

**FOUNDATION INVESTIGATION AND DESIGN REPORT
HIGHWAY 17
CPR OVERHEAD BRIDGE AT ROSSPORT
DISTRICT OF THUNDER BAY, ONTARIO**

G.W.P. 6103-10-00, Site No. 48C-24

Geocres Number: 42D-28

Report to

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on Hwy 17 at Rossport\CP Overhead at Rossport-FIDR-
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PART 1: FACTUAL INFORMATION

1 INTRODUCTION

This report presents the factual findings obtained from a foundation investigation conducted at the site of a proposed replacement of the existing bridge which carries Highway 17 over the CP tracks at Rosspport. The bridge is located approximately 300 m west of Main Street in the Rosspport Community, District of Thunder Bay, Ontario.

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

Thurber carried out the investigation as a sub-consultant to McCormick Rankin (MRC), under the Ministry of Transportation Ontario (MTO) Agreement Number 6010-E-0011.

In the preparation of this report and in addition to the boreholes drilled under the current assignment, reference has been made to information on subsurface conditions contained in a previous foundation report. The title of this report is listed as follows:

- Foundation Investigation Report, C.P.R. Overpass, T.C.H. #17, Revised Location, Rosspport, Geocres 58-F-277C, prepared by Trow, Soderman and Associates, dated May 26, 1958.

2 SITE DESCRIPTION

The site of this investigation is located at the crossing of Highway 17 over CP tracks, approximately 300 m west of Main Street in the Community of Rosspoint, Thunder Bay District, Ontario. At present, the highway crosses the railway tracks on a skewed three-span structure supported on two abutments and two piers. Each span is 19.8 m long. The total length of the bridge is 59.4 m and the width is 10.3 m. The existing embankment heights are approximately 5.0 m to 9.0 m for the west approach and 10.0 m to 15.0 m at the east approach.

The area surrounding the bridge site generally slopes gently towards the east-south. The immediate areas to north and east of the site are treed. Bedrock outcrops are present on the west side of the bridge. Cobbles, boulders and/or rockfill were observed on the embankment slope surface during the field investigation. Lake Superior is located approximately 200 m south from the existing bridge.

Photographs in Appendix G show the general nature of the site.

The site lies within the physiographic region known as the Wabigoon Subprovince of the Superior Province of the Canadian Shield. The region is characterized by granite rocks. Locally, a sand layer was encountered above the bedrock.

3 SITE INVESTIGATION AND FIELD TESTING

The site investigation and field testing for this project were carried out from May 23 to 31, 2012 and consisted of drilling and sampling ten boreholes (numbered RPT-01 to RPT-10) through the existing highway embankment in the area of the existing and proposed west and east approaches and abutments. Four test pits (numbered TP-01 to TP-04) were conducted along the toe of the existing west embankment (at railway track level between the existing west abutment and the west pier), to establish the depth to bedrock.

Initially, both a three span and a single span bridge replacement were considered at this site. The field investigation was completed for a three span option.

Boreholes RPT-01 and RPT-10 were drilled near the west and east approaches and terminated at 9.1 m and 9.8 m depth (elevations 199.6 and 193.7), respectively. Boreholes RPT-02, RPT-03, RPT-08 and RPT-09 were drilled near the existing west and east abutments and extended to 7.9 m to 16.2 m depth (elevations 188.5 to 200.1). Boreholes RPT-04, RPT-05, RPT-06 and RPT-07 were drilled near the location of the proposed piers for the three span option and near the location of the new east and west abutments and terminated at depths ranging from 9.9 m to 17.5 m (elevations 188.4 to 197.6).

Bedrock was proved in Boreholes RPT-02 to RPT-05, RPT-07 and RPT-08 by NQ size diamond coring.

Test pits excavated along the toe of the existing west embankment were terminated on bedrock at 0.6 m and 0.8 m depth. Bedrock was exposed at one test pit location.

Records of boreholes drilled during the current investigation are included in Appendix A.

Records of Boreholes drilled during the previous investigation (Geocres 58-F-277C) and their respective laboratory test results are enclosed in Appendix C.

The approximate locations of the boreholes are shown on the attached Borehole Locations and Soil Strata Drawing in Appendix H. The coordinates and elevations of the boreholes are listed on the drawings and are presented on the individual Record of Borehole Sheets in Appendix A. MRC provided plan drawings to obtain the co-ordinates and the ground surface elevations for the boreholes.

The borehole locations were marked in the field and utility clearances were obtained prior to drilling.

The drilling was carried out from the highway grade using a CME 55 truck-mounted drill rig. NW casing was used to advance the boreholes through the soils and NQ coring methods were used to advance the boreholes through the cobbles and boulders encountered in the highway embankment fill and through the bedrock. Soil samples were obtained at selected intervals using a split spoon sampler in conjunction with Standard Penetration Testing (SPT).

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 rock samples for transport to Thurber's laboratory for further examination and testing.

All rock cores were logged, and the Total Core Recovery (TCR), Rock Quality Designation (RQD) and the Fracture Indices (FI) were determined.

Two standpipe piezometers consisting of 19 mm PVC pipe with slotted screen and enclosed in filter sand were installed at this site to permit longer term groundwater level monitoring. The boreholes were backfilled with bentonite holeplug in general accordance with O.Reg. 903 upon completion. The location and completion details of the piezometer and boreholes are presented in Table 3.2. The piezometers were decommissioned on July 24, 2012 in accordance with O.Reg. 903.

Table 3.2 – Borehole Abandonment Details

Location	Borehole/ Test pit	Piezometer Tip Depth/ Elevation (m)	Abandonment Details
West approach	RPT-01	None installed	Borehole backfilled with bentonite from 9.1 m to 0.1 m, then asphalt to surface.
Existing west abutment	RPT-02	7.9/200.1	Sand from 7.9 m to 5.9 m, bentonite holeplug from 5.9 m to 2.0 m, sand from 2.0 m to 0.4 m, then asphalt to surface.
	RPT-03	None installed	Borehole backfilled with holeplug from 8.1 m to 0.1 m, then asphalt to surface.
Proposed west abutment	RPT-04	None installed	Borehole backfilled with holeplug from 11.0 m to 7.0 m, concrete from 0.3 m to 0.1 m, then asphalt to surface.
	RPT-05	None installed	Borehole backfilled with holeplug from 9.9 m to 5.7 m. At bridge deck, borehole backfilled with concrete from 0.3 m to 0.06 m, then asphalt to surface.
Proposed east abutment	RPT-06	None installed	Borehole backfilled with holeplug from 11.8 m to 5.0 m. At bridge deck, borehole backfilled with concrete to 0.3 m to surface.
	RPT-07	None installed	Borehole backfilled with holeplug from 17.5 m to 4.8 m. At bridge deck, borehole backfilled with concrete from 0.3 m to 0.15m, then asphalt to surface.
Existing east abutment	RPT-08	None installed	Borehole backfilled with holeplug from 16.2 m to 0.9 m, concrete from 0.9 m to 0.1 m, then asphalt to surface.
	RPT-09	14.6/190.3	Sand from 14.6 m to 12.5 m, holeplug form 12.5 m to 0.3 m, sand from 0.3 m to 0.1 m, then asphalt to surface.
East approach	RPT-10	None installed	Borehole backfilled with holeplug from 9.8 m to 0.1 m, then asphalt to surface.

4 LABORATORY TESTING

The recovered soil samples were subjected to Visual Identification (VI) and to natural moisture content determination. Selected samples were also subjected to grain size distribution analyses (sieve and hydrometer). The results of this testing program are summarized on the Record of Borehole sheets in Appendix A and shown on the figures contained in Appendix B.

Point load tests were carried out on selected samples of intact bedrock upon arrival at the laboratory to assist in evaluation of the compressive strength of the bedrock. Results of point load tests on the rock core samples are included in Appendix B and on the Record of Borehole sheets in Appendix A.

5 DESCRIPTION OF SUBSURFACE CONDITIONS

Reference is made to the Record of Borehole sheets in Appendix A. Details of the encountered soil stratigraphy are presented in these sheets and on the “Borehole Locations and Soil Strata” drawing in Appendix H. An overall description of the stratigraphy is given in the following paragraphs. However, the factual data presented in the Record of Borehole Sheets governs any interpretation of the site conditions. It must be recognized that soil conditions may vary between and beyond borehole locations.

In general terms, the stratigraphy encountered at this site consists of pavement structure overlying the embankment granular fill. Native sand and gravelly sand were encountered below the east approach embankment fill. A thin layer of clayey silt was encountered below the sand in one borehole drilled near the new east abutment. Grey, pink and white granite bedrock as well as auger refusal on probable bedrock were encountered below the approach fill on the west side of the CP overhead and below the native layers of sand, gravelly sand and clayey silt on the east side of the structure.

More detailed descriptions of the individual strata are presented below.

5.1 Pavement Structure

Pavement structure was encountered in all the boreholes at this site, which were drilled through the existing Highway 17 roadway. The thickness of the asphalt ranged from 50 mm to 88 mm. A layer of concrete ranging from 225 mm to 455 mm in thickness was encountered below the asphalt in Boreholes RPT-03 to RPT-07, which were drilled through the bridge deck and approach slabs. The concrete layer was 810 mm thick in Borehole RPT-08.

Granular fill was encountered below the asphalt and concrete.

5.2 Sand and Sand and Gravel Fill

Fill was encountered below the pavement structure in the boreholes drilled through Highway 17 embankment, except in Boreholes RPT-04 to RPT-07. These four boreholes encountered fill which forms the forward slope. Fill was also encountered surficially in Test pits TP-02 to TP-04, drilled near the toe of the west embankment slope, at the railway track level, between the existing west abutment and west pier.

The fill comprising the existing highway embankment, consisted of the following distinct soil layers:

- West approach and west abutment (Boreholes RPT-01 to RPT-05)

Brown sand and gravel fill containing cobbles, boulders, possible rockfill and trace to some silt and clay. Coring through cobbles and boulders encountered in the fill was required to advance the boreholes. Boulders and cobbles (and

possible rockfill) are visible near the lower part of the forward and side embankment slopes, below the existing abutments and along the side embankment slopes, as shown in photographs in Appendix G. It is not confirmed whether some rockfill is present in the approach embankment. It must be recognized that embankment fills are heterogeneous in nature and may contain obstructions such as cobbles, boulders or rockfill.

Test pits drilled near the west abutment, revealed surficial sand and gravel fill.

- East approach and east abutment (Boreholes RPT-06 to RPT-10)

An upper layer of brown sand and gravel fill was contacted surficially in Borehole RPT-06 and below the approach slab and asphalt in Boreholes RPT-08 to RPT-10. Below the sand and gravel fill, and surficially in Borehole RPT-07, sand fill was encountered.

The thickness of the granular fill ranged from 4.7 m to 10.7 m. In Boreholes RPT-04 to RPT-07, the thickness of the fill varied from 0.8 m to 7.2 m.

In Test pits TP-02 to TP-04, the thickness of the sand and gravel ranged from 0.6 m to 0.8 m.

In Boreholes RPT-02 to RPT-05, drilled near the west abutment, the depth to the base of the fill ranged from 5.2 m to 7.8 m (elevations 199.7 to 202.9). The depth to the base of the fill in Boreholes RPT-06 to RPT-09, drilled at the east abutment, varied from 10.7 m to 12.0 m (elevation 193.1 to 194.2). Boreholes RPT-01 and RPT-10 drilled at the west and east approaches, were terminated in the granular embankment fill at 9.1 m and 9.8 m depth (elevations 199.6 and 193.7), respectively.

SPT N-values recorded in the sand and gravel fill at the west abutment generally ranged from 15 blows per 0.3 m of penetration to 50 blows for not penetration, indicating a compact to very dense relative density. Only two samples revealed SPT N-values of 8 and 7 blows per 0.3 m of penetration, indicating a loose relative density.

At the east abutment, the SPT N-values are lower. The SPT N-values measured in the fill in Boreholes RPT-06 and RPT-07, drilled near the forward slope, typically ranged from 0 to 7 blows per 0.3 m of penetration, indicating a very loose to loose relative density. In Boreholes RPT-08 to RPT-10, the SPT N-values ranged from 8 to 61 blows per 0.3 m of penetration, indicating a loose to very dense relative density. In Borehole RPT-08, an SPT N-value of 153 blows per 0.225 m of penetration was recorded on a probable cobble near elevation 195.5.

The moisture content of samples of the sand/sand and gravel fill generally ranged from 1% to 29%.

Grain size distribution curves for sand fill and sand and gravel fill samples are presented on the Record of Borehole sheets and on Figures B1 to B3 of Appendix B. The results of the laboratory tests are summarized as follows:

Soil Particles	Sand and Gravel Fill Percentage (%)	Sand Fill Percentage (%)
Gravel	30 to 54	0 to 14
Sand	43 to 66	83 to 98
Silt and Clay	1 to 11	2 to 9

5.3 Sand

Native brown sand containing trace to some gravel, trace silt and clay and occasional cobbles and boulders was contacted below the east granular approach fill at depths ranging from 10.7 m to 12.0 m (elevations 193.1 to 194.2) in Boreholes RPT-07 to RPT-09.

The depths to the base of the sand in Boreholes RPT-07 and RPT-08 were at 13.7 m and 13.2 m (elevations 192.2 and 191.5), respectively.

Borehole RPT-09 was terminated within the sand layer at 14.6 m depth (elevation 190.3) upon refusal on probable bedrock.

SPT N-values recorded in the sand layers ranged from 19 to 50 blows per 0.3 m of penetration indicating a compact to dense relative density.

The moisture contents of samples of sand ranged from 7% to 23%.

A grain size distribution curve for one sample of the sand is presented on the Record of Borehole sheets and on Figure B4 of Appendix B. The results of the laboratory test are summarized as follows:

Soil Particles	Sand Percentage (%)
Gravel	20
Sand	78
Silt and Clay	2

5.4 Clayey Silt

A layer of grey clayey silt was contacted below the sand at 13.7 m depth (elevation 192.2) in Borehole RPT-07. The thickness of the clayey silt was 800 mm.

The depth to the base of the clayey silt was at 14.5 m (elevation 191.4).

The moisture content in the clayey silt was 24%.

5.5 Bedrock and Refusal

The overburden soils described above are underlain by granite bedrock, locally quartz diorite in Borehole RPT-04. The bedrock varied in colour from pink and grey to pink and white. The bedrock cores revealed occasional vertical and sub-vertical breaks. The bedrock was described as slightly weathered to fresh with the exception of the initial run in Borehole RPT-03 which was described as moderately weathered.

Bedrock was proved by coring in Boreholes RPT-02 to RPT-05, RPT-07 and RPT-08. Boreholes RPT-06 and RPT-09 were terminated upon auger refusal on probable bedrock or boulders. The depths and elevations of the bedrock surface encountered in the boreholes are summarized in Table 5.1.

Table 5.1 – Depths and Elevations of Top of Bedrock and Auger Refusal on Probable Bedrock or Boulders

Location	Borehole/DCPT	Top of Bedrock or Auger Refusal on Probable Bedrock or Boulders	
		Depth (m)	Elevation (m)
Behind west abutment	RPT-02 ⁽¹⁾	5.6	202.4
	RPT-03 ⁽¹⁾	5.2	202.9
Between west abutment and west pier	RPT-04 ⁽¹⁾	0.8	199.7
	RPT-05 ⁽¹⁾	1.8	200.0
Between east pier and east abutment	RPT-06	6.8	194.0
	RPT-07 ⁽¹⁾	9.7	191.4
Behind east abutment	RPT-08 ⁽¹⁾	13.2	191.5
	RPT-09	14.6	190.3

⁽¹⁾Bedrock proved by coring

Based on the borehole information, the bedrock surface generally slopes down approximately 10.9 to 12.6 m between the existing west abutment (Boreholes RPT-02 and RPT-03) and the existing east abutment (Boreholes RPT-08 and RPT-09), a distance of about 70 m. The bedrock is exposed adjacent to the west side of the west pier in an apparent rock cut (Photograph 9 in Appendix G). Four testpits excavated at the toe of the outcrop encountered bedrock at the ground surface (Testpit TP-01) and at depths ranging from 0.6 m to 0.8 m (Testpits TP-02 to TP-04).

Core recovery in the bedrock was 100%. The RQD values ranged from 81% to 100%, indicating a good to excellent rock quality. An RQD of 59%, indicating a fair rock quality, was observed in Borehole RPT-02 Run 1. An RQD of 0% was noted in Borehole RPT-04 Run 1.

The Fracture Index (FI) of the rock, expressed as fractures per 0.3 m of core, ranged from 0 to 5. A Fracture of Index greater than 15 was noted in Borehole RPT-02 Run 1. Highly

broken zones were noted in cores from Borehole RPT-03 near elevations 201.7 and 202.3, and in Borehole RPT-04 near elevation 198.3.

The estimated unconfined compressive strength of the rock cores (average per Run) generally ranged from 132 MPa to 316 MPa, indicating a very strong to extremely strong rock. Unconfined compressive strengths of 65 MPa and 99 MPa were estimated in Boreholes RPT-03 Run 1 and RPT-05 Run 1, indicating a strong rock. These estimated rock strength values are interpreted from point load tests that were conducted on rock cores recovered from the boreholes. A summary of the Point Load Test Results are presented in Appendix B.

5.6 Water Levels

Water levels were monitored in the open boreholes during and upon completion of drilling. Two standpipe piezometers were installed in Boreholes RPT-02 and RTP-09 to monitor water levels after completion of drilling. The water levels measured in the piezometer and open boreholes are summarized in Table 5.2

Table 5.2 – Water Level Measurements

Borehole	Date	Water Level (m)		Comments
		Depth	Elevation	
RPT-02	July 24, 2012	6.2	201.8	In piezometer
RPT-03	May 30, 2012	5.1	203.0	Open borehole
RPT-05	May 30, 2012	7.3	200.2	Open borehole
RPT-07	May 31, 2012	9.7	196.2	Open borehole, water observed in the fill layer
RPT-09	July 24, 2012	12.3	192.6	In piezometer

The water levels observed in Boreholes RPT-03, RPT-05 and RPT-07 are believed to represent water added into the borehole during wash-boring and rock coring operations. The piezometric reading in Borehole RPT-02, taken approximately two months after piezometer installation, likely represents water retained within the core hole from the rock coring operations or from seepage along the bedrock surface, and is not believed to represent a stabilized groundwater level.

The piezometric reading from Borehole RPT-09 indicates that the groundwater level is near elevation 192.6 at the east abutment. This level is consistent with the level of ponded water noted in the southeast quadrant of the site during the fieldwork.

The above values are short-term readings and seasonal fluctuations of the groundwater level are to be expected. In particular, the groundwater level may be at a higher elevation after the spring snowmelt or after periods of heavy rainfall.

5.7 Data from Previous Foundation Report

Five boreholes and two DCPTs were advanced at the site during the 1958 investigation carried out for the existing overhead structure. The approximate locations of these boreholes and DCPTs are included on the Borehole Locations drawing in Appendix H, and the borehole logs are reproduced in Appendix C.

The subsurface conditions encountered in the previous boreholes were variable, generally consisting of topsoil, native sand and native clay overlying granite bedrock to the south of the east abutment, and railway embankment fill (sand, gravel and boulders) over bedrock to the north of the structure.

