

**FOUNDATION INVESTIGATION AND DESIGN REPORT  
ST. LAURENT BLVD. OVERPASS REHABILITATION AND WIDENING  
HIGHWAY 417 EXPANSION FROM VANIER PARKWAY TO O.R.174  
OTTAWA, ONTARIO**

**G.W.P. 4320-06-00, SITE No. 3-072**

**Geocres Number: 31G5-243**

**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 and widening of the existing St. Laurent Blvd. Overpass in Ottawa, Ontario. This structure rehabilitation and widening is part of the Highway 417 Expansion project, from Vanier Parkway to O.R.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-00007.

**2 SITE DESCRIPTION**

The St. Laurent Blvd. Overpass structure is located on Highway 417 approximately 6 km east of Ottawa city centre. The structure is a 2 span bridge with a total length of approximately 38 m which crosses St. Laurent Blvd., a 6 lane roadway. The deck is approximately 46 m wide and carries 6 main lanes of traffic for Highway 417 as well as two on-ramp lanes (N-E Ramp and S-W Ramp). The pier and abutments are supported on spread footings founded on shale bedrock. Approach embankments on either side of the bridge are approximately 6.0 m high.

Land use surrounding the site is primarily commercial/industrial to the northeast and southeast, and retail (shopping centre) to the northwest. To the southwest, the lands adjacent to the site are undeveloped and further west the lands consist of residential developments.

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 overpass site however, the general stratigraphy comprises fill underlain by sand and glacial silty 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. The Billings Formation lies adjacent to the north end of the structure.

Photographs in Appendix C 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 during the period of July 11 to September 12, 2011 and consisted of drilling and sampling ten boreholes at the existing structure (Boreholes SLB-01 to SLB-10). Boreholes SLB-01, SLB-05, SLB-06, and SLB-10 were drilled on the shoulders of Highway 417 while Boreholes SLB-02 to SLB-04 and SLB-07 to SLB-09 were drilled along St. Laurent Blvd. The locations, termination depths and elevations of the ten boreholes drilled at this site are listed in Table 3.1, below.

**Table 3.1 – Borehole Details**

Location	Borehole	Ground Elevation (m)	Termination Elevation (m)	Termination Depth (m)
West Approach	SLB-01	72.8	60.9	11.9
	SLB-06	72.6	60.4	12.2
West Abutment	SLB-02	66.6	60.3	6.3
	SLB-07	67.0	60.0	7.0
Pier	SLB-03	67.3	61.2	6.1
	SLB-08	67.0	61.4	5.6
East Abutment	SLB-04	66.9	61.2	5.7
	SLB-09	67.0	60.6	6.4
East Approach	SLB-05	73.1	61.2	11.9
	SLB-10	73.1	60.9	12.2

The locations of the boreholes are shown on the attached Borehole Locations and Soil Strata Drawing in Appendix G. A list of the borehole coordinates and elevations is included on this drawing.

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 from the City of Ottawa



for boreholes drilled within the St. Laurent Blvd. right-of-way. An MTO encroachment permit was obtained for boreholes on Highway 417.

Both a CME 75 and CME 55 truck-mounted drill rig were used for the boreholes drilled at this site. 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). A minimum 3m length of rock core was recovered from each borehole. All rock cores were logged in the field, and the Total Core Recovery (TCR), Solid Core Recovery (SCR), Rock Quality Designation (RQD) and Fracture Index (FI) were determined for each core run.

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 were not recorded in the open boreholes during drilling operations since water was introduced into the borehole during coring and was not representative of groundwater conditions on site. Standpipe piezometers, consisting of 19 mm PVC pipe with a slotted screen, were installed in six boreholes at this site. The completion details of the piezometers are summarized in Table 3.2. Following the final water level reading, the piezometers will be decommissioned in general accordance with MOE Regulation 903. Upon completion of drilling, boreholes without a piezometer installation were backfilled with a mixture of bentonite holeplug and cuttings then asphalt cold patch at the surface, where appropriate.

**Table 3.2 – Piezometer Completion Details**

Location	Borehole	Tip Position (m)		Completion Details
		Depth	Elev.	
West Approach	SLB-06	12.2	60.4	Sand filter from 12.2 to 7.3 m, bentonite holeplug from 7.3 to 6.7 m, cuttings and bentonite mixture from 6.7 to 0.6 m, then asphalt cold patch to surface. Flushmount casing protector installed at surface.
West Abutment	SLB-07	4.9	62.1	Sand filter from 7.0 to 1.2 m, bentonite holeplug from 1.2 to 0.3 m, then cuttings to surface. Flushmount casing protector installed at surface.
Pier	SLB-08	4.6	62.4	Sand filter from 5.6 to 1.2 m, bentonite holeplug from 1.2 to 0.3 m, then cuttings to surface with 50 mm asphalt patch. Flushmount casing protector installed at surface.
East Abutment	SLB-04	4.6	62.3	Sand filter from 5.7 to 1.2 m, bentonite holeplug from 1.2 to 0.3 m, then cuttings to surface. Flushmount casing protector installed at surface.
East Approach	SLB-05	10.4	62.7	Sand filter from 11.9 to 6.7 m, bentonite holeplug from 6.7 to 5.8 m, cuttings and bentonite mixture from 5.8 m to 0.1 m, then asphalt cold patch to surface. Flushmount casing protector installed at surface.
	SLB-10	10.7	62.4	Sand filter from 12.2 to 7.0 m, bentonite holeplug from 7.0 to 6.1 m, cuttings and bentonite mixture from 6.1 to 0.3 m, then sand to surface with 100 mm asphalt cold patch. Flushmount casing protector installed at 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.

Point load tests were carried out on selected samples of intact bedrock core to assist in evaluation of the compressive strength of the bedrock. Results of the point load tests are included on the Record of Borehole sheets in Appendix A (as average per core run).



## **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 G. 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 should be used for interpretation of the site conditions.

In general terms, the stratigraphy encountered in the boreholes drilled on the shoulders of Highway 417 consists of pavement structure overlying silty clay fill, underlain by silty sand till which in turn is underlain by shale bedrock. The stratigraphy encountered in the boreholes drilled along St. Laurent Blvd generally consists of pavement structure overlying silt and sand fill, underlain by a sand layer in the west and underlain by silty sand till in the east. Shale bedrock was also encountered below the overburden deposits in the boreholes drilled along St. Laurent Blvd.

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

### **5.1 Pavement Structure**

#### **5.1.1 Highway 417**

In Boreholes SLB-01, SLB-05, SLB-06, and SLB-10 drilled on the shoulders of Highway 417 at the approaches to the overpass, the pavement structure consists of 150 mm to 225 mm of asphalt overlying 0.6 m to 1.9 m of granular fill.

The granular fill is brown to dark grey and varies from sand containing some gravel and some silt, to sand and gravel. The lower boundary of the granular fill was encountered at depths ranging from 0.8 m to 2.1 m (Elevations 72.0 to 70.5).

SPT 'N' values recorded in the granular fill ranged from 21 to 68 blows for 0.3 m penetration, indicating a compact to very dense relative density.

Moisture contents of 1 to 7% were measured in the granular fill.

#### **5.1.2 St. Laurent Boulevard**

In Boreholes SLB-03 and SLB-08 drilled on the left shoulder of northbound St. Laurent Blvd., the pavement structure consists of 50 mm of asphalt overlying 700 mm of brown to grey sand containing some gravel and trace silt.

Moisture contents of samples of the sand fill were 2% and 3%.

## 5.2 Fill

### 5.2.1 Silty Clay Fill

Cohesive embankment fill was encountered below the pavement structure in the boreholes drilled on the shoulders of Highway 417 (SLB-01, SLB-05, SLB-06, and SLB-10). The cohesive fill consists of greenish brown to grey silty clay containing trace to some sand.

The thickness of the silty clay fill ranged from 4.3 to 6.5 m with a lower boundary at a depth of 6.1 to 7.3 m (Elevations 67.0 to 65.5 m).

SPT 'N' values recorded in the cohesive fill typically ranged from 5 to 13 blows for 0.3 m penetration, indicating a firm to stiff consistency. A higher SPT 'N' value of 17 blows for 0.3 m penetration was recorded in Borehole SLB-10 at a depth of 6.5 m, where occasional gravelly sand seams were observed in the silty clay fill.

The moisture contents of the silty clay fill samples typically ranged from 39 to 51%, with two lower moisture contents (16% and 27%) measured in Borehole SLB-01.

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

Gravel %	0
Sand %	1 to 13
Silt %	20 to 23
Clay %	64 to 77

Atterberg Limits tests were also carried out on four samples of the cohesive fill. The results of the Atterberg Limits tests are plotted on Figure B4, Appendix B, and are summarized below.

Plastic Limit %	25 to 27
Liquid Limit %	67 to 74
Plasticity Index %	43 to 48

The results of the Atterberg Limits tests indicate that the silty clay fill has high plasticity with a group symbol of CH.

### 5.2.2 Sand Fill

Granular fill consisting primarily of sand was encountered at the surface in Boreholes SLB-02, SLB-04, and SLB-09, below the pavement structure in Boreholes SLB-03 and SLB-08, and below silt fill encountered at surface in Borehole SLB-07. The sand fill is grey to black and contains trace gravel to gravelly, trace to some silt, trace clay, and occasional cobbles. An obstruction was encountered at 1.5 m depth in Borehole SLB-04.



The thickness of the sand fill ranged from 0.8 m to 2.1 m with a lower boundary at a depth of 0.8 m to 3.2 m (Elevations 66.2 to 63.8).

SPT 'N' values measured in the sand fill generally ranged from 13 to 66 blows for 0.3 m penetration, indicating a compact to very dense relative density. Higher SPT 'N' values of 50 blows/ 0.025 m and 68 blows/ 0.275 m were measured in Boreholes SLB-07 and SLB-08, respectively, just above bedrock. An 'N' value of 7 (loose) was obtained in a gravelly sand layer in Borehole SLB-02.

Moisture contents of samples of the sand fill generally ranged from 2% to 15%. A higher moisture content of 41% was measured in Borehole SLB-07 at a depth of 3.2 m, just above bedrock.

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

Gravel %	6 to 23
Sand %	52 to 80
Silt %	11 to 18
Clay %	3 to 7

### 5.2.3 Silt Fill

Silt fill was encountered at the surface in Borehole SLB-07 and below gravelly sand fill in Borehole SLB-09. The silt fill is dark grey to brown/black and contains some sand, trace to some clay, trace gravel, some organics, shale fragments, glass and wood fragments.

The silt fill was 1.5 m thick in both boreholes, with a lower boundary at a depth of 1.5 m in Borehole SLB-07 and 2.3 m in Borehole SLB-09 (Elevations 65.5 and 64.7).

SPT 'N' values recorded in the silt fill ranged from 22 to 39 blows for 0.3 m penetration, indicating a compact to dense condition. Samples of the silt fill had measured moisture contents of 4% to 19%.

## 5.3 Sand

Native sand was encountered below the sand fill in Boreholes SLB-02 and SLB-03. The sand is brown to black, fine grained and contains trace gravel, clay and shale fragments.

The sand was 0.7 m thick in Borehole SLB-02, with a lower boundary at a depth of 2.8 m (Elevation 63.8). In Borehole SLB-03 the sand was 0.4 m thick, with a lower boundary at a depth of 2.7 m (Elevation 64.6).

SPT 'N' values recorded in the native sand ranged from 9 to 31 blows for 0.3 m penetration, indicating a loose to dense condition.

Moisture contents of the sand samples were 6% and 14%.

#### 5.4 Silty Sand Till

Silty sand till was encountered below silty clay fill in Boreholes SLB-01, SLB-05, SLB-06, and SLB-10 and below the sand/silt fill in Boreholes SLB-04 and SLB-09 (both located at the east abutment). The silty sand till is dark brown to black and contains trace to some clay and trace to some gravel.

The thickness of the silty sand till ranged from 1.5 m to 2.8 m in the approach boreholes (SLB-01, SLB-05, SLB-06, and SLB-10) and 0.5 m to 1.1 m in the boreholes at the east abutment (SLB-04 and SLB-09, respectively). The depth to the base of the silty sand till ranged from 8.8 m to 9.2 m (Elevations 64.3 to 63.4) at the approaches, and 2.6 m to 3.4 m (Elevations 64.3 and 63.6) at the east abutment.

SPT 'N' values recorded in the silty sand till typically ranged from 14 to 49 blows for 0.3 m penetration, indicating a compact to dense relative density. An SPT 'N' value of 5 blows for 0.3 m penetration, indicating a loose condition, was recorded in Borehole SLB-09 at 2.5 m depth. SPT 'N' values of 78 and 76 blows for less than 0.3 m penetration, indicating a very dense condition, were recorded in Borehole SLB-04 and SLB-09 directly above bedrock and therefore these 'N' values are likely a result of spoon refusal on bedrock. Difficult augering or auger grinding was encountered in the till in Boreholes SLB-01 and SLB-06.

