

**FOUNDATION INVESTIGATION AND DESIGN REPORT
HURDMAN BRIDGE REPLACEMENT
HIGHWAY 417 EXPANSION FROM NICHOLAS STREET TO VANIER PARKWAY
OTTAWA, ONTARIO**

G.W.P. 4091-07-00, SITE No. 3-073

Geocres Number: 31G5-245

Report to

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TABLE OF CONTENTS

PART 1 FACTUAL INFORMATION

1	INTRODUCTION	1
2	SITE DESCRIPTION	1
3	SITE INVESTIGATION AND FIELD TESTING	2
4	LABORATORY TESTING	5
5	DESCRIPTION OF SUBSURFACE CONDITIONS	5
5.1	Asphalt	6
5.2	Topsoil	6
5.3	Sand/ Gravelly Sand/ Sand and Gravel Fill	6
5.4	Silt.....	7
5.5	Sand and Gravel.....	8
5.6	Silty Sand Till	8
5.7	Shale Bedrock	9
5.8	Water Levels	11
6	MISCELLANEOUS	12

PART 2 ENGINEERING DISCUSSION AND RECOMMENDATIONS

7	GENERAL.....	13
8	FOUNDATION DESIGN	14
8.1	Spread Footings Bearing on Bedrock	14
8.2	Existing Utility Trenches	16
8.2.1	Trenches to the North	16
8.2.2	Trench to the South.....	17
8.3	Uplift Resistance.....	17
8.4	Scour Protection.....	18
8.5	Frost Cover.....	18
9	APPROACH EMBANKMENT WIDENING	18
10	ABUTMENT BACKFILL AND LATERAL EARTH PRESSURES.....	19
11	EXCAVATION AND GROUNDWATER CONTROL	21
12	ROADWAY PROTECTION.....	22

13	SEISMIC CONSIDERATIONS	23
14	CONSTRUCTION CONCERNS	25
15	CLOSURE	26

Appendices

Appendix A	Record of Borehole Sheets
Appendix B	Laboratory Test Results
Appendix C	Site Photographs
Appendix D	List of SPs and OPSS, and Suggested Text for Selected NSSPs
Appendix E	Slope Stability Analysis
Appendix F	Drawing titled “Borehole Locations and Soil Strata”

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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 replacement of the existing Hurdman Bridge which carries Highway 417 over the Rideau River in Ottawa, Ontario. This structure replacement is part of the Highway 417 Expansion project, from Nicholas Street to Vanier Parkway.

The purpose of this investigation was to explore the subsurface conditions at the site and, based on the data obtained, to provide a borehole location plan, 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 Hurdman Bridge is located on Highway 417 approximately 4 km east of Ottawa city centre. The structure is a five span bridge with a total length of approximately 157 m, which crosses the Rideau River. The bridge deck is approximately 36.8 m wide and carries eight lanes of traffic for Highway 417. The bridge is supported on four piers and two abutments. The piers are numbered Pier 1 to Pier 4 from west to east. The piers and abutments are on shallow footings founded on bedrock. Approach embankments on either side of the bridge are approximately 5 m high.

A paved pathway for both pedestrians and cyclists runs under the existing bridge in front of each abutment. A gas main (300 mm diameter) runs parallel to the north side of the bridge and a watermain (1220mm diameter) runs in a east-southeast direction across the river on the south side

of the bridge. A 900 mm diameter storm sewer runs parallel to the south side of the west approach and outlets in the Rideau River. There is also a pedestrian bridge approximately 40m to the south of Hurdman Bridge.

In the mid to late 1980s, the Hurdman Bridge was widened from a six lane deck to an eight lane deck. Archive information suggests that during construction of the works in the mid-1980s, the gas main was relocated further north to its current location and its previous trench abandoned. The original gas main trench had an invert level approximately 1.2 to 1.8 m below the existing pier footing. A site instruction was issued to excavate the trench and backfill it with concrete whenever widening of footings at that time came within 2 m of the trench.

Topography across the site is generally flat. The Rideau River flows from south to north and is approximately 150 m wide at this site. The river is typically shallow at this location, being less than 1.5 m deep. A small island exists in the river just south of Pier 3.

Land use surrounding the site is commercial/industrial in the northeast and northwest quadrants. In the southwest quadrant are educational institutions, and the southeast quadrant consists of undeveloped parklands.

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 bridge 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 C show the general nature of the site.

3 SITE INVESTIGATION AND FIELD TESTING

The site investigation and field testing for this project were carried out during the period of July 19 to September 15, 2011 and consisted of drilling and sampling twenty-one boreholes at the existing structure (Boreholes HD-01 to 16, HD-04B to 06B, HD-11B and HD-15B). Nine of the boreholes at the approaches and abutments were drilled on land, eleven of the boreholes at the piers were drilled in the Rideau River and one borehole was drilled on the small island south of Pier 3.

At the west approach, boreholes HD-01 and HD-09 were drilled on the shoulders of Highway 417 and Boreholes HD-02 and HD-10 were drilled near the river bank in front of the west abutment. At the east approach, boreholes HD-08 and HD-16 were drilled just off the shoulder of Highway 417, and Boreholes HD-07, HD-15 and HD-15B were drilled in front of the east abutment by the river bank. The locations and termination depths and elevations of the twenty-one boreholes drilled at this site are listed in Table 3.1.

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

Table 3.1 – Borehole Details

Location	Borehole	Water Elevation (m)	Depth of Water (m)	Ground Elevation (m)	Termination Depth (m)*	Termination Elevation (m)
West Approach	HD-01	-	-	61.5	10.4	51.1
	HD-09	-	-	60.6	9.5	51.0
West Abutment	HD-02	-	-	56.9	5.7	51.2
	HD-10	-	-	57.5	6.7	50.8
Pier 1	HD-03	55.1	0.4	54.7	3.5	51.2
	HD-11	55.3	0.3	55.0	5.7	49.3
	HD-11B	55.1	0.2	54.9	3.3	51.6
Pier 2	HD-04	55.1	0.4	54.7	3.4	51.3
	HD-04B	55.1	0.4	54.7	3.5	51.2
	HD-12	55.0	0.2	54.8	3.4	51.4
Pier 3	HD-05	55.1	0.4	54.7	3.5	51.2
	HD-05B	-	-	55.1	3.7	51.4
	HD-13	55.2	0.4	54.8	3.5	51.3
Pier 4	HD-06	55.2	0.6	54.6	3.4	51.2
	HD-06B	55.1	0.5	54.6	3.4	51.2
	HD-14	55.1	0.5	54.6	3.4	51.2
East Abutment	HD-07	-	-	57.3	6.6	50.7
	HD-15	-	-	57.4	6.1	51.3
	HD-15B	-	-	57.6	6.1	51.5
East Approach	HD-08	-	-	60.8	9.3	51.5
	HD-16	-	-	60.5	9.1	51.4

* Below ground elevation

The borehole locations were marked in the field, where possible and utility clearances were obtained prior to commencement of drilling operations. Consent was obtained from the National Capital Commission (NCC) for boreholes drilled near the east abutment and a permit was obtained from the City of Ottawa for boreholes drilled at the west abutment. A permit was also obtained from the Rideau Valley Conservation Authority (RVCA) for borehole drilling in the Rideau River and on the island south of Pier 3.

Three different drill rigs were used to complete the field investigation at this site. A truck-mounted CME 75 drill rig was used for boreholes located on the shoulder of Highway 417 and a track-mounted CME 45 drill rig was used for boreholes located adjacent to the highway. For these two rigs, a combination of hollow-stem auger drilling techniques and NQ coring methods were used to advance the boreholes. A portable Hilti drill was used for coring the boreholes located in and adjacent to the Rideau River. The portable drill was operated from a temporary scaffolding platform set up at each borehole location over water. For boreholes located on the east and west banks of the river, the portable Hilti drill was used in conjunction with a tripod and hammer set-up.

Overburden samples were obtained at selected intervals using a split spoon sampler in conjunction with Standard Penetration Testing (SPT). A minimum 3.0 m length of bedrock core was recovered from 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 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 were not recorded in the open boreholes during onshore drilling operations since water is introduced into the borehole during coring and is not representative of groundwater conditions on site. Standpipe piezometers consisting of 19 to 40 mm diameter PVC pipe with a slotted screen were installed in seven 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 bentonite chips or pellets and asphalt cold patch at the surface, where appropriate.

Table 3.2 – Piezometer Details

Borehole	Tip Position (m)		Completion Details
	Depth	Elev.	
HD-1	9.1	52.4	Sand filter from 10.4 to 5.8m, bentonite from 5.8 to 4.6m, cuttings and bentonite mixture from 4.6 to 0.1m, then asphalt cold patch to surface. Flush-mount casing protector at surface.
HD-7	6.6	50.7	Sand filter from 6.6 to 4.7m, bentonite from 4.7 to 0.1m, then sand and grass to surface. 0.6 m stick-up.
HD-8	7.8	53.0	Sand filter from 9.3 to 3.8m, bentonite from 3.8 to 3.1m, cuttings and bentonite mixture from 3.1 to surface. 0.6 m stick-up.
HD-9	6.7	53.9	Sand filter from 9.5 to 3.4m, bentonite from 3.4 to 1.8m, cuttings from 1.8 to 0.1m, then asphalt cold patch to surface. Flush-mount casing protector at surface.
HD-10	6.7	50.8	Sand filter from 6.7 to 4.9m, bentonite from 4.9 to 0.6m, then sand and gravel to surface. 0.6 m stick-up.
HD-15B	4.7	52.9	Bentonite from 6.1 to 4.7m, sand filter from 4.7 to 1.2m, bentonite from 1.2 to 0.05m, sand to surface. 0.7 m stick-up.
HD-16	7.6	52.9	Sand filter from 9.1 to 4.0m, cuttings and bentonite mixture from 4.0 to surface. 0.6 m stick-up.

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 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 drilled on land consists of asphalt or a thin layer of organics overlying various fills, which are underlain by silty sand till. Shale bedrock was encountered below the till. The stratigraphy encountered in the boreholes drilled in the Rideau River generally consists of shale bedrock at the surface of the river bed.

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

5.1 Asphalt

Asphalt was encountered in Boreholes HD-01 and HD-09 which were drilled on the shoulders of Highway 417. The asphalt was between 125 and 150 mm thick.

Borehole HD-02 which was drilled on the pedestrian pathway that passes under the Hurdman Bridge on the west bank of the Rideau River encountered 100 mm asphalt.

5.2 Topsoil

A thin layer of topsoil was encountered at surface in Boreholes HD-08 and HD-16, which were drilled at the edge of Highway 417 and in Boreholes HD-07, HD-15 and HD-15B, which were drilled on the east bank of the Rideau River. The topsoil was typically 25 mm thick.

A layer of brown silty topsoil was also encountered at surface in borehole HD-05B, which was drilled on the small island south of Pier 3. This layer of silty topsoil contains some sand, cobbles and boulders and was 0.5 m thick.

5.3 Sand/ Gravelly Sand/ Sand and Gravel Fill

A layer of fill was encountered beneath the asphalt and topsoil in all boreholes drilled at the abutments and approaches. The fill varies from sand, gravelly sand to sand and gravel.

Sand fill was encountered below the asphalt in Boreholes HD-01 and HD-09 at the west approach, and below the topsoil in Boreholes HD-08 and HD-16 at the east approach. The sand fill is brown and contains trace to some gravel, silt and clay. Hydrocarbon odour and dark grey to black staining of the fill were noted in Boreholes HD-09 and HD-16 at approximate depths of 3.2 and 3.4 m.

Brown to dark brown sand and gravel fill was encountered below the asphalt in Borehole HD-02 and at surface in Borehole HD-10 at the west abutment. In Borehole HD-10, a layer of concrete 150 mm thick was encountered at 0.6 m depth beneath the sand and gravel fill. Beneath the concrete layer, the fill consists of brownish grey silty sand with some gravel and trace of clay.

Gravelly sand fill was encountered below the topsoil in Boreholes HD-07, HD-15 and 15B at the east abutment. The gravelly sand fill is dark brown to brown and contains some silt, shale fragments and some rootlets.

In Borehole HD-11, drilled in the Rideau River near Pier 1, gravel fill containing concrete rubble and shale fragments was encountered below 0.9 m of concrete and concrete rubble. The gravel fill was 1.4 m thick with a lower boundary at a depth of 2.6 m (Elevation

52.7 m) below the river water level. This fill is believed to be backfill material in the abandoned gas main trench.

The thickness of the sand/gravel ranged from 3.0 to 4.5 m in the approaches and 0.6 to 2.1 m at the base of the abutments. The lower boundary was at Elev. 55.4 to 57.8 m. Details of the fill thickness and elevations are summarized in Table 5.1.

Table 5.1 – Fill thickness encountered in boreholes

Foundation Element	Borehole	Top Elevation of Fill (m)	Bottom Elevation of Fill (m)	Thickness of Fill (m)
West Approach	HD-01	61.4	56.9	4.5
	HD-09	60.4	56.6	3.8
West Abutment	HD-02	56.8	55.6	1.2
	HD-10	57.5	55.4	2.1
East Abutment	HD-07	57.3	56.7	0.6
	HD-15	57.4	56.8	0.6
	HD-15B	57.6	56.4	1.2
East Approach	HD-08	60.8	57.8	3.0
	HD-16	60.5	57.1	3.4
Pier 1	HD-11	55.0	52.7	2.3

SPT ‘N’ values recorded in the cohesionless fill typically ranged from 8 to 29 blows for 0.3 m penetration, indicating a loose to compact relative density. Locally, higher SPT ‘N’ values between 31 blows/ 0.3 m penetration and 60 blows for 0.025 m penetration were recorded in Boreholes HD-01, HD-08, HD-09 and HD-15B, indicating a dense to very dense condition or possible cobbles in the fill.

The moisture contents ranged from 2 to 16%.

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 %	6 to 18
Sand %	67 to 90
Silt and Clay%	4 to 15

5.4 Silt

A thin layer of silt containing some clay, to clayey silt with some sand was encountered below the sand fill in Boreholes HD-01 and HD-16. The silt to clayey silt layer is grey to greyish brown in colour and contains trace of organics in Borehole HD-01.

The thickness of the silt layer ranged from 0.7 to 1.0 m with a lower boundary at a depth of 4.1 to 5.6 m (Elevations 56.4 to 55.8 m).

An SPT 'N' value of 6 blows for 0.3 m penetration was recorded in this layer in Borehole HD-01, indicating a firm consistency. The moisture contents measured in two samples of the clayey silt were 27 and 30%.

5.5 Sand and Gravel

A thin layer of sand and gravel was encountered below the fill layer and overlying bedrock in borehole HD-10. The sand and gravel is brownish grey and contains trace of silt and shale fragments.

The sand and gravel layer was 0.6 m thick with a lower boundary at 2.7 m depth (Elevation 54.8).

An SPT 'N' value of 17 blows for 0.3 m penetration was recorded in this layer, indicating a compact condition. The moisture content measured in a single sample of the sand and gravel was 14%.

The results of a grain size distribution analysis on a sample of sand and gravel are plotted in Figure B2, Appendix B and summarized below:

Gravel %	50
Sand %	42
Silt and Clay%	8

5.6 Silty Sand Till

Dark brown to dark grey silty sand till containing trace to some clay and trace to some gravel was encountered overlying the bedrock in all of the boreholes which were drilled on land except for Borehole HD-10.

The thickness of the silty sand till varied from 0.9 to 2.9 m. The depth to the base of the silty sand till ranged from 2.2 m to 7.3 m (Elevations 54.9 to 54.2 m).

SPT 'N' values recorded in the silty sand till typically ranged from 10 blows for 0.3 m penetration to 50 blows/0.125 m penetration, indicating a compact to very dense condition. Lower 'N' values of 6 and 9 blows for 0.3 m penetration, indicating a loose condition, were recorded in Boreholes HD-02 and HD-15. Difficult augering was experienced in Boreholes HD-01, HD-08, and HD-16, and in some cases coring was required to advance these boreholes 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 typically ranged from 6 to 16%. Two samples of the silty sand till had higher moisture contents (26% and 52%, both in Borehole

HD-02). The moisture content of 52% can likely be attributed to the presence of some organics and wood fragments in the sample.

Grain size distribution analyses were carried out on eight samples of the silty sand till. The results of these tests are plotted on Figures B3 and B4, Appendix B, and are summarized below.

Gravel %	0 to 19
Sand %	41 to 64
Silt %	24 to 45
Clay %	4 to 14

Some samples of the silty sand till exhibited plasticity to enable Atterberg Limits testing. Three Atterberg Limits tests were carried out on the silty sand till. The results of these tests are plotted on Figure B5, Appendix B and are summarized below. The results suggest slight plasticity within the till.

Plastic Limit %	12 to 16
Liquid Limit %	19 to 26
Plasticity Index %	7 to 10

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

5.7 Shale Bedrock

Bedrock was encountered below the silty sand till in all the boreholes drilled at the bridge approaches and the abutments, except for Borehole HD-10 where bedrock was encountered below the sand and gravel layer. Bedrock was encountered at the surface of the river bed in all of the boreholes drilled at the pier locations in the river except for Boreholes HD-05B (where the bedrock was encountered below topsoil on a small island), HD-11 (which penetrated probable backfilled trench) and HD-11B where 200 mm of sand and gravel was encountered above bedrock.

Bedrock was proven by coring in all boreholes drilled at this site. The depths and elevations at which bedrock was encountered are summarized in Table 5.2.

The bedrock was described as dark grey shale. Locally in Boreholes HD-01 and HD-09, hard limestone interbeds were encountered. The bedrock was generally described as slightly weathered to fresh. At some locations the bedrock was highly weathered at surface. Highly fractured/ rubble zones, typically less than 250mm thick and locally up to 600mm thick, were noted, as was clay seams of up to 15mm in thickness.