The depths and elevations of the bedrock surface encountered in the previous boreholes are summarized in Table 5.3. In general, the bedrock surface identified in these boreholes slopes down from northwest to southeast.

Table 5.3 – Depths and Elevations of Top of Bedrock/Probable Bedrock in Previous Boreholes

Location	Borehole/DCPT	Top of Bedrock or Probable Bedrock	
		Depth below original grade (m)	Elevation (m)
South of structure	1	0.2	194.5
East pier	2	2.3	191.1
South of structure	3	6.4	186.6
East of structure	4	6.5	185.5
	5	6.7	184.6
North of structure	6	2.4	195.0
	7	1.8	195.4

It is noted that the original grades at the borehole locations varied from elevation 197.4 on the railway embankment (Borehole 6), to elevation 193.0 to the south of the highway alignment (Borehole 3), and elevation 191.3 to the east (Borehole 5).

6 MISCELLANEOUS

Borehole locations were selected and established in the field by Thurber Engineering Ltd. MRC provided plan drawings to obtain the co-ordinates and the ground surface elevations for the boreholes.

Thurber obtained utility clearances for the borehole locations prior to drilling.

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

The drilling and sampling operations in the field were supervised on a full time basis by Mr. George Azzopardi and Ms. Eckie Siu Mei of Thurber Engineering Ltd.

Routine laboratory testing was carried out by Thurber Engineering Ltd.

Overall planning and supervision of the field program was conducted by Mr. Mark Farrant, P. Eng.

Interpretation of the data and preparation of the report was carried out by Ms. R. Palomeque Reyna, P.Eng. 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 recommendations for design of a new bridge to replace the existing bridge located at the crossing of Highway 17 over the CP tracks at Rosspport. The CP overhead is located approximately 300 m west of Main Street in the Rosspport Community, District of Thunder Bay, Ontario.

The CP Overhead at Rosspport was constructed in 1959. At present, Highway 17 crosses the railway track on a skewed three-span structure supported on two abutments and two piers. The west abutment, west pier and south column of the east pier are supported on spread footings founded on bedrock. The east abutment and centre and north columns of the east pier are supported on H-piles (12 BP @53") driven to bedrock. Each span is 19.8 m long. The total length of the bridge is 59.4 m and the width is 10.3 m.

At the bridge location, Highway 17 grade varies from the west abutment to the east abutment from elevation 207.5 to 205.8. The ground surface elevation at the railway track level is near elevation 198.0. The existing embankment height at the west approach varies from about 5.0 m on the north side to 9.0 m on the south side. At the east approach, the height at the face of the embankment is approximately 8.0 m; however, the original ground surface slopes down to the east and the embankment fill height increases to about 14.5 m behind the abutment.

Based on the preliminary General Arrangement (GA) drawing provided by MRC, the current design concept calls for replacement of the existing structure with a single-span structure consisting of deck supported on a precast pre-stressed concrete box girders supported on caissons

socketted into bedrock. The abutments will consist of free-standing RSS structures retaining the approach fills. The total length and width of the structure will be 37.0 m and 12.9 m, respectively. Highway grades will not be revised.

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

8 STRUCTURE FOUNDATIONS

The subsurface stratigraphy and depth to bedrock encountered at this site vary between the west and east abutment locations:

- In Boreholes RPT-04 and RPT-05 drilled along the alignment of the proposed west abutment. Bedrock was encountered 0.8 m to 1.8 m (elevation 199.7 to 200.0) below the west forward slope surface. Bedrock outcrops at the toe of this slope. Test pits excavated near the toe of the west forward slopes encountered bedrock at 0.6 m to 0.8 m (elevations 197.2 to 197.4). At TP-01, bedrock was exposed at the surface (elevation 198.0).
- In Boreholes RPT-06 to RPT-07 drilled along the alignment of the proposed east abutment, bedrock/probable bedrock was encountered at 6.8 m to 9.7 m below the east forward slope surface (Elev. 194.0 and 191.4), underlying 5.2 m to 7.2 m of sand and gravel fill and 1.6 m to 1.7 m of native sand. An 800 mm thick layer of soft clayey silt was encountered immediately above the bedrock in Borehole RPT-07.

The investigation carried out in 1958 for the existing structure (Reference 1) identified a deposit of soft clay ranging in thickness from approximately 1.0 to 6.0 m below the original grade south and east of the existing east abutment (Boreholes 2 and 3 in Appendix C). It was recommended that this clay layer be removed and replaced with granular material prior to construction of the east approach embankment. Current Boreholes RPT-08 and RPT-09 did not encounter this soft clay and appear to confirm that the clay was removed from below the east approach embankment. The presence of a soft clayey silt layer in Borehole RPT-07, also encountered in an earlier borehole drilled at about the same location (Borehole 2, Appendix C), indicates that this layer was not removed from in front of the existing abutment.

The piezometric reading from Borehole RPT-09 indicates that the water level is near elevation 192.6 at the east abutment. Perched water may be present in the fill immediately above the bedrock surface.

Based on existing site conditions, initial consideration was given to the following foundation types:

- Spread footings on native soils
- Spread footings on bedrock
- Spread footings on engineered fill

- Augered Caissons (drilled shafts)
- Drilled-in pipe piles socketed into bedrock
- Driven steel H-piles founded on bedrock

A comparison of the foundation alternatives based on advantages and disadvantages of each one is included in Appendix D.

8.1 Spread Footings on Native Soils

Bedrock was contacted immediately below the existing embankment fill at the west abutment, and therefore spread footings on native soils are not a foundation option at this location.

In Boreholes RPT-06 and RPT-07 drilled at the proposed east abutment, compact native sand was contacted below the fill at depths of 5.2 m and 7.2 m (Elev. 195.6 and 193.9). However, the geotechnical resistance of the native sand is relatively low and is considered inadequate for design of foundations to support the abutment. Further, a soft clayey silt layer is present below the sand in Borehole RPT-07. The use of spread footings on native soils is therefore not recommended at the east abutment.

In light of the above factors, the foundation option of spread footings on native soils was not further developed.

8.2 Spread Footings on Bedrock

Consideration was given to supporting the bridge on spread footings founded on bedrock. Table 8.1 indicates the depths and elevations where bedrock was contacted at the west and east abutments.

Table 8.1 – Depth/Elevation of Bedrock Surface or Auger Refusal

Foundation Unit	Borehole	Top of Bedrock	
		Depth (m)	Elevation (m)
West Abutment	RPT-04 ⁽¹⁾	0.8	199.7
	RPT-05 ⁽¹⁾	1.8	200.0
East Abutment	RPT-06	6.8	194.0
	RPT-07 ⁽¹⁾	9.7	191.4

⁽¹⁾Bedrock proved by coring

At the west abutment, constructing spread footings at the elevations indicated in Table 8.1 is considered feasible. However, it is noted that the footings will be located above a bedrock cut apparently excavated for construction of the existing railway and west pier (refer to Photograph 9). Test pits excavated immediately west of the existing west pier

indicate that the bedrock level at the base of the cut face range from Elev. 197.2 to 198.0, approximately 1.7 to 2.8 m below the bedrock levels encountered along the proposed abutment alignment. It is recommended that the new west foundation footing be located no closer than 3.0 m from the exposed cut face of the bedrock outcrop. If this cannot be accommodated, the founding level should be lowered below a line inclined upward at 1H:1V from the cut base by excavation of the bedrock. The footing should be placed on level bedrock surface.

Prior to construction, the geometry of the bedrock surface under the west abutment footing should be inspected by exposing the bedrock surface. If a sloping bedrock is encountered under the footprint of the footing, the bedrock surface should be levelled by excavation of bedrock and placing mass concrete to the design base of the footings. As indicated above, the west perimeter of the footing must be set back at least 3.0 m from the crest of the bedrock cut.

At the east abutment, the bedrock surface is 6.8 m to 9.7 m below the east header slope surface and approximately 4.0 m to 6.6 m below grade at the railway level. Excavation for footing construction on bedrock will extend through cohesionless fill and native soils below the groundwater level requiring shoring and dewatering/groundwater control prior to excavation. Spread footings founded on bedrock are therefore not the preferred foundation alternative at the east abutment.

Spread footings bearing on undisturbed bedrock at or below the elevations presented in Table 8.1 may be designed for the following geotechnical resistance:

- Factored geotechnical resistance of 3,000 kPa at Ultimate Limit States (ULS)

The SLS condition will not govern design of footings founded on bedrock.

This resistance value is for vertical, concentric loads. Where eccentric or inclined loads are applied, the resistance used in design must be reduced in accordance with the CHBDC Clause 6.7.3 and Clause 6.7.4.

Excavation and backfilling for the footings must be in accordance with OPSS 902.

The bearing surface should be prepared by removing all loose/disturbed material and shattered/loosened rock fragments. Areas requiring subexcavation beneath the underside of footing should be backfilled with the same class of concrete as used in the footing. The same value of resistance as the bedrock may be used where concrete of suitable strength is poured in neat contact with a clean, sound bedrock surface.

If rock excavation is required, excavation must be carried out using pneumatic breakers or other methods that will avoid shattering and disturbing the bedrock on which foundations will be constructed.

8.2.1 Lateral Resistance of Footings on Bedrock

The horizontal resistance of footings on bedrock may be computed using an unfactored friction factor of 0.7 for concrete poured on clean sound bedrock.

If the frictional component is insufficient, the horizontal resistance may be increased by dowelling the footing into the rock mass. Using lower bound values for the strength of the rock, an ultimate horizontal resistance of 2.6 MN may be assumed for a 50 mm steel dowel embedded at least 1.0 m into the rock. The shearing resistance of the selected dowel must be checked structurally.

8.3 Spread Footings on Engineered Fill

Bedrock was contacted immediately below the existing embankment fill at the west abutment, and therefore spread footings on engineered fill are not a foundation option at this location.

In Boreholes RPT-06 and RPT-07 drilled at the proposed east abutment, compact native sand was contacted below the existing loose to very loose fill at depths of 5.2 m and 7.2 m (Elev. 195.6 and 193.9). Consideration can be given to placing spread footings on an engineered fill pad constructed over the native sand and bedrock.

The engineered fill should bear on compact native sand and the highest permitted founding/base elevations at which engineered fill should be placed are given in Table 8.2.

Table 8.2 – Highest Permitted Elevations for Base of Engineered Fill

Foundation Unit	Borehole	Engineered Fill Base	
		Depth below existing ground surface (m)	Founding Elevation
East Abutment	RPT-06	5.2	195.6
	RPT-07	7.2	193.9

In view of the 1.7 m difference between the base elevations recommended in Table 8.2, the base of the engineered fill pad should be stepped up from south to north or alternatively established at a consistent elevation of 193.9 for the full width of the abutment.

The engineered fill must consist of OPSS Granular “A” placed in 150 mm lifts and compacted to 100% of its SPMDD at $\pm 2\%$ of optimum moisture content and generally conforming to the geometry illustrated in Figure 1 in Appendix D.

Provided a minimum footing width of 2.0 m is maintained, footings bearing on an engineered fill pad at least 3.0 m thick over native sand or constructed on bedrock may be designed for the following values:

- Factored geotechnical resistance of 600 kPa at Ultimate Limit States (ULS)
- Geotechnical resistance of 350 kPa at Serviceability Limit States (SLS)

These resistance values 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.

For footings designed on the basis of the geotechnical resistance values given above, total settlement under a footing is expected to not exceed 25 mm. Differential settlements are not expected to exceed 20 mm across the width of the structure.

The lateral resistance of the footings founded on engineered fill may be computed using an unfactored friction coefficient of 0.6. This is an “ultimate” value and requires a degree of sliding movement to occur to fully mobilize the resistance.

In light of the relatively deep excavation required to construct the engineered fill pad, this option is not recommended.

8.4 Socketted Drilled Shafts/Caissons

Caissons socketed into bedrock may be employed to support the structural loads this site. Bedrock was encountered and proved by coring at elevations given in Table 8.1.

The vertical geotechnical resistance computed for 0.76 m, 0.91 m, 1.2 m and 1.5 m diameter sockets for various socket depths below the bedrock surface (defined in Table 8.1) are presented in Table 8.3.

Table 8.3 – Recommended Resistance Values for Caisson Design

Caisson Diameter (m)	Socket Length below bedrock surface (m)	Factored Geotechnical Resistance at ULS (kN)
0.76	1.5	2,500
	2.0	3,250
	3.0	5,000
0.91	1.5	3,000
	2.0	4,000
	3.0	6,000
1.2	1.5	3,500
	2.0	5,000
	3.0	7,500
1.5	1.5	4,500
	2.0	6,500
	3.0	9,500

The vertical geotechnical resistances were computed using the method outlined in the Canadian Foundation Engineering Manual, 4th Edition, Section 18.6.4. The resistance values are based on shaft resistance within the bedrock socket only; end-bearing resistance has been ignored in anticipation of difficulties cleaning and inspecting the caisson base below the water level.

The SLS condition will not govern for caissons socketted into the rock.

The selection of a suitable socket depth will be governed by axial loads, lateral load and maximum shear and moment demand on each caisson. The depth of rock socket should not be less than 1.5 m and the axial load, shear and moment demands may require a deeper depth of rock socket.

The assessment of rock socket depth allows for the presence of some weathered and broken up rock just below the bedrock surface. Since the elevation of the bedrock surface is variable across the site and there is evidence of cobbles and boulders above the bedrock, it is critical to determine in the field during inspection of caisson installation that the entire depth of socket is formed in bedrock and not partly in cobbles and boulders and partly in bedrock. This issue is addressed in an NSSP included in Appendix F.

8.4.1 Caisson Socket Lateral Resistance

The ultimate passive force that can be mobilized by the embedded portion of a socket within rock is constant with depth and is given by:

$$P_p = 6 \cdot c \cdot D \cdot L$$

Where

c = 2,000 kPa (equivalent Mohr-Coulomb cohesion based on Hoek and Brown rock mass classification)

L = Depth of socket in rock, m

D = Socket diameter, m

Lateral resistance of any section of caisson located a horizontal distance of less than 2 m from the rock cut face at the west abutment should be neglected.

8.4.2 Caisson Socket Installation

Caisson installation must be in accordance with OPSS 903.

Caisson installation will require excavation through cohesionless layers of fill, native sand and gravelly sand below the groundwater table and construction of sockets in the

underlying bedrock. The installation of caissons at this site must consider the following issues:

- The installation method must prevent collapse of caisson sidewall and washing of cohesionless soils into the rock socket.
- The caisson installation equipment must be able to get through cobbles, boulders and large fragments of rock in the soils above the bedrock.
- The strength and hardness of the bedrock at this site must be considered when selecting equipment to excavate the rock socket. Blasting to facilitate rock removal is not permitted.

Selection of the methods and equipment employed to address the above issues is the responsibility of the Contractor. The contract documents must contain a statement to alert bidders of the above facts. The wording for an NSSP addressing this issue is included in Appendix F. This NSSP must be included in the tender documents.

8.5 Drilled-in pipe pile socketed into bedrock

A foundation alternative to support the bridge consists of 610-mm diameter drilled-in steel pipe piles socketed into bedrock and filled with concrete. Driving pipe piles through the fill containing boulders will be difficult and hence not recommended.

For drilled in pipe piles socketed into bedrock, the vertical geotechnical resistance estimated for 455 mm and 610 mm diameter pipe piles for a socket depth of 1.5 m below the bedrock surface are presented in Table 8.4.

Table 8.4 – Recommended Resistance Values for Drilled in Pipe Piles

Pipe pile diameter (mm)	Socket length below bedrock surface (m)	Factored Geotechnical Resistance at ULS (kN) of each pipe pile filled with concrete
455	1.5	2,600
610	1.5	4,000

The factored Geotechnical Resistance at ULS is based on the structural resistance of the pile section, with the end-bearing resistance reduced to account for the following factors:

- The tip of the pipe pile will not be in direct contact with the bedrock. The area of contact between the bedrock and the teeth of the cutting shoe at the tip of a pipe

pile will be less than the full contact area between a shoeless pipe pile and the bedrock.

- Cleaning the rock socket from debris and water will be difficult.

The above resistances are for pipe wall thickness of 12.5 mm, steel yield strength of 245 MPa and concrete strength of 30 MPa. The depth of the socket will be governed by the lateral resistance requirement, base fixity requirement and shear and moment demand for each pile. The structural resistance of the pipe pile must be checked by the structural engineer.

8.5.1 Pipe Pile Installation

Installation of pipe piles must follow OPSS 903 specifications.

The method of installation of the pipe piles is the responsibility of the Contractor. One option for installing the pipe piles is to drill them in by a Rotary Duplex Drilling Method. The Contractor's drilling method must be capable of dislodging, removing or penetrating obstructions such as cobbles, boulders or large rock fragments in the overburden soils. Care must be exercised while drilling the socket within the bedrock; the drilling methodology must be capable of excavating the bedrock to the specified socket dimensions without disturbing or fracturing the bedrock forming the sidewalls and base of the socket. Blasting to facilitate rock removal is not permitted.

The drilling method must also maintain sidewall stability of the drilled hole and allow cleaning of the socket without cohesionless soils running into the socket.

Since the rock cutting shoe at the tip of a pipe pile will be slightly larger in diameter than the outside diameter of a pipe pile, there will be a small gap between the rock socket wall and the pipe pile. It is recommended that the annular space between the pipe pile and socket wall be grouted to the bedrock surface to achieve fixity.

During and subsequent to installation, the pipe pile will be partially filled with water and it may not be practical to dewater the pipe prior to concreting. Tremie concreting will be required for concreting these pipe piles.

An NSSP addressing the above issues is included in Appendix F.

8.5.2 Lateral Resistance for Pipe Pile Sockets

The ultimate passive force that can be mobilized by the embedded portion of a pipe pile socket within rock is constant with depth and is given by the lateral resistance formula presented in Section 8.4.1 of this report.

8.6 Steel H-piles driven to refusal on bedrock

Consideration was given to supporting the abutments on driven steel H-piles founded on bedrock.

H-piles driven to bedrock are not recommended at the west abutment due to the presence of very shallow bedrock. The H-piles will be very short and will require socketing into the rock to provide sufficient lateral stability or fixity. Additionally, cobbles, boulders and rockfill are present within the highway embankment fill, and pre-augering or removal of these obstructions will be required to install the H-piles.

H-piles driven to bedrock can be installed at the east abutment, where no major obstructions (cobbles, boulders, rockfill) were encountered in the boreholes. The elevations at which bedrock was contacted and the piles are expected to meet refusal are given in Table 8.1.

The axial, factored geotechnical resistance at Ultimate Limit States (ULS_f) for an H-Pile section 310x110 driven to refusal on bedrock is 2,000 kN.

The SLS condition will not govern for piles founded on the bedrock.

The factored structural resistance of the piles at ULS must be checked by the structural designer as per Section 6.8.8 of the CHBDC.

Oversize materials (e.g. greater than 75 mm nominal diameter) must not be used in any fills through which the piles will be driven.

8.6.1 Pile Tips

The tips of all piles must be fitted with pile tip protection from an approved manufacturer such as Titus Steel (Standard H-point) or approved equivalent. Pile tip protection is recommended for setting the piles on potentially sloping bedrock and tip protection if cobbles or boulders are encountered above the bedrock.

8.6.2 Pile Installation

Pile installation should be in accordance with OPSS 903.

For piles installed for the tolerances shown in Clause 903.07.05.01 of the Specification, the foundation drawing should include the note "Piles to be driven to bedrock".