The moisture content of samples of the silty sand till ranged from 9 to 28%.

Grain size distribution analyses were carried out on six samples of the silty sand till. The results of these tests are plotted on Figure B3, Appendix B, and are summarized below.

Gravel %	4 to 15
Sand %	46 to 54
Silt %	23 to 35
Clay %	7 to 15

For one sample of the silty sand till, the clay content was sufficient to allow for Atterberg Limits testing. The results of this test are plotted on Figure B5, Appendix B, and are summarized below.

Plastic Limit %	14
Liquid Limit %	22
Plasticity Index %	8

The results of this test indicate that the silty sand till has zones of low plasticity, with a group symbol of CL.

Glacial tills are known to contain cobbles, boulders and bedrock slabs.



## 5.5 Shale Bedrock

Bedrock was encountered below the silty sand till in the approach boreholes (SLB-01, SLB-05, SLB-06, and SLB-10) and the boreholes at the east abutment (SLB-04 and SLB-09), and below either sand or fill in the boreholes at the west abutment and pier. Bedrock was proved by coring in all boreholes drilled at this site. The depths and elevations at which bedrock was encountered are summarized in Table 5.1.

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

Location	Borehole	Bedrock Surface	
		Depth (m)	Elevation (m)
West Approach	SLB-01	8.8	64.0
	SLB-06	9.2	63.4
West Abutment	SLB-02	2.8	63.8
	SLB-07	3.2	63.8
Pier	SLB-03	2.7	64.6
	SLB-08	2.8	64.2
East Abutment	SLB-04	2.6	64.3
	SLB-09	3.4	63.6
East Approach	SLB-05	8.8	64.3
	SLB-10	9.1	64.0

The bedrock was described as laminated grey shale and typically contains hard limestone interbeds up to 50 mm in thickness. The shale was generally described as slightly weathered to fresh. Occasional vertical fractures, rubbles zones, and clay seams were observed in the bedrock cores. Total Core Recovery (TCR) in the bedrock ranged from 73 % to 100%. The RQD values ranged from 0 to 100%, indicating a widely variable rock quality ranging from very poor to excellent. RQD values typically ranged from 32 to 71%, which is indicative of poor to fair rock quality. The Fracture Index (FI) of the rock, expressed as fractures per 0.3 m of core, was also quite variable and 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 6 to 25 MPa, indicating a weak rock strength classification. Higher rock strengths may be obtained in the hard limestone interbeds.



## 5.6 Groundwater Levels

Groundwater was not recorded 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 six boreholes at this site upon completion of drilling. The groundwater depths and elevations measured in the piezometers are summarized in Table 5.2.

**Table 5.2 – Groundwater Depths and Elevations**

Location	Borehole	Date	Water Level	
			Depth (m)	Elevation (m)
West Approach	SLB-06	20-Sept-2011	8.8	63.8
		12-Oct-2011	8.6	64.0
West Abutment	SLB-07	21-Jul-2011	2.1	64.9
		26-Jul-2011	2.2	64.8
		18-Aug-2011	2.2	64.8
		2-Sep-2011	2.4	64.6
		8-Sep-2011	3.4	63.6
		20-Sep-2011	2.4	64.6
		12-Oct-2011	2.8	64.2
Pier	SLB-08	21-Jul-2011	3.0	64.0
		26-Jul-2011	3.0	64.0
		18-Aug-2011	3.0	64.0
		8-Sep-2011	3.4	63.6
		20-Sep-2011	3.2	63.8
		12-Oct-2011	2.9	64.1
East Abutment	SLB-04	21-Jul-2011	2.3	64.6
		26-Jul-2011	2.3	64.6
		18-Aug-2011	2.3	64.6
		8-Sep-2011	3.4	63.5
		20-Sep-2011	2.4	64.5
		12-Oct-2011	1.7	65.2
East Approach	SLB-05	20-Sep-2011	8.6	64.5
		12-Oct-2011	7.7	65.4
	SLB-10	20-Sep-2011	8.5	64.6
		12-Oct-2011	8.7	64.4

The groundwater level at the approach embankments ranged from 7.7 to 8.8 m below ground surface (Elevation 65.4 to 63.8). Groundwater measured along St. Laurent Blvd. was at depths of 1.7 to 3.4 m (Elevation 65.2 to 63.5).

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 borehole locations after completion of the site investigation.

Eastern Ontario Diamond Drilling Ltd. from Hawkesbury, Ontario supplied both a CME 75 and CME 55 truck mounted drill rig for this site and conducted the drilling, sampling and in-situ testing operations.

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

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 and widening works proposed at the St. Laurent Blvd. Overpass.

The existing structure is a two-span overpass supported by a pier and closed abutments founded on bedrock. The overpass spans a distance of approximately 38 m over the six-lane roadway of St. Laurent Blvd. The overpass deck is approximately 46 m wide, carrying Highway 417 which comprises six lanes of main traffic and two on-ramp lanes (N-E Ramp and S-W Ramp). The existing structure replaced a single span rigid frame structure with a deck span of 24 m and width of 33 m, previously at this location.

Rehabilitation and widening of the existing overpass is planned to accommodate an extra traffic lane in each direction on Highway 417. The works will include extension of the pier and abutments both northwards and southwards, widening of the deck by placement of two new 1.2m wide prestressed concrete box girders on each side of the deck, and widening of the approach embankments. The widened structure will have closed abutments, and the abutments will be converted to a semi-integral configuration. The structure widening will be 1.4 m at the southwest and northeast quadrants, and 2.7 m at the southeast and northwest quadrants.

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

- Extension of the pier and abutment footings for widening of bridge superstructure and provision of seismic/ uplift in accordance with CHBDC requirements;

- Widening of approach embankments; and
- Temporary roadway protection works.

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 EXTENSION OF PIER AND ABUTMENT FOUNDATIONS**

Archive drawings indicate that the existing abutments are founded on approximate 6 m wide, 47.4 m long and 1.6 to 1.8 m thick spread footings. The pier is founded on two 2 m wide, 20 m long and 1.0 m thick spread footings. The design founding levels are elevation 63.0 at the pier, elevation 63.5 at the east abutment, elevation 63.0 at the WBL west abutment and elevation 62.5 at the EBL west abutment. The footings are constructed over a 150 mm thick layer of mass concrete within shale bedrock.

It is recommended that the extensions for the pier and abutments be supported on spread footings founded within unweathered shale bedrock at the same levels as the existing footings. A foundation type similar to the existing foundation is recommended to simplify construction and minimise the potential for differential movements between the new and existing footings. Alternatively, drilled caissons could be employed to limit requirements for shoring/roadway protection during construction. Pile foundations are not recommended in view of the shallow shale bedrock at the site. A comparison of the foundation alternatives is provided in Appendix D.

### **8.1 Extension of Spread Footings on Rock**

It is recommended that the footing extensions be founded on unweathered shale bedrock below the level of any broken or highly weathered shale. The elevations of the bedrock encountered at the borehole locations, the design founding level of the existing footings, and the interpreted depth below the bedrock surface are summarized in Table 8.1. The as-built level of the existing footings must be confirmed in the field.

The founding elevation of the footing extension must match the founding level of the existing footing at the interface of the new and existing footing. To avoid disturbance of the shale on which the existing footing is founded, bedrock excavation for footing extension should not extend below the existing founding level adjacent to the existing footing.

Spread footings founded on unweathered shale bedrock may be designed based on the following geotechnical resistance:

- Factored geotechnical resistance of 1,500 kPa at Ultimate Limit State (ULS)

**Table 8.1 – Depths and Elevations of Unweathered Shale Surface**

Foundation Element		Borehole	Design Founding Level of Existing Footing (m)	Bedrock Surface Elevation (m)	Estimated Depth of Footing Below Shale Surface (m)
West Abutment	North	SLB-07	63.0	63.8	0.8
	South	SLB-02	62.5	63.8	1.3
Pier	North	SLB-08	63.0	64.2	1.2
	South	SLB-03	63.0	64.6	1.6
East Abutment	North	SLB-09	63.5	63.6	0.1
	South	SLB-04	63.5	64.3	0.8

The recommended ULS value includes a resistance factor of 0.5 as per Table 6.1 of the CHBDC. The SLS condition will not govern design for footings founded on shale bedrock.

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

The sliding resistance of mass concrete poured on the unweathered shale may be computed on the basis of an unfactored 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 bases of the foundation excavations must be inspected to confirm that the exposed surface conforms to the design requirements and has been adequately prepared to receive concrete. Where sub-excavation is required to remove unsuitable material from below the design founding level, the founding surface must be re-established using concrete fill of the same class of concrete as used in the footing.

Excavation of the bedrock to the founding level must be carried out using equipment and procedures that minimize the potential for disturbance/damage to the existing structure and bedrock on the founding surface (no blasting). The bearing surface should be prepared by removing all loose/ disturbed material and shattered rock.

The shale at the base of the footing excavation must be protected from deterioration by a concrete working slab. The working slab must be at least 100 mm thick and must be placed as soon as practical after completion of the excavation and in no case later than 4



hours after excavation. Suggested wording for an NSSP on the working slab is provided in Appendix F.

## 8.2 Caissons

The pier and abutment extensions may be founded on caissons (drilled shafts) socketed into the shale bedrock. The caissons will provide resistance to both axial foundation loads and seismic uplift.

Caissons socketed into bedrock to develop uplift resistance should be extended at least 3 m below the bedrock surface. Shale bedrock was encountered at depths of 2.6 to 3.4 m below existing grade on St. Laurent Boulevard.

The factored axial geotechnical resistances at ULS recommended for typical caisson designs socketed 3 m and 5 m into shale bedrock, are provided in Table 8.2. These values include the geotechnical resistance factors of 0.4 and 0.3 specified in the CHBDC for axial compression and uplift, respectively.

**Table 8.2- 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 SLS condition will not govern for caissons founded in bedrock.

Temporary steel liners must be used to support the sides of the caisson shaft through the overburden. The liners should be sealed into the bedrock to exclude groundwater and permit construction in the dry.

Caissons should be backfilled with concrete within 8 hours of excavation to minimize softening of the shale bedrock. Suggested wording for an NSSP in this regard is provided in Appendix F.

The Contractor's caisson drilling equipment must be able to penetrate shale bedrock with frequent hard limestone layers to prepare the rock sockets.

The horizontal loads imposed on the wall will be resisted by passive forces developed on the face of the caissons within the shale below the level of existing foundation backfill. 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:

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

The above equations and recommended parameters may be used to analyse the interaction between a pile and the surrounding rock. 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,  $K = k_s \cdot L \cdot D$  (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 on any one segment of pile,  $P_{ult}$ , may be obtained from the expression,  $P_{ult} = p_{ult} \cdot L \cdot D$ . This represents the ultimate load at which the pile fails and will not support any additional load at greater displacements.

### 8.3 Permanent Anchors

If the foundations require seismic upgrade, the use of rock anchors is considered feasible to provide uplift resistance. The anchorage should be developed within sound shale bedrock.

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. Anchors providing resistance only during seismic events are considered to be passive anchors. Accordingly, it is recommended that each anchor be proof tested to 100% of the factored ULS design load, then unloaded to a lock-off value of 20% of the design value.

The rock anchors should be provided with double corrosion protection.

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

#### **8.4 Frost Cover**

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

It is recommended that all footings be provided with a minimum of 1.8 m of earth cover above the underside of the footing. Frost protection is required for the shale bedrock at this site.

### **9 APPROACH EMBANKMENT WIDENING**

The existing approach embankments are approximately 6 m high. It is anticipated that the approach embankments will be widened by less than 3 m on each side to accommodate the bridge deck widening. The resulting thickness of additional fill placed over the existing 2H:1V embankment slope will be about 1.5 m.

Boreholes drilled through the approach embankments (Boreholes SLB -01, SLB-05, SLB-06 and SLB-10) indicate that the existing embankment fill consists of firm to stiff silty clay. The underlying foundation soil comprises compact to dense native silty sand till underlain by shale.

Settlement within the existing compacted clay embankment fill and underlying foundation soils due to placement of an additional 1.5 m of embankment fill is expected to be in the order of 25 mm. This settlement is expected to occur largely as the fill is constructed and within about three months after completion.

Slope stability analyses were carried out for a maximum 6 m high earth fill embankment with a side slope of 2H: 1V. The slope stability program GSLOPE developed by Mitre Software Inc. was used for the analysis. The analysis was based on Bishop's simplified method. The embankment was assessed under static and seismic loading with an acceleration of 0.16g, which is a conservative assumption as it is the value of peak acceleration at the site.

Factors of safety (FoS) of 1.3 and 1.5 are considered acceptable for static assessment of short term and long term conditions, respectively, in cohesive soils. A FoS greater than 1.0 is considered adequate for seismic assessment.

