



THURBER ENGINEERING LTD.

**FOUNDATION INVESTIGATION AND DESIGN REPORT
RETAINING WALL FROM LORETTA AVENUE NORTH
TO THE CPR/O-TRAIN BRIDGES
HIGHWAY 417, OTTAWA**

GWP 4245-05-00

Geocres No.: 31G5-294

Report to:

WSP

Latitude: 45.402422
Longitude: -75.714114

October 2018
Thurber File: 11189

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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 crib retaining wall along the north side of Highway 417 from Loretta Avenue North to the Canadian Pacific Rail (CPR) / O-Train bridges in 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.

Thurber carried out a previous investigation in the vicinity of the retaining wall for the CPR/O-Train bridge replacement portion of this project. One of the boreholes advanced for the bridge (Borehole 16-03) is located in the vicinity of the retaining wall and is included in this report.

A previous foundation investigation report that was obtained from the online Geocres library and reviewed in preparation of 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 exact locations of the boreholes in this document were not known and therefore not included in this report.

2 SITE DESCRIPTION

The existing crib retaining wall is about 138 m long and is up to about 4.7 m in height. A noise wall and the westbound lanes of Highway 417 are located above the retaining wall. At the toe of the retaining wall, the landscape consists of a narrow strip of grass and an asphalt parking lot on the City of Ottawa Traffic Operations property.

A large diameter storm sewer and a sanitary sewer transect the site along an alignment that extends from Champagne Avenue on the south side of the highway. It is understood that

that these sewers continue northerly beneath the highway, retaining wall, and the Traffic Operations building. The depth of these sewers is not currently known. These sewers will remain in service and will not be relocated as part of this contract.

At this location, Highway 417 is an eight-lane urban freeway. The start of the crib wall is approximately 350 m west of Rochester Street. The outside lane of Highway 417 westbound adjacent to the crib wall alignment consists of the speed change lane for the on-ramp from Rochester Street. The land adjacent to the highway is generally developed with both industrial and residential properties. Traffic volumes on Highway 417 are understood to be 184,100 AADT (2016).

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

3 SITE INVESTIGATION AND FIELD TESTING

The current site investigation and field testing program was carried out between February 13th and 27th, 2018. The field investigation consisted of advancing five boreholes identified as 18-01 through 18-05. Boreholes 18-04 and 18-05 were advanced through the highway embankment and were drilled with a truck-mounted CME 75 drill rig. The remaining off-road boreholes were drilled with a track-mounted CME LC-60 drill rig.

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 at Boreholes 18-04 and 18-05 were surveyed relative to the top of the expansion joint of the WB structure (Lane 4) which was identified on base plans as having a geodetic elevation of 72.1 m. The elevations at Boreholes 18-01 through 18-03 were surveyed relative to the top of the concrete curb in front of the base of the existing crib wall, at the east end beside the chain link fence; this feature was identified on base plans as having a geodetic elevation of 67.1 m. 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.	Northing (m)	Easting (m)	Ground Surface Elevation (m)	Borehole Termination Depth Below Existing Ground Surface (m)
18-01	5029442.5	366264.5	71.4	15.7
18-02	5029460.2	366308.1	69.1	11.9
18-03	5029479.7	366352.7	66.7	9.0
18-04	5029444.4	366289.8	74.5	17.9
18-05	5029454.8	366318.6	73.8	16.4

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.

Soil samples were obtained at selected intervals using a split spoon sampler in conjunction with Standard Penetration Testing (SPT). Where split spoon sampling was not possible due to boulders and/or very dense conditions, the boreholes were advanced with wash boring and diamond drilling techniques.

The boreholes were drilled and sampled through the overburden to the bedrock surface, which was encountered at depths ranging from 5.8 to 14.8 m (elev. 58.7 to 60.9 m) below the existing ground surface. The bedrock was then cored for drilled lengths ranging from 3.0 to 3.4 m using NQ sized diamond drilling techniques. The final depth of the boreholes ranged from 9.0 to 17.9 m (elev. 55.7 to 57.7 m) below the existing ground surface.

Two 19 mm diameter standpipe piezometers were installed in Boreholes 18-01 and 18-03 to allow for measurements of the groundwater level after completion of drilling. The piezometer installation details are illustrated on the respective Record of Borehole sheets provided in Appendix B. All other boreholes were backfilled with a low-permeability mixture of cuttings and bentonite pellets in general accordance with Ontario MOE Regulation 903. Boreholes advanced within paved areas were capped with granular fill followed by 150 mm of cold patch asphalt to reinstate the travelling surface.

The drilling and sampling operations were supervised on a full-time basis by a member of Thurber's geotechnical staff. The drilling supervisor logged the boreholes and processed the recovered soil and bedrock samples for transport to Thurber's Ottawa laboratory for further examination and testing.

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.

4 LABORATORY TESTING

Geotechnical laboratory testing consisted of visual identification and natural moisture content determination on all retained soil samples. Grain size distribution and Atterberg limits testing were 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), rock quality designation (RQD), and fracture index (FI) were measured. Four samples of the bedrock core were submitted for unconfined compressive strength (UCS) testing. Chemical analysis for determination of pH, conductivity, resistivity, soluble sulphate and chloride concentrations was carried out on four soil samples.

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

5 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 sections. 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 boreholes advanced through the highway embankment encountered pavement structure and embankment fill to a depth of about 6 m. Pavement structure and fill were also encountered in off-road Boreholes 18-01 and 18-03, which were advanced through the parking lot of the City of Ottawa Traffic Operations property. The fill materials, and the topsoil at Borehole 18-02, were underlain by native layers of silty sand and/or glacial till overlying limestone bedrock. A thin discontinuous layer of clay was also encountered underlying the fill in Borehole 18-04.

The subsurface conditions encountered in the previous borehole (BH 16-03) are included on the Record of Borehole sheet in Appendix B, but are not included in the following discussion.

5.1 Topsoil

A surficial layer of topsoil, with a thickness of 150 mm, was encountered at Borehole 18-02.

5.2 Highway Embankment Fill

5.2.1 Asphalt and Concrete

Boreholes 18-04 and 18-05 were drilled through the existing Highway 417 embankment and encountered a layer of asphalt with a thickness of 120 mm overlying a layer of Portland cement concrete that was 240 to 260 mm in thickness.

5.2.2 Embankment Fill: Sand

Embankment fill consisting of sand with trace to some gravel was encountered below the concrete in Boreholes 18-04 and 18-05. The thickness of the sand fill was 3.4 to 4.2 m with a base depth of 3.8 to 4.6 m (elev. 69.9 to 70.0 m) below the existing ground surface.

The SPT tests conducted in the sand fill gave N-values ranging from 7 to 63 blows indicating a loose to very dense state of packing. However, the higher blowcounts within the frost zone (1.8 m) may represent frozen conditions rather than the state of packing of the soil matrix.

Recorded moisture contents ranged from 7 to 15% within the sand fill. The results of grain size analyses conducted on three samples of the sand fill are summarized below and are illustrated on Figure C1 in Appendix C.

Soil Particle	Percentage (%)
Gravel	1 - 11
Sand	80 - 95
Silt and Clay	4 - 9

5.2.3 Embankment Fill: Sand and Gravel

Embankment fill consisting of silty sand and gravel was encountered below the sand fill in Boreholes 18-04 and 18-05. The thickness of the sand and gravel fill was 1.2 to 1.7 m with a base depth of 5.5 to 5.8 m (elev. 68.3 to 68.7 m) below the existing ground surface.

The SPT tests conducted in the sand and gravel fill gave N-values ranging from 10 to 33 blows indicating a loose to dense state of packing.

Recorded moisture contents ranged from 3 to 16% within the sand and gravel fill. The results of grain size analyses conducted on one sample of the sand and gravel fill are summarized below and are illustrated on Figure C2 in Appendix C.

Soil Particle	Percentage (%)
Gravel	43
Sand	40
Silt and Clay	17

5.3 Surrounding Area Fill

5.3.1 Asphalt

Boreholes 18-01 and 18-03 were drilled through the parking lot in front of the crib wall and encountered a layer of asphalt with a thickness ranging from 50 to 60 mm.

5.3.2 Granular Fill

Granular fill was encountered below the asphalt in Boreholes 18-01 and 18-03. The composition of the granular fill ranged from sand and gravel to silty sand. The thickness (and base depth) of the granular fill ranged from about 0.6 to 1.2 m (elev. 65.5 to 70.8 m).

One SPT test was attempted within the granular fill in Borehole 18-03, which gave an N-value of 100 blows per 50 mm of penetration (practical refusal). Within this sample, asphalt was encountered at a depth of 0.8 m. Below 0.8 m depth, significant resistance to augering was encountered to 1.2 m depth, indicating possible buried concrete or cobbles.

Recorded moisture contents ranged from 4 to 18% within the granular fill.

5.3.3 Clayey Sand Fill

Fill consisting of clayey sand, some gravel was encountered below the granular fill in Borehole 18-03. The thickness of this layer was 2.2 m with a base depth of 3.4 m (elev. 63.3 m) below the existing ground surface.

The SPT tests conducted in the clayey sand fill gave N-values ranging from 3 to 7 blows indicating a loose to very loose state of packing.

Recorded moisture contents ranged from 14 to 28% within the clayey sand fill. The results of grain size analyses conducted on one sample of the clayey sand fill are summarized below and are illustrated on Figure C3 in Appendix C.

Soil Particle	Percentage (%)
Gravel	14
Sand	40
Silt	28
Clay	18

Atterberg Limit testing was completed on one sample of the clayey sand fill (with the coarse sand and gravel fractions removed). The results are summarized on the respective Record of Borehole sheet in Appendix B and the plasticity chart included in Figure C8 of Appendix C. The laboratory results are summarized below and indicate that the fines consist of low plasticity clay.

Parameter	Value
Liquid Limit	27
Plastic Limit	15
Plasticity Index	12

5.4 Clay (CH)

A native deposit of clay was encountered below the fill in Borehole 18-04. The thickness of this deposit was 1.2 m with a base depth of 7.0 m (elev. 67.5 m).

The SPT test conducted in the clay gave an N-value of 17 blows indicating a very stiff consistency.

The recorded moisture content was 40%. The results of grain size analyses conducted on one sample of the clay are summarized below and are illustrated on Figure C4 in Appendix C.

Soil Particle	Percentage (%)
Gravel	0
Sand	5
Silt	42
Clay	53

Atterberg Limit testing was completed on one sample of the native clay. The results are summarized on the Record of Borehole sheets in Appendix B and the plasticity chart included in Figure C9 of Appendix C. The laboratory results are summarized below and indicate that the clay has high plasticity (CH).

Parameter	Value
Liquid Limit	56
Plastic Limit	23
Plasticity Index	33

5.5 Silty Sand

A native deposit of silty sand was encountered below the fill in Borehole 18-03. The thickness of this deposit was 1.8 m with a base depth of 5.2 m (elev. 61.5 m) below the existing ground surface. Layers of silty sand were also interbedded within the glacial till (see Section 5.6).

The SPT tests conducted in this deposit gave N-values ranging from 9 to 23 blows indicating a loose to compact state of packing.

Recorded moisture contents ranged from 8 to 19%. The results of grain size analyses conducted on one sample of silty sand are summarized below and are illustrated on Figure C5 in Appendix C.

Soil Particle	Percentage (%)
Gravel	7
Sand	69
Silt and Clay	24

5.6 Silty Sand with Gravel (Glacial Till)

A native deposit of glacial till was encountered below the topsoil in Borehole 18-02, below the fill in Boreholes 18-01 and 18-05, below the clay in 18-04, and below the silty sand in Borehole 18-03. In general, the glacial till consists of silty sand with gravel and contains occasional sand layers, cobbles, and boulders.

The thickness of the glacial till generally ranged from 0.6 to 12.1 m with a base depth ranging from 5.8 to 14.8 m (elev. 58.7 to 60.9 m) below the existing ground surface. In general, the lower portion of the glacial till deposit contained frequent cobbles and boulders and was very dense. Auger refusal was encountered within the glacial till on boulders in Boreholes 18-01 and 18-02; these boreholes were advanced deeper by means of wash boring and diamond drilling.

The SPT tests conducted in the glacial till gave N-values ranging from 6 blows per 300 mm of penetration to 100 blows per 100 mm of penetration indicating a loose to very dense state of packing, but more typically a compact to very dense state of packing. It is noted that the higher blowouts could represent cobbles or a boulder within the glacial till rather than the state of packing of the soil matrix.

Recorded moisture contents ranged from 7 to 18%. The results of grain size analyses conducted on eight samples of the glacial till are summarized below and are illustrated on Figures C6 and C7 in Appendix C. It is noted that one of the tested samples (BH 18-04, SS9) consists primarily of silty sand with trace gravel; however, significant resistance to

augering was encountered in the borehole at this depth, likely representing the presence of cobbles and/or boulders within the deposit and signifying that this is a glacial deposit.

Soil Particle	Percentage (%)	
Gravel	7 - 33	
Sand	39 - 76	
Silt	27 - 31	17 - 32
Clay	10 - 11	

Atterberg Limit testing was completed on two samples of the glacial till (with the coarse sand and gravel fractions removed). The results are summarized on the Record of Borehole sheets in Appendix B and the plasticity chart included in Figure C10 of Appendix C. The laboratory results are summarized below and indicate that the fines within the glacial till are non-plastic (NP) to slightly plastic (ML).

Parameter	Value
Liquid Limit	NP - 13
Plastic Limit	NP - 10
Plasticity Index	NP - 3

5.7 Bedrock

The overburden is underlain by grey limestone bedrock with shale interbeds. A summary of the bedrock surface depth and elevation is summarized in the table below:

Borehole No.	Depth to Bedrock (m)	Bedrock Elevation (m)
18-01	12.7	58.7
18-02	8.5	60.6
18-03	5.8	60.9
18-04	14.8	59.7
18-05	13.3	60.5

The total core recovery ranged from 94% to 100%, the solid core recovery ranged from 53% to 100% and the Rock Quality Designation (RQD) ranged from 30% to 100%. Based on the RQD values the bedrock is classified as poor to excellent quality. In general, the bedrock quality increases with depth. The fracture index ranged from 0 to greater than 5, but more typically 1 to 2, natural fractures per 0.3 m of bedrock core.

Four UCS tests carried out on samples of the bedrock core gave UCS values ranging from about 83 to 120 MPa, indicating that the bedrock is strong to very strong. Photographs of the bedrock core are provided in Appendix C.

5.8 Groundwater

Groundwater levels were measured in the standpipe piezometers installed in Boreholes 18-01 and 18-03. The water level measurements are presented in the table below.

Borehole	Groundwater Level		Date of Measurement
	Depth (mbgs)	Elevation (m)	
18-01	5.2	66.2	February 27, 2018
	5.6	65.8	March 12, 2018
18-03	5.2	61.5	February 28, 2018
	5.2	61.5	March 1, 2018
	5.2	61.5	March 12, 2018

It should be noted that variations 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 snowmelt and/or precipitation events.

5.9 Analytical Testing

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

Borehole	Sample/ Soil Type	Depth (m)	Sulphate (µg/g)	pH (-)	Resistivity (Ohm-cm)	Chloride (µg/g)
18-1	SS4/ Native till	2.3 – 2.9	76	7.98	4,040	63
18-2	SS2/ Native till	1.5 – 2.1	31	7.98	3,160	98
18-4	SS3/ Fill	2.3 – 2.9	143	8.05	1,070	391
18-5	SS4/ Fill	3.1 – 3.7	99	8.02	659	998

6 MISCELLANEOUS

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

George Downing Estate Drilling Ltd. of Hawksbury, Ontario supplied and operated the drilling equipment to conduct the drilling, soil sampling, in-situ testing, piezometer installations, and borehole decommissioning for all the boreholes on site. Beacon Lite Ltd. of Ottawa, Ontario supplied, erected, and dismantled the traffic protection required during the drilling of the on-road boreholes. The field investigation was supervised on a full-time basis by Mr. Justin Gray, E.I.T. and Ms. Katya Edney, P.Eng. of Thurber. Overall supervision of the investigation program was provided by Mr. Paul Carnaffan, 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, and the analytical testing was completed by Paracel Laboratories in Ottawa, Ontario. Interpretation of the factual data and preparation of this report were carried out by Mr. Stephen Dunlop, P.Eng. and Mr. Paul Carnaffan, P.Eng. The report was reviewed by Dr. P.K. Chatterji, P.Eng., a Designated Principal Contact for MTO Foundation Projects.



