tr> <tr> <td>Oct. 12/ 11</td> <td>3.2</td> <td>59.6</td> </tr> </tbody> </table>															DATE	DEPTH (m)	ELEV. (m)	July 26/ 11	2.9	59.9	Aug. 18/ 11	3.0	59.8	Oct. 12/ 11	3.2	59.6
DATE	DEPTH (m)	ELEV. (m)																								
July 26/ 11	2.9	59.9																								
Aug. 18/ 11	3.0	59.8																								
Oct. 12/ 11	3.2	59.6																								

ONTMT4S 1201B.GPJ 12/15/11

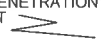
+ 3, X 3 Numbers refer to Sensitivity

RECORD OF BOREHOLE No 23S-02

1 OF 1

METRIC

W.P. 4320-06-00 LOCATION N 5 031 390.0 E 371 540.9 Belfast Rd. Retaining Wall South ORIGINATED BY LPG  
HWY 417 BOREHOLE TYPE Hollow Stem Augers/NQ Coring COMPILED BY AN  
DATUM Geodetic DATE 2011.07.22 - 2011.07.22 CHECKED BY LRB

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa				
62.5							20 40 60 80 100					
0.0	ASPHALT: (200mm)							○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB VANE				
0.2	SAND, some gravel, some silt Brown Moist (FILL)		1	AS								
61.7												
0.8	Silly SAND, some gravel, trace clay Compact Brown Moist (FILL)		2	SS	16							
61.1												
1.4	Silly SAND, some gravel, trace clay Very Dense Dark Grey Moist (TILL)		3	SS	50/ 0.05							
			4	SS	62							
			5	SS	50/ 0.025							
	Difficult augering at 4.1m, possible bedrock											
57.9												
4.6	SHALE, slightly weathered, laminated, very thin limestone interbeds through out, grey Highly fractured, occasional calcite infilling, sub-horizontal fractures		1	RUN								
	25mm thick limestone interbeds at 6.0m, 6.2m and 6.3m		2	RUN								
55.2												
7.3	END OF BOREHOLE AT 7.3m. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 3.0m slotted screen.											
	WATER LEVEL READINGS: DATE DEPTH (m) ELEV. (m) July 26/ 11 2.8 59.7 Aug. 18/ 11 2.8 59.7 Oct. 12/ 11 3.9 58.6											

+ 3, x 3: Numbers refer to  
Sensitivity

20  
15  
10

(%) STRAIN AT FAILURE

# RECORD OF BOREHOLE No BFS-1

1 OF 2

METRIC

W.P. 4320-06-00 LOCATION N 5 031 497.4 E 370 867.4 Belfast Road Staging Area ORIGINATED BY LPG  
 HWY 417 BOREHOLE TYPE Hollow Stem Augers/NQ Coring - CME 75 COMPILED BY AN  
 DATUM Geodetic DATE 2011.08.10 - 2011.08.10 CHECKED BY LRB

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	
61.4	ASPHALT: (50mm)											
60.6	GRAVEL Brown Moist (FILL)		1	AS			61					
59.9	Sandy GRAVEL Very Dense Brown Moist (FILL)		2	SS	61		60					
1.4	Silty SAND, trace clay, trace gravel Loose to Very Dense Grey to Dark Grey Moist (TILL)		3	SS	9		59					
			4	SS	5		58					6 55 32 7
			5	SS	36		57					
	Difficult augering from 3.7m to 5.5m		6	SS	82/ 0.280		56					
			7	SS	54		55					
	Difficult augering from 6.7m		8	SS	47		54					
53.0	SHAPE, slightly weathered, grey, very thin limestone interbeds through out		1	RUN			53					8 59 28 5
8.4			2	RUN			52					

Continued Next Page

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

METRIC

W.P.	4320-06-00	LOCATION	N 5 031 497.4 E 370 867.4 Belfast Road Staging Area	ORIGINATED BY	LPG
HWY	417	BOREHOLE TYPE	Hollow Stem Augers/NQ Coring - CME 75	COMPILED BY	AN
DATUM	Geodetic	DATE	2011.08.10 - 2011.08.10	CHECKED BY	LRB

[illegible]

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity

# RECORD OF BOREHOLE No BFS-2

1 OF 1

METRIC

W.P. 4320-06-00 LOCATION N 5 031 527.1 E 370 919.1 Belfast Road Staging Area ORIGINATED BY LPG  
HWY 417 BOREHOLE TYPE Hollow Stem Augers/NQ Coring - CME 75 COMPILED BY AN  
DATUM Geodetic DATE 2011.08.10 - 2011.08.10 CHECKED BY LRB

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT   NATURAL LIMIT   MOISTURE   CONTENT   LIQUID LIMIT			UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR   SA   SI   CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa				WATER CONTENT (%)				
61.7								20	40	60	80	100				
0.0	ASPHALT: (150mm)															
0.2	Sandy GRAVEL Grey Moist (FILL)		1	AS												
60.9																
0.8	Silty SAND, trace to some clay, trace gravel Compact to Very Dense Grey and Brown Moist (TILL)		2	SS	25											
			3	SS	37											4   46   38   12
			4	SS	34											
			5	SS	62/ 0.225											
			6	SS	24											7   54   31   8
			7	SS	50/ 0.125											
54.0	Shale and limestone fragments at 7.6m															
7.6	END OF BOREHOLE AT 7.6m. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 3.0m slotted screen.		8	SG	50/ 0.0											
	WATER LEVEL READINGS: DATE      DEPTH (m)      ELEV. (m) Sep.02/ 11      5.6      56.1 Sep.20/ 11      5.5      56.2															

ONTMT4S 1201B.GPJ 10/6/11

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity 15-25 (%) STRAIN AT FAILURE

# RECORD OF BOREHOLE No BFS-3

1 OF 2

METRIC

W.P. 4320-06-00 LOCATION N 5 031 470.3 E 370 938.3 Belfast Road Staging Area ORIGINATED BY LPG  
 HWY 417 BOREHOLE TYPE Hollow Stem Augers/NQ Coring - CME 75 COMPILED BY AN  
 DATUM Geodetic DATE 2011.08.10 - 2011.08.10 CHECKED BY LRB

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								WATER CONTENT (%)	
								○ UNCONFINED	+ FIELD VANE	● QUICK TRIAXIAL							× LAB VANE
								20	40	60							80
61.7																	
0.0	ASPHALT: (50mm)																
	Sandy GRAVEL Grey Moist (FILL)		1	AS													
60.9																	
0.8	Silty SAND, trace clay, trace gravel Compact to Very Dense Brown to Dark Grey Moist (TILL)		2	SS	20												
			3	SS	32												
			4	SS	45												
			5	SS	31												
			6	SS	90/ 0.200												
55.6																	
6.1	SHALE, slightly weathered to fresh, laminated, grey, very thin limestone interbeds through out		1	RUN													
			2	RUN													
			3	RUN													
52.2	Clay infill (50mm) at 9.3m																
9.4	END OF BOREHOLE AT 9.4m. BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG AND																

Continued Next Page

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to  
Sensitivity

20  
15  
10

(%) STRAIN AT FAILURE

# RECORD OF BOREHOLE No BFS-3

2 OF 2

METRIC

W.P. 4320-06-00 LOCATION N 5 031 470.3 E 370 938.3 Belfast Road Staging Area ORIGINATED BY LPG  
 HWY 417 BOREHOLE TYPE Hollow Stem Augers/NQ Coring - CME 75 COMPILED BY AN  
 DATUM Geodetic DATE 2011.08.10 - 2011.08.10 CHECKED BY LRB

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	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					
	CUTTINGS TO 0.1m, THEN ASPHALT COLD PATCH TO SURFACE.																