The input parameters and soil model used in the analyses are shown in Figures E1 to E3 in Appendix E along with the results. The results indicate a FoS of 1.6 and 1.5 for short term and long term static conditions, respectively, and a FoS of about 1.1 for seismic conditions. The stability of the widened embankment is therefore considered acceptable.

Embankment construction and widening must be in accordance with OPSS 206. The existing embankment must be benched in accordance with OPSD 208.010 prior to placement of new fill. Disturbed or re-graded earth slopes must be provided with erosion protection in accordance with OPSS 804.

## 10 ABUTMENT BACKFILL AND LATERAL EARTH PRESSURES

Backfill to the abutment walls should be in accordance with OPSS 902. Granular backfill should be placed to the extents shown in OPSD 3101.150. The design of the abutment must incorporate a subdrain as shown in OPSD 3101.150 and OPSD 3190.100.

Backfill to the abutments should consist of Granular A or Granular B Type II material meeting the requirements of Special Provision 110S13. Compaction equipment to be used adjacent to retaining structures should be restricted in accordance with OPSS 501.

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 $\phi = 32^\circ, \gamma = 21.2 \text{ kN/m}^3$	
	Horizontal Surface Behind Wall	Sloping Backfill (2H:1V)	Horizontal Surface Behind Wall	Sloping Backfill (2H:1V)
Active (Unrestrained Wall)	0.27	0.38*	0.31	0.46*
At Rest (Restrained Wall)	0.43	-	0.47	-
Passive (Movement towards soil mass)	3.7	-	3.3	-

\* For wing walls if applicable.

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.

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 extension of the pier footings is expected to encounter the existing pavement structure, sand fill, sand and shale bedrock. At the abutments, excavation will extend through a pavement structure, abutment backfill, silt and sand fill, sand, silty sand till and shale.

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 above the water table and Type 4 below water table.

The excavation and backfilling for foundations must be carried out in accordance with OPSS 902. Construction methodology and design of excavation shoring is the responsibility of the Contractor.

At all footings, excavation into the shale bedrock will be required to remove the upper highly weathered shale and match the founding level of the existing footings. Equipment must be supplied to excavate up to approximately 1.6 m below the shale surface without disturbing/fracturing the adjacent bedrock or undermining the existing footings. At the abutments, equipment suitable for excavating dense silty sand till with possible cobbles, boulders and shale slabs should be provided.

Suggested wording for a NSSP on excavation of the glacial till and shale bedrock is provided in Appendix F.

The groundwater level was measured at Elev. 63.8 to 65.4 at the approaches and at Elev. 63.5 to 65.2 along St. Laurent Blvd. Based on these water level measurements, excavation is expected to extend below the water level within cohesionless sand fill, sand and silty sand till. Excavation below the groundwater level without prior dewatering is not recommended due to the potential for instability and “flowing” of saturated material into the excavation.

Selection of the dewatering system is the responsibility of the Contractor. However, pumping from perimeter wells extended to the shale surface to draw down the water prior to excavation, along with sump pumping from within the excavation to remove any seepage accumulating on the founding surface, may be a suitable methodology. Suggested wording for a NSSP on excavation dewatering is provided in Appendix F.



## 12 ROADWAY PROTECTION

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

It is the Contractor's responsibility to select a suitable roadway protection system based on his evaluation of the data presented in the Foundation Investigation report. However, it is noted that installation of sheet piles or driven H-piles for a soldier pile and lagging system is expected to be unsuitable for excavations extended to bedrock from the level of St. Laurent Boulevard. A soldier pile and lagging system with piles set in pre-augered holes in the bedrock should be feasible for these foundation excavations.

Roadway protection will also be required on Highway 417 to facilitate conversion of the abutments to a semi-integral configuration. Excavation for this work is expected to extend to a depth of about 1.7 m below Highway 417 road grade. Use of a sheet pile shoring system or driven H-piles for a soldier pile and lagging system is expected to be feasible for these excavations.

Soil parameters for design of the roadway protection systems are provided in Section 10 for granular materials and in Table 12.1 for clay fill and silty sand till.

**Table 12.1 – Earth Pressure Coefficients (K)**

Condition	Earth Pressure Coefficient (K)	
	Silty Clay Fill $\phi = 28^\circ, \gamma = 20.0 \text{ kN/m}^3$	Silty Sand Till $\phi = 32^\circ, \gamma = 21.2 \text{ kN/m}^3$
Active (Unrestrained Wall)	0.36	0.31
At Rest (Restrained Wall)	0.53	0.47
Passive (Movement towards soil mass)	2.8	3.3

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

$$k_s = 67 * s_u / D \quad (\text{kN/m}^3)$$

$$p_{ult} = 6 * s_u \quad (\text{kPa})$$

where  $D$  = socket diameter in metres

$$s_u = 400 \text{ kPa (undrained shear strength of bedrock mass, kPa)}$$

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

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

The Contractor is responsible for the design of the roadway protection system and any dewatering system that may be required.

### 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 4.4.6.1 of the CHBDC, a Site Coefficient “S” (ground motion amplification factor) of 1.0 should be used in seismic design.

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

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

where:

- $p_{he}$         =        horizontal pressure on the wall at depth  $h$  (kPa)
- $K$             =        earth pressure coefficient (see Table 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 $\phi = 32^\circ, \gamma = 21.2 \text{ kN/m}^3$		Native Silty 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 CONSTRUCTION CONCERNS

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

- The soils on site include glacial till consisting of silty sand. The glacial till is dense to very dense and may contain cobbles, boulders and shale slabs. Excavation of the till may prove arduous and equipment capable of excavating very dense material and removing cobbles, boulders and shale slabs will be required.
- Excavation for foundation construction will require excavation of shale bedrock to achieve the required footing founding levels. Shale excavation must be carried out in a manner that does not disturb or fragment the shale adjacent to the excavation or undermine the existing overpass foundations.
- Exposed shale on the founding surfaces should be inspected, approved and protected within 4 hours to prevent softening of the shale.

Monitoring of the existing superstructure for movements is recommended during footing extension works to identify potential movement due to construction activities. The monitoring program and tolerance criteria should be established by the structural designer, but as a minimum should include monitoring of both vertical and lateral movements of the existing pier and abutments, and deck movement during adjacent footing construction. 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.

## 15 CLOSURE

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

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## **Appendix A**

### **Record of Borehole Sheets**



## SYMBOLS, ABBREVIATIONS AND TERMS USED ON RECORDS OF BOREHOLES

### 1. TEXTURAL CLASSIFICATION OF SOILS

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

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

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

### 3. TERMS DESCRIBING CONSISTENCY (COHESIVE SOILS ONLY)

DESCRIPTIVE TERM	UNDRAINED SHEAR STRENGTH (kPa)	APPROXIMATE SPT <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		Field Estimation of Hardness*
			(MPa)	(psi)	
Very thickly bedded	Greater than 2m	Extremely Strong	Greater than 250	Greater than 36,000	Specimen can only be chipped with a geological hammer
Thickly bedded	0.6 to 2m				
Medium bedded	0.2 to 0.6m	Very Strong	100-250	15,000 to 36,000	Requires many blows of geological hammer to break
Thinly bedded	60mm to 0.2m				
Very thinly bedded	20 to 60mm	Strong	50-100	7,500 to 15,000	Requires more than one blow of geological hammer to break
Laminated	6 to 20mm				
Thinly Laminated	Less than 6mm	Medium Strong	25.0 to 50.0	3,500 to 7,500	Breaks under single blow of geological hammer.

TERMS					
Total Core Recovery: (TCR)	Core recovered as a percentage of total core run length.	Weak	5.0 to 25.0	750 to 3,500	Can be peeled by a pocket knife with difficulty
Solid Core Recovery: (SCR)	Percent Ratio of solid core of full cylindrical shape recovered. Expressed with respect to the total length of core run.	Very Weak	1.0 to 5.0	150 to 750	Can be peeled by a pocket knife, crumbles under firm blows of geological pick.
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.	Extremely Weak (Rock)	0.25 to 1.0	35 to 150	Indented by thumbnail
Uniaxial Compressive Strength (UCS)	Axial stress required to break the specimen				
Fracture Index: (FI)	Frequency of natural fractures per 0.3m of core run.				

## METRIC

SOIL PROFILE						DYNAMIC CONE PENETRATION RESISTANCE PLOT		ELEVATION SCALE	SHEAR STRENGTH kPa	WATER CONTENT (%)	UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS					$\gamma$	GR SA SI CL
72.8 0.0	ASPHALT: (150mm)	[Solid Black]									kN/m <sup>3</sup>	
0.2  72.0 0.8	SAND, some gravel, some silt Brown Moist (FILL)  Silty CLAY, some sand, trace gravel Firm to Stiff Greenish Brown to Grey Moist (FILL)	[Cross-hatch pattern]	1	AS					O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL X LAB VANE			
			2	SS	7			72				
			3	SS	11			71				0 13 23 64
			4	SS	10			70				
			5	SS	11			69				
			6	SS	13			68				
			7	SS	10			67				
								66				
65.5 7.3	Silty SAND, some clay, trace gravel Compact Dark Grey Wet (TILL)  Difficult augering at 8.5m	[Dotted pattern]	8	SS	27			65				4 52 31 13
64.0 8.8	SHALE, slightly weathered to fresh, laminated, blue-grey, very thin limestone interbeds through out Limestone (25mm) at 9.3m	[Diagonal hatch pattern]	1	RUN				64			FI	RUN #1 TCR=100% SCR=100% RQD=100% UCS=17MPa (Average)
								63			3 2 1 0	

(%) STRAIN AT FAILURE

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RECORD OF BOREHOLE No SLB-01

2 OF 2

METRIC

W.P. 4320-06-00 LOCATION N 5 031 492.2 E 372 449.1 ORIGINATED BY LPG  
 HWY 417 BOREHOLE TYPE Hollow Stem Augers/NQ Coring COMPILED BY AN  
 DATUM Geodetic DATE 2011.09.12 - 2011.09.12 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										
								20 40 60 80 100										
Continued From Previous Page							<div><div>○ UNCONFINED</div><div>● QUICK TRIAXIAL</div></div> <div><div>+ FIELD VANE</div><div>× LAB VANE</div></div>					<div><div>PLASTIC LIMIT</div><div>NATURAL MOISTURE CONTENT</div><div>LIQUID LIMIT</div></div> <div><div>w<sub>P</sub></div><div>w</div><div>w<sub>L</sub></div></div> <div>WATER CONTENT (%)</div> <div>20 40 60</div>					0	RUN #2 TCR=100% SCR=100% RQD=100% UCS=11MPa (Average)
											0							
											0							
			2	RUN							0							
											1							
60.9												2						
11.9	END OF BOREHOLE AT 11.9m. BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG AND CUTTINGS TO 0.1m THEN ASPHALT COLD PATCH TO SURFACE.																	

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

20  
15 5  
10 (%) STRAIN AT FAILURE



## METRIC

[illegible]

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

RECORD OF BOREHOLE No SLB-03

1 OF 1

METRIC

W.P. 4320-06-00 LOCATION N 5 031 489.1 E 372 499.2 ORIGINATED BY LPG  
HWY 417 BOREHOLE TYPE Hollow Stem Augers/NQ Coring COMPILED BY AN  
DATUM Geodetic DATE 2011.07.11 - 2011.07.11 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			SHEAR STRENGTH kPa												
67.3								20	40	60	80	100								
0.0	ASPHALT: (50mm)																			
66.6	SAND, medium to coarse grained, some gravel Brown Moist (FILL)		1	AS			67													
0.7																				
	SAND, medium to coarse grained, some silt, trace gravel Very Dense to Compact Grey Moist (FILL)		2	SS	66		66													
			3	SS	16															
65.0																				
2.3	SAND, trace gravel Loose Brown		4	SS	9		65													
64.6																				
2.7	Wet																			
	SHALE, slightly weathered, laminated, grey to black, very thin limestone interbeds through out						64													
	Clay seam at 3.9m		1	RUN																
							63													
	Vertical fracture (300mm)																			
			2	RUN			62													
61.2																				
6.1	END OF BOREHOLE AT 6.1m. BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG AND CUTTINGS TO 0.05m, THEN ASPHALT COLD PATCH TO SURFACE.																			

