



THURBER ENGINEERING LTD.

**FOUNDATION INVESTIGATION AND DESIGN REPORT
REPLACEMENT OF CPR/O-TRAIN BRIDGES
HIGHWAY 417, OTTAWA
SITE No. 3-54**

GWP 4245-05-00

5016-E-0007

Geocres No.: 31G5-293

Report to:

WSP

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PART 1. FACTUAL INFORMATION

1 INTRODUCTION

This section of the report presents the factual findings obtained from a foundation investigation completed for the proposed replacement of the Canadian Pacific Rail (CPR) / O-Train Overpasses (Site Nos. 3-54.1 and 3-54.2) located on Highway 417 in the City of Ottawa, Ontario. Thurber Engineering Limited (Thurber) carried out the current investigation as a sub-consultant to WSP under Agreement No. 4014-E-0042.

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, laboratory test results and a written description of the subsurface conditions. A model of the subsurface conditions influencing design and construction was developed in the course of the current investigation.

2 SITE DESCRIPTION

The existing Highway 417 overhead structures at the CPR/O-Train tracks consists of two side-by-side structures supporting the eastbound lanes (EBL) and westbound lanes (WBL). The EBL structure is identified as Site #3-54.1 and the WBL structure is identified as Site #3-54.2.

Both structures were reportedly built between 1961 and 1962 and consist of five span concrete T-beam bridges with integral piers and conventional abutments. The abutments and piers are founded on spread footings bearing on bedrock. Both bridges are reported to have a width of 21.5 m and a length of 85.2 m (individual span lengths are 18.2 m, 18.2 m, 18.3 m, 18.3 m and 12.2 m). Concrete cantilever retaining walls are present adjacent to the structure. Beneath the bridges, the railway is located within a rock cut section with concrete crib walls in front of the middle piers. Rock faces are visible within the cut section. The top of rail elevation is approximately 55.3 m beneath the bridges. The top of the rock cut beside the rail tracks is typically in the range of elevation 59 to 60 m.

At the location of the overpasses (Linear Highway Referencing System Base Point: 49450, Offset: 0.26), Highway 417 is an urban freeway with four through lanes of traffic in each direction, plus one speed change lane in each direction. Traffic volumes on Highway 417 are understood to be 184,100 AADT (2016).

The land adjacent to the highway is generally developed with both industrial and residential properties. The slopes above the rock cut beside the railway are densely covered with

shrubs and trees. A pedestrian walking and biking pathway (Trillium Pathway) is located along the east side of the railway and crosses beneath the Highway 417 structures. A pedestrian bridge over the railway tracks is located approximately 40 m south of the Highway 417 structures and connects the Trillium Pathway to the City streets on the west side.

Select photographs showing the existing conditions in the area of the bridges are included in Appendix D for reference.

3 INVESTIGATION PROCEDURES

A previous foundation investigation report for this site was available from the Ministry's Geocres library. The reference for this report is as follows:

Foundation Investigation Report for C.P.R. Overpass – Bridge No. 14,
Queensway, Geocres 31G05-033, Site 59-F-220C, Ottawa, Ont., dated 1959.

The report was reviewed during the planning stages of the current investigation, however, it was noted that the investigation was carried out prior to the construction of the railway rock cut and the Highway 417 bridges and embankment. In addition, the exact locations of the boreholes in this document were not known. Therefore, the boreholes from the previous investigation were not incorporated into the current report.

It is noted that separate investigations were completed for a proposed retaining wall in the northwest quadrant of the site and for environmental purposes. This information is provided under separate cover.

3.1 Site Investigation and Field Testing

The site investigation and field testing program was carried out in two phases:

The first phase of the field investigation was carried out between March 28th and April 12th, 2017 and consisted of advancing nine boreholes identified as 16-01 through 16-03, 16-05 through 16-09 and 16-11.

The second phase of the field investigation was carried out between February 20th and March 7th, 2018, and consisted of advancing seven boreholes identified as 16-04, 16-10 and 18-08 through 18-12.

The northing, easting and elevation of the boreholes are shown on the Borehole Location and Soil Strata Drawing No. 1 in Appendix A and are summarized in Table 3-1. The site is within MTM Zone 9. The elevations were surveyed relative to site benchmarks HCP 105 and HCP 106 which have geodetic elevations of 62.659 m and 62.605 m, respectively. Northing and easting coordinates were derived from CAD files provided by WSP using measurements of offsets from site features.

Table 3-1: Borehole Summary

Borehole No.	Drilled Location	Northing (m)	Easting (m)	Ground Surface Elevation (m)	Borehole Termination Depth Below Existing Ground Surface (m)
16-01	East Abutment	5029507.6	366437.9	62.8	7.2
16-02	East Abutment	5029463.7	366465.5	62.5	13.7
16-03	Retaining Wall	5029487.8	366363.4	66.5	9.2
16-04	East Slide Path	5029516.9	366432.6	63.4	5.9
16-05	West Abutment	5029458.8	366420.7	63.8	6.2
16-06	Retaining Wall	5029453.8	366480.2	62.5	7.2
16-07	West Approach	5029458.7	366381.5	72.0	10.5
16-08	East Approach	5029491.6	366478.1	68.9	8.2
16-09	West Slide Path	5029527.1	366379.9	63.5	7.5
16-10	East Slide Path	5029544.4	366413.9	63.7	7.2
16-11	West Abutment	5029497.5	366400.8	63.8	12.3
18-08	West Slide Path	5029510.9	366392.1	63.7	10.1
18-09	West Abutment	5029489.7	366404.0	63.7	9.0
18-10	West Abutment	5029478.3	366410.8	63.7	9.0
18-11	West Abutment	5029468.5	366417.0	63.7	9.1
18-12	West Abutment	5029459.4	366422.0	63.4	9.1

Thurber contacted Ontario One Call in advance of the field investigation to obtain utility locates/clearances in the vicinity of the proposed boreholes. Private locate services were carried out for Boreholes located on private property.

Boreholes 16-07 and 16-08 were advanced through the highway embankment and were drilled with a truck-mounted CME 75 drill rig. All other off-road boreholes were drilled with a track-mounted CME 55LC drill rig.

The subsurface stratigraphy encountered in the boreholes was recorded in the field by Thurber personnel. Split spoon samples were collected at regular depth intervals in the boreholes via the completion of Standard Penetration Tests (SPT), following the methods described in ASTM Standard D1586. Rock was cored and collected using NQ coring equipment. All soil and rock core samples recovered from the boreholes were transported to Thurber's Ottawa geotechnical laboratory for further examination and testing.

A standpipe piezometer was installed in Borehole 16-04. The piezometer consisted of ¾" (19 mm) PVC pipe with a 1.5 m long slotted screen installed just above the bedrock surface and surrounded by filter sand. Solid pipe was used above the slotted screen and the backfill included a bentonite seal.

The boreholes were backfilled in accordance with MOEE requirements (O.Reg. 903). Boreholes 16-03, 16-07 and 16-08 were capped with 200 mm of cold patch asphalt to reinstate the traveling surface.

The approximate borehole locations are shown on the Borehole Locations and Soil Strata Drawing included in Appendix A. The coordinates and elevation of the boreholes are provided on this drawing and on the individual Record of Borehole sheets.

3.2 Laboratory Testing

Geotechnical laboratory testing consisted of natural moisture content determination and visual identification of all retained soil samples in accordance with the current MTO standards. Grain size distribution analyses and Atterberg Limit testing was also carried out on selected samples to MTO and ASTM standards. All rock cores were photographed and their total core recovery (TCR), solid core recovery (SCR) and rock quality designation (RQD) were determined. Twenty-three rock core samples were submitted to Stantec's laboratory in Ottawa for unconfined compression (UCS) testing. Chemical analysis for determination of pH, conductivity, resistivity, soluble sulphate and chloride concentrations was carried out on two soil samples.

The results of the geotechnical tests are summarized on the Record of Borehole sheets included in Appendix B and all laboratory results are included in Appendix C.

4 DESCRIPTION OF SUBSURFACE CONDITIONS

Details of the encountered soil stratigraphy are presented on the Record of Borehole sheets included in Appendix B and the Borehole Location and Soil Strata drawing included in Appendix A. A general description of the stratigraphy, based on the conditions encountered in the boreholes, is given in the following paragraphs. However, the factual data presented on the Record of Borehole sheets takes precedence over this general description for interpretation of the site conditions. It must be recognized that the soil and groundwater conditions may vary between and beyond borehole locations.

In general terms, the investigation advanced through the highway embankment found that the asphalt was underlain by a road base/subbase and embankment fill. Fill was also encountered from surface and underlying the topsoil and asphalt in the off-road boreholes. The fill materials were underlain by silt, sand and gravel materials and/or till overlying limestone bedrock with interbedded shale. A discontinuous layer of clay was also encountered underlying the fill materials.

4.1 Topsoil

A surficial layer of topsoil was encountered in Boreholes 16-01, 16-02, 16-04, 16-06, 16-09 and 16-10 with a thickness ranging from 100 to 200 mm.

4.2 Highway 417 Embankment Fill

Boreholes 16-07 and 17-08 were drilled through the existing Highway 417 pavement structure behind the existing bridge abutments.

4.2.1 Asphalt

The thickness of the asphalt was 250 mm in both boreholes.

4.2.2 Base/Subbase Fill: Gravel with Sand and Silt to Sand with Gravel and Silt

Fill material consisting of gravel with sand and silt to sand with gravel and silt was encountered below the asphalt in Boreholes 16-07 and 16-08. The thickness of the fill was 1.3 m with the base of the fill at 1.5 m depth below the existing highway surface (elev. 67.4 to 70.5 m).

The SPT tests conducted in the fill gave N-values ranging from 28 to 33 blows indicating a relative density of compact to dense.

Recorded moisture contents were 2% to 3% within the road base/subbase. The results of grain size analyses conducted on two samples of this material are summarized below and are illustrated on Figure C1 in Appendix C.

Soil Particle	Percentage (%)
Gravel	45 - 49
Sand	41 - 45
Silt and Clay	10

4.2.3 Embankment Fill: Sand

A sand fill was encountered below the pavement base/subbase in Boreholes 16-07 and 16-08. The thickness of the fill was 4.3 m in both boreholes with the base of the fill at 5.8 m below the existing ground surface (elev. 63.1 to 66.2 m).

The SPT tests conducted in the fill gave N-values typically ranging from 3 to 28 blows indicating a relative density of very loose to dense. One SPT N-value of 100 blows per 250 mm penetration was recorded in Borehole 16-07 and may be indicative of a cobble or boulder.

Recorded moisture contents were 4 to 10% within the embankment fill. The results of grain size analyses conducted on two samples of the embankment fill materials are summarized below and are illustrated on Figure C2 in Appendix C.

Soil Particle	Percentage (%)
Gravel	1
Sand	95 - 96
Silt and Clay	3 - 4

4.3 Fill: Beyond Highway 417 Embankment

4.3.1 Fill: Asphalt

Borehole 16-03 was drilled through a parking lot and encountered a 100 mm layer of asphalt at surface.

4.3.2 Fill: Gravel with Sand to Silty Gravel with Sand

A gravel with sand to silty gravel with sand layer was encountered below the asphalt in Borehole 16-03, below the topsoil in Borehole 16-09 and at surface in Boreholes 18-08 and 18-09. This layer contained occasional to frequent cobbles and boulders in Boreholes 16-09, 18-08 and 18-09. The thickness of this layer ranged from 1.0 to 2.3 m with a base depth ranging from 1.2 to 2.3 m below the existing ground surface (elev. 61.4 to 64.5 m).

The SPT tests conducted in the fill gave N-values typically ranging from 6 to 72 blows indicating a relative density of loose to very dense. One SPT N-value of 100 blows per 150 mm penetration was recorded in Borehole 18-08 and may be indicative of a cobble or boulder.

Recorded moisture contents were typically 4 to 14% within the fill. The results of grain size analyses conducted on four samples of the fill materials are summarized below and are illustrated on Figure C3 in Appendix C.

Soil Particle	Percentage (%)
Gravel	42 - 63
Sand	30 - 42
Silt and Clay	7 - 18

4.3.3 Fill: Silty Sand to Sand With Silt and Gravel

A silty sand to silty sand with gravel to sand with silt and gravel fill was encountered below the topsoil in Boreholes 16-01, 16-02, 16-04, 16-06 and 16-10, and from the ground surface in Boreholes 16-05, 16-11, 18-10, 18-11 and 18-12. The thickness of this layer ranged from 1.4 to 3.5 m with base depths ranging from 1.5 to 3.7 m (elev. 58.8 to 61.9 m) below the existing ground surface.

The SPT tests conducted in the fill gave N-values ranged from 4 to 47 blows indicating a relative density of loose to dense.

Recorded moisture contents were 5 to 22% within this fill unit. The results of grain size analyses conducted on eleven samples of the fill material are summarized below and are illustrated on Figures C4 and C5 in Appendix C.

Soil Particle	Percentage (%)
Gravel	1 - 41
Sand	41 - 86
Silt and Clay	8 - 24

4.4 Clay

A native deposit of clay with some sand was encountered below the fill in Boreholes 16-03 and 16-08. Trace amounts of rootlets were encountered within this layer at Borehole 16-03. The thickness of this deposit ranged from 0.9 to 2.4 m with an underside depth of 2.9 to 8.2 m below existing ground surface (elev. 60.7 and 63.6 m). The SPT N-values ranged from 3 to 9 blows.

The moisture content of the samples tested ranged from 32 to 41%. The results of grain size analyses conducted on two samples of the native clay are summarized below and are illustrated on Figure C6 in Appendix C.

Soil Particle	Percentage (%)
Gravel	0 - 3
Sand	14 - 21
Silt	42 - 51
Clay	34 - 35

Atterberg Limit testing was completed on two samples of the native clay deposit. The results are summarized on the Record of Borehole sheets in Appendix B and the Atterberg Limit graphs are included in Figure C11 of Appendix C. The laboratory results are summarized below and indicate that the clay is low to intermediate plasticity (CL-CI).

Parameter	Value
Liquid Limit	33 - 35
Plastic Limit	16 - 17
Plasticity Index	17 - 18

4.5 Sandy Silt to Silty Sand

A deposit of sandy silt to silty sand was encountered below the fill in Boreholes 16-07, 16-09, 16-10, 18-10 and 18-11 and below the clay layer in Borehole 16-03. Trace organics and/or wood fragments were noted in Boreholes 16-07 and 16-10. The thickness of this deposit ranged from 0.4 to 4.7 m with a base depth of 2.8 to 10.5 m below the existing ground surface (elev. 60.3 to 61.5 m).

The SPT tests conducted in this deposit gave typical N-values of 8 to 39 blows indicating a relative density of loose to dense. SPT N-values recorded near the suspected bedrock surface in Boreholes 16-07 and 16-10 were 100 blows for 25 mm of penetration.

Recorded moisture contents were 9 to 42%. The results of grain size analyses conducted on five samples of the material are summarized below and are illustrated on Figure C7 in Appendix C.

Soil Particle	Percentage (%)
Gravel	1 - 12
Sand	44 - 83
Silt and Clay	15 - 55

4.6 Silty Sand with Gravel to Gravel with Sand and Silt (Glacial Till)

A layer of glacial till ranging in composition from sandy silt to silty sand with gravel to gravel with sand and silt was encountered in all boreholes except 16-07, 16-8 and 16-10. The thickness of the till ranged from 0.2 to 1.4 m with a base depth ranging from 2.4 to 5.2 m below the existing ground surface (elev. 58.2 to 61.3 m). Occasional cobbles and boulders were noted in this layer.

The SPT tests conducted in the till gave N-values typically ranging from 5 to 36 blows indicating a relative density of loose to dense. SPT N-values recorded near the bedrock surface ranged from 100 blows per 50 to 275 mm penetration.

Recorded moisture contents ranged from 3 to 11%. The results of grain size analyses conducted on eight samples of the till are summarized below and are illustrated on Figures C8 and C9 in Appendix C.

Soil Particle	Percentage (%)
Gravel	0-51
Sand	37 - 94
Silt and Clay	6 - 52

4.7 Bedrock

The overburden materials were underlain by grey limestone bedrock with some interbedded shale. Bedding and fractures were both noted to be near horizontal. The bedrock depth ranged from 2.4 to 10.5 m below the existing ground surface (elev. 58.2 to 61.5 m). The depth to bedrock and bedrock surface elevation are summarized in the table below. Photographs of the bedrock core are provided in Appendix B.

Borehole	Depth to Bedrock (m)	Bedrock Surface Elevation (m)	Comments
16-01	3.3	59.5	Cored 3.9 m
16-02	4.3	58.2	Cored 9.4 m
16-03	5.2	61.3	Cored 4.0 m
16-04	2.9	60.5	Cored 3.0 m
16-05	2.9	60.9	Cored 3.3 m
16-06	3.4	59.1	Cored 3.8 m
16-07	10.5	61.5	Auger refusal on inferred bedrock
16-08	8.2	60.7	Auger refusal on inferred bedrock
16-09	3.0	60.5	Cored 4.5 m
16-10	3.4	60.3	Cored 3.8 m
16-11	3.5	60.3	Cored 8.8 m
18-08	2.4	61.3	Cored 7.7 m
18-09	3.1	60.6	Cored 5.9 m
18-10	3.3	60.5	Cored 5.7 m
18-11	3.3	60.4	Cored 5.8 m
18-12	3.1	60.3	Cored 6.0 m

The total core recovery ranged from 64% to 100%, the solid core recovery ranged from 56% to 100% and the Rock Quality Designation (RQD) ranged from 32% to 100%. Only 15% of the RQD values were determined to be less than 75%. Based on the RQD value the bedrock is classified as poor to excellent quality, but generally good to excellent. The results of the UCS testing on select cores of the bedrock yielded strengths ranging from 93 to 157 MPa, with an average of 124 MPa, indicating that it has an average strength classification of very strong.

4.8 Groundwater

The water levels were measured in the standpipe piezometer installed in Borehole 16-04 and the results are presented in the table below along with those observed in open Boreholes 16-07, 16-09 and 16-11 after drilling:

Borehole	Groundwater Level		Date of Measurement
	Depth (mbgs)	Elevation (m)	
16-04	2.7	60.7	March 12, 2018
	2.8	60.8	March 23, 2018
16-07	9.4	62.6	March 28, 2017
16-09	2.4	61.1	April 11, 2017
16-11	2.3	61.5	April 11, 2017

These observations are considered short term and it should be noted that the groundwater level at the time of construction and seasonal fluctuations of the groundwater level are to be expected. In particular, the groundwater level may be at a higher elevation after periods of significant and/or prolonged precipitation events.

4.9 Analytical Testing

Two samples of soil were submitted to Paracel Laboratories in Ottawa, Ontario for analysis of pH, water soluble sulphate and chloride concentrations and resistivity. The analysis results are included in Appendix C and are summarized in the table below:

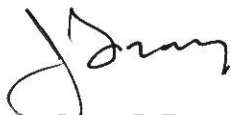
Borehole	Sample	Depth (m)	Sulphate (µg/g)	pH (-)	Resistivity (Ohm-cm)	Chloride (µg/g)
16-4	SS4	2.3 – 2.9	64	7.90	4410	33
18-11	SS4	3.1 – 3.7	122	7.89	1480	333

5 MISCELLANEOUS

Borehole locations were selected by Thurber relative to existing site features and the anticipated foundation locations. The as-drilled locations and ground surface elevation were measured by Thurber following completion of the field program.

George Downing Estate Drilling Ltd. of Hawkesbury, Ontario supplied and operated the drilling equipment to conduct the drilling, soil sampling, in-situ testing and borehole decommissioning for all the boreholes on site. The field investigation was supervised on a full time basis by Mr. Justin Gray, E.I.T., Ms. Katya Edney, P.Eng., and Sean O'Bryan of Thurber. Overall supervision of the investigation program was provided by Mr. Fred Griffiths, P.Eng.

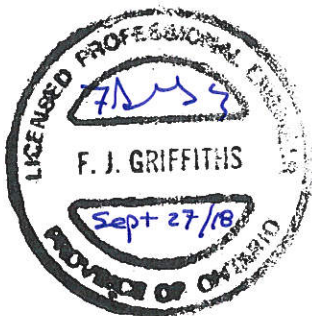
Routine geotechnical laboratory testing was completed by Thurber's laboratory in Ottawa, Ontario. UCS testing was completed by Stantec's laboratory in Ottawa, Ontario. Interpretation of the factual data and preparation of this report were carried out by Mr. Justin Gray, E.I.T. and Mr. Paul Carnaffan, P.Eng. The report was reviewed by Dr. Dr. P.K. Chatterji, P.Eng. a Designated Principal Contact for MTO Foundation Projects.



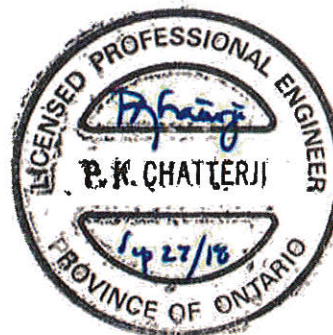
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REPLACEMENT OF CPR/O-TRAIN BRIDGES
HIGHWAY 417, OTTAWA
SITE 3-54**

**GWP 5170-13-00
Geocres No.: 31G5-293**

PART 2. ENGINEERING DISCUSSION AND RECOMMENDATIONS

6 GENERAL

This report presents the interpretation of the factual data obtained from a foundation investigation conducted by Thurber for the replacement of the Highway 417 overpass structures at the CPR/O-Train tracks in Ottawa, Ontario. Geotechnical recommendations are provided to assist the design team in designing a suitable foundation for the proposed bridge replacements.

This foundation investigation and design report with the interpretation and recommendations are intended for the use of the Ministry of Transportation, and shall not be used or relied upon for any other purposes or by any other parties including the construction or design-build contractor. Contractors must make their own interpretation based on the factual data in Part 1 of the report. Where comments are made on construction, they are provided only in order to highlight those aspects which could affect the design of the project. Contractors must make their own interpretation of the factual information provided as it may affect equipment selection, proposed construction methods and scheduling.

The following sections address geotechnical recommendations for the replacement of the existing overpass structures and related retaining walls. The discussions and recommendations presented in this report are based on the information provided by WSP and on the factual data obtained during the course of this investigation.

6.1 Existing Foundations

Based on the historical contract drawings (Works Project No. 938-59), the existing Highway 417 Overpass of the CPR/O-Train tracks was constructed in 1961 as twin five span structures (separate structures for EB and WB lanes). The historical drawings indicate that each structure had an overall length of approximately 280 ft (85.3 m) and width of 70.5 ft (21.5 m). The abutments and piers are supported by pedestal footings bearing on bedrock.

It is noted that the historical drawings show the CPR tracks slightly above the original bedrock surface at approximately elevation 61.8 m and show a "Future Cut in Rock" with a track profile lowered to approximately elevation 55.0 m. (copy provided in Appendix E).

The founding elevations for the existing structure are summarized as follows:

West Abutment	60.96 m
Pier B	59.44 m
Pier C	54.86 m
Pier D	54.86 m
Pier E	57.88 m
East Abutment	58.52 m

The ground in front of the abutments was sloped at 2H:1V and covered with concrete slope paving. The contract included a cast-in-place concrete counterfort retaining wall on the south side of the highway extending back from the west abutment for a distance of approximately 32.6 m. A concrete crib wall was provided west of the counterfort wall and on the north side of the west approach. The east approach was constructed with 2H:1V earth slopes.

Based on a visual inspection of the site, the existing rail tracks are set within a rock cut. A short, near vertical rock face is visible on both side of the track alignment. Concrete crib walls are present above the rock faces (see Photo 6 in Appendix D).

6.2 Proposed Structure

It is understood that the existing five-span bridge structures are to be replaced with a single span structure using rapid replacement techniques. The preliminary general arrangement drawing (dated April 2018, copy provided in Appendix E) indicates that the span of the replacement structure will be 40 m, with the new abutments located behind the middle piers of the existing structures.

The proposed span arrangement will allow for passage of the existing O-Train track, a future track to the east of the existing track, a multi-use pathway (MUP) on the east side and a future MUP on the west side. The minimum vertical clearance beneath the structure at the track alignment is 12.5 m. The profile of the multi-use pathway is about 8 m higher than the rail tracks and will be constructed above an RSS slope.

The abutment walls extend out beyond the edges of Highway 417 and taper down to tie into the adjacent highway embankment. The length of the abutment wall extensions ranges from approximately 7.5 m at the northwest quadrant to 14 m at the southeast quadrant.

At the northwest quadrant, the abutment wall will tie into a new retaining wall that is to be constructed and which is addressed in a separate Foundation Investigation and Design Report. At the other three quadrants, the Highway 417 embankment will slope down at 3H:1V.

Key elevations (approximate) related to the proposed design include:

Highway 417 top of pavement	69.8 to 71.0 m
Top of rails (O-Train Track)	55.1 m
East side MUP profile	62.8 m

The deck and girders of the existing bridge will be removed and the existing abutments and outer piers will be removed to at least 1.8 m below finished grade on Highway 417 prior to

backfill of the new structure. The columns of the existing inner piers will be removed to just below finished grade.

6.3 Applicable Codes and Design Considerations

The geotechnical assessment presented below has been prepared based on the available data regarding the proposed foundations and existing ground conditions and in accordance with the Canadian Highway Bridge Design Code, version CSA S6-14 (CHBDC).

In accordance with CHBDC, the analysis and design of structures take into consideration the importance of the structure and the consequence associated with exceeding limit states. The importance category and consequence classification are defined by the Regulatory Authority, which in this case is the Ministry of Transportation, Ontario (MTO).

It is understood that MTO has designated this structure as follows:

Table 6-1: Bridge Structure Classification

Criteria	Classification	CHBDC Section
Importance Category	Major Route Bridge	4.4.2
Consequence Classification	Typical Consequence	6.5.1

Accordingly, a consequence factor (Ψ) of 1.0, as per Table 6.1 of the CHBDC, has been used in assessing factored geotechnical resistances.