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PART 2. ENGINEERING DISCUSSION AND RECOMMENDATIONS

7 GENERAL

This report presents the interpretation of the factual data obtained from Part 1 of this report for the proposed replacement of the crib retaining wall along the north side of Highway 417 from Loretta Avenue North to the Canadian Pacific Rail (CPR) / O-Train bridges in Ottawa, Ontario. Geotechnical recommendations are provided to assist the design team in designing a suitable foundation for the proposed retaining wall.

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. The construction or design-build contractor 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 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 the foundation aspects of the new retaining wall. 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.

7.1 Proposed Structure

An evaluation of alternatives for replacement of the existing crib wall was carried out by WSP and presented in a memo to MTO dated January 26, 2018. The alternatives evaluated included:

- a) Cast-in-place concrete gravity wall
- b) Cast-in-place concrete wall on caissons
- c) Post and panel retaining wall

Alternative B (cast-in-place concrete wall on caissons) was identified as the preferred option based on an assessment of factors including cost, schedule, impacts to traffic, durability, and aesthetics. It is noted that a foundation investigation had not been completed at that

time. Foundation alternatives for the proposed wall are discussed and evaluated in Section 9 of this report.

Design drawings indicate that the new retaining wall will be located in front of the existing wall (as close as is practicable) and the existing wall will remain in place permanently behind the new wall. The new wall will consist of cast-in-place concrete and will be about 147.5 m long and 3.0 to 5.3 m in height. The undersides of the foundations are indicated to be 1.0 m below the existing grade to avoid undermining the existing crib wall. No grade changes are proposed at the base of the wall.

7.2 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).

A consequence factor (Ψ) of 1.0 for a "Typical Consequence" classification, as per Table 6.1 of the CHBDC, has been used in assessing factored geotechnical resistances in this report. If the consequence classification changes, the geotechnical assessment will need to be reviewed and revised.

7.3 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, the geotechnical foundation design considerations include:

- From a geotechnical perspective, the native glacial till deposit is capable of supporting shallow foundations with moderate bearing resistance; however, property restraints would necessitate the removal of the existing crib wall and a temporary protection system would be required to support the highway embankment for shallow foundations to be used.
- Deep foundations (caissons) do not require removal of the existing crib wall and are also feasible from a geotechnical perspective, with moderate to high bearing and lateral resistances provided by the glacial till and/or the limestone bedrock.
- The glacial till deposit contains frequent cobbles and boulders. Advancement of caissons will require appropriate equipment, such as down-the hole hammers, to penetrate these obstructions and to reach the design tip elevation.
- The limestone bedrock is strong to very strong and of good to excellent quality. Equipment supplied to construct rock sockets must be capable of excavating the bedrock to the specified socket dimensions without disturbing or fracturing the bedrock forming the sidewalls and base of the socket.
- Groundwater will enter the caisson shafts during construction, which could destabilize the sidewalls of the shafts. Temporary liners may be required to keep the shafts open during advancement.

- The design will need to take into consideration a backfilled sewer trench that transects the site at about Stn. 226+820. This trench contains a 1050 mm diameter sanitary sewer and a 1350 mm diameter storm sewer. It is understood that the trench extends into the bedrock and that the sewers have invert elevations of about 59 m. The existing fill within the trench has not been investigated due to conflicts with the existing sewers (the City utility locators required a wide clearance for borehole drilling); however, it is anticipated that the fill would not be suitable to support shallow foundations. The foundations for the retaining wall will need to span over the sewer trench, likely with deeper rock-socketed caissons on both sides of the trench (i.e., terminated below the base of the trench).

8 SEISMIC CONSIDERATIONS

8.1 CHBDC Seismic Site Classification

In accordance with the CHBDC, the selection of the seismic site classification is based on the soil and bedrock conditions encountered in the upper 30 m of the stratigraphy.

Based on the soil and bedrock conditions encountered below the anticipated pile cap elevation, the site is classified as a Seismic Site Class C in accordance with Table 4.1 of the CHBDC.

8.2 Seismic Liquefaction

The potential for liquefaction of the underlying soils 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 at this site.

9 FOUNDATION DESIGN ALTERNATIVES

9.1 General

Selection of the preferred wall must consider the height of the retained soil, the subsurface conditions, and any space restrictions affecting construction of the wall. Consideration was given to the following wall types/foundations for the proposed retaining wall:

- Caissons (either socketed into bedrock or terminated in the overburden)
- Driven steel piles
- Shallow spread footings
- RSS Wall

These foundation alternatives are presented in the following sections and evaluated from a geotechnical perspective in terms of their respective advantages, disadvantages, risks/consequences and relative costs. The evaluation is summarized in the table provided in Appendix E. A preferred foundation from a geotechnical engineering perspective is recommended.

9.2 Caissons

The new retaining wall could be founded on caissons, which are drilled/bored into the ground and derive their support from either the bedrock or the overburden. This option would allow for the existing crib wall to remain in place with all of the construction taking place at the base of the existing wall. The existing embankment and highway traffic flow would not be affected. It also provides high axial and lateral resistance, particularly if rock-socketed caissons are used. Caisson installations can also penetrate very dense and bouldery soils if appropriate equipment is used. The primary disadvantage of this option is that a specialized contractor will be required, with specialized drilling equipment to penetrate the very dense bouldery glacial till at this site, which will increase costs.

If caissons are used, there is also a risk that groundwater inflow could destabilize the shaft walls and base. This could result in settlement of the existing crib wall (due to loss of soil from the sidewalls) and/or insufficient bearing resistance (due to unconsolidated sediments accumulating at the toe of the caisson). To limit the potential negative effects of groundwater inflow, temporary liners will be required to support the sidewalls of the shafts. If basal 'blow-up' is also an issue due to an unbalanced hydrostatic head, a slurry or head of water will need to be maintained in the shaft during excavation to balance the hydrostatic force.

9.3 Driven Steel Piles

The new retaining wall could be founded on steel piles that are driven to refusal on the bedrock or the very dense bouldery glacial till. This option would allow for the existing crib wall to remain in place with all of the construction taking place at the base of the wall. The existing embankment and highway traffic flow would not be affected. It also provides high axial resistance. The primary disadvantage of this option is that the cobbles and boulders within the glacial till are likely to interfere with pile driving. Some piles may become damaged and need to be wasted. There is also a potential for refusals to be shallower than intended, resulting in design changes at the time of construction and the potential for supplemental piles. In addition, the lateral resistance of driven steel piles is much less than caissons. If this option were chosen, H piles would be recommended. In comparison, pipe piles are more likely to be damaged during pile driving in the very dense bouldery glacial till.

Vibrations from pile driving also have the potential to cause damage/movement of nearby structures, which would need to be monitored during construction. For example, structures at risk include: the City Traffic Operations building, nearby residential houses, the existing City sewers that transect the site, and the existing crib wall (and therefore the highway embankment).

9.4 Shallow Spread Footings

The new retaining wall could be supported on shallow spread footings bearing on the native glacial till (i.e., a cantilever retaining wall). Spread footings do not require a specialist contractor and are typically more cost effective than deep foundations. However, in this case there is not sufficient space at the site to install a cantilever retaining wall without removing the existing crib wall; therefore, this option would require excavating the existing embankment. A temporary protection system would be required to maintain some of the westbound traffic lanes of the highway, which would significantly increase the cost of this option. Significant traffic impacts would be necessary to install the protection system. The spread footing option is not well suited to spanning across the existing sewer trenches.

9.5 RSS Wall

The new retaining wall could consist of a retained soil system (RSS) wall. RSS walls do not require extensive concrete foundations and are flexible structures with more tolerance for differential settlement. They are also generally a cost-effective option, particularly for higher walls. However, in this case there is not sufficient space at the site to install an RSS wall without removing the existing crib wall; therefore, this option would require excavating the existing embankment. A temporary protection system would be required to maintain some of the westbound traffic lanes of the highway, which would significantly increase the cost of this option. Significant traffic impacts would be necessary to install the protection system.

9.6 Recommended Foundation

For this project it will be advantageous from a cost perspective to construct the new retaining wall in front of the existing crib wall to prevent an excavation from extending into the existing highway embankment, which would require an expensive temporary protection system. However, space constraints due to the existing property boundaries only permit options that can be constructed within a narrow corridor. Therefore, the preferred option is a cast-in-place concrete retaining wall supported by deep foundations. From a geotechnical perspective, the recommended deep foundation option is drilled/bored caissons, which derive their support from either the bedrock or the overburden.

Rock sockets are likely required within the eastern part of the wall alignment where the depth to bedrock is less and the overburden included extensive fill material. Rock sockets may not be required in the western part of the alignment where a thick dense glacial till deposit is present above the bedrock. However, it should be noted that differential settlement of up to 25 mm should be expected at the location where the foundation type changes from rock-socketed caissons to caissons terminating in glacial till. In that case, consideration may need to be given to using an expansion joint or, if that is not sufficient, to using a consistent foundation type (rock-socketed caissons) for the entire length of wall.

10 FOUNDATION DESIGN RECOMMENDATIONS

10.1 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 pile caps to serve as frost protection.

In addition, 1.8 m of non frost-susceptible backfill, or equivalent insulation, will need to be placed behind the new wall, since it cannot be confirmed that the soils within the existing crib wall are not frost-susceptible. Otherwise, lateral frost pressures may be applied to the new retaining wall.

10.2 Caissons

10.2.1 Axial Compression

The caissons can be designed to either terminate in the glacial till or the limestone bedrock.

Caissons that terminate in the glacial till will derive their support from a combination of shaft friction and end-bearing, both of which are a function of the in situ effective stress, which increases with depth and the diameter of the pile. The drawings available indicate that the piles will be 1070 mm in diameter. Caissons terminating in the glacial till can be designed based on the following geotechnical resistances for axial compression:

Axial Compression Geotechnical Resistances for Caissons Terminating in Till

Caisson Diameter (m)	Pile Length (m)	Factored Geotechnical Resistance at ULS (kN) (Axial Compression) ($\phi_{gu}=0.4$)	Factored Geotechnical Resistance at SLS (kN) (Axial Compression) ($\phi_{gs}=0.8$)
1.07	6.0	1,100	400
	7.0	1,250	500
	8.0	1,400	600

The geotechnical resistance values at SLS provided above correspond to a total settlement of 25 mm at the top of the caisson. This total settlement includes both elastic shortening of the concrete and settlement of the foundation soils.

Piles should be designed to be no shorter than 6.0 m in length. Based on the stratigraphy observed in the boreholes, these resistances would only apply to caissons located west of Borehole 18-05. Caissons located to the east of Borehole 18-05 will need to derive their axial support from the bedrock (i.e., rock socketed caissons). Caissons adjacent to the existing sewer trench should be designed so that the rock sockets start at the bottom of the sewer trench since the rock within the walls of the trench may be highly fractured.

The socket length of rock-socketed caissons may be relatively short if the purpose is solely to provide axial resistance. However, if the rock socket is also required to provide lateral resistance and/or fixity, a longer rock socket will likely be required. To provide full fixity, it is recommended that the design use rock socketed caissons with a minimum socket length of twice the caisson diameter (2.14 m) into sound bedrock. The upper 0.5 m of rock should be assumed to be weathered and neglected in the determination of socket length (i.e., for full fixity the caissons should be socketed a minimum of 2.64 m into the bedrock). The actual socket length required should be determined based on the required lateral capacity and moment capacity requirements. A deeper socket length may therefore be required. Under no circumstances should the rock-socket length be less than 1.0 m.

The limestone bedrock was measured to have an unconfined compressive strength ranging from approximately 83 to 120 MPa, indicating that the rock is strong to very strong. Below a surficial weathered zone of the bedrock, the Rock Quality Designation (RQD) results also classified the rock as good to excellent quality. An NSSP to alert the Contractor is provided.

Rock socketed caissons at this site may be designed based on the following factored geotechnical resistances for axial compression:

Axial Compression Geotechnical Resistances for Rock Socketed Caissons

Caisson Diameter (m)	Socket Length (m)	Factored Geotechnical Resistance at ULS (kN) (Axial Compression) ($\phi_{gu}=0.4$)	
		Shaft Resistance	End-Bearing Resistance
1.07	1.0	1,000	See Note**
	1.5	2,000	
	2.0	3,000	
	2.64	4,300	
	3.0	5,000	

**Note – Rock socketed caissons on this site would have theoretical factored end-bearing resistances in excess of 10,000 kN at ULS; however, for end-bearing resistance to be relied upon the base of the rock socket will need to be cleaned thoroughly and inspected prior to the placement of concrete, which can be challenging and impractical if water is present in the shaft. In that case, axial displacement (settlement) of the caissons would occur prior to engaging the end-bearing resistance. It is difficult to predict the magnitude of this settlement and therefore it is recommended that end-bearing resistance be neglected in the design (i.e., only shaft resistance be relied upon).

10.2.2 Lateral Resistance/Deflection

The lateral resistance in the soils above the bedrock may be calculated using p-y curves. The p-y curves are shown in Appendix F to allow for the calculation of the ultimate lateral capacity of an individual caisson. The values provided in Appendix F are unfactored. A reduction factor of 0.5 should be applied to these ultimate values in accordance with Table 6.2 of the CHBDC.

For caissons that are located between borehole locations, careful interpretation of the p-y data will be required. Within the soil the p-y curves are a function of depth, soil type, and the groundwater level. It is recommended that the p-y data from BH 18-03 be used for all caissons east of BH 18-02 (i.e., all caissons between 18-02 and 18-03 and easterly). Similarly, the p-y data from BH 18-02 should be used for caissons between BH's 18-01 and 18-02, and the p-y data from BH 18-01 for all caissons west of BH 18-01.

Within the bedrock, the factored lateral resistance at ULS is constant with depth and for a 1.07 m diameter caisson can be taken as 4,000 kN/m length of caisson into sound bedrock (neglecting the upper 0.5 m of weathered bedrock).

Where lateral spacing between an adjacent pile or another structural element is less than four equivalent pile diameters, the lateral resistance will need to be reduced based on the center-to-center spacing. The reduction factors to be used can be obtained from Figures C6.11.3(r), C6.11.3(s) and C6.11.3(t) of the CHBDC.

10.2.3 Caisson Installation

Caisson installation must be in accordance with OPSS 903.

The caisson drilling equipment supplied by the Contractor must be capable of advancing through the existing soils and penetrating or pushing aside potential obstructions. The drilling equipment must be able to penetrate cobbles and boulders within very dense glacial till, as well as strong to very strong limestone of generally good to excellent quality.

Construction of rock-socketed caissons will require the use of a steel liner advanced into the upper bedrock to support the sidewalls within the overburden, minimize groundwater inflow and enable machine-cleaning of the socket base. After the liner is installed, the bedrock socket must be advanced to found the caisson in sound bedrock with a minimum embedment length as recommended in Section 10.2.1.

Construction of caissons terminating in the overburden will also require the use of a steel liner to support the sidewalls of the shafts. If basal 'blow-up' is also an issue due to an unbalanced hydrostatic head, a slurry or head of water will need to be maintained in the shaft during excavation to balance the hydrostatic force.

An NSSP notifying the Contractor of the specific subsurface conditions and installation requirements at this site should be included in the contract documents. Suggested wording is presented in Appendix G. Selection of the type of equipment and method of installation is the responsibility of the Contractor.

10.3 Lateral Earth Pressures

The lateral earth pressure acting on the new retaining wall will be influenced by:

- The narrow zone of backfill between the new wall and existing crib wall, and
- The backfill within and behind the existing crib wall.