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

20  
15 5  
10 (%) STRAIN AT FAILURE

# RECORD OF BOREHOLE No SLB-04

1 OF 1

METRIC

W.P. 4320-06-00 LOCATION N 5 031 489.8 E 372 512.0 ORIGINATED BY LPG  
 HWY 417 BOREHOLE TYPE Hollow Stem Augers/NQ Coring COMPILED BY AN  
 DATUM Geodetic DATE 2011.07.12 - 2011.07.12 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			SHEAR STRENGTH kPa										
66.9								20	40	60	80	100						
0.0	<b>SAND</b> , some gravel to gravelly, some silt Dense Brown Moist (FILL)  Obstruction encountered at 1.49m (possibly concrete)		1	AS														
			2	SS	39													
65.4																		
1.5	Gravelly <b>SAND</b> , trace silt Compact Black Moist to Wet (FILL)		3	SS	15													
64.8																		
2.1	Silty <b>SAND</b> , trace clay, trace gravel Very Dense Black Moist (TILL)		4	SS	78/ 0.200													
64.3																		
2.6	<b>SHALE</b> , slightly weathered, laminated, dark grey, thin limestone interbeds through out, highly fractured		1	RUN														
			2	RUN														
61.2																		
5.7	END OF BOREHOLE AT 5.7m. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 3.05m slotted screen.  WATER LEVEL READINGS: DATE      DEPTH (m)      ELEV. (m) Jul.21/ 11    2.3            64.6 Jul.26/ 11    2.3            64.6 Aug.18/ 11    2.3            64.6 Sep.08/ 11    3.4            63.5 Sep.20/ 11    2.4            64.5 Oct.12/ 11    1.7            65.2																	

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

20  
15  
10  
5  
0  
(%) STRAIN AT FAILURE

# RECORD OF BOREHOLE No SLB-05

1 OF 2

METRIC

W.P. 4320-06-00 LOCATION N 5 031 499.7 E 372 532.8 ORIGINATED BY LPG  
 HWY 417 BOREHOLE TYPE Hollow Stem Augers/NQ Coring COMPILED BY AN  
 DATUM Geodetic DATE 2011.09.11 - 2011.09.11 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				
73.1								20 40 60 80 100	PLASTIC LIMIT w <sub>P</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	
0.0	ASPHALT: (200mm)								WATER CONTENT (%)			
0.2	SAND, some gravel, some silt Dark Grey Moist (FILL)		1	AS					○ UNCONFINED	+ FIELD VANE		
72.3									● QUICK TRIAXIAL	× LAB VANE		
0.8	Gravelly SAND Compact Dark Grey Moist (FILL)		2	SS	24							
71.6												
1.5	Silty CLAY, trace sand, trace gravel Firm to Stiff Greenish Brown to Grey Moist (FILL)		3	SS	12							
			4	SS	7							
			5	SS	12							
			6	SS	9							
67.0												
6.1	Silty SAND, some clay, trace gravel Dense Dark Grey Moist (TILL)		7	SS	49							
			8	SS	38							
64.3												
8.8	SHALE, slightly weathered, laminated, grey, occasional vertical fractures, very thin limestone interbeds through out		1	RUN								

Continued Next Page

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

20  
15  
10  
(%) STRAIN AT FAILURE



RECORD OF BOREHOLE No SLB-05

2 OF 2

METRIC

W.P. 4320-06-00 LOCATION N 5 031 499.7 E 372 532.8 ORIGINATED BY LPG  
HWY 417 BOREHOLE TYPE Hollow Stem Augers/NQ Coring COMPILED BY AN  
DATUM Geodetic DATE 2011.09.11 - 2011.09.11 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						
			2	RUN		63									1	RUN #2 TCR=100% SCR=100% RQD=52% UCS=20MPa (Average)	
						62									3		
															2		
															6		
61.2															6		
11.9	END OF BOREHOLE AT 11.9m. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 3.0m slotted screen.  WATER LEVEL READINGS: DATE    DEPTH (m)    ELEV. (m) Sep.20/ 11    8.6    64.5 Oct.12/ 11    7.7    65.4																

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

20  
15  
10  
(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No SLB-06

1 OF 2

METRIC

W.P. 4320-06-00 LOCATION N 5 031 527.5 E 372 446.0 ORIGINATED BY RK  
HWY 417 BOREHOLE TYPE Hollow Stem Augers/NQ Coring COMPILED BY AN  
DATUM Geodetic DATE 2011.08.27 - 2011.08.27 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	WATER CONTENT (%)					
72.6														
0.0	ASPHALT: (225mm)													
0.2	SAND and GRAVEL Dense to Compact Brown Dry (FILL)		1	SS	43									
			2	SS	21									
70.5														
2.1	Silty CLAY, some sand Firm Grey Damp (FILL)		3	SS	7									
			4	SS	5									
			5	SS	7									
66.2			6	SS	42									
6.4	Silty SAND, some clay, trace gravel Dense to Compact Dark Grey Moist (TILL)													
			7	SS	14									
	Auger grinding													
63.4														
9.2	SHALE, slightly weathered to fresh, thinly laminated, dark grey Rubble zone (100mm)													
	Rubble zone (150mm)		1	RUN										

Continued Next Page

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

20  
15  
10  
(%) STRAIN AT FAILURE

## METRIC

[illegible]

# RECORD OF BOREHOLE No SLB-07

1 OF 1

METRIC

W.P. 4320-06-00 LOCATION N 5 031 535.9 E 372 462.9 ORIGINATED BY LPG  
 HWY 417 BOREHOLE TYPE Hollow Stem Augers/NQ Coring COMPILED BY AN  
 DATUM Geodetic DATE 2011.07.13 - 2011.07.13 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				
67.0								20 40 60 80 100				
0.0	SILT, some sand, trace to some clay, trace gravel, some organics, shale fragments Dense Dark Grey Moist (FILL)		1	AS				○ UNCONFINED + FIELD VANE				
			2	SS	35			● QUICK TRIAXIAL × LAB VANE				
65.5								20 40 60 80 100				
1.5	SAND, some silt, some gravel, shale fragments Compact Black Moist (FILL) Trace organics		3	SS	13							19 62 16 3
			4	SS	13							
63.8	Weathered shale		5	SS	50/ 0.025							
3.2	SHALE, slightly weathered, laminated, grey, very thin limestone interbeds through out		1	RUN							FI	RUN #1 TCR=97% SCR=80% RQD=44% UCS=14MPa (Average)
											>10	
											1	
											2	
											>5	
											2	RUN #2 TCR=100% SCR=88% RQD=82% UCS=13MPa (Average)
	Calcite infilled sub-vertical fracture from 4.9m to 5.3m		2	RUN							0	
											1	
											>5	
	50mm thick clay seam at 6.1m and 6.3m		3	RUN							0	RUN #3 TCR=100% SCR=77% RQD=71% UCS=12MPa (Average)
											3	
	100mm thick clay seam at 6.5m										3	
60.0											2	
7.0	END OF BOREHOLE AT 7.0m. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 3.05m slotted screen.											
WATER LEVEL READINGS: DATE DEPTH (m) ELEV. (m) Jul.21/ 11 2.1 64.9 Jul.26/ 11 2.2 64.8 Aug.18/ 11 2.2 64.8 Sep.02/ 11 2.4 64.6 Sep.08/ 11 3.4 63.6 Sep.20/ 11 2.4 64.6 Oct.12/ 11 2.8 64.2												

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

20  
15  
10

(%) STRAIN AT FAILURE



# RECORD OF BOREHOLE No SLB-08

1 OF 1

METRIC

W.P. 4320-06-00 LOCATION N 5 031 539.2 E 372 478.9 ORIGINATED BY LPG  
 HWY 417 BOREHOLE TYPE Hollow Stem Augers/NQ Coring COMPILED BY AN  
 DATUM Geodetic DATE 2011.07.14 - 2011.07.14 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					
67.0							20 40 60 80 100						
66.3	ASPHALT: (50mm)		1	AS									
66.3	SAND, some gravel, trace silt Grey Moist (FILL)												
64.2	SAND, trace to some gravel, some silt Dense to Very Dense Grey Moist (FILL)		2	SS	66								13 66 15 6
			3	SS	42								
	Shale fragments		4	SS	68/ 0.275								
64.2	SHALE, slightly weathered to fresh, laminated, grey, very thin limestone interbeds through out		1	RUN								FI >4 3 3 0 2 1 2 >4 5	RUN #1 TCR=100% SCR=92% RQD=71% UCS=16MPa (Average)
	Fractures at 100mm intervals		2	RUN									RUN #2 TCR=100% SCR=97% RQD=52% UCS=21MPa (Average)
61.4	END OF BOREHOLE AT 5.6m. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 3.05m slotted screen.												
5.6	WATER LEVEL READINGS: DATE DEPTH (m) ELEV. (m) Jul.21/ 11 3.0 64.0 Jul.26/ 11 3.0 64.0 Aug.18/ 11 3.0 64.0 Sep.08/ 11 3.4 63.6 Sep.20/ 11 3.2 63.8 Oct.12/ 11 2.9 64.1												

## METRIC

[illegible]

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

RECORD OF BOREHOLE No SLB-10

1 OF 2

METRIC

W.P. 4320-06-00 LOCATION N 5 031 532.6 E 372 523.3 ORIGINATED BY RK  
HWY 417 BOREHOLE TYPE Hollow Stem Augers/NQ Coring COMPILED BY AN  
DATUM Geodetic DATE 2011.09.09 - 2011.09.09 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	w P      w      w L					
								● QUICK TRIAXIAL      × LAB VANE						
73.1							20 40 60 80 100			20 40 60				
0.0	ASPHALT: (175mm)													
0.2	SAND and GRAVEL Very Dense to Compact Brown Dry (FILL)		1	SS	68									
			2	SS	22									
71.0														
2.1	Silty CLAY, trace sand Firm to Very Stiff Grey (FILL)		3	SS	6									
			4	SS	7									
			5	SS	10									
	Occasional gravelly sand seams		6	SS	17									
65.8														
7.3	Silty SAND, some clay, trace to some gravel Dense Dark Brown Moist (TILL)		7	SS	39									
64.0														
9.1	SHALE, fresh, thinly laminated, horizontal, vertical and subvertical jointed, frequent rubble zone, dark grey		1	RUN										

Continued Next Page

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

20  
15  
10  
(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No SLB-10

2 OF 2

METRIC

W.P. 4320-06-00 LOCATION N 5 031 532.6 E 372 523.3 ORIGINATED BY RK  
HWY 417 BOREHOLE TYPE Hollow Stem Augers/NQ Coring COMPILED BY AN  
DATUM Geodetic DATE 2011.09.09 - 2011.09.09 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								
							20	40	60	80	100		20	40	60	
	Continued From Previous Page															
	Clay seam (50mm)		2	RUN		63										7
						62										1
						61										2
60.9	END OF BOREHOLE AT 12.2m. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 3.0m slotted screen.															
12.2	WATER LEVEL READINGS: DATE DEPTH (m) ELEV. (m) Sep.20/ 11 8.5 64.6 Oct.12/ 11 8.7 64.4															

## **Appendix B**

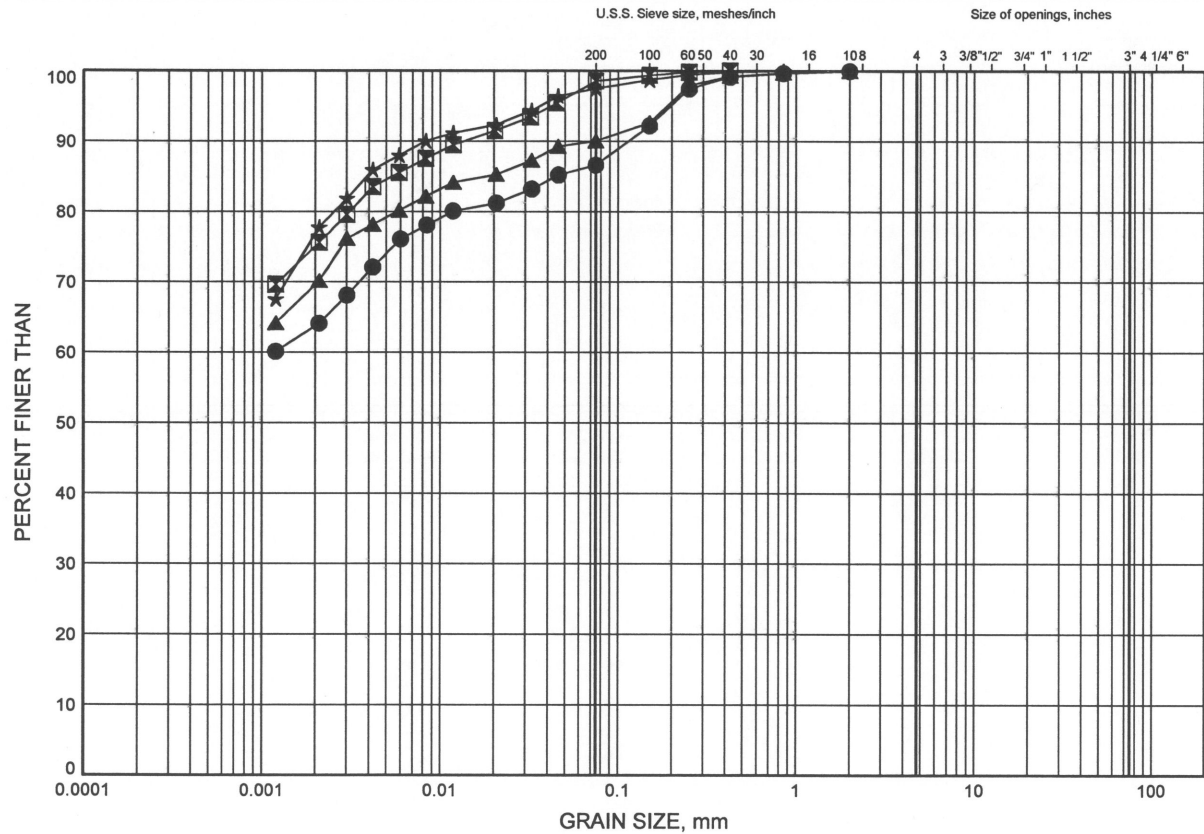
### **Laboratory Test Results**



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

FIGURE B1

## SILTY CLAY FILL



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

### LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	SLB-01	1.83	70.92
■	SLB-05	4.88	68.17
▲	SLB-06	3.35	69.21
★	SLB-10	4.88	68.18