## METRIC

+ 3, × 3: Numbers refer to Sensitivity

RECORD OF BOREHOLE No BRS-2

1 of 1

METRIC

W.P. 4320-06-00 LOCATION N 5 031 442.9 E 372 629.7 Belfast Rd. Staging Area at St. Laurent Blvd. ORIGINATED BY RK  
HWY 417 BOREHOLE TYPE Hollow Stem Augers/Coring COMPILED BY AN  
DATUM Geodetic DATE 2011.11.24 - 2011.11.24 CHECKED BY LRB

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								
68.9								20	40	60	80	100				
0.0 0.1	TOPSOIL: (75mm)															
	Silty SAND, some gravel, trace to some clay Loose to Compact Dark Brown Moist (FILL)		1	SS	7											
			2	SS	14											18 49 24 10
67.3																
1.7	SAND and GRAVEL Compact Grey-Brown Damp		3	SS	16											
66.6																
2.3	Silty CLAY, sandy Very Stiff Dark Brown Moist		4	SS	17											0 23 37 40
			5	SS	62/ 0.225											
65.5																
3.4	SHALE, dark grey, thinly laminated, horizontal jointed, occasional limestone interbeds		1	RUN												RUN #1 TCR=68% SCR=29% RQD=0% UCS=30MPa (Average)
	Clay from 4.2m to 4.4m															RUN #2 TCR=88% SCR=48% RQD=33% UCS=30MPa (Average)
			2	RUN												
	Clay (10mm) at 5.8m and 5.9m															RUN #3 TCR=98% SCR=70% RQD=37% UCS=48MPa (Average)
			3	RUN												
61.6																
7.3	END OF BOREHOLE AT 7.3m. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 3.0m slotted screen.															
	WATER LEVEL READINGS: DATE DEPTH (m) ELEV. (m) 2011.12.29 3.0 65.9															

+ 3 x 3 : Numbers refer to  
Sensitivity

20  
15 5  
10 (%) STRAIN AT FAILURE

## METRIC

[illegible]