The type of backfill that is used between the two walls will not govern the design since only a narrow gallery of soil backfill will be present. Clear crushed stone will be an acceptable backfill soil, noting that it should be nominally compacted (if space permits). Granular backfill such as Granular A and Granular B Type II, should only be used if space permits these materials to be compacted to 95 percent of the material's standard Proctor maximum dry density.

The lateral earth pressure parameters provided in this section are based on the following assumptions:

- The backfill is fully drained so that there are no unbalanced hydrostatic pressures.
- The backfill behind the wall is sloped at angles ranging from 3H:1V to 2H:1V and the height of soil above the wall is no greater than 1.0 m. If a different slope angle or height is proposed, the earth pressure parameters provided in this section will need to be revised.

10.3.1 Static Lateral Earth Pressure Coefficients

Lateral earth pressures acting on the retaining wall should be computed in accordance with the CHBDC, but are generally given by the expression:

$$P_h = K \cdot (\gamma \cdot h + q)$$

where:

P_h	=	horizontal pressure on the wall (kPa)
K	=	earth pressure coefficient
γ	=	unit weight of retained soil (kN/m ³)
h	=	depth below top of wall 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 the table below.

Static Lateral Earth Pressure Parameters	Design Value
Soil Unit Weight, kN/m ³ , γ	22
Angle of Internal Friction, ϕ	30°
Coefficient of At-Rest Earth Pressure, K_o (Restrained Wall) Assumes a flat backslope; sloping backfill to be added as a surcharge	0.50
Coefficient of Active Earth Pressure, K_a (Unrestrained Wall) Assumes a 2H:1V backslope	0.54
Coefficient of Active Earth Pressure, K_a (Unrestrained Wall) Assumes a 3H:1V backslope	0.43

For rigid structures, it is recommended that at-rest 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.

10.3.2 Combined Static and Seismic Lateral Earth Pressure Parameters

In accordance with Clause 4.6.5 of the CHBDC (S6-14), a structure should be designed using a dynamic earth pressure coefficient that incorporates the effects of earthquake loading. The following recommendations are as per Section C4.6.5 of the Commentary of the CHBDC which states that seismically induced lateral soil pressures may be calculated using the Generalized Limit Equilibrium (GLE) method with:

- $k_h = \frac{1}{2} * F(PGA) * PGA$, for structures that allow 25 to 50 mm of movement, and
- $k_h = F(PGA) * PGA$, for non-yielding walls

The ratio of wall movement to wall height required to mobilize the active conditions would be approximately 0.002 for a yielding structure with respect to the assessment of seismically induced lateral earth pressures.

The coefficients of horizontal earth pressure for seismic loading presented in the table below may be used. The provided earth pressure coefficients are based on a **Seismic Site Class C**, PGA with a 2% probability of exceedance in 50 years of 0.28g (Geological Survey of Canada – Fifth Generation) and a $F(PGA)$ of 1.00 as per Table 4.8 of the CHBDC (S6-14 update No. 1, April 2016).

Seismic Lateral Earth Pressure Parameters	Design Value
Coefficient of Active Earth Pressure, K_{AE} (Restrained Wall) Assumes a 2H:1V backslope with a height of 1.0 m	0.85
Coefficient of Active Earth Pressure, K_{AE} (Unrestrained Wall) Assumes a 2H:1V backslope with a height of 1.0 m	0.65
Coefficient of Active Earth Pressure, K_{AE} (Restrained Wall) Assumes a 3H:1V backslope with a height of 1.0 m	0.81
Coefficient of Active Earth Pressure, K_{AE} (Unrestrained Wall) Assumes a 3H:1V backslope with a height of 1.0 m	0.63

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:

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

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)

11 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. An 'S' exposure class is therefore not applicable to this site.

The pH, resistivity and chloride concentration provide an indication of the degree of corrosiveness of the sub-surface environment. The tests results provided in Section 5.9 may be used to aid in the selection of coatings and corrosion protection systems for buried steel objects. Based on criteria from the California Department of Transportation (Caltrans), one of the tested soil samples (BH 18-5, SS4) would be characterized as corrosive.

12 CONSTRUCTION CONSIDERATIONS

12.1 Excavations

All excavations must be conducted in accordance with the requirements of the Occupational Health & Safety Act & Regulations (OHSA) for Construction Projects. The fills and glacial till above the water level at the site should be classified as Type 3 in accordance with OHSA.

Subgrade preparation and placement of the backfill and pile caps 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. Cobbles and boulders were observed in the boreholes; a NSSP alerting bidders to their presence has been provided in Appendix G.

12.2 Dewatering

All excavations for foundation construction must be dewatered prior to the placement of concrete, as per OPSS 902 and NSSP FOUN0003.

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 open excavations.

Dewatering design and decisions regarding dewatering, must be carried out by the Contractor. Due to the shallow excavation depths being considered and the depth to groundwater at the site it is anticipated that conventional sump and pump techniques should be sufficient, therefore a preconstruction survey is not required and Designer Fill-in ** of FOUN0003 is not applicable (N/A). In addition, a temporary flow control system is not required for this project, therefore Design Fill-in * of FOUN0003 is not applicable (N/A).

12.3 Erosion Protection

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

12.4 Construction Concerns

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

- Cobbles and boulders, or other obstructions may be encountered within the fill and glacial till deposits. Recommended wording for an NSSP alerting the Contractor to this condition and the requirement to use appropriate equipment and techniques to penetrate the obstructions is provided in Appendix G.
- The limestone bedrock is strong to very strong and of good to excellent quality. The contractor will need to provide appropriate equipment to drill the bedrock for the rock-socketed caissons.
- Groundwater will enter the caisson shafts during construction, which will likely destabilize the sidewalls of the shafts. Temporary liners will be required to keep the shafts open during advancement. Basal 'blow-up' could also occur due to an unbalanced hydrostatic head. In that case, a slurry or head of water will need to be maintained in the shaft during excavation to balance the hydrostatic force.
- If end-bearing support for the caissons is required, the base of the rock socket will need to be cleaned and inspected, which can be challenging, particularly for deep shafts that are filled with water. In that case, the contractor will need to supply appropriate cleaning and dewatering equipment to allow for proper inspection of the base. Additional inspection equipment, such as down-hole cameras, may also be required.
- Confirmation that the granular backfill is adequately placed and compacted to specifications.

The successful performance of the retaining wall will depend largely upon good workmanship and quality control during construction. Observation of the caisson installations, excavations and backfilling operations will be required during construction to confirm that the foundation recommendations are correctly implemented and material specifications are met.

13 CLOSURE

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

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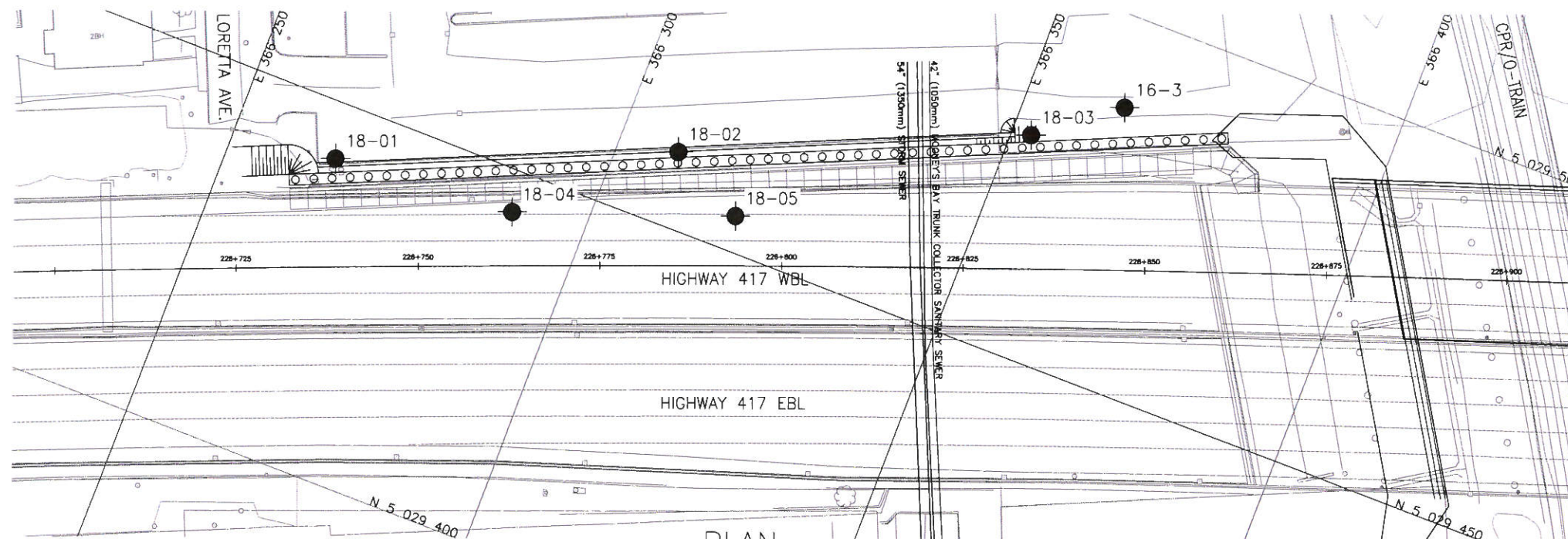


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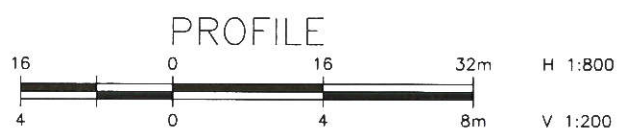
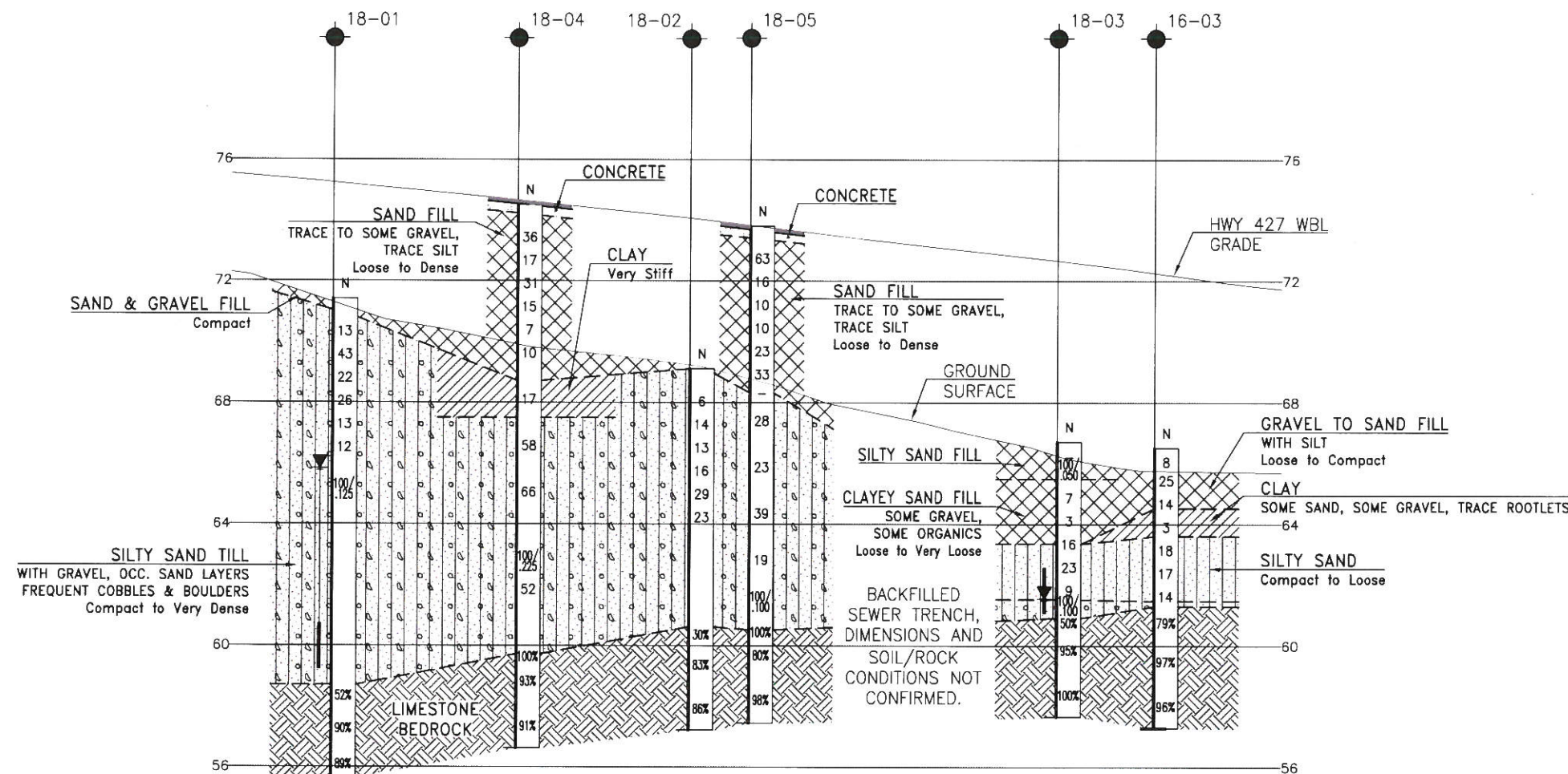
RETAINING WALL FROM LORETTA AVENUE NORTH
TO THE CPR/O-TRAIN BRIDGES
HIGHWAY 417, OTTAWA

Appendix A.

Borehole Location Plan and Stratigraphic Drawing



PLAN

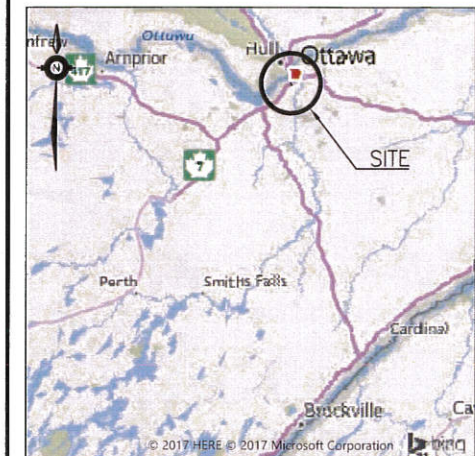


PROFILE

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

CONT No
GWP No 4245-05-00

HIGHWAY 417
CPR/O-TRAIN
LORETTA AVE. RETAINING WALL
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
- W Water Level
- ↑ Head Artesian Water
- ⊕ Piezometer
- 90% Rock Quality Designation (RQD)
- A/R Auger Refusal

NO	ELEVATION	NORTHING	EASTING
16-3	66.5	5 029 487.8	366 363.4
18-01	71.4	5 029 442.5	366 264.5
18-02	69.1	5 029 460.2	366 308.1
18-03	66.7	5 029 479.7	366 352.7
18-04	74.5	5 029 444.4	366 289.8
18-05	73.8	5 029 454.8	366 318.6

-NOTES-

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

GEOCRES No. 31G5-294



REVISIONS	DATE	BY	DESCRIPTION
DESIGN	SD	CHK PC	CODE
DRAWN	AN	CHK SD	SITE
			LOAD
			STRUCT
			DWG 1
			DATE OCT 2018

RETAINING WALL FROM LORETTA AVENUE NORTH
TO THE CPR/O-TRAIN BRIDGES
HIGHWAY 417, OTTAWA

Appendix B.