Table 5.2 – Depths and Elevations of Bedrock Surface

Location	Borehole	Weathered Bedrock Surface		Unweathered Bedrock Surface	
		Depth (m)*	Elevation (m)	Depth below Weathered Bedrock Surface (m)	Elevation (m)
West Approach	HD-01	7.3	54.2	0.0	54.2
	HD-09	6.2	54.3	0.7	53.6
West Abutment	HD-02	2.2	54.7	0.3	54.4
	HD-10	2.7	54.8	0.3	54.5
Pier 1	HD-03	0.0	54.7	0.4	54.3
	HD-11	2.3**	52.7	0.2	52.5
	HD-11B	0.2	54.7	0.5	54.2
Pier 2	HD-04	0.0	54.7	0.5	54.2
	HD-04B	0.0	54.7	0.3	54.4
	HD-12	0.0	54.8	0.8	54.0
Pier 3	HD-05	0.0	54.7	0.4	54.3
	HD-05B	0.5	54.6	0.1	54.5
	HD-13	0.0	54.8	0.6	54.2
Pier 4	HD-06	0.0	54.6	0.6	54.0
	HD-06B	0.0	54.6	0.6	54.0
	HD-14	0.0	54.6	0.5	54.1
East Abutment	HD-07	3.0	54.3	0.7	53.7
	HD-15	3.0	54.4	0.4	54.2
	HD-15B	2.7	54.9	0.4	54.5
East Approach	HD-08	5.9	54.9	0.4	54.5
	HD-16	6.1	54.4	0.1	54.3

* Excluding depth of water in river

**Presumed abandoned gas main trench

Total Core Recovery (TCR) in the bedrock ranged from 70 to 100%, but with some outlier values as low as 33%. The RQD values typically ranged from 0 to 80% indicating a variable rock quality ranging from very poor to good. An RQD value of 0% was recorded in the initial 100 to 600mm run at the bedrock surface in the majority of boreholes. The Fracture Index (FI) of the rock, expressed as fractures per 0.3 m of core, ranged from 0 to greater than 15.

The estimated unconfined compressive strength (UCS) of the rock, interpreted from point load tests conducted on intact rock cores, generally ranged from 8 to 45 MPa, indicating a

weak to medium strong strength classification. One value of 99 MPa (strong) was obtained on a sample of probable limestone from Borehole HD-05B. The variability of the rock strength is likely related to the limestone content and hard interbeds.

5.8 Water Levels

Groundwater levels were 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.

A standpipe piezometer was installed in seven boreholes at this site upon completion of drilling. The groundwater level at the approach embankments varied between 4.9 and 6.7 m depth (Elev. 53.8 and 56.6). The groundwater level measured at the river banks was between depths of 2.1 and 2.5 m (Elev. 55.0 and 55.3).

The groundwater depths and elevations measured in the piezometers are shown in Table 5.3.

Table 5.3 – Groundwater Depths and Elevations

Borehole	Date	Water Level (m)	
		Depth	Elevation
HD-1	12-Oct-2011	4.9	56.6
HD-07	2-Sept-2011	2.1	55.2
	20-Sept-2011	2.1	55.2
	12-Oct-2011	2.1	55.2
HD-08	20-Sept-2011	5.6	55.2
	12-Oct-2011	5.7	55.1
HD-09	18-Aug-2011	5.8	54.8
	12-Oct-2011	5.0	55.6
HD-10	2-Sept-2011	2.5	55.0
	20-Sept-2011	2.2	55.3
	12-Oct-2011	2.3	55.2
HD-15B	2-Sept-2011	2.3	55.3
	20-Sept-2011	2.4	55.2
	12-Oct-2011	2.5	55.1
HD-16	20-Sept-2011	5.9	54.6
	12-Oct-2011	6.7	53.8

Seasonal fluctuations of the groundwater level and the water level in the river 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.

The water level in the Rideau River at the time of investigation (July – September 2011) was approximate Elev. 55.1 m. The depth of water at the borehole locations ranged from 0.2 to 0.6 m.

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 both a truck mounted CME 75 drill rig and track mounted CME 45 drill rig and conducted the drilling, sampling and in-situ testing operations. OGS Inc. from Almonte, Ontario supplied the portable drilling and coring equipment and conducted the drilling, sampling and in-situ testing operations at the piers.

The field investigation was supervised by Mr. Ryan Kromer 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. 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 replacement of the Hurdman Bridge.

The Hurdman Bridge is a five-span structure supported by four piers and two abutments. The bridge spans a distance of approximately 157 m over the Rideau River. Based on the findings presented in Section 5 and on information shown on archive drawings, the piers and abutments are founded on footings within shale bedrock. The piers were numbered Pier 1 to Pier 4 from west to east.

In the mid-1980s, the bridge was widened from a six lane deck to an eight lane deck. The works in the mid-1980s involved widening the piers and footings both northwards and southwards by approximately 3.6 m which increased the width of the bridge deck from 29.7 m to its current width of approximately 36.8 m. The approach embankments were also widened.

A gas main running approximately parallel to the north side of the Hurdman Bridge was indicated in the archive drawings. In a MTO memo dated October 1, 1986, it was recommended that the gas main be relocated further north during the widening works and its original trench backfilled with concrete where it encroaches within 2 m of the footing extension. The current investigation indicates that this trench was backfilled with gravel, concrete and concrete rubble near Pier 1 (Borehole HD-11).

Adjacent to the south side of the Hurdman Bridge, a 1220 mm diameter watermain runs in a southeast direction from the west abutment across the Rideau River. A 900 mm diameter storm sewer runs parallel to the south side of the west approach and outlets in the Rideau River. A pedestrian bridge spans across the Rideau River approximately 40m south of Hurdman Bridge.

Replacement of the existing bridge superstructure and abutments with a wider superstructure and new wider abutments is planned. The existing piers will be rehabilitated and extended on both sides to support the new wider superstructure. The existing approach embankments are approximately 5 m high and will be widened on both sides with no raise in grade.

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. Reference must be made to the utility drawings for the locations of utility lines at the site.

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

- Extension of the pier foundations;
- Replacement of the abutments to accommodate the new superstructure;
- Widening of the approach embankments to suit the new abutments;
- Temporary roadway protection works.

8 FOUNDATION DESIGN

Each of the four existing piers at Hurdman Bridge is supported on an individual spread footing 38 m long, 2.1 m wide and 0.76 m deep. The abutments are also founded on spread footings, 38.6 m long and 0.76 m deep. Based on archive drawings, the top of the footings is indicated at elevation 55.169 with a founding level at approximately elevation 54.4, constructed over a 150 mm thick layer of mass concrete (lower boundary at elevation 54.3) within the shale bedrock.

Given the elevation of the shale bedrock and the existing construction, spread footings bearing on the relatively unweathered shale bedrock are recommended for new foundations and foundation extensions at this site. Deep foundations are not feasible on the shallow bedrock and recommendations for these have not been developed.

8.1 Spread Footings Bearing on Bedrock

It is recommended that the pier footing extensions and the new abutment footings be founded on unweathered shale bedrock below the level of all cobbles/boulders and broken or highly weathered shale. Spread footings founded in this manner may be designed for the following geotechnical resistance:

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

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

The sliding resistance of mass concrete poured on the native 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.

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 D.

For existing pier footings founding at Elevation of 54.4, excavation to the underside of the concrete working slab must extend to Elevation 54.3 or lower if necessary to accommodate the depth of weathered bedrock. In the case of the new abutment foundations where there is no need to match existing, the footing can be founded at any elevation that places it on unweathered shale.

The elevations of the unweathered shale encountered at the borehole locations and the recommended elevations for the base of excavation (corresponding to either the level of unweathered shale or the underside of the working slab beneath the existing footings, whichever is lower) are summarized in Table 8.1.

Construction of the foundations must be carried out in accordance with OPSS 902, including verification of the founding surface. In the event that the QVE determines that the founding surface is not in unweathered rock, the matter must be referred to the Contract Administrator for resolution by the designers.

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.

Table 8.1 – Depths and Elevations of Unweathered Bedrock Surface

Location	Borehole	Unweathered Bedrock Surface		Base of Excavation
		Depth below Bedrock Surface (m)	Elevation (m)	Elevation (m)
West Abutment	HD-02	0.3	54.4	54.3
	HD-10	0.3	54.5	54.3
Pier 1	HD-03	0.4	54.3	54.3
	HD-11	0.2	52.5*	See Section 8.2
	HD-11B	0.5	54.2	54.2
Pier 2	HD-04	0.5	54.2	54.2
	HD-04B	0.3	54.4	54.3
	HD-12	0.8	54.0	54.0
Pier 3	HD-05	0.4	54.3	54.3
	HD-05B	0.1	54.5	54.3
	HD-13	0.6	54.2	54.2
Pier 4	HD-06	0.6	54.0	54.0
	HD-06B	0.6	54.0	54.0
	HD-14	0.5	54.1	54.1
East Abutment	HD-07	0.7	53.6	53.6
	HD-15	0.4	54.0	54.0
	HD-15B	0.4	54.5	54.3

* Presumed abandoned gas main trench

8.2 Existing Utility Trenches

8.2.1 Trenches to the North

The gas main on the north side of the bridge was partially relocated for an earlier widening project and will be relocated again to accommodate the current widening. Northerly extension of the pier footings and the new abutment footings are likely to encounter at least one of the abandoned gas main trenches.

Borehole HD-11 appears to have been drilled in the backfill of the abandoned trench and encountered approximately 2.3 m of backfill consisting of concrete rubble, and gravel with shale fragments. It is not known if this represents the maximum depth of the trench

and no information was gathered regarding the areal extent. Considering the proximity to the existing Pier 1 footing, it is probable that this borehole encountered the backfill of the original trench and the trench containing the current gas main lies a short distance further north.

Utility markers observed in the field and a drawing received from MRC indicate that the current gas main runs parallel to the structure and passes under all the pier footing extensions and the new abutment footing. Based on this information, it must be assumed that all footings will encounter this trench north of the existing structure.

No records are available documenting the placement and composition of the existing trench backfill. Placement of footings to bear on this fill is not acceptable and it is recommended that all fill be removed to sound bedrock and the excavation be backfilled up to the design founding elevation using concrete of the same class as the footing. The concrete backfill may be placed by tremie methods.

8.2.2 Trench to the South

The preliminary GA drawings indicate that the south edge of the footing for the west abutment will be located approximately 2.6 m from the centreline of the existing 1220 mm diameter watermain and about 2 m above the watermain invert level. At Pier 1, the south edge of the extended footing will be approximately 4.3 m from the centreline of the watermain, with a similar elevation difference.

Based on information received from McCormick Rankin Corporation, the watermain is encased in concrete in the trench, with a minimum cover of 150 mm of concrete. The concrete encasement can be assumed to have maintained the integrity of the bedrock beyond the trench walls up to the level of the top of the concrete. Based on this information and to minimize the increase in stress on the pipe from the new construction, it is recommended that the edge of the new footings be kept behind a plane projected upward at 1H:1V from the near edge of the theoretical base of trench.

8.3 Uplift Resistance

Where structural analysis indicates seismic uplift to be a concern, anchorage may be provided using rock anchors or dowels grouted into the bedrock. The anchorage must be developed within sound shale bedrock.

The length of the dowel below the underside of the footing should be determined using a factored rock-grout bond strength at ULS of 200 kPa. This value includes a geotechnical resistance factor of 0.4 as per Table 6.1 of the CHBDC. The rock dowels should extend a minimum 2.0 m into the shale bedrock.

The NSSP for the installation of rock dowels is provided in Appendix D.

For rock anchors, the length of the unbonded zone below the underside of the footing should be at least 1.5 m. 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.

The rock dowels and anchors will be extended into shale bedrock which contains hard limestone interbeds. The Contractor's drilling equipment must be able to penetrate the sound bedrock and hard interbeds to achieve the design embedment length.

8.4 Scour Protection

A scour specialist must be consulted with regards to any requirement for scour protection around the pier foundations and riverbank slopes.

8.5 Frost Cover

The design depth of frost penetration at this site is 1.8 m. It is recommended that footings not within the Rideau River be provided with a minimum of 1.8 m of earth cover above the underside of the footing.

The minimum frost protection embedment is not considered to be necessary for the pier footings founded on unweathered bedrock within the Rideau River.

9 APPROACH EMBANKMENT WIDENING

The existing approach embankments are approximately 5.0 m high. Widening of the embankments by approximately 4.2 m on both sides will be required in connection with the bridge widening.

Based on boreholes drilled at the approaches (Boreholes HD-01, HD-08, HD-09 and HD-16), the foundation soils for the approach fill will generally consist of compact to very dense silty sand till overlying shale bedrock. A 0.7 to 1.0 m thick layer of firm clayey silt to silt was encountered over the till in two boreholes.

The founding soils are predominantly cohesionless and/or slightly plastic. Settlement in the foundation soils due to placement of 5 m of fill for embankment widening is estimated to be in the

order of 25 mm. The settlement is expected to be immediate and to occur essentially as the fill is constructed.

Slope stability analyses were carried out for a maximum 5 m high embankment with a side slope of 2H: 1V. The slope stability program GSLOPE developed by Mitre Software Inc. was used for the analysis with the option for Bishop's simplified method. The embankment was assessed under static and seismic loading with an acceleration of 0.16g, which is the peak horizontal ground acceleration having a probability of exceedance of 10% in 50 years.

A factor of safety (FoS) of 1.3 is considered acceptable for static assessment of both short term and long term conditions. A FoS greater than 1.0 is considered adequate for seismic assessment.

The input parameters and soil model used in the analyses are shown in Figures E-1 and E-2 in Appendix E along with the results. The results indicate a FoS of 1.6 for static conditions and 1.2 for the seismic condition. The stability of the embankment is therefore considered acceptable.

The preliminary GA drawing indicates that a slope inclination of 1.5H :1V is proposed for the forward facing slope immediately adjacent to the side of the abutments. Stability analyses were carried out for this case and the input parameters, soil models and results are presented in Figures E-3 and E-4 in Appendix E. Results indicate that the computed FoS values for this inclination are slightly below the recommended values for both static and seismic conditions, and the potential exists for surficial instability at this inclination. The use of rock fill is recommended to improve the surficial stability of the steepened slope, and the possible need for future maintenance should be anticipated.

Embankment construction and widening must be in accordance with OPSS 206. The existing embankment must be benched as per OPSD 208.010 prior to placement of new fill. To maintain drainage of the sand fill in the existing embankments, it is recommended that new fill placed to construct the widening consist of OPSS Granular B Type I material. Disturbed or re-graded earth slopes must be provided with erosion protection in accordance with OPSS 804.

A multi-use pathway will pass below the bridge between the water's edge and abutment face at both ends of the structure. It is understood that rock fill will be placed along the shoreline to construct the pathway. The rock fill should consist of hard durable rock other than shale and be placed at an inclination not exceeding 1.5H:1V.

Widening of the embankment may place additional earth loading on to the existing watermain, storm sewer and gas main which lie within the footprint of the widened embankment. The impact of the additional loads on these services must be assessed.

10 ABUTMENT BACKFILL AND LATERAL EARTH PRESSURES

Backfill to the abutment walls should be in accordance with OPSS 902 and 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.

The earth pressures acting on the abutments 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)

γ = 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 $\phi = 32^\circ, \gamma = 21.2 \text{ kN/m}^3$		Gravelly Sand 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
Active (Unrestrained Wall)	0.27	-	0.31	-	0.33
At Rest (Restrained Wall)	0.43	-	0.47	-	0.50
Passive (Movement towards soil mass)	3.7	2.1*	3.3	1.7*	3.0

* 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. For the assessment of existing abutments, the existing backfill may be assumed to comprise of OPSS Granular B Type I.

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 be limited to typically weathered shale and occasional cobbles or boulders on the river bed. Excavation of concrete rubble, concrete and gravel fill with shale fragments will be required where the abandoned gas main trench must be suitably backfilled with concrete. At the abutments, excavation through sand and gravelly sand fill, silt and/or sand and gravel layers, native silty sand till and weathered shale is expected.

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 water level and Type 4 below water level.

In general, the excavation and backfilling for foundations must be carried out in accordance with OPSS 902. Suggested wording for a NSSP on excavation of the glacial till and shale bedrock is provided in Appendix D.

Extension of the pier footings will typically require excavation to remove sediment, cobbles and boulders that may be encountered on top of the bedrock in the river and excavation of shale bedrock to the specified founding elevation. Additionally, excavation to remove the existing fill within the abandoned trench or trenches will be required along the north side of the structure.

Temporary causeways will be constructed across the river to facilitate construction of the pier footing extensions and some form of cofferdam must be incorporated at each pier to allow excavation to the founding elevation and construction of the footing. Cofferdams are temporary works to be designed and constructed by the contractor but two methods that might be appropriate are:

1. A steel cofferdam set on bedrock
2. A sandbag cofferdam.

The contract documents must alert bidders to the fact that cofferdam selection must take account of the possibility of encountering cobbles, boulders, slabs of limestone and loose sediment on top of the bedrock.

The inside dimensions of the cofferdam must be at least 2 m larger on each side than the dimensions of the pier footing to facilitate subexcavation of the abandoned trench backfill wherever it is encountered. The contract documents must contain an NSSP to flag the possibility

that subexcavation to depths in the range of 2.0 to 2.5 m below the founding elevation may be required in order to clean out the trench(es). The NSSP must also advise that the composition of the backfill has not been determined and it should be assumed that rock fragments and concrete rubble in a matrix of river sediment may be encountered.

Creation of a watertight cofferdam down through the trench is not considered practical, therefore it will have to be acceptable to construct a system to retain the material outside the footing excavation and allow the excavation to be cleaned out in a flooded condition. Hydro-vac methods could be considered for the final cleaning. Following cleaning, the subexcavation in the abandoned trench can be filled by placing concrete up to the founding level using tremie methods.

Suggested wording for a NSSP on excavation of the existing backfill in the abandoned gas main trenches and restoring with concrete is provided in Appendix D.

After the trench has been backfilled, the cofferdam should be dewatered to permit construction of the footing in the dry. Complete dewatering in areas of trench sub-excavation may not be effective in view of the locally fractured nature of the rock and the undetermined extent and position of the trench in relation to the proposed pier footing extensions. Where dewatering is found to be impractical, tremie concrete may be required to construct the pier footings.