With respect to Section 6.5.3.2 of the CHBDC, the degree of site and prediction model understanding is considered "Typical Understanding".

6.4 Frost Protection

The frost penetration depth at this site is 1.8 m as per OPSD 3090.101. Accordingly, a minimum of 1.8 m of earth cover, or equivalent insulation, must be provided above the base of the shallow foundations or pile caps to serve as frost protection. The requirement for frost protection can be waived where the foundations are placed on sound bedrock free of open fractures.

6.5 Geotechnical Assessment

Based on the results of the field and laboratory investigation and the information provided by WSP with regards to the proposed project requirements, foundation design considerations include:

- Shallow bedrock is present throughout the site; the bridge abutments and retaining walls should be founded on the bedrock.
- The current rock line is irregular due to previous rock excavation to create the cut section for the existing rail tracks. A short section of rock face is visible within the existing rail cut, however, the upper portion is behind concrete crib walls. The geometry and condition of the hidden rock face is not known. The abutments and piers of the existing bridges are founded on bedrock at elevations ranging from 54.86 m to 60.96 m.

- Spread footings founded on sound level bedrock at this site will offer a high bearing resistance. The available bearing resistance will be reduced where the footings are to be located close to and above a rock face.
- Due to the requirement for a rapid replacement, the foundations and sub-structure for the new bridge must be constructed beneath the existing bridge deck and/or from above with only short duration single lane closures. The deck replacement will occur during a weekend full-closure of Highway 417.
- The existing O-Train will remain operational during the construction of the new foundations and sub-structure.
- The allowance for a future second rail track and retaining wall supporting the multi-use pathway may require future excavation in front of and below the founding elevation for the new east abutment. Protection of the new abutment will need to be considered by the future wall designers and contractor.

7 SEISMIC CONSIDERATIONS

7.1 Spectral and Peak Acceleration Hazard Values

The seismic hazard data for the CHBDC is based on the fifth-generation seismic model developed by the Geological Survey of Canada (GSC). Seismic hazard data for this site has been obtained from the GSC's seismic hazard calculator. The data includes peak ground acceleration (PGA), peak ground velocity (PGV), and the 5% damped spectral response acceleration values ($S_a(T)$) for the reference ground condition (Site Class C) for a range of periods (T) and for a range of return periods including the 475-year, 975-year and 2475-year events. The GSC seismic hazard calculation data sheet for this site is attached.

The site coefficients used to determine the design spectral acceleration and displacement values are a function of the Site Class and the peak ground acceleration (PGA). The PGA value for this site is 0.280 g for Site Class C with a 2% probability of exceedance in 50 years. This value is to be scaled by the $F(PGA)$ based on the site specific Site Class.

7.2 CHBDC Seismic Site Classification

In accordance with the CHBDC, the selection of the seismic classification is based on the soil and rock within the upper 30 m of the stratigraphy. Review of data from published shear wave velocity measurements within 1.5 km of the site indicates that where rock is at a depth of less than 3 m the average shear wave velocity in the upper 30 m is greater than 760 m/s (GSC 6273 – Seismic Site Class Ottawa, Data Points 250 and 527). Since the existing overburden soils are to be removed and the structures are to be founded directly on bedrock, the site has been determined to be a seismic Site Class B in accordance with Table 4.1 of the CHBDC.

7.3 Seismic Liquefaction

The new structures are to be backfilled with new granular material placed and compacted in accordance with OPSS 501. The potential for liquefaction of the approach fills during a seismic event is considered low in accordance with CHBDC (S6-14) Clause C4.6.6. Therefore, liquefaction is not considered to be a concern for this site.

8 FOUNDATION DESIGN ALTERNATIVES

8.1 Abutment Foundation Alternatives

The results of the field and laboratory investigation and historical data indicate that the embankment fill is underlain by glacial till deposits.

Given the soil stratigraphy encountered and the requirements of the proposed structure provided by WSP, the following foundation alternatives were considered for the new bridge foundations:

- Caissons (drilled shaft socketed into bedrock)
- Micropiles
- Spread footings

These foundation alternatives are presented in the following sections and evaluated from a geotechnical perspective in terms of their respective advantages, disadvantages, risks and consequences. The evaluation is summarized in the table provided in Appendix F. A preferred foundation from a geotechnical engineering perspective is identified in Section 8.5.

8.2 Caissons (drilled shaft socketed into bedrock)

The new bridge could be founded on rock-socketed caissons. A series of retaining walls would be required, including the abutment walls and walls to retain the soil beneath the MUP on the east side of the tracks.

Advantages

- High axial and lateral resistance when socketed into sound bedrock

Disadvantages

- Headroom beneath the existing bridge deck limits the equipment that can be used for caisson installation. Consideration was given to drilling through the existing bridge deck, however, the work could not be completed within the permitted lane closure restrictions.
- Very strong rock conditions limit the equipment that can be used for caisson installation.
- If retaining walls need to be constructed in front of the caissons, the potential for load transfer between the caissons and the wall would need to be considered and would add to the design complexity.
- The use of caissons does not reduce the depth of excavation required compared to a spread footing design.

Relative Cost

- Highest cost alternative

Risk/Consequences

- Drilling progress is slower than planned due to the combination of very strong rock and equipment able to work within clearance restrictions leading to delays to project schedule.

8.3 Micropiles

The bridge abutments could be supported on micropiles installed into the bedrock. The micropiles could be installed through a combination of rock and granular fill but would derive their support from the bedrock. A series of retaining walls would be required including the abutment walls and to retain the soil beneath the MUP on the east side of the tracks.

Advantages

- Limited headroom beneath existing bridge deck is less of a constraint compared to caissons.
- Easy to install micropiles on a batter to help develop resistance to uplift or lateral loads.

Disadvantages

- The use of micropiles does not reduce the depth of excavation required compared to a spread footing design; a pilecap would be constructed at similar elevation as the spread footing option.
- Low lateral resistance.

Relative Cost

- Moderate cost: higher than spread footings but less than rock socketed caissons

Risk/Consequences

- Exact alignment and condition of existing rock face not known throughout new abutment limits. Inclined micropiles may intercept the rock face or fractured rock/ potential for difficulty maintaining micropile alignment and/or of binding of the drill head due to mixed ground conditions.

8.4 Spread Footings

The new abutments could be supported on spread footings bearing on bedrock.

Advantages

- Does not require specialty equipment
- Can be carried out within limited headroom conditions
- High bearing resistance when founded on sound level bedrock

Disadvantages

- Development of resistance to uplift, sliding or overturning may require a larger footing and/or use of rock anchors
- Reduced bearing resistance where founded close to top of existing rock cut
- Potential conflict between new abutment foundations and footings for existing bridge piers.

Relative Cost

- Lowest cost alternative

Risk/Consequences

- The exact alignment and condition of the existing rock face are not known throughout the new footing limits. The volume of rock excavation and/or mass concrete may be greater than initially estimated.

8.5 Recommended Foundation

Based on the proposed structure geometry and evaluation of foundation alternatives presented above, the recommended foundation approach from a geotechnical perspective is to support the abutments on spread footings on the bedrock above the railway cut and set back from the top edge of the cut.

9 FOUNDATION DESIGN RECOMMENDATIONS

9.1 Shallow Foundations

9.1.1 Subgrade Preparation

Shallow foundations for the bridge abutments and retaining walls for this project should be founded on clean level sound bedrock. The bedrock surface is expected to be encountered between approximately elevation 58.2 and 60.5 m at the east abutment/abutment walls and between approximately 60.3 and 61.3 m at the west abutment/abutment walls.

Excavation for the shallow foundations should be carried out in accordance with OPSS 902.

All overburden soil, fill, debris and loose rock must be removed from beneath the footings. If bedrock is found to be sloped, rock should be removed to provide a level or stepped footing.

It is noted that some bedrock excavation may be required to achieve the design foundation elevation. It is also possible that areas of overexcavated rock associated with construction of the existing bridge piers may be encountered and would need to be backfilled with concrete of the same class and strength as the footing concrete.

Since the existing bridge is to remain in service during construction of the new bridge abutments, the caution must be taken to ensure that the existing foundations are not negatively impacted. It is understood that a theoretical check has been carried out to ensure that there is no conflict between the proposed and existing footings, however, this should be verified once excavation has been completed to expose the existing foundations.

9.1.2 Design Parameters

Shallow foundations for the abutments and abutment walls, set back a minimum of 2 m from the face of the rock cut and prepared as described above may be designed based on the factored geotechnical resistances provided below:

- Factored geotechnical resistance at ULS 5,000 kPa
- Factored geotechnical resistance at SLS N/A*

* Settlement at this level of loading is expected to be less than 5 mm and therefore the SLS does not apply.

The factored geotechnical resistances include the following factors:

- Consequence factor (Ψ) of 1.0
- Geotechnical resistance factor $\phi_{gu} = 0.5$ (analysis; typical degree of understanding)

The geotechnical resistance is for vertical concentric loading and will need to be adjusted for the effects of inclined or eccentric loading, if applicable, as illustrated in the CHBDC Clause 6.10.3 and Clause 6.10.4.

Resistance to lateral forces through sliding resistance may be calculated based on the following unfactored interface friction factors:

- Concrete on clean sound bedrock 0.7
- Concrete on mass concrete 0.7

The factored resistance to sliding should include the following factors:

- Consequence factor (Ψ) of 1.0
- Geotechnical resistance factor $\phi_{gu} = 0.8$ (analysis; typical degree of understanding)

9.2 Abutment Backfill

The existing five-span structures are to be replaced with single span structures using rapid bridge replacement (RBR) techniques. The depth of backfill directly behind the abutments is approximately 8.5 m and the zone of backfill extends approximately 20 to 25 m back from the abutments. The new abutments will be constructed beneath the existing bridge deck and the intention is to place as much of the backfill as possible prior to the RBR; the remainder of the backfill will need to be placed during the RBR closure.

Design considerations include:

- Schedule. As noted above, demolition of the existing bridge superstructure, completion of backfilling operations, installation of the new superstructure and paving must be completed within a full closure of Highway 417 over a long weekend. Therefore, there is a strong desire to place as much backfill as possible in advance of the RBR closure and to place the remainder of the backfill as quickly as possible.

- The selection of the backfill material and methods for placement/compaction must ensure that post-construction settlement is within acceptable limits.
- The selection of the backfill material must ensure that adequate drainage is provided to prevent hydrostatic pressure build-up behind the walls. Good drainage at the subgrade level of the pavement structure must also be provided.
- It is understood that pavement reinstatement will be carried out with flexible pavement consisting of 230 mm of hot mix over 150 mm of Granular A over Granular B Type II. The minimum thickness of Granular B Type II would be 150 mm but may be thicker depending on the underlying backfill material. Overall, the pavement structure is expected to range from 530 to 680 mm thick.
- Due to time constraints, the new median sewer will be constructed during weekend lane closures following the bridge replacement. It is understood that the sewer will have 1.8 m of cover and therefore excavations up to about 2.5 m deep through the recently placed backfill will be required within the Highway 417 median shoulders.
- The risk and consequence of potential problems, both in terms of long term performance and during construction should be considered.
- Cost: the most cost-effective option that meets schedule and performance requirements without presenting excessive risk would be preferred.

Alternative backfill materials are presented and evaluated in a table in Appendix F. The alternatives that were evaluated include:

- 1) Conventional granular backfill (OPSS.PROV 1010, Granular A or B)
- 2) Clear stone (OPSS.PROV 1004, 19 mm Type II)
- 3) Cellular concrete (MTO DSM – Lightweight Fill Material); and
- 4) Unshrinkable fill (OPSS.PROV 578)

Although the alternatives are presented as separate alternatives, the preferred approach may include the use of a combination of backfill materials (e.g. conventional granular fill within the lower portion of the backfill with one of the other alternatives for the upper backfill during the RBR closure). Evaluation of the materials as separate alternatives allows for identification of the major advantages, disadvantages, risks/consequences and relative costs of the backfill materials.

Based on the review of the available options, the following backfill materials and approach are recommended:

- Place geosynthetic wall drain vertically against the back of the abutment and wingwalls.
- Backfill with conventional compacted granular backfill up to approximately 2 m below the existing bridge girders.
- Backfill to within approximately 0.1 m below the existing bridge girders using unshrinkable fill.
- After demolition of existing bridge, place remaining backfill (approximately 600 mm) with Granular B Type II (2 lifts) followed by approach slab and pavement structure (approximately 680 mm total thickness).

Some settlement of the backfill is expected. Delaying the installation of the sleeper slabs for one year would allow for some of the settlement to occur prior to final paving. This approach would be beneficial to the longer term pavement performance.

9.3 Lateral Earth Pressures

The abutment walls and retaining walls will be subject to lateral earth pressures from the backfill. The lateral earth pressure parameters provided in Table 9.1 and 9.2 in the sections below are based on the following assumptions:

- That the backfill is fully drained so that there are no unbalanced hydrostatic pressures. If adequate drainage cannot be confirmed, the potential for buildup of hydrostatic pressures should be considered in the design.
- The backfill behind the wall is horizontal. If sloping ground is present behind the wall, the earth pressure parameters will need to be reassessed to account for the sloping ground conditions.

9.3.1 Static Lateral Earth Pressure Coefficients

Lateral earth pressures acting on the abutment walls and retaining walls 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 (kPa)

K = earth pressure coefficient

γ = unit weight of retained soil (kN/m³)

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

q = value of any surcharge (kPa)

The recommended static lateral earth pressure parameters for use in design are provided in Table 9.1. The values for OPSS Granular A & B Type II or 19 mm clear stone should be used for abutment walls which include granular backfill as per OPSD 3101.150. The values for the existing fill are provided for conceptual design of protection systems if required.

Table 9.1: Static Lateral Earth Pressure Coefficients

Parameter	OPSS Granular A & B Type II	Existing Fill	Clear Stone
Soil Unit Weight, kN/m ³ , γ	21.0	20.0	18.5
Angle of Internal Friction, ϕ	35°	30°	39°
Coefficient of at Rest Earth Pressure, K_o (Restrained Wall)	0.43	0.50	0.37
Coefficient of Active Earth Pressure, K_a (Unrestrained Wall)	0.27	0.33	0.23

For rigid structures, it is recommended that at-rest horizontal lateral earth pressures be used for design. Active pressures should be used for the design of unrestrained walls.

For static analysis, passive earth resistance should be ignored and have therefore not been provided.

A lateral pressure due to backfill compaction should be added to the calculated lateral earth pressure in accordance with Section 6.12.3 of the CHBDC. In addition, surcharge loads due to other structures, traffic loading or construction equipment should be accounted for in design where applicable.

9.3.2 Combined Static and Seismic Lateral Earth Pressure Parameters

The following recommendations are per Section C4.6.5 of the Commentary of the CHBDC. The recommended seismic lateral earth pressure parameters provided in Table 9.2 below have been assessed based on the seismic hazard data discussed in Sections 8.1 and 8.2 (Seismic Site Class of B, and a PGA with a 2% probability of exceedance in 50 years of 0.280g) and an F(PGA) of 0.87 as per Table 4.8 of the CHBDC.

The seismically induced lateral soil pressures from the backfill may be calculated using the Mononobe-Okabe Method with:

- $k_h = \frac{1}{2} F(\text{PGA}) \cdot \text{PGA}$ for walls capable of moving 25 to 50 mm, and
- $k_h = F(\text{PGA}) \cdot \text{PGA}$ for non-yielding walls (< 25 mm movement)

Table 9.2: Lateral Earth Pressure (Under Seismic Loads)

Parameter	OPSS Granular A & B Type II	Clear Stone
Soil Unit Weight, kN/m^3 , γ	21.0	18.5
Angle of Internal Friction, ϕ	35°	39°
Coefficient of Active Earth Pressure, K_{AE} (Restrained Wall)	0.43	0.37
Coefficient of Active Earth Pressure, K_{AE} (Unrestrained Wall)	0.34	0.29

The total pressure due to combined static and seismic loads acting at a specific depth below the top of the wall may be determined using the following equation that includes consideration of material properties and the soil profile:

$$\sigma_h = K\gamma d + (K_{AE} - K_a) \gamma (H - d)$$

where:

σ_h = lateral earth pressure at depth, d (kPa)

d = depth below the top of the wall (m)

K = static earth pressure coefficient

(K_a for yielding walls, K_o for non-yielding walls)

γ = unit weight of the backfill soil (kN/m^3)

K_{AE} = combined static and seismic earth pressure coefficient

H = total height of the wall (m)

The design parameters provided above for lateral earth pressures associated with wall backfill should be considered preliminary and will need to be reviewed once the geometry of the walls, backslope above the walls and limits of backfill materials have been established.

9.4 Rock Anchors

If required, foundation resistance to sliding or overturning can be increased by providing rock anchors.

Rock anchors should be constructed in accordance with OPSS 942 – Construction Specification for Prestressed Soil and Rock Anchors.

The design of grouted rock anchors should be carried out in accordance with Section 6.13 of the CHBDC. Additional guidance is provided in the following document from the Post Tensioning Institute (PTI):

PTI DC35.1-14: Recommendations for Prestressed Rock and Soil Anchors (2014)

The PTI document is referenced in both the commentary to the CHBDC and the OPSS specification governing the construction of rock anchors (OPSS 942).

The design of the rock anchors must include both structural and geotechnical components. From a geotechnical perspective, the design must consider failure of the rock to grout bond as well as failure within the rock mass (sometimes referred to as rock cone pull-out).

The factored geotechnical resistance at ULS should include the following factors:

- Consequence factor (Ψ) of 1.0
- Geotechnical resistance factor (ϕ_{gu}) of 0.4 (analysis; typical degree of understanding)

The factored geotechnical resistance at ULS with respect to the rock to grout bond may be calculated based on an ultimate (un-factored) bond stress of 1500 kPa (the corresponding factored bond stress at ULS is 600 kPa).

A higher bond stress may be possible but would need to be verified by pre-production test anchors. The completion of pre-production test anchors would also allow for use of a higher geotechnical resistance factor ($\phi_{gu} = 0.6$).

The factored geotechnical resistance at ULS for a single anchor with respect to failure within the rock mass may be calculated based on the mass of an inverted cone having its apex at the mid-point of the bonded zone and having an apex angle of 90 degrees. The submerged unit weight should be used for rock located beneath the groundwater level. Where multiple anchors are present, the overlap of cones must be considered.

Guidance regarding the minimum bond length, minimum free-stressing length, anchor spacing, drill hole diameter, etc. is provided in PTI DC35.1-14.

All permanent rock anchors should include a double corrosion protection system.

9.5 Approach Embankments

Embankment construction should be carried out in accordance with OPSS.PROV 206. The embankment material behind the proposed structure walls should consist of imported Granular B Type II material or better.

Granular fill should be placed and compacted in accordance with OPSS.PROV 501. Where new embankment fill is placed against existing embankment slopes or fill, the existing slope must be benched in accordance with OPSD 208.010.

It is noted that compaction testing cannot be carried out on clear stone using a nuclear densometer, however, compaction of the clearstone material is still required to arrange the particles into their densest state to limit settlement.

An assessment of settlement, bearing and global stability is summarized in the sections that follow.

9.5.1 Assessment of Settlement

The proposed abutments and abutment walls will be founded on bedrock. Foundation settlement beneath the approach fills is not expected to occur. Settlement of the approach embankments is expected to be limited to settlement of the backfill material itself. The magnitude of the embankment compression constructed with compacted granular materials is in the order of 0.5% of the embankment height and is expected to occur following fill placement. The magnitude of settlement if the abutments are backfilled with uncompacted clearstone could be as high as 8% of the thickness of the clear stone. The use of uncompacted clearstone is not recommended.

9.5.2 Assessment of Global Stability

The maximum height of the embankment slopes is expected to be about 7 m (southwest quadrant). The embankment will be sloped at 3H:1V. The global stability for the proposed embankment constructed using OPSS.PROV 1010 Granular B Type II, with 3H:1V side slopes was evaluated using GeoStudio 2012 Slope/W software for limit equilibrium analysis. A seismic horizontal loading of 0.140, equal to ½ of the site adjusted PGA value (0.280g) was used for seismic analysis. The results of the analysis are summarized below and indicate that the embankments are considered stable under both static and seismic loading conditions.

Table 9-1: Global Stability Analysis Results

Location	Factor of Safety	
	Static Conditions	Seismic Conditions
Southwest Embankment	2.2	1.5

9.6 Multi-Use Pathway RSS

The multi-use pathway (MUP) on the east side of the O-Train tracks (Trillium Pathway) is to be reinstated following completion of the bridge replacement. It is understood that the profile of the pathway will approximately match that of the existing pathway but will be 6.5 m wide. The MUP will be supported on retained soil system (RSS) slope approximately 3 m in height and sloped at 1H:1V.

The RSS should be designed and supplied in accordance with SP 599S22. The following attributed should be specified:

Geometry: Slope (GS)
Performance: Medium
Appearance: Low

The RSS will be founded partially on the bedrock adjacent to the new east abutment and partially on a concrete slab placed to support the temporary CSP protection canopy over the O-Train Tracks, which is supported on bedrock. Bearing resistance and global stability are not a design concern based on the proposed geometry and founding conditions.

9.7 Staging Area

The existing bridge superstructure will be demolished in place. The new superstructure will be constructed immediately north of Highway 417 and then put in place using a jack and slide method.

Boreholes 16-04 and 16-10 were drilled along the proposed slide path alignment on the east side and Boreholes 16-09, 16-11 and 18-08 were drilled along the proposed slide path alignment on the west side. The depth to bedrock was generally found to be between 2.4 m and 3.5 m below ground surface. The overburden consisted of granular fill, silty sand/sandy silt and glacial till deposits.

It is understood that the settlement tolerances are very tight (5 to 10 mm) for the temporary supports. Therefore, it is recommended to support the slide towers on bedrock. Shallow foundations on bedrock may be designed using the recommendations provided in Section 9.1. Alternatively, the use of deep foundations such as rock socketed caissons or micropiles could be considered. It is understood that the temporary supports will be designed by engineers working with/for the specialty contractor.

10 CEMENT TYPE AND CORROSION POTENTIAL

Analytical tests were completed to determine the potential for degradation of the concrete in the presence of soluble sulphates and the potential for corrosion of exposed steel. The concentration of soluble sulphate provides an indication of the degree of sulphate attack that is expected for concrete in contact with soil and groundwater at the site. Soluble sulphate concentrations less than 1000 µg/g generally indicate that a low degree of sulphate attack is expected for concrete in contact with soil and groundwater.

The pH, resistivity and chloride concentration provide an indication of the degree of corrosiveness of the sub-surface environment. The test results provided in Table 4-2 were compared with Table 3.2 of the MTO Gravity Pipe Design Guideline and generally indicate a moderate to severe soil corrosiveness. The test results provided in Table 4-2 may be used to aid in the selection of coatings and corrosion protection systems for buried steel objects.

11 CONSTRUCTION CONSIDERATIONS

11.1 Excavations

All temporary excavations must be conducted in accordance with the requirements of the Occupational Health & Safety Act & Regulations (OHSA) for Construction Projects. The soil at the site should be classified as Type 3 in accordance with OHSA.

Subgrade preparation and construction of foundations must be carried out in the dry.

Selection of the equipment and methodology to excavate and prepare the founding surface is the responsibility of the Contractor.

11.2 Temporary Protection Systems

Temporary protection systems are not required to support Highway 417 during the construction since the new structure is to be constructed beneath the existing structure and then backfilled. However, if required for associated work, temporary protection systems should be provided in accordance with OPSS.PROV 539. Typically, Performance Level 2 would be appropriate unless the system must support foundation loads or utilities sensitive to movement; the performance level should be reviewed once the proximity to existing utilities and structures has been determined.

Design of the temporary protection systems is the responsibility of the contractor. All protection systems should be designed by a Professional Engineer experienced in such designs.

The Contractor will be required to provide a protection envelope above the O-Train tracks in order to isolate the O-Train tracks from the work zone. Foundation Engineering input regarding the proposed CSP Protection Canopy has been provided in a separate memorandum.

11.3 Dewatering

All excavations for foundations must be dewatered prior to the placement of concrete, as per OPSS 902 and Special Provision (SP) No. FOUN0003, dated March 8, 2018.

The deepest open excavation anticipated for this project is above the base of the railway cut. It is anticipated that groundwater may be perched within the soil above the bedrock, particularly following periods of precipitation.

The Contractor must be prepared to control the groundwater and surface water flow at the site to permit construction in a dry and stable excavation. Water from either surface flow and/or groundwater must be diverted away from the excavation at all times. Groundwater perched within the embankment fill and, surface runoff will tend to seep into, and accumulate in proposed excavations. Temporary groundwater and surface water control measures will be required to remain operational during construction until the footings are installed and backfilled.

The design of any dewatering system that may be required is the responsibility of the Contractor. The Contract Documents must alert them to this responsibility and to design the system in accordance with SP No. FOUN0003 which amends OPSS 902. A preconstruction survey is not considered necessary thus Designer Fill-In ** in the SP should be "N/A".

In accordance with SP FOUN0003, the dewatering system is to be designed in accordance with OPSS.PROV 517 and SP 517F01; Amendment to OPSS 517, July 2017. For this project, it is recommended that Designer Fill-In **** in SP517F01 be "No". A preconstruction survey is not considered necessary, thus designer Fill-In ***** in this SP should be "N/A".

A contaminant/waste management assessment was carried out by others and should be consulted regarding environmental requirements for disposal of soil, rock or groundwater generated during the construction.

11.4 Erosion Protection

The erodibility of the soil within the existing embankment fills was assessed based on the gradation results and the Wischmeier nomograph. The results indicate that the material has a low susceptibility to erosion. It is noted however, that the steepness of slopes also plays a role in erosion and steep temporary excavation slopes may still be subject to erosion. The contractor should provide silt fences and erosion control blankets, as required, throughout the duration of the construction to prevent silt/sediments from running off the site as per OPSS 805.