Record of Borehole Sheets



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 16-03

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								
66.5																
0.0																
0.1	ASPHALT 100 mm															
	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																
2.0	CLAY (CL) some sand, trace rootlets Soft Grey		4	SS	3		64								3 21 42 34	
63.6																
2.9	SILTY SAND (SM) Compact Brown to Grey		5	SS	18		63									
			6	SS	17										0 70 30 (SI+CL)	
			7	SS	14		62									
61.5																
60.9	SILTY SAND with gravel TILL														FI	
5.2	LIMESTONE BEDROCK with some shale interbedding Grey Fresh Very Thinly Bedded		1	RUN			61								2 1 1 1 2 2 1 1 0 1 1 0 1	
			2	RUN			60								2 1 1 0 1 1 0 1	
			3	RUN			59								1 1 0 1 1 0 1	
57.3							58								1 1 0 1 1 0 1	
9.2	End of Borehole														1	

ONTMT4S 11189 - HWY 417 O-TRAIN GPJ 2012TEMPLATE(MTO).GDT 3/4/18

+³, ×³: Numbers refer to
Sensitivity

20
15
10

(%) STRAIN AT FAILURE

METRIC

Continued Next Page

+³, ×³: Numbers refer to Sensitivity

METRIC

+³, ×³: Numbers refer to Sensitivity

RECORD OF BOREHOLE No 18-02

1 OF 2

METRIC

GWP# 4245-05-00 LOCATION Loretta Crib Wall, MTM z9: N 5 029 460.2 E 366 308.1 ORIGINATED BY KE
HWY 417 BOREHOLE TYPE HSA / NW Casing / NQ Coring COMPILED BY CM
DATUM Geodetic DATE 2018.02.21 - 2018.02.25 CHECKED BY SD

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					
								20 40 60 80 100					
69.1													
0.0	150 mm Topsoil												
0.2	Silty SAND with Gravel, occasional Sand layers, occasional Cobbles and Boulders TILL Loose to Compact Grey-Brown		1	SS	6								15 54 31 (SH+CL)
			2	SS	14								
			3	SS	13								
			4	SS	16								33 39 28 (SH+CL)
			5	SS	29								
			6	SS	23								
63.7	- Auger Refusal at 5.3 m, borehole advanced with casing and coring techniques												
5.3	Silty SAND with Gravel, Frequent Cobbles and Boulders TILL Very Dense - 400 mm Boulder at 5.3 m												

Continued Next Page

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

ONTMT4S 11189 - HWY 417 O-TRAIN LORETTA WALL.GPJ 2012TEMPLATE(MTO).GDT 3/4/18

RECORD OF BOREHOLE No 18-02

2 OF 2

METRIC

GWP# 4245-05-00 LOCATION Loretta Crib Wall, MTM z9: N 5 029 460.2 E 366 308.1 ORIGINATED BY KE
 HWY 417 BOREHOLE TYPE HSA / NW Casing / NQ Coring COMPILED BY CM
 DATUM Geodetic DATE 2018.02.21 - 2018.02.25 CHECKED BY SD

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
								20 40 60 80 100						
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE						
Continued From Previous Page							WATER CONTENT (%)				20 40 60			
57.2 <														

ONTMT4S 11189 - HWY 417 O-TRAIN LORETTA WALL.GPJ 2012TEMPLATE(MTO).GDT 3/4/18

RECORD OF BOREHOLE No 18-03

1 OF 1

METRIC

GWP# 4245-05-00 LOCATION Loretta Crib Wall, MTM z9: N 5 029 479.7 E 366 352.7 ORIGINATED BY JAG
 HWY 417 BOREHOLE TYPE HSA / NW Casing / NQ Coring COMPILED BY CM
 DATUM Geodetic DATE 2018.02.19 - 2018.02.20 CHECKED BY SD

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					
66.7													
66.7 0.1	60 mm ASPHALT												
	Silty SAND FILL Frozen Brown - Asphalt in SS2 and hard augering from 0.8 m to 1.2 m (possible concrete or cobbles)		1	BS	-								
			2	SS	100/ 50mm								
65.5													
1.2	Clayey SAND, some Gravel, some Organics FILL Loose to Very Loose Grey-Brown		3	SS	7								14 40 28 18
			4	SS	3								
			5	SS	16								
63.3													
3.4	Silty SAND Compact to Loose Brown		6	SS	23								7 69 24 (SI+CL)
			7	SS	9								
61.5													
5.2	Silty SAND with Gravel TILL Very Dense Grey		8	SS	100/ 100mm								28 40 32 (SI+CL)
60.9													
5.8	LIMESTONE BEDROCK Nodular with Shale Seams/Beds Slightly Weathered to Fresh Thinly Bedded Strong to Very Strong Grey		1	RUN									RUN #1 TCR=100% SCR=100% RQD=50% UCS = 120 MPa
			2	RUN									RUN #2 TCR=100% SCR=98% RQD=95%
			3	RUN									RUN #3 TCR=100% SCR=100% RQD=100%
57.7													
9.0	End of Borehole												
	DATE DEPTH (m) ELEV. (m)												
	2018.02.28 5.2 61.5												
	2018.03.01 5.2 61.5												
	2018.03.12 5.2 61.5												

ONTMT4S 11189 - HWY 417 O-TRAIN LORETTA WALL.GPJ 2012TEMPLATE(MTO).GDT 3/4/18

+³, ×³: Numbers refer to
Sensitivity

20
15
10
(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 18-04

1 OF 2

METRIC

GWP# 4245-05-00 LOCATION Loretta Crib Wall, MTM z9: N 5 029 444.4 E 366 289.8 ORIGINATED BY JAG
 HWY 417 BOREHOLE TYPE NW Casing / NQ Coring COMPILED BY CM
 DATUM Geodetic DATE 2018.02.14 - 2018.02.15 CHECKED BY SD

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa		WATER CONTENT (%)				
74.5							20 40 60 80 100							
0.0	120 mm ASPHALT													
0.1	240 mm CONCRETE													
74.1														
0.4	SAND trace to some Gravel, trace Silt FILL Loose to Dense Brown		1	SS	36									
			2	SS	17									1 95 4 (SH+CL)
			3	SS	31									
			4	SS	15									11 80 9 (SH+CL)
	- No recovery in SS5		5	SS	7									
69.9			6	SS	10									
4.6	Silty SAND and GRAVEL FILL Compact Grey													
68.7														
5.8	CLAY (CH) Very Stiff Grey-Brown		7	SS	17									0 5 42 53
67.5														
7.0	Silty SAND with Gravel, occasional Sand layers, occasional Cobbles and Boulders TILL Very Dense Grey		8	SS	58									
			9	SS	66									7 76 17 (SH+CL)

Continued Next Page

+³, ×³: Numbers refer to
Sensitivity

20
15
10

(%) STRAIN AT FAILURE

ONTMT4S 11189 - HWY 417 O-TRAIN LORETTA WALL.GPJ 2012TEMPLATE(MTO).GDT 3/4/18

METRIC

[illegible]

+³, ×³: Numbers refer to Sensitivity

RECORD OF BOREHOLE No 18-05

1 OF 2

METRIC

GWP# 4245-05-00 LOCATION Loretta Crib Wall, MTM z9: N 5 029 454.8 E 366 318.6 ORIGINATED BY JAG
 HWY 417 BOREHOLE TYPE NW Casing / NQ Coring COMPILED BY CM
 DATUM Geodetic DATE 2018.02.13 - 2018.02.13 CHECKED BY SD

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									
73.8								20	40	60	80	100					
0.0	120 mm ASPHALT																
0.1	260 mm CONCRETE																
73.4																	
0.4	SAND, trace to some Gravel, trace Silt FILL Loose to Dense Brown		1	SS	63		73										
			2	SS	16		72										
			3	SS	10		71									10 84 6 (SI+CL)	
			4	SS	10												
70.0							70										
3.8	Silty SAND and GRAVEL FILL Compact to Dense Grey		5	SS	23											43 40 17 (SI+CL)	
			6	SS	33		69										
68.3			7	NQ	-		68										
5.5	Silty SAND with Gravel, occasional Sand layers, occasional Cobbles and Boulders TILL Compact to Dense Brown to Grey		8	SS	28		67										
			9	SS	23		66									16 47 27 10 non-plastic	
			10	SS	39		65										
							64										

Continued Next Page

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

ONTMT4S 11189 - HWY 417 O-TRAIN LORETTA WALL.GPJ 2012TEMPLATE(MTO).GDT 3/4/18

RECORD OF BOREHOLE No 18-05

2 OF 2

METRIC

GWP# 4245-05-00 LOCATION Loretta Crib Wall, MTM z9: N 5 029 454.8 E 366 318.6 ORIGINATED BY JAG
 HWY 417 BOREHOLE TYPE NW Casing / NQ Coring COMPILED BY CM
 DATUM Geodetic DATE 2018.02.13 - 2018.02.13 CHECKED BY SD

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		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT				
							W P W L							
							WATER CONTENT (%)							
							20 40 60 80 100				20 40 60			
							○ UNCONFINED + FIELD VANE							
							● QUICK TRIAXIAL × LAB VANE							

ONTMT4S 11189 - HWY 417 O-TRAIN LORETTA WALL.GPJ 2012TEMPLATE(MTO).GDT 3/4/18

RETAINING WALL FROM LORETTA AVENUE NORTH
TO THE CPR/O-TRAIN BRIDGES
HIGHWAY 417, OTTAWA

Appendix C.

Laboratory Testing

RETAINING WALL FROM LORETTA AVENUE NORTH
TO THE CPR/O-TRAIN BRIDGES
HIGHWAY 417, OTTAWA

Appendix C.1

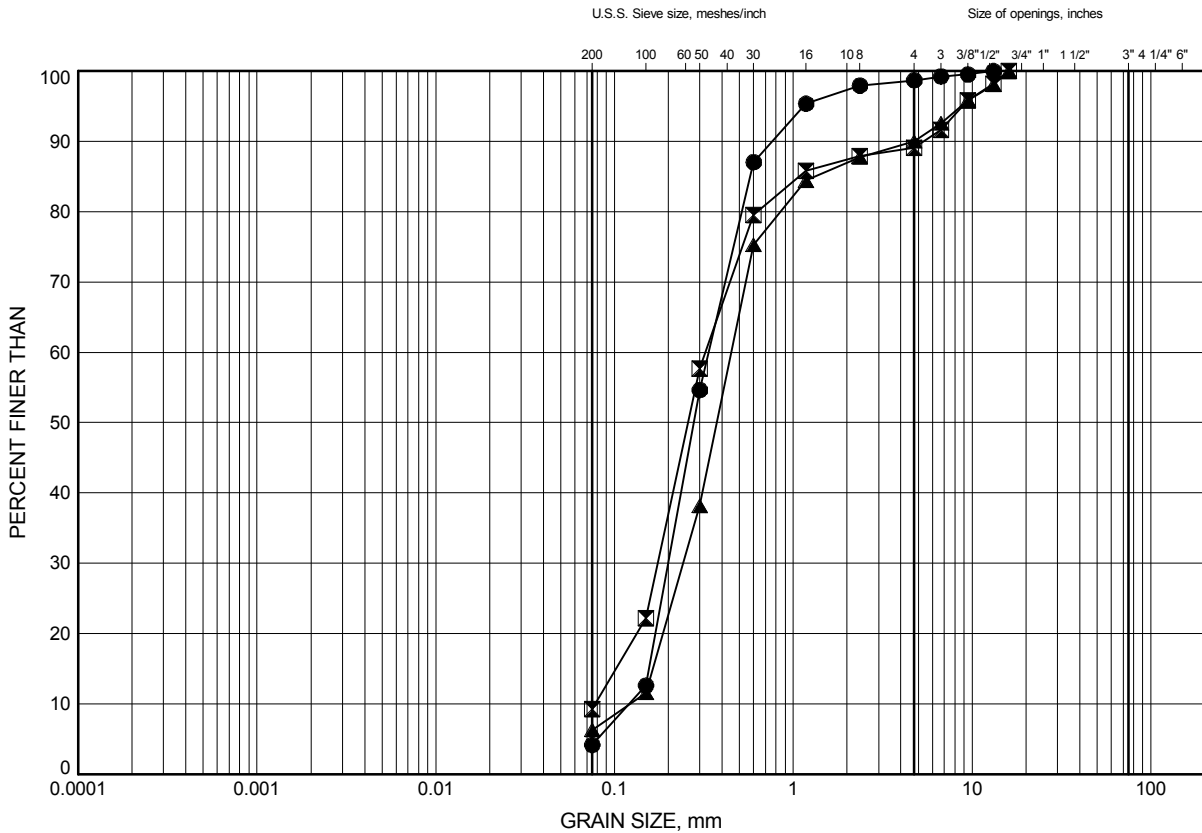
Grain Size Distribution Figures

Hwy 417 O-Train - Loretta Crib Wall

GRAIN SIZE DISTRIBUTION

FIGURE C1

Embankment Fill - Sand



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	18-04	1.83	72.67
⊠	18-04	3.35	71.15
▲	18-05	2.59	71.21