The discharge of water from dewatering the excavations must comply with all relevant regulations. A permit to take water may be required and the discharge water may require treatment, particularly if tremie concrete is placed.

Potential hydrocarbon contaminants were encountered at approximate depths of 3.2 and 3.4 m in Boreholes HD-09 and HD-16 drilled at the west and east approaches during the current ground investigation. Site specific handling and disposal procedures of the excavated material may be required and should be established by the environmental consultant. The contract documents should alert the contract bidders of the potential presence of hydrocarbon contaminants and any requirements for special disposal and handling of potential environmentally impacted soils.

12 ROADWAY PROTECTION

Where required, 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. A soldier pile and lagging system with piles set in pre-augered holes in the bedrock should be feasible. A braced excavation or system of rakers may also be considered.

The Contractor is responsible for the design the roadway protection system and any dewatering system required.

Soil parameters for design of the roadway protection systems are provided in Section 10.

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:

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

The spring constant, K , for analysis may be obtained by the expression, $K = k_s \cdot L \cdot D$ (kN/m), where k_s is the coefficient of horizontal subgrade reaction (kN/m³), D is the socket 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 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 rock. The lateral pressures obtained from the analysis should not exceed the ultimate lateral resistance. The lateral resistance of the pile in bedrock will not be significantly affected by pile spacing.

13 SEISMIC CONSIDERATIONS

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

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

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

In accordance with Clause 4.6.4 of the CHBDC, retaining structures should be designed using earth pressure coefficients that incorporate the effects of earthquake loading. The seismic

component of the earth pressure distribution is additional to the static earth pressure distribution and may be taken as an inverted triangle with the maximum pressure at the top of the wall and the minimum pressure at the toe. The total (static plus seismic) pressure distribution for earthquake loading is therefore as follows:

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

where:

p_{he}	=	horizontal pressure on the wall at depth h (kPa)
K	=	earth pressure coefficient (see Table 10.1)
ΔK_E	=	seismic earth pressure coefficient (see Table 13.1)
γ	=	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 (ΔK_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 (ΔK_E)				
	OPSS Granular A or OPSS Granular B Type II $\phi = 35^\circ, \gamma = 22.8 \text{ kN/m}^3$		OPSS Granular B Type I $\phi = 32^\circ, \gamma = 21.2 \text{ kN/m}^3$		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 (ΔK_{AE})*	0.07	0.22	0.07	0.23	0.08
At Rest (ΔK_{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 following:

- 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 unduly disturb or fragment the shale adjacent to the excavation or undermine the existing bridge foundations.
- Exposed shale on the founding surfaces should be inspected, approved and protected within 4 hours to prevent softening of the shale.
- Care must be taken to avoid disturbance to the existing underground utilities located within close proximity to the footing extensions and approach embankments.
- If the edge of the abandoned gas main trench is encountered within 2 m of the edge of the footing, that portion of the trench within the 2 m limit must be subexcavated and backfilled with concrete. Concrete and concrete rubble may be encountered during subexcavation of the trench backfill.
- Complete dewatering in areas of trench sub-excavation may not be effective in view of the potentially fractured nature of the rock and the undetermined extent and position of the trench in relation to the proposed pier footing extensions. Tremie concrete may be required to construct the pier footings.
- Special handling and disposal procedures may be required for potential environmentally impacted soils on site.

Monitoring of the existing superstructure for movements is recommended during extension of the pier footings and construction of new abutment footings to identify potential movement due to construction activities. 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 piers and abutments, and deck movements during adjacent footing construction. The monitoring must 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. 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 ⁽¹⁾ 'N' VALUE
Very Soft	12 or less	Less than 2
Soft	12 to 25	2 to 4
Firm	25 to 50	4 to 8
Stiff	50 to 100	8 to 15
Very Stiff	100 to 200	15 to 30
Hard	Greater than 200	Greater than 30

NOTE: Hierarchy of Soil Strength Prediction

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



4. TERMS DESCRIBING DENSITY (COHESIONLESS SOILS ONLY)

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

5. LEGEND FOR RECORDS OF BOREHOLES

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

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






 Water Level
 Shear Strength Determination by Pocket Penetrometer

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

UNIFIED SOILS CLASSIFICATION

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

EXPLANATION OF ROCK LOGGING TERMS

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

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

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

RECORD OF BOREHOLE No HD-01

1 OF 2

METRIC

W.P. 4091-07-00 LOCATION N 5 031 092.3 E 370 198.0 ORIGINATED BY LPG
 HWY 417 BOREHOLE TYPE Hollow Stem Augers/NQ Coring (CME 75) COMPILED BY AN
 DATUM Geodetic DATE 2011.09.15 - 2011.09.15 CHECKED BY LRB

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)									
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)								
								○ UNCONFINED	+ FIELD VANE	● QUICK TRIAXIAL			x LAB VANE	W _P	W	W _L					
61.5							20	40	60	80	100	20	40	60	GR	SA	SI	CL			
0.0	ASPHALT: (125mm)																				
0.1	SAND, trace to some gravel, some silt Dense to Compact Brown Moist (FILL)		1	AS																	
			2	SS	46												10	79	11 (SI+CL)		
			3	SS	28																
			4	SS	11																
			5	SS	12																
56.9																					
4.6	SILT, some clay, trace organics Firm Grey Moist		6	SS	6																
55.8																					
5.6	Silty SAND, some clay, trace gravel Very Dense Dark Brown Moist (TILL) Difficult augering at 5.6m		7	SS	76/ 0.225													10	41	35	14
54.2																					
7.3	SHALE, fresh, laminated, grey, very thin limestone interbeds through out		1	RUN																	TCR=100% SCR=100% RQD=92%
			2	RUN																	TCR=100% SCR=93% RQD=58%

Continued Next Page

+ 3 . X 3 : Numbers refer to
Sensitivity

20
15
10

(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No HD-01

2 OF 2

METRIC

W.P. 4091-07-00 LOCATION N 5 031 092.3 E 370 198.0 ORIGINATED BY LPG
 HWY 417 BOREHOLE TYPE Hollow Stem Augers/NQ Coring (CME 75) COMPILED BY AN
 DATUM Geodetic DATE 2011.09.15 - 2011.09.15 CHECKED BY LRB

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC NATURAL LIQUID LIMIT MOISTURE CONTENT LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)				
							20	40	60	80	100	20	40	60			
	Continued From Previous Page																
51.1																	
10.4	END OF BOREHOLE AT 10.4m. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 3.0m slotted screen. WATER LEVEL READINGS: DATE DEPTH (m) ELEV. (m) Oct.12/11 4.9 56.6					51											

RECORD OF BOREHOLE No HD-02

1 OF 1

METRIC

W.P. 4091-07-00 LOCATION N 5 031 095.1 E 370 247.8 ORIGINATED BY RK
 HWY 417 BOREHOLE TYPE Tripod with Hammer/Portable Hilti Drill COMPILED BY AN
 DATUM Geodetic DATE 2011.07.27 - 2011.07.27 CHECKED BY LRB



SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa				WATER CONTENT (%)				
56.9 0.0 0.1	ASPHALT: (100mm)						20	40	60	80	100	W _P	W	W _L		
	SAND and GRAVEL, some silt Compact Dark Brown Moist (FILL)		1	SS	15											
55.6 1.3	Silty SAND, trace clay, trace gravel, some wood fragments Loose to Compact Brown to Dark Grey Moist to Wet (TILL)		2	SS	6											0 49 45 6
54.7 2.2	SHALE, fresh, thinly laminated, occasional horizontal joints, dark grey		3	SS	118/ 0.20											
			1	RUN											FI	TCR=70% SCR=70% RQD=0%
			2	RUN											3	TCR=91% SCR=89% RQD=37% UCS=14MPa Average
			3	RUN											1	
			4	RUN											4	TCR=100% SCR=61% RQD=39% UCS=32MPa Average
51.2 5.7	END OF BOREHOLE AT 5.7m. BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG TO 0.6m, SAND AND GRAVEL TO 0.15m AND ASPHALT COLD PATCH TO SURFACE.														0	TCR=100% SCR=93% RQD=20% UCS=17MPa Average

RECORD OF BOREHOLE No HD-03

1 OF 1

METRIC

W.P. 4091-07-00 LOCATION N 5 031 097.0 E 370 269.3 ORIGINATED BY RK
 HWY 417 BOREHOLE TYPE Portable Hilti Drill - Coring COMPILED BY AN
 DATUM Geodetic DATE 2011.07.26 - 2011.07.26 CHECKED BY LRB

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB VANE			WATER CONTENT (%) w _P w w _L				
55.1								20	40	60	80	100			
0.0	WATER						55								
54.7															
0.4	SHALE, slightly weathered to fresh, thinly laminated, horizontal joints, dark grey Highly fractured from 0.4m to 0.6m Clay seam at 1.9m		1	RUN										FI	TCR=88% SCR=62% RQD=0%
			2	RUN			54							4	
														2	TCR=95% SCR=95% RQD=57% UCS=23MPa Average
			3	RUN			53							3	TCR=100% SCR=94% RQD=52% UCS=20MPa Average
														4	
														3	TCR=100% SCR=89% RQD=32% UCS=12MPa Average
			4	RUN			52							5	
														3	TCR=100% SCR=89% RQD=32% UCS=12MPa Average
51.2														4	
3.9	END OF BOREHOLE AT 3.9m. BOREHOLE BACKFILLED WITH BENTONITE PELLETS.														

+³, ×³: Numbers refer to
Sensitivity

20
15
10



(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No HD-04

1 OF 1

METRIC

W.P. 4091-07-00 LOCATION N 5 031 101.0 E 370 303.6 ORIGINATED BY RK
HWY 417 BOREHOLE TYPE Portable Hilti Drill - Coring COMPILED BY AN
DATUM Geodetic DATE 2011.07.26 - 2011.07.26 CHECKED BY LRB

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT NATURAL MOISTURE LIQUID CONTENT LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa				WATER CONTENT (%)				
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE				w _P w w _L				
55.1							20	40	60	80	100	20	40	60		
0.0	WATER															
54.7																
0.4	SHALE, slightly weathered to fresh, thinly laminated, occasional horizontal joints, dark grey Rubble zone (150mm) at 0.4m Highly fractured zone (250mm) at 0.6m		1	RUN												
			2	RUN												
			3	RUN												
51.3																
3.8	END OF BOREHOLE AT 3.8m. BOREHOLE BACKFILLED WITH BENTONITE PELLETS.															

+³, X³: Numbers refer to
Sensitivity

20
15 10 5
(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No HD-04B

1 OF 1

METRIC

W.P. 4091-07-00 LOCATION N 5 031 097.0 E 370 304.8 ORIGINATED BY RK
 HWY 417 BOREHOLE TYPE Portable Hilti Drill - Coring COMPILED BY AN
 DATUM Geodetic DATE 2011.08.31 - 2011.08.31 CHECKED BY LRB

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							WATER CONTENT (%)		
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB VANE							W P W W L		
55.1							20	40	60	80	100	20	40	60	FI	TCR=82% SCR=27% RQD=0% TCR=95% SCR=94% RQD=64% UCS=17MPa Average TCR=100% SCR=98% RQD=12% UCS=30MPa Average TCR=95% SCR=51% RQD=14% UCS=42MPa Average	
0.0	WATER					55									3		
54.7															2		
0.4	SHALE, slightly weathered to fresh, thinly laminated, occasional horizontal joints, dark grey Rubble zone (150mm) at surface Clay seam (5mm)		1	RUN											2		
			2	RUN		54									4		
	Clay seam in joint														3		
			3	RUN		53									4		
															5		
															1		
	Vertical joint (250mm long) at 3.1m Rubble zone (50mm) at 3.4m		4	RUN		52									2		
51.2															9		
3.9	END OF BOREHOLE AT 3.9m. BOREHOLE BACKFILLED WITH BENTONITE CHIPS TO SURFACE.																

RECORD OF BOREHOLE No HD-05

1 OF 1

METRIC

W.P. 4091-07-00 LOCATION N 5 031 106.0 E 670 338.5 ORIGINATED BY RK
 HWY 417 BOREHOLE TYPE Portable Hilti Drill - Coring COMPILED BY AN
 DATUM Geodetic DATE 2011.07.25 - 2011.07.25 CHECKED BY LRB

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa				
								20 40 60 80 100				
								20 40 60 80 100				
55.1												
0.0	WATER											
54.7												
0.4	SHALE, slightly weathered to fresh, thinly laminated, dark grey, occasional horizontal fractures, some rubble zones Rubble zone (50mm) at 0.7m		1	RUN								
			2	RUN								
	Rubble zone (75mm) at 2.0m Rubble zone (100mm) at 2.2m Rubble zone (150mm) at 2.5m		3	RUN								
			4	RUN								
51.2												
3.9	END OF BOREHOLE AT 3.9m. BOREHOLE BACKFILLED WITH BENTONITE PELLETS.											

+ 3, x 3: Numbers refer to
Sensitivity

20
15 10 5
(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No HD-05B

1 OF 1

METRIC

W.P. 4091-07-00 LOCATION N 5 031 099.0 E 370 339.7 ORIGINATED BY RK
HWY 417 BOREHOLE TYPE Portable Hilti Drill - Coring COMPILED BY AN
DATUM Geodetic DATE 2011.08.31 - 2011.08.31 CHECKED BY LRB

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					
55.1								20 40 60 80 100					
0.0	TOPSOIL, silty, some sand, some cobbles and boulders		1	GS			55						
54.6	Brown												
0.5	Saturated		1	RUN									
	SHALE, slightly weathered to fresh, thinly laminated, occasional horizontal joints, dark grey		2	RUN			54						
	Clay seam (10mm) at 1.6m												
			3	RUN			53						
	Clay seam (<5mm)												
			4	RUN			52						
51.4													
3.7	END OF BOREHOLE AT 3.7m. BOREHOLE BACKFILLED WITH BENTONITE CHIPS TO SURFACE.												

+³, X³: Numbers refer to
Sensitivity




20
15 10 5
(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No HD-06

1 OF 1

METRIC

W.P. 4091-07-00 LOCATION N 5 031 111.0 E 370 373.1 ORIGINATED BY RK
HWY 417 BOREHOLE TYPE Portable Hilti Drill - 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			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa			WATER CONTENT (%)						
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB VANE									
55.2								20	40	60	80	100					
0.0	WATER						55										
54.6	Scattered cobbles																
0.6	SHALE, slightly weathered to fresh, thinly laminated, occasional vertical joints, dark grey		1	RUN			54										TCR=84% SCR=23% RQD=0% UCS=35MPa Average
			2	RUN													TCR=96% SCR=69% RQD=19% UCS=27MPa Average
			3	RUN			53										TCR=98% SCR=98% RQD=50% UCS=35MPa Average
			4	RUN			52										TCR=100% SCR=100% RQD=66% UCS=17MPa Average
51.2	END OF BOREHOLE AT 4.0m. BOREHOLE BACKFILLED WITH BENTONITE PELLETS.																
4.0																	

RECORD OF BOREHOLE No HD-06B

1 OF 1

METRIC

W.P. 4091-07-00 LOCATION N 5 031 105.0 E 370 374.0 ORIGINATED BY RK
 HWY 417 BOREHOLE TYPE Portable Hilti Drill - Coring COMPILED BY AN
 DATUM Geodetic DATE 2011.08.30 - 2011.08.31 CHECKED BY LRB

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						WATER CONTENT (%)			
								○ UNCONFINED + FIELD VANE						PLASTIC NATURAL LIQUID LIMIT MOISTURE LIMIT			
								● QUICK TRIAXIAL × LAB VANE						w _P w w _L			
							20	40	60	80	100		20	40	60		
55.1																	
0.0	WATER																
54.6																	
0.5	SHALE, fresh, thinly laminated, dark grey, occasional horizontal joints, occasional clay seams		1	RUN													
	Clay seam (5mm)		2	RUN													
			3	RUN													
	Clay seam (5mm) at 3.7m Clay seam (5mm) at 3.8m		4	RUN													
51.2																	
3.9	END OF BOREHOLE AT 3.9m. BOREHOLE BACKFILLED WITH BENTONITE CHIPS.																

RECORD OF BOREHOLE No HD-07

1 OF 1

METRIC

W.P. 4091-07-00 LOCATION N 5 031 113.6 E 370 398.9 ORIGINATED BY RK
 HWY 417 BOREHOLE TYPE Tripod with Cathead/Portable Hilti Drill COMPILED BY AN
 DATUM Geodetic DATE 2011.07.20 - 2011.07.20 CHECKED BY LRB

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								WATER CONTENT (%)		
								○ UNCONFINED	+ FIELD VANE	● QUICK TRIAXIAL							× LAB VANE	
57.3							20	40	60	80	100	20	40	60	GR	SA	SI	CL
8.8	TOPSOIL: (25mm)		1	SS	8													
56.7	Gravelly SAND, some silt, trace of clay, shale fragments		2	SS	31													
0.6	Loose Dark Brown Moist (FILL)		3	SS	29													
	Silty SAND, trace to some clay, trace to some gravel, some shale fragments		4	SS	10													
	Loose to Dense Dark Brown Moist (TILL)		5	SS	12													
	180mm cobble (coring)																	
54.3	Becomes wet																	
3.0	SHALE, slightly weathered to fresh, thinly laminated, dark grey, some clay in fractures		1	RUN														
			2	RUN														
			3	RUN														
			4	RUN														
50.7	END OF BOREHOLE AT 6.6m. Piezometer installation consists of 30mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen.																	
6.6																		
	WATER LEVEL READINGS: DATE DEPTH (m) ELEV. (m) Sep.02/11 2.1 55.2 Sep.20/11 2.1 55.2 Oct.12/11 2.1 55.2																	

+ 3, X 3: Numbers refer to Sensitivity 20 15 10 5 0 5 10 15 20 (%) 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	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE			
60.8								<p>w_p ————— w ————— w_L</p> <p>20 40 60</p>	kN/m ³	GR SA SI CL	

Borehole ID	Depth (m)	Soil Description	Sample No.	Soil Type	Grain Size (%)	Moisture (%)	Plasticity Index (%)	Unconfined Compressive Strength (kPa)
B-8	0.0 - 57.8	TOPSOIL: (25mm) SAND, trace to some gravel Dense to Compact Brown (some orange staining) Damp (FILL)	1	SS	42			
			2	SS	37			
			3	SS	29			
			4	SS	10			
B-3	57.8 - 54.9	Silty SAND, some gravel, trace clay Compact to Very Dense Dark Brown Moist (TILL) Difficult augering at 4.0m No recovery	5	SS	14			
			6	SS	90			
			1	RUN				
			2	RUN				
			3	RUN				
B-5	54.9 - 51.5	SHALE, slightly weathered to fresh, thinly laminated, dark grey Rubble zone (150mm) at 5.9m Rubble zone (75mm) at 6.2m Clay seam (<5mm thick) at 6.3m Clay seam (<5mm thick) at 8.8m						
B-3	51.5 - 9.3	END OF BOREHOLE AT 9.3m. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 3.0m slotted screen.						