Slope protection and drainage measures will be required to ensure the long-term surficial stability of the embankment slopes. A vegetation cover should be established on all other exposed earth surfaces to protect against surficial erosion in general accordance with OPSS.PROV 804.

11.5 Construction Concerns

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

- Significant bedrock excavation has been carried out at this site in the past for construction of the existing bridge piers and railway cut. The bedrock surface may be irregular and loose/fractured bedrock may be present in some locations. All bedrock surfaces must be inspected to ensure that new structures are founded on competent rock. In addition, the contract should include a unit rate for additional rock excavation and for placement of mass concrete.
- Confirmation that the granular backfill is adequately placed and compacted to specifications.
- The existing bridge is to remain in service during the construction and backfilling of the new bridge abutments. All excavation and backfilling activities must be carried out in a way that protects the existing structure and its foundations and ensures that the existing structure is not subjected to unbalanced loading for which it was not designed.

The successful performance of the construction of this structure will depend largely upon good workmanship and quality control during construction. Observation of the excavation and backfilling operations will be required as per MTO SP No. 109S12, amendment to OPSS 902 during construction to confirm that the foundation recommendations are correctly implemented and material specifications are met.

12 CLOSURE

Engineering analysis and preparation of this report was carried out by Mr. Paul Carnaffan, M.Eng., P.Eng. The report was reviewed by Dr. Fred Griffiths and Dr. P.K. Chatterji, P.Eng a Designated Principal Contact for MTO Foundation Projects.

Thurber Engineering Ltd.
Report Prepared By:



Paul Carnaffan, M.Eng., P.Eng.
Principal | Senior Geotechnical Engineer



Dr. Fred Griffiths, P.Eng.
Senior Associate
Senior Geotechnical Engineer

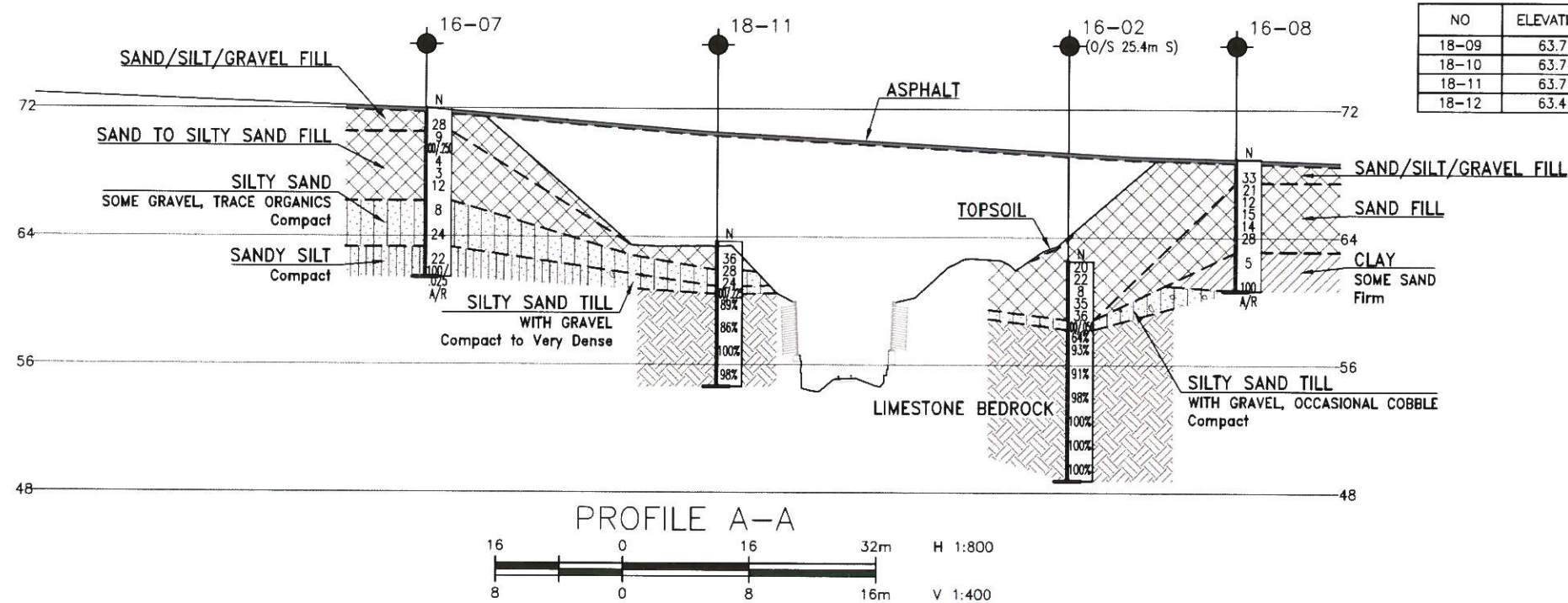


Dr. P.K. Chatterji, P.Eng.
Designated Principal Contact
Senior Geotechnical Engineer

h:\projects\10000 to 20000\11189 - hwy 417 o-train\reports & memos\final\bridge\3-54 tel_fidr final sept 26.docx

Appendix A.

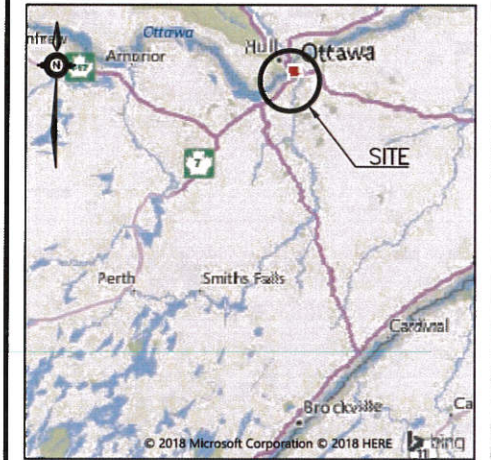
Borehole Location Plan and Stratigraphic Drawings



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

CONT No
GWP No 4245-05-00

HIGHWAY 417
CPR/O-TRAIN
OVERPASS
BOREHOLE LOCATIONS AND SOIL STRATA



KEYPLAN

LEGEND

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

NO	ELEVATION	NORTHING	EASTING
18-09	63.7	5 029 489.7	366 404.0
18-10	63.7	5 029 478.3	366 410.8
18-11	63.7	5 029 468.5	366 417.0
18-12	63.4	5 029 459.4	366 422.0

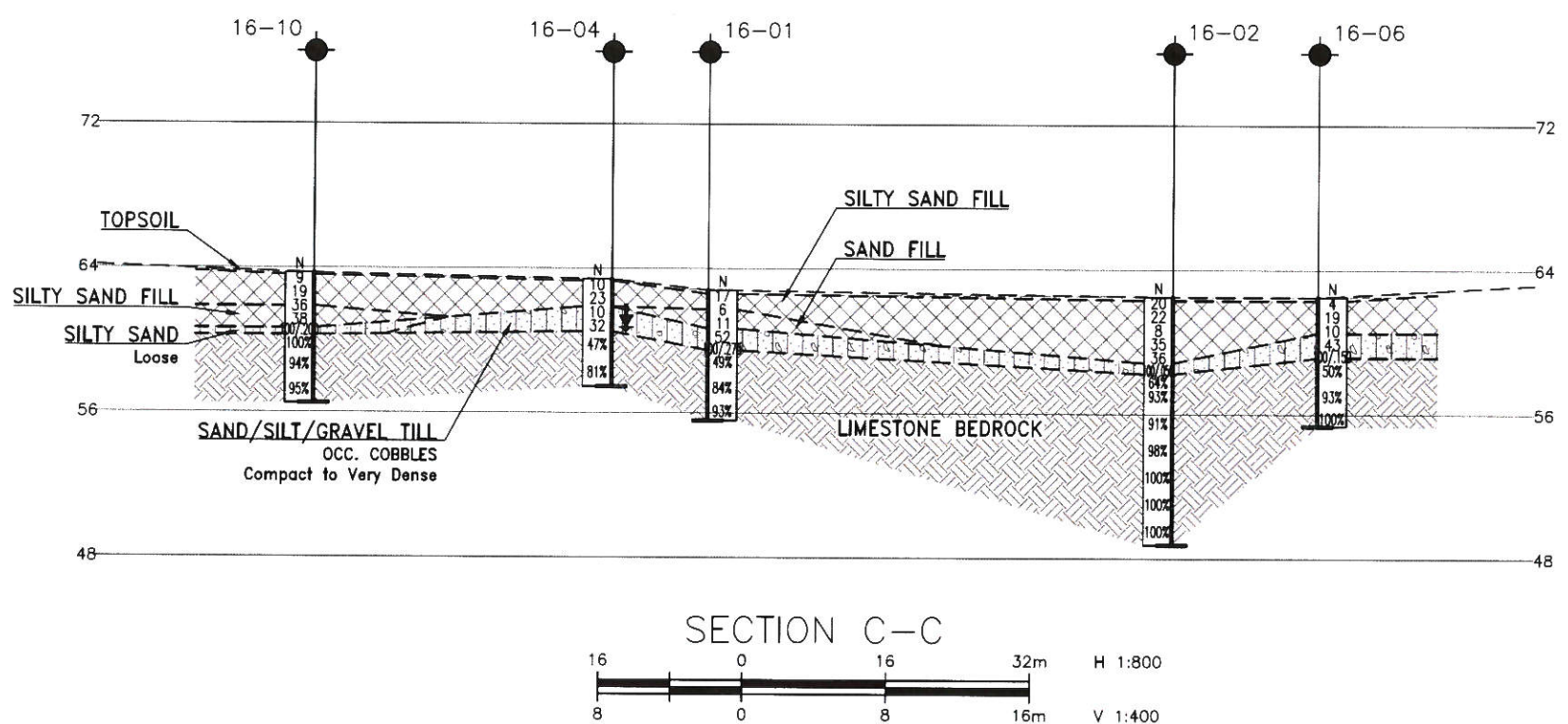
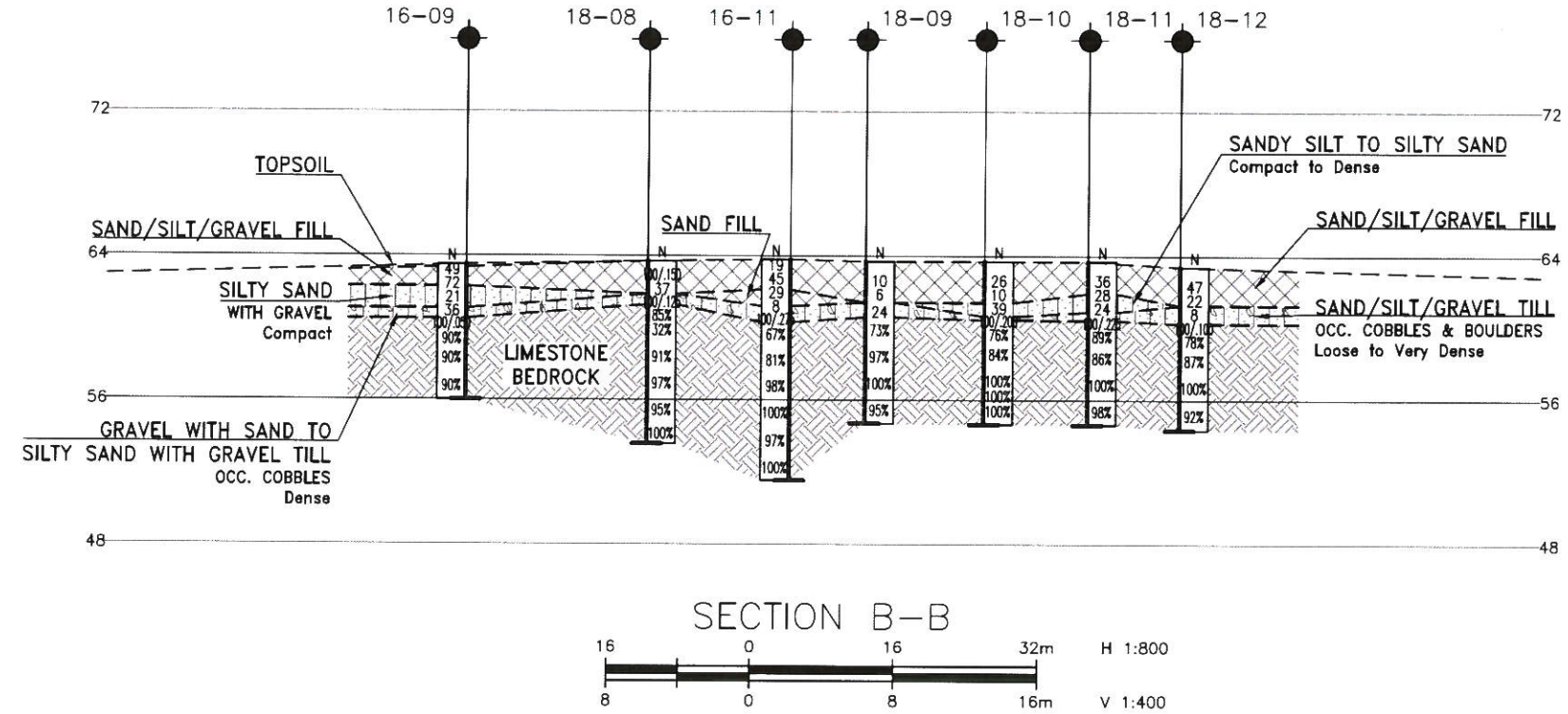
NO	ELEVATION	NORTHING	EASTING
16-01	62.8	5 029 507.6	366 437.9
16-02	62.5	5 029 463.7	366 465.5
16-03	66.5	5 029 487.8	366 363.4
16-04	63.4	5 029 516.9	366 432.6
16-05	63.8	5 029 458.8	366 420.7
16-06	62.5	5 029 453.8	366 480.2
16-07	72.0	5 029 458.7	366 381.5
16-08	68.9	5 029 491.6	366 478.1
16-09	63.5	5 029 527.1	366 379.9
16-10	63.7	5 029 544.4	366 413.9
16-11	63.8	5 029 497.5	366 400.8
18-08	63.7	5 029 510.9	366 392.1

NOTES

- The boundaries between soil strata have been established only at Borehole locations. Between Boreholes the boundaries are assumed from geological evidence.
- This drawing is for subsurface information only. Surface details and features are for conceptual illustration.
- Coordinate system is MTM NAD 83 Zone 9.

GEOCREs No. 31G5-293

REVISIONS	DATE	BY	DESCRIPTION
DESIGN	DJP	CHK PC	CODE
DRAWN	MFA	CHK DJP	SITE 3-54
			STRUCT
			DWG 2
			DATE SEP 2018



Appendix B.

**Record of Borehole Sheets
Rock Core Photographs**



SYMBOLS, ABBREVIATIONS AND TERMS USED ON TEST HOLE RECORDS

TERMINOLOGY DESCRIBING COMMON SOIL GENESIS

Topsoil	mixture of soil and humus capable of supporting vegetative growth
Peat	mixture of fragments of decayed organic matter
Till	unstratified glacial deposit which may include particles ranging in sizes from clay to boulder
Fill	material below the surface identified as placed by humans (excluding buried services)

TERMINOLOGY DESCRIBING SOIL STRUCTURE:

Desiccated	having visible signs of weathering by oxidization of clay materials, shrinkage cracks, etc.
Fissured	having cracks, and hence a blocky structure
Varved	composed of alternating layers of silt and clay
Stratified	composed of alternating successions of different soil types, e.g. silt and sand
Layer	> 75 mm in thickness
Seam	2 mm to 75 mm in thickness
Parting	< 2 mm in thickness

RECOVERY:

For soil samples, the recovery is recorded as the length of the soil sample recovered.

N-VALUE:

Numbers in this column are the field results of the Standard Penetration Test: the number of blows of a 63.5 kg hammer falling 0.76 m, required to drive a 50 mm O.D. split spoon sampler 0.3 m into undisturbed soil. For samples where insufficient penetration was achieved and N-value cannot be presented, the number of blows are reported over the sampler penetration in millimetres (e.g. 50/75).

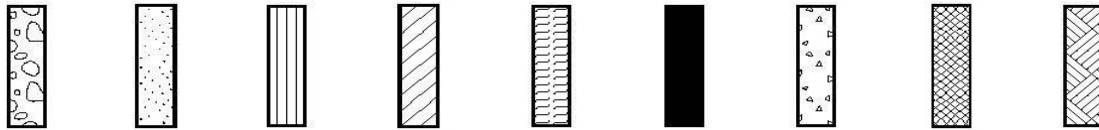
DYNAMIC CONE PENETRATION TEST (DCPT):

Dynamic cone penetration tests are performed using a standard 60 degree apex cone connected to an "A" size drill rods with the same standard fall height and weight as the Standard Penetration Test. The DCPT value is the number of blows of the hammer required to drive the cone 0.3 m into the soil. The DCPT is used as a probe to assess soil variability.



STRATA PLOT:

Strata plots symbolize the soil and bedrock description. They are combinations of the following basic symbols. The dimensions within the strata symbols are not indicative of the particle size, layer thickness, etc.



Boulders
Cobbles
Gravel Sand Silt Clay Organics Asphalt Concrete Fill Bedrock

TEXTURING CLASSIFICATION OF SOILS

Classification	Particle Size
Boulders	Greater than 200 mm
Cobbles	75 – 200 mm
Gravel	4.75 – 75 mm
Sand	0.075 – 4.75 mm
Silt	0.002 – 0.075 mm
Clay	Less than 0.002 mm

TERMS DESCRIBING CONSISTENCY (COHESIVE SOILS ONLY)

Descriptive Term	Undrained Shear Strength (kPa)
Very Soft	12 or less
Soft	12 – 25
Firm	25 – 50
Stiff	50 – 100
Very Stiff	100 – 200
Hard	Greater than 200

NOTE: Clay sensitivity is defined as the ratio of the undisturbed strength over the remolded strength.

SAMPLE TYPES

SS	Split spoon samples
ST	Shelby tube or thin wall tube
DP	Direct push sample
PS	Piston sample
BS	Bulk sample
WS	Wash sample
HQ, NQ, BQ etc.	Rock core sample obtained with the use of standard size diamond coring equipment

TERMS DESCRIBING CONSISTENCY (COHESIONLESS SOILS ONLY)

Descriptive Term	SPT "N" Value
Very Loose	Less than 4
Loose	4 – 10
Compact	10 – 30
Dense	30 – 50
Very Dense	Greater than 50

MODIFIED UNIFIED SOIL CLASSIFICATION

Major Divisions		Group Symbol	Typical Description
COARSE GRAINED SOIL	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	SILT AND CLAY SOILS $W_L < 35\%$	ML	Inorganic silts, 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.
		OL	Organic silts and organic silty-clays of low plasticity.
	SILT AND CLAY SOILS $35\% < W_L < 50\%$	MI	Inorganic compressible fine sandy silt with clay of medium plasticity, clayey silts.
		CI	Inorganic clays of medium plasticity, silty clays.
		OI	Organic silty clays of medium plasticity.
	SILT AND CLAY SOILS $W_L > 50\%$	MH	Inorganic silts, micaceous or diatomaceous fine sandy of silty soils, elastic silts.
		CH	Inorganic clays of high plasticity, fat clays.
		OH	Organic clays of high plasticity, organic silts.
HIGHLY ORGANIC SOILS		Pt	Peat and other organic soils.

Note - W_L = Liquid Limit



EXPLANATION OF ROCK LOGGING TERMS

ROCK WEATHERING CLASSIFICATION

Fresh (FR)	No visible signs of weathering.
Fresh Jointed (FJ)	Weathering limited to surface of major discontinuities.
Slightly Weathered (SW)	Penetrative weathering developed on open discontinuity surfaces, but only slight weathering of rock materials.
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 structures are preserved.

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 0.1 m in length or larger, as a percentage of total core length
Unconfined Compressive Strength: (UCS)	Axial stress required to break the specimen.
Fracture Index: (FI)	Frequency of natural fractures per 0.3 m of core run.

DISCONTINUITY SPACING

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

STRENGTH CLASSIFICATION

Rock Strength	Approximate Uniaxial Compressive Strength (MPa)
Extremely Strong	Greater than 250
Very Strong	100 – 250
Strong	50 – 100
Medium Strong	25 – 50
Weak	5 – 25
Very Weak	1 – 5
Extremely Weak	0.25 – 1

RECORD OF BOREHOLE No 16-01

1 OF 1

METRIC

GWP# 4245-05-00 LOCATION O-Train Bridge, MTM z9: N 5 029 507.6 E 366 437.9 ORIGINATED BY JG
 HWY 417 BOREHOLE TYPE HSA/NW Coring COMPILED BY JG
 DATUM Geodetic DATE 2017.04.12 - 2017.04.12 CHECKED BY FJG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC NATURAL LIQUID LIMIT MOISTURE CONTENT			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	○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE			W P W L				
62.8																
0.0	TOPSOIL (200 mm)															
0.2	SILTY SAND with gravel FILL Compact to Loose Dark Brown		1	SS	17											
61.7							62									
1.1	SAND with silt FILL Loose to Compact Brown		2	SS	6											
60.7							61								6 86 8 (SI+CL)	
2.1	GRAVEL (GW-GM) with sand and silt, occasional cobbles TILL Very Dense Brown		3	SS	11											
60.7							60								51 38 11 (SI+CL)	
59.5			4	SS	52											
59.5							60									
3.3	LIMESTONE BEDROCK with some shale interbedding Grey Slightly Weathered to Fresh Very Thinly Bedded		5	SS	100/ 275 mm									FI		
59.5							59							10	RUN #1 TCR=100% SCR=73% RQD=49%	
59.5			1	RUN										3		
59.5							58							2		
59.5							58							1	RUN #2 TCR=100% SCR=100% RQD=84%	
59.5			2	RUN										3		
59.5							57							2		
59.5							57							1		
59.5			3	RUN										0	RUN #3 TCR=100% SCR=100% RQD=93% UCS=123MPa	
55.6							56							1		
55.6							56							0		
7.2	End of Borehole Borehole open and dry prior to coring															

+³, ×³: Numbers refer to Sensitivity 20 15 10 5 0 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 16-02

1 OF 2

METRIC

GWP# 4245-05-00 LOCATION O-Train Bridge, MTM z9: N 5 029 463.7 E 366 465.5 ORIGINATED BY JG
 HWY 417 BOREHOLE TYPE HSA/NW Coring COMPILED BY JG
 DATUM Geodetic DATE 2017.04.12 - 2017.04.12 CHECKED BY FJG

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				W P W W L								
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE				WATER CONTENT (%)								
62.5							20	40	60	80	100						GR	SA	SI	CL
0.0	TOPSOIL (180 mm)																			
0.2	SAND with silt and gravel to SILTY SAND with gravel FILL Loose to dense Brown		1	SS	20								○							
			2	SS	22								○							41 50 9 (SI+CL)
			3	SS	8								○							
			4	SS	35								○							40 41 19 (SI+CL)
			5	SS	36								○							
58.8																				
3.7	SILTY SAND with gravel, occasional cobble TILL Very Dense Grey		6	SS	100/ 50 mm								○							
58.2																				
4.3	LIMESTONE BEDROCK with some shale interbedding Grey Fresh Very Thinly Bedded		1	RUN																
			2	RUN																
			3	RUN																
			4	RUN																
</																				

Continued Next Page

+³, ×³: Numbers refer to
Sensitivity

20
15
10


(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 16-02

2 OF 2

METRIC

GWP# 4245-05-00 LOCATION O-Train Bridge, MTM z9: N 5 029 463.7 E 366 465.5 ORIGINATED BY JG
 HWY 417 BOREHOLE TYPE HSA/NW Coring COMPILED BY JG
 DATUM Geodetic DATE 2017.04.12 - 2017.04.12 CHECKED BY FJG

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								
								20 40 60 80 100								
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE								
Continued From Previous Page							20 40 60 80 100				20 40 60					
48.8	LIMESTONE BEDROCK with some shale interbedding Grey Fresh Very Thinly Bedded		5	RUN			52								1	RUN #5 TCR=100% SCR=100% RQD=100%
			6	RUN			51								1	RUN #6 TCR=100% SCR=100% RQD=100% UCS=138MPa
													1			
													1			
			7	RUN			50								0	RUN #7 TCR=100% SCR=100% RQD=100%
													0			
									49							
13.7	End of Borehole Borehole open and dry prior to coring															

RECORD OF BOREHOLE No 16-03

1 OF 1

METRIC

GWP# 4245-05-00 LOCATION O-Train Bridge, MTM z9: N 5 029 487.8 E 366 363.4 ORIGINATED BY JG
 HWY 417 BOREHOLE TYPE HSA/NW Coring COMPILED BY JG
 DATUM Geodetic DATE 2017.04.11 - 2017.04.11 CHECKED BY FJG

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								
								20 40 60 80 100								
						WATER CONTENT (%)				20 40 60						
66.5																
0.0	ASPHALT 100 mm															
0.1	GRAVEL with sand and silt FILL Loose to Compact Brown		1	SS	8		66									
			2	SS	25											54 36 10 (SI+CL)
			3	SS	14		65									
64.5	CLAY (CL) some sand, trace rootlets Soft Grey		4	SS	3		64									3 21 42 34
2.0			5	SS	18		63									
63.6	SILTY SAND (SM) Compact Brown to Grey		6	SS	17		62									0 70 30 (SI+CL)
2.9			7	SS	14		61									
61.5	SILTY SAND with gravel TILL						60									
60.9	LIMESTONE BEDROCK with some shale interbedding Grey Fresh Very Thinly Bedded		1	RUN			59									RUN #1 TCR=100% SCR=100% RQD=79%
5.2			2	RUN			58									RUN #2 TCR=100% SCR=100% RQD=97% UCS=128MPa
			3	RUN												RUN #3 TCR=100% SCR=100% RQD=96%
57.3																
9.2	End of Borehole															