Date March 2018
GWP# 4245-05-00



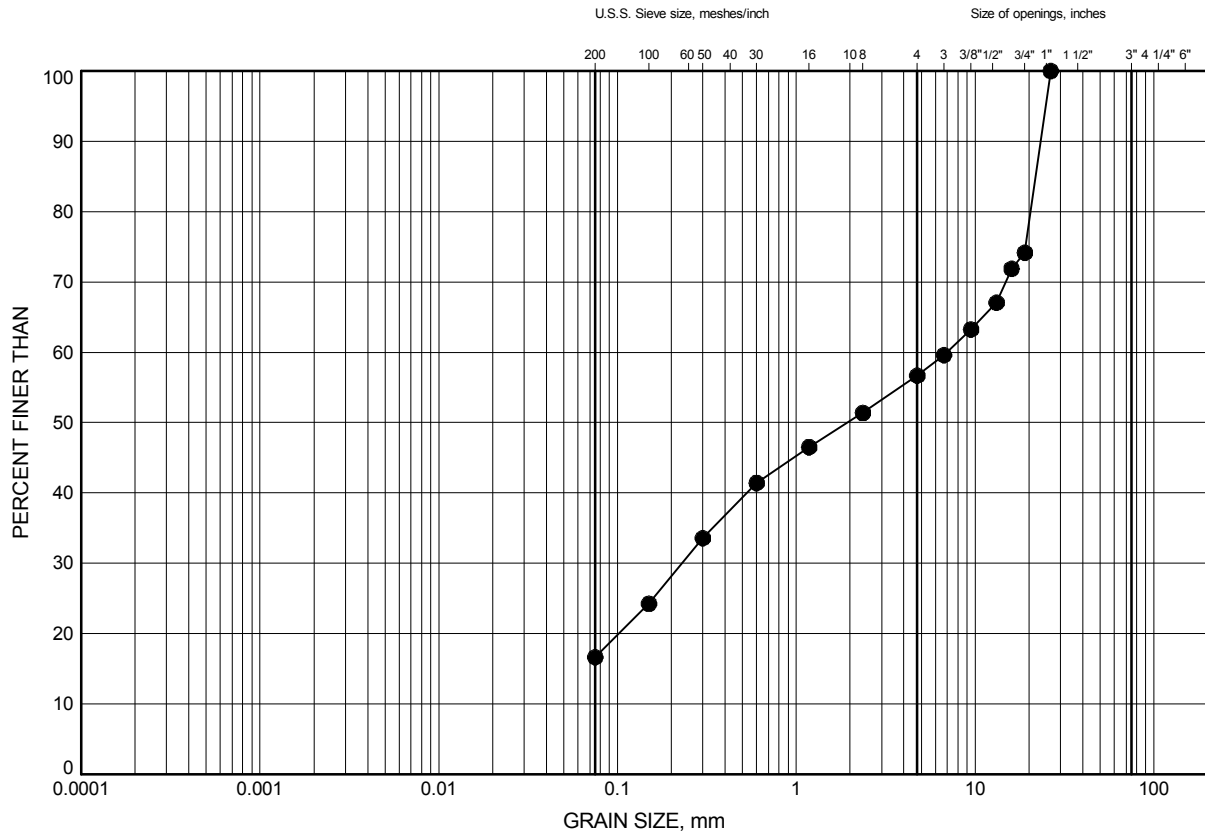
Prep'd CM
Chkd. SD

Hwy 417 O-Train - Loretta Crib Wall

GRAIN SIZE DISTRIBUTION

FIGURE C2

Embankment Fill - Silty Sand and Gravel



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	18-05	4.11	69.69

Date March 2018
GWP# 4245-05-00



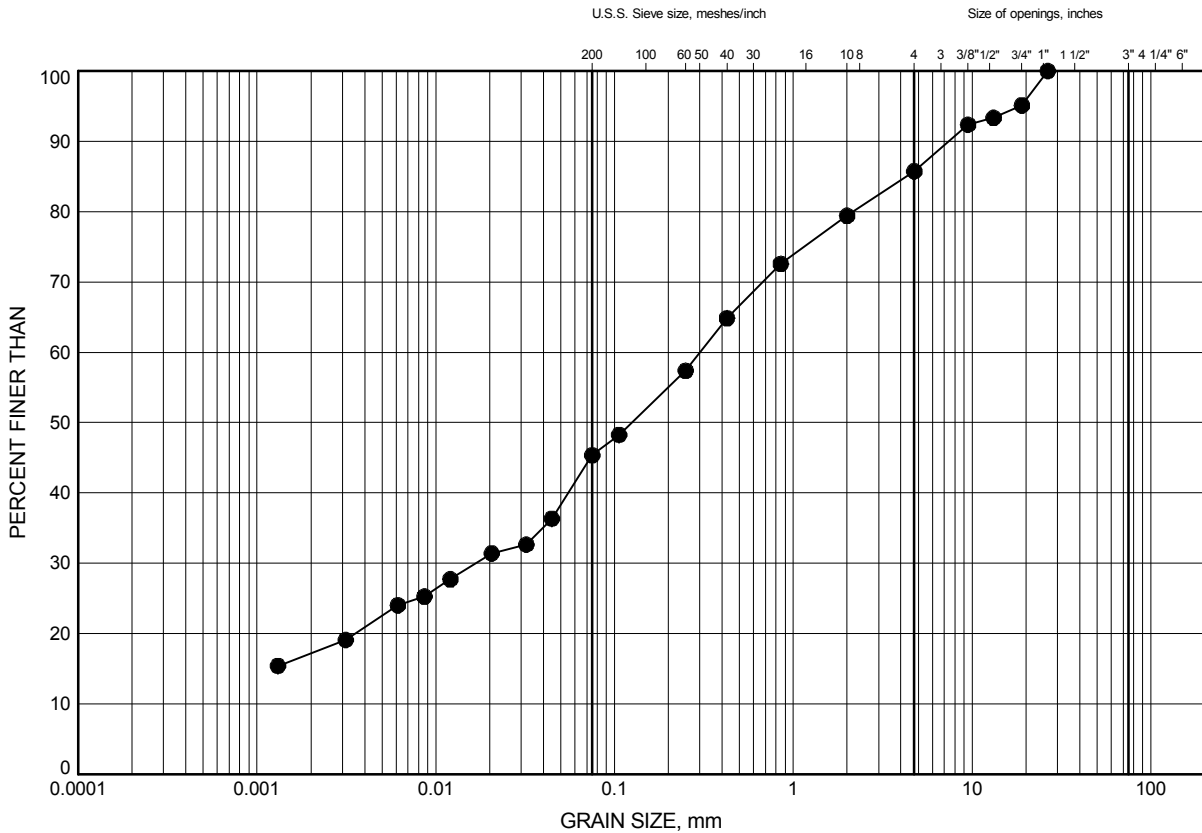
Prep'd CM
Chkd. SD

Hwy 417 O-Train - Loretta Crib Wall

GRAIN SIZE DISTRIBUTION

FIGURE C3

Fill - Clayey Sand



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	18-03	1.83	64.87

Date March 2018
GWP# 4245-05-00

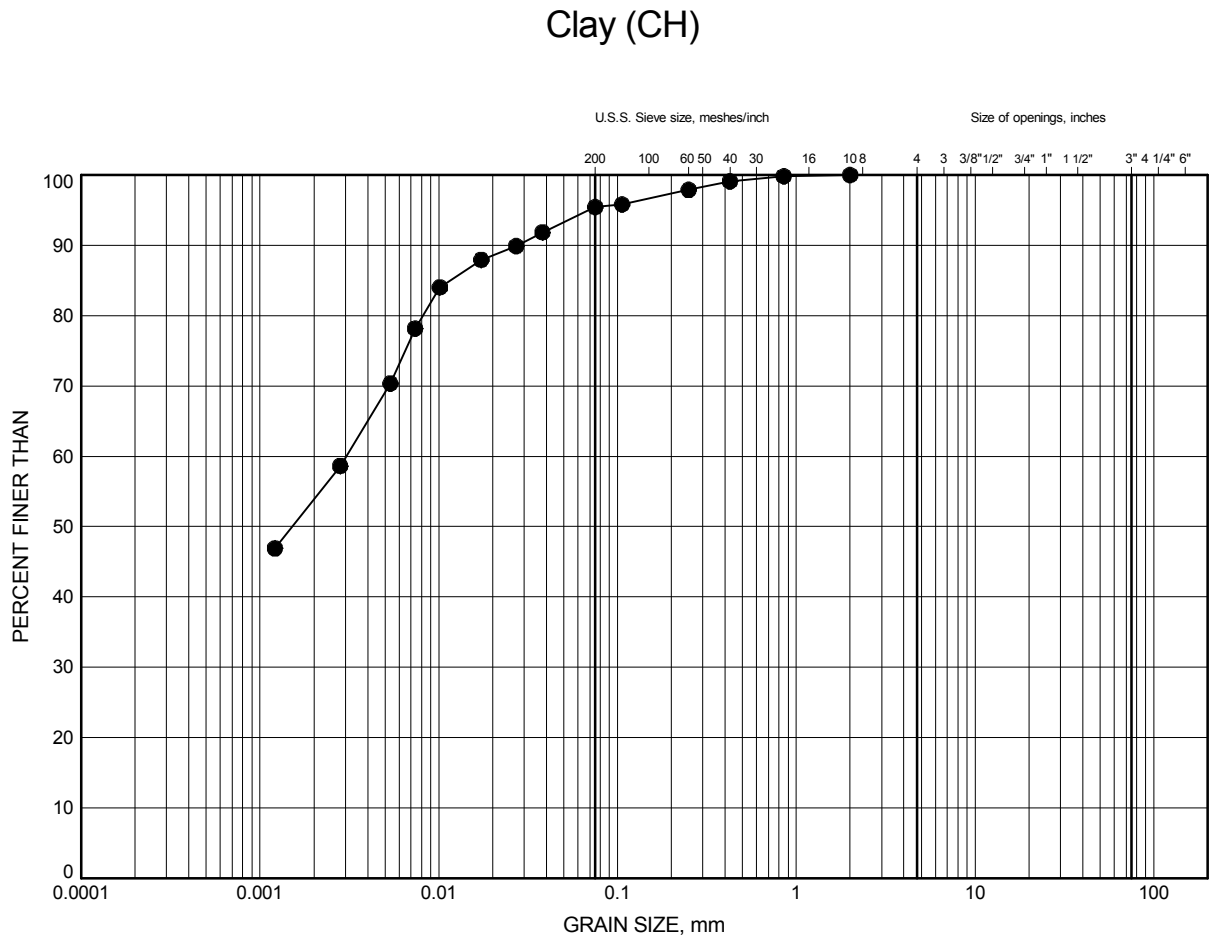


Prep'd CM
Chkd. SD

Hwy 417 O-Train - Loretta Crib Wall

GRAIN SIZE DISTRIBUTION

FIGURE C4



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	18-04	6.40	68.10

Date March 2018
GWP# 4245-05-00



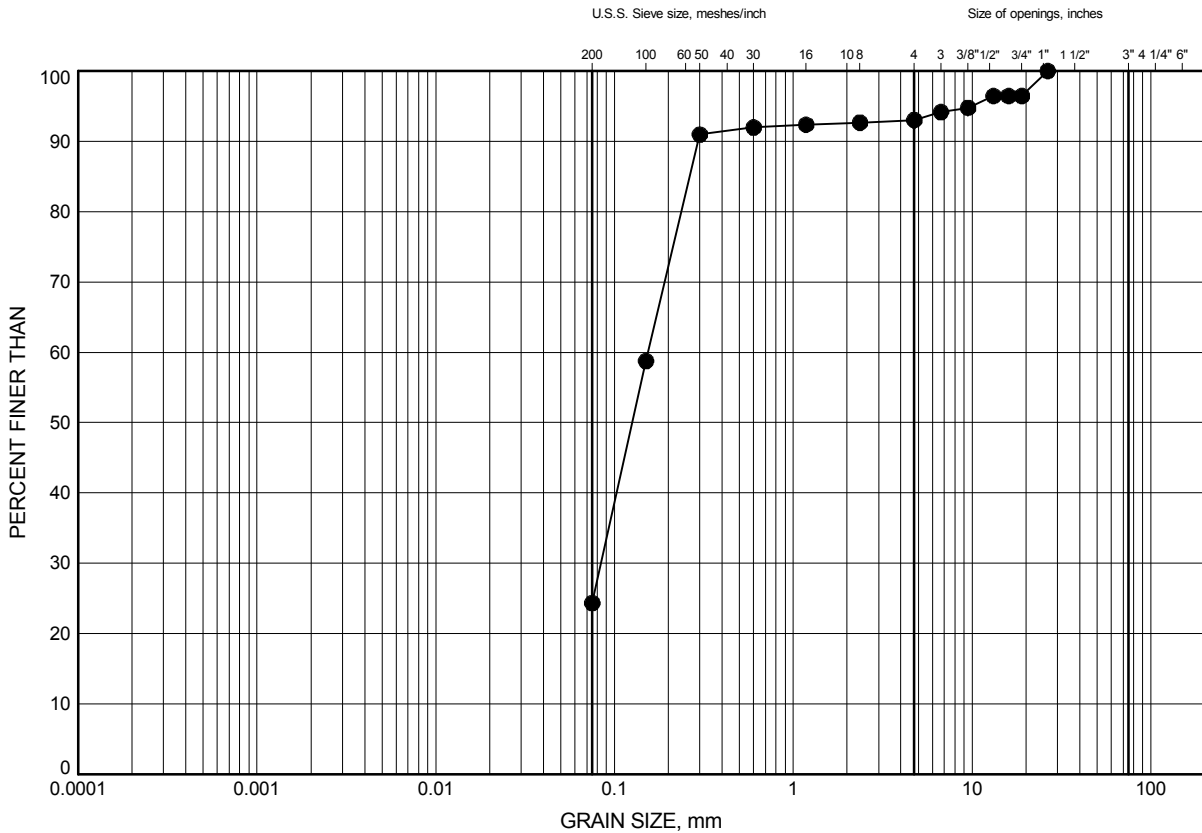
Prep'd CM
Chkd. SD

Hwy 417 O-Train - Loretta Crib Wall

GRAIN SIZE DISTRIBUTION

FIGURE C5

Silty Sand



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	18-03	4.11	62.59

Date April 2018
GWP# 4245-05-00



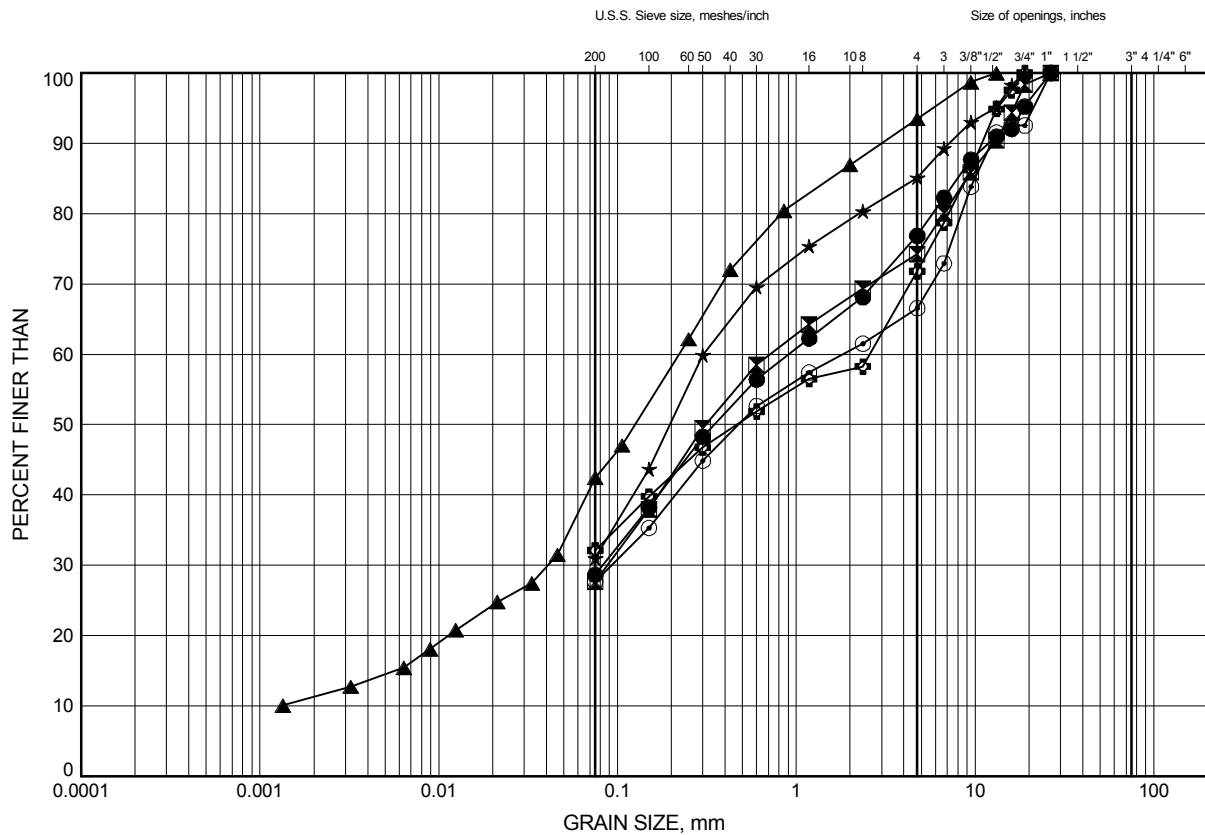
Prep'd CM
Chkd. SD

Hwy 417 O-Train - Loretta Crib Wall

GRAIN SIZE DISTRIBUTION

FIGURE C6

Glacial Till



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	18-01	1.07	70.33
⊠	18-01	3.35	68.05
▲	18-01	4.88	66.52
★	18-02	1.07	68.01
⊙	18-02	3.35	65.73
⊕	18-03	5.49	61.21

Date April 2018

GWP# 4245-05-00



Prep'd CM

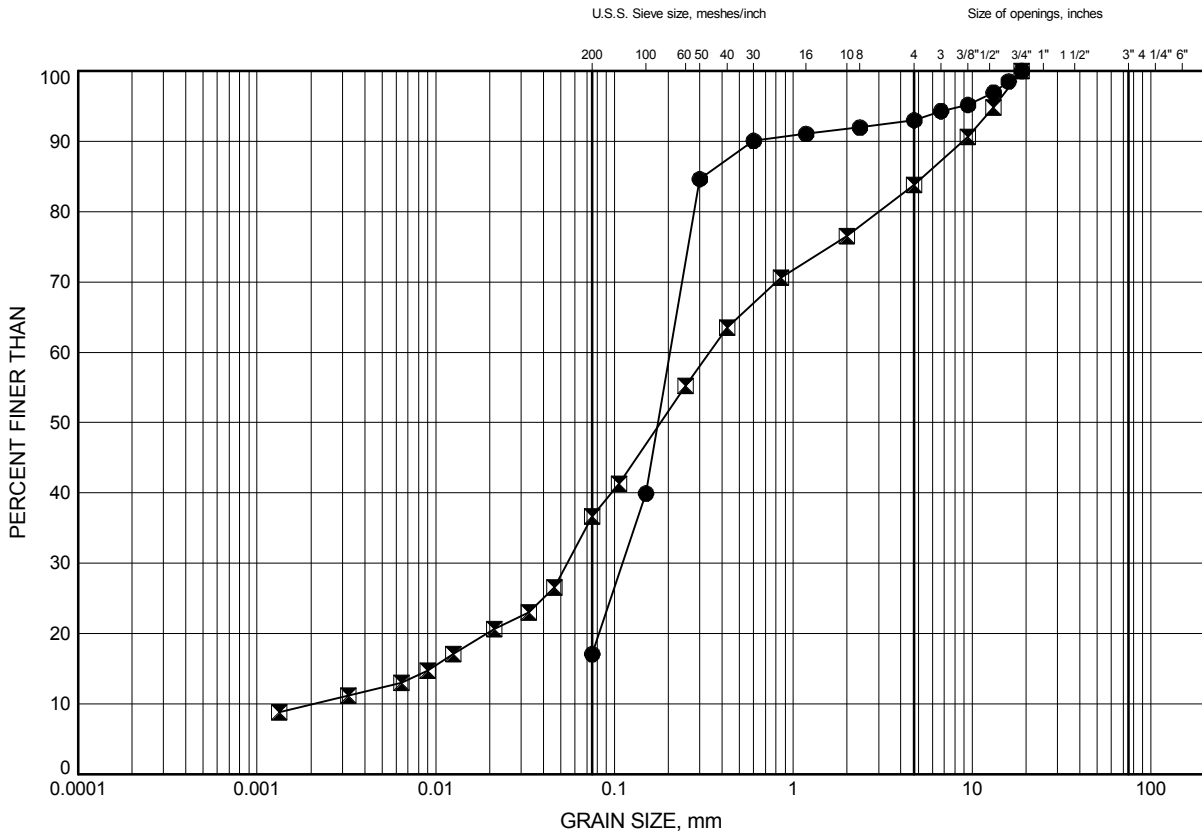
Chkd. SD

Hwy 417 O-Train - Loretta Crib Wall

GRAIN SIZE DISTRIBUTION

FIGURE C7

Glacial Till



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	18-04	9.45	65.05
◻	18-05	7.92	65.88