+³, ×³: Numbers refer to Sensitivity

ONTMT4S 1201A.GPJ 11/22/11

METRIC

ONTMT4S 1201A.GPJ 11/22/11

RECORD OF BOREHOLE No HD-09

1 OF 2

METRIC

W.P. 4091-07-00 LOCATION N 5 031 125.7 E 370 199.7 ORIGINATED BY LPG
 HWY 417 BOREHOLE TYPE Hollow Stem Augers/NQ Coring (CME 75) COMPILED BY AN
 DATUM Geodetic DATE 2011.08.15 - 2011.08.15 CHECKED BY LRB

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								
								○ UNCONFINED	+ FIELD VANE	● QUICK TRIAXIAL						× LAB VANE
60.6							20	40	60	80	100	20	40	60		GR SA SI CL
0.0	ASPHALT: (150mm)															
0.2	SAND, trace to some gravel, trace silt Compact Brown Moist (FILL)		1	AS												
			2	SS	23											
			3	SS	29											
			4	SS	10											
			5	SS	60/											
	Becomes very dense, possible hydrocarbon Dark Grey Probable boulder at 3.2m				0.025											6 90 4 (SI+CL)
56.6																
4.0	Silty SAND, some clay, trace gravel Dense to Very Dense Dark Brown to Dark Grey Moist (TILL)		6	SS	36											3 51 36 10
			7	SS	90											
54.3																
6.2	SHALE, slightly weathered to fresh, laminated, very fine, grey, very thin limestone interbeds through out		1	RUN											FI	
															>10	
															>10	TCR=100% SCR=67% RQD=19% UCS=11MPa Average
															>5	
															0	
	Highly fractured with clay infilling		2	RUN											>10	
															4	TCR=100% SCR=93% RQD=50% UCS=16MPa Average
															3	
															3	
															0	
															2	TCR=100% SCR=100% RQD=100%
51.0															0	UCS=15MPa Average
9.5	END OF BOREHOLE AT 9.5m. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe															

Continued Next Page

+³, x³: Numbers refer to
Sensitivity

20
15
10

(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No HD-09

2 OF 2

METRIC

W.P. 4091-07-00 LOCATION N 5 031 125.7 E 370 199.7 ORIGINATED BY LPG
 HWY 417 BOREHOLE TYPE Hollow Stem Augers/NQ Coring (CME 75) COMPILED BY AN
 DATUM Geodetic DATE 2011.08.15 - 2011.08.15 CHECKED BY LRB

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa					WATER CONTENT (%)			
							20	40	60	80	100	20	40	60		
	Continued From Previous Page															
	with a 3.0m slotted screen.															
	WATER LEVEL READINGS: DATE DEPTH (m) ELEV. (m) Aug.18/11 5.8 54.8 Oct.12/11 5.0 55.6															

+³, X³: Numbers refer to
Sensitivity

20
15 10 5
(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No HD-10

1 OF 1

METRIC

W.P. 4091-07-00 LOCATION N 5 031 133.9 E 370 237.4 ORIGINATED BY RK
 HWY 417 BOREHOLE TYPE Tripod with Hammer/Portable Hilti Drill COMPILED BY AN
 DATUM Geodetic DATE 2011.07.27 - 2011.07.27 CHECKED BY LRB

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																								
57.5							20	40	60	80	100																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																					

+³ X³: Numbers refer to
Sensitivity

20
15 10 5
10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No HD-11

1 OF 1

METRIC

W.P. 4091-07-00 LOCATION N 5 031 136.0 E 370 263.6 ORIGINATED BY RK
 HWY 417 BOREHOLE TYPE Portable Hilti Drill - Coring COMPILED BY AN
 DATUM Geodetic DATE 2011.07.26 - 2011.07.26 CHECKED BY LRB

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB VANE						WATER CONTENT (%) w _p w w _L			
55.3							20	40	60	80	100						
0.0	WATER																
55.0	Scattered cobbles, with sand		1	RUN													
0.3	CONCRETE RUBBLE		2	RUN													
54.5																	
0.8	CONCRETE																
54.1																	
1.2	GRAVEL, with rubble and shale fragments (FILL)		3	RUN													
52.7																	
2.6	SHALE, fresh, thinly laminated, occasional horizontal and vertical joints, dark grey Highly fractured from 2.6m to 2.8m		4	RUN													
	Rubble zone (50mm) at 3.3m																
			5	RUN													
	Rubble zone (125mm) at 4.3m																
			6	RUN													
	Rubble zone (50mm) at 4.8m																
49.3																	
6.0	END OF BOREHOLE AT 6.0m. BOREHOLE BACKFILLED WITH BENTONITE PELLETS.																

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

RECORD OF BOREHOLE No HD-11B

1 OF 1

METRIC

W.P. 4091-07-00 LOCATION N 5 031 144.0 E 370 262.9 ORIGINATED BY RK
 HWY 417 BOREHOLE TYPE Portable Hilti Drill - Coring COMPILED BY AN
 DATUM Geodetic DATE 2011.08.29 - 2011.08.29 CHECKED BY LRB

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																											
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																										
								○ UNCONFINED	+ FIELD VANE	● QUICK TRIAXIAL			x LAB VANE	w _P	w	w _L																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																							
55.1							20	40	60	80	100																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																												

+³ ×³: Numbers refer to
Sensitivity

20
15 5
10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No HD-12

1 OF 1

METRIC

W.P. 4091-07-00 LOCATION N 5 031 149.0 E 370 297.6 ORIGINATED BY RK
 HWY 417 BOREHOLE TYPE Portable Hilti Drill - Coring COMPILED BY AN
 DATUM Geodetic DATE 2011.07.28 - 2011.07.28 CHECKED BY LRB











SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL																
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa			WATER CONTENT (%)																				
55.0								20 40 60 80 100																							
0.0	WATER							20 40 60 80 100																							
0.2	SHALE, slightly weathered to fresh, thinly laminated, dark grey Highly fractured from 0.2m to 0.8m		1	RUN									FI	TCR=33% SCR=0% RQD=0%																	
			2	RUN											>10																
			3	RUN												>10															
					4	RUN											9														
51.4	Rubble zone (50mm) at 3.5m													5	TCR=92% SCR=63% RQD=27% UCS=21MPa Average																
																3	RUN											4			
																		4	RUN										4		
																3.6	END OF BOREHOLE AT 3.6m. BOREHOLE BACKFILLED WITH BENTONITE PELLETS.														4
																10	TCR=96% SCR=92% RQD=43% UCS=13MPa Average														
																		3	RUN												3
																				4	RUN										

RECORD OF BOREHOLE No HD-13

1 OF 1

METRIC

W.P. 4091-07-00 LOCATION N 5 031 154.0 E 370 332.5 ORIGINATED BY RK
 HWY 417 BOREHOLE TYPE Portable Hilti Drill - Coring COMPILED BY AN
 DATUM Geodetic DATE 2011.08.29 - 2011.08.29 CHECKED BY LRB

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	*N VALUES			SHEAR STRENGTH kPa				WATER CONTENT (%)				
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE	20 40 60 80 100	20 40 60	w _p w w _L					
55.2																
0.0	WATER						55							FI		
54.8														>15	TCR=77% SCR=15% RQD=0%	
0.4	SHALE, slightly weathered to fresh, thinly laminated, occasional horizontal joints, occasional clay infilling of joints, dark grey		1	RUN										12		
			2	RUN			54							6	TCR=96% SCR=90% RQD=0% UCS=17MPa Average	
	Clay seam (10mm)													5		
														10		
	Sub-vertical joint (50mm) at 2.5m		3	RUN			53							8	TCR=100% SCR=90% RQD=21% UCS=34MPa Average	
														6		
														4	TCR=97% SCR=94% RQD=19% UCS=11MPa Average	
			4	RUN			52							6		
														4		
51.3																
3.9	END OF BOREHOLE AT 3.9m. BOREHOLE BACKFILLED WITH BENTONITE CHIPS.															

+³ . X³ : Numbers refer to
Sensitivity

20
15 5
10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No HD-14

1 OF 1

METRIC

W.P. 4091-07-00 LOCATION N 5 031 159.0 E 370 366.2 ORIGINATED BY RK
 HWY 417 BOREHOLE TYPE Portable Hilti Drill - Coring COMPILED BY AN
 DATUM Geodetic DATE 2011.08.30 - 2011.08.30 CHECKED BY LRB

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					
55.1								20 40 60 80 100					
0.0	WATER						55						
54.6													
0.5	SHALE, slightly weathered to fresh, thinly laminated, occasional horizontal joints, dark grey Rubble zone (50mm) at 0.9m		1	RUN			54						
			2	RUN									
	Clay seam (5mm) at 1.9m						53						
			3	RUN									
							52						
			4	RUN									
	Clay seam (15mm) at 3.8m												
51.2													
3.9	END OF BOREHOLE AT 3.9m. BOREHOLE BACKFILLED WITH BENTONITE CHIPS.												

RECORD OF BOREHOLE No HD-15

1 OF 1

METRIC

W.P. 4091-07-00 LOCATION N 5 031 156.0 E 370 394.7 ORIGINATED BY RK
HWY 417 BOREHOLE TYPE Tripod with Cathead/Portable Hilti Drill COMPILED BY AN
DATUM Geodetic DATE 2011.07.19 - 2011.07.19 CHECKED BY LRB

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								
								20	40	60	80	100				
57.4																
56.8	TOPSOIL: (25mm)		1	SS	19											
56.6	Gravelly SAND, some silt															
56.4	Compact															
56.2	Brown															
56.0	Dry															
55.8	(FILL)		2	SS	22											
55.6	Silty SAND, trace to some gravel,															
55.4	trace to some clay															
55.2	Compact to Loose		3	SS	50/											
55.0	Dark Brown															
54.8	Moist to Wet		1	RUN	0.125											
54.6	(TILL)															
54.4	Cobble (150mm)															
54.2			4	SS	10											
54.0																
53.8	Shale fragments															
53.6			5	SS	9											
53.4																
53.2																
53.0																
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50.4																

+ 3, X 3: Numbers refer to
Sensitivity

20
15 10 5
(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No HD-15B

1 OF 1

METRIC

W.P. 4091-07-00 LOCATION N 5 031 160.9 E 370 393.2 ORIGINATED BY RK
HWY 417 BOREHOLE TYPE Tripod/Portable Hilti Drill COMPILED BY AN
DATUM Geodetic DATE 2011.09.01 - 2011.09.01 CHECKED BY LRB

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								
57.6																
56.4	TOPSOIL: (25mm) Gravelly SAND, some rootlets Very Dense to Dense Dark Brown Damp (FILL)		1	SS	75											
			2	SS	33											
54.9	Silty SAND, trace gravel, trace to some clay, some shale fragments Compact to Very Dense Dark Brown Damp (TILL) Shale fragments		3	SS	18											
			4	SS	53											8 56 32 4
			5	SS	106/											
52.7	SHALE, slightly weathered to fresh, thinly laminated, occasional clay seams, dark grey Sub-vertical joint from 4.8m to 5.0m		1	RUN	0.250										FI	TCR=61% SCR=35% RQD=0%
			2	RUN											5	TCR=100% SCR=93% RQD=43% UCS=27MPa Average
			3	RUN											5	
			4	RUN											2	TCR=95% SCR=65% ROD=46% UCS=29MPa Average
51.5															9	
															6	
															3	TCR=100% SCR=97% RQD=76% UCS=14MPa Average
6.1	END OF BOREHOLE AT 6.1m. Piezometer installation consists of 40mm diameter Schedule 40 PVC pipe with a 3.0m slotted screen. WATER LEVEL READINGS: DATE DEPTH (m) ELEV. (m) Sep.2/11 2.3 55.3 Sep.20/11 2.4 55.2 Oct.12/11 2.5 55.1															

+ 3, X 3: Numbers refer to
Sensitivity

20
15
10

(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No HD-16

1 OF 2

METRIC

W.P. 4091-07-00 LOCATION N 5 031 160.4 E 370 419.3 ORIGINATED BY RK
 HWY 417 BOREHOLE TYPE Hollow Stem Augers/NQ Coring - CME 45 COMPILED BY AN
 DATUM Geodetic DATE 2011.09.06 - 2011.09.06 CHECKED BY LRB

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								WATER CONTENT (%)
								○ UNCONFINED ● QUICK TRIAXIAL	+ FIELD VANE × LAB VANE							
60.5								20 40 60 80 100		20 40 60					GR SA SI CL	
59.8	TOPSOIL: (25mm)															
	SAND, some gravel, some silt Compact to Loose Brown Damp Moist (FILL)		1	SS	17						○					
			2	SS	12						○					
			3	SS	8						○					
			4	SS	11						○				18 67 15 (SI+CL)	
57.1	Black staining, hydrocarbon odour		5	SS	18						○					
56.4	Clayey SILT, some sand Greyish Brown										○					
54.1	Silty SAND, trace gravel, trace clay Very Dense Dark Brown Damp (TILL)		6	SS	77/ 0.280						○				9 54 28 9	
	Difficult augering at 5.2m															
54.4																
6.1	SHALE, fresh, thinly laminated, occasional horizontal and sub-horizontal joints, dark grey		1	RUN							○					
	Rubble zone (100mm thick) at 6.1m															
			2	RUN												
51.4																
9.1	END OF BOREHOLE AT 9.1m. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 3.0m slotted screen.															

Continued Next Page

+ 3, × 3: Numbers refer to
Sensitivity

20
15 10 5
(%) STRAIN AT FAILURE

METRIC

[illegible]