ONTMT4S 11189 - HWY 417 O-TRAIN GPJ 2012TEMPLATE(MTO).GDT 19/6/18

+³, ×³: Numbers refer to
Sensitivity

20
15
10

(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 16-04

1 OF 1

METRIC

GWP# 4245-05-00 LOCATION O-Train Bridge, MTM z9: N 5 029 516.9 E 366 432.6 ORIGINATED BY SOB
 HWY 417 BOREHOLE TYPE HSA / NQ Coring COMPILED BY SOB
 DATUM Geodetic DATE 2018.03.07 - 2018.03.07 CHECKED BY PC

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						
63.4														
0.0														
0.1	TOPSOIL (100 mm)													
	SILTY SAND some gravel FILL Compact Brown		1	SS	10		63							
			2	SS	23		62							
61.8														
1.5	GRAVEL (GP-GM), silty with sand TILL Compact to Dense Brown		3	SS	10		61							44 43 13 (SI+CL)
			4	SS	32									
60.5														
2.9	LIMESTONE BEDROCK Slightly weathered to fresh Very thinly bedded with shale seams Strong Grey		1	RUN			60							RUN #1 TCR=100% SCR=60% RQD=47%
							59							
			2	RUN			58							RUN #2 TCR=100% SCR=95% RQD=81%
57.4														
5.9	End of Borehole Borehole open and dry prior to coring WATER LEVEL READINGS: DATE DEPTH (m) ELEV. (m) 2018.03.12 2.7 60.7 2018.03.23 2.8 60.8													

ONTMT4S 11189 - HWY 417 O-TRAIN GPJ 2012TEMPLATE(MTO).GDT 19/6/18

RECORD OF BOREHOLE No 16-05

1 OF 1

METRIC

GWP# 4245-05-00 LOCATION O-Train Bridge, MTM z9: N 5 029 458.8 E 366 420.7 ORIGINATED BY JG
 HWY 417 BOREHOLE TYPE HSA/NW Coring COMPILED BY JG
 DATUM Geodetic DATE 2017.04.11 - 2017.04.11 CHECKED BY FJG

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					W P W W L																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																	
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63.8								20	40	60	80	100																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																		

+³, ×³: Numbers refer to
Sensitivity

20
15
10

(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 16-06

1 OF 1

METRIC

GWP# 4245-05-00 LOCATION O-Train Bridge, MTM z9: N 5 029 453.8 E 366 480.2 ORIGINATED BY JG
 HWY 417 BOREHOLE TYPE HSA/NW Coring COMPILED BY JG
 DATUM Geodetic DATE 2017.04.11 - 2017.04.11 CHECKED BY FJG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _P	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
								20 40 60 80 100									
62.5																	
0.0	TOPSOIL (180 mm)																
0.2	SILTY SAND with gravel FILL Loose to Compact Brown		1	SS	4		62							o			33 49 18 (SI+CL)
			2	SS	19									o			
							61										
60.5	- trace wood/ organics		3	SS	10									o			
2.0	SILTY SAND (SM) with gravel, occasional cobble TILL Dense Brown																
			4	SS	43		60							o			38 45 17 (SI+CL)
			5	SS	100/ 150 mm									o			
59.1																	
3.4	LIMESTONE BEDROCK with some shale interbedding Grey Fresh Very Thinly Bedded		1	RUN			59									FI	
																10	
																10	RUN #1 TCR=78% SCR=57% RQD=50%
																3	
							58									1	
																1	
			2	RUN			57									3	RUN #2 TCR=97% SCR=97% RQD=93% UCS=143MPa
																1	
																1	
																0	RUN #3 TCR=100% SCR=100% RQD=100%
55.3			3	RUN			56									3	
7.2	End of Borehole Borehole open and dry prior to coring															2	

+³, ×³: Numbers refer to Sensitivity 20 15 10 5 0 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 16-07

1 OF 2

METRIC

GWP# 4245-05-00 LOCATION Highway 417, West Approach MTM Zone 9: N 5 029 458.7 E 366 381.5 ORIGINATED BY JG
HWY 417 BOREHOLE TYPE HSA COMPILED BY JSM
DATUM Geodetic DATE 2017.03.28 - 2017.03.28 CHECKED BY FJG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
								20 40 60 80 100						
								20 40 60 80 100						
72.0														
0.0	ASPHALT 250 mm													
0.2	GRAVEL with sand and silt to SAND with gravel and silt FILL Compact Brown		1	AS									49 41 10 (SH+CL)	
			2	SS	28		71							
70.5														
1.5	SAND FILL Compact to very loose Brown		3	SS	9		70							
			4	SS	100 / 250 mm									
	- cobble						69							
			5	SS	4									
			6	SS	3		68						1 96 3 (SH+CL)	
			7	SS	12		67							
66.2							66							
5.8	SILTY SAND (SM) some gravel, trace organics Loose to compact Brown		8	SS	8								12 51 37 (SH+CL)	
							65							
			9	SS	24		64							
63.3														
8.7	Sandy SILT (ML) Compact Brown						63							
			10	SS	22								1 44 55 (SH+CL)	

Continued Next Page

+³, ×³: Numbers refer to
Sensitivity

20
15
10

(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 16-07

2 OF 2

METRIC

GWP# 4245-05-00 LOCATION Highway 417, West Approach MTM Zone 9: N 5 029 458.7 E 366 381.5 ORIGINATED BY JG
 HWY 417 BOREHOLE TYPE HSA COMPILED BY JSM
 DATUM Geodetic DATE 2017.03.28 - 2017.03.28 CHECKED BY FJG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)				
						20 40 60 80 100 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE					W P W W L 20 40 60						
	Continued From Previous Page																
61.5	Sandy SILT (ML) Compact Brown		11	SS	100												
10.5	End of Borehole Auger refusal on inferred bedrock Water level measured in open hole at 9.4 m BGS When pulling out augers, hole caved at 4.6 m BGS				25mm												

ONTMT4S 11189 - HWY 417 O-TRAIN GPJ 2012TEMPLATE(MTO).GDT 19/6/18

RECORD OF BOREHOLE No 16-08

1 OF 1

METRIC

GWP# 4245-05-00 LOCATION Highway 417, East Approach MTM Zone 9: N 5 029 491.6 E 366 478.1 ORIGINATED BY JG
HWY 417 BOREHOLE TYPE HSA COMPILED BY JSM
DATUM Geodetic DATE 2017.03.29 - 2017.03.29 CHECKED BY FJG

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				W P W W L																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																					
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ONTMT4S 11189 - HWY 417 O-TRAIN GPJ 2012TEMPLATE(MTO),GDT 19/6/18

+³, ×³: Numbers refer to
Sensitivity

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(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 16-09

1 OF 1

METRIC

GWP# 4245-05-00 LOCATION O-Train Bridge, MTM z9: N 5 029 527.1 E 366 379.9 ORIGINATED BY JG
 HWY 417 BOREHOLE TYPE HSA/NW Coring COMPILED BY JG
 DATUM Geodetic DATE 2017.04.10 - 2017.04.11 CHECKED BY FJG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
								20 40 60 80 100						
								20 40 60 80 100						
63.5														
0.0	TOPSOIL (150 mm)													
0.2	GRAVEL with sand and silt, frequent cobbles FILL Dense to Very Dense Grey		1	SS	49		63							
62.3			2	SS	72									63 30 7 (SI+CL)
1.2	SILTY SAND with gravel Compact Brown		3	SS	21		62							
61.1														
2.4	GRAVEL (GM) with sand and silt to SILTY SAND with gravel, occasional cobble, TILL Dense Grey		4	SS	36		61							42 41 17 (SI+CL)
60.5			5	SS	100/									
3.0	LIMESTONE BEDROCK with some shale interbedding Grey Slightly Weathered to Fresh Very Thinly Bedded		1	RUN	50 mm		60							RUN #1 TCR=98% SCR=97% RQD=90%
			2	RUN			59							RUN #2 TCR=100% SCR=100% RQD=90%
			3	RUN			58							RUN #3 TCR=100% SCR=100% RQD=90%
56.0							57							
7.5	End of Borehole Borehole open and dry prior to coring Overnight water level in open borehole at 2.4 m													

ONTMT4S 11189 - HWY 417 O-TRAIN GPJ 2012TEMPLATE(MTO).GDT 19/6/18

RECORD OF BOREHOLE No 16-10

1 OF 1

METRIC

GWP# 4245-05-00 LOCATION O-Train Bridge, MTM z9: N 5 029 544.4 E 366 413.9 ORIGINATED BY SOB
 HWY 417 BOREHOLE TYPE HSA / NQ Coring COMPILED BY SOB
 DATUM Geodetic DATE 2018.03.07 - 2018.03.07 CHECKED BY PC

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																						
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ONTMT4S 11189 - HWY 417 O-TRAIN GPJ 2012TEMPLATE(MTO).GDT 19/6/18





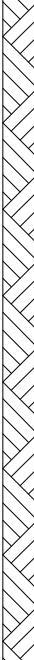
+³, ×³: Numbers refer to Sensitivity
 20
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(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 16-11

1 OF 2

METRIC

GWP# 4245-05-00 LOCATION O-Train Bridge, MTM z9: N 5 029 497.5 E 366 400.8 ORIGINATED BY JG
 HWY 417 BOREHOLE TYPE HSA/NW Coring COMPILED BY JG
 DATUM Geodetic DATE 2017.04.10 - 2017.04.10 CHECKED BY FJG

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				W P			W			W L						
63.8								20	40	60	80	100												
0.0	SILTY SAND with gravel, occasional cobble FILL Compact to Dense Brown to Black		1	SS	19																			
	- trace wood		2	SS	45																			
62.1																								
1.7	SAND with silt FILL Compact to Loose Brown		3	SS	29																			
61.2																								
2.6	Sandy SILT (ML) some gravel to SILTY SAND with gravel, occasional cobble, TILL Loose to Very Dense Grey		4	SS	8																			
				5	SS	100/ 275 mm																		
60.3																								
3.5	LIMESTONE BEDROCK with some shale interbedding Grey Fresh Very Thinly Bedded		1	RUN																				
				2	RUN																			
			3	RUN																				

Continued Next Page

+³, ×³: Numbers refer to
Sensitivity

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(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 16-11

2 OF 2

METRIC

GWP# 4245-05-00 LOCATION O-Train Bridge, MTM z9: N 5 029 497.5 E 366 400.8 ORIGINATED BY JG
 HWY 417 BOREHOLE TYPE HSA/NW Coring COMPILED BY JG
 DATUM Geodetic DATE 2017.04.10 - 2017.04.10 CHECKED BY FJG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)				
							20	40	60	80	100	W _p	W	W _L			
	Continued From Previous Page		5	RUN													
	LIMESTONE BEDROCK with some shale interbedding Grey Fresh Very Thinly Bedded					53											
			6	RUN													
						52											
51.5																	
12.3	End of Borehole Borehole open and dry prior to coring Overnight water level in open borehole at 2.3 m																

ONTMT4S 11189 - HWY 417 O-TRAIN GPJ 2012TEMPLATE(MTO).GDT 19/6/18

RECORD OF BOREHOLE No 18-08

1 OF 2

METRIC

GWP# 4245-05-00 LOCATION O-Train Bridge, MTM z9: N 5 029 510.9 E 366 392.1 ORIGINATED BY KE
 HWY 417 BOREHOLE TYPE HSA / NQ Coring COMPILED BY CM
 DATUM Geodetic DATE 2018.02.26 - 2018.02.26 CHECKED BY PC

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								
							20	40	60	80	100	20	40	60		
63.7																
0.0	GRAVEL, silty with sand FILL Frequent cobbles and boulders, trace wood Very dense to dense Grey-black to brown		1	SS	100/ 150mm											
61.9			2	SS	37											42 40 18 (SI+CL)
1.8	SAND (SP-SM) with silt TILL Occasional cobbles Dense to very dense Brown		3	SS	100/ 125mm											0 94 6 (SI+CL)
61.3			1	RUN												RUN #1 TCR=100% SCR=92% RQD=85% UCS=113MPa
2.4	LIMESTONE BEDROCK Slightly Weathered to Fresh Occasional Shale Seams Very Strong Grey		2	RUN												RUN #2 TCR=100% SCR=56% RQD=32%
			3	RUN												RUN #3 TCR=100% SCR=100% RQD=91%
			4	RUN												RUN #4 TCR=100% SCR=98% RQD=97% UCS=101MPa
			5	RUN												RUN #5 TCR=100% SCR=100% RQD=95%
			6	RUN												RUN #6 TCR=100% SCR=100% RQD=100%

Continued Next Page


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

RECORD OF BOREHOLE No 18-08

2 OF 2

METRIC

GWP# 4245-05-00 LOCATION O-Train Bridge, MTM z9: N 5 029 510.9 E 366 392.1 ORIGINATED BY KE
 HWY 417 BOREHOLE TYPE HSA / NQ Coring COMPILED BY CM
 DATUM Geodetic DATE 2018.02.26 - 2018.02.26 CHECKED BY PC

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									
53.6	Continued From Previous Page																
10.1	End of Borehole																

ONTMT4S 11189 - HWY 417 O-TRAIN GPJ 2012TEMPLATE(MTO).GDT 19/6/18

RECORD OF BOREHOLE No 18-09

1 OF 1

METRIC

GWP# 4245-05-00 LOCATION O-Train Bridge, MTM z9: N 5 029 489.7 E 366 404.0 ORIGINATED BY KE
 HWY 417 BOREHOLE TYPE NW Casing / NQ Coring COMPILED BY SOB
 DATUM Geodetic DATE 2018.03.01 - 2018.03.04 CHECKED BY PC

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 (%)				
												W P W W L				
63.7							20 40 60 80 100									
0.0	GRAVEL with silt and sand FILL Occasional cobbles and boulders Loose to compact Brown															
			1	SS	10										48 42 10 (SI+CL)	
			2	SS	6											
61.4																
2.3	SILTY SAND (SM) with gravel TILL Occasional cobbles and boulders Compact Grey															
			3	SS	24											
60.6																
3.1	LIMESTONE BEDROCK Slightly Weathered to Fresh With Shale Partings Very Strong Grey															
			1	RUN												
			2	RUN												

+³, ×³: Numbers refer to Sensitivity 20 15 10 5 0 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 18-10

1 OF 1

METRIC

GWP# 4245-05-00 LOCATION O-Train Bridge, MTM z9: N 5 029 478.3 E 366 410.8 ORIGINATED BY KE
 HWY 417 BOREHOLE TYPE NW Casing / NQ Coring COMPILED BY SOB
 DATUM Geodetic DATE 2018.03.01 - 2018.03.01 CHECKED BY PC

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
								20 40 60 80 100						
								20 40 60 80 100						
63.7						○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE	WATER CONTENT (%) W P W W L PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT							
0.0	SILTY SAND with gravel FILL Occasional cobbles and boulders Compact Brown													
			1	SS	26									30 54 16 (SI+CL)
			2	SS	10									
61.4														
2.3	Sandy SILT (ML) Dense Brown-grey		3	SS	39									3 47 50 (SI+CL)
60.7														
3.0	SILTY SAND (SM) with gravel TILL Very Dense Grey		4	SS100/200mm										
3.3	LIMESTONE BEDROCK Slightly Weathered to Fresh With Shale Partings Strong Grey		1	RUN										RUN #1 TCR=100% SCR=80% RQD=76%
			2	RUN										RUN #2 TCR=100% SCR=100% RQD=84% UCS=118MPa
			3	RUN										RUN #3 TCR=100% SCR=100% RQD=100%
			4	RUN										RUN #4 TCR=100% SCR=100% RQD=100% UCS=116MPa
			5	RUN										RUN #5 TCR=100% SCR=100% RQD=100%
54.7														
9.0	End of Borehole													

ONTMT4S 11189 - HWY 417 O-TRAIN GPJ 2012TEMPLATE(MTO).GDT 19/6/18

+³, ×³: Numbers refer to
Sensitivity

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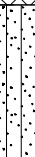


(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 18-11

1 OF 1

METRIC

GWP# 4245-05-00 LOCATION O-Train Bridge, MTM z9: N 5 029 468.5 E 366 417.0 ORIGINATED BY KE
 HWY 417 BOREHOLE TYPE NW Casing / NQ Coring COMPILED BY SOB
 DATUM Geodetic DATE 2018.02.27 - 2018.02.28 CHECKED BY PC

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	20	40	60	80	100	W _p			W	W _L
								● QUICK TRIAXIAL × LAB VANE	20	40	60	80	100					
63.7																		
0.0	SILTY SAND some gravel FILL Occasional cobbles and boulders Dense Grey		1	SS	36								○					
61.9																		
1.8	SILTY SAND (SM) Compact Brown to Grey		2	SS	28								○					
			3	SS	24								○					
60.9																		
2.8	SILTY SAND (SM) with gravel TILL Compact to Very Dense Grey		4	SS100/225mm									○					
60.4																		
3.3	LIMESTONE BEDROCK Fresh With Shale Laminations Very Strong Grey		1	RUN														
			2	RUN														
			3	RUN														
			4	RUN														

ONTMT4S 11189 - HWY 417 O-TRAIN GPJ 2012TEMPLATE(MTO).GDT 19/6/18

+³, ×³: Numbers refer to
Sensitivity

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(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 18-12

1 OF 1

METRIC

GWP# 4245-05-00 LOCATION O-Train Bridge, MTM z9: N 5 029 459.4 E 366 422.0 ORIGINATED BY JG / KE
 HWY 417 BOREHOLE TYPE HSA / NQ Coring COMPILED BY CM
 DATUM Geodetic DATE 2018.02.20 - 2018.02.22 CHECKED BY PC

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT W _P	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									WATER CONTENT (%)
								20	40	60	80	100					
								○ UNCONFINED									
								● QUICK TRIAXIAL									
									+	FIELD VANE							
									×	LAB VANE							
								20	40	60	80	100	20	40	60		
63.4																	
0.0	SILTY SAND with gravel FILL frequent Cobbles Very Dense Brown						63										
62.8																	
0.6	SILTY SAND, some gravel FILL Compact to Dense Brown		1	SS	47		62										
			2	SS	22												
61.3																	
2.1	SILTY SAND (SM) with gravel TILL Loose to Very Dense Grey		3	SS	8		61										
			4	SS	100/100mm												
60.3																	
3.1	LIMESTONE BEDROCK Slightly Weathered to Fresh Occasional Shale Seams Strong to Very Strong Grey		1	RUN			60										
			2	RUN			59										
			3	RUN			58										

+³, ×³: Numbers refer to Sensitivity
 20
15
10
5
0
5
10
(%) STRAIN AT FAILURE

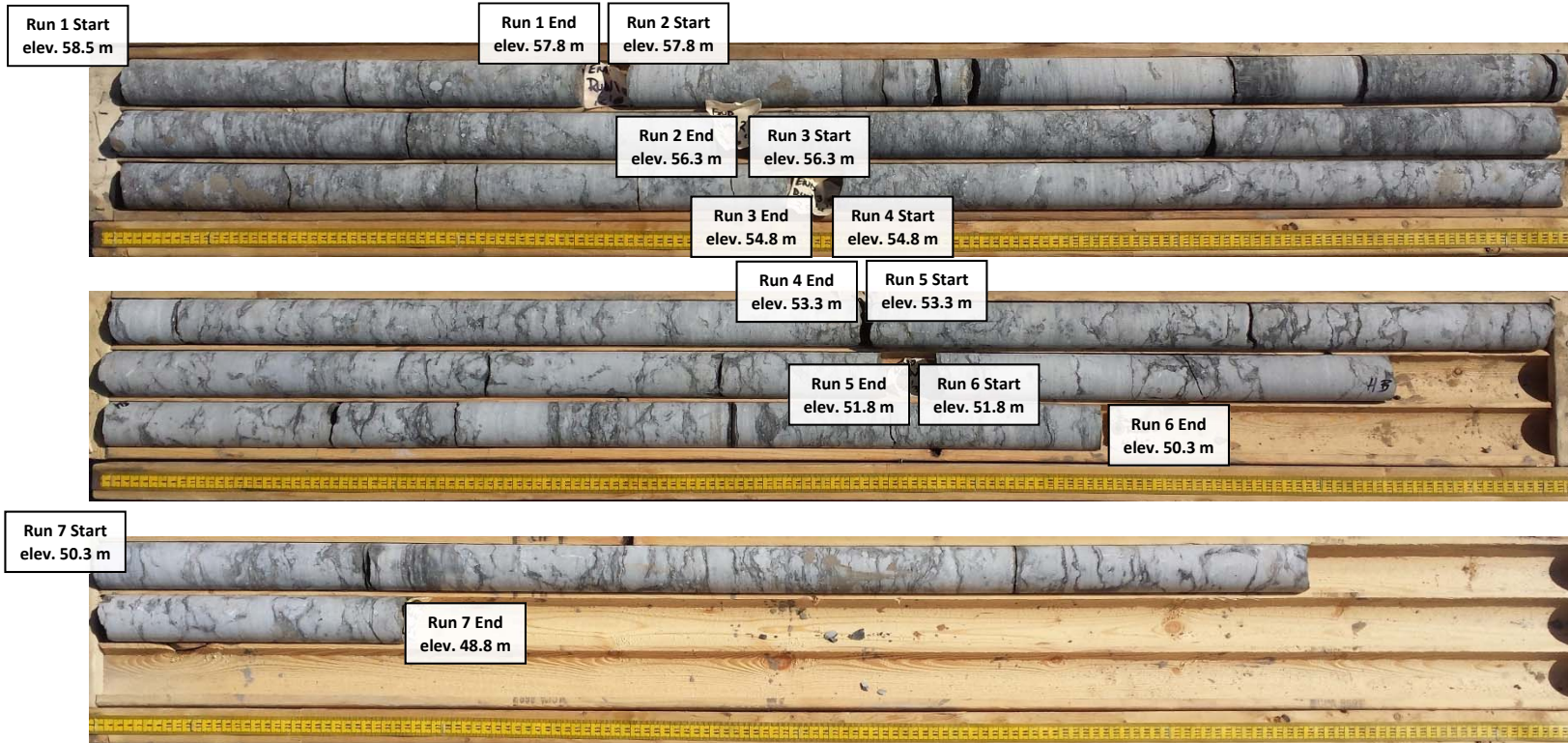
Borehole 16-1
Run 1 to 3 (of 3)
Elevation 59.5 m to 55.6 m



Borehole 16-2

Run 1 to 7 (of 7)

Elevation 58.5 m to 48.8 m



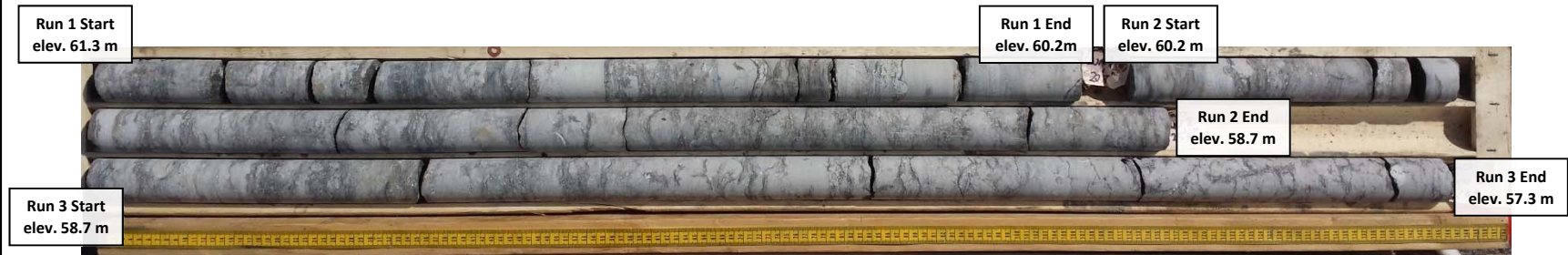
THURBER ENGINEERING LTD.