ONTMT4S 1201B.GPJ 1/18/12

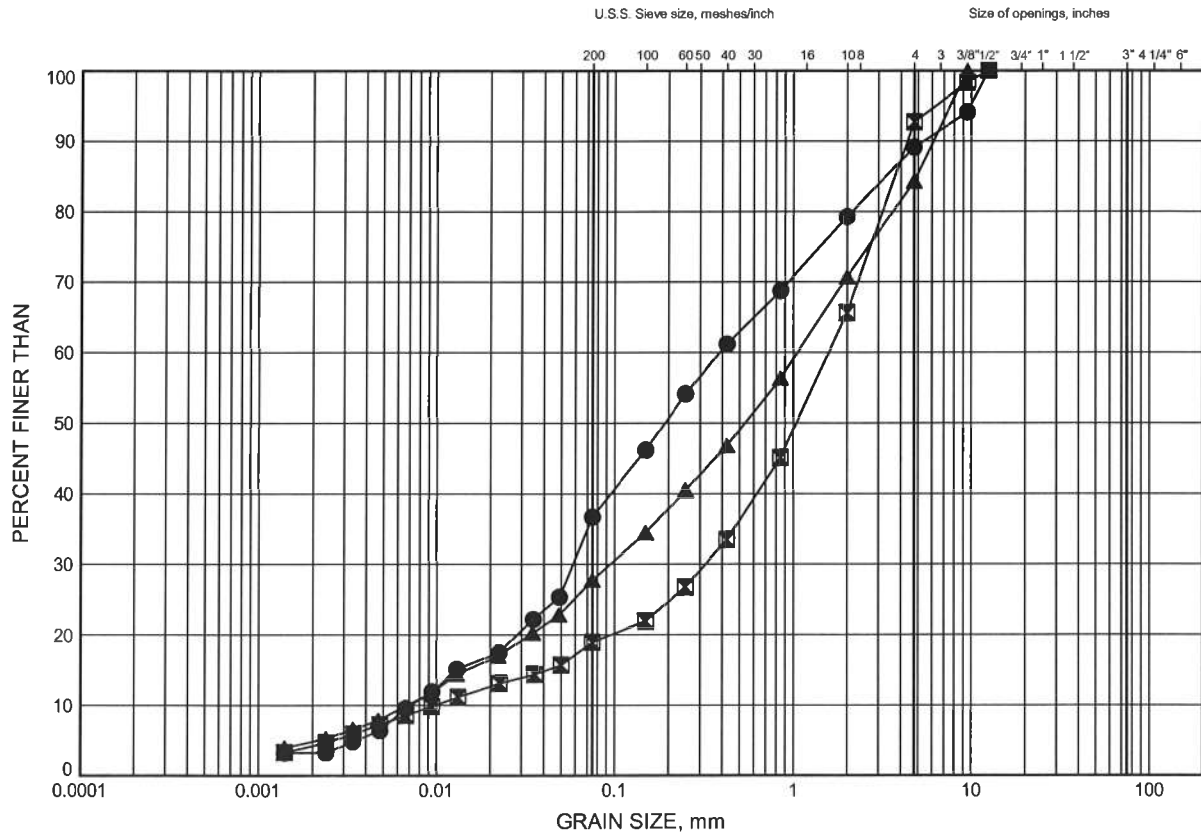
## **Appendix B**

### **Laboratory Test Results**

# Highway 417 Ottawa: Vanier to OR 174 GRAIN SIZE DISTRIBUTION

FIGURE B1

## SILTY SAND FILL



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

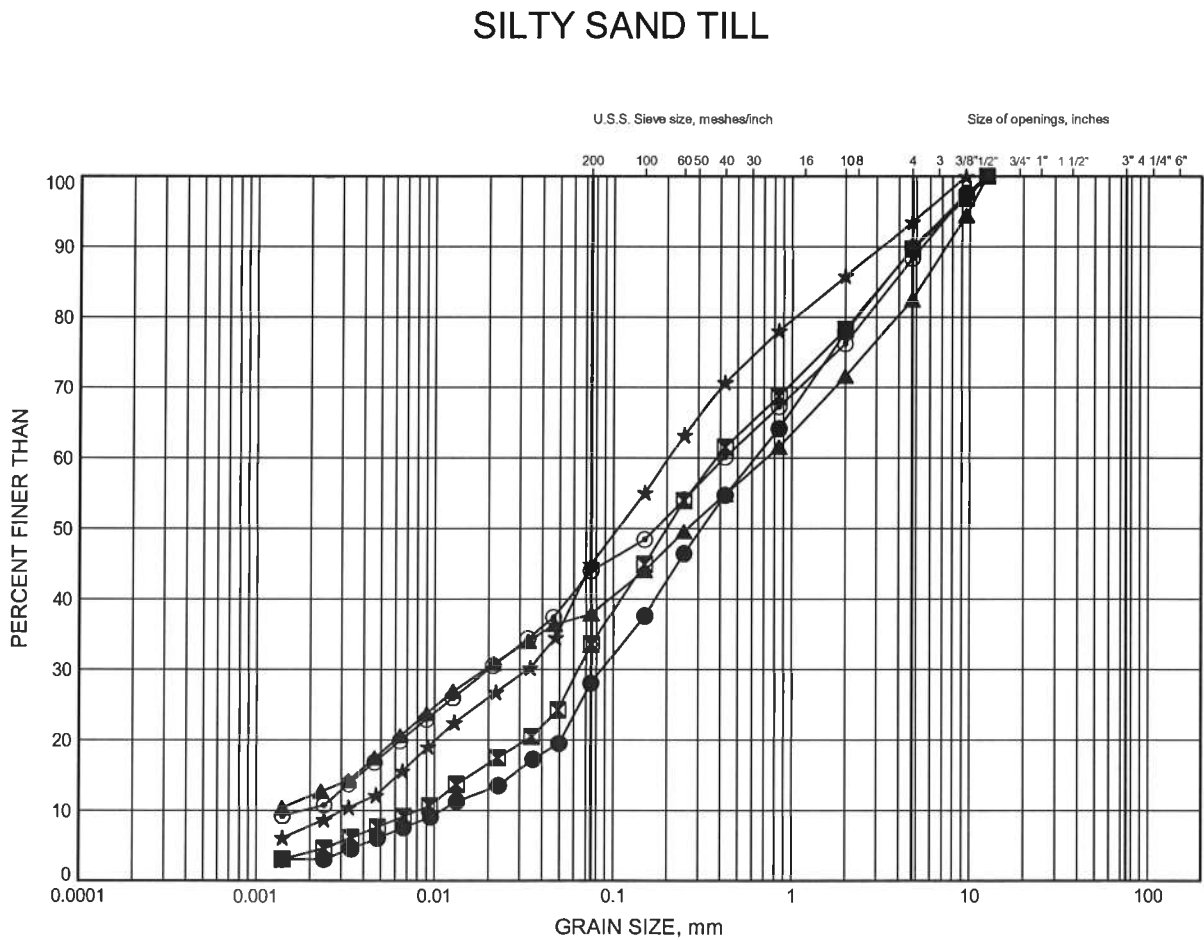
### LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	15N-05	1.83	60.94
⊠	BFR-01	1.83	67.18
▲	BFR-03	1.83	61.03

Highway 417 Ottawa: Vanier to OR 174

# GRAIN SIZE DISTRIBUTION

FIGURE B2



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

## LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	15N-04	2.59	59.68
⊠	23S-02	2.55	59.96
▲	BFR-01	6.40	62.61
★	BFR-02	1.83	60.86
⊙	BFR-04	7.92	60.78

GRAIN SIZE DISTRIBUTION - THURBER 1201B.GPJ 12/15/11

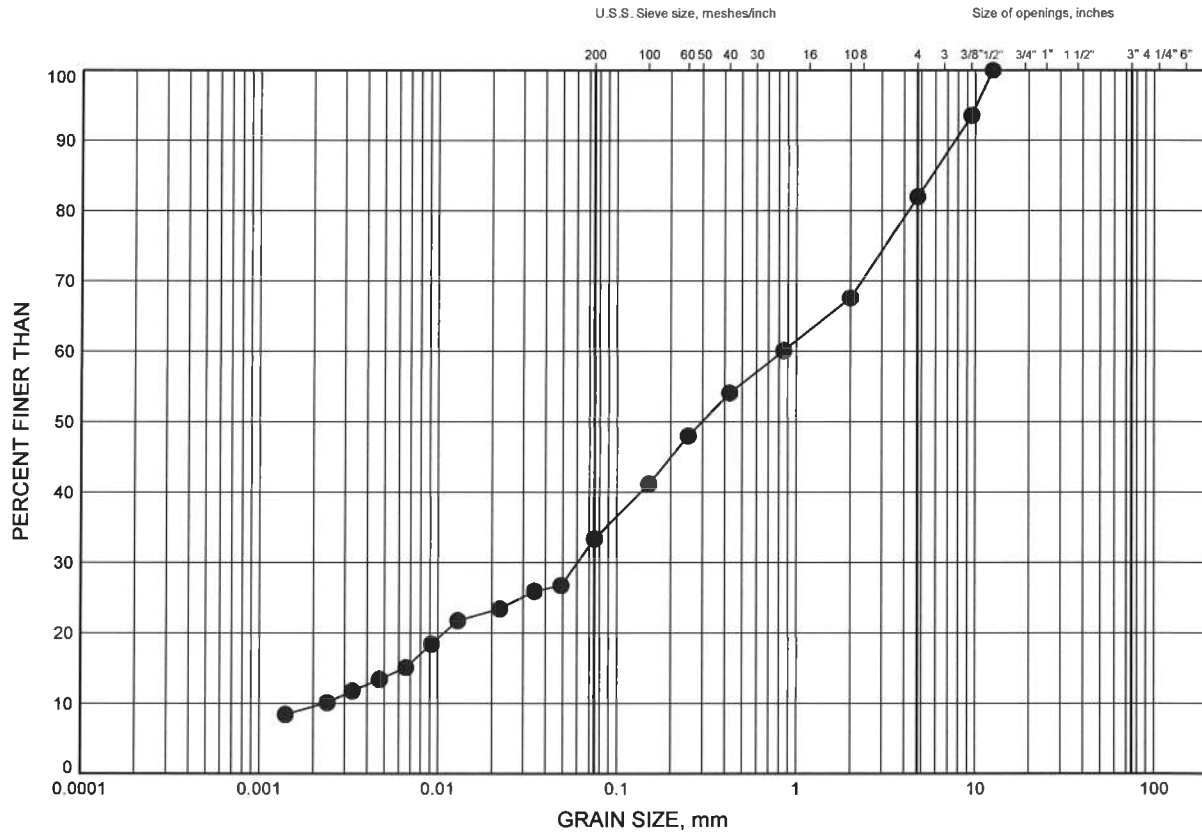
W.P.# 4320-06-00  
 Prepared By AN  
 Checked By MRA