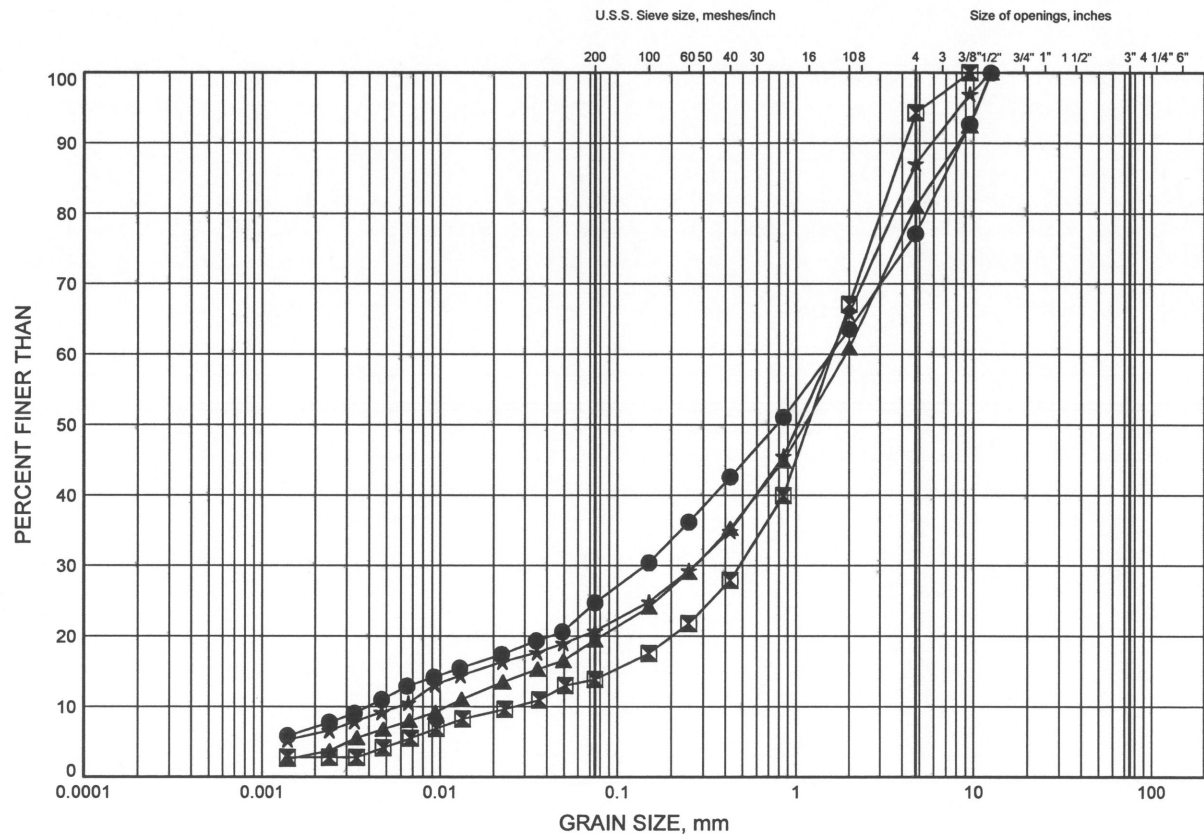


W.P.# 4320-06-00.....  
Prepared By AN.....  
Checked By LRB.....

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

FIGURE B2

## SAND FILL



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

## LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	SLB-02	0.30	66.32
⊠	SLB-03	1.83	65.51
▲	SLB-07	1.83	65.19
★	SLB-08	1.07	65.95

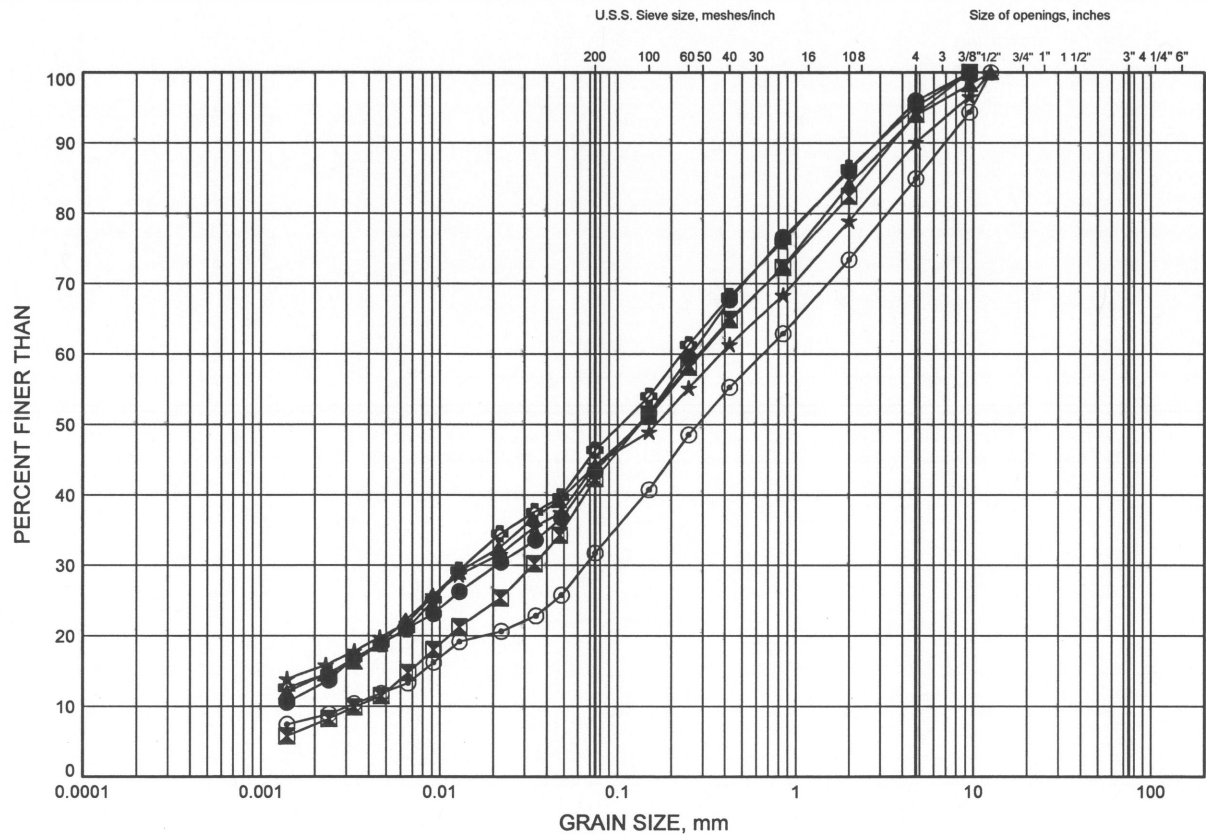


W.P.# 4320-06-00  
Prepared By AN  
Checked By LRB

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

FIGURE B3

## SILTY SAND TILL



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

### LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	SLB-01	7.92	64.82
⊠	SLB-04	2.59	64.32
▲	SLB-05	7.92	65.12
★	SLB-06	7.92	64.63
⊙	SLB-09	2.59	64.41
⊕	SLB-10	7.92	65.13

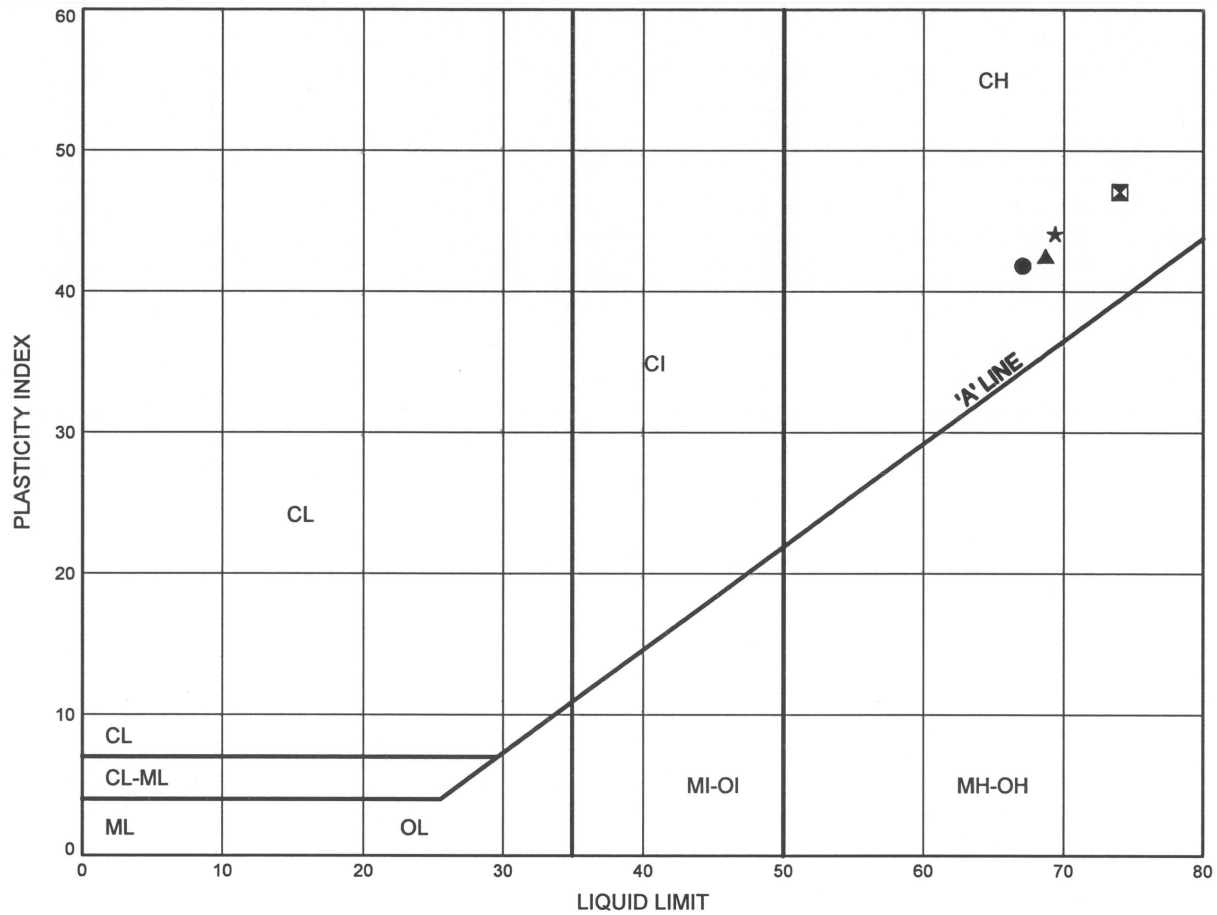


W.P.# 4320-06-00  
Prepared By AN  
Checked By LRB

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

FIGURE B4

**SILTY CLAY FILL**



SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	SLB-01	1.83	70.92
⊠	SLB-05	4.88	68.17
▲	SLB-06	3.35	69.21
★	SLB-10	4.88	68.18