Foundation Investigation
Replacement of CPR/O-Train Bridges,
Highway 417 Ottawa, Ontario

GWP 4245-05-00

Project No.: 11189

Borehole 16-3
Run 1 to 3 (of 3)
Elevation 61.3 m to 57.3 m



Borehole 16-4
Run 1 to 2 (of 2)
Elevation 60.5 m to 57.4 m



Borehole 16-5
Run 1 to 3 (of 3)
Elevation 60.9.0 m to 57.6 m



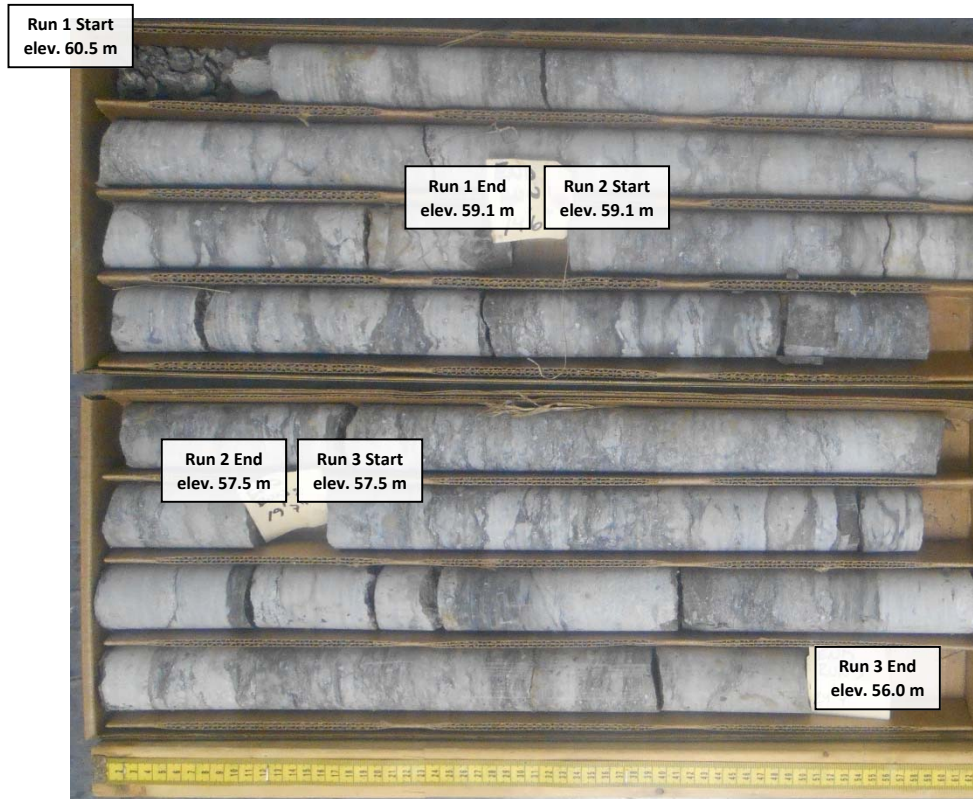
Borehole 16-6
Run 1 to 3 (of 3)
Elevation 59.1 m to 55.3 m



16-9

Run 1 to 3 (of 3)

Elevation 60.5 m to 56.0 m



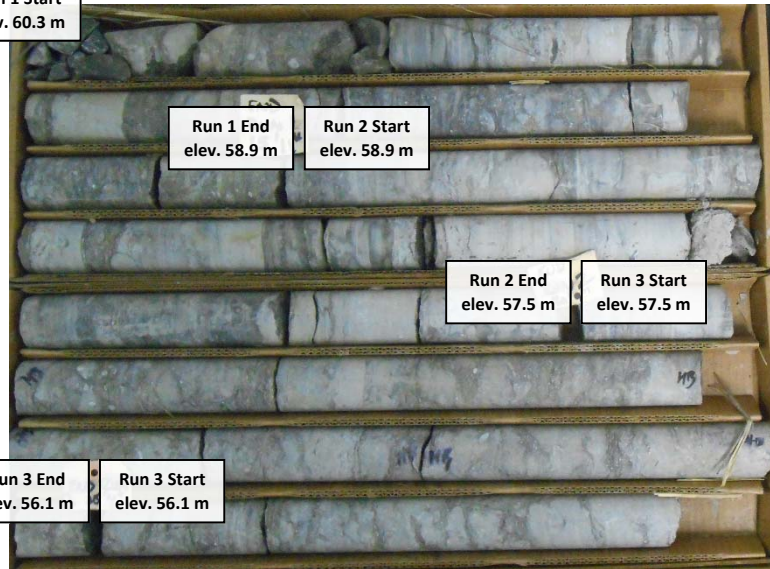
Borehole 16-10
Run 1 to 3 (of 3)
Elevation 60.3 m to 56.5 m



16-11

Run 1 to 6 (of 6)
Elevation 60.3 m to 51.5 m

Run 1 Start
elev. 60.3 m



Run 1 End
elev. 58.9 m

Run 2 Start
elev. 58.9 m

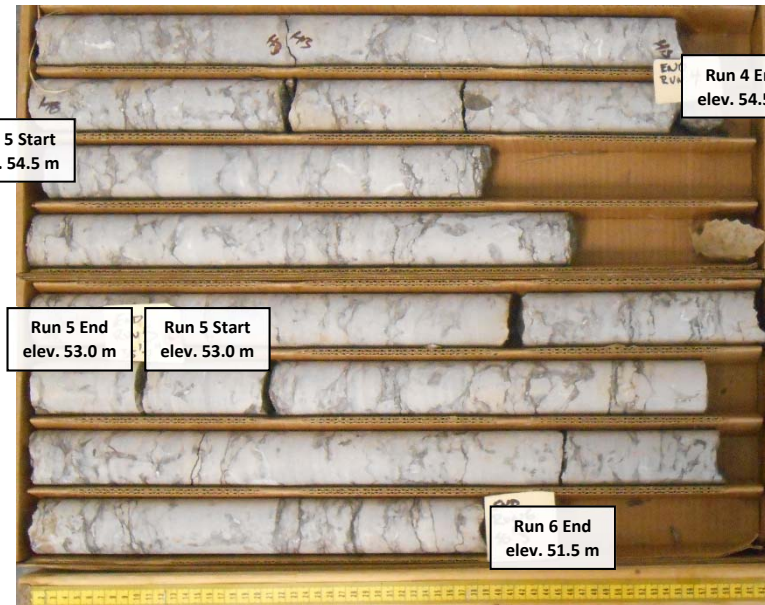
Run 2 End
elev. 57.5 m

Run 3 Start
elev. 57.5 m

Run 3 End
elev. 56.1 m

Run 3 Start
elev. 56.1 m

Run 5 Start
elev. 54.5 m



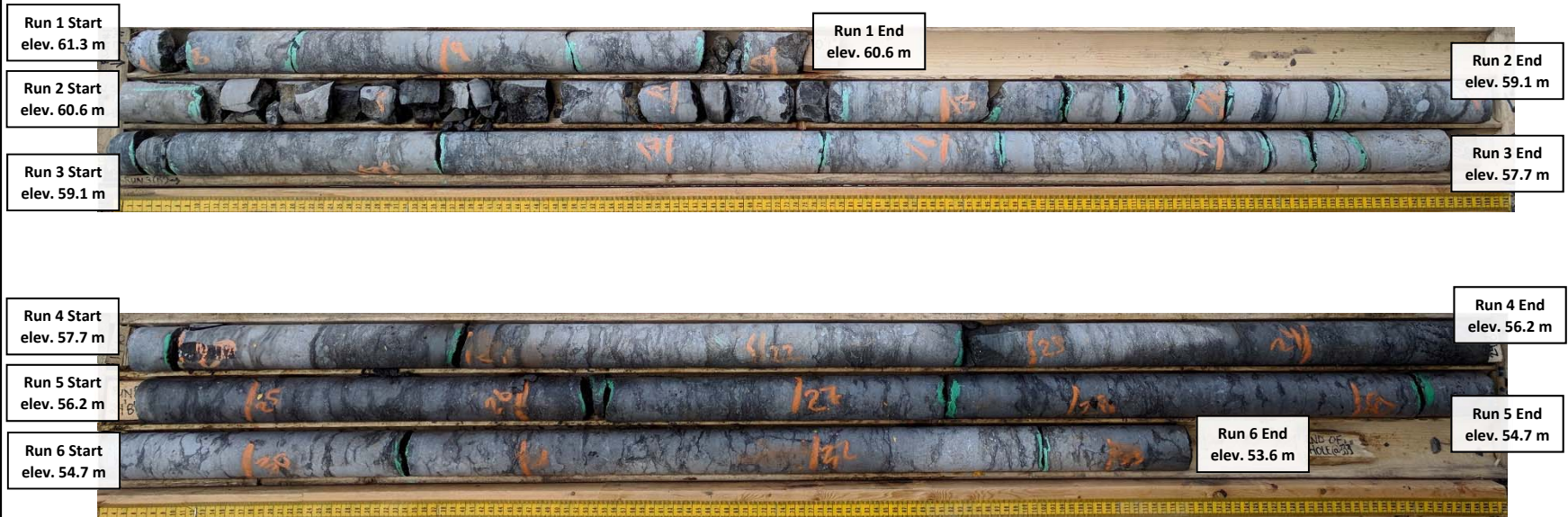
Run 4 End
elev. 54.5 m

Run 5 End
elev. 53.0 m

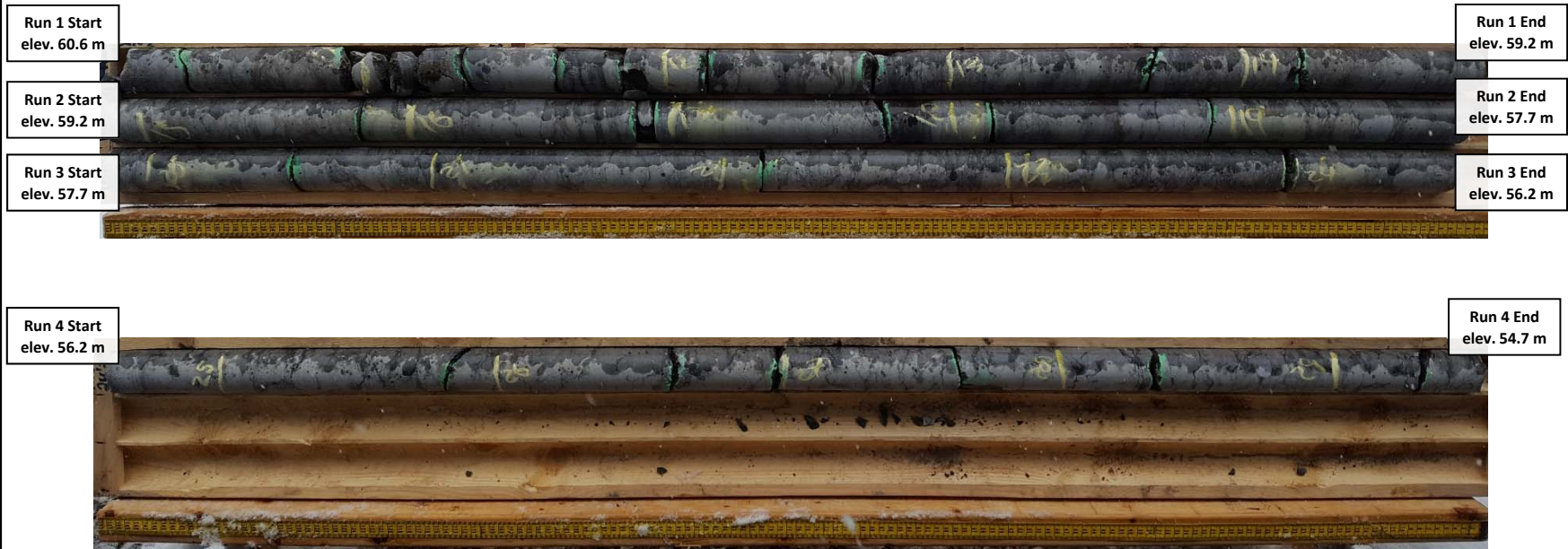
Run 5 Start
elev. 53.0 m

Run 6 End
elev. 51.5 m

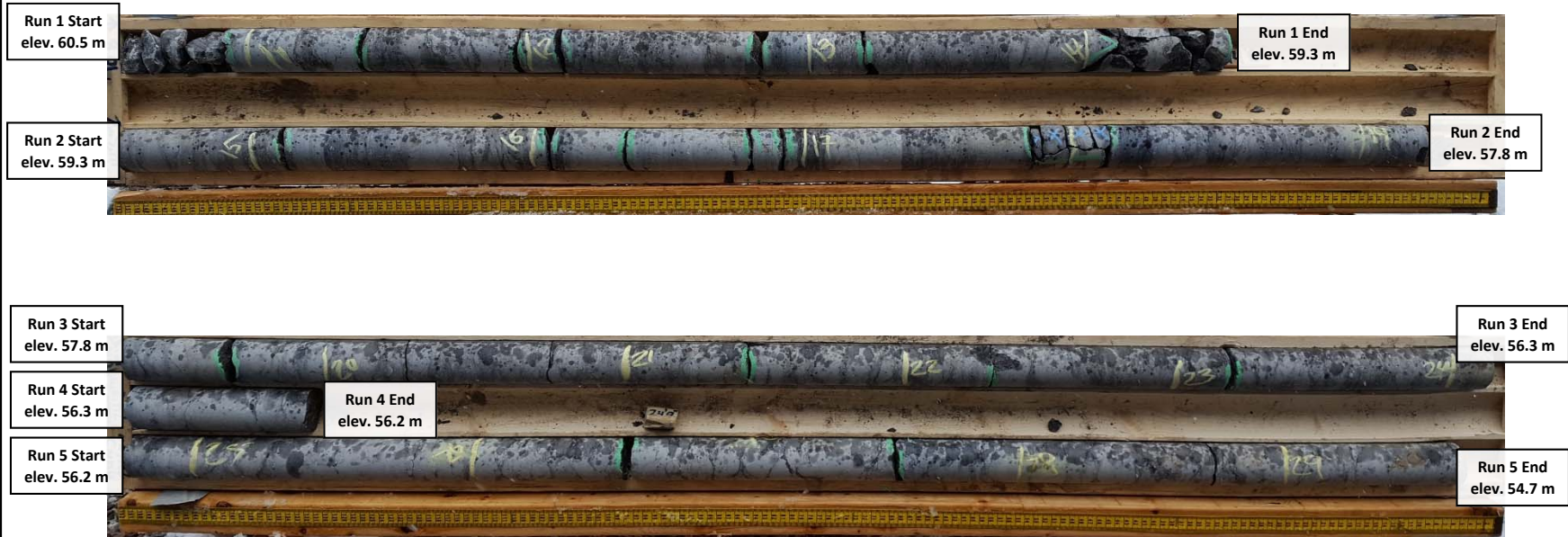
Borehole 18-08
Run 1 to 6 (of 6)
Elevation 61.3 m to 53.6 m



Borehole 18-09
Run 1 to 4 (of 4)
Elevation 60.6 m to 54.7 m



Borehole 18-10
Run 1 to 5 (of 5)
Elevation 60.5 m to 54.7 m



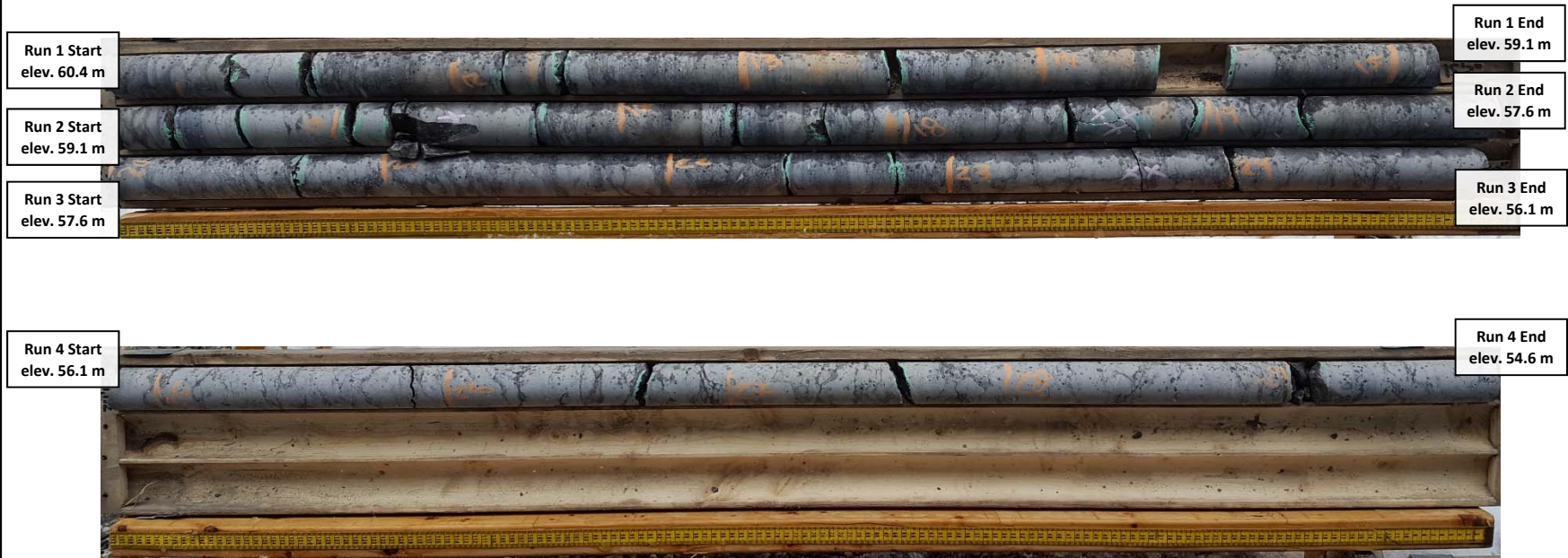
THURBER ENGINEERING LTD.

**Foundation Investigation
Replacement of CPR/O-Train Bridges,
Highway 417 Ottawa, Ontario**

GWP 4245-05-00

Project No.: 11189

Borehole 18-11
Run 1 to 4 (of 4)
Elevation 60.4 m to 54.6 m



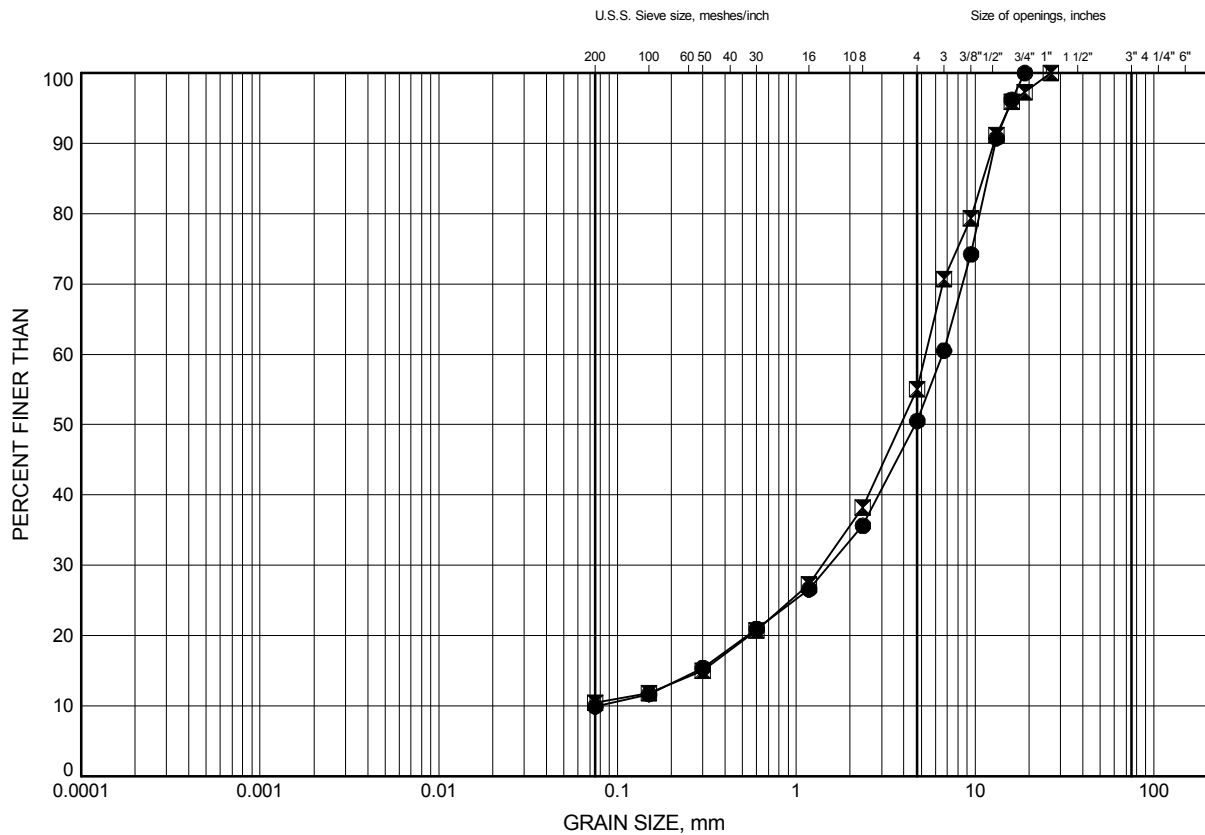
Borehole 18-12
Run 1 to 4 (of 4)
Elevation 60.3 m to 54.4 m



Appendix C.
Laboratory Test Results

GRAIN SIZE DISTRIBUTION

FILL: Road Base



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	16-07	0.46	71.54
⊠	16-08	0.46	68.44

Date June 2018

GWP# 4245-05-00

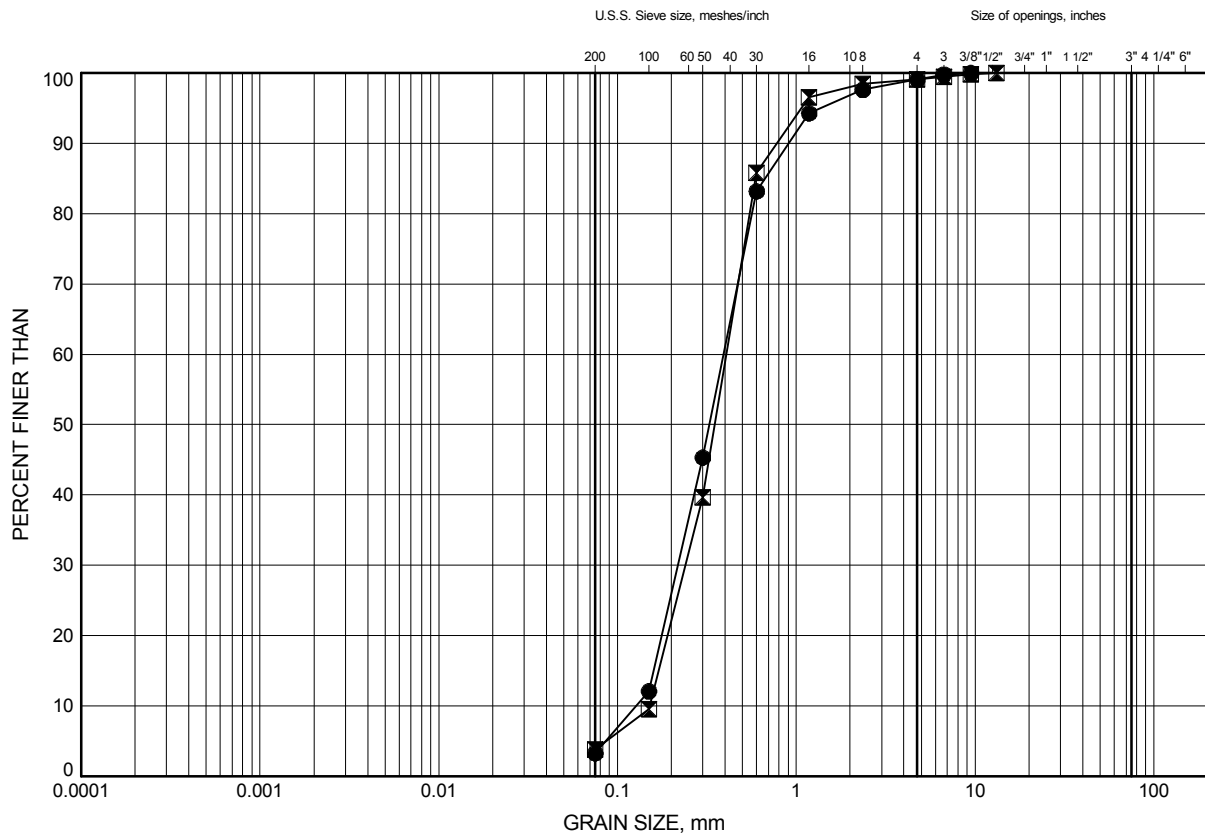


Prep'd JG

Chkd. PC

GRAIN SIZE DISTRIBUTION

FILL: Embankment



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	16-07	4.11	67.89
⊠	16-08	2.59	66.31

Date June 2018

GWP# 4245-05-00

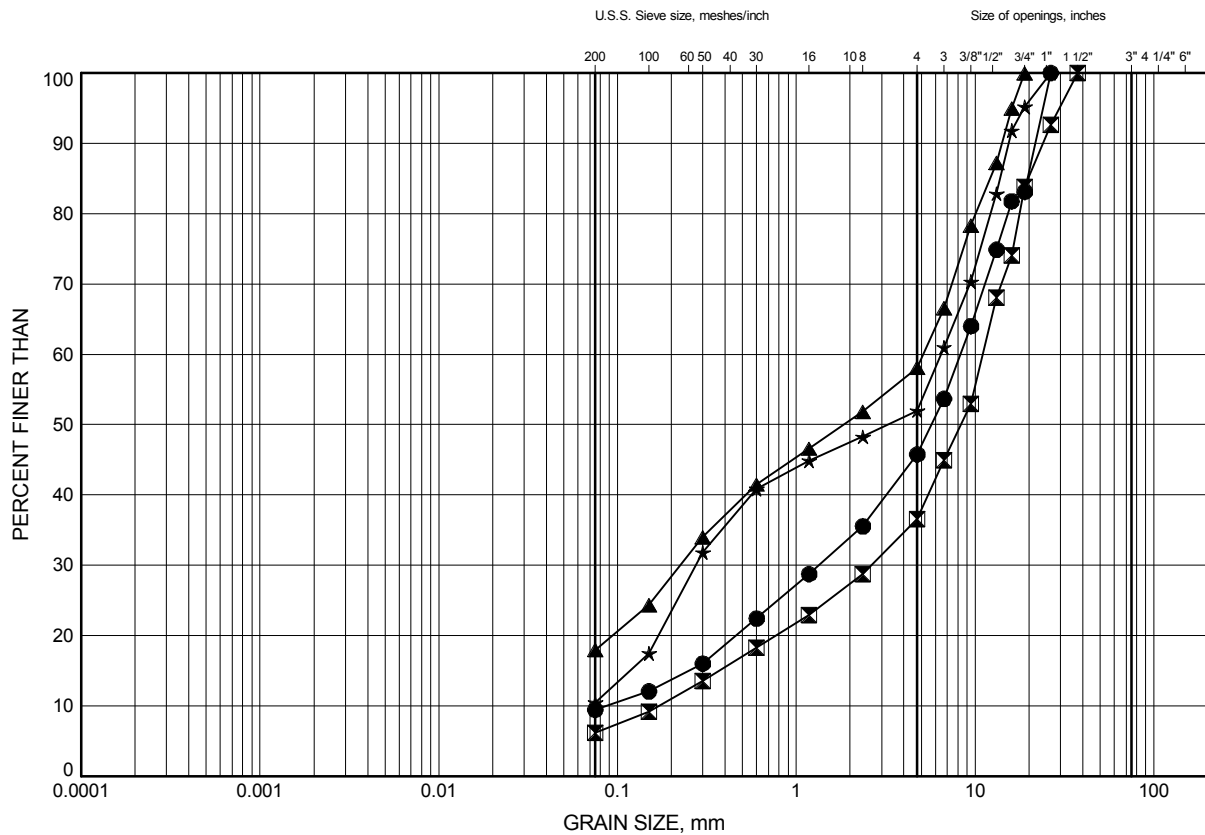


Prep'd JG

Chkd. PC

GRAIN SIZE DISTRIBUTION

FILL: Gravel



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	16-03	1.07	65.43
☒	16-09	1.07	62.43
▲	18-08	1.68	62.06
★	18-09	1.07	62.59