# Highway 417 Ottawa: Vanier to OR 174 GRAIN SIZE DISTRIBUTION

FIGURE B3

## SILTY SAND FILL



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

### LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	BRS-2	1.07	67.86

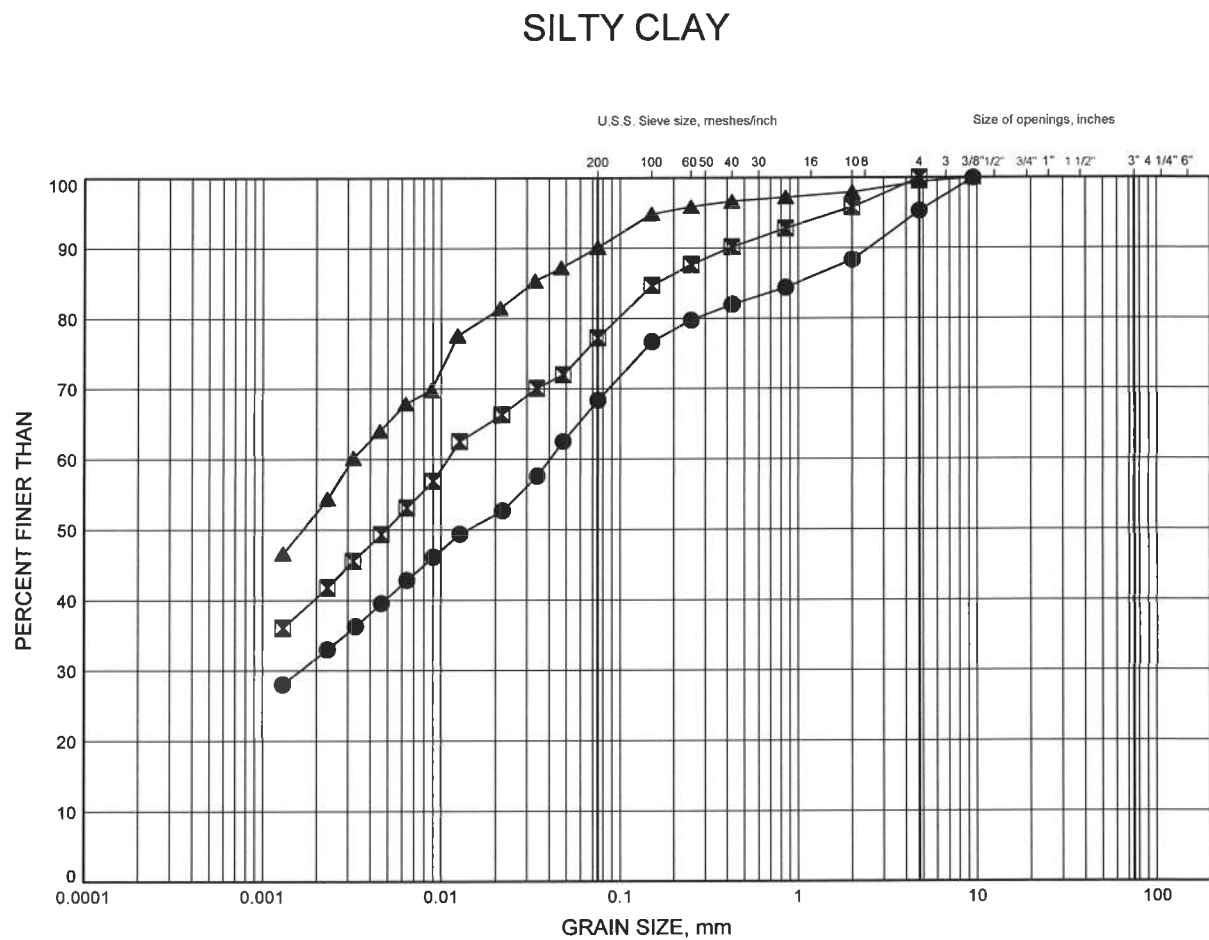


W.P.# 4320-06-00  
Prepared By MFA  
Checked By MRA

Highway 417 Ottawa: Vanier to OR 174

# GRAIN SIZE DISTRIBUTION

FIGURE B4



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	BRS-1	1.07	65.98
⊠	BRS-2	2.59	66.34
▲	BRS-3	1.83	66.74