If the proposed bridge design requires that the deviation at the top of the pile be limited to tight tolerances, a driving template or other means may be required to achieve the specified maximum deviation.

8.6.3 Downdrag

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

8.6.4 Lateral Resistance for H-piles

The lateral resistance of the piles within cohesionless fill and native sand at the east abutment may be calculated using a value for the coefficient of horizontal subgrade reaction (k_s) and ultimate lateral resistance (p_{ult}) as follows:

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

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

where

z = depth of embedment of pile in metres

D = pile width/diameter in metres

n_h = value from Table 8.5

γ = unit weight (Table 8.5)

K_p = passive earth pressure coefficient (Table 8.5)

Table 8.5 – Parameters for Lateral Pile Resistance

Location	Elevation	C_u (kPa)	n_h (kN/m ³)	K_p	Unit Weight (kN/m ³)	Soil Conditions
East Abutment North End (BH RPT-06)	198.0 to 195.6	-	2,000	3.0	20	Sand and Gravel, loose to very loose (FILL)
	195.6 to 194.0	-	3,500	3.3	10.5*	Sand, compact
East Abutment South End (BH RPT-07)	198.0 to 193.9	-	2,000	3.0	20	Sand, loose to very loose (FILL)
	193.9 to 192.2	-	3,500	3.3	10.5*	Sand, compact
	192.2 to 191.4	30	-	2.7	9*	Clayey Silt, soft

*Buoyant unit weight below the water table.

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

The spring constant, K , for analysis may be obtained by the expression, $K = k_s \cdot L \cdot D$ (kN/m), where k_s is the coefficient of horizontal subgrade reaction (kN/m³), D is the pile

width/diameter (m) and L is the length (m) of the pile segment or element used in the analysis. The ultimate lateral resistance on any one segment of pile, P_{ult} , may be obtained from the expression, $P_{ult} = p_{ult} * L * D$. This represents the ultimate load at which the pile fails and will not support any additional load at greater displacements. It is recommended, however, that the total lateral resistance assumed in a HP 310x110 pile driven to refusal on bedrock or be limited to no more than 110 kN at ULS and 40 kN at SLS.

For lateral soil/pile group interaction analysis, the modulus of subgrade reaction (k_s) may have to be reduced based on pile spacing.

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

Pile Spacing Perpendicular to Direction of Loading	Horizontal Subgrade Reaction Reduction Factor, R
4 D*	1.00
1 D*	0.50

* D is the width of the pile, and spacing is measured centre to centre

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

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

Intermediate values may be obtained by interpolation.

8.7 Recommended Foundation

From a geotechnical perspective, the recommended foundation alternative to support bridge abutments at this site would be spread footings on bedrock for the west abutment and H-piles driven to bedrock for the east abutment. However, considering the conceptual design of the structure, drilled shafts or caisson socketed into bedrock or drilled-in pipe piles socketed into bedrock are considered feasible alternatives.

8.8 Frost Cover

The design depth of frost penetration at this site is 2.2 m.

However, frost penetration is not an issue for footings bearing on bedrock or concrete fill placed on bedrock.

Frost protection should be provided for buried pile caps, if used, and should consist of a minimum of 2.2m of soil cover.

9 BACKFILL TO ABUTMENTS

The current design concept calls for construction of RSS abutments to contain the approach fills. Recommendations regarding RSS wall design are provided in a subsequent section. Recommendations regarding conventional abutments are provided in this section in the event that the design concept changes.

In the case of integral or semi-integral abutments, backfill to the abutment must be granular material. In the case of a conventional abutment, granular backfill is recommended but rock backfill can be permitted. The rock fill used as backfill for the abutment should be limited to fragments no greater than 250 mm as per OPSS 206.

Backfill to the abutments should consist of Granular A or Granular B Type II or Type III material meeting the requirements of Special Provision 110S13 “Amendment to OPSS 1010, April 2004”. The backfill must be in accordance with OPSS 902, and placed to the extents shown in OPSD 3101.150.

All new embankment fill should be placed in regular lifts and be compacted in accordance with OPSS 501. Also, compaction equipment to be used adjacent to retaining structures must be restricted in accordance OPSS 501.

The design of the abutment must incorporate a subdrain as shown in OPSD 3101.150 or OPSD 3101.200, as applicable.

10 LATERAL EARTH PRESSURES

Lateral earth pressures acting on the structure may be assumed to be triangular and to be governed by the characteristics of the abutment backfill. For a fully drained condition, the pressures should be computed in accordance with the CHBDC but generally are given by the expression:

$$p_h = K * (\gamma h + q)$$

Where:

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

K = earth pressure coefficient (see table below)

γ = unit weight of retained soil (see table below)

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

q = value of any surcharge (kPa)

Earth pressure coefficients for backfill to the abutment wall are dependent on the material used as backfill. Typical values are shown in Table 10.1.

Table 10.1 – Earth Pressure Coefficient (K)

Condition	Earth Pressure Coefficient (K)							
	OPSS Granular A or OPSS Granular B Type II $\phi = 35^\circ, \gamma = 22.8 \text{ kN/m}^3$		OPSS Granular B Type I and Type III $\phi = 32^\circ, \gamma = 21.2 \text{ kN/m}^3$		Rock Fill (Limited to 250 mm size) $\phi = 42^\circ, \gamma = 19 \text{ kN/m}^3$		Existing sand and gravel fill $\phi = 32^\circ$ $\gamma = 21.2 \text{ kN/m}^3$	
	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)
Active (Unrestrained Wall)	0.27	0.40*	0.31	0.48*	0.2	0.28*	0.31	0.48*
At rest (Restrained Wall)	0.43	-	0.47	-	0.33	-	0.47	-
Passive (Movement Towards Soil Mass)	3.7	-	3.3	-	5.0	-	3.3	-

* For wing walls if employed.

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

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

The factors in Table 10.1 are “ultimate” values and require certain movements for the respective conditions to be mobilized. The values to use in design can be estimated from C6.16 in the Commentary to the Canadian Highway Bridge Design Code.

11 RETAINED SOIL SYSTEMS

The current design concept calls for abutments comprising Retained Soil Systems (RSS). The preliminary RSS Layout drawings indicate that the RSS height above existing grade will vary from about 7.5 m at the south end to 6.5 m at the north end of the west abutment, and from about 5.5 m at the south end to 8.0 m at the north end of the east abutment. The base of the RSS walls will taper upwards behind the abutment walls parallel to the roadway alignment.

In general, RSS walls used in conjunction with the new abutments must be “High Performance” and “High Appearance”. The contract drawings should include information on the longitudinal

alignment of the wall in plan, the top and base elevations of the wall in profile, cross-sectional space constraints and an NSSP for the RSS wall.

To provide an acceptable foundation performance, the RSS mass must be founded on competent soils, bedrock or engineered fill. Failure to provide an adequate foundation may lead to settlement and distortion of the RSS and, in severe cases, to possible failure of the system. It is critical that the RSS walls are not subject to settlement due to compression of the foundation soils and embankment fill. The foundation of the entire RSS mass must be considered, i.e. from the face of the wall to the furthest extent of the reinforcement.

11.1 Retained soil system at the west abutment

RSS wall layouts provided by MRC in October 2012 indicate that the base of the RSS wall rises towards the west (away from the west abutment). The variation of the design founding levels for the RSS walls at the west approach, as well as soil conditions at the proposed founding levels are presented in Table 11.1.

Table 11.1 – RSS founding levels at the West Abutment

RSS	Location	Boreholes	Top of Bedrock	Proposed RSS Founding levels	Soil conditions at the RSS founding level
1	North side of West abutment (13 m long)	RPT-02 (15 m west of the west abutment)	202.4	From east to west 199.9 to 205.7	Dense to compact sand and gravel fill above bedrock
		RPT-04 (west abutment)	199.7		
	Abutment face (15.45 m long)	RPT-05 (west abutment)	200.0	199.2 (south side)	Bedrock
		RPT-04 (west abutment)	199.7	199.7 (north side)	Bedrock
2	South Side of West Abutment (24.0 m long)	RPT-05 (west abutment)	200.0	From east to west 199.2 to 206.5	Bedrock from the west abutment to 12 m west.
		RPT-03 (17 m west of the west abutment)	202.9		Loose to very dense sand and gravel fill above the bedrock.

At some locations, generally on the west side, the RSS structure will be founded on bedrock and some bedrock excavation will be required to achieve the design founding level. It may be possible to adjust the design founding level of the RSS wall to avoid rock excavation.

A factored geotechnical resistance at ULS of 3,000 kPa is available for design of the RSS system on bedrock. Settlement is not an issue for an RSS system constructed on bedrock and the SLS condition does not apply.

As the elevation of the wall base rises towards the west (away from the west abutment), the RSS wall will be founded on the existing sand and gravel fill with cobbles and

boulders. The RSS may be constructed on the existing compact to very dense granular fill and designed using the following geotechnical resistances:

Factored Geotechnical Resistance at ULS 450 kPa

Geotechnical Resistance at SLS 300 kPa

Prior to construction, the geometry of the bedrock surface under the RSS wall at the west abutment should be inspected by exposing the bedrock surface. If a sloping bedrock is encountered under the proposed RSS wall location, the bedrock surface should be levelled by excavation of bedrock and placing concrete fill to the design base of the RSS wall. It is recommended that the base on the RSS walls be located no closer than 3.0 m from the exposed cut face of the bedrock outcrop.

If rock excavation is required, excavation must be carried out using pneumatic breakers or other methods that will avoid shattering and disturbing the bedrock on which foundations will be constructed.

11.2 Retained soil system at the east abutment

Based on RSS wall layouts provided by MRC in October 2012, the base of the RSS wall rises towards the east (away from the east abutment). The variation of the design founding levels for the RSS walls at the east approach, as well as soil conditions at the proposed founding levels are presented in Table 11.2.

Table 11.2 – RSS founding levels at the East Abutment

RSS	Location	Boreholes	Top of Bedrock	Proposed Founding levels	Soil conditions at the RSS founding level
3	North side of East abutment (27.0 m)	RPT-08 (18 m east of the east abutment)	191.5	From west to east 197.1 to 202.1	Very loose to compact sand and gravel fill near the abutment to loose to compact sand fill
		RPT-06 (east abutment)	194.0		
	Abutment face (15.45 m)	RPT-06 (east abutment)	194.0	From north to south 197.1 to 200.1	Very loose to loose sand and gravel fill
		RPT-07 (east abutment)	191.4		
4	South Side of East Abutment (15.0 m)	RPT-07 (east abutment)	191.4	From west to east 200.1 to 203.2	Very loose to loose sand fill to elevation 201.0, then compact sand/sand and gravel fill to elevation 204.9
		RPT-09 (16 m east of the east abutment)	190.3		

At the east abutment, sand to sand and gravel fill and native sand (and a thin layer of soft clayey silt locally) are present over the bedrock. In general, the fill is loose to very loose in front of (west of) the existing abutment, and loose to compact behind (east of) the abutment. To provide a competent founding surface and to minimize the potential for post-construction settlement of the RSS, the existing embankment fill should be excavated and replaced with granular engineered fill to a depth of 1.5 m below the proposed founding base of the RSS indicated in the GA drawings.

Engineered fill placed under the RSS mass to achieve the design founding level should consist of OPSS Granular A material placed in 150 mm lifts and compacted to 100% of its SPMDD at a moisture content within 2% of optimum. The engineered pad must extend at least 1.0 m beyond the limits of the RSS mass and levelling strip.

RSS walls founded on a minimum 1.5 m thick pad of granular engineered fill overlying the fill at the east abutment may be designed for the following geotechnical resistances:

Factored Geotechnical Resistance at ULS	300 kPa
Geotechnical Resistance at SLS	200 kPa

The geotechnical resistances provided above are for concentric, vertical loading. The effects of load inclination and eccentricity need to be taken into account according to the CHBDC 2006 Section 6.7.

In order to avoid or minimize dewatering at the east abutment, it is recommended that the base of the RSS walls be above elevation 196.2.

In general, for all the RSS walls at the west and east abutments, the entire block of reinforced earth must be designed against various modes of failure including sliding and overturning. Sliding resistance along the base of the wall on engineered granular fill may be estimated using an ultimate friction coefficient of 0.55. A value of 0.7 may be used for a wall base on bedrock.

11.3 Slope Stability of the Retained soil system

The stability of an RSS abutment founded on bedrock or on the existing sand/granular fill at the west approach is not a concern.

A preliminary analysis of the global stability of the RSS at the east abutment was conducted to assess stability of a maximum 8.0 m high wall founded above the toe of the forward slope. The stability analysis was carried out using the commercially available slope stability program GEO-SLOPE, applying the Morgenstern-Price method. The slope model and the analysis results are shown in Appendix E. The computed factor of safety for the proposed RSS/slope geometry was 1.8, which exceeds the minimum value of 1.3

normally accepted for this type of analysis. Stability of the RSS abutment is therefore not considered to be an issue.

11.4 Settlement of the Retained soil system

Settlement of the RSS founded on bedrock at the west approach should be minimal and is not an issue.

At the east abutment, the construction of a maximum 8.0 m high RSS wall on a 1.5 m thick pad of granular engineered fill will induce immediate (elastic) settlement in the underlying very loose to compact cohesionless fill.

The immediate settlements were assessed using elastic methods. Based on these analyses, the settlement at the east abutment varies from 50 mm to 150 mm. The settlement increases as the wall extends behind the east abutment, where the layer of the existing fill below the base of the RSS wall is generally thicker.

Due to the non-cohesive nature of the foundation soils, this settlement will be immediate and essentially complete when construction of the RSS wall is completed.

Inspection of the RSS walls and placing of additional granular material to re-establish grades as necessary should be implemented during and after construction. If possible, placement of the wall facing should be delayed until settlement is completed.

If this magnitude of settlement and maintenance is not acceptable, consideration should be given to using lightweight fill or Elastizell as RSS backfill in order to reduce the settlements. The estimated settlement of the cohesionless fill foundation induced by 8.0 m of Elastizell and 1.0 m of granular fill varies from 10 mm to 40 mm. The differential settlement between these two values is expected to occur over a wall length of about 15 m, corresponding to rate of less than 1%. This rate of differential settlement should be acceptable but must be confirmed by the wall supplier.

For placement of new fill and where warranted, the existing slope surfaces should be appropriately benched, as per OPSD 208.010, after stripping of vegetation, topsoil, organics, soft soils or otherwise unsuitable overburden materials.

12 EXCAVATION AND GROUNDWATER CONTROL

In general, excavation is expected to be limited to the existing embankment fill for removal of the existing abutment stems, construction of the new RSS abutments, and possibly spread footing construction on bedrock at the west abutment. A deeper excavation extending approximately 1.0 m below existing grade adjacent to the railway (to approximate Elev. 197) will be required in front of the existing east abutment for subgrade improvement below the new RSS abutment.

All excavation must be carried out in accordance with the Occupational Health and Safety Act (OHSA). For the purposes of the OHSA, the cohesionless fill and native soils within the probable depth of excavation at this site may be classed as Type 3 soils above the water table and Type 4 below the water level.

The excavation of the cohesionless soils and backfilling for foundations must be carried out in accordance with OPSS 902. Roadway protection should be supplied in accordance with OPSS 539 and designed for Performance Level 2.

Discussions with the railway authorities should be undertaken to determine the required level of track protection. CP Rail may require a more stringent performance level for track protection (Performance Level 1).

If rock excavation is required, excavation must be carried out using pneumatic breakers or other methods that will avoid shattering and disturbing the bedrock on which foundations will be constructed.

Groundwater levels observed in the boreholes at the west abutment indicate that seepage or perched water may be encountered in the fill immediately above the bedrock surface. It is anticipated that this water can be handled by proper surface drainage measures and localized sump pumping.

Groundwater was measured at 12.3 m depth (Elev. 192.6) in a piezometer installed in Borehole RPT-09 at the east approach. It is anticipated that excavation for RSS construction will be maintained above the groundwater level at the east abutment, and sump pumping should be suitable to handle any seepage entering temporary excavations.

If excavation of the cohesionless native soils below the groundwater level is planned at the west and east abutments, dewatering to lower the groundwater level below the base of the excavation will be required prior to excavation. Excavation without prior dewatering is not recommended since the inflow of groundwater will cause boiling and sloughing of the soil below the water table making it difficult to maintain a dry, sound base on which to work. Unwatering must remain operational and effective until the footing is constructed and backfilled.

If excavations are kept above the groundwater level, a Ministry of Environment (MOE) Permit to Take Water (PTTW) will not be required.

The design of the dewatering system and any road/rail protection that may be required is the responsibility of the Contractor and the Contract Documents must alert him to this responsibility. All shoring systems should be designed by a Professional Engineer experienced in such designs, who will determine an appropriate support system.

Fibre optic cables or other buried utilities might be present along the CP track in the vicinity of the new foundation areas. These utilities must not be undermined or damaged during construction

of the foundation system for the new structure. The locations of any utilities should be established in relation to potential work zones, and if necessary exposed to protect them during construction of the new foundations. Relocation of, and/or special protective measures for affected utilities may be required.

13 SEISMIC CONSIDERATIONS

13.1 Seismic Design Parameters

The following seismic parameters should be used for design:

- Velocity Related Seismic Zone 0
- Zonal Velocity Ratio 0.0
- Acceleration Related Seismic Zone 0
- Zonal Acceleration Ratio 0.0
- Peak Horizontal Acceleration 0.02

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 active (K_{AE}) and passive (K_{PE}) earth pressure coefficients that incorporate the effects of earthquake loading. The coefficients of horizontal earth pressure for seismic loading presented in Table 13.1 may be used:

Table 13.1 – Earth Pressure Coefficients for Earthquake Loading

Condition	Earth Pressure Coefficient (K)			
	OPSS Granular A or OPSS Granular B Type II $\phi = 35^\circ$ $\gamma = 22.8 \text{ kN/m}^3$	OPSS Granular B Type I and Type III $\phi = 32^\circ$, $\gamma = 21.2 \text{ kN/m}^3$	Rock Fill $\phi = 42^\circ$; $\gamma = 19.0 \text{ kN/m}^3$	Existing sand and gravel fill $\phi = 32^\circ$ $\gamma = 21.2 \text{ kN/m}^3$
Active (K_{AE})*	0.28	0.32	0.21	0.32
Passive (K_{PE})	3.7	3.2	5.0	3.2
At Rest (K_{OE})**	0.45	0.50	0.36	0.50

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

** After Woods

The potential for liquefaction of the foundations soils was assessed using the Seed and Idriss (1971) method for cohesionless soils.

Using the method, it is estimated that under the existing conditions, the foundation soils at the abutments are not prone to liquefaction.

14 CONSTRUCTION CONCERNS

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

1. Caisson installation and excavation of rock sockets

Caisson installation and excavation of rock sockets at this site must consider the issues of the sloughing of cohesionless soils below the groundwater table into the socket and the hardness of the bedrock. An NSSP addressing these issues is attached.

2. Variable depth to top of bedrock

The surface of the bedrock is variable at this site. It is possible that higher or lower elevations will be encountered during construction. This may impact the length of caisson or pile required and or necessitate the use of concrete fill to achieve the design footing base elevation if footings on bedrock are selected.