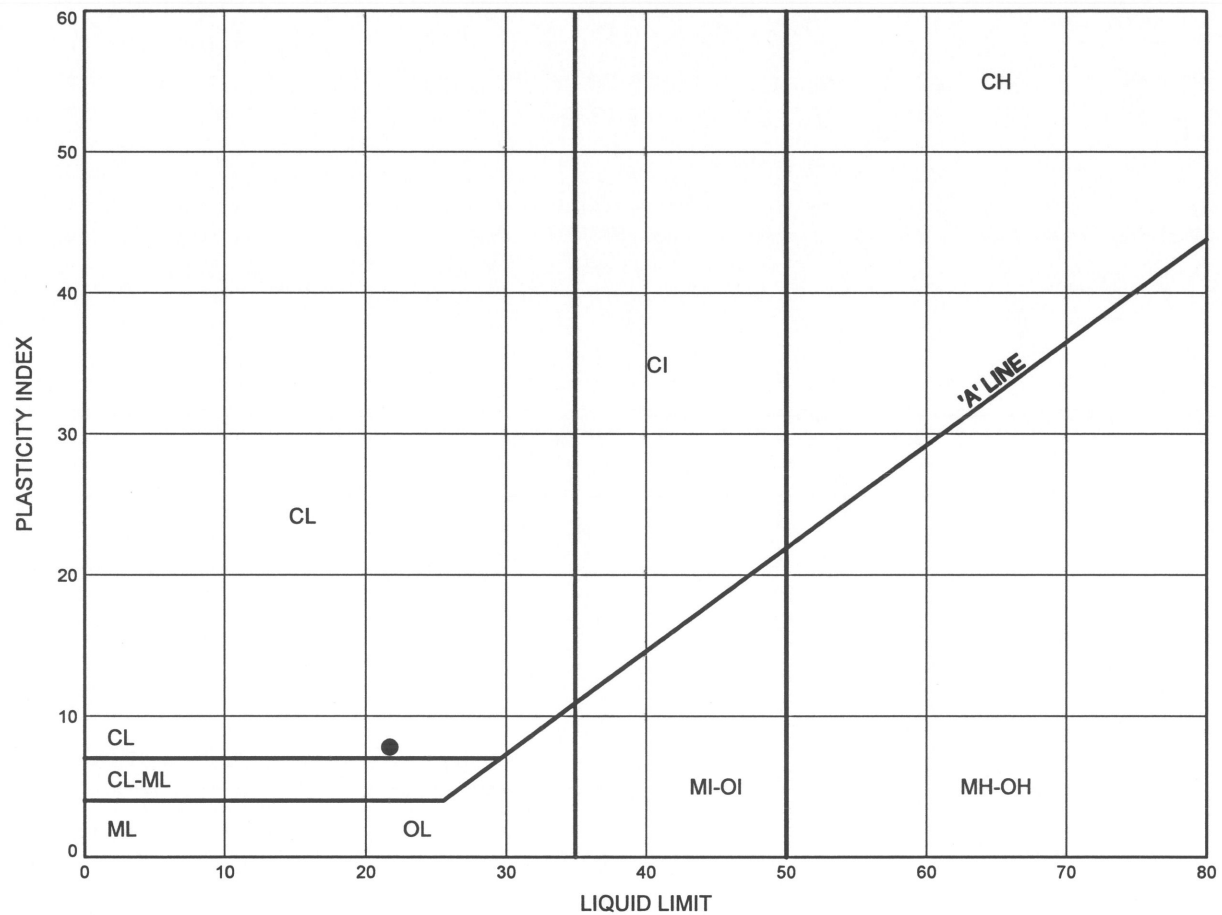
Date November 2011  
 Project 4320-06-00

Prep'd AN  
 Chkd. LRB

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

FIGURE B5

**SILTY SAND TILL**



SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	SLB-05	7.92	65.12

Date November 2011  
 Project 4320-06-00



Prep'd AN  
 Chkd. LRB



## **Appendix C**

### **Site Photographs**

St. Laurent Blvd. Overpass Rehabilitation and Widening  
Highway 417 – Ottawa, Ontario

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**Photograph 1: Looking east on north side of St. Laurent Blvd. Overpass**



**Photograph 2: Looking south under St. Laurent Blvd. Overpass**



St. Laurent Blvd. Overpass Rehabilitation and Widening  
Highway 417 – Ottawa, Ontario

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**Photograph 3 : Southwest abutment**



**Photograph 4 : Northeast abutment**

## **Appendix D**

### **Comparison of Foundation Alternatives**



### COMPARISON OF FOUNDATION ALTERNATIVES FOR PIER

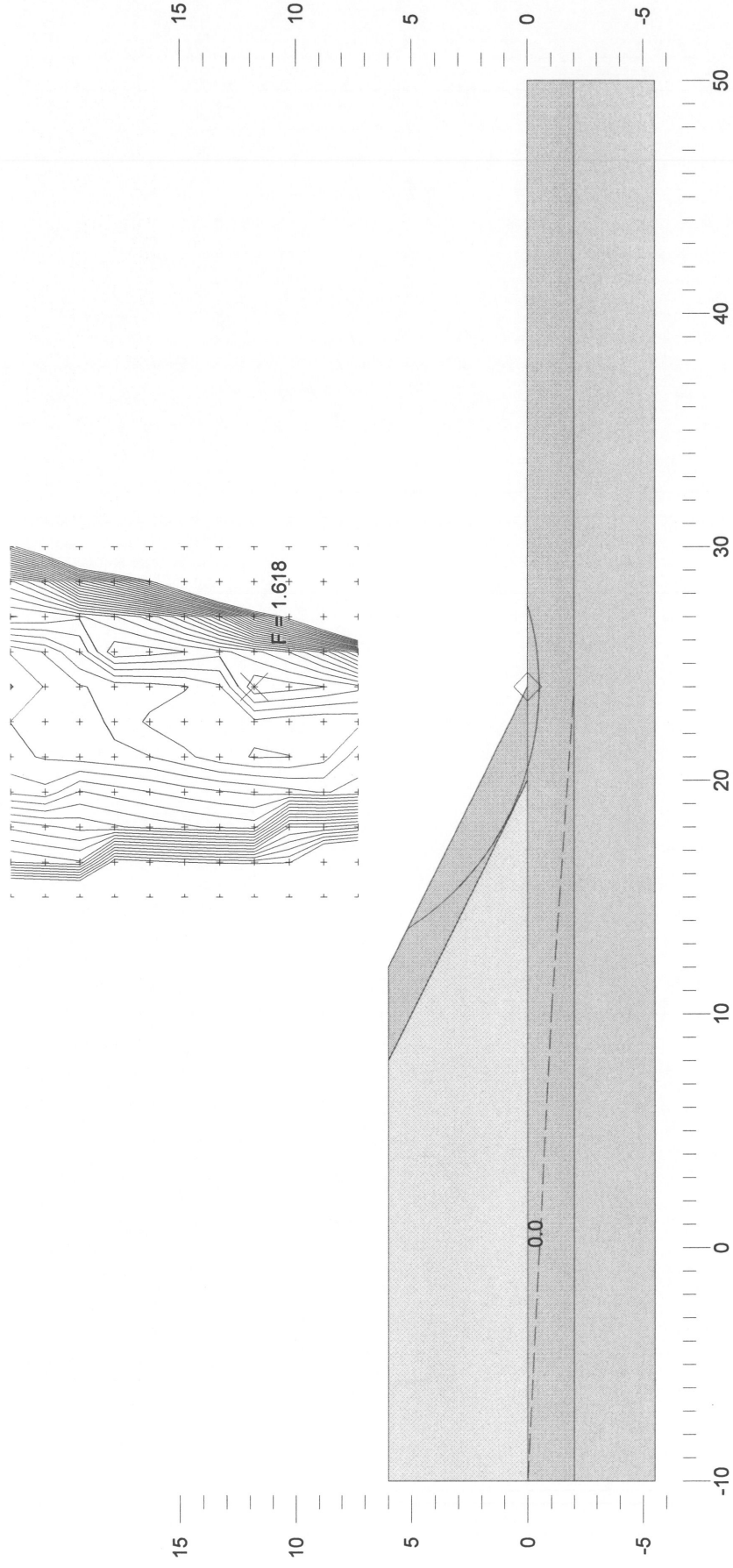
Footings on Rock	Caissons Socketed into Bedrock	Steel H-Piles
<p><b>Advantages:</b></p> <ul style="list-style-type: none"> <li>i. High geotechnical resistance available.</li> <li>ii. Economical to construct.</li> <li>iii. Matches existing foundation type.</li> </ul>	<p><b>Advantages:</b></p> <ul style="list-style-type: 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 rock not required, reducing roadway protection costs.</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 rock not required.</li> </ul>
<p><b>Disadvantages:</b></p> <ul style="list-style-type: none"> <li>i. Anticipated higher costs for roadway protection than for deep foundations, given the need for socketing of soldier piles.</li> <li>ii. Relatively deep excavation and possible dewatering required.</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. Potential difficulty in unwatering, cleaning and inspecting bases.</li> <li>ii. Generally higher unit cost compared to other foundation options, subject to roadway protection requirements.</li> </ul>	<p><b>Disadvantages:</b></p> <ul style="list-style-type: none"> <li>i. Impractical for shallow depth to rock.</li> <li>ii. Piles will need to be socketed into bedrock.</li> <li>iii. Concrete placement may require tremie pipe.</li> <li>iv. Higher unit cost compared to footings.</li> </ul>
<b>RECOMMENDED</b>	<b>FEASIBLE</b>	<b>NOT RECOMMENDED</b>



## **Appendix E**

### **Slope Stability Analyses**

	Gamma C	Phi	Piezo
	kN/m <sup>3</sup>	kPa	deg Surf.
Fill (Granular)	21.2	0	32 1
Fill (Clayey Silt)	19.5	40	0 1
Sand/Till	21	0	33 1
Bedrock	23	0	45 1



Thurber Engineering Ltd. - Toronto  
 19-1351-201  
 St Laurent Avenue  
 Nov 2011  
 6m Emb. Long Term

	Gamma C	Phi	Piezo
	kN/m <sup>3</sup>	kPa	deg Surf.
Fill (Granular)	21.2	0	32 1
Fill (Clayey Silt)	19.5	2	28 1
Sand/Till	21	0	33 1
Bedrock	23	0	45 1

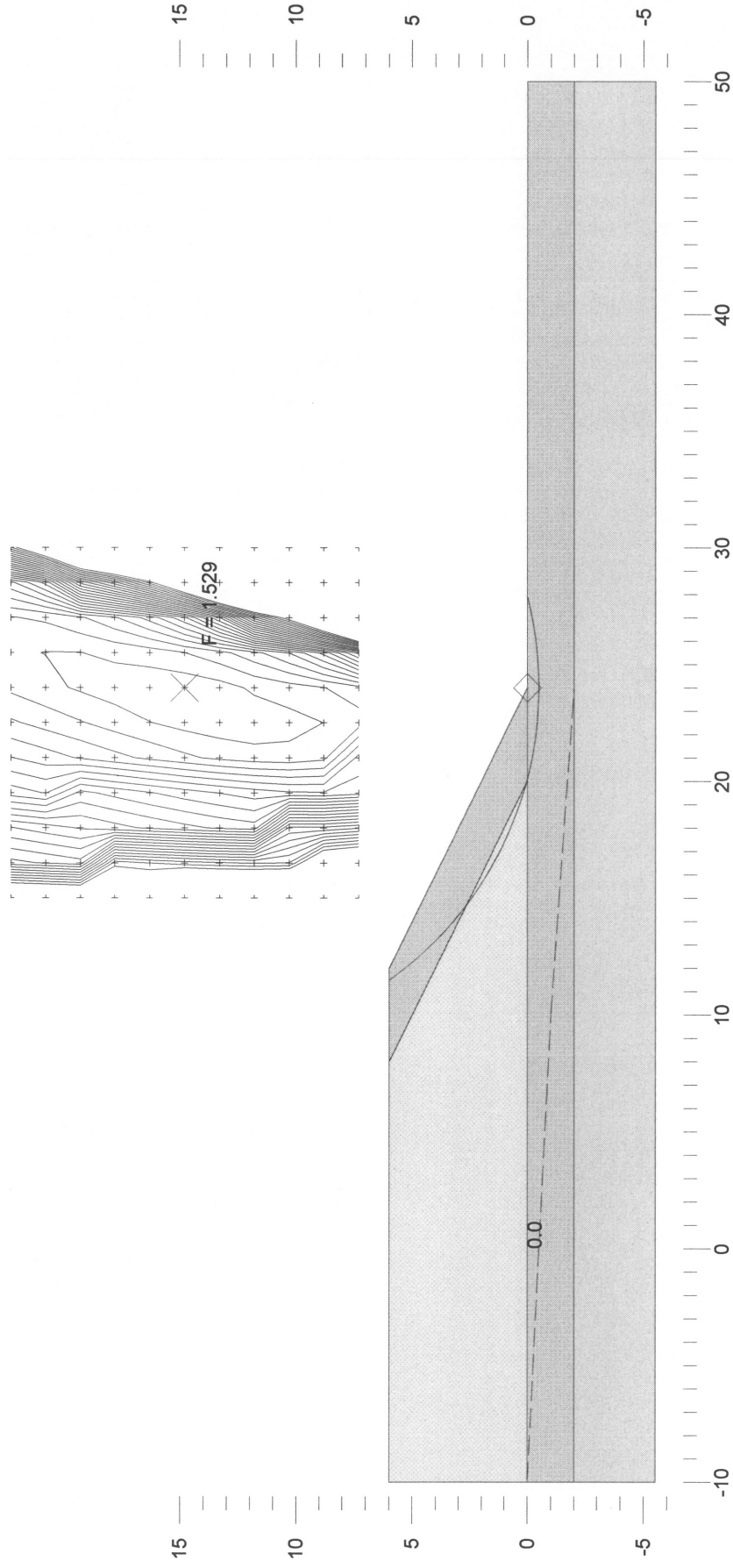


Figure E2

Thurber Engineering Ltd. - Toronto  
 19-1351-201  
 St Laurent Avenue  
 Nov 2011  
 6m Emb. Long Term  
 Seismic