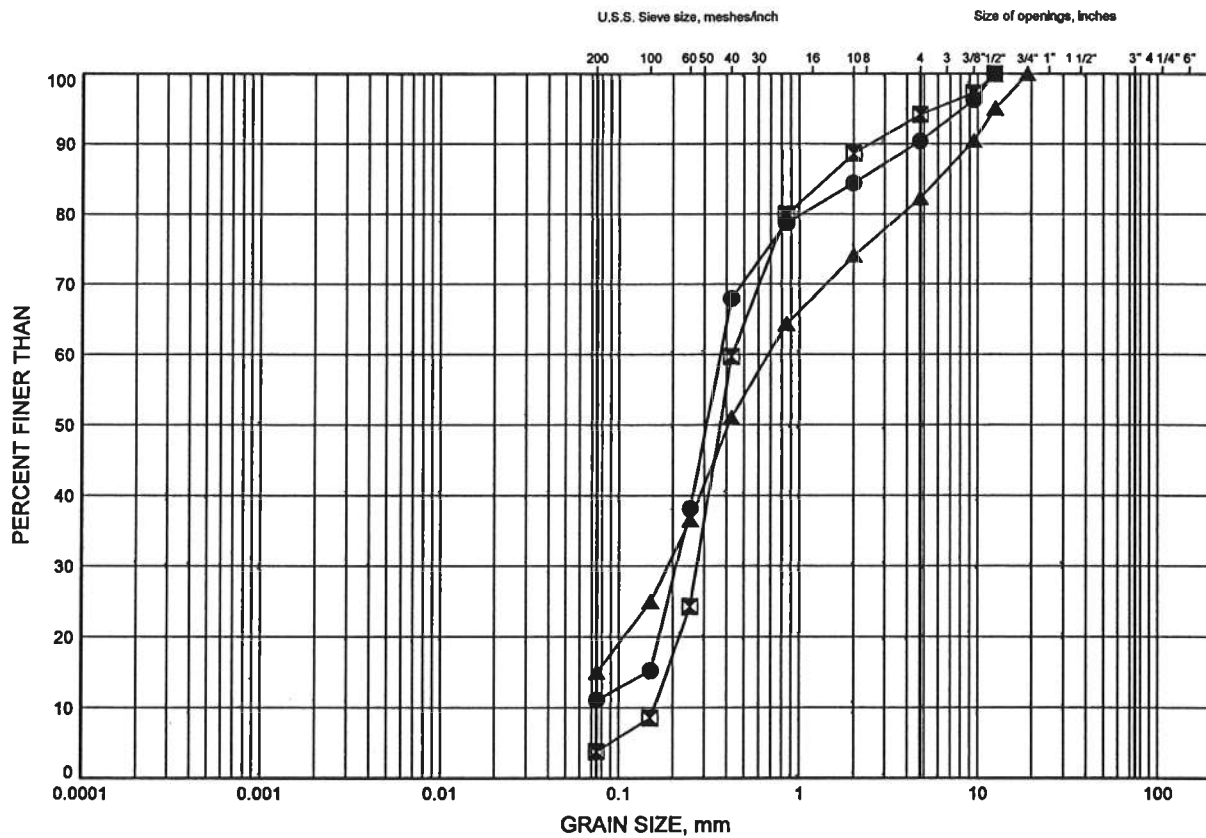
Appendix B

Laboratory Test Results

Highway 417 Ottawa: Nicholas to Vanier GRAIN SIZE DISTRIBUTION

FIGURE B1

SAND FILL



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

LEGEND

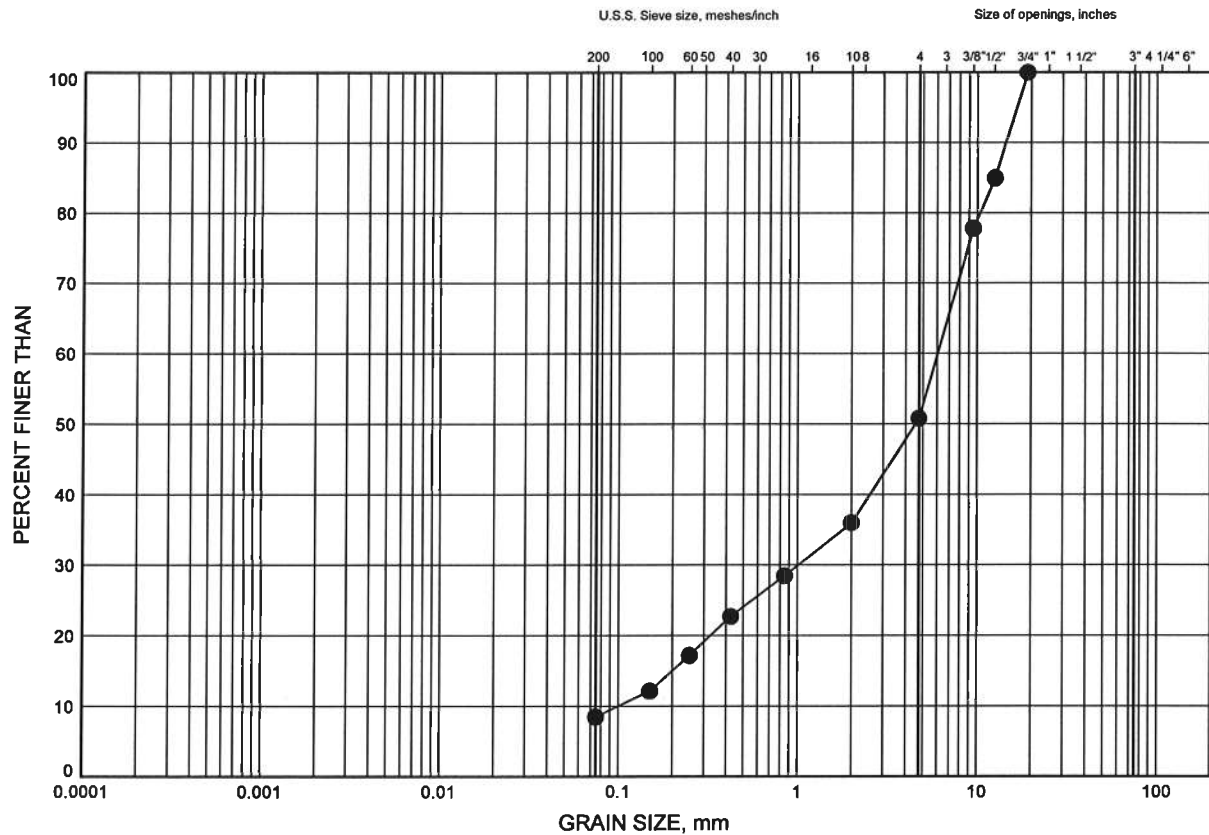
SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	HD-01	1.07	60.40
⊠	HD-09	2.59	57.97
▲	HD-16	2.59	57.91

Highway 417 Ottawa: Nicholas to Vanier

GRAIN SIZE DISTRIBUTION

FIGURE B2

SAND & GRAVEL



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	HD-10	2.44	55.04

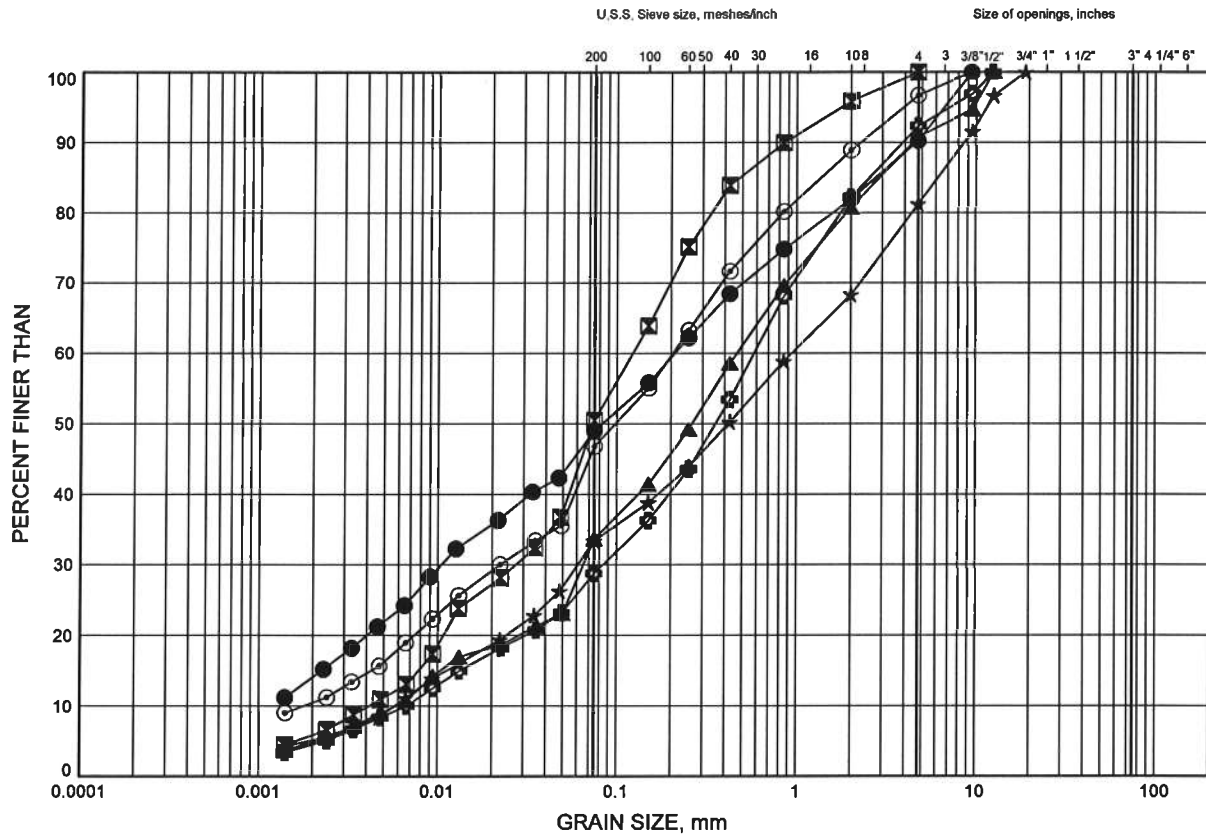


W.P.# 4091-07-00
 Prepared By AN
 Checked By LRB

Highway 417 Ottawa: Nicholas to Vanier GRAIN SIZE DISTRIBUTION

FIGURE B3

SILTY SAND TILL



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	HD-01	6.29	55.18
⊠	HD-02	1.60	55.32
▲	HD-07	2.74	54.52
★	HD-08	3.35	57.42
⊙	HD-09	4.57	55.99
⊕	HD-15	2.27	55.13

GRAIN SIZE DISTRIBUTION - THURBER 1201A GPJ 11/21/11

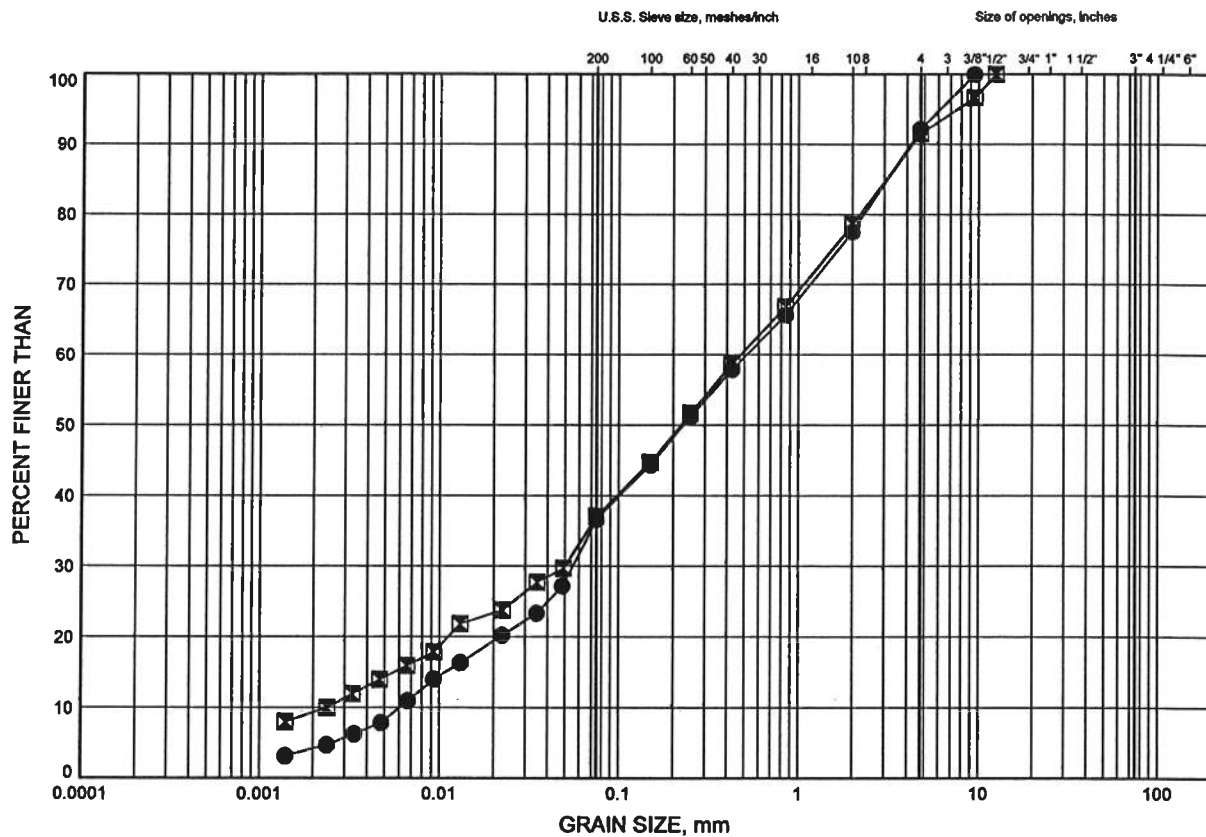
W.P.# .4091-07-00.....
Prepared By .AN.....
Checked By .LRB.....



Highway 417 Ottawa: Nicholas to Vanier GRAIN SIZE DISTRIBUTION

FIGURE B4

SILTY SAND TILL



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	HD-15B	2.13	55.49
⊠	HD-16	4.88	55.62

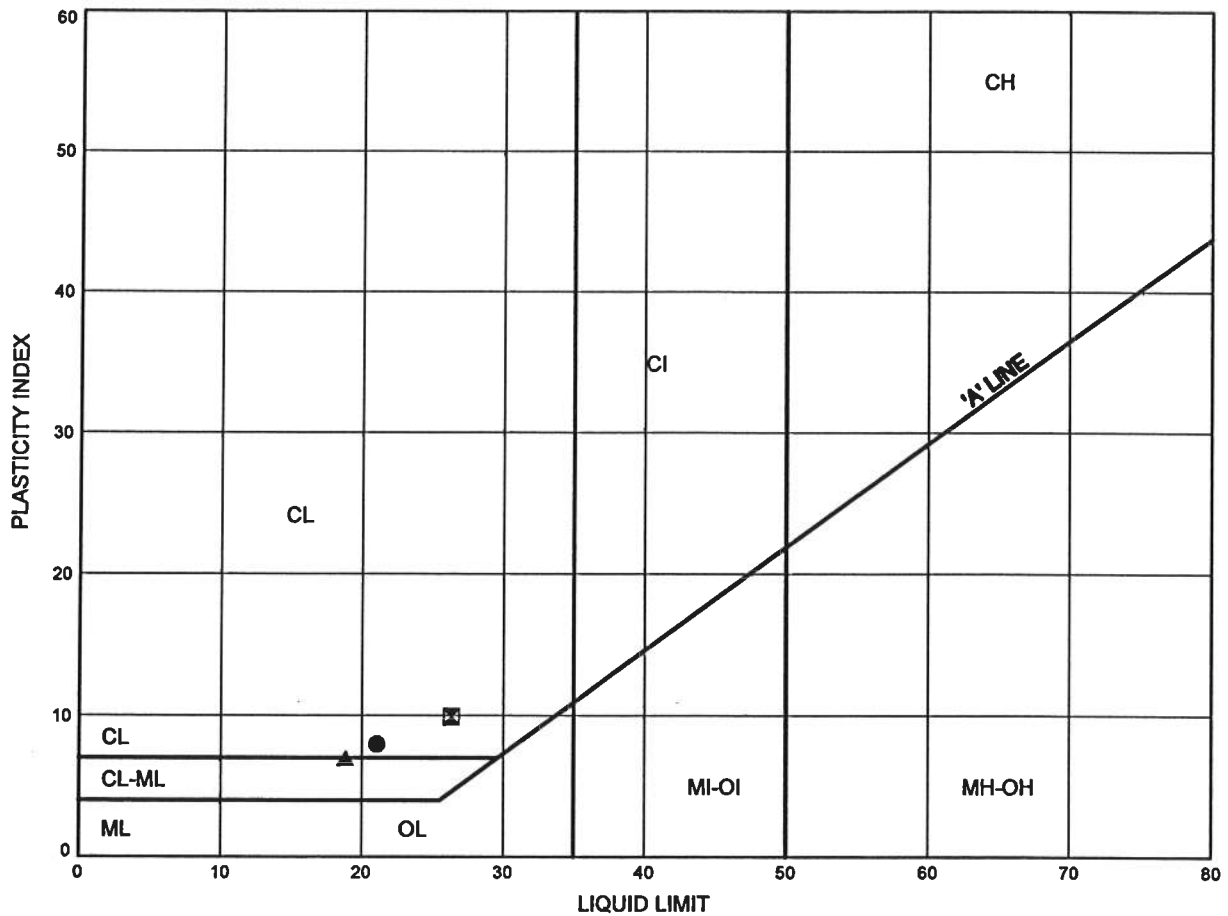


W.P.# 4091-07-00
Prepared By AN
Checked By LRB

Highway 417 Ottawa: Nicholas to Vanier
ATTERBERG LIMITS TEST RESULTS

FIGURE B5

SILTY SAND TILL - Trace to Some Clay



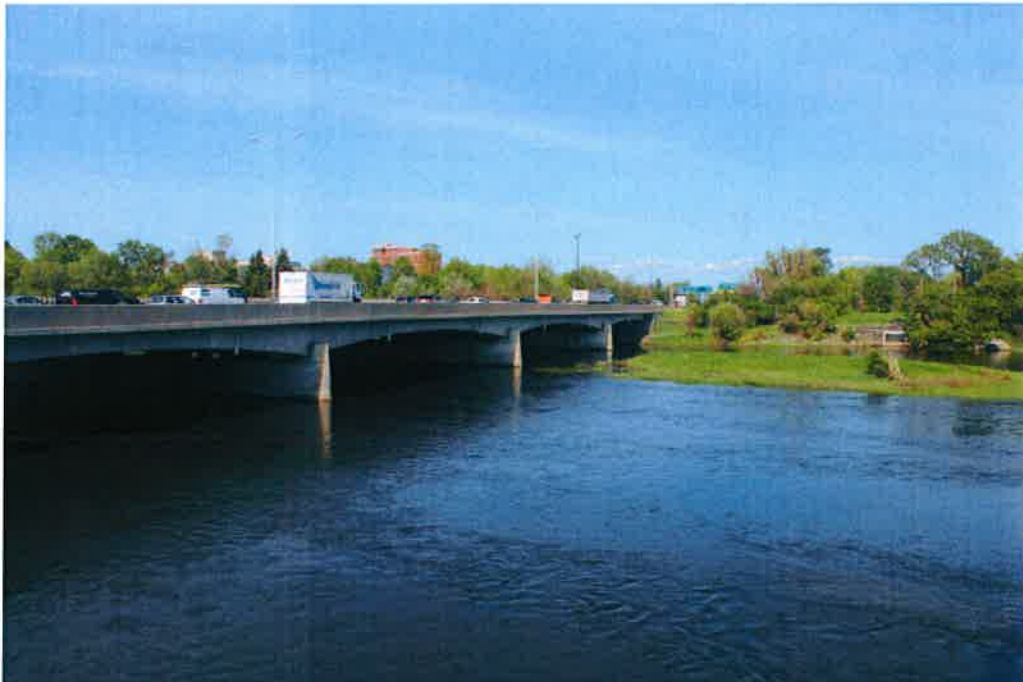
SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	HD-01	6.29	55.18
■	HD-07	2.74	54.52
▲	HD-09	4.57	55.99

Appendix C

Site Photographs



Photograph 1: North side of Hurdman Bridge, looking east



Photograph 2: South side of Hurdman Bridge, looking east

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 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

WORKING SLAB - Item No.

Special Provision

1.0 SCOPE

This Special Provision covers the requirements for the supply and placement of a concrete working slab under structure foundations.

2.0 REFERENCES

This Special Provision refers to the following standards, specifications or publications:

Ontario Provincial Standard Specifications, Construction

OPSS 902 Excavating and Backfilling - Structures

3.0 DEFINITIONS - Not Used

4.0 DESIGN AND SUBMISSION REQUIREMENTS - Not Used

5.0 MATERIALS

Concrete for working slabs shall be of the same class of concrete as the footing to be placed on it.

6.0 EQUIPMENT - Not Used

7.0 CONSTRUCTION

7.01 Excavation

Excavation for the working slab shall be according to OPSS 902.

7.02 Protection of Founding Soil – Not Used

7.03 Protection of Founding Bedrock

The surface of the footing founding rock shall be exposed, cleaned and any loose or fractured parts removed so that sound rock is exposed. The thickness of the working slab shall be a minimum 100 mm and depend on the slope and irregularities in the exposed founding rock surface. A footprint plan view area has been specified on the Contract Documents.