Date June 2018

GWP# 4245-05-00

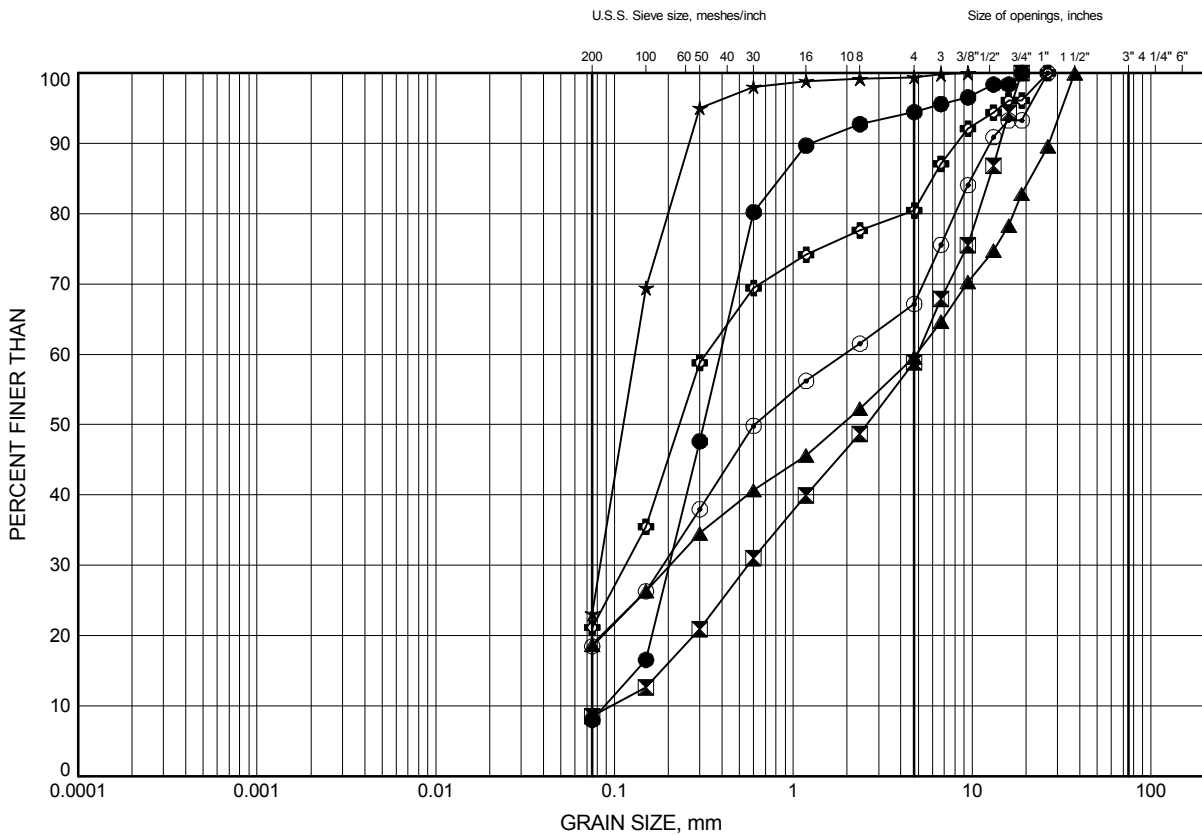


Prep'd JG

Chkd. PC

GRAIN SIZE DISTRIBUTION

FILL: Sand



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	16-01	1.83	60.97
⊠	16-02	1.07	61.43
▲	16-02	2.59	59.91
★	16-05	1.83	61.97
⊙	16-06	0.30	62.20
⊕	16-10	0.30	63.40

Date June 2018

GWP# 4245-05-00

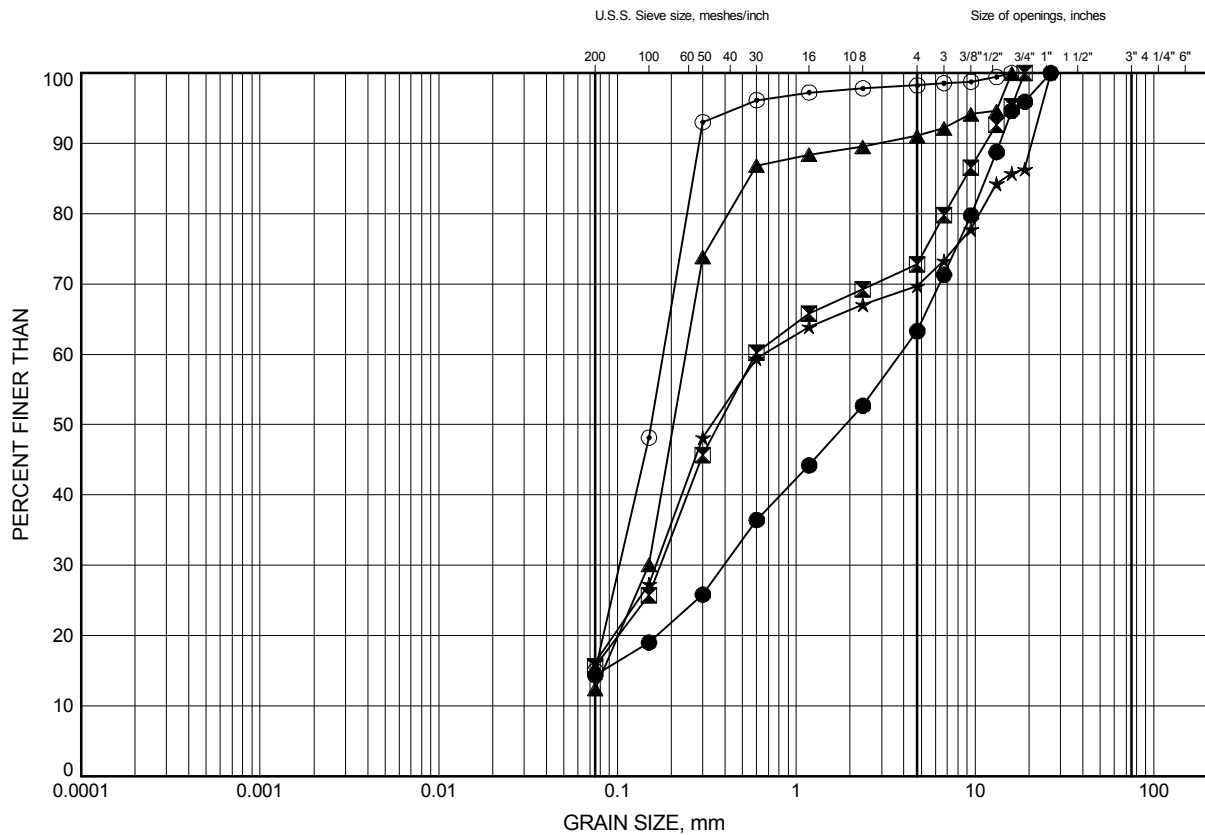


Prep'd JG

Chkd. PC

GRAIN SIZE DISTRIBUTION

FILL: Sand



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	16-10	1.22	62.48
⊠	16-11	1.07	62.73
▲	16-11	1.91	61.89
★	18-10	1.07	62.65
⊙	18-11	1.98	61.72

Date June 2018

GWP# 4245-05-00

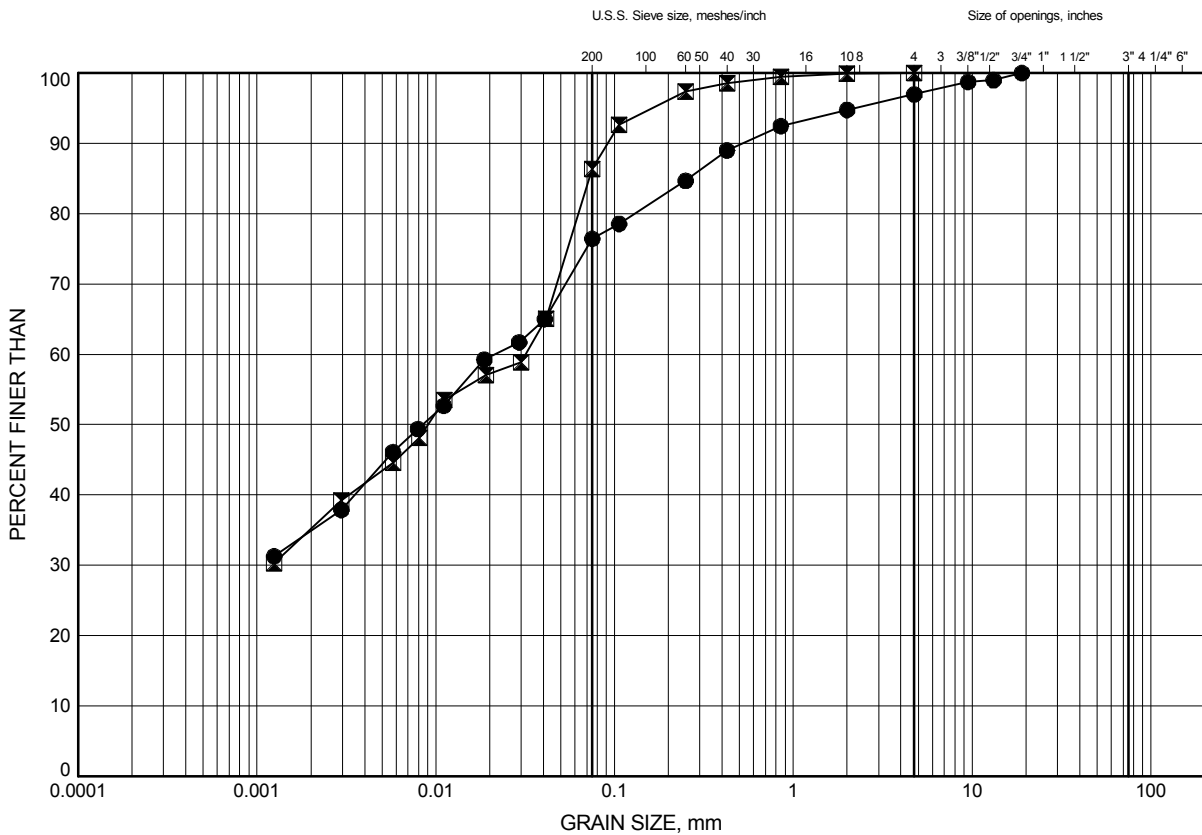


Prep'd JG

Chkd. PC

GRAIN SIZE DISTRIBUTION

CLAY



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	16-03	2.59	63.91
⊠	16-08	6.40	62.50

Date June 2018

GWP# 4245-05-00

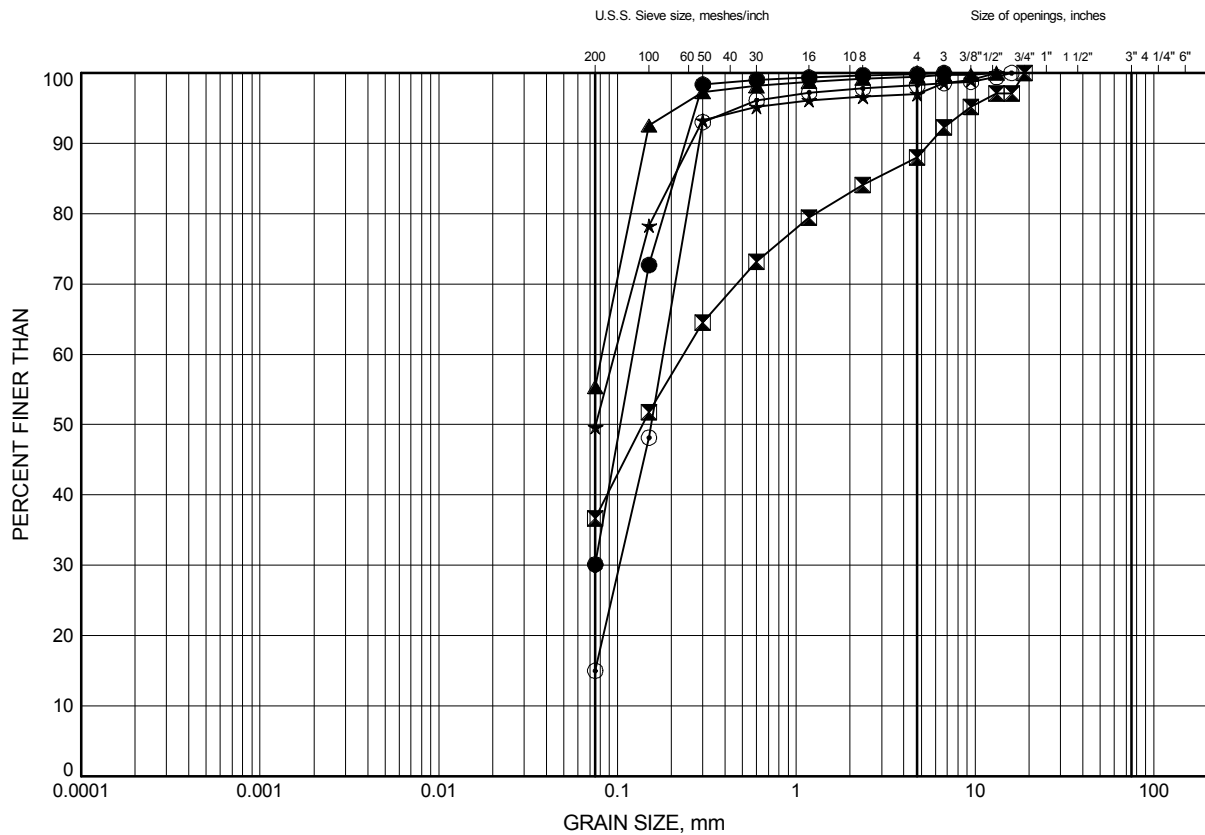


Prep'd JG

Chkd. PC

GRAIN SIZE DISTRIBUTION

SANDY SILT TO SILTY SAND



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	16-03	4.11	62.39
⊠	16-07	6.40	65.60
▲	16-07	9.45	62.55
★	18-10	2.59	61.13
◉	18-11	1.98	61.72

Date June 2018

GWP# 4245-05-00

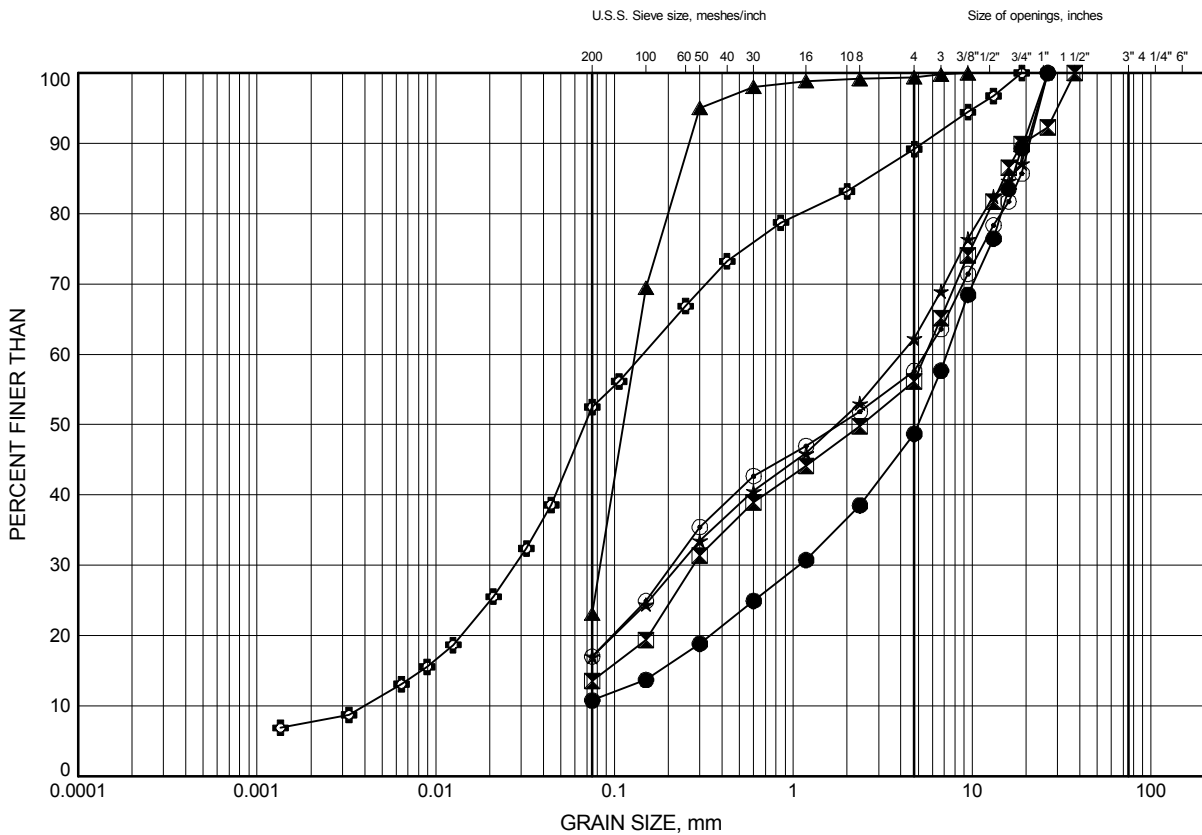


Prep'd JG

Chkd. PC

GRAIN SIZE DISTRIBUTION

GLACIAL TILL



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	16-01	2.59	60.21
⊠	16-04	1.83	61.53
▲	16-05	1.83	61.97
★	16-06	2.59	59.91
⊙	16-09	2.59	60.91
⊕	16-11	2.74	61.06

Date June 2018

GWP# 4245-05-00

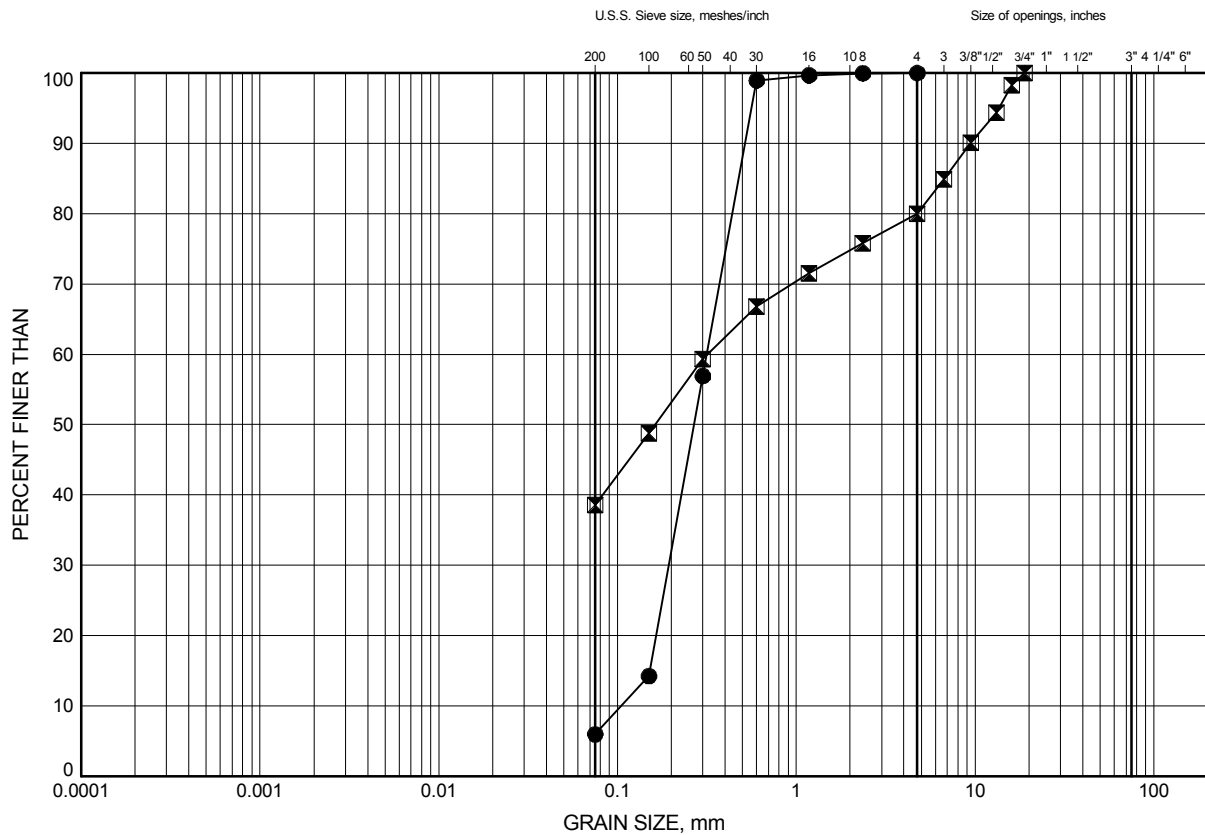


Prep'd JG

Chkd. PC

GRAIN SIZE DISTRIBUTION

GLACIAL TILL



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	18-08	1.98	61.76
⊠	18-12	2.59	60.85

Date June 2018

GWP# 4245-05-00



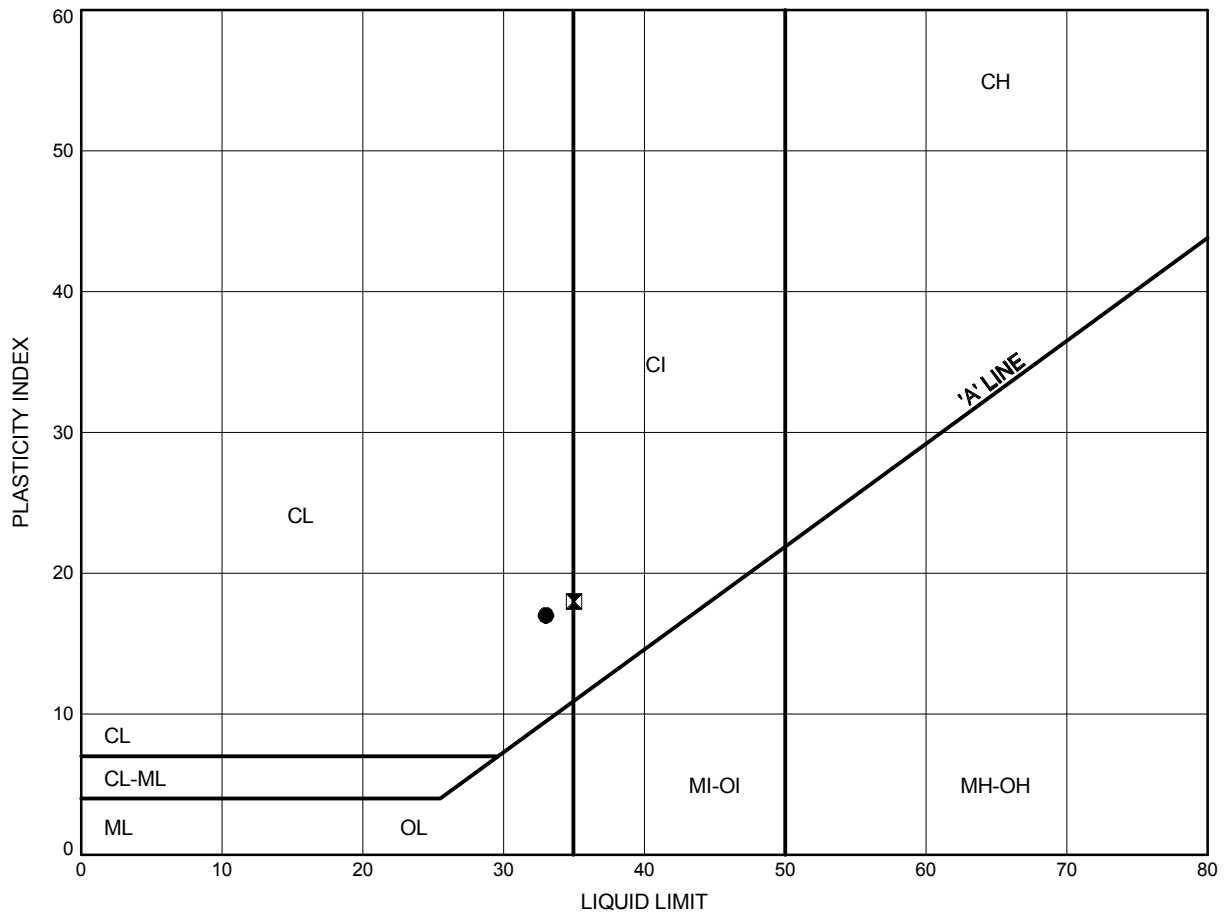
Prep'd JG

Chkd. PC

Replacement of CPR/O-Train Bridges, Highway 417 Ottawa

ATTERBERG LIMITS TEST RESULTS

FIGURE C10



LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	16-03	2.59	63.91
◻	16-08	6.40	62.50

Date June 2018
GWP# 4245-05-00



Prep'd JG
Chkd. PC



Stantec Consulting Ltd
2781 Lancaster Rd, Suite 100 A&B
Ottawa, ON K1B 1A7
Tel: (613) 738-6075
Fax: (613) 722-2799

Stantec

April 25, 2017
File: 122410864

Attention: Thurber Engineering Ltd., File #11189

Reference: ASTM D7012, Method C, Unconfined Compressive Strength of Intact Rock Core

The table below summarizes twelve rock core unconfined compressive strength results.