3. Destabilization of excavations

If spread footings (founded on bedrock at the west abutment or/and founded on engineered fill at the east abutment) are selected as foundation alternative, seepage may be encountered within the granular embankment fill during excavation. The impact of seepage or surface water could destabilize the sides and or base of the excavation. Proper groundwater and surface water control measures must be in place prior to commencing excavation. Groundwater control measures should be implemented if excavation extends below groundwater level.

4. Pile and caisson installation

The embankment fill may contain cobbles and boulders which may affect installation of H-piles, drilled-in pipe piles or caissons. The Contractor shall be prepared to remove, drill through and/or penetrate these obstructions and extend the piles to bedrock.

5. Existing underground utilities

Fibre optic cables or other buried utilities might be present along the CP track in the vicinity of the new foundation areas. These utilities must not be undermined or damaged during construction of the foundation system for the new structure.

Engineering analysis and preparation of the foundation design report were carried out by Ms. R. Palomeque Reyna 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.

Thurber Engineering Ltd.

Rocio Palomeque Reyna, P.Eng., M.Eng.
Geotechnical Engineer



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



Report reviewed by:
P.K. Chatterji, P.Eng., Ph.D.
Review Principal



Appendix A
Record of Borehole Sheets
(Present investigation)

SYMBOLS AND TERMS USED ON TEST HOLE LOGS

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 naked eye

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

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

TERMS DESCRIBING CONSISTENCY (COHESIVE SOILS ONLY)

DESCRIPTIVE TERM	UNDRAINED SHEAR STRENGTH (kPa)	APPROX. SPT ⁽¹⁾ "N" VALUE
Very Soft	< 10	< 2
Soft	10 to 25 (POCKET PEN)	2 to 4
Firm	25 to 50 (0.5-1)	4 to 8
Stiff	50 to 100 (1-2)	8 to 15
Very Stiff	100 to 200 (2-4)	15 to 30
Hard	> 200 (>4)	> 30

(1) Standard Penetration Test – the number of blows from a 63.5kg hammer falling through 0.76m to advance a 60 degree truncated cone 0.3m

TERMS DESCRIBING DENSITY(COHESIONLESS SOILS)

DESCRIPTIVE TERM	SPT "N" VALUE
Very Loose	< 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very Dense	> 50


HIERARCHY OF SOIL STRENGTH PREDICTION

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

LEGEND FOR TEST HOLE LOGS

 Shelby Tube
 A – Casing
  SPT
  Grab/Auger sample
  Core
  No Recovery

- MC – Moisture Content (% by Weight) as determined by sample

 Water Level
 C_{vane} Shear Strength Determination by Field Insitu Vane
 C_{pen} Shear Strength Determination by Pocket Penetrometer
 C_{lab} Shear Strength Determination using a Laboratory Vane Apparatus
 C_u Undrained Shear Strength determined by Unconfined Compression Test
 AS/GS/BS Auger Sample/Grab Sample/ Block Sample
 SS Split-spoon
 SC Soil core
 AED Oedometer test
 TXL Triaxial test

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


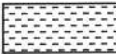



ROCK WEATHERING CLASSIFICATION

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

DISCONTINUITY SPACING

Bedding	Bedding Plane Spacing
Very thickly bedded	Greater than 2m
Thickly bedded	0.6 to 2m
Medium bedded	0.2 to 0.6m
Thinly bedded	60mm to 0.2m
Very thinly bedded	20 to 60mm
Laminated	6 to 20mm
Thinly Laminated	Less than 6mm

SYMBOLS

	CLAYSTONE
	SILTSTONE
	SANDSTONE
	COAL
	BEDROCK

STRENGTH CLASSIFICATION

Rock Strength	Approximate Uniaxial Compressive Strength		Field Estimation of Hardness*
	(MPa)	(psi)	
Extremely Strong	Greater than 250	Greater than 36,000	Specimen can only be chipped with a geological hammer
Very Strong	100-250	15,000 to 36,000	Requires many blows of geological hammer to break
Strong	50-100	7,500 to 15,000	Requires more than one blow of geological hammer to break
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 % 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 RPT-01

1 OF 2

METRIC

W.P. 6103-10-00 LOCATION N 5 411 129.3 E 267 279.2 CP Overhead at Rosspoint, Mile 14.11 ORIGINATED BY ES/GA
HWY 17 BOREHOLE TYPE Casing/NO Coring COMPILED BY AN
DATUM Geodetic DATE 2012.05.28 - 2012.05.29 CHECKED BY RPR

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						
208.7								20 40 60 80 100						
0.0	ASPHALT: (88mm)							○ UNCONFINED + FIELD VANE						
0.1	SAND and GRAVEL, trace silt and clay Dense to Compact Brown Moist (FILL)		1	SS	44			● QUICK TRIAXIAL x LAB VANE					36 56 7 (SI+CL)	
			2	SS	15									
	Cored through cobbles and boulders Cobbles (75mm) at 1.6m		3	SS	36									
	Cobbles from 2.1m to 2.3m No recovery		4	SS	15									
	Cobbles from 2.8m to 3.0m No recovery		5	SS	58/ 0.275									
	Cobbles and boudlers from 3.4m to 4.6m													
	Cored through cobbles and boulders from 4.3m to 9.1m (FILL)		6	SS	100/ 0.050									
			7	SS	50/ 0.100									
	No recovery													
			8	SS	50/ 0.150									
	No recovery													
199.6			9	SS	50/ 0.0									
9.1	END OF BOREHOLE AT 9.1m. BOREHOLE OPEN TO 9.1m AND DRY UPON COMPLETION OF DRILLING. BOREHOLE BACKFILLED WITH BENTONITE FROM 9.1m TO 0.1m													

Continued Next Page

+³.X³: Numbers refer to
Sensitivity

20
15 10 5
(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No RPT-01

2 OF 2

METRIC

W.P. 6103-10-00 LOCATION N 5 411 129.3 E 267 279.2 CP Overhead at Rosspoint, Mile 14.11 ORIGINATED BY ES/GA
HWY 17 BOREHOLE TYPE Casing/NQ Coring COMPILED BY AN
DATUM Geodetic DATE 2012.05.28 - 2012.05.29 CHECKED BY RPR

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									
							20	40	60	80	100	W _p	W	W _L			
	Continued From Previous Page THEN ASPHALT TO SURFACE.																

RECORD OF BOREHOLE No RPT-02

1 OF 1

METRIC

W.P. 6103-10-00 LOCATION N 5 411 125.4 E 267 304.7 CP Overhead at Rosspoint, Mile 14.11 ORIGINATED BY ES
 HWY 17 BOREHOLE TYPE Hollow Stem Augers/NQ Coring COMPILED BY AN
 DATUM Geodetic DATE 2012 05 25 - 2012 05 25 CHECKED BY RPR

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES		20	40	60	80	100	
208.0	ASPHALT: (63mm)											
0.0												
0.1	SAND and GRAVEL, trace silt and clay, occasional cobbles and boulders Dense to Compact Brown Moist to Damp (FILL)		1	GS								41 54 6 (SI+CL)
			1	SS	43							
			2	SS	20							
			3	SS	30							
	Very Dense		4	SS	76							
	Cored through cobbles and boulders											
	Compact Start coring at 5.4m		5	SS	27							
202.4												
5.6	BEDROCK, granite, slightly weathered to fresh, pink and grey, occasional vertical and subvertical breaks Sub-vertical fracture (125mm) at 5.8m Sub-horizontal fracture (25mm) at 5.9m		1	RUN								RUN #1 TCR=100% SCR=69% RQD=59% UCS=191MPa (Average)
			2A	RUN								RUN #2A Core barrel jammed. No core recovery
			2B	RUN								RUN #2B TCR=100% SCR=100% RQD=100% UCS=165MPa (Average)
200.1			3	RUN								RUN #3 TCR=100% SCR=100% RQD=100%
7.9	END OF BOREHOLE AT 7.9m. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen. WATER LEVEL READINGS: DATE DEPTH (m) ELEV. (m) Jul.24/12 6.2 201.8											

RECORD OF BOREHOLE No RPT-03

1 OF 1

METRIC

W.P. 6103-10-00 LOCATION N 5 411 120.8 E 267 297.3 CP Overhead at Rosspoint, Mile 14.11 ORIGINATED BY GA
HWY 17 BOREHOLE TYPE Casing/NQ Coring COMPILED BY AN
DATUM Geodetic DATE 2012.05.30 - 2012.05.30 CHECKED BY RPR

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	
208.1												
0.0												
0.1	APPROACH SLAB						208					
207.6	75mm of asphalt over 455mm of concrete											
0.5	SAND and GRAVEL											
	Compact to Loose											
	Brown		1	SS	15		207					54 43 3
	Damp											(SI+CL)
	(FILL)											
	Reddish Brown		2	SS	8		206					
	Very Dense		3	SS	50/		205					
	No recovery, spoon bouncing				0.0							
	Cored through cobbles and boulders from 3.0m to 4.5m		4	SS	50/		204					
	No recovery				0.150							
	No recovery, spoon bouncing		5	SS	50/		203					
					0.0							
202.9	Start coring at 5.2m											
5.2	BEDROCK, granite, moderately weathered, grey, occasional vertical and subvertical breaks											
	Horizontal joint at 5.6m, 6.5m		1	RUN			202					RUN #1 TCR=100% SCR=59% RQD=81% UCS=146MPa (Average)
	Highly broken zones: 125mm at 5.8 100mm at 6.4m											
	Pink and Grey											
	Fresh		2	RUN			201					RUN #2 TCR=100% SCR=100% RQD=100% UCS=178MPa (Average)
200.0												
8.1	END OF BOREHOLE AT 8.1m. WATER LEVEL AT 5.1m UPON COMPLETION OF DRILLING. BOREHOLE BACKFILLED WITH HOLEPLUG FROM 8.1m TO 0.1m, THEN ASPHALT TO SURFACE.											

+ 3 , x 3 ; Numbers refer to
Sensitivity

20
15
10

(%) STRAIN AT FAILURE

METRIC

ELEV DEPTH	SOIL PROFILE DESCRIPTION	STRAT PLOT	SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC NATURAL LIQUID LIMIT MOISTURE CONTENT		UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION (%)
			NUMBER	TYPE			"N" VALUES	20 40 60 80 100	W _p W W _L	WATER CONTENT (%) 20 40 60		
207.5							SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100				kN/m ³	GR SA SI C

BRIDGE DECK
75mm of asphalt over 265mm of concrete

Gap between underside of bridge deck and ground surface

SAND and GRAVEL, occasional cobbles
Compact
Brown
Moist
(FILL)
Layer of cobbles and boulders
Start coring at 7.6m
Cobbles from 7.5m to 7.8m

BEDROCK, quartz diorite, slightly weathered to fresh, grey, occasional vertical and subvertical breaks

Sub-vertical fracture (125mm) at 7.8m

Sub-vertical fractures (25mm to 75mm) at 8.1m, 8.5m, 8.8m, 9.3m
100mm at 8.2m
Broken zone:
75mm at 9.2m

Run #	TCR	SCR	RQD	UCS
207				
206				
205				
204				
203				
202				
201				
200	FI	>5	3	
199	3			
198	3			

Run #1
TCR=100%
SCR=0%
RQD=0%

Run #2
TCR=100%
SCR=95%
RQD=90%
UCS=215MPa (Average)

Run #3

(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No RPT-04

2 OF 2

METRIC

W.P. 6103-10-00 LOCATION N 5 411 118.6 E 267 318.7 CP Overhead at Rosspoint, Mile 14.11 ORIGINATED BY ES
HWY 17 BOREHOLE TYPE Casing/NQ Coring COMPILED BY AN
DATUM Geodetic DATE 2012.05.25 - 2012.05.25 CHECKED BY RPR

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa			WATER CONTENT (%)				
	Continued From Previous Page							20 40 60 80 100							
	BEDROCK , quartz diorite, fresh, vertical and subvertical breaks Sub-vertical fractures: 125mm at 9.7m 175mm at 10.0m Sub-vertical fractures at: 50mm at 10.3m 100mm at 10.5m		3	RUN			197							1	TCR=100% SCR=100% RQD=100% UCS=245MPa (Average)
196.4 11.0	END OF BOREHOLE AT 11.0m. BOREHOLE BACKFILLED WITH HOLEPLUG TO 0.3m, CONCRETE FROM 0.3m TO 0.1m, THEN ASPHALT TO SURFACE.													1	
														2	
														0	

+ 3 , × 3 : Numbers refer to
Sensitivity

20
15 5
10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No RPT-05

2 OF 2

METRIC

W.P. 6103-10-00 LOCATION N 5 411 113.2 E 267 313.2 CP Overhead at Rosspoint, Mile 14.11 ORIGINATED BY GA
HWY 17 BOREHOLE TYPE Casing/NQ Coring COMPILED BY AN
DATUM Geodetic DATE 2012.05.30 - 2012.05.30 CHECKED BY RPR

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa		WATER CONTENT (%)				
	Continued From Previous Page							20 40 60 80 100		20 40 60				
9.9	END OF THE BOREHOLE AT 9.9m. WATER LEVEL AT 7.3m UPON COMPLETION OF DRILLING. BOREHOLE BACKFILLED WITH HOLEPLUG FROM 9.9m TO 5.7m. AT BRIDGE DECK, BOREHOLE BACKFILLED WITH CONCRETE TO 0.06m, THEN ASPHALT TO SURFACE.							○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE						GR SA SI CL

METRIC

[illegible]

+ 3, × 3; Numbers refer to Sensitivity

RECORD OF BOREHOLE No RPT-06

2 OF 2

METRIC

W.P. 6103-10-00 LOCATION N 5 411 102.6 E 267 352.4 CP Overhead at Rosspoint, Mile 14.11 ORIGINATED BY ES
HWY 17 BOREHOLE TYPE Casing/NQ Coring COMPILED BY AN
DATUM Geodetic DATE 2012.05.25 - 2012.05.25 CHECKED BY RPR





SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE								
	Continued From Previous Page						20	40	60	80	100					
	SAND Compact Brown Moist (FILL)		7	SS	15											
194.0																
11.8	END OF BOREHOLE AT 11.8m UPON REFUSAL ON PROBABLE BEDROCK. BOREHOLE BACKFILLED WITH HOLEPLUG TO 5.0m. AT BRIDGE DECK, BOREHOLE BACKFILLED WITH CONCRETE TO SURFACE.															

RECORD OF BOREHOLE No RPT-07

2 OF 2

METRIC

W.P. 6103-10-00 LOCATION N 5 411 097.4 E 267 346.7 CP Overhead at Rosspoint, Mile 14.11 ORIGINATED BY GA
HWY 17 BOREHOLE TYPE NW Casing/NQ Coring COMPILED BY AN
DATUM Geodetic DATE 2012.05.31 - 2012.05.31 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	× LAB VANE			
	Continued From Previous Page					20	40	60	80	100	20	40	60		
193.9	SAND, trace silt and clay Loose Brown Wet (FILL)		7	SS	6										
12.0	SAND, trace silt and clay Compact Brown Wet		8	SS	19										
192.2															
13.7	Clayey SILT Soft Grey Start coring at 14.4m		9	SS	50/ 0.150										
191.4															
14.5	BEDROCK, granite, slightly weathered to fresh, pink and white, occasional vertical breaks Horizontal joints at 14.8m		1	RUN											
			2	RUN											
188.4															
17.5	END OF BOREHOLE AT 17.5m. WATER LEVEL AT 9.7m UPON COMPLETION OF DRILLING. BOREHOLE BACKFILLED WITH HOLEPLUG FROM 17.5m TO 4.8m. AT BRIDGE DECK, BOREHOLE BACKFILLED WITH CONCRETE FROM 0.3m TO 0.15m, THEN ASPHALT TO SURFACE.														

ONTMT4S 1197.GPJ 2012TEMPLATE(MTO).GDT 3/20/13

METRIC

[illegible]

+ 3, × 3: Numbers refer to Sensitivity

RECORD OF BOREHOLE No RPT-08

2 OF 2

METRIC

W.P. 6103-10-00 LOCATION N 5 411 094 8 E 267 369.0 CP Overhead at Rosspoint, Mile 14.11 ORIGINATED BY ES
HWY 17 BOREHOLE TYPE Hollow Stem Augers/Casing/NQ Coring COMPILED BY AN
DATUM Geodetic DATE 2012.05.23 - 2012.05.24 CHECKED BY RPR

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 (%)				
	Continued From Previous Page							20 40 60 80 100						
	SAND , trace gravel, trace silt and clay Compact Brown Wet (FILL)							○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE						
193.1			9	SS	22		194							
11.6	SAND , some gravel, occasional cobbles Dense Brown Wet						193							
	Layer of cobbles and boulders		10	SS	35		192							
191.5	Start coring at 13.1m													
13.2	BEDROCK , granite, slightly weathered to fresh, pink and grey, occasional vertical and subvertical breaks		1	RUN			191							
			2	RUN			190							
			3	RUN			189							
188.5														
16.2	END OF BOREHOLE AT 16.2m. BOREHOLE BACKFILLED WITH HOLEPLUG TO 0.9m, CONCRETE TO 0.1m, THEN ASPHALT TO SURFACE.													

ONTMT4S 1197.GPJ 2012TEMPLATE(MTO).GDT 3/20/13

RECORD OF BOREHOLE No RPT-09

1 OF 2

METRIC

W.P. 6103-10-00 LOCATION N 5 411 090.2 E 267 361.7 CP Overhead at Rosspoint, Mile 14.11 ORIGINATED BY GA
 HWY 17 BOREHOLE TYPE Hollow Stem Augers COMPILED BY AN
 DATUM Geodetic DATE 2012.05.31 - 2012.05.31 CHECKED BY RPR

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	
204.9	ASPHALT: (75mm)											
0.0												
0.1	SAND and GRAVEL, trace silt and clay Compact to Dense Brown (FILL)		1	SS	20							
			2	SS	24		204					30 66 4 (SI+CL)
203.4												
1.4	SAND, trace gravel, trace silt and clay Compact to Dense Brown Damp (FILL)		3	SS	15		203					
			4	SS	32		202					
			5	SS	16							
							201					
			6	SS	8		200					
	Loose											
			7	SS	9		199					
							198					
			8	SS	8		197					0 91 9 (SI+CL)
							196					
	Compact Damp		9	SS	13							
							195					

Continued Next Page

+ 3 x 3 : Numbers refer to
Sensitivity

20
15 5
10 (%) STRAIN AT FAILURE

METRIC

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT	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			20 40 60 80 100	w _p	w	w _L			WATER CONTENT (%)
								SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE						
	Continued From Previous Page.						20 40 60 80 100				20 40 60		GR SA SI C	

[illegible]

END OF BOREHOLE AT 14.6m
UPON REFUSAL ON PROBABLE
BEDROCK.
BOREHOLE OPEN TO 14.6m AND
WATER LEVEL AT 11.8m.
Piezometer installation consists of
19mm diameter Schedule 40 PVC pipe
with a 1.52m slotted screen.