GRAIN SIZE DISTRIBUTION - THURBER 1201B GPJ 1/18/12

W.P.# 4320-06-00  
Prepared By MFA  
Checked By MRA

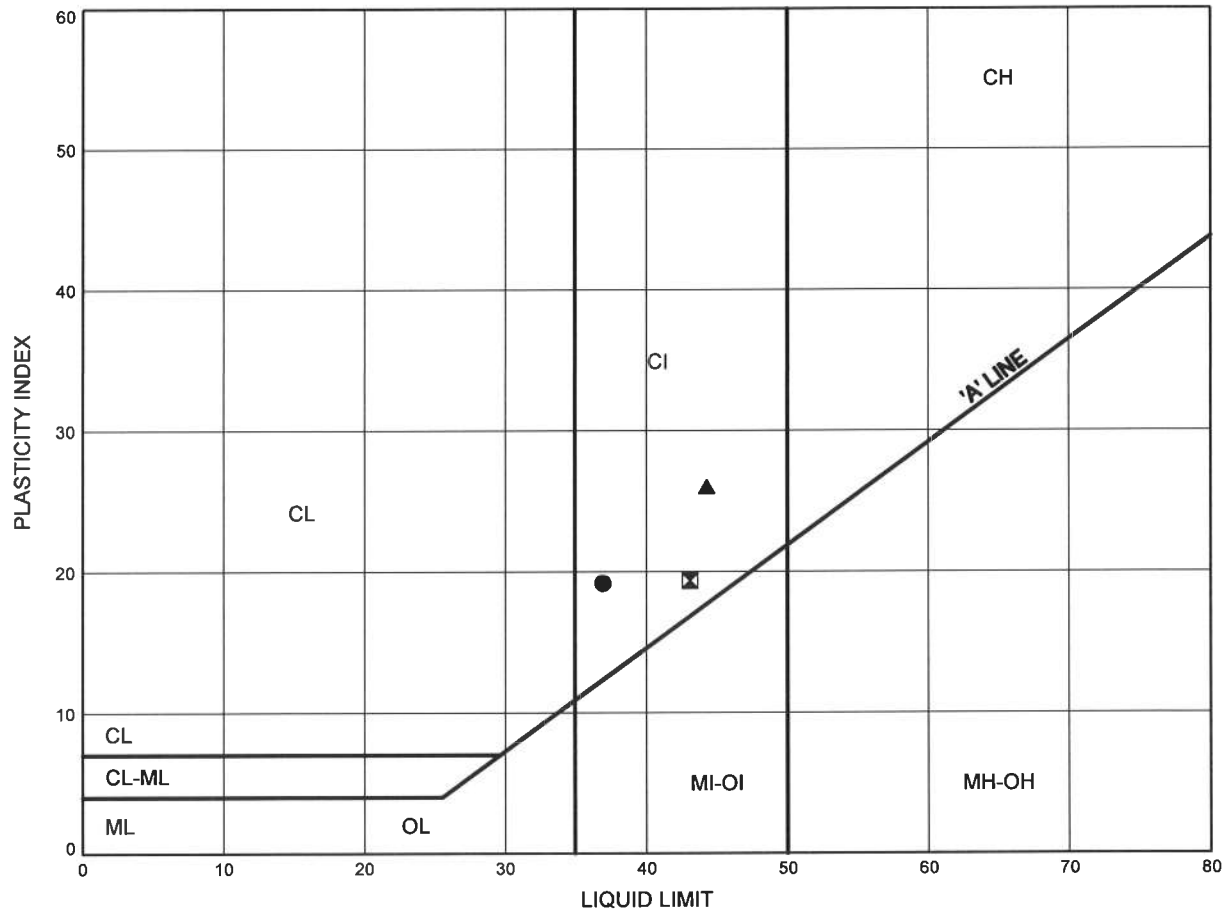




Highway 417 Ottawa: Vanier to OR 174  
**ATTERBERG LIMITS TEST RESULTS**

FIGURE B5

**SILTY CLAY**



SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	BRS-1	1.07	65.98
⊠	BRS-2	2.59	66.34
▲	BRS-3	1.83	66.74

## **Appendix C**

### **Foundation Comparison**

**COMPARISON OF FOUNDATION ALTERNATIVES FOR PIER**

Footings on Native Soil	Footing on Rock	Caissons Socketed into Bedrock	Steel H-Piles
<p><b>Advantages:</b></p> <ul style="list-style-type: none"> <li>i. Economical to install.</li> <li>ii. High geotechnical resistance available in native soil.</li> <li>iii. Overhead clearance should not be an issue.</li> <li>iv. Economical option for temporary pier support.</li> </ul>	<p><b>Advantages:</b></p> <ul style="list-style-type: none"> <li>i. Higher geotechnical resistance than footing on native soil.</li> <li>ii. Overhead clearance should not be an issue.</li> </ul>	<p><b>Advantages:</b></p> <ul style="list-style-type: none"> <li>i. High geotechnical resistance available by socketing caissons into bedrock.</li> <li>ii. Provide uplift and overturning resistance</li> <li>iii. Construction of caissons could continue in freezing weather.</li> <li>iv. Subexcavation to native soil or rock not required.</li> </ul>	<p><b>Advantages:</b></p> <ul style="list-style-type: none"> <li>i. High geotechnical resistance available for piles on/in bedrock.</li> <li>ii. Socketed piles provide uplift and overturning resistance</li> <li>iii. Installation less influenced by weather and groundwater than spread footings.</li> <li>iv. Subexcavation to native soil or rock not required</li> </ul>
<p><b>Disadvantages:</b></p> <ul style="list-style-type: none"> <li>i. Not compatible with existing pile foundation system</li> <li>ii. Ineffective for resistance to uplift or overturning.</li> <li>iii. Roadway protection required.</li> </ul>	<p><b>Disadvantages:</b></p> <ul style="list-style-type: none"> <li>i. Not compatible with existing pile foundation system.</li> <li>ii. Relatively deep excavation and possible dewatering.</li> <li>iii. Ineffective for resistance to uplift or overturning.</li> </ul>	<p><b>Disadvantages:</b></p> <ul style="list-style-type: none"> <li>i. Difficulty in unwatering, cleaning and inspecting bases.</li> <li>ii. Higher unit cost compared to other foundation options such as footings or driven piles.</li> <li>iii. May only be possible outside of existing superstructure envelope due to low clearance.</li> </ul>	<p><b>Disadvantages:</b></p> <ul style="list-style-type: none"> <li>i. Limited overhead clearance may preclude use of driven piles.</li> <li>ii. Installation under the existing superstructure will require low clearance drilling equipment.</li> <li>iii. Concrete placement may require tremie pipe.</li> <li>iv. Higher unit cost compared to footings.</li> </ul>
<b>NOT RECOMMENDED</b>	<b>NOT RECOMMENDED</b>	<b>FEASIBLE</b>	<b>RECOMMENDED</b>

## **Appendix D**

### **List of SPs and OPSS, and Suggested Text for Selected NSSPs**

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

- OPSS 501
- OPSS 539
- OPSS 902
- OPSS 903
- OPSS 1010
- OPSD 3101.150
- OPSD 3121.150
- OPSD 3190.100
- Special Provision 110S13
- Special Provision 999S26

**2. Suggested text for NSSP on Pile Installation**

The soils on site consist of very dense silty sand till containing 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 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 piles 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 alignment of the piles may be affected by the obstructions.

**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, 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.

- Construction of the rock socket shall include provision of a steel liner advanced below the bedrock surface as required to support the sidewalls, minimize groundwater inflow, and enable machine-cleaning of the socket base.

**4. Suggested Text for NSSP on Caisson Concrete**

The shale in the caisson 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 E**