Location	Sample Depth	Compressive Strength (MPa)	Description of Break
BH16-1 Run-3	23'	123.4	No well-formed cones, vertical cracks throughout
BH16-3 Run-2	25'3"	128.1	No well-formed cones on either end
Bh16-5 Run-2	17'5"	141.2	No well-formed cones, vertical cracks throughout
BH16-6 Run-2	20'3"	142.7	Well-formed cone on bottom, vertical cracks through top
Bh16-2 Run-2	18'3"	127.9	Two well-formed cones on either end
BH16-2 Run-3	24'2"	119.6	Two well-formed cones on either end
BH16-2 Run-4	35'	136.3	Well-formed cone on bottom, vertical cracks through top
BH16-2 Run-6	39'1"	137.5	No well-formed cones, vertical cracks throughout
BH16-11 Run-2	17'	156.6	Two well-formed cones on either end
BH16-11 Run-3	23'3"	131.7	Two well-formed cones on either end
BH16-11 Run-5	31'2"	139.5	Two well-formed cones on either end
BH16-11 Run-6	38'9"	106.5	No well-formed cones, long vertical cracks throughout

Sincerely,

Stantec Consulting Ltd

Brian Prevost
Laboratory Supervisor
Tel: 613-738-6075
brian.prevost@stantec.com



Stantec Consulting Ltd
2781 Lancaster Rd, Suite 100 A&B
Ottawa, ON K1B 1A7
Tel: (613) 738-6075
Fax: (613) 722-2799

Stantec

March 7, 2018
File: 122410864

Attention: Thurber Engineering Ltd., File #11189

Reference: ASTM D7012, Method C, Unconfined Compressive Strength of Intact Rock Core

The table below summarizes three (8) rock core unconfined compressive strength results.

Location	Sample Depth	Compressive Strength (MPa)	Description of Break
18-8	8'5"-9'5"	112.5	One large diagonal crack through centre of core
18-8	20'-21'	101.4	Two well formed cones on either end
18-12	11'3"-12'1"	106.9	One large straight crack down centre of core
18-12	24'4"-25'2"	93	Well formed cone on one bottom, cracks through rest
18-3	20' 20'0"	120.2	Long vertical cracks throughout core
18-5	11' 15'	96.4	Two well formed cones on either end
18-4	10' 10'0"	98.8	Two well formed cones on either end
18-2	30'4"-31'3"	93.3	Well formed cone on bottom, vertical cracks through rest

Sincerely,

Stantec Consulting Ltd

Brian Prevost

Brian Prevost
Laboratory Supervisor
Tel: 613-738-6075
brian.prevost@stantec.com



Stantec

Stantec Consulting Ltd
2781 Lancaster Rd, Suite 100 A&B
Ottawa, ON K1B 1A7
Tel: (613) 738-6075
Fax: (613) 722-2799

March 20, 2018
File: 122410864

Attention: Thurber Engineering Ltd., File #11189

Reference: ASTM D7012, Method C, Unconfined Compressive Strength of Intact Rock Core

The table below summarizes seven (7) rock core unconfined compressive strength results.

Location	Sample Depth	Compressive Strength (MPa)	Description of Break
16-10	15'10"	101.1	Two well-formed cones on either end
18-9	14'6"	150.3	Well-formed cone on bottom, vertical cracks through top
18-9	24'3"	123.0	Two well-formed cones on either end
18-10	14'10"	118.0	No well-formed cones, vertical cracks throughout
18-10	24'6"	116.3	Two well-formed cones on either end
18-11	14'7"	108.7	Two well-formed cones on either end
18-11	20'3"	139.0	No well-formed cones, vertical cracks throughout

Sincerely,

Stantec Consulting Ltd

Brian Prevost

Brian Prevost
Laboratory Supervisor
Tel: 613-738-6075
brian.prevost@stantec.com

Certificate of Analysis

Thurber Engineering Ltd.

2460 Lancaster Rd, Suite 104
Ottawa, ON K1B 4S5
Attn: Justin Gray

Client PO: 11189
Project: OTrain Bridge
Custody: 39592

Report Date: 21-Jun-2018
Order Date: 8-Mar-2018

Revised Report

Order #: 1810380

This Certificate of Analysis contains analytical data applicable to the following samples as submitted:

Paracel ID	Client ID
1810380-01	16-4, SS4, 7'6"-9'6"
1810380-02	18-11, SS4, 10'-10'9"

Approved By:



Mark Foto, M.Sc.
Lab Supervisor

Certificate of Analysis
Client: Thurber Engineering Ltd.
Client PO: 11189

Report Date: 21-Jun-2018

Order Date: 8-Mar-2018

Project Description: OTrain Bridge

Analysis Summary Table

Analysis	Method Reference/Description	Extraction Date	Analysis Date
Anions	EPA 300.1 - IC, water extraction	9-Mar-18	10-Mar-18
Conductivity	MOE E3138 - probe @25 °C, water ext	9-Mar-18	9-Mar-18
pH, soil	EPA 150.1 - pH probe @ 25 °C, CaCl buffered ext.	11-Mar-18	12-Mar-18
Resistivity	EPA 120.1 - probe, water extraction	9-Mar-18	9-Mar-18
Solids, %	Gravimetric, calculation	13-Mar-18	13-Mar-18

Certificate of Analysis
Client: Thurber Engineering Ltd.
Client PO: 11189

Report Date: 21-Jun-2018

Order Date: 8-Mar-2018

Project Description: OTrain Bridge

Client ID:	16-4, SS4, 7'6"-9'6"	18-11, SS4, 10'-10'9"	-	-
Sample Date:	03/07/2018 00:00	02/27/2018 00:00	-	-
Sample ID:	1810380-01	1810380-02	-	-
MDL/Units	Soil	Soil	-	-

Physical Characteristics

% Solids	0.1 % by Wt.	88.7	91.8	-	-
----------	--------------	------	------	---	---

General Inorganics

Conductivity	5 uS/cm	227	675	-	-
pH	0.05 pH Units	7.90	7.89	-	-
Resistivity	0.10 Ohm.m	44.1	14.8	-	-

Anions

Chloride	5 ug/g dry	33	333	-	-
Sulphate	5 ug/g dry	64	122	-	-

Certificate of Analysis
Client: Thurber Engineering Ltd.
Client PO: 11189

Report Date: 21-Jun-2018

Order Date: 8-Mar-2018

Project Description: OTrain Bridge

Method Quality Control: Blank

Analyte	Result	Reporting Limit	Units	Source Result	%REC	%REC Limit	RPD	RPD Limit	Notes
Anions									
Chloride	ND	5	ug/g						
Sulphate	ND	5	ug/g						
General Inorganics									
Conductivity	ND	5	uS/cm						
Resistivity	ND	0.10	Ohm.m						

Certificate of Analysis
Client: Thurber Engineering Ltd.
Client PO: 11189

Report Date: 21-Jun-2018

Order Date: 8-Mar-2018

Project Description: OTrain Bridge

Method Quality Control: Duplicate

Analyte	Result	Reporting Limit	Units	Source Result	%REC	%REC Limit	RPD	RPD Limit	Notes
Anions									
Chloride	30.7	5	ug/g dry	32.9			6.8	20	
Sulphate	58.9	5	ug/g dry	64.0			8.3	20	
General Inorganics									
Conductivity	163	5	uS/cm	165			1.3	6.2	
pH	7.00	0.05	pH Units	7.02			0.3	10	
Resistivity	61.3	0.10	Ohm.m	60.5			1.3	20	
Physical Characteristics									
% Solids	82.6	0.1	% by Wt.	82.7			0.2	25	

Certificate of Analysis
Client: Thurber Engineering Ltd.
Client PO: 11189

Report Date: 21-Jun-2018

Order Date: 8-Mar-2018

Project Description: OTrain Bridge

Method Quality Control: Spike

Analyte	Result	Reporting Limit	Units	Source Result	%REC	%REC Limit	RPD	RPD Limit	Notes
Anions									
Chloride	127	5	ug/g	32.9	93.7	78-113			
Sulphate	160	5	ug/g	64.0	96.5	78-111			

Certificate of Analysis
Client: Thurber Engineering Ltd.
Client PO: 11189

Report Date: 21-Jun-2018
Order Date: 8-Mar-2018
Project Description: OTrain Bridge

Qualifier Notes:

Login Qualifiers :

Container(s) - Bottle and COC sample ID don't match -
Applies to samples: 18-11, SS4, 10'-10'9"

Sample Data Revisions

None

Work Order Revisions / Comments:

Revision 1 - this report includes an updated client Sample ID.

Other Report Notes:

n/a: not applicable
ND: Not Detected
MDL: Method Detection Limit
Source Result: Data used as source for matrix and duplicate samples
%REC: Percent recovery.
RPD: Relative percent difference.

Soil results are reported on a dry weight basis when the units are denoted with 'dry'.
Where %Solids is reported, moisture loss includes the loss of volatile hydrocarbons.

Paracel ID: 1810380



RELIABLE

Office
9 St. Laurent Blvd.
Ontario K1G 4J8
749-1947
e: paracel@paracellabs.com

Chain of Custody
(Lab Use Only)

No 39592

Page 1 of 1

Client Name: <u>Thurber Engineering Ltd.</u>	Project Reference: <u>OTrain Bridge</u>	Turnaround Time: <input type="checkbox"/> 1 Day <input type="checkbox"/> 3 Day <input type="checkbox"/> 2 Day <input checked="" type="checkbox"/> Regular Date Required: _____
Contact Name: <u>Justin Gray</u>	Quote # _____	
Address: <u>2460 Lancaster Rd Ottawa.</u>	PO # <u>11189</u>	
Telephone: <u>613-247-2121</u>	Email Address: <u>jgray@thurber.ca</u>	

Criteria: ☐ O. Reg. 153/04 (As Amended) Table __ ☐ RSC Filing ☐ O. Reg. 558/00 ☐ PWQO ☐ CCME ☐ SUB (Storm) ☐ SUB (Sanitary) Municipality: _____ ☐ Other: _____

Matrix Type: S (Soil Sed.) GW (Ground Water) SW (Surface Water) SS (Storm Sanitary Sewer) P (Paint) A (Air) O (Other)

Required Analyses

Parcel Order Number:			Matrix	Air Volume	# of Containers	Sample Taken		pH	conductivity	resistivity	sulphates chlorides								
Sample ID/Location Name						Date	Time												
1	16-4, SS4, 7'6"-9'6"			S		1	Mar 7/2018	✓	✓	✓	✓								
2	16-10, SS4, 10'-10'9"			S		1	Feb 27/2018	✓	✓	✓	✓								
3	↳ Borehole 18-11																		
4																			
5																			
6																			
7																			
8																			
9																			
10																			

Comments:			Method of Delivery: <u>Walkin</u>		
Relinquished By (Sign): <u>[Signature]</u>	Received by Driver/Depot:	Received at Lab: <u>[Signature]</u>	Verified By: <u>Rachel Subject</u>		
Relinquished By (Print): <u>Justin Gray</u>	Date/Time:	Date/Time: <u>March 8, 2018</u>	Date/Time: <u>Mar 8/18</u>		
Date/Time: <u>March 8, 2018</u>	Temperature: _____ °C	Temperature: <u>22.4</u> °C	pH Verified <input checked="" type="checkbox"/> By: <u>N/A</u> <u>3:14</u>		

Chain of Custody (Blank) - Rev 0.4 Feb 2016

Appendix D.
Site Photographs

REPLACEMENT OF CPR/O-TRAIN BRIDGES
HIGHWAY 417, OTTAWA



Photo 1. Looking south under the 417 (along the Trillium Pathway) (2017-02-02) .



Photo 2. Looking east at bridge abutment (near 16-03) (2017-02-02).

REPLACEMENT OF CPR/O-TRAIN BRIDGES
HIGHWAY 417, OTTAWA

v



Photo 3. Looking north under Highway 417 (near 16-05) (2017-02-02).



Photo 4. Looking west from east abutment (near 16-11) (2017-02-02).

REPLACEMENT OF CPR/O-TRAIN BRIDGES
HIGHWAY 417, OTTAWA



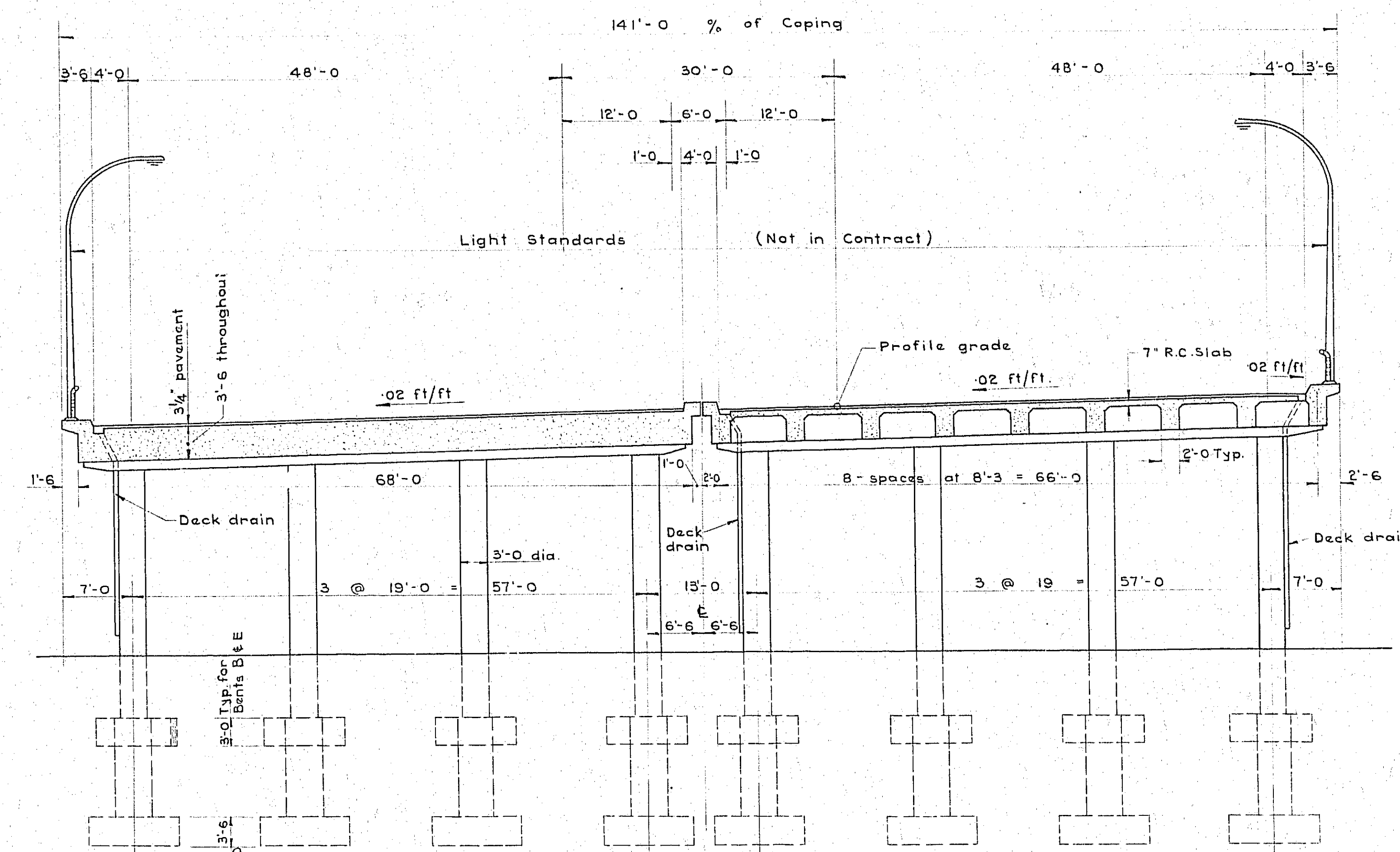
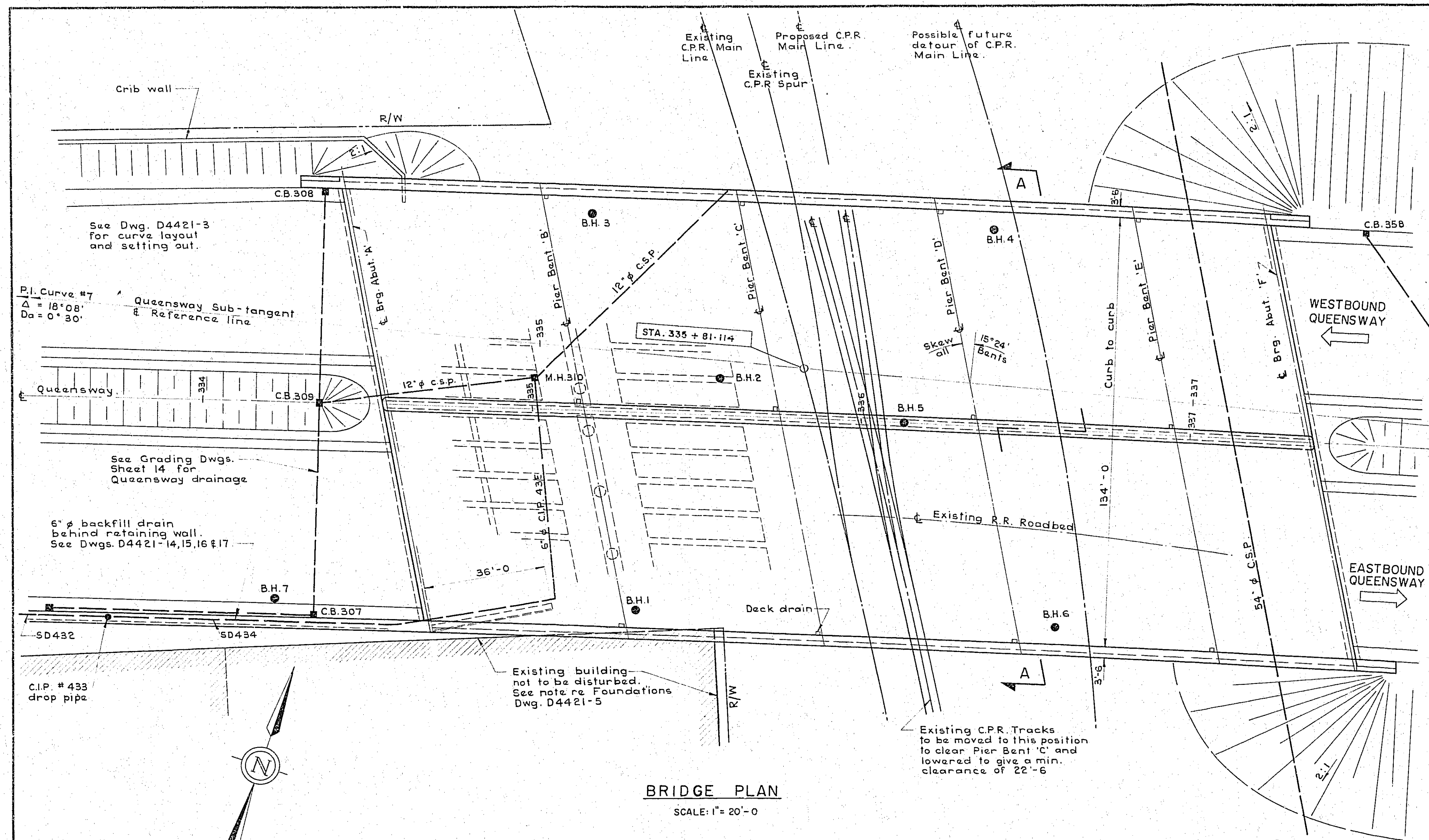
Photo 5. Looking north from pedestrian bridge (2017-01-27).



Photo 6. Looking west showing exposed rock in railway cut (2017-02-02).

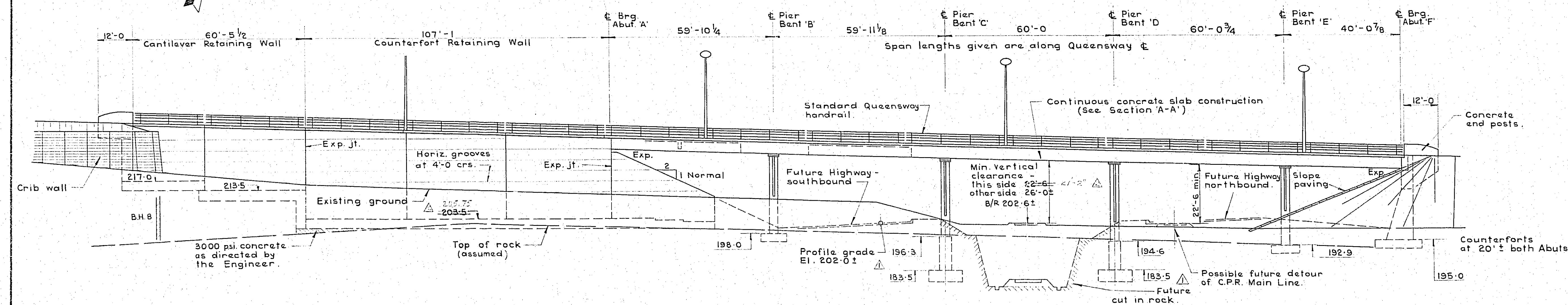
Appendix E.

**Historical Contract Drawings – Existing Bridges
Preliminary General Arrangement – Proposed Structure**



GENERAL NOTES

- These drawings to be read in conjunction with the specifications.
- Concrete work on this structure must not be commenced until monuments to fix control points have been erected and checked by the District Engineer.
- Structures to be built in accordance with Form 9, latest revision and Special Provisions, extra copies of which may be obtained from the District Engineer.
- The complete Soils Investigation Report BA-919 may be examined at the Bridge Office, Toronto. The Department does not guarantee the accuracy of this report or the abridged version shown on Dwg. D4421-4.
- Minimum strength of all concrete in structure to be 3000 psi at 28 days. Approved admixtures will be added to all concrete as specified by the Engineer.
- Clear cover to reinforcement to be as follows: 3 in. for concrete faces against earth or rock 2 in. for exposed faces of retaining walls, abutments and pier bent columns.
- Deck, curbs, coping and and posts as noted.
- All exposed edges to be chamfered 3/4 in. x 3/4 in. except as noted.
- All construction joints must be approved by the Engineer.
- The General Contractor must carry out the work subject to whatever regulations the C.P.R. may consider necessary for the protection of its traffic.
- Any existing utilities located during excavation to be carefully exposed by the General Contractor and relocated by the Utility concerned or encased as directed by the Engineer.