WATER LEVEL READINGS:		
DATE	DEPTH (m)	ELEV. (m)
Jul. 24/12	12.3	192.6

RECORD OF BOREHOLE No RPT-10

1 OF 2

METRIC

W.P. 6103-10-00 LOCATION N 5 411 086.2 E 267 387.0 CP Overhead at Rosspoint, Mile 14.11 ORIGINATED BY ES
 HWY 17 BOREHOLE TYPE Hollow Stem Augers COMPILED BY AN
 DATUM Geodetic DATE 2012.05.23 - 2012.05.23 CHECKED BY RPR

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 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE						WATER CONTENT (%) PLASTIC LIMIT (w _P) NATURAL MOISTURE CONTENT (w) LIQUID LIMIT (w _L)						
203.5								20	40	60	80	100								
0.0	ASPHALT: (88mm)		1	SS	44		203													
0.1	SAND and GRAVEL Dense to Very Dense Brown Damp (FILL)		2	SS	55															
201.6			3	SS	23		202													
1.9	SAND, fine grained, trace gravel, trace silt and clay Compact to Dense Brown Damp (FILL)		4	SS	29		201													
			5	SS	33															
							200													
			6	SS	19		199													
							198													
			7	SS	47		197													
							195													
	Very Dense to Dense		8	SS	59															
							195													
			9	SS	47		194													
193.7																				
9.8	END OF BOREHOLE AT 9.8m																			

Continued Next Page

+³, ×³: Numbers refer to
Sensitivity

20
15 5
10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No RPT-10

2 OF 2

METRIC

W.P. 6103-10-00 LOCATION N 5 411 086.2 E 267 387.0 CP Overhead at Rossport, Mile 14.11 ORIGINATED BY ES
HWY 17 BOREHOLE TYPE Hollow Stem Augers COMPILED BY AN
DATUM Geodetic DATE 2012.05.23 - 2012.05.23 CHECKED BY RPR

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	SHFAR STRENGTH kPa					WATER CONTENT (%)			
							20	40	60	80	100	W _p	W	W _L		
	Continued From Previous Page															
	BOREHOLE DRY UPON COMPLETION OF DRILLING. BOREHOLE BACKFILLED WITH HOLEPLUG TO 0.1m, THEN ASPHALT TO SURFACE.															

RECORD OF BOREHOLE No TP-01

1 OF 1

METRIC

W.P. 6103-10-00 LOCATION N 5 411 110.1 E 267 313.6 CP Overhead at Rosspoint, Mile 14.11 ORIGINATED BY GA
 HWY 17 BOREHOLE TYPE Hand Shovel COMPILED BY AN
 DATUM Geodetic DATE 2012 05 29 - 2012 05 29 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	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa	WATER CONTENT (%)					
198.0 0.0	BEDROCK AT SURFACE. BEDROCK EXPOSED ON SOUTH WEST SIDE OF BRIDGE.													

RECORD OF BOREHOLE No TP-02

1 OF 1

METRIC

W.P. 6103-10-00 LOCATION N 5 411 112.1 E 267 320.9 CP Overhead at Rosspoint, Mile 14.11 ORIGINATED BY GA
HWY 17 BOREHOLE TYPE Hand Shovel COMPILED BY AN
DATUM Geodetic DATE 2012.05.29 - 2012.05.29 CHECKED BY RPR

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 × LAB VANE					WATER CONTENT (%) w _P w w _L				
198.0							20	40	60	80	100						
0.0	SAND and GRAVEL, occasional rootlets																
197.4	Brown																
0.6	Damp (FILL)																
	END OF TEST PIT ON BEDROCK ENCOUNTERED AT 0.6m.																

+³ . X³ : Numbers refer to Sensitivity


20
15 5
10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No TP-03

1 OF 1

METRIC

W.P. 6103-10-00 LOCATION N 5 411 113.8 E 267 327.2 CP Overhead al Rosspport, Mile 14.11 ORIGINATED BY GA
HWY 17 BOREHOLE TYPE Hand Shovel COMPILED BY AN
DATUM Geodetic DATE 2012.05.29 - 2012.05.29 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 (%) GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							WATER CONTENT (%)
								20 40 60 80 100							
198.0															
0.0	SAND and GRAVEL Brown Damp (FILL)						198								
197.2															
0.8	END OF TEST PIT ON BEDROCK ENCOUNTERED AT 0.8m.														

RECORD OF BOREHOLE No TP-04

1 OF 1

METRIC

W.P. 6103-10-00 LOCATION N 5 411 115.2 E 267 332.3 CP Overhead at Rosspoint, Mile 14.11 ORIGINATED BY GA
HWY 17 BOREHOLE TYPE Hand Shovel COMPILED BY AN
DATUM Geodetic DATE 2012.05.29 - 2012.05.29 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 (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100					
198.0 0.0	SAND and GRAVEL Brown Damp (FILL)						198							
197.4 0.6	END OF TEST PIT ON BEDROCK ENCOUNTERED AT 0.6m.													

Appendix B

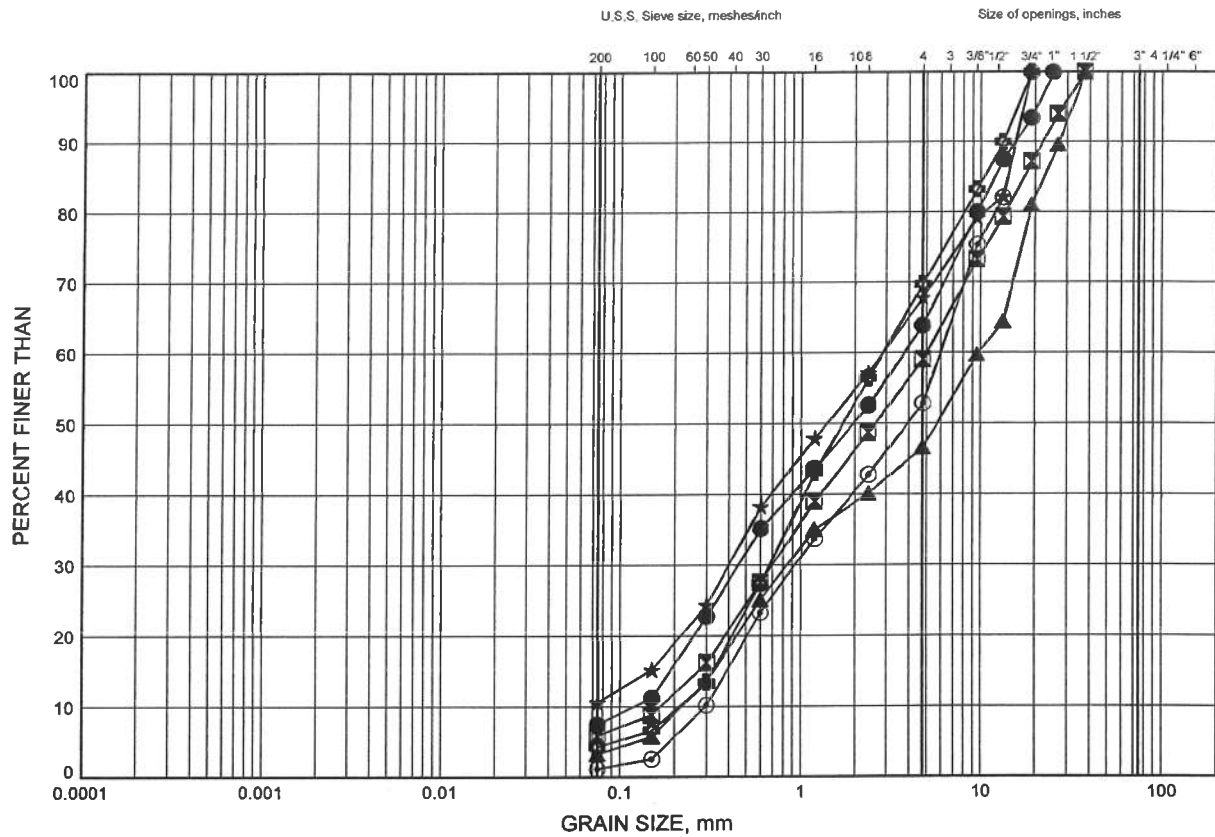
Laboratory Test Results (Present investigation)

NWR 32 Rehabs

GRAIN SIZE DISTRIBUTION

FIGURE B1

SAND & GRAVEL FILL



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	RPT-01	0.38	208.32
⊠	RPT-02	0.38	207.62
▲	RPT-03	1.07	207.03
★	RPT-05	6.10	201.40
⊙	RPT-06	5.33	200.46
⊕	RPT-09	1.07	203.83

Date July 2012

W.P.# 6103-10-00



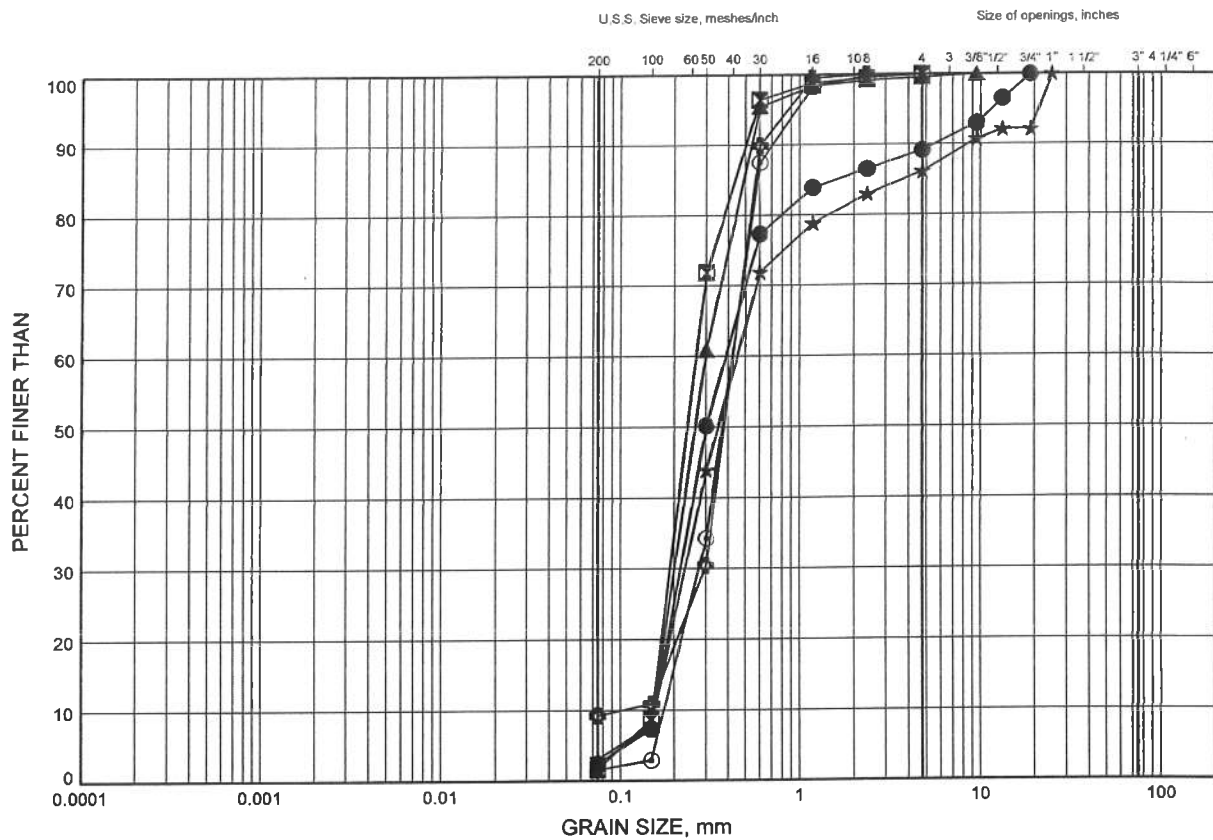
Prep'd AN

Chkd. RPR

NWR 32 Rehabs
GRAIN SIZE DISTRIBUTION

FIGURE B2

SAND FILL



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	RPT-06	8.69	197.11
⊠	RPT-07	6.71	199.19
▲	RPT-07	9.75	196.14
★	RPT-08	3.35	201.35
⊙	RPT-08	6.40	198.30
⊕	RPT-09	7.92	196.97

Date August 2012
W.P.# 6103-10-00



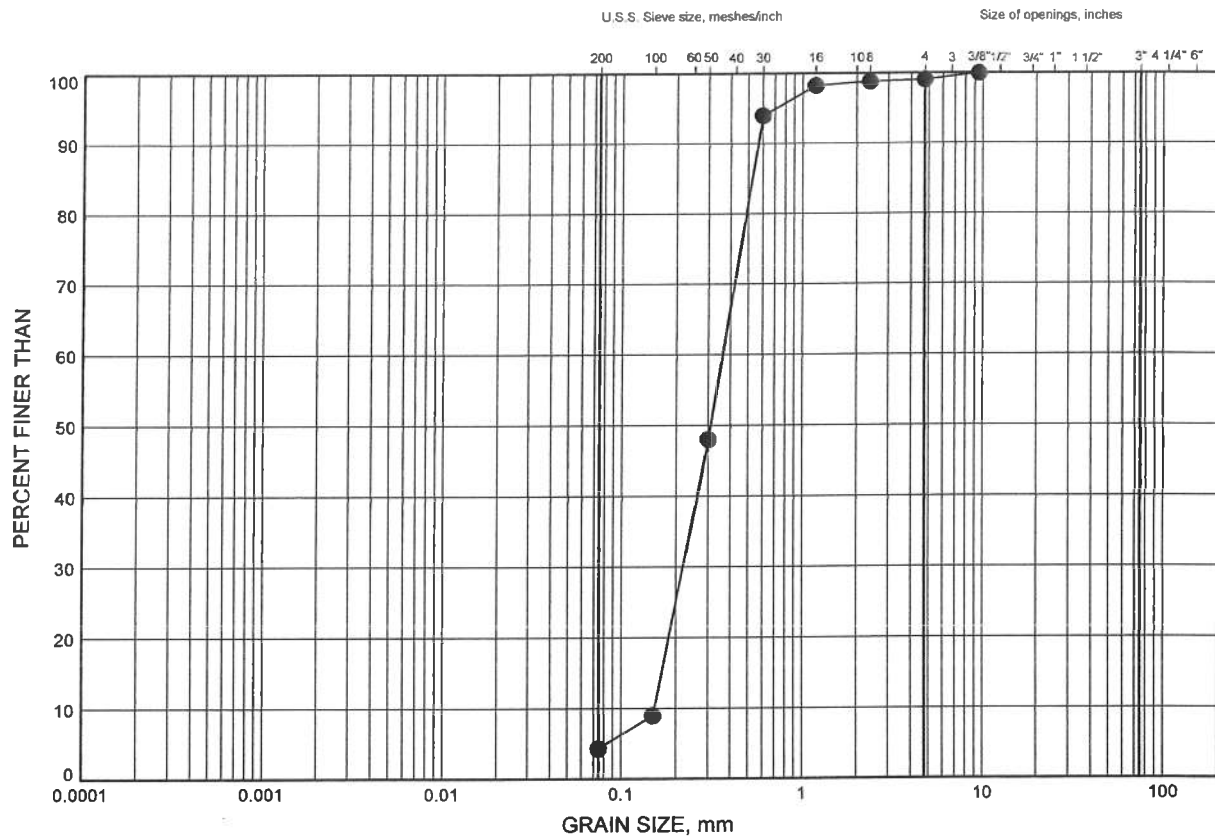
Prep'd AN
Chkd. RPR

NWR 32 Rehabs

GRAIN SIZE DISTRIBUTION

FIGURE B3

SAND FILL



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	RPT-10	2.59	200.91

Date August 2012
W.P.# 6103-10-00

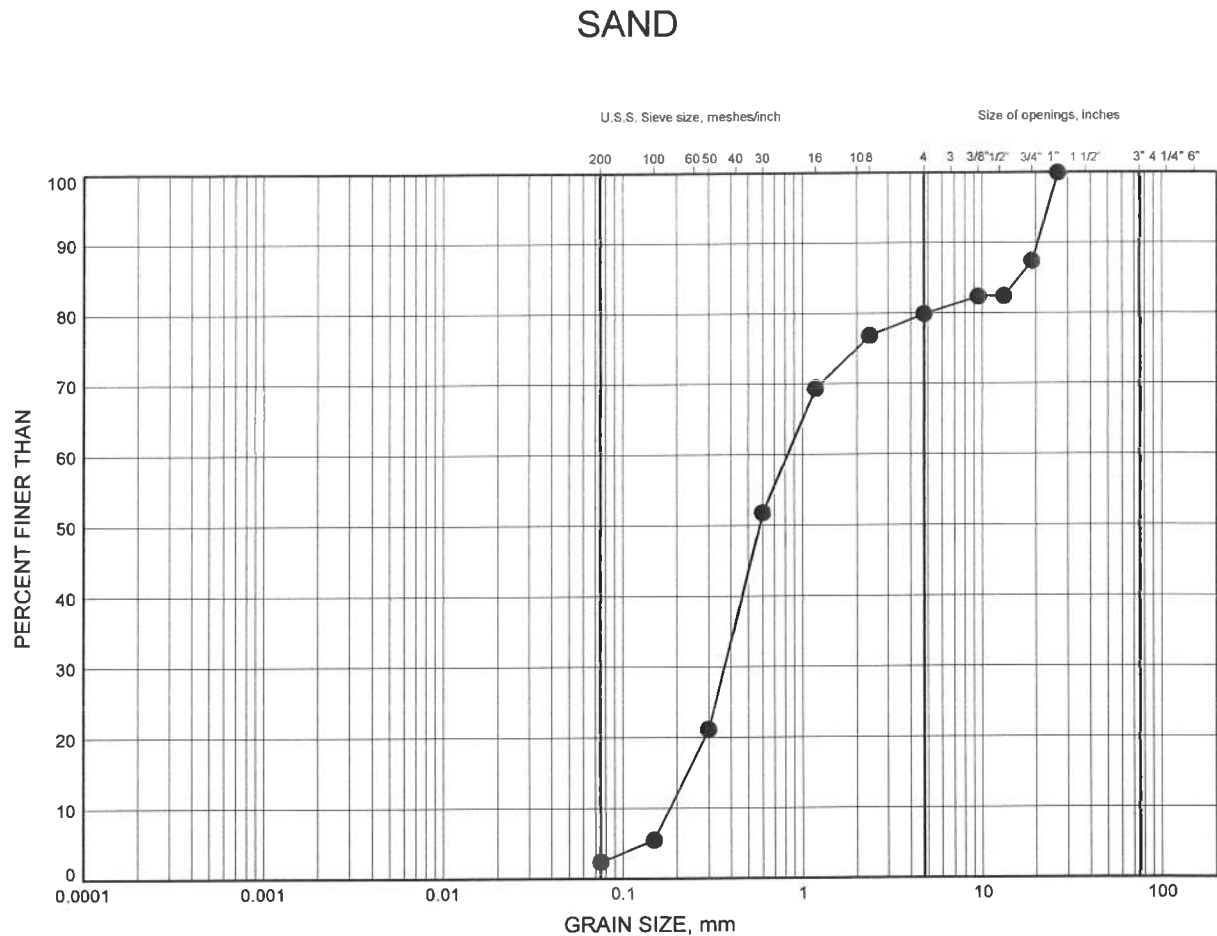


Prep'd AN
Chkd. RPR

NWR 32 Rehabs

GRAIN SIZE DISTRIBUTION

FIGURE B4



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	RPT-09	14.02	190.88

Date March 2013
W.P. 6103-10-00



Prep'd AN
Chkd. RPR



POINT LOAD TEST SHEET

Job No : 19-1351-197 Client : MRC
Date Drilled : 5/24/2012
Project Name : CP Overhead at Rosspoint, Mileage 14.11 Date Tested : 6/1/2012
Core Size : NQ BH No : RPT-08 Tester : SLL

Test No.	Run No.	Depth (m)	Axial or Diametral	Force (kN)	Diameter (mm)	Length (mm)	UCS (MPa)	Rock Type	Notes
1	1	13.3	D	13.2	47.3	80.1	138.1	Granite	Very Strong
2	1	13.6	D	14.2	47.3	78.6	148.1	Granite	Very Strong
3	1	13.9	D	12.7	47.3	84.5	132.8	Granite	Very Strong
4	2	14.2	D	15.4	47.3	79.1	161.6	Granite	Very Strong
5	2	14.5	D	15.5	47.3	91.3	162.5	Granite	Very Strong
6	2	14.9	D	17.1	47.3	79.2	179.3	Granite	Very Strong
7	2	15.2	D	14.1	47.3	96.3	147.5	Granite	Very Strong
8	2	15.4	D	13.5	47.3	81.1	141.5	Granite	Very Strong
9	3	15.8	D	16.4	47.3	93.2	171.5	Granite	Very Strong
10	3	16.1	D	16.0	47.3	79.6	167.3	Granite	Very Strong
11									
12									
13									
14									
15									
16									
17									
18									
19									
20									
21									
22									
23									
24									
25									
26									
27									
28									
29									
30									

* It is ideal to perform axial test on core specimens with D/L ratio of 1.1 ± 0.1

Long pieces of core can be tested diametrically to produce suitable lengths for axial testing

* Diametral Test should have $0.7 \times D$ on either side of test point.