Following inspection and approval of the prepared subgrade, the working slab shall be placed on the exposed cleaned sound founding rock surface as specified in the Contract Documents. The working slab shall be placed as soon as practical after completion of the excavation and in no case later than 4 hours after excavation.

The Contractor shall carry out all necessary work to place the working slabs in dewatered conditions.

Notwithstanding the above, if site conditions do not permit effective dewatering of the excavations, the Contractor shall submit a request in writing to the Contract Administrator for placing the working slabs using tremie concrete methods. The Contract Administrator shall review the request and the dewatering procedures employed, and based on this review, authorize placement of the working slab using tremie methods where effective dewatering is found impractical. The Contractor shall not proceed with tremie concrete placement until written permission has been provided by the Contract Administrator.

7.04 Dewatering

Dewatering shall be carried out according to OPSS 902.

8.0 QUALITY ASSURANCE - Not Used

9.0 MEASUREMENT FOR PAYMENT - Not Used

10.0 BASIS OF PAYMENT

10.01 Working Slab - Item

Payment at the contract unit price for the tender item shall be full compensation for all labour, equipment and materials required to supply and place concrete in the working slabs.

No payment will be made for the supply and placement of concrete on working slabs due to over-excavation of the rock beyond the limits shown on the contract drawings and/or to replace additional rock excavation that may be required due to non-compliance with the four (4) hours timeframe.

Payment for tremie concrete, if required, shall be made at the contract unit price for this item as specified elsewhere in the contract.

EXCAVATION - Item Nos. XX

Special Provision

Site Conditions

The geotechnical conditions at the site are described in the Foundation Investigation Report, Hurdman Bridge Replacement, Highway 417 Expansion from Nicholas Street to Vanier Parkway, Ottawa Ontario by Thurber Engineering Ltd, dated xx/xxx/xxxx.

In particular, the Contractor's attention is drawn to the following:

- The soils at the abutments consist of fill overlying glacial till overlying shale bedrock. The glacial till contains cobbles and boulders. The shale bedrock may contain hard limestone interbeds of varying thickness
- At the piers in river, the bedrock is typically overlain by less than 1 m of sediment, including cobbles, boulders and possible slabs of limestone

There is an abandoned gas main along the north side of the structure, as shown on the Contract Drawings. The northerly extensions of the pier foundations and the north ends of the abutment foundations will overlap the alignment of the abandoned gas main and will encounter the trench excavated for that main. It is possible that a second trench, for an earlier alignment of the gas main, will be encountered along part, or all, of the structure.

Apart from the information on the Record of Borehole HD-11, which is believed to have been drilled within the earlier gas main alignment, there is no information regarding the infill material in the trenches. The trench backfill encountered in Borehole HD-11 consisted of concrete rubble, concrete and gravel with rubble and shale fragments.

Cofferdams

It is anticipated that cofferdams will be required around foundation excavations, particularly in the river. At the north pier foundation extensions and the north end of the abutment foundations, the cofferdams must be a minimum of 2.0 m beyond the edge of the foundation. Locally, cofferdams may have to penetrate below the specified founding elevations in order to facilitate subexcavation of existing trench infill as described later in this NSSP.

Construction

The soils at the site contain cobbles and boulders. The presence of cobbles and boulders must be taken into account when selecting excavation equipment and when designing such works as roadway protection and cofferdams.

Excavation of bedrock will be required in order to achieve the specified founding elevations. Bedrock excavation must be carried out using excavators and pneumatic or hydraulic breakers in a manner that does not disturb the bedrock below the founding elevations. Blasting is not permitted.

The planning of work in the river must take account of the fact that cobbles, boulder and slabs of limestone

may be encountered loose on top of the bedrock or embedded in recent river sediment.

Where an abandoned trench, or trenches, is encountered within 2.0 m of the edge of the foundation, all materials within that portion of the abandoned trench must be subexcavated to undisturbed bedrock. This work will require shoring to prevent the inflow of material from outside the required extent of excavation.

The abandoned gas main may be left in place.

After excavation and removal of all infill in the portion of trench to be excavated, the excavation must be filled to the specified founding elevation using 30 MPa concrete. Concrete placement up to the specified founding elevation may be carried out using tremie methods.

After the specified working slab has been placed, the excavation must be dewatered to permit construction of the foundation in the dry.

Payment

Payment at the contract price for the above tender item shall include full compensation for labour, equipment and materials to do the work.

DOWELS INTO ROCK – Item No.

Special Provision

CONSTRUCTION SPECIFICATION FOR THE SUPPLY, INSTALLATION AND TESTING OF DOWELS INTO ROCK FOR PIER FOOTINGS

1.0 SCOPE

The work for the above noted tender item shall be in accordance with OPSS 904, including all Special Provisions, except as extended herein. This document specifies additional requirements for the supply, installation and testing of Dowels into Rock for the pier footing.

2.0 REFERENCES

This Special Provision refers to the following standards, specifications, or publications:

ASTM International

D1143M Standard Test Methods for Deep Foundations Under Static Axial Compressive Load

3.0 DEFINITIONS

For the purpose of this Special Provision, the following definitions apply:

Dowels into Rock: reinforcing steel bar and non-shrink grout.

Design Engineer: An Engineer who has a minimum of five (5) years experience in all aspects associated with the installation of Dowels into Rock, including drilling, grouting and doweling work. The Design Engineer shall be retained by the Contractor to design various components for the installation and testing for the Dowels into Rock.

Quality Verification Engineer: An Engineer who has a minimum of five (5) years experience in all aspects associated with the installation of Dowels into Rock, including drilling, grouting and doweling work. The Quality Verification Engineer shall be retained by the Contractor to ensure conformance with the contract documents and issue certificate(s) of conformance.

4.0 DESIGN AND SUBMISSION REQUIREMENTS

4.01 Working Drawings

Working Drawings shall consist of drawings, testing and installation records, procedures and reports, and work plans.

The Contractor shall submit Working Drawings to the Contract Administrator as follows:

- a) All Working Drawings that include drawing, testing and installation procedures and reports, and work plans shall be sealed and signed by the Design Engineer.

- b) All Working Drawings that include testing and installation results and reports shall be signed and sealed by the Quality Verification Engineer.

Upon completion of testing or installation and testing for each component, the Contractor shall submit to the Contract Administrator a Certificate of Conformance sealed and signed by a Quality Verification Engineer. The Certificate shall state that the work has been carried out in conformance with the Working Drawings and in general conformance with the contract documents.

Working Drawings consisting of testing and installation records and reports shall be submitted four days after completion of testing and installation. All other Working Drawings shall be submitted two weeks prior to construction.

Working Drawings to be submitted include the following with further details outlined in the remainder of this specification:

- a) Design calculations, specifications and shop drawings covering all aspects of fabrication, installation and acceptance testing of Dowels into Rock.
- b) Test results verifying the 28 day strength of non-shrink grout.
- c) The method for constructing of the holes, maintaining the holes, and placing reinforcing steel bars, grout and other materials in the holes, including casing sizes, bit sizes and tremie grouting methods.
- d) The procedures to verify hole length. Records of measurements that verify the hole length.
- e) Records of all drilling procedures, rock conditions encountered, and installation times.
- f) Test procedures for Dowels into Rock.
- g) Drawings and design calculations for a suitable reaction system for the applied test loads.
- h) Records of vertical and horizontal movements of the reaction system, and elongation of the reinforcing steel bar.
- i) Drawings and details for reference system arrangement.
- j) Current calibration curves shall be provided for all gauges.
- k) Complete test records for all tests including plots of dowel movement versus dowel load, dowel load versus time, and dowel movement versus time.
- l) Remedial measures for unacceptable stressing results.

5.0 MATERIALS

5.01 Non-Shrink Grout

The non-shrink grout shall be an approved product from the MTO's Pre-Qualified Products List.

5.02 Anti-Washout Agent

The anti-washout agent shall be used with the non-shrink grout for the Dowels into Rock. The anti-washout agent shall be one of the following proprietary products:

- 1) Sikament 100 SC Anti-Washout Admixture
Sika Canada Inc.
6915 Davand Drive
Mississauga, ON, L5T 1L5
Toll Free Phone: 800-933-7452
- 2) Rheomac UW 450 Anti-Washout Admixture
BASF Construction Chemicals Canada Ltd (Master Builders)
1800 Clark Blvd
Brampton, ON, L6T 4M7
Toll Free Phone: 416-520-1392

5.03 Manufacturer Information

The Contractor shall provide the following information from the manufacturer for non-shrink grout and anti-washout agent:

- a) Data sheets for the non-shrink grout and anti-washout agent,
- b) Technical information that proves that the non-shrink grout and anti-washout agent are compatible, and
- c) installation procedures

6.0 EQUIPMENT

All equipment for the installation of the Dowels into Rock shall be suitable for the intended purposes and capable of working on the site under the prevailing access and clearance conditions.

The equipment shall not cause damage to the reinforcing steel bars.

7.0 CONSTRUCTION

7.01 Instructions to Contractor

These instructions are to be read in conjunction with the Contract Drawings.

A total of 2 test Dowels into Rock are required for the Dowels into Rock at the pier.

Dowels into rock at the pier shall be installed into sound bedrock to the specified embedment depth.

7.02 Responsibilities of the Contractor

The Contractor shall prove the allowable bond stress by tests of the Dowels into Rock on non-production Dowels into Rock.

The Contractor shall supply equipment, materials and skilled personnel to install production Dowels into Rock and conduct the specified acceptance tests. It shall be the responsibility of the Contractor to constantly monitor the acceptance tests, maintain specified test loads and record test measurements as specified by the Contract Administrator.

The Contractor is responsible for materials and workmanship. Any remedial measures, required because of defects in materials or workmanship, shall be completed by the Contractor at no cost to the Owner.

The Contractor shall submit 4 copies of all Working Drawings to the Contract Administrator as outlined in Section 4.0.

7.03 Subsurface Conditions

Rock and groundwater conditions are described in the Foundation Investigation Report for this Contract.

7.04 Construction of Holes

The sides and end of the hole shall not be disturbed. The Contractor shall submit Working Drawings to the Contract Administrator that include the method for constructing of the holes, maintaining the holes, and placing reinforcing steel bar, grout and other materials in the holes. All excavated material shall be removed from the site.

The hole diameters and hole length for this project are as specified on the Contract Drawings. Prior to commencing drilling operations, the Contractor shall submit Working Drawings to the Contract Administrator outlining devised procedures to verify hole length. The Contractor shall submit Working Drawings that include drilling operations records to the Contract Administrator that include the above noted records.

At all times, the Contractor shall keep a record of all drilling procedures, rock conditions encountered, and installation times. The Contractor shall submit Working Drawings to the Contract Administrator that include the above noted records.

7.05 Installation of Reinforcing Steel Bar

Reinforcing steel bar shall be installed in strict accordance with the Contract Drawings and installation procedures.

Centering devices shall be provided to ensure that the reinforcing steel bar is located centrally in the hole.

Dowels into Rock at the pier shall be installed into sound bedrock.

Reinforcing steel bar shall be installed after the dowel hole has been filled with non-shrink grout.

7.06 Grout and Anti-Washout Agent

The non-shrink grout shall entirely fill the annular space between the reinforcing steel bar and side for the dowel hole.

The placement of grout for the test Dowels into Rock shall be identical to the production Dowels into Rock.

Anti-washout agent shall be used in accordance with the specifications of the manufacturer.

Non-shrink grout shall be placed into the dowel hole using tremie placement methods.

8.0 QUALITY ASSURANCE

All work for the installation of Dowels into Rock shall be inspected by the Quality Verification Engineer.

8.01 Qualifications

8.01.01 Qualifications of Staff from Contractor or Sub-Contractor Completing Work for the Dowels into Rock

All work shall be performed under the direction of personnel experienced with all aspects associated with the installation of Dowels into Rock. Such experience shall have been obtained within the preceding five (5) years on projects of similar nature and scope to the work required for this project.

8.01.02 Qualifications of the Quality Verification Engineer

A resume of the work experience of the Quality Verification Engineer shall be submitted to the Contract Administrator for record purposes. The Quality Verification Engineer shall be a Professional Engineer licensed in the Province of Ontario having a minimum of five years of experience on projects of similar nature and scope to the work required for this project.

8.01.03 Qualifications of the Design Engineer

A resume of the work experience of the Design Engineer shall be submitted to the Contract Administrator for record purposes. The Design Engineer shall be a Professional Engineer licensed in the Province of Ontario having a minimum of five years of experience of projects of similar nature and scope to the work required for this project.

8.02 Testing Requirements

All work for the testing of Dowels into Rock shall be inspected by the Quality Verification Engineer.

8.02.01 General Testing Requirements

Refer to the attached Instructions to Contractor and the Contract Drawings for specific test details.

The Contractor shall install the number of Dowels into Rock specified in the contract documents for testing purposes. The purpose of the testing the Dowels into Rock is to prove the adequacy of the proposed anchor configuration and installation procedures under the site conditions, and to provide design parameters.

The equipment, labour and materials for test dowels shall be identical to Dowels into Rock at the pier. The Dowels into Rock for testing shall be __M dowels grouted into ___ mm diameter holes filled with an approved non-shrink grout with a minimum ___ mm embedment into sound bedrock.

The Contractor shall submit Working Drawings that include proposed procedures for testing of the Dowels into Rock to the Contract Administrator. Such testing shall be executed in strict accordance with the proposed procedures of the Contractor.

The Quality Verification Engineer shall supervise the testing of the Dowels into Rock. The Contractor will notify the Contract Administrator of the testing schedule at least 10 days prior to commencement of the

testing program. Testing for Dowels into Rock shall be conducted concurrently, as scheduled by the Contract Administrator. The tests shall normally be conducted between 8:00 hrs and 20:00 hrs from Monday to Friday, unless otherwise directed by the Contract Administrator.

The Contractor shall supply materials and skilled personnel to conduct the tests for the Dowels into Rock. The equipment and materials shall be capable of stressing the Dowels into Rock to the specified loads. It shall be the responsibility of the Contractor to constantly monitor the test, maintain specified test loads and to record test measurements as specified by the Quality Verification Engineer.

The test site shall be restored to its pre-test condition. Reinforcing steel bars used in tests shall be cut down 25 mm below the top of the sound bedrock.

8.02.02 Testing Location

The Contractor shall remove all loose rock down to sound bedrock at the test location.

The test Dowels into Rock shall be constructed at locations specified by the Contract Administrator.

If site conditions dictate, changes to the test locations will be considered. The Contractor shall provide the Contract Administrator at least 2 days notice in writing of this operation.

8.02.03 Testing Equipment

The dowels into rock will be carried out generally in accordance with the prevailing requirements of ASTM International D1143M superseded where applicable by the procedures specified in this document.

The Contractor shall submit Working Drawings for a suitable reaction system for the applied test loads to the Contract Administrator. Jacks must be secured with chains to provide adequate protection for the personnel in the event of breakage of the reinforcing steel bar or stressing system.

The Contractor shall submit Working Drawings for the reference system arrangement to the Contract Administrator. All reference beams shall be as follows:

The beams shall be independently supported with the support firmly embedded in the ground.

The testing device shall not apply compression to the bedrock surrounding the test for the Dowels into Rock, within a circle concentric with the dowel hole and a diameter equal to 4.0 m.

Reference beams shall be sufficiently rigid to support instrumentation such that variations in readings do not occur.

The Contractor shall construct suitable enclosures to provide complete protection for equipment and instruments from variations in the weather conditions and disturbances during the test program. These provisions must meet the approval of the Quality Verification Engineer and will include that the test enclosures must be weather-proof and provide a consistent temperature in order to eliminate temperature variations that could affect instrumentation.

8.02.04 Testing for Dowels Into Rock, and Report

At all times, the Contractor shall keep records of vertical and horizontal movements of the reaction system, elongation of reinforcing steel bar, and the record of test enclosure temperature. The movements shall be recorded with respect to an independent fixed reference point. The Contractor shall submit Working Drawings that include the above noted records to the Contract Administrator.

Dial gauges shall have at least a 76.2 mm (3.0 in.) travel. Longer gauge stems or sufficient gauge blocks shall be provided to allow for greater travel if required. Gauges shall have precision of at least 0.025 mm (0.0001 in.). The dial gauges shall be placed on smooth bearing surfaces mounted perpendicular to the direction of movement. All gauges, scales or reference points attached to the test anchor shall be mounted so as to prevent movement relative to the test anchor during the test. The Contractor shall submit Working Drawings that include details for current calibration and curves for all gauges to the Contract Administrator.

Jacks used for reinforcing steel bars shall have a minimum ram dimension of 152.6 mm (6.0 in.). The Contractor shall submit Working Drawings that include details for current calibration and curves for all gauges to the Contract Administrator.

Requirements for Clauses 5.4.1 to 5.4.4 shall be repeated as required at different testing locations.

8.02.05 Testing Loading

The testing procedures shall safely load test the Dowels into Rock in tension at a rate of approximately 100kN per minute to the test load of ____ kN. The load shall be increased by an additional 50 kN beyond this level as directed by the Quality Verification Engineer.

Each load shall be maintained for a minimum time of 15 minutes and until the rate of displacement is not greater than 0.25 mm (0.01 inches) per hour.

8.03 Acceptance Criteria

The following acceptance criteria apply:

- a) The testing of dowels shall be carried out in advance of the instalment of Dowels into Rock at the pier footing.
- b) Tests for Dowels into Rock shall have a capacity of at least ____ kN. The Quality Verification Engineer shall report on the acceptance of the tests for Dowels into Rock. The Quality Verification Engineer shall report on the testing of the Dowels into Rock including recommendations for increasing embedment depth, if necessary.

9.0 MEASUREMENT FOR PAYMENT

For measurement purposes, a count shall be made of the number of dowels installed.

10.0 BASIS OF PAYMENT

Payment at the contract unit price for the above tender item shall include full compensation for all labour, equipment, and materials to do the work. No additional payment will be made for tests for Dowels into Rock which are deemed as included as part of the work for the above noted item.