### **Site Photographs**



**Photograph 1: Belfast Road Underpass – Hwy 417 WBL**



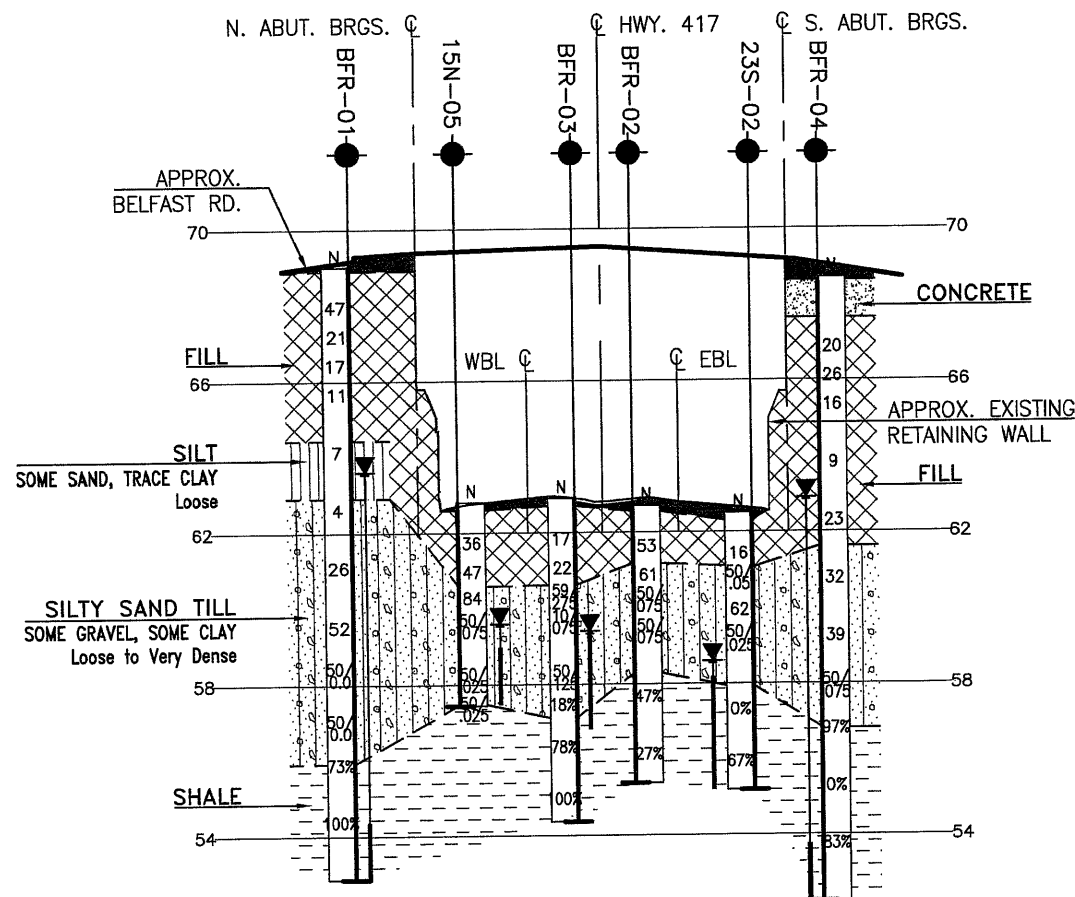
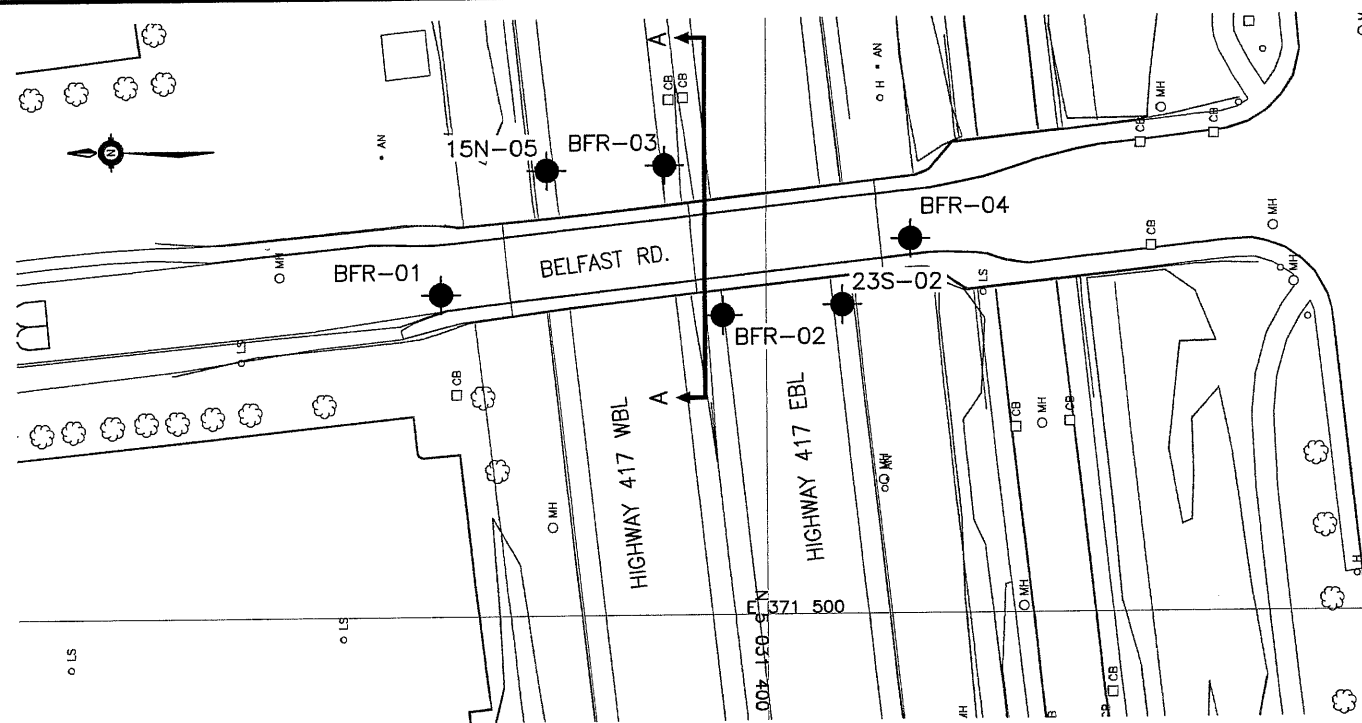
**Photograph 2: Belfast Road Underpass – South End of Structure**