	Gamma C kN/m <sup>3</sup>	Phi deg	Piezo Surf.
Fill (Granular)	21.2 0	32	1
Fill (Clayey Silt)	19.5 2	28	1
Sand/Till	21 0	33	1
Bedrock	23 0	45	1

Seismic coefficient = 0.16

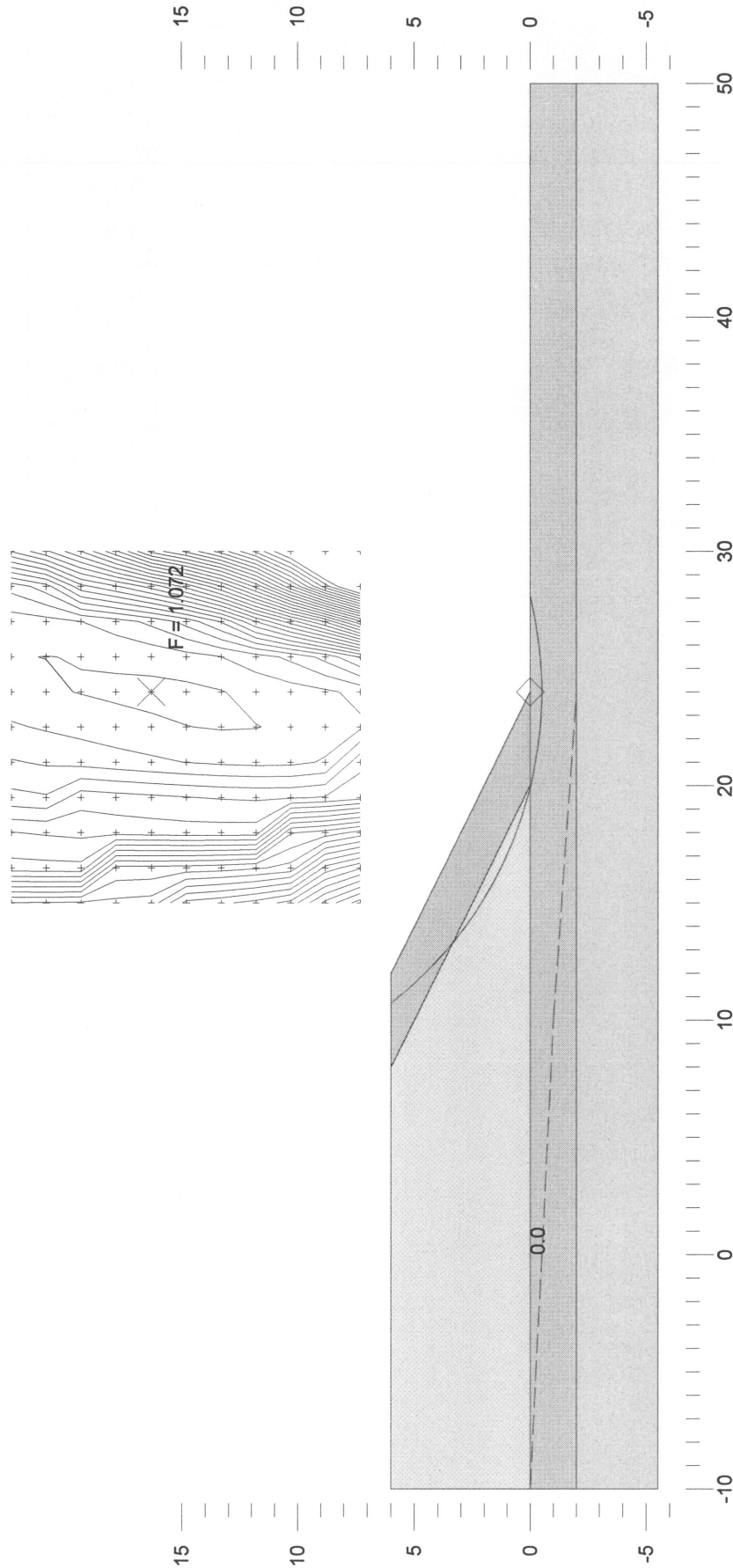


Figure E3

## **Appendix F**

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



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

- OPSS 206
- OPSS 501
- OPSS 539
- OPSS 804
- OPSS 902
- OPSS 1010
- OPSD 208.010
- OPSD 3101.150
- OPSD 3190.100
- Special Provision 110S13
- Special Provision 999S26

**2. Suggested Text for NSSP on Excavation of Glacial Till and Shale Bedrock**

The soils on site primarily consist of silty clay fill, sand fill, and glacial till consisting of silty sand. The glacial till may contain cobbles, boulders and shale slabs. Equipment capable of excavating very dense material and removing cobbles, boulders and shale slabs will be required.

Excavation for foundation construction will require excavation of shale bedrock to achieve the required footing founding levels. The shale bedrock generally increases in strength with depth and contains hard limestone interbeds. Excavation equipment capable of penetrating and removing this rock material without disturbing or fragmenting the shale adjacent to the excavation or undermining the existing overpass foundation must be supplied. Blasting is not permitted.

**3. Suggested Text for NSSP on Concrete Working Slab on Shale Bedrock**

The shale at the base of the footing excavation must be protected from deterioration by a concrete working slab. The working slab must be at least 100 mm thick and must be placed as soon as practical after completion of the excavation and in no case later than 4 hours after excavation.

**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.

**5. Suggested Text for NSSP on Excavation Dewatering**

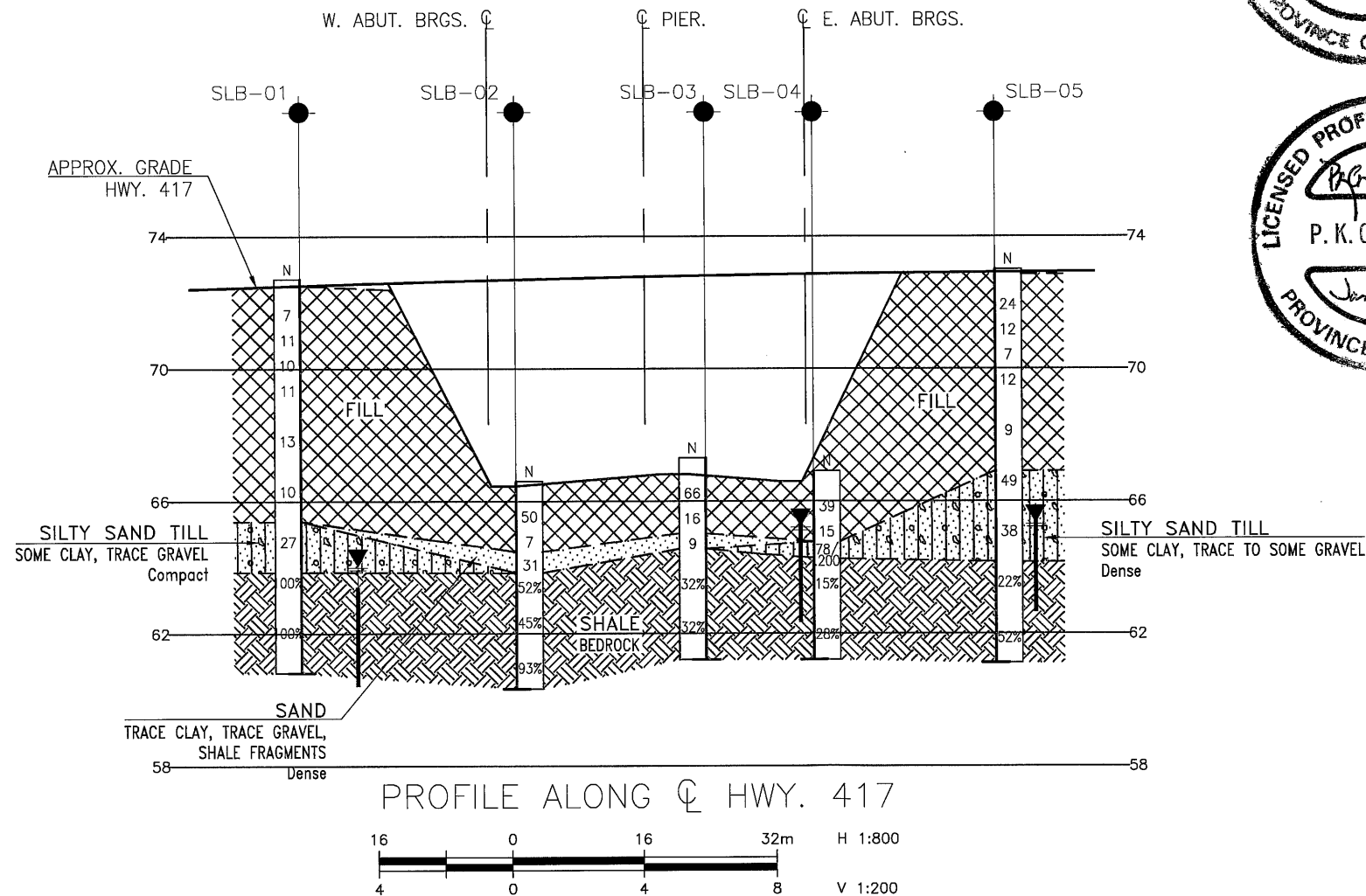
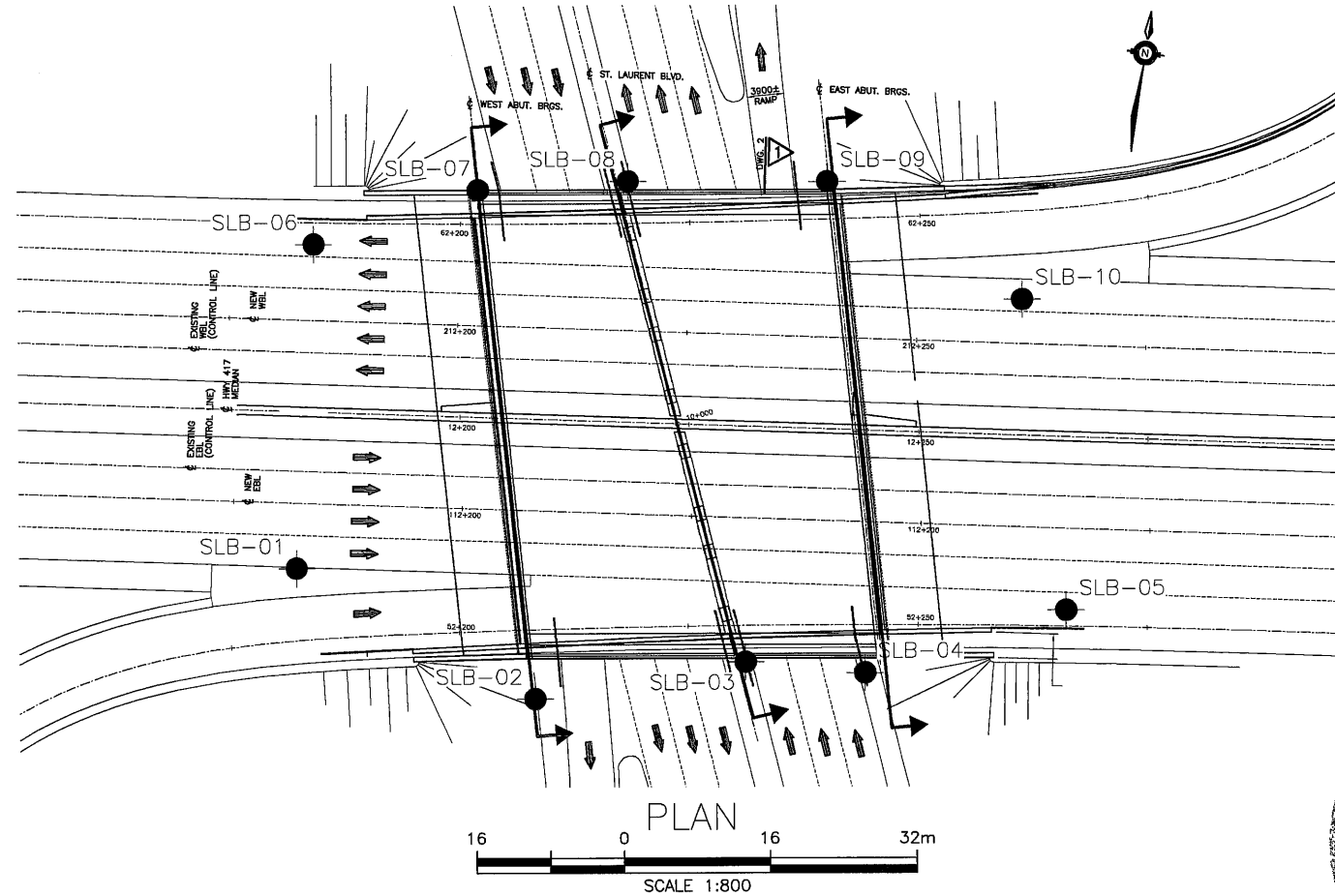
The Contractor must note that groundwater was measured at levels as high as 2.4 m above the anticipated excavation depth at this site. Excavation is expected to extend below the water level within cohesionless sand fill, silty sand till, and shale bedrock. Excavation below the groundwater level without prior dewatering is not recommended due to the potential for instability and “flowing” of saturated material into the excavation.