Date April 2018
GWP# 4245-05-00



Prep'd CM
Chkd. SD

RETAINING WALL FROM LORETTA AVENUE NORTH
TO THE CPR/O-TRAIN BRIDGES
HIGHWAY 417, OTTAWA

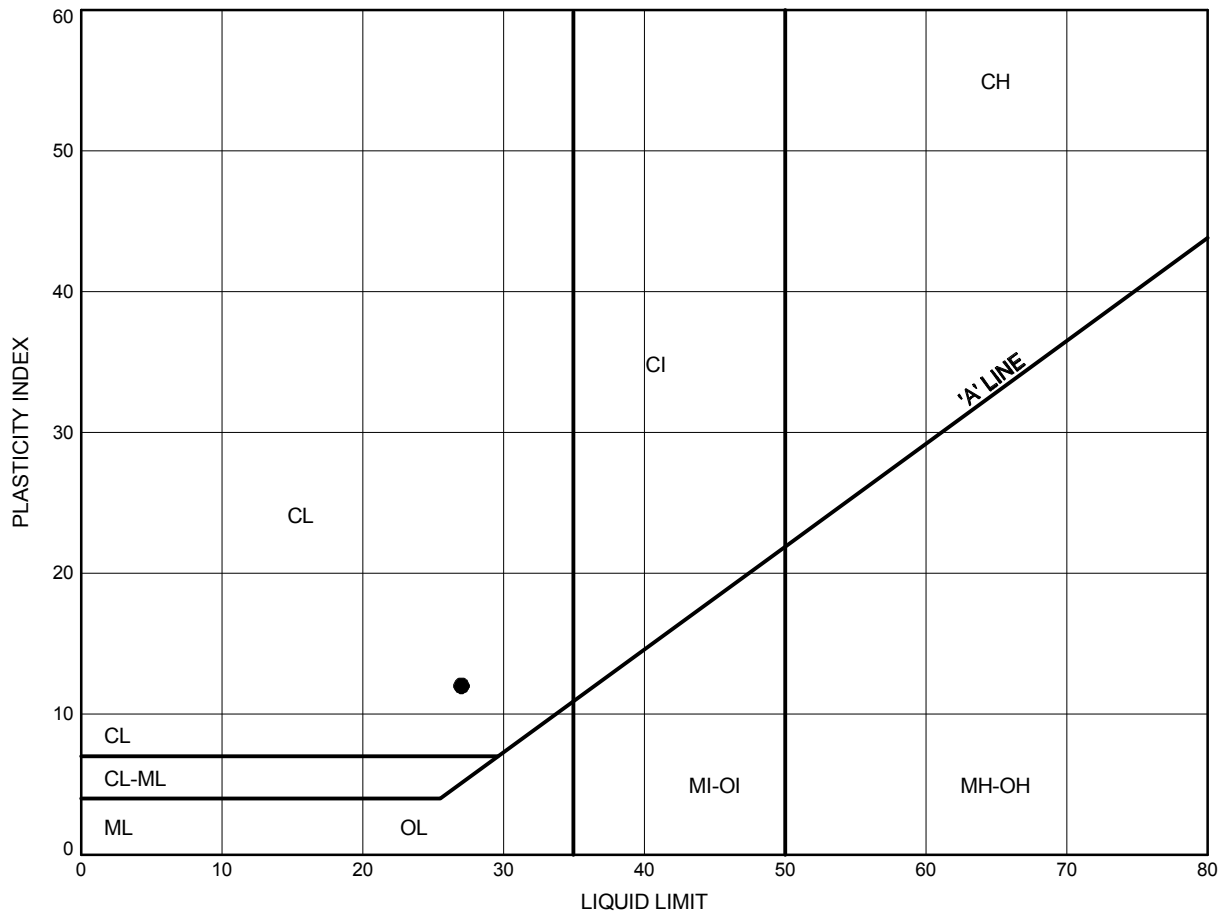
Appendix C.2
Plasticity Chart Figures

Hwy 417 O-Train - Loretta Crib Wall

ATTERBERG LIMITS TEST RESULTS

FIGURE C8

Fill - Clayey Sand



LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	18-03	1.83	64.87

Date .. March 2018 ..
GWP# .. 4245-05-00 ..

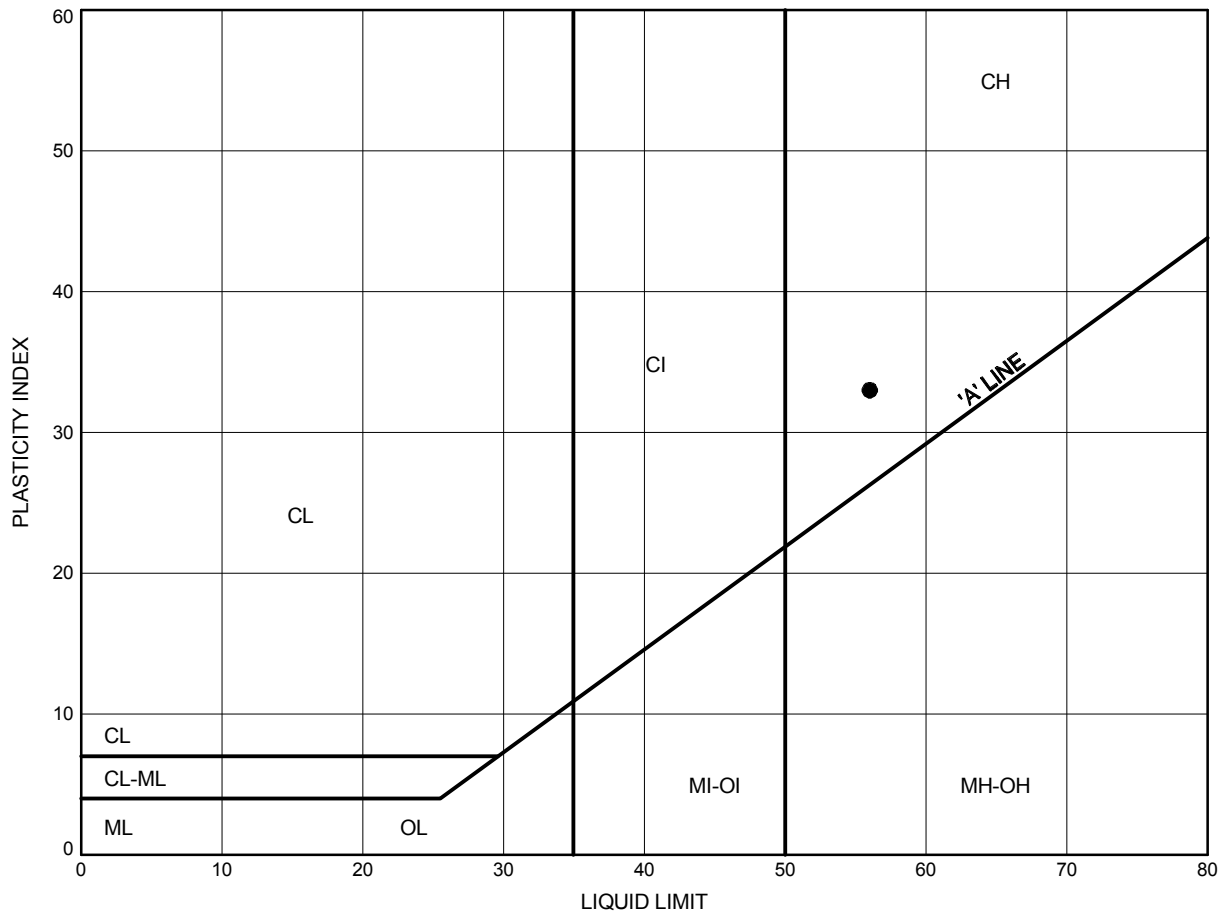


Prep'dCM.....
Chkd.SD.....

Hwy 417 O-Train - Loretta Crib Wall
ATTERBERG LIMITS TEST RESULTS

FIGURE C9

Clay (CH)



LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	18-04	6.40	68.10

Date March 2018
 GWP# 4245-05-00

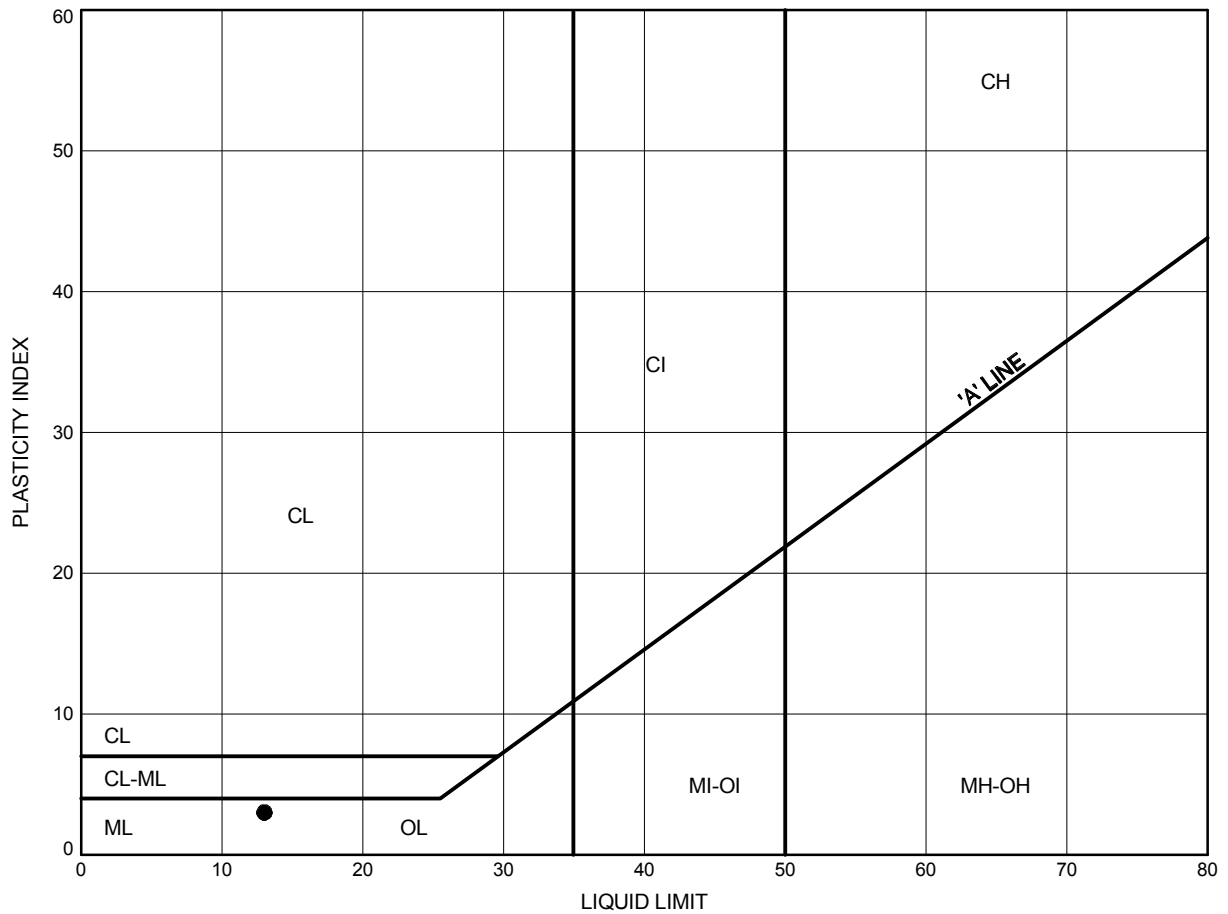


Prep'd CM
 Chkd. SD

Hwy 417 O-Train - Loretta Crib Wall
ATTERBERG LIMITS TEST RESULTS

FIGURE C10

Glacial Till (Fines Only)



LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	18-01	4.88	66.52

Date March 2018
 GWP# 4245-05-00



Prep'd CM
 Chkd. SD

RETAINING WALL FROM LORETTA AVENUE NORTH
TO THE CPR/O-TRAIN BRIDGES
HIGHWAY 417, OTTAWA

Appendix C.3

Unconfined Compression Test Results



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,

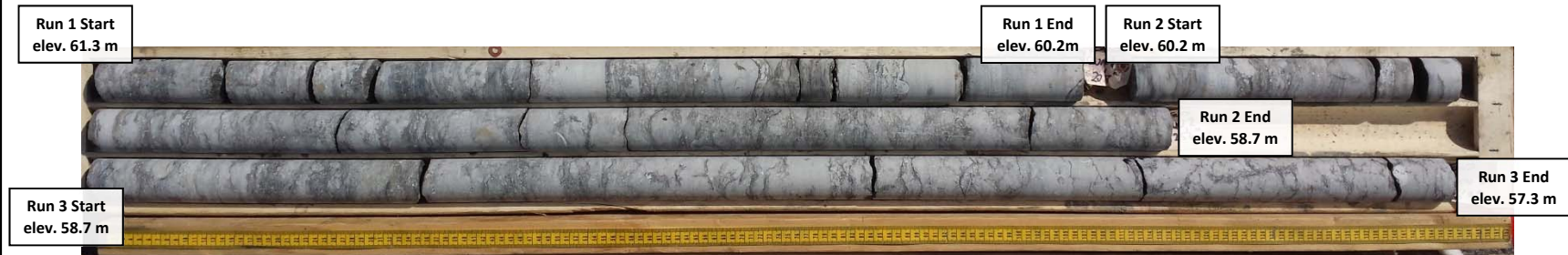
Stantec Consulting Ltd

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

RETAINING WALL FROM LORETTA AVENUE NORTH
TO THE CPR/O-TRAIN BRIDGES
HIGHWAY 417, OTTAWA

Appendix C.4
Bedrock Core Photographs

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



THURBER ENGINEERING LTD.