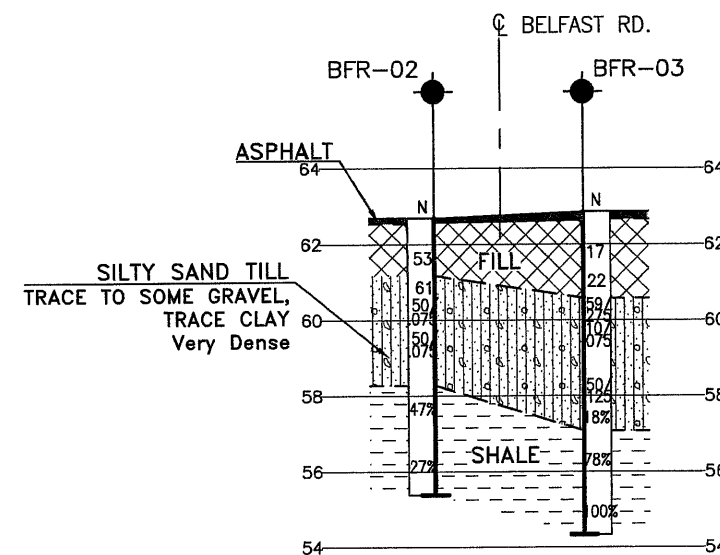
## **Appendix F**

### **Drawing**

#### **Borehole Locations and Soil Strata**



PROFILE ALONG BELFAST RD. UNDERPASS



SECTION A-A

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

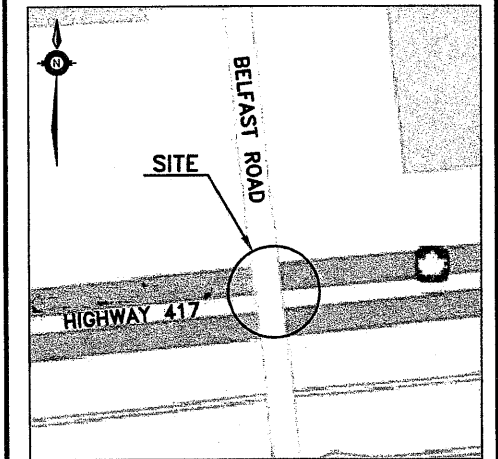


CONT No  
WP No 4320-06-00

HIGHWAY 417  
BELFAST ROAD UNDERPASS  
BOREHOLE LOCATIONS AND SOIL STRATA

**MRC** 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

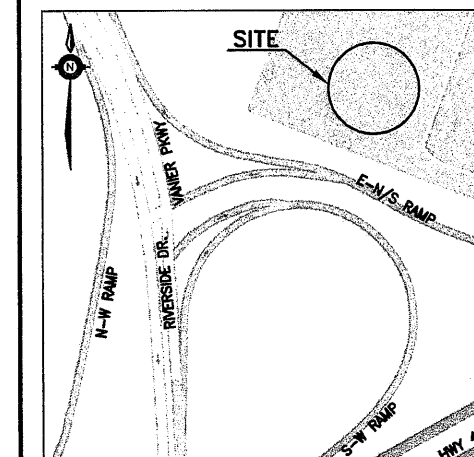
NO	ELEVATION	NORTHING	EASTING
BFR-01	69.0	5 031 442.8	371 542.6
BFR-02	62.7	5 031 405.7	371 539.6
BFR-03	62.9	5 031 413.4	371 559.6
BFR-04	68.7	5 031 381.0	371 549.6
15N-05	62.8	5 031 429.0	371 559.0
23S-02	62.5	5 031 390.0	371 540.9

**-NOTES-**

- The boundaries between soil strata have been established only at Borehole locations. Between Boreholes the boundaries are assumed from geological evidence.
- This drawing is for subsurface information only. Surface details and features are for conceptual illustration.






**GEOCRES No. 31G5-242**

REVISIONS	DATE	BY	DESCRIPTION
DESIGN	MC	CHK MC	CODE
DRAWN	AN	CHK PKC	SITE
			3-071
			STRUCT
			DWG 1
			DATE JAN. 2012



## KEYPLAN

## LEGEND

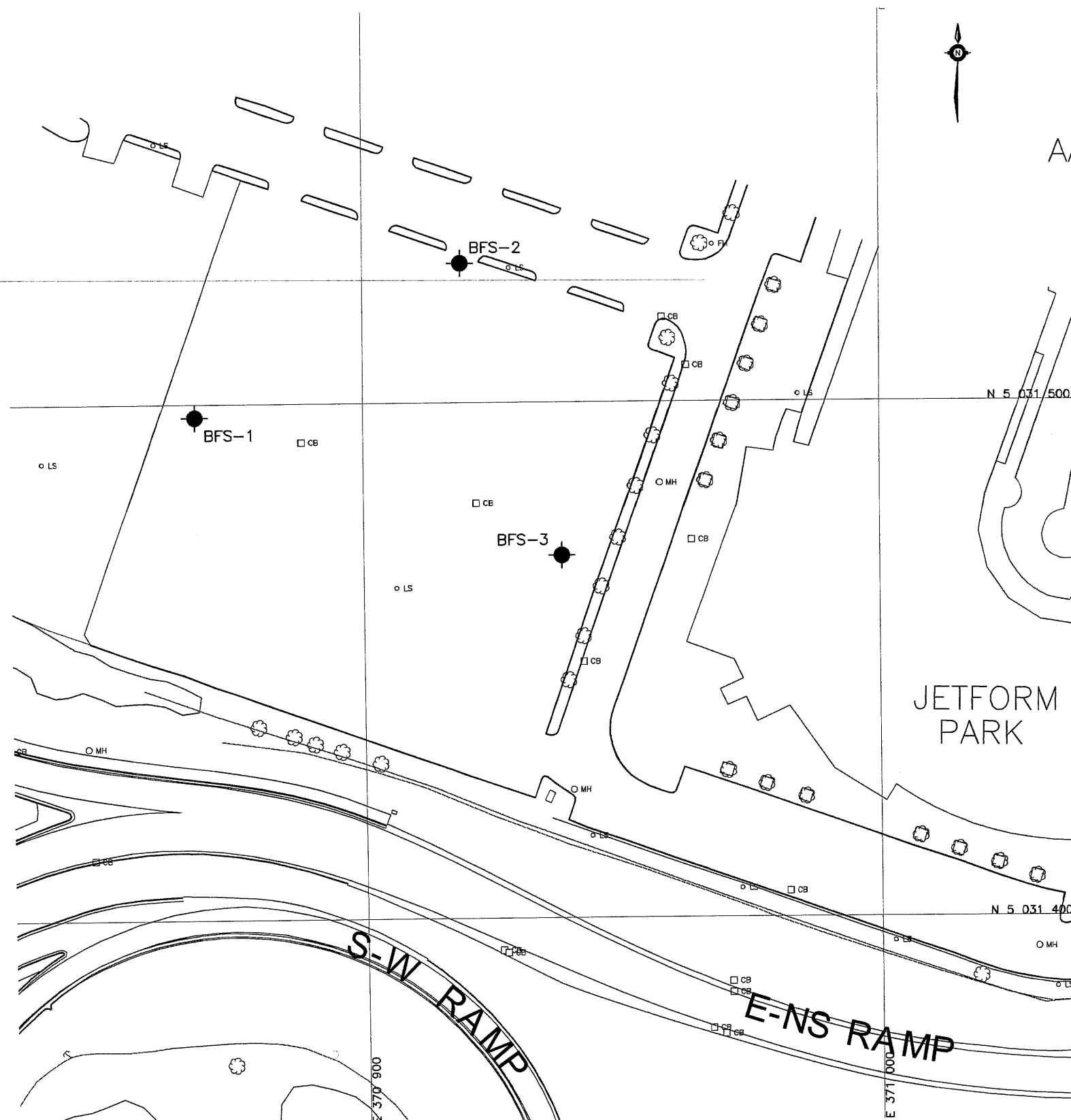
	Borehole
	Borehole and Cone
N	Blows /0.3m (Std Pen Test, 475J/blow)
CONE	Blows /0.3m (60° Cone, 475J/blow)
PH	Pressure, Hydraulic
	Water Level
	Head Artesian Water
	Piezometer
90%	Rock Quality Designation (RQD)
A/R	Auger Refusal

NO	ELEVATION	NORTHING	EASTING
BFS-1	61.4	5 031 497.4	370 867.4
BFS-2	61.7	5 031 527.1	370 919.1
BFS-3	61.7	5 031 470.3	370 938.3

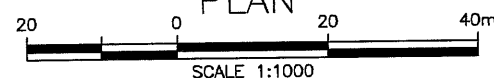
-NOTES-

- 1) The boundaries between soil strata have been established only at Borehole locations. Between Boreholes the boundaries are assumed from geological evidence.
- 2) This drawing is for subsurface information only. Surface details and features are for conceptual illustration.