The Contractor must be prepared to provide a suitable dewatering system to enable excavation to the design founding level, maintain the stability of the excavation sidewalls and roadway protection, and provide a relatively dry stable base on which to construct foundations.

## **Appendix G**

### **Drawing**

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



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

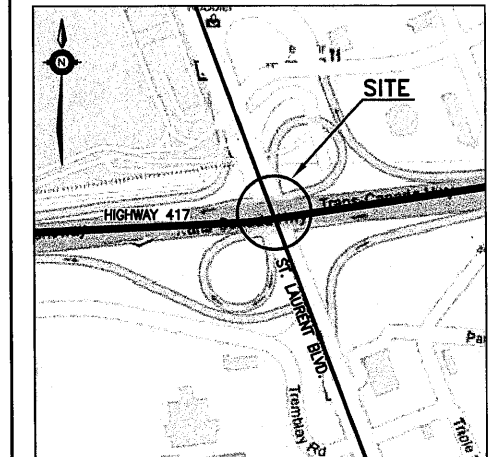


CONT No  
WP No 4320-06-00

ST. LAURENT BLVD.  
OVERPASS  
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
- W Water Level
- HA Head Artesian Water
- PZ Piezometer
- 90% Rock Quality Designation (RQD)
- A/R Auger Refusal

NO	ELEVATION	NORTHING	EASTING
SLB-01	72.8	5 031 492.2	372 449.1
SLB-02	66.6	5 031 481.8	372 476.8
SLB-03	67.3	5 031 489.1	372 499.2
SLB-04	66.9	5 031 489.8	372 512.0
SLB-05	73.1	5 031 499.7	372 532.8
SLB-06	72.6	5 031 527.5	372 446.0
SLB-07	67.0	5 031 535.9	372 462.9
SLB-08	67.0	5 031 539.2	372 478.9
SLB-09	67.0	5 031 542.3	372 500.4
SLB-10	73.1	5 031 532.6	372 523.3

**-NOTES-**

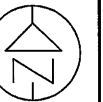
- The boundaries between soil strata have been established only at Borehole locations. Between Boreholes the boundaries are assumed from geological evidence.
- This drawing is for subsurface information only. Surface details and features are for conceptual illustration.

**GEOCRES No. 31G5-243**

REVISIONS	DATE	BY	DESCRIPTION
DESIGN	LRB	CHK	LRB
DRAWN	AN	CHK	SITE
LOAD	DATE	JAN. 2012	DWG 1

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

CONT No  
WP No 4320-06-00



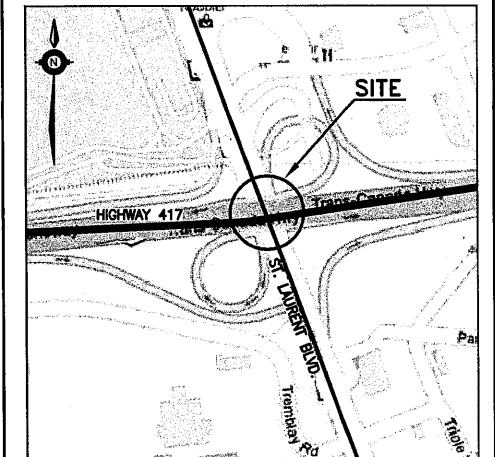
ST. LAURENT BLVD.  
OVERPASS  
BOREHOLE LOCATIONS AND SOIL STRATA

SHEET

MRC McCORMICK RANKIN  
CORPORATION



THURBER ENGINEERING LTD.



KEYPLAN

LEGEND

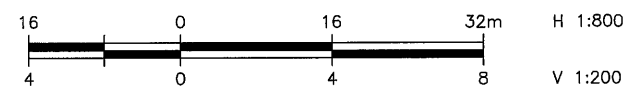
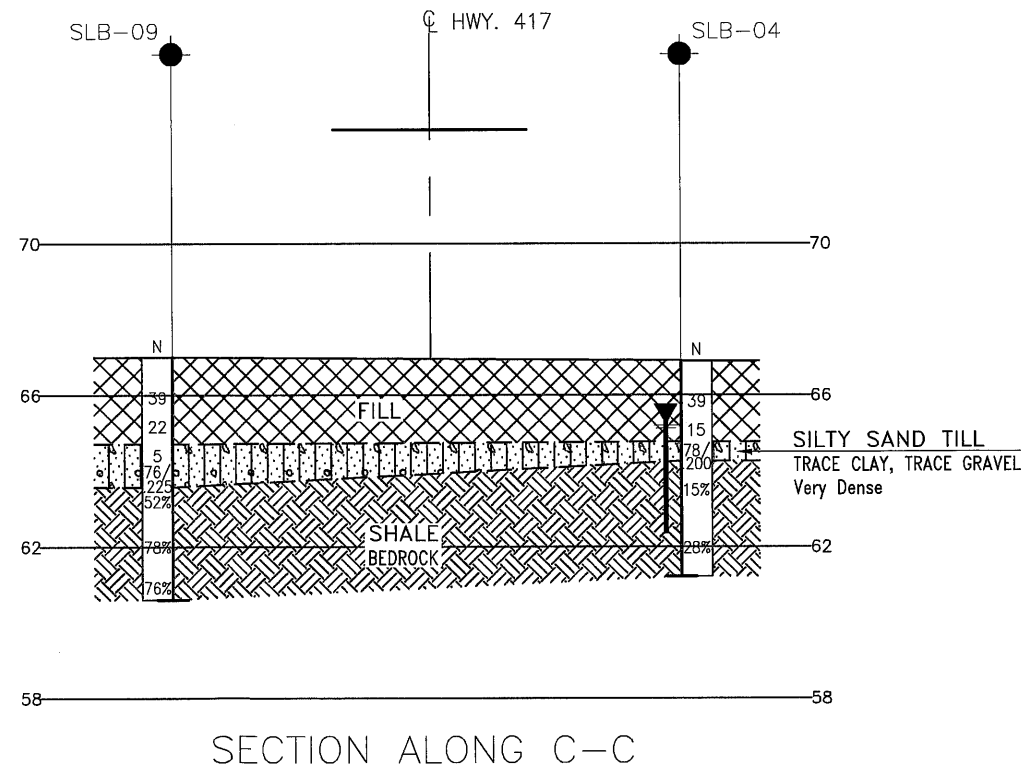
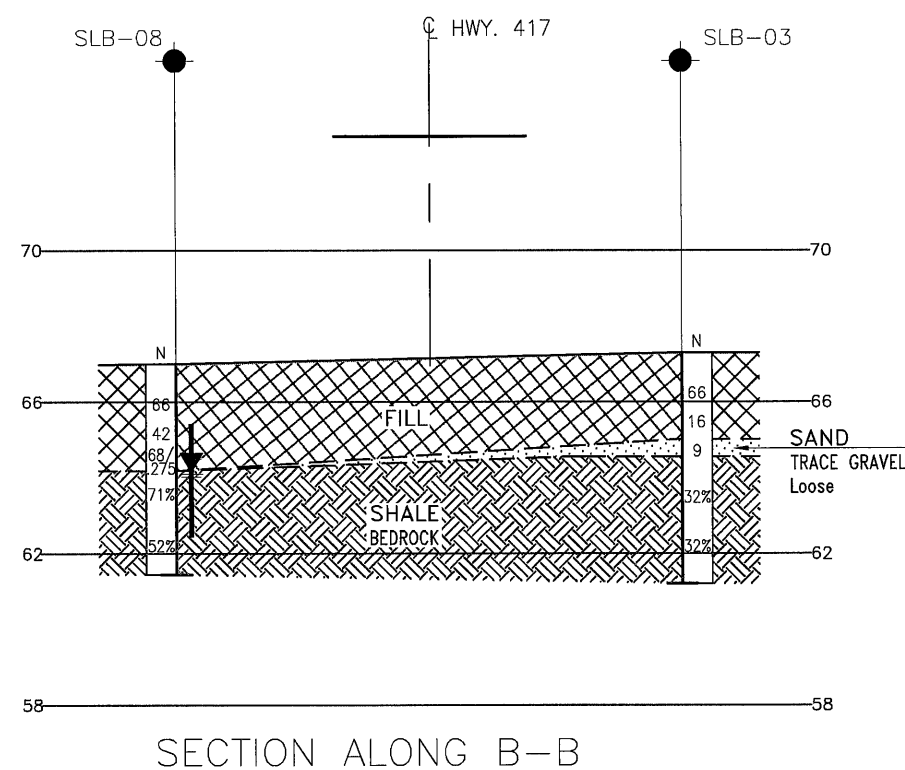
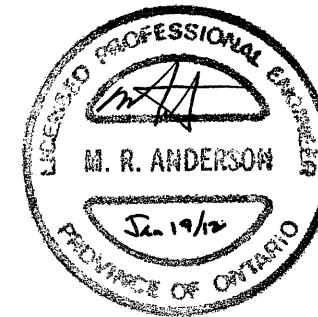
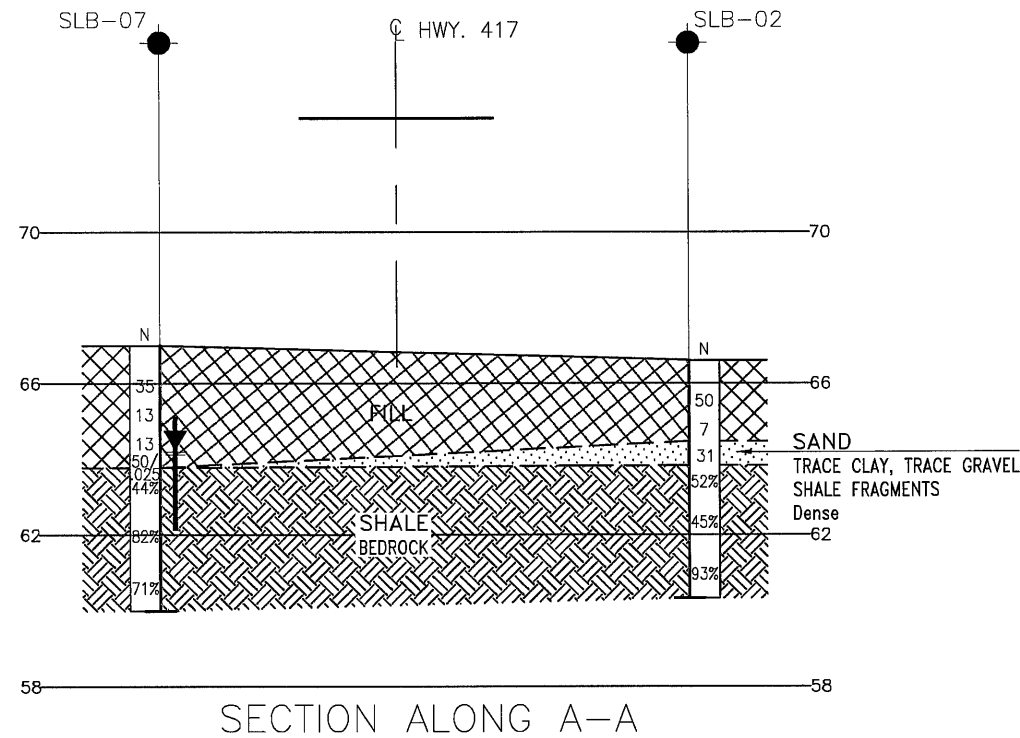
- Borehole
- ⊕ Borehole and Cone
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- CONE Blows /0.3m (60° Cone, 475J/blow)
- PH Pressure, Hydraulic
- ⊕ Water Level
- ⊕ Head Artesian Water
- ⊕ Piezometer
- 90% Rock Quality Designation (RQD)
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GEOCRES No. 31G5-243



REVISIONS	DATE	BY	DESCRIPTION
DESIGN	LRB	CHK	LRB
DRAWN	AN	CHK	SITE
LOAD	STRUCT	DWG	2
DATE	JAN. 2012		