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

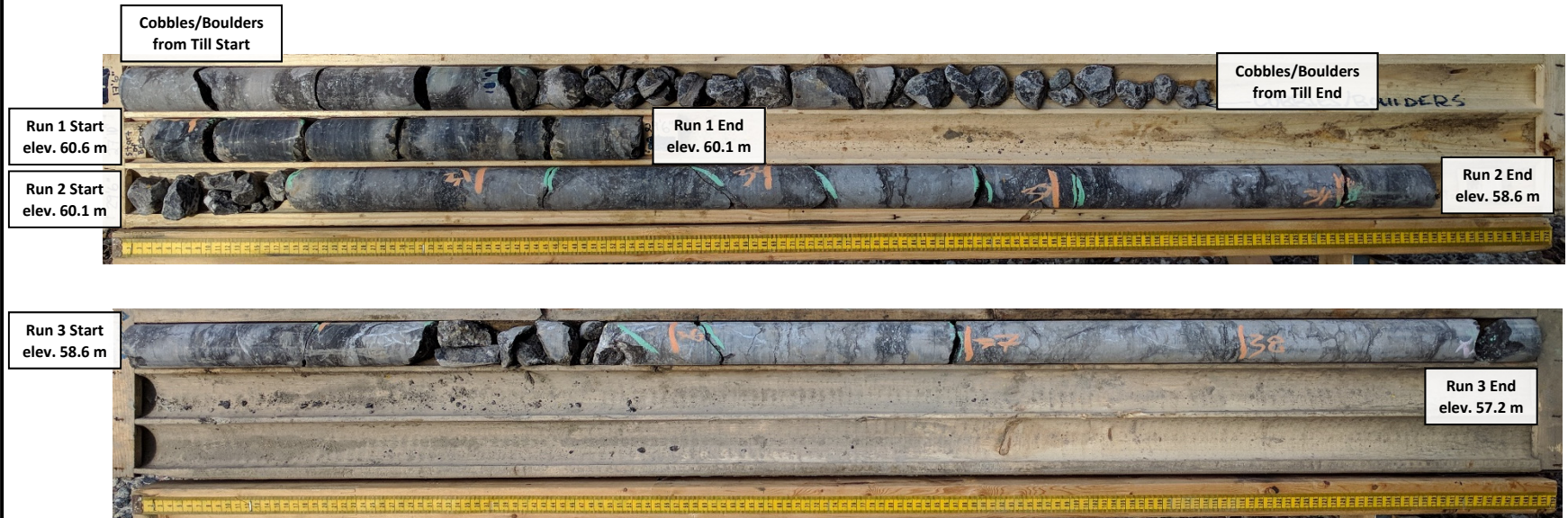
GWP 4245-05-00

Project No.: 11189

Borehole 18-01
Run 1 to 3 (of 3)
Elevation 58.7 m to 55.7 m



Borehole 18-02
Run 1 to 3 (of 3)
Elevation 60.6 m to 57.2 m



Borehole 18-03
Run 1 to 3 (of 3)
Elevation 60.9 m to 57.7 m



Borehole 18-04
Run 1 to 3 (of 3)
Elevation 59.7 m to 56.6 m



Borehole 18-05
Run 1 to 3 (of 3)
Elevation 60.5 m to 57.4 m



RETAINING WALL FROM LORETTA AVENUE NORTH
TO THE CPR/O-TRAIN BRIDGES
HIGHWAY 417, OTTAWA

Appendix C.5
Analytical Test Results

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

Report Date: 05-Mar-2018

Order Date: 28-Feb-2018

Project Description: O Train- Crib Wall

Client ID:	18-1, SS4, 7'6"-9'6"	18-2, SS2, 5'-7'	18-5, SS4, 10'-12'	18-4, SS3, 7'6"-9'6"
Sample Date:	26-Feb-18	21-Feb-18	12-Feb-18	13-Feb-18
Sample ID:	1809301-01	1809301-02	1809301-03	1809301-04
MDL/Units	Soil	Soil	Soil	Soil

Physical Characteristics

% Solids	0.1 % by Wt.	92.2	90.3	87.4	88.4
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General Inorganics

Conductivity	5 uS/cm	247	317	1520	938
pH	0.05 pH Units	7.98	7.98	8.02	8.05
Resistivity	0.10 Ohm.m	40.4	31.6	6.59	10.7

Anions

Chloride	5 ug/g dry	63	98	998	391
Sulphate	5 ug/g dry	76	31	99	143

RETAINING WALL FROM LORETTA AVENUE NORTH
TO THE CPR/O-TRAIN BRIDGES
HIGHWAY 417, OTTAWA

Appendix D.

Site Photographs

RETAINING WALL FROM LORETTA AVENUE NORTH
TO THE CPR/O-TRAIN BRIDGES
HIGHWAY 417, OTTAWA



Photo 1. Looking west along the existing crib retaining wall



Photo 2. Looking at east end of crib retaining wall

RETAINING WALL FROM LORETTA AVENUE NORTH
TO THE CPR/O-TRAIN BRIDGES
HIGHWAY 417, OTTAWA

Appendix E.

Foundation Comparison

RETAINING WALL FROM LORETTA AVENUE NORTH
TO THE CPR/O-TRAIN BRIDGES
HIGHWAY 417, OTTAWA

Comparison of Foundation Alternatives

Comment	Caissons	Driven Steel Piles (H-Piles)	Shallow Spread Footings	RSS Wall
<i>Advantages</i>	<ul style="list-style-type: none"> - Existing crib wall can remain in place - Minimal impact to traffic - High axial and lateral resistance, particularly if socketed into bedrock - Robust compared to driven piles, which are more likely to be wasted due to obstructions 	<ul style="list-style-type: none"> - Existing crib wall can remain in place - Minimal impact to traffic - Higher geotechnical capacity than spread footings - Requires less concrete than spread footings or caissons - Lateral resistance provided by native soil or with battered piles 	<ul style="list-style-type: none"> - A specialist contractor is not required - Frost protection detail is straightforward - Typically less costly than deep foundations if there are no mitigating factors 	<ul style="list-style-type: none"> - Flexible structure with more tolerance for differential settlement - cost effective for high walls
<i>Disadvantages</i>	<ul style="list-style-type: none"> - Cobbles and boulders in the glacial till are likely to slow progress - Requires a specialist contractor - Specialized installation measures such as liners and pneumatic drilling equipment to penetrate boulders will be required. A slurry may also be required to prevent basal 'blow-up'. - Higher unit cost than spread footings 	<ul style="list-style-type: none"> - Cannot penetrate/displace large cobbles or boulders - Requires a specialist contractor - Vibrations could cause damage/movement to adjacent structures - Higher unit cost than spread footings 	<ul style="list-style-type: none"> - lower geotechnical resistances than deep foundations - A shear key may be required to provide lateral resistance - the existing crib wall will need to be removed to provide the necessary space to construct spread footings - highway traffic will be impacted 	<ul style="list-style-type: none"> - the existing crib wall will need to be removed to provide the necessary space to construct an RSS wall - highway traffic will be impacted
<i>Risks/ Consequences</i>	<ul style="list-style-type: none"> - May not be able to dewater socket for cleaning and inspection; end-bearing resistance may be questionable - inadequate equipment to penetrate the bouldery conditions could slow progress 	<ul style="list-style-type: none"> - Cobbles and boulders in the glacial till are likely to interfere with pile driving; some piles may be wasted and/or supplemental piles may be required; design changes during construction may be required 	<ul style="list-style-type: none"> - excavations to remove the existing crib wall will encroach on the existing highway embankment, which would need to be supported with a temporary protection system that would increase costs significantly and affect traffic on the highway 	<ul style="list-style-type: none"> - excavations to remove the existing crib wall will encroach on the existing highway embankment, which would need to be supported with a temporary protection system that would increase costs significantly and affect traffic on the highway
<i>Relative Cost</i>	High	High, but typically less than caissons	Very high due to temporary protection system	Very high due to temporary protection system
<i>Assessment</i>	Recommended	Not recommended	Not recommended	Not recommended

RETAINING WALL FROM LORETTA AVENUE NORTH
TO THE CPR/O-TRAIN BRIDGES
HIGHWAY 417, OTTAWA

Appendix F.

P-Y Curves



Project No.: Thurber - 11189
Project: Highway 417 - O-Train Loretta Crib Wall
Date: October 2018

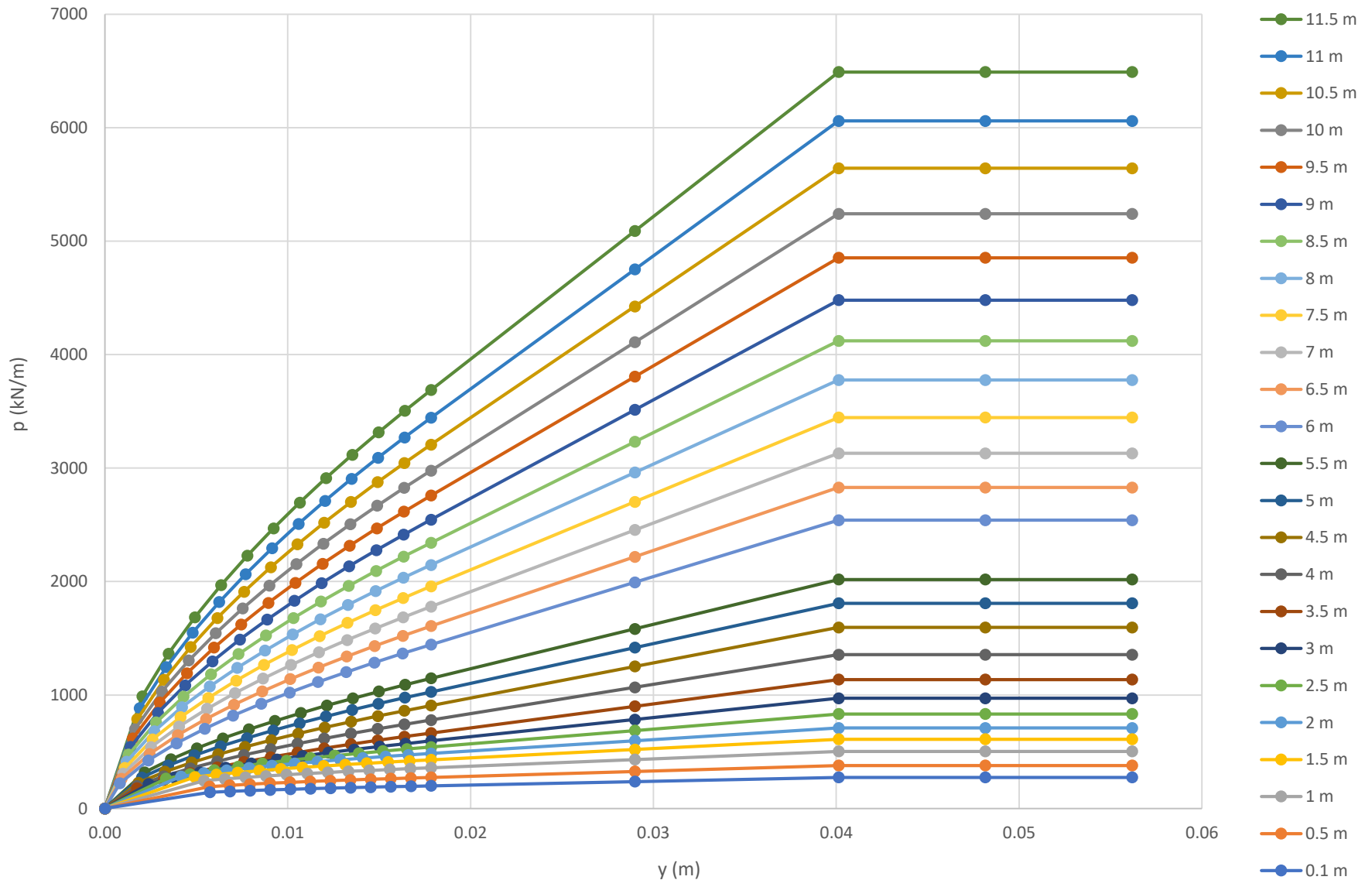
1.07 m Diameter Caisson
Borehole 18-01
Depth Below Elevation 70.4 m (underside of footing)

STATIC	Depth (m)																																																					
	0.1		0.5		1		1.5		2		2.5		3		3.5		4		4.5		5		5.5		6		6.5		7		7.5		8		8.5		9		9.5		10		10.5		11		11.5							
	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)										
	0.0000	0.00	0.0000	0.00	0.0000	0.00	0.0000	0.00	0.0000	0.00	0.0000	0.00	0.0000	0.00	0.0000	0.00	0.0000	0.00	0.0000	0.00	0.0000	0.00	0.0000	0.00	0.0000	0.00	0.0000	0.00	0.0000	0.00	0.0000	0.00	0.0000	0.00	0.0000	0.00	0.0000	0.00	0.0000	0.00	0.0000	0.00	0.0000	0.00	0.0000	0.00	0.0000	0.00						
	0.0057	142.52	0.0057	193.53	0.0054	244.65	0.0049	277.42	0.0042	281.70	0.0033	263.12	0.0024	215.18	0.0017	177.53	0.0018	197.81	0.0018	227.19	0.0020	272.98	0.0022	318.91	0.0008	221.84	0.0009	261.96	0.0010	306.92	0.0011	357.06	0.0012	412.71	0.0013	474.23	0.0014	541.97	0.0015	616.30	0.0016	697.59	0.0018	786.23	0.0019	882.59	0.0020	987.08						
	0.0068	150.12	0.0068	204.24	0.0065	260.01	0.0061	298.24	0.0054	310.49	0.0046	303.68	0.0038	271.96	0.0032	250.76	0.0032	283.09	0.0033	324.21	0.0035	378.68	0.0036	433.16	0.0024	422.77	0.0024	479.71	0.0025	541.48	0.0026	608.35	0.0027	680.62	0.0028	758.60	0.0029	842.60	0.0030	932.95	0.0031	1029.97	0.0032	1134.00	0.0033	1245.41	0.0035	1364.55						
	0.0079	156.91	0.0079	213.81	0.0077	273.66	0.0073	316.55	0.0066	335.36	0.0060	338.08	0.0052	318.98	0.0047	310.36	0.0047	353.30	0.0047	405.15	0.0049	467.92	0.0050	530.49	0.0039	574.03	0.0040	645.71	0.0041	722.48	0.0041	804.56	0.0042	892.20	0.0043	985.65	0.0044	1085.17	0.0045	1191.04	0.0046	1303.53	0.0047	1422.93	0.0048	1549.55	0.0049	1683.69						
	0.0090	163.06	0.0090	222.49	0.0088	286.01	0.0084	332.99	0.0079	357.47	0.0073	368.38	0.0066	360.02	0.0061	362.29	0.0061	414.93	0.0062	476.75	0.0063	547.28	0.0064	617.42	0.0055	702.91	0.0055	787.67	0.0056	877.86	0.0057	973.67	0.0057	1075.28	0.0058	1182.90	0.0059	1296.76	0.0060	1417.07	0.0061	1544.07	0.0061	1678.01	0.0062	1819.14	0.0063	1967.72						
	0.0101	168.71	0.0101	230.47	0.0099	297.33	0.0096	347.98	0.0091	377.48	0.0086	395.69	0.0080	396.93	0.0076	409.08	0.0076	470.77	0.0076	541.99	0.0078	619.82	0.0079	697.07	0.0070	818.02	0.0071	914.72	0.0071	1017.19	0.0072	1125.59	0.0072	1240.08	0.0073	1360.84	0.0074	1488.03	0.0075	1621.86	0.0075	1762.50	0.0076	1910.16	0.0077	2065.05	0.0078	2227.40						
	0.0112	173.94	0.0112	237.86	0.0111	307.81	0.0108	361.80	0.0104	395.84	0.0099	420.70	0.0094	430.75	0.0091	452.11	0.0091	522.37	0.0091	602.51	0.0092	687.24	0.0093	771.23	0.0086	923.50	0.0086	1031.25	0.0086	1145.14	0.0087	1265.28	0.0088	1391.81	0.0088	1524.87	0.0089	1664.60	0.0089	1811.15	0.0090	1964.68	0.0091	2125.36	0.0091	2293.36	0.0092	2468.85						

1.07 m Diameter Caisson
Borehole 18-02
Depth Below Elevation 68.1 m (underside of footing)