POINT LOAD TEST SHEET

Job No : 19-1351-197 Client : MRC
Date Drilled : 5/26/2012
Project Name : CP Overhead at Rosspoint, Mileage 14.11 Date Tested : 6/12/2012
Core Size : NQ BH No : RPT-02 Tester : SLL

Test No.	Run No.	Depth (m)	Axial or Diametral	Force (kN)	Diameter (mm)	Length (mm)	UCS (MPa)	Rock Type	Notes
1	1	5.8	D	18.0	47.2	96.7	188.2	Granite	Very Strong
2	1	6.1	D	18.5	47.2	86.1	194.1	Granite	Very Strong
3	2	6.4	D	15.3	47.2	76.5	160.2	Granite	Very Strong
4	2	6.7	D	15.2	47.2	69.3	159.4	Granite	Very Strong
5	2	7.0	D	16.2	47.2	71.3	170.2	Granite	Very Strong
6	2	7.3	A	22.7	47.2	56.6	171.7	Granite	Very Strong
7									
8									
9									
10									
11									
12									
13									
14									
15									
16									
17									
18									
19									
20									
21									
22									
23									
24									
25									
26									
27									
28									
29									
30									

* It is ideal to perform axial test on core specimens with D/L ratio of 1.1 ± 0.1

Long pieces of core can be tested diametrically to produce suitable lengths for axial testing

* Diametral Test should have $0.7 \times D$ on either side of test point.



POINT LOAD TEST SHEET

Job No : 19-1351-197 Client : HMM
Date Drilled : 5/30/2012
Project Name : CP Overhead at Rossport, Mileage 14.11 Date Tested : 6/12/2012
Core Size : NQ BH No : RPT-03 Tester : SLL

Test No.	Run No.	Depth (m)	Axial or Diametral	Force (kN)	Diameter (mm)	Length (mm)	UCS (MPa)	Rock Type	Notes
1	1	5.5	D	9.4	47.2	77.7	98.9	Granite	Strong
2	1	6.1	D	18.4	47.2	79.1	193.2	Granite	Very Strong
3	2	6.7	D	16.0	47.2	86.4	167.4	Granite	Very Strong
4	2	7.1	D	11.4	47.2	90.3	119.5	Granite	Very Strong
5	2	7.4	D	15.1	47.2	86.9	157.9	Granite	Very Strong
6	2	7.7	D	18.4	47.2	87.3	192.8	Granite	Very Strong
7	2	8.1	D	24.1	47.2	78.1	253.3	Granite	Extremely Strong
8									
9									
10									
11									
12									
13									
14									
15									
16									
17									
18									
19									
20									
21									
22									
23									
24									
25									
26									
27									
28									
29									
30									

* It is ideal to perform axial test on core specimens with D/L ratio of 1.1 ± 0.1

Long pieces of core can be tested diametrically to produce suitable lengths for axial testing

* Diametral Test should have $0.7 \times D$ on either side of test point.



POINT LOAD TEST SHEET

Job No : 19-1351-197 Client : MRC
Date Drilled : 5/25/2012
Project Name : CP Overhead at Rosspoint, Mileage 14.11 Date Tested : 6/1/2012
Core Size : NQ BH No : RPT-04 Tester : SLL

Test No.	Run No.	Depth (m)	Axial or Diametral	Force (kN)	Diameter (mm)	Length (mm)	UCS (MPa)	Rock Type	Notes
1	2	8.1	D	18.8	47.1	95.4	198.0	Granite	Very Strong
2	2	8.4	D	22.0	47.1	110.4	231.5	Granite	Very Strong
3	2	9.0	D	19.4	47.1	76.4	204.2	Granite	Very Strong
4	2	9.4	D	21.5	47.1	68.2	226.6	Granite	Very Strong
5	3	9.6	D	28.3	47.1	74.9	298.3	Granite	Extremely Strong
6	3	9.9	D	19.8	47.1	88.0	209.3	Granite	Very Strong
7	3	10.3	D	24.0	47.1	79.4	253.2	Granite	Extremely Strong
8	3	10.5	D	21.0	47.1	91.2	221.5	Granite	Very Strong
9	3	10.8	D	23.0	47.1	87.8	242.1	Granite	Very Strong
10									
11									
12									
13									
14									
15									
16									
17									
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26									
27									
28									
29									
30									

* It is ideal to perform axial test on core specimens with D/L ratio of 1.1 ± 0.1

Long pieces of core can be tested diametrically to produce suitable lengths for axial testing

* Diametral Test should have $0.7 \times D$ on either side of test point.



POINT LOAD TEST SHEET

Job No : 19-1351-197 Client : HMM
Date Drilled : 5/30/2012
Project Name : CP Overhead at Rosspoint, Mileage 14.11 Date Tested : 6/12/2012
Core Size : NQ BH No : RPT-05 Tester : SLL

Test No.	Run No.	Depth (m)	Axial or Diametral	Force (kN)	Diameter (mm)	Length (mm)	UCS (MPa)	Rock Type	Notes
1	1	7.7	D	15.2	47.1	86.3	160.0	Granite	Very Strong
2	1	8.0	D	19.0	47.1	79.4	200.5	Granite	Very Strong
3	1	8.3	D	17.1	47.1	81.8	180.4	Granite	Very Strong
4	1	8.6	D	6.2	47.1	93.4	65.2	Granite	Strong
5	2	8.9	D	17.4	47.1	79.1	183.4	Granite	Very Strong
6	2	9.2	D	17.2	47.1	86.9	181.5	Granite	Very Strong
7	2	9.6	D	30.0	47.1	80.8	316.5	Granite	Extremely Strong
8	2	9.9	D	17.5	47.1	93.4	184.3	Granite	Very Strong
9									
10									
11									
12									
13									
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25									
26									
27									
28									
29									
30									

* It is ideal to perform axial test on core specimens with D/L ratio of 1.1 ± 0.1

Long pieces of core can be tested diametrically to produce suitable lengths for axial testing

* Diametral Test should have $0.7 \times D$ on either side of test point.



POINT LOAD TEST SHEET

Job No : 19-1351-197 Client : HMM
Date Drilled : 5/31/2012
Project Name : CP Overhead at Rosspoint, Mileage 14.11 Date Tested : 6/12/2012
Core Size : NQ BH No : RPT-07 Tester : SLL

Test No.	Run No.	Depth (m)	Axial or Diametral	Force (kN)	Diameter (mm)	Length (mm)	UCS (MPa)	Rock Type	Notes
1	1	14.8	D	17.5	47.1	83.9	184.4	Granite	Very Strong
2	1	15.1	D	16.5	47.1	84.8	173.5	Granite	Very Strong
3	1	15.5	D	19.3	47.1	98.1	203.7	Granite	Very Strong
4	1	15.7	D	18.4	47.1	75.6	194.0	Granite	Very Strong
5	2	16.1	D	15.8	47.1	88.0	166.4	Granite	Very Strong
6	2	16.4	D	19.8	47.1	76.2	208.6	Granite	Very Strong
7	2	16.7	D	18.9	47.1	80.0	199.5	Granite	Very Strong
8	2	17.0	D	13.5	47.1	81.1	142.1	Granite	Very Strong
9	2	17.3	D	20.1	47.1	94.6	212.0	Granite	Very Strong
10									
11									
12									
13									
14									
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30									

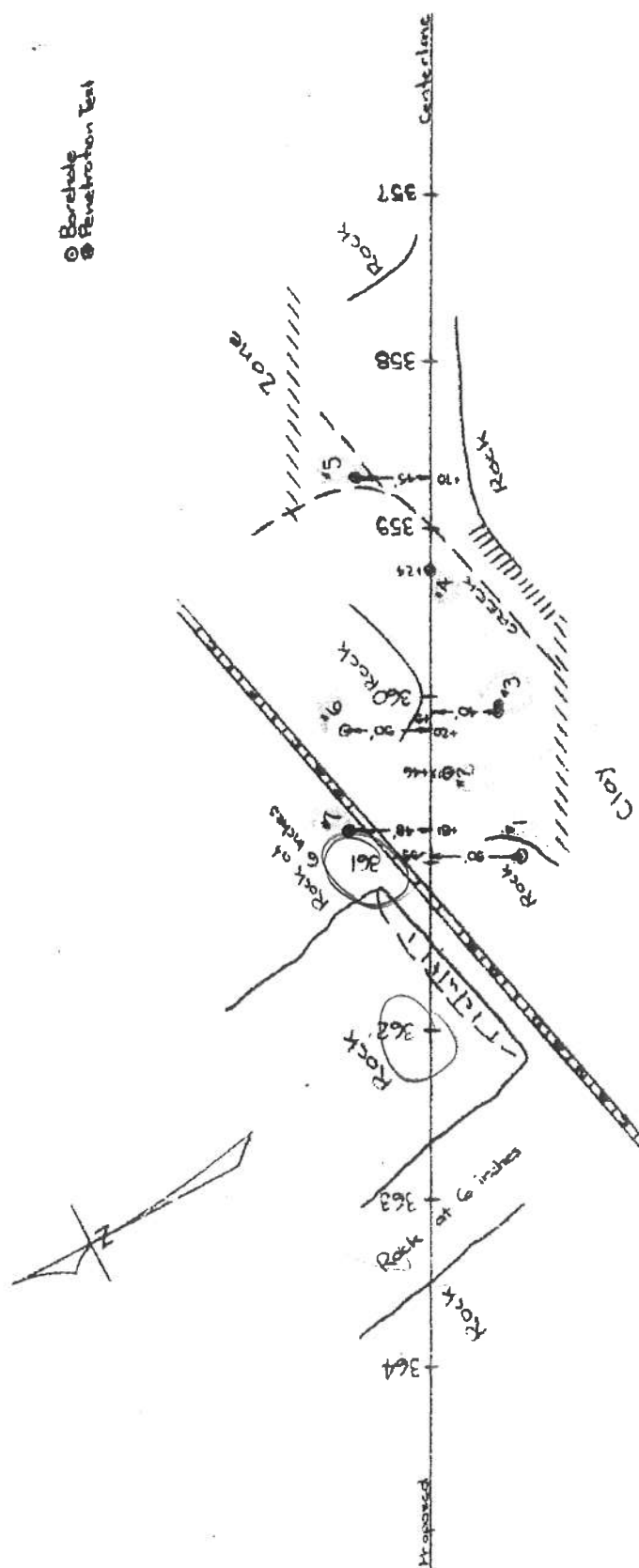
* It is ideal to perform axial test on core specimens with D/L ratio of 1.1 ± 0.1

Long pieces of core can be tested diametrically to produce suitable lengths for axial testing

* Diametral Test should have $0.7 \times D$ on either side of test point.

Appendix C

Record of Borehole Sheets and Laboratory Results (previous investigation)



TROW SODERMAN & ASSOCIATES

PROJECT NO. 6129 J213

TROW SODERMAN AND ASSOCIATES

SITE INVESTIGATIONS AND SOIL MECH. NICS CONSULTATION

PROJECT **Rossport Overpass**
 LOCATION **Rossport, Ont.**
 HOLE LOCATION **See Dwg. #1**

HOLE ELEVATION AND DATUM **638.7**
B/R C.P.R. @ Sta. 361+18 = 648.9

BOREHOLE NO. **1**
 FIELD SUPERVISOR **D8**
 DRILLER **AA**
 PREP. **D8**

DRAWING NO. **2**

LEGEND

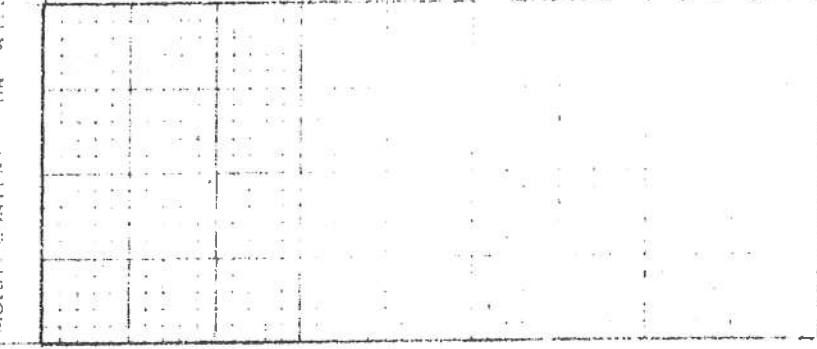
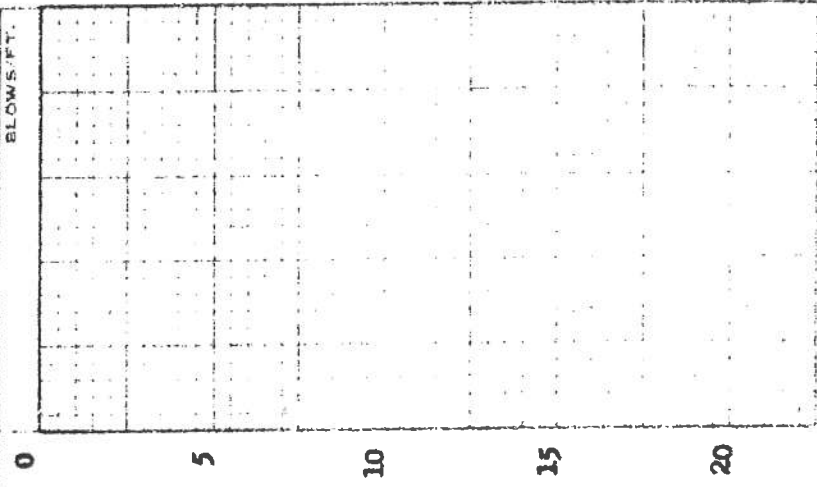
- 2" DIA. SPLIT TUBE
- 2" SHELBY TUBE
- 2" SPLIT TUBE
- 2" DIA. CONE
- CASING
- 2" SHELBY
- 1/2 UNCONFINED COMPRESSION (QU)
- VANE TEST (C) AND SENSITIVITY (S)
- NATURAL MOISTURE AND LIQUIDITY INDEX
- LIQUID LIMIT
- PLASTIC LIMIT

CONSISTENCY	NATURAL SAMPLE UNIT WT
MOIST. CONTENT - 3 GR. WT.	P.C.F.

SYMBOL	DESCRIPTION	ELEV. FEET	DEPTH FEET	STRENGTH AND PENETRATION RESISTANCE P.S.F.	BLOWS/FT.
--------	-------------	------------	------------	--	-----------

Topsoil
 Bedrock - granite

End of hole 634.2



0129/J213

TROW SODERMAN AND ASSOCIATES

SITE INVESTIGATIONS AND SOIL MECHANICS CONSULTATION

PRODUCT **Rossport Overpass,**
LOCATION **Rossport, Ont.**
SOLE LOCATION **See Dwg. #1** 634-3

W/2 C.P.R. @ Sta. 361 + 18 = 648.9

BOREHOLE NO. 2

FIELD SUPERVISOR

08:17:50

पञ्चमः

或

3

22

DEPTH	STRENGTH AND PENETRATION RESISTANCE	P.S.F.
FEET		1000
		500
		BLOWS/FT

Torresill

SAND - clean brown beddins

Clay - grey, soft with sand sizes present.