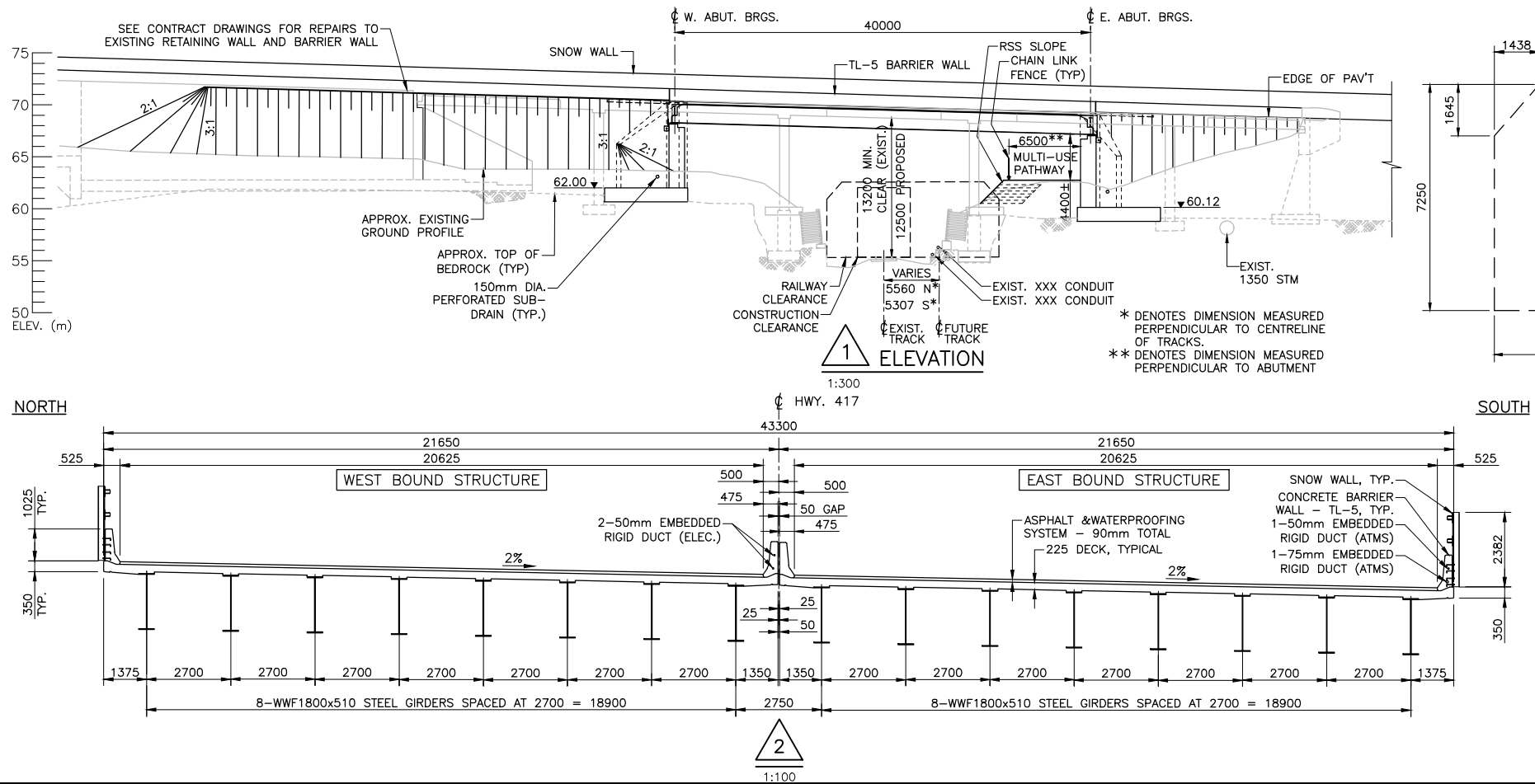
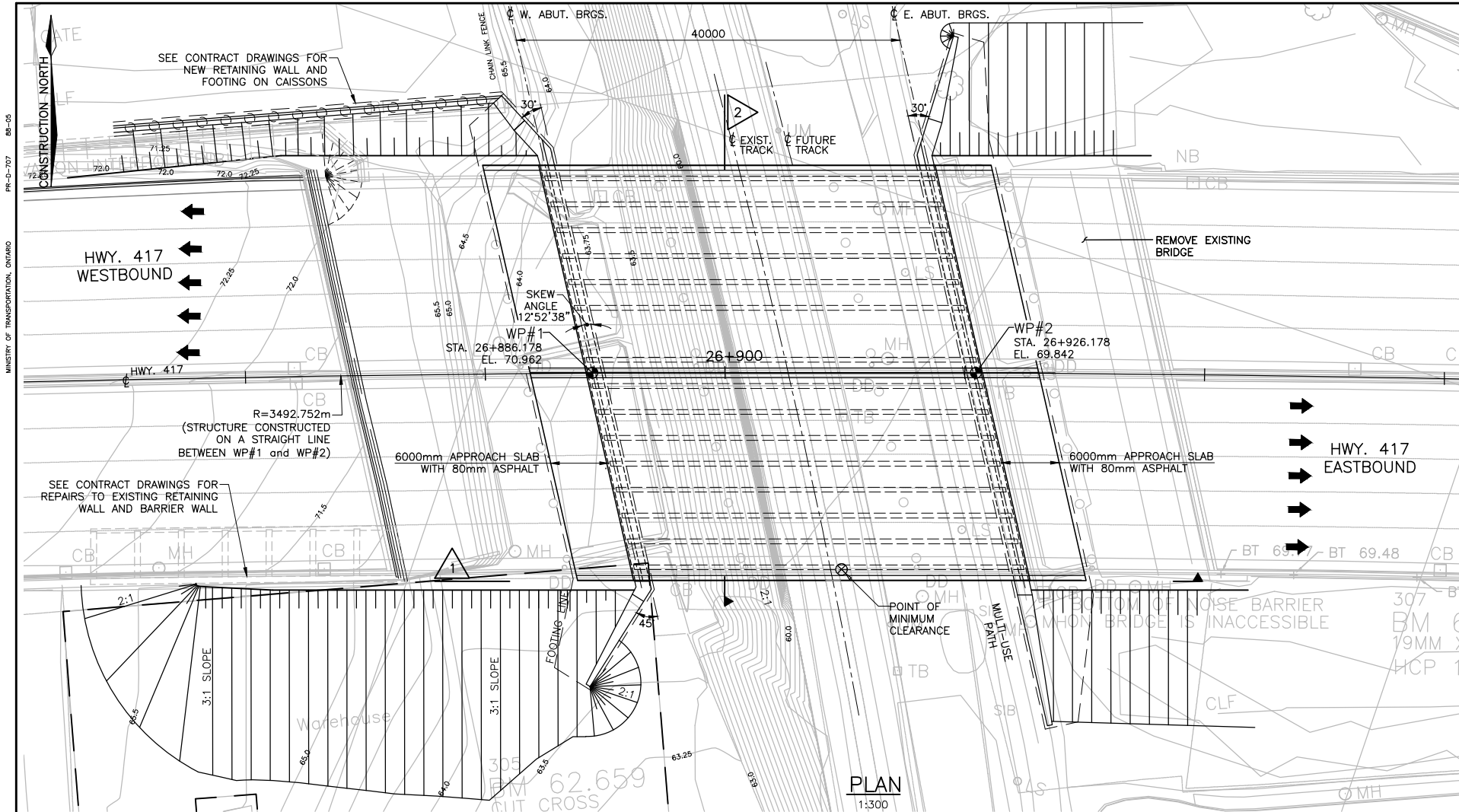


Y:\34 16014 HWY 417 CPR [O-TRAIN BRIDGE REPLACEMENT] 34 16014 Hwy 417 CPR O-Train Main Structure Contracts34 16014 - 01 GA.dwg, 03/04/2018 2:37:44 PM, DWG To PDF.pc3

CAD FILE LOCATION AND NAME: Y:\34 16014 HWY 417 CPR [O-TRAIN BRIDGE REPLACEMENT] 34 16014 Hwy 417 CPR O-Train Main Structure Contracts34 16014 - 01 GA.dwg

MODIFIED: 4/3/2018 2:03:09 PM BY: MASONP

DATE PLOTTED: 4/3/2018 2:37:59 PM BY: MASONP



WORKING POINT COORDINATES (m)			
LOCATION	STATION	NORTHING	EASTING
WP#1	26+886.024	5029472.144	366411.382
WP#2	26+926.024	5029485.007	366449.257

NOTE:



SKEW ANGLE IS MEASURED FROM A LINE PERPENDICULAR TO THE STRAIGHT LINE JOINING WP#1 AND WP#2

CONSTRUCTION NOTES:

1. THE CONTRACTOR SHALL VERIFY ALL DIMENSIONS OF THE WORK AND ALL DETAILS ON SITE AND REPORT ANY DISCREPANCIES TO THE CONTRACT ADMINISTRATOR BEFORE PROCEEDING WITH THE REPAIR WORK. THE CONTRACTOR SHALL ADJUST DIMENSIONS OF THE WORK AS REQUIRED TO SUIT EXISTING CONDITIONS.
2. THE CONTRACTOR SHALL ESTABLISH THE BEARING SEAT ELEVATIONS BY DEDUCTING THE ACTUAL BEARING THICKNESSES FROM THE TOP OF BEARING ELEVATIONS. IF THE ACTUAL BEARING THICKNESSES ARE DIFFERENT FROM THE GIVEN WITH THE BEARING DATA, THE CONTRACTOR SHALL ADJUST THE REINFORCING STEEL TO SUIT.
3. AT NO TIME SHALL THE DIFFERENCE IN HEIGHT OF BACKFILL BEHIND THE EAST AND WEST DECK ENDS BE GREATER THAN 500mm.
4. ALL EXPOSED EDGES TO RECEIVE A 20x20 CHAMFER.

LIST of DRAWINGS:

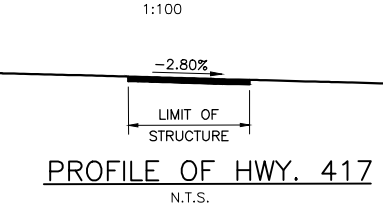
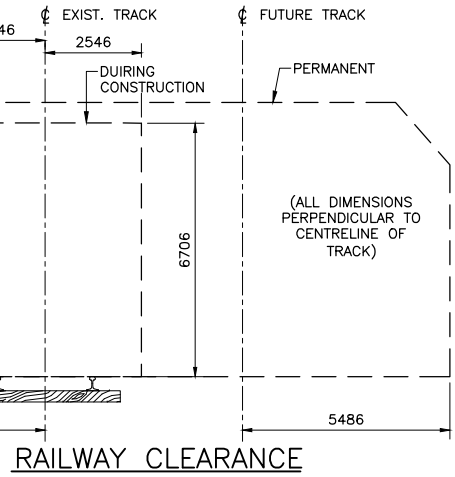
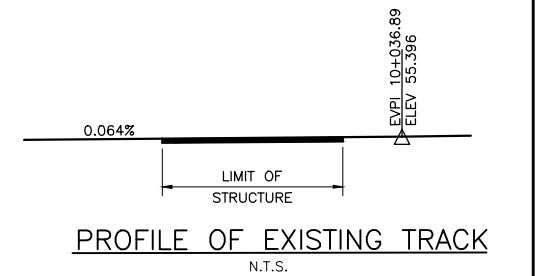
- 1 GENERAL ARRANGEMENT
- 2 STAGING AND REMOVALS
- 3 BOREHOLE LOCATIONS
- 4 SOIL STRATA
- 5 ~~MASS STRUCTURAL CONCRETE NOT USED~~
- 6 WEST and EAST FOUNDATION GEOMETRY
- 7 WEST and EAST FOUNDATION REINFORCEMENT
- 8 WEST ABUTMENT WALL GEOMETRY
- 9 WEST ABUTMENT WALL REINFORCEMENT I
- 10 WEST ABUTMENT WALL REINFORCEMENT II
- 11 WEST ABUTMENT WALL REINFORCEMENT III NOT USED
- 12 EAST ABUTMENT WALL GEOMETRY
- 13 EAST ABUTMENT WALL REINFORCEMENT
- 14 MISCELLANEOUS RETAINING WALLS NOT USED
- 15 STRUCTURAL STEEL I
- 16 STRUCTURAL STEEL II
- 17 DECK GEOMETRY PLAN, SECTION and DETAILS
- 18 DECK REINFORCEMENT
- 19 SCREED ELEVATIONS
- 20 NOISE BARRIER DETAILS
- 21 BARRIER WALL WITHOUT RAILING, TL-5 S.S. REBAR
- 22 6000mm APPROACH SLAB
- 23 STANDARD DETAILS
- 24 EMBEDDED WORK

DISTRICT		
CONT. No. WP No. 4245-05-00		
OTTAWA QUEENSWAY CPR/O-TRAIN OVERHEAD		SHEET
GENERAL ARRANGEMENT (40m SPAN)		
		METRIC

GENERAL NOTES:

1. ALL DIMENSIONS ARE BASED ON THE ORIGINAL DRAWINGS OF THE EXISTING BRIDGE BY THE DEPARTMENT OF HIGHWAYS ONTARIO, APRIL 1961 WP No. 938-59 AND BY MINISTRY OF TRANSPORTATION ONTARIO, AUGUST 1982, WP No. 175-06.
2. DESIGN STANDARDS AND CODES:
DESIGN CODE: CAN/CSA-S6-14 AND SUPPLEMENTS
MTO STRUCTURAL MANUAL
ONTARIO PROVINCIAL STANDARD SPECIFICATIONS
3. LIVE LOAD: CL-625 ONT
4. CLASS OF CONCRETE: ALL CONCRETE.....30MPa
5. CLEAR COVER TO REINFORCING STEEL:
FOOTING BOTTOM.....100±25
TOP AND SIDES.....70±20
COLUMNS.....70±20
DECK TOP.....70±20
BOTTOM.....40±10
REMAINDER.....70±20 UNLESS OTHERWISE NOTED
6. REINFORCING STEEL SHALL BE GRADE 400W UNLESS OTHERWISE SPECIFIED.
7. UNLESS SHOWN OTHERWISE, TENSION LAP SPLICES SHALL BE CLASS B.
8. BAR MARKS WITH PREFIX 'S' DENOTE STAINLESS STEEL BARS. BARS MARKED WITH PREFIX GIII DENOTE GRADE III GLASS FIBRE REINFORCED POLYMER BARS.
9. STAINLESS REINFORCING STEEL SHALL BE TYPE 316 LN OR DUPLEX 2205 AND HAVE A MINIMUM YIELD STRENGTH OF 500 MPa, UNLESS OTHERWISE SPECIFIED.

GLASS FIBRE REINFORCED POLYMER REINFORCING BARS SHALL BE GRADE III, THE NOMINAL DIAMETER, TENSILE MODULUS OF ELASTICITY AND GUARANTEED MINIMUM TENSILE STRENGTH SHALL BE AS SPECIFIED IN THE CONTRACT DOCUMENTS.
10. BAR HOOKS SHALL HAVE STANDARD HOOK DIMENSIONS USING MINIMUM BEND DIAMETERS, WHILE STIRRUPS AND TIES SHALL HAVE MINIMUM HOOK DIMENSIONS. ALL HOOKS SHALL BE IN ACCORDANCE WITH THE STRUCTURAL STANDARD DRAWING SS12-1 UNLESS INDICATED OTHERWISE.
11. ALL DIMENSIONS ARE IN MILLIMETERS ALL ELEVATIONS AND STATIONS ARE IN METERS UNLESS OTHERWISE SHOWN



PRELIMINARY
3 APR 2018
NOT FOR CONSTRUCTION

DRAWING NOT TO BE SCALED
100mm ON ORIGINAL DRAWING

APPLICABLE STANDARD DRAWINGS:	
OPSD 911.3810	GUIDE RAIL SYSTEM, CONCRETE BARRIER PERMANENT TRANSITION INSTALLATION, ROADSIDE CONCRETE BARRIER TO STRUCTURE
OPSD 911.3820	GUIDE RAIL SYSTEM, CONCRETE BARRIER DOWEL CONNECTION DETAIL
OPSD 3370.100	DECK, WATERPROOFING HOT APPLIED ASPHALT MEMBRANE WITH PROTECTION BOARD
OPSD 3370.101	DECK, WATERPROOFING HOT APPLIED ASPHALT MEMBRANE AT ACTIVE CRACKS GREATER THAN 2mm WIDE AND CONSTRUCTION JOINTS
OPSD 3419.100	BARRIERS AND RAILINGS STEEL GUIDE RAIL AND CHANNEL ANCHORAGE
OPSD 3941.200	FIGURES IN CONCRETE - SITE NUMBER AND DATE LAYOUT
REVISIONS	
NO.	DESCRIPTION
DESIGN FJP	CHK FJP
DRAWN PCM	CHK FJP
CODE CHBDC	2014 LOAD CL-625-ONT
DATE	APR/18
SITE	3-54
STRUCT	SCHEME
DWG	1

Appendix F.
Foundation Comparison

Comparison of Foundation Alternatives

Comment	Caissons	Micropiles	Spread Footings
Advantages:	<ul style="list-style-type: none">High axial and lateral resistance when socketed into sound bedrock	<ul style="list-style-type: none">Limited headroom beneath existing bridge deck less of a constraint compared to caissons.	<ul style="list-style-type: none">Does not require specialty equipmentCan be carried out within limited headroom conditionsHigh bearing resistance when founded on sound level bedrock
Disadvantages:	<ul style="list-style-type: none">Headroom beneath the existing bridge deck limits the equipment that can be used for caisson installation. Consideration was given to drilling through the existing bridge deck, however, the work could not be completed within the permitted lane closure restrictions.Very strong rock conditions limit the equipment that can be used for caisson installation.If retaining walls need to be constructed in front of the caissons, the potential for load transfer between the caissons and the wall would need to be considered and would add front of the caissons. The potential for load transfer between the caissons and the wall adds to design complexity.The use of caissons does not reduce the depth of excavation required compared to a spread footing design.	<ul style="list-style-type: none">The use of micropiles does not reduce the depth of excavation required compared to a spread footing design; a pilecap would be constructed at similar elevation as the spread footing.Additional construction time required relative to spread footing option.	<ul style="list-style-type: none">Development of resistance to uplift, sliding or overturning may require a larger footing and/or use of rock anchorsModerate bearing resistance where founded above a rock cut
Risks / Consequences	<ul style="list-style-type: none">Exact alignment and condition of existing rock face not known throughout new abutment limits. Drilling for caissons may intercept the rock face / potential for difficulty maintaining caisson alignment and/or of binding of the drill head due to mixed ground conditions.	<ul style="list-style-type: none">Exact alignment and condition of existing rock face not known throughout new abutment limits. Drilling for micropiles may intercept the rock face / potential for difficulty maintaining caisson alignment and/or of binding of the drill head due to mixed ground conditions.	<ul style="list-style-type: none">Exact alignment and condition of existing rock face not known throughout new footing limits / volume of rock excavation and mass concrete to achieve design founding elevation may be greater than initially estimated
Relative Cost	<ul style="list-style-type: none">Highest cost alternative	<ul style="list-style-type: none">Moderate cost: higher than spread footings but less than rock socketed caissons	<ul style="list-style-type: none">Lowest cost alternative
Conclusion			Recommended

**HIGHWAY 417 CPR/O-TRAIN BRIDGE REPLACEMENTS
W.P. 4245-05-00**

COMPARISON OF BACKFILL MATERIALS

Backfill Material	Conventional Granular Backfill (OPSS.PROV 1010, Granular A or B)	Clear Stone (OPSS.PROV 1004, 19 mm Type II)	Cellular Concrete	Unshrinkable Fill (OPSS.PROV 578)
Comments	<ul style="list-style-type: none"> Lower portion of backfill would be placed prior to demolition of existing deck and girders. Upper portion of backfill would need to be placed after demolition of existing deck and girders, during full closure of Hwy. 	<ul style="list-style-type: none"> No standard specification for compaction or quality control/verification of placement 	<ul style="list-style-type: none"> Although no OPSS for this material, it is available through Designated Sources List (Lightweight Fill Material) Compressive strength in the range of 500 to 700 kPa 	<ul style="list-style-type: none"> Compressive strength \leq 400 kPa
Advantages	<ul style="list-style-type: none"> Readily available Conventional compaction equipment and methods Provides required drainage behind abutment Very low post construction settlement Relatively stable excavation sidewalls for future median sewer trench work 	<ul style="list-style-type: none"> Readily available Provides required drainage behind abutment Requires less compaction effort than conventional granular backfill which should reduce time required for placement Contractor indicates that the clear stone could be placed to within 1 m below existing girder prior to bridge demolition (NOTE: the upper portion would not be roller compacted). Therefore, less backfill to be placed during RBR closure compared to conventional granular fill but still more than cellular concrete or unshrinkable fill options. 	<ul style="list-style-type: none"> No appreciable post construction settlement Flowable material could allow for placement to higher elevation prior to demolition of existing bridge since no need for compaction (within 0.1 m of underside of existing girders) Allows for future excavations for service trenches (e.g. for median sewer) with stable excavation slopes 	<ul style="list-style-type: none"> No appreciable post construction settlement Flowable material could allow for placement to higher elevation prior to demolition of existing bridge since no need for compaction (within 0.1 m of underside of existing girders) Less expensive than cellular concrete Allows for future excavations for service trenches (e.g. for median sewer) with stable excavation slopes
Disadvantages	<ul style="list-style-type: none"> Placement and compaction in lifts will take valuable time during the full closure of Highway 417 for the RBR Placement and compaction prior to the RBR would only reach to within about 2 m below the existing girders, leaving more backfill to be placed during the RBR closure 	<ul style="list-style-type: none"> Misconception by many contractors who believe that compaction is not required High post construction settlement if not compacted adequately (some estimates in the range of 8% of thickness) Requires geotextile wrap to prevent loss of fines from adjacent and overlying materials into voids within the clear stone. Excavation through clear stone for installation of median sewer will disturb geotextile wrap and excavation slopes in clear stone less stable than other backfill options. 	<ul style="list-style-type: none"> Must be placed in lifts and allowed to set between placement of lifts (max 650 mm typical). Therefore, not likely to save time compared to conventional granular backfill High cost Does not provide drainage; geosynthetic drainage layer could be used behind abutment and walls; shaping of pavement subgrade and possibly longitudinal subdrains may be required 	<ul style="list-style-type: none"> OPSS requires that, where placed behind structures, the material be placed in max 500 mm lifts with min 4 hours set time between placement of lifts. Therefore, not likely to save time compared to conventional granular backfill Does not provide drainage; geosynthetic drainage layer could be used behind abutment and walls; shaping of pavement subgrade and possibly longitudinal subdrains may be required
Risks / Consequences	<ul style="list-style-type: none"> Time for placement and compaction results in delayed completion of RBR and delayed re-opening of highway 	<ul style="list-style-type: none"> Contractor does not provide adequate compaction / excessive post construction settlement Loss of clear stone from sidewall of trench excavation for median sewer / undermining of adjacent pavement structure requiring additional lane closures for repairs 	<ul style="list-style-type: none"> Time for placement results in delayed completion of RBR and delayed re-opening of highway 	<ul style="list-style-type: none"> Time for placement results in delayed completion of RBR and delayed re-opening of highway
Relative Cost	Low	Low	High	Low to Medium

Appendix G.
GSC Seismic Hazard Calculation

2015 National Building Code Seismic Hazard Calculation

INFORMATION: Eastern Canada English (613) 995-5548 français (613) 995-0600 Facsimile (613) 992-8836
Western Canada English (250) 363-6500 Facsimile (250) 363-6565

February 15, 2017

Site: 45.4025 N, 75.7127 W User File Reference: Highway 17 / O-Train

Requested by: Ottawa, TEL

National Building Code ground motions: 2% probability of exceedance in 50 years (0.000404 per annum)

Sa(0.05)	Sa(0.1)	Sa(0.2)	Sa(0.3)	Sa(0.5)	Sa(1.0)	Sa(2.0)	Sa(5.0)	Sa(10.0)	PGA (g)	PGV (m/s)
0.445	0.521	0.437	0.332	0.236	0.118	0.056	0.015	0.0054	0.280	0.196

Notes. Spectral ($S_a(T)$, where T is the period in seconds) and peak ground acceleration (PGA) values are given in units of g (9.81 m/s^2). Peak ground velocity is given in m/s . Values are for "firm ground" (NBCC 2015 Site Class C, average shear wave velocity 450 m/s). NBCC2015 and CSAS6-14 values are specified in **bold** font. Three additional periods are provided - their use is discussed in the NBCC2015 Commentary. Only 2 significant figures are to be used. *These values have been interpolated from a 10-km-spaced grid of points. Depending on the gradient of the nearby points, values at this location calculated directly from the hazard program may vary. More than 95 percent of interpolated values are within 2 percent of the directly calculated values.*

Ground motions for other probabilities:

Probability of exceedance per annum	0.010	0.0021	0.001
Probability of exceedance in 50 years	40%	10%	5%
Sa(0.05)	0.044	0.147	0.246
Sa(0.1)	0.060	0.185	0.298
Sa(0.2)	0.055	0.160	0.254
Sa(0.3)	0.043	0.124	0.194
Sa(0.5)	0.031	0.088	0.138
Sa(1.0)	0.015	0.044	0.069
Sa(2.0)	0.0061	0.020	0.033
Sa(5.0)	0.0012	0.0047	0.0081
Sa(10.0)	0.0006	0.0019	0.0032
PGA	0.032	0.101	0.162
PGV	0.021	0.068	0.110

References

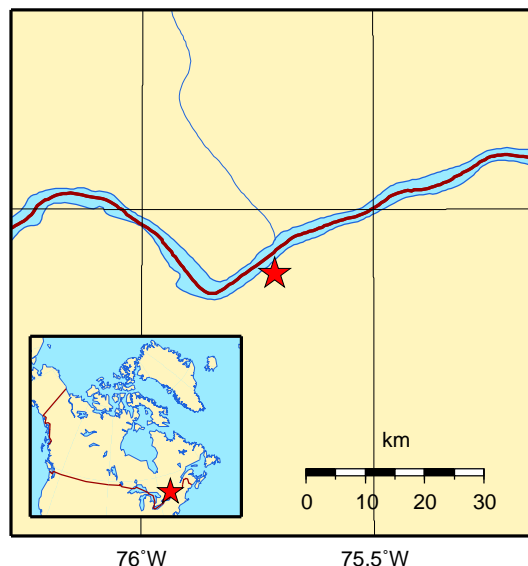
National Building Code of Canada 2015 NRCC no. 56190;
Appendix C: Table C-3, Seismic Design Data for Selected Locations in Canada

User's Guide - NBC 2015, Structural Commentaries NRCC no. 45.5"N
xxxxxx (in preparation)
Commentary J: Design for Seismic Effects

Geological Survey of Canada Open File 7893 Fifth Generation
Seismic Hazard Model for Canada: Grid values of mean hazard to be
used with the 2015 National Building Code of Canada

See the websites www.EarthquakesCanada.ca
and www.nationalcodes.ca for more information

Aussi disponible en français



Natural Resources
Canada

Ressources naturelles
Canada



Appendix H.

**List of Special Provisions and OPSS Documents Referenced in this Report
NSSP - Obstructions**

REPLACEMENT OF CPR/O-TRAIN BRIDGES
HIGHWAY 417, OTTAWA

1. The following Special Provisions and OPSS Documents are referenced in this report:

OPSS.PROV 206	Construction Specification for Grading
OPSD 3090.101	Foundation, Frost Penetration Depths for Southern Ontario
OPSS.PROV 501	Construction Specification for Compacting
OPSS.PROV 517	Construction Specification for Dewatering of Pipeline, Utility and Associated Structure Excavation
OPSS.PROV 539	Construction Specification for Temporary Protection Systems
OPSS.PROV 804	Construction Specification for Seed and Cover
OPSS 805	Construction Specification for Temporary Erosion and Sediment Control Measures
OPSS 902	Construction Specification for Excavating and Backfilling Structures
OPSS.PROV 1010	Material Specification for Aggregates Base, Subbase, Select Subgrade, and Backfill Material
OPSS 1860	Material Specification for Geotextile
OPSD 208.010	Benching of Earth Slopes

Special Provision 109S12	Amendment to OPSS 902, March 2018
Special Provision 517F01	Amendment to OPSS 517, July 2017
Special Provision Foun0003	Dewatering Structure Excavations, March 2018

NON-STANDARD SPECIAL PROVISIONS

NSSP – 902.07.05 EXCAVATION

Subsection 902.07.05 of OPSS 902 is amended by the addition of the following:

Excavations at the site may be impeded by obstructions such as boulders or debris within the existing fill and glacial till. The contractor shall be prepared to dislodge and remove these obstructions and extend the excavations to the design depths.

Reference can be made to the Foundation Investigation Report for the Replacement of the Highway 417 CPR/O-Train Bridges, prepared by Thurber Engineering Ltd., 2018, for further details on likely subsurface conditions at the foundation locations.