Appendix E

Slope Stability Analysis

	Gamma C kN/m3	Phi deg	Piezo Surf.
Fill	22.8	1	35
Native Till	21	0	33
Shale	22	0	38

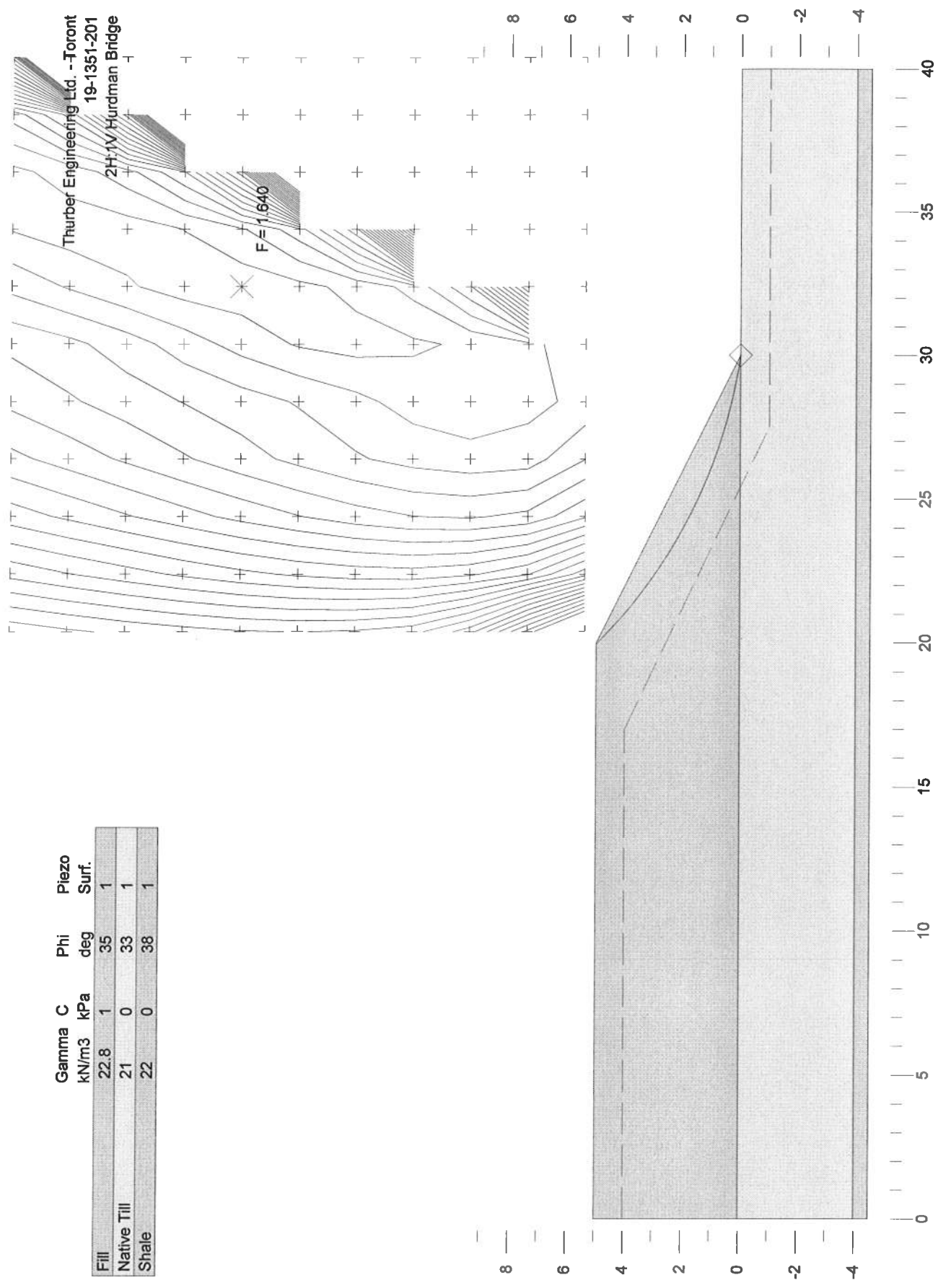


FIGURE E-1

	Gamma C kN/m3	Phi deg	Piezo Surf.
Fill	22.8	1	35
Native Till	21	0	33
Shale	22	0	38

Seismic coefficient = 0.16

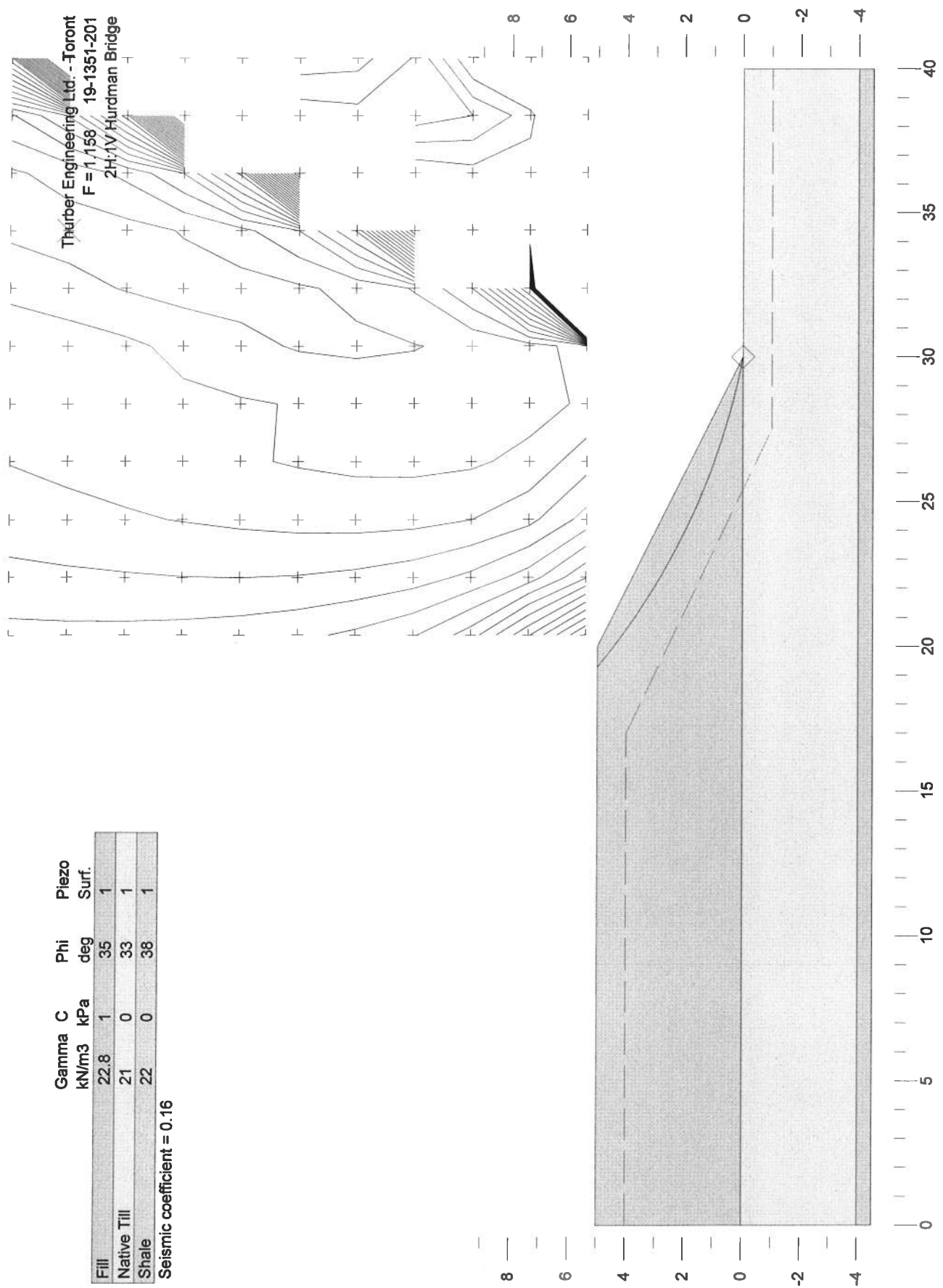


FIGURE E-2

	Gamma C kN/m ³	Phi deg	Piezo Surf.
Fill	22.8	1	35
Native Till	21	0	33
Shale	22	0	38

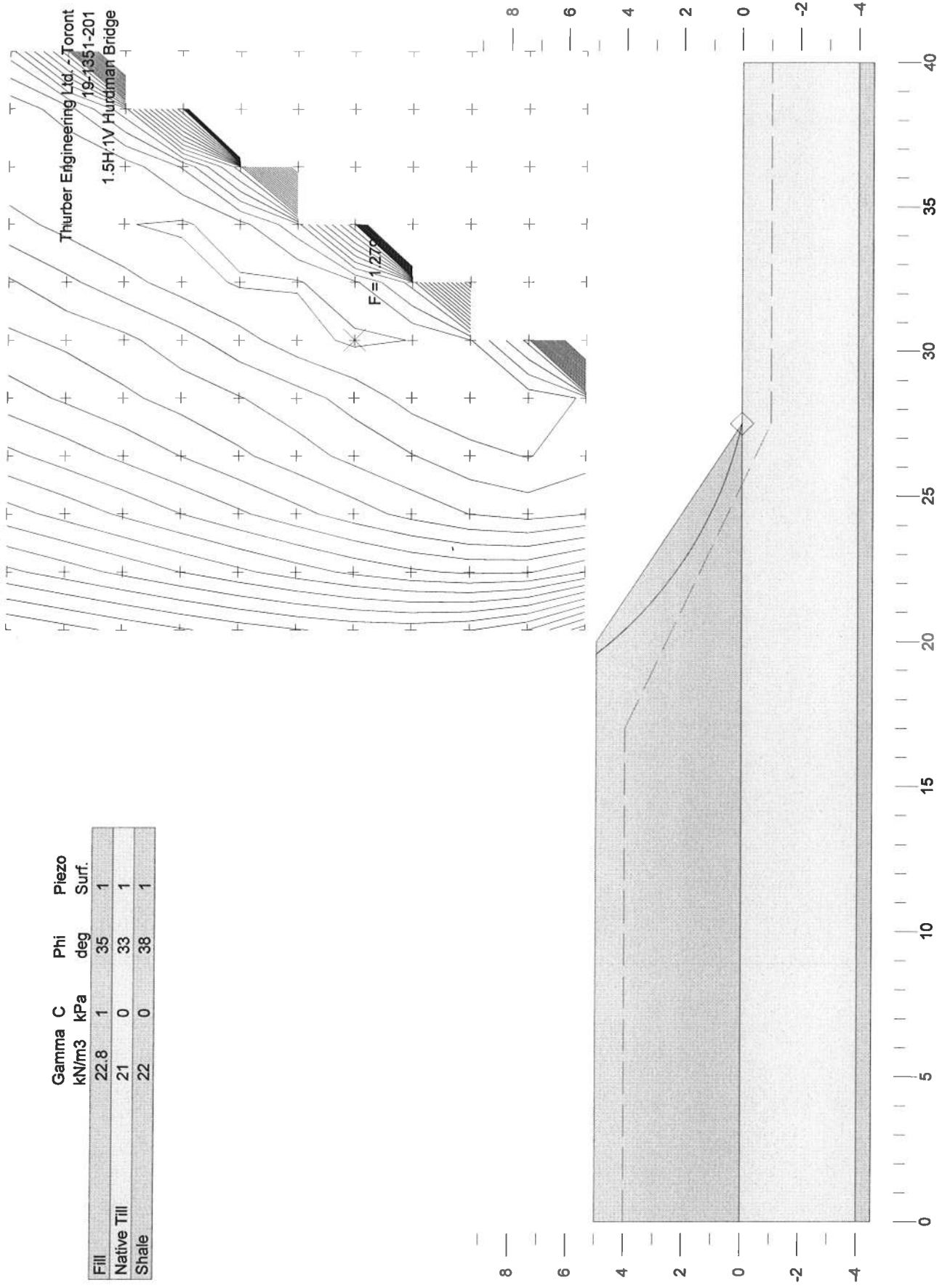


FIGURE E-3

	Gamma C kN/m3	Phi deg	Piezo Surf.
Fill	22.8	1	35
Native Till	21	0	33
Shale	22	0	38

Seismic coefficient = 0.16

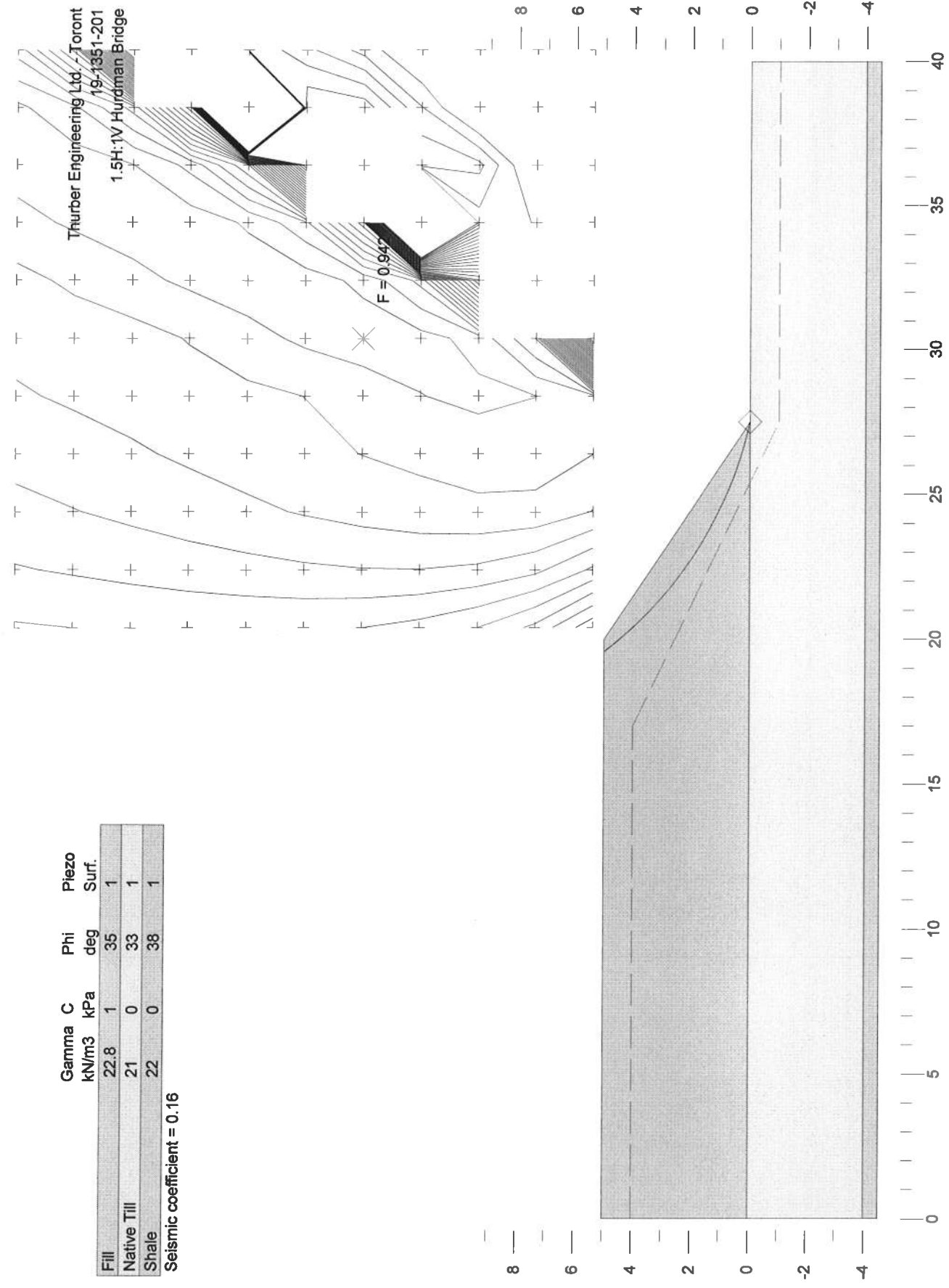
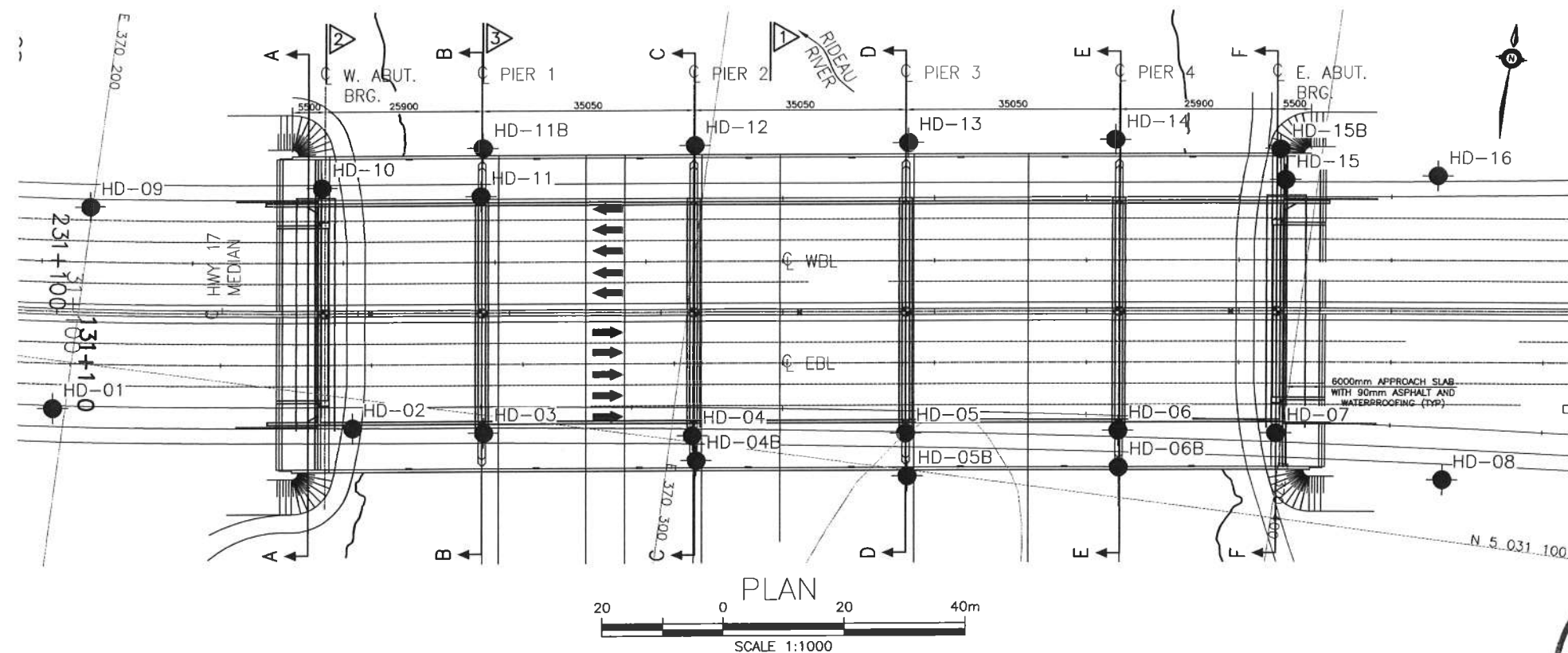


FIGURE E-4

Appendix F

Drawing

Borehole Locations and Soil Strata

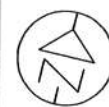


METRIC
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN



CONT No
WP No 4157-11-01

HIGHWAY 417 EXPANSION
NICHOLAS STREET TO OR 174
RIDEAU RIVER (HURDMAN) BRIDGE
BOREHOLE LOCATIONS AND SOIL STRATA I



SHEET
41



McCORMICK RANKIN
A member of MCM GROUP



THURBER ENGINEERING LTD.