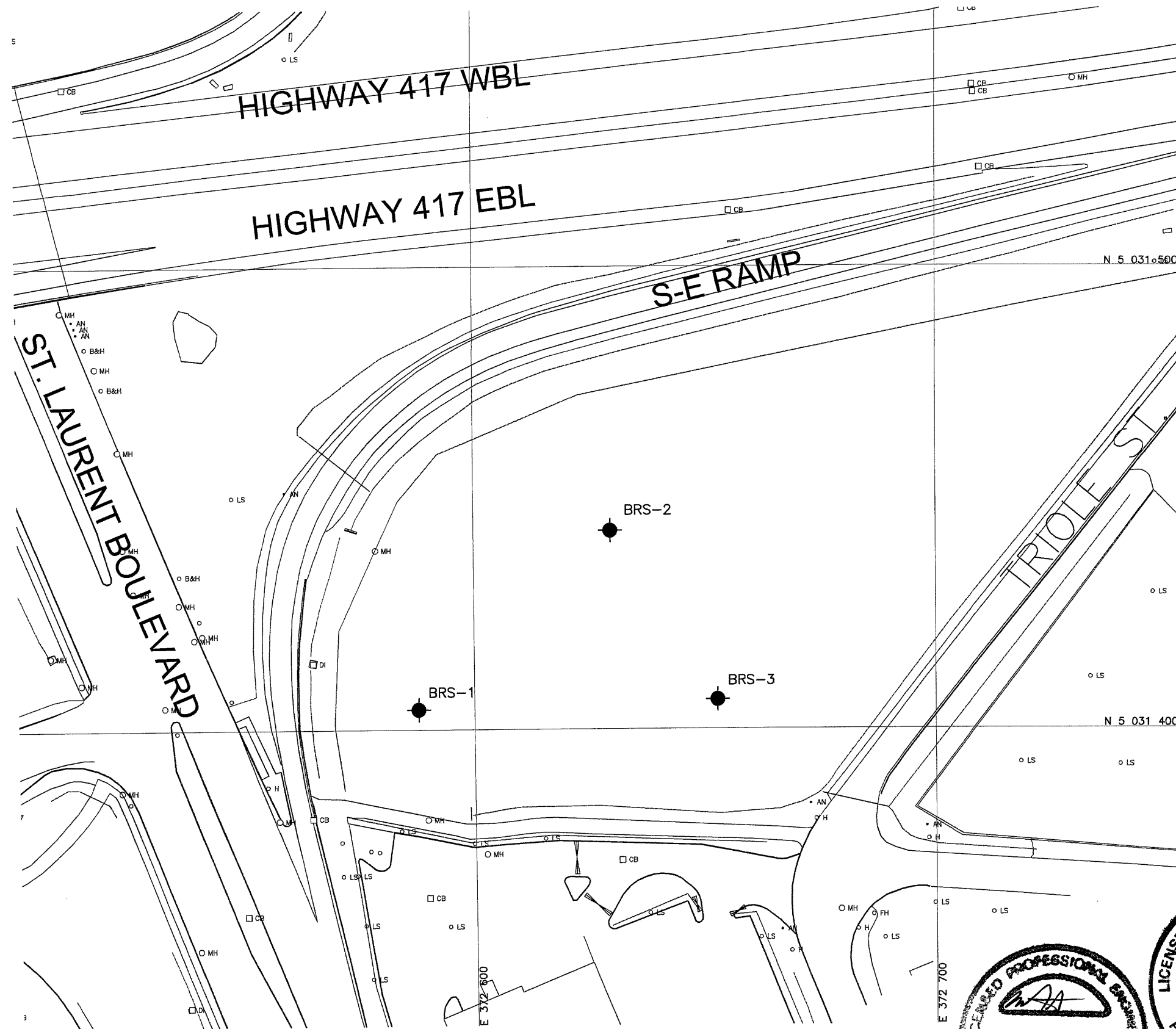
**GEOCRES No. 31G5-242**



## PLAN



REVISIONS									
	DATE	BY	DESCRIPTION						
DESIGN	MC	CHK	MC	CODE	LOAD		DATE	JAN. 2011	
DRAWN	AN	CHK	PKC	SITE	3-071	STRUCT	DWG	1	



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

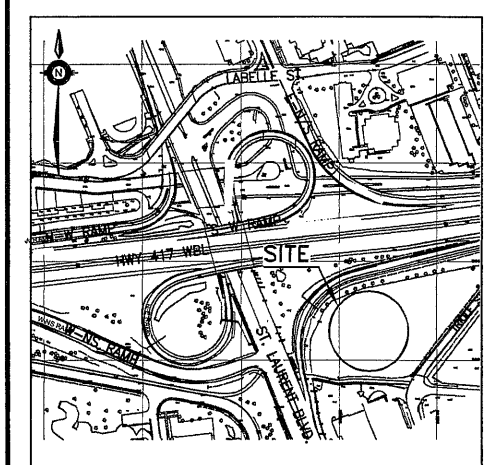
CONT No  
WP No 4320-06-00

HIGHWAY 417  
BELFAST ROAD STAGING AREA  
BOREHOLE LOCATION PLAN

MRC McCORMICK RANKIN  
CORPORATION

SHEET

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
BRS-1	67.1	5 031 404.4	372 587.9
BRS-2	68.9	5 031 442.9	372 629.7
BRS-3	68.6	5 031 406.3	372 652.7

- NOTES-**
- The boundaries between soil strata have been established only at Borehole locations. Between Boreholes the boundaries are assumed from geological evidence.
  - This drawing is for subsurface information only. Surface details and features are for conceptual illustration.

GEOCRES No. 31G5-242

REVISIONS	DATE	BY	DESCRIPTION
DESIGN	MC	CHK MC	CODE
DRAWN	AN	CHK PKC	SITE 3-071
			STRUCT
			DWG 1