BEDROCK - granite

End of hole

DRAWING NO.:

3

02541

- 2 1 DIA. SPLIT TUBE
2 1 SHELLEY TUBE
1 1 SPLIT TUBE
2 1 DIA. CONE

- CASING
1. SHELBY

12 UNCONF(NEO COMPRESSION [QU]

NAME TEST (C) AND SENSITIVITY (S)

XEROX ALIQUOT
THERMAL MODULINE A-6

LIQUID LIMIT

PLASTIC LIMIT

MOIST	CONTENT	W. OR	WT	CONSISTENCY	SAMPLE UNIT	NATURAL	WT

55

CL29/J213

TROW SODERMAN AND ASSOCIATES

Rossport Overpass
 Rossport, Ont.
 See DWG. #1
 633.3
 B/R CPR @ Sta. 361+18 = 648.9

3
 DS
 AA
 DS

Topsoil 633.3
 632.3

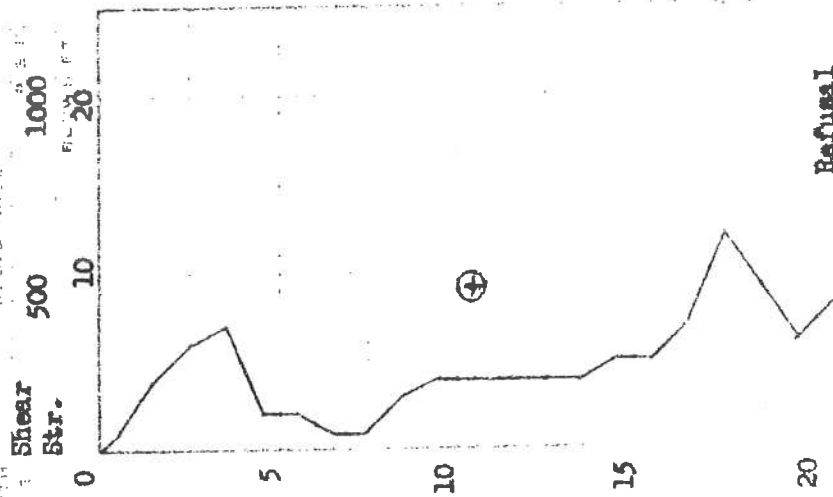
SAND - brown, medium

CLAY - grey, stiff at first,
 then contains sand aizes

Coarse sand and gravel
 present

End of hole

614.3



Refusal

16' clay

DRAWING NO. 4

- 1. 4" DIA. SPLIT TUBE
- 2. SPLIT TUBE
- 3. SPLIT TUBE
- 4. SPLIT TUBE
- 5. SPLIT TUBE
- 6. SPLIT TUBE
- 7. SPLIT TUBE
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- 15. SPLIT TUBE
- 16. SPLIT TUBE
- 17. SPLIT TUBE
- 18. SPLIT TUBE
- 19. SPLIT TUBE
- 20. SPLIT TUBE

- TW1 No recovery
- TW2 "
- TW3 "
- TW4 "
- TW5 "
- TW6 Damaged
- TW7 No recovery

PROJECT NO. 0129/3213

TROW SODERMAN AND ASSOCIATES

PITC INVESTIGATIONS AND SOIL MECHANICS CONSULTATION

PROJECT
LOCATION
SITE LOCATION
ELEVATION AND DATUM

See Dwg. #1

630.0
B/A CPR Sta. 361+18 = 648.9

BOREHOLE NO. 4
FIELD SUPERVISOR DS
DRILLER AA
PREP. DS

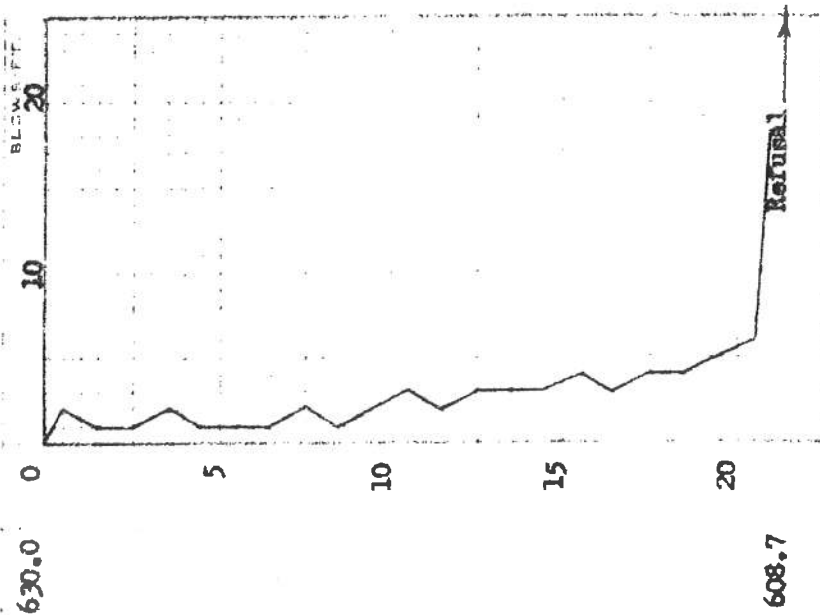
LEGEND

2" DIA. SPLIT TUBE
2" SHELBY TUBE
2" SPLIT TUBE
2" DIA. CONE
CASING
2" SHELBY
1-2 UNCONFINED COMPRESSION (QU)
VANE TEST (C) AND SENSITIVITY (S)
NATURAL MOISTURE AND
LIQUIDITY INDEX
LIQUID LIMIT
PLASTIC LIMIT

9
5
X
-0

STRENGTH AND PENETRATION
RESISTANCE
P.S.F.
630.0 0 10 20
608.7

Penetration test



21.3' day

DRAWING NO. 5

CONSISTENCY
MOIST. CONTENT - VERN. WT. %
NATURAL
SAMPLE UNIT WT.
P.L.F.



CL29/J213

PROJECT NO.

TROW SODERMAN AND ASSOCIATES

SITE INVESTIGATIONS AND SOIL MECHANICS CONSULTATION

PROJECT **Rosport Overpass**
 LOCATION **Rosport, Ont.**
 HOLE LOCATION **See DWG.1**
 HOLE ELEVATION AND DATUM **627.6**
B/A CPR @ Sta. 361+18 = 648.9

BOREHOLE NO. **5**
 FIELD SUPERVISOR **DS**
 DRILLER **AA**
 PREP. **DS**

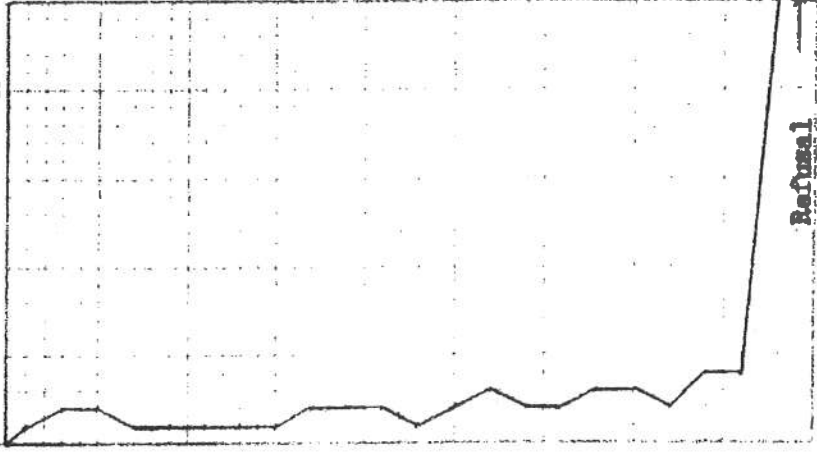
LEGEND
 2" DIA. SPLIT TUBE
 2" SHELBY TUBE
 2" SPLIT TUBE
 2" DIA. CONE
 CASING
 2" SHELBY
 1/2 UNCONFINED COMPRESSION (QU)
 PLANE TEST (C) AND SENSITIVITY (S)
 NATURAL MOISTURE AND
 LIQUIDITY INDEX
 LIQUID LIMIT
 PLASTIC LIMIT

DRAWING NO. **6**

STRENGTH AND PENETRATION
 RESISTANCE
 P.S.F.
 ELEV. FEET
 DEPTH FEET

627.6 0 10 20 30 40 50 60 70 80 90 100 110 120 130 140 150 160 170 180 190 200 210 220 230 240 250 260 270 280 290 300 310 320 330 340 350 360 370 380 390 400 410 420 430 440 450 460 470 480 490 500 510 520 530 540 550 560 570 580 590 600 610 620 630 640 650 660 670 680 690 700 710 720 730 740 750 760 770 780 790 800 810 820 830 840 850 860 870 880 890 900 910 920 930 940 950 960 970 980 990 1000

Penetration test



605.6

Refusal

20' clay

PROJECT NO.

TROW SODERMAN AND ASSOCIATES

SITE INVESTIGATIONS AND SOIL MECHANICS CONSULTATION

Report Overpass

LOCATION	REPORT, Ont.
1. 1000	1000
2. 1000	1000
3. 1000	1000
4. 1000	1000
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6. 1000	1000
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91. 1000	1000
92. 1000	1000
93. 1000	1000
94. 1000	1000
95. 1000	1000
96. 1000	1000
97. 1000	1000
98. 1000	1000
99. 1000	1000
100. 1000	1000

HOLE LOCATION

WALLS AND NOT REATCH
647.8

$$B/A \text{ CPR @ Sta. } 361+18 = 648.9$$

ВОЯЧЕНЕНОЕ НО:
6

FIELD SUPERVISOR

DRILLER

PREP.

8

3

TOBACCO REGISTRATION

FILL: sand gravel boulders

BEDROCK - granite

End of bol.	634.8
-------------	-------

[illegible]

CONSISTENCY	SAMPLE UNIT WT.	NATURAL
MOIST. CONTENT - % DRY WT.	P. C. F.	
		SSL No record

STAY NO RECOVERY

Appendix D

Foundation Comparison

COMPARISON OF FOUNDATION ALTERNATIVES FOR EACH FOUNDATION ELEMENT

Foundation Unit	Footings on Native Soil	Footings on Bedrock	Footings on Engineered Fill	Augered Caissons (drilled shafts) socketed into bedrock	Drilled in Pipe Piles socketed into bedrock	Driven Piles to Bedrock
	<p>Advantages:</p> <ul style="list-style-type: none"> i. Generally less costly construction than deep foundation elements. <p>Disadvantages:</p> <ul style="list-style-type: none"> i. Low available geotechnical resistance in native soils. ii. Potential for settlements. iii. Relatively deep excavation is required at the east abutment. This will necessitate excavation shoring. 	<p>Advantages:</p> <ul style="list-style-type: none"> i. High values of geotechnical resistance are available on the bedrock. <p>Disadvantages:</p> <ul style="list-style-type: none"> i. Deep excavation in cohesionless soils extending below the groundwater level is required. This will necessitate prior dewatering and shoring. ii. High cost of excavation. iii. Mass concrete fill required to create a level founding surface. iv. Possible sloping bedrock at the west abutment. 	<p>Advantages:</p> <ul style="list-style-type: none"> i. Higher geotechnical resistance than is available on native soil. ii. Lower cost compared to deep foundations. <p>Disadvantages:</p> <ul style="list-style-type: none"> i. Lower geotechnical resistance than piles. ii. Deep excavation and shoring required to construct engineered fill at the east abutment. iii. High cost of constructing engineered fill. iv. Potential settlements. 	<p>Advantages:</p> <ul style="list-style-type: none"> i. High geotechnical resistance available for caissons socketed into bedrock ii. Construction of caissons could continue in freezing weather. iii. Subexcavation of fill not required. <p>Disadvantages:</p> <ul style="list-style-type: none"> i. Higher unit cost compared to spread footings. ii. Specialized installation measures such as temporary liners and drilling mud will be required to install caissons under the water table. iii. Potential difficulty in cleaning and inspecting rock sockets. iv. Installation through cobbles and boulders will be difficult. v. Potential sloping bedrock surface. 	<p>Advantages:</p> <ul style="list-style-type: none"> i. High geotechnical resistance available for units socketed into bedrock. ii. Construction could continue in freezing weather. iii. Subexcavation of fill not required. iv. Cleaning and inspection of socket base not required <p>Disadvantages:</p> <ul style="list-style-type: none"> i. Possibly higher unit cost compared to other foundation options such as footings. ii. Difficulties in obtaining seal below the liner to pour concrete in dry conditions. iii. Specialized installation. 	<p>Advantages:</p> <ul style="list-style-type: none"> i. High geotechnical resistance available by driving piles to achieve resistance on the bedrock at the east abutment. ii. Installation of piles could continue in freezing weather. iii. Independent of groundwater conditions. iv. Foundation construction requires less volume of excavation than footings v. Readily installed. <p>Disadvantages:</p> <ul style="list-style-type: none"> i. Higher unit costs than footings. ii. Pile lengths required to achieve design resistance may vary. iii. Pre-augering might be required if cobbles and boulders are encountered within the fill. iv. Piles would need to be socketed into bedrock at the west abutment.
West Abutment	NOT APPLICABLE	RECOMMENDED	NOT RECOMMENDED	FEASIBLE	FEASIBLE	NOT RECOMMENDED
East Abutment	NOT RECOMMENDED	NOT RECOMMENDED	FEASIBLE	FEASIBLE	FEASIBLE	RECOMMENDED

Appendix E
Slope Stability Output

Title: HWY 17 - CPR Overhead Bridge at Rossport

Name: East Abutment

Description: RSS Wall

Last Solved Date: 3/21/2013, 2:18:07 PM

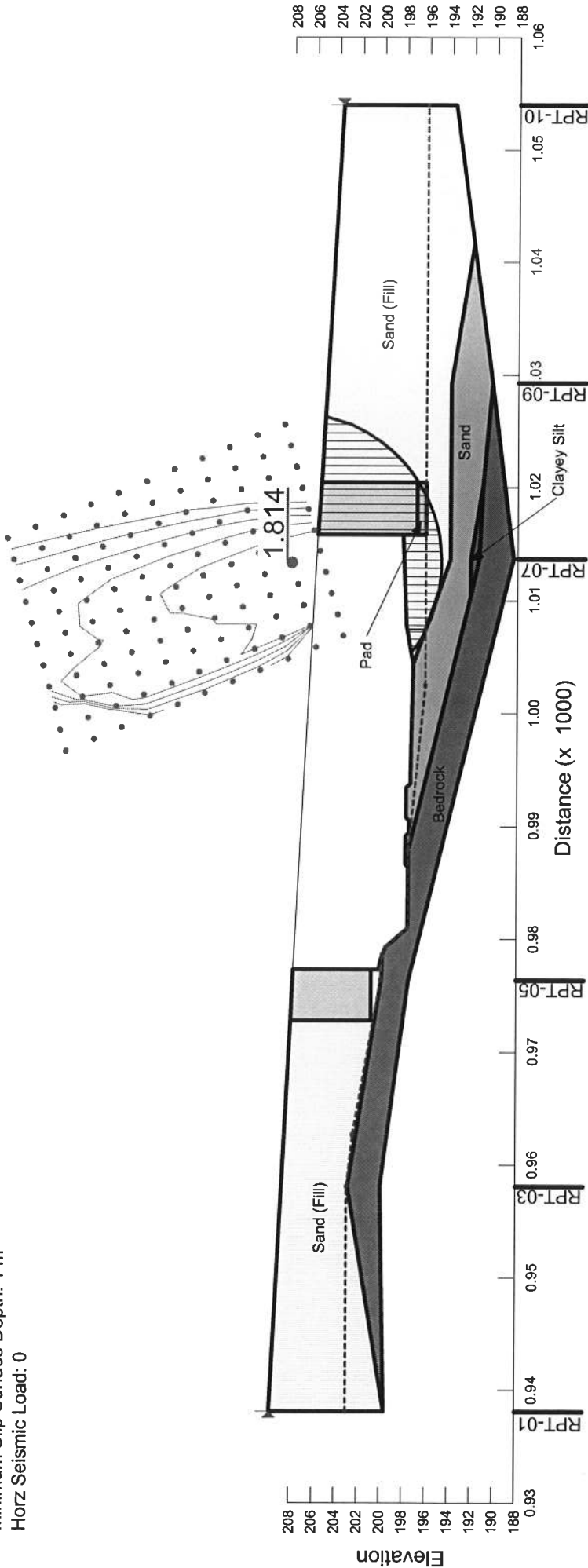
Method: Morgenstern-Price

Interslice force function option: Half-Sine

Minimum Slip Surface Depth: 1 m

Horz Seismic Load: 0

Name: RSS Wall	Unit Weight: 20 kN/m ³	Cohesion: 200 kPa	Phi: 45 °	Piezometric Line: 1
Name: SAND (Fill)	Unit Weight: 20 kN/m ³	Cohesion: 0 kPa	Phi: 32 °	Piezometric Line: 1
Name: SAND	Unit Weight: 20 kN/m ³	Cohesion: 0 kPa	Phi: 32 °	Piezometric Line: 1
Name: Clayey SILT	Unit Weight: 18 kN/m ³	Cohesion: 0 kPa	Phi: 26 °	Piezometric Line: 1
Name: GRANITE (Bedrock)				
Name: pad	Unit Weight: 21 kN/m ³	Cohesion: 0 kPa	Phi: 35 °	Piezometric Line: 1



Appendix F

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

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

- OPSS 903
- OPSS 902
- OPSS 804
- OPSD 208.010
- OPSD 3101.150
- OPSS 539
- Special Provision 110S13 “Amendment to OPSS 1010, April 2004”.

2. Suggested Text for NSSP on “Construction of Caissons”

Caisson installation shall be in accordance with OPSS 903 and the following.

Caisson installation at this site will require excavation through cohesionless soils below the groundwater table and construction of sockets in the underlying bedrock. The Contractor is advised of the following:

- The cohesionless soil above the bedrock is susceptible to disturbance under conditions of unbalanced hydrostatic head, and measures must be employed to maintain sidewall stability in the caisson excavation and prevent collapse/washing of cohesionless soils into the rock socket. Selection of the methods and equipment employed to achieve this is the responsibility of the Contractor.
- Caisson installation may encounter cobbles, boulders and/or large rock fragments in the soils overlying the bedrock. The installation methods and equipment must be capable of dislodging, removing or otherwise penetrating such obstructions.
- The bedrock consists of strong to extremely strong granite rock. The strength and hardness of this rock must be taken into account when selecting equipment to advance the caisson into rock. Equipment supplied to construct the rock socket must be capable of excavating the bedrock to the specified socket dimensions without disturbing or fracturing the bedrock forming the sidewalls and base of the socket. Blasting to facilitate the removal of bedrock is not permitted.
- The rock socket must be formed entirely within the bedrock below the level of any cobbles and boulders. Any length of caisson above the bedrock surface will not be considered part of the specified length of rock socket.

3. Suggested Text for NSSP on “Construction of Drilled-in pipe piles”

Drilled-in pipe pile installation shall be in accordance with OPSS 903 and the following.

Drilled-in pipe pile installation at this site will require excavation through cohesionless soils below the groundwater table and construction of sockets in the underlying bedrock. The Contractor is advised of the following:

- The cohesionless soil above the bedrock is susceptible to disturbance under conditions of unbalanced hydrostatic head, and measures must be employed to maintain sidewall stability during installation of the pipe piles and prevent collapse/washing of cohesionless soils into the rock socket. Selection of the methods and equipment employed to achieve this is the responsibility of the Contractor.
- Drilled-in pipe pile installation may encounter cobbles, boulders and/or large rock fragments in the soils overlying the bedrock. The installation methods and equipment must be capable of dislodging, removing or otherwise penetrating such obstructions.
- The bedrock consists of strong to very strong granite rock. The strength and hardness of this rock must be taken into account when selecting equipment to advance the drilled-in pipe pile into rock. Equipment supplied to construct or drill the rock socket must be capable of excavating the bedrock to the specified socket dimensions without disturbing or fracturing the bedrock forming the sidewalls and base of the socket. Blasting to facilitate the removal of bedrock is not permitted.
- The rock socket must be formed entirely within the bedrock below the level of any cobbles and boulders. Any length of drilled-in pipe pile above the bedrock surface will not be considered part of the specified length of rock socket.
- Pipe piles shall be placed centred into the holes, bearing directly on the sound rock at the bottom of the hole. Pipe piles shall be stabilized in place by temporary supports.
- The annular space between the rock socket wall and the pipe pile shall be filled with 30 MPa concrete to the top of the bedrock surface at the location. Concrete may be tremied into the pipe pile. The plumbness and alignment of the pipe pile shall be maintained during concreting