KEYPLAN

LEGEND

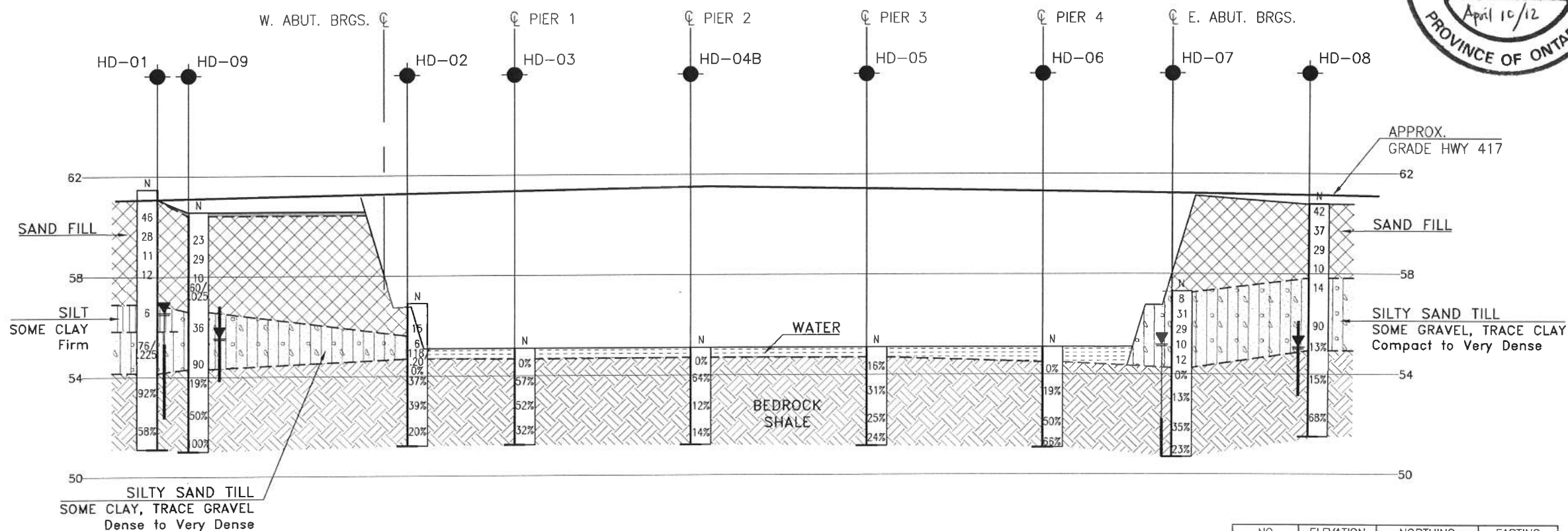
◆	Borehole
◆	Borehole and Cone
N	Blows /0.3m (Std Pen Test, 475J/blow)
CONE	Blows /0.3m (60° Cone, 475J/blow)
PH	Pressure, Hydraulic
▽	Water Level
↑	Head Artesian Water
⊥	Piezometer
90%	Rock Quality Designation (RQD)
A/R	Auger Refusal

NO	ELEVATION	NORTHING	EASTING
HD-01	61.5	5 031 092.3	370 198.0
HD-02	56.9	5 031 095.1	370 247.8
HD-03	54.7	5 031 097.0	370 269.3
HD-04	54.7	5 031 101.0	370 303.6
HD-04B	55.1	5 031 097.0	370 304.8
HD-05	55.1	5 031 106.0	370 338.5
HD-05B	55.1	5 031 099.0	370 339.7
HD-06	55.2	5 031 111.0	370 373.1
HD-06B	55.1	5 031 105.0	370 374.0
HD-07	57.3	5 031 113.6	370 398.9
HD-08	60.8	5 031 110.4	370 427.0
HD-09	60.6	5 031 125.7	370 199.7
HD-10	57.5	5 031 133.9	370 237.4

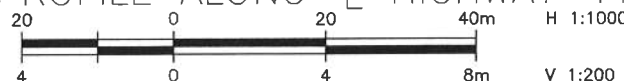
-NOTES-

- 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. 31G5-245



PROFILE ALONG CL HIGHWAY 417



NO	ELEVATION	NORTHING	EASTING
HD-11	55.3	5 031 136.0	370 263.6
HD-11B	55.1	5 031 144.0	370 262.9
HD-12	55.0	5 031 149.0	370 297.6
HD-13	55.2	5 031 154.0	370 332.5
HD-14	55.1	5 031 159.0	370 366.2
HD-15	57.4	5 031 156.0	370 394.7
HD-15B	57.6	5 031 160.9	370 393.2
HD-16	60.5	5 031 160.4	370 419.3

DATE	BY	DESCRIPTION
DESIGN	LRB	CHK LRB
DRAWN	AN	CHK
DATE	APR. 2012	
STRUCT	LDWG	2

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

CONT No
WP No 4157-11-01

HIGHWAY 417 EXPANSION
NICHOLAS STREET TO OR 174
RIDEAU RIVER (HURDMAN) BRIDGE
BOREHOLE LOCATIONS AND SOIL STRATA II



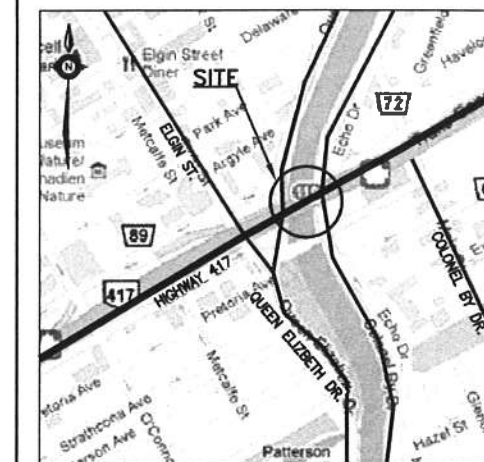
SHEET
42



McCORMICK RANKIN
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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

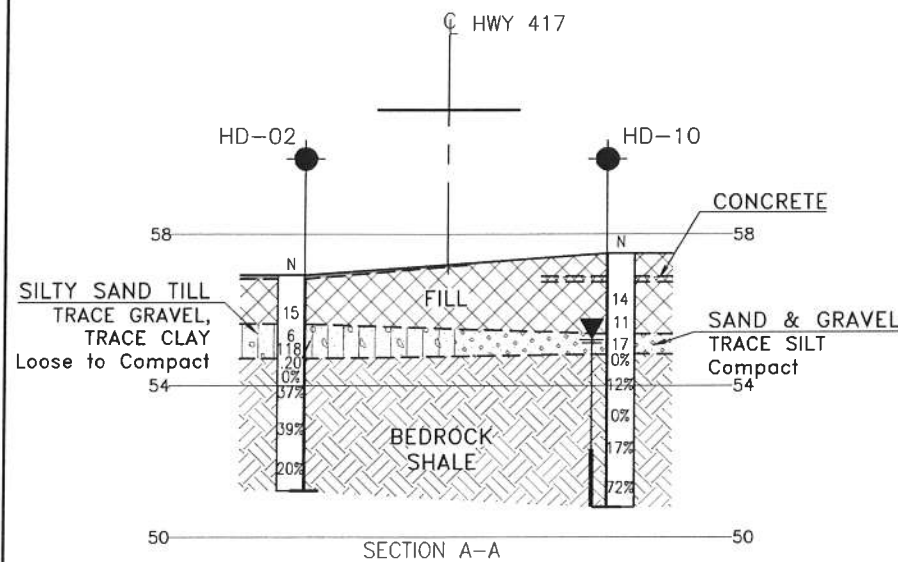
NO	ELEVATION	NORTHING	EASTING
HD-01	61.5	5 031 092.3	370 198.0
HD-02	56.9	5 031 095.1	370 247.8
HD-03	54.7	5 031 097.0	370 269.3
HD-04	54.7	5 031 101.0	370 303.6
HD-04B	55.1	5 031 097.0	370 304.8
HD-05	55.1	5 031 106.0	370 338.5
HD-05B	55.1	5 031 099.0	370 339.7
HD-06	55.2	5 031 111.0	370 373.1
HD-06B	55.1	5 031 105.0	370 374.0
HD-07	57.3	5 031 113.6	370 398.9
HD-08	60.8	5 031 110.4	370 427.0
HD-09	60.6	5 031 125.7	370 199.7
HD-10	57.5	5 031 133.9	370 237.4

-NOTES-

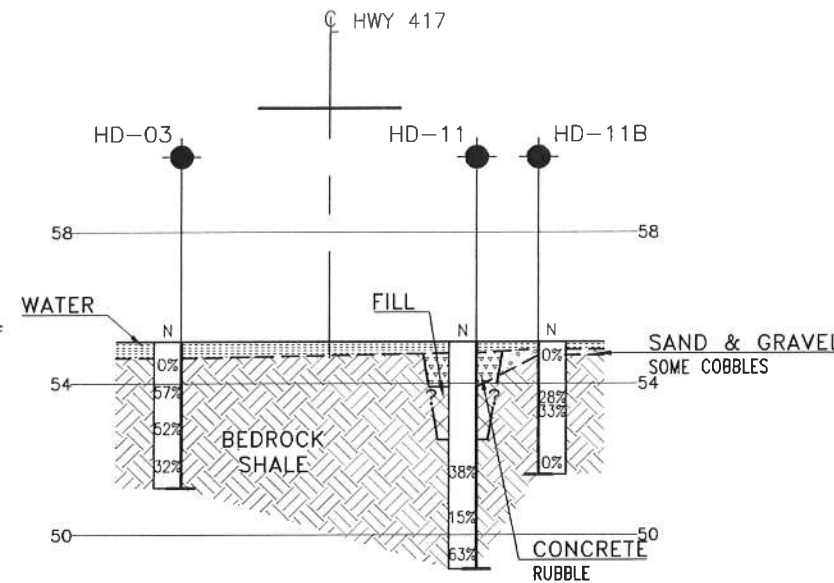
- The boundaries between soil strata have been established only at Borehole locations. Between Boreholes the boundaries are assumed from geological evidence.
- This drawing is for subsurface information only. Surface details and features are for conceptual illustration.

GEOCRES No. 31G5-245

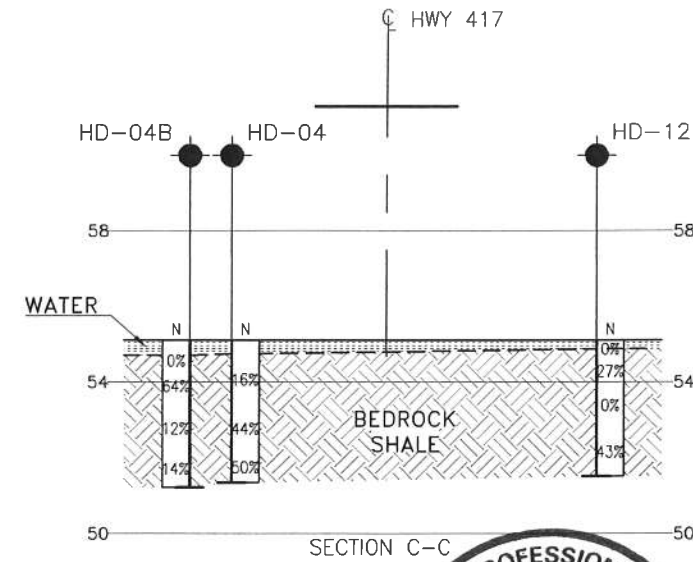
DATE	BY	DESCRIPTION
DESIGN	LRB	CHK LRB
DRAWN	AN	CHK
		SITE
		STRUCT
		DWG 3



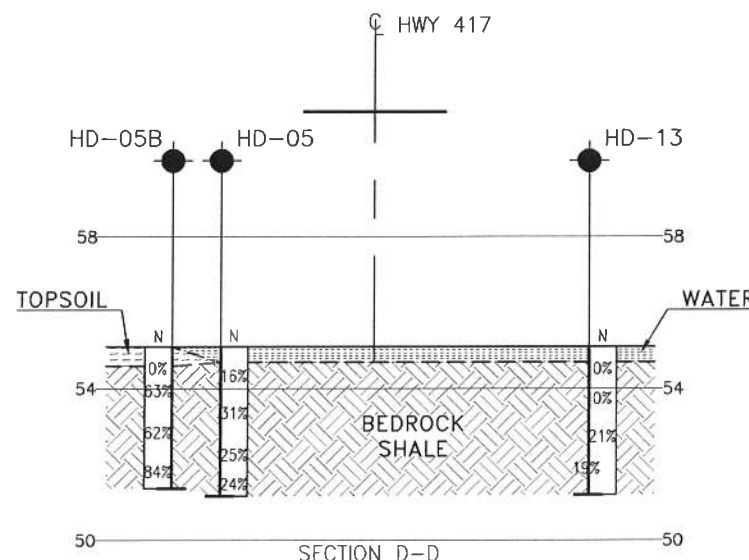
SECTION A-A



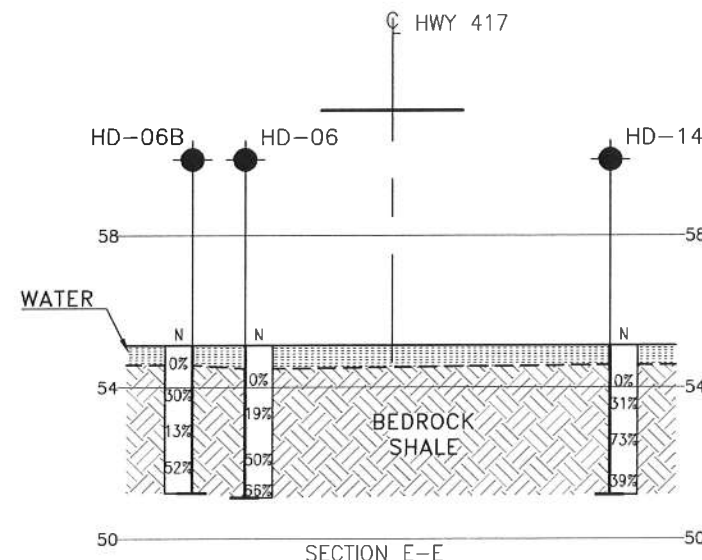
SECTION B-B



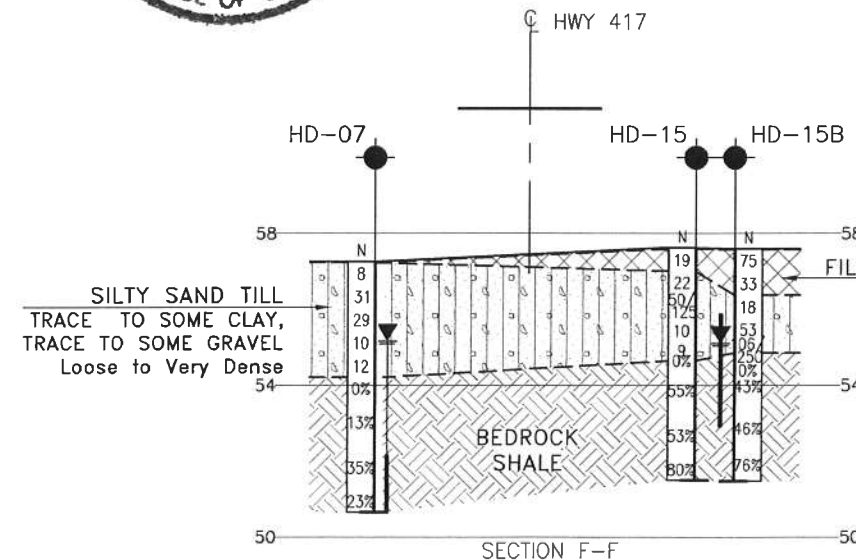
SECTION C-C



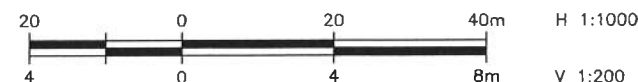
SECTION D-D



SECTION E-E



SECTION F-F



H 1:1000

V 1:200

NO	ELEVATION	NORTHING	EASTING
HD-11	55.3	5 031 136.0	370 263.6
HD-11B	55.1	5 031 144.0	370 262.9
HD-12	55.0	5 031 149.0	370 297.6
HD-13	55.2	5 031 154.0	370 332.5
HD-14	55.1	5 031 159.0	370 366.2
HD-15	57.4	5 031 156.0	370 394.7
HD-15B	57.6	5 031 160.9	370 393.2
HD-16	60.5	5 031 160.4	370 419.3