STATIC	Depth (m)																																	
	0.1		0.5		1		1.5		2		2.5		3		3.5		4		4.5		5		5.5		6		6.5		7					
	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)				
	0.0000	0.00	0.0000	0.00	0.0000	0.00	0.0000	0.00	0.0000	0.00	0.0000	0.00	0.0000	0.00	0.0000	0.00	0.0000	0.00	0.0000	0.00	0.0000	0.00	0.0000	0.00	0.0000	0.00	0.0000	0.00	0.0000	0.00	0.0000	0.00		
	0.0057	142.52	0.0057	193.53	0.0054	244.65	0.0049	277.42	0.0042	281.70	0.0033	263.12	0.0024	215.18	0.0017	177.53	0.0018	197.81	0.0006	135.76	0.0006	136.69	0.0007	165.67	0.0007	198.65	0.0008	235.95	0.0009	277.88				
	0.0068	150.12	0.0068	204.24	0.0056	260.01	0.0061	298.24	0.0054	310.49	0.0046	303.68	0.0038	271.96	0.0032	250.76	0.0032	283.09	0.0022	185.12	0.0022	300.96	0.0022	346.77	0.0023	396.72	0.0024	451.04	0.0024	510.01				
	0.0079	156.91	0.0079	213.81	0.0077	273.66	0.0073	316.55	0.0066	335.36	0.0060	338.08	0.0052	318.98	0.0047	310.36	0.0047	353.30	0.0038	387.39	0.0037	419.54	0.0038	479.36	0.0038	543.72	0.0039	612.81	0.0040	686.84				
	0.0090	165.06	0.0090	222.49	0.0088	286.01	0.0084	332.99	0.0079	357.47	0.0073	368.38	0.0066	360.02	0.0061	362.29	0.0061	414.93	0.0053	475.63	0.0053	519.44	0.0054	593.47	0.0054	668.46	0.0055	750.59	0.0055	838.03				
	0.0101	168.71	0.0101	230.47	0.0099	297.33	0.0096	347.98	0.0091	377.48	0.0086	395.69	0.0080	396.93	0.0076	409.08	0.0076	470.77	0.0069	553.67	0.0069	608.20	0.0069	691.23	0.0070	779.67	0.0070	873.65	0.0071	973.31				
	0.0112	173.04	0.0112	237.86	0.0111	307.81	0.0108	361.80	0.0104	395.84	0.0099	420.70	0.0094	430.75	0.0091	452.11	0.0091	522.37	0.0085	624.75	0.0084	689.28	0.0085	782.45	0.0085	881.46	0.0085	986.40	0.0086	1097.41				
0.0123	178.82	0.0123	244.77	0.0122	317.58	0.0120	374.66	0.0116	412.87	0.0112	443.89	0.0108	462.15	0.0105	492.23	0.0105	570.65	0.0100	690.61	0.0100	764.62	0.0100	867.27	0.0101	976.16	0.0101	1091.39	0.0101	1213.03					
0.0134	183.41	0.0134	251.26	0.0133	326.76	0.0131	386.70	0.0129	428.80	0.0126	465.57	0.0122	491.59	0.0120	530.01	0.0120	616.25	0.0116	752.37	0.0116	835.45	0.0116	947.04	0.0116	1065.27	0.0116	1190.22	0.0117	1321.95					
0.0145	187.74	0.0145	257.39	0.0144	335.43	0.0143	398.06	0.0141	443.78	0.0139	485.99	0.0136	519.40	0.0134	565.84	0.0134	659.64	0.0131	810.81	0.0131	902.59	0.0131	1022.69	0.0132	1149.81	0.0132	1284.02	0.0132	1425.35					
0.0156	191.84	0.0156	263.21	0.0156	343.65	0.0155	408.81	0.0153	457.96	0.0152	505.32	0.0150	545.83	0.0149	600.02	0.0149	701.13	0.0147	866.46	0.0147	966.66	0.0147	1094.89	0.0147	1230.52	0.0147	1373.59	0.0148	1524.13					
0.0167	195.75	0.0167	268.74	0.0167	351.47	0.0167	419.03	0.0166	471.43	0.0165	523.73	0.0164	571.06	0.0164	632.78	0.0164	740.99	0.0163	917.93	0.0163	1028.10	0.0163	1164.14	0.0163	1307.95	0.0163	1459.54	0.0163	1618.94					
0.0178	198.48	0.0178	274.04	0.0178	358.95	0.0178	428.79	0.0178	484.28	0.0178	541.32	0.0178	595.25	0.0178	664.30	0.0178	779.42	0.0178	970.75	0.0178	1087.25	0.0178	1230.83	0.0178	1382.52	0.0178	1542.34	0.0178	1710.30					
0.0290	236.47	0.0290	326.41	0.0290	431.17	0.0290	519.38	0.0290	596.98	0.0290	686.80	0.0290	783.36	0.0290	900.22	0.0290	1067.40	0.0290	1329.43	0.0290	1500.41	0.0290	1698.55	0.0290	1907.88	0.0290	2128.44	0.0290	2360.21					
0.0401	273.46	0.0401	378.78	0.0401	503.39	0.0401	609.97	0.0401	709.67	0.0401	832.29	0.0401	971.48	0.0401	1136.13	0.0401	1355.38	0.0401	1687.91	0.0401	1913.56	0.0401	2166.26	0.0401	2433.24	0.0401	2714.53	0.0401	3010.13					
0.0482	273.46	0.0482	378.78	0.0482	503.39	0.0482	609.97	0.0482	709.67	0.0482	832.29	0.0482	971.48	0.0482	1136.13	0.0482	1355.38	0.0482	1687.91	0.0482	1913.56	0.0482	2166.26	0.0482	2433.24	0.0482	2714.53	0.0482	3010.13					
0.0562	273.46	0.0562	378.78	0.0562	503.39	0.0562	609.97	0.0562	709.67	0.0562	832.29	0.0562	971.48	0.0562	1136.13	0.0562	1355.38	0.0562	1687.91	0.0562	1913.56	0.0562	2166.26	0.0562	2433.24	0.0562	2714.53	0.0562	3010.13					

P-Y Curves for 1.07 m Diameter Caisson - Borehole 18-01

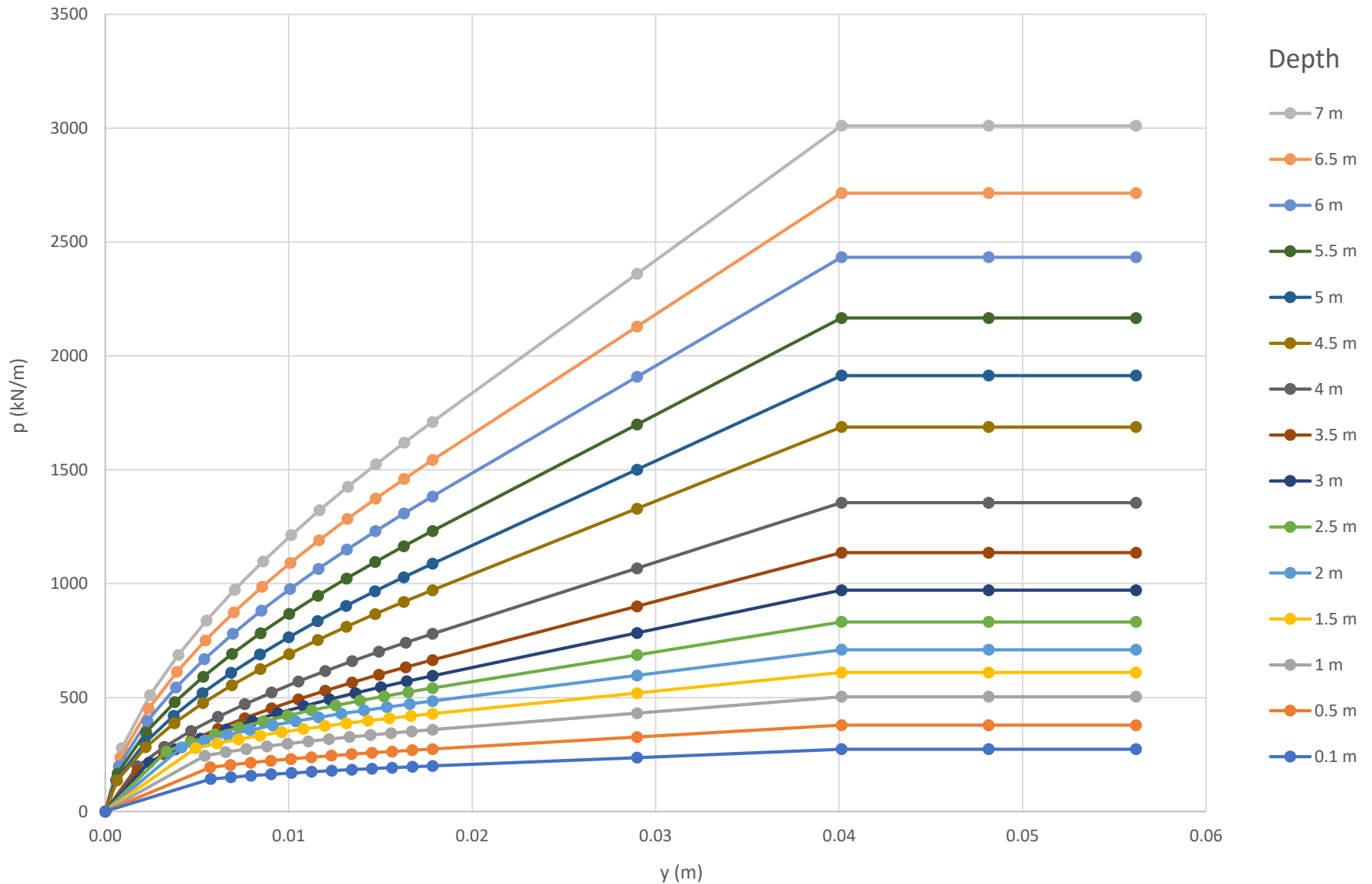


File: 11189

FIGURE F-1



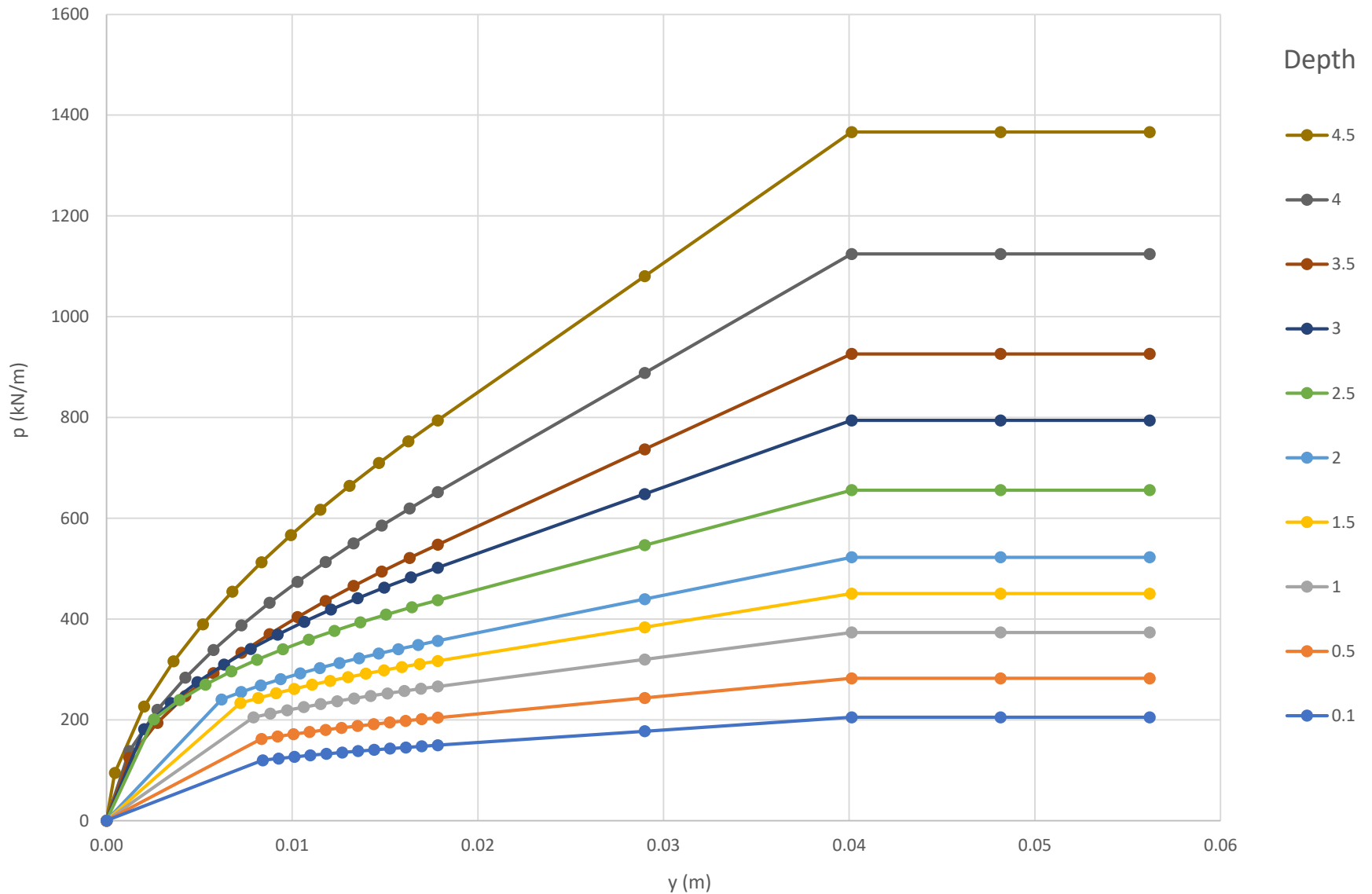
P-Y Curves for 1.07 m Diameter Caisson - Borehole 18-02



File: 11189

FIGURE F-2

P-Y Curves for 1.07 m Diameter Caisson - Borehole 18-03



File: 11189

FIGURE F-3

RETAINING WALL FROM LORETTA AVENUE NORTH
TO THE CPR/O-TRAIN BRIDGES
HIGHWAY 417, OTTAWA

Appendix G.

List of Special Provisions and OPSS Documents Referenced in this Report

RETAINING WALL FROM LORETTA AVENUE NORTH
TO THE CPR/O-TRAIN BRIDGES
HIGHWAY 417, OTTAWA

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

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 903	Construction Specification for Deep Foundations
OPSD 3090.101	Foundation Frost Penetration Depths for Southern Ontario
NSSP Foun003	Dewatering Structure Excavations

2. Suggested text for an NSSP on "Construction of Caissons".

The Contractor is advised that variable types of subsurface materials may be encountered during caisson advancement. For additional information regarding soil and rock conditions, the Contractor is referred to the Foundation Investigation Report. For bidding purposes, the Contractor shall assume the following:

- 1) There is a high probability that cobbles and boulders will be encountered within the glacial till. Caisson installation equipment must be able to penetrate or remove these obstructions.
- 2) The Contractor is responsible for constructing the caissons without disturbing the existing materials at the sides or bases of the foundations.
- 3) Water seepage and/or soil sloughing into the caisson hole, and/or basal 'blow-up' due to an unbalanced hydrostatic head, may occur from existing cohesionless soils during excavation. The destabilizing effects of water seepage, including sloughing of the sidewalls and basal instability must be controlled to maintain the design bearing resistances and to prevent settlement of the existing crib wall. Temporary liners must be used in accordance with OPSS 903 to support the caisson sidewalls during excavation. If basal 'blow-up' occurs during shaft excavation, the contractor shall use a slurry or head of water in the shaft to balance the hydrostatic force and maintain an undisturbed base.
- 4) The contractor must not cause structural damage to, or movement of, the existing crib wall during construction.
- 5) The contractor shall place concrete in the shaft on the same day that the excavation is completed. Open shafts left overnight will result in softening of the foundation soils and will need to re-drilled/deepened to the satisfaction of the Contract Administrator.
- 6) The limestone bedrock is strong to very strong and of good to excellent quality. These conditions must be taken into account when selecting equipment to advance rock sockets. Equipment supplied to construct rock sockets must be capable of excavating the bedrock to the specified socket dimensions without disturbing or fracturing the bedrock forming the sidewalls and base of the socket. Blasting to facilitate the removal of bedrock is not permitted.
- 7) The contractor is responsible for proper disposal of materials generated from the site.