Appendix G

Site Photographs



Photograph 1– Highway 17 and CP Overhead at Rosspoint crossing, south side



Photograph 2– Highway 17 and CP Overhead at Rossport crossing, north side



Photograph 3 Highway 17 and CP Overhead at Rossport crossing, northwest side



Photograph 4– Highway 17 and CP Overhead at Rossport crossing, north side



Photograph 5– Highway 17 and CP Overhead at Rossport crossing, southeast side



Photographs 6 and 7—CP Overhead at Rossport existing embankments

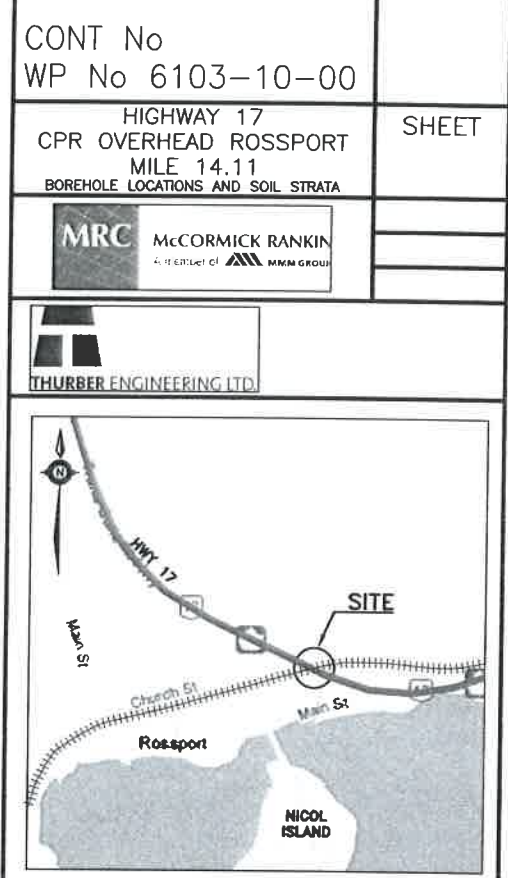
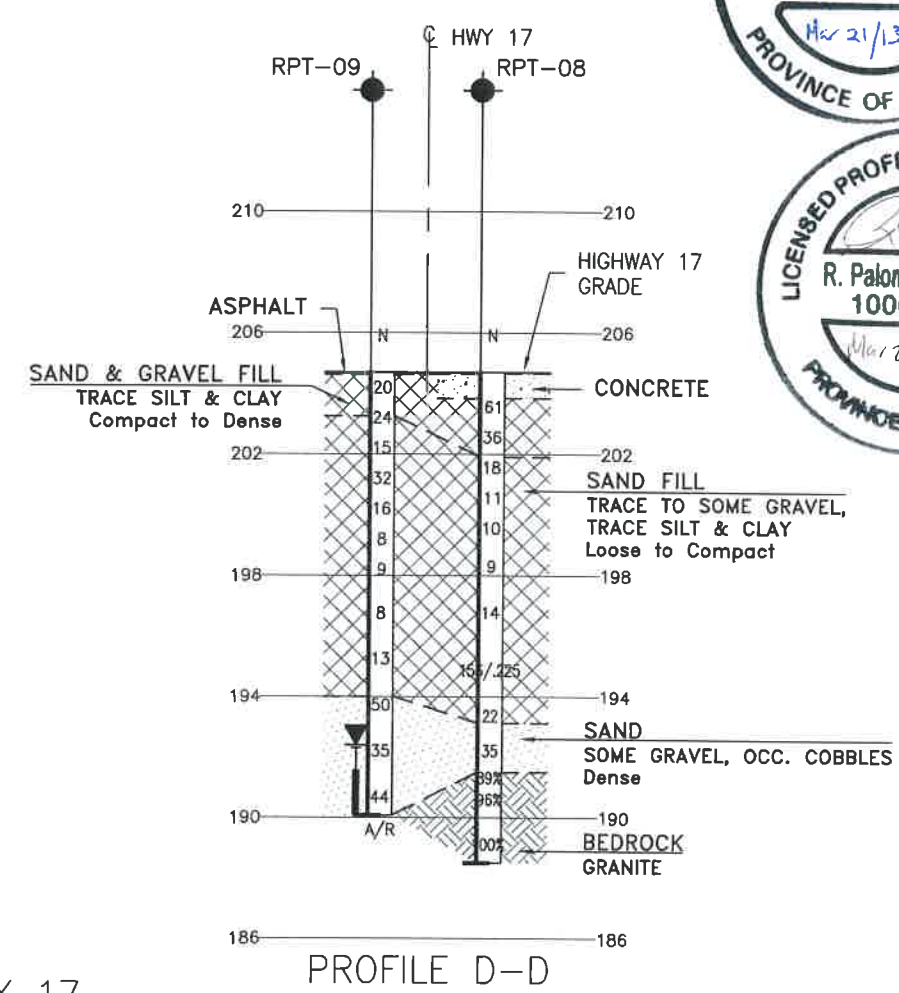
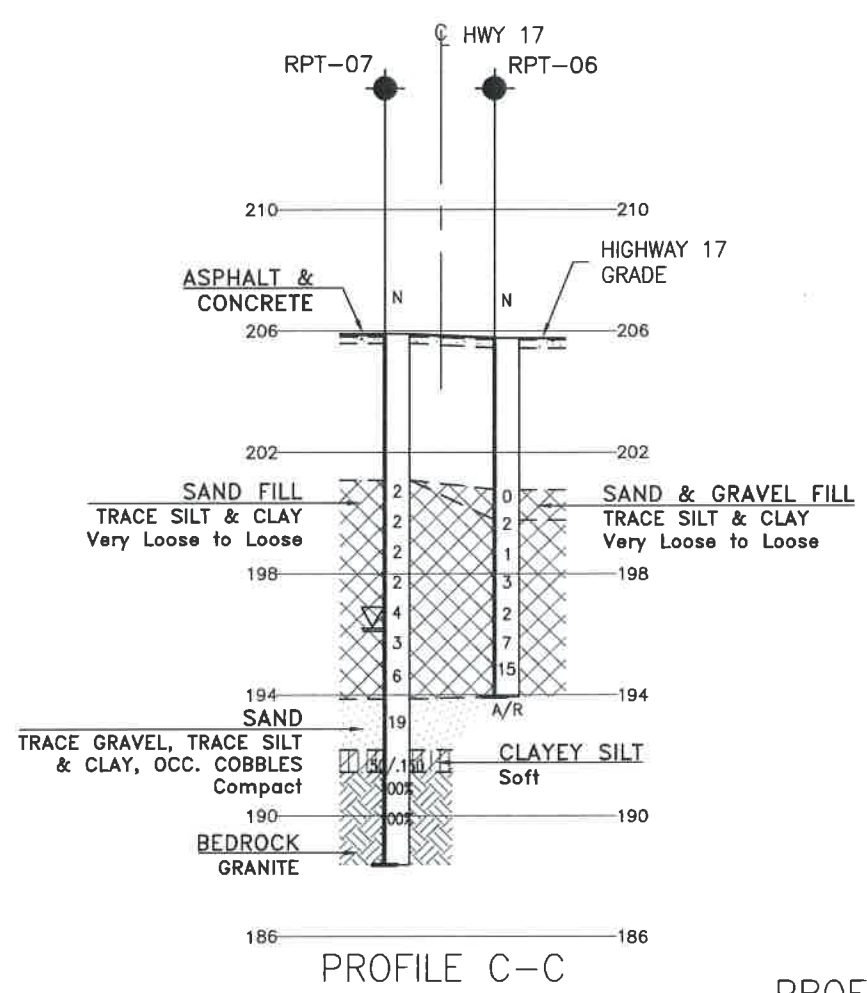
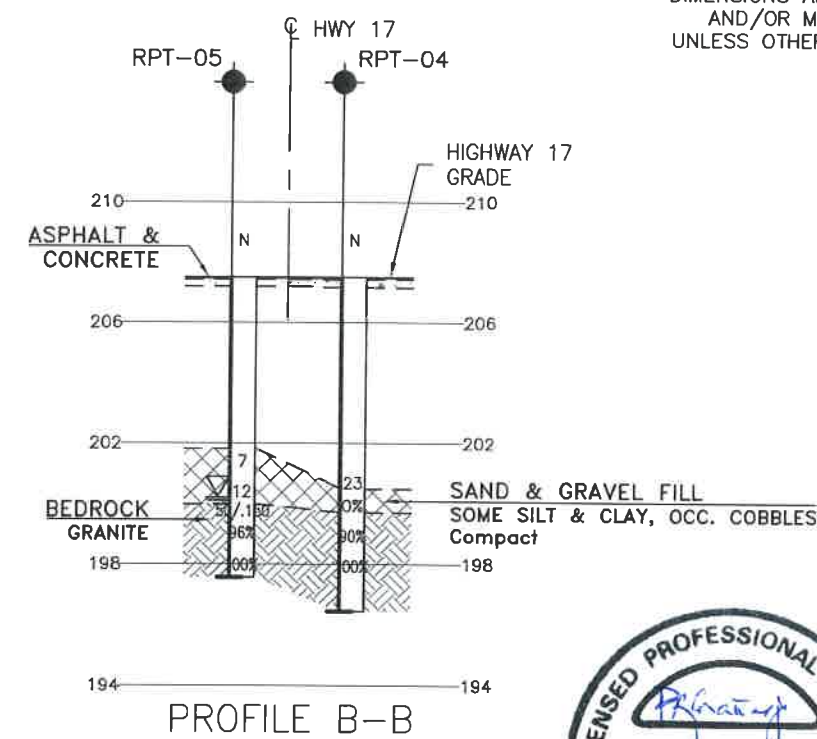
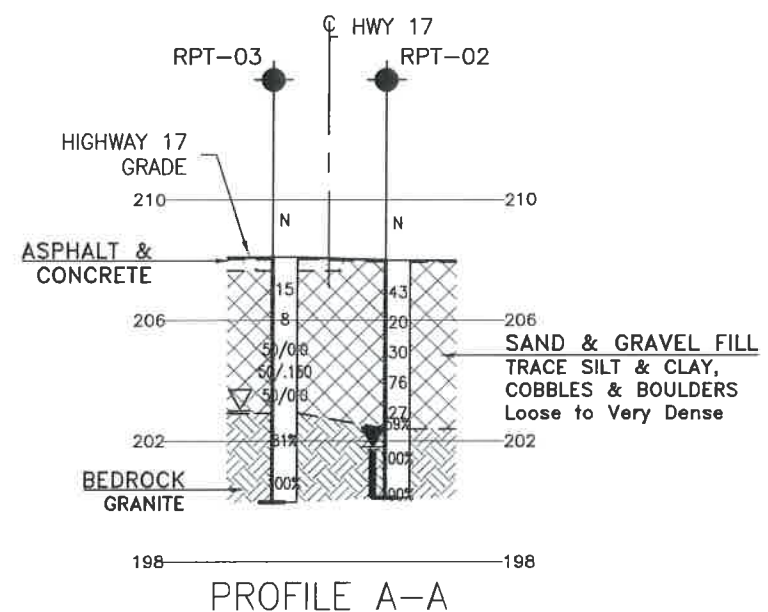
CPR Overhead at Rosspoint Replacement
Highway 17, Site 48C-24








Photographs 8 and 9—CP Overhead at Rosspoint existing embankments

Appendix H

Drawing titled “Borehole Locations and Soil Strata”



L E G E N D	
	Borehole (Current Investigation)
	Test Pit (Current Investigation)
	Borehole (Previous Investigation)
N	Blows /0.3m (Std Pen Test, 475J/blow)
CONE	Blows /0.3m (60° Cone, 475J/blow)
	Water Level During Drilling
	Water Level in Piezometer
90%	Rock Quality Designation (RQD)
A/R	Auger Refusal

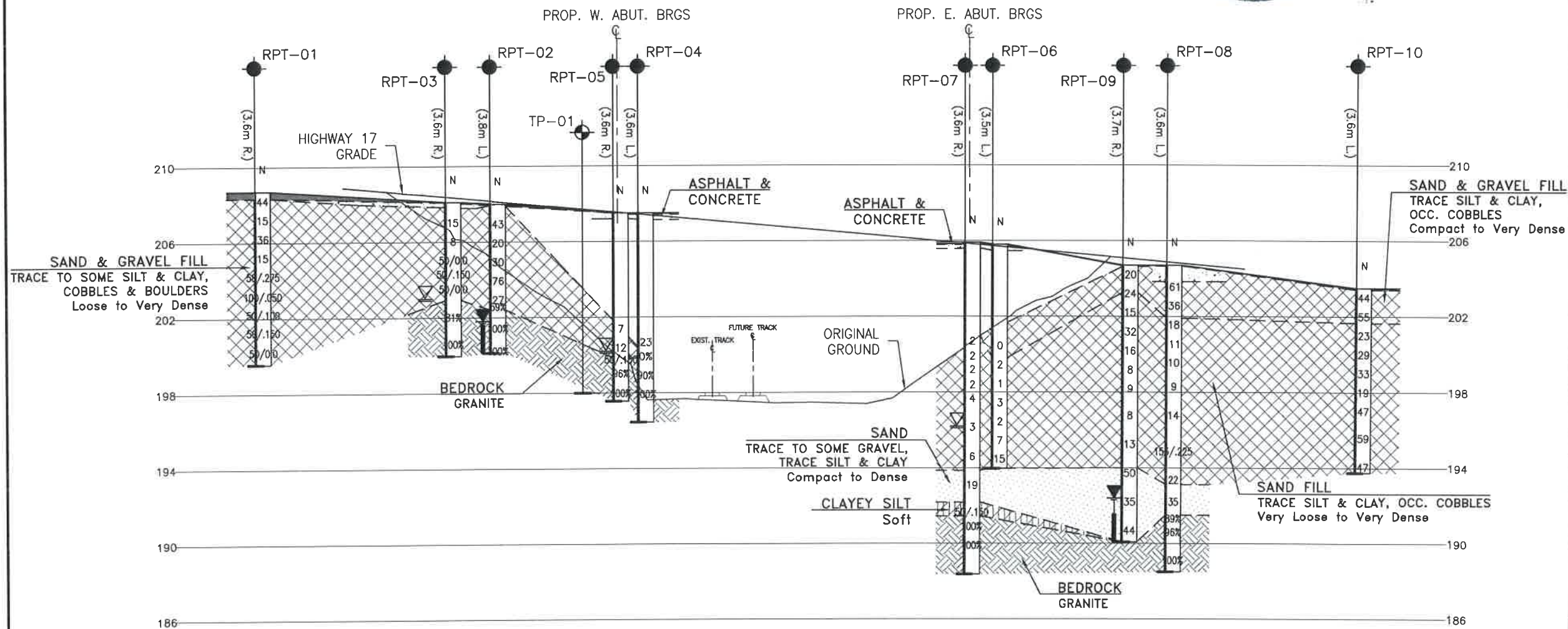
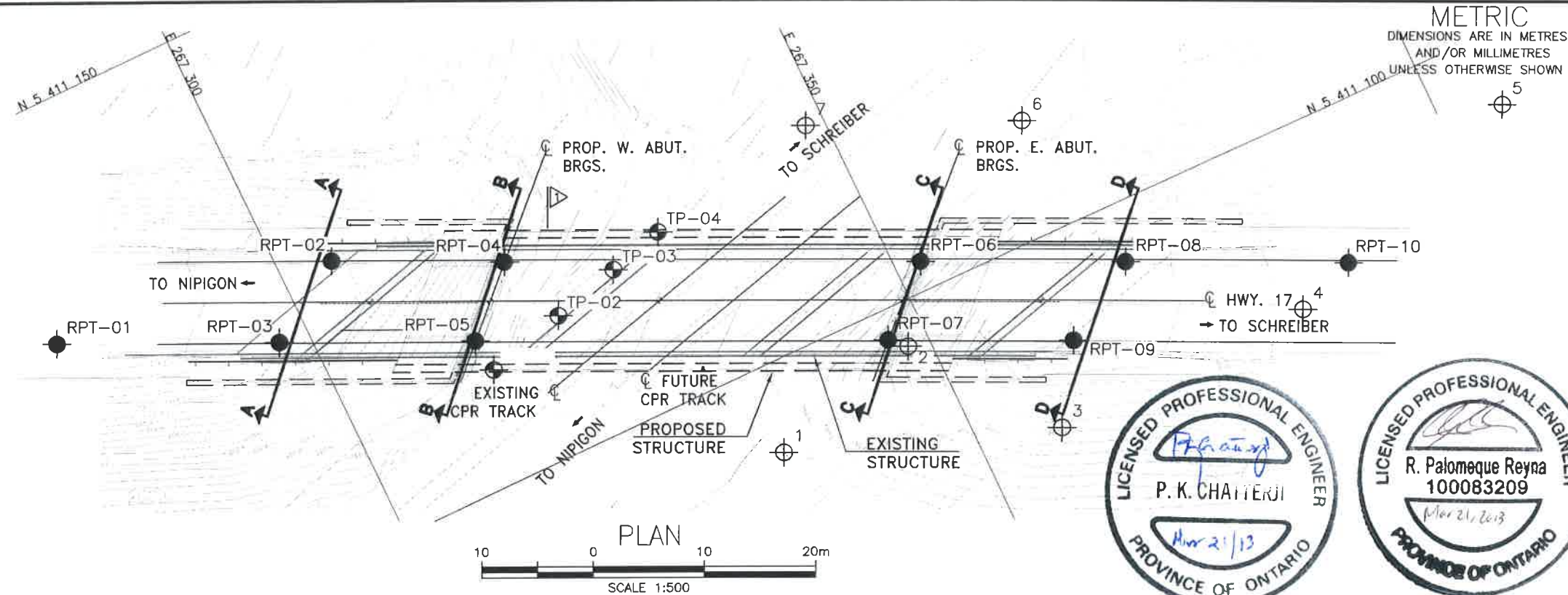
NO	ELEVATION	NORTHING	EASTING
RPT-01	208.7	5 411 129.3	267 279.2
RPT-02	208.0	5 411 125.4	267 304.7
RPT-03	208.1	5 411 120.8	267 297.3
RPT-04	207.5	5 411 118.6	267 318.7
RPT-05	207.5	5 411 113.2	267 313.2
RPT-06	205.8	5 411 102.6	267 352.4
RPT-07	205.9	5 411 097.4	267 346.7
RPT-08	204.7	5 411 094.8	267 369.0
RPT-09	204.9	5 411 090.2	267 361.7
RPT-10	203.5	5 411 086.2	267 387.0
TP-01	198.0	5 411 110.1	267 313.6
TP-02	198.0	5 411 112.1	267 320.9
TP-03	198.0	5 411 113.8	267 327.2
TP-04	198.0	5 411 115.2	267 332.3

-NOTES-

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

GEOCRES No. 42D-28

REVISIONS								
	DATE	BY	DESCRIPTION					
DESIGN	RPR	CHK	RPR	CODE	LOAD		DATE	MAR. 2013
DRAWN	AN	CHK		SITE	STRUCT	DWG	2	

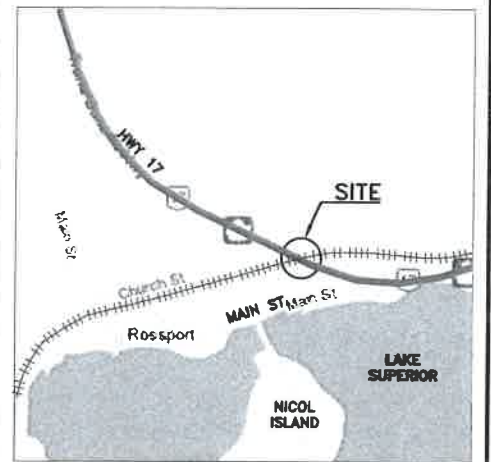


CONT No
WP No 6103-10-00

HIGHWAY 17
CPR OVERHEAD ROSSPORT
MILE 14.11
BOREHOLE LOCATIONS AND SOIL STRATA

MRC
McCORMICK RANKIN
A MEMBER OF THE
PETER GROUP

THURBER ENGINEERING LTD.



KEYPLAN

LEGEND

- Borehole (Current Investigation)
- ⊕ Test Pit (Current Investigation)
- ⊕ Borehole (Previous Investigation)
- N Blows /0.3m (Std Pen Test, 475J/blow)
- CONE Blows /0.3m (60° Cone, 475J/blow)
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NO	ELEVATION	NORTHING	EASTING
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GEOCREs No. 42D-28

REVISIONS	DATE	BY	DESCRIPTION
DESIGN	RPR	CHK RPR	CODE
DRAWN	AN	CHK	SITE
			STRUCT
			DWG 1